ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

SEISMIC PERFORMANCE EVALUATION OF STIFFNESS IRREGULAR RC BUILDINGS DUE TO STORY HEIGHT DIFFERENCE

BY

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DECLARATION

I hereby declare that this thesis entitled "**SEISMIC PERFORMANCE EVALUATION OF STIFFNESS IRREGULAR RC BUILDINGS DUE TO STORY HEIGHT DIFFERENCE**" was composed by myself, with the guidance of my advisor, that the work contained herein is my own except where explicitly stated otherwise in the next, and that this work has not been submitted, in whole or in part, for any other degree of professional qualification.

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ABSTRACT

In high rise buildings or multi story buildings, soft-story construction is common because of story height difference due to architectural consideration. These provide stiffness irregularities on the building structures, which reduce the stiffness of the lateral load resisting system and sudden reduction in stiffness causes higher stress to be concentrated at the stiffness reduced story columns, which lead the columns to be unable to provide adequate resistance during the earth quake.

Taking into consideration the above points, the structural building frames must be evaluated and designed by considering the effect of soft story on seismic performance of the building. The main focus of this research is to study the seismic performance of stiffness irregular building due to significant story height difference on RC buildings by implementing numerical models on the basis of finite element principles. For these two regular buildings G+10 and G+20 is developed to be considered as base case structure, and cases that represent irregular structures are defined by modifying vertical distribution stiffness of the base case. The stiffness irregularity is created by increasing the height of ground floor and second floor of base case 200%. Accordingly, different model cases of RC building are analyzed and designed on ETABS 2016.2.1 (CSI ETABS 2016, Integrated Building Design Software, Computers and Structures Inc. Berkeley). Analysis and design of the proposed building model cases followed the conventional design approach as prescribed on the new Ethiopian Buildings Code Standards (ES EN: 2015). The design outputs of main structural elements are then used for the numerical model on SeismoStruct [SeismoSoft, 2016]. Pushover and nonlinear dynamic analyses are carried out for obtaining the response in terms fundamental periods, base shear-top displacement, inter-story drift, lateral displacements, and fragility curve at different performance levels.

The result of this research shows the following: the effect of stiffness irregularity on fundamental period have more significant contributions as the soft-story number and number of story or building height increase, for all cases the Story drifts demands increase in the soft story and decrease in most of the other stories and drift of most floors are greater than the allowable drift according to ES EN limit ($drv \leq 0,005h$), the seismic base shear for stiffness regular building are greater than the stiffness irregular one and finally the probability of failure for each limit state capacities of stiffness irregular building model is greater than stiffness regular buildings models in all cases.

Keywords: *soft-story, fragility curves, pushover and limit state capacities*

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ABBRIVATION

$\psi_{2,i}$	Combination coefficient for the quasi-permanent value of a variable action i
ATC	Applied Technology Council
CP	Collapse Prevention
FEMA	Federal Emergency Management Agency
IBC	International Building Code
DL	Damage Limitation
LS	Life Safety
RC	Reinforced Concrete
q	Behaviour Factor
UBC	Universal building code
Sd(T)	Design spectrum
β	Lower bound factor for the horizontal design spectrum
T	Fundamental period of vibration
αg	Design ground acceleration
TB	Lower limit of the period of the constant spectral acceleration branch
TC	Upper limit of the period of the constant spectral acceleration branch
TD	The value defining the beginning of the constant displacement response range of the spectrum
S	Soil factor
η	Damping correction factor

CHAPTER ONE

1 INTRODUCTION

1.1 GENERAL

During an earthquake, failure of structure starts at points of weakness. This weakness arises due to discontinuity in mass, stiffness and geometry of structure which is termed as irregularity. These weaknesses tend to emphasize and concentrate the structural degradation, often leading to complete collapse. Therefore, the structural engineer needs a good understanding of the seismic response of different types and configurations of buildings. Uncertainties in both seismic demands and structural capacities should be considered in comprehensive design procedure that addresses the response of the structures with various configurations which might result from different constraints imposed by owners, architects and planners. Many buildings in the present situation have irregular configurations both in plan and elevation due to aesthetic consideration and city regulation, which in future may be subjected to devastating earthquakes. Irregularities are not avoidable in construction of buildings; however, the behavior of structures with these irregularities during earthquake needs to be studied. Vertical irregularities are one of the major reasons of failures of structures during earthquakes. For example, structures with soft story and irregular distribution of mass were the most notable structures which collapsed during the past earthquakes. Excess mass can lead to abrupt increase in lateral inertial forces, reduced ductility in column, and increased tendency of collapse due to P-delta effects. The basic fundamental earthquake resistant design concept is the strong columns-weak beam criteria, so as to ensure safety of occupants, i.e. during earthquake the beams yield before the columns get collapsed. From the past earthquake we see that many building structures that collapsed exhibited the opposite strong beam-weak columns behavior i.e. the columns collapsed before the beams yielded this is because of the effects of soft-story provision.

Soft stories can result from omission of infill in single story, as often occurs in the first story, difference in height of floors, also from partial height infill which are also common in many buildings. In many buildings, first story columns are badly damaged as shown in fig 1.1 due to shear in earthquake. So, the effect of vertical irregularities on the seismic performance of structures becomes really important. Height-wise changes in stiffness and mass make the dynamic characteristics of the buildings different from the regular building. Also such type of irregularities causes concentration of forces and stresses in some floor, where there is change in mass, stiffness and strength. When a structure has an irregularity in mass, stiffness, strength or vertical geometric irregularity, the method of analysis to be used is the most important point.

Now a day's most of the mixed use building configuration have soft-story, which allows to have different story height like, double height first stories, a mezzanine for storage and cafe, intermediate double story height when there is cinema and swimming pool and double height show case facing. The need for construction of RC buildings that deviate from the normal design dimensions, i.e., buildings of variable story heights, particularly with considerably greater first story height compared to the other stories, i.e., building with the so called soft-story (Guevara-Perez, 2012). To satisfy all the requirement of the investors and the architects, it is first of all necessary to provide complete seismic stability of the structure that will guarantee safety of the people in the case of large scale natural disaster, i.e., earthquake (Kanno et al., 2014)

According to current technical and scientific advances, seismic performance evaluation of reinforced concrete structures can be done by two different approaches: deterministic (code based) and probabilistic (performance based) approaches. Now a day the emphasis on seismic design and assessment of reinforced concrete frame structures have shifted from code based (force-based design) to performance-based design so as to assess the strength and ductility for required performance of building. RC frame structure may suffer different levels of damage under seismic-induced ground motions, with potentials for formation of hinges in structural elements, depending on the level of stringency in design. In the code based design the typical building design process is not performance-based and in this typical design process, design professionals select, proportion, and detail building components to satisfy prescriptive criteria contained within the building code. Many of these criteria were developed with the intent to provide some level of seismic performance; however, the intended performance is often not obvious, and the actual ability of the resulting designs to provide the intended performance is seldom evaluated or understood.

While performance-based seismic design is a formal process for design of new buildings, or seismic upgrade of existing buildings, which includes a specific intent to achieve defined performance objectives in future earthquakes. Performance objectives relate to expectations regarding the amount of damage a building may experience in response to earthquake shaking, and the consequences of that damage on overall end users of the building and equipment's attached thereto. In present-generation procedures, performance is expressed in terms of a series of discrete performance levels identified as Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. These performance levels are applied to both structural and nonstructural components, and are assessed at a specified seismic hazard level.

Many parameters are involved in seismic analysis and design of building structures and have uncertainty associated with them. Accordingly, due to the uncertainty in the nature of seismic analysis for building structures, it is not always possible to do deterministic (code based) approach to get accurate and reliable results. Uncertainty caused may be due to change in Material properties, in the structural properties of the building, in the dynamic excitation, Time history data, loading profiles etc.

A probabilistic based approach is the most appropriate to account uncertainties. Accordingly, fragility curve is probabilistic based approach to represent the safety of the structure incorporating the uncertainties involved. Mathematically, fragility curves can be defined as the probability of exceedance of damage at various levels of ground motion, which is considered as an Intensity Measure. Out of the various existing methodologies for development of fragility curves, a method based on nonlinear time history analysis and the probabilistic demand model suggested by Cornell et al (2002) is considered in this study and fragility curves are developed.

Taking into consideration of the above points, the structural building frames must be evaluated and designed by considering the effect of soft-story on seismic performance of the building. The main focus of this research is to study the seismic performance of stiffness irregular building due to significant story height difference on RC buildings by implementing numerical models on the basis of finite element principles. For these different model cases of RC building are analyzed and designed on ETABS 2016.2.1 (CSI ETABS 2016, Integrated Building Design Software, Computers and Structures Inc. Berkeley). Analysis and design of the proposed building model cases followed the conventional design approach as prescribed on the new Ethiopian Buildings Code Standards (ES EN: 2015). While numerical modeling and nonlinear time history analysis of designed building model cases are computationally done on SeismoStruct [SeismoSoft, 2016] which is a fiber-based finite element software package capable of predicting the large displacement behavior of space frames.

Figure 1-1- Shear Failure of Ground Story Columns

1.2 STATEMENT OF THE PROBLEM

A building is referred as a soft-story building when a story level has lower stiffness than the story level above it (according to some codes, the difference could be around 70%). These stiffness irregularity are common to multi story buildings and are not avoidable due to architectural consideration, which makes the soft-story becomes inadequate stiffness to resist horizontal seismic force and susceptible to partial or complete damage of the structure. Therefore, in order to prevent and reduce the collapse of the building it is important to consider and evaluate the effects of stiffness irregularity on seismic performance of the building. It generally accepted that most building structures shall reveal a nonlinear response when subjected to medium-high intensity earthquakes. It is currently known, however, that this phenomenon is not properly modelled in the majority of cases, especially at the design stage, where only simple linear methods have effectively been used. Most of the previous studies have focused on stiffness irregularities due to infill wall; therefore, a stiffness irregularity due to difference in height of stories needs to be studied.

Accordingly, the aim of this study is to evaluate the effect of soft-story on seismic performance of the RC building; and the research is mainly designed reinforced concrete buildings of variable story heights and number of stories using finite element software packages to determine their effects on the seismic performance. Two distinct buildings (G+10 and G+20) are proposed reinforced concrete frames for numerical analysis purpose. This RC building are designed based on the conventional method as per ES EN: 2015 on ETABS 2016.2.1. The design outputs of main structural elements are then used for the numerical model on SeismoStruct [SeismoSoft, 2016]. Pushover and nonlinear dynamic analyses are carried out for obtaining the response in terms fundamental periods, base shear-top displacement, inter-story drift, lateral displacements, and fragility curve at different performance levels.

1.3 OBJECTIVES

1.3.1 GENERAL OBJECTIVE

The general objective of this thesis is to evaluate the seismic performance of stiffness irregular building due to significant story height difference on RC building under seismic excitation.

1.3.2 SPECIFIC OBJECTIVE

- Establishment of limit state capacities at different damage limit states.
- Development of probabilistic seismic demand models (PSDM).
- Development of fragility curves for various performance levels of buildings under the excitation of seismic hazard.

- To evaluate the effect of stiffness irregularity on the distribution inelastic seismic demand and fundamental period of reinforced concrete building.

1.4 SIGNIFICANCE OF THE STUDY

The study used to help and give awareness to design engineers in order to make their design by considering the effects of story height difference on RC buildings in seismically active regions.

The researcher proposes to answer the following problems

- What is the seismic performance of stiffness irregular building due to significant story height difference on RC building under seismic excitation?
- What is the probability of failure of stiffness irregular building due to significant story height difference?
- Does story height difference have an effect on seismic performance as number of stories increase?

1.5 METHODOLOGY AND SCOPE

Seismic Design of proposed RC buildings is performed on ETABS 2016.2.1 following the new Ethiopian Building code analysis and design approach. All building model cases are analyzed both for gravitational loads and earthquake loads by situating proposed study site area in Addis Ababa (earthquake zone-III) using response spectrum method. The numerical values found from the design section are then used for numerical modeling of RC frames on finite element software package (SeismoStruct 2016). Pushover and nonlinear time history analysis are performed on all model cases. Finally, the performance of the model structures at different performance levels has been investigated and their results are discussed in terms of the leading response parameters such as fundamental periods, total base shears, inter-story drifts, lateral displacements, and seismic fragility curves.

The scope and methodology of the research includes the following;

- Detailed literature review was conducted to identify research needs, building selection, seismic record selection, analysis method, performance criteria and the effects of many parameters on the seismic performance and development of fragility curves.
- Six RC building model cases are proposed for mixed use and located at Addis Ababa are designed based on ES EN: 2015 on ETABS 2016.2.1. Earthquake analysis followed modal response spectrum method. Buildings are conventionally designed and optimized in such a way that the whole analysis and designed process followed the current code approach, construction practice, and overall safety and economy.

- Structural details for the above designed building model cases are clearly presented so as to easily use in the numerical modeling for the next following step.
- Six building model cases for each designed models are prepared and modelled on SeismoStruct 2016.

  Model 1 ………. G+20 similar story height building (base case)

  Model 2…………. G+20 (ground floor height is 200% of the story height of other floors)

  Model 3…………. G+20 (ground floor height and second floor height is 200% of the story Height of other floors)

  Model 4………... G+10 similar story height building (base case)

  Model 5…………. G+10 (ground floor height is 200% of the story height of other floors)

  Model 6…………. G+10 (ground floor height and second floor height is 200% of the story Height of other floors)
- 30 artificial accelerograms having different magnitudes are generated, scaled and matched with Ethiopian response spectrum on SeismoArtif [SeismoSoft, 2016] so as to use as earthquake records for nonlinear time history analysis. SeismoArtif is an application capable of generating artificial earthquake accelerograms matched to a specific target response spectrum using different calculation methods and varied assumptions for nonlinear dynamic analysis of new or existing structures.
- Accordingly, six building model cases are loaded with each 30 artificial accelerograms and a total 180 model cases are prepared for the nonlinear time history analysis.
- Nonlinear time history analysis is performed for 180 numerically modeled building cases to develop Probabilistic Seismic Demand Models (PSDMs) and generate seismic fragility curves in Microsoft excel.
- Fundamental periods and capacity curves are generated from the static pushover analysis; and roof displacements and inter-story drifts are sorted from the nonlinear time history analysis.
- Seismic fragility curves are then developed by combining results from pushover and nonlinear time history analysis for all building model cases at various performance levels. Fragility responses are computed and fragility curves (indicator of the probability of failure) for each building model case are developed for different performance levels in terms of PGA by combining the limit state capacities and the PSDMs using Microsoft excel. Out of the various existing methodologies for development of fragility curves, a method based on nonlinear time history analysis and the probabilistic demand model suggested by Cornell et al (2002) is considered in this study. Accordingly, fragility curves are developed for the selected buildings.

- Detail results for each building model cases are presented and final remarks and conclusions have been drawn.

1.6 THESIS ORGANIZATION

This thesis is organized into seven chapters. Introduction, background and statement of problem, objectives of the research, research questions and methodology of the research are discussed in this introductory chapter.

Chapter two discusses the state of the art literature review on different topics related to the current study. An overview of the seismic performance evaluation of the previous researches on mass and stiffness irregularities are given. Limit state capacities, pushover analysis, nonlinear dynamic analysis and fragility curve were reviewed and discussed here.

Chapter three talks a detailed methodology adapted for the overall work of the research is discussed. Performance evaluation involves explicit determination of local and global parameters for detail discussion. Well defined methodologies with regard to design of reinforced concrete buildings, pushover analysis and nonlinear dynamic time history analysis are covered in this chapter. Determination of development of probabilistic seismic demand models (PSDMs), establishment of limit state capacities and fragility curve developments are discussed in detail in this chapter.

Chapter four presents analysis and design of reinforced concrete buildings under the case study and it describes the whole aspects of analysis and design approach as per the new codes. This chapter briefly describes the whole conventional design process and presents the design out puts for the ongoing numerical models simulated on the SeismoStruct software.

Chapter five discusses the non-linear modelling and analysis procedure used in the present study. The chapter briefly describes modelling approaches of infill panels, nonlinear material modeling, nonlinear geometric definition and other modeling aspects of the building. This chapter also describes in detail about the static nonlinear pushover analysis, nonlinear dynamic time history analysis.

Chapter six is all about the results and discussions of the present study, this chapter present the pushover analyses of the designed frames carried out to obtain the structural capacities at different limit states. PSDM and corresponding fragility curves are developed for all the selected frames at each limit state and their comparisons are discussed in this chapter.

Chapter seven presents summary, conclusion, recommendations and scopes of future works.

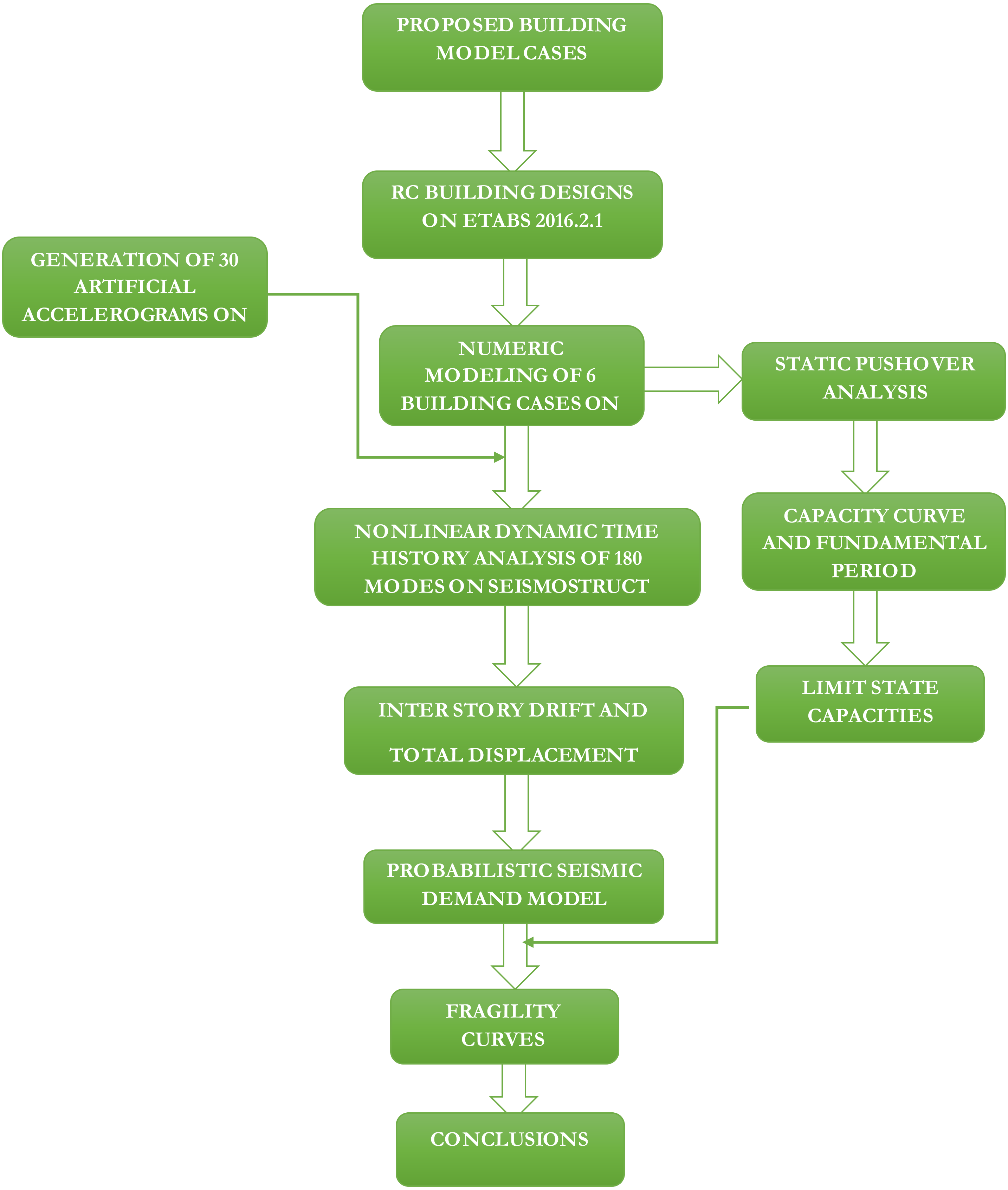


Figure 1-2- Flow Chart Diagram Representing the Overflow Methodology

CHAPTER TWO

2 LITERATURE REVIEW

2.1 INTRODUCTION

This chapter presents an overview of previous work on related topics that provide the necessary background for the purpose of this research. The literature review concentrates on the current state of the art in the seismic performance evaluation of existing buildings, and discusses overview of the relevant published literatures related to the current study. The discussion starts with the literatures on the seismic performance evaluation and then the basics of earthquake design, the review on pushover analysis and limit state capacities followed by a review of published literatures on the fragility curves. The probabilistic seismic demand models are also included. Detailed review on nonlinear structural dynamic analysis and probability-based assessment of building response is also presented.

2.2 SEISMIC PERFORMANCE EVALUATION

A brief review of previous studies on seismic performance evaluation of structures is presented in this section. This literature review focuses on evaluation of seismic performance of structures and past efforts most closely related to the needs of the present study.

Inel et.al. (2008) studied the evaluation of the buildings reflecting existing construction practice. The paper also covered some models with a soft story. It is concluded in that study that, (a) the increase in the confinement level increases the sustained level of damage, (b) the effect of infill are significant in low rise buildings with weaker members, (c) the main reason for a collapse is found to be weak columns and strong beams, (c) the structural irregularities like short column, soft story and heavy overhangs are quite dangerous but the soft story irregularity with a heavy overhang is the most dangerous one, (d) the irregularity effect are found to be more significant in mid-rise structures than the low rise ones, the soft story irregularity formed by the absence of infill at the ground story is found to be more dangerous than the stiffness based ones.

Esteva L., (1992) studied the nonlinear response of buildings with excessive stiffness and strength above the first story. It is stated that the response of a building is quite sensitive to the stiffness variation along the height of the structure and the p-delta effects are significant on the response. The use of a safety factor to meet the local ductility demands in a soft story, which is dependent to the natural period of a structure, is offered.

Chang and Kim (1994) investigated a 20-story building with a soft story by nonlinear time-history and nonlinear pushover analysis. It is stated that low strength reduction factor with perfectly yielding mechanisms are required for effective protection and it is also advised that an amplification factor must be applied to soft stories for which the displacements might be reduced by this way.

Chopra et al. (1973) investigated the yielding point of a soft first story for the adequate protection of upper stories from significant yielding. It is concluded that, to limit the force transmitted to the adjacent story above, an elastic-perfectly plastic mechanism is needed as any residual stiffness increase the shear force transmitted. Even if the first story limits the forces transmitted to upper stories, the resulting shear wave propagates and any weakness of strength in an upper story may lead to collapse. In this paper it is also stated that the first soft story mechanisms must be designed according to very large displacements.

Fernandez, J. (1983) evaluated the effects of uneven distribution of mass and stiffness on the elastic and inelastic response of multi-story buildings. It was noted that the type of earthquake record did not have an appreciable influence in the response of low rise (5story) buildings compared to high rise (10story) buildings. It was further observed that a reduction of first story stiffness by 17% to 67% increased the first story drift by 20% to 100. In general, it was noted that good behavior of structure was obtained when the structure had a continuous variation of mass and stiffness along the height. Costa, A.G. (1990) studied the seismic behavior of reinforced concrete buildings exhibiting vertical irregularities. Sixteen story buildings were studied for three different horizontal layouts and five vertical configurations. The buildings were idealized as a set of plane moment resisting frames connected to shear walls by rigid diaphragms. Based on their study, Costa et al., put forward the following observations. A discontinuity in the frame markedly increased the ductility demand in the shear wall. Further, the distribution of ductility demand was irregular in shear walls but was fairly regular in the frames, except for stories immediately above a discontinuity, where there was a significant increase in frame ductility demand. For irregular buildings, the ductility demands were observed to be nearly twice as high as those of regular buildings. In general, it was noted that if the irregularity occurred in the frame, the shear wall exhibited an increase in ductility demand and vice versa.

All-Ali et.al (1998) showed that mass irregularity only had a limited influence on the seismic performance of buildings, they assumed beams were rigid and plastic hinge formed at all columns ends. Moreover, they showed that increase of mass at the top floor produces a relatively larger effect on story drifts than an increase at middle floor or at the bottom floor of the building.

According to study of Sharany Haque et.al (2008), open ground story buildings should not be treated as ordinary RC framed buildings. In this study, sway characteristics of RC framed buildings with open ground floor reveals that the columns of open ground floor demands much higher allowance for drift. Drift demand of these columns are, in general, about 75% higher than that predicted by conventional equivalent static force method. Thus special detailing of reinforcement, based on designing the building as special moment resisting frame, may be adopted to meet that high ductility demand of the ground floor columns. However, they feel that more research in this area is needed. It has been found that calculation of earthquake forces by treating the common RC framed buildings with open ground floor as ordinary frames results in an underestimation of design force and moment for ground floor columns. Result shows that, when RC framed buildings having brick masonry infill on upper floor with soft ground floor is subjected to earthquake loading, base shear can be more than twice to that predicted by equivalent earthquake force method with or without infill or even by response spectrum method when no infill in the analysis model. Since response spectrum method is seldom used in practice for the design of such buildings, it can be suggested that the design shear and moment calculated by equivalent static method may at least be doubled for the safer design of the columns of soft ground floor.

Davis and Menon (2010) examined the presence of masonry infill panels modifies the structural force distribution significantly in an open ground story building. They considered verities of building case studies by increasing the story heights and bays in open ground story buildings to study the change in the behavior of the performance of the buildings with the increase in the number of story and bays as well as the story heights. They observed that with the total story shear force increases as the stiffness of the building increases in the presence of masonry infill at the upper floor of the building. Also, the bending moments in the ground floor columns increase and the failure is formed due to soft story mechanism that is the formation of hinges in ground story columns.

Moehle et.al. (1986) studied the seismic response of four irregular reinforced concrete test structures. These test structures were simplified models of nine-story three bay building frames comprised of moment frames and frame-wall combinations. Irregularities in the vertical plane of these structures were introduced by discontinuing the structural wall at various levels. Based upon measured displacements and distributions of story shears between frames and walls, it was apparent that the extent of the irregularity could not be gaged solely by comparing the strengths and stiffness's of adjacent stories in a structure. Structures having the same stiffness interruption, but occurring in different stories did not perform equally. It was observed that the curvature ductility demand in beams varied from 3.9 to7.2 and for columns, from 1.8 to 2.9 for an abrupt termination of shear walls at different levels along the height.

Hidalgo, Arias and Cruz (1994) presented an analytical study to determine the influence of vertical structural irregularities on the results of static and response spectrum analyses. Two shear-wall building models, typical of Chilean reinforced concrete construction, were used. The number of stories in each of these models was 20 and 15 respectively. Stiffness irregularities in these models were introduced by reducing the stiffness of one or more floors. The stiffness ratio studied were in the range of 17% to 83% of the original stiffness's the depth of the coupling beam in the case of the coupled shear-wall system was also varied. In order to study the variation of lateral strength over the height, it was assumed that changes in strength were usually associated with changes in stiffness. Effects of mass irregularities were also studied by either increasing or decreasing the mass of one floor with respect to the adjacent floor. The mass ratios studied were in the range of 25% to 175%. The locations of these stiffness, strength, and mass ratios were also varied along the height of the structure and were conceptualized as setbacks. The most relevant conclusions obtained from this study may now be summarized. Considering all cases of vertical structural irregularities, the most critical was that of a setback at mid-height of the building, involving simultaneous reductions in plan geometry, stiffness, and mass. The second most critical was that of reduction in stiffness in the lower stories. Also, an irregular distribution of strength could imply a larger demand of ductility at weak sections near the irregularity during severe earthquakes. It was also pointed out by the authors that for the type of buildings considered in their study, the UBC limitations for using the static analysis procedure were too stringent.

Magliulo et. Al. (2002) investigated 5 and 9 stories reinforced concrete buildings with mass irregularity that has been designed according to Europe building codes. They are concluded that international code can't realizes building regularity and irregularity spatially in distribution of strength and mass.

Poncet and Trembly (2004) concluded design of frame with vertical mass irregularity by linear static procedure gives lower performance than regular frame.
Sadashiva et.al. (2008) illustrated that the effect of irregularity depends on the structural model used, the location and amount of the irregularity, and the analysis method used.

A paper by Arlekar, Jain and Murty (1997) highlights the importance of explicitly recognizing the presence of the open first story in the analysis of the building. The error involved in modelling such buildings as complete bare frames, neglecting the presence of infill in the upper story, is brought out through the study of an example building with different analytical models. This paper argues for immediate measures to prevent the indiscriminate use of soft first story in buildings, which are designed without regard to the increased displacement, ductility and force demands in the first story columns.

Alternate measures, involving stiffness balance of the open first story and the story above, are proposed to reduce the irregularity introduced by the open first story. In this paper, stiffness balancing is proposed between the first and second story of a reinforced concrete moment- resisting frame building with open first story and brick infill in the upper story. A simple example building is analyzed with different models. The stiffness effect on the first story is demonstrated through the lateral displacement profile of the building, and through the bending moment and shear force in the columns in the first story.

In this research (Zubair Ahmed, S; et al. (2014)), aG+5 building is modeled and analyzed in ETABS software for three different cases i.e. model with no infill wall (bare frame), model with bottom story open and model with steel bracing in the bottom story. Dynamic analysis carried out using response spectrum method performance of building evaluated in terms of story drifts, lateral displacement, lateral forces, story stiffness, base shear, time period and torsion.

Hirde S.; Tepugade G. (2004) discussed the performance of a G+20 RC building with soft story at different level along with at GL using nonlinear pushover analysis, founded that plastic hinges developed in columns of ground level soft story which is not acceptable criteria for safe design. Displacement reduces when the soft story is provided at higher level. Hence models retrofitted shear walls.

Kasushik H. B; et al (2009) in this study, several strengthening schemes evaluated for improving the performance of open ground story buildings. Nonlinear analysis was carried out. Also developed a rational method for the calculation of the required increase in strength of open first-story columns. Other strengthening schemes such as providing additional columns, diagonal bracings, and lateral bitterness in the open first story. Code method increased only lateral strength whereas, some of the alternate schemes studied improved both lateral and ductility.

Setia S.; Sharma V. (2012) typical six storied RC frame is analyzed and modelled in STAAD-pro software. Equivalent static performed on five different models. Concluded minimum displacement for corner column is observed in the building in which a shear wall is introduced in X-direction as well as in Z-direction. Buildings with increased column stiffness of ground story perform well in case of story shear.

Maazemd R; Dyavanal S. (2013), they modeled bare frame and soft frame considering them as special and ordinary moment resisting frame (SMRF & OMRF) for medium soil profile 5 under zone III using SAP2000 V 15 software. Equivalent static, response spectrum and nonlinear static pushover analysis was carried out for default hinge properties. It was concluded that the performance of buildings having non-ductile moment resisting frames can be improved by adding infill walls SMRF building models are found to more resistant to earthquake loads as compared to the OMRF building levels.

The study of Nagae, Suita, and Nakashima (2006) focuses on the seismic performance of soft-first-story buildings, which are demanded specially in urban areas. Six-story reinforced concrete buildings are focused on and the seismic response of the soft-first-story structures and typical frame structures are statically assessed based on the results of dynamic response analysis. The mean annual frequency of the maximum of the inter story drift ratio exceeding the specified value is computed, and the mean annual frequencies for the plural values are shown as a seismic hazard curve for each case. Eventually the probabilities of the maximum inter story drift ratios exceeding safety limit states are computed and compared. The soft-first-story buildings with the yield strength coefficient of more than 0.7 showed the same level of safety in comparison with typical frame structures, on the condition that the same deformation capacities are given to the main structural members.

S.Haque, Khan Mahmud Amanat (2008) studied the behavior of the columns at ground level of multi-storied buildings with soft ground floor subjected to dynamic earthquake loading. The structural action of masonry infill panels of upper floors has been taken into account by modelling them as diagonal struts. Finite element models of six, nine and twelve storied buildings are subjected to earthquake load in accordance with equivalent static force method as well as response spectrum method. It has been found that when infill is incorporated in the FE model, modal analysis shows different mode shapes indicating that dynamic behavior of buildings changes when infill is incorporated in the model. Natural period of the buildings obtained from modal analysis are close to values obtained from code equations when infill is present in the model. This indicates that for better dynamic analysis of RC frame buildings with masonry walls, infill should be present in the model as well. Equivalent static force method produces same magnitude of earthquake force regardless of the infill present in the model. However, when the same buildings are subjected to response spectrum method, significant increase in column shear and moment as well as total base shear has been observed in presence of infill. In general, a two-fold increase in base shear has been observed when infill is present on upper floors with ground floor open when compared to the base shear given by equivalent static force method. The study suggests that the design of the columns of the open ground floor would be safer if these are design for shear and moment twice the magnitude obtained from conventional equivalent static force method. Study of the sway characteristics also reveals significantly high demand for ductility for columns at ground floor level. Presence of in filled wall on upper floors demands significant enhancement of column capacity or ductility to cope up with increased sway or drift.

Samir Helou and Abdul Razzaq Touqan (2008), illustrates the importance of the judicious distribution of shear walls. The selected building is analyzed through nine numerical models which address the behavior of framed structures. The parameters discussed include, the fundamental period of vibration, lateral displacements and ending moment. It is noticed that an abrupt change in stiffness between the soft story and the level above is responsible for increasing the strength demand on first story columns.

Extending the elevator shafts throughout the soft story is strongly recommended. Tuladhar and Kusunoki (2008), investigated the seismic performance and design of the masonry infill reinforced concrete frame structure with the soft first story under a strong ground motion. The study also highlighted the error involved in modeling of the RC frame buildings as completely bare frame neglecting stiffness and strength of the masonry wall in the upper floors. The attempt was made to determine the strength increasing factor to account the effect of the story through various 2D analytical models using capacity spectrum method and established its relationship with initial stiffness ratio. In this study, the nonlinear dynamic time history analysis was also carried out with a 3D practical model in order to verify the proposed strength increasing factor.

According to Yong Lu,Tassios, Zhang and Vintzileou (1999) two six-story, three-bay, reinforced concrete frames, one having a tall first story, and the other a discontinuous interior column, were designed in accordance with Euro code 8,2004, and their models were constructed and tested on an earthquake simulator. The main objectives of the investigation were to study the structural effects of these particular irregularities and to check the relevant design code provisions. During tests, Frame having tall first story performed in a reasonably regular manner. For discontinuous interior column frame, the response during moderate earthquakes was strongly influenced by the increased flexibility in the direction towards the missing column side, combined with the gravitational effects on the suspended beam spans. The response of discontinuous interior column frame to strong earthquakes was dominated by an apparent soft first-story mechanism. Both frames exhibited a weakness at the fifth story where discontinuity in stiffness occurred. A base shear over strength factor, with respect to the design required base shear, approximately of the order of three and two, was achieved for tall first story frame and discontinuous interior column frame, respectively.

R.K.L. Su (2008), studied how to improve the general understanding of the seismic response of concrete buildings with transfer structures in low-to moderate seismicity regions. This paper summarizes and discusses the existing codified requirements for transfer structure design under seismic conditions. Based on the previous shaking table test results and numerical findings, the seismic effects on the inelastic behaviors of transfer structures are investigated. The mechanisms for the formation of a soft story below transfer floors, the abrupt change in inter-story drift near transfer story and shear concentration due to local deformation of transfer structures are developed. Design principles have been established for controlling soft-story type failure and minimizing shear concentration in exterior walls supported by transfer structures. The influence of the vertical positioning of transfer floors on the seismic response of buildings has also been reviewed.

A paper of Yong Lu (2002), highlights a comparative study of the nonlinear behavior of reinforced concrete (RC) multi-story structures is carried out on the basis of measured response of four six story, three-bay framed structures, namely a regular bare frame, a discontinuous column frame, a partially masonry-infill frame and a wall-frame system. The structures were designed for seismic requirements in accordance with Euro code 8, 2004, and their models were subjected to similar earthquake simulation tests. Experimental observations and numerical analyses show that the distribution of the story shear over strength is a rather stable indicator of the general inelastic behavior of frames and hence can be employed as a characteristic parameter to quantify the frame irregularity for design purposes. Abrupt discontinuity of the geometry or arrangement of structurally effective elements, where unavoidable, may be compensated by strength enhancement targeting a smoothed over strength profile to allow for distributed inelastic deformation, and this principle applies as well to non-uniformly masonry infill frames. For the wall-frame system, adequate countermeasures against rocking of the RC wall are shown to be a key to maintaining the effectiveness of the system at advanced inelastic response.

According to paper of Poonam et al. (2012), results of the numerical analysis showed that any story, especially the first story, must not be softer/weaker than the story above or below. Irregularity in mass distribution also contributes to the increased response of the buildings. The irregularities, if required to be provided, need to be provided by appropriate and extensive analysis and design processes.

Bariola (1988) investigated the influence of strength and stiffness variation on seismic behavior of structures. He studied the nonlinear response of an 8 story building, with five bays per floor, subjected to five different earthquakes. Three different categories of building periods were considered-low, medium, and high. For every building, two cases were considered, one weak building and one strong. The weak building had base shear strength of 15% of the total weight of the building, while the strong building had base shear strength of 30% of the total weight. The results of this study indicated that the period of a structure increases during an earthquake, with larger period elongation for weaker structures. He stated that if this increases in period is considered along with an increase in damping, a standard linearly-elastic response spectrum can be used to estimate the building response.

Sharooz and Moehle (1990) studied the effects of setbacks on the earthquake response of multi-storey buildings. In an effort to improve design methods for setback structures, an experimental and analytical study was undertaken. In the experimental study, a six story moment-resisting reinforced concrete space frame with 50% setback in one direction at mid-height was selected. The analytical study focused on the test structure and on several analytical representations of setback buildings. The following were the observations from the experimental study. The displacement profiles were relatively smooth over height. Relatively large inter story drifts at the base of the tower were accompanied by a moderate increase in damage at that level. Overall, the predominance of the fundamental mode on the global translational response in the direction parallel to the setback was clear from the lateral displacement and inertia force profiles. Furthermore, the distribution of lateral forces was almost always similar to the distribution specified by UBC. The abrupt reduction in forces at the setback exceeded the static analysis threshold of the UBC; no significant peculiarities in dynamic response were detected. To investigate further, an analytical study was also carried out on six generic reinforced concrete setback frames. These frames were designed by the UBC static method and by a modal analysis procedure. All the setback frames were classified as having an irregular configuration according to the current building code (UBC-1998). The following observations were made from the analytical study. For each of the six setback configurations, all the frames indicated a similar amount and distribution o ductility demand. For all six frames, the floor plan dimensions and mass ratios ranged between 300% and 900% respectively. These were well above the threshold limits for applicability of the static design approach. Nevertheless, the response of only a few frames incurred damage concentration in the tower as indicated by relatively high rotational ductility's. Furthermore, frames having identical plan dimensions and mass ratios but setback at different heights did not experience the same degree of damage. Thus, the approach (UBC1998) in which regular and setback structures were differentiated according to plan dimension or mass ratio appeared to be insufficient.

Wood, S.L. (1992) investigated the seismic behaviour of reinforced concrete frames with setbacks. Two small-scale reinforced concrete test structures with setbacks were constructed and subjected to simulated ground motion. The displacement, acceleration, and shear responses of the setback frames were compared with those of seven previously tested frames with uniform profiles. Each structure considered by Wood in this study was comprised of two identical planar frames. The tower structure was a symmetrical arrangement with a seven story tower and a two story base. The stepped structure was an unsymmetrical arrangement of a three-story tower, a three-story middle section, and a three story base. The first story height was approximately 1.4 times the height of the upper stories. These nine structures were classified using the UBC-1998 definitions for vertical structural irregularities. Based on this study, the following conclusions were drawn. The displacement and shear responses of the setback frames were governed primarily by the first mode. Acceleration response at all levels exhibited the contribution of higher modes. The linear mode shapes for setback frames exhibited kinks that were not present in uniform frames; however, there was no evidence to suggest that kinks adversely influenced the dynamic response of the setback frames. Distributions of maximum story Further based on observations, Wood pointed out that the difference between the nonlinear behavior of regular and

Set back frames do not warrant different design procedures required in building codes (UBC-1998; BOCA-1989).

Pinto and Costa (1995), evaluated the nonlinear seismic behavior of setback buildings with reinforced concrete frames. In the study, a set of 17 different buildings were considered: nine 4 story, four 8 story and four 20 story buildings. All of these structures had the same plan configuration. Hover, with regard to elevation, some were regular but others were irregular with different degrees of setback. The fundamental frequencies of these buildings ranged from 0.49 Hz to 3.20 Hz. This covered all of the key frequencies of the design response spectrum in the Portuguese Code. The main conclusions suggested by the authors may be summarized as follows. For most buildings, it was evident that a greater concentration of the largest values of ductility demands occurred in lower stories. However, some critical zones at intermediate heights were also observed. For buildings with similar frequencies and different heights, the tallest exhibited the greatest values either for the ductility demands or for the story forces at the floor level. The consequences of the irregularities were evident on the shear forces for all of the buildings and on the ductility demands of the 4 story buildings. However, the influence of the irregularities was not evident on the displacements of the 8 and 20 story buildings. The influences of the characteristics of ground motion on the response parameters of the buildings analyzed were also observed.

Aranda et.al. (1982) studied the nonlinear response of irregular reinforced concrete frames. Two reinforced concrete frames, irregular in height, were idealized as single stick models with masses lumped at the floor levels. It was found that irregularities in elevation increase the ductility demand by a factor of 2. This effect was more pronounced where there was a sudden change in the stiffness distribution along the vertical height of the building.

Moehle and Alarcon (1986) presented a combined experimental and analytical study to examine the seismic response behavior of reinforced concrete frame-shear wall structures. In one of the models, vertical irregularity in the frame-shear wall system was introduced by interrupting the shear wall at the first story level. Inelastic dynamic analysis was capable of adequately reproducing measured displacement waveforms, but accurate matching of responses required a trial and error approach to establish the best behavior assumptions. It was observed that in the vicinity of the discontinuity, the elements exhibited a curvature ductility demand 4 to5 times higher in the case of the model without any interruption of the shear wall.

Wong and Tso (1994), who studied the elastic response of set-back structures by means of response spectrum analysis, found that the modal weights of higher order modes for setback structures are Large, leading to a seismic load distribution that is different from static code procedures. They also found that for setback structures, although higher order modes may contribute more to the base shear than the fundamental mode, the first mode still dominates the displacement response.

Humar and Wright (1997), using one ground motion record in their study, found that the difference in elastic and inelastic interstory drifts between setback and regular structures depends on the level of the story considered. For the tower, interstory drifts were found to be larger than for regular structures. For the base, interstory drifts were found to be smaller in "setback" structures than in the regular ones.

Ruiz and Diederich (1989) also studied structures with soft and weak first story subjected to the same soft soil ground motion record from the 1985 Mexico earthquake. They concluded that the behavior of these structures depends on the ratios of the dominant period of the excitation to the effective period of the nonlinear response. They also noted that their results are based on one soft soil record and that different results might be obtained for other kinds of ground motions.

Nassar and Krawinkler [1997] and Seneviratna and Krawinkler (1997) studied an extreme case of a "weak" first story structure, in which all the stories in the MDOF structure stay elastic except for the first story. Both studies concluded that extreme strength discontinuities, such as the "weak" first story they studied, lead to large amplifications in ductility and overturning moment demands and should be avoided whenever possible.

2.3 EARTHQUAKE DESIGN

The objective of design codes is to have structures that will behave elastically under earthquakes that can be expected to occur more than once in the life of the building. It is also expected that the structure would survive major earthquakes without collapse that might occur during the life of the building. To avoid collapse during a large earthquake, members must be ductile enough to absorb and dissipate energy by post-elastic deformations. Nevertheless, during a large earthquake the deflection of the structure should not be such as to endanger life or cause a loss of structural integrity. Ideally, the damage should be repairable. The repair may require the replacement of crushed concrete and/or the injection of epoxy resin into cracks in the concrete caused by yielding of reinforcement. In some cases, the order of ductility involved during a severe earthquake may be associated with large permanent deformations and in those cases; the resulting damage could be beyond repair.

Even in the most seismically active areas of the world, the occurrence of a design earthquake is a rare event. In areas of the world recognized as being prone to major earthquakes, the design engineer is faced with the dilemma of being required to design for an event, which has a small chance of occurring during the design life time of the building. If the designer adopts conservative performance criteria for the design of the building, the client will be faced with extra costs, which may be out of proportion to the risks involved. On the other hand, to ignore the possibility of a major earthquake could be construed as negligence in these circumstances. To overcome this problem, buildings designed to these prescriptive provisions would;

- Not collapse under very rare earthquakes;
- Provide life safety for rare earthquakes;
- Suffer only limited repairable damage in moderate shaking; and
- Be undamaged in more frequent, minor earthquakes.

The design seismic forces acting on a structure as a result of ground shaking are usually determined by one of the following methods:

- Static analysis, using equivalent seismic forces obtained from response spectra for horizontal earthquake motions.
- Dynamic analysis, either modal response spectrum analysis or time history analysis with numerical integration using earthquake records.

2.3.1 DYNAMIC ANALYSIS

The dynamic time-history analysis can be classified as either linear elastic or inelastic (Chopra, 1995). The linear elastic modelling and analysis of Reinforced Concrete (RC) structures is a well-established technique. Several commercial packages for the 3-D elastic analysis of structures are available and are in widespread use (e.g. SAP2000, ETABS, SPACE GASS, etc.). However, the results of the linear analysis are not useful in the determination of the actual behavior of the RC structures and the seismic safety analysis which depends more on inelastic displacement and deformation up to collapse than on forces. It is necessary to take advantage of the inelastic capacity of various components of the structure. The response spectrum approach is based on the linear force response of an equivalent single degree of freedom (SDOF) system. There have been several developments in the response spectrum approach including modification to account for some non-linear effects such as inelasticity, ductility and the response modification factor. The use of the capacity-spectrum technique in the evaluation of RC buildings have been suggested (ATC40, 1996). The recent development in the field of displacement-based response spectra (Bommer et al., 1988; Priestley et al., 2000) represents a promising approach that may be adapted to the simple seismic assessment of buildings. In general, the response spectrum approach has its limitations. It does not account for the different failure modes and sequence of component failure. It does not provide information on the degree of damage or the ultimate collapse mechanism of a deficient RC structure. The inelastic analysis of structures requires a non-linear dynamic time-history procedure past the elastic response and up to collapse (Chopra, 1995).

2.3.2 PUSHOVER ANALYSIS AND LIMIT STATE CAPACITIES

Structures are expected to deform in-elastically when subjected to earthquake, so seismic performance evaluation of structures should be conducted considering this behavior. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as this behavior cannot be determined directly by an elastic analysis. Moreover, maximum in-elastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behavior.

Various simplified nonlinear analysis procedures and approximate methods to estimate maximum inelastic displacement demand of structures are proposed in literature. The widely used simplified nonlinear analysis procedure, pushover analysis is discussed here.

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral force with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate the force-displacement curve of the overall structure.

A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. The structure is subjected to predefined lateral load patterns which are distributed along the building height. The lateral forces are increased until some member's yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional member's yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

The published reports ATC 40 (1996) and FEMA 273 (1997) highlighted the non-linear static pushover analysis. It is an efficient method for the performance evaluation of a structure subjected to seismic loads. The step by step procedure of the pushover analysis is to determine the capacity curve, demand curve and performance point. These reports deal with modelling aspects of the hinge behavior, acceptance criteria and procedures to locate the performance point.

Jaswant (1997) studied nine different models of the building. The buildings were considered to be located in seismic zone III. Linear elastic analysis was performed for the models of the building using ETABs analysis package. Two different analyses were performed on the models of the building considered in this study, namely the equivalent static analysis and the multi model dynamic analysis. Finally suggested that, the buildings are located in Zone-III will exhibit poor performance during a strong earthquake. This hazardous feature of Indian RC frame buildings needs to be addressed immediately and necessary measures should be taken to improve the performance of the buildings.

Habibullah and Stephen (1998) described the use of SAP2000 for the performing a pushover analysis of a simple three dimensional building. SAP2000 is a state-of-the-art, general purpose, and three dimensional structural analysis programs. SAP2000 has static pushover analysis capabilities which were fully integrated into the program; allow quick and easy implementation of the pushover procedures for both two and three dimensional frames.

Helmut and Seneviratna (1998) discussed that, the pushover analysis would be a great improvement over presently employed elastic evaluation procedures and they also pointed out that a carefully performed pushover analysis would provide insight into structural aspects that control performances during severe earthquakes. Further it was concluded that, for structures that vibrate primarily in the fundamental mode, the pushover analysis would provide good estimates of global as well as local inelastic, deformation demands. These analyses also expose design weaknesses that may remain hidden in an elastic analysis.
Elnashai (2001) analyzed the dynamic response of structures using static pushover analysis. The significance of pushover analysis as an alternative to inelastic dynamic analysis in seismic design and assessment were discussed. New developments towards a fully adaptive pushover method accounting for spread of inelasticity, geometric non-linearity, full multimodal, spectral amplification and period elongation within a framework of fiber modelling of materials were discussed and preliminary results were given. These developments lead to static analysis results that were closer than ever to inelastic time-history analysis.

Santosh kumar (2003) studied the evaluation of multi-story buildings with and without considering the stiffness of infill located in zone III. The study compromised of seismic loads, gravity load analysis and lateral load analysis as per the seismic code for the bare and infill structure by considering different analytical models, and their evaluation was carried out using pushover analysis. The results in terms of natural periods, lateral deformation and ductility ratio were compared for the different building models. It was concluded that the performance point of all the building models considered for the study falls before the life safety point. Hence the buildings need not be retrofitted. Base shear capacity was observed to be 20 greater than the design base shear; therefore, the building has safe under design basis earthquake.

2.4 FRAGILITY CURVES AND PROBABLISTIC SEISMIC DEMAND MODELS

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

Mosalam [1997] studied on behavior of low-rise Lightly Reinforced Concrete (LRC) frames with and without masonry infill walls using fragility curves. Adaptive nonlinear static pushover analyses were performed for the frame models. Monte Carlo simulation was used to generate the frame models considering uncertainties in material properties.

Ellingwood [2001] highlighted the importance of the probabilistic analysis of building response in understanding the perspective of building behavior. This paper outlined a relatively simple procedure for evaluating earthquake risk based on seismic fragility curve and seismic hazard curve. This study shows the importance of inherent randomness and modelling uncertainty in forecasting building performance through a building fragility of a steel frame.

Cornell et al [2002] investigated a formal probabilistic framework for seismic design and assessment of structures and its application to steel moment-resisting frame buildings based on the 2000 SAC, Federal Emergency Management Agency (FEMA) steel moment frame guidelines. The framework is based on realizing a performance objective expressed as the probability of exceedance for a specified performance level. That related to demand and capacities of that are described by nonlinear, dynamic displacements of the structure. One of the spectral acceleration at the approximate first. Probabilistic models distributions were used to describe the randomness and uncertainty in the structural demand given the ground motion level, and the structural capacity. A common probabilistic tool the total probability theorem was used to convolve the probability distributions for demand, capacity, and ground motion intensity hazard. This provided an analytical expression for the probability of exceeding the performance level as the primary product of development of framework. Consideration of uncertainty in the probabilistic modelling of demand and capacity allowed for the definition of confidence statements for the likelihood performance objective being achieved.

Tantala and Deodatis [2002] considered a 25 story of reinforced concrete moment resisting frame Building having three-bays. They have generated fragility curves for a wide range of ground motion intensities. They have used time histories are modelled by stochastic processes. Simulation is done by power spectrum probability and duration of earthquake by conducting 1000 simulation for each parameter. The nonlinear analysis is done by considering the P-Δ effects and by ignoring soil-structure interaction. They have considered the nonlinearity in material properties in model with nonlinear rotational springs a bilinear moment-curvature relationship by considering the stiffness degradation through hysteretic energy dissipation capacity over successive cycles of the hysteresis. They have used Monte Carlo simulation approach for simulation of the ground motion. The simulation for the durations of strong ground motions is done at 2, 7 and 12 seconds labels to observe the effects. They considered the effects of the assumption of Gaussianity and duration. They have adopted stochastic process for modelling. The analyses were done by using DRAIN-2D as a dynamic analysis with inelastic time histories data. The random material strengths were simulated for every beam and column using Latin Hypercube sampling.

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

Celik and Ellingwood [2010] studied the effects of uncertainties in material, structural properties and modelling parameters for gravity load designed RC frames. It was found that damping, concrete strength, and joint cracking have the greatest impact on the response statistics. However, the uncertainty in ground motion dominated the overall uncertainty in structural response. The study concluded that fragility curves developed using median (or mean) values of structural parameters may be sufficient for earthquake damage and loss estimation in moderate seismic regions.

Elnashai (2004) was followed to derive the fragility curves. The frames were modelled with randomly generated material strength parameters. The statistical analysis of structural demand indicated that the effect of material uncertainty is negligible with respect to that of ground motion uncertainty. The comparison of fragility curves developed using different sets of ground motions revealed that the fragility curves depend considerably on the choice of the ground motions.

Ramamoorthy et al. (2006) developed fragility curves for low-rise RC frames. Cloud analysis was carried out based on NTHA to develop the structural demand. A bilinear function was used here to represent the median demand instead of a linear function given in Cornell et al. (2002).

Kircil and Polat (2006) developed fragility curves for mid-rise RC buildings in Istanbul region designed according to the Turkish seismic design code. Typical buildings with different stories were considered ranging from 3 to 7 stories. Twelve artificial ground motions were used to perform incremental dynamic analyses to determine the yielding and collapse capacity of each sample building. This study proposes an equation for immediate occupancy (IO) and collapse prevention (CP) performance levels as a function of number of stories and concluded that these equations may be used for the preliminary evaluation of mid-rise RC framed structures designed with 1975 version of the Turkish seismic design code.

Lagaros (2008) conducted fragility analyses for two groups of reinforced concrete buildings. The first group of structures was composed of fully infilled, weak ground story and short columns frames and the second group consists of building frames designed with different values of behavioral factors. Four limit state fragility curves were developed on the basis of nonlinear static analysis and 95% confidence intervals of the fragility curves were calculated. This study concludes that the probability of exceedance of the slight damage state for the design earthquake (0.30g) is of the same order for first group of building frames. On the other hand, it was found that the probability of exceedance for the fully infilled frame is one and three orders of magnitude less than that of the weak ground story and short column frames for the moderate and complete damage states, respectively. This study shows that the behavior factor significantly affects the fragility curves of the buildings.

CHAPTER THREE

3 METHODOLOGY

3.1 GENERAL

The seismic performance evaluation requires performance based approaches rather than code based approaches due to uncertainties involved. The major uncertainties are in the material properties of concrete and steel, time history data, building geometries etc. The seismic performance of the buildings depends on these uncertainties.

The methodology of this research yields one of the performance evaluation procedures that explicitly accounts for the randomness (measure of our inability to precisely understand the factors that affect phenomena such as seismic loadings and capacity of structures) and uncertainty (measure of the error introduced into calculations as a result of our inability to precisely characterize reality, e.g. seismic methods, structural models and etc.) inherent in performance prediction. Acceptable seismic performance is in this way defined by an explicit quantification of the confidence level at which the performance objective has been achieved.

The objective of this study was to evaluate the seismic performance of stiffness irregular buildings due to significant story height difference on RC buildings.

Seismic Design of proposed RC buildings is performed on ETABS 2016.2.1 following the new Ethiopian Building code analysis and design approach. All building model cases are analyzed both for gravitational loads and earthquake loads by situating proposed study site area in Addis Ababa (earthquake zone-III) using response spectrum method. The numerical values found from the design section are then used for numerical modeling of RC frames on finite element software package (SeismoStruct 2016). Pushover and nonlinear time history analysis are performed on all model cases. Finally, the performance of the model structures at different performance levels has been investigated and their results are discussed in terms of the leading response parameters such as fundamental periods, total base shears, inter-story drifts, lateral displacements, and seismic fragility curves.

3.2 BUILDING SELECTION

In order to evaluate seismic performance of stiffness irregular building due to significant story height difference different moment resisting frames was selected and designed as a case study for the evaluation. The selected buildings are eleven-story (G+10) and twenty-one-story (G+20) buildings with different stiffness irregularity at different level. The main purpose of having varying number of story and story height as the case study is to investigate the seismic performance of stiffness irregular building due to significant story height difference. All building models are proposed to be situated at Addis Ababa where the current building code classified as seismic zone III. After preparing general architectural plans for the proposed building models, analysis and seismic design of frame elements are performed according to new Ethiopian Buildings Code Standards (ES EN: 2015). The design process comprised preparing a basic structural analysis model of the building with the dimensions and details obtained from preliminary design strategies. Then apply design lateral forces, perform structural analysis, and then design structural elements based on stress resultants obtained from structural analysis. Seismic action is used as governing lateral force on the building structures and the analysis for the lateral action followed modal response spectrum method. The proposed building models are classified as regular both in plan and elevation that the parameters and results of the intended study could easily be interpreted. All analyses and designs are performed on ETABS 2016 software (CSI 2016. ETABS. Integrated Building Design Software, Computers and Structures Inc. Berkeley). A three dimensional (spatial) structural model is used for all cases. The model cases are multistory reinforced concrete buildings composed of frame system and solid slab floors. Beams, supporting floors and columns are continuous and meet at nodes, often called "rigid" joints. Such frames can readily carry gravity loads while providing adequate resistance to horizontal forces, acting in any direction. Once the designs of the building model cases are complete, structural details of the members are prepared and presented in the way that they are clear for numerical modeling purpose with infill wall configurations. The details of analysis and design of the building model cases are presented under distinct section in chapter four (4).

3.3 SEISMIC PERFORMANCE EVALUATION

Through advances in computer analysis techniques as the computer technology, nonlinear structural analysis becomes possible (Irtem et al., 2007). Moreover, probabilistic analysis could be added in to the seismic assessments, which makes the analyses more accurate and more dependable. In structural behavior assessment analysis, with the technological developments in computing in civil engineering, the deterministic assessment methods are thought to be insufficient to define structural behavior under earthquake effect. Due to the uncertainty and random variables in the analysis it is necessary to include probabilistic assessment into the analysis. Including the probabilistic approaches into the analyses for definition of seismic structural behavior will give more rational results. A probabilistic methodology is realized for using to make a rehabilitation decision according to seismic hazard and system performance (FEMA 273, 274, 356, 1997; FEMA 440, 2005).

The push-over and nonlinear dynamic time history analyses are performed using finite element analysis software to evaluate the seismic performance of the case study buildings. To predict the response of the selected structures during an earthquake, representative earthquake data record for that location should be used. However, there is not adequate recorded ground motion data to characterize the high seismicity of specific locations in the Addis Ababa Region. Therefore, 30 artificial accelerograms using Seismo-Artif 2016 are generated, scaled, and matched with Ethiopian response spectrum and loaded on all building model cases for nonlinear dynamic time history analysis.

3.4 ARTIFICIAL ACCELEROGRAMS

The assessment of performances and demands for bare and infilled frames necessitated the availability of a set of acceleration time histories with amplitude, frequency content, and duration enclosed into certain limits in order to reduce the dispersion of the corresponding demand parameters. However, in most cases, using time histories from actual earthquake data has many limitations for many reasons. Hence, artificial time history sets, generated from response spectra, are widely used instead. ES EN 1998-1:2015 recommends using artificial accelerograms for seismic motion input depending on the information available and nature of application. It stipulates that artificial accelerograms shall be generated so as to match the elastic response spectra used in the design for 5% viscous damping. Also it has been stated that with the absence of site specific data, the minimum duration of stationary part of part of the accelerograms should be equal to 10sec; and a minimum of 3 accelerograms should be used. Accordingly, in this paper 30 artificial accelerograms are generated on SeismoArtif (SeismoSoft 2016) having 30sec duration and different frequencies and magnitudes. Generated artificial accelerograms suits with elastic response spectra conforming to the used code. SeismoArtif is an application capable of generating artificial earthquake accelerograms matched to a specific target response spectrum using different calculation methods and varied assumptions. The program is capable of reading accelerograms and spectra saved in different text file formats. The generated artificial accelerograms are then used in the simulation of nonlinear dynamic time history analysis for numerically modeled building cases.

3.5 STATIC PUSHOVER ANALYSIS

Pushover analysis is a static, nonlinear procedure using simplified nonlinear techniques to estimate structural deformations. It is an incremental static analysis used to determine the force-displacement relationships, or the capacity curve, for structure or structural element. 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 at each increment, until the structure collapse condition. Pushover analysis is the preferred tool for seismic performance evaluation of structures by the major rehabilitation guidelines and codes. In pushover analysis an inelastic model is developed and is subjected to gravity load followed by a monotonically increasing static lateral load. The load pattern is defined in that way that incremental static load is applied by the program until the specified target displacement is attained. The analysis is continued till the structure collapses, or the building reaches certain level of lateral displacement. It provides a load versus deflection curve of the structure starting from the state of rest to the ultimate failure of the structure. The load is representative of the equivalent static load of the fundamental mode of the structure. It is generally taken as the total base shear of the structure and the deflection is selected as the top-story deflection. In this paper pushover analysis is monitored is to evaluate the expected performance of a simulated structural models case systems by estimating their strength and deformation demands in design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift, and inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behavior.

3.6 NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

Nonlinear dynamic analysis using the combination of ground motion records with a detailed structural model theoretically is capable of producing results with relatively low uncertainty. In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model. Higher-level demands (element distortions, story drifts, and roof displacement) are derived directly from the basic component actions. There is still uncertainty with the detailed models, associated primarily with the lack of data on actual component behavior, particularly at high ductility. In addition, the variability of ground motion results in significant dispersion in engineering demand parameters. Accordingly, this uncertainty is taken care of by developing probabilistic seismic demand model (PSDM) which depicts results from a series of nonlinear dynamic analyses for increasingly larger intensities of ground shaking. At each level of intensity, the multiple time histories produce a probabilistic distribution of results in terms of a selected engineering demand parameter.

3.7 BUILDING PERFORMANCE LEVELS

To define the fragility function demand, parameters are compared with the selected structural limit states or building performance levels. Building performance levels are defined as approximate limiting levels of structural and non -structural damage that may be expected during an earthquake. It can be described qualitatively in terms of the following parameters:

- Safety afforded to building occupants, during and after an earthquake.
- cost and feasibility of restoring the building to pre-earthquake conditions
- Length of time, the building is removed from service to conduct repairs.
- Economic, architectural, or historic impacts on the community at large.

These performance characteristics will be directly related to the extent of damage sustained by the building during a damaging earthquake. Three important performance levels (Damage Limitation, Significant Damage and Collapse Prevention) are being considered in the present study as discussed in the following sections and illustrated graphically in Figs. 3.1 for bare and frame (Dolsek and Fajfar, 2008).

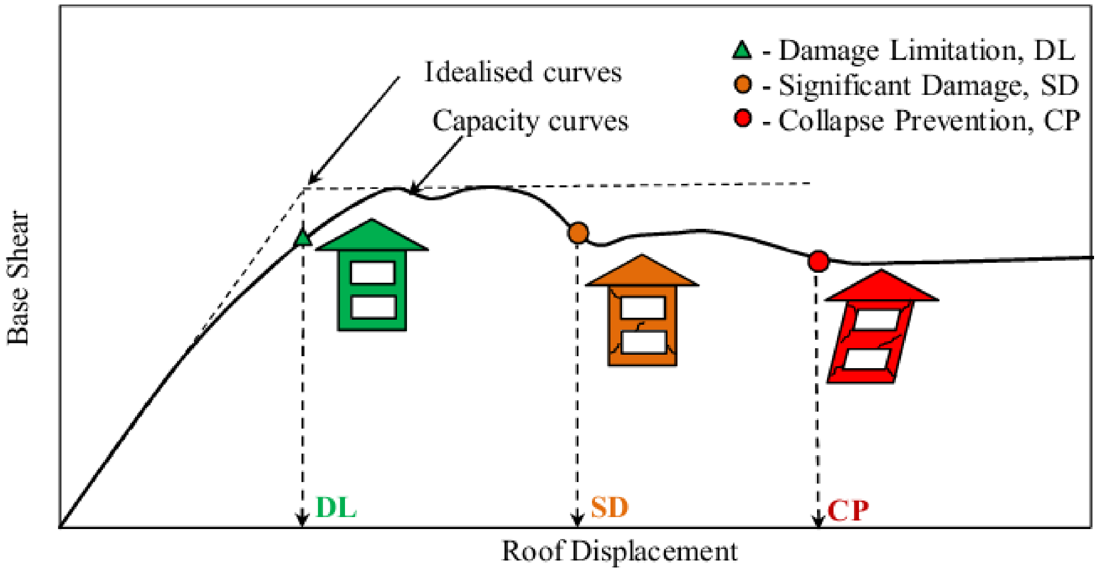


Figure 3-1- Typical Performance Levels for Bare Frame

3.7.1 DAMAGE LIMITATION (DL)

In this performance level, overall damage to the building is light. Damage to the structural systems is very less, however, somewhat more damage to non-structural systems is expected. Non-structural components such as cladding and ceilings and mechanical and electrical components remain secured; however, repair and cleanup may be needed. It is expected that utilities necessary for normal function of all systems will not be available, although those necessary for life safety systems will be available. Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake ground motion. In addition, some owners may desire such performance for very important buildings, under severe levels of earthquake ground shaking. At this limit state, masonry infill walls attain its maximum strength.

3.7.2 SIGNIFICANCE DAMAGE (SD)

Structural and non-structural damage in this performance level is significant. The building may lose a substantial amount of its pre-earthquake lateral strength and stiffness, but the gravity-load bearing elements function. Out-of-plane wall failures and tipping of parapets are not expected, but there will be some permanent drift and select elements of the lateral -force resisting system may have substantial cracking, spalling, yielding, and buckling. Non-structural components are secured and do not present any a falling hazard, but many architectural, mechanical, and electrical systems are damaged. The building may not be safe for continued occupancy until repairs are done. Repair of the structure is feasible, but it may not be economically attractive to do so. Masonry infill walls lose its complete strength at this level.

3.7.3 COLLAPSE PREVENTION LEVEL OR NEAR COLLAPSE LEVEL (CP)

The structure sustains severe damage. The lateral-force resisting system loses most of its pre- earthquake strength and stiffness. Load-bearing columns and walls function, but the building is near collapse. Substantial degradation of structural elements occurs, including extensive cracking and spalling of masonry and concrete elements, and buckling and fracture of steel elements. Infills are completely failed. The building has large permanent drifts. Non-structural components experience substantial damage and may be falling hazards. The building is unsafe for occupancy. Repair and restoration is probably not practically achievable. This building performance level results in mitigation of the most severe life-safety hazards at relatively low cost.

3.8 DEVELOPMENT OF PROBABLISTIC SEISMIC DEMAND MODEL (PSDM)

In order to obtain a probabilistic evaluation of seismic structural performance of a given structure for a given seismic hazard, it is necessary to know the relationships between ground motion intensity measures, IMs, and engineering demand parameters, EDPs. These relationships are denoted as the probabilistic seismic demand models, PSDMs. The seismic demand (SD) is usually described through probabilistic seismic demand models (PSDMs) particularly for nonlinear time history analyses which are given in terms of an appropriate intensity measure (IM). It has been suggested by Cornell et. al. (2002) (also known as 2000 SAC FEMA method), that the estimate of the median demand EDP (SD) can be represented in a generalized form by a power model as given in the following equation.

$$EDP = a\,(IM)^{b} \quad \text{……..Equation 3-1}$$

Where: ‘a’ and ‘b’ are the regression coefficients of the PSDM. Eq. 3.1 can be rewritten for system fragilities as follows:

$$P\left(D \geq \frac{C}{IM}\right) = 1 - \emptyset\left(\frac{ln(S_c) - ln(a.IM^b}{\sqrt{\beta_{D/IM} + \beta_c + \beta_M}}\right) \text{............Equation 3-2}$$

The dispersion, βD/IM, of inter-story drifts (di) from the time history analysis can be calculated using Eq. 3.3 where a (IM) ^{b} represents the mean inter-story drift.

$$\beta_{D/IM} \cong \sqrt{\frac{\Sigma\left(ln(d_i) - ln(aIM)^b\right)^2}{N-2}} \text{................Equation 3-3}$$

Uncertainty associated with building definition and construction quality (βc) accounts for the possibility that the actual properties of structural elements (e.g., material strength, section properties, and details such as rebar location) might be different than those otherwise believed to exist. Values of βc are assigned based on the quality and confidence associated with building definition. For existing buildings, this will depend on the quality of the available drawings documenting the as-built construction, and the level of field investigation performed to verify their accuracy. For new buildings, this will be determined based on 60 assumptions regarding how well the actual construction will match the design. ATC 58 (2012) recommends values for βc under representative conditions. In the present study, βc is considered as 0.25 which represents the building design is completed to a level typical of design development, construction quality assurance and inspection are anticipated to be of limited quality. According to ATC 58 (2012), modelling uncertainty (βm) is the result from inaccuracies in component modelling, damping and mass assumptions. For the purpose of estimating βm, this uncertainty has been associated with the dispersion of building definition and construction quality assurance (βc) and the quality and completeness of the nonlinear analysis model (βq).

The total modelling dispersion can be estimated as follows:

$$\beta_m = \sqrt{\beta_c^2 + \beta_q^2} \text{................Equation 3-4}$$

β_q recognizes that hysteretic models may not accurately capture the behavior of structural components, even if the details of construction are precisely known. Values of β_q are assigned based on the completeness of the mathematical model and how well the components deterioration and failure mechanisms are understood and implemented. Dispersion should be selected based on an understanding of how sensitive response predictions are to key structural parameters (e.g., strength, stiffness, deformation capacity, in -cycle versus cyclic degradation) and the likely degree of inelastic response in this study, βq is assumed to be 0.25 representing that numerical model for each component is robust over the anticipated range of displacement or deformation response. Strength and stiffness deterioration is fairly well represented though some failure modes are simulated indirectly. The mathematical model includes most structural components and non-structural components in the building that contribute significant strength or stiffness.

To obtain P [EDP|IM], it is necessary to define pairs of IM and EDP, i.e. to define probabilistic seismic demand models (PSDMs). Relationships between IM and EDP are obtained by analyzing the results of nonlinear time-history analyses of structure responses under earthquakes of different intensity. In this paper different PSDMs, which are used in the probabilistic seismic performance evaluation of RC frame structures, are examined. Different reinforced concrete frame building structures with different cases, all designed according to the EBCS design provisions, and are used as the prototypes for the analysis. All RC framed buildings are exposed to the ground motions of different intensity.

The 30 ground motions are scaled linearly from 0.1g to 1g and each computational model is analyzed for a particular earthquake (randomly selected) with a particular PGA. A total of 30 nonlinear dynamic times history analyses are performed and the maximum inter-story displacement (EDP) for selected story are monitored. The inter-story drifts (maximum of all stories) along with the corresponding PGAs (IM) are plotted in a logarithmic graph. Each point in the plot represents the PGA values and the corresponding percentage of maximum inter -story drift in each of the 30-time history analysis for all the frames. A power law relationship for each building is fitted using regression analysis, which represents PSDM model for the corresponding frames. The regression coefficients 'a' and 'b' are found for each building. The PSDM model provides the most likely value of inter-storey drift (in mm) in the event of an earthquake of certain PGA (up to 1g) in each frame. Depending on the values of parameters 'a' and 'b' the vulnerability of the particular building is identified.

PSDMs are generally developed from the analysis of NTHA results. Step by step: procedure for development of PSDM models is presented as follows:

I. Select a suite of ground motions (N" number of records) representing a broad range of values for the chosen intensity measure.
II. Create 'N' number of statistical models of the subject structure. These models should be created by sampling on various modelling parameters which may be deemed significant (e.g. material strength, damping ratio). Thus, N statistically significant yet nominally identical samples are made.
III. Perform a nonlinear time history analyses for each ground motion for set of developed structures.
IV. For each analysis, peak responses are recorded and plotted against the value of the intensity measure for that ground motion. A regression analysis of these data is then used to develop PSDM models.

3.9 DEVELOPMENT OF FRAGILITY CURVES

In last decades, through further development of computer technology in civil engineering, so many different seismic analyses became possible and accuracy of the analysis is increased. Therefore, there are lots of methodologies for seismic assessment in use. Including the probabilistic approaches into the seismic assessment offer more realistic approaches and it is the most appropriate to account uncertainties. Recently, seismic assessments are done with this consideration. Fragility analysis is one of them to represent the safety of the structure incorporating the uncertainties involved. The fragility analysis which is a system reliability analysis with correlated demands and capacity is performed with different methodologies to establish the probabilistic characterization of the demands in different aspects. In the present study, probabilistic seismic analyses to define the structural seismic behaviors are evaluated. A representative RC frame structure is taken in to consideration in the analytical part.

A probabilistic based approach is the most appropriate to account the uncertainties. Characterizing the probabilistic nature of structural parameters is done through the use of Fragility Curves in the present study. Fragility curve is probabilistic based approach to represent the safety of the structure incorporating the uncertainties involved. Out of the various existing the safety of the structure incorporating the uncertainties involved. Out of the various existing methodologies for development of fragility curves, a method based on nonlinear time history analysis and the probabilistic demand model suggested by Cornell et al (2002) is considered in the present study.

A fragility analysis assesses the probability that the seismic demand placed on the structure exceeds the capacity conditioned on a chosen Intensity Measure (IM), representative of the seismic loading. Demand (D) and capacity (C) are assumed to follow a lognormal distribution, and the probability of exceeding a specific damage state for a particular component can be estimated with the standard normal cumulative distribution function as per Cornell et. al, (2002).

Fragility curves represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation, they also involve uncertainties associated with structural capacity, damage limit state definition and records of ground motion accelerations. And mathematically, fragility curves can be defined as the probability of exceedance of damage at various levels of ground motion, which is considered as an Intensity Measure. The fragility function represents the probability of exceedance of a selected Demand Parameter (EDP) for a selected structural limit state (LS) for a specific ground motion intensity measure (IM). Fragility curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand. The seismic fragility, FR (x) can be expressed in closed form using the following equation as per Cornell et. al. (2002); and a fragility curve is obtained for different limit states using this equation.

$$P\left(D \geq \frac{C}{IM}\right) = 1 - \emptyset\left\{\frac{ln\frac{S_C}{S_D}}{\sqrt{\beta_{D/IM}^2+\beta_C^2+\beta_M^2}}\right\} \text{……………….Equation 3-5}$$

Where,

- ✓ 'D' is the drift demand,
- ✓ 'C' is the drift capacity at chosen state,
- ✓ S_C and S_D are the chosen limit state and the median of the demand (LS) respectively.
- ✓ βd/IM, βc and βM are dispersions in the intensity measure, capacities and modelling respectively.

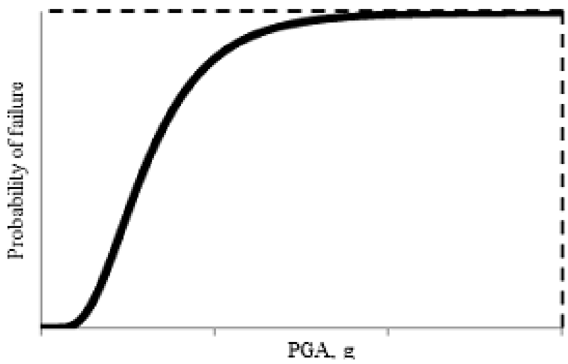


Figure 3-2- Schematic of Seismic Fragility Curve

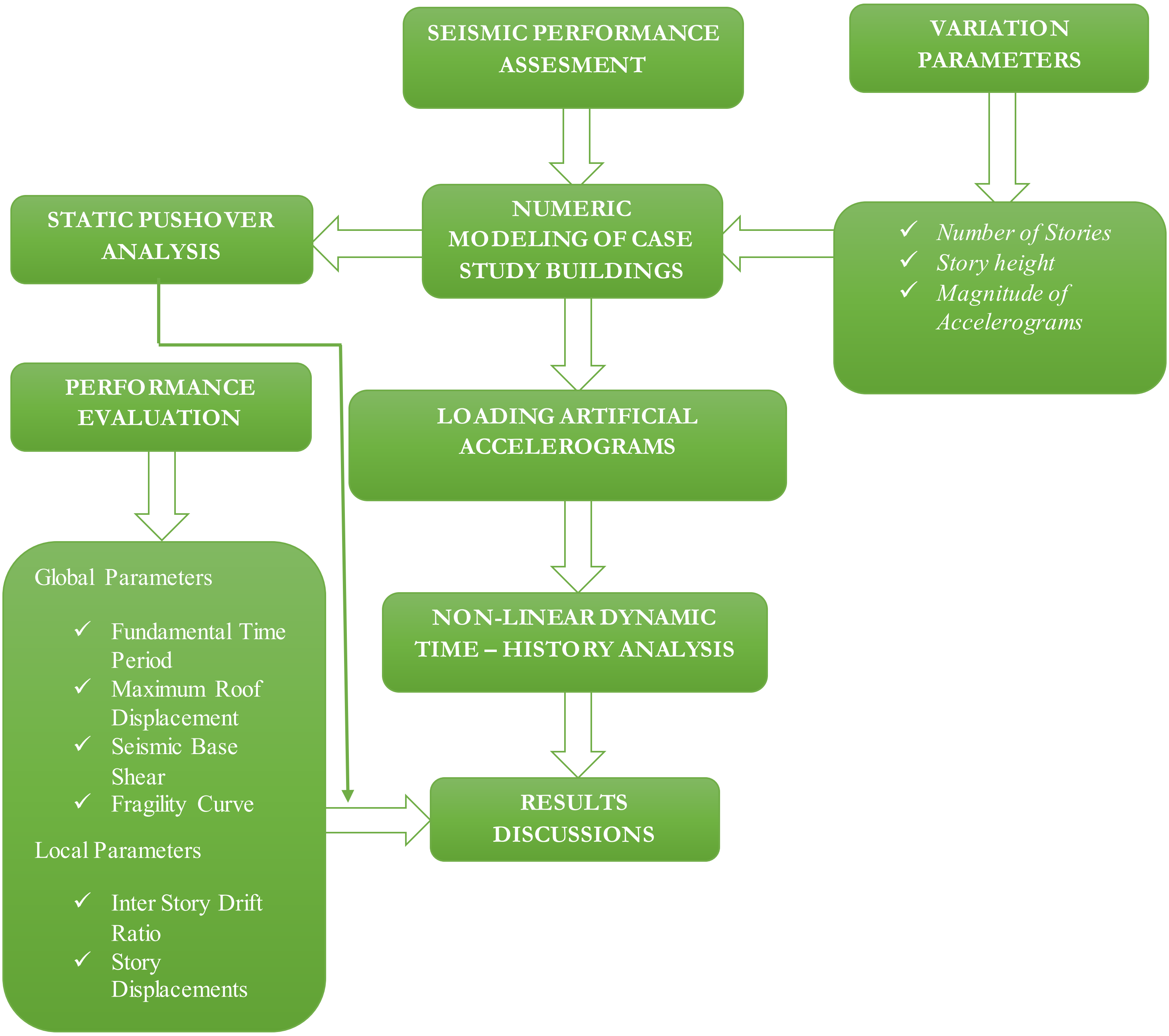


Figure 3-3- Seismic Performance Assessment Framework

CHAPTER FOUR

4 ANALYSIS AND DESIGN OF REINFORCED CONCRET BUILDINGS

4.1 GENERAL DESCRIPTION OF THE BUILDINGS

The investigated buildings are a multi-story reinforced concrete mixed use buildings for apartment, office and shop, and two building models having different number of story and story height eleven-story (G+10) and twenty-one-story (G+20) with similar floor plans and functions are used for the study. Seismic action is used as governing lateral force on the building structures and the analysis for the lateral action followed modal response spectrum method. The proposed building models are classified as regular both in plan and elevation that the parameters and results of the intended study could easily be interpreted. All analyses and designs are performed on ETABS 2016 software (CSI 2016. ETABS. Integrated Building Design Software, Computers and Structures Inc. Berkeley). A three dimensional (spatial) structural model is used for all cases. The model cases are multistory reinforced concrete buildings composed of frame system and solid slab floors. Beams, supporting floors and columns are continuous and meet at nodes, often called "rigid" joints. Such frames can readily carry gravity loads while providing adequate resistance to horizontal forces, acting in any direction.

Six building model cases for each designed models are prepared and modelled on ETABS 2016 software (CSI 2016. ETABS. Integrated Building Design Software, Computers and Structures Inc. Berkeley).

Model 1 ………. G+20 similar story height building (base case)

Model 2…………. G+20 (ground floor height is 200% of the story height of other floors)

Model 3…………. G+20 (ground floor height and second floor height is 200% of the story Height of other floors)

Model 4………... G+10 similar story height building (base case)

Model 5…………. G+10 (ground floor height is 200% of the story height of other floors)

Model 6…………. G+10 (ground floor height and second floor height is 200% of the story height of other floors)

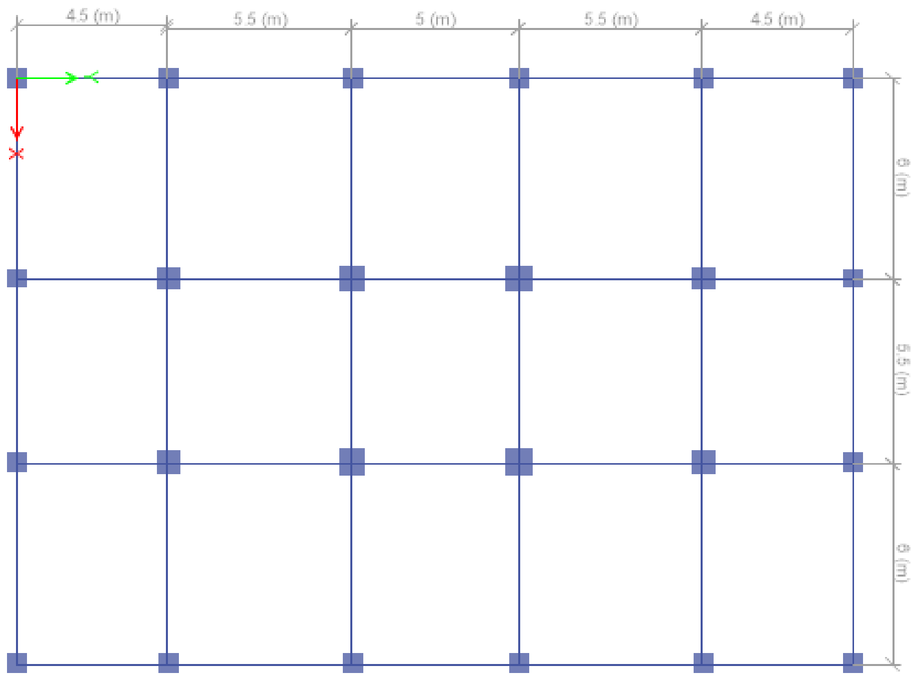
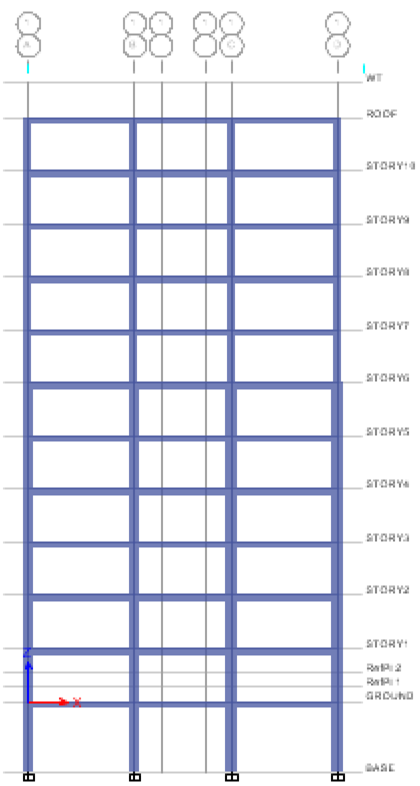


Figure 4-1- Typical Plan Considered for Analysis



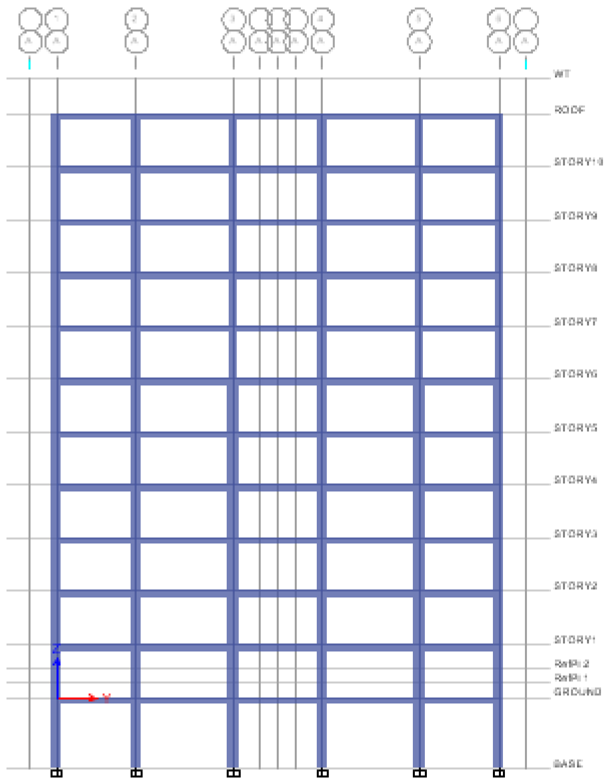


Figure 4-2- Model4 (along x-axis) **Figure 4-3-** Model4 (along y-axis)

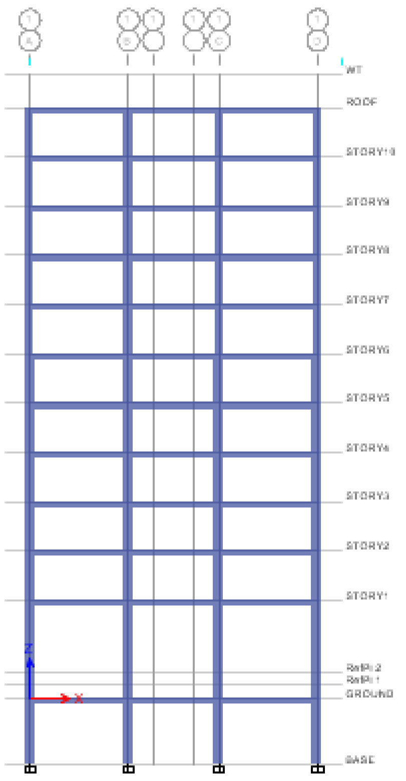


Figure 4-4- Model 5 (along x-axis)

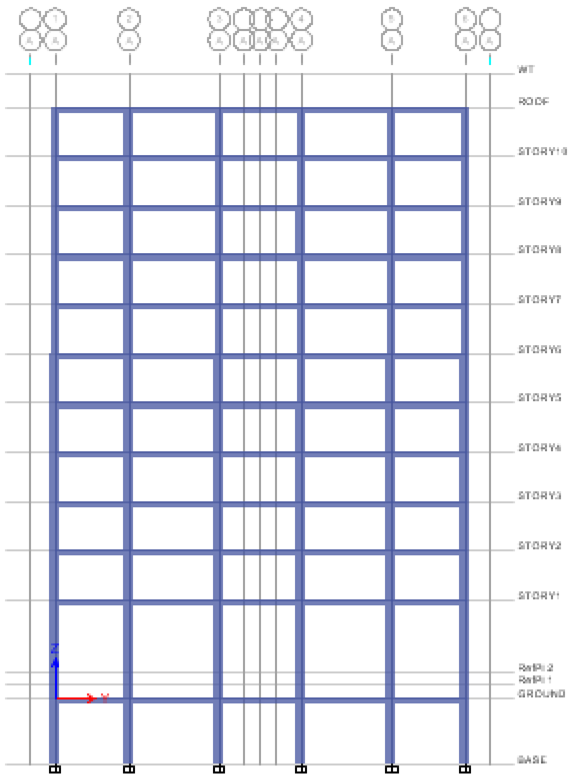


Figure 4-5- Model 5 (along Y-axis)

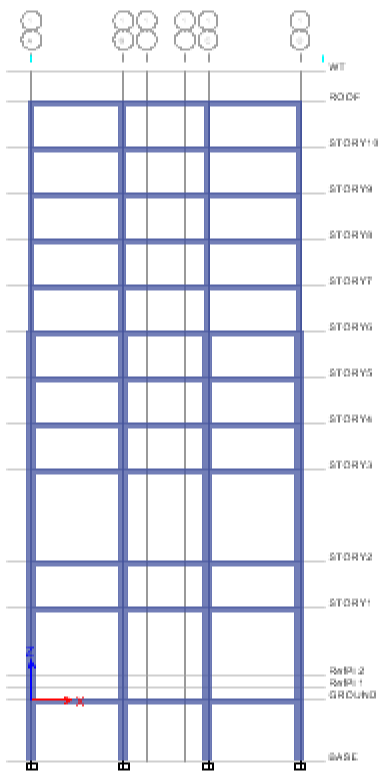


Figure 4-6- Model 6 (along X-axis)

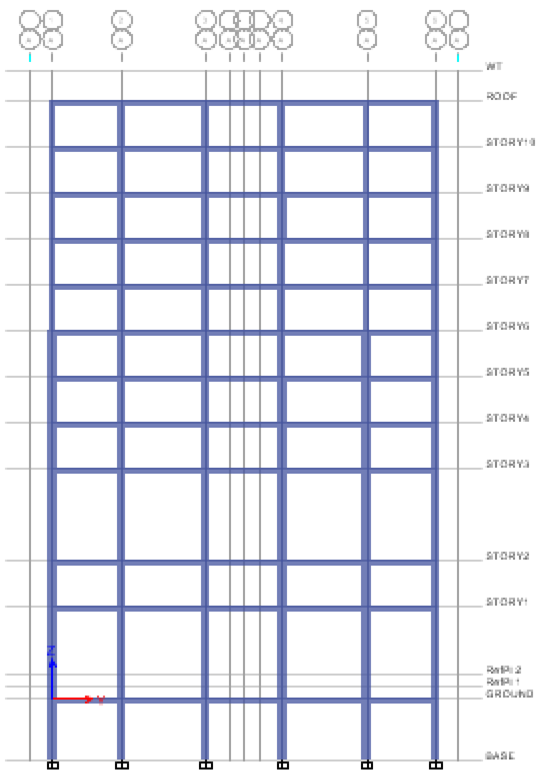


Figure 4-7- Model 6 (along y-axis)

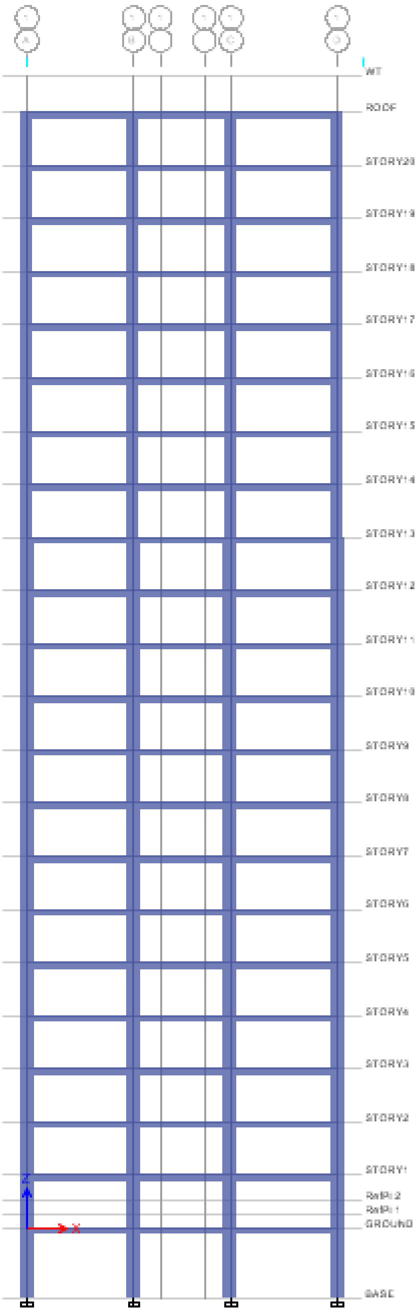


Figure 4-8- Model 1 (along x-axis)

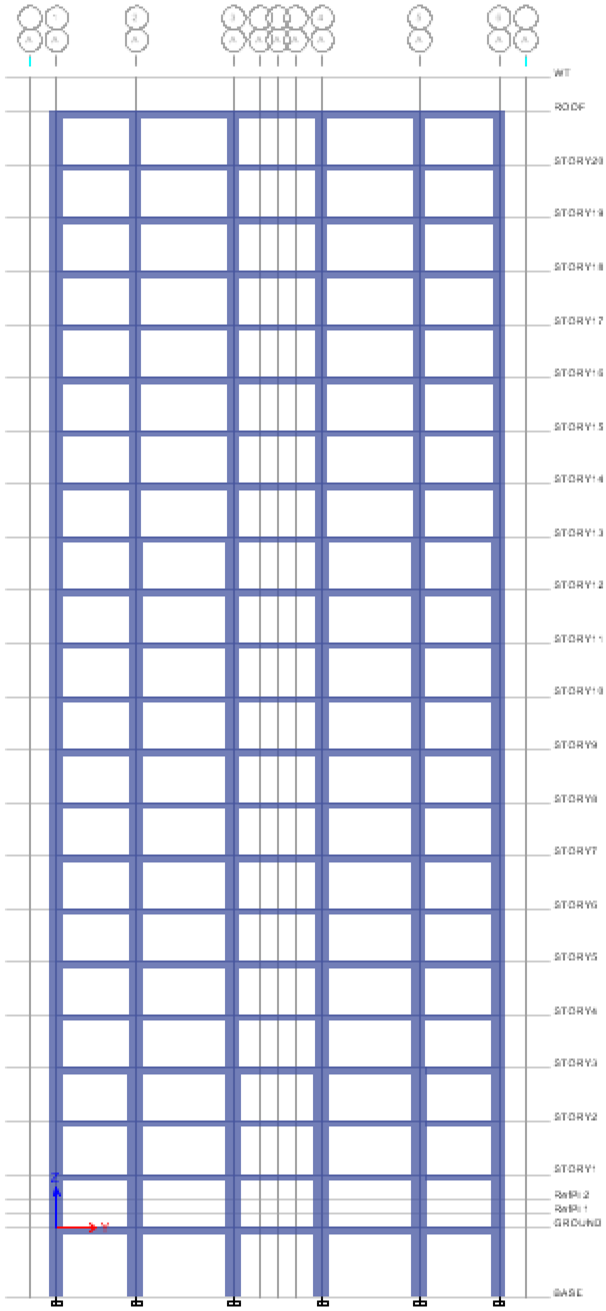


Figure 4-9- Model 1 (along y-axis)

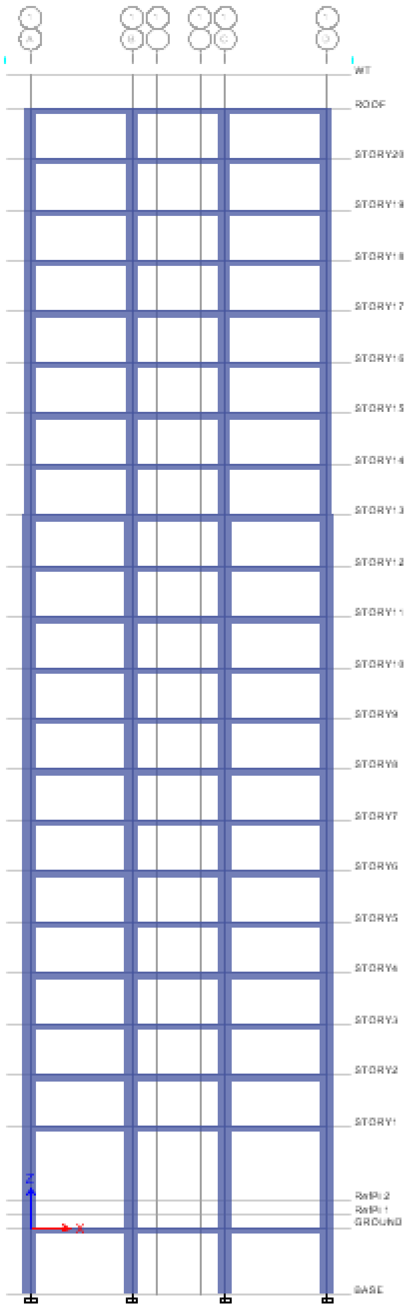


Figure 4-10- Model2 (along x-axis)

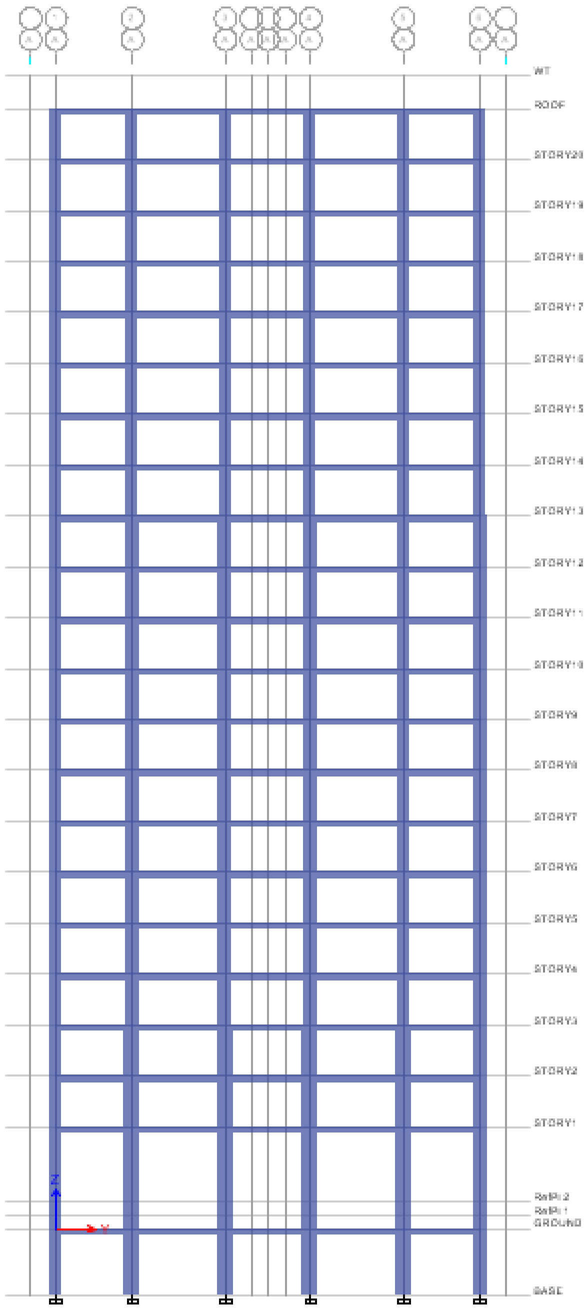


Figure 4-11- Model2 (along y-axis)

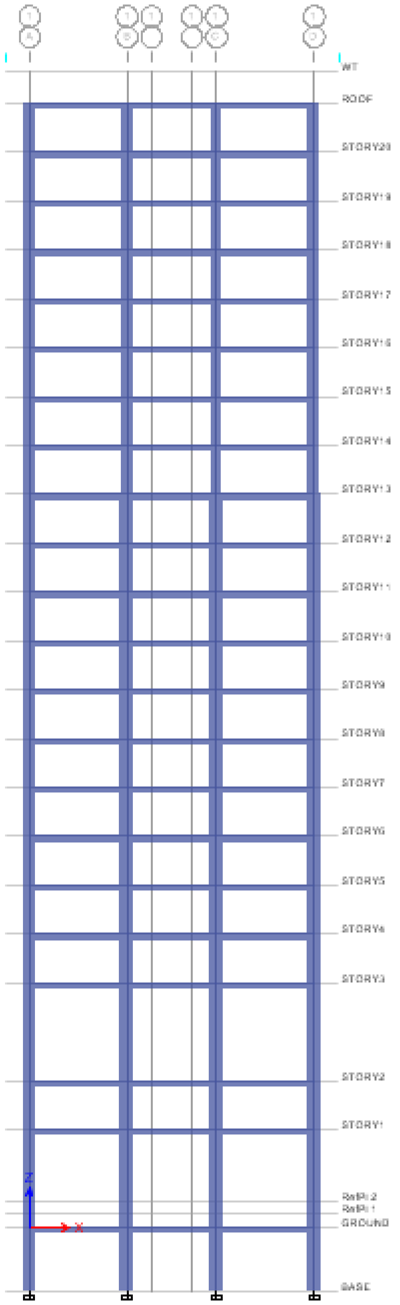


Figure 4-12- Model 3 (along x-axis)

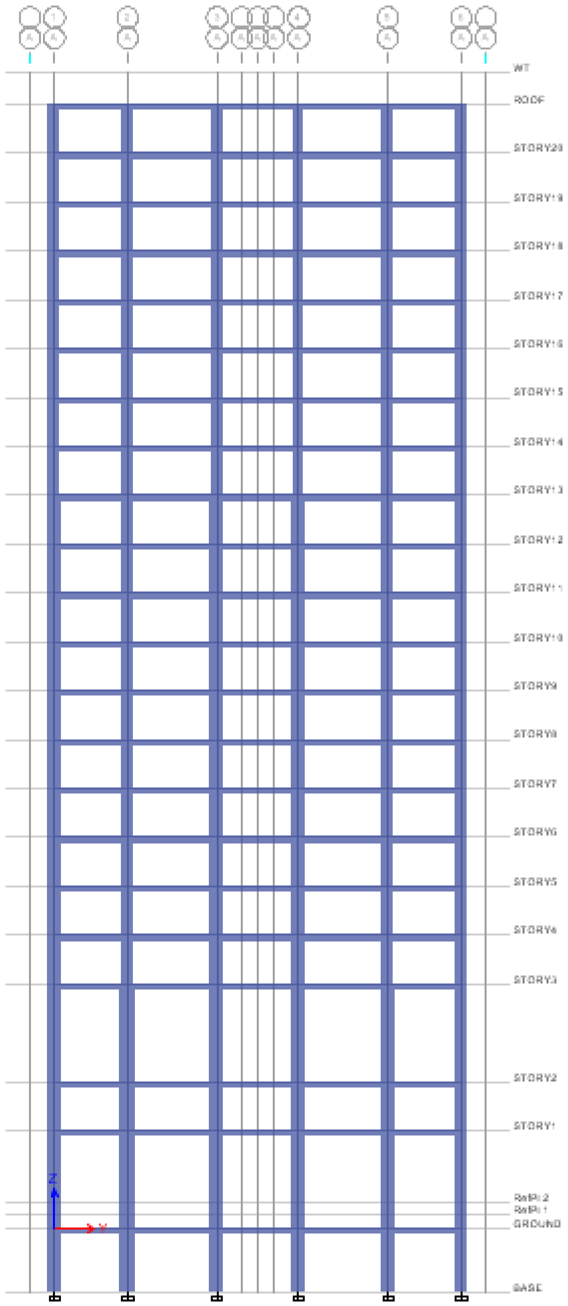


Figure 4-13- Model 3 (along y-axis)

4.2 ANALYSIS APPROACH

The structure is modeled, analyzed, and designed in computer software "ETABS 2016.2.1" The software has very good analysis and design capability which are verified in the verification problems included in the package. It is a Finite Element Method (FEM) based software and thus requires modeling of structures by finite elements. Beams and columns are modeled with line or frame elements, shear walls are modeled with wall elements, and slabs and roof floors are modeled with area elements. Analysis and design of slabs entirely followed coefficient method where the approach depends upon whether it is a one – way or two - way slab, support conditions and the loadings. Accordingly, slabs are analyzed on spread sheets/excel sheets based on their support conditions and corresponding parameters as per EBCS. The calculated partition loads, floor finishes, and live loads are then assigned on the modeled area elements on ETABS 2016.2.1 so as to consider for their respective applied gravity loads.

- Partition load on floors (average value):

  $((14kN/m3 * 3.1 * 0.15 * 5) + (0.04 * 3.1 * 5 * 23)) / (4*5) = 2.35kN/m^2$

- Finishing load on floors:

  Floor finish = 0.03 * 23 = 0.69kN/m2

  Ceiling plaster = 0.02 * 23 = 0.46kN/m2] Total = $0.69 + 0.46 + 0.69 = 1.84kN/m^2$

  Cement screed = 0.03 * 23 = 0.69kN/m2

- Wall loads on beam:

  For 15cm thick HCB = 14kN/m3* 0.15 * 2.9 + 0.04 * 2.9 * 23 = 8.75kN/m

  For 20cm thick HCB = 14kN/m3* 0.2 * 2.9 + 0.04 * 2.9 * 23 = 10.8kN/m

- Live load

  Apartment.......... = 2 KN/m^2

  Office... .... = 3 KN/m^2

  Shop................ = 5 KN/m^2

4.3 DESIGN PHILOSOPHIES

Structural design methods are selected based on the local practices. The current design philosophy is based on the Capacity Design Method which is adopted from Limit State Design Method. These are the methods used for the design of structural members and are guided by the relevant standard code of practice. The design philosophies used in the design of this particular case study projects entirely followed the rules as per in the new code of Ethiopian Building Code Standards adopted from European Norm (ES EN 2015) listed as in below.

1. ES EN 1990:2015 (Basis of Structural Design)
2. ES EN 1991:2015 (Basis of Design and Actions on Structures)
3. ES EN 1992:2015 (Design of Concrete Structures)
4. ES EN 1997:2015 (Design of Foundations)
5. ES EN 1998:2015 (Design of Structures for Earthquake Resistance)

Moreover, structures in seismic regions shall be designed and constructed in such a way that the following requirements are met, each with an adequate degree of reliability.

4.3.1 NO-COLLAPSE REQUIRMENT

The structure shall be designed and constructed to withstand the designed seismic actions without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events. The design seismic action is expressed in terms of:

a) The reference seismic action associated with reference probability of 10 % exceedance, PNCR in 50 years or a reference return period, TNCR= 475 years

b) The importance factor γ_1 to take into account reliability differentiation.

The reliability differentiation is implemented by classifying structures into different importance classes. An importance factor $\gamma 1$ is assigned to each importance class. Accordingly, an importance factor $\gamma 1 = 1$ is assigned for the above reference return period (50 years).

4.3.2 DAMAGE LIMITATION REQUIREMENT

The structure shall be designed and constructed to withstand a seismic action having a larger probability of occurrence that the designed seismic action, without the occurrence of damage and the associated limitations of use, the costs of would be disproportionately high in comparison with the costs of the structure itself. The seismic action to be taken into account for the “damage limitation requirement” has a probability of exceedance, PDLR, in 10 years and a return period, TDLR. The recommended values are PDLR = 10% and TDLR= 95 years.

4.4 SEISMIC ACTIONS

For the purpose of ES EN 1998:2015, national territories shall be subdivided into seismic zones, depending on the local hazard. By definition, the hazard within each zone is assumed to be constant. The reference peak ground acceleration, chosen for each seismic zone, corresponds to the reference return period TNCR of the seismic action for the no-collapse requirement (or equivalently the reference probability of exceedance in 50 years, PNCR). An importance factor $\gamma 1$ equal to 1.0 is assigned to this reference period.

4.4.1 MODAL RESPONSE SPECTRUM METHOD

It is the reference method for determining the seismic effects using the linear-elastic model of the structure and the corresponding site specific design spectrum. Modal response spectrum analysis is applicable to all types of buildings. In response spectrum method the peak response of structure during an earthquake is obtain directly from the earthquake response spectrum. This procedure gives an approximate peak response, but this is quite accurate for structural design applications. In this approach, the multiple modes of response of building to an earthquake are taken in account. For each mode, a response is read from design spectrum, based on modal frequency and modal mass. Generally, a site specific response spectra is required based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. In absence of a site specific response spectrum, the normalized response spectra for damping ratio 5% shall be used in the dynamic analysis. In this method the load vectors are calculated corresponding to predefined number of modes. These load vectors are applied at the design center of mass to calculate the respective modal responses. The loads acting on the structure are contributed from slabs, beams, columns, walls, ceilings and finishes; and these loads are directly considered by the ETABS 2016.2.1 software through the definition of mass sources for seismic load.

Accordingly, the design ground acceleration $\alpha g = \alpha g_R * \gamma_1$ or $\alpha o I$

Where:

αg= the design ground/bed rock acceleration

$\alpha g_R(\alpha o)$= the ratio of design bed rock acceleration to acceleration due to gravity

$\gamma_1(I)$=importance factor assigned to the reference return period

Thus for Addis Ababa (seismic zone 3), αo=0.01

γ_1= 1.0 (for ordinary building of reference return period associated with no-collapse requirement)

αg= 0.1*1.0 = 0.10

For the horizontal component of the seismic action the design spectrum, Sd(T), shall be defined by the following expressions. To avoid explicit inelastic structural analysis design, the capacity of the structure to dissipate energy, through mainly ductile behavior of its elements/and other mechanisms, is taken into account by performing an elastic analysis based on a response spectrum reduced with respect to the elastic one called “Design Spectrum”. This reduction is accomplished by introducing the behavior factor, q.

$0 \le T \le T_B : S_d(T) = a_g . S . \left[\frac{2}{3} + \frac{T}{T_B} . \left(\frac{2.5}{q} - \frac{2}{3}\right)\right]$……………Equation 4-1

$T_B \le T \le T_c : S_d(T) = a_g . S . \frac{2.5}{q}$………………Equation 4-2

$$T_C \leq T \leq T_D : S_d(T) = \begin{Bmatrix} = a_g . S . \frac{2.5}{q} . \left[\frac{T_C}{T}\right] \\ \geq \beta . a_g \end{Bmatrix}$$ ………………..Equation 4-3

$$T_D \leq T : S_d(T) = \begin{Bmatrix} = a_g . S . \frac{2.5}{q} . \left[\frac{T_C T_D}{T^2}\right] \\ \geq \beta . a_g \end{Bmatrix}$$ …………………Equation 4-4

Where:

Sd(T) = is the design spectrum

q = is the behavior

β=is the lower bound factor for the horizontal design spectrum (0.2)

T = is the vibration period of a linear single-degree of freedom system

αg= is the design ground acceleration on type A ground (ag= $\gamma \alpha g_R$)

TB= is the lower limit of the period of the constant spectral acceleration branch

TC= is the upper limit of the period of the constant spectral acceleration branch

TD= is the value defining the beginning of the constant displacement response range of the spectrum

S = is the soil factor

η=is the damping correction factor with a reference value of η=1 for 5% viscous damping.

If the earthquake that contributes most to the seismic hazard defined for the site for the probability hazard assessment have a surface-wave magnitude, Ms, less than 5.5, it is recommended that the Type 2 Spectrum is adopted.

Table 4-1- Values of the Parameters describing Type 2 Elastic Response Spectra

Ground Type	S	TB	TC	TD
A	1.0	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

4.5 STRUCTURAL TYPE AND BEHAVIORAL FACTOR

Structural type is the property of the building, but in general (especially in the case when the structure consists of walls and frames), it could not be defined without appropriate analyses. So, the mathematical (structural) model is needed for the determination of the structural type of the building. There are various classifications of structural types for concrete buildings based on their behavior under horizontal seismic actions. The buildings under the considerations are frame systems in which both the vertical and lateral loads are mainly resisted by special frames whose shear resistance at the building base is greater than 65% of the total shear resistance of the whole structural system.

Behavioral factor to account for energy dissipation capacity is calculated as:

$q = q_o k_W \geq 1.5$

Where

q_o= is the basic value of the behavior factor, dependent on type of structural system and its regularity on elevation

k_W= is the factor reflecting the prevailing failure mode in structural system with walls

q_o= 3.0αu/α1(for DCM, frame system, dual system, coupled wall system)

αu/α1= 1.3 (for multistory, multi-bay frames or frames-equivalent dual structures)

kw= 1 (for frames or frames-equivalent dual structures)

Accordingly; q = qokw= 3.0*1.3*1 = 3.3

Table 4-2- Design Response Spectrum Parameters

Parameters	Values	Remark
Site Class	C	Deep deposit of dense or medium-dense sand, gravel or stiff clay
Seismic Group/Seismic Zone	III	Addis Ababa
Design Spectrum Type	Type 1	Recommended Type
Response Modification Facto	3.3	Frame System
Bed Rock Acceleration Ratio	0.1	Addis Ababa (Zone-3)

4.6 DESIGN APPROACH

Structural design is an art and science of understanding the behavior of structural members subjected to loads and designing them with economy and elegance to give a safe, serviceable and durable structure. Structural design basis starts with the type of structure to be designed. Once the form of the structure is selected and analyzed on the pertinent structural software, the structural design process starts. The new ES EN 2015 code is based on the Capacity Design Method and this project is designed based on the recommended capacity design method. For any structural design to commence, we require certain data. These data include information about type of structure, site conditions, loading conditions, type of environmental exposure, earthquake zone and wind zone Structural Parameters.

Slab Design

The vertical loads on the slabs were calculated and applied to the different slab panels. Designing of slabs depends upon whether it is a one – way, two - way slab or cantilever slab, the end conditions and the loadings. Accordingly, slabs are analyzed on spread sheets/excel sheets based on their end conditions and corresponding parameters as per ES EN 2015.

Beam Design

The force envelope i.e. maximum positive and negative moments, and maximum shear (envelope) is automatically selected by the ETABS software. The beam reinforcements are designed to resist these loads.

Column Design

Columns are designed for the first order effects as well as for the second order effects. First order effects are those caused by direct application of the loads. Second order effects are those that occur from either of two sources.

- P- Delta Effects
- Slenderness Effects

The 3D ETABS analysis was setup so that it will take into account any p- delta effects resulting from lateral loads. Therefore, the analysis results from ETABS give both the first and second order effects of the loads.

4.7 MATERIAL PROPERTIES

In computation of dead loads and associated loads on the structure, the following unit weight of materials were used

- Concrete……………………....25KN/m^3
- Cement Screed………………...23KN/m^3
- Ceramic floor finish……………23KN/m^3
- Plastering ………………………23KN/m^3
- HCB……………………………..14 KN /m^3

Partial Safety Factors

Dead load = 1.35

Live load = 1.50

Concrete

Grade C-25 (Columns, Beams, slabs & foundations)

Fck = 20 Mpa

Fctk =1.5 Mpa

Partial Safety Factor = 1.5 for concrete

Fcd = 0.85[20/1.5] = 11.33 Mpa

Fctd = 1.5/1.5 = 1.00 Mpa

Ecm = 29Gpa

Reinforcing Steel:

fyk = 400 Mpa

Partial Safety Factor =1.15

fyd = 400/1.15 = 347.82 Mpa

Es = 200Gpa

4.8 LOAD COMBINATIONS

Loading for the different occupancies are clearly identified in the loading section of the structural calculations.

1. COMB-1 = 1.35 DL + 1.5 LL
2. COMB-2 = DL + ψ2LL + EQx + 0.3EQy
3. COMB-3 = DL + ψ2LL + EQx - 0.3EQy
4. COMB-4 = DL + ψ2LL - EQx + 0.3EQy
5. COMB-5 = DL + ψ2LL - EQx - 0.3EQy
6. COMB-6 = DL + ψ2LL + EQy + 0.3EQx
7. COMB-7 = DL + ψ2LL + EQy - 0.3EQx
8. COMB-8 = DL + ψ2LL - EQy + 0.3EQx
9. COMB-9 = DL + ψ2LL - EQy - 0.3EQx
10. COMB-10 = DL + LL + WL

4.9 DESIGN OUTPUTS AND DETAIL OF STRUCTURAL MEMBERS

All building model cases (G+10 and G+20) were analyzed on ETABS 2016.2.1 and cross section of structural members were adjusted in such a way that safety requirements, optimization and most practical way of provisions have been met.

CHAPTER FIVE

5 NON-LINEAR MODELLING AND ANALYSIS

5.1 GENERAL

Non-linear Modelling a structure and representing it in a computer program for suitable analysis are important steps in performance evaluation. Several well established computer programs, which can be used in the modelling and analysis of a structure to evaluate its seismic performance, are available. And the best alternative of all are used for this study. The modelling techniques are discussed in this chapter. 30 models are considered for each case of building, which is modelled in Seismostruct software for nonlinear analysis. Concrete is modelled as per, Mander (1988). Steel reinforcing bars are modelled using uniaxial Giuffre Menegotto Pinto (1973) steel material model with isotropic strain hardening.

The non-linear analysis of this study is based upon pushover and nonlinear dynamic time history analysis of the selected buildings. Accordingly, accurate modelling of the nonlinear properties of various structural elements is very important for nonlinear analysis. In this study, the non-linear structural and material properties are considered for the non-linear analysis.

5.2 ABOUT SEISMOSTRUCT SOFTWARE

SeismoStruct is a Finite Element package capable of predicting the large displacement behavior of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity, and the spread of inelasticity along the member length and across the section depth is explicitly modelled in SeismoStruct allowing for accurate estimation of damage. Concrete, steel, and other material non-linear models are available, together with a large library of 3D elements that may be used with a wide variety of pre-defined steel, concrete and composite section configurations. This software is also capable of doing eight different types of analysis: dynamic and static time-history, conventional and adaptive pushover, incremental dynamic analysis, eigenvalue, non-variable static loading, and response spectrum analysis. Performance criteria can also be set using SeismoStruct, allowing to identify the instants at which different performance limit states (e.g. non-structural damage, structural damage, collapse) are reached. The sequence of cracking, yielding, failure of members throughout the structure can also be. Accordingly, this software is found to be the best preference for this research; so many researchers have also used this software successfully.

5.3 NONLINEAR MODELING

Seismostruct has been used throughout the study for developing nonlinear analytical models. In Seismostruct, fibre approach is made use of to represent the cross-sectional behaviour, where each fibre is associated with a sectional stress-strain state of beam-column elements is then obtained through the integration of uniaxial stress-strain relationship; the nonlinear stress-strain response of individual fibers with which the section has been discretized (Seismostruct, 2016). Both force-based (infrmFB) and displacement-based (infrmDB) formulations are available in the program to simulate inelastic behavior of the beam-column elements. Here we have chosen displacement formulation for all the elements. Each element is assigned five integration points along its length where the nonlinear axial-flexural behavior of the cross-section is monitored. The fibers in each cross-section are assigned material properties to represent unconfined concrete, confined concrete and the steel reinforcement. Here Mander's nonlinear model has been chosen to represent both confined and unconfined concrete whereas a bilinear model is assigned for steel reinforcement. The main advantages of the fibre include the ability to capture axial-flexural interaction and the effects of concrete tensile strength and tension stiffening along with user-friendly inputs. Skyline solver method and Hilber-Hughes-Taylor integration scheme has been used for dynamic time history analysis.

5.4 MATERIAL MODELS

An elemental cross-section in an RC member is composed of three types of materials: unconfined concrete, confined concrete and reinforcing steel. All reinforced concrete components are detailed with transverse steel which provide both shear resistance and confining action. The confining effects of transverse steel are considered implicitly by modifying the stress-strain response of the core concrete. Numerous researchers have developed stress-strain models of confined concrete based on observed experimental behavior. The concrete cover will typically spall at relatively small strain levels; therefore, the modelling of unconfined concrete is generally not critical for damage limit states in the inelastic range. The response of RC components and consequently the system is a function of the behavior of the confined core concrete and the longitudinal steel.

5.4.1 CONCRETE MODELLING

Concrete outside the transverse reinforcements in the RC section has no confinement, whereas concrete inside the transverse reinforcements is confined. In order to consider the effect of confinement, cover concrete (outside the transverse reinforcement) and core concrete (inside the transverse reinforcement) materials are considered separately. Behavior of confined concrete is different from that of unconfined concrete. Concrete can be considered confined when it is subjected to triaxial compressions; the triaxial compression increases the concrete's capacity to sustain larger compressive strengths and deformations. When a concrete element is laterally reinforced (e.g., by shear reinforcements) and subjected to axial compression, lateral expansion of the element in the plane perpendicular to the axial compression activates the lateral steel, which confines the element by exerting lateral pressure. Confined concrete generally fails in a ductile manner, whereas unconfined concrete fails in a brittle manner. As tensile strains develop in unconfined concrete subjected to compression, concrete softens and strength decreases. In this study the uniaxial nonlinear constant confinement model, initially programmed by Madas (1993), that follows the constitutive relationship proposed by Mander et al. (1988) and the cyclic rules proposed by Martinez-Rueda and Elnashai (1997) in used for the concrete modelling for the seismic performance evaluation. The confinement effects provided by the lateral transverse reinforcement are incorporated through the rules proposed by Mander et al. (1988) whereby constant confining pressure is assumed throughout the entire stress- strain range.

5.4.2 REINFORCEMENT MODELLING

Steel reinforcing bars are modelled using Menegotto and Pinto, (1973) model with Isotropic Strain Hardening as shown in Fig. 4.2 with a schematic cyclic behavior. This model consists of explicit algebraic stress-strain relationship, in finite terms, for branches between two subsequent reversal points (loading branches). The parameters involved are updated after each strain reversal. The Menegotto-Pinto $\sigma = f(\varepsilon)$ expression is:

$$\sigma^* = b\varepsilon^* + \frac{(1-b)\varepsilon^*}{(1+\varepsilon^{*R})^{\frac{1}{R}}} \text{……………..Equation 5-1}$$

$$\varepsilon^* = \frac{\varepsilon-\varepsilon_r}{\varepsilon_0-\varepsilon_r} \quad and \quad \sigma^* = \frac{\sigma-\sigma_r}{\sigma_0-\sigma_r} \text{…………………..Equation 5-2}$$

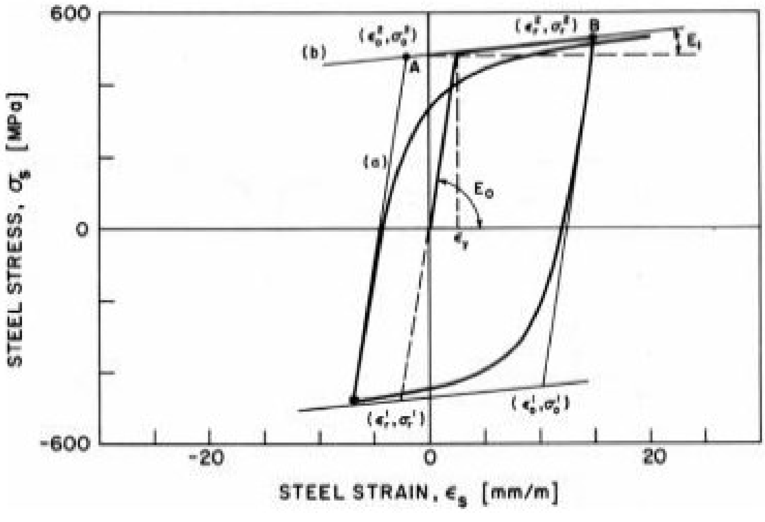


Figure 5-1- Manegotto-Pinto Steel Model (Menegotto-Pinto, 1973

5.5 BUILDING DESCRIPTION

The plan layout of a typical eleven-story (G+10) and twenty-one story (G+20) RC moment resisting frame as shown in the Fig.1 is considered for the analysis. The building has plan dimensions of 25m x 17.5m. The frame is assumed to be of moment-resisting type (MRF). The building is intended for mixed use. The other relevant details are as given in the Table-5.1.

Table 5-1- Preliminary Data

Story height	3.0 m
Building size	25mx17.5m
Live load intensity on apartment floor	2.0 KN/m^2
Live load intensity on shop floor	5.0 KN/m^2
Weight of floor finishes	2.42 KN/m^2
Grade of concrete	C25/30
Grade of steel	S-400
Modulus of elasticity of concrete	33Gpa
Modulus of elasticity of steel	200Gpa
Seismic zone	III
Peak ground acceleration a_g	0.1_g
Soil type	C
Damping ratio	5%
Importance factor	1
Building type	Moment resisting RC frame(MRF)
Beam size (mm) for G+10 story	400 x 300
Beam size (mm) for G+20 story	500 x300
Column size (mm) for G+10 story	500x500,600x600,700x700 and 800x800
Column size (mm) for G+20 story	700x700,800x800 and 900x900

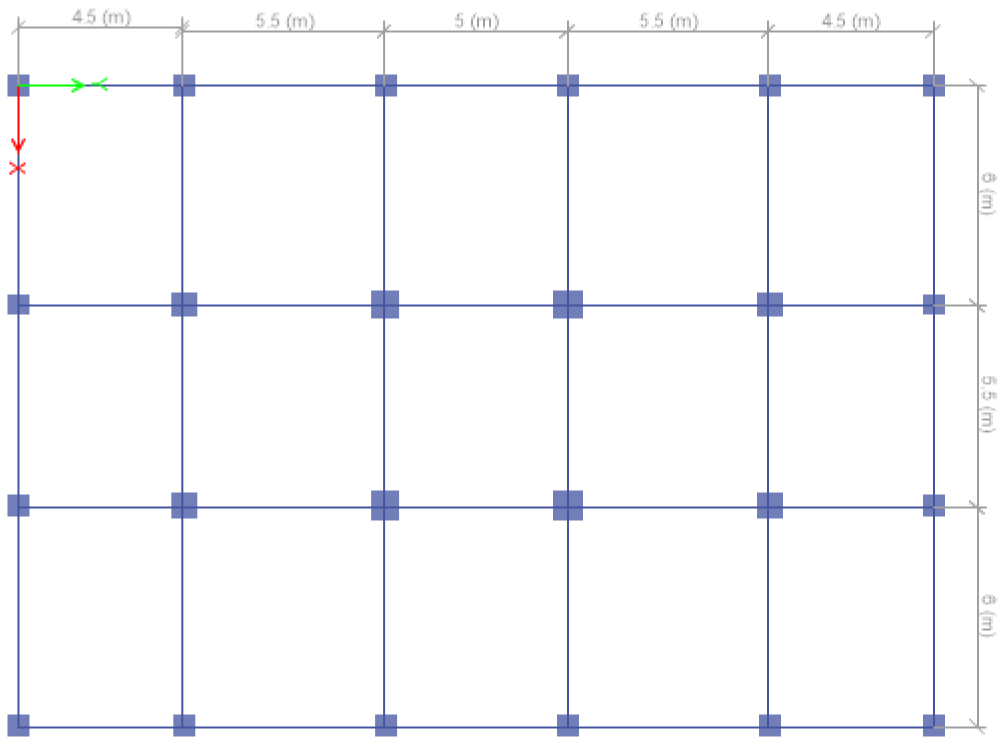


Figure 5-2- Typical Plan considered for Analysis

5.6 MODEL DESCRIPTION

Six building model cases for each designed models are prepared and modelled on SeismoStruct 2016.

Model 1 ………. G+20 similar story height building (base case)

Model 4………... G+10 similar story height building (base case)

Model 2…………. G+20 (ground floor height is 200% of the story height of other floors)

Model 5…………. G+10 (ground floor height is 200% of the story height of other floors)

Model 3…………. G+20 (ground floor height and second floor height is 200% of the story height of other floors)

Model 6…………. G+10 (ground floor height and second floor height is 200% of the story height of other floors)

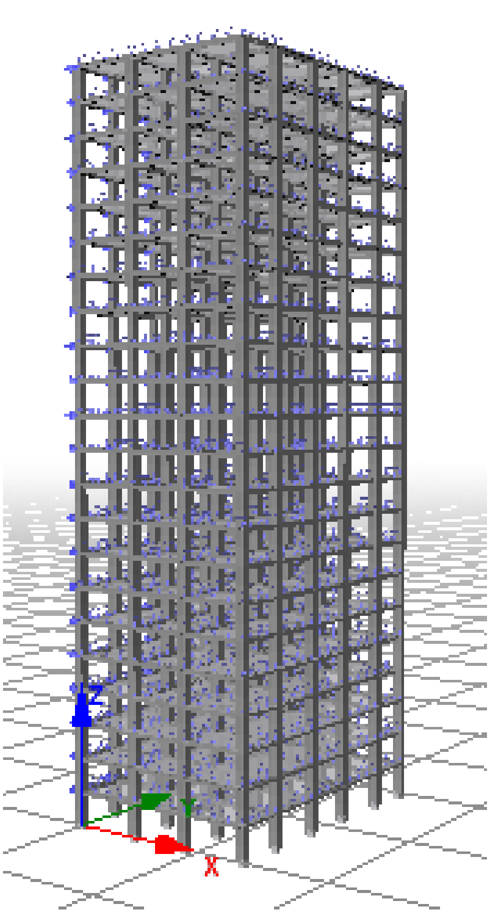


Figure 5-3- Model 1

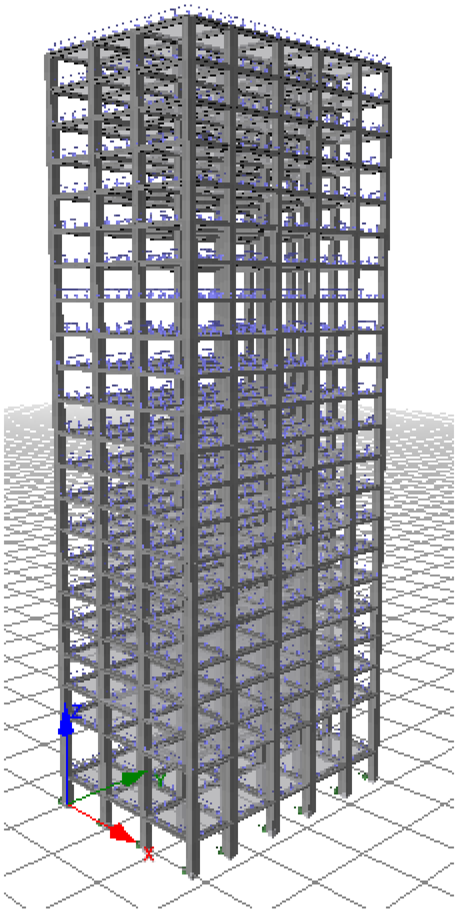


Figure 5-4- Model 2

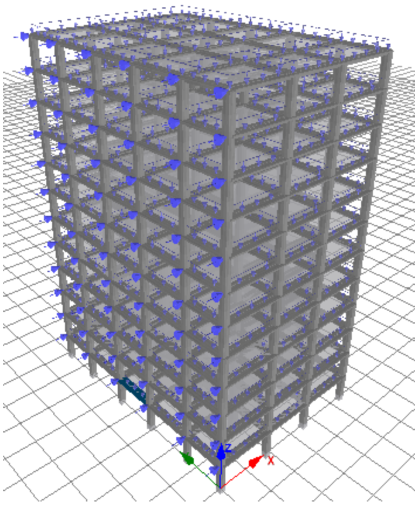


Figure 5-5- Model 3

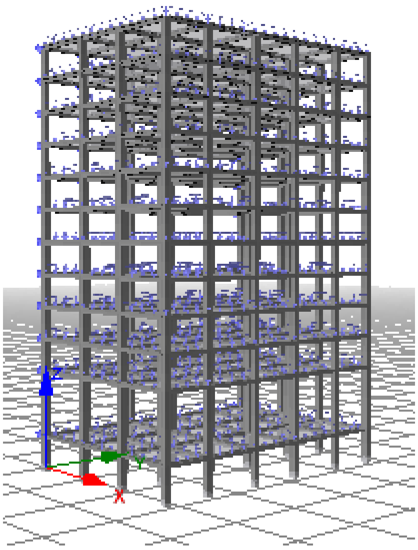


Figure 5-6- Model 4

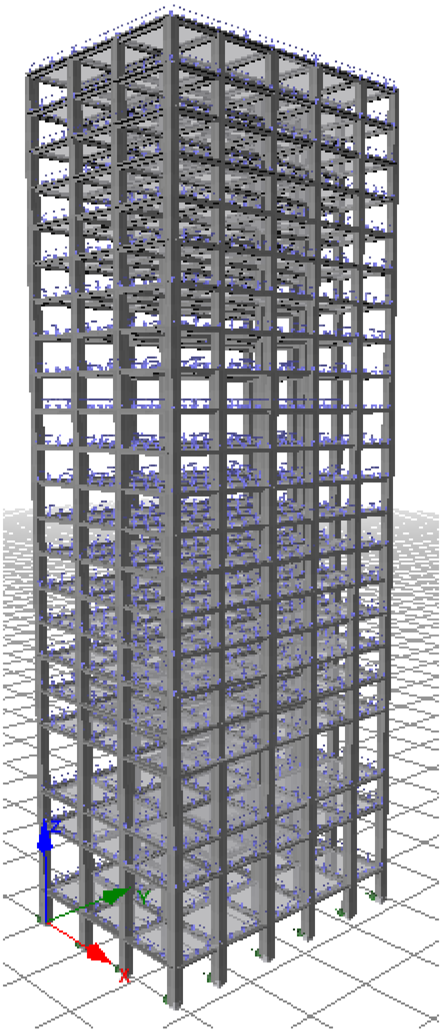


Figure 5-7- Model 5

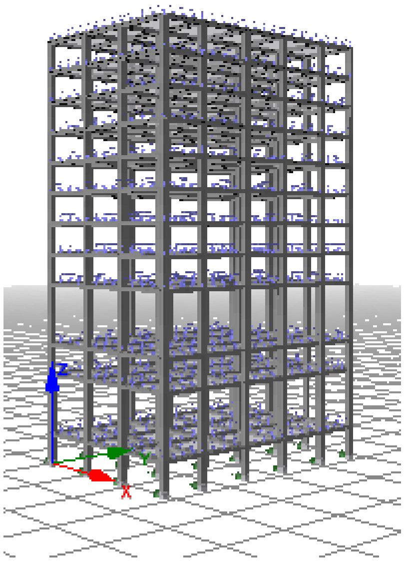


Figure 5-8- Model 6

CHAPTER SIX

6 RESULTS AND DESCUSSION

6.1 FUNDAMENTAL NATURAL VIBRATION PERIOD

Every building has a number of natural frequencies; at which it offers minimum resistance to shaking induced by the earthquakes. Each of this natural frequencies and the associated deformations shape of a building constitutes a natural mode of oscillation. The mode of oscillation with the smallest natural frequencies (and largest natural period) is called as fundamental mode; and the associated natural period is called as the fundamental natural period and the associated natural frequencies is known as fundamental natural frequency. The natural vibration period of the building is thus the time taken by it to undergo one complete cycle of the oscillation. It is an inherent property of the building controlled by its mass and stiffness. The building offers least resistance when shaken at its natural frequency (or natural period). Hence it undergoes larger oscillation when shaken at its natural frequencies than at other frequencies.

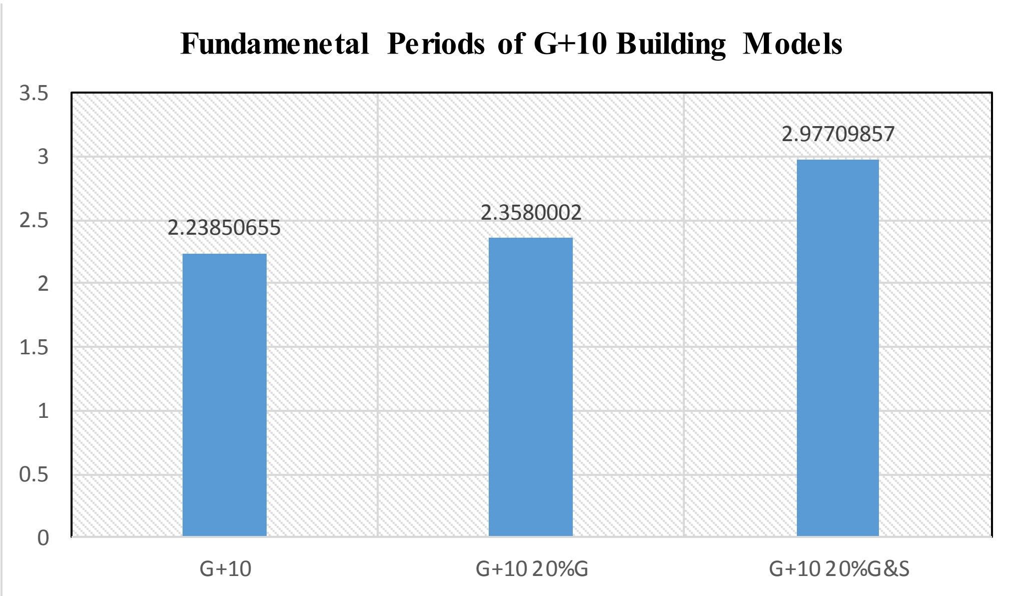
When a structure is excited by seismic forces, it starts to vibrate. The lowest natural frequency (f) of vibration of a structure corresponds to the longest time period (T) of vibration, as frequency and time period are inversely proportional ($T = 1/f$). This is also referred to as the first mode vibration or fundamental period of vibration. Structure will have multiple natural modes of vibration for which frequency will be higher and time period will be shorter than the fundamental period. The reliable and sufficient estimation of the natural period of vibration could play an essential role in the understanding of the global demands on the structure under an earthquake. Its evaluation is an essential step in estimating the seismic response both in seismic design and assessment. This important property of the building's seismic behavior is mainly dependent on mass, strength and stiffness, and consequently on all the factors which affect them (dimensions in height and plan, morphology, irregularities, section properties, stiffness, cracking, etc.). In the current research the fundamental period of vibration is estimated for the different model cases of building and comparative study is made in the section follows.

From the monitored pushover analysis in the governing direction (+X) of the simulated three dimensional G+10 and G+20 buildings, the following response was found in terms of the fundamental periods.

6.1.1 G+10 BUILDING MODEL CASES

Table 6-1- Fundamental Periods of G+10 Building Models

Building Model Types	Case Designation	Fundamental Period (Sec.)	Deviation From Model-4 (%)
G+10	Model-4	2.238	0
G+10 20%G	Model-5	2.358	5.36
G+10 20%G&S	Model-6	2.977	33.02



Building Model Types

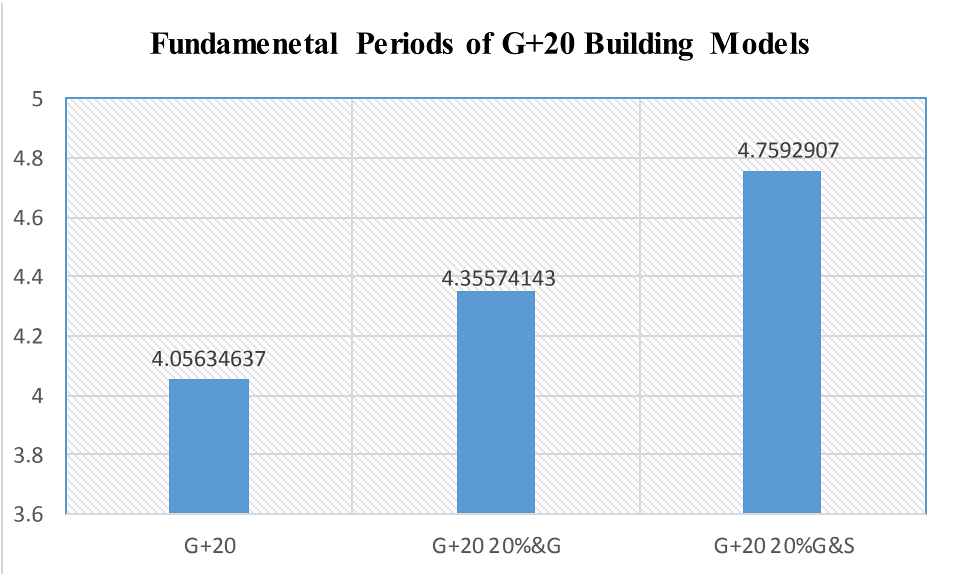
Table 6-2- Fundamental Periods of G+10 Building Models

As shown in table 6.1 and figure 6.1 the stiffness regular building model (G+10) was found to have a lowest fundamental natural vibration period (i.e. 2.238 Sec.) compared to other stiffness irregular building model cases with varying story height. Increase story height of ground floor by 200% had increase the fundamental period to 2.358 Sec (5.36% increment). Highest fundamental natural vibration period corresponds to lowest natural frequency of vibration of a structure and in turn corresponds to relatively lowest structural stiffness. It can be seen that increase the story height of selected floors of the building increase the fundamental periods of the structure which reduce the lateral stiffness of the building. Accordingly, introducing 200% increase story height on the ground floor and 200% increase both on the ground and second floors story height, increase the fundamental periods to 2.358 (5.36% increment) and 2.977 (33.02% increment) Sec respectively. Due to the increase of the ground floor story height the fundamental time period of the structures increase 5.36%, and increase both the ground and second floors story height increase the fundamental time period more than 33%.

6.1.2 G+20 BUILDINGS MODEL CASES

Table 6-3- Fundamental Periods of G+20 Building Models

Building Model Types	Case Designation	Fundamental Period (Sec.)	Deviation From Model-1 (%)
G+20	Model-1	4.056	0
G+20 200%G	Model-2	4.355	7.37
G+20 200%G&S	Model-3	4.759	17.33



Building Model Types

Figure 6-1- Fundamental Periods of G+20 Building Models

As shown in table 6.2 and figure 6.2 the stiffness regular building model (G+20) was found to have a lowest fundamental natural vibration period (i.e. 4.056 Sec.) compared to other stiffness irregular building model cases with varying story height. Increase story height of ground floor by 200% had increase the fundamental period to 4.355 Sec (7.37% increment). Highest fundamental natural vibration period corresponds to lowest natural frequency of vibration of a structure and in turn corresponds to relatively lowest structural stiffness. It can be seen that increase the story height of selected floors of the building increase the fundamental periods of the structure which reduce the lateral stiffness of the building. Accordingly, introducing 200% increase story height on the ground floor and 200% increase both on the ground and second floors story height, increase the fundamental periods to 4.355 (7.37% increment) and 4.759 (17.33% increment) Sec respectively. Due to increase off the ground floor story height the fundamental time period of the structures increase 5.37%, and increase both the ground and second floors story height increase the fundamental period more than 17%.

6.1.3 CONCLUSION ON FUNDAMENTAL PERIODS

Table 6-4- summary of Fundamental Periods of all Building Models

Building Model Types	Case Designation	Fundamental Period (Sec.)	Deviation From Model-1 (%)
G+20	Model-1	4.056	0
G+20 200%G	Model-2	4.355	5.37
G+20 200%G&S	Model-3	4.759	17.33
Building Model Types	Case Designation	Fundamental Period (Sec.)	Deviation From Model-4 (%)
G+10	Model-4	2.238	0
G+10 200%G	Model-5	2.358	5.36
G+10 200%G&S	Model-6	2.977	33.02

As it has been seen above the percentage deviations of fundamental periods of two stories (G+10 200%G&S) stiffness irregular building from the stiff one (G+10) for G+10 building models are greater than the corresponding values G+20 building model, this shows that the effect of stiffness irregularity on fundamental period have more significant contributions as the soft-story number increase. Also it has more significant contributions as story number increase or as the building gets high-rise.

6.2 CAPACITY CURVE PARAMETERS

Amongst the natural hazards, earthquakes have the potential for causing the greatest damages. Since earthquake forces are random in nature & unpredictable, the engineering tools need to be sharpened for analyzing structures under the action of these forces. Earthquake loads are to be carefully modelled so as to assess the real behavior of structure with a clear understanding that damage is expected but it should be regulated. In this context pushover analysis which is an iterative procedure is looked upon as an alternative for the conventional analysis procedures. It is generally assumed that the behavior of the structure is controlled by its fundamental mode and the predefined pattern is expressed either in terms of story shear or in terms of fundamental mode shape. It provides a load versus deflection curve of the structure starting from the state of rest to the ultimate failure of the structure. The load is representative of the equivalent static load of the fundamental mode of the structure. It is generally taken as the total base shear of the structure and the deflection is selected as the top-story deflection.

The seismic performance of a building can be evaluated in terms of pushover curve, performance point, displacement ductility, plastic hinge formation etc. The base shear vs. roof displacement curve is obtained from the pushover analysis from which the maximum base shear capacity of structure can be obtained. As it has been noted the two parameters that are involved in making up the push over curve are seismic base shear and roof displacement. Seismic base shear is an estimate of the maximum expected lateral forces that will occur due to seismic ground motion at the base of a structure. It depends upon the soil conditions at the site, seismic weight of the structure, stiffness and ductility, and overall response of the structure for the seismic action. And roof displacement is the measured top floor displacement of the structures subjected to the incremental load (push load). Carrying out the pushover analysis on typical structure gives a curve having seismic base shear and monitored roof displacement at various performance levels. Pushover analyses of proposed building models subjected to increasing lateral forces were carried out until the pre-set performance level (target displacement) is reached. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance.

6.3 CAPACITY CURVES OF G+10 BUILDING MODELS

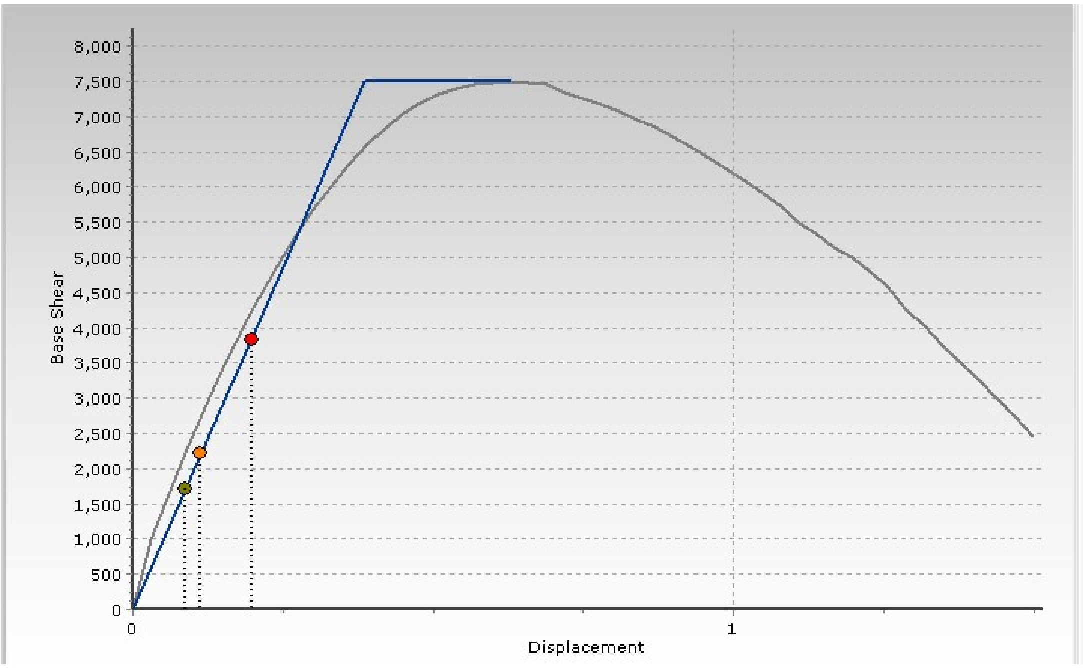


Figure 6-2- Capacity Curves of G+10 (Similar Story Height) Building Models

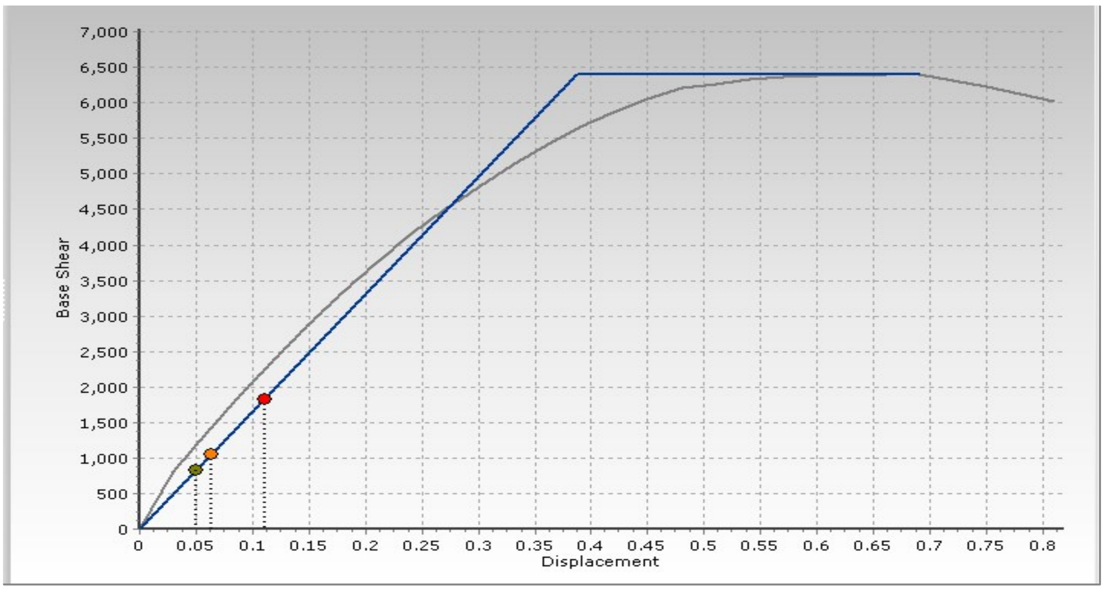


Figure 6-3- Capacity Curves of G+10 200%G Floor Height Building Model

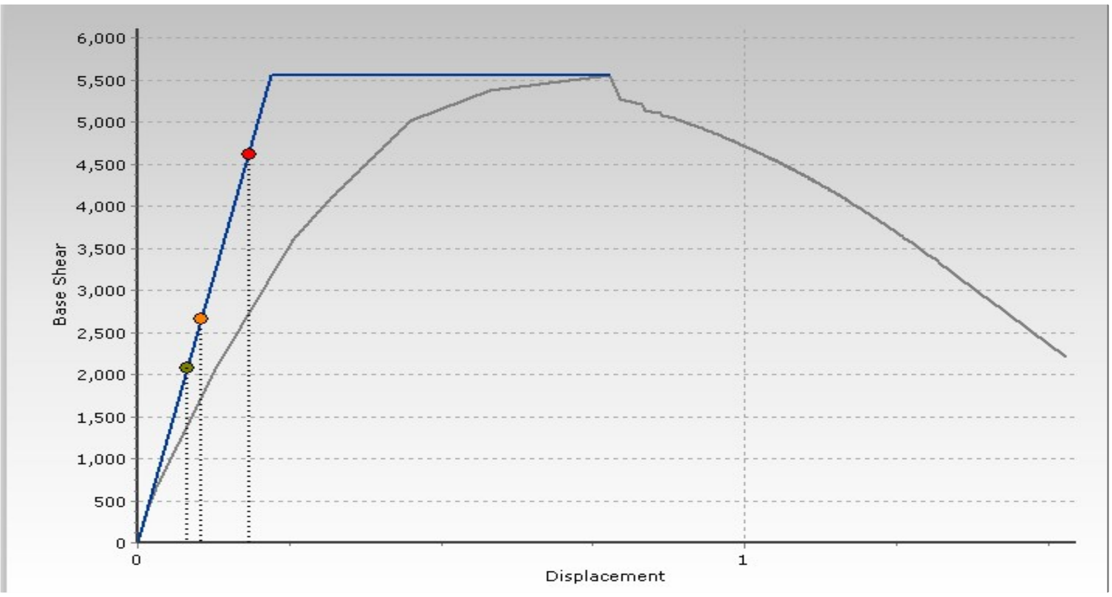


Figure 6-4- capacity Curves of G+10 200%G & S Floors Height Building Model

Table 6-5- Summary of Seismic Base Shears at DL Performance Level for G+10 Building Cases

Building Model Types	Case Designation	Base Shear at DL Performance Level (KN)	Deviation From Model-4 (%)
G+10	Model-4	5670	0
G+10 200%G	Model-5	4820	15.00
G+10 200%G&S	Model-6	3900	31.22

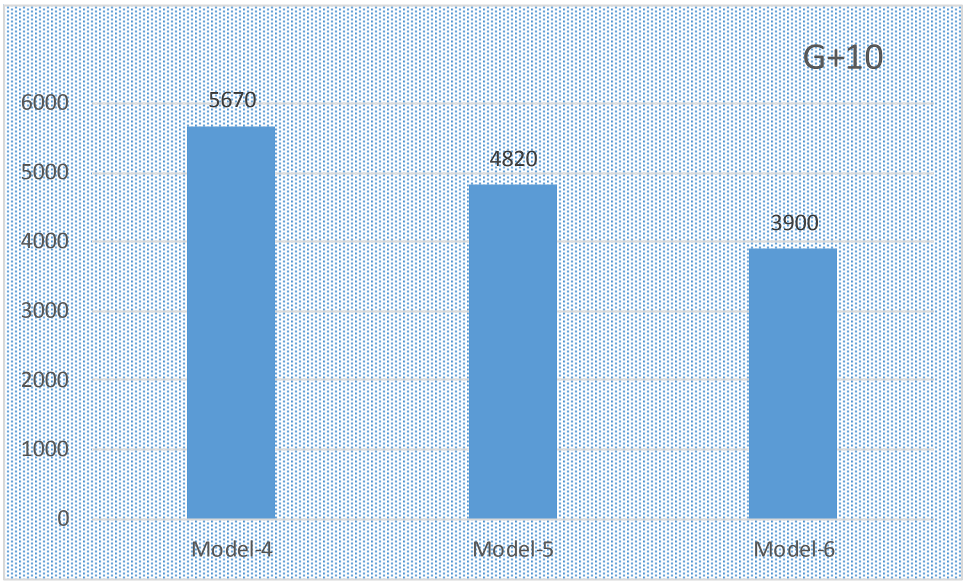


Figure 6-5- Seismic Base Shear at DL Performance Level for G+10 Building Model Types

The above figures illustrate the capacity curves generated from static pushover analysis on SeismoStruct 2016. The curve shows seismic base shear versus roof displacement. It was found that seismic base shear for stiffness regular building model (G+10) is greater than stiffness irregular buildings models (G+10 200%G and G+10 200%G&S).

It was noted that increase the ground story height by 200% which decrease the base shear from 5,670kN to 4,820KN (15%). Similarly increase the ground and second floor height by 200% which decrease the base shear from 5,670kN to 3,900KN (25.9%). Also in the table above seismic base shear has been presented at Damage Limitation (DL) performance level for G+10 building model cases. Damage limitation performance level is an operational state where no significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness.

6.4 CAPACITY CURVES OF G+20 BUILDING MODELS

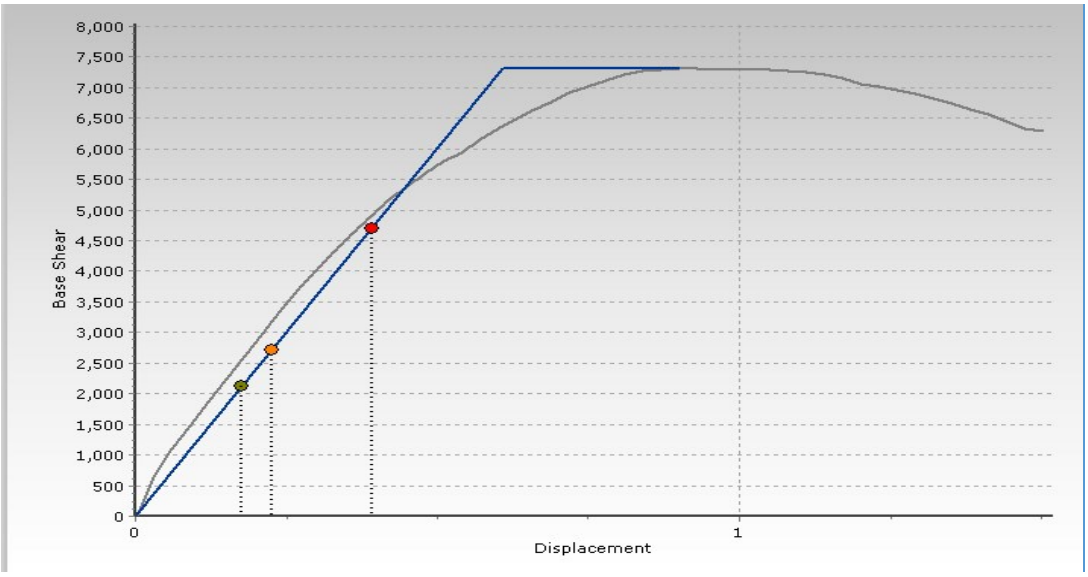


Figure 6-6- Capacity Curves of G+20 (Similar Story Height) Building Models

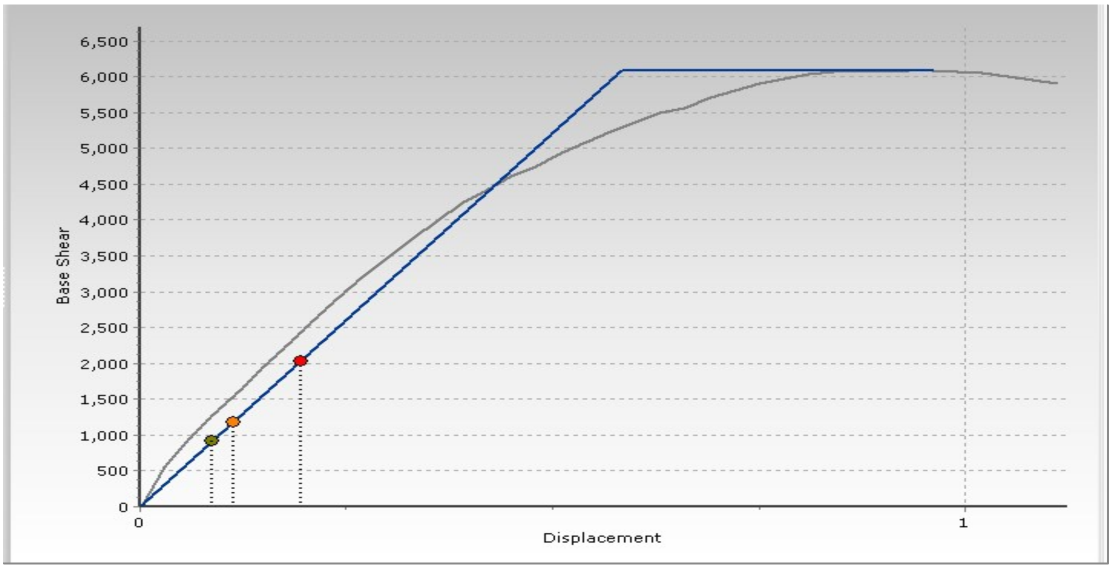


Figure 6-7- Capacity Curves of G+20 200% Floor Height Building Model

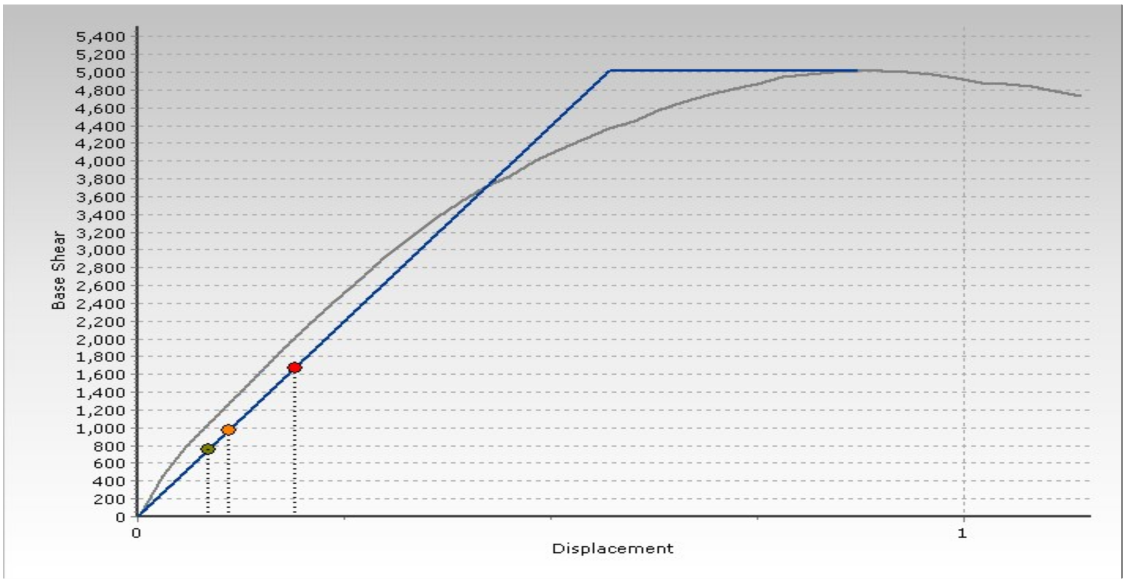


Figure 6-8- Capacity Curves of G+20 200%G & S Floors Height Building Model

Table 6-6- Summary of Seismic Base Shears at IO Performance Level for G+20 Building Cases

Building Model Types	Case Designation	Base Shear at DL Performance Level (kN)	Deviation From Model-4 (%)
G+20	Model-1	5170	0
G+20 200%G	Model-2	4420	14.5
G+20 200%G&S	Model-3	3710	28.24

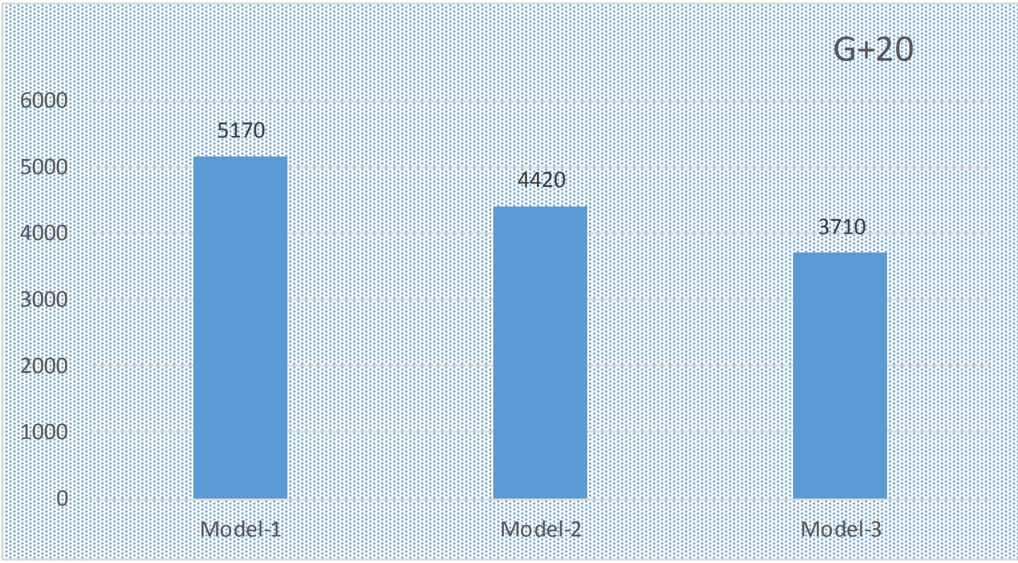


Figure 6-9- Seismic Base Shear at DL Performance Level for G+20 Building Models Types

The above figures illustrate the capacity curves generated from static pushover analysis on SeismoStruct 2016. The curve shows seismic base shear versus roof displacement. It was found that seismic base shear for stiffness regular building model (G+20) is greater than stiffness irregular buildings models (G+20 200%G and G+20 200%G&S).

It was noted that increase the ground story height by 200% which decrease the base shear from 5,170kN to 4,420KN (14.5%). Similarly increase the ground and second floor height by 200% which decrease the base shear from 5,170kN to 3,710KN (28.24%). Also in the table above seismic base shear has been presented at Damage Limitation (DL) performance level for G+20 building model cases. Damage limitation performance level is an operational state where no significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness.

6.5 CONCLUSION ON CAPACITY CURVES

Table 6-7- summary of Seismic Base Shears at DL Performance Level for all Building Cases

Building Model Types	Case Designation	Base Shear at DL Performance Level (kN)	Deviation From Model-4 (%)
G+10	Model-4	5670	0
G+10 200%G	Model-5	4820	15.00
G+10 200%G&S	Model-6	3900	31.22
Building Model Types	Case Designation	Base Shear at DL Performance Level (kN)	Deviation From Model-1 (%)
G+20	Model-1	5170	0
G+20 200%G	Model-2	4420	14.5
G+20 200%G&S	Model-3	3710	28.24

Capacity curves (base shear versus roof displacement) are the load-displacement envelopes of the structures and represent the global response of the structures. Capacity curves for case study frames were obtained from the pushover analyses using aforementioned lateral load patterns on methodology and numerical modelling sections. Also the deformation level and its corresponding seismic base shear at immediate occupancy/operational level were discussed in detail. From the results obtained so far the effect of stiffness irregularity introduced in the frame models has found to be very significant and impressive results were reported. Decrease in the seismic base shear was remarkable with increasing story height of the selected floor levels (ground and second floors) which makes the story soft. The increase in stiffness of the structure will decrease a fundamental period of vibration and in turn results in increase of design spectrum ordinate which would obviously rise up the seismic base shear. In this building model cases, all the structural aspects of the models were designed in efficient and most appropriate way that the variations in the number of story was found to be perfectly proportional to variation of stiffness and fundamental vibrations periods. Therefore, as it has been seen above the percentage deviations of seismic base shear for G+10 building models are almost similar to the corresponding values G+20 building model, this shows that the effect of stiffness irregularity on seismic base have more significant contributions as the number of soft- story increase.

6.6 NONLINEAR DYNAMIC TIME HISTORY ANALYSIS

Time history analysis is a powerful tool for the study of structural seismic response. It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history. Recorded ground motion from past natural earthquakes can be used for time history analysis. It has been stipulated that artificial accelerograms shall be generated so as to match the elastic response spectra used in the design for 5% viscous damping. Accordingly, Sample artificial accelerogram (TH-5) used in the simulation of models

Nonlinear responses of structures using nonlinear dynamic time history analysis are very sensitive to their modelling method and the character of chosen earthquake excitations. Therefore, to predict the modes of deformations of the structure, a series of time history ground motions with different intensity, frequency, and various time history features were used in this research. All response spectra of the artificial accelerograms were scaled based on the Ethiopian building code response spectrum for Addis Ababa under soil type C. the scaled response spectra for the generated ground motions are shown in figures below.

In nonlinear dynamic analyses, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model. Higher-level demands (element distortions, story drifts, and roof displacement) are derived directly from the basic component actions. Accordingly, in this paper 30 artificial accelerograms are generated on SeismoArtif (SeismoSoft 2016) having 30sec duration and different frequencies and magnitudes. Generated artificial accelerograms suits with elastic response spectra conforming to the used code and has been employed in the simulation of nonlinear dynamic time history analysis for numerically modelled building cases. The discussion parameters and performance evaluations in terms of story displacements, inter-story drift and fragility curve is entirely based on the results obtained from this nonlinear dynamic time history analysis; and they are explicitly presented in the next sections of this document.

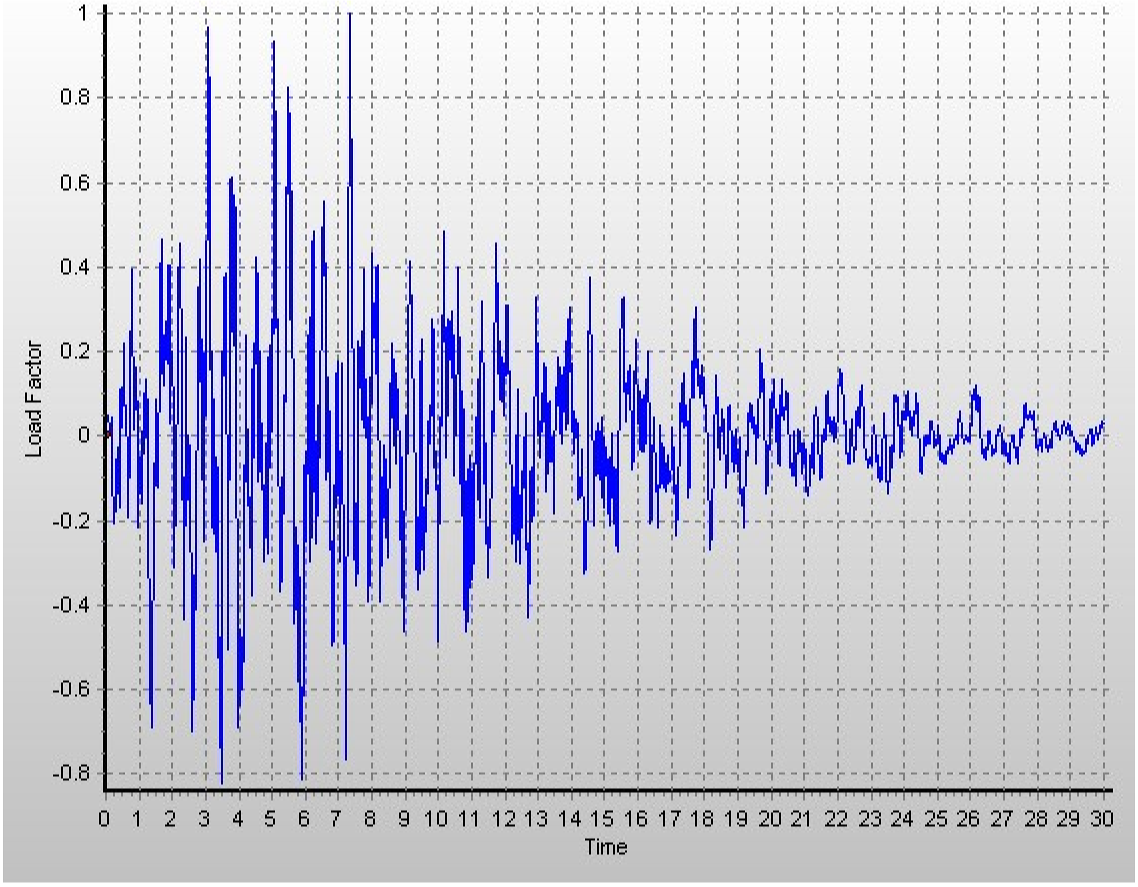


Figure 6-10- Sample Artificial Accelerogram (TH-5) Used in the Simulation of Models

6.7 STORY DISPLACEMENTS

Seismic performance evaluation is directly related to displacement or deformation and thus estimation of seismic deformation demand is a primary or fundamental concern in performance evaluation of reinforced concrete structures under seismic excitation. The basic analysis approach consists of performing nonlinear dynamic time history analysis for a given structure and ground motion, using three-dimensional nonlinear analysis on SeismoStruct software. The story displacement of the case study building models were studied under randomly selected individual ground motions. Accordingly, out of employed 30 ground motions set in the dynamic analysis, one ground motions were considered for evaluation of building performance with respect to story displacements.

6.8 G+10 BUILDINGS MODEL CASES

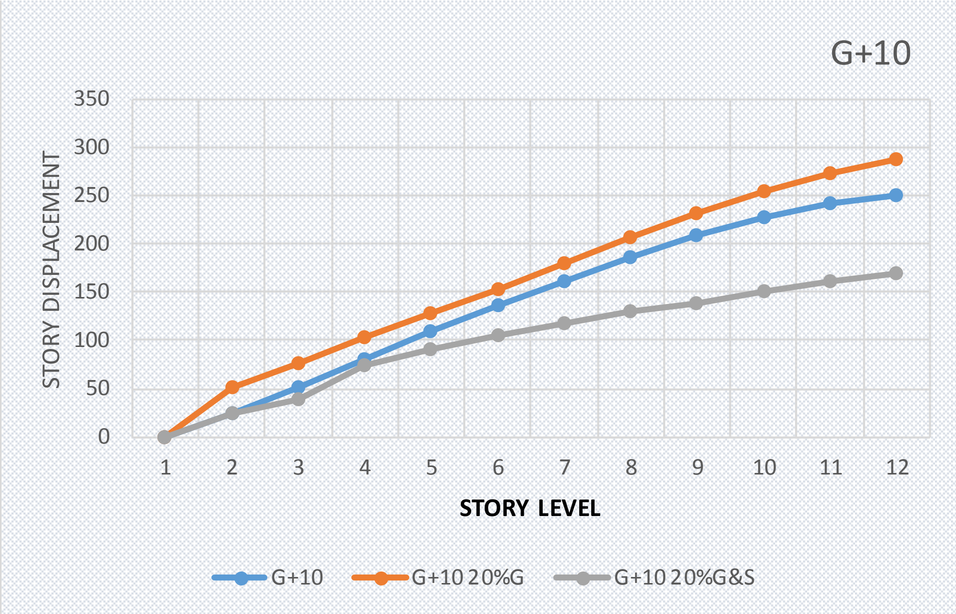


Figure 6-11- Story Displacements of G+10 Building Model Cases TH-5 Ground Motion

Referring to the above figures, the story displacements of G+10 building model cases were studied under randomly selected ground motions under simulated ground motions the G+10 200%G frame model displaced in largely compared to other building model. It was found that the roof displacements of G+10 200%G, G+10 and G+10 200%G&S building models are 287, 251, and 169 mm respectively under TH-5 ground motion. Also from the figures it can be seen that ground story of G+10 200%G and ground and second stories of G+10 200%G&S buildings have displaced abruptly and the displacements then followed gradual increase with level of stories. This shows that floors which have double story height relative to others are more susceptible for soft story problems and they attract more stresses and thus subjected to larger deformation at the instant of lateral action.

6.9 G+20 BUILDING MODEL CASES

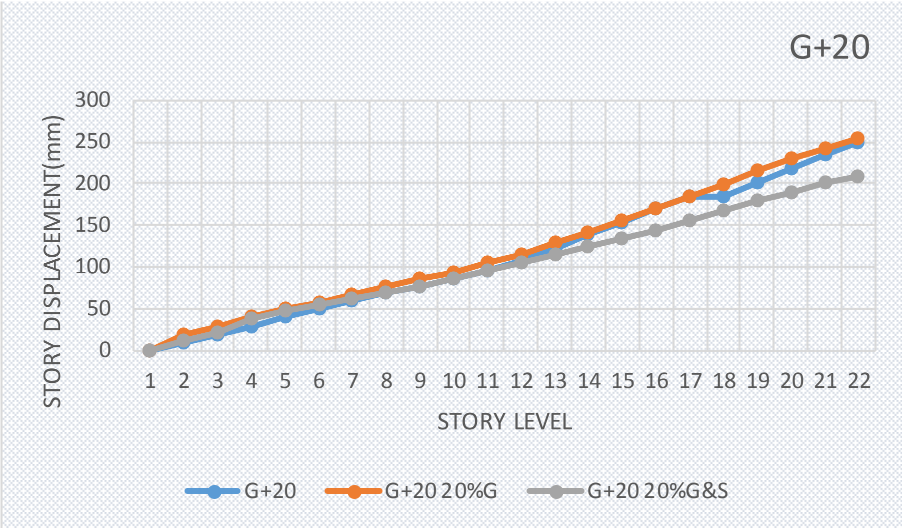


Figure 6-12- Story Displacement of G+20 Building Cases Under TH-5 Ground Motion

Referring to the above figures, the story displacements of G+20 building model cases were studied under randomly selected ground motions under simulated ground motions the G+20 200%G frame model displaced in largely compared to other building model. It was found that the roof displacements of G+20 200%G, G+20 and G+20 200%G&S building models are 254, 248, and 208 mm respectively under TH-5 ground motion. Also from the figures it can be seen that ground story of G+20 200%G and ground and second stories of G+20 200%G&S buildings have displaced abruptly and the displacements then followed gradual increase with level of stories. This shows that floors which have double story height relative to others are more susceptible for soft story problems and they attract more stresses and thus subjected to larger deformation at the instant of lateral action.

6.10 INTER-STORY DRIFT

Seismic performance evaluation is directly related to displacement or deformation and thus estimation of seismic deformation demand is a primary or fundamental concern in performance evaluation of reinforced concrete structures under seismic excitation. The basic analysis approach consists of performing nonlinear dynamic time history analysis for a given structure and ground motion, using three-dimensional nonlinear analysis on SeismoStruct software. The inter story displacement of the case study building models were studied under randomly selected individual ground motion. Accordingly, out of employed 30 ground motions set in the dynamic analysis, one ground motions were considered for evaluation of building performance with respect to story displacements.

6.10.1 STORY DRIFT FOR G+10 BUILDING MODEL CASES

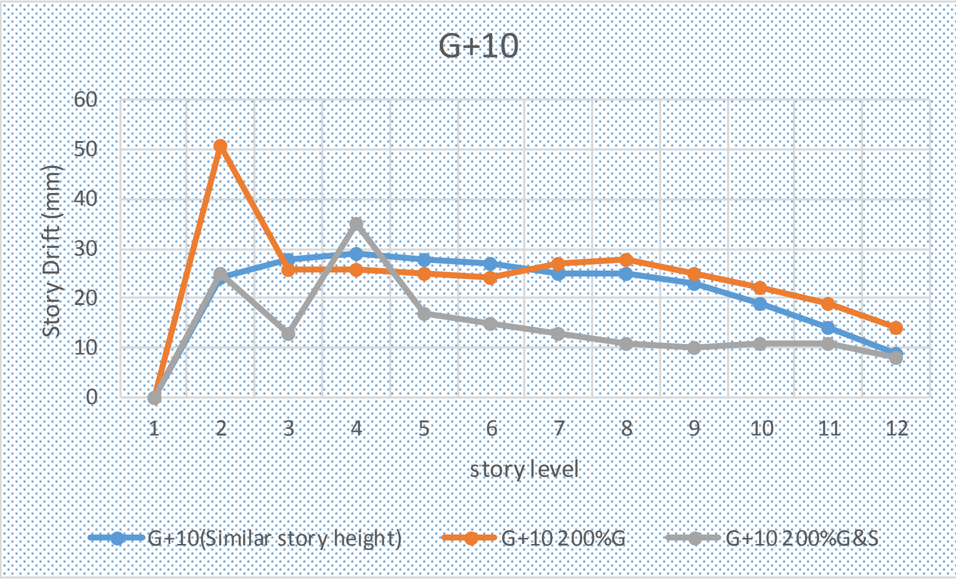


Figure 6-13- Story Drift of G+10 Building Model Cases Under TH-5 Ground Motion

Referring to the above figures, the distributions of story drift demands over height for the cases with stiffness irregularities are very different from the base case (G+10). The above figure shows the values of maximum story drifts in millimeter for the case studied. The presence of a soft story drastically increases the drifts in the soft story and decreases the drifts in the other stories. The abrupt change in the slope of the profile indicates the stiffness irregularity. All drift profiles corresponding to models having stiffness irregularity have a sudden change of slope at ground floor level and second floor; however, the regular building shows smooth displacement profiles. The buildings with excess height at ground and second stories are weaker than the regular building, because lateral stiffness at soft story is quite less than the regular building. From the graph we can conclude that drift of most floors are greater than the allowable drift according to ES EN limit (drv≤0,005h).Also storey drift demands are more sensitive to stiffness irregular one.

6.10.2 STORY DRIFT FOR G+20 BUILDING MODEL CASES

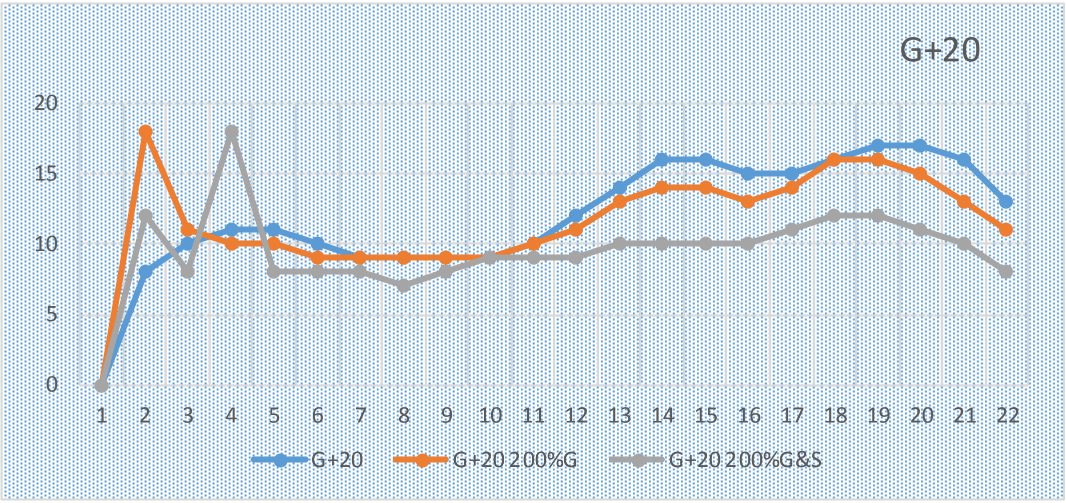


Figure 6-14- Story Drift of G+20 Building Model Cases Under TH-5 Ground motion

Referring to the above figures, the distributions of story drift demands over height for the cases with stiffness irregularities are very different from the base case (G+10). The above figure shows the values of maximum story drifts in millimeter for the case studied. The presence of a soft story drastically increases the drifts in the soft story and decreases the drifts in the other stories. The abrupt change in the slope of the profile indicates the stiffness irregularity. All drift profiles corresponding to models having stiffness irregularity have a sudden change of slope at ground floor level and second floor; however, the regular building shows smooth displacement profiles. The buildings with excess height at ground and second stories are weaker than the regular building, because lateral stiffness at soft story is quite less than the regular building. From the graph we can conclude that drift of most floors are greater than the allowable drift according to ES EN limit ($dr_v \leq 0{,}005h$). Also storey drift demands are more sensitive to stiffness irregular one.

6.10.3 Conclusion on Story Drift of G+10 and G+20 Model Cases

From the graph we can conclude that drift of most floors are greater than the allowable drift according to ES EN limit ($dr_v \leq 0{,}005h$). Also storey drift demands are more sensitive to stiffness irregular one and the the sorey drift demand of G+10 buildings are greater than the corresponding G+20 buildings.

6.11 DEVELOPMENT OF FRAGILITY CURVE

Fragility curves represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation, they also involve uncertainties associated with structural capacity, damage limit state definition and records of ground motion accelerations. And mathematically, fragility curves can be defined as the probability of exceedance of damage at various levels of ground motion, which is considered as an Intensity Measure. The fragility function represents the probability of exceedance of a selected Demand Parameter (EDP) for a selected structural limit state (LS) for a specific ground motion intensity measure (IM). Fragility curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand.

In order to develop fragility curve 30 artificial accelerograms having different magnitudes are generated, scaled and matched with Ethiopian response spectrum on SeismoArtif [SeismoSoft, 2016] so as to use as earthquake records for nonlinear time history analysis. SeismoArtif is an application capable of generating artificial earthquake accelerograms matched to a specific target response spectrum using different calculation methods and varied assumptions for nonlinear dynamic analysis of new or existing structures. Accordingly, six building model cases are loaded with each 30 artificial accelerograms and a total 180 model cases are prepared for the nonlinear time history analysis.

Seismic fragility curves are then developed by combining results from pushover and nonlinear time history analysis for all building model cases at various performance levels. Fragility responses are computed and fragility curves (indicator of the probability of failure) for each building model case are developed for different performance levels in terms of PGA by combining the limit state capacities and the PSDMs using Microsoft excel. Out of the various existing methodologies for development of fragility curves, a method based on nonlinear time history analysis and the probabilistic demand model suggested by Cornell et al (2002) is considered in this study. Accordingly, fragility curves are developed for the selected buildings.

6.11.1 FRAGILITY CURVES OF G+20 BUILDING MODELS

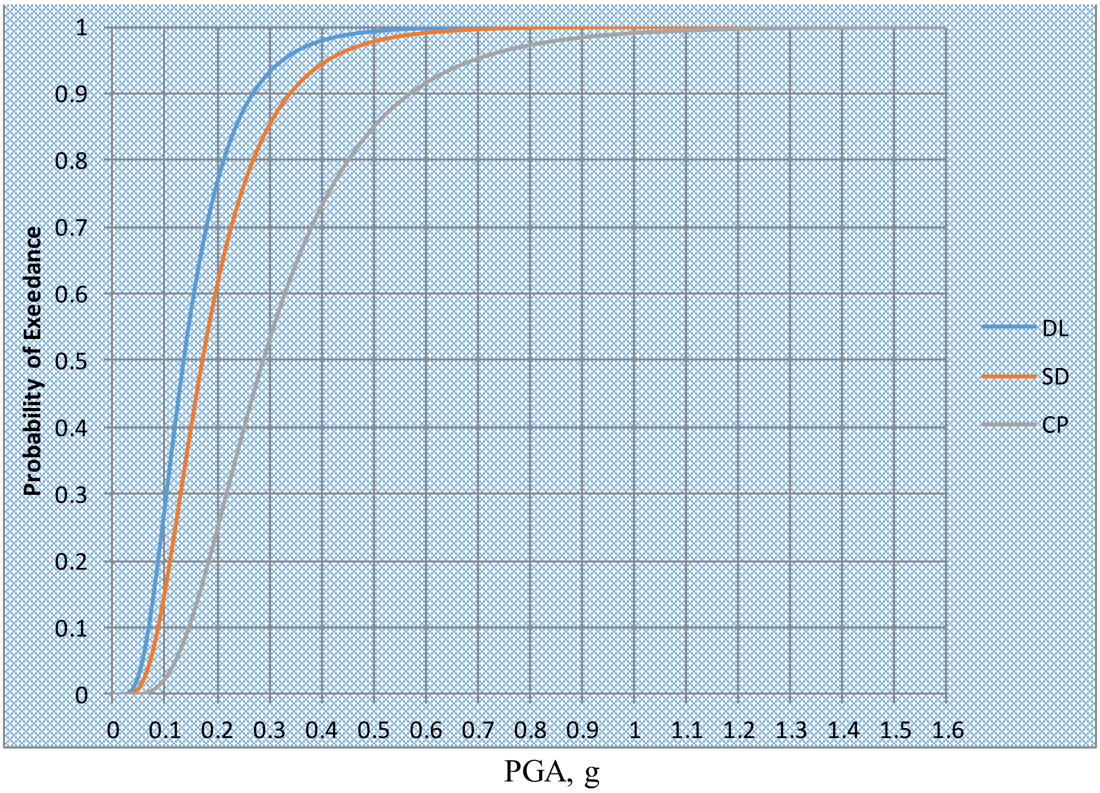


Figure 6-15- Fragility Curve of G+20 (Similar Story Height) Building Model for Three Limit States

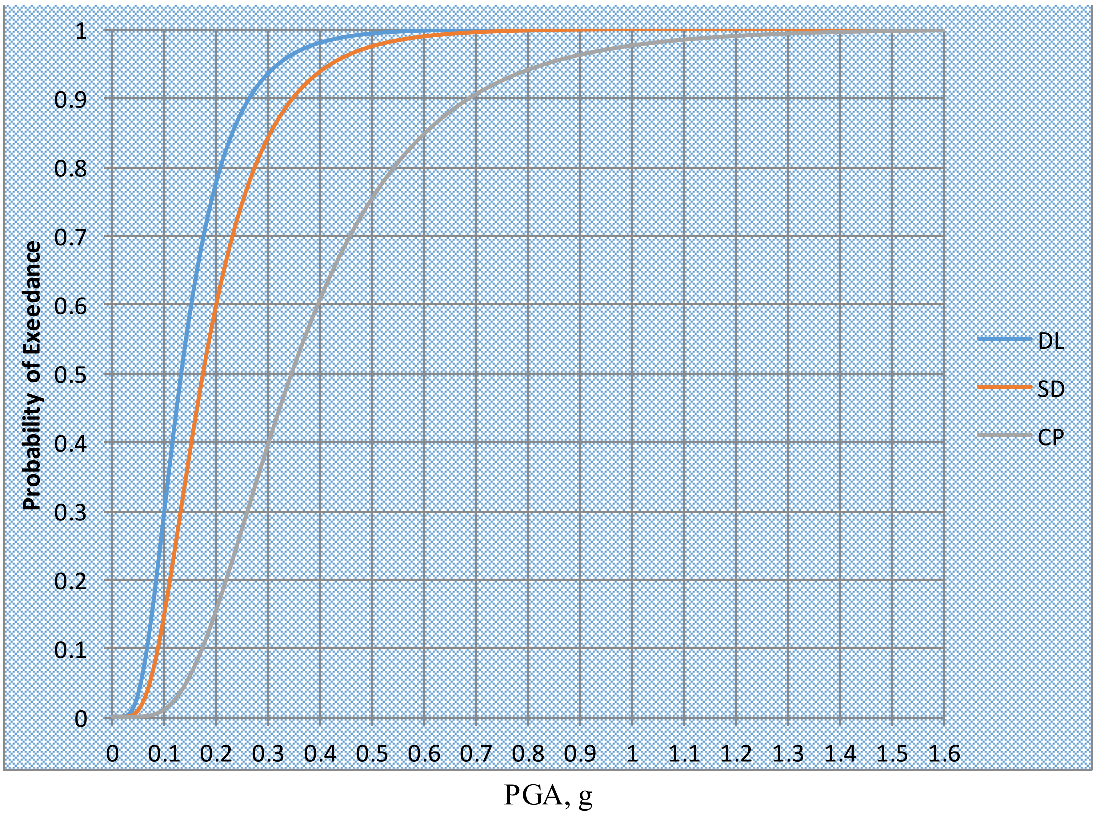


Figure 6-16- Fragility Curve of G+20 200%G Building Model for Three Limit States

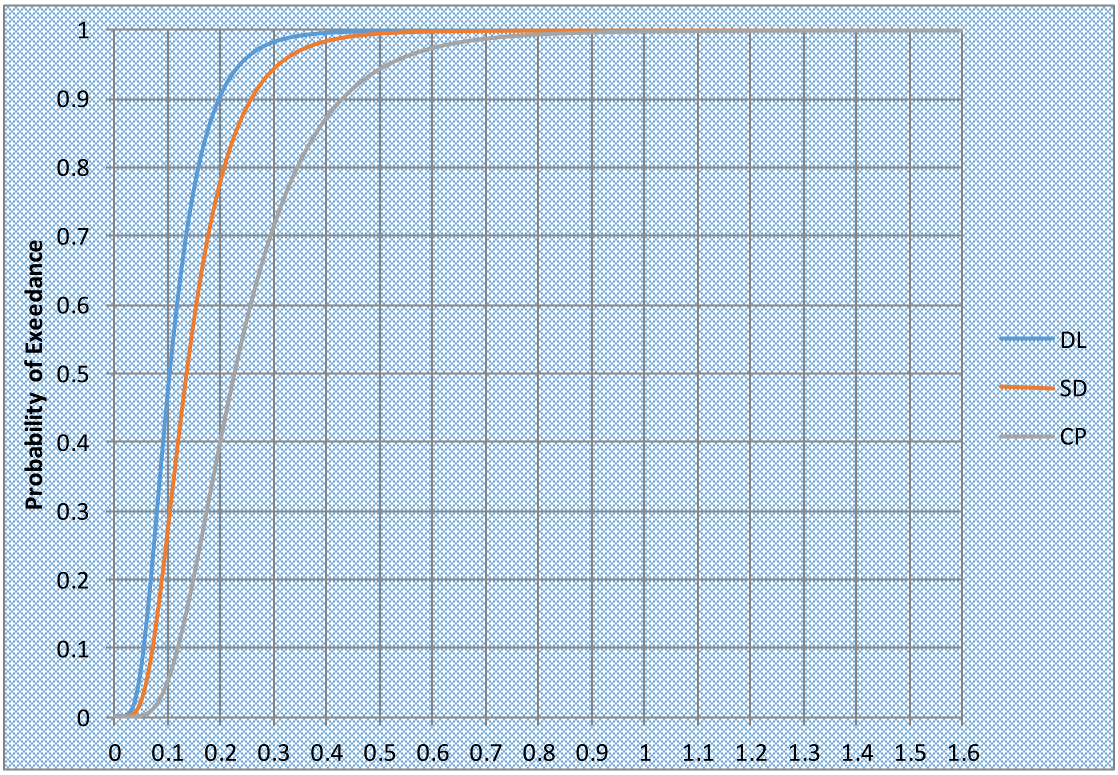


Figure 6-17- Summary of Fragility Curve of G+20 200%G & S Building Model for Three Limit State

Table 6-8- Summary of Fragility Curve of G+20 Building Models in Terms of Probability of Failure

Building Model Type	Case Designation	Probability of failure for each performance level (%)		
		DL	SD	CP
G+20(similar story height)	Model-1	25	15	2.4
G+20 200%G	Model-2	30	20	2.8
G+20 200%G&S	Model-3	40	23	4

The above figures illustrate the fragility curves developed by combining results from pushover and nonlinear time history analysis for all building model cases at various performance levels. The curve shows probability of exceedance versus peak ground acceleration. It was found that the probability of failure for each performance level for stiffness irregular building model (G+20 200%G and G+20200%G&S) is greater than stiffness regular buildings models (G+20).

It was noted that increase the ground story height by 200% which increase the probability of failure at DL, SD and CP from 25%to 30%, 15%to 20% and 2.4%to 2.8% respectively. Similarly increase the ground and second floor height by 200% which increase. The probability of failure at DL, SD and CP from 25%to 40%, 15%to 23% and 2.4%to 4% respectively. This shows that buildings which have double story height relative to others are more fragile at all performance level.

6.11.2 FRAGILITY CURVES OF G+10 BUILDING MODELS

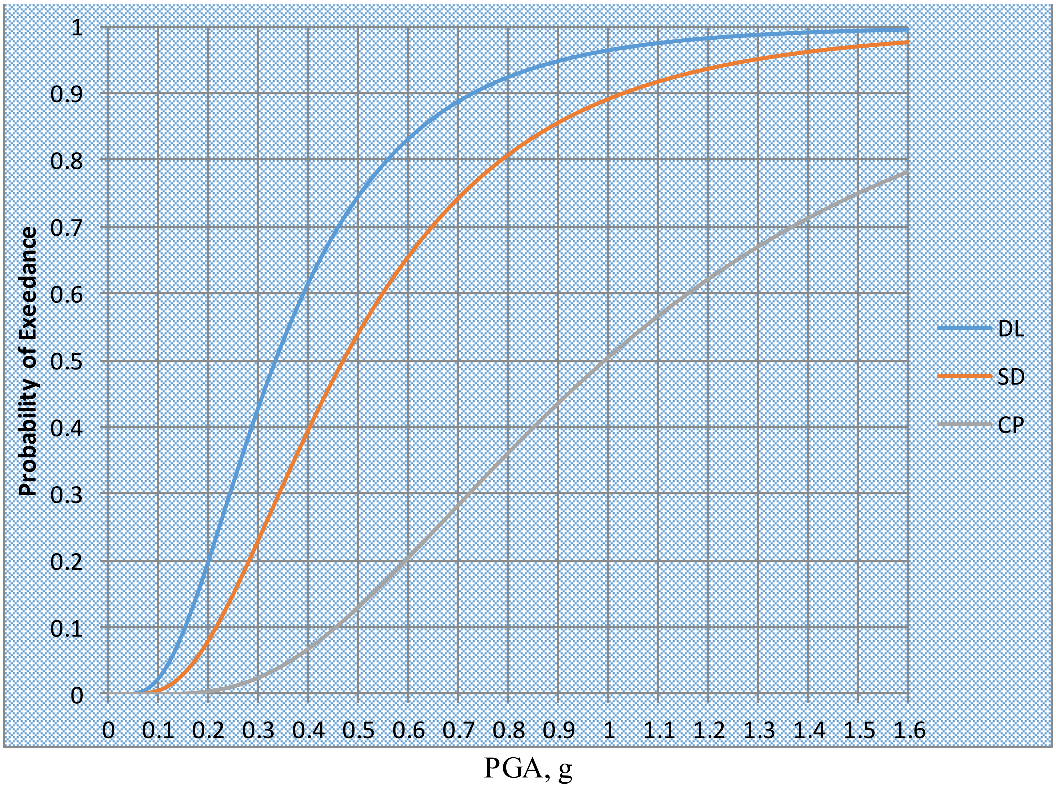


Figure 6-18- Fragility Curves of G+10 (Similar Story Height) Building Model for Three Limit States

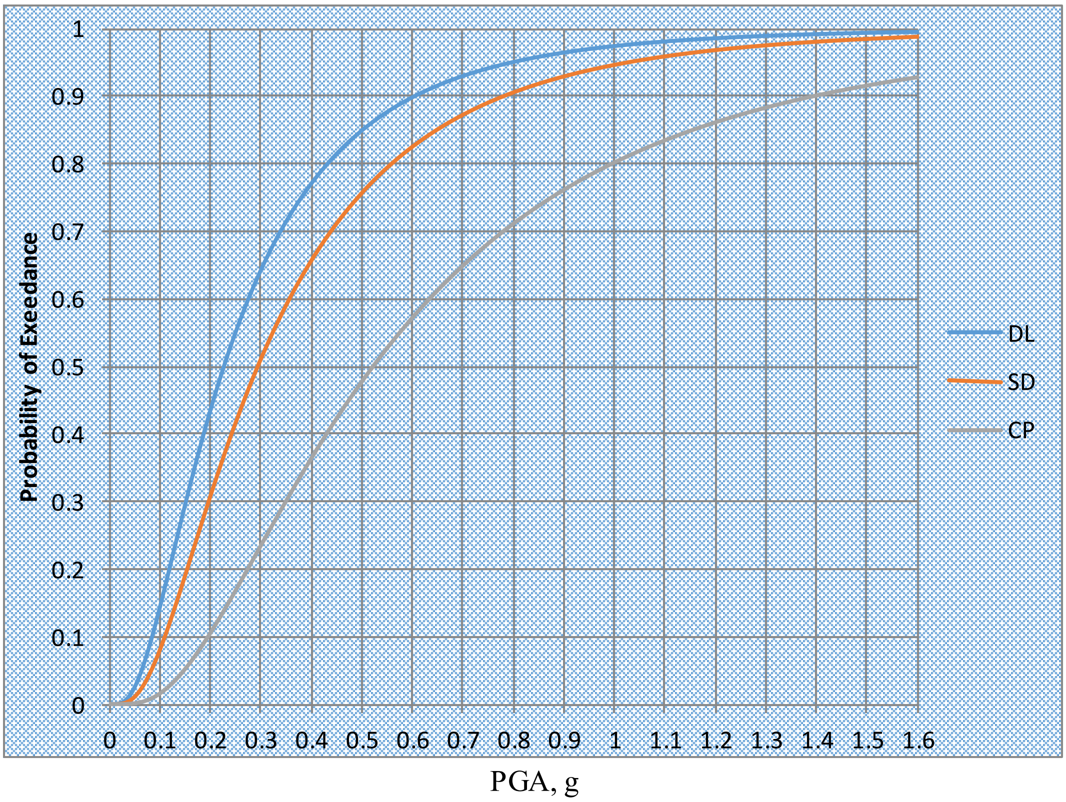


Figure 6-19- Fragility Curve of G+10 200% Building Model for Three Limit States

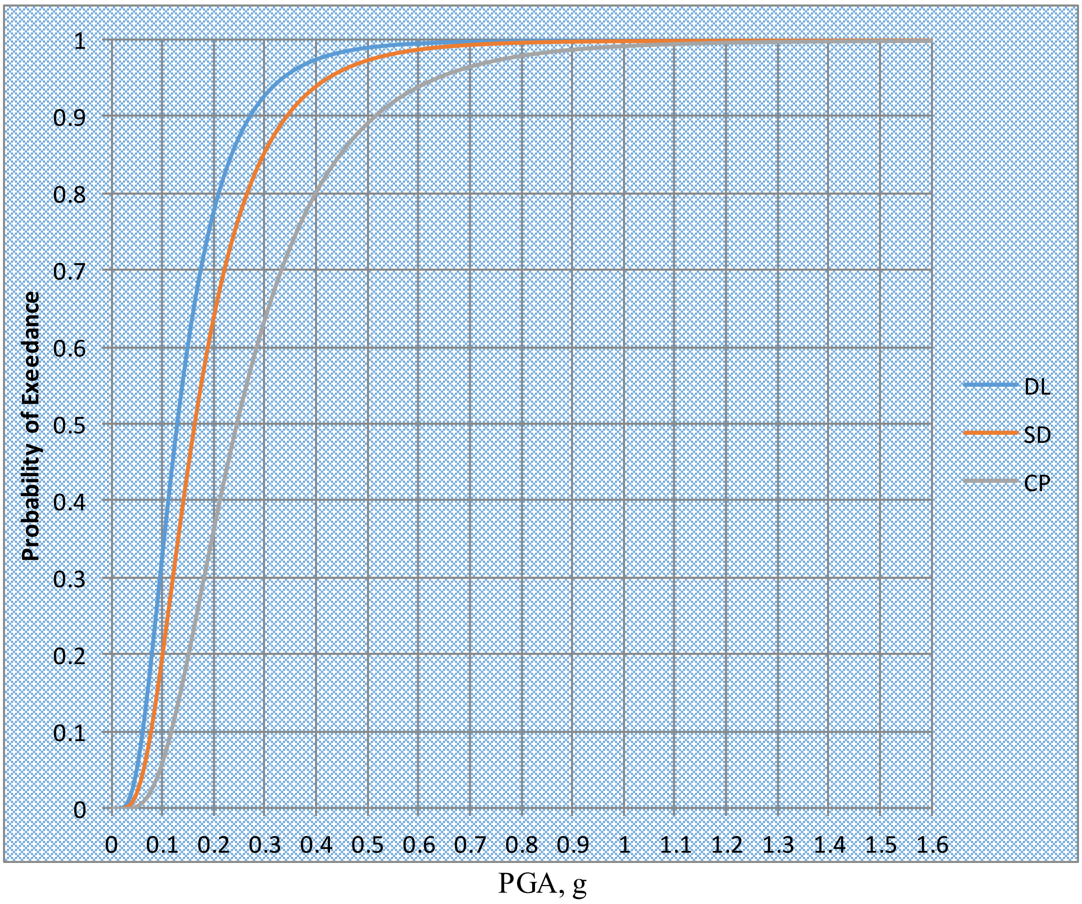


Figure 6-20- Fragility Curve of G+10 200%G Building Model for Three Limit States

Table 6-9- Summary of Fragility Curves of G+10 Building Models in Terms of Probability of Failure

Building Model Type	Case Designation	Probability of failure for each performance level (%)		
		DL	SD	CP
G+10(similar story height)	Model-4	5	0	0
G+10 200%G	Model-5	14	7	1.5
G+10 200%G&S	Model-6	30	19	4

The above figures illustrate the fragility curves developed by combining results from pushover and nonlinear time history analysis for all building model cases at various performance levels. The curve shows probability of exceedance versus peak ground acceleration. It was found that the probability of failure for each performance level for stiffness irregular building model (G+10 200%G and G+10200%G&S) is greater than stiffness regular buildings models (G+10).

It was noted that increase the ground story height by 200% which increase the probability of failure at DL, SD and CP from 5%to 14%, 0%to 7% and 0%to 1.5% respectively. Similarly increase the ground and second floor height by 200% which increase. The probability of failure at DL, SD and CP from 5%to 30%, 0%to 19% and 0%to 4% respectively. This shows that buildings which have double story height relative to others are more fragile at all performance level.

6.11.3 CONCLUSION ON FRAGILITY CURVES

Table 6-10- Summary of Fragility Curve of G+10 and G+20 Building Models in Terms of Probability of Failure

Building Model Type	Case Designation	Probability of failure for each performance level(%)		
		DL	SD	CP
G+20(similar story height)	Model-1	25	15	2.4
G+20 200%G	Model-2	30	20	2.8
G+20200%G&S	Model-3	40	23	4
G+10(similar story height)	Model-4	5	0	0
G+10 200%G	Model-5	14	7	1.5
G+10200%G&S	Model-6	30	19	4

From the above summarized result, it was found that the probability of failure for each performance level for stiffness irregular building model is greater than stiffness regular buildings models in all case.

CHAPTER SEVEN

7 SUMMARY AND CONCLUSIONS

7.1 SUMMARY

The main objective of this study was to evaluate the seismic performance of stiffness irregular buildings due to significant story height difference on RC buildings. To achieve this objective, the following specific objectives were proposed.

- Establishment of limit state capacities at different damage limit states.
- Development of probabilistic seismic demand models (PSDM).
- Development of fragility curves for various performance levels of buildings under the excitation of seismic hazard.
- To evaluate the effect of stiffness irregularity on the distribution inelastic seismic demand and fundamental period of reinforced concrete building.

To achieve the above objectives a detailed literature review was conducted to identify research needs, building selection, seismic record selection, analysis methods, performance criteria and performance objectives, effects of many parameters on the seismic performance and development of fragility curves.

Accordingly, the proposed RC buildings is performed on ETABS 2016.2.1 following the new Ethiopian Building code analysis and design approach. All building model cases are analyzed both for gravitational loads and earthquake loads by situating proposed study site area in Addis Ababa (earthquake zone-III) using response spectrum method. The numerical values found from the design section are then used for numerical modeling of RC frames on finite element software package (SeismoStruct 2016). Pushover and nonlinear time history analysis are performed on all model cases. Finally, the performance of the model structures at different performance levels has been investigated and their results are discussed in terms of the leading response parameters such as fundamental periods, total base shears, inter-story drifts, lateral displacements, and seismic fragility curves.

7.2 CONCLUSION

In this research work, the seismic performance of building with stiffness irregularities are studied Stiffness irregularity is created at ground and second floors by increasing the height of the floors 200% of other floors. Pushover and nonlinear time history analysis are performed on all model cases. Finally, the performance of the model structures at different performance levels has been investigated and their results are discussed in terms of the leading response parameters such as fundamental periods, total base shears, inter-story drifts, lateral displacements, and seismic fragility curves. Therefore, conclusions from this study are drawn as follows:

- The percentage deviations of fundamental periods of two stories (G+10 200%G&S) stiffness irregular building from the stiff one(G+10) for G+10 building models are greater the corresponding values G+20 building model, this shows that the effect of stiffness irregularity on fundamental period have more significant contributions as the soft-story number increase. Also it has more significant contributions as story number increase or as the building gets high-rise.
- The seismic base shear for both stiffness regular building models of G+10 and G+20 are greater than stiffness irregular buildings models (G+10 200%G, G+10 200%G&S, G+20 200%G and G+20 200%G&S).
- For the cases of inelastic analysis, roof displacement demands are not sensitive to the presence of stiffness irregularities. For the cases of stiffness modification factor, roof displacement demands do not change by more than 15% from the base case.
- The change in distribution of story drift over height due to stiffness irregularities are non-uniform for all cases studied. Story drifts demands increase in the soft story and decrease in most of the other stories. From the results we can conclude that drift of most floors are greater than the allowable drift according to ES EN limit ($d_r v \leq 0{,}005h$).
- The probability of failure for each performance level (DL, SD and CP) for stiffness irregular building model is greater than stiffness regular buildings models in all cases.
- The probability of failure for each performance level (DL, SD and CP) for G+20 building models are greater than G+10 building models in all cases.

7.3 RECOMMENDATION

Recommendations for potential extended studies and selected future research needs related to seismic fragility analysis and probabilistic analysis are listed below:

- The structures considered in this study are regular in plan and the lateral forces are assumed to be applied at the mass center, therefore, future study can cover unsymmetrical building with significant torsion.
- Reinforced buildings with basement, shear walls and infill walls are not considered in this study. The present methodology can be extended to such buildings also.
- In this study, mass irregularity is not considering, so future study will cover mass irregularity with stiffness irregularity at a time in many floors over height of the building.

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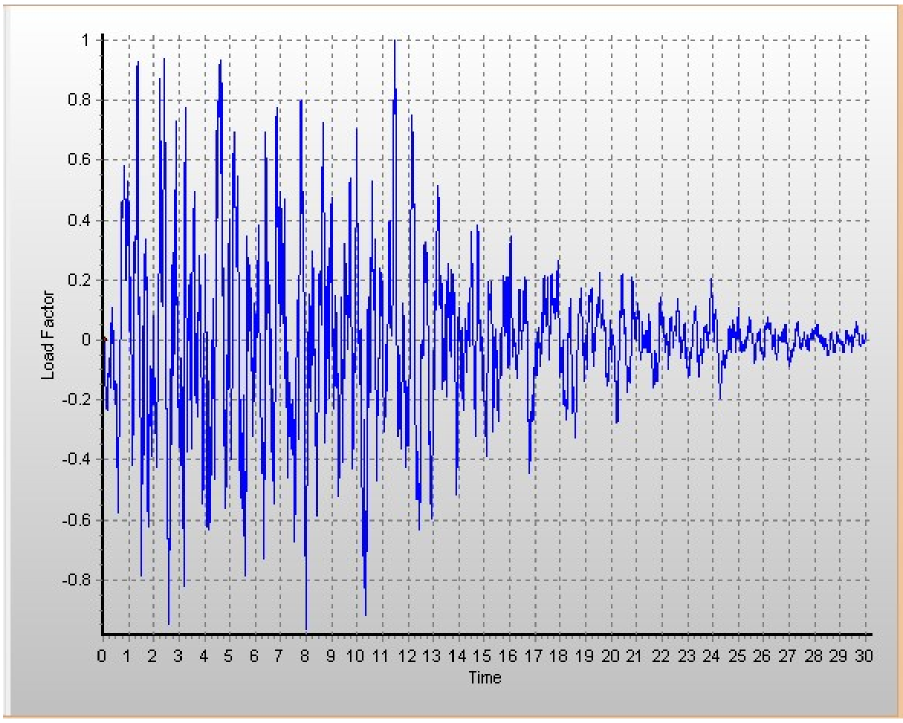
APPENDIX – A

ARTIFICIAL GROUND MOTIONS



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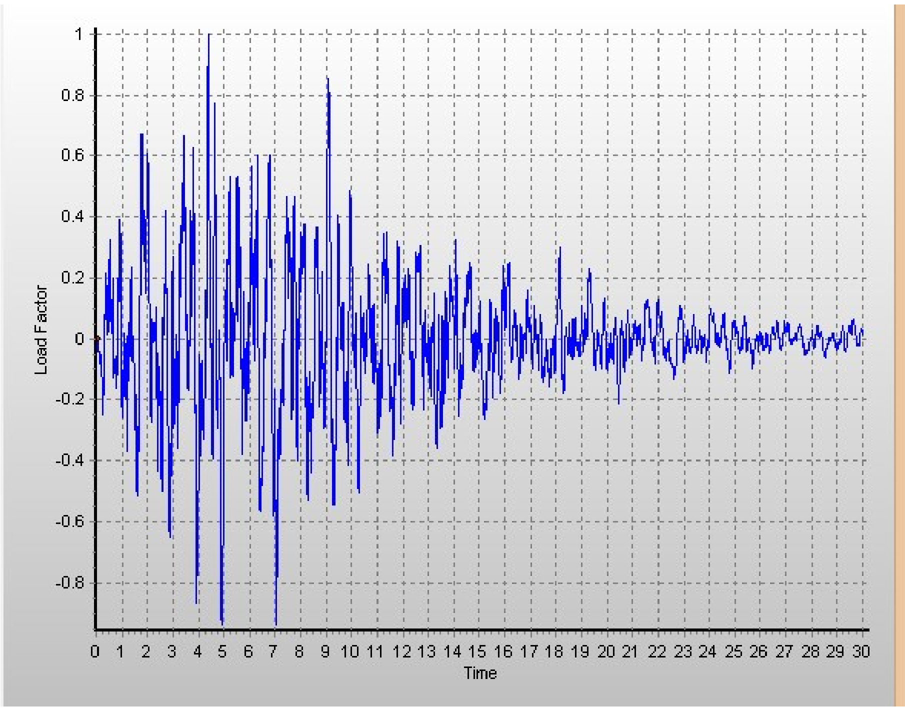

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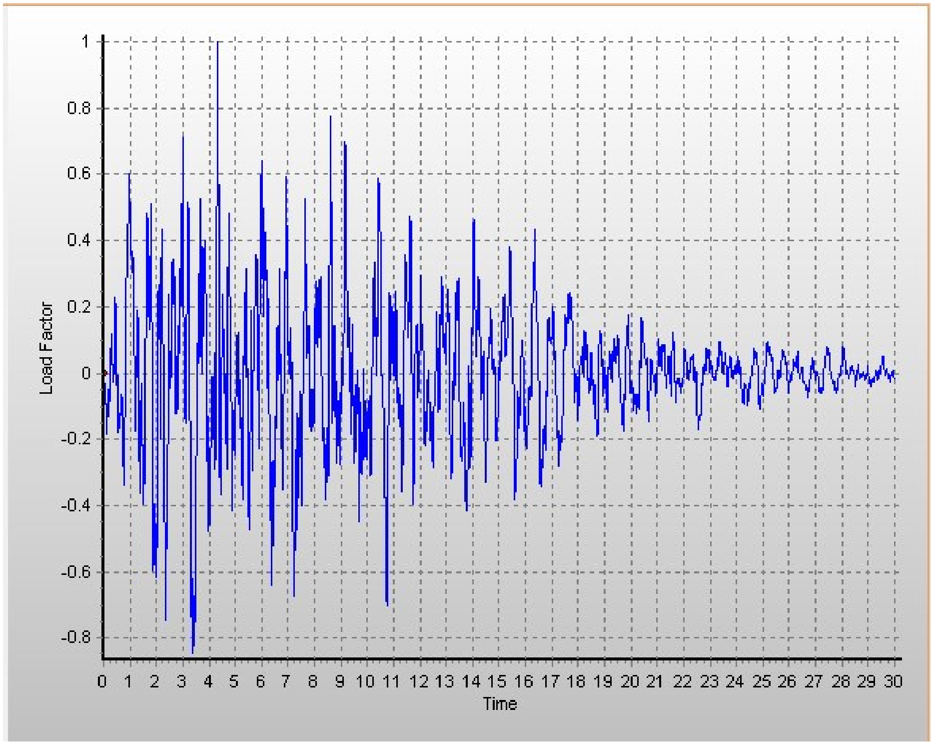
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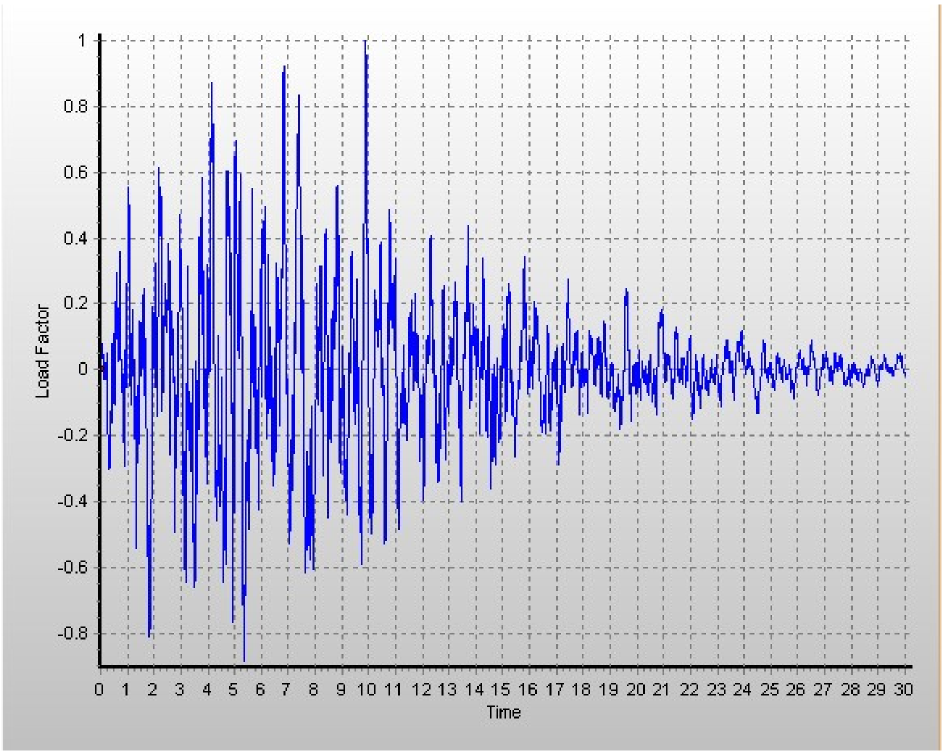
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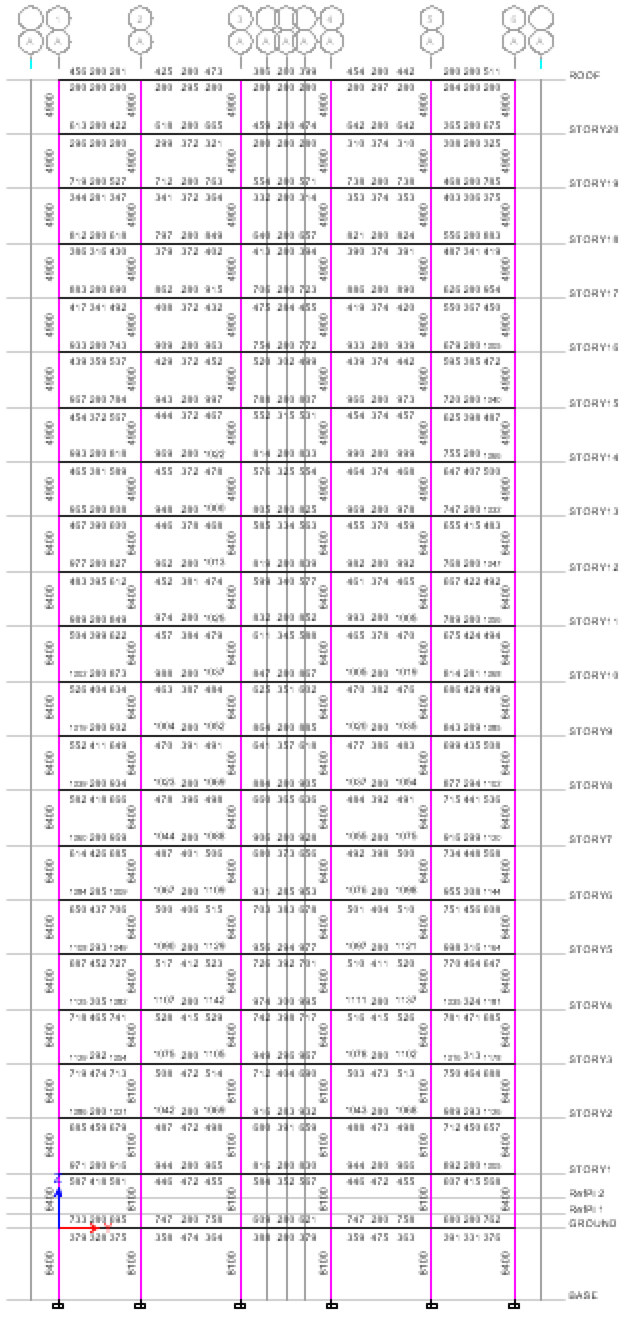
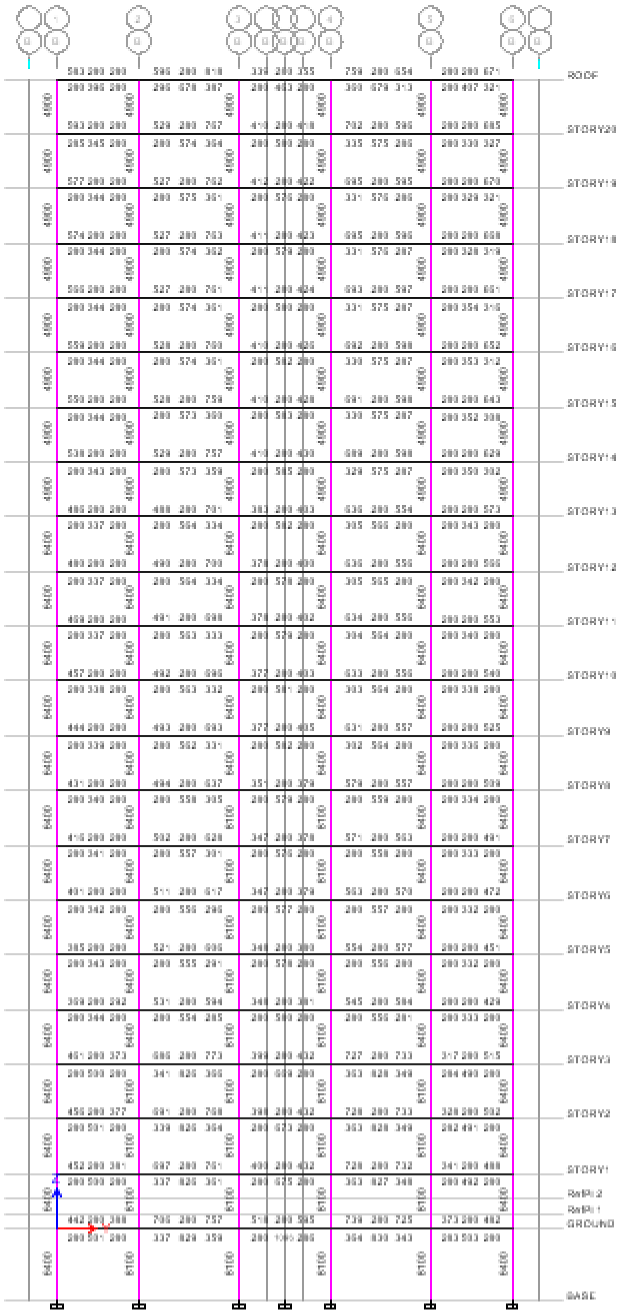
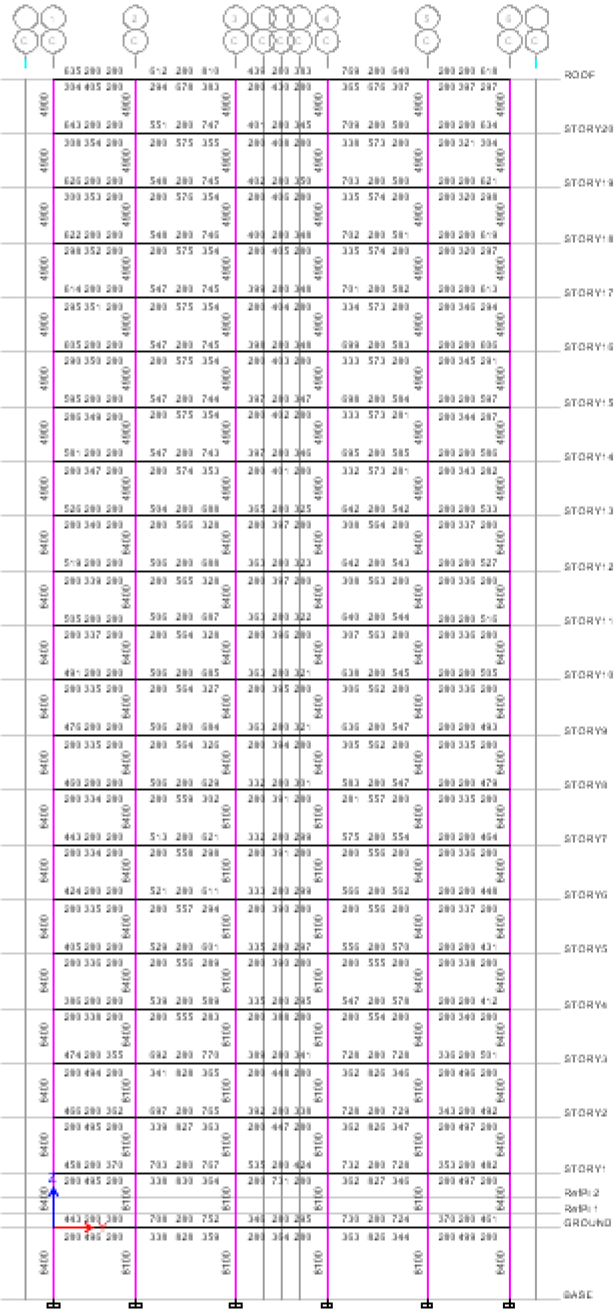
TH-4

APPENDIX-B

STRUCTURAL DESIGN OUTPUTS



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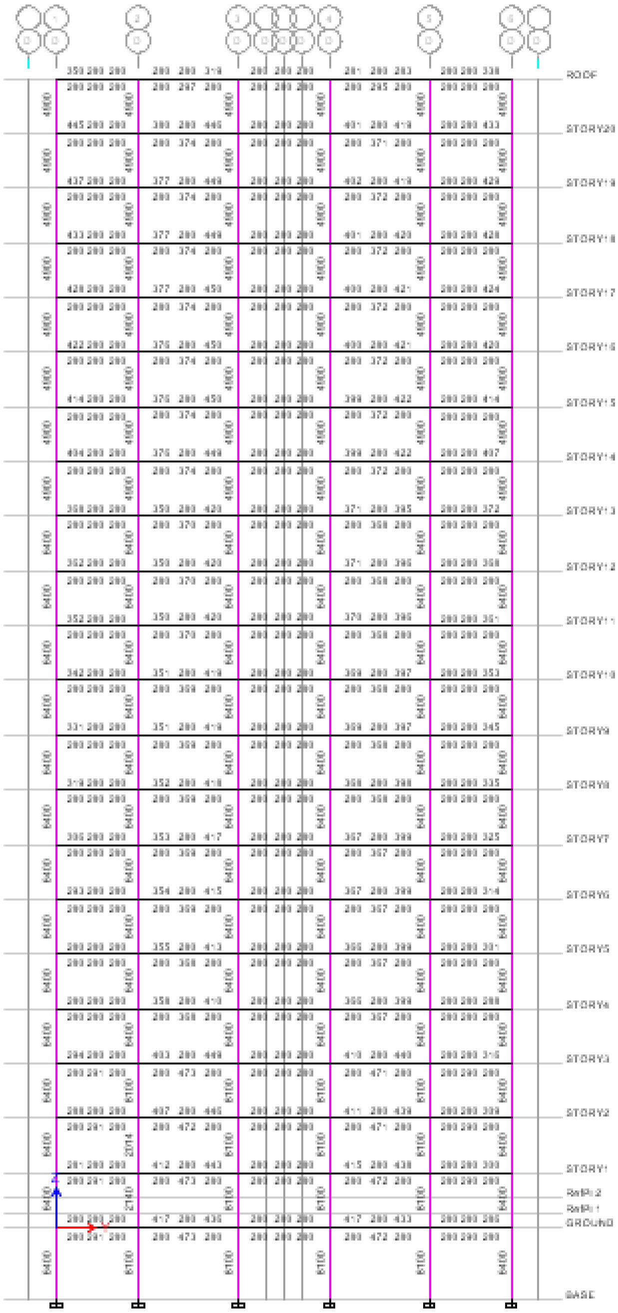
STORY5
STORY4
STORY3
STORY2
STORY1
RetPit2
RetPit1
GROUND
BASE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GROUND
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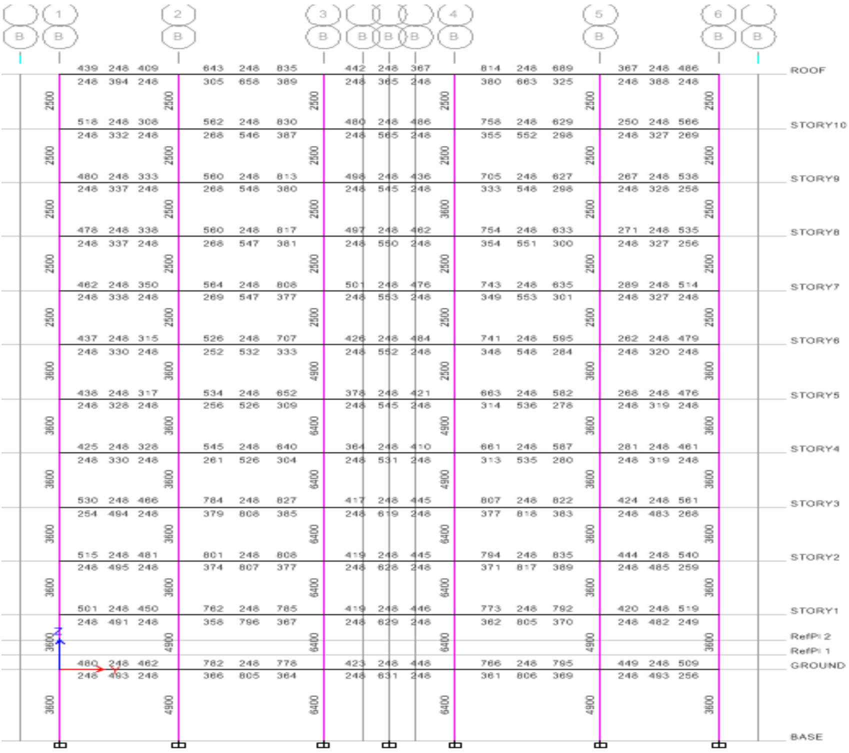
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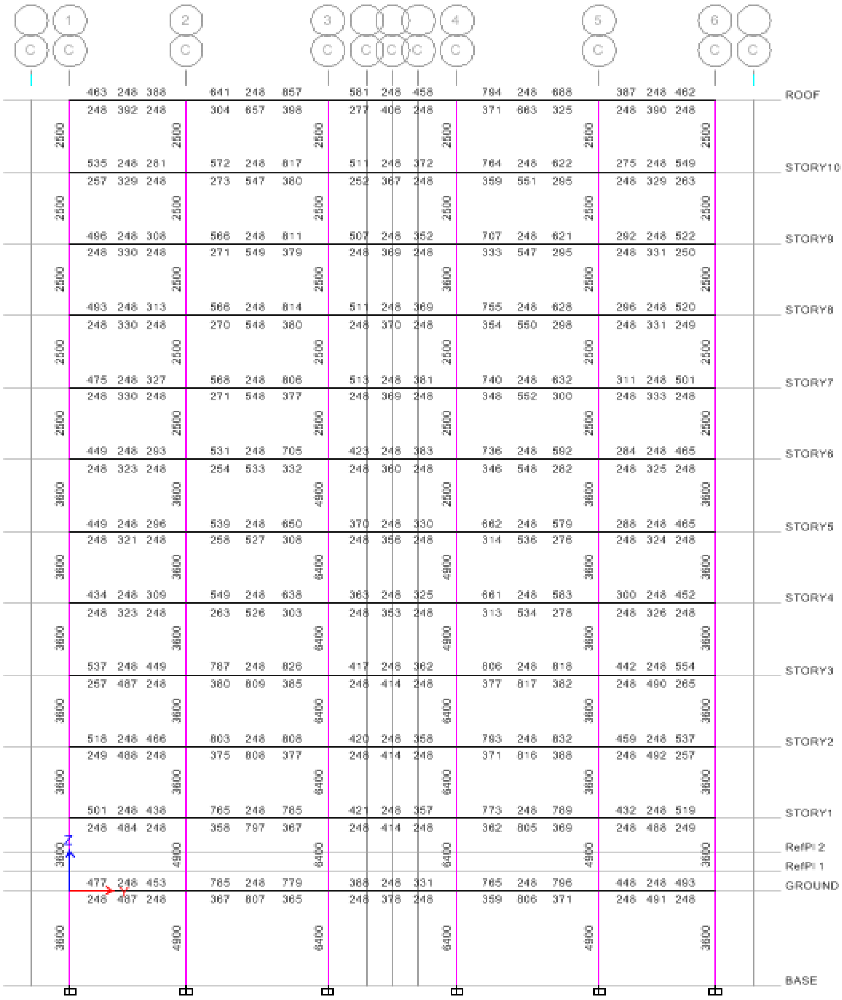
RefPl 1
GROUND
BASE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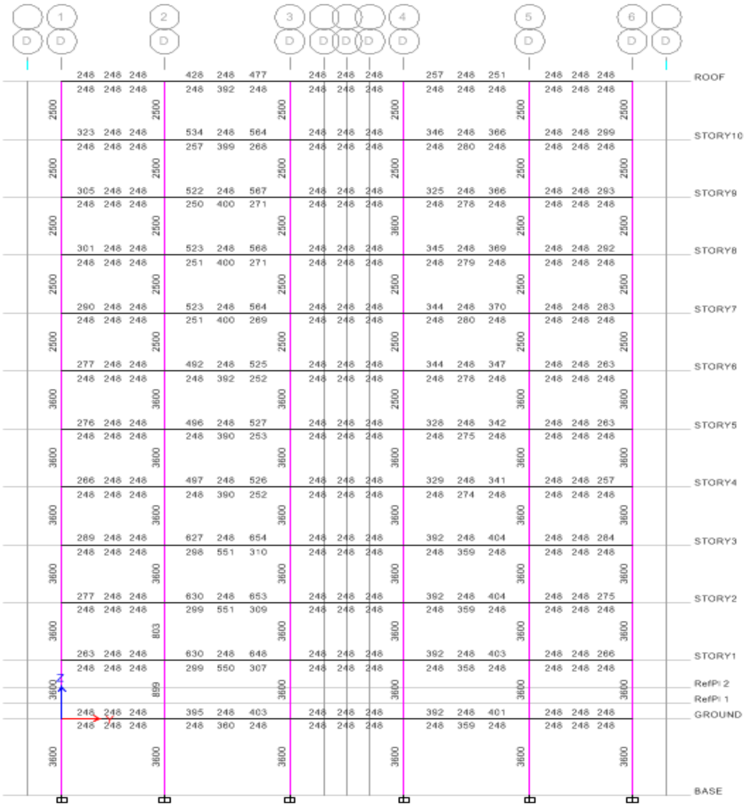
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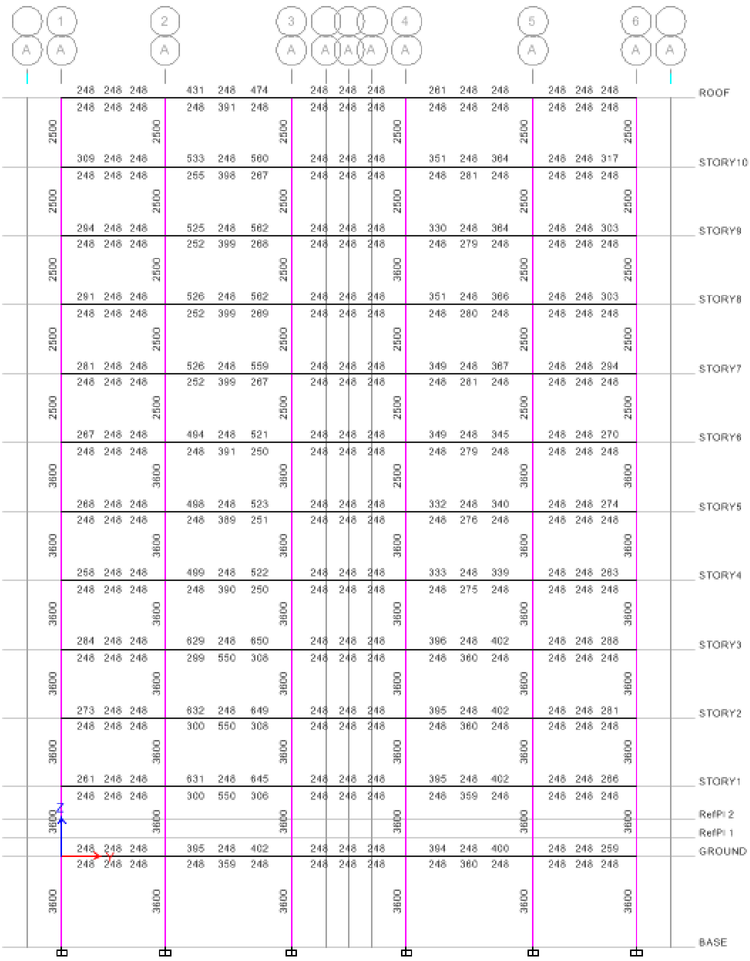
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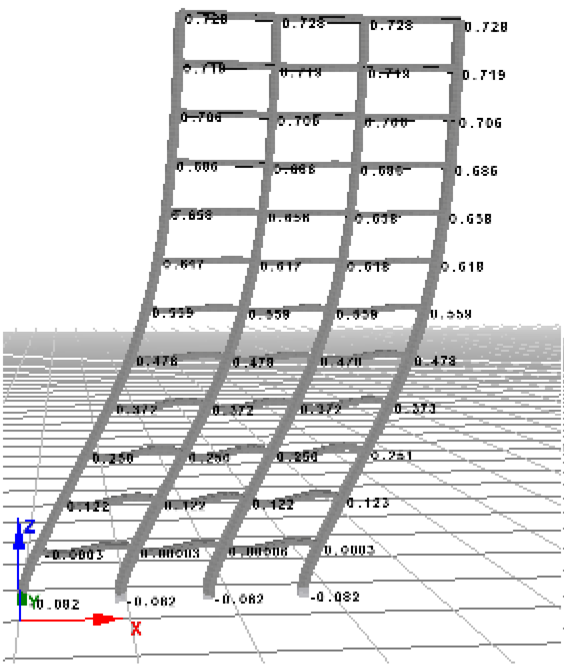
STORY3
STORY2
STORY1
RefPl 2
RefPl 1
GROUND
BASE

APPENDIX-C

DEFORMED SHAPE OF PUSH OVER ANALYSIS RESULT



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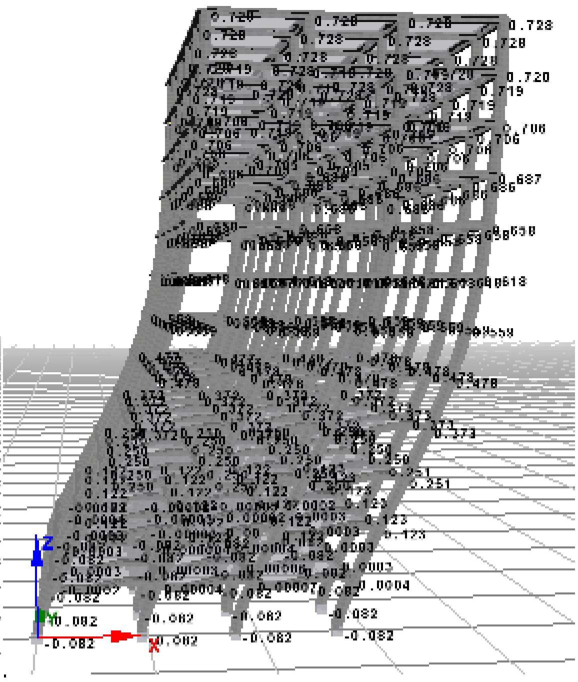

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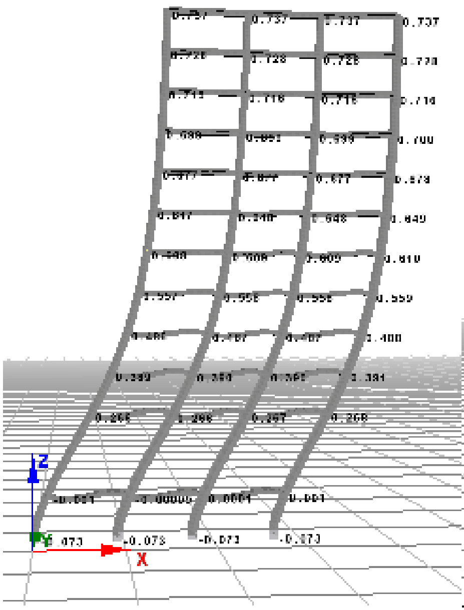

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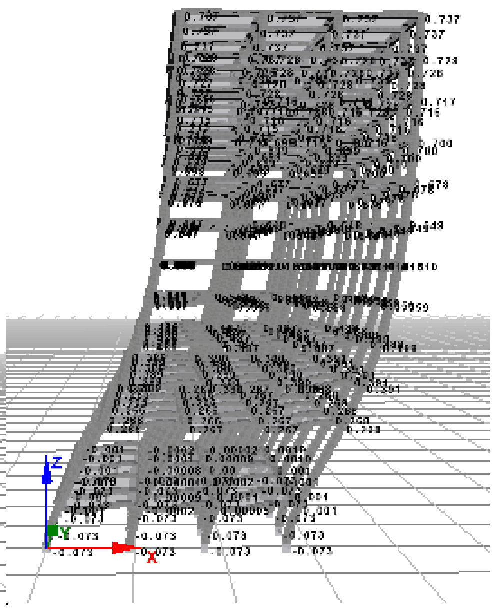

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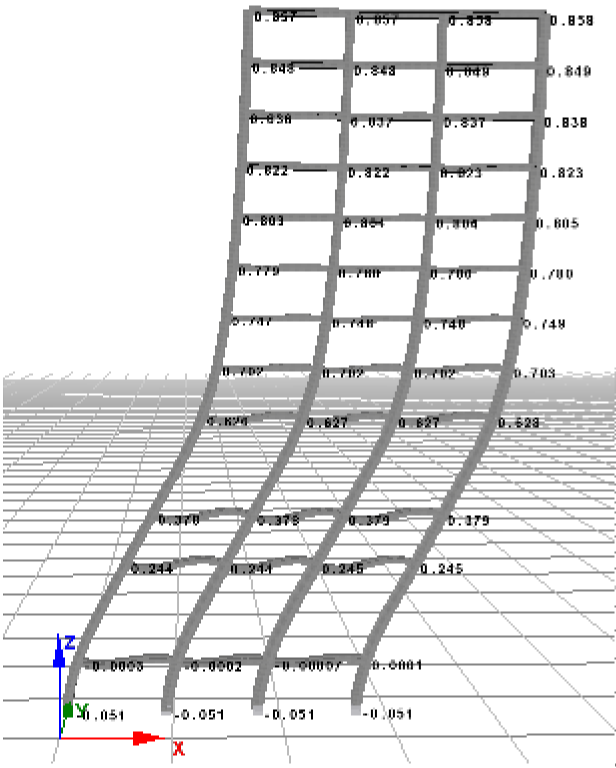

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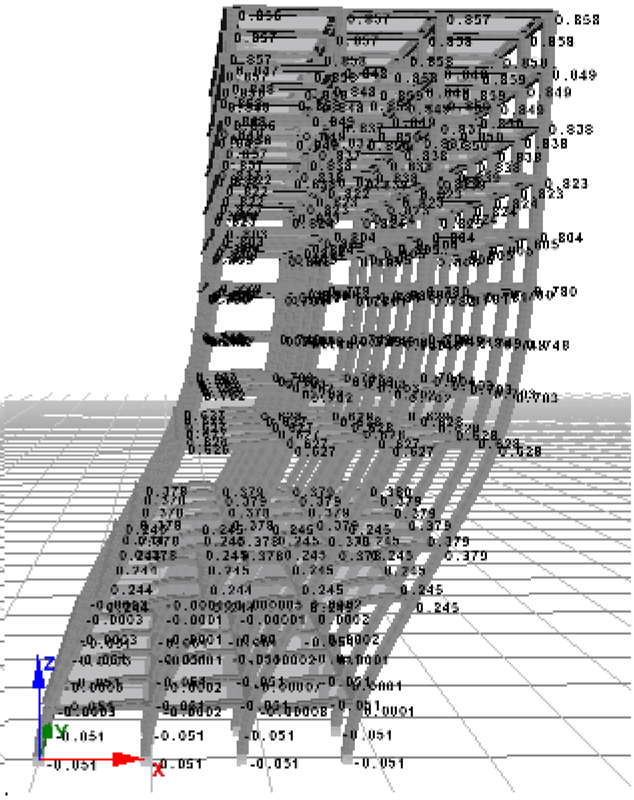

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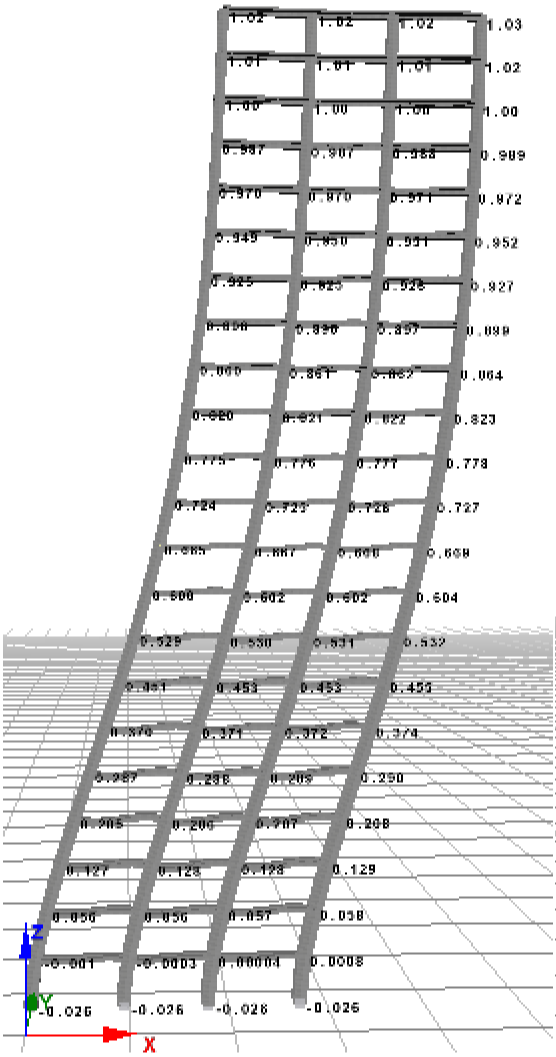

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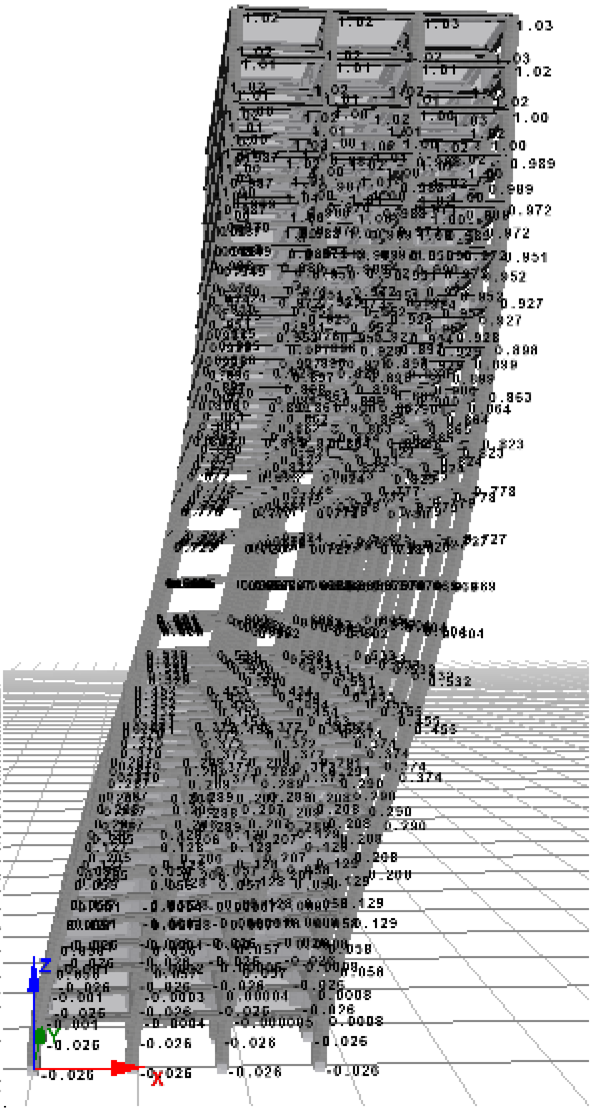

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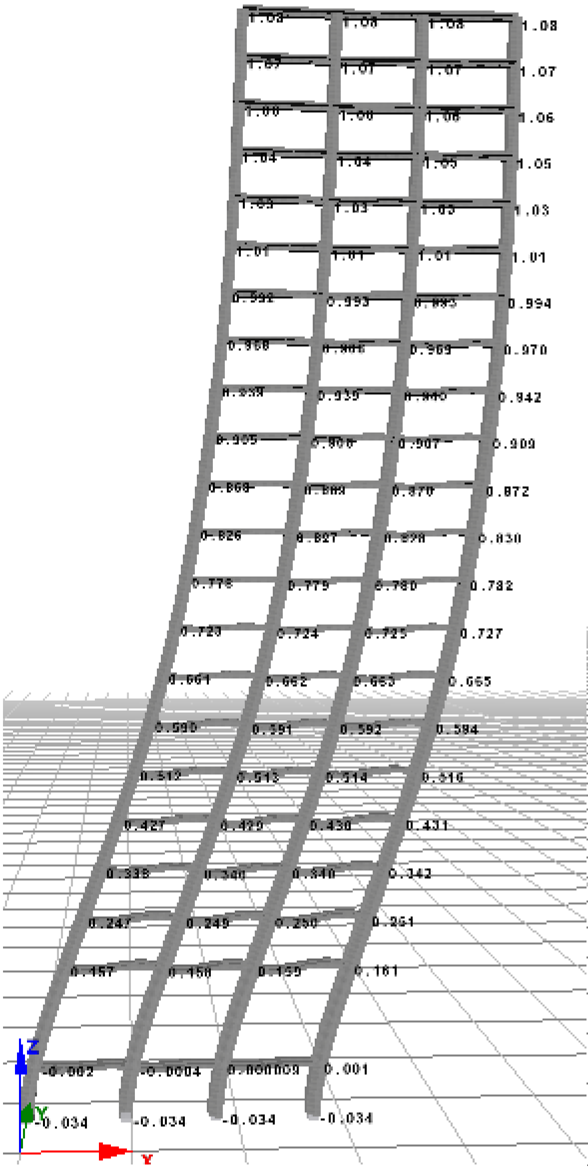

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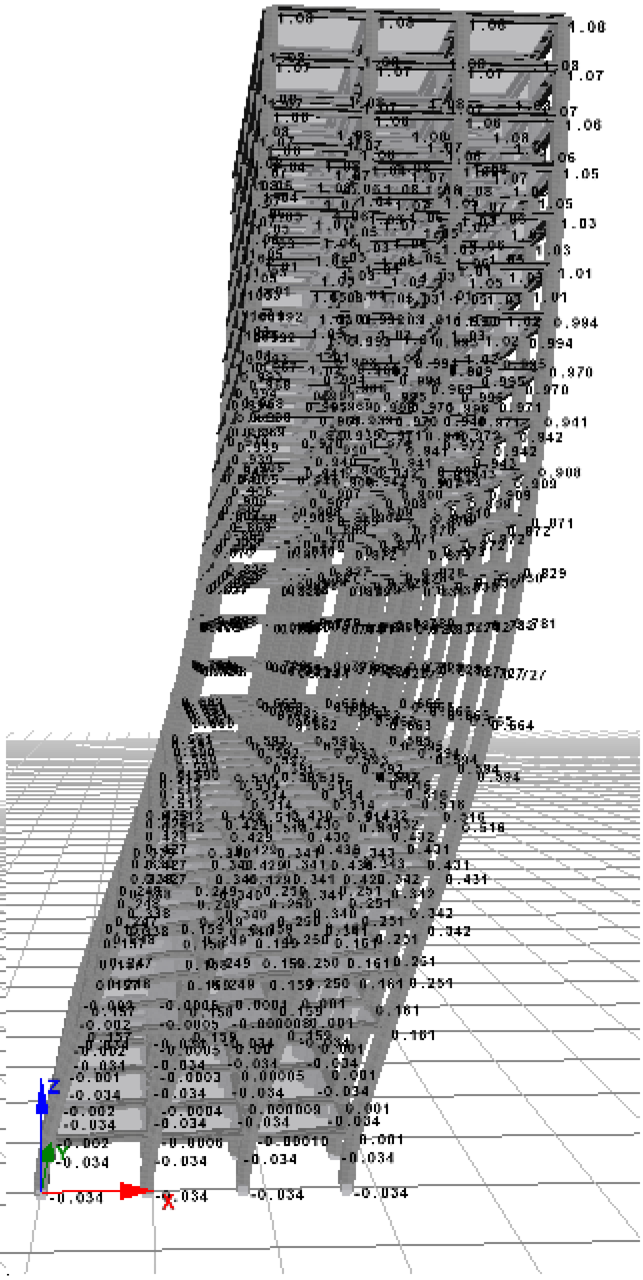

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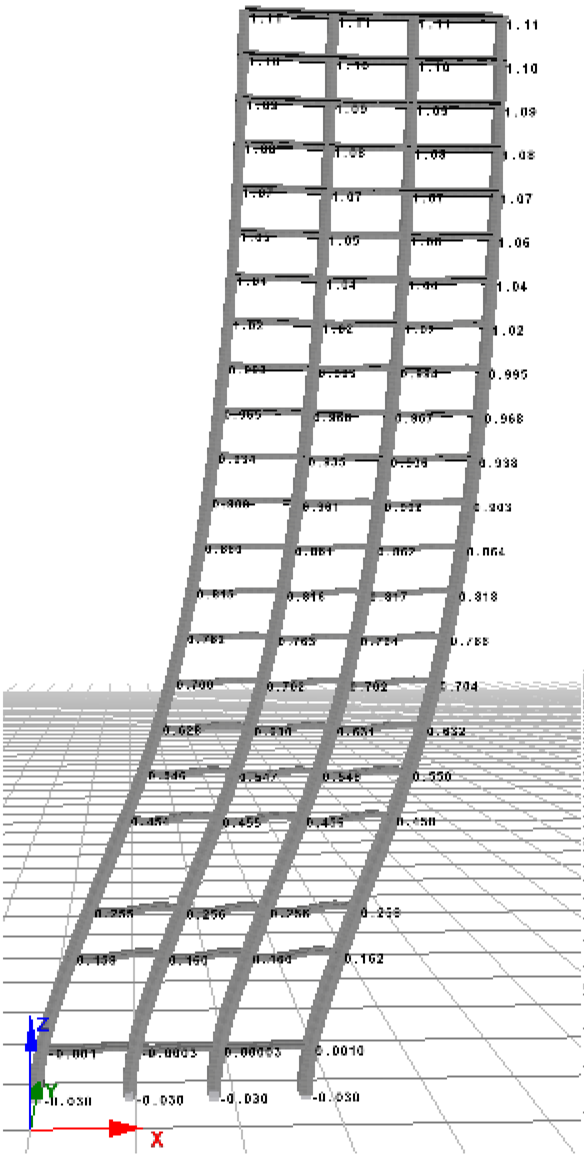

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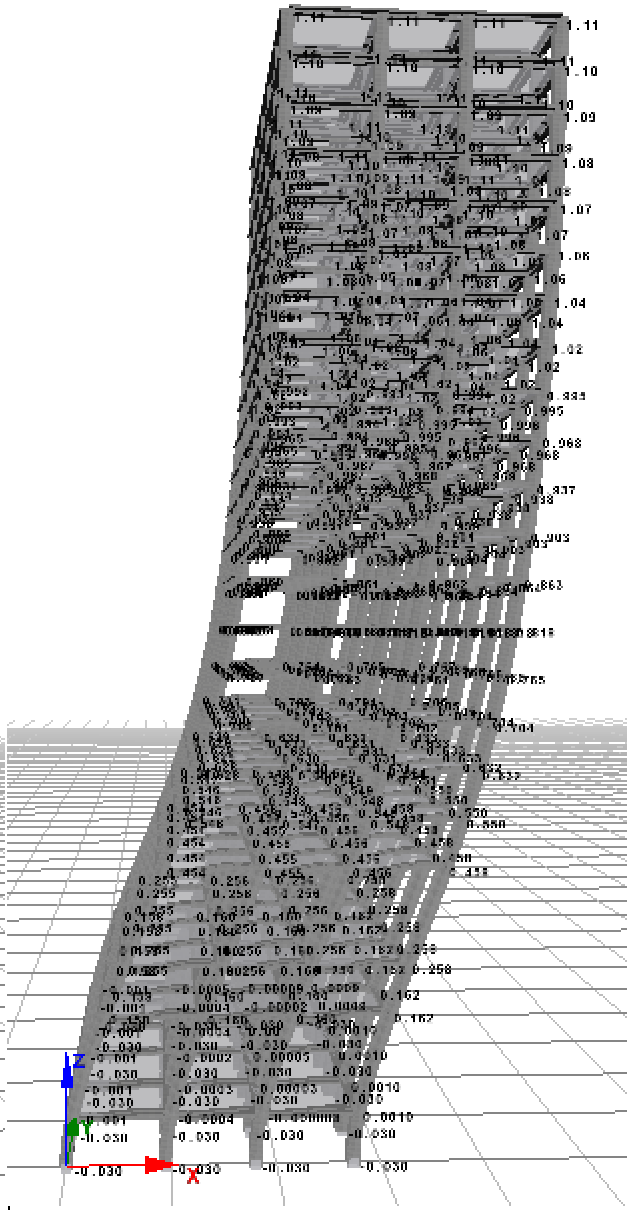

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