PERFORMANCE BASED DESIGN OF MULTI-STOREYED BUILDINGS

A THESIS REPORT

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

With specialization in

STRUCTURAL ENGINEERING

Under the supervision

of

Dr. Tanmay Gupta

(Assistant Professor)

by

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173234

HIMACHAL PRADESH, INDIA

May 2021

STUDENT'S DECLARATION

I hereby declare that the work presented in the Project report entitled **"Performance Based Design of multi- storeyed buildings"** submitted for partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering with specialization in Structural Engineering at **Jaypee University of Information Technology, Waknaghat,** is an authentic record of my work carried out under the supervision of **Dr. Tanmay Gupta, Assistant Professor.** This work has not been submitted elsewhere for the reward of any other degree/diploma. I am fully responsible for the contents of my project report.

Pushkar Sharma

Signature of Student

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Jaypee University of Information Technology, Waknaghat

May 2021

CERTIFICATE

This is to certify that the work which is being presented in the project report titled "**Performance Based Design of multi- storeyed buildings**" in partial fulfillment of the requirements for the award of the degree of Master of Technology in Civil Engineering with specialization in "Structural Engineering" and submitted to the Department of Civil Engineering, **Jaypee University of Information Technology, Waknaghat** is an authentic record of work carried out by **Pushkar Sharma (192651)** during a period from July 2020 to December 2020 under the supervision of **Dr. Tanmay Gupta, Assistant Professor**, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of our knowledge.

Date: - 25 May 2021

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ABSTRACT

In general, the building is designed as per codal provisions, which has various constraints while analysing with dynamic loads. This analysis procedure takes a lot of time and is complex. Therefore, most of the Civil Engineering structures are designed taking the assumption of applied loading to be static. The process of neglecting the dynamic forces may lead to the collapse of the structure as a whole in case of a catastrophe such as an earthquake. Some recent earthquakes have shown the need for dynamic analysis. Nowadays, a lot of research is going on the field of performance-based design such that the structure can withstand earthquake-induced loads. This study confers the need to shift the design practice from force based to performance based for getting actual response. Three different analysis have been performed using empirical formulae and numerical modelling software to estimate the natural period of oscillation of building and the parameters are discussed on which it depends. Research and development in the field of earthquake resistant design has put emphasis on non-linear analysis methods to estimate seismic demands. Nonlinear time history and nonlinear static pushover analysis are the main methods. In this study, pushover analysis is carried out on multi-story reinforced residential concrete building in India. A non-linear structure is taken and with the help of two modern finite element programs, pushover analysis is performed. For SAP2000, a lumped plasticity model is taken and fibre based finite elements are established in SeismoStruct to determine the plastic behaviour of the vulnerable parts of the structure. A comparison between the results obtained from the two computer programs is presented in the study. It was observed that SeismoStruct showed the actual degradation curve and the softening behaviour due to deformation. The structure satisfied the concept of Strong column weak beam concept as the first plastic hinge was formed in the beams.

Keywords: Performance Based-Design, Pushover Analysis, Dynamic Analysis, SeismoStruct, Reinforced Concrete Buildings

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LIST OF ABBREVIATIONS

PBD	Performance Based Design
PBSD	Performance Based Seismic Design
ASCE	American Society of Civil Engineers
PE	Probability of Exceedance
O	Operational
IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention
RC	Reinforced Concrete
FEA	Finite Element Analysis
SEAOC	Structural Engineers Association of California
SMRF	Special Moment-Resisting Frame
RCR	Repair Cost Ratio
FRP	Fiber-Reinforced Polymer
NLTH	Non Linear Time History
CFRP	Carbon Fiber Reinforced Polymer
PGA	Peak Ground Acceleration
IDA	Incremental Dynamic Analysis
DCR	Demand Capacity Ratio
E	Modulus of Elasticity
I	Moment of Inertia
SFD	Shear Force Diagram
BMD	Bending Moment Diagram
Fz	Force in Z direction
Fy	Force in Y direction
Fx	Force in X direction
Mz	Moment in Z direction

My	Moment in Y direction
Mx	Moment in X direction
RCC	Reinforced Cement Concrete
IS	Indian Standard
Ta	Natural period
K	Lateral Stiffness
M	Mass
ω	Angular frequency
ATC	Applied Technology Council
NSP	Nonlinear Static Procedure
P-M2-M3	Axial force with biaxial moment
M3	Flexural moment

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CHAPTER 1

INTRODUCTION

1.1 GENERAL

The seismic activities in past few decades in several countries has raised the necessity for an elementary change in the current earthquake engineering design process [1]. The major earthquake events during the past few years, such as the 2001 M7.7 Bhuj (India), 2004 M9.1 (Indonesia), 2011 M9 (Japan) and 2015 M7.8 (Nepal) have shown the world, the destructive power of earthquakes. Although the structures designed as per the current codes performed well taking the life safety viewpoint but the measure of destruction to the engineered structures led to great economical losses as well as high repairing costs.

With every revision of building and seismic codes, the clauses advances itself with respect to latest research and advancement ensuring the state of collapse and serviceability for plain and reinforced concrete and proper ductile detailing for structures which falls in different seismic zones. These codes mainly focuses on forces as input and output comes in the form of displacement, moments, drift. These input forces are designed under elastic analysis and for earthquake the inelastic forces, stiffness are taken indirectly by Response Reduction factor, which results in misjudgment in the actual building response with such indirect approach [2]. In present for elastic analysis, we use force-based codes for designing the primary and secondary components of any structure. Ductile detailing and displacement limits are designed using Serviceability checks. Now for important buildings and structures like skyscrapers, water retaining structures, dams and tunnels, bridges etc, we need to shift the design practice from force based to performance based for getting actual response.

1.2 PERFORMANCE BASED DESIGN

PBD is a method which was developed over the period from past experiences of earthquakes and provide the realistic approach by incorporating the dynamics which gives us the output similar to time history analysis which depicts the actual results and used as verification methods. The trace of comprehensive design can be traced to earlier 1960,s where performance level of structure (structure response to various failure stage) are correlated with hazard level or the return period of earthquakes so that the loss direct and indirect can be minimized.. A philosophy regarding three design objectives was introduced in the commentary of SEAOC Blue Book in the year 1967 for earthquake resistant design of building other than essential and hazard facilities.

Performance based seismic design (PBSD) is a design practice whose main focus is to design a reliable structure based on performance objectives which direct towards achieving target performance for design earthquake. The PBSD design procedure gives the realistic and reliable assessment of damage indices, loss of strength in members with respect to time [3, 4]. The first step of PBSD is to fix performance objective depending upon the owner, designer or building official. After assessing the location and seismic intensity record of site, performance level for structure is decided considering the frequent level earthquakes, design basic earthquakes, and Maximum consideration earthquakes whose objectives are serviceability, code level moderate damage to primary components and life or collapse prevention. After performance objective and performance level the goals are decided in which criteria like strength and serviceability is considered with limited ductility which will reduce the probability of damage to acceptable level and allows a proper load combination with respect to Maximum design Earthquake to ensure the structural performance.

Based on the objectives and goals an assessment is made whether the structure is designed according to objective and goals set by the owner, engineer. If structure performance meets the objective then the design is ready for actual construction but if desired performance is not achieved then the design is revised or objective are modified or altered until the desired performance is met [4]. A flow diagram is shown in Figure 1.1, which define the basic systematic procedure described above. By following these steps, the designer can proceed with higher level of confidence in designing structure beyond elastic limit and getting higher level of performance from design codes.

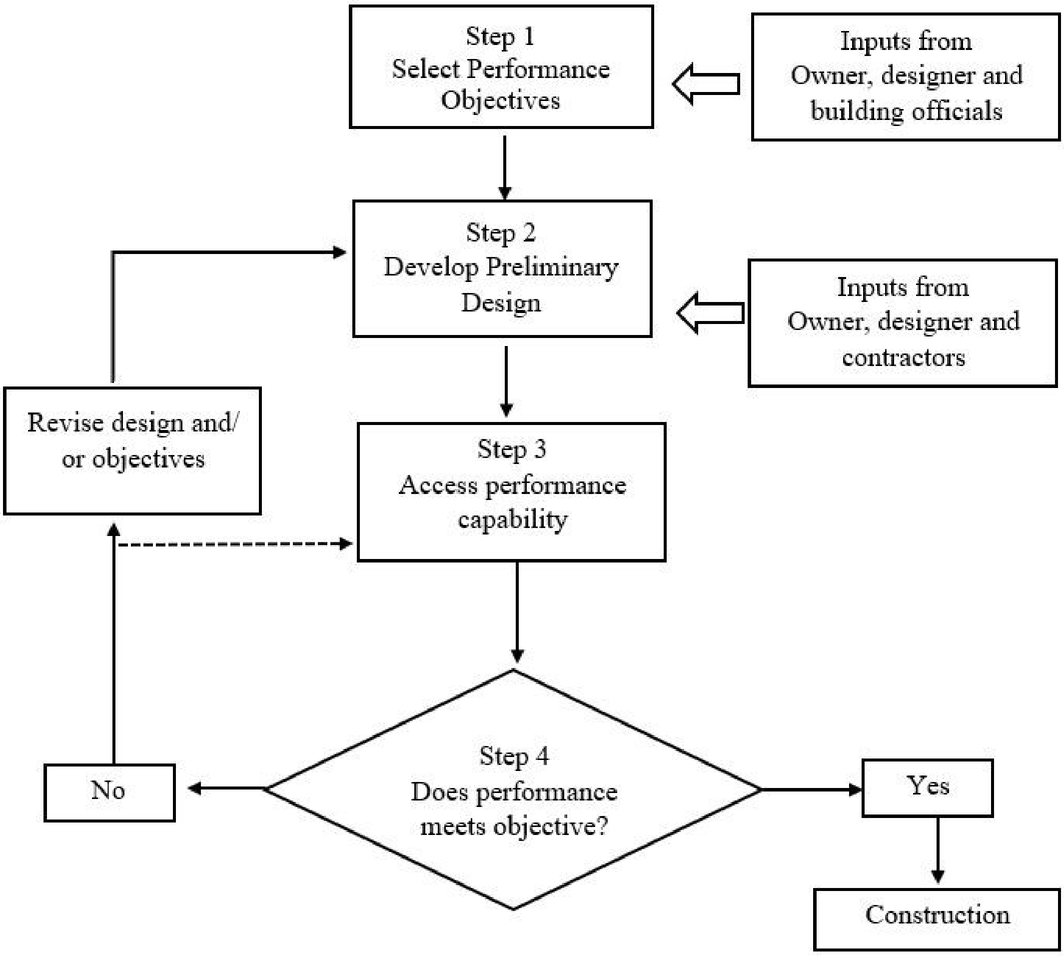

Fig. 1.1 PBD Flow Diagram [4]

Performance levels are based on ground motion level, damage state, displacement and drift. These seemed to be oversimplified but no of cycle, duration, acceleration, transfer of forces on member and their behavior or response to seismic forces, which reduced stiffness in members and their failure modes influences the performance levels. Sometimes displacement-based design is used in terms of performance based design due to the major significance of displacement. In PBSD we have capacity spectrum method, N2 method and Displacement based design method all methods have some advancement over the former and these methods are used for finding target performance in the form of displacement, drifts and damage levels.

1.3 IMPORTANT POINTS OF PBD

1. Performance based seismic approach can be used to design new as well as existing structures.

2. Performance based design considers both material and geometric nonlinearity.
3. Performance based design is performed using target displacements to determine the damage level.
4. Performance based design do not use response reduction factor.
5. Performance based design provides the ease to identify the damage in vulnerable members which can be modified by retrofitting.
6. Performance based design can also be called as displacement based design because performance levels are based on ground motion level, damage state, displacement and drift.

1.4 PERFORMANCE OBJECTIVE

Performance objectives are the affirmations of allowable performance of a building. A Performance Objective comprises of two key elements. They are Damage state and Seismic hazard. The performance of a structure is expressed by the delegation of permissible damage to an earthquake hazard. The level is indicated by the damage and the hazard of earthquake by the ground movements. The standard performance level is divided into two stages of damage: structural and non-structural damage. The accumulation of both damages gives the structure an overall level of performance. The four types are depicted in the following part.

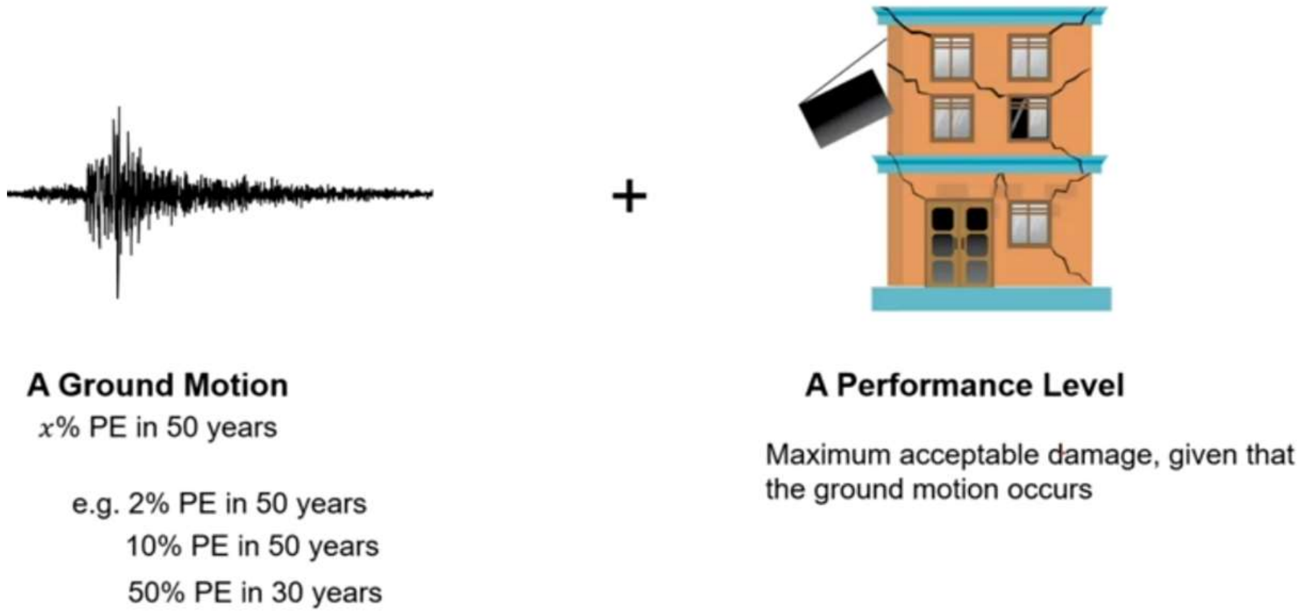

Fig. 1.2 Performance Objectives

1.5 PERFORMANCE LEVELS

The performance standards elucidated by ASCE 41 [5] for both structural and nonstructural components, are the widely accepted for performance-based design. The different performance levels being used are in the ascending order of structural deformation.

Fig. 1.3 Performance Levels of building

1.5.1 Operational Level

Structures accepting this level of efficiency do not harm the structural and non-structural elements. The building will usually run without interruption, but minor modifications for fuel, water, and so on should be made. Even the building's tenants are not expected to vacate. This is called the most competitive standard of efficiency, but it cannot be done with any structure because it is ineffective from an economic standpoint.

1.5.2 Immediate Occupancy

Structures with such an immediate occupancy degree of success are projected to have minimal structural damage and only minor damage to the non-structural elements. The structure after an earthquake is secure to reoccupy. However, certain non-structural elements might therefore reparate particularly vulnerable non-structural elements. At this stage of success, the risk for the inhabitants is much smaller. This efficiency standard is also not so economical. Life protection is the fundamental safety provision in accordance with the code.

1.5.3 Life Safety

It is anticipated that structures with this standard can do significant harm to the structural as well as non-structural components. The residents will not be able to relocate the house; repairs are needed before returning to the building. At this amount, the vulnerability of the inhabitants of buildings increases slightly. Life protection under the design basis earthquake according to the FEMA code 356[6] is considered as a specific output goal.

1.5.4 Collapse Prevention

Structures that reach this standard of performance can provide an elevated risk to the life of the tenants, due to failure of the non-structural components, but loss of life can be prevented because the plastic hinges are developed. The restoration work is not advisable, the house has to be destroyed in most circumstances.

Table 1.1 Performance Levels

Parameter	O Level	I O Level	L S Level	C P Level
Structural damage	Negligible	Negligible	Significant	Extensive
Non-structural damage	Negligible	Minor	Extensive	Extensive
Injury	No	No	Some	More but no loss of life
Repair	No	No	Required	May not be practical
Loss	5%	15%	30%	>>30%

1.6 ORGANISATION OF THESIS

The thesis is comprised of six chapters. The description of each chapter is given briefly as follows:

Chapter 1 Introduction This chapter presents the explanation of Performance based design approach and its techniques, performance objectives and levels. It discusses about the current aspect of this methodology.

Chapter 2 Literature Review This chapter is basically the foundation of this study. It discusses the past, current and future scope of research in field of PBD. The knowledge is accumulated from various research articles, books and seismic codes and from them research gap and objectives are formed for this study.

Chapter 3 Structural Modelling and Verification In this chapter various models linear as well as nonlinear are modelled in SeismoStruct and verified either by hand calculations or by other computer softwares whose results are known to us. In this chapter a case study from a journal paper is verified.

Chapter 4 Investigation on natural frequency This chapter deals with the current seismic analysis procedure described by Indian seismic code to determine the natural period of oscillation of structure. The same structure is modelled and analyzed in finite element software SeismoStruct, the modes shapes are plotted, and results are compared.

Chapter 5 Comparative study on non linear static analysis This chapter deals with modelling and analysis of low-rise RC frame building. Two FEA softwares are used in this study and the results are compared to draw conclusions regarding the reliability and accuracy of the programs.

Chapter 6 Conclusion This chapter concludes and summarizes the entire thesis. The conclusions are established from the analysis results obtained from the study.

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

This chapter covers the brief review of literature about performance based engineering and finite element programs used in the analysis and design. Literature regarding performance evaluation techniques are also examined. This chapter focuses on the past research and development in the field of performance based evaluation methods. This chapter provides the various linear and nonlinear analysis techniques, which can be performed on SeismoStruct.

2.2 LITERATURE REVIEW

A M. Chandler and N T K. Lam [7] studied the historical development linked with Performance based earthquake engineering. The fields on which they research were Seismology, Geology, Soil and System Dynamics and Mechanics of Materials. Detailed review was done for each part by studying all the research and development till that time. They emphasized on the future study to determine maximum considered earthquake. The major consequence of the study led to the identification of fundamental constraints in current seismic procedures.

A Ghobarah [1] conducted a study on various developments in the discipline of PBD. Three testimonials laid the foundation of performance-based design. SEAOC Vision 2000 was the main testimonial. The study stated that current design codes were not reliable as they focused on life safety viewpoint and collapse prevention aspect for an earthquake. However, the design criteria should be demonstrated in respect of reaching specific performance goals considering particular level of seismic hazard. The paper discussed the various design evaluation methods, challenges and future scope of performance based earthquake engineering.

S M. Easa and W Y. Yan [8] conducted a study on performance based design and reviewed its applications in three major civil engineering fields: Structural Engineering, Transportation Engineering and Environmental Engineering. They presented 187 publications and 122 application papers from 23 countries in these fields. The study showed that the United States and Canada are the main countries, which has encouraged the use of PBD in their specifications. In Asian countries such as China, Japan, Iran and India, this is a matter of research. Australia, France and the United Kingdom have least publications in this field.

Q. Zhang and M S. Alam [9] studied the practices of performance based design for bridges. In the study, codes from the US, Canada, China, Japan, New Zealand and Europe were reviewed and a case study was executed to draw comparison among these design codes. Canadian Code (CSA S6-14) found out to be having the most stringent criteria. Challenges and future scope were discussed in this paper. Bridge damage states creates difficulty in prediction of traffic interruptions, as it is not associated with residual vertical load capacity. Therefore, the study suggests that more research and investigation should be done on residual vertical load capacity factor.

Y E. Ibrahim and M M. El-Shami [10] developed vulnerability curves for typical RCC moment-resisting frames in KSA. Two models of 4-storey and 8-storey buildings were surveyed for three geographical areas having different seismic intensities. They were considered to get diversity in the amount of spectral accelerations. The structural prototypes were designed according to the country design code and IDA was executed with the ground motions of twelve different earthquakes in finite element software SeismoStruct. In this study, fragility curves were presented taking in account five preferred performance levels. The structures performed well and showed good seismic performance under earthquakes. Seismic performance in Al-Sharaf city was better in comparison with Jazan and Abha.

M. Rashid and N. Ahmad [11] presented the seismic performance evaluation of RC framed structures who were studied according to seismic codes. In this study, four frame models varying in the number of stories were modelled and designed to estimate the economic loss due to earthquakes. The four models having three, five, eight and ten floor levels were taken in this study. Quasi-static cyclic tests implemented on the beams to estimate the damage scale. IDA was run on the modelled structures by applying 7 ground motion records. The structure RCR was linked with seismic intensity for developing seismic vulnerability curves. These curves were used for the calculation of economic loss of structures. RCR of 20.21%, 14.91%, 14.94% and 12.17% were obtained in this study.

A M. Elshihy et al [12] stated an assessment of seismic performance of RCC buildings in Egypt. In this study, five structural models were designed. Three analysis were run on finite element software named SeismoStruct. The modes shapes and fundamental modes with natural frequency were determined and nonlinear analysis were also done by using 12 ground motions. The horizontal capacity of the structures were evaluated with the analysis. Four performance levels were considered in this paper.

R. Latifi and R. Rouhi [13] conducted a study to determine the most convenient retrofitting technique for reinforced concrete structures making use of current standards and design codes. In this, various techniques were used to retrofit a 2-story RC building in SeismoStruct. The seismic assessment of the retrofitted structures were compared with the original building. The results showed that demand capacity ratio (DCR), natural period and roof displacements could be reduced by using RC walls at the boundary of the original structure. Nonlinear static analysis of the structure with RC jackets and reinforced concrete walls showed an augmentation in the capacity curves.

M C. Porcu et al [14] conducted a study on the seismic retrofitting of old buildings by exectuing nonlinear time history analysis. In this paper, the merits of adopting this procedure were presented to indicate the critical characteristics in seismic response of old structures. In this study, the critical sections were strengthened using carbon fibre reinforced polymer. The behaviour of the retrofitted building was evaluated in two finite element softwares: SeismoStruct and SAP2000. The differences in each model were elaborated. In this study, the results from both the approaches were compared and it was found that SeismoStruct performed well in non linear dynamic analysis comparative to SAP2000. The process to model CFRP model is time consuming in SAP2000 whereas in SeismoStruct, one can directly model it.

A. Ismail [15] investigated the seismic behaviour of an old building located in Cairo. In this study, nonlinear static analysis was executed for both retrofitted and non-retrofitted building. Reinforced concrete, steel sections and CFRP composite jackets were the retrofitting methods adopted and comparison was conducted considering the performance levels. The results showed higher lateral strength when jacketing with CFRP sheets was done. The steel and reinforced concrete jacketing also improved the lateral displacement capacity with significant increase in lateral strength.

F. Cheraghi and A S. Moghadam [16] evaluated an existing hospital structure in Karaj by performing IDA in SeismoStruct. In this study, two 2D frames are taken and analysis is done considering five levels i.e. IO, LD, LS, LLS and CP. The IDA results with the peak ground acceleration of 0.6g showed the probability of exceedance of 99% for IO, 96% for LD, 82% for LS, 53% for LLS, and 43% for CP.

H. Crowley and R. Pinho [17] conducted a study on relationship of period and height for existing RCC structures. They reviewed the various design codes, which has various empirical formulae for calculating the natural period of vibration. In this study, height was found the main parameter. All the codes work on force-based design but displacement- based demand provides the exact indication of damage, so in this study the displacement based assessment of European buildings is focused. Various analysis were performed to obtain the yield period of numerous buildings with varying height.

P G. Asteris et al [18] conducted a study on 14 storey designed and non-designed RC building. The natural period of high-rise building was studied with modal analysis and finite element approach. Stiffness, mass and strength along the height of a building influence the natural period. Building height is the main parameter. There are many other factors, which affect this property such as section dimensions, structural regularity, number of bays and storeys, load position, soil flexibility, reinforcement ratio, and infill and shear walls. The research showed that change in span length could change the period. However, the soft storey do not contribute to high fundamental period.

2.3 SUMMARY OF LITERATURE REVIEW

1 PBD provides superior results compared to Code based design.
2 PBD can be used in calculating the Repair Cost Ratio.
3 PBD is realistic approach, which can be used for seismic retrofitting of structures.
4 IDA and NLTHA are adopted to obtain fragility curves.
5 Indian codes don’t have any provision of PBD.

2.4 RESEARCH GAP

1. Traditional codes are not efficient enough as they work upon force based design.
2. A comprehensive study is needed to incorporate PBD in Indian seismic codes.
3. Indian seismic code focusses on ‘safety’ but not on ‘performance’.
4. IS 1893 has many limitations which can be resolved by PBD.
5. PBD is more generalised and reliable as compared to seismic codes.

2.4 RESEARCH OBJECTIVES

1 To study the need of Performance Based Design.

2 To verify a well-known problem with software modelling based on PBD.

3 To compare and investigate the mode shapes and natural frequency of low-rise building.

4 To compare the results of case study by two finite element programs.

CHAPTER 3
STRUCTURAL MODELLING & VERIFICATION

3.1 GENERAL

In this chapter, different models of elements of a building are analyzed using Seismostruct. The results are verified either with hand calculations or with structural analysis softwares such as STAAD Pro and SAP2000. Starting from a basic 2D beam element to 3D portal frame the global parameters like displacement, reactions, moments and other results are found out with the SeismoStruct. Eigen value analysis is carried out on a 3-storey building and then the results are compared with seismic analysis as per Indian code.

3.2 SOFTWARE DESCRIPTION

Seismosoft is an established leading company in the discipline of seismic engineering. It provides the engineering background, the access to a robust and systematic tool, which can be used by designers and researchers even if they are not experts in finite element analysis. SeismoStruct is one of the latest softwares of Seismosoft.

SeismoStruct [19] is a nonlinear finite element program SeismoStruct which is used in this study. SeismoStruct is proficient in forecasting displacement behaviour of space as well as plane frames under dynamic and static loading taking both material inelasticity as well as geometric nonlinearity. There are many material models available such as concrete, steel, fibre reinforced polymer and shape-memory alloy. Only a few finite element softwares have this super elastic shape memory alloy. There is a wide range of three-dimensional elements, which can be used with various types of steel, concrete and composite sections. Numerous successes in Blind Test Prediction Exercises shows its accuracy.

The software has three main sectors: a Pre-Processor, a Processor and a Post-Processor. The former is used to input the data required for the analysis of structural model, Processor is used to run the analysis and all the output files and results are obtained in Post-Processor. Moreover, it includes two more components which are known as Building Modeller and Wizard. These facilities help the designer in creating regular and irregular shaped 2-dimensional as well as 3-

dimensional models. With them, the analyses can be run quickly as it takes only a few minutes for the whole process.

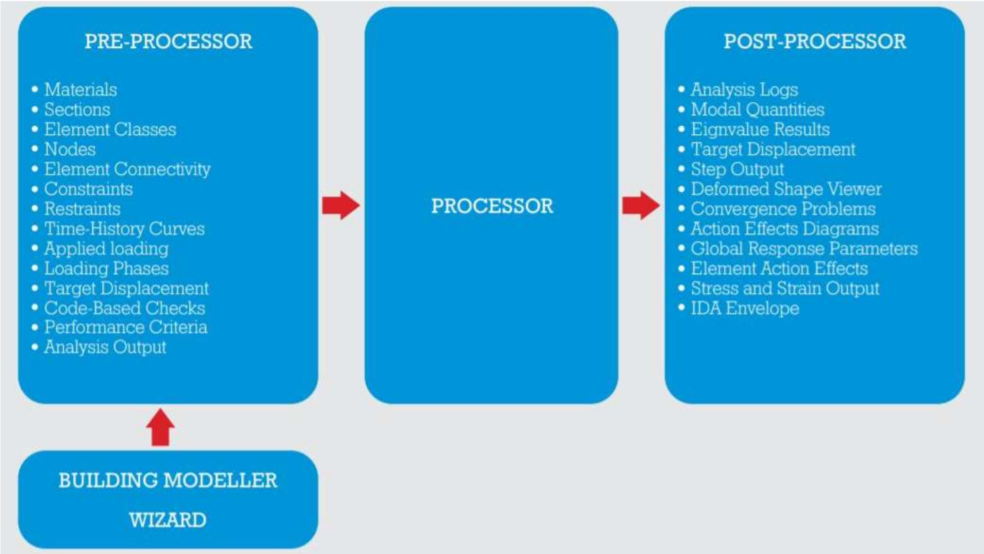

Fig. 3.1 SeismoStruct flowchart

Eight different types of analysis can be performed in SeismoStruct and it supports six seismic design codes, which belong to countries such as America, Europe, Italy, Greece and Turkey.

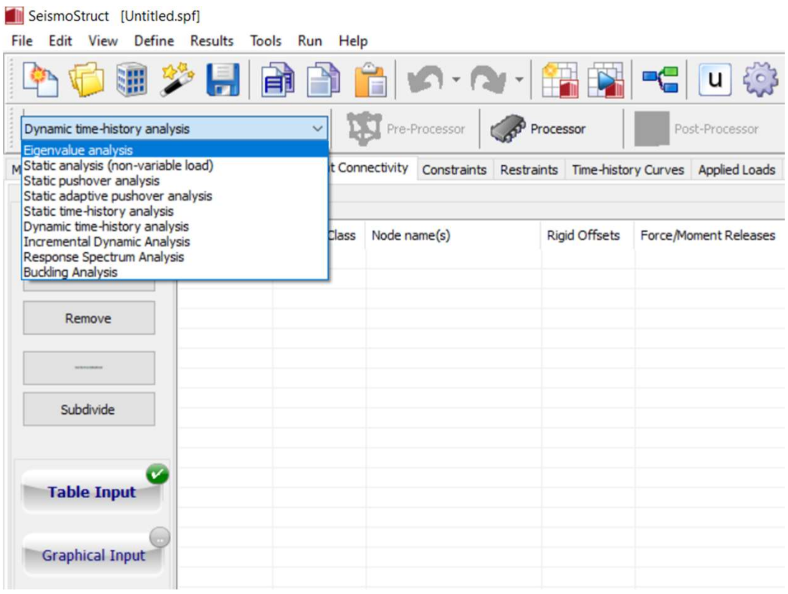

Fig. 3.2 Analysis types performed in SeismoStruct

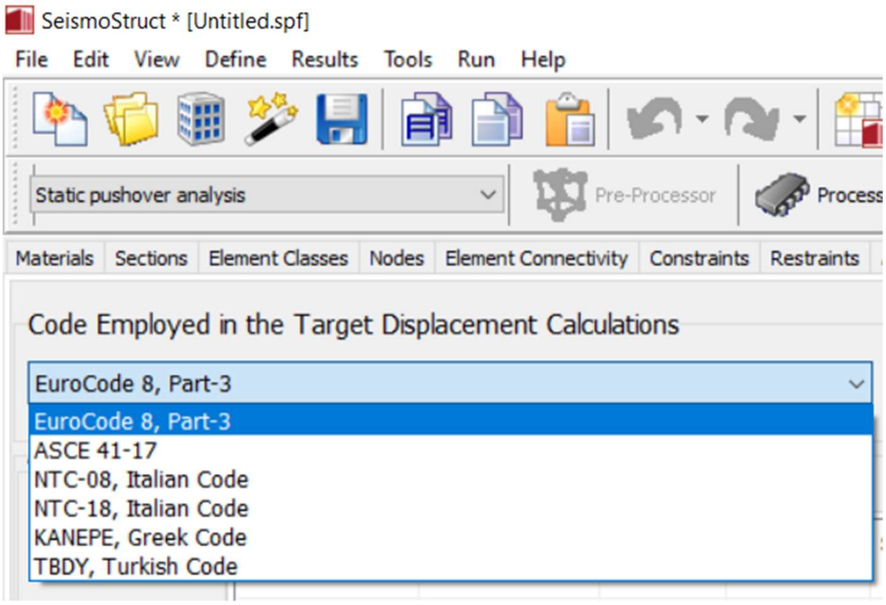

Fig. 3.3 Codes in SeismoStruct

3.3 VERIFICATION OF CANTILEVER BEAM

Analysis of a cantilever beam having a point load at free end is carried out and the deflections, slopes, reactions and moments are verified with hand calculations. The beam diagram, input data, hand calculation and software results are as follows:

Fig. 3.4 Cantilever with point load

Table 3.1 Input data for cantilever beam model

Length of beam (L)	5m
Area of Cross Section (b × d)	0.4m x 0.4m
Modulus of Elasticity (E)	2 x 10^5N/mm^2
Point Load (w)	4kN

Table 3.2 Hand calculation results for cantilever beam model

PARAMETER	FORMULA	VALUE
BM at n1	wl	20kNm
Reaction at n1	w	4kN
Deflection at n2	$wl^3/3EI$	0.00039m
Slope at n2	$wl^2/2EI$	0.00011718 rad

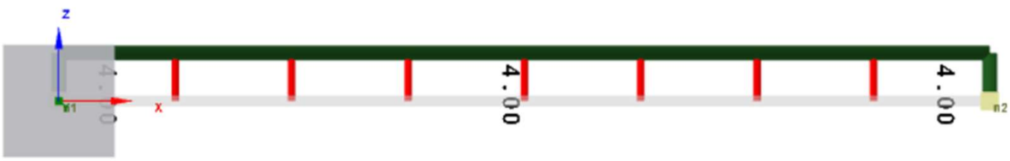

Fig. 3.5: SFD

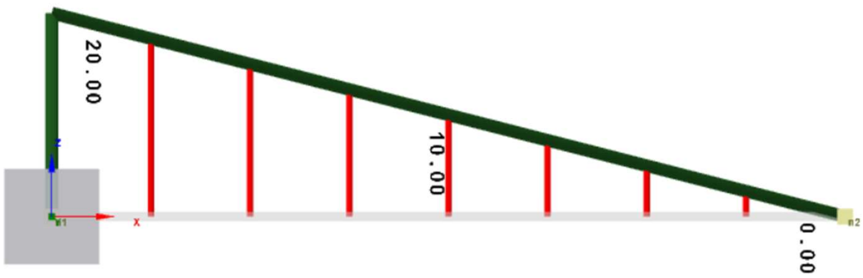

Fig. 3.6 BMD

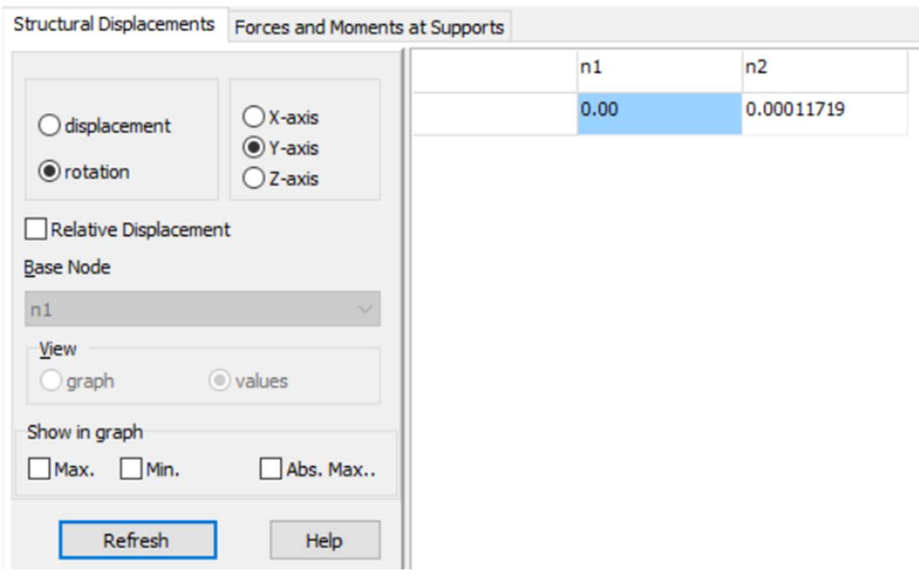

Fig. 3.7 Slope values of cantilever beam

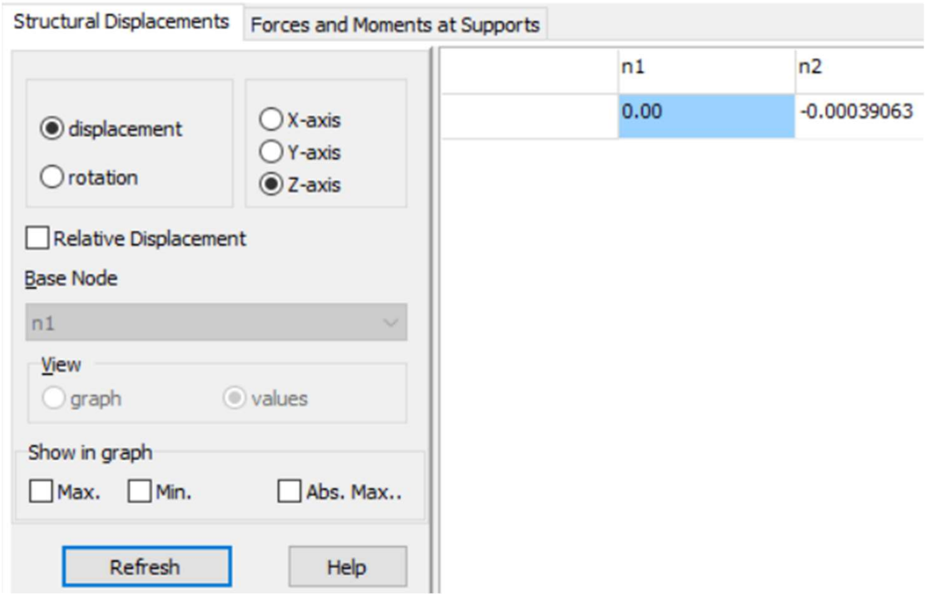

Fig. 3.8 Deflection values of cantilever beam

3.4 VERIFICATION OF SIMPLY SUPPORTED BEAM

In this, a simply supported beam is analyzed which has a distributed load throughout the length of the beam. The deflections, slopes, reactions and moments are verified with hand calculations. The beam diagram, input data, hand calculation and software results are as follows:

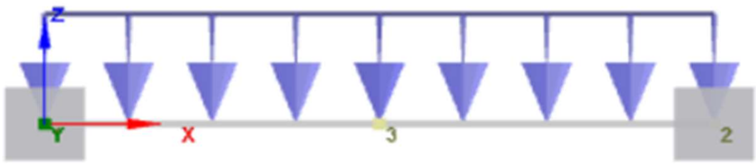

Fig. 3.9 Simply supported beam with udl

Table 3.3 Input data for simply supported beam model

Length of beam (L)	5m
Area of Cross Section (bxd)	0.1m x 0.2m
Modulus of Elasticity (E)	$2x10^5 N/mm^2$
Point Load (w)	10kN/m

Table 3.4 Hand calculation results for simply supported beam model

PARAMETER	FORMULA	VALUE
BM at n3	$wl^2/8$	31.25kNm
Reaction at n1 & n2	$wl/2$	25kN
Max Deflection at n3	$5wl^4/384EI$	0.006103m
Max Slope at n2 & n3	$wl^3/24EI$	0.00011718 rad

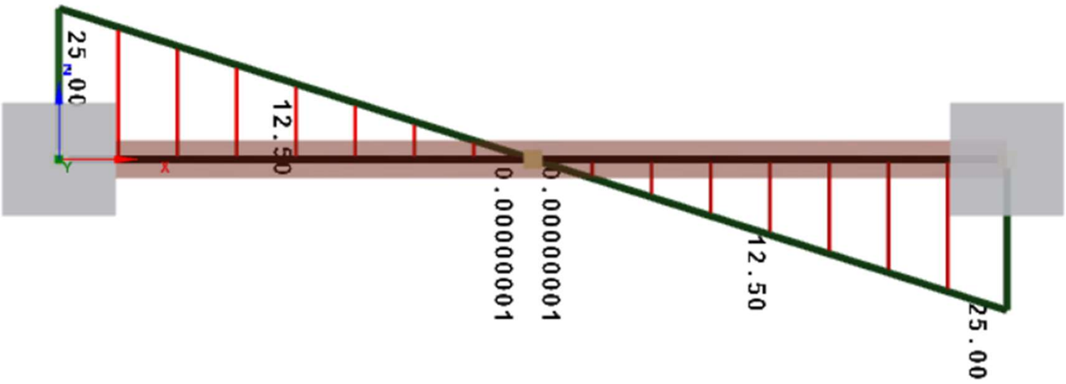

Fig. 3.10 SFD

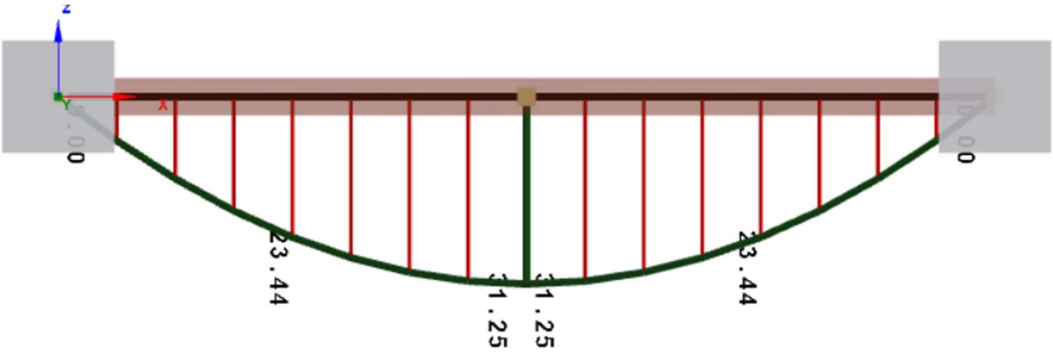

Fig. 3.11 BMD

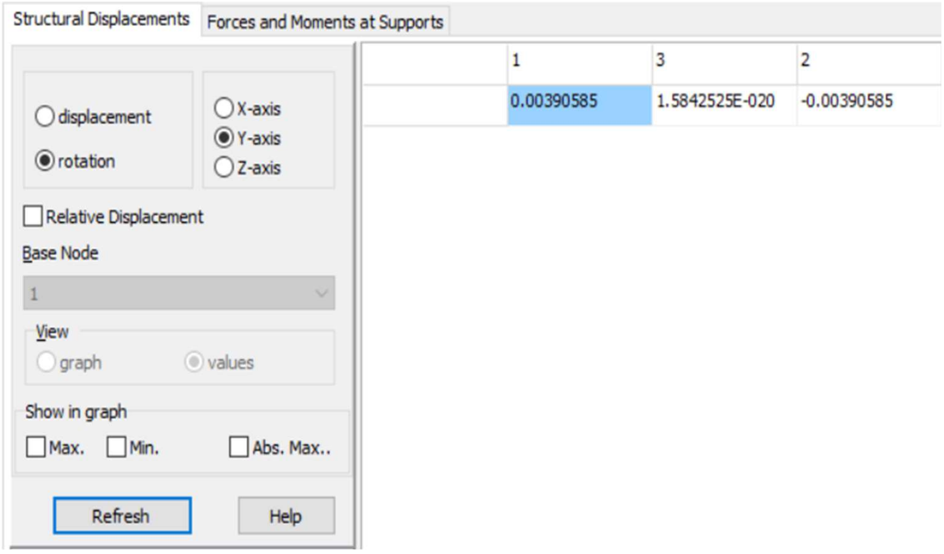

Fig. 3.12 Slope values of simply supported beam

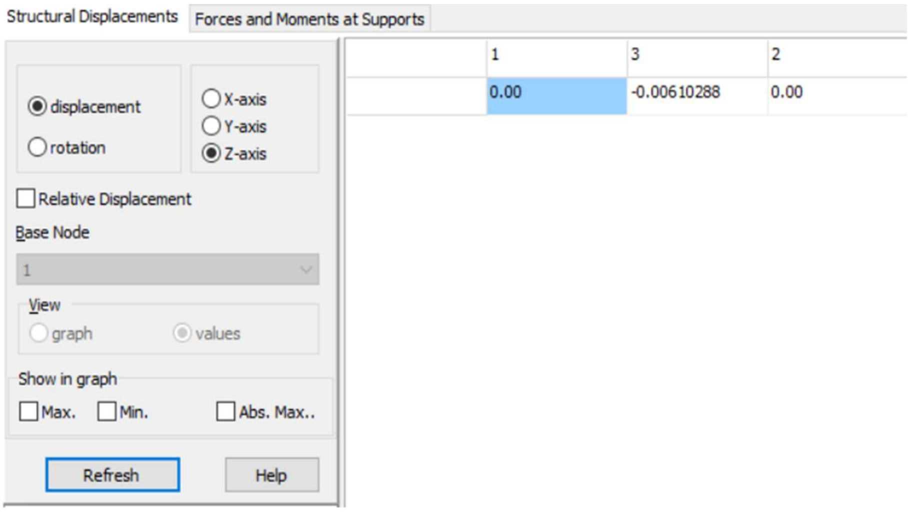

Fig. 3.13 Deflection values of simply supported beam

3.5 VERIFICATION OF 3D PORTAL FRAME

Analysis of one bay one story 3D frame having uniformly distributed load throughout the beam is carried out and results of the analysis are verified. The frame is analysed on STAAD Pro which is the most common software used for analysis and SeismoStruct. The frame diagram and analysis results from both the software are as follows:

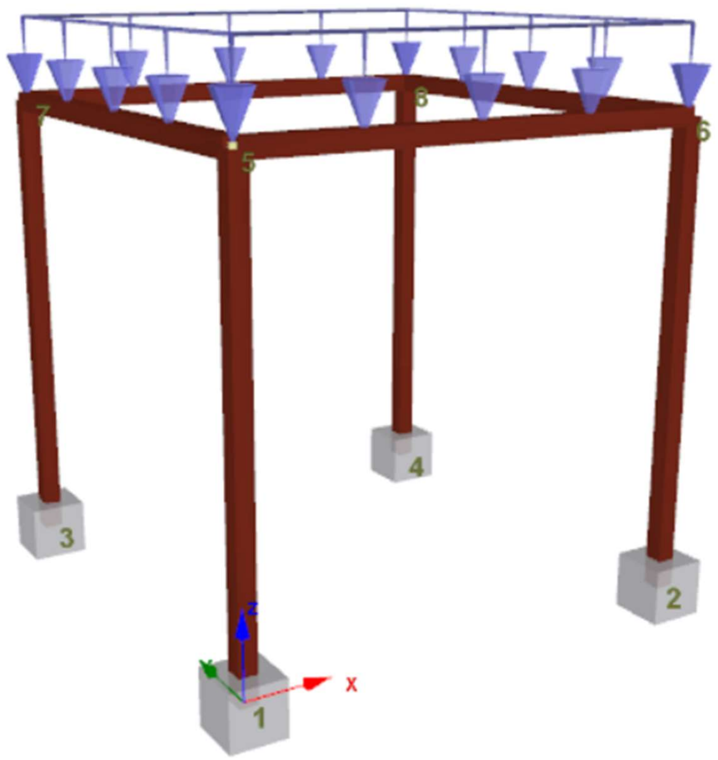

Fig. 3.14 3D frame with loading

Table 3.5 SeismoStruct analysis results of frame

PARAMETER	N 1	N 2	N 3	N 4
Fz	50	50	50	50
Fy	4.16504	4.16504	-4.16504	-4.16504
Fx	4.16504	-4.16504	4.16504	-4.16504
Mz	7.023×10^{-17}	6.97×10^{-17}	7.023×10^{-17}	6.97×10^{-17}
My	6.93889	-6.93889	6.93889	-6.93889
Mx	-6.93889	-6.93889	6.93889	6.93889

Table 3.6 STAAD Pro analysis results of frame

PARAMETER	N1	N2	N3	N4
Fz	50	50	50	50
Fy	4.155	4.155	-4.155	-4.155
Fx	4.155	-4.155	4.155	-4.155

Mz	0	0	0	0
My	6.905	-6.906	6.905	-6.905
Mx	-6.905	-6.905	6.905	6.905

3.6 VERIFICATION OF MULTI-STOREYED BUILDING

In this study, Eigen value analysis is performed on a case study of 4-storeyed building located in Delhi having zone IV which is an earthquake prone area [20]. The building is symmetric along X and Y-axes having plan dimensions 10m × 10m. The height of the structure is 12.5 m. All storey heights are of 3 m except ground storey whose height is 3.5m. The values of natural frequency and time period are compared with the case study results.

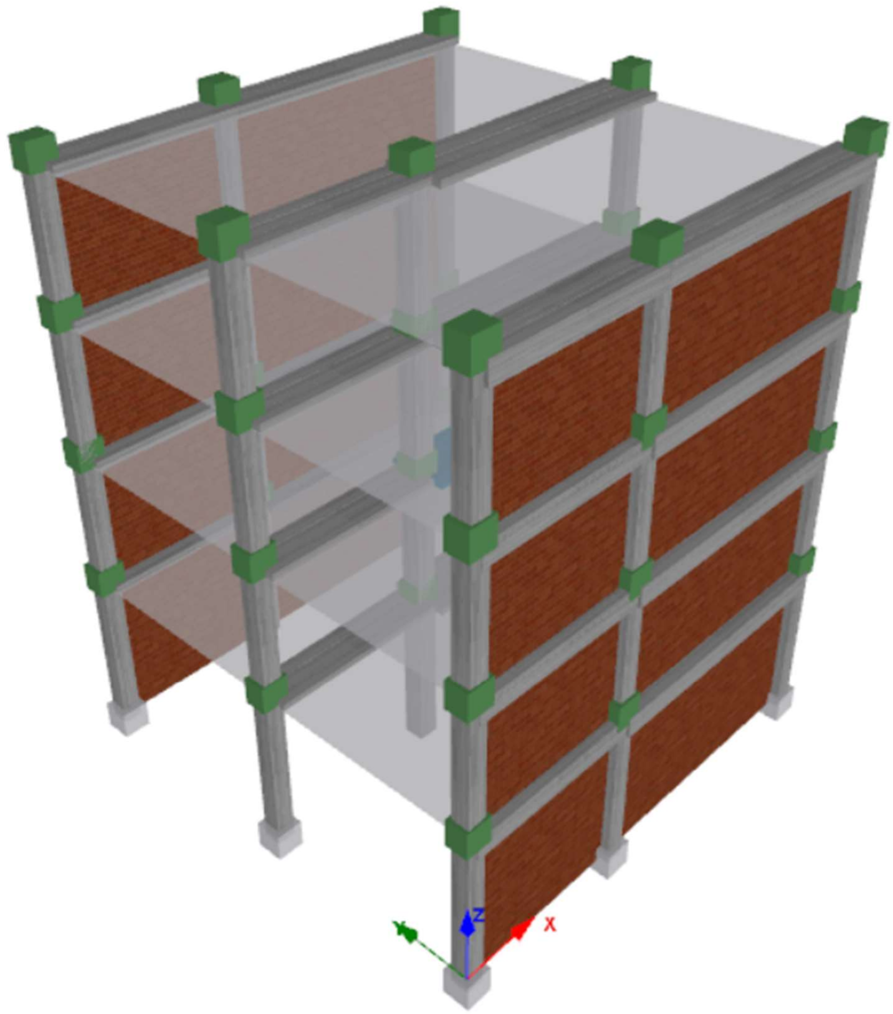

Fig. 3.15 3D model

3.6.1 Material Specifications

The models of Mander et al. [21] have been used for concrete specimens, which considers the cyclic nature of concrete. In general, nonlinearity of RCC is very dependent on the reinforcement. Consequently, steel models have an utmost significance for the evaluation of flexural nature of a RCC section, and mainly when exposed to load reversals. The Menegotto and Pinto [22] model is used in this case study. The Menegotto and Pinto model has also been included in several studies for its simplicity and efficiency because it considers the softening of curves in reloading automatically.

M25 concrete and Fe415 steel is used to model concrete elements and steel reinforcements respectively. The material properties are shown in figures.

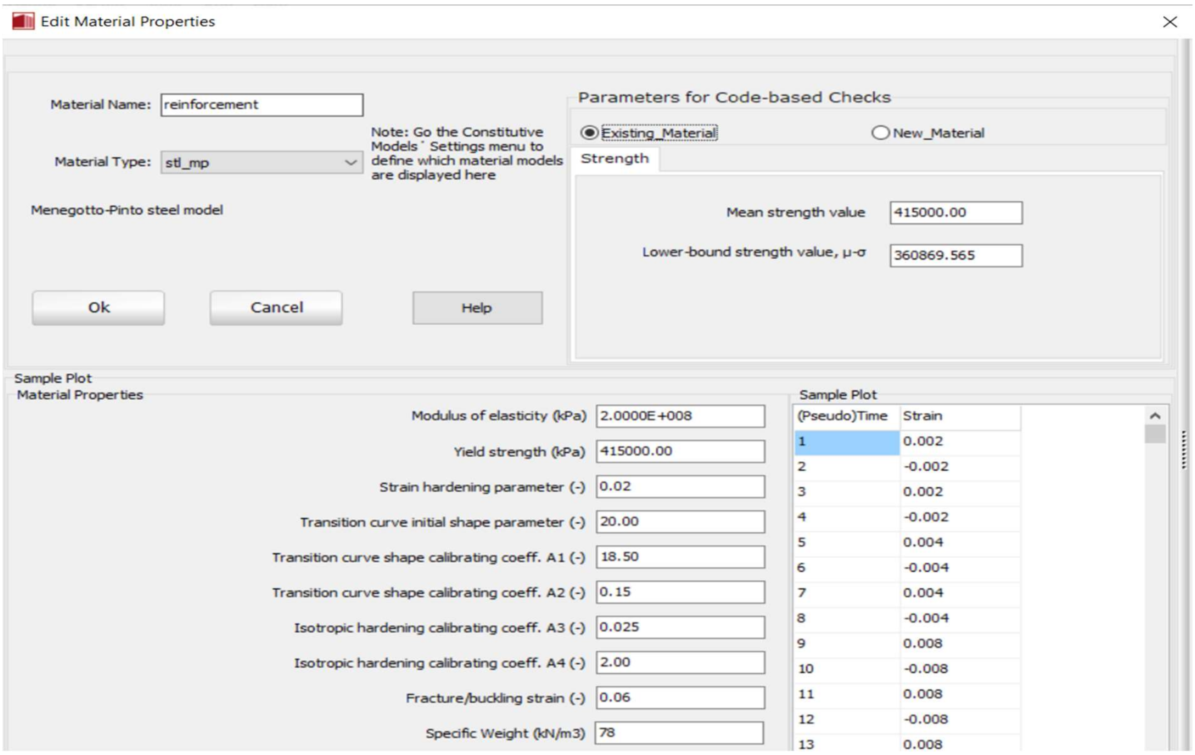

Fig. 3.16 Reinforcement properties

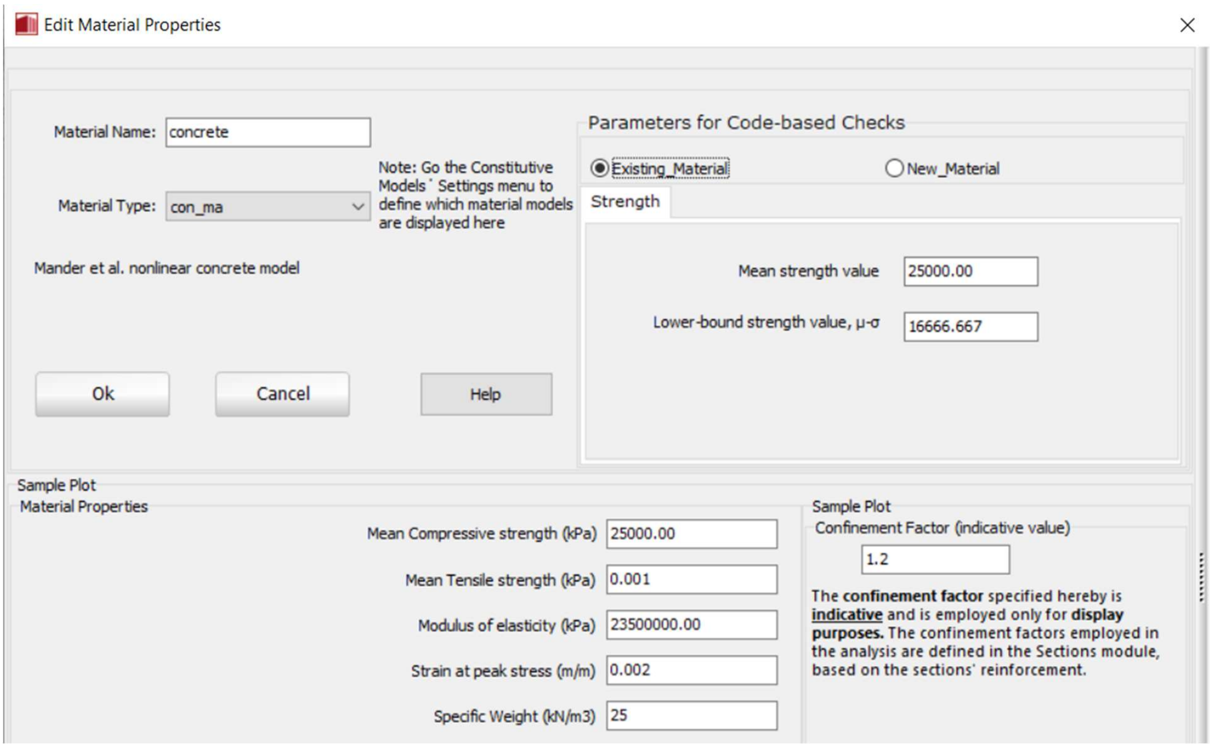

Fig. 3.17 Concrete properties

3.6.2 Section and Element classes

Reinforced concrete sections are defined for column and beam sections in SeismoStruct. Reinforced concrete rectangular sections are used for columns and reinforced concrete T-sections are used for beams throughout the model. The longitudinal reinforcement for columns are corners (4@20mm), top & bottom (4@20mm) and left & right (4@20mm); and transverse reinforcement is 10mm @ 100mm c/c. The longitudinal reinforcement for beams are lower (4@14mm), upper (5@14mm), lower flange (2@6mm) and upper flange (2@6mm); and transverse reinforcement is 8mm @ 200mm c/c. The column and beam dimensions are shown in figure.

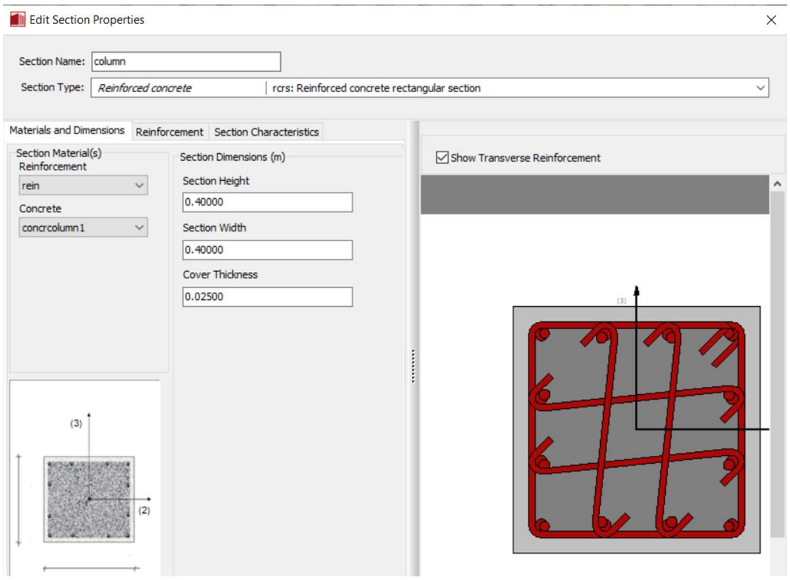

Fig. 3.18 Column section properties

Inelastic force-based frame elements are used for both beams and columns. Infill having specific weight of $10kN/m^3$ is considered to model the exterior walls. The bottom nodes are restrained and then the Eigen value analysis is done to compute the vital parameters of the structure.

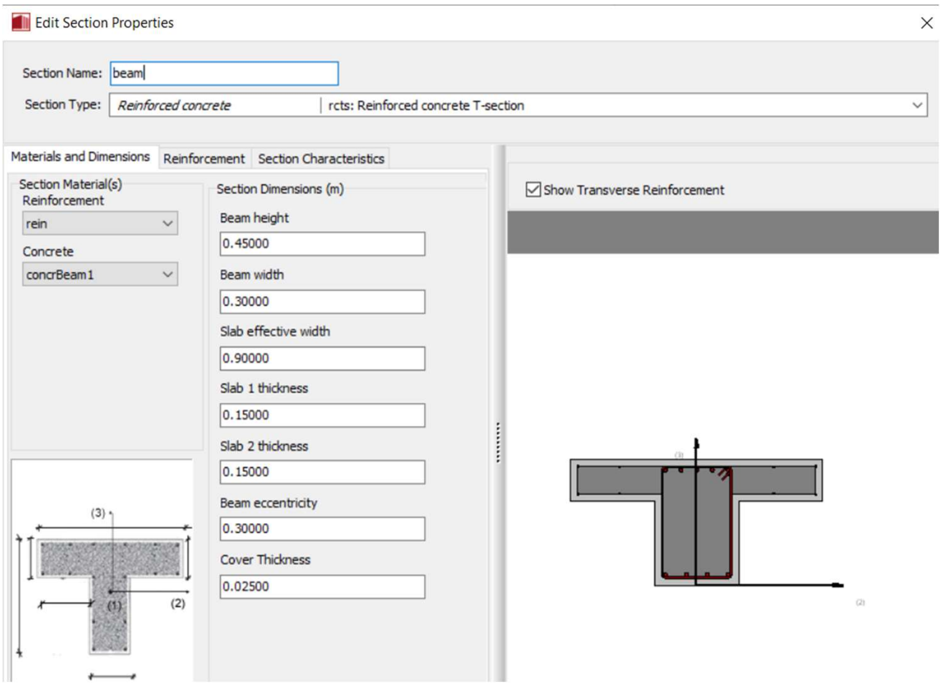

Fig. 3.19 Beam section properties

The discretization of the column and beam members is done for the fibres of the model. Each section is divided into a number of areas. In this case, number of monitoring points are taken 200 for each element.

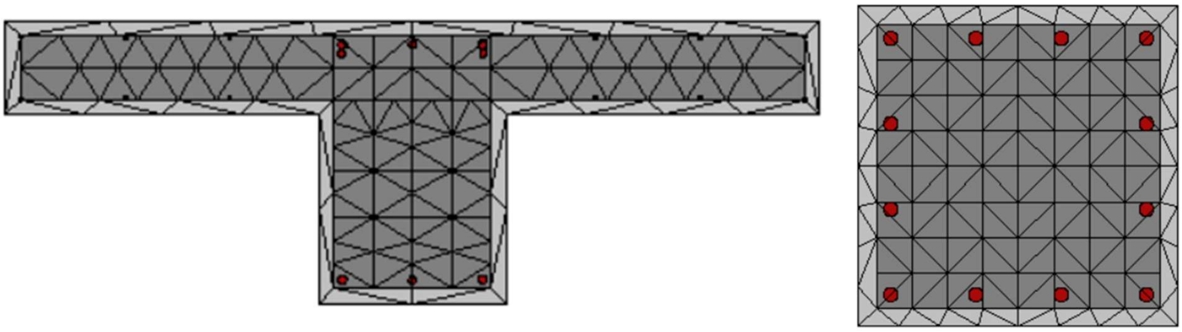

Fig. 3.20 Discretization of beams and columns

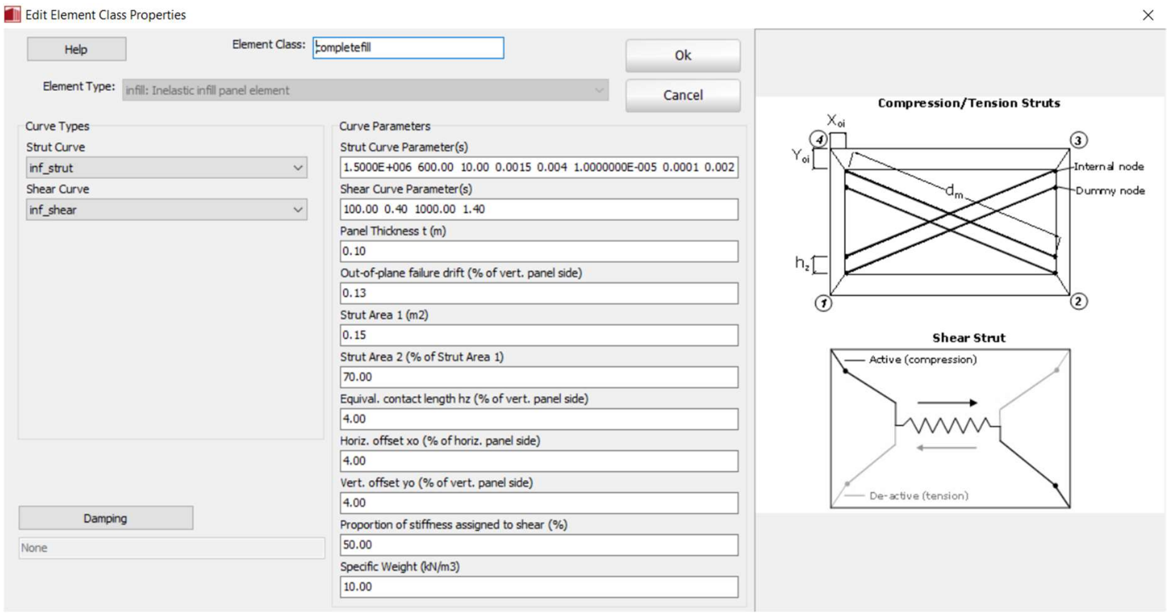

Fig. 3.21 Infill properties

3.6.3 Results and Discussion

The Eigen value results from this study are verified with the case study and the values of fundamental period, frequency and angular frequency of all the modes matches with it. So with this, it is stated that SeismoStruct is reliable and accurate software to analyze different kinds of analysis. After the verification of structures, the other objectives of the study are achieved by comparing the analysis values with other approaches.

Analysis Logs | Modal/Mass Quantities | Step Output | Deformed Shape Viewer

Modal Periods and Frequencies | Nodal Masses

M O D A L   P E R I O D S   A N D   F R E Q U E N C I E S

Mode	Period (sec)	Frequency (Hertz)	Angular Frequency (rad/sec)
1	2.19477061	0.45562848	2.86279818
2	0.35073214	2.85117870	17.91448409
3	0.29272215	3.41620891	21.46467360
4	0.27897844	3.58450642	22.52211809
5	0.12600016	7.93649788	49.86648688
6	0.09861487	10.14045827	63.71437838
7	0.09459091	10.57183983	66.42482866
8	0.06828141	14.64527361	92.01896793
9	0.05930873	16.86092440	105.94031246
10	0.05567077	17.96274866	112.86327845

Fig. 3.22 Fundamental modes and frequencies

SeismoStruct gives two options of eigensolvers. In this study, the Lanczos algorithm is used to compute the results. The natural mode with period 2.19s is the fundamental mode of the building.

M O D A L   P A R T I C I P A T I O N   F A C T O R S

For Unit Acceleration Loads in Global Coordinates

Mode	Period	[Ux]	[Uy]	[Uz]	[Rx]	[Ry]	[Rz]
1	2.19477061	0.0000	16.2647	0.0000	-35.8066	0.0000	0.1177
2	0.35073214	0.0049	-8.9231	0.0001	-42.9614	0.0031	-3.9890
3	0.29272215	0.3321	0.3675	-0.0028	1.3287	0.3710	-92.8701
4	0.27897844	-18.2510	0.0045	0.0785	0.0211	-23.50057	-1.6447
5	0.12600016	0.0000	-4.7926	-0.0006	-28.1082	-0.0116	-0.9060
6	0.09861487	-0.1036	-0.1981	-0.0093	-1.9845	1.2421	29.4171
7	0.09459091	-5.6200	0.0033	-0.2986	0.0171	60.7055	-0.6138
8	0.06828141	0.0017	2.3762	-0.0017	14.8525	-0.0096	0.6655
9	0.05930873	0.0527	0.1784	-0.0155	0.4344	-0.2513	-14.8686
10	0.05567077	0.1356	-0.0001	10.0021	-0.0017	7.6292	0.0096

E F F E C T I V E   M O D A L   M A S S E S

[Individual Mode]

Mode	Period	[Ux]	[Uy]	[Uz]	[Rx]	[Ry]	[Rz]
1	2.19477061	0.000000	264.540278	0.000000	1282.112285	0.000000	0.013862
2	0.35073214	0.000024	79.622280	0.000000	1845.684039	0.000010	15.912485
3	0.29272215	0.110308	0.135047	0.000008	1.765357	0.137663	8624.851201
4	0.27897844	333.098349	0.000020	0.006168	0.000445	529.260850	2.705117
5	0.12600016	0.000000	22.969416	0.000000	790.072277	0.000135	0.820842
6	0.09861487	0.010733	0.039248	0.000087	3.938179	1.542864	865.363257
7	0.09459091	31.584680	0.000011	0.089172	0.000292	3685.159661	0.376803
8	0.06828141	0.000003	5.646158	0.000003	220.596581	0.000092	0.442842
9	0.05930873	0.002775	0.031831	0.000241	0.188692	0.063142	221.075930
10	0.05567077	0.018377	0.000000	100.043002	0.000003	58.204429	0.000092

Fig. 3.23 Modal participation factors and modal masses

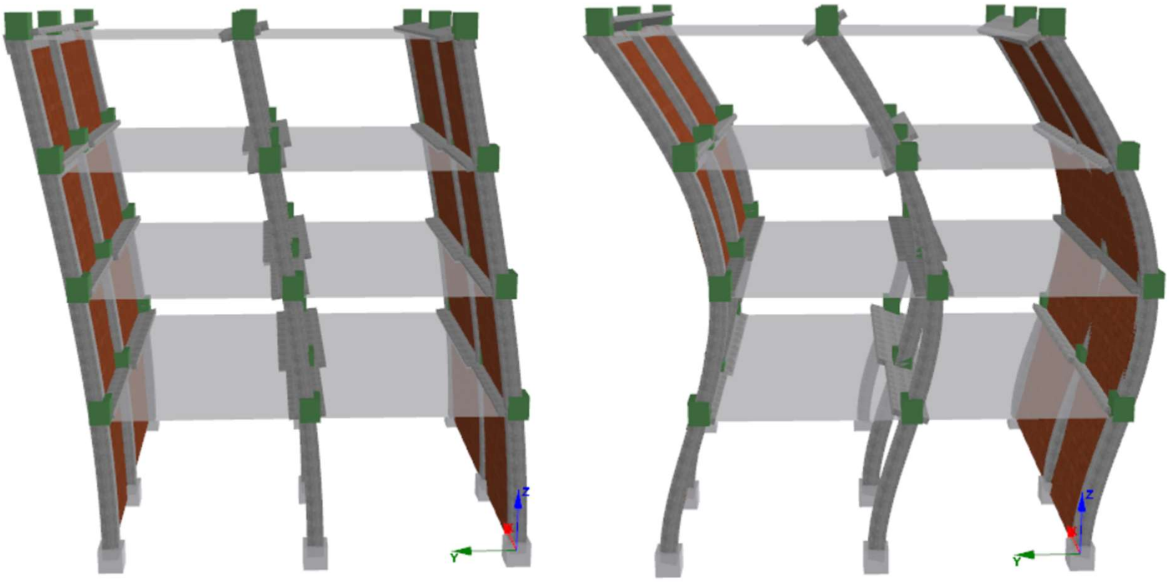

Fig. 3.24 Deformed shapes for mode 1 and mode 2

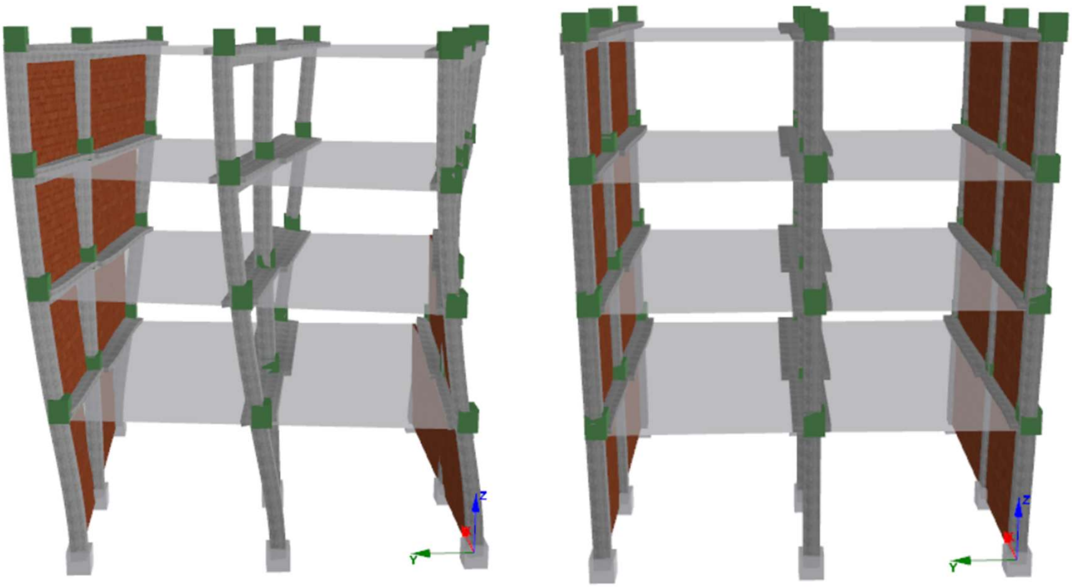

Fig. 3.25 Deformed shapes for mode 2 and mode 3

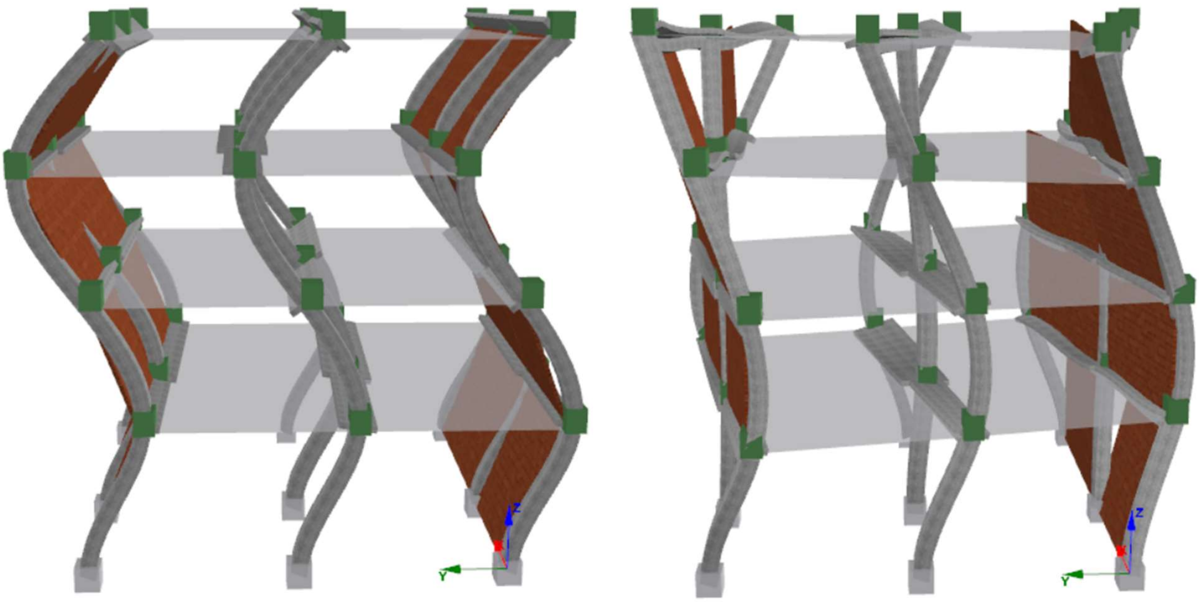

Fig. 3.26 Deformed shapes for mode 5 and mode 6

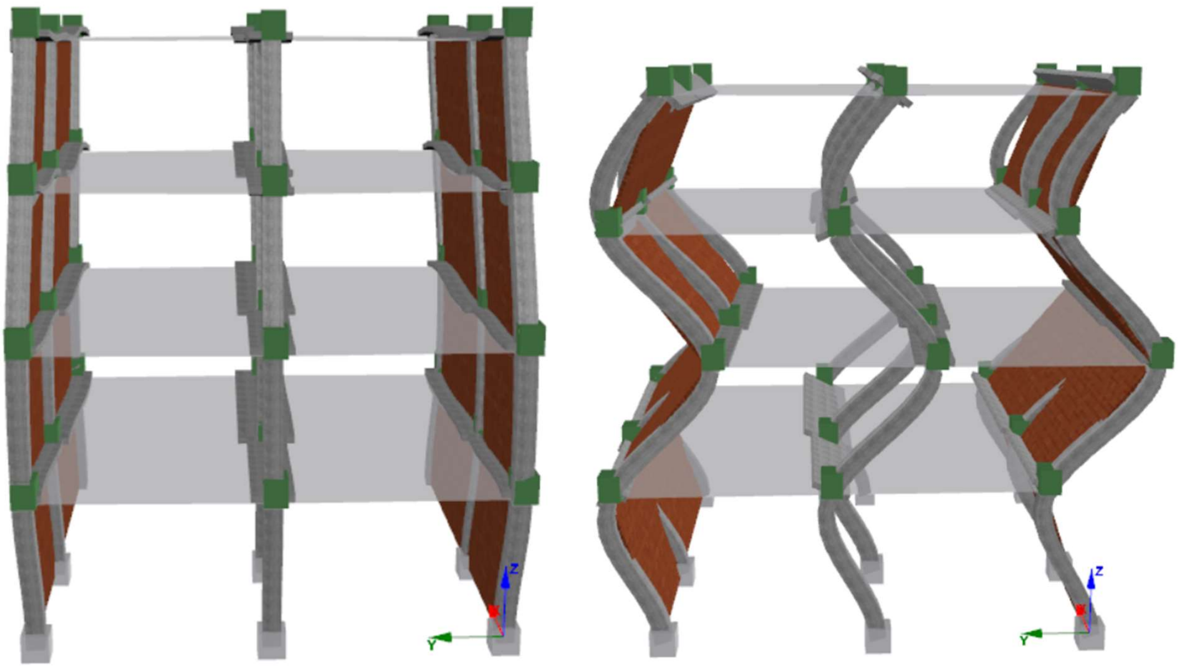

Fig. 3.27 Deformed shapes for mode 7 and mode 8

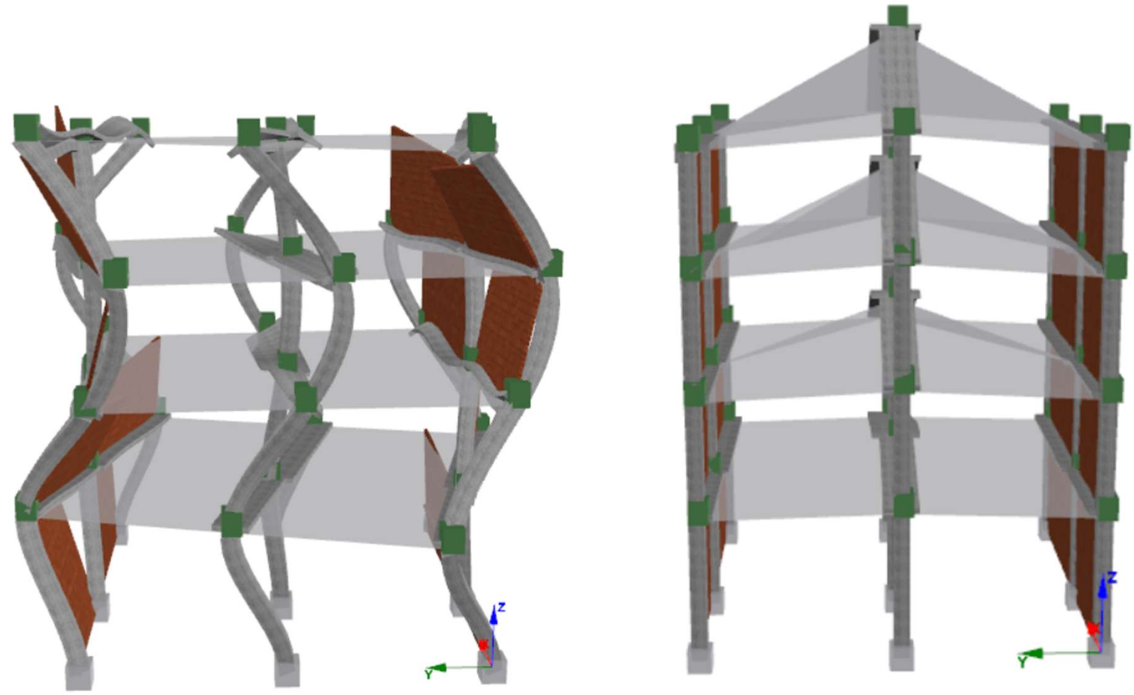

Fig. 3.28 Deformed shapes for mode 9 and mode 10

CHAPTER 4

INVESTIGATION ON NATURAL FREQUENCY

4.1 GENERAL

In general, the building is designed as per codal provisions, which has various constraints while analysing with dynamic loads. As, the process is time-consuming, most of the Civil Engineering structures are designed taking the assumption of applied loading to be static. The process of neglecting the dynamic forces may become the cause of collapse of the structure as a whole in case of a catastrophe such as an earthquake. Some recent earthquakes have shown the need for dynamic analysis. There are many empirical relationships available in seismic codes of different countries, which relate the height of the structure with the natural period of oscillation. In this study, the fundamental periods of a case study of a building are investigated using the Equivalent Static Method and Response Spectrum Method using IS 1893(Part 1) and finite element modelling in SeismoStruct.

4.2 EQUIVALENT STATIC METHOD (IS 1893:2016)

IS 1893(Part 1): 2016 [23] provides the guidelines and provisions for earthquake resistant design. It adopts equivalent static method to perform linear static analysis. This is the simplest and easiest method of analysis, which requires less computational effort. The fundamental period of vibration can be estimated by various expressions. Clause 7.6.2 give some empirical expressions to calculate the natural period.

For Reinforced Concrete frames without infills,

$$Ta = 0.075\ h^{0.75}\ sec \tag{1}$$

For Steel constructed frames without infills,

$$Ta = 0.080\ h^{0.75}\ sec \tag{2}$$

For all other buildings with infills

$$Ta = \frac{0.09h}{\sqrt{d}}\ sec \tag{3}$$

Where

h = height (in metre) excluding basement storey, when ground floor deck is connected with walls but including basement storeys when there is no connection between the two.

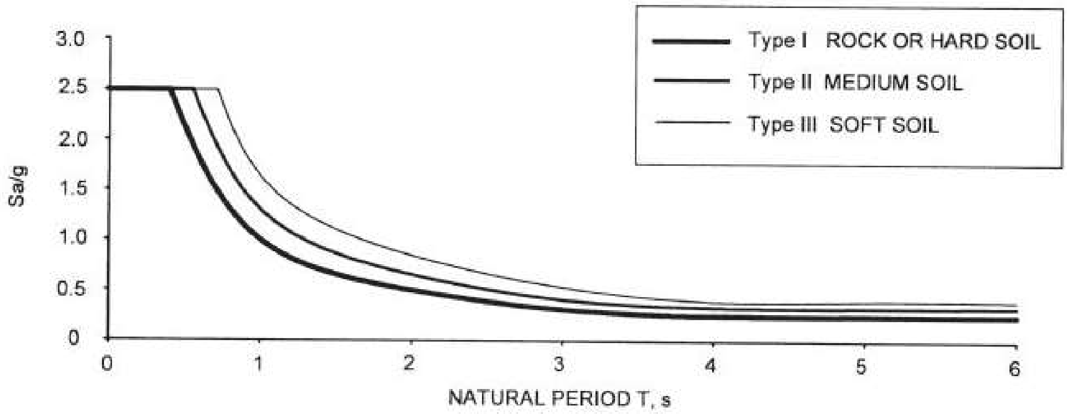

Fig. 4.1 Design Spectra for Equivalent Static Method

4.3 RESPONSE SPECTRUM METHOD (IS 1893:2016)

IS 1893 recommends the use of response spectrum and time history analysis to perform linear dynamic analysis. This technique is appropriate for such cases where other modes besides the fundamental mode influence the seismic response of the building. The multi degree of freedom system is idealized in terms of single degree of freedom system having lumped mass at each level. In this procedure, the mass of the building is lumped at every storey. The time period depends upon stiffness and mass of the structure, so the code specifies the use of dynamic analysis which requires other periods and shapes of natural modes. In this procedure, mass matrix and stiffness matrix are calculated for equivalent model and using these matrices, an eigenvalue problem is formulated to calculate the natural frequencies and Eigen values using the following equation

$$|K- \omega^2 M| = 0 \tag{4}$$

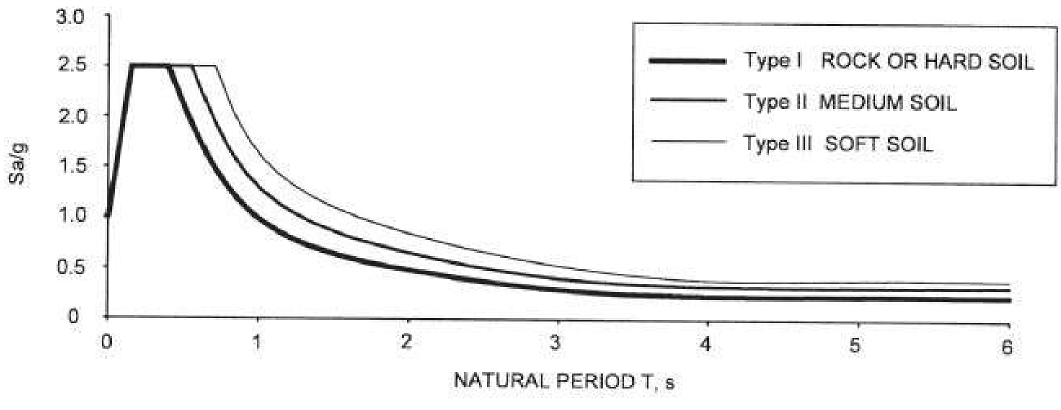

Fig. 4.2 Design Spectra for Response Spectrum Method

4.4 CASE STUDY

The case study [24] chosen for the current investigation is an existing low-rise residential building located in Zone V. The building is selected from a set of previous studies on nonlinear static analysis. The natural period of low-rise RC frame structure is examined of the building frame having regular plan, consisting of columns and beams. The building is symmetric along X and Y-axes having plan dimensions 50m × 8m and floors having same height of 3.1m. The building parameters of the study are as follows and the elevation, lumped mass model and plan of the building are shown in figure 1, 2 and 3.

Table 4.1 Building Parameters from case study [18]

Structure type	Moment resisting frame
Number of stories	Three , (G+2)
Height of floor	3.1 m
Materials	Concrete (M 25) and Reinforcement (Fe415)
Live load	$3kN/m^3$
Size of columns	500mm X 500 mm
Size of beams	400mm X 500 mm
Specific weight of RCC	$25\ kN/m^3$

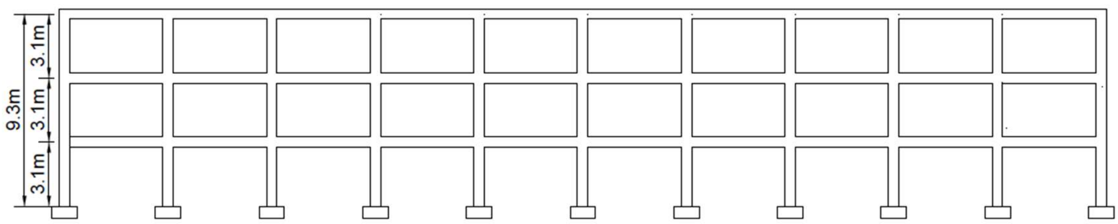

Fig. 4.3 Building Elevation

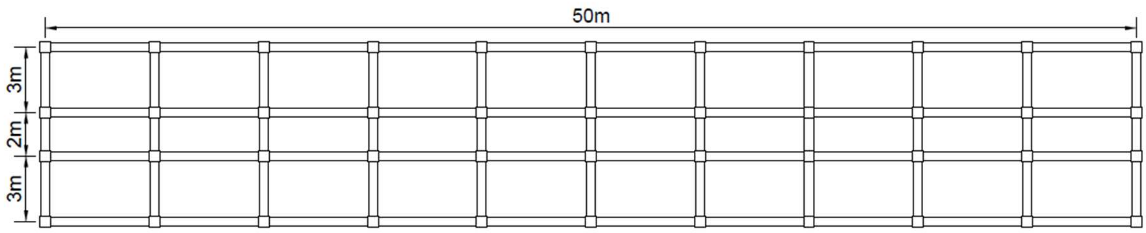

Fig. 4.4 Building Plan

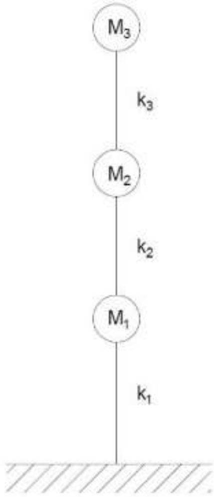

Fig. 4.5 Lumped mass model

4.5 CODE BASED SEISMIC ANALYSIS

The analysis is performed by two processes: equivalent static method and response spectrum method. Putting the value of height in Equation 1, the fundamental period of the structure can be found out by equivalent static method. To compute the fundamental period by response spectrum method, the seismic weights, the lumped masses and lateral stiffness for each floor level is calculated.

Table 4.2 Calculated results of seismic analysis

Floor level	Seismic Weights(W) kN	Lumped mass (M) kg	Lateral stiffness (k) N/m
1	2592.5	264271	2307743950
2	2592.5	264271	2307743950
3	1866.25	190239	2307743950

The mass (M) and stiffness (k) matrices are formed. These are as follows:

$$M = \begin{bmatrix} M1 & 0 & 0 \\ 0 & M2 & 0 \\ 0 & 0 & M3 \end{bmatrix} = \begin{bmatrix} 264271 & 0 & 0 \\ 0 & 264271 & 0 \\ 0 & 0 & 190239 \end{bmatrix}$$

$$K = \begin{bmatrix} k1 + k2 & -k2 & 0 \\ -k2 & k2 + k3 & -k3 \\ 0 & -k3 & k3 \end{bmatrix} = \begin{bmatrix} 4615487900 & -2307743950 & 0 \\ -2307743950 & 461548790 & -2307743950 \\ 0 & -2307743950 & 2307743950 \end{bmatrix}$$

Solving equation (4) the natural frequency, time period and Eigen values are calculated.

$$\omega^2 = \begin{bmatrix} 2032.131 & 0 & 0 \\ 0 & 15362.206 & 0 \\ 0 & 0 & 29668.915 \end{bmatrix}$$

Frome the above values, the Eigen vectors are computed

$$\varphi = \begin{bmatrix} 0.471 & -1.105 & 1.035 \\ 0.832 & -0.266 & -1.446 \\ 1 & 1 & 1 \end{bmatrix}$$

The modal mass M_k are calculated as per IS 1893(Part 1): 2016 using equation

$$M_k = \frac{[\sum_{i=1}^{n} W_i \varphi_{ik}]2}{g[\sum_{i=1}^{n} W_i \varphi_{ik}]2} \tag{5}$$

Using the seismic weights of each floor and Eigen vectors a table is formed and using that table the modal mass is estimated.

Table 4.3 Analysis values of response spectrum method

Level	W_i	φ_{ik}	$W_i\varphi_{ik}$	$W_i(\varphi_{ik})^2$	φ_{ik}	$W_i\varphi_{ik}$	$W_i(\varphi_{ik})^2$	φ_{ik}	$W_i\varphi_{ik}$	$W_i(\varphi_{ik})^2$
3	1866.25	1	1866.25	1866.25	1	1866.25	1866.25	1	1866.25	1866.25
2	2592.5	0.83	2156.96	1794.59	-0.26	-689.6	183.43	-1.44	-3748.7	5420.6
1	2592.5	0.47	1221.07	575.12	-1.11	-2863.6	3163.21	1.03	2683.2	2777.2

4.6 FINITE ELEMENT ANALYSIS IN SEISMOSTRUCT

Building has been idealized as three-dimensional space frame using two node frame elements in SeismoStruct. The 3D model is shown in the figure having plan dimensions 50m x 8m. In this study, the Jacobi algorithm is used to compute the Eigen values.

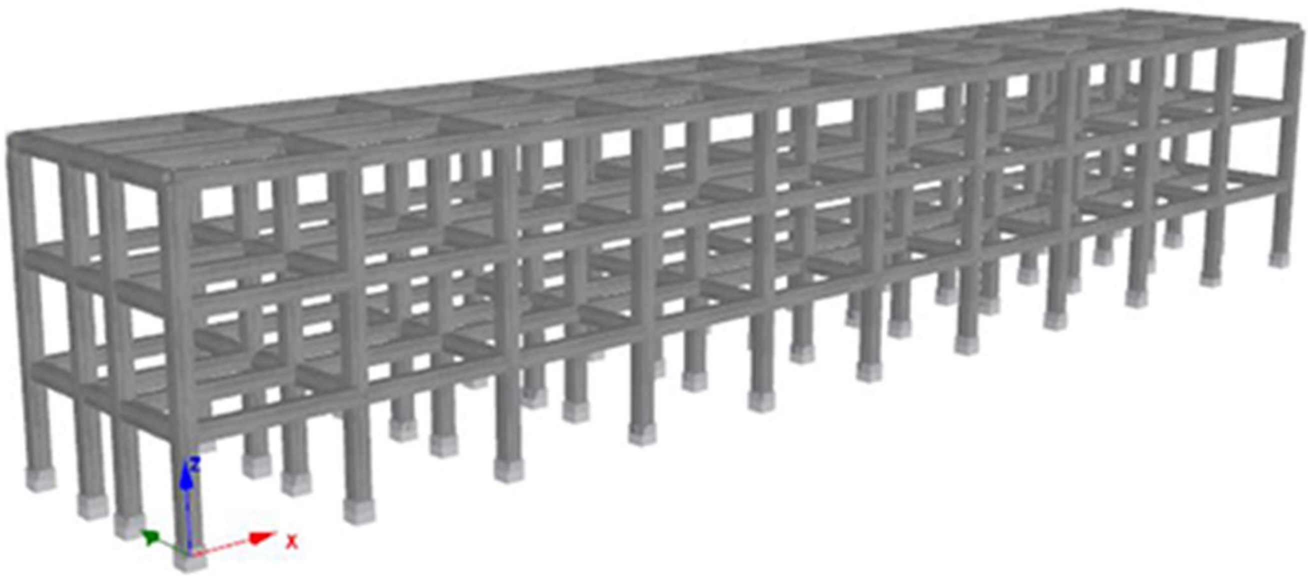

Fig. 4.6 3D model

4.6.1 Material Specifications

Mander et al. nonlinear concrete model have been used for M25 grade of concrete and Menegotto and Pinto steel model is used for steel reinforcements in this case study. The material properties are same as in previous chapter. The stress strain relationships of both the models are shown in the figure.

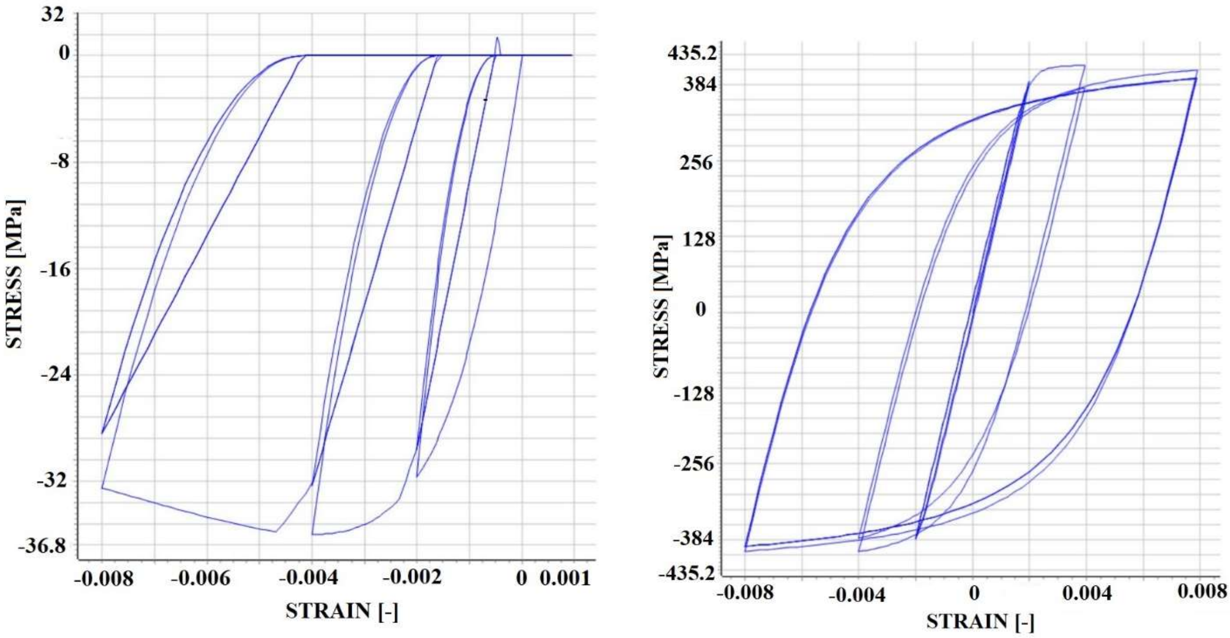

Fig. 4.7 Constitutive relationships for concrete and reinforcement steel.

4.6.2 Section and Element classes

Reinforced concrete sections are defined for column and beam sections in SeismoStruct. Reinforced concrete rectangular sections are used for both columns and beams throughout the model. The longitudinal reinforcement for columns are corners (4@20mm), top & bottom (2@20mm) and left & right (2@20mm); and transverse reinforcement is 10mm @ 150mm c/c. The longitudinal reinforcement for beams are corners (4@20mm) and top & bottom (4@20mm); and transverse reinforcement is 10mm @ 200mm c/c. The column and beam dimensions are shown in figure.

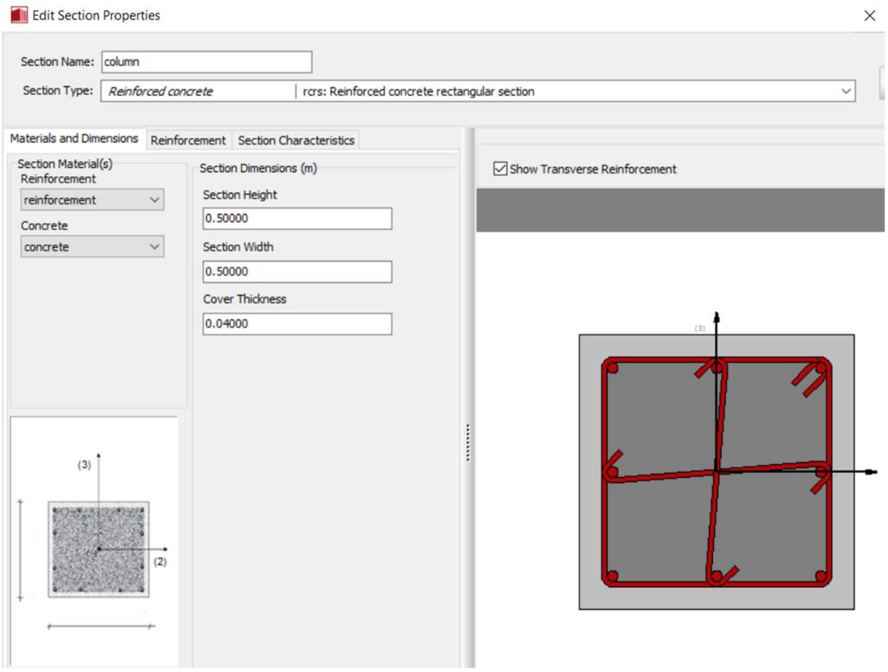

Fig. 4.8 Column section properties

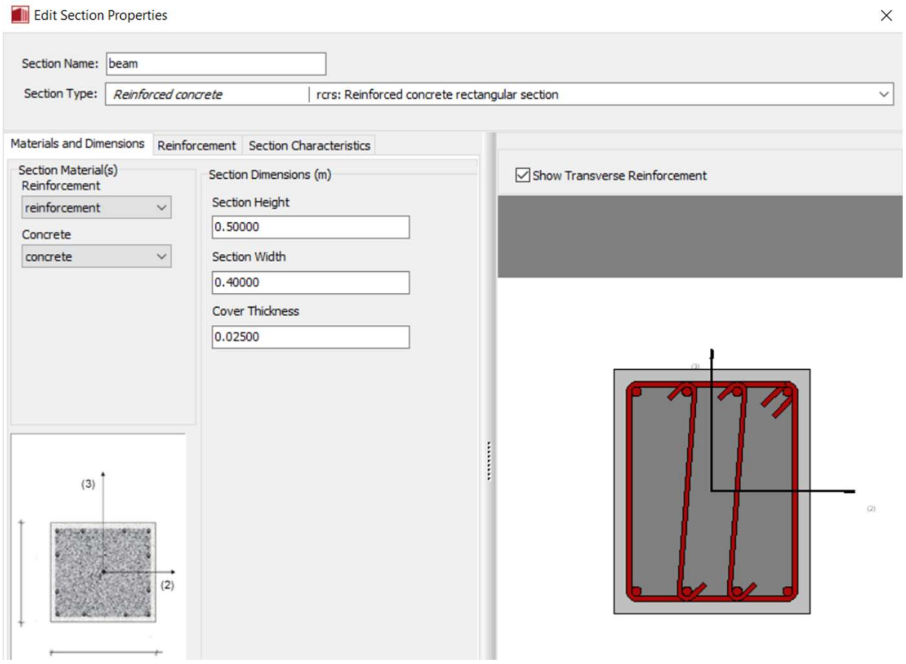

Fig. 4.9 Beam section properties

Inelastic plastic hinge force-based frame elements are used in SeismoStruct to define both beam and column elements. The bottom nodes of the building are restrained. In the Processor module, the analysis is run and then the Eigen value analysis is done. The output results are shown in the Post-Processor module.

4.7 RESULTS AND DISCUSSION

The fundamental period from both the approaches are tabulated and compared. The fundamental mode for Equivalent Static method, Response Spectrum method and numerical analyses are estimated to be 0.39s, 0.14s and 0.178s. Indian Seismic code suggests to perform Equivalent static method for natural period less than 0.4s. As the value is almost equal to 0.4s, this method is not adopted to calculate the lateral seismic forces. The fundamental frequencies and natural periods from both methods are shown as follows:

Table 4.4 Fundamental frequencies and time periods from both approaches

Modes	Response Spectrum Analysis		SeismoStruct Analysis	
	ω (rad/sec)	T (sec)	ω (rad/sec)	T (sec)
Mode 1	45.1	0.140	33.42	0.178
Mode 2	124. 21	0.051	35.62	0.176
Mode 3	172.24	0.036	40.50	0.155
Mode 4	-	-	51.53	0.122
Mode 5	-	-	102.77	0.0611
Mode 6	-	-	105.60	0.0595
Mode 7	-	-	108.89	0.0577
Mode 8	-	-	145.91	0.0431
Mode 9	-	-	178.44	0.0352
Mode 10	-	-	181.40	0.0346
Mode 11	-	-	294.33	0.021
Mode 12	-	-	349.62	0.018

SeismoStruct uses Jacobi algorithm with Ritz transformation, to solve the Eigen values so 12 modes are generated whereas in Response Spectrum analysis three modes are generated as the multi-degree freedom system has been converted into finite degree of freedom system. The shapes of modes of first 3 modes have been plotted and the comparison shows that the first mode has similar shape in both cases.

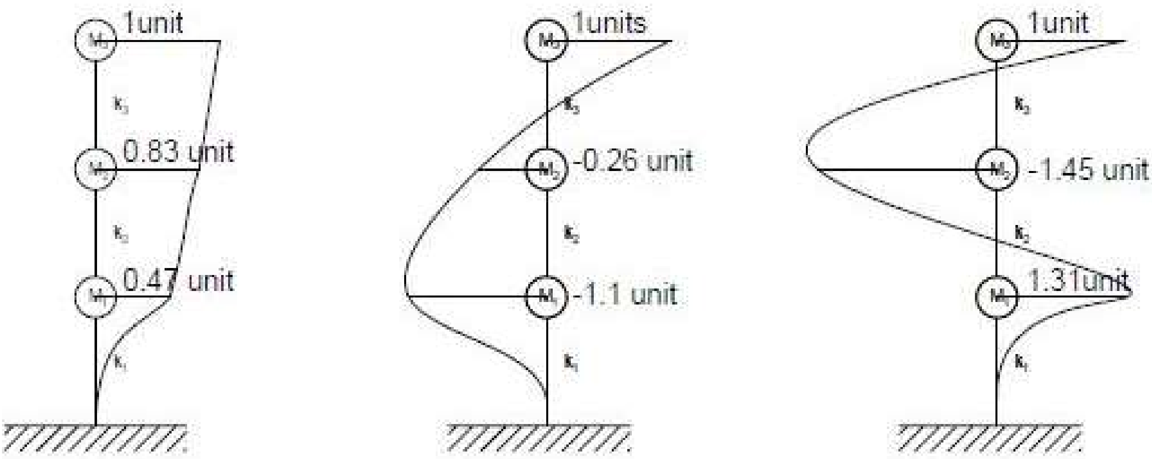

Fig. 4.10 Mode shapes from Response Spectrum method

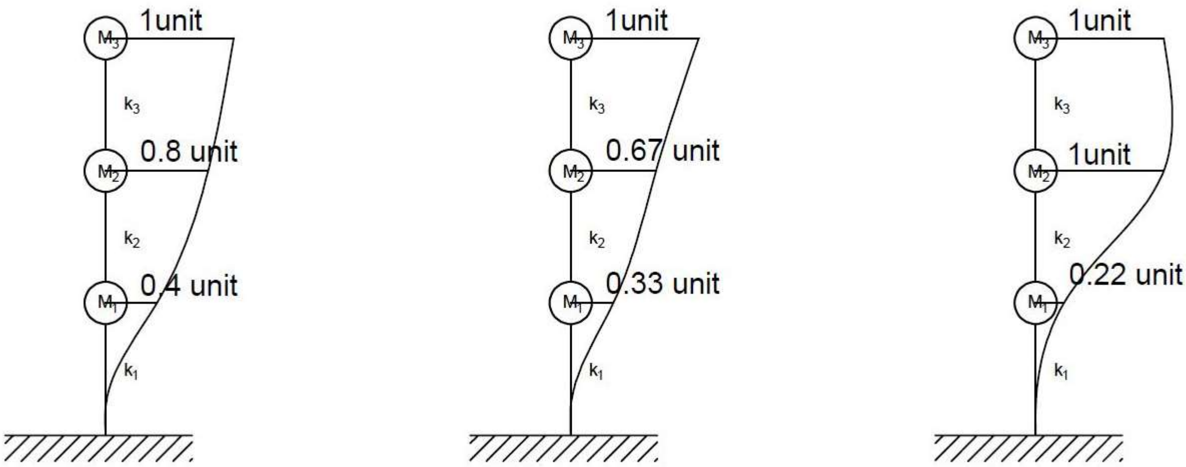

Fig. 4.11 Mode shapes from SeismoStruct

The fundamental frequencies and modal mass of the three modes are compared and relative error can be computed. The relative error in range (1% -30%). The reason behind this variation is that IS 1893 is based on assumptions and has many limitations. IS 1893 states that the fundamental mode dominates the response of the structure whereas ground motions are complex having several frequencies. The material elasticity and structural rigidity with lumped mass are considered in Response Spectrum method and in SeismoStruct, the material non-linearity is considered. The study proved that SeismoStruct provides more generalised and realistic results of fundamental period.

Table 4.5 Comparison of natural frequencies

S.N	ω(Response Spectrum)			ω(SeismoStruct)		
	ω_1	ω_2	ω_3	ω_1	ω_2	ω_3
1	45.1	124.41	172.24	33.4	102.77	178.4

Table 4.6 Comparison of modal mass

S.N	M(Response Spectrum)			M(SeismoStruct)		
	M_1	M_2	M_3	M_1	M_2	M_3
1	91%	7.6%	2.89%	86%	10.84%	3.19%

CHAPTER 5
COMPARATIVE STUDY ON NON LINEAR STATIC ANALYSIS

5.1 GENERAL

Research and development in the field of earthquake resistant design has put emphasis on non-linear analysis methods to estimate seismic demands. Nonlinear time history and nonlinear static pushover analysis are the main methods. In this chapter, pushover analysis is carried out on multi-story reinforced residential concrete building in India. A non-linear structure is taken and with the help of two modern finite element programs, pushover analysis is performed. For SAP2000, a lumped plasticity model is taken and fibre based finite elements are established in SeismoStruct to determine the plastic behaviour of the vulnerable parts of the structure. A comparison between the results obtained from the two computer programs is presented in the study.

5.2 PUSHOVER ANALYSIS

Due to time consuming computational in nonlinear dynamic analysis, nonlinear static pushover analysis is preferred by researchers and designers all across the globe. Some of the codes such as ATC 40, FEMA 273, FEMA 356 and ASCE 41 recommend the use of NSP. Nonlinear force-deformation curves are obtained from pushover analysis, which helps in estimation of seismic performance of the structure.

Pushover analysis is a non-linear static practice relating to the implementation of increasing lateral forces or motions to a non-linear construction model. Each factor of the structure's nonlinear load-deformation relationship is modelled separately. In this procedure, the nonlinear effects are simulated and the structure moved to the end of a collapse process. Pushover curve is formed by plotting the shear versus displacement at each step.

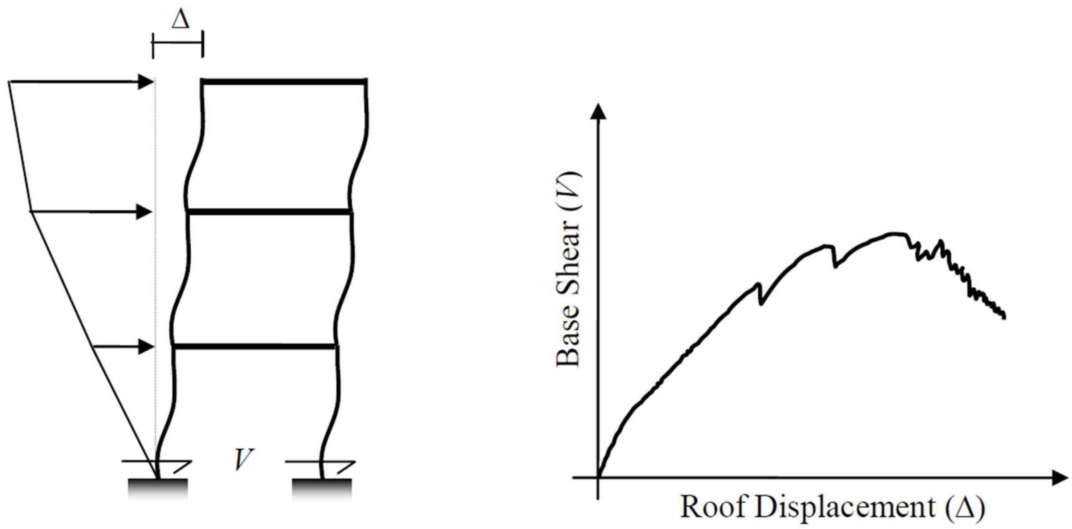

Fig. 5.1 Pushover analysis approach

5.3 STRUCTURAL MODELLING

A three-storey reinforced concrete building located in India is considered in this study. The details of the building are described in Chapter 4. The building is symmetric along X and Y axes having plan dimensions 50mx8m and floors having same height of 3.1m. Both geometric and material nonlinearity are incorporated into building models in both SAP2000 and SeismoStruct. Lumped plasticity model is taken in SAP2000 for the building and distributed plasticity model in SeismoStruct. For simplicity, open framed structure is considered without taking the walls and slabs in this analysis.

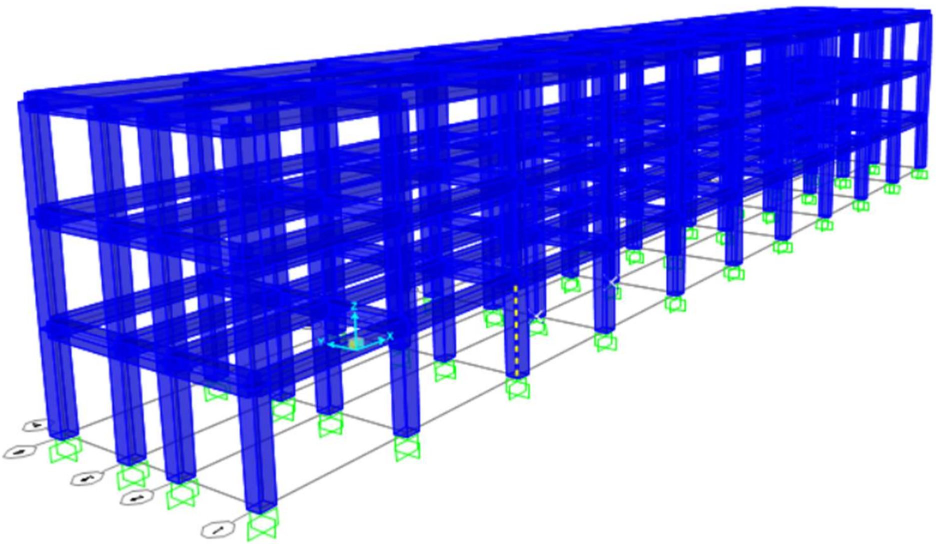

Fig. 5.2 3D model

5.4 MODELLING IN SAP2000

Modelling in SAP can be done by clicking ‘New Model’ from ‘File” dropdown menu. There are many templates present. In this study after selecting the units, 3D frames template is chosen. A new dialogue box will open and if the structure is regular, one can directly assign the values. But if the spacing is non-uniform, one can edit the grid and assign the ordinates or spacing.

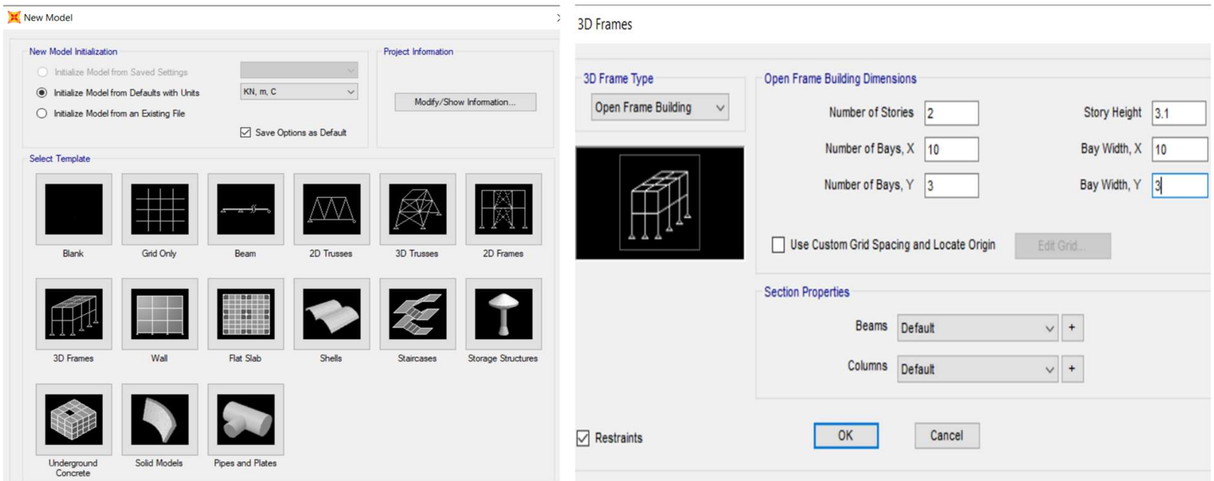

Fig. 5.3 Input table in SAP2000

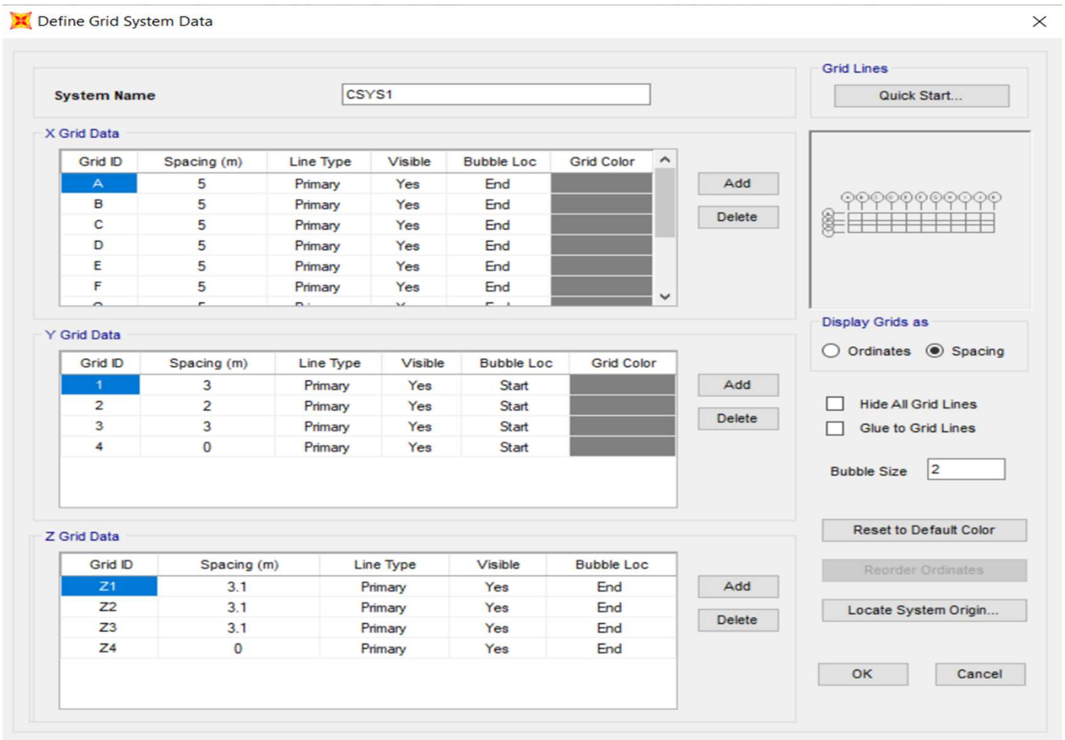

Fig. 5.4 Grid system in SAP 2000

The nonlinear material properties are assigned for M25 concrete and Fe415 steel in SAP2000. The material nonlinearity is incorporated by assuming Takeda behavior in hysteresis type for concrete and kinematic behavior for reinforcement steel. The material properties in SAP2000 are shown in figure

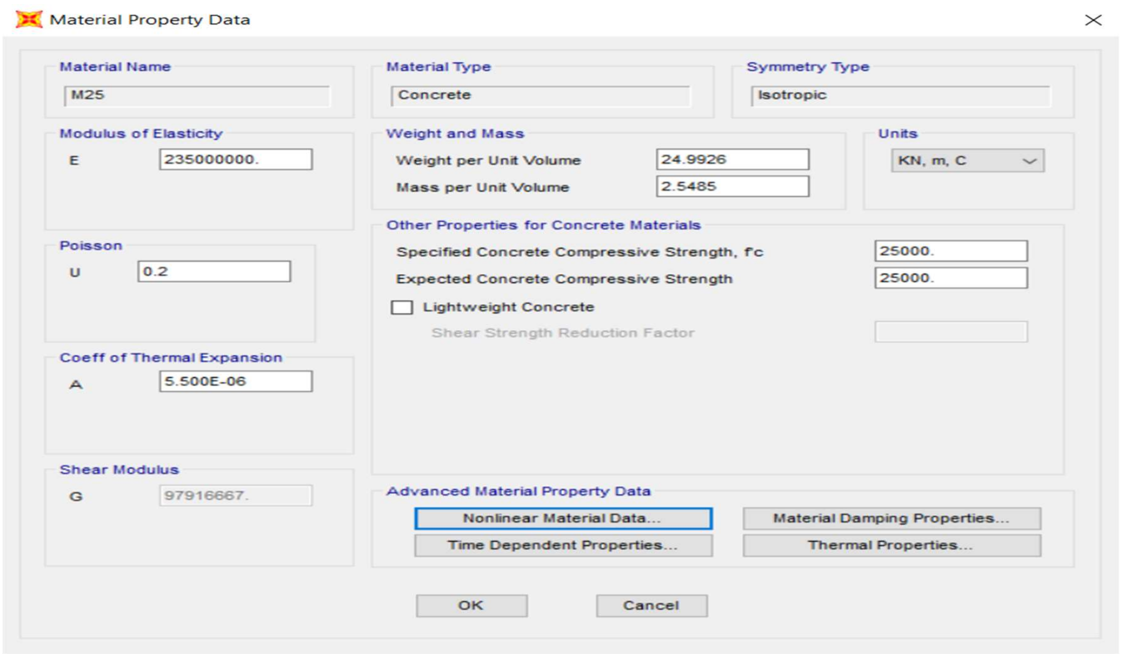

Fig. 5.5 Concrete properties

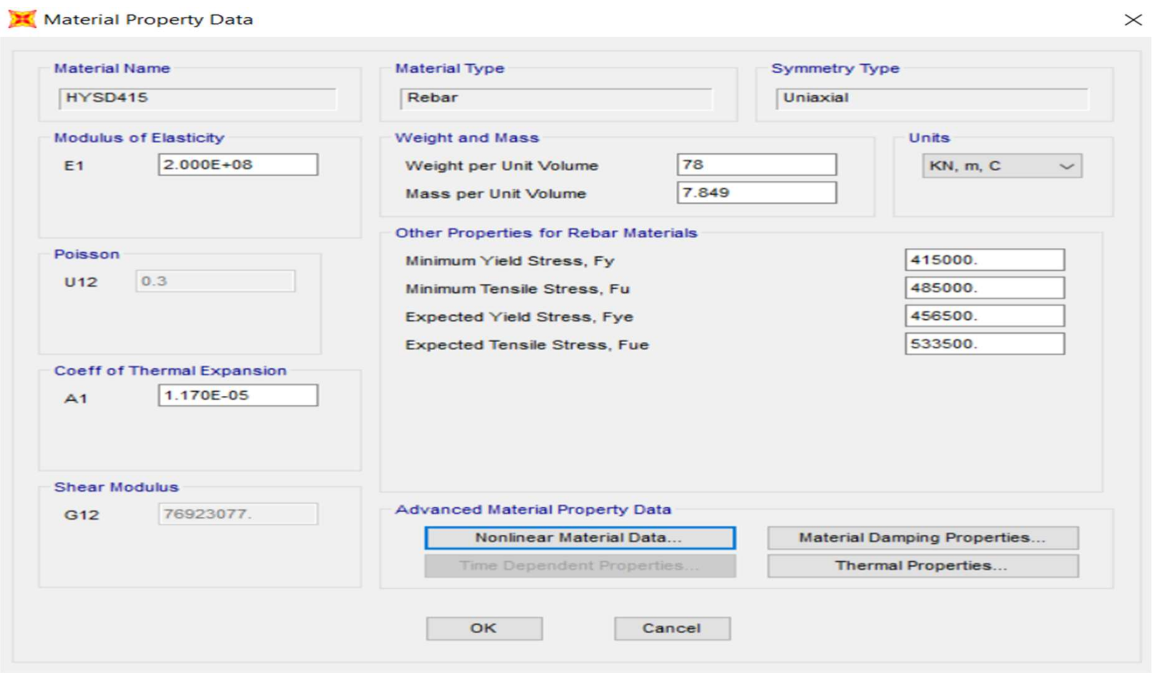

Fig. 5.6 Reinforcement properties

Reinforced concrete rectangular sections are used for both columns and beams throughout the model. Columns and beams of cross section 0.5m × 0.5m and 0.4m × 0.5m have been used to model the whole structure respectively. The details are shown in figure.

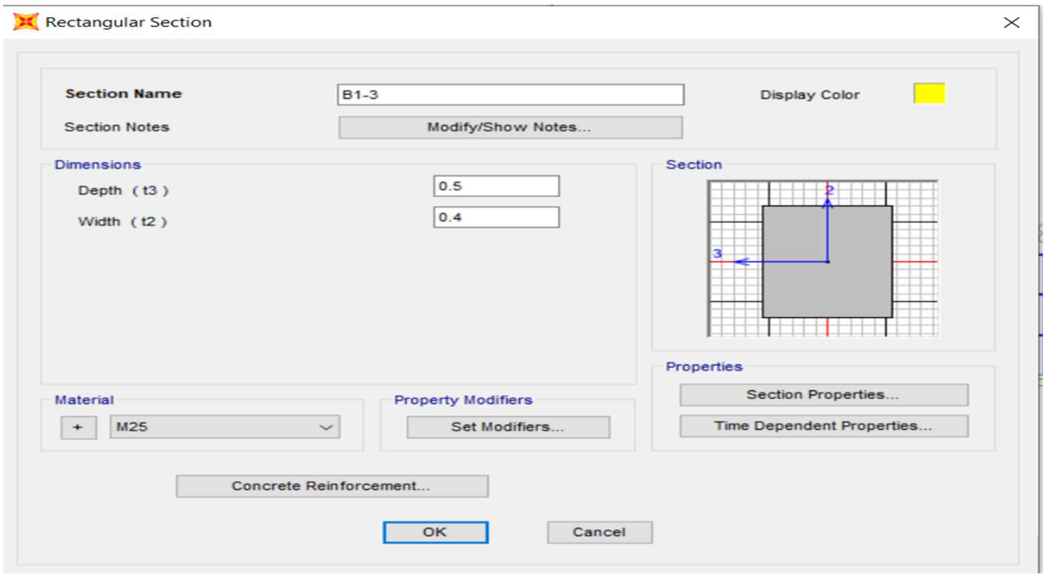

Fig. 5.7 Beam section properties

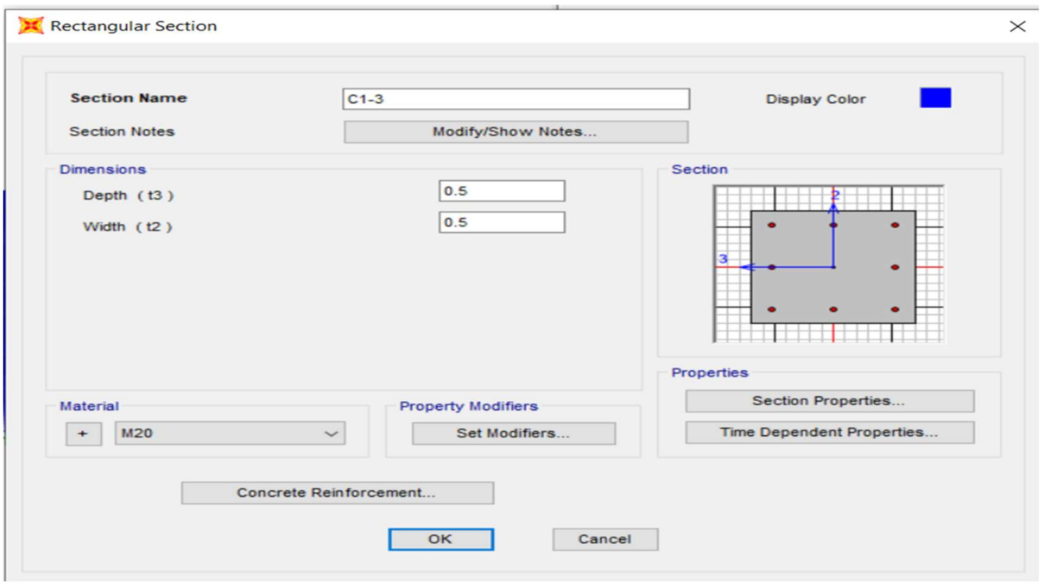

Fig. 5.8 Column section properties

Plastic hinges in this study are determined by moment curvature curves. These are established by calculating the area of cross section and details of the steel reinforcement at the feasible hinge positions. FEMA356 and ASCE41 suggests P-M2-M3 hinges for columns and M3 (flexural moment) hinges for beams. This is default values of ASCE 41-13 in SAP2000 are assumed in the study.

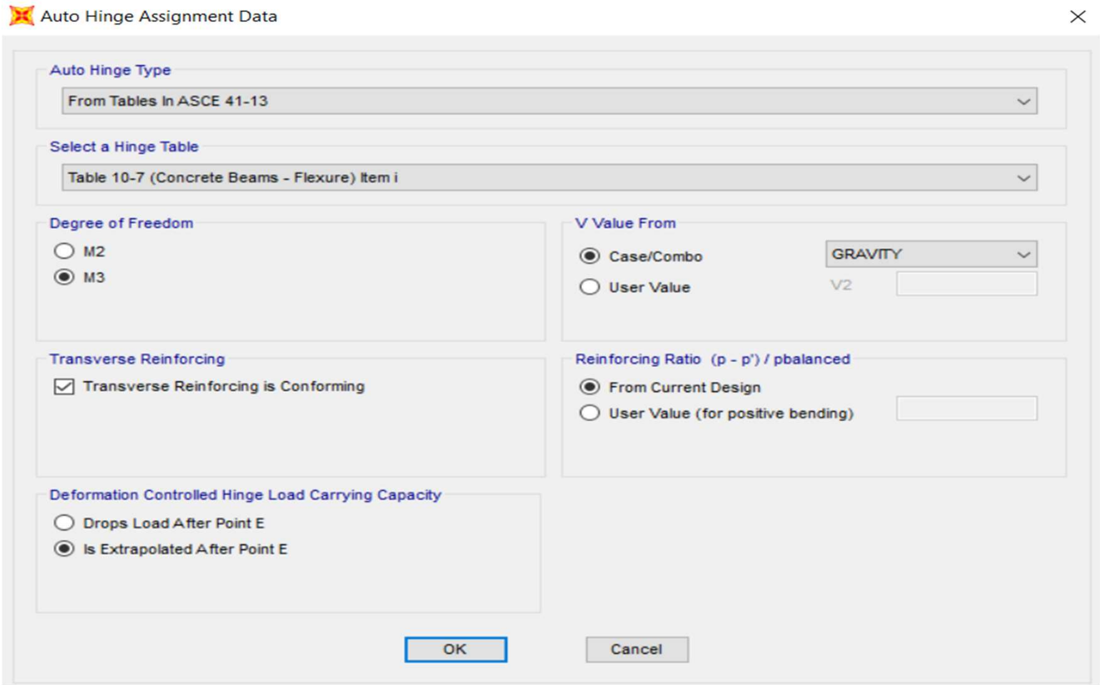

Fig. 5.9 Hinge properties for beams

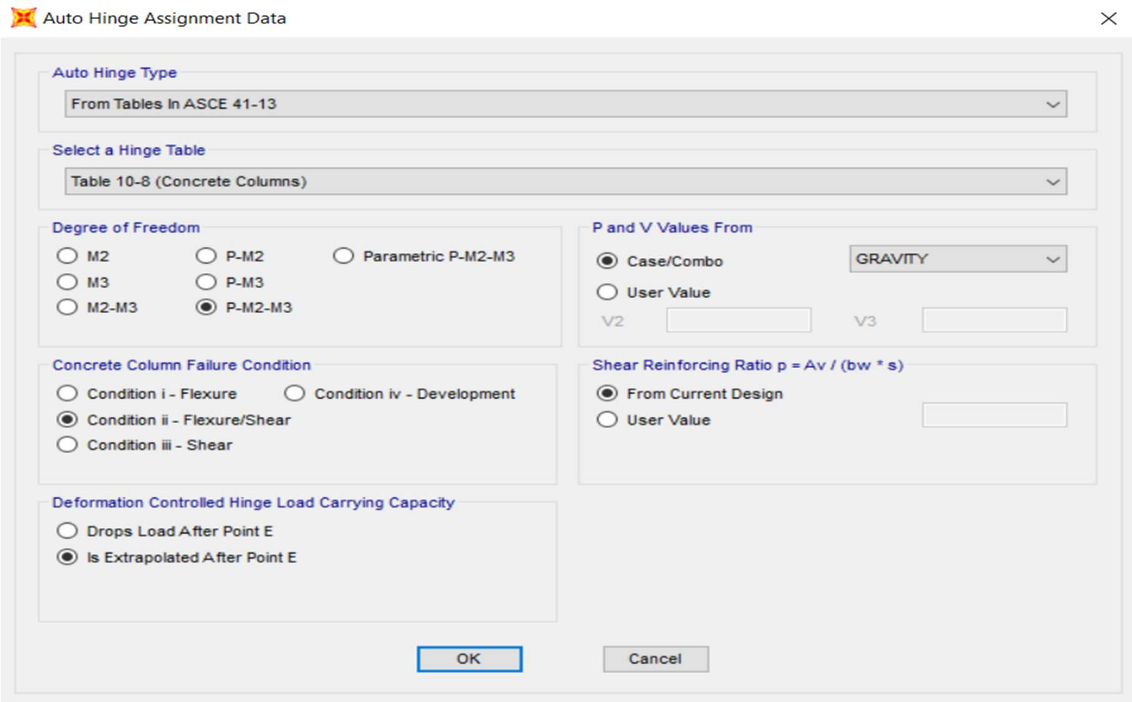

Fig. 5.10 Hinge properties for columns

5.5 MODELLING IN SEISMOSTRUCT

Modelling in SeismoStruct can be done by using 'Wizard' or 'Building Modeller' on the toolbar. 2D or 3D frames can be modelled in Wizard. By clicking on the icon, a dialogue box will open and if the structure is regular, one can directly assign the values of number of bays, height and spacing. But if the spacing is non-uniform, one can edit the grid and assign the ordinates or spacing.

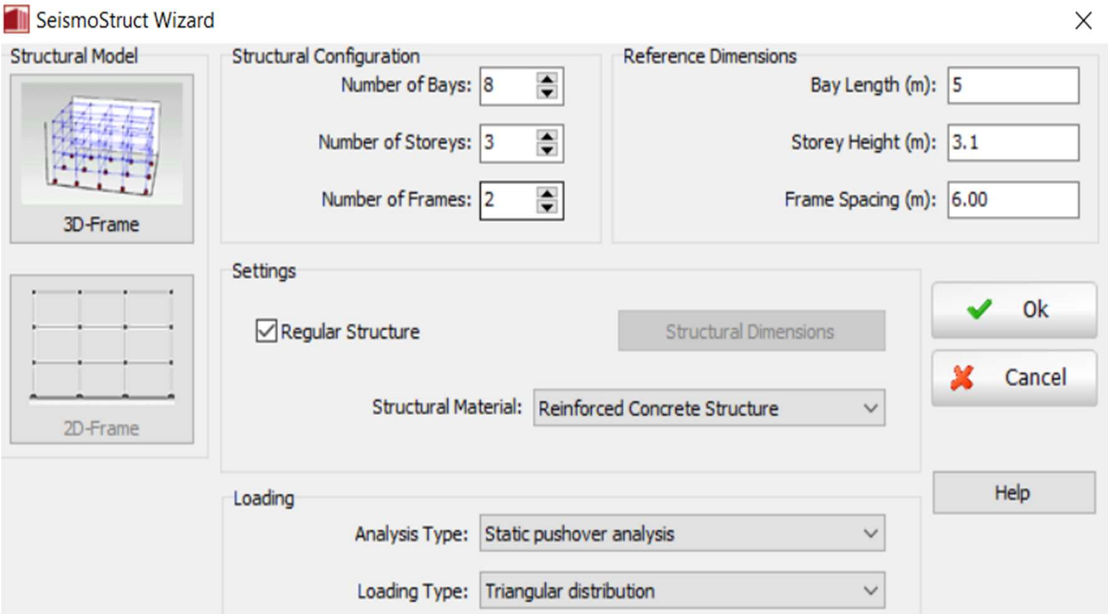

Fig. 5.11 SeismoStruct wizard

Building Modeller can also be used to model the building. All the frame elements, load combinations, analysis type and other parameters can selected from the dialog box.

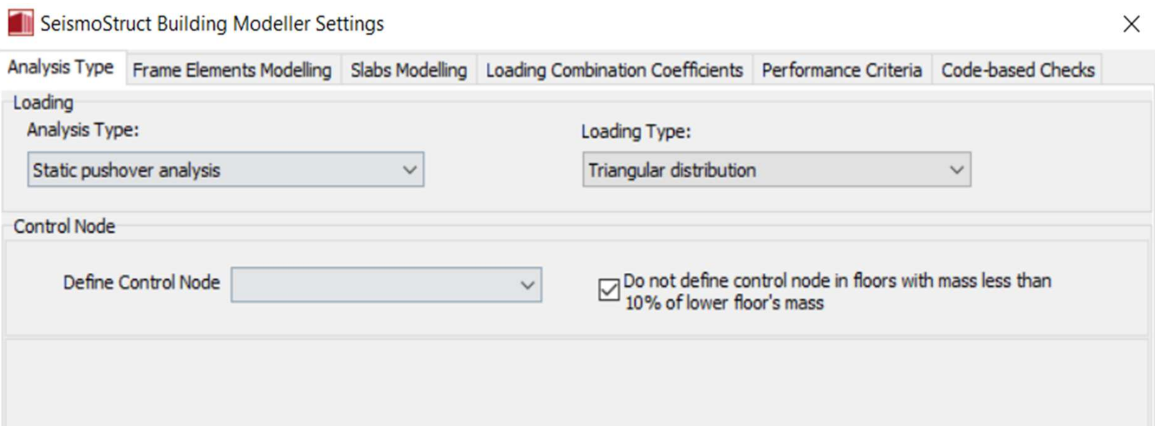

Fig. 5.12 SeismoStruct building modeller settings

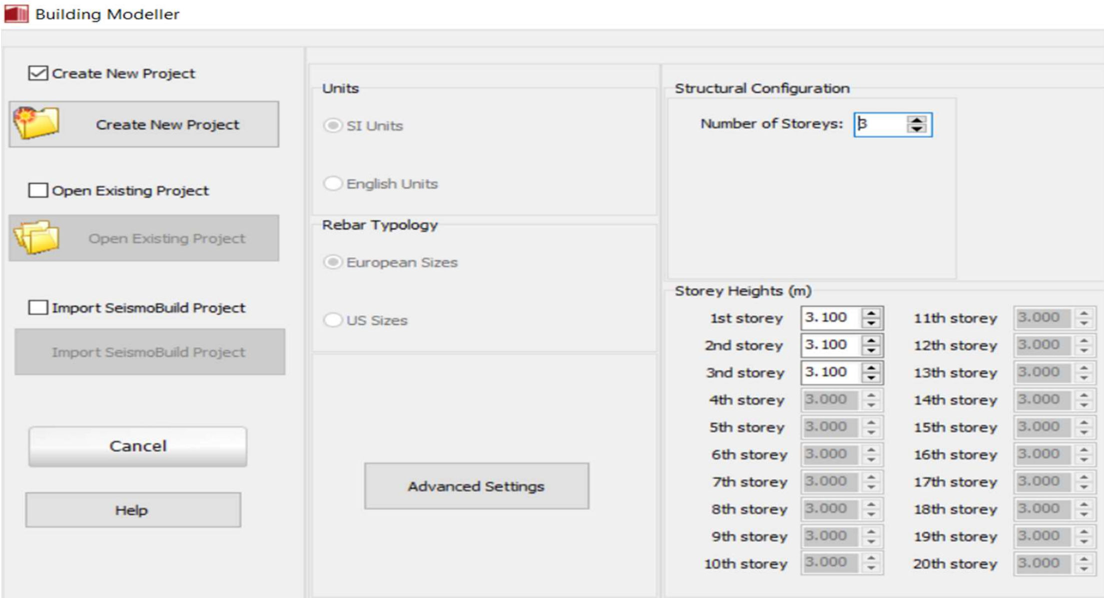

Fig. 5.13 SeismoStruct building modeller

Building has been idealized as three-dimensional space frame using two node frame elements in SeismoStruct. Mander nonlinear concrete model have been used for M25 grade of concrete and Menegotto and Pinto steel model is used for steel reinforcements to incorporate material nonlinearity. The material properties in SeismoStruct are shown as follows

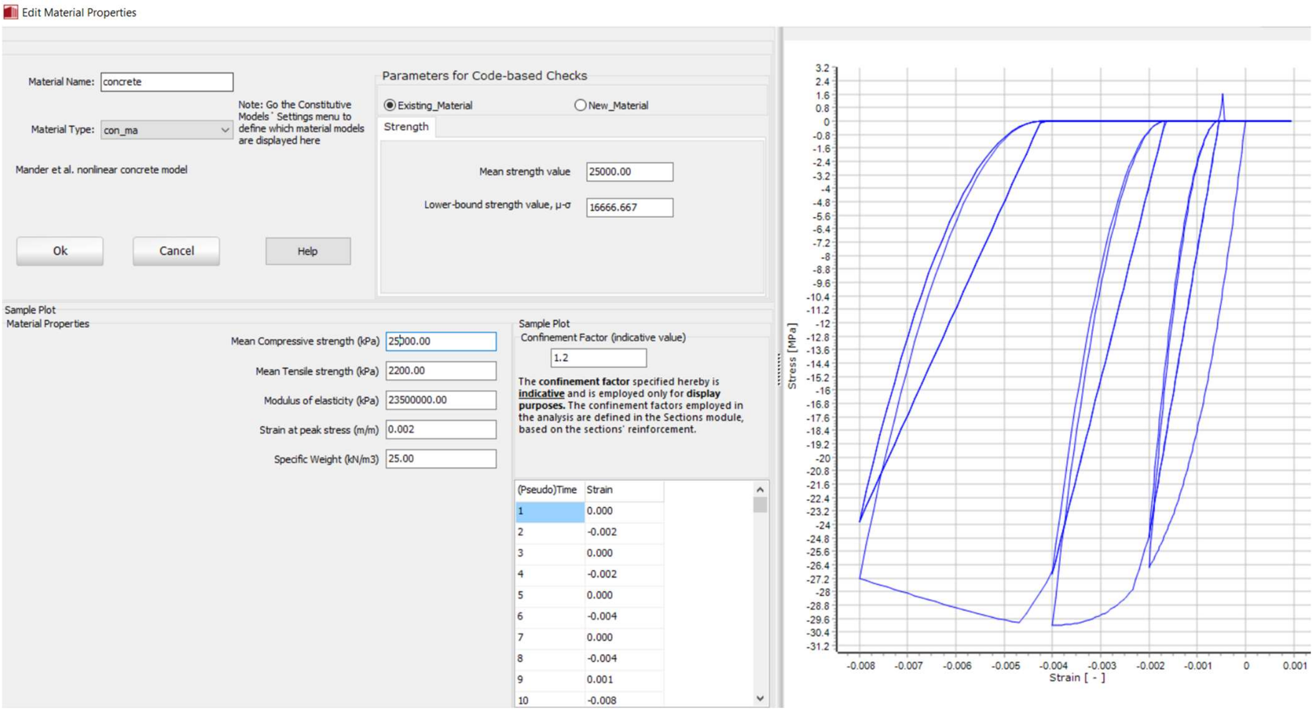

Fig. 5.14 Concrete properties

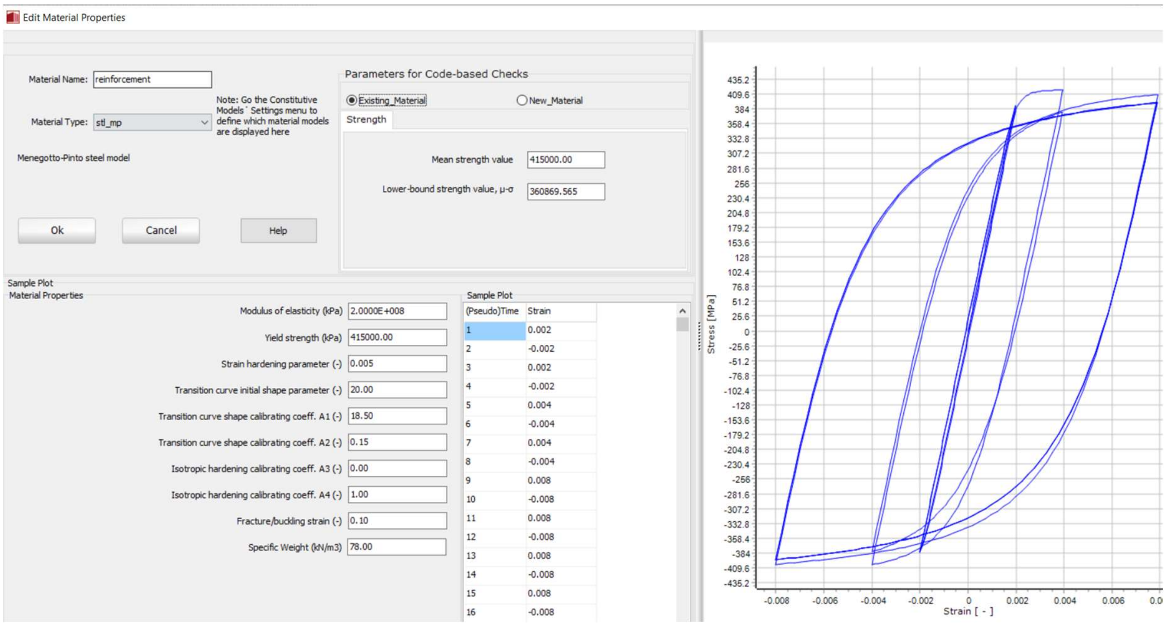

Fig. 5.15 Reinforcement properties

Rectangular reinforced concrete sections are defined for all the sections in SeismoStruct. The reinforcement details in transverse and longitudinal directions are shown in figure.

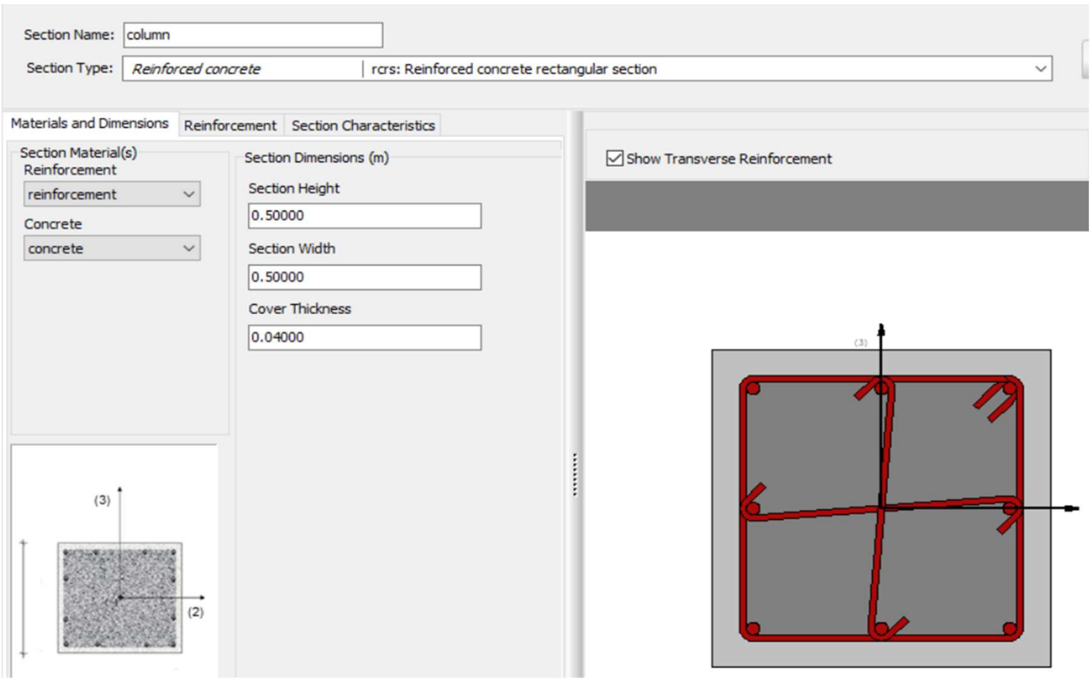

Fig. 5.16 Column section properties

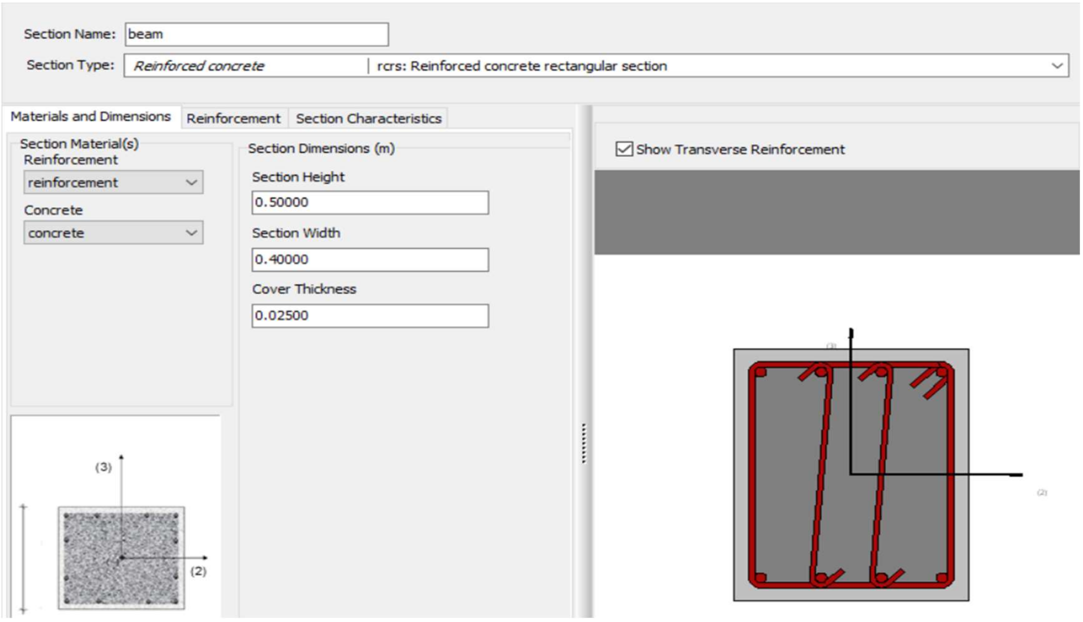

Fig. 5.17 Beam section properties

Inelastic plastic hinge force-based frame elements are used in SeismoStruct to define both beam and column elements. The location of plastic hinge is kept at 5% for both elements. The figure shows the element class properties. The discretization of the column and beam members is done for the fibres of the model. Each section is divided into a number of areas. In this case, number of monitoring points are taken 150 for each element.

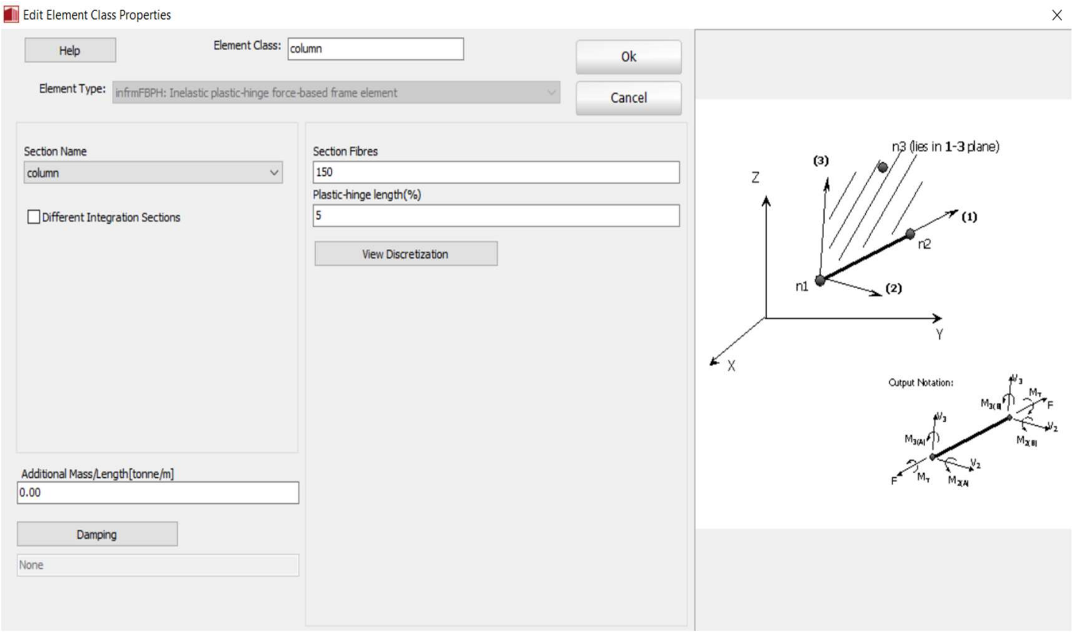

Fig. 5.18 Column element properties

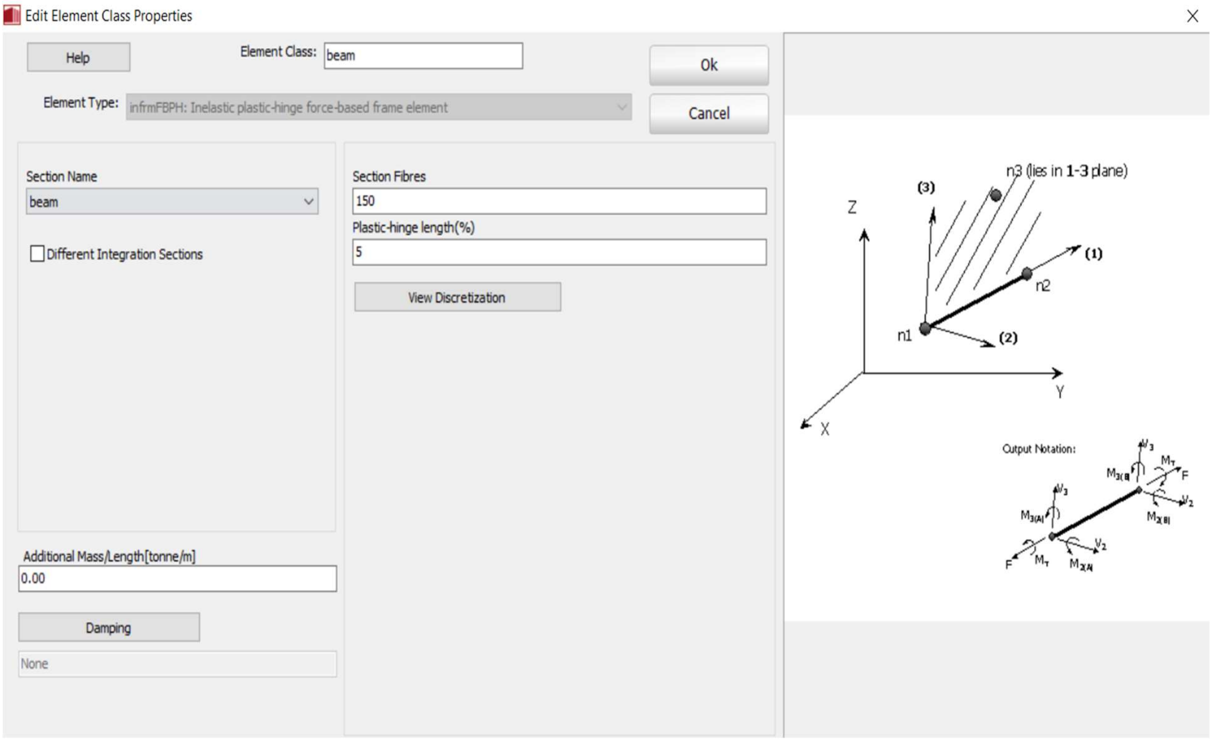

Fig. 5.19 Beam element properties

5.6 PUSHOVER LOAD

Pushover load can be force controlled as well as displacement controlled. The loading is increased monotonically by the force-driven push, so far as the overall load approaches a predefined value, or the structure displays a collapsing function, and the displacing controlled push increases monotonously, so long as it exceeds the target or if the structure exhibits collapse.

In this study, displacement type incremental load is used as a pushover load in X direction in both SAP 2000 and SeismoStruct. The target displacement of 0.6m in 1000 steps is assigned to the software and the values of base shear versus displacement are used to plot the pushover curve. ASCE 41-17 is used to determine the performance levels of the building. Generally, control node is selected as the top node of the building so in this study, the top left node on YZ plane is considered as control node and pushover load is applied to this node. Node number 9244 is the control node in this study. The figure shows the control node and the pushover load with blue arrow.

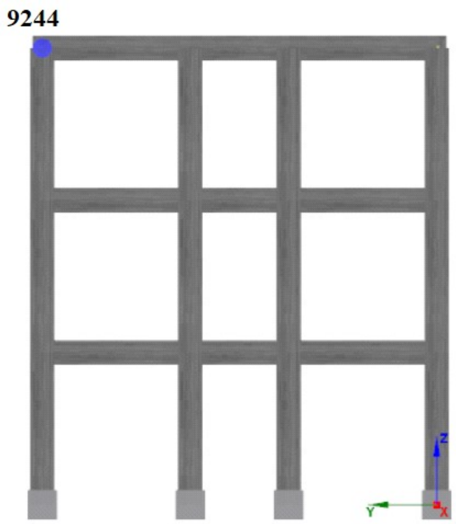

Fig. 5.20 Control node point

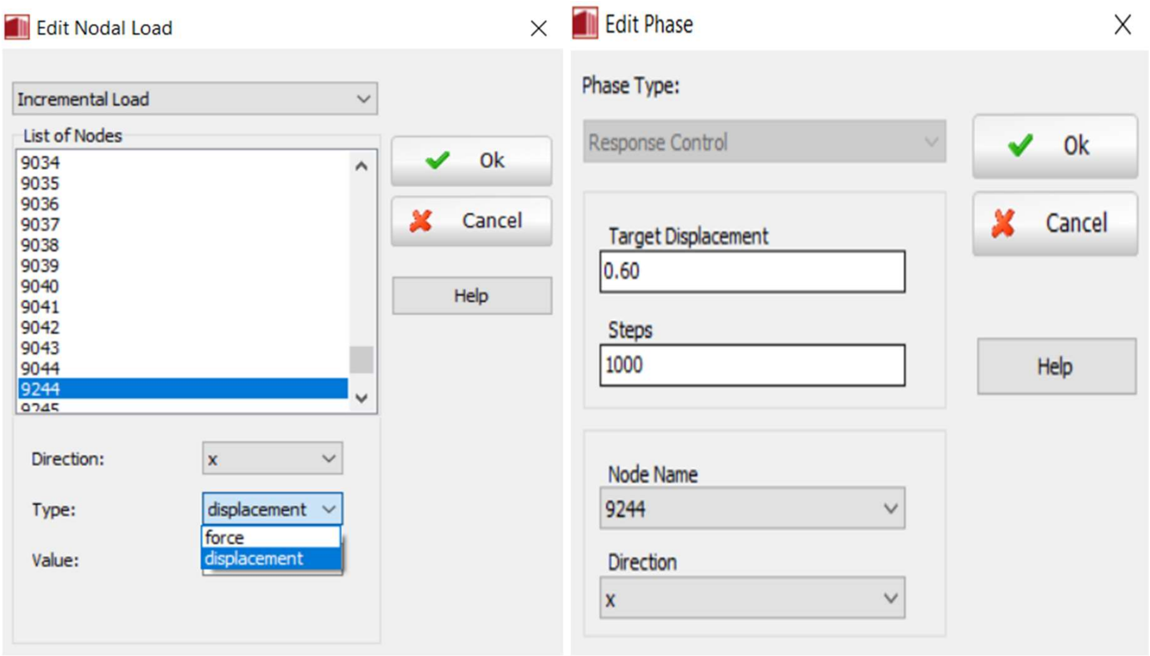

Fig. 5.21 Pushover loading

The performance level can be chosen in SeismoStruct as per ASCE 41-17 for particular seismic hazard. The software automatically generates the Design spectra. One can also modify the spectral acceleration. In this study Soil class A is taken and damping of 5% is assumed for the whole structure.

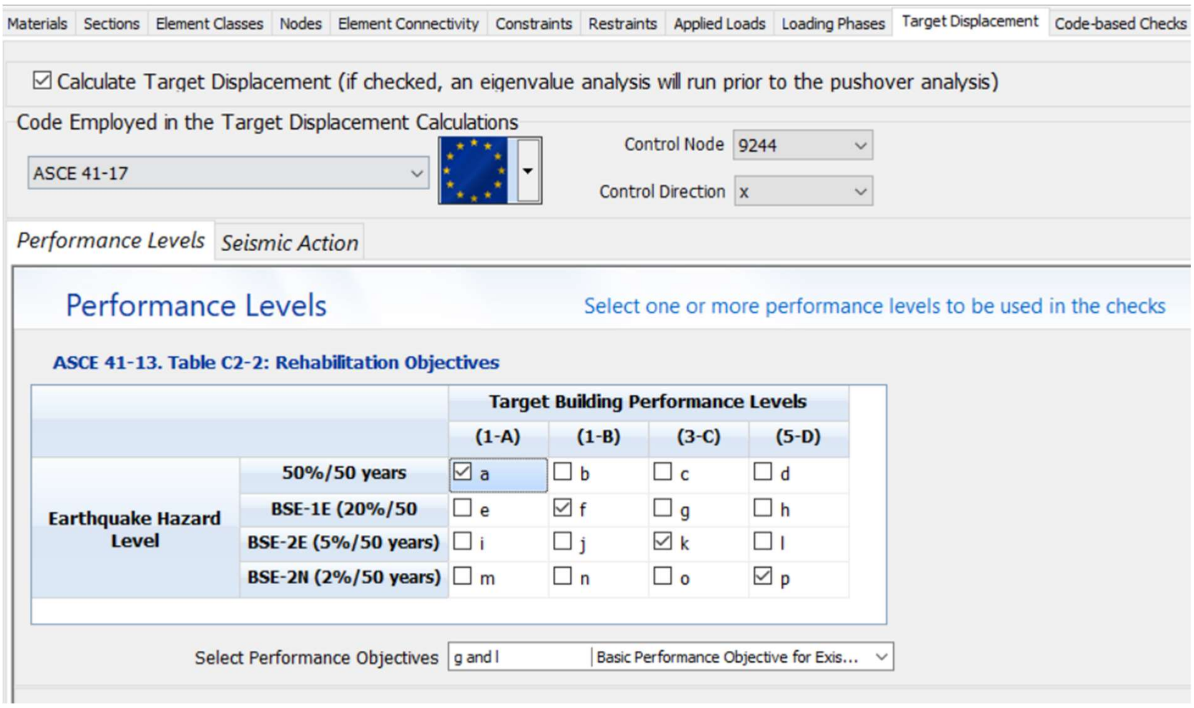

Fig. 5.22 Performance Levels in SeismoStruct

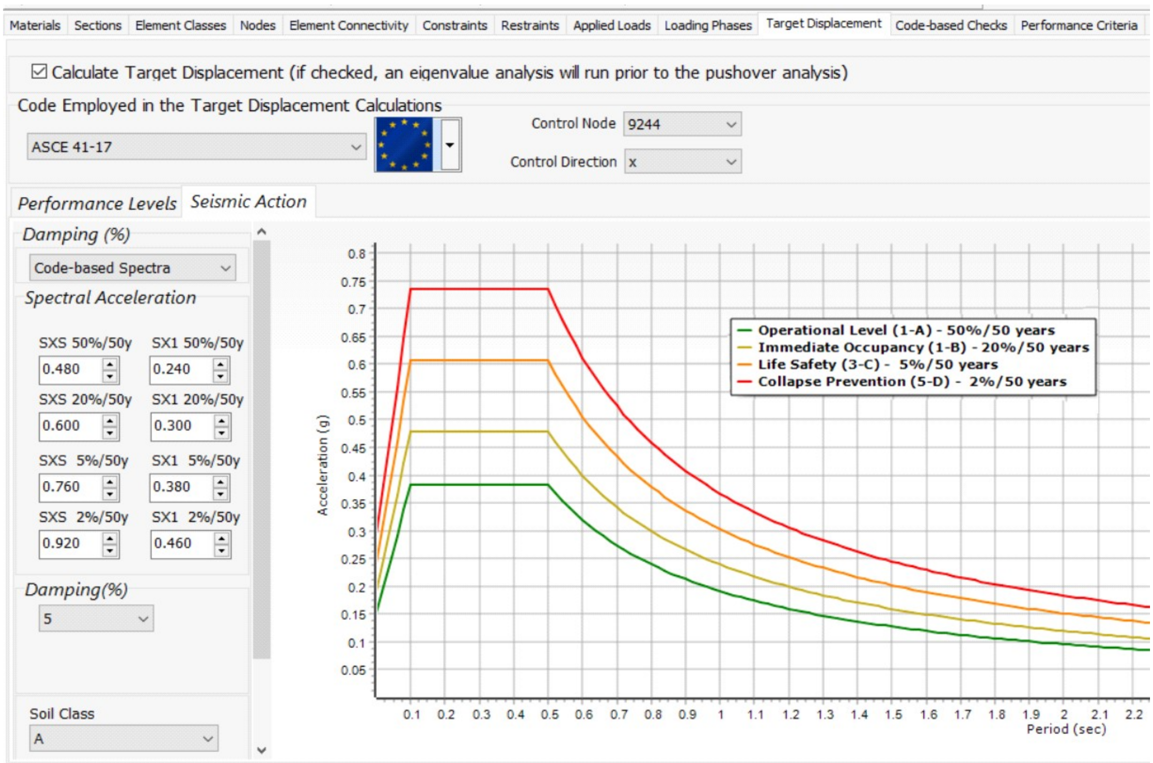

Fig. 5.23 Design Spectra as per ASCE 41

5.7 RESULTS AND DISCUSSION

Once all the modeling steps are completed in the pre-processing stage. The analysis is run on both the programs. The deformed shapes and capacity curves are obtained from the analysis. The results are compared for the values of both the softwares. The deflected shape of the building is shown in figure as follows

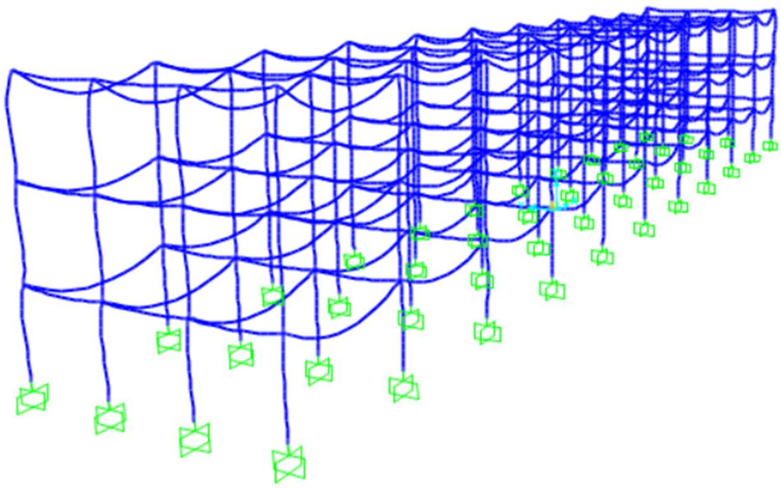

Fig. 5.24 Deformed shape

In SeismoStruct, the value of maximum base shear is 3028.8kN, which occurs at a roof displacement of 0.467 m. The base shear for O, IO, L S and C P are 920kN, 1010kN, 1195kN, 1380kN respectively.

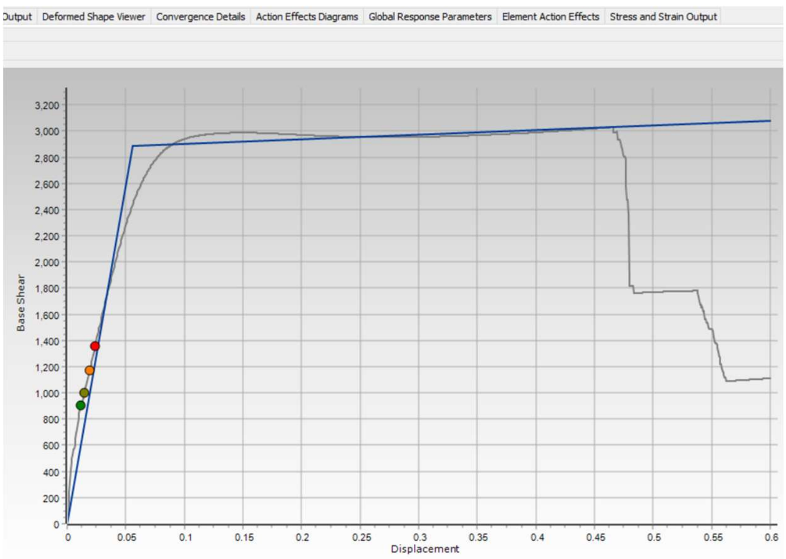

Fig. 5.25 Pushover curve in SeismoStruct

In SAP2000, the maximum value of base shear is 3762kN, which occurs at a roof displacement of 0.43m. SAP2000 does not provide specific base shear values for performance levels. So from SAP2000, nature and number of plastic hinges for each performance level is noted.

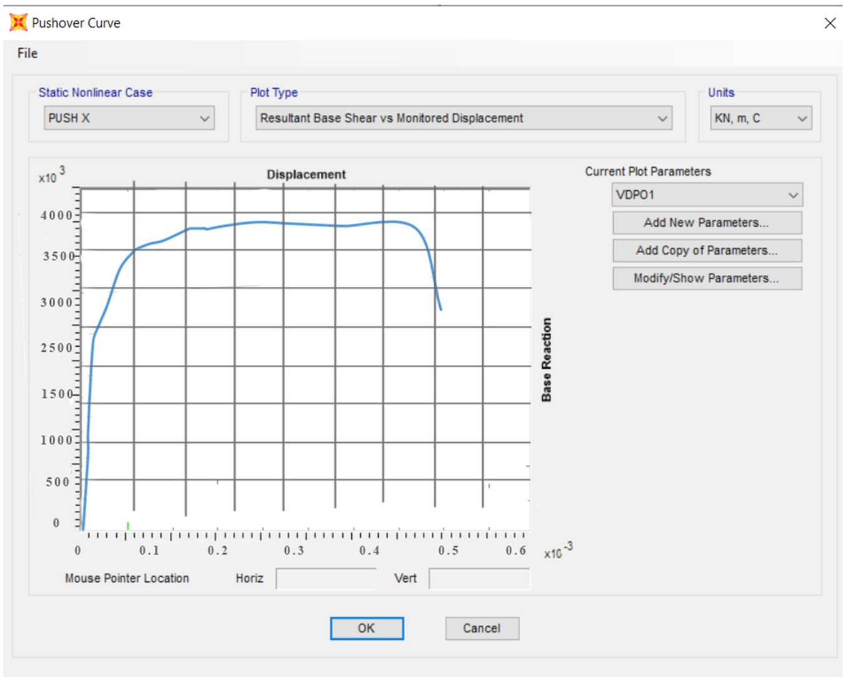

Fig. 5.26 Pushover curve in SAP2000

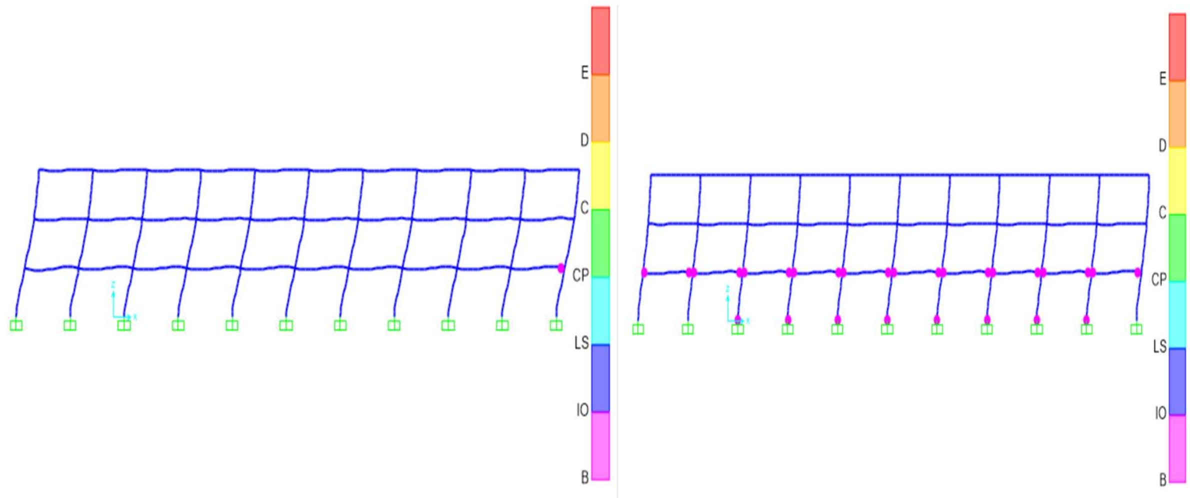

Fig. 5.27 Nature of plastic hinge in step 1 and step 5

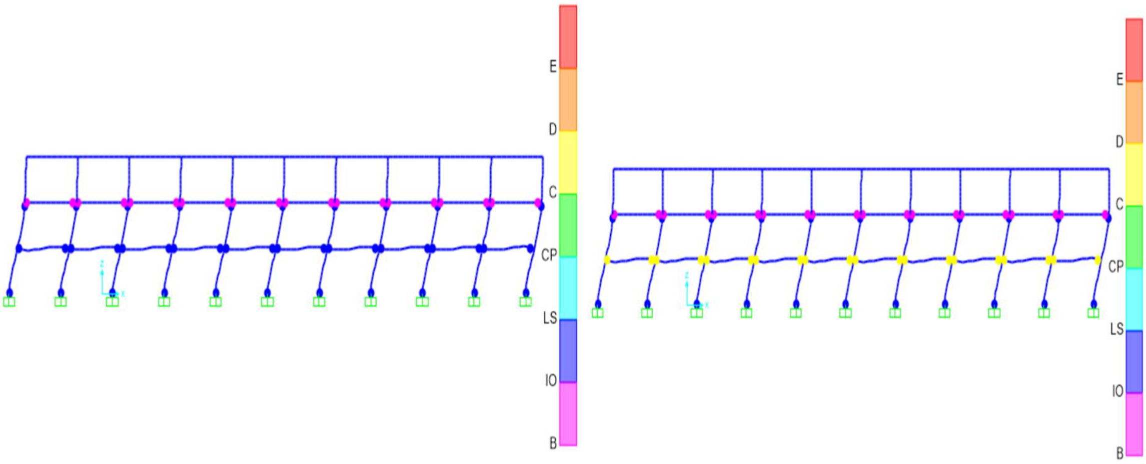

Fig. 5.28 Nature of plastic hinge in step 8 and step 13

The capacity curves of building from both softwares are plotted and compared. It is found that SeismoStruct and SAP2000 computed almost equal base shear for the structure. SAP2000 gave a little higher values. SeismoStruct showed the actual degradation curve and the softening behaviour due to deformation. The curve is well-defined whereas SAP2000 pushover results do not show degradation curve. Strong column weak beam concept is satisfied from the analysis as the first plastic hinge is formed in the beam than in column. The pushover curves are plotted and differences are noted.

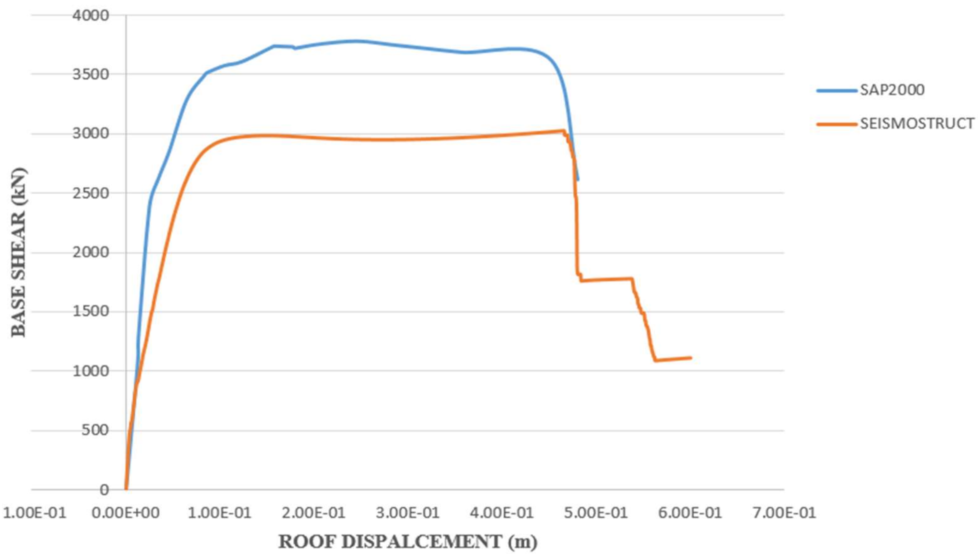

Fig. 5.29 Comparison of pushover curves

CONCLUSION

In this study, a residential structure located in zone V is considered for analysis. Two types of analysis are performed which are recommended by IS 1893:2016. These methods are the simplest and easiest methods to determine the fundamental period of vibration. The same analysis is adopted in finite element software, SeismoStruct. Performance based design is performed on low-rise RC frame building using non-linear static analysis with the help of two softwares. Some of the conclusions drawn from this study are:

- Performance based design methodology is a boon to the earthquake engineering background. Various codes have provisions in their seismic code as it focuses on displacement-based design considering material and geometric nonlinearity.
- Indian seismic code has many limitations and assumptions. IS 1893 states that the fundamental mode dominates the response of the structure whereas ground motions are complex having several frequencies. The idealisation is done by converting the multi degree-of-freedom system to finite degree of freedom system with lumped mass model. The material elasticity and structural rigidity with lumped mass are also considered which needs to be amended in new versions.
- There are many parameters which influence the natural frequency of a structure. As the stiffness decreases, the natural frequency of structure also decreases but a decrease in height of structures leads to increase in the natural frequency. The column elements have a remarkable place for the mass and stiffness of a building, as they are very reliant on its dimensions. Hence, a change in dimension can give rise to a sudden change in the dynamic nature.
- Non-linear static analysis is a significant tool practice to visualize the nature of hazard of both old as well as new buildings under a given earthquake. The capacity curve helps in determining the maximum base shear which in turn will help in effective and efficient construction.
- There are many finite element application softwares, which work in estimating the earthquake capacity and demand. Two computer programs are used to conduct analysis on 3-storey building and results are compared.

- SeismoStruct and SAP2000 computed nearly equal base shear values for the structure with SAP2000 giving a little higher. SeismoStruct showed the actual degradation curve and the softening behaviour due to deformation while SAP2000 pushover results did not.
- Strong column weak beam concept was satisfied from the analysis as the first plastic hinge was formed on the beam. Failure of any beam in a structure can be less remedial but failure of a single column can be the cause of collapse as a whole.

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