ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATION OF THE CRITICAL ANGLE OF SEISMIC INCIDENCE FOR THE SEISMIC SAFETY ASSESSMENT OF ASYMMETRIC REINFORCED CONCRETE BUILDINGS WITH RE-ENTRANT CORNER PLAN CONFIGURATIONS

A thesis submitted to the Graduate Studies of Addis Ababa University in Partial fulfillment of the Requirements for the Degree of Master of Science (MSc) in Structural Engineering.

BY

HAMZA AHMEDNUR HASSEN

ADVISOR: Dr. SHIFFERAW TAYE

NOVEMBER, 2018
ADDIS ABABA, ETHIOPIA

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

INVESTIGATION OF THE CRITICAL ANGLE OF SEISMIC INCIDENCE FOR THE SEISMIC SAFETY ASSESSMENT OF ASYMMETRIC REINFORCED CONCRETE BUILDINGS WITH RE-ENTRANT CORNER PLAN CONFIGURATIONS

BY

HAMZA AHMEDNUR

Approved by Board of Examiners

Dr. Shifferaw Taye	____________________	____________
Advisor	Signature	Date
Dr.-Ing. Adil Zekaria	____________________	____________
External Examiner	Signature	Date
Dr. Esayas Gebreyohannes	____________________	____________
Internal Examiner	Signature	Date
Dr. Agizew Nigussie	____________________	____________
Chair person	Signature	Date

DECLARATION

I, the undersigned, declare that this thesis is my own original work and all sources of materials used for the thesis have been duly acknowledged/ referred.

Name: Hamza Ahmednur Hassen

Signature: -----------------------

Place: Addis Ababa University

Addis Ababa Institute of Technology

ABSTRACT

In this thesis, the methodology to estimate the possible critical angle of seismic incidence and its corresponding structural demand has been investigated in the context of LTHA, to assess the seismic behavior of asymmetric reinforced concrete buildings with re-entrant corner plan configurations under different orientations of seismic excitation. This was done by defining the structural demand as the function of input seismic incidence angle (α) with in the range (0^0, 180^0) at 15^0 increment, in order to assess the seismic incidence angle that leads to the maximum global and local structural demand of reinforced concrete buildings under different orientations of seismic excitation. To do so, different 3D computer model of single and four-story symmetric (torsionally stiff) and asymmetric (torsionally flexible) reinforced concrete buildings has been generated with the aspect of changing plan configuration. Finite element based linear time history dynamic analysis were carried out to assess seismic response of the structural model in all cases with the help of elastic demands (maximum story drift, and maximum column axial force, torsion & moment), by using ETABS.V16 software.

It is observed from the study that, the structural demand produced varies as a function of input seismic incidence angle and the highest demand occurs in the most flexible structural direction. The structure gets its maximum value of column axial forces, torsion and moment with a specific angle of seismic excitation which is different for different shapes of building as well as from column to column. It is concluded from the study that the angle of seismic incidence input motion considerably influences the response of RC structures. Additionally, considering ASIcri values of all models, asymmetric buildings with re-entrant plan have shown considerable change in maximum column axial forces and moment due to the change in building plan configurations & column locations. But, among various internal forces irregular buildings with re-entrant corner plans are more sensitive to the maximum column torsion when compared with corresponding regular building.

KEYWORDS: Seismic incidence angle, re-entrant corner plan, linear response history analysis

ACKNOWLEDGMENTS

First and foremost, I would like to thank the Almighty God, Who gave me the tolerance and strength to pass various difficulties and come up to the accomplishment of this thesis. There is no doubt that a single step further is unthinkable without his involvement.

Next, I would like to express the deepest and unlimited appreciation to my advisor Dr.Shifferaw Taye, for his innovative ideas, knowledge, guidance and encouragement to the highest standards inspired and motivated me from the early beginning to end. Really, I consider myself very lucky for being able to work with a very considerate and encouraging doctor like him. Without his fruitful guidance and encouragement this thesis would not have been realized.

Besides, I would like to thank Ethiopian Road Authority, Addis Ababa institute of Technology and Haramaya University which sponsored me to have this great opportunity. And I would like to express my deepest gratitude to my lovely wife, parents and brother engineer Abulnasir mohammed for their prayers and critical supports.

Finally, I would like to put on record my gratitude and appreciation to all my instructors, classmates and all individuals who contributed directly or indirectly to this thesis and provided the necessary materials and moral support.

TABLE OF CONTENTS

LIST OF TABLES

.

LIST OF FIGURES

LIST OF SYMBOLS AND NOTATIONS

α, ASI	Angle of seismic incidence
α crit, ASIcri	Critical angle of seismic incidence
β	Percentile used for the percentage combination rule, 30 or 40%
3D	Three Dimensional
EDPs	Engineering demand parameters
ES	Ethiopian standards
EQ	Earth quake
FN/FP	Force to the normal/parallel directions
Ag	Acceleration due to gravity
I	important factor
LTHA	Linear time history analysis
NLTHA (NLRHA)	Nonlinear time history analysis
S	Soil factor
Sd	Design spectrum (for elastic analysis).
T	Fundamental period of the building
Z	Seismic zone factor

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Earthquake is one of the most unpredictable and disturbing natural disasters not only due to its magnitude but also its direction of occurrence. Although the occurrence of earthquakes cannot be predicted and prevented, the structures should be designed to resist earthquake motions equally from all possible directions. Indeed, this is one of the greatest challenges for Structural Engineers in today’s scenario. Most of the earthquake resistant design of buildings comprises the implementation of an analysis procedure adopted by seismic codes. According to the code provisions and regardless of the chosen analysis procedure, seismic analysis of structures is usually performed by applying horizontal design seismic loading representing the earthquake actions along the certain orthogonal directions of the structure. E.g. ASCE/SEI41-06 (section 3.2.7) (American Society of Civil Engineers,2008) specifies that two orthogonal axes X and Y shall be established, while EC-8 (European Committee for Standardization,2004) or ES-8 EN-1998:2015 mentions the application of the seismic loads in all relevant horizontal directions (with regard to the structural layout of the building) and their orthogonal horizontal axes.

It is really known that, in most tectonic regions around the world the earthquake ground motion can act along any horizontal direction, this implies the existence of possible different direction of seismic incidence that would leads to an increase of structural response so called critical angle of seismic incidence. Thus, due to uncertainty of the several epicenter locations at a site, it is reasonable as the structural analysis and design standard to consider all possible values of the seismic incidence angle α in the range (0^0, 360^0) relative to the structure which needs to be analyzed to assess maximum structural demand [18]. Applying the main seismic component along a direction different from the principal structural axes may lead to higher demand on the structure and Cause unexpected collapse. More specifically, for spatial structural model or asymmetric complex multistory reinforced concrete building either in plan, in elevation or both, the analysis and design outcomes may substantially unsafe if the ASIcrit is not taken into account. Since the main direction of earthquake action and principle axis of structure are a prior unknown and may not be identical, several incidence angles should be considered in order to assess the maximum structural demand. Therefore, Seismic incidence angle is one of the basic structural analysis parameter that affects the potential seismic behavior of reinforced concrete structures where earthquake is prevalent.

In light of this, many of the past research indicated that the different engineering demand parameters (EDP) reach their maximum value at different angles of seismic incidence which may not coincide with the main structural axes, even for structures that possess two main orthogonal structural axes (e.g. see Athanatopoulou 2005; Morfidis, Athanatopoulou and etal 2008; Magliulo G, Maddaloni G and etal 2014). So, it is difficult to determine a ***"unique"*** angle of incidence for a given structure that maximizes demands for all engineering demand parameters (EDPs) simultaneously.

Therefore, linear time history analysis has been used here as one of engineering approach systems to investigate ASIcrit and its corresponding structural demands, in order to assess the seismic behaviors of asymmetric reinforced concrete buildings with reentrant corner plan configurations under different orientations of seismic excitation. This has been done by defining the structural demand as the function of input seismic incidence angle α in the range [0^0, 180^0] at 15^0 increments. The building models have been analyzed using finite element software's, ETABs.V16.

1.2 Statement of the Problem

ES 8 EN 1998-1-1:2015 states in §4.3.3.1(11) P: "Whenever a spatial structural model is used, the design seismic action shall be applied along all relevant horizontal directions and their orthogonal horizontal axes." For buildings with resisting elements in two perpendicular directions these two directions shall be considered as the relevant directions. Of course, due to uncertainty of the several epicenter locations at a site, it is reasonable as a design standard to consider all possible values of the seismic angle α in the range (0^0, 360^0) relative to the structure, in order to assess the maximum structural demand. Specifically, for spatial structural model or asymmetric reinforced concrete (RC) building with different typologies in plan, or in elevation, or both. [E.g. reentrant corner plan irregularity configuration], the analysis and design outcomes may substantially unsafe if the ASIcrit is not taken into account. Since the main direction of earthquake and principle axes of structure are a prior unknown and may not be identical, the determination of the seismic orientation that leads to the highest demand (ASIcrit) is a complex issue.

Thus, Linear Time History Analysis (LTHA) has been used here as one of engineering approach systems to investigate ASIcrit and its corresponding structural demands of reinforced concrete structures under different orientations of seismic excitation.

1.3 Significance of the study

Seismic incidence angle is one of the basic structural analysis parameter that affects the potential seismic demand of reinforced concrete structures where earthquake is prevalent. Additionally, the building with re-entrant corner plan irregularities configurations more vulnerable to earthquake damage, because earthquakes are likely to find the weakest link in any complex system and cause damage to the most vulnerable element.

Thus, investigating the methodology to estimate the possible critical angle of seismic incidence and its corresponding structural demand is vital task, to assess seismic behavior of reinforced concrete buildings under different orientations of seismic excitation, or to reduce the potential seismic damages. To do so, finite element based linear time history dynamic analysis has been conducted by considering different models of single and multi-story symmetric and asymmetric RC buildings with various re-entrant corner plan configurations with the aspect of changing plan configuration, under different orientations of seismic excitation, using commercial software ETABS.V16.

These can develop a better insight and awareness about the methodology to estimate the possible ASIcrit and its corresponding structural demand in context of LTHA, to assess seismic behaviors of reinforced concrete buildings under different orientations of seismic excitation on one side and the influence seismic incidence angle and change in plan configurations on the seismic behavior reinforced concrete buildings, especially with asymmetric reentrant corner plan configurations on the other side.

1.4 Objective of the study

1.4.1 General objective

To investigate the ASIcrit and its corresponding structural demand in context of LTHA, in order to assess the seismic behaviors of asymmetric reinforced concrete buildings with re-entrant corner plan configurations.

1.4.2 Specific Objective

- To assess the seismic behavior of reinforced concrete buildings under the different orientations of seismic excitation (i.e.0^0 to 180^0 @ 15^0 increments).
- To compare the analysis results with the help of elastic structural demand parameters like; maximum story drifts, column axial force, torsion and moment.
- To assess the influence seismic incidence angle (ASI) and change in plan configurations on the potential seismic behavior of asymmetric reinforced concrete buildings with re-entrant corner plan configurations.
- To create a better insight and awareness regarding the influence of ASI on the potential seismic performance of asymmetric reinforced concrete buildings.

1.5 Methodology

To achieve the objectives of the study, deep investigation has been carried out in accordance with the methodology outlined below.

- Literature has been reviewed***:*** regarding the topic specified.
- The material, details demission of building plan, structural systems and all other essential parameter has been selected.
- Different models of single and multi-story symmetric and asymmetric plan buildings were generated with the aspect of changing plan configuration.
- Dynamic analysis has been done; using finite element commercial software (ETABS .V16) with LTHA method.
- Result and discussion has been made.
- Finally the conclusion and recommendations has been made up on finding.

1.6 Scope of the study

In this study, only Linear Time History Analyses (LRHAs) were conducted to investigate the possible critical angle of seismic incidence and its corresponding structural demand, to assess seismic behaviors of single and multi-story [4 stories] reinforced concrete buildings with symmetric and asymmetric re-entrant corner plan configurations under different orientations of seismic excitation in the range [0^0, 180^0] with an increment of 15 degrees. The effect of soil structure interaction is ignored in analysis. The columns are assumed to be fixed at the ground level. The effects of secondary structural components and nonstructural components are assumed to be negligible; these include staircases and partitions. The expected building is located in high seismic region (zone IV), as per ES 8 EN 1998:2015 seismic designs manual.

1.7 Organization of the thesis

The thesis is organized in different chapters which are arranged as follows:

Chapter 1: Deals with introductory parts which include background of the study, statement of the problem, significance of the thesis, objective and scope of the study and organization of the thesis.

Chapter 2: The literature reviews i.e. the review of the previous research regarding the topic specified.

Chapter 3: Discusses about the methods of analysis.

Chapter 4: Tells about the modeling of structural systems.

Chapter 5: Presents the result and discussion of the analyses results.

Chapter 6: Contains the conclusions of the study, and the recommendations for future studies.

[This page intentionally left blank]

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction to seismic excitation Angle

During seismic analysis and design of structures, the earthquake motions should be considered in principal directions of structure. But the main direction of earthquake and principal axes of structure is not identical and the response of structure will change with variation of earthquake excitation angle. The directions of application of seismic excitations (forces) used in the analysis and design shall be those which will produce the most critical load effects. To satisfy this requirement most of seismic resistance design codes of practice use different techniques or combination rules (like; Uni-directional or Bi-directional earthquake excitation technique) to estimate simultaneous effects of two horizontal components of earthquake excitations.

I. Uni-directional earthquake excitation is means of considering earthquake action separately in the two orthogonal directions of the building and then combine them using superposition by using (SRSS and 100/β , with β = 30, 40, 60) to estimate the Bi-directional response from the uni-directional response. The 30% combination rule (100/30) is commonly used in seismic resistance design code of practice, meaning simultaneous effects of two horizontal components of earthquake excitations is taken into account by applying 100% of the force resulting from a uni-directional earthquake lateral action in the direction of one of the building main axes and 30% of those forces in other orthogonal direction of the main axis. This is while in reality the direction of the dominant component of excitations might not be one of the main directions of the building axes, and applying the main component in a direction other than main axes direction may lead to higher internal forces and stresses in the building’s structural elements.

II. Bi-directional earthquake excitation is the means of applying the two orthogonal horizontal components earthquake actions simultaneously (Bi-directionally) at the same time with/without several angles of incidence which called as **"*exact response*.”**

The simultaneous application of two horizontal seismic components is very important for an accurate determination of the response, mainly for uncertain relations of the lateral stiffness in the two orthogonal directions of the plan. It is recommended that seismic codes impose the simultaneous application of the two horizontal components of seismic motion, in order to reach an adequate design for serviceability earthquakes and avoiding premature incursions in the inelastic range. It is known that some conditions which increase the elastic response can as well increase the inelastic one. But, the one that implicates the maximum response considerate like "exact" should be evaluated more preferably through a nonlinear time history analysis not could be esteemed for a spectral modal superposition dynamic analysis (Julio J Hernandez and Oscar A Lopez, 2000).

In most tectonic regions around the world the earthquake ground motion can acts along any horizontal directions and the major principal axis at a site can be directed to several epicenter locations, this implies the existence of possible different direction of seismic incidence that would leads to an increase of structural response so called critical angle of seismic incidence. Due to uncertainty of the next earthquake epicenter locations at site, it is reasonable as structural analysis and design criterion to consider all possible values of the angle α in the range (0^0, 360^0) in order to asses maximum structural demands. Thus, seismic incidence angle is one of the basic structural analysis and design parameter that affects the potential seismic behavior of reinforced concrete structures.

It has been stressed by many researchers (e.g. Lopez and Torres, 1997; Athanatopoulou, 2005; Morfidis, Athanatopoulou and etal, 2008; Cantagallo and Spacone, 2012; Magliulo.G, Maddaloni.G and etal, 2014; Quadri and Madhuri, 2014; M.sri kanya and Chandra Mohan, 2015; Ramchandani and Mangulkar, 2016; Kostinakis, Athanatopoulou and etal, 2017) that, in general, the different engineering demand parameters (EDP) of structure reach their maximum value at different random angles of seismic incidence so called critical seismic orientation which may not coincide with the main structural axes, even for structures that possess two main orthogonal structural axes as some codes specify.

Lopez and Torres (1997) determined the critical incident angle and the associated maximum structural response by means of the response spectrum method, for the general case of three ground motion components that may have either different or identical spectral shapes.

Athanatopoulou (2005) developed analytical formulae for the determination of the critical angle of seismic incidence and the corresponding maximum value of any response quantity under three seismic components within the context of linear response history analysis. The analytical formulae require the results of three specific time history loading cases: two for the horizontal components and one for the vertical one. The suggested formulae have been verified through extensive parametric studies (Athanatopoulou, 2005; Athanatopoulou et al., 2005; Athanatopoulou and Avramidis, 2006; Kostinakis et al., 2008) which have shown that the same earthquake records have different critical angles for different response quantities, while different earthquake records have different critical angles for the same response quantity. Also, it has been indicated that the maximum value of a response quantity corresponding to the critical orientation of the horizontal ground motion components can be up to 80% larger than the value obtained by the application of the seismic excitation along the structural axes. Finally, it has been demonstrated that the variation of a response quantity in the interval (0^0-180^0) of the seismic incident angle can be up to 80%. Similar conclusions have been drawn by Rigato and Medina (2007) who studied the influence of the angle of incidence on the seismic demands of single-story inelastic buildings subjected to bi-directional excitations. In particular, they concluded that the maximum response (column displacement ductility ratios, slab rotations and drifts) obtained by nonlinear dynamic analyses conducted for different incident angles (varying from 0^0 to 180^0 at increasing 5^0 increments relative to the X structural axis) can be on average up to 60% larger than those obtained by taking into account only one orientation of the seismic components (along the structural axes). This value varies with the degree of inelasticity and the fundamental period of vibration.

C. Cantagallo, G. Camata & etal. (2012), studied the effect of the seismic incidence angle on the seismic response of RC buildings, by considering four different model of three-dimensional reinforced concrete structures under different orientation bi-directional ground motion (both scaled and un-scaled), using NLTHA. Each ground motion record is applied along nine different incidence angles, ranging between 0 and 180 degrees, with 22.5 degree increments. Finally the principal results presented in their study can be summarized as follows: 1**.** the structural demand on a doubly-symmetric single-story reinforced concrete structure does not vary significantly as a function of the incidence angle. Conversely, MIDRs for plan-irregular reinforced concrete structures vary considerably depending on the incidence angle. This behavior is due to the fact that plan-irregular buildings tend to have significantly different stiffness and capacity in different directions. The applied ground motion generates the highest demand when applied in the most flexible direction. 2. The ground motion records generate the highest demand when applied in the most flexible structural direction and a high energy content of the records increases the structural demand corresponding to this direction. Therefore, the influence of the incidence angle on the seismic demand varies depending on both structural configuration and specific characteristics of each examined record.

Morfidis, K. E., A. M. Athanatopoulou and etal (2008), studied the effects of seismic directivity on the longitudinal steel reinforcement of R/C buildings within the framework of the lateral force procedure. It is proved that, in general, different orientations of the seismic action may lead to differences of up to 29% concerning the required longitudinal steel reinforcement. The only case in which the structural response does not depend on the loading direction is that of structures which fulfill the criterion TB<Tα<TC for any value of α, where α is the angle of incidence, Tα is the uncoupled fundamental period of the structure along the α-direction and TB, TC are the lower and upper limits of the constant spectral acceleration branch.

Despoina skoulidou and Xavier Roma (2016), develops an analytical methodology to determine the critical angle of seismic incidence in the context of LFA. The proposed expression was defined based on the geometrical and mechanical characteristics of the structure. The demand parameter under consideration was the maximum total displacement of single story and isotropic multi-story buildings, while the maximum inter-story drift may also be employed. The validity of the developed framework was demonstrated by two case studies: a single story building and a 3-storey isotropic building. Although in both case studies the parameter under consideration was the maximum total displacement, the methodology and the results can be extended for inter-story drifts. Finally he concludes, although the presented framework introduces a direct methodology to determine the ASIcrit, its applicability is still bounded by the requirement for the existence of a real elastic axis. Despoina Skoulidou and Xavier Romao (2017), propose the analytical methodology enabling the determination of the critical angle of seismic incidence for the story displacements and the inter story drifts of buildings that conforms to standard-based provisions for linear static analysis. The applicability and accuracy of the methodology was illustrated for three buildings comprising different typologies in plan and in elevation, by performing a parametric LFA of the buildings for different orientations of the bidirectional seismic action.

Quadri. S. A. and M. Madhurin (2014) studied the critical angle of seismic incidence for the analysis of RCC frames in the context of LFA. In their investigation 4-story reinforced concrete building with moment resisting frame have been analyzed by the equivalent static analysis method. A set of values from 0 to 90 degrees with an increment of 10 degrees, have been used for angle of excitation of seismic force. Buildings' columns have been divided into three main categories, including corner, side, and internal columns and column forces have been investigated in all cases. The result shows that column forces exceeds by varying the angle of excitation of seismic force and the value of axial forces may exceed the ordinary cases up to 13%. The structure gets its maximum value of column forces with a specific angle of excitation of seismic force which is different from column to column.

Magliulo.G, Maddaloni.G,and etal (2014), studied the influence of earthquake direction on the seismic response of irregular plan RC frame buildings. Three multi-story RC buildings (rectangular shaped plan, the L-shaped plan and a rectangular shaped plan with courtyard), representing a very common structural typology in Italy, were assumed in the paper as case-studies for the evaluation. Nonlinear static and dynamic analyses were performed considering different seismic levels, characterized by peak ground acceleration on stiff soil equal to 0.35g, 0.25g and 0.15g as per EC-8. Nonlinear dynamic analyses are carried out considering twelve different earthquake directions, rotating the direction of both the orthogonal components by 30° for each analysis (from 0° to 330°). The survey is carried out on the L-shaped plan structure. The results show that the angle of the seismic input motion significantly influences the response of RC structures: the critical seismic angle, i.e. the incidence angle that produces the maximum demand, provides an increase up to 37% both in roof displacements and in terms of plastic hinge rotations.

Konstantinos G. Kostinakis, Grigorios E. Manoukas, and et al. (2017), investigated the influence of the seismic incident angle on the response values of symmetric in plan of RC buildings, by considering two model of two story three-dimensional reinforced concrete buildings (referred to as structural model A with equal stiffness and structural model B with different stiffness along the two structural axes), under different orientation of bi-directional horizontal ground motion using both linear and non-linear response history analysis. Linear response history analysis is performed using the program SAP2000, while nonlinear response history analysis is performed using the program RUAUMOKO (Carr, 2004). Firstly, a set of symmetric buildings was studied by means of linear response history analysis. The maximum response values of overall incident angles were determined using well established analytical formulae. It is demonstrated that for symmetric buildings possessing equal stiffness along two orthogonal horizontal axes the maximum value of some resultant response quantities (resultant displacements, resultant moments of some columns) does not depend on the orientation of the seismic action. On the contrary, the seismic incident angle was essential for the rest response quantities of such buildings, as well as for all the response quantities of symmetric buildings with different stiffness along the two structural axes. In addition, the same buildings were analyzed by non-linear response history analysis, with seismic components having several different orientations with regard to the structural axes. Similar conclusions are derived for the nonlinear range of behavior too.

Ramchandani Jaya and Mangulkar Madhuri (2016), studied the critical angle of seismic incidence for the analysis of reinforced concrete building with moment resisting frame, of different shapes i.e., square, rectangular, L-shaped, T-shaped and irregular structures, subjected to the same ground motion in different orientation whose values ranges between 0 to 90 degrees, with an increment of 10 degrees by using time history dynamic analysis of earthquake. The analysis has been carried out by using STAAD PROS. Buildings' columns have been divided into three main categories, including corner, side & internal columns and then the column forces(i.e. shear force and bending moment) have been investigated in all cases. Finally they develop the following conclusions; the structure gets its maximum value of column forces with a specific angle of seismic excitation which is different from column to column as well as for different shapes of building. The variation in percentages of bending moment is different not only for different structures but also for different types of columns such corner, side and middle. i.e. the variation in percentages column moments at critical angle with respect to 0 degrees. E.g.1. For square shape the variation percent of column moment (My) is 66.176%, 20.734% and 99.445% for corner, side and middle column respectively. 2. For corner column of Rectangular shape it is 29.691% whereas for side column it is 16.940% and for middle column it is 0.006413%. 3. For corner column of T shape it is 99.993% whereas for side column it is 27.499% and for middle column it is 76.239%. 4. For corner column of L shape it is 42.707% whereas for side column it is 57.257% and for middle column it is 62.906%. [5]. for corner column of Irregular shape it is 57.257% whereas for side column it is 3.863% and for middle column it is 61.462%.

M.sri kanya, & BDV.Chandra (2015), studied the influence of the earthquake direction on the seismic response of RC buildings, by considering three different model of three-dimensional reinforced concrete buildings [one symmetric and two asymmetric (Plus-shape, H-shape)] subjected to the same ground accelerations (Northridge earthquake record) in twelve different orientation whose values ranges between 0 to 180 degrees, with an increment of 15 degrees. The analysis has been carried out by using ETABS.V15 software with linear time history analysis (LTHA). The various response parameters studied are axial forces in the columns, maximum story displacement and story shear all varies as the function of incidence angle of the seismic input. Finally, they develop the following conclusions; 1. It is observed that regular and irregular buildings have shown considerable increase in maximum column forces when the peak ground acceleration is subjected at various incidence angles. There have been no considerable changes in maximum story displacement and maximum story shear. 2. The internal forces of structural elements are highly depends on the angle of incidence of seismic wave with respect to the axes of building plan. Among various internal forces the axial forces of columns are more sensitive to the angle of incidence. 3. For both regular and plus shape buildings the maximum column axial forces occurred when the peak ground acceleration is applied at 135 degrees, the value increased by 54% and 50% respectively with respect to 0 degrees. Whereas for H shaped building the maximum column axial forces occur at 45 degrees, the value increased by 45% with respect to 0 degrees. 5. It is observed that H shaped building is more vulnerable to earthquakes than the plus and regular shaped buildings under the same conditions. meaning, considering critical incidence angle values of all models, Plus shaped building has shown negligible variation in maximum column force compared to regular building whereas, H shaped building has shown 13.2% increase in maximum column force when compared to regular shaped building.

Fernandez Davila, Cominetti Silvana and etal (2000), examined different methodologies to consider the bi-directional seismic effects in the building design process, by considering the following combination rule:

1) Applying the two horizontal components of the earthquake action simultaneously (Bi-Directional) at the same time with several angles of incidence, which called as "exact *Response*."

2) Applying the higher component of the earthquake in each direction of the building independently.3) 30% combination rule (applying 100% of the force resulting from a uni-directional earthquake lateral action in the direction of one of the building main axes and 30% of those forces in other orthogonal direction of the main axis). i.e: $R = R^{0^\circ} + 0.3R^{90^\circ}$

4) The square root of the sum of the square (SRSS) combination rule [the forces resulting from the use of a uni-directional earthquake applied in both directions]

$$R(\text{SRSS}) = \sqrt{(R^{0^\circ})^2 + (R^{90^\circ})^2}\ .$$

5) 20% amplification of the maximum response obtained of the application of the higher component in the two principal directions of the building in independently (maximum force resulting from a uni-directional earthquake applied in the most unfavorable direction for the element) i.e.: $R_{20\%} = 1.2 \times Max\{R^{0^\circ}: R^{90^\circ}\}$

To do so, 6 models of 5story 3D RC buildings structured by frames and with a wall central double "T" representative of the nucleus of elevators which have been subjected to the uni and bi-directional seismic movement, with angles of incidence variables each 15º were studied. Considering that the axial effect is relevant in column elements under the occurrence of bi-directional seismic movement. Finally develop the following concussion: 1. the maximum response that is obtained in any structural element due to the application of a bi-directional seismic movement with angle of variable incidence not necessarily coincides with any of the two principal directions of the buildings. 2. It is observed that the 30% and square root of the sum of the square (SRSS) combination rule, underestimate the structural response by 25% with respect to the 'exact' response. 3. A rule that consists of amplifying in 20% the response that results higher upon only applying the higher horizontal component of the seismic in each one of the two principal directions of the building, meaning for the analyzed structures the building analysis by an uni-directional seismic input, amplified by a factor of 1.2 gives similar responses to those coming from bi-directional seismic analysis acting in the most unfavorable incidence angle. From this methodology the responses are overestimated and the maximum observed error is of 25%. 4. It is observed the response obtained by applying a bi-directional seismic movement simultaneously with several angles of incidence with respect building reference axis, which called as **"*exact response*”** is best combination rules.

2.2 Types of Irregularity

In addition to the magnitude and direction of earth quack actions; geometric configuration, type of structural members, materials and details of connections, all have a profound effect on the structural-dynamic response of a building [21]**.** ES 8 EN 1998-1:2015, article 4.2.3.1(1) P; for the purpose of seismic design, building structures are categorized into being regular or irregular. The irregularities are again divided into two broad categories: (I) vertical and (II) plan irregularities.

I. Vertical Irregularity: Vertical irregularities are defined in terms of strength, stiffness, geometry, and mass. This may include I. Stiffness Irregularity (Soft or Extreme Soft Story), II. Mass Irregularity (large changes in mass from floor to floor) III. Vertical

Geometric Irregularity, IV. In-plane discontinuity in vertical elements resisting lateral force. V. Discontinuity in capacity /strength (Weak story). Although all these are evaluated separately, they are related to one another, and may occur simultaneously. But Vertical Irregularity is not part of this thesis work.

II. Plan or Horizontal Irregularity: Plan irregularity generally of five types:

1. Re-entrant corners: Re-entrant corners plan configurations should be considered, if length in one direction > 25% or re-entrant area > 5% of the total plan area (ES 8 EN 1998:1-2015). A building having square, rectangular, or circular configuration with minor reentrant corners (i.e. < 5% of the total plan area) would be considered as regular, but large reentrant corners creating a crucifix form would be classified as an irregular configuration.

The response of the wings of this type of building is generally different from the response of the building as a whole, and this produces higher local forces than would be determined by analysis. Other a plan configurations such as L, H, T, E, I, C, Plus (+) shapes, that have a geometrical symmetry would also be classified as irregular because of the response of the wings (See Fig2.1). The most common problems for building plans with re-entrant corner irregularity configurations [like L–shape building] is that; The local stress concentrations, extra shear and torsion at the re-entrant corner which is due to sudden changes in stiffness or differential motions between different wings of the building. This may develop dis-continuity between structural components and inability of their parts to work together in resisting lateral forces or safely transfer of the seismic forces to the ground (See Fig2.2) [15].

2. Torsion irregularity: To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum story drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure.

3. Diaphragm discontinuity: Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than

50 percent from one story to the next. A building plan having square with core, rectangle with core that have significant differences in stiffness between portions of a diaphragm at a level are classified as plan irregularities since they may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular building.

4. Out of plane offsets: Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements. A building may have a symmetrical geometric shape without reentrant corners or wings but still be classified as irregular in plan because of the distribution of mass or vertical seismic force resisting elements. Where there are discontinuities in the path of lateral force resistance, the structure can no longer be considered “regular.” The most critical of discontinuities is the out-of-plane offset of vertical elements of the seismic force resisting elements.

5. Non-parallel systems: The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements. Thus, in case where vertical elements of the lateral force resisting system are not parallel to or symmetric about major orthogonal axes, the static lateral force procedures cannot be applied and, thus, the Structure must be considered to be “irregular” [21].

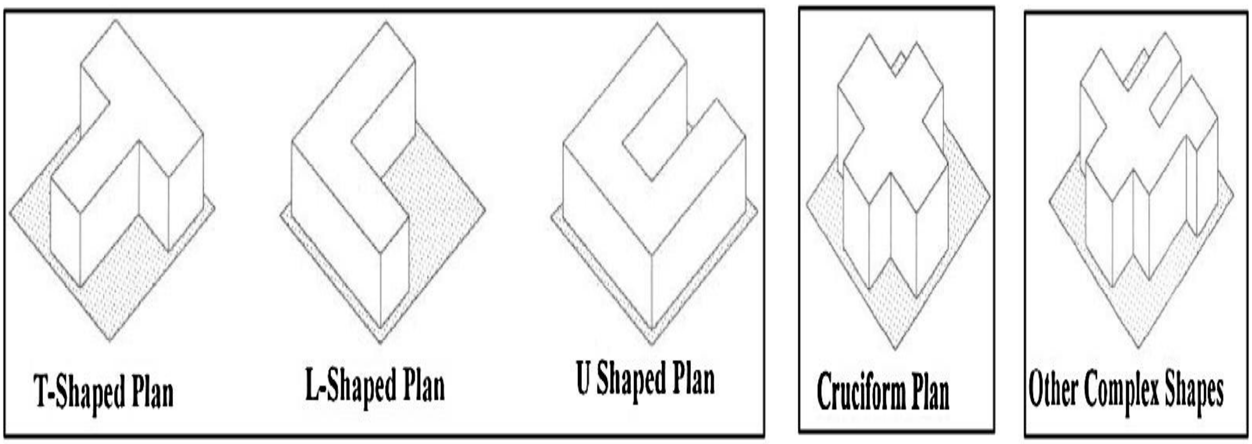


Fig2.1 building with re-entrant corners plans configuration

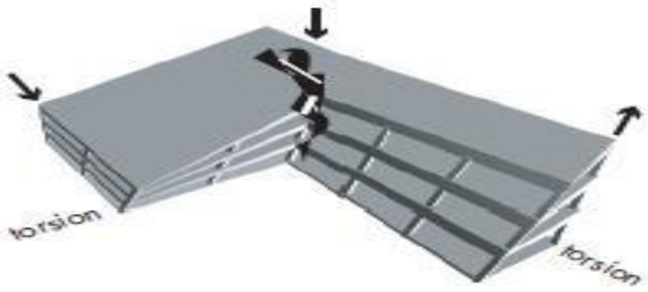


Fig2.2 Damage at re-entrant corner in L-shaped buildings

2.2.1 Criteria for regularity in plan as per ES EN 1998-1:2015 Code.

ES 8 EN 1998-1:2015 states in §4.2.3.2 (1) P "For a building to be categorized as being regular in plan, it shall satisfy all the conditions listed in the following paragraphs".

(I) With respect to the lateral stiffness and mass distribution: the building structure shall be symmetrical in distribution of mass and stiffness in plan with respect to two orthogonal axes.

(II) The plan configuration shall be compact: re-entrant area < 5% of the total plan area.

(III) The in-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements. In this respect, the L, C, H, I, and X plan shapes should be carefully examined, notably as concerns the stiffness of the lateral branches, which should be comparable to that of the central part, in order to satisfy the rigid diaphragm condition. The application of this paragraph should be considered for the global behavior of the building.

(IV) The ratio of longer side to shorter sides in plan does not exceed four.

Here in my study, Linear Time History Analyses were conducted to investigate the possible critical angle of seismic incidence and its corresponding structural demands, in order to assess the seismic behaviors of asymmetric reinforced concrete buildings with re-entrant corner plan configurations under different orientations of seismic excitation. Seismic incidence angle and re-entrant building plan configurations were considered as dependent (study) variables, while all other perimeters are considered as independent (constant) variables.

CHAPTER THREE

METHODS OF ANALYSIS

3.1 General

Nowadays, seismic analysis is the most powerful and advanced tool in earthquake engineering which is used to analyze the response a building or other structures under earthquake excitations in a simpler manner. During earthquake many buildings may collapse due to lack of understanding of the inelastic behavior of structure. Inelastic analysis gives capacity of the structure deforms to their inelastic or post yielding range during intense ground shaking, which is critical part of seismic analysis and design of any structures where earthquake is prevalent. While, elastic analysis gives only the elastic capacity of the structure that can indicates where the first yielding occurs. It cannot give any information about redistribution of forces and moments and failure mechanism [5]. But, in both case the structure should be designed well to resist earthquake motions equally from all possible directions. Thus, seismic analysis can be performed on the basis of external action, the behavior of structure, structural materials, and the type of structural model selected. Based on the type of external action it is classified as static and dynamic, while based on behavior of structure it is elastic and inelastic analysis. Generally, the modern seismic codes, like EC 8 CEN, 2004 (ES 8 EN 1998-1; 2015) allow four possible methodologies to analyses a mathematical model of a building or other structure:

1. Lateral Force /Equivalent Static Analysis (Linear Static)
2. Push Over Analysis (Nonlinear Static)
3. Modal Response Spectrum /Response Spectrum Analysis (Linear Dynamic)
4. Time History Analysis (Linear or Nonlinear Dynamic)

Their level of reliability decreases from 4 to 1 and, consequently, the safety margin with respect to the same limit state should increase according to the same order (Magliulo &

etal, 2014). However, the investigation for a more useful and rational analysis & design process is a big issue and open for the future study.

Thus, Linear Modal Time History Analyses (LRHAs) with transient analysis was selected in this thesis work based on the nature of structural model and objectives of this study.

3.1.1 EQUIVALENT STATIC ANALYSIS (ESA)

Lateral force analysis is a basic linear static analysis technique used to provide structural response with in elastic range, which is simple to apply than the multi-modal response method. This approach defines a series of forces acting on a building to represent the effect of earthquake ground motion, typically defined by a seismic design response spectrum. It assumes that the building responds in its fundamental mode. For this to be true the building must be regular, low-rise and not twist significantly when the ground moves. The response is read from a design response spectrum, given the natural frequency of the building. The applicability of this method is extended in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces. Likewise, ES 8 EN 1998-1:2015 state that ESA type of analysis method shall be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The above requirement is deemed to be satisfied in buildings which fulfill the following conditions.

a) They have fundamental periods of vibration T_1 in the two main directions which are smaller than $T_1 \leq \{4T_c, 2s\}$. Where, Tc is given in Table 3-1 for type 1 elastic response spectra
b) They meet the criteria for regularity in elevation (given in 4.2.3.3 ES 8 EN 1998-1:2015).

For the determination of the fundamental period of vibration $T1$ of the building, expressions based on methods of structural dynamics (for example the Rayleigh method) may be used. Or for buildings with heights of up to 40 m the value of T1 (in S) may be approximated by the following expression: $T_1 = C_t . H^{3/4}$. Where, Ct = 0.085 for moment resistant space steel frames, Ct = 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames, Ct =0.050 for all other structures; H is the height of the building, in m from the top of a rigid basement.

3.1.2 RESPONSE SPECTRUM ANALYSIS (RSA)

Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Therefore, for tall buildings, buildings with torsional irregularities, or non-orthogonal systems, a dynamic procedure is required. Thus, Response spectrum analysis is one of elastic dynamic analysis so called modal super-position method. In the linear dynamic procedure, the building is modeled as a multi degree- of-freedom (MDOF) system with a linear elastic stiffness matrix and an equivalent viscous damping matrix. The seismic input is modeled using either modal spectral analysis or time history analysis but in both cases, the corresponding internal forces and displacements are determined using linear elastic analysis. However, since they are based on linear elastic response the applicability decreases with increasing nonlinear behavior. The analytical method can use modal decomposition as a means of reducing the degrees of freedom in the analysis.

Likewise, ES 8 EN 1998-1:2015 state that RSA types of analysis method shall be applied to buildings whose response is significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction. The response of all modes of vibration contributing significantly to the global response shall be taken into account. This method is based on the fact that, for certain forms of damping which are reasonable models for many buildings the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation (mode shape), with its own frequency (the modal frequency), and with its own modal damping.

These requirements may be deemed to be satisfied if:

- ➢ All modes with effective modal masses greater than 5% of the total mass are taken into account.
- ➢ The sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure and the minimum number k of

modes to be taken into account in a spatial analysis should satisfy both the two following conditions.

$$k \geq 3.\sqrt{n}$$

$$\mathrm{Tk} \leq 0.20s$$

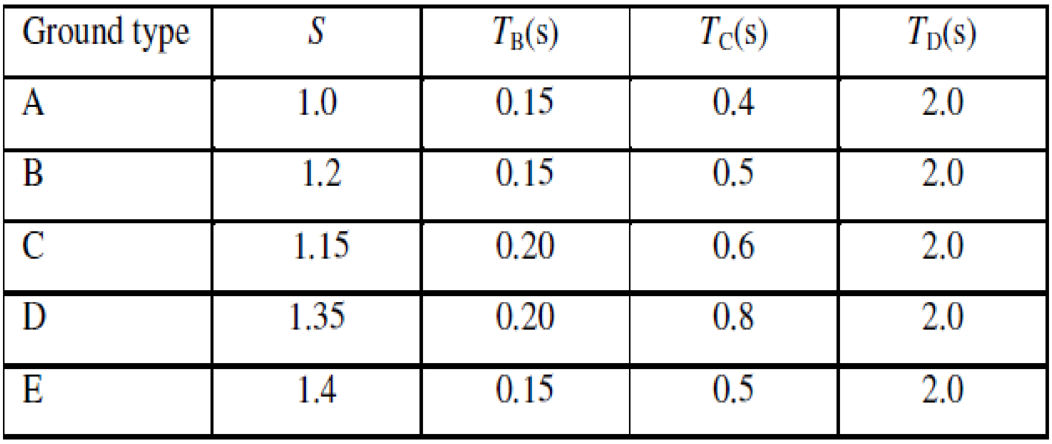
Where**:** k- is the number of modes taken into account; n- is the number of stories above the foundation or the top of a rigid basement; Tk- is the period of vibration of mode k.

Euro code (EC-8) 1998:2004 suggests two different design spectrums.

a) Type 1 for High and moderate seismicity regions (distance EQ, MS>5.5) (southern Europe)

b) Type 2 for Low seismically active regions (local EQs <5.5) (central and northern Europe) and (NDP, recommended: PGA on rock ≤0.08g). But, in this study, Type1 design spectrum was considered.

Table 3.1: Values of the parameters describing the recommended type 1 elastic response spectra.



Ground type	S	T_B(s)	T_C(s)	T_D(s)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

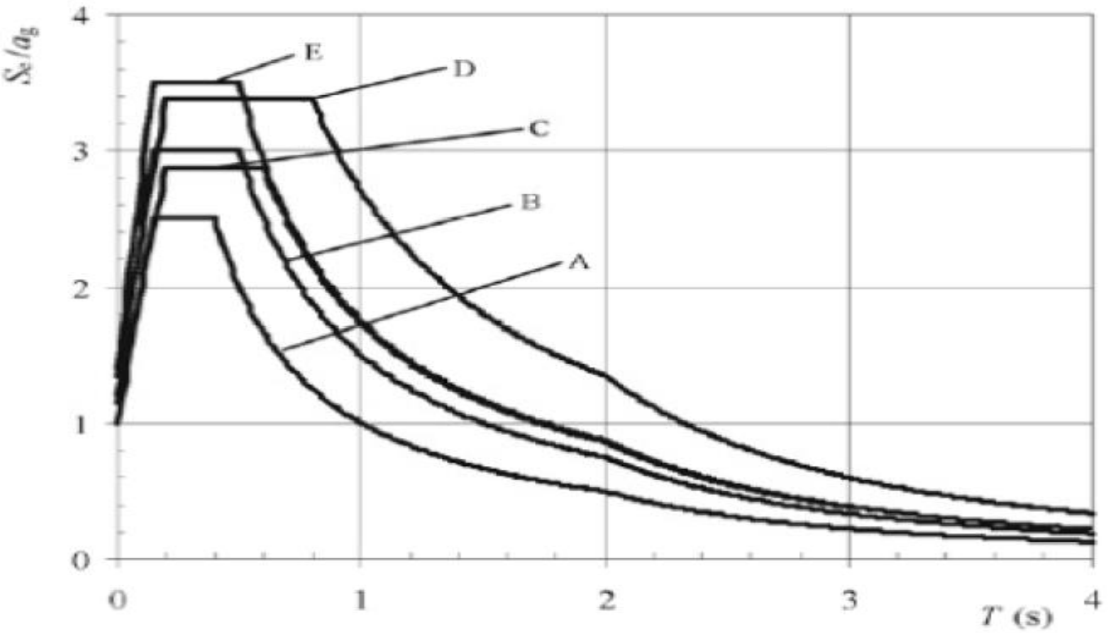


Figure 3.1: Recommended Type 1 elastic response spectra for ground types A to E (5% damping)

3.1.3 PUSHOVER ANALYSIS

According ES 8 EN 1998-1:2015, Pushover analysis is a non-linear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for detailed performance evaluation of building. It may be applied to verify the structural performance of newly designed and of existing buildings for the following purposes:

I. To verify or revise the over strength ratio values $\alpha u / \alpha 1$

II. To estimate the expected plastic mechanisms and the distribution of damage
III. To assess the structural performance of existing or retrofitted buildings

IV. As an alternative to the design based on linear-elastic analysis which uses the behavior factor q.

3.1.4 TIME HISTORY ANALYSIS

Time-history analysis is a step by step procedure, provides for linear or nonlinear evaluation of dynamic structural response under a specified loading that may vary with time function. The dynamic equation can be solved either modal or direct integration method. Time history analysis is the only realistic method that describes the actual behavior the structure during earthquake actions, and also used to determine the accurate response of a structure under dynamic loading of representative earthquake. It is mostly adopted for asymmetrical and high-rise buildings to provide a better check to the safety of structural analysis and design. The requirements for the mathematical model for time history analysis are identical to those developed for response spectrum analysis. In THA response parameters shall be calculated for each time-history analysis. If three time-history analysis are performed, the maximum response of the parameter of interest shall be used for design. If seven or more pairs of horizontal ground motion records are used for time-history analysis, the average response of the parameter of interest may be used for analysis and design [9].

Indeed, in most tectonic regions around the world the earthquake ground motion can act along any horizontal direction, this implies the existence of possible different direction of seismic incidence that would leads to an increase of structural response so called critical angle of seismic incidence [18]. Therefore, RHA is one of a dynamic analysis method used to investigate critical angle of seismic incidence (ASIcri) for a given structure that maximizes demands for all EDPs under different orientation of seismic excitations.

There are several options that determine the type of time-history analysis to be performed:

- Linear vs. Nonlinear.

- Modal vs. Direct-integration: These are two different solution methods, each with advantages and dis advantages. Under ideal circumstances, both methods should yield the same results to a given problem.

- Transient vs. Periodic: Transient analysis considers the applied load as a one-time event, with a beginning and end. Periodic analysis considers the load to repeat indefinitely, with all transient response damped out. Periodic analysis is only available for linear modal time-history analysis [6].

3.1.4.1 Linear Time History Analysis (LTHA)

Linear time history analysis is one of dynamic analysis procedure that provides structural response as function of time with in elastic range. The maximum (peak) value of response of the parameter of interest over the duration of earthquake-induced response shall be used for structural analysis and design [5]. For linear THA the time function curve is start from zero, while for nonlinear time history analysis the time function curve may start from zero or continue from previse analysis [6].

3.1.4.2 NONLINEAR TIME HISTORY ANALYSIS (NLTHA)

NLTHA is the most accurate dynamic analysis method used to predict the force and deformation demands at various components of the structure to inelastic range. It requires proper modeling of the cyclic load-deformation characteristics, and careful consideration of the deterioration properties of all the important components.

It can be of two type nonlinear modal time history (FNA) and nonlinear direct integration time history. These are two different solution methods, each with advantages and dis advantages. Under ideal circumstances, both methods should yield the same results to a given problem. For nonlinear direct-integration time-history analysis, all of the available nonlinearities may be considered. While for nonlinear modal time-history analysis, only the nonlinear behavior of the Link/Support elements is included. If the modes used for this analysis were computed using the stiffness from the end of a nonlinear load case, all other types of nonlinearities are locked into the state that existed at the end of that nonlinear load case. Therefore, compared with the direct integration the modal analysis is more efficient because it is considerably reduces the size of the system and the computational time [6].

3.2 Alternative representations of the seismic action (ES 8 EN 1998-1:2015)

3.2.1 Time - history representation (see 3.2.3.1 ES 8 EN 1998-1:2015)
3.2.1.1 General
(1)P The seismic motion may also be represented in terms of ground acceleration time-histories and related quantities (velocity and displacement).

(2)P When a spatial model is required; the seismic motion shall consist of three simultaneously acting accelerograms. The same accelerogram may not be used simultaneously along both horizontal directions. Simplifications are possible in accordance with the relevant Parts of ES 8 EN 1998-:2015.

(3) Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2 ES 8 EN 1998-1:2015) and recorded or simulated accelerograms (see 3.2.3.1.3 ES 8 EN 1998-1:2015).

3.2.1.2 Artificial accelerograms. (See 3.2.3.1.2 ES 8 EN 1998-1:2015)

(1)P Artificial accelerograms shall be generated so as to match the elastic response spectra given in 3.2.2.2 and 3.2.2.3 for 5% viscous damping.

(2)P The duration of the accelerograms shall be consistent with the magnitude and the other relevant features of the seismic event underlying the establishment of ag.

(3) When site-specific data are not available, the minimum duration Ts of the stationary part of the accelerograms should be equal to 10s.

(4) The suite of artificial accelerograms should observe the following rules:

a) A minimum of 3 accelerograms should be used;

b) The mean of the zero period spectral response acceleration values (calculated from the individual time histories) should not be smaller than the value of ag×S for the site in question.

c) In the range of periods between 0.2T1 and 2T1, where T1 is the fundamental period of the structure in the direction where the accelerogram will be applied; no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

3.2.1.3 Recorded or simulated accelerograms (see 3.2.3.1.3 ES 8 EN 1998:2015)

(1)P Recorded accelerograms, or accelerograms generated through a numerical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismo genetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of ag×S for the zone under consideration.

(2)P For soil amplification analyses and for dynamic slope stability verifications (see ES 8 EN 1998-5:2015, 2.2.)

(3) The suite of recorded or simulated accelerograms to be used should satisfy 3.2.3.1.2(4).

CHAPTER FOUR

MODELING OF STRUCTURAL SYSTEMS

4.1 Description of Structural Modeling

For the analysis work, different 3-D computer models of single and multistory symmetric and asymmetric reinforced concrete moment frame buildings were generated on with the aspect of changing plan configurations. The building models were represented as part (I) for single story models and part (II) for multistory models depending on story level and their plan configuration. Each building models were subjected to the same ground motion in 13 different orientation angles α in range of 0 to 180 degree in 15º increments to find the critical angle where the global and local structural responses are maximized.

The critical angle of seismic incidence and its corresponding structural response was examined using RHA in linear-elastic domains using ETABs v.16. In addition to the influence of the ground motion incidence angle on seismic behavior of symmetric and asymmetric buildings with reentrant plan configurations, the influence of change in plan configurations and column locations on seismic behavior of asymmetric building with reentrant plan configurations was also examined in this study.

4.1.1 Assumptions and Analysis Considerations

The assumption and considerations for analyzing the structure are:

1. Modal damping 5% is considered.

2. Plan dimension and size of structural elements (beams and columns) are kept similar to all Story.

3. The effect of soil structure interaction is ignored in analysis. The columns are assumed to be fixed at the ground level.

4. Beams and columns are modeled as frame element and joined node to nodes.

5. Participating Components: Only the primary structural components are assumed to participate in the overall behavior. The effects of secondary structural components and nonstructural components are assumed to be negligible; these include staircases, partitions, cladding, and openings.

6. Gravity loads are not considered except self-weight of structural elements.

7. Modal linear time history analysis method is considered.

8. The buildings are assumed to be founded on a “C” soil type (Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters), according to ES 8 EN 1998-1-1:2015 classification, i.e. soil with an average shear wave velocity in the range 180 m/s and 360 m/s.

9. Modal analyses of the structures were performed as per Euro code 8 (CEN, 2004); Euro code 8 allows reducing the flexural stiffness properties of concrete elements up to one-half (50%) of the corresponding stiffness of the un cracked elements, in order to take into account the effect of cracking.

10. Description of loading condition and other material and cross sectional property presented in the following tables**.**

I. Material Properties

Table 4.1 material property

Material properties	
Grade of concrete	C-20/25
Mass Density of reinforced concrete	24.99kN/m3
Poisons ratio of concrete	0.2
Modulus of elasticity of concrete	30GPa
Coefficient of thermal expansion of concrete	10*10-6 per o C
Grade of steel(rebar)	S-420
Density of reinforcing steel	7851.815 kg/m3
Coefficient of thermal expansion of steel	11.7*10-6 per oC
Modulus of elasticity of steel	200GPa
Poisons ratio of steel	0.3

II. Cross Section Property

Table 4.2 Cross section property

Building model	Beam section (mm)	Column section(mm)	Slab thickness(mm)	Number of stories	Story height(m)
I. Model 1	350X300	400X400	150	1	3
I. Model 2	350X300	400X400	150	1	3
II. Model 1	350X300	400X400	150	4	3
II. Model 2	350X300	400X400	150	4	3
II. Model 3	350X300	400X400	150	4	3
II. Model 4	350X300	400X400	150	4	3
II. Model 5	350X300	400X400	150	4	3

iii. Loading condition:

a. Gravity Loads**:** Only self-weight of the structural elements were considered.

b. Seismic loading:

Table 4.3 earth quake data

Seismic Zone	IV
Bedrock acceleration ratio ($\alpha_o = a_{og}$)	0.15g
Importance factor, I	1
Subsoil class	C
Damping Ratio	5%

Where, ($\alpha_o = a_{og}$) is ratio of design bedrock acceleration to acceleration of gravity

4.1.2 Selection of Ground Motions Time History Data

ES 8 EN 1998-1-1:2015 states in § 3.2.3.1.1 (3)P: “Depending on the nature of the application and on the information actually available, the description of the seismic motion may be made by using artificial accelerograms (see 3.2.3.1.2) and recorded or simulated accelerograms (see 3.2.3.1.3)”. Thus, to achieve the objective of the study, the recorded pair ground motion of Landers earthquake data occurred in yermo fire station (in 28/6/1992 with 7.4Ms) were selected as time history function from the program file of ETABS V16 software manual.

The selected pair of TH function has been scaled to PGA of 0.15g (seismic zone IV), soil type C (with S=1.15), and 5% damping ratio as per ES 8 EN 1998-1-1:2015.(ie ES 8 EN 1998-1-1:2015 states in § 3.2.3.1.3 (1) P: “Recorded accelerograms, or accelerograms generated through a numerical simulation of source and travel path mechanisms, may be used, provided that the samples used are adequately qualified with regard to the seismo genetic features of the sources and to the soil conditions appropriate to the site, and their values are scaled to the value of ag×S for the zone under consideration”.

Up on this, the relative intensities of the two components of recorded accelerograms were scaled to a peak ground acceleration of 0.15g (zone 4) and soil type C (S=1.15) from type 1 elastic response spectra, and 1.7 scale factor has been used in both X and Y component of time history data. Then all building models were analyzed, under the same ground motion data in different orientation angle α (0^0-180^0) @ 15^0 increments.

Note; soil type C taken as average of A, B, C, D and E from Table 3.1: Ground types EBCS EN 1998-1-1:2015.

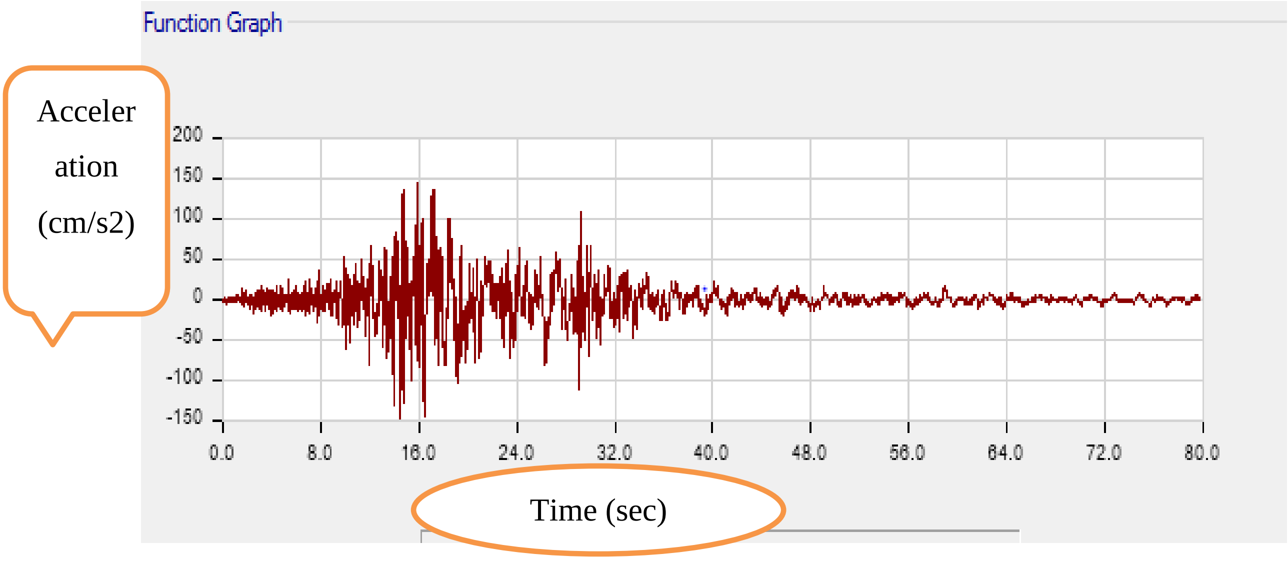


Figure 4.1 Yermo fire station earthquake time history data (YERMO-1 THX)

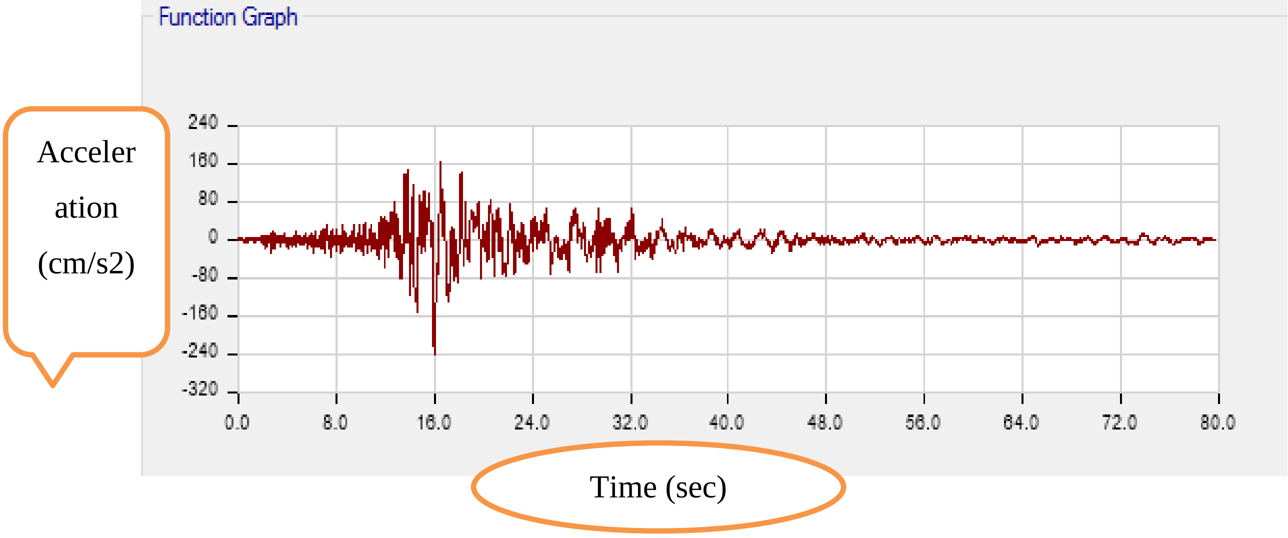


Figure 4.2 Yermo fire station earthquake time history data (YERMO-2 THY)

4.2 Description of Studied structural configuration

The test-bed systems used are the 3D computer models of single-story and multistory symmetric- and asymmetric-plan buildings. Descriptions of their structural systems and computer models are discussed below.

4.2.1 Part I: Single-Story Symmetric- and Asymmetric-Plan Buildings

The first set of test-bed structures are single-story symmetric and asymmetric-plan buildings.

Table4.4 single-story buildings models

Model type	Plan configuration	Plan dimension (X,Y) in [m]
1	Regular with square shape	5*5
2	Irregular L-shape with 60% re-entrant along X and Y directions (56.25%re-entrant area)	5*5

1. Model (1):Regular, square shaped model

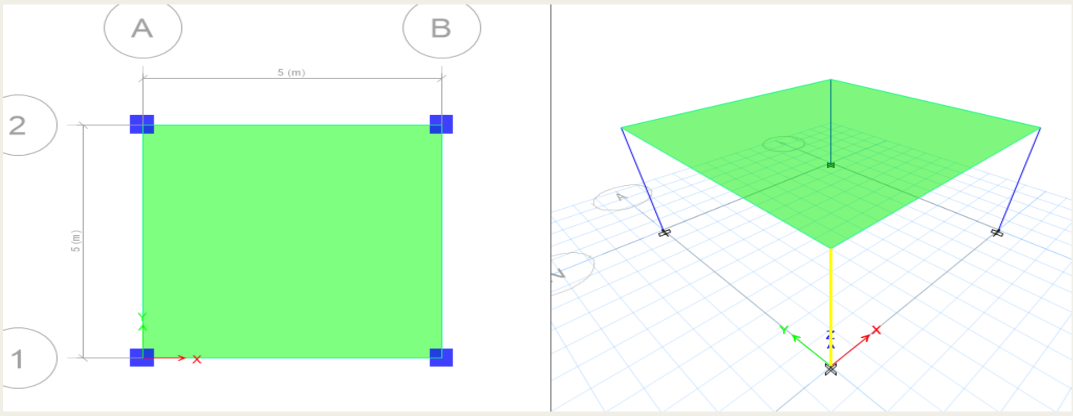


Figure 4.3 Plan and 3D view of part I model (1)

2. Model (2): Irregular L-shaped model with 56.25%re-entrant area.

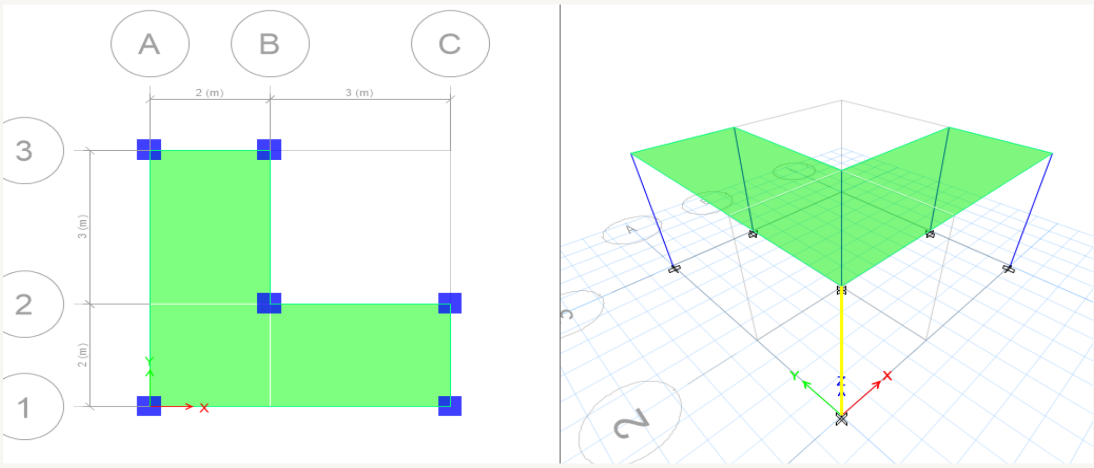


Figure 4.4 Plan and 3D view of part I model (2)

4.2.2 Part II: Multistory Symmetric- and Asymmetric-Plan Buildings

The second set of test-bed structures are four-story symmetric- and asymmetric-plan buildings.

Table 4.5 multi-story buildings models

Model type	Plan configuration type	Plan dimension (X,Y) in [m]
1	Regular with rectangular shape with 1.67 aspect ratios	15*25
2	Regular with Square shape	25*25
3	Irregular L-shape with 60 and 40% re-entrant along X and Y directions respectively (31.6% re-entrant area)	25*25
4	Irregular L-shape with 60% re-entrant along X and Y directions (56% re-entrant area)	25*25
5	Irregular with 36% re-entrant area	15*25

1. Model (1) [regular rectangular shape with 1.67 aspect ratios]: is a four-story building with 3 and 5 bays of 5m each along the X and Y-direction respectively.

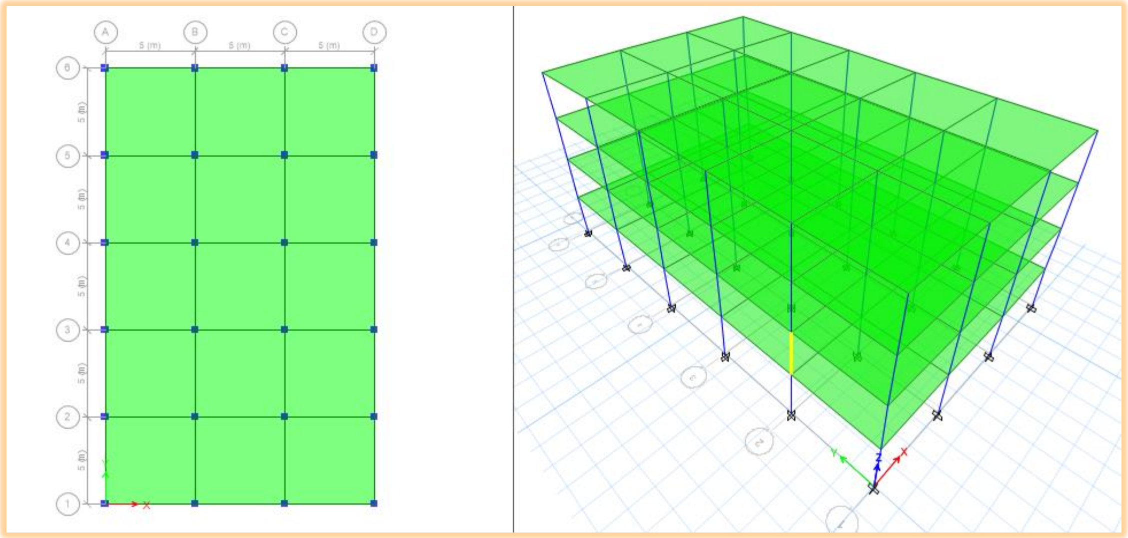


Fig. 4.5 Plan and 3D view of part II model (1)

2. Model (2) [Regular Square Shaped building]: is a four-story building with 5 and 5 bays of 5m each along the X and Y-direction respectively.

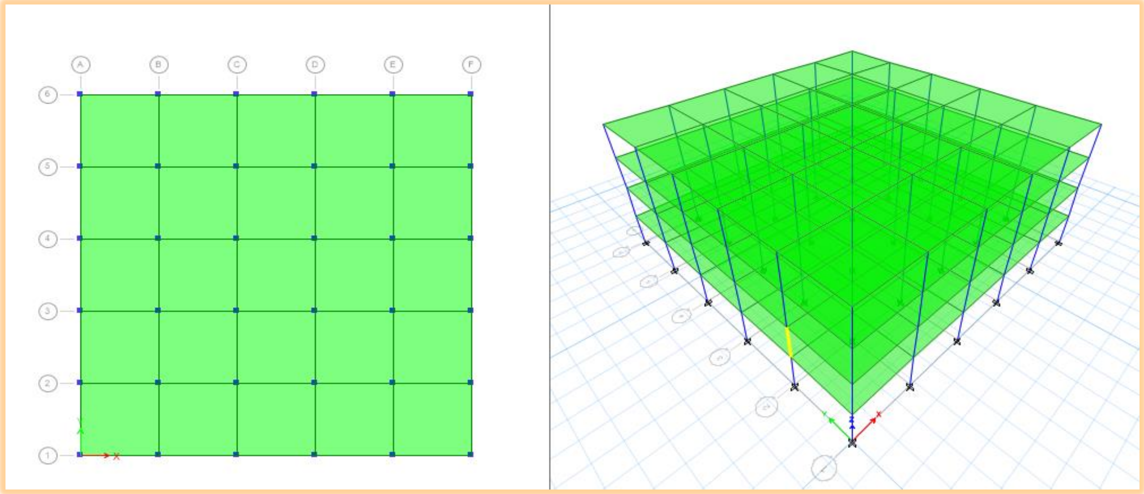


Fig. 4.6 Plan and 3D view of part II model (2)

3. Model (3) [irregular L-Shaped building with 31.6% re-entrant area]: is four-story L shaped buildings with 5 bays of 5m each along both X and Y -direction.

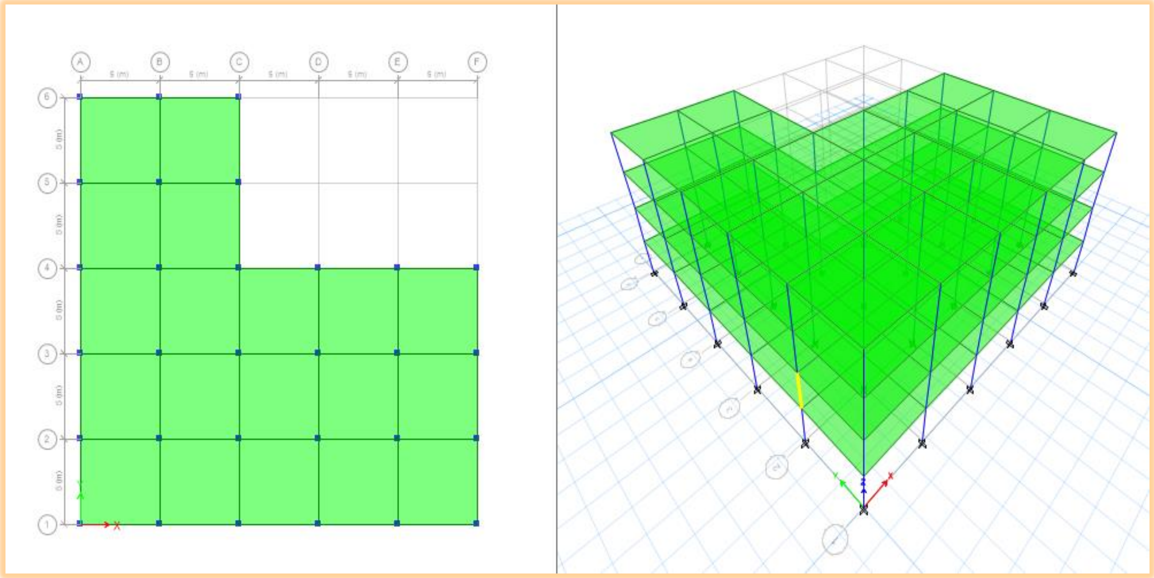


Fig. 4.7 Plan and 3D view of part II model (3)

4. Model (4) [irregular L-Shaped building with 56% re-entrant area]: is four-story L shaped buildings with 5 bays of 5m each along both X and Y -direction.

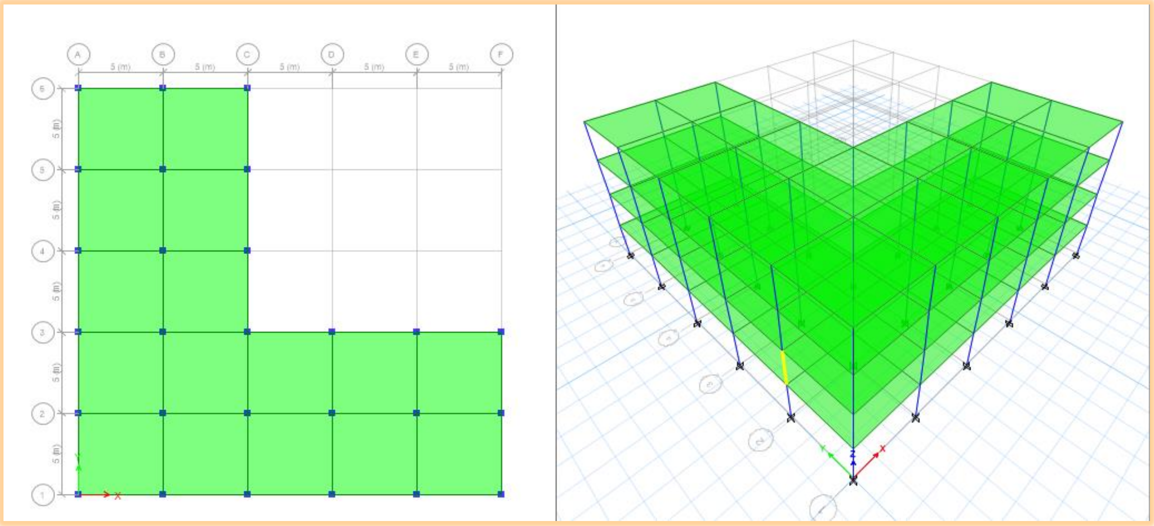


Fig. 4.8 Plan and 3D view of part II model (4)

5. Model (5) [Irregular building with 36.36% re-entrant area]: is a four-story asymmetric building with 3 and 5 bays of 5m each along the X and Y-direction respectively.

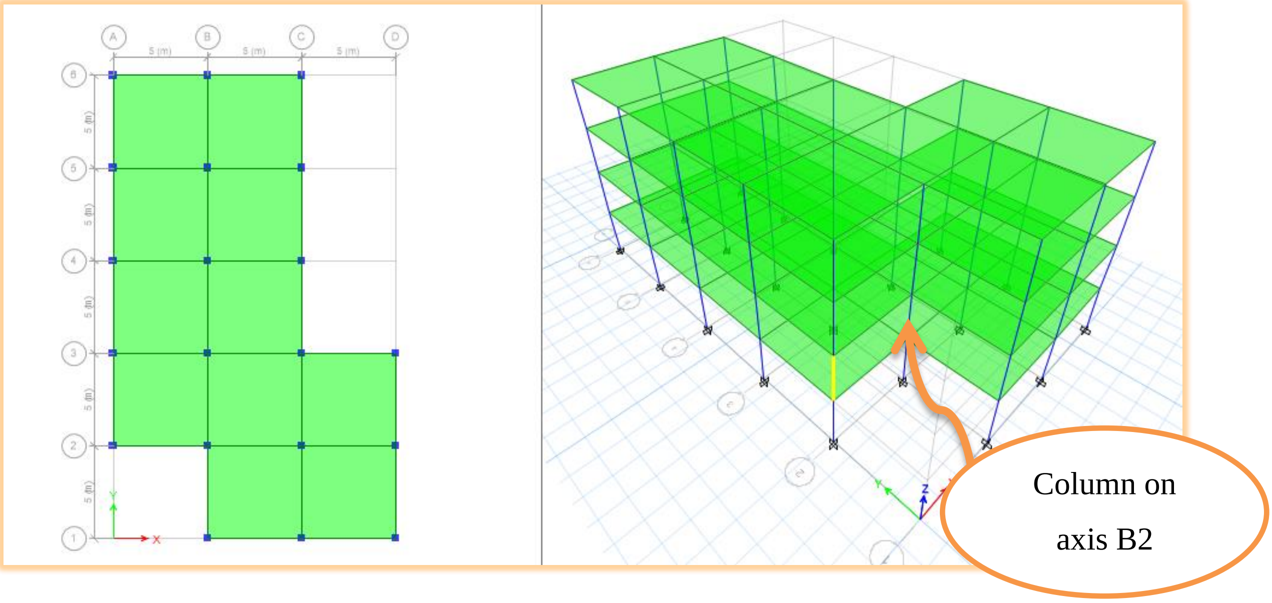


Fig. 4.9 Plan and 3D view of part II model (5)

CHAPTER FIVE

RESULTS AND DISCUSSION

Linear Time History Analyses (LRHAs) were conducted to investigate the possible critical angle of seismic incidence and its corresponding structural demand, in order to assess the seismic behaviors of reinforced concrete buildings with symmetric and asymmetric re-entrant corner plan configurations under different orientations of seismic excitation. The result of single story and four-story symmetric and asymmetric buildings have been used to examine and compare the percentage variations in structural response due to change in ASI and plan configurations, in terms of the **maximum story drift, maximum column axial force, torsion and bending moment**.

Note: 1. Maximum story drift has been studied at first story level for single story models and second story level for multi-story models.

2. Maximum column axial force, torsion and bending moment has been studied on axis A1 for single story models and on axis A2 at second (2nd) story for all multi-story models and also on axis B2 for model 1&5 at the same story (see section 4.2.1 & 4.2.2 above).

5.1 Maximum Story Drifts

The values of maximum story drifts and the variation with seismic incidence angle are shown in Table and Figure (5.1A&B).

Table 5.1A Maximum governing story drift for single story model at 1st story

	Max governing story drift	Max governing story drift
Angle	model 1	model 2
0	0.000272	0.000167
15	0.000292	0.000187
30	**0.000298**	0.00021
45	0.000283	0.000225
60	0.000248	0.000225
75	0.000224	0.00021
90	0.000267	0.00018
105	0.000292	0.000161
120	**0.000298**	0.000189
135	0.000283	0.000225
150	0.000248	0.000245
165	0.000197	**0.000249**
180	0.000183	0.000245

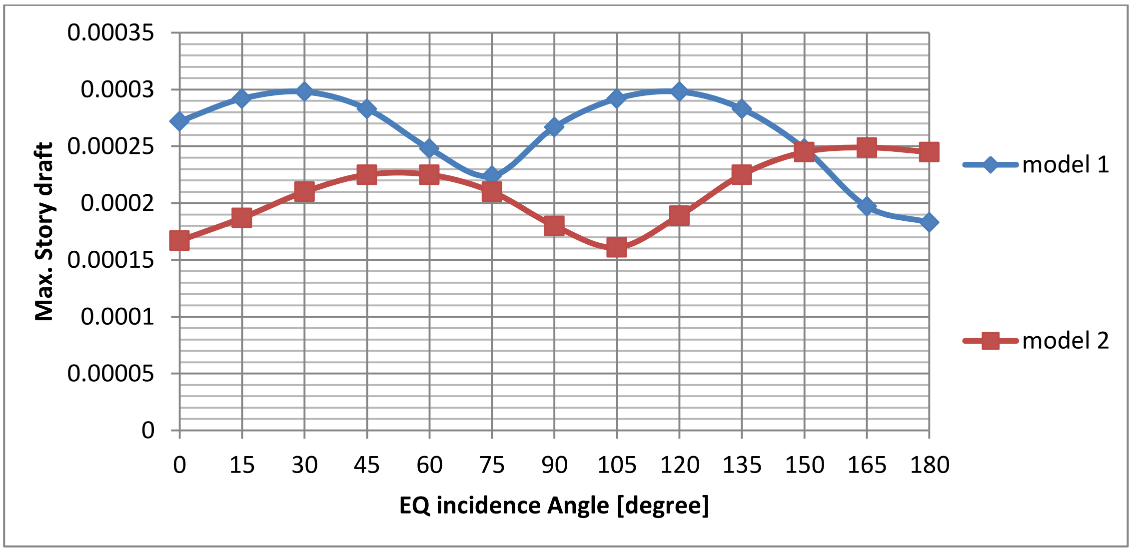


Fig 5.1A: Variation of maximum story drifts with EQ incidence angle for single story model

As shown in Figure 5.1A and Table 5.1A above, for single story regular shaped building model (1), the maximum story drifts occurred when the peak ground acceleration is applied at 30,120 degrees, the value increased by 8.72% with respect to 0 degrees. While, for single story L-shaped building model (2) the maximum story drifts occurred when the peak ground acceleration is applied at 165 degrees, the value increased by 32.93% with respect to the value at 0 degrees.

Additionally, considering critical incidence angle values of each model, as building plan configuration changes form regular square shaped model (1) to irregular L-shaped model (2) the value of maximum story drift is decreased by 16.44%, which is the reverse of what expected due to additional column in model (2). This decrease in maximum story drift implies an increase in the story stiffness and stability issues (LF resistance capacity). (ie. Model (2) has more story stiffness & Lateral load resistance capacity than Model (1)).

Table 5.1B Maximum governing story drift for multi-story model at 2nd story

Angle	model(1) Rectangular	model(2) Square	model (3)	model (4)	model (5)
0	0.000971	0.000916	0.000991	0.000981	0.000999
15	**0.001013**	0.000987	**0.001048**	0.001016	0.000992
30	0.000986	**0.000991**	0.001033	0.000986	0.000917
45	0.000893	0.000927	0.000949	0.00091	0.00096
60	0.000927	0.000882	0.000967	0.001001	0.001002
75	0.000979	0.000954	0.001003	**0.001024**	0.001017
90	0.000964	0.000962	0.000971	0.000976	0.00097
105	0.000884	0.000903	0.000875	0.000863	0.000858
120	0.000796	0.000784	0.000815	0.000841	0.000885
135	0.000883	0.000861	0.000895	0.000916	0.000998
150	0.000964	0.000895	0.000966	0.000987	**0.001043**
165	0.000993	0.000954	0.001017	0.001004	0.001016
180	0.000954	0.000962	0.000999	0.000951	0.000933

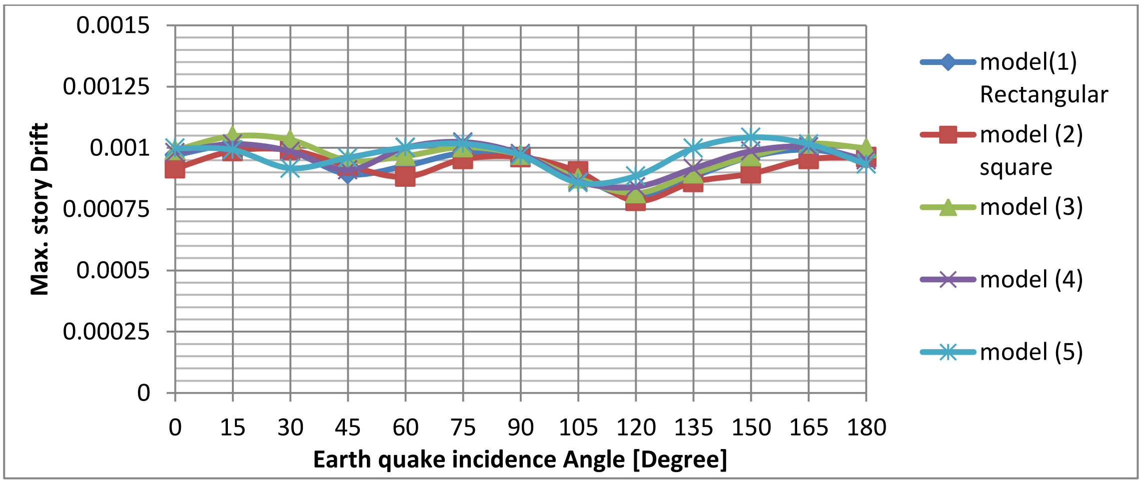


Fig 5.1B: Variation of maximum story drifts with EQ incidence angle for multi-story models at 2nd story

As shown in Figure 5.1B and Table 5.1B, for multi-story regular rectangular shaped building model (1), the maximum story drifts have occurred when the peak ground acceleration is applied @ 15 degree, the value increased by 4.15% with respect to 0 degrees. While, for multi-story regular square shaped building model (2) the maximum story drifts has occurred when the peak ground acceleration is applied at 30 degrees, the value increased by 7.57% with respect to 0 degrees. For multi-story irregular L-shaped building model (3), the maximum story drifts has occurred when the peak ground acceleration is applied at 15 degrees, the value increased by 5.44% with respect to 0 degrees. For multi-story irregular L-shaped building model (4), the maximum story drifts has occurred when the peak ground acceleration is applied at 75 degrees, the value increased by 4.20% with respect to 0 degrees. For multi-story irregular shaped building model (5), the maximum story drifts has occurred when the peak ground acceleration is applied at 150 degrees, the value increased by 4.22% with respect to 0 degrees.

Additionally, considering critical incidence angle values of all irregular multi story building models, as the building plan configuration changes from regular rectangular shaped model (1) to irregular shaped model (5) the value of maximum story drift is increased by 2.88%. Similarly, as the building plan configuration changes from model (2) to model (1) and also to model (3) and model (4) the value of maximum story drift is increased by 2.17%, 5.44%, and 3.22% respectively.

Hence, reentrant corner plan irregular multi story building models has shown relatively slight variation in maximum story drift due to the change in seismic incidence angle and building plan configurations.

5.2 Maximum Column Axial Forces

The values of maximum column axial forces and the variation with seismic incidence angle are shown in Table and Figure 5.2(A&B).

Table 5.2A Maximum column axial force P [KN] for single story model @ axis A1 on 1st story

	Max column Axial force	Max column Axial force
	Corner column[C1-5]	Corner column[C1-5]
Angle	Model 1 P [KN]	Model 2 P [KN]
0	2.6544	2.6911
15	2.5985	2.5963
30	3.5148	2.3741
45	4.1916	2.4179
60	4.5828	2.9055
75	**4.6616**	3.7077
90	4.4228	4.2571
105	3.8825	**4.5164**
120	3.0777	4.468
135	2.0631	4.115
150	1.8299	3.4817
165	2.392	3.1729
180	2.8015	3.0923

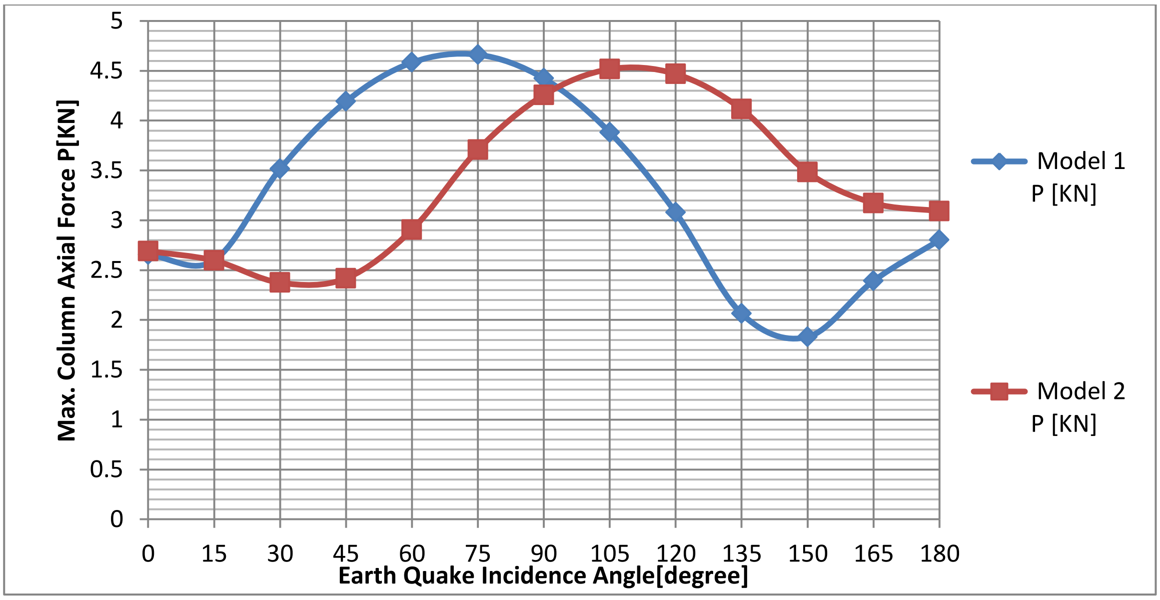


Fig 5.2A: Variation of maximum column axial forces with seismic incidence angle for single-story model on axis A1

As shown in Figure 5.2A and Table 5.2A, for single story regular shaped model (1), the maximum axial forces of column on axis A1 occurred when the peak ground acceleration is applied @ 75 degrees, the value increased by 43% with respect to 0 degrees. While, for irregular L-shaped building model (2) the maximum axial forces of column on axis A1 occurred when the peak ground acceleration is applied at @ 105 degrees, the value increased by 40.4% with respect to 0 degrees.

Additionally, considering critical incidence angle values of each building models, as the building plan configuration changes form regular square shaped model (1) to irregular L-shaped model (2) the values of maximum axial force is decreased by 3.11%.

Table 5.2B Maximum column axial force P [KN] for multi-story model @ axis A2 on 2nd story

	Max.Column Axial force	Max.Column Axial force	Max.Column Axial force	Max.Column Axial force	Max.Column Axial force
	Side Column [C1-234]	Side Column [C3-493]	Side Column [C3-317]	Side Column [C6-203]	Corner Column [C2-155]
Angle	Model(1) Rectangular P[KN]	Model(2) square P[KN]	Model(3) P[KN]	Model(4) P [KNm]	Model(5) P [KNm]
0	17.4639	16.2161	17.4687	17.9733	21.6009
15	15.8793	13.9706	16.0322	18.1328	22.8593
30	19.9531	18.1735	20.0816	21.7489	22.5598
45	22.6672	21.1379	22.7625	23.883	20.723
60	**23.8366**	**22.6618**	**23.8921**	**24.3894**	20.2421
75	23.3815	22.6414	23.3936	23.2338	24.4203
90	21.3331	21.0779	21.3008	20.4948	26.9342
105	17.8308	18.0781	17.7564	16.7049	**27.6126**
120	16.18	16.1464	15.9032	16.1342	26.4092
135	18.5621	18.3611	18.2921	18.5833	23.4062
150	19.6792	19.3245	19.4344	19.8514	20.6221
165	19.4553	18.971	19.2523	19.7666	19.8865
180	17.9054	17.3247	17.7582	18.3348	20.2685

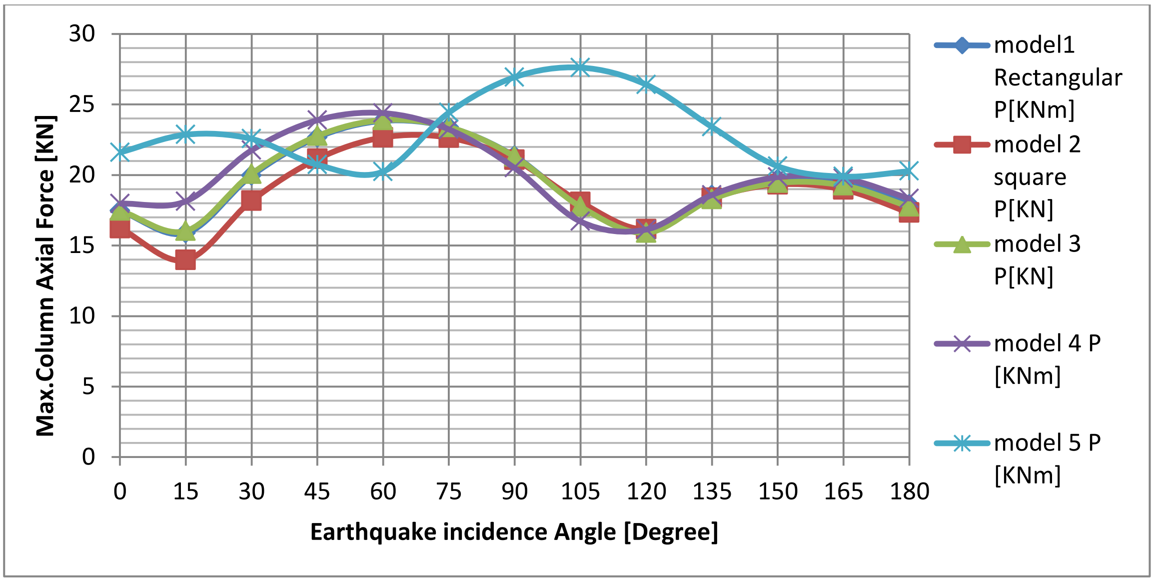


Fig 5.2B: Variation of maximum column axial forces on axis A2 @ 2nd story with seismic incidence angle for multi-story models.

As shown in Figure 5.2B and Table 5.2B, for multi-story regular rectangular shaped building model (1), regular square shaped building model (2), irregular L-shaped building model (3) and model (4) the maximum axial force for side column on axis A2 @ 2nd story have occurred when the peak ground acceleration is applied at 60 degrees, the value increased by 26.7%, 28.4%, 26.9%, 26.3% respectively with respect to the value at 0 degrees. While, for multi-story irregular shaped building model (5) the maximum column axil force has occurred when the peak ground acceleration is applied at 105 degrees, the value increased by 21.8% with respect to 0 degrees.

Additionally, considering critical incidence angle values of each building models, as building plan configuration changes from regular square shaped model (2) to regular rectangular shaped model (1) the value of maximum column axial force is increased by 4.93%. Similarly, as building plan configuration changes from regular square shaped model (2) to irregular L-shaped models model (3) & model (4), the value of maximum axial force of side column in model (3) & model (4) is increased by 5.15 % & 7.1% respectively. Plus, as the column changes form side column in model (1) to corner column in model (5) the value of maximum column axial forces is increased by 13.67 %.

Table 5.2C Maximum column axial force P [KN] for multi -story model 1&5 on axis B2 @ 2nd story

	Max column Axial force	Max column Axial force
	Middle column[C2-238]	Re-entrant column[C16-237]
Angle	model 1 P [KN]	model 5 P [KN]
0	4.3871	3.9482
15	4.743	4.1675
30	4.7756	4.1028
45	4.4828	3.7585
60	3.9464	3.2595
75	3.8087	4.0695
90	4.1949	4.6023
105	4.4787	**4.8214**
120	5.1775	4.712
135	**5.5235**	4.2814
150	5.4931	3.5591
165	5.0883	3.3308
180	4.7103	3.6641

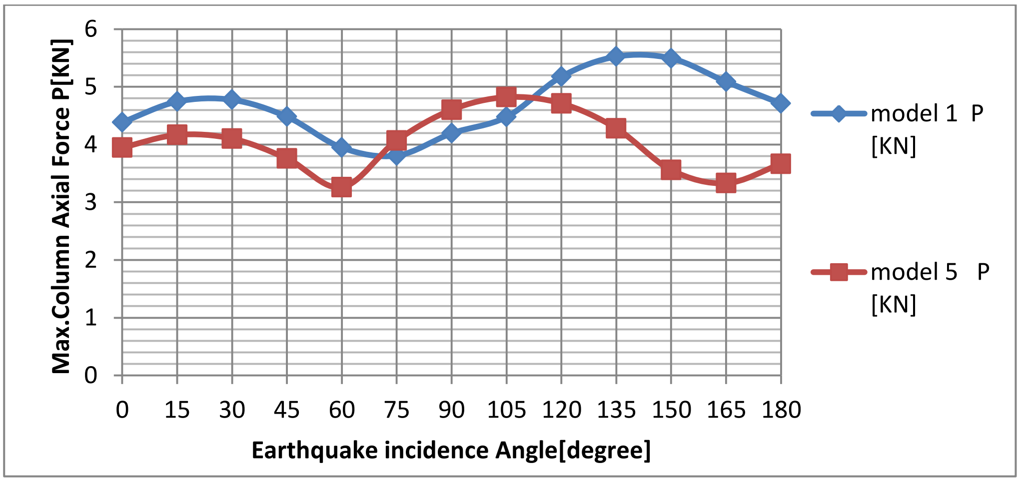


Fig 5.2C: Variation of maximum column axial force with incidence angle for multi-story model 1&5 on axis B2 @ 2nd story

As shown in table and figure 5.2C, for multi-story regular shaped rectangular building model (1), the maximum axial force of middle column on axis B2 @ 2nd story occurred when the peak ground acceleration is applied @ 135 degrees, the value increased by 20.57% with respect to 0 degrees. Whereas, the maximum axial force of the same **re-entrant corner column** in irregular shaped model (5) occurred when the peak ground acceleration is applied @ 105 degrees, the value increased by 18.11% with respect to the value at 0 degrees.

Additionally, considering critical incidence angle values of each building models, as the column changes form middle column in model (1) to **re-entrant corner column** in model (5) the value maximum column axial forces is decreased by 12.71%.

Summary: It is observed that the regular and irregular buildings have shown considerable increase in maximum column axial forces when the peak ground acceleration is subjected at various incidence angles with respect to 0 degree or the axes of building plan. Additionally, considering critical incidence angle values of all models, the reentrant corner plan irregular buildings have shown considerable variations in maximum column axial forces due to the change in building plan configurations & column locations when compared with the corresponding regular building. As the column changes form side column in model (2) to the same side column in model (3) and (4) there has been considerable change (increase) in the value of maximum column axial forces (increased by 5.15 % & 7.1% respectively). While, as the column changes form side column in model (1) to corner column in model (5) again there has been considerable change (increase) in the value of maximum column axial forces (increased by 13.67%). On the other hand as the column changes form middle column in model (1) to reentrant corner column in model (5) there has been considerable change (decrease) in the value of maximum column axial forces (decreased by 12.71%).

Therefore, reentrant corner plan irregular buildings have shown considerable variations in maximum column axial forces due to the change in angle of seismic incidence, building plan configurations & column locations.

5.3 Maximum Column Torsion

The values of maximum column torsion and the variation with seismic incidence angle are shown in Table 5.3 and Figure 5.3.

Table5.3A. Maximum column torsion (T) [KNm] for single-story model @ axis A1 on 1st story

	Max Column Torsion	Max Column Torsion
	Corner column[C1-5]	Corner column[C1-5]
Angle	Model 1 T [KNm]	Model 2 T [KNm]
0	**0**	**1.1076**
15	0	1.0935
30	0	1.0049
45	0	0.919
60	0	0.8759
75	0	0.7731
90	0	0.6801
105	0	0.7053
120	0	0.7232
135	0	0.8346
150	0	0.9379
165	0	0.9772
180	0	0.95

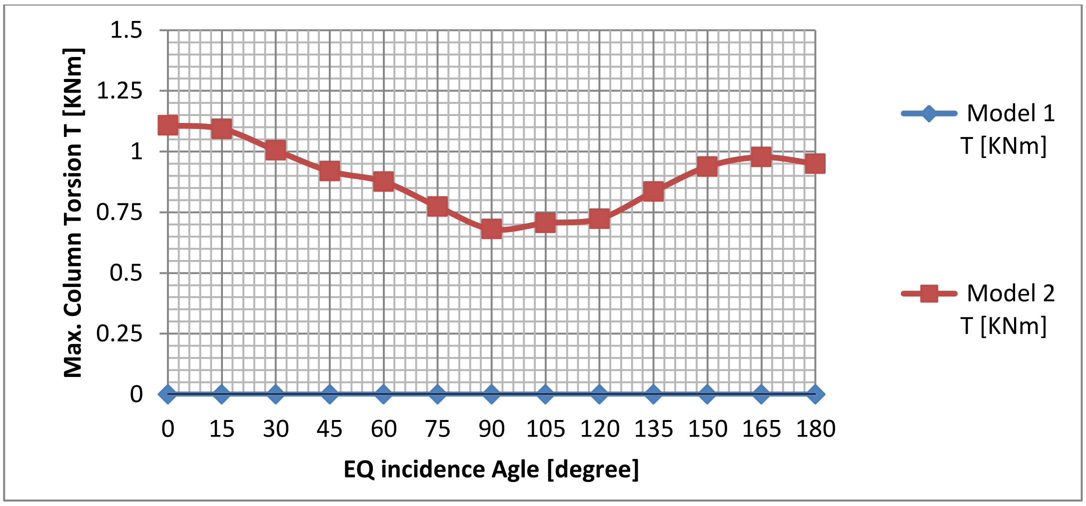


Fig 5.3A: Variation of maximum column torsion with seismic incidence angle for single-story models**.**

As shown in Figure 5.3A and Table 5.3A, for single story regular shaped building model (1), the maximum column torsion on axis A1 is zero. This mean that regular building is not affect by torsions. While, for single story L-shaped building the maximum column torsion on axis A1 occurred when the peak ground acceleration is applied at @ 0 degrees (along building plan axis), the value of maximum column torsion increased by 100% with respect to corresponding regular shaped building.

Table5.3B. Maximum column torsion (T) [KNm] for multi-story models on axis A2 @ 2nd story

	Max.Column Torsion (T)	Max.Column Torsion (T)	Max.Column Torsion (T)	Max.Column Torsion (T)	Max.Column Torsion (T)
	side column [C1-234]	Side column [C3-493]	Side column [C3-317]	Side column [C6-203]	Corner column [C2-155]
Angle	Model(1) Rectangular T [KNm]	Model(2) Square T[KNm]	Model(3) T[KNm]	Model(4) T [KNm]	Model(5) T [KNm]
0	0.00001431	0.00003522	**0.1289**	0.0379	0.1422
15	0.00001235	0.00003954	0.1221	0.0483	0.1468
30	0.00001324	0.00004289	0.1219	0.0575	0.1413
45	0.00001405	0.00004331	0.1139	0.0628	0.1262
60	0.0000139	0.00004079	0.1015	0.0638	0.105
75	0.0000128	0.00004178	0.094	0.0622	0.1286
90	0.00001082	0.00004044	0.0952	0.0637	0.1479
105	0.000009076	0.00004236	0.0959	0.0644	**0.157**
120	0.000009655	0.0001	0.1129	**0.0652**	0.1555
135	0.000009961	0.0001	0.1222	0.0616	0.1433
150	0.000009589	0.0001	0.1232	0.0538	0.1477
165	0.000008668	0.0001	0.1158	0.0543	0.1507
180	0.000009639	0.0001	0.1274	0.0514	0.1505

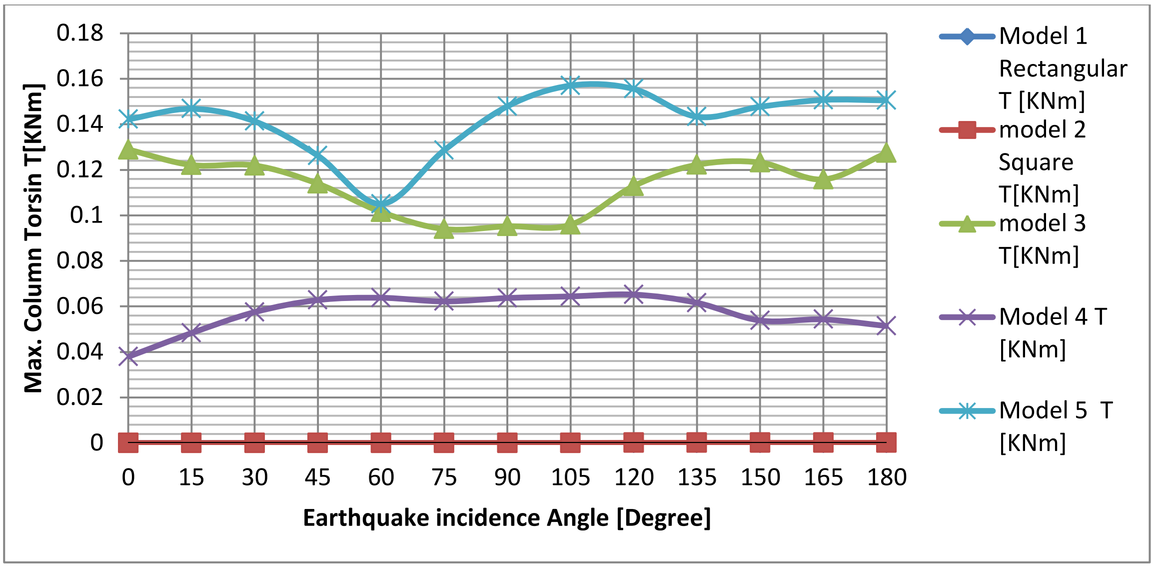


Fig 5.3B: Variation of maximum column torsion with seismic incidence angle for multi-story models

As shown in Figure 5.3B and Table 5.3.B, for multi-story regular shaped building model (1) and model (2), the value of maximum column torsion is almost zero and its variations with seismic incidence angle was also neglected. This mean that regular building is not affect by torsion. While, for multi-story irregular L-shaped building model (3), the maximum column torsion on axis A2 have occurred when the peak ground acceleration is applied at 0 degrees and the value is 0.1289 KNm. For multi-story irregular L-shaped building model (4), the maximum column torsion on axis A2 @2nd story have occurred when the peak ground acceleration is applied @ 120 degrees, the value increased by 41.87% with respect to 0 degrees. For multi-story irregular shaped building model (5), the maximum column torsion on axis A2 have occurred when the peak ground acceleration is applied @ 105 degrees, the value increased by 9.43% with respect to 0 degrees.

Additionally, considering critical incidence angle values of each building models, as the plan configuration change from regular square shaped model (2) to irregular L-shaped model (3) & (4) or from model (1) to corner column in irregular shaped model (5) the value of maximum column torsion (T) is increased by 100%.

Summary: It is observed that the single and multi-story asymmetric buildings have shown considerable increase in maximum column torsion when the peak ground acceleration is subjected at various incidence angles with respect to the axes of building plan. Plus, the re-entrant corner plan irregular buildings are more sensitive to maximum column torsion when compared with the corresponding regular building.

5.4 Maximum Column Bending Moment

The values of maximum column moment and the variation with seismic incidence angle are shown in Table 5.4 and Figure 5.4.

Table 5.4A1 Maximum column moment M3 [MX] [KNm] for single-story models on axis A1 @ 1st story [bottom moment MX]

	Max Column Moment	Max Column Moment
	Corner column[C1-5]	Corner column[C1-5]
Angle	Model 1 M3 [KNm]	Model 2 M3 [KNm]
0	12.0456	5.8526
15	13.2596	7.8501
30	**13.5699**	9.3248
45	12.9555	10.164
60	11.4582	10.3106
75	9.1801	9.7546
90	6.2763	8.5337
105	5.1951	7.4724
120	6.8674	6.4531
135	8.1018	6.1654
150	8.7841	7.421
165	8.8678	9.5578
180	8.3471	**11.0432**

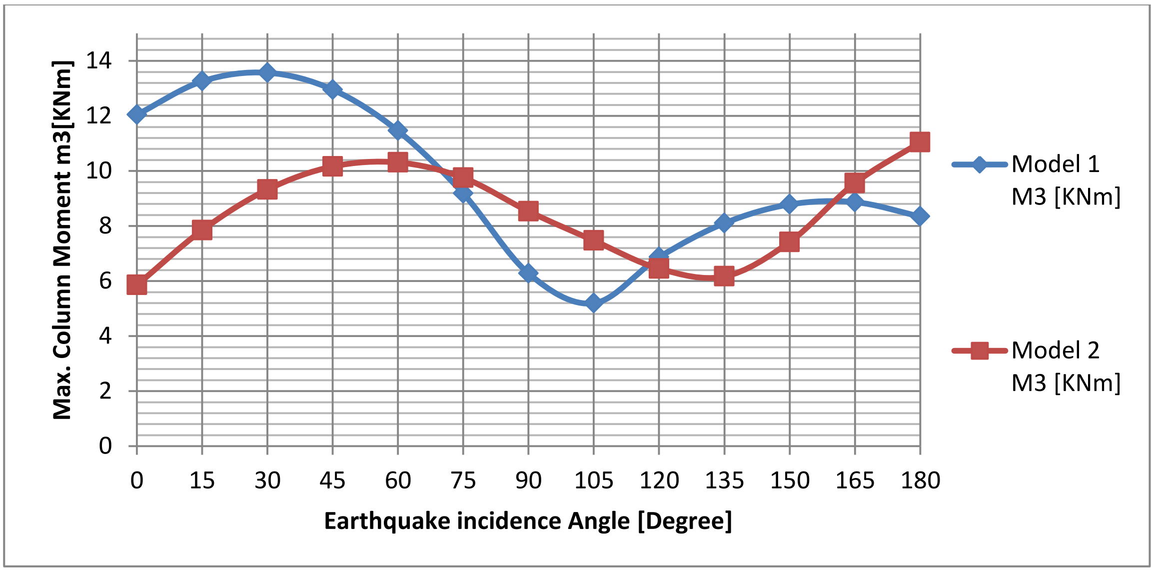


Fig 5.4A1: Variation of maximum column moment M3 with seismic incidence angle for singe-story models

Table 5.4A2 Maximum column moment M2 [MY] [KNm] for single-story models on axis A1 @ 1st story [bottom moment MY]

	Max Column Moment	Max Column Moment
	Corner column[C1-5]	Corner column[C1-5]
Angle	Model 1 M2 [KNm]	Model 2 M2 [KNm]
0	12.3307	7.4779
15	11.4755	8.6367
30	9.8383	9.289
45	7.6446	9.3082
60	7.7748	8.6932
75	10.392	7.5179
90	12.3011	6.9198
105	13.3719	7.1892
120	**13.5314**	9.013
135	12.7687	10.522
150	11.1359	11.3138
165	8.7442	**11.3347**
180	5.9228	10.5831

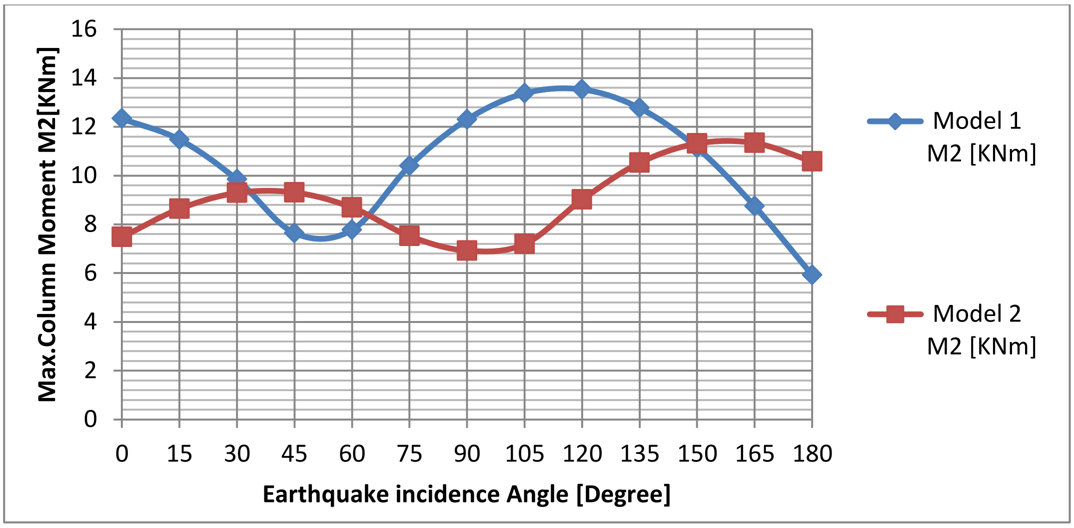


Fig 5.4A2: Variation of maximum column moment M2 with seismic incidence angle for singe- story models

As shown in Figure 5.4(A1,A2) and Table 5.4 (A1,A2), for single story regular shaped building model(1), the maximum bending moment M3 (MX) & M2 (MY) of column on axis A1 occurred when the peak ground acceleration is applied @ 30 and 120 degrees, the value increased by 11.23% and 8.9% with respect to 0 degrees respectively. While, for single story irregular L-shaped building model(2); the maximum bending moment M3 (MX) & M2 (MY) of column on axis A1 occurred when the peak ground acceleration is applied @ 180 and 165 degrees, the value increased by 47.0% and 34.03% with respect to 0 degrees respectively.

Additionally, considering critical incidence angle values of each building models, as building plan configurations changes form regular square shaped model (1) to irregular L-shaped in model (2) the value of maximum moment M3&M2 for corner columns is decreased by 18.62% and 16.23% respectively.

Table 5.4B1 Maximum column moment M3 [MX] [KNm] for multi-story models on axis A2 @ 2nd story [bottom moment MX]

	Max.Column Moment	Max.Column Moment	Max.Column Moment	Max.Column Moment	Max.Column Moment
	Side column [C1-234]	Side column [C3-493]	Side column [C3-317]	Side column [C6-203]	Corner column [C2-155]
Angle	Model(1) Rectangular M3 [KNm]	Model(2) Square M3 [KNm]	Model 3 M3 [KNm]	Model 4 M3 [KNm]	Model (5) M3 [KNm]
0	13.0446	12.3191	13.0346	13.5375	11.3536
15	11.4953	10.1195	11.6159	13.265	11.0737
30	14.4696	12.6654	14.505	16.3762	13.3541
45	17.3358	15.6705	17.2902	18.7969	15.2901
60	19.0206	17.6077	18.8972	**19.9365**	**16.1841**
75	**19.4092**	**18.345**	**19.2163**	19.7176	15.9751
90	18.4751	17.8321	18.2259	18.1549	14.6775
105	16.2819	16.104	15.9934	15.3549	12.3796
120	12.9792	13.2784	12.671	13.1209	9.619
135	14.3655	13.8405	14.1806	14.7638	11.2668
150	15.221	14.8335	15.0482	15.4005	12.1468
165	15.0392	14.8157	14.8904	16.4093	13.0563
180	15.2564	13.7882	15.1903	16.611	13.4208

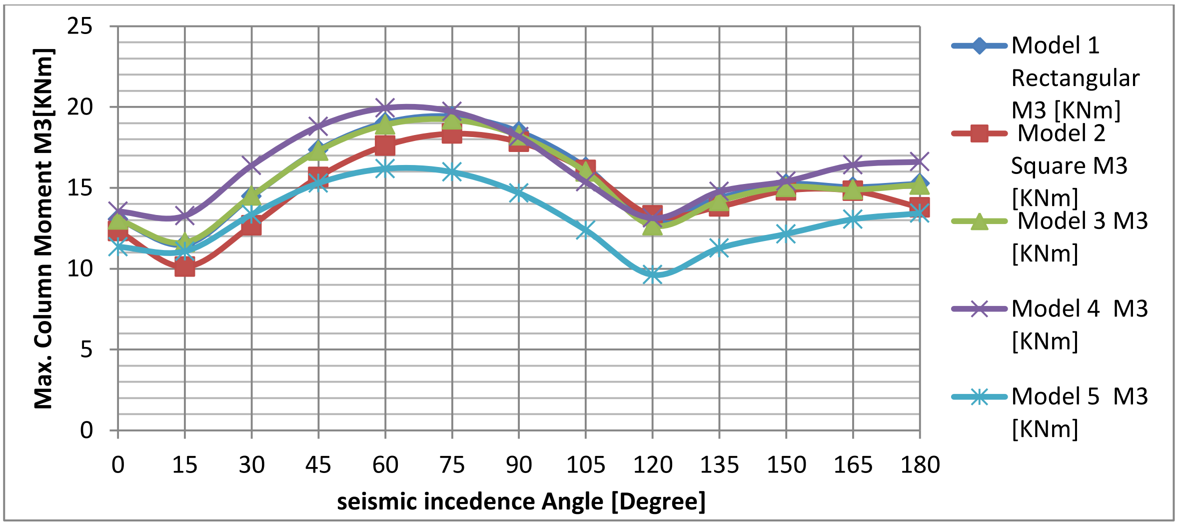


Fig 5.4B1: Variation of maximum column moment M3 with seismic incidence angle for multi- story models on axis A2 @ 2nd story.

As shown in Figure 5.4B1 and Table5.4B1, the maximum bending moment MX (M3) of side column on axis A2 @ 2nd story; For multi-story regular rectangular shaped building model (1), regular square shaped building model (2), and irregular L-shaped building model (3) have occurred when the peak ground acceleration is applied at 75 degrees, the value increased by 32.79%, 32.85%, 32.17% respectively with respect to the value @ 0 degrees. While, for multi-story irregular L-shaped buildings model (4) and model (5) has occurred when the peak ground acceleration is applied at 60 degrees, the value increased by 32.1% & 29.85% respectively with respect to 0 degrees.

Additionally, considering critical incidence angle values of all building models, as the building plan configurations changes form regular square shaped model (2) to regular rectangular shaped model (1) the value of maximum column moment [M3] is increased by 5.48%. Similarly, as the building plan configurations changes form regular square shaped model (2) to irregular L-shaped model (3) and (4), the value of maximum column moment [M3] for side column is increased by 4.53% & 7.98% respectively. While, as the column changes form side column in model (1) to corner column in model (5) the value of maximum column moment (M3) is decreased by 16.62%.

Table 5.4B2 Maximum column moment M2 [MY] [KNm] for multi-story models on axis A2 @ 2nd story

	Max.Column Moment	Max.Column Moment	Max.Column Moment	Max.Column Moment	Max.Column Moment
	Side column [C1-234]	Side column [C3-493]	Side column [C3-317]	Side column [C6-203]	Corner column [C2-155]
Angle	Model(1) Rectangular M2 [KNm]	Model(2) Sqaure M2 [KNm]	Model(3) M2 [KNm]	Model(4) M2 [KNm]	Model(5) M2 [KNm]
0	22.2372	20.5315	21.1205	22.678	13.5141
15	23.5862	22.6902	22.72	23.6606	13.3244
30	23.3279	23.3027	22.7713	23.0307	12.2265
45	21.4798	22.3271	21.2707	20.8314	12.7975
60	20.5076	19.8299	19.6368	20.9314	12.8954
75	19.9682	18.592	19.1474	20.2708	12.1145
90	18.068	16.9238	17.353	18.2288	12.3945
105	16.3165	14.1024	14.9723	17.3181	13.8375
120	19.1515	15.238	17.6791	20.3291	16.0486
135	22.6736	19.3946	21.2859	23.6492	**17.166**
150	24.6505	22.2295	23.4421	**25.3576**	17.1135
165	**24.9476**	**23.5494**	**24.0007**	25.3379	15.8948
180	23.5445	23.2646	22.9237	23.5915	13.5929

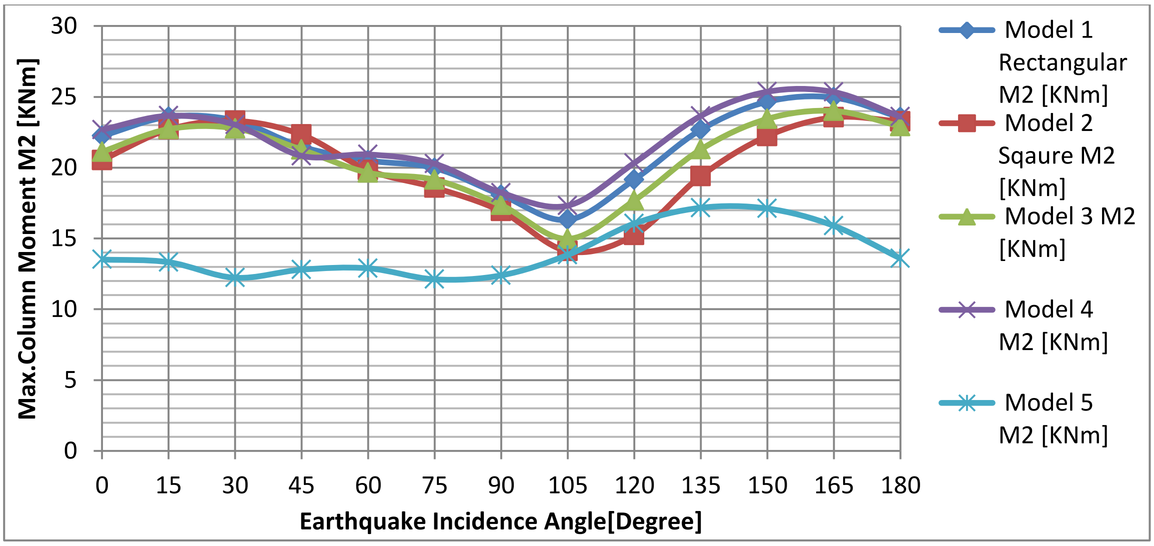


Fig 5.4B2: Variation of maximum column moment M2 with incidence angle for multi-story models on axis A2 @ 2nd story

As shown in Figure 5.4B2 and Table5.4B2, the maximum bending moment MY (M2) of side column on axis A2 at 2nd story; For multi-story regular rectangular shaped building model (1), regular square shaped building model (2), and irregular L-shaped building model (3) have occurred when the peak ground acceleration is applied at 165 degrees, the value increased by 10.9%, 12.8%, 12.0% respectively with respect to 0 degrees. While, for multi-story irregular L-shaped buildings model (4) and model (5) has occurred when the peak ground acceleration is applied at 150 & 135 degrees, the value increased by 10.6%, & 21.3% respectively with respect to 0 degrees.

Additionally, considering critical incidence angle values of each building models, as building plan configuration changes from regular square shaped model (2) to regular rectangular shaped model (1) the value of maximum column moment [M2] is increased by 5.605%. Similarly, as building plan configuration changes from regular square shaped model (2) to irregular L-shaped model (3) and (4), the value of maximum column moment [M2] is increased by 1.88% &7.13% respectively.

Furthermore, as column locations changes form side column in model (1) to corner column in model (5) the value of maximum column moment (M2) is decreased by 31.19%.

Table 5.4C1 Maximum column moment M3 [MX] [KNm] for multi-story models 1&5 on axis B2 @ 2nd story [at bottom]

	Max Column Moment	Max Column Moment
	Middle column[C2-238]	Re-entrant column[C16-237]
Angle	MODEL (1) M3 [KNm]	MODEL (5) M3 [KNm]
0	22.0172	20.4305
15	18.4205	19.9734
30	20.4961	23.7427
45	25.344	27.0261
60	28.4647	**28.4678**
75	**29.6456**	27.9694
90	28.8062	25.5649
105	26.0037	21.4182
120	21.8576	21.767
135	24.5866	23.564
150	25.6401	23.7552
165	24.9462	25.0111
180	24.4964	25.1317

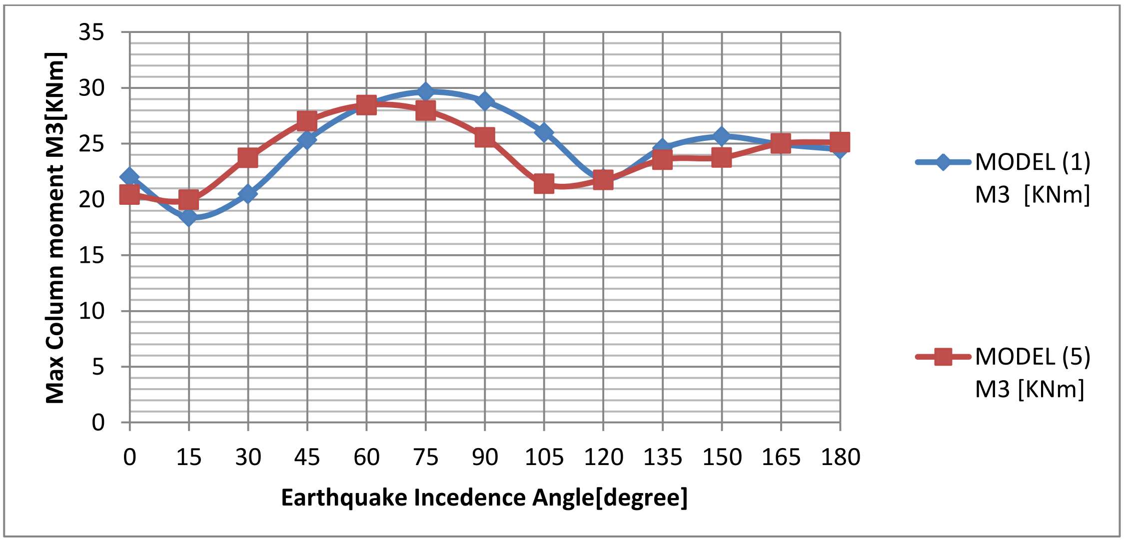


Fig 5.4C1: Variation of maximum column moment M3 with incidence angle for multi-story models1&5 on axis B2 @ 2nd story [at bottom]

Table 5.4C2 Maximum column moment M2 [MY] [KNm] for multi-story models 1&5 on axis B2 @ 2nd story [at bottom]

	Max Column Moment	Max Column Moment
	Middle column[C2-238]	Re-entrant column[C16-237]
Angle	model 1 M2 [KNm]	model 5 M2 [KNm]
0	27.6233	25.5777
15	29.1455	25.8924
30	28.6815	24.4426
45	26.2629	24.5276
60	25.5513	25.2159
75	24.8464	24.1857
90	22.4483	21.6983
105	19.84	21.4798
120	22.6768	24.5425
135	27.0469	27.5773
150	29.5738	**28.7328**
165	**30.0853**	27.9302
180	28.5466	25.2242

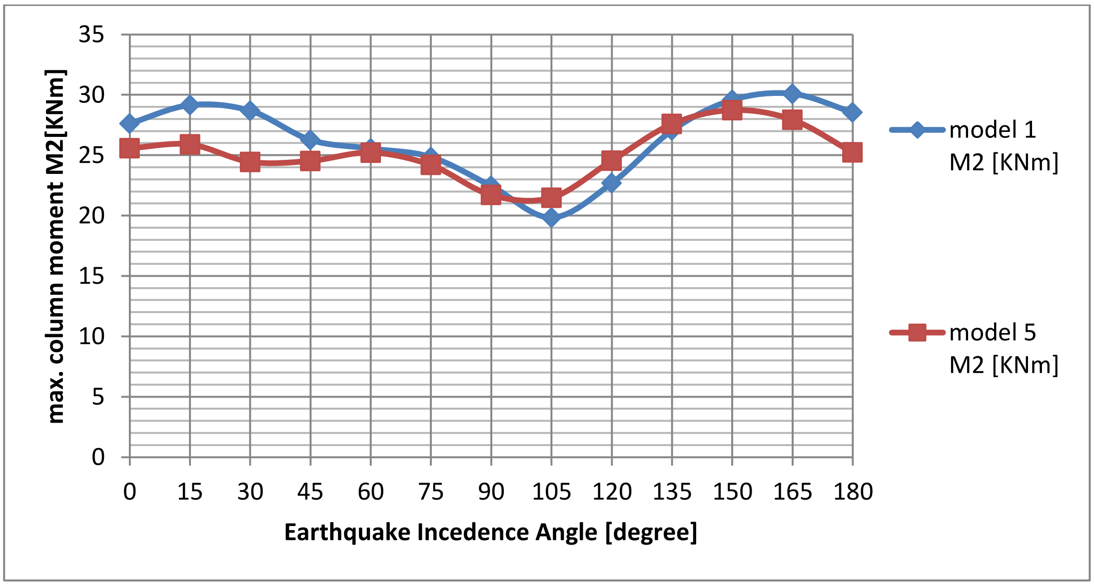


Fig 5.4C2: Variation of maximum column moment M2 with incidence angle for multi-story models 1&5 on axis B2 @ 2nd story [at bottom]

As shown in table and figure 5.4C, for multi-story regular shaped rectangular building model (1), the maximum bending moment MX (M3) & MY (M2) of middle column on axis B2 @ 2nd story has occurred when the peak ground acceleration is applied @ 75 & 165 degrees, the value increased by 25.7% & 8.2% with respect to 0 degrees respectively. While, for multi-story irregular shaped building model (5) the maximum bending moment MX (M3) & MY (M2) of **re-entrant corner column** on axis B2 @ 2nd story occurred when the peak ground acceleration is applied @ 60 & 150degrees, the value is increased by 28.23% & 10.98% with respect to 0 degrees respectively.

Furthermore, considering critical incidence angle values of each building models, as column location changes form middle column in model (1) to re-entrant corner column in model (5) the value of maximum column moment (M3&M2) is decreased by 3.97% & 4.5% respectively.

Summary: It is observed that the single and multi-story systematic and asymmetric buildings have shown considerable increase in maximum column moment when the peak ground acceleration is subjected at various incidence angles with respect to the axes of building plan.

Additionally, considering critical incidence angle values of all models, reentrant corner plan irregular buildings have shown considerable variations in maximum column moment due to the change in building plan configurations & column locations when compared with the corresponding regular building.

For single story model, as building plan configurations changes form regular square shaped in model (1) to irregular L-Shaped in model (2) the value of maximum moment M3&M2 for corner columns on axis A1 is decreased by 18.62% and 16.23% respectively.

While, for multi-story model, as building plan configurations changes form regular square shaped model (2) to irregular L-shaped buildings model (3) and model (4), the value of maximum moment (M3, M2) for side columns is increased by (4.53%, 1.88%) and (7.98%, 7.13%) respectively with respect to the same side column in model 2 on axis A2 @ 2nd story. Similarly, as building plan configurations changes form regular square shaped model (2) to regular rectangular shaped model (1) the value of maximum moment M3&M2 for side columns is increased by 5.48% and 5.60% respectively.

Furthermore, as column changes form side column in model (1), on axis A2 @ 2nd story, to corner column in model (5) the value of maximum column moment M3 & M2 are decreased by 16.62% and 31.19% respectively. While, as the column changes form middle column in model (1), on axis B2 @ 2nd story, to re-entrant corner column in model (5) the value of maximum column moment M3 and M2 is decreased by 3.97% & 4.5% respectively.

Therefore, reentrant corner plan irregular buildings have shown considerable variation in maximum column moment due to the change in seismic incidence angle, building plan configurations & column locations.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

In this research work, the methodology to estimate the possible critical angle of seismic incidence and its corresponding structural demand were investigated in the context of Liner time history analysis, to assess the seismic behavior of reinforced concrete buildings under the different orientations of the seismic excitation. To do so, different 3D computer models of single and four-story symmetric and asymmetric reinforced concrete buildings were generated with the aspect of change in plan configurations. Then the seismic behavior of reinforced concrete buildings under the different orientations of seismic action in range of 0^0 to 180^0 @ 15^0 increments was studied. The influence of seismic incidence angle (ASI) and change in plan configurations on the potential seismic behaviors of asymmetric reinforced concrete buildings with re-entrant corner plan configurations was evaluated and compared with corresponding regular building, with help of the maximum global and local structural elastic demand (like: maximum story drift, maximum column axial force, torsion and bending moment).

Therefore, based on the analysis result, evaluation and discussion made in the previous chapter, the following conclusions are drawn:

1. It is concluded from the study that regular and irregular buildings have shown considerable increase in maximum column axial forces, moment and torsion (but torsion effect is only for irregular building) when the peak ground acceleration is subjected at various incidence angles. But, relatively there has been slight variation in maximum story drift.
2. It is observed from the study that, the structural demand produced varies as a function of input seismic incidence angle and the highest demand occurs in the most flexible structural direction. For multi-story irregular shaped building model (5) the maximum moment MX & MY of re-entrant corner column on axis B2 @

2nd story has occurred when the PGA is applied @ 60 & 150 degrees, the value is increased by 28.23% & 10.98% with respect to 0 degrees respectively.

3. But, each structural model gets its maximum value of column axial forces, torsion and moment at a specific angle of seismic excitation which is different for different shapes of building as well as from column to column. This implies that, there is no unique ASIcri for a given structure that maximizes all engineering demand parameter (EDPs) simultaneously, the critical angle of seismic incidence depends on the chosen engineering demand parameter, structural configurations & locations, and other parameter like specific characteristics of ground motions.
4. Additionally, considering ASIcri values of all models, asymmetric buildings with re-entrant corner plan configurations have shown considerable variation in maximum column axial forces and moment due to the change in building plan configurations & column locations. But, among various internal forces irregular buildings with re-entrant corner plans are more sensitive to the maximum column torsion when compared with the corresponding regular building.
5. For multistory model (1) and (5); As column changed from side column in model (1) to corner column in model (5) the values of maximum column moment MX&MY is decreased by 16.62%. & 31.19% respectively. This indicates that, the column is weaker when it served as corner or re-entrant corner column in model (5) than as side or middle column in model (1). Because, for moment frame building the resistance capacity to lateral load and lateral deflection depends on moments in the column.
6. Thus, the angle of seismic incidence and change in building plan configurations & column locations considerably influences the behavior of asymmetric buildings with re-entrant corner plan configurations.
7. Generally; since angle of seismic incidence considerably influences the response of RC structures, it is reasonable as structural analysis and design criteria to consider the structural demands from all possible values of the seismic angle relative to the structure, to ensure seismic performance assessment and design safety verification.

6.2 Recommendations for Future Study

The results presented herein have important implications for current earthquake engineering practice, suggesting that ground motions applied to FN/FP directions may not necessarily provide the most critical estimates of EDPs. Additionally, for a given record the seismic incident angle leading to maximum elastic response may be different from that for maximum inelastic response for the same response quantity. Therefore, Non Linear Time History Analyses (NLTHA) of this study under similar scenarios is open for future study. Furthermore, among the possibilities for future study, the following are the main points deserve attention.

1. A study on the seismic behavior of asymmetric RC building with varies re-entrant corner plan configurations per story under different orientations of bidirectional seismic excitation in the range of [0,360] @ 15 degree increment angle in context of NLTHA under different earthquake data (as per new EBCS) by considering gravity load and compare the results with nonlinear static analyses is left for future investigation.

2. A study on the seismic behavior of asymmetric RC building both in plan and elevation, under different orientation of bidirectional seismic excitation in the range of [0,360] @ 15 degree increment angle in context of NLTHA under different earthquake data (as per new EBCS) by considering gravity load and compare with nonlinear static analyses can be considered for future study.

REFERENCES

[1] American Society of Civil Engineers (2008).ASCE/SEI41-06, Seismic Rehabilitation of Existing Buildings, Reston, Virginia.

[2] Athanatopoulou, A. M. (2005). Critical orientation of three correlated seismic components. Eng. Struct., Vol. 27, pp. 301-312, DOI: 10.1016/j.engstruct.2004.10.011.

[3] Athanatopoulou, A. M. and Avramidis, I. E. (2006). Effects of seismic directivity on structural response. Proc., Second FIB Congress, Naples, Italy.

[4] C. Cantagallo, G. Camata & E. Spacone, (2012). Effect of earthquake incidence angle on seismic demand of RC structures. University G. D Annunzio of Chieti-Pescara, Italy

[5] Chopra A.K. (1995).Dynamics of structures: theory and applications to earthquake engineering. New Jersey: Prentice-Hall.

[6] Computers and Structures, Inc. (2016). CSI Analysis reference manual for SAP2000, ETABS, SAFE and CSiBridge, Berkeley, California.

[7] Despoina skoulidou and Xavier Roma, (2016). Determination of the angle of seismic incidence in standardized procedures for the seismic safety assessment of 3D RC buildings.

[8] Despoina Skoulidou and Xavier Romao, (2017). Critical orientation of earthquake loading for building performance assessment using lateral force analysis.

[9] Euro code 8 (1998-1:2004). Design of structures for earthquake resistance, European Committee for Standardization, English version.

[10] Ethiopian Building Code Standard (ES EN1998:1-2015). Design of structures for earth quake resistance. Addis Ababa, Ethiopia.

[11] Fernandez Davila, Cominetti.S and etal, (2000). Considering the bi-directional effects and the seismic angle variations in building design.12th World Conference on Earthquake Engineering, E.Q.C., Auckland, paper 0435.

[12] Julio J Hernandez and Oscar A Lopez, (2000). Influence of bidirectional seismic motion on the seismic response of asymmetric buildings. 12th World Conference on Earthquake Engineering, E.Q.C., paper 1813.

[13] Konstantinos.G, and et al, (2017). Influence of seismic incident angle on response of symmetric in plan buildings. Bull Earthquake Eng DOI 10.1007/s10518-017-0176-9.

[14] Magliulo G, and etal, (2014). Influence of earthquake direction on the seismic response of irregular plan RC frame buildings. Earthq Eng Eng Vib 13(2):243–256.

[15] Momen M. M. Ahmed, and A. Abdel-Shafy, (September 2016). Irregularity effects on the seismic performance of L-shaped multi story buildings. Journal of Engineering Sciences Assiut University Faculty of Engineering Vol. 44 No.5 PP. 513-536.

[16] Morfidis, K. E., A. M. Athanatopoulou and I. E. Avramidis, (2008). Effects of seismic directivity within the framework of the lateral force procedure.14th World Conference on Earthquake Engineering Beijing, China.

[17] M.sri kanya, & BDV Chandra Mohan, (2015). Effect of earth quake incidence angle on seismic performance of RC Buildings. IJRET Volume 04 No.13 PP. 2319-1163.

[18] Oscara .Lo pez and Ronald Torres, (1997). The critical angle of seismic incidence and the maximum structural response. Earthquake Engineering and structural dynamics, VOL. 26, No. 9, PP. 881-894.

[19] Quadri, S. A. and M. Madhurin, (2014). Investigation of critical angle of seismic incidence for the analysis of RCC frames. 12th IRF International Conference, Pune, India.

[20] Ramchandani J and Mangulkar M, (2016). Investigation of critical angle of seismic incidence for the analysis of RCC frames by time history method of dynamic analysis using STAAD PRO International Journal of Modern Engineering Research, India.

[21] Taranath, Bungale S., (2010). Reinforced concrete design of tall buildings. Taylor and Francis Group, LLC, New York, USA.

APPENDIX A

Summary of ASIcri and the percentage variation of structural response

The following table shows the variations of structural demands (i.e. Maximum story drift, maximum column axial force, torsion, and bending moment) results from LTHA, with respect to ASI and change in building plan configurations.

Table A1.Summary in percentage variation of maximum story drift with respect to the value @ 0 degree and corresponding regular plan for single story model at 1st story

Building model	Critical Angle (degree)	Max.governing storydrift @critical angle	Max.governing storydrift @0degrees	% Variation wrt 0 degree	%Variation wrt matching regular plan configuration
model 1	30/120	0.000298	0.000272	8.72	0
model 2	165	0.000249	0.000167	32.93	16.44

Table A2.Summary in percentage variation of maximum story drift with respect to the value @ 0 degree and corresponding regular plan for multistory models at 2nd story

Building model	Critical Angle (degree)	Max.gov. Story drift @critical angle	Max.gov. Story drift @0degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan
model 1	15	0.001013	0.000971	4.15	2.17
model 2	30	0.000991	0.000916	7.57	0
model 3	15	0.001048	0.000991	5.44	5.44
model 4	75	0.001024	0.000981	4.20	3.22
model 5	150	0.001043	0.000999	4.22	2.88

Table A3.Summary in percentage variation of maximum column axial force P [KN] with respect to the value @ 0 degree and corresponding regular plan for single story model @ axis A1 on 1st story

Building model	Column Category	Critical Angle (degree)	Max.Axial force[kN] @critical angle	Max.Axial force[kN] @0degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Corner [C1-5]	75	4.6616	2.6544	43	0
model 2	Corner [C1-5]	105	4.5164	2.6911	40.4	3.11

Table A4.Summary in percentage variation of maximum column axial force P[KN] wrt the value @ 0 degree and corresponding regular plan for multi-story model @ axis A2 on 2nd story

Building model	Column Category	Critical Angle (degree)	Max.Axial force[kN] @critical angle	Max.Axial force[kN] @0degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Side [C1-234]	60	23.8366	17.4639	26.73	4.93
model 2	Side [C3-493]	60	22.6618	16.2161	28.44	0
model 3	Side [C3-317]	60	23.8921	17.4687	26.89	5.15
model 4	Side [C6-203]	60	24.3894	17.9733	26.31	7.1
model 5	Corner [C2-155]	105	27.6126	21.6009	21.77	13.67

Table A5.Summary in percentage variation of maximum column axial force P [KN] with respect to the value @ 0 degree and corresponding regular plan for multistory model 1&5 on axis B2 @ 2nd story

Building model	Column Category	Critical Angle (degree)	Max.Axial force[kN] @critical angle	Max.Axial force[kN] @0degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Middle [C2-238]	135	5.5235	4.3871	20.57	0
model 5	Reentrant Corner [C16-237]	105	4.8214	3.9482	18.11	12.71

Table A6.Summary in percentage variation of maximum column torsion (T) [KNm] with respect to the value @ 0 degree and corresponding regular plan for single-story model @ axis A1 on 1st story

Building model	Column Category	Critical Angle (degree)	Max.Torsion [kNm]@ critical angle	Max.Torsion [kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Corner [C1-5]	0	0	0	0	0
model 2	Corner [C1-5]	0	1.1076	1.1076	0	100

Table A7.Summary in percentage variation of maximum column torsion (T) [KNm] with respect to the value @ 0 degree and corresponding regular plan for multi-story models on axis A2 @ 2nd story

Building model	Column Category	Critical Angle (degree)	Max.Torsion [kNm] @critical angle	Max.Torsion [kNm] @0degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Side [C1-234]	0	0	0	0	0
model 2	Side [C3-493]	0	0	0	0	0
model 3	Side [C3-317]	0	0.1289	0.1289	0	100
model 4	Side [C6-203]	120	0.0652	0.0379	41.87	100
model 5	Corner [C2-155]	105	0.157	0.1422	9.43	100

Table A8.Summary in percentage variation of maximum column torsion (T) [KNm] with respect to the value @ 0 degree and corresponding regular plan for multi-story models 1&5 on axis B2 @ 2nd story

Building model	Column Category	Critical Angle (degree)	Max.Torsion [kNm]@ critical angle	Max.Torsion [kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Middle [C2-238]	0	0	0	0	0
model 5	Reentrant corner [C16-237]	105	0.157	0.1422	9.43	100

Table A9.Summary in percentage variation of maximum column moment M3 [MX] [KNm] with respect to the value @ 0 degree and corresponding regular plan for single-story models on axis A1 @ 1st story [@bottom]

Building model	Column Category	Critical Angle (degree)	Max.Moment M3[kNm] @ critical angle	Max.Moment M3[kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Corner [C1-5]	30	13.5699	12.0456	11.23	0
model 2	Corner [C1-5]	180	11.0432	5.8526	47	18.62

Table A10.Summary in percentage variation of maximum column moment M2 [MY] [KNm] with respect to the value @ 0 degree and corresponding regular plan for single-story models on axis A1 @ 1st story [@bottom]

Building model	Column Category	Critical Angle (degree)	Max.Moment M2[kNm] @ critical angle	Max.Moment M2[kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Corner [C1-5]	120	13.5314	12.3307	8.9	0
model 2	Corner [C1-5]	165	11.3347	7.4779	34.03	16.23

Table A11.Summary in percentage variation of maximum column moment M3 [MX] [KNm] with respect to the value @ 0 degree and corresponding regular plan for multi-story models on axis A2 @ 2nd story [@ bottom]

Building model	Column Category	Critical Angle (degree)	Max.MomentM3 [kNm] @ critical angle	Max.MomentM3 [kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Side [C1-234]	75	19.4092	13.0446	32.79	5.48
model 2	Side [C3-493]	75	18.345	12.3191	32.85	0
model 3	Side [C3-317]	75	19.2163	13.0346	32.17	4.53
model 4	Side [C6-203]	60	19.9365	13.5375	32.1	7.98
model 5	Corner [C2-155]	60	16.1841	11.3536	29.85	16.62

Table A12.Summary in percentage variation of maximum column moment M2 [MY] [KNm] with respect to the value @ 0 degree and corresponding regular plan for multi-story models on axis A2 @ 2nd story

Building model	Column Category	Critical Angle (degre)	Max.MomentM2 [kNm] @ critical angle	Max.MomentM2 [kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Side [C1-234]	165	24.9476	22.2372	10.9	5.605
model 2	Side [C3-493]	165	23.5494	20.5315	12.8	0
model 3	Side [C3-317]	165	24.0007	21.1205	12.0	1.88
model 4	Side [C6-203]	150	25.3576	22.678	10.6	7.13
model 5	Corner	135	17.166	13.5141	21.3	31.19

Table A13.Summary in percentage variation of maximum column moment M3 [MX] [KNm] with respect to the value @ 0 degree and corresponding regular plan for multi-story models 1&5 on axis B2 @ 2nd story [@ bottom]

Building model	Column Category	Critical Angle (degre)	Max.Moment M3[kNm] @ critical angle	Max.Moment M3[kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Middle [C2-238]	75	29.6456	22.0172	25.73	0
model 5	Reentrant [C16-237]	60	28.4678	20.4305	28.23	3.97

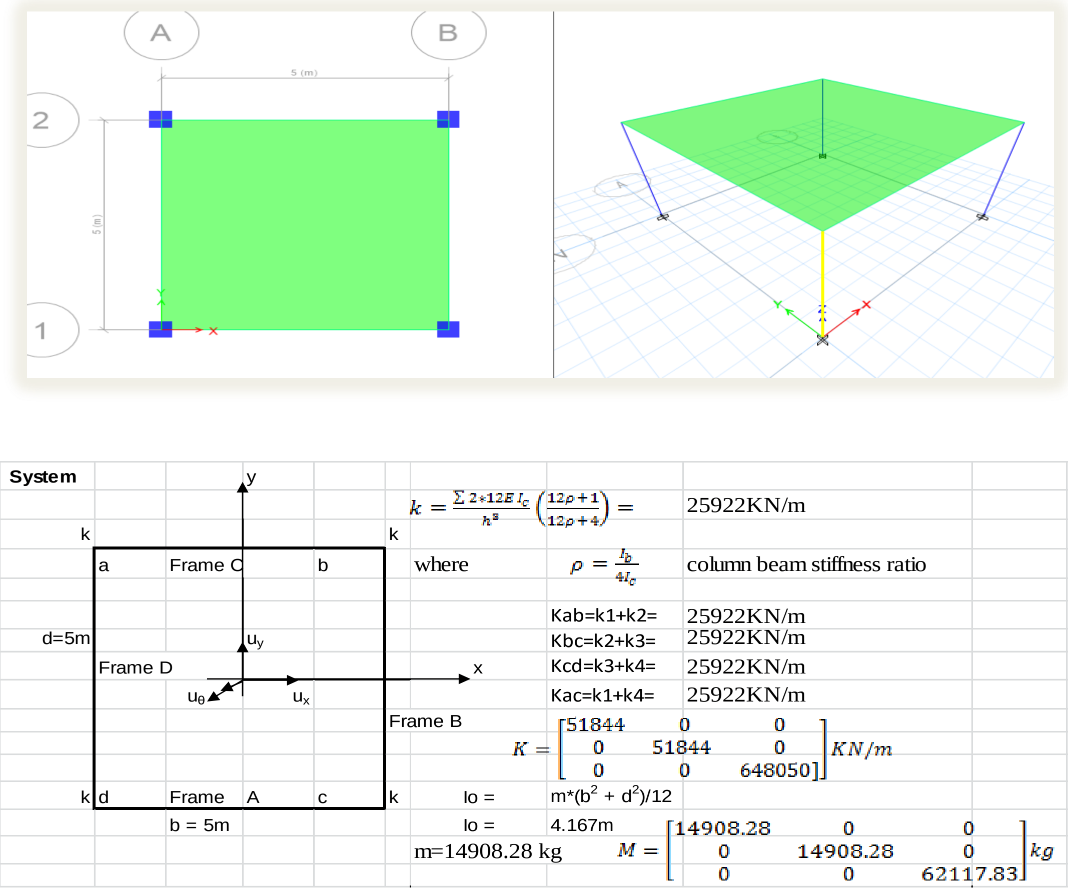
Table A14.Summary in % variation of max.column moment M2[KNm] wrt the value @ 0^0 and corresponding regular plan for multi-story models 1&5 on axis B2 @ 2nd story

Building model	Column Category	Critical Angle (degree)	Max.Moment M2[kNm] @ critical angle	Max.Moment M2[kNm] @ 0 degrees	% Variation wrt 0 degree	%Variation wrt corresponding regular plan configuration
model 1	Middle [C2-238]	165	30.0853	27.6233	8.2	0
model 5	Reentrant [C16-237]	150	28.7328	25.5777	10.98	4.5

APPENDIX B

Verification of the Analysis Results

The single-story, three-dimensional, symmetric building model with 5m by 5m rigid floor system is used here as a set of test-bed structure and benchmark to verify software results using ESA and modal dynamic analysis manual (theoretical) solution.



$k = \frac{\sum 2*12EI_c}{h^3}\left(\frac{12\rho+1}{12\rho+4}\right) =$ 25922KN/m

where $\rho = \frac{I_b}{4I_c}$ column beam stiffness ratio

Kab=k1+k2= 25922KN/m

Kbc=k2+k3= 25922KN/m

Kcd=k3+k4= 25922KN/m

Kac=k1+k4= 25922KN/m

$$K = \begin{bmatrix} 51844 & 0 & 0 \\ 0 & 51844 & 0 \\ 0 & 0 & 648050 \end{bmatrix} KN/m$$

Io = m*(b^2 + d^2)/12

Io = 4.167m

m=14908.28 kg

$$M = \begin{bmatrix} 14908.28 & 0 & 0 \\ 0 & 14908.28 & 0 \\ 0 & 0 & 62117.83 \end{bmatrix} kg$$

Fig B1; Single-story, three-dimensional, symmetric building model

- ✓ Equivalent static analysis

Fundamental period (T): $T = C_t(H)^{\frac{3}{4}}$= 0.075*3 $^{3/4}$ =0.171sec;

For soil type C and type1 elastic response spectra; Since T_1=0.171 sec is between 0 and T_b (T_b= 0.2 sec; Tc=0.6sec, Td=2 sec, S=1.15, ag=0.15g zone4 & q=3.3).

So, $\mathbf{0 \leq T \leq T_B : S_d(T)}$; Sd(T) $= \boldsymbol{a_g . S}\left[\frac{2}{3}+\frac{T}{T_B}\left(\frac{2.5}{q}-\frac{2}{3}\right)\right]$=1.26m/s^2.

Base shear (Fb) =1.26*19801.22Kg *1.0 = 24.95KN

Peak story shear =$F_i = \frac{F_b * Z_i m_i}{\sum Z_j * m_j}$ = 24.95KN

Displacement @ CM is; $\Delta = Fi/k$ = (24.95 KN)⁄(51844KN/m)= 0.4815mm.

Drift== $\frac{\Delta}{h}$=0.0001604

- Modal analysis

$$M = \begin{bmatrix} m & 0 & 0 \\ 0 & m & 0 \\ 0 & 0 & I_0 \end{bmatrix}$$

[m] = [14908.28 0 0; 0 14908.28 0; 0 0 62117.83] kg,

$$K = \begin{bmatrix} K_{xx} & K_{xy} & K_{x\theta} \\ K_{yx} & K_{yy} & K_{y\theta} \\ K_{\theta x} & K_{\theta y} & K_{\theta\theta} \end{bmatrix} = \begin{bmatrix} K_{ab}+K_{cd} & 0 & 0 \\ 0 & K_{ad}+K_{bc} & 0 \\ 0 & 0 & \frac{a^2}{4}(K_{yB}+K_{yA}+K_{xA}+K_{xD}) \end{bmatrix}$$

[k] = [51844000 0 0; 0 51844000 0; 0 0 648050000] N/m.

$W_n = \sqrt{k/m}$; = (58.970591, 58.970591 and 102.14006) rad/sec, $T_n = 2\pi/W_n$; T_n= (0.1065446, 0.1065446 and 0.0615136) sec and $Fi = -m\iota\ddot{u}_g(t)$ = F= (21.9375, 21.9375, 0) KN

- ETABS Results

TABLE B1: Centers of Mass and Rigidity

Story	Mass kg	Stiffness KN/m	CM (m)	CR (m)
Story1X	16927.39	32716.363	2.5	2.5
Story1Y	16927.39	32716.363	2.5	2.5

TABLE B2: Modal Periods and Frequencies

Case	Mode	Period	Frequency	Circular Frequency	Eigenvalue
		sec	cyc/sec	rad/sec	rad^2/sec^2
Modal1	1	0.143	6.991	43.9241	1929.3287
Modal1	2	0.143	6.991	43.9241	1929.3287
Modal1	3	0.035	28.694	180.2874	32503.5633

TABLE B3: Story Stiffness

Story	Load Case	Shear	Stiffness	Displacement X	Displacement Y
		kN	kN/m	mm	mm
Story1	EQX1	21.3154	32716.363	0.652	0.028
Story1	EQY1	21.3154	32716.363	0.028	0.652

TABLE B4: Diaphragm Center of Mass Displacements

Story	Diaphragm	Load Case/Combo	UX	UY	RZ
			mm	mm	rad
Story1	Rigid DP1	EQX1	0.652	0	0.000011
Story1	Rigid DP1	EQY1	0	0.652	-0.000011
Story1	Rigid DP1	LCase 0 TH Max	0.802	0.815	1.336E-11

TABLEB5: Diaphragm Max/Avg Drifts

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
Story1	EQX1	Diaph Rigid DP1 X	0.000227	0.000217	1.044
Story1	EQY1	Diaph Rigid DP1 Y	0.000227	0.000217	1.044
Story1	LCase 0 TH Max	Diaph Rigid DP1 X	0.000267	0.000267	1
Story1	LCase 0 TH Max	Diaph Rigid DP1 Y	0.000272	0.000272	1

Table B6: Summary of theoretical and ETABS results

Quantity	Theoretical	Theoretical	ETABS	ETABS	ETABS
	ESA	Modal Analysis	ESA	LTHA @0 degree	LTHA @ critical angle
T(sec)	0.171	0.106545	0.143	0.143	0.143
Fb (KN)	24.95	21.9375	21.3153	26.6087	29.176 @30^0
D(mm)	0.4815	0.4231	0.6515	0.8146	0.8929 @30^0
Drift	0.0001604	0.000141	0.000272	0.000272	0.000298@30^0

The ESA hand computation seems to overestimate the fundamental period and base shear value with respect to ETBS results. Here the fundamental period is only depends on the height of structure rather than its mass and stiffness. This is accounted to the increase in the fundamental period by about 16.4% by hand computation. While in modal analysis case the fundamental period is depend on the mass and stiffness of structures. Thus, modal analysis overestimates the stiffness of the structure and gives smaller fundamental period, top story displacement and story drift. But, the ESA software results underestimates the base shear and stiffness of the structure and gives larger values of top story displacement and story drift than the hand calculation by about 26 &29% respectively. The values of maximum top story displacement and story drift from ESA software result is decreased by 20% & 8.7% respectively when compared with THA values @ ASIcrit.

- Comparisons of maximum column moment (M3, M2) for column on axis A1 Using THA (i.e. uni & bi-directional seismic excitation) method with RSA & 100/30% combination rule.

Table B7: Summary of % variation of maximum column moment (M3, M2) for column on axis A1 wrt bi-directional seismic excitation method

Method	M3(KNm)	M2(KNm)	%variation Max.Moment wrt bidirectional method
Bi directional	15.95 @30^0	9.43 @120^0	0
$R_{20\%}$ =$(1.2 \times Max\{R^{0^\circ}: R^{90^\circ}\}$	16.25 @30^0	16.25 @120^0	-1.84
RSA	9.718	9.718	39
R_0+0.3R_{90}	10.26	2.881	35
0.3R_0+R_{90}	2.881	10.26	35

It is observed from table B7, the 30% and square root of the sum of the square (SRSS) combination rule underestimate the values of maximum column moment on axis A1 by 35% and 39% respectively when compared with Bi directional seismic excitation method. A rule that consists of amplifying in 20% the response that results higher upon only applying the higher horizontal component of the seismic in each one of the two principal directions of the building, meaning for the analyzed structures the building analysis by an uni-directional seismic input, amplified by a factor of 1.2 gives similar responses to those coming from bi-directional seismic analysis acting in the most unfavorable incidence angle. From this methodology the responses are overestimated and the maximum observed error is of 1.8%. But this is out of the scope of this thesis work.