



**ADDIS ABABA INSTITUTE OF TECHNOLOGY  
SCHOOL OF GRADUATE STUDIES**

# **Fiber Based Pushover Analysis of Reinforced Concrete Frame**

**(A case study on the structural performance assessment of  
existing School Building in Addis Ababa city)**

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the  
Requirements for the Degree of Master of Science in Civil Engineering  
(Structures)

by

Shewangizaw Tesfaye

Adviser Dr. Esayas Gebreyohannes

(February 2011)



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Approved by Board of Examiners

Dr Esayas Gebreyohannes \_\_\_\_\_

Advisor

\_\_\_\_\_  
Signature

\_\_\_\_\_  
Date

Dr \_\_\_\_\_

External Examiner

\_\_\_\_\_  
Signature

\_\_\_\_\_  
Date

Dr \_\_\_\_\_

Internal Examiner

\_\_\_\_\_  
Signature

\_\_\_\_\_  
Date

Ato \_\_\_\_\_

Chairman

\_\_\_\_\_  
Signature

\_\_\_\_\_  
Date

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## Abstract

There is a significant variation in seismic zoning adopted in Ethiopia for urban areas between the seismic building code (EBCS-8:1995) and those reported in other research publications. Adopting a conservative value is, therefore, crucial for particularly in places such as Ethiopia where quality control in the construction industry is not at adequate level yet and reinforcement detailing in practice may not complying with the codes. This paper deals with structural performance assessment of a G+4 ribbed slab School Building (Reinforced Concrete Frame) as a case study located in Addis Ababa City, Ethiopia where proposed peak ground acceleration value by RADIUS 1999 project and others is more than twice as of EBCS-8:1995 (RADIUS 1999 project).

Two verification problems: rectangular reinforced concrete column at element level and six story four bay regular reinforced concrete planar frame have been taken to check the reliability of fiber beam column model and the validation of software outputs through the performance assessment of reinforced concrete buildings using conventional pushover analysis. Material nonlinearity behavior was included through Mander's confined concrete compression model with tension stiffening effect; and bilinear steel reinforcement model with some strain-hardening effect. There is some variation between capacity curves from Response2000 and SAP2000 with the experiment result at element level while the result from Seismostruct is acceptably similar. Furthermore, Seismostruct (for both uniform and triangular lateral load patterns) give nearly similar result with the reference capacity curves for the frame while the capacity curves from SAP2000 V14 (not licensed) are not comparable. Pushover curve results of SAP2000 V14 and the research application software Seismostruct revealed that they are not comparable. Therefore, the structural performance assessment was done based on Seismostruct results. The results show that capacity of the case study building is below the force demand for PGA of 0.1g and 0.05g for all soil class and soil class C respectively at life safety structural performance level set by ATC-40. The building complies with inter-story and roof displacement requirements for demand of 0.05g. Since it is possible only to capture flexural failures in fiber model, premature failure (e.g. shear) was checked before criterion are reached.

**Keywords:** Buildings, earthquakes, performance, life safety, safety factor, seismic codes, material model, tension stiffening model, fiber elements, capacity curve, Peak Ground Acceleration.

## Contents

<b>Acknowledgements</b> .....	<b>I</b>
<b>Abstract</b> .....	<b>II</b>
<b>List of Figures</b> .....	<b>V</b>
<b>List of Tables</b> .....	<b>VII</b>
<b>Notations</b> .....	<b>VIII</b>
<b>CHAPTER ONE INTRODUCTION</b> .....	<b>1</b>
1.1 OBJECTIVE AND SCOPE.....	2
1.1.1 Statement of the problem.....	2
1.1.2 Objective of the study .....	3
1.1.3 Scope of the Study .....	3
<b>CHAPTER TWO LITRETURE REVIEW</b> .....	<b>4</b>
<b>CHAPTER THREE ZONATION OF ADDIS ABABA</b> .....	<b>11</b>
3.1 Background.....	11
3.2 Seismic Zoning for Ethiopia.....	11
3.3 Proposed Interim Solutions.....	13
<b>CHAPTER FOUR PUSHOVER ANALYSIS</b> .....	<b>15</b>
4.1.1 Use of Pushover Results.....	15
4.1.2 Limitations of Pushover Analysis .....	16
4.1.3 Pushover Analysis – SAP2000 .....	17
4.1.4 Assumptions .....	18
4.1.5 Analysis Parameters .....	18
4.1.6 Non-linear properties.....	19
4.1.7 Non-linear hinges.....	19
4.2 Vertical Loads.....	19
4.3 Lateral Load Pattern.....	19
4.4 Capacity Spectrum Method .....	20
4.5 Calculating Performance Point .....	20
<b>CHAPTER FIVE VERIFICATION</b> .....	<b>21</b>
5.1 Column Verification.....	25
5.1.1 Verification Data.....	25
5.1.2 Verification column model.....	26
5.1.3 Seismostruct Response2000 and SAP2000 Results.....	26
5.2 Frame Verification .....	28

5.2.1	Verification Data.....	28
5.2.2	Verification Frame Model.....	29
5.2.3	Seismostruct and SAP2000 Results.....	29
<b>CHAPTER SIX PERFORMANCE ASSESSMENT OF THE CASE STUDY BUILDING</b>		
6.1	Case Study Building – Description and Details.....	32
6.1.1	Case Study Building.....	32
6.1.2	Building Description.....	32
6.1.3	Structural System Design Details.....	33
6.2	Results and Discussion.....	37
6.2.1	Seismostruct.....	37
6.2.2	Check for Shear.....	44
6.2.3	SAP2000.....	46
<b>CHAPTER SEVEN CONCLUSION AND RECOMMENDATIONS.....</b>		<b>52</b>
<b>References.....</b>		<b>53</b>
<b>APPENDIX-A Plastic Hinge Locations.....</b>		<b>55</b>
<b>APPENDIX-B Seismic Performance Criteria.....</b>		<b>60</b>
<b>APPENDIX-C FORTRAN 95 Code: Moment-Curvature Relationship.....</b>		<b>63</b>
<b>APPENDIX-D Section Details.....</b>		<b>70</b>

## List of Figures

Figure 2-1: Static Approximation Used In the Pushover Analysis .....	4
Figure 2-2: Building Performance Level.....	4
Figure 2-3: Response Spectrum .....	5
Figure 2-4: Target Displacement .....	5
Figure 2-5: Lumped Plasticity Elements: (a) Parallel Model (Clough and Johnston, 1966); (b) Seies Model (Giberson, 1967) [10].....	9
Figure 2-6: Hinge Behaviour Curve (Figure Adopted from CSI, 2005).....	10
Figure 2-7: Fiber Element: (a) Distribution of Control Sections; (b) Section Subdivision [10] .....	10
Figure 3-1: Seismic zoning of Ethiopia as per Gouin (1976) [6] .....	12
Figure 3-2: Seismic zoning of Ethiopia as per ESCP-1:1983 [6] .....	12
Figure 3-3: Seismic zoning of Ethiopia as per EBCS-8:1995 [6].....	13
Figure 3-4: Seismic zoning of Addis Ababa as per RADIUS Project (1999) [6].....	13
Figure 4-1: Global Capacity (Pushover) Curve of a Structure .....	15
Figure 5-1: Mander’s model for confined and unconfined concrete.....	21
Figure 5-2: Model for reinforcing steel.....	21
Figure 5-3: Stress distribution and the space-averaged tension Stiffening model [12] .....	23
Figure 5-4: Takeda Hysteresis Model .....	24
Figure 5-5: Verification Column.....	26
Figure 5-6: Verification Column Model.....	26
Figure 5-7: Seismostruct Capacity Curve and Column at Steel Rupture .....	27
Figure 5-8: Verification Column Results of Seismostruct, Response2000 and SAP2000.....	27
Figure 5-9: Verification Frame .....	29
Figure 5-10: Verification Frame Models.....	29
Figure 5-11: Seismostruct Capacity Curves and Deformed Shapes at first Steel Rupture .....	30
Figure 5-12: Reference Capacity Curves and PCs of Seismostruct, Response2000 and SAP2000 .....	31
Figure 6-1: Floor plan.....	32
Figure 6-2: Front elevation .....	33
Figure 6-3: Section view.....	33
Figure 6-4: Three dimensional structural model.....	33
Figure 6-5: Foundation layout and frame of interest.....	34

Figure 6-6: Capacity Curve: Triangular/Opposite direction.....	37
Figure 6-7: Deflected shapes at first rupture of steel .....	38
Figure 6-8: Story displacement profiles .....	39
Figure 6-9: Inter-Story drift ratio profiles .....	40
Figure 6-10: Plastic hinges at Life Safety .....	42
Figure 6-11: Creep Rupture of Concrete under Concentric Loading.....	45
Figure 6-12: Capacity and Demand Curves (Triangular Pattern).....	46
Figure 6-13: Capacity and Demand Curves (Uniform Pattern).....	47
Figure 6-14: Pushover Curves: Triangular/opposite .....	48
Figure 6-15: Story displacement profiles .....	50
Figure 6-16: Inter-Story drift ratio profiles .....	51

## **List of Tables**

Table 3-1: Predicted PGA in Addis Ababa and Afar Depression as per different sources .....	14
Table 5-1: General structural dimension of the reference building .....	28
Table 6-1: Reinforcement Details for Columns and Beams .....	35
Table 6-2: Roof Displacement and Total Drift Ratio at Rupture of Steel.....	38
Table 6-3: Story-Drift Requirement .....	41
Table 6-4: Section Curvature limits for LS, in radians .....	42
Table 6-5: Factor of safety for PGA [0.05g] at Life Safety: Seismostruct.....	43
Table 6-6: Factor of safety for PGA [0.1g] at Life Safety: Seismostruct.....	43
Table 6-7: Factor of Safety for PGA [0.05g] at Steel Rupture: Seismostruct .....	43
Table 6-8: Factor of Safety for PGA [0.1g] at Steel Rupture: Seismostruct .....	44
Table 6-9: Performance Point PGA [0.05g] .....	48
Table 6-10: Performance Point PGA [0.1g] .....	49

## Notations

- $V_{Rd,c}$  is the design shear resistance of the member without shear reinforcement
- $V_{Rd,s}$  is the design value of the shear force which can be sustained by the yielding shear reinforcement
- $V_{Rd,max}$  is the design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts
- $A_{s1}$  is the area of the tensile reinforcement
- $b_w$  is the smallest width of the cross-section in the tensile area
- $N_{Ed}$  is the axial force in the cross section due to loading ( $N_{Ed} > 0$  for compression)
- $A_c$  is the area of concrete cross section
- $A_{sw}$  is the cross sectional area of the shear reinforcement
- $s$  is the spacing of the stirrups
- $f_{ywd}$  is the design yield strength of the shear reinforcement
- $\theta$  is the angle between the stirrup and the member axis
- $f_t$  is tensile strength of concrete
- $\varepsilon_{tu}$  is tensile cracking strain
- C and NC are abbreviations for conforming and non-conforming details
- IO Immediate Occupancy
- LS Life Safety
- SS Structural Stability
- DC Damage Control
- V Design Shear
- $V_i$  Shear force in story i
- $P_i$  Total gravity load at story i

## CHAPTER ONE INTRODUCTION

The need for changes in the existing seismic design methodology implemented in codes has been widely recognized. The structural engineering community has developed a new generation of design and rehabilitation procedures that incorporate performance based engineering concepts. This aim can be achieved only by introducing some kind of nonlinear analysis into the seismic design methodology. In a short term, the most appropriate approach seems to be a combination of the nonlinear static (pushover) analysis and the response spectrum approach. A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral forces, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness.

Using a pushover analysis, a characteristic nonlinear force - displacement relationship of the MDOF system can be determined. In principle, any force and displacement can be chosen. In this paper, base shear and roof (top) displacement have been used as representative of force and displacement, respectively. The selection of an appropriate lateral load distribution is an important step within the pushover analysis. A unique solution does not exist. Fortunately, the range of reasonable assumptions is usually relatively narrow and, within this range, different assumptions produce similar results. One practical possibility is to use two different displacement shapes (load patterns) and to envelope the results. Nonlinear static (pushover) *analysis* can provide an insight into the structural aspects which control performance during severe earthquakes. The analysis provides data on the strength and ductility of the structure which cannot be obtained by elastic analysis. Furthermore, it exposes design weaknesses that may remain hidden in an elastic analysis. Pushover analysis is based on a very restrictive assumption, i.e. a time-independent displacement shape. Thus, it is in principle inaccurate for structures where higher mode effects are significant, and it may not detect the structural weaknesses which may be generated when the structure's dynamic characteristics change after the formation of the first local plastic mechanism. The softening post peak load-deflection relation for cracking reinforced concrete beams and frames is analyzed by layered finite elements. Concrete is assumed to exhibit strain softening in both tension and compression, and the steel reinforcement is elastic-plastic. The bending theory assumptions are used and bond slip of reinforcement is neglected.

Current Ethiopian codes do not address the evaluation of seismic resistance of existing building stock, which may be expected to suffer and result in loss of life and money in the future earthquake forces. An appropriate level of safety needs to be ensured for occupants of these buildings through strengthening measures, if found deficient. This paper used a conventional nonlinear static (pushover) analysis method to assess the capacity of an existing building to reach an adequate level of performance related to the life safety of its occupants. Seismostruct and SAP2000 V14 Finite Element Package are used for seismic evaluation and the software are verified by taking single RC column and 6 story four bay regular RC planar frame as verification problem.

Many models for the non-linear analysis of RC frames are proposed in the literature. The choice of the most suitable model depends on the goals of the analysis and by the structural properties. For structures with flexural collapse mechanisms, member by member or global models can be used to obtain reliable predictions. A good balance between computational effort and level of reliability of the results should be achieved in choosing the model by taking into account the amount of basic information that each model requires. In this thesis Discrete Finite Member Models is used with fiber discretization model for beam and column in Seismostruct; and fiber discretization model for beam columns and lumped plasticity model for beams in SAP2000 V14.

## **1.1 OBJECTIVE AND SCOPE**

### **1.1.1 Statement of the problem**

Currently, building's structural design practice for earthquake in Ethiopia is based on the rules and regulations of Ethiopian Building Code Standards as a reference. The proposed seismic data (Peak Ground Acceleration) to Addis Ababa by EBCS-8 is very low compared to various research projects which depict the need for code revision and give interim solution until seismic zonation based on recent and strong data is done. Furthermore, in the design and construction process the structural design may not be checked rigorously by the respective authority which aggravates the problem in earthquake performance of buildings.

## **1.1.2 Objective of the study**

### **1.1.2.1 General Objective**

The aim of this study is to propose a way for structural performance assessment of existing reinforced concrete frame buildings. A case study building located in Addis Ababa city is taken to assess its performance for the future earthquake expected to occur as per EBCS-8 and proposed interim solution data.

### **1.1.2.2 Specific Objectives**

- To verify the reliability of fiber model and results of software (SAP2000 V14, Seismostruct, Response 2000) in structural performance evaluation using conventional pushover analysis:
  - To verify the results of SAP2000 and Seismostruct (for column and frame) and Response2000 (for column).
  - Compare results of SAP2000 with that of Seismostruct
- To evaluate the global and element level structural performance according to the performance level set by ATC-40 and find safety factor of the case study building considering different parameters using Seismostruct and SAP2000 V14.
- To evaluate progress of damages.
- Check failures against different criteria.

### **1.1.3 Scope of the Study**

Though the structural performance assessment of the case study building applying fiber beam column model and conventional pushover analysis based on the working drawing data; the following considerations were not taken in to account: existing damages, soil structure interaction, effects of walls, bond slip.

## CHAPTER TWO LITRETURE REVIEW

The Non – Linear Static Procedure or Pushover Analysis is defined in the Federal Emergency Management Agency document 273 (FEMA 273) as a non – linear static approximation of the response a structure will undergo when subjected to dynamic earthquake loading. The static approximation consists of applying a vertical distribution of lateral loads to a model which captures the material non–linearity of an existing or previously designed structure, and monotonically increasing those loads until the peak response of the structure is obtained on a base shear vs. roof displacement plot as shown in Figure 2-1.

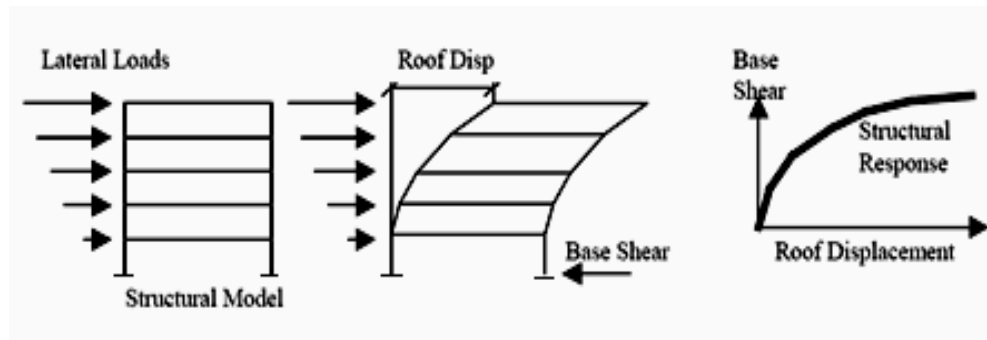


Figure 2-1: Static Approximation Used In the Pushover Analysis

The desired condition of the structure after a range of ground shakings, or Building Performance Level, is then decided upon by the owner, architect, and structural engineer. The Building Performance Level is a function of the post event conditions of the structural and non – structural components of the structure. Some common Building Performance Levels are shown in Figure 2-2.

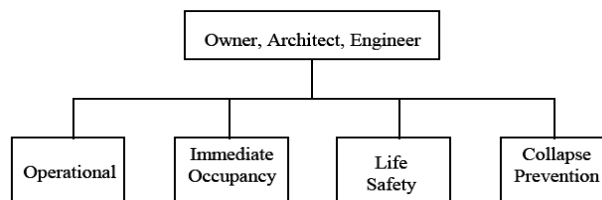


Figure 2-2: Building Performance Level

Based on the desired Building Performance Level, the Response Spectrum for the design earthquake may be determined. The Response Spectrum gives the maximum acceleration, or Spectral Response Acceleration, a structure is likely to experience under the design ground shaking given the structure’s fundamental period of vibration, T. This relation is shown qualitatively in Figure 2-3.

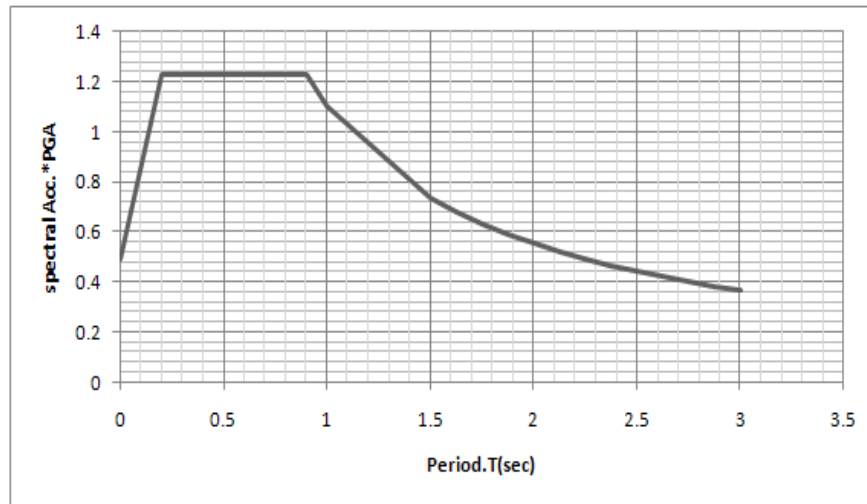


Figure 2-3: Response Spectrum

From the Response Spectrum and Base Shear vs. Roof Displacement plot, the Target Displacement,  $\delta_t$ , may be determined. The Target Displacement represents the maximum displacement the structure will undergo during the design event. One can then find the maximum expected deformations within each element of the structure at the Target Displacement and redesign them accordingly. The Target Displacement is shown qualitatively in Figure 2-4.

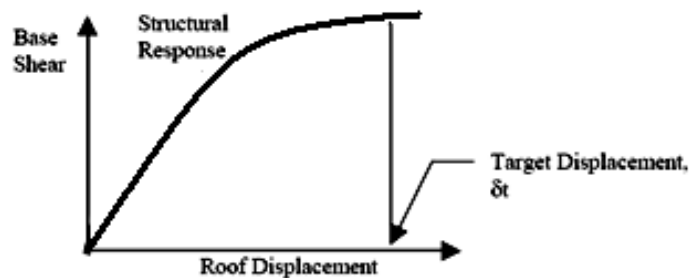


Figure 2-4: Target Displacement

The analysis involves applying horizontal loads, in a prescribed manner, to a model of the structure incrementally; i.e. “pushing “the structure; and plotting the total applied shear force and associated lateral displacement at each increment, until the structure reaches a limit state or collapse condition. In this method, forcing functions, expressed either in terms of horizontal forces or displacements, are applied to the lateral action - resisting system. Static forces or displacements are distributed along the height of the structure so as to simulate the inertia forces or their effects. The forcing functions are increased in intensity and the pushover analysis (PA) terminates when the ultimate capacity corresponding to a set of ultimate limit

states reached. These forcing functions correspond to one or more mode shapes. If the pattern of forcing function (loads or displacements) is kept constant throughout the analysis, the method is referred to as conventional pushover. If the pattern changes to account for variations in the mode shapes of the structure in the inelastic range, the method is referred to as adaptive pushover.

Conventional pushover is an inelastic static analysis method in which the idealized representation of the structure is subjected to constant gravity loads and to monotonically increasing lateral force or displacement pattern (also termed 'forcing function') of a constant shape. Because the structural model accounts directly for effects of both material inelasticity and geometric non-linearity, the Pushover analysis is a capacity estimation method under a set of functions that represent inertial effects from the earthquake. This method is capable of shedding light on design weaknesses that elastic analysis cannot detect. For example in the equivalent static analysis or in the simplified code method, weaknesses such as storey mechanisms cannot be readily detected. However, such satisfactory predictions of seismic demands are mostly restricted to low and medium-rise structures in which inelastic action is distributed throughout the height of the structure [Krawinkler and Seneviratna, 1998; Gupta and Krawinkler, 1999]. [1]

None of the invariant force distributions can account for the contributions of higher modes to response, or for a redistribution of inertia forces due to structural yielding and the associated changes in the vibration properties of the structure. To overcome these limitations, several researchers have proposed adaptive force distributions that attempt to follow more closely the time-variant distributions of inertia forces [Fajfar and Fischinger, 1988; Bracci et al. 1997; Gupta and Kunnath, 2000]. While these adaptive force distributions may provide better estimates of seismic demands [Gupta and Kunnath, 2000]; they are conceptually complicated and computationally demanding for routine application in structural engineering practice. Attempts have also been made to consider more than the fundamental vibration mode in pushover analysis [paret et al., 1996; Sasaki et al., 1998; Gupta and Kunnath, 2000; Kunnath and Gupta, 2000; Matsumori et al. 2000]. [1]

A modal pushover analysis (MPA) procedure that includes the contributions of several "modes" of vibration, thus providing superior accuracy in estimating inelastic seismic demands on buildings, while retaining the conceptual simplicity and computational attractiveness of the procedure with invariant force distribution, has recently been developed [Chopra and Goel, 2001, 2002]. It seems reasonable because the lateral force distribution for

each mode appears to be the most rational choice among all invariant distributions of forces and it provides results for elastic buildings that are identical to the well-known response spectrum analysis (RSA) procedure.

Since no single load pattern can capture the variability in the local demands expected in a design earthquake, the use of at least two load patterns that are expected to bound inertia force distributions is recommended (Krawinkler, 1998). Most common load distributions are, a uniform distribution, a linear load distribution and that recommended by the FEMA 356 guidelines in which the normalized story load is a function of the floor height,  $h$ , and the fundamental period of the structure. The load pattern suggested by FEMA 356 applies increased lateral forces to the upper levels of the building. This distribution is intended to capture the higher mode effects in the seismic response. The results of the pushover analyses using the baseline model suggest that the predicted failure mechanism is relatively insensitive to the load distribution. A similar failure mechanism was observed for different load distributions (uniform, triangular and FEMA 356). [2]

The Nonlinear Static Procedure (NSP) may be used for any structure and any Rehabilitation Objective, with the following exceptions and limitations. [4]

- The NSP should not be used for structures in which higher mode effects are significant. To determine if higher modes are significant, a modal response spectrum analysis should be performed for the structure using sufficient modes to capture 90% mass participation, and a second response spectrum analysis should be performed considering only the first mode participation. Higher mode effects should be considered significant if the shear in any story calculated from the modal analysis considering all modes required to obtain 90% mass participation exceeds 130% of the corresponding story shear resulting from the analysis considering only the first mode response.
- The NSP should not be used unless comprehensive knowledge of the structure has been obtained.

There are two reasons why the Pushover Analysis may be preferred to a full dynamic analysis. The first reason is computational time. To run a full dynamic, non – linear analysis on even a simple structure takes a long time. If the Pushover Analysis is deemed applicable to the structure at hand, accurate results can be obtained in fractions of the time it would take to get any useful results from the fully dynamic analysis. This makes the Pushover Analysis much more applicable in a design office. The second reason has to do with earthquake

unpredictability. When performing a dynamic analysis, it is best to use a series of earthquakes. This further increases the computational time. If we were to redesign a structure based on a maximum displacement achieved from a full dynamic analysis based on one particular earthquake, it is easy to imagine that there could be an earthquake which had the same probability of exceedance but had different frequency content. Based on the fundamental period of the structure, this would increase or decrease the maximum response. So, one would not know if the design was the maximum that could be expected until a great number of earthquake ground motion records were tested. The Pushover Analysis naturally accounts for all earthquakes with the same probability of exceedance by predicting the maximum displacement that can be expected in the form of the Target Displacement. Now, computational time has been further reduced, since only one analysis must be run for each exceedance probability that the designer is interested in, strengthening the idea that the Pushover Analysis is much more practical in a design office.

Significant effort has been devoted, in the last several decades, in developing models for accurate simulation of the behavior of reinforced concrete frame elements. One of the earliest motivations for this was the desire to simulate the behavior of reinforced concrete elements subjected to seismic excitations. In some cases, it was the desire to assess the remaining capacity of a structure after a strong ground motion. The determination of the behaviour of structural components was essential for the assessment of the inelastic response of the complete structure. The initial stiffness, ultimate capacity and ductility demand were some of the parameters needed for this purpose. These difficulties have largely been overcome by static tests on structural components (e.g., beams, columns and shear walls) and small-scale structural subassemblies (e.g., beam-column joints) under cyclic load reversals.

Several models have been proposed to date for the simulation of the nonlinear behavior of reinforced concrete frame structures. These range from simple nonlinear springs which lump the behavior of an entire storey into a one degree-of-freedom system to complex three-dimensional finite element formulations that describe the structural behaviour by integrating the stress-strain relationships of the constituent materials (Filippou and Issa, 1988). These nonlinear models can be categorized, in a more broad sense, into three categories: [10]

(1) Global Models: These models constitute the simplest form of all nonlinear models, requiring the least computational power. In these models, the nonlinear behaviour of the entire structure is concentrated at selected degrees of freedom.

(2) Discrete Finite Member Models: These models possess more advanced formulations compared to global models, and require more computational power. In these models, the structure is represented by an assemblage of interconnected elements that describe the nonlinear behavior of reinforced concrete members with Lumped Nonlinearity model (as shown in Figure 2-5) and Distributed Nonlinearity model.

(3) Microscopic Finite Element Models: These models possess the most advanced formulations developed to date for nonlinear analyses, requiring significant computational power and analysis time. In these models, members and joints are discretized into a large number of finite elements. Constitutive and geometric nonlinearity are typically accounted for at the stress-strain level or averaged over a finite region. Bond modelling between concrete and reinforcement, interface friction at the cracks, creep, relaxation, thermal effects and geometric crack discontinuities are among the physical nonlinearities usually considered by this class of models.

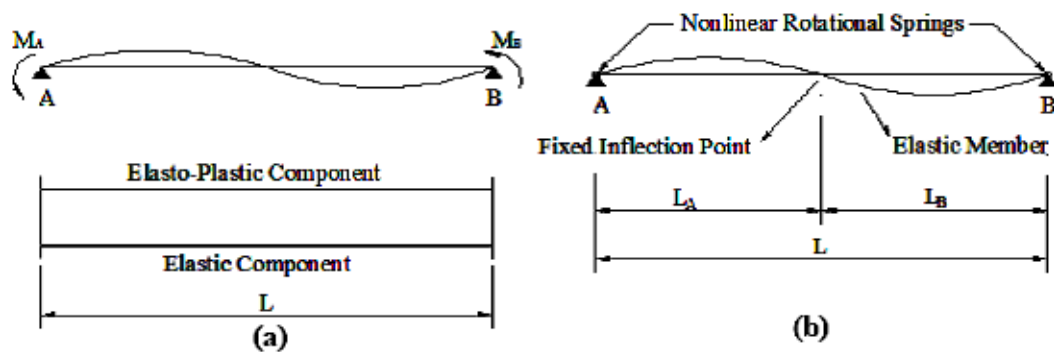


Figure 2-5: Lumped Plasticity Elements: (a) Parallel Model (Clough and Johnston, 1966); (b) Seis Model (Giberson, 1967) [10]

In this thesis Fiber model that constitutes the most advanced formulation in distributed nonlinearity models is used for beams and columns model in Seismostruct, and for columns in SAP2000 while user defined hinge property with non zero hinge length as of Figure 2-6 for beams were used. In Fiber model, the element is subdivided into longitudinal fibers as shown in Figure 2-7.

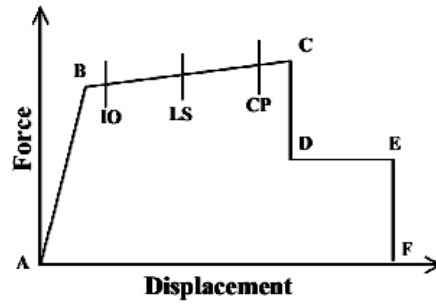


Figure 2-6: Hinge Behaviour Curve (Figure Adopted from CSI, 2005)

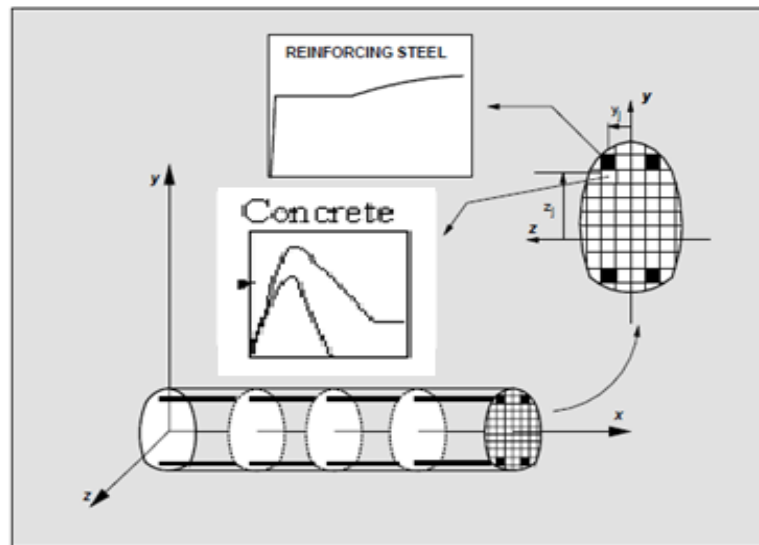


Figure 2-7: Fiber Element: (a) Distribution of Control Sections; (b) Section Subdivision [10]

The fiber location in the local  $y, z$  reference system and the fiber area  $A_{fib}$  are geometric characteristics of the cross section. The governing compatibility relationship is based on the “plane sections remain plane” hypothesis (Hooke, 1678; Bernoulli, 1705; Navier, 1826), which forms the basis of the engineering beam theory used in the sectional analysis of concrete members. Equilibrium is satisfied through integrating the responses of the fibers and equating them to the required sectional forces. Appropriate stress-strain relationships are used for the constituent materials so as to determine the stress distribution on the section for a given strain profile. [10] Fiber models are only able to consider flexural effects. In other words, the accurate simulation of reinforced concrete members dominated by shear or shear flexure cannot be achieved through these models.

## **CHAPTER THREE      ZONATION OF ADDIS ABABA**

### **3.1 Background**

Addis Ababa, capital of Ethiopia, was founded around 125 years ago and is located on western edge of the Ethiopian rift system. The area of the greater metropolitan city is about 54000 hectares with a population of around 3.2 millions. Several earthquakes have occurred along the rift and its vicinity and were felt in the city. Notable cases are:

- 1906 earthquake in Langano (epicenter 110 km from Addis Ababa) with an intensity of Mercalli Scale 8 in the city at a time when fewer than 50000 people were living in Addis Ababa; and [7]
- 1961 Kara Kore earthquake (epicenter 150 km from Addis Ababa) with an intensity of Mercalli scale 7 felt in Addis Ababa, which caused some damage in the city. [7]

There is a significant variation in seismic zoning adopted in Ethiopia for urban areas between the seismic building code (EBCS-8:1995) and those reported in other research publications. The seismic zone of an area determines the level of equivalent static forces that will be applied in a building frame in seismic design using the ESL (Equivalent Static Load) procedure. Likewise, the seismic zone also determines the level of dynamic loads to be considered for design based on response spectra analysis. However, among the published seismic zone maps for Ethiopia, there are distinct and significant differences in the predicted peak ground accelerations (PGA) for various seismically important regions of the country. It is therefore argued that, the variations in predicted PGA are significant enough to warrant a review of the seismic zoning of urban areas such as Addis Ababa.

### **3.2 Seismic Zoning for Ethiopia**

Gouin (1976) who used probabilistic approach is credited for the initial attempts in producing the first seismic hazard map of Ethiopia as shown in Figure 3-1. Gouin's work served as a basis for the seismic zoning adopted by the ESCP-1:1983 building code of Ethiopia (see Figure 3-2). Asfaw (1994) served as a basis for the seismic zoning adopted by the current Ethiopian building code - EBCS-8:1995 as shown in Figure 3-3. Further, there have been other attempts on seismic zoning of some of the country's important economic regions such as the city of Addis Ababa. The work of the RADIUS project (1999) is a notable example as shown in Figure 3-4. [6]

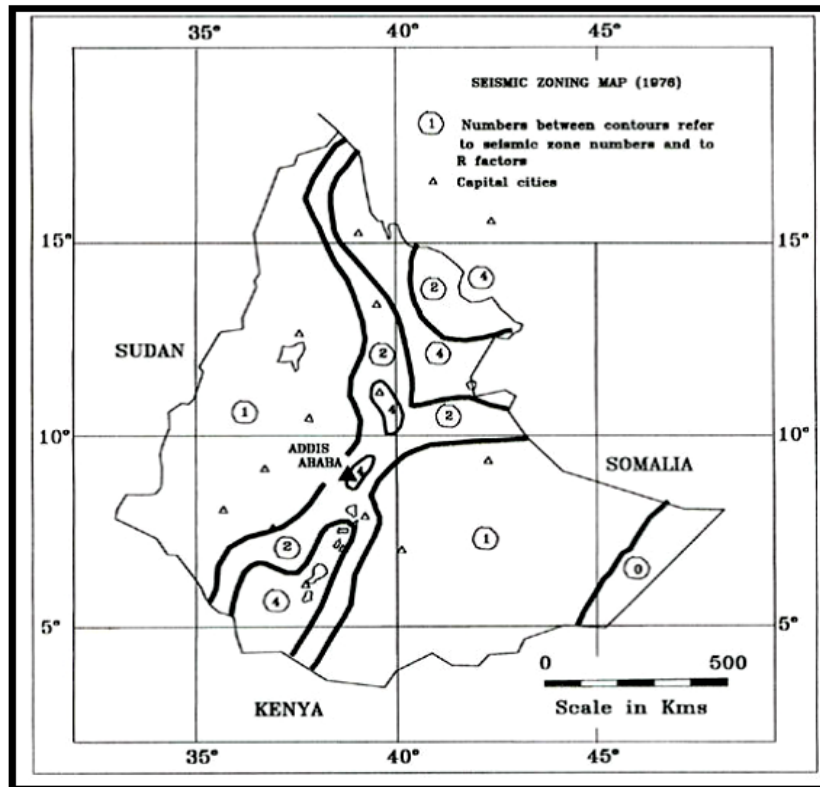


Figure 3-1: Seismic zoning of Ethiopia as per Gouin (1976) [6]

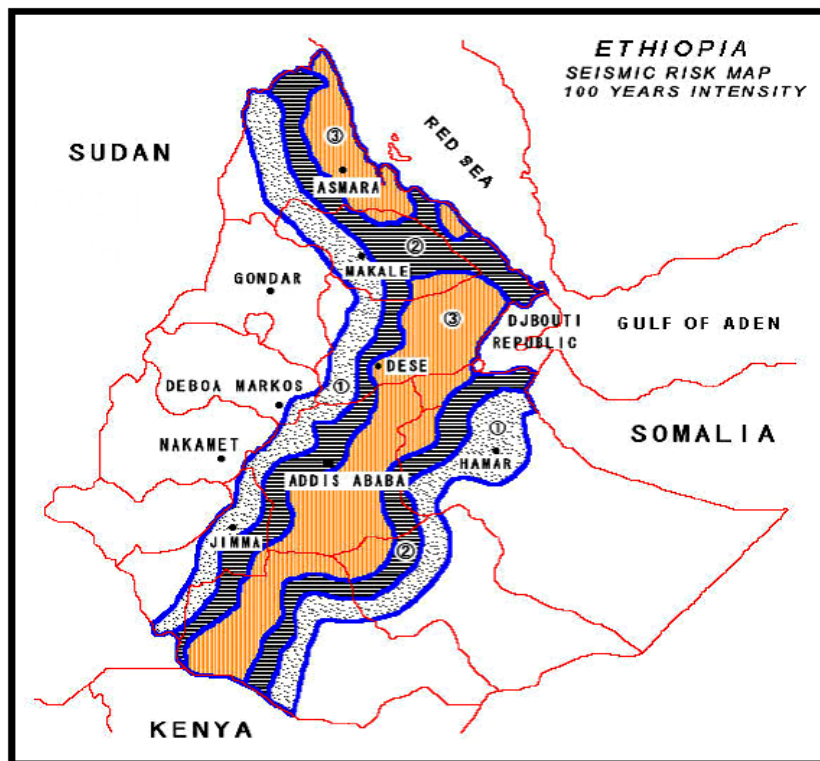


Figure 3-2: Seismic zoning of Ethiopia as per ESCP-1:1983 [6]

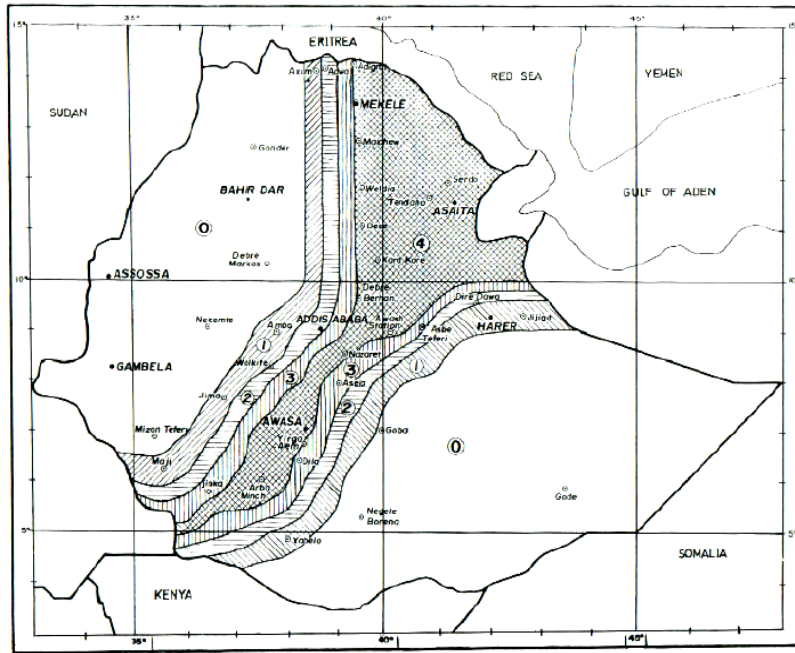


Figure 3-3: Seismic zoning of Ethiopia as per EBCS-8:1995 [6]

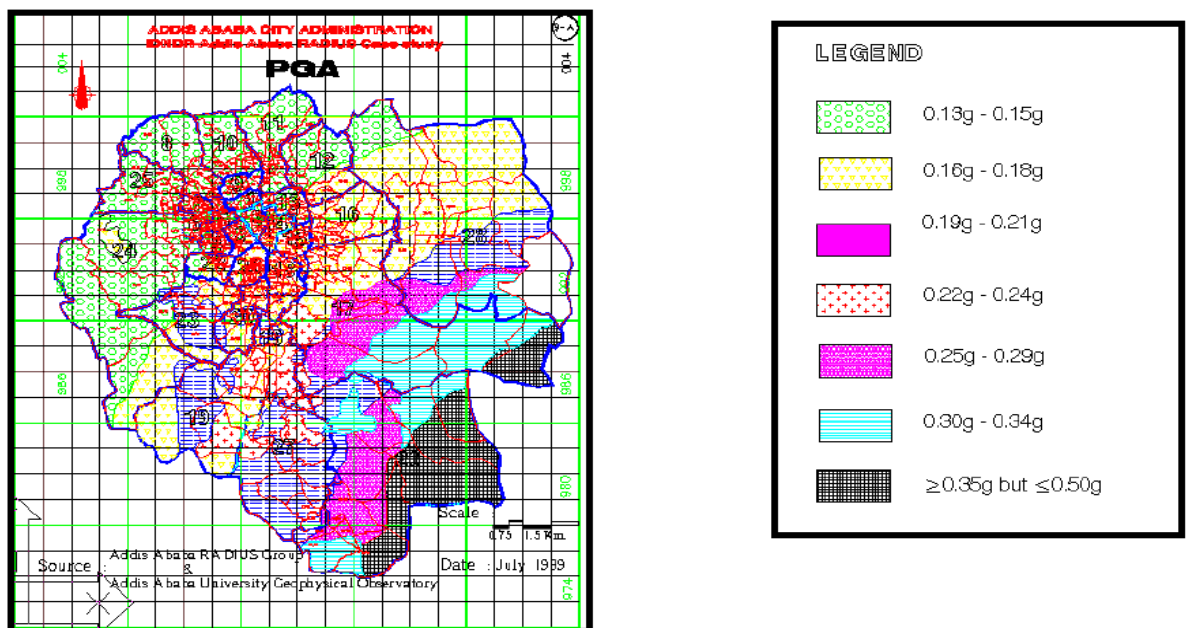


Figure 3-4: Seismic zoning of Addis Ababa as per RADIUS Project (1999) [6]

### 3.3 Proposed Interim Solutions

Further investigation and collaboration between seismologists and earthquake engineers is required before the controversy could be settled. For current design practice, until such information is widely available, however, it still remains difficult to support the current EBCS-8:1995 classifications that put the maximum peak ground acceleration in Addis Ababa

as 0.05g. In the same token, it also looks like the peak ground accelerations predicted by the RADIUS project are on the high end and are not supported by strong data. As a temporary solution, until further data is available, it is proposed that a more conservative zoning than the EBCS-8:1995 be adopted that increases the PGA to at least in range of PGA of 0.1g – 0.12g.

Table 3-1: Predicted PGA in Addis Ababa and Afar Depression as per different sources

		Kebede and Asfaw (1996)	EBCS-8:1995 (1995)	RADIUS (1999)	UBC-97 (1997)
<b>Addis Ababa</b>	PGA for 0.01p	0.08g	0.05g	NA	NA
	PGA for 0.005p	0.12g	NA	0.13g-0.5g (extrapolated)	0.3g
<b>Afar Depression</b>	PGA for 0.01p	0.14g/0.53g	0.16g	0.25g-0.75g (extrapolated)	NA
	PGA for 0.005p	0.19g/1.05g	NA		0.3g

P stands for 100 years return period and the corresponding values are probability of occurrence

## CHAPTER FOUR PUSHOVER ANALYSIS

Pushover analysis can be performed as force-controlled or displacement controlled. In force-controlled pushover procedure, full load combination is applied as specified, i.e, force-controlled procedure should be used when the load is known. Also, in force controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects. Generally, pushover analysis is performed as displacement-controlled to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

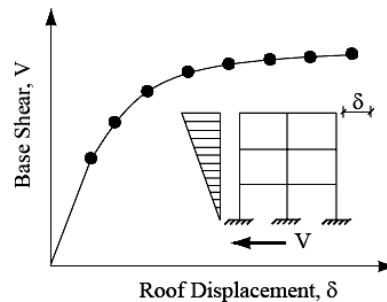


Figure 4-1: Global Capacity (Pushover) Curve of a Structure

### 4.1.1 Use of Pushover Results

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analysis provides information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis.

These are:

- estimates of inter-story drifts and its distribution along the height
- determination of force demands on brittle members, such as axial force demands on columns, moment demands on beam-column connections
- determination of deformation demands for ductile members
- identification of location of weak points in the structure (or potential failure modes)
- consequences of strength deterioration of individual members on the behavior of structural system
- identification of strength discontinuities in plan that will lead to changes in dynamic characteristics in the inelastic range
- verification of the completeness and adequacy of load path
- Pushover analysis also exposes design weaknesses that may remain hidden in an elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

#### **4.1.2 Limitations of Pushover Analysis**

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. In pushover analysis, the target displacement for a multi degree of freedom (MDOF) system is usually estimated as the displacement demand for the corresponding equivalent single degree of freedom (SDOF) system. The basic properties of an equivalent SDOF system are obtained by using a shape vector which represents the deflected shape of the MDOF system. Most of the researchers recommend the use of normalized displacement profile at the target displacement level as a shape vector but iteration is needed since this displacement is not known a priori. Thus, a fixed shape vector, elastic first mode, is used for simplicity without regards to higher modes by most of the approaches. And, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. Moreover, hysteretic characteristics of MDOF should be incorporated into the equivalent SDOF model, if displacement demand is affected from stiffness degradation or pinching, strength deterioration and P- $\Delta$  effects.

The above discussion on target displacement and lateral load pattern reveals that pushover analysis assumes that response of structure can be related to that of an equivalent SDOF system. In other words, the response is controlled by fundamental mode which remains

constant throughout the response history without considering progressive yielding. Although this assumption is incorrect, some researchers obtained satisfactory local and global pushover predictions on low to mid-rise structures in which response is dominated by fundamental mode and inelasticity is distributed throughout the height of the structure.

#### **4.1.3 Pushover Analysis – SAP2000**

SAP2000 provides the following tools needed for pushover analysis (CSI, 2008):

- Material nonlinearity at discrete, user-defined hinges in frame elements
- Nonlinear static analysis procedures specially designed to handle the sharp drop off in load carrying capacity typical of frame hinges used in pushover analysis
- Nonlinear static analysis procedures that allow displacement control, so that unstable structures can be pushed to desired displacement targets
- Display capabilities in the graphical user interface to generate and plot pushover curves, including capacity and demand curves in spectral ordinates
- Capabilities in the graphical user interface to plot and output the state of every hinge at each step in the pushover analysis

In addition to these, it is also possible to include non-linear link/support behavior, geometric nonlinearity and staged construction. Out of these, material and geometric nonlinearities were implemented in this study.

The general steps required to conduct pushover analysis using SAP 2000, as presented in (CSI, 2008), are as summarized here under.

- a. Create a model just like for any other analysis.
- b. Define frame hinge properties and assign them to the frame elements.
- c. Define any Load Patterns and static and dynamic Load Cases that may be needed for concrete design of the frame elements, particularly if default hinges are used.
- d. Run the Load Cases needed for design.
- e. If any concrete hinge properties are based on default values to be computed by the program, perform concrete design so that reinforcing steel is determined.
- f. Define the Load Patterns that are needed for use in the pushover analysis, including:
  - Gravity loads and other loads that may be acting on the structure before the lateral seismic loads are applied. These loads patterns may have already been defined.
  - Lateral loads that will be used to push the structure. If Acceleration Loads or modal

loads are going to be used, no new Load Patterns need to be defined, although modal loads require definition of a Modal Load Case.

- g. Define the nonlinear static Load Cases to be used for pushover analysis, including:
  - a. A sequence of one or more cases that start from zero and apply gravity and other fixed loads using load control. These cases can include staged construction and geometric nonlinearity.
  - b. One or more pushover cases that start from this sequence and apply lateral pushover loads. These loads should be applied under displacement control. The monitored displacement is usually at the top of the structure and will be used to plot the pushover curve.
- h. Run the pushover Load Cases
- i. Review the pushover results: Plot the pushover curve, the deflected shape showing the hinge states, force and moment plots, and print or display any other results needed.
- j. Revise the model as necessary and repeat.

#### **4.1.4 Assumptions**

The following assumptions were made to formulate the research problem and develop the analyses models

- a. Member failures due to inadequate anchorage and splicing were not considered as potential failures
- b. Rigid beam-column joints were assumed for modeling and the capacity of adjacent components was not limited by the joint strength
- c. No bond-slip
- d. No shear failure
- e. No infill wall effect and diaphragm action
- f. The design spectrum from EBCS-8:1995 was scaled by peak ground accelerations

#### **4.1.5 Analysis Parameters**

Sub-soil classification

- I. Sub-soil class A.
- II. Sub-soil class B.
- III. Sub-soil class C.

Peak Ground Acceleration

- I. PGA 0.05g
- II. PGA 0.1g

#### 4.1.6 Non-linear properties

Two types of non-linear properties were considered for the model. The first one, material non-linearity, was considered by introducing nonlinear material property. And the second, geometric non-linearity was taken into account by considering second order effects by the iterative P- $\Delta$  method (Seismostruct and SAP 2000).

#### 4.1.7 Non-linear hinges

SAP2000 presents different types of non-linear hinges in order to be able to account for the possible sources of non-linearity: uncoupled moment, torsion, axial force and shear hinges and also coupled P-M2-M3 hinges which yield based on the interaction of axial force and uni-axial or bi-axial bending moments at the potential hinge locations as stated in CSI, 2008. In this study user defined hinge properties (FORTRAN-95 code) were assigned for beam elements and user defined fiber elements for the columns in SAP2000 and fibers for beams and columns in Seismostruct.

#### 4.2 Vertical Loads

The vertical load assumed in the structural performance assessment was 75% of the combination of design dead load and design live load, i.e.:

$$\text{Vertical load} = 0.75(1.3\text{DL} + 1.6\text{LL})$$

#### 4.3 Lateral Load Pattern

Different invariant lateral load patterns were utilized to represent the likely distribution of inertia forces imposed on the frames during an earthquake and the utilized lateral load patterns are described as follows:

- **'Uniform' Lateral Load Pattern**

The lateral force at any story is proportional to the mass at that story, i.e.

$$F_i = m_i / \sum m_i$$

Where  $F_i$  : lateral force at i-th story

$m_i$  : mass of i-th story

- **'Elastic First Mode' Lateral Load Pattern**

The lateral force at any story is proportional to the product of the amplitude of the elastic first mode and the mass at that story, i.e.

$$F_i = \frac{m_i \Phi_i}{\sum m_i \Phi_i}$$

Where  $\Phi_i$ : amplitude of the elastic first mode at i-th story

(Note that the story forces are normalized with the base shear to have a total base shear equals to unity).

#### **4.4 Capacity Spectrum Method**

According to ACT-40 Capacity Spectrum Method includes

1. Conversion of the Capacity Curve to the Capacity Spectrum (ADRS format)
2. Estimation of Damping and Reduction of 5 percent Damped Response Spectrum
3. Development of the Demand Spectrum (ADRS format)
4. Construction of Bilinear Representation of Capacity Spectrum

#### **4.5 Calculating Performance Point**

The measure of the extent to which the actual building hysteresis is well represented by the parallelogram is taken in to account by k-factor which depends on the structural behavior of the building, which in turn depends on the quality of the seismic resisting system and the duration of ground shaking.

## CHAPTER FIVE VERIFICATION

Two verification examples (single column and planar frame) are presented in this chapter to verify the results of SAP2000 V14 (not licensed), Seismostruct and Response2000; and verify the reliability of fiber model through the structural performance assessment of reinforced concrete frame buildings. In the analysis of reinforced concrete structure, the nonlinearity material behavior is approximately accounted using the implementation of fiber section and the uni-axial material model that can incorporate effects of confinement. There are two important factors in the modeling of discrete member using fiber section model that are a member discretization along the element length and a section discretization into many fibers. The member discretization should be done as the requirement of stiffness based formulation in a nonlinear analysis problem. The fiber discretization should be done as well due to the constant stress assumption of each fiber. Figure 5-1 shows Mander's concrete model (1988) used for behavior modeling of the unconfined and confined concrete fibers under compression which is applicable to rectangular and circular cross sections based on equation 5.1.

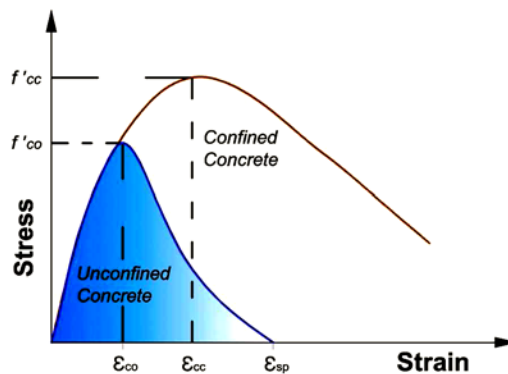


Figure 5-1: Mander's model for confined and unconfined concrete

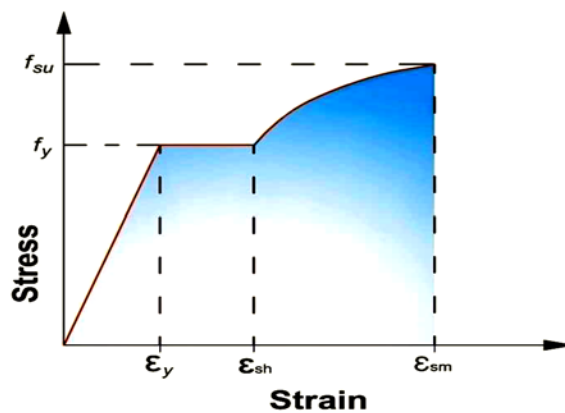


Figure 5-2: Model for reinforcing steel

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \quad (5.1)$$

Where:

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}$$

$\varepsilon_c$ : Longitudinal compressive concrete strain

$$\varepsilon_{cc} = \varepsilon_{co} \left[ 1 + 5 \left( \frac{f'_{cc}}{f'_{co}} - 1 \right) \right]$$

$\varepsilon_{co}$ : Unconfined concrete strain

$f'_{co}$ : Unconfined concrete stress

$$r = \frac{E_c}{E_c - E_{sec}}$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$$

$$\varepsilon_{cu} = 1.4 \left( 0.004 + \frac{1.4 \rho_s f_{yh} \varepsilon_{su}}{f'_{cc}} \right)$$

For circular section:

$$f'_{cc} = f'_{co} \left( -1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_1}{f'_{co}}} - \frac{2 f'_1}{f'_{co}} \right)$$

$$f'_1 = \frac{1}{2} K_e \rho_s f_{yh}$$

$f_{yh}$ : Yielding stress of transverse steel

$$\rho_s = \frac{4 A_{sp}}{d_s s}$$

$A_{sp}$ : Cross sectional area of spiral or hoop

$d_s$ : Diameter of the core (center to center of spirals)

$$K_e = \frac{\left(1 - \frac{s}{2d_s}\right)^2}{1 - \rho_{cc}} \text{ For circular hoops}$$

$$K_e = \frac{\left(1 - \frac{s}{2d_s}\right)^2}{1 - \rho_{cc}} \text{ For circular spirals}$$

$s$ : Clear distance between spirals or hoops

$\rho_{cc}$ : Ratio of area longitudinal reinforcement to area of core section

For rectangular sections:

$$f'_{lx} = K_e \rho_x f_{yh}$$

$$f'_{ly} = \frac{1}{2} K_e \rho_y f_{yh}$$

$$\rho_x = \frac{A_{sx}}{H_c s}$$

$$\rho_y = \frac{A_{sy}}{B_c s}$$

$A_{sx}, A_{sy}$ : Total area of transverse steel running in the x and y directions, respectively

$H_c, B_c$ : Core dimensions to centerline of perimeter hoop in the x and y directions

$s$ : Distance between hoops, center to center

$$K_e = \frac{(1 - \sum_{i=1}^n \frac{(wi)^2}{6H_c B_c}) (1 - \frac{s}{2B_c}) (1 - \frac{s}{2H_c})}{(1 - \rho_{cc})}$$

$w_i; i^{th}$ : Clear distance between longitudinal adjacent bars

The tension strength of the concrete was taken in to account by tension stiffening model. Okamura (et al.) include the tension stiffness in their proposed average stress-strain relation of plain concrete in tension as shown in Figure 5-3. [12] Bilinear reinforcement steel model with 0.01 strain hardening ratio shown in Figure 5-2 was used for behavior modeling of steel fibers.

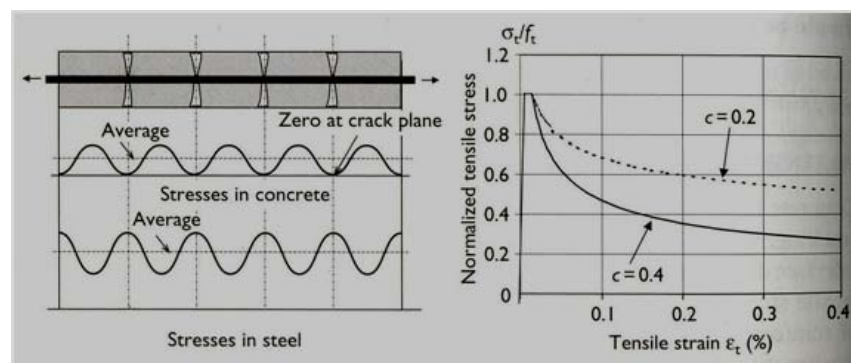


Figure 5-3: Stress distribution and the space-averaged tension Stiffening model [12]

The equation for tensile stress of concrete in RC zone is: [12]

$$\sigma_t = f_t \cdot \left(\frac{\varepsilon_{tu}}{\varepsilon}\right)^c$$

Where  $f_t$  is tensile strength of concrete,  $\varepsilon_{tu}$  is tensile cracking strain and  $c$  is the coefficient depending on bond characteristics, e.g. is 0.4 for deformed bar.

In lumped plasticity model for beams in SAP2000 the nonlinear flexural behavior of reinforced concrete sections can be assessed by a special type of section analysis called Moment-Curvature analysis. The outcome of this analysis is the relation between applied moment and related curvature in the section. Moment-curvature relation is then very important since gives the correspondence between moment and curvature necessary to the calculation of displacement due to forces acting in inelastic elements. In SAP2000 the Takeda hysteresis model was used to model the flexural behavior of all elements. Its rules define the moment-rotation behavior of nonlinear rotational springs in column and beam elements.

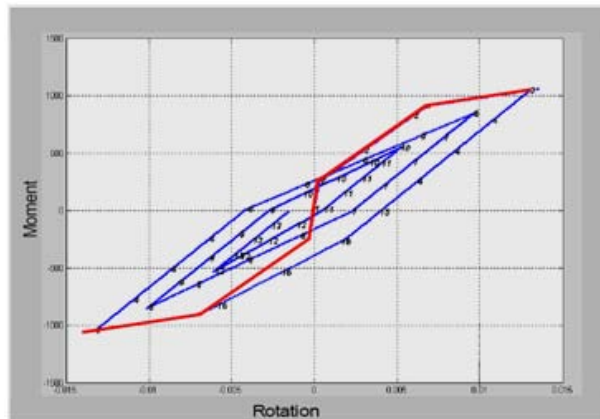


Figure 5-4: Takeda Hysteresis Model

## 5.1 Column Verification

### 5.1.1 Verification Data

This rectangular column example is taken from the previous numerical study of Maekawa *et al.* The cyclic experiment was initially performed by Osada *et al.* (1997). The column has two side reinforcement uniformly distributed along the load direction. This column is not well-confined which can be seen from the distant spacing of the transversal steel compared to its longitudinal and sectional dimension. The column is not under reinforced so that all region of the confined concrete is assumed to be RC zone. The overall dimension and structural configuration can be seen in Figure 5-5.

#### Specimen Specification and Property:

$$P_{\text{axial}} = 78.5 \text{ KN} = 17,648.38 \text{ lb. (constant)}$$

#### Concrete

$$f_c' = 26.5 \text{ MPa} = 3,843.56 \text{ lb/in}^2$$

#### Reinforced Bar

##### Longitudinal Steel

$$\text{Number of steel} = 22$$

$$\text{Diameter} = 13 \text{ mm} = 0.512''$$

$$f_{y\text{long}} = 331.6 \text{ MPa} = 48,095.26 \text{ lb/in}^2$$

##### Side Steel

$$\text{Number of steel} = 6$$

$$\text{Diameter} = 10 \text{ mm} = 0.394''$$

$$f_{y\text{long}} = 343.4 \text{ MPa} = 49,806.74 \text{ lb/in}^2$$

##### Transversal Steel

$$\text{Diameter} = 6 \text{ mm} = 0.236''$$

$$\text{Spacing} = 250 \text{ mm} = 9.843''$$

$$F_{y\text{hoop}} = 353.2 \text{ MPa} = 51,228.13 \text{ lb/in}^2$$

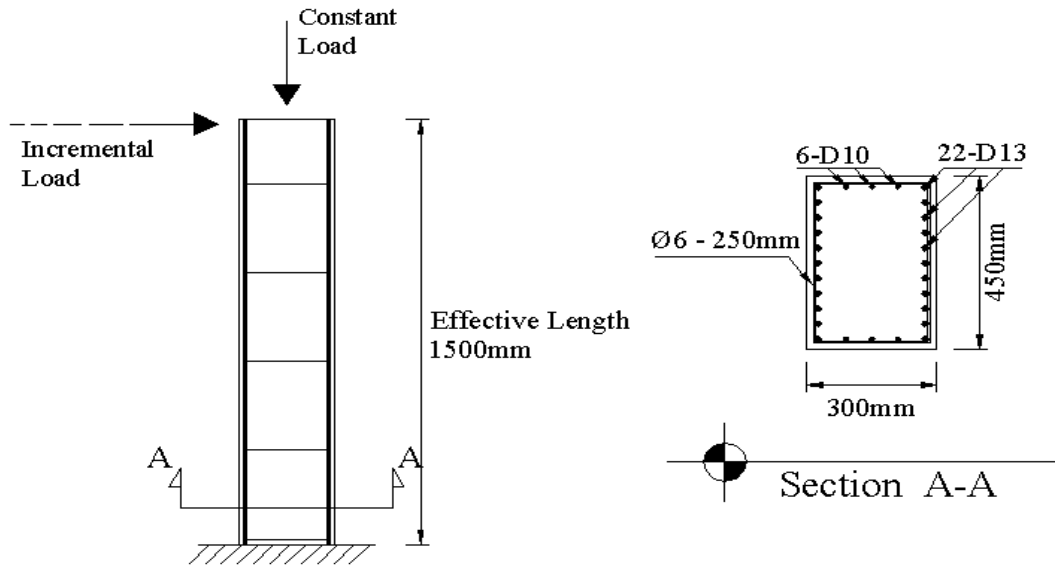


Figure 5-5: Verification Column

### 5.1.2 Verification column model

The figure below shows models of the column used to verify the results of SAP2000, Seismostruct and Response2000; and the reliability of the fiber model.

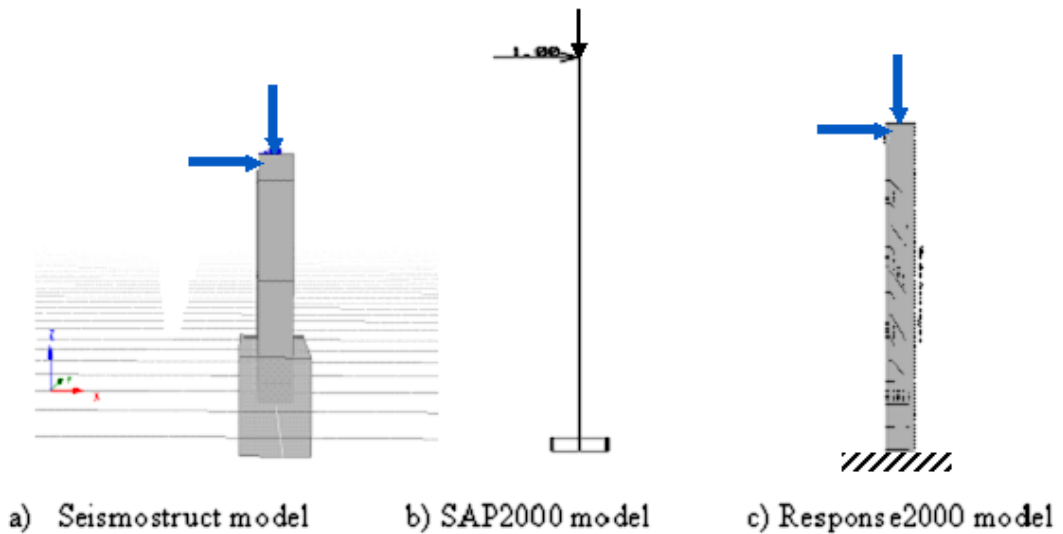


Figure 5-6: Verification Column Model

### 5.1.3 Seismostruct Response2000 and SAP2000 Results

As depicted on Figure 5-8 there is some variation between capacity curves from Response2000 and SAP2000 with the experiment result while the result from Seismostruct is acceptably similar. The pushover curve of the Response2000 is below from the others due to Bentz's 1999 lower bound concrete tension capacity model. Due to zero strain hardening ratio assumption in bilinear reinforcement steel model the Seismostruct capacity curve could be below the experiment one in yielding zone. But the material models should capture the real behavior of materials as much as possible through the performance assessment as could be

seen from the Seismostruct results with and without strain hardening ratio parameters. Considering the effects of tension capacity after cracking can be observed on the initial pattern of the pushover curves that has an impact on the serviceability of the structures.

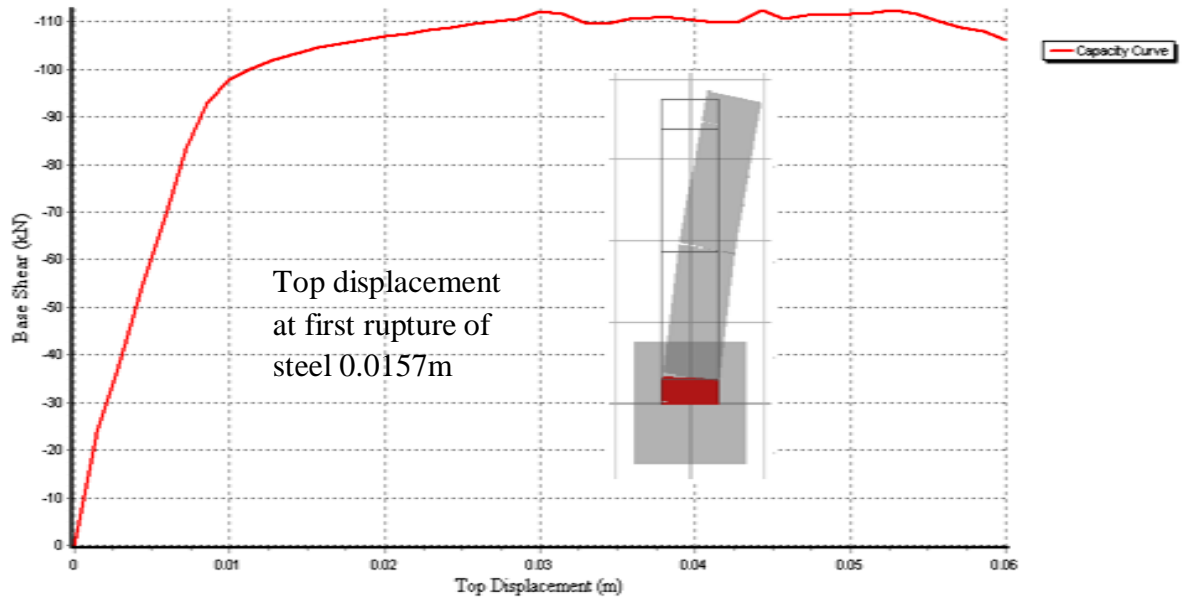


Figure 5-7: Seismostruct Capacity Curve and Column at Steel Rupture

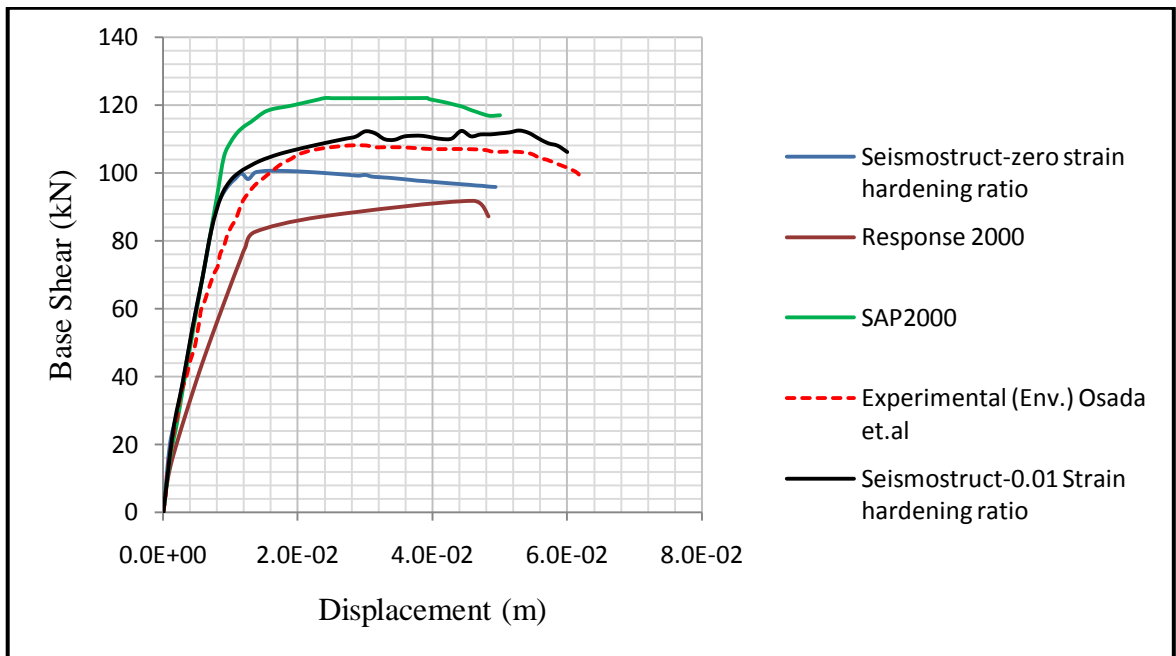


Figure 5-8: Verification Column Results of Seismostruct, Response2000 and SAP2000

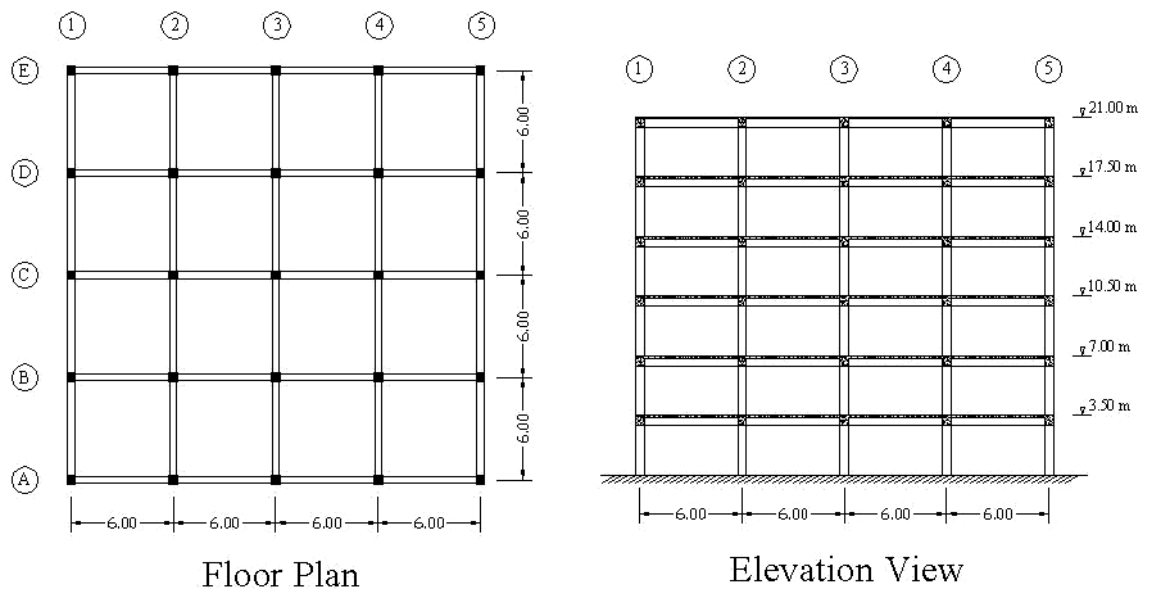
## 5.2 Frame Verification

### 5.2.1 Verification Data

A planar frame of a 6-story regular reinforced concrete structure which has 4x4 bays and 6 stories is used as a reference in this verification chapter. This structure has been designed earlier in the thesis study of Tandian *et al* (2003) based on the national concrete and seismic standard in Indonesia. The overall dimension and structural configuration can be seen in Figure 5-9 and Table 5-1.

Table 5-1: General structural dimension of the reference building

Element Description	Dimension
Area per floor	36 x 36 m <sup>2</sup>
All Beams	0.40 x 0.60 m <sup>2</sup>
All Columns	0.50 x 0.50 m <sup>2</sup>
Plate Thickness	0.12 m
Height each story	3.50 m



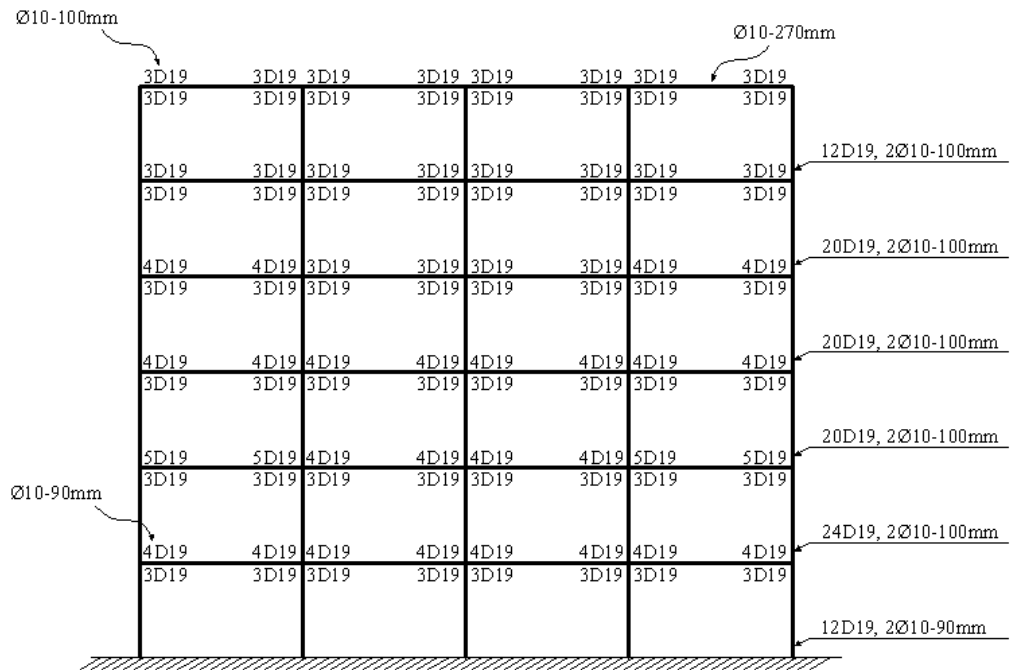
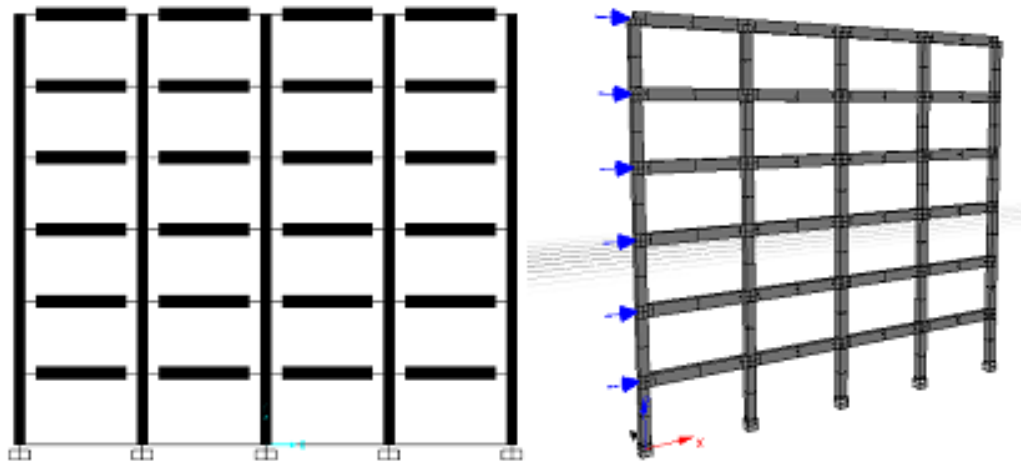


Figure 5-9: Verification Frame

### 5.2.2 Verification Frame Model

The figure below shows the model of the frame used to verify the results of SAP2000 and Seismostruct; and the reliability of the fiber model.



a) SAP2000 Model

b) Seismostruct Model

Figure 5-10: Verification Frame Models

### 5.2.3 Seismostruct and SAP2000 Results

According to the results of Seismostruct and SAP2000 (inconsistent result) shown in

Figure 5-12 there is a huge gap between the pushover curves of SAP2000 and the reference Curve while the results of the Seismostruct and that of the reference pushover curves are acceptably the same for both triangular and uniform lateral load pattern distributions. Figure 5-11 shows the capacity curves for both load patterns and deformed shapes at first rupture of steel with the corresponding roof displacement.

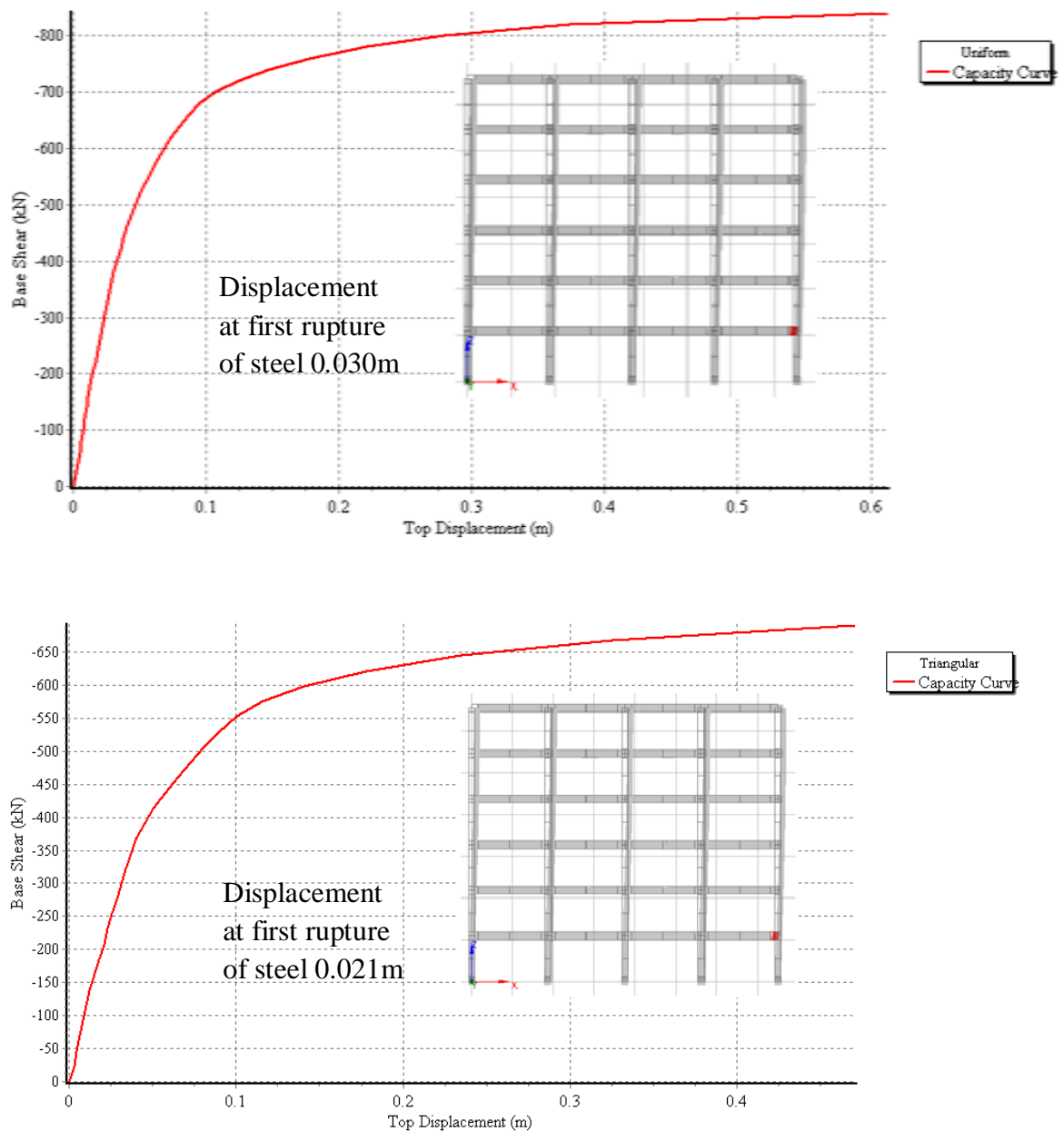


Figure 5-11: Seismostruct Capacity Curves and Deformed Shapes at first Steel Rupture

The sources of the acceptable difference between the Seismostruct and that of the reference pushover curves may arise due to the different software used and assumptions in the modeling of the planar frame in general. The sequences of hinge formations are the same for both lateral

load patterns. Therefore, looking back the verification results it may not be impossible to use Seismostruct as a tool for the structural performance assessment of our case study building with fiber element model.

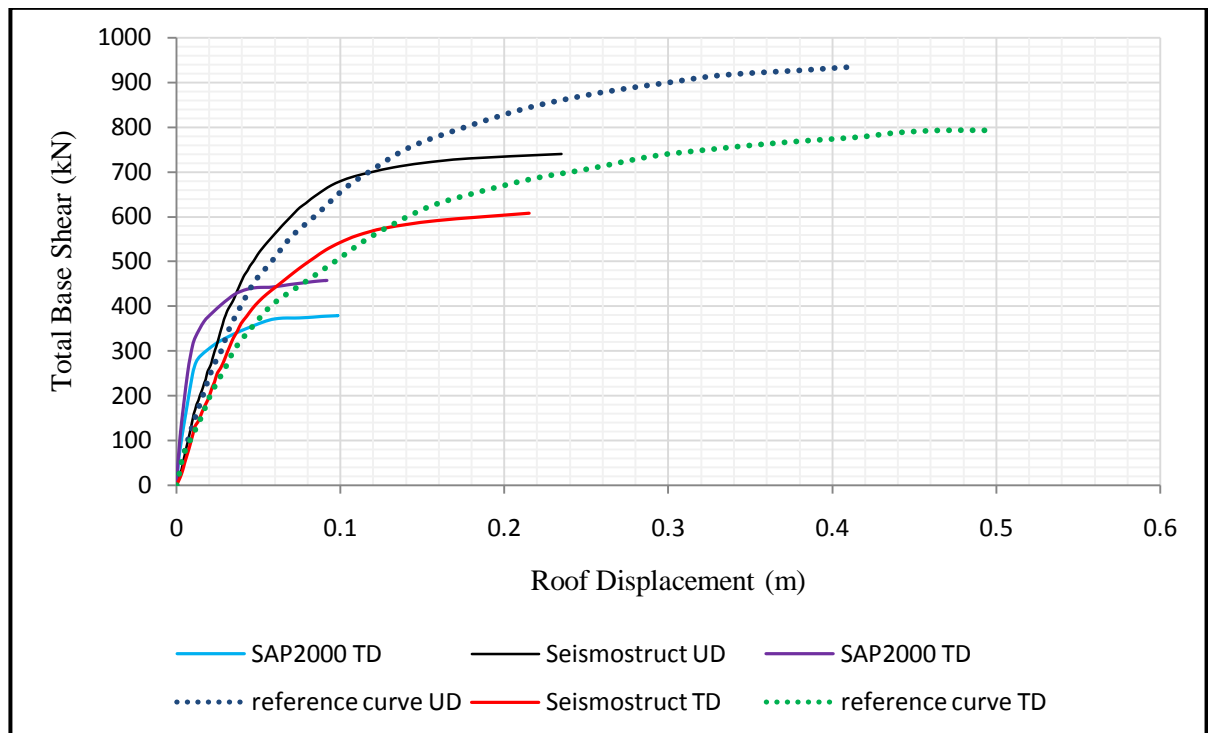
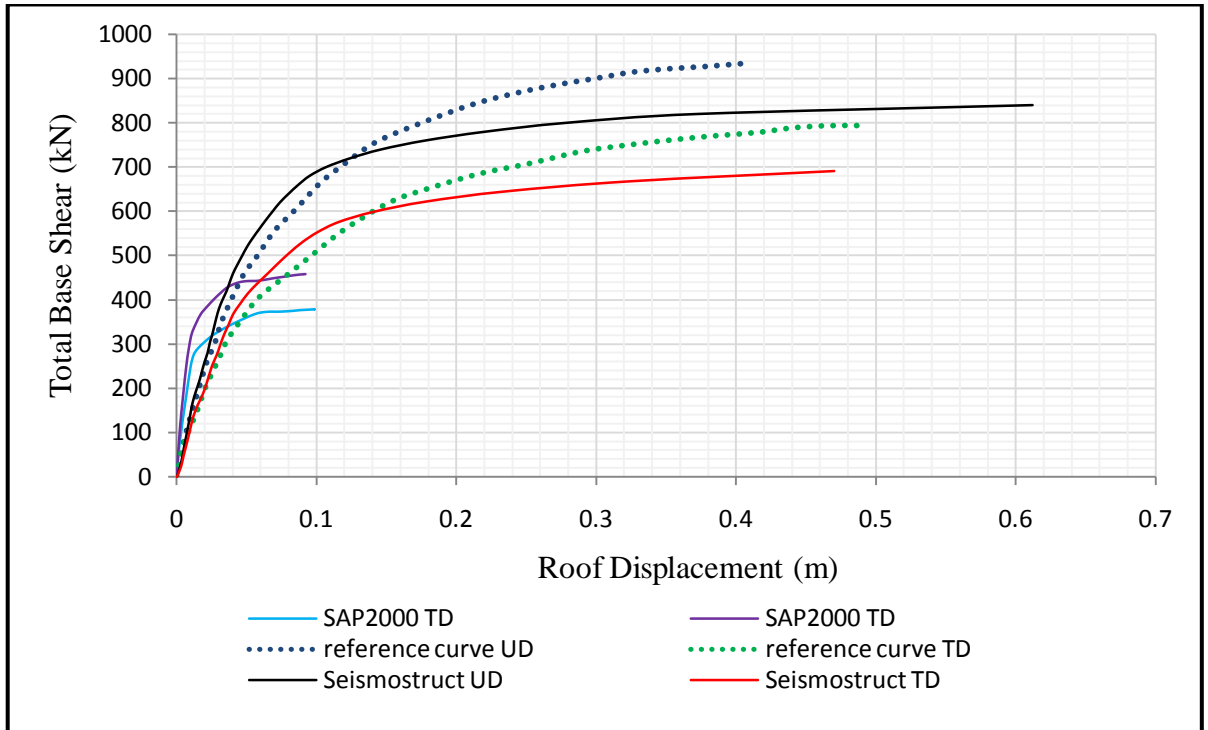


Figure 5-12: Reference Capacity Curves and PCs of Seismostruct, Response2000 and SAP2000

# CHAPTER SIX PERFORMANCE ASSESSMENT OF THE CASE STUDY BUILDING

## 6.1 Case Study Building – Description and Details

### 6.1.1 Case Study Building

The building selected for study is a regular G+4 ribbed slab school building located in Addis Ababa city which is categorized as zone two according to the seismic hazard map of Ethiopia. The building was chosen for study primarily because this building is high in number in the city.

### 6.1.2 Building Description

Figure 6-1 through Figure 6-3 show the architectural plans, elevation and section of the building. The building has six stories and the roof of the building is at an elevation of 18.75m. The first story of the building is 1.75m in height and all other stories are approximately 3.2m in height. The structure system of the building is RC beams and columns frame system with ribbed slab made of pre-cast beam elements. The frame acts as the lateral load resisting system for the building. The reinforced concrete frame structures have been designed according to the rules of the Ethiopian Building Code Standards. The frames are assumed fixed at their base and no diaphragm constraint at each story level and rigid beam column joints. Unfactored material strengths were used in which Yield strength of the steel reinforcements is  $400 \text{ N/mm}^2$  and cubic compressive strength of concrete is  $25 \text{ N/mm}^2$ .

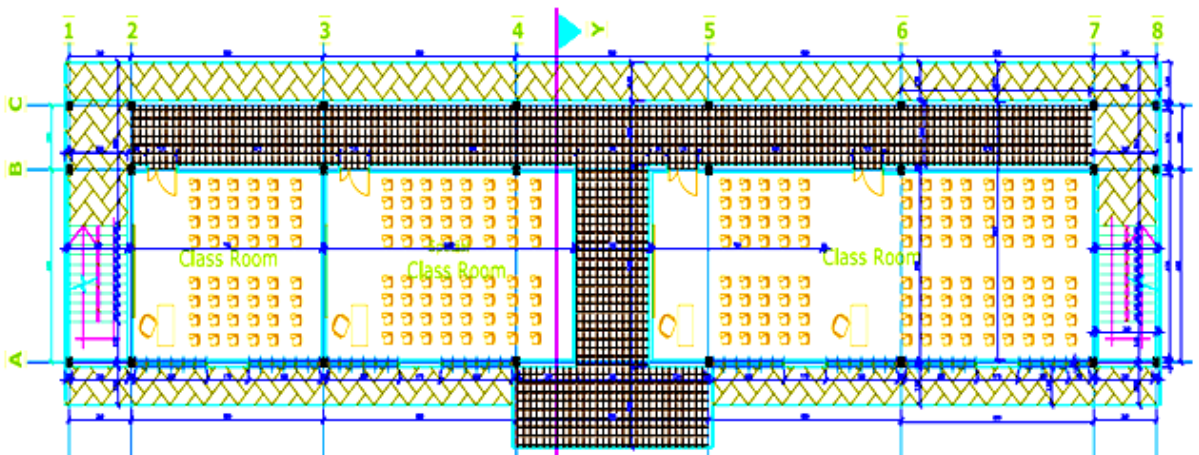


Figure 6-1: Floor plan

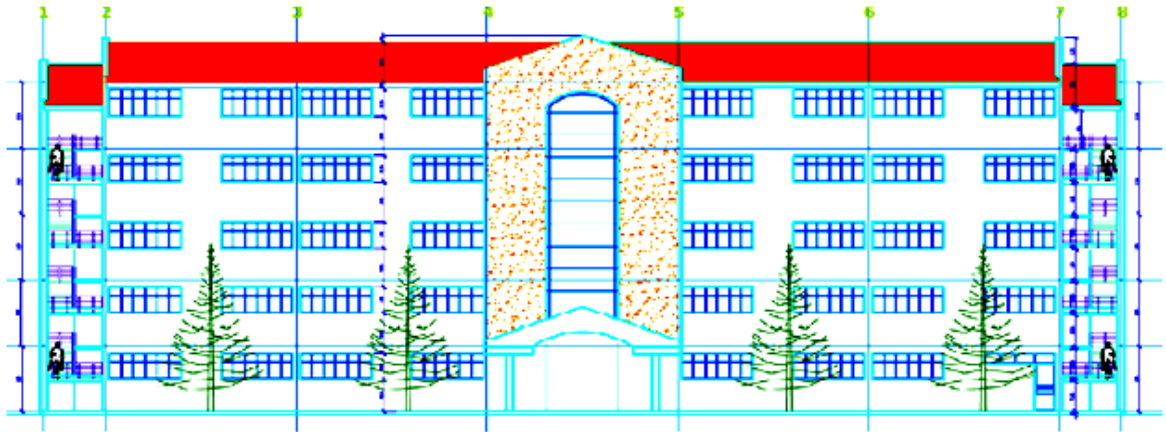


Figure 6-2: Front elevation

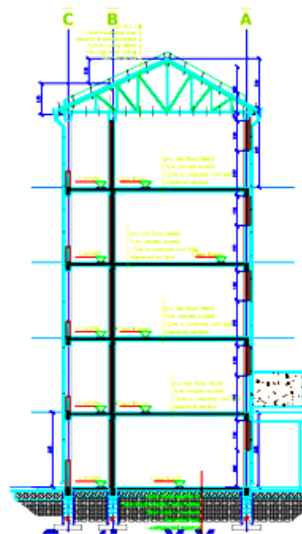


Figure 6-3: Section view

### 6.1.3 Structural System Design Details

Figure 6-4 shows the three dimensional structural model of the case study building.

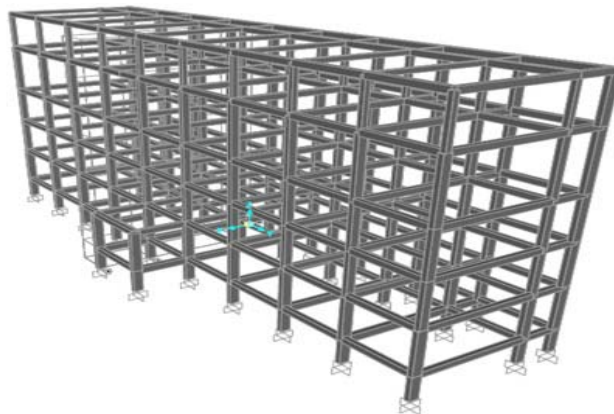


Figure 6-4: Three dimensional structural model



Table 6-1: Reinforcement Details for Columns and Beams

Story	Reinf. and Group ID	Column ID		
		C1	C3	C5
1	Long. Bars	8Φ20	8Φ20	6Φ14
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col A-A	Col A-A	Col P-P
	Section	Sec A-A	Sec A-A	Sec P-P
2	Long. Bars	8Φ20	8Φ20	8Φ20
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col A-A	Col A-A	Col P-P
	Section	Sec A-A	Sec A-A	Sec P-P
3	Long. Bars	8Φ20	8Φ20	8Φ20
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col C-C	Col C-C	Col C-C
	Section	Sec C-C	Sec C-C	Sec C-C
4	Long. Bars	8Φ20	8Φ20	8Φ20
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col C-C	Col C-C	Col C-C
	Section	Sec C-C	Sec C-C	Sec C-C
5	Long. Bars	8Φ20	8Φ20	8Φ20
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col C-C	Col C-C	Col C-C
	Section	Sec C-C	Sec C-C	Sec C-C
6	Long. Bars	8Φ14	8Φ14	8Φ14
	Ties	Φ6@20	Φ6@20	Φ6@20
	Group ID	Col D-D	Col D-D	Col D-D
	Section	Sec D-D	Sec D-D	Sec D-D

Story	Reinf.	Group ID			
		GB A-A	FB D-D	FB E-E	RB A-A
1	Long. Bars Ties Section	6Φ14 Φ8@15 Sec A-A			
2	Long. Bars Ties Section		6Φ14 Φ8@15 Sec D-D		
3,4,5	Long. Bars Ties Section			6Φ14 Φ8@15 Sec E-E	
6	Long. Bars Ties Section				4Φ12 Φ8@15 RB SecA-A

## 6.2 Results and Discussion

### 6.2.1 Seismostruct

#### 6.2.1.1 Global Structure Behavior

##### 6.2.1.1.1 Capacity Curve

Capacity curves (base shear versus roof displacement) are the load-displacement envelopes of the structures and represent the global response of the structures. Capacity curves for case study frames were obtained from the pushover analyses using Triangular and uniform lateral load patterns. The absolute maximum values of roof displacements and base shears experienced under ground motion excitations were determined for each deformation level to approximate a dynamic capacity curve for the frames. Figure 6-6 is a capacity curve of the frame in the negative x-direction for triangular lateral load distribution.

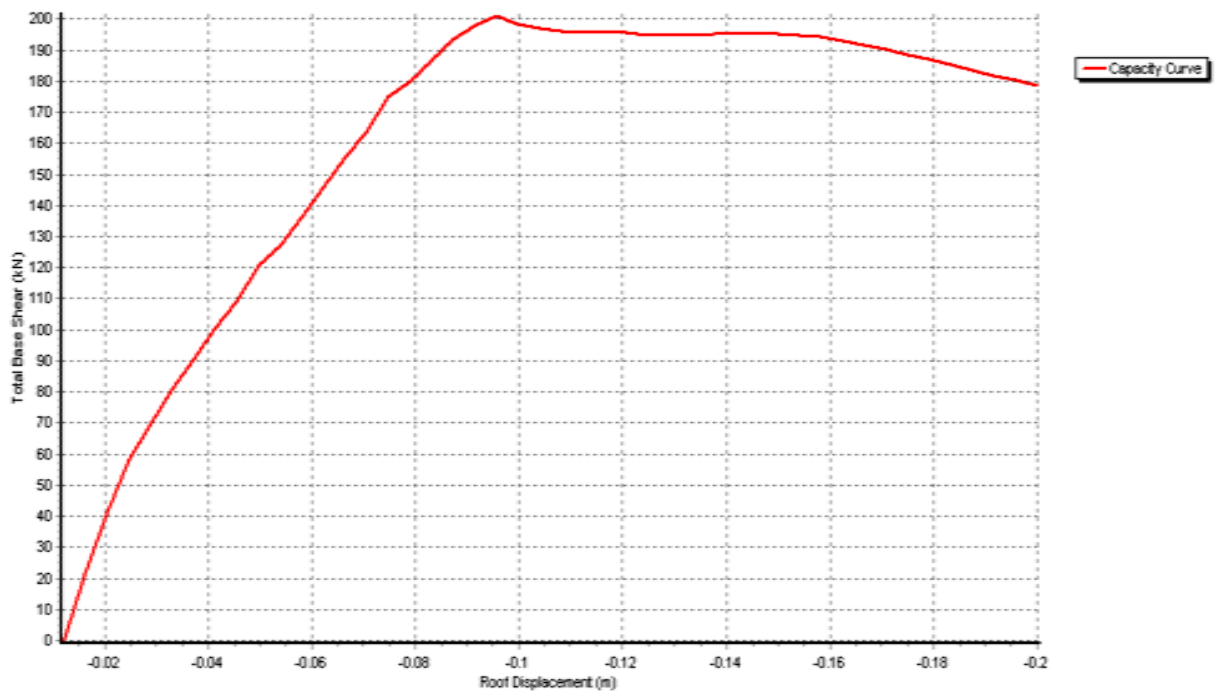
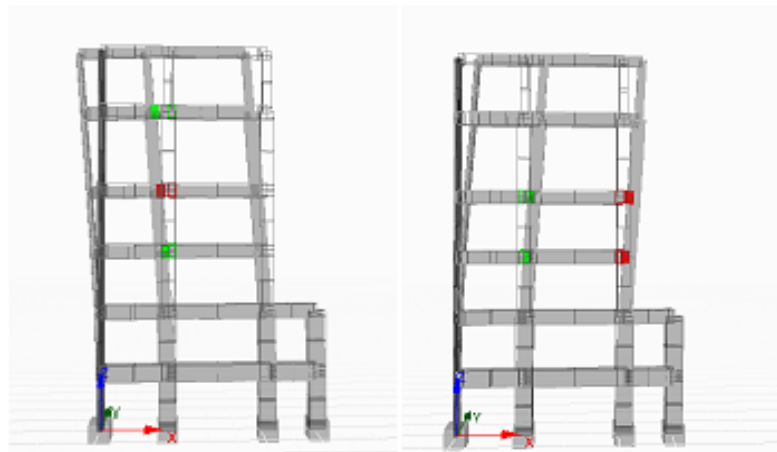


Figure 6-6: Capacity Curve: Triangular/Opposite direction

Table 6-2 gives roof displacement and total drift ratio at first rupture of steel. It is to show that how much the roof level is displaced relative to the base until the first rupture of steel while Figure 6-7 shows the deflected shapes of the case study frame at the same damage level for Uniform and Triangular lateral load patterns.

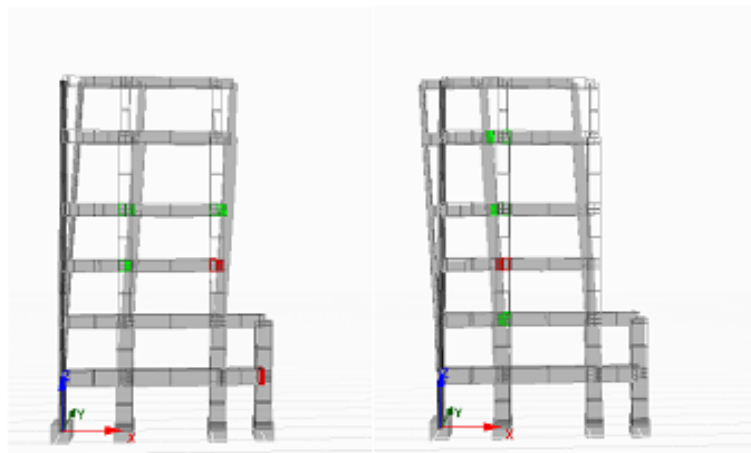
Table 6-2: Roof Displacement and Total Drift Ratio at Rupture of Steel

Lateral Load Pattern	Base Shear at Rupture of Steel (kN)	Roof Displacement (m)	Total Drift Ratio (%)
Triangular	228.5	0.0513	0.2736
Triangular/opposite	148.1	0.0637	0.3397
Uniform	325.	0.0514	0.2741
Uniform/opposite	213.1	0.0647	0.3450



a) Triangular/opposite

b) Triangular



a) Uniform

b) Uniform/opposite

■ Rupture of Steel

Figure 6-7: Deflected shapes at first rupture of steel

### 6.2.1.1.2 Story displacements

The figure below shows displacement profile at the performance point in which statically calculated force demands are used to find the points.

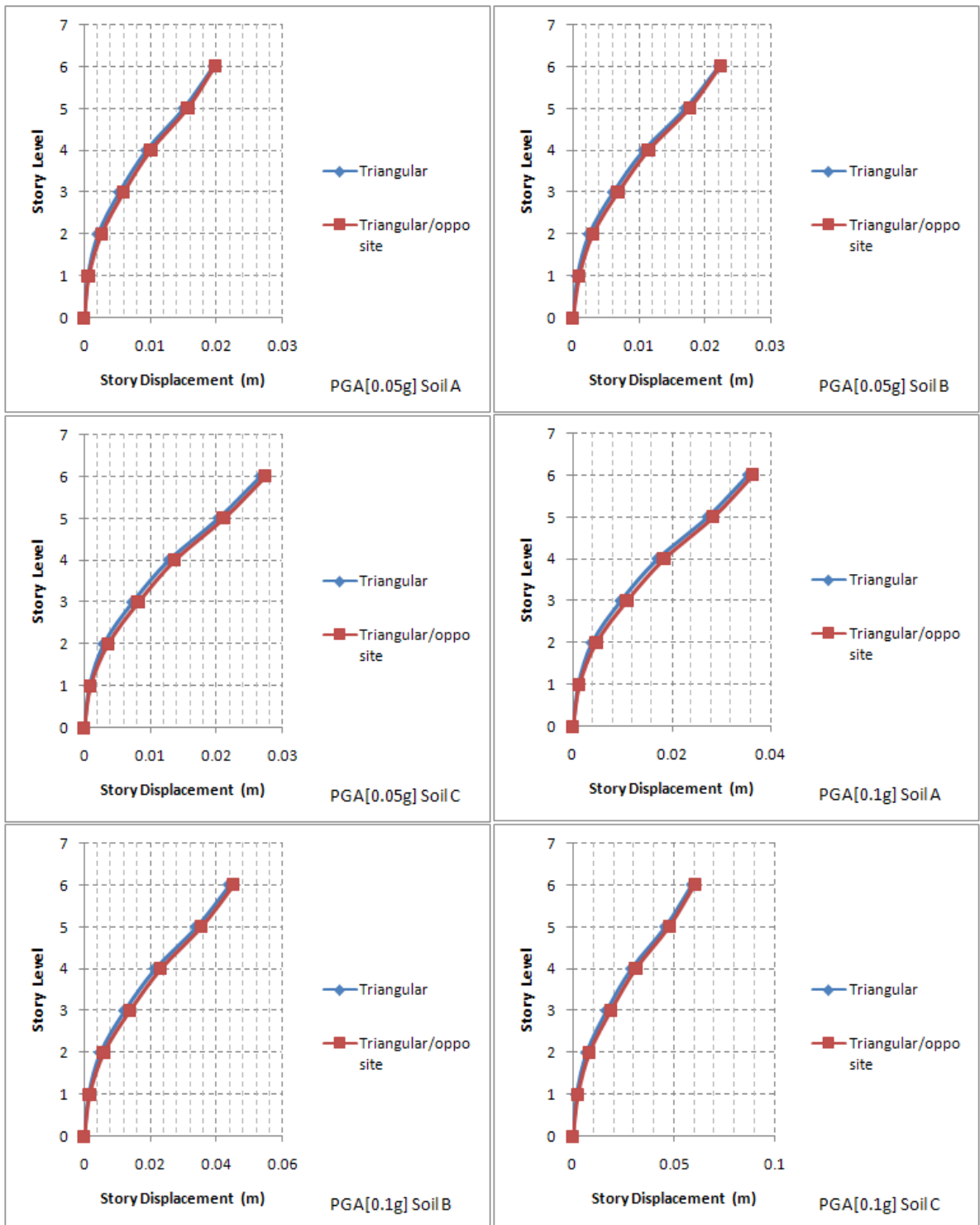


Figure 6-8: Story displacement profiles

### 6.2.1.1.3 Check Story-Drift Requirements

The accurate estimation of inter-story drift ratio and its distribution along the height of the structure is very critical for seismic performance evaluation purposes since the structural damage is directly related to the inter-story drift ratio.

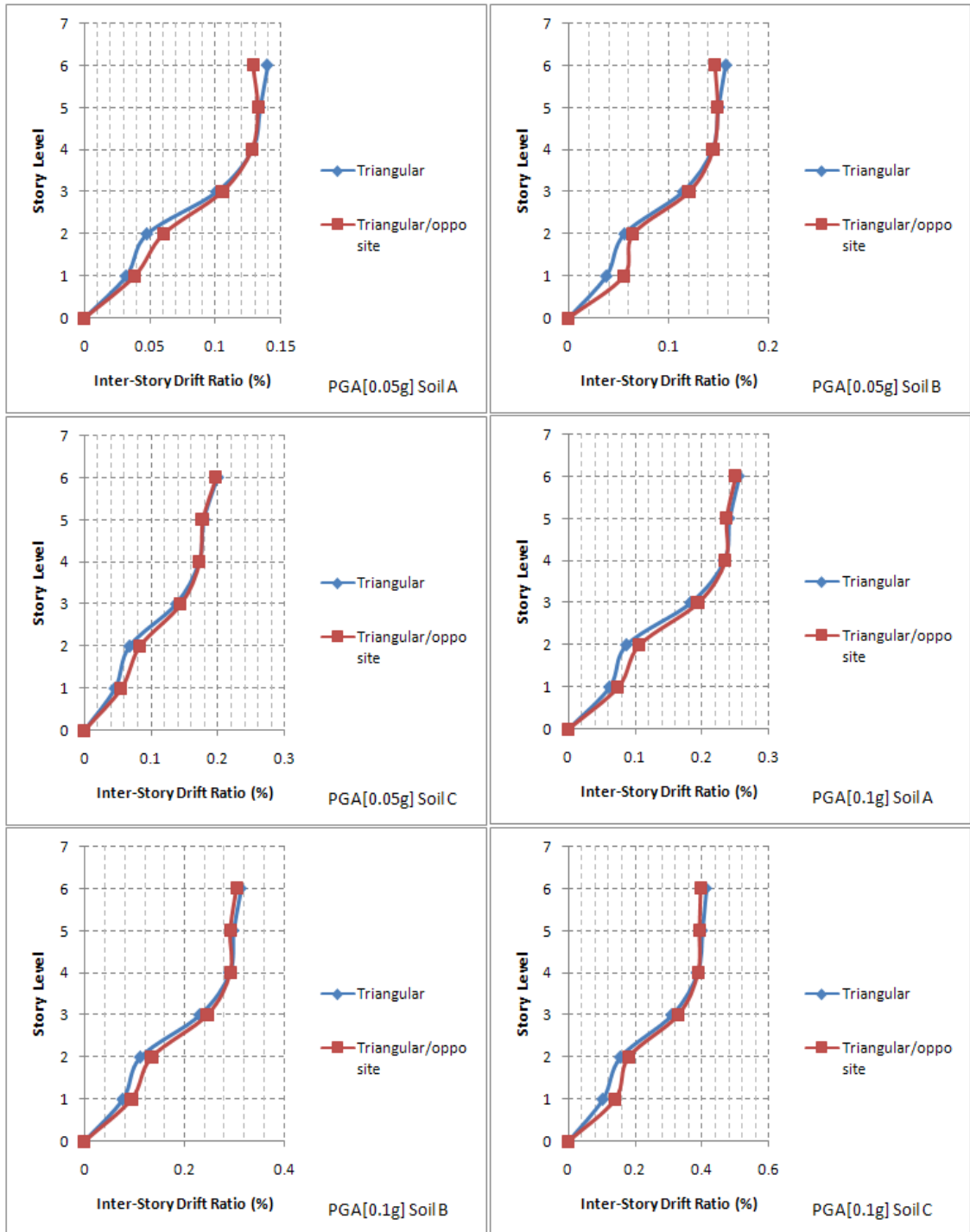


Figure 6-9: Inter-Story drift ratio profiles

Figure 6-9 shows nonlinear height wise distribution of stiffness shifted inelastic demands to the upper stories. Based on the response limits for story drift ratio described in APPENDIX-B Seismic Performance C, the performance level of the building was determined as shown in table below.

Table 6-3: Story-Drift Requirement

		Lateral Load pattern: Triangular/Opposite				
		Story Level	Story Drift	IO	DC	LS
<b>PGA [0.05g]</b>	<b>Soil Class A</b>	6	0.00413058	OK!	OK!	OK!
		5	0.00557491	OK!	OK!	OK!
		4	0.00410437	OK!	OK!	OK!
		3	0.00337798	OK!	OK!	OK!
		2	0.0019416	OK!	OK!	OK!
		1	0.00067199	OK!	OK!	OK!
	<b>Soil Class B</b>	6	0.00465729	OK!	OK!	OK!
		5	0.00622111	OK!	OK!	OK!
		4	0.00460399	OK!	OK!	OK!
		3	0.00383681	OK!	OK!	OK!
		2	0.00205602	OK!	OK!	OK!
		1	0.0009626	OK!	OK!	OK!
	<b>Soil Class C</b>	6	0.00628008	OK!	OK!	OK!
		5	0.00742087	OK!	OK!	OK!
		4	0.00550149	OK!	OK!	OK!
		3	0.00461109	OK!	OK!	OK!
		2	0.00265176	OK!	OK!	OK!
		1	0.0009626	OK!	OK!	OK!
<b>PGA [0.1g]</b>	<b>Soil Class A</b>	6	0.00798242	OK!	OK!	OK!
		5	0.00991884	NOT OK!	OK!	OK!
		4	0.00750152	OK!	OK!	OK!
		3	0.00620809	OK!	OK!	OK!
		2	0.00339951	OK!	OK!	OK!
		1	0.00129872	OK!	OK!	OK!
	<b>Soil Class B</b>	6	0.00973194	NOT OK!	OK!	OK!
		5	0.01224994	NOT OK!	OK!	OK!
		4	0.00933799	NOT OK!	OK!	OK!
		3	0.00786379	OK!	OK!	OK!
		2	0.00430668	OK!	OK!	OK!
		1	0.00164442	OK!	OK!	OK!
	<b>Soil Class C</b>	6	0.01267569	NOT OK!	OK!	OK!
		5	0.0164986	NOT OK!	OK!	OK!
		4	0.01242344	NOT OK!	OK!	OK!
		3	0.01052288	NOT OK!	OK!	OK!
		2	0.00576625	OK!	OK!	OK!
		1	0.00245041	OK!	OK!	OK!

#### 6.2.1.1.4 Check Roof Displacement Ratios at Performance points

The limit used for roof drift ratio is 0.2%. The table below shows (for the building located on Soil Class B and Soil Class C) the structural performance is not conformal with the requirement for PGA of 0.1g.

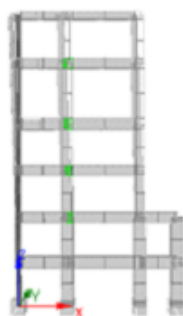
PGA [0.05g]	Soil Class	Roof Drift Ratio (%)	Check Limit (0.2%)
	A	0.12348	Ok!
	B	0.139893	Ok!
PGA [0.1g]	C	0.171952	Ok!
	A	0.22643	Not Ok!
	B	0.282736	Not Ok!
	C	0.36464	Not Ok!

#### 6.2.1.2 Plastic hinge rotations

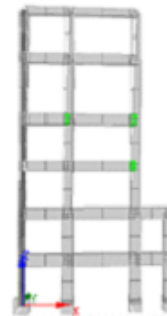
The section curvatures of the case study frame were below the limits for life safety (LS) for PGA of 0.05g for all except Soil Class C. The curvature limits for beams are given below.

Table 6-4: Section Curvature limits for LS, in radians

Beam ID	Curvature at first yield of steel In radians	Curvature for LS In radians
GB A-A	0.00524	0.01024
FB D-D	0.0067	0.0117
FB E-E	0.0072	0.012202
RB A-A	0.01162	0.02162



Triangular/opposite at base shear 74.7kN



Triangular at base shear 168kN

■ LS

Figure 6-10: Plastic hinges at Life Safety

### 6.2.1.3 Safety factors of the case study building

Table 6-5 through Table 6-8 show safety factors of the building at life safety structural performance level and collapse prevention performance level set by ATC-40. The expected factor of safety of the building is a combination of material and loads safety factors. From the results of Seismostruct, capacity of the case study building is below the force demand for PGA of 0.1g and 0.05g for all soil class and soil class C respectively at life safety structural performance level. The buildings comply with inter-story and roof displacement requirements at the force demands, and also checked for premature (e.g. shear) and creep failures in addition to flexural failures.

Table 6-5: Factor of safety for PGA [0.05g] at Life Safety: Seismostruct

	<b>Triangular/opposite</b>		
	Soil A	Soil B	Soil C
Capacity (kN)	74.7	74.7	74.7
Demand (kN)	51.98	62.5	77.96
Safety Factor	1.437	1.195	0.958

Table 6-6: Factor of safety for PGA [0.1g] at Life Safety: Seismostruct

	<b>Triangular/opposite</b>		
	Soil A	Soil B	Soil C
Capacity (kN)	74.7	74.7	74.7
Base Shear (kN)	103.95	124.74	155.93
Safety Factor	0.718	0.599	0.479

Table 6-7: Factor of Safety for PGA [0.05g] at Steel Rupture: Seismostruct

	<b>Triangular/opposite</b>		
	Soil A	Soil B	Soil C
Capacity (kN)	148.1	148.1	148.1
Demand (kN)	51.98	62.5	77.96
Safety Factor	2.847	2.368	1.898

Table 6-8: Factor of Safety for PGA [0.1g] at Steel Rupture: Seismostruct

	Triangular/opposite		
	Soil A	Soil B	Soil C
Capacity (kN)	148.1	148.1	148.1
Base Shear (kN)	103.95	124.74	155.93
Safety Factor	1.423	1.186	0.949

### 6.2.2 Check for Shear

Since it is not possible to control the premature (e.g. shear) failure in fiber model, it is important to check the level of shear capacity reserve. The factor of safety for shear at the first rupture of steel is given in table below that shows no premature failure.

Element	Capacity (kN)	Design Shear (kN)	Check	Safety factor
Col A-A	230.7	139	Ok!	1.65
Beam	245.9	159.3	Ok!	1.25

Shear capacity taking in to account the size, axial force, strut action is: [11]

$$V_{Rd} = V_{Rd,s} + V_{Rd,c}$$

$$V_{Rd,c} = \left[ C_{Rd,c} K (100 \rho_1 f_{ck})^{\frac{1}{3}} \left( \frac{2d}{a_v} \right) + K_1 \sigma_{cp} \right] b_w d$$

$$V_{Rd,c} = (V_{min} + K_1 \sigma_{cp}) b_w d$$

$$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

$$\rho_1 = \frac{A_{s1}}{b_w d} \leq 0.02$$

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2 f_{cd}$$

$$C_{Rd,c} = 0.18$$

$$V_{min} = 0.035 K^{\frac{3}{2}} f_{ck}^{\frac{1}{2}}$$

$$K_1 = 0.15$$

$$V_{Rd,c} \leq 0.5 b_w d v f_{cd}$$

$$v = 0.6 \left[ 1 - \frac{f_{ck}}{250} \right]$$

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

$$V_{Rd,max} = \frac{\alpha_c b_w z v f_{cd}}{\cot \theta + \tan \theta}$$

$\tan\alpha_c$  is:

$$\begin{aligned} & \left(1 + \frac{\sigma_{cp}}{f_{cd}}\right) && \text{for } 0 < \sigma_{cp} \leq 0.25f_{cd} \\ & 1.25 && \text{for } 0.25f_{cd} < \sigma_{cp} \leq 0.5f_{cd} \\ & 2.5 \left(1 - \frac{\sigma_{cp}}{f_{cd}}\right) && \text{for } 0.25f_{cd} < \sigma_{cp} \leq 0.5f_{cd} \\ & z = 0.9d \\ & \theta = 45^\circ \end{aligned}$$

### 6.2.2.1 Check for Creep Failure

The design of concrete and reinforced concrete is based on the short-time compressive strength of concrete  $f'_c$ . Normal strength concrete loaded at late age with sustained stress in excess of approximately 70 to 75 percent of its short-time of loading may fail under the sustained loads after a period of several minutes to several months. Figure 6-11 shows the nonlinear creep and delayed fracture test of concrete by Rüsçh [1960]. Provided that the sustained stress is approximately below 70% long-term creep failure hardly occurs.

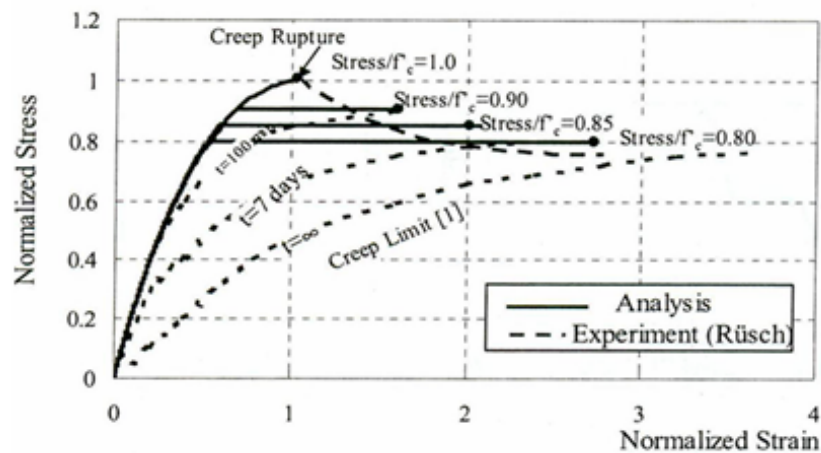


Figure 6-11: Creep Rupture of Concrete under Concentric Loading

Therefore, from the results shown below the case study building is safe from creep failure.

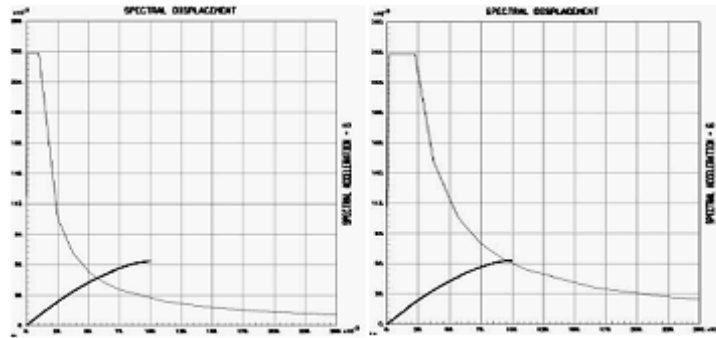
	75% strength	Stress (N/mm <sup>2</sup> )	Check	SF
<b>Critical Column</b>	15	-9.4	OK!	1.6

## 6.2.3 SAP2000

### 6.2.3.1 Global Structure Behavior

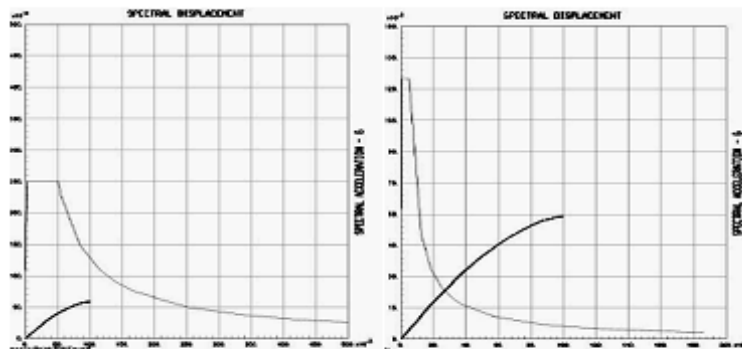
#### 6.2.3.1.1 Capacity Curves

The figures below are plots of demand and capacity curves in Acceleration-Displacement (AD) format.



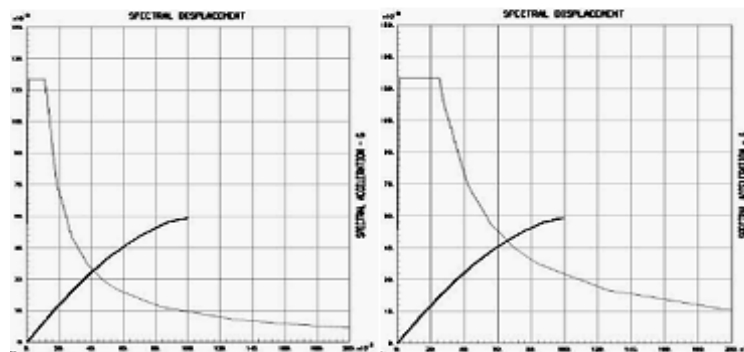
PGA [0.1g] Soil A

PGA [0.1g] Soil B



PGA [0.1g] Soil C

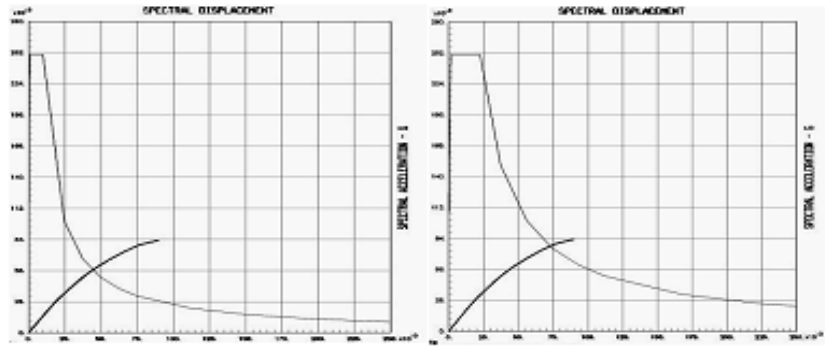
PGA [0.05g] Soil A



PGA [0.05g] Soil B

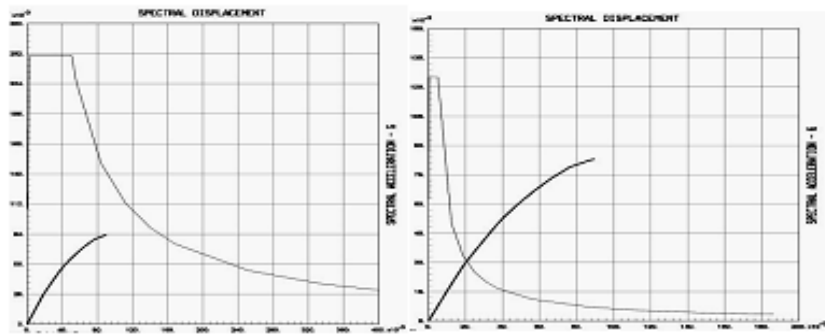
PGA [0.05g] Soil C

Figure 6-12: Capacity and Demand Curves (Triangular Pattern)



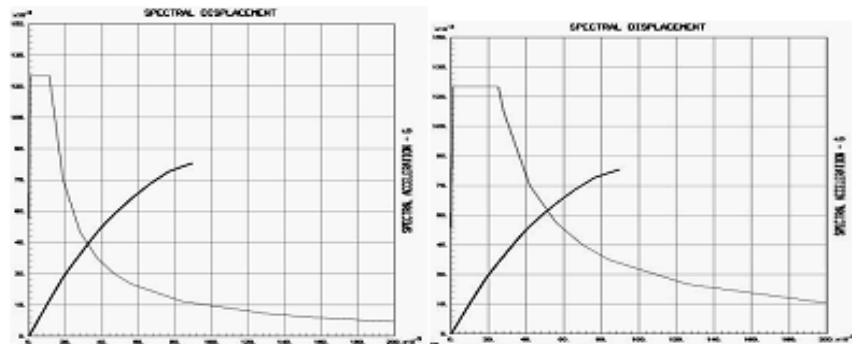
PGA [0.1] Soil A

PGA [0.1] Soil B



PGA [0.1g] Soil C

PGA [0.05g] Soil A



PGA [0.05g] Soil B

PGA [0.05g] Soil C

Figure 6-13: Capacity and Demand Curves (Uniform Pattern)

Figure 6-14 shows the pushover curve of the case study frame for the Triangular lateral load pattern in the opposite direction (negative x-direction).

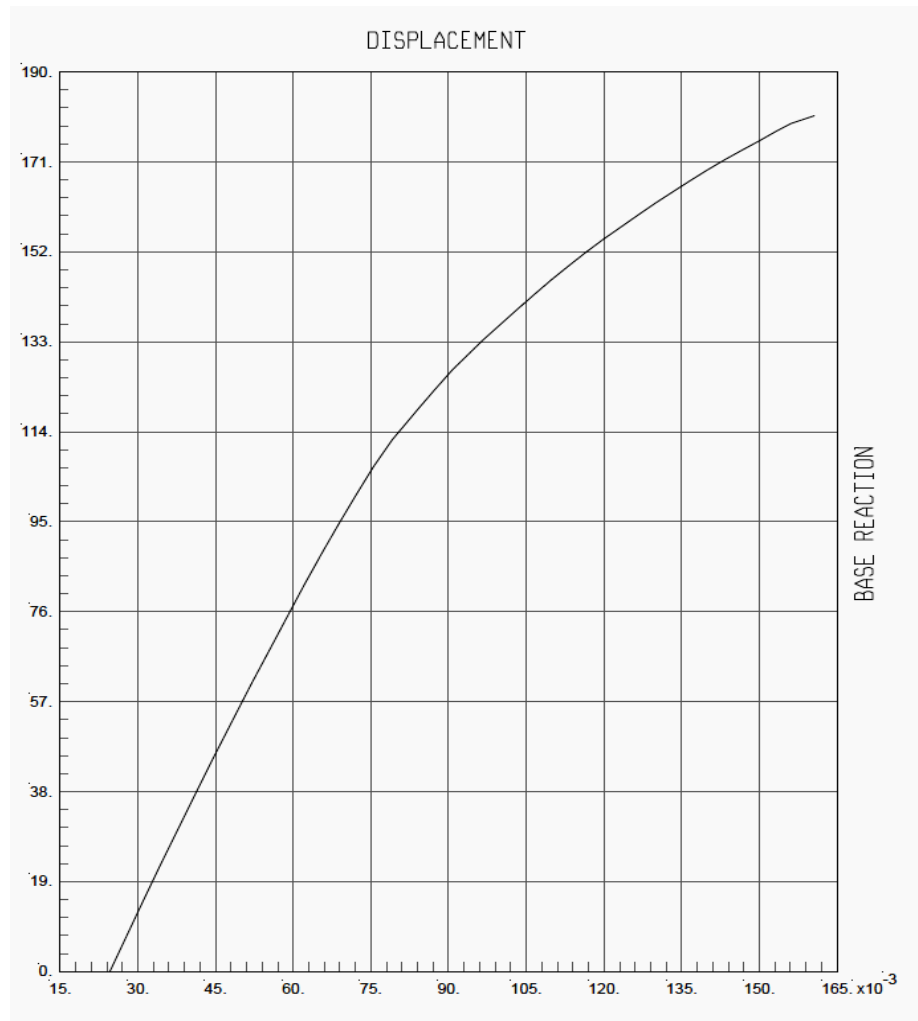


Figure 6-14: Pushover Curves: Triangular/opposite

### 6.2.3.1.2 Performance Points

Table 6-9: Performance Point PGA [0.05g]

	Triangular			Triangular/opposite		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
Performance Point (m)	0.051	0.066	0.088	-0.011	-0.028	-0.052
Base Shear (kN)	82.5	115.3	145.6	77	107.2	135.8

	Uniform			Uniform/opposite		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
Performance Point	0.0053	0.018	0.035	-0.044	-0.057	-0.074
Base Shear (kN)	105	149	192	109	155	202

Table 6-10: Performance Point PGA [0.1g]

	<b>Triangular</b>			<b>Triangular/opposite</b>		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
Performance Point (m)	0.081	0.114	0.174	-0.044	-0.08	N.A
Base Shear (kN)	136	173	215	126.7	165	N.A

	<b>Uniform</b>			<b>Uniform/opposite</b>		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
Performance Point (m)	0.029	0.057	N.A	-0.068	-0.095	-0.140
Base Shear (kN)	179	229	N.A	188	242	309

### 6.2.3.1.3 Story Displacement Profiles

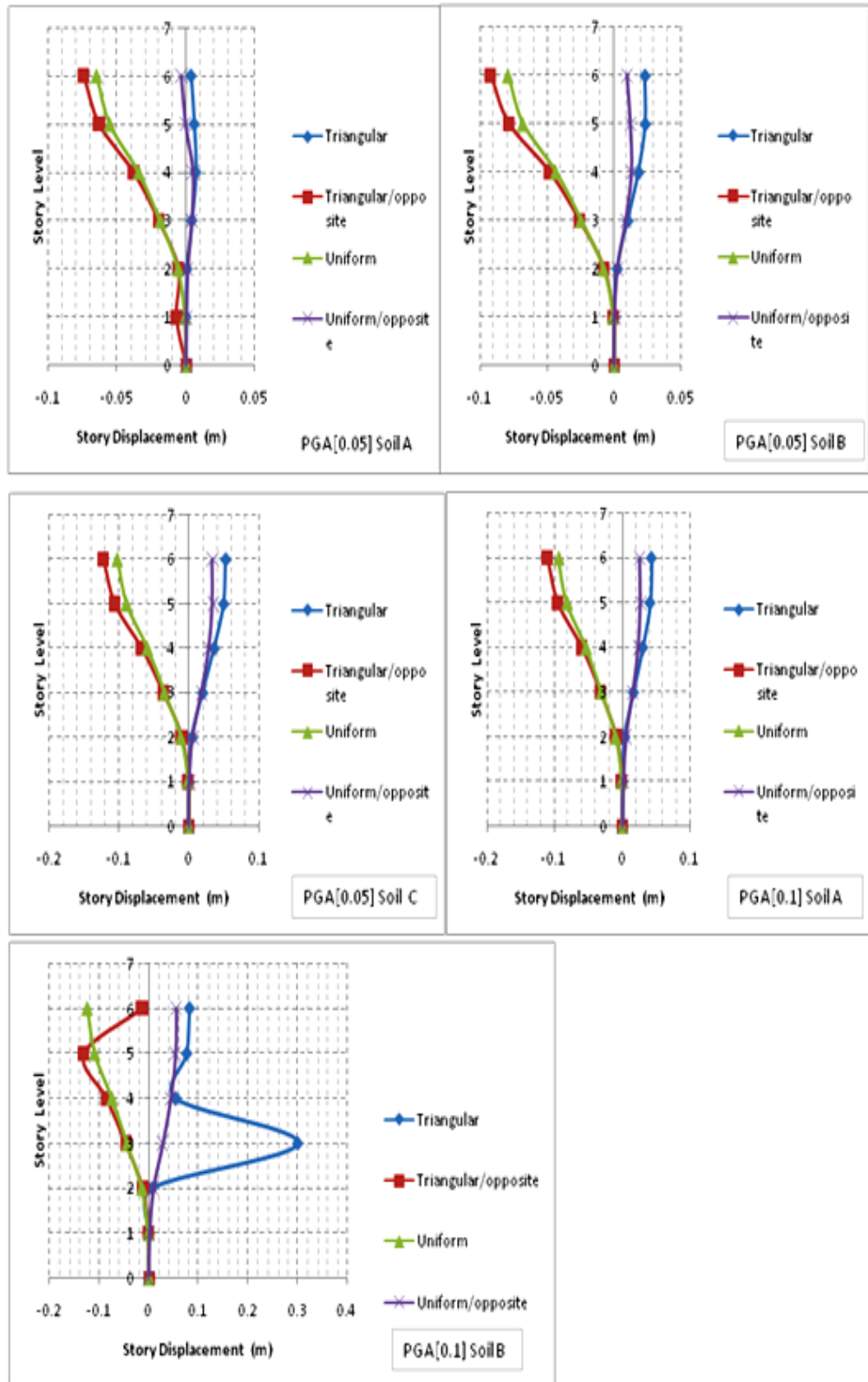


Figure 6-15: Story displacement profiles

### 6.2.3.1.4 Inter-Story Drift Ratios

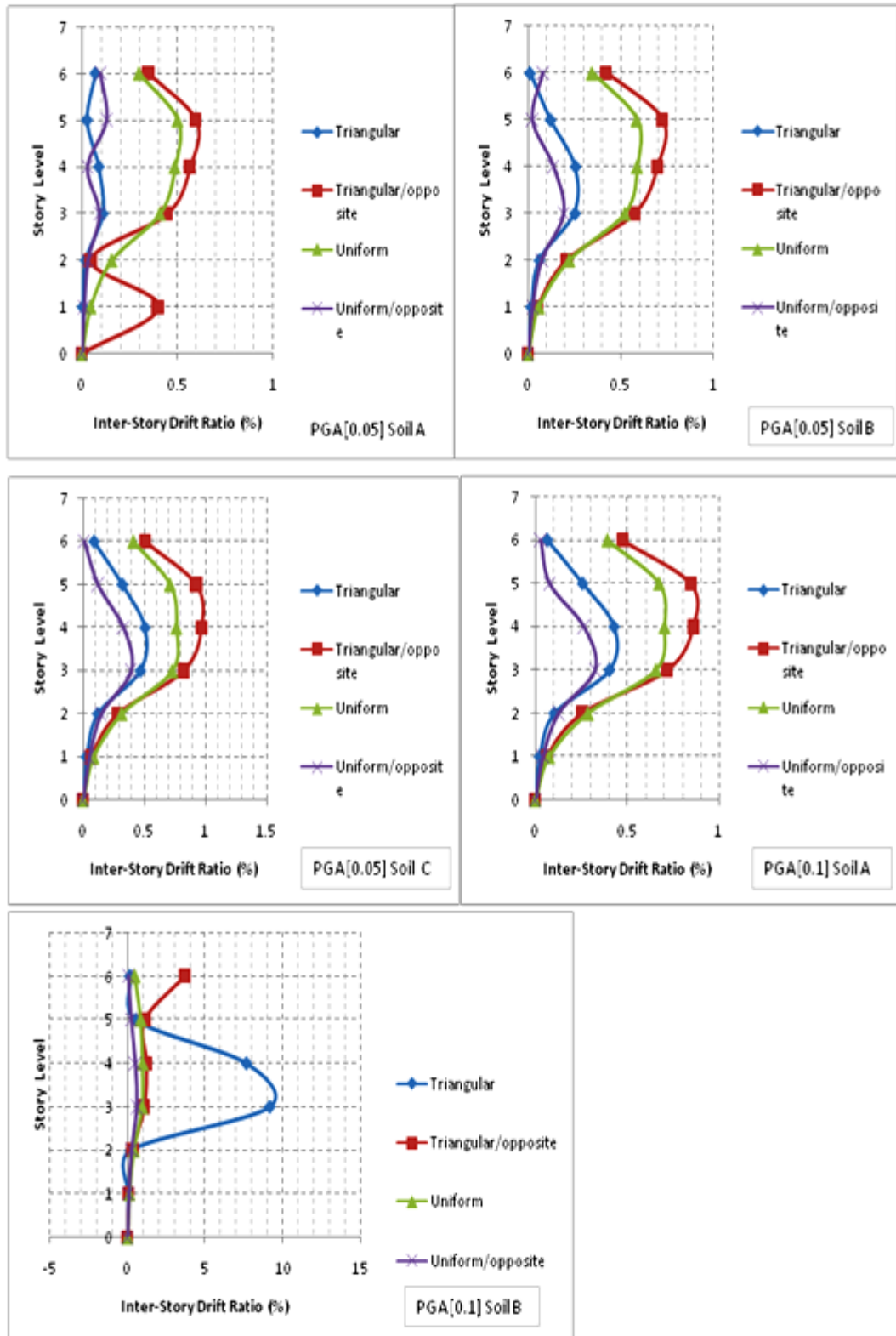


Figure 6-16: Inter-Story drift ratio profiles

## CHAPTER SEVEN CONCLUSION AND RECOMMENDATIONS

- The case study and the frame verification problem show that further study is needed before utilizing SAP2000 as application software for structural performance assessment.
- The results of Seismostruct show that it is reliable to use fiber based model for the structural performance assessment of RC frame buildings.
- The sequences of hinge formation for all lateral load patterns are the same in SAP2000 and Seismostruct. And Seismostruct is more convenient in controlling material failure criterion directly.
- Conventional pushover analysis with triangular load pattern distribution gives higher displacement while the uniform lateral load pattern gives higher base shear.
- Performance evaluations of existing buildings which are expected to have high risk of life and/or money loss for future earthquake is necessary. So that it is possible to categorize buildings based on their performance levels.
- A way to improve seismic performance of the built environment (e.g. Buildings) through the development of performance-oriented procedures and guidelines should be adopted in Ethiopian Building Code Standards for the design of new buildings and performance evaluation of existing buildings to control damages in structure's life.
- It is better to design our buildings in such a way that an adequate reserve of resistance is available to prevent failure in the case of major earthquake.

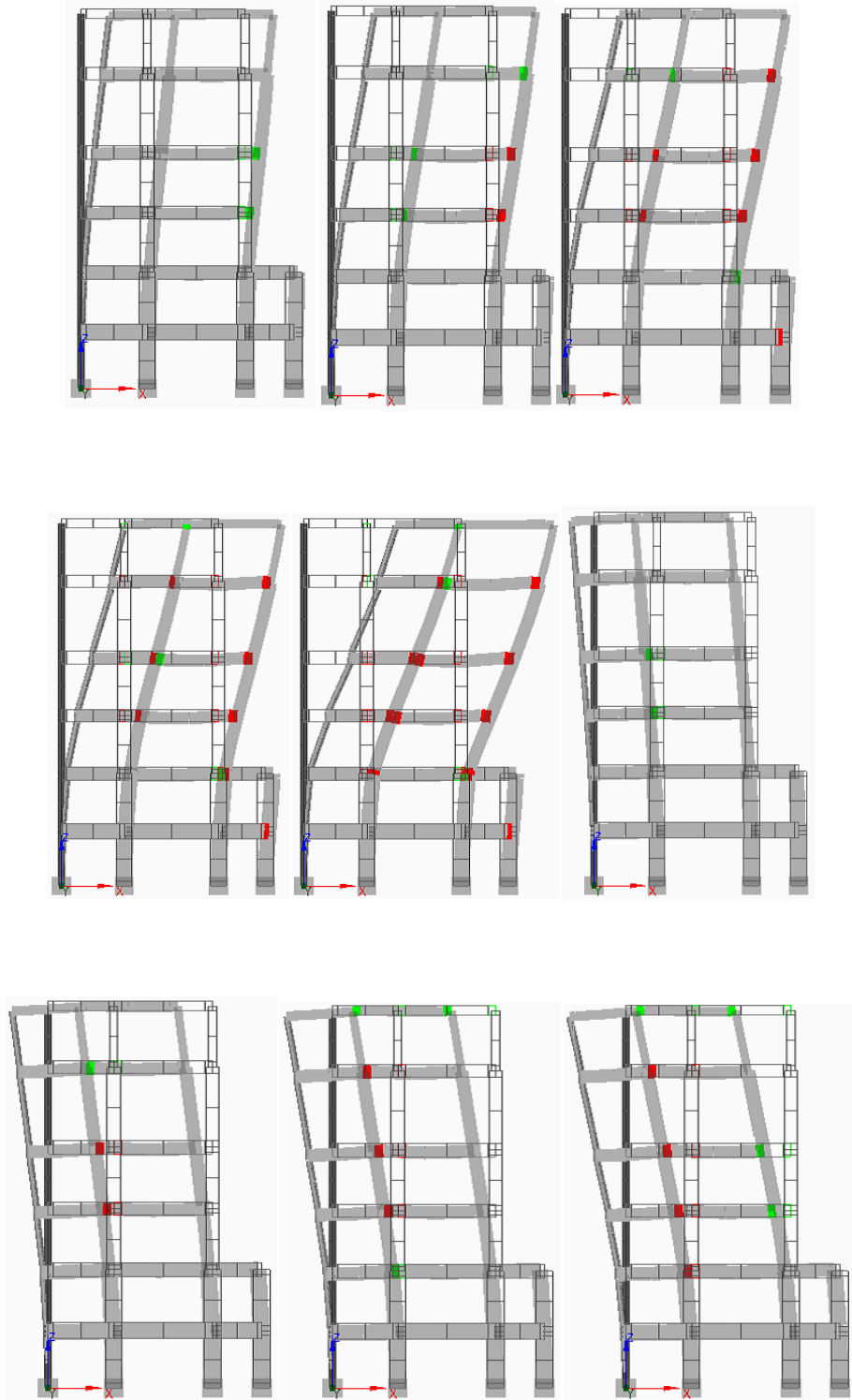
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## APPENDIX-A Plastic Hinge Locations

Location of weak points and potential failure modes that structure would experience in case of a seismic event is expected to be identified by pushover analyses.



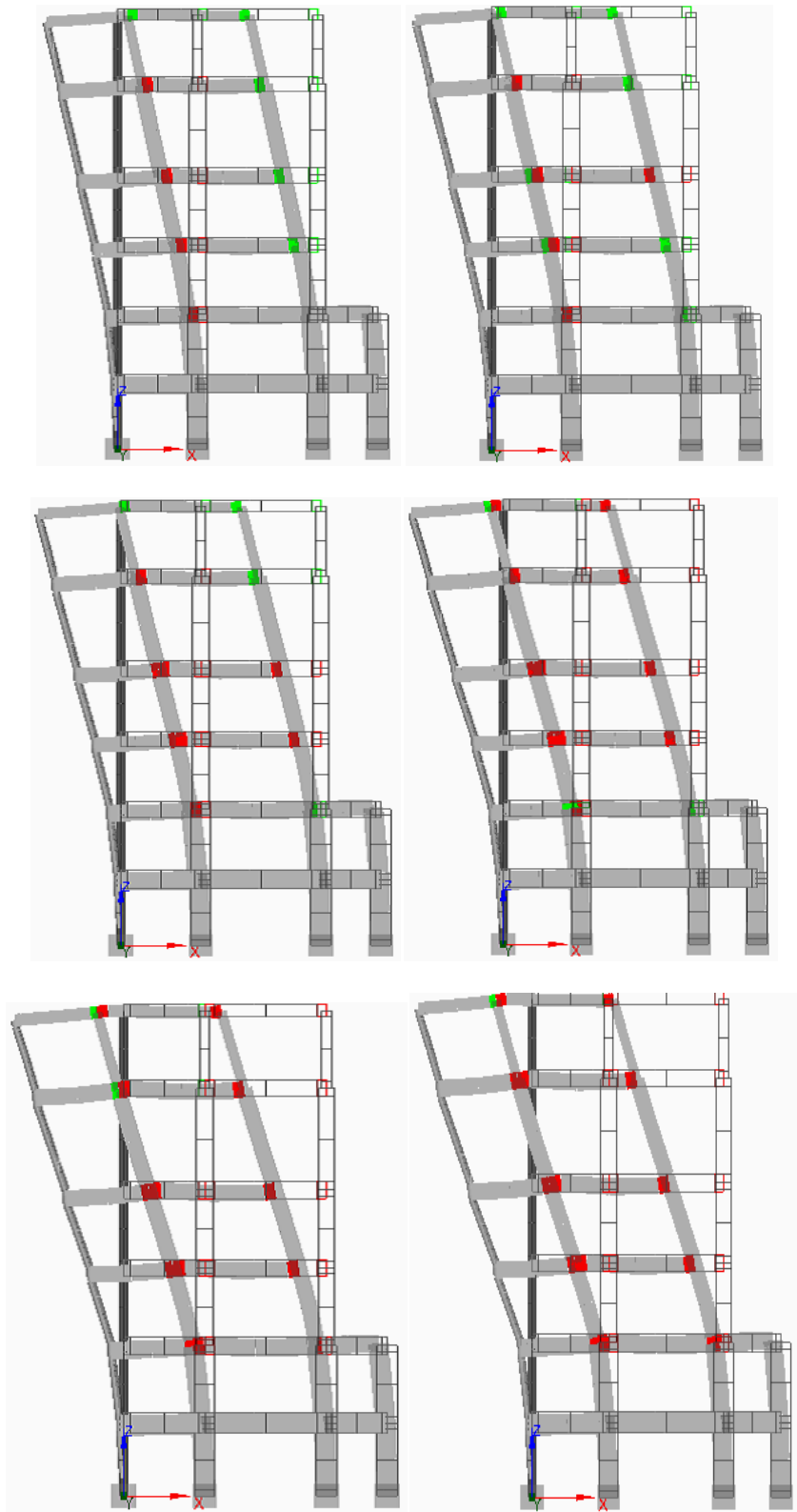
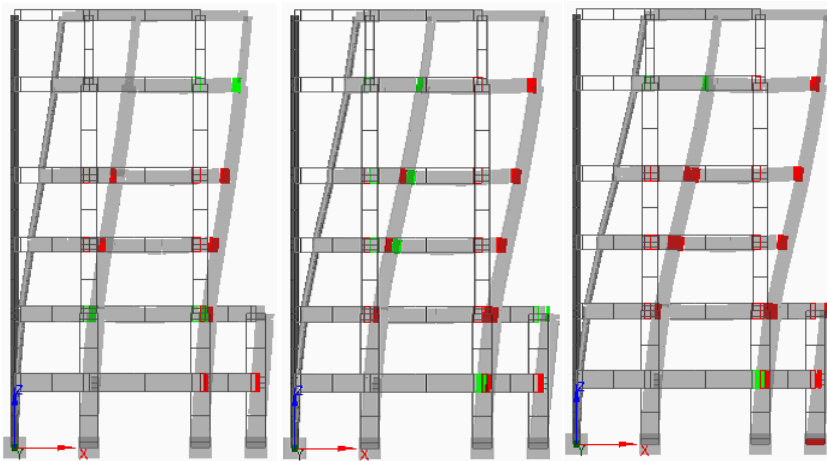
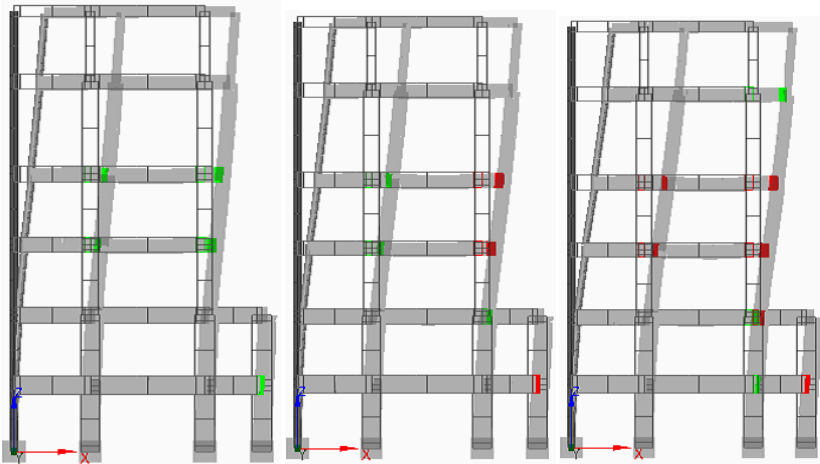
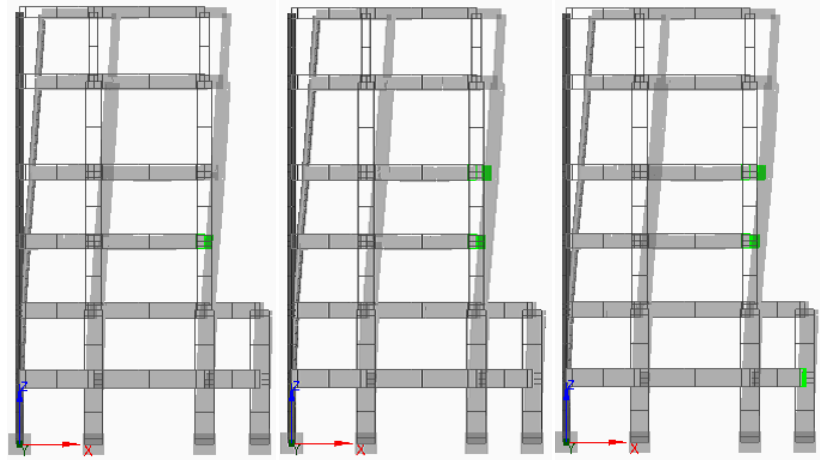
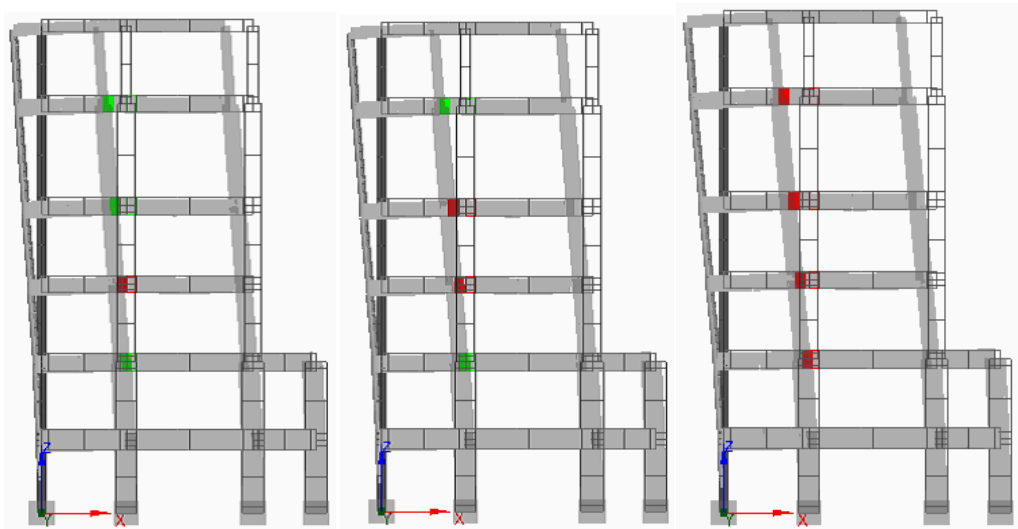
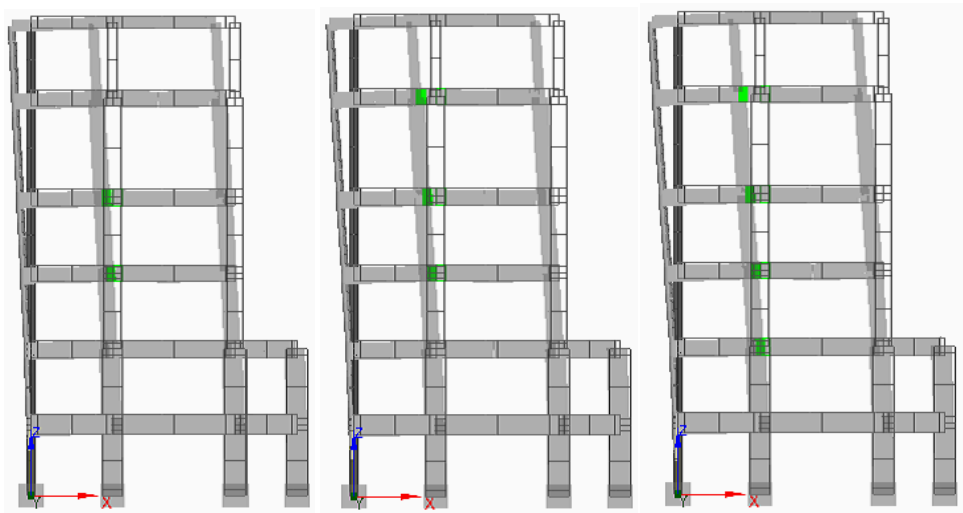
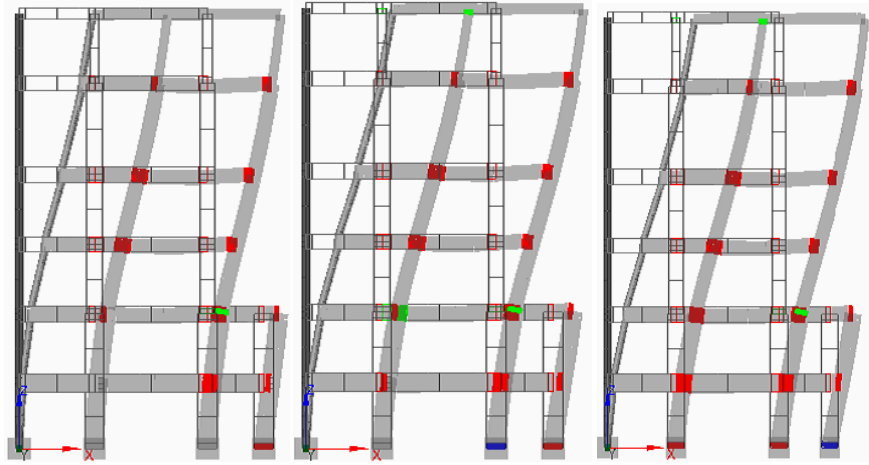
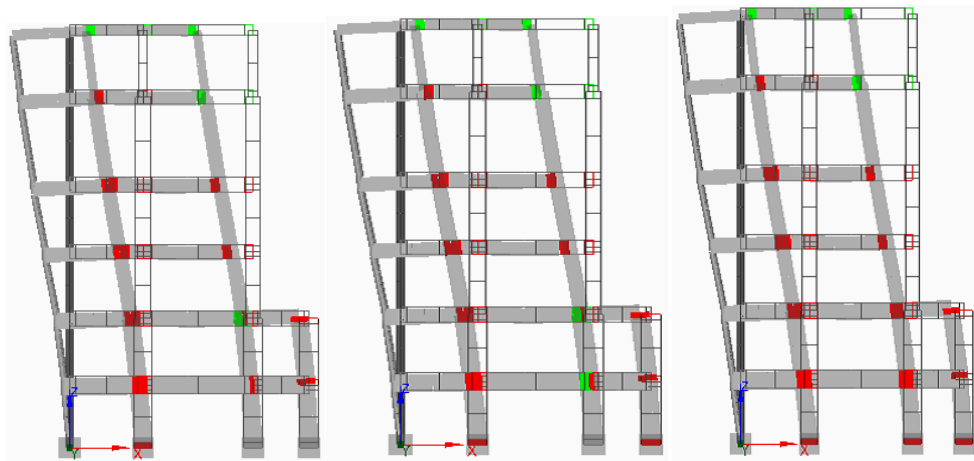
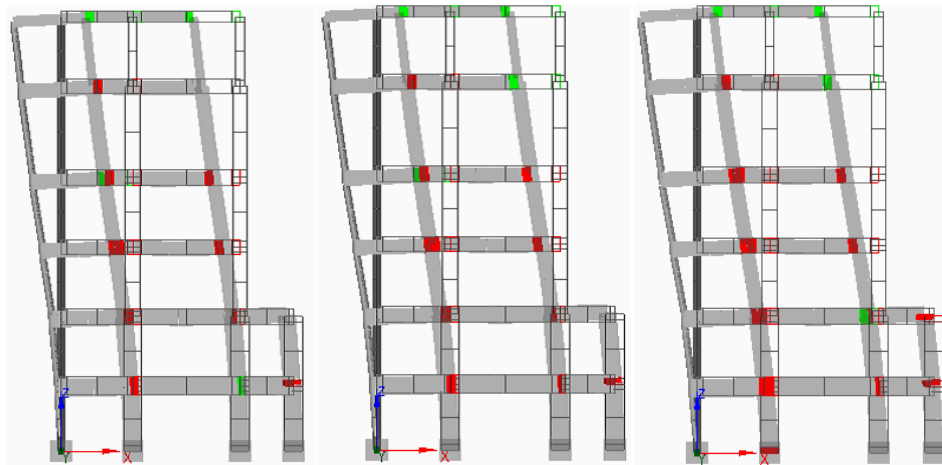
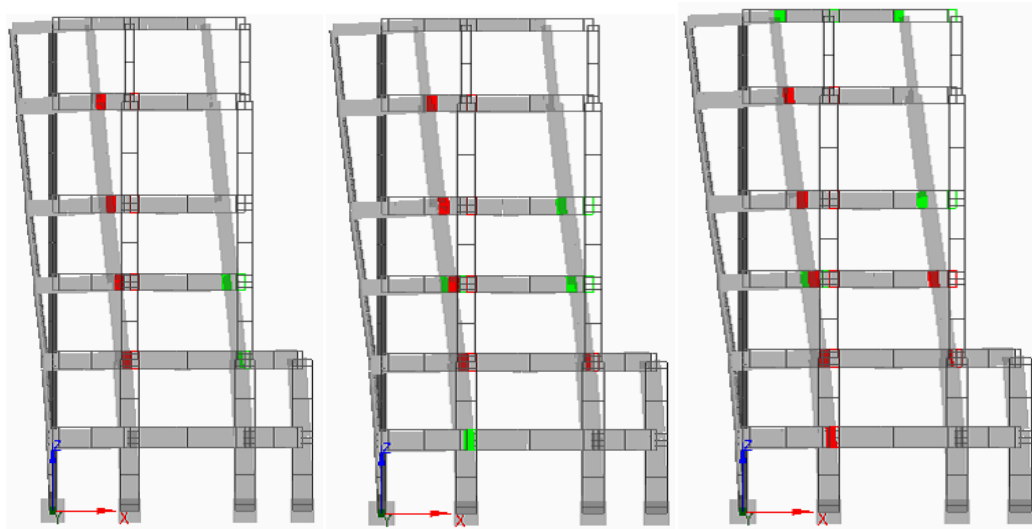


Figure A-1: Sequences of failures: Seismostruct (Triangular/Triangular Opposite)







● LS Rupture of steel

Figure A-2: Sequences of failures: Seismostruct (Uniform/Uniform Opposite)

## **APPENDIX-B Seismic Performance Criteria**

### Target Building Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Due to inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected. The Structural Performance Level of a building shall be selected from four discrete Structural Performance Levels and two intermediate Structural Performance Ranges defined in this section.

The four discrete structural Performance Levels are:

- I. Immediate Occupancy (S-1),*
- II. Life Safety (S-3),*
- III. Collapse Prevention (S-5),*
- IV. Structural Performance not Considered (S-6).*

The intermediate Structural Performance Ranges are:

- I. Damage Control Range (S-2) ,and*
- II. Limited Safety Range (S-4).*

### **Response Limits**

According to ATC-40 the response limits fall into two categories: [8]

1. Global building acceptability limits. These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift. (Table B-1)
2. Element and component acceptability limits. Each element (frame, wall, diaphragm, or foundation) must be checked to determine if its components respond within acceptable limits. (Table B-2 and Table B-3)

Table B-1: Deformation Limits

<i>Performance Level</i>				
<i>Inter story Drift Limits</i>	<i>Immediate Occupancy</i>	<i>Damage Control</i>	<i>Life Safety</i>	<i>Structural Stability</i>
<b>Maximum Total drift</b>	0.01	0.01-0.02	0.02	$0.33 \frac{V_i}{P_i}$
<b>Maximum inelastic drift</b>	0.005	0.005-0.015	No limit	No limit

Table B-2: Numerical Acceptance Criteria For Plastic Hinge Rotations in Reinforced concrete beams, In radians [8]

<i>Component Type</i>	<i>Performance Level</i>				
	<i>Primary</i>			<i>Secondary</i>	
	<i>IO</i>	<i>LS</i>	<i>SS</i>	<i>LS</i>	<i>SS</i>

**1. Beams controlled by flexure**

$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.	$\frac{V}{b_w d \sqrt{f'_c}}$					
$\leq 0.0$	C	$\leq 3$	0.005	0.02	0.025	0.02	0.05
$\leq 0.0$	C	$\geq 6$	0.005	0.01	0.02	0.02	0.04
$\geq 0.5$	C	$\leq 3$	0.005	0.01	0.02	0.02	0.03
$\geq 0.5$	C	$\geq 6$	0.005	0.005	0.015	0.015	0.02
$\leq 0.0$	NC	$\leq 3$	0.005	0.01	0.02	0.02	0.03
$\leq 0.0$	NC	$\geq 6$	0.0	0.005	0.01	0.01	0.015
$\geq 0.5$	NC	$\leq 3$	0.005	0.01	0.01	0.01	0.015
$\geq 0.5$	NC	$\geq 6$	0.0	0.005	0.005	0.005	0.01

Table B-3: Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete columns, in radians [8]

<i>Component Type</i>	<i>Performance Level</i>				
	<i>Primary</i>			<i>Secondary</i>	
	<i>IO</i>	<i>LS</i>	<i>SS</i>	<i>LS</i>	<i>SS</i>

**Columns controlled by flexure**

$\frac{P}{A_g f'_c}$	Trans. Reinf.	$\frac{V}{b_w d \sqrt{f'_c}}$					
$\leq 0.1$	C	$\leq 3$	0.005	0.01	0.02	0.015	0.03
$\leq 0.1$	C	$\geq 6$	0.005	0.01	0.015	0.01	0.025
$\geq 0.4$	C	$\leq 3$	0.0	0.005	0.015	0.01	0.025
$\geq 0.4$	C	$\geq 6$	0.0	0.005	0.01	0.01	0.015
$\leq 0.1$	NC	$\leq 3$	0.005	0.005	0.01	0.005	0.015
$\leq 0.1$	NC	$\geq 6$	0.005	0.005	0.005	0.005	0.005
$\geq 0.4$	NC	$\leq 3$	0.0	0.0	0.005	0.0	0.005
$\geq 0.4$	NC	$\geq 6$	0.0	0.0	0.0	0.0	0.0

## APPENDIX-C FORTRAN 95 Code: Moment-Curvature Relationship

```
! FUNCTIONS:

! Moment Curvature Relationship - Doubly reinforced concrete.

!*****

! PROGRAM: Moment curvature relationship

! PURPOSE: To establish the moment curvature relationship

!*****

  Program MKAPPA

  Implicit none

  INTEGER I, N, k

  Real :: Ems, Emc, fc, FSS, FCC, FCCp, FCCR, FCT, AS, t, h, d, b, P, fyd, esyd, fctm, ChPt

  REAL :: STLSTRS, STLSTRSnet, STLSTRS'net, STLSTRS', Straintenmax, Maxststrn = 0.01

  REAL :: alpha = 0.85, X, ecrk,

  REAL, ALLOCATABLE :: ecm(:), NXD(:), es(:), Moment(:), CURVATURE(:)

  CHARACTER (len = 20) FILENAME

  WRITE (*,*) "ENTER THE FILE NAME"

  READ*, FILENAME

  OPEN (UNIT=1, FILE=FILENAME, Status =' REPLACE', ACTION=' WRITE' )

! ++++++

  100 FORMAT (5F10.2)

  110 FORMAT (I10, F10.2, 5F10.6)

  105 FORMAT (F12.0)

! ++++++

  WRITE (*,*) "ENTER THE COMPRESSIVE STRENGTH(cylindrical)"

  READ*, fc
```

```

WRITE (*,*) "ENTER THE STEEL YIELD STRENGTH"

READ*, fyd

WRITE (*,*) "ENTER THE BOTTOM STEEL AREA, ENTER THE TOP STEEL AREA, total height,
effective depth, d prime and width of the beam"

READ*, AS, AS', h, d, d', b

WRITE (*,*) "ENTER THE NUMBER OF DATA POINTS"

READ*, N

ALLOCATE (MOMENT(N+1))

ALLOCATE (CURVATURE(N+1))

ALLOCATE (ecm(N+1))

ALLOCATE (NXD(N+1))

ALLOCATE (es(N+1))

EMC = 4700*SQRT(fc)

EMS = 200000

esyd = fyd/EMS

fctm = 0.21*(fc**(2./3.))

ecrk = 2.0*0.21*(fc**(1./6.))/4700.0

X = 0.5*d

Do I = 1, N

    ecm(i) = i*0.0035/N

1000 Straintenmax = (h-X)/X*ecm(i)

    es(i) = (d-X)/X*ecm(i)

    es'(i) = (X-d')/X*ecm(i)

    IF (es(i).GE. Maxststrn) EXIT

    IF (es'(i).GE. Maxststrn) EXIT

    IF (ecm(i) .LE. 0.002) THEN

```

```

FCC = (b*fc)*1000*alpha*ecm(i)*X*(0.5-250*ecm(i)/3.0)

IF (Straintenmax .LE. 0.5*ecrk) THEN

    FCT = EMC*ecm(i)*b*(h-X)*(h-X)/(2*X)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

        STLSTRS = fyd

    END IF

    STLSTRSnet= STLSTRS- (d-X)/X* ecm(i)* EMC

ELSE IF (Straintenmax .LE. ecrk) THEN

    ChPt= 0.5*ecrk*(h-X)/Straintenmax

    FCT = EMC*ecm(i)*b*ChPt*ChPt/(2*X) + fctm*b*(h-X-ChPt)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

        STLSTRS = fyd

    END IF

    STLSTRSnet= STLSTRS- fctm

ELSE

    ChPt= 0.5*ecrk*(h-X)/Straintenmax

    FCT = EMC*ecm(i)*b*ChPt*ChPt/(2*X) +
    fctm*b*(ChPt)+fctm*b/0.6*((ecrk*X/ecm(i))**0.4)*((h-X)**0.6-(2.*ChPt)**0.6)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

        STLSTRS = fyd

```

```

        END IF

        STLSTRSnet= STLSTRS- fctm

        END IF

        FSS = STLSTRSnet*AS !net force

        IF (es'(i).LE. esyd) THEN

            STLSTRS' = EMS*es(i)

        ELSE

            STLSTRS' = fyd

        END IF

        STRESS'net= STLSTRS'-1000*alpha*(X-d')/X* ecm(i)*(1-250*(X-d')/X* ecm(i))* fc

        FSS' = STLSTRS'net*AS' !net force

        ELSE

            FCCp = 2*alpha*fc*b*0.002*X/(3.0*ecm(i))

            FCCR = fc*alpha*b*X*(1-0.002/ecm(i))

            FCC = FCCp + FCCR

            IF (es'(i).LE. esyd) THEN

                STLSTRS' = EMS*es(i)

            ELSE

                STLSTRS' = fyd

            END IF

            IF (4*X/7. LE. X-d') THEN

                STRESS'net= STLSTRS'-1000*alpha*(X-d')/X* ecm(i)*(1-250*(X-d')/X* ecm(i))* fc

            ELSE

                STRESS'net= STLSTRS'-fc

            END IF

            FSS' = STLSTRS'net*AS' !net force

```

```

IF (Straintenmax .LE. ecrk/2.0) THEN

    FCT = EMC*ecm(i)*b*(h-X)*(h-X)/(2*X)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

        STLSTRS = fyd

    END IF

    STLSTRSnet= STLSTRS- (d-X)/X* ecm(i)* EMC

    FSS = STTRSnet*AS    !net force

ELSE IF (Straintenmax .LE. ecrk) THEN

    ChPt= 0.5*ecrk*(h-X)/Straintenmax

    FCT = EMC*ecm(i)*b*ChPt*ChPt/(2*X) + fctm*b*(h-X-ChPt)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

        STLSTRS = fyd

    END IF

    STLSTRSnet= STLSTRS- fctm

    FSS = STLSTRSnet*AS    !net force

ELSE

    ChPt= 0.5*ecrk*(h-X)/Straintenmax

    FCT = EMC*ecm(i)*b*ChPt*ChPt/(2*X) +
    fctm*b*(ChPt) + fctm*b/0.6*((ecrk*X/ecm(i))**0.4)*((h-X)**0.6-(2.*ChPt)**0.6)

    IF (es(i).LE. esyd) THEN

        STLSTRS = EMS*es(i)

    ELSE

```

```

        STLSTRS = fyd

        END IF

        IF ((d-X)/(h-X)* ecm(i).GE. ecrk) THEN

        STLSTRSnet= STLSTRS- fctm*(ecrk/((d-X)/(h-X)* ecm(i)))**0.4

        ELSE

        STLSTRSnet= STLSTRS-fctm

        END IF

        FSS = STLSTRSnet*AS      !net force

        END IF

    END IF

    t= P-(FCC+ FSS'-FCT-FSS)

    If (t>10.0) Then

        X = X - 0.001

        GO TO 1000

    Else if (t < -10.0) Then

        X = X + 0.001

        GO TO 1000

    ELSE

        NXD(i) = X

    END IF

    IF (es(i).GE.0.01) EXIT

    IF (ecm(i) .LE. 0.002) THEN

    Moment (i) = FCC*(X*(1/3.0-62.5*ecm(i))/(0.5 -250*ecm(i)/3.0)+d-X) - FCT*((h-X)/3-
    h+d)+FSS*(d-d')-P*(d-0.5*h)

```

```

ELSE

      Moment (i) = FCCp*(d-X + 0.625*0.002*X/ecm(i)) + FCCR*(d-0.5*X+0.001*X/ecm(i))-
FCT*((h-X)/3-h+d)+ FSS'*(d-d')- P*(d-0.5*h)

      END IF

      CURVATURE(i) = ecm(i)/NXD(i)*1E06

      END DO

      k = i -1

      WRITE (1,100) 0,0,0,0,0

      DO I = 1,k

            WRITE (1,100) CURVATURE(i), Moment(i)*1E-06, NXD(i), es(i)*1E06, ecm(i)*1E06

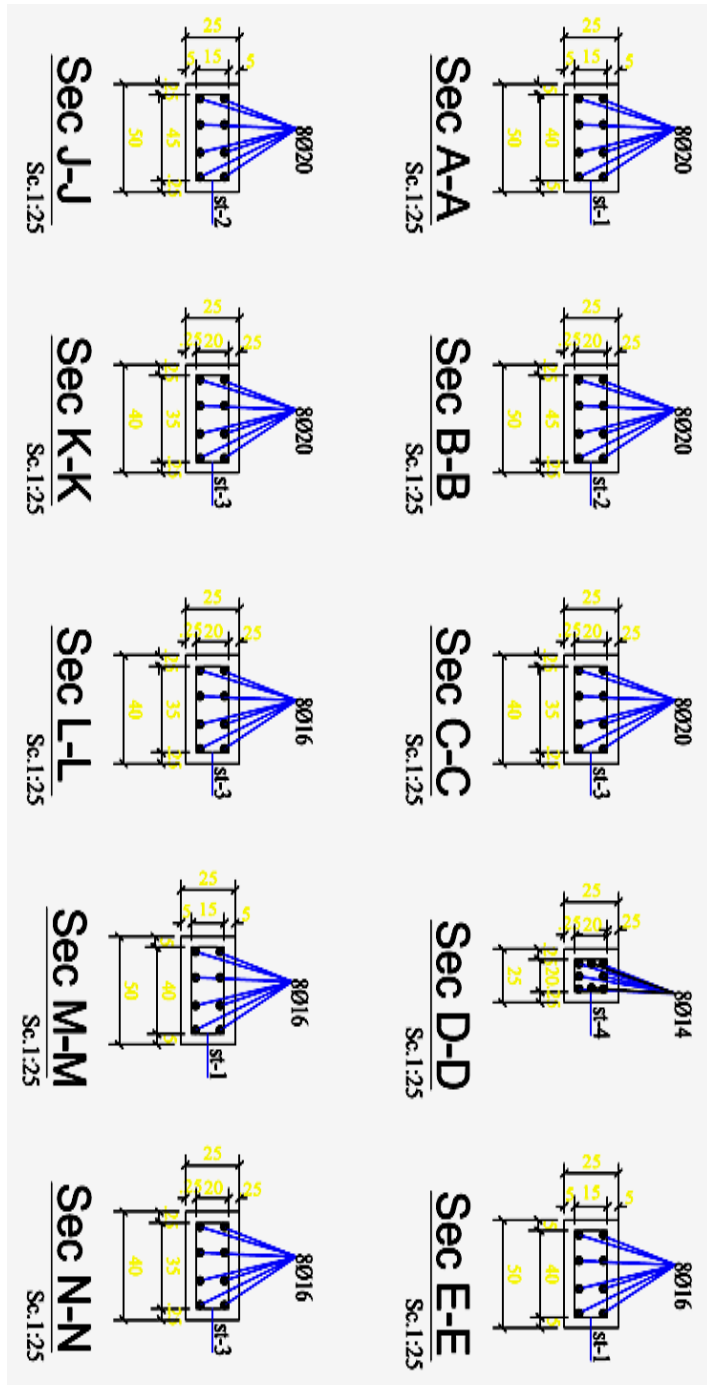
      END DO

      CLOSE (UNIT = 1)

      End Program MKAPPA

```

## APPENDIX-D Section Details



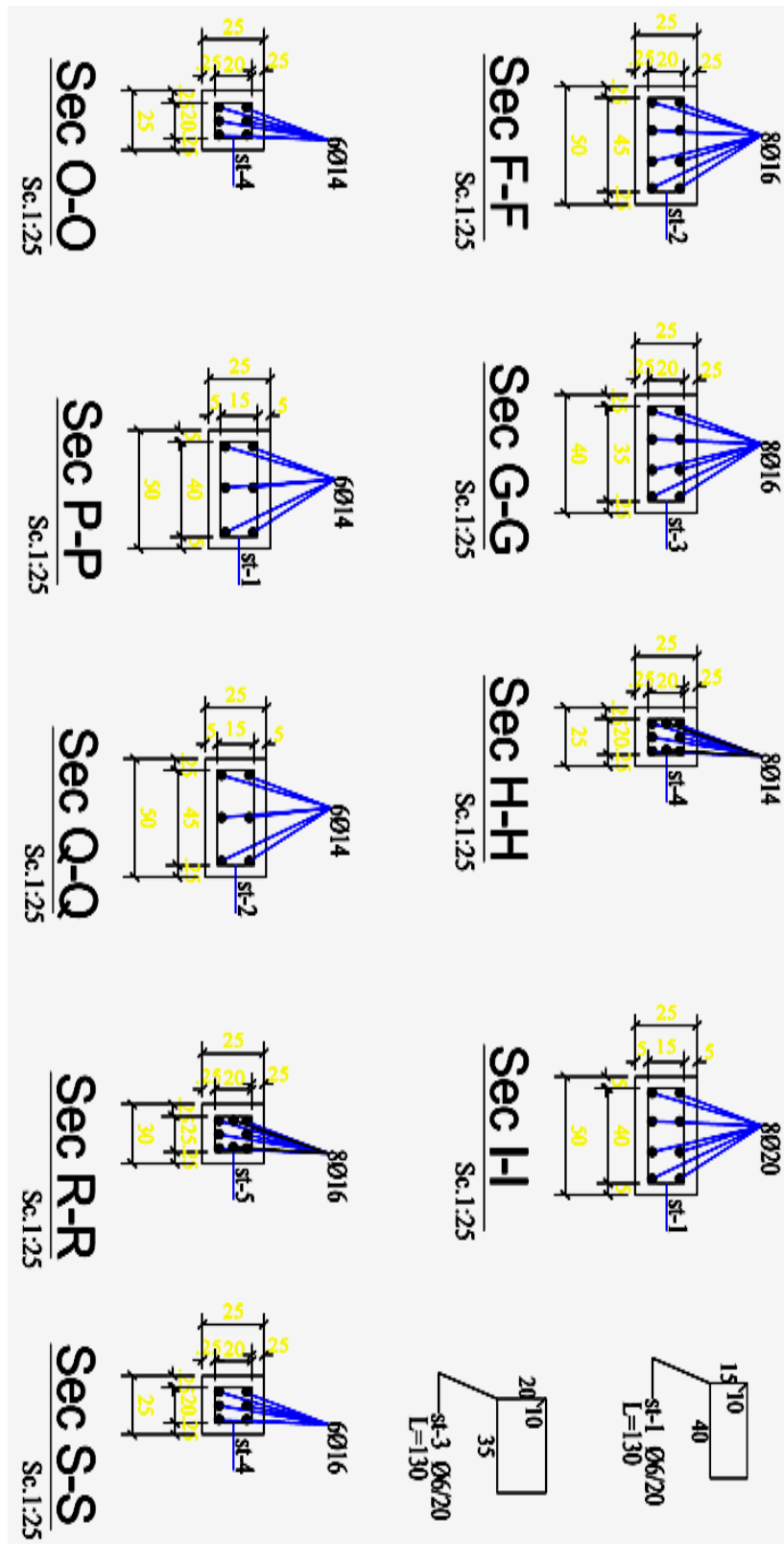


Figure D-1: Column Section Details

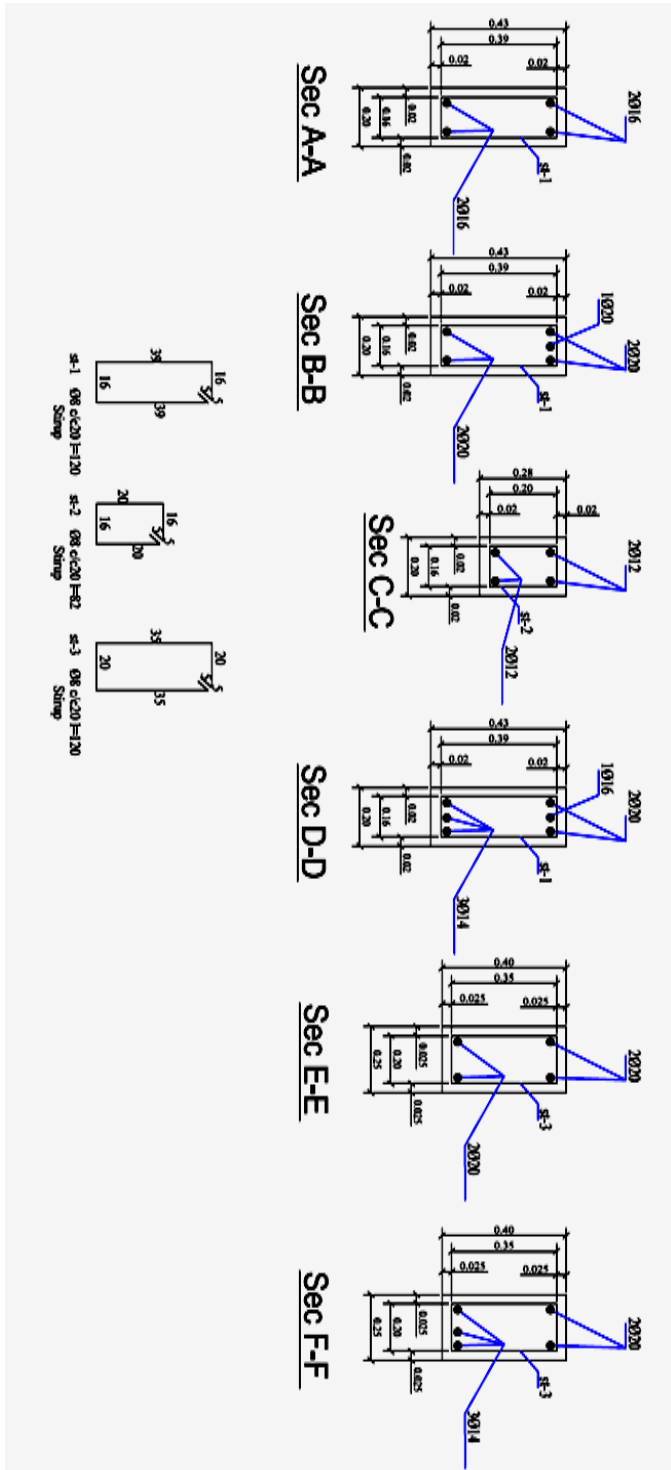


Figure D-2: Beam Section Details