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INFLUENCE OF OVERSTRENGTH FACTOR ON THE GLOBAL SEISMIC PERFORMANCE OF REINFORCED CONCRETE STRUCTURE

A Thesis in Structural Engineering

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The undersigned have examined the thesis entitled '**INFLUENCE OF OVERSTRENGTH FACTOR ON THE GLOBAL SEISMIC PERFORMANCE OF REINFORCED CONCRETE STRUCTURE**' presented by Henok Abera, a candidate for the degree of Master of Science and hereby certify that it is worthy of acceptance.

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UNDERTAKING

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ABSTRACT

In recent years, designs of earthquake resistance structures using performance based design philosophies are developed. Nowadays, prediction of inelastic seismic responses and the evaluation of seismic performance of a structural building are very important subjects. This study is focused on capacity design philosophy approach. The criteria and requirements used for the design of the building using capacity design philosophies are adopted according to the provision of EBCS EN 1998-1:2014. The comparison of global seismic performance of the structures is assessed in detail using four structural parameters quantitatively.

In this study a ten story regular framed reinforced structure was selected and structural modeling and analysis of the structure was done using ETABS 2016.1.0 software. The lateral force method of analysis was used for seismic action effects. Then the structure was designed using capacity design philosophy using four different column-beam overstrength factors according to the requirements of the code. The detailing of critical regions was done carefully for the required energy dissipation. After the buildings have been designed for the incoming earthquake, the performance evaluation of the buildings was done using static nonlinear analysis methods.

Finally, the influence of overstrength factor on the global seismic performance of the structure was discussed. The parameters used for the evaluation of seismic performance of the structures are capacity curve, story displacement, interstory drift and plastic hinge distribution of the building.

Keywords: Capacity Design, Capacity Curve, Ductility Class, Performance Level, Overstrength Factor, Plastic Hinge, Pushover Analysis.

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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
T_c	Corner period at the upper limit of the constant acceleration region of the elastic spectrum
q	Behaviour factor
a_g	Design ground acceleration on type A ground
DCM	Ductility Class Medium
DCH	Ductility Class High
T_1	Fundamental period of vibration of a building
NSA	Nonlinear static analysis
γ_{Rd}	Overstrength factor
RC	Reinforced concrete
γ	Unite weight of materials
ULS	Ultimate limit state

CHAPTER 1 INTRODUCTION

1.1 Background

The dynamic response of the buildings to earthquakes motion is a complex, three dimensional, nonlinear and dynamic problems. Different limitations in technology and the depth of our understanding of this dynamic problem have led to the profession developing a number of simplified methods for representing it, most of which neglect one or more of its fundamental aspects of the structure: the Linear Static Procedure ignores both nonlinearity and dynamic effects; Linear Dynamic Procedure ignores nonlinearity; the Nonlinear Static Procedure ignores dynamic effects; the Linear Static Procedure ignores both (Mourad & Sabah, 2015; Chambers & Kelly, 2004).

The design of earthquake resistance building needs the determination of all the loads that acts on the structure. The actual force that occurs during the earthquake motion cannot be determined exactly (Dowrick, 2009). An accurate estimate of the incoming earthquake force is very important, however, since the cost of construction, and therefore the economic viability of the project depends on a safe and cost efficient final product.

High seismic regions have experienced different small earthquake levels in different times. However, a structural damage does not usually occur until the magnitude of motion approaches 5.0 (Chen & Lui, 2006). Damages of structural buildings during earthquake motion are mainly caused by the failure of surrounding soil and strong shaking. And also there are different causes of damage of the structure including surface rupture and collapse of vulnerable structures in the surrounding of the structure.

Nowadays, the term Performance of the structure is being used as a popular word in the field of earthquake engineering, with the structural engineer taking deep interest in its concepts due to its potential benefits in assessment, design and better understanding of structural behavior during strong ground motions (Sextos, Simopoulos, & Skoulidou, 2015; Bento, Falcão, & Rodrigues, 2004).

Due to an advanced understanding of the response of the structure and the development of structural dynamics concepts new design method is developed which is known as the capacity design philosophy. These design methods provides how the structure is failed and to know the expected plastic mechanism on the structure (Paulay & Priestley, 1992). Additionally capacity design method gives well known behavior of the structure under earthquake motion and plastic deformations are occurs at the pre-defined plastic hinges.

Most building structures exhibit a nonlinear dynamic response when subjected to medium up to high intensity earthquake motion. In the current studies (Carvalho, Bento, & Bhatt, 2012; Causevic & Mitrovic, 2010) however, that this phenomenon is not properly modeled in the majority of the cases, especially at the design stage, where only simple linear methods have effectively been used.

Recently, as a result of dramatic development of analysis tools, nonlinear modeling and analysis have gradually been brought to a more promising level. A wide range of modeling alternatives developed by (Fardis, 2010) over the years is hence at the designer's disposal for the seismic design and assessment of engineering structures.

Nowadays, modern seismic design codes allow engineers to use either linear or nonlinear analyses to compute design forces and design displacements. In particular, (Ethiopian Building Code Standard-8 [EBCS EN 1998-1], 2014) contains four methods of analysis: Lateral force method (elastic), modal response spectrum analysis (elastic), nonlinear pushover analysis and nonlinear time-history analysis. These methods refer to the design and analysis of structures, mainly buildings and bridges. The two nonlinear methods require advanced models and advanced nonlinear procedures in order to be fully applicable by design engineers (Mourad & Sabah, 2015).

The static pushover analysis is relatively simple analysis method to produce an approximate solution to the complex and three dimensional problem of predicting force and deformation demands imposed on the structures and their elements by severe earthquake motion and evaluation of structures. The static pushover analysis method also gives approximate results of the structural system because this analysis method ignores the dynamic behavior of the structure (Causevic & Mitrovic, 2010).

1.2 Statement of the Problem

Recent modern seismic codes in different countries including Ethiopia the provisions for structural buildings rely on energy dissipation through inelastic structural deformation during the designed seismic load to the necessity to compromise damages with economic consideration. Structural buildings designed by strength design philosophy under the sever earthquake motion results unknown plastic mechanism formation and plastic hinge regions and the global ductility of the structure is not capture exactly. And also plastic mechanisms are arbitrary and not identified. Because of this limitation most of seismic codes now a day adopt the capacity design philosophy. The aim of this design philosophy is providing hierarchy of strengths in the structures under seismic load; plastic deformations are possible only in the pre-defined critical plastic hinge regions and the structure is designed with a known failure mode.

In the capacity design philosophy the column for flexure is that any beam-column joint the strength of the column must be larger than the beam to obtain strong column with weak beam.

$$\sum \mathrm{M_{Rc}} \geq \gamma_{\mathrm{Rd}} \sum \mathrm{M_{Rb}} \tag{1.1}$$

Where M_{Rb} is the sum of the design values of the moments of resistance of the beams framing the joint, M_{RC} is the sum of the design values of the moments of resistance of the columns framing the joint and γ_{Rd} is the overstrength factor.

From Eq. (1-1) it is now clear that different source of uncertainties associated with the capacity design of structures. Uncertainties related to column understrength are the first uncertainties. These column uncertainties arise from variation in axial loading, biaxial strength, construction tolerance and material variability. Secondly, uncertainties comes from the beam overstrength from strain hardening of the steel bar, due to slab reinforcement, two directional yielding of beams and material variability associated with yield strength.

These sources of uncertainties affect the structural response of the building that are designed based on capacity design method. The overstrength factor for the beam is assumed that all sources of overstrengths are to be considered to design the column as elastic member. From the previous discussions variation of column-beam overstrength factor is affect the global seismic performance of the structure. So the influence of this overstrength factor on the global seismic performance of reinforced concrete structures was assessed in this research.

1.3 Objectives

1.3.1 General Objective

The general objective of this research is to study the influence of overstrength factor on the global seismic performance of newly designed regular frame reinforced concrete structure.

1.3.2 Specific Objectives

While in the assessment of the global seismic performance of RC frame structure design according to capacity design philosophy using different overstrength factor the following specific objective carried out:

- To know capacity design Philosophy procedure according to EBCS-EN-1998-1-2014.
- To know the expected distribution of plastic hinges.
- To know the detail analysis procedure and requirement of the non-linear static methods.

1.4 Scope of the Study

The scope of this thesis is to study the influence of column-beam overstrength factor on the global seismic performance of newly designed regular frame reinforced concrete structure on the bases of different structural parameters. In this research a ten story four by three bay building is selected. Lateral force method of analysis is used for the elastic analysis of the building. The design method used in this thesis is capacity design philosophy based on the new Ethiopian building code ([EBCS EN 1998-1], 2014). Both ductility classes, ductility class medium and high buildings are considered in this research. The structures are designed using four different overstrength factors. After a carful design and detailing of critical regions of the structural elements, performance evaluation of the structure is done using static pushover analysis.

1.5 Significance of the Research

The significance of this research is to understand the effect of column-beam overstrength factor on the global seismic performance of the structures that are designed by capacity design philosophy. And also a detail concept and design procedure of capacity design philosophy according to the new Ethiopian building code was described ([EBCS EN 1998-1], 2014). The performance evaluation of new and existing structure is always the intention of earthquake engineering to protect undesirable failures and to serve the desired function without any failure signs throughout its design life. So this research provides the detailed analysis procedures of the performance assessment of structures under earthquake motion using nonlinear static analysis.

The concepts of overstrength factor described in detail and structural parameters that are affect the overstrength factor of the structure also discussed. An understanding of the structural overstrength factor leads to which structural parameters are affected in the design of earthquake resistance buildings due to this factor. The analysis procedure and clear concepts of nonlinear static analysis also described. Advantage and limitation of nonlinear static analysis method and different uncertainties related with strong column and week beam design method are described. Structural parameters which influence the column-beam overstrength as well as the performance of the structures are discussed.

Detailing and dimensioning of primary seismic columns for the two structural elements beam and column are summarized in the tabular form based on the new Ethiopian building code. This summary provides simple reference for the designers to use the capacity design method. Advantage of the capacity design method is also discussed.

Generally, the result of this study provides essential base line to practice capacity design philosophy and also provides an understanding of the influence of overstrength factor on the performance of newly designed reinforced concrete structures. The criteria for the design of frame elements in the capacity design method are described in detail. Different assumptions of nonlinear static pushover analysis also discussed.

CHAPTER 2 LITERATURE REVIEW

2.1 Basic Concept of Seismic Design

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, Kumar, Kumar, & Murari, 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 (Dowrick, 2009).

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 (Chen & Lui, 2006; Paulay & Priestley, 1992).

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.

Secondly, Damage control limit state for large intensity earthquakes that is greater than serviceable limit state and cause adverse 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 (Causevic & Mitrovic, 2010).

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. ([ATC], 1996; Kumar et al., 2014).

2.1.1 Design Concepts for Safety under Design Seismic Action

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 ([EBCS EN 1998-1], 2014; New Zealand Standard, 2006).

Global ductility:

- Structure forced to remain straight in elevation through shear walls or strong columns ($\Sigma M_{Rc} > 1.3\Sigma M_{Rb}$ in frames).

Local ductility:

- Plastic hinges detailed for the required ductility capacity;
- Brittle failures prevented by overdesign/capacity design

Capacity design of foundations & foundation elements:

- On the basis of overstrength of ductile elements of superstructure. (Or: Foundation elements - incl. piles - designed & detailed for ductility).

2.1.2 Types of Seismic Design Philosophy

To satisfy the requirements provided in different country codes and for acceptable design of seismic resistance structures different design philosophy's are developed by different literatures and country codes (Necevska-Cvetanovska & Petrusevska, 2000; Sextos et al., 2015; Sahoo, 2008; Victorsson, 2011).

Strength Design Philosophy

This is most common design approach adopted in the previous period. This philosophy allows the structure remains elastic during the earthquake motion and no ductility is insured in this design philosophy and only some simple construction detail rules are needed to be satisfied.

Capacity Design Philosophy

In this design approach the structures are designed in such a way so that plastic hinges can form only in predetermined positions and in predetermined sequences. The concept of this method is to avoid brittle mode of failure. This is achieved by designing the brittle modes of failure to have higher strength than ductile modes.

Capacity Design is a concept or a method of designing flexural capacities of critical member sections of a building structure based on the actual response of the structure under earthquake motion. This design provides strong column and weak beam of structural elements and the plastic mechanism of the structure is known.

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. Therefor 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.

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.

2.2 Performance Based Seismic Design

As per the conventional earthquake-resistant design philosophy, the structures are designed for forces which are much less than the expected design earthquake forces. Hence, when a structure is hit with large earthquake ground motion, it undergoes inelastic deformations through different structural elements. Even though the structure may not collapse, the damages cannot be controlled by simple maintenance. These methods usually don't consider the expected performance objectives and seismic risk levels of the structure after an earthquake event.

Since, these methods give high base shear, high ductility demand and also don't give the actual performance of structure after an earthquake event need of new method comes which would give the actual performance of the structure after the occurs of an earthquake event.

Performance based seismic design is described in (Bagchi, 2001) a process of designing new buildings or seismic up-gradation of existing buildings, which includes a specific concentration to achieve defined performance objectives in future severe earthquakes.

The performance level or objective of structural buildings are relate to expectations regarding the amount of damage a structural building may experience in response to earthquake shaking and the consequences of that damage. There are different performance objectives that 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 structures (Chaudhari & Dhoot, 2016).

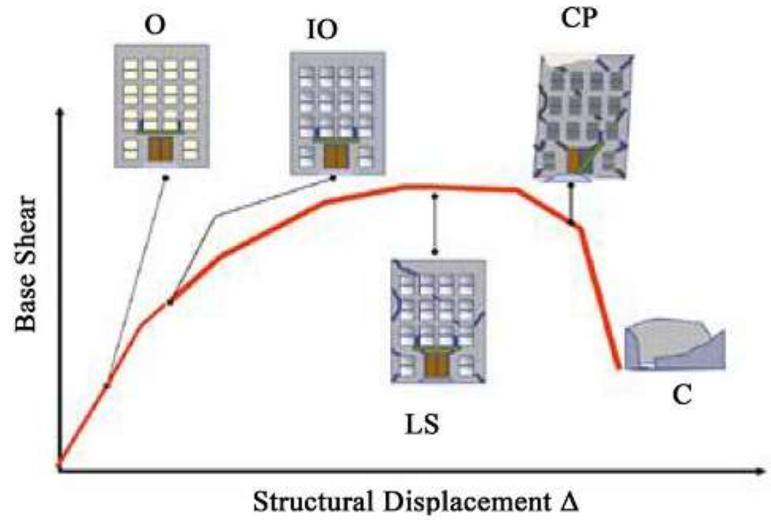

Figure 2.1 Performance Levels, (Chaudhari & Dhoot, 2016)

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 2-2 shows 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.

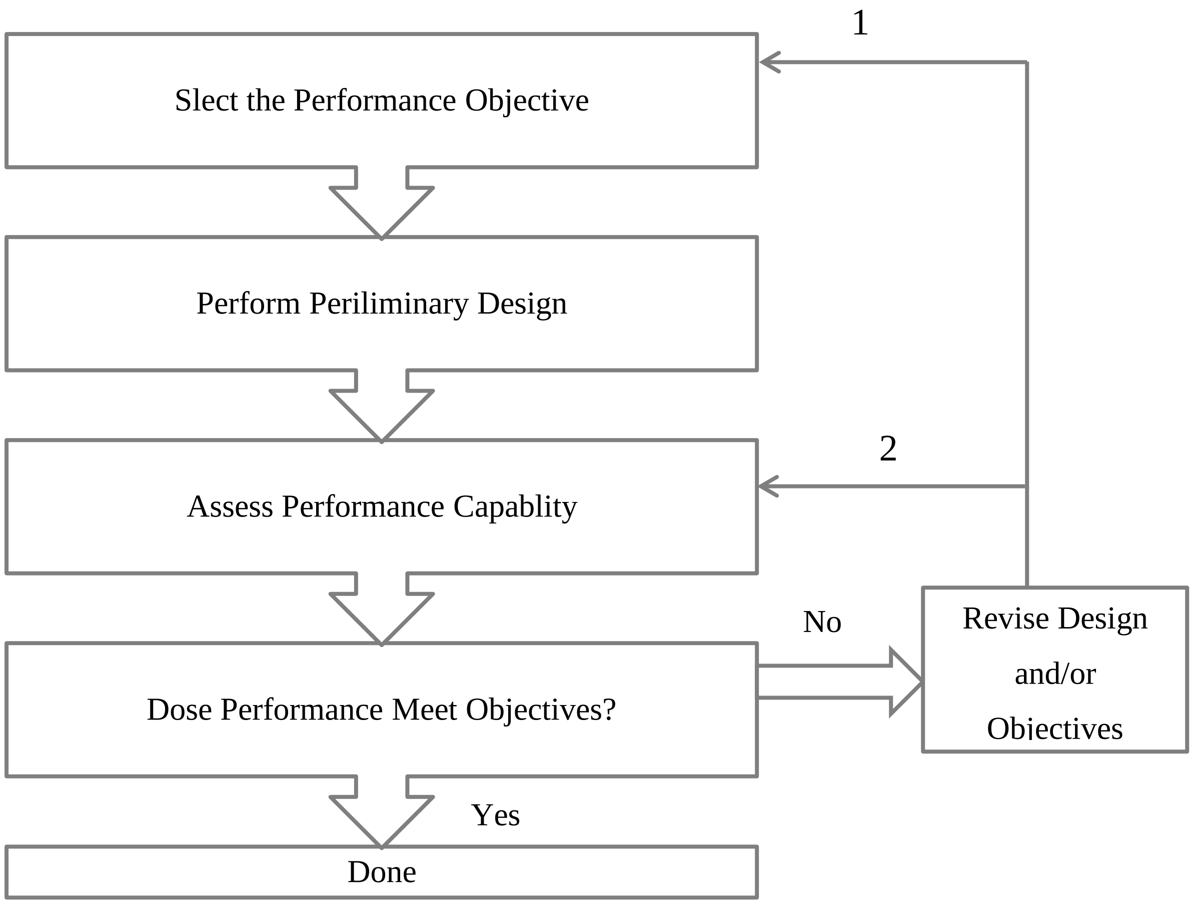

Figure 2.2 Flowchart of Performance Based Design Process (Chaudhari & Dhoot, 2016).

The performance based philosophy measures acceptability and changes in acceptability caused by previous damage on the basis of the degree to which the structure achieves one or more performance levels for the hazard caused by one or more expected future earthquakes. A performance objective typically is defined by a particular damage state for a building for the incoming earthquake (Chaudhari & Dhoot, 2016; [FEMA], 2000).

2.3 Capacity Design Philosophy

2.3.1 Features of Capacity Design Method

Capacity Design method is a concept of designing of critical frame members based on a hypothetical behavior of the structure in responding to dynamic earthquake actions. The hierarchy of strength of the structure is reflected by (Paulay & Priestley, 1992) the assumptions that the earthquake action is of a static equivalent nature increasing gradually until the structure reaches its state of near collapse and that the critical sections are starting the development of plastic hinging simultaneously at the predetermined locations to form a collapse mechanism simulating ductile characteristic of the structure.

The actual behavior of a building structure during a maximum earthquake action is far from that described above, with seismic actions having a vibratory and dynamic character and plastic hinging occurring in the critical regions rather randomly. However, by applying the Capacity Design philosophy in the design of the flexural structural members of the structure, it is believed that the structure will possess adequate earthquake force resistance, as has been proven in many strong earthquakes in the past (Kumar et al., 2014).

A feature in the capacity design concept is the ductility level of the structure, expressed by the displacement ductility factor or briefly ductility factor. This concept defined by (Sahoo, 2008) ductility factor is the ratio of the lateral displacement of the structure due to the design earthquake at near collapse and that at the point of first yielding.

The basic of capacity based design lies on weak beam and strong column concept. The earthquake inertia forces generated at its story levels are transferred through the various structural elements that are beams and columns to the ground. The structural members need to be made ductile to dissipate the incoming seismic force. Based on the principle of capacity design philosophy the failure of columns causes global failure but the failure of beams causes local failure. Therefore, it is better to make the column is stronger than the beam element. This method of designing RC buildings is called the strong-column weak-beam design method ([FEMA], 2000; Victorsson, 2011).

In the capacity design of structures for earthquake resistance, distinct elements of the primary lateral force resisting system are chosen, suitably designed and detailed for energy dissipation under severe imposed deformations. As mentioned in (Paulay & Priestley, 1992) all other elements are, then, to be protected against actions that could cause failure by providing them with strength greater than that corresponding to the maximum feasible strength in the potential plastic hinge regions.

Ductile reinforced concrete structures are design according to this design philosophy. In capacity design method of seismic resistance structures, primary lateral resisting systems are chosen and carefully design and detailed for energy dissipation under severe earthquake load ([EBCS EN 1998-1], 2014). These critical regions of the primary elements are known as plastic hinges. Plastic hinges are carefully design and detailed for inelastic flexural actions and protect from shear failure by providing strength differential from the former case. All other secondary structure elements are design and detailed to the strength greater than the maximum strengths provided to the critical regions.

All other structural elements except primary elements are remains in the elastic range under the earthquake motion and primary elements subjected to ductility demand are designed below its maximum elastic response.

Nowadays, most building codes include capacity design philosophy to help ensure ductile response and energy dissipation capacity in earthquake resistance designs. The design philosophy are drive towards limiting significant nonlinear deformations to those structural elements that are designed with sufficient inelastic deformation capacity (Victorsson, 2011). Those components of the structure are generally referred to as displacement or deformation controlled components. Other components of the structure, referred to as force-controlled components, are designed with sufficient strength to remain essentially elastic under the design earthquake force.

Capacity design method allows the designer to tell to the structure what to do by providing hierarchy of strength in the structure by ensuring the beam strength less than that of column strength by providing over strength factor (Paulay & Priestley, 1992; Dowrick, 2009) and the structural failure is follow the predetermined paths through plastic hinge regions. The designed structure after the occurrence of the maximum earthquake force does not fail randomly.

Establishment of margins between demand and capacity to ensure the desired behavior of the designed structure requires consideration of uncertainties in both local component strengths and overall global indeterminate system response. Moreover, (Victorsson, 2011; Necevska-Cvetanovska & Petrusevska, 2000) describes the extent to which capacity design is or should be enforced to create ideal mechanisms is a matter of debate and involves a trade-off between structural strength and economy. There are also cases where it can prove almost physically impossible to create the ideal mechanism.

2.3.2 Illustrative Analogy of Capacity Design Method

To illustrate the basic concept of capacity design philosophy consider the following tube welded by ductile and brittle material.

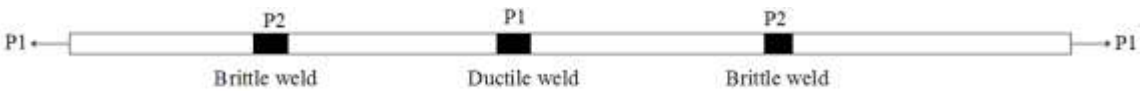

Figure 2.3 Principle of Strength Limitation in Ductile Tube

As we see from the Figure above the middle weld ductile and the outer welds are brittle. So the design strength of weld that susceptible to brittle failure must be greater than that of ductile failure welds. If both welds are design according to their nominal strength the probability of the tube fail in brittle mode is higher possibility and brittle failure is not recommended (Paulay & Priestley, 1992). Therefore to avoid this brittle failure the strength of brittle welded regions must be designed to the strength greater than that of over the strength capacity of ductile welded regions. But the final design failure load is similar. The difference comes when the ductile members reach the yielding strength the brittle weld regions remains elastic. From the figure P_1 is the nominal strength of ductile weld and if the tube is designed for the maximum tensile earthquake force P_S and

$P_1 = \phi . P_S$. To avoid the brittle failure the brittle weld should be designed by establishing over strength capacity $P_O = \lambda_O P_S$. Therefore the nominal strength of the brittle weld needs to be

$$P_2 > \frac{P_O}{\phi_b} \tag{2.1}$$

Where subscript b refers to brittle link

ϕ is the strength reduction factor and λ_O is over strength factor.

2.3.3 Basic Design Procedures

Basic procedures in the capacity based design according to ([EBCS EN 1998-1], 2014; Paulay & Priestley, 1992; Sahoo, 2008).

I. Design loads such as dead loads, live loads and earthquake loads are calculated and seismic analysis of the frame for all load combination specified in codes are done.
II. Critical or plastic hinge regions within the structure must be defined and Members are designed for maximum forces obtained from all load combinations. Beams are designed for maximum negative and maximum positive moments. Provided reinforcements by checking all the requirements.
III. The flexural capacities of the beams under positive and negative condition for the provided reinforcements are calculated.
IV. The flexural capacity of columns at a joint is compared with actual flexural capacity of the beams at the joint. If the sum of the beam moment is less than the sum of column moment, no need of factoring the design moment. If the sum of the beam moment is greater than the sum of the column moment multiply the column moment by overstrength factor until the column moment was greater than the sum of the beam moment.
V. The critical regions must be designed to prevent shear failure by providing strength greater than the over strength capacity of critical regions.
VI. Other structural element remains in the elastic range under the maximum seismic intensity. And brittle failure elements must be carefully designed to the strength greater than the over strength capacity of the critical or plastic regions.
VII. Carefully done the detailing of the structural elements for critical and non-critical regions by checking all requirements provided in the codes.

2.4 Concept of Overstrength Factor

Observation of structural performance under many pervious earthquake motions have led to the conclusion that code deigned buildings must possess significant overstrength in order for them to have continued without damage earthquake forces considerably larger than those considered in design of the building. Many researchers have tried to identify the factors that may have contributed to the observed overstrength. These challenges have been useful in understanding the phenomenon of overstrength but may have also led to the confidence that the identified source of overstrength can be add up upon in designing new building structures (Humar & Rahgozar, 1996; Taïeb & Sofiane, 2014). A critical determination of the factors that contribute to the reserve strength is essential to understand when and if a particular source of overstrength can be relied upon.

The main sources of overstrength includes: according to (Taïeb & Sofiane, 2014) the variation of designed and actual (in the construction) material strength; the design and analysis method used for structural designs and analysis; factor of safety for load and material; additional accidental eccentricity usage; conservative criteria for serviceability limit states; the effect of non-structural elements and structural elements that are not considered in the lateral resistance; minimum requirements provided in the code larger than the design requirement; effect of redundancy and properties of redistribution stress of structural elements; the strength increment due to the confinement of concrete and strain hardening of the reinforcement.

2.4.1 Flexural Overstrength Factor

To know the hierarchy of strength in the design of ductile structures using capacity design philosophy, it is convenient to express the overstrength of a member in flexure at a section, such as at the end of beam element , in terms of the required flexural strength at the same section, derived by an elastic analysis (Paulay & Priestley, 1992; Taïeb & Sofiane, 2014).

There are two types of overstrength factors considered in the structural systems. These are overstrength factor from material and flexure at the section. The material overstrength obtained from the strength difference between actual and design strength difference. But the source of flexural overstrength factors are includes the following source of overstrength in addition to material overstrength;

- Conservative strength requirements by considering overestimated design forces.
- Design moment changes due to the redistribution of moment from one section to other sections.
- Deviation from the required strength of the section due to the code requirement and practical availability of construction materials (Paulay & Priestley, 1992).

2.4.2 System Overstrength Factor

The system overstrength factor describes the overstrength of the whole structural system by comparing with the demand needs. But the flexural overstrength factor is measure at the node point locally. In certain situation the system overstrength factor is also important. For example, the system overstrength shows the domination of the load compared with earthquake force alone and the sum of the flexural overstrengths of all column sections of a framed building at the bottom and the top of a story may be compared with the total story moment demand due to the total story shear force (Humar & Rahgozar, 1996; Manola & Koumousis, 2010; Paulay & Priestley, 1992).

2.5 Structural Performance

Seismic performance is described by designating the maximum allowable damage state (performance level) for a known seismic hazard (earthquake ground motion). A performance objective specifies the chosen seismic performance of the building ([ATC], 1996). A performance objective may include consideration of damage states for several objective levels of ground motion and would then be termed a dual or multiple level performance objectives.

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).

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 ([EBCS EN 1998-1], 2014) 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 (Causevic & Mitrovic, 2010; Mourad & Sabah, 2015).

2.6.1 Non-Linear Static (Pushover) Analysis

Pushover analysis method is nonlinear static analysis method. This method is carried out under constant gravity loading and monotonically increasing lateral loads. Pushover analysis is very practical to evaluate the performance of new and existing structures.

The capacity of structure is represented by pushover curve. The most convenient way to draw the load deformation curve is by plotting the bas shear and the top displacement in the vertical and horizontal direction respectively. The pushover procedure can be presented in various forms and it can be used in a variety of forms for the use of different performance check. As the name implies it is a process of pushing horizontally, with a prescribed loading pattern, incrementally, until the structure reaches failure state or near collapse. The maximum failure states are described in different limit states that are described in different codes (Causevic & Mitrovic, 2010; Fajfar & Eeri, 2000).

This method also described with different groups (Bento et al., 2004; [ATC], 1996).

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.

Adaptive Pushover

This method is also nonlinear static analysis method and the method accounts the possible change of inertia force distribution due to the change in the stiffness of the structural elements.

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 specially 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; [EBCS EN 1998-1], 2014).

The main advantages of pushover analysis over the two linear methods (Linear Static and Linear Dynamic analysis) are (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.

2.6.1.1 Lateral Load Distribution

For an adequate performance evaluation, the suitable selection of the load pattern is very important. These selected load patterns should bound approximately the likely distribution of inertia forces in a design seismic force, thus requiring to incorporate, in some cases, higher mode effects into the selected lateral load pattern (Bento et al., 2004).

As no single load distribution can identify the variation of the local demands expected in a design earthquake, the use of at least two load patterns is recommended. For instance the ([EBCS EN 1998-1], 2014; [FEMA], 2000) propose two lateral load patterns in the non-linear static procedure:

- The uniform load pattern, leads to conservative values of demands in lower stories, compared to the upper values, and emphasizes the importance of story shear forces compared with overturning moments;
- A modal pattern, which can account for elastic higher mode effects, makes a good choice for the second load pattern.

The main concern in using the invariant load pattern for the pushover analysis is that, it is possible to identify only confined mechanism that could occur in an earthquake while weaknesses due to dynamic characteristics change may not be identifiable. It is apparent that none of the invariant lateral load patterns referred can reflect correctly the inertia forces redistribution when some elements undergo non-linear behavior (Bento et al., 2004; [EBCS EN 1998-1], 2014; Krawinkler, 1997).

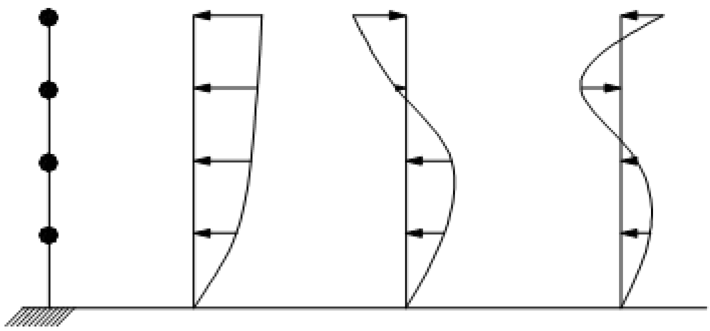

Figure 2.4 Load Patterns due to Higher Modes (Bento et al., 2004)

2.6.1.2 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.

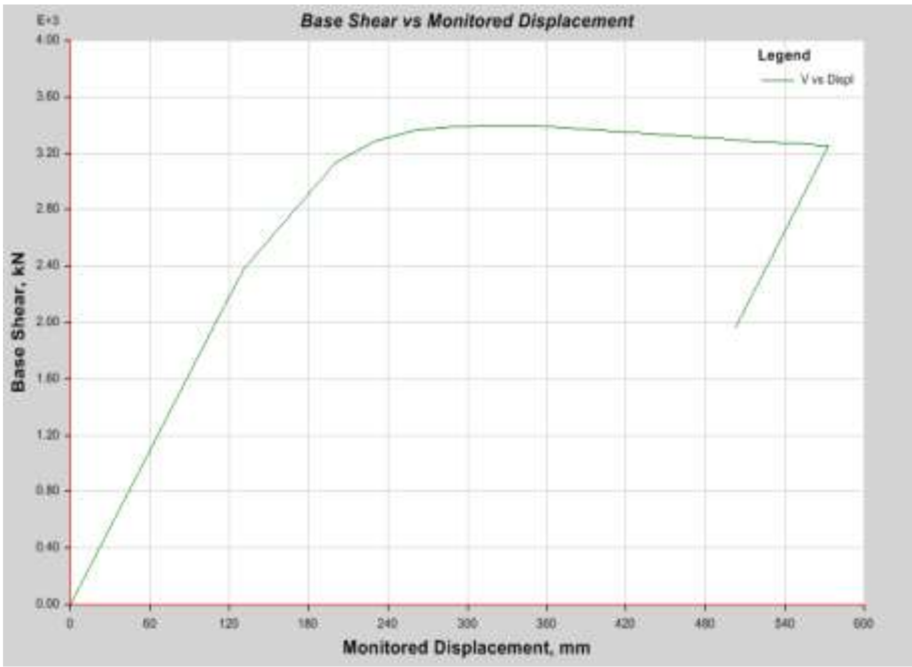

Figure 2.5 Capacity Curve of the Building

The control displacement is in the range of zero up to the value corresponding to 150% of the target displacement according to ([EBCS EN 1998-1], 2014).

2.6.1.3 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 (Chopra, 1995; [EBCS EN 1998-1], 2014).

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 ([ATC], 1996; [EBCS EN 1998-1], 2014; Federal Emergency Management Agency [FEMA], 2000).

CHAPTER 3 METHODOLOGY

3.1 General

A ten story reinforced concrete building for mixed use building is selected for this research. The floor plan and the chosen framing system are shown in the Figure A.1, Figure A.2 and Figure A.3 in the appendix part. The building was analyzed and designed for the two ductility classes and the global performance of the building also done for the two ductility classes DCM and DCH.

3.2 Materials and Software Used

Table 3.1 List of Materials and Software Used for the Analysis and Design

Materials Used	**Beam**	**Column**
Concrete strength	C25/30	C25/30
Yield strength of rebar-G60 (MPa)	420	420
Software Used	**Purpose**	
ETABS 2016	To analyze (both structural and non-linear static analysis) the structure	
AutoCAD	For detailing	

3.3 Methods of the Research

3.3.1 Geometry of the Building

The general layout of the building is shown in Figure A.1, Figure A.2 and Figure A.3 in the appendix part. The beam size used in this thesis is selected based on the incoming load and criteria provided in the Ethiopian building code for different ductility class. The column size used for the analysis was based on the load and the criteria provided for ductility class medium and ductility class high structures.

3.3.2 Loading

The dead and live load are calculated based on the function of the building and the size of the structural elements. The slab thickness used for this analysis is 150mm. The selected material property and live loads are used according to ([EBCS EN 1998-1], 2014; Ethiopian Building Code Standard-2 [EBCS-2], 2014).

Table 3.2 Material Limitations for Primary Seismic Elements

Ductility Class	DCH	DCM
Concrete grade	≥C20/25	≥C16/20
Steel class	Only C	B or C
Longitudinal bars	only ribbed	only ribbed
Steel overstrength	$f_{yk,0.95} \leq 1.25 f_{yk}$	No limit

3.3.2.1 Gravity Load

Design gravity loads are determined from dead load and live load. Dead load is the load comes from the self-weight of the structural elements used in the model. Self-weights of the structural elements are dependent on the size and the characteristic strength of the material used in the model. The characteristic strength of the material used in this research is based on the new Ethiopian building code ([EBCS EN 1998-1], 2014) Table A.1-construction materials-concrete and mortar. Characteristic strength of normal weight concrete used in this research is γ=24 kN/m^3.

Live load of the structure is dependent on the function of the structure. Mixed used building is used for this thesis and based on the new Ethiopian building code EBCS EN 1998-1], 2014) Table 6.1 and Table 6.2. Imposed load used for the determination of gravity load is calculated multiplying by the area of the floor.

3.3.2.2 Earthquake Load

Design earthquake force is dependent on the seismic weight of the structure and the method of analysis used. The earthquake load is done using ETABS 2016 software. Sample seismic weight calculation and analysis method is described below.

Seismic Weight Determination

The weights used for the calculation of seismic loads are similar to the calculation of gravity load. 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 seismic load. The Combination coefficient is determined based on the function of the building from ([EBCS EN 1998-1], 2014; Ethiopian Building Code Standard-0 [EBCS EN 1990], 2014).
Combination coefficient, ψ_{Ei}, calculated from the following expression:

$$\Psi_{Ei} = \varphi . \Psi_{2i} \tag{3.1}$$

Whereφ: 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.

Design Seismic Load

The method of analysis used for the determination of design seismic load is Lateral force method of analysis. Ten story regular framed reinforced concrete building used in this thesis is satisfying the entire requirement of the code ([EBCS EN 1998-1], 2014).

(a) The fundamental period of the building satisfies:

$$T_1 \leq \begin{cases} 4.T_c \\ 2.0\text{sec} \end{cases}$$

Where T_C is given in Table C.1

$$T_1 = C_t H^{3/4} \tag{3.2}$$

C_t is 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures;

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

(b) The building is regular in plan and elevation

Determination of base shear force

For each horizontal direction base she force is calculated from the following equation:

$$F_b = S_d(T_1).m.\lambda \tag{3.3}$$

$S_d(T_1)$ is calculated using the following expression at period T_1;

$$T_B \le T \le T_C: \qquad S_d(T) = a_g.S.\frac{2.5}{q}.\left[\frac{T_c}{T}\right] \ge \beta.a_g \tag{3.4}$$

T_1 is the fundamental period of vibration of the building for lateral motion in the direction considered;

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

λ is the correction factor, the value of which is equal to: λ= 0.85 if $T_1<2\times T_C$=2*0.4=0.8 and the building has more than two storeys, or λ= 1.0 otherwise. λ= 1.0, used for this thesis.

β is the lower bound factor for the horizontal design spectrum.

The recommended value for β is 0.2.

Distribution of total horizontal load to different story levels

$$F_i = F_b \frac{Z_i m_i}{\sum_{j=1}^{10} Z_j m_j} \tag{3.5}$$

Ground Type= Type A

Ground acceleration (a_g/g)=0.15

Soil factor (S) =1

Accidental torsional effect is considered using the following expression.

$$e_{ai} = \pm 0.05 L_i \tag{3.6}$$

e_{ai} is the accidental eccentricity of story mass i from its nominal location, applied in the same direction at all floors.

L_i is the floor dimension perpendicular to the direction of seismic action considered.

Behavior factor is calculated using the following two expressions for each ductility class
For ductility class medium structures,

$$\mathrm{q} = 3 \times \frac{\alpha_u}{\alpha_1} \tag{3.7}$$

For ductility class high structures,

$$\mathrm{q} = 4.5 \times \frac{\alpha_u}{\alpha_1} \tag{3.8}$$

$\frac{\alpha_u}{\alpha_1}$ is the overstrength factor

3.3.2.3 Imperfection Load

The imperfection load due to the possible deviation of geometry of the structure and load considered in this research using the following expressions;

$$\theta_i = \theta_o \alpha_h \alpha_m \tag{3.9}$$

θ_o is the basic value and θ_o =1/200 recommended

α_h is height or length reduction factor, $\alpha_h = \frac{2}{\sqrt{L}}$

α_h is reduction factor for number of members, $\alpha_m = \sqrt{0.5(1 + \frac{1}{m})}$

L is length or height [m].

m is number of vertical members contributing to the total effect

For unbraced members

$$H_i = \theta_i N \tag{3.10}$$

N is the axial load at the story required

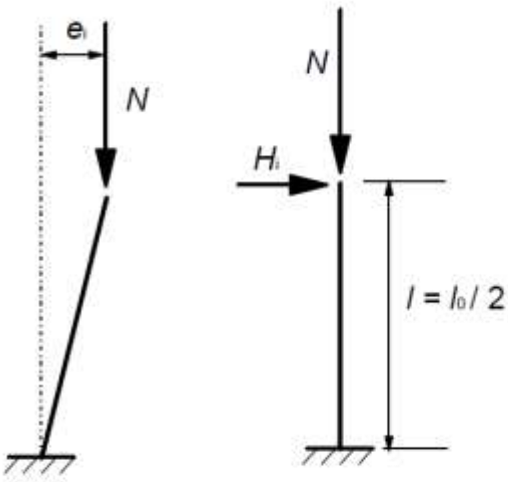

Figure 3.1 Imperfection Load in Unbraced Member ([EBCS EN 1992], 2014)

3.3.3 Modeling and Analysis of Structure

Modeling and structural analysis of the building was done using ETABS 2016 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 section elastic flexural and shear property modifier or stiffness modifier used in this research was one-half of the corresponding uncracked gross stiffness of the section. All the incoming loads including: dead, live and earthquake loads including imperfection loads was defined in the load pattern option in ETABS 2016 software.

In this research all possible load combination was defined to get maximum action effect of the structural elements.

Table 3.3 Load Combination

Comb	Number	Loads
1	1	1.35DL+1.5LL
	2	DL+0.3LL
2	1	Comb1-2±EQ_{X1}±0.3EQ_{Y1}±Imp_x
	2	Comb 1-2±EQ_{X2}±0.3EQ_{Y1}± Imp_x
	3	Comb 1-2± EQ_{Y1}±0.3 EQ_{X1} ± Imp_x
	4	Comb 1-2± EQ_{Y2}±0.3 EQ_{X1} ± Imp_x

One of the elastic analysis method Lateral force method of analysis was used in this research. All the requirements used to this elastic analysis method were satisfied. The fundamental period of the structure was small and the contribution of higher modes for the structural response was insignificant. Draw the structural elements and assign all additional dead loads and calculated live loads to the structural model.

At the end analysis was done and extracts maximum effect for different structural responses such as flexure, axial and shear.

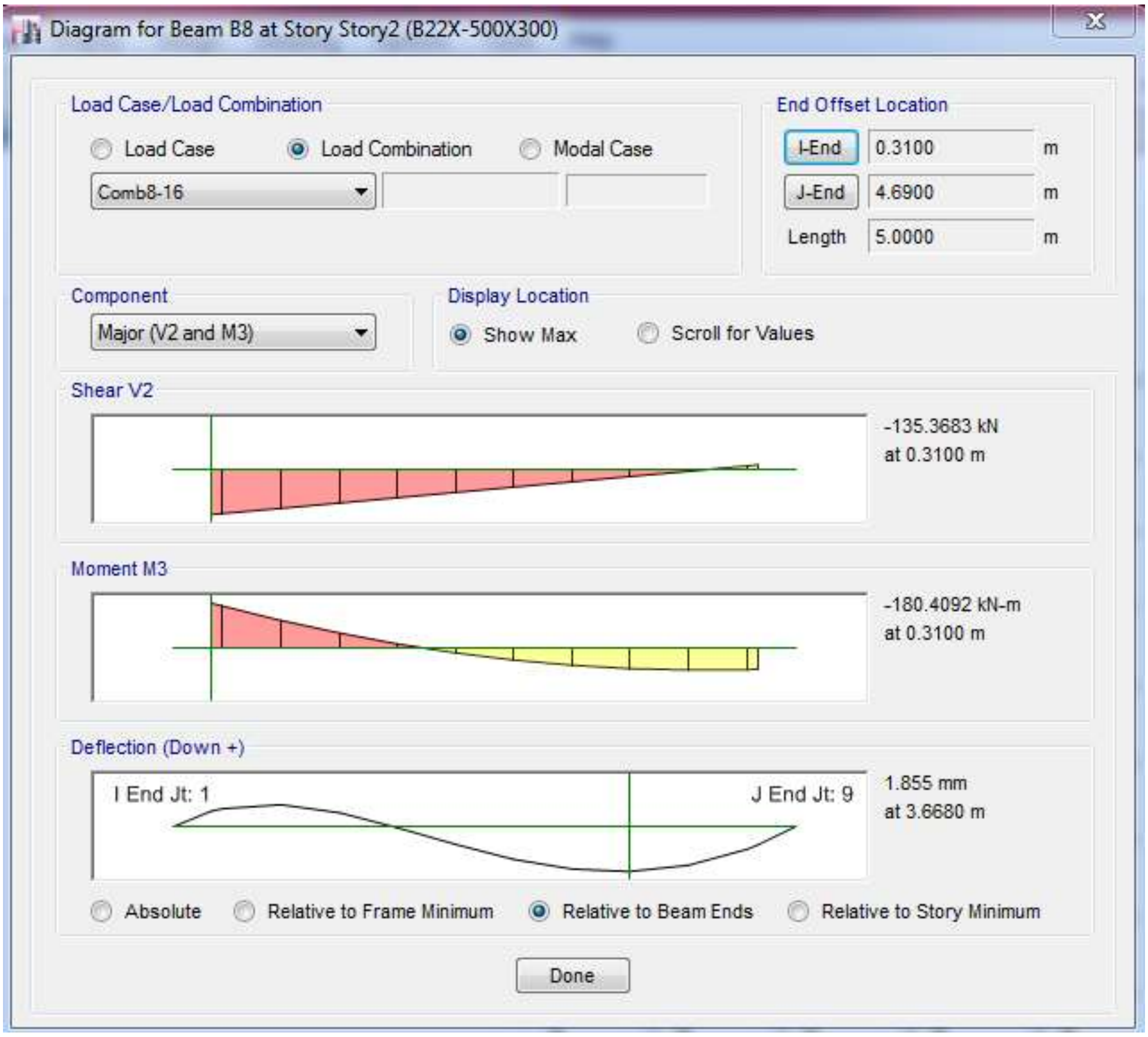

Figure 3.2 Sample Beam Analysis Output

3.3.4 Design of Reinforced Concrete Ductile Structure

The design of structural elements of the building was done by using capacity design philosophy based on the new Ethiopian building code ([EBCS EN 1998-1], 2014) requirement. The design procedure of the frame elements was described in detail in the following sections.

3.3.4.1 Design of Beam

Geometrical Constraints

In the design of ductile frames, limitation of section geometry of the element is important to minimize eccentricity of the axis of the beam relative to the column to achieve efficient transfer of cyclic moment.
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 was obtained by considering second order effect by using iterative P-Δ option and 10% moment redistribution of bending moment was done.
II) The design values of shear forces of primary seismic beams can be calculated as follows

The calculation of shear force was done 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, $M_{i,d}$

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} \min\left[1, \frac{\sum M_{Rc}}{\sum M_{Rb}}\right] \tag{3.11}$$

γ_{Rd} is the factor accounting for possible overstrength 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.

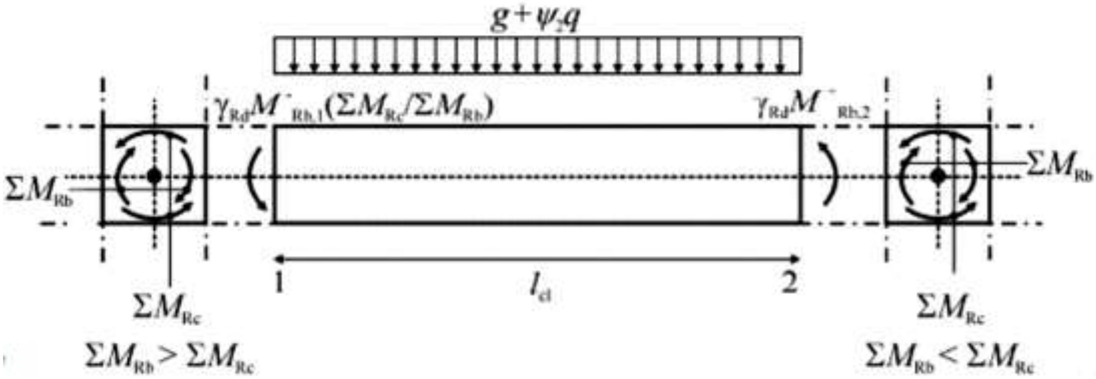

Figure 3.3 Capacity Design Values of Shear Forces on Beams ([EBCS EN 1998-1], 2014)

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

$$V_{Ed} = V_{o,g+\psi_2 q} \pm \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} \tag{3.12}$$

I) Flexural Design of Beam

The design of the beam element was designed using bending moment obtained from the structural analysis with 10% moment redistribution. The moment redistribution was performed by checking the requirement to get the required rotational capacity. For 10% moment redistribution the corresponding maximum design moment ratio, μ_{sd}, was 0.252. 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.

(1) Calculate

$$\mu_{sd} = \frac{M_{Sd}}{f_{cd} b d^2} \tag{3.13}$$

Where: M_{sd} is the design bending moment after redistribution

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \tag{3.14}$$

(a) If $\mu_{sd} \leq \mu_{sd,lim}$, then the section is single reinforced section.

Read ω from design table corresponding to μ_{sd} and calculate the tension reinforcement using the following equation:

$$A_{s1} = \frac{1}{f_{yd}} (\omega . b . d . f_{cd}) \tag{3.15}$$

Number of reinforcement, n_R, $= \frac{A_{s1}}{a_s}$

Where: a_s is the area of single rebar used, $a_s = \frac{\pi D^2}{4}$

D is the diameter of the rebar

(b) If $\mu_{sd} > \mu_{sd,lim}$, then the section is double reinforced section.

Read ω_{lim} from design table corresponding to $\mu_{sd,lim}$

$$\omega' = \frac{\mu_{sd} - \mu_{sd,lim}}{(1 - d_2/d)} \tag{3.16}$$

Then calculate tension reinforcement

$$A_{s1} = \frac{1}{f_{yd}} ((\omega_{lim} + \omega') . b . d . f_{cd}) \tag{3.17}$$

And calculate compression reinforcement

$$A_{s2} = \omega' . b . d . \frac{f_{cd}}{|\sigma_{s2} - \sigma_{cd,s2}|} \tag{3.18}$$

$$\sigma_{s2} = \varepsilon_{s2} . E_s \tag{3.19}$$

$$\varepsilon_{s2} = \left(1 - \frac{d_2/d}{k_x}\right) \varepsilon_c \tag{3.20}$$

$$\sigma_{cd,s2} = f_{cd}\left[1-\left(1-\frac{\varepsilon_{cs2}}{\varepsilon_{c2}}\right)^{n}\right] \qquad 0 \le \varepsilon_c \le \varepsilon_{c2} \qquad (3.21)$$

$$\sigma_{cd,s2} = f_{cd} \qquad \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \qquad (3.22)$$

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 3-5 below.

Table 3.4 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)	$\rho' + 0.0018 f_{cd}/(\mu_\phi \varepsilon_{sy,d} f_{yd})$ (3)	
$A_{s,min}$, top & bottom	-	$2\varphi14$
$A_{s,min}$, top-span	-	$A_{s,}$top-supports/4
$A_{s,min}$, critical regions bottom	$0.5A_{s,top}$ (2)	
Hoops or Transverse bars		
(a) outside critical region		
spacing $s_w \le$	$0.75d\,(1+\cot\alpha)$ (4)	
$\rho_{w,min}$	$0.08\sqrt{f_{ck}}/f_{yk}$, f_{ck} and f_{yk} in MPa	
(b) In critical regions		
$d_{bw} \ge$	6mm	
Spacing $s_w \le$	$h_w/4$; $24d_{bw}$; 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.

$$\mu_\phi = 2q_o - 1 \qquad \text{if } T_1 \ge T_C \qquad (3.23)$$

$$\mu_\phi = 1 + 2(q_o - 1)\frac{T_c}{T_1} \qquad \text{if } T_1 < T_C \qquad (3.24)$$

(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).

II) 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 overstrength 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 & Macgregor, 2012).

In this research vertical shear reinforcement was used for shear resistance of the section and the shear resistance of the beam element was the minimum of the following two expressions:

$$V_{Rd,s} = \frac{A_{sw}}{s} Z f_{ywd} \cot\theta \tag{3.25}$$

$$V_{Rd,s} = \alpha_{cw} b_w Z v_1 \frac{f_{cd}}{(\cot\theta + \tan\theta)} \tag{3.26}$$

Where:

A_{sw} is the cross-sectional area of the shear reinforcement

s is the spacing of the stirrups

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

ν_1 is a strength reduction factor for concrete cracked in shear

$$V_1 = 0.6\left[1 - \frac{f_{ck}}{250}\right], \quad (f_{ck} \text{ in MPa}) \tag{3.27}$$

α_{cw} is a coefficient taking account of the state of the stress in the compression chord, for non-prestressed structures α_{cw}=1 used in this research.

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.

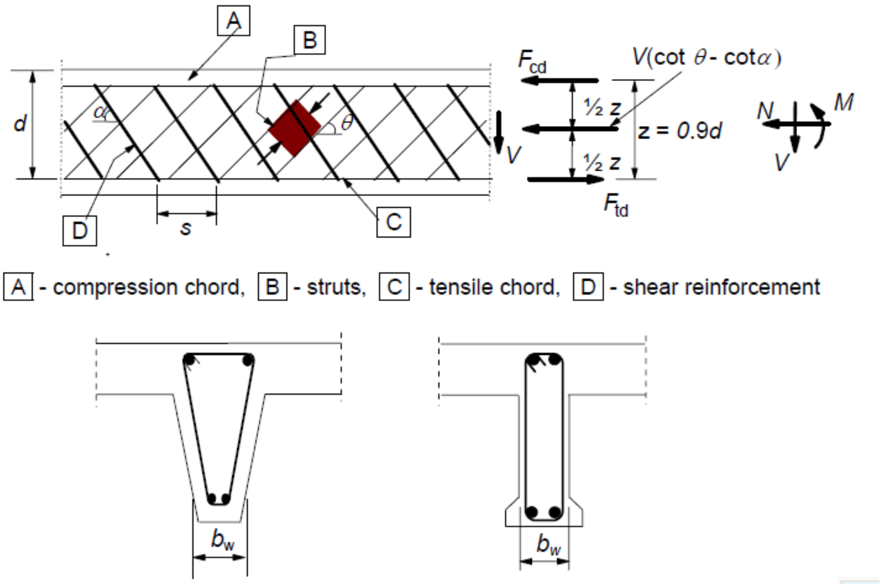

Figure 3.4 Truss Model and Notation for Shear Reinforced ([EBCS EN 1992], 2014)

3.3.4.2 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 interstorey 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:

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb} \tag{3.28}$$

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} overstrength factor on beam strengths

In this research different overstrength factor was used to obtain different design action effects for columns.

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

Column end moment $M_{i,d}$

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} \min\left[1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right] \tag{3.29}$$

Where

γ_{Rd} is the factor accounting for possible overstrength 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;

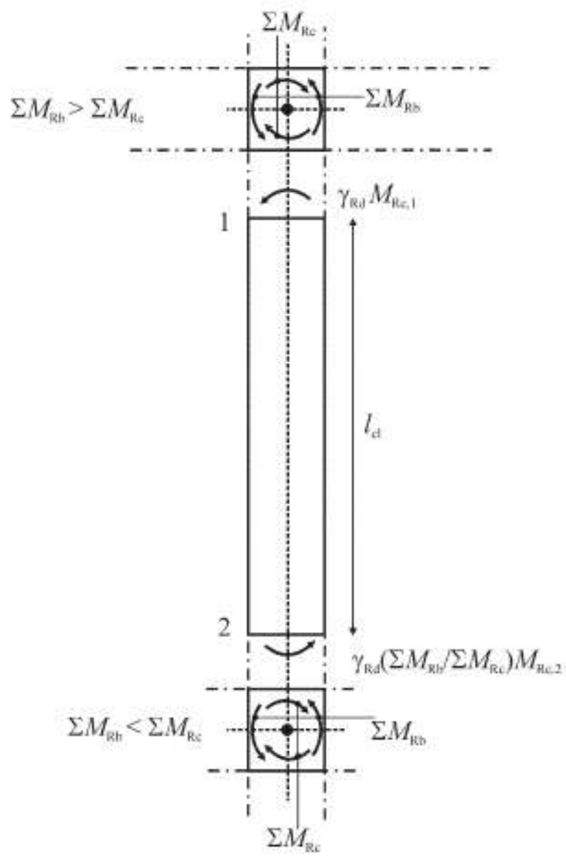

Figure 3.5 Capacity Design Shear Force in Columns ([EBCS-8], 2014).

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

$$V_{Ed} = \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} \tag{3.30}$$

$\boldsymbol{\gamma_{Rd}}$ is the factor accounting for possible overstrengths.

I) 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}

$$\upsilon_{sd} = \frac{N_{sd}}{A_c f_{cd}} \tag{3.31}$$

Where

N_{sd} is design axial load obtained from the overstrength 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}

$$\mu_{sdx} = \frac{M_{sd}x}{A_c h f_{cd}} \text{ and } \mu_{sdy} = \frac{M_{sd}y}{A_c h f_{cd}} \tag{3.32}$$

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

$$A_{s,tot} = \frac{\omega A_c f_{cd}}{f_{yd}} \tag{3.33}$$

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 3.5 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; $0.4d_{bl,Max}(f_{ydl}/f_{ydw})^{1/2}$
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

$$\omega_{wd} = \frac{\text{Volume of confining hoops}}{\text{Volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} \tag{3.34}$$

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

Where

$$\alpha_n = 1 - \frac{\sum_n b_i^2}{6 \times b_o h_o} \tag{3.35}$$

$$\alpha_s = \left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right) \tag{3.36}$$

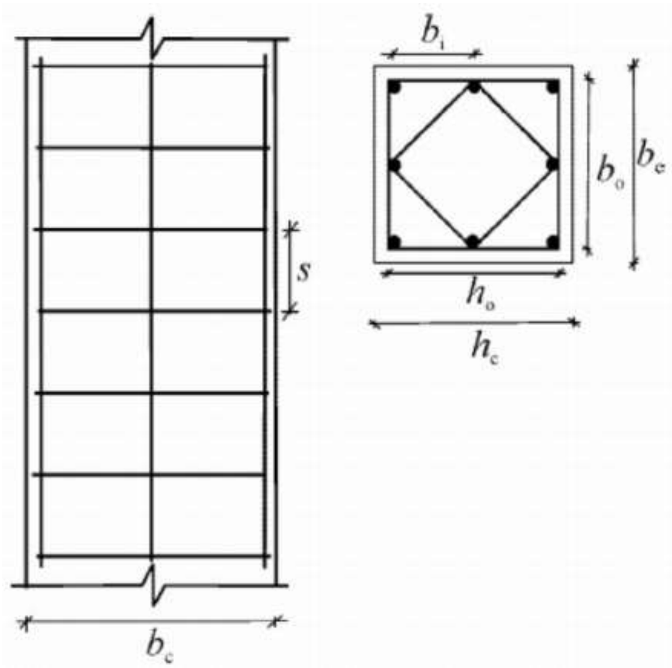

Figure 3.6 Confinement of Concrete Core ([EBCS-8], 2014).

II) 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.

III) 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:

$$V_{jhd} = \gamma_{Rd} A_{s1} f_{yd} - V_c \tag{3.37}$$

ii) For interior beam-column joints:

$$V_{jhd} = \gamma_{Rd}(A_{s1} + A_{s2}) f_{yd} - V_c \tag{3.38}$$

Where

A_{s1} 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.

$$V_{jhd} \leq \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_c \tag{3.39}$$

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

V_d the normalized axial force in the column above the joint

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

$$V_{jhd} \leq 0.8 \times \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}} b_j h_c \tag{3.40}$$

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

i) If $b_c > b_w$: $b_j = \min\{b_c; (b_w + 0.5.h_c)\}$ (3.41)

ii) If $b_c > b_w$: $b_j = \min\{b_w; (b_w + 0.5.h_c)\}$ (3.42)

Adequate confinement also check according to the new code ([EBCS EN 1998-1], 2014).

3.3.5 Performance Assessment According to Non-Linear Static Analysis

The performance of the building designed with different overstrength factor was assed using non-linear static analysis method to capture the nonlinear property of the building. The analysis was performed using ETABS 2016 software. To analyze the global performance of the buildings with different overstrength factor using the software mentioned previously the following steps were performed.

(a) Development of Structural Model

The development of modeling for the structure was the first step and three dimensional models were adopted in this research. The material property and cross-section of the structure was defined with the reinforcement obtained from capacity design of the element.

(b) Definition of Load Case

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

Load Application

The load applied on the structure to assess the performance of the structure was displacement controlled type was used.

The target displacement corresponding to the control node determined according to the following procedure.

Determination of Target Displacement

The calculation of target displacement was performed according to the method described in the new Ethiopian building code ([EBCS EN 1998-1], 2014). Elastic response spectrum was used for the determination of target displacement.

(c) Non-Linear Behavior Modeling of Critical Structural Members

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.
- The nonlinear behavior of plastic hinge regions was modeled with the simplified multi-linear force-displacement relationship. The characteristics of these simplified relationships was depend on different material and structural properties such as material property, geometry of the cross-section and assumed ductility class.

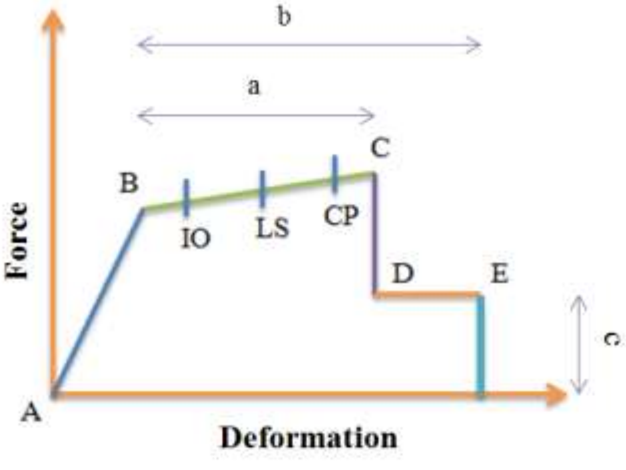

Figure 3.7 Force-Displacement Relationship of Plastic Hinges

a, b and c refers to the inelastic portion of the deformation. a stands for B-C portion; b indicates portion between B and E and c the force up to the point E. a, b and c ([ATC], 1996).

In the figure 3.9

- ✓ 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.

(d) 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 overstrength factor were analyzed and extract different global parameters for the performance comparison of the structure.

3.3.6 Comparison of the Performance of the Building

Finally the performance of structures asses using non-linear static analysis was compared and discussed with four structural parameters. The first parameter was capacity curve of the structure. The capacity curve was draw top displacement with base shear of the building. This parameter was describes the effect of overstrength factor on the capacity of the building. The story displacement was the second parameter used for the comparison of the performance of the building for different overstrength factors.

The thread parameter was interstory drift of the building. This parameter is also used for compare the performance of the building for different overstrength factor. This parameter tells as how the capacity of the building was decrease from story to story. The last parameter is plastic hinge distribution of the building. The plastic hinge distribution of the building was varying for different overstrength factor. So the effect of overstrength factor on the plastic distribution of the building was described using this parameter.

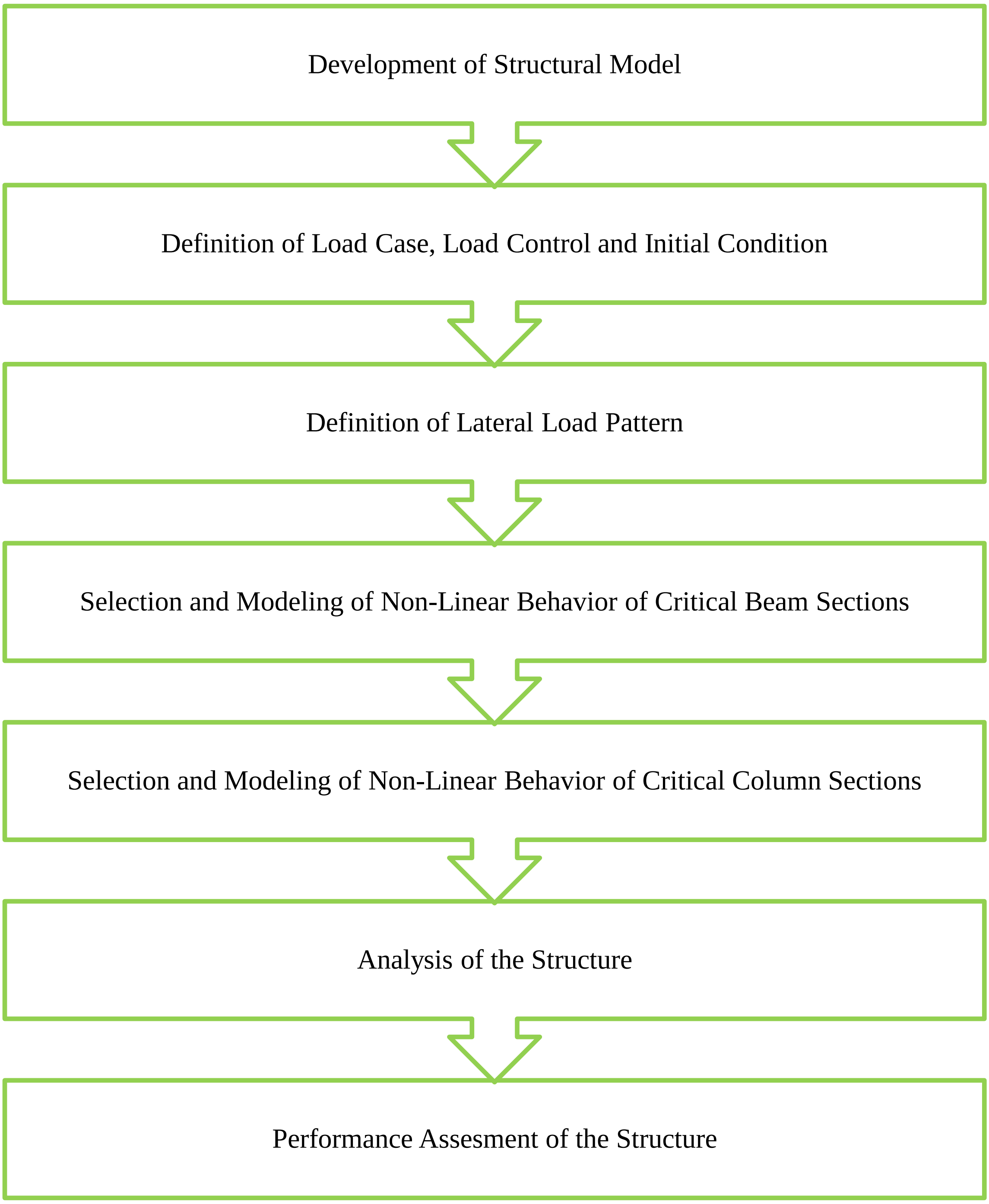

Figure 3.8 Main Steps of Non-linear Static (Pushover) Analysis Method

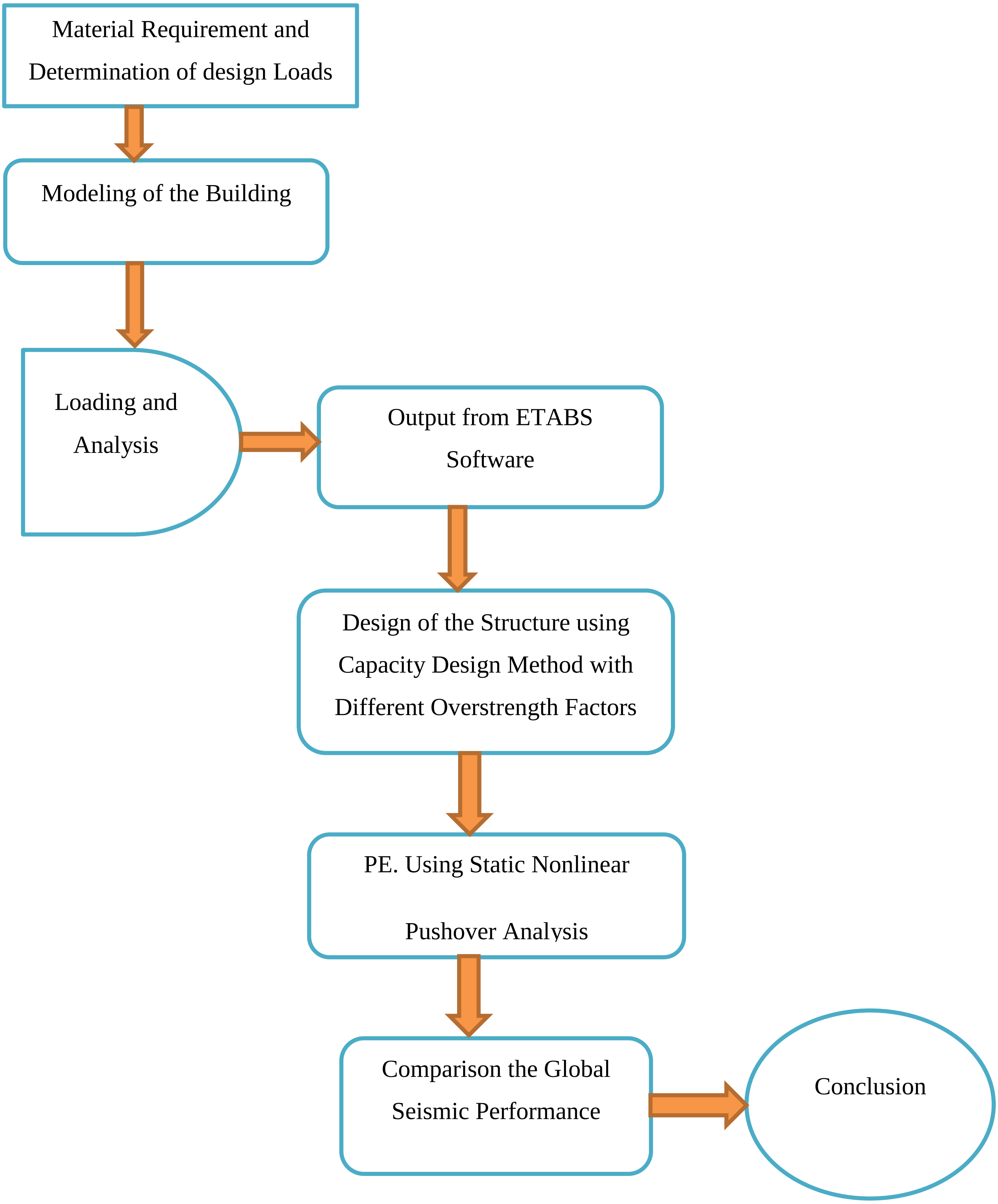

Figure 3.9 Methodology of the Thesis

3.4 Capacity Design Procedure Example in the Ductile Frame

3.4.1 General Description

Ten story 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 code ([EBCS EN 1998-1], 2014). Whenever necessary reference was made from previously discussed equations, tables and figures.

The floor plan and the framing system were shown in the above Figure A.1; A.2 and A.2 in the appendix part. The design of one selected beam and column from the selected framing system was described in detail.

3.4.2 Material Properties

Material used in the design of the structural elements

Table 3.6 Material Property

Concrete strength	γ_c	α_c	f_{ck} (MPa)	$f_{cd}=\alpha_c f_{ck}/\gamma_c$ (MPa)	f_{ctm} (MPa)	E_c (GPa)
Beam-C25/30	1.5	0.85	25	14.17	2.6	31
Column-25/30	1.5	0.85	25	14.17	2.6	31
Strength of rebar	γ_s	f_{yk}(MPa)	$f_{yd}=f_{yk}/\gamma_s$(MPa)	E_s(GPa)		
Beam and Column	1.15	420	365.22	200		

3.4.3 Geometry of the Structure

The geometry of the structure used in this research was ten 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 from 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.

3.4.4 Loading and Design Forces Determination

3.4.4.1 Gravity Load

Slab Self-Weight Determination

Characteristic strength of normal weight concrete used in this research is γ=24 kN/m^3. Size of the slab, $L_X \times L_Y$=5m×5.5m

Number of slabs per floor, N_{slab},=12

Thickness of the slab,t,=150mm=0.15m

$DL_S = L_X \times L_Y \times t \times \gamma$=5m×5.5m×0.15m×24kN/m^3=<u>99kN</u> or

DL_s=<u>3.6kN/m^2</u>, dead load per unit area.

Finishing materials

Cement screed, (t=3cm) =0.03m×23kN/m^3=0.69kN/m^2
Marble floor finish, (t=2cm) =0.02m×27kN/m^3=0.54kN/m^2
Bottom plastering, (t=1.5cm) =0.015m×23 kN/m^3=0.35kN/m^2

DL_f=<u>1.58kN/m^2</u>

Total dead load on the slab, $DL=DL_S+DL_f$= (3.6+1.58)kN/m^2=<u>5.18kN/m^2</u>

Live Load

The live load of the structure was used based on the function of the building (Ethiopian Building Code Standard-1 [EBCS-1], 2014)

Live load from story 1-5, LL=<u>3.5kN/m^2</u>
Live load from story 6-9, LL=<u>3kN/m^2</u>
Combination coefficient, ψ_{Ei}, calculated from the following expression:

$$\Psi_{Ei} = \varphi.\Psi_{2i} \tag{3.43}$$

Where φ: value from Table C.2
φ=0.5 used for our case
ψ_2=0.6 from Table C.3

Ψ_E=0.5×0.6=0.3

3.4.4.2 Earthquake Load

i) Seismic Weight Determination

Weight of slab,

DL_S=12×5.5m×5m×0.15mx24kN/m^3=1188kN

Weight of the beam (H/b=300×250mm),

DL_b= (16×5m+15×5.5m) ×0.3m×0.25m×24kN/m^3=292.5kN

Weight of column, (H/b=300×300mm),

DL_c=20×3m×0.3m×0.3m×24kN/m^3=129.6kN

Live Load,

DL_L=12×5.5m×5m×3kN/m^2=990kN

Total DL+0.3LL=1545.3kN+297kN

Similar procedure was used for the remaining floors.

M_T=33,223.2kN

ii) Designed Seismic Load

In this research lateral force method was used for the seismic analysis of the structure by checking all the requirements to use this method according to ([EBCS EN 1998-1], 2014).

$$F_b = S_d(T_1).m.\lambda \qquad (3.3)$$

First determine the fundamental period of the structure

$$T_1 = C_t H^{3/4} \qquad (3.2)$$

C_t=0.075 for moment resistant space concrete frames

H=30m < 40m--------Satisfied.

T_1=0.075× $(30m)^{3/4}$= 0.96sec.

Check the two requirements to use the lateral force method

(a) T_1=0.96sec $\leq \begin{cases} 4.T_c=4\times0.4\text{sec.}=1.6\text{sec} \\ 2.0\text{sec} \end{cases}$

T_c from the Table C.1

(b) The structure was regular in elevation

The two requirements are satisfied.

The structure was ordinary building and the importance factor becomes γ_I =1.0

Ground type A was used in this research

Designed ground acceleration, a_{gR}/g=0.15

$a_g/g = \gamma_I a_{gR}$=1×0.15=0.15

a_g=0.15×9.81m/s^2=1.47m/s^2

The fundamental period, T_1 was between T_c=0.4sec. and T_d=2.0sec.

Ordinate of the design spectrum was calculated as follows:

$$S_d(T_1) = a_g.S.\frac{2.5}{q}\left[\frac{T_c}{T_1}\right] \geq \beta.a_g \qquad (3.4)$$

The recommended value of β=0.2

Behavior factor,q,=$q_o k_w \geq 1.5$

q_o= 3×α_u/α_1=3×1.3=3.9; k_w=1 for frame system

q=3.9×1=<u>3.9</u>

S=1 for ground type A Table C.1

$S_d(T_1)$=1.47×1×2.5/3.9×0.4/0.96=<u>0.393m/s^2</u> ≥ 0.2×1.47=<u>0.294 m/s^2</u>

F_b=0.393m/s^2×33,223.2kN/9.81m/s^2=<u>1330.96kN</u>

iii) Distribution of Horizontal Seismic Forces

Distribution of the total horizontal seismic load to different floor levels was obtained as follows.

$$F_i = F_b \frac{Z_i m_i}{\sum_{j=1}^{10} Z_j m_j} \tag{3.5}$$

Story 10

$$F_{10} = 1330.96 \times \frac{30 \times 1545.3}{400{,}454.14} = 154.08\text{kN}$$

Similar calculation procedure was used for the remaining storys and summarize in the following Table 3.7.

Table 3.7 Distribution of Total Horizontal Load to Different Story Levels

F_b (kN)	Story	Z_i (m)	F_i (kN)
1330.96	1	3	28.29
	2	6	56.10
	3	9	83.45
	4	12	110.37
	5	15	134.31
	6	18	157.61
	7	21	181.12
	8	24	201.69
	9	27	223.99
	10	30	154.08

3.4.4.3 Imperfection Load

The imperfection load was obtained using the following expression.

$$\theta_i = \theta_o \alpha_h \alpha_m \tag{3.8}$$

θ_o=1/200, recommended basic value

Hight reduction factor, $\alpha_h = \frac{2}{\sqrt{L}} = \frac{2}{\sqrt{30}} = 0.365 \leq \frac{2}{3} = 0.667$

$\alpha_h = \underline{0.667}$

Number of member's reduction factor, $\alpha_m = \sqrt{0.5(1+\frac{1}{m})}$

m=5 in the x direction and m=4 in the y direction

$$\alpha_{mx} = \sqrt{0.5(1+\frac{1}{5})} = 0.775$$

$$\alpha_{my} = \sqrt{0.5(1+\frac{1}{4})} = 0.791$$

$$\theta_{ix} = \theta_o \alpha_h \alpha_m = \frac{1}{200} \times 0.667 \times 0.775 = 2.58 \times 10^{-3}$$

$$\theta_{iy} = \theta_o \alpha_h \alpha_m = \frac{1}{200} \times 0.667 \times 0.791 = 2.64 \times 10^{-3}$$

For unbraced members

$$H_i = \theta_i N \tag{3.9}$$

N is the axial load at the story required

From analysis result without imperfection load was used for axial load calculation. The determination was summarized in the table below.

Table 3.8 Story Axial Load, N

			N_{ij} (kN)			
	X-Direction	Axis/Story	1	2	3	4
θ_i	0.00258	1	8711.4	13571.1	13571.1	8711.4
		2	7732.2	12076.9	12076.9	7732.2
		3	6732.5	10623.7	10623.7	6732.5
		4	5729.3	9196.3	9196.3	5729.3
		5	4718.9	7796.5	7796.5	4718.9
		6	3786.9	6394.6	6394.6	3786.9
		7	2684.5	5224.0	5224.0	2684.5
		8	2014.3	3642.3	3642.3	2014.3
		9	1235.6	2236.1	2236.1	1235.6
		10	419.7	885.4	885.4	419.7

Table 3.9 Horizontal Imperfection Load in the X Direction

	H_{ijX} (kN)			
Axis/story	1	2	3	4
1	22.5	35.0	35.0	22.5
2	19.9	31.2	31.2	19.9
3	17.4	27.4	27.4	17.4
4	14.8	23.7	23.7	14.8
5	12.2	20.1	20.1	12.2
6	9.8	16.5	16.5	9.8
7	6.9	13.5	13.5	6.9
8	5.2	9.4	9.4	5.2
9	3.2	5.8	5.8	3.2
10	1.1	2.3	2.3	1.1

Horizontal imperfection load in the y direction obtained using similar procedure with X direction.

3.4.4.4 Design Forces Determination

For this example second story beam (B8) on axis 3 between axis A and B shown in the figure 3-12 and second story column on axis 3-A was designed in detail. Maximum design force was obtained from maximum load combination as follows.

Comb 8-16= DL+0.3LL-EQ$_{X2}$-0.3EQ$_{Y1}$-Imp$_x$

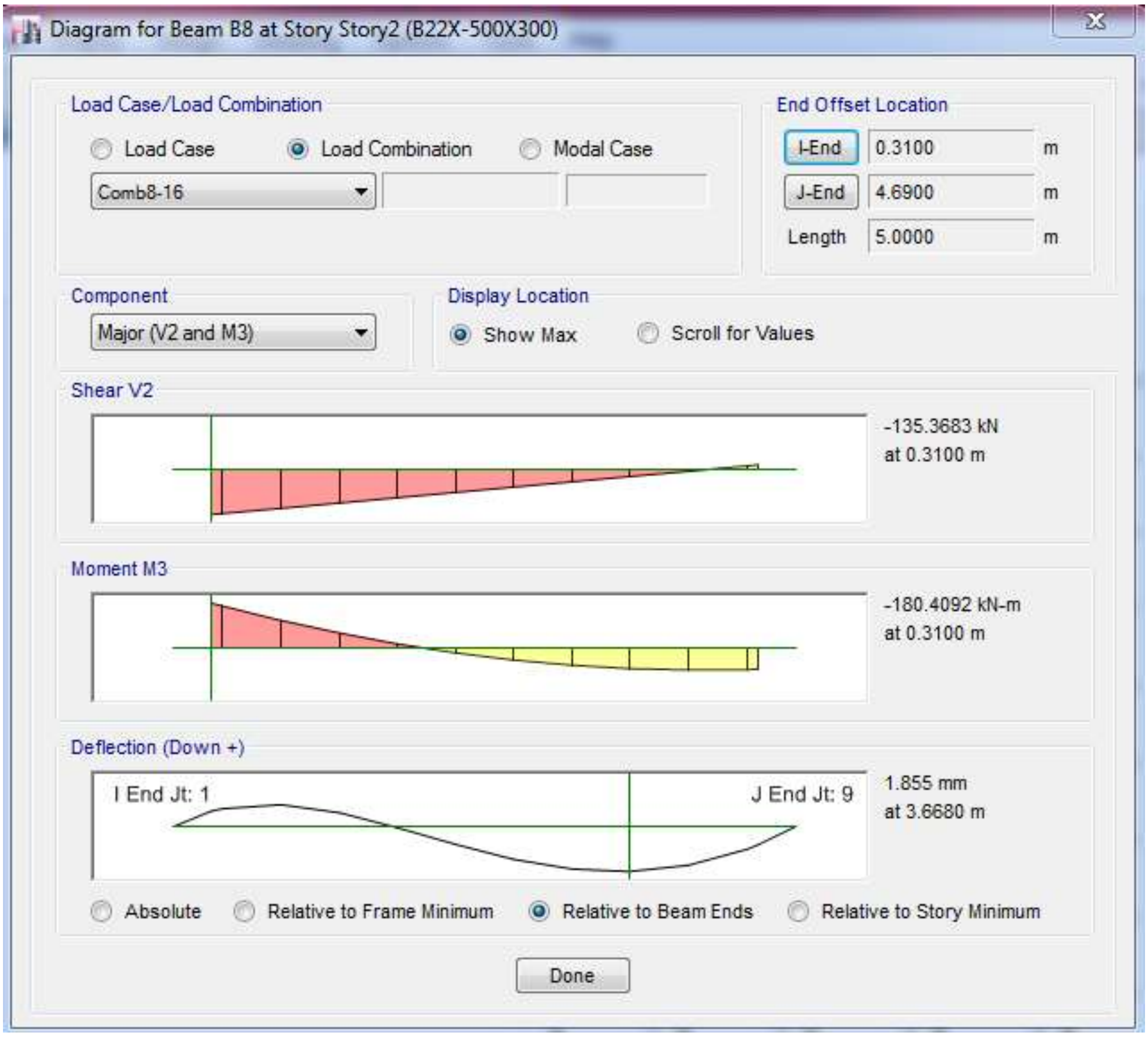

Figure 3.10 Design Moment Diagram for Beam Element

3.4.5 Design of Structural Elements

3.4.5.1 Flexural Design of Beams

Beam size, H/b=500×300mm

Use $Ø_L$=16mm rebar for longitudinal reinforcement

Use $Ø_s$=8mm rebar for hoops or transverse reinforcement

Concrete cover used=25mm

b_c=600mm

(a) Chick the Geometric Constraint

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

b_w=300mm ≤ min {600+500mm; 2×600}≤1100mm------- Satisfied.

(b) Design Moment Determination

M_{se}=180.41kNm

Moment redistribution

10 % moment redistribution was used in this research.

δ=90%=0.9

Design moment after redistribution becomes

M_{sd}=0.9×180.41kNm=<u>162.37kNm</u>

(c) Deign of Flexural Reinforcement

The design of beam done using design table

<u>Step 1</u>

Calculate μ_{sd}

$$\mu_{sd} = \frac{M_{sd}}{f_{cd} b d^2} \tag{3.12}$$

b=300mm

d=H−cover−$ø_s$−$ø_L$/2=500−25−8−16/2=459mm

f_{cd}=14.17MPa

f_{yd}=365.21MPa

M_{sd}=162.37kNm

$$\mu_{sd} = \frac{M_{sd}}{f_{cd}bd^2} = \frac{162.37 \times 10^6}{14.17 \times 300 \times 459^2} = 0.181 \leq \mu_{sd,lim} = 0.252$$

The section is singly reinforced section.

<u>Step 2</u>

Read ω from design Table B.1 corresponding to μ_{sd}

ω=0.202

Tension reinforcement at the critical end of the beam

$$A_{s1} = \frac{1}{f_{yd}}(\omega.b.d.f_{cd}) = \frac{1}{365.21}(0.202 \times 300 \times 459 \times 14.17) = 1079.22mm^2$$

Number of bars, $n_b = \frac{A_{s1}}{a_s} = \frac{1079.22}{201} = 5.37$

Use 6ø16 longitudinal reinforcement

$A_{s,provided}$=6×201=1206mm^2

$$\rho = \frac{A_{s,pro}}{bd} = \frac{1206}{300 \times 459} = 8.76 \times 10^{-3}$$

Compression reinforcement

A_{s2}=0.5A_{s1}, recommended for ductility class medium structures

A_{s2}=0.5×1206=603mm^2

$$\rho' = \frac{A_{s,pro}}{bd} = \frac{603}{300 \times 459} = 4.38 \times 10^{-3}$$

<u>Step 3</u>

Check minimum and maximum reinforcement using Table 3-5

$\rho_{min,tension} = 0.5\frac{f_{ctm}}{f_{yk}} = 0.5\frac{2.6}{420} = 3.09 \times 10^{-3}$…….. Satisfied.

$$\rho_{\mathrm{Max,cr}} = \rho' + \frac{0.0018}{\mu_{\varphi}\varepsilon_{\mathrm{sy,d}}} . \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd}}}$$

$T_1 > T_c$ calculate $\mu_{\varphi} = 2q_o - 1 = 2 \times 3.9 - 1 = 6.8$

The reinforcement used in this research was class B, because of this

$\mu_{\varphi f} = 1.5 \times \mu_{\varphi} = 1.5 \times 6.8 = 10.2$

$$\varepsilon_{\mathrm{sy,d}} = \frac{f_{\mathrm{yd}}}{E} = \frac{365.22 \times 10^6}{200 \times 10^9} = 1.83 \times 10^{-3}$$

$\rho_{\mathrm{Max,cr}} = 4.38 \times 10^{-3} + \frac{0.0018}{10.2 \times 1.83 \times 10^{-3}} . \frac{14.17}{365.22} = 8.12 \times 10^{-3}$

$\rho = 8.76 \times 10^{-3} > \rho_{Max,cr} = 8.12 \times 10^{-3}$………Not satisfied.

Revise the Design

Increasing $\rho' = 0.6\rho = 0.6 \times 8.76 \times 10^{-3} = 5.26 \times 10^{-3}$, check $\rho_{\mathrm{Max,cr}}$

$\rho_{\mathrm{Max,cr}} = 5.26 \times 10^{-3} + \frac{0.0018}{10.2 \times 1.83 \times 10^{-3}} . \frac{14.17}{365.22} = 9 \times 10^{-3} > \rho = 8.76 \times 10^{-3}$

Use for tension side (top end) 6ø16; $A_{s,provided}$=6×201=<u>1206mm²</u>

Use for compression side (bottom end) 4ø16; $A_{s,provided}$=4×201=<u>804mm²</u>

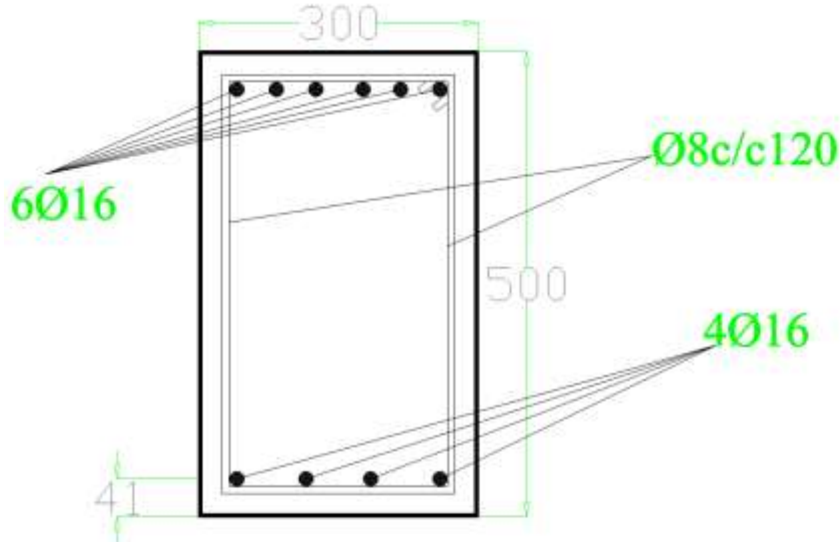

Figure 3.11 Detailing of Beam Critical Section

3.4.5.2 Flexural Design of Column

Column size, H/b=600×600mm

Use $Ø_L$=20mm rebar for longitudinal reinforcement

Use $Ø_s$=10mm rebar for hoops or transverse reinforcement

Concrete cover used=25mm

a) Chick the Geometric Constraint

Check the interstorey drift sensitivity coefficient, θ≤0.1.

Where $\theta = \frac{P_{tot} d_r}{V_{tot} h} \leq 0.1$

h is the interstorey height.

d_r is the design interstorey drift, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration.

P_{tot} is the total gravity load at and above the storey considered in the seismic design situation; and

V_{tot} is the total seismic storey shear

From the above definition and from ETABS output for DCM and $\gamma_{Rd} = 1.3$

h=3000mm

Check the the interstorey drift sensitivity coefficient in the tabular form below.

From the Table 3.10 the drift sensitive coefficient is less than 0.1, therefore the size of column is dependent on the incoming action effect on the column. The interstory drift coeffient calculated below is the X direction of the selected frame ductility class medium with $\gamma_{Rd} = 1.3$.

Table 3.10 Interstorey Drift Sensitivity Coefficient

Story	P_{tot} (kN)	d_T (mm)	d_r	V_{tot} (kN)	h (mm)	ϴ
10	2610.27	85.27	3.1	158.09	3000	0.017
9	6943.49	82.17	7.92	392.71	3000	0.047
8	11313.21	74.25	10.03	611.70	3000	0.062
7	15817.08	64.22	11.48	816.84	3000	0.074
6	20362.93	52.74	11.39	1005.09	3000	0.077
5	25030.74	41.35	10.8	1177.10	3000	0.077
4	29851.14	30.55	9.47	1332.49	3000	0.071
3	34712.39	21.08	9.1	1468.13	3000	0.072
2	39618.26	11.98	8.05	1583.65	3000	0.067
1	44565.00	3.93	3.93	1675.80	3000	0.035
Base		0				

b) Design Moment Determination

Moment resistance of the beam in the X direction due to seismic action in the X direction

$$\sum M_{Rbx} = 6 \times 201 \times 365.21 \times (500 - 82) \times 10^{-6} = 184.11\text{kNm}$$

For γ_{Rd}=1.3

M_{Rcx}=1.3×184.11kNm=239.34kN, at the top and bottom of the column center because the beam design provides almost similar moment resistance at the bottom and at the top of the column.

Moment resistance in the Y direction due to seismic in the X direction

$$\sum M_{Rby}=122.74\text{kNm}$$

For γ_{Rd}=1.3

M_{Rcy}=1.3× 122.74kNm=159.56kN, at the top and bottom of the column center because the beam design provides almost similar moment resistance at the bottom and at the top of the column.

For maximum compression in the column

$P_{d,max}$=DL+0.3LL+$\gamma_{Rd}P_{EQ}$+I_{mpx}

=1633.4kN+0.3×454.63kN+1.3×286.6kN+35.5kN=2177.87kN

Determine the equivalent moment in each direction

In the X direction

e_{o1}= e_{o2}=239.34kNm÷2177.87kN=0.11m

The equivalent eccentricity is selected from the maximum of the two values below.

$e_e=0.6e_{o2}+0.4e_{o1}$ or $e_e=0.4e_{o2}$ and e_{o1} is negative because the column bents in double curvature.

e_e=0.6×0.11−0.4×0.11=0.022m or e_e=0.4×0.11=0.044m

e_{ex}=0.044m

M_{sdx}=2177.87kN×0.044m=95.83kNm

Similarly procedure was used in the Y direction

M_{sdy}=63.59kNm

Compute

Axial load ratio, $\upsilon_{sd} = \frac{N_{sd}}{A_c f_{cd}} = \frac{2177.87\times10^3}{600\times600\times14.17} = 0.427$

Moment ratio, $\mu_{sdx} = \frac{M_{sdx}}{A_c h f_{cd}} = \frac{95.83\times10^6}{600^3\times14.17} = 0.031$

$$\mu_{sdy} = \frac{M_{sdy}}{A_c h f_{cd}} = \frac{63.59\times10^6}{600^3\times14.17} = 0.0208$$

Using the above value

$(\upsilon_{sd}, \mu_{sdx}, \mu_{sdy})$ = (0.427, 0.031, 0.0208)

Read ω from Biaxial Chart C12/15-C50/60

h'/h=b'/b=0.1 and $A_{s,tot}$=4A

ω<0.0

Provide minimum reinforcement

$\rho_{min}=0.01$

$A_{s,min}=0.01bd=0.01\times600\times555=3330mm^2$

$n_b=3330mm^2\div314mm^2=10.6$, use 12ø20 rebars;

4ø16, rebar's in each direction.

Check maxiumum reinforcement

$A_{smax}=0.04\times600\times559=13416mm^2\geq A_{s,provi}=12\times314=3768mm^2$… Satisfied.

For ductility medium structures the spacing between restrained bars ≤ 200mm

Calculate the spacing of rebars

$b=2\times cover+(n_b-1)S+n_b x ø_R+2ø_s$

$600=2\times25+(4-1)S+4\times20+2\times10$

S=150mm≤200mm……….Satisfied.

3.4.5.3 Shear Design of Beam

The design of shear strength follows the procedure described in the section 3.3.4.1.

Determination of design action effects

Calculate the the overstrrength end moment resistance of the beam.

$$M_{i,d}=\gamma_{Rd}M_{Rb,i}\min\left[1,\frac{\sum M_{Rc}}{\sum M_{Rb}}\right] \tag{3.10}$$

$$\frac{\sum M_{Rc}}{\sum M_{Rb}} = \frac{3768 \times 365.21 \times (600-90)}{184.11} = 3.81 > 1$$

$$M_{1,d} = \gamma_{Rd} M_{Rb,1} = 1.3 \times 184.1\text{kNm} = 239.33\text{kNm}$$

$$M_{2,d} = \gamma_{Rd} M_{Rb,2} = 1.3 \times 85.13\text{kNm} = 110.67\text{kNm}$$

1 and 2 indicates left and right ends of the beam respectively.

V_g=76.66kN

l_{cl}=5m

$$V_{Ed} = V_g + \gamma_{Rd} \frac{\sum M_{i,d}}{l_{cl}} = 76.66kN + \frac{1.3 \times (184.11+85.13)kNm}{5m}$$

$$V_{Ed} = 146.66\text{kN}$$

Shear Resistance Determination

The design resistance of the critical section for shear effect is obtained from the minimum of the following two expressions.

(a) $V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} \text{Cot}\theta$

From Table 3-5, in critical region of the beam obtained as

$S_w \leq (h_w/4;\ 24d_{bw};\ 225;\ 8d_{bl}) \leq (500\text{mm} \div 4;\ 24 \times 8\text{mm};\ 225;\ 8 \times 16)$

S_w=125mm, Use ø8 c/c 120mm

$A_{sw} = \pi d^2/4 = \pi \times 8 \times 8/4 = 50.3\text{mm}^2$

z=0.9d=0.9×459=413.1mm

$1 \leq \text{Cot}\theta \leq 2.5$, $\theta = 34^o$, used in this research

$$V_{Rd,s} = \frac{2\times50.3}{120} \times 413.1 \times 365.22 \times \mathrm{Cot}34^{o} = 187.51kN$$

(b) $V_{Rd,max} = \alpha_{cw} b_w z v_1 {}^{f_{cd}}/_{(\mathrm{Cot}\theta + \tan\theta)}$

α_{cw}=1, for non prestressed structures

$$v_1 = 0.6 \times \left[1 - \frac{f_{ck}}{250}\right] = 0.6 \times \left[1 - \frac{25}{250}\right] = 0.54$$

$$V_{Rd,max} = 1 \times 300 \times 413.1 \times 0.54 \times \frac{14.17}{(Cot34^{o} + tan34^{o})} = 439.62kN$$

V_{Rd}=min ($V_{Rd,S}$; $V_{Rd,max}$)=187.51kN > V_{Ed}=108.0.8kN……Satisfied.

<u>Use ø8 c/c 120mm</u>

3.4.5.4 Shear Design of Column

The design of shear strength follows the procedure similar to the beam shear strength design.

Determination of Design Action Effects

Calculate the the overstrrength end moment resistance of the column.

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} \min\left[1, \frac{\sum M_{Rb}}{\sum M_{Rc}}\right] \quad (3.28)$$

Top end: $\frac{\sum M_{Rb}}{\sum M_{Rc}} = \frac{184.11}{701.82} = 0.262 < 1;$

Bottom end: $\frac{\sum M_{Rb}}{\sum M_{Rc}} = \frac{184.11}{701.82} = 0.262 < 1$

$$M_{1,d} = \gamma_{Rd} M_{Rc,1} = 1.3 \times 701.82\mathrm{kNm} \times 0.262 = 239.04\mathrm{kNm}$$

$$M_{2,d} = \gamma_{Rd} M_{Rc,2} = 1.3 \times 701.82\mathrm{kNm} \times 0.262 = 239.04\mathrm{kNm}$$

1 and 2 indicates top and bottom ends of the column respectively.

$$V_{Ed} = \gamma_{Rd}\frac{\sum M_{i,d}}{l_{cl}} = \frac{1.3 \times (701.82 \times 0.262 + 701.82 \times 0.262)kNm}{3m}$$

$$V_{Ed} = 159.36kN$$

Shear Resistance Determination

The design resistance of the section for shear effect was obtained from the minimum of the following two expressions.

(a) $V_{Rd,s} = \frac{A_{sw}}{S} z f_{ywd} Cot\theta$ (3.24)

From Table 3-6, in critical region of the beam obtained as

$S_w \leq (b_o/2;\ 175;\ 8d_{bl})$

$b_o = b_c - (2\times cover + \varnothing_s) = 600 - (2\times25+10) = 540mm$

$S_w \leq (b_o/2;\ 175;\ 8d_{bl}) \leq (540mm \div 2;\ 175;\ 8\times20)$

S_w=120mm, Use ø10 c/c 120mm

$A_{sw} = \pi d^2/4 = \pi\times10\times10/4 = 78.54mm^2$

$z = 0.9d = 0.9\times555 = 499.5mm$

$1 \leq Cot\theta \leq 2.5$, $\theta = 34^o$, used in this research

$$V_{Rd,s} = \frac{2 \times 78.54}{120} \times 499.5 \times 365.22 \times Cot34^o = 354.03kN$$

(b) $V_{Rd,max} = \alpha_{cw} b_w z v_1 \, {f_{cd}}/{(Cot\theta + tan\theta)}$ (3.25)

α_{cw}=1, for non prestressed structures

$$V_1 = 0.6 \times \left[1 - \frac{f_{ck}}{250}\right] = 0.6 \times \left[1 - \frac{25}{250}\right] = 0.54$$

$$V_{Rd,max} = 1 \times 600 \times 499.5 \times 0.54 \times \frac{14.17}{(Cot34^o + tan34^o)} = 1063.13kN$$

V_{Rd}=min ($V_{Rd,S}$; $V_{Rd,max}$)=354.03kN > V_{Ed}=159.36kN……Satisfied.

Use ø10 c/c 120mm

(c) Check the mechanical volumetric ratio of the designed column above

$$\omega_{wd} = \frac{\text{Volume of confining hoops}}{\text{Volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}}$$

ω_{wd} is the mechanical volumetric ratio of confining hoops within the critical region

Length of critical region,

L_{cr}= Max (h_c;l_{cl}/6;450mm) for Ductility class medium structure

L_{cr}= Max (600mm; 333.3mm; 450mm) =600mm

Number of hoops within the critical regions (N_h),

N_h=600÷120=5

Length of each hoops=4 × (600−50) = 2200mm

Area of each hoops=78.54mm^2

Volume of confining hoops, V_h, = 4× 78.54 ×2200=691,152mm^3

Volume of concrete core, V_c, = b_o×h_o×L_{cr}=540mm×540mm×600mm=174,960,000mm^3

$$\omega_{wd} = \frac{\text{Volume of confining hoops}}{\text{Volume of concrete core}} \cdot \frac{f_{yd}}{f_{cd}} = \frac{691{,}152}{174{,}960{,}000} \cdot \frac{365.2}{14.17} = 0.102$$

$$\geq 0.08 \ldots\ldots \text{Satisified.}$$

Check

$$\alpha\omega_{wd} \geq 30\mu_{\varphi}.v_d.\varepsilon_{sy,d}.\frac{b_c}{b_o} - 0.035$$

μ_{φ}=10.2… ……....calculated previously.

υ_d=0.427………...calculated previously.

$\varepsilon_{sy,d}$=1.83x10^{-3}…..calculated previously.

ω_{wd}=0.102

$\alpha=\alpha_n.\alpha_s$

Where

$$\alpha_n = 1 - \frac{\sum_n b_i^2}{6\times b_o h_o} = 1 - \frac{4\times 170^2}{6\times 540\times 540} = 0.934$$

$$\alpha_s = \left(1 - \frac{s}{2b_o}\right)\left(1 - \frac{s}{2h_o}\right) = \left(1 - \frac{120}{2\times 540}\right)^2 = 0.79$$

$\alpha=\alpha_n.\alpha_s$=0.934×0.79=0.738

$\alpha\omega_{wd} = 0.738 \times 0.102 = 0.075$

$30\mu_{\varphi}.v_d.\varepsilon_{sy,d}.\frac{b_c}{b_o} - 0.035 = 30 \times 10.2 \times 0.427 \times 1.83 \times 10^{-3} \times \frac{600}{540} - 0.035$

$$= 0.23 \geq \alpha\omega_{wd} = 0.075 \text{ ... Not satisified.}$$

Revise the detailing to increase the volume of confining hoops

Change the diameter of hoops to 10mm and add the hoops configuration

Number of hoops within the critical regions (N_h),

N_h=600÷120=5

From figure 3.13 hoop configuration determine the volume of each hoops in the critical region as follows.

i) Exterior Hoops Volume

Length of each hoops=4 × (600−50) = 2200mm

Area of each hoops=78.54mm^2

Volume of confining hoops, V_h, = 5×78.54×2200=863,940mm^3

ii) Interior Hoops Volume

Length of each hoops=2 × (530+190) = 1440mm

Length of each diagonal hoops =4 × (228+207)=1740mm

Area of each hoops=78.54mm^2

Volume of confining hoops, V_h, = 5×78.54× (2×1440+1740) =1,814,274mm^3

Total volume of confining hoops=2,678,214mm^3

Volume of concrete core, V_c, = b_o×h_o×L_{cr}=540mm×540mm×600mm=174,960,000mm^3

$$\omega_{wd} = \frac{\text{Volume of confining hoops}}{\text{Volume of concrete core}} . \frac{f_{yd}}{f_{cd}}$$

$$= \frac{2,678,214}{174,960,000} . \frac{365.2}{14.17}$$

$$= 0.395 \ \geq 0.08 \ldots \text{Satisified.}$$

α=α_n.α_s

Where $$\alpha_n = 1 - \frac{\sum_n b_i^2}{6 \times b_o h_o}$$

$$= 1 - \frac{4 \times 170^2}{6 \times 540 \times 540} = 0.934$$

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

$$= \left(1 - \frac{120}{2\times540}\right)^2 = 0.79$$

$\alpha=\alpha_n.\alpha_s=0.934\times0.79=0.74$

$\alpha\omega_{wd} = 0.74 \times 0.395 = 0.292 \geq 0.23$………The requirement is satisfied.

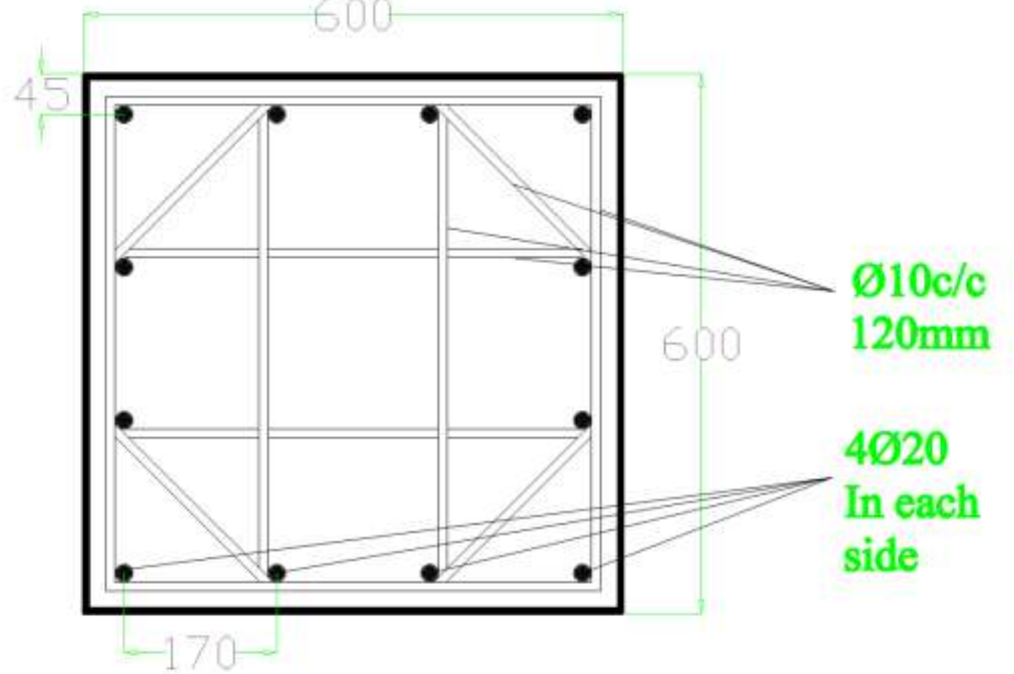

Figure 3.12 Detailing of Column Section

3.4.5.5 Beam-Column Joint Design

The beam-column joint design was satisfied when the joining column and beams are satisfied all the requirements provided in the code. In this thesis all the requirements need for column and beam sections were satisfied including the volumetric ratio of the column and one intermediate bar was found between the column corner bars.

CHAPTER 4 RESULT AND DISCUSSION

4.1 Analysis of the Building

Analysis of the selected ten-story building in this thesis was done by defining the necessary input material, loads, frame sections and load combinations to get maximum action effects on the structure for the design of structural elements. The analysis of regular frame reinforced concrete structure was done for four different column-beam overstrength factors using ETABS 2016.1.0 computer software. Loading, material property and earthquake data's were similar for all building models. The material used for the analysis and the determination of imperfection load used for this thesis was described in the previous methodology part.

Table 4.1 Loading Data for Analysis

Dead load		
Frame elements		From the cross section size
Slab thickness (mm)		150
Live Load (kN/m^2)		
Story	1-5	3.5
	6-9	3
Earthquake data		
Ground acceleration (a_g/g)		0.15
Spectrum Type		1
Ground Type		A

The type of analysis method used for this thesis was lateral force method of analysis. All the requirements need to use this method was fulfilled as mentioned in the previous chapter according to the new Ethiopian building code ([EBCS EN 1998-1], 2014).

After the analysis was finished the design of the ductile frame elements was done using the incoming maximum action effect and discussed in the following section.

4.2 Design of the Building

The design of ten story regular frame reinforced concrete elements was done using capacity design philosophy. The basic steps of the design of the building are summarized in the following flow chart.

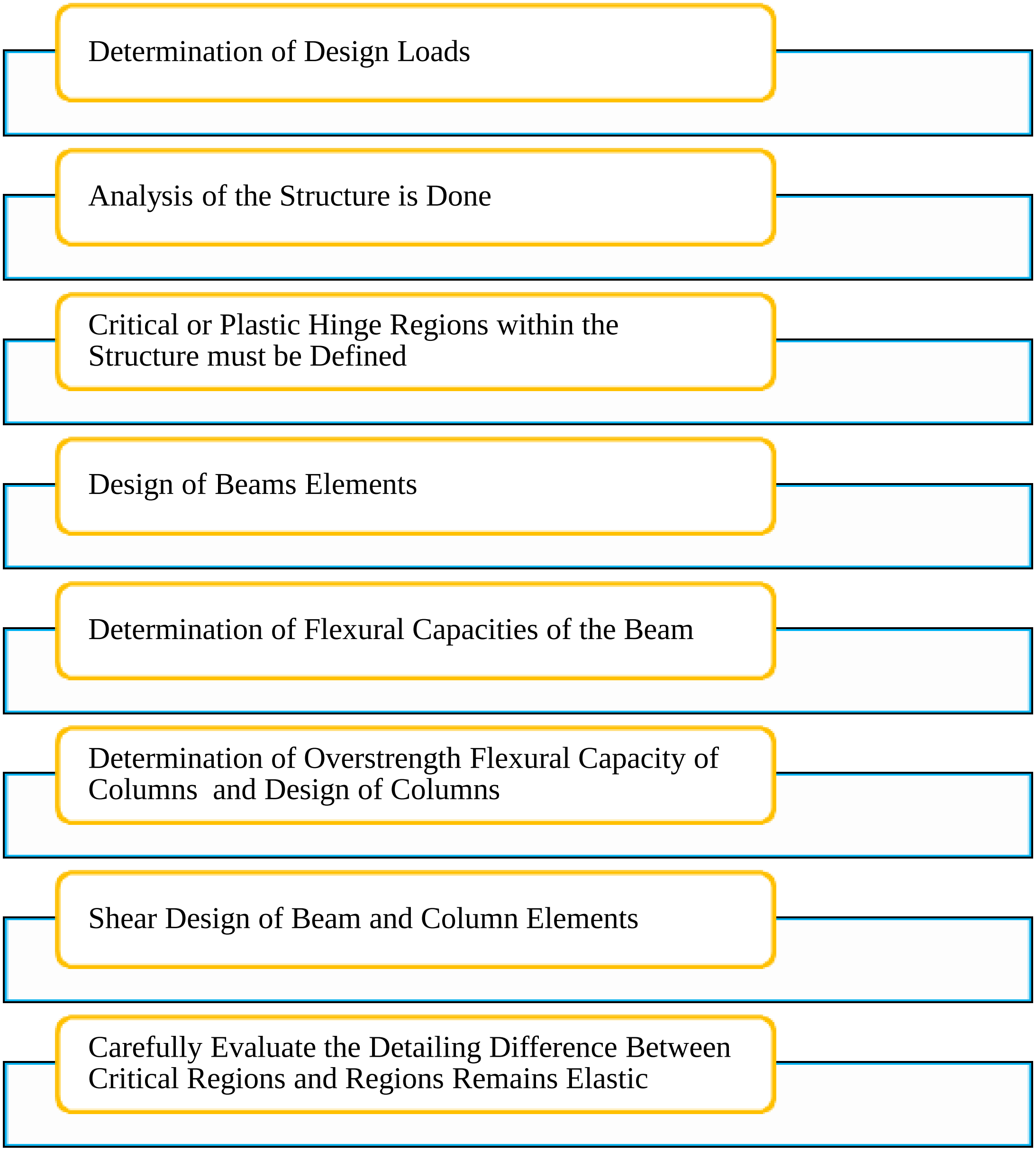

Figure 4.1 Basic Design Steps

.

Table 4.2 Beam Design Output Data for DCM and $\gamma_{Rd} = 1.15$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1206	1206	1206	1005	804	804	603	603	603
		Bottom	603	804	804	804	804	603	603	603	603	603
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1407	1407	1407	1407	1206	804	603	603
		Bottom	804	1005	1206	1206	1407	1407	1206	603	603	402
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	603
		Bottom	603	1005	1005	804	804	804	804	804	804	402
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	1005	1206	1206	1206	1005	1005	804	804	804	603
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1005	804	804	603
		Bottom	1005	1206	1206	1206	1407	1407	1005	804	804	603

Sample design output for ductility class medium structure and for $\gamma_{\mathrm{Rd}} = 1.15$ overstrength factor was shown in the above table and design output for other buildings models are described in the Appendix A.2.

From the above table H is depth of the beam (mm), B is width of the beam (mm) and A_{sLc} is the total top and bottom flexural longitudinal reinforcement at the two critical regions of the beam ends. Use ϕ8 c/c 120 hoops in the critical regions of the beam and ϕ8 c/c 200 out of the critical beam section.

Table 4.3 Column Design Output Data for DCM and $\boldsymbol{\gamma_{\mathrm{Rd}} = 1.15}$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	550	550	500	500	500	400	350	350	300	300
	B (mm)	550	550	500	500	500	400	350	350	300	300
	A_{sLT} (mm^2)	3768	3768	2512	2512	2512	1608	1608	1356	1356	904
2&3	H (mm)	650	600	600	550	500	500	400	350	300	300
	B (mm)	650	600	600	550	500	500	400	350	300	300
	A_{sLT} (mm^2)	5024	3768	3768	3768	2512	2512	1608	1356	1356	904

From the above Table H is depth of the column (mm), B is width of the column (mm) and A_{sLT} is the total flexural longitudinal reinforcement at the column and distribute equally for each side of the column. Use ϕ10 c/c 200 shear reinforcement or hoops for each column for non-critical regions and three types hoops used at the critical regions using ϕ10 c/c 120 as shown in the Figure 3.13.

4.3 Global Seismic Performance of Ductile Frames

Global seismic performance of the newly designed regular framed reinforced concrete structure based on capacity design method was done using static nonlinear pushover analysis. The performance of the building for each ductility class and each overstrength factor was discussed in the following sections in terms of four structural performance parameters known as capacity curve, story displacement, interstorey drift and plastic hinge distribution.

4.3.1 Seismic Performance Evaluation of Ductility Class Medium Buildings

This section presents the global seismic performance of ductility class medium regular framed reinforced concrete structure designed in the previous section. The global seismic performance of the building was discussed for four different overstrength factors. As mentioned earlier the analysis was done using nonlinear static analysis (Pushover). The results obtained from pushover analysis were described graphically in both principal X and Y directions in the following sections.

4.3.1.1 Capacity-Curve of the Building

The first parameter to describe the performance of the structure was capacity curve of the building. Capacity curve is represented in terms of top displacement and base shear of the building shown in the Figure 4.1 for four different overstrength factors. The pushover cure is drawn until the top displacement corresponding to the available capacity of the building at the near collapse limit state. The capacity curve shows as the capacity progress of the structure due to the applied loads on the building.

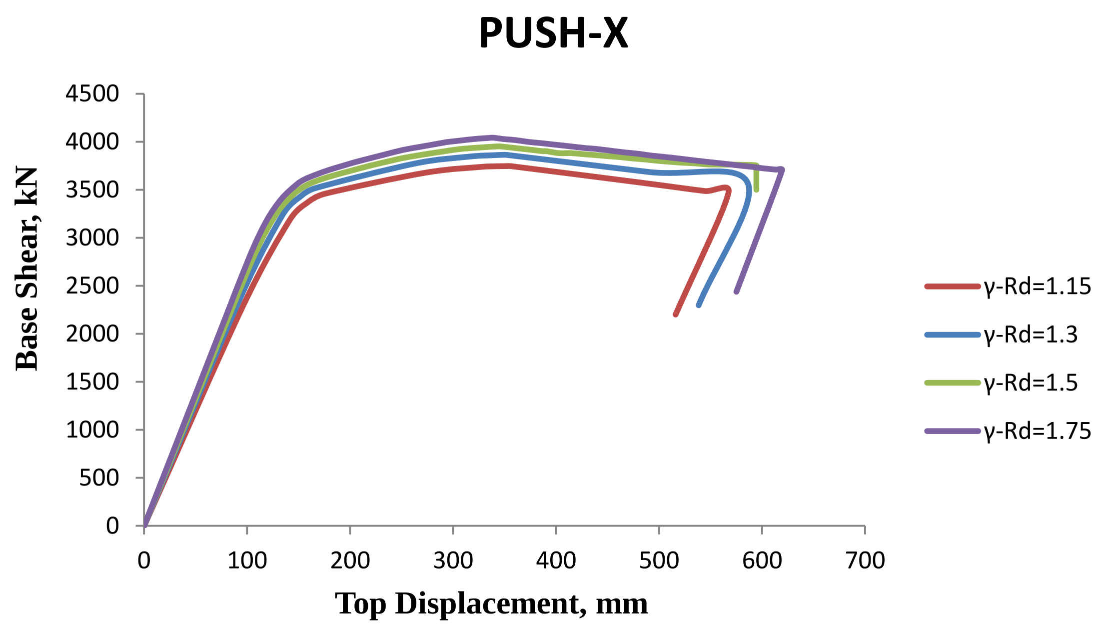

Figure 4.2 Capacity Curve for Different Overstrength Factor in the X Direction-DCM

From the above Figure 4.1 the capacity and the displacement of the structure are not differing significantly when the overstrength factor is increase. This indicates that the design of the building with different overstrength factors is governed by the code minimum requirements for the selected ductility class. From the results the capacity of the structure is increased by 3.14% (117.79kN) when the overstrength factor increased by 15% and also when the overstrength factor increased by 35% the capacity of the building also increased by 5.48% (from 3746.02kN to 3951.16kN). Further the overstrength factor increased by 60% the capacity of the building for ductility class medium structure becomes 4041.84kN which is 7.89% larger than the building designed with overstrength factor ($\gamma_{Rd} = 1.15$).

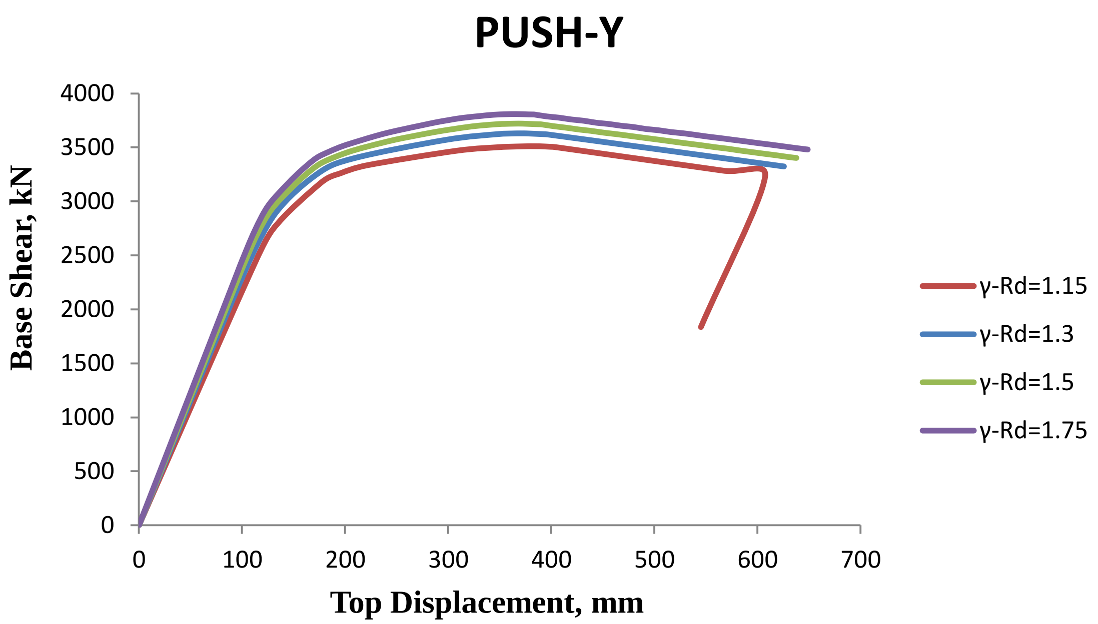

Figure 4.3 Capacity Curve for Different Overstrength Factor in the Y Direction-DCM

Similarly in the Y direction when the overstrength factor increased by 15%, 35% and 60% the capacity of the building is also increased by 3.39% (from 3511.16 to 3630.27), 5.96% and 8.49% respectively. The above result shows when the overstrength factors is increase the performance of the building in terms of capacity and displacement also increase even if the increment is small.

4.3.1.2 Story Displacement

Story displacement was the second parameter to describe the performance the building designed according to capacity design method. The story displacement of regular framed reinforced concrete structure for different overstrength factor γ_{Rd} is shown in the Figure 4.3 below in the global X direction. From the figure the story displacement of the building up to the 5th story for different overstrength factors are nearly similar.

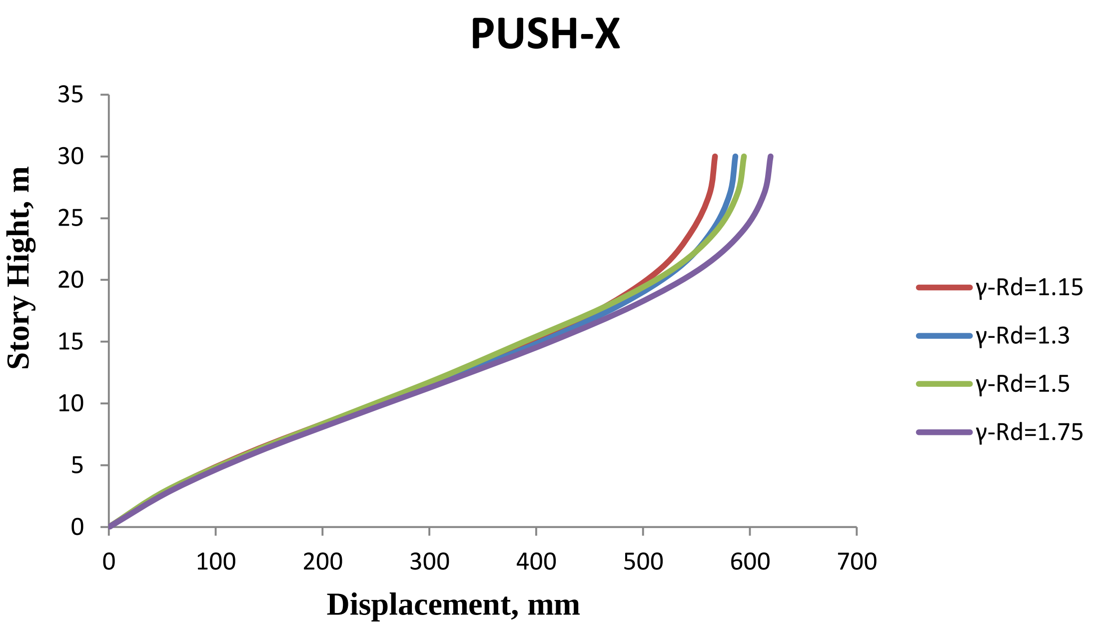

Figure 4.4 X-Direction Story Displacement for Different Overstrength Factors-DCM

Figure 4.3 represents the top displacement of the structure designed with overstrength factor $\gamma_{Rd} = 1.15$ is becomes 567.39mm in the global X direction. The overstrength factor of the building also affects the displacement of the building. The result shows in the figure 4.3 the displacement increases when the overstrength factor is increases. But the percentage of increment is different. The top displacement is becomes 586.45mm (3.36%) when the overstrength factor increases by 15%. The 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. Similarly the top displacement of the structure becomes 594.48mm and 619.43mm when the overstrength factor is increased by 35% and 60%.

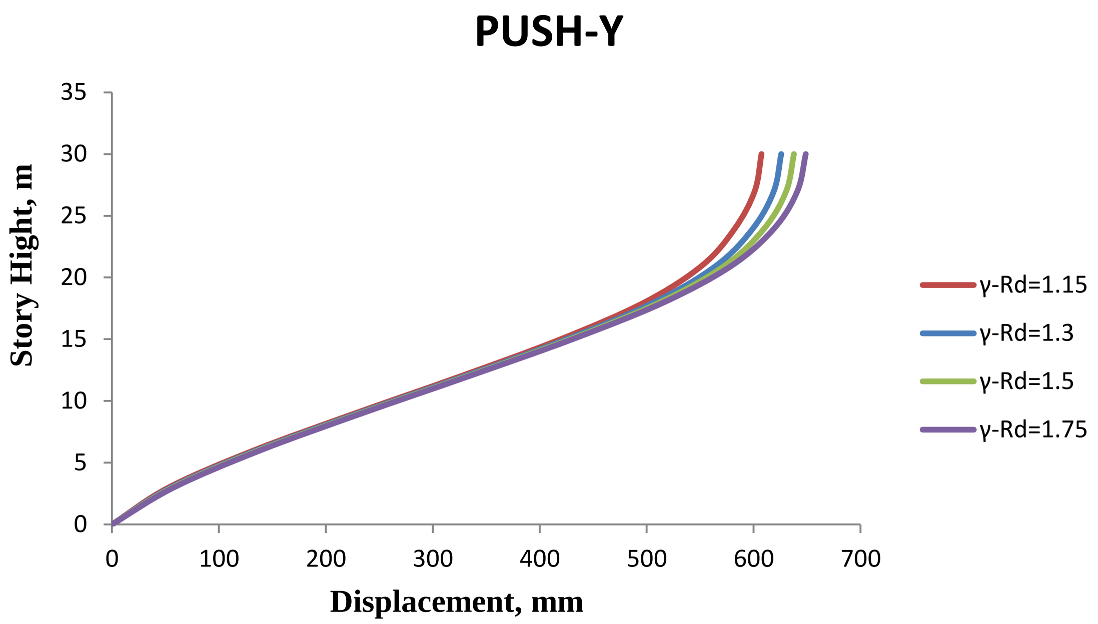

Figure 4.5 Y-Direction Story Displacement for Different Overstrength Factors-DCM

The top displacement obtained in the Y direction is 607.47mm for $\gamma_{Rd} = 1.15$. Similar to X direction the top displacement was increased with the increment of the capacity of the building. With similar manner the overstrength factor is increased by 15%, 35% and 60% and the top displacements are also increased by 3.03%, 4.99% and 6.81%. The result indicates when the overstrength factor is increased the ductility of the structure also increased and dissipates more energy.

4.3.1.3 Interstorey Drift

Interstory drift is the parameter used to describe the performance of the building in terms of the capacity difference in the consecutive storeys of the building. The interstory drift difference of the building also tells about the sudden decrease of capacity of the structural element. The interstory drift for each direction was shown graphically in the Figure 4.5 and 4.6. So from the figure below the maximum interstory drift was obtained from the building designed with overstrength factor $\gamma_{Rd} = 1.15$ is 3.04%. This maximum value is observed between 3rd and 4th storys this indicates the capacity is decreased at this story is larger than the rest of the storys. The maximum drift observed in the case of $\gamma_{Rd} = 1.3$ is 3.09% in the global X direction. This value also occurs in the

2nd and 3rd story. When the overstrength factor of the building is increased by 35% the maximum drift is 3.05% and 3.5% of maximum drift is observed when $\gamma_{Rd} = 1.75$.

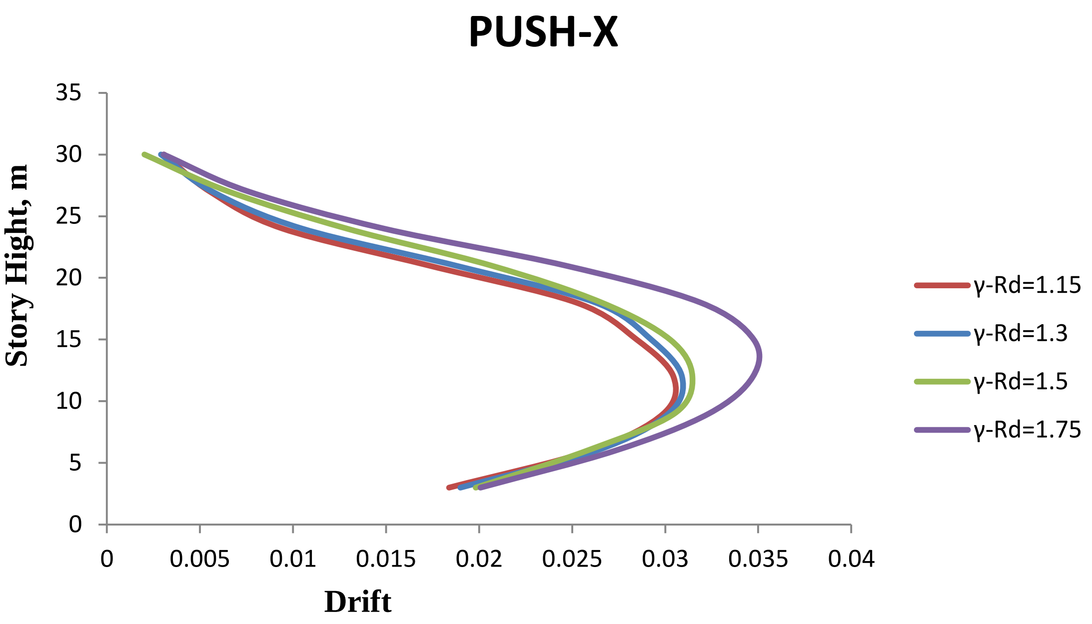

Figure 4.6 Maximum Drift for Different Overstrength Factor in the X Direction-DCM

So from the result the maximum drift observed in the building is not observed at the same story and the inter storey drift also increases when the capacity of structure as well as displacement increases due to the increment of overstrength factors. Similarly in the global Y direction the maximum drifts are observed between 2nd up to 6th story of the building. The maximum drift observed for the building designed with overstrength factor $\gamma_{Rd} = 1.15$ is 3.28% between 3rd and 4th storys. When the building overstrength factor is increased by 15%, 35% and 60% the maximum interstory drift are obtained 3.31%, 3.44% and 3.4% but the values are occur at the different storys which indicates the capacity difference is observed at different stories of the building.

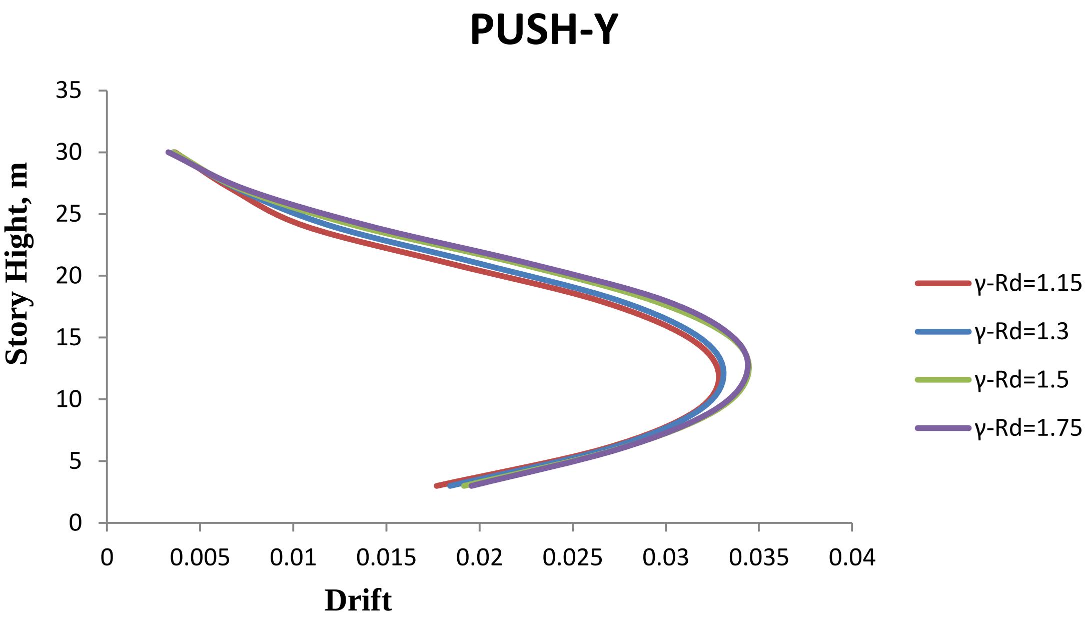

Figure 4.7 Maximum Drift for Different Overstrength Factor in the Y Direction-DCM

4.3.1.4 Plastic Hinge Distribution

The plastic hinges are occurring through predefined plastic hinge positions as shown in the figure 4.7. The main feature of capacity design method is the failure of the structure is through pre-defined critical regions or plastic hinge regions. For our building in this thesis the plastic hinges are defined at the end of each beam and bottom end of the first story column. From the Figure 4.7 below the plastic hinges are observed according to the position defined before.

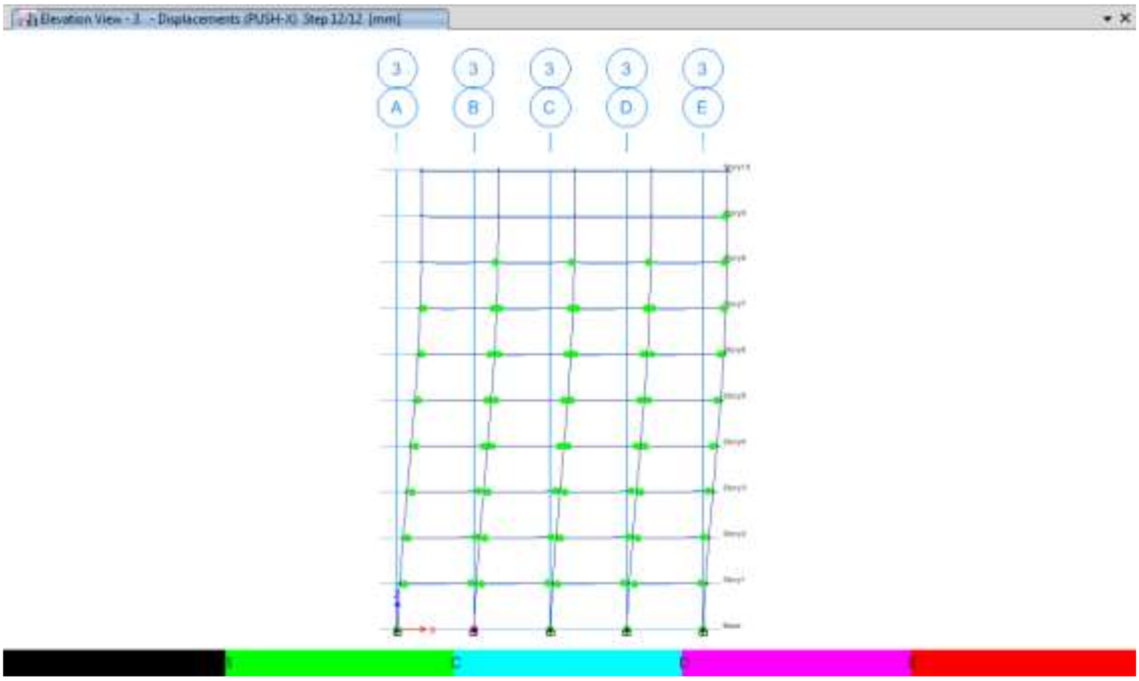

Figure 4.8 Plastic Hinge Distribution Near to Collapse for DCM Building-$\gamma_{Rd} = 1.3$

Figure 4.7 represents the plastic hinge distribution for the building designed with $\gamma_{Rd} = 1.3$ overstrength factor and for ductility class medium buildings and observed that the plastic hinges are occur predominantly in the first 6^{th} stories and this is the expected position. From the figure point B shows the starting point for yielding of hinges and point C represents the ultimate capacity of the pushover analysis. All the beams are in the range between B-C before the column is passing point D. The plastic hinges are formed according to the maximum incoming earthquake load. The first plastic hinges are formed in the bottom story beams because maximum magnification of earthquake motions are expected at the bottom and top storys of reinforced concrete structures as mentioned in (Paulay & Priestley, 1992).

4.3.2 Seismic Performance Evaluation of Ductility Class High Buildings

In this section the seismic performance of buildings designed using capacity design philosophy and for ductility class high requirements are described. The parameter used for the performance evaluation of the building is similar to the buildings that are designed by medium ductility class.

4.3.2.1 Capacity-Curve of the Building

The capacity curve of the building for ductility class high structures for different overstrength factor is shown in the figure 4.8 below.

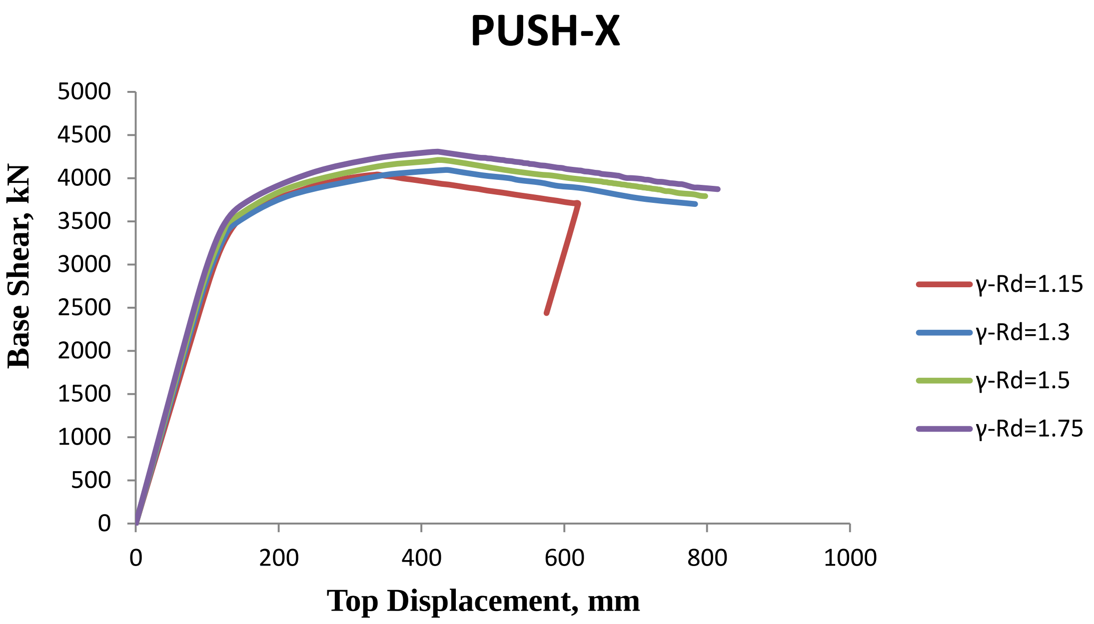

Figure 4.9 Capacity Curve for Different Overstrength Factors in the X Direction-DCH

The above figure represents the overstrength factors of the building is affect the capacity as well as the top displacement of the building considered in this thesis. In the global X direction the capacity and top displacement of the building is increased with increasing overstrength factor similar to the medium ductility class structure. The capacity of the building is increased from 4041.84kN to 4094.95kN (1.31%) when the overstrength factor is increased by 15%. Similarly the capacity is increased from 4041.84kN to 4206.79 (4.08%) and from 4041.84kN to 4306.68kN (6.55%) when the overstrength factor is increased by 35% and 60% respectively.

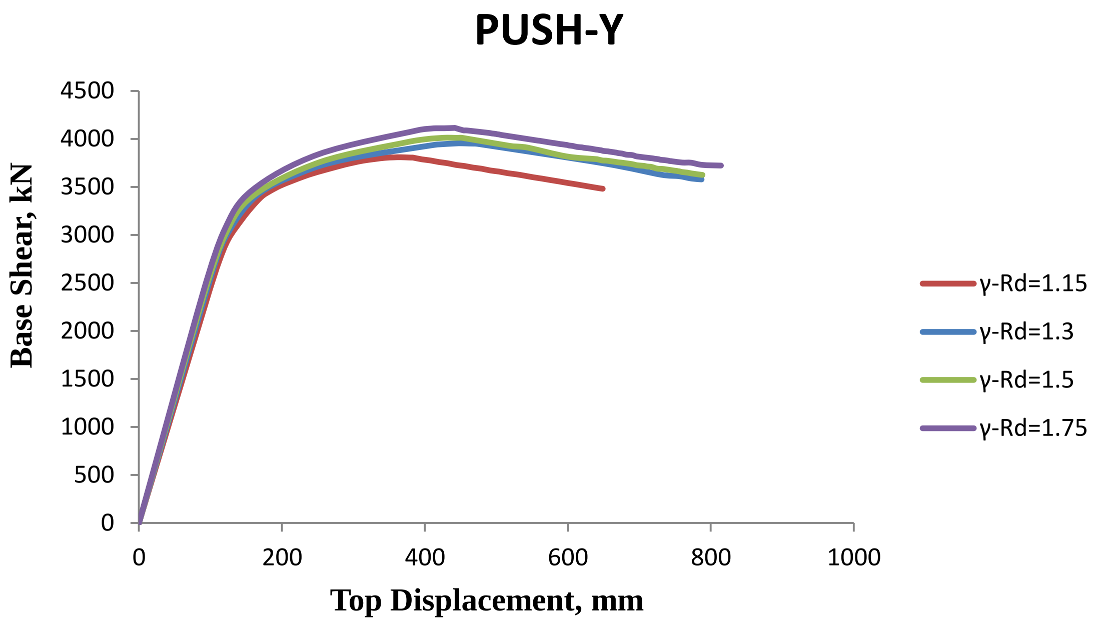

Figure 4.10 Capacity Curve for Different Overstrength Factors in the Y Direction-DCH

Based on the result obtained from the above figure 4.9 the capacity of the structure is increased when the overstrength factor is increase because the required section and area of reinforcement correspondingly increases. From the global Y direction in the figure 4.9 above the maximum capacity is obtained 4113.98kN when the overstrength is increased by 60% and when the overstrength factor is increased by 15%, 35% the capacity also increases by 3.81% and 5.34% respectively and when 60% of overstrength increased the capacity is also increased by 8.00%. From the figure 4.9 the ductility of the structure is also increased when the overstrength factor is increased.

4.3.2.2 Story Displacement

The story displacement of the building in the high ductility class building is also affected by the overstrength factor of the building in the capacity design method. Based on the following figure the displacement of each story is increased when the overstrength factor is increases.

The top displacement of the ductility class high building considered in this thesis is increased by 26.44%, 28.74% and 31.59% when the overstrength factor is increased by 15%, 35% and 60% respectively. The displacement variation for different overstrength factor is shown in the following figure.

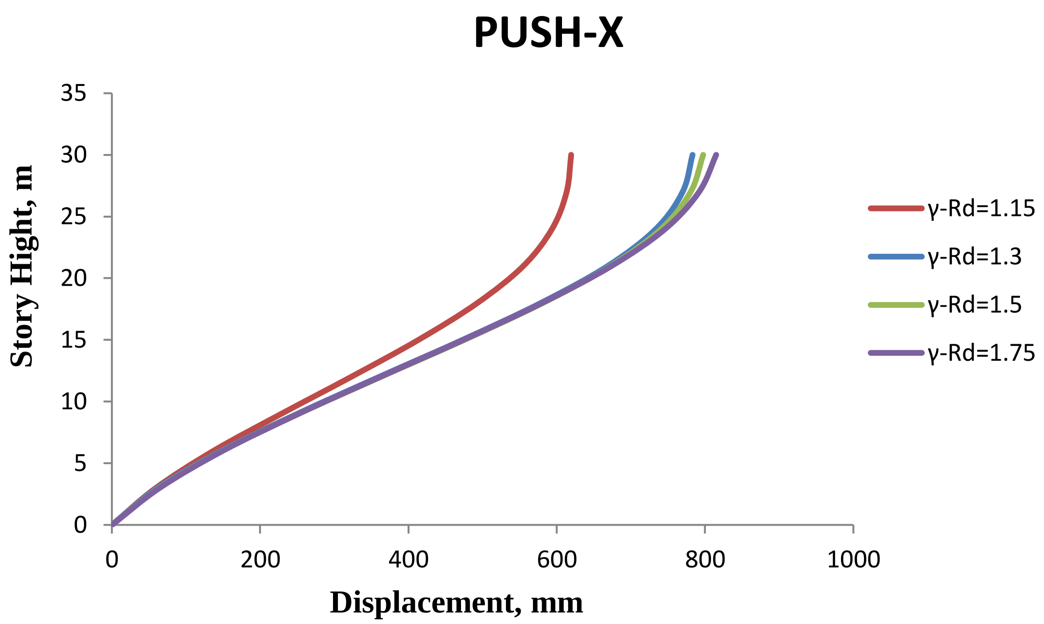

Figure 4.11 X-Direction Story Displacement for Different Overstrength Factors-DCH

The displacement difference in the upper storys of the building in both directions is large when the overstrength factor is increased. This indicates the top displacement of the building is sensitive to the variation of the required cross sectional size and area of reinforcement. Similarly in the global Y direction top displacement is increased by 21.32%, 21.52%, 25.45% when the overstrength factor is increased by 15%, 35% and 60% respectively.

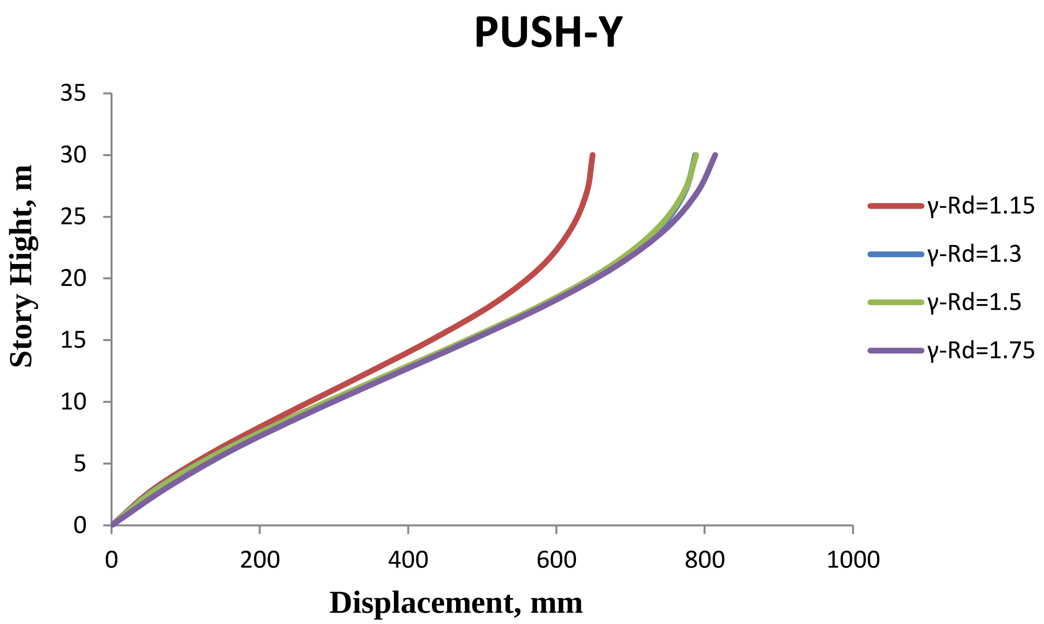

Figure 4.12 Y-Direction Story Displacement for Different Overstrength Factors-DCH

4.3.2.3 Interstorey Drift

The maximum interstory drift for each direction for different overstrength factors is shown graphically in the figure 4.12 and 4.13.

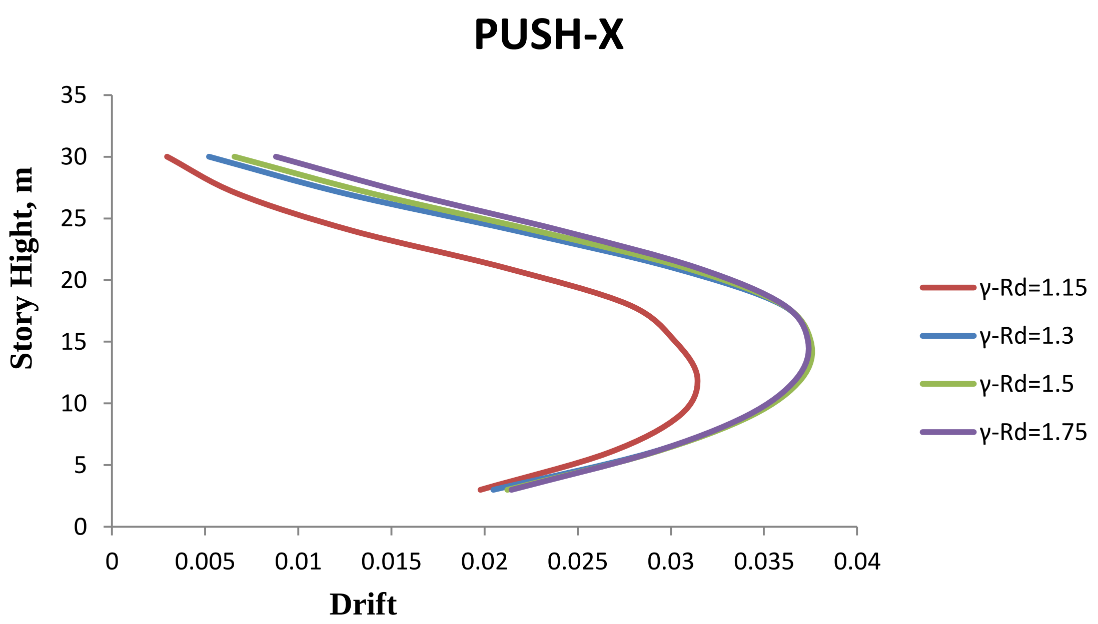

Figure 4.13 Maximum Drift for Different Overstrength Factor in the X Direction-DCH

From Figure 4.12 the maximum interstory of the building designed with $\gamma_{Rd} = 1.3$, $\gamma_{Rd} = 1.5$, $\gamma_{Rd} = 1.75$ overstrength factor are nearly similar. But the interstory of the building designed by $\gamma_{Rd} = 1.15$ is far from the buildings designed compared to the other three overstrength factors. This result shows in the global X direction the displacement variation is large between successive storys of the building that are designed with $\gamma_{Rd} = 1.15$ and with the buildings designed using the other three overstrength factor. The maximum interstory drift observed in the global X direction are 3.14%, 3.75%, 3.75% and 3.74% for $\gamma_{Rd} = 1.15$, $\gamma_{Rd} = 1.3$, $\gamma_{Rd} = 1.5$ and $\gamma_{Rd} = 1.75$ overstrength factors respectively.

Similarly in the global Y direction the maximum drifts are observed between 2nd up to 6th story of the building. The maximum drift for the building designed with overstrength factor $\gamma_{Rd} = 1.15$ is 3.33% between 4th and 5th story. When the building overstrength factor is increased by 15%, 35% and 60% the maximum interstory drift is obtained 3.80%, 3.78% and 3.73%.

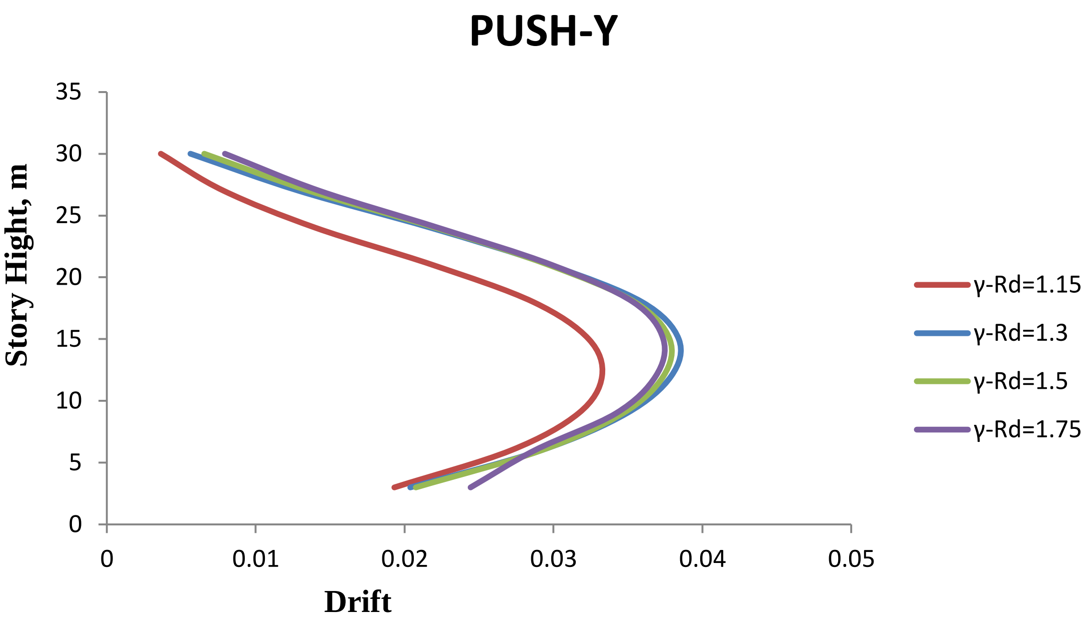

Figure 4.14 Maximum Drift for Different Overstrength Factor in the Y Direction-DCH

4.3.2.4 Plastic Hinge Distribution

Figure 4.14 represents the plastic hinge distribution of ductility class high ten story building considered in this thesis designed by $\gamma_{Rd} = 1.3$ overstrength factor. The plastic hinges are formed in all ends of the beam except the left end of the two top storys. The plastic hinges formed in the beams are in the range between point B and Point C. But the bottom end of the first story column on the axis B and D are passed point C. From the result obtained the structure is collapsed with predefined plastic hinge positions and this is expected failure mechanism for buildings designed using capacity design method.

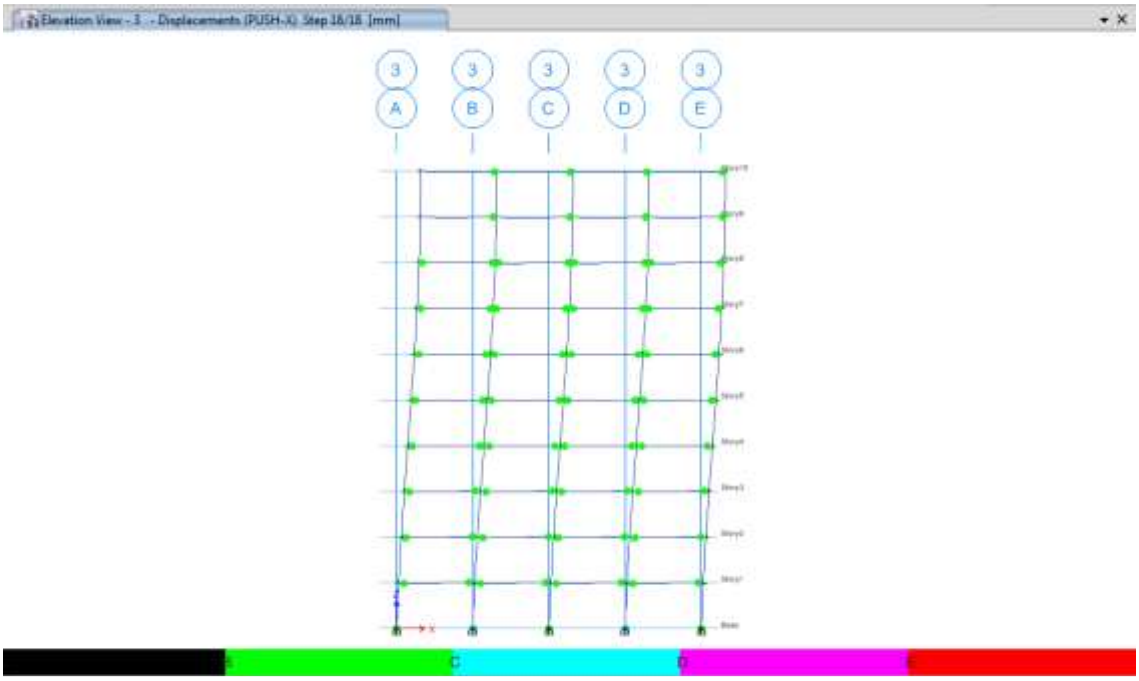

Figure 4.15 Plastic Hinge Distribution Near to Collapse for DCH Building-$\gamma_{Rd} = 1.3$

4.3.3 Performance Comparison of the Two Ductility Class Buildings

The structural performance of the structure is influenced by ductility class of the structure. The requirements and criteria for the two ductility class are different according to Ethiopian building code. This difference is leads to the usage of different cross sectional size, different reinforcement areas and other parameters. The comparisons of ductility class high and ductility class medium buildings that are assessed in this thesis are summarized in the following figure 4.15 in terms of base shear in the X direction. The difference in the Y direction is almost similar to the X direction.

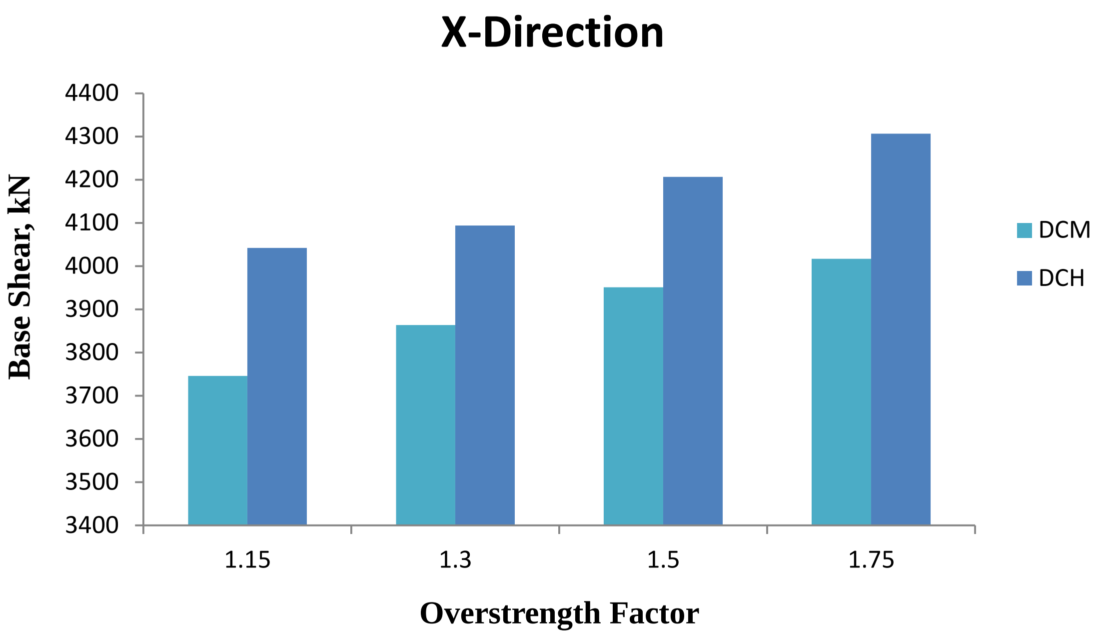

Figure 4.16 Performance Comparison of Buildings in terms of Base Shear

From the above figure the effect of ductility class on the capacity of the building is nearly similar for all column-beam overstrength factors. The influence of ductility class for the specific overstrength factor is large as shown in the figure 4.15.

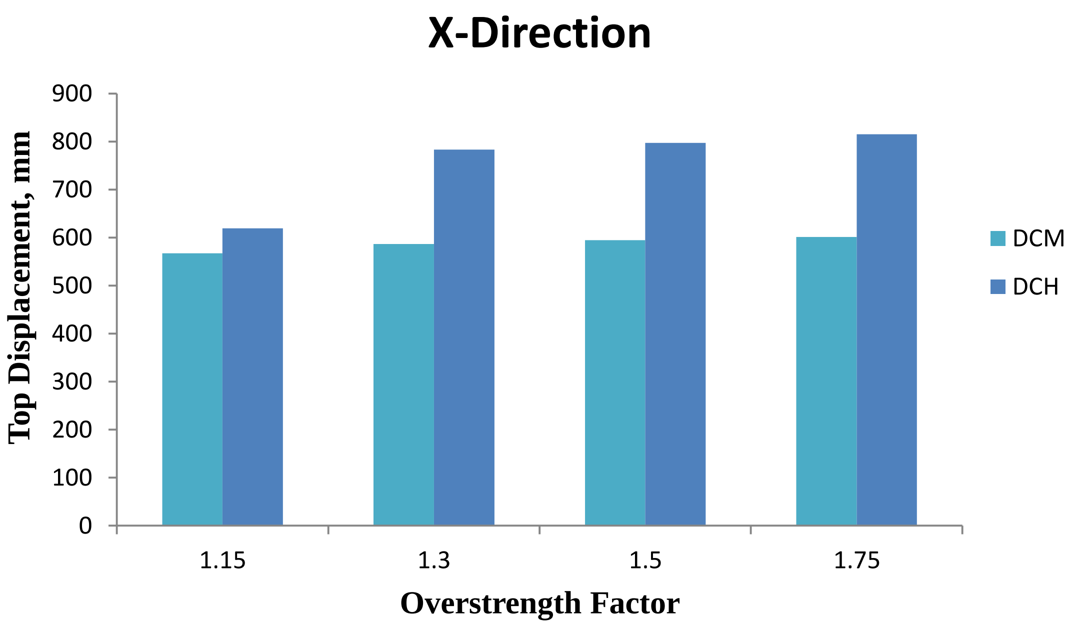

Figure 4.17 Performance Comparison of Buildings in terms of Top Displacement

This difference is mainly due to the size of the members and the required reinforcements for each ductility class. Similarly the top displacement of the building is also affected by the ductility class used in the modeling of the building. The effect of overstrength is not large in the lower column-beam overstrength factor. But almost similar effect on the high overstrength factors as shown in the figure 4.16. In both directions the effect of ductility class on the performance of the structures are nearly similar. From this result ductility class is relatively higher effect on the capacity of the building than the top displacement of the buildings that are considered in this thesis.

CHAPTER 5 CONCLUSIONS AND RECCOMENDATIONS

5.1 CONCLUSIONS

In this paper the seismic performance assessment of ten story regular framed reinforced concrete structure has been investigated through the use of nonlinear static analysis method. The buildings assessed in this thesis were designed using capacity design method for different column-beam overstrength factors. The effect of ductility class has been also investigated in this paper using ductility class medium and ductility class high buildings. The result obtained from pushover analysis by considering the influence of overstrength factor and ductility class leads to the following conclusions.

In the ductility class medium building the capacity of the structure is increased when the overstrength factor is increases. The increments of the base shear of the buildings are relatively small. This is due to the design of the buildings are mostly governed by the minimum requirements provided in the new Ethiopian building code. At the critical sections special attention is given for the detailing of reinforcement relative to the section remains elastic. So the minimum requirements are governed in most cases.

The top displacement of the building also increased when the capacity of the building as well as the overstrength factor is increased in medium ductility class buildings. But the increment is relatively smaller than the base shear of the buildings considered in this thesis. The base shear and top displacement of the building in the case of ductility class high structure is also increased when overstrength factor is increased.

The influences of overstrength factor in the capacity of the structure in medium ductility class buildings are nearly similar to ductility class high buildings with similar overstrength factor value in both principal directions. This difference is due to the difference in the required cross sectional size and reinforcements used in the design of structural members. The top displacement increment due to the increasing of overstrength factor in the ductility medium structures are relatively smaller than ductility class high buildings with equal overstrength increment.

Generally, column-beam overstrength factor influence the overall seismic performance of the buildings that are designed with capacity design method in both ductility class buildings. The ductility of the building also improved when the overstrength factor is increases.

5.2 RECCOMENDATIONS

The seismic performance of reinforced concrete structure is affected by the column-beam overstrength factor of the structure. The overstrength factor of the structure is also dependent on different parameters such as the strength of material used, the ductility class considered, the regularity of the structure, the story height of the building, the soil type used, the magnitude of the incoming ground acceleration and the structural type used to resist the lateral forces. The seismic performance of the structure is influenced when the pervious parameters are changed.

Therefore different researches should be done by considering different parameters that affects the overstrength factor of the structure as well as the seismic performance of the building and the following researches provides for the future researchers.

- The effect of irregularity on the overstrength factor and on the seismic performance of the building.
- The effect of story height on the overstrength factor of the building and on the seismic performance of the building.
- The effect of Lateral resisting systems on the overstrength factor and on the seismic performance of the building.

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APPENDIX A

A.1 The Floor Plan and Chosen Framing System

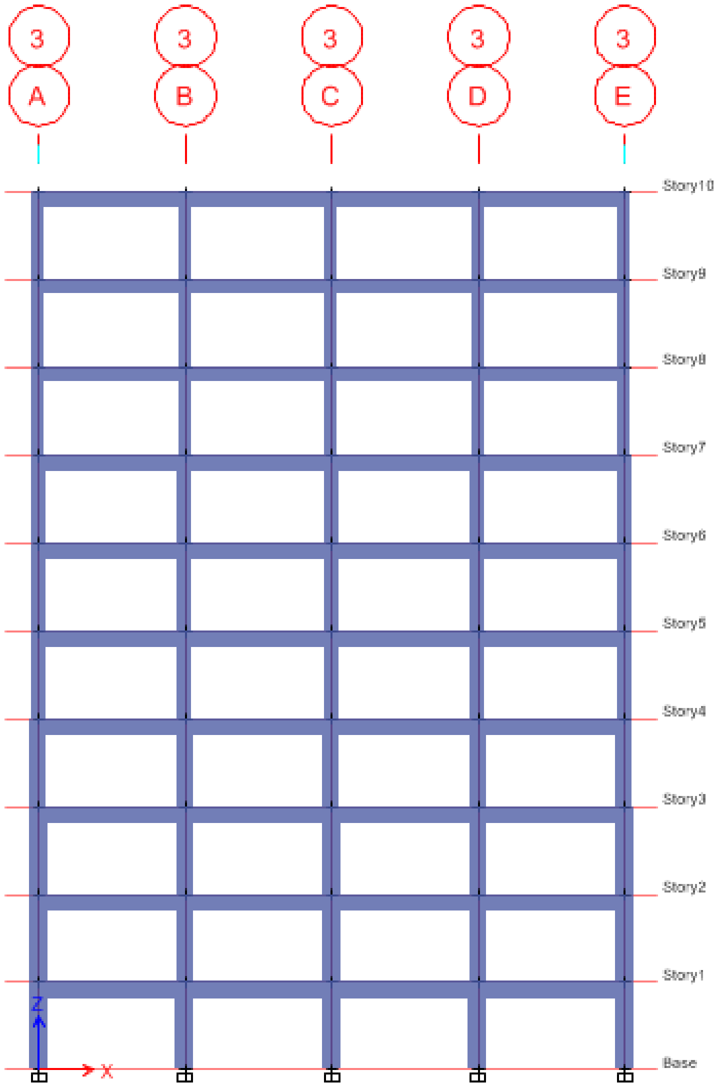

Figure A. 1 Elevation View-3 in the X Direction

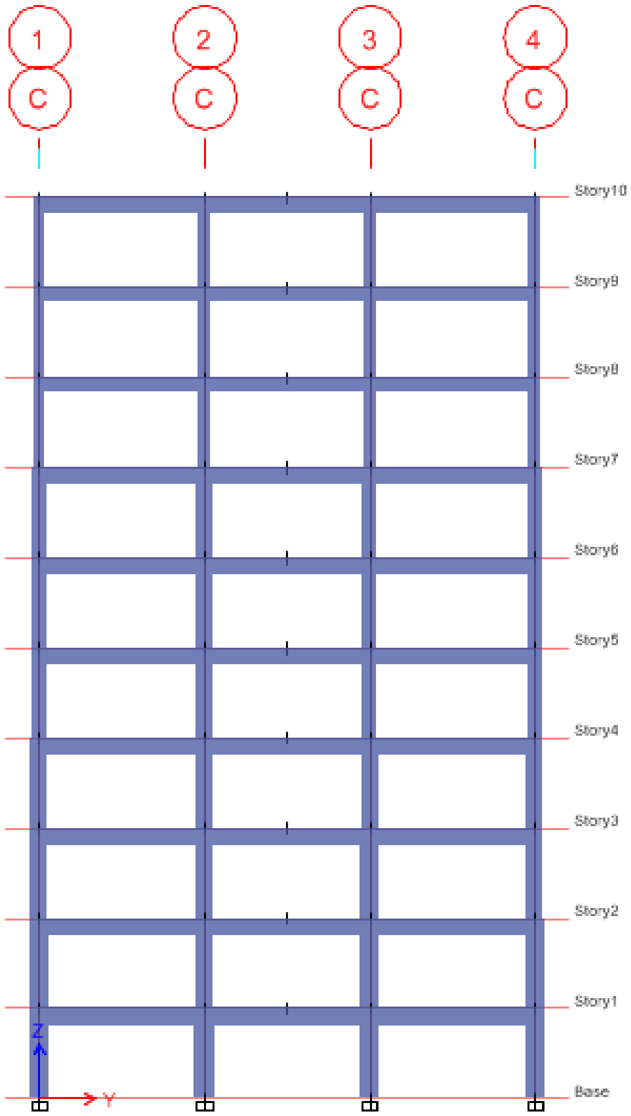

Figure A. 2 Elevation View-C in the Y Direction

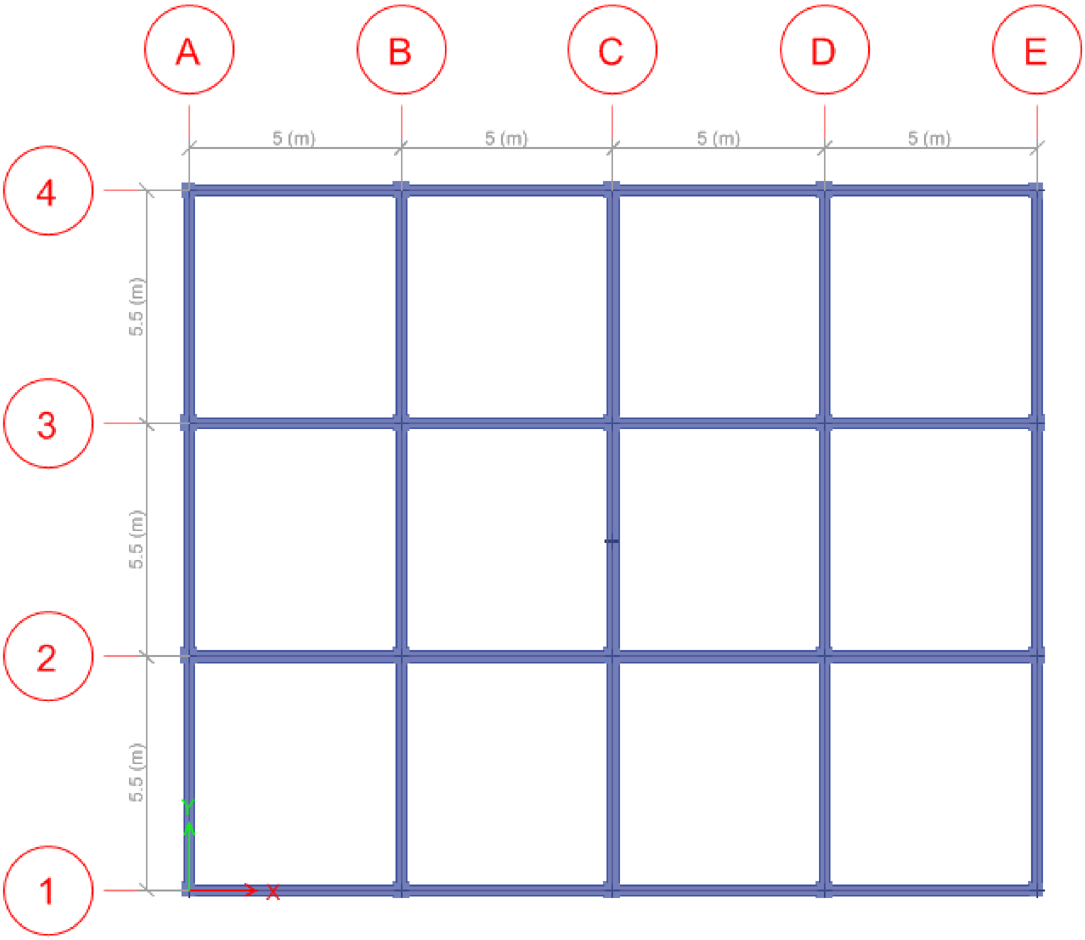

Figure A. 3 Plan View of Story 10-Z=30m

A.2 Design Outputs of DCM and DCH Buildings for Diffrent Overstrength Factor

Table A. 1 Beam Flexural Design Output for DCM and $\gamma_{Rd} = 1.3$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1608	1206	1206	1005	804	804	603	603	603
		Bottom	603	1206	804	804	804	603	603	603	603	603
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1407	1407	1407	1407	1206	804	603	603
		Bottom	804	1005	1206	1206	1407	1407	1206	603	603	603
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	603
		Bottom	603	1005	1005	1005	804	804	804	804	804	603
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	1005	1206	1206	1206	1005	1005	804	804	804	603
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	1005	1206	1206	1206	1005	1005	804	804	804	603

Table A. 2 Column Flexural Design Output for DCM and $\gamma_{Rd} = 1.3$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	550	550	500	500	500	400	400	400	350	300
	B (mm)	550	550	500	500	500	400	400	400	350	300
	A_{sLT} (mm^2)	3768	3768	3216	3216	3216	2412	1608	1608	1608	1356
2&3	H (mm)	650	600	600	550	500	500	450	400	350	300
	B (mm)	650	600	600	550	500	500	450	400	350	300
	A_{sLT} (mm^2)	5024	5024	5024	3768	3216	3216	2412	1608	1608	1356

Table A. 3 Beam Flexural Design Output for DCM and $\gamma_{Rd} = 1.5$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1570	1206	1206	1005	804	804	603	603	603
		Bottom	603	1178	784	784	754	603	603	603	603	603
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1407	1407	1407	1407	1206	804	603	603
		Bottom	704	965	1126	1126	1267	1267	1086	603	452	452
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	603
		Bottom	563	754	754	754	804	804	804	644	683	452
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	965	1196	1126	1126	905	965	804	804	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1407	804	804	603
		Bottom	905	1196	1126	1126	1267	1267	1267	643	643	512

Table A. 4 Column Flexural Design Output for DCM and $\gamma_{Rd} = 1.5$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	550	550	550	500	500	450	400	400	350	300
	B (mm)	550	550	550	500	500	450	400	400	350	300
	A_{sLT} (mm^2)	3768	3768	3768	3216	3216	3216	1608	1608	1608	1356
2&3	H (mm)	650	600	600	550	550	550	450	400	350	300
	B (mm)	650	600	600	550	550	550	450	400	350	300
	A_{sLT} (mm^2)	5024	5024	5024	3768	3768	3768	3216	1608	1608	1356

Table A. 5 Beam Flexural Design Output for DCM and $\gamma_{Rd} = 1.75$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1571	1206	1206	1005	804	804	603	603	603
		Bottom	563	1178	784	784	804	643	603	452	452	452
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1407	1407	1407	1407	1206	804	603	603
		Bottom	704	965	1126	1126	1247	1247	1086	603	483	483
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	603
		Bottom	563	754	754	704	804	754	804	643	684	452
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	804	804	603
		Bottom	965	1126	1056	1267	845	965	804	643	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1005	804	804	603
		Bottom	905	1126	1126	1267	1247	1267	754	643	643	513

Table A. 6 Column Flexural Design Output for DCM and $\gamma_{Rd} = 1.75$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	600	600	550	550	550	450	400	400	350	350
	B (mm)	600	600	550	550	550	450	400	400	350	350
	A_{sLT} (mm^2)	3768	3768	3216	3216	3216	2412	1608	1608	1356	1356
2&3	H (mm)	700	650	650	600	550	550	450	400	350	350
	B (mm)	700	650	650	600	550	550	450	400	350	350
	A_{sLT} (mm^2)	5024	5024	5024	3768	3216	3216	2412	1608	1356	1356

Table A. 7 Beam Flexural Design Output for DCH and $\gamma_{Rd} = 1.15$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1571	1206	1206	1005	804	804	603	603	603
		Bottom	563	1178	784	784	804	563	603	452	452	603
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1407	1407	1407	1407	1407	1206	804	603	603
		Bottom	704	1126	1126	1056	1267	1056	1086	563	452	452
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	804	804	804	603
		Bottom	643	754	754	704	804	804	643	643	683	452
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	965	1126	1126	1056	965	965	804	754	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1005	804	804	603
		Bottom	905	1056	1126	1126	1267	1337	905	643	643	513

Table A. 8 Column Flexural Design Output for DCH and $\gamma_{Rd} = 1.15$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	600	600	550	550	550	450	400	400	350	350
	B (mm)	600	600	550	550	550	450	400	400	350	350
	A_{sLT} (mm^2)	3768	3768	3768	3768	3768	2412	1608	1608	1356	1356
2&3	H (mm)	700	650	650	600	550	550	450	400	350	350
	B (mm)	700	650	650	600	550	550	450	400	350	350
	A_{sLT} (mm^2)	5024	5024	5024	3768	3768	3768	2412	1608	1356	1356

Table A. 9 Beam Flexural Design Output for DCH and $\gamma_{Rd} = 1.3$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1571	1005	1005	1005	804	804	603	603	402
		Bottom	563	1178	754	754	804	563	603	452	452	402
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1407	1407	1407	1407	1206	804	603	402
		Bottom	704	965	1126	1126	1247	1247	1086	603	452	402
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	603	804	402
		Bottom	563	754	754	704	804	804	754	483	684	402
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	965	1126	1126	1126	905	965	804	804	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1407	1005	804	603
		Bottom	905	1126	1126	1126	1247	1267	1267	754	643	513

Table A. 10 Column Flexural Design Output for DCH and $\gamma_{Rd} = 1.3$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	600	600	550	550	550	450	450	450	400	350
	B (mm)	600	600	550	550	550	450	450	450	400	350
	A_{sLT} (mm^2)	5024	5024	5024	5024	3768	2412	2412	2412	2412	1608
2&3	H (mm)	700	650	650	600	550	550	500	450	400	350
	B (mm)	700	650	650	600	550	550	500	450	400	350
	A_{sLT} (mm^2)	6280	5024	5024	5024	5024	3768	2512	2412	2412	1608

Table A. 11 Beam Flexural Design Output for DCH and $\gamma_{Rd} = 1.5$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1206	1206	1206	1005	804	804	603	603	402
		Bottom	603	905	965	965	804	563	603	452	452	402
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1206	1206	1206	1206	1206	804	603	402
		Bottom	704	965	1086	1086	1025	1086	1146	603	452	402
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	402
		Bottom	563	603	603	754	804	804	804	523	684	402
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	965	1196	1196	1126	965	965	804	804	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1005	804	804	603
		Bottom	905	985	1126	1126	1267	1267	754	523	643	513

Table A. 12 Column Flexural Design Output for DCH and $\gamma_{Rd} = 1.5$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	600	600	550	550	550	500	450	450	400	400
	B (mm)	600	600	550	550	550	500	450	450	400	400
	A_{sLT} (mm^2)	5024	5024	3768	3768	3768	2512	2412	2412	2412	1608
2&3	H (mm)	700	650	650	600	600	600	500	450	400	400
	B (mm)	700	650	650	600	600	600	500	450	400	400
	A_{sLT} (mm^2)	6280	5024	5024	5024	5024	5024	2512	2412	2412	1608

Table A. 13 Beam Flexural Design Output for DCH and $\gamma_{Rd} = 1.75$

			Story									
Axis			1	2	3	4	5	6	7	8	9	10
1&4	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1571	1206	1206	1005	804	804	603	603	402
		Bottom	603	1206	905	905	804	603	563	452	452	402
2&3	H (mm)		500	500	500	500	400	400	400	400	400	350
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1005	1206	1206	1206	1407	1407	1206	804	603	402
		Bottom	754	965	1086	1086	1247	1247	1086	603	603	402
A&E	H (mm)		500	500	500	500	450	450	400	350	350	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	804	1005	1005	1005	1005	1005	1005	804	804	402
		Bottom	523	754	754	754	804	804	804	644	684	402
B&D	H (mm)		500	500	500	500	450	450	450	400	400	300
	B (mm)		300	300	300	300	300	300	300	300	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1206	1206	1005	1005	804	603
		Bottom	965	1196	1196	1126	905	965	804	804	643	483
C	H (mm)		500	500	500	500	400	400	400	400	400	300
	B (mm)		300	300	300	300	300	300	300	250	250	250
	A_{sLc} (mm^2)	Top	1206	1407	1407	1407	1407	1407	1407	1005	804	603
		Bottom	905	985	1126	1126	1247	1267	1267	654	643	513

Table A. 14 Column Flexural Design Output for DCH and $\gamma_{Rd} = 1.75$

		Story									
Axis		1	2	3	4	5	6	7	8	9	10
1&4	H (mm)	650	650	600	600	600	500	450	450	400	400
	B (mm)	650	650	600	600	600	500	450	450	400	400
	A_{sLT} (mm^2)	5024	5024	3768	3768	3768	2512	2412	2412	1848	1848
2&3	H (mm)	750	700	700	650	600	600	500	450	400	400
	B (mm)	750	700	700	650	600	600	500	450	400	400
	A_{sLT} (mm^2)	6280	5024	5024	5024	3768	3768	2512	2412	1848	1848

APPENDIX B

B.1 Design Table and Design Charts for the Design of Beam and Column

Table B. 1 Beam Flexural Design Table for C 12/15-C 50/60

$\mu_{Sd} = \frac{M_{Sd}}{f_{cd}bd^2}$	$\omega = \frac{A_{S1}f_{yd}}{f_{cd}bd}$	$k_x = \frac{x}{d}$	$k_x = \frac{z}{d}$	ε_c $(^o/_{oo})$	ε_{S1} $(^o/_{oo})$	Percentage Redistribution
0.000	0.000	0.000	1.000	0.000	25.000	
0.010	0.010	0.030	0.990	0.773	25.000	
0.020	0.020	0.044	0.985	1.146	25.000	
0.030	0.031	0.055	0.980	1.464	25.000	
0.040	0.041	0.066	0.976	1.763	25.000	
0.050	0.051	0.076	0.971	2.060	25.000	
0.060	0.062	0.086	0.967	2.365	25.000	
0.070	0.073	0.097	0.962	2.682	25.000	
0.080	0.084	0.107	0.956	3.009	25.000	
0.090	0.095	0.118	0.951	3.349	25.000	
0.100	0.106	0.131	0.946	3.500	23.294	
0.110	0.117	0.145	0.940	3.500	20.709	
0.120	0.128	0.159	0.934	3.500	18.552	
0.130	0.140	0.173	0.928	3.500	16.726	
0.140	0.152	0.188	0.922	3.500	15.159	
0.150	0.164	0.202	0.916	3.500	13.799	
0.160	0.176	0.217	0.910	3.500	12.608	
0.170	0.188	0.232	0.903	3.500	11.555	
0.180	0.201	0.248	0.897	3.500	10.618	
0.190	0.213	0.264	0.890	3.500	9.777	
0.200	0.226	0.280	0.884	3.500	9.019	
0.205	0.233	0.288	0.880	3.500	8.653	20%
0.210	0.239	0.296	0.877	3.500	8.332	
0.220	0.253	0.312	0.870	3.500	7.706	
0.230	0.266	0.329	0.863	3.500	7.132	
0.400	0.280	0.346	0.856	3.500	6.605	
0.250	0.295	0.364	0.849	3.500	6.118	
0.252	0.298	0.368	0.847	3.500	6.011	10%
0.260	0.309	0.382	0.841	3.500	5.667	
0.270	0.324	0.400	0.834	3.500	5.247	
0.280	0.339	0.419	0.826	3.500	4.856	
0.290	0.355	0.438	0.818	3.500	4.490	
0.295	0.363	0.448	0.814	3.500	4.313	0%

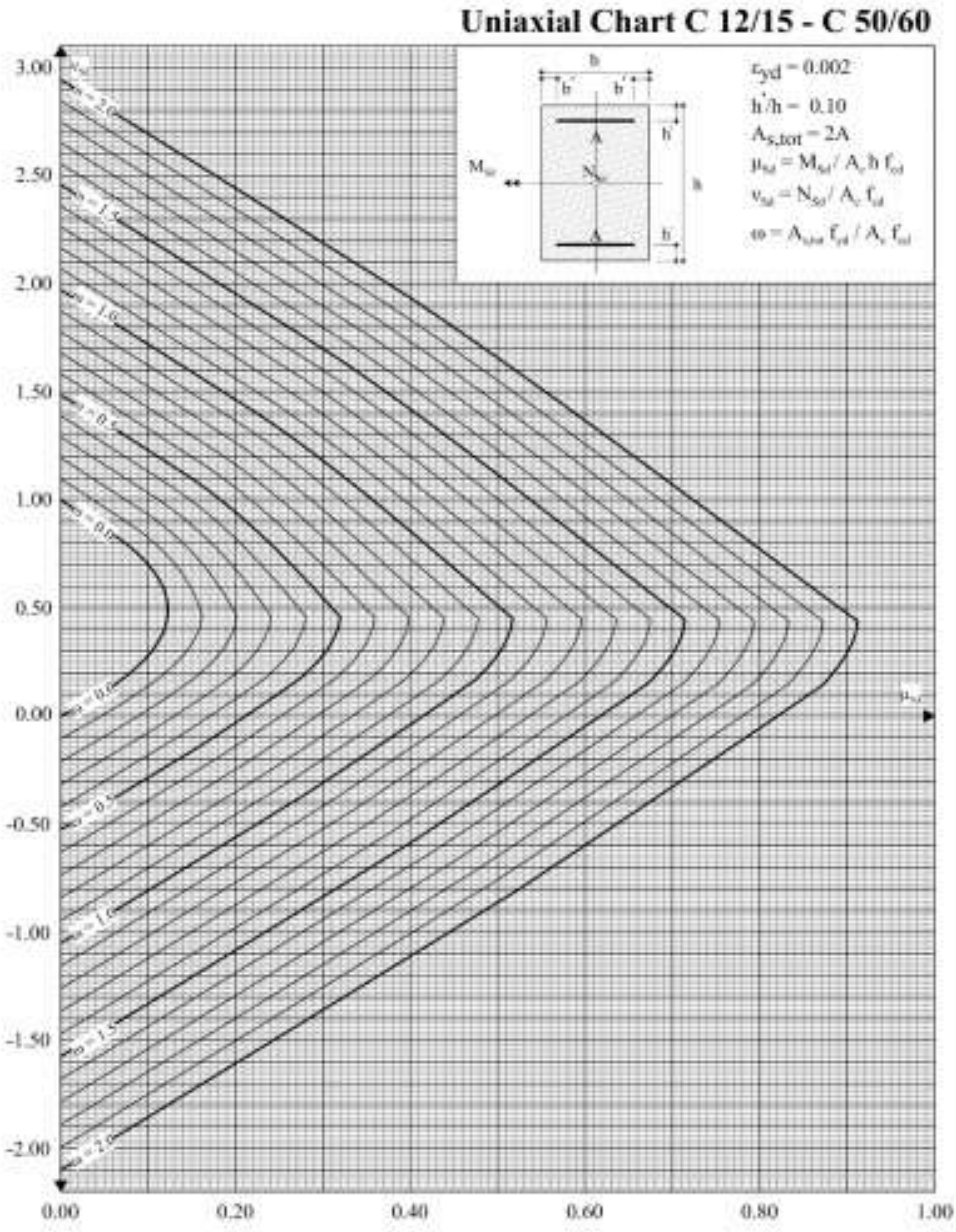

Figure B. 1 Uniaxial Chart for C 12/15 – C 50/60 Column

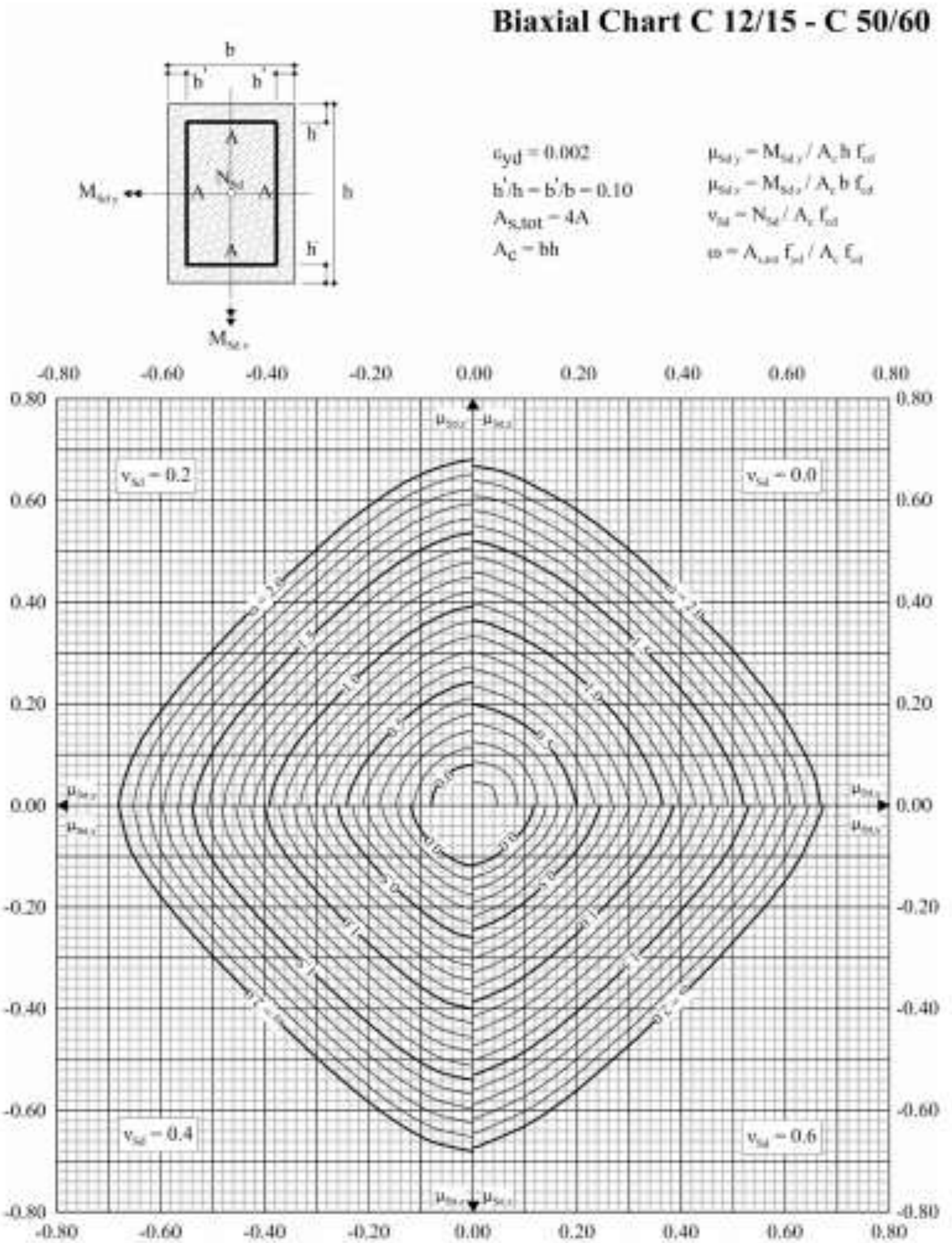

Figure B. 2 Biaxial Chart for C 12/15 - C 50/60 Column

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
Torsionally 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.

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