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JIMMA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

STRUCTURAL ENGINEERING STREAM

EFFECT OF DUCTILITY CLASS ON THE SEISMIC PERFORMANCE OF IRREGULAR REINFORCED CONCRETE FRAME STRUCTURES ACCORDING TO ES-EN PROVISION

A Thesis Submitted to the Graduate School of the Jimma University in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structural Engineering)

By

Ali Hassen Feleke

July , 2021

Jimma, Ethiopia

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Ali Hassen Feleke

Advisor: Engr. Elmer C. Agon (Asso. Prof)

Co-adviser: Engr. Eden Shukri (MSc.)

July , 2021

Jimma, Ethiopia

DECLARATION

I hereby declare that this thesis entitled "**Effect of ductility class on the seismic performance of irregular reinforced concrete frame structures according to ES-EN provision**" was composed by myself, with the guidance of my advisor, that the work contained herein is my own except where explicitly stated otherwise in the text, and that this work has not been submitted, in whole or in part, for any other degree or professional qualification.

Ali Hassen	________________.	____________________.
Name of Student	Signature	Date

This thesis has been submitted for the examination with my approval as a university supervisor.

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Co. Advisor	Signature	Date

ABSTRACT

Earthquake is one of disastrous consequences and vulnerability of inadequate structures. The main cause of failure of multi-storey multi-bay reinforced concrete frames during seismic motion was in appropriate design of structures for ductility class. This cause in addition to irregularity (pane or vertical) future the failure is high so the effect of the ductility class on the performance of seismic load of vertical geometric irregular reinforced concrete structure should be evaluated. In this study, the effect of ductility classes on the seismic performance of reinforced concrete frame structures was investigated in detail. In Ethiopian new code ES EN-1998-1-2015, three types of ductility classes were provided with their respective requirements. And to know the effect of ductility classes on the seismic performance of the structure, design of geometrical vertical irregular reinforced concrete structures will be done according to the new code capacity rule provision for each ductility classes. The evaluation is done by designing vertical geometric irregular structures for different ductility classes with the provision of new seismic code and by checking their performance through non-linear (pushover) analysis using CSI ETABS 2016 V.2.1. So structures designed for high ductility class were found to be better than structures designed for medium ductility regarding to capacity of the structures base shear increased by 7.04%, 12.14%, 5.18%, 13.77%, 12.37%, 5.33%, 4.52% and 8.78% and increased by 2.51%, 22.05%, 21.23%, 6.67%, 4.91%, 0.33%, 3.51% and 0.69% along in the X and Y whereas for inter storey drift the medium ductility class was performed better resistance than high ductility class. This research is done only for moment resisting frames, the evaluation of the effect of ductility class on the performance of reinforced concrete for dual and wall systems is recommended. As well as evaluation of the effect of ductility class on the performance of reinforced concrete for irregular structures (stiffness, mass and the combination of both) uses statics nonlinear analysis or dynamic non-linear analysis.

Keywords: *Ductility, Geometric irregularities, top story drift, inter story drift and plastic hinge.*

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ABBREVIATIONS

ψ_{Ei}	Combination coefficient for a variable action i, to be used when determining the effects of the design seismic action
$\psi_{2,I}$	Combination coefficient for the quasi-permanent value of a variable action i
Tc	Corner period at the upper limit of the constant acceleration region of the elastic spectrum
q	Behavior factor
a_g	Design ground acceleration on type A ground
DCM	Ductility Class Medium
DCH	Ductility Class High
ES	Ethiopian Standard
T_1	Fundamental period of vibration of a building
NSA	Nonlinear Static Analysis
γ_{Rd}	Over strength factor
RC	Reinforced Concrete
γ	Unite weight of materials
ULS	Ultimate Limit State

CHAPTER ONE

INTRODUCTION

1.1 Background

Earthquakes are most unpredictable and devastating of all-natural disasters. Earthquakes have the potential for causing the greatest damages among all the natural hazards. Since earthquake forces are random in nature and unpredictable. They not only cause great destruction in human casualties, but also have a tremendous economic impact on the affected area. The concern about seismic hazards has led to an increasing awareness and demand for structure designed to withstand seismic forces. When a structure is subjected to ground motions in an earthquake, it responds by vibrating. Those ground motion causes the structure to vibrate or shake in all three directions; the predominant direction of shaking is horizontal. During an earthquake, the damage in a structure generally initiates at location of the structural weakness present in the building systems. High-Rise RC structures are a special class of structures with their own peculiar characteristics and requirements. These structures are often occupied by a large number of people. Thus, their damage, loss of functionality, or collapse can have very severe and adverse consequences on the life and on the economy of the affected regions. Each high-rise structure represents a significant investment and as such high-rise structure analysis is generally performed using more sophisticated techniques and methodologies. Thus, to understand modern approaches for seismic analysis of high-rise RC structures are valuable to structural engineers and researchers. In the modern era, most of the structures are delineated by irregular in both plan and vertical configurations. Moreover, to analyze or design such irregular structures high level of effort is needed.(Pawade, *et.al*, 2018)

Many structures are designed with vertical irregularities due to functional, aesthetic, or economic reasons. Vertical irregularities are due to sudden changes in stiffness, strength and/or mass between adjacent stories. Sudden changes in stiffness and strength between adjacent stories are associated with changes in structural system along the height, changes in story height, setbacks, changes in materials and unanticipated participation of non-structural components (Das, 2000). Many structures have suffered unexpected damage or collapse due to these types of discontinuities.

The modern design approach allows some local failure in pre-determined structural members to occur during an earthquake event, where extra ductility has been provided. The formation of plastic hinges at these places enables the dissipation of energy and re-distribution of moments and forces. Contrary to the classic design the energy released from the earthquake is not totally transformed to kinetic energy of the building and the structural response in terms of lateral acceleration and shear forces is not as high. Although that the modern earthquake resistant construction techniques through the correct detailing, the design checks and limits are being provided by the codes, there is not any clear guidance for obtaining the optimum structural performance. A high (DCH) and a medium ductility class (DCM) multi-storey building has been designed according to Euro code 8 in this study. (SALAWDEH S., 2009)

Vertical geometric irregular reinforced concrete structures designed for different ductility classes have different performance characteristics. To make such evaluation, pushover analysis which is a nonlinear method performance based method has been developed by structural engineers. Pushover analysis determines expected seismic performance of structural system by estimating its strength and deformation demands in design earthquake. Also this method determines the capacity curve for the building based on series of incremental static analysis. Based on this capacity curve, a targeted displacement (an estimation of design displacement during earthquake) is determined. These results are to be determined and for assessing the importance of ductility towards the seismic load and the effect of the ductility class on the performance of the vertical geometric irregular reinforced concrete structure to be assess in this study.

1.2 Problem of statement

Recent modern seismic codes provisions for structural buildings rely on energy dissipation through inelastic deformation during the designed seismic load to the necessity to compromise damages with economic consideration. This energy dissipation based on the ductile properties of the structure due to the material. In order to ensure that the designed resistance of the structure will be maintained with negligible decay, it is of high importance to perform correct detailing and provide appropriate selection of the ductility class of the structure.

Vertical irregularities nowadays have a lot of interest in seismic research investigations. This study will concentrate on the design of this type of structures for ductility class according to the new Ethiopian building code of standard.

So this research conducted to check of the effect of each ductility classes on performance of a newly designed vertical geometric irregular reinforced concrete frame structures to design the structure and to select the optimum ductility class according to the new Ethiopian code ES-EN 1998-1-2015 with the aid of nonlinear static (i.e., pushover) analysis and ETABS 2016.2.1 software to be used as a guide in the future design of Earthquake resistance structure.

1.3 Research Questions

This research will be mainly focus to answer the following research questions**:**

- How the plastic mechanism is formed and distribution of critical regions (plastic hinges) looks like?
- What is the relationship between demand curve and capacity curve in performance level evaluation?
- What are the parameters used to evaluate the performance of the structures in static non-linear analysis?
- What are the effects of each ductility classes on the performance of RC structures?

1.4 Objectives

1.4.1 General Objective

The general objective of this study was to assess the effect of ductility classes on the performance of vertical geometric irregular RC frame structure according to the provision of ES EN 1998-1-2015.

1.4.2 Specific Objectives

While the assessment of the performance of RC frame structure designed with different ductility classes according to provision of ES-EN 1998-1-2015 the following specific objectives have been carried out:

- To understand the expected plastic mechanism formation and distribution of plastic hinges.
- To discuss the relationship between capacity curve and demand curve in performance evaluation.
- To understand and discuss the parameters used to determine the performance of the structures in static non-linear analysis.
- To compare the seismic effect of ductility class medium and ductility class high of selected buildings.

1.5 Significance of the Study

Significant of this study is to figure out influence of ductility class on the seismic performance of vertical geometric irregular reinforced concrete structures. And also this study is providing a detail concept and design procedure of capacity design philosophy according to the new Ethiopian code ES-EN-1998-1-2015. The performance evaluation of structure is always to plan earthquake engineering to protect undesirable failures and to serve the desired function without any failure signs throughout its design life. So this study provides how the ductility classes are influence performance of the structure under seismic loading.

This study provides clear design procedure of this method by referring some international codes in addition to our new code and how it also improves performance of the structure. This study also provides the analysis procedure and clear concepts of nonlinear static analysis.

The new Ethiopian code ES-EN-1998-1-2015 as a provision code for seismic design in Ethiopia, ES-EN-1998-1-2015 recommends three ductility classes for seismic design, however it is not clear for the effect of the ductility class on the performance of the building. It is not easy decision to decide suitable ductility class for the structure to resist seismic actions in high performance with low.

Generally, the result of this study will give an essential baseline to show the effect of the ductility class and it will also give which ductility class is performing high resistance for the earthquake for vertical geometric irregular structures. These studies also give the detailed concept and design procedure of capacity design philosophy according to ES-EN-1998-1-2015 and distribution of plastic hinges on the structure.

1.6 Scope and Limitation of the Study

The present study be limited to three-dimensional RC frames with vertical geometric irregularities. The stiffness and strength of Infill walls not be considered. The design G+13, G+14, and G+15 structures according to ES-EN 1998-1-2015 provision and on a medium dens soil (Ground-type C, ES- EN 1998-1-2015 soil classification), and the buildings be designed for DCM and DCH. The soil structure interface effects not be considered in the study. The flexibility of floor diaphragms is ignored and considered as stiff diaphragms. The column bases assumed fixed in the study. A none-linear analysis for different ductility classes is done using CSI ETABS 2016 v2.1 software that considers several important effects such as P-delta and stiffens reduction factors.

The study limited to do the regular and vertical irregular structure in stiffness and mass as well as the plane direction of possible irregularity and regularity. In this research, the other irregularity conditions are not considering for the design and analysis of the frame. And also the material irregularity is not considered in the nonlinear pushover analysis of this paper

CHAPTER TWO

RELATED LITERATURE

In multi-storeyed framed buildings, damage from earthquake ground motion generally initiates at locations of structural weaknesses present in the lateral load resisting frames. In some cases, these weaknesses may be created by discontinuities in stiffness, strength or mass between adjacent storeys. Such discontinuities between storeys are often associated with sudden variations in the frame geometry along the height. There are many examples [1, 2] of failure of buildings in past earthquakes due to such vertical discontinuities.

A common form of vertical discontinuity arises from reduction of the lateral dimension of the building along its height. This building category is labelled as ‘stepped’ building in this paper. This building form is becoming increasingly popular in modern multistory building construction mainly because of its functional and aesthetic architecture. In particular, such a stepped form provides for adequate daylight and ventilation for the lower storeys in an urban locality with closely spaced tall buildings… Stepped buildings are characterized by staggered abrupt reductions in floor area along the height of the building, with consequent drops in mass, strength and stiffness (not necessarily at the same rate). Height-wise changes in stiffness and mass render the dynamic characteristics of these buildings different from the ‘regular’ building. Design codes have not given particular attention to the stepped building form. This is perhaps due to the paucity of research on stepped buildings reported in the literature. (Sarkar, Prasad and Menon, 2010)

The seismic response of vertically irregular frames, the subject of numerous research investigations, was reviewed in two recent comprehensive investigations by Valmundsson and Nau (1997) and Al-Ali and Krawinkler (1998), both studies considering mass, stiffness, and strength irregularities separately and in various combinations. The first of these investigations focused on evaluating building code requirements for vertically irregular frame buildings, whereas the latter emphasized the effects of vertical irregularities on height-wise variation of seismic demands and behavior of frame buildings. It was found that among the four types of irregularity, the effect of mass irregularity is the smallest, the effect of strength irregularity is larger than the effect of stiffness irregularity, and the effect of combined-stiffness-and strength irregularity is the largest. The roof displacement was shown to be a stable parameter not affected significantly by vertical irregularities (Al-Ali and Krawinkler 1998).

This Chapter provides a short description of effects of ductility in structural performance, the nature of performance-based earthquake engineering and its goals in seismic assessment and design. The procedures that are recommended for seismic design and assessment purposes are briefly described and their shortcomings are addressed. The theoretical background of the nonlinear static ‘pushover’ analysis method, POA, is then described together with the various pushover analysis procedures. Finally, a review of the state-of-the-art of research on pushover analysis is presented together with general conclusions on the efficiency of the method derived from the literature (THEMELIS, 2008).

2.2 Earthquake resistance building

Structural buildings are designed to carry different types of loads without failure. From the different types of load earthquake motion is the one that consequences destructive failure in buildings if the building is not analyze and design for the probable maximum earthquake force. Structural buildings design for earthquake motion primarily it concerns about the safety of occupants and the safety of the structure in addition to these economy and serviceability requirements also satisfied according to the code of the country. (Jonathan *et al*, 2004)

Earthquake resistance design of structures requires a deep understanding of the structure during earthquake motion and inelastic properties of structures also known. Conventionally the analysis and the design of seismic loading are focused on minimization of risk of loss of life under the probable maximum incoming earthquake motion. (Chen and Lui, 2006)

Recent seismic codes are described different criteria and requirements for the new design and existing structures subjected to earthquake ground motions. The main objective of requirement and criteria specify in the seismic codes are to minimize loss of life, to increase the performance of the structure, to protect drastic failure of the structure and to improve the capacity of the structure after the earthquake motion. (Dowrick, 2009)

In most structural buildings that subjected to moderate to strong earthquake motion economical design of the earthquake resistance building is achieved by allowing yielding in some structural element. If the design is permitted the structure will remain in the elastic range after the earthquake motion the structure becomes uneconomical. In the earthquake resistance design of structures the structure becomes strong and sustain small seismic, allow only small non-structural damage and negligible structural damage under the moderate earthquake and the structure expected large deformation with structural yielding critical regions without collapse. (Sextos, Simopoulos and Skoulidou, 2015)

An effective earthquake resistance design is started from the conceptual design of the structures. In the design process the following important concepts also remained for success full design of the structure under seismic load. (Paulay.T and Priestley M.J.N., 1992).

2.3 Code perspective on irregular building

Same researchers and the EBCS EN 1998, 2014 are describing the difference classifications and condition of the irregular structure acceding plan and elevation irregularity. Thus are

i. Plan and Elevation Irregularity of structures

Buildings shall be classified as regular or irregular based on the criteria in Euro code 8. Such classification shall be based on the plan and vertical configuration. The seismic design of regular buildings is based on two concepts. First, the linearly varying lateral force distribution is a reasonable and conservative representation of the actual response distribution due to earthquake ground motions. Second, the cyclic inelastic deformation demands are reasonably uniform in all of the seismic force-resisting elements. However, when a structure has irregularities, these concepts may not be valid, requiring corrective factors and procedures to meet the design objectives.

According to, Euro Code 8 vertical irregularity due to irregular distributions in their mass, strength and stiffness along the height of building. When such buildings are constructed in high seismic zones, the analysis and design becomes more complicated.

Vertical Irregularities are mainly of five types

i) Stiffness Irregularity:

a) Soft Story-A soft story is one in which the lateral stiffness is less than 70 percent of the story above or less than 80 percent of the average lateral stiffness of the three stories above.

b) Extreme Soft Story-An extreme soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of the three stories above.

ii) Mass Irregularity-Mass irregularity shall be considered to exist where the seismic weight of any story is more than 200 percent of that of its adjacent stories. In case of roofs irregularity need not be considered.

iii) Vertical Geometric Irregularity- A structure is considered to be Vertical geometric irregular when the horizontal dimension of the lateral force resisting system in any story is more than 150 percent of that in its adjacent story.

iv) In-Plane Discontinuity in Vertical Elements Resisting Lateral Force-An in-plane offset of the lateral force resisting elements greater than the length of those elements.

Discontinuity in Capacity-Weak Story-A weak story is one in which the story lateral strength is less than 80 percent of that in the story above. (Shelke and Ansari, 2017)

In addition to the above requirements ES EN 1998-1, 2015 in section 4.2.3.3 there are the criteria for irregularity in elevation this are

1) P For a building to be categorized as being regular in elevation, it shall satisfy all the conditions listed in the following paragraphs.
2) All lateral load resisting systems, such as cores, structural walls, or frames, shall run without interruption from their foundations to the top of the building or, if setbacks at different heights are present, to the top of the relevant zone of the building.
3) Both the lateral stiffness and the mass of the individual stories shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
4) In framed buildings the ratio of the actual story resistance to the resistance required by the analysis should not vary disproportionately between adjacent

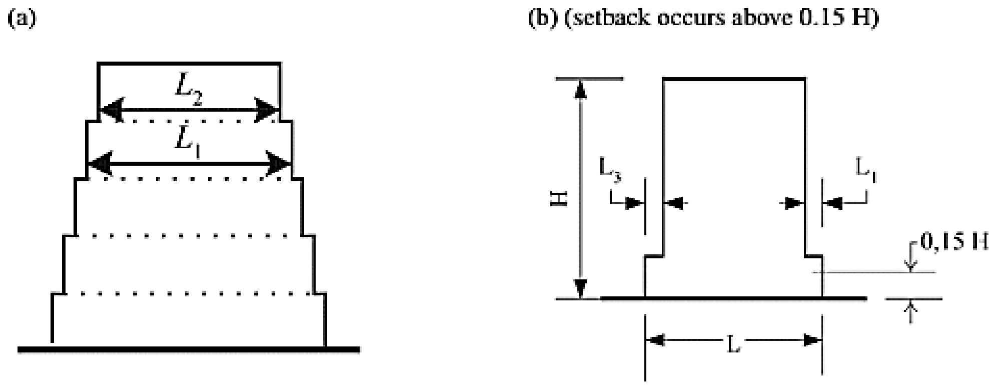
stores. Within this context the special aspects of masonry in filled frames are treated in 4.3.6.3.2.

5) When setbacks are present, the following additional conditions apply:

a). for gradual setbacks preserving axial symmetry, the setback at any floor shall be not greater than 20 % of the previous plan dimension in the direction of the setback;

b). for a single setback within the lower 15 % of the total height of the main structural system, the setback shall be not greater than 50 % of the previous plan dimension. In this case the structure of the base zone within the vertically projected perimeter of the upper stories should be designed to resist at least 75% of the horizontal shear forces that would develop in that zone in a similar building without the base enlargement;

c) If the setbacks do not preserve symmetry, in each face the sum of the setbacks at all stories shall be not greater than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be not greater than 10 % of the previous plan dimension.



$$\frac{L_1 - L_2}{L_1} \le 0.20 \qquad \frac{L_3 + L_1}{L} \le 0.20$$

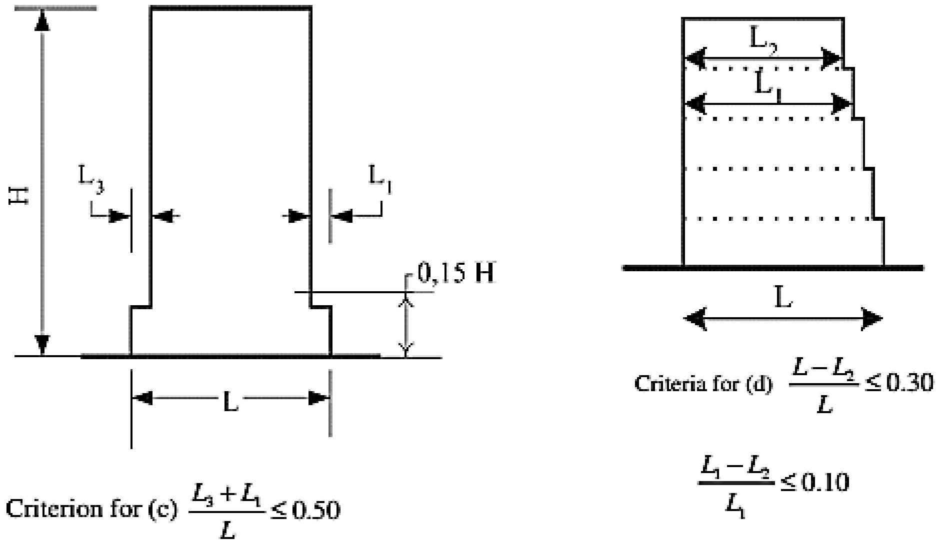


Figure 2. 1 Criteria for irregularity of buildings with setbacks [ES EN 1998-1, 2015]

2.4 Basic Concept of Seismic Design

Seismic design has evolved during the last decade to become a major area of application of performance-based earthquake engineering. This development has opened a new door to structural design engineers who were struggling to overcome the structural restrictions imposed on tall buildings by traditional prescriptive seismic design codes. Earthquake resistant design of buildings is based on the concept of acceptable levels of damage and performance level under the incoming earthquake motion. The performance objective of the building is related to the need of the designer and the client based on acceptable level of damage. The performance should be specified as an acceptable integrated probability of the building exceeding certain limit states during the maximum designed earthquake events that the building is likely to experience in the designed period. (Kumar *et al.*, 2014).

Specifying an integrated probability is complexity process and the requirements are often limit to specified intensity. For example, the objective may be specified in the form of a requirement that the building is fully operational with small damage or no damage during an earthquake.

The generally accepted objectives in the earthquake resistance design of a building are to ensure that the life safety of the users and the general public is preserved in the event of the maximum expected incoming earthquake that the building may experience within the design life, and structural damages are prevented for frequent earthquake. Additional performance objective may be defined for the structures needs special attention(Federal Emergency Management Agency [FEMA], 2000). For earthquake resistance design of normal buildings most codes specify only a single design earthquake which the building and its components are required to sustain without collapse. Some structural and nonstructural distress during the design earthquake of the building is expected. The building designed in this manner automatically satisfies the goal of no damage in a moderate earthquake.

Seismic design of the structure is the design of the structure according to the incoming reference earthquake load for the protection of human lives; limit the damage of the structure to acceptable limit and operational continuity of civil works import for civil safety. In the seismic design of structures two basic design steps involved firstly, the determination of the resultant seismic force applied to the structure and secondly, the design of the structures that satisfy all the requirements proposed by country codes.

In design of seismic resistance structures there are different limit states requirements that are satisfied for the good performance of the structure and to protect adverse damage. Serviceability limit state is the first requirement of design of seismic resistances structures and in this limit state frequent and minor intensity earthquakes should not affect day to day function of the structures. And also the damage occurs in structure and nonstructural elements must not needed repair. The structural elements also design within acceptable story displacement and to ensure adequate strength to resist the incoming earthquake force the structures elements design in the elastic range. This limit state is very important for some structures such as hospitals, fire stations etc. (Dowrick, 2009)

In design of seismic resistance structures there are different limit states requirements that are satisfied for the good performance of the structure and to protect adverse damage. Serviceability limit state is the first requirement of design of seismic resistances structures and in this limit state, frequent earthquakes inducing comparatively minor intensity of ground shake should not affect day to day function of the structures.

Secondly, Damage control limit state in this state for ground shaking of large intensity than the corresponding serviceable limit state causes some damage on the structure. This limit state requirement must be checked. Possible second order effect, strength and ductility of the structures must be within acceptable limit. Finally specific measures that are related to the criteria that cause loss of human life should be prevented even in the strong intensity earthquake and critical sections are carefully detailed for the transmission of incoming seismic load.

Occurs of large earthquakes is relatively infrequent. Although it is technically possible to design and construct structural buildings for this earthquake, it is generally considered as uneconomical and unnecessary to do this. The seismic design is performed in the study (Kumar *et al.*, 2014) with the anticipation that the large ground motions would cause some damage, and a seismic design philosophy on this basis has been developed over the years. The goal of the seismic design is to limit the damage in structures to an acceptable level. The buildings designed with the specific objective should be able to resist minor levels of earthquake ground motion without damage, resist moderate levels of earthquake ground motion without structural damage, but possibly with small non -structural damage, and resist major levels of earthquake ground motion without collapse, but with more structural as well as non -structural damage.

The seismic designs of the structures are dependent on different structural parameters as well as the incoming earthquake location, magnitude, soil property and the nearby buildings. All this considerations are advantageous to design the building for the incoming earthquake in the design period of the building.

The design of earthquake resistant concrete buildings shall provide the structure with an adequate capacity to dissipate energy without substantial reduction of its overall resistance against horizontal and vertical loading**.** Structure forced to remain straight in elevation through shear walls or strong columns ($\Sigma M_{Rc} > 1.3 \Sigma M_{Rb}$ in frames). (Awoke B.A, 2019)

2.4.1 Types of Seismic Design Philosophy

The deterministic design philosophy for anti-seismic design of buildings requires that the plastic hinges will develop in ductile reinforced concrete structures and only in specific desirable locations selected by the designer. In order.to satisfy the requirements provided in different country codes and for acceptable design of seismic resistance structures different design philosophies are developed by different literatures and country codes.(Papamichalopoulos, 2014; Awoke B.A, 2019)

2.4.1.1 Strength Design Philosophy

This is most common seismic design approach adopted nowadays. It is based on providing the structure with the minimum lateral strength to resist seismic loads, assuming that the structure will behave adequately in the non-linear range. For this reason only some simple construction detail rules are needed to be satisfied. (Awoke B.A, 2019)

2.4.1.2 Capacity Design Philosophy

Most modern building codes employ capacity design principles to help ensure ductile response and energy dissipation capacity in seismic resisting systems. The design provisions are geared toward restricting significant inelastic deformations to those structural components that are designed with sufficient inelastic deformation capacity. Those are generally referred to as deformation-controlled components. Other structural components, referred to as force-controlled components, are designed with sufficient strength to remain essentially elastic. Examples of applications of capacity design principles in building codes are the design provisions for brace connections, columns and beams in steel Special Concentrically Braced Frames in the 2010 AISC *Seismic Provisions*. (AISC, 2010a) The design provisions aim to confine significant inelastic deformation in the braces while the brace connections, columns and beams remain essentially elastic. To help ensure this behavior, the required design strengths of brace connections, columns and beams are to exceed the expected strength of the braces.

Capacity design provisions for force-controlled components can be further differentiated between those that can be defined solely based on the strength of adjacent members, as the brace and brace connection example above, to those that require information of overall system behavior, such as columns in steel braced frames. The required axial strength for columns in seismic resistant steel frames is based on the load from all yielding members exerting demand on them, including the effects of material over strength and strain hardening.

Another example of capacity design provisions that require information of overall system behavior are the design provisions for columns in reinforced concrete Special Moment Frames in the 2008 ACI 318 *Building Code Requirements for Structural Concrete and Commentary* (ACI 318, 2008). To confine inelastic deformations to beams (weak beam – strong column), the minimum required nominal flexural strength of columns is to exceed the factored nominal flexural strength of beams joining into the column where the column flexural strengths depend on the axial loads.(Victorsson, 2011)

2.4.1.3 **Performance Based Design Philosophy**

In the former design philosophy's only a few performance criteria are considers that are avoidance of collapse and damage protection of human lives, but experience in the earthquake engineering suggested that large damages are occurred on the structures designed according to the country codes. Therefore this concept leads to the birth of performance based design philosophy. The aim of this design philosophy is seismic resistance structures must design, construct and evaluated according to the need of the client under different seismic loads for different performance objectives.

2.4.1.4 **Displacement Based Design Philosophy**

In this method the structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive the ground motion. This method operates directly with deformation quantities hence gives better insight on the expected performance of the structures. This design philosophy addresses the deficiency of the former force based design method. (Awoke B.A, 2019)

2.5 Ductility and Seismic Response of Structures

The structures designed for different ductility classes have different performance characteristics: The structures designed for high ductility class are the most economically effective due to the largest reduction of the design seismic force (high q factor). Under an earthquake the structural system experiences large inelastic excursions and the overall damage is extensive. High ductility class (DCH) structures have lower base-shear resistance, higher damage rates, an increased displacement response and ductility demand comparing to the other two. In terms of the performance based design the difficulty for DCH to meet the displacement based design criteria is higher.

The DCL structures require relatively high resistance as the q factor and the ductility provided are small. Their responses are rather stiff due to the small yield excursion under the designed earthquakes. Main concerns are the structural vulnerability to fragility which may arise in excessive ground motions. The low ductility frames depict a worse performance, due to the crushing of concrete and buckling of the longitudinal bars at the bottom regions of the columns from insufficient confining. The hysteretic response of DCL is not usually satisfactory, as pinching in the plastic hinges occurs. Medium ductility class structures usually behave in an intermediate manner. The medium ductility class frames usually experience less damage with no obvious signs of material failure. Although their satisfactory performance due to their reduced overall damage and good hysteretic behaviour, efforts to gain enhanced ductility for the same cost should be encouraged [3].

Increasing the amount of confinement in the critical regions of columns, improves the local performance and the overall ductility through the hysteretic behaviour and increased hysteretic damping.(Papamichalopoulos, 2014)

2.5.1 Ductility in Seismic Design

2.5.1.1 Ductility definition

Ductility is the ability of a structure system, a component of a system, or a structural material to sustain plastic deformation prior to collapse, without substantial loss of resistance. Before the 1960s the term ductility was used only for characterizing the material behavior, after Baker’s studies in plastic design and Housner’s research works in earthquake problems (1997), this concept was extended to the level of structure and associated with the notions of strength and stiffness of the whole structure. But after years of use this concept continues to be an ambiguous parameter. Earthquake resistant concrete buildings shall be designed to provide energy dissipation capacity and an overall ductile behaviour. Overall ductile behaviour is ensured if the ductility demand involves globally a large volume of the structure spread to different elements and locations of all its storeys. To this end, ductile modes of failure (e.g. flexure) should precede brittle failure modes (e.g. shear) with sufficient reliability. (GRAVINA R.J, 2002; Ferrario, 2004)

At the material level the ductility of reinforcing steel depends up on the extent of the plastic deformation region. The larger the extent of the plastic deformation region, the mare ductile the steel. Although the ductility of a concrete is minimal compared to steel, the strain softening characteristics of concrete allow it to deform well beyond the peak strength under decreasing level of stress. The post-peak behaviour of concrete gives an indication of ductility. The less gradient on the softening curve, the more ductility the concrete. The ductile response of the concrete can be improved with confining effects. (GRAVINA R.J, 2002; Ferrario, 2004; ([ES EN 1998-1], 2015)

In other word, among many aspects required in RC beams and slabs design, ductility has become mandatory by the standard codes (ABNT NBR 6118: 2014; EUROCODE 2: 2004; ACI 318: 2002). In this context, ductility can be defined as the ability to support large plastic deformations before failure without significant resistance loss. The main reasons to consider ductility as a mandatory characteristic in the modern structural design are: ductility prevents brittle ruptures, which is a failure mode that must always be avoided; elements with ductile behavior have higher plastic rotation capacities when compared to brittle elements and contribute to large deformations/displacements before a physic rupture (Ko et al. 2001); ductility of cross sections are essential to provide bending moment redistribution along the beam as longitudinal reinforcement steel yields ensuring the redundant behavior of hyperstatic structures (Kara and Ashour 2013). Another important application in which the ductility is essential to guarantee safe behaviors of RC structural systems is related to dynamic loads generated by seismic tremors. In such cases, the ductility of the structural elements must be predicted and quantified in a detailed way to avoid severe damage and brittle failures of the buildings (Lopes *et al*., 2012; Arslan 2012; Demir *et al*. 2016).

To avoid demanding nonlinear analysis in the framework of everyday design purposes, an equivalent lateral load or modal response spectrum analysis is permitted, using spectral accelerations that result from a response spectrum, appropriately reduced by a, so called, behavior factor (q in Europe) . Ultimately, the structural system is designed for a lower level of strength, relying that stable energy absorption will be made feasible through specific geometric and minimum reinforcement requirements along with the associated detailing rules. Fundamental requirements (i.e. collapse prevention, damage limitation, and minimum level of serviceability) are also achieved through capacity design for the enhancement of global ductility. According to Euro code 8 (CEN 2004) in particular, the above philosophy is materialized for reinforced concrete buildings through the choice of the Ductility Class, i.e., Low (DCL), Medium (DCM) and High (DCH), each corresponding to different structural and detailing requirements. Notably, the Lower ductility class is only recommended by National Annexes in low seismicity areas or for base-isolated structures.

It has to be noted herein, that according to Euro code 8, the behaviour factor q depends not only on the structural system and the Ductility Class adopted but also on the degree of regularity in plan and height, while it represents a maximum permissible and not a recommended value. As a result, given the present challenging architectural forms, the actual seismic performance and the associated cost of three dimensional, dual building systems cannot be easily assessed in advance. Along these lines, the scope of this work is to study further the impact of Ductility Class on the construction cost and performance of such spatial buildings of different degrees of regularity, designed with distinct behavior factors along the two principal directions, within the permissible minimum and maximum limits.(Sextos *et al*, 2015)

A very important value in seismic design is the ductility limit. This limit is not necessarily the largest possible energy dissipation, but a significant changing of structural behavior must be expected at ductility larger than this limit. Two ductility limit types can be defined:

- ❖ available ductility, resulting from the behavior of structures and taking in to account its information, material properties, cross-section type, gravitational loads, degradation in stiffness and strength due to plastic excursions, etc.;
- ❖ required ductility, resulting from earthquake actions, in which all factors influencing these action are considered: magnitude, ground motion type, soil influence, natural period of the structure versus ground motion period, number of important cycles, etc. (Ferrario, 2004).

2.5.2 Performance Based Seismic Design

Performance based seismic design is a process of designing new buildings or seismic up-gradation of existing buildings, which includes a specific intent to achieve defined performance objectives in future earthquakes. Performance objectives relate to expectations regarding the amount of damage a building may experience in response to earthquake shaking and the consequences of that damage. Performance objectives are operational (O),immediate occupancy (IO), life safety (LS), collapse prevention (CP), in which Life safety is the major focus to reduce the threats to the life safety of the structure.

Performance based design approach in which performance levels are described in terms of displacement as damage is better correlated to displacements rather than forces. The fundamental goal of PSBD is to obtain a structure which will reach a target displacement profile when subjected to earthquakes consistent with a given reference response spectrum. The performance levels of the structure are governed through the selection of suitable values of the maximum displacement and maximum inter storey drift.(Chaudhari and More, 2017)

2.5.3 Performance Levels

A building can be subjected to low, moderate, or severe earthquakes. It may cross these events undamaged, it can undergo slight, moderate or heavy damage, it may be partially destroyed or it can collapse. These levels of damage depend on the earthquake intensities. Low intensity earthquakes occur frequently, moderate earthquakes more rarely, while strong earthquakes may occur once or maximum twice during the life of the structure. It is also possible that no devastating earthquake will affect the structure during its life. In these conditions, the checks, required to guarantee a good behavior of a structure during a seismic attack, must be examined in the light of a multi-level design approach. The structure design procedure on the basis of multi-level criteria is not a new concept. Under gravity, live, snow, wind loads, the limit state design considers the service and ultimate levels. In the case of seismic loading, the declared intent of building codes is to produce buildings capable of achieving the following performance objectives (Fajfar, 1998):

- To resist minor earthquakes without significant damage;
- To resist moderate earthquakes with repairable damage;
- To resist major earthquakes without collapse.

However, as a rule, the majority of codes considers explicitly only one performance objective, defined as protection, in cases of rare major earthquakes, of occupants against injury or death.

Earthquake resistant design of buildings is based on the concept of allowable levels of damage under the incoming earthquake. The required level of damage is related to the performance objective for the building(Bagchi, 2001)**.**

ATC-40 and FEMA-356 codes define the acceptance criteria depending on the plastic hinge rotations by considering various performance levels. In Figure 2.2, the five points (A, B, C, D and E) which are used to define the hinge rotation behaviour of RC members and the acceptance criteria on a force versus deformation diagram are given. In this diagram, points marked as IO, LS and CP represent immediate occupancy, life safety and collapse prevention, respectively

Table 2. 1 Structural Performance Level Definition (taken from Antoniou 2002)

Performance Level	Description
Operational	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate Occupancy	No significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness. Nonstructural elements are secure and most would function, if utilities were Available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Significant damage to structural elements with substantial reduction in stiffness, however margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted
Collapse Prevention	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

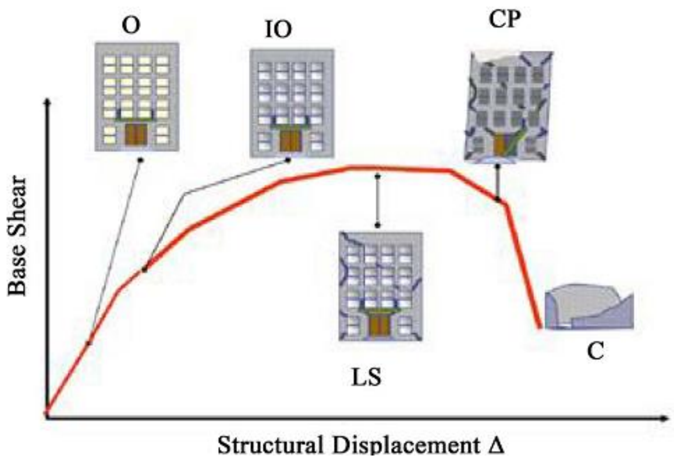


Figure 2. 2 Structural Performance Level Definition (taken from Antoniou 2002)

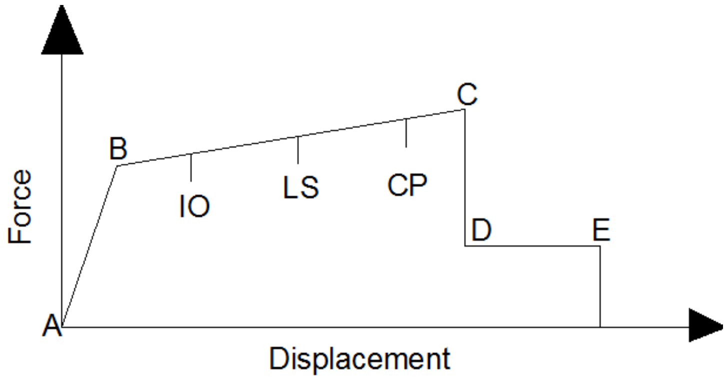


Figure 2. 3 Force-Displacement Relationship of Plastic Hinges

The load-deformation relation is defined by linear response (or elastic response) until point B. At point B, the member yields and again a linear response is observed with a reduced stiffness between the points B and C. At point C, a sudden reduction in the load resistance of the element occurs and the graph drops to point D. The residual resistance is observed until point E, where the final loss of resistance takes place.

The initial slope of this diagram between points A and B defines the elastic stiffness of the structure. Point C in this diagram represents the ultimate strength of the element where the significant stiffness degradation begins.

In the figure

- ✓ Point A represents always the origin of the curve
- ✓ Point B represents the yielding point.
- ✓ Point C represents the ultimate capacity

- ✓ Point D represents a residual strength and Point E represents total failure point.

Type of inelastic behavior of plastic hinges was flexural type desirable within capacity design philosophy. ([ATC], 1996)

Performance based philosophy in which performance objectives are defined in terms of displacement as damage is better correlated to displacements rather than forces. The basic feature of performance based design philosophy is to obtain a structure which will reach a target displacement profile when subjected to maximum earthquakes motion in the design period. The performance levels of the structure are governed through the selection of suitable values of the maximum inter story drift and maximum displacement(Bagchi, 2001; Chaudhari & Dhoot, 2016). Figure 2-3shows the typical process of design is to be followed.

Specified deformation states are often taken as a measure of building performance at corresponding load levels. ([FEMA], 2000)identifies the operational, IO, LS, CP performance levels and adopts the roof level lateral displacement at the corresponding load levels as a measure of the associated behavior states of the building. One of the performance based design method is capacity design method which is described in detail in the subsequent sections.

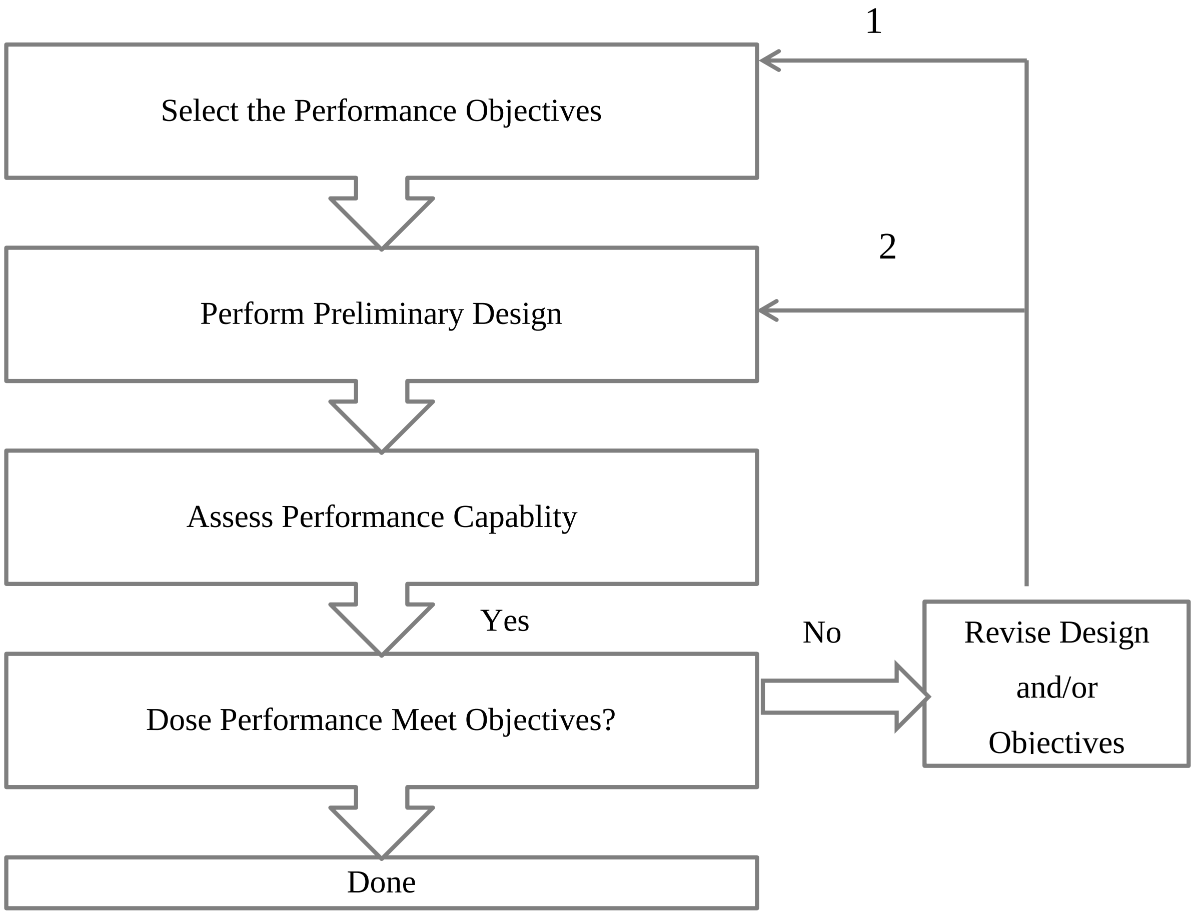


Figure 2. 4 Flowchart of Performance Based Design Process (Chaudhari and Dhoot, 2016)

2.5.4 Performance-Based Methodology Tools

The performance-based methodology necessitates the estimation of two quantities for assessment and design purposes. These are the seismic capacity and the seismic demand. Seismic capacity signifies the ability of the building to resist the seismic effects. Seismic demand is a description of the earthquake effects on the building. The performance is evaluated in a manner such that the capacity is greater than the demand (ATC-40, 1996). These quantities can be determined by performing either inelastic time-history analyses or nonlinear static ‘pushover’ analyses. The former is the most realistic analytical approach for assessing the performance of a structure, but it is usually very complex and time consuming mainly because of the complex nature of strong ground motions. This complexity has led to the adaptation of nonlinear static analysis methods as necessary assessment and design tools.

There are four analytical procedures for design and assessment purposes recommended in the guidelines of FEMA, ATC, and EC8. These are the Linear Static Procedure, LSP, Linear Dynamic Procedure, LDP, Nonlinear Static Procedure, NSP, and the Nonlinear Dynamic Procedure, NDP, with ascending order of complexity. (Awoke, B. A. 2019)

2.5.4.1 Linear Static Analysis, LSA

Linear static analysis is carried out under lateral forces applied separately in two orthogonal horizontal directions, X and Y. These forces are meant to simulate the peak inertia loads induced by the horizontal component of the seismic action in these directions, with the structure vibrating in its fundamental mode in the corresponding direction. As designers are familiar and conversant with elastic analysis for static loads (due to gravity or wind actions, etc.), this analysis is the workhorse of practical seismic design.

Accordingly, design codes limit the application of this method to buildings with a height wise distribution of mass and stiffness which is sufficiently regular for assumption 2 to be made with some confidence. Most codes, especially those adopting a standard 1st mode drift pattern independent of the value of the 1st natural period, e.g. (CEN 2004a), do not allow application of the method to tall flexible structures where higher modes dominate the response. Euro-code 8 in particular, allows applying linear static analysis only if both conditions (a) and (b) are met:

a. The building is regular in elevation, according to the criteria in the code which can be checked by inspection of the framing and the architectural drawings, without any structural calculations. The rationale for the exclusion of height wise irregular buildings is that their 1st mode shape may be far from the simple approximation assumed in linear static analysis. Moreover, higher mode effects may be locally significant (notably, around discontinuities or abrupt changes along the height), even though they may not be important for the global response (e.g., for the base shear and overturning moment)
b. The fundamental period of the building is not-longer than 2 s or four times the corner period TC between the constant-spectral-pseudo acceleration and constant-spectral-pseudo velocity ranges of the elastic spectrum. Recall that at periods above 2 s or 4TC spectral pseudo accelerations are low and that, if the 1st mode is in that range, the 2nd and/or 3rd modes may be at, or close to, the range where spectral pseudo-accelerations are constant and highest. So, their

contribution to the response may be comparable to that of the 1st mode, notwithstanding their normally lower participation mass and factors. (MICHAEL N. FARDIS, 2009)

2.5.4.2 Linear Dynamic Analysis, LDA

The linear dynamic approach is similar to the linear static approach and uses the structural model linearly elastic in nature. However, this analysis adopts the dynamic forces contrary to the linear static approach which employs the static forces. The dynamic forces in this method are applied in the form of the code specified response spectrum to the structure. Therefore, it provides a greater insight into the structural response as compared to the linear static approach. Furthermore, the representative ground motion is not reduced by the response modification factor *Err.* (*Chopra 1973*).

This method requires an eigen-value analysis of the building analytical model to determine the natural frequencies and the mode shapes. By use of the mathematical procedures and a response spectrum corresponding to the specified damping, the modal frequencies and shapes are further used to compute the spectral demands. These spectral demands are used to calculate the member forces, displacements, storey shears, base reactions etc. These modal forces are then combined using an established rule *(SRSS, ABS, and CQC)* to calculate the total response quantity to achieve better accuracy. The equation of dynamic equilibrium of a structure with N degrees of freedom under seismic excitation.(Varadharajan S., 2014)

2.5.4.3 Nonlinear Static Analysis, NSA, or Pushover Analysis, POA

As the name suggests this procedure is essentially a static analysis, in which the static loads are applied in an incremental fashion until the ultimate state of the structure is attained. The non-linear designation comes from the fact that the various components/elements are modeled using a non-linear mathematical model.(BENTO1 *al et*, 2004)

Unlike linear analysis, which has long been the basis of practical seismic design of new buildings, and nonlinear dynamic analysis, which has been extensively used since the 1970s for research, code-calibration or other special tasks, nonlinear static analysis (commonly called “pushover” analysis) was not widely known or used until the first new-generation guidelines for seismic rehabilitation of existing buildings (ATC 1997) adopted it as the reference method. Since then, its appealing simplicity and intuitiveness and the wide availability of reliable and user-friendly analysis software have made it the analysis method of choice for seismic assessment and retrofitting of buildings. (MICHAEL N. FARDIS, 2009)

The employment of the non-linear static procedure involves four distinct phases as described below

1. Define the mathematical model with the non-linear force deformation relationships for the various components/elements;
2. Define a suitable lateral load pattern and use the same pattern to define the capacity of the structure;
3. Define the seismic demand in the form of an elastic response spectrum;
4. Evaluate the performance of the building.(BENTO1 *et al*. 2004)

2.5.4.4 Nonlinear Dynamic Analysis, NDA

In this method, the seismic response of the structure is evaluated using step-by-step time history analysis. The main methodology of this procedure is almost similar to the static method of analysis. However, this approach differs in the concept that the design displacements are not established using the target displacement; but, is estimated through dynamic analysis by subjecting the building model to an ensemble of the ground motions. The calculated seismic response is very sensitive to the ground motion characteristics, and the analysis is carried out for more than one ground motion record. To perform the non-linear dynamic analysis, the equation prescribed by the Newmark"s method (*Chopra 2001; Cook 1988 and Humar 1990)* can be suitably extended. Based on review of analytical methods, the non-linear dynamic analysis method is adopted for the analytical study due to its accuracy and efficiency in determining the inelastic seismic response of a system subjected to the ground motion data. The review of previous research works show that the past research works have adopted static methods in majority for simplicity. However, the present research works in majority have adopted dynamic analysis (especially non-linear dynamic analysis) to achieve better accuracy to estimate the realistic seismic demands. Moreover, different seismic design codes prescribe dynamic analysis for medium and tall structures and it has been used by recent researchers as well *(Karavasilis et al. 2008 a, b; Panda and Ramachandra 2010).* Therefore, non-linear dynamic analysis method has been adopted in the present study to determine the seismic response of the building models.(Varadharajan S., 2014)

2.6 Non-Linear Analysis Methods

Nonlinear structural analysis methods of structural analysis are one of the analysis methods of structures under seismic loading and which considers the nonlinearity property of the structure and the material. Conventionally linear methods are dominant over the nonlinear method because of linear methods are relatively simple for analysis purpose and availability of linear analysis software's. And also this method is given approximate results but it does not consider the property of the structural response after the earthquake motion. Nonlinear methods in the opposite consider post-earthquake response of the structure properly and this method is appropriate for the investigation of structural performance after the seismic motion. Post-earthquake functions of some buildings are very important, therefore this type of structures are analyze using nonlinear methods are very important. In our new code ([ES EN 1998-1], 2015)and in different literatures two types of nonlinear methods are provided for the performance analysis of structures under earthquake motion. These are nonlinear static (pushover) analysis and nonlinear time history (dynamic) analysis methods.(Awoke, B. A. 2019)

2.6.1 Non-Linear Static (Pushover) Analysis

Nonlinear static procedure starts with the definition of control node in a structure. The previous research works *(Moghadam 1998; Fajfar et al. 2002; Fajfar et al. 2005)* pertaining to non-linear static analysis have considered the control node at center of mass of roof of the building. In this procedure, the mathematical model of the structure using this approach is prepared incorporating the aspect of material and geometrical nonlinearities. Then the modeled structure is subjected to monotonically increasing loads resulting in the increased displacement. This process is repeated until the occurrence of structure collapse. Since, the mathematical model directly incorporates the effects of inelastic material response; it results in fairly accurate estimate of the seismic response. Before initiating the static analysis procedure, the gravity loads are applied to the structure. Then the lateral load profiles of the building model are selected approximately to represent the distribution of the inertia forces during an earthquake. These forces vary in a complex manner during the seismic excitation. In elastic range, the inertia forces mainly depend upon factors like ground motion characteristics and mode shapes of the building. If the building response is in the non-linear range, then the distribution of these forces is influenced by localized yielding of the structural components. For performing seismic analysis and design, simplified procedures are required that can capture the worst possible scenarios of the building. The different patterns of force distribution methods are discussed in detail in FEMA 273 and FEMA 274. The non-linear static procedure although more accurate than linear elastic analysis fails to give an exact estimate of the seismic response. The main disadvantage with this procedure is that it does not account for variation in the dynamic response and inertial load patterns which vary with degrading strength and stiffness. Furthermore, it ignores the effect of higher modal contributions. Therefore, a more rational nonlinear approach needs to be adopted to get realistic estimate of seismic demands.

A. Conventional Pushover Analysis

Conventional pushover analysis method is static nonlinear analysis method and used to generate force-displacement relationship or capacity curve by incremental static lateral loads. This method is appropriate for simple and regular structure. The regularity criteria are provided in different codes and literatures.

B. Adaptive Pushover

Adaptive POA procedures have been developed by Bracci *et al.* (1997), Gupta (1998), and Requena *et al.* (2000). They all differ from the conventional POA procedures in the execution of nonlinear static analysis of the MDOF model.

The Adaptive POA procedures are mostly concerned with an appropriate estimation of the force vector that is going to 'push' the structure at each static force increment. The monitoring in the change of the incremental force vector could ensure that the stiffness degradation or strength deterioration of the structure is accounted for more realistically, than conventional nonlinear static analyses. When the new force vector has been determined, the remaining steps of the Adaptive POAs follow those of the Conventional POAs.

Bracci *et al.* (1997) introduced the Adaptive Pushover Analysis, APA, by utilizing an adaptive load pattern. The basis of the load pattern was an inverted triangular distribution, however it was stated that any assumed lateral force distribution could be equally used. The nonlinear static analysis in the APA method comprised the identification of four distinct response phases: elastic, first yield, incipient failure mechanism and full failure mechanism.

C. Modal Pushover

Modal pushover method is the third type of nonlinear static analysis which considers the effect of higher modes in the response of the structure. This type of analysis method is used for both regular and irregular structures especially for irregular structures because the effects of higher modes are significant. Acceptance criteria for primary elements, that are required to have a ductile behavior, are typically within the elastic or plastic ranges, depending on the performance level.

The employment of the non-linear static procedure involves four distinct phases as described below:

- Define the mathematical model with the non-linear force deformation relationships for the various components/elements;
- Define a suitable lateral load pattern and use the same pattern to define the capacity of the structure;
- Define the seismic demand in the form of an elastic response spectrum and calculate target displacement;
- Evaluate the performance of the building using different parameters.

Various methodologies have been developed for the performance evaluation using nonlinear static procedure. From the various methods the new Ethiopian building code adopted the N2 method(Bento et al., 2004; [ES EN 1998-1], 2015).

The main advantages of pushover analysis over the two linear methods (Linear Static and Linear Dynamic analysis) are(R.Bento et al., 2004, Krawinkler, 1997):

- The design is achieved by monitoring the deformations in the structure;
- The non-linear behavior is considered

- Gives the hierarchy of plastic hinge formations or yielding and failure on the member and the structure levels, as well as the progress of the overall capacity curve of the structure;
- It is convenient for performance-based seismic design approaches as it permits different design levels to verify the performance targets of the structure.

To develop a pushover analysis procedure consistent with RSA, we note that static analysis of the structure subjected to lateral forces

$$f_{no} = \Gamma_n m \phi_n A_n$$

Will provide the same value of r_{no}, the peak *n*th-mode response as in Eq. (3.17) [Chopra, 2001; Section 13.8.1]. Alternatively, this response value can be obtained by static analysis of the structure subjected to lateral forces distributed over the building height according to

$$s_n{}^* = m\phi_n$$

And the structure is pushed to the roof displacement, u_{rno}, the peak value of the roof displacement due to the *n*th-mode, which from Eq. (3.12) is $u_{rno} = \Gamma_n \phi_{\Gamma n} D_n \pi$

Where $D_n = A_n/\omega_n{}^2$Obviously *Dn* and *An* are available from the response (or design) spectrum. The peak modal responses, r_{no}, each determined by one pushover analysis(Chopra and Goel, 2003)

2.7 Capacity Curve

The force-displacement relation output of nonlinear push over analysis is called capacity curve. This capacity curve describes the relation between base shear of the structure and the control displacement.

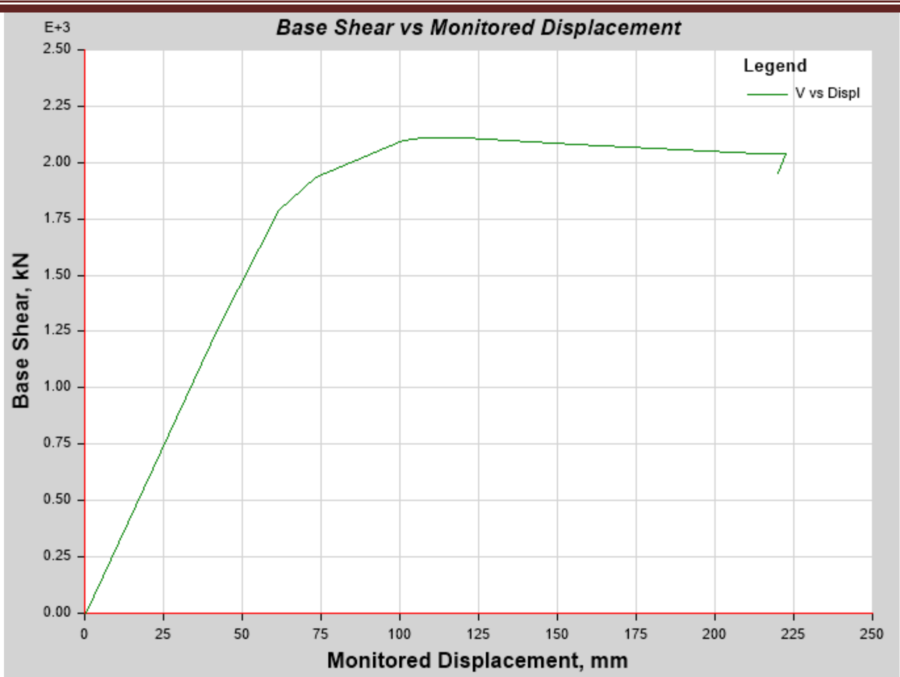


Figure 2. 5 Capacity Curve of the Building

The control displacement is in the range of zero up to the value corresponding to 150% of the target displacement according to ([ES EN 1998-1], 2015).

2.8 Target Displacement

The target displacement is the most useful parameter in the pushover analysis. Target displacement is the seismic demand derived from elastic response spectrum in terms of displacement of an equivalent single degree freedom system. Target displacement is calculated from elastic response spectrum using modal structural analysis methods by converting multi degree of freedom systems to single freedom systems.

Generally nonlinear static (pushover) analysis is used for many practical works for different purposes such as: to estimate the expected plastic mechanism and distribution of damage and to assess the structural performance of new, existing and retrofitted buildings. In the analysis of structure depending upon the regularity of the structure select which type of method is appropriate for the given structure among different types of pushover analysis methods. (Awoke, B. A. 2019)

CHAPTER THREE

METHODOLOGY

3.1 General

The main method used for this research is literature review on analysis, design, and pushover analysis of vertical geometric irregular reinforced concrete moment resisting frame structures. In addition to literature review on analysis, design and pushover analysis, assessment of the effect of the ductility classes in the vertical geometric irregular reinforced concrete be evaluated.

3.1 Description of the study area

The earth can be compared to an egg whose shell is cracked. The breaks are areas of least resistance, weak points around which most earthquakes occur. One of these faults in, the earth's crust is Africa's Great Rift Valley. Stretching some 3 000 kilometers, from the Red Sea in the north to Kenya in the south. Three active rift zones -- the Ethiopian Rift, the Red Sea Rift, and the Gulf of Aden Rift -- meet within this seismic fault system that slices through the heart of Ethiopia. (Hibler, 1973). The study area selected for this research is Addis Ababa city which is the capital city of Ethiopia is found at the horn of Africa with geographical coordinates 9˚1’48’’ North and 38˚44’24’’ East and an average elevation of 2355 above sea level. According to the new Ethiopian Standard for seismic design ES-EN-1998-1-2015, the seismic hazard of the city is in zone IV.

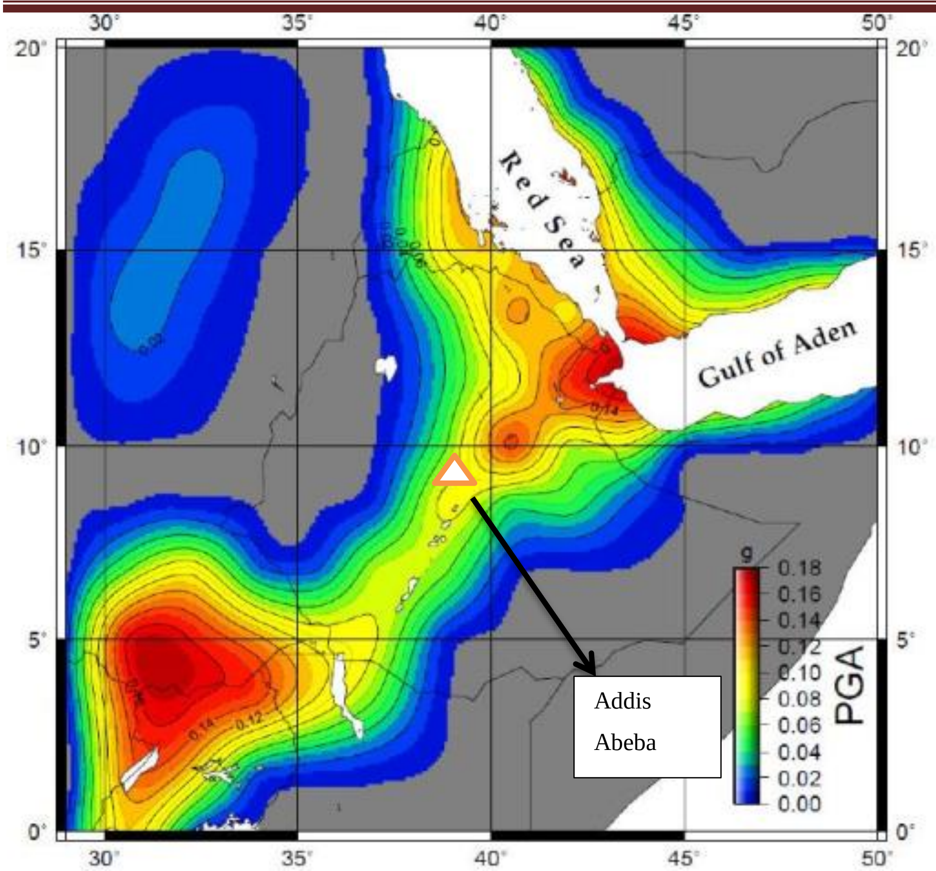


Figure 3 1 Study Area [ES-EN-1998-1-2015]

3.2 Research design

This section describes the performance of vertical geometric irregular reinforced concrete structures including the design of the structure according to ES-EN-1998-1-2015 and the method used to investigate the effect of ductility class for the seismic load. To meet the goals of the study the method describes in brief, how to execute the work, what be done, what tools are proposed, and the methods of analysis. The start of designing the structure and the performance of the structure in pushover analysis is generally tackled by the software development industry in one or more ways. Several powerful software packages have become commercially available. But for this study, ETABS 2016 was used to design the structure and to do none linear pushover.

For past years, ETABS software is used to design and do performance evaluation of any structure in many types of research works have been performed successfully and to show the analysis of seismic performances for regular and irregular reinforced concrete and steel structures.

The research significantly reveals the effect of ductility class on the performance of vertical geometric irregularity reinforced concrete structures. This chapter explains the process to draw the model and the process to run the nonlinear analysis using ETABS software programs. These analyses are done in the fastest way using 2016 a version of ETABS software. The major topics of this section are materials properties, modeled frames, loads used, designing of sections, and performing non-linear static (pushover) analysis.

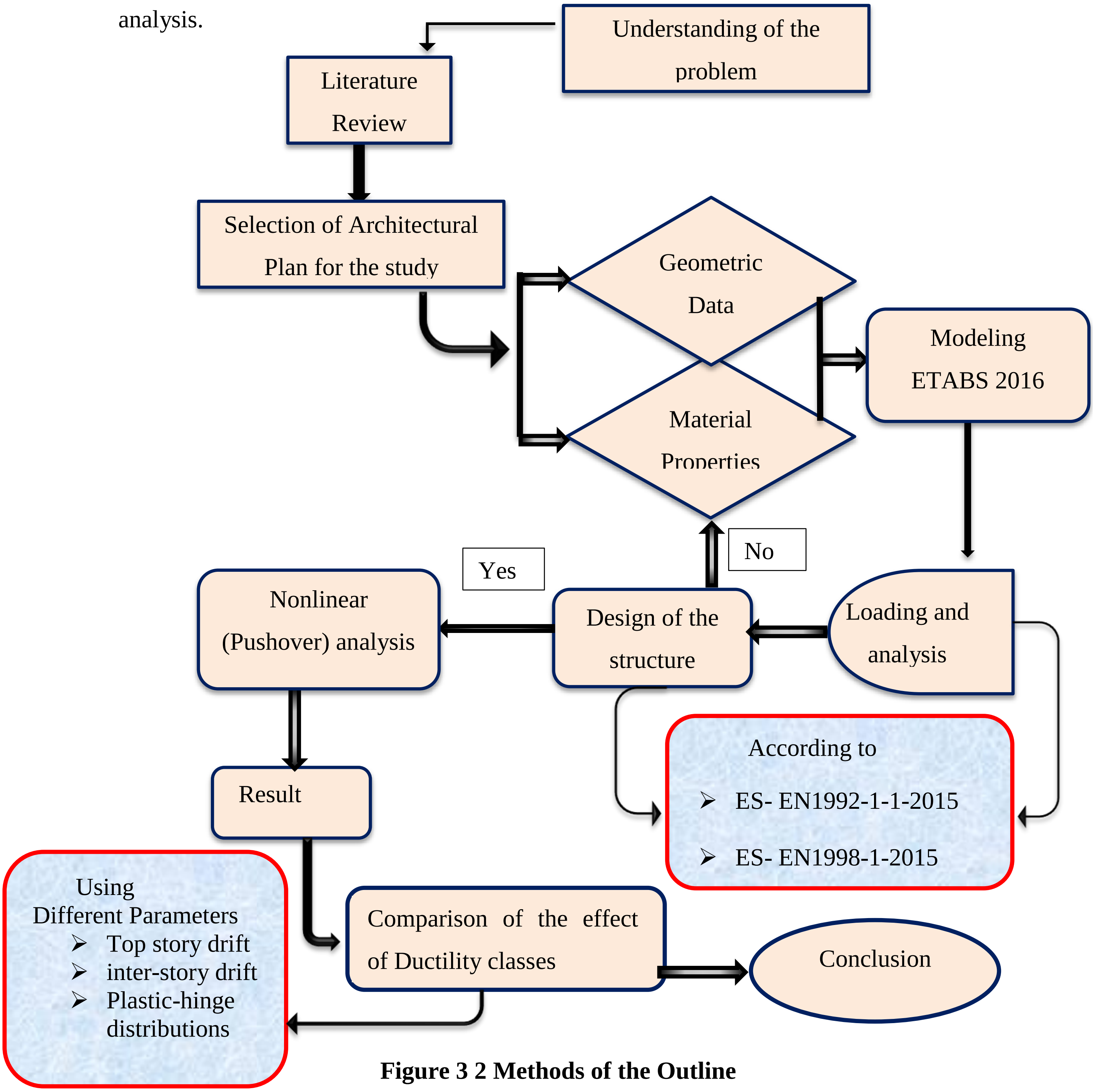


Figure 3 2 Methods of the Outline

3.3 Population

The total number of populations that considered in this the study is only the population existing within the range of my study area, which covers according to the code and classification of the vertical geometric irregular reinforced concrete frame according to setback ratio. From the existing population sampling be taken by using purposive sampling choice.

3.4 Study Variables

There are two variables that will be taken into consideration; the dependent variable and independent variables

I. Dependent variable

- Seismic performance of RC frame structure
  - ✓ Top Story drift
  - ✓ Inter story drift
  - ✓ Plastic-hinge distributions

II. Independent variable

- ✓ Loading (seismic)
- ✓ Vertical irregularity
- ✓ Ductility class

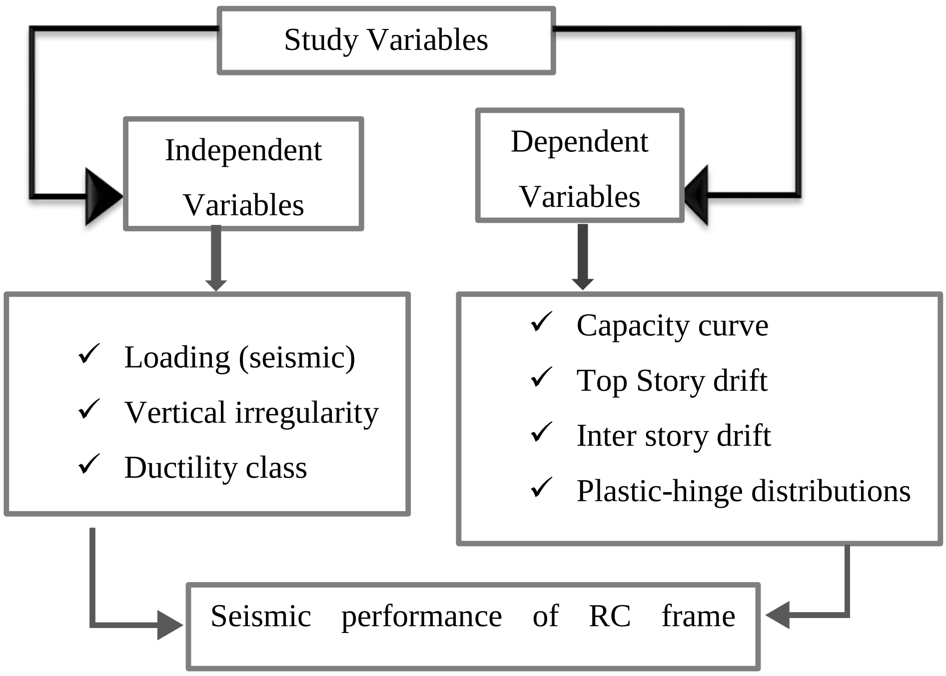


Figure 3 3 flow chart study Variables

3.5 Data processing and analysis

3.5.1 Modeling and material properties

3.5.1.1 Material properties

The material used in this design be according to the code provision for all seismic elements and in all seismic regions including critical regions reinforcing steel also based on ES EN 1992-1-1:2015, Table C.1 be used which is follows Euro code 2-2004. So the material properties are assigning in the software according to the Euro code for the design.

Table 3. 1 List of Materials and Software Used for the Analysis and Design

<table>
<tr><th rowspan="2">Materials Used</th><th colspan="2">DCH</th><th colspan="2">DCM</th></tr>
<tr><th>Beam</th><th>Column</th><th>Beam</th><th>Column</th></tr>
<tr><td>Concrete strength</td><td>C25/30</td><td>C25/30</td><td>C20/25</td><td>C20/25</td></tr>
<tr><td>Yield strength of rebar-S-420 (MPa)</td><td>420</td><td>420</td><td>420</td><td>420</td></tr>
<tr><th>Software Used</th><th colspan="4">Purpose</th></tr>
<tr><td>ETABS 2016</td><td colspan="4">To analyze(both structural and non-linear static analysis) the structure</td></tr>
<tr><th colspan="5">Parameters</th></tr>
<tr><td>Peak ground acceleration a_g</td><td colspan="4">0.1g</td></tr>
<tr><td>Importance factor</td><td colspan="4">1</td></tr>
<tr><td>Damping ratio</td><td colspan="4">5%</td></tr>
<tr><td>Earthquake zone</td><td colspan="4">Zone 3</td></tr>
<tr><td>Soil class</td><td colspan="4">C</td></tr>
<tr><td>Building type</td><td colspan="4">Special moment resisting RC frame</td></tr>
<tr><td>Modulus of elasticity of concrete</td><td colspan="4">31Gpa (C25/30) and 30Gpa (C20/25)</td></tr>
<tr><td>Modulus of elasticity of steel</td><td colspan="4">200Gpa</td></tr>
<tr><td>Spectrum Type</td><td colspan="4">1</td></tr>
</table>

3.5.2 Loading

Live, dead, and super dead loads are three types of load used in all 16 models. Among them, the Live load on the slab, self-weight of the slab, and partition load on the beam and slab were in-put because the dead load of the beam is automatically calculated by the software itself. The live load and self-weight applied on the slabs are transferred to the beam. Live load, as well as super dead load, were define and assigned to beams.

Table 3. 2 Slab thickness and live load

Dead load	
Frame elements	From the cross section of each element
Slab thickness (mm)	150
0.15*24=3.6 kN/m^2	
Portion load on the beam =2.8*0.1*24= 6.75 kN/m	
Live Load (kN/m^2)	
Story live load on slab	3kN/m^2

3.5.3 Modeling

In this research, for the analytical study of the performance of irregular reinforced concrete structure considered based on 3D RC building with varying heights and widths. Different building geometries were taken for the study. These building geometries represent varying degrees of irregularity or amount of setback. The bay widths and length shall be taken under the code by varying the setback. In this research G+13, G+14 and G+15 reinforced concrete buildings for mixed-use buildings are selected uniform story height of 3m. Altogether 16 building frames with different amounts of setback irregularities due to the reduction in width and height and also the ductility class with high and medium were selected. To prevent the torsion effect all frames are assumed to be rigid.

For the condition of vertical geometric irregularity according to ES-EN-1998-1-2015 it shall satisfy the following condition.

A. For gradual set-backs preserving axial symmetry, the set back at any floor shall be not greater than 20% of the previous plan dimension in the direction of the setback.
B. For a single setback within the lower 15% of the total height of the main structure system, the setback shall not be greater than 50% of the previous plan dimension. In this case structure of the base zone within the vertically projected perimeter of the upper storeys should not be designed to resist at least 75% of the horizontal shear force that would develop in the zone in a similar building without the base enlargement;
C. If the setback do not preserve symmetry, in each face the sum of the setback at all storey shall be not greater than 30% of the plan dimension at the ground floor

above the foundation or above the top of a rigid basement, and the individual set back shall be not greater than 10% of the previous plan dimension.

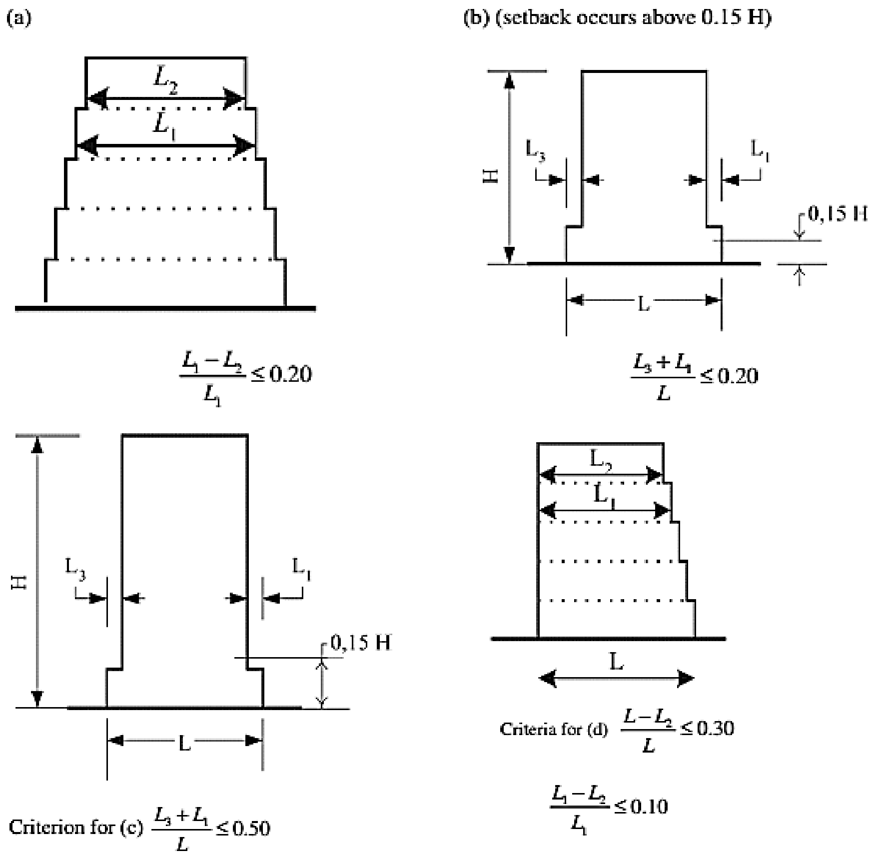
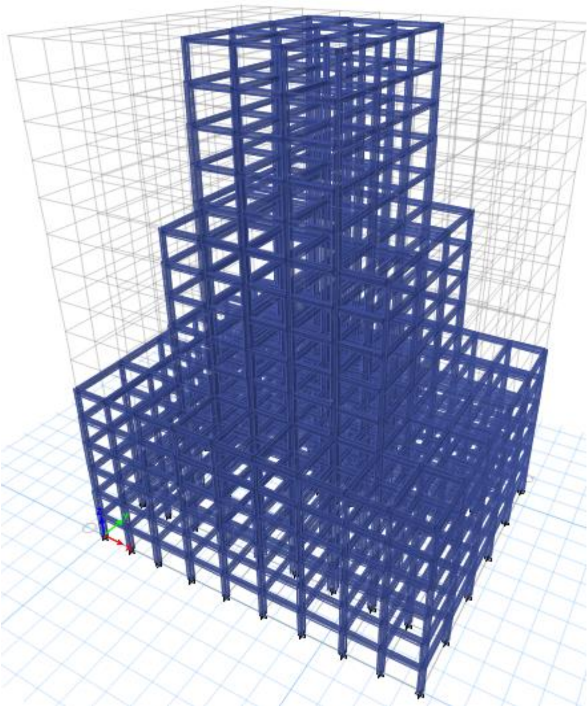


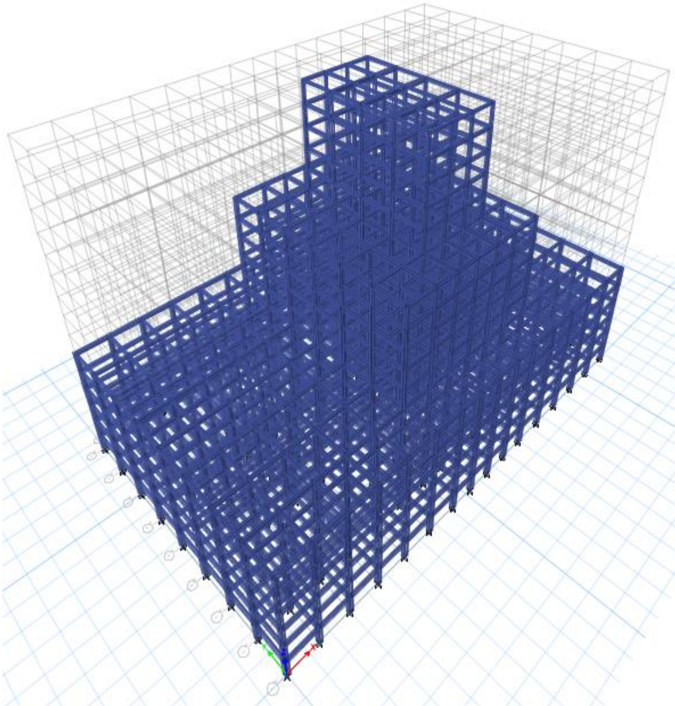
Figure3 4 Criteria for irregularity of buildings with setbacks

According to the above conditions buildings are three-dimensional, with the irregularity in the direction of setback, in the other horizontal direction the building is just repeating its geometric configuration. Setback frames are hereafter denoted as Model one (M-1), Model two (M-2), Model three (M-3), Model four (M-4), Model five (M-5), Model six (M-6), Model seven (M-7), and Model eight (M-8) depending on the percentage reduction of floor area and height according to ES-EN-1998-2015 as shown in the Fig. 3.1.

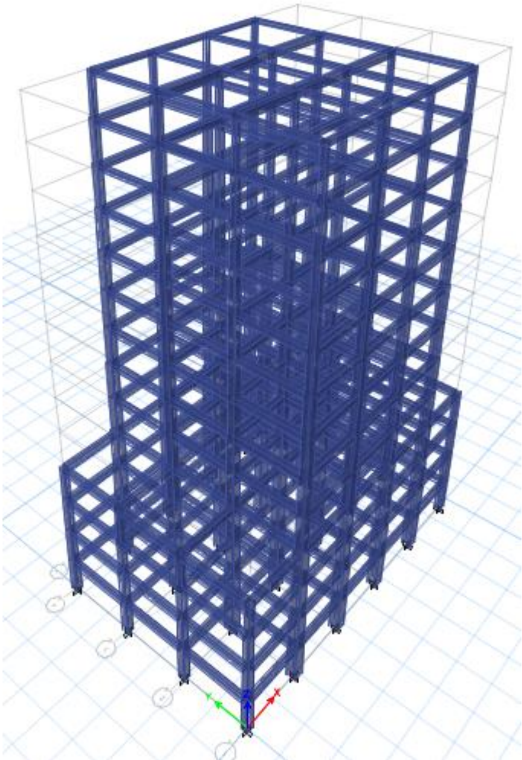
Modeling and structural analysis of the building was done using ETABS 2016. V 2.1 software. Frame section property was defined including the size of the section, select material properties, and section property modifier was defined. The size of the section was dependent on the ductility requirement of the section and the incoming design load. The design of the reinforcement for the cross-section of the beam and the column is according to ES-EN-1992-2015 which is followed Euro code 2-2004 provisions. So in the definition of material properties and design constants relevant for design is selected according to Euro code 2-2004 and Euro code 8-2004 for seismic parameters.



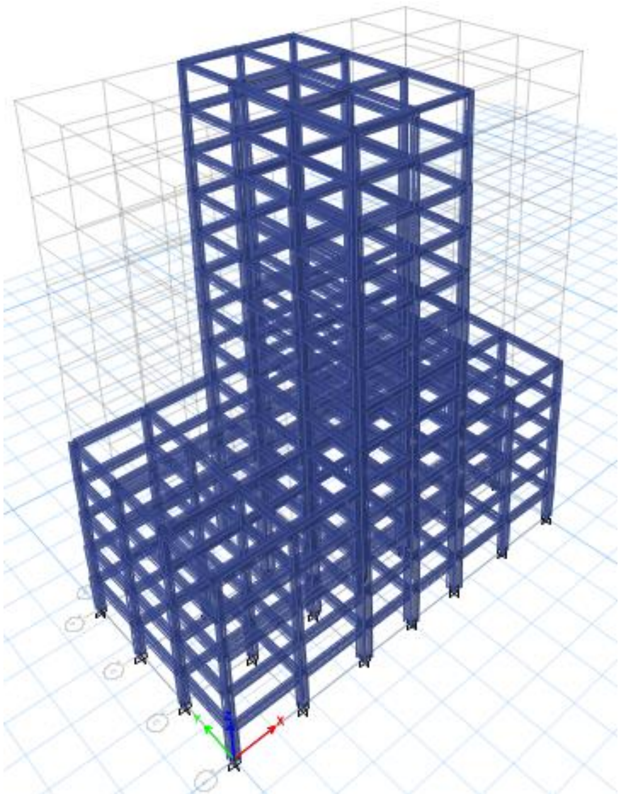
a. Gradual setback one direction M-1 $\frac{L_1-L_2}{L}=40\%$



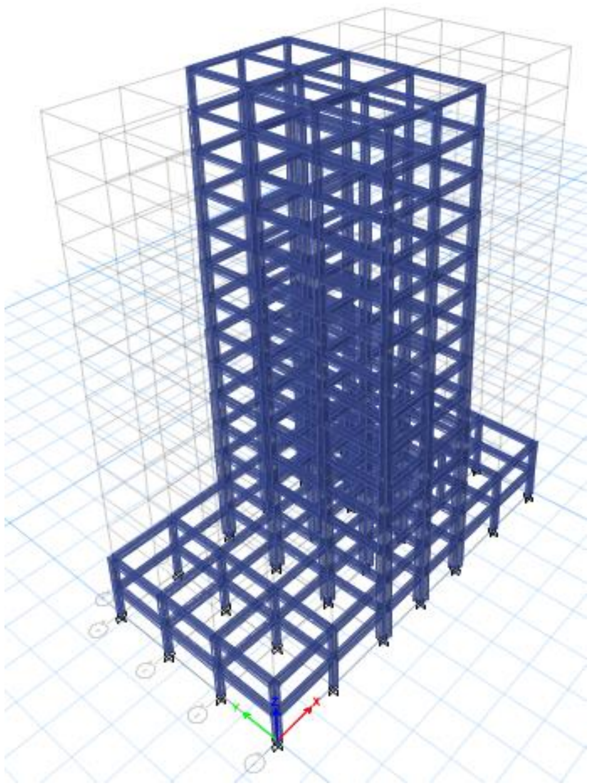
b. Gradual setback one direction M-2 $\frac{L_1-L_2}{L}=60\%$



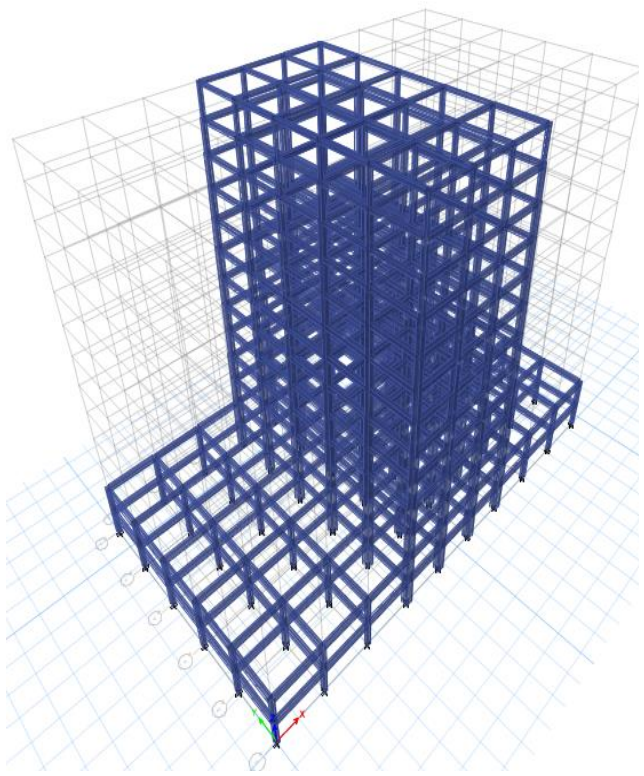
C.For setbackoccur above 0.15HM-3 $\frac{L_3-L_1}{L}=40\%$



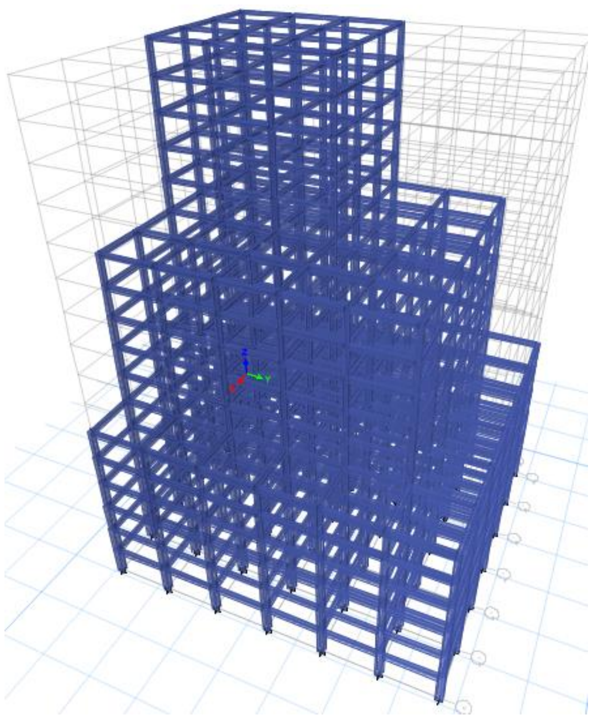
d. For setback occur above 0.15H M-4 $\frac{L_3-L_1}{L}=60\%$



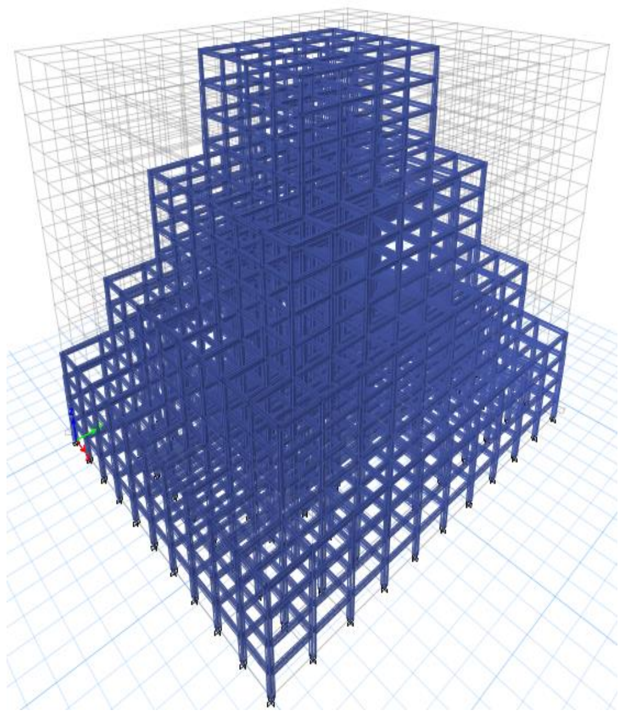
e.For setbackoccur below 0.15HM-5 $\frac{L_3+L_1}{L} = 60\%$



f. For setbackoccur below 0.15H M-6 $\frac{L_3+L_1}{L} = 70\%$



g. Gradual setback in both direction M-7 $\frac{L-L_2}{L} = 50\%$



h. Gradual setback in both direction M-8 $\frac{L-L_2}{L} = 70\%$

Figure3 5 Typical building models considered for the present study.

3.2.1 Seismic Mass Determination

Seismic mass will first need to be defined. The weight of the structure used in the calculation of automatic seismic loads is based on the specified mass of the structure and is termed mass source in ETABS 2016. The self-weight of columns in any story shall be equally distributed to the floors above and below the story and lumped with the beam self-weight found in that story. The reduced live load is used for the calculation of seismic load. The mass taken when calculating the earthquake loads should comprise the full permanent (or dead) load plus the variable (or live) load multiplied by a factor ψ_{Ei}. The combination coefficient is determined based on the function of the building from ([ES-EN 1998-1], 2015.

Combination coefficient, ψ_{Ei}, calculated from the following expression:

$$\boldsymbol{\Psi_{Ei} = \varphi . \Psi_{2i}} \qquad \textbf{(3. 1)}$$

Where φ: value from Table C.2 of this paper

Ψ_{2i} Recommended values for buildings from Table C.3 of this paper

The seismic weight of the floor is lumped weight, which acts at the center of mass of the floor.

3.2.2 Load Combination

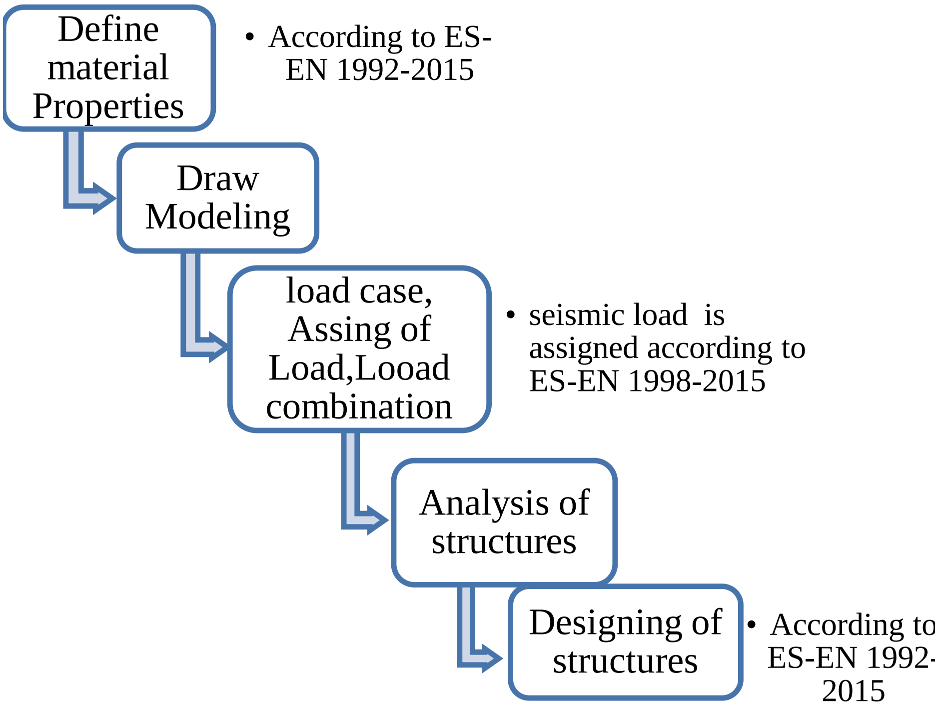
In this research all possible load the combination was defined from the software default combination for the given assigned load to get the largest action effect of the structural elements.

Table 3. 3 Load Combination

TABLE: Load Combinations		
Combo	**Combinations**	**Load case**
UDCon1	1.35DL+1.35SDL	Serviceability Limit State
UDCon2	UDCon1+1.5LL	Static case
UDCon3	DL+SDL+0.3LL+Eqx	Seismic Case
UDCon4	DL+SDL+0.3LL-Eqx	
UDCon5	DL+SDL+0.3LL+(Eq-x)	
UDCon6	DL+SDL+0.3LL-(Eq-x)	
UDCon7	DL+SDL+0.3LL+Eqy	
UDCon8	DL+SDL+0.3LL-Eqy	
UDCon9	DL+SDL+0.3LL+(Eq-y)	
UDCon10	DL+SDL+0.3LL-(Eq-y)	
UDCon11	DL+ SDL + Eqx	
UDCon12	DL+SDL- Eqx	
UDCon13	DL+SDL+ (Eq-x)	
UDCon14	DL+SDL-(Eq-x)	
UDCon15	DL+SDL+ Eqy	
UDCon16	DL+SDL- Eqy	
UDCon17	DL+SDL+ (Eq-y)	
UDCon18	DL+SDL- (Eq-y)	

3.6 Design of structure

The buildings to be considered in this study will be designed according to the provision of ES- EN 1998-1-2015. Analytical works will be based on the comparison of the reinforced concrete buildings designed as high ductility class (DCH) and medium ductility class (DCM) upon deep deposits of dense or medium-dense sand, gravel, or stiff clay (Ground type C, ES-EN 1998-1-2015 soil classification). In the designing of the structure, ETABS software is used but in the software the parameters to be filled according to Euro code. The code filled is similar to ES-EN-2015. In the design of the structure the following steps will be followed;



- **Material requirement**:- The material used in this design will be according to the code provision for all seismic elements and in all seismic regions including critical regions reinforcing steel also based on ES-EN 1992-1-1:2015, Table C.1 will be used.
- **Design of beam and column**: - Beam and column frame elements are designed according the requirement given in ES-EN 1998-1-2015. In the primary seismic beam and column the design action effects including shear force are determined according to capacity design rule. In the design action effects redistribution of moments will be permitted. And it will provide appropriate longitudinal and transverse reinforcement based on the imposed maximum action effect.
- **ULS verifications and detailing**: - This verification will be done according to the new Ethiopian building code and the resistance and local ductility of primary beam and column sections are verify and careful detailing will done based on the provision of the code. The lengths of critical regions are will determine and carefully detailed. The ductility factor, spacing of stirrups and reinforcement ratios are also determined accordingly.

3.7 Performance assessment according to Non-linear pushover analysis

In this study first the performance evaluation of the newly designed structure will be done using nonlinear static (Pushover) analysis and ETABS software used for this non-linear pushover analysis. In the modeling of the structure strength and stiffness degradation will also consider and this finite element also allows for strength and stiffness degradation in the components by providing the force-deformation criteria for hinges used in pushover analysis.

The values used to define the force-deformation curves for critical regions will be defined depending upon the ratio of the reinforcement, failure mechanism, and ductility class. The target displacement used for calculating the controlled displacement will be used according to ES EN 1998-1-2015 provision. In this study the building select for this comparison is vertical geometric irregular frame structure so the analysis will be performed using two planar models, one for each main horizontal direction.

Finally after a careful modeling and analysis of the structures using ETABS, It will extract different outputs such as capacity curve, top displacement, and inter-story drift and see the distribution of critical regions and plastic mechanisms.

3.7.1 Comparison of the seismic performance of structures

In this study the outcome is seismic performance evaluation of the buildings designed by different ductility classes. The evaluation will do with ETABS 2016 using nonlinear static analysis. The comparison of seismic performance will be done according to different parameters. The first value is the inter-story drift of the structure will compare quantitatively. Secondly, the seismic demand of the structure will be assessed. The distribution of plastic regions and plastic mechanisms are also considered and the number and location of hinge formation in the column will be investigated. Finally, the output difference of the buildings will be discussed and describe qualitatively.

CHAPTRE FOUR

RESULT AND DISCUSSION

4.1 General

This chapter presents the results of the effect of the ductility class on the performance of vertical geometric irregular reinforced concrete structures with different setback ratios in the accordance with ES-EN 1998-1-2015 (i.e.G+13, G+14, and G+15 buildings) designed as a DCM and DCH by static non-linear (pushover analysis). The study was undertaken by considering the effect of ductility on the performance of the reinforced concrete structure. This evaluation can be discussed in the form of a capacity curve, top storey displacement, inter-story drift, and plastic mechanism distribution for critical regions. These outputs are plotted to show the different performances between DCM and DCH for the reinforced concrete moment resisting frame.

4.2 Seismic Performance Evaluation of DCM and DCH Buildings

The nature of structural design under seismic actions ideally should directly rely on energy-based (hysteresis based) formulations because in reality the seismic energy input (demand) should be dissipated by seismic energy supply (capacity). To compare the seismic performance of the effect of the ductility class for vertical geometric irregular reinforced concrete structure non-linear analysis (Pushover) are used to analyze 16 different irregulars reinforced concrete frame structures. The result of the pushover analysis plotted as the capacity cure, top story displacement; inter-story drift, and plastic mechanism for the two ductility to evaluate the seismic performance of the structure. The following results were get from pushover analysis will be presented below.

4.2.1 Capacity-Curve of the Building

Capacity curve of the buildings are represented the variation base shear with the roof displacement used to evaluate the performance the structure. As the specific objective of this study is to compare the performance of structure designed for different ductility classes. It can be observed that there is significant variation in capacity curves when ductility class increases (from DCM to DCH) the energy dissipation (absorption) of the structure increases. The capacity (base shear) and the top displacement of the structure are differing significantly in same structures when the ductility class increased. From the pushover x results the maximum capacity of the structure for M-1, M-2, M-4, M-3, M-5, M-6, M-7 and M-8 increased the bas shear by 7.04%, 12.14%, 5.18%, 13.77%, 12.37%, 5.33%, 4.52% and 8.78% respectively and for pushover y 0.51%, 22.05%, 21.23%, 6.67%, 4.91%, 0.33%, 3.51% and 0.69% respectively when it changes from DCM to DCH. So the ductility class changes the capacity of the structure is increase and the dissipation or absorption of the energy is increase due to the ductile properties of the material in the reinforcement. Thus, in all cases the performance in terms of ductility (deformation capacity) of structures designed for ductility class high was found to be better than structures designed for ductility class medium. The percentage increase in of maximum base shear result is summarized on the table 4-2 and the graph below.

Table 4. 1 Maximum Bas Shear increment for push-X

PUSH – X				
Building	Maximum Base shear			
	DCM	DCH	Net	%
M-1	6657.5836	7161.8817	504.2981	7.04
M-2	19170.613	21819.775	2649.1621	12.14
M-3	2884.3661	3041.9408	157.5747	5.18
M-4	3591.2778	4164.7933	573.5155	13.77
M-5	2037.8957	2325.5412	287.6455	12.37
M-6	4769.8017	5038.5728	268.7711	5.33
M-7	7167.498	7507.1339	339.6359	4.52
M-8	11568.873	12681.695	1112.8228	8.78

Table 4. 2 Maximum Base Shear increment for push-Y

PUSH – Y				
Building	Maximum Base shear			
	DCM	DCH	Net	%
M-1	6220.48	6380.77	160.288	2.51
M-2	20032.8	25700.4	5667.54	22.05
M-3	2494.11	3166.43	672.323	21.23
M-4	3681.39	3944.6	263.212	6.67
M-5	2152.53	2263.62	111.088	4.91
M-6	4836.13	4852.25	16.119	0.33
M-7	7474.21	7745.99	271.779	3.51
M-8	12245.5	12331.1	85.5427	0.69

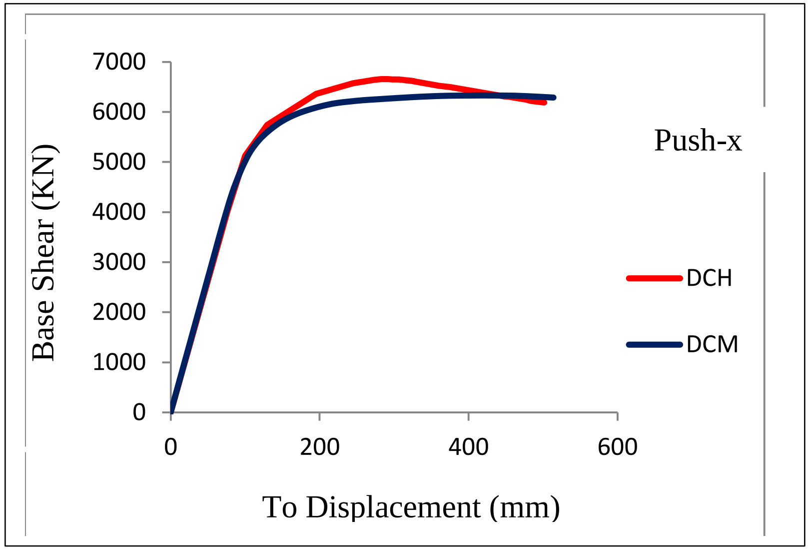


Figure 4 2Capacity Curve G+15 DCM and DCH Building ...PUSH – X M-1

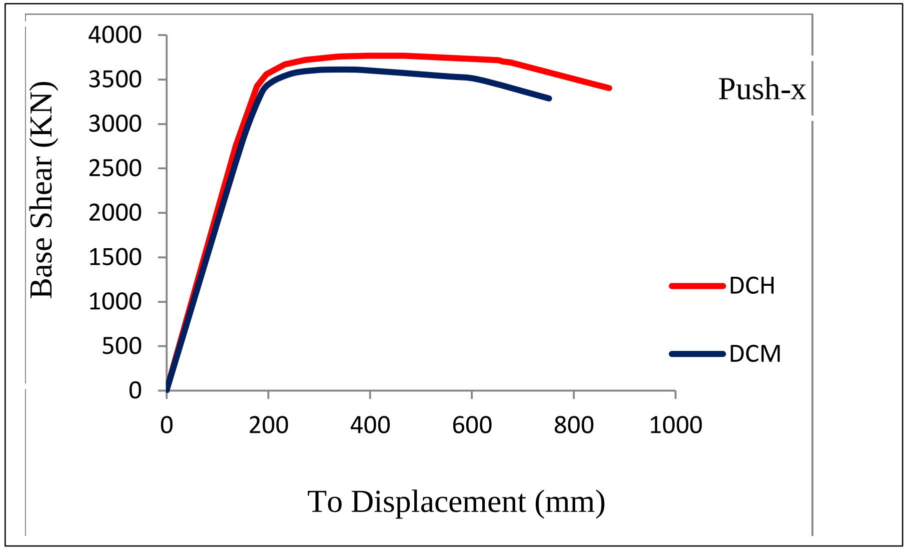


Figure 4 1Capacity Curve G+15 DCM and DCH Building ...PUSH – X M-2

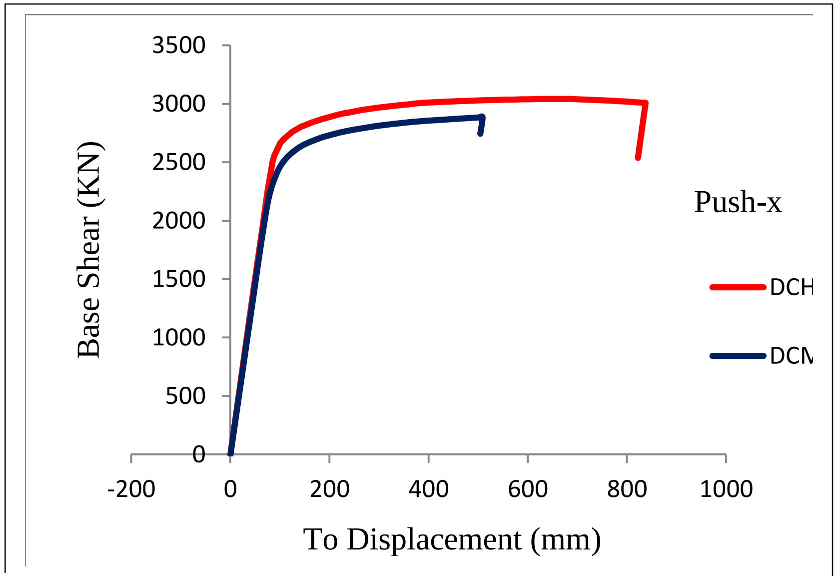


Figure 4 4Capacity Curve G+13 DCM and DCH Building ...PUSH – X M-3

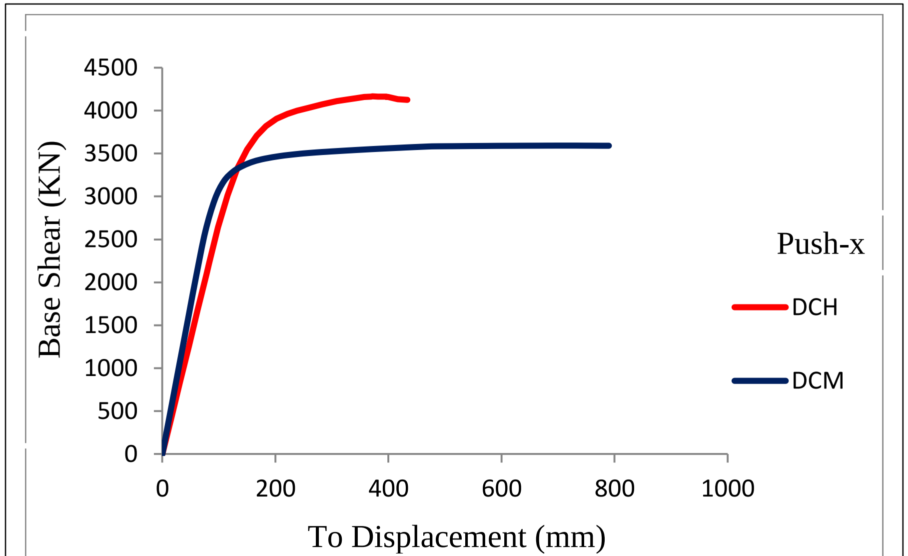


Figure 4 3Capacity Curve G+13 DCM and DCH Building ...PUSH – X M-4

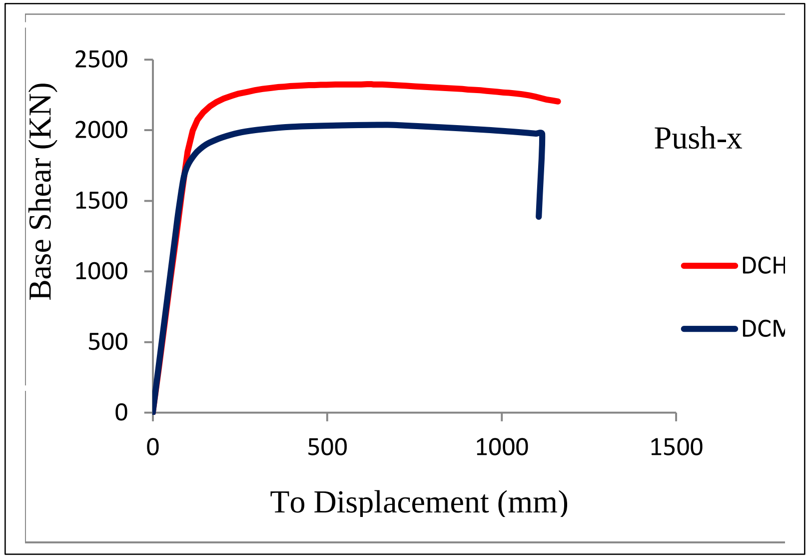


Figure 4 6Capacity Curve G+13 DCM and DCH Building ...PUSH – X M-5

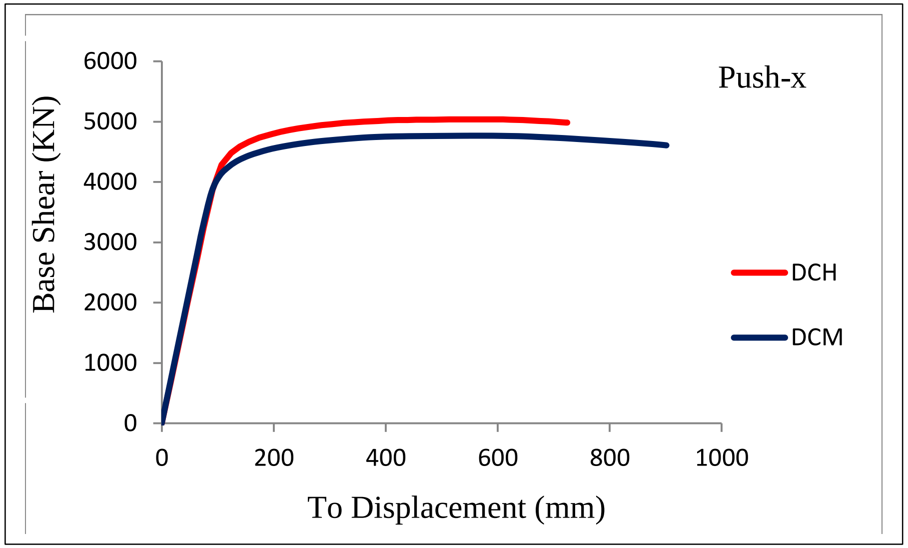


Figure 4 5Capacity Curve G+13 DCM and DCH Building ...PUSH – X M-6

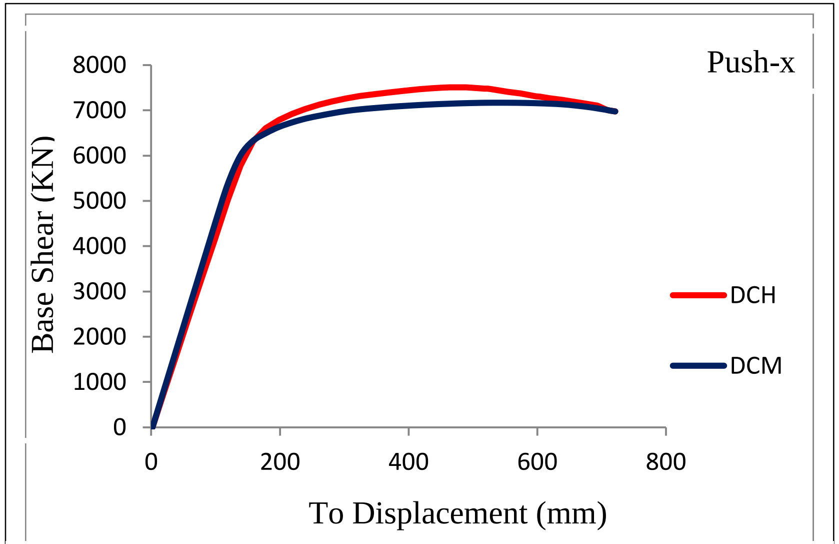


Figure 4 8Capacity Curve G+14 DCM and DCH Building ...PUSH – X M-7

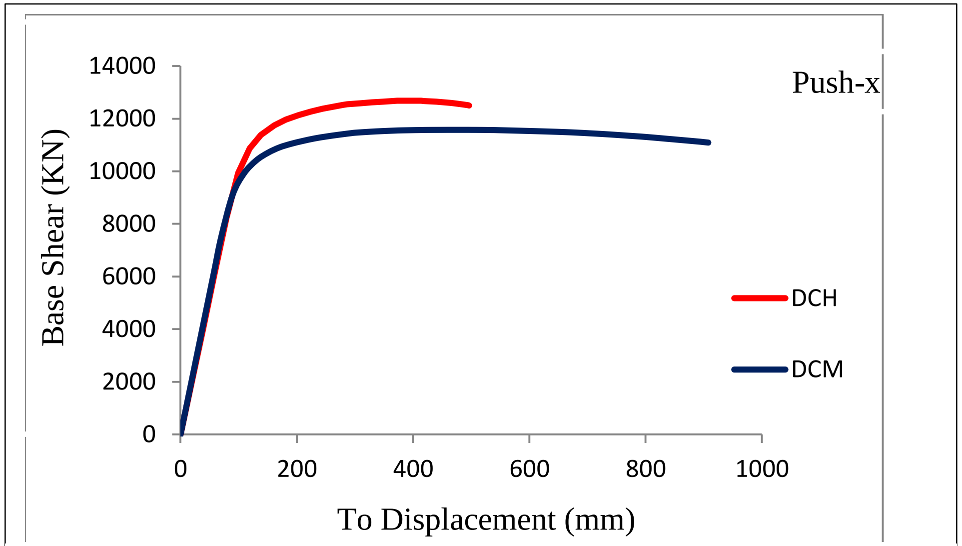


Figure 4 7Capacity Curve G+14 DCM and DCH Building ...PUSH – X M-8

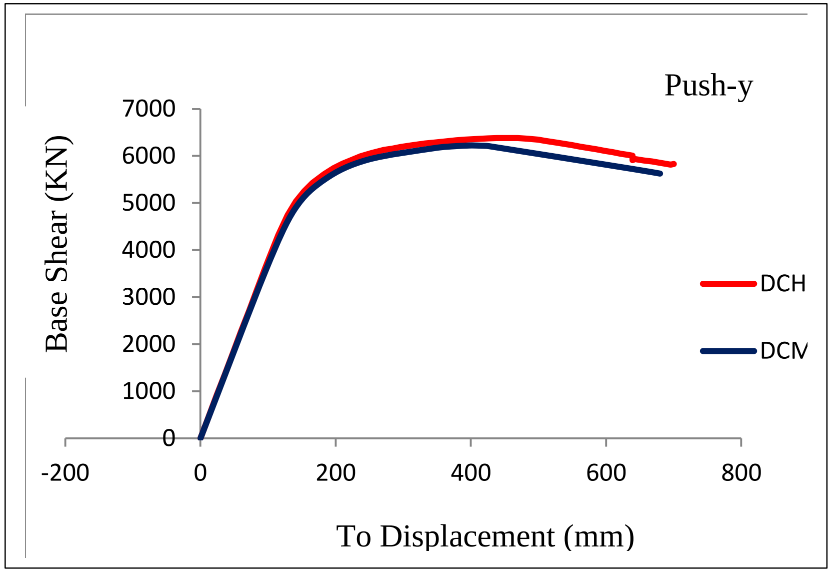


Figure 4 9Capacity Curve G+15 DCM and DCH Building ...PUSH – Y M-1

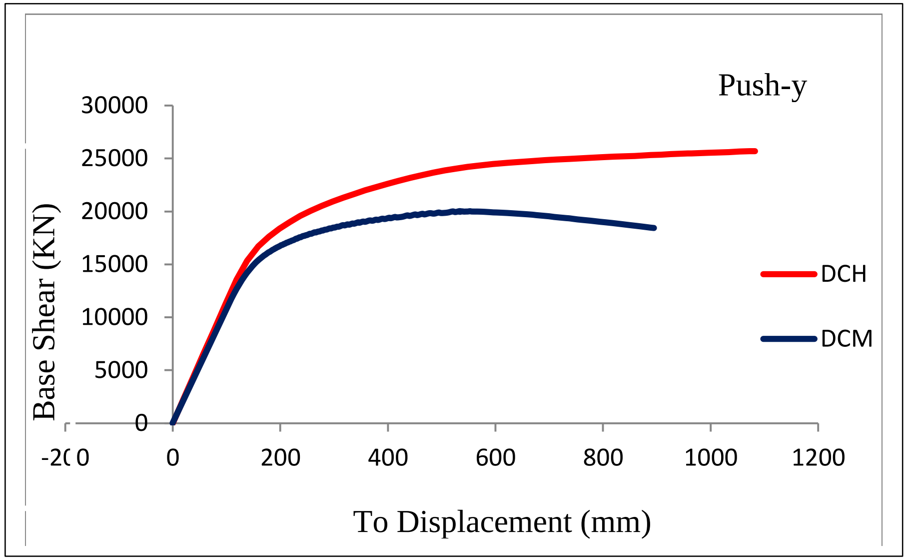


Figure 4 10Capacity Curve G+15 DCM and DCH Building ...PUSH – Y M-2

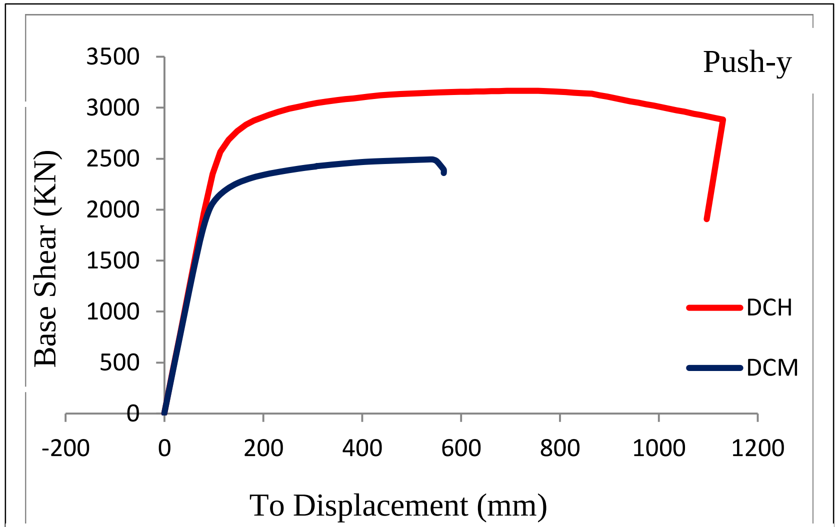


Figure 4 11Capacity Curve G+13 DCM and DCH Building ...PUSH – Y M-3

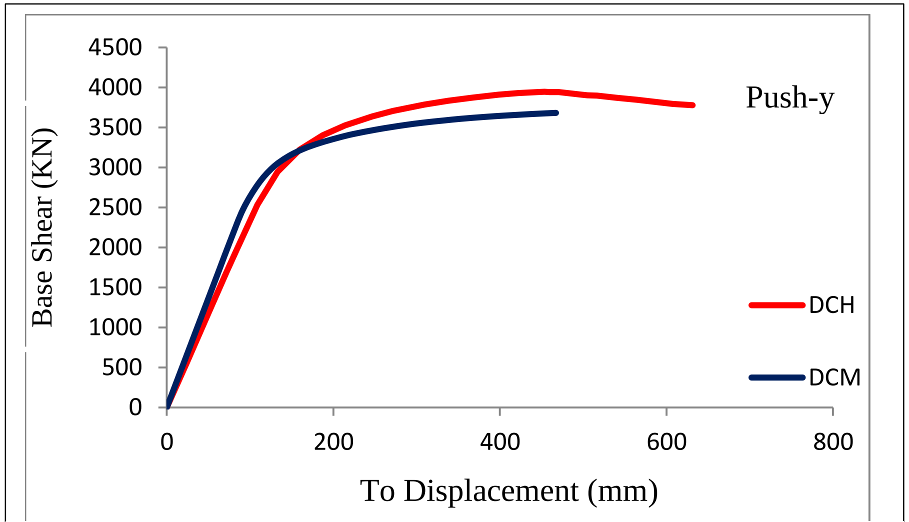


Figure 4 12Capacity Curve G+13 DCM and DCH Building ...PUSH – Y M-4

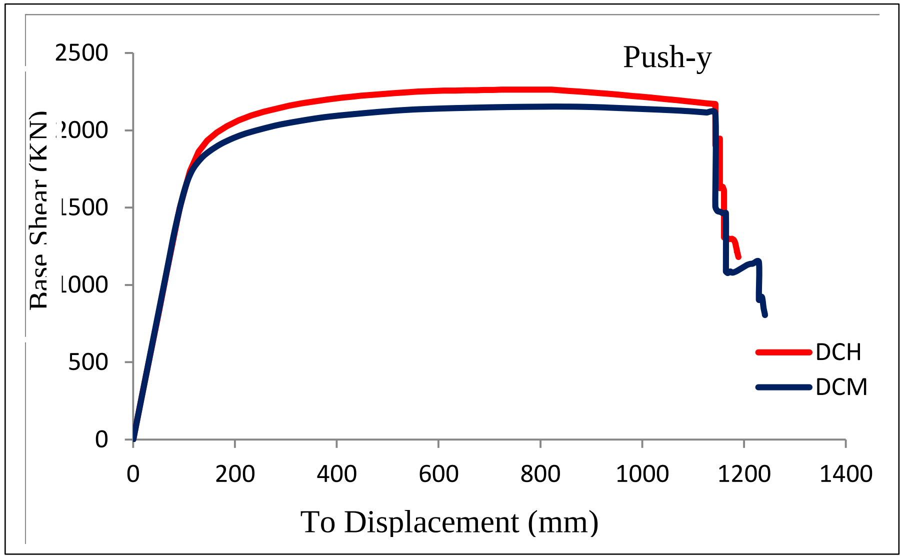


Figure 4 14Capacity Curve G+13 DCM and DCH Building ...PUSH – Y M- 5

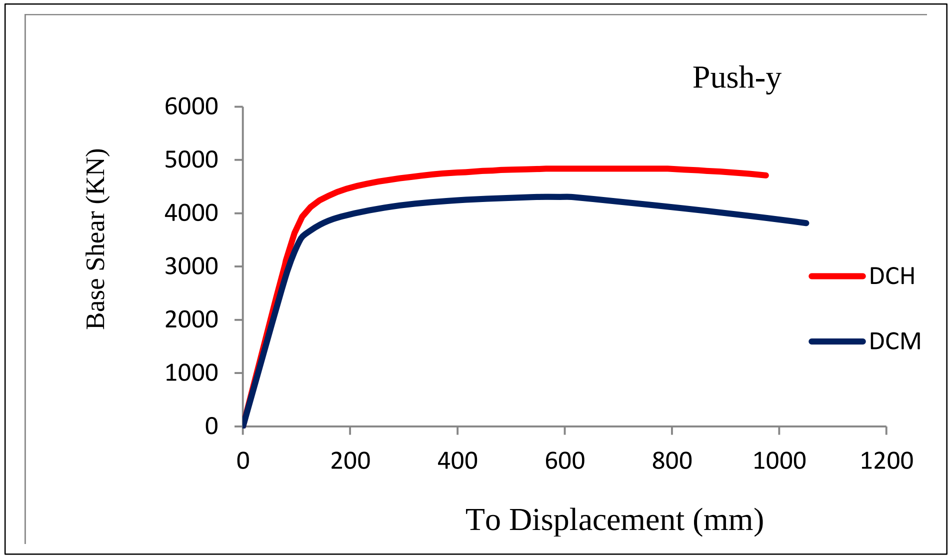


Figure 4 13Capacity Curve G+13 DCM and DCH Building ...PUSH – Y M-6

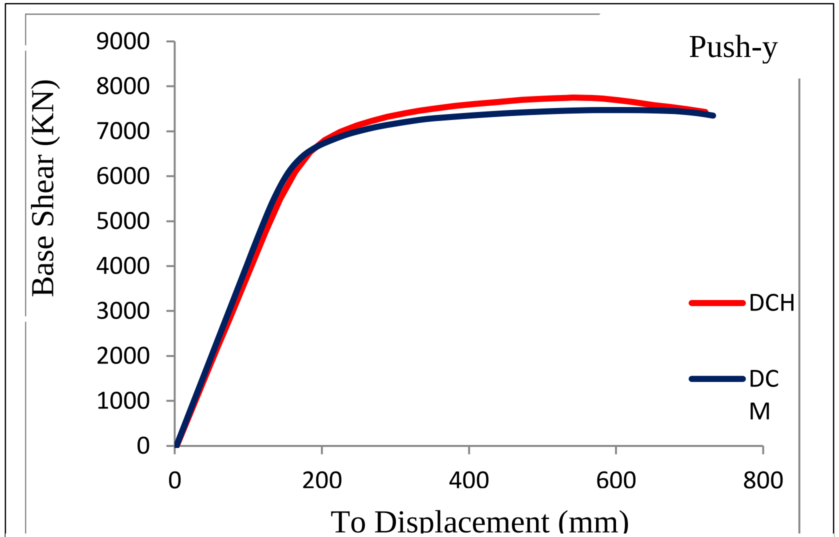


Figure 4 16Capacity Curve G+14 DCM and DCH Building ...PUSH – Y M-7

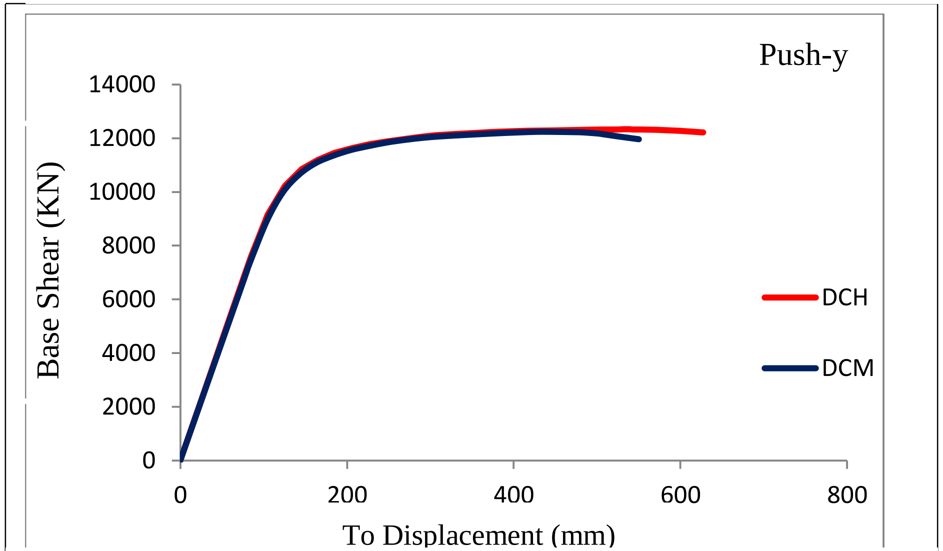


Figure 4 15Capacity Curve G+14 DCM and DCH Building ...PUSH – Y M-8

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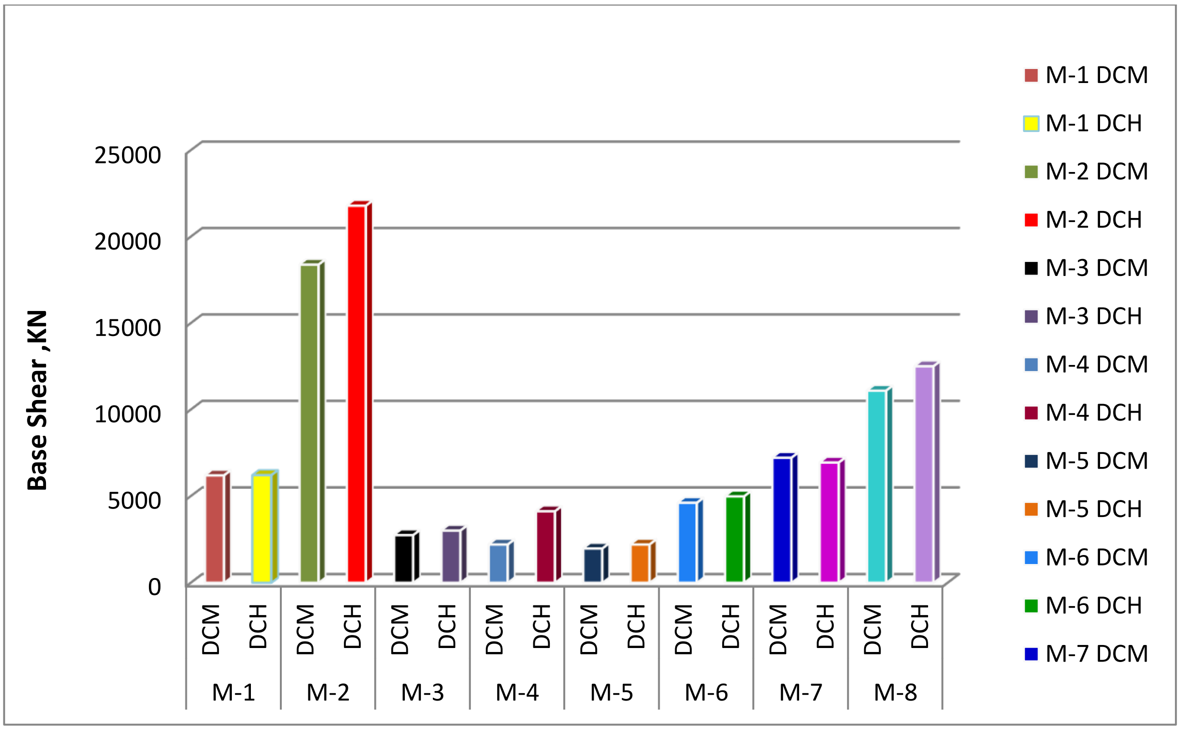

Figure 4 17Base Shear Bar graph DCM and DCH for M-1 up to M-8

4.2.2 Top Story Displacement

Top story displacement is an important second step to evaluate the seismic performance of the structure for this study. It can be observed that there is significant variation in top story displacement (roof displacement) when ductility class increases (from DCM to DCH). For the pushover-x the top-displacement the percentage increment of top displacements for M-1, M-2, M-3, M-4, M-6, M-7, and M-8 by 11.94%, 97.51%, 38.99%, 101.46%, 19.51%, 17.31% and 44.78% respectively. But for M-5 the top story displacement is decreased by 4.06 % as the ductility class increases from DCM to DCH. And For pushover-y the percentage increment of top displacements for M-1, M-2, M-3, M-4, M-5, M-6, M-7, and M-8 by 2.26%, 17.15%, 76.38%, 45.27%, 4.17%, 4.55%, 13.38%, and 12.24% respectively as the ductility class increase from DCM to DCH. This top displacement of the building is dependent on different structural parameters such as the material strength of the building and the stiffness of structural elements which is dependent on the size of the element. Thus, in all cases, the performance in terms of top displacement of structures designed for ductility class medium was found to be better than structures designed for ductility class high. The percentage increase in of top story displacement result is summarized in table 4.3, table 4.4.

Table 4. 3 Top Displacement increment for push-X

PUSH – X				
Building	Top Displacement			
	DCM	DCH	Net	%
M-1	506.257	574.903	68.646	11.94
M-2	300.555	593.619	293.064	97.51
M-3	511.416	838.232	326.816	38.99
M-4	437.809	882.016	444.207	101.46
M-5	1161.7	1116.37	-45.325	-4.06
M-6	729.213	906.019	176.806	19.51
M-7	82.138	99.332	17.194	17.31
-M-8	503.867	912.492	408.625	44.78

Table 4. 4 Top Displacement increment for push-Y

PUSH – Y				
Building	Top Displacement			
	DCM	DCH	Net	%
M-1	681.134	696.903	15.769	2.26
M-2	900.605	1087.07	186.46	17.15
M-3	266.817	1129.7	862.883	76.38
M-4	854.902	467.881	387.021	45.27
M-5	1189.74	1241.51	51.771	4.17
M-6	1111.19	1164.14	52.952	4.55
M-7	98.648	113.884	15.236	13.38
M-8	553.735	630.987	77.252	12.24

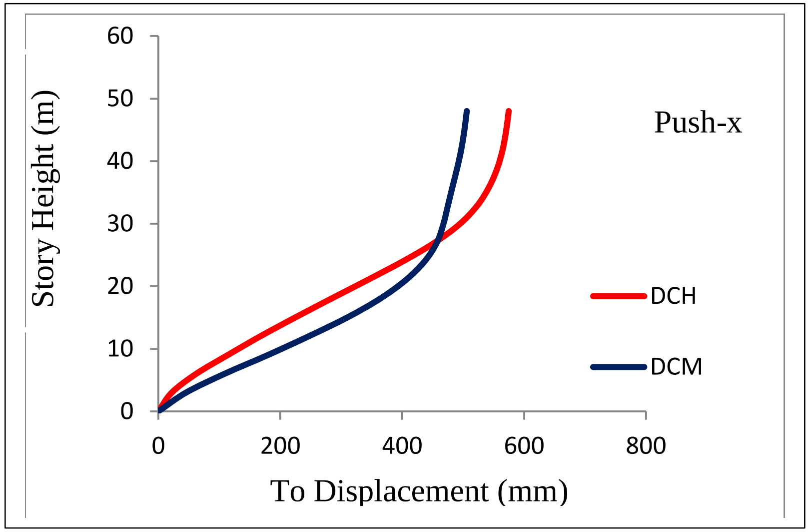

Figure 4 19Top Story Displacement G+15 DCM and DCH Building ...PUSH – X M-1

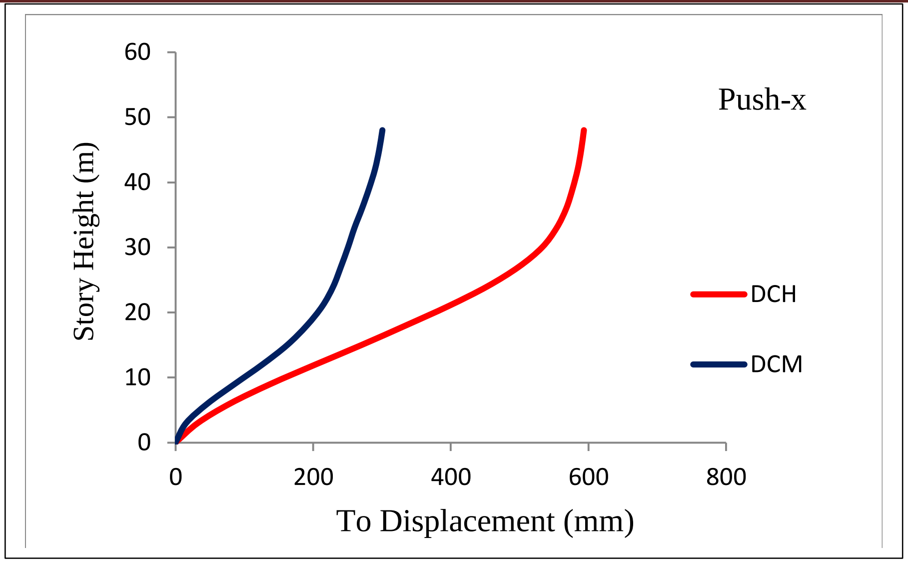

Figure 4 18Top Story Displacement G+15 DCM and DCH Building ...PUSH – X M-2

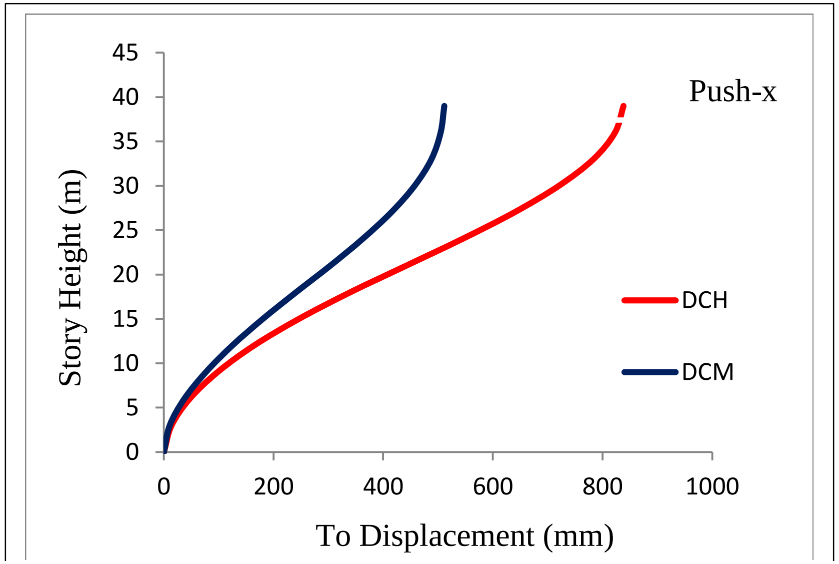

Figure 4 21Top Story Displacement G+13 DCM and DCH Building ...PUSH – X M-3

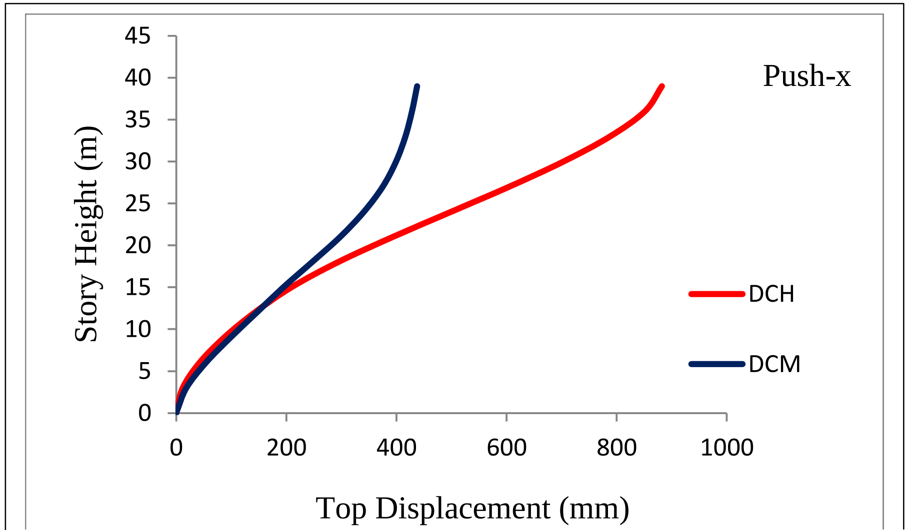

Figure 4 20Top Story Displacement G+13 DCM and DCH Building ...PUSH – X M-4

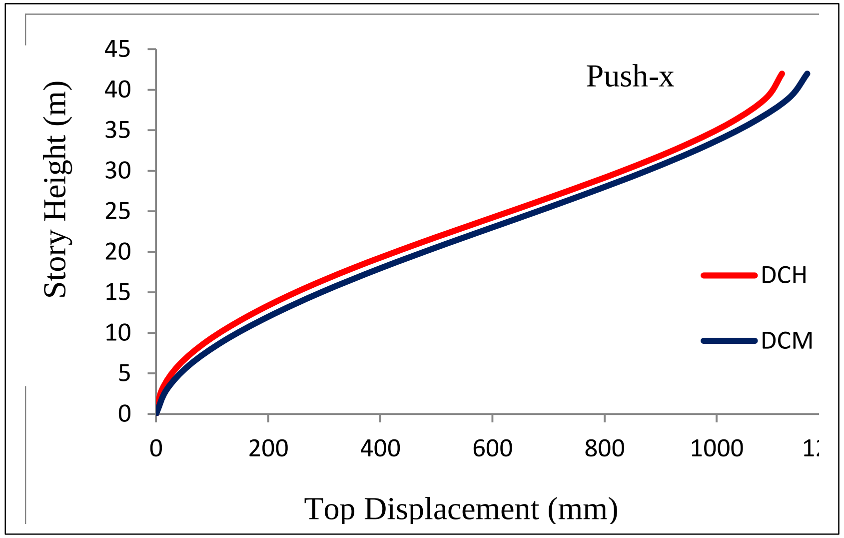

Figure 4 23Top Story Displacement G+13 DCM and DCH Building ...PUSH – X M-5

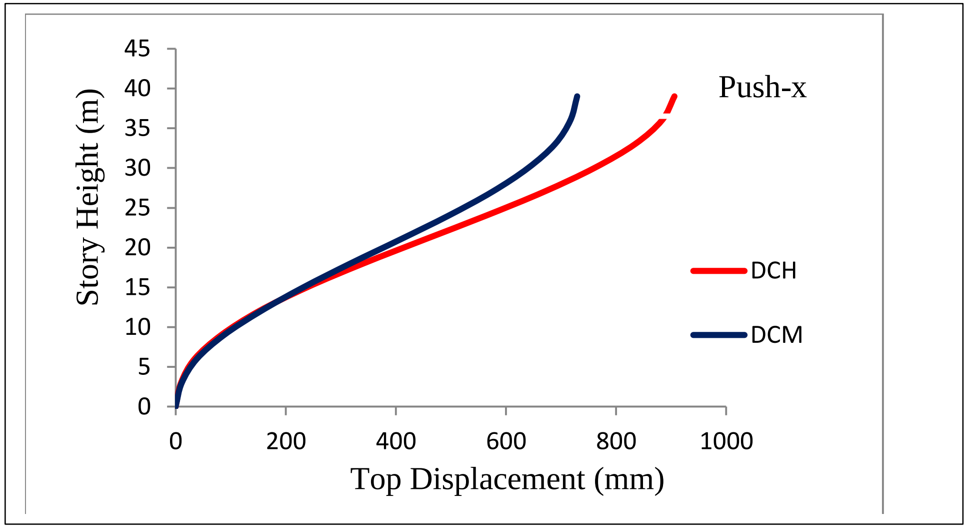

Figure 4 22Top Story Displacement G+13 DCM and DCH Building ...PUSH – X M-6

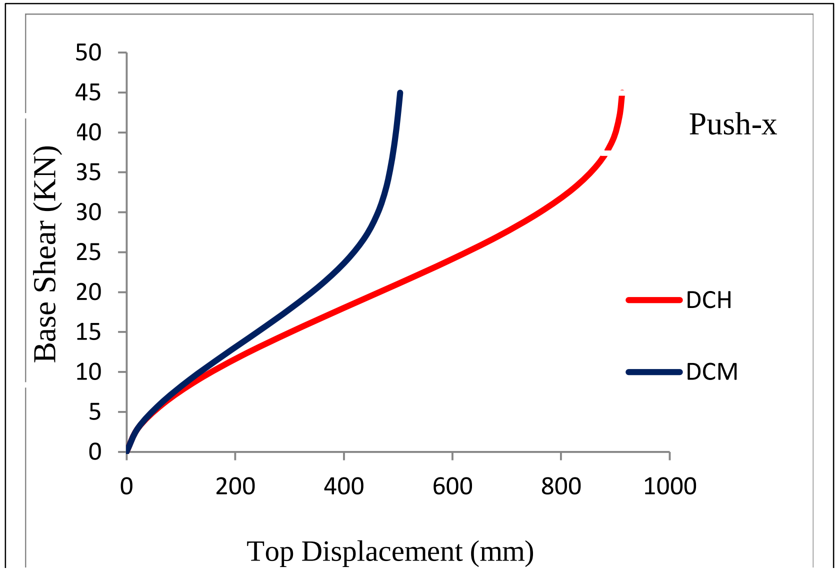

Figure 4 25Top Story Displacement G+14 DCM and DCH Building ...PUSH – X M-7

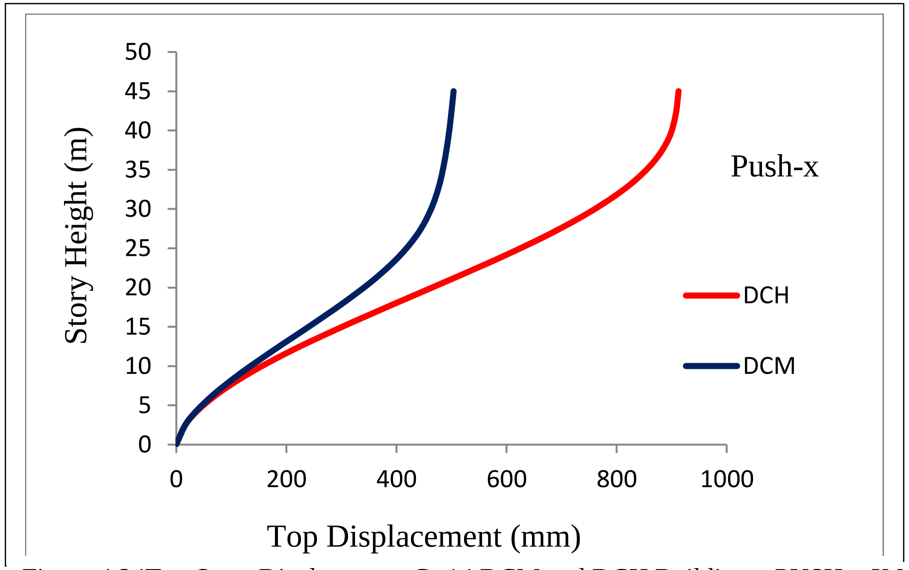

Figure 4 24Top Story Displacement G+14 DCM and DCH Building ...PUSH – X M-8

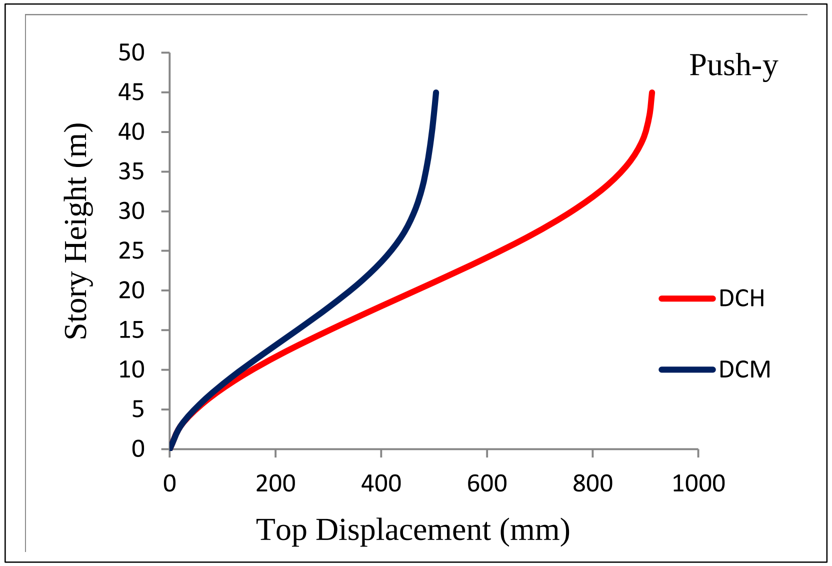

Figure 4 27Top Story Displacement G+15 DCM and DCH Building ...PUSH – Y M- 1

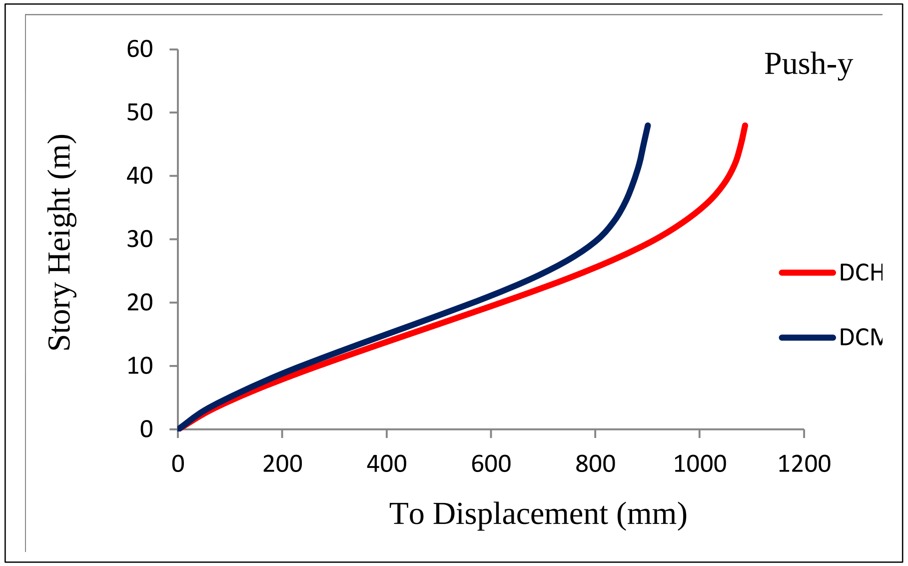

Figure 4 26Top Story Displacement G+15 DCM and DCH Building ...PUSH – Y M-2

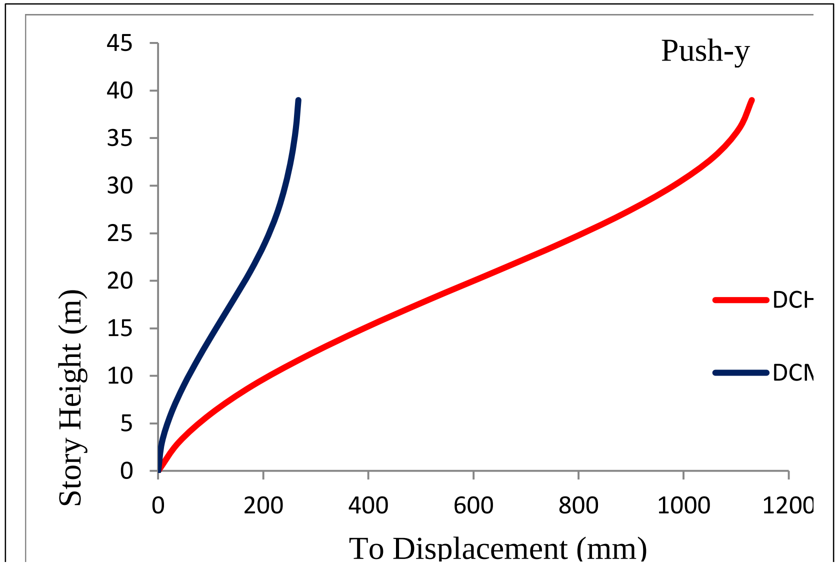

Figure 4 29Top Story Displacement G+13 DCM and DCH Building ...PUSH – Y M-3

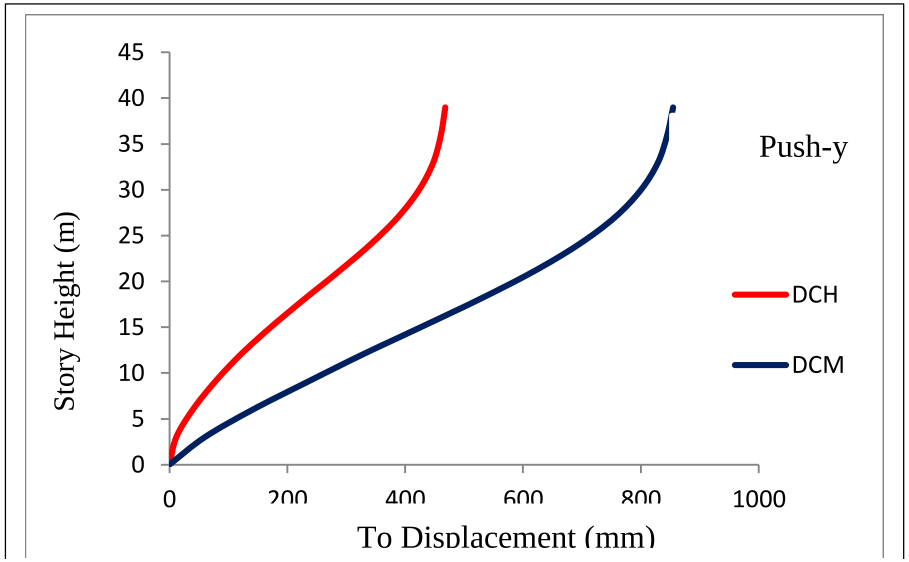

Figure 4 28Top Story Displacement G+13 DCM and DCH Building ...PUSH – Y M-4

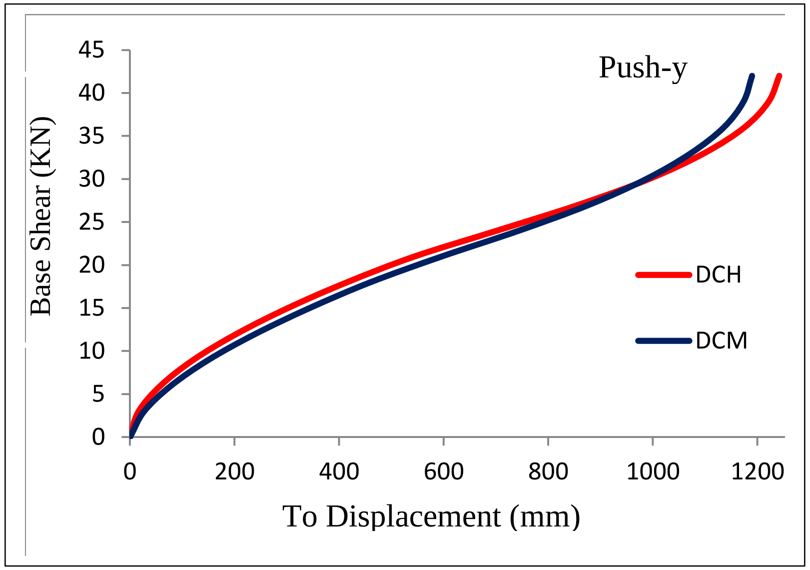

Figure 4 31Top Story Displacement G+13DCM and DCH Building ...PUSH – Y M-5

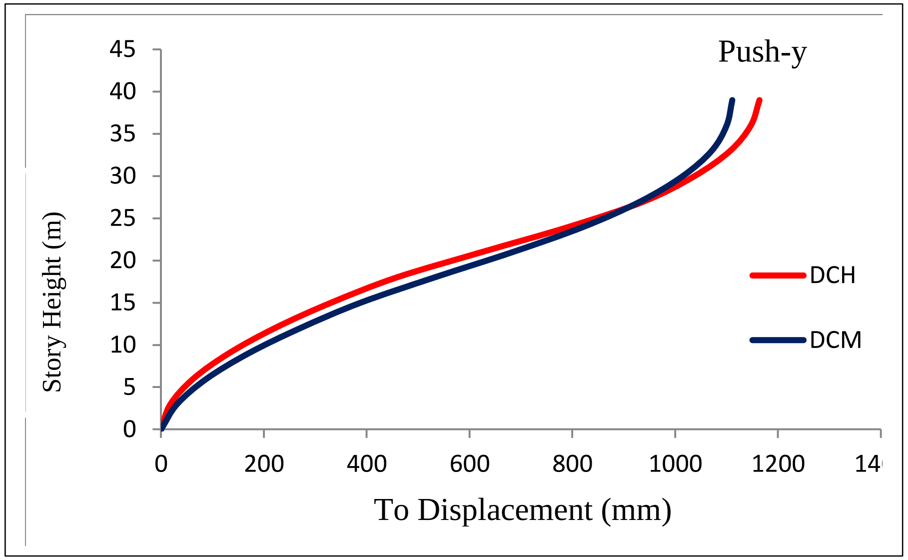

Figure 4 30Top Story Displacement G+13 DCM and DCH Building ...PUSH – Y M-6

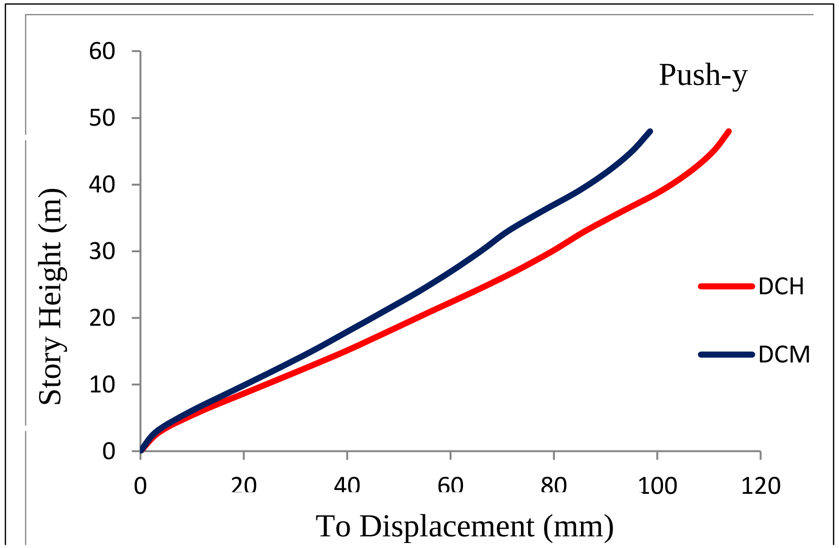

Figure 4 32Top Story Displacement G+14 DCM and DCH Building ...PUSH – Y M-7

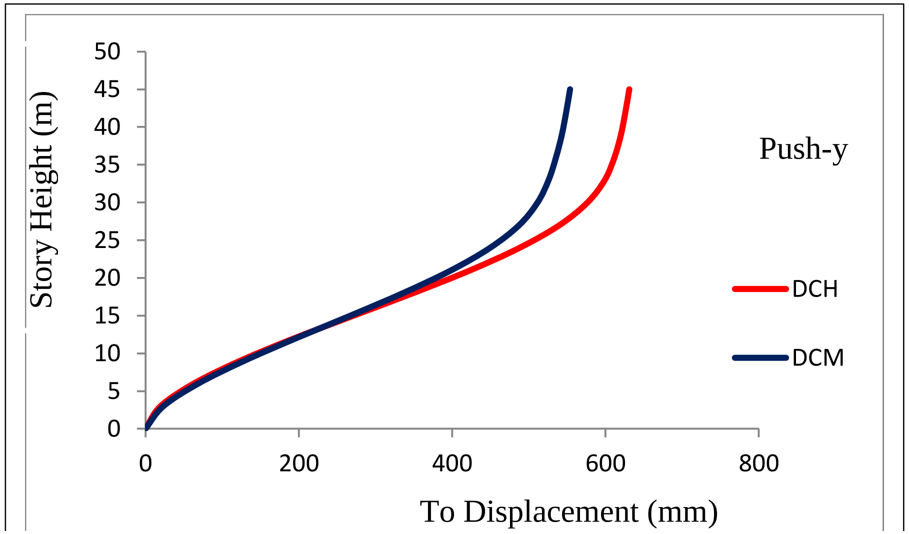

Figure 4 33Top Story Displacement G+14 DCM and DCH Building ...PUSH – Y M-8

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4.2.3 Inter-story Drift

Anther performance evaluation for this study is inter-story drift. The story drift of structures designed for medium ductility is found lesser than structures designed for ductility class high. This shows as the performance of structures designed for ductility class medium in drift resistance, are better than structures designed for ductility class high for both pushover-x and pushover-y. For the irregular reinforced concrete structure, the difference of the stiffness due to the setback ratio and behavioral factor results in the inter-story drifts vary consecutive models under their set back ratio. The story drifts result at the collapse point of the selected models for performance comparison is shown in the following figures.

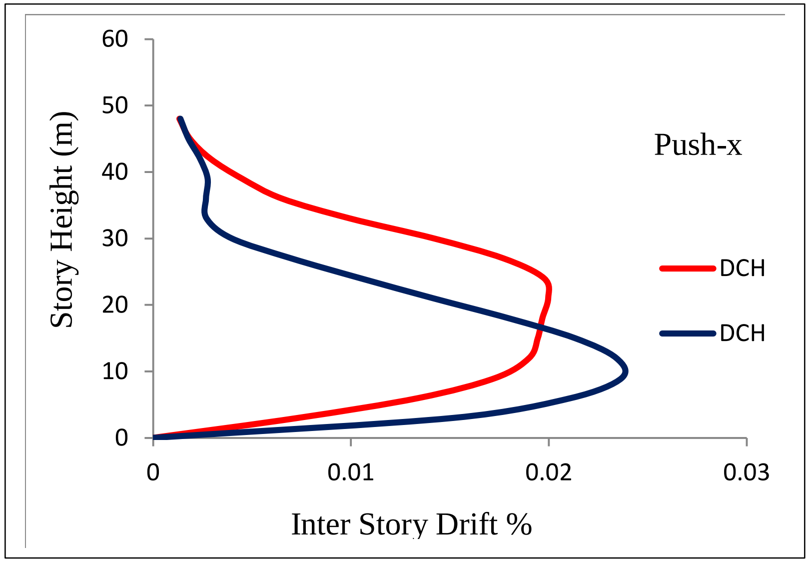

Figure 4 35Inter Story Drift G+15 DCM and DCH Building ...PUSH – X M-1

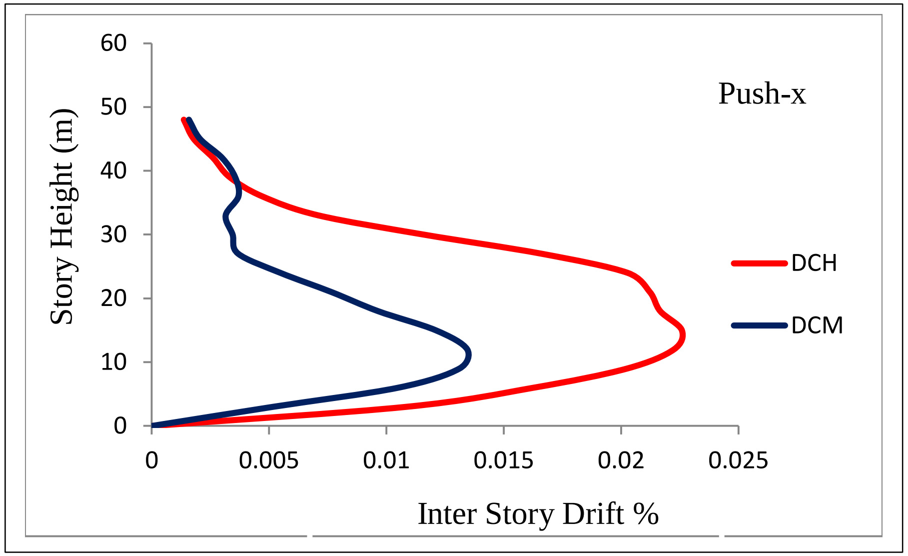

Figure 4 34Inter Story Drift G+15 DCM and DCH Building ...PUSH – X M-2

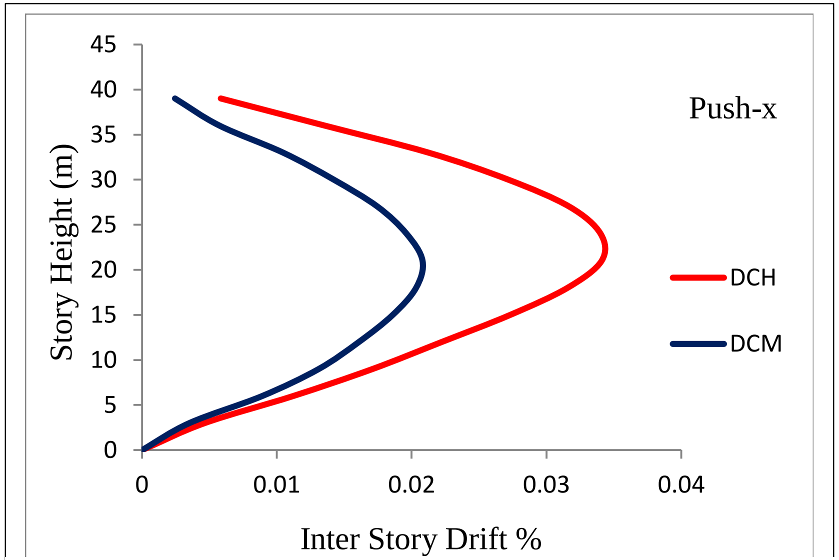

Figure 4 37Inter Story Drift G+13 DCM and DCH Building ...PUSH – X M-3

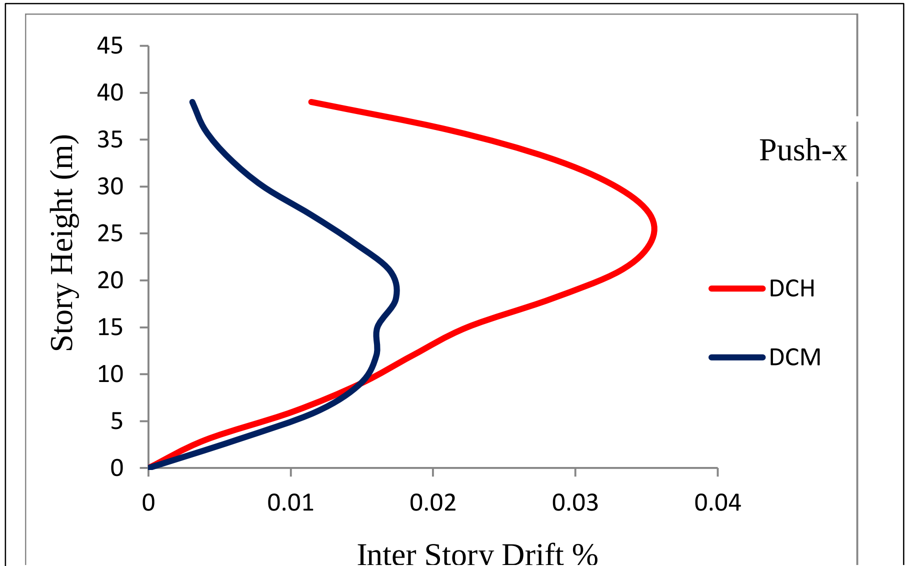

Figure 4 36Inter Story Drift G+13 DCM and DCH Building ...PUSH – X M-4

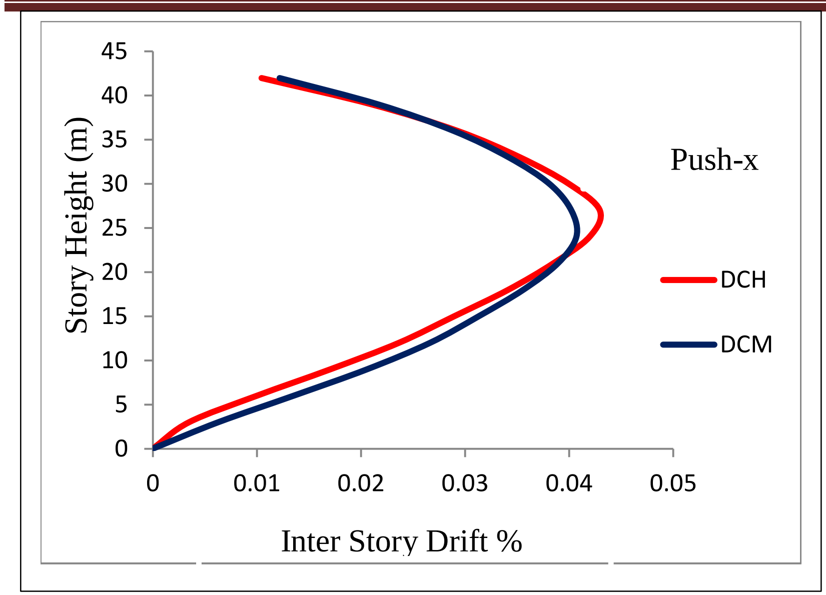

Figure 4 38Inter Story Drift G+13 DCM and DCH Building ...PUSH – X M-5

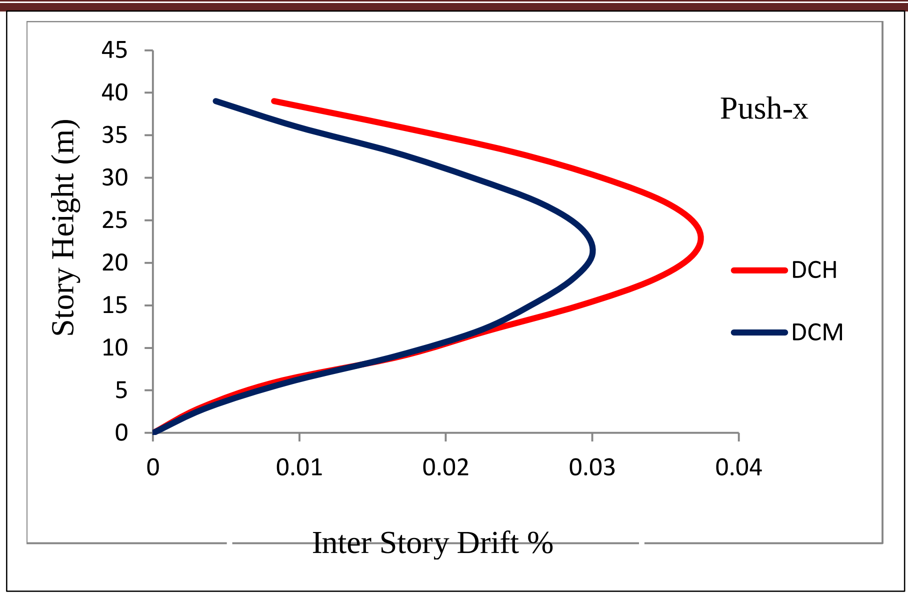

Figure 4 39Inter Story Drift G+13 DCM and DCH Building ...PUSH – X M-6

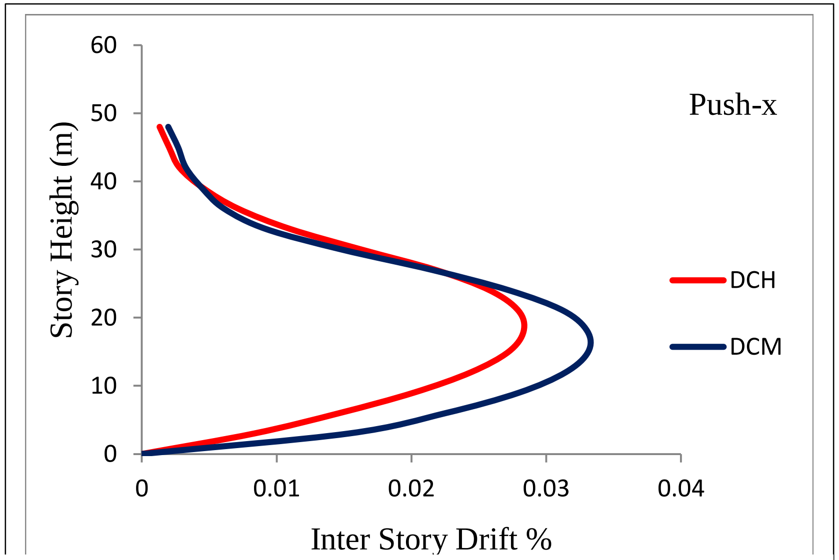

Figure 4 41Inter Story Drift G+14 DCM and DCH Building ...PUSH – X M-7

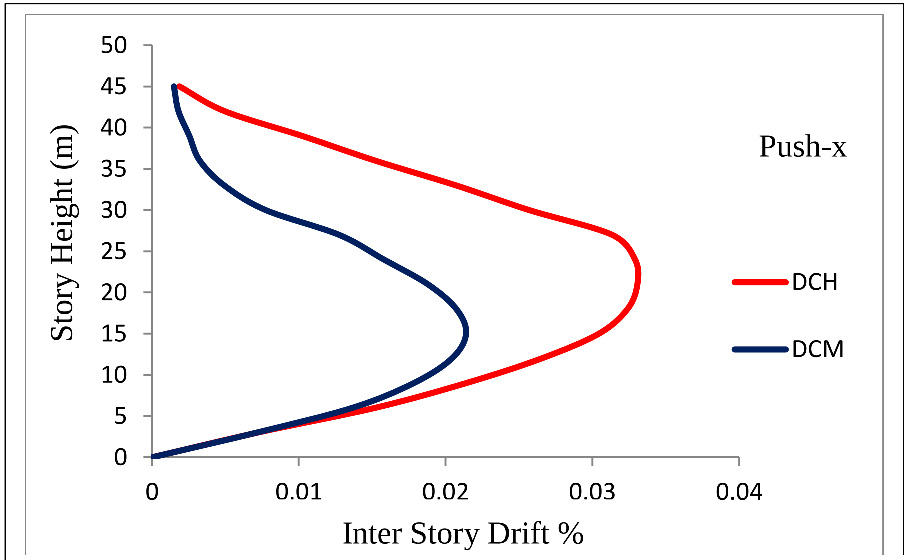

Figure 4 40Inter Story Drift G+14 DCM and DCH Building ...PUSH – X M-8

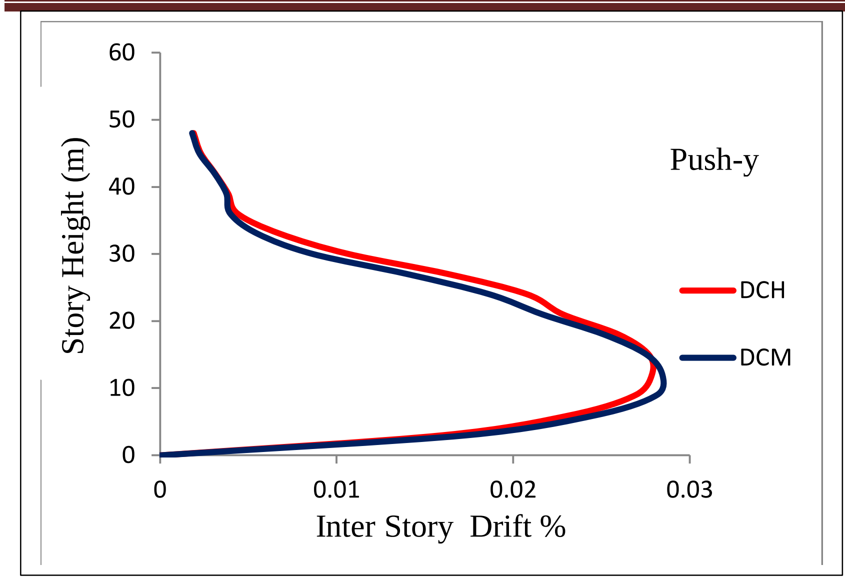

Figure 4 42Inter Story Drift G+15 DCM and DCH Building ...PUSH – Y M-1

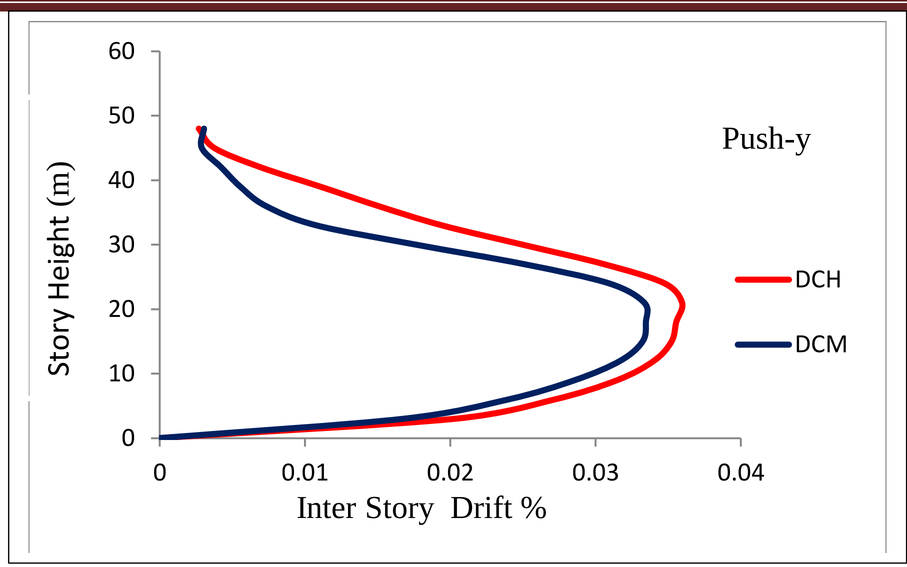

Figure 4 43Inter Story Drift G+15 DCM and DCH Building ...PUSH – Y M-2

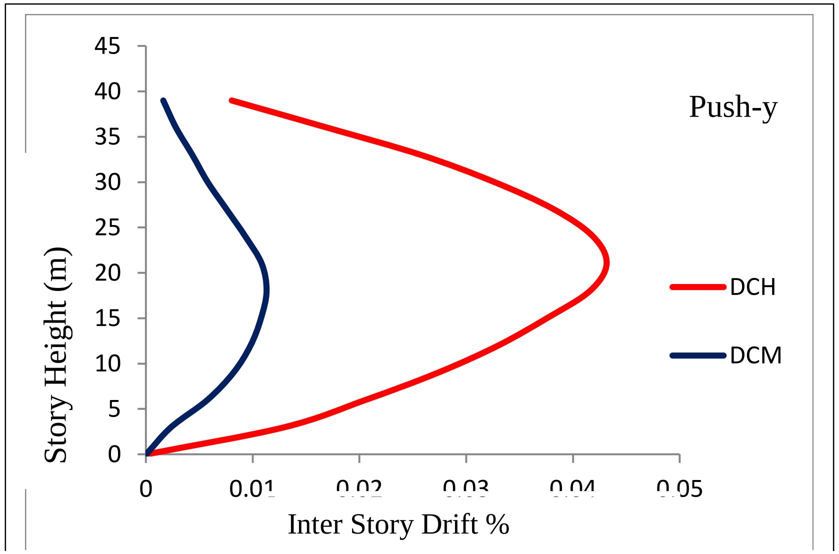

Figure 4 44Inter Story Drift G+13 DCM and DCH Building ...PUSH – Y M-3

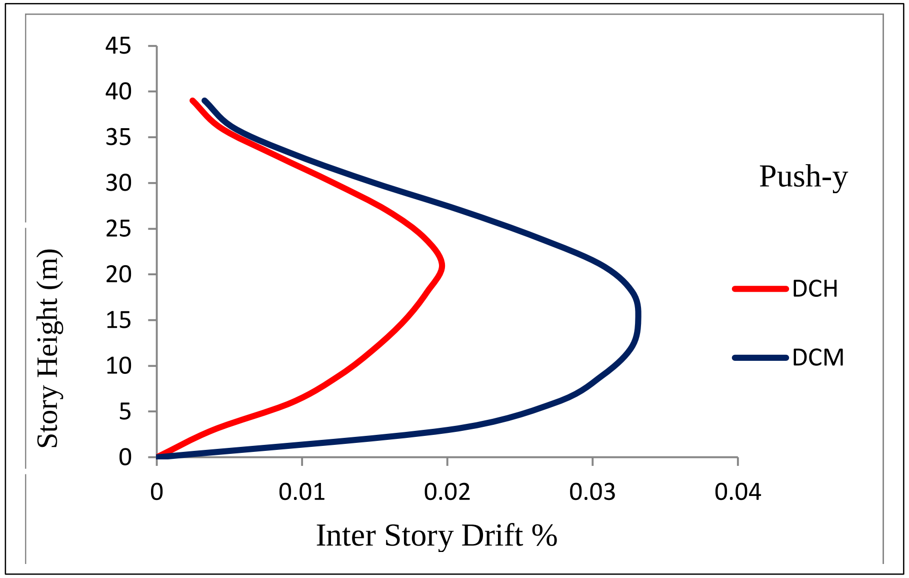

Figure 4 45Inter Story Drift G+13 DCM and DCH Building ...PUSH – Y M-4

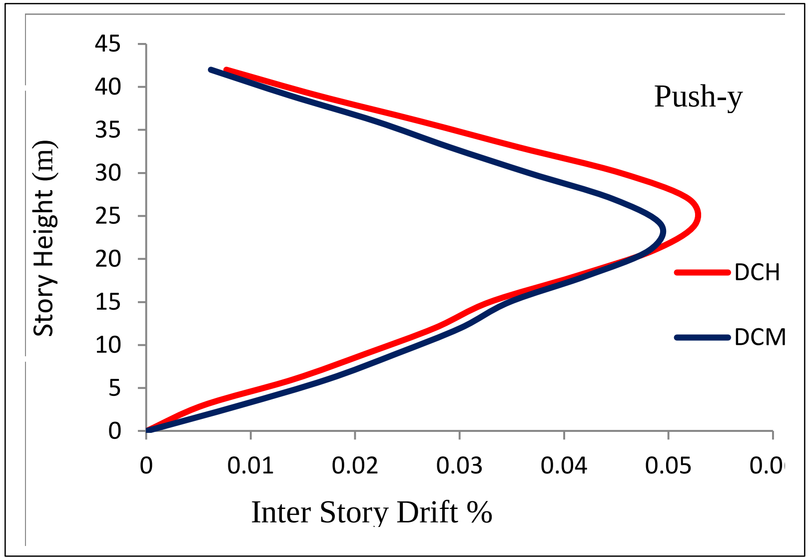

Figure 4 47Inter Story Drift G+13 DCM and DCH Building ...PUSH – Y M-5

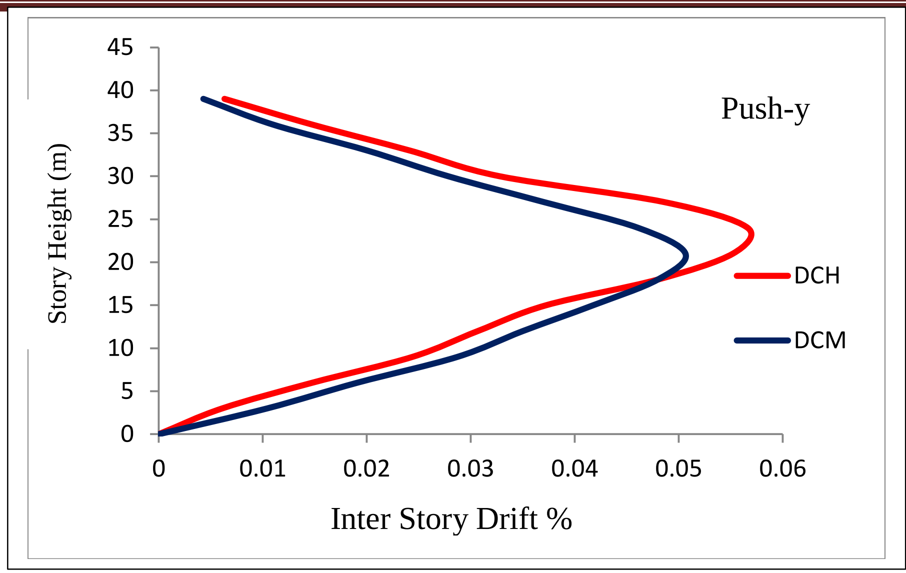

Figure 4 46Inter Story Drift G+13 DCM and DCH Building ...PUSH – Y M-6

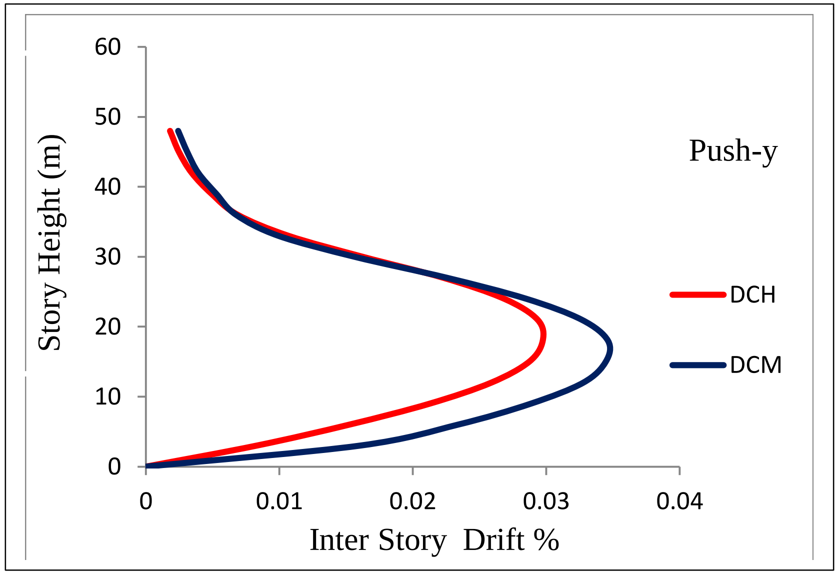

Figure 4 49Inter Story Drift G+14 DCM and DCH Building ...PUSH – Y M-7

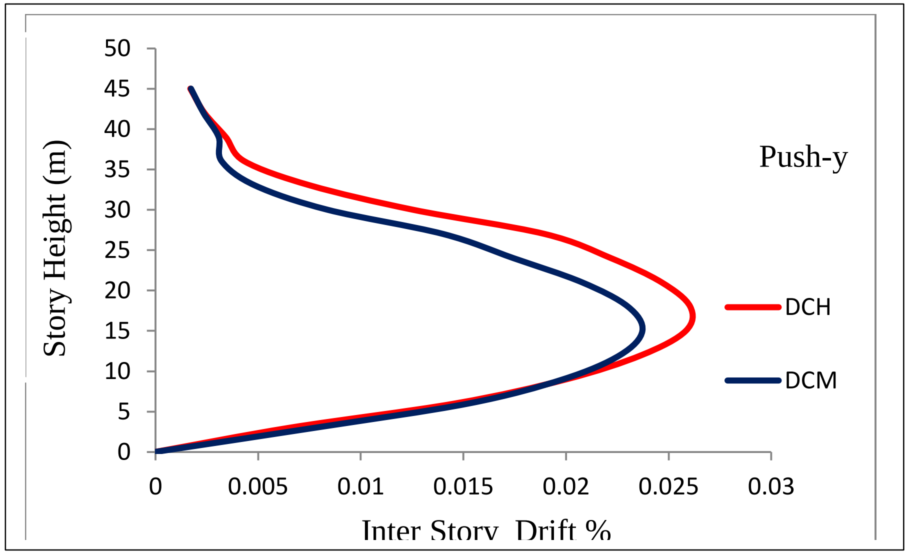

Figure 4 48Inter Story Drift G+14 DCM and DCH Building ...PUSH – Y M-8

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4.3.3 Plastic Hinge Distribution

This is a simple inelastic beam element for modeling beam columns. Plastic deformations are assumed to occur only at the two ends of the beam and column. Rigid zones at the ends of each element can be specified. The investigation revealed that in all the beams and columns of these study buildings, the critical region, potential location of the plastic hinge formation. The study shows from the ductility medium to the ductility high in each point or in each range the number of plastic hinges is differing.

Table 4. 5 Performance level for DCM and DCH in X and Y direction

Building	Ductility class	PUSH	Total hinges	A-IO	IO-LS	LS-CP	>CP	Net No. of A-IO	Net No. of IO-LS	Net No. of LS-CP	Net No. of >CP	Percentage A-IO	Percentage IO-LS	Percentage LS-CP	Percentage >CP
M-1	DCM	X	5976	4812	1124	20	20	812	784	20	8	16.87%	69.75%	100%	40%
	DCH			5624	340	0	12								
	DCM	Y		4676	1264	24	12	680	656	18	6	14.54%	51.90%	75%	50%
	DCH			5356	608	6	6								
M-2	DCM	X	13448	13418	0	0	30	14	0	-	14	0.10%	-	-	46.67%
	DCH			13432	0	0	16								
	DCM	Y		11644	1744	0	60	308	312	-	32	2.65%	17.89%	-	53.33%
	DCH			11952	1432	0	28								
M-3	DCM	X	2432	2394	0	0	38	38	0	-	38	1.59%	-	-	100%
	DCH			2432	0	0	0								
	DCM	Y		2062	184	32	154	-182	332	-4	146	-8.83%	180.43 %	-	95%
	DCH			1880	516	28	8								
M-4	DCM	X	2388	2320	56	8	4	68	56	8	4	2.93%	100%	100%	100%
	DCH			2388	0	0	0								
	DCM	Y		2266	94	16	12	450	476	16	10	19.86%	506.38 %	100%	83.33%
	DCH			1816	570	0	2								

Building	Ductility class	PUSH	Total hinges	A-IO	IO-LS	LS-CP	>CP	Net No. of A-IO	Net No. of IO-LS	Net No. of LS-CP	Net No. of >CP	Percentage A-IO	Percentage IO-LS	Percentage LS-CP	Percentage >CP
M-5	DCM	X	1684	1392	288	0	4	288	288	-	0	20.69%	100%	-	-
	DCH			1680	0	0	4								
	DCM	Y		1302	368	10	4	232	228	2	2	17.82%	61.96%	20%	50%
	DCH			1534	140	8	2								
M-6	DCM	X	3632	2968	576	0	8	656	576	-	0	22.10%	100%	-	-
	DCH			3624	0	0	8								
	DCM	Y		2720	808	16	36	756	692	12	-28	27.79%	85.64%	75%	77.78%
	DCH			3476	116	4	8								
M-7	DCM	X	5912	5284	610	0	14	-348	-352	-	-12	-6.59%	-57.70%	-	55.56%
	DCH			4936	962	0	9								
	DCM	Y		4766	1112	0	18	114	90	-	8	2.39%	8.09%	-	44.44%
	DCH			4880	1022	0	10								
M-8	DCM	X	11324	9360	1940	0	24	1960	1940	-	20	20.94%	100%	-	83.33%
	DCH			11320	0	0	4								
	DCM	Y		10114	1210	0	0	1210	1210	-	0	11.96%	100%	-	-
	DCH			11324	0	0	0								

From table 4-5, the number of hinges above CP in all buildings decreases as the ductility class increase from DCM to DCH at the last step of the analysis. This indicates that the week points of DCM buildings are greater than DCH Buildings.

For the M-1 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from the base up to 12 stories. From the total of 5976 hinges at step 14, the percentage of the number of the plastic hinge is 16.87% is increased in the range of Immediate Occupancy (IO), 69.75% is decreased in the range of IO – LS and the remaining 40% is decreased in the range greater than CP along push X. The percentage of the number of the plastic hinge is 14.5% is increased in the range of Immediate Occupancy (IO), 51.9% is decreased in the range of IO – LS and the remaining 50% is decreased in the range greater than CP along push Y.

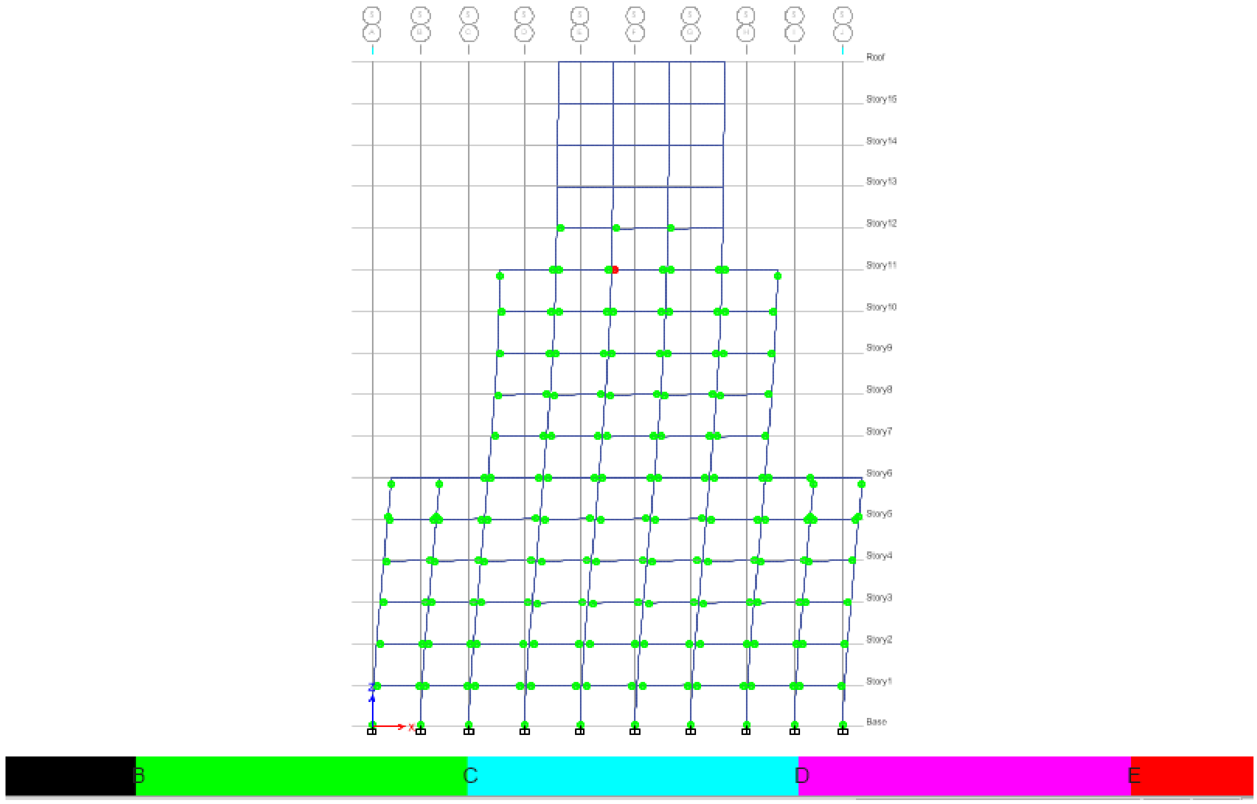


Figure 4 50Plastic Hinge Distribution DCM for M-1

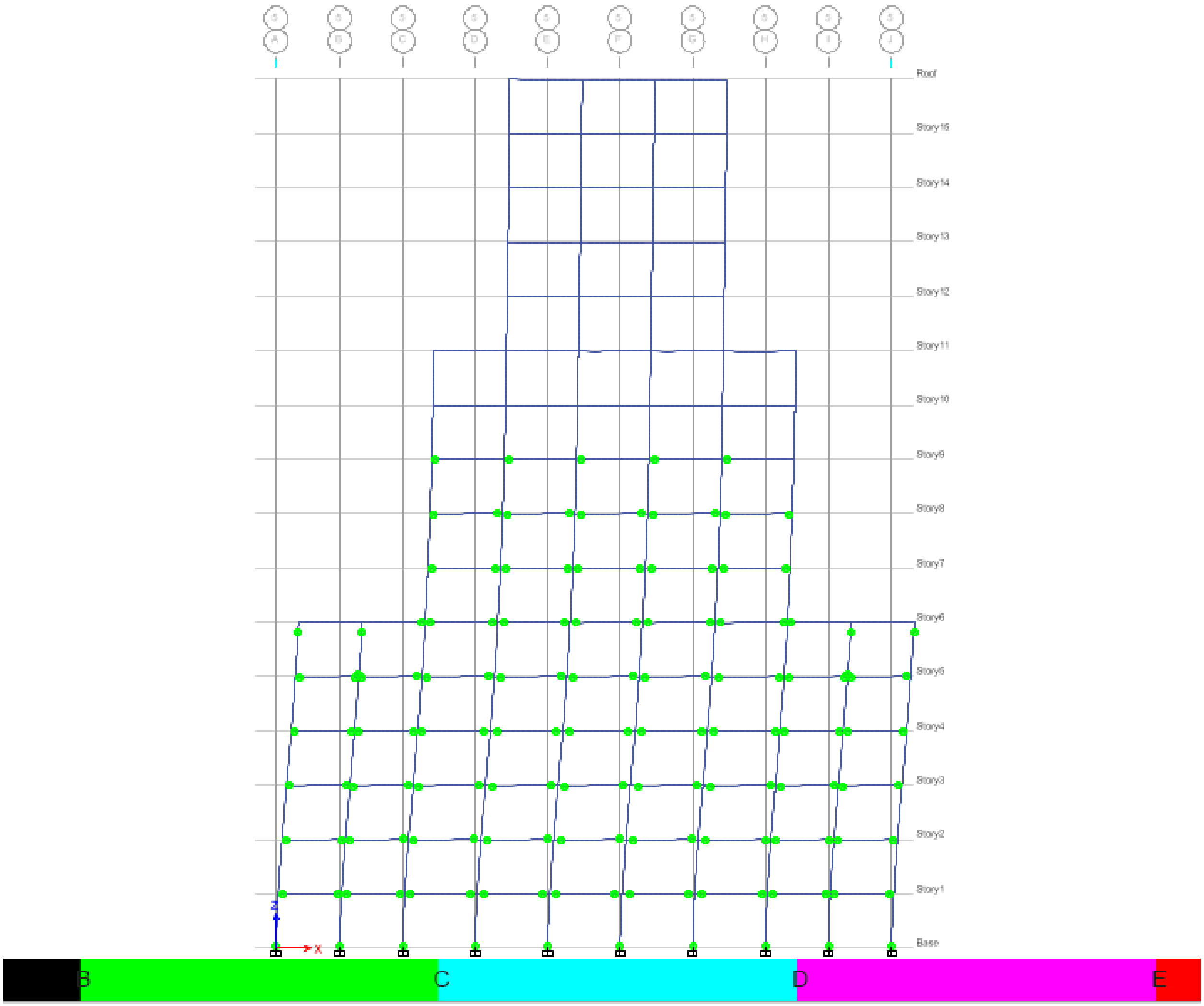


Figure 4 51Plastic Hinge Distribution DCH for M-1

The M-2 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 13448 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 0.10% in increased the range of Immediate Occupancy (IO), 0% is in the range of IO – LS, and the remaining 46.69% is decreased in the range greater than CP along push X. The percentage of the numbers of plastic hinges is 2.65% is increased in the range of Immediate Occupancy (IO), 17.89% is decreased in the range of IO – LS, and the remaining 53.33% is decreased in the range greater than CP along push Y.

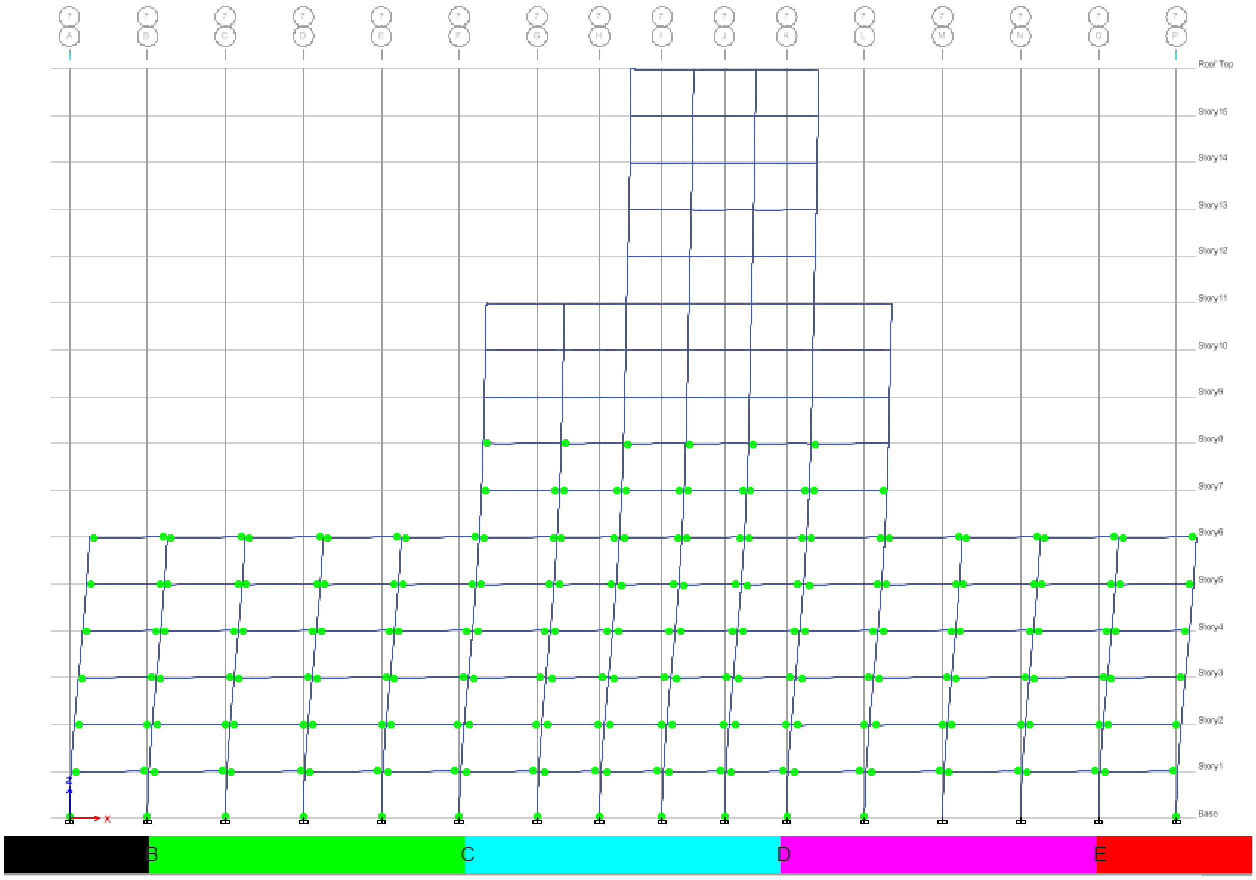


Figure 4 52Plastic hinge Distribution DCH For M-2

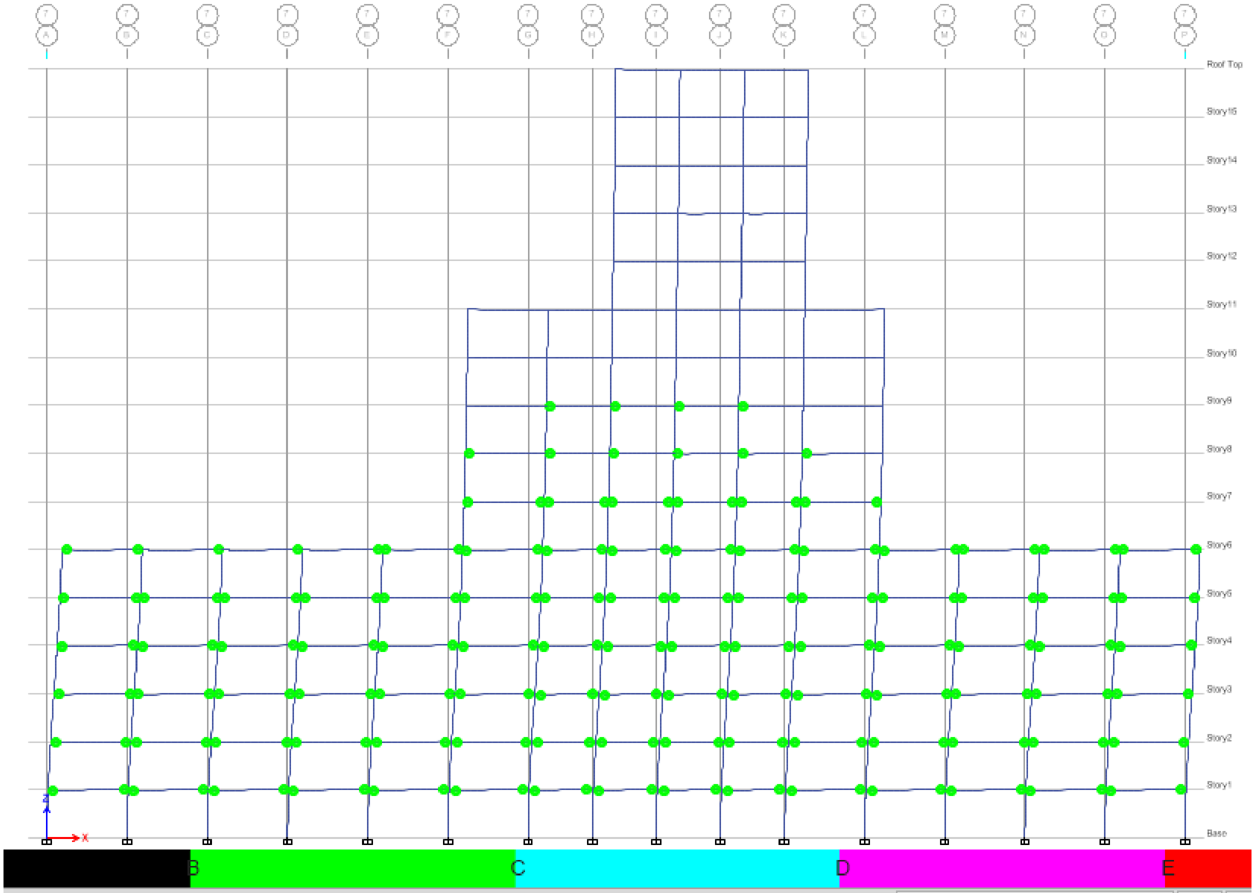
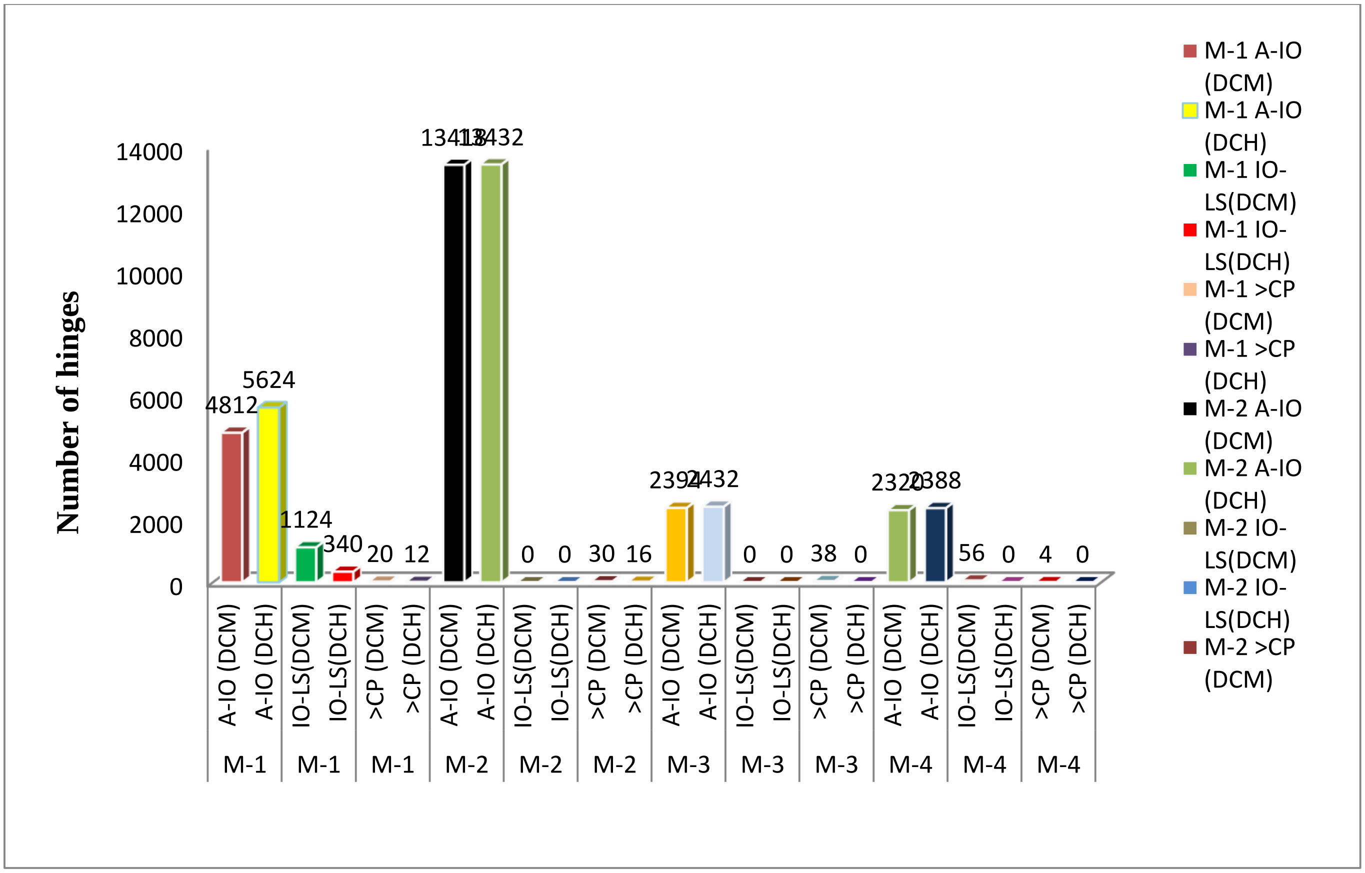


Figure 4 53Plastic hinge Distribution DCM for M-2

The M-3 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 13448 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 1.59% is increased in the range of Immediate Occupancy (IO), 0% is in the range of IO – LS, and the remaining 100% is decreased in the range greater than CP along push X. The percentage of the number of the plastic hinge is 8.83% is decreased in the range of Immediate Occupancy (IO), 180.43% is increased in the range of IO – LS, and the remaining 95% is decreased in the range greater than CP along push Y.

For the M-4 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from the base up to 8 stories. From the total of 2388 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 2.93% is increased in the range of Immediate

Occupancy (IO), 100% is decreased in the range of IO – LS and the remaining 100% is decreased in the range greater than CP along push X. The percentage of the number of the plastic hinge is 19.86% is decreased in the range of Immediate Occupancy (IO), 506.38% is increased in the range of IO – LS and the remaining 83.33% is decreased in the range greater than CP along push Y.



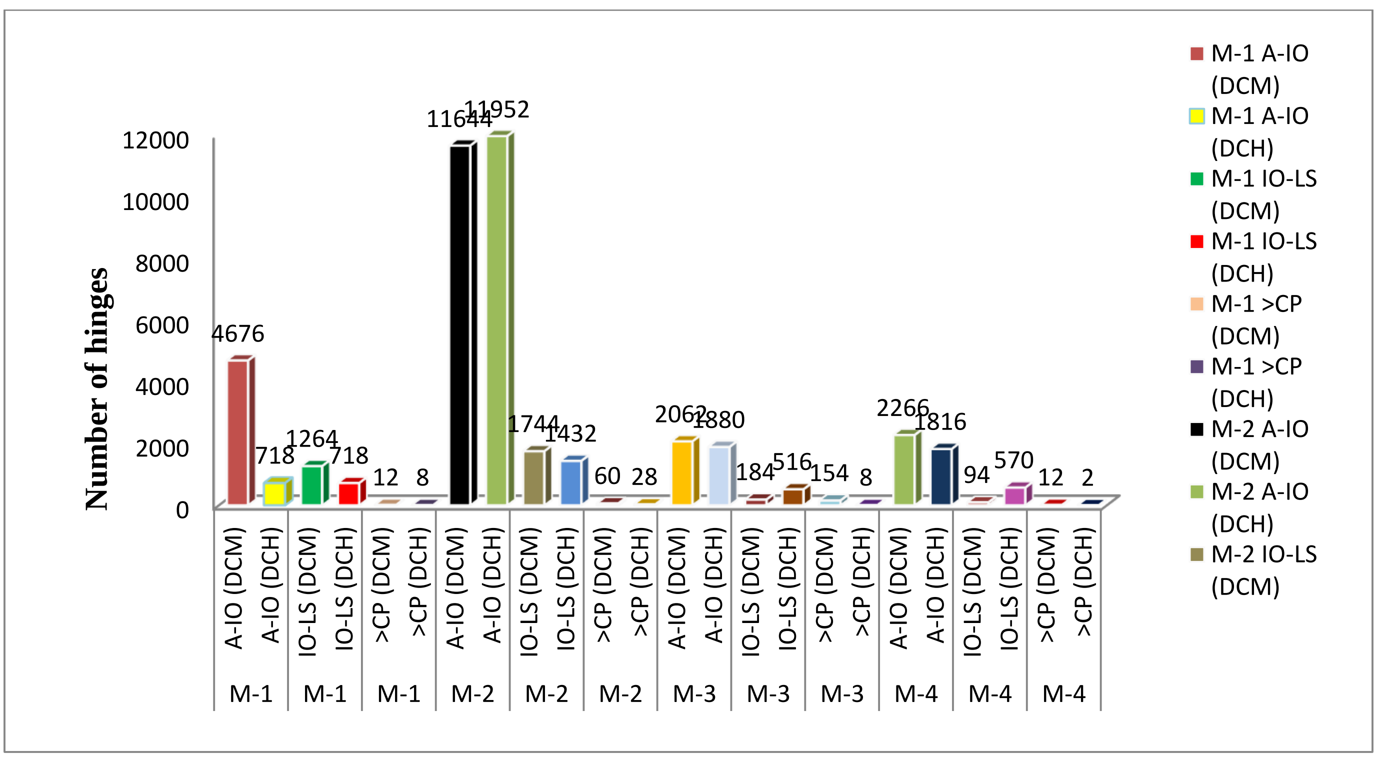


Figure 4 54Types and Number of Hinge from M-1 up to M-4 for push x and push y

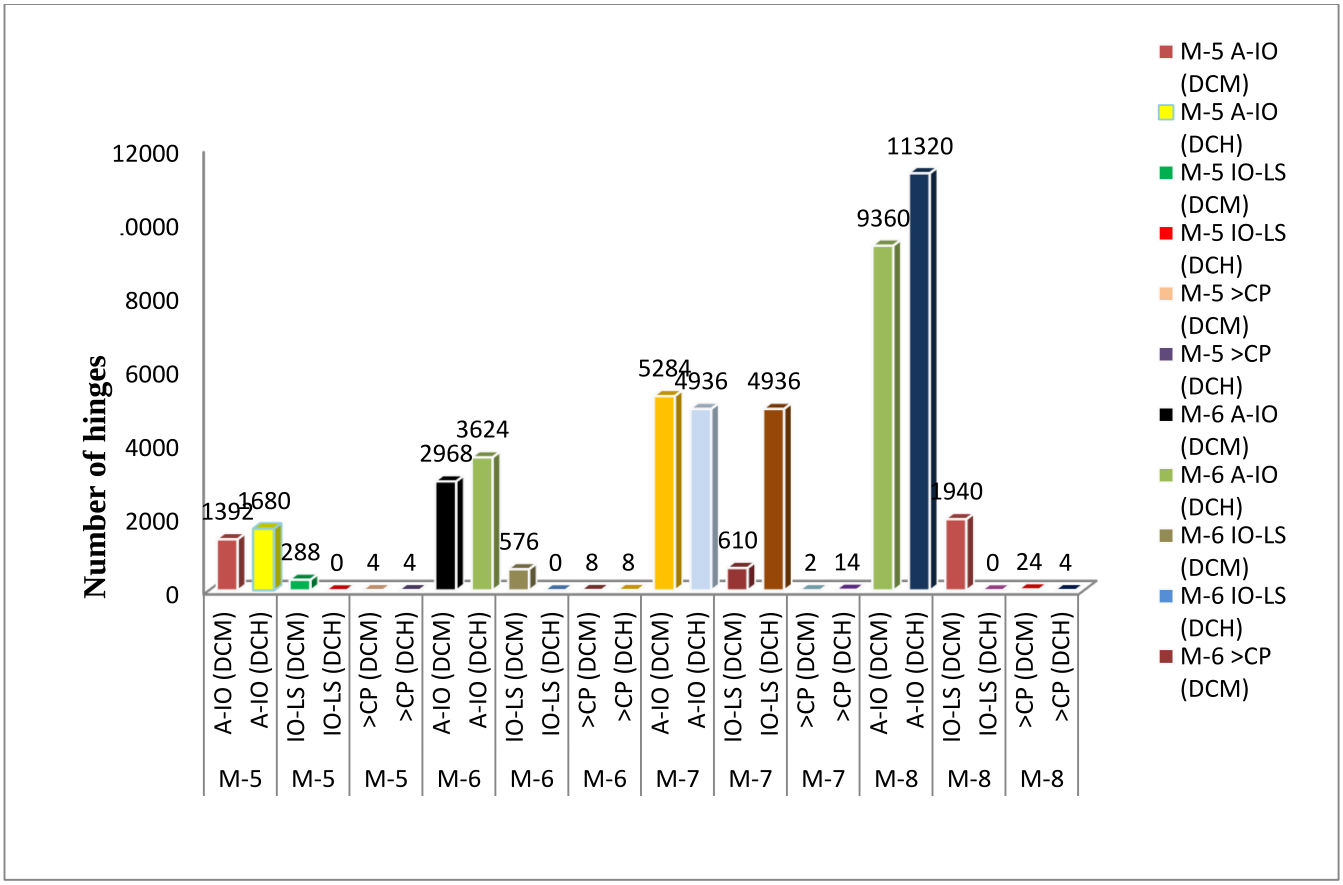
The M-5 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 1684 hinges at step 26 & 31, the percentage of the number of the plastic hinge is 20.69% is increased in the range of Immediate Occupancy (IO), 100% is decreased in the range of IO – LS and the remaining is the same number in the range greater than CP along push X. The percentage of the number of the plastic hinge is 17.82% is increased in the range of Immediate Occupancy (IO), 61.96% is decreased in the range of IO – LS and the remaining 50% is decreased in the range greater than CP along push Y.

The M-6 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 3632 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 22.1% is increased in the range of Immediate Occupancy (IO), 100% is decreased in the range of IO – LS and the remaining is the same number in the range greater than CP along push X. The percentage of the number of the plastic hinge is 27.79% is increased in the range of Immediate Occupancy (IO), 85.64% is decreased in the range of IO – LS and the remaining 77.77% is increased in the range greater than CP along with push Y.

The M-7 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 5912 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 6.59% is decreased in the range of Immediate Occupancy (IO), 57.7% is increased in the range of IO – LS, and the remaining 85.71% is increased in the range greater than CP along push X. The percentage of the number of the plastic hinge is 2.39% is increased in the range of Immediate Occupancy (IO), 8.09% is decreased in the range of IO – LS, and the remaining 44.44% is decreased in the range greater than CP along push Y.

The M-8 represents the plastic hinge distribution from the building designed for ductility class medium (DCM) to ductility class high (DCH) buildings. It has been observed that the plastic hinges occur from base up to 8 stories. From the total of 11324 hinges at steps 26 & 31, the percentage of the number of the plastic hinge is 20.94% is increased in the range of Immediate

Occupancy (IO), 100% is decreased in the range of IO – LS, and the remaining 8.33% is decreased in the range greater than CP along push X. The percentage of the number of the plastic hinge is 11.96% is increased in the range of Immediate Occupancy (IO), 100% is decreased in the range of IO – LS, and the remaining is the same in the range greater than CP along push Y.



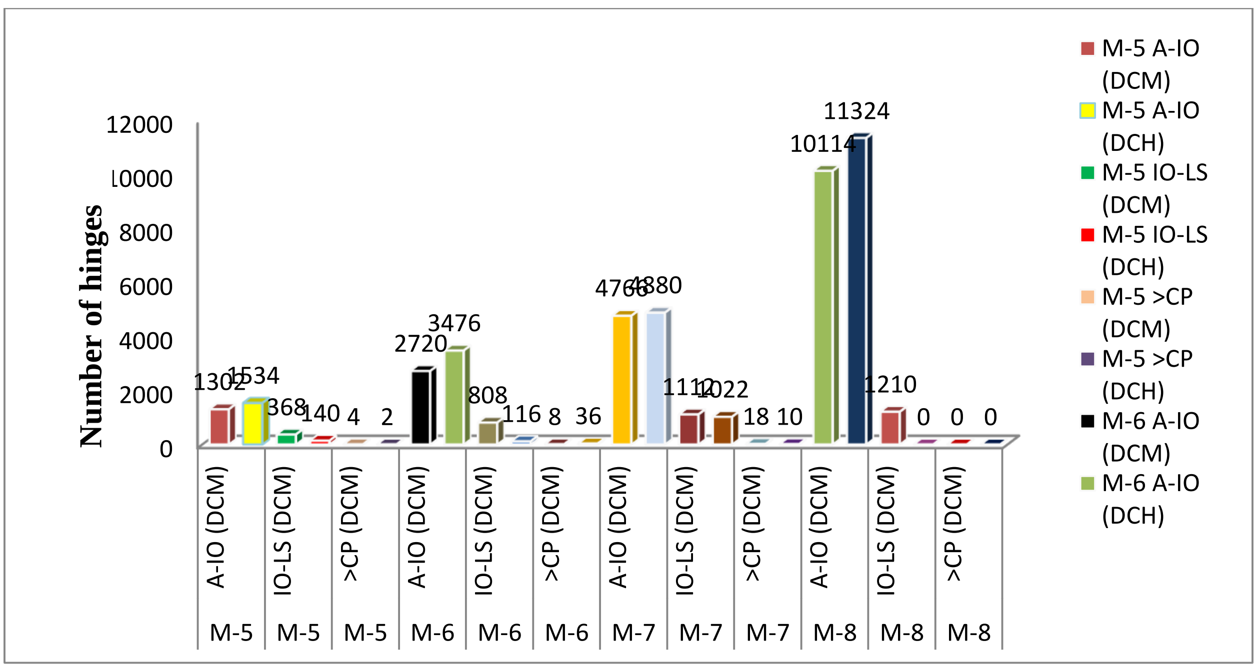


Figure 4 55Types and Number of hinge from M-5 up to M-8 push x and push y

Table 4. 6 Plastic hinge distribution for DCM and DCH buildings in X & Y Direction

Building	Ductility class	PUSH	Target Displacement	Base shear	A-B	B-C	Total hinge	Performance level
M-1	DCM	X	58.903	3103.613	5964	12	5976	IO
	DCH		38.98	2095.4437	5908	4	5912	IO
	DCM	Y	70.55	2609.068	5960	16	5976	IO
	DCH		50.621	1909.75	5960	16	5976	IO
M-2	DCM	X	62.757	9122.4261	13424	24	13448	IO
	DCH		41.138	6108.85	13440	8	13448	IO
	DCM	Y	70.929	7637.7292	13424	24	13448	IO
	DCH		46	5269	13440	8	13448	IO
M-3	DCM	X	51.726	1509.787	2424	8	2432	IO
	DCH		34.005	1026.548	2428	4	2432	IO
	DCM	Y	58.089	1394.787	2424	8	2432	IO
	DCH		39.916	993.8128	2428	4	2432	IO
M-4	DCM	X	53.683	1441.543	2384	4	2388	IO
	DCH		32.159	1113.2595	2384	4	2388	IO
	DCM	Y	57.259	1348.88	2384	4	2388	IO
	DCH		35.896	980.5708	2386	2	2388	IO
M-5	DCM	X	52.65	983.262	1968	8	1976	IO
	DCH		34.837	676.1887	1972	4	1976	IO
	DCM	Y	57	931.916	1950	26	1976	IO
	DCH		40.496	671.229	1966	10	1976	IO
M-6	DCM	X	51.475	2230.288	3808	8	3816	IO
	DCH		34.261	1521.9333	3888	8	3896	IO
	DCM	Y	55.834	2122.988	3804	12	3816	IO
	DCH		38.837	1503.8749	3884	12	3896	IO
M-7	DCM	X	77.787	3256.3434	5842	2	5844	IO
	DCH		73.54	3324.315	5908	4	5912	IO
	DCM	Y	83.465	3187.54	5842	2	5844	IO
	DCH		80.008	3271.7687	5908	4	5912	IO
M-8	DCM	X	59.837	6208.325	11300	24	11324	IO
	DCH		39.264	4211.4144	11320	4	11324	IO
	DCM	Y	66.214	5840.914	11312	12	11324	IO
	DCH		46.403	4148.3572	11318	6	11324	IO

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 CONCLUSIONS

The effect of ductility class for seismic performance of earthquake resistant irregular RC buildings are getting a lot of attention and the major building code authorities around the world are stressing the need for evaluating the performance seismic design of buildings. Various performance goals, building categories based on regular or irregular, and general seismic performance evaluation technic like capacity curve, top story displacement, inter-story drift and plastic hinge mechanisms are briefly reviewed in the research.

Conclusions derived based on this thesis study, are presented as follows

- ✓ As the seismic performance of structures designed for different ductility classes was investigated in this study, in terms of deformation capacity, structures designed for high ductility class performs well better than structures designed for medium ductility class, since the detailing rules for ductility class high is tighter.
- ✓ The capacity curve is one of evaluating the performance of a building seismic loads depends on types of structure, ductility type and regularity requirements, in terms of base shear capacity, structures designed for high ductility class were found better than structures designed for medium ductility, because for these types of structures, bigger sections and the larger amount of longitudinal reinforcement bar especially in beams were used. In addition, because bigger sections were used in structures designed for High ductility as a result of greater base sear.
- ✓ The drift resistances of structures, designed for high ductility were found better than structures designed for medium ductility. The largest drift value of structures designed for high ductility was within life safety performance limit while the largest story drifts for medium ductility structures.
- ✓ Number and pattern of the hinge formation observed in push-over analyses for ductility classes it can be concluded from the results of failure mechanism of the planner frame used

to show the formation of plastic hinges (i.e. location and sequence), edge and corner base columns were more susceptible to nonlinear rotation. Thus, the critical length of those columns should be detailed tighter to make better deformation capacity.

- ✓ From the response demand versus capacity curve of the sample buildings in this research the demand curve intersects the capacity envelope near the elastic range. At the performance point, Most of the hinges formed in the sample buildings are at performance level A to B which shows that the buildings are safe and has a good resistance for expected earthquake forces and the performance level of all buildings are Immediate Occupancy (IO) but the number of plastic hinges distribution on the levels of A-B and B-C are not the same in all sample buildings as the ductility classes increased from DCM to DCH. Then it can conclude that the number of hinges in level A-B is greater in DCH than in DCM samples and the reverse is true in level B-C.

Generally, the seismic performance of structures designed for different ductility classes and irregularity was investigated in this study, from the performance evaluation parameters in this study like capacity curve, inter story drift, roof displacement and plastic hinges. As the irregularity present in each type increases the high ductility class performs well better than structures designed for medium ductility class.

5.2 RECOMMENDATION

The research work presented in this thesis evaluation of the performance of ductility class on the vertical geometric irregular RC moment-resisting frame buildings. And further possible research areas are recommended hereunder:

- ➢ This research studies only the new designing of reinforced concrete structures with a deferent cross-section of members for the ductility class. So it is better to take the boundary section which satisfies the design need for both ductility classes to preferable performance evaluation for irregular structures.
- ➢ The irregular setback ratio percent is taken in this research is not close to each other. Another researcher can take the percentage of the setback with closest each other to get the perfect evaluation for the performance of the irregular reinforced concrete structure.

- Evaluation of the effect of ductility class on the performance of reinforced concrete for irregular structures (stiffness, mass, and the combination of both)
- In this research, the effect of soil-structure interactions and infill walls were not included. Therefore, it is required to study the evaluation of the effect of ductility class on the performance of reinforced concrete the effect of these factors be considered.
- Since this research is done only for moment-resisting frames, the evaluation of the effect of ductility class on the performance of reinforced concrete for dual and wall systems is recommended.
- The study presented here should be extended to include a variety of other buildings with various configuration and heights, and different building materials such as steel and composite construction.

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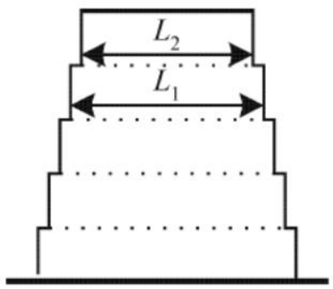
APPENDIX-A

A.1. **The Floor Plan and Chosen Framing System**

1. Condition one

$\boldsymbol{Case\ I}$ $\frac{L_1-L_2}{L_1} \le 0.2$ $Take\ 0.4\ or\ 40\%$ $and\ L_2 = 12\ with\ 3\ bays\ @4m\ \ will\ be\ L_1 = 20$

(a)



$$\frac{L_1 - L_2}{L_1} \le 0.20$$

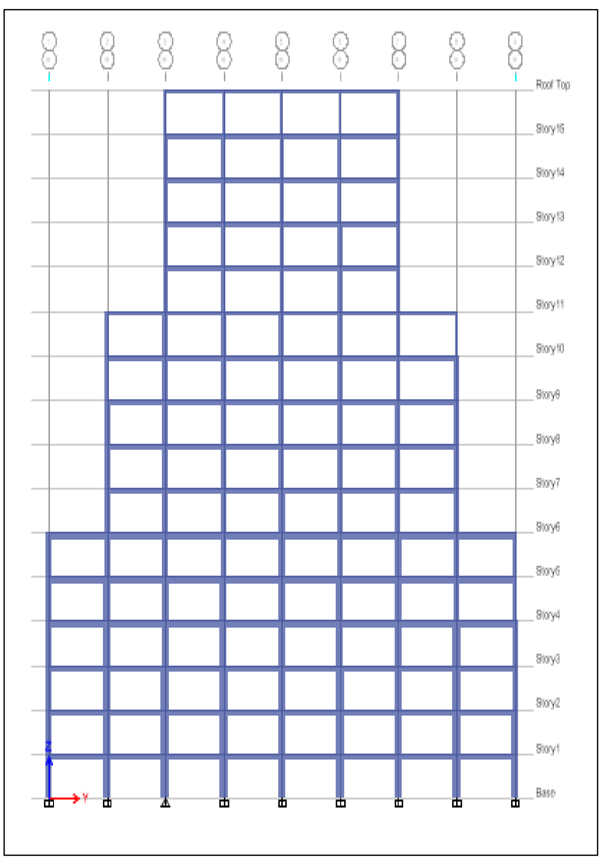


Figure A1 1 Elevation M-1 X-axis

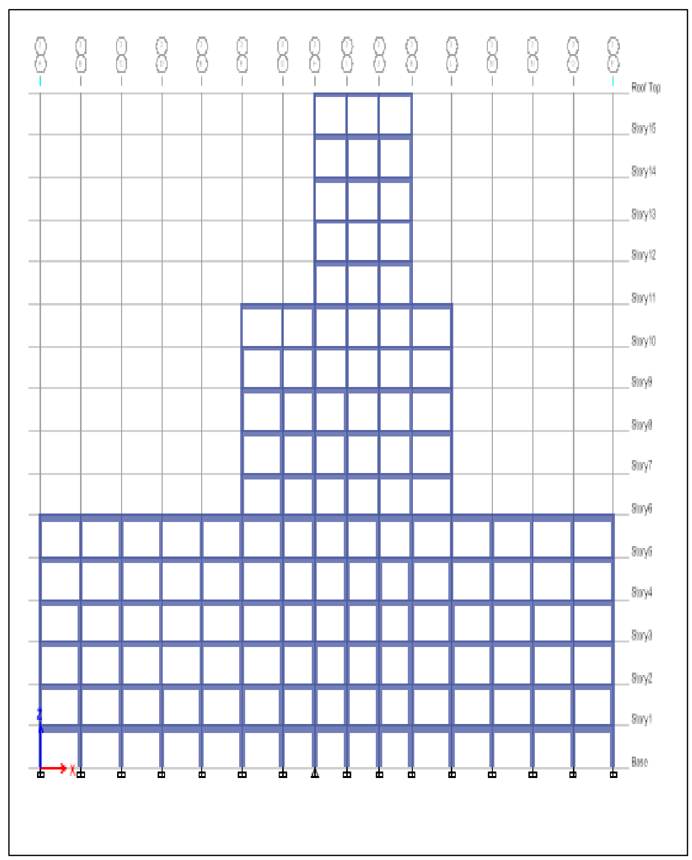


Figure A1 2 Elevation for M-1 Y axis

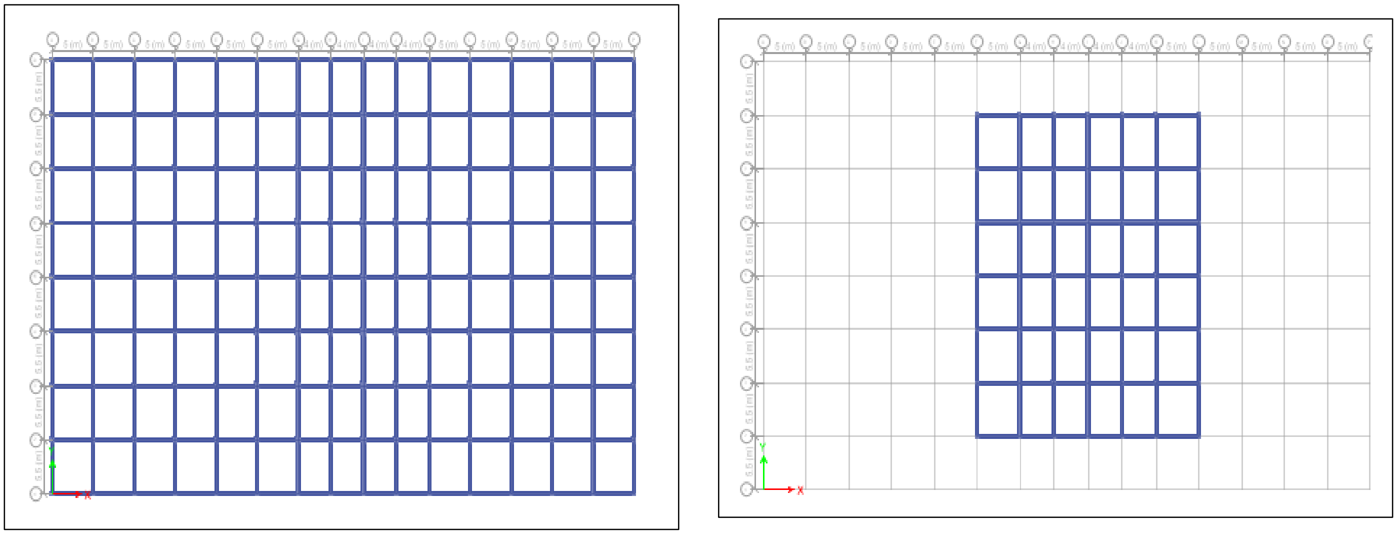


Figure A1 3Sample floor plan for M-1

2. Condition two

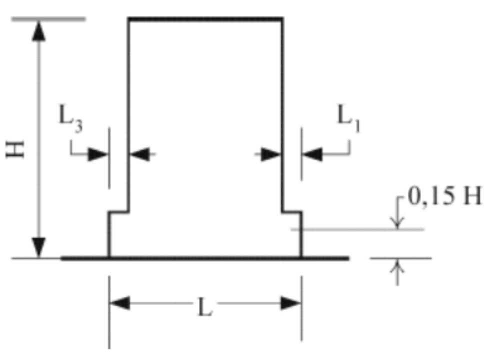
Case I

$$\boldsymbol{MTp-3} \quad \frac{L_3+L_1}{L} > 0.2 \; take \; 0.4 \; or \; 40\%$$

$and \; L_1 = L_3$

$and \; the \; top \; 3 \; bays \; @ \; 5m \;\; so \; L = 25m \; and$

(b) (setback occurs above 0.15 H)



$$\frac{L_3+L_1}{L} \leq 0.20$$

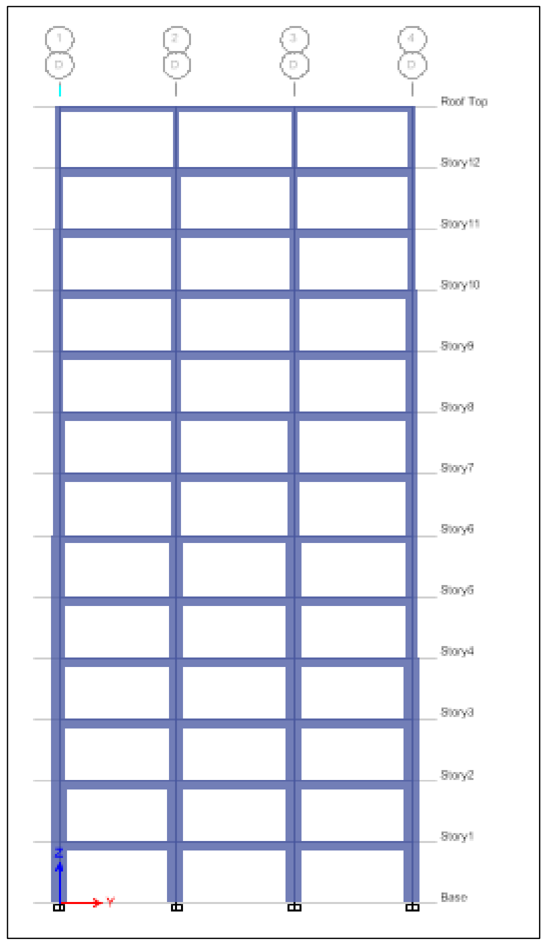
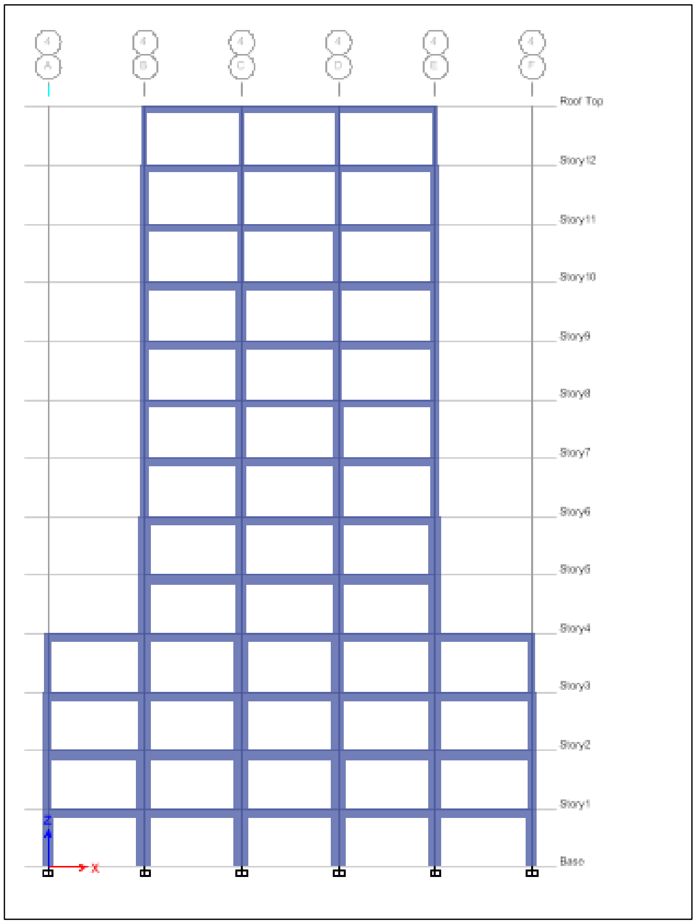


Figure A1 4Elevation for M-2 Y-axis



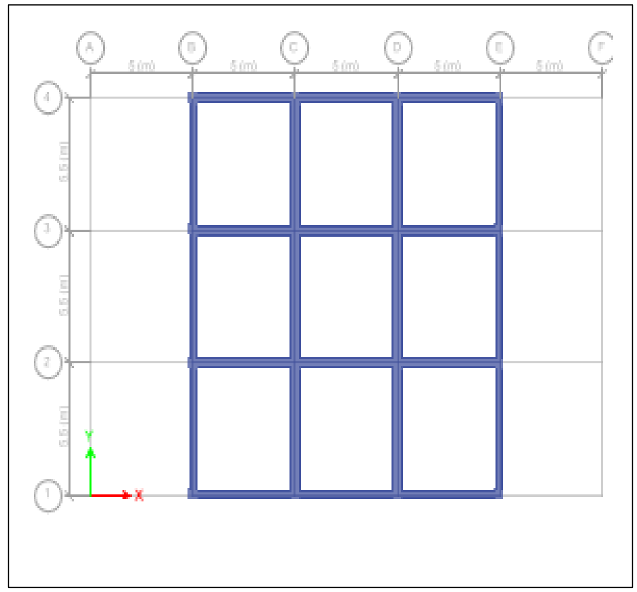


Figure A1 5 Elevation for M-3 X-axis

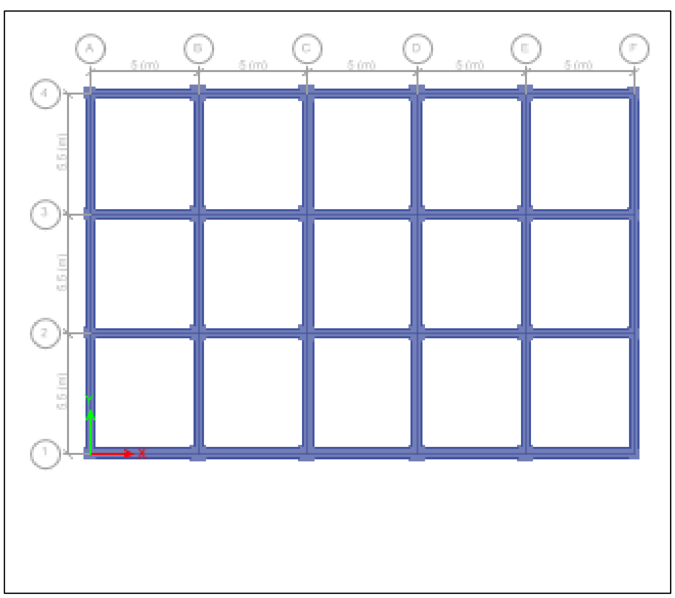
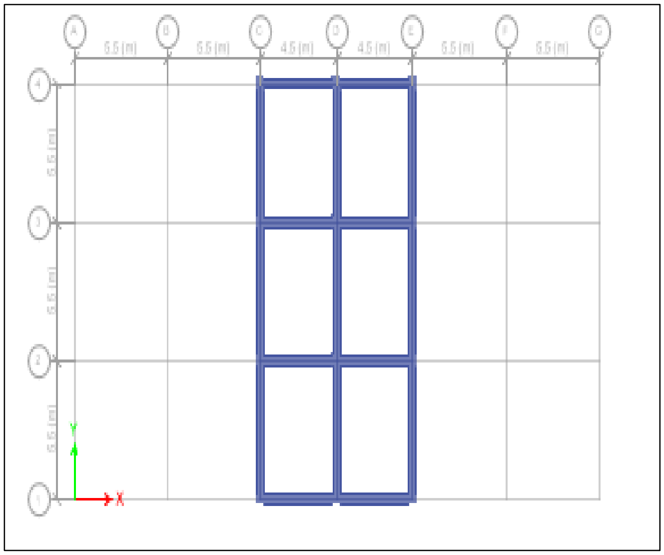


Figure A1 6Sample floor for M-3

$\underline{\boldsymbol{Case\ II}}\ \boldsymbol{MTp-4}\quad \frac{L_3+L_1}{L} > 0.2\ take\ 0.7\ or\ 70\%\ and\ L_1 = L_3\ and\ the\ top\ 2\ bays\ @\ 4.5m\ \ so\ L = 31m\ and$



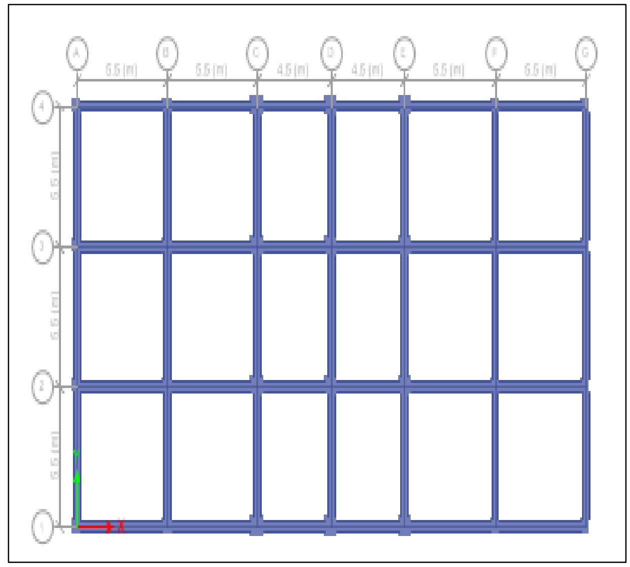


Figure A1 7 Sample floor plane for M-4

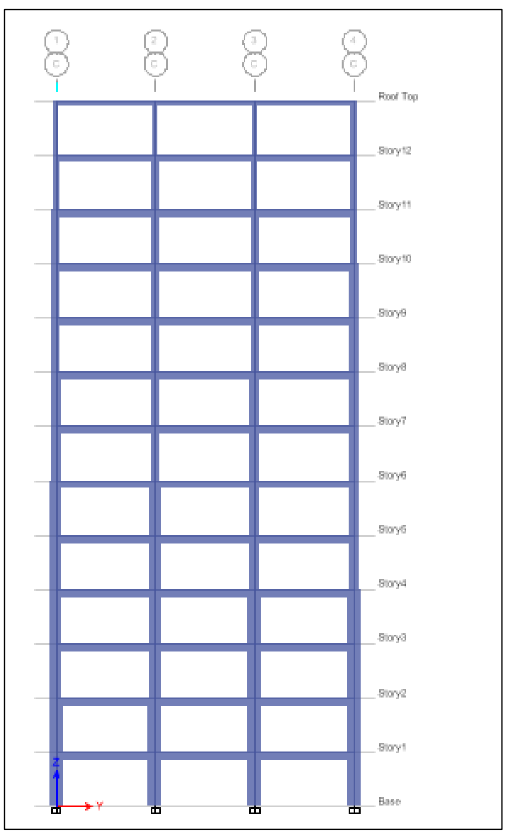


Figure A1 9 Elevation for M-4 for axis Y

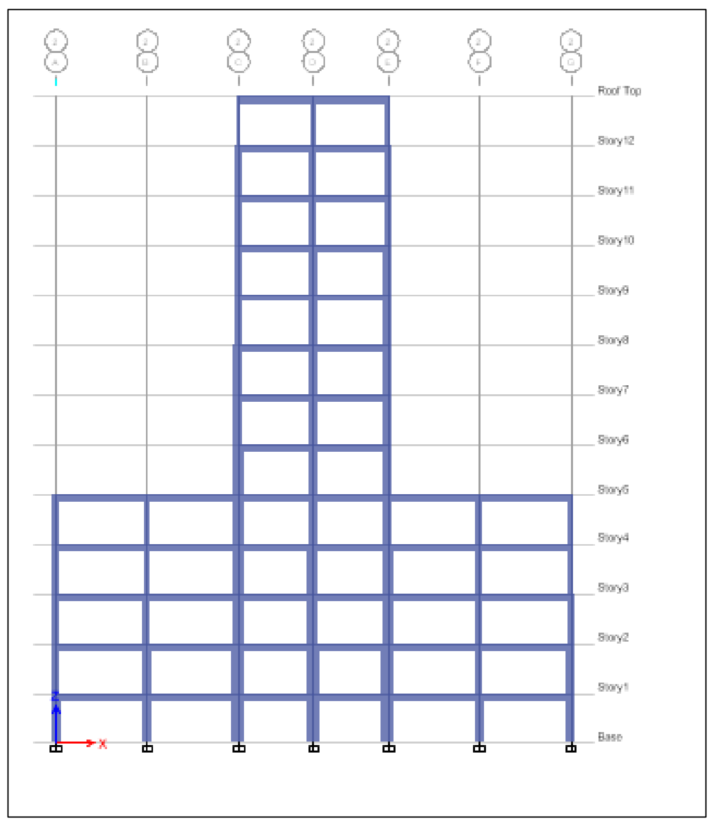
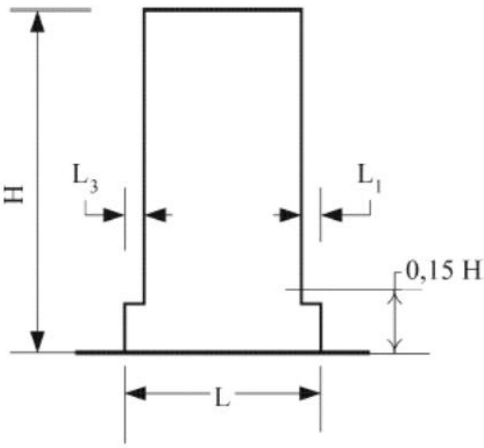


Figure A1 8 Elevation for M-4 axis X

3. Condition three

<u>***Case I***</u>__________________________ $\boldsymbol{MTp-5}$ $\frac{L_3+L_1}{L} > 0.2$ $take\ 0.6\ or\ 60\%\ and\ L_1 = L_3\ \ and\ the\ top\ 2\ bays\ @\ 4.5m\ \ so\ L = 31m\ and\ L_1 = 11m\ and\ L_3 = 11m$



Criterion for (c) $\frac{L_3+L_1}{L} \leq 0.50$

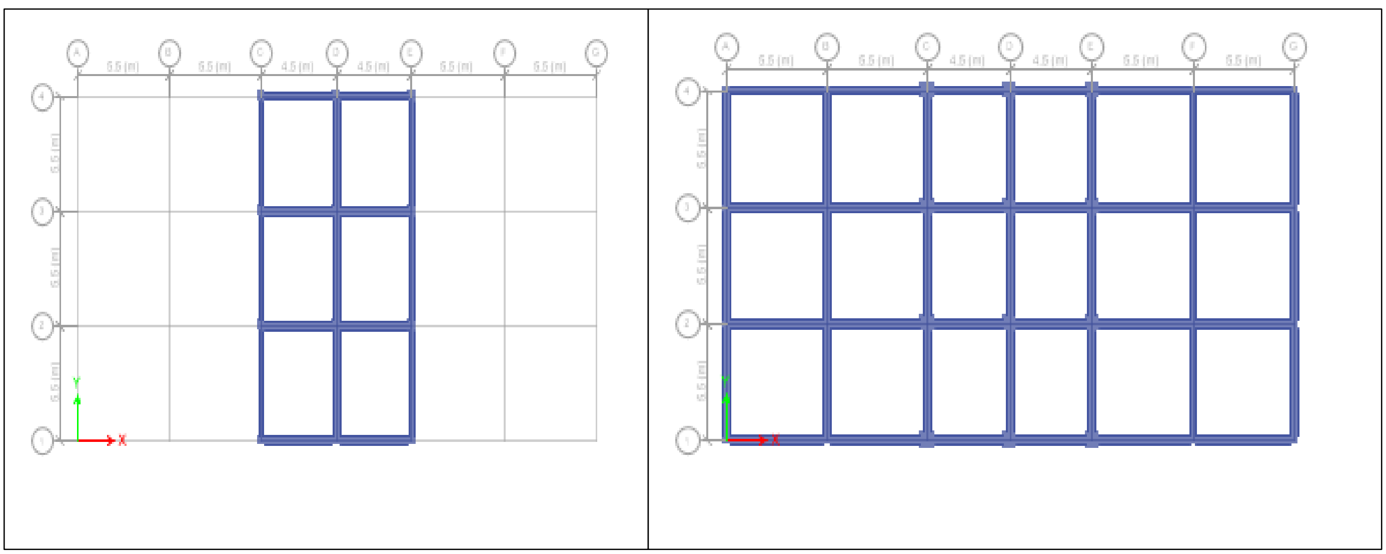


Figure A1 10Sample floor plan for M-5

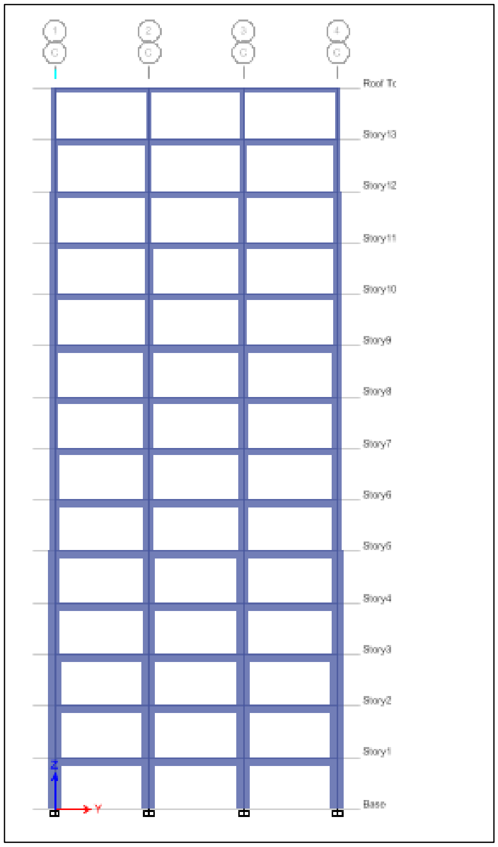


Figure A1 11 Elevation for M-5 axis Y

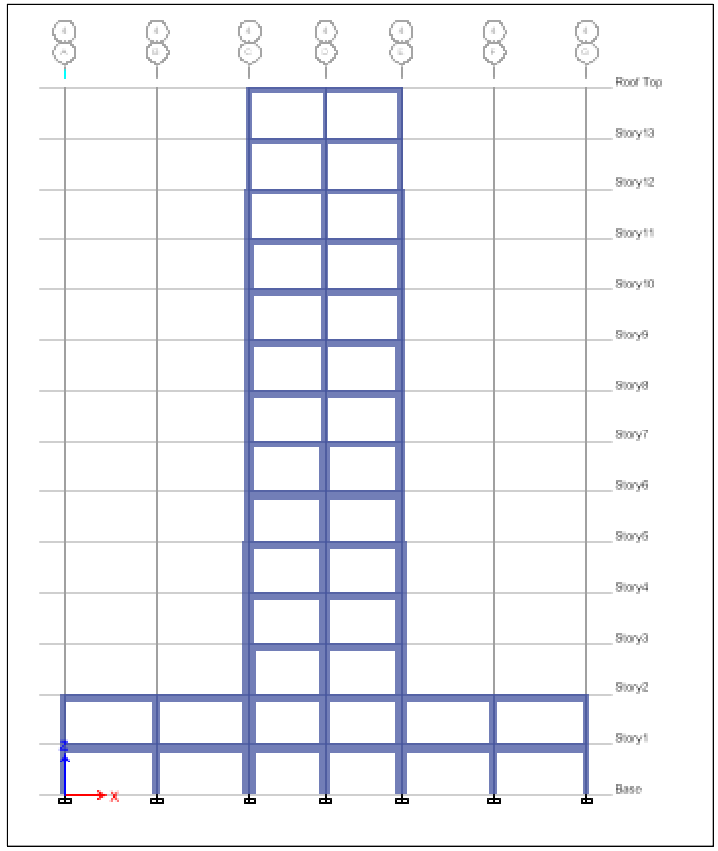
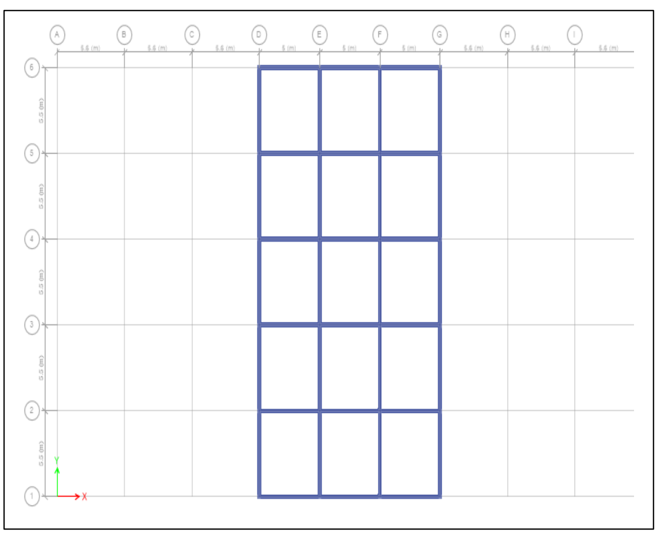


Figure A1 12 Elevation for M-5 axis X

Case II $\boldsymbol{MTp-6}$ $\frac{L_3+L_1}{L} > 0.2\ take\ 0.7\ or\ 70\%\ and\ L_1 = L_3\ and\ the\ top\ 3\ bays\ @\ 5m\ \ so\ L = 48m\ and\ \ L_1 = 16.5m\ and\ L_3 = 16.5m$



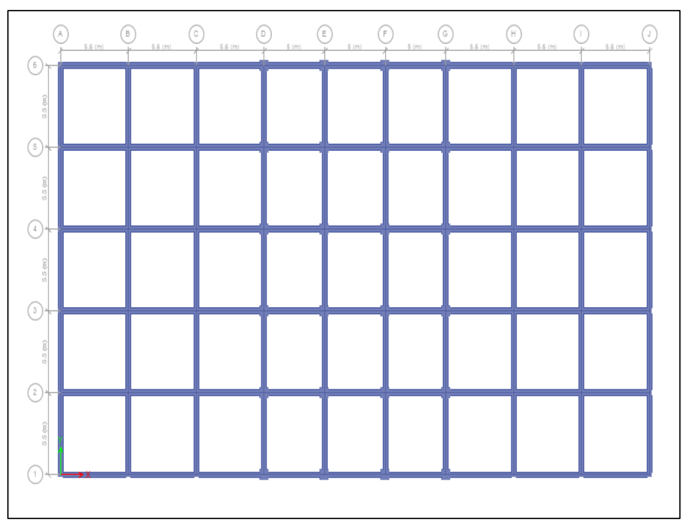
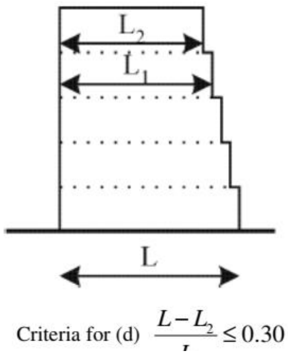


Figure A1 13Sample floor plane

4. Condition four

Case I

$\boldsymbol{MTp-7}$ $\frac{L-L_2}{L} > 0.3$ *take* 0.5 *or* 50% *and* $L_1 = 15, L_2 = 25$ *and the top* 3 *bays* @ $5m$ *so* $L = 30m$



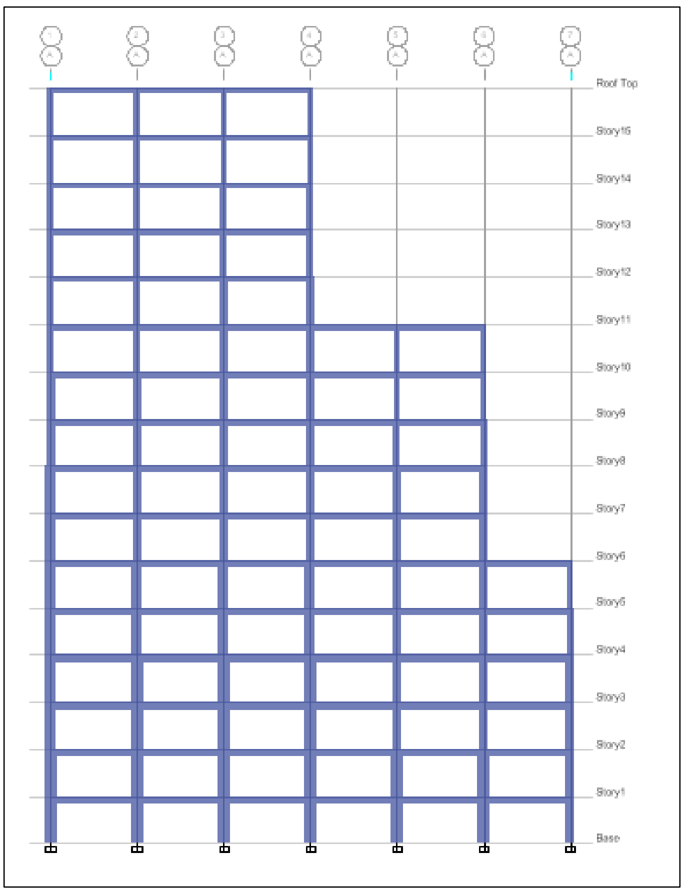


Figure A1 14Elevation for M-7 axis X

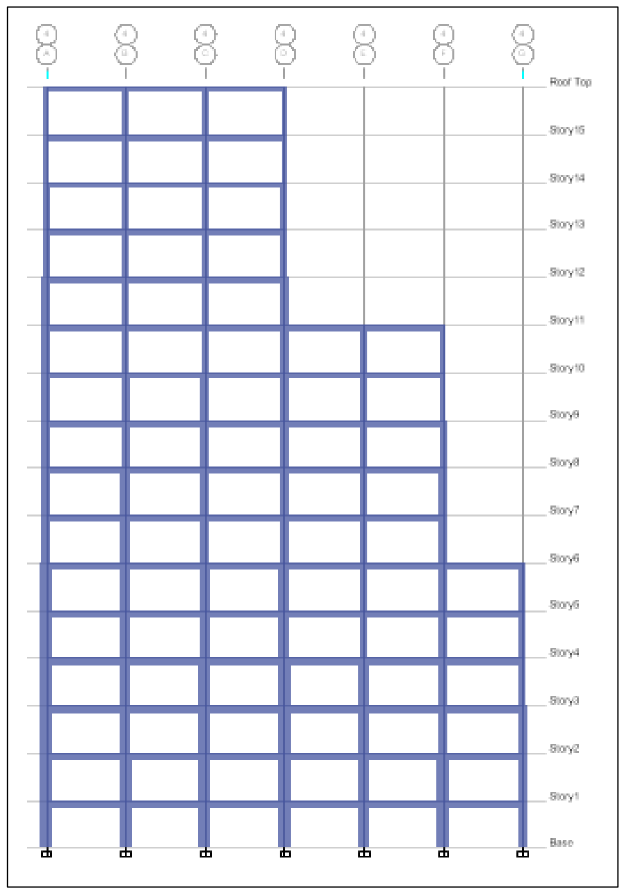


Figure A1 15Elevatioon for M-7 axis Y

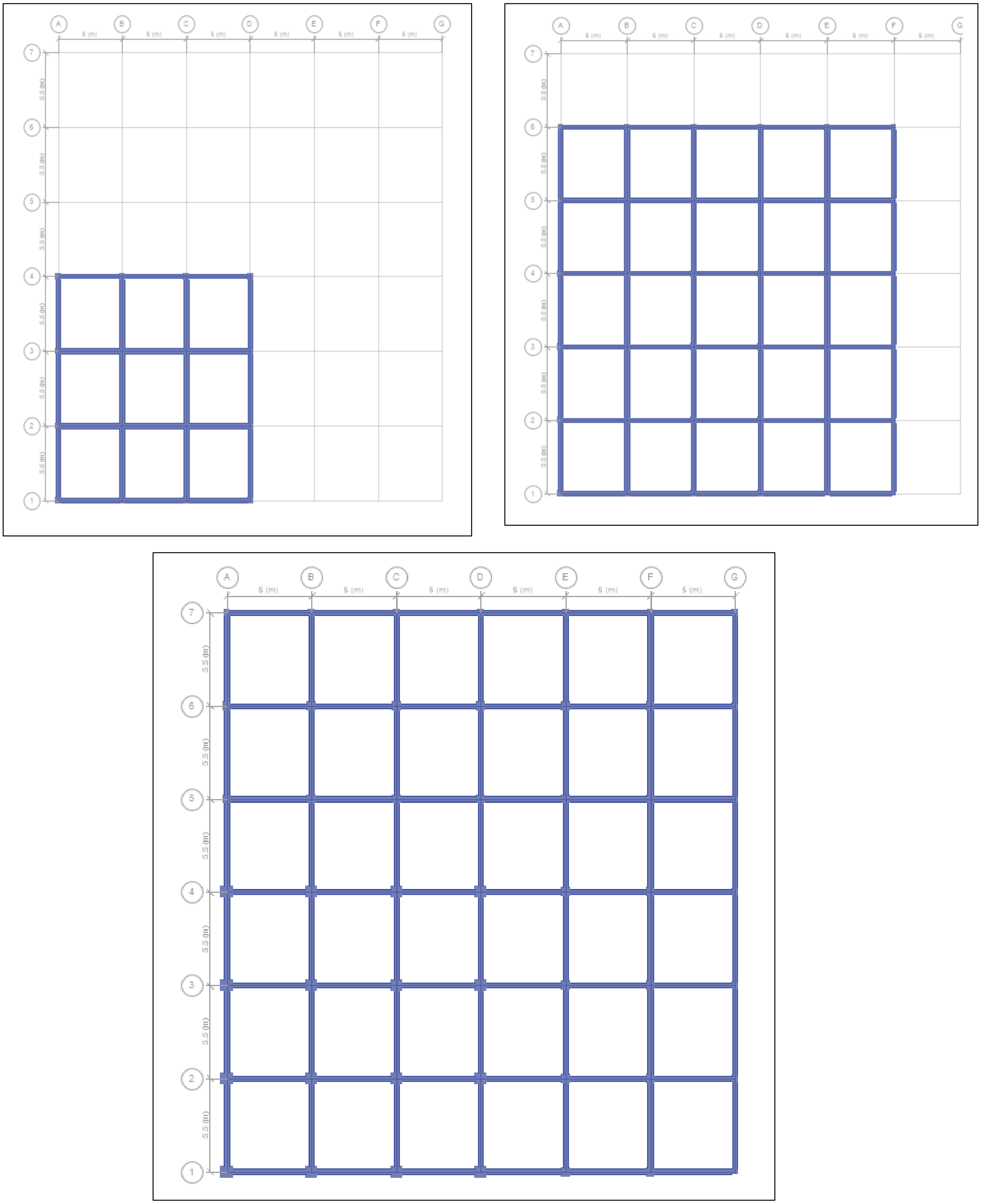


Figure A1 16 Sample floor plane for M-7

Case II

$\boldsymbol{MTp-8}$ $\frac{L-L_2}{L} > 0.3\ take\ 0.5\ or\ 50\%\ and\ L_1 = 15, L_2 = 25$

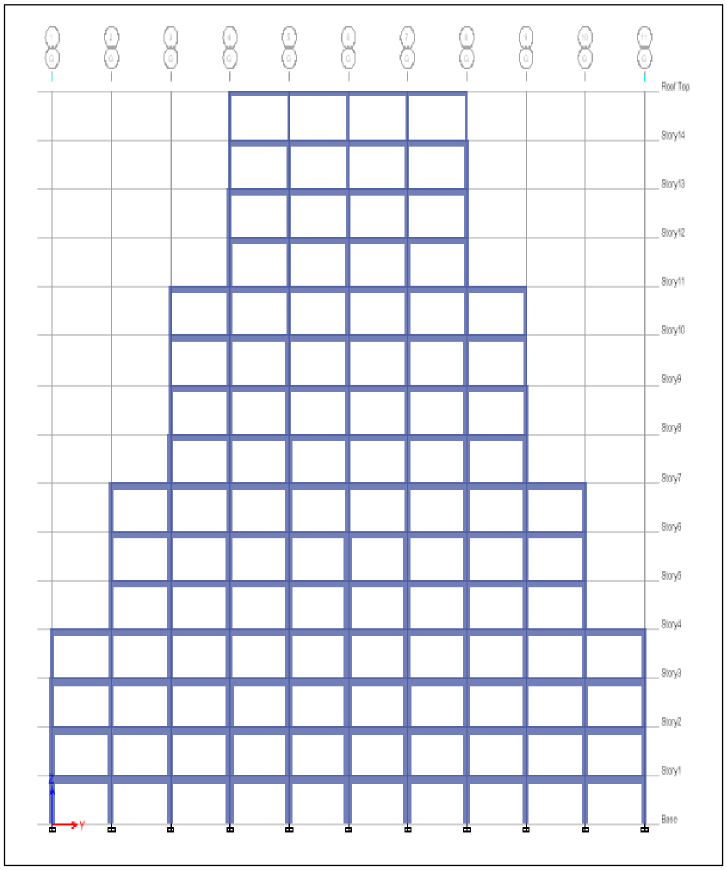


Figure A1 17Elevation for M-8 axis Y

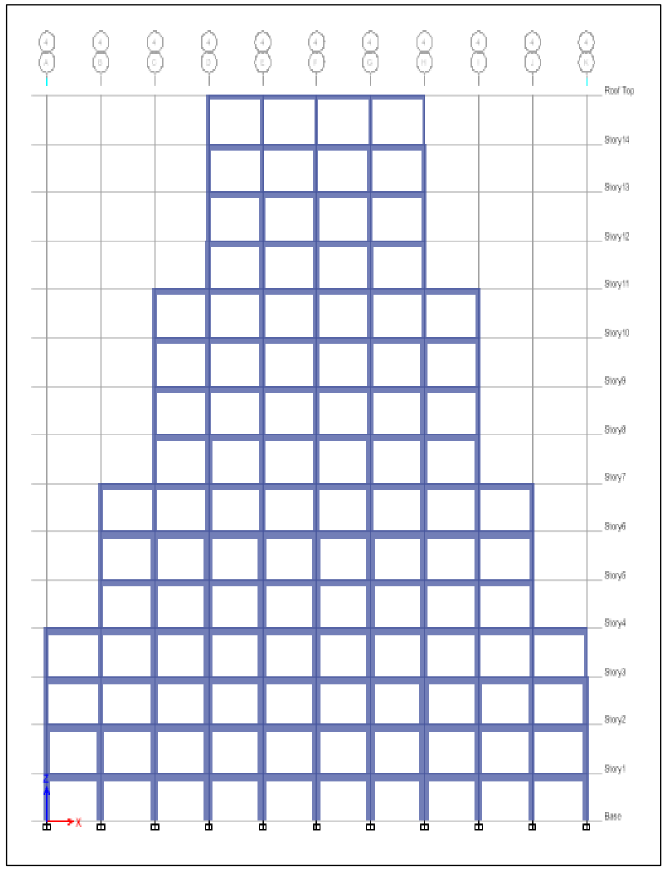


Figure A1 18Elevation for M-8 axis X

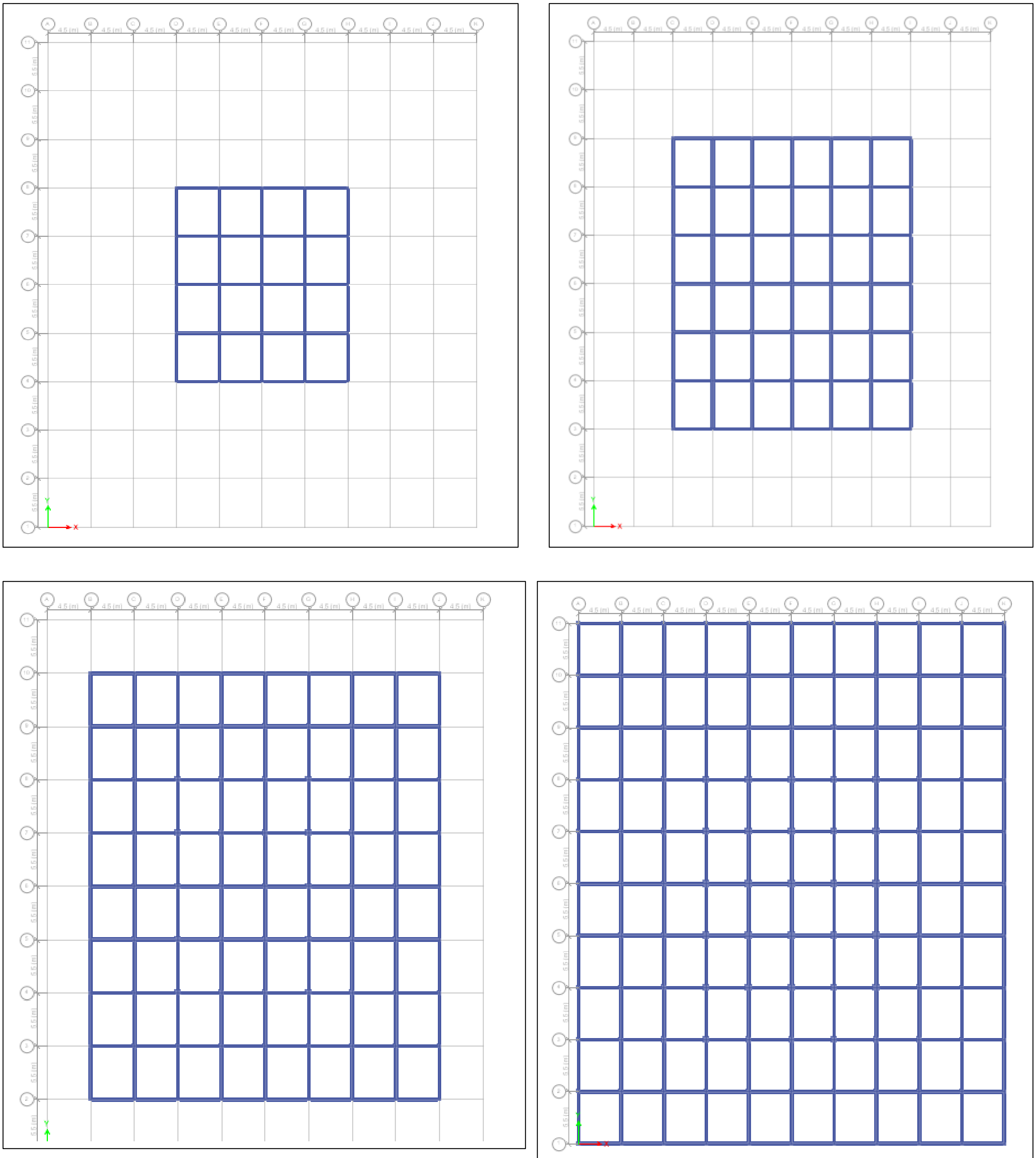


Figure A1 19Sample floor plan for M-8

APPENDIX-B

B.1 Analysis and Design of frame element

Design Seismic Load

The method of analysis used for the determination of design seismic load is Lateral force method of analysis. For G+12,G+14 & G+15 vertical geometric irregular framed RC buildings used in this thesis is satisfying the entire requirement of the code([EBCS EN 1998-1], 2014).

Design of Beam

Geometrical Constraints

In the design of ductile frames, the eccentricity of the beam axis shall be limited relative to that of the column in to which the frame enables efficient transfer of cyclic moments from a primary seismic beam to a column to be achieved.

The following expression is satisfied.

$$b_w \leq \min \{b_c + h_w; 2b_c\}$$

Where

h_w is the depth of the beam

b_c is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam.

Design Action Effects

I) The design value of bending moment and axial forces was obtained from the structural analysis of structural model. The design values from the structural analysis were obtained by considering second order effect by using iterative P-Δ option.

II) The design values of shear forces of primary seismic beams can be calculated as follows

The calculation of shear force was done in accordance with the requirement of capacity design method based on ([EBCS EN 1998-1], 2014). To avoid brittle failure mode the design shear force was calculated from the over strength moment corresponding to plastic hinge formation. The plastic hinge was assign to occur at the end of the beam section.

Beam end moment, $M_{i,d}$

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min \left[1, \frac{\sum M_{RC}}{\sum M_{RB}}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9.0)$$

$\boldsymbol{\gamma_{Rd}}$ The factor accounting for possible over-strength due to steel strain hardening.

$M_{Rb,i}$ is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;

ΣM_{Rc} and ΣM_{Rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively.

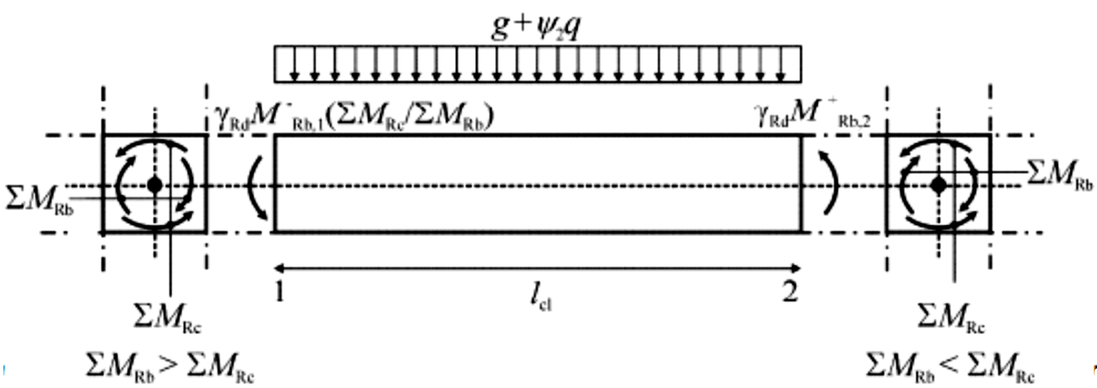


Figure B-1 Capacity Design Values of Shear Forces on Beams ([EBCS EN 1998-1], 2014)

After the end moments obtained from the above expression the design shear force was calculated by using following the equation.

$$V_{Ed} = V_{o,g+\psi_2 q} \pm \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10.0)$$

Flexural Design of Beam

The design of the beam element was designed using bending moment obtained from the structural analysis. The design moment ratio greater than the required value, the section was designed as double reinforced section. The two ends of the beam were designed as critical section or plastic hinge regions the remaining section of the beam was designed as elastic region.

The required reinforcement for the section was calculated by using the moment obtained after redistribution. The reinforcement calculation was done using general design table for C12/15-C50/60(Ethiopian Building Code Standard-2 [EBCS EN 1992], 2014). The procedure used in this research was described below.

I. Design Ultimate Moment (M_{ED})

This Value is obtained from analysis result

II. Ultimate Moment of resistance (M_{RD})

$$M_{RD} = 0.167 f_{ck} b d^2 \ldots \ldots \ldots (11.0)$$

I. Compare M_{ED} and M_{RD}

If the design ultimate moment is greater than the ultimate moment of resistance i.e. $M_{ED} < M_{RD}$, design the member as a single reinforced and if $M_{ED} \geq M_{RD}$ design the member as a double reinforced beam.

II. Design a single Reinforced beam

$$A_{s1} = \frac{M_{ED}}{0.87 f_{ck} z} \ldots \ldots \ldots (12.0)$$

Where

$$z = d[0.5 + \sqrt{(0.25 - \frac{3k_0}{3.4})}$$

$$K_O = \frac{M_{ED}}{f_{ck} b d^2}$$

III. Design a double Reinforced beam

$$A_{s2} = \frac{M_{ED} - M_{RD}}{0.87 f_{yk}(d - d_2)} \ldots \ldots \ldots (13.0)$$

Where

d_2 = is depth of compression steel from the compression face

d = is depth of Tensile steel from the tension face

$$A_{s1} = \frac{M_{RD}}{0.87 f_{yk} Z} + A_{s2}$$

Where

$$z = d[0.5 + \sqrt{(0.25 - \frac{3k'_0}{3.4})}$$

$$K'_O = 0.167$$

The required dimension and detailing of the beam section in this research for the two ductility class was satisfying all the requirements written in the Table B-1below.

Table B-1 Detailing/Dimensioning of Primary Seismic Beams

	DCM	DCH
Length of critical region	h_w	$1.5h_w$
Longitudinal bars (L)		
ρ_{min}, tension side	$0.5f_{ctm}/f_{yk}$	
ρ_{max}, critical regions(1)	$\rho'+0.0018f_{cd}/(\mu_\phi \varepsilon_{sy,d} f_{yd})$(3)	
$A_{s,min}$, top & bottom	-	2φ14
$A_{s,min}$, top-span	-	$A_{s,top\text{-}supports}/4$
$A_{s,min}$, critical regions bottom	$0.5A_{s,top}$(2)	
Hoops or Transverse bars		
(a) outside critical region		
spacing $s_w \leq$	$0.75d\,(1 + \cot\alpha)$(4)	
$\rho_{w,min}$	$0.08\sqrt{f_{ck}}/f_{yk}$, f_{ck} and f_{yk} in MPa	
(b) In critical regions		
$d_{bw} \geq$	6mm	
Spacing $s_w \leq$	$h_w/4$; $24d_{bw}$; 225; $8d_{bL}$(4)	$h_w/4$; $24d_{bw}$; 225; $8d_{bL}$(4)

(1) μ_ϕ: value of the curvature ductility factor corresponding to the basic value, q_o, of the behavior factor used in the design. The local ductility of the section was satisfied by using the following expression related with basic behavior factor.

$$\mu_\phi = 2q_o - 1 \quad \text{if } T1 \geq TC \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(14.0)$$

$$\mu_\phi = 1 + 2(q_o - 1)\frac{T_c}{T_1} \quad \text{if } T_l < T_C \ldots\ldots\ldots\ldots..\ldots\ldots\ldots\ldots\ldots (15.0)$$

(2) The minimum area of bottom steel, $A_{s,min}$, is in addition to any compression steel that may be needed for the ULS in bending moment from the analysis for the seismic design situation, M_{Ed}.

(3) ρ and ρ' are tension zone and compression zone reinforcement ratio and both normalised to bd, where d is the effective depth of the section, and b is the width of the compression flange of the beam.

(4) d_{bw} is the diameter of the hoops; d_{bL} is the minimum longitudinal bar diameter (in millimetres); α is the inclination of the shear reinforcement to the longitudinal axis of the beam and h_w the beam depth (in millimetres).

Shear Design of Beam

The design of the beam was designed for the required resistance of the section against shear failure due to the incoming shear action effect obtained from the above. The shear failure was brittle failure so the design of the section due to shear failure was design using the over strength moment to precede the ductile failure of the beam ends. The resistance of the section was obtained from the following expression. The shear resistance is also affected by the angle between the concrete compression strut and the beam axis perpendicular to the shear force(Wight and Macgregor, 2012).

EBCS EN 1992-1-1:2014 identifies four basic shear forces for design purpose, namely Design shear force (V_{ED}), shear resistance of the member without reinforcement(
($V_{Rd,c}$), compression capacity of compression strut($V_{Rd,max}$)and Design shear resistance of the member without shear reinforcement ($V_{Rd,s}$)

I. Design Shear Force (V_{ED})

The capacity design shear force in a beam weaker than the column is calculated as follows:

$$V_{Ed} = V_{o,g+\psi_2 q} \pm \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} \qquad (16.0)$$

II. Design shear resistance of the member without shear reinforcement ($V_{Rd,c}$)

Design shear resistance of the member without shear reinforcement ($V_{Rd,c}$) and given by:

$$V_{Rd,c} = [c_{Rd,c} k (100 p_1 f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp}] b_w d \geq (V_{min} + k_1 \sigma_{cp}) b_w d \qquad (17.0)$$

Where

$c_{Rd,c} = 0.18/\gamma$

$k = 1+\sqrt{\frac{200}{d}} < 2.0$ with the effective depth, in mm

$$\rho_1 = \frac{A_{S1}}{b_w d} < 0.02$$

In which

A_{S1} is the area of tensile reinforcement which extends $\geq (l_{bd} + d)$ beyond the section Considered

b_w The smallest width of the cross section in the tensile area

$$V_{min} = 0.035k^{3/2}{f_{ck}}^{1/2}$$

$k_1 = 0.15$

$\sigma_{cp} = N_{ED/A_C} < 0.2f_{cd}$

In which

N_{ED} is the axial force in the cross section

A_C is the cross section of the concrete

III. concrete strut capacity($V_{Rd,max}$)

The concrete compression capacity of compression strut is given by:

$$V_{Rd,max} = \frac{b_w z v_1 f_{cd}}{\cot\theta + \tan\theta} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots . (18.0)$$

Where

$z \approx 0.9d$

$f_{cd} = \alpha_{cc} f_{ck}/\gamma_m$ (For $f_{ck} \leq 50N/mm^2$)(Note: $\alpha_{cc} = 1$ may be used)

$v_1 = 0.6(1 - \frac{f_{ck}}{250})$For $f_{ck} \leq 50N/mm^2$)

θ Is the angle between the concrete compression strut and the beam axis perpendicular to the shear force; for outside the critical region and DCM $1 \leq \cot\theta \leq 2.5$(or 21.8°-45°) used and in this research θ_{ave}=34° used for shear strength calculation. θ for DCH 45° were used in the critical region.

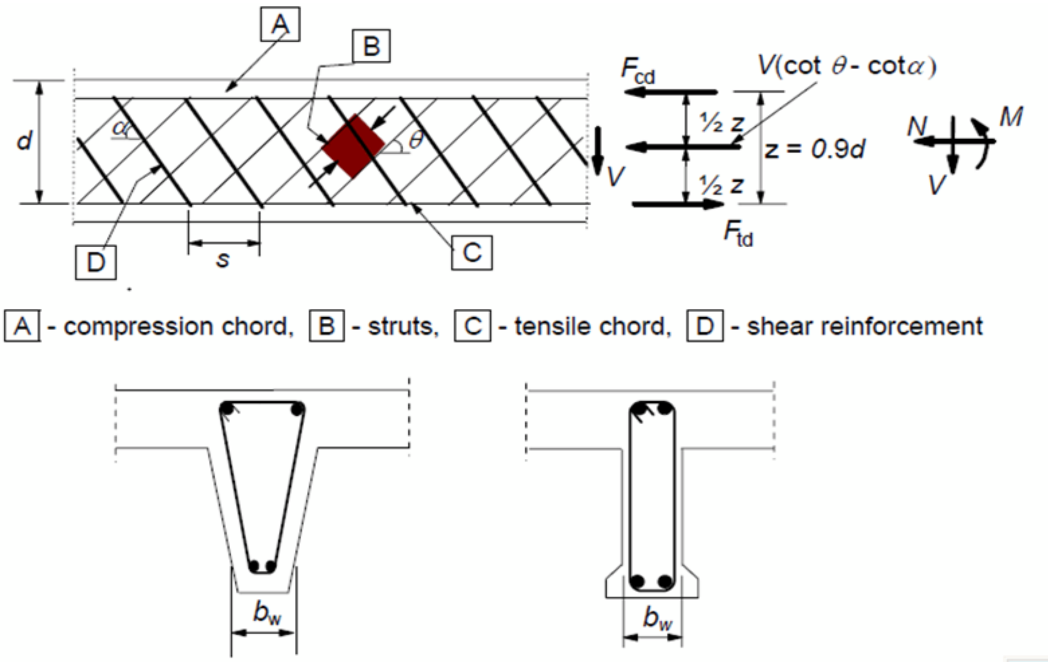


Figure B-2 Truss Model and Notation for Shear Reinforced([ES EN 1992], 2015)

Note: if $V_{ED} < V_{Rd,c}$ no shear reinforcement is required

IV. Design shear resistance of the member without shear reinforcement($V_{Rd,s}$)

If $V_{ED} > V_{Rd,c}$, shear reinforcement must be provided. Provided$V_{ED} > V_{Rd,max}$, the area of shear reinforcement can be estimated from the following expression by equating $V_{ED} = V_{Rd,s}$

$$V_{Rd,s} = \frac{A_{SW}}{S} z f_{ywd} \cot\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19.0)$$

Where

$V_{Rd,s}$is the shear resistance of the member governed by ‘failure ‘of stirrup

$A_{SW}$is the cross sectional area of shear reinforcements

S is the spacing of shear reinforcement

f_{ywd} is the design yield strength of the shear reinforcement

Design of Column

Geometrical Constraints

The geometric constraint for column element was different in the two ductility class. For ductility class medium structures the size of the column depending on the incoming action effect and axial ratio. In this research all the columns inter-story drift sensitivity coefficient was less than unity. The cross sectional dimension of the column was greater than 250mm in ductility class high column designs because the code limits the cross section of the column in ductile sections.

Design Action Effects

I) The design action effect for columns was obtained from capacity design philosophy by satisfying strong column and weak beam design rule. In this research the flexural design moment of the column determined first from the designed beam resistance of the section.

The flexural moment was obtained from the following expression:

$$\sum \mathbf{M_{Rc}} \geq \boldsymbol{\gamma_{Rd}} \sum \mathbf{M_{Rb}} \quad \text{(20.0)}$$

Where

$\sum M_{Rc}$ is the sum of the design values of the moments of resistance of the columns framing the joint.

$\sum M_{Rb}$ is the sum of the design values of the moments of resistance of the beams framing the joint.

γ_{Rd} over strength factor on beam strengths

In this research over strength factor 1.3 was used for the two ductility classes to obtain design action effects for columns.

II) Design shear force for the column was obtained from the over strength end moment of the columns similar to the beam shear force determination.

Column end moment $M_{i,d}$

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} \min\left[1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right] \quad \text{(21.0)}$$

Where

$\boldsymbol{\gamma_{Rd}}$ is the factor accounting for possible over-strength due to steel strain hardening.

$M_{Rc,\,I}$ is the design value of the column moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;

After the end moments obtained from the above expression the design shear force was calculated the equation below.

$$V_{Ed} = \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (22.0)$$

$\boldsymbol{\gamma_{Rd}}$ is the factor accounting for possible over strengths

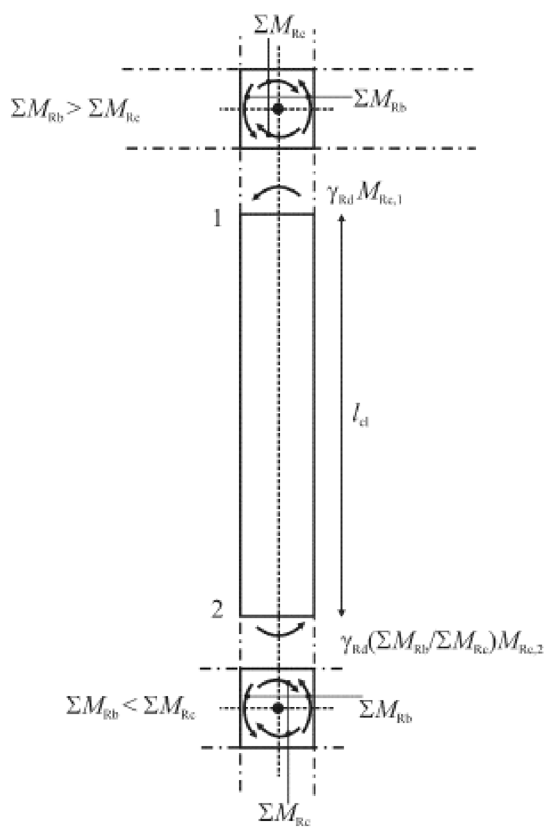


Figure B-3 Capacity Design Shear Force in Columns ([ES-8], 2015).

Flexural

Design of Column

The design of the column was designed using uniaxial interaction charts based on ([EBCS EN 1998-1], 2014) and design again using biaxial design chart if the required quantity of reinforcement was greater than the design reinforcement using uniaxial chart.

The quantity of reinforcement required for the incoming action effect was calculated based on the following expressions:

(a) Calculate axial load ratio, υ_{sd}

$$\boldsymbol{\upsilon_{sd} = \frac{N_{sd}}{A_c f_{cd}}} \quad \ldots(23.0)$$

Where

N_{sd} is design axial load obtained from the over-strength beam shear force and its own gravity loads.

A_c is cross-sectional area

f_{cd} design strength of the concrete

(b) Calculate moment ratio, μ_{sd}

$$\boldsymbol{\mu_{sdx} = \frac{M_{sd}x}{A_c h f_{cd}} \text{ and } \mu_{sdy} = \frac{M_{sd}y}{A_c h f_{cd}}} \quad \ldots(24.0)$$

Where

μ_{sdx} is moment ratio in the x direction.

μ_{sdy} is moment ratio in the y direction and not needed in the case of uniaxial design.

M_{sd} is design moment

h depth of the column in the considered sense of the design moment

f_{cd} and A_c, defined in (a).

(c) Read ω from the chart and calculate the total reinforcement

$$\boldsymbol{A_{s,tot} = \frac{\omega A_c f_{cd}}{f_{yd}}} \quad \ldots(25.0)$$

Divide $A_{s,tot}$ in each side of the cross-section uniformly depending on used chart.

In this research all the requirements below in the Table 3-6 detailing/dimensioning of primary seismic column was satisfied in the design of column above.

Table B-2 Detailing/Dimensioning of Primary Seismic Columns

	DCM	DCH
Length of critical region ≥	h_c;l_{cl}/6;0.45m	1.5h_c; l_{cl}/6;0.6
Axial load ratio, $\upsilon_{sd} = \frac{N_{sd}}{A_c f_{cd}}$	≤0.65	≤0.55
Longitudinal bars (L)		
ρ_{min}, tension side	0.01	
ρ_{max}, critical regions$^{(1)}$	0.04	
bars per side ≥	3	
d_{bl}	8mm	
Spacing between restrained bars	≤200mm	≤150mm
Hoops or Transverse bars		
(a) outside critical region		
d_{bw} ≥	6mm; d_{bl}/4	
spacing s_w(mm) ≤	20d_{bl}; h_c; b_c; 400mm	
(b) In critical regions		
d_{bw}≥	6mm	6mm; 0.4$d_{bl,Max}(f_{ydl}/f_{ydw})^{1/2}$
Spacing s_w≤	b_o/2; 175; 8d_{bl}	b_o/3; 125; 6d_{bl}
ω_{wd} ≥	-	0.08
(c) In column base critical region		
ω_{wd} ≥	0.08	0.12
$\alpha\omega_{wd}$ ≥	$30\mu_\phi \upsilon_d \varepsilon_{sy,d} b_c/b_o - 0.035$	

(1) h_c is the largest cross sectional dimension of the column; l_{cl} is the clear length of the column (meter); d_{bl} is the minimum diameter of longitudinal bars; b_c is the gross cross sectional width; d_{bw} diameter of the hoops; $b_o$is the width of confined core (to the centerline of the hoops), b_i is the distance between consecutive engaged bars; n the total number of longitudinal bars engaged by hoops and s is the spacing of the hoops.

(2) ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical region.

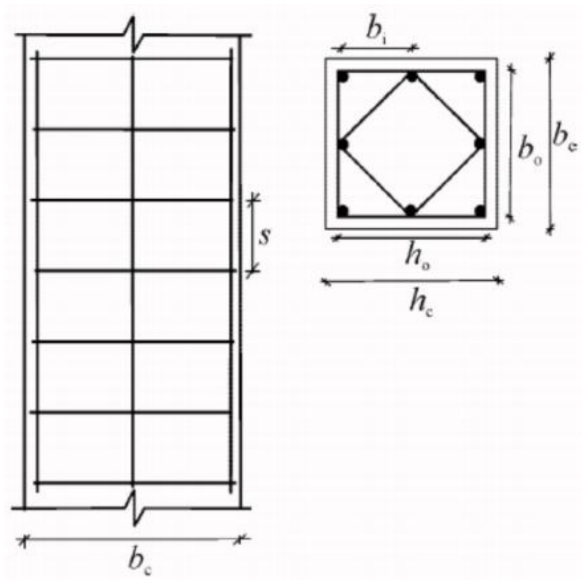
$$\boldsymbol{\omega}_{\mathbf{wd}} = \frac{\textbf{Volume of confining hoops}}{\textbf{Volume of concrete core}} \cdot \frac{\mathbf{f_{yd}}}{\mathbf{f_{cd}}} \dots\dots\dots\dots\dots\dots(26.0)$$

(3) $\alpha = \alpha_n . \alpha_s$, is the confinement effectiveness factor.

Where

$$\alpha_n = 1 - \frac{\sum_n b_i^2}{6 \times b_o h_o} \dots\dots\dots\dots\dots\dots(27.0)$$

$$\alpha_s = \left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right) \dots\dots\dots\dots\dots\dots(28.0)$$



1.1.1.1 Shear Design of Column

The shear design of the column was designed with the maximum action effect obtained from the above design action effect calculation for columns. The resistance determination of shear capacity for column section was similar to the procedure followed in beam shear resistance calculation.

1.1.1.2 Design of Beam-Column Joint

Beam-column joint in ductility class medium structures was designed by checking the confinement requirements of the frame element are satisfied or not. One intermediate column was provided between column corners.

Beam-column joint in ductility class high structures was designed by obtaining the shear force acting on the concrete core using the following expressions.

i) For exterior beam-column joint:

$$\mathbf{V_{jhd} = \gamma_{Rd} A_{s1} f_{yd} - V_c} \text{ ... ...} \quad (29.0)$$

ii) For interior beam-column joints:

$$\mathbf{V_{jhd} = \gamma_{Rd}(A_{s1} + A_{s2}) f_{yd} - V_c} \text{ ... ...} \quad (30.0)$$

Where

As1 is the area of beam top reinforcement

A_{s2} is the area of beam bottom reinforcement

V_c is the column shear force, from the analysis in seismic design

γ_{Rd} is over strength factor

After the determination of shear force acting on the concrete core the diagonal compression force on the joint does not exceed the compressive strength of concrete in the presence of transverse tensile strain. This requirement is satisfied by fulfilling the following expression:

a) For interior beam-column joint the following expression is satisfied.

$$\mathbf{V_{jhd} \le \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_c} \text{ ... ...} \quad (31.0)$$

Where $\eta = 0.6(1 - f_{ck}/250)$; $\quad f_{ck}\ in\ MPa$

V_d the normalized axial force in the column above the joint

b) For interior beam-column joint the following expression is satisfied.

$$\mathbf{V_{jhd} \le 0.8 \times \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_c} \text{ ... ...} \quad (32.0)$$

And the effective joint width b_j is calculated using the following expression:

i) If b_c >bw: $\mathbf{b_j = min\{b_c; (b_w + 0.5.h_c)\}}$ ... ...(33.0)

ii) If b_c >b_w: $\mathbf{b_j = min\{b_w; (b_w + 0.5.h_c)\}}$ ... ...(34.0)

Adequate confinement also check according to the new code ([ES EN 1998-1], 2015).

1.1.2 General Description

G+12, G+14 and G+15 regular frame reinforced structures for mixed use building was design in this thesis. In the following example only ductility class medium structure was considered to show detail procedure of capacity design philosophy according to the new Ethiopian building codes ([ES EN 1998-1], 2015 and ([ES EN 1998-1], 2015). Whenever necessary reference was made from previously discussed equations, tables and figures.

The floor plan and the framing system were shown in the Figure A1 1-18 in the appendix part. The design of one selected beam and column from the selected framing system was described in detail.

Material Properties

Material used in the design of the structural elements

Table B-3 Material Property

Material	Member	γ_c	α_c	f_{ck}(MPa)	$f_{cd}=\alpha_c f_{ck}/\gamma_c$ (MPa)	f_{ctm} (MPa)	E_c (GPa)
Concrete strength	Beam-C20/25	1.5	0.85	20	11.33	2.2	30
	Column-25/30	1.5	0.85	25	14.17	2.6	31
		γ_s	f_{yk}(MPa)	$f_{yd}=f_{yk}/\gamma_s$(MPa)	E_s(GPa)		
Strength of Rebar	Beam and Column	1.15	420	362.217	200		

Table B-4 Properties of Reinforcement

Production form	Bars and de-coiled rods			Wire Fabrics			Requirement or quantile value (%)
Class	A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600						5.0
Minimum value of $K=(f_t/f_y)_k$	≥1.05	≥ 1.08	≥1.15 <1.35	≥1.05	≥ 1.08	≥1.15 < 1.35	10.0
Characteristic strain at maximum force, ε_{uk} (%)	≥ 2.5	≥ 5.0	≥ 7.5	≥ 2.5	≥ 5.0	≥ 7.5	10.0
Bendability	Bend/ Re-bend test			-			
Shear strength	-			0.25 A f_{yk} (A is area of wire)			Minimum
Maximum Nominal deviation from bar size (mm) Nominal mass ≤ 8≥ (individual bar > 8 or wire) (%)	± 6.0 ± 4.5						5.0

Geometry of the Structure

The geometry of the structure used in this sample was four story and four by three bay regular frame structure. The framing system is shown in the Figure A.1; A.2 and A.2.

The cross-sectional property of the frame structure for this particular example was used for medium ductility class and$\gamma_{Rd} = 1.3$ .The size of the building elements was selected based on the geometric constraint and maximum axial load ratio criteria.

APPENDIX-C

C.1 Analysis Parameters and Values

Table C-1 Values of the Recommended Type 2 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

Table C-2 Values of φ for Calculating ψ_{Ei}

Type of variable action	Storey	φ
Categories A-C*	Roof Storeys with correlated occupancies Independently occupied storeys	1.0 0.8 0.5
Categories D-F* and Archives		1.0

Table C-3 Recommended Value of Ψ Factors for Buildings

Action		Ψ_0	Ψ_1	Ψ_2
Imposed loads in buildings				
Category A: Domestic, Residential areas		0.7	0.5	0.3
Category B: Office areas		0.7	0.5	0.3
Category C: Congregation areas		0.7	0.7	0.6
Category D: Shopping areas		0.7	0.7	0.6
Category E: Storage areas		1.0	0.9	0.8
Category F:	Traffic areas; vehicle weight ≤30kN	0.7	0.7	0.6
Category G:	Traffic areas; 30kN<vehicle weight ≤30kN	0.7	0.5	0.3
Category H: Roofs		0	0	0

Table C-4 Basic Value of the Behavior Factor, q_o, for Systems Regular in Elevation

Structural Type	DCH	DCM
Frame system, dual system, coupled wall system	$4.5\alpha_u/\alpha_1$	$3\alpha_u/\alpha_1$
Uncoupled wall system	$4\alpha_u/\alpha_1$	3
Torsional flexible system	3	2
Inverted pendulum system	2	1.5

α_l and α_u are defined as follows:

α_1 is the value by which the horizontal seismic design action is multiplied in order to first reach the flexural resistance in any member in the structure, while all other design actions remain constant;

α_u is the value by which the horizontal seismic design action is multiplied, in order to form plastic hinges in a number of sections sufficient for the development of overall structural instability, while all other design actions remain constant. The factor α_u may be obtained from a nonlinear static (pushover) global analysis.

APPENDIX-D

D.1 Modal Pushover analysis Procedure

i. Mass Source

- Go to Define>Mass source> Modify/Show mass source > load pattern Dead load/live load/ supper dead load>Add for each load pattern with their multiplayer

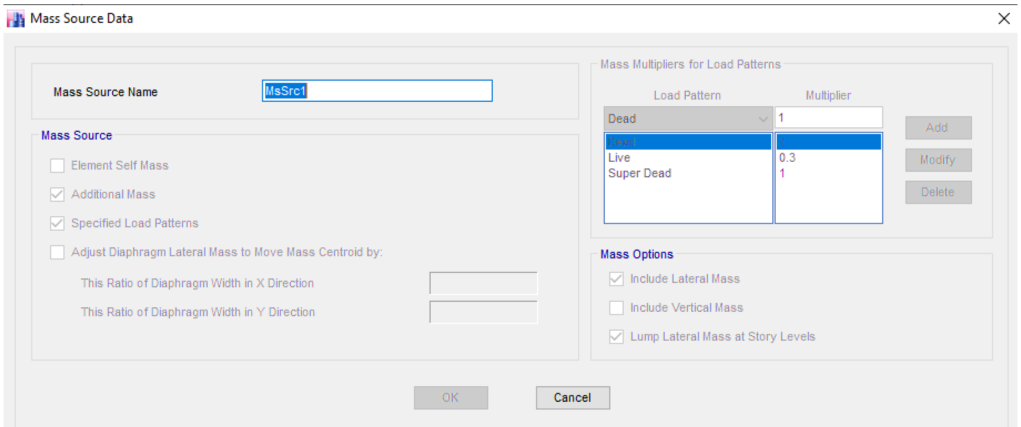


Figure D1 1 Mass source assigning

ii. P-Delta Option

- Go to Define >P-Delta>Iterative Based on load>load pattern Dead load/live load/ supper dead load>Add for each load pattern with their scale factor

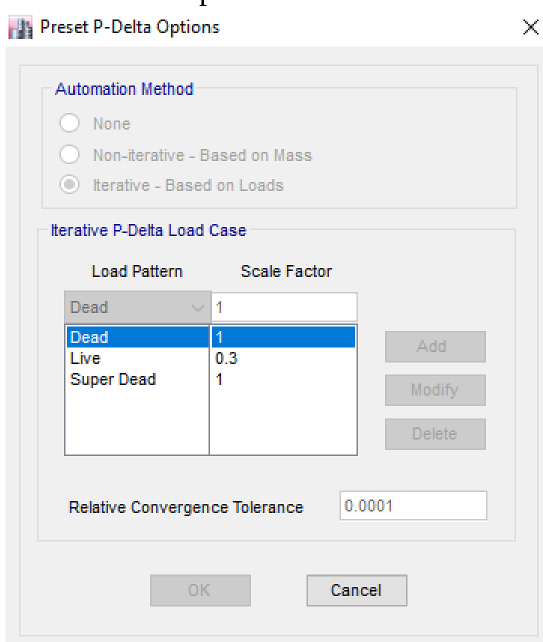


Figure D1 2 P-Delta option

iii. Modal Case

In this research the nonlinear analysis for irregular reinforced concrete structure according to ES-EN-8-1998-2015 should be modal analysis. So

- Go to Define > Modal Case> Modify/Show case>

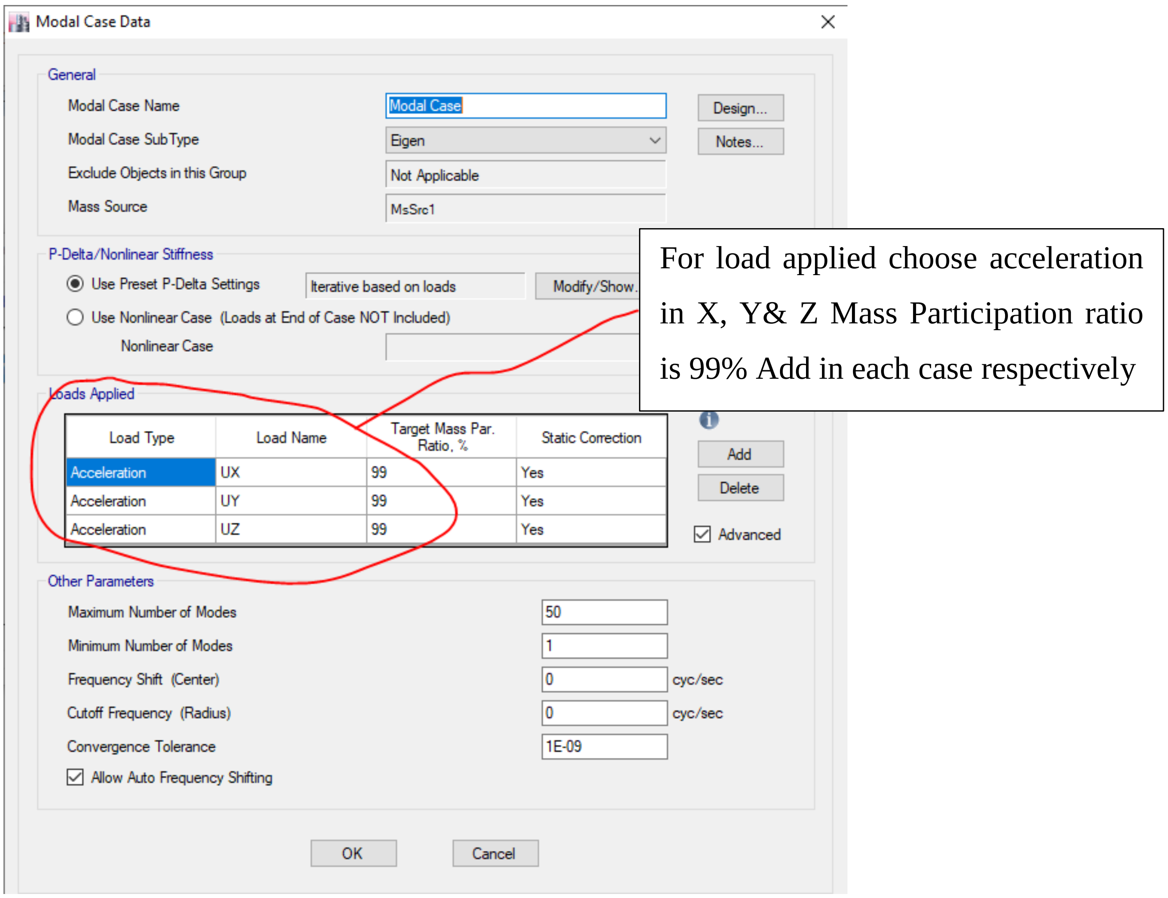


Figure D1 3Modal Case Assigning

And the modal number should be taken the 90% of mass participation in this case take 50 modes.

iv. Define new Load case for non-linear pushover analysis

In this research defined new load patterns as Gravity-x, Gravity-y, Modal push x and Modal push y.

- Go to Define > load Case >Modify/show case >Gravity-x/Gravity-y/Modal-push x/Modal-Push y> Add new load > ok

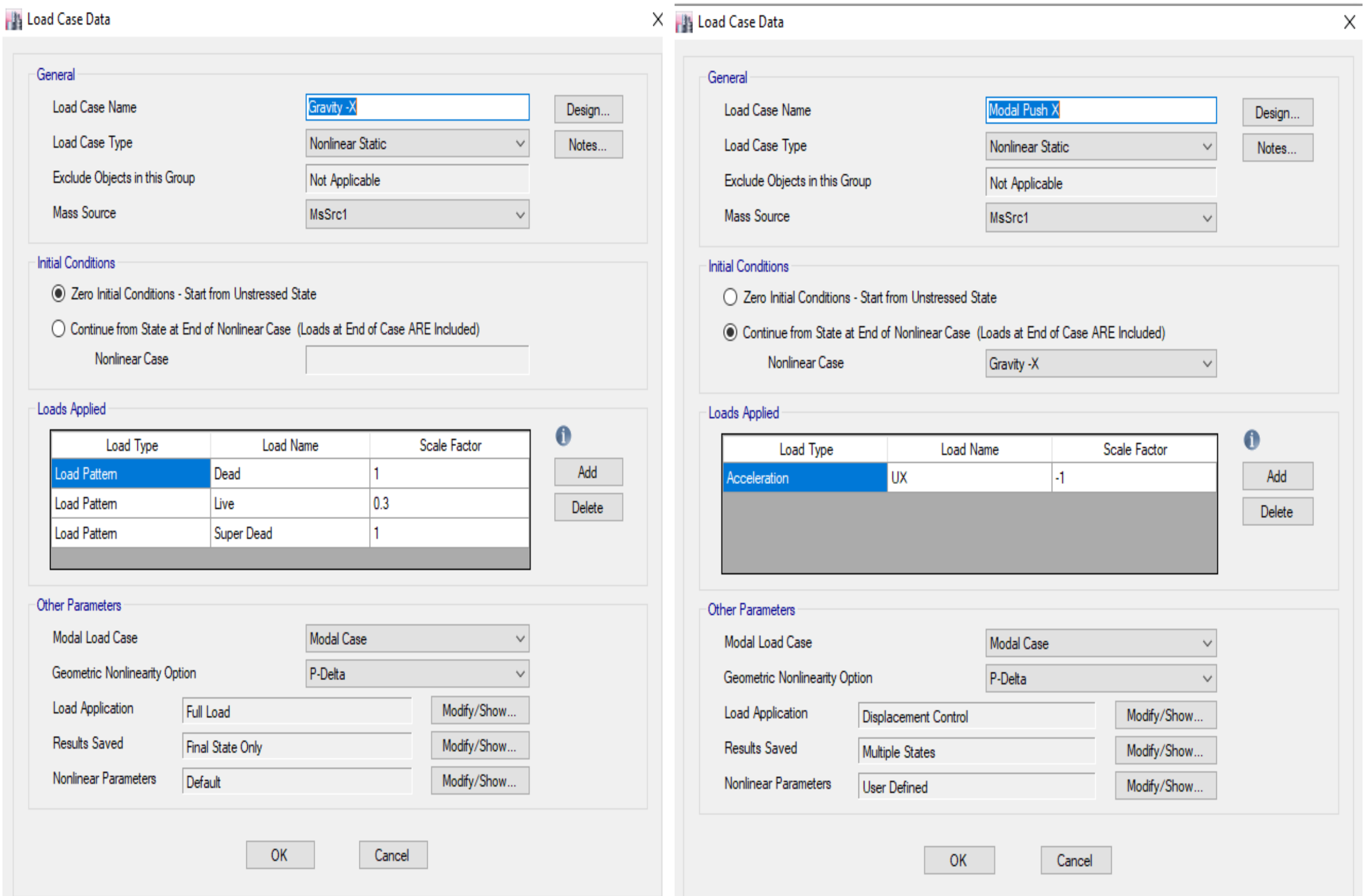


Figure D1 4 Load case for Gravity load and Modal pushover cases

A non-linear static load case was defined in the two principal directions. In this step mass source, Acceleration load type, load application type and how the results are saved parameters were filled in the software.

- ➢ The load applied used to assess the performance of the structure was displacement controlled type.
- ➢ Go to Define > load case>Gravity x/Gravity y>Add new case >load case (nonlinear static) > it started from zero initial condition>load type (acceleration)(load pattern of DL+0.3LL)>>load name Ux / Uy>scale factor 1> Geometric nonlinearity parameters as P-Delta>load application (Full load)>used monitor displacement U1/U2(kept as equal to 4% of the height of the building)>Result saved>Final states only.

- Go to Define > load case> push X/push Y>Add new case >load case (nonlinear static > continuous from state at end of nonlinear case >load type (acceleration) Ux / Uy>scale factor 1> Geometric nonlinearity parameters as P-Delta>load application (displacement control)>used monitor displacement U1/U2 (kept as equal to 4% of the height of the building)>Result saved>Multiple states.

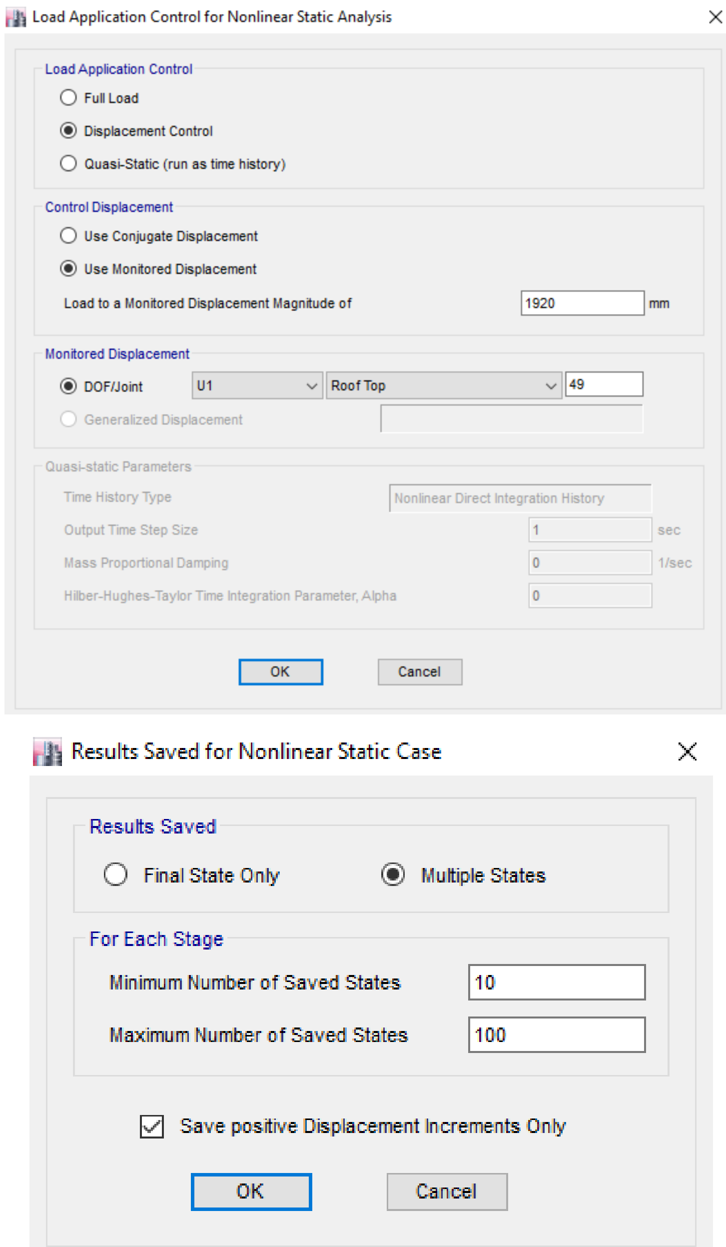


Figure D1 5Result saved for non-linear static load case a

v. Assignment of Hinges to the Frame Elements

The modeling of the plastic hinges was performed at the end of beam element and the bottom end of base column. For further performance investigation plastic hinges also defined at the end of structural column elements. When the plastic hinges were defined deferent assumptions are made and described as follows:

- The plastic hinges formations in the nonlinear deformation of the building was concentrated or lumped in the critical length (single point) of the element.

➢ Select all the beams in the model.

➢ Go to Assign>Frame>Hinges (the hinges assigned at both ends of the beam which means at the relative distance of 0.05 and 0.95.

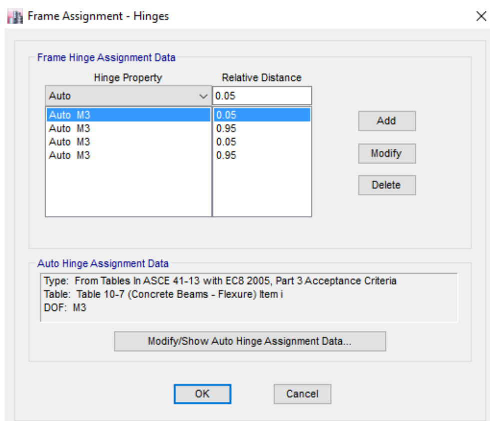


Figure D1 6Hinge at both ends for Beams

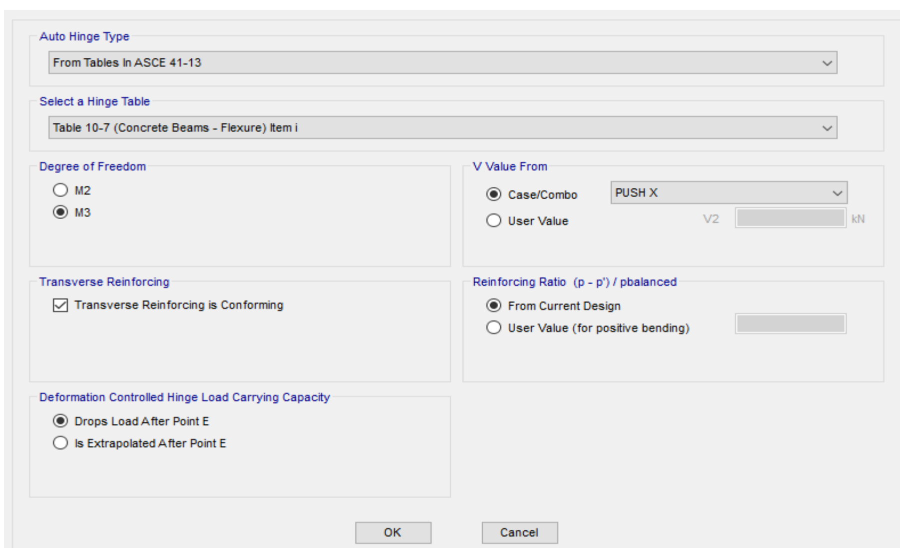


Figure D1 7Hinge Properties for Beams

- In similar manner assigned hinges to all columns by repeating steps as previously carried out for beams ,the only difference is that column assigned P-M2-M3 hinges instead of M3 hinges for beams.

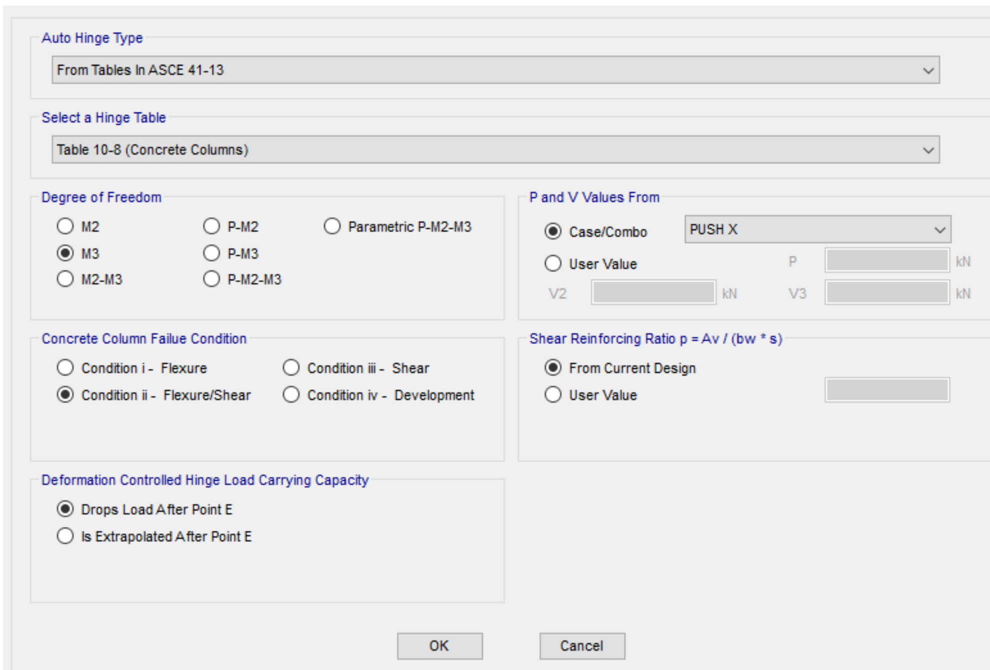


Figure D1 8Hinge Properties for Columns

vi. Analysis of the Structure

In this research the analysis of the structures was performed using the above data and procedure. The structures that are designed with different ductility class were analyzed and extract different parameters for the performance comparison of the structure.

- Go to Analyze>Set load case to run>don’t run earth quake, modal load cases> Run now (only push X, push Y, dead and live load cases).

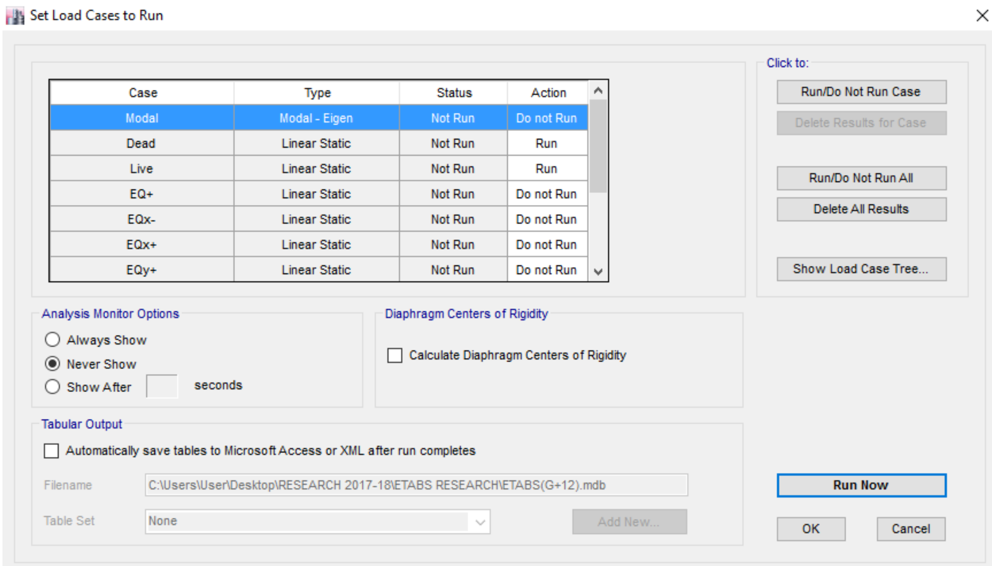


Figure D1 9Set Load Case to Run the Analysis