ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

Performance Assessment of Reinforced Concrete Planar Frame Using Nonlinear Analysis

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 stream)

By

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(October 2015)

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Abstract

Ground excitation is the major cause of failure in reinforced concrete structures constructed in seismically active zone. This can be due to either the unpredictable nature of the seismic excitation or due to lack of proper structural design, detailing and construction. During seismic excitation the response of structures should be estimated appropriately. For response determination of structures, codes fail to predict the real time response of structures. Hence existing performance assessment should be done using a reliable and easily applicable analysis procedure.

The revised Ethiopian building code (EBCS-EN-2014) shows a major change on the expected peak ground acceleration (from 0.05g to 0.1g) for Addis Ababa. For the past several years large numbers of buildings are designed and constructed using the previous building code. The performance of these structures is highly affected by this change. There for it is mandatory to assess the performance of the existing structures using provisions of revised code. This study focuses on condominium buildings as they are constructed in large scale all over the country and as they are areas where people are congregated. The study is done under the assumption that premature failures are not expected to happen.

A planar frame was taken from the representative 3D case study building. The capacity demand comparison was done for each member of the frame using EURO CODE and ACI provision for stiffness reduction factor. The frame fails to satisfy minimum code requirements even after the consideration of moment redistribution.

In the aim of saving the structure load reduction was performed as engineering solution. The solution improve the structural performance but still it is unsafe.

A mechanistic solution was also proposed to utilize the reserve capacity of the structure. The solution being pushover analysis with fiber based modeling which capture the nonlinear properties. Seismostruct and SAP2000 software were used for analysis.

Element level verification was done using both software followed by frame level verification which will address effect of redundancy. At relatively low loads prediction obtained from both software give consistent result with the experimentally obtained data At a relatively high load the result obtained from Seismostruct gives a better prediction of the experimental result.

With the aim of investigating the stiffness reduction factor recommended by EURO CODE and ACI extent and progress of crack checked. For this particular frame it is found out that the columns reduction factor is closer to ACI provision while the beam reduction factor is closer to EURO CODE provision.

Finally the case study building was assessed for its performance level, for soil class B the result shows that the structure's performance level do not satisfies drift requirements for operational and immediate occupancy but it is adequate for life safety and collapse prevention.

From the study it is found that the use of pushover analysis results in 25.6% increment in shear capacity from Seismostruct and 16% increment in shear capacity from SAP2000 for this particular frame. The higher increment from Seismostruct is expected due to the plastic zone consideration which is a plastic hinge in SAP2000. The advantage obtained by using the pushover analysis may increase or decrease depending on the rate of progressive hinge formation, length of the plastic hinge and other parameters.

Keywords: seismic excitation, base shear, pushover, performance level, plastic hinge, plastic zone, fiber based modeling

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NOTATIONS

Fb is the base shear

Ki is the base shear distribution coefficients

ESA is equivalent static analysis

V Design Shear

V_i Shear force in story i

P_i Total gravity load at story i

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

C is for conforming

IO Immediate Occupancy

LS Life Safety

SS Structural Stability

DC Damage Control

1 CHAPTER ONE - INTRODUCTION

As the knowledge and understanding of human beings increases towards seismic action it will result in better understanding of the performance of structures. Structures that are constructed during the past several years will not behave in the same manner as those structures which are constructed currently and in the future. In order to study the real time behavior of structures after construction, a new method of studying the structure which is different from code based analysis should be implemented.

The need for changes in the existing seismic design methodology available 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 the load various structural elements yields sequentially. Consequently, at each event, the structure experiences decrease in stiffness which results in increase in the lateral deformation.

1.1 OBJECTIVE

1.1.1 GENERAL OBJECTIVE

The aim of this study is to evaluate the structural performance of an existing building to seismic action based on code provision and provide an engineering solution and also provide a simpler seismic analysis approach.

1.2.1 SPECIFIC OBJECTIVES

- To take a case study buildings located in Addis Ababa city and assess its performance for the revised peak ground acceleration of EBCS-8 and propose a feasible engineering solution for the same structure and structures with a similar complexity.

- To verify the reliability of the pushover analysis approach of software (SAP2000 V14 and SEISMOSTRUCT) in structural performance evaluation using conventional pushover analysis.
- Element level and frame level verification of pushover analysis result of SAP2000 and 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 SAP2000 V14 and SEISMOSTRUCT.

1.2 SCOPE OF THE STUDY

This paper will address only low rise buildings, specifically condominium building. For the assessment of the case study only working drawing is used. Any type of damage is not considered and so the following parameters are neglected.

the following parameters are neglected on the planar analysis.

- existing damages
- soil structure interaction
- bond slip

For the plot of the capacity curve soil type is not considered. The capacity is compared with the base shear demand for different soil class. Any premature failure like shear failure is not expected to happen and if a member is shear liable then it will be checked at that specific member.

1.3 STATEMENT OF THE PROBLEM

The current structural design practice for earthquake resistant structures 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 1995 is very low compared to various research projects which depict the need for code revision and on the revised code the Peak Ground Acceleration is changed from 0.05g to 0.1g this will dramatically change the structural performance of the existing structures as they are designed and constructed using the former code. Furthermore, in the design and construction process the structural design may not be checked rigorously which aggravates the problem in earthquake performance of buildings.

1.4 RESEARCH STRATEGY

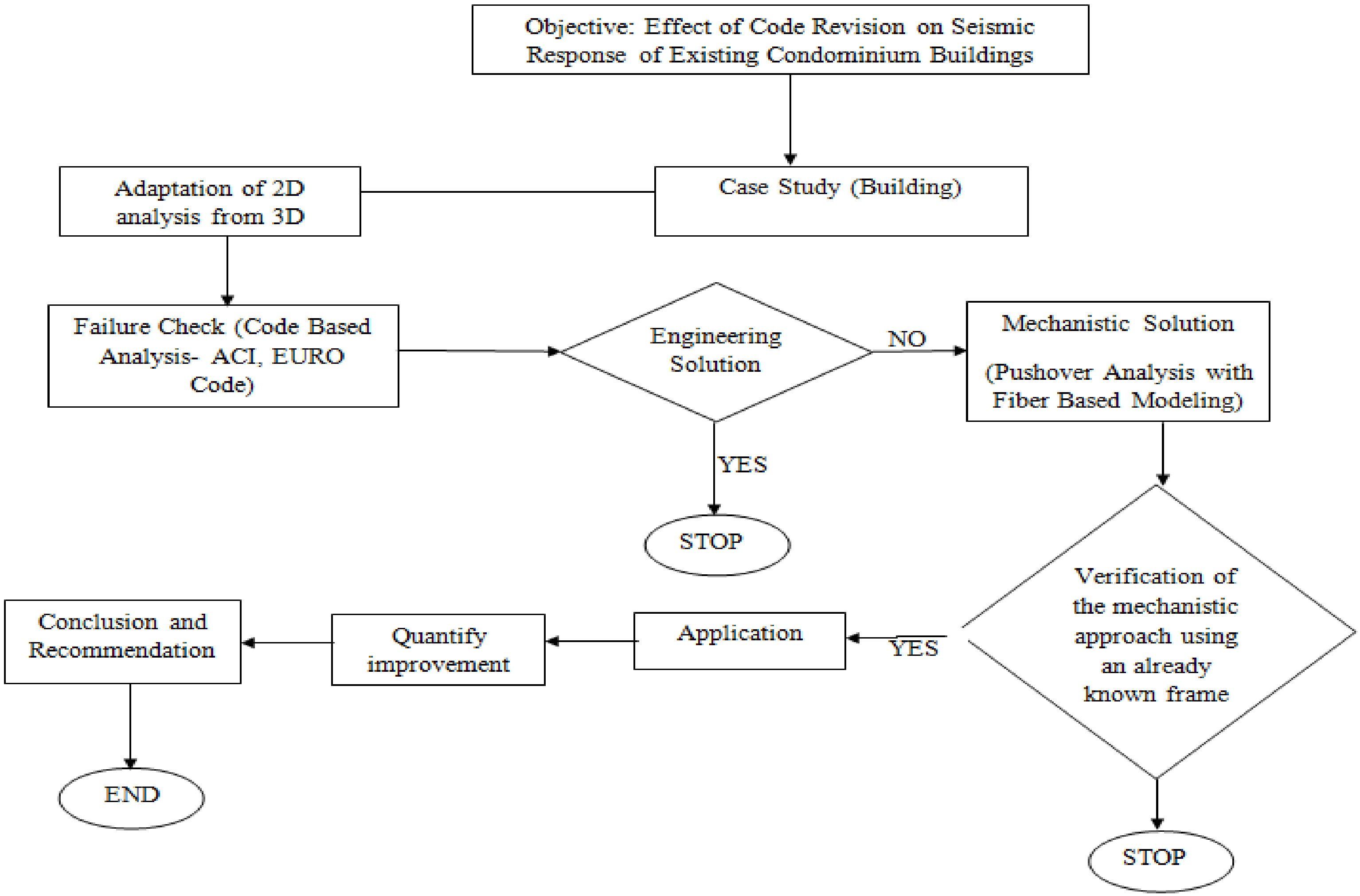
Currently the Ethiopian building code standard is revised, approved and is ready for application. On the revised code there is a major change on the peak ground acceleration, which is a changed from 0.05g to 0.1g for Addis Ababa. This change will highly affect the seismic performance and response of structures that are designed and constructed using the previous code.

There are many structures constructed using the previous codes which needs structural assessment and here it is chosen to assess the structural performance of precast condominium buildings as they are constructed in large scale all over the country. The performance assessment is done by using the new code provision at first.

The case study building was taken and analyzed in 3D model and then a frame is taken out of the 3D model to compare with a planar frame as a verification of planar frame analysis, then capacity demand comparison was done using the EURO CODE and ACI provision, because failure of structure was observed, engineering solution was proposed.

Since the proposed engineering solution was not adequate mechanistic solution by using fiber based modeling and pushover analysis was done after verification .Finally performance assessment was done on the case study building using the proposed method of analysis by SAP2000 and SEISMOSTRUCT. Finally the result was discussed following by conclusion and recommendation.

RESEARCH STRATEGY



2 CHAPTER TWO - LITRETURE REVIEW

The static pushover analysis is becoming a popular tool for seismic performance evaluation of existing and new structures. The expectation is that the pushover analysis will provide adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The existing building can become seismically deficient since seismic design code requirements are constantly. Further, Ethiopian buildings built over past years are seismically deficient because of lack of awareness regarding seismic behavior of structures and in the new revised building code the Peak Ground Acceleration will be changed. The seismic data (Peak Ground Acceleration) of Addis Ababa is 0.1g which is double compared to the previous code which was 0.05g. The widespread damage especially to RC buildings during earthquakes exposed the construction practices being adopted around the world, and generated a great demand for seismic evaluation and retrofitting of existing building stocks.

The pushover analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, and plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure is analytically computed. This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.

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]. [6]

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). [4]

Pushover analysis is nonlinear static analysis in which provide ‘capacity curve’ of the structure, it is a plot of total base force vs. roof displacement. The analysis is carried out up to failure; it helps determination of collapse load and ductility capacity of the structure. The pushover analysis is a method to observe the successive damage state of the building. In Pushover analysis structure is subjected to monotonically increasing lateral load until the peak deformation of the structure is obtained as shown in *Figure 2.1*[3]

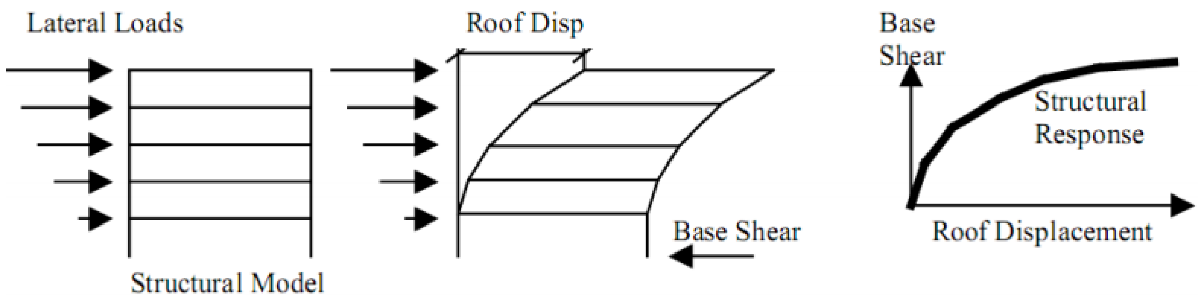


Figure 2.1 Static Approximation Used In the Pushover Analysis

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating its strength and deformation demands in the design of structures for earthquakes resistant by means of static nonlinear analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift

The nonlinear- static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behavior.

The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformations demands for elements that have to form in elastically in order to dissipate the energy imparted to the structure.
- Consequences of the strength deterioration of individual elements on behavior of structural system.
- Consequences of the strength deterioration of the individual elements on the behavior of the structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.
- Estimates of the inter-story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength, and the foundation system.

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. The performance levels are as follows:

- Immediate Occupancy (IO)

- Life Safety (LS)
- Collapse Prevent (CP)

Immediate Occupancy Performance Level (S-1):- Immediate Occupancy is the post-earthquake damage state in which only very limited structural damage has occurred. In the primary concrete frames, there will be hairline cracking.

Damage Control Performance Range (S-2):- Structural Performance Range S-2, Damage Control, is the continuous range of damage states that less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level.

Life Safety Performance Level (S-3):- Structural Performance Level S-3, Life Safety, is the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. In the primary concrete frames, there will be extensive damage in the beams. There will be spilling of concrete cover and shear cracking in the ductile columns

Limited Safety Performance Range (S-4):- Structural Performance Range S-4, Limited Safety is the continuous range of damage states between the Life Safety and Collapse Prevention levels

Collapse Prevention Performance Level (S-5):- Structural Performance Level S-5, Collapse Prevention, is the building is on the verge of experiencing partial or total collapse. In the primary concrete frames, there will be extensive cracking and formation of hinges in the ductile elements

Performance point – The performance point is the point where capacity curve crosses demand curve.

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 structures fundamental period of vibration, T

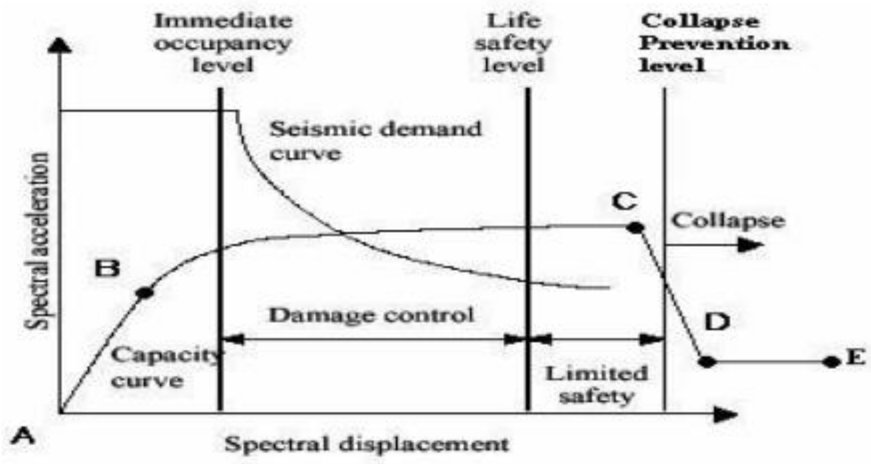


Figure 2.2 Building Performance Level

From the Response Spectrum and Base Shear vs. Roof Displacement plot, the Target Displacement, δt, 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 fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which seismic performance evaluation of the structure is to be performed. The target displacement serves as an estimate of the global displacement of the structure is expected to experience in a design earthquake. It is the roof displacement at the center of mass of the structure. In the pushover analysis it is assumed that the target displacement for the MDOF (Multiple degree of freedom) structure can be estimated as the displacement demand for the corresponding equivalent SDOF (Single degree of freedom) system transformed to the SDOF domain through the use of a shape factor. This assumption, which is always an approximation, can only be accepted within limitations and only be accepted within limitations and only if great care is taken in incorporating in the predicted SDOF displacement demand all the important ground motion and structural response characteristics that significantly affect the maximum displacement of the MDOF structure. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to higher mode effects.

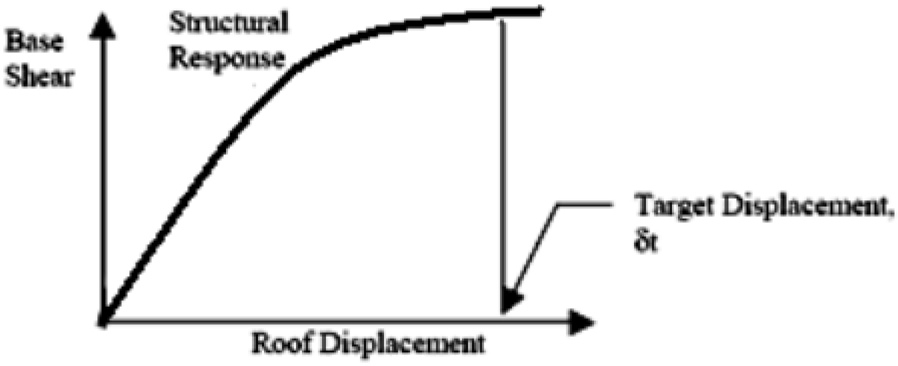


Figure 2.3 Target Displacement

Seismic analysis of buildings can be categorized depending upon the sophistication of modeling adopted for the analysis. Buildings loaded beyond the elastic range can be analyzed using Non-Linear static analysis, but in this method one would not be able to capture the dynamic response, especially the higher mode effects. This is pushover analysis. There is no specific code for NLSA. This procedure leads to the capacity curve which can be compared with design spectrum/DCR of members and one can determine whether the building is safe or needs strengthening and its extent.

The capacity of structure is represented by pushover curve. The most convenient way to plot the load deformation curve is by tracking the base shear and the roof displacement. The pushover procedure can be presented in various forms can be used in a variety of forms for the use in a variety of methodologies. As the name implies it is a process of pushing horizontally, with a prescribed loading pattern, incrementally, until the structure reaches the limit state.

Level-1: It is generally used for single story building, where at a single concentrated horizontal force equal to base shear applied at the top of the structure and displacement is obtained.

Level-2: In this level, lateral force in proportion to story mass is applied at different floor levels in accordance with IS:1893-2002 (Part-I) procedure, and story drift is obtained.

Level-3: In this level lateral force is applied in proportion to the product of story masses and first mode shape elastic model of the structure. The pushover curve is constructed to represent the first mode response of structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This procedure is valid for tall buildings with fundamental period of vibration up to 1 sec.

Level-4: This procedure is applied to soft story buildings, wherein lateral force in proportion to product of story masses and first mode of shape of elastic model of the structure, until first yielding, the forces are adjusted with the changing the deflected shape.

Level-5: This procedure is similar to level 3 and level 4 but the effect of higher mode of vibration in determining yielding in individual structural element are included while plotting the pushover curve for the building in terms of the first mode lateral forces and displacements. The higher mode effects can be determined by doing higher mode pushover analysis. For the higher modes, structure is pushed and pulled concurrently to maintain the mode shape.

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 behavior 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 story into a one degree-of-freedom system to complex three dimensional finite element formulations that describe the structural behavior 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:[7]

(1) Global Models: These models constitute the simplest form of all nonlinear models, requiring the least computational power. In these models, the nonlinear behavior 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 2-4) 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.

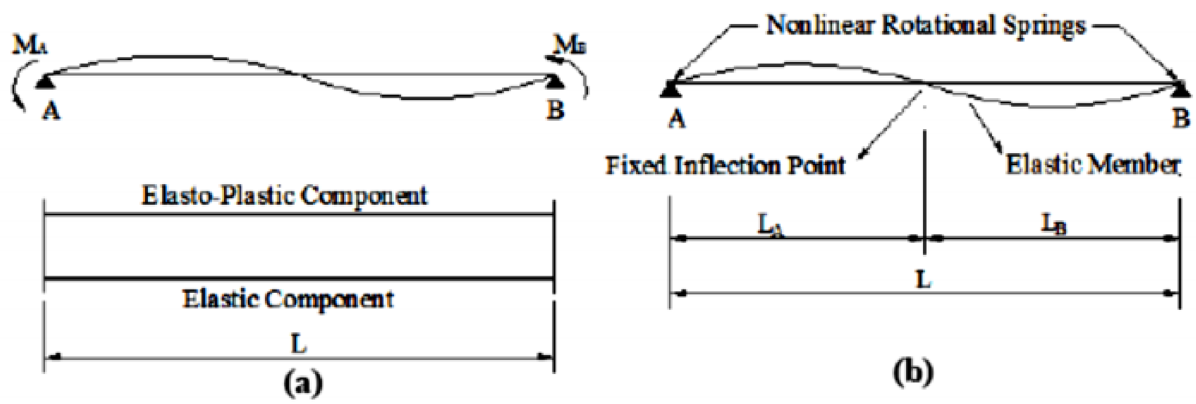


Figure 2.4 Lumped Plasticity Elements: (a) Parallel Model (Clough and Johnston, 1966); (b) Seies Model (Giberson, 1967)[5]

In this thesis Fiber model (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 2-5 for beams were used. In Fiber model, the element is subdivided into longitudinal fibers as shown in Figure 2-6.

Force deformation behavior of hinges:

- Point A corresponds to unloaded condition.
- Point B represents yielding of the element.
- The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins.
- The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable.
- The residual resistance from D to E allows the frame elements to sustain gravity loads.

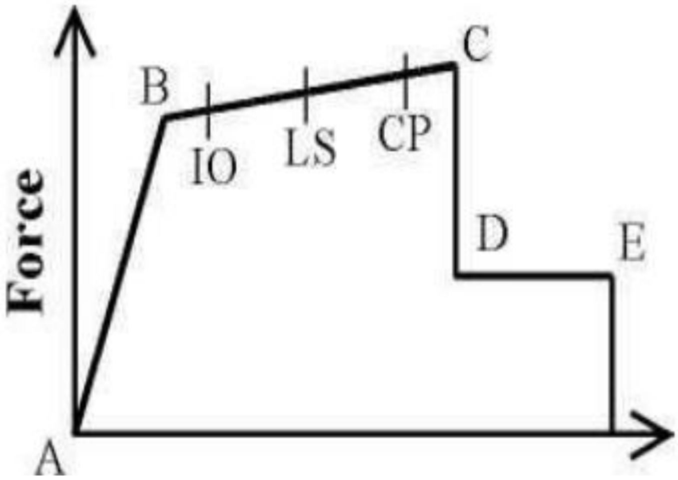


Figure 2.5 Hinge Behavior Curve

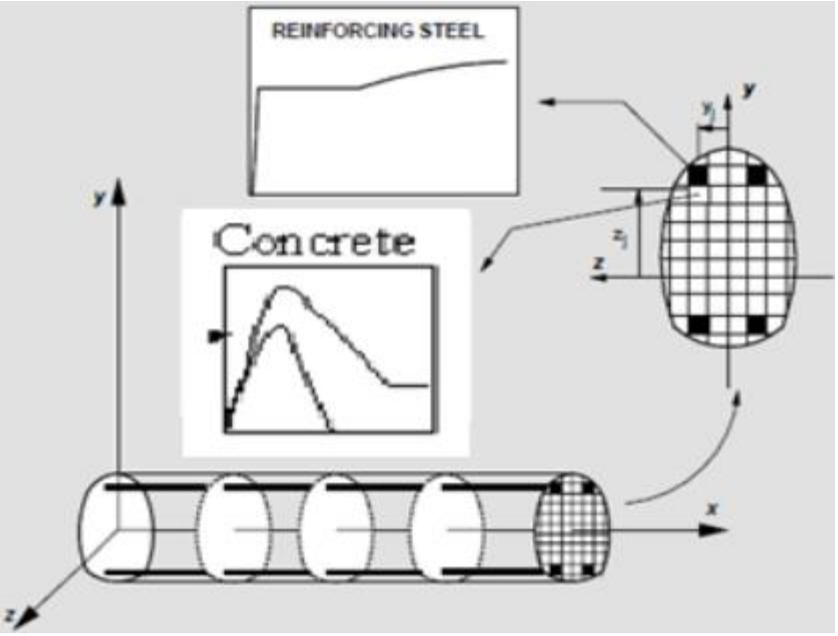


Figure 2.6 Fiber Element

Method of analysis can be either linear or nonlinear and the preference of analysis method depends on the objective of the study.

The advantage of analytical nonlinear analysis procedure under monotonic and pushover loading lies in its inherent and accurate consideration of shear effects and significant second order mechanisms within a simple modeling process suitable for use in practice.[1]

The modern use of nonlinear analysis focuses mostly on these three fields: [2]

- Complex / stringent safety requirement structures (e.g. nuclear plants, dams, bridges)
- Virtual laboratory for parametric studies
- Existing structures (evaluation, repair, rehabilitation)

Three types of non-linarites may arise:

Material (or physical) nonlinearity

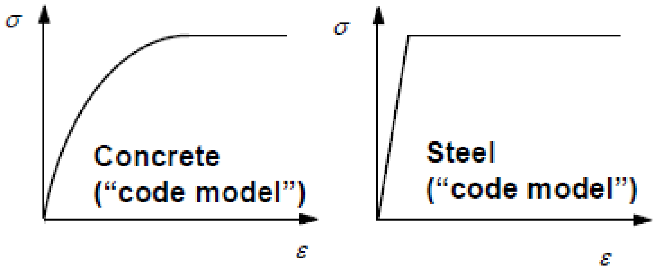


Figure 2.7 Material nonlinearity

Geometrical nonlinearity

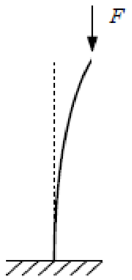


Figure 2.8 Geometric Nonlinearity

Contact nonlinearity

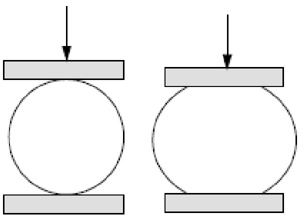


Figure 2.9 Contact Nonlinearity

3 CHAPTER THREE - CODE BASED PERFORMANCE ASSESSMENT

3.1 INTRODUCTION

This chapter contains the structural assessment report of an office building constructed in 2001 around ‘Enkulal Fabrica’ Addis Ababa, Ethiopia. The photographic view is shown in Figure 1-1. It is a 5 story building with flat roof and four bays in East-West direction and eight bays in North-South direction.

Figure 3.1 Photographic View of the building

For the assessment of the building a planar frame is used, but first the analysis outputs from the (3D) analysis are compared with the planar analysis. planar frames are preferred from three dimensional (3D) frames due to the time required to analyze and determine all parameters necessary for the performance analysis of the structure being considered and also it is convenient to model planar frame using **SeismoStruct** which will be done later on this paper.

The design ground acceleration $a_g = \gamma_I \times a_{gR} = 1.0 \times 0.10g = 0.1g$ is determined using the reference peak ground acceleration of 0.1g for Addis Ababa and building importance class II.

The building assessment is conducted by carrying out a new structural analysis using as-built measurements for the overall and cross-sectional dimensions of the structure and verifying whether or not the compliance criteria for the ultimate limit state, i.e. no-collapse requirement is satisfied.

3.2 VERIFICATION OF PLANAR FRAME ANALYSIS

First applicability of planar frame analysis is verified on one of the frames. This is done by analyzing the 3D frame with its loads, and then the planar frame is also analyzed by using the same loading. The result of these two analyses is compared based on shear force, bending moment and story drift values using the Ethiopian Building Code Standard (EBCS).

Since the analysis result obtained from planar fame analysis is close enough to the 3D frame analysis, planar frame is used for performance evaluation of the building from the results obtained the building fails to satisfy minimum requirement for ultimate limit state according to both ACI and Euro Codes.

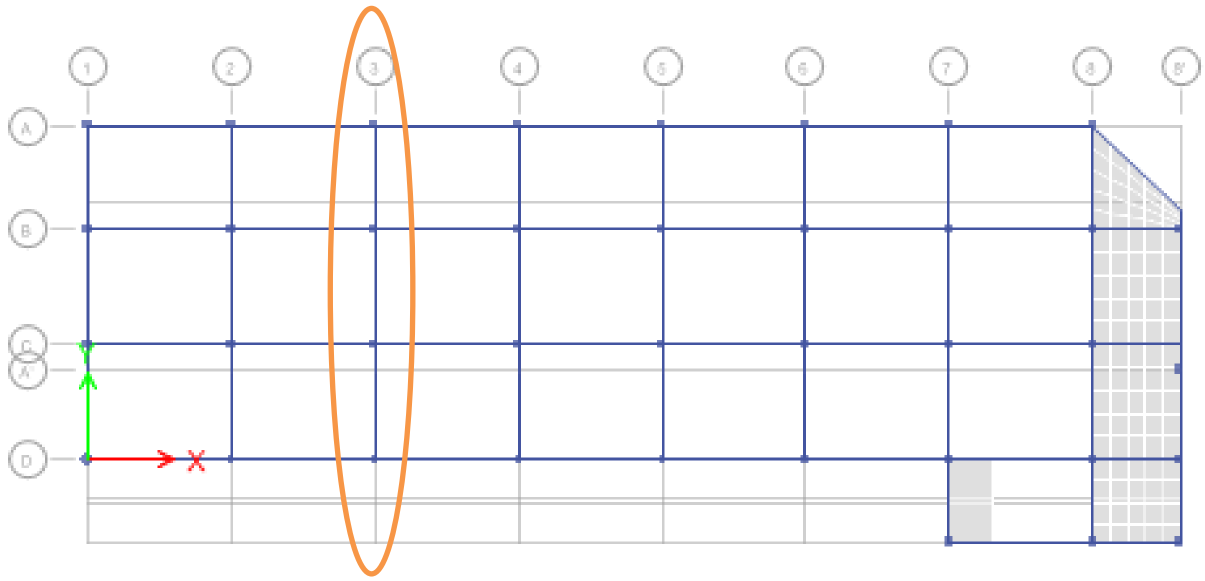


Figure 3.2 Selected axis from the 3D ‘axis-3’

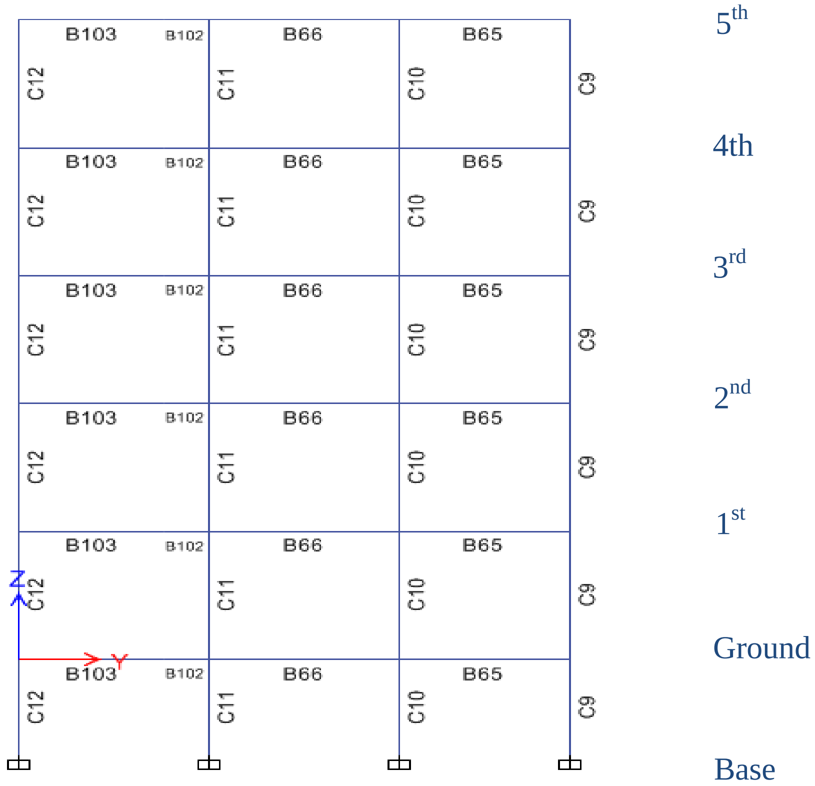


Figure 3.3 Column labels of 3D frame on axis-3

3.3 VERIFICATION OF COLUMN SHEAR DISTRIBUTION

A closer value of column shear force distribution is obtained between the planner and the 3D frame. This is because, in the analysis of the three dimensional (3D) frame torsional constant of all beams and columns is reduced to 0.1 which makes the three dimensional (3D) frame to act as planar frame.

The analysis result of the three dimensional (3D) frame is summarized in the table below.

Table 3.1 Shear force of columns on axis-3 from 3D frame analysis

	Column Shear (kN)					
	5^{th}	4^{th}	3^{rd}	2^{nd}	1^{st}	Ground
C9	14.2815	36.8674	42.1073	55.521	61.8044	67.1418
C10	25.9454	48.8202	89.5982	104.804	114.667	116.61
C11	26.2724	48.3388	87.87	101.679	110.611	113.816
C12	11.3909	28.3425	32.5176	42.4149	46.7826	50.4613

In similar manner, the shear force on each column of the planar frame is analyzed and summarized on the table shown below. *Figure 3.4* shows the planar frame on axis-3 with its story shear force at each story level.

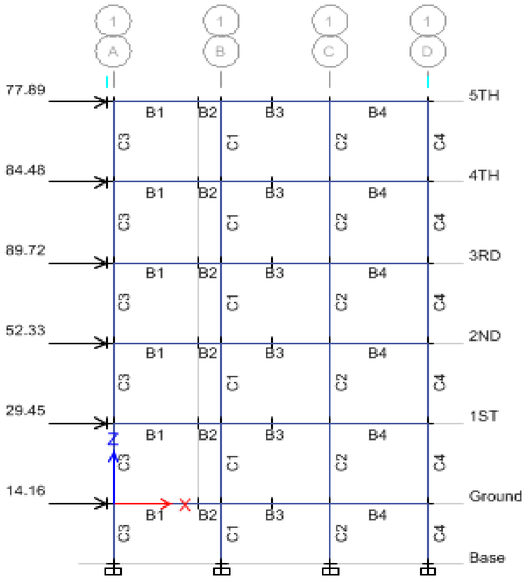


Figure 3.4 Planar frame on axis-3 with its story shear

Table 3.2 Shear force of columns on axis-3 from planar frame analysis

	Column Shear(kN)					
	5th	**4th**	**3rd**	**2nd**	**1st**	**Ground**
C4	13.8201	34.9432	40.9579	53.7298	59.7903	64.5458
C2	24.9359	46.8073	86.3969	100.999	110.469	111.752
C1	25.2402	46.5092	85.1334	98.5664	107.26	109.512
C3	13.894	34.1092	39.6049	51.1231	56.3461	62.2199

3.4 VERIFICATION OF STORY DRIFT

The following table summarizes the drift value obtained from the two analyses.

Table 3.3 Drift value of 3D and planar frames

	Drift		
	3D frame	Planar frame	Percentage Variation
5th	0.002163	0.001777	18%
4th	0.004289	0.003412	20.4%
3rd	0.005196	0.004088	19.6%
2nd	0.004831	0.003793	20.8%
1st	0.00416	0.003282	21.4%
Ground	0.002106	0.001688	19.9%

From the above two analysis it is clearly shown that if planar frame analysis is used the result will be close to the analysis result of the three dimensional (3D) frame. So it is possible to use planar frame for performance analysis.

3.5 CAPACITY DEMAND RATIO

In order to know whether the elements (i.e. beam and columns) fail under the applied load, it is necessary to determine the capacity to demand ratio. This value is the ratio between the required capacities of the member to the actual capacity of that member. The comparison is done based on moment ratio for beams and based on reinforcement ratio for the case of columns

The capacity to demand ratio is calculated according to both ACI and Euro Codes. The basic difference observed between this codes applied here in the analysis is, the stiffness modifiers used for beams and columns. The value of dead and live loads are shown on the following diagrams.

The capacity to demand ratio for both beams and columns according to ACI Code is summarized below on the tables. The values shown here below are only for typical results.

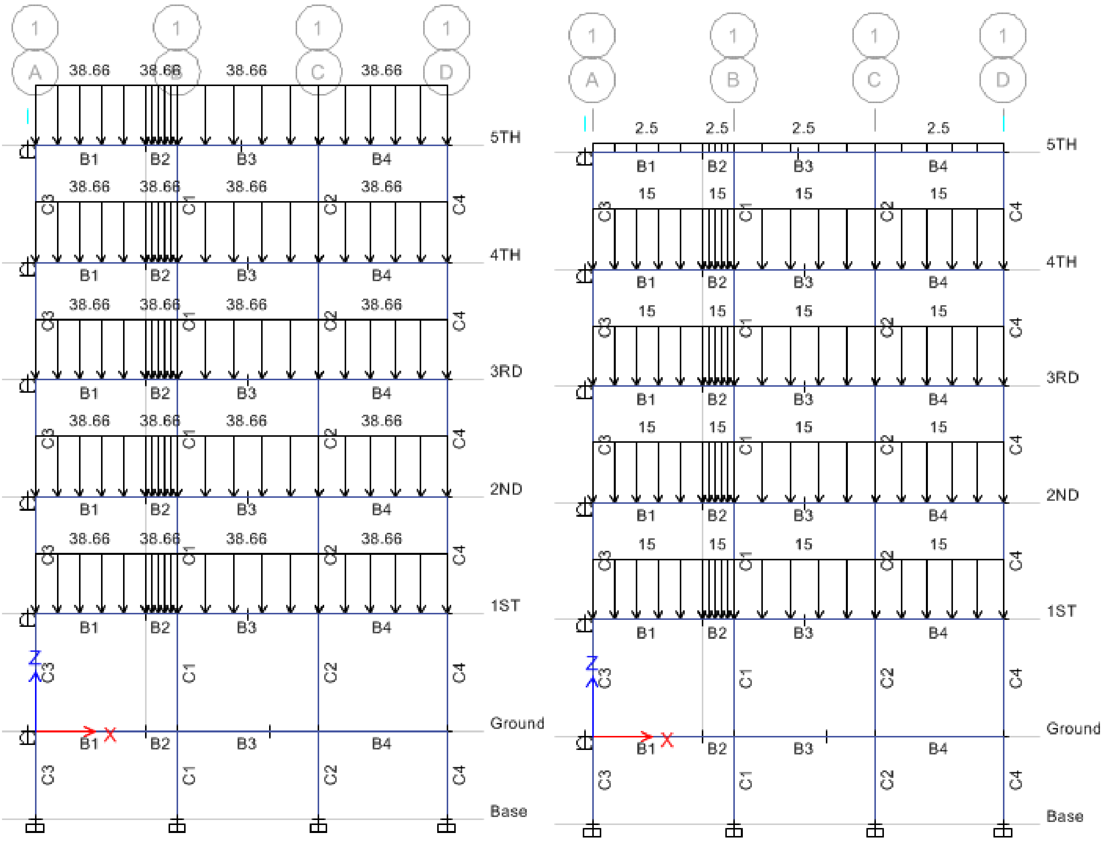


Figure 3.5 a) Dead load and b) Live load on the planar frame

Table 3.4 Typical capacity to demand ratio for moment capacity of grade beam according to ACI

Ground Floor Beam									
GFB-1 250/400									
Axis 3	**Axis A-B**			**Axis B-C**			**Axis C-D**		
Moment	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve
Provided reinforcement	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16
Action, M_{sd}	-111.7	107.9	-64.8	-107.9	103.7	-107.4	-114	110.6	-118.1
Capacity, M_{cap}	-78	61	-78	-78	61	-78	-78	61	-78
Demand to Capacity	1.43	1.77	0.83	1.38	1.70	1.38	1.46	1.81	1.51

Table 3.5 Typical capacity to demand ratio for moment capacity of base column according to ACI

COL Id.	b	h	P	M_2	M_3	ν	μ_2	μ_3	ω	$A_{s,tot}$	No ø20 bars	As, Provided	Demand to Capacity
1-A	400	400	255.4	5.1	179.0	0.141	0.007	0.247	0.58	3022	9.62	8ø20	1.06
2-B	450	450	1310.7	26.9	-290.6	0.571	0.026	-0.281	0.59	3890	12.39	10ø20	1.15
3-C	450	450	1375.6	28.3	-289.6	0.600	0.027	-0.280	0.58	3824	12.18	10ø20	1.13
1-D	400	400	216.0	4.3	-178.9	0.119	0.006	-0.247	0.59	3074	9.79	8ø20	1.1

Table 3.6 Typical capacity to demand ratio for moment capacity of grade beam according to Euro Code

Ground Floor Beam									
GFB-1 250/400									
Axis 3	**Axis A-B**			**Axis B-C**			**Axis C-D**		
Moment	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve
Provided reinforcement	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16
Action, M_{sd}	-121.53	109.9	-67.2	-119.72	101.05	-120.18	-125.56	113.5	-133.2
Capacity, M_{cap}	-78	61	-78	-78	61	-78	-78	61	-78
Demand to Capacity	1.56	1.80	0.86	1.53	1.66	1.54	1.61	1.86	1.71

Table 3.7 Typical capacity to demand ratio for moment capacity of base column according to Euro Code

COL Id.	b	h	P	M_2	M_3	ν	μ_2	μ_3	ω	$A_{s,tot}$	No ø20 bars	As, Provided	Demand to Capacity
1-A	400	400	222.2	4.4	141.3	0.123	0.006	0.195	0.37	1928	6.14	8ø20	0.90
2-B	450	450	1312.9	26.9	-230.8	0.572	0.026	-0.224	0.45	2967	9.45	10ø20	0.92
3-C	450	450	1396.9	28.7	-229.6	0.609	0.028	-0.222	0.50	3297	10.50	10ø20	0.96
1-D	400	400	183.6	3.7	-140.3	0.101	0.005	-0.194	0.37	1928	6.14	8ø20	0.9

The above tables show that most beams and columns fail under the application of design load. The beam capacity demand ratio according to Euro code is higher than the ratio according to ACI code. Only one column fails in the analysis made by Euro Code whereas all columns fails in the analysis made by ACI Code. This is because the stiffness modifier for beams in Euro Code is higher than modifiers used in ACI Code.

3.6 ENGINEERING SOLUTION

The first engineering solution for this building applied is moment redistribution, so that the loads will be distributed to members with lower stress from those members that are over stressed. But the capacity to demand ratio of most members is greater than 1.3. This means more than 30% moment redistribution is required which is not allowed according to codes.

The next measure that is taken is to reduce the dead load of building by removing the hollow concrete block (HCB) blocks of the ribbed slab and then the building is analyzed using the new loading.

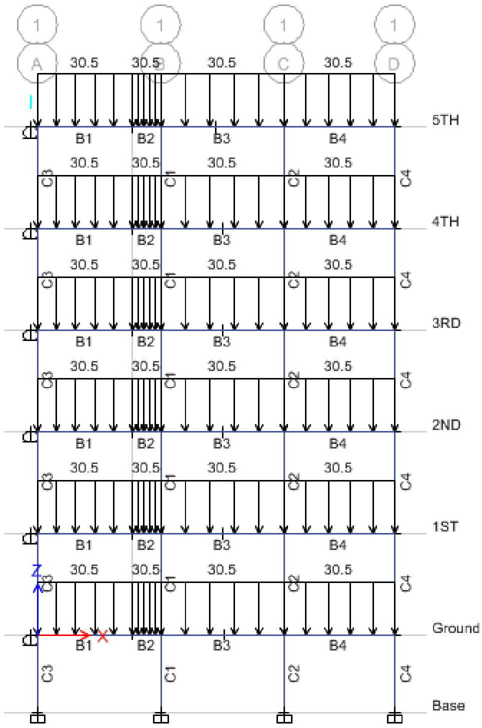


Figure 3.6 Reduced Dead Load

The following tables summarize the story shear according to ACI and EURO code after the removal of the hollow concrete block.

Table 3.8 Column shear force according to ACI

	Column Shear(kN)					
	5th	**4th**	**3rd**	**2nd**	**1st**	**Ground**
C4	11.1156	30.451	35.9462	47.3484	53.0344	57.3213
C2	22.4579	41.0617	74.9363	86.7355	95.167	96.947
C1	22.1451	40.0414	72.6344	83.383	91.6748	94.9808
C3	10.5815	28.9459	33.883	44.0331	49.6238	55.45

Table 3.9 Column shear force according to EURO Code

	5th	4th	3rd	2nd	1st	Ground
C4	12.0922	31.0853	36.1802	47.663	53.012	56.9564
C2	20.6103	39.3333	74.319	87.082	95.5682	97.5252
C1	20.533	38.7082	72.6693	84.3369	92.2019	95.3209
C3	11.7644	29.9732	34.6315	44.9182	49.618	54.7975

Capacity Demand Ratio

Table 3.10 Typical capacity to demand ratio for moment capacity of grade beam according to ACI

Ground Floor Beam									
GFB-1 250/400									
Axis 3	Axis A-B			Axis B-C			Axis C-D		
Moment	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve
Provided reinforcement	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16
Action, M_{sd}	-88.12	79.12	-49.02	-91.19	71.41	-91.38	-94.71	80.69	-99.23
Capacity, M_{cap}	-78	61	-78	-78	61	-78	-78	61	-78
Demand to Capacity ratio	1.13	1.30	0.63	1.17	1.17	1.17	1.21	1.32	1.27

Table 3.11 Typical capacity to demand ratio for moment capacity of base column according to ACI

COL Id.	b	h	P	M_2	M_3	ν	μ_2	μ_3	ω	$A_{s,tot}$	No. ø20 bars	As, Provided	Demand to Capacity
1-A	400	400	281	0.0	148.0	0.155	0.000	0.204	0.35	1824	9.07	8ø20	0.86
1-B	450	450	1269	26.0	254.0	0.553	0.025	0.246	0.35	2308	11.48	10ø20	1.01
1-C	450	450	1326	27.0	252.0	0.578	0.026	0.244	0.39	2572	12.80	10ø20	1.03
1-D	400	400	251	0.0	150.0	0.138	0.000	0.207	0.36	1876	9.33	8ø16	0.89

Table 3.12 Typical capacity to demand ratio for moment capacity of grade beam according to Euro Code

Ground Floor Beam									
GFB-1 250/400									
Axis 3	Axis A-B			Axis B-C			Axis C-D		
Moment	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve	-Ve	+Ve	-Ve
Provided reinforcement	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16	3ø16	3ø14	3ø16
Action, M_{sd}	-107.68	96.01	-59.00	-106.5	87.83	-107.03	-111.3	98.77	-118.49
Capacity, M_{cap}	-78	61	-78	-78	61	-78	-78	61	-78
Demand to Capacity ratio	1.38	1.57	0.76	1.37	1.31	1.37	1.43	1.62	1.52

Table 3.13 Typical capacity to demand ratio for moment capacity of base column according to Euro

COL Id.	b	h	P	M_2	M_3	ν	μ_2	μ_3	ω	$A_{s,tot}$	No. ø20 bars	As, Provided	Demand to Capacity
1-A	400	400	251.0	5.0	114.0	0.138	0.007	0.157	0.26	1355	6.74	8ø16	0.67
1-B	450	450	1276.0	26.0	201.0	0.556	0.025	0.195	0.23	1517	7.55	10ø20	0.75
1-C	450	450	1345.0	27.0	200.0	0.586	0.026	0.194	0.25	1649	8.20	10ø20	0.79
1-D	400	400	222.0	0.0	116.0	0.122	0.000	0.160	0.28	1459	7.26	8ø16	0.71

From the analysis result it is seen that the beams are more safe in ACI than in EURO code and the columns are more safe in EURO code than the in ACI this is because of the stiffness modification factors, since the stiffness of the beam increase in case of EURO code it took more load than the beam in ACI while the column stiffness is reduced in case of EURO code so the column took less load and become safe.

Even though the load is reduced the structure still fails to satisfy minimum requirement for ultimate limit state condition we still need to provide another solution. Again here the capacity to demanded ratio is greater than 1.3, hence moment redistribution is not applicable.

Since the engineering solution provided is not sufficient enough to satisfy minimum code requirements it calls for another solution which will not relay on code provisions but focus on the real time structural response of the structure during seismic excitation. Now, a mechanistic solution is provided as a third option in which the capacity of the structure is analyzed by using pushover analysis by SAP2000 software but first the reliability of the software is checked using an existing data frame level.

Table 3.14 shows the percentage improvement in capacity demand ratio of the beam and the column element before and after the engineering solution

Table 3.14 summary of capacity demand ratio before and after the engineering solution

	Euro Code		**ACI Code**	
	Beam	**Column**	**Beam**	**Column**
With HCB	57%	0.913	48%	1.995
No HCB	37%	0.563	15%	1.67

In case of euro code the demand to capacity ratio for beams was 1.57 in average and this was improved to 1.37 by removing the hollow concrete block. While in case of the columns it was needed only one diameter 20bar additional but after the engineering solution the requirement reduces by half from 0.913 to 0.563 diameter 20 bar.

In case of ACI code the demand to capacity ratio for beams was 1.48 in average and this was improved to 1.15 by removing the hollow concrete block. While in case of the columns it was needed only 1.995 diameter 20bar additional but after the engineering solution the requirement reduces to 1.65 diameter 20bars.

4 CHAPTER FOUR - VERIFICATION

4.1 INTRODUCTION

In this chapter the verification of the SAP2000 and Seismostruct software is made. First element level verification is done. Then frame level verification is followed since element level verification does not consider the high degree of redundancy. For element level verification an isolated column modeled with Seismostruct (which is fiber based modeling software) and SAP2000 software is made and the result was plotted. For frame level verification two frames one by Seismostract and the other by SAP2000 software were done. The column and one of the frames used for verification were experimentally done before, while for the other frame we have only an analytical data.

In the analysis of reinforced concrete structure, for the Seismostruct the nonlinear material behavior is approximately accounted by the implementation of fiber section and the uniaxial material model that can incorporate effects of confinement. For the modeling of discrete member we have section discretization into many fibers and member discretization along the element length. The member discretization is for stiffness based formulation during non-linear analysis. The fiber discretization should be done as well due to the constant stress assumption.
Figure 4.2 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.

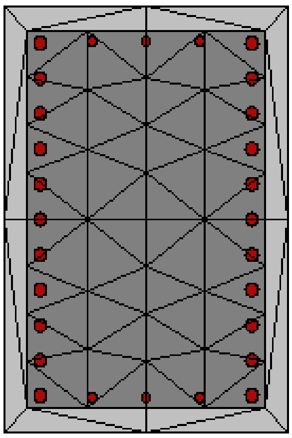


Figure 4.1Fiber based modeled section discretization

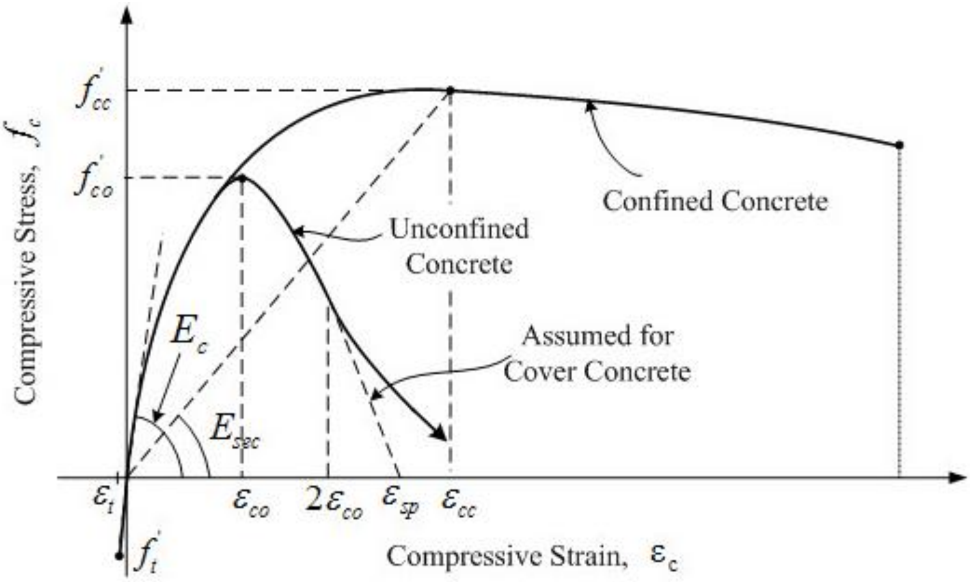


Figure 4.2 Mender's model for confined and unconfined concrete

Elastic Modulus of Concrete: $E_c = 5{,}000\sqrt{f'_{co}}$ MPa

Tensile Strength of Concrete: $f_t = 0.62\sqrt{f'_{co}}$ MPa

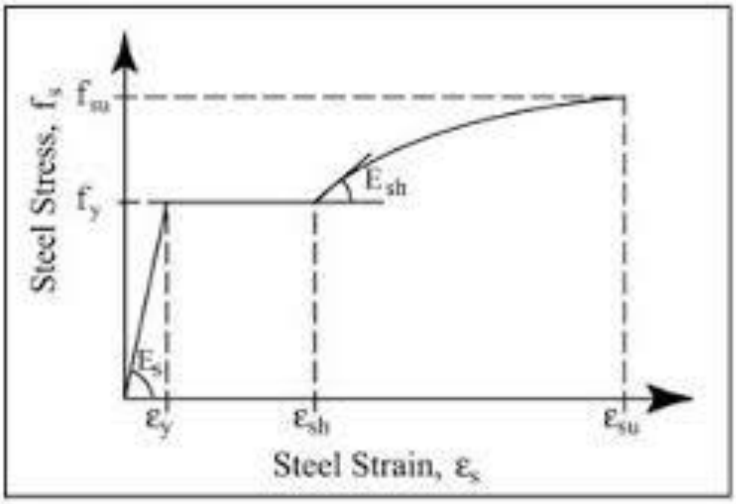


Figure 4.3 Model for reinforcing steel

4.2 ELEMENT LEVEL VERIFICATION - ISOLATED COLUMN

4.2.1 BASIC ASSUMPTION

The basic assumption used here in the verification process for both Seismostruct and SAP2000 are:

- For the Seismostruct model strain hardening of 0.01 is used.
- For the Seismostruct a 10% of the total element length is used for a plastic hinge zone. Paulay and Priestley (1992), $L_p = 0.08z + 0.022d_b\ fy$ (for RC beams and columns)
- For Seismostruct two discretization is applied at section level with fiber definition and at member level by segmenting
- For SAP2000 no strain hardening is used
- For SAP2000 plastic hinge is concentrated at a point at a point and so lamped inelasticity is used

4.2.2 COLUMN VERIFICATION DATA

For an isolated column cyclic experiment was initially performed by Osada et al (1997). This column is a rectangular column. Now it is taken from the previous numerical study by Maekawa et al. It is reinforcements are uniformly distributed along the load direction and on the other direction. It is loaded on the shorter direction of the cross section. This column has less transversal steel compared to its longitudinal steel. The column is adequately reinforced so all regions of the confined concrete is assumed to be reinforced concrete. The overall dimension and structural configuration can be seen in *Figure 4.4*

<u>Specimen Specification and Property:</u>

P_{axial} = 78.5 KN (constant)

<u>Concrete</u>

fc' = 26.5 MPa

<u>Reinforcement Bar</u>

Longitudinal Steel		Transversal Steel
Along the left and right	Along the top and bottom	Along the height

6 φ 10mm	22 φ 13mm	φ 6mm @250 mm C/C.
$F_{y,long}$ = 331.6 MPa	$F_{y,long}$ = 331.6 MPa	$F_{y,hoop}$= 353.2 MPa

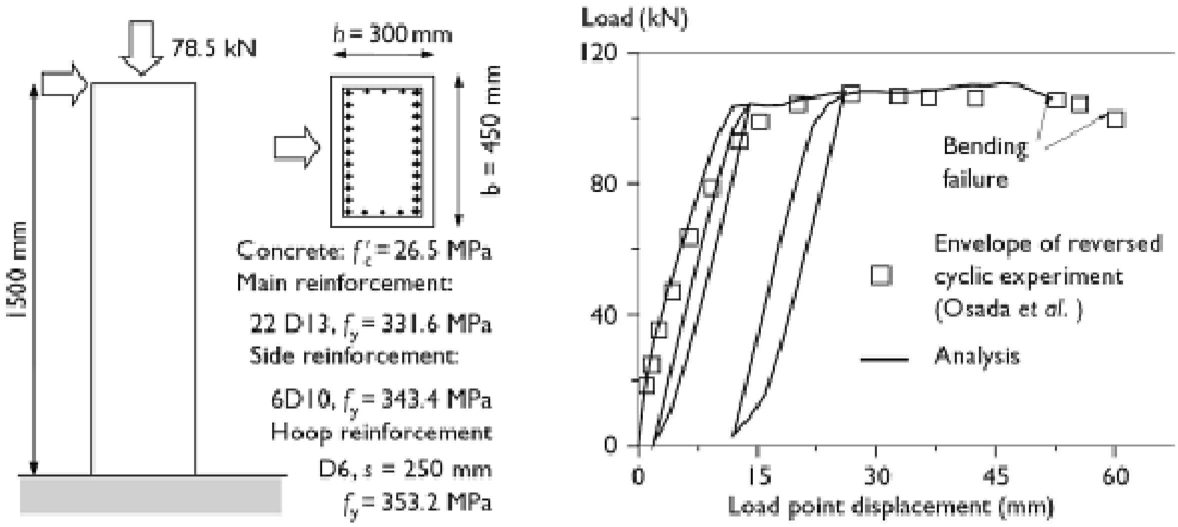
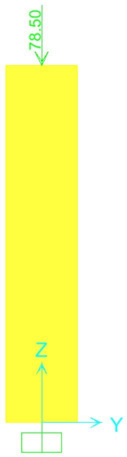


Figure 4.4 isolated column cross-section and experimental results

4.2.3 COLUMN VERIFICATION MODEL

The following two figures shows the model used in SAP-2000 and Seismostruct software.



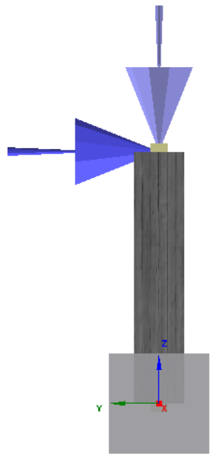


Figure 4.5 Verification Column Model a) SAP2000 b) Seismostruct models

4.2.4 RESULTS OF SAP2000, SEISMOSTRUCT AND EXPERIMENTAL DATA

As shown on *Figure 4.6* there is major variation between capacity curves from SAP2000 with the experiment result while the result from Seismostruct have acceptably similar pattern. When zero strain hardening ratio assumption in used the Seismostruct capacity curve is below the experiment around the yielding zone. But the material models we use for performance assessment should capture the real behavior of materials so the use of strain hardening will results in a better resemblance with the experiment.

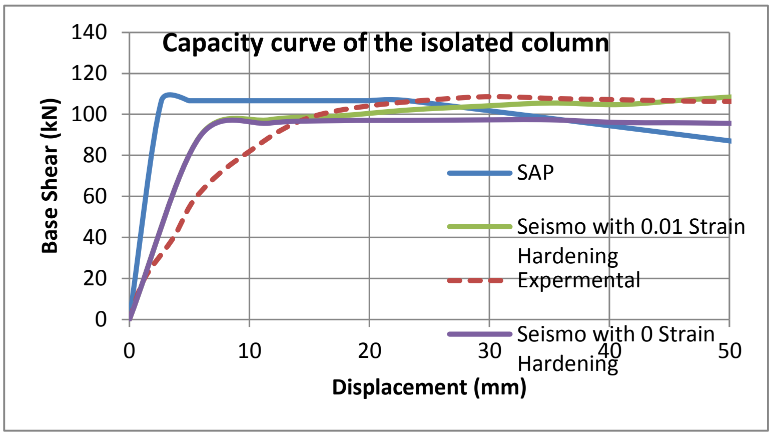


Figure 4.6 Response curve of SAP2000, Seismostruct and Experiment

4.2.5 STIFFNESS REDUCTION

The amount of reduced stiffness of the column is an important parameter that can be determined from fiber based discretization (i.e from Seismostruct). The gross moment of inertia and the cracked moment of inertias are calculated based on the un-cracked and cracked transformed sections of the column. The ratio between the cracked and un-cracked moment of inertia is determined which is the stiffness reduction value. In the figures below sample fiber element is taken from both tension and compression side is selected and their respective stress-strain curve is provided.

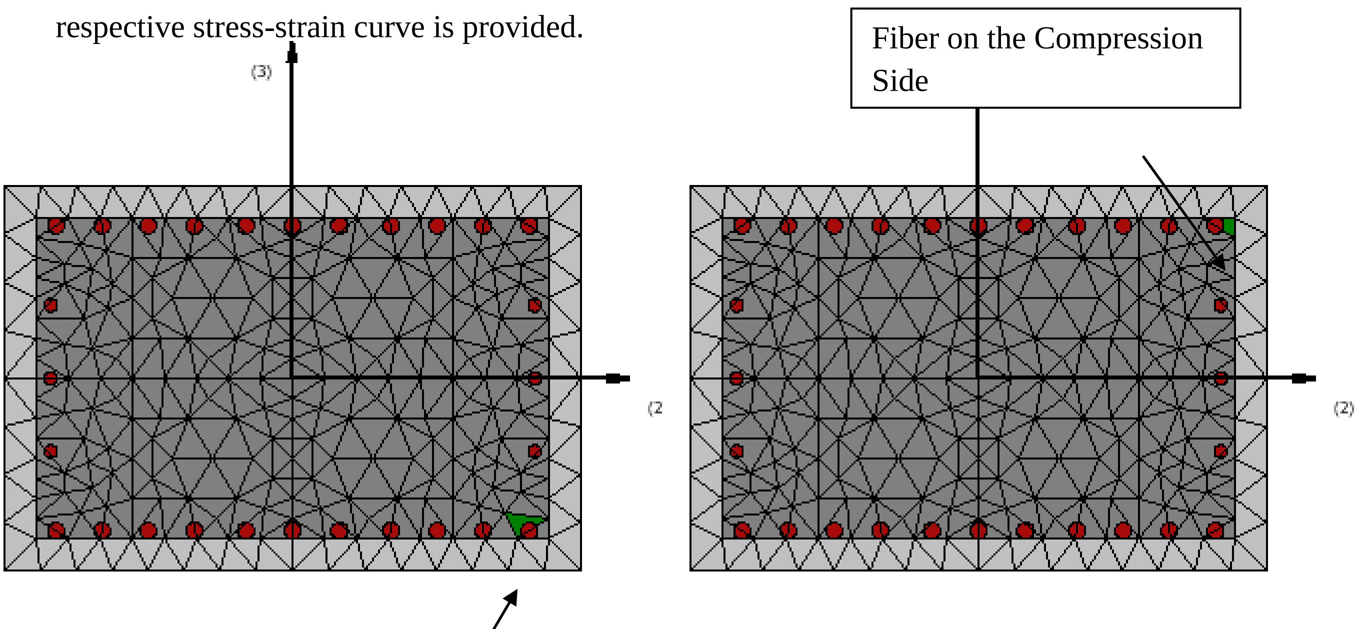


Figure 4.7 Fiber based modeling a) Fiber on the tension side b) Fiber on the compression side

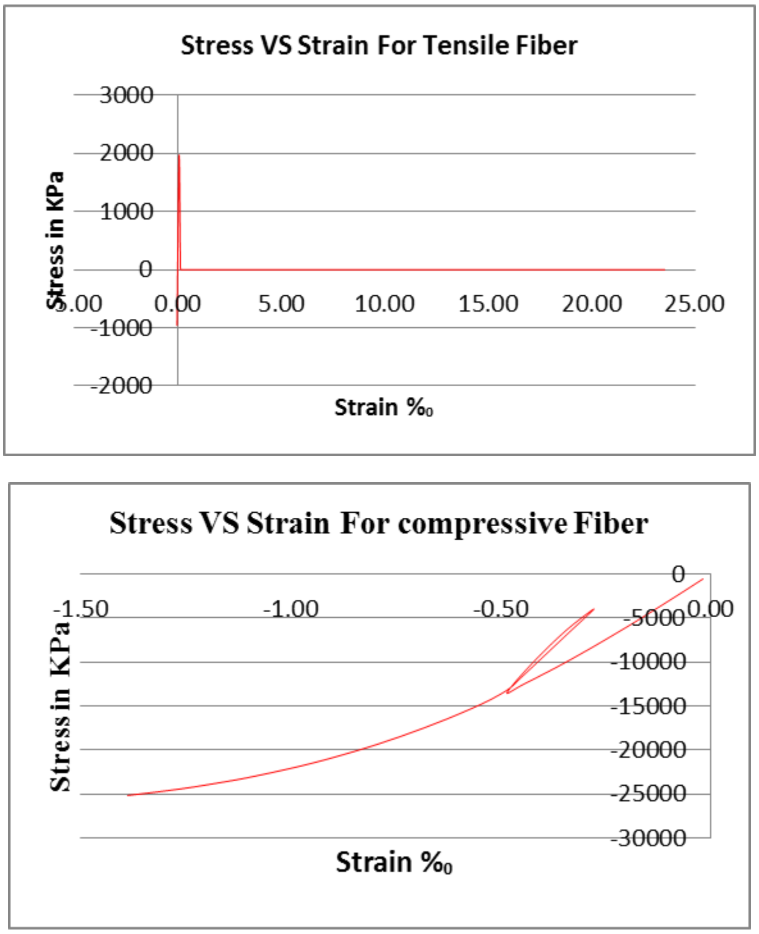


Figure 4.8 Stress-strain profile for a) Fiber on tension side b) Fiber on compression side

The calculation of moment of inertia for both un-cracked and cracked section is provided on the next table.

Table 4.1 Un-cracked section Moment of Inertia of Column

Un-cracked section Moment of Inertia					
Concrete		**Top and Bottom bar**		**Left and Right bar**	
B	450	**n**	6.666666667	**n**	6.666667
H	300	**As**	1460.055186	**As**	157.0796
I	1012500000	**dx**	119	**dx**	59.5
		I	275677886.5	**I**	7414682

Total $I_{uncracked}$ **1,295,592,568.73** mm^4

Table 4.2 Cracked section Moment of Inertia of Column

cracked section Moment of Inertia									
Concrete		**Bottom bar**		**Left and Right bar**		**Top bar**		**Left and Right bar**	
B	450	**n**	6.666666667	**N**	6.666667	**n**	6.66667	**N**	6.66667
H	48.4375	**As**	1460.55186	**As**	157.0796	**As**	1460.05	**As**	157.096
I	17046546.9	**dx**	220.5625	**dx**	161.0625	**dx**	17.4375	**Dx**	-42.0625
		I	473523310.8	**I**	27165487	**I**	2959692	**I**	1852758

Total I_{cr} **522,547,794.31mm⁴**

The stiffness reduction value is 0.403, $\frac{I_{un}}{I_{cr}} = 0.403$

This value is very close to the ACI recommendation of beam stiffness factor which is 0.35. Since this column is predominantly subjected to lateral load and also subjected to a very small axial load (78.5KN), the column is behaving as a cantilever beam. Therefore for this column the calculated stiffness reduction value is reasonably close to ACI recommendation for beam.

4.3 FRAME LEVEL VERIFICATION

4.3.1 VERIFICATION OF SAP2000

The following frame is a two bay, three story frame taken from *Ahmet Yakut (2001)* from his numerical study. It is going to be analyzed using hand calculation of nonlinear static pushover analysis in which an incremental load is applied to the structure at each story and the corresponding internal moments are recorded and the load is applied until hinge is formed and final a progressive failure is achieved

4.3.1.1 BASIC ASSUMPTIONS

- Column axial load is estimated from the equivalent static method of analysis and it is assumed not to change.
- The moment curvature is assumed to be linear until yielding of reinforcement and constant after yielding of reinforcement until rupture is reached with rigid plastic hinge (no hardening of steel or softening of concrete)

- Plastic hinge is expected to form when the moment at a joint reach 5% closes to the yield moment resistance

Analysis Procedure

- Create the model using SAP2000 software like any other modeling.
- Define member properties for both beams and columns and determine the section capacity and moment curvature relation.
- Apply gravity load on the frame elements, no lateral load is applied here.
- Define the load to be applied, it can be a rectangular, inverted triangle or the first mode shape can be used. Here the inverted triangle lateral load distribution is used.
- apply the lateral load incrementally. Find the lateral load that will induce the first hinge point and once the first hinge is formed elastic analysis is no applicable; record the load effects due to this lateral load at each element.
- Apply an actual hinge at the location of first hinge point by assigning a frame release option then remove the vertical load and apply an incremental lateral load on the frame so that the new load effect on each element and the load from the previous action will result in a new hinge at a new location. Continue this procedure until collapse or mechanism.
- The base shear is the sum of the incremental load at each step. Plot the base shear verses the top displacement this will results in the capacity curve.

4.3.1.2 FRAME VERIFICATION DATA

Figure 4.9 shows the element allocation with its joint assignment of the frame elements which is used during section capacity estimation and moment curvature relation.

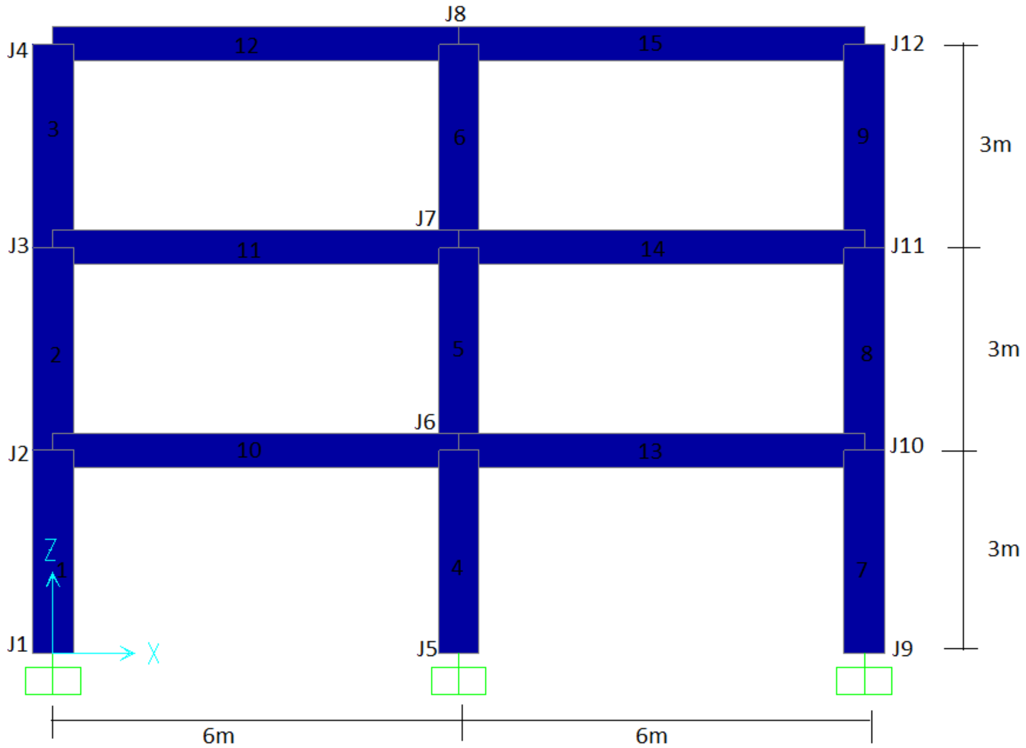


Figure 4.9 Frame element and joint allocation

4.3.1.3 VERIFICATION FRAME LOADING AND MODELING

4.3.1.3.1 Loading on SAP2000

The frame is loaded with gravity load and lateral load. A super imposed dead load of magnitude 10KN/m at third story and 15KN/m at first and second story and service live load of 2KN/m on every story. The lateral load is not known at first so it is increased at every step until mechanism is formed. For the pushover analysis the following loading is used DL + 0.3LL + EQ and it is named “PUSH”.

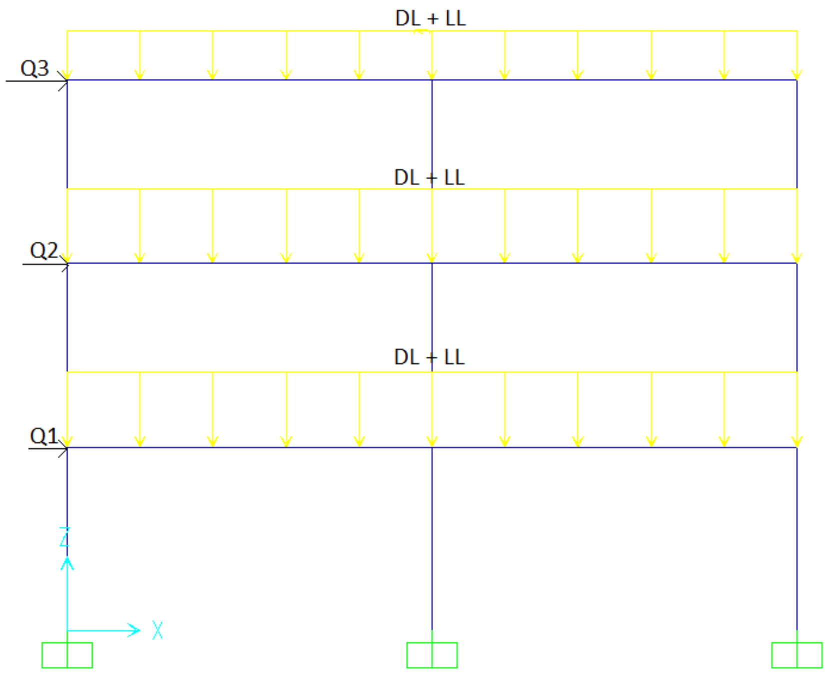


Figure 4.10 Frame Loading

Material Property

C-55 concrete and S-570 steel is used and concrete cover of 50mm is used.

Figure 4.11- Column and Beam Cross section

4.3.1.3.2 Modeling using SAP2000

The moment curvature relation used for the section capacity determination for both SAP2000 and for the incremental approach (the handmade pushover analysis with an incremental load for the verification frame) is shown in Figure 4.12. From the graph it is seen that there is a variation between them this variation results in the reduction of capacity in case of incremental approach.

Table 4.3 summarizes the moment capacity of each element and their corresponding moment curvature relation.

Table 4.3 Moment Curvature Relation for beams and columns on the sample frame

Member	Element Type	Yield Moment, My (KNm)	Yield Curvature Φy (rad/m)	Ultimate Curvature, Φy (rad/m)	Remark
1	Column	124	0.0055	0.111	The yield moment is varied for the same cross section due to the presences of the axial load
2	Column	115.5	0.0056	0.115	
3	Column	107.5	0.0056	0.119	
4	Column	166	0.0059	0.085	
5	Column	143	0.006	0.099	
6	Column	119	0.006	0.113	
7	Column	133.5	0.0056	0.105	
8	Column	122	0.0057	0.112	
9	Column	110	0.0054	0.118	
10	Beam	49	0.0073	0.103	
11	Beam	50	0.0069	0.102	
12	Beam	53	0.0069	0.099	
13	Beam	49	0.0073	0.103	
14	Beam	50	0.0069	0.102	
15	Beam	53	0.0069	0.099	

Figure 4.12 Shows the moment curvature relation variation used for SAP2000 and for the incremental approach, SAP2000 definition allows rigid plastic hinge definition which results in M-K variation.

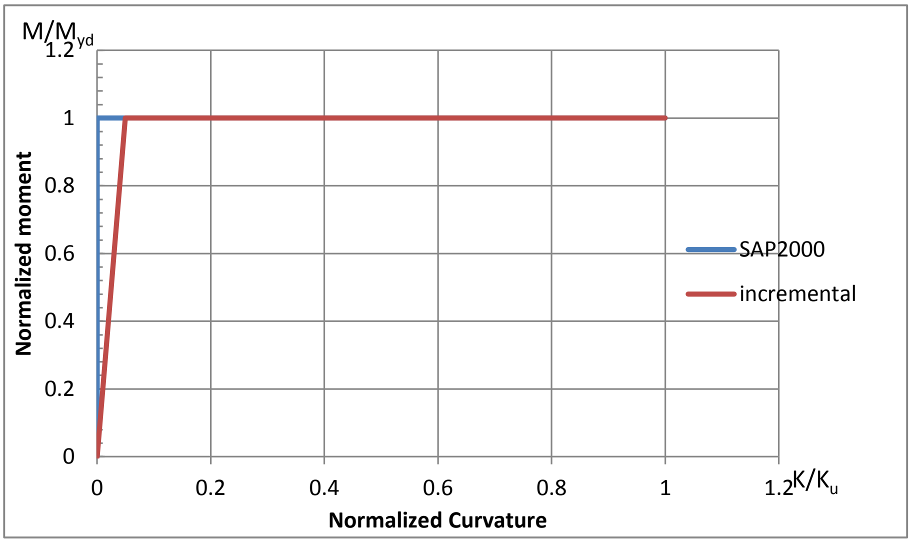


Figure 4.12 Normalized Moment Curvature relation for the first column

4.3.1.4 RESULTS OF SAP2000 AND INCREMENTAL ANALYSIS

From the verification analysis shown *Figure 4.13*, it is seen that in both cases the sequence of hinge formation is similar while the result of the hand calculation is smaller than that of the SAP2000. In case of the incremental approach the analysis is terminated when mechanism is formed and so cannot continue to the full displacement. In case of SAP2000 analysis is terminated when mechanism is formed or when target displacement is reached.

Form the result it is seen that the capacity obtained from SAP2000 is higher than the actual capacity, this is mainly due to the moment curvature used in SAP2000 is rigid plastic as shown in *Figure 4.12*

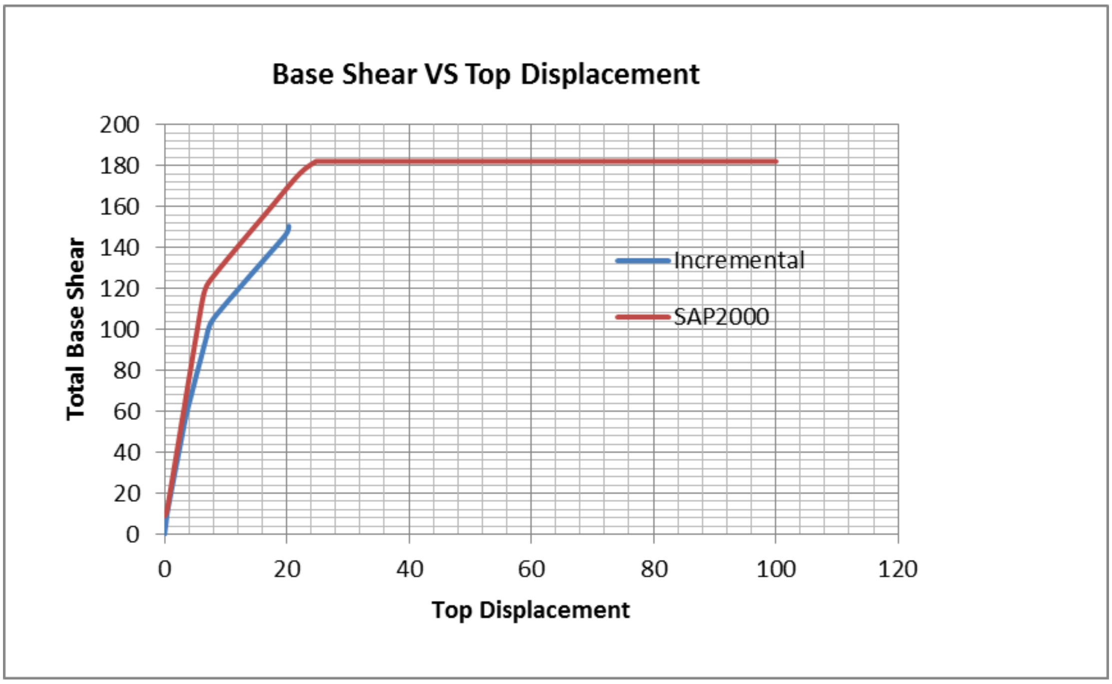


Figure 4.13 Capacity Curve For Incremental and SAP2000

The analytical result of the analysis and the progressive hinge formation of the verification frame are found on the annex attached at the end of this paper.

4.3.2 VERIFICATION OF SEISMOSTRUCT

4.3.2.1 BASIC ASSUMPTIONS

The basic assumption used here in the verification process is:

- The moment curvature is based on the software output
- For the Seismostruct model strain hardening of 0.01 is used.
- Plastic hinge length is assumed to be 10% of the total length (different researchers suggests different values, to be conservative lower value is taken)

4.3.2.2 VERIFICATION DATA

This frame system example is taken from the previous numerical study of Vecchio and Emara (1992). The frame was flexure-critical with well-confined cross sections; the beams' span-depth ratios were larger at 8.75; and, lastly, higher column axial loads were applied on this experiment.

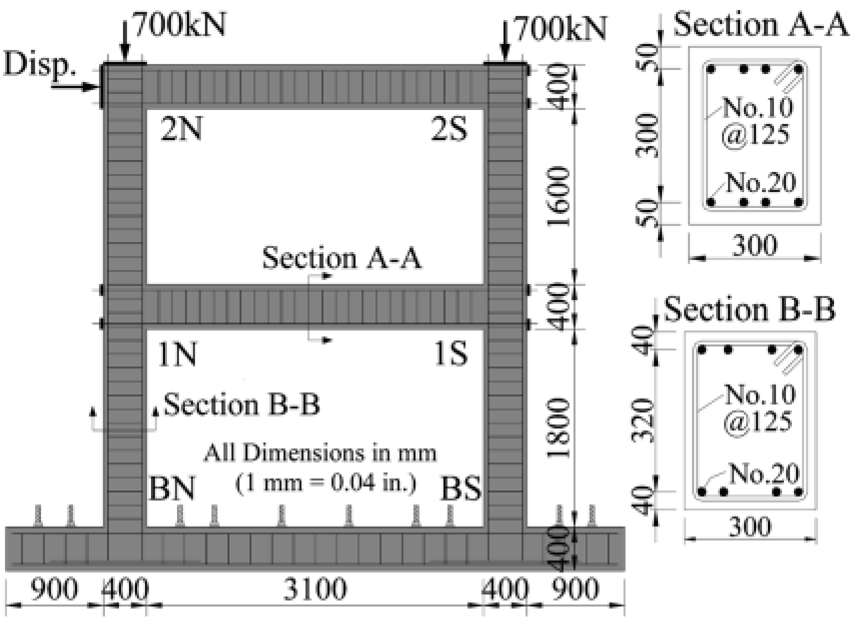


Figure 4.14 Frame used For Verification

Table 4.4 Material Property for the Verification Frame

Reinforcement Bars									Concrete
Bar No.	A_s (mm^2)	D_b (mm)	f_y (MPa)	f_u (MPa)	E_s (GPa)	E_{sh} (MPa)	ε_{sh}	ε_u	f'_c (MPa)
No. 20	300	19.5	418	596	192.5	3100	0.0095	0.0669	30
No. 10	100	11.3	454	640	200	3100	0.0095	0.0695	

4.3.2.3 VERIFICATION FRAME MODEL

Figure 4.15 shows the modeling and loading of the frame in Seismostruct software.

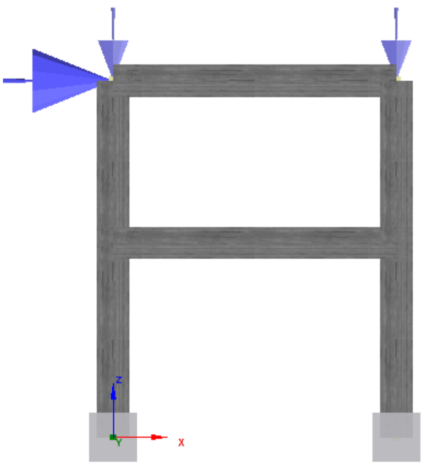


Figure 4.15 Frame Modeling

4.3.2.4 RESULTS OF SEISMOSTRUCT SAP2000 AND EXPEIMENTAL DATA

As seen from *Figure 4.16* the result from Seismostruct have acceptably similar pattern especially at the beginning. Therefore now the required frame (frame taken from the case study building) can be modeled in this software and the output will be used to assess the building performance under extreme loading especial Earthquake loading.

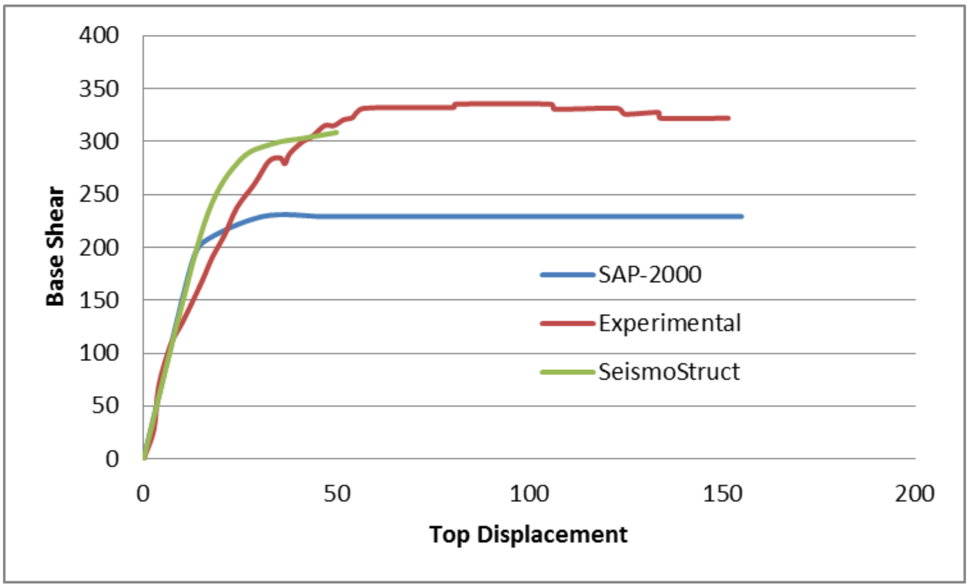


Figure 4.16 Capacity Curve from Experiment, SAP2000 and Seismostruct

4.3.2.5 STIFFNESS REDUCTION

The quantification of damages on the frame (i.e. damage of all beams and column) is summarized on the next table. *Figure 4.17* shown below indicates the name of the elements.

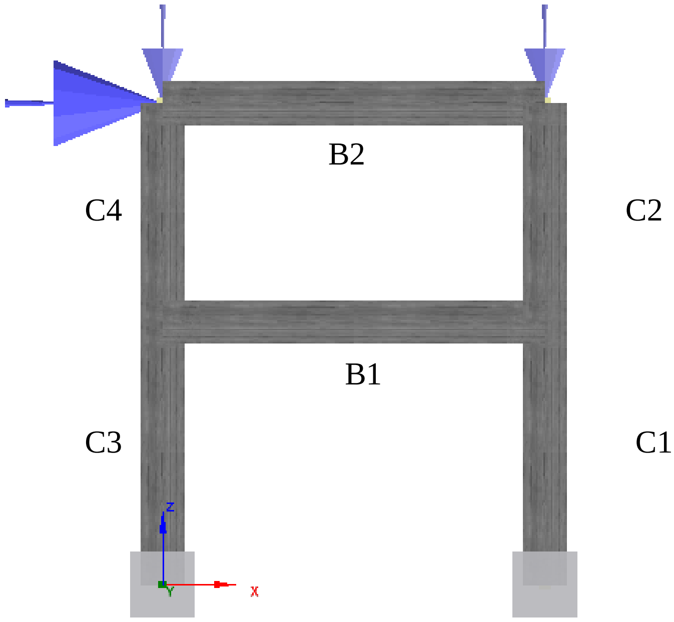


Figure 4.17 Frame Labeling

Table 4.5 Stiffness Reductions of Beams and Columns at Failure

Member ID	$\mathbf{I_{uncracked}}$	$\mathbf{I_{cracked}}$	**Ratio**
C1	1960000180	1574355087	0.803
C2	1960000180	2079905923	≈1
C3	1960000180	620583463.2	0.317
C4	1960000180	1357437682	0.693
B1	2009600205	1392770606	0.693
B2	2009600205	1392770606	0.693

The moment of inertia is calculated at the neutral axis depth which is different for each element of the frame. The stiffness reduction of different element of the frame is shown and from this table it can be seen clearly that the stiffness of the elements in the tension side has very low value compared to the stiffness of elements on the compression side. The stiffness reduction value of the beam is closer to the EURO code than ACI code recommendation. Whereas the stiffness reduction value obtained from this analysis for columns are closer to the ACI Code than EURO code recommendations. Therefore from this analysis it can be concluded the ACI code is conservative for beams and except for C3 EURO code is conservative for columns.

5 CHAPTER FIVE – PERFORMANCE ASSESMENT OF THE CASE STUDY BUILDING

5.1 CASE STUDY BUILDING DESCRIPTIONS

5.1.1 LOCATION

The case study building is selected so that it will have similar property with low rise pre cast condominium building. The building is located in Addis Ababa which is zone two with PGA of 0.05g previously and 0.1g on the revised building code of Ethiopia. The building is also located on a site of soil class B.

Other properties of the case study building like the geometry, support system frame allocation direction of rib and any other description were presented on chapter three.

5.1.2 STRUCTURAL MODELING

The frame is three bays and six story frames with column dimension of 45x45cm, 40x40cm, 35x35cm and 30x30cm, and the beams are 25x40cm and 25x43cm. For SAP2000 The section capacity and moment curvature relation are computed using response2000 software, as done before in the code based assessment stage and verification stage. For the columns the expected axial loads are computed using the equivalent static method.

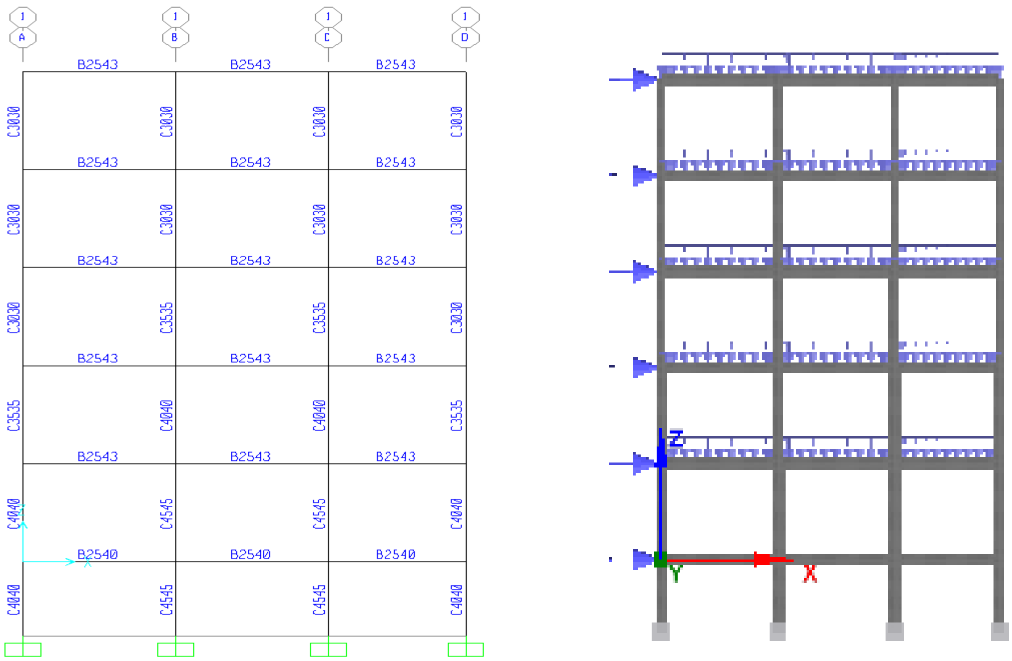


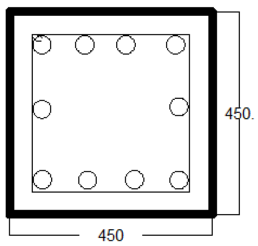
Figure 5.1 Case Study Frame a) SAP2000 model b) Seismostruct model

The longitudinal and the transvers reinforcement detail of columns and beams are shown in *Table* 5.1

Table 5.1 Column Section and Reinforcement

Story	**Reinf. and Group ID**	**Column ID**			
		C1	**C2**	**C3**	**C4**
1 (45cmX45cm C2 &C3) **1 (40cmX40cm C1 &C4)**	Long. Bars Ties	8Φ20 Φ8@20	10Φ20 Φ8@20	10Φ20 Φ8@20	8Φ20 Φ8@20
2 (45cmX45cm C2 &C3) **2 (40cmX40cm C1 &C4)**	Long. Bars Ties	8Φ20 Φ8@20	10Φ20 Φ8@20	10Φ20 Φ8@20	8Φ20 Φ8@20
3 (40cmX40cm C2 &C3) **3 (35cmX35cm C1 &C4)**	Long. Bars Ties	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20
4 (35cmX35cm C2 &C3) **4 (30cmX30cm C1 &C4)**	Long. Bars Ties	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20
5 (30cmX30cm C2 &C3) **5 (30cmX30cm C1 &C4)**	Long. Bars Ties	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20	8Φ20 Φ8@20
6 (30cmX30cm C2 &C3) **6 (30cmX30cm C1 &C4)**	Long. Bars Ties	8Φ14 Φ8@20	8Φ14 Φ8@20	8Φ14 Φ8@20	8Φ14 Φ8@20
7 (45cmX45cm C2 &C3) **7 (30cmX30cm C1 &C4)**	Long. Bars Ties	8Φ14 Φ8@20	8Φ14 Φ8@20	8Φ14 Φ8@20	8Φ14 Φ8@20

For every story C1 and C4 are exterior columns and C2 and C3 are the interior columns.



Longitudinal bar 8Φ20

Transverse bas Φ8@200mm

Figure 5.2 typical column cross section

Plastic Hinge Defenation

The plstic hinge zone to be used is affected by different parameters like the compersive strain. Different researchers gives different values, If we use the Paulay and Priestley (1992), L_p= 0.08z + 0.022d_b fy (for RC beams and columns) we will have different values depending on the member length, bar size and contra flexure points. Even though the quantification of the length needs further study For Seismostruct 10% of the member length is used while for SAP2000 a hinge point with 0 strain hardening is used so there is no plasitc zone.

Figure 5.3 shows the hinge properites defination of one of the columns on SAP2000 uisng the momnet curvature relation obetained from response 2000.

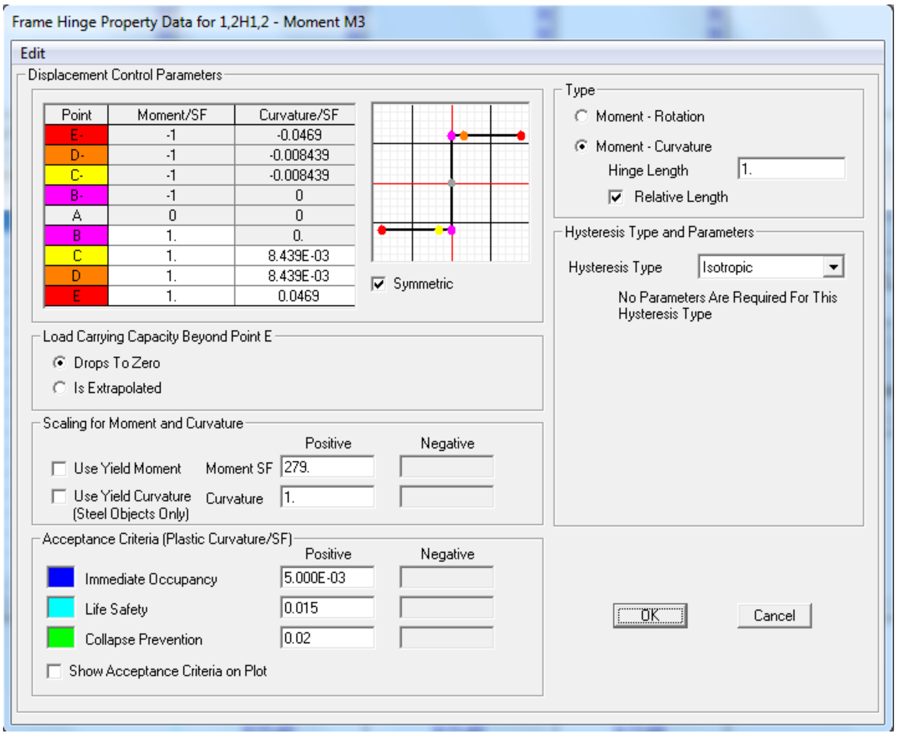


Figure 5.3 Hinge Definition on SAP2000

5.2 RESULT AND DISCUSSION OF SEISMOSTRUCT

5.2.1 PUSHOVER CURVE

Pushover curves also called the capacity curve is the plot of base shear versus displacement usually roof displacement. It represents the global response of the structures. The capacity curves for the study frames were done by using pushover analyses. The load distribution used was the triangular one as we have regular structure. Figure 5.4 is a capacity curve of the frame in the negative x-direction for triangular lateral load distribution.

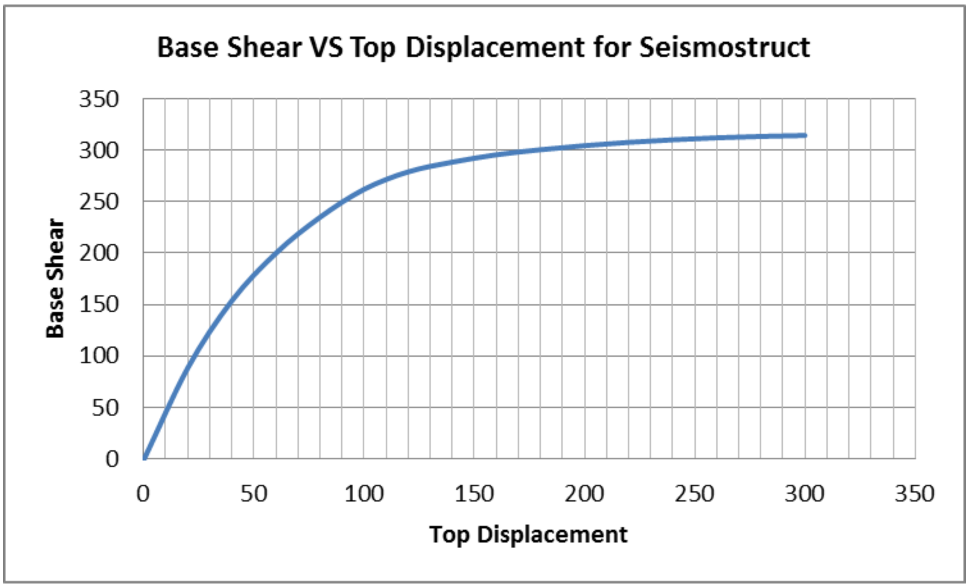


Figure 5.4 capacity curve

Table 5.2 gives roof displacement and 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. From the table it is seen that the maximum drift at first rupture of still is below the limit for immediate occupancy. Even though the maximum drift does not exceed immediate occupancy limit the base shear at this stage is below the base shear that the building will experience for this specific soil class (soil class B).

Table 5.2 Roof Displacement and Total Drift at Rupture of Steel

Base Shear at Rupture of Steel (kN)	**Roof Displacement (m)**	**Maximum Drift at Steel Rupture**
280	0.120	0.00818

Figure 5.5 shows the deformed shape of the case study frame at first rupture of steel and four joints has reached at rupture strain,

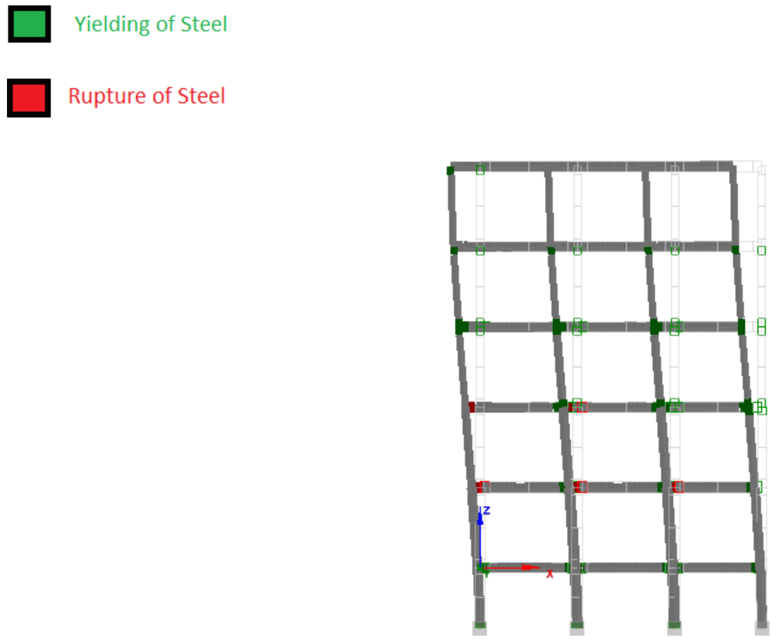


Figure 5.5 Deformed Shape at First Rupture of Steel

5.2.2 SAFETY FACTOR FOR BASE SHEAR

Here we have base shear form two analyses one is ESA and the other is pushover analysis. The base shear obtained from ESA is unique for a specific soil class while the base shear from pushover analysis is not related the soil condition. So we will compare the two base shears and we will have the building's factor of safety

The case study building satisfy force demand requirement at life safety performance level for soil class A and fails to satisfy force demand for soil class B and C

Table 5.3: Factor of Safety at First Steel Rupture: Seismostruct

	PGA [0.1g]		
	Soil A	**Soil B**	**Soil C**
Capacity (kN)	280	280	280
Demand (kN)	253.8	304.6	380.7
Safety Factor	1.103	0.920	0.750

5.2.3 STORY DISPLACEMENT

The displacement of the frame can be computed by using the base shear from the equivalent static analysis. The base shear for soil class A and B are below the base shear capacity of the frame so their displacement can be obtained from the capacity curve.

Figure 5.6 shows displacement profile at rupture of reinforcement while Figure 5.7 and Figure 5.8. Shows the displacement for soil class A and soil class B respectively

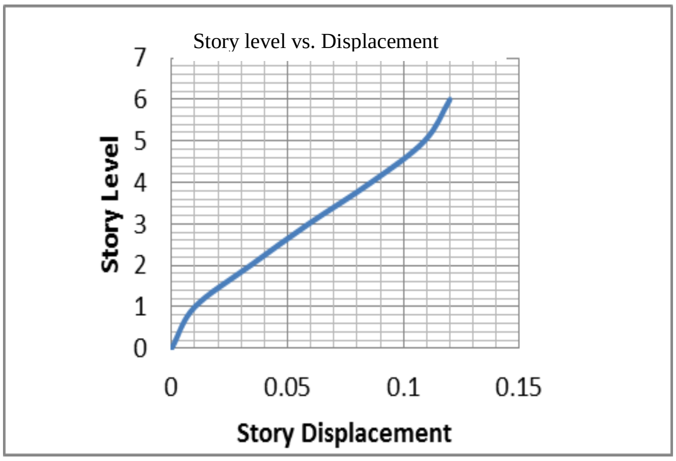


Figure 5.6 Story Displacement at Steel Rupture

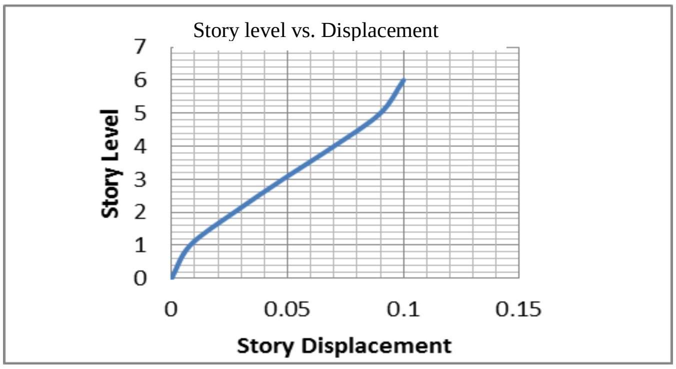


Figure 5.7 Story Displacement for Soil Class A

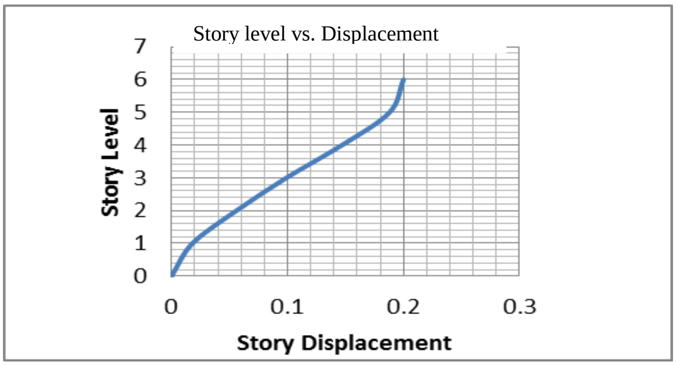


Figure 5.8 Story Displacement for Soil Class B

5.2.4 STORY DRIFT

The expected performance level of a given structure is evaluated by comparing the drift of the structure with the limit drift. So it is important to accurately quantify the story drift in order to have a clear image of the damage on the structure and the attained performance level.

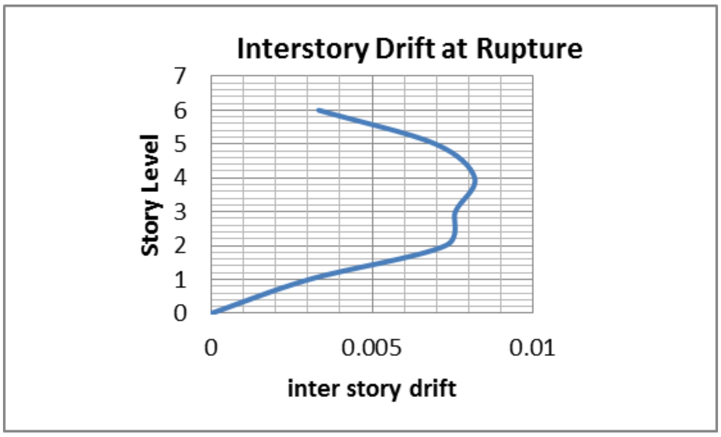


Figure 5.9 Inter Story Drift at Steel Rupture

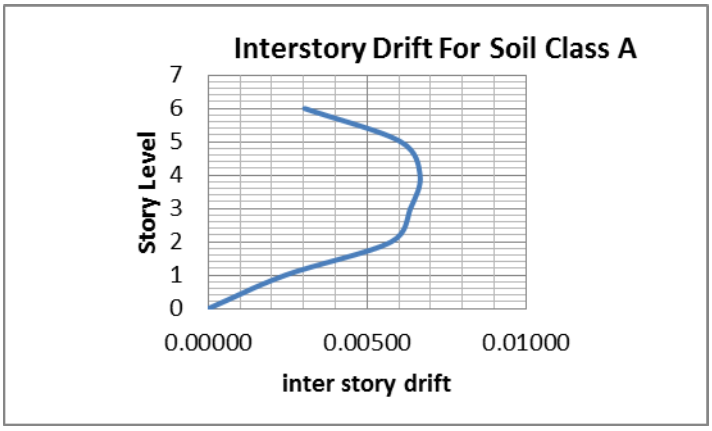


Figure 5.10 Inter Story Drift for Soil Class A

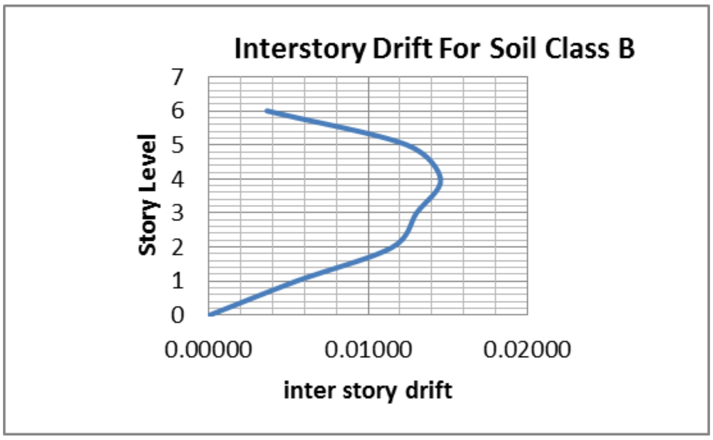


Figure 5.11 Inter Story Drift for Soil Class B

Table 5.4 Summarize the performance level of the case study building for different soil class

Table 5.4 Story Drift requirement

	Story Level	Story Drift	IO	DC	LS
Soil Class A	6	0.00303	OK!	OK!	OK!
	5	0.00606	OK!	OK!	OK!
	4	0.00667	OK!	OK!	OK!
	3	0.00636	OK!	OK!	OK!
	2	0.00576	OK!	OK!	OK!
	1	0.00242	OK!	OK!	OK!
Soil Class B	6	0.00364	OK!	OK!	OK!
	5	0.01242	NOT OK!	OK!	OK!
	4	0.01455	NOT OK!	OK!	OK!
	3	0.01303	NOT OK!	OK!	OK!
	2	0.01152	NOT OK!	OK!	OK!
	1	0.00545	OK!	OK!	OK!

5.3 RESULT AND DISCUSSION OF SAP2000

5.3.1 PUSHOVER CURVE

Figure 5.12 shows the plot of the base shear against the top story displacement

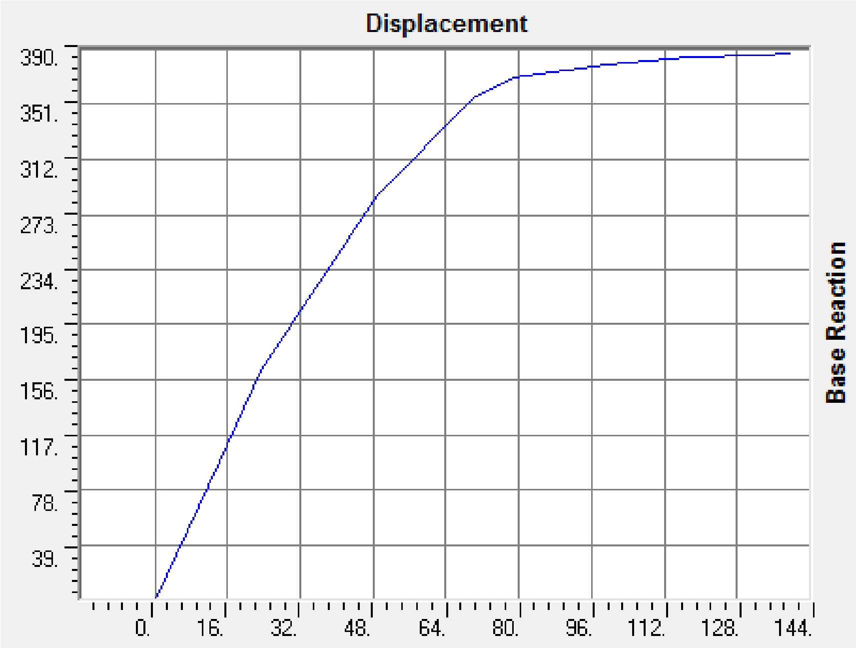


Figure 5.12 Pushover Curve from SAP2000

The performance curve plot from SAP2000 is higher than the same performance plotted by using the base shear at each incremental stage. Below shows the variation in performance curve

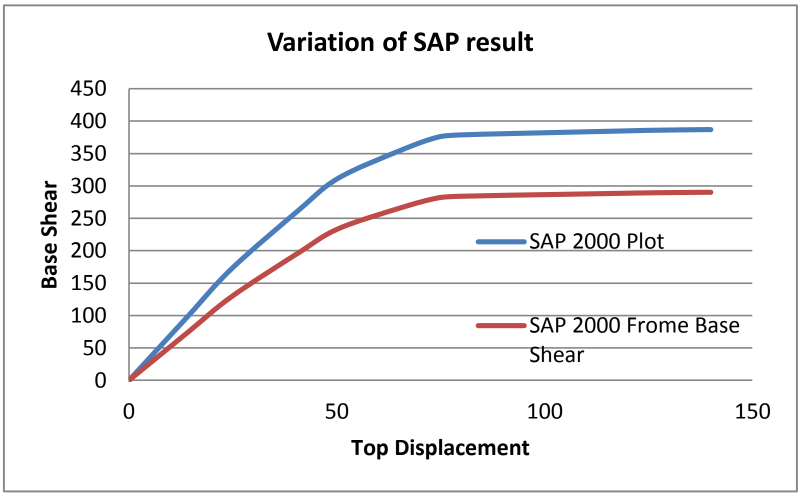


Figure 5.13 performance curve Variation of SAP2000

Table 5.5 Base Shear from ESA and Pushover Analysis

Maximum Pushover Base Shear Before Rupture (kN)	Base Shear for Soil Class B ESA (kN)	Roof Displacement Before Rupture (mm)
290	304.58	140

Percentage variation of the ESA base shear and base shear due to pushover analysis is 4.8%, even though the use of pushover analysis provides better capacity estimation than the one which is done by the ESA still there is a deficiency in the capacity of the structure.

The use of pushover analysis will allow the structure to carry larger amount of base shear as the structure undergoes large amount of moment redistribution which intern allows utilization of members that are less stressed at the formation of the first plastic hinge.

5.3.2 LIMITING BASE SHEAR FOR ESA

In order to quantify the amount of additional capacity utilized by the application of pushover analysis we have to know the base shear before member failure. This is done by applying a base shear at the frame and then we compare the reinforcement requirement with the actual amount of reinforcement, we will continue doing this until the two reinforcements are the same.

Table 5.6 Limiting Base Shear for ESA for the case study frame

Story Level	Actual Story Shear	Distribution Coefficient	New Story Shear
6	66.28	0.218	54.383
5	74.21	0.244	60.890
4	76.89	0.252	63.089
2	44.1	0.145	36.184
3	28	0.092	22.974
1	15.21	0.050	12.480
0	0	0.000	0.000
Total	Fb = 304.69KN	K = 1	Fb = 250KN

There for the limiting base shear for ESA is 250KN and this load will induce an axial load of 1150KN, bending moment of 29KNm and 215KNm which can be resisted by the foundation column any load higher than this value will results in failure of the column.

From the pushover analysis we have a base shear capacity of 290KN which is 16% higher than the capacity from the ESA this shows that it is advantageous to use pushover analysis in order to assess the performance of an existing structures.

Figure 5.14 Show the distribution of the the lateral load on the frame during the applicaiotn of the limiting base shear which is 250KN

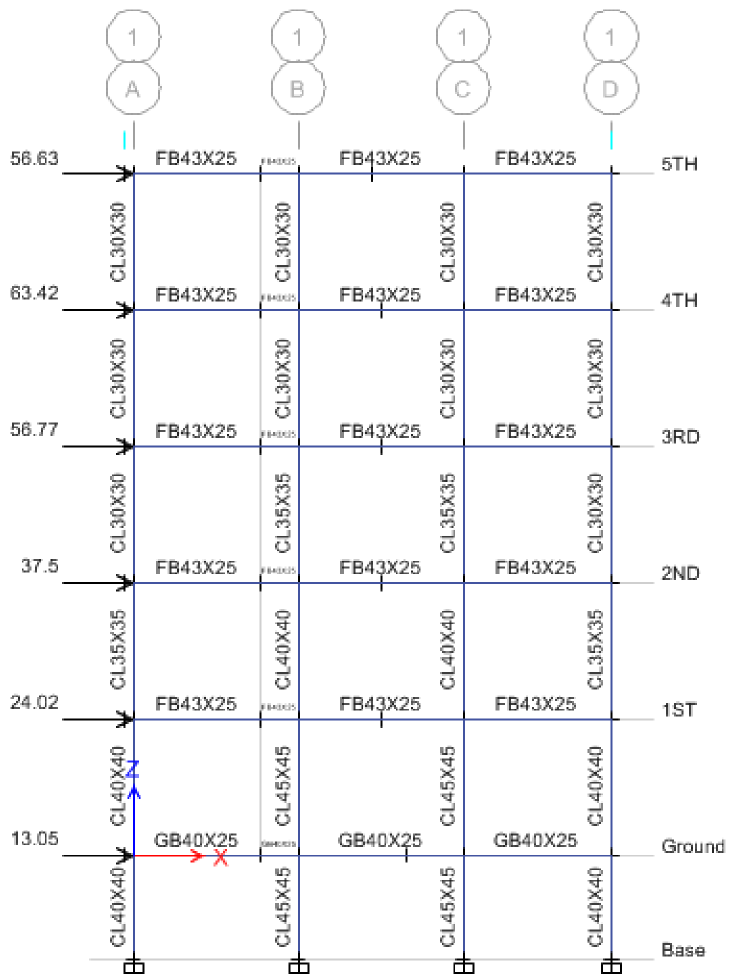


Figure 5.14 Limiting Lateral Load on the Frame

5.3.3 EFFECT OF GROUND CONDITION

Table 5.7 Show the base shear from the ESA approach for the different soil class and the base shear from the capacity curve.

In the construction of the capacity curve the structure is given a displacement and the internal forces are estimated irrespective of the ground condition and hence the soil date, once the internal force (the base shear) is known then the demand is imposed depending on the soil condition.

Table 5.7 Base Shear for Different Soil Class

	Soil Class	**Base Shear from ESA, (KN)**	**Base Shear from Pushover, (KN)**	**Base Shear Comparison**
PGA [0.1g]	A	253.82092	290	Ok!
	B	304.65069	290	Not Ok!
	C	380.73137	290	Not Ok!

5.3.4 DRIFT REQUIREMENT

Table 5.8 Show the limiting requirement for the expected performance level

Table 5.8 Drift comparison for soil class B

Story Level	**Displacement (mm)**	**Story Drift**	**IO (0.01)**	**DC (0.01-0.02)**	**LS (0.02)**
6	140	0.00136364	OK!	OK!	OK!
5	135.5	0.00283939	OK!	OK!	OK!
4	126.13	0.00538182	OK!	OK!	OK!
3	108.37	0.01087273	NOT OK!	OK!	OK!
2	72.49	0.01356061	NOT OK!	OK!	OK!
1	27.74	0.00840606	OK!	OK!	OK!
0	0				

Table 5.8 shows the story drift of the frame during the seismic excitation, and from the results it can be concluded that all the stories satisfy the life safety and collapse prevention

requirements but they fails to satisfy the operational and immediate occupancy requirement. Therefore those condominium buildings that are located on soil class B are expected to be safe for life safety and collapse prevention but they fail to satisfy operational and immediate occupancy. Most of all the structure fails to carry the expected base shear force for soil class B.

5.3.5 HINGES FORMATION BEFORE COLLAPSE

Figure 5.15 shows the number and position of hinges formed during the application of the displacement force, the detailed progressive hinge formation is shown on the attached appendix.

The major advantage of pushover analysis lays on the progressive hinge formation which will show the type and mode of failure, it even shows the extent of additional loads that can be carried by the frame through moment redistribution.

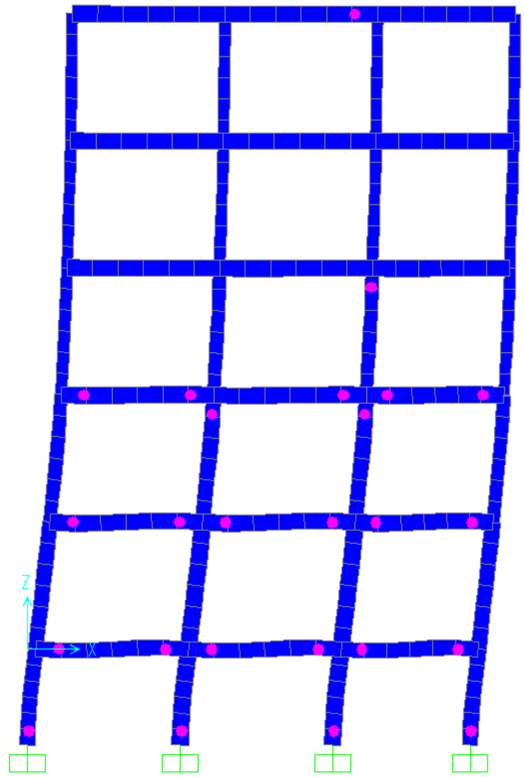


Figure 5.15 Hinge Formation before Failure

It is seen on the case study frame that most of the hinges are formed relatively at the same time which is a very good indication of the reserved capacity, the frame will have limited amount of reserved capacity as most of the elements reach their capacity at the same time this justify the small amount of increased performance (16%) from the code based capacity.

6 CHAPTER SIX - CONCLUSION AND RECOMMENDATION

- The increased value of the peak ground acceleration on the revised building code has a negative effect on the structural performance of existing condominium buildings, therefore they should be investigated thoroughly and appropriate mitigation must be done in order to avoid catastrophic failure during occurrence of the expected seismic excitation.
- Static nonlinear pushover analysis is found to be reliable in predicting the performance of constructed precast condominium housings
- Pushover analysis is advantageous not only in predicting the real time response of the structures but also in indicating progressive hinge formation. The progressive hinge formation can be utilized as a valuable input for strengthening purpose.
- Representation of failure using limiting force fails to address actual behavior of a given structure so it is highly advantageous to prescribe failure using displacement limitations.
- As the study indicates, performance assessment of an existing structure should not relay on code provisions as cods are very conservative which will undermined the actual capacity and fails to give the real time response of structures.
- The performance of a given structure depends on plastic hinge definition, the use of plastic hinge zone results in a better performance of the structure as compared with hinge points.
- Further study should be done by considering shear hinge in addition to flexural hinge in order to address all possible type of failure.
- Target displacement and global ductility should be estimated in a more comprehensive way for the result to be conclusive.
- The government should perform more assessment and impose a performance requirement based its current priority which doesn’t compromise the life safety of the occupants on the existing condominium buildings.

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24. XD- COSec Version 1.0.0 “Reinforced Concert Section Analysis According to Euro Code”

APPENDIX-A 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 A-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 A-2 and Table A-3)

Table A-1: Deformation Limits

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

Table A-2: NumericalAcceptance Criteria For Plastic Hinge Rotations in Reinforced concrete beams, In radians [12]

Component Type			***Performance Level***				
			Primary			***Secondary***	
			IO	***LS***	***SS***	***LS***	***SS***
1. Beams controlled by flexure							
$\frac{\rho-\rho'}{\rho_{bal}}$	**Trans. Reinf.**	$\frac{V}{b_w d\sqrt{f_c'}}$					
≤ 0.0	C	≤ 3	0.005	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.0	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.0	0.005	0.005	0.005	0.01

Table A-3: Numerical Acceptance Criteria for Plastic Hinge Rotations in Reinforced Concrete columns, in radians [12]

Component Type			***Performance Level***				
			Primary			***Secondary***	
			IO	***LS***	***SS***	***LS***	***SS***
Columns controlled by flexure							
$\frac{P}{A_g f_c'}$	**Trans. Reinf.**	$\frac{V}{b_w d\sqrt{f_c'}}$					
≤ 0.1	C	≤ 3	0.005	0.01	0.02	0.015	0.03
≤ 0.1	C	≥ 6	0.005	0.01	0.015	0.01	0.025
≥ 0.4	C	≤ 3	0.0	0.005	0.015	0.01	0.025
≥ 0.4	C	≥ 6	0.0	0.005	0.01	0.01	0.015
≤ 0.1	NC	≤ 3	0.005	0.005	0.01	0.005	0.015
≤ 0.1	NC	≥ 6	0.005	0.005	0.005	0.005	0.005
≥ 0.4	NC	≤ 3	0.0	0.0	0.005	0.0	0.005
≥ 0.4	NC	≥ 6	0.0	0.0	0.0	0.0	0.0

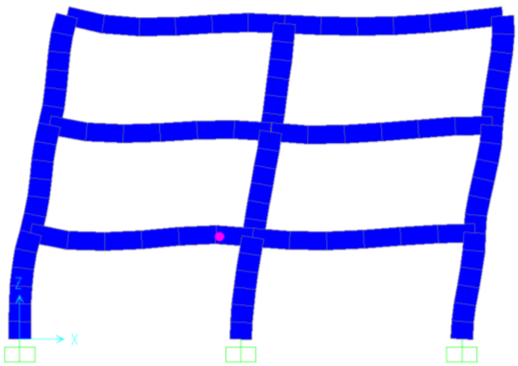
APPENDIX B1- PUSHOVER ANALYSIS WITH INCEREMENTAL APPROACH

Step 1					
Total Base Shear = 1+2+3 = 6KN					
Lateral Displacement at the top = 0.273mm					
Member	**Element Type**	**joint**	**Yield Moment, My (KNm)**	**Acting Moment, My (KNm)**	**Joint Condition**
1	Column	J1	124	-4.33	
		J2	124	20.6	
2	Column	J2	115.5	-22.14	
		J3	115.5	21	
3	Column	J3	107.5	-22.23	
		J4	107.5	27.35	
4	Column	J5	166	6.23	
		J6	166	-0.6	
5	Column	J6	143	3.5	
		J7	143	-2.94	
6	Column	J7	119	1.52	
		J8	119	-3.29	
7	Column	J9	133.5	16.03	
		J10	133.5	-20.07	
8	Column	J10	122	26.88	
		J11	122	-24.83	
9	Column	J11	110	22.95	
		J12	110	-30.82	
10	Beam	J2	49	-42.74	
		J6	49	-49.58	Yielded
11	Beam	J3	50	-43.24	
		J7	50	-49.28	
12	Beam	J4	53	-27.35	
		J8	53	-34.34	
13	Beam	J6	49	-45.48	
		J10	49	-46.95	
14	Beam	J7	50	-44.83	
		J11	50	-47.79	
15	Beam	J8	53	-31.05	
		J12	53	-30.82	

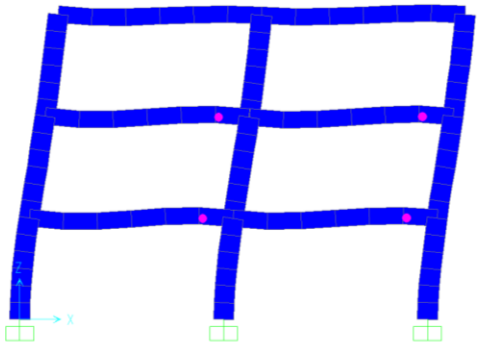
APPENDIX B2- PUSHOVER ANALYSIS WITH INCEREMENTAL APPROACH, PROGRESSIVE YIELDING AND HINGE FORMATION OF THE FRAME ELEMENT WITH THE ACTION EFFECT AT EACH STEPS

Step			**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
M	J	My , (KNm)	M1	M2	M3	M4	M5	M6	M7	M8	M9	M10	M11
1	J1	124	-4.33	2.06	59.85	68.44	123.78	124.03	124.03	124.03	124.03	124.03	YIELDED
	J2	124	20.6	21.36	33.48	35.48	51.34	51.37	55.41	8.77	6.94	1.6	
2	J2	115.5	-22.1	-20.0	4.59	8.5	37.16	37.24	46.05	51.79	52.23	54.41	
	J3	115.5	21	18.82	2.63	0.67	-5.71	-5.74	-9.37	-53.52	-55.26	-59.3	
3	J3	107.5	-22.2	-21.9	-24.1	-23.8	-13.4	-13.37	-11.3	-10.01	-9.71	-6.57	
	J4	107.5	27.35	25.53	6.59	4.63	-11.19	-11.25	-16.4	-55.09	-55.09	-55.09	
4	J5	166	6.23	12.7	69.55	77.98	132.48	132.74	167.9	167.9	167.9	167.9	YIELDED
	J6	166	-0.6	-0.21	11.97	14.04	30.07	30.09	26.46	-19.76	-21.58	-26.75	
5	J6	143	3.5	6.29	30.87	34.82	63.52	63.59	65.62	71.67	72.11	74.46	
	J7	143	-2.94	-6.09	-19.5	-21.2	-27.27	-27.29	-29.85	-73.59	-75.03	-79.22	
6	J7	119	1.52	3.08	4.07	4.57	15.32	15.36	18.37	20.09	20.73	23.73	
	J8	119	-3.29	-6.72	-41.6	-45.0	-60.89	-60.95	-66.13	-104.9	-106.7	-106.7	
7	J9	133.5	16.03	22.51	76.16	84.11	135.59	135.59	135.59	135.59	135.59	135.59	YIELDED
	J10	133.5	-20.0	-19.8	-1.87	1.03	22.46	22.55	28.5	4.35	3.51	1.42	
8	J10	122	26.88	29.45	47.45	50.35	71.78	71.87	77.82	53.67	52.83	50.74	
	J11	122	-24.8	-27.0	-35.2	-35.7	-34.56	-34.56	-35.58	-57.56	-58.1	-57.94	
9	J11	110	22.95	23.1	14.95	14.45	15.63	15.63	14.61	-7.37	-7.91	-7.75	
	J12	110	-30.8	-32.6	-51	-54.3	-54.35	-54.35	-54.35	-54.35	-54.35	-54.35	
10	J2	49	-42.7	-41.5	-28.9	-27.0	-14.27	-14.22	-9.45	42.92	45.19	52.71	YIELDED
	J6	49	-49.5	-49.5	-49.5	-49.5	-49.58	-49.58	-49.58	-49.58	-49.58	-49.58	YIELDED
11	J3	50	-43.2	-40.8	-26.7	-24.4	-7.69	-7.63	-1.93	43.5	45.53	52.71	YIELDED
	J7	50	-49.2	-51.6	-51.6	-51.6	-51.64	-51.64	-51.64	-51.64	-51.64	-51.64	YIELDED
12	J4	53	-27.3	-25.5	-6.59	-4.63	11.19	11.25	16.4	55.09	55.09	55.09	YIELDED
	J8	53	-34.3	-36.0	-53.6	-53.6	-53.68	-53.68	-53.68	-53.68	-53.68	-53.68	YIELDED
13	J6	49	-45.4	-43.0	-30.8	-28.8	-16.12	-16.07	-10.4	41.87	44.13	51.65	YIELDED
	J10	49	-46.95	-49.33	-49.33	-49.33	-49.33	-49.33	-49.33	-49.33	-49.33	-49.33	YIELDED
14	J7	50	-44.8	-42.4	-28.0	-25.8	-9.06	-9	-3.43	42.03	44.11	51.29	YIELDED
	J11	50	-47.7	-50.2	-50.2	-50.2	-50.2	-50.2	-50.2	-50.2	-50.2	-50.2	YIELDED
15	J8	53	-31.0	-29.3	-12.0	-8.61	7.22	7.28	12.46	51.24	53.1	53.1	YIELDED
	J12	53	-30.8	-32.6	-51	-54.3	-54.35	-54.35	-54.35	-55.7	-55.7	-55.7	YIELDED

APPENDIX B2- PUSHOVER ANALYSIS WITH INCEREMENTAL APPROACH, THE PLACE OF HINGE FORMATION OF BOTH THE INCREMENTAL APPROACH AND SAP2000 PUSHOVER ANALYSIS



Step 1



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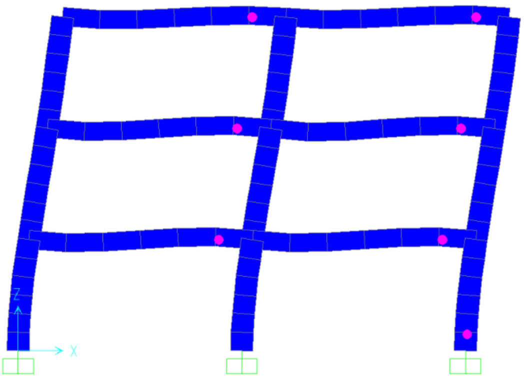

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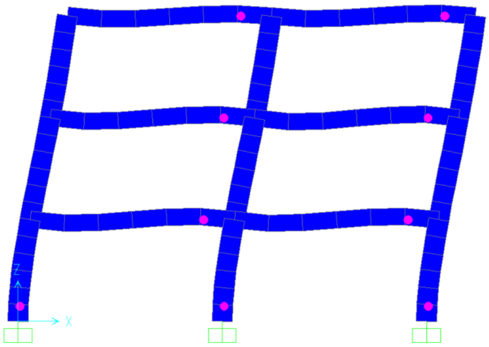

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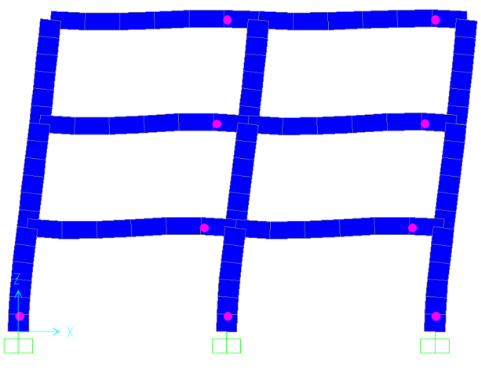

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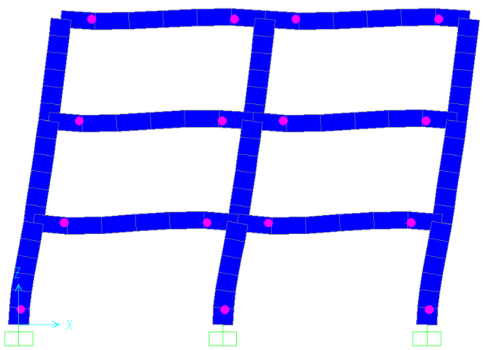
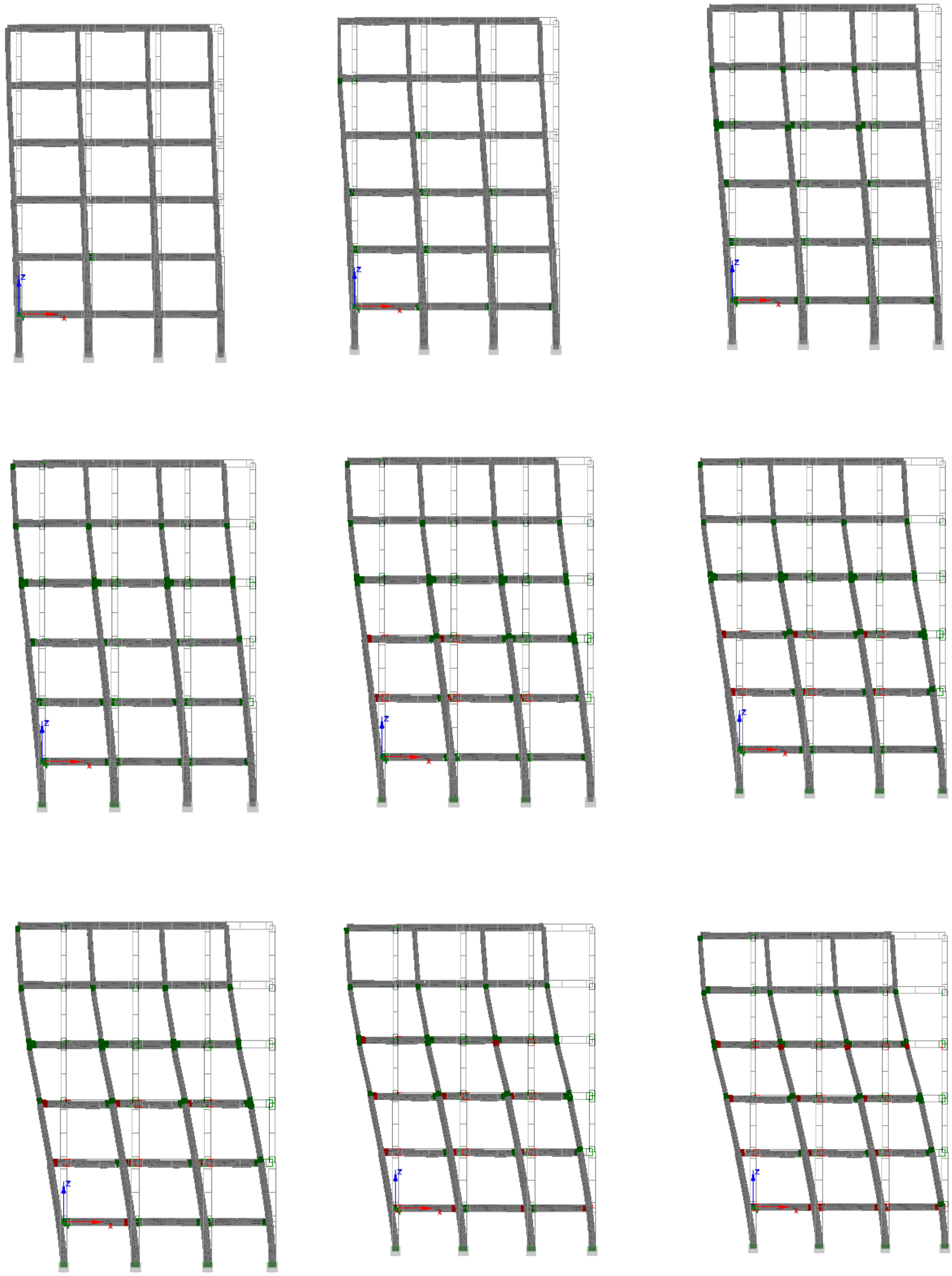
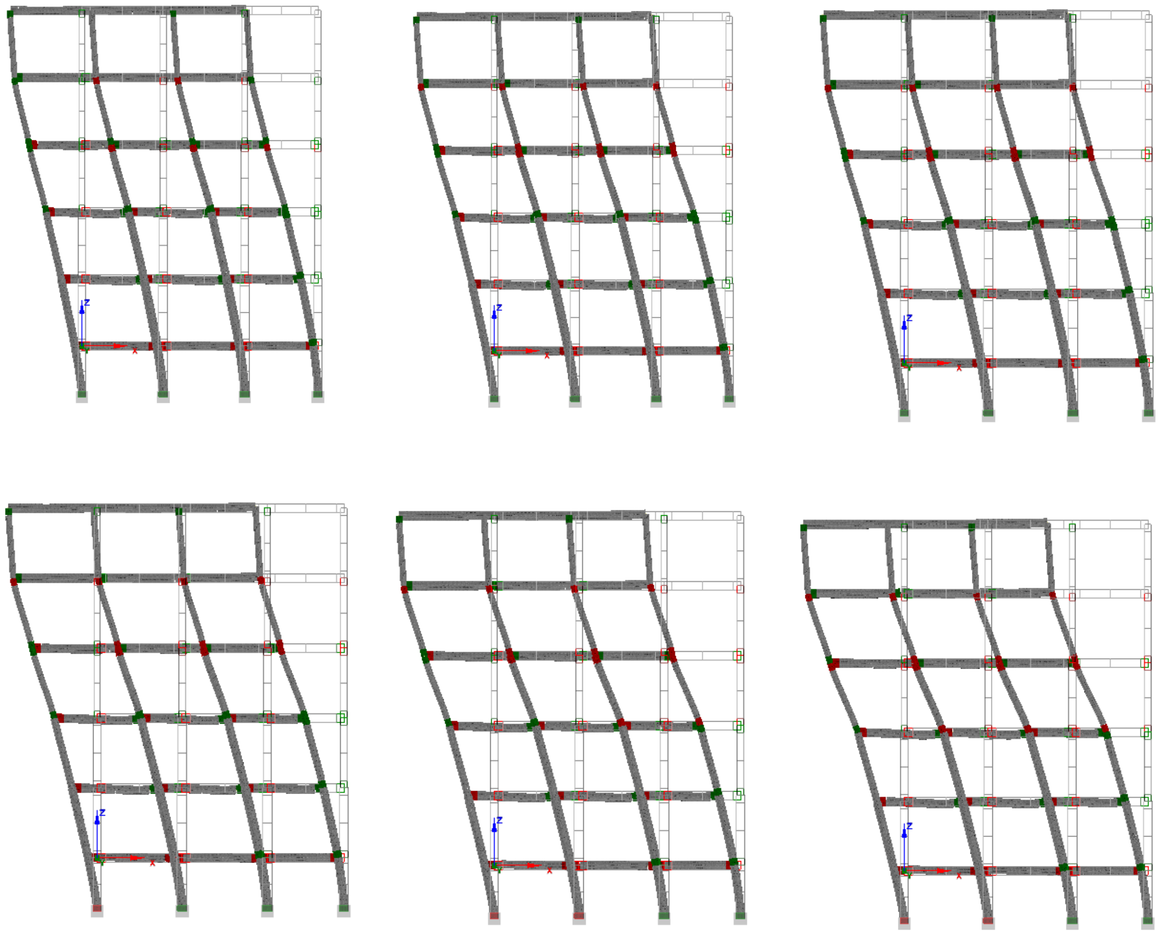


Step 6

APPENDIX C-PROGRESSIVE PLASTIC HINGE FORMATION FOR SEISMOSTRUCT





APPENDIX D-PROGRESSIVE PLASTIC HINGE FORMATION FOR SAP2000