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Seismic Risk Evaluation of Existing Building

A Thesis in Structural Engineering

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A Thesis
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The undersigned have examined the thesis entitled 'Seismic Risk Evaluation of Existing Building' by Genet H/Michael, a candidate for the degree of Master of Science and hereby certify that it is worthy of acceptance.

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all materials and results that are not original to this work.

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ABSTRACT

In spite of the advances of research in earthquake engineering in general and particularly on seismic design codes, catastrophic losses have occurred recently in many countries in the world, including countries in which earthquake engineering studies are priority tasks. It is clear that new developments in earthquake resistant design can only be applied to new projects, which represent a small part of the existing structures in a seismic area.

This thesis work investigates seismic risk assessment of nine story building located in Addis Ababa City. ETABS software is used for non-linear static pushover analysis whereas Federal Emergency Management Agency (FEAM) earthquake loss estimation methodology is employed for earthquake loss estimation during risk assessment for the building model.

Pushover analysis is used in order to investigate the capacity of the study building and from intersection of capacity curve and demand curve performance point of the building has been determined with this estimated performance value of the building probability of reaching or exceeding different damage state are calculated and finally with these probability of reaching or exceeding four different damage states the total loss due to structural damage becomes 1.585B birr The estimated value of economic losses based on the method FEMA-1999 approach obtained by MDP (mean damage probability) value of 18.09%. This means that the building has a small damage economic loss probability of 18.09% due to the expected earthquake scenario.

And human life loss is calculated based on the assumption that 5 people dwell in single household. From the result it is observed that night time earthquake scenario causes high causality than day time scenario and rush hour scenario. And for particular time earthquake scenario the number of casualty decreases as level of severity increase.

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CHAPTER 1 INTRODUCTION

1.1 Background

The so called "natural" disasters, that is, those related to phenomena of Nature, have caused throughout the centuries great convolutions in the process of human development. Despite advances in science and technology have produced a great deal of knowledge on the causes of those disasters, Loss of human life is only slightly decreasing with time; but the economic losses have dramatically increased in the last decades. The rise in world population and the complexity of societal organization, among others, are factors that may explain this unfortunate fact. Nowadays inadequate non-sustainable uses of the territory and construction practices, especially in developing countries, are clear causes of the too frequent "natural" disasters. The number of victims and the economic losses from earthquakes are strongly dependent on the seismic magnitude and focal distances to urban areas. Moreover, the relation between economic losses and number of victims is dependent on social and economic factors associated to the level of development of the affected country. For the same range of magnitudes a larger number of victims occur in less developed countries; the larger economic losses occur in the most developed countries. [1, 2]

Developing countries are more vulnerable to hazards because of their increasing rate of development and urban growth. The lack of proper disaster management leads to increase in risk in more densely populated cities. Most of the growth in terms of civil structures and infrastructure will concentrate in the developing countries for the next few decades. These countries are already loaded with various urban problems like population growth, urban sprawl, building density and lack of financial strength. The risk is continuously increasing in these countries at an alarming rate.[3]

The current economic expansion in Ethiopia which seems to be driven by a number of enabling factors has had substantial impact in the transportation, energy, and water supply sectors with a growing number of large scale infrastructure projects such as dams, power-plants, highways, water reservoirs, and expansion of railways either coming or entering construction phase. Furthermore, pressure from other natural developments - the staggering population growth of the country being a primary one - continue to force rapid implementation of large-scale engineering infrastructure works such as mass-housing, water-supply reservoirs, power-plants, dams, new cities, etc. As things stand, the country's population is projected to reach a staggering 120 million by 2025 positioning Ethiopia to be among the top 10-15 populous countries on the planet (see Figure 1.1). In addition to a multitude of other threats that this population growth could bring, the issue of housing these additional 30-40 million Ethiopians in the next few decades will pose a huge risk factor. In a recent paper, it has been argued that 25 new cities with size equivalent to present Dire Dawa are needed or the current 10 cities such as Addis Ababa and Dire Dawa will have to become mega cities of 10 million or more to accommodate this growth. While these projections regarding urbanization may be a little bit on the high-side, there is no denying regarding the need for housing these additional millions of citizens in the next several decades.[4]

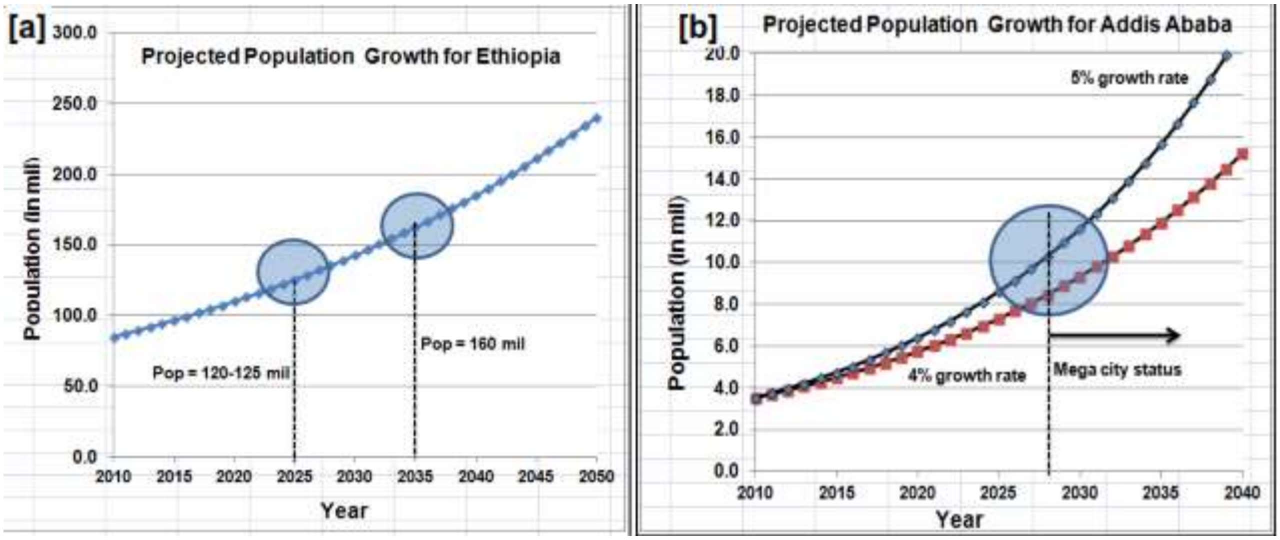

Figure 1-1 Population Projections - (a) Ethiopia (b) Addis Ababa

Interestingly, however, a substantial amount of these large infrastructure works already lie or will be in or in close proximity to some of the most seismically active regions of the country such as Afar Triangle, the Main Ethiopian Rift (MER), and the Southern Most Rift (SMR) where well-documented damage-causing earthquakes are common. A review of the engineering reports associated with some of the largest and most expensive infrastructure projects in the country suggest that - despite the presence of a substantial amount of published literature on the significant seismicity of the region - the severity of threats posed by seismic hazards on the safety and serviceability of these structures is not well-understood by the main stake-holders such as policy-makers, insurance companies, real-estate developers, capital investors, building design-checkers and, not infrequently, the engineering community itself as well.[4]

Against this background, therefore, the need for preparing for this real and substantial threat of seismic hazards in the country is pressing and requires attention at all levels. Ethiopia and Global Earthquake Model (GEM) Foundation have signed a Memorandum of Understanding (MoU) that will enable them to work together on earthquake risk assessment in Addis Ababa.

Studying earthquake damage and loss estimation is a complex process. A loss estimation study for a major metropolitan area could take months to collect the underlying data and would require the participation of experts from several fields. Despite their complexity, loss estimation studies have proven to be a very useful tool for developing emergency preparedness plans and for promoting seismic risk mitigation. It is relevant to mention that, in this paper due to lack of advanced space technology, remote sensing image, high resolution satellite images, time and resource discussion of seismic hazard as well as seismic risk will focus on methodology of seismic risk assessment of Addis Ababa, Addis Ababa lies in one of the most seismically active regions in Ethiopia with a population of approximately 4million it is important to understand the possible damage and loss that could occur as a result of future earthquakes in order to reduce or eliminate the potential for catastrophic effects.

1.2 Problem Definition

The pressure of population in Addis Ababa city, expressed both in terms of absolute number of additional people every year, their distributional pattern, and their decadal growth rate of population, have resulted in several constraints in the development process, and have also resulted in unsustainable exploitation of the existing resource base. The process of urbanization in Addis in terms of net accretion of population, has been faster in recent decades, and coupled with changes in economic activities, have resulted in certain changes in the socioeconomic structure of the population. Migration from the suburban and rural areas to the city in search of jobs has also been an important factor in shaping the socio-economic profile of the city's population.

Seismic resiliency of new buildings has improved over the years due to improved seismic codes and design practices. However, vulnerability of seismically deficient older buildings, designed and built on the basis of older codes of practice, poses a significant threat to life safety and survivability of buildings. It is economically not feasible to retrofit the entire seismically deficient infrastructure. Therefore, there is need for a comprehensive plan to identify critical structures and prioritize their retrofit requirements.

Responding to earthquakes can be a very difficult task if nothing has been done to prepare for such events in advance. Therefore, preventive and preparedness actions should be carried out to reduce risks and facilitate emergency response.

1.3 Goals, Purpose, Objectives and Tasks

The main purpose of the earthquake risk assessment is to assess the potential damage and losses that can be created by potential future earthquakes and inform the authorities and the public about the risk their city and its residents face. A reliable earthquake risk assessment also helps identify the most vulnerable building typologies, which, if used properly, can lead to a targeted and economically viable strengthening strategy.

The main purpose of this study is to develop a methodology to assess seismic vulnerability of reinforced concrete building structures and to estimate direct losses related to structural damage due to future seismic events. The goal of this thesis is to contribute to the knowledge and understanding of the effects of earthquake related disasters. This knowledge will improve both short and long term disaster planning as well as community awareness. These improvements will help to mitigate the negative consequences of potential seismic event.

The purpose of this study is to supply how geo-scientific and engineering knowledge is transferred to civil protection and insurance agents, and how the close collaboration between geoscientists, engineers and emergency managers can contribute to more efficient earthquake mitigation.

The objectives of the thesis are to:

1. Assess methodologies for seismic risk assessment;
2. Identification of case study structures: Identify representative building structures in the Addis Ababa Region. The building inventory data are collected as much as possible and used to find the most common types of structures. The focus of this paper is Reinforced concrete building structures.
3. Evaluation of seismic performance*:* Evaluate the seismic performance of existing Reinforced concrete structures in Region. To estimate the seismic performance, structural analyses are conducted using nonlinear pushover analyses

4. Estimation of seismic fragility
5. Assess Structural Losses.
6. Provide documentation on how the risk estimates shall be implemented.

This study aims to present a comprehensive seismic risk assessment in Addis Ababa, where the different stages of the calculation process are explained and discussed. The stages of this assessment can be summarized as follows:

- Probabilistic seismic hazard assessment.
- Assembly of the exposure database.
- Seismic vulnerability assessment.
- Damage and loss calculation

For the accomplishment purpose of this thesis work there is a methodology that designed to achieve the general and specific objective of this thesis work. Among the methodology that used are listed as follows.

- Review of related literatures.
- Earthquake hazard analysis for Addis Ababa
- Selection of types of reinforced concrete buildings used for analysis and modeling of the selected buildings with appropriate software. In order to investigate the abovementioned objectives, three-dimensional reinforced concrete building models are formed and analyzed according to the EBCS /Euro code
- Performing non-linear analysis on selected building.
- Identify damage state and quantify the loss

1.4 Scope

This thesis focuses on the methodology for seismic risk assessment (SRA). And because assessment of seismic risk involves different discipline more focus will be given to related discipline (Structural engineering) while most methodologies are appropriate for any region, the work presented in this thesis is specific to seismic risk assessment in Addis Ababa regions. Due to time and resource shortage more focus is given to the vulnerability assessment for seismic evaluation of buildings and quantification of the damage and loss.

Introductory work has been done in the area of estimating the seismic risk of a building after an earthquake has occurred. The methodology presented in this thesis represents good start, but further research and involvement of experts from other filed would be is needed in order to develop confidence in the results.

1.5 Relevance of Study

Addis Ababa is the capital and the largest city of Ethiopia and home to 25% of the urban population in the country. This city is urbanizing and growing in an accelerated way, and it is considered the growth engine for Ethiopia.[5] The economy of Addis Ababa is growing annually by 14%, and it contributes approximately 50% towards the national GDP. In fact Addis Ababa is not highly prone to natural disasters meaning it is categorized under moderate seismicity. However there are big city which are categorized under high seismicity and it is known that the earthquake risk is not only a factor of the intensity of the disaster the density of the population, building exposer and the vulnerability are also factors. Yet some moderate intense earthquakes have occurred in Addis Ababa but fortunately, none of these have resulted significant damage. Now Addis is highly populous cities, located in zones of moderate seismic risk and in addition not ever seen from low to high rising buildings are constructing and large national capital is invested. Typically, the majority of the constructions in such cities would turn into a major disaster. It is most important in medium and long term to formulate strategies to reduce the vulnerability and losses arising from possible earthquake striking of the city.

Buildings in urban areas are highly vulnerable structures in seismic events especially in developing countries. There is a direct relationship between the damage of civil structures to the number of casualties. Most casualties, damage and economic losses caused by earthquake result from ground motion acting upon buildings incapable of withstanding such motion (Montoya, 2002). Damage to buildings also causes a variety of secondary effects that can be greatly destructive. In developing countries lack of capacity of buildings, lead to increase in risk of property loss and damage to essential buildings substantially increases the rate of casualties.

Therefore, in the absence of adequate information and data required for risk assessment, it becomes difficult to assess the loss in a post-earthquake event. The risk assessment process helps in the preparing of a proper disaster management plan and plays a major role in the process of preparedness, mitigation, response and recovery. The proper implementation of building permits and controls, building codes, and awareness-raising can effectively reduce the earthquake vulnerability to large extent.

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

It is well know that earthquake event is one of undesirable and unavoidable natural hazard that cause damage to buildings, bridges, highways and loss to human life and monetary. These loss and destruction depends on one hand on the characteristics of the event like the intensity, epicentral distance and geologic condition of the area, on other hand the construction method, distribution of buildings and population can be factors. Generally earthquake hazard can have direct and indirect effects.

For different reasons, buildings and other structures may not fully resist the coming earthquake hazard events. The structural damage caused by the earthquake may vary between light damage to total collapse. In addition to that nonstructural damage which consists of failure or malfunctioning of architectural, mechanical and electrical system and component may lead to large financial losses, as well as pose significant risk to life. Due to different reason some buildings types may are more vulnerable than others, but even when a building don’t sustain structural damage, the content of building may suffer from damage especially for certain occupancy such damage can be very important.

As described on chapter one of this paper under scope and limitation seismic risk assessment includes different discipline areas. However, on this paper any discussion is tried to limit on area of structural engineering.

Reliable earthquake loss estimation (in monetary terms) for buildings struck by an earthquake is of growing importance both for the planning of appropriate and cost effective earthquake mitigation measures and for insurance purposes, and also for the definition of criteria for prioritizing seismic strengthening (rehabilitation) programs for existing buildings. Decisions regarding the seismic rehabilitation of existing buildings require both engineering and economic studies and consideration of social priorities.

2.2 Damage to Buildings

During earthquake event buildings, bridges, highways and lifelines suffers extensive structural and non-structural damages which leads to huge amount of loss either in terms of economy or human life.

The damage imposed on buildings is continuous function of earthquake demand. However it is not practical to have such a continuous scale so ranges of damages are used in different assessment methodology. According to FEMA/NIBS (national institute of building science) there are four damage states.[6]

- None
- Slight,
- Moderate,
- Extensive or Complete

2.3 Human and financial losses

Earthquake doesn't kill peoples buildings do. As a result of structural or nonstructural damage of buildings the monetary and human life loss will be very substantial. Furthermore for developing countries like Ethiopian, where expansion of urban development and construction investment are increasing, the economic impact of earthquake will be very high. Ethiopia is crossed by active Great Rift Valley and this makes susceptible to earthquake and volcanic eruptions. May be in the past years the losses were not that much significant relative to other country struck by more or less equivalent magnitude of earthquake. This can be seen from one of the best known disaster database-the EM-DAT database data see Table 2.1 shows that from 1900 to 2013 there were a total of ten earthquakes and eruptions – leading to a total of 93 deaths, 165 injured, 420 homeless and affecting 11,000 people. These are estimated to have an economic cost of more than US$7 million [7].

Table 2.1 Number and impact (human and economic) of earthquakes and volcanoes in Ethiopia (1900-2013)[1]

Disaster type	Number of disasters	Number of people killed	Number of people injured	Number of people affected	Number of homeless	Total number of people affected	Total economic damage (US$ '000)
Earthquake	7	24	165	0	420	585	7,070
Volcano	3	69	0	11,000	0	11,000	0
Total	10	93	165	11,000	420	11,585	7,070

2.4 Seismic risk

Earthquake hazard can occur due to faulting, which is the surface expression of differential movement of blocks of earth’s crust or shaking which is the primary hazard due to earthquake. The former one is typically along narrow features and therefore affects a relatively small fraction of the total affected persons and structure whereas shaking affects a much greater number of structure and human life. Always damage results in loss; the loss can be either in the form of human life or in monitory loss. In general one can conclude that anything subjected to earthquake hazard is at risk from earthquake. Three types of things of value (assets) are at risk; people, money and function.

Earthquake risk is defined by Bruce A.Bolt as “the potential economic, social and environmental consequences of hazardous events that may occur in a specified area unit and period of time”.[1] The same author implies that risk estimation requires a multidisciplinary approach that takes in to account not only the physical damage on structures, human or economic losses, but also social, organizational and institutional factors.

Another definition of seismic risk by author called Katerine “seismic risk, therefore, refers to expected loss due to future earthquakes and it is comprised of four elements : hazard, location, exposure and vulnerability’’.[8] According to this definition in order to have seismic risk the four elements should exist simultaneously.

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[1] Source: Data downloaded from EM-DAT database on 17 January 2013. - http://www.emdat.be/result-country-profile?disgroup=natural&country=eth&period=1900$2013

Definition by Dowrick is “the probability that social or economic consequences of earthquake will equal or exceed specified values at a site, at several sites, or in an area, during specified exposure time’’[9]

Therefore seismic risk assessment and loss estimation comprise hazard analysis, exposure information (structural inventory), Vulnerability analysis and estimation of risk and loss. Since the standard definition of risk is a probability or likelihood of happening loss, which numerical represent between zero and one, Therefore it will be more appropriate to express risk as hazard ×Vulnerability while loss depends on the value of the exposure at risk, which can be given by Loss = Hazard × Vulnerability ×Exposure. Thus, while seismic hazard is a product of natural processes, seismic risk and loss are dependent on the vulnerability and social exposure in terms of the built environment, human population, and value of operations. Assessment of seismic risk will comprise the following components.

Figure 2-1 Components of Seismic Risk [10]

2.5 Component of seismic risk

2.5.1 Seismic hazard

Seismic hazard is a natural phenomenon which produces adverse effects on human life and built up areas. Thus, it can be either descriptive or quantitatively evaluated. Katherine defined seismic hazard as “the study of expected earthquake ground motions at any point on earth and the expected level of shaking at the site or region of interest is calculated based on characteristic of the areas seismic source, the attenuation of seismic waves from epicenter to the site and the local site condition (location)".[8]

Seismic hazard is assessed from instrumental, historical, and geological observations. In other words, seismic hazard is assessed from Earth sciences. Therefore, Earth scientists, in particular, seismologists play a key role in seismic hazard assessment.

In evaluating seismic risk initially seismic hazard assessment should be carried out and known. Seismic hazard assessment could be deterministic or probabilistic. In deterministic seismic hazard a single earthquake scenario is assessed and quantified whereas in probabilistic hazard assessment all possible earthquakes are taken into account and quantified. The discussion among seismologist about deterministic and probabilistic seismic hazard analysis was controversial. While seismic design of critical infrastructures such as nuclear plants and dams were and still are based on deterministic design procedures. In our country Nani building, the new Ethiopian commercial building and the renaissance dam structures could be examples. Whereas the probabilistic method was formulated as risk based approach and has been introduced over the years for national building codes of many countries. Thus, probabilistic seismic hazard analysis (PSHA) is becoming the most widely used method to assess seismic hazard and risk for different regions and Ethiopian is one of them. In our code the result of hazard analysis is given to be used by engineers and the building code ES EN 1998-1 subdivide the national territories into seismic zones based on probabilistic seismic hazard assessment and the hazard map resulted from the assessment is preliminary and processed from instrumentally recorded earthquake catalog. The hazard is described in terms of a single parameter, i.e. the value a_g of the effective peak ground acceleration in rock of firm soil. [11]

Since seismic hazard assessment is essentially forecast of future situation, it always inherent uncertainties. Those uncertainties arose partly because of the process involved, which are not fully understandable and partly because relevant data are scarce and variable. Even if there is a dynamic nature in the process and large uncertainties, studying and understanding such natural hazards and its assessment and quantification are very essential and required for preparation of earthquake loading regulations and for various earthquake risk management purpose.

2.5.2 Seismic vulnerability

According to Katherine seismic vulnerability of a structure refers to how well it will perform under earthquake loading. It is essentially the sensitivity of the exposed structures to the expected seismic hazard in a region. The same author implies that structural vulnerability is typically defined by motion-damage relationships which define the probability of damage to a structure given the level of ground shaking.[8]

Mohammad M. A. et al describe structural vulnerability refers to the susceptibility of those parts of a building which includes foundations, floor slabs, supporting walls, beams, and columns that are required for physical support when subjected to an earthquake or other hazard.[12]

An evaluation of seismic risk to exposed assets or element from hazardous events requires a consideration of element's vulnerability, which is the tendency of the element to suffer from damage. The occurrence of hazardous event to the built up area is not enough to quantify the risk, vulnerability of the structure is also very important.

Seismic vulnerability assessment, ranges from seismic hazard analysis to determination of a structure's response and is a method that estimate amount of damage likely to be imposed on structure as a whole and its elements. There are a lot of seismic vulnerability assessment methods available with their own strength and purpose of application. However, all the methodologies have common objective to determine or quantify the future damage of building due to earthquake and is good always to choose the suitable one.

There are four distinct analytical procedures used for seismic vulnerability assessment of structures [13]: Linear Static, Linear Dynamic, Nonlinear Static (Pushover) and Nonlinear Dynamic Procedures (NDP).

Due to their simplicity linear procedures either static or dynamic are the most common procedures in seismic analysis and design of structures. Such procedures are efficient as long as the structure behaves within elastic limits but not expected during earthquake. If the structure responds beyond the elastic limit, linear analyses may indicate the location of first yielding, but cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. On the other hand, Nonlinear (static and dynamic) procedures are the solutions that can overcome this problem and show the performance level of structures at any loading level. These procedure help demonstrate how structures work by identifying modes of failure and the potential for progressive collapse Nonlinear procedures help engineers to understand how a structure will behave when subjected to major earthquakes.

In addition to the intensity of the earthquake several structural features could be factors affecting seismic vulnerability of buildings for instance soft story, heavy overhang, short column, pounding possibility between adjacent buildings, visible ground settlement, and topographic effect could be among the factors. Furthermore, the existing building codes, regulations and the building types also play a vital role in seismic vulnerability that is why building codes are updated through the times. Therefore, the need to understand the vulnerability of existing buildings is very important so that proper measure can be taken to reduce the vulnerability.

According to Vazurkar U.Y and Chaudhur,D.J seismic vulnerability of structures could be represented either in terms of Damage Probability Matrices (DPM) or vulnerability (fragility) curves. It is expressed commonly by probabilistic fragility function which represents the conditional probability of structure being reaching or exceeding a predefined damage state, such as slight, moderate, or extensive damage given the measure of earthquake. [14]

Fragility Curves are function which represents the conditional probability of reaching or exceeding limiting damage conditioned on the full range of loads to which the structure might be exposed. Simply it can be defined as plot of probability verses ground motion parameter. The data derived from fragility curve could be used to analyze, evaluate and improve the seismic performance of the buildings and to evaluate the losses. It is developed from behavior model of structure. In order to asses seismic risk or potential losses of building availability of relevant fragility curve is essential and different methods can be employed or used to develop. It is commonly agreed to classify the methods in to four generic groups.

- Empirical curves based on observation of actual damage and post-seismic surveys; utilizes damage data collected in the field after earthquakes at different sites.
- Expert opinion-based curves, directly estimated by experts, or based on vulnerability index models that use expert judgment;

- Analytical or numerical curves, obtained from the results of static or dynamic analyses of structural models where no earthquake field data exists, or for which experimentation would be prohibitively expensive.
- Hybrid curves, which can combine any of the above-mentioned techniques, in order to compensate for their respective drawbacks.

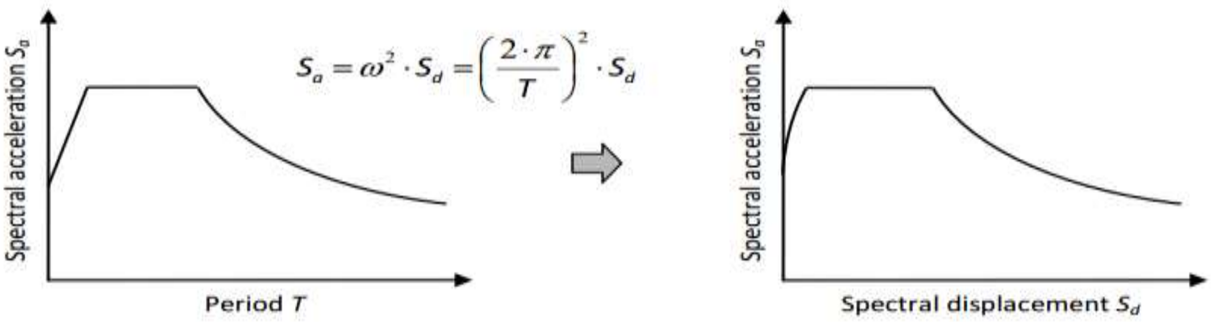
2.5.2.1 Fragility curve and damage state probability

When there is no field data observed or experimental data numerical analysis could be employed to develop fragility curve. The analysis can be repeated for different ground motion and structural configuration without much cost but the quality of the result will depend on ground motion and structural model used. Two different levels of analytical analyses are used; the simplified capacity spectrum method, and more detailed time-history analysis. The former one is used to estimate the response of a structure from spectrum demand and spectral capacity curve.

Building spectral capacity curve is defined on Hazus (US hazard) manual as a plot of building's lateral load resistance as a function of characteristic lateral displacement (i.e. a force –deflection plot) derived from a plot of static equivalent base shear versus displacement at roof (pushover curve) and demand spectrum represent 5% damped response spectrum which is reduced for effective damping when effective damping exceeds the 5% damping level of the input spectrum. In order to facilitate the comparison between two curves, base shear is converted to spectral acceleration and roof displacement is converted to spectral displacement. And in the same way the response spectrum converted from conventional Sa-T domain into the domain of capacity curve, i.e. spectral acceleration –spectral displacement (Sa-Sd). Every point on response spectrum curve has associated with a unique spectral acceleration S_a, velocity, S_v, displacement, S_d and period T. To convert from the standard S_a vs T format which found in building codes to acceleration –displacement response spectra (ADRS) format it is necessary to determine the value of S_{di} for each point on the curve S_{ai},-$T_{i.}$ Using the following equations:[15]

$$S_{di} = \frac{Ti^2}{4\pi^2} S_{ai} \tag{2-1}$$

Where S_{di} is spectral displacement, Ti is period and S_{ai} spectral acceleration



(a) Conventional representation of elastic Response spectrum (design spectrum)

b) Transformed response spectrum into S_a–S_d domain

Figure 2-2 Conversion of design response spectrum into Sa–Sd domain

To develop the capacity spectrum from pushover curve it is necessary to do a point by point conversation to first mode spectral coordinate .That mean any point base shear vs top roof displacement (V_i,Δ_{roof}) on the capacity curve is converted to the corresponding point spectral acceleration ,spectral displacement (Sa,Sd)on the capacity spectrum using the following equation:

$$S_{ai} = {}^{Vi}/_{W}\Big/_{\alpha 1} \qquad (2\text{-}2)$$

$$S_{di} = {}^{\Delta roof}\Big/_{(PF1X\text{Ø}1, roof)} \qquad (2\text{-}3)$$

Where α_1 and PF_1 are modal mass coefficient and participation factors for the first natural mode of the structure respectively and $Ø_{1roof}$ is the roof level amplitude of the first mode. ETABs software work with these data and give capacity curve in acceleration –displacement response spectra (ADRS).[15]

Therefore, building response is determined by the intersection of the demand spectrum and capacity curve.

Fragility curves take into account the uncertainty and variability related with building capacity curve properties, damage states and ground shaking. The fragility curves distribute damage among Slight, Moderate, Extensive and Complete damage States. For any given value of spectral response, discrete damage-state probabilities are calculated as the difference of the cumulative probabilities of reaching, or exceeding, successive damage states. The probabilities of a building reaching or exceeding the various damage levels at given response level sum to 100%. Discrete damage-state probabilities are used as inputs to the calculation of various types of building-related loss.

The conditional probability of being in, or exceeding, a particular damage state, k, given the spectral displacement, sd, (or other seismic demand parameter) is defined by Equation:

$$P[k/sd] = \phi[\frac{1}{\beta s}\ln(\frac{sd}{\bar{S}d,k})] \tag{2-4}$$

Where $\bar{S}d$,k = is the median value of spectral displacement at which the building reaches the threshold of damage state, k,

βs = is the standard deviation of the natural logarithm of spectral displacement for damage state, ds, and
Φ= is the standard normal cumulative distribution function.

FEMA/NIBS (National Institute of Building Sciences) damage state classification methodology provides detailed descriptions of the damaging effects to each different building typology and in addition, median values of threshold spectral displacements $\bar{S}d,k$ are provided for each damage state k, which are based on drift ratio δ_k (at the threshold of damage state k), fraction of the building height α_2 at the location of the first mode displacement, and typical roof level height H:

$$\bar{\mathrm{S}}\mathrm{d,k} = \delta_k * \alpha_2 * H \tag{2-5}$$

In addition in Hazus both mean displacement threshold of damage state and corresponding standard deviation are given in table. They depend on model building type and seismic design level. However due to difference in construction technique and detailing different countries used special care has be given.

Other damage classifications characterize the thresholds of damage states in terms of capacity curve characteristics, i.e. yield and ultimate spectral displacement, S_y and S_u, respectively. These classifications have been developed by Giovinazzi (2005), Barbat et al (2006) and Kappos et al. (2006). Their proposed damage classifications and damage state thresholds are listed in Table 2.2, respectively.[16]

Table 2.2 Damage state thresholds dependent on capacity curve parameters Sy and Su (Giovinazzi, 2005, Barbat et al., 2006, Kappos et al., 2006).

Median value of threshold spectral displacement $\overline{Sd}$,k						
Damage state	k	Giovinazzi (2005)	Barbat et al.(2006)	Kappos et al.(2006)	k	Damage state
Slight	1	$0.7S_y$	$0.7\ S_y$	$0.7\ S_y$	1	Slight
Moderate	2	$1.5S_y$	$1.0\ S_y$	$1.0\ S_y$	2	Moderate
Extensive	3	$0.5(S_y+ S_u)$	Sy+ 0.25(Su– Sy)	$2.0\ S_y$	3	Substantial to heavy
Complete	4	S_u	S_u	$0.7S_u$	4	Heavy to very heavy
				S_u	5	Collapse

Now each damage state k will have a corresponding fragility function, which is basically described by median values of threshold spectral displacements $\overline{Sd}$,k and a standard deviation β_s.

Uncertainty of the fragility parameters is estimated through the standard deviation, βtot, that describes the total variability associated with each fragility and obtained by combining three types of uncertainties.

- Uncertainty on the definition of damage state, β_{ds},
- Uncertainty on the structural response of elements (capacity curve). β_c
- Uncertainty On seismic demand (response spectrum). β_D

$$\beta_s = \sqrt{\beta_{ds}^2 + \beta_C^2 + \beta_D^2} \tag{2-6}$$

Where:-β_{ds} is the lognormal standard deviation parameter that describes the total variability of damage state ds,

β_C is the lognormal standard deviation parameter that describes the variability of the capacity curve,

β_D is the lognormal standard deviation parameter that describes the variability of the demand spectrum (values of β_D= 0.45 at short periods and β_D = 0.50 at long periods).

Hazus manual advocate the value of β_{ds}=0.4 for all damage state, β_c=0.25 for code compliant elements and β_c=0.3 for pre code construction.

2.5.3 Building exposure

Exposer is defined by Kishor et al as the valuables that could suffer losses as result of earthquake shaking.[17] The valuables exposed to the coming hazard could be economic or social and could include human lives and business revenue.

In order to develop seismic vulnerability and risk models classification system that characterizes the earthquake-exposed building stock and description of its damage are required. A complete set of data (i.e. covering the entire city) is able to be provided only by the National Statistics agency of the country.

In order to assess the loss and impact of earthquake on built up area, it is very important to know the structural system, their performance, engineering standards adopted during construction and location and distribution of vulnerable assets in the area. For instance in capital city of Ethiopia, the distribution of exposed assets and population before 10 or 15 years and now is too much incomparable. The dramatically increasing distribution of building and population will make the city to suffer largely from expected earthquake.

For assessing seismic risk at least the following building inventory and information are required

- Building location: geographical coordinate.
- Building occupants: number of people at different time scenario.
- Building size and replacement value.

2.6 Earthquake loss estimation

For planning and cost effective mitigation measures and insurance purpose reliable earthquake loss estimation (in monetary terms) of buildings struck by an earthquake is very important and in addition to that for definition of criteria for prioritizing seismic strengthening (rehabilitation) programs for existing buildings is also required. Decisions regarding the seismic rehabilitation of existing buildings require both engineering and economic studies and consideration of social priorities. Engineers and scientists have contributed to the development of a number of useful tools and methods designed to facilitate planning and mitigation to reduce future losses from earthquakes

2.6.1 Monetary loss

The building use will be major factor in estimating monetary losses for instance for the same level of shaking a residential and hospital building will not have equivalent amount of loss so taking this factor in to account will be very good in loss assessments. The total replacement value of a building will be the sum of construction cost and replacement value of its content. Again the construction cost is divided in to structural and nonstructural costs by means of repair cost ratio (RCS). This ratio represents the fraction of construction cost that is attributed to structural and non-structural component cost. So, monetary losses are determined by multiplying the structural damages by replacement value of respective component and its content.

In order to establish loss estimates in birr or any other currency, the damage state probabilities must be converted to loss equivalents of that birr or currency. Losses will be due to both structural and non-structural damage. For a given occupancy and damage state, building repair and replacement costs are estimated as the product of the floor area of each building type within the given occupancy, the probability of the building type being in the given damage state, and repair costs of the building type per square unit for the given damage state, summed overall building types within the occupancy.

For structural damage, losses are calculated as follows:

$$CS_{k,i} = BRC_i * \sum_{i-1}^{33} P[k / sd]_{,i} * RCS_{k,i} \tag{2-7}$$

$$CS_i = \sum_{k=1}^{4} CS_{k,i} \tag{2-8}$$

Where:

$CS_{k,i}$= cost of structural damage (repair and replacement costs) for damage state k and occupancy i

BRC: building replacement cost of occupancy i

$P[k/sd]_i$: probability of occupancy i being in structural damage state k,

$RCS_{k,i}$: structural repair and replacement ratio for occupancy i in damage

k and i refers damage states (slight, moderate ,extensive and complete) and occupancy class for this case only one occupancy class is taken for analysis.

2.6.2 Human loss/causalities

Estimating the number of life lost and injured is based on structural and non-structural damage to the building and the number of occupant present inside and outside of the building in the moment of hazardous event.

To calculate the number of causalities the population present in the building is multiplied by the probability of being in a certain damage state for a given size or intensity of earthquake and probability of an injury of certain severity level for a given damage state. The probability of an injury of a certain severity occurring given the damage sate is termed as causality rate. The causality rates are given in the form of tables for each damage states on Hazus manual.

For the purpose of vulnerability assessment FEMA defined damage states in to four whereas for the purpose of causalities estimation it define in to five damage state (slight, moderate, extensive, complete without collapse and with collapse) just to highlight the influence of building collapse and partial collapse. In addition four levels of injury severities are defined and showed below in Table 2.3. They range from injuries requiring no assistance to instantaneous death.

Table 2.3 Injury Classification Scale according to HAZUS.

Injury Level	Description	Examples
Severity 1	Injuries requiring basic medical aid that could be administered by paraprofessionals. These types of injuries would require bandages or observation.*	• Sprains • severe cuts requiring stitches • minor burns (first or second degree on a small part of the body) • Bumps on the head without loss of consciousness.
• Severity 2	• Injuries requiring a greater degree of medical care and use of medical technology such as x-rays or surgery, but not expected to progress to a life threatening status.	• bump on the head that causes loss of consciousness • fractured bones • dehydration or exposure
• Severity 3	• Injuries that pose an immediate life threatening condition if not treated adequately and expeditiously.	• punctured organs • other internal injuries • spinal column injuries • crush syndrome
• Severity 4	• Instantaneously killed or mortally injured.	•

* Injuries of lesser severity which can be self-treated are not covered by HAZUS.

Population distribution is another important parameter in estimating causalities. In case of that no detail information on population distribution is available or cannot infer from available data optional basic methodology can be used. This methodology is proposed by the earthquake loss estimation tool SELENA (SEismic Loss EstimatioN using a logic tree Approach). This the tool follows fully Hazus's approach.[18]

Table 2.4 population percentages indoors and outdoors dependent on the time of the day[18]

Occupancy class	Night (2:00am)	Day (10:00am	Commuting (5:00pm)
Indoor	98%	90%	36%
outdoor	2%	10%	64%
sum	100%	100%	100%

Table 2.4 shows the population distribution proposed by SELENA as an option to use when there is no detail information and census data. These values are dependent on the country's culture and consequently may vary. This may not be applicable for our country to develop population distribution applicable for us needs intensify research. However, just to study methodology for seismic risk evaluation one can use developed population distribution. Of course Hazus manual population distribution can be used but more or less it needs more detail information. The Hazus methodology classified the total population in to five different groups for the three times of the day. The default distribution for residential occupancy is shown below in Table 2.5.

Table 2.5 Default Relationships for Estimating Population Distribution of Hazus

	2:00a.m		2:00p.m		5:00p.m	
% of people at home	99% NRES		75%DRES		50%NRES	
	In door	outdoor	In door	outdoor	In door	outdoor
	99.9% of 99 NRES	0.01% of 99 NRES	70% of 75% DRES	30% of 75% DRES	70% of 50% DRES	30% of 50% DRES

Where NRES is night time resident and DRES is day time resident and this table can be read as, for instance the 1st column state that 99% of night time residents are expected to be at home, 99.9% of these occupants are expected to be in doors and the remaining 0.01% will be outdoor. Since there is no census data of number of occupants at day and night time of the study building an assumption is made and the above default distribution is taken to estimate the number of causality.[6]

The number of casualties due to direct structural damage for any given structural type, level of building damage, and injury severity can be calculated by:

$$K_i^s = \left(injuries(severity)i = \sum_{j=1}^{N_{BT}} \sum_{k=1}^{N_{DS}} C_{j,k}^{CSR} P_{j,k} N_j^{POP} \right) \tag{2-9}$$ [2]

Where

$C_{j,k}^{CSR}$ = Causality rate of severity i for damage state k

$P_{j,k}$ = Structural damage probability for damage state k

N_j^{POP} =Number of people in the model building type j

The total number of people in all buildings of model building type j, for one geographical unit at a specific time period (time of the day), is computed in a simplified way:

$$N_j^{POP} = N^{TP} C^{PO} C_j^{OMBT} \tag{2-10}$$ [3]

Where:

N^{TP}= total number of people living in the respective geographical unit

C^{PO} = Percentage of people staying indoors or outdoors dependent on the time of the day

C_j^{OMBT} = Percentage of population by occupancy class for jth model building type

2.7 Seismic Risk Assessment Methodology

There are a lot of research works done by different researchers in different country on seismic risk assessment of buildings for both existing and as well as new one. Some works are reviewed and summarized as follows and this review is by no means extensive, but it can give the reader a sense of the type of work that has been done in different regions.

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[2] SELENA v6.5 User and Technical Manual v6.5

[3] SELENA v6.5 User and Technical Manual v6.5

2.7.1 HAZUS Risk Assessment Methodology

HAZUS (HAZards U.S.) is a standardized software package developed for the Federal Emergency Management Agency (FEMA) to assess the multi-hazard loss within United States by the National Institute of Building Sciences (NIBS). It is nationally applicable earthquake loss estimation methodology (NIBS). The package includes other hazards like floods and hurricanes. It is GIS (Geographic information system) based software which deliver map and display hazard data and result of probable risk estimate for buildings and infrastructures. The software is very reach in building inventory data and groups buildings with similar characteristics and classifies them in predefined building class. This methodology is more of generic there is another module called Advanced Engineering Building Module (AEBM) which is developed to assess building specific damage and loss function. The AEBM procedures are an extension of the more general of FEMA earthquake loss estimation methodology.

On this methodology buildings are classified in terms of use, occupancy class and structural system then damage is predicated based on model building type. The buildings are classified into five structural framing such as wood framing, steel framing, concrete framing, reinforced concrete framing and unreinforced concrete framing. These framed structures are further classified into 36 different structural classes based on their structural design and material used(all 36 structural classes are found listed on Hazus technical manual on table 3.1).[10] Figure 2.3 shows the flow chart of HAZUS methodology for earthquake risk assessment (ERA) of model building type. The methodology is divided into the seven steps. In the first step, input requirement are shown. The second and third steps shows the parameters required to generate the response curve and capacity curve respectively. The output from second and third steps is peak building response. It is calculated from the intersection of these two curves. The output of fourth step is used to calculate the cumulative probabilities of model building type, shown in step 5. The sixth step shows the calculation of discrete probabilities for all four damage states and finally the damage matrix is developed in step7.

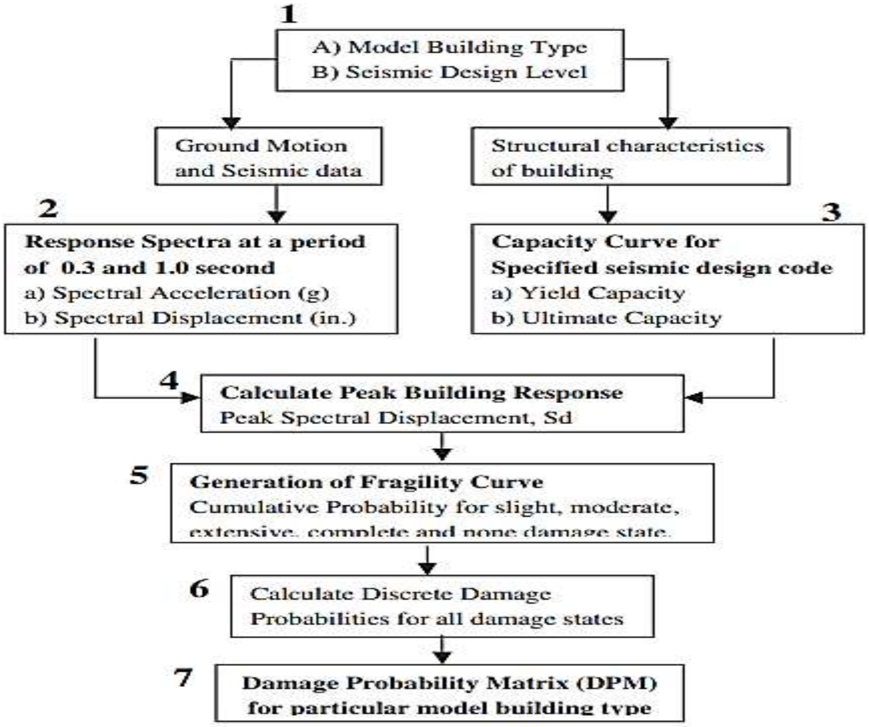


Figure 2-3 HAZUS methodology flow chart

The first thing in this methodology is selecting model building including all features of that building. Building response will be determined by the intersection of the demand spectrum (which is based on 5% damped response spectrum at the building site reduced for effective damping) and building capacity curve. Building capacity curve represent the level of lateral force for a given loading condition or it is an expected performance of that building. This curve characterized by three points: design capacity, yield capacity, and ultimate capacity. To compare the earthquake demand and the capacity of the building the pushover curve converted to spectral acceleration versus spectral displacement axis and can compare easily. Having the peak spectral displacement of the building, which is the intersection point of the capacity curve and demand curve, able to estimate the damage state probabilities. In estimating cumulative damage probabilities two values are first calculated median value of spectral displacement for each damage state and value of lognormal standard deviation (β) for damage state. Finally discrete damage probabilities will be calculated as shown below and Damage Probability Matrix (DPM) for model class will be developed, example of DPM is shown in Table 2.6 below .[10]

Calculate the discrete damage probabilities

Probability of complete damage, P [C] $= P [C|S_d]$

Probability of extensive damage, P [E] $= P [E|S_d] - P [C|S_d]$

Probability of moderate damage, P [M] $= P [M|S_d] - P [E|S_d]$

Probability of slight damage, P [S] $= P [S|S_d] - P [M|Sd]$

Probability of no damage, P [None] $= 1 - P [S|S_d]$

Table 2.6 Example of Damage Probability Matrix (DPM)

Damage probability Matrix				
Model type	Slight	Moderate	Extensive	Complete
Probability	P[S]	P[M]	P[E]	P[C]

The steps followed by this methodology is summarized diagrammatically below in Figure 2.4 .The capacity of the building with the intensity of the hazard will tell the exact performance of the building, with this performance capacity of the building probability of different damage states will be determined. This all process depends on the quality of analysis and modeling.

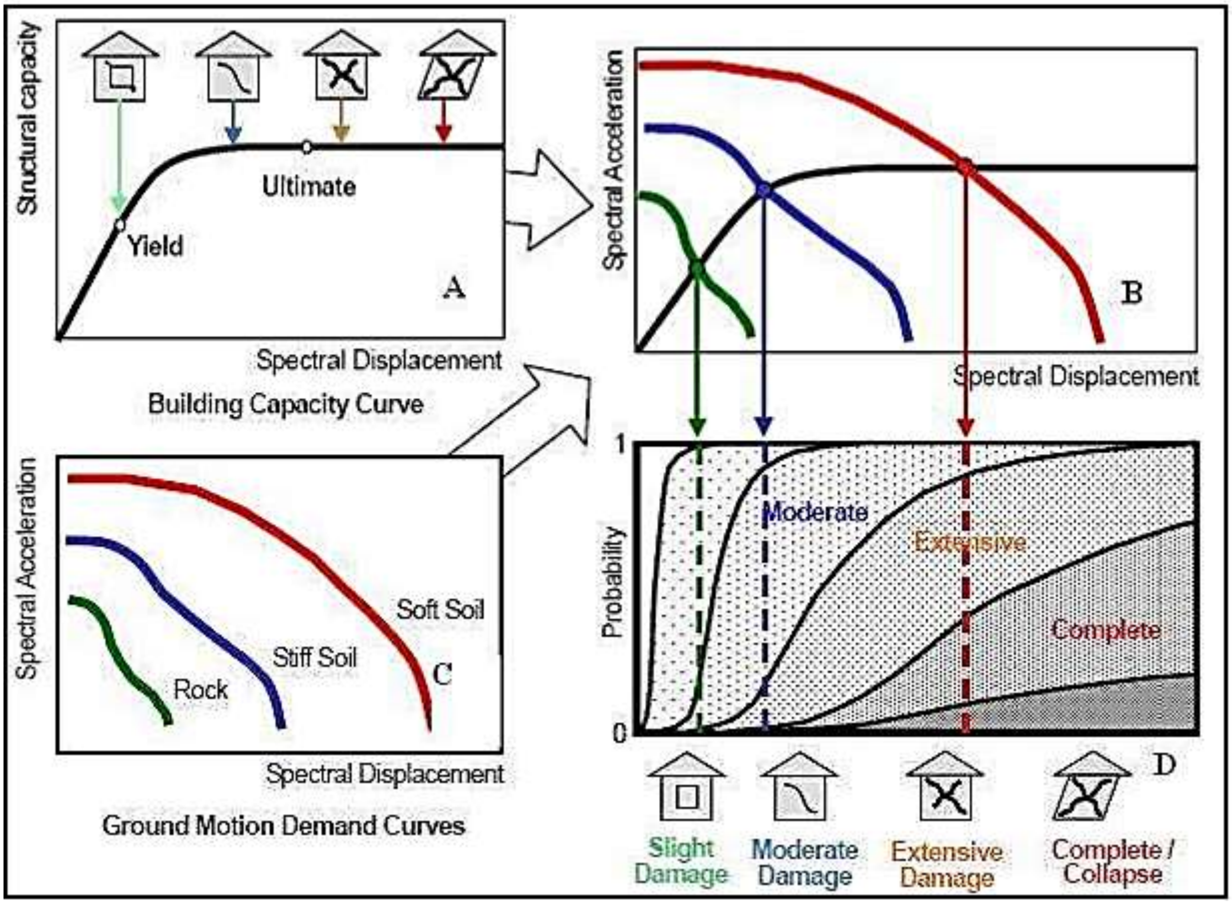


Figure 2-4 Schematic diagram of HAZUS method [10]

CHAPTER 3 CASE STUDY

3.1 Introduction

In this chapter following HazUs manual seismic risk assessment method is discussed for nine floor reinforced concrete building which is considered as representative of 43 similar blocks. This case study mainly focuses on the assessment of structural damage and human loss. A description of the site location/zone and the configuration of the study building, the parameter taken for analysis, member sizes and material properties used in the analysis and the procedure followed are described.

Figure 3-1 Case study building.

3.2 Modeling and building description

In this thesis work nine floor building is considered to represent high-rise RC buildings for the study. This building can be taken as representative of group of buildings blocks at different area of Addis Ababa city. The building could be classified under building model type of HazUS building classification system. It consists of different section size beam-column Rc frame building with shear walls located in moderate –seismicity region. Since it is an existing building as built in characteristics of the building is modeled and analyzed using ETABS 2016 integrated building design software version 16.2.1 Model is formed using finite element for beams and columns. Floor construction is not modeled and load transfers carried out manually with help of Microsoft excel (attached as appendix), but rigid floor diaphragm effect was considered. The seismic parameters and property of building considered are listed below.

Table 3.1 Properties of the study building

Floor height	3.3m
Total height	35.35m (High rise building)
Categories of use	Category A : Areas for domestic and residential activities prEN 1991-1-1:2001 Table 6.1 - Categories of use
Beam size	200 x 400 mm, 250x 500 mm,300 x 500 mm, 250 x 400 mm.
Column size	700 x 700 mm, 600 x 600 mm,500 x 500 mm, 400 x 400 mm, 500 x 400 mm
Slab type thickness	160mm, 170mm, Ribbed and solid slab
Wall thickness	300mm , 250mm
Support	Fixed

Table 3.2 Parameters used in the computation of seismic action

Input	Value	Code and reference
Behavior factor q	1.5	AS per section 5.3.3 ES EN 1998-1:2015
Seismic class IV	1.4	Section 4.2.5 of table 4.3 ES EN 1998-1:2015
Ground acceleration (ag)	0.1g	Annex D of ES EN 1998-1:2015
Soil type	C	Section 3.1.2 of ES EN 1998-1:2015
Spectrum type	1	Section 3.2.2 of ES EN 1998-1:2015

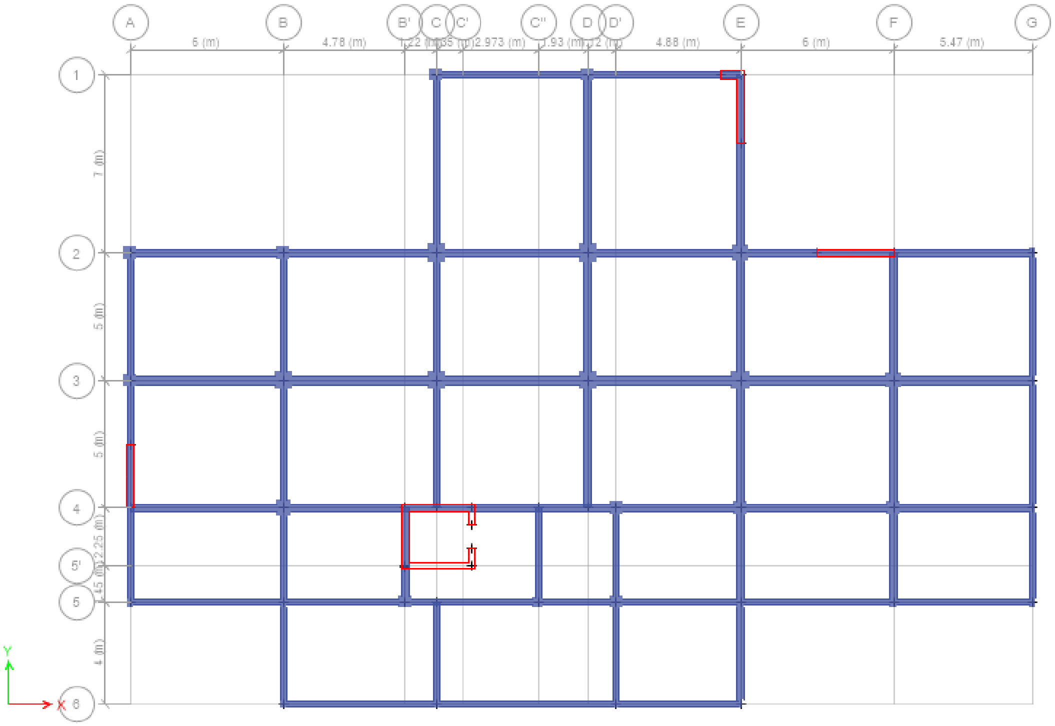


Figure 3-2 Plan view of the model: first floor plan

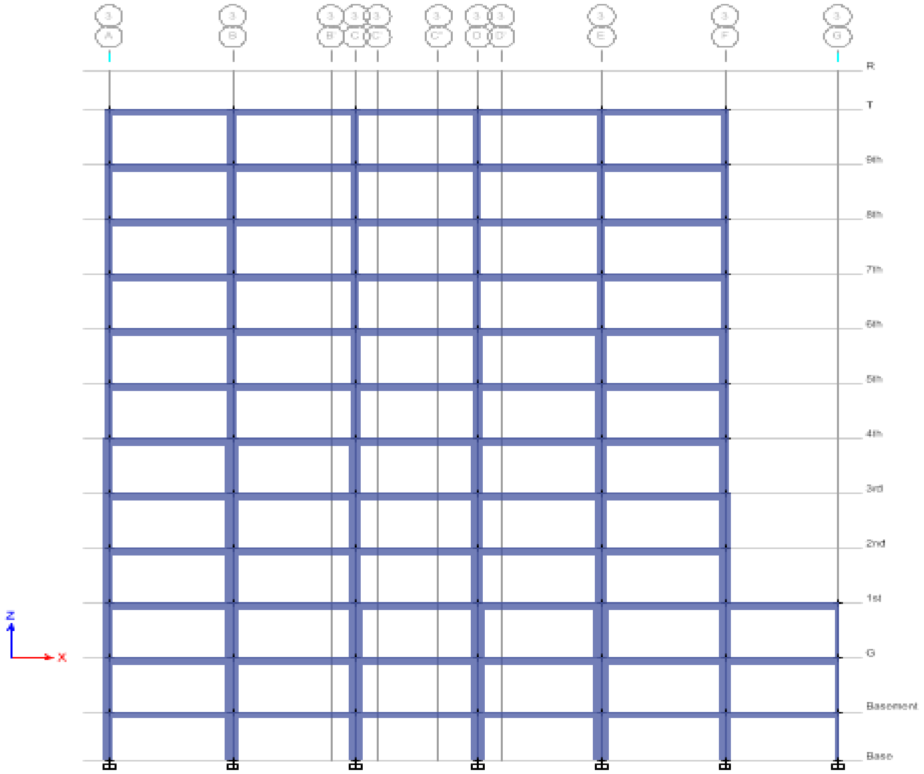


Figure 3-3 Elevation view of the model: 9 story building

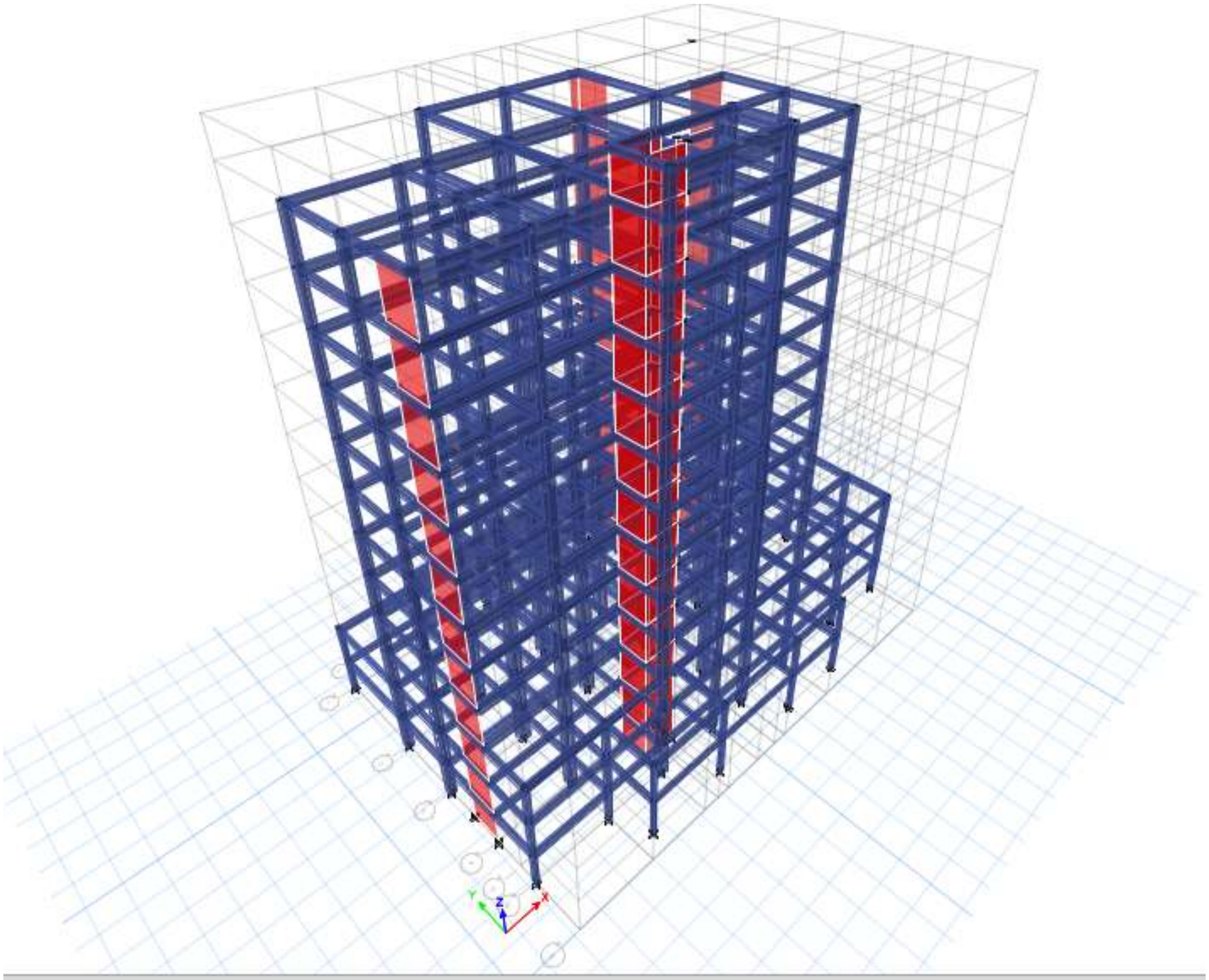


Figure 3-4 3D view of the model: 9 story building

Table 3.3 Material properties used

Grade of concrete	C-25 and C-30
Density of reinforced concrete	25kN/m3
Grade of steel	G-60 for Φ≥12 mm G-40 for Φ (6-12) mm bars
Density of reinforcing steel	7850 kg/m3
Modulus of elasticity of steel (E)	200GP

3.3 Seismic Hazard of the Study Area

As discussed under section 2.5.1 seismic hazard analysis that evaluates the intensity of ground shaking at a given location can be determined either as probabilistic seismic hazard analysis (PSHA) or deterministic seismic hazard analysis (DSHA). Since these analyses are beyond the scope of this thesis and in addition to that it requires knowledge of earth science. For the purpose of this work the seismic hazard defined by the national code is used.

The national code subdivided national territories into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant. The hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, a_{gR}.

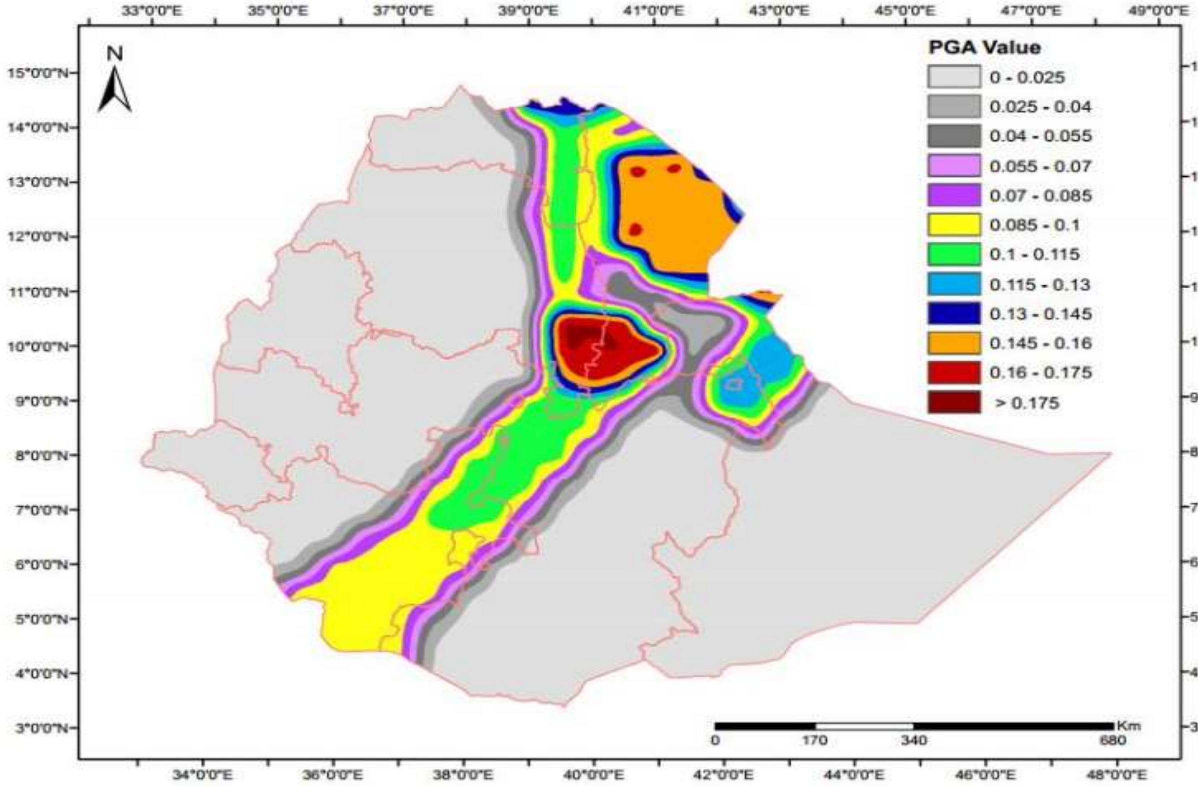


Figure 3-5 Ethiopia's Seismic Hazard map in terms of peak ground acceleration

In the present study, the response of the building is studied in Addis Ababa which categorized as seismic zones III of Ethiopia as per EBCS EN 1998-1:2014 Ethiopian building code.

Table 3.4 Bedrock Acceleration Ratio αo (EBCS EN 1998-1, 2014)

zone	5	4	3	2	1	0
α_o=ag/g	0.2	0.15	0.1	0.07	0.04	0

3.3.1 Representation of the seismic action

In the context of Euro code 8 (EC8) as well as in EBCS-8 the earthquake motion at a given point of the surface is represented, for a reference return period, by the elastic response spectrum Se(T), where T is the vibration period of a linear single degree of freedom system. The elastic response spectrum is defined by the values of parameters ag , the design ground acceleration for the reference return period, β_0 , the spectral acceleration amplification factor for 5% viscous damping, T_B and T_C , the limits of the constant spectral acceleration branch, T_D , the value defining the beginning of the constant displacement range of the spectrum, k_1 and k_2 , the exponents which influence the shape of the spectrum for a vibration period greater than T_C and T_D , respectively, S, the soil parameter and η the damping correction factor with reference value of 1 for 5 % viscous damping.

Depending on the characteristics of the most significant earthquake contributing to the local hazard Type 1-High and moderate seismicity regions (M_s>5.5) is used.

Table 3.5 Parameters describing Type 1 spectrum (EBCS EN 1998-1:2014)

Ground type	soil factor	TB (Sec)	Tc (Sec)	TD(sec)
A	1	0.15	0.4	2
B	1.2	0.15	0.5	2
C	1.15	0.20	0.6	2
D	1.35	0.20	0.8	2
E	1.4	0.15	0.5	2

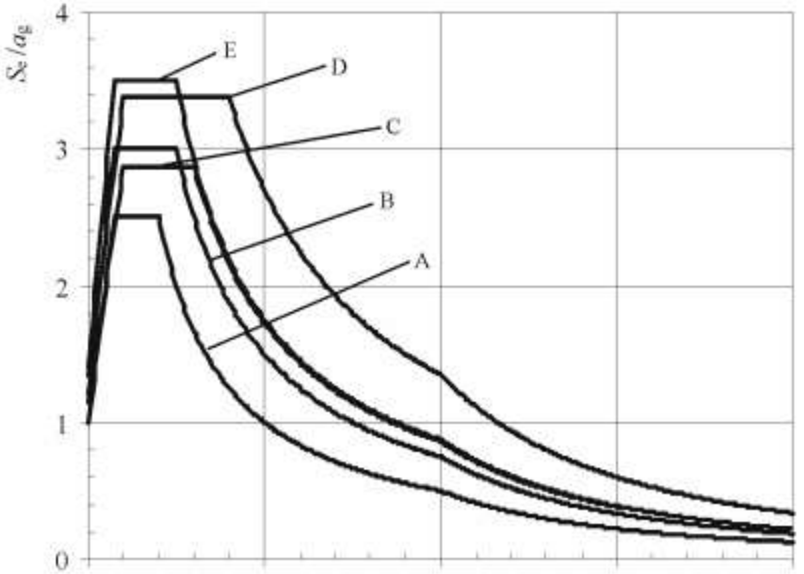


Figure 3-6 Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

The horizontal components of the seismic action the design spectrum, Sd(T), defined by the following expressions:

$$0 \le T \le TB : sd(T) = ag.S[\frac{2}{3} + \frac{T}{TB}.[\frac{2.5}{q} - \frac{2}{3}]] \tag{3-1}$$

$$TB \le T \le TC : sd(T) = ag.S.[\frac{2.5}{q}]$$

$$TC \le T \le TD : sd(T) = ag.S[\frac{2.5}{q}.\frac{TC}{T}] \ge \beta.ag$$

$$TD \le T :: sd(T) = ag.S[.[\frac{2.5}{q}.\frac{TCTD}{T^2}] \ge \beta.ag$$

Where $S_d(T)$ is the design spectrum;
q is the behavior factor;
β is the lower bound factor for the horizontal design spectrum.(The recommended value for β is 0,2.is taken)
T_B is the lower limit of the period of the constant spectral acceleration branch;
T_C is the upper limit of the period of the constant spectral acceleration branch;
T_D is the value defining the beginning of the constant displacement response range of the spectrum;
S is the soil factor;

3.3.2 Lateral force method of analysis

Depending on the structural characteristics of the building for this study equivalent elastic analysis is used. The seismic base shear force F_b, for each horizontal direction in which the building is analyzed is determined by the following expression

$$F = Sd(T1).m.\lambda \tag{3-2}$$

Where Sd (T1) is the ordinate of the design spectrum at period T1;

T1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

m is the total mass of the building, above the foundation or above the top of a rigid basement

λ is the correction factor, the value of which is equal to: λ= 0,85 if T1<2 TC and the building has more than two storeys, or λ= 1,0 otherwise.

For the determination of the fundamental period of vibration period $T_1$of the building,

$$T_1 = C_t.H^{\frac{3}{4}} \tag{3-3}$$

Where ;- Ct is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

The horizontal seismic forces Fi distributed to all story determined as

$$F_t = F_b \frac{Z_i . m_i}{\sum Z_j . m_j} \tag{3-4}$$

Where: F_i is the horizontal force acting on story i;

F_b is the seismic base shear

Z_i, Z_j are the heights of the masses $m_{i,}$ m_j above the level of application of the seismic action (foundation or top of a rigid basement).

ETABS is extremely powerful and productive structural analysis and design program, partially due to the high level of intelligence embedded within the software that means many of the capabilities are highly automated, allowing the user to create and analyze the models in such a way that is both natural and efficient for a structural engineer. As a result, automated lateral load generation is used for this study. The data inputted for auto lateral load calculations are described below. Since the building was designed for low ductility class the behavior factor taken as 1.5 according to code provision.

Table 3.6 Parameter inputted for auto lateral load generation

Ground Acceleration ag/g	0.1
Spectrum Type	1
Ground type	C
Soil factor, S	1.15
Spectrum Period Tb	0.2
Spectrum Period Tc	0.6
Spectrum Period Td	2
Lower Bound factor Beta	0.2
Behavior Factor	1.5
Correction factor, Lamda	1

3.3.3 Load Combinations

As recommended by the code the following load combinations are used for analysis.

$$\mathrm{D}L + LL \tag{3-5}$$

$$1.35EL + 1.5LL$$

$$\mathrm{D}L + 0.3LL \pm \mathrm{EQX1} \pm 0.3\mathrm{EQY1}$$

$$\mathrm{D}L + 0.3LL \pm \mathrm{EQX1} \pm 0.3\mathrm{EQY2}$$

$DL + 0.3LL \pm EQX2 \pm 0.3EQY1$

$DL + 0.3LL \pm EQX2 \pm 0.3EQY2$

$DL + 0.3LL \pm 0.3EQX1 \pm EQY1$

$DL + 0.3LL \pm 0.3\ EQX1 \pm EQY2$

$DL + 0.3LL \pm 0.3EQX2 \pm EQY1$

$DL + 0.3LL \pm 0.3\ EQX2 \pm EQY2$

3.4 Nonlinear Static Pushover (NSP) analysis methods

An elastic analysis cannot determine post-elastic behavior of structure, because structures are expected to deform in elastically when subjected to severe earthquakes, So seismic performance evaluation of structures during post-elastic behavior could be conducted by nonlinear analysis procedures. In this study nonlinear static pushover analysis method is used to capture inelastic behavior of the structures under earth quake loading.

A pushover analysis is a series of incremental static analyses carried out to develop a capacity curve for the building. With the increase in the magnitude of the loads, weak links and failure modes of the building are found. The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement.

In the present work, pushover analysis of the structure is done to obtain the pushover curve of the structure. The lateral loads for pushover analysis were determined based on the code recommendation. Pushover load case called Push is used to apply in direction of x and made to start from the final condition of gravity loads. Both the pushover curve and capacity spectrum are generated from the analysis. The point of intersection of capacity spectrum and reduced response spectrum is taken as the performance point.

3.4.1 Considerations of plastic hinges

Nonlinear static pushover analysis capabilities are provided in the nonlinear version of ETABS and in the implementation of pushover analysis, modeling is one of the important steps. The model must consider nonlinear behavior of structure and its elements. The nonlinear behavior occurs in discrete user defined or default hinges and can be introduced in to both frame and vertical wall objects. Hinges may be assigned at any location along the frame element but are restricted to mid –high in the wall objects. Nonlinear wall hinges assignment may only be made to wall that made of concrete material with geometry that is vertical, planer and rectangular in shape. As there are different types of modeling for wall, hinges may not be assigned to walls with modeling types of layered. In this thesis, all plastic hinge aspects are approached as considered relevant to the analysis.

Nonlinearity modeling can be concentrated plastic hinge models, distributed plastic hinge models, or other models whose nonlinear behavior has been demonstrated to adequately represent. For this thesis, the chosen method to model nonlinearity is through discrete flexural plastic hinges because it is commonly used in academia and practical design.

The location of a hinge is specified as a relative distance along the member, chosen by the user. It is important to ensure that the location of the hinges is representative of where the moment demands are greatest so that the hinges will be activated appropriately and the structure will perform as expected and realistically. To be consistent with the intended first-mode, strong column weak-beam hinge mechanism. Plastic hinges were assigned at the ground level for columns and at each end (i.e. beam-column faces) of all beams. Since everything below grade is expected to remain elastic, hinges are not modeled beneath the ground level.

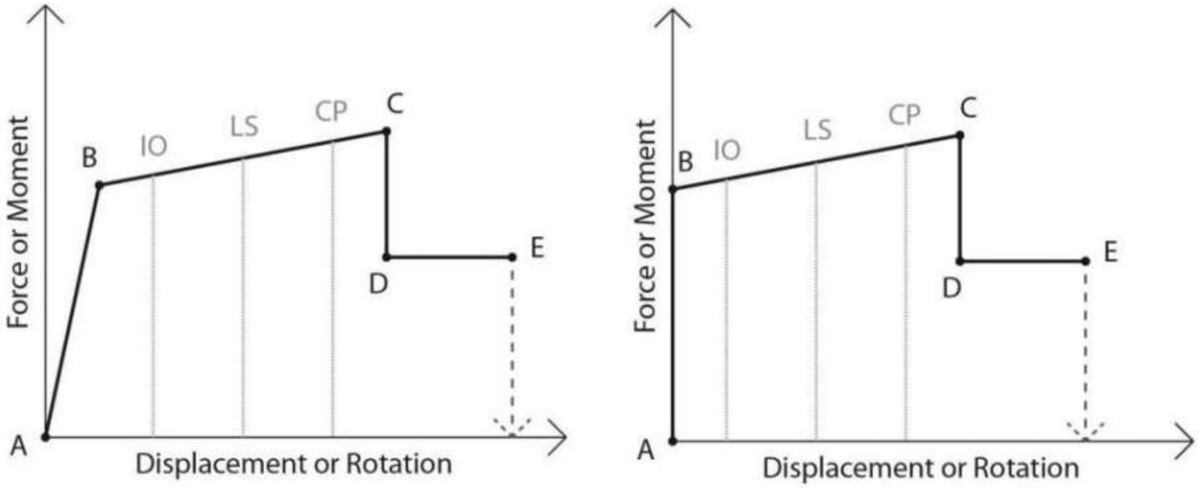
There are three types of hinge properties in ETABS. They are default hinge properties; user defined hinge properties and generated hinge properties. The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. Only default hinge properties and user -defined hinge properties can be assigned to frame elements. These built in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses. For this study User-defined hinge properties are implemented.

User-defined hinge properties are based on plastic force displacement (FD), for axial and shear degrees of freedom, or moment-rotation (M-θ), for bending and torsion moment degrees of freedom, relationships. Fully user-defined properties allow for symmetric F-D/ M-θ backbone curves, or the curves can be different in the negative and positive directions. To define the plastic deformation curve, several points must be input into ETABS. Figure 3.6 (a) shows the points on the plastic deformation curve and while the shape shown is typically used for pushover analysis, any shape can be defined as long as they comply with the following point definitions.

Point A is always the origin and cannot be changed. From Point A to Point B, all deformation is linear and occurs inside the frame element. Point B represents yielding, but in the ETABS version used, the hinges are rigid-plastic, so the deformation at B is always defined as zero, as shown in Figure 3.6 (b). Thus, no deformation occurs in the hinge up to Point B and only plastic deformation beyond B will be exhibited by the hinge. For a typical pushover analysis, Point C should be defined as ultimate capacity and Point D represents residual strength. For other purposes, a positive slope can be defined from C to D and/ or D to E. For any shape of backbone curve, Point E represents total failure. Beyond E, the hinge will drop the load to zero force/ moment as shown in the Figure 3.7. Additionally, the moment-rotation backbone curves were chosen to be symmetric, such that the absolute value of each point coordinate in the negative direction is the same as the positive direction coordinate.

In addition to the five points required for defining the plastic deformation backbone curve, the user has the option of specifying the deformation values at the Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) levels used for performance-based design. These measures are used only as recorders for obtaining analysis results at the defined points, and do not affect the structure's behavior. As such, they can be used to measure other points of interest and are not required to be specified.

Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 and M3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupled P-M2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location. Also, more than one type of hinge can be assigned at the same location of a frame element.



(a) (b)

Figure 3-7 a) Plastic deformation backbone curve and b) Actual rigid-plastic deformation curve used for hinges

User-defined hinges were used to model material nonlinearity for the structure under study because the default hinges were not able to accurately account for confinement without the transverse steel modeled. Since all hinges intended for use in this thesis are purely flexural hinges, the bending moment-rotation relationships were obtained for each element. For the vertical walls default fiber hinge is assigned at mid-high of the wall

For this study the calculation of user-defined hinge properties of the members, the moment-curvature relationships for the beams and columns were determined using SAP2000 Section Designer. Moment curvature values are generated based on material model for concrete (confined or unconfined) and steel by using equilibrium and compatibility equations for each element. Curvature values are multiplied with the length of the plastic hinge to get the rotation values. Some authors have proposed various expressions in order to establish the correct hinge length. Park and Paulay presented the following expression and used for this study in which Lp is the plastic hinge length and h is the section height:

$$L_p = 0.5 * h \tag{3-6}$$

The parameters used for the generation of moment curvature and sample hinge properties of beams and columns are presented in Tables 3.7 through 3.9.

Table 3.7 Parameters used for moment-curvature generation for ground Column

Axis	Longitudinal Rebar	Stirrup	Axial force (kN)	Section (mm)
5B’	14Φ20	Φ10/110	-2013.32	500*500
3C	16Φ20	Φ10/110	-6821.98	700*700

Table 3.8 Parameters used for moment-curvature generation for typical beams

Axis	Joint	Section (mm)	Longitudinal		Stirrup
			Top	Bottom	
4	A	300*500	3Φ16+3Φ20	4Φ16	Φ10/90
	B	300*500	3Φ16+3Φ20	4Φ16	Φ10/90
	C	300*500	3Φ16+3Φ20	4Φ16	Φ10/90
	D	300*500	3Φ16+3Φ20	4Φ16	Φ10/90
	E	300*500	3Φ16	6Φ20	Φ10/90
	F	300*500	3Φ16+4Φ20	6Φ20	Φ12/100

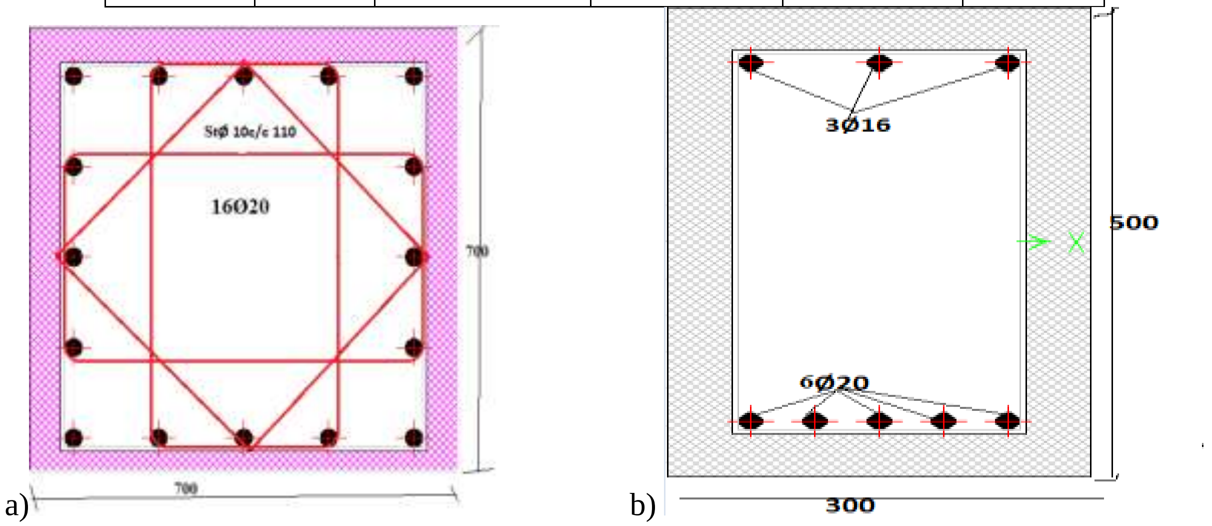


Figure 3-8 a) Sample column cross-section b) Sample Beam cross-section

Table 3.9 Sample beam moment curvature results for hinge property

Id	Section	my	ky	mu	ku
FFB1	B+,B'+,C+,D'	60.419	0.007465	70.996	0.1742
FFb-2	A+,B,E,F,G	76.732	0.00551	93.613	0.1571
	B',D'	128.056	0.006138	149.405	0.1803
FFB-3	A+,	193.084	0.006345	226.505	0.112
	B,	195.9	0.006125	233.883	0.1162
	C-	225.109	0.006608	251.496	0.0667
	C+	299.961	0.006863	323.79	0.0452
	D',E	181.06	0.006117	209.004	0.0799
	F	195.938	0.00623	224.271	0.0697
	G	85.451	0.005599	118.534	0.1697

Table 3.10 Sample column moment curvature results for hinge property

Id		Section	Axial load	my	ky	mu	ku
C-1/7	G-1st	500*500	-3010	985.181	0.007784	1028.36	0.0151
	B-G		-3380.71	1023.006	0.008214	1047.71	0.0137
	Base-B		-3604.08	1038.423	0.008543	1053.48	0.013
C-1/14	G-1st	500*500	-2068.84	860.2	0.006879	953.933	0.0191
	B-G		-2492.88	919.206	0.007268	989.915	0.0169
	Base-B		-2775.88	956.886	0.007546	1011.76	0.0159
C2/9	G-1st	500*500	-890	358.09	0.007434	410.99	0.0336
	B-G		-1117.44	390.211	0.007789	439.844	0.029
	Base-B		-1210.98	403.704	0.007943	449.189	0.0269
C3/19	G-1st	700*700	-5474.08	1432.26	0.00769	1436.78	0.0102
	B-G		-6247.86	1427.53	0.007818	1436.72	0.00912
	Base-B		-6745.3	1399.103	0.007355	1412.73	0.008306

3.8 Damage/vulnerability estimation

As described in section 2.5.2 there are different approaches for earthquake damage assessment method. When there is no observational and experimental data the analytical/numerical approach which is based on the theoretical simulation (i.e. prediction) of structural damage under earthquake loading is used. Therefore, in this study numerical approach is chosen.

Building vulnerability is expressed in terms of a capacity curve that represents the nonlinear behavior of the structure under lateral displacement. In this study to identify a capacity curve, which is defined as the relationship between the base shear force and the lateral displacement of a control node of the building a nonlinear structural analysis method (pushover analysis method) is used.

As described in above section 2.5.2 the seismic ground motion (or seismic demand), i.e. accelerations and displacements to correlate with the capacity curve converted from the (conventional) Sa–T domain into the domain of the capacity curve, i.e. spectral acceleration–spectral displacement domain (Sa–Sd) ADRS (acceleration displacement response spectrum) format. Under section 2.5.2.1 .how this conversion is carried out is discussed briefly.

In order to predict analytically the structural damage that a building of a given capacity will produce under a given seismic impact, different methods are available among which Capacity Spectrum Methods (CSM) is selected because it has received the greatest attention to date, mainly because these procedures were published as various FEMA provisions and established the basis for FEMA's HAZUS methodology.

The chosen procedure, CSM, helps to obtain the peak displacement (or performance point) dp. This displacement stands for the mean displacement a building typology under study reach under the respective seismic demand. Hence, it represents the mean damage individual buildings of this building typology will experience. Having this value the corresponding damage probabilities are calculated by fragility functions for damage states k, which are closely connected to the capacity curve of the respective building typology as follow.

3.8.1 Fragility function/vulnerability

For development of realistic fragility functions which incorporate the distinct uncertainties from the geometrical building model, material parameters, seismic demand etc. damage classification and quantification concept comes first. In this work the fragility curves obtained based on capacity spectrum curve which was developed using pushover analysis.

As discussed in section 2.5.2 for purpose of assessing structural damage FEMA's four damage state definition are taken and median value of threshold spectral displacement $\bar{S}d$,k proposed by Barbat et al is used for this study due to its simplicity and commonly used to estimate the damage states from capacity curve.

Therefore, the cumulative probability of reaching or exceeding damage state k or a cumulative lognormal function with respect to the spectral displacement at the performance point is determined from the following equation.

$$P[k / sd] = \phi[\frac{1}{\beta s}\ln(\frac{sd}{\overline{Sd},k})] \qquad (3\text{-}7)$$

Initially to evaluate median value $\overline{Sd}$,k of different threshold damage states the yield and ultimate deformation parameter has to be determined. The capacity spectral curve is used to obtain yield spectral displacement and ultimate displacement. To determine those values among different definitions by different authors the definition by park is used. Park defined yield point deformation as point of equivalent elasto-plastic system with reduced stiffness computed as the secant stiffness at 75% of the ultimate lateral load of the real system. And for ultimate deformation definition by the same author deformation which corresponds to the apex of the load-displacement curve is taken.[19]

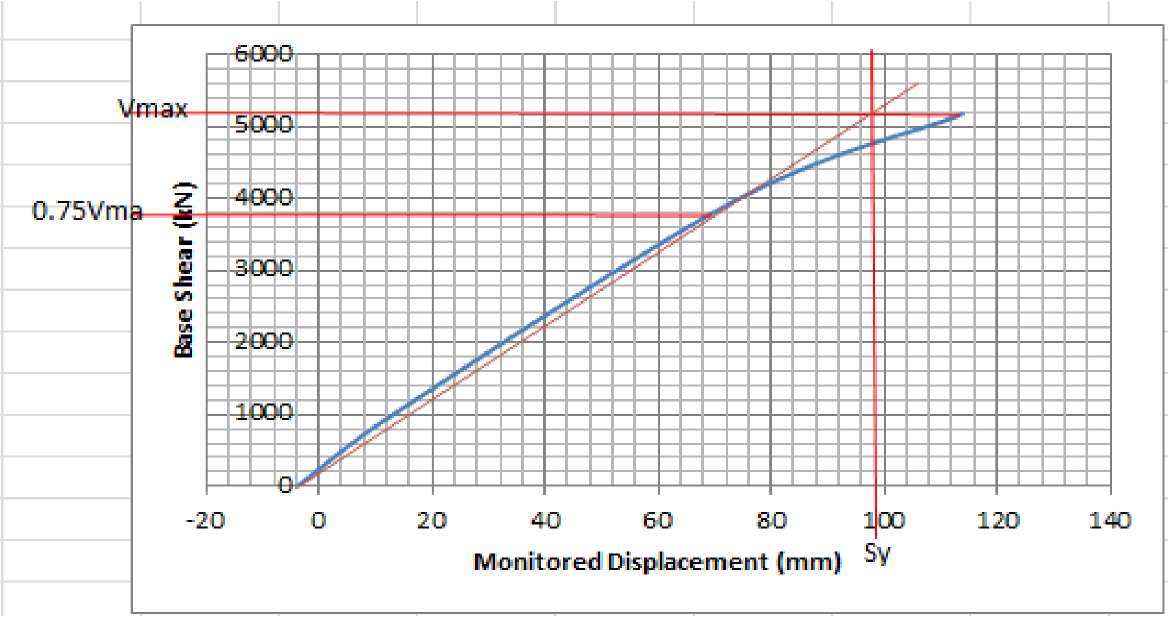


Figure 3-9 Definitions of yield deformation, based on reduced stiffness equivalent Elastic-plastic yield (Park, 1988)

Therefore from the above curve result yield displacement and ultimate displacement (98mm and 113.89mm) are found and used for evaluation of median value $\overline{Sd}$,k. Having median value and log standard deviation it is possible to develop the fragility curve or cumulative probability for each damage states.

The lognormal bata or standard deviation β_s which describes the total variability of the damage states can be taken directly from tables given in Hazus technical manual by choosing appropriate values of degradation or kappa factors and β_C and β_d values for different types of buildings .However ,as described in section 2.5.2 for this work the recommended values are taken.

3.9 Earthquake loss estimation (ELE) method

3.9.1 Monetary losses

The economic losses for building repair (mostly for damage states slight and moderate) and replacement (mostly for damage states extensive and complete) are computed in the following way:

Seismic economic loss can be caused by direct economic loss due to structural and nonstructural damage. In the current study only the direct economic losses caused by structural damage are computed. Those being caused by non-structural damage (acceleration sensitive damage) and indirect economic loss like loss due to business interruption are not considered.

For this study to estimate the direct economic loss of a building damage due to earthquake FEMA (Federal Emergency Management Agency) loss estimation methodology is followed.

As discussed in section 2.6.1 to evaluate monitory loss, repair cost ratio which will distribute the construction cost to structural and non-structural cost is required. This cost ratio may differ for different country based on their construction techniques. However, since there is no previous study in our country repair cost ratio which is exactly applicable to our country could not be found. Therefore, for this study appropriate default values for the structural repair cost ratios among different occupancy classifications from HazUs manual is taken. According to the data found the building under study costs 14,739,376.49 birr. This cost was estimated before 8 years (2012 G.C).

Table 3.11 Structural Repair Cost Ratios (in % of building replacement cost)[6]

Occupancy Class	Structural Damage state			
	Slight	Moderate	Extensive	Complete
Multi Family dwelling	0.3	1.4	6.9	13.8

Since the study building is residential building from Hazus manual multi family dwelling occupancy class will represent and repair cost ration taken accordingly.

As described under section 2.6.1 to estimate monitory loss the building replacement cost (BRC) is multiplied with probability of damage state and repair cost ratio.

3.9.2 Human Losses – Casualties

Methodology for the estimation of casualties is based on the assumption that there is a strong correlation between building damage (both structural and nonstructural) and the number of occupant present inside and outside and severity of casualties. In smaller earthquakes, nonstructural damage will most likely control the casualty estimates. In severe earthquakes where there will be a large number of collapses and partial collapses, there will be a proportionately larger number of fatalities.

In order to cover extreme cases of occupancy which are dependent of time of a day; three time scenarios are taken to compute number of casualties. The three specific times of day are 2 AM, 10Am and 5PM. These scenarios represent the times when population is at home, at work or school and commuting/rush respectively.

In order to calculate the number of casualties the number of occupant is multiplied by probability of damage state and probability of an injury of certain severity occurring given the damage state. For this purpose the manual classifies injuries in to four different severities as described under section 2.6.2 and the probability of an injury of a certain severity occurring given the damage state is termed as causality rate. Or in other word it can be defined as probabilities of fatality/severity as the result of given damage state.

Hazus causality rates take in to account people indoor and outdoor at a time of event. Outdoor causality are caused by failing hazards like parapet failure and only three damage states except slight are considered because it is unlikely that slight damage will cause failing hazard. For each occupancy class and for the three time scenario default population distribution is given in Hazus manual table based on census tract of that country. In our country there is no detail information on population distribution or cannot be inferred from available data. In such case as described in section 2.6.2 SELENA tool developers put an option for population distribution so considering residential building for the study building this distribution of peoples is taken and presented as follow in Table 3.12 .

Based on assumption that 5 peoples will dwell in single house unit the following number of population is estimated and the distribution as well.

Table 3.12 Estimated number of population

No of beds	No of unit	No of population
3beds	24	120
2beds	8	40
Shops	9	45
	Total	=205* 43 blocks=8815

Table 3.13 Estimated population percentage indoors and outdoors dependent on the time of the day

Occupancy class	2:00am(night)	10:00am(day)	5:00pm(commuting)
Indoor	8639	7933.5	3173.4
Outdoor	176	881.5	5641.6
total	8815	8815	8815

The loss model applied in the current study is only considering the direct human losses caused by structural damage not due to non-structural damage or follow-on hazards.

The following default casualty rates are defined by the methodology and used for this study.

Table 3.14 Indoor Casualty Rates for reinforced concrete frame Model Building Type for Structural Damage[4]

	Causality severity level			
Structural Damage	Severity 1 (%)	Severity 2 (%)	Severity 3 (%)	Severity 4 (%)
Slight	0.05	0	0	0
Moderate	0.25	0.03	0	0
Extensive	1	0.1	0.001	0.001
Collapse	40	20	5	10

Each number inside the table indicate that the probability of an injury of certain severity occurring given damage state Finally having the above data the number of causalities due to direct structural damage is estimated by multiplying indoor causality rate with number of people and probability of structural damage.

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[4] HAZUS-MH MR4 Technical Manual Table 13.3-13.7

CHAPTER 4 RESULTS AND DISCUSSIONS

In this chapter the results from ETABS 2016 based on Nonlinear static pushover analysis and Earth quake loss estimation method based on Hazus methodology are discussed respectively. The residential B+G+ 9 building which represent a group of blocks is analyzed and discussed based on the aim of study. A three-dimensional model for single building has been created to undertake the nonlinear analysis.

Once the non-linear analysis is carried out, the pushover curve in x-direction for a model are generated. Based on the analysis the pushover base shear verse monitored displacement diagram of the building under study is shown below.

4.1 Pushover Curve.

Table 4.1 Pushover base shear and top displacement for user defined properties.

Step	Droof	Vb	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN										
0	-3.879	0	1242	5	0	0	0	1247	0	0	0	1247
1	13.519	1021.366	1241	6	0	0	0	1247	0	0	0	1247
2	74.001	3985.974	1217	30	0	0	0	1245	2	0	0	1247
3	112.512	5127.441	1188	58	1	0	0	1230	16	0	1	1247
4	112.529	5127.938	1188	58	1	0	0	1230	16	0	1	1247
5	113.899	5174.998	1185	61	1	0	0	1229	17	0	1	1247

Table 4.1 shows the hinge state details at each step of the analysis. It can be seen that for the 99% of hinges are within A- IO.

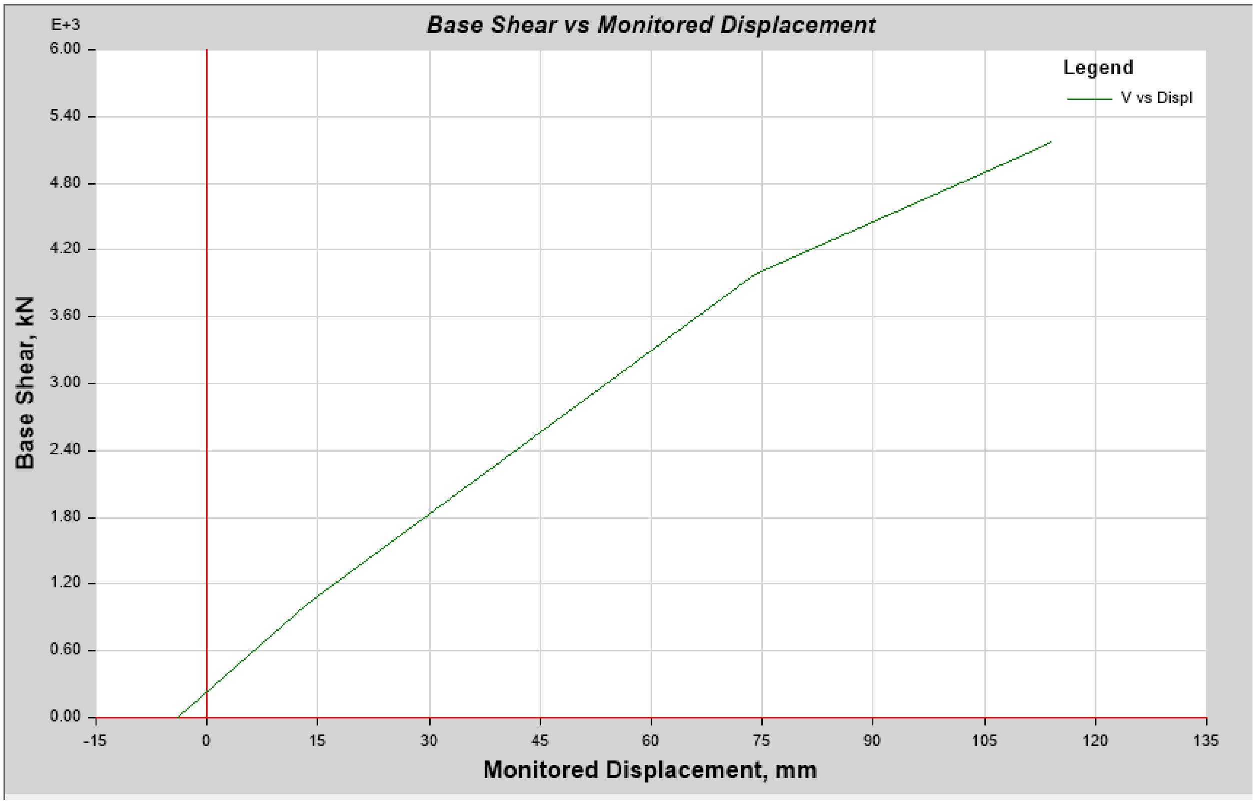


Figure 4-1 Pushover curve of the building under study.

4.2 Capacity curve

A building capacity curve is a relation between base shear force and the control displacement and derived from a plot of static-equivalent base shear versus building displacement at the roof, known commonly as a pushover curve.

To develop the capacity spectrum from pushover curve it is necessary to do a point by point conversation to first mode spectral coordinate .That mean any point base shear vs top roof displacement (V_i,Δ_{roof}) on the capacity curve is converted to the corresponding point spectral acceleration ,spectral displacement (Sa,Sd)on the capacity spectrum using the following equation:

$$S_{ai} = {}^{Vi}/_{W}\Big/_{\alpha 1} \quad (4\text{-}1)$$

$$S_{di} = {}^{\Delta roof}\Big/_{(PF1X\emptyset 1, roof)} \quad (4\text{-}2)$$

Where α_1 and PF_1 are modal mass coefficient and participation factors for the first natural mode of the structure respectively and $Ø_{1roof}$ is the roof level amplitude of the first mode. ETABs software work with these data and give capacity curve in acceleration –displacement response spectra (ADRS).

Building capacity curves are constructed for model building type and represent different levels of lateral force design and for a given loading condition, expected building performance. And the results of the analysis of the typical building represent median properties of the group.

Table 4.2 Spectral displacement verse acceleration result for capacity curve

Sd(mm)	**Sa(g)**
0	0
13.594	0.024798
60.852	0.09639
90.942	0.123697
90.955	0.123709
92.026	0.124914

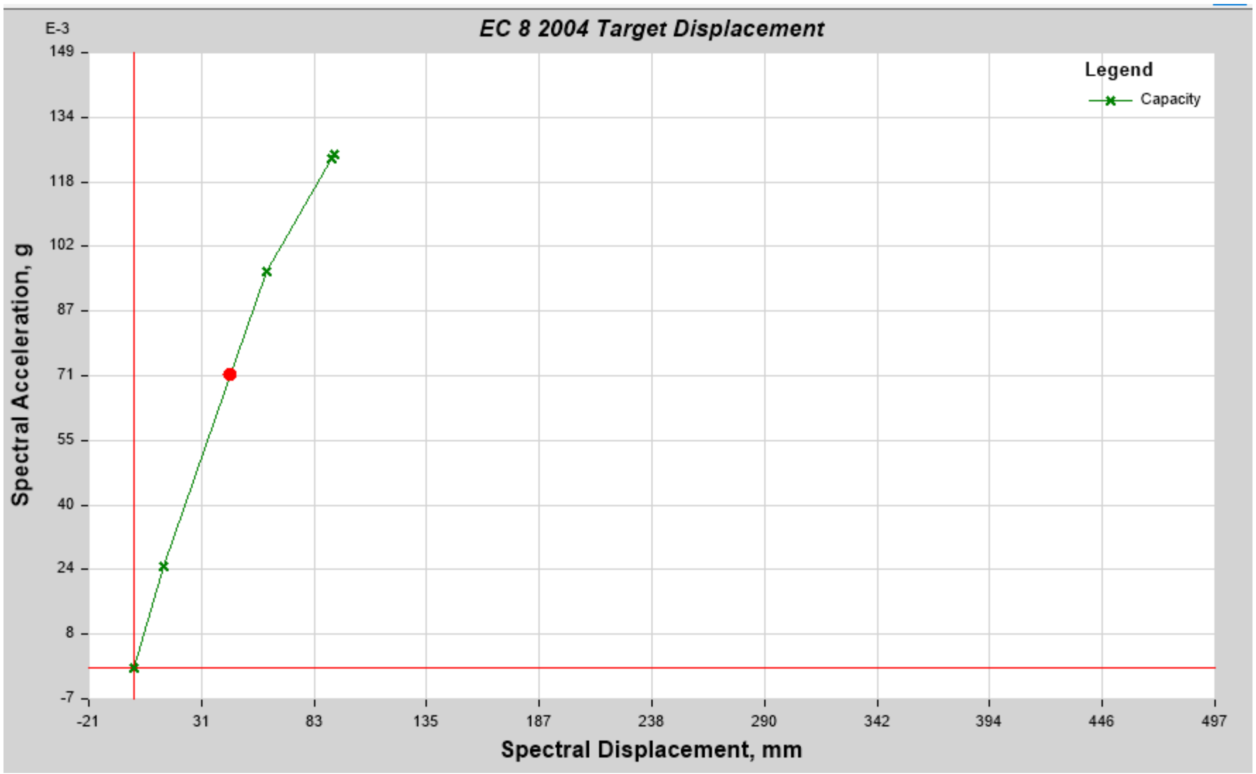


Figure 4-2 Capacity curve of the building

4.3 Performance Point.

Peak Building Response or Peak Spectral displacement is obtained by the intersection point of demand spectrum curve and capacity curve. It actually means that shaking is experienced by building till the peak building response point is achieved. Maximum damage in a building is seen when it reaches or crosses the peak building response point. The two curves i.e., demand spectrum curve and capacity curve are overlaid to achieve peak building response. However in order to compare capacity curve and demand curve both curve should be plotted in spectral acceleration vs spectral displacement domain. Every point on response spectrum curve has associated with a unique spectral acceleration S_a, velocity, Sv, displacement, S_d and period T. To convert from the standard S_a vs T format which found in building codes to ADRS format it is necessary to determine the value of S_{di} for each point on the curve S_{ai}, $T_{i.}$ Using the following equations

$$S_{di} = \frac{Ti^2}{4\pi^2} S_{ai} g \tag{4-3}$$

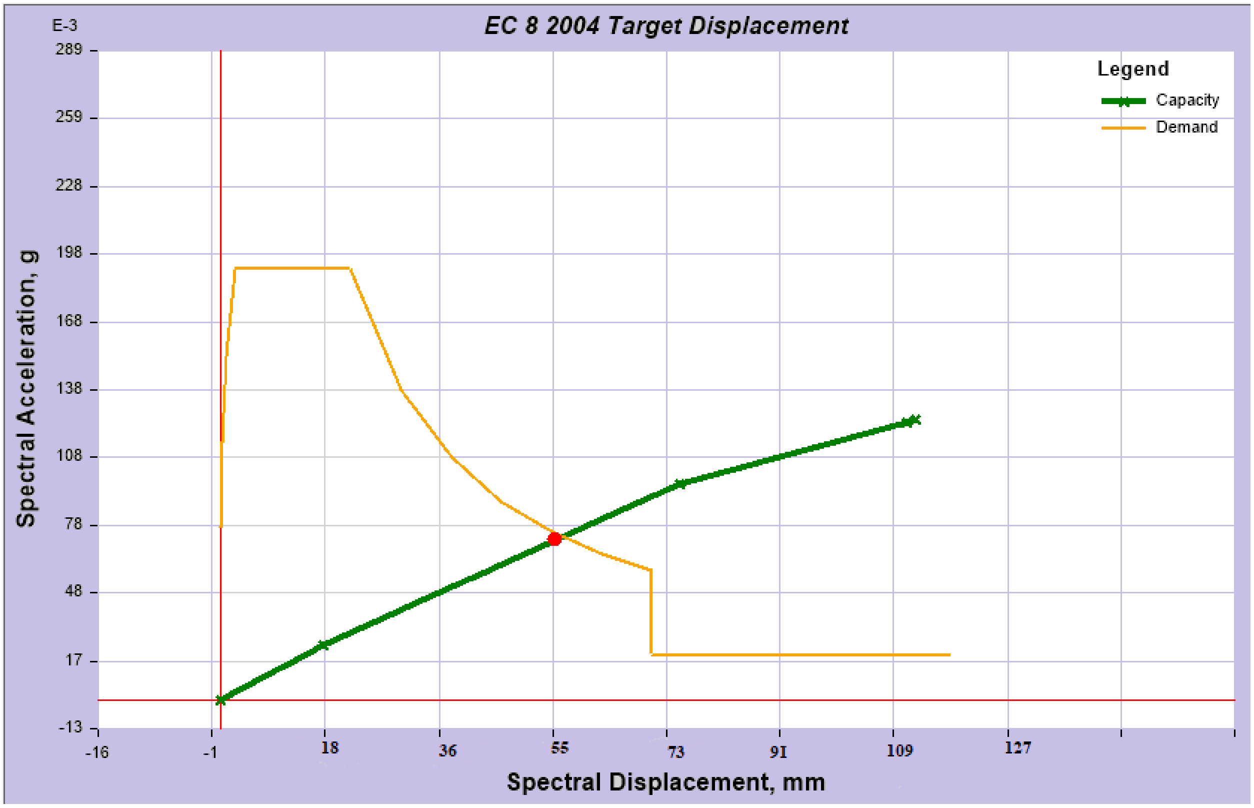


Figure 4-3 Peak building response plots

From the peak building response plots, peak response point in terms of spectral displacement are obtained. This is shown in Table 4-3. These values are further used in damage probability calculations. Damage states are calculated based on the ultimate and yield displacement of the building.

Table 4.3 Peak building response values for representative building type.

Building Type	Peak Building Response (Sd in mm)
RC frame system	56.97

The expected peak displacement (obtained from the performance point) is overlaid with the fragility curves in order to compute the damage probability in each one of the different damage states.

4.4 Fragility Curve and damage state probability

Fragility curves describe the probability of a damage state (k) sustained by an studied building type, being equal or exceeded given a level of ground motion intensity measure .In seismic vulnerability studies the curves are assumed to take the form of a lognormal cumulative distribution function having a median value and logarithmic standard deviation, or dispersion. The mathematical form for such a fragility curves is described in section 3.8.1 equation 3-7.

Cumulative damage probability is calculated for the building type for four damage types namely slight damage, moderate damage, extensive damage and complete damage. The sd value obtained (56.97mm) used to find the parameters of fragility curve Table 4.4 shows the two parameters i.e. Median Spectral displacement and Log standard Deviation (Beta) taken for each types of damages. Table 4.5 represents the cumulative damage probabilities calculated using above mentioned parameters.

Four disaggregated structural damage states are used, i.e. Slight, Moderate, Extensive, and Complete. Thus four median value of the spectral displacement ($\overline{Sd}$,k) at which the building reaches the threshold of damage state are required to differentiate among the four damage states.

Table 4.4 Damage state threshold values based on yield displacement S_y and Ultimate displacement S_u

Damage Level	Threshold values		Log standard Deviation βds
Yield Displacement(S_y)	98mm		
Ultimate Displacement(S_u)	113.89mm		
Slight	$0.7S_y$	68.6mm	0.687
Moderate	$1S_y$	147mm	0.687
Extensive	$S_y+0.25(S_u-S_y)$	105.94mm	0.687
Complete	S_u	113.89mm	0.687

Substituting all those values in equation (3-7) fragility curves are developed for each damage states which shows the continuous distribution of damage state.

Table 4.5 Cumulative probability damage state result

Sd	SLIGHT	MODERATE	EXTENSIVE	COMPLETE
1	7.4912E-09	3.19189E-10	8.1582E-11	3.19189E-10
5	0.00044948	6.1702E-05	2.54842E-05	2.71463E-06
10	0.01038892	0.002320987	0.001172093	0.000200749
15	0.0425228	0.012512462	0.007085567	0.001592823
20	0.09618223	0.034189429	0.020956645	0.005691495
25	0.16379242	0.067082755	0.043655688	0.013688844
30	0.23769048	0.108855611	0.074297062	0.026138157
35	0.31224562	0.156635851	0.111183373	0.043026592
40	0.383907	0.207787768	0.152445955	0.063962521
45	0.45070262	0.260186003	0.196372413	0.088349898
50	0.51172347	0.312245619	0.241535069	0.115515088
55	0.56672666	0.362854505	0.286813701	0.144787714
60	0.61586152	0.411278962	0.33136962	0.175546701
65	0.65949103	0.457072715	0.374601911	0.207242764
70	0.69807977	0.5	0.416101056	0.239406092
75	0.73212599	0.539974651	0.455606653	0.271645324
80	0.76212207	0.577013688	0.492971584	0.303641827
85	0.78853291	0.611202845	0.528132915	0.335141782
90	0.8117853	0.642671457	0.561088908	0.365947591
95	0.83226403	0.671574473	0.591881199	0.395909464
100	0.85031175	0.698079772	0.620581192	0.424917636

Where Sd refer to sepctral displacement.

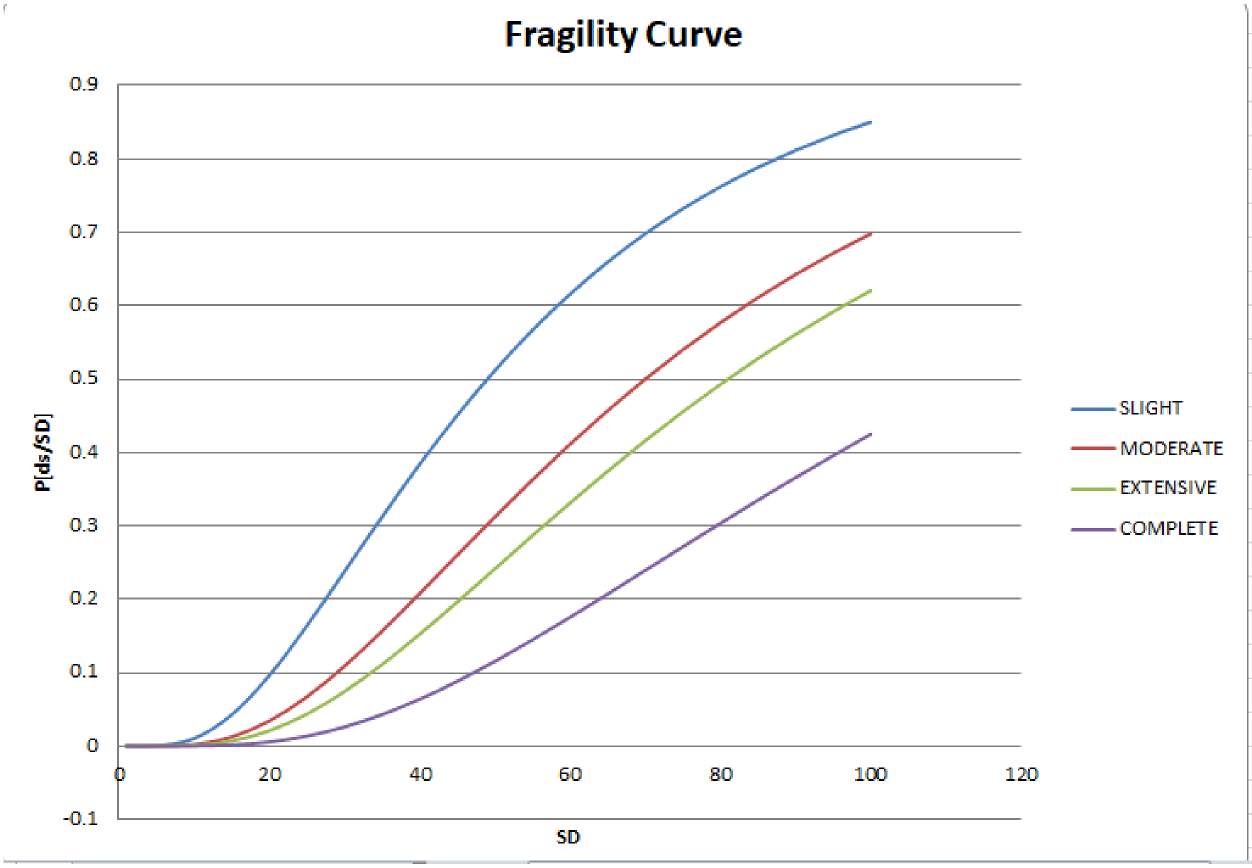


Figure 4-4 Continuous probability of damage or Fragility curve for the model building.

Fragility curve may different for different models, but same for a model irrespective of type of soils and seismic zones. Soil type and seismic zone does not have influence on fragility curve. It is purely dependent on the building properties. Type of soil and seismic zone are responsible for the position of performance point on the fragility curve. When those parameter changes the position of performance point in fragility curve changes. The vulnerability of structure increases as the performance point slides towards right of the graph. It is observed that the performance point is more towards left for this model building and the probability of reaching or excedence of the three damage state is more or less close to each other.

For any given value of spectral response, discrete damage state probabilities are calculated as the difference of the cumulative probabilities of reaching or exceeding successive damage states. The probabilities of a building reaching or exceeding the various damage levels at given response level sum to 100%. Then discrete damage state probabilities are used as inputs to the calculation of various types of building-related loss. The cumulative and discrete damage probability of the model building estimated and presented as follow.

Table 4.6 Discrete damage probability result

Cumulative probability of damage state	P[k/sd]
P[S/sd]	0.3907485
P[M/sd]	0.2129421
P[E/sd]	0.1965266
P[C/sd]	0.1550592

Table 4.7 Discrete damage probability result

Probability of damage	Level of damage	Discrete Damage probability	Discrete Damage probability in (%)
P(C)=P[C/sd]	Complete	0.155059	15.50
P(E)=P[E/sd]- P[C/sd]	extensive	0.041467	4.14
P(M)=P[M/sd]- P[E/sd]	Moderate	0.016415	1.64
P(S)=P[S/sd]- P[M/sd]	slight	0.177806	17.78
Total vulnerability		0.39074	39.07
P(None)=1- P[S/sd]-	No damage	0.60925	60.92

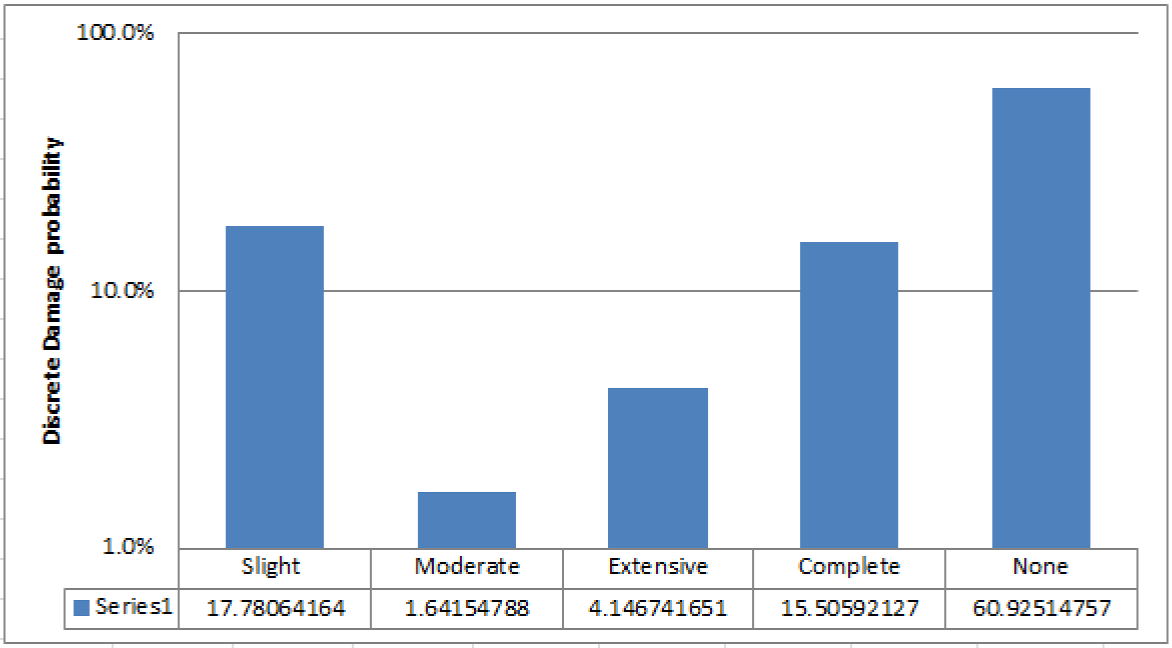


Figure 4-5 Discrete damage probabilities derived from the cumulative damage probabilities for an expected displacement as illustrated in Figure 4.4.

From Figure 4.5 and Table 4.7 one can see that the probability of no damage to the study building is 60.92% and relative to other damage states slight damage has greater probability.

4.5 Loss estimation

Based on the data found on this model building on 2012 G.C the building cost estimation (Building replacement cost) was 14,739,376.32birr (BRC) There are 43 blocks of this type of building in three different area of Addis Ababa. Cost of structural damage for damage level k found by multiplying the probability of that damage with repair cost ratio and replacement cost of the building. Finally the total cost of structural damage will be sum of cost of the four damage levels. Accordingly as shown below in Table 4.8 the building total cost of structural damage could be estimated as 1,585,919,087.15 birr of that time value. It is fact that money can have different values at different times. This is because money can be used to earn more money between the different instances of time. Obviously, 7.1 million birr of that time is worth more than now. The cost of structural damage for the study building is calculated as follow:

$$CS_{k,} = BRC \ * P[k/sd] * RCS_k \tag{4-4}$$

$$CS \ = \sum_{k=2}^{4} CS_{ki} \tag{4-5}$$

For this case Building replacement cost (BRC) is fourteen million seven hundred thirty nine thousand three hundred seventy six birr and thirty two cent (14,739,376.32birr)

Table 4.8 Cost of structural damage

	Damage levels			
	S	M	E	C
P[k/sd]	0.177806	0.016415	0.041467	0.155059
RCS_k	0.3	1.4	6.9	13.8
Cs_k	786226.71	338735.49	4217307	31539570.37
Cs			Sum Cs_k	36,881,839.24
For 43 block				1,585,919,087.15

Where P[k/sd] is probability of damage sate k, RCS_k is repair cost ratio, Cs_k is cost of structural damage for damage level k and Cs is cost of structural damage.

Repair cost ratio represents the fraction of construction cost that is attributed to structural and non-structural component cost and taken from default values from HAZuS manual.

Based on probability analysis the value of economic loss can be calculated for each level of damage. Mean damage probability (MDP) of the building is calculated and the following results are obtained.

Table 4.9 Damage loss analysis (FEMA, 1999)

Level of damage	P[k]	Loss Economic value	MDP (%)
Complete	0.155059	100%	15.50
Extensive	0.041467	50%	2.073
Moderate	0.016415	10%	0.164
Slight	0.177806	2%	0.3556
		Total MDP	18.0976

The estimated value of economic losses based on the method FEMA-1999 approach obtained by MDP value of 18.09%. This means that the building has a damage economic loss probability of 18.09% due to the expected earthquake scenario.

Human life loss is calculated based on the assumption that 5 people dwell in single household and the result for severity 1 at 2:00 am time scenario (night time) is showed below in Table 4.9 From the result it is observed that night time earthquake scenario causes high causality than day time scenario and rush hour scenario. And for particular time earthquake scenario the number of casualty decreases as level of severity increase. For the same severity level the number of loss for night time will be higher than day and rush time. The results are described by chart as shown in Figure 4.6 to 4.8.

$$K_i^s = \left(injuries(severity)i = \sum_{j=1}^{N_{BT}} \sum_{k=1}^{N_{DS}} C_{j,k}^{CSR} P_{j,k} N_j^{POP} \right) \tag{4-6}$$

Where

$C_{j,k}^{CSR}$ = Causality rate of severity i for damage state k

$P_{j,k}$ = Structural damage probability for damage state k

N_j^{POP} =Number of people in the model building type j

For this case j=1 and after assuming number of peoples in the study building number of population are distributed in three different time of a day.

Table 4.10 Number of people

Assuming 5 people in one unit house

	No room	No Pop	
3 Beds	24	120	
2 Beds	8	40	
shops	9	45	
		205	single building
		8815	43 Blocks

Table 4.11 Population distribution indoor and outdoor

	2:00 AM	2:00Pm	5:00pm
	98% in door	90% in door	36% in door
Indoor	8638.7	7933.5	3173.4
outdoor	176.3	881.5	5641.6
	8638.7	7933.5	3173.4

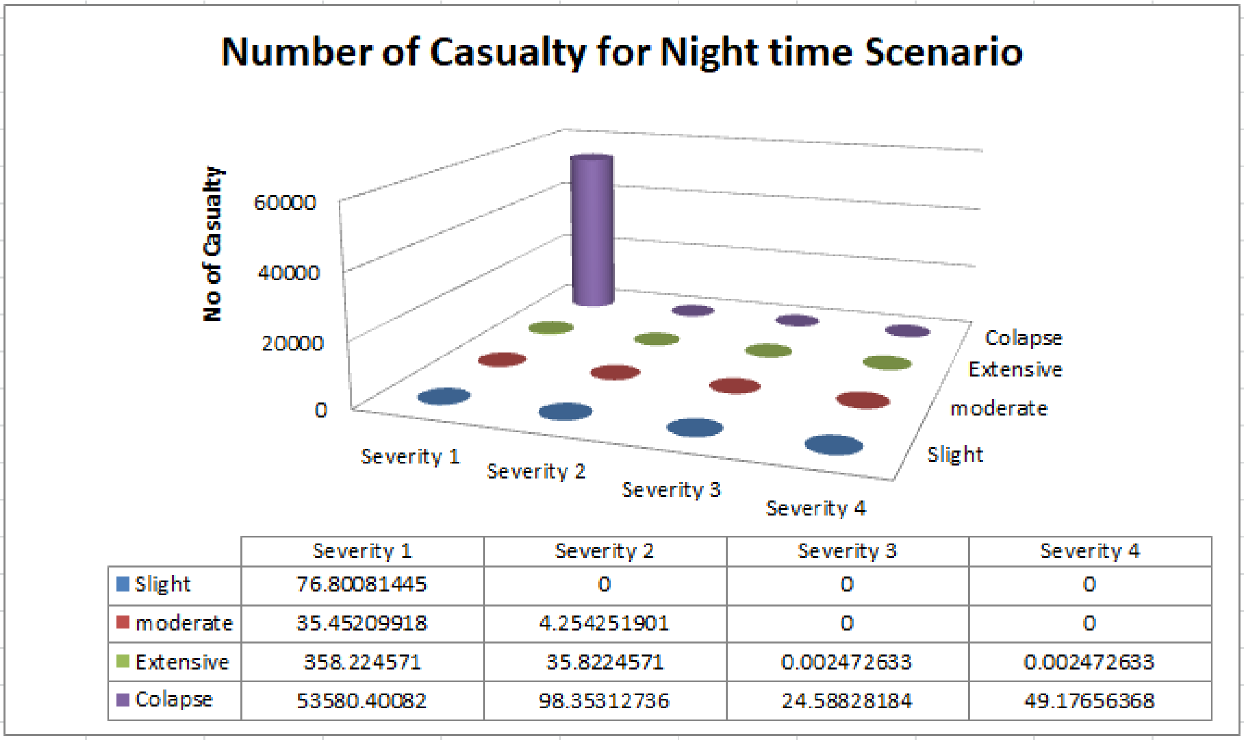
Table 4.12 Number of causality estimation at 2:00Am (Night time)

Severity 1	Ki	CJ,k	PJ,K	Nj
Slight	76.80081445	0.05	0.177806416	8638.7
moderate	35.45209918	0.25	0.016415479	8638.7
Extensive	358.224571	1	0.041467417	8638.7
Collapse	53580.40082	40	0.155059213	8638.7

Table 4.13 Number of causality estimation at 2:00pm (Day time)

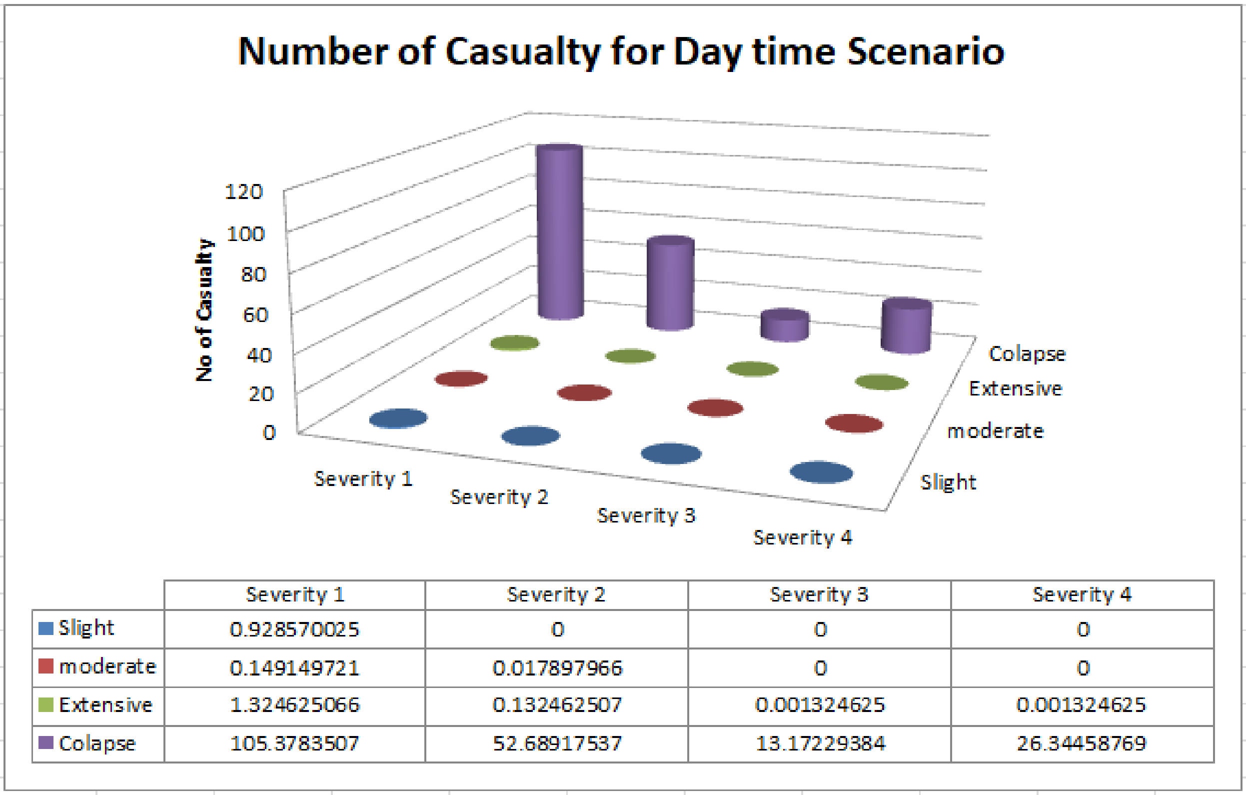
Severity 1	Ki	CJ,k	PJ,K	Nj
Slight	0.928570025	0.05	0.004012943	4627.875
moderate	0.149149721	0.25	0.000128914	4627.875
Extensive	1.324625066	1	0.000286227	4627.875
Collapse	105.3783507	40	0.000569259	4627.875

Table 4.12 and Table 4.13 shows the number of causality for severity level 1 for night and day time scenario respectively .the other severity level and time scenario are summarized below in figures. The number indicated on table shows the number of causality.



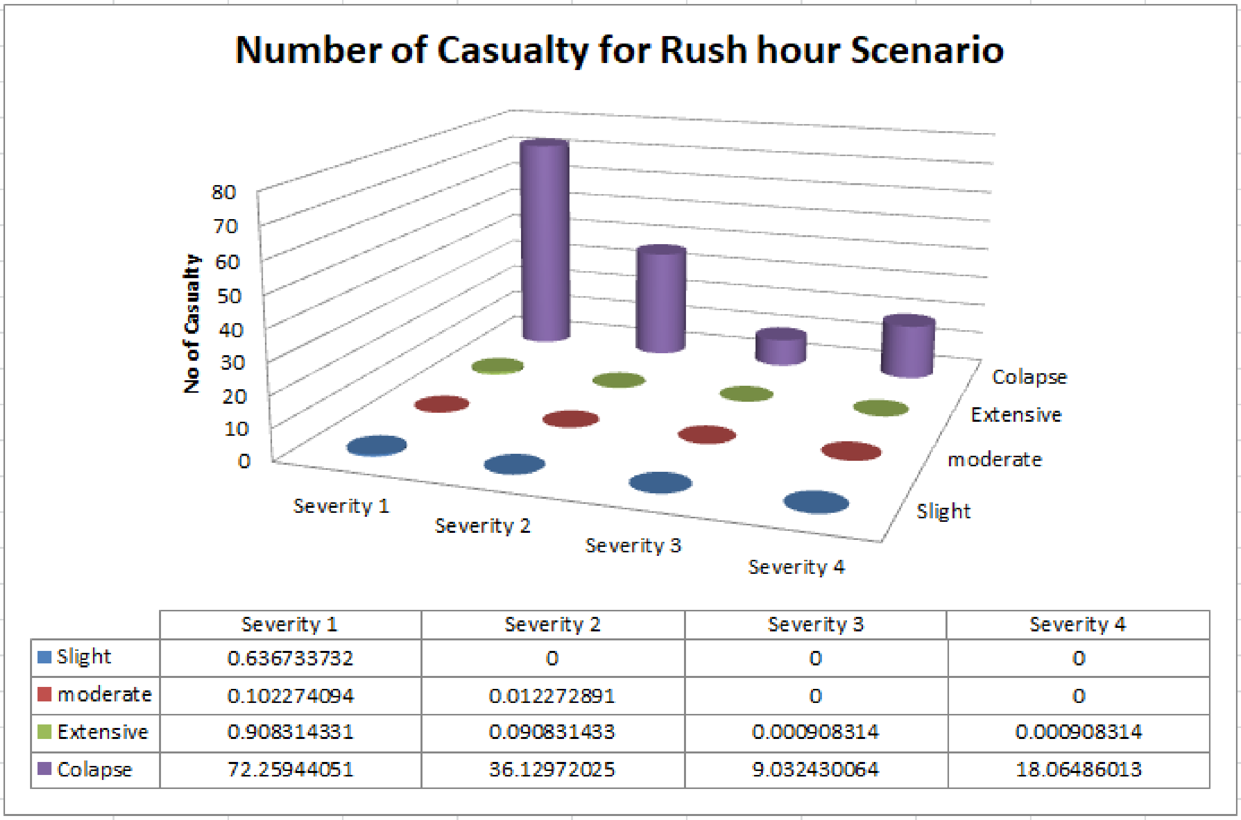
	Severity 1	Severity 2	Severity 3	Severity 4
🟦 Slight	76.80081445	0	0	0
🟥 moderate	35.45209918	4.254251901	0	0
🟩 Extensive	358.224571	35.8224571	0.002472633	0.002472633
🟪 Colapse	53580.40082	98.35312736	24.58828184	49.17656368

Figure 4-6 Number of Casualty for Night time Earthquake scenario



	Severity 1	Severity 2	Severity 3	Severity 4
🟦 Slight	0.928570025	0	0	0
🟥 moderate	0.149149721	0.017897966	0	0
🟩 Extensive	1.324625066	0.132462507	0.001324625	0.001324625
🟪 Colapse	105.3783507	52.68917537	13.17229384	26.34458769

Figure 4-7 Number of Casualty for Day time Earthquake scenario



	Severity 1	Severity 2	Severity 3	Severity 4
🟦 Slight	0.636733732	0	0	0
🟥 moderate	0.102274094	0.012272891	0	0
🟩 Extensive	0.908314331	0.090831433	0.000908314	0.000908314
🟪 Colapse	72.25944051	36.12972025	9.032430064	18.06486013

Figure 4-8 Number of Casualty for Rush hour Earthquake Scenario

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In the present study, an attempt has been made to study the seismic risk of RC structures by conducting vulnerability and loss assessment. Following are the conclusions drawn from the present study.

- The Pushover Analysis was including 5steps it has been observed that one subsequent push to building, hinges started forming in beams first. Initially hinges were in A-B stage and subsequently proceeding to C-D stage. Out of 7428 hinges 7261 in A-B stage, 218 in B-C stage. Overall performance of building is said to be collapse prevention (CP) stage.
- The performance point of the building model was around 56.97mm.with this performance point values the building discrete probability of reaching or exceeding slight, moderate, extensive and complete damage state are 0.177806, 0.016415, 0.041467and 0.155059 respectively.
- The probability of no damage to the study building is 60.9% and relative to other damage states slight damage has greater probability.
- Based on probability analysis the building has a damage economic loss probability of 18.09% due to the expected earthquake scenario.
- With given probability of reaching or exceeding four damage states the total loss due to structural damage is 1,585,919,087.15 birr.
- The economic loss due to expected earthquake scenario is very large relative to building value in this case instead of maintenance replacement would be recommended
- For particular time earthquake scenario the number of casualty decreases as level of severity increase.

5.2 Recommendation for future study

- Since this study is restricted to residential buildings further study can be conducted on essential buildings like on police stations, fire stations, hospitals and schools. For buildings located in the earth quake prone area of Ethiopia.
- Nonlinear dynamic analysis needs to be done for more accurate and exact estimation.
- There are different software tools like SELENA and ELER for further studies which can carry out estimation of damage to the general building stock, the economic and human

losses related to these physical damages, and optionally the estimation of the amount of debris and number of uninhabitable buildings as well as number of displaced households.

- Further studies can be carried out by selecting different retrofitting technique and compare cost benefit analysis.
- In this study just one type of model is checked, However different type of models can be carried out with different story number, zone, construction material and year of construction.
- GIS supported extensive national databases, containing information such as demographic aspects of the population in a study region, square footage for different occupancies of buildings, and numbers and locations are needed to be documented; Using this information, users can carry out general loss estimates for a region.
- Since there are different seismic risk assessment methodologies, it is good to compare different methodology and propose the best one which could be suitable for our country.

REFERENCE

1) Burce A. Bolt (2006) Assessing and Managing Earthquake, Volume 2, Netherlands, 2006
2) Georgia E.Thermou, Stavroula J.Pantazopoulou and Amr S. Elnashai (2009), Design and Assessment models and spectra for repaired reinforced concrete structures, Mid America Earthquake center, MAE center report No.09-01.
3) Berijesh Gulati (2006), Earthquake risk assessment of buildings: Applicability of Hazus in Dehradun, India,2006
4) Samuel Kinde Kassegne, Samson Engeda , Asnake Kebede and Eyob Tessema , Notes and proposed Guidelines on updated seismic codes in Ethiopia-Implication for large-scale infrastructures.
5) WB and GEDRR(2015) Addis Ababa, Ethiopia Enhancing urban resilience, Retrieved fromhttp://documents.worldbank.org/curated/en/559781468196153638/pdf/100980-REVISED-WPPUBLIC-Box394816B-Addis-Ababa-CityStrength-ESpread-S.pdf," 2015.
6) National institute of building science (NIBS), Multi-hazard loss estimation methodology, earthquake model, HAZUS-MH4 technical manual. Washington .D.c 2003
7) Sian Herbert (2013), Assessing seismic risk in Ethiopia ,Helpdesk research report , 2013
8) Katherine Marie Thibert (2008), Methodology for Assessing the seismic risk of buildings, University of British Columbia., 2008
9) David Dowrick (2009) Earthquake resistant design and risk reduction
10) Federal emergency management agency (FEMA) (2003), Earthquake Loss estimation methodology, Hazus-MH2.1, Advanced Engineering building module (AEBM).
11) EBCS-8 part 1 (2015), Design of structures for earthquake resistance, Ethiopian building code standard.
12) Mohammed Mohimuddin Ahmed, Israt Jahan and Dr.Md.Jahangir Alam (2014) , Earthquake vulnerability assessment of existing buildings in Cox's-Bazar using Filed survey and GIs, International Journal of Engineering research and technology (IJERT), Vol 3 ,No 8
13) Federal Emergency Management Agency (FEMA) (1997), NEHRP Guidelines for the seismic rehabilitation of building.
14) Vazurkar U.y.and Chaudhari D.J (2016), Development of fragility curves for Rc buildings, International journal of engineering research volume No 5 pp 591-594,
15) Applied Technology Councile (ATC 40), 1996, Seismic evaluation and retrofit of concrete buildings, Vol 3.
16) Dominik H.Lang (2012) Earthquake damage and loss assessment- predicating the unpredictable, Bergen , Norway
17) Kishor Jaiswal , David Wald and Keith Porter (2010), A glopal building inventory for earthquake loss estimation and risk management ,Earthquake spectra volume 26 No 3 Pages 731-748, earthquake research institute.
18) Sergio Molina, Dominik H.Lang, Abdelghani Meslem and Conrad D. Lindholm, SELENA v6.5 User and Technical manual, Norway 2015.

19) Park, R. (1988). Ductility evaluation from laboratory and analytical testing Vol &VIII. Proceedings of the 9th world conference on earthquake engineering, Tokyo, Japan.
20) Serhan Guner (2010). Pushover Analysis of Shear Critical Frames: Verification and Application. ACI structural journal, 2010, USA

APPENDIX

Appendix A: Verification of software ETSBS 2016 V 16.2.1 for pushover analysis

The objective of verification is to demonstrate the performance of ETABS 2016 V 16.2.1 for static nonlinear or pushover analysis. The experiment investigated by Vecchio and Emara (1992) is used for verification .The model frame used by the researcher is described below in Figure appendex-1. In the experiment the frame was loaded to a lateral displacement of 155mm and then unloaded to a net lateral load of zero.[20]

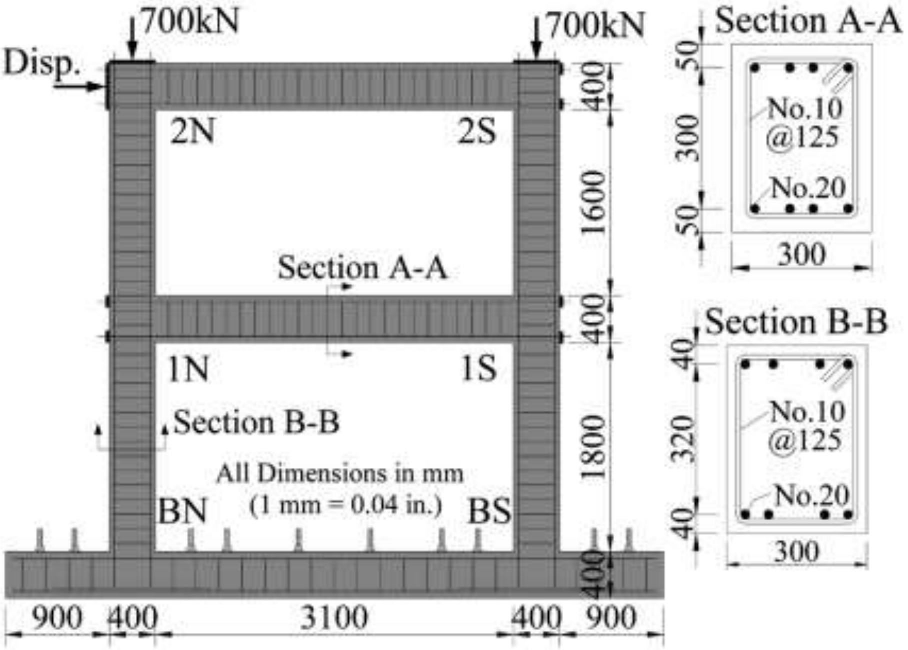


Figure A-1 structural detail of Vecchio and Emara (1992) frame

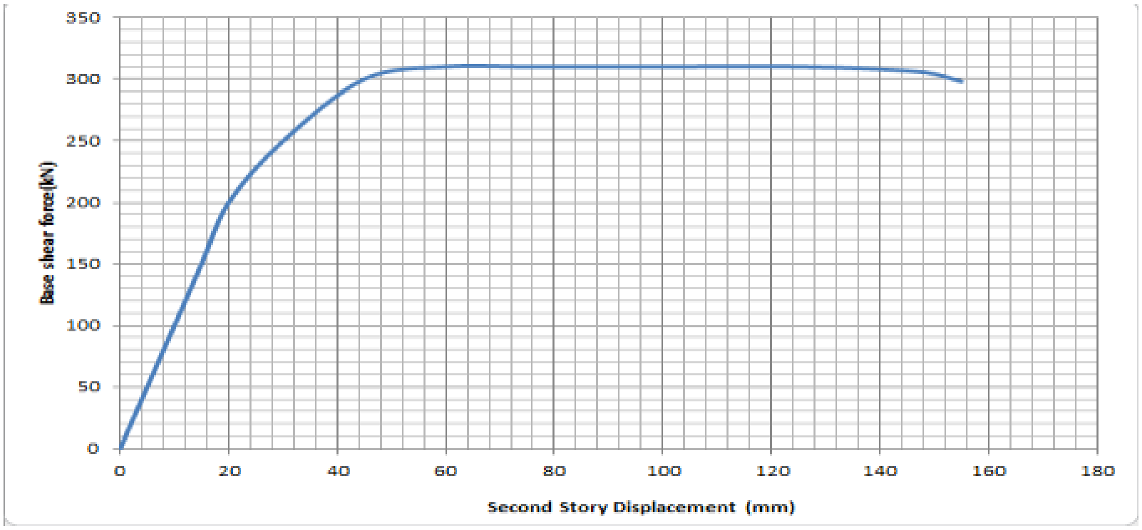
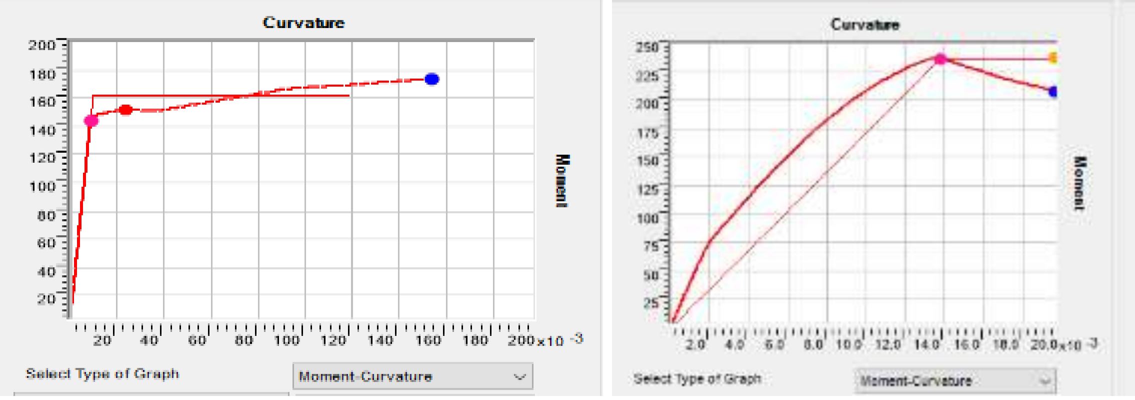


Figure A-2 Pushover curve for experimental frame model

Hinge characteristics (moment curvature data) of beam and column section are carried out with SAP2000 v.21 software.



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Select Type of Graph: Moment-Curvature

Figure A-3 M-k diagram for beam and column section

Table A-1 Moment curvature result from SAP2000.

Section Id	my	ky	mu	ku
Beam	143.773	0.008681	173.556	0.154
Column	232.53	0.0124	243.84	0.0196

Table A-2 Moment curvature relation for beam and column

Section	point	m	Ɵ=lp*k	m/sf	Ɵ/sf
Beam 400x300	A	0	0	0	0
	B	143.773	0.001736	1	0
	C	173.556	0.0308	1.207	0.0291
	D	28.75	0.0308	0.2	0.0291
	E	28.75	0.0616	0.2	0.0581
Column 400x300	A	0	0	0	0
	B	232.53	0.00248	1	0
	C	234.84	0.00392	1.01	0.0014
	D	46.506	0.00392	0.2	0.0014
	E	46.506	0.00784	0.2	0.0029

Axial load for the column section is used as given above on the experiment

Pushover analysis using auto equivalent static earthquake analysis were carried out using ETABS 2016 Vs 16.2.1 In order to capture nonlinearity characteristics the above experimental frame was modeled with user defined and default hinge characteristics and the result are shown below in figure and tables

Table A.3 Top story displacement versus base shear result of UDH and DH

Pushover (UDH)		Pushover (DH)	
D(mm)	F_b(kN)	D(mm)	F_b(kN)
0.001	0	0.001	0
17.717	195.6009	20.001	220.8207
26.129	246.4954	22.348	246.7355
38.211	277.4779	32.997	306.6057
58.211	286.6041	33.728	307.9561
78.211	295.8425	53.728	318.1368
98.211	305.0509	70.304	326.6784
118.211	314.2703	70.301	261.6477
138.211	323.4896	71.184	268.5537
140.409	324.5347	71.186	254.1081
140.409	205.8682	72.327	263.0569
144.987	231.4491	76.307	277.8162
148.367	238.7738	77.54	278.33

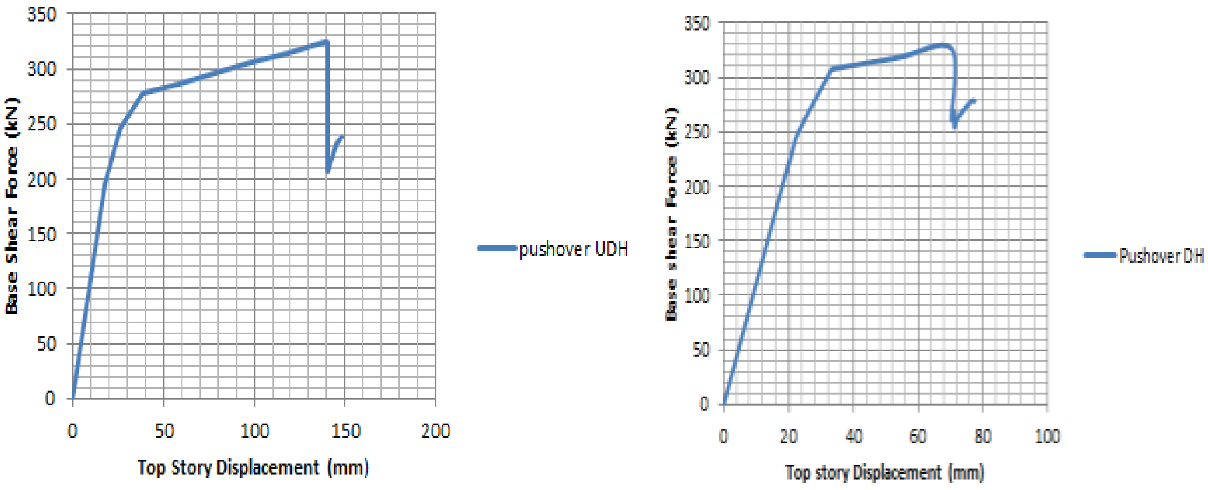

Figure A-4 Pushover curve for user defined hinges (UDH) and default hinges (DH)

Comparing of the three pushover curve the following result obtained

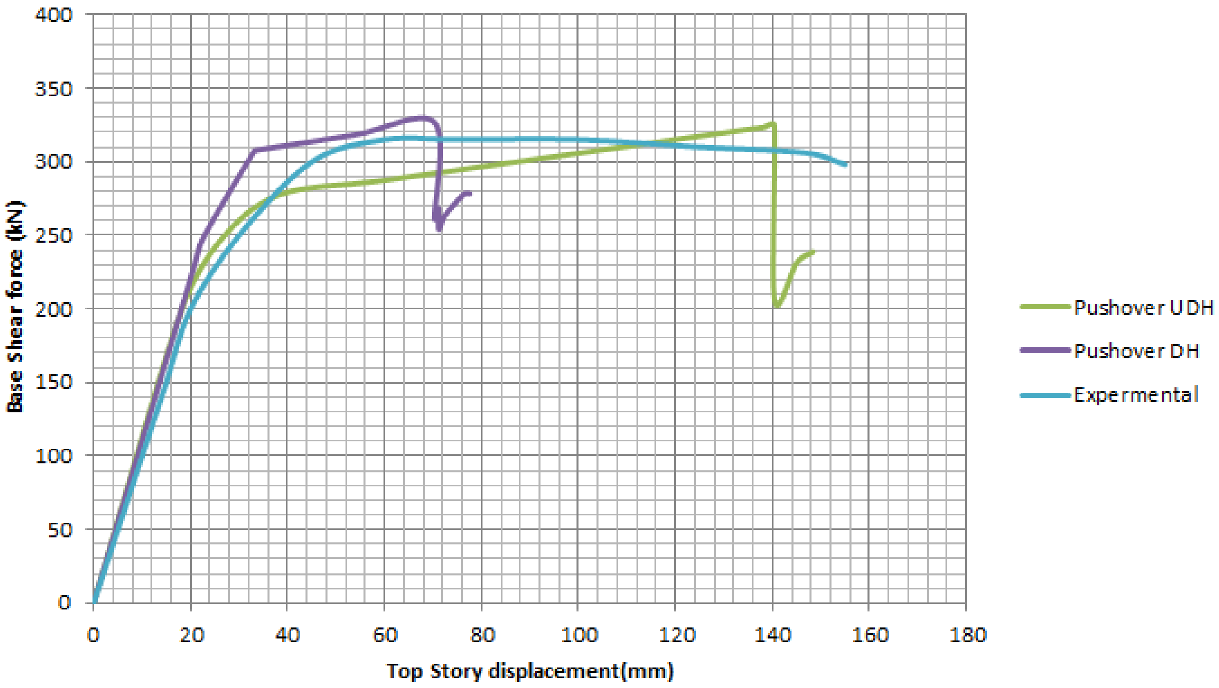

Figure A-5 Pushover curve of Experimental and analytical result

As seen from the figure the maximum base shear force for experimental, user defined and default hinge are 315kN, 325kN and 327kN respectively. Hence both user-defined and default hinge assignment for pushover analysis could be acceptable. However, pushover analysis using user-defined hinge is more acceptable than default and used for this study.

Appendix B: Design sections and reinforcement for sample model

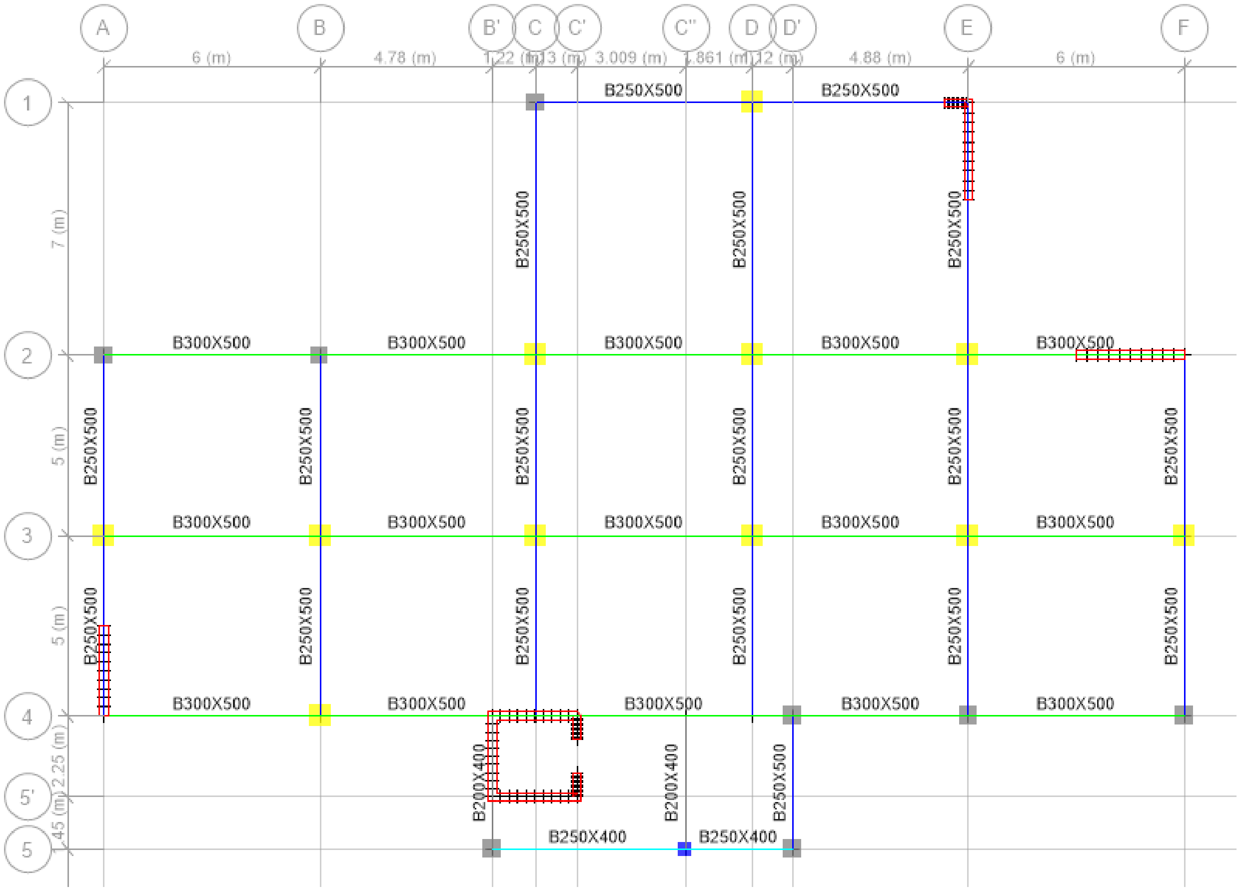

Figure A-6 Typical floor beam and column layout

Table A-4 sample design section and reinforcement

Name	SIZE	AXIS									
TFB-1	250*400	A	B	B	B	B'	C	C	D	D'	E
	TOP					3Φ14+1Φ16	5Φ14				
	BOTTOM					3Φ14	3Φ14				
	STIRUP					Φ8/110	Φ8/110				
TFB-2	300*500	A	B	B	C	C	D	D'		E	F
	TOP	5Φ20	3Φ20+2Φ16	3Φ20+2Φ16	3Φ20+3Φ24	6Φ24	3Φ16+3Φ20	3Φ16+3Φ20	3Φ20+3Φ16	3Φ16+3Φ20	3Φ20+3Φ16
	BOTTOM	5Φ16	5Φ16	5Φ16	5Φ16	4Φ16	3Φ16	3Φ16	3Φ16	5Φ16	5Φ16
	STIRUP	Φ10/90	Φ10/90	Φ10/90	Φ10/90	Φ10/90	Φ10/90	Φ10/100	Φ10/100	Φ8/120	Φ8/120
TFB-3	300*500	A	B	B	C	C	D	D	E	E	F
	TOP	3Φ20+3Φ24	3Φ20+3Φ24	3Φ20+3Φ24	3Φ20+3Φ24	3Φ20+3Φ24	3Φ20+3Φ2	3Φ20+3Φ24	3Φ20+3Φ2	3Φ20+3Φ24	3Φ20+3Φ24
	BOTTOM	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20	4Φ20
	STIRUP	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90	Φ12/90
TFB-4	300*500	A	B	B	C	C	D	D		E	F
	TOP	3Φ16+3Φ20	3Φ16+3Φ20	3Φ16+3Φ20	3Φ16+3Φ2	3Φ16+3Φ20	3Φ16+3Φ2	3Φ16+3Φ20	3Φ1	3Φ16	3Φ16+4Φ20
	BOTTOM	4Φ16	4Φ16	4Φ16	4Φ16	4Φ16	4Φ16	4Φ16	6Φ2	6Φ20	6Φ20
	STIRUP	Φ10/90	Φ10/90	Φ10/90	Φ10/9	Φ10/90	Φ10/90	Φ10/90	Φ10/90	Φ12/100	Φ12/10
TFB-5	250*500	A	B	B	C	C	D	D	E		
	TOP					6Φ16	3Φ16+3Φ14	3Φ16+3Φ14	3Φ16+3Φ20		
	BOTTOM					3Φ16	3Φ16	3Φ16	4Φ16		
	STIRUP					Φ10/150	Φ10/150	Φ8/100	Φ8/100		
TFB-6	250*500	6	5	5		4	3	3	2	2	1
	TOP					6Φ20	3Φ20+1Φ1	3Φ20+1Φ16	3Φ20+1Φ14		
	BOTTOM					4Φ20	4Φ2	4Φ16	4Φ16		
	STIRUP					Φ10/90	Φ10/9	Φ8/120	Φ8/120		
TFB-7	250*500	6		5	4	4	3	3		2	1
	TOP					3Φ14+3Φ16	2Φ16+3Φ1	2Φ16+3Φ14	3Φ14+3Φ16		
	BOTTOM					4Φ16	4Φ1	4Φ16	4Φ16		
	STIRUP					Φ8/120	Φ8/120	Φ8/110	Φ8/110		

Appendix D: Hinge properties for sample models

		A		B		C		D		E	
Id	Section	m	θ	m	θ	m	θ	m	θ	m	θ
TFB-1	B'	0	0	1	0	1.121	0.0186	0.2	0.0186	0.2	0.0372
	C'	0	0	1	0	1.07	0.014	0.2	0.014	0.2	0.028
TFB-2	A	0	0	1	0	1.082	0.0125	0.2	0.0125	0.2	0.025
	B	0	0	1	0	1.131	0.0168	0.2	0.0168	0.2	0.0335
	C-	0	0	1	0	1.027	0.0061	0.2	0.0061	0.2	0.0121
	C+	0	0	1	0	1.008	0.0027	0.2	0.0027	0.2	0.0054
	D',E-	0	0	1	0	1.043	0.0092	0.2	0.0092	0.2	0.0185
	E+,F	0	0	1	0	1.108	0.0141	0.2	0.0141	0.2	0.0282
TFB-3	A-F	0	0	1	0	1.04	0.008	0.2	0.008	0.2	0.016
TFB-4	A-D+	0	0	1	0	1.058	0.0063	0.2	0.0063	0.2	0.0125
	E	0	0	1	0	1.246	0.0464	0.2	0.0464	0.2	0.0928
	F	0	0	1	0	1.177	0.0257	0.2	0.0257	0.2	0.0514
TFB-5	C+	0	0	1	0	1.056	0.0101	0.2	0.0101	0.2	0.0202
	D	0	0	1	0	1.109	0.0133	0.2	0.0133	0.2	0.0266
	E-	0	0	1	0	1.062	0.0093	0.2	0.0093	0.2	0.0186
TFB-6	3	0	0	1	0	1.19	0.0287	0.2	0.0287	0.2	0.0573
	2	0	0	1	0	1.133	0.0158	0.2	0.0158	0.2	0.0315
	4	0	0	1	0	1.063	0.0087	0.2	0.0087	0.2	0.0175
TFB-7	4+,2-	0	0	1	0	1.146	0.0193	0.2	0.0193	0.2	0.0387
	3	0	0	1	0	1.176	0.0299	0.2	0.0299	0.2	0.0597
TFB-8	5+,4	0	0	1	0	1.154	0.0237	0.2	0.0237	0.2	0.0475
TFB-9	4	0	0	1	0	1.03	0.008	0.2	0.008	0.2	0.016
	3	0	0	1	0	1.116	0.0157	0.2	0.0157	0.2	0.0315
	2-	0	0	1	0	1.052	0.0035	0.2	0.0035	0.2	0.007
	2+,1	0	0	1	0	1.104	0.0141	0.2	0.0141	0.2	0.0283
TFB-10	4	0	0	1	0	1.212	0.0428	0.2	0.0428	0.2	0.0856
	3	0	0	1	0	1.156	0.0238	0.2	0.0238	0.2	0.0476
	2-	0	0	1	0	1.02	0.004	0.2	0.004	0.2	0.008
	2+	0	0	1	0	1.165	0.0227	0.2	0.0227	0.2	0.0454
	1	0	0	1	0	1.084	0.0114	0.2	0.0114	0.2	0.0228