Performance Evaluation of RCC Building A Direct Displacement Based Approach

BY

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DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

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Performance Evaluation of RCC Building A Direct Displacement Based Approach

Major Project

Submitted in partial fulfillment of the requirements

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NIRMA UNIVERSITY
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MAY 2018

Declaration

This is to certify that

a The major project comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b Due acknowledgement has been made in text to all other material used.

Kiran Tiwari

Certificate

This is to certify that the Major Project entitled **"Performance Evaluation of RCC Building : A Direct Displacement Based Approach"** submitted by **Ms. Kiran Tiwari (16MCLC10)**, towards the partial fulfillment of the requirement for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad, is the record of work carried out by her under my supervision and guidance. The work submitted has in my opinion reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, have not been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Current seismic design codes used for seismic design of structures are based on Force-Based Design approach. The traditional seismic design of structure is primarily based on forces. This methodology completely misses an important parameter, structural displacement, which is a measure of damage in structures subjected to earthquake excitations. Strain and drift can be integrated to give displacement. Therefore, both structural and non-structural damage can be related to displacement. Many researchers have shown that Direct Displacement Based approach performed well in predicting seismic demands of the structure.

The present study is done to close the gap between existing research on Displacement-Based Design (DBD) and its implementation for the design of conventional reinforced concrete (RC) frame structures. This approach has been utilized on reinforced concrete regular moment resisting frame structure. Story drift ratios were chosen as deformation limits to define the performance levels for specific earthquake hazard levels. In this study frames designed using DBD approach has been utilized in compliance with response spectra of IS 1893 (Part 1). Nonlinear static analysis and time history analysis has been carried out to evaluate whether the postulated performance criteria are met.The base shear forces and the top displacements, in addition, for each frame were also checked with the ones obtained through DBD approach. Comparison of buildings analysed using DBD approach has been done with the same building analysed using FBD. Results show that DBD gives low base shear as compared to FBD. Also, performance assessment of frames by nonlinear static and time history analysis show that DBD is well within the desired performance limit.

Also effect of the site specific response spectrum on analysis of reinforced concrete regular frame structure is presented. Present study includes ground response analysis of of Ahmedabad and Delhi site using one dimensional equivalent linear analysis based software ProSHAKE. Acceleration time history of Bhuj Earthquake (0.106g) and El Centro Earthquake (0.344g) are considered as input motion to get response spectra at ground surface for low and high PGA. Various sites considered in Ahmedabad are Paldi, Chandkheda, Thaltej and Passport Office at same soil depth of 15m. Delhi site is considered

with varying bedrock depth to take account of variation in soil depth. Seismic response of 3, 6-storey regular RC frame structures are analysed using DBD methodology at Ahmedabad and Delhi site for low and high PGA earthquakes. Results of the Ahmedabad show similar results in all the sites for a given height of building and earthquake because of the reason that all the sites have same soil depth. Whereas, sites in Delhi show variation in base shear forces corresponding to variation in soil depth.

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First and foremost, I would like to extend my heartfelt gratitude to my faculty guide,**Prof. J. M. Suthar**, Assistant Professor and **Dr. S. P. Purohit**, Professor, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for helping me throughout the preparation of my thesis. I was fortunate to have such guides who were always there to solve my problems and difficulties during my thesis work.

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Chapter 1

Introduction

1.1 General

The seismic design in all current codes and standards has been based on force rather than displacement. In the past, structures were designed based on forces. The reason for which is that earlier force considerations were critical and structures were designed to have sufficient strength so as to be able to resist applied loads. Prior to 1930's few structures were designed for seismic actions. In the 1920's and early 1930's several major earthquakes occurred where structures designed for lateral wind forces were observed to perform better. As a result the importance of lateral force design was realised and the design codes started including seismic design for the structures in high seismic zones. Earlier, approximately 10% of the building weight, regardless of building period was distributed and applied vertically along the height of building. Later concepts of ductility and force-reduction factors came into consideration while designing structures for seismic forces.

Past studies and researches aimed at better understanding of mechanics of concrete behaviour and detail design methods applied to structural and non-structural elements. This has reached to an extent that in some forms, code requirements are such that newly designed buildings can be considered sufficiently safe under seismic excitation.Earthquakes induce forces and displacements in structures. For elastic systems these responses are related to stiffness but for inelastic systems, these are dependent on both the current displacement and history of displacement during the seismic response. In the traditional earthquake resistant design, ductility demands are obtained from, the calculated force demand-capacity ratio. This methodology completely misses an important parameter, structural displacement, which is a measure of damage in structures subjected to earthquake excitations. It has been discussed in literature that damages in building due to earthquake can be better quantified by displacement rather than the forces.

In force-based design external forces are applied to the structure, which are equivalent to the inertial forces induced by ground accelerations. The basic assumption used in these types of design methods is that the structure will behave with first mode response. First mode dominates may be a valid assumption but the presence and effect of higher modes of vibration cannot be neglected.Force based approach assumes that the elastic characteristics of structure best indicators of inelastic performance which is not true. Force based design utilizes initial stiffness in order to determine time period of structure. Forces between different elements are distributed in proportion to their initial stiffness. Outdated height dependent equations are used to calculate time period of structure. Different codes provide different force-reduction factors for identical systems and materials which is inappropriate. Detailed analysis and experimental investigations have shown that stiffness is directly dependent on strength. It has now been understood and slowly being accepted, that the damage is more related to relative displacements than to forces.

In the 1990's, concept of "Displacement Based Design (DBD)" was proposed based on displacement. In this approach, displacement of structures is the primary response variable. Hence, design or acceptance criteria and capacity-demand comparisons are expressed in terms of displacements rather than the forces. The stiffness and time period are the end products rather than input quantities. Displacement based concepts are more used for the seismic assessment of existing structures rather than for design of new ones. The fundamental philosophy behind "Displacement Based Design" approach is to design a structure to achieve a given performance limit state under a given seismic intensity. In the Displacement Based Design (DBD) procedure, the seismic design is carried out by specifying a target displacement. Strength and stiffness are not the design variables in the procedure; instead they are the end-products.

1.2 Objective of the Study

The major objective of study is to consider an effect of site-specific characteristic of soil on Displacement Based Design of reinforced concrete building.

1.3 Scope of the Work

In order to achieve above objective following scope of work is prepared:

- To understand fundamental difference between Force Based Design(FBD) and Displacement Based Design(DBD).
- To study Displacement Based Design procedure in detail and its application to building structures.
- To carryout Displacement Based Design of regular RC frame building for IS 1893 (Part 1):2016 and site-specific displacement response spectrum.
- To compute various response quantities of RC building using Displacement Based Design.

1.4 Organization of the Report

The report is organized chapter-wise with details as follows.

Chapter 2 presents literature review regarding Displacement Based Design (DBD) and its implementation on RC frame structures. Also literatures concerning site specific response spectra has been discussed.

Chapter 3 contains the basic information of DBD procedure on RC frame structures. Application of DBD procedure on single storey RC frame building has been presented. Also comparison of the same has made with traditional FBD as per IS 1893 (Part 1): 2016

Chapter 4 Application of DBD procedure on multistorey buildings has been discussed. Similar buildings are analysed using traditional force based design as per IS 1893 (Part 1): 2016

Chapter 5 Performance evaluation of the frames designed using DBD procedure is carried out using nonlinear static and time history analysis.

Chapter 6 presents development of the response spectra at different sites using the one dimensional equivalent linear analysis based software ProSHAKE. Comparison of results of building analysed using site specific displacement spectra.

Chapter 7 summarizes work carried out in present study. Conclusion and future scope of the work has also been discussed.

Chapter 2

Literature Survey

2.1 General

The present chapter is focused towards literature study involving theory and application of DBD in RC structures. Several literatures have been studied to understand parameters involving DBD procedure. The basic philosophy behind DBD is that structural damage is better characterized by deformations. Literatures referring to performance evaluation of DBD to understand seismic response of RC building under design level earthquake have also been studied.

2.2 Literature Review

Various literatures related to study of displacement based design of R.C. structures are given in brief:

2.2.1 Basics of Displacement Based Design

Priestley et al.[1] discussed procedure for performing DBD on RC frame building.The equations for computing yield and design displacement are presented. Capacity design procedure is explained so that the hinges do not form on column. The problems with current force-based design,seismic input for displacement-based design , fundamentals of direct displacement- based design, and analytical tools appropriate for displacement-based design are also discussed. DBD approach is explained for other structures such as bridges, shear wall buildings, wall-frame buildings, etc.

Paulay T. and Priestley M. J. N.[2] discussed the shortcomings of FBD, design seismic input for DBD, fundamental considerations of DBD and analytical tools appropriate for displacement-based design. Important parameters related to DBD are also discussed. The design procedure is based on secant stiffness at maximum displacement and equivalent viscous damping. This is a simple approach and allows an inelastic system to be designed and analyzed using elastic displacement response spectra. The theory and application of DBD is explained to all kinds of structures such as bridges, shear wall buildings, wall-frame buildings, etc.

Shibata and Sozen[3] developed the substitute-structure method capable of representing a multi-degree of freedom structure to a single degree of freedom structure. The substitute-structure method is a procedure for determining the design forces, corresponding to a given type and intensity of earthquake motion represented by the design spectrum, for a reinforced concrete MDOF structure. The method is explicitly a design (and not an analysis) procedure. The central and significant feature of the substitute-structure method is that it provides a simple vehicle for taking account of inelastic response of reinforced concrete in the design of multi-degree-of-freedom structures.

Moehle J. P[4] identified the importance of structural displacement as a main determinant of structural and non-structural damage during an earthquake. Displacement concepts for multi-storey frames are discussed. Two approaches to design and evaluate using drift information are discussed. First is the ductility based approach that uses displacement information indirectly, establishing ductility requirements as a function of provided strength and the strength required for elastic response. The second is displacement based approach that uses displacement information directly and forms the main subject of this paper. The displacement based approach can be used to establish proportions and layout that will control drift demand, and to determine structural and non-structural details that will ensure adequate performance. The examples demonstrate that the displacement based approach is a simple and effective tool for design.

Kennedy D. J. L and Medhekar M. S.[5] discussed acceleration-based design method used in seismic codes and its limitations. Alternative method that uses displacement as basis for design procedure is discussed, that is applied to SDOF and MDOF structures for inelastic seismic design. The advantages of DBD method over the spectral acceleration-based design method are also discussed. The theoretical basis for displacement based design and design procedure for displacement based design for SDOF and MDOF system are discussed. The effect of torsion is also incorporated in the paper.

Massena B.,Bento R., and Degee[12] discussed DBD method for reinforced concrete frames. Case study of application of DBD method to the interior frame of four storey reinforced concrete structure is presented. The structure is irregular in terms of spans. Two cases were discussed where the first frame building is designed according to DBD considering Portigal seismic codes taking PGA of 0.35g. The second frame building is considered when the displacement capacity exceeds the spectral demand where PGA of 0.27g is taken.

Park R. and Paulay T.[14] discussed useful information on ultimate deformation and ductility of members with flexure. Theoretical moment-curvature for reinforced concrete sections with flexure and axial load are derived. Ductility of unconfined beam sections and unconfined column sections are discussed. Compressive stress block parameters for concrete confined by rectangular hoops and theoretical moment-curvature curves for sections with confined concrete are discussed in detail.

Thacker, P.K. and Purohit, S.P.[15] discussed implementation of DBD on single storey building (SDOF) and multi storeyed building (MDOF). The advantages of DBD over FBD are also presented. The traditional FBD approach given in IS 1893 (Part 1) :2002 is reviewed and its limitations are discussed. Single storey R.C.C. building (SDOF) is analyzed by FBD and DBD and the parameters like time period, stiffness and base shear are compared. The influence of percentage reinforcement on curvature and stiffness are also examined by performing parametric study. The influence of beam aspect ratio on yield displacement, displacement ductility and equivalent viscous damping are examined through parametric studies. FBD is proved to be conservative in nature while DBD is quite realistic and simple in implementation.

Jinadni M, Purohit S.P. and Suthar J.M.[16] discussed implementation of DBD to single storey building modeled as Single Degree of Freedom (SDOF) system. Comparison among FBD and DBD of single storey building is also presented. DBD is implemented to two buildings with different structural systems, namely, Wall-Frame and Shear Wall. A parametric study is carried for shear wall building with respect to the height of the building. A parametric study with respect to base shear contribution of wall and frame is carried out. An inelastic design spectrum is developed from design spectrum given in

IS 1893 (Part 1): 2002. A four Storey Reinforced Concrete Frame building is considered to implement DBD using Inelastic Design Spectrum.

2.2.2 Performance Evaluation of RC Buildings using Direct Displacement Design(DBD)

Aidcer L. Vidot-Vega and Mervyn J. Kowalsky[13] performed non-linear time history analysis of six different reinforced concrete moment frames which are designed designed using displacement-based design (DBD) and traditional force-based design methods. The interstorey drifts, displacements, and material strains obtained from the analyses of the frames designed using both design methods are compared. The implications of code implied ductility and allowable drifts were also studied. Target steel tensile strains and interstorey drifts for the frames designed using DDBD correlated well with the values obtained from the analysis.

Muljati I., Kusuma Amelinda and Hindarto Fonny[17] observed the effect of out-of-plane offset of frame in a six-storey moment-resisting frame system designed using DBD method for two different earthquake level. Direct displacement based design approach has been discussed where multi –degree of freedom (MDOF) system is transferred into a substitute structure in a single degree of freedom (SDOF) system model. The offset frame is assumed to be in-plane with the adjacent frame, the existence of offset is ignored and the structure is designed as regular MRF. The target storey drift is set as 2.5% .Non-linear time history analysis is used to verify the structural performance based on three parameters: storey drift, damage indices and structure failure.

Muljati I., Asisi Fransiscus and Willyanto Kevin[18] evaluated performance of DBD on a regular concrete special moment resisting frame compared to two varians of FBD, equivalent lateral force procedure and response spectrum analysis. The internal forces are generated by ETABS. The storey drift design target is set 2% for all methods. All design methods run in a single cycle of design without any effort to improve the performance level to experience the effectiveness of the method. All methods were designed using the latest Indonesian seismic code and verified using the exact method nonlinear time history analysis. Response spectra for two cities of low and high seismicity were selected. The parameters used for evaluating structural performance are storey drift, damage indices and structure failure mechanism.

The aim of the work presented by **Muljati I., Lumantarna Benjamin, Intan Reynaldo P. and Valentino Arygianny** [19] is to verify the performance of regular plan concrete building designed using DBD with nonlinear time history analysis based on parameters: drift, damage indices and structure failure mechanism. The maximum storey drift target is set 2%. The excitation is spectrum consistent accelerogram based on El-Centro 1940 N-S, to match with the Indonesian response spectrum for soft soil in low- and high- intensity area.

2.2.3 Site Specific Ground Response Analysis

Importance of site specific ground response analysis and difficulties faced in conducting a complete ground response analysis were presented in **Govinda Raju et al.**[20]. In this paper, ground response analysis of a site in Ahmedabad City during Bhuj earthquake is also discussed. One dimensional equivalent linear analysis was taken for ground response analysis in Ahmedabad during Bhuj earthquake. Data of soil exploration composed from an agency in Ahmedabad. SHAKE91 software based on One Dimensional Equivalent Linear Analysis is used to evaluate effect of local soil condition on ground response during earthquake. Finite Element Analysis Package (NISA) software was used for analysis of RC multistorey building plane frames. Deep alluvial deposits with partially saturated conditions were observed which suggest that there are minimum possibility of occurrence of liquefaction and surface settlement. Because of transfer of large accelerations to high rise buildings by soil amplification, high degree of damage to multi-storey buildings was observed.

Dharna Dilip M.,Patel P.V.[21] presented the study on ground response analysis of eleven sites of Ahmedabad city using one dimensional equivalent linear analysis. ProSHAKE software is used to develop site specific response spectra . Acceleration time history of Bhuj Earthquake recorded at Passport Office, Ahmedabad is considered as input motion for different sites taken to obtain acceleration time history and response spectra at ground. The site specific response spectrum for various sites are compared with the response spectrum plot provided in IS: 1893 (Part I)-2002. Response of multi-storied frame structures is evaluated using site specific response spectra with ETABS software. 3 to 20 storied regular frame structures are analysed to calculate base shear of various sites. Also shear forces, bending moments and axial forces for ground floor columns of the structures are obtained using site specific response spectra. Comparison of analysis result of site specific response spectrum analysis is made with that obtained using IS: 1893 (Part I)-2002 response spectrum. Two multi-storied frame structures are also designed.

Adhikary Shrabony, Singh Yogendra, Paul D.K.[24] discusses the effect of soil depth on inelastic displacement of three typical structures namely, a tower, a four storey building and a continuous bridge. Adequacy of the site amplification models of the current design codes and available empirical relationships were examined. The structures were considered to be situated on sites with varying bedrock depths. Impact of of depth on elastic response, spectrum site amplification factor,displacement modification factor and inelastic displacement was discussed. Here, two PGA values were considered. The results show that elastic and inelastic response of structure is greatly effected by soil depth. Although, elastic response is more effected by soil amplification in contrast to inelastic displacement. It was thus observed that estimation of inelastic response by use of empirical site amplification models on the basis of elastic response may be too conservative.

2.3 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes the development of displacement-based design method, and implementation of this method to SDOF and MDOF systems. It also includes performance evaluation of RC building designed using DBD. This review helps to develop basic understanding of DBD. Also concepts of site specific response analysis, characteristics and geotechnical parameters of soil are presented.

Chapter 3

Displacement Based Seismic Design of RC Structure

3.1 General

Current seismic design codes used for seismic design of structures are based on Force-Based Design approach. The traditional seismic design of structure is primarily based on forces. This methodology completely misses an important parameter, structural displacement, which is a measure of damage in structures subjected to earthquake excitations. Strain and drift can be integrated to give displacement. Therefore, both structural and non-structural damage can be related to displacement. Many researchers have shown that Direct Displacement Based approach performed well in predicting seismic demands of the structure.

3.2 Force Based Design

The seismic design in all current codes and standards uses force based approach. Force based design [7] procedure as per IS 1893 (Part1):2016 has been discussed as follows:

1. Assume structural dimensions and calculate the seismic weight of the building

2. Calculate member stiffness based on primarily estimated member sizes.

3. Based on the assumed member stiffness, the fundamental period is calculated.

$$T = 2\pi\sqrt{\frac{m_{eff}}{K}} \tag{3.1}$$

where m_{eff} is the seismic mass of structure.

Generally in codes height dependent time period is provided which is independent of member stiffness, mass distribution or structural geometry.

4. Zone factor (Z), Importance factor (I), Response reduction factor R and Damping percentage are chosen. Here, Z depends on seismic activities estimation in various regions of country. Importance factor defined based on implementation of structure under consideration. R is a function of parameters like over strength and ductility of material. Damping coefficient depends on type of structure.

5. Calculate the Spectral acceleration coefficient (S_a/g) from Design Response Spectra depending on type of the soil which is based on site conditions viz. hard, medium or soft.

6. Calculate design horizontal seismic coefficient (A_h)

$$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} \tag{3.2}$$

7. Calculate Base Shear (V_b)

$$V_b = A_h \times W \tag{3.3}$$

8. Distribute design base shear (V_b) along the height

$$Q = V_b \frac{W_i h_i^2}{\sum_{j=1^n} W_j h_j^2} \tag{3.4}$$

where

Q_i = design lateral force at floor i,

W_i = seismic weight of floor i,

h_i = height of floor i measured from base,

n = number of storeys in building

9. Analyze structure under seismic force.

10. Check the Displacement.

3.2.1 Problems with Force Based Design

- FBD relies on elastic characteristics of structure to determine inelastic performance of the structure which is inappropriate. Forces between different elements are distributed in proportion to their assumed initial stiffness which indicates stiffness is independent of strength. Detailed analysis and experimental evidence shows that this assumption is invalid and stiffness is essentially proportional to strength. It is incorrectly assumed that the different elements can be forced to yield simultaneously.

- Seismic codes consider height dependent equations to estimate time period of structure resulting in low time periods. This leads to conservative design of structure.

- Different codes provide different force-reduction factors for identical systems and materials. These values appear to be arbitrary, are difficult to justify and do not appear to have been established consistently by experiments or analysis.

- Displacement check is made at the end of design process which shows lack of concern about implied inelastic displacements.

3.3 Displacement Based Design

In Displacement Based Design [1], structures are designed to achieve a specified deformation state under the design level earthquake, rather than achieve a displacement that is less than a specified displacement limit. Thus, different structures designed by DBD have uniform risk of damage. The DBD utilizes secant stiffness to maximum displacement based on Substitute Structure characterization and an equivalent elastic representation of hysteretic damping at maximum response. The stiffness and time period are the end products rather than input quantities.

3.3.1 Basic Formulation of Displacement Based Design

The structure under consideration is represented as a single degree of freedom system. Its performance at peak displacement response is considered, instead of its initial elastic characteristics. The Substitute-Structure Approach is used to model an inelastic MDOF system as an equivalent elastic SDOF system.

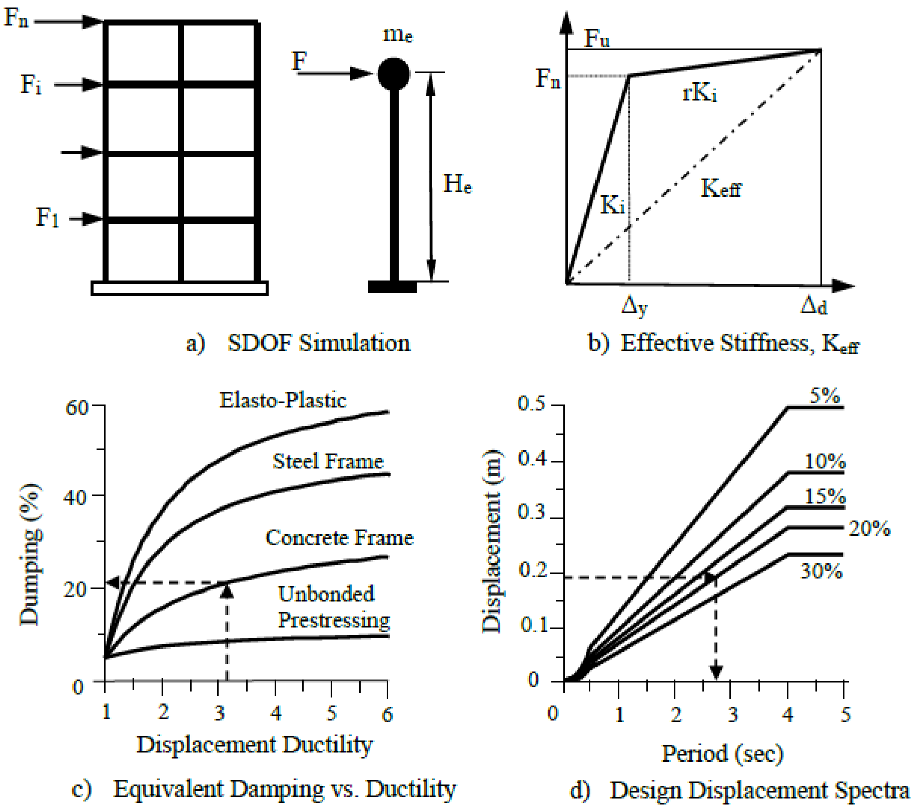


Figure 3.1: Fundamentals of Displacement Based Design [12]

DBD characterizes the structure by secant stiffness K_e at maximum displacement $\triangle_d$ .The damping considered for the system is an equivalent viscous damping, ξ_{eq} which consists of combined elastic damping and hysteretic energy absorbed during inelastic response. Since the effective properties of the substitute-structure are elastic, a set of elastic displacement response spectra can be used for design. The substitute structure-approach allows an inelastic system to be designed and analyzed using elastic displacement response spectra.

Effective time period, T_e at maximum displacement $\triangle_d$ response can be obtained from a set of displacement spectra for different damping levels as shown in Figure 3.1. The effective stiffness K_e of the equivalent SDOF system at maximum displacement can be obtained as:

$$K_e = \frac{4\pi^2 m_{eff}}{T_e^2} \tag{3.5}$$

Design base shear force is given as:

$$F = V_{base} = K_e \triangle_d \tag{3.6}$$

3.4 Comparison of FBD and DBD

The comparison of FBD and DBD is being discussed as given in the following Table 3.1 below:

Table 3.1: Comparison of FBD and DBD

Sr. No.	Force Based Design	Displacement Based Design
1	Structures are characterized by initial stiffness and elastic damping .	Structures are characterized by secant stiffness and an equivalent damping.
2	Elastic natural vibration period is considered.	Effective period is considered.
3	Design acceleration response spectrum is used to find the elastic base shear force which is reduced by using force reduction factor.	Design displacement response spectrum is used to find the base shear and no force reduction factor is applied.
4	Initial stiffness is assumed based on the plan dimensions.	Secant stiffness is used.
5	Check for displacement is carried out at the end of the procedure, in order to maintain displacement within limit.	Displacement limit is chosen, and the analysis are carried out for that displacement.
6	Force reduction factor depends on type of material and type of structure.	Ductility ratio depends on design displacement and yield displacement.

3.5 Displacement-Based Seismic Design Procedure

In this approach, multi degree of freedom (MDOF) system is represented by single degree of freedom (SDOF) system based on substitute structure approach [3]. This procedure is presented in Figure. 3.2

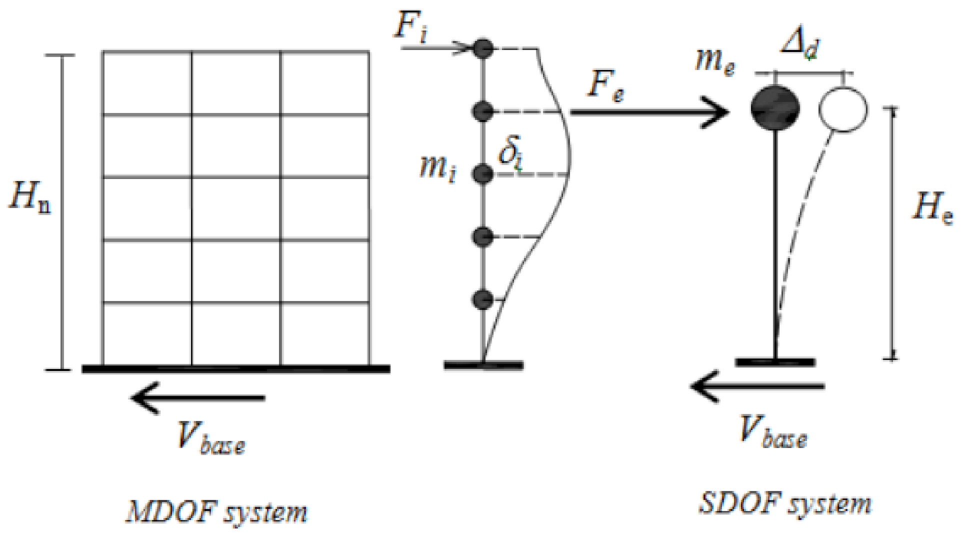


Figure 3.2: Simplified model of a multi-storey building [12]

The procedure for Displacement-Based Seismic Design [1] has been described as follows.

1. **Design Story Displacement**

   The design storey displacement is related to normalized inelastic mode shape which is given as:

$$\triangle_i = \delta_i \left(\frac{\triangle_c}{\delta_c} \right) \tag{3.7}$$

   where, δ_c = mode shape at critical storey, $\triangle_c$ = storey displacement at critical storey The inelastic mode shape δ_i, where i=1 to n is given as:

$$\delta_i = \frac{H_i}{H_n} \qquad \text{for } n \leq 4 \text{ ;} \tag{3.8}$$

$$\delta_i = \frac{4}{3} \left(\frac{H_i}{H_n} \right) \left(1 - \frac{H_i}{4H_n} \right) \qquad \text{for } n > 4 \text{ ;} \tag{3.9}$$

   where H_i= height of storey i, H_n= total height of structure and n= number of stories

2. **Design Displacement of the System**

   The design displacement of the equivalent SDOF system is given as:

$$\delta_d = \frac{\sum_{i=1}^{n} m_i \triangle_i^2}{\sum_{i=1}^{n} m_i \triangle_i} \tag{3.10}$$

   where m_i = mass of storey i, $\triangle_i$= height of storey i

3. **Effective Mass of System**

The effective mass of the equivalent SDOF system is given as:

$$m_e = \sum_{i=1}^{n} \frac{m_i \triangle_i}{\triangle_d} \tag{3.11}$$

4. **Effective Height of System**

The effective height of the equivalent SDOF system is given as:

$$H_e = \frac{\sum_{i=1}^{n} m_i \triangle_i H_i}{\sum_{i=1}^{n} m_i \triangle_i} \tag{3.12}$$

5. **Consider Higher Mode Effects**

In tall structures higher mode effects are also taken in consideration by taking into account amplification factor ω_θ which transforms the target design displacement through:

$$\triangle_{i,\omega} = \omega_\theta \triangle_i \tag{3.13}$$

$$\omega_\theta = 1.15 - 0.0034 H_n \leq 1 \tag{3.14}$$

6. **Design Displacement Ductility of System**

The design displacement ductility of the equivalent SDOF system is given as:

$$\mu = \frac{\triangle_d}{\triangle_y} \tag{3.15}$$

where $\triangle_y$ is yield displacement which is given as:

$$\triangle_y = \theta_y H_e \tag{3.16}$$

Here, θ_y is yield rotation defined as:

$$\theta_y = \frac{0.5\epsilon_y . L_b}{H_b} \tag{3.17}$$

where, L_b and H_b are the beam span and overall depth of beam repectively.

7. **Equivalent Viscous Damping of System** Then the equivalent viscous damping of the equivalent SDOF system is obtained as:

$$\xi_{eq} = 0.05 + 0.565 \left(\frac{\mu - 1}{\mu\pi}\right) \tag{3.18}$$

Here, 0.05 is the elastic damping and the remaining part is hysteretic damping.

8. **Effective Period of System**

The effective period of the system at peak displacement is obtained from displacement spectra as shown in Figure. 3.3.

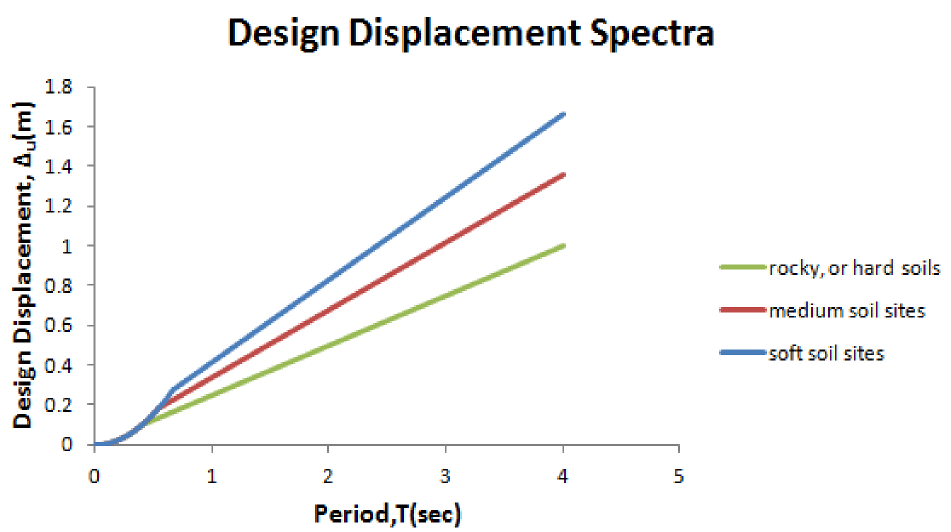


Figure 3.3: Design Displacement Spectra as per IS 1893 (Part 1):2016

9. **Effective Stiffness of System**

The effective stiffness of the equivalent SDOF system is obtained as:

$$K_e = \frac{4\pi^2 m_e}{T_e^2} \tag{3.19}$$

10. **Design Base Shear Force**

The design base of the system is obtained as:

$$F = V_{base} = K_e \triangle_d \tag{3.20}$$

11. **Distribution of Design Base Shear Force to Floor Levels**

The base shear force is distributed throughout the height of the building as:

$$F_i = 0.9V_{base}\left[\frac{m_i\triangle_i}{\sum_{i=1}^{n} m_i\triangle_i}\right] \qquad \text{for typical storey} \tag{3.21}$$

$$F_i = 0.1V_{base} + 0.9V_{base}\left[\frac{m_i\triangle_i}{\sum_{i=1}^{n} m_i\triangle_i}\right] \qquad \text{for roof level} \tag{3.22}$$

12. **P-Delta Effects**

$P - \triangle$ effect is considered in most seismic design codes. This effect is controlled using stability index $\theta_\triangle$:

$$\theta_\triangle = P\frac{\triangle_d}{M_d} \tag{3.23}$$

If stability index $\theta_\triangle \leq 0.1$, then design base shear force is given by eq.3.20, else if $\theta_\triangle > 0.1$ design base shear force is given by eq.3.24

$$V_{base} = K_e\triangle_d + C.P\frac{\triangle_d}{M_d} \tag{3.24}$$

where C=0.5 for concrete structure,

P = total wight of the structure,

M_d=total overturning moment and

H= height of the structure

3.6 Illustrative Example

In this example , application of DBD and FBD method has been carried out on single story building (SDOF) system. The parameters that are compared are time period, stiffness, base shear force and ductility.

Following data defines the building configuration [15]:

Slab thickness: 125mm, M20 grade

Beam size: 300mm × 450mm, M25 grade

Column size: 450mm × 300mm , M25 grade

Panel size: 5m × 3m

Height of column: 3m

Density of concrete: 25 kN/m^3

Parapet wall: 150mm thick, 1 m high, Density: 20 kN/m^3

Floor Finish, FF = 1 kN/m^2, Live Load, LL = 2 kN/m^2

Modulus of elasticity of steel, E_s = 2 × 10^5 N/mm^2

Characteristic yield strength of longitudinal steel, f_y = 415 N/mm^2

Characteristic yield strength of transverse steel, f_{yh} = 415 N/mm^2

Zone V, Importance factor, I = 1,Response Reduction Factor, R =3, Medium Soil

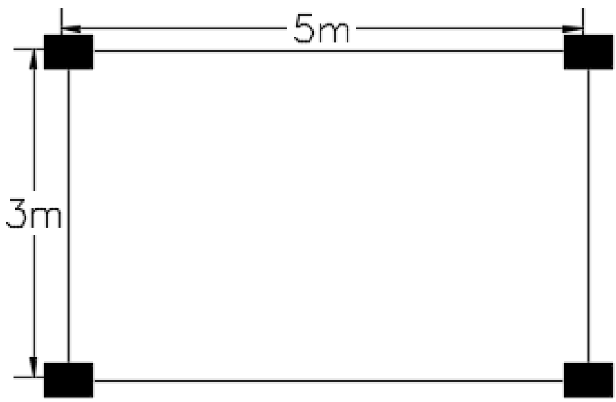


Figure 3.4: Plan of Single Story Building

3.6.1 Displacement Based Design

Total load = DL + LL = 211.0875 kN

Factored load on each column = $\frac{1.5(DL+LL)}{4}$ = 79.16 kN

Provide 6 nos. of bar of 16 mm diameter and 2-legged 8mm ϕ stirrups at 150mm c/c in x and y-direction.

From RC Analysis [23], following results are obtained :-

Yield curvature in x-direction,ϕ_y = 0.0087 m^{-1}

Ultimate curvature in x-direction,ϕ_u = 0.179 m^{-1}

Yield displacement of the system is given by eq.3.25

$$\triangle_y = \phi_y \frac{(H + L_{sp})^2}{3} \tag{3.25}$$

Therefore, Yield displacement in x-direction, $\triangle_y$ = 28.703 mm

Design displacement, $\triangle_u = \triangle_y + (\phi_u - \phi_y)L_pH$

where,plastic hinge length, $L_p = kL_c + L_{sp} \geq 2L_{sp}$

and $k = 0.2(f_u/f_y - 1) \leq 0.08$

Here, L_c = length from the critical section to the point of contraflexure in the member.

f_u = ultimate steel stress

H = height of column So, k = 0.06

and L_p = 326.08 mm

Therefore, design displacement = 195.3 mm

Now, displacement ductility,$\mu_\triangle = \triangle_u/\triangle_y$ =195.3/28.703 =6.8

Equivalent viscous damping for concrete frame building,

$\xi_{eq} = 0.05 + 0.565(\mu_\triangle - 1)/(\mu_\triangle \pi)$ = 20.34 %

Zone specific design displacement, $\triangle_u{}'$= 195.3 x (0.36/2)=0.035 m

Time period obtained from IS 1893 displacement spectra is T= 0.751 sec

Effective stiffness, K= $4\pi^2$m/T^2 = 1.35 x 10^6 N/m

Base shear force in x-direction, V_b= K$\triangle_u$ =47.25 kN

Similarly, Base shear force obtained in y-direction, V_b= 31.59 kN

3.6.2 Force Based Design

Total Dead load, DL= 181.075 kN

Total Live Load, LL = 0.25x30= 7.5 kN (as per IS 1893 (Part 1)

Therefore, Total Load, W =189 kN

Stiffness of structure in x-direction,K = 12EI/L^3=4.5x10^7 N/m

Mass of the building, m= W/g =19266.055 kg

Natural frequency of the system in x-direction, ω= $\sqrt{k/m}$=72.49 rad/sec

Time period of the system in x-direction, T= $2\pi/\omega$=0.087 sec

Design acceleration coefficient, S_a/g=2.305 (as per IS 1893:2002)

Now, design acceleration in x-direction, A_h = Z/2 x I/R x S_a/g=0.1383

Base shear force in x-direction,V_b= A_hW = 26.1387 kN

Similarly, Base shear force in y-direction,V_b= A_hW = 28.35 kN

3.6.3 Comparison of FBD and DBD method on single storey building

As can be seen from Table 3.2, time period obtained from FBD method is less as compared to DBD method. Also, the stiffness of the building is more in FBD method because it takes the elastic stiffness of structure whereas in DBD method stiffness is less because of the fact that it takes secant stiffness at maximum displacement which is less than elastic stiffness due to cracks formed in structure. Base shear force in FBD is less than that obtained in DBD method. The ductility obtained in FBD is less than that in DBD method.

Table 3.2: Comparison of FBD and DBD method on single story building

Response Quantities	Force Based Design		Displacement Based Design	
	x-direction	y-direction	x-direction	y-direction
Time period (sec)	0.087	0.13	0.74	1.13
Stiffness, (N/m)	1.0124×10^8	4.5×10^7	1.35×10^6	0.596×10^6
Base Shear (kN)	26.1387	28.35	47.25	31.59

3.7 Summary

In this chapter a basic information regarding application of displacement based design on RC Frame Buildings has been discussed in detail. Representation of multi degree of freedom system into single degree of freedom system has also been discussed. Comparison of DBD and FBD method on single story building has also been discussed. It can be concluded that time period obtained from FBD method is less as compared to DBD method. Also, the stiffness of the building is more in FBD method because it takes the elastic stiffness of structure whereas in DBD method stiffness is less because it takes secant stiffness at maximum displacement which is less than elastic stiffness due to cracks formed in structure. Base shear force in FBD is less than that obtained in DBD method. The ductility obtained in FBD is less than that in DBD method.

Chapter 4

Application of DBD on Multistorey Building

In this chapter application of displacement based approach for design of RC moment resisting frame buildings has discussed. In this regard,four buidings are selected which are same in plan but different in height i.e. 3, 6, 10, 12 storey buildings. For this, one interior frame which is critical in gravitational load has been considered. The typical plan of the building is as shown in Figure 4.1

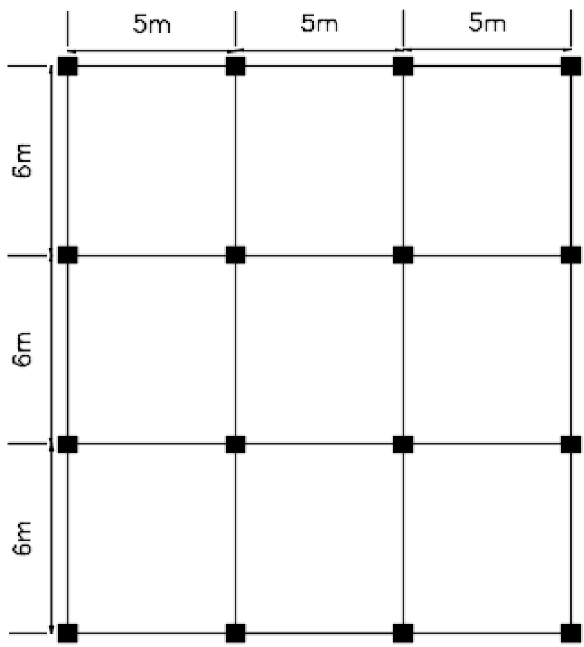


Figure 4.1: Plan Dimension of Building

4.1 Case Study

In this chapter six-storey RC moment resisting frame building is considered. The results of other buildings are discussed in appendices. Plan of the building spans in 15m and 18m in x- and y-direction respectively. Height of each storey is taken as 3m. Following parameters defines the building configuration:

Building Data:

No. Of storeys: 6

Building height: 18m

Plan Dimensions: 15m × 18m

Concrete Grade: M25

Steel Grade: Fe415

No. Of bays in Y-Direction: 3

Size of Beams: 300mm × 450mm

Size of Columns: 500mm × 500mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Response Reduction Factor: 3

Soil type: Medium

4.1.1 Displacement Based Design

A deformation level corresponding to operational and life safety performance levels is selected. According to ATC-40 (Table 11-2 of ATC-40) [10], drift ratios for operational and life safety performance levels are 1% and 2%, respectively. In this case study, 2% drift ratio is selected as target drift limit corresponding to life safety limit. The procedure for analysis of RC frame building by DBD [1] method is:

1. **Design Storey Displacement** Design storey displacement is shown in Table 4.1. The seismic mass at top of each storey is calculated considering 100% dead load and 25% live load as per IS 1893.

Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System

$\triangle_d$= 50.657/240.659 = 0.210 m

Table 4.1: Calculations of Design Displacement

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$ (m)	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_i H_i$
6	18	212.08	1	0.282	59.751	16.834	1075.520
5	15	227.37	0.88	0.248	56.348	13.965	845.223
4	12	227.37	0.741	0.209	47.451	9.903	569.414
3	9	227.37	0.583	0.164	37.368	6.141	336.310
2	6	227.37	0.407	0.115	26.098	2.996	156.589
1	3	227.37	0.213	0.06	13.642	0.819	40.927
Total	-	1348.93	-	-	240.659	50.657	3023.982

2. **Effective Height of the System**

   H_e = 3023.982/240.659 = 12.536 m

3. **Effective Mass of the System**

   m_e = 240.659/0.21 = 1143.306 ton

4. **Design Ductility Factor of the System** Yield strain is given as:

$$\epsilon_y = \frac{f_{ye}}{E_s} = \frac{1.1 \times 415}{2 \times 10^5} = 2.283 \times 10^{-3} \tag{4.1}$$

Yield rotation is given as:

$$\theta_y = 0.5\epsilon_y \frac{L_b}{H_b} = 0.015 \tag{4.2}$$

Therefore, yield displacement is obtained as:

$$\triangle_y = 0.015 \times 12.56 = 0.191m \tag{4.3}$$

Hence, ductility is found as:

$$\mu = \frac{0.21}{0.191} = 1.101$$

5. **Equivalent Viscous Damping of the System**

Equivalent viscous damping for the system is found as:

$$\xi_{eq} = 0.05 + 0.565(\frac{1.101 - 1}{1.101\pi}) = 0.066 = 6.65\% \tag{4.4}$$

6. **Effective Period at Peak Displacement Response**

In this case study, Zone V and medium soil is considered as per IS 1893 (Part 1):2016. For zone V ,Z_2=0.36.

Modification factor corresponding to 6.649% damping is 0.918.

The displacement spectra used for this case study is shown in Figure. 4.2

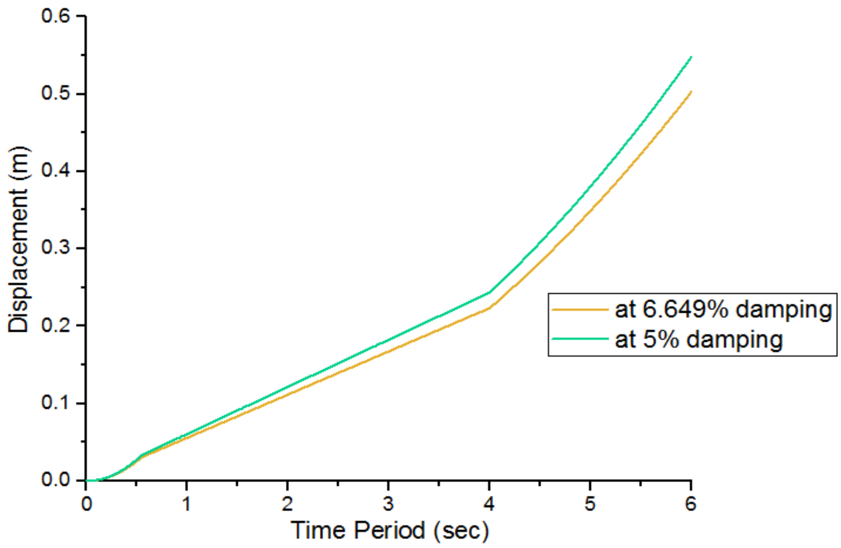


Figure 4.2: Design Displacement Response Spectra

Time period corresponding to design displacement $\triangle_d$= 0.21 m is:

T_e=3.77 sec.

7. **Effective Stiffness of the System**

Effective stiffness of the system,

$$K_{eff} = \frac{4\pi^2 m_e}{{T_e}^2} = 3176.799 kN/m$$

8. **Design Base Shear Force of the System**

Design base shear of the building,

$$V = K_{eff}\triangle_u = 668.696 kN$$

Design base shear of internal frame of building= $V_{base} = V/3$=222.899 kN

9. **Distribution of Design Base Shear Force**

   Distribution of design shear force over the height of building is shown in Table 4.2

Table 4.2: Distribution of Design Shear Force

Storey, i	Height , H_i (m)	$\triangle_i$ (m)	Mass (ton), m_i	$m_i\triangle_i$	$F_{i,x}$ (kN)
6	18	0.282	212.080	59.751	72.097
5	15	0.248	227.370	56.348	46.971
4	12	0.209	227.370	47.451	39.554
3	9	0.164	227.370	37.368	31.149
2	6	0.115	227.370	26.098	21.755
1	3	0.060	227.370	13.642	11.372
Total	-	-	1348.930	240.659	222.899

10. **P-Delta Effects**

    Overturning moment, $M_d = V_b H_n = 668.696 \times 18 = 12036.52$ kNm

    In order find out whether P-Delta effect is to be considered or not, stability index is estimated as: $\theta_{\triangle}$= 0.231,

    Since, $\theta_{\triangle} > 0.1$, so effect of P-Delta is considered and base shear of the frame is modified as V_{base}=248.69 kN

4.1.2 Force Based Design

Seismic Weight of building, W = 13233 kN

Time period of the building, $T = 0.075h^{0.75} = 0.655sec$

For medium soil, $S_a/g = 2.075$

Horizontal seismic coefficient,

$$A_h = \frac{0.36}{2} \times \frac{1}{3} \times 2.075 = 0.124m/sec^2$$

Base shear force of building, $V = 0.124 \times 13233 = 1647.528kN$

Design base shear force of internal frame, $V_b = V/3 = 549.176kN$

Table 4.3 shows distribution of design lateral force along height which is given by,

$$Q_i = \frac{W_i {h_i}^2}{\sum_{i=1}^{n} W_i {h_i}^2} V_b$$

Table 4.3: Design lateral force

Storey, i	Height , H_i (m)	Weight (kN), W_i	$W_i {h_i}^2$	Lateral Force, Q_i (kN)
6	18	2080.5	674082	208.185
5	15	2230.5	501862.5	154.996
4	12	2230.5	321192	99.197
3	9	2230.5	180670.5	55.798
2	6	2230.5	80298	24.799
1	3	2230.5	20074.5	6.199
Total	-	13233	1778180	-

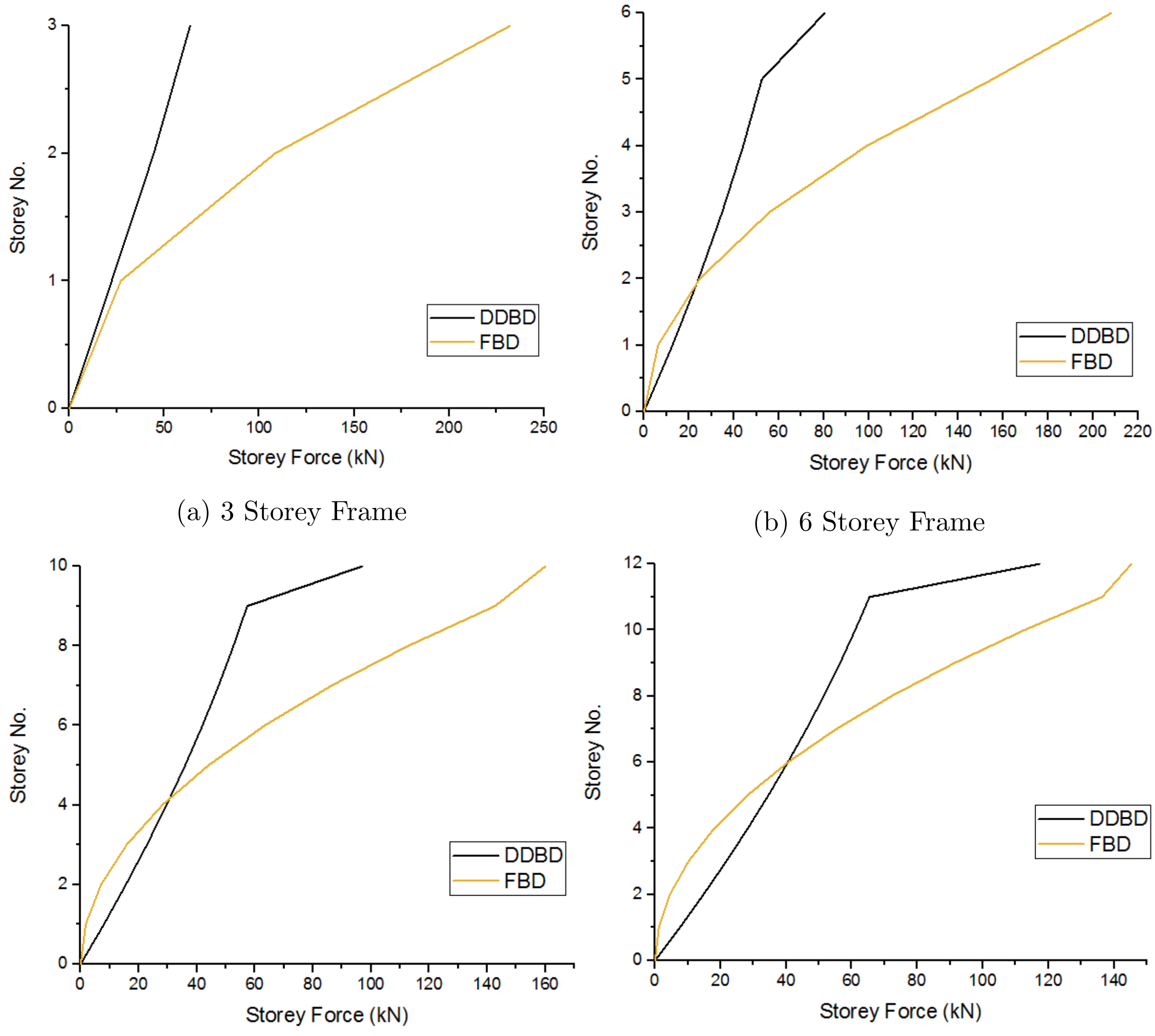
4.2 Results and Comparison

This section discusses the results obtained for 3, 6, 10, 12-storey frames designed with DBD and FBD method. Table 4.4 shows comparison of time period, equivalent damping, design base shear force of 3, 6, 10, 12-storey frames.

Table 4.4: Comparison of DBD and FBD Approach

Total No. of Storeys	3		6		10		12	
	DBD	FBD	DBD	FBD	DBD	FBD	DBD	FBD
Time Period, T (sec)	2.865	0.39	3.767	0.655	4.886	0.961	5.319	1.102
Equivalent Damping, ξ(%)	10.159	5	6.649	5	6.474	5	6.430	5
Design Base Shear of Internal Frame, V_b (kN)	130.374	367.588	248.69	549.176	408.512	662.292	545.012	715.53

Figure 4.3 shows variation of lateral forces along the height of building for DDBD and FBD approach.



(a) 3 Storey Frame

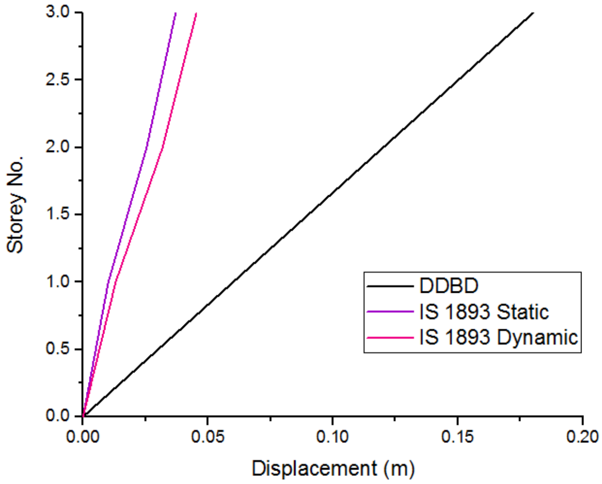
(b) 6 Storey Frame

(c) 10 Storey Frame

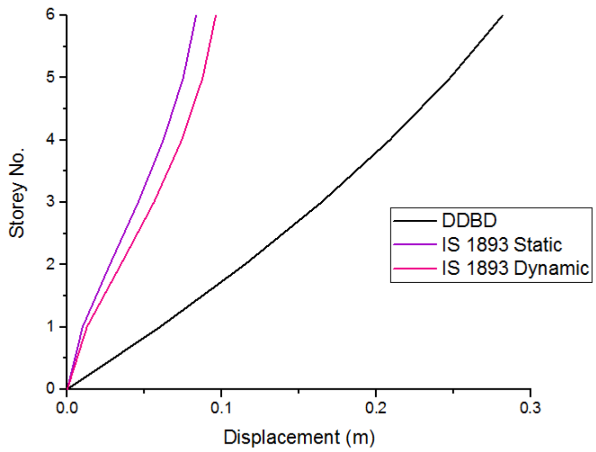
(d) 12 Storey Frame

Figure 4.3: Comparison of Lateral Force by FBD and DBD Approach

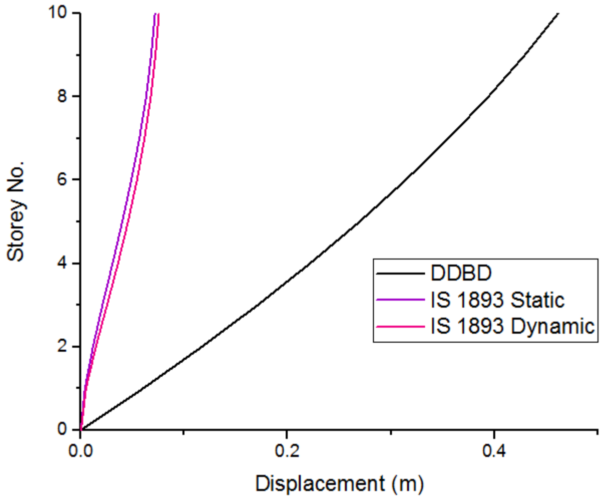
Figure 4.4 shows comparison of displacement along the height of building for DDBD and FBD approach.



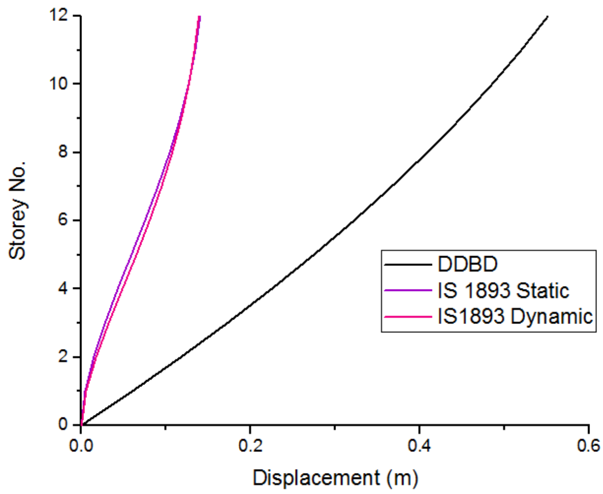
(a) 3 Storey Frame



(b) 6 Storey Frame



(c) 10 Storey Frame



(d) 12 Storey Frame

Figure 4.4: Comparison of Displacement by FBD and DBD Approach

It can be concluded that time period obtained through DBD approach is more than FBD approach. And the stiffness of the system obtained for DBD is less than that obtained for FBD approach because in DBD approach secant stiffness to maximum displacement is taken which gets reduced due to building undergoing inelastic action and thus forming cracks. While in FBD approach, elastic stiffness is taken to estimate inelastic response of structure. Due to reduction in stiffness as a result of formation of cracks, base shear force in DBD is less than FBD approach.

4.3 Summary

In this chapter, implementation of displacement based approach and force based approach on 3, 6, 10, 12-storey RC moment resisting frame building has been presented.For DBD, 2% target drift limit corresponding to life safety limit has been considered. Also comparison of 3, 6, 10, 12-storey building analysed using DBD and FBD has been presented.

The comparison results of DBD and FBD approach show that time period obtained for DBD is more compared to that obtained through FBD method. In turn, the stiffness of building is more in FBD method because it takes the elastic stiffness of structure whereas in DBD method stiffness is less because it takes account of secant stiffness at maximum displacement which is less than elastic stiffness due to cracks formed in structure. As a result, base shear force in FBD is more than that obtained in DBD method. This shows that ductility obtained in FBD is less than that in DBD method.

Chapter 5

Nonlinear Static and Time History Analysis

5.1 Performance Assessment of RC Moment Resisting Frames

In this chapter, performance assessment of 3, 6, 10, 12-storey moment resisting frames that has been presented in previous chapter are discussed. The performance of frames has been assessed through nonlinear static pushover analysis and time history analysis. In this study, damage states of the members of the frames are checked, and the drift ratios are controlled with the one chosen in DBD approach, and it is assumed that there are no nonstructural members in the system.

As per ATC-40, four performance levels have been provided namely, Immediate Occupancy (IO), Damage Control, Life Safety (LS) and Structural Stability. In the present study only Life Safety performance levels has been considered. It includes consideration of damage states for several levels of ground motion. Performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. Target performance level is specified independently. Structural performance levels are given names and number designations while nonstructural performance levels are given names and letter designations.

As per ATC 40 [10], structural performance levels are defined as:

1. **Immediate Occupancy, SP-1** The post-earthquake damage state in which only

very limited structural damage has occured. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristics and capacities. The risk of life-threatening injury from structural failure is negligible, and the building should be free from unlimited egress, ingress, and occupancy.

2. **Damage Control, SP-2** It is not a specific level but provides a range of damage states between SP-1 and SP-3.

3. **Life Safety, SP-3** The post-earthquake damage state in which significant damage to structure may have occured but in some margin against either total or partial structure collapse remains.

4. **Limited Safety, SP-4** It is not a specific level but provides a range of damage states between SP-3 and SP-4.

5. **Structural Stability, SP-5** This state is the limited post-earthquake structural damage state in which the building's structural system is on the verge of collapse.

5.2 Nonlinear Static Analysis

Pushover analysis is a useful and practical tool for the performance evaluation of existing structures and newly designed structures. It is used for inelastic analysis of the structure, under monotonically increasing vector of forces or a vector of displacements. The main outcome of pushover analysis is a capacity curve (Base shear force vs top displacement), from which target displacement, a displacement induced by design earthquake, can be determined. The capacity curve shows nonlinear behavior of the structure under increasing base shear force. Pushover analysis mostly considers first mode of vibration.

In this study, nonlinear pushover analysis is performed in SAP2000 v19. Some points that have been taken into consideration for modelling frames in SAP2000 v19 are discussed below:

- All beams and columns are modeled as 2D frames.

- Cross sectional properties are defined and to consider cracked section stiffness, the gross stiffness of the section is reduced as recommended in IS 1893 (Part 1):2016.

- For beams and columns “Moment-M_3” and “P-M_2” type of hinges are considered. Plastic hinges are assigned at 0.05 and 0.95 relative distance of each member.
- Gravity loads are applied on the frames as distributed loads and concentrated loads on the beams and on joints respectively. Lateral forces obtained through DBD approach are applied at the top of each story at the beam-column joints. As per IS:1893(2002), 100% of gravity loads and 25% of live load is used.
- The displacement is chosen as the controlling parameter for load where monitored displacements at the top of the frames are chosen. P-Delta effects are also considered.

5.3 Nonlinear Time History Analysis

Time history analysis is a step-by-step analysis for dynamic response of structure to specified loading that may vary with respect to time. Ideally time histories need to be reasonably close to the design response spectra. There are two main methods we use to get out time histories to match the design spectra which are as follows:

- **Time History Scaling**- It involves modifying a time history's scale and/or time step in order to try to closely match the design response spectrum. All the ground motions are fudged at a certain time period in which they match the target spectrum. It is done so that the structure can be hit with right spectral quantity at critical time period. The idea is to take average from all these time histories which should be pretty close to our target response spectrum. In addition, the smoothened average spectrum is made to fall within a specified tolerance of the target spectrum. As it is impossible to get this tolerance over entire spectrum, therefore, focus is made on period of range of interest. As per NEHRP (NIST GCR 11-917-15) guidelines, it is to ensure that the average time history remains between the tolerance limit, often, taken as $0.2T_n$ to $1.5T_n$.
- **Spectral Matching**- It involves a time domain modification of an acceleration time history to make it compatible with a user-specified target spectrum.This method is based on the original method proposed by Linhanad and Tseng (1987,1988). Abrahamson (1993) wrote the first widely used computer code for spectral matching

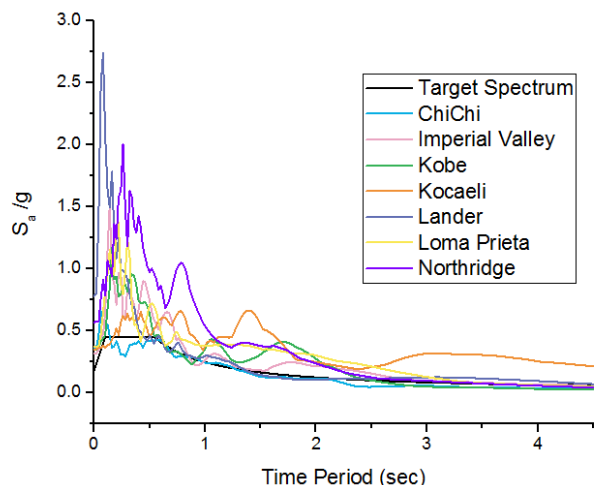
called RSPMatch. It was subsequently updated by Hancock et. al(2006). Spectral matching has the advantage of significantly reducing variability in the computer response because time histories are often very close to the target response spectrum.

In this study, time history scaling method has been adopted to conduct nonlinear time history analysis. Therefore, seven ground motions have been taken and matched with target response spectrum in SeismoMatch Version 2016. For this purpose, elastic acceleration response spectrum utilized in DDBD approach is used as target response spectrum. The ground motions used in the study are presented in Table 5.1.

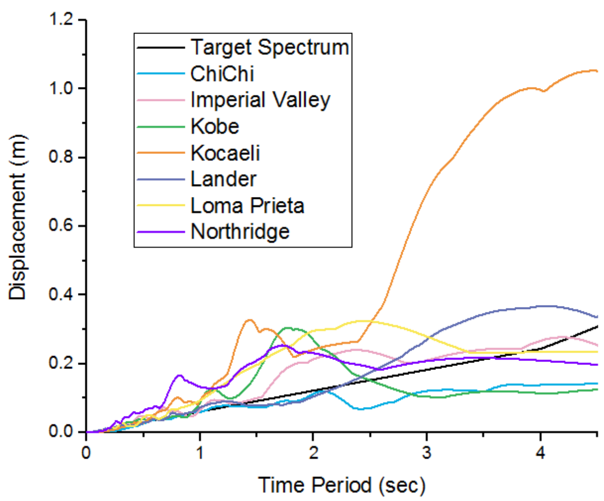
Table 5.1: Time History Motion

Earthquake Name	**Year**	**Recording Station**	**PGA (g)**	**PGV (cm/sec)**	**PGD (cm)**
Imperial Valley (USA)	1979	USGS STATION 5115	0.034	1.003	0.061
Chi Chi (Taiwan)	1999	TCU045	0.356	20.57	21.175
Friuli (Italy)	1976	Tolmezzo	0.277	10.503	3.0552
Kobe (Japan)	1995	Kakogawa	0.171	10.965	3.4036
Kocaeli (Turkey)	1999	Yarimca	0.241	30.779	29.543
Landers (USA)	1992	SCE STATION 24	0.823	41.086	29.829
Loma Prieta (USA)	1989	CDMG STATION 47381	0.342	15.654	7.0354
Northridge (USA)	1994	CDMG STATION 24278	0.217	12.269	5.3028

Original acceleration and displacement of seven motion time histories is shown in Figure 5.1.



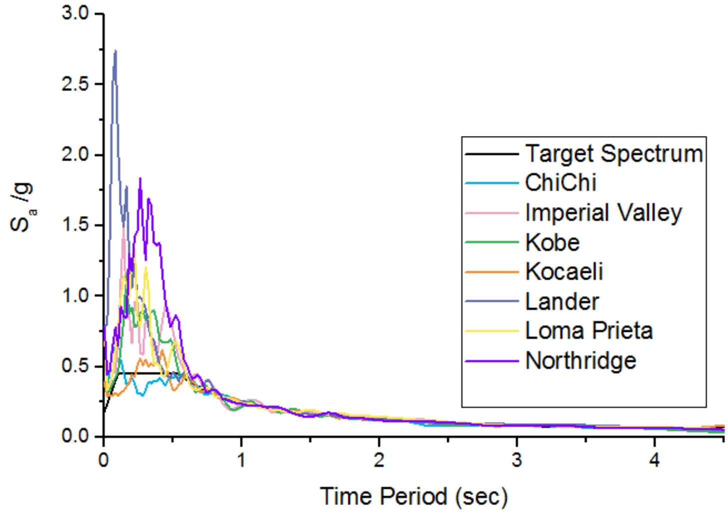
(a) Original Acceleration Time Histories



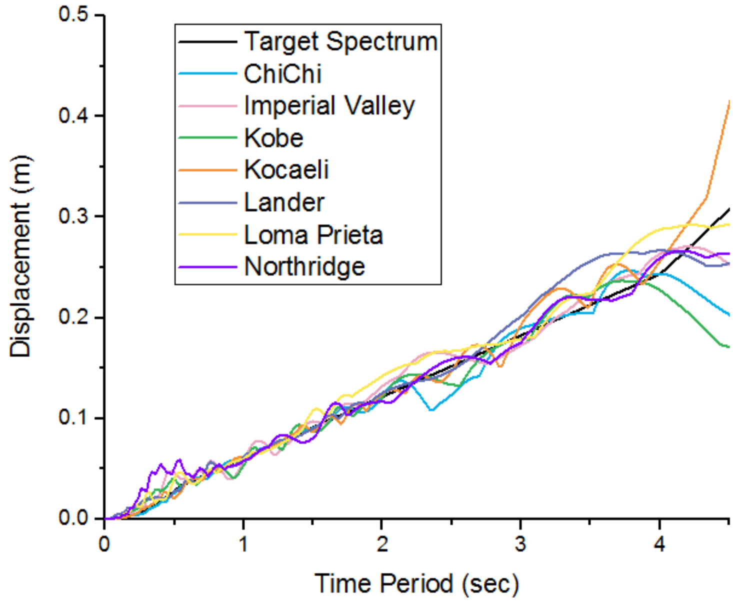
(b) Original Displacement Time Histories

Figure 5.1: Original Acceleration and Displacement of Seven Motion Time Histories

Figure 5.2 shows acceleration and displacement spectra of 3-storey frame for seven time histories that are matched with target response spectra. Here, target response spectra is design basis earthquake (DBE) spectra of zone V. The time histories are matched between $0.2\ T_n = 0.573$ sec and $1.5\ T_n = 4.2975$ sec using SeismoMatch Version.2016



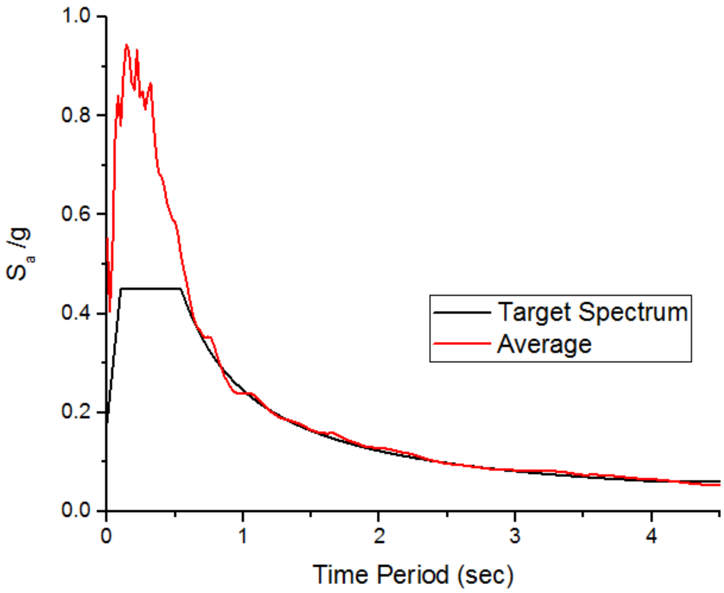
(a) Matched Acceleration Time Histories



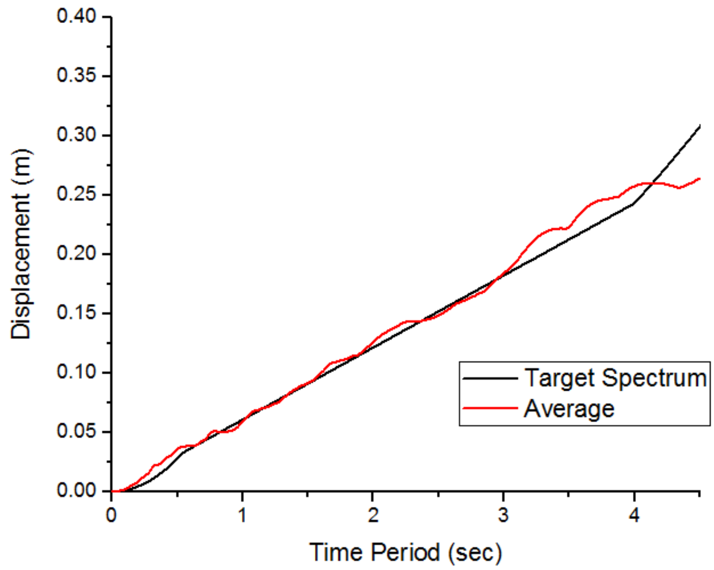
(b) Matched Displacement Time Histories

Figure 5.2: Matched Acceleration and Displacement of Seven Motion Time Histories

Figure 5.3a shows average acceleration spectra of seven motion time histories and target acceleration spectra. Similarly, Figure 5.3b shows average displacement spectra of seven motion time histories and target displacement spectra. It can be seen that the average of these seven time histories are matching finely with target response spectra. Similarly, time history motions are matched with target response spectra for remaining frames.



(a) Acceleration Spectra



(b) Displacement Spectra

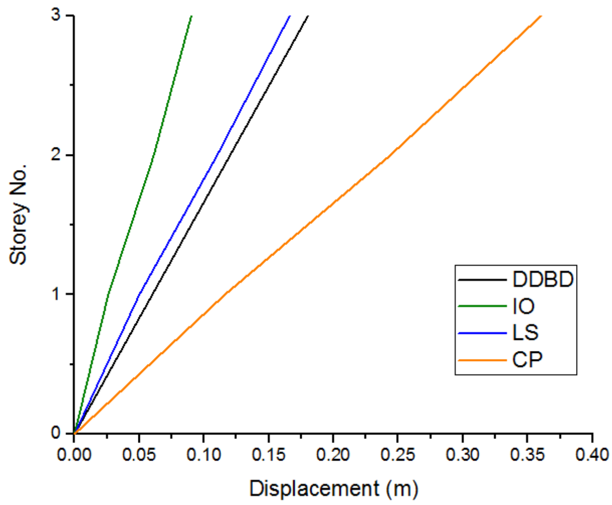
Figure 5.3: Target Acceleration and Displacement Spectra Matched with Average of Seven Motion Time Histories

Modelling of the frames has been done similar to that in nonlinear pushover analysis. Two types of solutions are available for nonlinear time history analysis in SAP2000, modal and direct integration. In modal type of solution, mode superposition is used, while in direct integration type of solution, the equations of motions are being solved for the structure at each time step. In this study, direct integration type of solution is considered for performing nonlinear time history analysis.

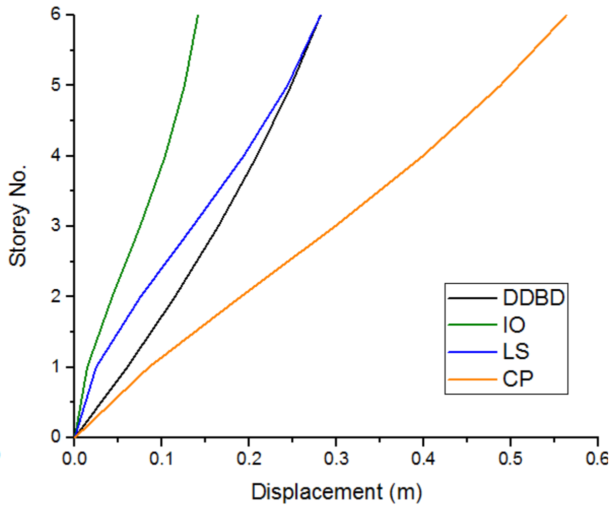
5.4 Results of Nonlinear Analysis

5.4.1 Results of Nonlinear Static Analysis

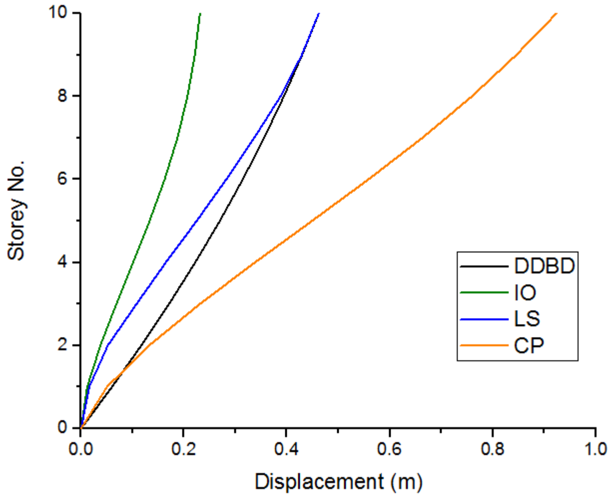
Results of nonlinear static pushover analysis has been presented in terms of capacity curves, displacement profile, interstorey drift ratios and sway mechanism for life safety performance level. In these curves limits for different performance levels are also shown.



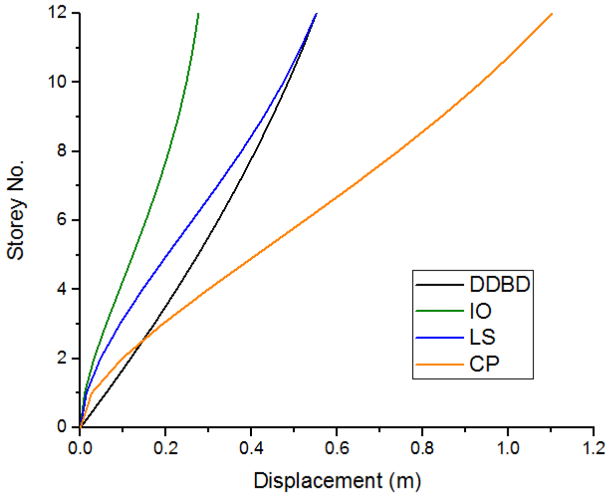
(a) 3 Storey Frame



(b) 6 Storey Frame



(c) 10 Storey Frame

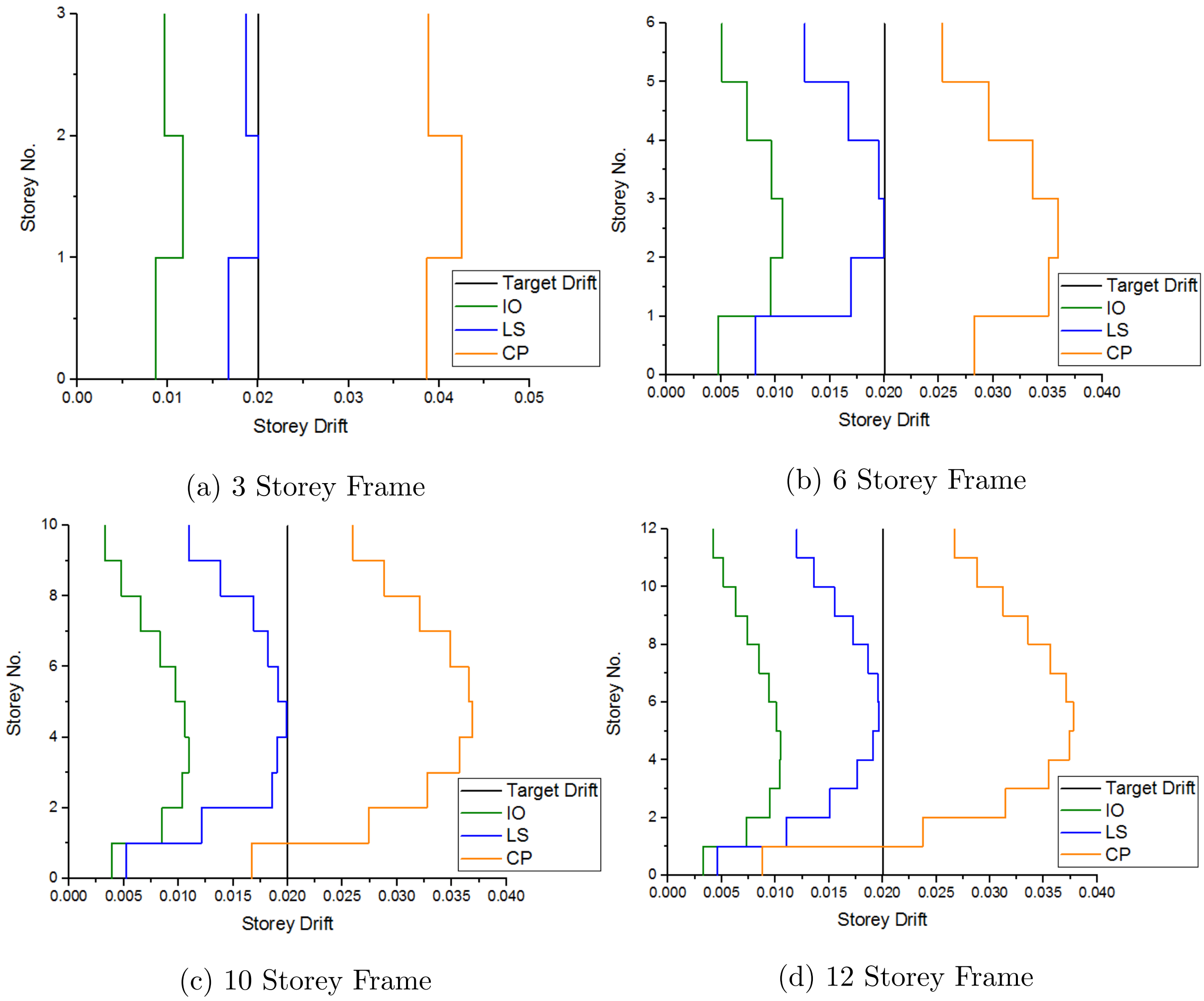


(d) 12 Storey Frame

Figure 5.4: Displacement Obtained Through Nonlinear Static Pushover Analysis

Figure 5.4 shows displacement profile of 3, 6, 10, 12-storey frames. As can be seen, displacement profile for IO is within the displacement profile obtained through DDBD approach. Displacement profile for LS limit is also well within displacement profile of DDBD approach whereas displacement profile for CP limit exceeds it.

Figure 5.5 shows interstorey drift ratio of 3, 6, 10, 12-storey frames.The interstorey drift ratios are shown for Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). As can be seen, interstorey drift ratio for IO is within the life safety drift limit of 0.02. Interstorey drift ratio for LS limit achieves life safety drift limit of 0.02 while interstorey drift ratio of CP exceeds it.



(a) 3 Storey Frame

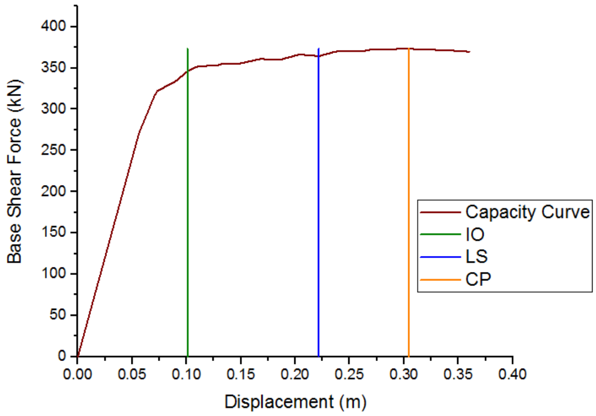
(b) 6 Storey Frame

(c) 10 Storey Frame

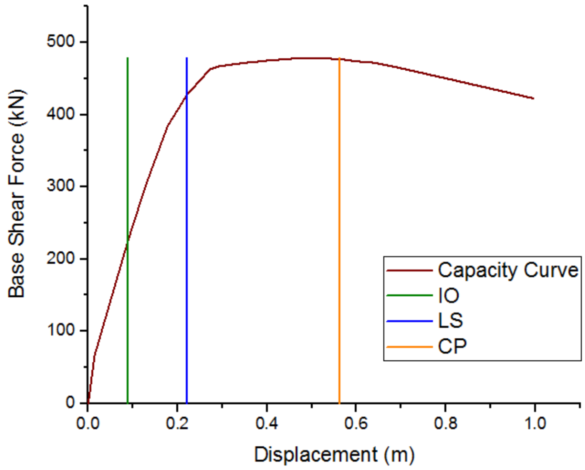
(d) 12 Storey Frame

Figure 5.5: Interstorey Drift Obtained Through Nonlinear Static Pushover Analysis

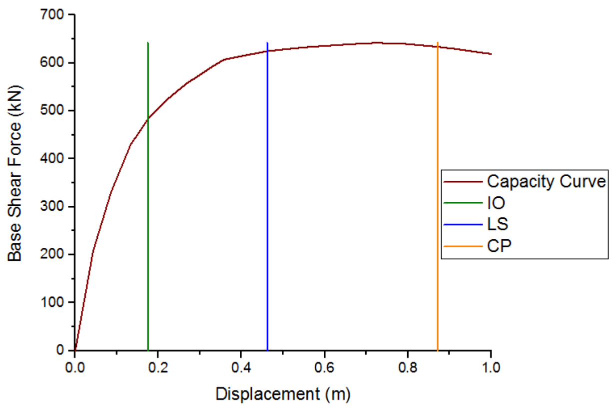
Figure 5.6 shows capacity curves of 3, 6, 10, 12-storey frames for Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) performance limit obtained through nonlinear pushover analysis.



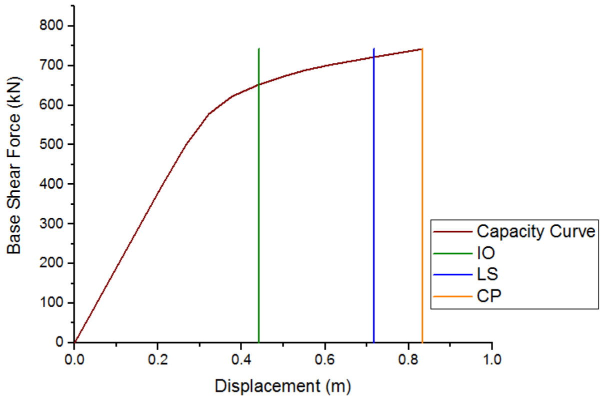
(a) 3 Storey Frame



(b) 6 Storey Frame



(c) 10 Storey Frame



(d) 12 Storey Frame

Figure 5.6: Capacity Curves Obtained Through Nonlinear Static Pushover Analysis

It can be seen from figure 5.6, limits for life safety limit for 3, 6, 10, 12-storey frames in terms of displacement is 0.222 m, 0.22 m, 0.481 m and 0.786 m corresponding to base shear force of 364.347 kN, 427.097 kN, 624.536 kN and 733.813 kN respectively.

The design displacement for 3-storey frame obtained through DDBD approach is 0.139 m corresponding to 130.374 kN. Similarly, design displacement for 6-storey frame obtained through DDBD approach is 0.211 m corresponding to 226.917 kN. Also, design displacement for 10 and 12-storey frame obtained through DDBD approach is 0.337 m and 0.4 m corresponding to 376.521 kN and 545.012 kN. The base shear forces obtained through nonlinear static pushover analysis is significantly larger compared to that obtained through DDBD procedure.

Tables 5.2 to 5.5 shows formation of hinges in 3, 6, 10, 12-storey fames as obtained through nonlinear static pushover analysis.

Table 5.2: Hinge Formation for 3-Storey Frame

Step	Displace- -ment(m)	Base Force(kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	Total
0	0	0	42	0	0	0	0	42
1	0.018	86.706	42	0	0	0	0	42
2	0.036	173.412	42	0	0	0	0	42
3	0.054	260.119	42	0	0	0	0	42
4	0.056282	271.11	41	1	0	0	0	42
5	0.070654	316.964	31	11	0	0	0	42
6	0.073238	322.316	28	14	0	0	0	42
7	0.089955	334.009	27	15	0	0	0	42
8	0.100773	345.896	25	17	0	0	0	42
9	0.108831	351.204	22	14	6	0	0	42
10	0.11098	352.112	21	14	7	0	0	42
11	0.128622	353.166	21	10	11	0	0	42
12	0.131698	354.592	20	11	11	0	0	42
13	0.149698	355.575	20	9	13	0	0	42
14	0.167698	360.926	20	9	13	0	0	42
15	0.185698	359.912	20	6	16	0	0	42
16	0.203698	366.408	20	3	19	0	0	42
17	0.221698	364.347	20	3	19	0	0	42
18	0.236942	369.782	20	2	18	2	0	42
19	0.242061	370.805	20	2	14	6	0	42
20	0.260061	369.939	20	2	14	6	0	42
21	0.269205	372.229	20	2	13	7	0	42
22	0.287205	372.023	20	2	13	7	0	42
23	0.293903	373.163	20	2	10	10	0	42
24	0.304315	373.092	20	2	9	11	0	42
25	0.30714	373.094	20	2	6	13	1	42
26	0.330997	371.833	20	2	5	13	2	42
27	0.337218	371.67	20	2	4	12	4	42
28	0.356198	370.071	20	2	2	13	5	42
29	0.36	369.711	20	2	2	11	7	42

Table 5.3: Hinge Formation for 6-Storey Frame

Step	Displace--ment(m)	Base Force(kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	Total
0	0	0	84	0	0	0	0	84
1	0.014191	66.82	80	4	0	0	0	84
2	0.088712	225.19	66	18	0	0	0	84
3	0.128712	302.242	66	9	9	0	0	84
4	0.177208	384	59	10	15	0	0	84
5	0.220313	427.097	57	9	18	0	0	84
6	0.272597	463.146	47	19	17	1	0	84
7	0.29349	467.247	44	19	17	4	0	84
8	0.362228	472.72	44	9	22	9	0	84
9	0.379685	473.445	44	9	21	10	0	84
10	0.392009	474.553	44	6	22	12	0	84
11	0.439999	476.692	44	3	21	16	0	84
12	0.458923	477.797	44	0	22	18	0	84
13	0.498064	477.69	44	0	22	18	0	84
14	0.504436	478.302	44	0	20	20	0	84
15	0.523608	478.414	44	0	17	23	0	84
16	0.55151	477.077	44	0	14	26	0	84
17	0.561795	476.989	44	0	11	29	0	84
18	0.601044	473.681	44	0	11	28	1	84
19	0.632574	472.297	44	0	8	29	3	84
20	0.638071	471.933	44	0	7	28	5	84
21	0.683757	466.414	44	0	3	31	6	84
22	0.744264	458.097	44	0	0	34	6	84
23	0.784261	452.436	44	0	0	33	7	84
24	0.824265	446.787	44	0	0	32	8	84
25	0.864261	441.119	44	0	0	32	8	84
26	0.904258	435.451	44	0	0	32	8	84
27	0.944265	429.804	44	0	0	32	8	84
28	0.984262	424.136	44	0	0	32	8	84
29	0.995397	422.558	44	0	0	32	8	84

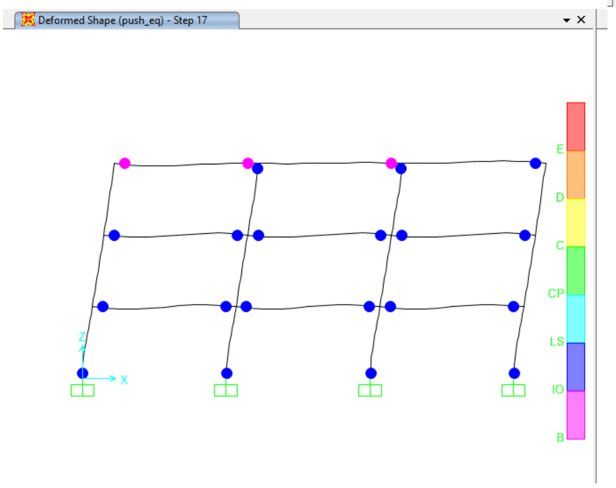
Table 5.4: Hinge Formation for 10-Storey Frame

Step	Displace- -ment(m)	Base Force(kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	Total
0	0	0	140	0	0	0	0	140
1	0.04	193.567	140	0	0	0	0	140
2	0.043253	209.311	138	2	0	0	0	140
3	0.086203	330.894	118	22	0	0	0	140
4	0.1326	428.977	113	27	0	0	0	140
5	0.174654	483.189	98	42	0	0	0	140
6	0.21995	523.148	93	38	9	0	0	140
7	0.265515	555.856	90	35	15	0	0	140
8	0.327567	591.388	85	34	21	0	0	140
9	0.356926	606.608	81	32	27	0	0	140
10	0.421006	617.891	79	22	39	0	0	140
11	0.461474	624.536	76	22	42	0	0	140
12	0.501474	628.33	76	15	48	1	0	140
13	0.549684	632.391	76	14	41	9	0	140
14	0.589684	635.055	75	7	49	9	0	140
15	0.668281	639.451	75	7	43	15	0	140
16	0.713256	641.409	75	4	38	23	0	140
17	0.720335	641.65	75	4	34	27	0	140
18	0.727416	641.746	75	4	32	29	0	140
19	0.767416	640.619	75	1	35	29	0	140
20	0.795366	639.798	75	1	28	36	0	140
21	0.871411	633.659	75	1	22	42	0	140
22	0.912243	629.426	75	1	18	44	2	140
23	0.988353	620.238	75	1	12	49	3	140
24	1	618.755	75	1	11	48	5	140

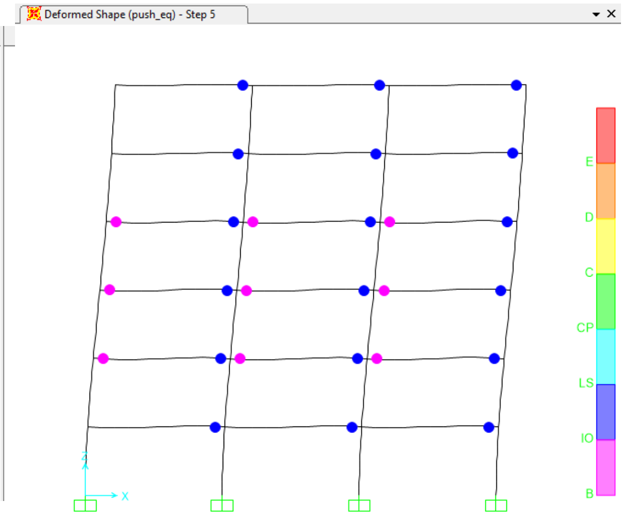
Table 5.5: Hinge Formation for 12-Storey Frame

Step	Displace- -ment(m)	Base Force(kN)	A to B	B to IO	IO to LS	LS to CP	CP to C	Total
0	0	0	168	0	0	0	0	168
1	0.048	91.156	168	0	0	0	0	168
2	0.096	182.311	168	0	0	0	0	168
3	0.144	273.467	168	0	0	0	0	168
4	0.192	364.622	167	1	0	0	0	168
5	0.209949	398.36	166	2	0	0	0	168
6	0.267151	501.051	160	8	0	0	0	168
7	0.321715	578.251	146	22	0	0	0	168
8	0.376615	622.14	126	42	0	0	0	168
9	0.441636	652.187	117	51	0	0	0	168
10	0.498498	672.275	109	46	13	0	0	168
11	0.551653	688.403	104	40	24	0	0	168
12	0.612222	702.173	98	33	37	0	0	168
13	0.668845	712.728	97	29	42	0	0	168
14	0.716845	721.655	96	25	47	0	0	168
15	0.785609	733.813	94	21	52	1	0	168
16	0.833596	742.281	93	20	53	1	1	168

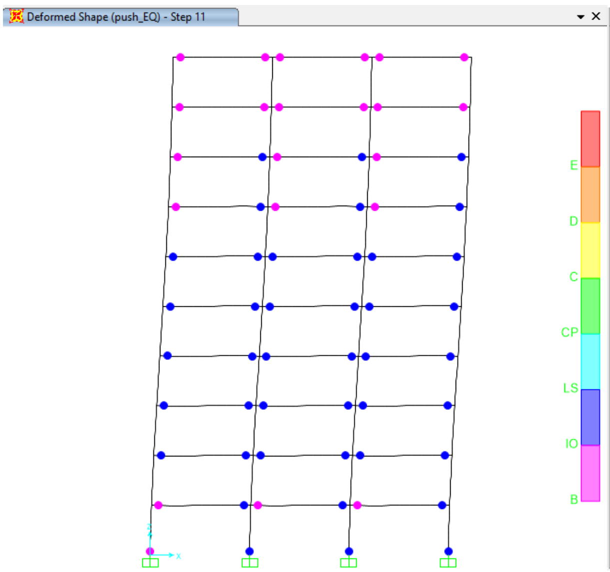
Figure 5.7 shows sway mechanism of 3, 6, 10, 12-storey frames for life safety performance level. As be seen, no hinges are formed in columns, so the structure follows weak beam-strong column failure mechanism.



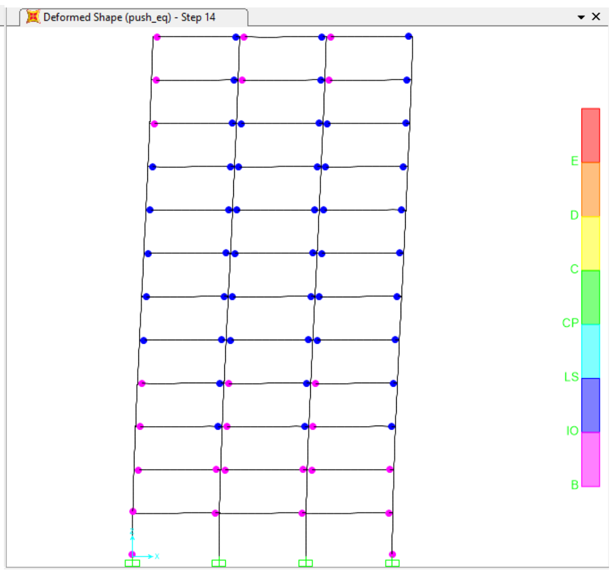
(a) 3 Storey Frame



(b) 6 Storey Frame



(c) 10 Storey Frame



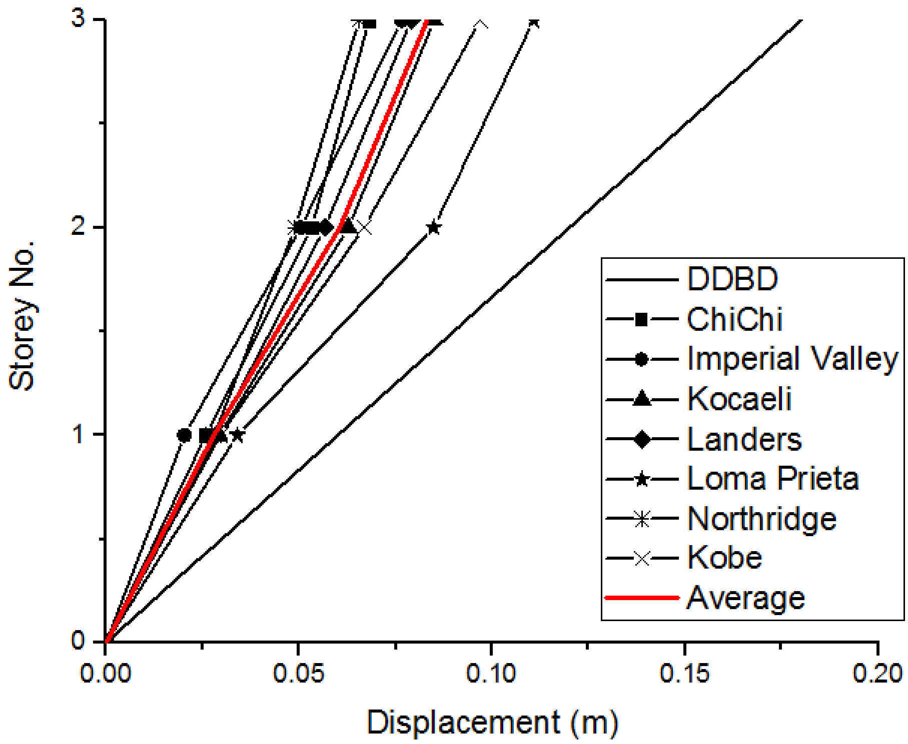
(d) 12 Storey Frame

Figure 5.7: Sway Mechanism for Life Safety Performance Level Obtained Through Non-linear Static Pushover Analysis

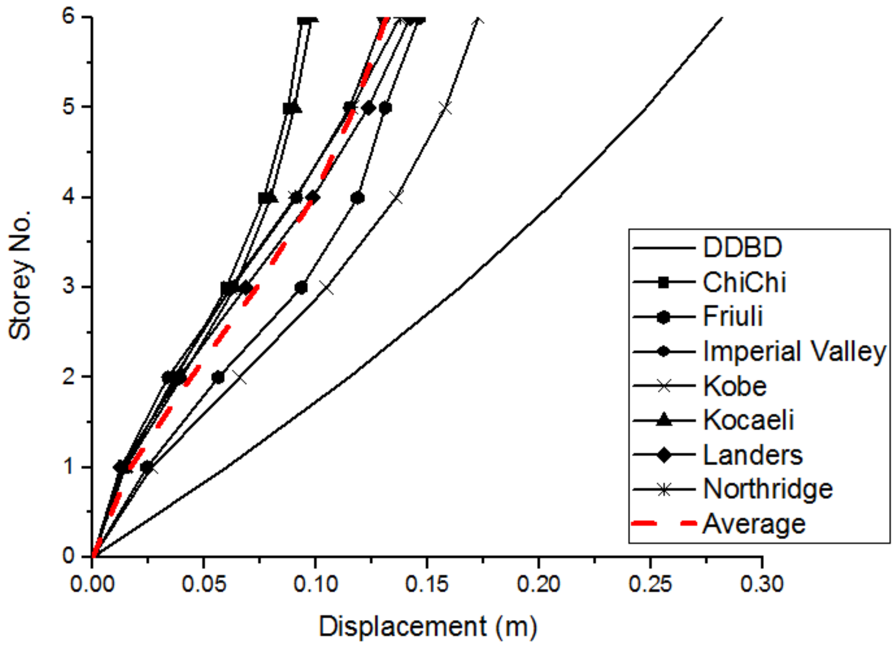
5.4.2 Results of Nonlinear Time History Analysis

The results of nonlinear time history analysis are also presented in terms of displacement profile and storey drift ratio similar to the results shown in nonlinear pushover analysis.

Figure 5.8 and 5.9 shows displacement of 3, 6, 10, 12-storey frames obtained through time history analysis. It can be seen that displacement profile of seven ground motion time histories are well within the target displacement all along the height of building.

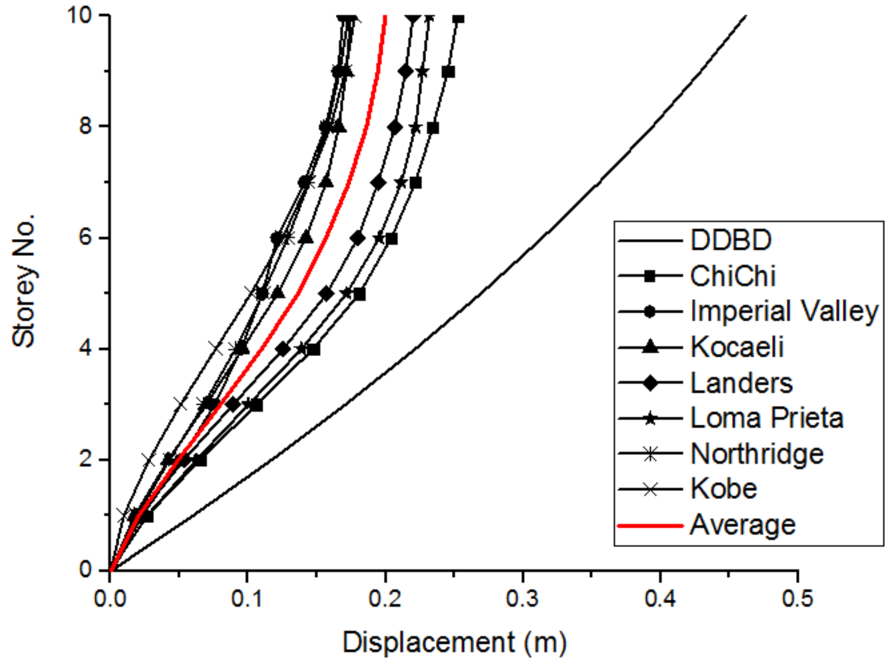


(a) 3 Storey Frame

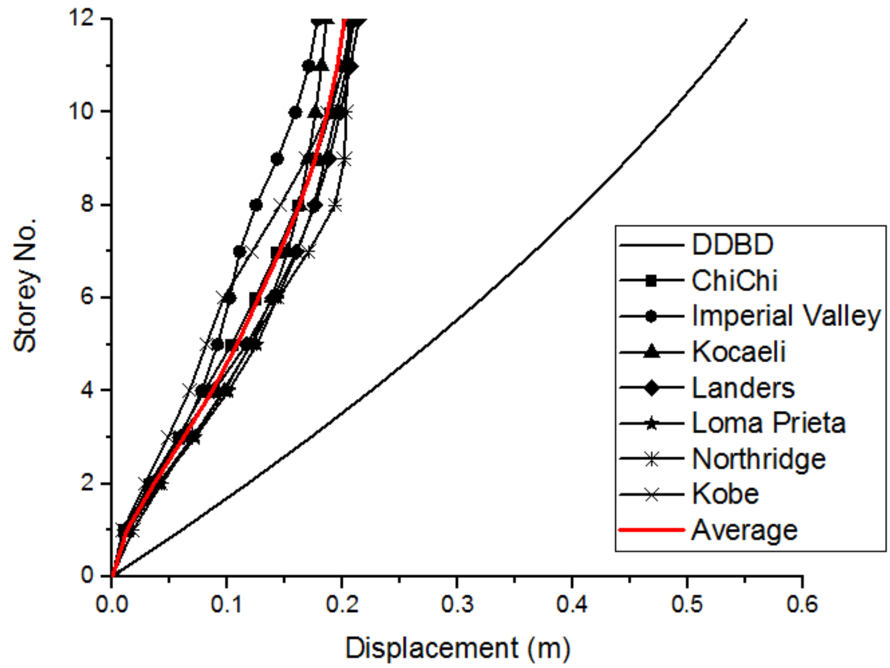


(b) 6 Storey Frame

Figure 5.8: Displacement of 3, 6- Storey Frames Obtained Through Time History Analysis



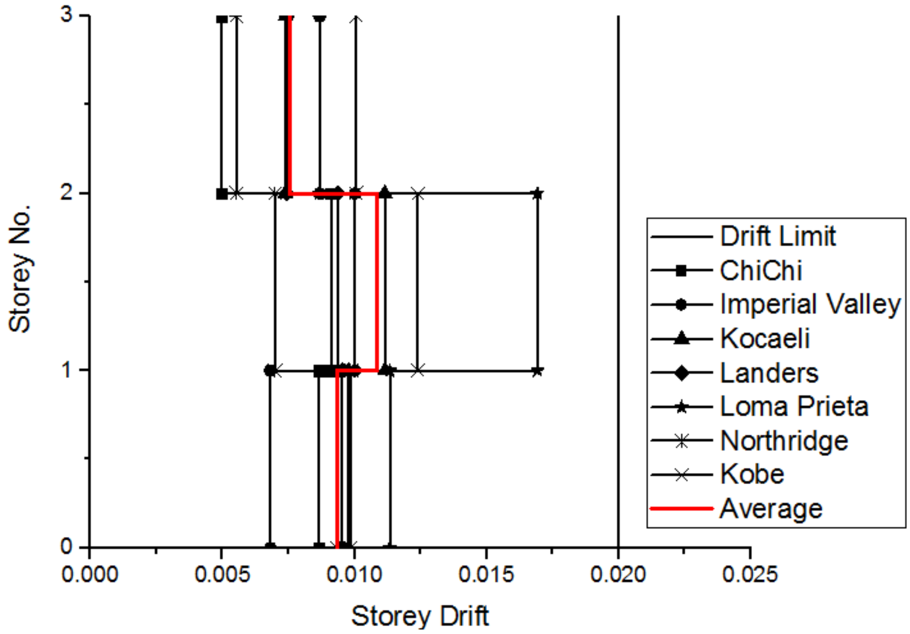
(a) 10 Storey Frame



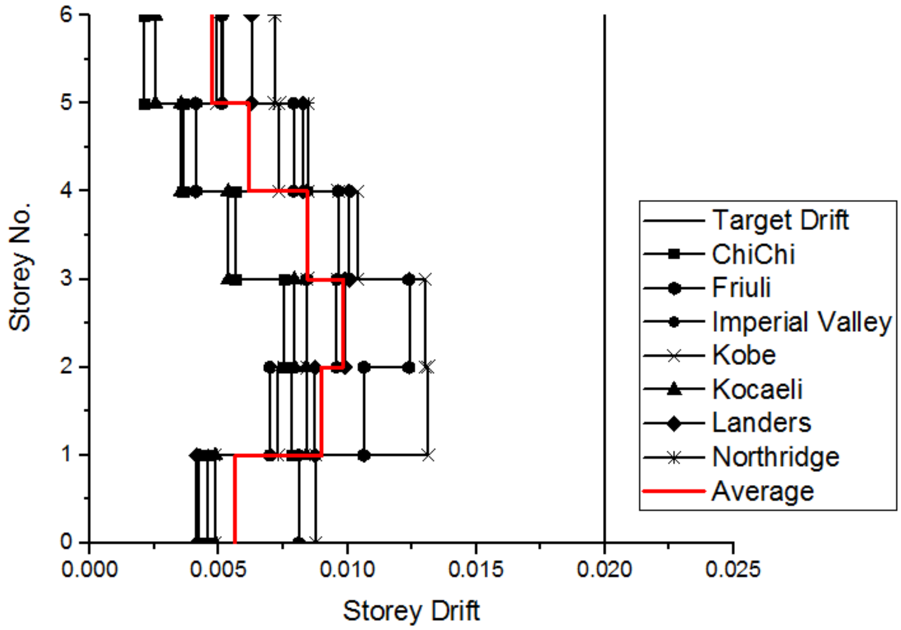
(b) 12 Storey Frame

Figure 5.9: Displacement of 10, 12- Storey Frames Obtained Through Time History Analysis

Figure 5.10 and 5.11 shows interstorey drift ratio of 3,6, 10, 12-storey frames obtained through time history analysis. It can be seen that interstorey drift ratio of seven ground motion time histories are well within the target drift limit of life safety performance limit throughout the height of building.

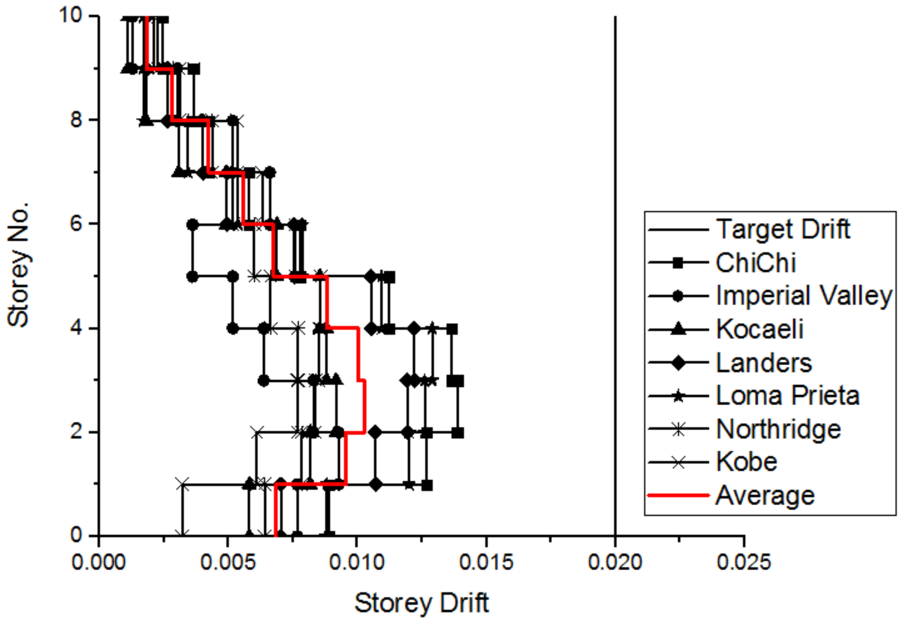


(a) 3 storey building

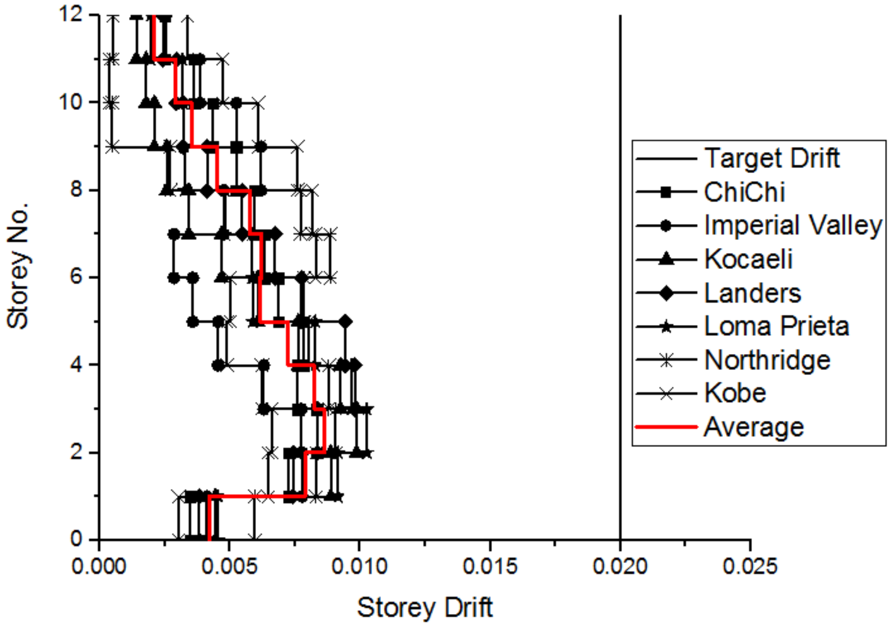


(b) 6 storey building

Figure 5.10: Interstorey Drift of 3, 6-Storey Frames Obtained Through Time History Analysis



(a) 10 storey building



(b) 12 storey building

Figure 5.11: Interstorey Drift of 10, 12-Storey Frames Obtained Through Time History Analysis

5.5 Summary

In this chapter, performance assessment of RC moment resisting frames analysed using DBD approach has been discussed. For the same purpose, nonlinear static analysis and time history analysis has been implemented. SAP2000 v19 has been used to perform nonlinear static and time history analysis. Target performance limit of 2% is considered corresponding to life safety performance level. Interior frames of 3, 6, 10, 12-storey frames analysed using DBD method is taken for performance assessment. It is seen that the performance limit of life safety is achieved satisfactorily in nonlinear static analysis. For time history analysis, seven ground motion time histories are selected. Each acceleration time history is matched with target response spectrum so that the structure can be hit with right spectral quantity at critical time period. In case of time history analysis, the responses obtained for seven time histories are well within the life safety performance level.

Chapter 6

Displacement Based Design using Site Specific Response Spectra

6.1 General

Site specific response analysis is required to determine the response of a soil deposit to the motion of the bedrock directly below the soil and also analysing the effect of local soil conditions on amplification of seismic waves. Therefore, the analysis is required for estimating the ground response spectra and time history plots obtained at the ground surface due to earthquakes. The response spectrum is processed based on the time history observation at regular intervals on the ground surface during the earthquake.

The effect of local soil conditions on ground response during earthquake is evaluated using computer software ProSHAKE which is based on one dimensional equivalent linear analysis. Soil profile modelling of soil in terms of soil properties layer is required for analysis.

ProSHAKE is a computer software used for one-dimensional, equivalent linear seismic ground response analysis of horizontal layered soil deposits. The software allows for more accurate analysis of ground-based responses. The results of the analysis allow the determination of surface response spectra in terms of acceleration and displacement spectra.

In this chapter, site specific displacement spectra obtained through analysis of soil profiles at various sites in Ahmedabad region and Delhi site using one dimensional ground response analysis software ProSHAKE is utilized for displacement based design in 3, 6-storey frames. Effect of soil properties such as depth of soil, unit weight of soil and shear wave velocity on displacement spectra on the event of an earthquake is studied. In both sites, Bhuj and El Centro earthquake time histories are taken as input motion file.

6.2 Program Structure of ProSHAKE Software

To perform ground-based analysis through ProSHAKE, different soil parameters needs to be provided. ProShake is designed with an interactive interface that facilitates and accelerates the process to implement and interpret the results of ground response analyses. The program structure of ProSHAKE is significantly divided in three sections namely Input Manager, Solution Manager and Output Manager. The report is developed with a word processor which is built in ProShake. The user can retain the records of each analysis in report.All input data are automatically generated in the Report and updated whenever the report is accessed.

6.2.1 Input Manager

The Input Manager of the software allows the entry of soil profiles and input motion data. The input manager requires the input of these geotechnical dynamic parameters for the soil profile.The input manager lets the user enter data, check and save the data before execution of program. It allows user to view the input data graphically and allows checking data for entry errors. All input data and graphs generated in the Input Manager can be copied to the report.
In order to carry out the ground response analysis by means of ProSHAKE software, the parameters that must be provided are as follows:

- **Soil Profile** For the development of site specific response spectra at ground surface , the soil profile data of the sites is required as input data. In input Manager, material name , thickness, unit weight of soil, maximum shear modulus, shear wave velocity and soil model. The user can enter either enter maximum shear modulus or shear wave velocity with unit weight of soil, the other will be automatically calculated.

The soil input data entered in the program is as shown in Figure 6.1. The input motion time history provided in the program can be seen in Figure 6.2.

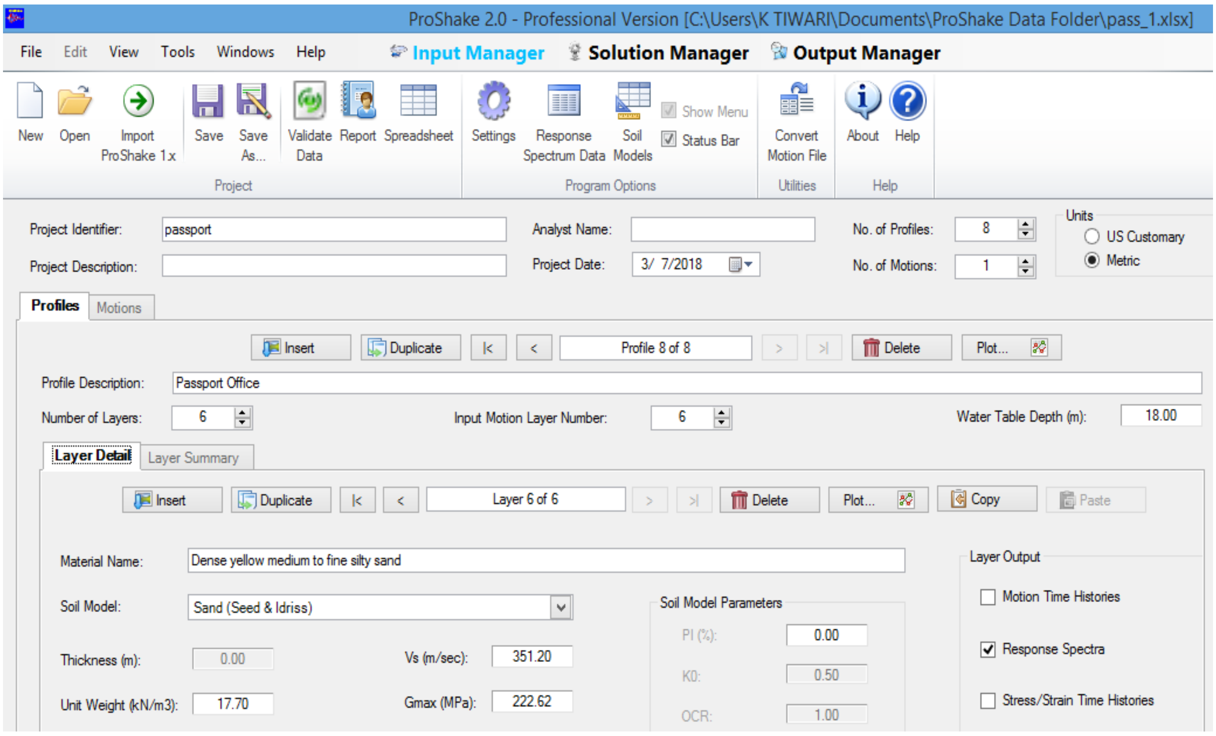


Figure 6.1: Soil Data

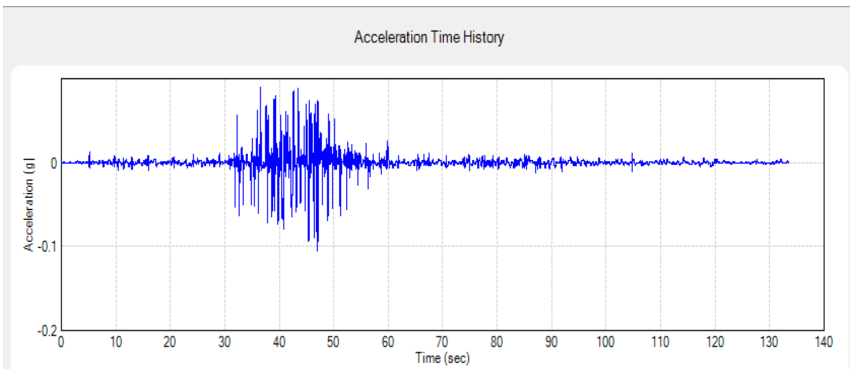


Figure 6.2: Input Motion Time History

- **Input Time History** Selection of input bedrock motions are important step for site specific response analysis. Appropriate rock motions i.e. natural acceleration time history or synthetic acceleration time history are selected to represent the design bed rock motion for the site. Therefore, the strong motion recorded on the ground floor at the Passport Office site in Ahmedabad during Bhuj earthquake on January

26, 2001 is used as input motion and are shown in Figure 6.2. Figure 6.3 shows input time history being assigned in the program.

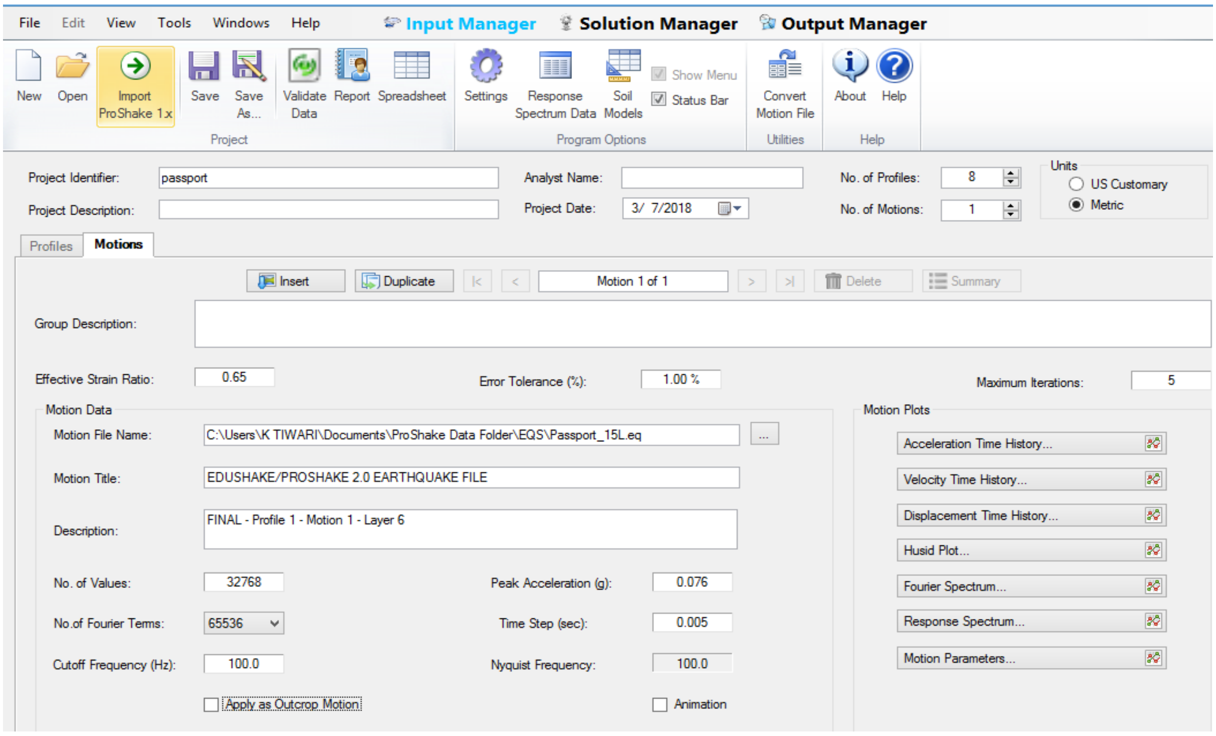


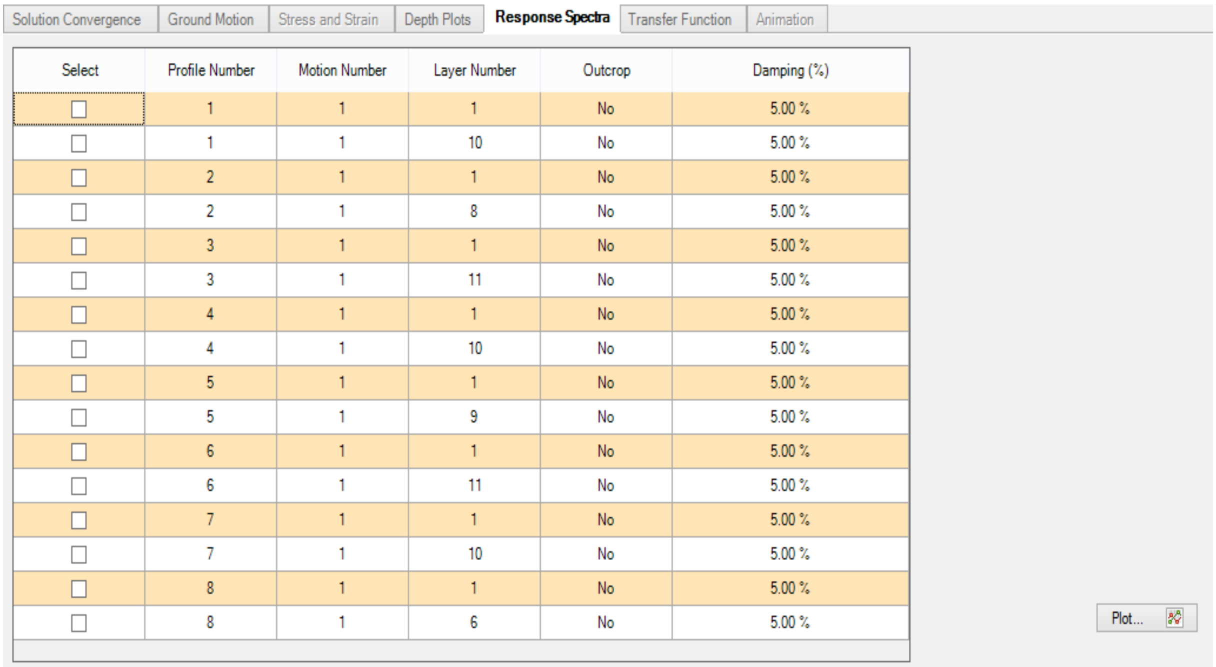
Figure 6.3: Input Motion File

6.2.2 Solution Manger

The solution manager is used to execute the ground response analysis in ProSHAKE software. The Solution Manager analyzes any input data file that is specified in the Input Manager.

6.2.3 Output Manger

The Output Manager has a graphical interface that allows the user to generate a wide range of plots of the result of the analysis. Several types of plots including ground motion plot, stress strain plot, response spectrum plot, and depth plot are available as can be seen in Figure 6.4. ProSHAKE allows saving numerical and graphical data in the form of a report. Report contains an analysis summary of soil profile, Input motion, and output selections by default. Report is generated in a format that can be read by other, more powerful word processors.



Select	Profile Number	Motion Number	Layer Number	Outcrop	Damping (%)
☐	1	1	1	No	5.00 %
☐	1	1	10	No	5.00 %
☐	2	1	1	No	5.00 %
☐	2	1	8	No	5.00 %
☐	3	1	1	No	5.00 %
☐	3	1	11	No	5.00 %
☐	4	1	1	No	5.00 %
☐	4	1	10	No	5.00 %
☐	5	1	1	No	5.00 %
☐	5	1	9	No	5.00 %
☐	6	1	1	No	5.00 %
☐	6	1	11	No	5.00 %
☐	7	1	1	No	5.00 %
☐	7	1	10	No	5.00 %
☐	8	1	1	No	5.00 %
☐	8	1	6	No	5.00 %

Figure 6.4: Output File Options

6.3 Site Specific Displacement Spectra of Ahmedabad and Delhi Site

In this study, sites in Ahmedabad and Delhi are taken into consideration. In Ahmedabad four sites are considered namely, Passport Office, Chandkheda, Paldi and Thaltej. The depth of soil profiles in Ahmedabad region is same for all sites i.e. 15 m. In order to understand the effect of depth of soil profile on seismic response of structures, Delhi site is taken into consideration. The same soil profile at Delhi is considered with variation in bedrock depth of 10m, 30m, 60m, 100m, 200m. Soil profile in this case is taken from literature [24]. The geotechnical data required for such analysis are thickness of material, shear wave velocity and soil model curve.

The ground motion recorded at ground floor of passport office building during Bhuj earthquake of 0.106g PGA on 26^{th} January 2001 is used to develop the ground motion at the rock or hard soil level. To consider high PGA earthquake motion, El Centro earthquake of 0.344g PGA is also considered. The site specific response spectra at ground surface is developed using one dimensional ground response analysis software ProSHAKE. For the development of artificial time history, longitudinal direction is considered because it is critical compared to other directions.

Priestley et al.[1] presented that the displacement spectra is linear upto corner period T_c. Above the corner period displacement essentially remains constant. The ground motion time histories that are used in present study are Bhuj Earthquake (0.106g) and El Centro Earthquake (0.344g).

Moment Magnitude of Bhuj Earthquake, Mw = 7.7

Moment Magnitude of El Centro Earthquake, Mw = 6.9

Faccioli et al.[?] presented following relationship to obtain corner period for earthquakes with moment magnitude greater than Mw =5.7

$$T_c = 1.1 + 2.5(M_w - 5.7) seconds$$

So, for Bhuj Earthquake, corner time period,Tc = 6 sec

So, for El Centro Earthquake, corner time period, Tc = 4 sec

However, in Indian seismic codes no mention of corner time period is mentioned after which displacement becomes constant.

Figure 6.5 and 6.6 shows displacement response spectra on ground surface at various sites in Ahmedabad. It can be seen from the figures below that for same soil depth of 15m, there is no variation between ground surface and bedrock motion.

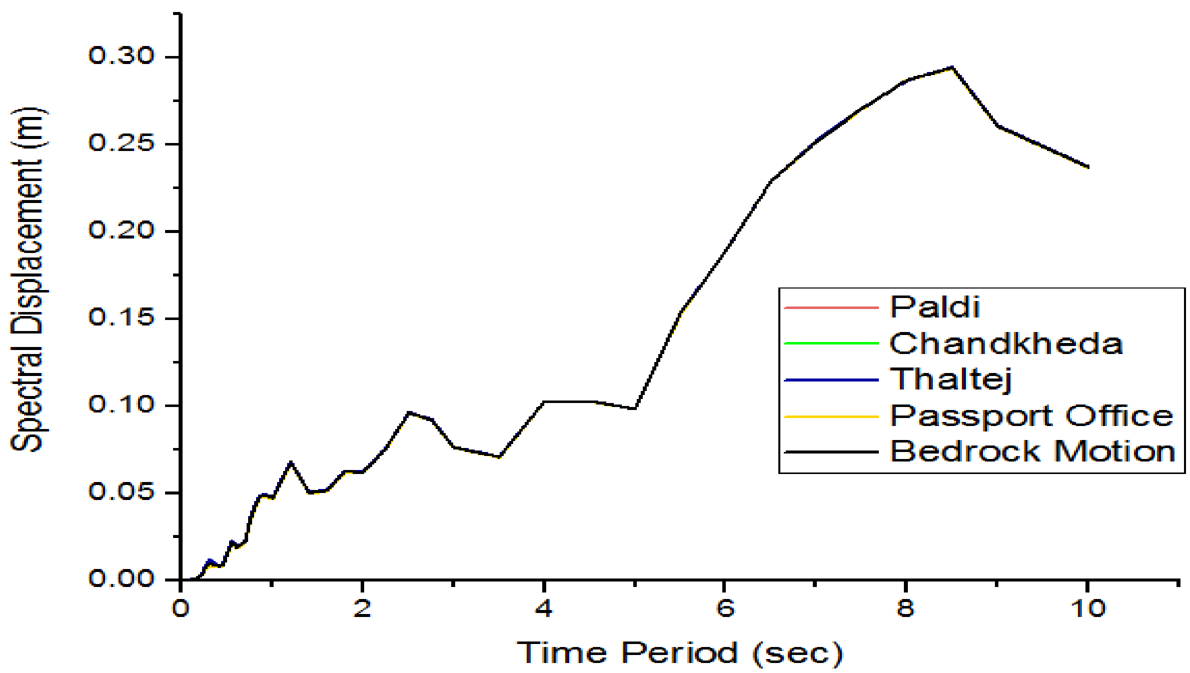


Figure 6.5: Elastic Displacement Spectra at Ahmedabad Site for Bhuj Earthquake

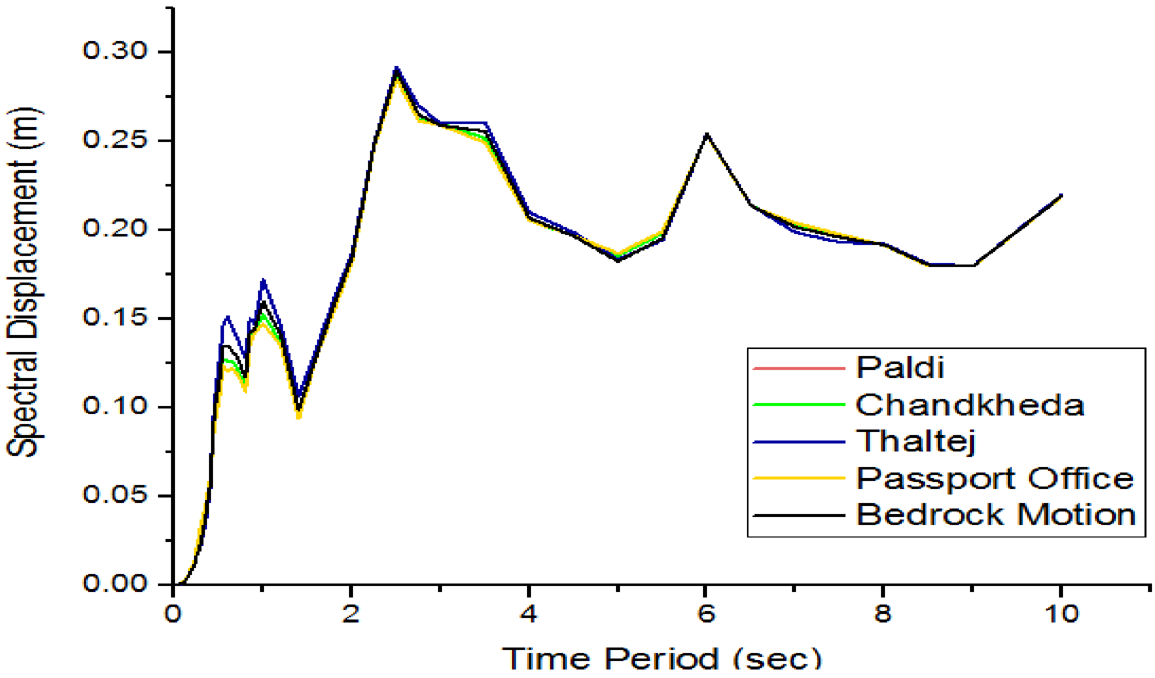


Figure 6.6: Elastic Displacement Spectra at Ahmedabad Site for El Centro Earthquake

Figure 6.7 and 6.8 shows displacement response spectra on ground surface at Delhi for varying bedrock depth of 10m, 30m, 60m, 100m and 200m.

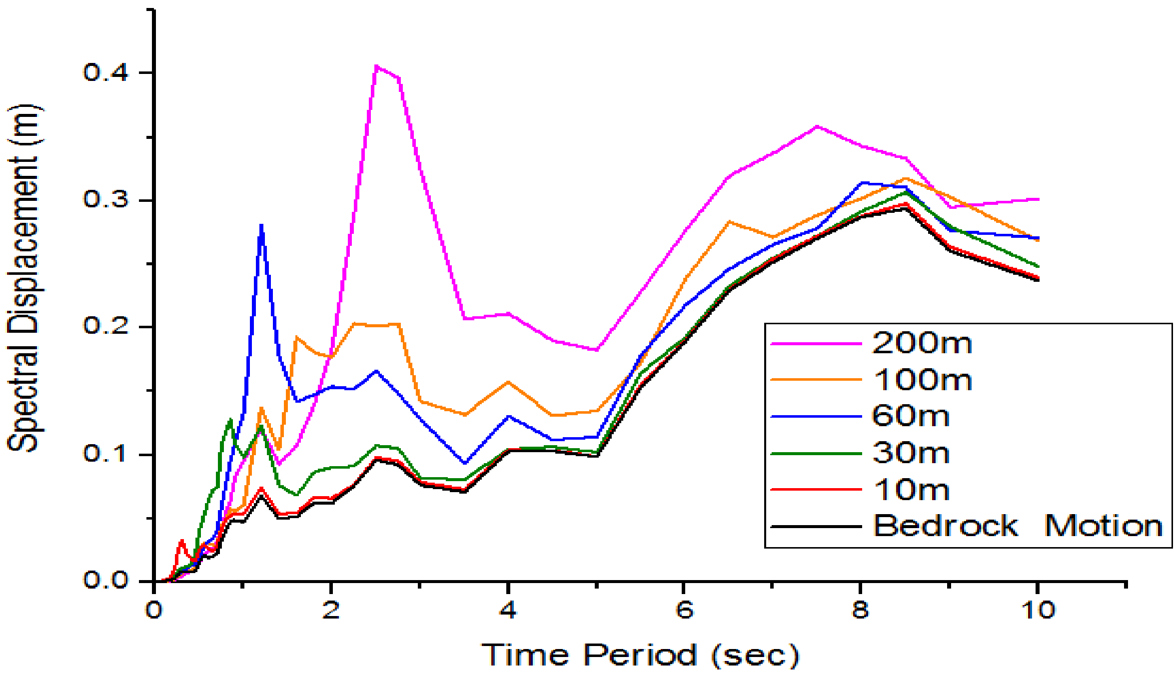


Figure 6.7: Elastic Displacement Spectra at Delhi Site for Bhuj Earthquake

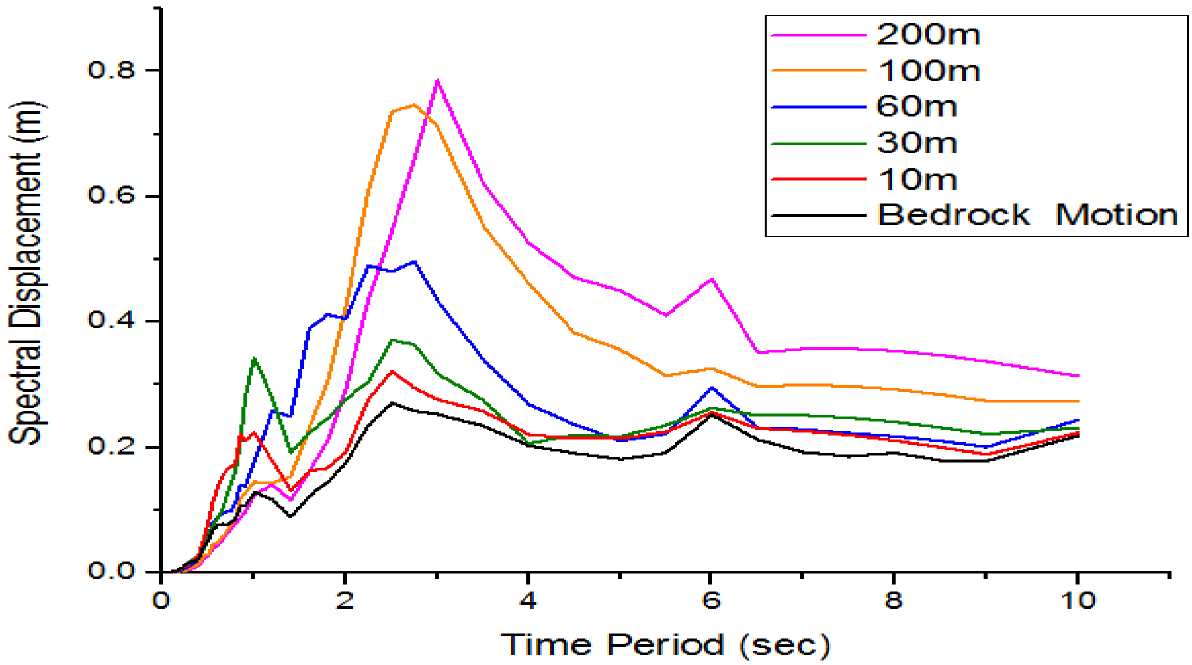


Figure 6.8: Elastic Displacement Spectra at Delhi Site for El Centro Earthquake

It can be seen from displacement spectra at Delhi site that, for a given soil stratum, the elastic displacement spectrum, for short period range, decreases with increase in soil depth. Whereas, for long period range, the elastic displacement spectrum increases with increase in soil depth. This observation is in compliance with the study shown by Adhikary et al.[24].

6.4 Displacement Based Design Using Site Specific Response Spectra

In this section, site specific displacement spectra obtained through ground response analysis of Ahmedabad region and Delhi site using ProSHAKE is implemented for displacement based design of 3,6-storey RC moment resisting frames.

The parameters defining the building are same as presented in Chapter 5 except seismic parameters and dimensions of beams and columns. So, dimensions of beams and columns considered in various buildings are:

For 3-storey building:

Beam dimension: 250 mm × 400 mm

Column dimension: 400 mm × 400 mm

For 6-storey building:

Beam dimension: 300 mm × 600 mm

Column dimension: 500 mm × 500 mm

Plan of all the buildings are same as described in Chapter 5. In this section, DBD analysis of 3, 6-storey buildings using site specific displacement spectra in Ahmedabad and Delhi site has been done. Since, design displacement of the system is required to estimate time period of structure.

Therefore, design displacement of 3-storey building = 0.139 m

6-storey building = 0.211 m

Results of the analysis of 3, 6-storey RC frame building analysed using site specific displacement spectra at Ahmedabad and Delhi site are shown in Tables 6.1 to 6.8.

Table 6.1: Results of analysis of 3-storey building analysed using site specific spectra of Bhuj Earthquake (0.106g) at Ahmedabad site

Response Quantities	Paldi	Chandkheda	Thaltej	Passport Office
Time Period, T (sec)	5.79	5.79	5.79	5.79
Stiffness, Ke (kN/m)	608.349	608.349	608.349	608.349
Base Shear Force (kN)	84.573	84.573	84.573	84.573

As can be seen from Table 6.1,the results of all the sites in Ahmedabad are same because the ground surface displacement spectra as shown in Figure 6.5 has no variation at same soil depth of 15m.

Also the time period obtained is also large because at equivalent damping of 10.159% the displacement spectra decreases. Bhuj earthquake has low PGA of 0.106g which makes the displacement spectra very low.Due to this stiffness of structure decreases as structures becomes flexible resulting in low base shear force indicating inelastic response of structure.

Table 6.2: Results of analysis of 3-storey building analysed using site specific spectra of El Centro Earthquake (0.344g) at Ahmedabad site

Response Quantities	Paldi	Chandkheda	Thaltej	Passport Office
Time Period, T (sec)	1.92	1.92	1.92	1.92
Stiffness, Ke (kN/m)	5532.326	5532.326	5532.326	5532.326
Base Shear Force (kN)	769.103	769.103	769.103	769.103

From Table 6.2 it is seen that results are similar for all Ahmedabad sites because the ground surface displacement spectra as shown in Figure 6.6 has almost no variation at similar bedrock depth of 15m.

El Centro earthquake has high PGA of 0.344g so the displacement spectra is high or amplified more than that of Bhuj earthquake.So, the time period obtained is also small. Due to this stiffness of structure increases as structures becomes quite stiff attracting more force resulting in high base shear force.

Table 6.3: Results of analysis of 6-storey building analysed using site specific spectra of Bhuj Earthquake (0.106g) at Ahmedabad site

Response Quantities	Paldi	Chandkheda	Thaltej	Passport Office
Time Period, T (sec)	6	6	6	6
Stiffness, Ke (kN/m)	1548.33	1548.33	1548.33	1548.33
Base Shear Force (kN)	253.926	253.926	253.926	253.926

Here also results of all the sites in Ahmedabad are similar as can be seen from Table 6.1 because of the reason mentioned previously.

As seen in Table 6.3, since the earthquake considered has low PGA and the design displacement is greater than corner displacement at equivalent damping of 10.374%. So, the time period of the structure is considered to be corner period and design displacement is reduced by iterative procedure. The corner period for Bhuj Earthquake is 6 sec. This results in low stiffness and low base shear force.

Table 6.4: Results of analysis of 6-storey building analysed using site specific spectra of El Centro Earthquake (0.344g) at Ahmedabad site

Response Quantities	Paldi	Chandkheda	Thaltej	Passport Office
Time Period, T (sec)	2.17	2.18	2.17	2.18
Stiffness, Ke (kN/m)	9575.52	9487.87	9575.52	9487.87
Base Shear Force (kN)	2015.585	1997.136	2015.585	1997.136

As seen in Table 6.4 El Centro earthquake having large PGAs is considered. So, time period is less which makes structure stiff, which means high stiffness resulting in high base shear force.Since, there is no difference in bedrock spectra and ground surface spectra, that is why, all sites have same time period, thus similar base shear force for a given PGA.

Table 6.5: Results of analysis of 3-storey building analysed using site specific spectra of Bhuj Earthquake (0.106g) at Delhi site

Response Quantities	200 m	100 m	60 m	30 m	10 m
Time Period, T (sec)	1.95	1.56	1.056	5.67	5.78
Stiffness, Ke (kN/m)	12656.85	19776.33	43158.57	1497.024	1440.586
Base Shear Force (kN)	888.398	1388.121	3029.345	105.078	101.116

From displacement spectra, it can be seen that for small PGA and small depth there is little variation between bedrock spectra and ground surface spectra.Displacement spectra at 10m and 30m depth is very flat, so, time period is long leading to low stiffness and low base shear force. Also, displacement spectra decreases with increase in depth, for small time periods, time period increases with increase in depth resulting in decrease of base shear force.

Table 6.6: Results of analysis of 3-storey building analysed using site specific spectra of El Centro Earthquake (0.344g) at Delhi site

Response Quantities	200 m	100 m	60 m	30 m	10 m
Time Period, T (sec)	1.65	1.45	0.99	0.81	0.8
Stiffness, Ke (kN/m)	17677.75	22890.69	49104.86	733541.2	75199.5
Base Shear Force (kN)	1041.4	1348.5	2892.79	4321.33	4430.036

For small height buildings with high PGA, displacement spectra increases with increase in soil depth for short period range. So, time period increases with increase in soil depth leading to decrease in base shear force with increase in soil depth.

Table 6.7: Results of analysis of 6-storey building analysed using site specific spectra of Bhuj Earthquake (0.106g) at Delhi site

Response Quantities	200 m	100 m	60 m	30 m	10 m
Time Period, T (sec)	2.25	6	0.98	6	6
Stiffness, Ke (kN/m)	9506.7	1548.33	50112.12	1548.33	1548.33
Base Shear Force (kN)	667.285	297.28	3517.421	261.67	261.67

For small PGAs and increasing height of building, design displacement of building exceeds the corner displacement at equivalent damping. An inelastic response will occur but not at the level of ductility corresponding to the displacement or drift capacity of the structure. An iterative procedure is adopted to obtain final displacement. The inelastic response develops at a lower ductility than the structural capacity.

Table 6.8: Results of analysis of 6-storey building analysed using site specific spectra of El Centro Earthquake (0.344g) at Delhi site

Response Quantities	200 m	100 m	60 m	30 m	10 m
Time Period, T (sec)	1.941	1.701	1.43	0.891	2.25
Stiffness, K_e (kN/m)	12774.5	16633.6	23535.47	60623.29	3483.75
Base Shear Force (kN)	896.655	1167.53	1651.979	4255.211	731.59

For low soil depth and increasing height of building, displacement spectra is low leading to high time period and low base shear force. As discussed earlier with high PGA, displacement spectra decreases with increase in soil depth for short period range. So, time period increases with increase in soil depth leading to decrease in base shear force with increase in soil depth. For tall structures with low PGA, seismic response is generally elastic, because the yield displacement exceeds design displacement at corner period of 5% damping.

6.5 Limitations of Indian Seismic Code

The limitations that are encountered in displacement based design of RC moment resisting frame building when using IS 1893 (Part 1):2016 displacement spectra are as follows:-

1. **Classification of Sites for Seismic Design**

   Seismic codes around the world classify soils based on parameters such as unconfined shear strength, average shear wave velocity, (V_s) which is usually taken for top 30m soil deposit, SPT Value, N, etc. But IS 1893 (Part 1):2016 classifies soil on into Type I (Rock or Hard soil), Type II (Medium soil) and Type III (Soft soil) the basis of SPT value only. However, soil amplification of seismic waves is more importantly dependent on shear wave velocity for which no mention is made in the current seismic codes.

2. **Design Acceleration Spectra**

   In IS 1893 (Part 1), shape of acceleration spectra is scaled by zone factor. Also, soil amplification is considered in longer periods based on soil type only. Also, in short period range, amplification of soil is completely ignored by Indian code. Effect of amplitude of motion is also ignored. While codes of other countries such as ASCE 7, Turkish seismic codes etc, scale the design spectrum based on two spectral ordinates which are dependent on site class. They also cover effect of soil type and amplitude of ground motion on soil amplification, de-amplification and shape of spectra on which Indian code is completely silent.

3. **Design Displacement Spectra**

   With displacement based design approach, even buildings of moderate height seem to lie in displacement controlled region. Indian code is completely silent on corner period lying between velocity controlled and displacement controlled region which is an important parameter when designing with displacement based approach. The corner period depends on magnitude of earthquake, distance and source mechanism. Also, it has been seen that IS 1893 (Part 1) gives low values of spectral displacement for all site classes as compared to other codes.

4. **Storey Drift Limitation**

   Interstorey drift is limited to 0.4% at design loads in case of Indian codes. However, other codes limit the total storey drift to 1.5-2.5%. Also, other codes amplify the elastic displacement at floor levels by deflection amplification factor to take into account P-$\triangle$ effects. Deflection amplification factor depends on structure type.

So, this shows that Indian seismic code needs revision and upgradation to get reliable estimates of displacement.

6.6 Summary

In this chapter, study of PROSHAKE software is done. The input parameters required for the ground response analysis has been discussed. The development of site specific response spectra for various sites in Ahmedabad region and Delhi site using Bhuj Earthquake and El Centro time history is presented. The two earthquakes motion taken account for low and high PGA motion earthquakes. The sites considered in Ahmedabad are Paldi, Chandkheda, Thaltej and Passport Office where the soil depth is considered at 15m in all cases. To take account of effect of soil depth on seismic response of structure same site in Delhi is considered but with different soil depth of 10 m, 30 m, 60 m, 100 m and 200 m. Also, the use of site specific response spectra to analyse 3,6-storey RC frame building is discussed. The results show that all sites in Ahmedabad give similar base shear force due because of same soil depth at all sites. Whereas sites in Delhi show variation in base shear force corresponding to soil depth. The limitations of Indian seismic code that are encountered while displacement based design of RC moment resisting frame buildings has also been discussed.

Chapter 7

Summary, Conclusions and Future Scope of the Work

7.1 Summary

In the present study, Displacement Based Design of RC moment resisting frames has been studied. The main objective of the present study is to understand the implementation of Displacement Based Design approach on RC frame structure for analysis of the structure. Substitute Structure method used to convert MDOF system to SDOF system for implementation of DBD is studied. Substitute-Structure Approach has been used to convert MDOF system to an equivalent SDOF system, and consider its performance at peak displacement response, instead of its initial elastic characteristics. Various literatures are studied for to understand DBD method and its implementation in RC frame structures. Displacement Based Design has been used in compliance with Indian seismic code IS 1893 (Part1):2016. Also performance of the same has been evaluated. Story drift ratios have been selected as deformation limits to define the performance levels for specific intensity of ground motions. In present study, for all buildings 2% drift limit corresponding to life safety performance level has been considered.

Implementation of displacement based and force based approach on 3, 6, 10, 12-storey RC moment resisting frame building has been presented.For DBD, 2% target drift limit corresponding to life safety limit has been considered. Also comparison of 3, 6, 10, 12-storey building analysed using DBD and FBD has been presented.

The comparison results of DBD and FBD approach show that time period obtained for

DBD is more compared to that obtained through FBD method. In turn, the stiffness of building is more in FBD method because it takes the elastic stiffness of structure whereas in DBD method stiffness is less because it takes account of secant stiffness at maximum displacement which is less than elastic stiffness due to cracks formed in structure. As a result, base shear force in FBD is more than that obtained in DBD method. This shows that ductility obtained in FBD is less than that in DBD method.

The performance assessment of RC moment resisting frames analysed using DBD approach has also been discussed. For the same purpose, nonlinear static analysis and time history analysis has been implemented. SAP2000 v19 has been used to perform nonlinear static and time history analysis. Target performance limit of 2% is considered corresponding to life safety performance level. Interior frames of 3, 6, 10, 12-storey frames analysed using DBD method is taken for performance assessment. It is seen that the performance limit of life safety is achieved satisfactorily in nonlinear static analysis. For time history analysis, seven ground motion time histories are selected. Each acceleration time history is matched with target response spectrum so that the structure can be hit with right spectral quantity at critical time period. In case of time history analysis, the responses obtained for seven time histories are well within the life safety performance level.

Effect of site conditions on building designed using DBD has been considered. For this site specific response spectra is generated using PROSHAKE software which takes into account soil profile parameters like layer thickness, shear wave velocity and soil model. The input parameters required for the ground response analysis has been discussed. The development of site specific response spectra for various sites in Ahmedabad region and Delhi site using Bhuj Earthquake and El Centro time history is presented. The two earthquakes motion taken account for low and high PGA motion earthquakes. The sites considered in Ahmedabad are Paldi, Chandkheda, Thaltej and Passport Office where the soil depth is considered at 15m in all cases. To take account of effect of soil depth on seismic response of structure same site in Delhi is considered but with different soil depth of 10 m, 30 m, 60 m, 100 m and 200 m. Also, the use of site specific response spectra to analyse 3,6-storey RC frame building is discussed. The results show that all sites in Ahmedabad give similar base shear force due because of same soil depth at all sites. Whereas sites in Delhi show variation in base shear force corresponding to soil depth. The limitations of Indian seismic code that are encountered while displacement based design of RC moment resisting frame buildings has also been discussed.

7.2 Conclusions

Following conclusions have been made based on present study:

- The time period of RC moment resisting buildings, obtained through DBD approach using IS 1893 spectra is in displacement controlled region even for low rise buildings.
- Thus, making the buildings quite flexible resulting in low base shear force as compared to that obtained for buildings designed with FBD approach.
- Displacement based design of high rise frame buildings using IS 1893 (Part1):2016 displacement spectra gives unrealistic large periods.
- The results obtained through DBD approach and nonlinear pushover analysis indicate that the frames are well able to achieve the desired performance level of life safety limit.
- Non-linear pushover analysis of frames designed using DBD approach shows that frames perform well in terms of plastic failure mechanism as they follow “strong column - weak beam” failure mechanism since no plastic hinges are formed in beams.
- Nonlinear time history analysis shows that response of frames designed for lateral forces obtained through DBD approach are well within desired drift limit of life safety.
- Site specific response spectra for various sites of Ahmedabad region show little variation between ground surface and bedrock spectra.
- Site specific response spectra of Delhi site shows amplification in displacement spectra in long period range as bedrock depth increases.
- Whereas elastic displacement spectra decreases with increase in soil depth at small period range.
- Tall buildings with low PGA show elastic seismic response of structure as the yield displacement exceeds elastic design displacement at 5% damping in corner period.

7.3 Future Scope of the Work

The future scope of the work presented in this study is:

- Displacement based design of Bridge structures and Steel structures.
- Displacement based design of Masonry buildings and Timber Structures.
- Performance assessment of Wall Frame and Shear Wall Buildings using non-linear static pushover analysis and non-linear time history analysis.

Bibliography

[1] Calvi G. M. Priestley M. J. N. and Kowalsky M. J. *Displacement-based seismic design of structures.*, IUSS Press,Italy, First edition, 2007.

[2] Paulay T. and Priestley M. J. N. *Seismic Design of Reinforced Concrete Structures.*John Wiley and Sons, New York, 1992.

[3] Shibata A. and Sozen M. A. *Substitute-structure method for seismic design in reinforced concrete*, ASCE Journal of the Structural Engineering, 02(1):1-18,1976.

[4] Moehle J. P. *Displacement based design of rc structures*, In Earthquake Engineering, Tenth World Conference, Balkema,Rotterdam, 1992.

[5] Kennedy D. J. L., Medhekar M. S. *Displacement-based sesimic design of buildings-theory*, Engineering Structures,22:201-209, 2000.

[6] Pettinga J. D. and Priestley M. J. N. *Dynamic behaviour of reinforced concrete frames designed with direct displacement-based design*, Technical Report ROSE-2005/02, European School for Advanced Studies in Reduction of Seismic Risk, May 2005.

[7] Bureau of Indian Standards, New Delhi. *IS 1893 (Part 1) : 2002, Indian Standard, Criteria for Earthquake Resisitant Design of Structures Part 1 General Provisions and Buildings*, fifth revision edition.

[8] Priestley M. J. N. Mander J. B. and Park R. *Theoretical stress-strain model for confined concrete*, ASCE Journal of Structural Engineering, 114(8), August 1988.

[9] Bureau of Indian Standards, New Delhi. *IS 456 : 2000, Indian Standard, Plain and Reinforced Concrete Code for Practice*, fourth revision edition.

[10] Applied Technology Council (ATC)-40,*Seismic Evolution and Retrofit of Concrete Building*,Report No.SSC 96-01, Volume I, Applied Technology Council, California, November,1996.

[11] SAP 2000 manual *Three Dimensional Static and Dynamic Analysis and Design of Structures*, Computer and Structure Inc. Berkeley, USA.

[12] Massena, B., Bento, R., and Degée H.,*Direct Displacement Based Design of a RC Frame – Case of Study*, ISSN: 0871-7869,2010.

[13] Aidcer L. Vidot-Vega, Mervyn J. Kowalsky. *Drift, strain limits and ductility demands for RC moment frames designed with displacement-based and force-based design methods*. Department of Civil, Construction, and Environmental Engineering, North Carolina State University, Raleigh, NC, United States,2013, PP 128-140.

[14] Park R. and Paulay T., *Reinforced Concrete Structures*. John Wiley and Sons,Inc, U.K., reprint 2010 edition.

[15] Thacker, P.K., Purohit, S.P., *Displacement-Based Design of R.C.C. Buildings* M.Tech Dissertation, Civil Engg. Department, Nirma University, Ahmedabad, May 2011.

[16] Jinadni M, Purohit S.P., Suthar J.M., *Displacement Based Design of Wall-Frame and Shear Wall Buildings*, M.Tech Dissertation, Civil Engg. Department, Nirma University, Ahmedabad, May 2014.

[17] Muljati I., Kusuma Amelinda, Hindarto Fonny, *Direct displacement based design on moment resisting frame with out-of-plane offset of frame*, Procedia Engineering, Vol.125,2015,pp. 1057-1064

[18] Muljati I. , Asisi Fransiscus, Willyanto Kevin, *Performance of force based design versus direct displacement based design in predicting seismic demands of regular concrete special moment resisting frames*, Procedia Engineering, Vol. 125, 2015,pp. 1050 – 1056

[19] Muljati I. , Lumantarna Benjamin , Intan Reynaldo P.,Valentino Arygianny, *Performance of direct displacement based design on regular concrete building Indonesian response spectrum*,Vol.171,2017,pp. 1019-1024

[20] Govinda Raju, L., Ramana, G. V., Hanumantha Rao, C. and Sitharam, T.G., *Site-Specific Ground Response Analysis*, Current Science, Volume.87, No.10, 25 November (2004), pp.1354-1362.

[21] Dharna Dilip M. , Patel, P.V., *Site specific response analysis of high rise & low rise buildings* M.Tech Dissertation, Civil Engg. Department, Nirma University, Ahmedabad, May 2007.

[22] Siyani Prakash K., Patel, P.V., *Analysis and design of shear wall building using site specific response spectrum* M.Tech Dissertation, Civil Engg. Department, Nirma University, Ahmedabad, May 2008.

[23] Suarez, V. A. and Hurtado, J. C., *RC-Anlaysis. Virtual Laboratory for Earthquake Engineering*, 2008.

[24] Adhikary Shrabony ,Singh Yogendra ,Paul D.K., *Effect of soil depth on inelastic seismic response of structures*, Soil Dynamics and Earthquake Engineering 61-62 (2014) 13–28

[25] Faccioli, E., Paolucci, R., and Rey, J., *Displacement Spectra for Long Periods, Earthquake Spectra*, Vol. 20(2), 2004,pp 347-376.

Appendix A

Displacement Based Design

A.1 3-Storey Building

Building Data:

No. Of storeys:3

Building height: 9m

Plan Dimensions: 15m x 18m

Concrete Grade: M25

Steel Grade: Fe415

Size of Beams: 250mm x 400mm

Size of Columns: 400mm x 400mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Soil type: Medium

Design Storey Displacement Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System

$\triangle_d$= 0.139 m

Table A.1: Design Displacement of 3-Storey Building

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_i H_i$
3	9	194.801	1.000	0.180	35.064	6.312	315.578
2	6	204.587	0.667	0.120	24.550	2.946	147.303
1	3	204.587	0.333	0.060	12.275	0.737	36.826
Total	-	603.976	-	-	71.890	9.994	499.706

Effective Height of the System H_e = 6.951 m

Effective Mass of the System m_e = 517.12 ton

Design Ductility Factor of the System

Yield rotation is given as: θ_y= 0.014

Therefore, yield displacement is obtained as: $\triangle_y$= 0.099 m

Hence, ductility is found as:μ = 1.402

Equivalent Viscous Damping of the System ξ_{eq}= 10.159%

Effective Period at Peak Displacement Response T_e= 2.865 sec.

Effective Stiffness of the System K_{eff}=2484.257 kN/m

Stability index $\theta_\triangle$= 0.133,

Base shear force of building,V_{base}=391.122 kN

Base shear force of internal frame,V_{base}=130.374 kN

Table A.2: Distribution of Base Shear Force

Storey, i	Height , H_i (m)	$\triangle_i$	Mass (ton), m_i	$m_i\triangle_i$	$F_{i,x}$ (kN)
3	9	0.180	194.801	35.064	63.590
2	6	0.120	204.587	24.550	44.523
1	3	0.060	204.587	12.275	22.261
Total	-	-	603.976	71.890	130.374

A.2 10-Storey Building

Building Data:

No. Of storeys:10

Building height: 30m

Plan Dimensions: 15m x 18m

Concrete Grade: M25

Steel Grade: Fe415

Size of Beams: 300mm x 450mm

Size of Columns: 600mm x 600mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Soil type: Medium

Table A.3: Design Displacement of 10-Storey Building

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_i H_i$
10	30	218.807	1.000	0.4615	100.988	46.610	3029.640
9	27	240.826	0.930	0.4292	103.370	44.369	2790.984
8	24	240.826	0.853	0.3938	94.848	37.356	2276.359
7	21	240.826	0.770	0.3554	85.586	30.416	1797.301
6	18	240.826	0.680	0.3138	75.582	23.721	1360.480
5	15	240.826	0.583	0.2692	64.838	17.456	972.565
4	12	240.826	0.480	0.2215	53.352	11.820	640.226
3	9	240.826	0.370	0.1708	41.126	7.023	370.131
2	6	240.826	0.253	0.1169	28.158	3.292	168.948
1	3	240.826	0.130	0.0600	14.450	0.867	43.349
Total	-	2386.239	-	-	662.297	222.930	13449.982

Design Storey Displacement

Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System $\triangle_d$= 0.337 m

Effective Height of the System H_e = 20.308 m

Effective Mass of the System m_e = 1967.6 ton

Design Ductility Factor of the System Yield rotation is given as: θ_y= 0.015

Therefore, yield displacement is obtained as: $\triangle_y$= 0.309 m

Hence, ductility is found as:μ = 1.089

Equivalent Viscous Damping of the System ξ_{eq}= 6.474%

Effective Period at Peak Displacement Response T_e= 4.886 sec.

Effective Stiffness of the System K_{eff}= 3250.763 kN/m

Stability index $\theta_\triangle$= 0.24

Base shear force of building,V_{base}=1225.537 kN

Base shear force of internal frame,V_{base}= 408.512 kN

Table A.4: Distribution of Base Shear Force

Storey, i	Height , H_i (m)	$\triangle_i$	Mass (ton), m_i	$m_i\triangle_i$	$F_{i,x}$ (kN)
10	30	0.462	218.807	100.9880028	96.91268739
9	27	0.429	240.826	103.3697953	57.3836721
8	24	0.394	240.826	94.848271	52.65311848
7	21	0.355	240.826	85.58574453	47.51121238
6	18	0.314	240.826	75.58221595	41.95795379
5	15	0.269	240.826	64.83768525	35.99334271
4	12	0.222	240.826	53.35215243	29.61737915
3	9	0.171	240.826	41.1256175	22.83006309
2	6	0.117	240.826	28.15808045	15.63139455
1	3	0.060	240.826	14.44954128	8.021373519
Total			2386.239	662.297	408.5121972

A.3 12-Storey Building

Building Data:

No. Of storeys:12

Building height: 36m

Plan Dimensions: 15m x 18m

Concrete Grade: M25

Steel Grade: Fe415

Size of Beams: 300mm x 450mm

Size of Columns: 650mm x 650mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Soil type: Medium

Table A.5: Design Displacement of 12-Storey Building

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_i H_i$
12	36	243.077	1.000	0.551	134.054	73.929	4825.953
11	33	268.918	0.942	0.520	139.723	72.596	4610.852
10	30	268.918	0.880	0.485	130.454	63.284	3913.611
9	27	268.918	0.813	0.448	120.498	53.993	3253.447
8	24	268.918	0.741	0.409	109.856	44.877	2636.538
7	21	268.918	0.664	0.366	98.527	36.099	2069.065
6	18	268.918	0.583	0.322	86.511	27.831	1557.205
5	15	268.918	0.498	0.274	73.809	20.258	1107.140
4	12	268.918	0.407	0.225	60.421	13.575	725.048
3	9	268.918	0.313	0.172	46.345	7.987	417.109
2	6	268.918	0.213	0.117	31.584	3.709	189.501
1	3	268.918	0.109	0.060	16.135	0.968	48.405
Total	-	3201.172	-	-	1047.917	419.108	25353.874

Design Storey Displacement

Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System $\triangle_d$= 0.4 m

Effective Height of the System H_e = 24.194 m

Effective Mass of the System m_e = 2620.158 ton

Design Ductility Factor of the System

Yield rotation is given as: θ_y= 0.015

Therefore, yield displacement is obtained as: $\triangle_y$= 0.368 m

Hence, ductility is found as:μ = 1.086

Equivalent Viscous Damping of the System ξ_{eq}= 6.430%

Effective Period at Peak Displacement Response T_e= 5.319 sec.

Effective Stiffness of the System K_{eff}= 3652.004 kN/m

Stability index $\theta_{\triangle}$= 0.239

Base shear force of building,V_{base}= 1635.037 kN

Base shear force of internal frame,V_{base}= 545.012 kN

Table A.6: Distribution of Base Shear Force

Storey, i	Height , H_i (m)	$\triangle_i$	Mass (ton), m_i	$m_i\triangle_i$	$F_{i,x}$ (kN)
12	36	0.551	243.077	134.0542463	117.2496502
11	33	0.520	268.918	139.7227902	65.40174993
10	30	0.485	268.918	130.4537107	61.0630589
9	27	0.448	268.918	120.4980328	56.40298335
8	24	0.409	268.918	109.8557564	51.42152328
7	21	0.366	268.918	98.52688151	46.1186787
6	18	0.322	268.918	86.51140816	40.49444959
5	15	0.274	268.918	73.80933633	34.54883596
4	12	0.225	268.918	60.42066602	28.28183781
3	9	0.172	268.918	46.34539723	21.69345514
2	6	0.117	268.918	31.58352996	14.78368794
1	3	0.060	268.918	16.13506422	7.552536232
Total			3201.172	1047.917	545.012

Table A.7: Results of 12-Storey Building Zone III and IV

Response Quantities	Z=0.16 (MCE)	Z=0.08 (DBE)	Z=0.24 (MCE)	Z=0.12 (DBE)
Time Period, T (sec)	5.642	7.98	4.61	6.515
Stiffness, Ke (kN/m)	3246.227	1623.155	4869.349	2167.175
Base Shear Force (kN)	1472.749	823.611	1590.273	860.799

A.4 15-Storey Building

Building Data:

No. Of storeys:15

Building height: 45m

Plan Dimensions: 15m x 18m

Concrete Grade: M25

Steel Grade: Fe415

Size of Beams: 300mm x 500mm

Size of Columns: 650mm x 650mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Importance Factor: 1

Design Storey Displacement

Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System $\triangle_d$= 0.493 m

Effective Height of the System H_e = 30.029 m

Effective Mass of the System m_e = 3067.993 ton

Design Ductility Factor of the System

Yield rotation is given as: θ_y= 0.014

Therefore, yield displacement is obtained as: $\triangle_y$= 0.411 m

Hence, ductility is found as:μ = 1.2

Equivalent Viscous Damping of the System ξ_{eq}= 8.001%

Effective Period at Peak Displacement Response T_e= 6.117 sec.

Effective Stiffness of the System K_{eff}= 3234.092 kN/m

Stability index $\theta_\triangle$= 0.255

Base shear force of building,V_{base}= 1799.452 kN

Table A.8: Design Displacement of 15-Storey Building

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_iH_i$
15	45	227.676	1.000	0.684	155.817	106.638	7011.770
14	42	253.517	0.954	0.653	165.534	108.085	6952.425
13	39	253.517	0.905	0.619	157.052	97.292	6125.013
12	36	253.517	0.853	0.584	148.055	86.465	5329.987
11	33	253.517	0.799	0.546	138.545	75.713	4571.975
10	30	253.517	0.741	0.507	128.520	65.153	3855.604
9	27	253.517	0.680	0.465	117.981	54.906	3185.500
8	24	253.517	0.616	0.422	106.929	45.101	2566.290
7	21	253.517	0.550	0.376	95.362	35.871	2002.601
6	18	253.517	0.480	0.329	83.281	27.358	1499.059
5	15	253.517	0.407	0.279	70.686	19.709	1060.291
4	12	253.517	0.332	0.227	57.577	13.077	690.924
3	9	253.517	0.253	0.173	43.954	7.621	395.585
2	6	253.517	0.172	0.118	29.817	3.507	178.900
1	3	253.517	0.087	0.060	15.165	0.907	45.496
Total	-	3776.911	-	-	1514.275	747.403	45471.420

Table A.9: Results of 15-Storey Building Zone III and IV

Response Quantities	Z=0.16 (MCE)	Z=0.08 (DBE)	Z=0.24 (MCE)	Z=0.12 (DBE)
Time Period, T (sec)	6.49	9.17	5.3	7.49
Stiffness, Ke (kN/m)	2874.748	1437.374	4312.123	2156.061
Base Shear Force (kN)	1622.09	912.642	2331	1267.366

A.5 25-Storey Building

Building Data:

No. Of storeys:25

Building height: 75m

Plan Dimensions: 15m x 18m

Concrete Grade: M25

Steel Grade: Fe415

Size of Beams: 300mm x 500mm

Size of Columns: 700mm x 700mm

Slab Thickness : 150 mm

Floor Finish = 1kN/m^2

Live Load = 3 kN/m^2 on all floors

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Design Storey Displacement

Critical storey displacement, $\triangle_c = 0.02 \times 3 = 0.06$m

Design Displacement of the System

$\triangle_d$= 0.81 m

Table A.10: Design Displacement of 25-Storey Building

Storey, i	Height , H_i (m)	Mass (ton), m_i	δ_i	$\triangle_i$	$m_i\triangle_i$	$m_i{\triangle_i}^2$	$m_i\triangle_i H_i$
25	75	231.804	1.000	1.133	262.624	297.541	19696.779
24	72	261.774	0.973	1.102	288.511	317.979	20772.777
23	69	261.774	0.945	1.070	280.128	299.768	19328.799
22	66	261.774	0.915	1.037	271.428	281.438	17914.242
21	63	261.774	0.885	1.002	262.412	263.052	16531.953
20	60	261.774	0.853	0.967	253.080	244.674	15184.778
19	57	261.774	0.821	0.930	243.431	226.374	13875.566
18	54	261.774	0.787	0.892	233.466	208.219	12607.162
17	51	261.774	0.753	0.853	223.185	190.284	11382.415
16	48	261.774	0.717	0.812	212.587	172.642	10204.171
15	45	261.774	0.680	0.770	201.673	155.371	9075.278
14	42	261.774	0.642	0.728	190.442	138.548	7998.582
13	39	261.774	0.603	0.683	178.896	122.257	6976.931
12	36	261.774	0.563	0.638	167.033	106.580	6013.172
11	33	261.774	0.522	0.592	154.853	91.604	5110.152
10	30	261.774	0.480	0.544	142.357	77.416	4270.719
9	27	261.774	0.437	0.495	129.545	64.109	3497.719
8	24	261.774	0.393	0.445	116.417	51.773	2793.999
7	21	261.774	0.347	0.393	102.972	40.505	2162.407
6	18	261.774	0.301	0.341	89.211	30.402	1605.790
5	15	261.774	0.253	0.287	75.133	21.564	1126.995
4	12	261.774	0.205	0.232	60.739	14.093	728.869
3	9	261.774	0.155	0.176	46.029	8.093	414.260
2	6	261.774	0.105	0.118	31.002	3.672	186.014
1	3	261.774	0.053	0.060	15.659	0.937	46.978
Total	-	6514.373	-	-	4232.811	3428.896	209506.509

Table A.11: Results of 25-Storey Building Zone III and IV

Response Quantities	Z=0.16 (MCE)	Z=0.08 (DBE)	Z=0.24 (MCE)	Z=0.12 (DBE)
Time Period, T (sec)	8.3	11.74	6.78	9.58
Stiffness, Ke (kN/m)	2990.52	1495.156	4485.949	5105.96
Base Shear Force (kN)	2997.75	1786.4	4209.165	2392.12

Appendix B

Force Based Design

Seismic parameters of building are:

Earthquake Zone: Zone-V (Z=0.36)

Importance Factor: 1

Response Reduction Factor: 3

Soil Type: Medium

3-Storey Frame

Time Period, T = 0.39 sec

Spectral acceleration coefficient, S_a/g= 3.49

Horizontal seismic coefficient, A_h= 0.186 m/sec^2

Base shear force of building,V_{base}= 1102.675 kN

Base shear force of internal frame,V_{base}= 367.5882 kN

6-Storey Frame

Time Period, T = 0.655 sec

Spectral acceleration coefficient, S_a/g= 2.075

Horizontal seismic coefficient, A_h= 0.124 m/sec^2

Base shear force of building,V_{base}= 1647.528 kN

Base shear force of internal frame,V_{base}= 549.167 kN

10-Storey Frame

Time Period, T = 0.961 sec

Spectral acceleration coefficient, S_a/g= 1.414

Horizontal seismic coefficient, A_h= 0.085 m/sec^2

Base shear force of building,V_{base}= 1986.876 kN

Base shear force of internal frame,V_{base}= 662.29 kN

12-Storey Frame

Time Period, T = 1.102 sec

Spectral acceleration coefficient, S_a/g= 1.234

Horizontal seismic coefficient, A_h= 0.074 m/sec^2

Base shear force of building,V_{base}= 2146.583 kN

Base shear force of internal frame,V_{base}= 715.53 kN