CHAPTER 1

1.1 Introduction

To perform seismic evaluation of a residential building using pushover analysis and provide method for retrofitting of members in case the members fail under the load combinations prescribed in IS 1893-2002.The methodology adopted to perform the seismic evaluation of the building requires an understanding of equivalent lateral force procedure also recognized as equivalent static procedure in literature. And in depth knowledge of ETAB software is required as the building was modeled in ETAB and post analysis data obtained from it was used in the analysis of the structure. The various hinges formation and capacity curve of members was calculated as per FEMA and ATC-40 approach to analyze the seismic stability of the structure under the various load combinations in accordance with IS 1893-2002 (part 1) .Suitable retrofit measures were proposed for beams and columns failing in shear and flexure

1.1.1 Scope of the study:

In the response spectrum procedure of seismic analysis, the seismic loads are applied to the centre of mass of the storey, but in ETAB, I have assumed the seismic loads to be nodal loads and applied it to nodes dividing the total lateral storey loads in equal proportion per node and not at the exact centre of mass of the storey. While considering retrofit measures for the structure, analysis of structure post concrete jacketing was kept outside the scope of this study.

1.1.2 Basics of Seismic evaluation:

Response spectra:

Interaction between ground accelerations and structural systems are reported through response spectrum. Plots peak responses over time for a range of single-degree-of-freedom (SDOF) systems subjected to a particular base motion as a function of their natural frequency ω_i, or vibration period T_i. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. Response spectra are used by earthquake engineers for analyzing the performance of structures in earthquakes, since many behave principally as single degree of freedom systems. The purpose of the response spectrum is to know the response of a single degree of freedom system if the ground moves as per the given accelerogram. An accelerogram is the recording of the acceleration of the ground during an earthquake. Response may mean any quantity like acceleration, velocity or deformation[1,2].

The figure below shows the accelerogram for the earthquake that hit the El-Centro city in Imperial valley of California.
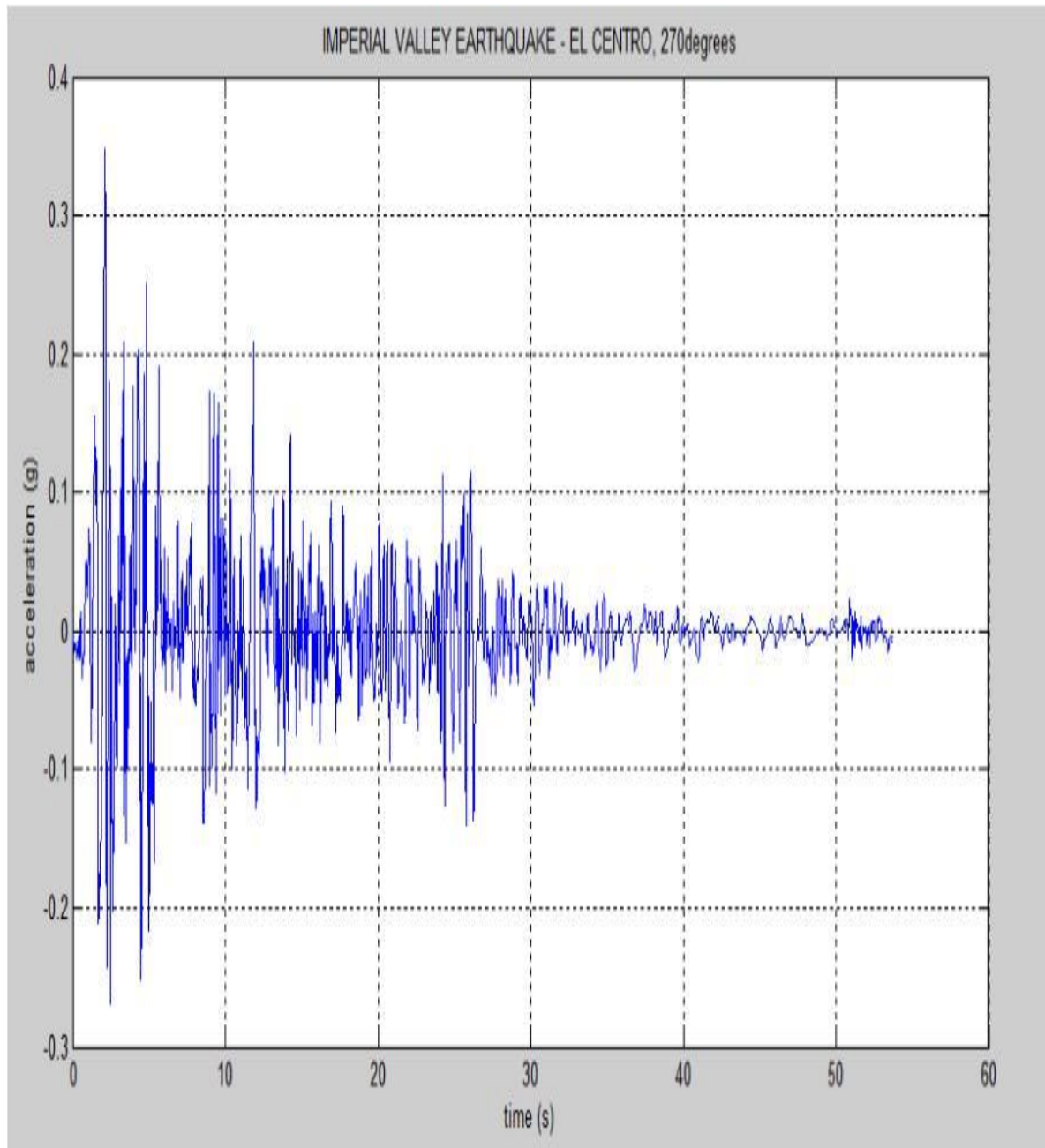

Figure 1: Accelerogram for the earthquake affecting El-Centro city in 1940 [3]

The strength depends on the following factors:

- Frequency

- Maximum velocity

- Maximum displacement

- Maximum acceleration

Design spectrum must also be accompanied by:

Load factors, as different choices of load factors will lead to different seismic safety of the structure. Damping, variations in the values of damping used in the design will affect the design force. Method of calculation of natural period of the structures, which depends upon the assumptions made while modeling.[2,4]
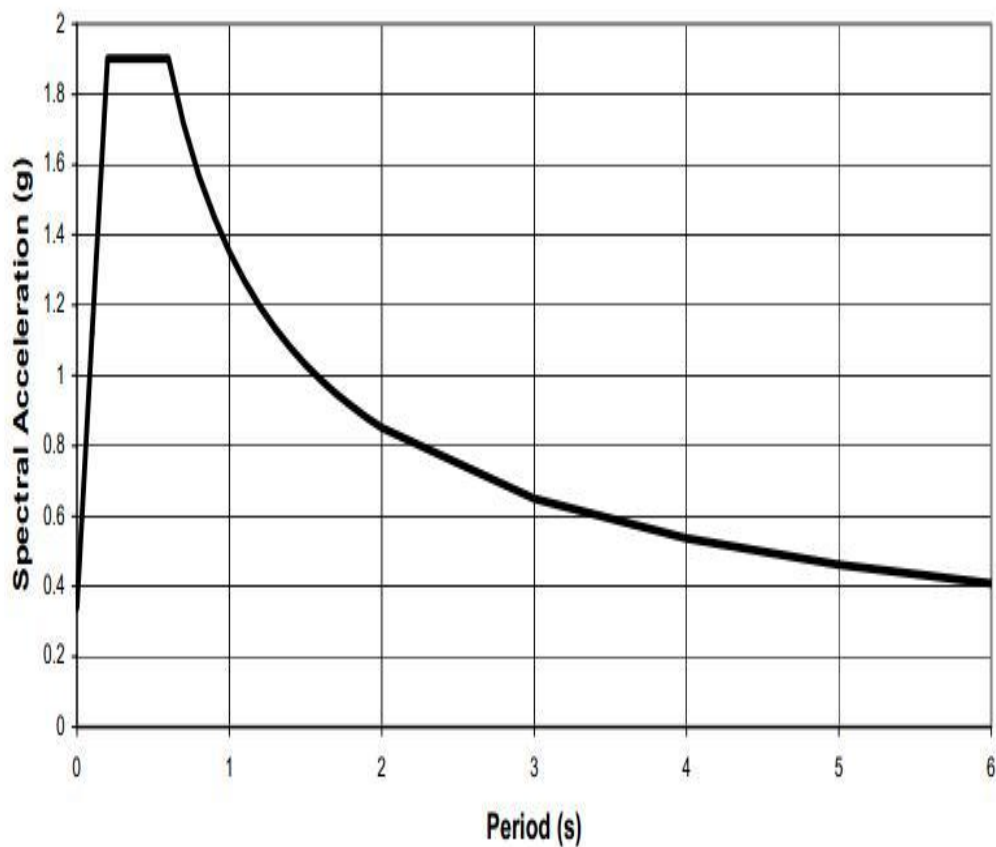

Figure 2: A typical design spectrum [5]

Pushover analysis

The pushover analysis of a high rise building is a static non-linear analysis under permanent vertical loads and monotonically increasing transverse loads. The equivalent static transverse loads represent earthquake induced forces. Premature failure can be represented by A graph of the total base shear vs top displacement in a building .failure can be analyzed by this method .Analysis is done upto failure. Thus collapse load and ductility capacity can be determined by pushover analysis.   In pushover analysis plastic rotation is monitored. The lateral inelastic forces vs displacement graph of the complete building is computed. This analysis is helpful in weakness in building to be identified. To make the decision to retrofit can be done in such studies. The earthquake design can be done as a two step process. The first one which is basically most important , is the concept of an effective structural system that needs to be configured with due care to all very important earthquake performance targets, ranging from serviceability considerations. The thumb rules for the strength and stiffness goals is based on the understanding of ground motion, elastic and inelastic dynamic response characteristics, should to be enough to configure and rough-size an effective structural system. Detailed mathematical or physical models can only be made when a structural system has been created. Such models are needed to evaluate earthquake performance of an existing building. They also improve component behavior characteristics (stiffness, strength, deflection capacity) for better fit the specified performance conditions. The 2nd step consists of the design process that contains demand/capacity evaluation at all important capacity parameters, as well as the prediction of demands imposed by ground motions.. The distribution of design forces are based on initial estimate of   stiffness and as stiffness is dependent on the strength of elements, this cannot be know until the design process is complete. Distribution of seismic forces between elements based on initial stiffness is illogical, as it incorrectly assumes different elements can be forced to yield simultaneously

1.2 Statement of the problem:

- The distribution of design forces are based on initial estimate of   stiffness and as stiffness is dependent on the strength of elements, this cannot be known until the design process is complete

- The distribution of design forces are based on initial estimate of   stiffness and as stiffness is dependent on the strength of elements, this cannot be known until the design process is complete.

- Distribution of seismic forces between elements based on initial stiffness is illogical, as it incorrectly assumes different elements can be forced to yield simultaneously

1.3 Objectives

- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.

- Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.

- Estimates of the inter story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.
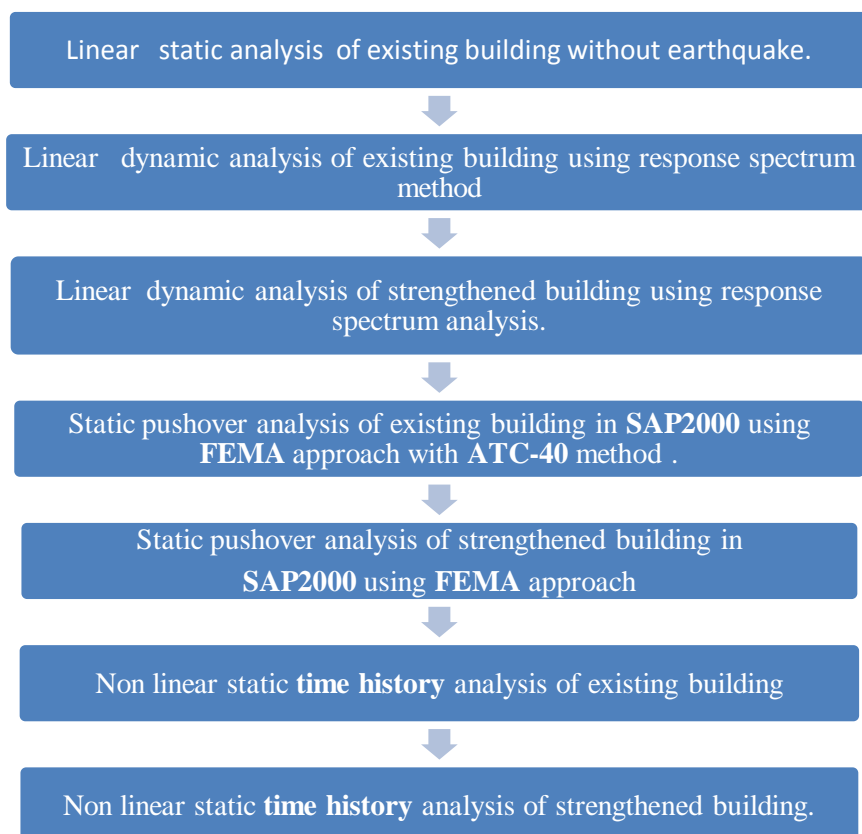
Work flow chart

Linear static analysis of existing building without earthquake.
Linear dynamic analysis of existing building using response spectrum method
Linear dynamic analysis of strengthened building using response spectrum analysis.
Static pushover analysis of existing building in **SAP2000** using **FEMA** approach with **ATC-40** method .
Static pushover analysis of strengthened building in **SAP2000** using **FEMA** approach
Non linear static **time history** analysis of existing building
Non linear static **time history** analysis of strengthened building.

Figure 3

CHAPTER 2

2.1 Litrature review

Appropriate capacity parameters and their permissible values, as well as suitable methods for demand prediction will depend on the performance level to be evaluated. The application of this solution requires the availability of as set of ground motion records (each with three components) that account for the uncertainties and differences in severity, frequency characteristics, and duration due rapture characteristics distances of the various faults that may cause certain ground motions at the field condition. It needs further the ability to model adequately the cyclic load-deflection characteristics of all important components of the three dimensional soil foundation structural system, and the availability of efficient tools to apply the solution process within the time and economical constraints on an technical problem.

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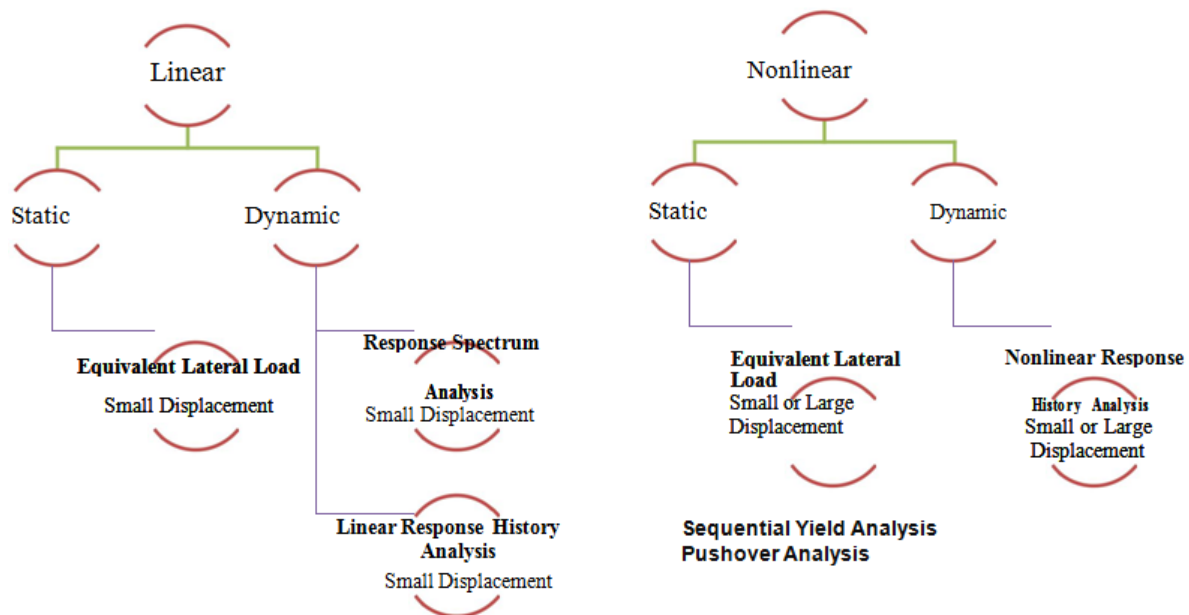
The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are the examples of such response characteristics:

• The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.

• Estimates of the deflections demands for elements that have to form in-elastically in order to dissipate the energy imparted to the building.

• Consequences of the strength deterioration of individual elements on behavior of structural system.

• For location of the critical regions in which the deflection demands are expected to be high and that have to become the focus through detailing.

• For location of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.

• Estimates of the inter-story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.

• Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff nonstructural elements of significant strength, and the foundation system.

   The last item is the most relevant one as the analytical model incorporates all elements, whether structural or non structural, that contribute significantly to the lateral load distribution. Load transfer through across the connections through the ductile elements can be checked with realistic forces; the effects of stiff partial-height infill walls on shear forces in columns can be evaluated; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements can be estimated.   These benefits come at the cost of the additional analysis effort, associated with incorporating all important elements, modeling their inelastic load-deflection characteristics, and executing incremental inelastic analysis, preferably with three dimensional analytical models.

2.1.2 Target Displacement

The fundamental question in the execution of the pushover analysis is the magnitude of the target displacement at which earthquake performance evaluation of the building is to be performed. It is the roof displacement at the center of mass of the building. In the pushover analysis it is assumed that the target displacement for the MDOF building can be estimated as the displacement demand for the corresponding equivalent SDOF system transformed to the SDOF domain through the use of a shape factor. Inherent in this approach is the assumption that the maximum MDOF displacement is controlled by a single shape factor without regards to higher mode effects.   Under the Non-linear Static Procedure, a model directly incorporating inelastic material response is displaced to a target displacement, and resulting internal deflections and forces are determined. The mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement is exceeded, or the building collapses. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake.

CHAPTER 3

3.1 Methology

Earthquake analysis of buildings can be categorized depending upon the sophistication of modeling adopted for the analysis. Buildings loaded beyond the elastic range can be analyzed using Non-Linear static analysis, but in this method one would not be able to capture the dynamic response, especially the higher mode effects. This is pushover analysis. There is no specific code for NLSA. This procedure leads to the capacity curve which can be compared with design spectrum/DCR of members and one can determine whether the building is safe or needs strengthening and its extent. The capacity of building is represented by pushover curve. The most convenient way to plot he load deflection curve is by tracking the bas shear and the roof displacement. The pushover procedure can be presented in various forms can be used in a variety of forms for the use in a variety of methodologies. As the name implies it is a process of pushing horizontally, with a prescribed loading pattern, incrementally, until the building reaches the limit state. There are several types of sophistication that can be used over for pushover curve analysis.

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

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

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

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

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

3.2 Limitations of Non-Linear Static Analysis

There are many unsolved issues that need to be addressed through more research and development. Examples of the important issues that need to be investigated are:

1. Incorporation of torsional effects (due to mass, stiffness and strength irregularities).

2. 3-D problems (orthogonality effects, direction of loading, semi-rigid diaphragms, etc)

3. Use of site specific spectra.

4. Cumulative damage issues.

5. Most importantly, the consideration of higher mode effects once a local mechanism has formed. Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deflection modes that occur in a building subjected to severe earthquakes, and it may significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.

Time history analysis is the use of an earthquake time history to evaluate the response of your system. Can be performed for both buildings and soil. Ideally, we want to use time histories that are reasonably close to our design response spectrum Target Spectrum.

3.3 Time History Analysis

There are two main methods we use to get our time histories to match the design spectrum:

1. Time History Scaling

2. Spectral Matching

3.3.1 Time History Scaling

Time history scaling involves modifying a time history's scale and/or time step in order to try to closely match the design response spectrum. In addition, we try to make the smoothed average spectrum fall within a specified tolerance of the target spectrum. Because it's impossible to get this tolerance over the entire spectrum, we typically focus on the period range of interest: Often 0.2TN to 1.5TN Advantages:

1) Uses "Real" EQ Time histories

2) Retains natural variances in the EQ time histories 3) Allows you to focus response on the period(s) of interest

4) Relatively easy to do with proper tools

Disadvantages:

1) Usually requires LOTS of time histories

2) Greater variance in your computed response

3) Can take a lot of time to do.

3.3.2 Spectral Matching

Spectral matching involves a time domain modification of an acceleration time history to make it compatible with a user-specified target spectrum. Abrahamson (1993) wrote the first widely-used computer code for spectral matching called RSP Match. It has been subsequently updated. Spectral matching has the advantage of significantly reducing variability in the computed response because the time histories are often very close to the target response spectrum.

Advantages:

1) Less variability

2) Usually requires fewer   ground motions

3) Sometimes easier to spot design flaws from the system response

Disadvantages:

1) Can result in unrealistic EQ ground motions

2) Certain valuable aspects of EQ time history can be lost

3) Can take a lot of time to do correctly

4) Easy to screw up if you don't know what you're doing

Selecting Time Histories

Part of the challenge of developing time histories is selecting appropriate time histories to use.

What kinds of factors should we consider when selecting time histories?

- Magnitude (within ± 1 Mag)
- Distance (within ±10 km if possible)
- Faulting mechanism (more important for reverse/thrust faults, subduction zones)
- Spectral Acceleration (within 20-30% at the natural period(s) of interest, if possible)
- Soil/Rock Type (same site class)
- Directivity effects (if applicable)

CHAPTER 4

4.1 Project part 1

Modeling with E -Tabs

This work presents the analysis of a framed building (G+10) is analyzed using E-Tabs. This research work is an approach, to study the effect on the lateral forces on the response of a 3-D framed building, when it is subjected to seismic forces. The analysis method used in E-Tabs, is the Response Spectrum Method conforming to IS 1893:2002. E-TABS is equipped with advanced finite element method and dynamic analysis capabilities. It features rapid analysis, and visualization tools. A G+10 storey building [3 D Frame] was analyzed for all possible load combinations (i.e live load, dead load, earthquakes loads). E-Tabs is user friendly and allows us to easily draw a frame, assign dimensions and thus input geometrical properties of all components along with definition of loads. Then as per the assigned specifications, we analyze the RCC frames.The loads to which the structure was subjected were, Dead load, Live load and Seismic loads, which had their calculation basis same as mentioned in IS 875 (Part 1): 1987 and IS1893 (Part 1): 2002 . The dimensions were assigned to the columns and the beams along with the material specification. The supports at the base of the building are fixed and the code of practice, for the analysis in E-Tabs is defined. Thereafter, the program is run for analysis. It is the post processing mode by which, the deflection of the members is studied, taking into account a suitable load combination.

4.2 Load type

4.2.1 Dead Load

The dead load comprises of weight due to all structural components of a structure in other words these are the permanent parts of the construction work or rather remain always with the structure. The dead consists of self weight of all structural components like floor, walls, beams, columns, roof etc. also floor finishes, false ceilings, false doors and other permanent constructions in the structure. The unit weight of RCC and Plain concrete made with sand and gravel or crushed natural aggregate are 25 KN/mm^2 and 24 KN/mm^2 respectively as per IS 875.

4.2.2 Imposed Load

It includes weight of furniture, mobile partitions, vibratory effects, impact loads, point loads and uniform loads.

4.2.3 Earthquake Load

Design Lateral Force: The lateral seismic force is evaluated for the building as a whole. This force shall then be used for the design of the entire structure. The floor diaphragm action further decides its distribution to horizontal load resisting components. Design Seismic Base Shear: The summation of all design lateral seismic forces at the base of the building is referred to as the Design Seismic Base Shear (V_B). The expression for this force as given in IS 1893(Part 1) : 2002, clause no. 7.5.3 is :

$$V_B = A_h W$$

Where,

A_h = Design horizontal acceleration spectrum, using the fundamental natural period T_a , in the direction under consideration.

W = Seismic Weight of the Building.

4.2.4 Fundamental Natural Period

It is the time period with long modal vibration period. The fundamental natural period of vibration (T_a), in seconds of a moment resisting frame building without brick infill is given as:

$$T_a = 0.075 \ h^{0.75} \qquad \text{for RCC frame building}$$
$$= 0.085 \ h^{0.75} \qquad \text{for steel frame building}$$

Where,

h = height of the building which does not include the basement walls connected to columns of the building or to the ground floor deck. The inclusion of the basement storeys is when they are not connected this way.

In case of moment resisting frame with brick infill, the evaluation of the fundamental period is done by the following empirical formula given in the IS code:

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

Where,

h = height of the building in mm

d = base dimension of the building at the plinth level, in the direction of considered horizontal seismic force.

4.2.5 Distribution of Design Force

The distribution of base shear at different heights of the building is given by the following expression, as per IS 1893:2002.

$$P_i = V_B \frac{W_i h_i^2}{\sum W_{jH_j^2}}$$

Where,

P_i = Design lateral force at ith floor,

W_i = Seismic weight of ith floor,

h_i = ith floor height from the base,

n = no. of storeys in the building, where the weights are positioned.

4.2.6 Dynamic Analysis

The buildings for dynamic analysis are:

a) Regular Buildings: Frame buildings exceeding the height of 40 m in seismic Zones IV and V and the ones in Zones II and III with height greater than 90 m.

b) Irregular Buildings: Frame building in Zones IV and V with height greater than 12 m and with height more than 40 m Zones II and III.

If there lies irregularities in the design of a building then the analytical model thus framed for the prototype shall represent it accurately. Buildings that are not regular in plan cannot be analyzed for dynamic analysis. Dynamic method for frame buildings can be carried out by Response Spectrum Method or Time History Method.

4.3 Analytical study of G+10 framed building

In the present study a G+10 Storey framed building has been analyzed. The following screen shots have been taken from E-TAB.

4.3.1 Characteristics of the Framed Building

Plan view and elevation view
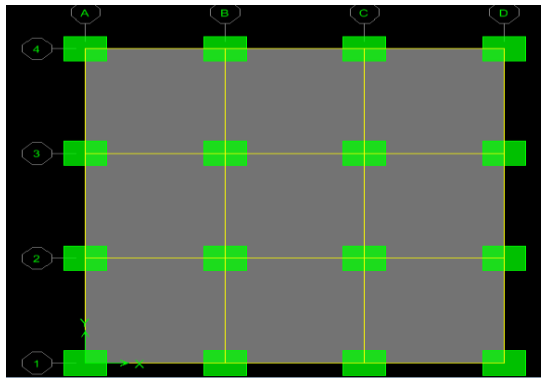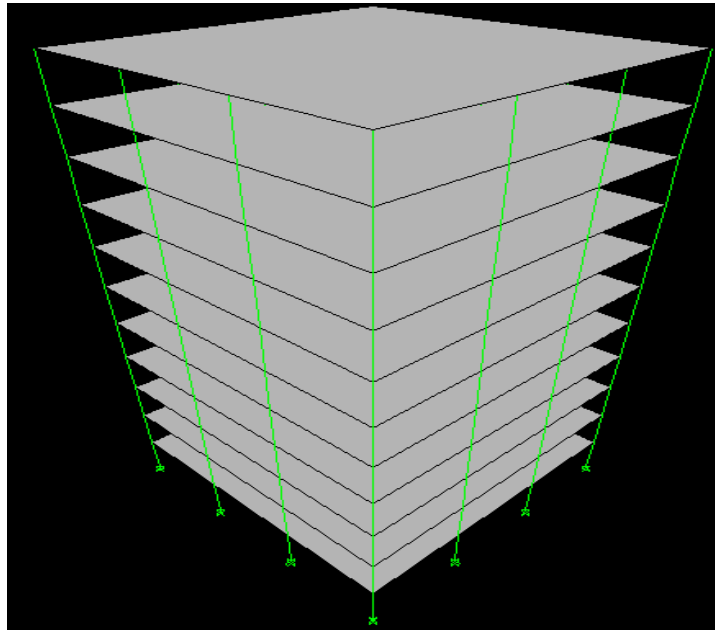
Figure4 and figure 5: Plan and elevation of a G+10 storey framed building

4.3.2 Defining Sectional Properties

Member property could be generated in E-Tabs. by using the sub tab "property" in the window of the program.

4.3.2.1Specifications of existing building

Table 1

Member	Size
Column 1	900*700mm up to 5th storey
Column 2	600*500mm above 5th storey
Beam	500*300mm
Slab	150mm
Wall	230mm

Column 1(reinforcement)	7 nos. in x-direction, 9 nos. in y-direction
Column 2(reinforcement)	4 nos. in x-direction, 6 nos. in y-direction

Column 1	900*700mm up to 5th storey
Column 2	600*500mm above 5th storey
Beam	400*300mm
Slab	150mm
Wall	230mm
Column 1(reinforcement)	7 nos. in x-direction, 9 nos. in y-direction
Column 2(reinforcement)	4 nos. in x-direction, 6 nos. in y-direction
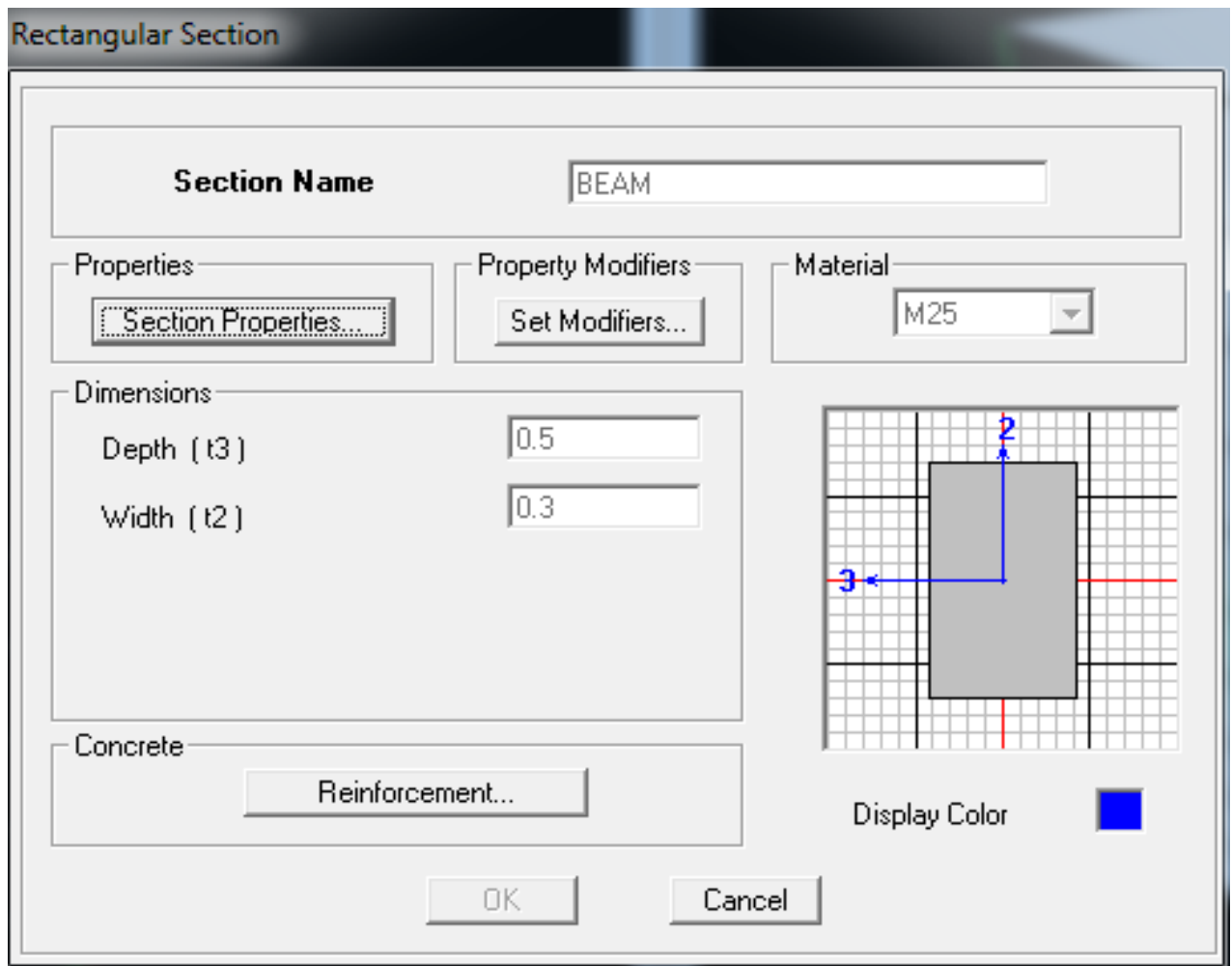
Figure 6

COLUMNS:
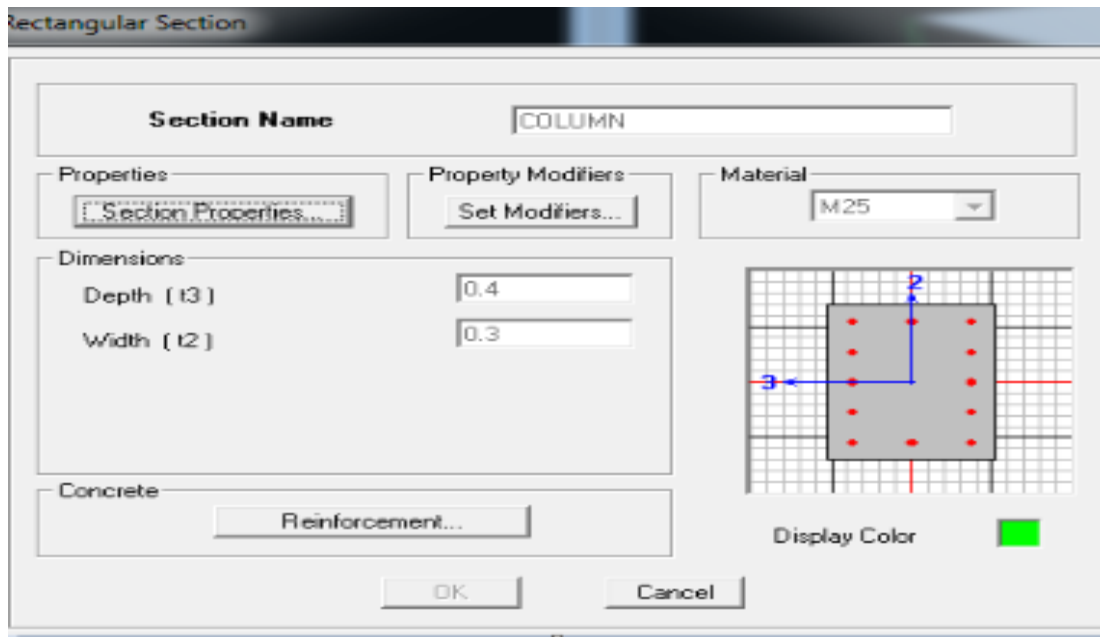
Figure 7

Slab and wall thickness:
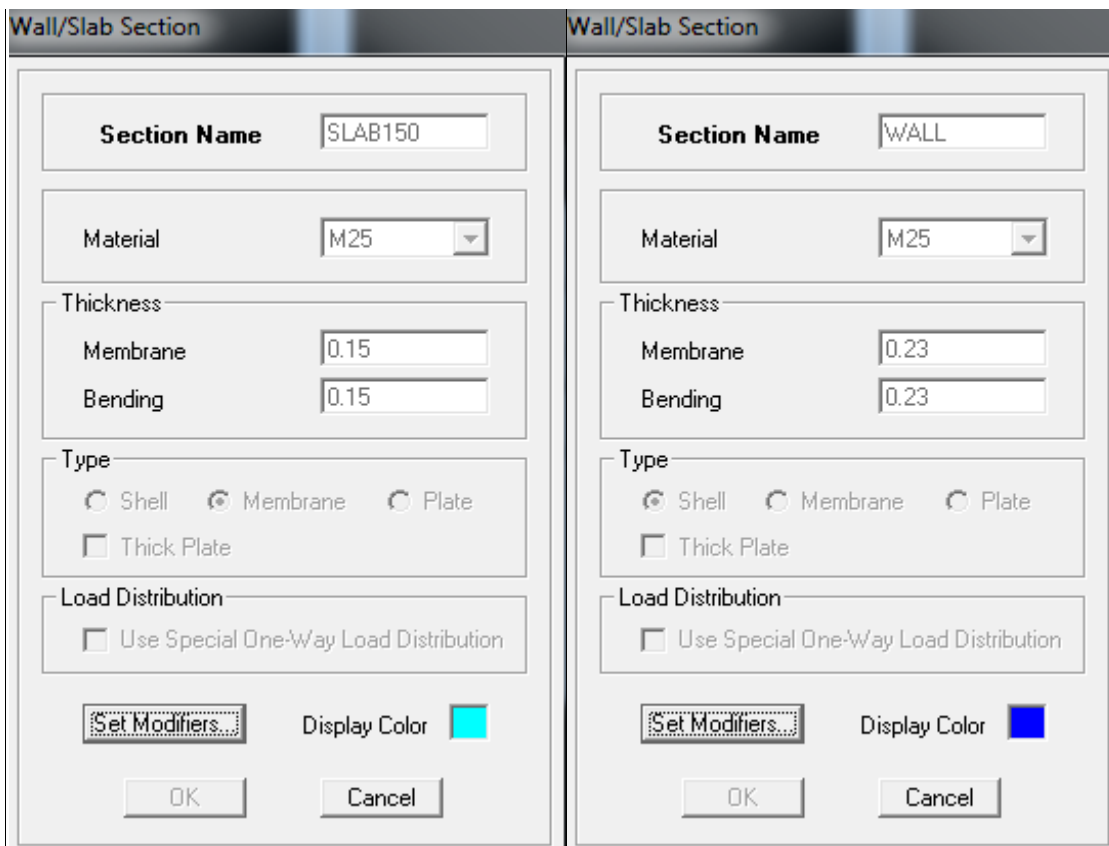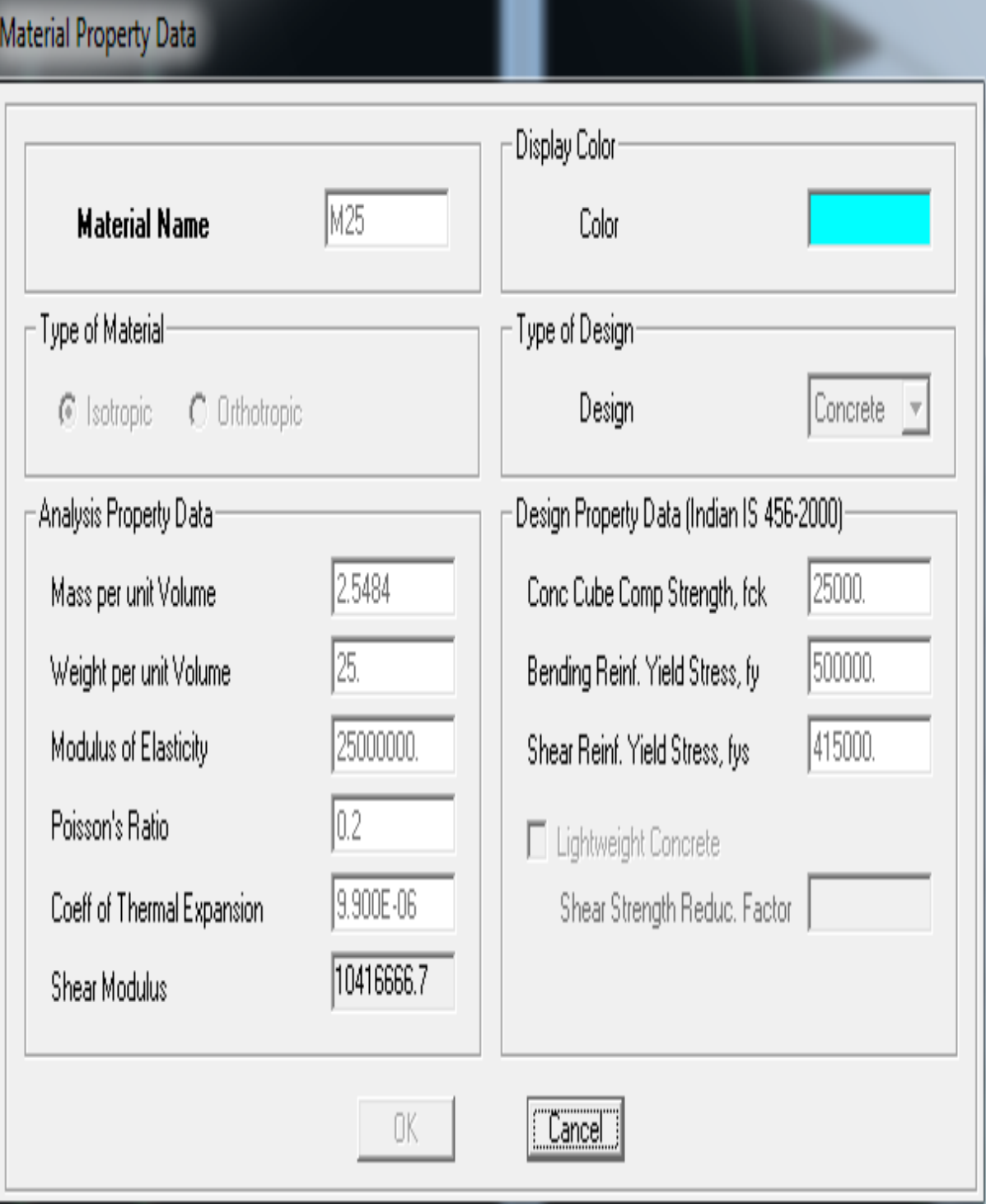
Figure 8 and figure 9

Material properties: concrete M25 Grade, bending reinforcement F_y500, shear reinforcement F_y 415

figure 10

4.3.3 Assigning Supports

The base of the building is provided with a fixed support. This fixed support is assigned to the building.
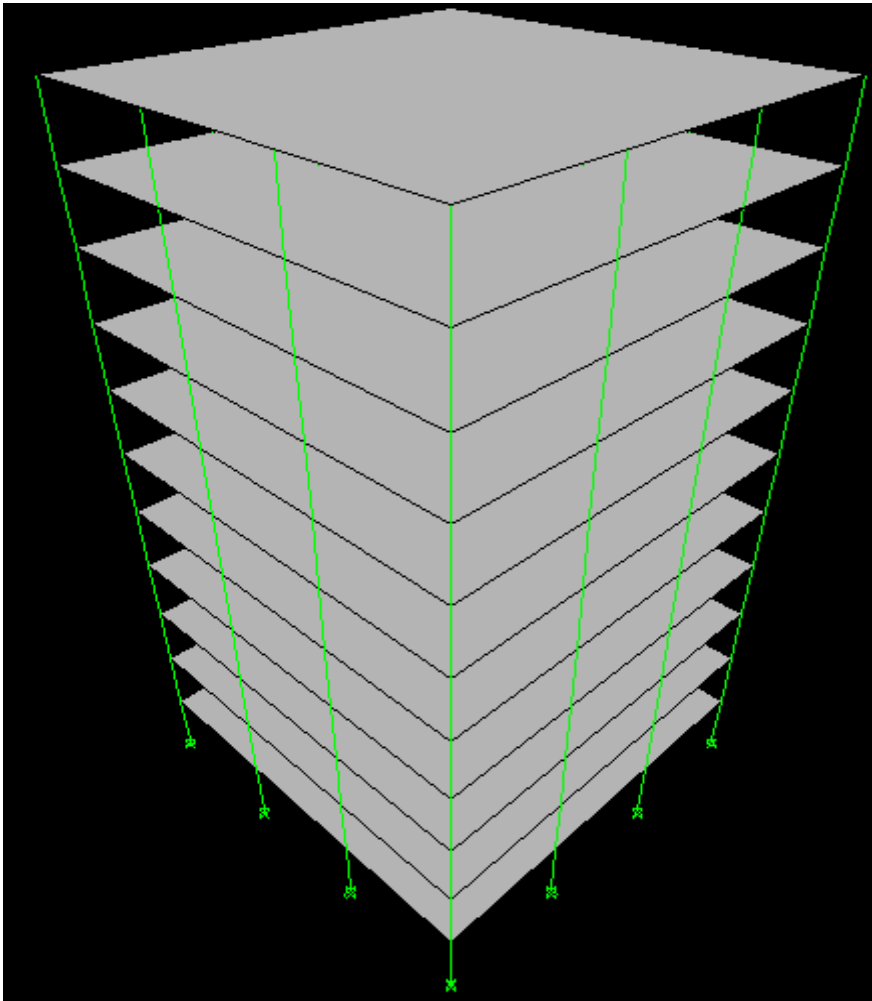
Figure 11: Fixed supports at the base.

4.3.4 Generation of Loads

Loads are assigned to the structure and its members by using the load generator in E-Tabs. In E-Tabs the categories of load are defined. The types of loads which has been used in analysis are as following-

 a. Dead load.
 b. Live load.
 c. Slab load.
 d. Wall load.
 e. Earthquake Load.
 f. Wind load.

Load cases:

Table 2: Load Pattern Definitions

LoadPat	Design Type	Self WtMult	Auto Load
DEAD	DEAD	1.000000	
LIVE	DEAD	0.000000	
WALL	SUPER DEAD	0.000000	
FF	SUPER DEAD	0.000000	
EQXPY	QUAKE	0.000000	IS1893 2002
EQYPX	QUAKE	0.000000	IS1893 2002
WX	WIND	0.000000	IS875 1987
WY	WIND	0.000000	IS875 1987
SLAB	SUPER DEAD	0.000000	

4.3.5 Dead load

The dead load comprises of weight due to all structural components of a structure.

4.3.6 Live Load:

The live load applied on the framed building is of different intensities. Live load on roof is 3 kN/m . Floor finishing load 1.5 KN/m^2.

Table 3: Auto Wind - IS875-1987, Part 1 of 2

Load Pat	Angle	Wind ward Cp	Lee ward Cp	Wind Speed
	Degrees			meter/sec
WX	0.000	0.800000	0.500000	50.000
WY	0.000	0.800000	0.500000	50.000

Table 4: Auto Wind - IS875-1987, Part 2 of 2

Load Pat	Terrain Cat	Struct Class	k1	k3
WX	2	B	1.000000	1.000000
WY	2	B	1.000000	1.000000

Table 5: Auto Seismic - IS1893:2002, Part 1 of 2

Load Pat	Dir	Percent Ecc	Z Code	Soil Type
EQXPY	X	0.050000	0.24	II
EQYPX	Y	0.050000	0.24	II

Table 6: Auto Seismic - IS1893:2002, Part 2 of 2

Load Pat	I	R	T Used Sec	Coeff Used	Weight Used KN	Base Shear KN
EQXPY	1.000000	5.000000	0.9610	0.707596	51138.006	1736.883
EQYPX	1.000000	5.000000	0.9610	0.707596	51138.006	1736.883

4.3.7 Slab load

Self weight is generated by the software itself by defining the primary load case as the slab weight in the "load & definition" tab.

4.3.8 Wall Load

Loading due to wall is distributed to the members .It is taken by software it self.

4.3.9 Seismic Load

The earthquake forces are calculated following the guidelines, mentioned in IS 1893(Part I):2002. The software can generate seismic force provided that, the code which is to be followed, is defined. The lateral seismic forces are generated in the two horizontal directions i.e. X and Z directions. The Base Shear calculated by the E-Tabs is as per the IS code:

$$V_B = A_h W$$

Where,

A_h = Design horizontal acceleration spectrum, using the fundamental natural period T_a, in the direction under consideration.

W = Seismic Weight of the Building.

4.3.10 Load Combination

The analysis of the structure proceeds with the consideration of the load combinations. The various loads like the dead load, live load, wall loads, seismic loads etc. are considered in a proper proportion and thus the structure is analyzed using the load cases which are the combinations of these.

Load combination data:
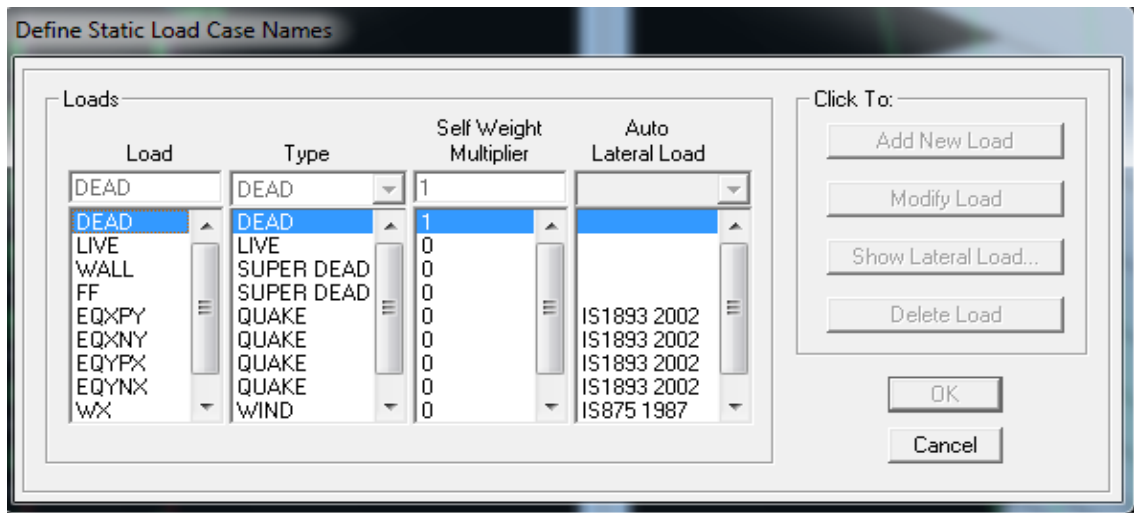
Figure12: Primary load combinations
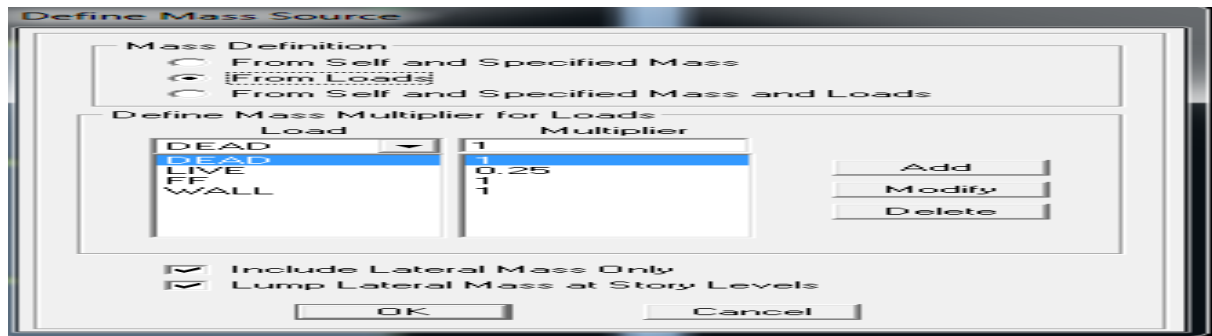
Reduction factor for loads:

Figure 13

4.4 Project Part 2

4.4.1 Procedure of Non-Linear Static Analysis using SAP 2000

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the building are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deflection criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the building and it promises to be a useful and effective tool for performance based design.

The ATC-40 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force deflection criteria for hinges used in pushover analysis. As shown in Figure 1, five points labeled A, B, C, D, and E are used

to define the force deflection behavior of the hinge and three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents.

This article presents the steps used in performing a pushover analysis of a simple three dimensional building. SAP2000, a state-of-the-art, general-purpose, three-dimensional structural analysis program, is used as a tool for performing the pushover. The SAP2000 static pushover analysis capabilities, which are fully integrated into the program, allow quick and easy implementation of the pushover procedures prescribed in the ATC-40 and FEMA-273 documents for both two and three-dimensional buildings.

Step-1 Linear Analysis & Unlock the Model

Design the building as per linear analysis using expected strength and modified partial safety factors. After completing the design of the building Unlock the model as shown in Figure 14.
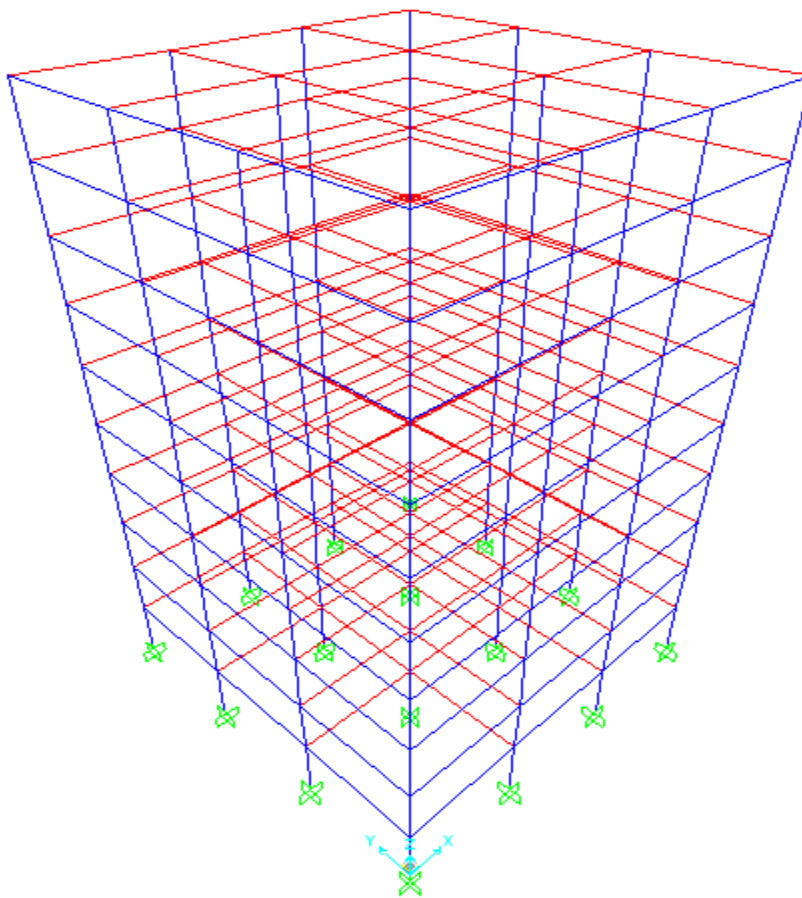
Figure 14: Unlock Model Form

Step-2 Define New Load Case of Gravity Loads

Go to Define > Load Case> Add New Load Case consisting of Gravity loads (i.e. dead load and % of live load). This load case consists of force controlled loads as shown in Figure 15 since load application type is full load.
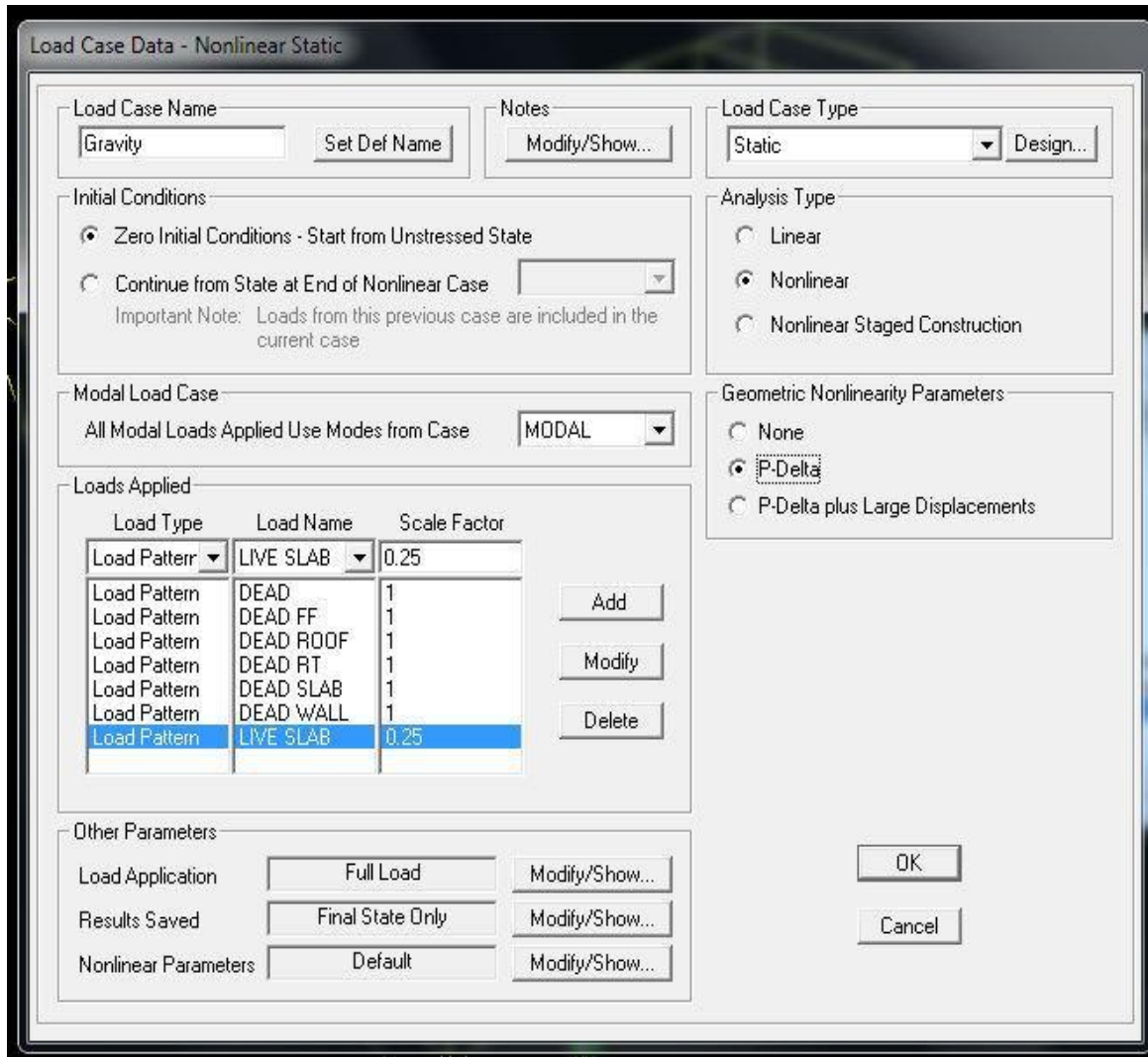
Figure 15

Select Load Case Type > Static, Analysis Type> Nonlinear and Geometric Non-linearity Parameters as P-Delta. The non-linear parameters (other parameters as shown in Figure 15) should not be changed in gravity load case. It will start from zero initial stiffness.

Step-3 Assignment of Hinges to Frame Elements

Select all the beams in the model. Go to Assign > Frame > Hinges. The hinge form will appear as shown in Figure 16
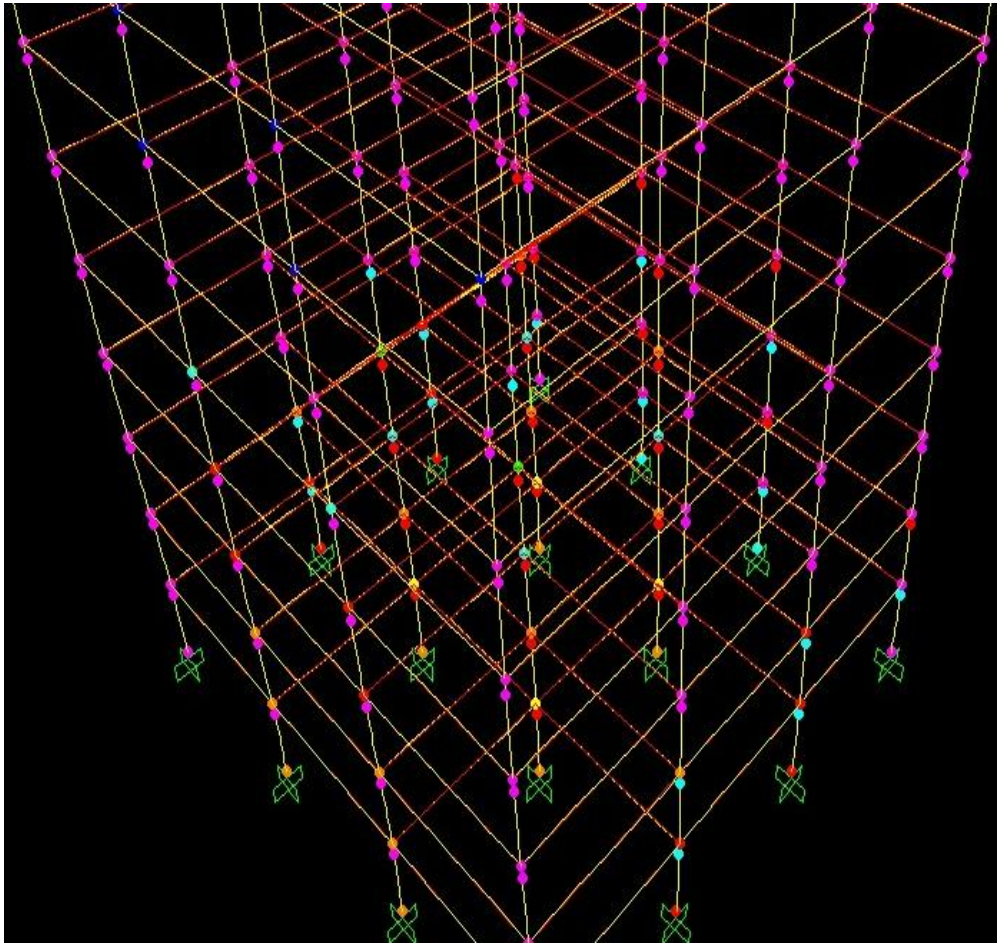
Figure 16: Hinge Assignment Form

Add hinges to the selected beams the hinge type form will appear as shown in Figure 16.
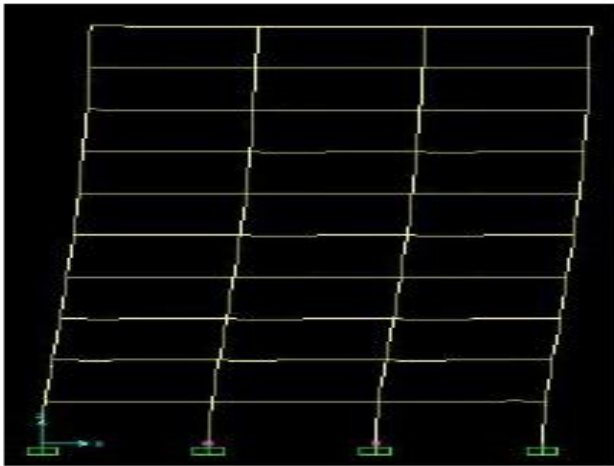
Figure 17

The hinges should be assigned at both the ends which means at the relative distance of 0 and 1 and form as shown in Figure 17 will appear. In similar manner assign hinges to all columns by repeating steps as previously carried out for beams the only difference is that column should be assigned P-M2-M3 hinges instead of M3 hinges as shown in Figure 17.

24
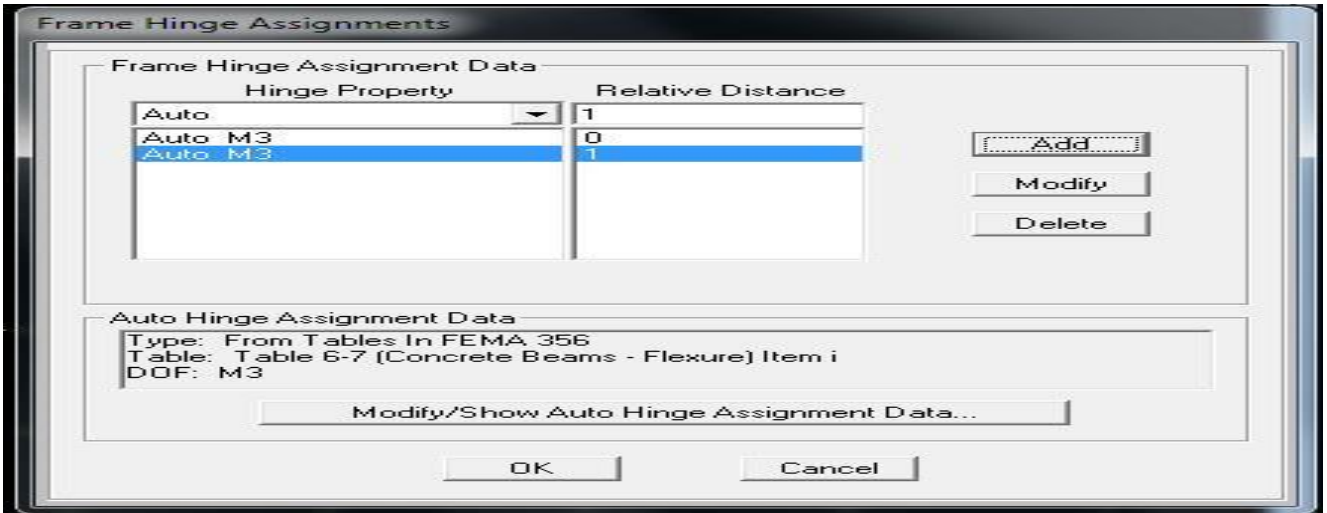
Figure 18: Hinge Properties for Beams Form

The hinges should be assigned at both the ends which means at the relative distance of 0 and 1 and form as shown in Figure 19 will appear.
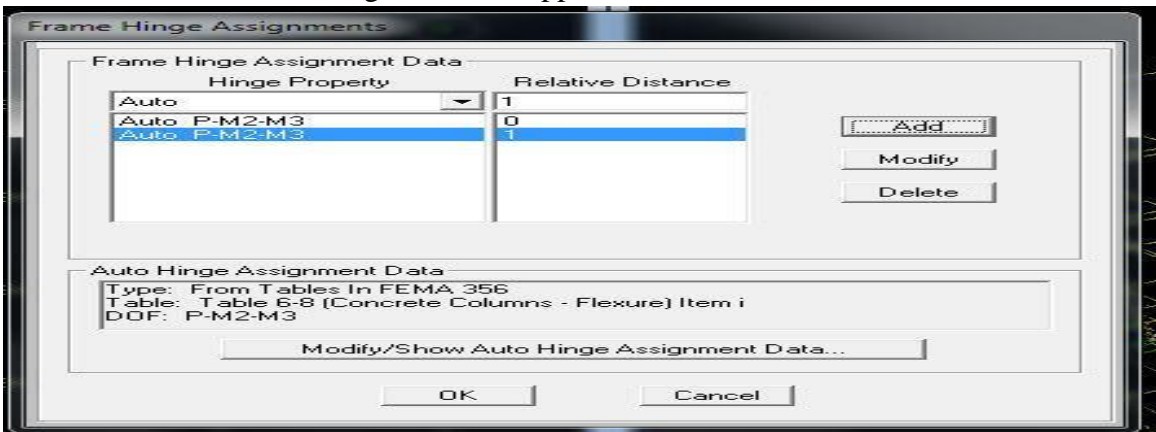
Figure 19:Hinge at both ends Form for Column

Step-4 Define Pushover load Case

Go to Define > Load Case> Add New Load Case >PUSH consisting of load in proportion to the fundamental mode. This load case is deformation controlled load case as shown in Figure 20.
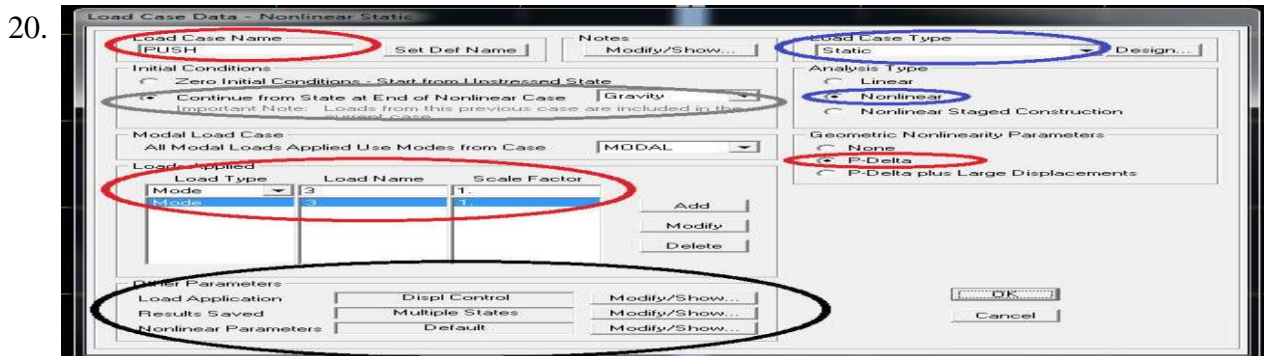
Figure 20:Pushover Load Case Form

Select Load Case Type> Static, Analysis Type> Nonlinear and Geometric Non-linearity Parameters as P-Delta. This load case should be started from a previous load case Gravity since gravity load will always be acting on the structure. Select Loads Applied proportional to Mode in the considered direction of the analysis. The scale factor for this load case should be kept equal to 1. In pushover load case for other parameters, to modify the displacement upto which the force-deformation curve needs to be monitored click Modify and the load application control for non-linear static analysis form will appear as shown in Figure 9. In the non-linear load application control parameters Load Application should be Displacement control with Monitored Displacement. Generally, the monitored displacement is kept equal to 2% of the height of the building. This displacement should be monitored in the considered direction of analysis (i.e. either U1 or U2 degree of freedom). The joint at which the force deformation curve is monitored is genera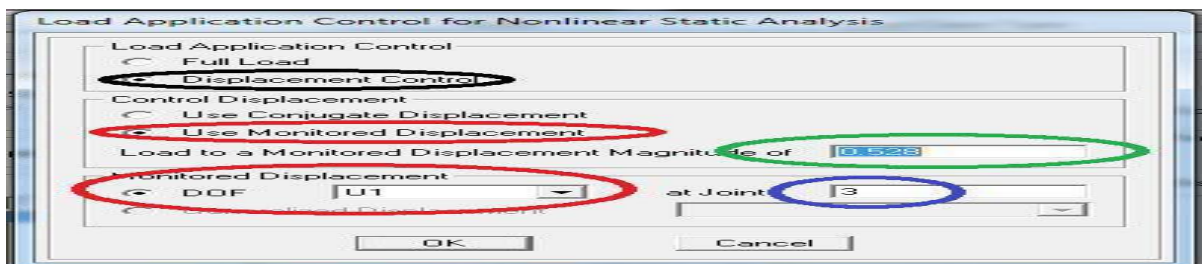lly taken as at the Centre of mass of the building.Displacement is kept equal to 2% of the height of the building. This displacement should be monitored in the considered direction of analysis (i.e. either U1 or U2 degree of freedom). The joint at which the force deformation curve is monitored is generally taken as at the Centre of mass of the building

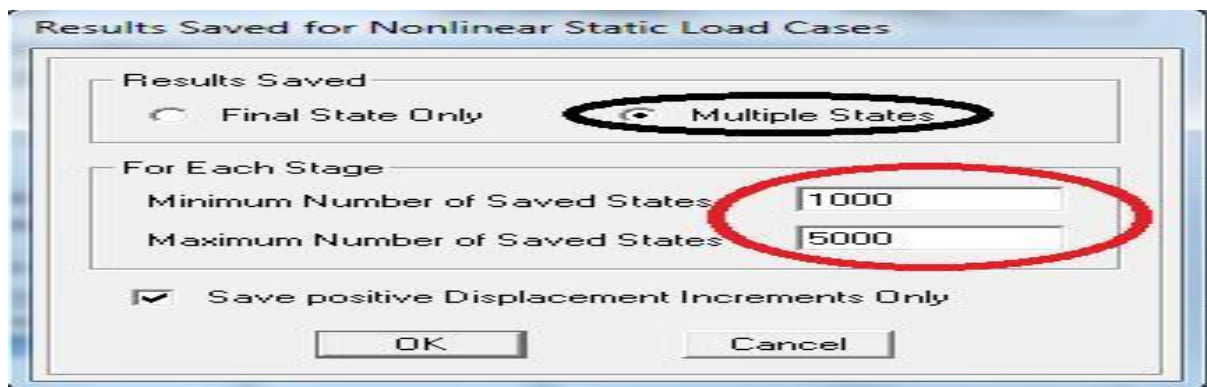Figure 21: Load Application Control for Non-Linear Static Analysis Form

Figure 22 : Results Saved for Non-Linear Static Load Case Form

26

In pushover load case for other parameters, to modify the steps at which results needs to be saved click Modify the results saved for non-linear static load case form will appear as shown in Figure 23. In this form Multiple Steps should be selected in order to save the results at the intermediate steps. In this form for each stage Minimum and Maximum number of saved steps should be kept 1000 and 5000 in order to avoid solution converge. In pushover load case for other parameters, the non-linear parameters should be by Default. If convergence problem occurs than Number of Null Steps and Number of Constant Stiffness Iteration should be increased. For unloading the hinge, Unload Entire Structure method should be used as shown non-linear parameter form in Figure 24. The hinge unloading method controls the time requirements for completing the analysis. The Local Redistribution method is the Fastest while the Unloading Entire Structure method is the Slowest. It is important to note that the non-linear parameters affect the solution control; hence the results obtained using different solution control parameters and Hinge unloading method may deviate slightly, hence no unique solution is obtained for some problems.
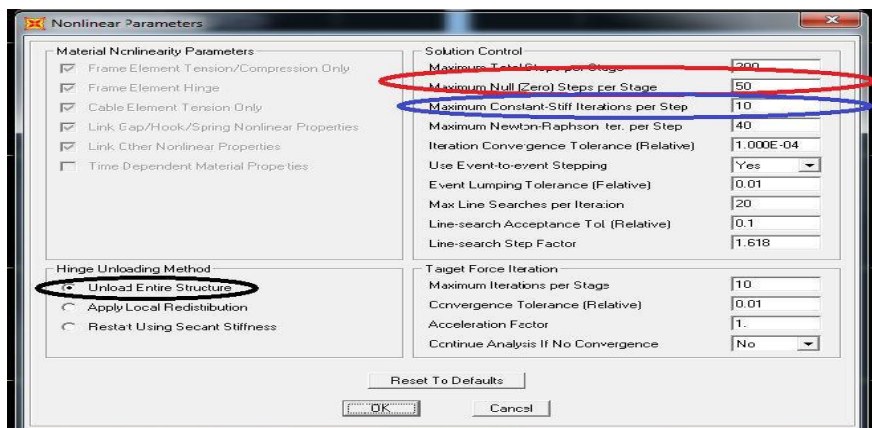

Figure 23: Non-Linear Parameters Form

Step-4 Run Analysis

While running the analysis it is important to Run the Modal and Gravity analysis with pushover load case since Pushover load case takes stiffness from gravity load case and mode shape from Modal load case. While the SAP 2000 is analyzing the structure the analysis monitor form as shown in Figure 24 will appear. After completing the analysis the analysis complete form as shown in Figure 25 will appear. The pushover analysis is a non-linear static analysis so depending upon system configuration it takes time to complete the analysis.
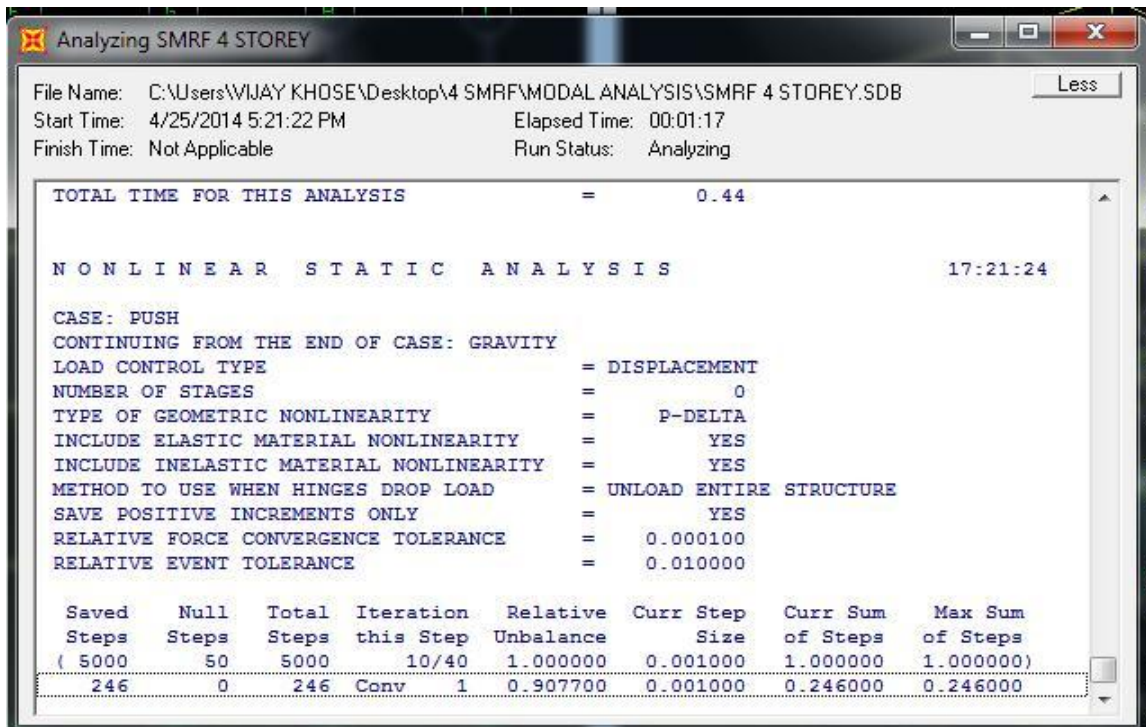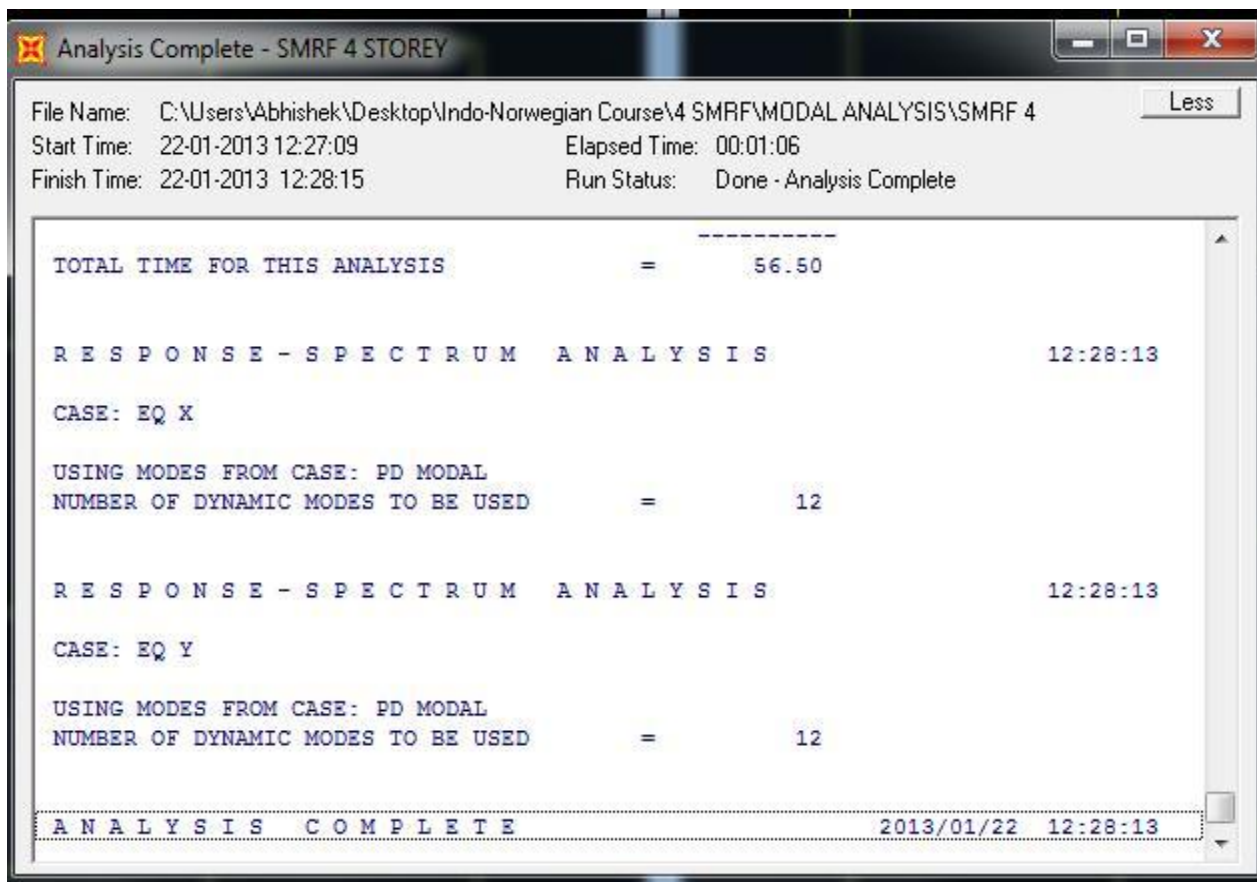
Figure 24: Analysis Monitor Form

Figure 25: Analysis Complete Form

Step-5 Graphically review the pushover analysis results

To see the pushover curve Go to > Display > Show Static Pushover Curve, the display static pushover curve form as shown in Figure 26.The Static Pushover Curve will appear as shown in Figure 26. Go to > File > Display Tables it will show the results of Pushover analysis in Tabular Form.
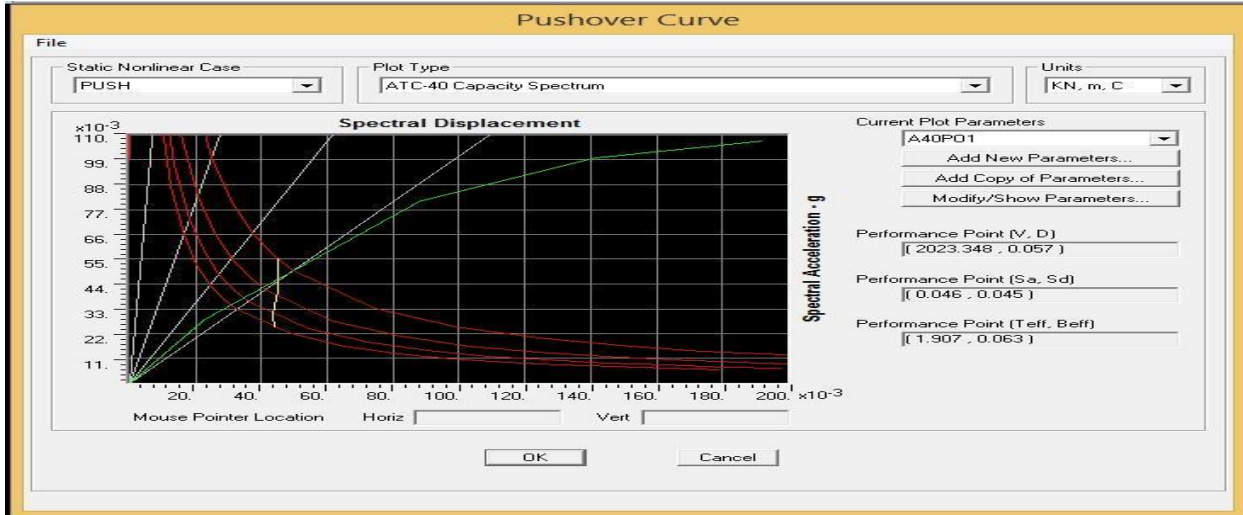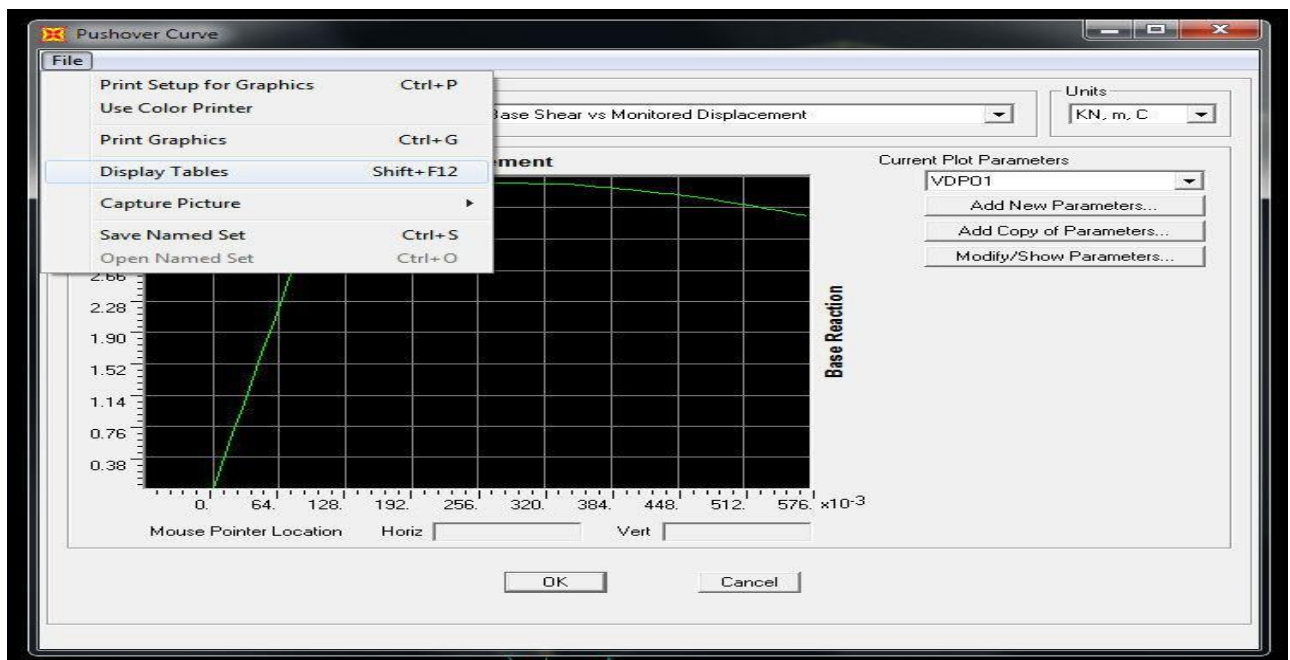

Figure26: Display Static Pushover Curve Form

Figure 27

To see the deformed shape under pushover load case and hinges at any step, Go to Display> Show Deformed Shape> Select load case> PUSH. The deformed shape form will appear as

shown in Figure 28.

In deformed shape form the step at which the result is to be seen should be given. For the result at last step it should be 1000 and deformed shape of structure at that step is as shown in Figure 29.
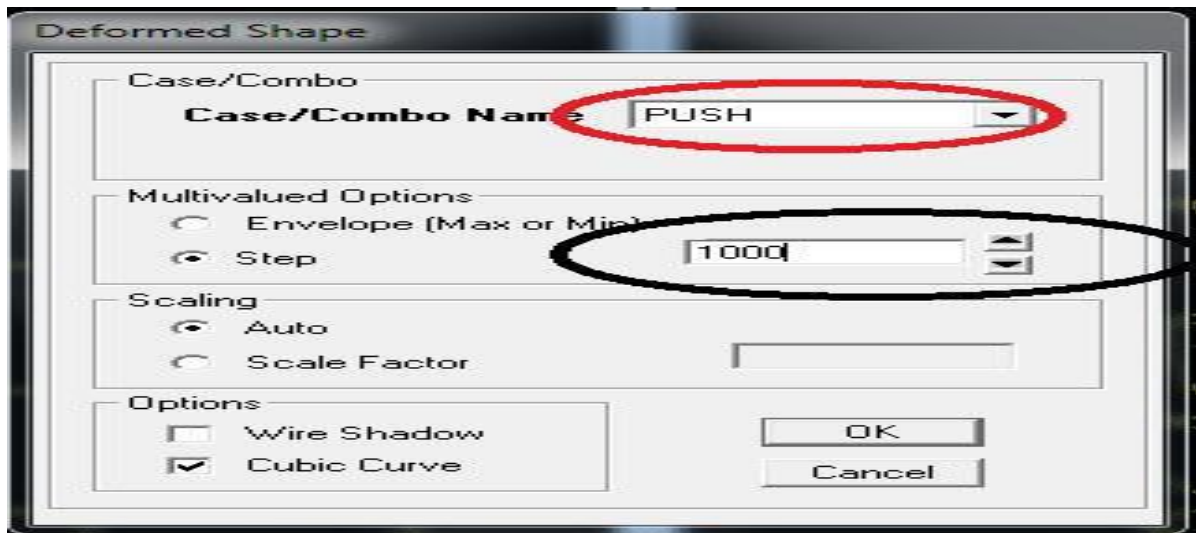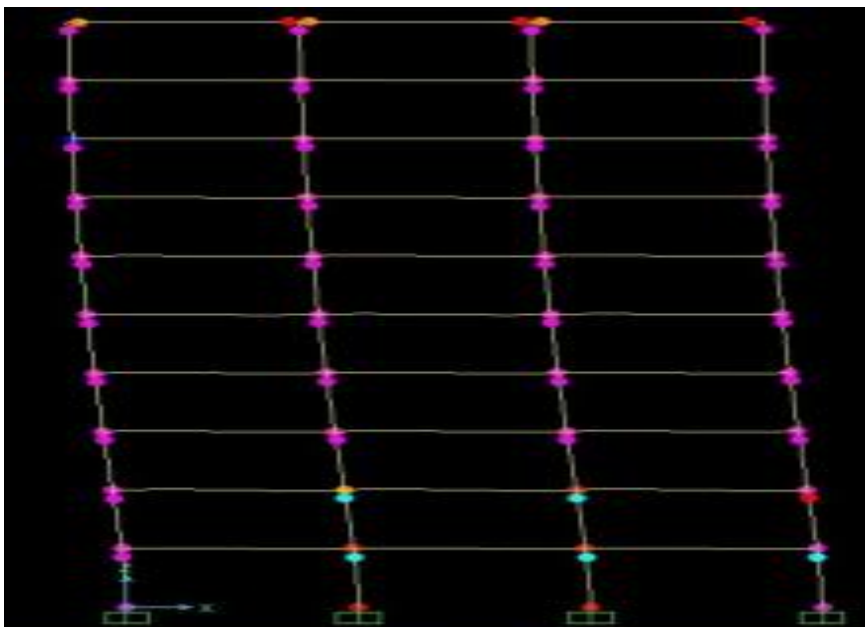

Figure 28: Deformed Shape Form

Figure 29: Deformed Shape at Last Step

To see the hinge results Right Click on Hinge the hinge results will appear as shown in Figure 30.
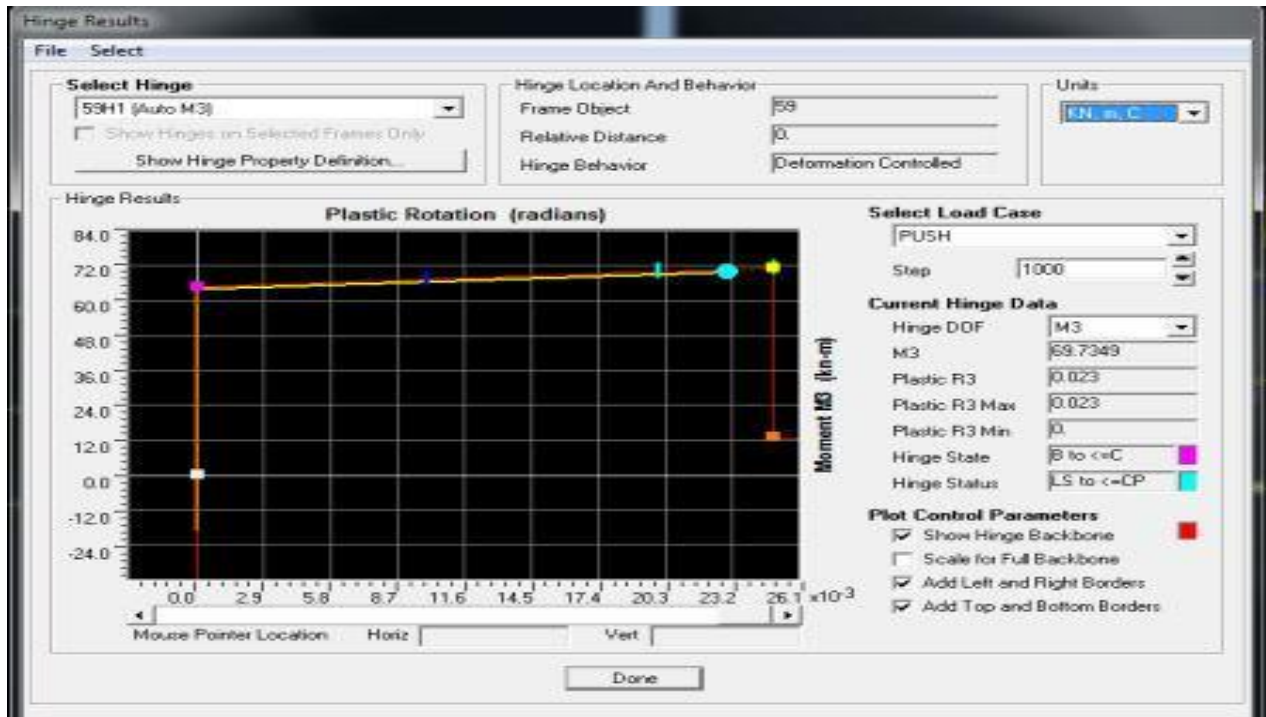

Figure 30 Hinge Result

In order to perform the time history analysis the step 1 to step 3 should be repeated as discussed above.

Step-6 Define time history

Go to Define > Functions> Time History the following form will appear as shown in Figure 31. In this form choose function type to add from file.
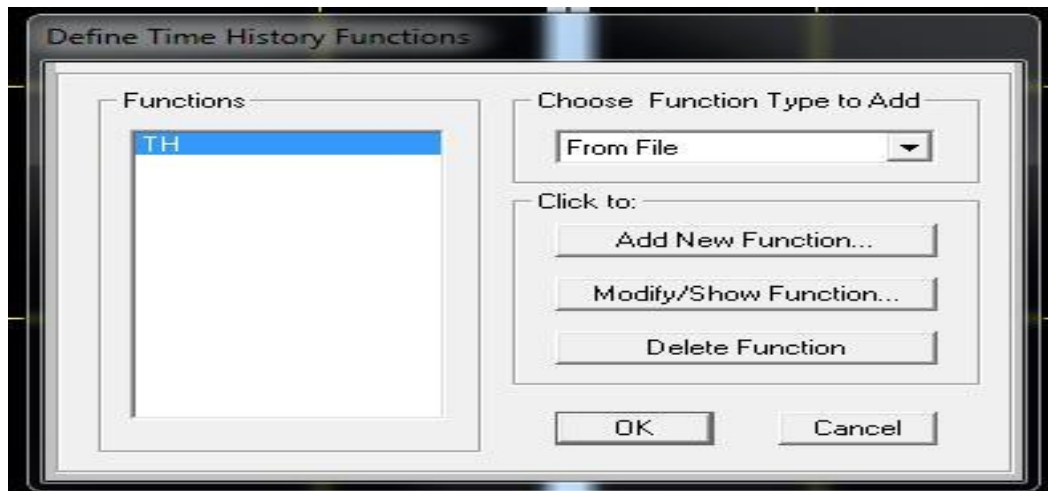

Figure31

Go to Add New Function and add the time history file from system in text format and convert it to user defined than time history file form will appear as shown in Figure 32
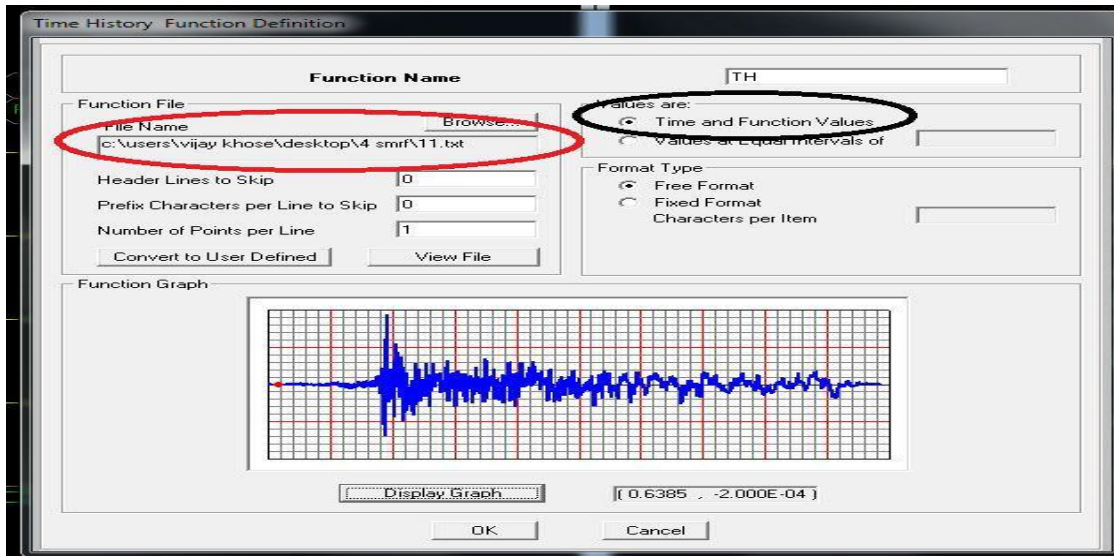

Figure 32**:** Time History Function Definition

Step-7 Define time history load case

Go to Define > Load Case> Add New Load Case >TH consisting of time history load case and the time history load case form will appear as shown in Figure 33.
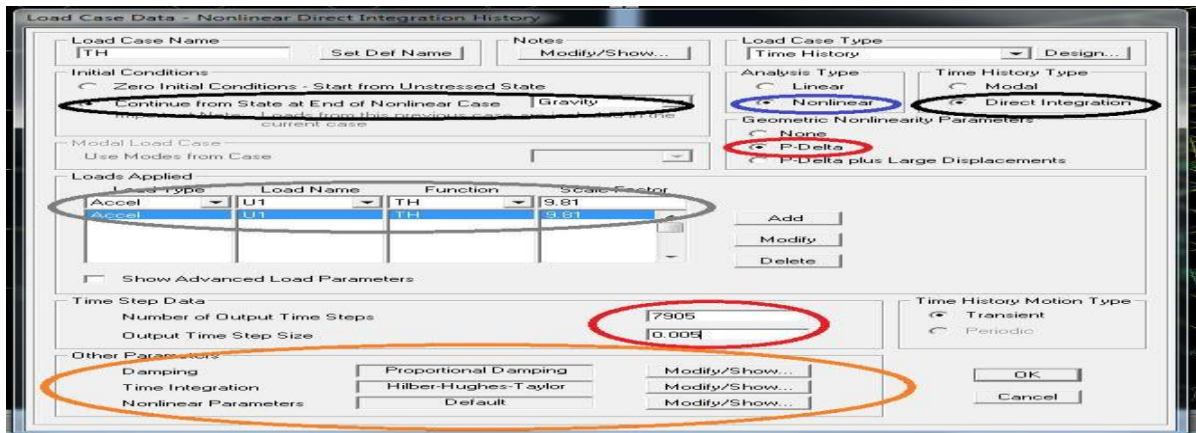

Figure 33 **:** Time History Load Case Form

Select Load Case Type> Time History, Analysis Type> Nonlinear, Time History Type> Direct Integration and

Geometric Non-linearity Parameters as P-Delta. This load case should be started from a previous load case Gravity since gravity load will always be acting on the structure. Select Loads Applied > Acceleration in the considered direction (i.e. either U1 or U2 degree of freedom) of the analysis. The scale factor for this load case should be kept equal to 1or 9.81 depending upon the input time history file whether provided in units of g or m/s^2, respectively.
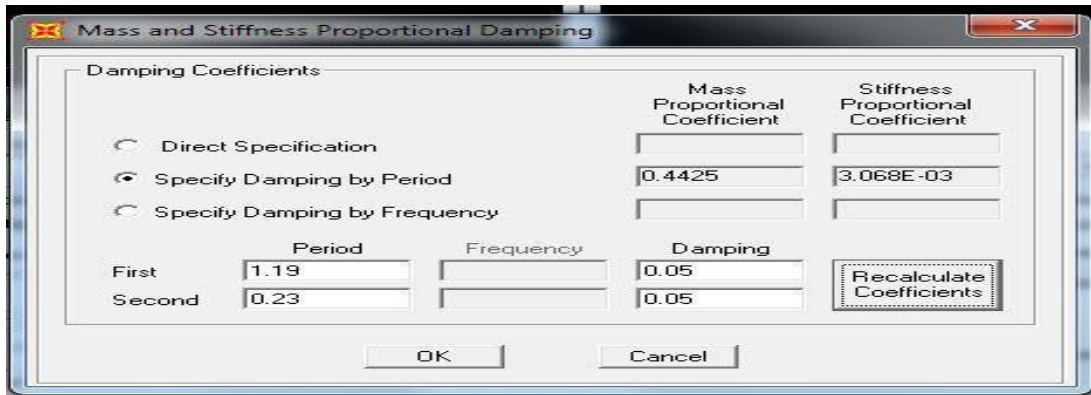

Figure 34: Mass and Stiffness Proportional Damping Form

In Time History Load Case for other parameters, to modify the damping which needs to be considered click Modify and mass and stiffness proportional damping form will appear as shown in Figure 35. It is important to note that when damping is specified it is specified by period and period corresponding to 1st and 3rd mode of vibration in the considered direction of analysis should be used. The damping used should be equal to 2%, 5% and 10% for Steel, RC and Masonry buildings, respectively. In Time History load case for other parameters, to modify the time integration technique to be considered click Modify and time integration parameter form will appear as shown in Figure 35. The Hilber-Hughes-Taylor method of time integration should be used with alpha as zero and if convergence does not occur than alpha equal to minus 0.33 should be used.
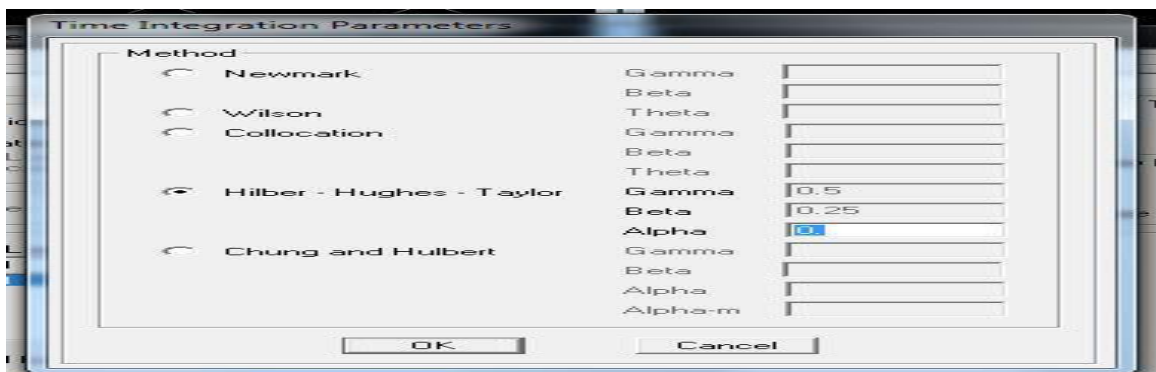

Figure 35: Time Integration Parameters Form

In Time History Load Case for other parameters, the non-linear parameters should be set to default. After

defining Time History Load Case, Run the analysis.The non-linear time history analysis is a time consuming analysis and depending upon the size of the problem it takes large amount of time as well as space for completion of the analysis.

Step-8 Graphically review the time history analysis results

The deformed shape and hinge pattern at any instant can be viewed in similar manner as viewed in case of pushover analysis. The peak displacement during time history analysis can be viewed through plot function. Go to > Display > Show Plot Functions and define the plot function i.e. joint whose displacement is required. The Figure 36 shows the plot functions for the joint displacement for the time history load case.
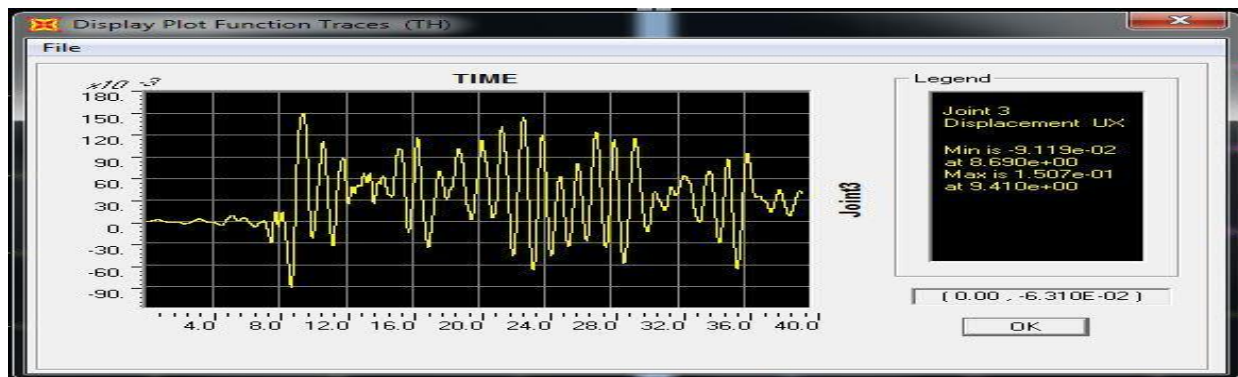

Figure 36 :Plot Function for Joint 3 Displacements

CHAPTER 5

5.1Results

On the above building frame the non linear static pushover analysis is performed to investigate the performance point of the building frame in terms of base shear and displacement. For pushover analysis the various pushover cases are considered such as push gravity, push X (i.e. loads are applied in X direction), push Y (i.e. loads are applied in Y direction). The various load combinations are also used for this purpose. After pushover analysis the demand curve and capacity curves are obtained to get the performance point of the structure. The performance point is obtained as per ATC 40 capacity spectrum method. The base shear for PUSH X load case is (904.612 KN). And for PUSH Y base shear at performance point is (915.197) as shown in figure

Column interaction curves and tables:

Table 7

Curve 1	0. degrees	
P	M3	M2
-3810.3	0	0
-3511.06	147.9328	0
-3099.22	195.128	0
-2517.27	250.8787	0
-1674.35	314.8028	0
-612.957	382.123	0
127.0696	386.7478	0
834.2274	361.1915	0
1662.192	275.0303	0
2655.853	132.1866	0
3492.041	0	0
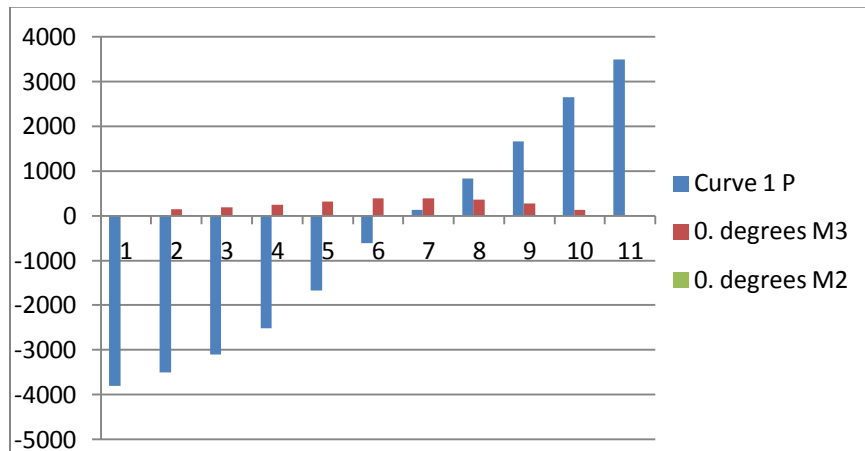
Figure 37

Table 8:

Curve 2	15. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	117.4401	21.5068
-3263.16	167.8358	26.8775
-2645.34	224.1198	33.7359
-1811.12	286.5337	40.7831
-604.55	354.7683	50.3025
298.0932	353.4224	50.6083
1208.793	301.5445	52.476
2191.1	194.1542	45.665
3249.103	38.5784	25.1397
3492.041	0	0
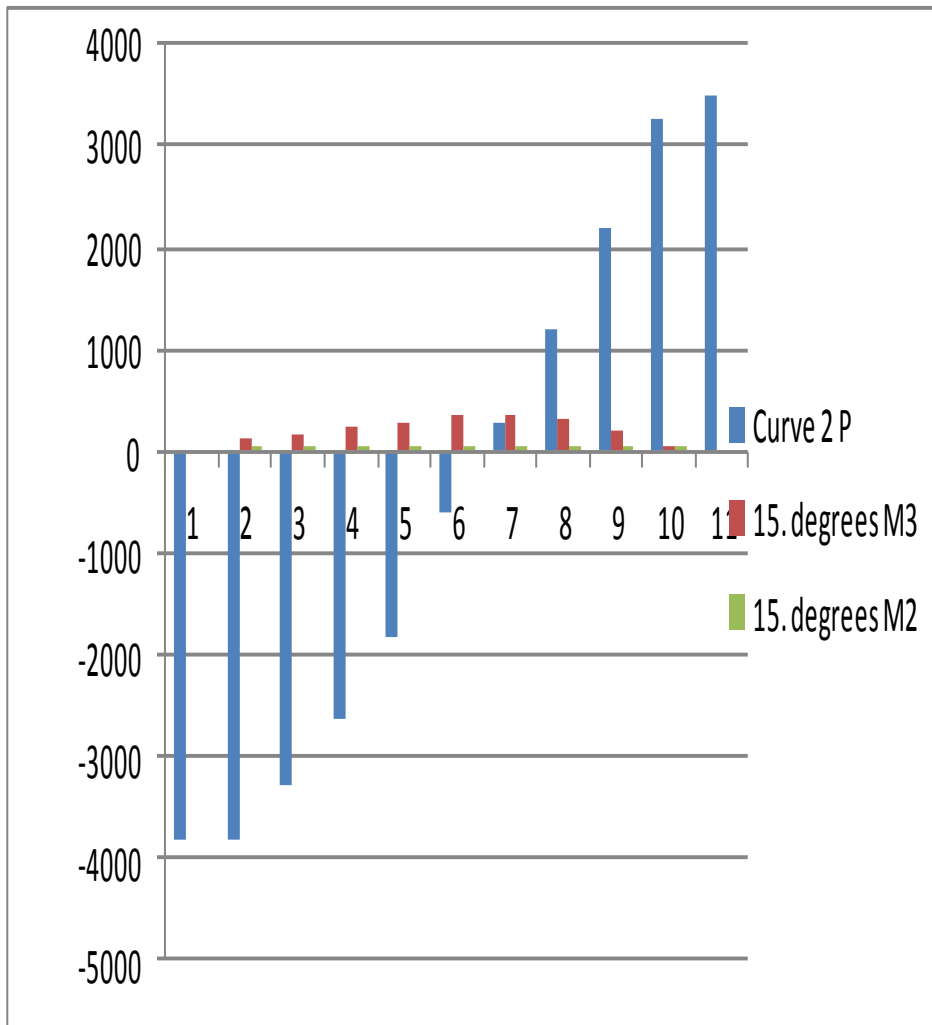
Figure 38

Table 9

Curve 3	30. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	99.3414	36.8919
-3384.4	141.047	51.1633
-2740	194.043	65.8957
-1846	251.394	82.8778
-566.68	304.501	102.069
515.542	307.873	101.008
1531.83	247.027	95.5849
2548.06	134.352	73.5455
3287.81	32.1687	21.6891
3492.04	0	0
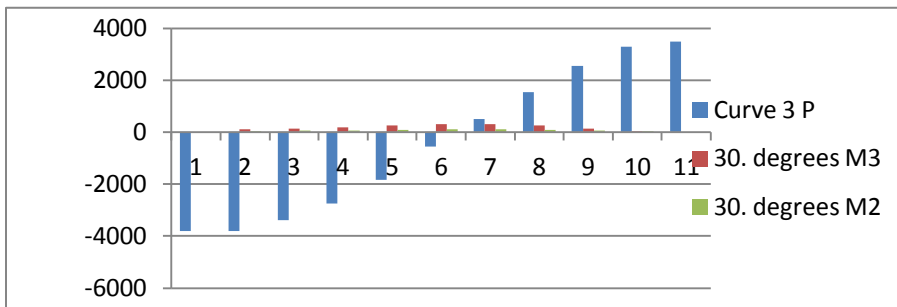
Figure 39

Table 10

Curve 4	45. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	81.1813	52.9308
-3425.61	114.1201	74.7396
-2788.53	158.0658	99.6959
-1855.68	207.3965	126.858
-513.94	249.5861	154.9376
615.5763	246.8412	158.9419
1690.337	200.926	130.4128
2647.731	115.0349	76.639
3316.568	27.5675	18.7938
3492.041	0	0
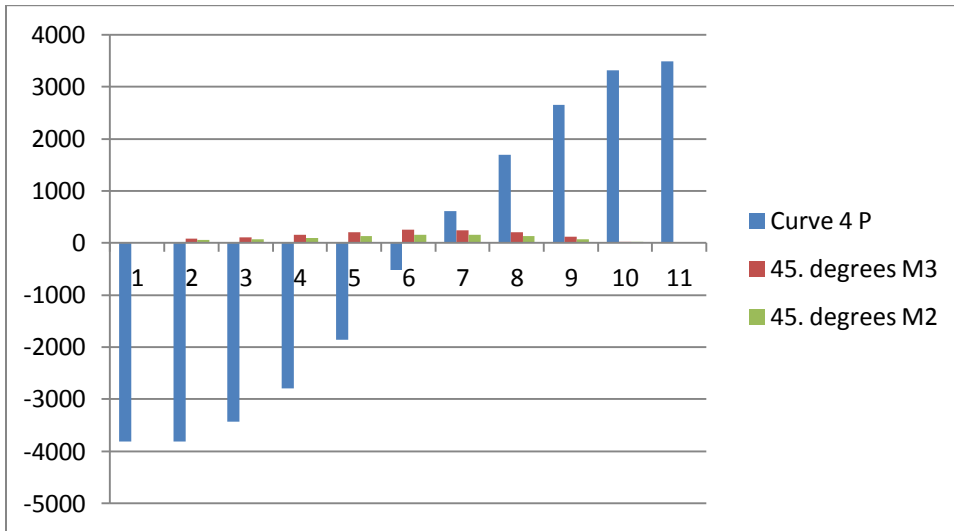
Figure 40

Table 11

Curve 5	60. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	59.7368	71.5918
-3424.97	86.1574	98.111
-2786.26	117.4013	134.0994
-1824.96	149.7527	174.5037
-471.643	184.4768	212.721
638.9825	182.0104	214.7496
1645.874	157.0087	167.1303
2676.669	97.7512	86.2232
3344.638	23.1146	15.9504
3492.041	0	0
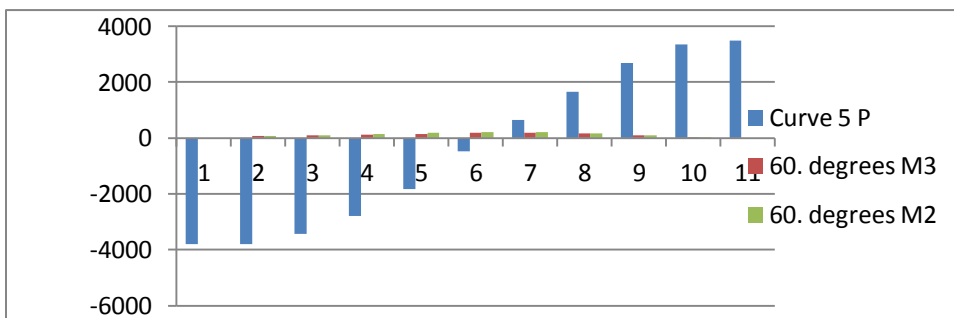
Figure 41

Table 12

Curve 6	75. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	36.5504	91.4335
-3320.87	49.1761	129.1021
-2676.62	62.649	176.3359
-1772.67	80.6622	226.6884
-463.69	103.3424	281.1802
455.6915	95.0449	279.7358
1270.447	95.9407	219.7757
2413.218	93.779	114.6648
3375.416	18.0684	12.9671
3492.041	0	0
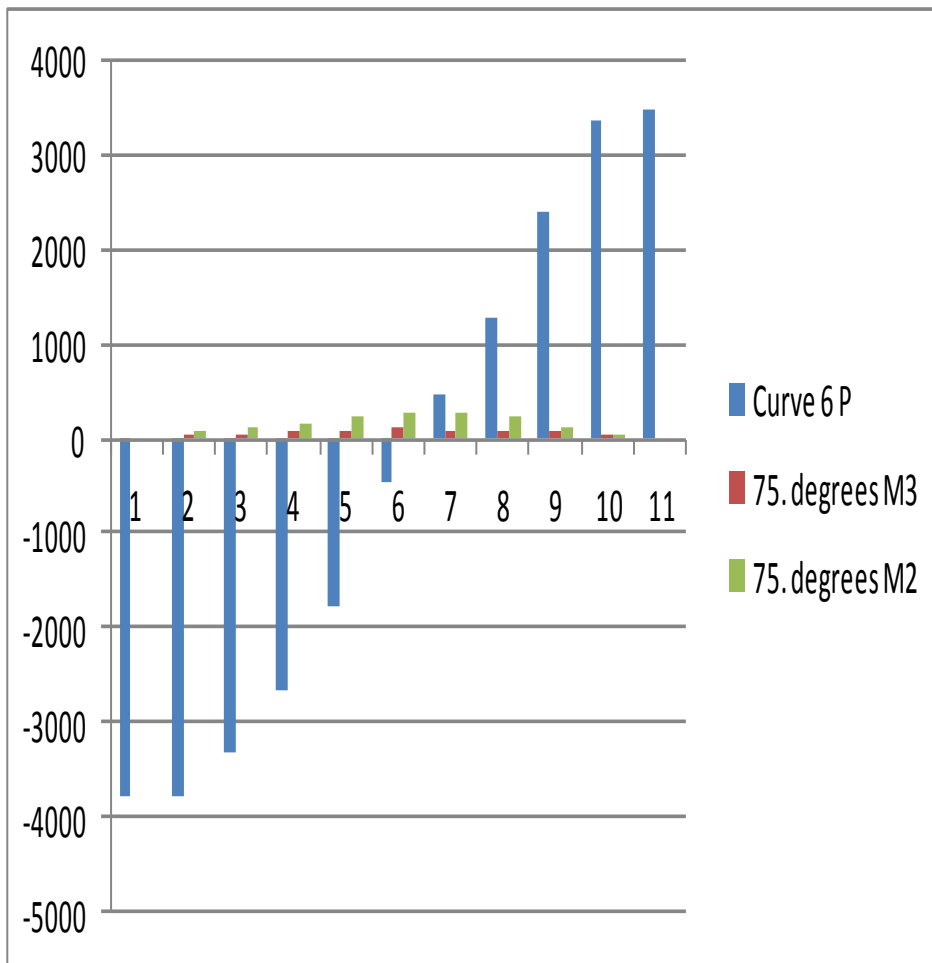
Figure 42

Table 13

Curve 7	90. degrees	
P	M3	M2
-3810.3	0	0
-3416.06	0	126.0205
-2978.64	0	165.3185
-2343.66	0	215.5833
-1495.76	0	278.5432
-487.699	0	346.6093
-129.472	0	341.0493
612.6906	0	302.9814
1320.9	0	230.8886
3196.06	0	34.421
3492.041	0	0
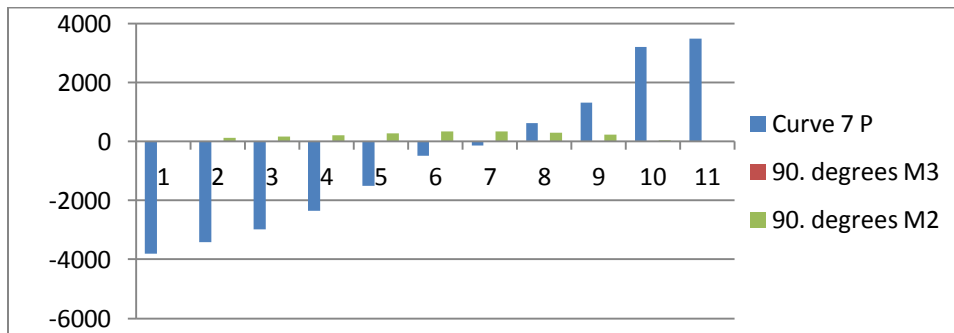
Figure 43

Table 14

Curve 8	105. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-36.5504	91.4335
-3320.87	-49.1761	129.1021
-2676.62	-62.649	176.3359
-1772.67	-80.6622	226.6884
-463.69	-103.342	281.1802
455.6915	-95.0449	279.7358
1270.447	-95.9407	219.7757
2413.218	-93.779	114.6648
3375.416	-18.0684	12.9671
3492.041	0	0
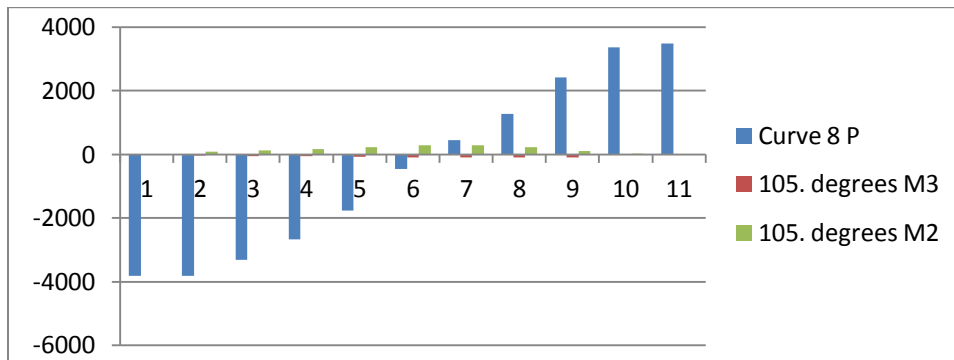
Figure 44

Table 15

Curve 9	120. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-59.7368	71.5918
-3424.97	-86.1574	98.111
-2786.26	-117.401	134.0994
-1824.96	-149.753	174.5037
-471.643	-184.477	212.721
638.9825	-182.01	214.7496
1645.874	-157.009	167.1303
2676.669	-97.7512	86.2232
3344.638	-23.1146	15.9504
3492.041	0	0
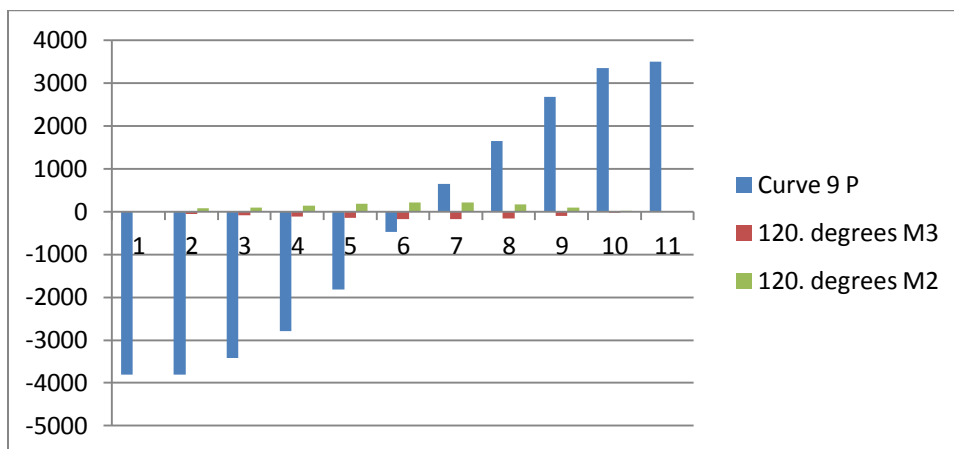
Figure 45

Table 16

Curve 10	135. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-81.1813	52.9308
-3425.61	-114.12	74.7396
-2788.53	-158.066	99.6959
-1855.68	-207.397	126.858
-513.94	-249.586	154.9376
615.5763	-246.841	158.9419
1690.337	-200.926	130.4128
2647.731	-115.035	76.639
3316.568	-27.5675	18.7938
3492.041	0	0
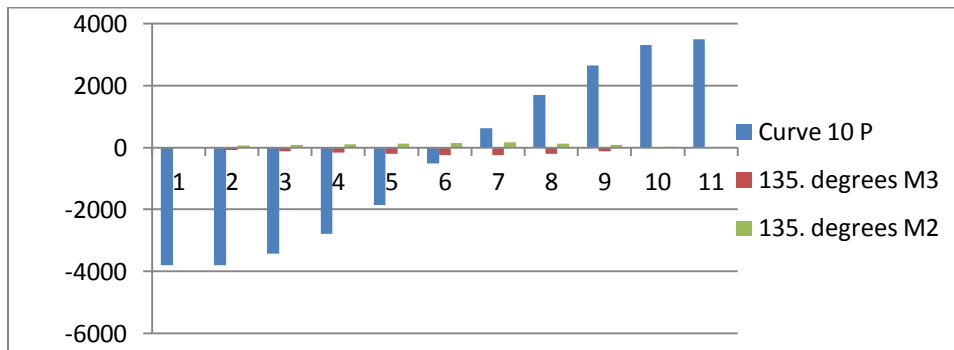
Figure 46

Table 17

Curve 11	150. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-99.3414	36.8919
-3384.43	-141.047	51.1633
-2740.04	-194.043	65.8957
-1846.04	-251.394	82.8778
-566.681	-304.501	102.0691
515.5423	-307.873	101.0081
1531.831	-247.027	95.5849
2548.06	-134.352	73.5455
3287.813	-32.1687	21.6891
3492.041	0	0
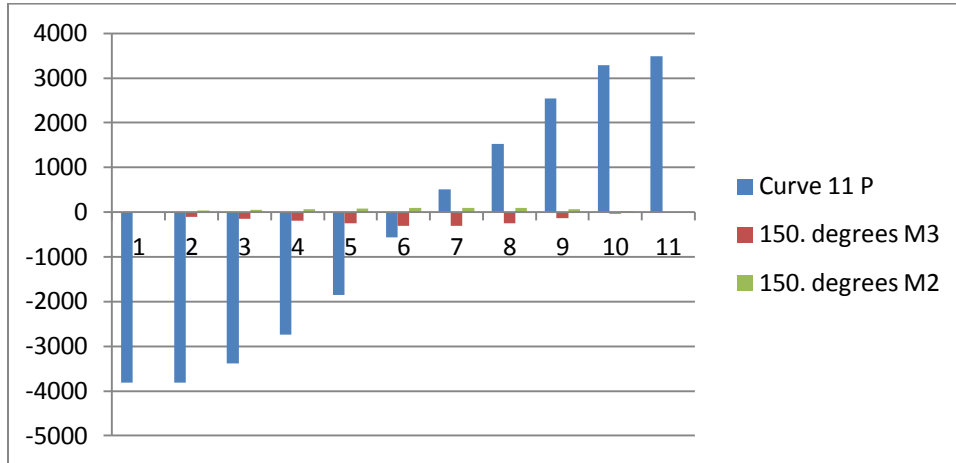
Figure 47

Table 18

Curve 12	165. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-117.44	21.5068
-3263.16	-167.836	26.8775
-2645.34	-224.12	33.7359
-1811.12	-286.534	40.7831
-604.55	-354.768	50.3025
298.0932	-353.422	50.6083
1208.793	-301.545	52.476
2191.1	-194.154	45.665
3249.103	-38.5784	25.1397
3492.041	0	0
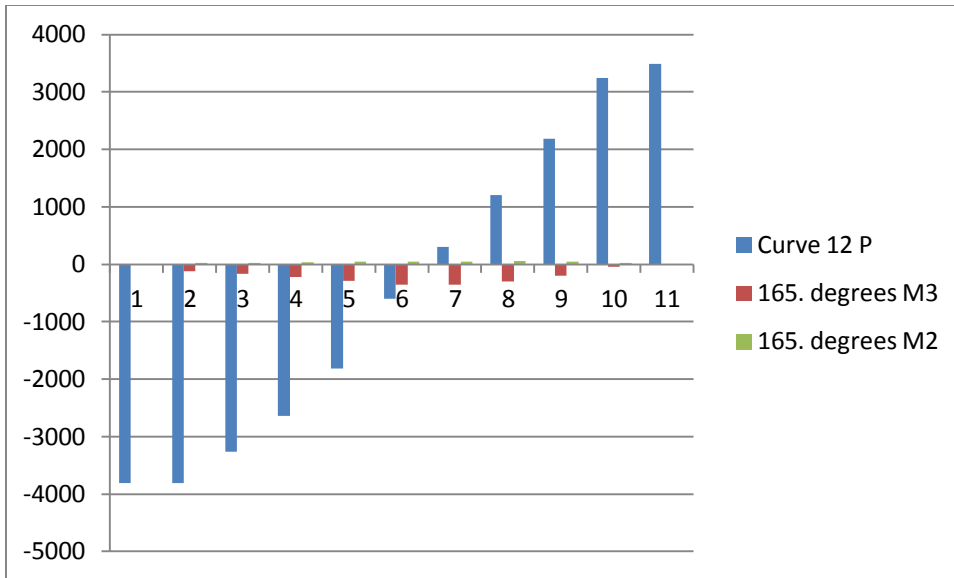
Figure 48

Table 19

Curve 13	180. degrees	
P	M3	M2
-3810.3	0	0
-3511.06	-147.933	0
-3099.22	-195.128	0
-2517.27	-250.879	0
-1674.35	-314.803	0
-612.957	-382.123	0
127.0696	-386.748	0
834.2274	-361.192	0
1662.192	-275.03	0
2655.853	-132.187	0
3492.041	0	0
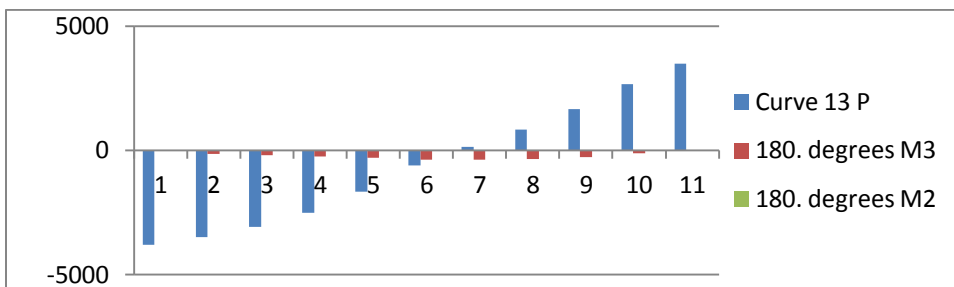
Figure 49

Table 20

Curve 14	195. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-117.44	-21.5068
-3263.16	-167.836	-26.8775
-2645.34	-224.12	-33.7359
-1811.12	-286.534	-40.7831
-604.55	-354.768	-50.3025
298.0932	-353.422	-50.6083
1208.793	-301.545	-52.476
2191.1	-194.154	-45.665
3249.103	-38.5784	-25.1397
3492.041	0	0
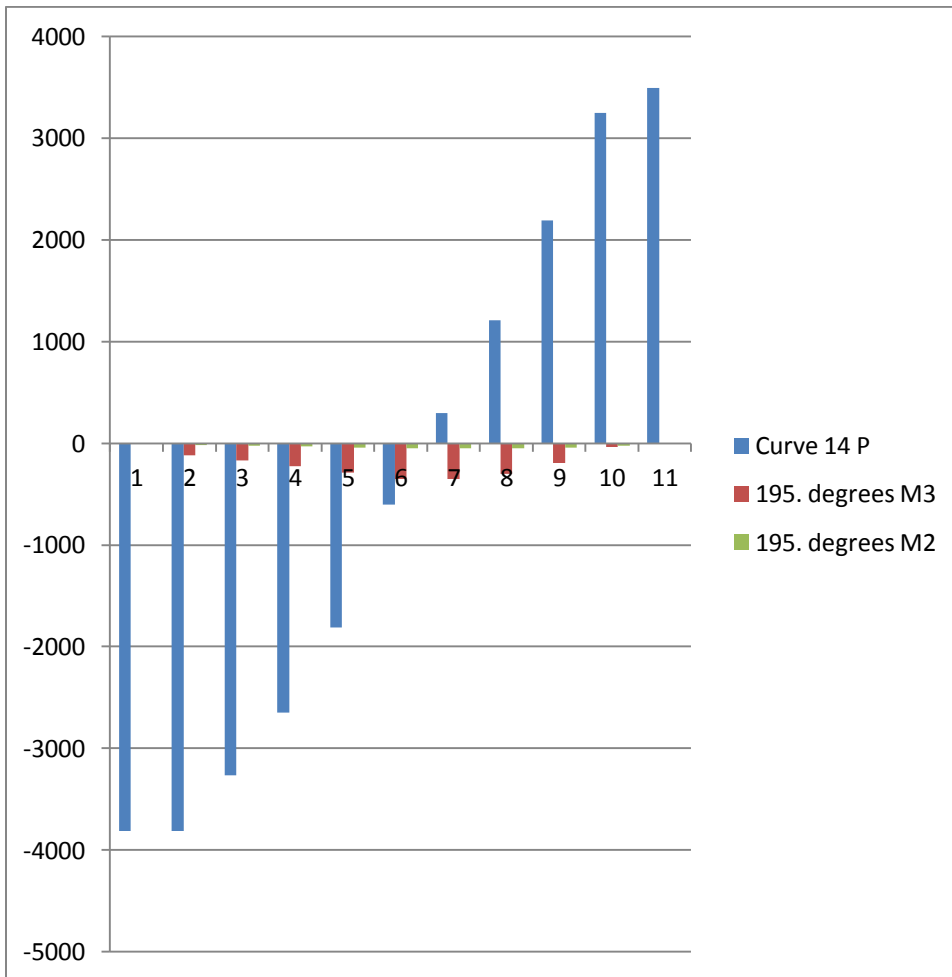
Figure 50

Table 21

Curve 15	210. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-99.3414	-36.8919
-3384.43	-141.047	-51.1633
-2740.04	-194.043	-65.8957
-1846.04	-251.394	-82.8778
-566.681	-304.501	-102.069
515.5423	-307.873	-101.008
1531.831	-247.027	-95.5849
2548.06	-134.352	-73.5455
3287.813	-32.1687	-21.6891
3492.041	0	0
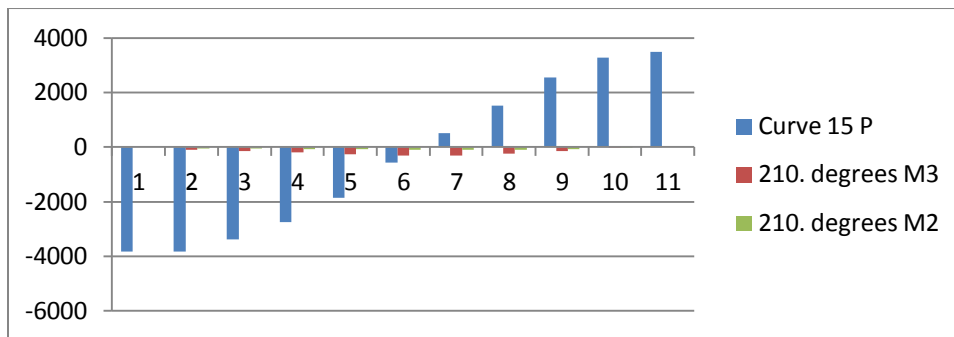
Figure 51

Table 22

Curve 16	225. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-81.1813	-52.9308
-3425.61	-114.12	-74.7396
-2788.53	-158.066	-99.6959
-1855.68	-207.397	-126.858
-513.94	-249.586	-154.938
615.5763	-246.841	-158.942
1690.337	-200.926	-130.413
2647.731	-115.035	-76.639
3316.568	-27.5675	-18.7938
3492.041	0	0
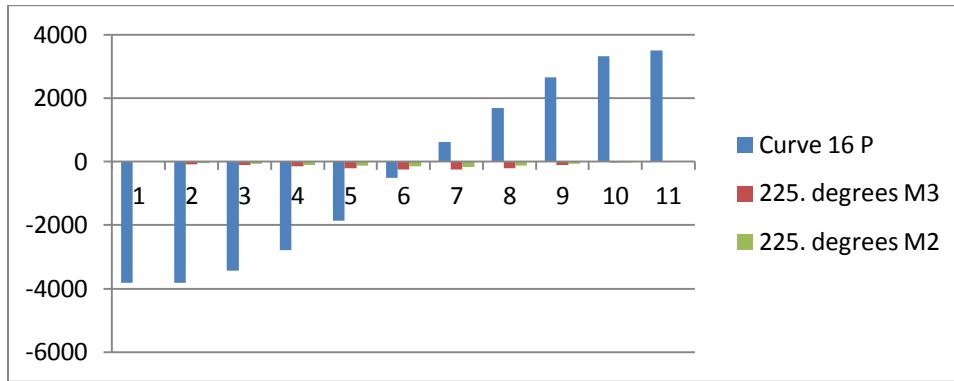
Figure 52

Table 23

Curve 17	240. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-59.7368	-71.5918
-3424.97	-86.1574	-98.111
-2786.26	-117.401	-134.099
-1824.96	-149.753	-174.504
-471.643	-184.477	-212.721
638.9825	-182.01	-214.75
1645.874	-157.009	-167.13
2676.669	-97.7512	-86.2232
3344.638	-23.1146	-15.9504
3492.041	0	0
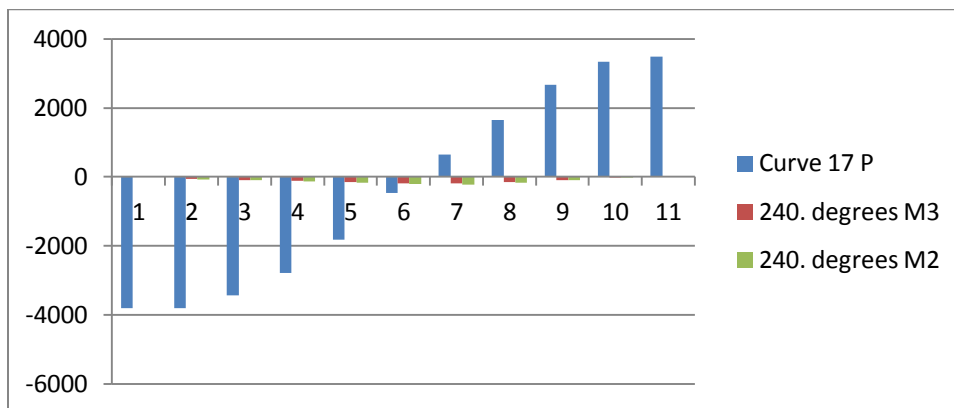
Figure 53

Table 24

Curve 18	255. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	-36.5504	-91.4335
-3320.87	-49.1761	-129.102
-2676.62	-62.649	-176.336
-1772.67	-80.6622	-226.688
-463.69	-103.342	-281.18
455.6915	-95.0449	-279.736
1270.447	-95.9407	-219.776
2413.218	-93.779	-114.665
3375.416	-18.0684	-12.9671
3492.041	0	0
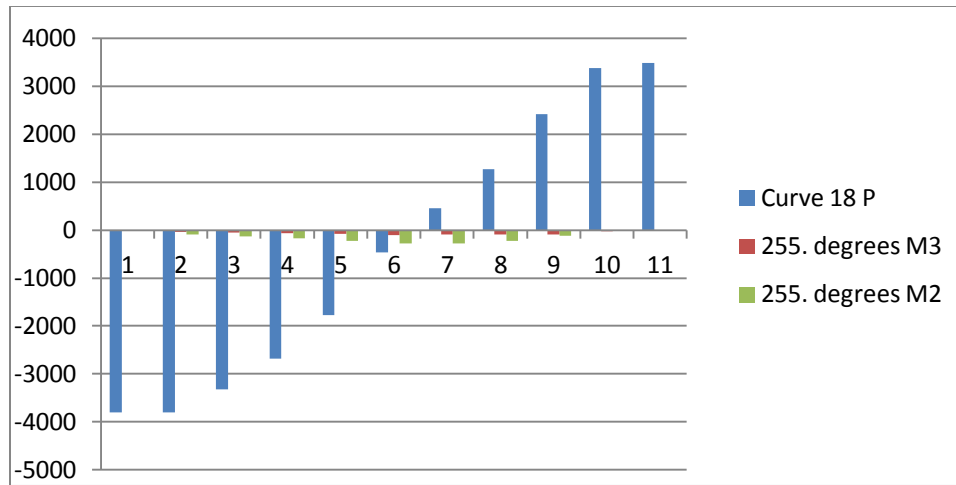
Figure 54

Table 25

Curve 19	270. degrees	
P	M3	M2
-3810.3	0	0
-3416.06	0	-126.021
-2978.64	0	-165.319
-2343.66	0	-215.583
-1495.76	0	-278.543
-487.699	0	-346.609
-129.472	0	-341.049
612.6906	0	-302.981
1320.9	0	-230.889
3196.06	0	-34.421
3492.041	0	0
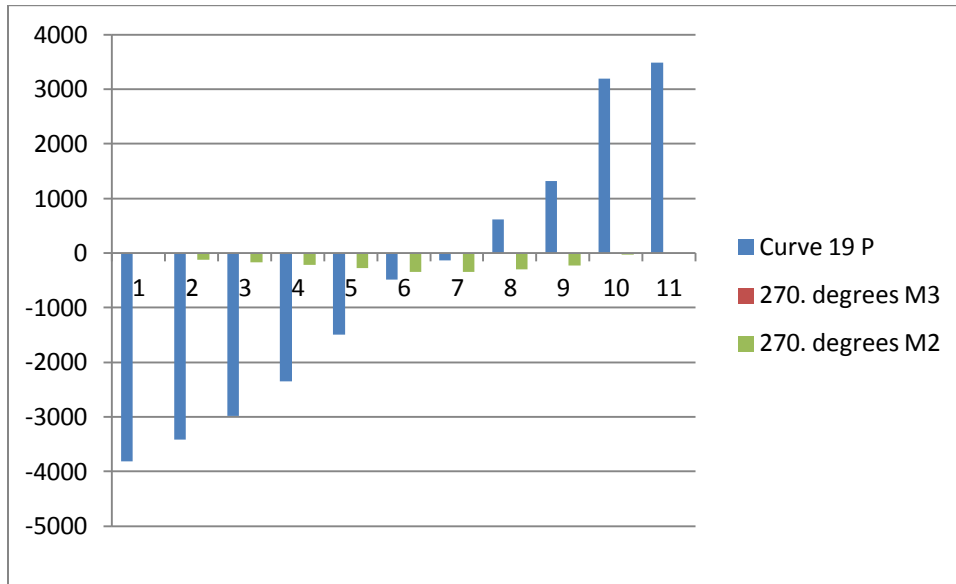
Figure 55

Table 26

Curve 20	285. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	36.5504	-91.4335
-3320.87	49.1761	-129.102
-2676.62	62.649	-176.336
-1772.67	80.6622	-226.688
-463.69	103.3424	-281.18
455.6915	95.0449	-279.736
1270.447	95.9407	-219.776
2413.218	93.779	-114.665
3375.416	18.0684	-12.9671
3492.041	0	0
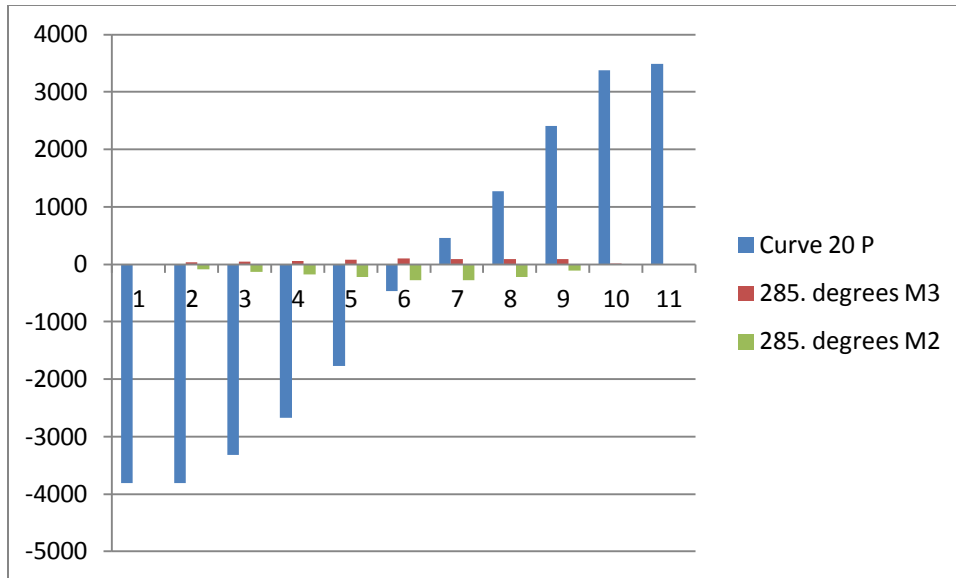
Figure 56

Table 27

Curve 21	300. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	59.7368	-71.5918
-3424.97	86.1574	-98.111
-2786.26	117.4013	-134.099
-1824.96	149.7527	-174.504
-471.643	184.4768	-212.721
638.9825	182.0104	-214.75
1645.874	157.0087	-167.13
2676.669	97.7512	-86.2232
3344.638	23.1146	-15.9504
3492.041	0	0

Figure 57

50
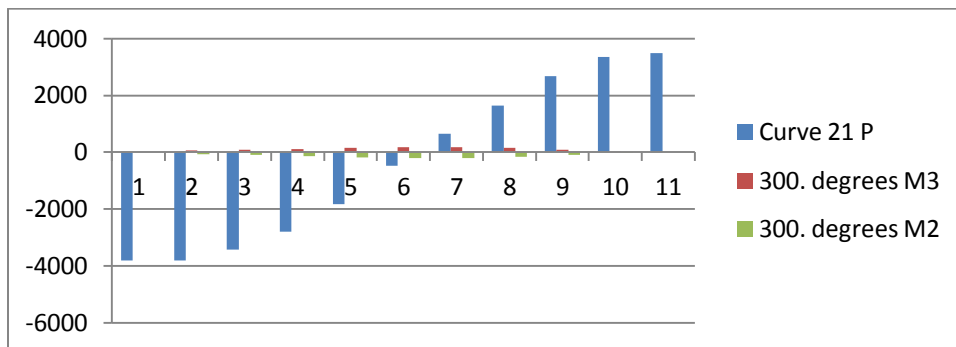
Table 28

Curve 22	315. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	81.1813	-52.9308
-3425.61	114.1201	-74.7396
-2788.53	158.0658	-99.6959
-1855.68	207.3965	-126.858
-513.94	249.5861	-154.938
615.5763	246.8412	-158.942
1690.337	200.926	-130.413
2647.731	115.0349	-76.639
3316.568	27.5675	-18.7938
3492.041	0	0
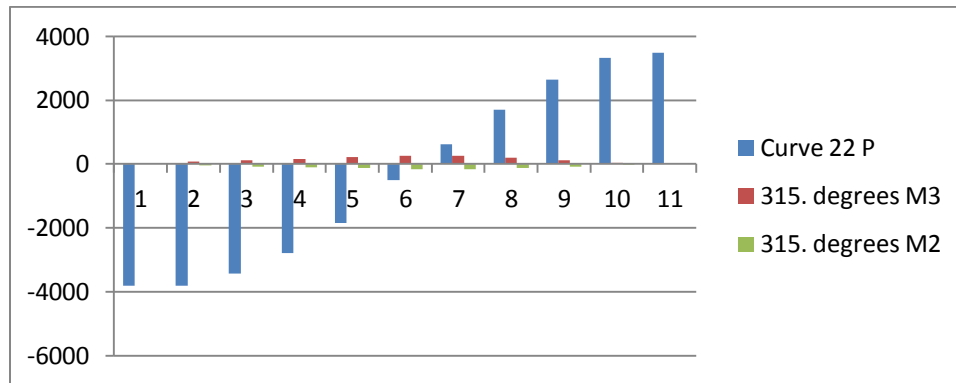
Figure 58

Table 29

Curve 23	330. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	99.3414	-36.8919
-3384.43	141.0471	-51.1633
-2740.04	194.0429	-65.8957
-1846.04	251.3937	-82.8778
-566.681	304.501	-102.069
515.5423	307.8733	-101.008
1531.831	247.0267	-95.5849
2548.06	134.3524	-73.5455
3287.813	32.1687	-21.6891
3492.041	0	0
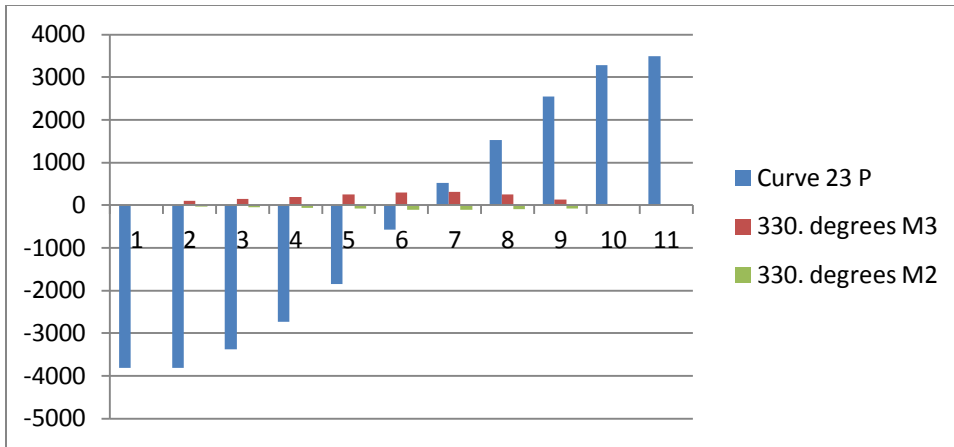
Figure 59

Table 30

Curve 24	345. degrees	
P	M3	M2
-3810.3	0	0
-3810.3	117.4401	-21.5068
-3263.16	167.8358	-26.8775
-2645.34	224.1198	-33.7359
-1811.12	286.5337	-40.7831
-604.55	354.7683	-50.3025
298.0932	353.4224	-50.6083
1208.793	301.5445	-52.476
2191.1	194.1542	-45.665
3249.103	38.5784	-25.1397
3492.041	0	0
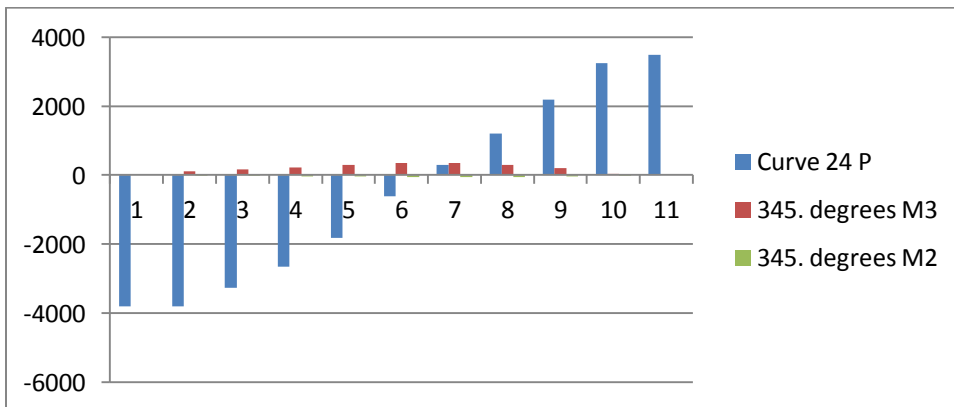
Figure 60

Table 31

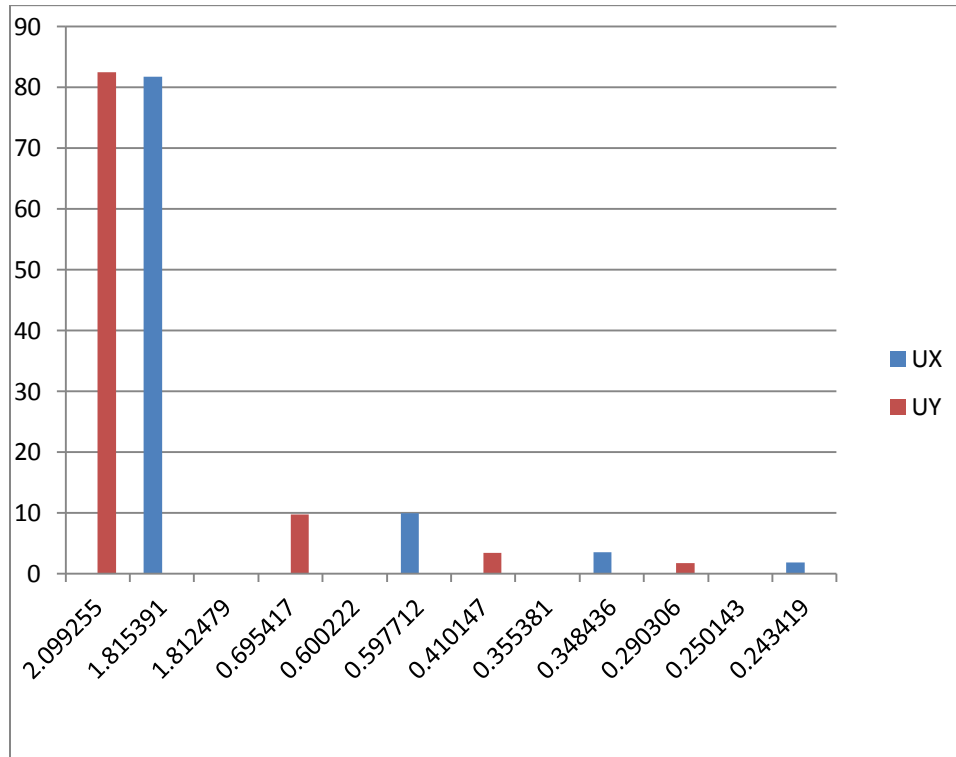| Period	UX	UY	UZ
2.099255	0	82.4199	0
1.815391	81.6922	0	0
1.812479	0	0	0
0.695417	0	9.7496	0
0.600222	0	0	0
0.597712	9.9013	0	0
0.410147	0	3.4065	0
0.355381	0	0	0
0.348436	3.5018	0	0
0.290306	0	1.7429	0
0.250143	0	0	0
0.243419	1.842	0	0

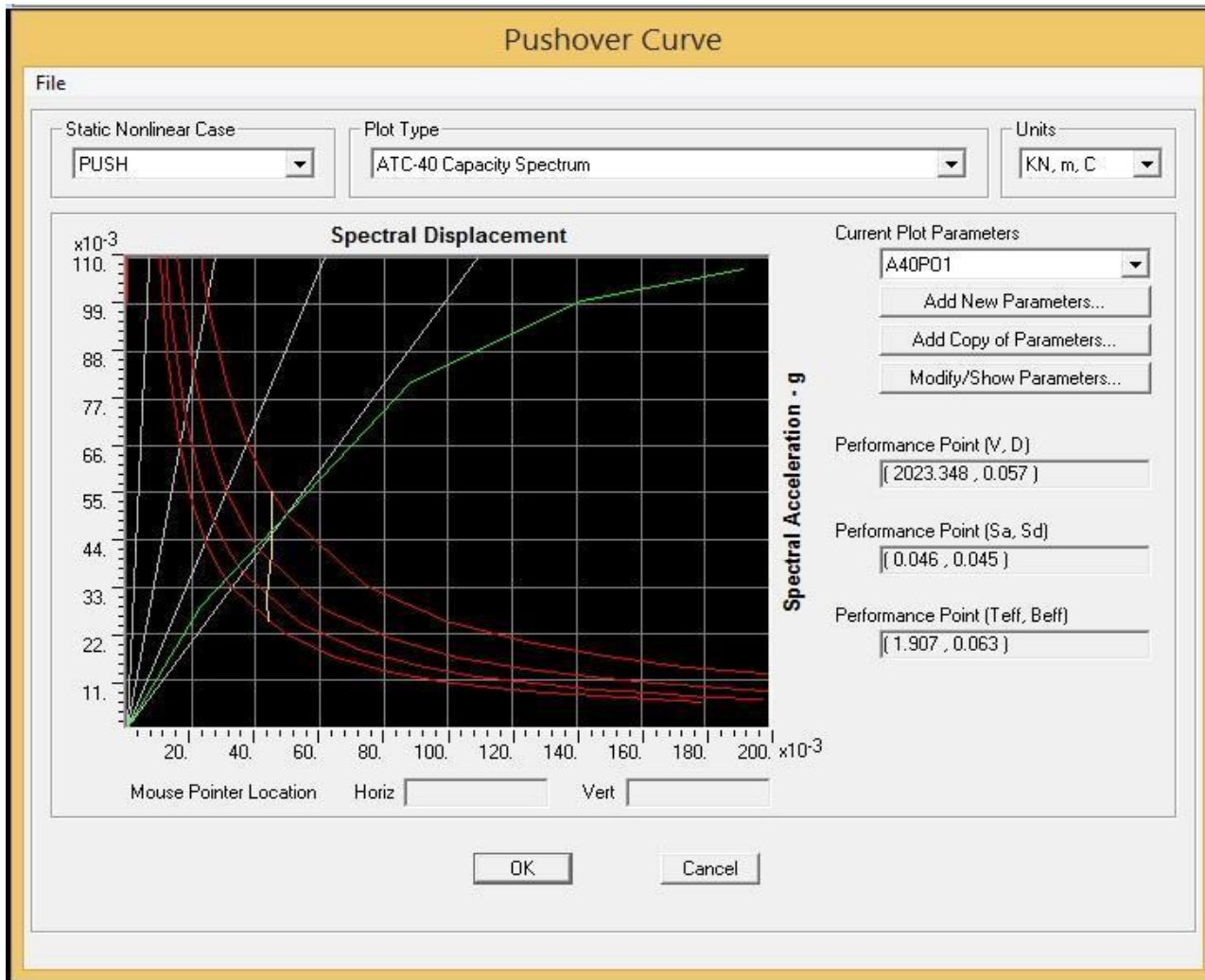Figure 61

Push over capacity curve

Figure 62

5.2 Performance point

The point at which the capacity curve intersects the reduced demand curve represents the performance point at which capacity and demand are equal. As displacement increase, the period of the structure lengthens and reduces demand. Hence, optimum point should have a higher capacity for a lesser displacement.

Lateral Force vs displacement table and curve

Table 32

Displacement	BaseForce
m	KN
0	0
0.029391	1194.955
0.111313	3660.505
0.174341	4582.99
0.230977	4848.772
0.221975	4276.953

Figure 63

Base shear :

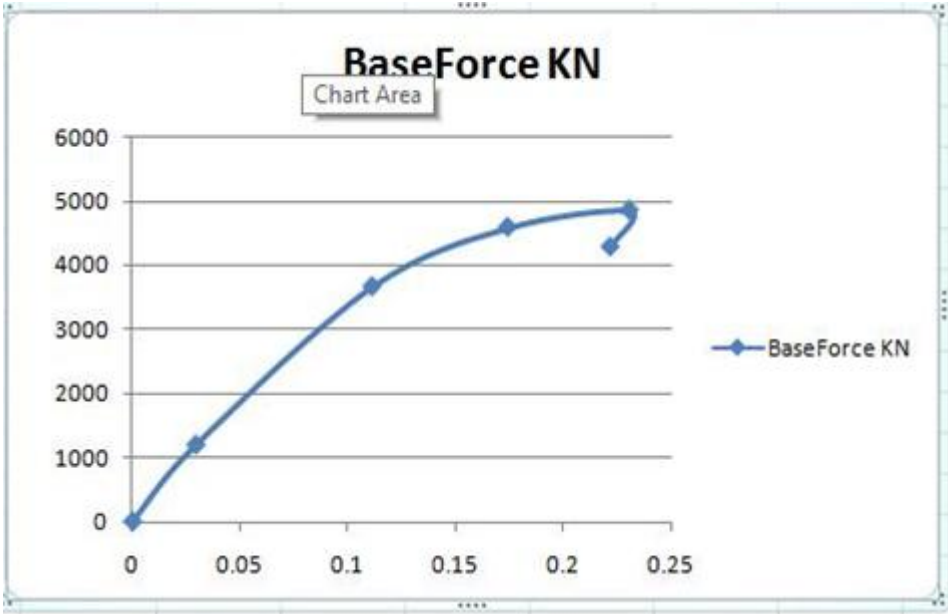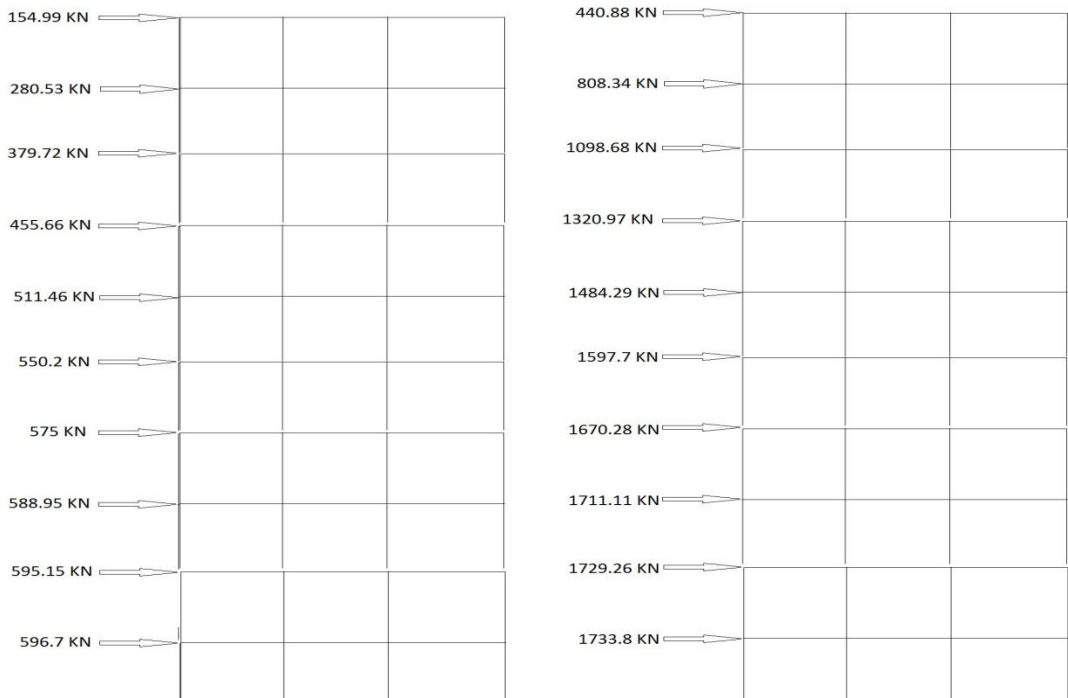| 154.99 KN | 440.88 KN |
280.53 KN	808.34 KN
379.72 KN	1098.68 KN
455.66 KN	1320.97 KN
511.46 KN	1484.29 KN
550.2 KN	1597.7 KN
575 KN	1670.28 KN
588.95 KN	1711.11 KN
595.15 KN	1729.26 KN
596.7 KN	1733.8 KN

Figure 64

CHAPTER 6

6.1 Conclusion

- Building designed with IS 1893:2002 found to have a better performance under given earthquake.

- The design base shear of the building frame is found to be 596.7 KN as per calculation. After performing the analysis the base shear at performance point is found to be 1733.8KN which is greater than design base shear. Since at the performance point base shear is greater than the design base shear the building frame is safe under the earthquake loading.

6.2 Future work

- Time history analysis for actual behavior under earthquake on existing and strengthened building.
- Comparison of existing and strengthened building at various performance levels using pushover and time history hinges results.

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