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Seismic Behavior of Multi Storied RCC Structures Using Different Sets of Stiffness Modification Factors

by

Sadia Afzal

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To my most loving, respected and worthy parents, who assisted me through each hard time of my life, always prayed for my achievements, sacrificed for my success and blessed me with their ethical support at all times. Their presence is a source of inspiration and made me strengthen throughout my life. I would also bestow my modest effort to my honorable supervisor who best guide me to complete the work.

CAPITAL UNIVERSITY OF SCIENCE & TECHNOLOGY
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List of Publications

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Abstract

Stiffness Modification Factors (SMFs) are utilized as a part of linear analysis of reinforced concrete structures to represent the impact of concrete cracking. Different sets of SMFs are recommended in different codes. Before availability of finite element softwares and even after the availability, designers have been using gross stiffness of the members for reinforcement design as this practice yields more reinforcement for beams. However, this practice may be against the seismic design philosophy of weak beam-strong column. Therefore, there is a need to verify the seismic performance of these buildings through advanced techniques under framework of performance based seismic design (PBSD). In this study, seismic behavior of a realistic 7-storied building designed with code-based, in-practice (uncracked stiffness for reinf. design) and hypothetical (slab and beams uncracked) sets of SMFs has been investigated. The performance of the building thus determined is compared with code-based, (American Concrete Intitute (ACI)) SMFs as reference for each set of SMFs. Decrease in reduction factor due to inherent ductility "R_μ", storey displacement and storey drift has been observed while increase in over strength factor "Ω", storey shears and overturning moment inelastic demand has been observed for in-practice and hypothetical sets of SMFs indicating more capacity requirement i.e. more reinforcement demand. Higher demand may greatly affect the elements/ behavior that intended to be elastic i.e. formation of hinges in columns in upper stories, shear in frame sections, bond and slip failure and foundation pressure. In code-based SMFs case, the load is observed to be transferred from slab to beam approximately equally along the length of the beam and from beam to column. This mechanism is according to the philosophy of strength based design mechanism of load transfer in RC structures. In other SMFs cases, significant load is observed to be directly transferred from slabs to columns deviating from realistic behavior. Plastic hinge formation in code-based SMFs indicates code intended behavior, i.e. reasonable formation of plastic hinges at ends of beams and bottom of bottom storey columns. Whereas for other stiffness modeling cases, formation/ status of hinges is rather low. This may result into formation of hinges at undesirable locations such as columns at upper stories. Reinforcement quantity with code-based modifiers is observed to be 20% more economical than the other systems. It is concluded that Code-based SMFs are better and economical than other sets in terms of load transfer mechanism, shear and moment inelastic demand, reinforcement demand and seismic behavior.

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Abbreviations

PC	Plain concrete
RC	Reinforced concrete
SMFs	Stiffness Modification Factors
SDoF	Single Degree of Freedom
MDoF	Multi Degree of Freedom
E-SDoF	Equivalent Single Degree of Freedom
UBC	Uniform Building Code
BCP	Building Code of Pakistan
ACI	American Concrete Institute
CEB	Comite Euro-International
FEMA	Federal Emergency Management Agency
ACSE	American Society of Civil Engineers
TBI	Tall Buildings Initiative
LATBSDC	Los Angeles Tall Buildings Structural Design Council
CSA	Canadian Standards Association
SD	Stiff Soil
SB	Rock Soil
FEM	Finite Element Modelling
ESA	Equivalent Static Analysis
RSA	Response-spectrum analysis
PBSD	Performance based Seismic Design
PoA	Pushover Analysis
IO	Immediate Occupancy level
LS	Life Safety level

CP	Collapse Prevention level
ICC PC	International Code Council, Performance Code
NEHRP	National Earthquake Hazards Reduction Program
CSI	Computer Structures International

Symbols

$f'c$	Compressive strength of concrete
f_y	Yielding strength of steel
I	Moment of inertia
I_g	Gross moment of inertia
M_u	Ultimate moment
M_{cr}	Cracking moment
A_g	Gross cross-sectional area of section
A_s	Area of steel
P_u	Factored axial load
E_c	Modulus of elasticity of concrete
σ	Stress of concrete
ε	Strain of concrete
S_y	Yield or ideal strength of concrete component
k	Stiffness of the specified component
D	Dead Load
L	Live Load
W	Wind Load
E	Earthquake Load
l_p	length of plastic hinge
d_b	diameter of reinforcement bar
C_0	Modification factor to relate spectral displacement.
C_1	Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2	Modification factor to represent the effect of hysteresis shape on the maximum displacement response.
S_a	Response spectrum acceleration.
T_e	Effective fundamental time period
g	Acceleration of gravity
R	Response modification factor
R_μ	Reduction factor due to inherent ductility
Ω	Over strength factor
$V_{Elastic}$	Elastic base-shear
V_{Design}	Design base shear
$V_{Inelastic}$	Inelastic / Actual base shear

Chapter 1

Introduction

1.1 Preface

Seismic and gravity response of a structure depends upon the seismic region, soil at which it is placed as well as its member properties. Reinforced concrete has been widely used as a major structural material in civil engineering buildings due to its strength, workability, durability and ease in handling and molding. It is a design requirement to satisfy both strength and serviceability when designing a reinforced concrete structure. Serviceability of reinforced concrete buildings could be ensured by precisely measuring and setting the deflection under gravity and lateral forces with in tolerable limits. To avoid second-order P-delta effects, control of lateral drift and deformation are most important serviceability criterion. (Chan, C. M. et al. 2000; Kara, I. F., & Dundar, C. 2009).

ACI-318 (2011) describes concrete weak and brittle in tension. Thus, cracks when load is applied. Consequently, mixed configuration including concrete and steel reinforcement is being utilized in structures. However, reinforcing bars start taking load after cracking of concrete (Arthur, H. Nilson, 2004). The phenomenon of concrete cracking results in the reduction of stiffness, hence moment carrying capacity (Zhou and Zheng, 2010). The cracking of concrete not only occurs at design basis earthquake level but also at much lower level and even under gravity loads.

As a result of cracking, moment is redistributed to other uncracked elements of the structure (Lopes, S. M., et al., 1997; Do Carmo and Lopes, 2005). Effects of concrete cracking are incorporated in most of the codes by reducing the stiffness of members using modification factors (Paulay, T., & Priestley, M. N., 1992).

Most of the concrete buildings are designed and analyzed through linear elastic practices. To extract a reasonable structural response, modification in stiffness of concrete elements is required. Different codes permit the designers to use modification factors less than 1 for stiffness to tackle with the effect of cracking in case of seismic event. (Wong, J. M., et al. 2016). Priestely (2003), however, reported that most seismic codes do not mention precisely the use of effective stiffness for seismic analysis of RC structures; structural stiffness of elements is, therefore, computed using un-cracked section properties. It is observed in professional practice that un-cracked stiffness modifiers are being used for reinforcement design. Pique and Burgos, (2008) reported that use of uncracked elements will result in increment of design moments and seismic shears. As per author's knowledge there is no study which investigated the seismic performance of a realistic RC building with code-based and in-practice used SMFs. Performance of the buildings designed with code-based and in-practice SMFs through advanced techniques is needed to be explored. In this study, therefore, seismic performance of a realistic building designed with code based stiffness modification factors (SET-1), un-cracked stiffness modification factors (SET-4), and hypothetical stiffness modification factors (SET-2 and 3) has been investigated under the frame work of performance based seismic design. The performance level in our case is life safety against design basis earthquake. The design basis earthquake is an event with 10

To determine the true seismic behavior of the buildings, an advanced method known as performance-based earthquake engineering is used (FEMA 356; ATC 40; Themelis, S., 2008). Performance based seismic design (PBSD) which explicitly evaluate the performance level against hazard level, probability of damage of a structure in case of seismic event could be analyzed. The prime achievement of PBSD is to attain a desired performance objective when structure is subjected to a specific hazard level. In PBSD, FEMA Pushover analysis (PoA) is an advanced technique utilized to assess the behavior of the fundamental mode dominant building. Response of nonlinear push-over procedures is represented by nonlinear load-deformation relations/ curve (FEMA 356).

1.2 Research Motivation and Problem Statement

Stiffness Modification Factors (SMFs) are used in the linear analysis of reinforced concrete (RC) structures to account for the effect of concrete cracking. Various Sets of SMFs are recommended in different codes. Designers are using different Sets of SMFs in different regions of the world. Even Sets of SMFs for the deflection check and reinforcement design are different in the same design. Moreover, seismic performance of RC structures with and without these SMFs is not studied so far. Therefore, there is need to explore seismic behavior of buildings with different sets of SMFs and what could be the consequences if building is designed without using SMFs. In this study, seismic behavior of a realistic building is examined with code-based, in practice and hypothetical sets of SMFs. The performance of the building with in-practice and hypothetical sets is compared with code-based (ACI) SMFs.

1.3 Objectives

Different factors have been proposed by different researchers and codes. Code-based factors are in practice to some extent. Therefore, the buildings have been and are being designed using uncracked stiffness of concrete elements. The prime objective of this study is therefore to explore seismic behavior of multistoried RCC structures located in zone 2B and soil type SD (stiff soil) and SB (rock) for different set of SMFs. The specific objectives of this research are:

1. Seismic performance Verification of real mid-rise building using Code-based SMFs

Different Sets of SMFs

2. Quantity of Steel rebars comparison.

1.4 Scope of Work and Research Methodology

In order to meet the objectives of this study, a realistic 7-storied building is studied in seismic zone 2B for two different soil types SD and SB. Building is first designed in accordance with code using different Set of stiffness modifiers for moment of inertia i.e. code-based factors (ACI defined as Set-1), uncracked sections (Set-4) and hypothetical SMFs (Set-2 and Set-3 described below). Assessment of seismic behavior of these code-based designed buildings is then performed using FEMA static non-linear pushover procedure in context of performance based seismic design to predict extent of damage in terms of plastic rotations, seismic responses and lateral deflection along with the performance of the building.

TABLE 1.1: Models created for this study.

<table>
<tr><th rowspan="2">S. No.</th><th rowspan="2">Set No.</th><th rowspan="2">Model</th><th colspan="4">Set of Stiffness Modification Factors to be used</th></tr>
<tr><th>Column</th><th>Wall</th><th>Beam</th><th>Slab</th></tr>
<tr><td>1</td><td rowspan="2">1</td><td>Code-based Design</td><td rowspan="2">0.7</td><td rowspan="2">0.7</td><td rowspan="2">0.35</td><td rowspan="2">0.35</td></tr>
<tr><td>2</td><td>Push-over Analysis</td></tr>
<tr><td>3</td><td rowspan="2">2</td><td>Code-based Design except SMFs</td><td rowspan="2">0.7</td><td rowspan="2">0.7</td><td rowspan="2">0.35</td><td rowspan="2">1</td></tr>
<tr><td>4</td><td>Push-over Analysis</td></tr>
<tr><td>5</td><td rowspan="2">3</td><td>Code-based Design except SMFs</td><td rowspan="2">0.7</td><td rowspan="2">0.7</td><td rowspan="2">1</td><td rowspan="2">1</td></tr>
<tr><td>6</td><td>Push-over Analysis</td></tr>
<tr><td>7</td><td rowspan="2">4</td><td>Code-based Design except SMFs</td><td rowspan="2">1</td><td rowspan="2">1</td><td rowspan="2">1</td><td rowspan="2">1</td></tr>
<tr><td>8</td><td>Push-over Analysis</td></tr>
</table>

Total sixteen models are prepared (8 with soil type SD and Zone 2B, 8 with soil type SB and Zone 2B as shown in Table 1.1) comprising of different Set of modification factors. For different models, seismic response parameters such as base shear, storey shear, storey moment, plastic hinges rotation and their status have been compared. Effect of variation of modification factors over cost of structure is also assessed. Based on the comparison, most suitable approach which results economical design yet satisfying desired seismic performance has been recommended.

1.5 Limitations of the Study

The limitations of this study are:

1. Only numerical study has been conducted using equivalent static analysis (ESA) and push-over analysis (PoA).
2. Only two soil types SD (stiff soil) and SB (rock) have been considered in this study.
3. Quantity of steel has been compared only.
4. Nonlinearity has been assigned only at specified places not throughout the elements.
5. Bond and slip failure of connections have not been considered in the research.
6. Determination of new stiffness modification factors is beyond the scope of this study.

1.6 Thesis Outline

Chapter 1: This chapter includes research gap and motivation. Objectives, limitations and methodology have been outlined.

Chapter 2: This chapter gives a detail review about concrete cracking behavior, moment redistribution in case of a seismic event and use of stiffness modification factors.

Chapter 3: This chapter provides an overview about modeling of stiffness and approaches used to design and evaluate the structure. Modeling and assignment of non-linear hinge property is also explained. A case study of a 7-storied building is given in detail. First elastic design of intermediate moment resisting frame (IMRF) is performed, and then non-linear design technique is applied.

Chapter 4: This chapter covers comparison of different seismic parameters for different cases. Non-linear behavior and plastic hinge formation is compared. Furthermore, steel comparison has been presented for different cases using different stiffness modification factors.

Chapter 5: This chapter is a summary of research work performed. Conclusions have been drawn and future recommendations have been illustrated.

Chapter 2

Literature Review

2.1 Background

This chapter illustrates a short description of behavior of RC structures. Concrete is very strong in compression, while very weak in tension. Therefore, reinforced concrete (RC) sections are usually employed to withstand bending moments and axial forces neglecting any resistance of concrete to tensile stresses. Concrete cracking under applied forces and moment redistribution phenomena as a result of the cracking is discussed. To tackle the cracking phenomenon, modification in moment of inertia of concrete member allowed by different researchers and codes is presented. Moreover, technique used for seismic assessment of structures is also described.

2.2 Reinforced Concrete Behavior

Plain concrete (PC) beams are insufficient as structural members due to the fact that tensile strength of concrete in bending is very low as compared to that of compressive strength (Arthur, H. Nilson, 2004). ACI-318 (2011) stated that ability of concrete against tensile forces is much less than that of its ability to resist compressive forces. Thus, the tensile strength is approximately 10% than that of compressive strength of concrete. The Concrete, being weak in tension, cracks in tensile zone when load is applied as shown in Figure 2.1. Therefore, heterogenous composition comprising of concrete and steel reinforcement bars in tension area of concrete is being utilized in reinforced concrete (RC) structures. However, reinforcing bars starts taking load after cracking of concrete (Arthur, H. Nilson, 2004).

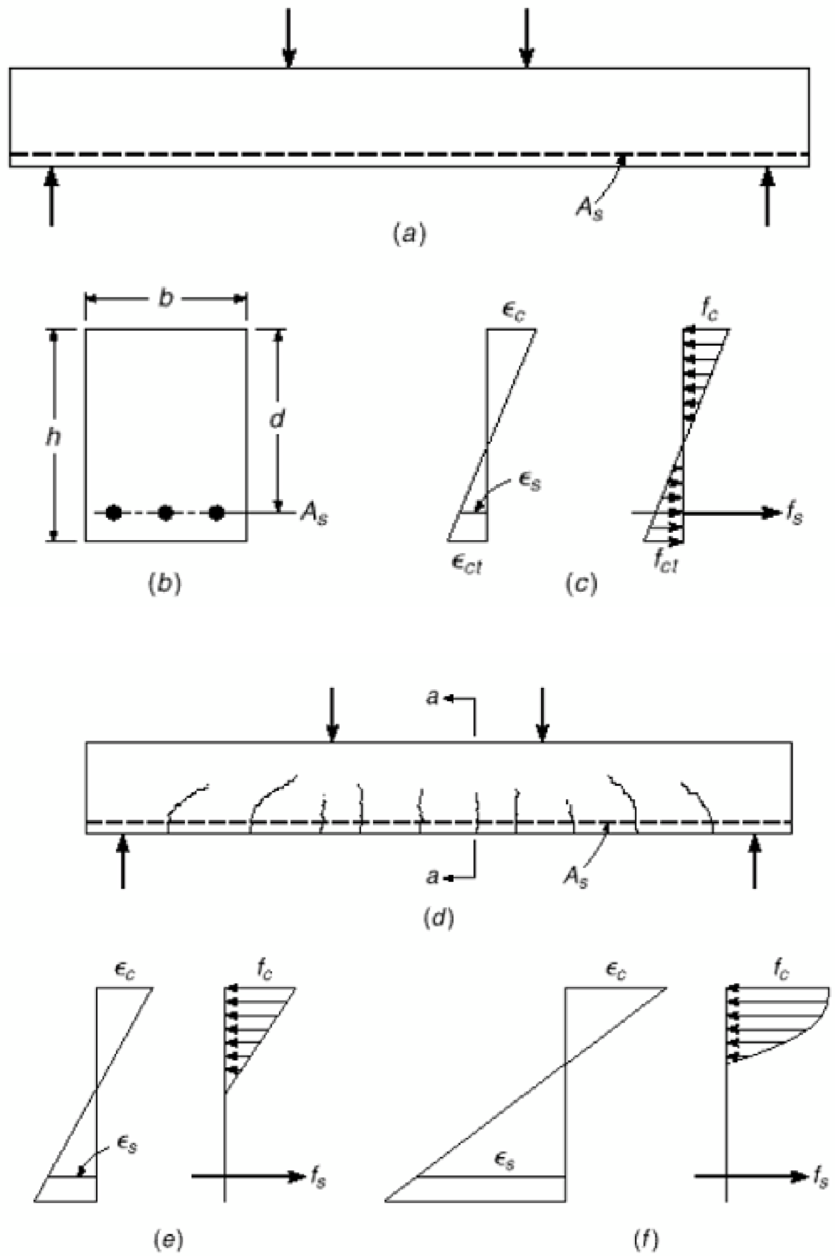


FIGURE 2.1: Behavior of reinforced concrete beam under increasing load (Arthur, H. Nilson, 2004).

In Figure 2.1 (a) a simple beam with steel reinforcement bars on tension side is shown along with its cross-section and distribution of stress and strain in Figure 2.1 (b) and (c) respectively. Upward propagation of cracks towards neutral plane can be seen from Figure 2.1 (d) as a result of applied load on the beam. In a well design beam, the width of these cracks must be very small. However, their presence greatly disturbs the behavior of the beam. In cracked section under loading, concrete does not transmit any tensile stresses. This can be seen from distribution of stress and strain at the cracked position of the beam and at ultimate load as shown in Figure 2.1 (e) and (f) respectively.

Cosenza E. (1990) established a analytical method for the finite element analysis of RC beams in a cracked position. Method for the analysis of two dimensional reinforced concrete beams developed by American Concrete institute (ACI) and Comite Euro-International (CEB) were considered for simply supported uniformly loaded beams. These methods or model equations deliberate the involvement of tensile resistance of concrete to flexural rigidities by moment-curvature relationships. ACI Committee 224 report stated expression to calculate tensile strength of concrete as function of its compressive strength. These approaches were used to evaluate the effective moment of inertia.

Linear tension stiffening was also taken into account. Tension stiffening is linked to the contact of the reinforcement and the concrete and has a substantial effect on the deflection of reinforced concrete components. In cracked state, tensile stresses are transferred to the concrete in uncracked regions present between the cracked regions. Thus, in uncracked portion of the concrete, steel stresses are reduced and the stiffness is increased with respect to the cracked state in which there is no contribution of concrete in tension. This is known as tension stiffening effect of the concrete (Moosecker, W., & Grasser, E., 1981). Cracked, uncracked and effective behavior of reinforced concrete sections is shown in Figure 2.2.

With reference to Figure 2.2, in uncracked state of concrete section (state 1) governing inertia I_1, considering the reinforcement, results the value of moment

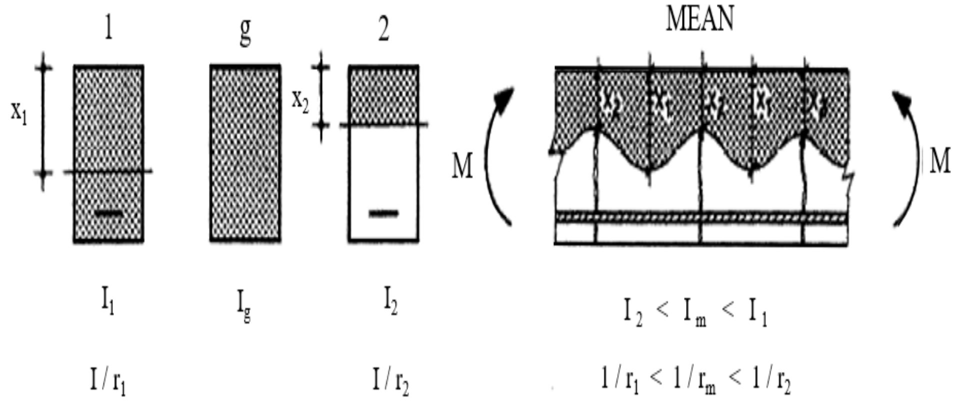


FIGURE 2.2: Cracked, uncracked and effective behavior of reinforced concrete sections (Cosenza E., 1990).

less than that of cracking moment with respect to gross section (state g) without any reinforcement. As concrete is not cracked yet, hence, stresses are not transferring to steel reinforcement. Concrete is taking stresses as shown at x_1. However, moment is constant and moment of inertia is varying along the length of the beam. In the cracked section (state 2), inertia I_2 is evaluated without the contribution of the concrete in tension because of the cracks. At position x_2 in Figure 2.2 cracks in concrete can be seen, all the tensile forces are taken by steel reinforcement at this stage. In fact, the sections where the cracks are localized are separated by regions where the concrete in tension is uncracked, with an increase of the stiffness, this phenomenon is known as tension stiffening (Cosenza E. 1990).

As a result of which effective inertia lies between I_1 and I_2, in a region of moment more than that of cracking moment as presented in Figure 2.2. The same phenomenon is proved for moment-curvature relationship as shown in Figure 2.3. It can be noticed that moment of inertia in state 1 is slightly less than that of gross moment of inertia in state g. For state 1 and 2 moment-curvature relationship resulted as linear. The real behavior for values of M greater than Mcr moment-curvature relationship lies in between states 1 and 2, due to the tension stiffening as shown by m line in Figure 2.3.

Lopes, S. M., et al. (1997) studied redistribution of moment in concrete beams by testing seven prestressed beams and stated that behavior of beams deviate

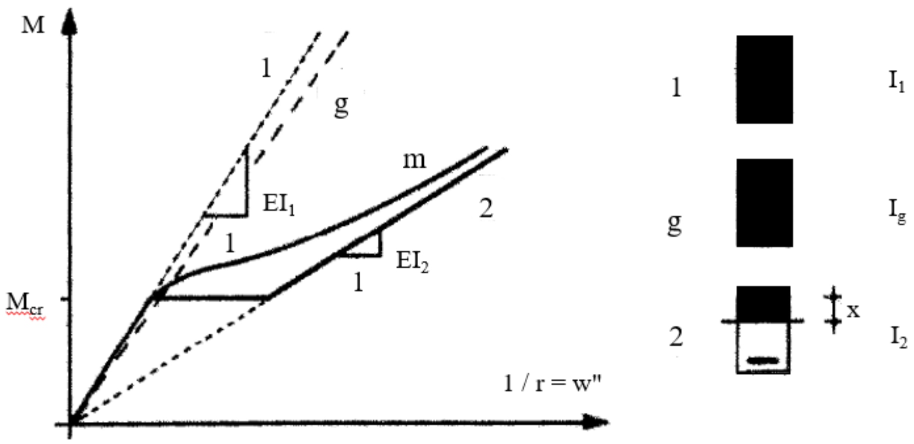


FIGURE 2.3: Moment-curvature relationship of reinforced concrete section (Cosenza E., 1990).

from elastic theory as load increased. A typical moment curvature relationship for concrete section is shown in Figure 2.4. Where M_{cr} is the cracking moment of the section and M_u is the ultimate moment. For moments more than M_{cr} the stiffness is significantly decreased by cracking. Maximum moment concentration happened prior to failure over a very short length of the member. These short lengths can be considered as plastic hinges. This phenomenon of hinge formation lasts until failure of the structure happened. Three different types of moment-loading relationships could be possible (Tfchy and Rakosnfc, 1977).

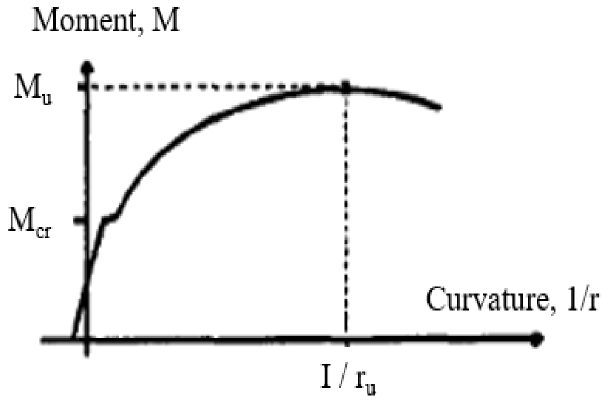


FIGURE 2.4: Moment-curvature relationship for concrete section (Lopes, S. M., et al., 1997).

Partial or full moment redistribution happened in beams at ultimate load as presented in Figure 2.5. In the Figure, M_u is ultimate moment and M_{cr} is cracking moment. Both M_u and M_{cr} is same for the sections having point load or distributed load. In the Figure, moment 1 and 3 is at ends of beams while moment at center of beam is represented as 2. For beam under point load moment at ends and center is same hence $M_1 = M_2 = M_3$ as shown in Figure 2.5 (a). For beam having distributed load moment at both ends is same but twice of the moment at center of the beam i.e. $M_1 = M_3 = 2M_2$. Cracking would be happened when edge moment reached M_{cr}. After this stage a decrease in stiffness can be observed from Figure 2.5 associated with development of cracks. If the three moment-loading curves ends at one-point M_u, then full redistribution is supposed to be taken place as in Figure 2.5 (b). If the ultimate moment of resistance of section 2 is not completed, then partial redistribution is said to have taken place as in Figure 2.5 (c).

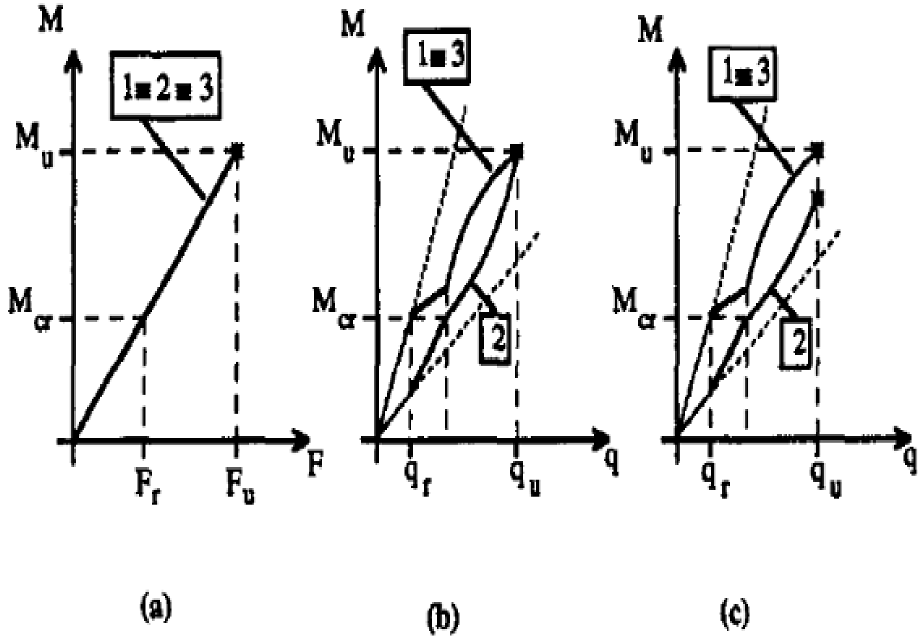


FIGURE 2.5: Moment redistribution in Fixed-Ended beam (Lopes, S. M., et al., 1997).

The perception of ductility of a structure is related to the moment redistribution capability and subsequently the safety of the structure (Do Carmo and Lopes 2005). The phenomenon of concrete cracking results in the reduction of stiffness, hence moment carrying capacity. As a result of this, some moment of that cracked section shifts to another uncracked section. However, upon further cracking in that section, again moment reshuffling happened. This process of redistribution of moment continuous till all the members cracked (Zhou and Zheng, 2010). Do Carmo and Lopes (2005) determined that moment reshuffling due to concrete cracking could be up to 20% of the moment inelastic demand of certain section.

Do Carmo and Lopes (2005) studied moment redistribution in high strength concrete beams. Ten (10) continuous beams of 6 m length were tested with variation of flexure and transverse reinforcements at mid-point support. Gradual loading of 0.05 kN/s was applied until failure. Moment redistribution was observed through moment diagrams varied from 4% to 20% from original moment due to the concrete cracking and the stiffness variation along the beam caused by cracking.

In RCC buildings response, cracking of concrete is a dominant element. There are many estimates presented in the analysis and extensive distinctions exist in the material parameters associated to the cracking of the concrete. Bažant, Z. P., (1985) described that cracking is an important aspect of the behavior of concrete structures. Concrete structures are generally cracks, even under service/ gravity loads, redistribution of moments accured and behavior of elements deviate from elastic theory. Therefore, cracking should be taken into account in forecasting ultimate load capacity as well as behavior under gravity and lateral loads (Gérard, B., et al., 1996).

2.3 Stiffness Modification

Stiffness is the rigidness of a body, the degree to which it resists deformation in response to a force. Paulay, T., & Priestley, M. N., (1992) explains stiffness as link between applied forces and resulted deformations. This relationship is established considering mechanics of structures, geometry and material properties of components and the modulus of elasticity of material. For sustainable reinforced concrete structures, extent and impact of cracking in members and the input of concrete in tension required to be considered. Nonlinear relationship between applied forces or loads and resulted deformations presenting the response of a reinforced concrete constituent subjected to monolithically cumulative displacement is shown in Figure 2.7.

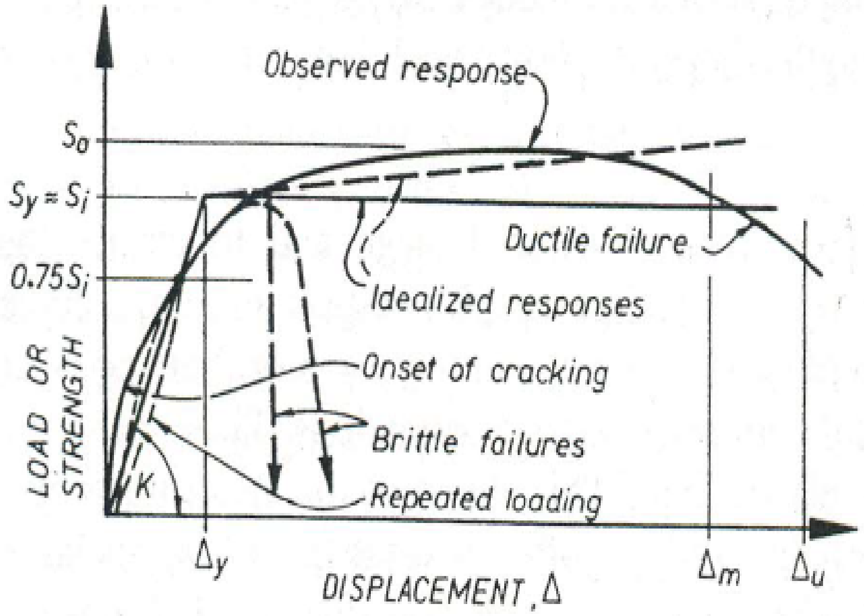


FIGURE 2.6: Typical load-displacement relationship for a reinforced concrete element (Paulay, T., & Priestley, M. N., 1992).

In the above figure, S_y expresses yield or ideal strength of concrete component and the slope of idealized linear elastic response, $K = S_y/\Delta y$ could be defined as stiffness of the specified component.

ACl-318 (2011) describes that inelastic action happened in structural members prior to yielding and degree of cracking could be expressed by stiffness of structural members. Presence of cracking in concrete structural members means variable moment of inertia. For this variable moment of inertia any consistent Set of reasonable assumptions could be utilized in design to compute relative flexural and torsional stiffnesses of structural members i.e. Columns, Beams, Walls, Floors etc. Reduced stiffness is necessary to compute the effect of long-term deflection correctly.

Cracked regions are present along the length of the members. To account for the effect of cracking, elastic second order analysis needs to be considered. When loads are approaching ultimate load capacity, elastic second order analysis predicts the lateral deflections. The stiffness or moment of inertia used for designing of members must be the depiction of stiffness of member immediately prior to failure. Shear deformation is large after the development of cracks in concrete (ACl 318-2011).

Stafford Smith and Coull (1991) stated that in design of concrete structures, cracking can start happening at service loads and reduces lateral stiffness and increase lateral deflection of a building. A specified ratio of inertia of columns and beams decreases to estimate the lateral drift. The moment of inertia of beams decreased to 50% of their groos inertia values, whereas the moment of inertia of columns is decreased to 80% of their uncracked values for all types of the buildings without considering extent of loading.

Al-Zaid, R., & Al-Shaikh, A. H., (1993) studied impact of reinforcement ratio on the effective moment of inertia of RC simply supported beams with rectangular section under a mid-span concentrated load. It was examined that for lightly reinforced beams, effective moment of inertia was approximately 55 percent to that of for heavily reinforced beams.

Kara I. F., & Dundar, C. (2007) have developed an iterative procedure to analyze three dimensional RC frames with cracked beams and columns elements by utilizing probability based effective stiffness mode (PBESM)l. In the PBESM, the effective moment of inertia is calculated as the ratio of the area of the moment diagram segment over which the working moment exceeds the cracking moment M_{cr} to the total area of the moment diagram as shown in Figure 2.8. Cracked sections are localized and separated by regions where the concrete in tension is uncracked. Experimental results with applied loads up to 78% of the ultimate load inelastic demand of the frame have been found similar as in analytical procedures. Beyond this limit theoretical and experimental results vary (Kara I. F., & Dundar, C., 2007 and Chan, C. M. et al., 2000).

Kara I. F., & Dundar, C. (2009) investigated impact of loading and reinforcement ratio on effective moment of inertia and deflection of reinforced concrete beams.

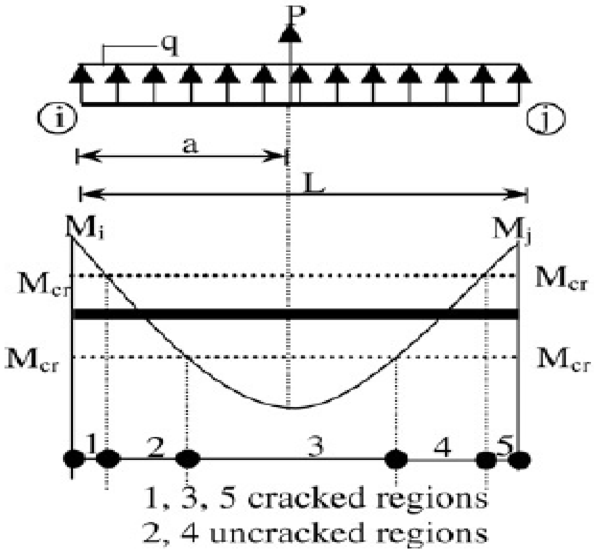


FIGURE 2.7: Cracked and uncracked regions of a member (Kara I. F., & Dundar, C., 2007).

Stiffness matrix method was used to conclude probability of analytical procedure for beams under different loading situations. It was reported that for mid-point loading case, reinforcement ratio has substantial effect on effective moment of inertia while under two-point loading case and uniformly distributed loading case, the effect is less significant.

Ahmed, et al. (2008) studied consequence of concrete cracking on the lateral response of RCC buildings. Linear and cracked analysis has been performed with different aspect ratios and heights of the building. It was reported that concrete cracking has significant effect on the deflection up to of 50% enhancement and an increase of 40% in the drift of the building as shown in Figure 2.6 when compared with that of linear analysis. Increase of drift is more in high-rise buildings.

J.R. Pique and M. Burgos, (2008) studied the effective rigidities of reinforced concrete elements in seismic analysis and design. Application of reduction factors proposed by Paulay and Priestley (1992) has been made on a four storey existing building. Increase in seismic shear and decrease in drift and distortion has been reported for uncracked case. It was reported that to obtained realistic distortions cracking must be incorporated in seismic analysis. Furthermore, design moments will results more if seismic analysis has been performed using uncracked analysis.

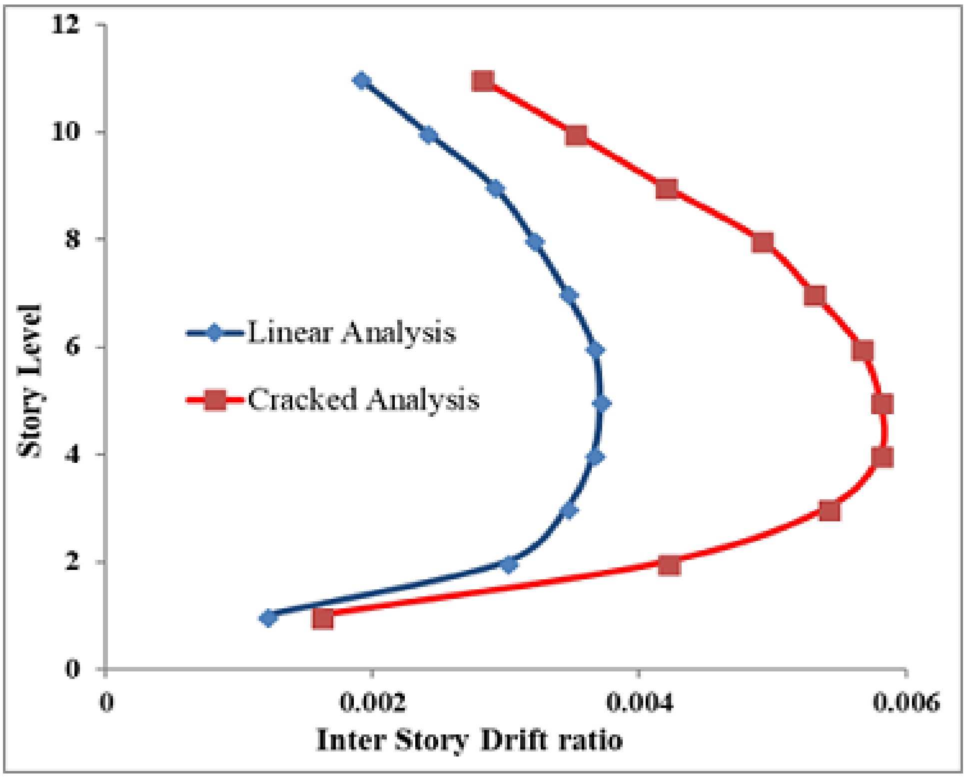


FIGURE 2.8: Inter storey drift ratio for linear and cracked concrete analysis (Ahmed et al., 2008).

Concrete buildings are designed and analysed through linear elastic practices. This is challenging technique for reinforced concrete building design due to the reason that complex interaction between its materials results in non-linear behaviour. Modelling of concrete as liner elastic approach requires simplifying the behaviour of concrete involving stiffness of concrete components. However, this method is challenging as reinforced concrete elements perform contrarily under altered loading conditions i.e. tension, compression, flexure etc. as well as different rates of loading i.e. impact, short term, long term etc. (Wong, J. M., et al. 2016).

As a result of an earthquake, phenomena of moment redistribution happened in concrete sections throughout their length. Concrete will crack in flexure at said time resulting variable moment of inertia (I) along the length. Moment at any point along the length of beam, quantity of flexural steel, cross section of concrete member, axial load on the member and tension stiffening effects are some of the main parameters that has an impact on moment of inertia. Diagonal cracking of a member due to shear intensity, direction of axial load and reverse cyclic loading are additional phenomena affecting member stiffness. For sake of durability control of concrete, cracking is essential in RC structures (Robert, P. et al., 1975). Thus, the moment of inertia of a section should be amended and an average value of EI is applied to the entire length of the beam for estimating flexure stiffness of a member (Paulay, T., & Priestley, M. N., 1992). Impacts of concrete cracking are integrated in most of the codes by decreasing the stiffness of members using modification factors for moment of inertia (Paulay, T., & Priestley, M. N., 1992).

2.4 Modification Factors in Different Codes

Stiffness Modification factors (SMFs) used for modeling concrete structures in different codes is presented in Table 2.1. Building Code Requirements for Structural Concrete, ACI-318, (2011) Sections 8.8.1 through 8.8.3 stated principles and procedures for effective stiffness values to be used to determine deflections under lateral loading. The stiffnesses EI defined for the analysis should present the stiffnesses of the members immediately prior to failure. Section 10.10.4.1 allows use of stiffness modification factors of less than 1 in the member properties for effective stiffness in the analysis. For uncracked compression member, 70% of the total moment of inertia can be utilized in the analysis, while for cracked compression as well as flexure members 35% of gross moment of inertia should be used in the analysis. For flat plates and flat slabs ACI allowed use of 25 percent of gross moment of inertia. For all other area elements 1.0 Ag has been recommended.

For compression and flexure members reduction of 30% and 70% has been allowed respectively in ASCE/SEI 41-13, Seismic Evaluation and Retrofit of Existing Buildings, Table 10-5.

Federal Emergency Management Agency (FEMA) 356, stated that stiffness of any component can be computed taking into account the deformations and behavior in shear, flexure and axial. FEMA 356 permits to use 50% moment of inertia in design and analysis of flexure member like beams. For compression members, if compression force due to gravity loads has a greater value than half of the product of gross cross-sectional area and concrete compressive strength as mentioned in Table 2.1, then 70% of moment of inertia could be used. Contrary, if compression force due to gravity loads is less than thirty percent of the product of gross cross-sectional area and concrete compressive strength as mentioned in Table 2.1, reduction of 50% in moment of inertia has been permitted in FEMA 356.

Guidelines for Performance based Seismic Design of Tall Buildings, also referred as PEER (2010) Tall Buildings Initiative (TBI) provisions also gives recommendations for effective stiffness modifications to be used in linear static analysis subjected to a service level earthquake (having return period of 43 years or 50% posibility of exceedance in 30 years). Both for beams and columns, PEER TBI allows to use reduce moment of inertia up to half of the gross moment of inertia.

Los Angeles Tall Buildings Structural Design Council (LATBSDC-2014) permits to incorporate actual stiffness and strength taking into account the expected level of excitation and damage. Stiffness of beams could be reduced by 30% according to recommendations of LATBSDC (2014). For compression members i.e., columns 0.9Ig (moment or inertia) could be used in design and analysis.

Paulay, T., & Priestley, M. N., 1992 and Priestley et al. 2007 concluded that the stiffness of a member is related to its strength. Recommendations have been made regarding assignment of stiffness modifiers to structural elements when designing any structure using code-based analysis procedure. Ranges defined regarding reduction in moment of inertia in different codes have been presented in Table 2.1.

Table 2.1: Stiffness Modifiers presented in codes for modeling concrete structures (Wong, J. M., et al. 2016).

	Elements	**ACI 318-11 10.10.4.1 ACI 318-14 6.6.3.1.1**	**ASCE 41-13 Table 10-5**	**FEMA 356 Table 6-5**	**PEER TBI Guidelines Service level**	**LATBSDC MCE-level (2014)**	**CSA A23.3-14**	**Paulay & Priestley (1992)**	**Priestley, Calvi & Kowalsky (2007)**
Beams	Regular Beams	0.35Ig	0.35Ig	0.5Ig	0.5Ig	0.35Ig	0.35Ig	0.4Ig	0.7Ig-0.44Ig
	Prestressed Beams	n/a	1.0Ig	1.0Ig	1.0Ig	n/a	0.35Ig	n/a	n/a
Columns	Columns- Pu $\geq$ 0.5Agf'c	0.7Ig	0.7Ig	0.7Ig	0.5Ig	0.9Ig	0.7Ig	0.18Ig	0.12Ig-0.86Ig
	Columns- Pu $\leq$ 0.3Agf'c	0.7Ig	0.7Ig	0.5Ig	0.5Ig	0.9Ig	0.7Ig	0.6Ig	
	Columns- Pu $\leq$ 0.1Agf'c	0.7Ig	0.3Ig	0.5Ig	0.5Ig	n/a	0.7Ig	0.4Ig	
Walls	Uncracked	0.7Ig	n/a	0.8Ig	0.75Ig	n/a	0.7Ig	(1)	n/a
	Cracked	0.35Ig	0.5Ig	0.5Ig	0.75Ig	1.0E_c (1)	0.35Ig		0.2Ig-0.3Ig
Slab	Conventional Slab	0.25Ig	See 10.4.4.2	n/a	0.5Ig	0.25Ig	0.25Ig	(1)	n/a
	Post-tensioned Slabs	n/a	See 10.4.4.2	n/a	0.5Ig	0.25Ig	0.25Ig	n/a	n/a

Notes:

1. Non-linear fiber elements naturally account for cracking of concrete as concrete fibers has no tension stiffness.
2. Effective stiffness as per equation according to the case.
3. Paulay & Priestley (1992) suggested to use stiffness modifier 0.35 Ig for T and L beams.
4. No reduction is recommended in shear stiffness of walls in ASCE 41-13, thus, suggested to use 1.0 modifiers for shear stiffness of concrete shear walls.

Definitions:

Ig = Gross moment of inertia

Ability of a body to withstand angular acceleration, which is the sum of the products of the mass of each particle in the body with the square of its distance from the axis of rotation.

P_u = Factored axial load

Axial load is a force along the line of axis of a body. Factored axial load is product of magnitude of load with that of factors defined in code.

$A_g = A_c$ = Gross (uncracked) area

Gross area is total cross-sectional area of any section.

f'c = Compressive strength of concrete

Compressive strength is the inelastic demand of a material or structure to withstand loads tending to reduce size.

E_c = Modulus of elasticity of concrete

The modulus of elasticity (Young's modulus) E is a material property, that defines its stiffness. From the Hook's law the modulus of elasticity is defined as the ratio of the stress to the strain ($E = \rho/\varepsilon$ (MPa)).

2.5 Seismic Assessment of the Structure

The need of seismic evaluation and design of structures is because of the occurrence of the earthquakes. Earthquakes are caused by sudden rupture and distinctive actions of the geological fault (Kramer, 1996). As a result of these actions 'ground shaking' produced that can cause substantial damage and/or collapse of buildings and infrastructure systems. A recent and advanced method to handle the design and/or assessment problems introduced is performance-based earthquake engineering (FEMA 356; ATC 40; Themelis, S., 2008). Inelastic seismic behavior of the structures during an earthquake is controlled by response modification factor known as "R" factor. A simplified "R" factor equation was formulated by Newmark and Hall (1973) on the basis of elastic and inelastic response spectrum of EL-centro1940 earthquake, which depends upon target period and ductility ratio of that structure. "R" factor includes inelastic performance of the structure. It is a nominal coefficient representative of the inherent over strength and global ductility inelastic demand of lateral-force-resisting systems (FEMA, 2005 and Abdollahzadeh, G., et. al., 2013). "R" factor is the ratio of elastic strength demand to design strength/base shear as expressed in Equation 2.1.

$$R = \frac{V_{Elastic}(\text{elastic base-shear})}{V_{Design}(\text{design base-shear})} \tag{2.1}$$

Figure 2.9 shows seismic response of a structure in case of earthquake.

- The red line indicates the force and displacement values if the structure responded elastically.
- The green line shows the actual force vs. displacement response of the structure.
- The pink line indicates the minimum strength needed to hold the structure during inelastic behavior.
- The blue line is the force for which building has been designed.

In Figure 2.9, the ratio of elastic response (base shear) to actual response i.e. inelastic base shear is known as "R_μ" and the ratio of inelastic base shear to design base shear is called over strength factor "Ω". Total "R" value is the product of "R_μ" and "Ω".

$$R = R_\mu \times \Omega = \frac{V_E}{V_{In}} \times \frac{V_{In}}{V_D} = \frac{V_E}{V_D} \tag{2.2}$$

Code-based "R" factor used for the current study has a value of 6.5 because of the dual frame structural system. Over strength factor "Ω" has been defined as 2.8 in UBC-97, thus code-based value of "R_μ" is 2.32.

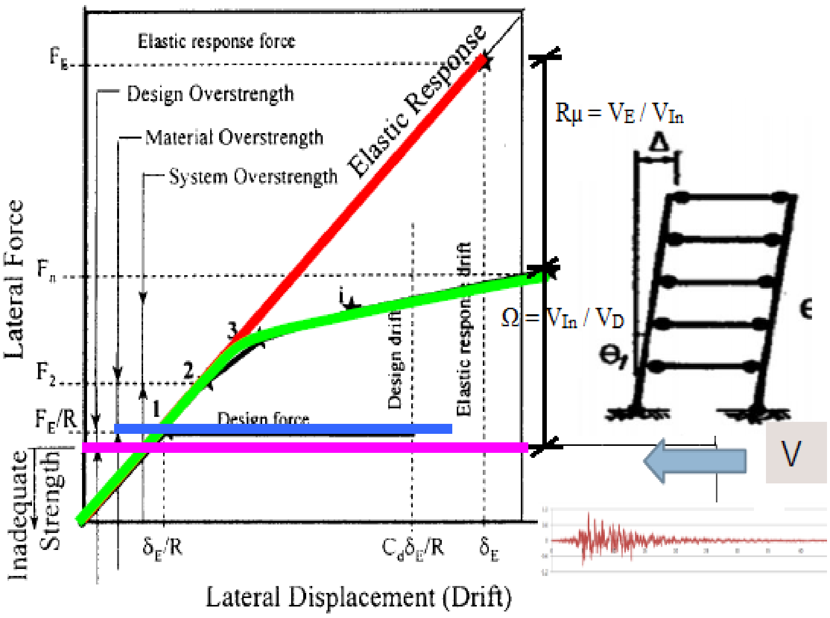


FIGURE 2.9: Elastic vs. Inelastic Structural Response (Uang, C. M., 1991).

2.5.1 Equivalent Static Analysis (ESA)

The simple equivalent static analysis (ESA) is the most common, useful and practical method in many of the codes for analysis of structures. The concept of static lateral force procedures is to apply static forces on a structure with magnitudes and direction that approximate the consequences of dynamic loading caused by ground shaking. UBC-97 permits design of a structure using equivalent static force procedure or a dynamic analysis for not more than 240 feet tall in case of regular structures and 65 feet tall in case of irregular structures. When the structure height exceeds the limit of 240 feet in case of regular structures, 65 feet in case of irregular structures and in case of buildings which are located on soil type-SF and having a time period more than 0.7 seconds, dynamic response spectrum analysis is required. The equivalent static force procedure is most commonly used for the case of regular structures. For irregular structures dynamic analysis must be adopted (Di Julio, R. M. 2001; ACl-318, 2011).

Bourahla (2013) explained equivalent static lateral force procedure as a simplified method. A static force distributed laterally on a structure for evaluation to account for the effect of dynamic loading of a predictable seismic event has been described. The total induced seismic pressure or impact V is generally assessed in two horizontal directions parallel to the main axes of the building. To obtain precise design of a structure, building should be symmetrical to avoid torsional effects and responds in its fundamental lateral mode.

2.5.2 Performance Based Seismic Design (PBSD) Approach

Performance-based seismic design (PBSD) practice has been started in earthquake engineering with the interrogation to design the building with known performance level in a seismic event. The main objective of PBSD is to design the building with identified probability of damage in case of earthquake. Outlined design philosophy has two main goals: suitably measuring the reservations related with the assessment of performance and reasonably portraying the related structural damage for direct integration into the plan or execution assessment system.

There are many restrictions and limitations in code-based design techniques while designing complex, high-rise and irregular buildings. In PBSD, probability of damage of a structure in case of seismic event could be analyzed. However, the classification of the several performance levels has headed to performance-based earthquake engineering; the most advanced procedure of seismic design and assessment.

THE ICC PC, (International Code Council, Performance Code, 2015) defines performance-based design as, "An engineering approach to design elements of a building based on agreed upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against the design goals and objectives using accepted engineering tools and methodologies".

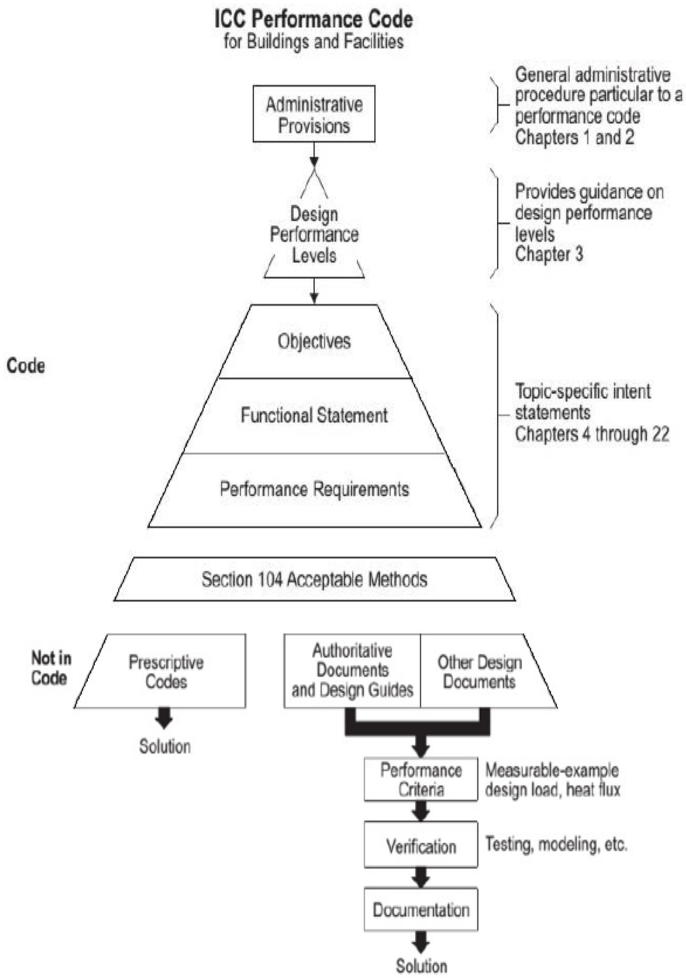


FIGURE 2.10: ICC performance code steps.

FEMA 356 and ATC 40 suggested four analysis procedures i.e. Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP). In this academic study only nonlinear static 'pushover' analyses, has been used to evaluate the inelastic seismic design. A performance objective has two components namely performance level and hazard level as described in Table 2.2 and Table 2.3 respectively. PBSD linked desired performance level to specified hazard level.

TABLE 2.2: Structural Performance level definition (FEMA 356; ATC 40; Antoniou 2002).

Performance level	**Description**
Operational	No substantial impact or destruction has happened to structural and non-structural elements. Building is appropriate for regular occupancy and usage.
Immediate-Occupancy	No substantial destruction has taken place and structure possesses almost all its pre-earthquake strength and stiffness Building is safe to occupy but possibly not good until cleanup and repair has performed.
Life-Safety	Notable damage to structural and non-structural elements could be occurred. Building is safe during seismic event but not afterwards. Occupancy may be prevented until repairs can be established.
Collapse Prevention	Remarkable damage. Structural strength and stiffness substantially degraded. Building is on verge of collapse, probable total loss.

TABLE 2.3: Structural Hazard level definition (FEMA 356; ATC 40).

Hazard Level	**Description**
Frequent, minor earthquakes (Service Level Earthquake)	100 yrs. (43% probability of occurance in 50 years)
Infrequent, moderate earthquakes (Design Basis Earthquake)	500 yrs. (10% probability of occurance in 50 years)
Worst earthquakes ever likely to occur (Maximum Considered Earthquake)	2,500 yrs. (2% probability of occurance in 50 years)

ATC-40, (1996) stated that assessment of seismic inelastic demand and seismic demand is required for PBSD procedure. The seismic capacity is the capacity of structure to resist the seismic effects; while, seismic demand is the earthquake effects imposed to the building. The structure must be designed for more seismic

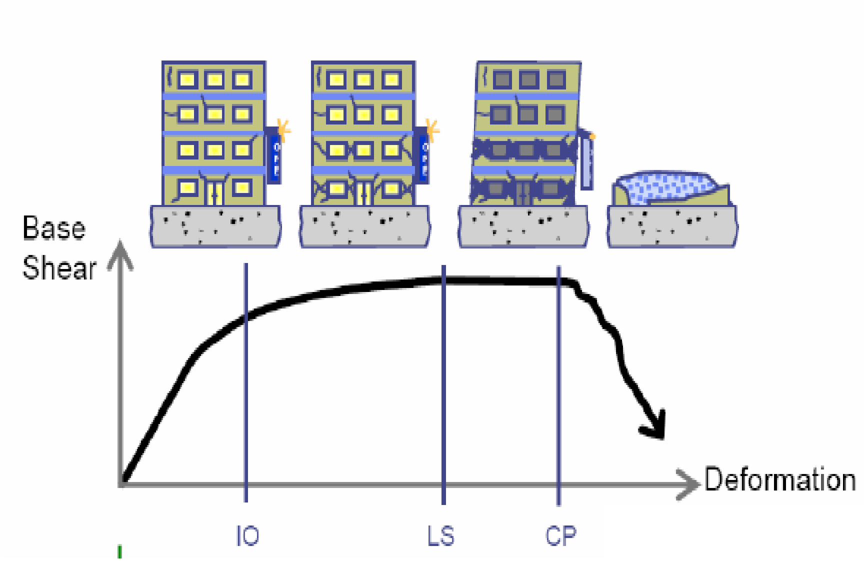


FIGURE 2.11: FEMA 273/356 performance levels.

inelastic demand than required by seismic demand. Association of a performance level, Table 2.2, to a hazard level, Table 2.3, results performance objective. The prime achievement of PBSD is to attain any desired performance objective when structure is subjected to any specific hazard level.

Goel, S. C., et al. (2010) illustrated that computation of base shear up to a particular or desire hazard level required a target displacement and yield mechanism. These two design parameters are directly related to the degree and distribution of structural damage respectively. The structure is then pushed as a whole up to the calculated target displacement based on work-energy balance principle (Zhang, Q. et al. 2017; Goel, S. C., et al. 2010). Plastic design is performed to detail the frame components and connections in order to achieve the targeted yield mechanism and behavior. This method is quite often useful for tall structures (Goel, S. C., et al., 2010 and Zhang, Q. et al. 2017). PBSD has been extensively recognized as an ideal method for use in the future for seismic design. (Wei, L., & Qing, Ning, L., 2012).

2.5.3 Static Push-over Analysis

Pushover analysis (PoA) is an advanced static nonlinear procedure for design and assessment of structures beyond their elastic limit. Push-over analysis as a combination of nonlinear static analysis and earthquake response spectrum and is engaged as a simple, but effective tool for the assessment of structural seismic inelastic demand (Ye, L., & Pan, W., 2000). This procedure was first adopted by Freeman et al. (1975), for seismically risky project of U. S. Navy, which is called as Inelastic demand Spectrum Method. Further, it was used as a relation between earthquake ground motion and building performance (Freeman, 1998 & ATC 1982). This method is then in 1986, being incorporated in the Tri-services Seismic Design Guidelines for Essential Buildings, as two-level approach to seismic design. The prime purpose of PoA is to judge performance of the building. It is a practical approach and being gradually put into practice around the world.

Pushover procedure is established on the assumption that structures lies primarily in the fundamental mode or in the lower modes of vibration during an earthquake. This leads to a decrease of the multi-degree-of-freedom (MDoF) systems, to an equivalent single-degree of- freedom (ESDoF) system, with properties forecasted by a nonlinear static analysis of the MDoF system. (Themelis, S., 2008).

Hinge definition in CSI SAP2000 define only the plastic behavior of the hinge. The elastic behavior of the frame element is determined by materail properties of the frame section assigned to the element. Thus, assignment of hinges to the frame elements does not changed the linear behavior of the structure. The main steps followed for static push over analysis are described in Figure 2.12.

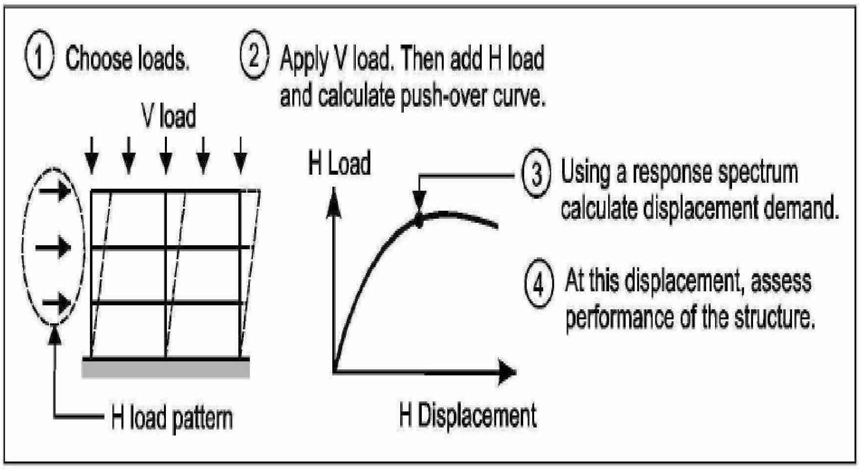


FIGURE 2.12: Steps for static push over analysis.

Response of nonlinear push-over procedures is represented by nonlinear load-deformation relations/ curve as shown in figure 2.13. The points categorized as A, B, C, D, and E on the push-over curve is used to express deflected actions of the hinge. Point A is unloaded position depicts linear response towards point B i.e. yielding condition. However, stiffness of structure is reduced after yielding, representing response from point B to C. Then there is sudden reduction in lateral load resistance to point D. After that from D to E is the response of the structure at decreased resistance and finally complete loss of resistance. The slope from point A to B shall be determined using linear elastic procedures. The slope from point B to C shall be taken between zero and 10% of the initial slope unless an alternate slope is justified by experiment or analysis. Other points labeled as IO, LS and CP is used to describe the performance of the hinge. The sections IO, LS and CP as shown in curve, defined Immediate Occupancy, Life Safety and Collapse Prevention level respectively.

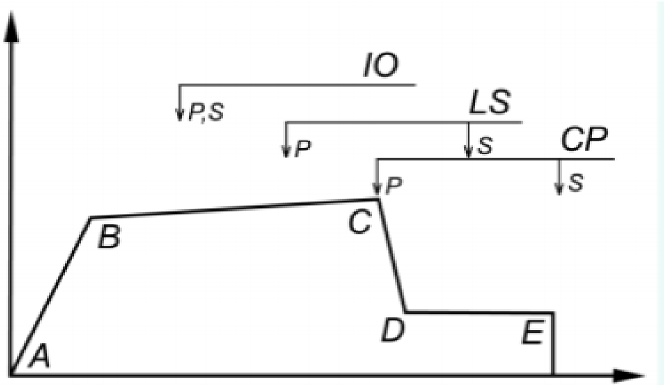


FIGURE 2.13: Displacement-Deformation curve for Pushover Hinges.

2.6 Summary

Behavior of RC structures and cracking phenomena of concrete under tensile loading has been discussed. Concrete being weak and brittle in tension does not take any tensile forces/ stresses and cracks even at service loads (ACI 318). Once cracking happened, reshuffling of moment occurred as a result of which moment shift toward uncracked section (Do Carmo and Lopes 2005 and Zhou and Zheng 2010). In consideration of this phenomenon, design of concrete using its full stiffness is uneconomical. Different researchers and codes stated modifiers for moment of inertia to be used in analysis. Recommendations about stiffness reduction are discussed in the light of different codes. For seismic assessment PBSD approach, structural performance and hazard levels and pushover analysis procedure used for this study is briefly presented. In subsequent chapter, a realistic 7-storied building is taken into account using ESA and FEMA non-linear PoA for two soil types SD and SB for different sets of stiffness modification as described in Table 1.1. Different seismic performance parameters have been evaluated and compared along with the load transfer mechanism and steel reinforcement. Based on results, most economical system of modification factors yet resulting defined seismic behavior is recommended.

Chapter 3

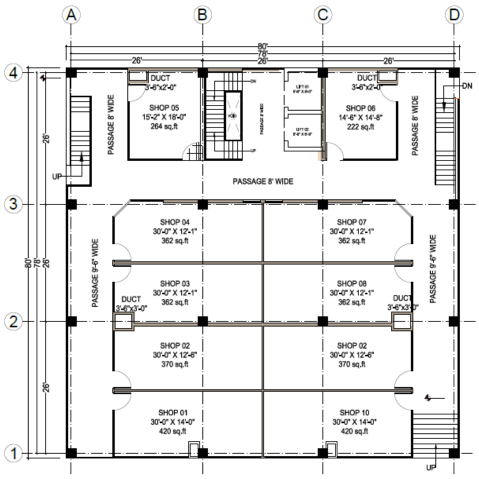
Modeling and Design of Case Study Building

3.1 Introduction

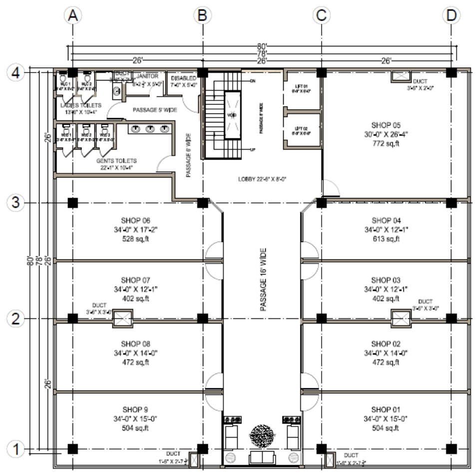
The current study is carried out to prescribe the performance of the structure considering concrete cracking behavior and stiffness reduction philosophy in RC structures. The design and evaluation of a realistic 7-storied building is taken into account using FEMA non-linear push-over analysis for two soil types SD (Stiff soil) and SB (Rock). Commercial Computer Structures International (CSI) Software SAP-2000 v15.0.0 has been used for the analysis. Equivalent linear static analysis and non-linear static pushover analyses have been performed. Different parameters i.e. elastic and inelastic base shear, storey shear force, storey moments, storey displacements and storey drifts has been compared to evaluate and recommend safe and economical option. Formation of plastic hinges from nonlinear static pushover analysis has also been compared. Comparison of steel reinforcement has also been made to recommend the economical option. This chapter briefly describes the methodology adopted for this study.

3.2 Description of the Building

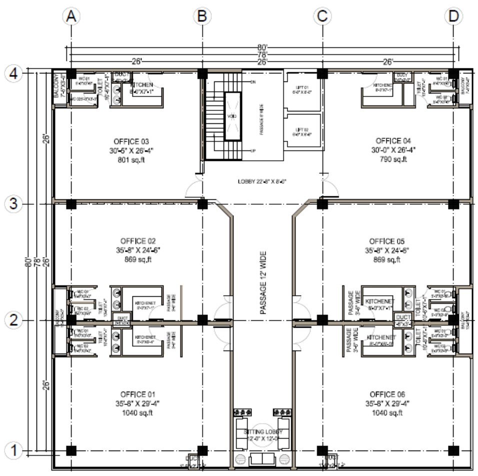
A realistic 7-storied building (Lower Ground, Ground Floor + five Floors) has been considered as a case study. The plane dimensions of the buildings are 80 feet in both length and width. The building is mix-used commercial building in which the lower ground, ground and 1st floors are to be used for commercial shops, 2nd and 3rd for offices and the remaining two stories are for residential apartments. The architectural plans at different levels of the building are shown in Figure 3.1 (a) to (d).



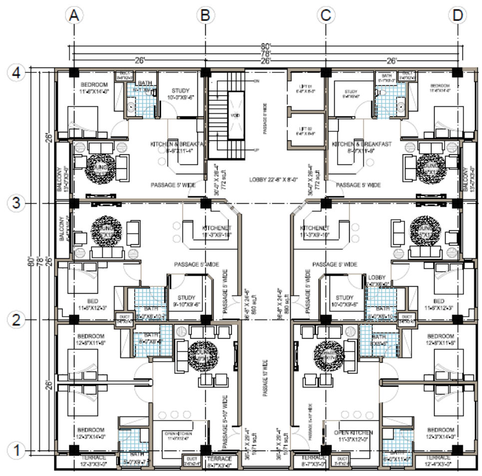
(a)



(b)



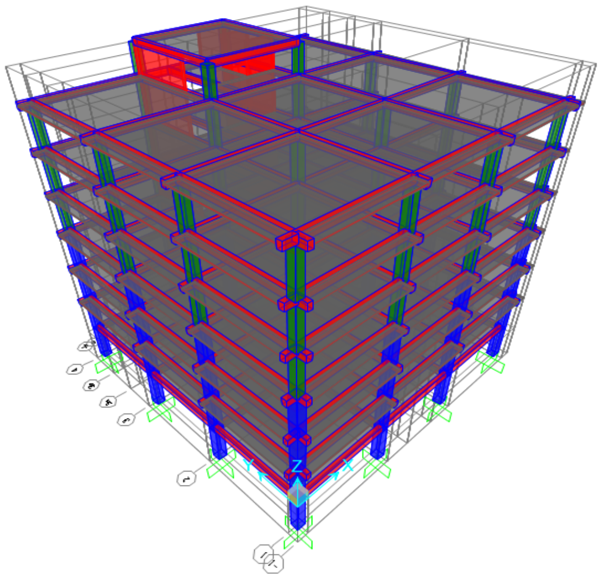
(c)



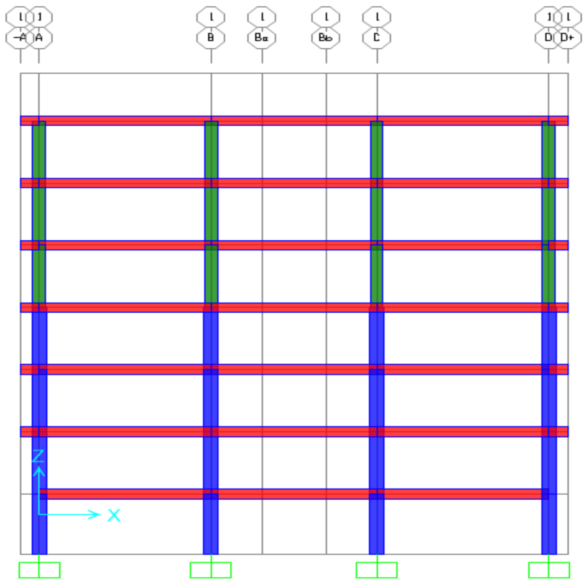
(d)

FIGURE 3.1: (a) Lower Ground and Ground Floor Plan, (b) First Floor Plan, (c) 2^{nd} and 3^{rd} Floor Plan, (d) 4^{th} and 5^{th} Floor Plan.

The grid spacing is 27 feet, 26 feet and 27 feet in both directions considering ease in construction. The case study structure is analyzed using UBC-97 design code for seismic zone-2B and soil profile type SD and SB. UBC-97 is used since it has been extensively used as a model code for seismic design of buildings. The elevation and 3D view of the building is shown in Figure 3.2.



(a)



(b)

FIGURE 3.2: (a) 3D view (b) elevation of the building.

Columns and beams have been modeled as frame elements, while slabs and walls have been modeled as shell elements. Concrete compressive strength has been taken as 3000 psi for beams and slabs, whereas 4000 psi for columns and walls. Slab thickness has been taken as 7.5 inches. Lateral force resisting system is dual comprising of "Structural frame system" and "Structural wall system" has been considered for the analysis. Sizes of frame elements are given in Table 3.1.

Table 3.1: Cross-sectional detail of frame elements of the structure.

Structural Member	**Member sizes**	**Floors to which assigned**
C1	33" x 33"	Lower ground and ground floor at grid 4B and 4C (at edge of wall)
C2	30" x 30"	Internal four columns of lower ground and ground floor
C3	27" x 27"	1st and 2nd floor along with external column of lower ground and ground floor
C4	24" x 24"	3rd, 4th and 5th
B1	15" x 24"	Ground, 1st, 2nd, 3rd, 4th, 5th and roof
B2	12" x 24"	Mumty

3.3 Stiffness Modeling

Stiffness of structural elements has been modeled as cracked and uncracked section properties according to different sets described below. Four different sets of stiffness modification factors (Ref: Table 1.1) are considered for comparison of seismic behavior of the structure are as follows:

1. Set-1: Code-based stiffness modification factors used for modeling of all frame and area elements.

2. Set-2: Slabs are modeled as uncracked elements using property modifiers for moment of inertia as 1. Other frame elements are as per code-based stiffness modification factors.

3. Set-3: In this case slabs and beams are modeled as uncracked elements while compression frame members and walls as per code modifiers.

4. Set-4: All the frame and area elements are modeled as uncracked elements using property modifiers for moment of inertia as 1.

Set-2 and Set-3 are hypothetical sets while Set-4 is being used in professional practice. Total sixteen models have been prepared, four with SD soil profile type and four with SB soil profile type using above mentioned four Sets of stiffness modification factors for equivalent static analysis and four for nonlinear static pushover analysis for both soil types as described in Table 1.1. All the analyses were performed using structural software SAP2000.

3.4 Equivalent Static Analysis (ESA)

The equivalent static lateral force method is a simplified procedure for seismic analysis of the building. For linear analysis ESA is being used extensively in civil engineering construction industry. The case study building is 1st mode dominant and there is not any type of irregularity found, thus building has been fulfilled the limitation for use of ESA and it can be performed for this building. Following gravity loading has been considered for gravity analysis and seismic mass calculation. Self-weight of the structure, load of 3-inch finishes and partitions wall loads is taken as dead load. For partition walls, load of 4-inch and 8-inch concrete block masonry has been considered and applied as per architectural drawings. According to UBC-97, live load for retail floors have been taken as 100 psf, 50 psf for offices floors, and 40 psf for apartment floors and roof. Building location is in the seismic zone 2B, whereas dual structural system is used, thus value of "R" factor is taken as 6.5 and importance factor value is taken as 1. Seismic coefficient

C_a and C_v are taken as 0.28 and 0.40 for soil profile type SD and 0.20 each for soil profile type SB. Seismic mass includes all the dead load of the structure i.e. self-weight, finishes load and partition load, and 25% for live load for retail floors in accordance with BCP-2007 section 5.30.1.1 and UBC-97 section 1630.1.1. The building is analyzed in both the X and Y-directions. Different load combinations and strength reduction factors given below and taken from UBC-97 are used in designing of the buildings.

(Ref. UBC 1997 & BCP 2007)

1. U = 1.4 D
2. U = 1.2 D + 1.6 LFLOOR + 0.5 LROOF
3. U = 1.2 D + (0.5 LFLOOR or 0.8 W) + 1.6 LROOF
4. U = 1.2 D + 1.3 W + 0.5 LFLOOR + 0.5 LROOF
5. U = 1.32 D + 1.1E + 0.55 LFLOOR
6. U = 0.99 D + 1.1E

Notation

U = Required ultimate strength for concrete structures to resist design loads or their related internal moments and forces, as defined in ACI 318-05.

D = Dead Load

L = Live Load

W = Wind Load

E = Earthquake Load

The time period from code-based procedure for different Sets of modification factors is given in Table 3.2. For both soil type soil type SD and SB time period comes out to be same for all the cases. This time period is supposed to be based on stiffness contribution from both structural and non-structural components (Williams A, 1997).

Table 3.2: Time period resulted from Equivalent Static Analysis.

S. No.	**Set of Stiffness Modification Factors used**				**Time Period in EX (sec)**		**Time Period in EY (sec)**	
	Column	**Wall**	**Beam**	**Slab**				
1	0.7	0.7	0.35	0.35	2.05	1.4 Method A	0.90	Method B
2	0.7	0.7	0.35	1	1.89	1.4 Method A	0.88	Method B
3	0.7	0.7	1	1	1.52	1.4 Method A	0.79	Method B
4	1	1	1	1	1.45	Method B	0.76	Method B

3.5 Preparation of Non-linear Models

The non-linear model has been prepared by inducing non-linearity at both ends of the beams, at bottom of the bottom story column and bottom story shear walls in accordance with the physical admissible plastic hinge mechanism.

3.5.1 Assignment of Plastic Hinges

In SAP-2000, the Axial P, Shear V2, Shear V3, Torsion T, Moment M2 and Moment M3 frame hinge types are all uncoupled and can be used independently. The interacting P-M2-M3 frame hinge type is a coupled hinge property. For beams Moment M3 auto hinge property integrated in SAP was assigned at both ends of each beam. The hinges are based on the X-section and reinforcement from equivalent static analysis, load combinations including the gravity and seismic moments and shears. Definition of hinge in SAP is shown in Figure 3.3. It can be seen from figure that auto hinge type is selected according to criteria described in FEMA 356, Table 6-7. Moment and rotation are displacement control parameters in hinge definition. Moment-rotation relationship for a typical hinge is shown in Figure 3.4. Acceptance criteria of FEMA 356 can be seen in auto hinge property data for IO, LS and CP levels. In manual hinge values of moment and rotation could be obtained by modeling exact reinforcement resulted in static analysis, whereas in auto hinge integrated in SAP hinges were taken positive and negative moment from results of static analysis of the structure as per FEAM criteria.

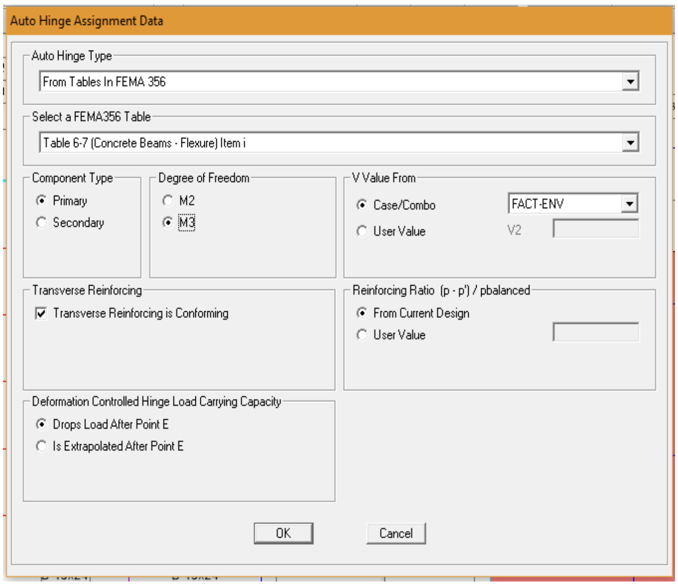


FIGURE 3.3: Definition of Moment M3 hinge for beams.

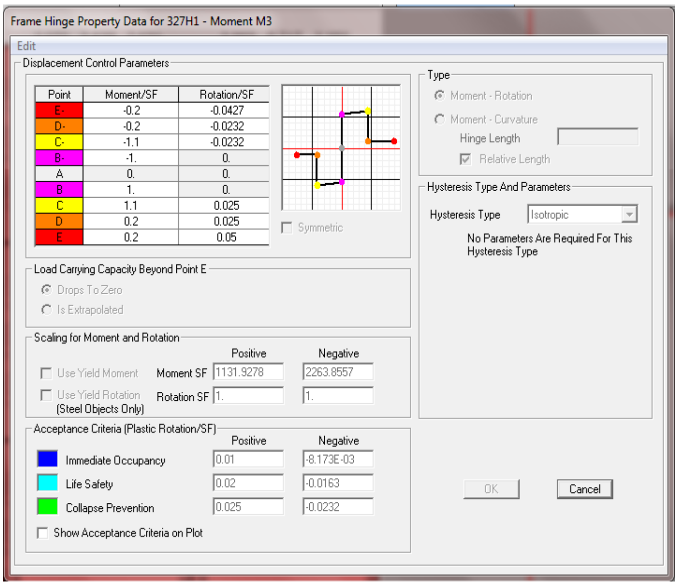


FIGURE 3.4: Auto Hinge Property data.

A typical hinge is selected and by comparing moments it was evaluated that hinge is taken approximately same moment resulted after static design. It can be seen from Figure 3.4 that hinge is taking negative moment of 2263 kip-in and positive moment of 1132 kip-in. Moment for resulted reinforcement is calculated using equation 3.1. Negative moment of 2252 kip-in while positive moment of 1245 kip-in has been resulted which shows that hinge is behaving correct and taken approximately same values of moment.

$$M = Asfy\left(d - \frac{Asfy}{0.85f'c2b}\right) \tag{3.1}$$

Where:

M = Resulted moment

As = Area of steel

fy = Yielding strength of steel

$f'c$ = Compressive strength of concrete

d = Depth of beam

b = Width of beam

For columns P-M2-M3 fiber hinges are defined for case study building as shown in Figure 3.5. In fiber hinge definition 6 concrete fibers have been defined in each hinge while steel fibers have been taken as per number of rebars resulted in static linear analysis of the structure. Fiber hinges have been assigned to the bottom of the bottom storey columns. Column hinge property data is shown in Figure 3.6 and column hinge moment curvature relationship is shown in Figure 3.7.

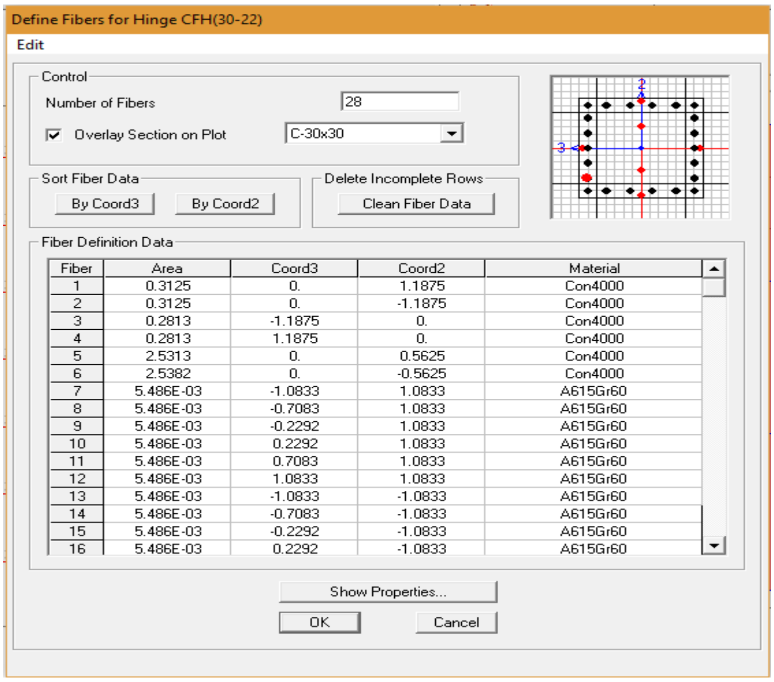


FIGURE 3.5: Definition of Moment P-M2-M3 fiber hinge for columns.

In FEMA 356, Table 6-8 modeling parameters and numerical acceptance criteria for nonlinear column hinges has been described. Force deformation curve of FEMA 356 is shown in Figure 2.13. Hinge is said to be in IO range if plastic rotation angle does not exceed 0.005 radians. While, hinge is said to be in LS and CP range if plastic rotation angle does not exceed 0.01 radians and 0.02 radians respectively. Acceptance criteria of FEMA 356 can be seen in Figure 3.6, column hinge property data for IO, LS and CP levels.

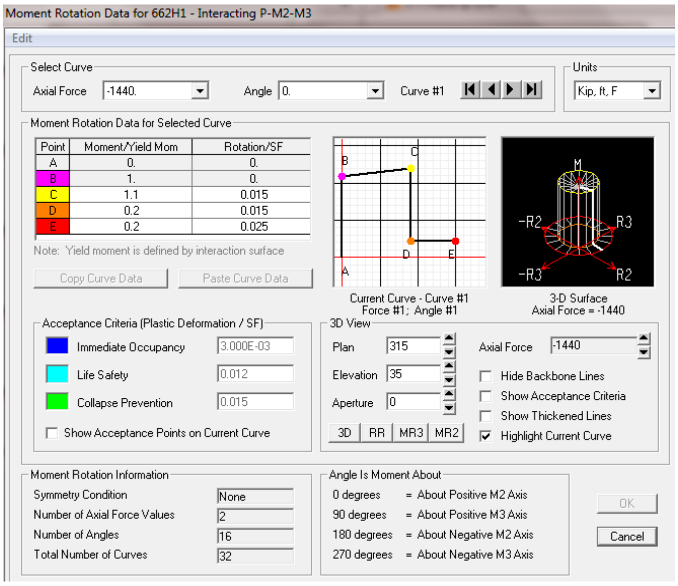


FIGURE 3.6: Column Hinge Property data.

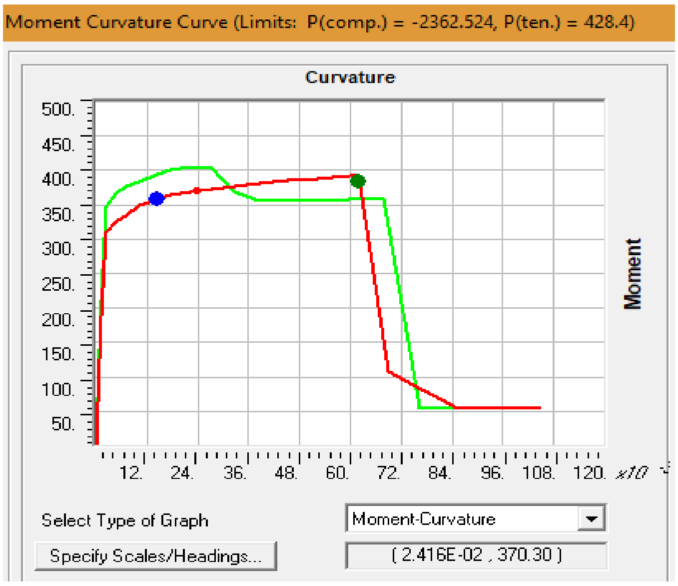


FIGURE 3.7: Column hinge moment-curvature relationship.

Nonlinear walls have been modeled as shell layered area elements in SAP2000. Longitudinal/ Vertical steel reinforcement (i.e. S22 in SAP2000) has been assigned as nonlinear as per number of rebars resulted in code-based design as shown in Figure 3.8. Figure 3.9 illustrates the definition of non-linear shell layered wall element. It can be seen that top and bottom bar 2M has been assigned as nonlinear.

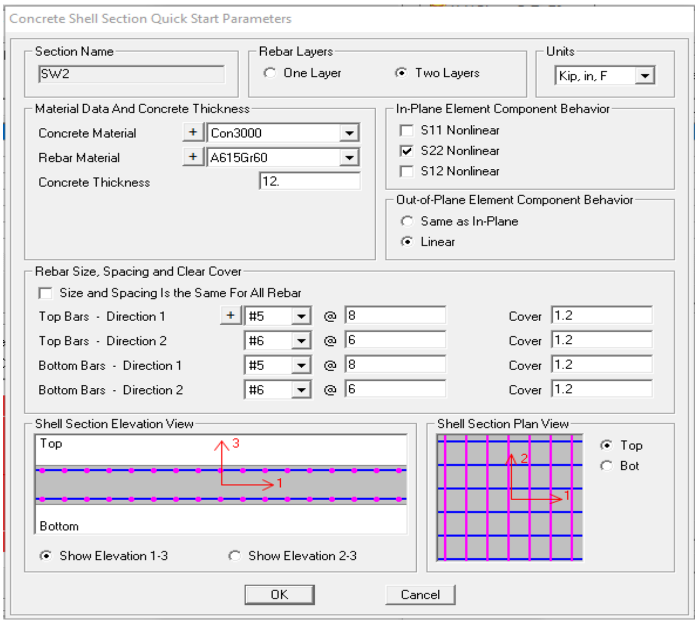


FIGURE 3.8: Non-linear wall definition.

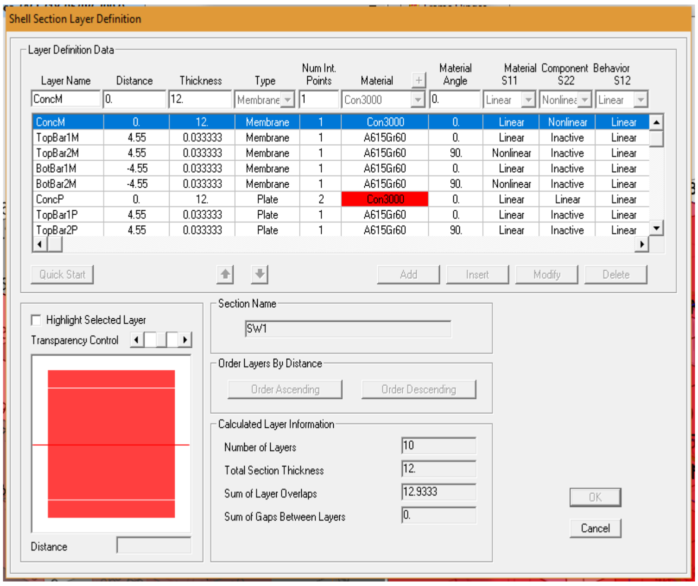


FIGURE 3.9: Non-linear shell layered section definition.

3.5.2 Plastic Hinge Length

Plastic hinge length needs to be defined for columns, beams and walls. Theoretical amounts for the equivalent plastic hinge length l_p based on incorporation of the curvature dispersal for typical compenents would make l_p directly proportional to l. Effective plastic hinge length may be obtained using following expression:

$$l_p = 0.08l + 0.15d_b f_y(ksi) \tag{3.2}$$

Where

l_p = Length of plastic hinge

d_b = Diameter of reinforcement bar

f_y = Yield strength of steel

A value of $l_p = 0.5h$, where h is depth of section could be utilized for typical beam and column proportion. A difference required to be made among the equivalent plastic hinge length l_p, defined above, and the area of plasticity over which special detailing requirements are needed to guarantee dependable inelastic rotation inelastic demand (Paulay, T., & Priestley, M. N., 1992). For the case study building relative length for hinge using #8 steel reinforcement bars is resulted as 0.1025 and for those elements using #6 steel reinforcement bars is resulted as 0.087.

3.6 Push-over Analysis (PoA)

Push-over analysis is non-linear static procedure as discussed in Chapter 2. Result of push over analysis is plot of the total base shear established against roof displacement is known as inelastic demand curve of a structure with respect to the roof displacement. This curve is shown in Figure 3.10. Behavior of structure beyond its elastic limit could be estimated from inelastic demand curve.

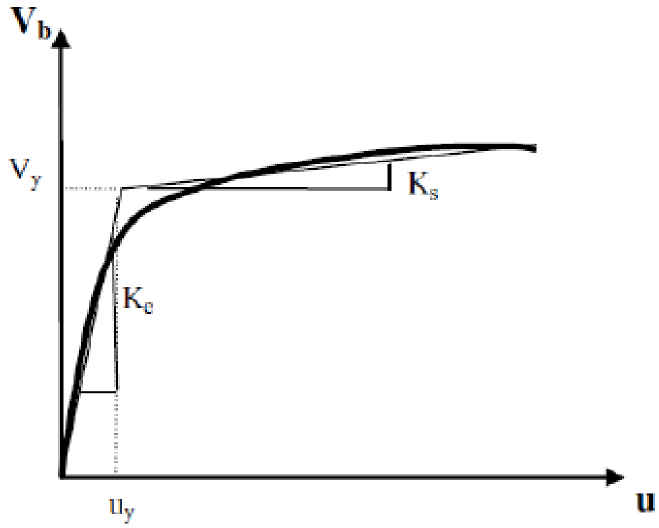


FIGURE 3.10: Inelastic demand curve (Themelis, S., 2008; M. Belgasmia, et al, 2014).

From two generally used approaches for PoA that are inelastic demand Spectrum Method (CSM) and Displacement Coefficient Method (DCM), the DCM is used for estimating nonlinear behavior of the building studied. There is required target displacement, for DCM method that can be calculated by Equation 3.3 (FEMA, 273).

$$\Delta t = \left(\frac{C_0 C_1 C_2 S_a T_e^2}{4\pi^2} \right) g \tag{3.3}$$

Where:

C_0 = Modification factor related to spectral displacement, probably roof displacement.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor represent the effect of hysteresis shape.

S_a = Response spectrum acceleration, at the fundamental time period and damping ratio of the building in the direction under consideration, "g".

T_e = Effective fundamental time period of the building, sec.

g = Acceleration of gravity. Its value is 9.8 m/s^2 (32.15 ft/s^2) on Earth.

In this method, structure is pushed horizontally in a certain direction with a pre-defined loading pattern up to a pre-defined target displacement. Plot of the total base shear versus roof displacement is then obtained. Pre-mature failure lines and weak connections are established, this phenomenon is called plastic hinge formation. (FEMA-273; ATC 40; M. Belgasmia et al, 2014). In this study, static non-linear push-over analysis has been performed using modeling guidelines of FEMA-273 and ATC 40. PoA is selected for non-linear static analysis as building is first mode dominant and credibility of PoA is established for static non-linear analysis for such buildings. Auto hinge property integrated in SAP-2000 was assigned to ends of the beams of each floor. Fiber hinges have been assigned to lower ends of bottom storey column. The hinges are based on the X-section and reinforcement obtained from the load combinations including the gravity and seismic moments and shears from equivalent static analysis.

Target displacement was calculated using the equation 3.1 of FEMA 273 (Eq. 3.3 described above). The values of effective time period Te, 1st mode displacement and modal participation factors are obtained from modal analysis using cracked section stiffness's of the structural members whereas, the values of spectral acceleration Sa are taken from response spectrum against the effective time period. For target displacement in Y-direction, time period of mode 3 of SAP2000 has been taken into account as 3rd mode has dominant mass participation in Y-direction Target displacements for each model for design basis earthquake level are given in Table 3.3 for soil all cases of soil type SD and in Table 3.4 for all cases of soil type SB.

Nonlinear static gravity load case has been first defined as 1.2Dead + Live$_{special}$ + 0.5Live. To simulate the directional effects nonlinear static Push-X case has been defined including 100% load in X-direction and 30% load in Y-direction, similarly Push-Y case with 100% load in Y-direction and 30% load in X-direction. Definition of push-X and push-Y cases in SAP2000 is shown in Figure 3.11. Figure 3.12 is

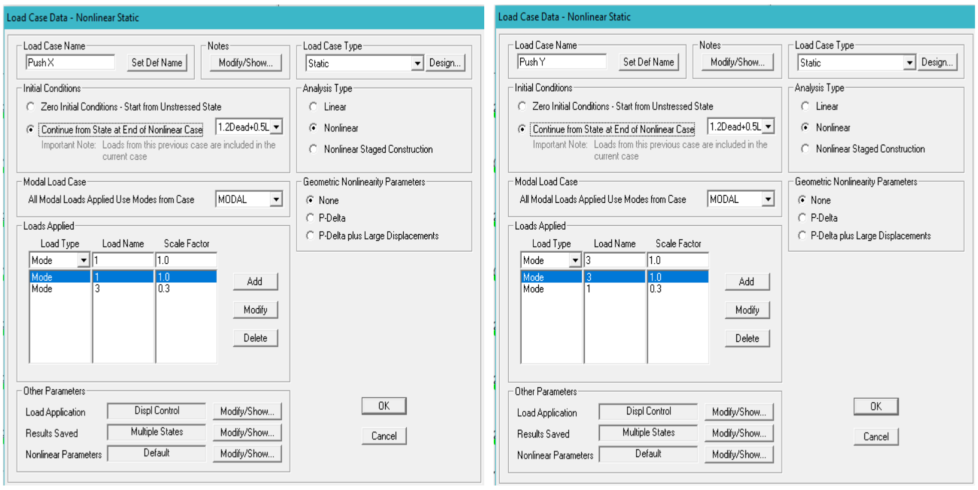
TABLE 3.3: Values of target displacement for PoA for soil type SD.

Values	**Set of Stiffness Modification Factors for soil type SD**							
	Set-1		**Set-2**		**Set-3**		**Set-4**	
	EX	EY	EX	EY	EX	EY	EX	EY
C_0	1.5		1.5		1.5		1.5	
C_1	1		1		1		1	
C_2	1		1		1		1	
S_a	0.197	0.4467	0.213	0.461	0.266	0.507	0.276	0.532
Time Period (sec)	2.04	0.89	1.89	0.87	1.51	0.79	1.45	0.754
Target Displacement (in)	12.03	5.2	11.13	5.13	8.92	4.65	8.5	4.45
Target Displacement (ft)	1.00	0.433	0.928	0.427	0.743	0.387	0.709	0.371

TABLE 3.4: Values of target displacement for PoA for soil type SB.

Values	**Set of Stiffness Modification Factors for soil type SD**							
	Set-1		**Set-2**		**Set-3**		**Set-4**	
	EX	EY	EX	EY	EX	EY	EX	EY
C_0	1.5		1.5		1.5		1.5	
C_1	1		1		1		1	
C_2	1		1		1		1	
S_a	0.098	0.224	0.106	0.233	0.266	0.507	0.139	0.268
Time Period (sec)	2.04	0.89	1.88	0.87	1.51	0.79	1.45	0.75
Target Displacement (in)	6.00	2.61	5.57	2.59	8.92	4.65	4.28	2.24
Target Displacement (ft)	0.50	0.217	0.464	0.216	0.743	0.388	0.357	0.186

symbolical presentation of distribution of loads over height. A modal load is a specialized type of loading used for pushover analysis. It is a pattern of forces on the joints that is proportional to the product of a specified mode shape times its circular frequency squared times the mass tributary to the joint. SAP mode 1 is dominant in X-direction and mode 3 participation in Y-direction is the maximum. This can also be observed in load cases definition. These load cases started from state at end of nonlinear case 1.2Dead + Live$_{\text{special}}$ +0.5Live. For load case in each direction 30% of perpendicular direction has been incorporated as per UBC-97 section 1633.1. UBC-97 section 1633.1 stated that "The requirements that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction". The non-linear load cases have been defined for each case and soil type SD and SB with the target displacement as described in above Tables. Once the inelastic demand curve is obtained, the performance of the structure was assessed by comparing the base shear, deflection, storey drift, and stages of number of hinges formed.



(a) (b)

FIGURE 3.11: Nonlinear load case definition (a) Push-X (b) Push-Y.

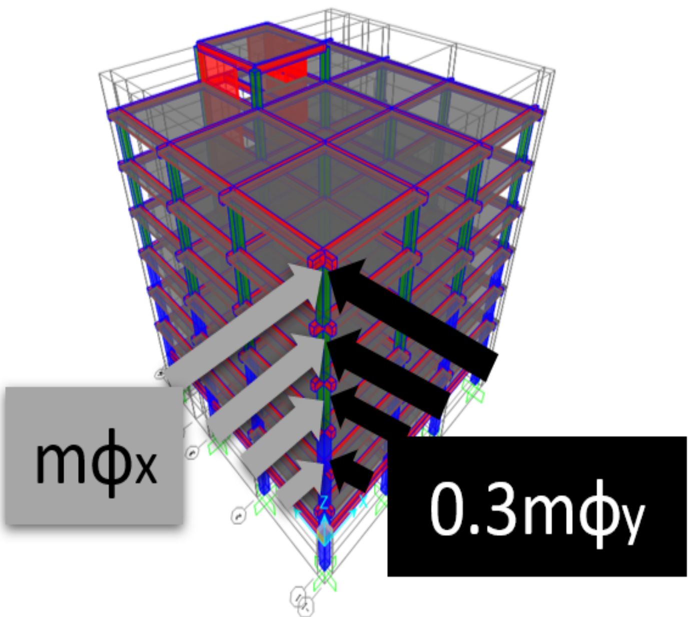


FIGURE 3.12: Symbolical presentation of load distribution over height.

3.7 Summary

The main objective of this study is to asses the seismic performance of RC building in case of any seismic event. To achieve the objective, linear and nonlinear analysis and design has been performed. All the required parameters for linear and nonlinear modeling in SAP-2000 has been described in this Chapter. Nonlinearity has been assigned at specified locations of the building. Moment M3 hinges have been assigned to both ends of each beam and P-M2-M3 fiber hinges to the bottom of columns of bottom storey. Non-linearity in walls has been introduced by assigning shell layered wall property. For non-linear analysis, nonlinear Push-X and Push-Y load cases have been defined using target displacement calculated as per FEMA-356 procedure. After analyses, the response of different parameters i.e. storey shear, overturning moment, storey displacement and drift have been compared for all the cases for soil type SD and SB for Push-X and Push-Y-directions and presented in Chapter 4.

Chapter 4

Results and Discussion

4.1 Introduction

In current study a realistic 7-storied building has been modeled and analyzed using elastic code-based technique and FEMA non-linear push-over analysis for two soil types SD (Stiff soil) and SB (Rock) as discussed in Chapter 3. Commercial Computer Structures International (CSI) Software SAP-2000 v15.0.0 has been used for the analysis. Equivalent linear static and non-linear static pushover analyses have been performed. Based on the results of static linear and static non-linear analysis, different responses i.e. elastic and inelastic base shear, storey shear force, storey moments, storey displacements and storey drifts have been compared to evaluate performance of the structure considering concrete cracking behavior. Further, changes in load transfer mechanism and formation of hinges by changing stiffness of the structure has been discussed. Comparison of steel reinforcement has also been made to predict effect of stiffness modifiers on cost of the structure. In subsequent sections, these parameters have been compared and discussed in detail.

4.2 Evaluation of Strength Reduction Factor

Inelastic seismic behavior of the structures during an earthquake is controlled by response modification factor known as "R" factor as described in Chapter 2. "R" factor is the ratio of elastic strength demand to design strength/base shear and product of "R_μ" and "Ω". Code-based "R" factor used for the current study has a value of 6.5 because of the dual frame structural system. Over strength factor "Ω" has been defined as 2.8 in UBC-97, thus code-based value of "R_μ" is 2.32. Both factors "R_μ" and "Ω" have been calculated for all the cases and presented in Table 4.1 and 4.2.

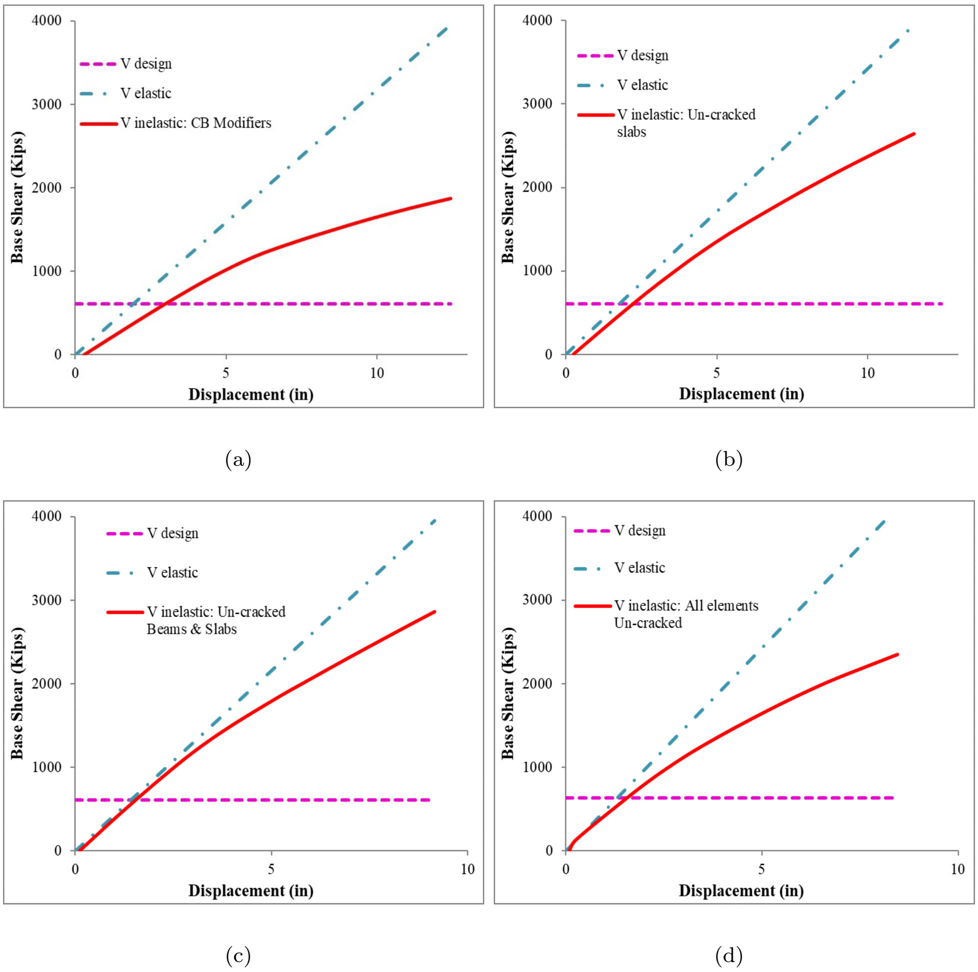


FIGURE 4.1: Comparison of V_{Design}, $V_{Elastic}$ and $V_{Inelastic}$ in push-X-direction for soil type SD (a) Code-based modifiers (b) Uncracked slabs case (c) Uncracked slabs and beams case (d) All elements uncracked case.

Elastic, inelastic and design base shears for push-X load case in the X-direction for all the cases of stiffness modification factors under study for soil type SD are shown in Figure 4.1 (a) to (d). Comparison of elastic, inelastic and design base shears in push-Y load case of soil type SD and in both directions of SB can be seen in Annexure-1. Elastic base shear as well as design base shear in the X-direction has approximately same value for all sets, whereas in case of Y-direction, $V_{Elastic}$ is increasing as the structural elements shifts towards uncracked sections except for the model with all elements uncracked. This trend results in more difference between the code-based "R" factor and modified "R" factor values as shown in the tables. $V_{Inelastic}$ has the same increasing pattern as that of $V_{Elastic}$.

Modified values of "R_μ" and "Ω" are given in Table 4.1 and 4.2. From tables, it can be seen for code-based modification case that the modified "R_μ" obtained from nonlinear analysis has been reduced a little bit as compared to that of code-based values, whereas over strength factor "Ω" has been increased in the same manner. This may be due to strain hardening effect of the rebar and effect of vertical seismic forces. Furthermore, results of "R_μ" and "Ω" depicts that as the stiffness of the elements of the structure is shifting from cracks towards uncracked, difference in code-based "R_μ" and "Ω" and modified values "$R_{\mu'}$" and "Ω'" is increasing. Modified values "$R_{\mu'}$" has been decreased by 9%, 35%, 40% and 25% in case of Set-1, Set-2, Set-3 and Set-4 as compared to that of code-based values in case of soil type SD. For soil type SB modified values "$R_{\mu'}$" has been decreased by 20%, 36%, 24% and 44% in case of Set-1, Set-2, Set-3 and Set-4 as compared to that of code-based values. Modified values "Ω'" has been increased by 9%, 35%, 40% and 25% in case of Set-1, Set-2, Set-3 and Set-4 as compared to that of code-based values in case of soil type SD. For soil type SB modified values "Ω'" has been increased by 20%, 36%, 24% and 44% in case of Set-1, Set-2, Set-3 and Set-4 as compared to that of code-based values. This may be due to more strength (more reinforcement) and stiffness of the cases. Hence, resulting more stiffer and less ductile system.

TABLE 4.1: Code-based and modified "R" values for soil type SD.

Model	**Code-based** R_μ/Ω	$R_{\mu'}$ **in push X**/Ω'	$R_{\mu'}$ **in push Y**/Ω'
Code-based Modifiers	2.32 / 2.8	2.11 / 3.08	2.82 / 2.30
Uncracked slabs case	2.32 / 2.8	1.50 / 4.33	2.38 / 2.73
Uncracked slabs and beams case	2.32 / 2.8	1.38 / 4.71	2.22 / 2.93
All elements uncracked case	2.32 / 2.8	1.75 / 3.71	2.47 / 2.63

TABLE 4.2: Code-based and modified "R" values for soil type SB.

Model	**Code-based** R_μ/Ω	$R_{\mu'}$ **in push X**/Ω'	$R_{\mu'}$ **in push Y**/Ω'
Code-based Modifiers	2.32 / 2.8	1.85 / 3.51	2.19 / 2.97
Uncracked slabs case	2.32 / 2.8	1.49 / 4.36	1.94 / 3.35
Uncracked slabs and beams case	2.32 / 2.8	1.77 / 3.67	1.09 / 5.96
All elements uncracked case	2.32 / 2.8	1.30 / 5.0	1.92 / 3.38

Resulted"R_μ" and "Ω" in PoA in Set-1 are comparable with code-based "R_μ" and "Ω" suggesting the correlation of SMFs with code-based "R_μ" and "Ω".

4.3 Storey Shear

In any seismic event, reactive forces are generated at different levels i.e. floors of the building because of the application of lateral loads and these forces are known as storey shears. These forces vary from floor to floor along the height of the building because of different stiffnesses and masses in the storey levels. Storey shear force is cumulative of lateral forces from top of the building towards bottom and results maximum value at base, equal to base shear. The values of storey shears for linear equivalent static case (EX and EY) and non-linear static (Push-X) and (Push-Y) cases for all the Sets of SMFs and for both soil types has been compared. Graphical presentation of storey shears resulted from linear and nonlinear analysis in X-direction for soil type SB is given in Figure 4.2. Graphical presentation of storey shears in Y-direction of soil type SB and both directions of soil type SD has been given in Annexure-2.

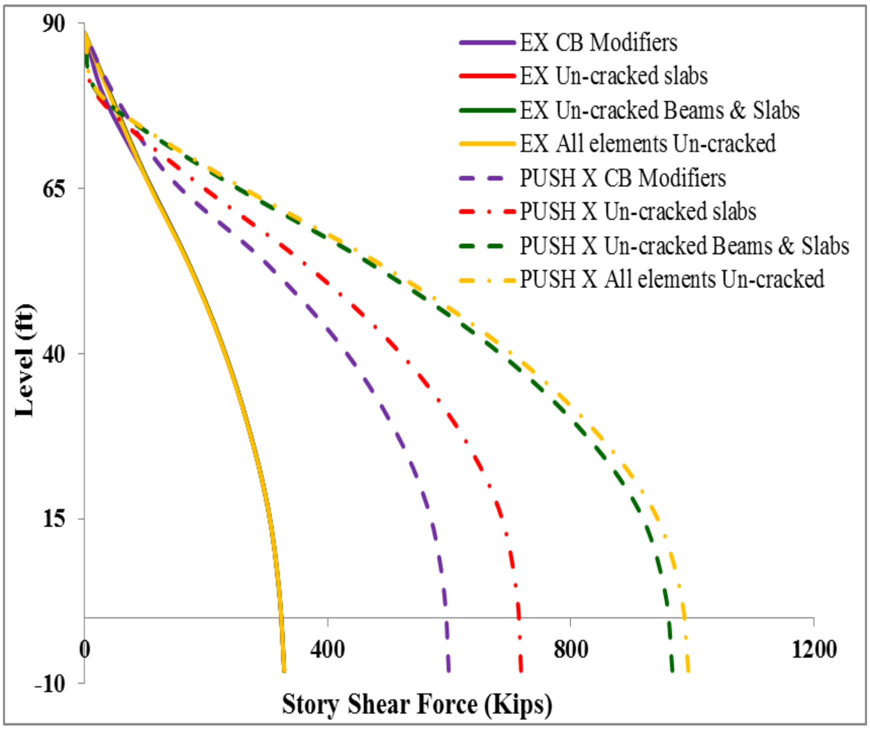


FIGURE 4.2: Storey Shears comparison for EX and Push-X case for soil type ‘SB’.

For ESA, storey shear for all the cases is approximately same. Significant difference in values of storey shear forces between linear static and non-linear static cases in both directions for both soil types for all sets has been observed. Inelastic storey shear has been increased significantly as compared to that of equivalent static storey shear. Percentage increase/ decrease in storey shears in Set-2, 3 & 4 w.r.t Set-1 resulted in Push-over analysis has been presented in Table 4.3.

TABLE 4.3: Percentage increase/ decrease in storey shears in Set-2, 3 & 4 w.r.t Set-1 for Push-over analysis.

Model	**SD X-direction**	**SD Y-direction**	**SB X-direction**	**SB Y-direction**
Set-2	27%	18%	17%	14%
Set3	41%	28%	38%	26%
Set-4	37%	27%	40%	26%

The difference between static linear and non-linear analysis in case of code-based modification factors is less as compared to all other systems. The storey shear inelastic demand is enhanced by 27%, 41% and 37% in case of Set-2, Set-3 and Set-4 respectively as compared to that of code-based modification factors case in Push-X-direction for soil type SD. Whereas, the storey shear inelastic demand is increased by 18%, 28% and 27% in case of Set-2, Set-3 and Set-4 respectively as compared to that of code-based modification factors case in Push-Y-direction for soil type SD.

Further, the inelastic shear demand is enhanced by 17%, 38% and 40% for Set-2, Set-3 and Set-4 respectively as compared to that of code-based modification factors case in Push-X-direction for soil type SB. Whereas, the storey shear inelastic demand is increased by 14%, 26% and 26% for Set-2, Set-3 and Set-4 respectively as compared to that of code-based modification factors case in Push-Y-direction for soil type SB.

It can be observed from Table 3.3 and 3.4 that values of time period and target displacement are reducing as structural model shifts towards uncracked member stiffness. Whereas, the shear inelastic demand determined through pushover analysis is increasing as structural model shifts towards uncracked member. This may be due to more flexural elastic demand (more designed reinforcement) provided in beams for uncracked models.

4.4 Over Turning Moment

Overturning moment of a storey is defined as the cumulative product of lateral forces and moment arm up to that storey level. Overturning moment for linear static cases (EX and EY) and non-linear static push-over cases in the X and Y directions for all the sets of stiffness modification factors for soil type SB is shown in Figure 4.3. Graphical presentation of overturning moments in Y-direction of soil type SD and both directions of soil type SB has been given in Annexure-3. The overturning moment is approximately same for all cases for equivalent static case in both directions for both the soil types. Substantial difference can be seen in the values of overturning moment between linear static and non-linear static analysis. There is significant increase in inelastic overturning moment as compared to that of equivalent static case. Percentage increase/ decrease in storey shears in Set-2, 3 & 4 w.r.t Set-1 resulted in Push-over analysis has been presented in Table 4.4.

The variation between equivalent static and non-linear static push-over case is significant in X-direction. This difference is substantially increased as uncracked elements increased in the model. The overturning moment inelastic demand is enhanced by 12%, 32% and 25% in Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in Push-X direction for soil type SD. Whereas, overturning moment in Y-direction in case of static push-over analysis results exactly same for Set-1 and Set-2. For Set-3 ans Set-4 moment demand is has been increased by 16% and 30% respectively.

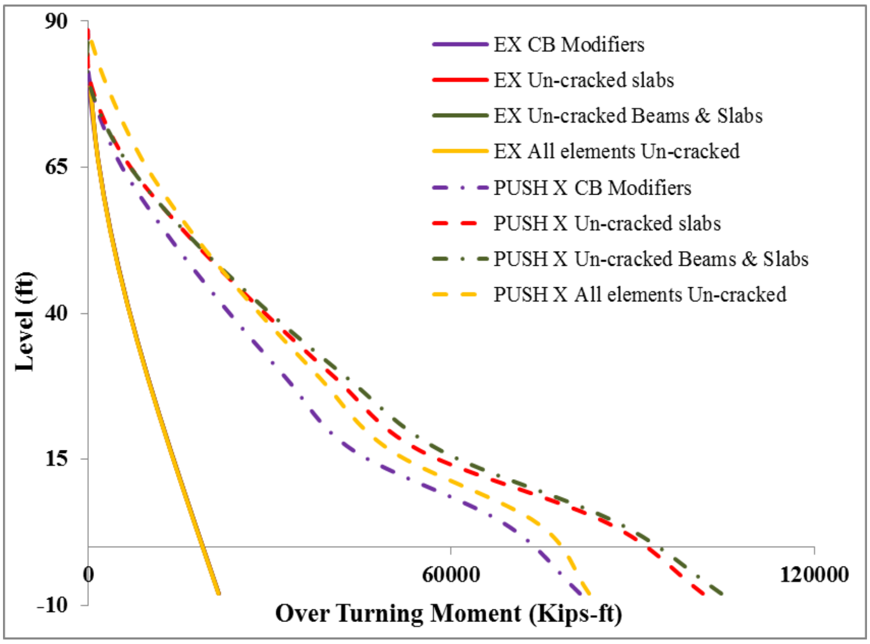


FIGURE 4.3: Over turning moment comparison for EX and Push-X case for soil type 'SB'.

TABLE 4.4: Percentage increase/ decrease in overturning moments in Set-2, 3 & 4 w.r.t Set-1 for Push-over analysis.

Model	**SD X-direction**	**SD Y-direction**	**SB X-direction**	**SB Y-direction**
Set-2	12%	00%	20%	04%
Set3	32%	16%	22%	-22%
Set-4	25%	30%	20%	15%

Moreover, increase in the overturning moment inelastic demand in the Push-X-direction for soil type SB is 20%, 22% and 20% in case of Set-2, Set-3 and Set-4 respectively as compared to that of Set-1. The overturning moment in the Y-direction in case of static push-over analysis is increased by 4% and 15% in Set-2 and Set-4 as compared to Set-1. In Set-3 moment demand is reduced by 22% with respect to moment demand in Set-1. This could be due to yielding of wall segments in this case unlike set-2 & 4.

Shear inelastic demand is associated with flexure inelastic demand and as result of increase in flexure demand, shear demand shall also be increased. As a result of more demand, more capacity requirement (i.e. more reinforcement) will be required. This increased demand will have great effect on elements and actions that intended to be elastic such as beams in terms of shear, columns above the bottom storey in terms of shear and development of undesired hinges and foundation in terms of reinforcement and stability (bearing pressure).

4.5 Storey Displacement

Storey displacement is the lateral displacement of the storey relative to the base. Figure 4.4 is representation of storey displacement for linear equivalent static case (EX and EY) and non-linear static (Push-X) and (Push-Y) cases for all the Sets of stiffness modification under consideration for soil type SD. Percentage increase/ decrease in storey displacements in Set-2, 3 & 4 w.r.t Set-1 resulted in Push-over analysis has been presented in Table 4.5. Graphical presentation of storey displacement for other direction and other soil can be seen in Annexure-4. In static linear analysis reduction in storey displacement has been observed as uncracked elements increased in structure. The less storey displacement in these cases for both soil types SD and SB has been observed. This is obvious due to stiffer model in these cases. The elastic displacement (from equivalent lateral force without "R" factor) and inelastic displacement from Push over Analysis (Target displacement) are little bit different only because they are evaluated by different methods. In case of storey displacement in the EY-direction for both soil type SD and SB, displacement in the EY-direction are same for all cases due to the same time period originating from code-based limitation. For the same reason, displacement from Push-Y case (Target displacement) is almost same for all cases, however, this is significantly larger than the EY-direction. This difference is attributed to rigid body displacement originating from yielding of the wall.

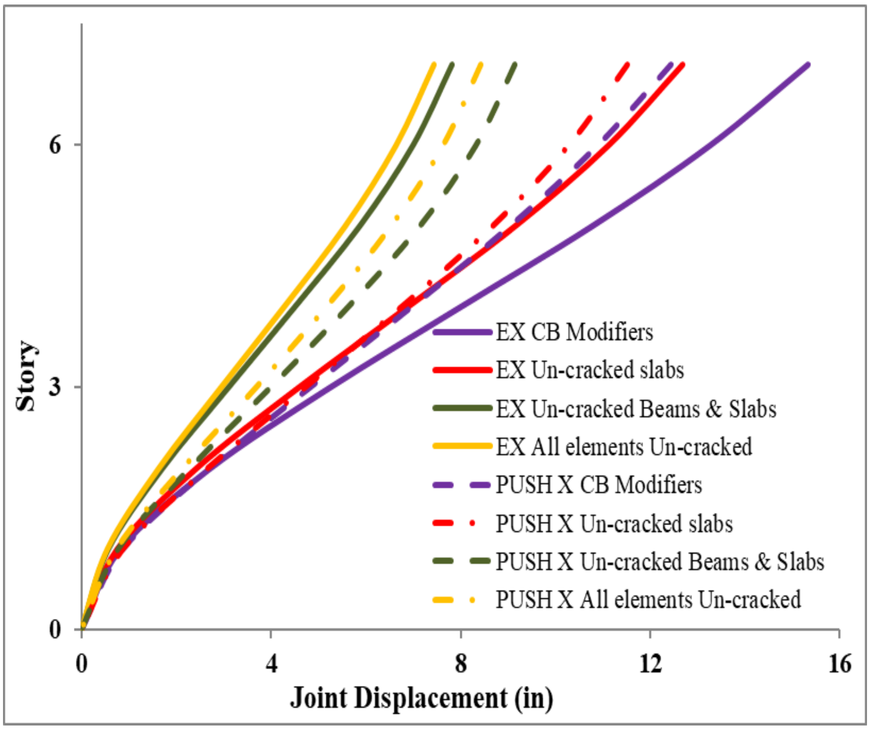


FIGURE 4.4: Storey displacements comparison for EX and Push-X case for soil type 'SD'.

TABLE 4.5: Percentage increase/ decrease in storey displacements in Set-2, 3 & 4 w.r.t Set-1 for Push-over analysis.

Model	**SD X-direction**	**SD Y-direction**	**SB X-direction**	**SB Y-direction**
Set-2	-08%	-05%	-16%	00%
Set3	-25%	-16%	-37%	-26%
Set-4	-32%	-21%	-31%	-40%

It can be observed from Table 4.5 that the inelastic storey displacement has been reduced as model shift towards uncracked elements. Storey displacement is decreased by 8%, 25% and 32% in Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in Push-X direction for soil type SD. Whereas, storey displacement is reducind by 5%, 16% and 21% in case of Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in Push-Y-direction for soil type SD. In the case of EX and EY-direction for soil type SB, results of storey displacement have been showed same trend as observed in soil type SD discussed above. As compared to that of Set-1 in Push-X-direction for soil type SB, storey displacement has been reduced by 16%, 37% and 31% in case of Set-2, Set-3 and Set-4 repectively. Whereas, in Y-direction, storey displacement results are exactly same as for Set-1 and Set-2. For Set-3 and Set-4 storey displacement has been reduced by 26% and 40% respectively. The reason of reduced displacement is more stiffness of the structural elements.

4.6 Storey Drift

Storey drift is usually interpreted as inter-storey drift - the lateral displacement of one floor level relative to the other level above or below. It is defined as ratio of displacement of two consecutive floors up to height of that floor. Storey drift for both soil types and for all the models of stiffness variations under consideration is compared and shown in Figure 4.5 for X-direction od soil type SD. Comparison for Y-direction of soi type SD and both directions of soil type SB has been presented in Annexure-5. Storey drift variation along height and cases shows the same trend as that of displacements.

For both soil types for the EX case, it can be noticed that, there is 25% increment in storey drift in case of Set-1 as compared to Set-2. For Set-3 and Set-4 storey drift is significantly less than that of first two cases for both soil types. In case of storey drift in the Y-direction, storey drift has same values for Set-1 and Set-2. Whereas, storey drift is decreased by 8% and 18% in Set-3 and Set-4 respectively as compared to that of Set-1.

Percentage increase/ decrease in storey drifts in Set-2, 3 & 4 w.r.t Set-1 resulted in Push-over analysis has been presented in Table 4.6. The inelastic storey drift ratio is decreasing as model shift towards uncracked structural elements. Storey drift has been decreased by 20%, 47% and 49% in Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in Push-X direction for soil type SD. Whereas, storey drift has been reduced by 5%, 21% and 31% in case of Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in the Push-Y-direction for soil type SD.

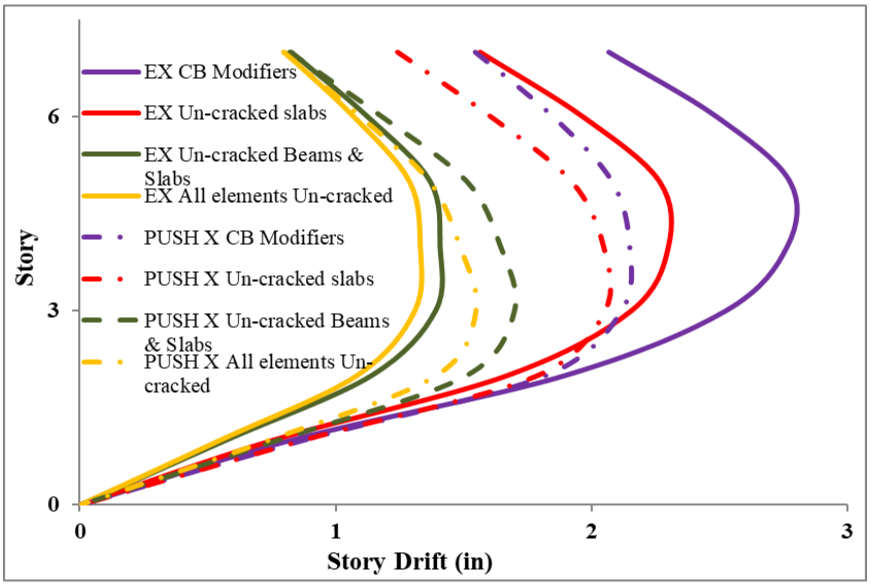


FIGURE 4.5: Storey drift comparison for EX and Push-X case for soil type 'SD'.

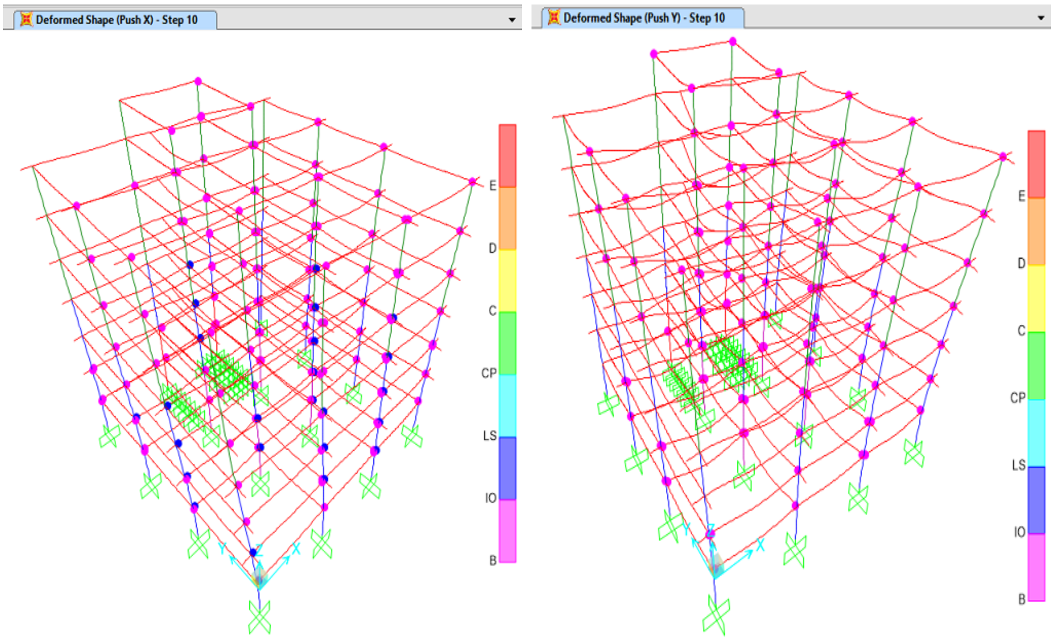
TABLE 4.6: Percentage increase/ decrease in storey drifts in Set-2, 3 & 4 w.r.t Set-1 for Push-over analysis.

Model	**SD X-direction**	**SD Y-direction**	**SB X-direction**	**SB Y-direction**
Set-2	-20%	-05%	-16%	00%
Set3	-47%	-21%	-48%	-25%
Set-4	-49%	-31%	-33%	-46%

For soil type SB, inelastic storey drift ratio has been decreased by 16%, 48% and 33% in Set-2, Set-3 and Set-4 respectively as compared to that of Set-1 in the Push-X direction. Whereas, in the Y-direction, storey displacement results are approximately same for Set-1 and Set-2. While reduction in storey displacement by 25% and 46% can be observed in Set-3 and Set-4 with respect to Set-1.

4.7 Formation of Plastic Hinges in Beams in PoA

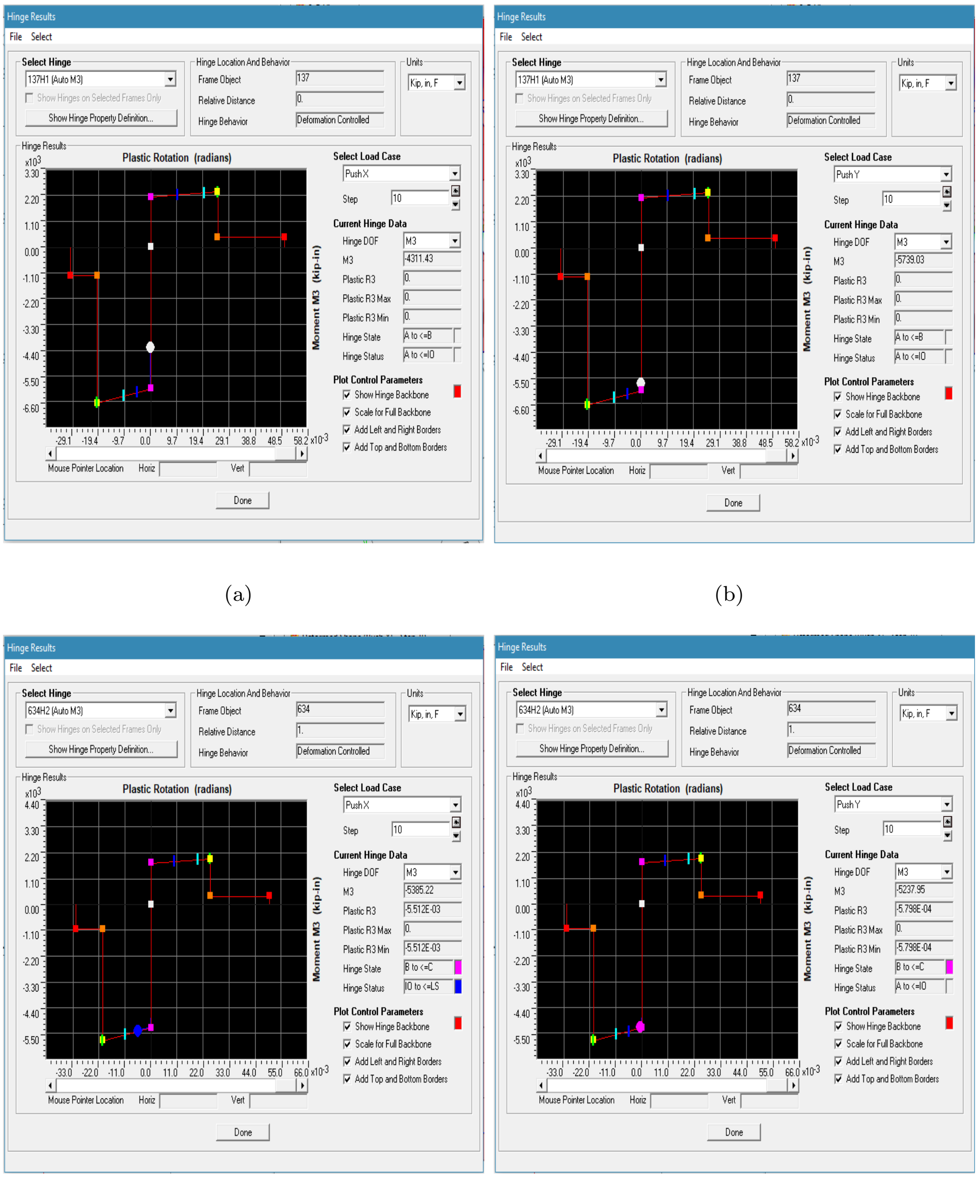
Figure 4.6 show the formation of plastic hinges in nonlinear static PoA for Set-1 in both push-X and push-Y-direction for the soil type SD. For both the soil types, formation of hinges in beams can be seen at all levels. In Push-X-direction, most of the hinges yield up to immediate occupancy level, however, some of the hinges in push-X case for soil type SD yield up to the life safety level as per criteria described in FEMA, 356, Table 6-7. Whereas in Push-Y-direction, all the hinges have been yielded up to immediate occupancy level. Formation of plastic hinges in nonlinear static PoA for Set-2, 3 and 4 in both Push-X and Push-Y-direction for both the soil type SD and SB has been presented in Annexure-6.



(a) (b)

FIGURE 4.6: Formation of plastic hinges in Set-1 code-based stiffness modifiers (a) Push-X (b) Push-Y for soil 'SD'.

Moment rotation curve resulted for a typical hinges is shown in Figure 4.7 for both Push X and Push Y-directions. Figure 4.7 (a) and (b) shows the result of a typical hinges which were remained in the elastic range. The moment rotation relationship of a representative hinges yielded beyond the elastic range is shown in Figure 4.7 (c) and (d) for Push-X and Y-direction respectively. The hinge in the Push-X-direction yielded up to the life safety region with the rotation of 0.0055 radian, whereas hinge for the Push-Y-direction lies in B to C range (up to immediate occupancy state) with the rotation of 0.00058 radian.



(a) (b) (c) (d)

FIGURE 4.7: Typical beam hinges results in Push X and Push Y.

Total number of plastic hinges observed in Push-X-direction for all the stiffness combinations models under study is shown in Figure 4.8. Total number of beams in each case is 179. Hinges have been assigned on both ends of each beam and thus total numbers of hinges are 358. Evaluation of hinges as per FEMA 356 displacement-deformation curve and rotation limits for different states shows that the most of the hinges for all the cases remain in elastic range A to B and B to C (but before immediate occupancy level). Only 25, 3, 6 and 2 hinges went beyond life safety region for Set-1, 2, 3 and 4 respectively for soil type SD. For soil type SB only for Set-3, 10 hinges undergo beyond their elastic range up to life safety region as defined in FEMA 356. Rotation of the hinges yielded up to the life safety level for both soil types is shown in Figure 4.9. It can be noticed that rotation of hinges remains less than 0.01 radian, hence remained in life safety region as per criteria described in FEMA, 356, Table 6-7.

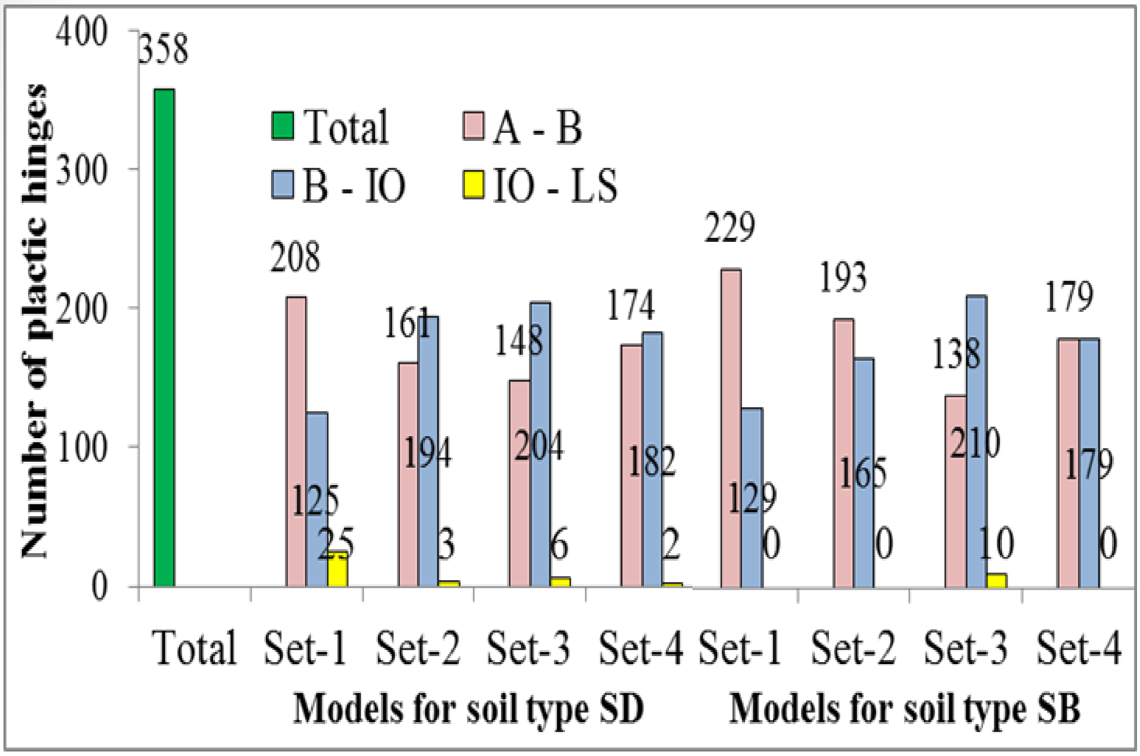
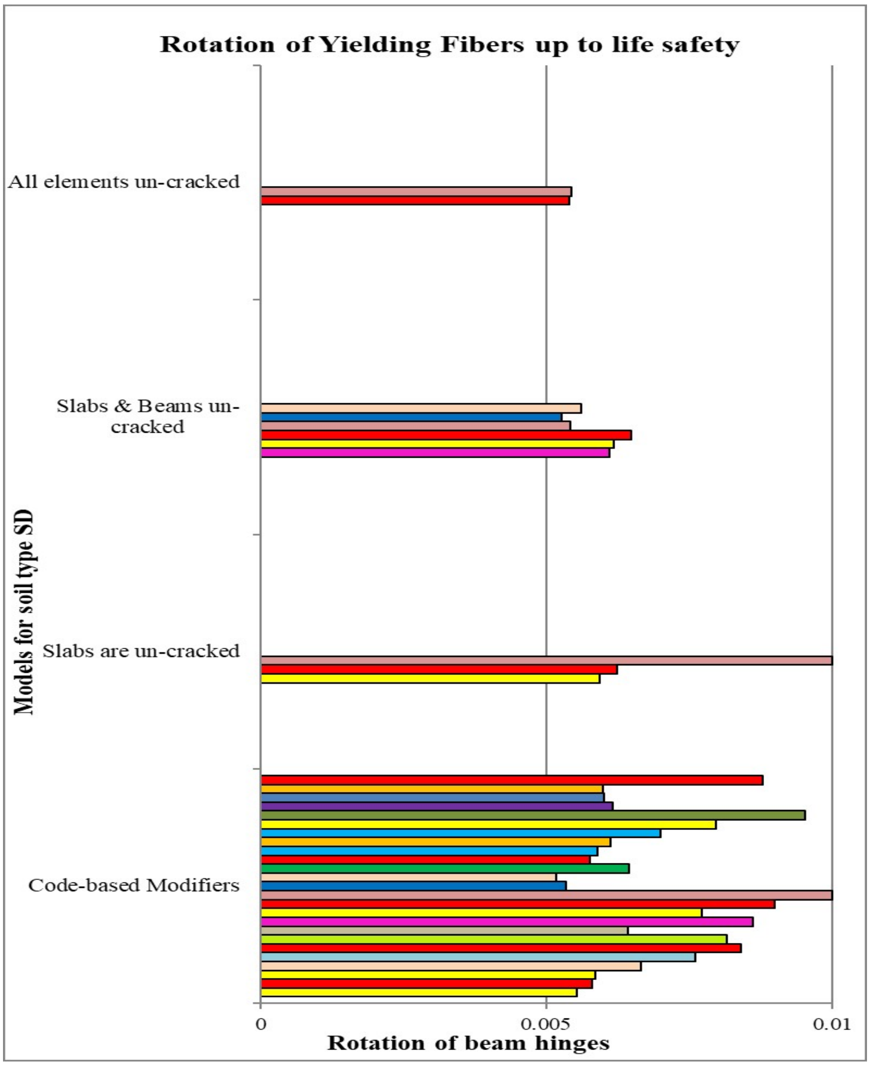
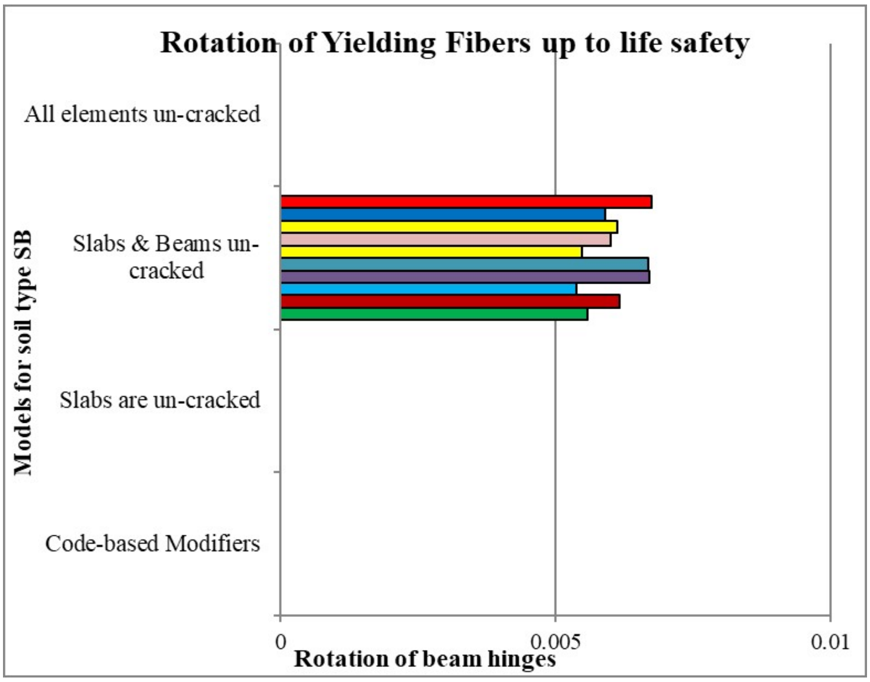


Figure 4.8: Plastic Hinge Formation in Push-X-direction for soil type SD and SB.



(a)



(b)

FIGURE 4.9: Rotation of Yielding Fibers up to life safety (a) Push-X-direction for soil type SD (b) Push-X-direction for soil type SB.

Plastic hinge formation for both soil types in Push-Y-direction for all the stiffness combinations models under study is shown in Figure 4.10. The trend of hinges formation is somehow similar to the Push-X-direction. For all the cases, number of hinges remained in elastic range is more than that of Push-X-direction. For soil type SD in Set1, Set2, Set3 and Set4, out of 358 hinges 109, 99, 106 and 112 hinges fall in B to IO range respectively. Only two hinges go up to life safety levels in Set-2. Rotation of these 2 hinges remained less than 0.01 radian as shown in Figure 4.11. In soil type SB number of hinges remain in B to IO range is 85, 114, 131 and 113 in Set1, Set2, Set3 and Set4 respectively. For soil type SB in push-Y-direction not any single hinge yielded up to life safety level as per criteria defined in FEMA, 356, Table 6-7.

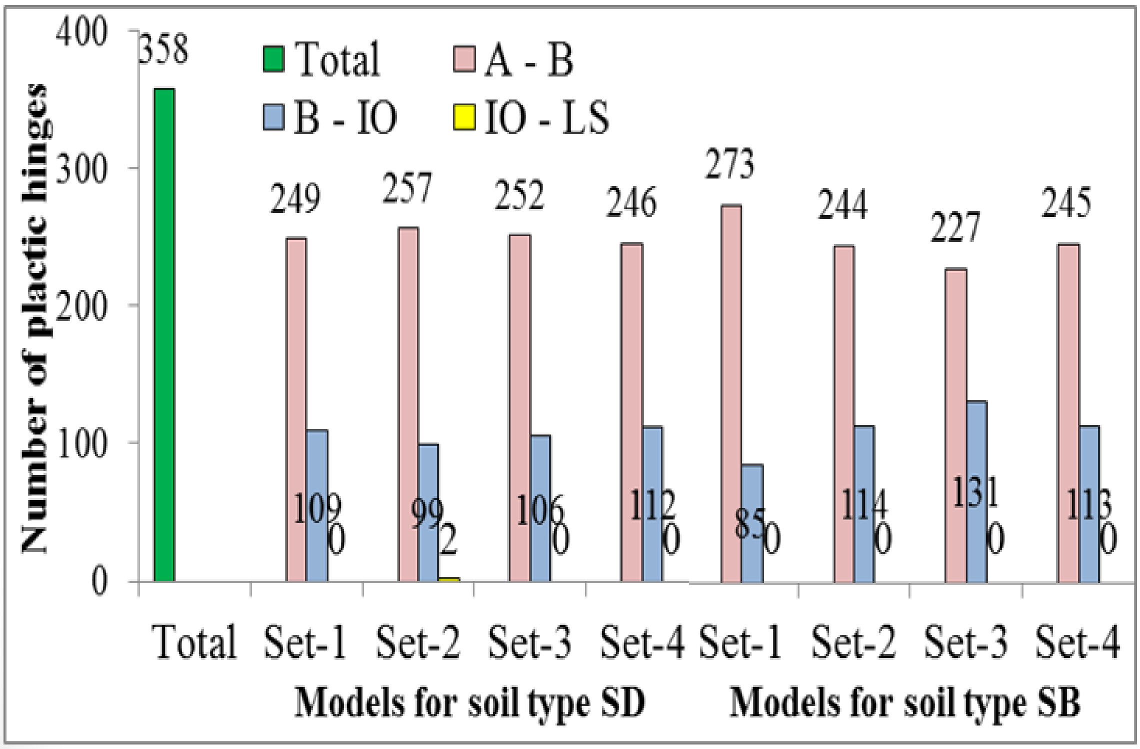


FIGURE 4.10: Plastic Hinge Formation in Push-Y-direction for soil type SD and SB.

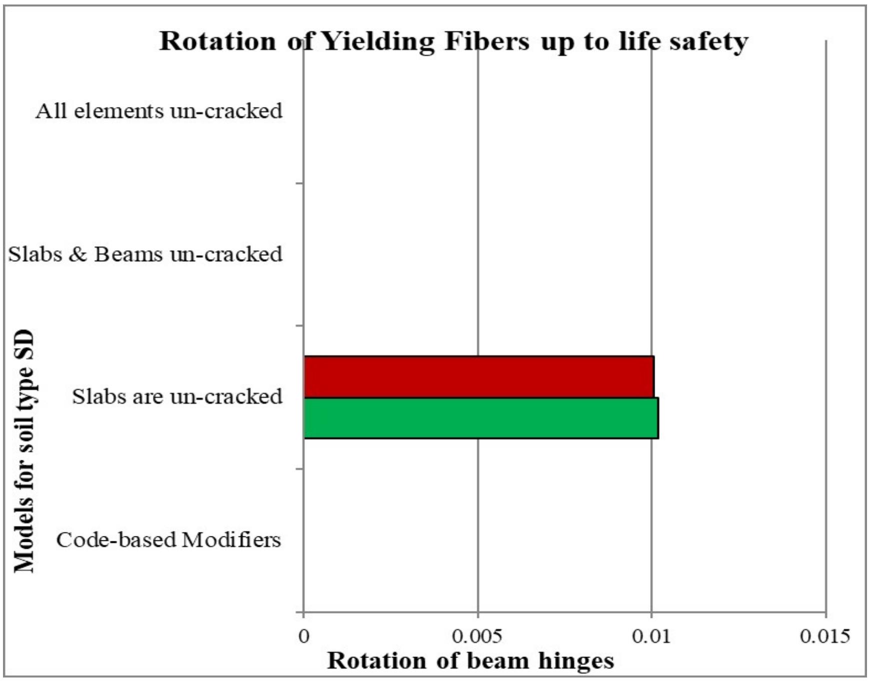
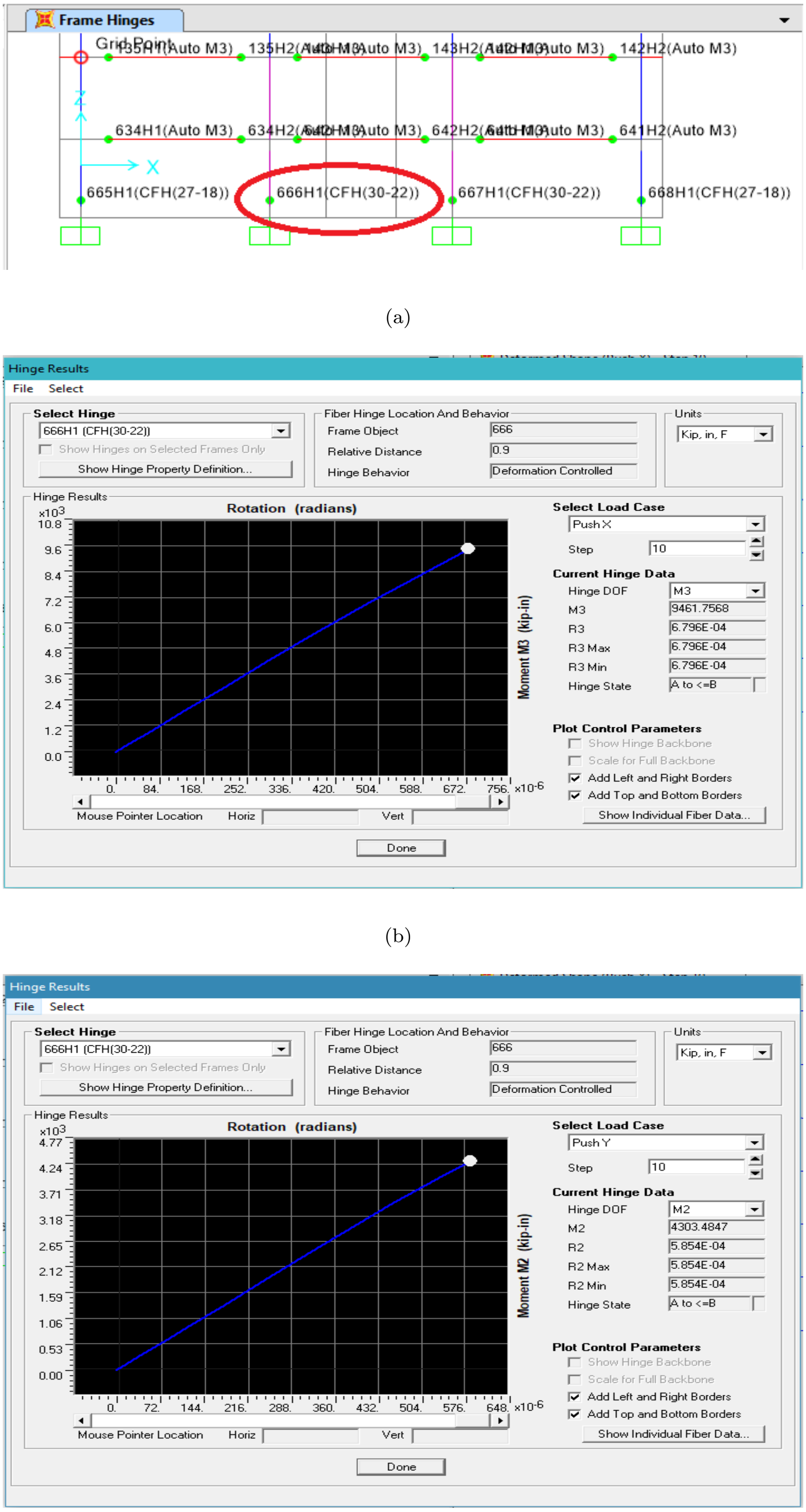


FIGURE 4.11: Rotation of Yielding Fibers up to life safety in push-Y-direction for soil type SD.

4.8 Formation of Plastic Hinges in Columns in PoA

Fiber hinges have been assigned at the bottom of all the columns of the bottom storey in accordance with physical admissible plastic hinge mechanism. Assignment of fiber plastic hinges to columns of bottom storey and State of one representative hinge, namely 666H1 for Set-1 for the soil type "SD" in Push-X and Push-Y-directions is shown in Figure 4.12. Comparison of moment M3 and rotation R3 for Push-X case to that of FEMA 356, Table 6-8 showed that hinge would remain in the elastic range.



(a)

(b)

(c)

FIGURE 4.12: (a) 666H1 column fiber hinge (b) State of column fiber hinge 666H1 in Push-X-direction (c) State of column fiber hinge 666H1 in Push-Y-direction.

Total number of hinges assigned to columns are 12. As design and analysis is done for design basis level earthquake, thus all the fiber hinges for all cases of stiffness modification under consideration remain in elastic range i.e. in A to B range of FEMA 356 force-deformation relation for concrete elements. Furthermore, to judge the behavior of fiber hinges, stresses and strains of individual fibers have been checked to verify if hinge is yielding or not. Steel of Grade-60 is used in analysis, thus fibers of hinges having stress more than 60 ksi and strain more than 0.002 have been considered as yielding. In Figure 4.13, number of column hinges yielding can be observed in each case under consideration. For the Set-1, 2 and 4 fibers of only 3, 2, and 2 hinges undergo yielding in Push-X-direction, whereas in Push-Y-direction, only 1 hinge went beyond yielding in Set-1 and only 2 hinges in Set-2 and 4. For Set-3l, fibers of any single hinge did not go beyond yielding in both Push-X-direction and Push-Y-directions. Figure 4.14 (a and b) shows strain of fibers, for which stress has been gone beyond 60 ksi.

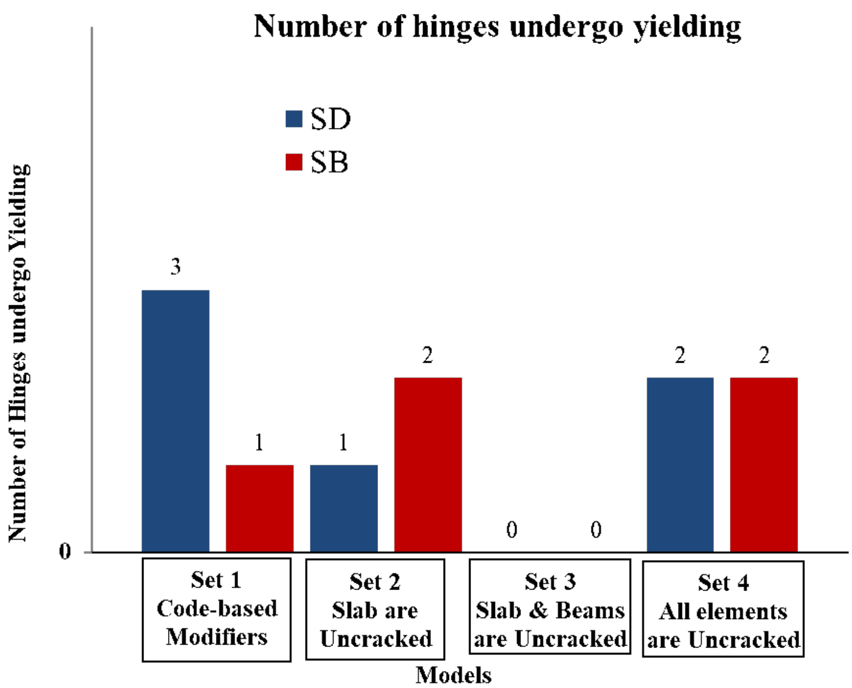
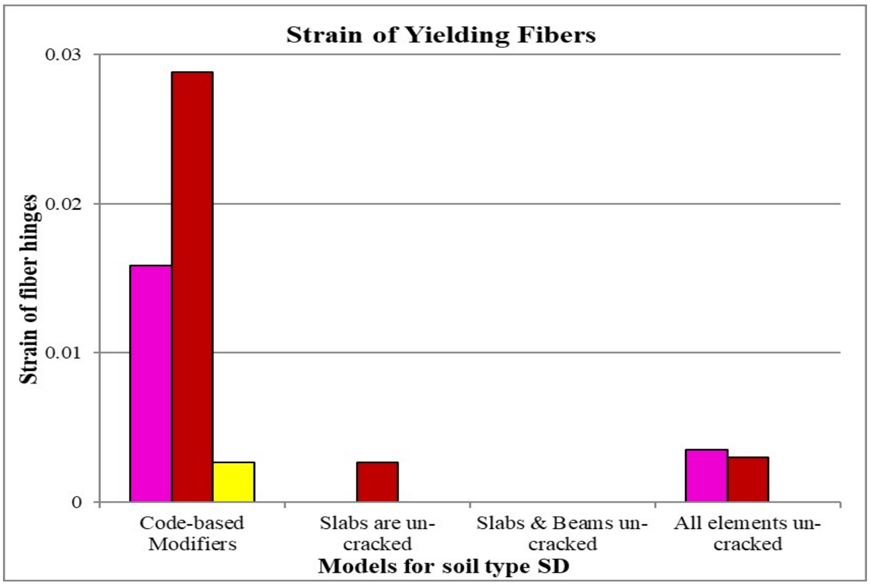
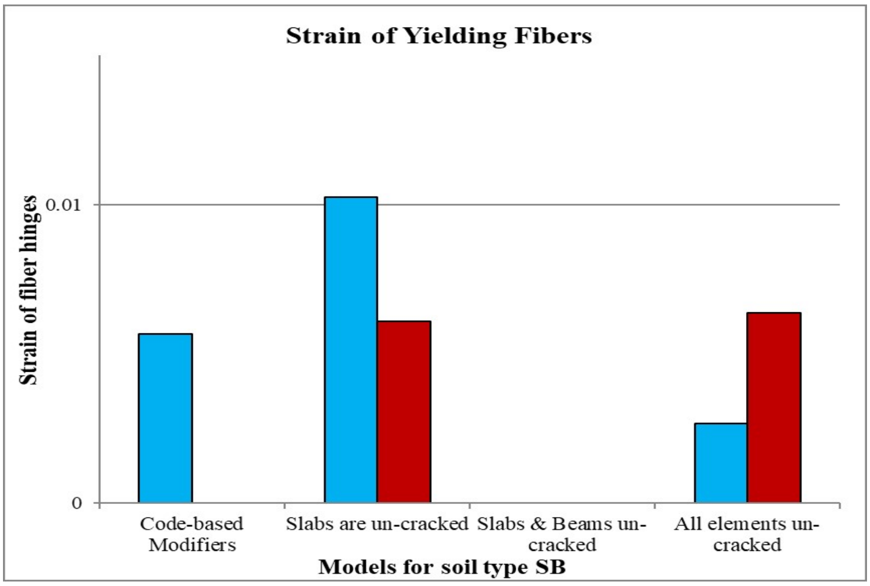


FIGURE 4.13: Number of hinges yielding in all cases.



(a)



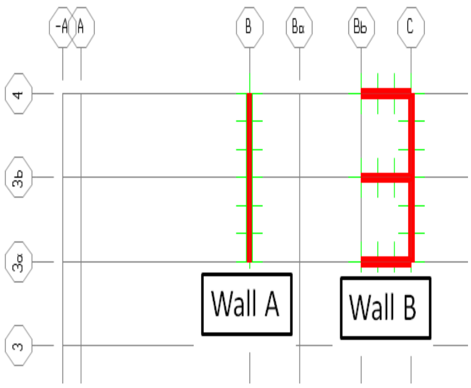
(b)

FIGURE 4.14: Strain of yielding fibers of hinges (a) Soil type SD (b) Soil type SB.

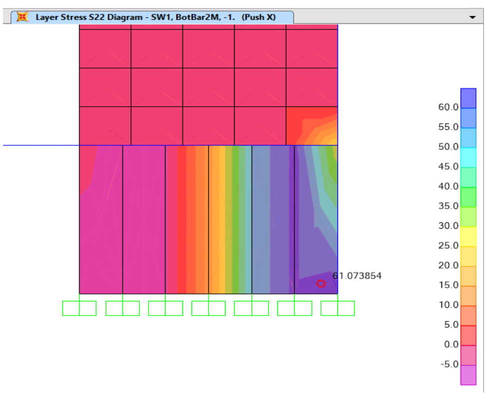
From all above-mentioned plastic hinge formation in beams and columns, formation of hinges in Set-1 indicates code intended behavior, i.e. formation of plastic hinges at ends of beams and bottom of bottom storey columns. Due to non/ less formation of hinges in beams & columns for set 2 to 4 may result into formation of hinges at undesirable locations. Moreover, failure in undesirable modes, such as shear failure of beams & columns, may occur.

4.9 Stresses in Walls

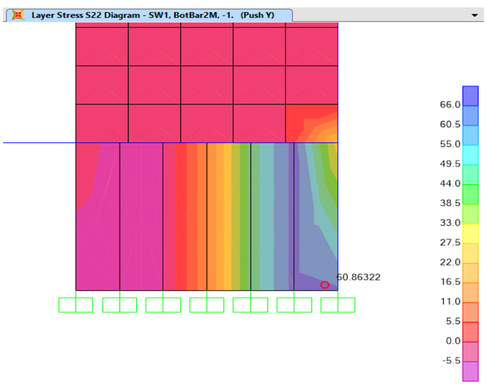
Walls of the building have been catagorised as wall A and wall B. Wall A has been divided into 6 segments and wall in 15 segments. Total 21 segments of wall shell layered areas have been assigned in the designing of the building under consideration as shown in Figure 4.15 (a). Non-linear wall has been modeled at bottom most floor only. For static push over analysis in the Push-X and Push-Y case, it is evaluated that how shear wall acts and how many number of segments or fibers of the wall are undergoing yielding. In Figure 4.15 (b) and (c), stresses in a typical wall section both for Push-X and Push-Y-direction are shown. The value more than 60 depicts that there is yielding in that specific segment of wall.



(a)



(b)

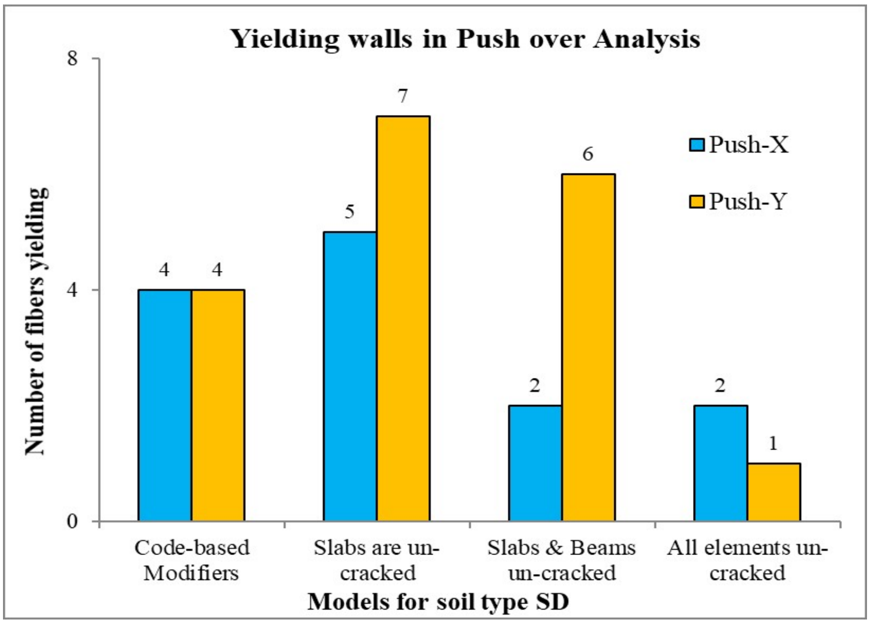


(c)

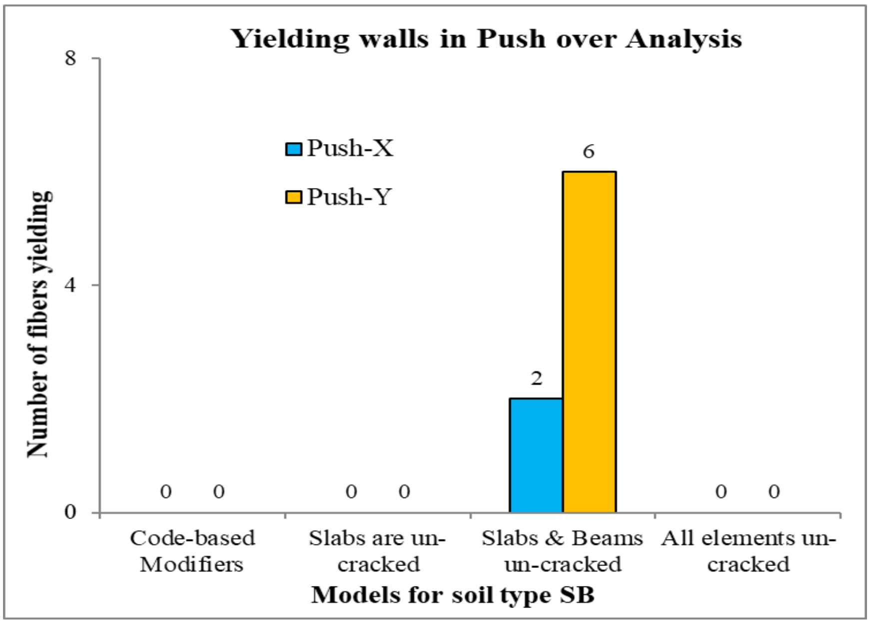
FIGURE 4.15: (a) Shell layered wall assignment (b) Stresses in wall in Push-X (c) Stresses in wall in Push-Y.

For evaluation of seismic behavior of the wall, reinforcement used against M22 moment in vertical direction (main reinforcement of wall) is considered as fibers of wall. It is evaluated that how much numbers of fibers (vertical reinforcement) experience yielding. Every segment of wall contains 16 fibers resulting to total of 336 fibers. Number of fibers yielding in both Push-X and Push-Y-direction for all Sets of models is shown in Figure 4.16.

It can be seen from Figure 4.16 (a) that for the code-based stiffness modifiers case for soil type SD, fibers of only 4 wall segments undergo yielding both in the X and Y-directions. InSet-2, however, maximum number of fibers of wall segments undergo yielding with 5 in X-direction and 7 in Y-direction. For Set-3, the yielded segments count to 2 and 6 in X and Y-direction respectively. For Set-4 having all elements uncracked, number of yielded wall segments are less than all the other cases. For soil type SB, fibers of wall segments undergo yielding in Set-3 only as shown in Figure 1.16 (b) as 2 in the X-direction and 6 in the Y-direction.



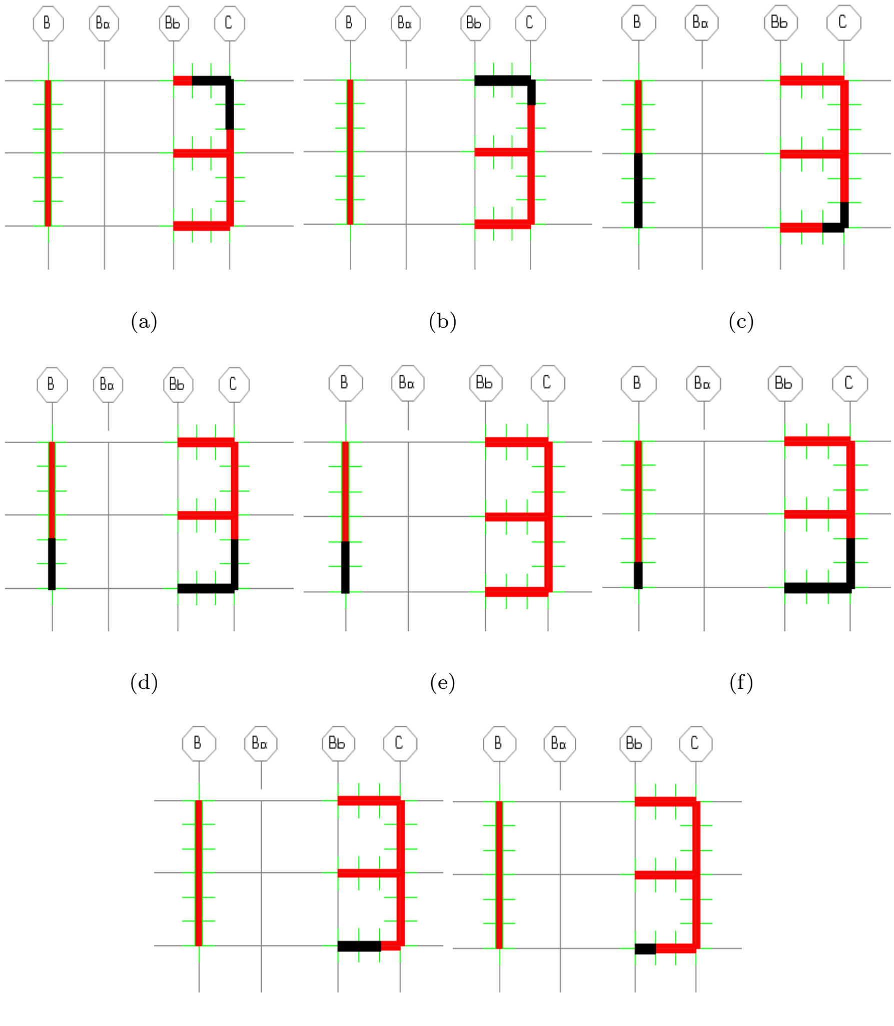
(a)



(b)

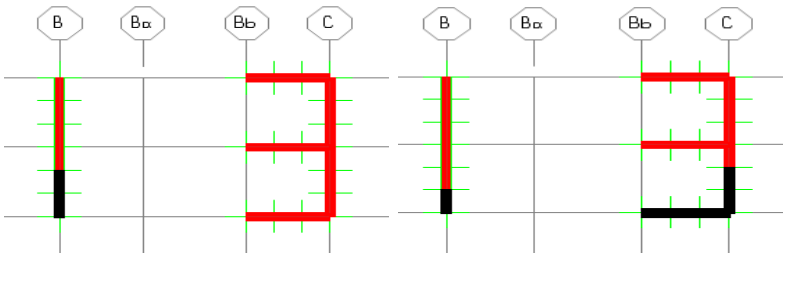
FIGURE 4.16: Yielding fibers of walls (a) Soil type SD (b) Soil type SB.

In figures 4.17 and 4.18, highlighted black segments show yielding fibers in each case. It can be noticed that, there is minor difference between yielding wall sections / segments location assigned in plan.



(a) (b) (c) (d) (e) (f) (g) (h)

FIGURE 4.17: Wall segments undergo yielding (a) Code-based Modifiers Push-X (b) Code-based Modifiers Push-Y (c) Uncracked Slabs Push-X (d) Uncracked Slabs Push-Y (e) Uncracked Slabs and Beams Push-X (f) Uncracked Slabs and Beams Push-Y (g) All elements Uncracked Push-X (h) All elements Uncracked Push-Y.



(a) (b)

FIGURE 4.18: Wall segments undergo yielding (a) Uncracked Slabs and Beams Push-X (b) Uncracked Slabs and Beams Push-Y.

4.10 Stresses in Slabs

The different set of stiffness modification factors in RC structures results in different load transfer mechanism. To demonstrate the load transfer mechanism, a typical slab panel from first floor level is considered as enclosed by grid 1, 2, B and C and shown in Figure 4.19.

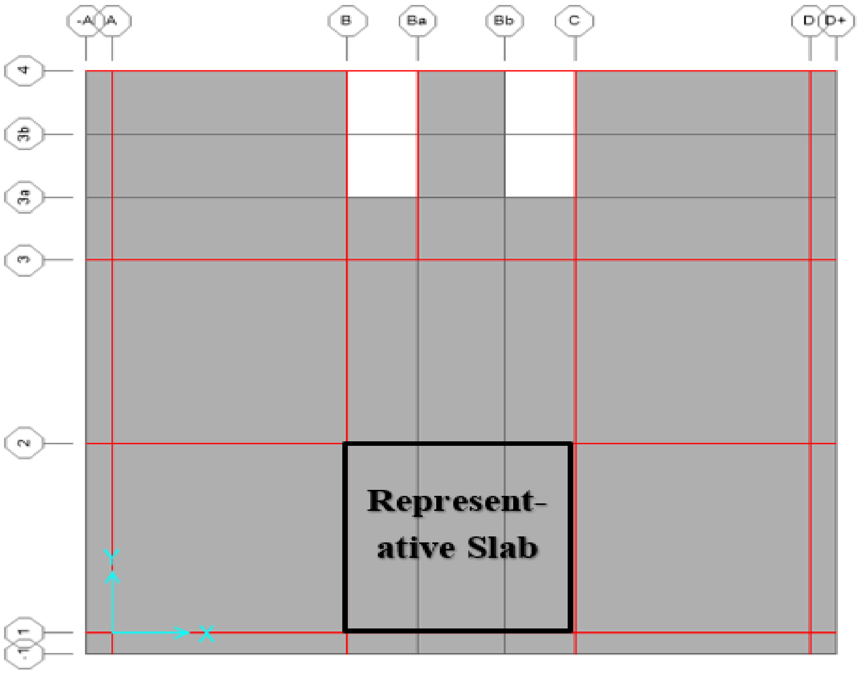


FIGURE 4.19: Representative slab for stress transfer comparison.

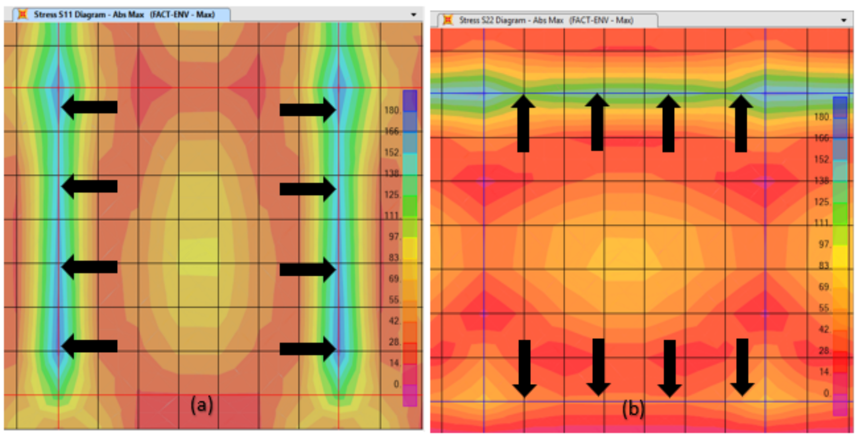


FIGURE 4.20: Stress transfer in code-based modifiers case for 'SD' (a) S11 Abs Max (b) S22 Abs Max.

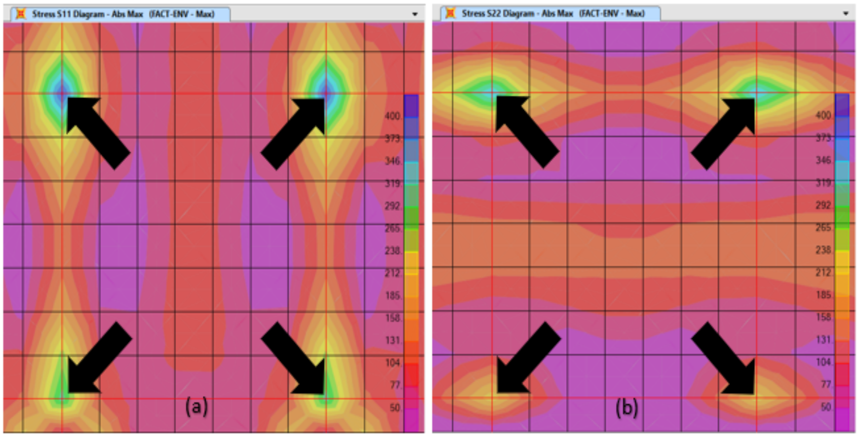


FIGURE 4.21: Stress transfer in uncracked slabs case for 'SD' (a) S11 Abs Max (b) S22 Abs Max.

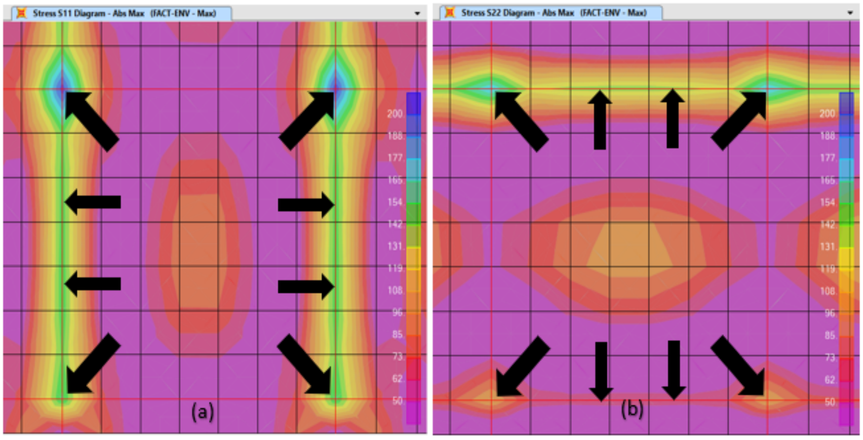


FIGURE 4.22: Stress transfer in uncracked slabs and beams case for 'SD' (a) S11 Abs Max (b) S22 Abs Max.

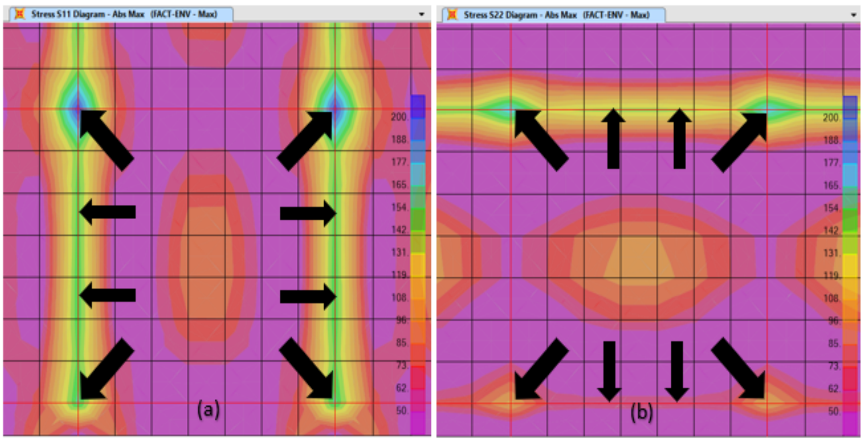


FIGURE 4.23: Stress transfer in all element uncracked case for 'SD' (a) S11 Abs Max (b) S22 Abs Max.

It can be observed from the Figure 4.20 that for design of structure using code-based stiffness modifies, the load is transferring from slab to beam equally along the length of the beam and from beam to column. Whereas, for Set-2 as shown in Figure 4.21, it was observed that 70% of load is directly transferring from slab to column and only 30% load is being transferred along the length of the beam. In this case stiffness of slabs is modeled as uncracked and hence more stiffness of the slabs could be the reason of this load transfer mechanism. For soil type SB, load transfer mechanism for Set-1 and Set-2 are presented in Figure 4.24 and 4.25 respectively. In both soil types load transfer mechanism has same behavior.

In Figures-22 and 23, load transfer mechanism in Set-3 and Set-4 has been presented for soil type SD. It was observed that 30% of load is being transferred directly to columns and 70% of load is being transferred along the length of the beam unlike Set-1. While in case of soil type SB, shifting of load along the length of beam is 75% and 25% is directly transferring to columns for Set-3 and Set-4 as presented in Figure 4.26 and Figure 4.27.

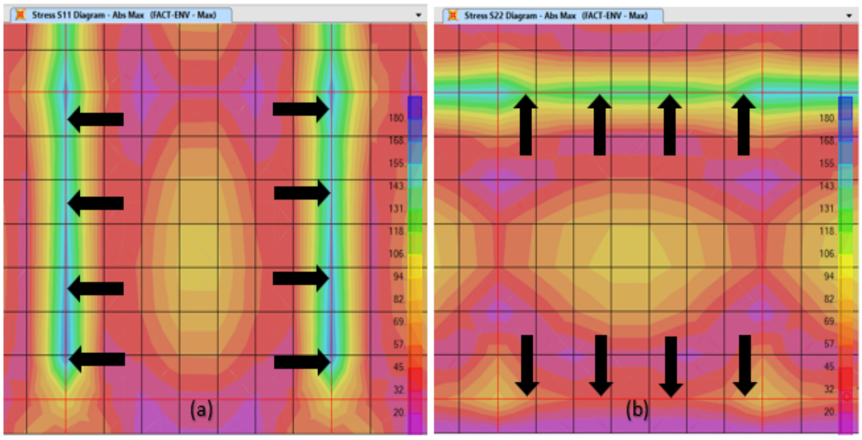


FIGURE 4.24: Stress transfer in code-based modifiers case for 'SB' (a) S11 Abs Max (b) S22 Abs Max.

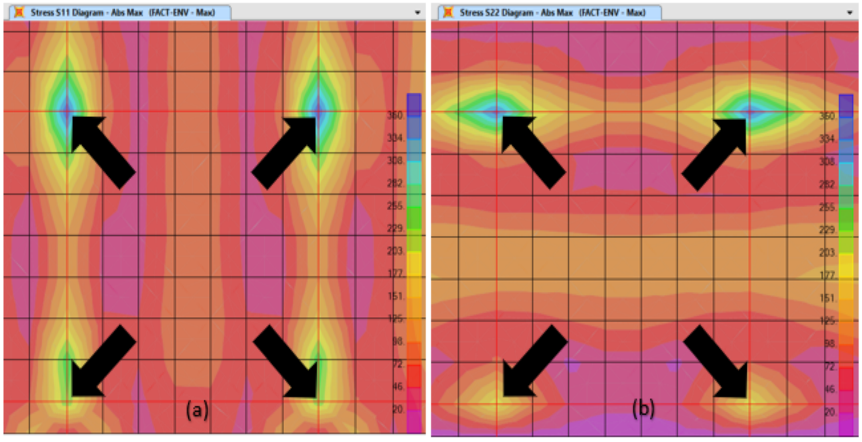


FIGURE 4.25: Stress transfer in uncracked slabs case for 'SB' (a) S11 Abs Max (b) S22 Abs Max.

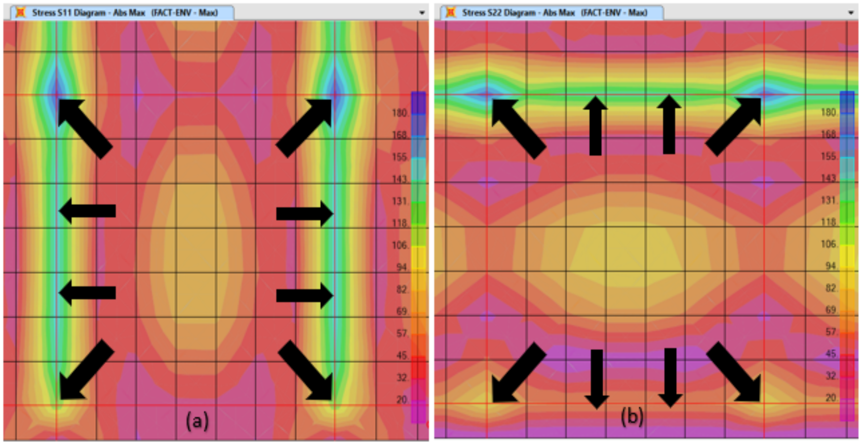


FIGURE 4.26: Stress transfer in uncracked slabs and beams case for 'SB' (a) S11 Abs Max (b) S22 Abs Max.

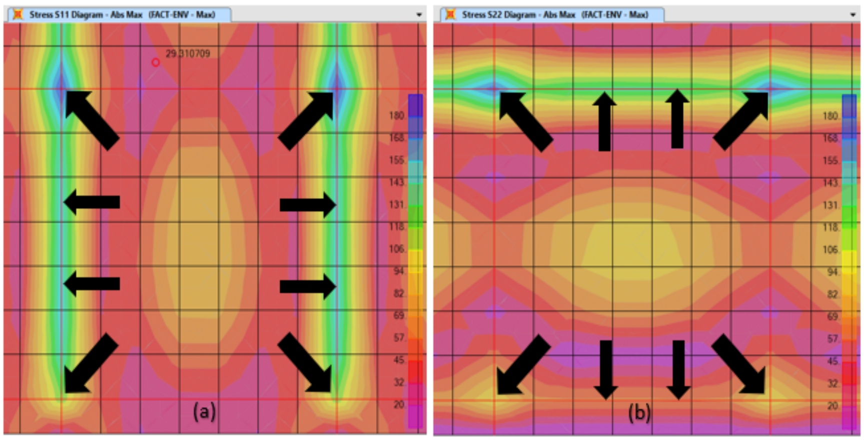


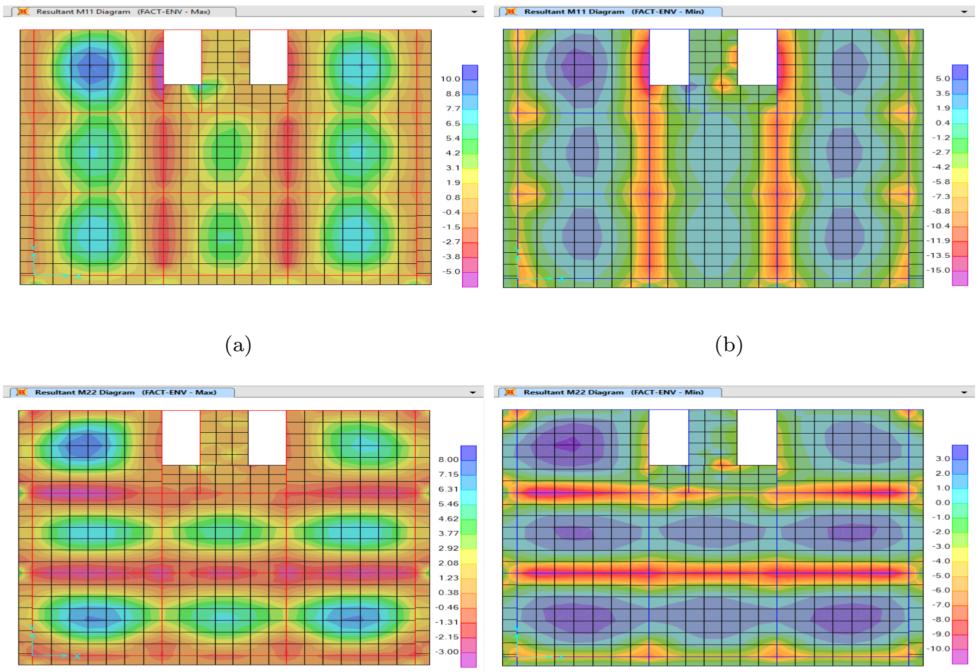
FIGURE 4.27: Stress transfer in all element uncracked case for 'SB' (a) S11 Abs Max (b) S22 Abs Max.

4.11 Calculation of Steel Reinforcement

Steel deformed bars are used as reinforcement in RC structures to cater tensile and flexural loading. Steel reinforcement has been calculated and compared for structural elements of all the cases of stiffness modification factors under consideration. The shear and moment demand in different cases has been verified through required reinforcement.

4.11.1 Slab Reinforcement

First floor level is considered as typical floor for the calculation and comparison of steel reinforcement in slabs for all the models under consideration. In Figure 28 (a) to (d), the maximum and minimum resultant forces in M11 and M22 direction for Set-1 for soil type SD i.e. code-based modification factors are shown. Positive and negative steel reinforcement has been calculated for each slab panel.



(a) (b) (c) (d)

FIGURE 4.28: Resultant slab stresses (a) M11 maximum (b) M11 minimum (c) M22 maximum (d) M22 minimum.

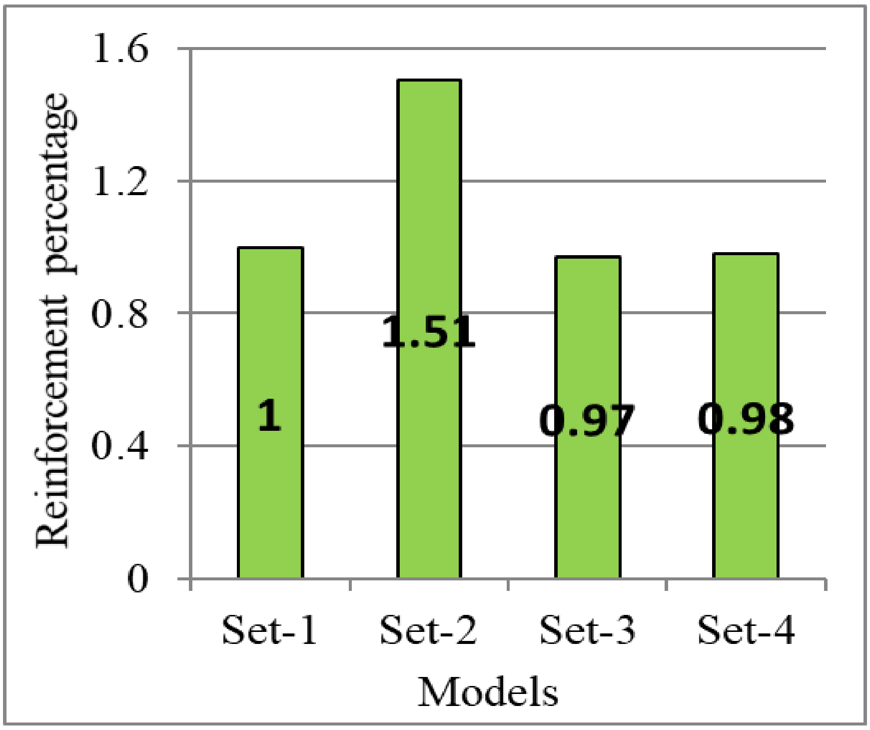


FIGURE 4.29: Slab reinforcement comparison for soil type SD.

It can be noticed from Figure 29 that keeping slabs uncracked in modeling results in more reinforcement of slabs confirming the results of section 4.3 and 4.4. There is large difference, up to 51% increaseas compared to Set-1. Reason for increase in set-2 is that most of the load is being transferred from slab to columns directly, so negative steel in slabs will increase.

4.11.2 Beam and Column Reinforcement

Flexure and torsion reinforcement quantity has been calculated by code-based analysis for all the Sets and is compared in Table 4.3 and 4.4 and has been shown in Figure 4.30 and 4.31 for beams and columns respectively.

Table 4.7: Reinforcement for SD soil type cases.

Model	Beam Flexural reinf.	Beam Torsional reinf.	Column reinf.	Total reinf. (Tonns)
Set-1	28.80	3.32	17.61	49.72
Set-2	22.83	2.33	17.17	42.32
Set3	30.15	9.34	17.90	57.38
Set-4	30.40	9.46	18.26	58.12

Table 4.8: Reinforcement for SB soil type cases.

Model	Beam Flexural reinf.	Beam Torsional reinf.	Column reinf.	Total reinf. (Tonns)
Set-1	27.71	3.49	16.11	47.31
Set-2	21.76	2.26	15.93	39.95
Set3	28.17	8.86	15.63	52.66
Set-4	28.8	8.95	15.91	53.14

It can be clearly observed that for Set-2, uncracked slabs case, reinforcement ratio is less than all other cases as the reinforcement of slabs is not included. Reason for reduction in set-2 is that most of the load is being transferred from slab to columns directly in axial direction, so steel is less in beams and columns in Set-2. For Set-3 and 4 increase in reinforcement of beams and columns can be observed. In crease of reinforcement in beams is more that that of columns seggesting opposite to weak beam strong column philosophy for these sets. Comparison of shear reinforcement of beams and columns of all the cases is shown in Figure 4.32. The reason for less steel in Set-2 is same as for flexure and torsional reinforcement. Increase of 9% and 13% shear reinforcement can be seen in Set-3 and 4 respectivelt as compared to Set-1. As in these 2 cases beams and columns have been modeled as uncracked thus, uncracked elements result in increase in shear reinforcement. This result is in line with the result of section 4.3 (storey shear inelastic demand) and section 4.4 (inelastic moment demand). Results of reinforcement are same for both soil types under consideration.

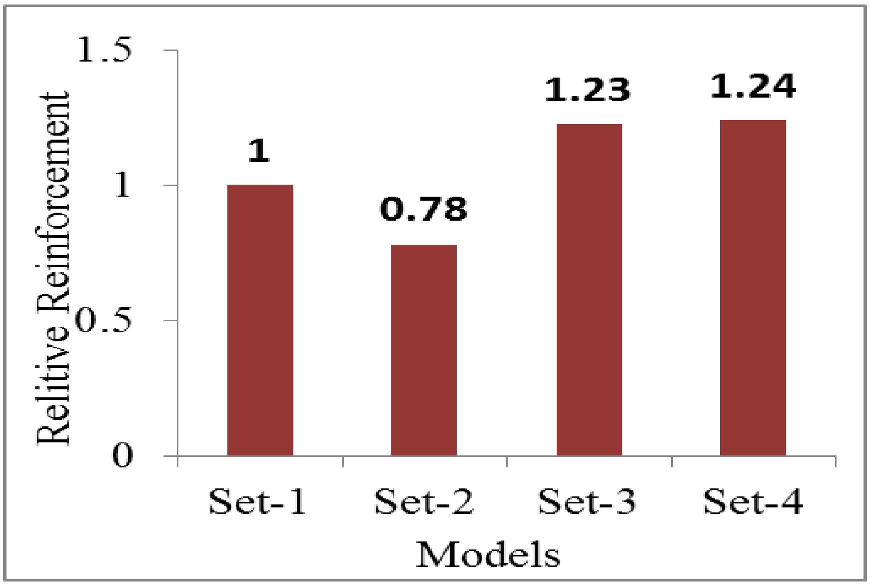


FIGURE 4.30: Reinforcements comparison for beams.

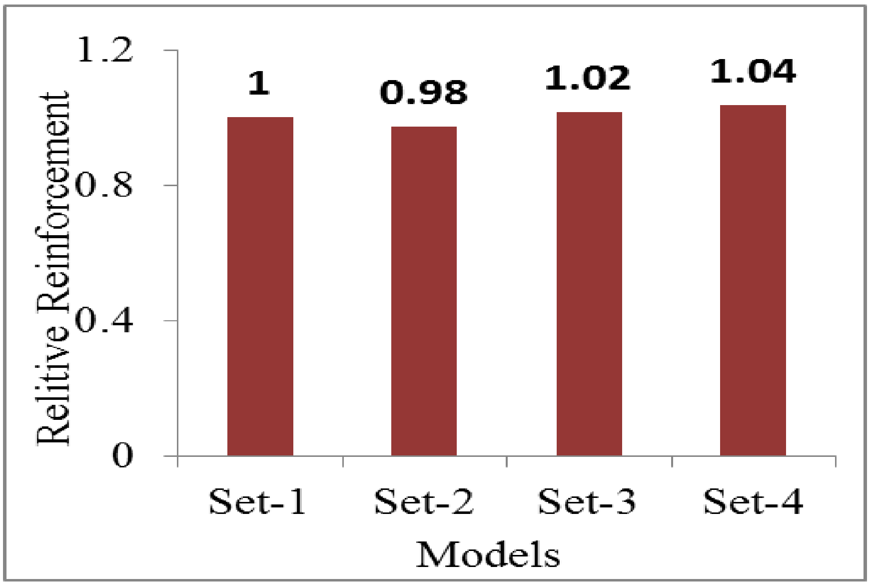


FIGURE 4.31: Reinforcements comparison for Columns.

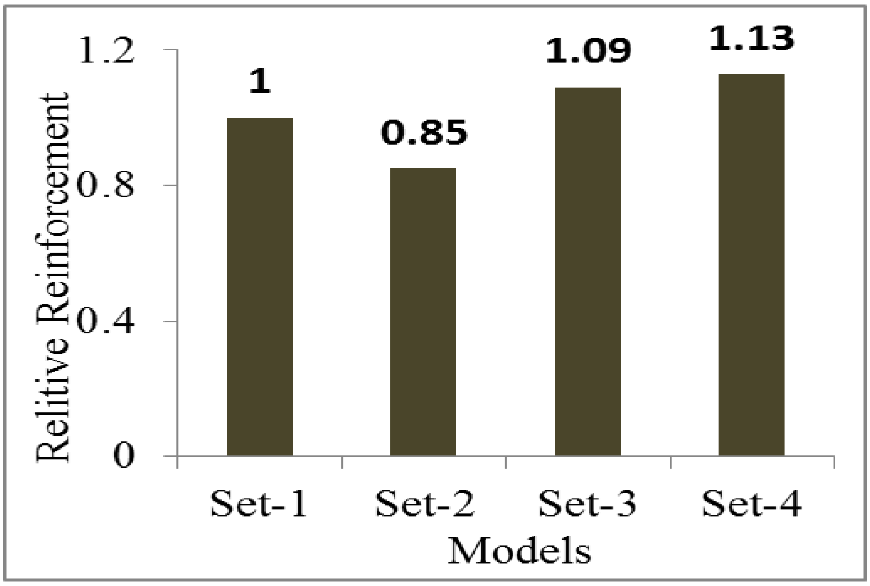


FIGURE 4.32: Comparison for Shear Reinforcement.

4.11.3 Wall Reinforcement

Reinforcement in walls has been compared in Figure 4.33. It can be observed that wall reinforcement is same in Set-1 and Set-2. An increment of 17% and 35% can be seen in Set-3 and Set-4 respectively as compared to that of Set-1. In Set-4 maximum increase in steel has been resulted as walls have been modeled as uncracked in this case.

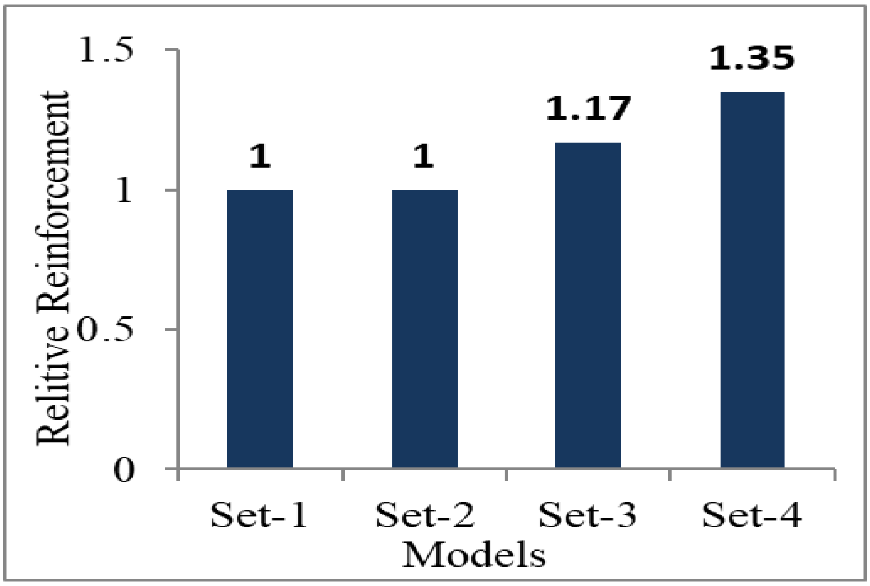


FIGURE 4.33: Comparison for Wall Reinforcement.

4.12 Summary of Discussions

In this chapter, seismic behavior of building designed with code-based, in-practice and hypothetical sets of SMFs has been analyzed and compared with code-based SMFs as reference. Decrease in reduction factor due to inherent ductility "R_μ", and increase in over strength factor "Ω" has been observed as un-cracked elements increased in the models. Storey shears and overturning moment inelastic demand has been increased. Whereas, storey displacement and storey drift has been decreased from Set-1 to Set-4. Increase in flexure strength means more capacity requirement i.e. more reinforcement. Higher demand greatly affects the elements/ behavior that intended to be elastic i.e. formation of hinges in columns in upper stories, shear in frame sections, bond and slip failure and foundation pressure.

Load transfer mechanism has been observed in all cases and suggested that load in tranferring in Set-1 that is ACI defined SMFs is realistic and according to strength based design philosophy. Formation of hinges in X- and Y-directions of each Set has been presented. In beams hinges have been formed at almost all levels and remained in immediate occupancy level and life safety level. The damage level in beams in Set-1 is less in X-direction and almost same for all cases in Y-direction. For columns all the fiber hinges for all sets of stiffness modification under consideration remain in elastic range. Formation of hinges in Set-1 indicates code intended behavior, i.e. formation of plastic hinges at ends of beams and bottom of bottom storey columns. Due to non/ less formation of hinges in beams & columns for set 2, 3 & 4 may result into formation of hinges at undesirable locations. Moreover, failure in undesirable modes, such as shear failure of beams & columns, may occur.

Cross-sections used are same for all cases therefore, concrete quantity is same. Reinforecement requirement has been observed more as less SMFs used in the analysis. It is depicted that non-incorporation of cracking effects of concrete results in more reinforcement, hence more cost. However, use of uncracked section of concrete in designing and modeling does not mean that cracking will not happen. There will be cracking in concrete elements but use of uncracked section results in more reinforcement. All the cases are designed for the same loading and load combinations in the analysis. The increased reinforcement ratio could be due to different stiffness, cracking of concrete, load transfer mechanism and lateral forces owing to different stiffness of the members.

Chapter 5

Conclusion and Recommendations

In this study, a realistic 7-storied building has been considered as the case study. Dual Intermediate moment resisting frame (IMRF) comprising of structural frame and structural wall system located in zone 2B with two different soil types SD and SB has been considered. Seismic behavior of the building for four different sets of stiffness modification factors (Ref: Table 1.1) is analysed and discussed. All the systems have been first analyzed according to code-based equivalent static analysis and designed using code-based load combinations and strength reduction factors. Formation of plastic hinges in frame elements, stresses, strains and rotation of yielding fibers of column and wall and load transfer pattern has also been compared to predict seismic performance of the structure. Furthermore, steel reinforcement resulted in all the cases have been compared to see which Set of SMFs lead more economical design. The conclusions of the study are given in following section.

5.1 Conclusions

Following conclusions have been made out of this study:

- The time period used in determination of equivalent static base shear for all the sets of stiffness modification factors under consideration in both X and Y-directions remains same for both soil types i.e. SD and SB. The reason of the same time period in all the cases is the limitation of the code-based time period using method-A and limitation on time period B. The parameter involved is the height of the building. This means equivalent base shear is almost same for all models, however, both gravity load and lateral load transfer mechanism would be different and as a result beams reinforcement as well as inelastic base shear is increased with the increasing stiffness of the model.

- Story shear forces obtained from linear equivalent static force procedure in X-direction for all the cases of stiffness modification remains approximately same for both soil types SD and SB. Whereas, in case of Y-direction for soil type SD, story shear for Set-1 and Set-2 has the same value, while for Set-3 and Set-4, story shear has been increased marginally for both soil types.

- As stiffness of the model is shifting towards uncracked structural elements, modified "R_μ" values obtained from nonlinear static analysis is significantly reducing as compared to that of code-based values, whereas, modified over strength factor "Ω" is increasing. This may be due to strain hardening effect of the rebar, increased stiffness of the concrete members, load transfer mechanism and effect of vertical seismic forces. Resulted values of "R_μ" and "Ω" in PoA in Set-1 are comparable with code-based values suggesting the correlation of SMFs with code-based "R_μ" and "Ω". As a result of increased Ω, more demand (more reinforcement) will be required to avoid premature failure in shear in beams and columns, foundation flexure and bearing pressure failure, and induction of plastic hinges in columns in upper stories.

- Shear inelastic demand has been increased as uncracked structural elements increased in the design. The reason of this is due to different load transfer mechanism and consequently more flexural inelastic capacity (more designed reinforcement) provided in beams for uncracked models.

- The overturning moment in all the systems having different SMFs remain exactly same for linear static analysis in EX-direction for both soil types i.e. SD and SB. Whereas, in EY-direction minor difference has been observed between all four cases. The inelastic overturning moment for Push-X case of both soil types increases with the increase in stiffness i.e., as elements of system getting stiffer. For Push-Y-direction for both soil types, increase in overturning moment demand with increase in stiffness is less than that of push-X case. The reason of this could be the presence of non-linear shear wall in Y-direction. The nonlinear behavior of walls controls the behavior of the structure.

- Story displacement in the Y-direction for EQA remained same for all the cases due to presence of wall in Y-direction. While, variation in both static linear and nonlinear storey displacement has been observed in all other cases for both soil types. Decreasing trend has been observed as uncracked elements increased. This is due to increased stiffness of the structural members.

- For both the soil types, storey drift has been decreased as stiffness of the system increased in trend similar to story displacement for linear static and nonlinear static analysis in X and Y-directions.

- The status of hinges in beams, yielded fibers of column hinges and yielding fibers of wall segments in all systems for both soil types suggests that uncracked systems behaves better in flexure as compared to that of code-based modifiers system. However, better flexure behavior does not necessarily guarantee better performance for other member and actions. Formation of hinges in Set-1 indicates code intended behavior, i.e. formation of plastic hinges at ends of beams and bottom of bottom storey columns. Less number of low levels hinges formation in beams and columns of hypothetical and in-practice SMFs cases may result into formation of hinges at undesired locations i.e.

columns in upper stories, at mid span of beams and damage shift to foundation. As mentioned earlier, the actions/ elements that desired to behave in the elastic range i.e. columns in upper stories, shear in beams and columns, bond and slip failure, foundation bearing pressure and reinforcement shall also be increased with the increase in flexure strength leading to undesirable premature brittle failure.

- Although applied loads, loading combinations and modeling technique is same for all the systems, but change in stiffness of the elements has significant effect on load transfer mechanism as building design is strength based. In code-based stiffness modifiers, the load is transferring from slab to beam equally along the length of the beam and from beam to column. Whereas, for other three cases having uncracked structural elements, 70% of load is directly transferring from slab to column and only 30% load is being transferred along the length of the beam. Load transfer mechanism in code-based stiffness modifiers case is as per guidelines of codes.

- The total reinforcement comparison depicts that design with code-based modifiers is about 20% economical than the other systems yet satisfying code intended performance more than other systems.

It is concluded that the system with code-based stiffness modification factors is better and economical than other systems in terms of load transfer mechanism, shear and moment inelastic demand, reinforcement demand and seismic behavior. Furthermore, it can be said that if structure design of any building is done using code defined loads, load combinations and modeling techniques, code-based stiffness modifiers are essential to be used.

5.2 Recommendations for Future Studies

The main objective of the study was to evaluate seismic behavior of the structure by using different in practice sets of stiffness modification. In this study nonlinear hinges are assigned only at specified places in the model. Further research can be done by modeling nonlinearity and fiber hinges throughout the concrete frame and area elements. This will ensure incorporation of actual cracking in concrete elements. Effect on columns of upper storey may be taken into account. Effect on foundation may be evaluated. Bond and slip failure of connections may be considered for further evaluation. Moreover, pushover analysis has been performed only for design basis earthquake level, it should be done for other hazard levels (Service and MCE) and performance should be verified. Presently the consequences of not using SMFs are not mentioned in the code. It can be recommended to elaborate these aspects at least in commentary in code after detailed evaluation of these aspects.

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Annexure-1

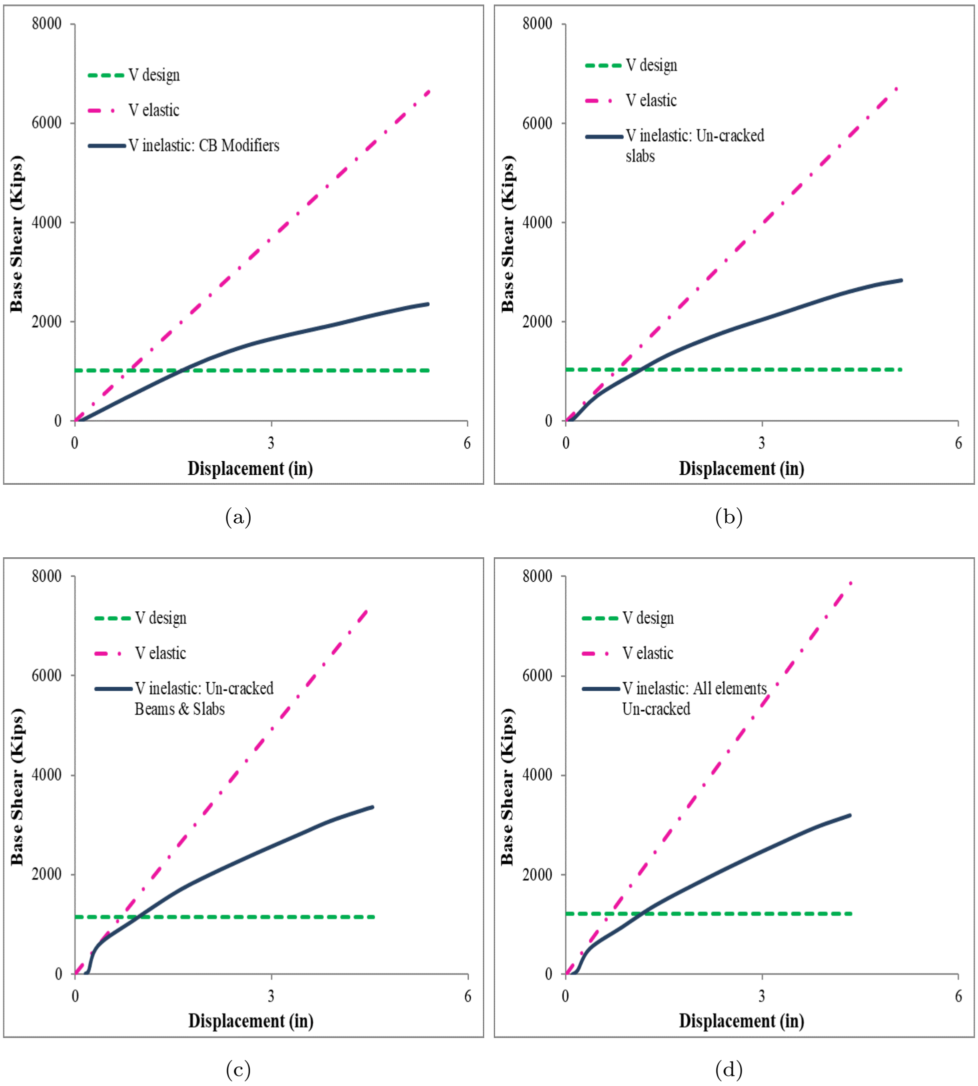


FIGURE A.1: Comparison of V_{Design}, $V_{Elastic}$ and $V_{Inelastic}$ in push-Y-direction for soil type SD (a) Code-based modifiers (b) Uncracked slabs case (c) Uncracked slabs and beams case (d) All elements uncracked case.

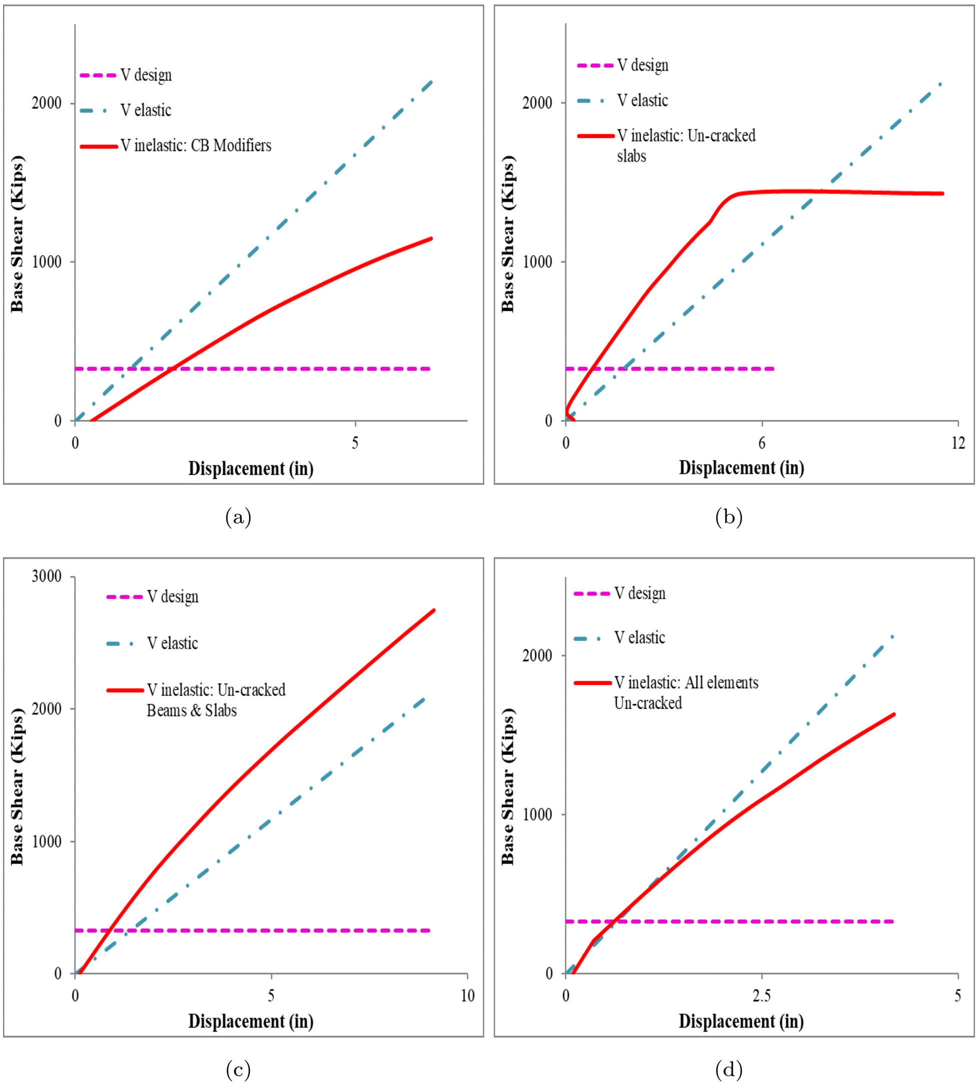
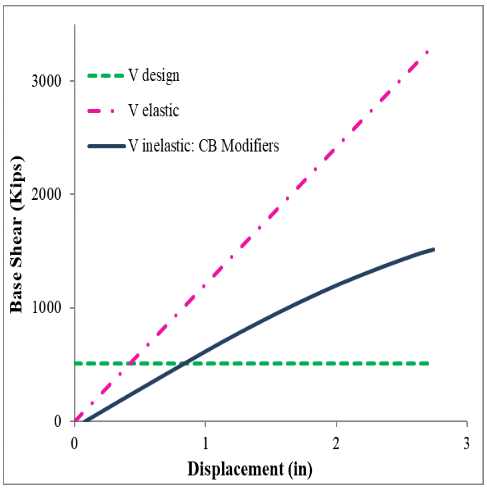
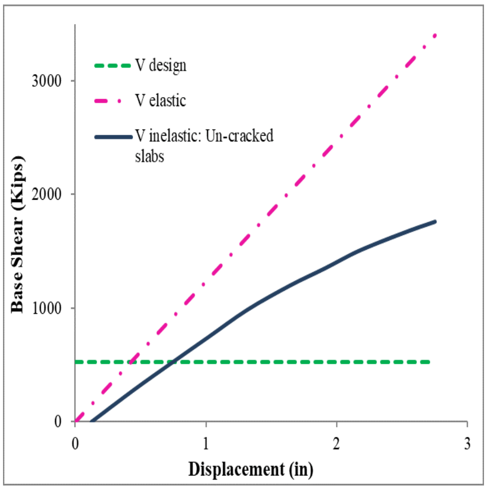


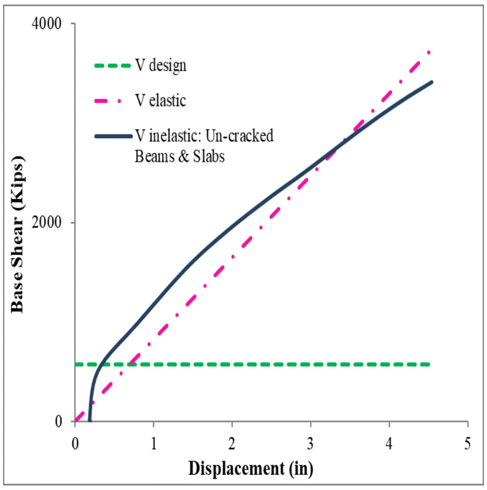
FIGURE A.2: Comparison of V_{Design}, $V_{Elastic}$ and $V_{Inelastic}$ in push-X-direction for soil type SB (a) Code-based modifiers (b) Uncracked slabs case (c) Uncracked slabs and beams case (d) All elements uncracked case.



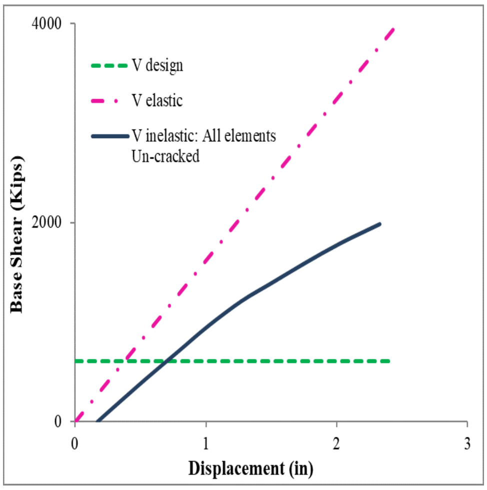
(a)



(b)



(c)



(d)

FIGURE A.3: Comparison of V_{Design}, $V_{Elastic}$ and $V_{Inelastic}$ in push-Y-direction for soil type SB (a) Code-based modifiers (b) Uncracked slabs case (c) Uncracked slabs and beams case (d) All elements uncracked case.

Annexure-2

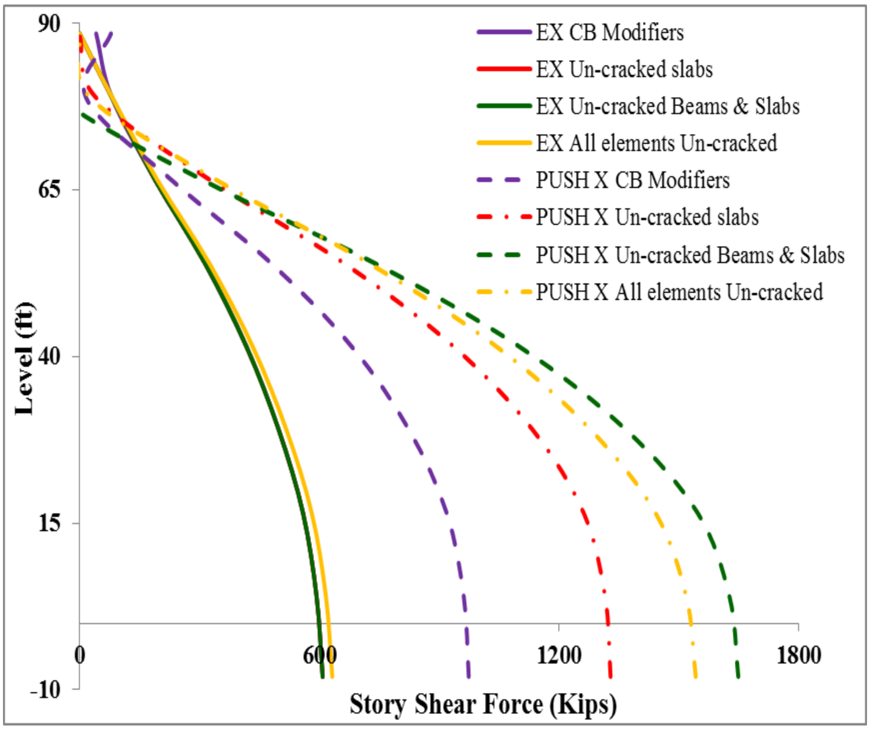


FIGURE A.4: Storey Shears comparison for EX and Push-X case for soil type 'SD'.

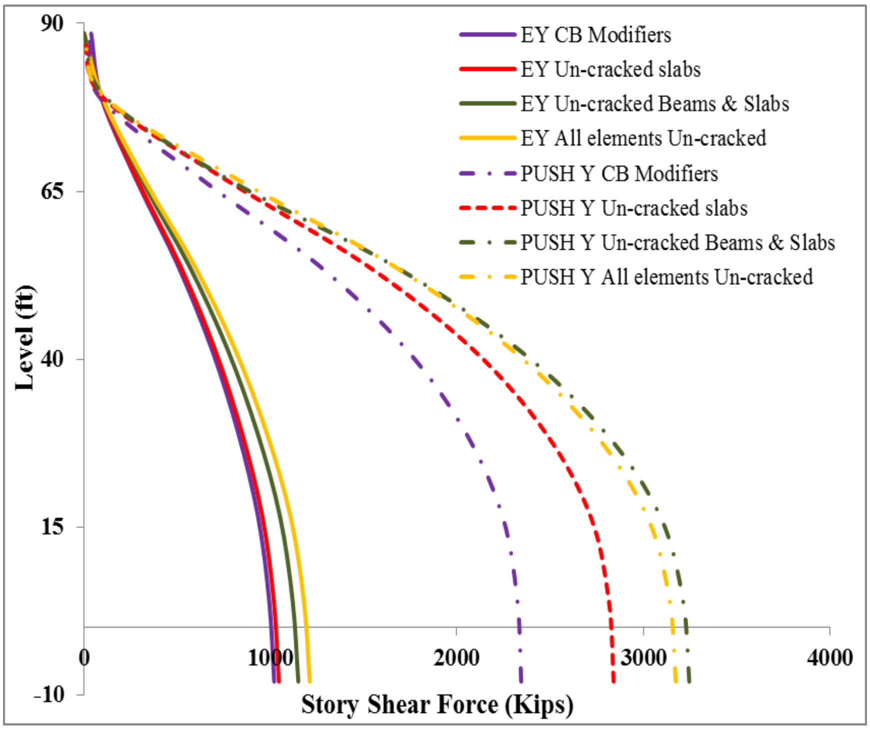


FIGURE A.5: Storey Shears comparison for EY and Push-Y case for soil type 'SD'.

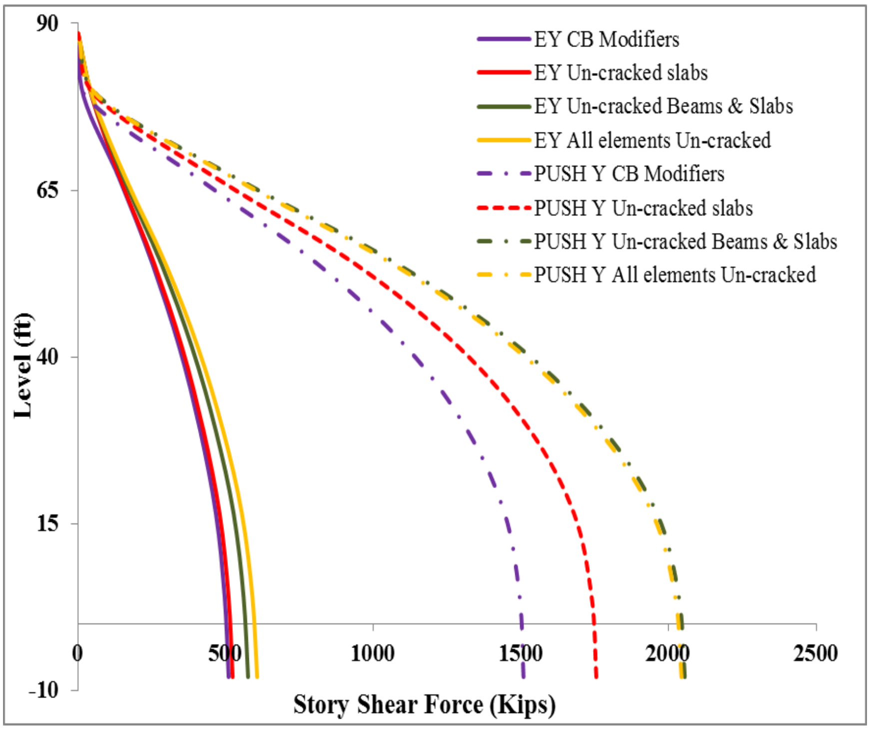


FIGURE A.6: Storey Shears comparison for EY and Push-Y case for soil type 'SB'.

Annexure-3

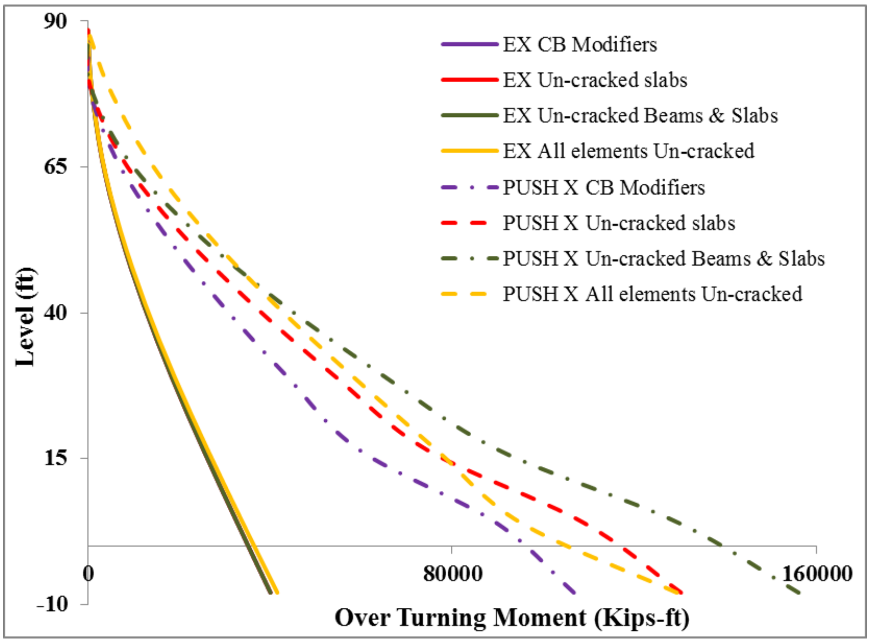


FIGURE A.7: Over turning moment comparison for EX and Push-X case for soil type 'SD'.

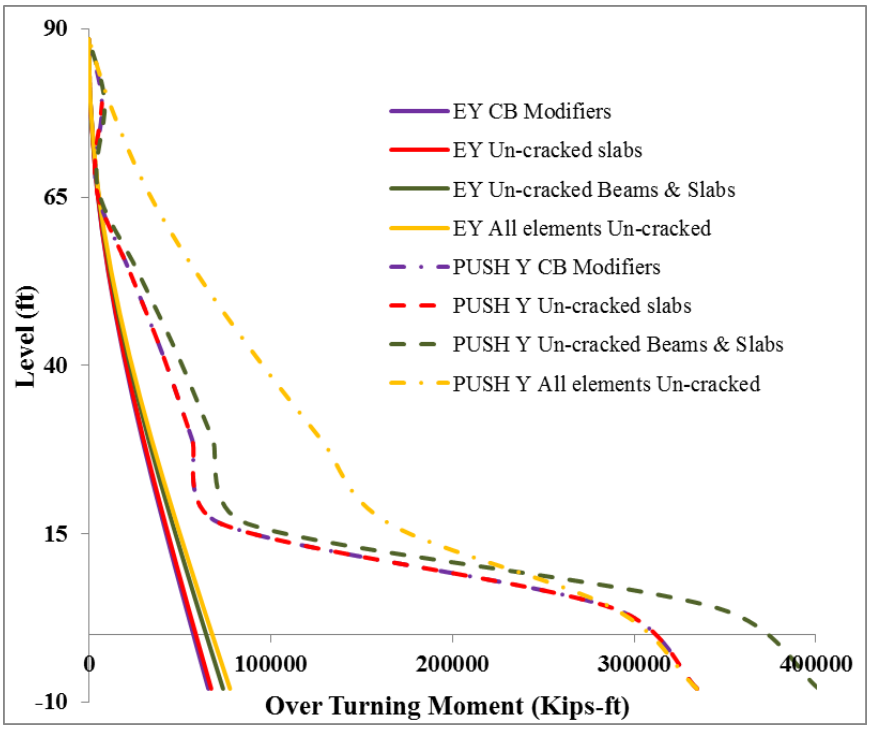


FIGURE A.8: Over turning moment comparison for EY and Push-Y case for soil type 'SD'.

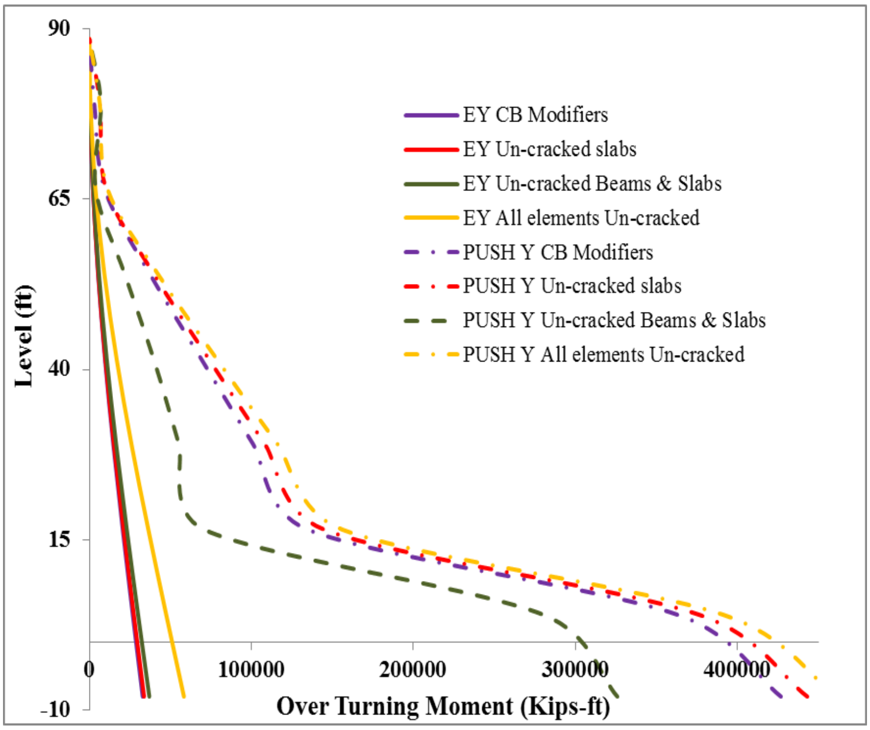


FIGURE A.9: Over turning moment comparison for EY and Push-Y case for soil type 'SB'.

Annexure-4

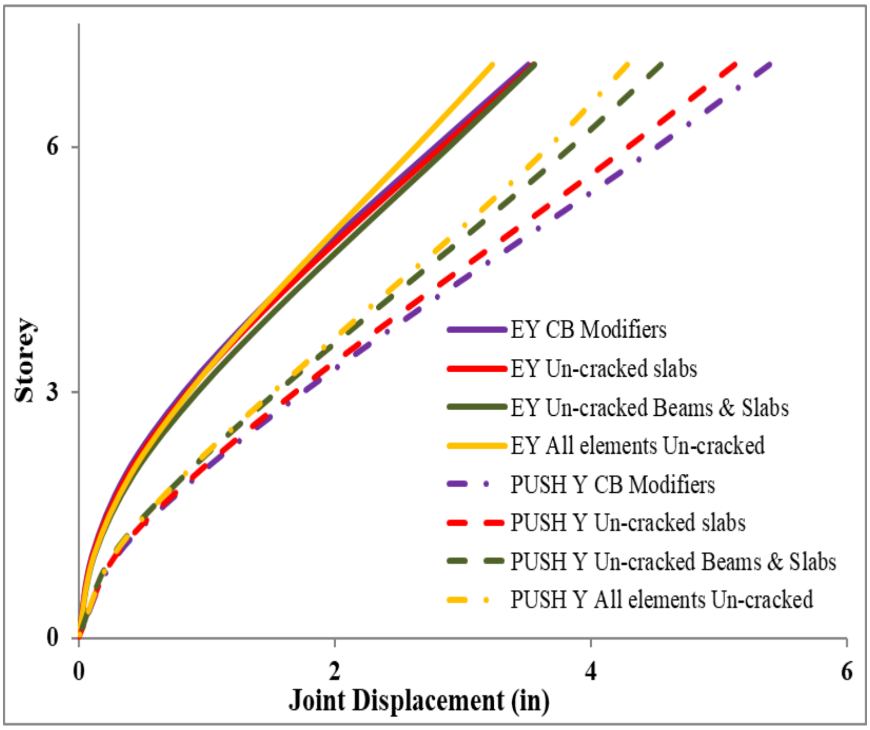


Figure A.10: Storey displacements comparison for EY and Push-Y case for soil type 'SD'.

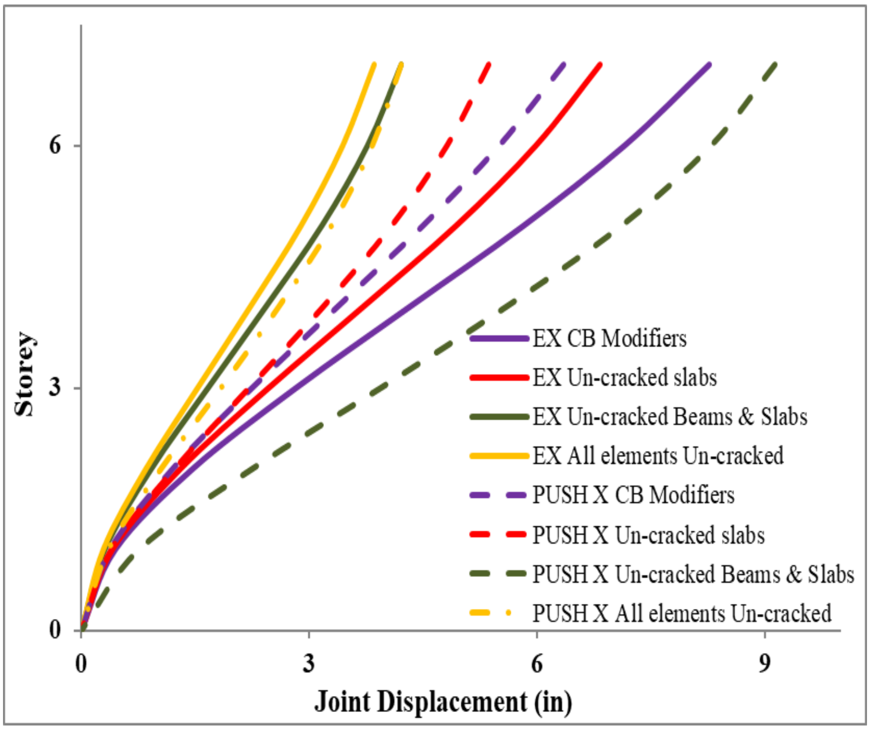


Figure A.11: Storey displacements comparison for EX and Push-X case for soil type 'SB'.

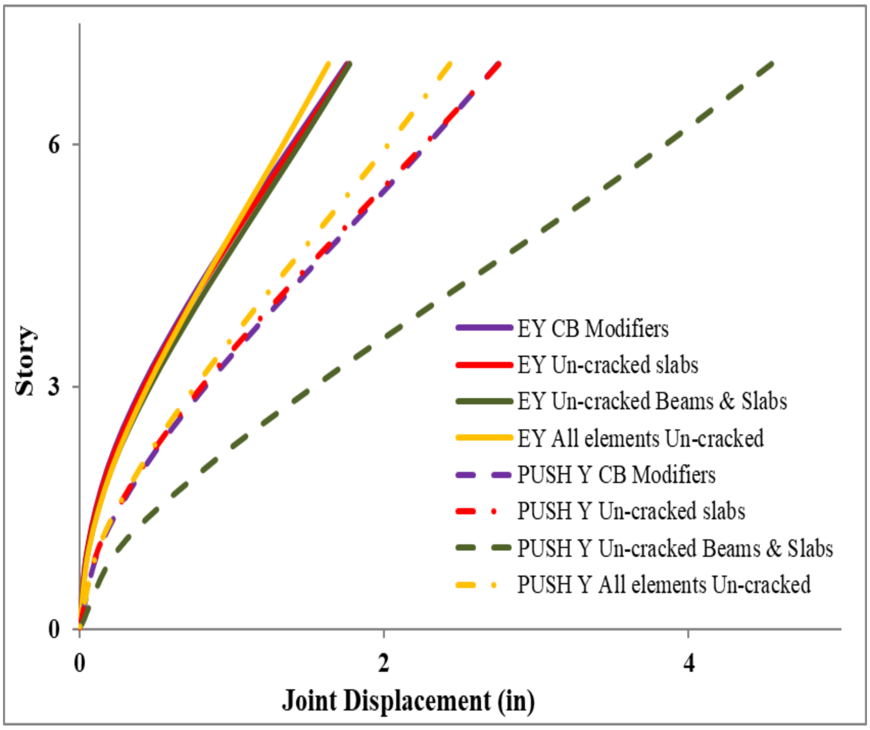


Figure A.12: Storey displacements comparison for EY and Push-Y case for soil type 'SB'.

Annexure-5

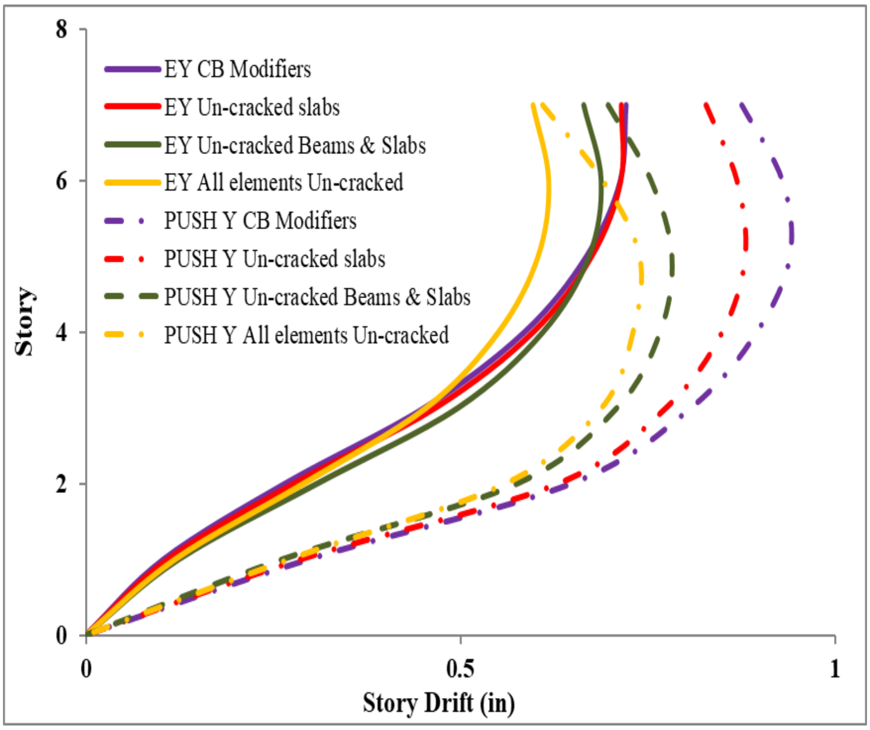


Figure A.13: Storey drift comparison for EY and Push-Y case for soil type 'SD'.

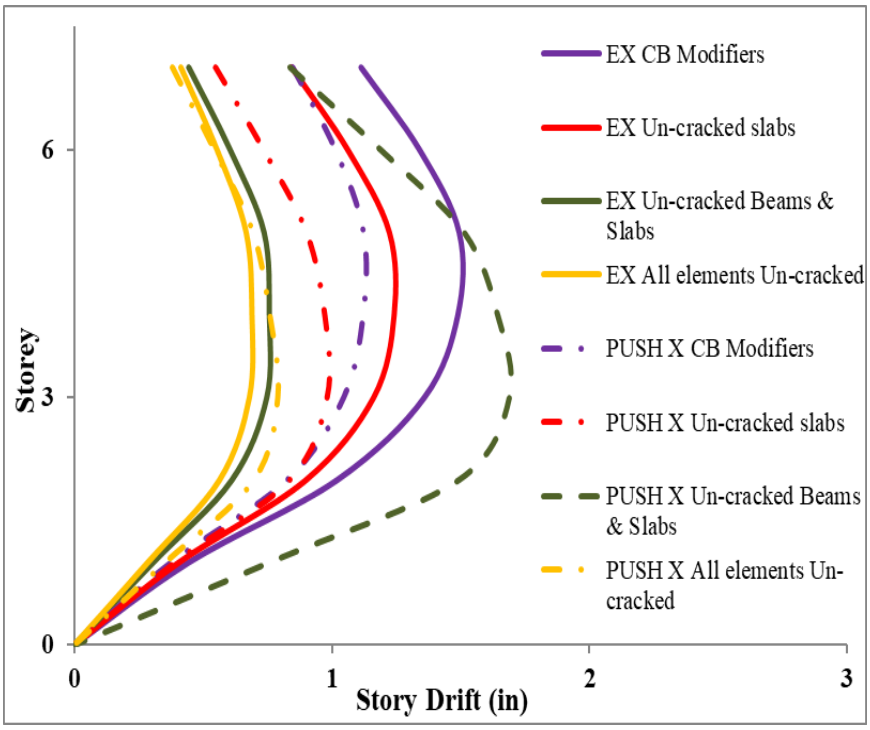


FIGURE A.14: Storey drift comparison for EX and Push-X case for soil type 'SB'.

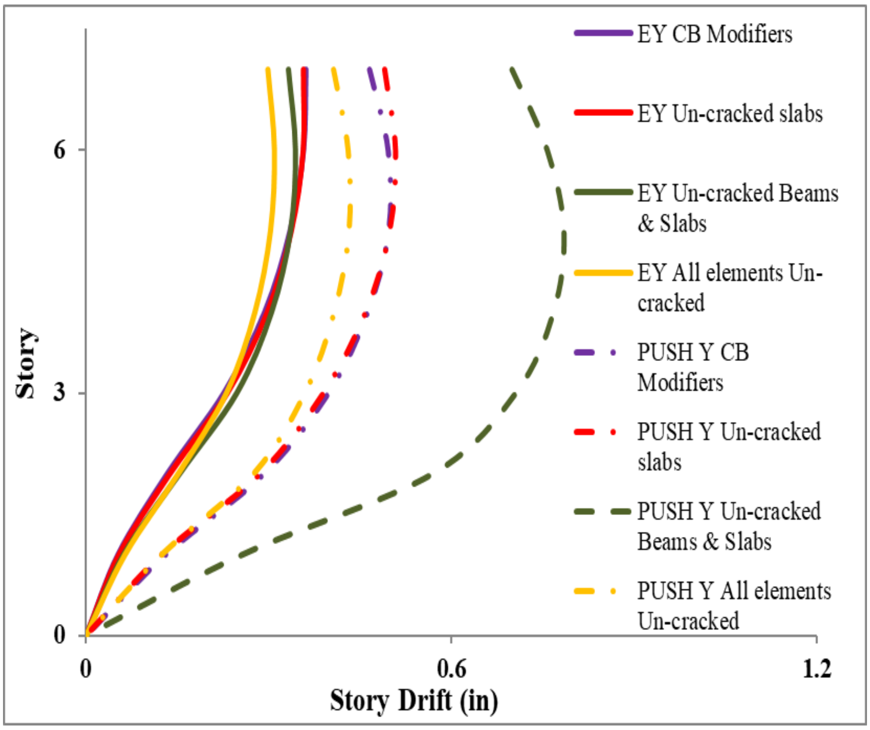
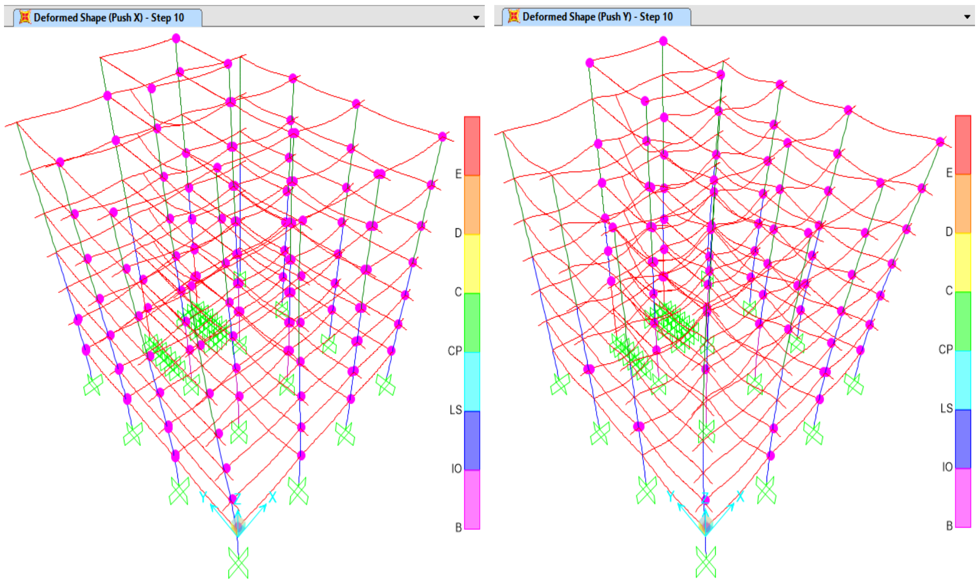


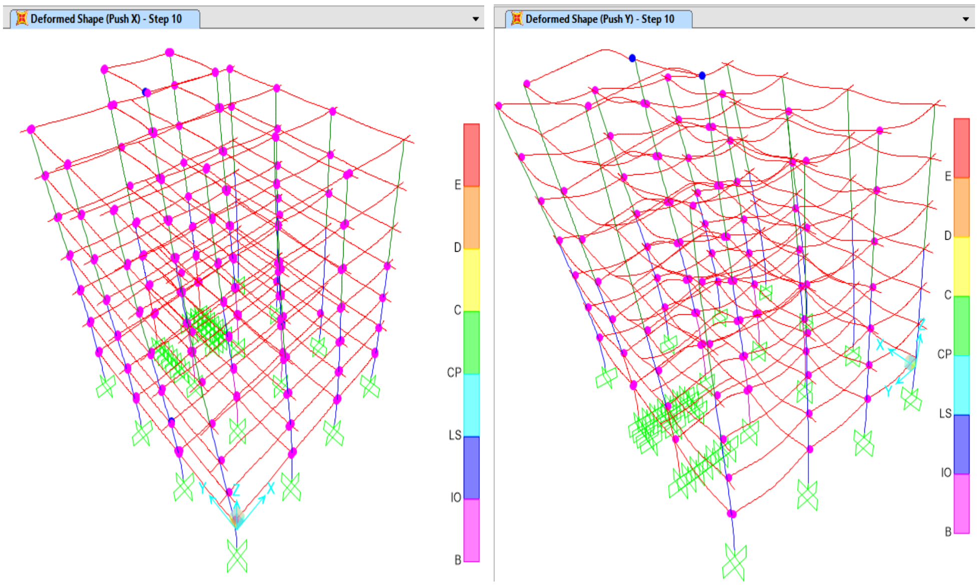
FIGURE A.15: Storey drift comparison for EY and Push-Y case for soil type 'SB'.

Annexure-6



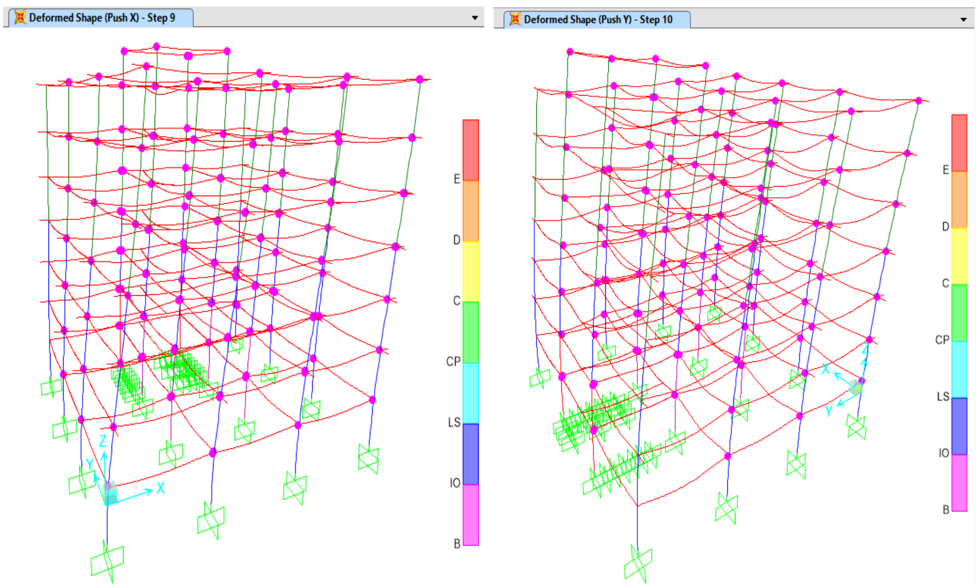
(a) (b)

FIGURE A.16: Formation of plastic hinges in Set-1 code-based stiffness modifiers (a) Push-X (b) Push-Y for soil ‘SB’.



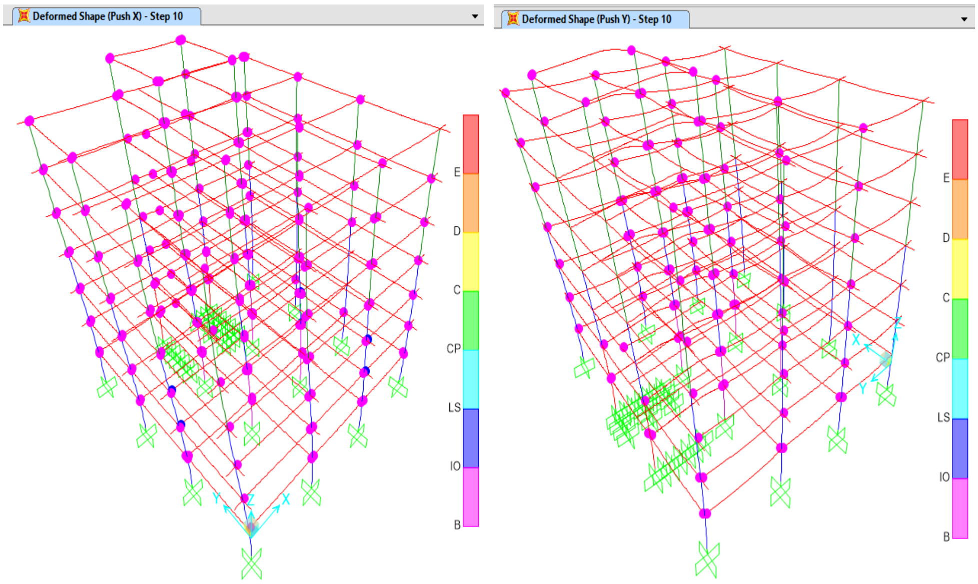
(a) (b)

FIGURE A.17: Formation of plastic hinges in Set-2 uncracked slabs case (a) Push-X (b) Push-Y for soil 'SD'.



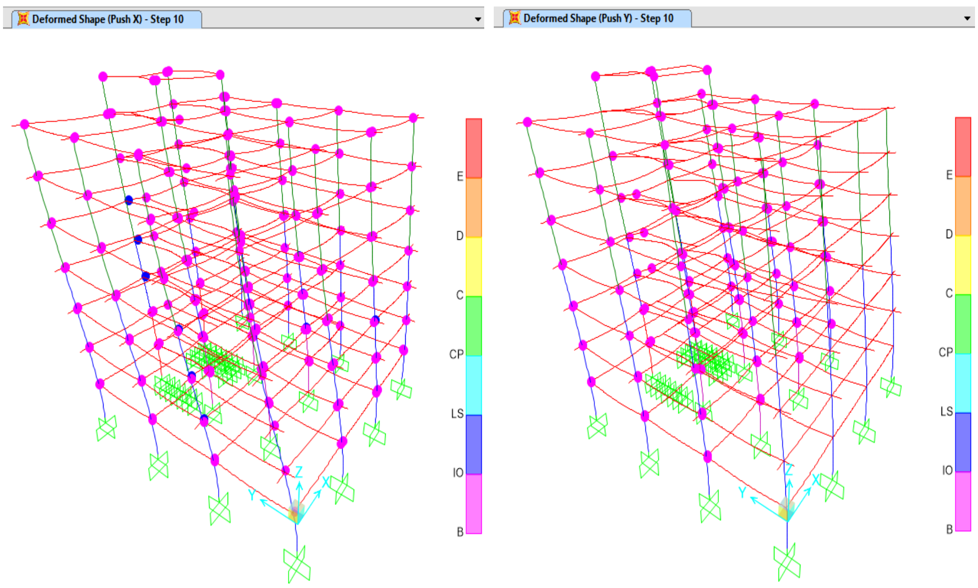
(a) (b)

FIGURE A.18: Formation of plastic hinges in Set-2 uncracked slabs case (a) Push-X (b) Push-Y for soil 'SB'.



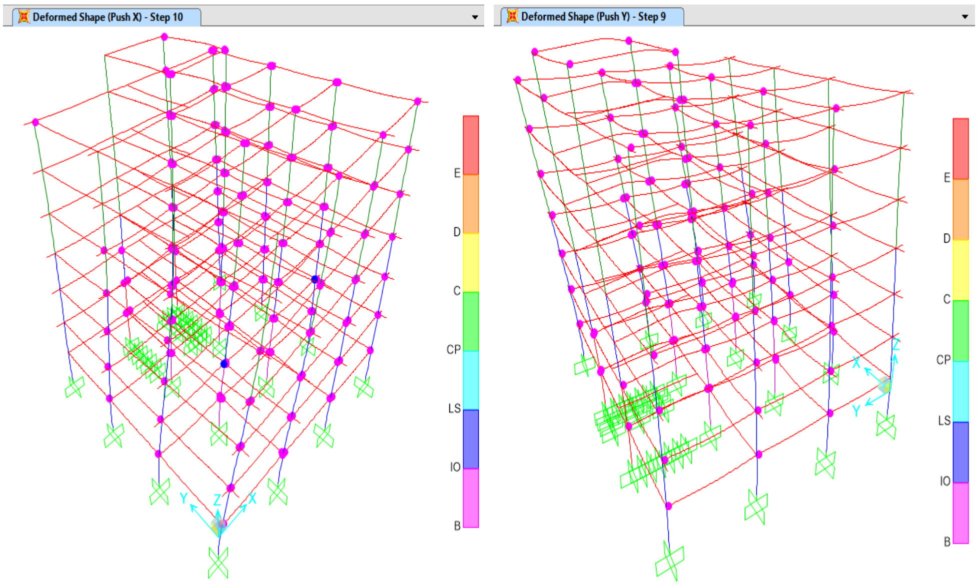
(a) (b)

FIGURE A.19: Formation of plastic hinges in Set-3 uncracked slabs & beams (a) Push-X (b) Push-Y for soil 'SD'.



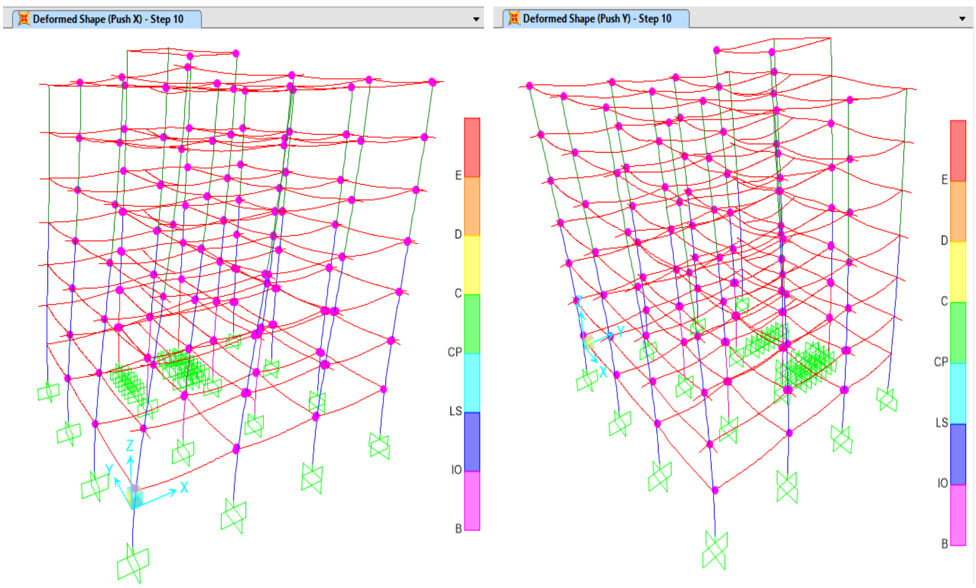
(a) (b)

FIGURE A.20: Formation of plastic hinges in Set-3 uncracked slabs & beams (a) Push-X (b) Push-Y for soil 'SB'.



(a) (b)

FIGURE A.21: Formation of plastic hinges in Set-4 all elements uncracked (a) Push-X (b) Push-Y for soil 'SD'.



(a) (b)

FIGURE A.22: Formation of plastic hinges in Set-4 all elements uncracked (a) Push-X (b) Push-Y for soil 'SB'.