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"PUSHOVER ANALYSIS OF 7 STOREY BUILDING USING SAP2000"

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Submitted by

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I, Rohit Sumbria, Roll No. 2K16/STE/16 Student of M.Tech. (Structural Engineering),hereby declare that the project Dissertation titled "Pushover Analysis of 7 storey building using sap2000" which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associateship, Fellowship or other similar title or recognition.

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I hereby certify that the Project Dissertation titled " Pushover Analysis of 7 storey building using sap2000" which is submitted by Rohit Sumbria, Roll No 2K16/STE/16, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere

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ABSTRACT

Elastic method of analysis indicates elastic capacity of structures and tells us about location of first yielding but it cannot capture important phenomena's that control seismic performance of structures during severe ground shaking. Thus, for design and evaluation of structures, inelastic procedures are being used by engineers to understand structural behaviour during earthquakes with the assumption that elastic capacity of structure will be exceeded. Pushover analysis includes pushing the structure using invariant load pattern to get force – deformation relationship. While performing the pushover analysis, it is assumed that structural response is dominated by the fundamental mode. Development of plastic hinges can be monitored during the analysis. In the present study, non-linear pushover analysis using SAP2000 using invariant loading pattern has been carried out with the intention to investigate the relative importance of several factors in the non-linear analysis of frames. Relevant codes (ATC 40, FEMA273, FEMA356 and FEMA440) have been referred.

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LIST OF SYMBOLS, ABBREVIATIONS

K_S - average post- elastic stiffness

Ki - initial stiffness

Ke - effective elastic stiffness

V : base shear (kN)

W : total weight of building (kN)

U_r : roof displacement (m)

a_1 : modal mass coefficient for the fundamental mode

Γ1 : modal participation factor for the fundamental mode

$\emptyset_{1,x}$: amplitude of first mode at roof level

Sa : spectral acceleration (m/s^2)

Sd : spectral displacement (m)

T : period (s)

ζ_{eq} : equivalent damping ratio

ζ_o : hysteretic damping ratio represented as equivalent viscous damping ratio

κ: damping modification factor to simulate the probable imperfections in actual building hysteresis loops

E_D : the energy dissipated in the inelastic system given by the area enclosed by the hysteresis loop

E_S : maximum strain energy

μ : displacement ductility ratio

a : ratio of average post-elastic stiffness of capacity curve to effective elastic stiffness of the capacity curve

Te : effective fundamental period (in seconds)

Ti : elastic fundamental period (in seconds) in the direction under consideration

Ki : elastic lateral stiffness of the structure in the direction under consideration

Ke : effective lateral stiffness of structure in the direction under consideration

Te : effective fundamental period of the structure.

CHAPTER 1 INTRODUCTION

1.1 GENERAL BACKROUND

Sudden release of energy in the earth's lithosphere creates seismic waves which causes shaking of the earth's surface resulting in an earthquake. It occurs because of sudden slip on fault line. Outer layer of the earth pushes the sides of the fault that builds up stress. Rocks slip releases energy which causes shaking during the earthquake. Earthquakes may also occur due to human activities. Gradual movement of tectonic plates (Plate Tectonics) causes earthquakes. Fault occurs due to movement of rocks along either side of fracture. Length of faults may be in kilometres. Normal, reverse or strike faults are few of its types. Any seismic activity is expected to arise from faults. Classification of earthquakes includes Tectonic, volcanic, Collapse, etc. Humans have also played a vital role in inducing earthquake motions. Waves generated are basically of two types either body or surface waves. When earthquake comes under water surface it causes Tsunamis. Earthquakes generally come with no initial warning and have been very destructive to life and property. These can cause physical damage to human settlements, roads and bridges, water pipelines etc. Old structures can be damaged too if not properly strengthen. Structures should be able to sustain severe ground motions that may occur during their construction or normal use.

For analysing steel frames subjected to ground motions, elastic and inelastic methods are used. Elastic Procedures include linear static and linear dynamic procedures. Inelastic method includes complete time history analysis which is overly complex and time consuming. It requires detailed mathematical models of MDOF systems along with characteristics of ground motion making it impractical for everyday use. Characteristics of material models and ground motion affects the response derived from time history analysis. Capacity spectrum method (CSM) and Displacement coefficient method are simplified nonlinear static analysis procedures. Capacity spectrum method provides force displacement curve and compares it with earthquake demand. Structure must have the capacity to handle demand of earthquake. Elastic analysis gives a good insight into the elastic capacity of the structure but fails to account for the redistribution of forces. Inelastic method of analysis demonstrates how building really works during severe ground shaking considering that elastic capacity of the structure is exceeded.

To assess the performance of steel structures, a simple option is pushover analysis. Due to its relative simplicity, methods applicability is increasing continuously. It assumes that the response of the structure may be predicted by fundamental mode or the first few modes of vibration which remains constant through the response.

1.2 PUSHOVER ANALYSIS

Pushover analysis is done by applying monotonically increasing lateral loads to the structure representing the inertial forces that would be experienced by the structure during severe earthquakes. Magnitude of lateral load increases until the structure reaches target displacement. Target displacement represents the top deformation that the structure will be subjected during earthquake. Various structural elements yield during load increment. Loss in stiffness occurs at each event. Capacity curve (Pushover curve) is generated during pushover analysis which shows the relationship between base shear force and roof top displacement. Capacity curve is dependent on strength and deformation capacities of the structure .It enable us to understand the behaviour of the structure beyond elastic limit. Because of the complex nature of the structural properties, structural response cannot be adequately predicted during ground shaking. Displacement values give an estimate of the maximum expected response of the structure during earthquakes.

Pushover analysis is computationally simple to use hence it is being highly preferred for seismic design of steel. structures by major rehabilitation guidelines and codes.

1.3 PURPOSE OF DOING STATIC NON-LINEAR STATIC ANALYSIS

The main purpose of doing pushover analysis to find performance of the structural elements by estimating strength and deformation demands and comparing these demands with available capacities. Main performance parameters include global drift; inter storey drift, deformation between elements etc. Pushover analysis provides information on many responses characteristic’s which elastic analysis fails to provide. Some examples of such response characteristic's includes Realistic force demands on potentially brittle elements, deformation demands of the elements that dissipates energy imparted to the structure in elastically, identification of the critical regions in which the deformation demands is expected to be high, identification of the strength discontinuities that affects dynamic characteristic's in elastic range, estimating inter storey drifts to evaluate p- delta effects.

1.4 OBJECTIVE AND SCOPE OF WORK

Several researchers have studied the various fundamental aspects of Pushover analysis with specially designed frames or using old structure. Firstly, the advantages and limitations of the pushover analysis are discussed to show the benefits of pushover analysis over elastic analysis. A pushover analysis is performed frames using SAP200. The response parameters such as story deformation, inter-story drift ratios, shear at different storeys. Maximum deformation demands also known as target displacements were estimated at elastic and various levels of nonlinear deformation levels. Identification of the assumptions and the accuracy of approximate procedures are done. Pushover analysis can predict seismic demands faster as compared to time history analysis but it produces approximate results as strong theoretical background is still not available. Thus, further research is required in this area.

CHAPTER 2 LITERATURE REVIEW

2.1 INTRODUCTION

It is instructive to review old as well as recent work on nonlinear pushover analysis. It is provided in order to provide an insight into the research that has been done to verify advantages as well as limitations of this method. Detailed reviews will be difficult to present here. Different research done in this area is provided below in chronological order.

2.2 LITERATURE RIVIEW

Gulkan and Sozen (1974) performed inelastic analysis considering SDOF systems to represent equivalently MDOF systems. The main objective was to describe the basic principle behind energy dissipation in building during earthquakes and to provide a basic procedure to calculate the design base shear corresponding to inelastic response. Manual computations or FEM methods were used to obtain stiffness and strength parameters. Reduction in stiffness and increase in energy dissipation capacity influences response of structure during ground shaking. During excitation of the building to large deformation, stiffness decreases and and its capacity to dissipate energy increases. Experiments were done on one storey one bay frames. Results confirmed that response of the STEEL framed structure can be approximated by linear response using a reduced stiffness and substitute damping. Substitute damping helps in understanding the effects of inelastic response in RC structures.

Humar and Wright (1977) studied the behaviour of steel frames with setbacks. Many observations were made which are listed as follows :

Setback of 90% will decrease fundamental period by 35%. As tower becomes more slender the contribution of higher modes becomes more and more prominent. With setback of 90 % contribution of higher modes increased by 40%.Further for the increase in setback of 90%, shear increased as high as 300% to 400%. Storey drift ratios were also very high for setbacks. The increment in inter-storey drift was approximately equal to four times with respect to the regular structure in the tower portion. Similar is the case with coloumn ductility.

Shahrooz and Moehle (1990) studied the response of buildings during earthquakes having setbacks. Both analytical and experimental methods were used .50% setback was given initially at mid height of the building. Displacement profile was studied and were found to be smooth. Drift also occurred at junction of tower and base. Further, fundamental mode dominance on the translational response towards setback direction was understood from the force and displacement profiles. The distribution of lateral forces was almost always similar to the distribution specified by the UBC code; no significant peculiarities were detected in dynamic response. Further studies were done on other reinforced concrete setback frames.

Wood (1992) studied the seismic behaviour of RCC buildings with setbacks. Two nine storey frame structures with steps and setbacks respectively were considered for seismic evaluation. Responses like displacement, acceleration and shear force were studied and compared. The first structure i.e the setback structure consists of two levels as basement and seven storeys above it. The other structure includes a three storey tower with three storeys at middle and base. First mode was considered for analysing the response. The effect of higher modes was considered only in acceleration response. Kinks were also formed at few locations. Lateral force distribution represented the maximum storey shear. There were hardly any differences between different types of analysis done**.**

Vojko Kilar Peter Fajfar (1996) explained the concept of simplified pushover analysis of building structures on which monotonically increasing horizontal loading is applied. Step by step analysis was done on the structure to develop an approximate relationship between base shear and top displacement. Plastic hinges were developed during the analyses which were continuously monitored. Seven storeys RC building was studied for the analysis purpose. They concluded that simplified pushover analysis is capable to estimate important characteristics of non-linear structural behaviour like strength and global plastic mechanism. Less effort was needed during the analysis and thus this method was considered an adequate method for analysis and design purposes as well as for evaluation of existing structures.

Helmut Krawinkler (1996) pointed out advantages, implementation and application of pushover analysis. He explained the importance of pushover analysis in evaluation and retrofit of existing structures and to design the new ones. He showed that pushover analysis, if carefully performed will provide insight into structural aspects that control performance during severe earthquakes. Pushover analysis provides good estimates of local as well as global inelastic deformation demands for structures vibrating in fundamental mode. However estimation of deformation can be inaccurate for structures in which there is significant effect of higher modes. Only the first local mechanism is detected and the other shortcomings are not exposed when the structure's dynamic characterstics change after the formation of first local mechanism. By applying more than one load pattern, effect of higher modes can be mitigated. Many recommendations have also been given in FEMA 273.

Peter Fajfar and Metej Fischinger (1988) proposed inelastic analysis procedures for SDOF system of regular buildings. They used 7 – Storey RC frame building and used N2 method for rational design of building which provided more meaningful conclusion regarding structural response during earthquakes.

A.S. Elnashai (2001) discussed critical issues in the application of inelastic analysis and the areas that can be improved to make the inelastic procedure more adequate to predict dynamic response. The effects of geometric nonlinearity, period elongation, full multimodal effects etc. were discussed which gave better results that were closed to inelastic time history analysis. The uncertainty in the results of pushover procedure was expected because of the contribution of higher modes and the continuous change in resistance distribution of the structure. Results obtained were much promising and closer fit to inelastic dynamic results than existing attempts.

Erol kalkan (2004) explained the validity of lateral load configurations. He explained that the use of invariant force distributions don't exactly incorporates the effects of varying dynamic characteristics due to the influence of higher modes. New approaches were discussed to overcome shortcomings of FEMA procedures. He took eight and twelve storey moment frame buildings for carrying out the analysis. He basically developed an alternative multi-mode pushover analysis methodology for estimating inelastic response quantities. Multi modal pushover analysis procedure was developed which helps in avoiding the complexity of adaptive methods by using invariant load patterns. He showed that FEMA – 356 procedures were not capable if predicting storey level at which critical demand occur. The study indicated that modal combination method results were promising for estimating response quantities such as lateral inter storey drifts and plastic hinge rotations.

Oguz, Sermin (2005) studied invariant lateral load patterns, their effects and accuracy and how to utilize it in non linear pushover analysis to guess the structural behaviour due to earthquake resulting in deformations. For this purpose, various invariant load patterns were used in pushover analysis along with modal pushover analysis were performed on RCC and steel structures in which many fundamental periods were considered. The accuracy of the procedures were also studied to estimate the accuracy of the procedures used to evaluate displacement was also studied on the structures. SAP 2000 and DRAIN-2DX were used to perform analysis. It was observed that accuracy of the pushover analysis depends mainly on the load path, ground shaking and properties of structure also.

P. P. Diotallevi & L. Landi (2005) studied pushover analysis on RC buildings by considering various simplified methods. They compared non-linear static as well as dynamic procedures. Seismic demand was evaluated by using non-linear pushover procedures .Six storey RC frame was studied. It was shown that lateral load distribution strongly affects pushover curves. Dynamic analysis results and modal shape provided were in agreement. Pushover curves provided a conservative estimation of lateral; strength and deformation capacity. A good estimation of deformation capacity and lateral strength was done. Procedures of ATC 40 were in best agreement with the dynamic analysis results.

In 2006, they explained the adequacy of several Non-linear static procedures in predicting the salient response characteristics of RC and structural frame buildings and compared it with the response obtained from time history analysis. It was shown that peak structural response such as plastic rotations of components and inter-storey drift were more consistent than the other NSP with Adaptive Modal Combination procedure. Four different types of nonlinear static procedures were studied.

Ima M., Benjamin L., Irma J.H., Hartanto W. (2015) studied the effectiveness and applicability of Modal Pushover Analysis (MPA). It was shown that MPA estimates well the peak deformation of the structure in non-linear range. A twelve – storey fully ductile moment resisting frame was considered. It was concluded that MPA is sufficient to predict the structural response with one mode consideration. It was further shown that capacity spectrum method and modal pushover analysis tends to be more conservative as compared to complete time history analysis.

Shuraim et al., (2007) used the pushover analysis (ATC-40) for evaluating existing design of new reinforced concrete frame. Various structural defects were analysed and studied using pushover analysis. Members were redesigned and revaluated to check which members require additional reinforcement considering the given seismic conditions. More reinforcement was needed in many column members and the risk to failure of columns were also shown. It was shown by pushover analysis that the structure can withstand the earthquake loading with some deformations at beams and columns.

The above review is non-exhaustive and there are many other investigations possible that are not mentioned here. However, the brief review gives the general reference about the work that has been done in pushover analysis. Further research work needs to be done.

CHAPTER 3 PUSHOVER ANALYSIS

3.1 DESCRIPTION OF PUSHOVER ANALYSIS

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. It consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three dimensional model which includes bilinear or trilinear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. Lateral loads that are predefined are distributed along the building height. Loads are increased until yielding of some members occurs. Modification of structural model is done to account for the reduced stiffness of yielded members and lateral loads are again increased until additional members yield. The process continues until structure becomes unstable or a control displacement at the top of building reaches a certain level of deformation. Roof displacement vs base shear is plotted to get the global capacity curve. Pushover analysis can be performed as following as:

- Force-controlled
- Displacement controlled.

Force-controlled pushover method is used when load is known such as gravity load. Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

Displacement-controlled method is generally used to perform pushover analysis. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Roof displacement at the centre of mass of structure is chosen as the control displacement. The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

3.1.1 CAPACITY

Strength and deformation limits of individual structural components define the overall capacity of the structure. Non-linear pushover analysis is performed to find capacity beyond elastic limits. It uses series of sequential elastic analysis to find force – displacement curve of the RC frame model. Modification to mathematical model is done to account for reduced stiffness. Lateral loads are applied in sequential steps until additional components yield. Till the structure becomes unstable, above steps are continued. The capacity curve gives an approximate idea about the behaviour of the structure once the elastic limit is exceeded.

3.1.2 DEMAND

During ground shaking, deformations are produced in the structure whose pattern may vary with time. It is very time consuming to analyse this motion at each and every step of time. Non-linear methods include set of lateral displacements as a design condition. Displacement demand of the RC frame during earthquake can be defined as maximum expected response of the structure during ground shaking.

3.1.3 PERFORMANCE

Performance check includes extent of damage to structural and non-structural components beyond the acceptable limits. It is done after finding capacity and demand of the structure.

3.2 CONVENTIONAL PUSHOVER ANALYSIS

Conventional Pushover analysis includes incremental step by step solution of the equilibrium equations. A function including forces and displacement is created which is kept constant during the analysis. Depending on the iterative procedure adopted, structural resistance is calculated from internal equilibrium conditions and stiffness matrix is updated. The unbalanced forces are reapplied. The above steps are repeated until the convergence criteria is satisfied or predefined limit is reached. The three critical elements of the procedure are forcing function nature, distribution and Magnitude i.e. what value of applied action be chosen at each load step if they are not held constant.

3.3 USE OF PUSHOVER RESULTS

Pushover analysis helps in finding structural response which is difficult to find through elastic or dynamic analysis. Following response quantities can be easily find out through pushover analysis.

- calculating interstorey drifts and its distribution along the height of the frame
- determining deformation demands for members which are ductile as well as force demands on brittle members
- finding out weak point location in the building
- effects of deterioration in strength of individual members on the overall behaviour of structural system
- identifying location of discontinuities in strength in top vies or front view of the structure that affects dynamic characteristic's
- verifying the adequacy of load path

Elastic analysis doesn't depicts certain weakness like excessive deformation, strength irregularities etc. which can easily be shown by pushover analysis.

3.4 PUSHOVER ANALYSIS LIMITATIONS

The identification of limitations of pushover analysis is must do get its accuracy. Due to higher modes of vibration, the estimated target displacement, selection of lateral force patterns and failure mechanism identification may get affected during pushover analysis.

The value of target displacement (global displacement) is taken from roof displacement value at mass center. Proper target displacement estimation is important as it affects the accuracy of seismic demand prediction if nonlinear pushover analysis. Target displacement for MDOF system is taken as equivalent to SDOF system. Shape vector is used to define properties of SDOF system which represents the deflected shape of MDOF system. A fixed shape vector is used without considering higher mode effects. Torsional effects and foundation uplift are also expected to affect the target displacement.

In pushover analysis, an invariant lateral load pattern is used assuming the distribution of inertial forces to be constant during ground shaking. However, the distribution of inertia forces varies with the severity of earthquake and time during earthquake. Thus the choice of lateral load pattern affects the capacity curve in the analysis. Further, in post elastic range, fixed load patterns have very limited capacity to predict the effects of higher modes. Since, lateral loads are applied statically; it cannot represent accurately the inelastic dynamic response of the structure.

3.5 THE HINGES

Hinges are points on the structure which show high shear or flexural displacement during loading. Location of hinges can be at either ends of beams or columns. During severe ground shaking, one can expect cross diagonal cracks in RC frame structure. Hinges can be of following types:

- Axial hinges
- Shear hinges
- Flexural hinges

Flexural and shear hinges are placed at the ends of beams and columns while axial hinges are inserted at either ends of diagonal strut members.

Force – deformation relation can be represented by hinges under earthquake loading. For example, moment – rotation relation is represented by flexural hinges. While doing Pushover analysis, hinges are inserted in RC frame structure as shown in fig. The non – linear state of hinges within its ductile range is defined by:

- Immediate Occupancy (IO)
- Life safety (LS)
- Collapse prevention (CP)

Table 3.1 – Performance Levels

Performance Level	Structural Performance
Immediate Occupancy (IO)	Structure undergoes a very light damage during ground shaking. Strength of the building is unaltered. Cracking may be seen at weak structural points. There is no permanent deformation in the structure
Life Safety (LS)	Moderate damage occurs to the structure during earthquakes. Strength and stiffness of the building reduces . Some permanent drift occurs
Collapse Prevention (CP)	Structure undergoes Severe damage during ground shaking. Structure may collapse at any time. Structure undergoes large permanent drifts. Structure has no strength after the earthquake.

3.6METHODS OF ANALYSIS

3.6.1 ELASTIC METHOD OF ANALYSIS

It is performed to find force demand on each structural component and compared with available capacities. It includes following methods:

- Code static lateral force procedure
- Code dynamic procedure
- Elastic procedure using demand capacity ratio

In code static lateral force procedure,

Structure is subjected to lateral forces obtained by scaling down the smoothened soil-dependent

Elastic response spectrum by a structural system dependent response reduction factor.

This approach assumes that, designed strength is lower than the actual strength and the structural energy is dissipated through yielding.

In code dynamic procedure, elastic dynamic analysis is performed to find force demands on different components of structure. The dynamic analysis may be of two types:

- response spectrum analysis
- Elastic time history analysis.

Number of modes considered should be such that mass participation is at least 90% for response spectrum analysis. Time history analysis automatically includes the effect of higher modes.

In demand/capacity ratio (DCR) procedure, the force actions and corresponding capacities are compared as demand/capacity ratios. The DCR approach does not consider response reduction factor and takes the full earthquake demand and adds it to the gravity demands.

Force-based methods have certain drawbacks also. Post-elastic behaviour of structural components could not be identified as they are evaluated for serviceability in the elastic range only. However, post-elastic behaviour should be considered as during strong ground shaking, structure is expected to deform in inelastic range.

Elastic methods can predict elastic capacity of structure but failure mechanisms are not predicted as well as redistribution of forces is not considered. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair.

The above said drawbacks of elastic procedures have led the researches to develop nonlinear analysis procedures considering seismic demands and available capacities

3.6.2 INELASTIC METHOD OF ANALYSIS

During strong earthquakes, RC structures undergo inelastic deformations which change its dynamic characteristics with time. Hence, inelastic analytical procedures need to be considered to investigate performance of a structure. Inelastic analytical procedures enable us to understand the real structural behaviour by identifying modes of failure and the potential for progressive collapse. Inelastic analysis procedures include inelastic pushover analysis and time history analysis.

To predict the force and deformation demands at various structural components, time history analysis is the most accurate method. However, the use of inelastic time history analysis is limited because modelling and ground motion characteristics affects dynamic response of structure as it requires proper modelling of cyclic load deformation characteristics. Also, a set of representative ground motion records is required which considers uncertainties and differences in severity, frequency and duration characteristics. Further, high time required for input preparation, computation time etc. make the use of inelastic time history analysis impractical for seismic performance evaluation.

Due, to its simplicity, pushover analysis is highly preferred. As it directly incorporates material characteristics. Inelastic static analysis procedures include:

- Capacity Spectrum Method
- Displacement Coefficient Method
- The Secant method.

3.7 ANALYSIS PROCEDURE

3.7.1 LINEAR STATIC PROCEDURE (LSP)

IN LSP, linear elastic static analysis is used for seismic analysis of structures, distribution of seismic forces over the building height and finding out corresponding internal forces and displacements. It uses a pseudo – lateral static load pattern to find force and displacement demands on structural elements due to earthquake. Demand and capacities are compared at later stage. Strength or stiffness irregularity cannot be accounted for in linear static procedure. Linear elastic stiffness and equivalent viscous damping values ae used to model structures.

3.7.2 LINEAR DYNAMIC ANALYSIS PROCEDURE (LDP)

IN LDP, linear elastic dynamic analysis is used for seismic analysis of structures, distribution of seismic forces over the building height and finding out corresponding internal forces and displacements. Force and displacement demands are computed using a modal analysis, a response spectrum analysis, or a time- history analysis. Response spectrum analysis is more preferred over modal analysis as it avoids the complete time history analysis of number of single degree of freedom systems that corresponds to each mode of vibration of interest. Maximum ground acceleration is obtained from the response spectrum of the ground motion and demands are calculated.

3.7.3 NON LINEAR STATIC PROCEDURE (NSP)

In NSP (Pushover analysis) , non – linear load deformation characteristics of individual structural elements is incorporated in a mathematical model (accounting the effects of material inelastic response) and are subjected to monotonically increasing lateral loads until the target displacement is reached. Target displacement represents the maximum displacement that the structure is expected to experience during ground shaking. During lateral load application, cracks, yielding and formation of plastic hinges is recorded.

3.7.4 NON LINEAR DYNAMIC PROCEDURE (NDP)

NDP is expected to remove the shortcomings of other analysis procedure. Basic modelling procedure is similar to static procedure with the exception that time history analysis is done to calculate responses. Design displacements are determined directly through dynamic analysis using ground motion time histories. The calculated internal forces are expected to be in close approximations of those expected during the design earthquake. Modelling of the structure, non-linear material models used and ground motion characteristic's affects the accuracy of the method.

3.8 PLASTIC HINGE (NONLINEAR)

For critical section of beams and columns, Pushover analysis requires the development of force deformation curve as shown below.

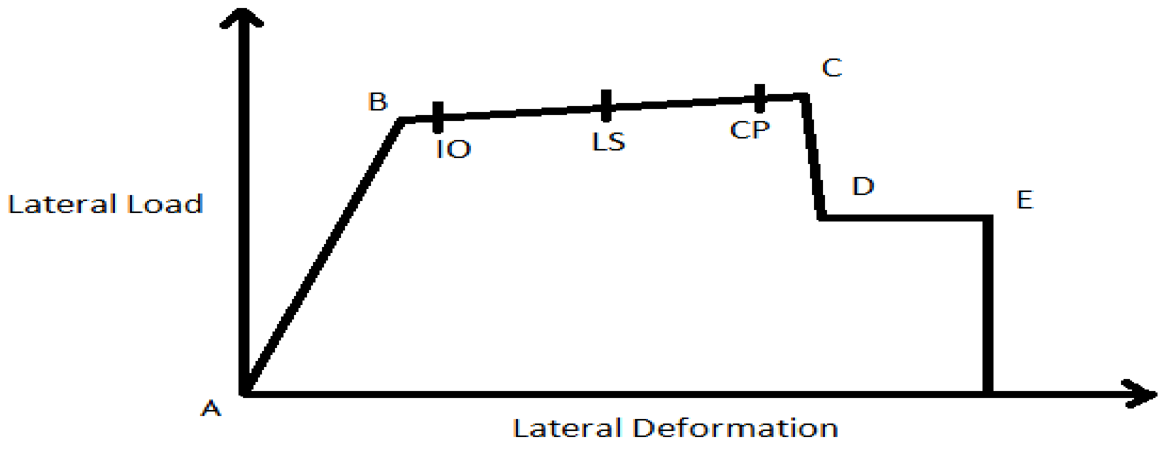


Figure 3.1 Load Deformation Relation

- Point A – It represents the unloaded condition.
- A to B - Load deformation relation shall be described by the linear response from A to an effective yield B.
- B to C -The stiffness reduction occurs from point B to C. Its slope is generally taken between 0 and 10% of the initial slope. Points between B and C represent acceptance criteria for the hinge, which are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP).
- Point C - It has a resistance equal to the nominal strength
- C to D -Decrease in lateral load resistance occurs from C to D, It corresponds to an initial failure of the member.
- D to E - The DE Line represents the residual strength of the member. These points are specified according to FEMA to determine hinge rotation behaviour of RC members.
- Point E - The response is at reduced resistance at E and final loss of resistance occurs

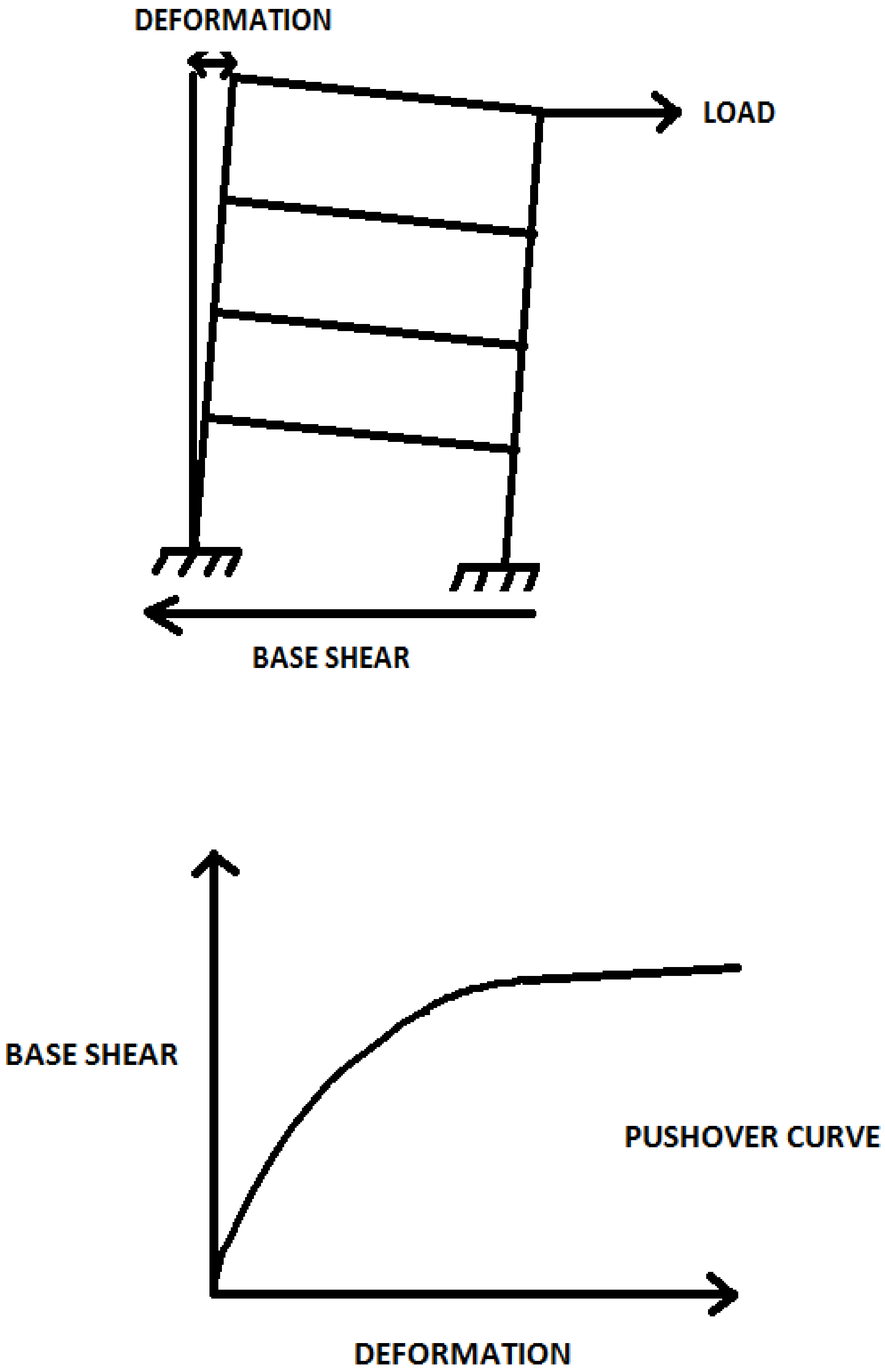


Figure 3.2 Pushover curve development

CHAPTER 4 CAPACITY SPECTRUM METHOD AND PUSHOVER CURVE

4.1 CAPACITY SPECTRUM METHOD (ATC-40 PROCEDURE A)

Capacity spectrum method has three versions as given in ATC – 40 which helps in finding demand due to displacement occurring due to ground. All the procedures are said to be approximate as dynamic analysis is not considered of the inelastic system. The three procedures are approximate since they avoid the dynamic analysis of inelastic system. Demand for series of linear systems can be founded with the help of dynamic analysis. Different procedures are used for hand as well as computer analysis. ATC-40 Procedure is utilised here which is defined as below.

1. Capacity curve of the building is developed initially with the help of pushover analysis.

2. Displacement Coefficient Method gives a method to develop a capacity curve which is known as Bilinear representation approach. A line is drawn initially which represents the average post – elastic stiffness, Ks, which belongs to capacity curve. Then a secant line is drawn which represents effective elastic stiffness, Ke, in such a way that it intersects the capacity curve. It is preferred that the point of intersection should be at 60% of the yield base shear. The whole process involves an iteration. It may also happen that effective stiffness (Ke) and initial stiffness (Ki) are equal. Following curve shows an example of capacity curve and its bilinear representation.

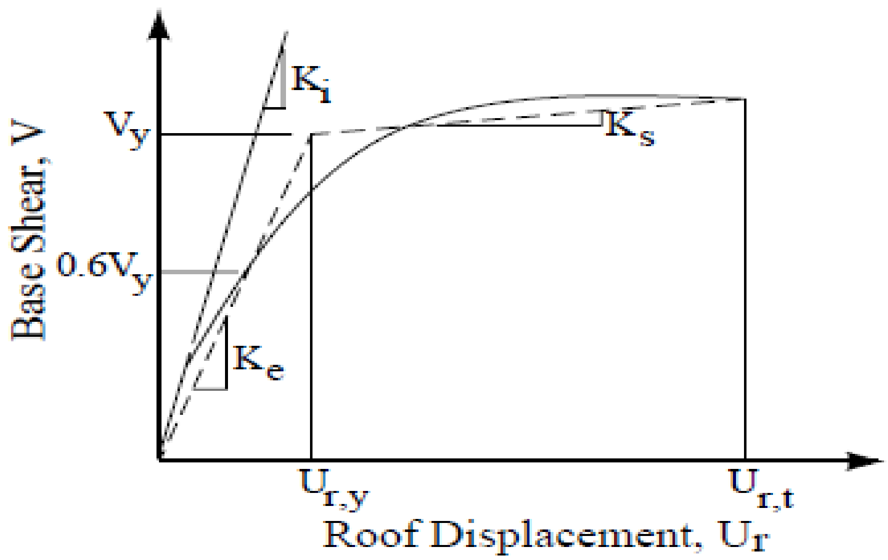


Figure 4.1 Bilinear representation of capacity curve

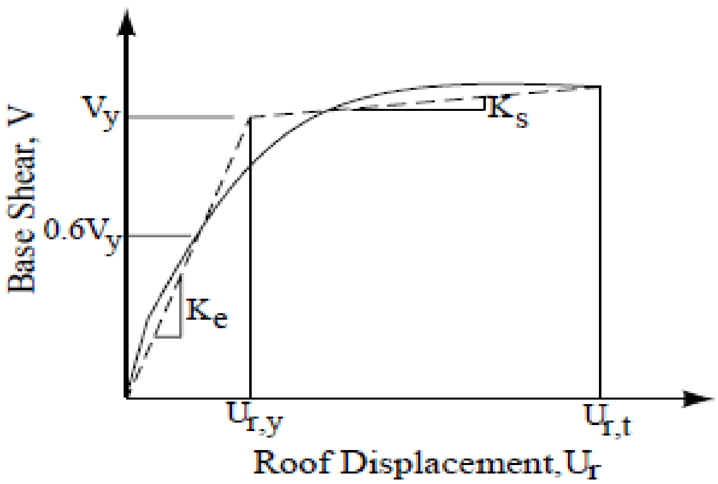
3. Next step includes converting the bilinear capacity curve into Acceleration-displacement response Spectrum (ADRS) format and for this the following equations are used :

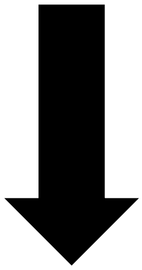
$$Sa = \frac{V/W}{\alpha_1}$$

$$Sd = \frac{U_r}{\Gamma_1 \cdot \phi_{1,r}}$$

Where,

- W - Building weight (KN)
- V - base shear (KN)
- U_r- Displacement at roof (m)
- a_1- modal mass coefficient for the fundamental mode
- Γ1- Fundamental mode's modal participation factor
- $\emptyset_{1,x}$ - It shows the amplitude at roof level of the first mode
- Sa - spectral acceleration (m/s^2)
- Sd - spectral displacement (m)





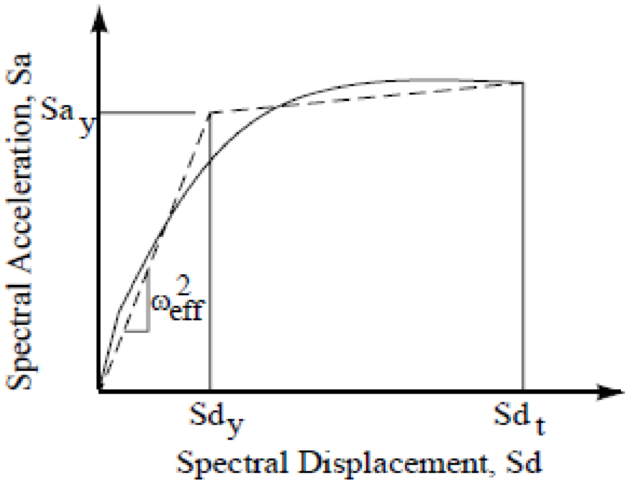


Figure 4.2 Capacity curve conversion to capacity spectrum

2. 5% elastic response (demand) spectrum is converted from standard Sa vs T format to Sa vs Sd (ADRS) format. The spectral displacement, Sd, is computed with the help of following equation for any point on standard response spectrum as shown in the following figure

$$Sd = \frac{1}{4\pi^2} Sa T^2$$

Where,

- Sa - spectral acceleration (m/s^2)
- Sd - spectral displacement (m)
- T - period (s)

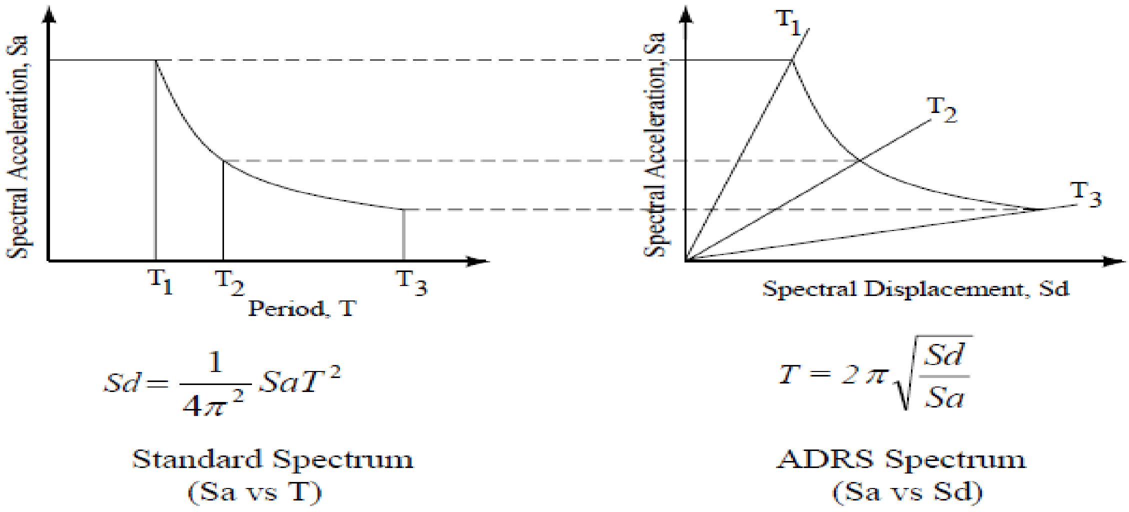


Figure 4.3 Response spectrum is shown in standard and ADRS Formats

3. Next step includes assuming peak spectral displacement demand Sd_i = Sd (T_1, ζ = 5%) which is determined for T_1 period from the elastic response spectrum.

4. Then we need to find the displacement ductility ratio which may be defined as follows:

$$\mu = Sd_i / Sd_y$$

5. Equivalent damping ratio (ζ_{eq}) is then computed by using the equation as follows:

$$\xi_{eq} = 0.05 + \kappa \cdot \xi_o$$

where ,

- ζ_{eq} - This term defines the equivalent damping ratio
- 0.05 – 5% viscous damping inherent in the structure (assumed to be constant)
- κ – It defines the damping modification factor to incorporate the imperfections that may develop in building hysteresis loops
- ζ_o – It represents the hysteretic damping ratio which can also be defined as equivalent viscous damping ratio

The energy dissipated in a vibration cycle of the inelastic system and that of the equivalent linear system should be equated which is indirectly defined as equivalent viscous damping ratio. This concept is used to define viscous damping ratio(Defined By Chopra) as follows ;

$$\xi_o = \frac{1}{4\pi}\frac{E_D}{E_S}$$

Where,

- E_D – It is defined as the energy which gets dissipated in the system that is elastic given by the enclosed area in the hysteresis loop
- E_S – It gives the value of maximum strain energy

Further, If we Substitute E_s and E_d in the above Equation, we get the following term :

$$\xi_o = \frac{2}{\pi}\frac{(\mu-1)(1-\alpha)}{\mu(1+\alpha\mu-\alpha)}$$

Where,

- μ - It is taken as the ratio of displacement and ductility
- a – It is the ratio of average post-elastic stiffness of capacity curve to effective elastic stiffness of the capacity curve

The structural behaviour of the building affects the κ-factor . Further, the building structural behaviour depends on the seismic resisting system's quality and the time interval during which earthquake lasts. Following three types of structural behaviour is defined by ATC-40:

- Type A represents hysteretic behavior with stable and full hysteresis loops
- Type B denotes hysteresis behavior intermediate between Type A and Type C (following table)
- Type C represents poor hysteretic behavior with severely pinched and/or degraded loops

Shaking Duration	Essentially New Building	Average Existing Building	Poor Existing Building
Short	Type A	Type B	Type C
Long	Type B	Type C	Type C

Table 4.1 : Structural Behavior Types as given by ATC-40

Following table gives the ranges and limits for the values of κ allocated to the three structural behavior types :

Structural Behavior Type	ζ_o (percent)	κ
Type A	≤ 16.25	1.0
	> 16.25	$1.13 - \dfrac{0.51(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type B	≤ 25	0.67
	> 25	$0.845 - \dfrac{0.446(Sa_y Sd_i - Sd_y Sa_i)}{(Sa_i Sd_i)}$
Type C	Any value	0.33

Table 4.2 - Damping Modification Factor values given by ATC-40

6. Plot elastic demand spectrum for ζ_{eq} determined in Step 5 and bilinear capacity spectrum on the same chart and finally obtain the spectral displacement demand Sd_j at the intersection (see Figure 5.8)

7. The a Check for convergence is done. Following criteria is used for this check:

If (SDj-SDi)/(SDj) ≤tolerance (=0.05) then spectral displacement demand induced due to earthquake is $Sd=Sd_j$. If it is not the case, then we have to manually set $Sd_i = Sd_j$ or some other estimated value can be taken and we have to repeat Steps 6-9.

8. The spectral displacement demand determined in Step 7 is converted to global displacement at roof by multiplication of the spectral displacement demand estimated which is of equivalent Single degree of freedom system with first modal participation factor at the roof level.

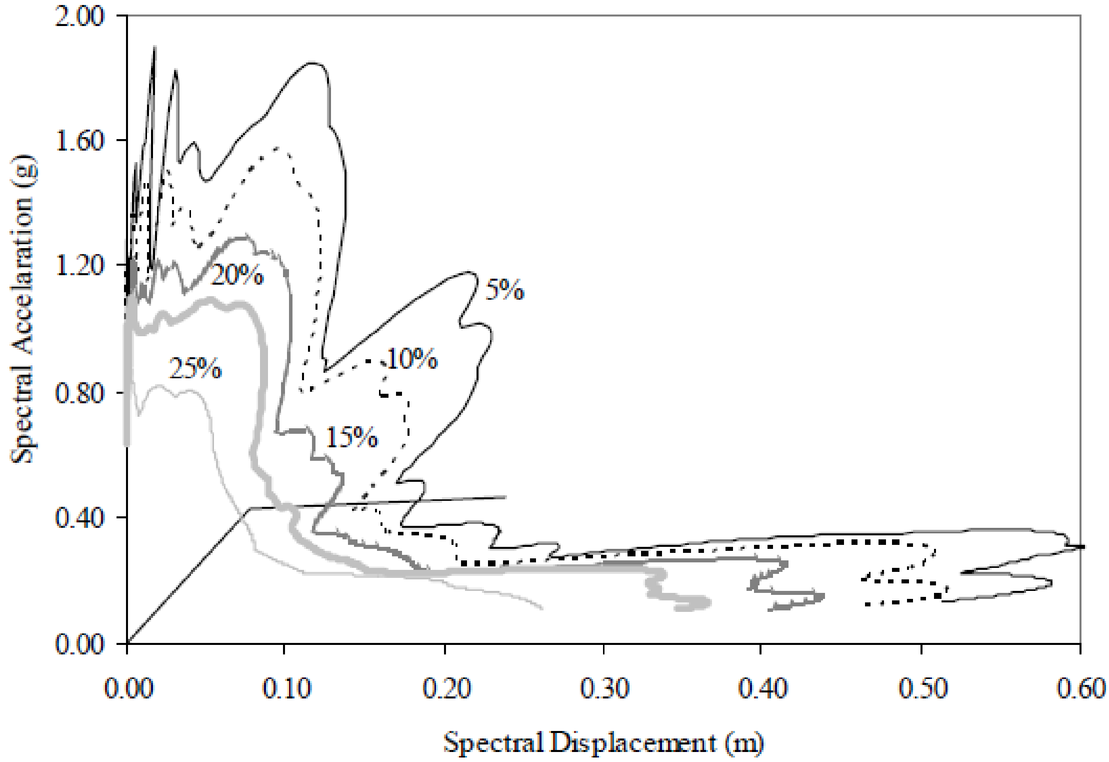


Figure 4.4 Capacity Spectrum Method given in ATC – 40 Procedure A

4.2 DISPLACEMENT COEFFICIENT METHOD (FEMA-356)

The Displacement Coefficient Method is an approximate method which is given by FEMA-356. It provides a method to directly calculate numerical value of maximum global displacement demand of structures. Deformation demand (dt) which is inelastic, is calculated by changing deformation elastic demand with the help of displacement modification factors. Bilinear representation of capacity curve is needed which can be used in the method. Once the bilinear curve is constructed, effective fundamental period (Te) of the building structure is calculated with the help of following equation :

$$T_e = T_i\sqrt{\frac{K_i}{K_e}}$$

Where,

- Te - It is the effictive fundamental period (sec)
- Ti - For the given direction, it is the elastic fundamental period (sec)
- Ki - For the given direction under consideration, it is defined as the elastic lateral stiffness of the building.
- Ke - For the given direction, it is the effective lateral stiffness of building in the direction under consideration .

Then the target displacement, dt , is calculated by changing the spectral displacement of an equivalent single displacement system using the coefficients as shown below.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

where

- C0 is the modification factor which relates spectral displacement with the roof displacement of the given structure. Generally, the first modal participation factor is used here at the roof level.
- C1 is the modification factor which is used to relate maximum inelastic displacements that is expected to the displacements that are calculated for linear elastic response.
- C2 defines the modification factor which represents the effect that will arise due to the shape of the hystersis on the maximum response due to displacement. In the present case, value of C2 was taken equal to 1.1 for both elastic and inelastic displacement levels. Since the values obtained by Displacement Coefficient procedure as given in FEMA-356 depends

further on the value of coefficient C2, for that the value of coefficient C2 should be taken as unity and should be in the elastic range. Further it should take the particular value for finding the performance level in the inelastic range for seismic performance evaluation purposes.

- C3 gives the modification factor which represents the increased displacements that arise due to second order effects.
- Sa gives the acceleration response spectrum which is calculated at effective fundamental period of the building.
- Te gives the fundamental period (effective) of the structure.

For the given method, target deformations can be found out for different seismic performance levels. In the given method, the target displacements for each ground motion were calculated for the life safety at performance levels.

4.3 CONSTANT DUCTILITY PROCEDURE (CHOPRA&GOEL)

Chopra and Goel proposed an improvement to Capacity Spectrum Method described in ATC-40 [3]. The improved capacity-demand diagram method uses constant ductility demand spectrum to estimate seismic deformation of equivalent SDOF system representation of MDOF structure. There are the three types of the proposed better procedure; Procedure A, Procedure B and Numerical procedure. Procedures A and B are graphically similar to the procedures given in ATC-40 Procedures A and B. In the given study, Procedure A was used to find the seismic displacement demand of the inelastic Single degree of freedom systems. Following steps are used here:

1. Firstly we need to perform the same Steps starting with 1 ending at 3 which are given in the procedure defined by ATC-40.

2. Then we need to find elastic damped response spectrum (5%) along with the set of inelastic response

spectra for various ductility levels.

3. Graph between the bilinear capacity spectrum and demand spectra together is plotted in the next step.

4. The displacement demand is determined as follows:

Initially, the ductility value is calculated at the junction of capacity spectrum and demand spectrum . Mathematically, it can be written as : (um / uy). When the calculated value of the ductility matches with the ductility which we find at the intersection of the demand spectrum, the same intersection point is selected as inelastic displacement demand of single degree of freedom system.

5. Then we need to Convert the spectral displacement demand determined in the Step 4 to the global (roof) displacement by doing a multiplication that is estimated by the spectral displacement demand of equivalent single degree of freedom system by including the first modal participation factor at the roof level. Further In the next step, the inelastic response spectra which is for different ductility levels can be found from the elastic response spectrum by using the relation defined as Ry-μ-Tn relation. Ry-μ-Tn relations are suggested by many researches such as Nassar and Krawinkler, Newmark and Hall etc. But here, Seismo Signal was utilised for developing both the elastic and inelastic response that the spectra are computed by the method of time integration of the equation of motion of a series of Single degree of freedom systems from which the calculation of the peak acceleration response quantities were done.

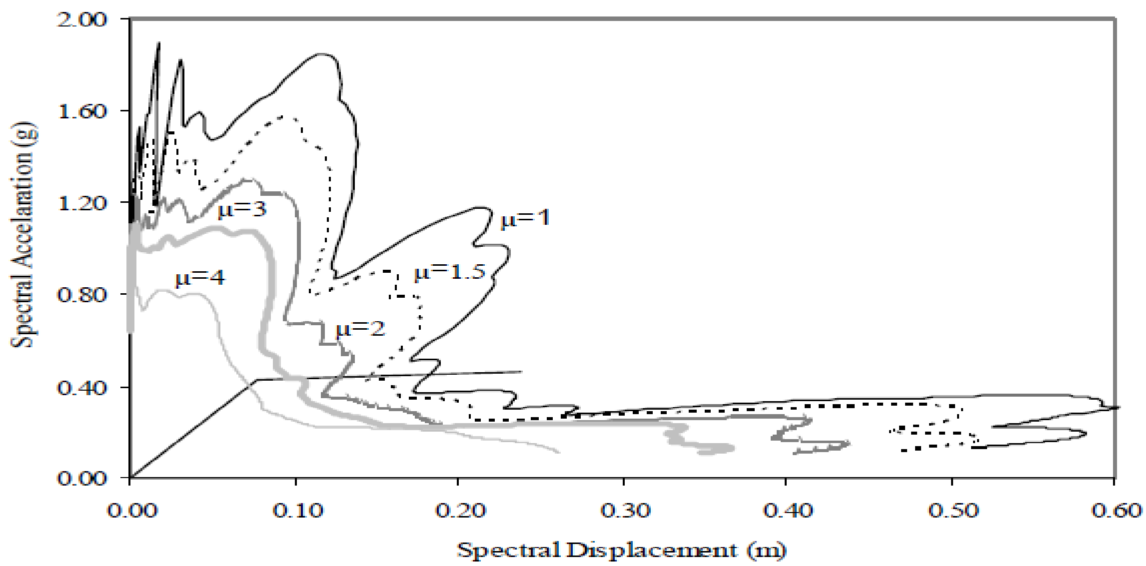


Figure 4.5 Constant ductility procedure

The displacement demand is determined at the intersection of capacity and demand spectra in both Capacity Spectrum Method (ATC-40 Procedure A) and Constant Ductility Procedure. However, the demand is calculated by analysing an inelastic system in improved procedure instead of equivalent linear systems in Capacity Spectrum Method (ATC-40 Procedure A).

CHAPTER 5 PUSHOVER ANALYSIS ON SAP2000

5.1 GENERAL

Computer program is used to model nonlinear behavior of structural elements in order to perform pushover analysis. Different softwares may give variations in the pushover results and hence, the basic principles of different software used for doing analysis should be analysed adequately to get better results.

In the present study, pushover analyses is done on Frames (7 storey) in SAP2000 .

5.2 PROPERTIES OF MATERIALS

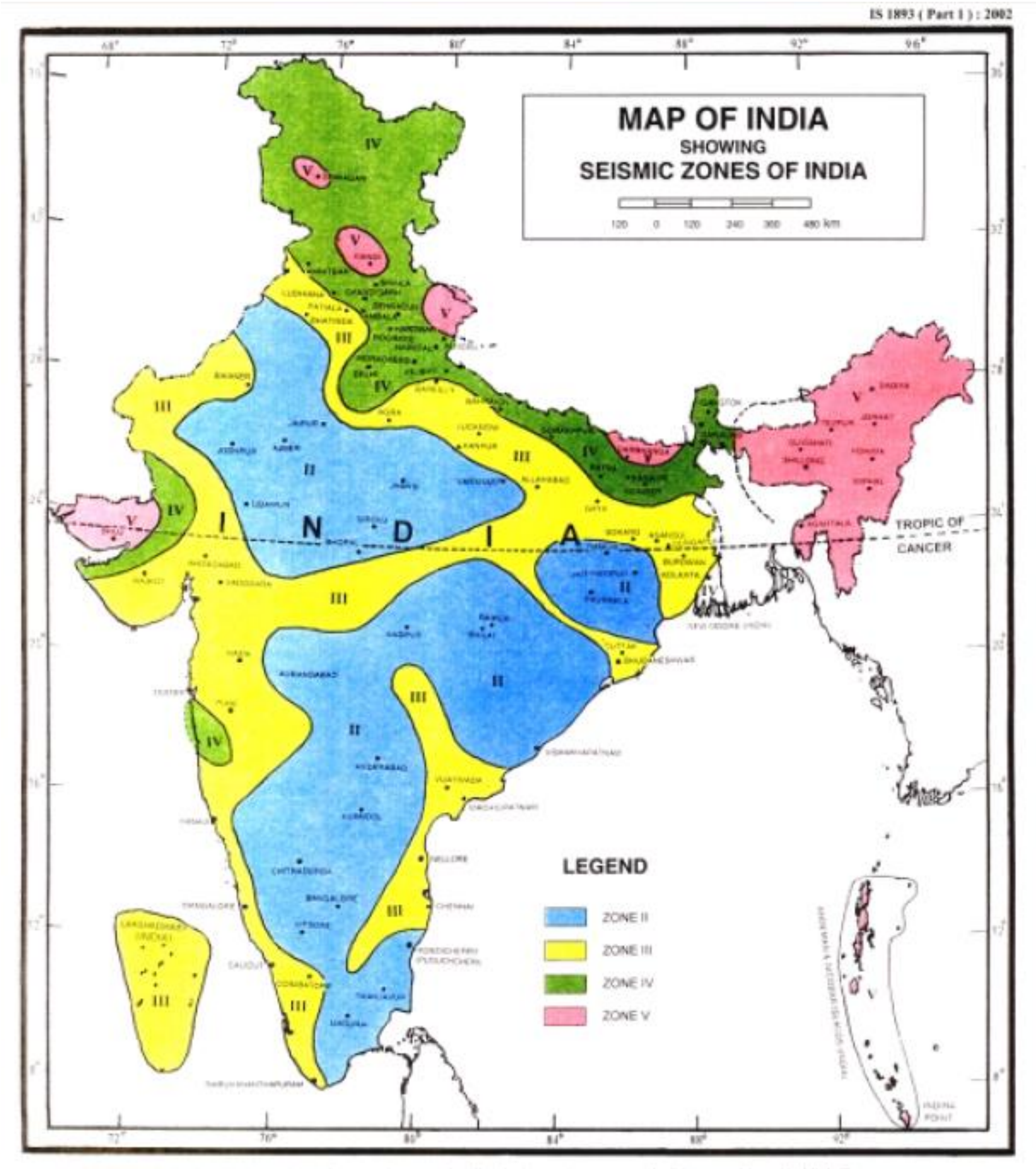
- Grade of - FE415
- Shear modulus - 76.9GPA
- Poison ratio - .3
- Yield stress – 415N/mm^2
- Density – 7690 kg/m^3

5.3 Model Details

A seven storey 3D frame is considered with storey height as 3.1m. A symmetrical building with 3 bays along each X & Y direction is considered with bay width as 4m along each direction. Rigid joints are assumed at beam column junction. Fixed End supports are provided to simulate the actual behaviour of the structure.

5.4 SEISMIC DEFINATION

ZONE – V



ZONE FACTOR - .36

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

IMPORTANCE FACTOR – 1.5

SOIL FACTOR – 2

DAMPING RATIO - .05

TIME PERIOD - .564 SEC

$$T_a = \frac{0.09h}{\sqrt{d}}$$

H- Height of the building

d- Base dimension along the considered direction at the plinth level

Response reduction factor – 5

5.5 LOADING

Loadings are calculated by considering IS 875

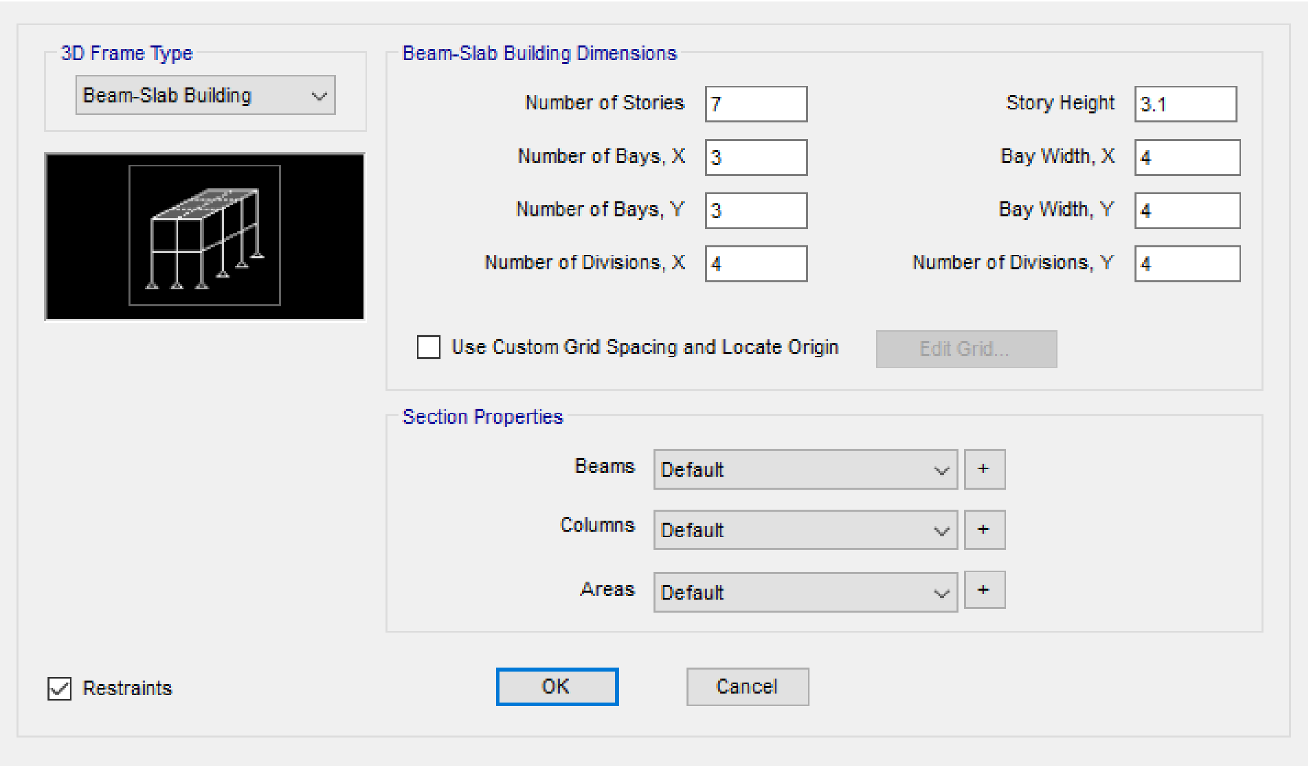
TABLE 5.1 LOADING

DEAD LOAD OF WALL	.23*20 = 4.6 KN/M^2
PLASTER	.016*21 = .336 KN/M^2
PLASTER INT SIDE	.012*21=.252 KN/M^2
TOTAL	5.188 KN/M^2
INT WALL	.115*20=2.3 KN/M^2
TOTAL	2.888 KN/M^2

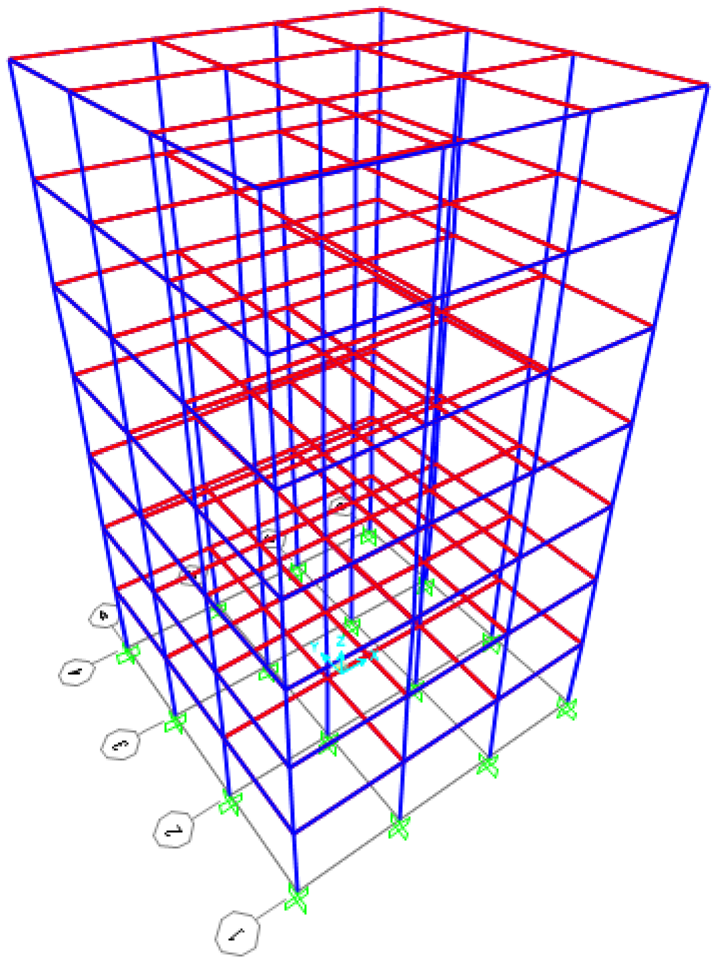
FLOOR FINISH	.05*24=1.2 KN/M^2
SLAB	.115*25=2.875
TOTAL	12.151 KN/M^2
LIVE	2 KN/M^2 (RESIDENTIAL)
LUMPED WEIGHT	DL+.5LL = 13.151 KN/m^2

5.6 ANALYSIS ON SAP2000 (STEP BY STEP PROCEDURE)

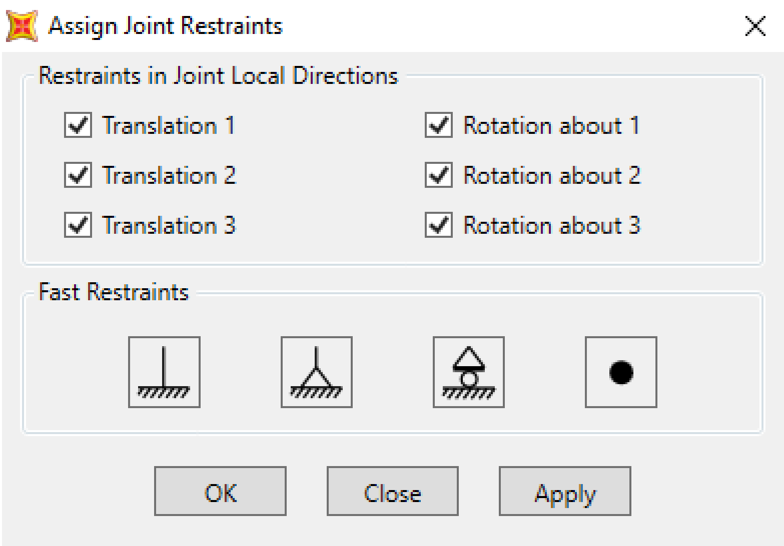
Step 1 – New 3D frame model of 7 storey is selected for analysis as shown below. Three bays are taken along X and Y direction with bay with as 4m along each direction. Storey height is taken as 3.1m.



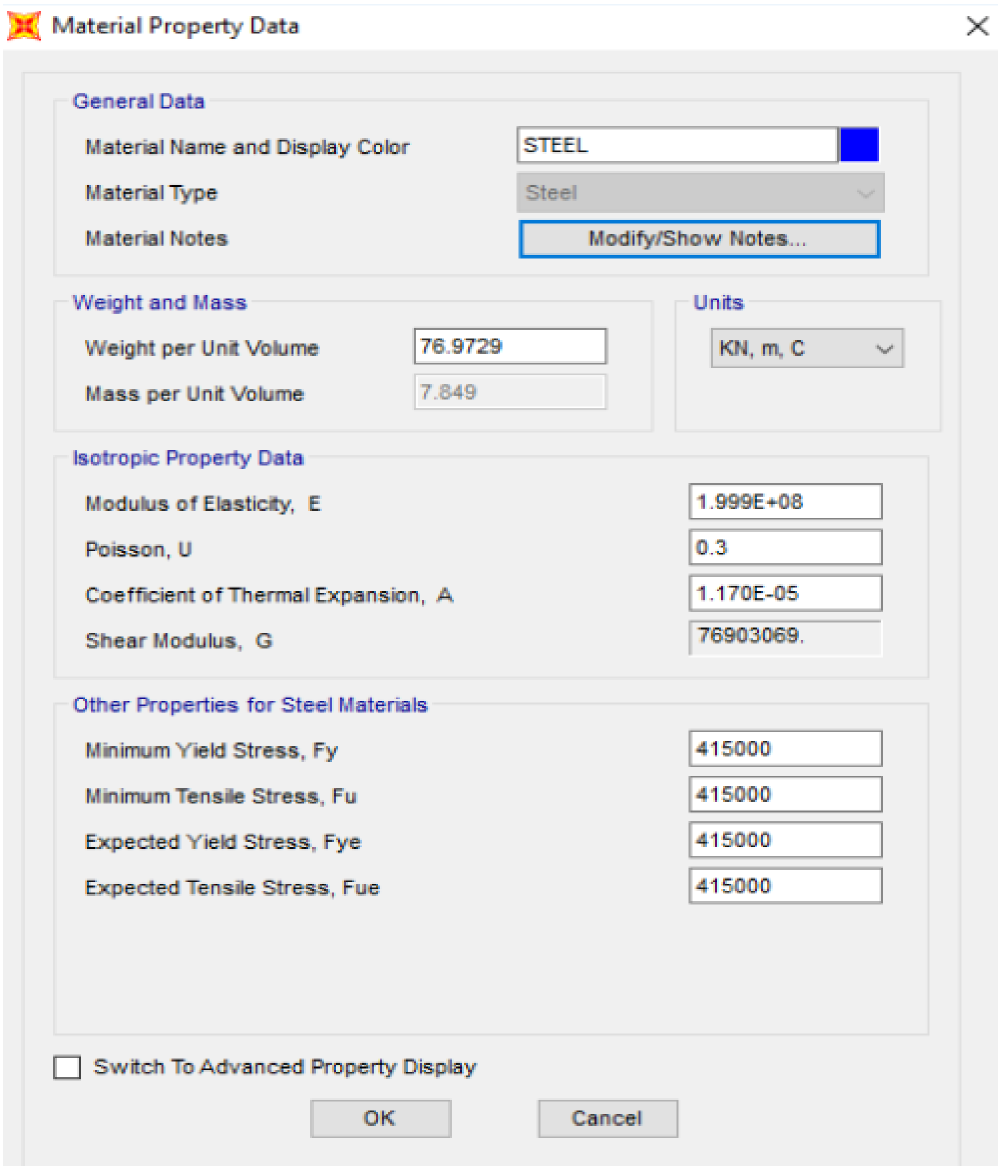
Following diagram shows 3d frame model of the building showing x, y & z direction.

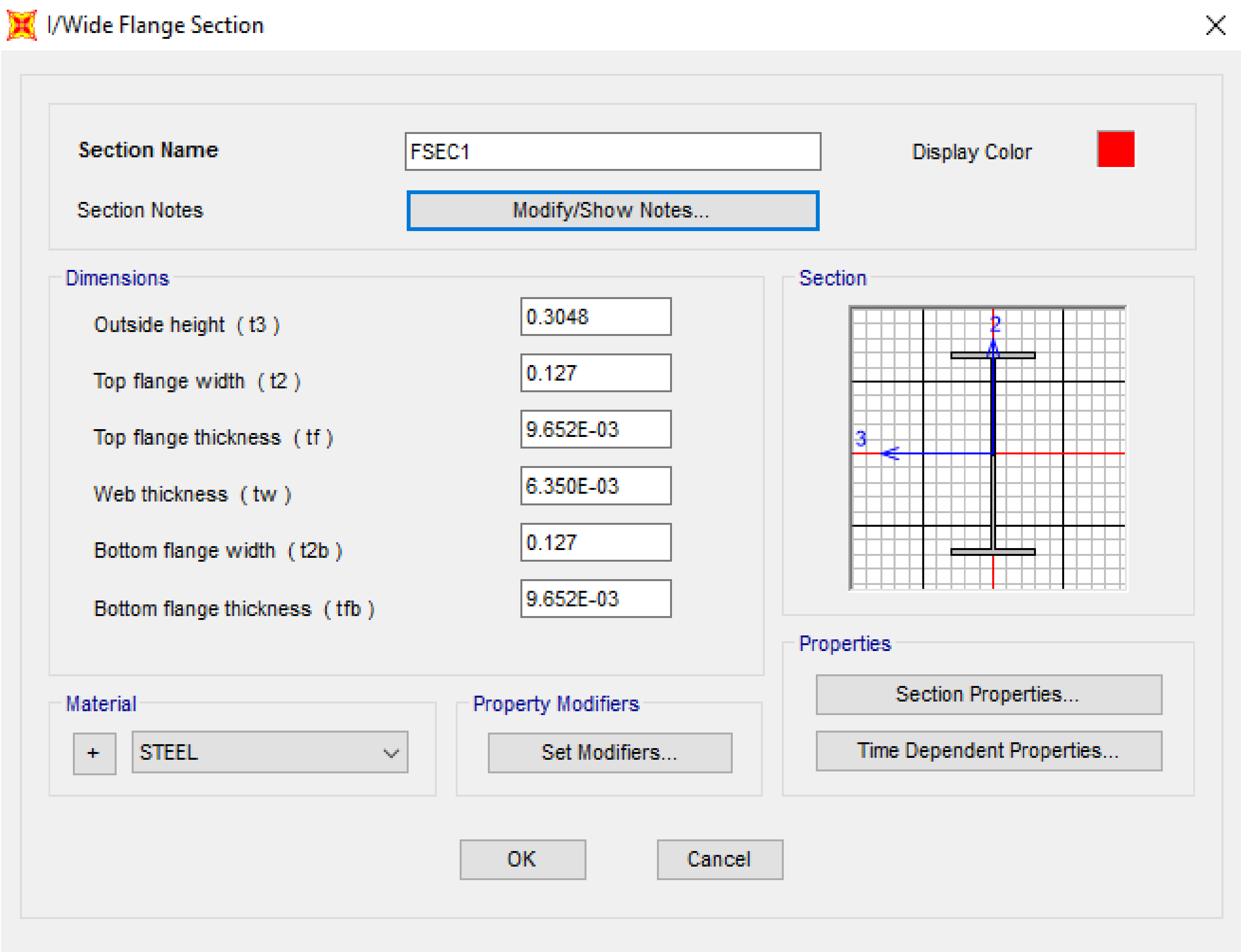


STEP 2 – In the next step, fixed support is provided by restraining translational and rotational movements along different axis. Different types of support can be provided in STAAD pro as fixed, hinged and roller as required.

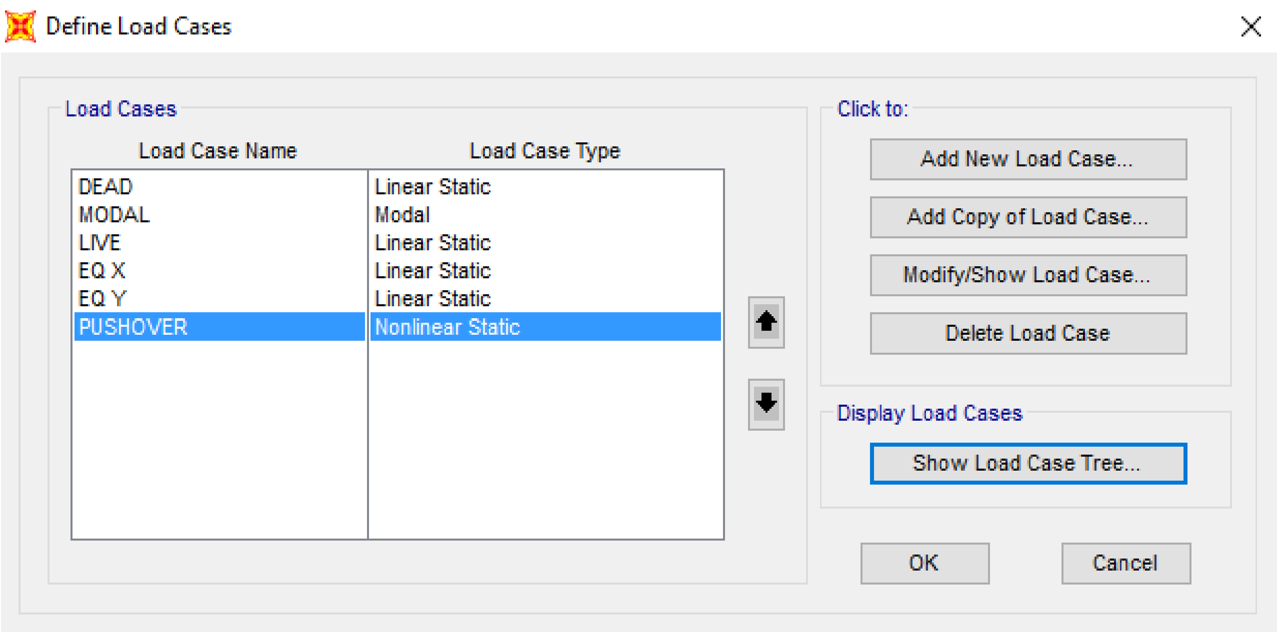


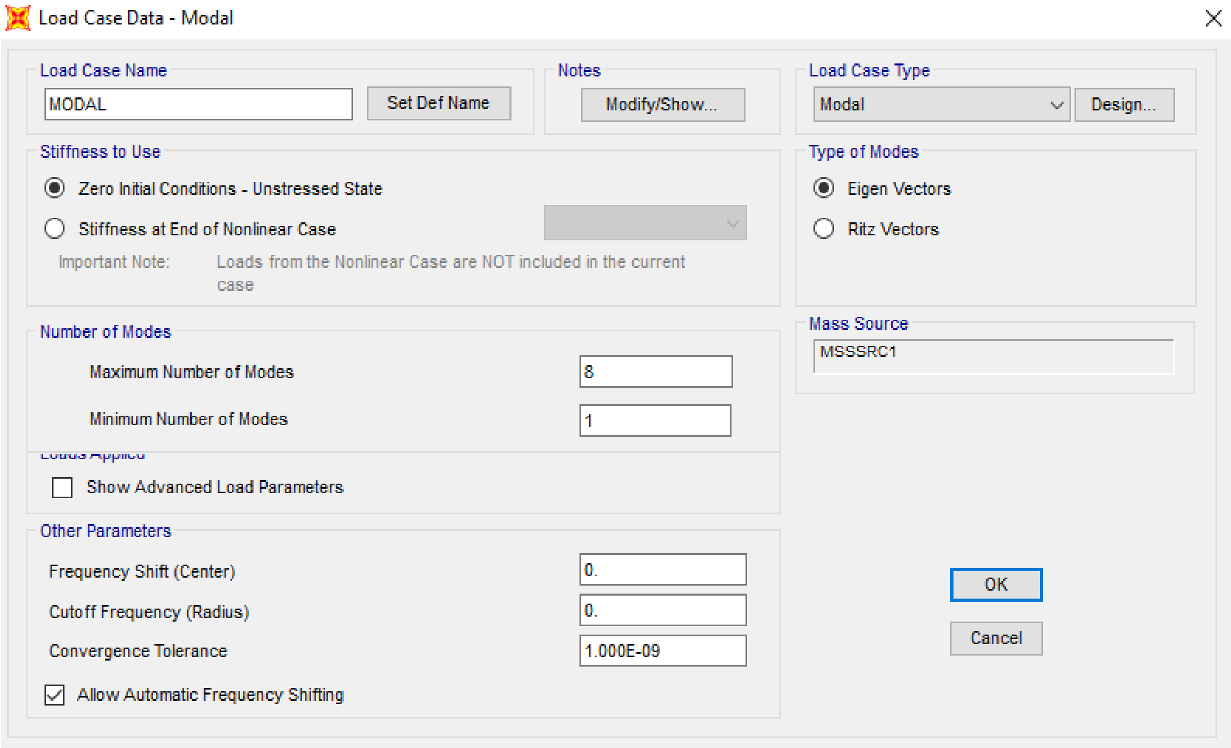
STEP 3 – Material properties are provided in the next step.

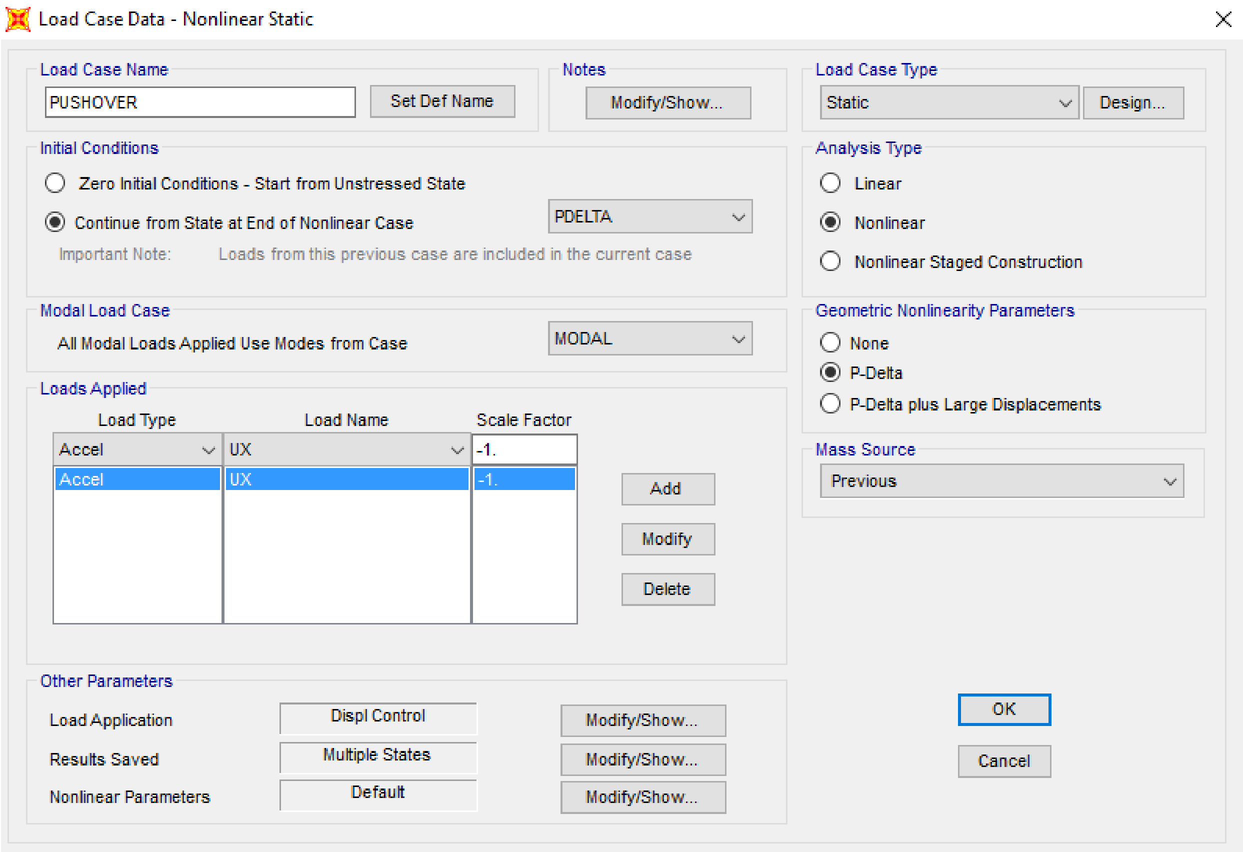


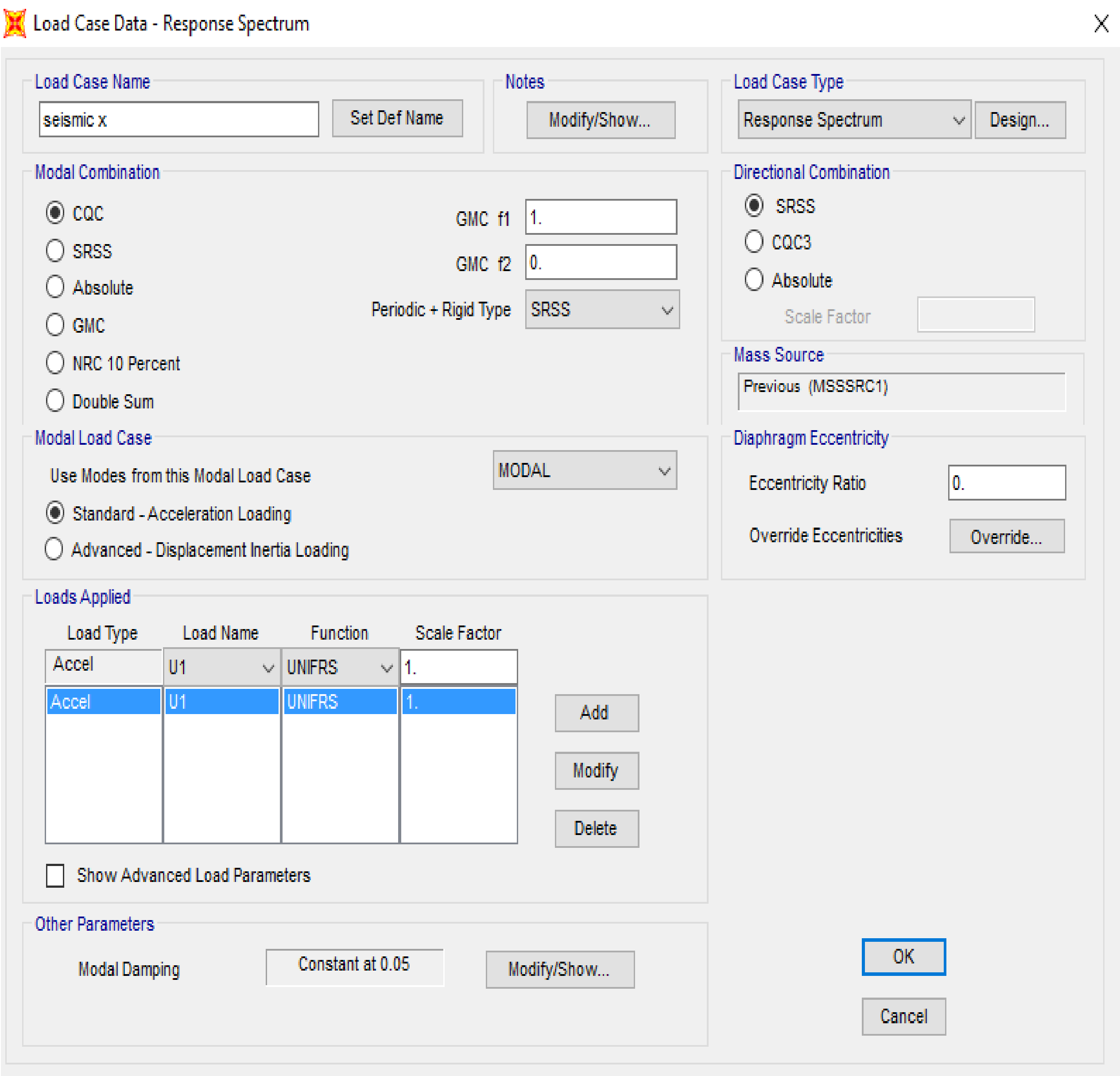


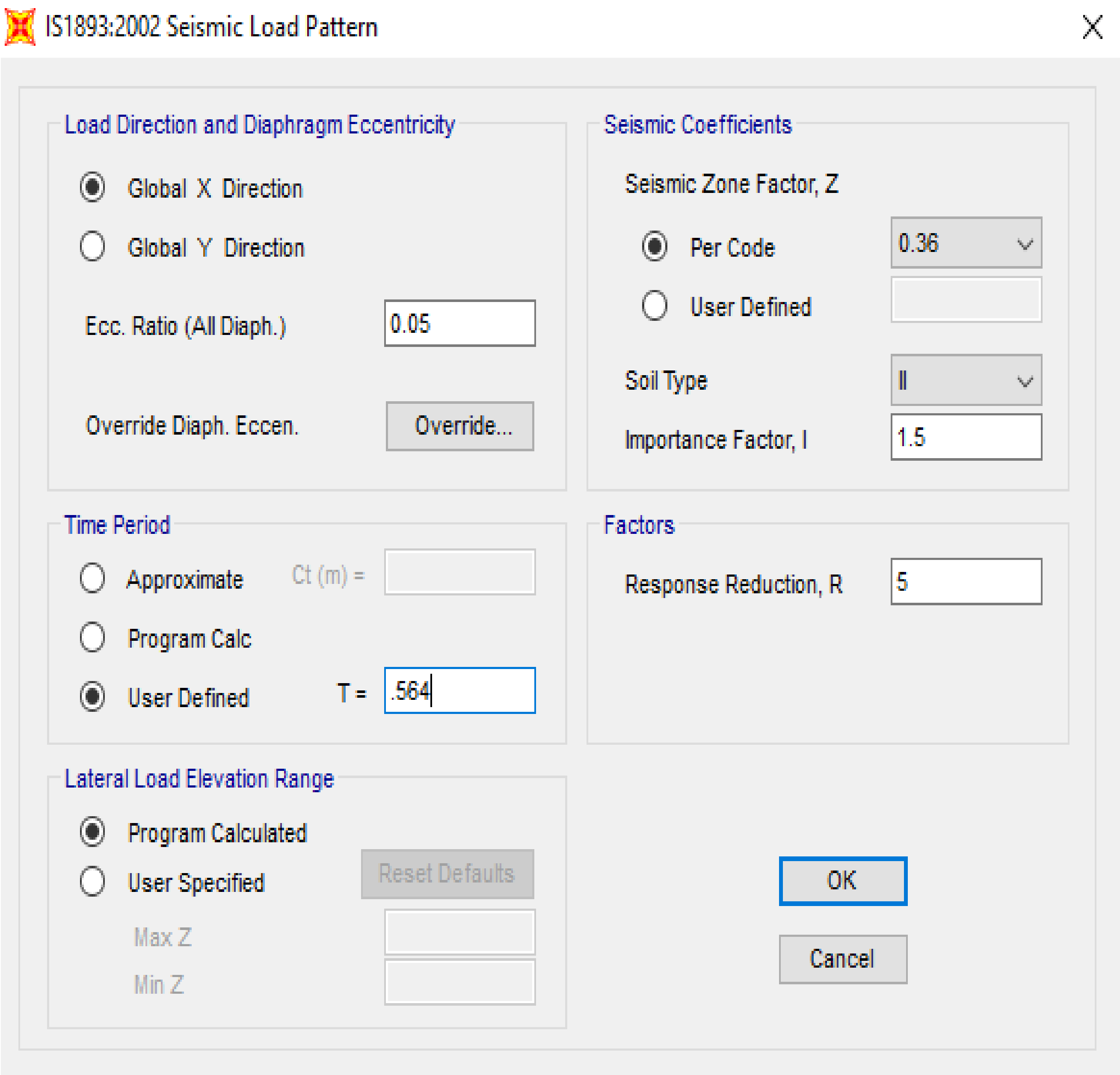
STEP 4 – Load cases are defined here. Load cases include dead, live, Pushover loads etc.



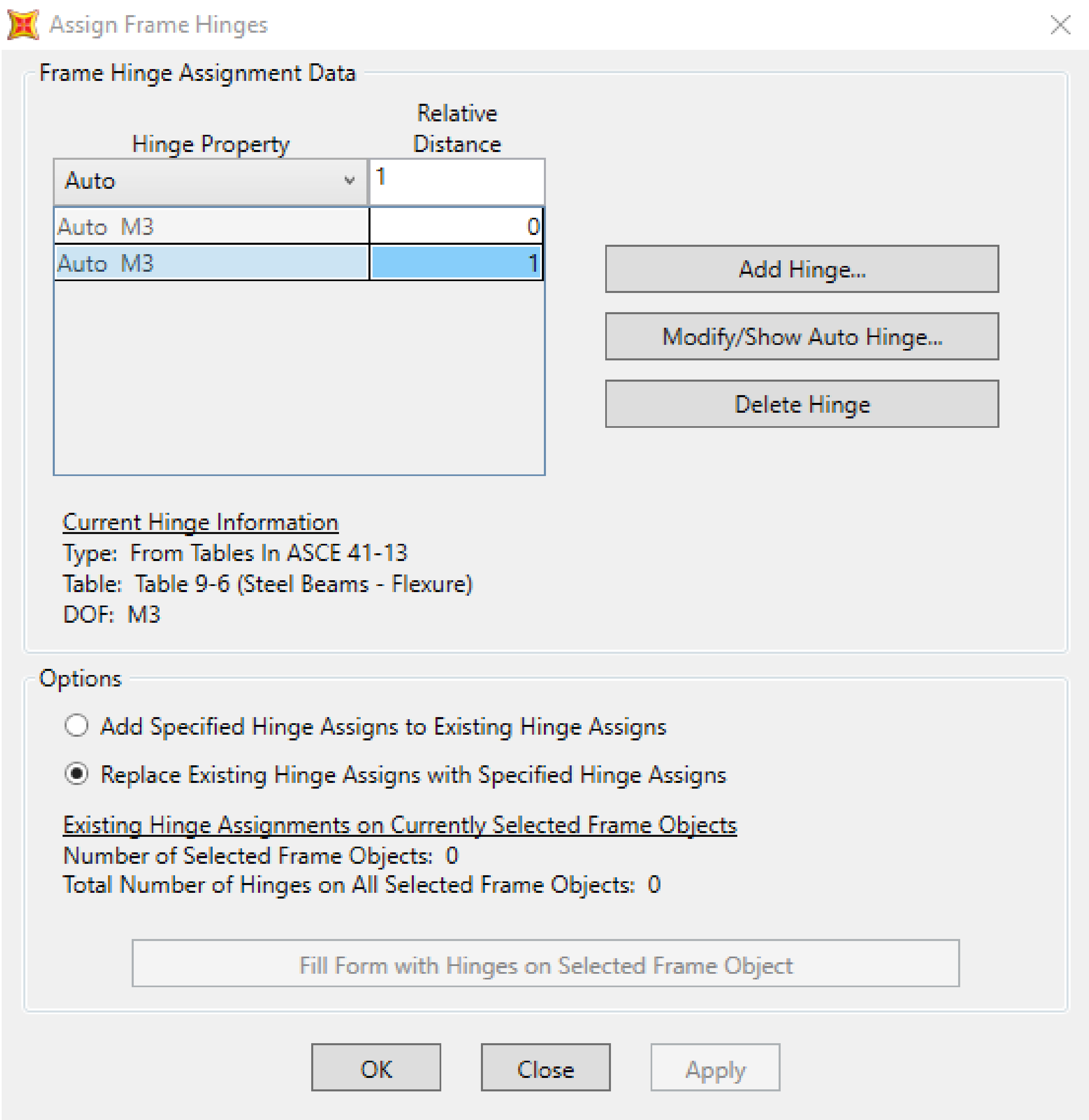

Load Case Data - Modal
Load Case Name
MODAL
Set Def Name
Notes
Modify/Show...
Load Case Type
Modal
Design...
Stiffness to Use
Zero Initial Conditions - Unstressed State
Stiffness at End of Nonlinear Case
Important Note: Loads from the Nonlinear Case are NOT included in the current case
Type of Modes
Eigen Vectors
Ritz Vectors
Number of Modes
Maximum Number of Modes
8
Minimum Number of Modes
1
Mass Source
MSSSRC1
Show Advanced Load Parameters
Other Parameters
Frequency Shift (Center)
0.
Cutoff Frequency (Radius)
0.
Convergence Tolerance
1.000E-09
Allow Automatic Frequency Shifting
OK
Cancel

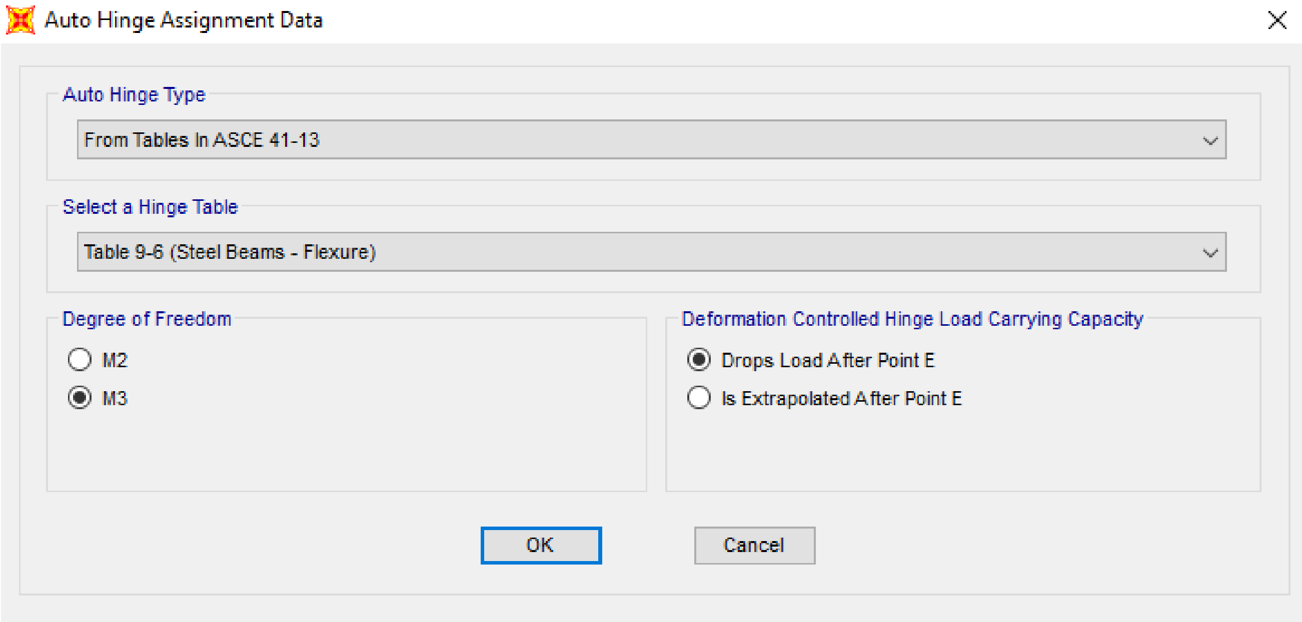

Load Case Data - Nonlinear Static
Load Case Name
PUSHOVER
Set Def Name
Notes
Modify/Show...
Load Case Type
Static
Design...
Initial Conditions
Zero Initial Conditions - Start from Unstressed State
Continue from State at End of Nonlinear Case
PDELTA
Important Note: Loads from this previous case are included in the current case
Analysis Type
Linear
Nonlinear
Nonlinear Staged Construction
Modal Load Case
All Modal Loads Applied Use Modes from Case
MODAL
Geometric Nonlinearity Parameters
None
P-Delta
P-Delta plus Large Displacements
Loads Applied
Load Type
Load Name
Scale Factor
Accel
UX
-1.
Accel
UX
-1.
Add
Modify
Delete
Mass Source
Previous
Other Parameters
Load Application
Displ Control
Modify/Show...
Results Saved
Multiple States
Modify/Show...
Nonlinear Parameters
Default
Modify/Show...
OK
Cancel

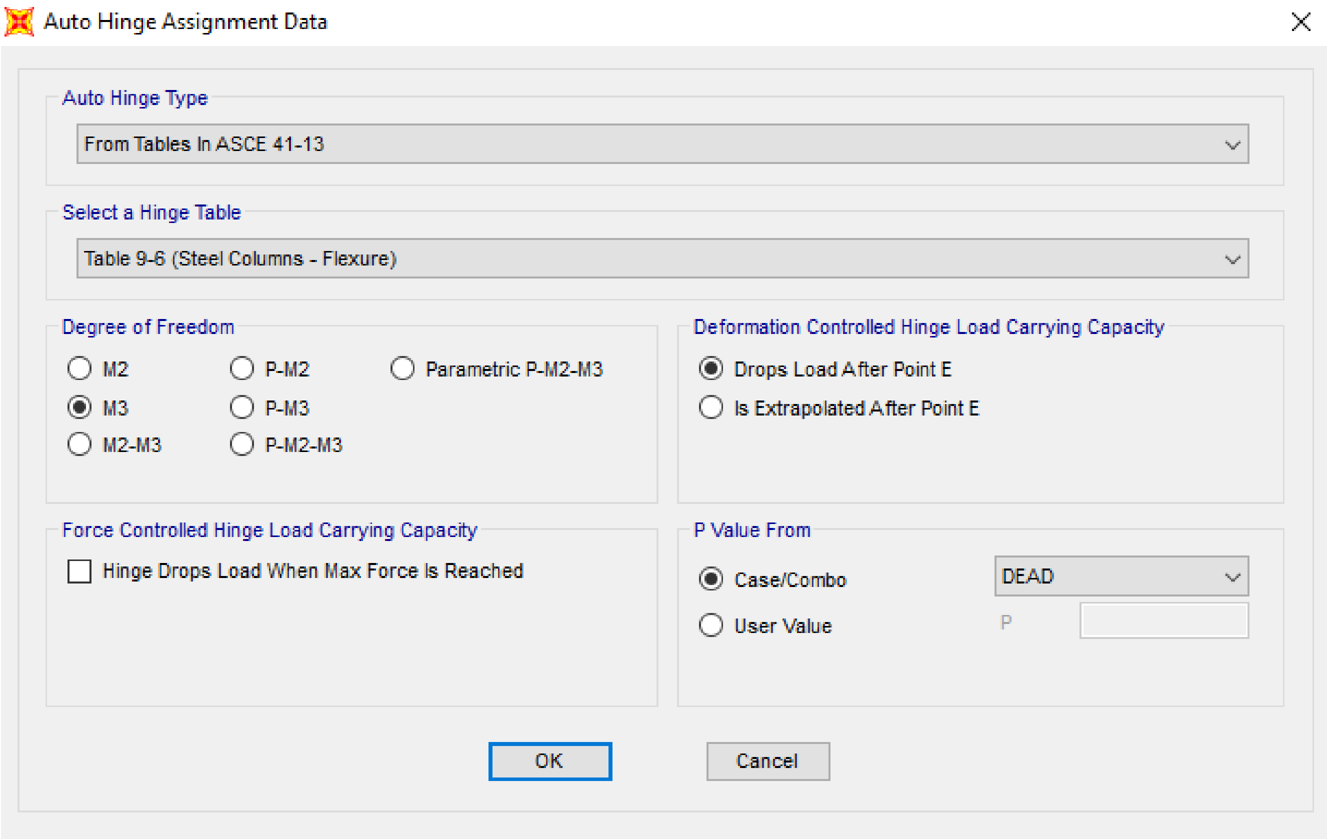

Load Case Data - Response Spectrum
Load Case Name
seismic x
Set Def Name
Notes
Modify/Show...
Load Case Type
Response Spectrum
Design...
Modal Combination
CQC
SRSS
Absolute
GMC
NRC 10 Percent
Double Sum
GMC f1
1.
GMC f2
0.
Periodic + Rigid Type
SRSS
Directional Combination
SRSS
CQC3
Absolute
Scale Factor
Mass Source
Previous (MSSSRC1)
Modal Load Case
Use Modes from this Modal Load Case
MODAL
Standard - Acceleration Loading
Advanced - Displacement Inertia Loading
Diaphragm Eccentricity
Eccentricity Ratio
0.
Override Eccentricities
Override...
Loads Applied
Load Type
Load Name
Function
Scale Factor
Accel
U1
UNIFRS
1.
Accel
U1
UNIFRS
1.
Add
Modify
Delete
Show Advanced Load Parameters
Other Parameters
Modal Damping
Constant at 0.05
Modify/Show...
OK
Cancel



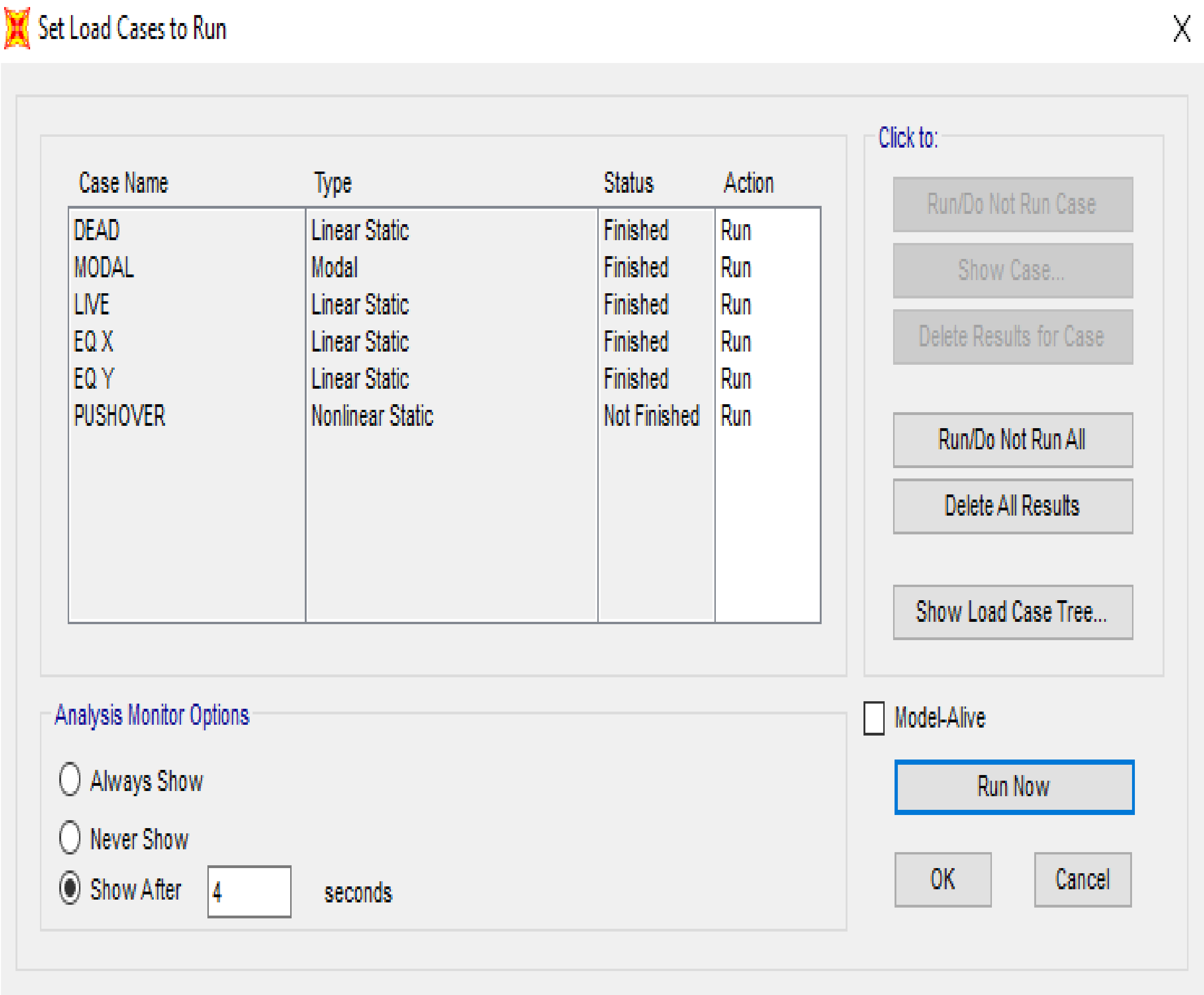
STEP 6 – This step includes providing information regarding hinges. Hinges are provided at ends of beams and columns.


Assign Frame Hinges
Frame Hinge Assignment Data
Hinge Property
Relative Distance
Auto
1
Auto M3
0
Auto M3
1
Add Hinge...
Modify/Show Auto Hinge...
Delete Hinge
Current Hinge Information
Type: From Tables In ASCE 41-13
Table: Table 9-6 (Steel Beams - Flexure)
DOF: M3
Options
Add Specified Hinge Assigns to Existing Hinge Assigns
Replace Existing Hinge Assigns with Specified Hinge Assigns
Existing Hinge Assignments on Currently Selected Frame Objects
Number of Selected Frame Objects: 0
Total Number of Hinges on All Selected Frame Objects: 0
Fill Form with Hinges on Selected Frame Object
OK
Close
Apply



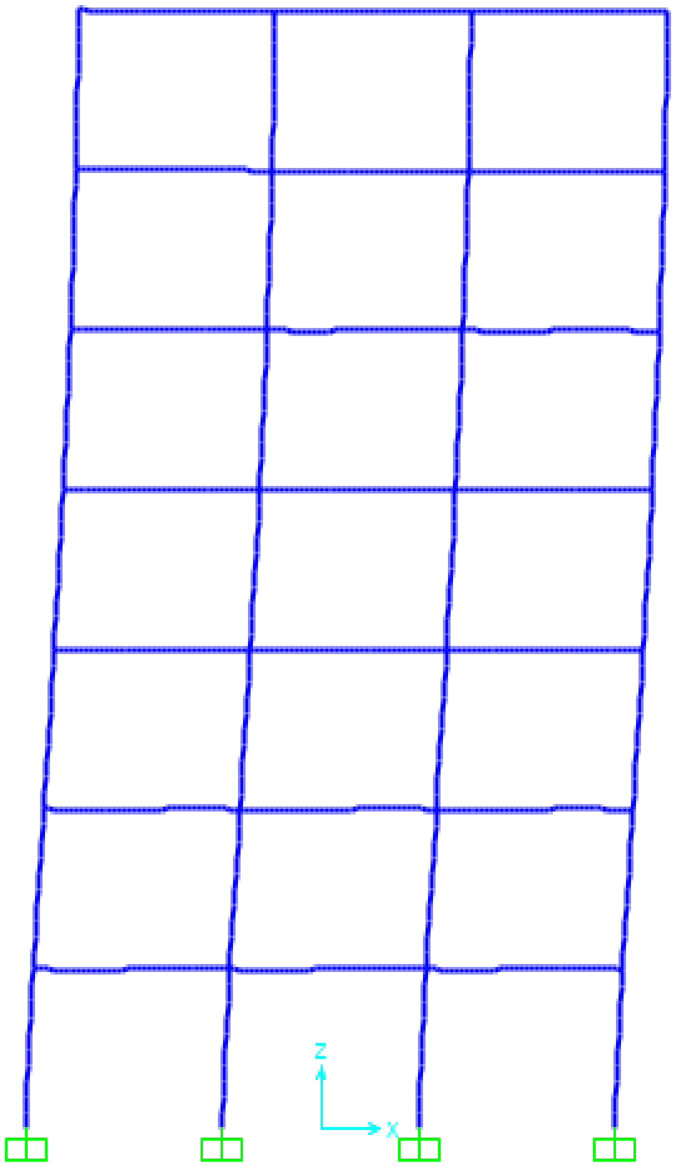


STEP 7 – “Run Analysis” command is given in the last step .


Set Load Cases to Run
Case Name
Type
Status
Action
DEAD
Linear Static
Finished
Run
MODAL
Modal
Finished
Run
LIVE
Linear Static
Finished
Run
EQ X
Linear Static
Finished
Run
EQ Y
Linear Static
Finished
Run
PUSHOVER
Nonlinear Static
Not Finished
Run
Click to:
Run/Do Not Run Case
Show Case...
Delete Results for Case
Run/Do Not Run All
Delete All Results
Show Load Case Tree...
Analysis Monitor Options
Always Show
Never Show
Show After
4
seconds
Model-Alive
Run Now
OK
Cancel

CHAPTER 6 RESULTS AND CONCLUSIONS

6.1 Deformed shape



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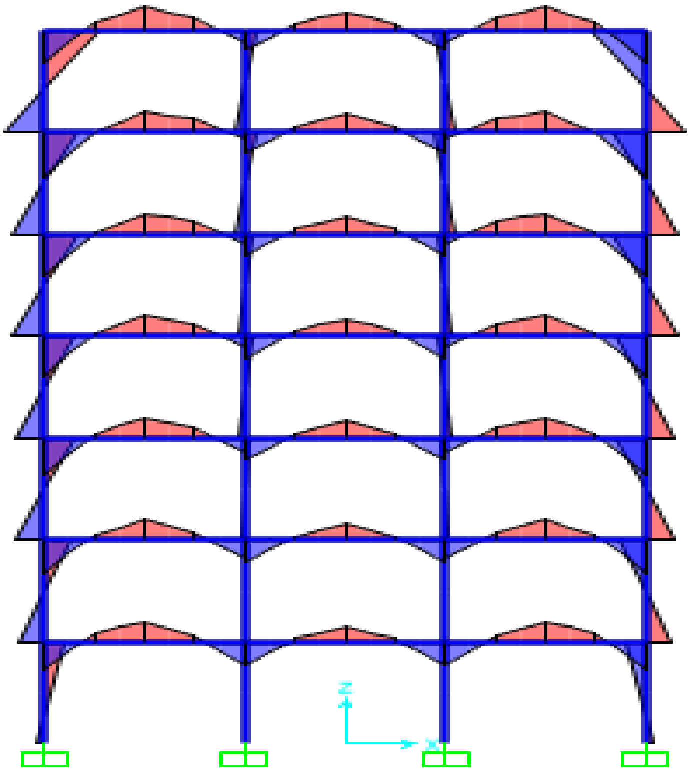
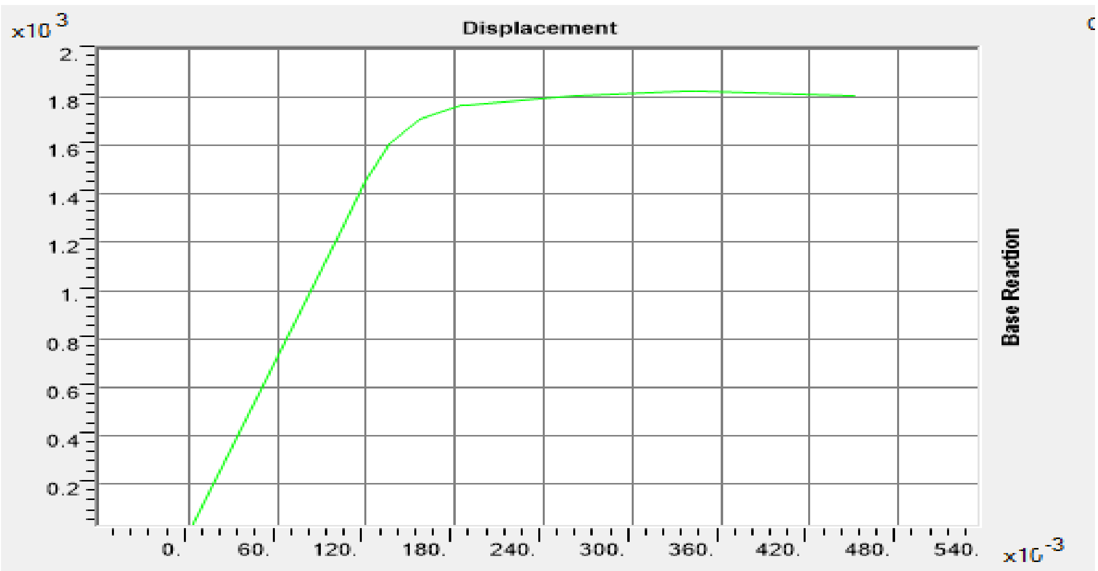
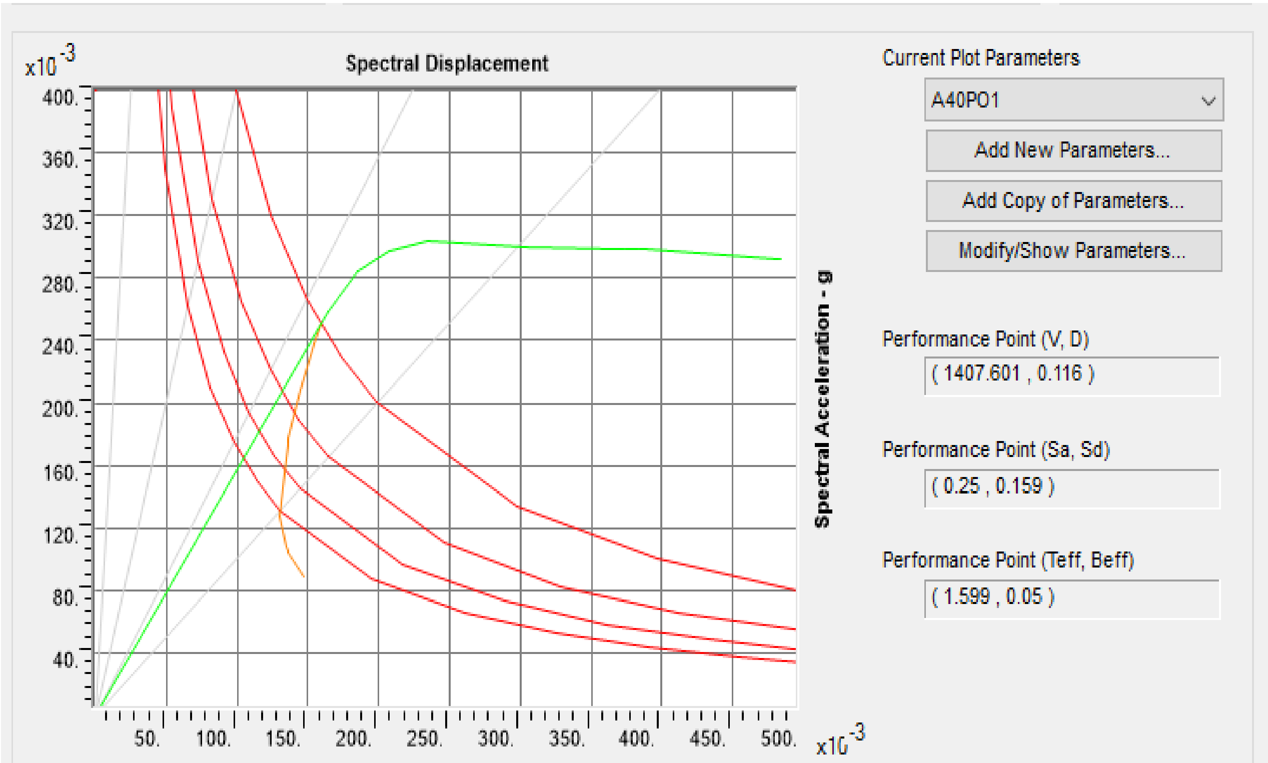
6.2 Bending moment diagram



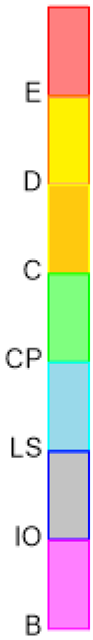
6.3 Base shear vs displacement



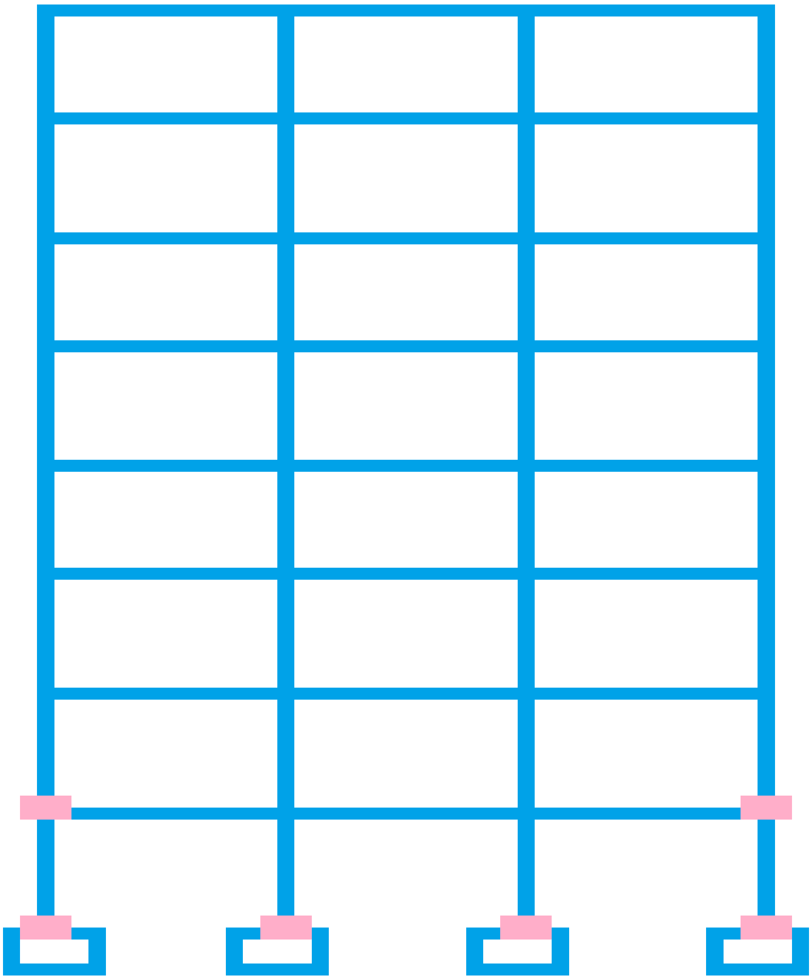
6.4 Pushover Curve



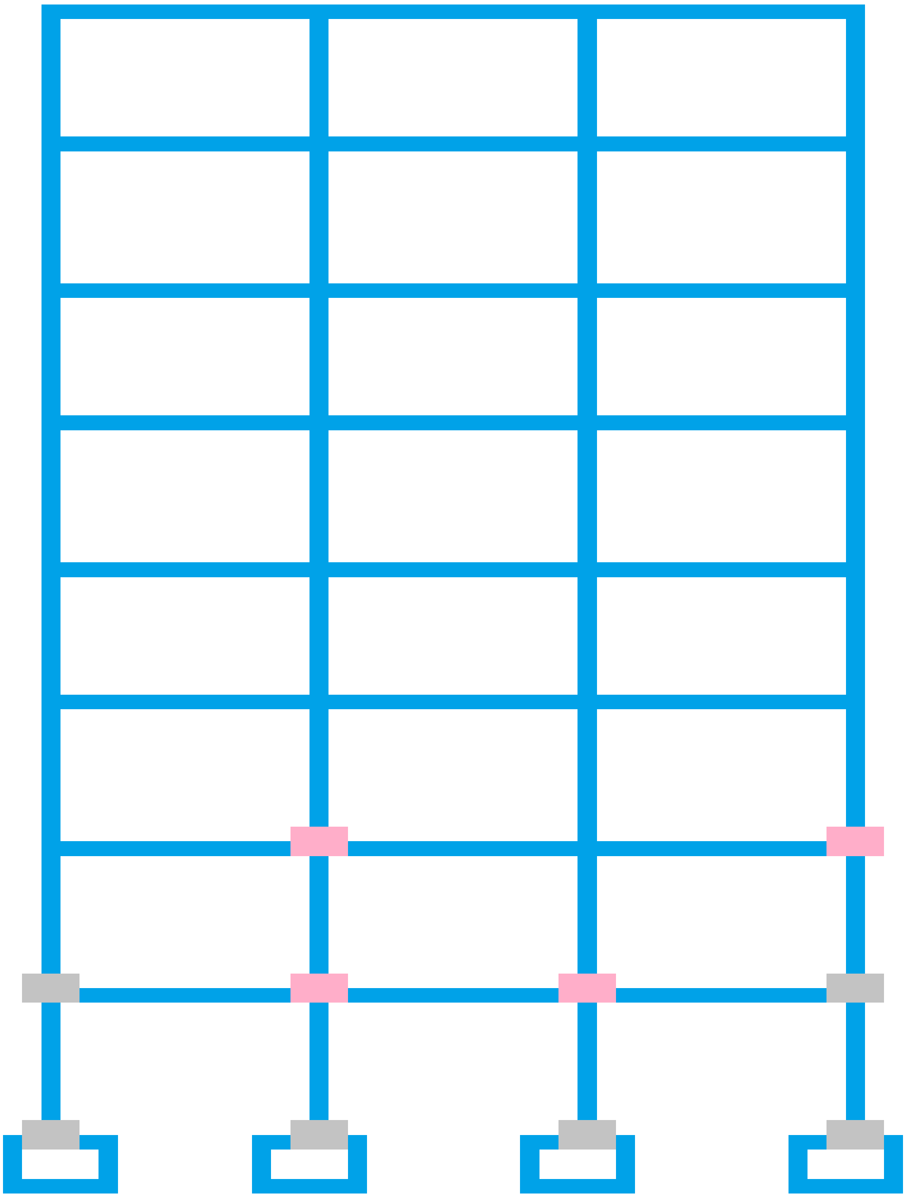
6.5 Hinge Results (Step by step)



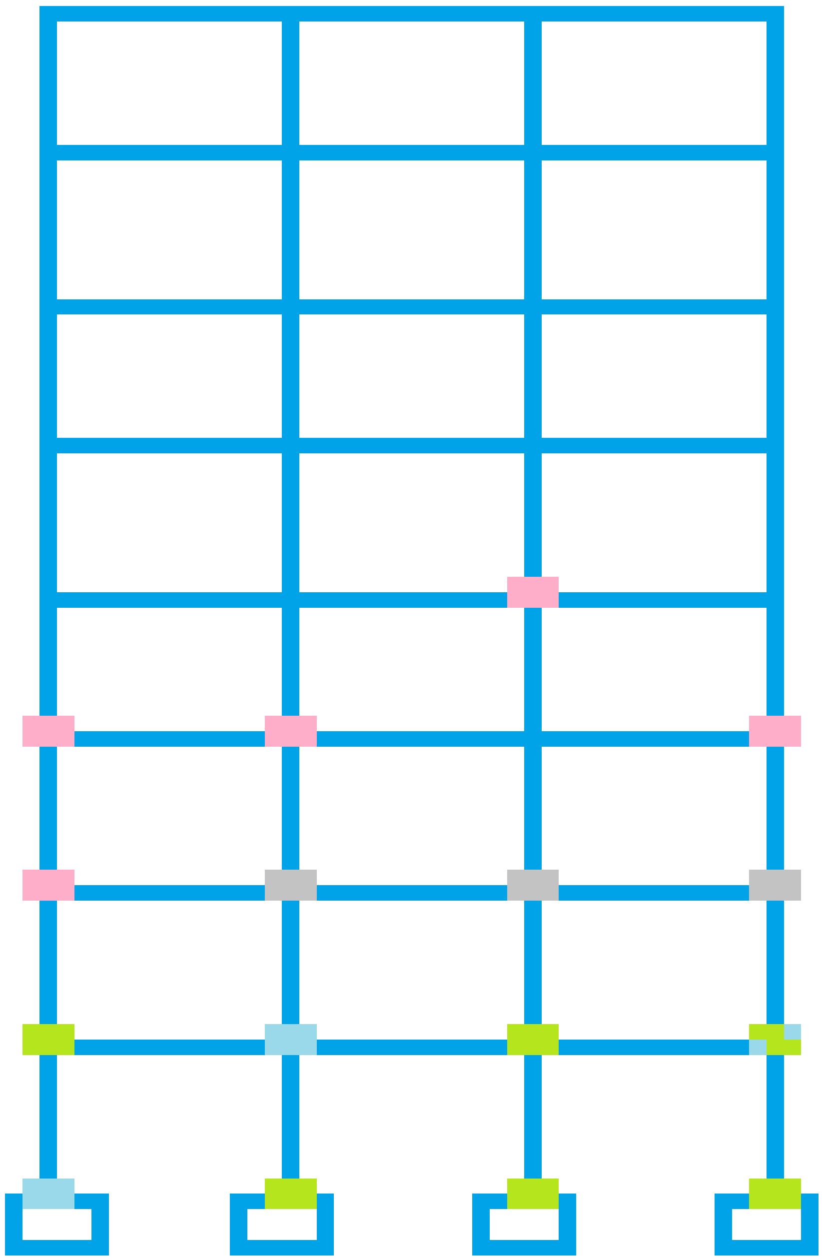
Step 1



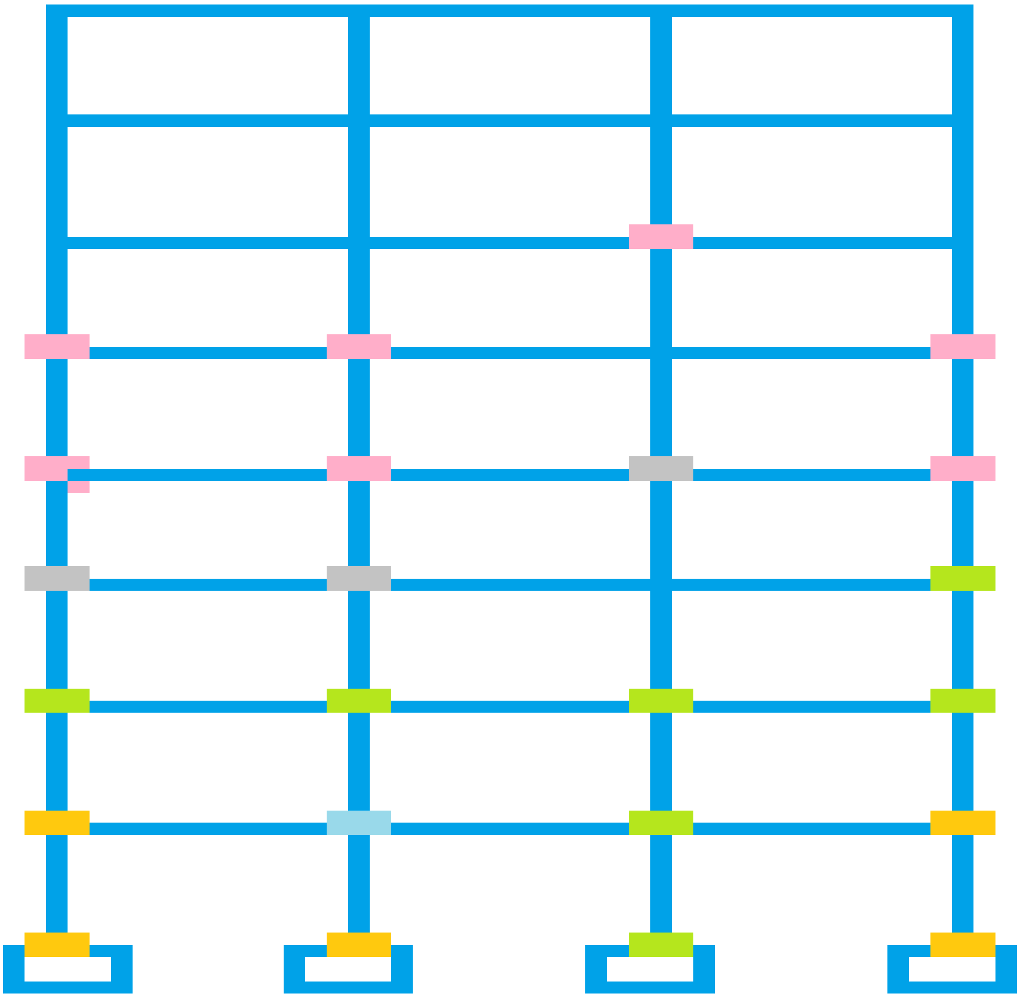
Step 2



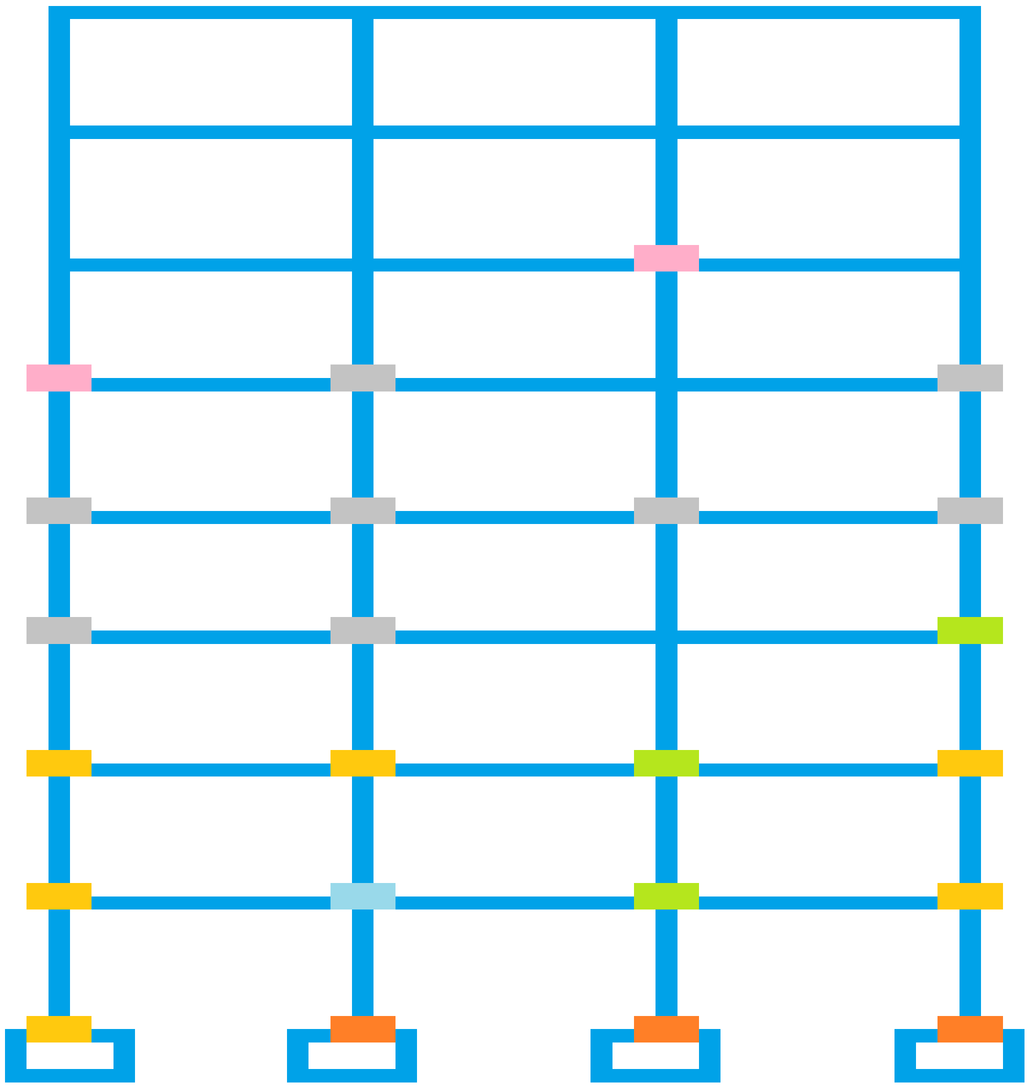
STEP 3



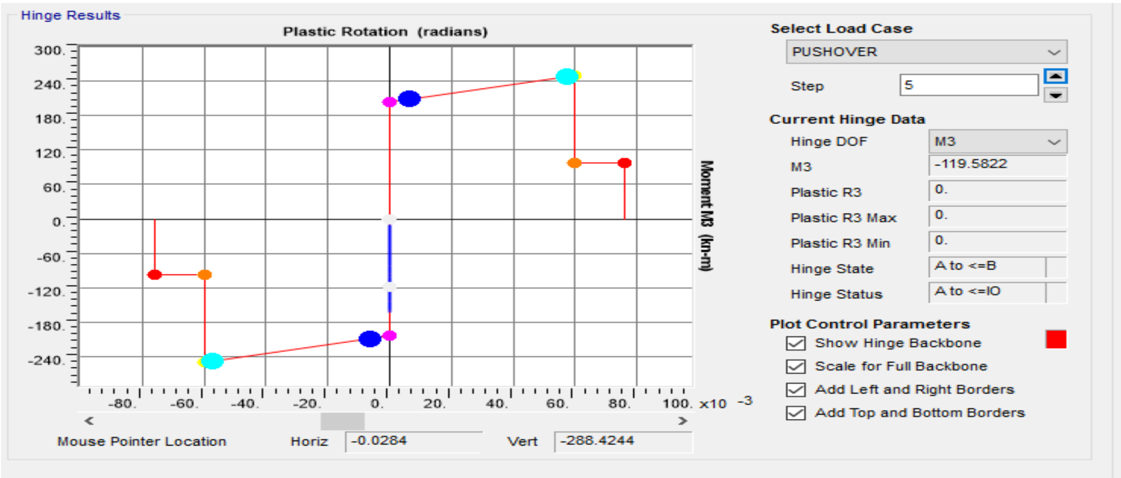
Step 4

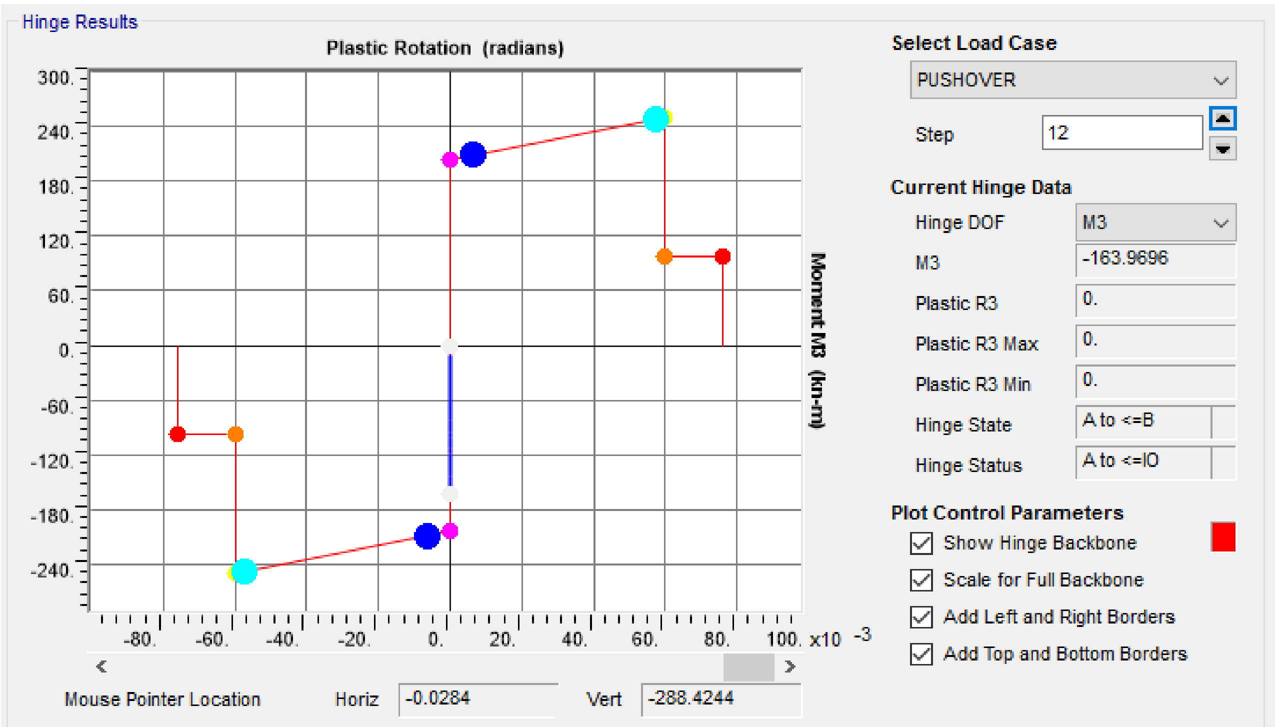


Step 5



Moment vs Plastic Rotation curve





CONCLUSIONS

- Pushover analysis is relatively a simple method to analyse the structure in non-linear range. Weak elements in the structure can be identified with the help of pushover analysis which also accounts for redistribution of the forces. However, Pushover analysis may not accurately represents dynamic behaviour of the structure as it is an approximate method based on static loading. The efforts needed for computation process and interpretation of results are much less as compared to other methods of non linear analysis.
- Formation of the hinges starts at the supports and progressively moves towards the upper stories with the increment of load. Step By step development of hinges is depicted in results. Moment vs Plastic rotation curve is also depicted.
- The general idea about the behaviour of the structure is depicted through plastic hinges, demand and capacity of the structure.
- Behaviour of the frame is linearly elastic upto base shear value of 1600 KN. Further it shows a nonlinear behaviour upto the base shear value of 1800 KN. After that it shows a complete plastic behaviour.

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