CAPITAL UNIVERSITY OF SCIENCE AND TECHNOLOGY, ISLAMABAD

# Optimal Design of Mid Rise Building Using Performance Based Seismic Design Approach

by

Kamran Muhammad

A thesis submitted in partial fulfillment for the degree of Master of Science

in the

Faculty of Engineering
Department of Civil Engineering

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*This work is dedicated to my lovely parents who helped me throughout my life. This work is dedicated to my honorable teachers who guided me to face the challenges of life with patience and courage.*

## CERTIFICATE OF APPROVAL

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# *Abstract*

Due to the high seismic demands and growing material cost, structural design optimization techniques need to be explored. Thus, the need for having an effective optimum design approach for the reinforced concrete (RC) building is in growing demand. The aim of this research thesis is to optimize code based design of RC framing members by using performance based design (PBD) approach. In this research, the probable economization in term of construction materials for RC framing members is assessed by achieving the performance objectives set by code for different seismic conditions. The specific goal of this research is to achieve the performance objectives set by code for seismic conditions by having minimal usage of construction materials. The seismic behavior of a code-based designed, 7-storey structure has been observed. Hinges are asserted at appropriate locations into the code based designed reinforced concrete frame structure to calculate its capacity against seismic demands by using nonlinear pushover analysis.

In PBD approach, the steel reinforcement in beams is taken as design variable to achieve the intended optimal RC design. In connection to the reinforced structural element optimization, Federal Emergency Management Agency (FEMA) guidelines have been used to evaluate the capacity of the element section being subjected to nonlinear interaction. Resultantly, an optimum design model using PBD approach with reduction of beam flexural reinforcement up to 9.09% is achieved. Therefore, it is demonstrated that the PBD strategy leads to optimal design reinforcement and better performance against seismic hazards.

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# Abbreviations

| | |
|---|---|
| **ACI** | American Concrete Institute |
| **ASCE** | American Society of Civil Engineering |
| **ATC** | Applied Technology Council |
| **BCP** | Building Code of Pakistan |
| **CBD** | Code Based Design |
| **CFRP** | Carbon Fiber Reinforced Polymer |
| **Cp** | Collapse Prevention |
| **DBE** | Design Based Earthquake |
| **DR** | Damage Ratio |
| **EC8** | Euro Code 8 |
| **EQ** | Earthquake |
| **FD** | Force Displacement |
| **FEMA** | Federal Emergency Management Agency |
| **FEM** | Finite Element Modelling |
| **IBC** | International Building Code |
| **IDR** | Interstorey Drift Ratio |
| **IMRS** | Intermediate Moment Resistant Frame |
| **IO** | Immediate Occupancy |
| **LDP** | Linear Dynamic Procedure |
| **LSP** | Linear Static Procedure |
| **LS** | Life Safety |
| **MCE** | Maximum Considered Earthquake |
| **MOOF** | Multi degree of Freedom System |
| **NEHPR** | National Earthquake Hazards Reduction Program |

| | |
|---|---|
| **NSP** | Nonlinear Static Procedure |
| **OC** | Optimum Citerion |
| **PBEE** | Performance Based Earthquake Engineering |
| **PBD** | Performance Based Design |
| **PC** | Plain Concrete |
| **PoA** | Push-over Analysis |
| **PP** | Performance Point |
| **RC** | Reinforced Concrete |
| **RSA** | Response Spectrum Analysis |
| **SDoF** | Single Degree of Freedom |
| **SLE** | Service Level Earthquake |
| **UBC** | Uniform Building Code |

# Symbols

| | |
|---|---|
| $f'_c$ | Compressive Strength of concrete |
| $f_y$ | Yield strength of steel |
| D | Daed load |
| L | Live load |
| E | Earthquake load |
| $l_p$ | Plastic hinge length |
| $d_b$ | diameter of reinforced bar |
| $C_0$ | Modification factor to relate spectral displacement |
| $C_1$ | Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response |
| $C_2$ | Modification factor o represent the effect of hysteresis shape on the maximum displacement response |
| $C_3$ | Modification factor to represent the increase displacement due to P-$\Delta$ effect |
| $S_a$ | Spectral acceleration |
| $T_e$ | Effective time period |
| g | Acceleration of gravity |
| b | width of beam |
| d | Depth of Beam |
| M | Mass |

# Chapter 1

# Introduction

## 1.1 Preface

The adequate designing of a structure is based on satisfying the criteria set by local design codes or guidelines. After accomplishing this, the designer has to check whether the structure is over-designed and to what degree the structural behavior can be adjusted to get an economical solution that still fulfills the design prerequisites. However, it is quite difficult to determine if the design of the structure can be further optimized or the current design is suitable. Seismic design of a structure is significantly influenced by the seismic zone and the soil profile type of the site.

Engineers do not make attempts to design earthquake-proof buildings owing to their exorbitant expenses that are usually resistant to strong intensity earthquakes. Therefore, the engineering objective is to design earthquake-resistant buildings although they may suffer severe damage but ultimately do not get collapse while subjected to strong and intensive ground shaking. In order to achieve earthquake-resistant structures, the seismic design philosophy is applied which makes the frame elements of the building ductile and flexible. Resultantly, the structure is able to sway to and fro in case of a seismic event without suffering total collapse. The earthquake design approach predetermines locations that are prone to collapse when face earthquake events. After identifying such positions in the proposed, Structure, the designer provides for certain details to make the building ductile and be able to cope with ground shaking forces.

On the contrary, in Code Based Design (CBD) method there is only one "R" (Strength Reduction Factor) value incorporated for the entire building. Therefore to reduce the cost of building a structure, a new approach for seismic design based on cost optimization is proposed with performance of the structure predetermined under seismic hazards. This strategy uses Performance-based Criteria. These criteria are taken from the National Guidelines for Seismic Rehabilitation of Buildings, Federal Emergency Management Agency (FEMA) and National Earthquake Hazards Reduction Program (NEHRP).

Using performance based seismic demand, nonlinear static procedure provides a better estimates of seismic demands (Bracci et al. 1997). Performance based seismic design (PBSD) assess the performance level of structure defined by codes against seismic hazards and the damage at desired locations. Performance based seismic design give designers the possibility to obtain a desired performance objective when the structure is subjected to a certain hazard level. In PBSD, non-linear static procedure or pushover analysis (PoA) technique the behavior of building is evaluated against target displacement.

## 1.2 Research Motivation and Problem Statement

Nowadays, the construction of reinforced concrete (RC) buildings is gradually increasing day by day. Thus, the quantity of materials used during their construction phase is found to be an important aspect in term of economy. Without compromising on safety against seismic hazards the quantity of materials could be reduced up to some extent and can contribute towards the economy of the construction of building.

It is almost impossible to completely diminish the effects of earthquakes, but their risk can be reduced by taking suitable measures. However, buildings could be designed and constructed to perform satisfactorily in an event of earthquake. In order to mitigate the harmful effects of earthquake events, codes have been developed by the engineers and researchers all over the world. One problem with Code Based Design (CBD) approach is that, the behavior of buildings is unknown under seismic events. Therefore, nonlinear analysis are required to be applied to assess the behavior of the structure under seismic effects. In most of the studies, performance based design (PBD) approach has been used in developing structural optimization technique. However, the performance based approach using prevailing code based design from the perspective of their effectiveness in terms of structural members optimization still needs to be explored in detail.

## 1.3 Objectives

Different regional codes are in practice of the designers. These codes are not true representatives of the building site and the behavior of the building is also unknown during a seismic hazard. The prime objective is to study the behavior of CBD and PBD of structure located in Zone 2B and soil type SB. The specific objective are:

1. To compare seismic performance of CBD and PBD with steel reinforcement as design variable.

2. To economize steel reinforcement of the structure.

## 1.4 Scope of Work and Research Methodology

The research project is based on real architecture seven storey RC located in Zone 2B for soil type SB. The building reinforcement is designed according to UBC-97 and ACI-318-11 and the design model is named as CBD. Capacity of the CBD model is checked by using nonlinear static pushover analysis. In PBD approach the reinforcement in beams is intentionally reduced and its performance is studied against different levels of earthquake (Service Level Earthquake, Design Based Earthquake, and Maximum Considered Earthquake) to achieve economy and ensure safety at the same time. Total 12- number of models are made, six for CBD and six for PBD.

TABLE 1.1: Number of models created for this study

| Sr. No. | Model | EQ level | X-direction | Y-direction |
|---|---|---|---|---|
| 1 | Code Based Design | SLE | 1 | 1 |
| | | DBE | 1 | 1 |
| | | MCE | 1 | 1 |
| 2 | Performance Based Design | SLE | 1 | 1 |
| | | DBE | 1 | 1 |
| | | MCE | 1 | 1 |
| | | Sub-total | 6 | 6 |
| | | | Total | 12 |

Nonlinear static pushover analysis is opted to assess seismic behavior of the building performance as per guidelines described in FEMA-273 and ATC-40. Performance level of the building is assessed by studying formation of plastic hinges and parameters such as inelastic demand curve, storey shear, storey displacements, storey drifts and overturning moments. Effect of reduction of reinforcement in each model is also assessed. Based on the comparison, most suitable approach which results economical design yet satisfying desired seismic performance, has been recommended.

## 1.5 Limitations of the Study

The limitations of the current research study are:

1. Non-linear hinges have been assigned only at specified locations.

2. The quantity of flexural reinforcement reduction has been compared only for beams.

3. The study has been conducted using linear static analysis (LSA) and non-linear static procedure (NSP).

4. The study of connections bond and slip failure is out of consideration in this research.

5. Higher mode dominant building and application of non-linear time history analysis is beyond the scope of this study.

## 1.6 Thesis Outline

Chapter 1: In first chapter, research gap has been identified. Motivation and problem statement is discussed. Limitations, scope of work, and research methodology have been outlined.

Chapter 2: Literature review available on the PBSD methods is presented. All the methods are explained along with their examples from literature and their advantages and disadvantages. As the analytical method will be used for this study so this method is presented with more detail. Various elements of behavior of structure from the perspective of optimal design are discussed afterwards.

Chapter 3: In this chapter, details regarding linear and non-linear models are included. Design of Intermediate Moment Resisting Frame (IMFR), assignment of non-linear hinge property is also described. A 7-storey building is designed as case study using Equivalent Static Analysis (ESA) and then non-linear design technique is applied.

Chapter 4: In this chapter, comparison of different seismic response parameters like storey shear, storey moment, storey displacement, storey drift etc. have been made. Development of hinges and their states has been studied and economization in term of reinforcement has been discussed.

Chapter 5: This chapter covers the summary of the whole research work performed. Conclusions of the research work have been portrayed and future recommendations are presented.

# Chapter 2

# Literature Review

## 2.1 Background

This chapter narrates a detailed literature study on response of reinforced concrete (RC) structure against seismic hazard. The response of main RC elements due to the applied seismic forces and its design capacity has been discussed. To evaluate the damage phenomena and design capacity, studies conducted by different researchers is presented. A brief background on the topic follows the description of all broadly used methods of seismic assessment and their effectiveness to RC structures is reviewed further to come up with a suitable method having ability to simulate the damage potential. Different methodologies used for seismic assessment is also discussed. As analytical method is utilized in this research thesis so this method is mainly focused.

## 2.2 Reinforced Concrete Frame Structure

Modern RC structures are built with ductility in their main elements. Therefore, such RC structures are able to move to and fro during a seismic event, and to survive with acceptable damage, but without total collapse. Moment resisting RC frames are used as seismic force resisting system for design of earthquake-resistant structure. Columns, beams and their joints are detailed with such amount of reinforcements that resist shear, flexural and axial actions. As a reaction the building sway back and forth multiple cycles in an event of earthquake ground shaking. During an earthquake as the building moves backward and forward, the damage is distributed over the height. If the structure holds weak columns, drift gravitate to focus on particular stories (Figure 2.1 (a)). Resultantly the drift may surpass the columns drift capacity. On the contrary, if columns provide firm support throughout the height of the building, drift will be distributed uniformly (Figure 2.1 (b)) thus minimize the chances of localized damage. Further, it is necessary to understand that the columns in a particular storey carries the entire building weight above those columns. On the other hand beams supports only gravity of that particular storey, therefore column failure is of more danger than beam failure. Due to this phenomena, building codes states that columns must be built stronger in frame as compare to beams. This principle is known as strong column/weak beam which is essential to accomplish safe behavior during seismic hazards. Studies have shown that the full structural mechanism of Figure 2.1 (c) can be only achieved if column to beam strength ratio is relatively large.

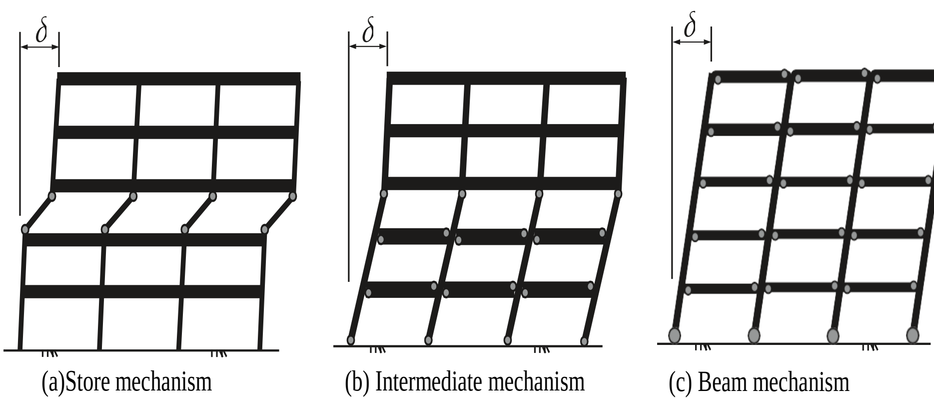


FIGURE 2.1: The three main frame mechanis to avoid(a) and achieve (b) or (c) (Moehle and Hooper, 2016)

Inducing ductility requires that members yield in flexure and avoid shear failure. Shear failure in column is avoided because it leads to brittle failure and can result into loss of axial load carrying capacity. Using capacity design method it is decided which object within a structural system will be designed as ductile component and permitted to yield and which object as brittle component and remain elastic.

Ductile components are designed with sufficient deformation capacity while brittle components are designed to achieve sufficient strength levels.

## 2.3 Design Optimization Method

The basic objective of CBD is that the building will remain in Immediate Occupancy (IO) state against Service Level Earthquake (SLE), Life Safety (LS) state against Design Based Earthquake (DBE) and Collapse Prevention (CP) state against Maximum Considered Earthquake (MCE) (UBC-97, 1997). These objectives are called performance objectives. Code based design (CBD) approach does not directly address the response of structure against seismic forces thus cannot predict the behavior of structure and its failure during earthquake (Zou et al. 2007).

The prime goal of Performance based design (PBD) approach is to enhance the safety of the structure against seismic hazards, and to resist seismic events in a computable manner to pre-define levels of possible damage (FEMA-356, 2000). FEMA-356 establishes criteria for design of new building against seismic hazards with performance constrains. The numerical results of Performance base seismic design demonstrate non-linear analysis which studies characteristics of the RC structure and its seismic response which gives designer better understanding of the structure and results into better seismic design (Nikos D. Lagaros et al. 2006). Non-linear static pushover analysis (PoA) and non-linear dynamic time history analysis are two main inelastic analysis methods which are much applicable analysis procedures for assessment of structural response against seismic hazards (Nikos D. Lagaros et al. 2006). Fredrick et al. (2015) analyzed the performance of regular and irregular buildings using non-linear PoA. It was observed that in regular building damage is uniformly distributed in comparison to the building which is not regular. The soft storey effect occurs in the lower stories where the collapse hinges are located.

Several studies on the optimization of reinforced concrete structure against seismic demands have been carried out. An Optimal design is set to economize the structure yet satisfying performance objectives of reinforced concrete frame set by building codes (S. Ganzerli et al. 2000). In optimization of structure using performance based design the design objective can be divided into two parts, initial material cost economization and expected behavior of structure in event of earthquake (Foley, 2002).

Ganzerli et al. (2000) Studied performance levels (IO, LS and CP) of reinforced concrete intending to minimize the structural for optimal design. 2D reinforced concrete frame on which nonlinear pushover analysis was performed by applying the lateral force monotonically. Using nonlinear analysis IO level was achieved just beyond the yielding point, this show that Structure had enough reserved capacity to which can be optimize for achieving minimum cost.

Vu phan et al. (2007) tested two large scale columns under near-fault ground motion on a shake table. The results showed there is still enough residual capacity after strong ground motion. Vijayakumar and Venkatesh (2012) conducted pushover analysis on existing buildings in India. It was noted that the structure most of the hinges developed were falling below immediate occupancy level and few went to life safety level. They concluded that local code is may be overestimate the seismic demands.

Fragiadakis et al. (2006) designed moment resisting reinforced concrete frames and analyzed them by using non-linear response history analysis. From their design and analysis they concluded that probabilistic and deterministic both approaches result into better seismic performance as compared to prevailing code design practice. Further economy can be achieved by reliability-optimization technique.

Mamaghani and Khaloo (2019) studied glass fiber reinforced polymer (GFRP) bars in four concrete moment frame by assessing its seismic behavior using pushover analysis. Using ATC-40 criteria the frames Performance level were assessed. Strength and ductility of moment frame reinforced with GFPR bars is compared with steel bar reinforced frames. Based on the results, they concluded that under seismic loads GFRP reinforced frames exhibit higher strength than steel reinforced frames. Moreover the drift ratio of GFRP reinforced frame was fond to be less than 2% and is almost the same as steel reinforced frame.

Mehmet and Hayri (2006) studied default (auto) and used defined (manual) hinges. Pushover analysis on 4 and 7 storey structures with default and user defined hinges were performed. It was demonstrated that used defined hinges model performed better than default hinges model.

Fragiadakisa and Lagaros (2011) designed 4-storey steel braced frame using performance based design procedure. They demonstrated that structural design performance objectives set by codes can be verified when non-linear analysis procedure are adopted to see if the performance objectives set by code are fulfilled by the design.

Chun and Zou (2005) carried out PBD using a technique called Optimality Criterion (OC) technique which is established for RC structural design optimization. The results exhibit that PBD approach can effectively achieve economical design of reinforces concrete structure subjected to drift limits and performance levels. In addition uniform ductility can be achieved over all the stories which overcomes the development of soft story mechanisms

## 2.4 Seismic Assessment of the Structure

Prediction of occurrence of an earthquake event and quantification of the damage caused by it always remained a challenge for scientists and engineers. Now with the advancement in technology and with increasing research on the topic, we have begun to understand what actually earthquake is, and have devised methods to evaluate and design structure in accordance to modern earthquake engineering.

Whitman et al. (1973) first time attempted to quantify the potential of damage after 1971 San Fernando earthquake and compiled statistically, the damaged data of over 1600 buildings having 5 or more storeys. The concept of damage ratio

(DR) was first time introduced after Whitman et al. (1973). DR is now used as economy damage indicator, is simply the ratio of repair and replacement value (Kyriakides 2007). Different methods for assessment of structure under seismic condition have been developed are having different level of details and precision. The choice of the method mainly depends on the assessment objectives, available technology and data. These methods provide with quantification of the damage for different levels of seismicity which differ for each method (Kyriakides, 2007).

In this age of computer, availability of different FEA software along with versatile inelastic elements modeling its behavior under seismic conditions allows the designers estimation of damage against certain performance levels. Simple and detailed analytical procedures are available in literature for vulnerability assessment. Main difference between the two is the refinement used for the building modelling.

## 2.5 Analytical Vulnerability Assessment

Simple methods just rely on simple equation rather complete analysis of the building to drive its capacity. Simple methods were evolved basically to be able to analyze large number of structures in a short time (Kyriakides, 2007). Therefore, modelling should be based on a very few input parameters including number of storeys, construction period and material. For reliable results these input parameters must be able to apprehend the behavior of the structure in Earthquake events. Clavi (1999) proposed a simple procedure for seismic assessment based on a concept of assessing the building displacement capacity analogues to certain limit states and earthquake displacement demand from related displacement spectrum. The model was based only on certain parameters (construction period, storeys, and material of construction etc.) and four limit states were applied for structural and non-structural damages (slight non-structural damage to collapse). Force displacement relationship (capacity curve) was obtained after idealizing each building as single degree of freedom (SDoF) system, based on simple equations of yield and ultimate capacity. For minimum and maximum drifts, the secant periods and their related ground displacement was obtained using particular damage state equations. These points drawn on displacement demand spectrum, the ratio of the area above and below this line represents probability of occurrence of a particular damage state of the building. In another attempt FEMA (1997) funded National Institute of Building Science (NIBS) for the development of a simple analytical procedure which after some modification resulted in software for risk assessment known as HAZUS99 (1999). Thirty six buildings having categorized on the basis of structural system, height, level of seismic design, usage and seismic zone were considered. Four damage states (slight, moderate, extensive and complete damage) were used to express damage potential. The capacity spectrum method (CSM) of ATC-40 was utilized for the evaluation of performance point (PP) at the superimposition of demand spectrum and capacity curve (Figure 2.2 (a, b)). Spectral displacement (SD) was used as a hazard parameter to define variation in damage states and the threshold values for each buildings assigned by the experts. Evaluation of the buildings exceeding a particular damage state was extracted using performance point (PP).

When detailed information are required and the structures with certain importance or having no empirical data available, are assessed for seismic damage then more thorough detailed analytical vulnerability procedures are used. These procedures with more refined models are time consuming and are used for the seismic evaluation of individual buildings. Such procedures are unsuitable for projects involving seismic evaluation of large number of buildings (Lang, 2002). New simple analytical methods can be developed by using concepts behind them. This method mainly depends on the determination of earthquake hazard parameter, determination of structural response through modelling and analysis, and finally relating this structural response with the damage and capacity. These procedures are classified as linear (static and dynamic) and nonlinear (static and dynamic).

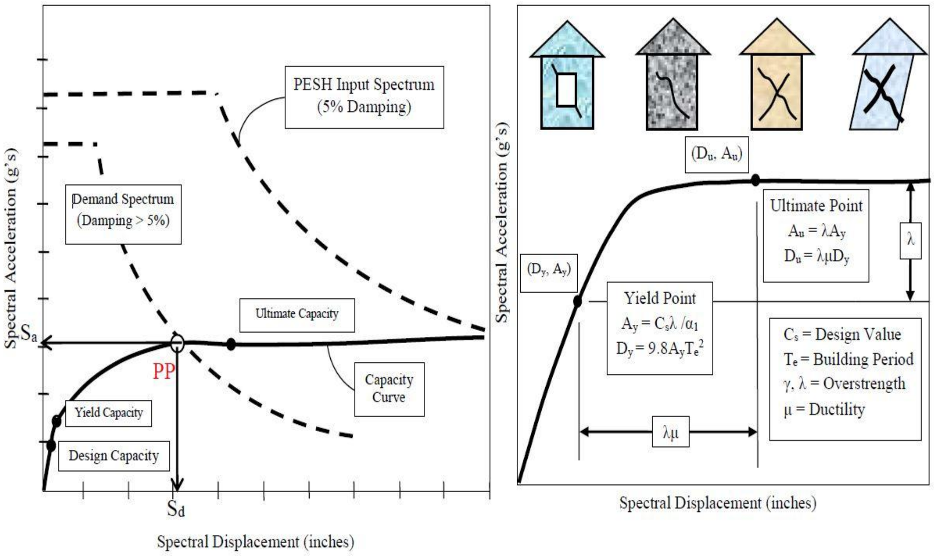


FIGURE 2.2: a) PP evaluation using ATC-40 CSM in HAZUS (HAZUS, 1999)
b) Determination of Capacity spectrum (HAZUS, 1999)

## 2.6 Linear Static Analysis (LSA)

Linear static analysis (LSA) is the most frequently used method for analysis of structures. Buildings are modeled as a Single Degree of Freedom (SDoF) system in linear static methods along with linear elastic stiffness and equivalent viscous damping and the input of seismic excitation is modeled by an equivalent lateral force. Spectral acceleration (SA) is determined from the appropriate response spectrum and on the basis of the estimation of fundamental frequency mode using Rayleighs method or some empirical relation. Equivalent lateral force is then obtained by the multiplication of this SA with the building mass as follows:

$\mathrm{V} = \mathrm{Sa}{*}\mathrm{m}{*}\Sigma\mathrm{Ci} - - - Equation(2.1)$

Issues of second order effects, stiffness degradation are addressed by the coefficient Ci in the above equation. Displacements and internal forces corresponding to the applied lateral force are determined after distributing this lateral force over the full height of the building using linear elastic analysis (LSA). LSA procedures find their extensive use in most building codes for the seismic analysis and design. They are applicable only to regular buildings having predominant first mode of vibration i.e. responds in its fundamental lateral mode (Lang, 2002).

Contrary to the linear static procedures, in linear dynamic procedures the buildings are modeled as a multi degree of freedom (MDoF) system with linear elastic stiffness and equivalent viscous damping like static procedures. Seismic inputs are modeled as timehistory analysis or modal spectral analysis.

UBC-97 permits design of a structure using equivalent static force procedure or a dynamic analysis for not more than 240 feet tall in case of regular structures and 65 feet tall in case of irregular structures. When the structure height exceeds the limit of 240 feet in case of regular structures, 65 feet in case of irregular structures and in case of buildings which are located on soil type-SF and having a time period (T) more than 0.7 seconds, dynamic response spectrum analysis is required. The equivalent static force procedure is most commonly used for the case of regular structures. For irregular structures dynamic analysis must be adopted (Di Julio, R. M. 2001; ACl-318, 2011).

## 2.7 Nonlinear Static Analysis

Nonlinear static procedures are most extensively used by most of the detailed analytical procedures, developed to overcome the complexities involved in the use of nonlinear dynamic analysis procedures. Many design assessment codes (ATC-40 and FEMA-356) propose nonlinear static procedures. In all of these methods building capacity is expressed by push-over curve (Figure 2.5.3). In ATC-40 the highly damped spectra and in FEMA-356 displacement coefficient method is used to compute the maximum displacement that is expected to be experienced by the building in case of a seismic event. FEMA 440 made an attempt to improve these procedures and finally achieve considerable achievement in for ATC-40.

Equivalent static lateral force distribution is used to approximate the seismic demand. An adaptive static analysis procedure was proposed by Antoniou (2002) for the efficient representation of the higher mode effects. At each increment of load the distribution of lateral loads is updated based on the modal characteristics, instantaneous structural stiffness and the resulting seismic demand. The method was utilized in the INDYAS (Elnashai et al. 2000) Finite element analysis tool which was used in the analytical vulnerability assessment study conducted by Rossetto (2005).

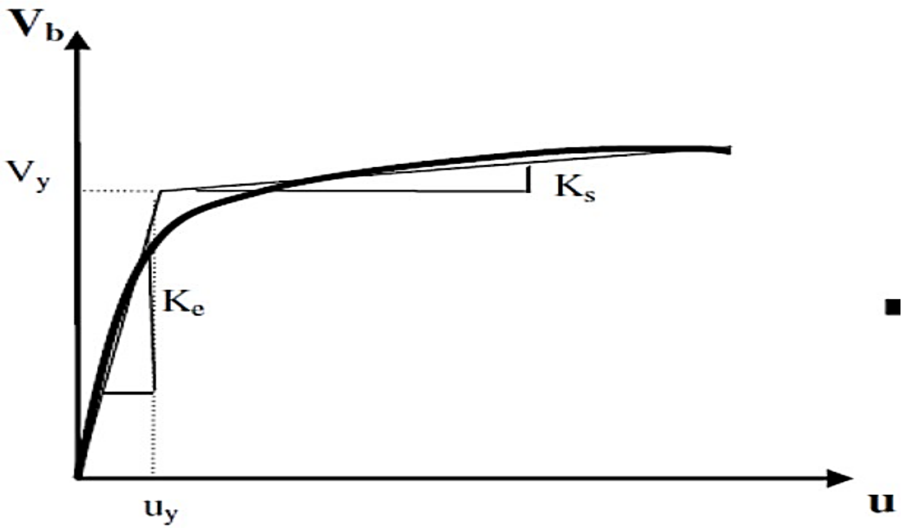


FIGURE 2.3: Capacity curve for MDOF system (Themelis, S., 2008)

A study for the comparison was conducted by Kalkan (2004), and showed that the top storey displacement for a 4 storey building as calculated by him using FEMA 356 (2000) and the Chopra (2002) procedures were in close agreement with the actual displacement obtained by using time history analysis. Although the displacement values decreases in case of FEMA 356 (2000) with the building height, still due to his simplicity and fairly accurate results it can be used as a good alternative method. Many individual researches have been carried out for analytical vulnerability assessment in the previous decade and the previous years of this decade as well (Rossetto, 2005; Kappos, 2007; Kyriakides 2007; Ahmed 2011; Kamran 2011; Qayyum 2012; Aleem 2012) by using capacity spectrum method.

## 2.8 Performance Based Seismic Design (PBSD) Approach

About all modern design code guidelines are targeted to achieve some intended performance level, such criteria are deemed to result in structures capability of achieving acceptable performance without clearly defining the performance expectations. Furthermore, the engineer using such prescriptive procedures does not explicitly verify the capability of the structure to attain the intended performance. As a result, structural engineers currently are unable to put in their full potential to the process of design. The designers are falling into limits of codes, Who add value by being able to negotiate the complication of prescriptive provisions promptly, rather than by applying crucial creative and inventive solutions to multi-faceted structural engineering problems. As a result, the stakeholders served by the expertise of designers are not getting maximum value from their limited resources of time, money, energy, and materials. Rather, the stakeholders are getting proposals that are limited by local codes that are generic, with unpredictable reliability because such design neither enumerates nor directly evaluates the performance. PBD is a process that enables the analysis and design of structures that will have known performance when subjected to defined loading. PBD turns the traditional design paradigm upside down in the sense that the required performance is the initial point for the design. Considering the desired performance of the structure and selecting the scenarios that match the goals for structural function in the presence of a specific hazard, the designer works toward achieving that stated, desired goal. The performance of the design is demonstrated through analysis, simulation, prototype testing, or a combination thereof. PBD is based on the principle that structural systems and the nonstructural systems they support, must meet specific performance objectives. The basic steps are as follows:

- Establish the performance objectives,
- Conduct initial design, and

- Verify performance through analytical simulation, prototype testing, or a combination thereof.

The work breakdown chart of performance based design methodology is illustrated in figure 2.4.

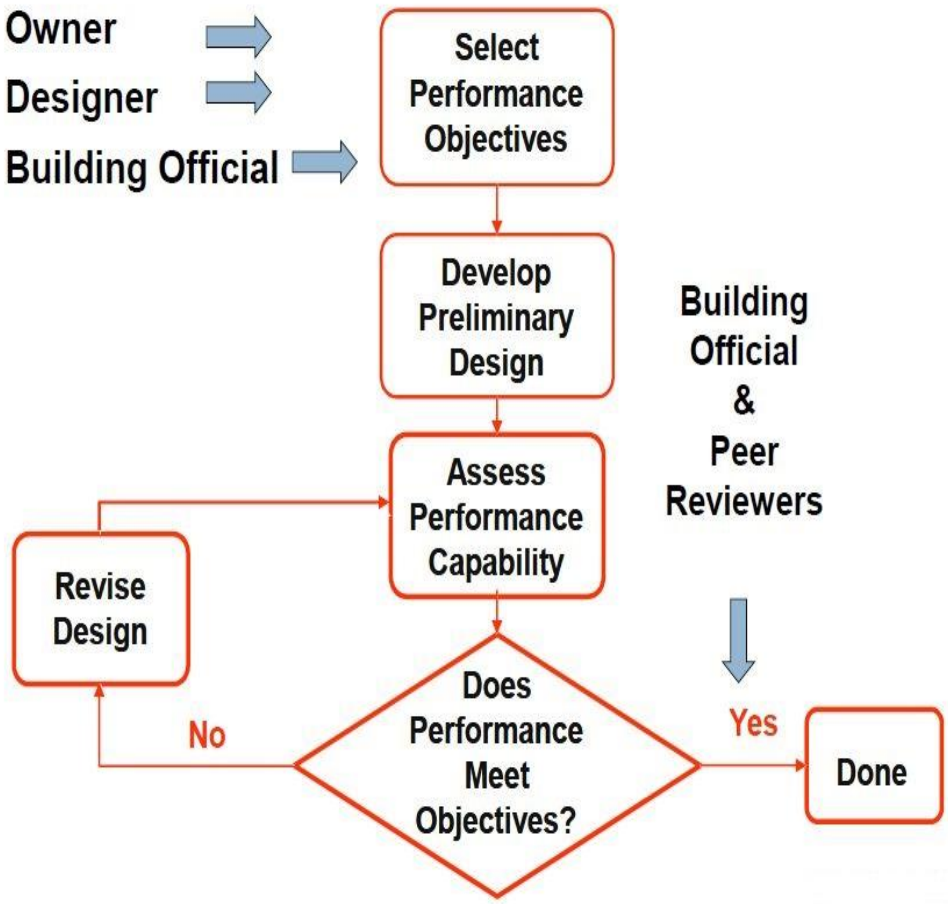


FIGURE 2.4: work breakdown chart for performance based design procedure.

Pre-set performance objectives are key to the procedure, as it establishes the goals for the design. Performance objectives may be qualitative (Structural Performance level) when working with clients or building owners, but engineering practice requires quantitative criteria (Structural Hazard level definition) for design and evaluation. Such performance objectives most often include statements of the likelihood that a damage level or service state will be exceeded over the structures life or if a specified event occurs. Examples of both types of performance objectives include the following:

TABLE 2.1: Structural Performance level definition (FEMA 356, ATC 40)

| Performance Level | Description |
|---|---|
| Operational (O) | Negligible impact on building. Building can be occupied |
| Immediate Occupancy (IO) | Building is safe to occupy but will need little repair work |
| Life Safety (LS) | Building is safe during the event but possibly not afterward |
| Collapse Prevention (CP) | Building is on the verge of collapse, probable total loss |

TABLE 2.2: Structural Hazard level definition (FEMA 356; ATC 40)

| Hazard Level | Description |
|---|---|
| Frequent, minor EQ (Service level EQ) | Return period: 100 years (43% probability of occurrence in 50 years) |
| Infrequent, moderate EQ (Design basis EQ) | Return period: 500 years (10% probability of occurrence in 50 years) |
| Worst EQ ever likely to occur (Maximum considered EQ) | Return period: 2500 years (2% probability of occurrence in 50 years) |

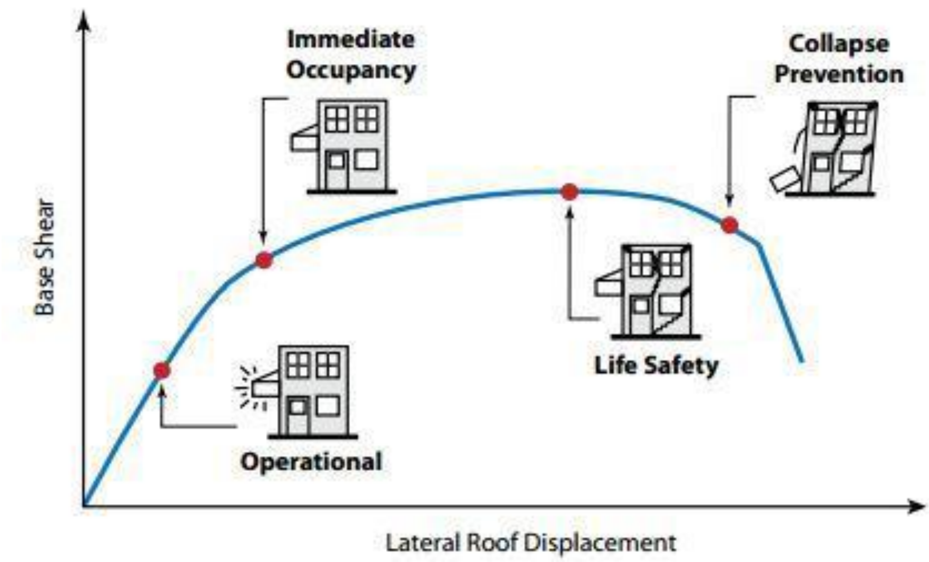


FIGURE 2.5: FEMA 273/356 performance levels

By starting with the end goal in mind, structural engineers have more flexibility and opportunities to add value, as well as to develop innovative solutions. With performance objectives set in advance, the engineer develops a design that can be verified through analytical or physical means. This establishes the mechanism by which structural performance can be assured. Once performance is verified against the performance objectives, the design can be completed, and implemented with confidence.

## 2.9 Summary

As observed in the above researches designers have made serious efforts to find cost-efficient solutions to engineering problems. Buildings are designed by applying the CBD Approach. Resultantly, designers end up with uneconomic sections and reinforcement. Further, the behavior of buildings is unknown under seismic events. Therefore, nonlinear analysis be applied to assess the behavior of the structure under seismic effects. In most studies PBD approach has been used in developing structural optimization technique. However, the performance based approach using prevailing CBD from the perspective of their effectiveness in terms of structural members optimization still needs to be explored in detail. In this research, based on the available literature study and FEMA guideline an attempt has been made to optimize structural members of reinforced concrete frame structures using PBD approach.

# Chapter 3

# Design and Methodology

## 3.1 Introduction

Prediction of structural response required most appropriate structural modelling, analytical tool with capability of simulating all the expected failure modes and a most suitable methodology for determination of damage potential for different seismic scenarios considering probabilistic nature of the problem. This chapter is commenced with brief description of the design model carried out in widely used design and analysis software SAP2000. A brief description of suitable element models to evaluate and verify their effectiveness against seismic excitations is elaborated in this chapter.

Reinforced concrete construction constitute 10-15% of the building stock and is on the rise from last two decades, especially in the major cities of Pakistan (i.e. Peshawar, Islamabad, Faisalabad, Lahore and Karachi). The RC buildings are mostly constructed in urban areas their total amount in sub urban areas is very low (1 to 2%), (Badrashi et al. 2010). Majority of buildings are regular in plane so, only regular buildings is considered in present study. In the current study, a midrise structure has been designed as case study using prevailing codes. The case study has been done in seismic zone 2B with soil profile SB.

## 3.2 Description of the Building

After detailed literature review, the very next task is to design the buildings and to do analysis. The designed structure is a midrise seven storey reinforced concrete (RC) frame building located in Zone 2B as per UBC-97. The research models consist of a Lower Ground plus Ground plus 5 storey commercial frame structure. The storey height is 11ft and geometry is such that spans in Y direction is 20 feet and spans in X direction is 21 and 18 feet respectively.

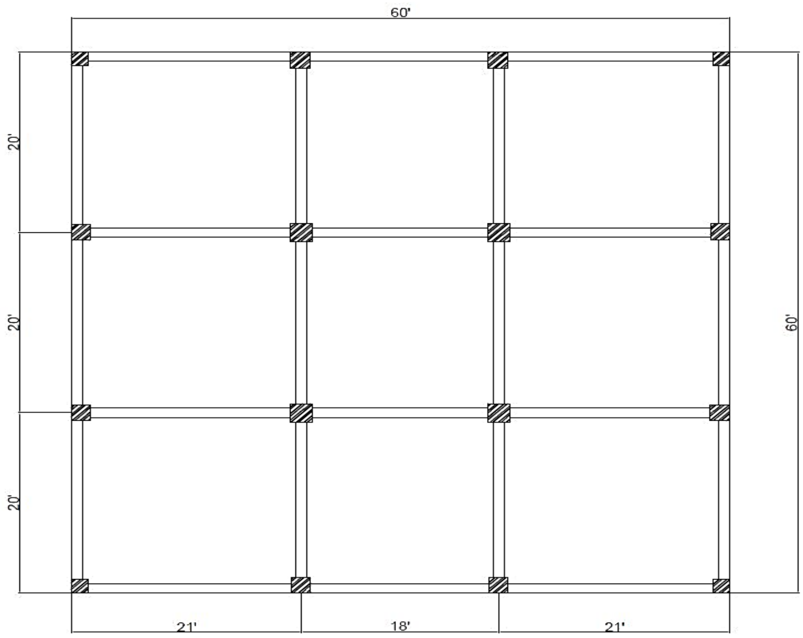


FIGURE 3.1: (a) Architectural Plan of bilding.

Framing type of the structure is Intermediate Moment Resistant Frame (IMRF) with R factor 5.5 and importance factor for the structure is 1. The material properties are: concrete with compressive strength (fc) 3000psi for beams and slabs and 3500psi for columns with steel reinforcement of ASTM A615 Grade 60 (Fy =60 Ksi).

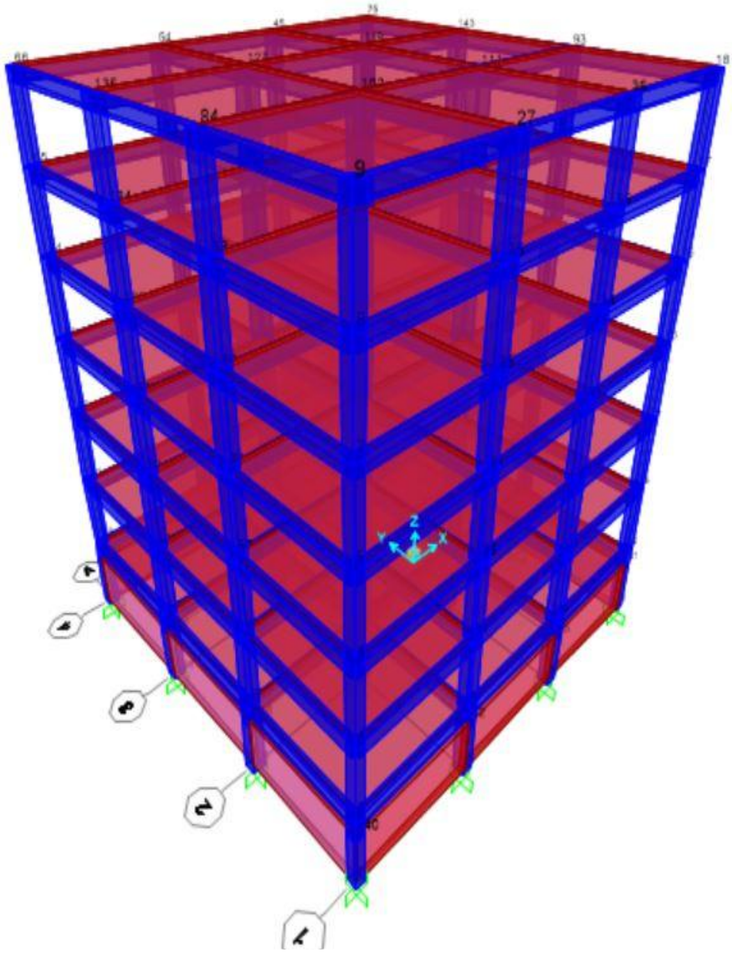

FIGURE 3.2: 3D view of the building

## 3.3 Linear Static Analysis (LSA) and Design of Structure

Linear static analysis (LSA) is the most universally used method for analysis of structures. Buildings are modeled as a Single Degree of Freedom (SDoF) system in linear static methods along with linear elastic stiffness and equivalent viscous damping and the input of seismic excitation is simulated by an equivalent lateral force. The building is free of any type of irregularity. Modal analysis is first done to examine the vibration modes of the building. From modal analysis, the modal participation factor for the building is first mode dominant as the mass participation in first mode is found to be 79%. Thus, the building fullfills the limitation for use of LSA.

The Software used in this study for the modeling, analysis and design of frames is SAP2000. SAP is widely used as practical structural software that allows modeling, analysis and design of structures constructed with different materials. It provides a platform for both 3D and 2D modeling of structures. SAP2000 has integrated SAP-Fire analysis engine that uses a sophisticated finite-element analysis (FEA) procedure. The structure is modeled as IMRF with RC beams and columns modeled as frame elements and RC slabs modeled as shell elements. The connections used or modeling the joints are fully rigid. The gross moment of inertia (Ig) as per ACI-318 for beams and columns are taken 35% and 70% respectively. Fixed supports are assigned at the base for soil and super structure interaction. The response modification factor (R) is taken 5.5 for building frame system. Linear static analysis (LSA) as per UBC-97 is used for analysis of building in seismic zone 2B and soil profile SB. Structure is designed against gravity loads and seismic load combinations. Loads are applied as per ASCE-07-05, for commercial floor loads are taken as per UBC-97. In accordance to section 1630.1.1 of UBC-97 the seismic mass source incorporates 25% off the live load and 100% of the structure dead load (self-weight, finishes load and partition load). The structure is designed to resist two types of loads mainly, which are gravity and lateral loads for evaluation of seismic performance.

The effective time period (Te) of the building is calculated in both X- direction and Y-direction. Code based response spectrum analysis is then performed using UBC-97.

## 3.4 Developing Non-linear Model

The non-linear model designed according to codes (UBC-97 and ACI-318-11) is named as Code based design (CBD) Model. For nonlinear static analysis nonlinearity is induced in CBD model. Inducing non-linearity involves assigning plastic hinge mechanism in structure at suitable locations. The model with intended optimized reinforcement is named as Performance based design (PBD) Model.

Service level earthquake (SLE), Design based earthquake (DBE) and Maximum considered earthquake (MCE) response spectrums with seismic zone 2B is used for response spectrum analysis (RSA).

### 3.4.1 Assignment of Plastic Hinges

In SAP 2000, there are two types of hinge definitions: i) default auto hinges and ii) user-defined hinges. default auto hinges includes type of hinges like Moment M3, Moment M2 hinges, Axial P, Shear V2, V3 and Torsion T hinges independently to frame and shell elements. These hinges are uncoupled and can be utilized individually. Also, the interacting P-M2-M3 coupled hinge is a default auto hinge type. Default auto hinges are based on geometry of structural member and area of reinforcement calculated from LSA. In user-defined hinges, different parameters and acceptance criteria is defined by the user as per requirement. In user-defined hinges, exact reinforcement resulted from static analysis is to be modeled to get the values of moments and rotations. While in auto hinges, values of positive and negative moments are taken from results of static analysis. Plastic hinges can be assigned at any point through the member but are usually assigned to critical and physical admissible locations.

For beams, M3 hinges (Auto) are defined and assigned at both ends of each beam. Figure 3.3 shows auto M3 hinge detail defined in SAP2000. Acceptance criteria for IO, LS and CP levels as per FEMA 356 and relationship for displacement controlled parameters, moment and rotation, for a typical hinge is shown in Figure 3.4.

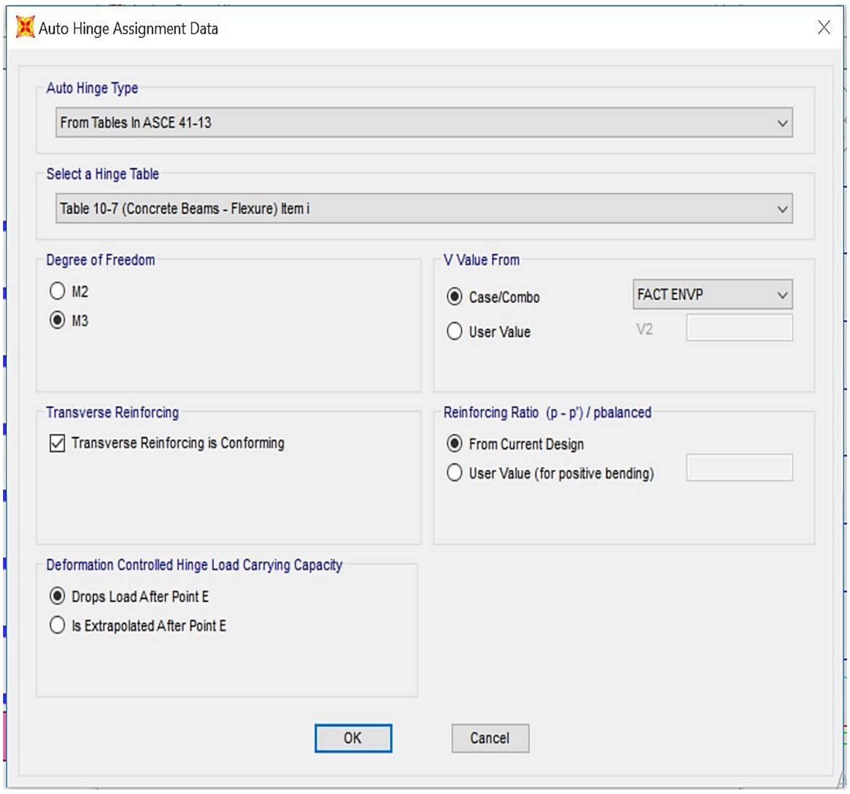


FIGURE 3.3: Definition of moment M3 hinges for beams.

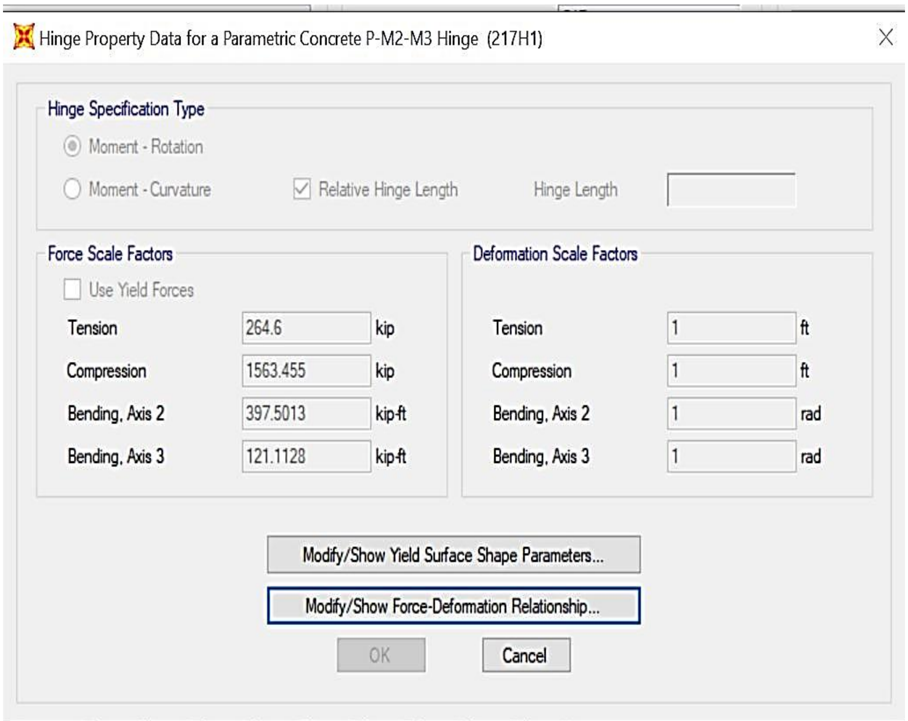


FIGURE 3.6: Column P-M2-M3 Hinge property detail.

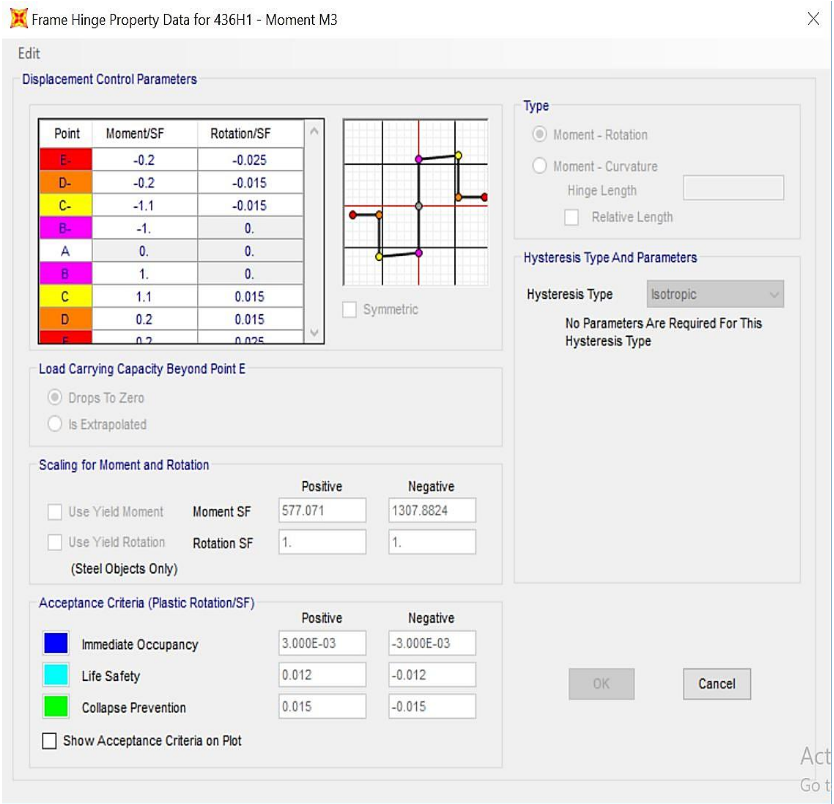


FIGURE 3.4: Beam Auto Hinge M3 property detail.

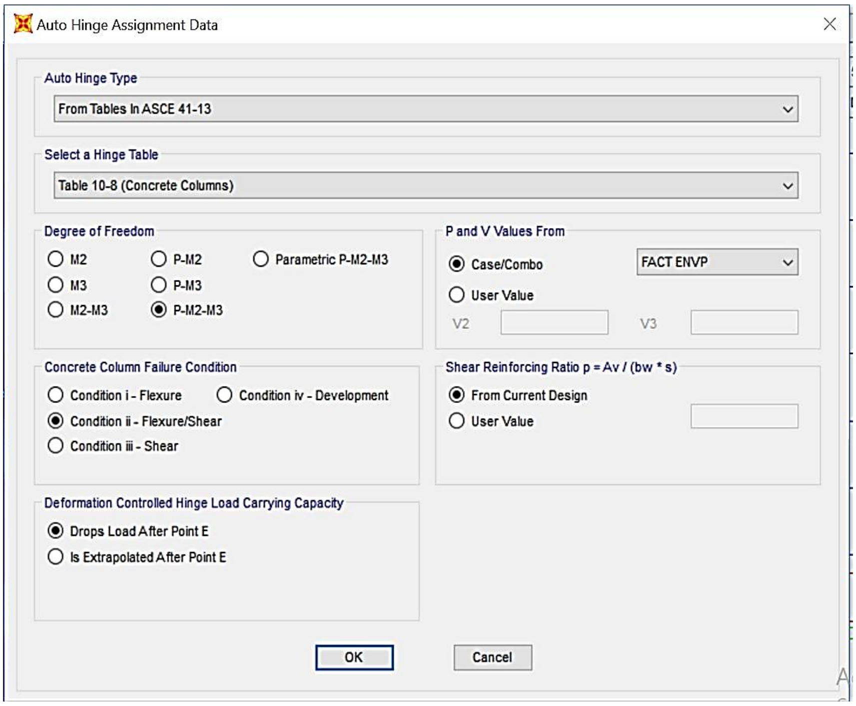


FIGURE 3.5: P-M2-M3 hinge definition for columns

It can be noted that the typical hinge shown in Figure 3.4 is taking a positive moment of 577 kip-in and a negative moment of 1307 kip-in. To verify either the hinge is taking the right amount of moment or not, the moment was calculated manually using Equation 3.1. It comes out to be positive moment of 594 kip-in and negative moment of 1329 kip-in which shows the hinge is performing well and taking approximately same amount of moment.

$M = A_s f_y (d - \frac{A_s f_y}{0.85 f_c' 2b}) - - - Equation(3.1)$

where:

M = Resulted Moment (kip-in)

$A_s$ = Area of steel (in$^2$)

$f_y$ = Yeild strength of steel (ksi)

$f_c$ = Compressive strength of concrete (ksi)

d = effective depth of beam (in)

b = width of beam (in)

P-M2-M3 auto hinges is selected for columns. P-M2-M3 coupled hinge is a default auto hinge type, and like other default hinges it is based geometry of structural column and area of reinforcement calculated from linear static analysis. For column, P-M2-M3 hinges (Auto) are defined and assigned at both ends of each column. Figure 3.4 shows (beam) auto M3 hinge detail defined in SAP2000. Acceptance criteria for IO, LS and CP levels as per FEMA 356 and relationship for displacement controlled parameters, Axial Load P, M2 and M3 moment and detail are shown in Figure 3.6.

### 3.4.2 Plastic Hinge Length

The performance of plastic hinge is critical to the members load carrying and deformation capacities. In modern age, many finite element analysis (FEA) tools for the nonlinear analysis of structures are available. Most of these tools use elasticity models by defining finite element hinges at the ends of elements to introduce nonlinearity. Elements plasticity is just an approximation of the true plastic hinge zone, and this may distribute significantly both in the member and in the joint.

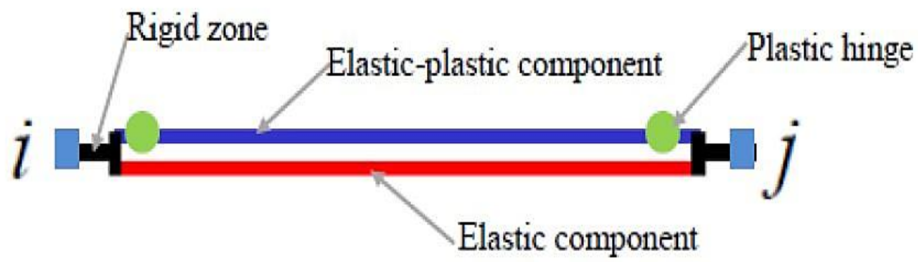


FIGURE 3.7: Elastic and elastic-plastic element combination (Ahmad, 2011)

The term hinge length (Lp) is not the actual length, but it is the region over which the plasticity spreads. (Park and Paulay 1975).

Plastic hinge length has substantial effect on deformation capacity of a structure. Although plastic hinge length does not affect base shear capacity but has substantial effect on displacement capacity of RC frame structures (Zhao et al., 2011). A number of plastic hinge length expressions have been proposed by different researchers as shown in. Upto 30% variation in displacement capacity frames is observed when different plastic hinge lengths $L_p$ are used (Inel and Ozmen, 2006). Effective plastic hinge length can be calculated using the formula proposed by Priestley et al. (1996) as expressed below in Equation 3.2 and is also used in guidelines of ATC-32.

$l_p = 0.08l + 0.15 d_b f_y - - - Equation(3.2)$

where:

$l_p$ = Plastic hinge length (in)

l = distance from critical section to point of contraflexure (in)

$d_b$ = Dia of rebar (in)

$f_y$ = Yield strength of rebar (ksi)

Alternatively, $l_p = 0.5h$, where h is the depth of the member is the easiest and the simplest expression for plastic hinge length and can be used for typical beam and column section (Park and Paulay, 1975).

## 3.5 Optimal Inelastic Design Methodology

In seismic design the codes intend that the building will linear-elastic when subjected to minor earthquake and will behave nonlinear-inelastic under moderated and severe earthquake. FEMA has set proper guidelines for seismic behavior of structure against hazard levels. The building will remain in IO performance level against SLE, in LS performance level DBE and will remain in CP performance level against MCE. The performance level of building can be analyzed by the drift limits defined by FEMA-356, ATC 40, by performance level of individual hinges assigned to the structural elements.

Table 3.1: Target Displacements for Soil profile SB.

| Performance Level | Immediate Occupancy | Life Safety | Collapse Prevention |
|---|---|---|---|
| Drift Limit | 1% | 2% | 0.33Vi/Pi≈3-7% |

If only one span (beam) reaches the point of collapse, then the whole structure should be regarded as partial collapse and the particular storey should be considered as overall collapse (Structural Steel Work: Limited State Design). For PBD, the CBD is first analyzed using non-linear static pushover analysis (PoA). The behavior of structure designed by linear-static analysis is studied against the three hazard levels (SLE, DBE and MCE). Against the optimal structural member sizes, the same frame structure is analyzed for soil profile type SB against three hazard levels (SLE, DBE and MCE). In handling the performance level of assigned hinges and drift limits the application of flexural reinforcement in beam is studied. In PBD phase the member sizes are kept constant and the flexural reinforcement are considered as design variables subjected to the performance constrains of hinges and drift limits. Transverse shear reinforcement is not considered variable, therefore, sufficient shear strength is provided. The emphasis of this research thesis is on the performance based design optimization in term of flexural reinforcement.

Figure 3.8 shows the flexural design input option used in performance based design procedure. The top and bottom reinforcement from both ends of the beam is first recorded from linear static analysis. The derived reinforcement from linear static analysis is than minimized or maximized accordingly to achieve the desire performance objectives. The top and bottom reinforcement variation is taken linear to each other.

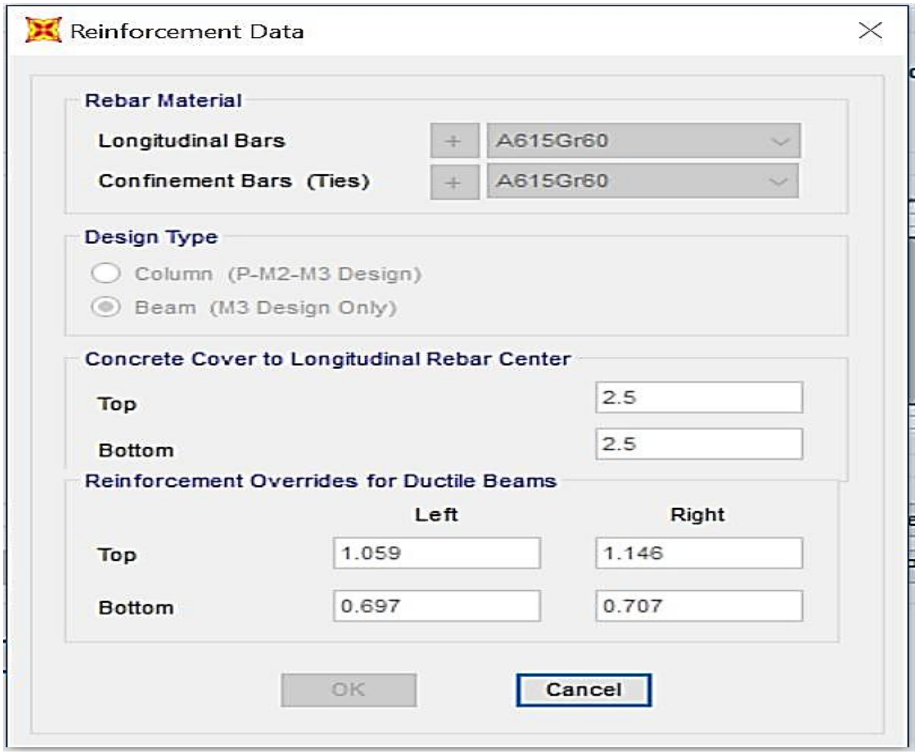


FIGURE 3.8: Beam reinforcement input detail.

## 3.6 Push-over Analysis (PoA)

Non-linear static Pushover analysis (PoA) is opted to assess seismic performance of the building performed as per guidelines described in FEMA-273 and ATC-40. PoA is most suitable for systems in which behavior of the building is dominated by the fundamental mode. When higher-order modes contribute as in skyscraper buildings, dynamic analysis is most effective. PoA is a static-nonlinear procedure in which a structural system is subjected to a monotonic load which increases iteratively, through an ultimate condition, to indicate a range of elastic and inelastic performance. As a function of both strength and deformation, the resultant nonlinear force-deformation (F-D) relationship provides insight into ductility and limit-state behavior. Deformation parameters may be translational or rotational. Target displacement is calculated using equation 3-3 given in FEMA-356 (2000).

$\delta_t = (C_0 C_1 C_2 C_3 S_a \frac{(T_e^2)}{(4\pi^2)})\mathrm{g} - - - Equation(3.3)$

$C_0$: Modification Factor to relate spectral displacement and likely roof displacement. (FEMA-273, Table 3-2)

$C_1$: Modification factor to relate expected maximum inelastic displacements to displacements calculated from linear elastic response. (FEMA-273, Section 3.3.1.3)

$C_2$: Modification factor to represent the effect of hyestic shape on the maximum displacement response. (FEMA-273, Section 3.3.1.3)

$C_3$: Calculated if the relation between base shear force and control node displacement exhibits negative post-yield stiffness using equation 3-13 (FEMA-273).

$C_3 = 1.0 + \frac{|\alpha|(R-1)^{\frac{3}{2}}}{Te}$

$S_a$: Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

g: Acceleration due to gravity.

The equation given above is used calculating target displacement for DBE. Target displacement for MCE and SLE is calculated by multiplying and dividing the spectral acceleration by 1.5 and 1.4 respectively. The target displacements for soil profile SB is shown in Table 3.2 and 3.3 respectively.

Table 3.2: Target Displacements Push-X.

| Values | SLE | DBE | MCE |
|---|---|---|---|
| Co | 1.4 | 1.4 | 1.4 |
| C1 | 1 | 1 | 1 |
| C2 | 1 | 1 | 1 |
| C3 | 1 | 1 | 1 |
| Sa | 0.110 | 0.155 | 0.232 |
| Time Period (sec) | 1.298 | 1.298 | 1.298 |
| Target Displacement (in) | 2.56 | 3.58 | 5.37 |
| Target Displacement (ft) | 0.213 | 0.298 | 0.447 |

Table 3.3: Target Displacements Push-Y.

| Values | SLE | DBE | MCE |
|---|---|---|---|
| Co | 1.4 | 1.4 | 1.4 |
| C1 | 1 | 1 | 1 |
| C2 | 1 | 1 | 1 |
| C3 | 1 | 1 | 1 |
| Sa | 0.111 | 0.156 | 0.234 |
| Time Period (sec) | 1.29 | 1.29 | 1.29 |
| Target Displacement (in) | 2.56 | 3.58 | 5.37 |
| Target Displacement (ft) | 0.213 | 0.298 | 0.447 |

The time period and spectral acceleration is determined for calculation of target displacement, load case is defined as nonlinear static gravity having loads of 1.2Dead+Livespecial+0.5Live. It is a model load combination and is a specialized type of loading used for pushover analysis. It is a pattern of forces on the joints that is proportional to the product of a specialized mode shape times its circular frequency squared times the mass tributary to the joint. After that nonlinear static pushover case has been defined with target displacement assigned as per UBC-97 section 1633.1. For Pushover in X direction, in Mode-1 100% of displacement is applied while in Mode-2 30% of the calculated is applied, vice versa for Pushover in Y direction.

After the performance of pushover analysis inelastic demand curve is obtained. The inelastic demand curve is representation of base shear against roof displacement. Storey shear, storey displacements, Storey drifts overturning moments, and formation of plastic hinges has been also assessed.

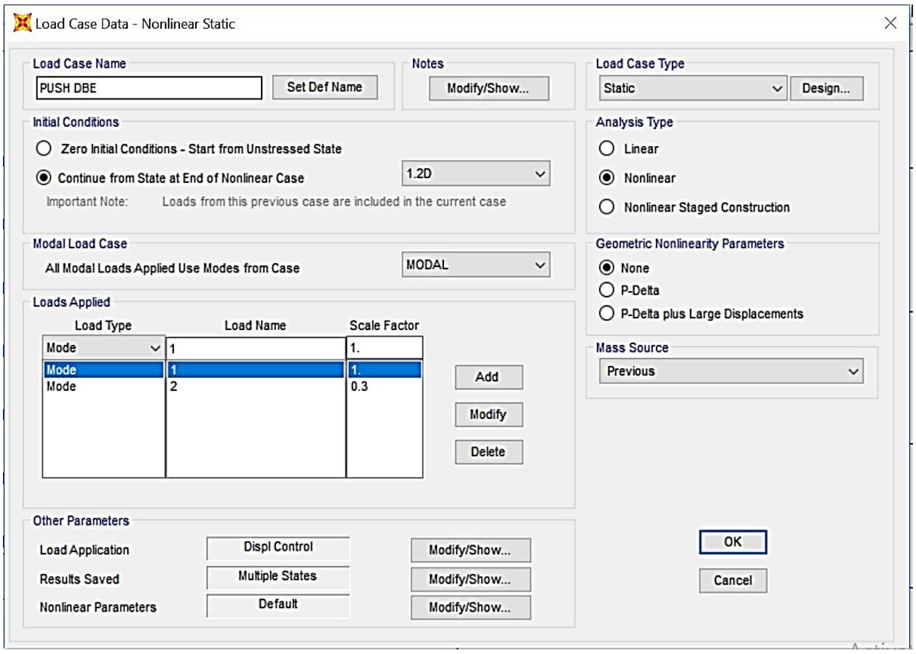


FIGURE 3.9: Nonlinear Pushover Case definition.

## 3.7 Summary

In the current study, a midrise structure has been designed as case study using linear static analysis. The case study has been done in seismic zone 2B and soil type SB. PBD is performed by using nonlinear static method. The building is analyzed and designed following UBC-97 using linear Static method. Non-linear static PoA is than performed by pushing the building to maximum Considered Earthquake (MCE) level target displacement. The performance of building is assessed by studding the development of hinges at assigned locations following FEMA (FEMA 273, 1997, FEMA 356, 2000) guideline. Reinforcement of the structure and its possible variation in beams has been assessed using performance based design approach.

# Chapter 4

# Results and Discussion

## 4.1 Introduction

Results of the parametric study are presented in this chapter. Seismic performance of the reinforced concrete structure is evaluated by performing nonlinear pushover analysis. The results of this study will help in developing the seismic guidance for design of RC structures.

Structural performance is evaluated in terms of global output parameter, theses parameters obtained from different models of nonlinear static analysis are evaluated against linear static analysis design. Design of structures are discussed by comparing the linear static analysis design versus nonlinear static design in terms of global parameters of roof displacement, structural period and base shear and one local parameter of Interstory drift ratio and drift limits set by code. Development of hinges according to assigned reinforcement is discussed according to FEMA-356 guideline. After comparing each variation parameter is individually discussed in detail and results are presented in terms of selected performance evaluation parameters. Table 4.1 explain the assign acronyms to different models in X-direction, the same acronyms are used for models in Y-direction.

Table 4.1: The Model acronyms in X direction are defined as.

| No | Model Acronym | Definition |
|---|---|---|
| 1 | EX CBD REINFORCEMNET SLE | Model with reinforcement designed using code based design approach analyzing it using equivalent static analysis against Service Level Earthquake in X direction. |
| 2 | EX CBD REINFORCEMNET DBE | Model with reinforcement designed using code based design approach analyzing it using equivalent static analysis against Design Based Earthquake X direction. |
| 3 | EX CBD REINFORCEMNET MCE | Model with reinforcement designed using code based design approach analyzing it using equivalent static analysis against Maximum Considered Earthquake X direction. |
| 4 | PUSHX CBD REINFORCEMNET | Model with reinforcement designed using code based design approach analyzing it using Pushover analysis against Service Level Earthquake X direction. |
| 5 | PUSHX CBD REINFORCEMNET DBE | Model with reinforcement designed using code based design approach analyzing it using Pushover analysis against Design Based Earthquake X direction. |
| 6 | PUSHX CBD REINFORCEMNET MCE | Model with reinforcement designed using code based design approach analyzing it using Pushover analysis against Maximum Considered Earthquake X direction. |
| 7 | PUSHX PBD REINFORCEMNET SLE | Model with reinforcement designed using Performance based design approach analyzing it using Pushover analysis against Service Level Earthquake X direction. |
| 8 | PUSHX PBD REINFORCEMNET DBE | Model with reinforcement designed using Performance based design approach analyzing it using Pushover analysis against Design Based Earthquake X direction. |
| 9 | PUSHX PBD REINFORCEMNET MCE | Model with reinforcement designed using Performance based design approach analyzing it using Pushover analysis against Maximum Considered Earthquake X direction. |

## 4.2 Storey Shear

Storey shear is the lateral force generated at each level of the building in case of a seismic event. Storey shear is calculated at each storey as it varies from storey to storey across the height depending on masses and stiffness. It varies from maximum at the bottom to minimum at the top of the building. The maximum lateral force that the structure experience at the base of a structure due to seismic forces is equal to base shear. Base shear is also a global response parameter which narrate the lateral reaction at the base of the structure. It primarily depends on the mass of the structure, lateral load magnitude and lateral resistance offered by the structure.

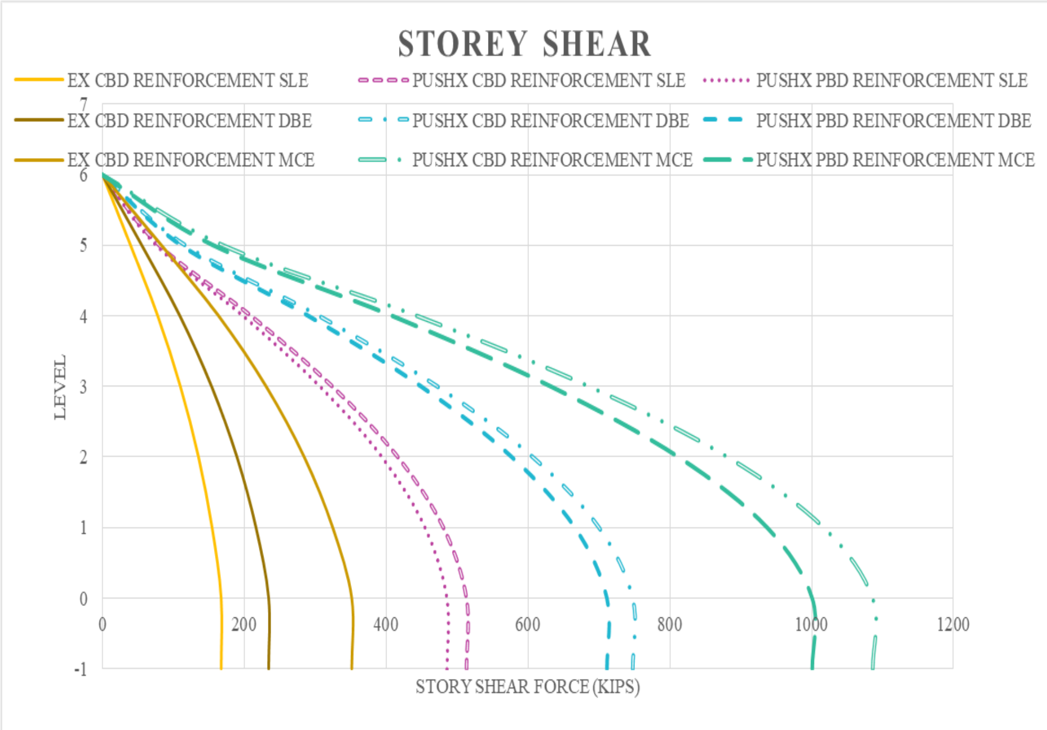


Figure 4.1: Storey Shears comparison of Code Based design and Performance Based design Ex and Push-X.

Comparison of storey shear at ground floor for respective earthquake levels is made which gives the following results. For SLE the storey shear of linear ststic (EX) is found to 306% less than nonlinar Push-X CBD reinorcement Model. Comparing the PBD model with CBD model the storey shear has decreased upto 5.5% in case of nonlinear Push-x.

For SLE the storey shear of linear ststic (EX) is found to 318% less than nonlinar Push-X CBD reinforcement Model. Comparing the PBD model with CBD model the storey shear has decreased upto 4.5% in case of nonlinear Push-x.

For SLE the storey shear of linear ststic (EX) is found to 309% less than nonlinar Push-X CBD reinforcement Model. Comparing the PBD model with code based design model the storey shear has decreased upto 4% in case of nonlinear Push-x.

Comparison of storey shear at ground floor for respective earthquake levels is made which gives the following results. For SLE the storey shear of linear ststic (EY) is found to 314% less than nonlinar Push-Y CBD reinforcement Model. Comparing the PBD model with code based design model the storey shear has decreased upto 6.3% in case of nonlinear Push-Y.

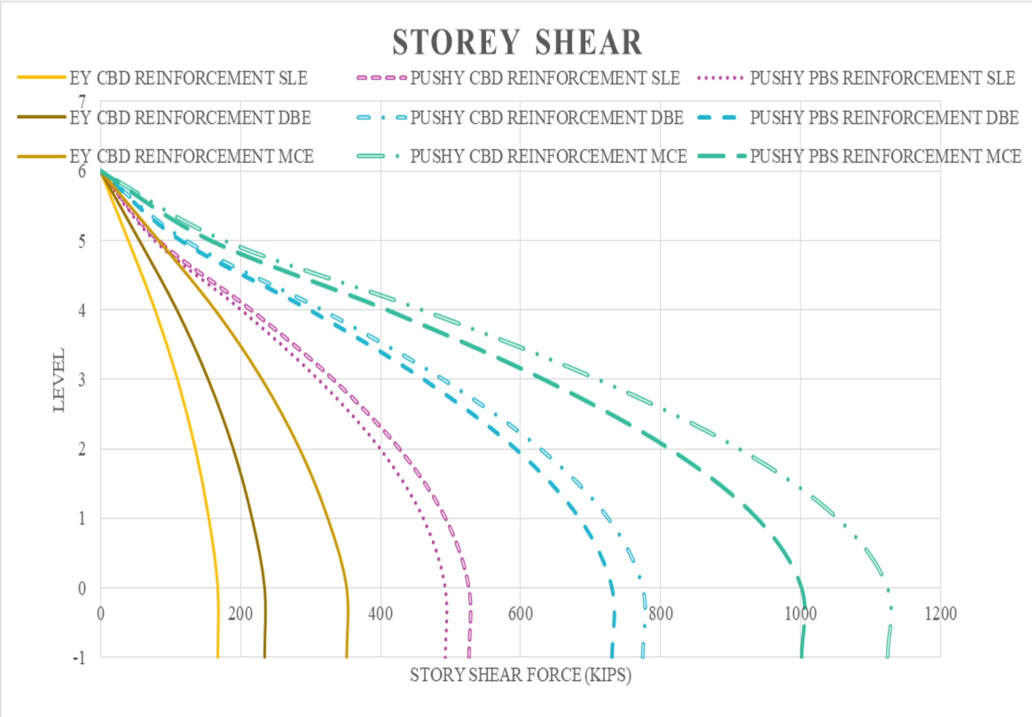


FIGURE 4.2: Storey Shears comparison of Code Based design and Performance Based design EY and Push-Y.

For SLE the storey shear of linear ststic (EY) is found to 330% less than nonlinar Push-Y code based design reinforcement Model. Comparing the PBD model with CBD model the storey shear has decreased upto 5.7% in case of nonlinear Push-Y.

For SLE the storey shear of linear static (EY) is found to 319% less than nonlinar Push-Y CBD Model. Comparing the PBD model with code based design model the storey shear has decreased upto 4.8% in case of nonlinear Push-Y.

The shear shear determined through nonlinear pushover has decreased in case of performancr based design. This is deu to less design reinforcement (less flexural demend) provided in beams.

## 4.3 Over Turning Moment

Overturning moment of storey is the torque due to the resulting applied lateral forces about the points of contact with the ground or base. Overturning moment of a storey is defined as the cumulative product of lateral forces and moment arm up to that storey level.

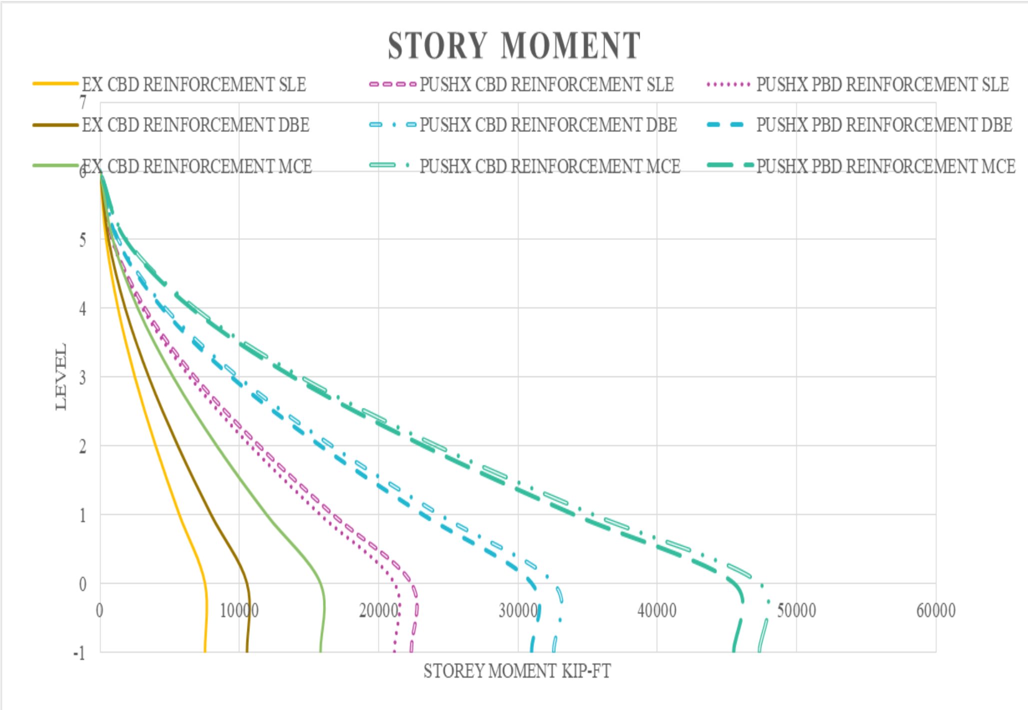


Figure 4.3: Storey Moment comparison of Code Based design and Performance Based design Ex and Push-X.

Comparison of storey moment at ground floor for respective earthquake levels is made which gives the following results. For SLE the storey moment of linear ststic (EX) is found to 296% less than nonlinar Push-X CBD reinforcement Model. Comparing the PBD model with CBD model the storey shear has decreased upto 5.5% in case of nonlinear Push-x.

For SLE the storey shear of linear ststic (EX) is found to 308% less than nonlinar Push-X code based design reinforcement Model. Comparing the PBD model with CBD model the storey shear has decreased upto 4.5% in case of nonlinear Push-x.

For SLE the storey shear of linear ststic (EX) is found to 299% less than nonlinar Push-X CBD reinforcement Model. Comparing the PBD with code based design model the storey shear has decreased upto 4% in case of nonlinear Push-x.

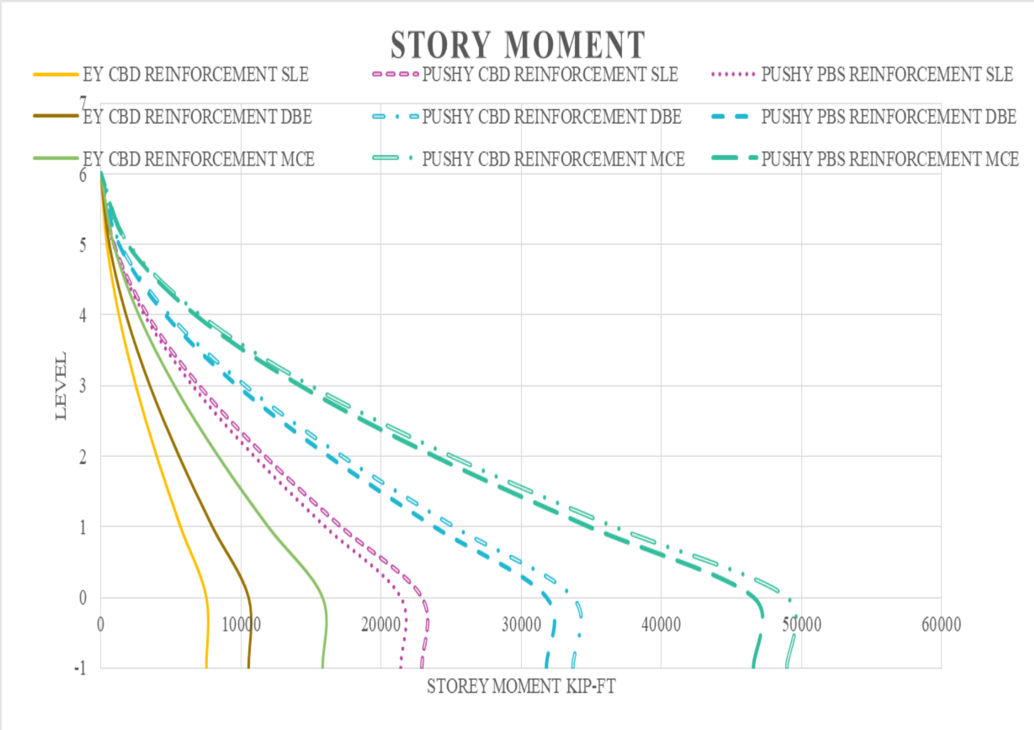


FIGURE 4.4: Storey Moment comparison of Code Based design and Performance Based design EY and Push-Y.

Comparison of storey moment at ground floor for respective earthquake levels is made which gives the following results. For SLE the storey moment of linear ststic (EY) is found to 303% less than nonlinar Push-Y CBD reinforcement Model. Comparing the PBD model with code based design model the storey shear has decreased upto 6.3% in case of nonlinear Push-Y.

For SLE the storey shear of linear ststic (EY) is found to 319% less than nonlinar Push-Y CBD reinforcement Model. Comparing the PBD model with code based design model the storey shear has decreased upto 5.7% in case of nonlinear Push-Y.

For SLE the storey shear of linear ststic (EY) is found to 309% less than nonlinar Push-Y CBD design reinforcement Model. Comparing the PBD model with code based design model the storey shear has decreased upto 4.8% in case of nonlinear Push-Y.

## 4.4 Storey Displacement

Storey displacement is global parameter which refers to the lateral displacement of the roof of the structure with respect to its base. The roof displacement is a parameter of measure of lateral displacement response of storey against lateral loading relative to the base.

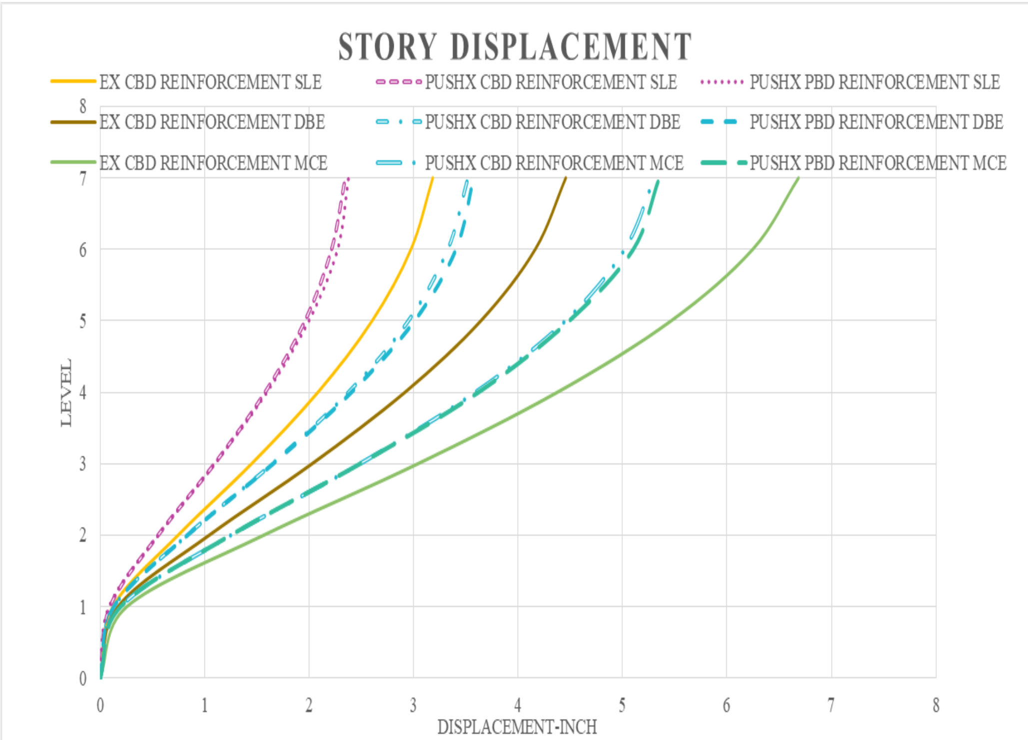


FIGURE 4.5: Storey Displacement comparison of Code Based design and Performance Based design Ex and Push-X.

In case of Service Level Earthquake the storey drift of nonlinear Push-X with CBD reinforcement is 22% less as compared static (EX). In PBD optimized reinforcement model the storey displacement has increased upto 2% as compared to CBD reinforcement model in case of nonlinear Push-X.

In case of DBE the storey drift of nonlinear Push-X with CBD reinforcement is 24.5% less as compared to linear static (EX). In PBD reinforcement model the storey displacement has increased up to 2.4% as compared to CBD reinforcement model in case of nonlinear Push-X.

In case of MCE the storey drift of nonlinear Push-X with CBD reinforcement is 24% as compared to linear static (EX). In PBD reinforcement model the storey displacement has increased upto 2.5% as compared to CBD reinforcement model in case of nonlinear Push-X.


Figure 4.6: Storey Displacement comparison of Code Based design and Performance Based design EY and Push-Y.

In case of SLE the storey drift of nonlinear Push-Y with CBD reinforcement is 24% less as compared ststic (EY). In PBD reinforcement model the storey displacement has increased upto 2.2% as compared to CBD reinforcement model in case of nonlinear Push-Y.

In case of DBE the storey drift of nonlinear Push-Y with CBD reinforcement is 19% less as compared to linear static (EY). In PBD reinforcement model the storey displacement has reduced upto 2.8% as compared to CBD reinforcement model in case of nonlinear Push-Y.

In case of MCE the storey drift of nonlinear Push-Y with CBD reinforcement is 20% as compared to linear static (EY). In PBD reinforcement model the storey displacement has reduced up to 2.7% as compared to CBD reinforcement model in case of nonlinear Push-Y.

## 4.5 Storey Drift

Storey drift or inter storey drift is an important parameter which represent the structural behavior. It is the relative horizontal displacement between two adjacent floors and it is generally represented in terms of drift ratios which is storey drift divided by the corresponding storey height. Drift ratio is a local parameter which gives the localized drift or displacement capacities for the structure at individual storey level Roof displacement divided by the structure height. Storey drift is calculated.

If excessive deformation is allowed in the structure, it may cause structural and non-structural damages due to the material or members failure. To avoid this, building codes provide limitations on the drifts to meet the structural and serviceability requirements. According to FEMA, ATC-40 the allowable drifts for (SLE), (DBE) and (MCE) is 1, 2 and 0.33Vi/pi≈3-7 respectively.

For SLE it is noted that storey drift of linear analysis (EX) is 37% more that that of nonlinear Push-X with CBD reinforcement. In PBD reinforcement model the storey drift has increased upto 2.7% W.R.T Push-X CBD reinforcement model.

For DBE it is noted that storey drift of linear analysis (EX) is 33% more that that of nonlinear Push-X with CBD reinforcement. In PBD reinforcement model the storey drift has increased upto 3.1% W.R.T Push-X CBD reinforcement model.

In case of MCE it is noted that storey drift of linear analysis (EX) is 28% more that that of nonlinear Push-X with CBD reinforcement. In PBD reinforcement model the storey drift has increased upto 3% W.R.T Push-X CBD reinforcement model.

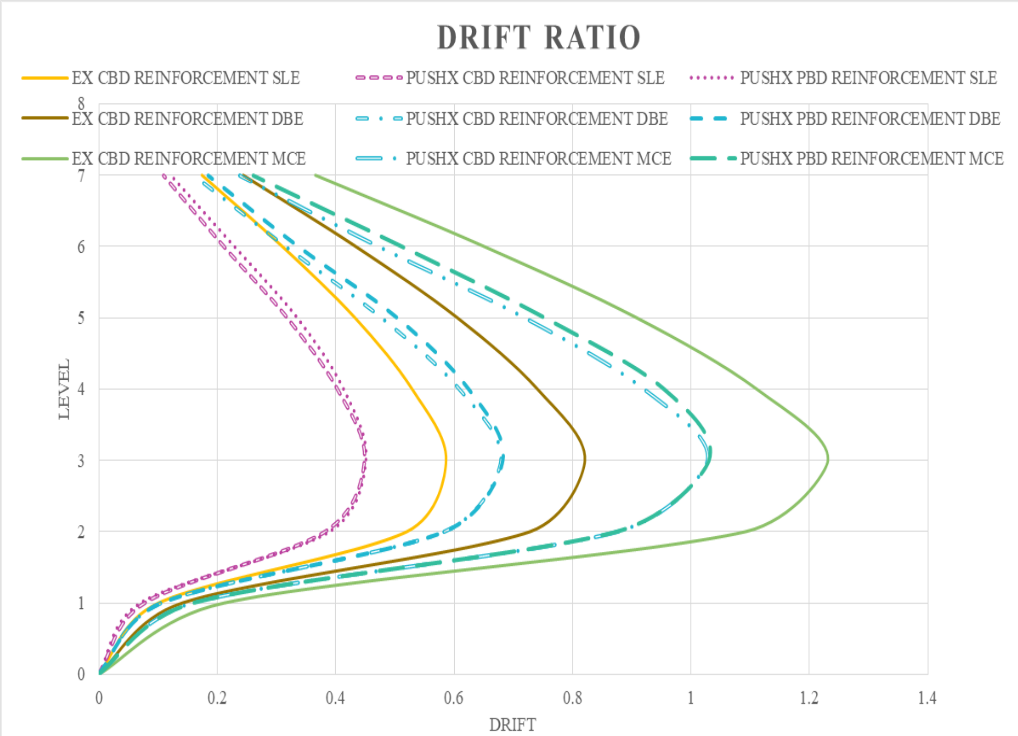


FIGURE 4.7: Storey Drift comparison of Code Based design and Performance Based design Ex and Push-X.

For SLE it is noted that storey drift of linear analysis (EY) is 41% more that that of nonlinear Push-Y with CBD reinforcement. In PBD reinforcement model the storey drift has increased upto 2.9% W.R.T Push-X CBD reinforcement model.

For DBE it is noted that storey drift of linear analysis (EY) is 36% more that that of nonlinear Push-Y with CBD reinforcement. In PBD reinforcement model the storey drift has increased upto 3.3% W.R.T Push-X CBD reinforcement model.

In case of MCE it is noted that storey drift of linear analysis (EY) is 37% more that that of nonlinear Push-Y with CBD reinforcement. In PBD reinforcement the behaviour of storey drift has changed. In PBD reinforcement model the storey drift has increased upto 3.5% W.R.T Push-X CBD reinforcement model.

The storey drift of Performance based design model with optimized reinforcement has slightly shifted towards more ideal uniform drift. The reason behing the variation in storey drift is because the structure has achieved more uniform ductility over all stories.

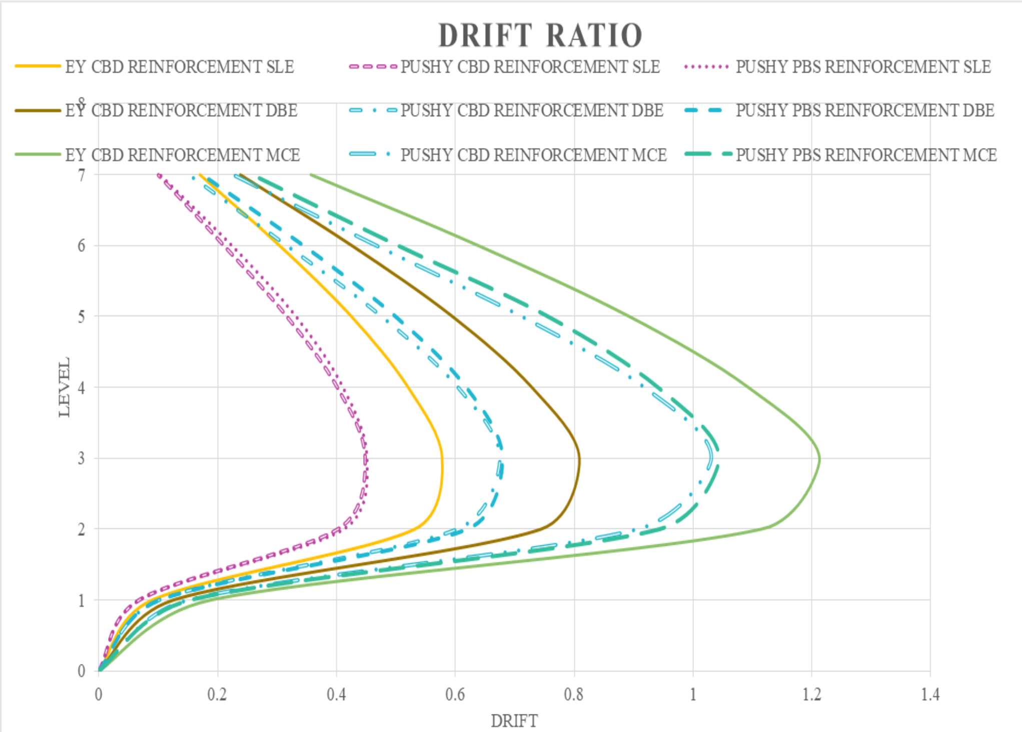


FIGURE 4.8: Storey Drift comparison of Code Based design and Performance Based design EY and Push-Y.

## 4.6 Performance Evaluation and Formation of Plastic Hinges

Nonlinear static pushover analysis is performed to apply pre-defined loading to push the structure horizontally up to a target displacement. At every deformation step of the pushover analysis, the program can do the following. (a) Determine the limit state (IO, LS, and CP) and plastic rotation of hinges in beams and columns. (B) Determine which hinges have reached one of the three FEMA limit states: IO, LS and CP using suitable colors for their identification (As shown in Figure 4.9).

Figure 4.9 shows plastic hinges formation in a model against design reinforcement. The formation of hinges and limit state against each earthquake level (SLE, DBE and MCE) has been discussed at each storey level.

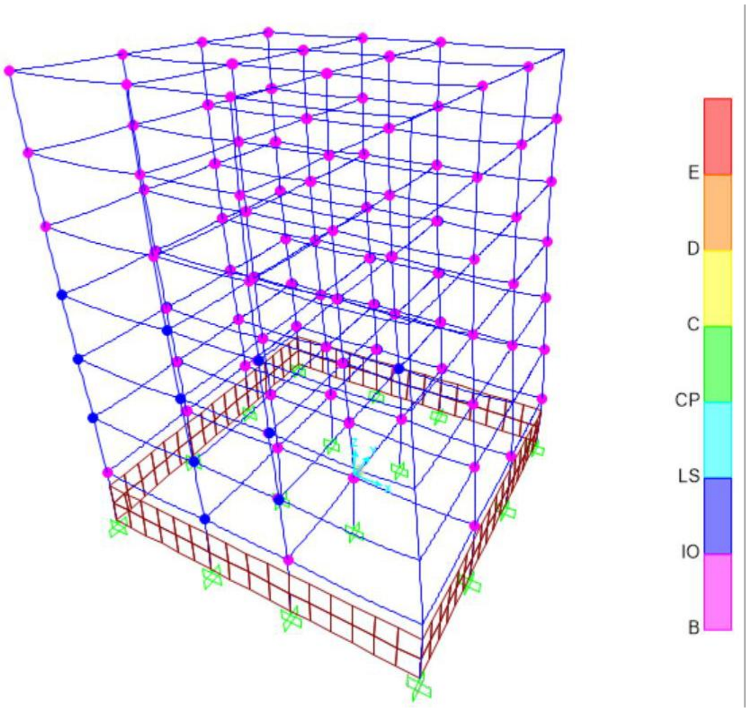


FIGURE 4.9: Formation of Plastic Hinges against Design Basis Earthquake.

The number of hinges in beams for both case models recorded are 336. The following tables shows the initial and final plastic hinge distribution at each storey under the pushover loading. In each beam two hinges are assigned, to check performance state of the whole beam the critical of both the hinges assigned is taken as state of the beam. No plastic hinge rotation has exceeded the specified threshold of plastic rotation in Performance based design.

It is noted that the formation of hinges starts from bottom storey and progress towards higher storey. This means that there is less capacity in lower storey structural members and the capacity increases as we go higher. In Code Bases Design (CBD) study the hinges developed for SLE, DBE and MCE are found to be in the preset limit states. In case of SLE 180 hinges lies below the Immediate Occupancy state, and 156 in the state of Immediate occupancy. In case of DBE 136 hinges are found in operational performance level, 122 in Immediate Occupancy and 78 number of hinges in Life Safety region. Similarly in CBD case of Maximum Considered Earthquake the 88 hinges are lies in operational level, 108 in Immediate

Occupancy and 140 in Life Safety region. None of the hinge reached the Collapse Prevention level for MCE.

TABLE 4.2: Detail of plastic hinges formation for Service Level Earthquake (SLE).

| Storey | Model | A-B | B-IO | IO-LS | LS-CP | Total |
|---|---|---|---|---|---|---|
| 1 | CBD | 24 | 24 | - | - | 48 |
| | PBD (-9% Reinforcement) | 16 | 32 | - | - | 48 |
| 2 | CBD | 24 | 24 | - | - | 48 |
| | PBD (-4% Reinforcement) | 16 | 32 | - | - | 48 |
| 3 | CBD | 24 | 24 | - | - | 48 |
| | PBD (-6% Reinforcement) | 22 | 26 | - | - | 48 |
| 4 | CBD | 24 | 24 | - | - | 48 |
| | PBD (-8% Reinforcement) | 20 | 28 | - | - | 48 |
| 5 | CBD | 24 | 24 | - | - | 48 |
| | PBD (-11% Reinforcement) | 18 | 30 | - | - | 48 |
| 6 | CBD | 28 | 20 | - | - | 48 |
| | PBD (-14% Reinforcement) | 8 | 40 | - | - | 48 |
| 7 | CBD | 32 | 16 | - | - | 48 |
| | PBD (-17% Reinforcement) | 16 | 8 | - | - | 48 |

TABLE 4.3: Performance Level of Beams against Service Level Earthquake (SLE).

| Model | Performance Level | Storey | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Code Based Design | Operational | 0 | 0 | 0 | 0 | 2 | 6 | 12 |
| | Immediate Occupancy | 24 | 24 | 24 | 24 | 22 | 18 | 12 |
| | Life Safety | - | - | - | - | - | - | - |
| Performance Based Design | Operational | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | Immediate Occupancy | 24 | 24 | 24 | 24 | 24 | 24 | 24 |
| | Life Safety | - | - | - | - | - | - | - |

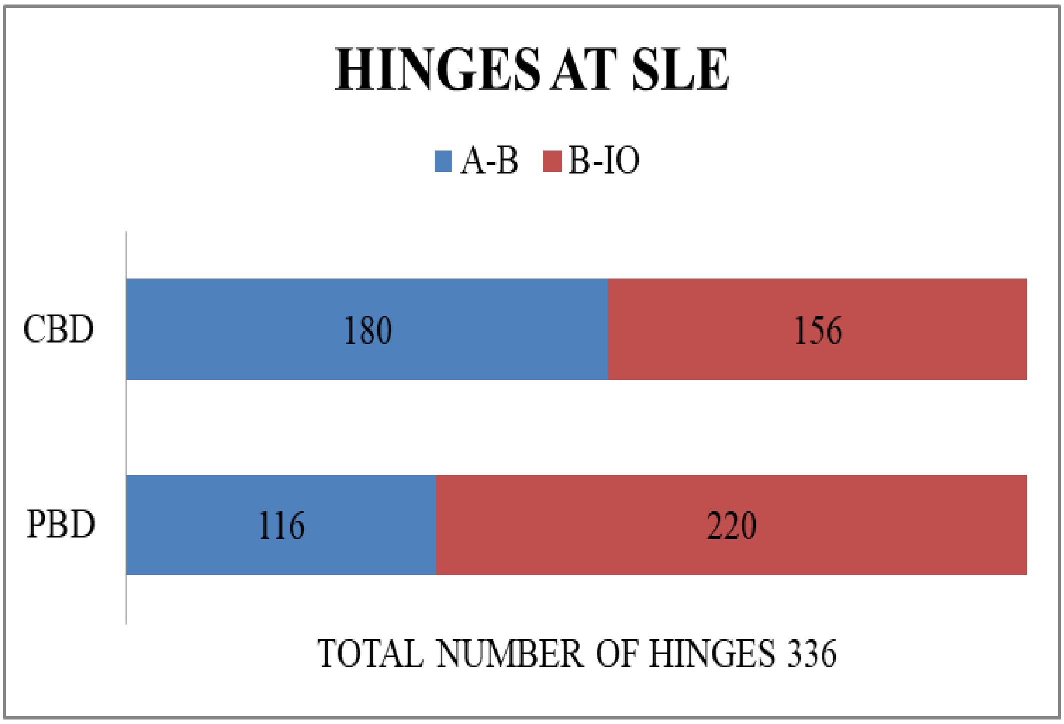


FIGURE 4.10: Summary of plastic hinges formation at (SLE).

TABLE 4.4: Detail of plastic hinges formation for Design Base Earthquake (DBE).

| Storey | Model | A-B | B-IO | IO-LS | LS-CP | Total |
|---|---|---|---|---|---|---|
| 1 | CBD | 24 | 18 | 6 | - | 48 |
| | PBD (-9% Reinforcement) | 16 | 16 | 16 | - | 48 |
| 2 | CBD | 16 | 8 | 24 | - | 48 |
| | PBD (-4% Reinforcement) | 12 | 12 | 24 | - | 48 |
| 3 | CBD | 16 | 8 | 24 | - | 48 |
| | PBD (-6% Reinforcement) | 14 | 10 | 24 | - | 48 |
| 4 | CBD | 16 | 8 | 24 | - | 48 |
| | PBD (-8% Reinforcement) | 10 | 14 | 24 | - | 48 |
| 5 | CBD | 24 | 24 | 0 | - | 48 |
| | PBD (-11% Reinforcement) | 18 | 8 | 22 | - | 48 |
| 6 | CBD | 24 | 24 | 0 | - | 48 |
| | PBD (-14% Reinforcement) | 8 | 18 | 22 | - | 48 |
| 7 | CBD | 16 | 32 | 0 | - | 48 |
| | PBD (-17% Reinforcement) | 8 | 24 | 16 | - | 48 |

TABLE 4.5: Performance Level of Beams against for Design Base Earthquake (DBE).

| Model | Performance Level | Storey | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Code Based Design | Operational | 0 | 0 | 0 | 0 | 0 | 0 | 4 |
| | Immediate Occupancy | 18 | 0 | 0 | 0 | 22 | 24 | 20 |
| | Life Safety | 16 | 24 | 24 | 24 | 2 | 0 | 0 |
| Performance Based Design | Operational | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | Immediate Occupancy | 8 | 0 | 0 | 0 | 0 | 8 | 18 |
| | Life Safety | 16 | 24 | 24 | 24 | 24 | 16 | 6 |

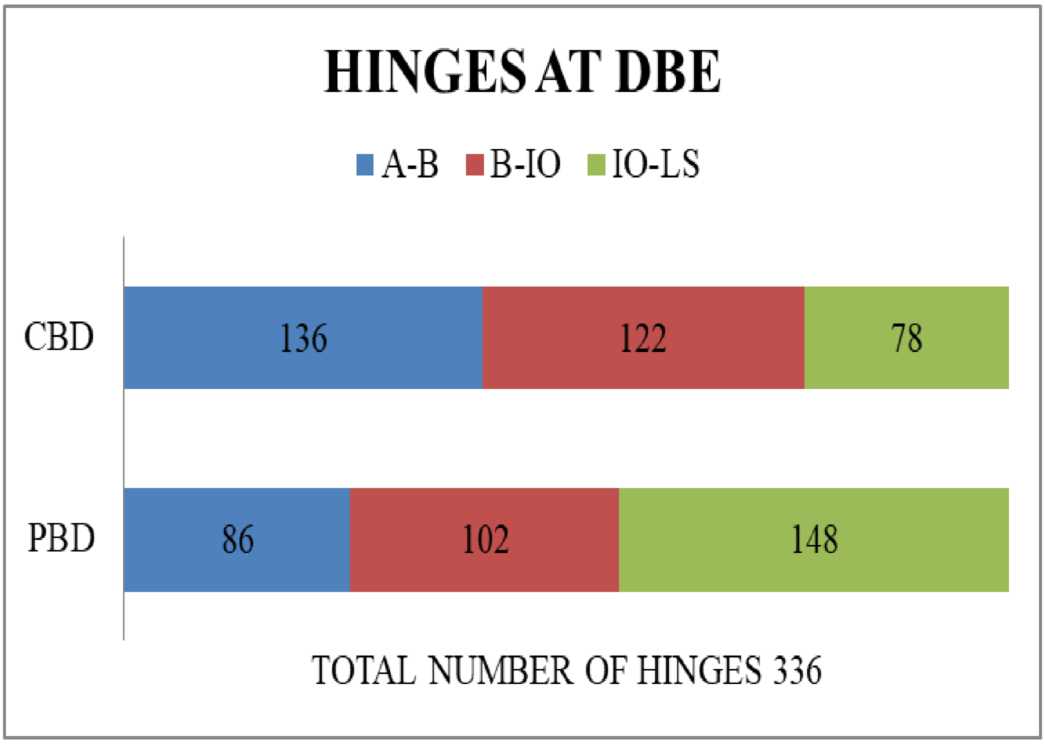


FIGURE 4.11: Summary of plastic hinges formation at (DBE).

Table 4.6: Detail of plastic hinges formation for Maximum Considered Earthquake (MCE).

| Storey | Model | A-B | B-IO | IO-LS | LS-CP | Total |
|---|---|---|---|---|---|---|
| 1 | CBD | 16 | 8 | 24 | 0 | 48 |
| | PBD (-9% Reinforcement) | 8 | 16 | 24 | 0 | 48 |
| 2 | CBD | 0 | 16 | 32 | 0 | 48 |
| | PBD (-4% Reinforcement) | 0 | 10 | 34 | 8 | 48 |
| 3 | CBD | 0 | 16 | 32 | 0 | 48 |
| | PBD (-6% Reinforcement) | 0 | 14 | 32 | 2 | 48 |
| 4 | CBD | 16 | 4 | 28 | 0 | 48 |
| | PBD (-8% Reinforcement) | 8 | 10 | 28 | 2 | 48 |
| 5 | CBD | 16 | 8 | 24 | 0 | 48 |
| | PBD (-11% Reinforcement) | 10 | 14 | 24 | 0 | 48 |
| 6 | CBD | 24 | 24 | 0 | 0 | 48 |
| | PBD (-14% Reinforcement) | 8 | 22 | 18 | 0 | |
| 7 | CBD | 16 | 32 | 0 | 0 | 48 |
| | PBD (-17% Reinforcement) | 8 | 36 | 4 | 0 | 48 |

For optimal design following performance based design approach the beam reinforcement has been reduced starting from 1% in beams across all stories. The reduction at particular storey is stopped if any of the member in that storey crosses the FEMA Structural performance level (Immediate Occupancy for SLE, Life Safety for DBE and Collapse Prevention for MCE). If only one span reaches the point of collapse than the whole structure should be regarded as partial collapse and the particular storey should be considered as overall collapse (Structural Steel Work: Limited State Design). With the varying reduction of reinforcement storey wise the optimized structure with number of hinges developed all satisfy the FEMA guideline.

TABLE 4.7: Performance Level of Beams against Maximum Considered Earthquake (MCE).

| Model | Performance Level | Storey | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| Code Based Design | Operational | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | Immediate Occupancy | 0 | 0 | 0 | 0 | 0 | 24 | 24 |
| | Life Safety | 24 | 24 | 24 | 24 | 24 | 0 | 0 |
| | Collapse Prevention | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Performance Based Design | Operational | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | Immediate Occupancy | 0 | 0 | 0 | 0 | 0 | 6 | 20 |
| | Life Safety | 24 | 20 | 22 | 22 | 24 | 18 | 4 |
| | Collapse Prevention | 0 | 4 | 2 | 2 | 0 | 0 | 0 |

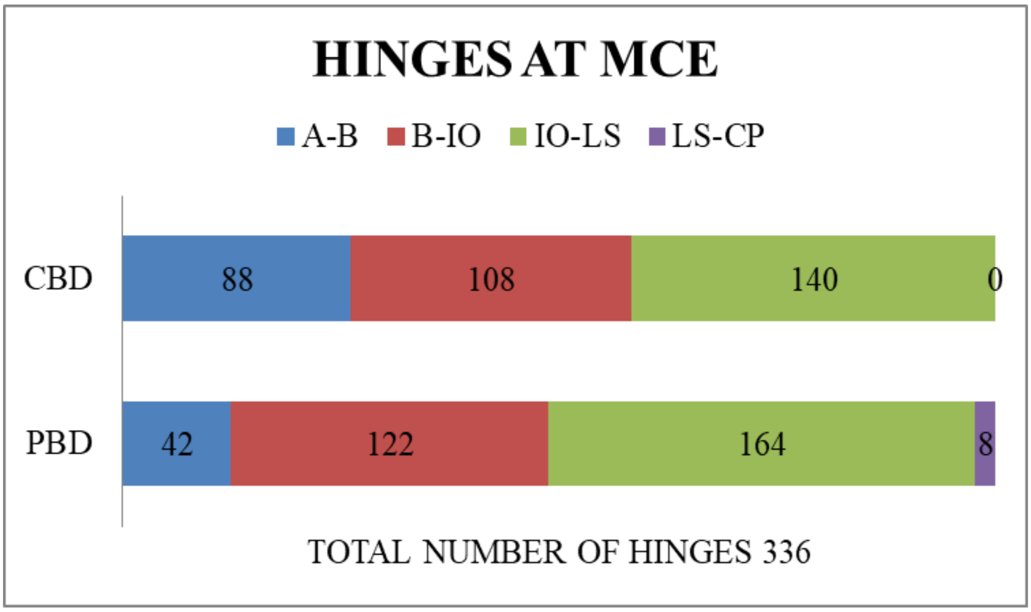


FIGURE 4.12: Summary of plastic hinges formation at (MCE).

In performance based design case of SLE 116 hinges lies below the Immediate Occupancy state, and 220 in the state of Immediate occupancy. In case DBE 86 hinges are found in operational performance level, 102 in Immediate Occupancy and 148 number of hinges in Life Safety region. Similarly case of MCE the 42 hinges are lies in operational level, 122 in Immediate Occupancy, 164 in Life Safety region and 8 of the hinge reached the Collapse Prevention level for Maximum Considered Earthquake.

The formation of plastic hinges and their states in conventional code base design with typical reinforcement and Performance Based Design shows that both designs meet the intended performance objective.

In the final Optimized Performance based Design with proper reduction of reinforcement the overall structure is found to be safe for Service Level Earthquake, Design Basis Earthquake and Maximum Considered Earthquake.

## 4.7 Beam Reinforcement

As elaborated the flexural reinforcement in beams has been reduced up to a reasonable extent. Using the performance based design approach the flexural reinforcement in beams in beams has been reduced up to 9.09%.

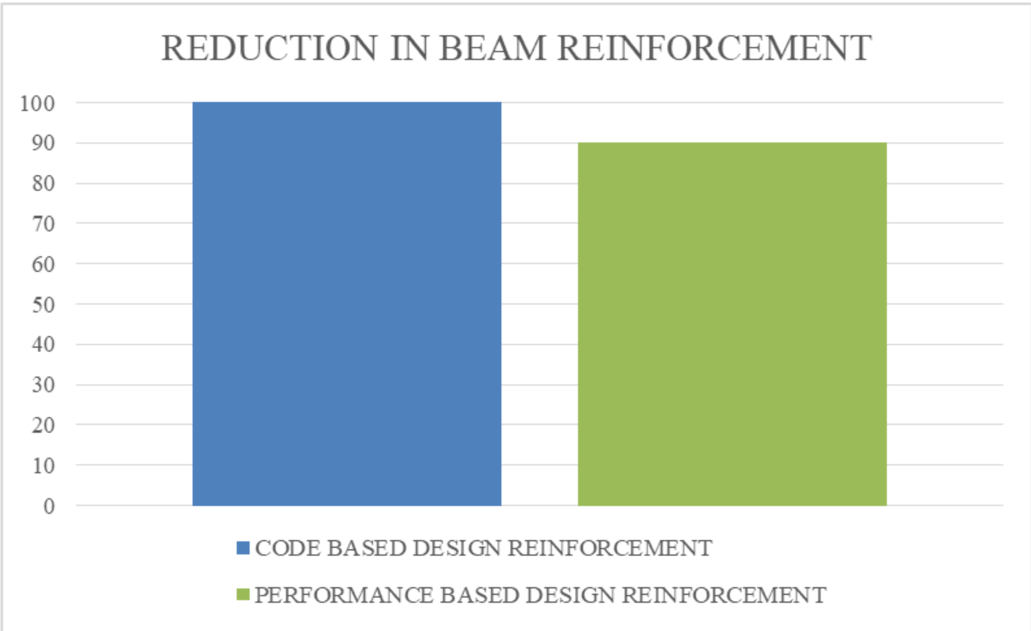


FIGURE 4.13: Percent reduction in flexural reinforcement

## 4.8 Hinges in Column

As P-M2-M3 hinges are assigned to the columns. The formations of hinges are studied for code based design and performance based design. All of the column hinges were found in the recommended performance levels. This ensures strong column weak beam concept.

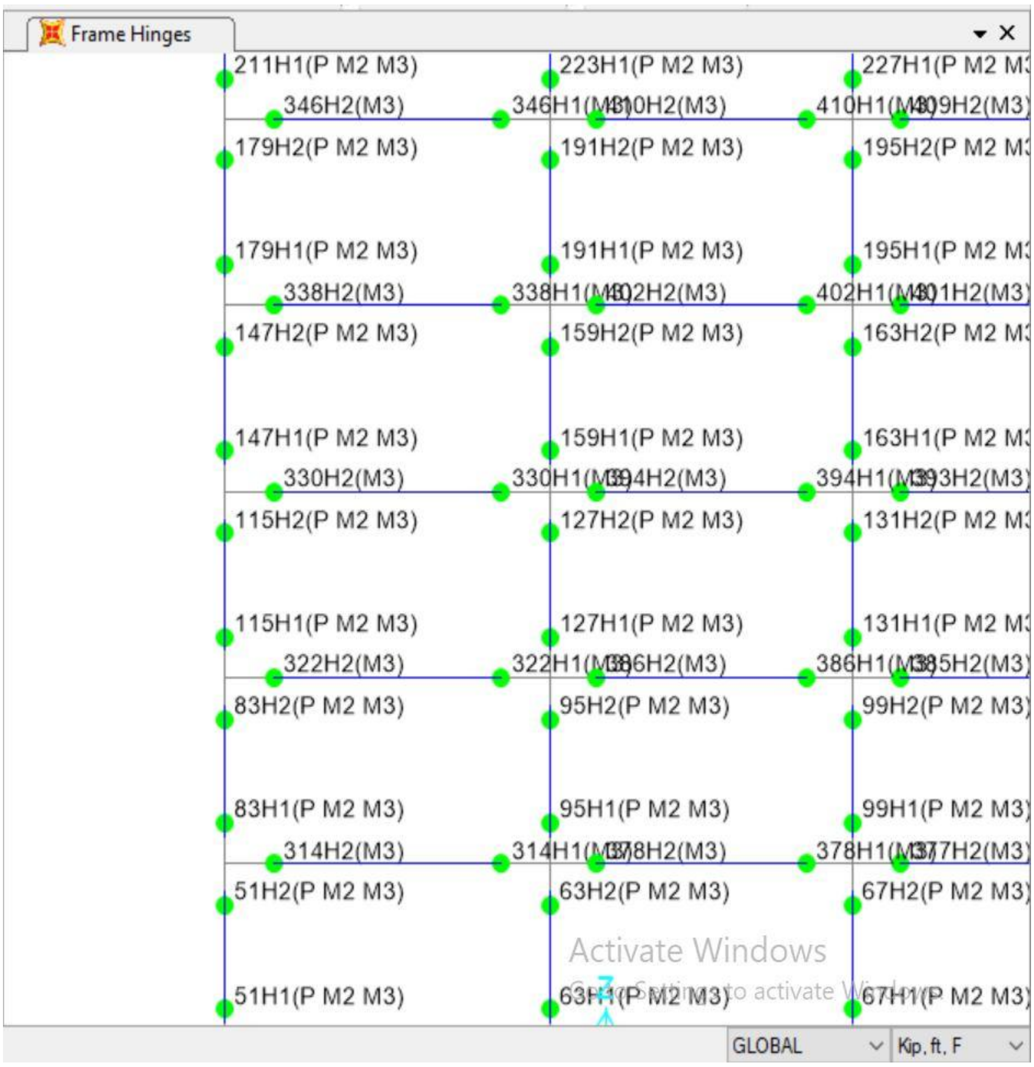


FIGURE 4.14: P-M2-M3 Column Assigned Hinges.

Further the design reinforcement demand for column was studied and no difference was found in the reinforcement demand of Code Based design and performance based design. This indicates that the performance based design of beam with respect to code based design column reinforcement is safe and applicable.

## 4.9 Summary of Discussions

In this chapter results obtained from the seismic performance assessment of building designed with code-based, in-practice and building designed with different sets off reinforcement were discussed in detail and reasonable justification of the results was presented. For this purpose an analytical seismic framework was developed for analyzing the seismic response of structures which could indicate the failure pattern due to earthquake.

Failure mode greatly depends on the geometric parameters of the structures. It has been observed that stiffness and ductility greatly influences the capacity and failure mode of the structure. It is noted that Time period of both code based design and performance based design is same which implies that both code base design and performance base design will have that same pushover lateral displacement and there is no heed the reiterate the displacement demand for performance based design.

In next chapter which is the last one, conclusion of the all results and recommendations for the further research are stated.

# Chapter 5

# Conclusion and Recommendations

In this research, a 7-storey structure has been designed in seismic zone 2B with soil profile type SB. After doing so, the seismic behavior of the designed structure is analyzed by using Linear Static Analysis method of UBC-97 and ACI-318 respectively. The building has been investigated for SLE, DBE and MCE. Using performance based design approach the nonlinear seismic design of reinforced concrete structure has been clearly illustrated in term of beam flexural reinforcement as design input variables. The seismic performance of CBD Model and PBD is compared in term of performance objectives, like development of hinges and Drift limits. Other parameter like, storey shear, overturning moment, storey displacement are also studies. The following conclusion can be withdrawn from the study.

## 5.1 Conclusions

Following conclusions have been made out of this study:

- At MCE in CDB model, there are 48(28.57%) and 120(71.42%) beams at immediate occpancy, and life safety respectively. While in PDB model with optimized reinforcement, there are 26(15.47%), 134 (79.76%) and (4.76%) beams at immediate occpancy, life safety and collapse prevention respectively.

- At DBE in CDB model, there are 4(2.38%), 84(50%) and 80(47.61%) beams at operational, immediate occpancy, and life safety respectively. While in PDB model with optimized reinforcement, there are 34(20.23%) and 134 (79.7%) beams at immediate occpancy, life safety respectively.
- At SLE in CDB model, there are 20(11.9%), 148(88.09%) beams at operational and immediate respectively. While in PDB model with optimized reinforcement, there all of 168(100%) beams are at immediate occpancy level.
- It is demonstrated that with beam flexural reinforcement it is possible to change the state of hinge according to desire performance objective.
- Using PBD approach, the total flexural reinforcement in RC beam has been reduced up-to 9.09%. Therefore, it is demonstrated that the PBD strategy leads to optimal design reinforcement and better performance against seismic hazards.
- Result indicates that performance based design method can effectively achieve economical design of reinforced concrete building frame works.

## 5.2 Recommendations for Future Studies

The aim of this research is to propose an effective computer based optimum design of RC structure using performance based design approach to meet designer specified needs and code requirements for reinforced concrete buildings. The performance based design establishes a good basis for more comprehensive optimization.

As research on nonlinear pushover suggests that pushover analysis is sufficient for first mode dominant buildings, still further research can be done by applying nonlinear time history analysis to explore if there is room for further optimization.

Using the performance based design method optimization of columns in term of steel reinforcement can be studied.

The performance based design strategy used in this research thesis is time-consuming and tedious, therefore it is recommended to address this issue in structural analysis and design programs.

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