“PUSHOVER ANALYSIS OF 2D and 3D RC FRAMES”

A PROJECT

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

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

Under the supervision of

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By

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to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

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HIMACHAL PRADESH INDIA

May, 2015

DECLARATION

We hereby declare that the project work presented in this report entitled "*Pushover Analysis of 2D and 3D RC Frames*" submitted for the award of the degree of Bachelor of Technology in Civil Engineering to the Department of Civil Engineering, Jaypee University of Information and Technology Wakhnaghat, has been carried out by us. This work is independent and its main content work has not been submitted for degree at any University in India or Abroad.

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CERTIFICATE

This is to certify that the work which is being presented in the project title **"TITLE OF PROJECT"** in partial fulfillment of the requirements for the award of the degree of Bachelor of technology and submitted in Civil Engineering Department, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out byAditya Sharma and Ankit Mishra during a period from August 2014 to May 2015 under the supervision of **Mr.Anil Dhiman** Assistant Professor, Civil Engineering Department, Jaypee University of Information Technology, Waknaghat.

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

Date: - 27/05/2015

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ABSTRACT

Modelling of complex behaviour of reinforced concrete analytically in its non-linear zone is a challenging task. This has led engineers in the past to rely heavily on empirical formulas which were derived from numerous experiments for the design of reinforced concrete structures. For structural design and assessment of reinforced concrete members, the non-linear analysis has become an important tool. The method can be used to study the behaviour of reinforced concrete structures including force redistribution. This analysis of the nonlinear response of RC structures to be carried out in a routine fashion. It helps in the investigation of the behaviour of the structure under different loading conditions, its load deflection behaviour and the cracks pattern. In the present study, the non-linear response of RCC frame using SAP2000 under the loading has been carried out with the intention to investigate the relative importance of several factors in the non-linear analysis of RCC frames. This includes the variation in load displacement graph.

LIST OF FIGURES

CONTENTS

CHAPTER 1

INTRODUCTION TO PUSHOVER ANALYSIS

1.1 PERFORMANCE BASED SEISMIC DESIGN

Seismic hazard in the context of engineering design is generally defined as the predicted level of ground acceleration which would be exceeded with 10% probability at the site under consideration due to the occurrence of an earthquake anywhere in the region, in the next 50 years. A lot of complex scientific perception and analytical modelling is involved in seismic hazard estimation.

A computational scheme involves the following steps: delineation of seismic source zones and their characterisation, selection of an appropriate ground motion attenuation relation and a predictive model of seismic hazard. Although these steps are region specific, certain standardisation of the approaches is highly essential so that reasonably comparable estimates of seismic hazard can be made worldwide, which are consistent across the regional boundaries.

The National Geophysical Research Institute (NGRI), Hyderabad, India is identified as one such center, responsible for estimating the seismic hazard for the Indian region. As it is well known, earthquake catalogues and data bases make the first essential input for the delineation of seismic source zones and their characterisation.

Thus, preparation of a homogeneous catalogue for a region under consideration is an important task. The data from historic time to recent can broadly be divided in to three temporal categories: 1) since 1964, for which modern instrumentation based data are available 2) 1900-1963, the era of early instrumental data, and 3) pre 1900, consisting of pre-instrumental data, which is based primarily on historical and macro-seismic information. In India, the scenario is somewhat similar.

The next key component of seismic hazard assessment is the creation of seismic source models, which demand translating seismic-tectonic information into a spatial approximation of earthquake localisation and temporal recurrence. For this purpose, all the available data on neo-tectonics, geodynamics, morpho structures etc., need to be compiled and viewed, overlain on a seismicity map. These maps then need to be critically studied for defining areal seismic source zones and active faults.

An earthquake recurrence model is then fitted to these source zones, for defining the parameters that characterise the seismicity of the source region, which go as inputs to the algorithm for the computation of seismic hazard viz. Fig. 1.1 shows a flow chart that presents the key steps in the performance-based design process.

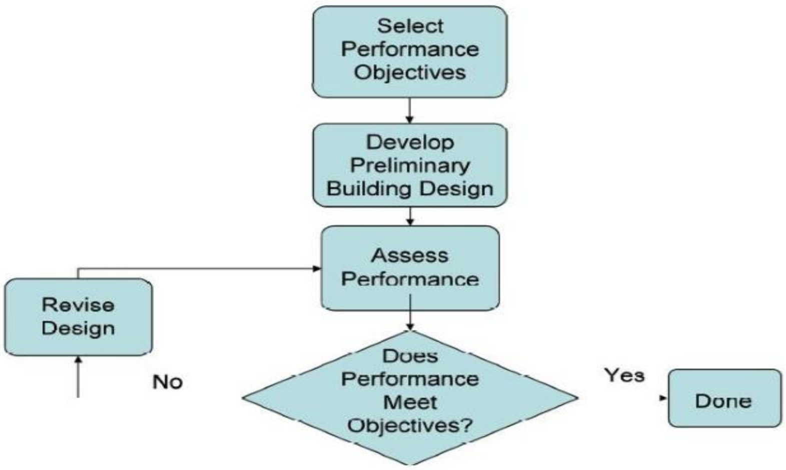


Fig. 1.1 Performance-Based Design Flow Diagram (ATC, 1997a)

1.2 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. Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure.

A two or three dimensional model which includes bilinear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern which is distributed along the building height is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

Pushover analysis can be performed as force-controlled or displacement-controlled. In force controlled pushover procedure, full load combination is applied as specified, i.e, force controlled procedure should be used when the load is known (such as gravity loading).

Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. (Girgin, et al., 2007).

1.3 PURPOSE OF DOING PUSHOVER ANALYSIS

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, moment demands on beam to column connections, shear force demands in reinforced concrete beams, etc.
- Estimates of the deformations demands for elements that have to form in-elastically in order to dissipate the energy imparted to the structure.
- Consequences of the strength deterioration of individual elements on behaviour of the structural system.
- Identification of the critical regions in which the deformation demands are expected to be high and that have to become the focus through detailing.
- Identification of the strength discontinuous 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 (www.architectjaved.com).

1.4 BACKGROUND

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty years and has become the preferred analysis procedure for design and seismic performance evaluation purposes as the procedure is relatively simple and considers post- elastic behaviour. However, the procedure involves certain approximations and simplifications that some amount of variation is always expected to exist in seismic demand prediction of pushover analysis.

Although, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and the reliability of pushover analysis in predicting global and local seismic demands for all structures have been a subject of discussion and improved pushover procedures have been proposed to overcome the certain limitations of traditional pushover procedures.

However, the improved procedures are mostly computationally demanding and conceptually complex that use of such procedures is impractical in engineering profession and codes. As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions.

In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues such as modelling nonlinear member behaviour, computational scheme of the procedure, variations in the predictions of various lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed (Wang. et., 2007).

1.5 DIFFERENT HINGE PROPERTIES FOR PUSHOVER ANALYSIS IN SAP2000

There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements. When these hinge properties are assigned to a frame element, the program automatically creates a different generated hinge property for each and every hinge. Default hinge properties cannot be modified. They also cannot be viewed because the default properties are section dependent. The default properties cannot be fully defined by the program until the section that they apply to is identified.

Thus to see the effect of the default properties, the default property should be assigned to a frame element, and then the resulting generated hinge property should be viewed. The built-in default hinge properties are typically based on FEMA-273 and/or ATC-40 criteria. User-defined hinge properties can be either be based on default properties or they can be fully user-defined. When user-defined properties are based on default properties, the hinge properties cannot be viewed because, again, the default properties are section dependent.

When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They can be viewed, but they cannot be modified. Generated hinge properties have an automatic naming convention of Label H, where Label is the frame element label, H stands for hinge, and # represents the hinge number. The program starts with hinge number 1 and increments the hinge number by one for each consecutive hinge applied to the frame element. For example if a frame element label is F23, the generated hinge property name for the second hinge applied to the frame element is F23H2.

The main reason for the differentiation between defined properties (in this context, defined means both default and user-defined) and generated properties is that typically the hinge properties are section dependent. Thus different frame section type in the model. This could potentially mean that a very large number of hinge properties would need to be defined by the user (SAP2000 tutorials).

1.6 OBJECTIVES

The various aspects of pushover analysis and the accuracy of pushover analysis in predicting seismic demands is investigated by several researchers. However, most of these researches made use of specifically designed structures in the context of the study or specific forms of pushover procedure. The following are the objectives of the present work:

(1) To identify the superiority of pushover analysis over elastic procedures in evaluating the seismic performance of a structure with the advantages and limitations of the procedure.

(2) To perform pushover analyses on case study frames using SAP2000.

(3) To estimate story displacements, inter-story drift ratios and plastic hinge locations performing an improved pushover procedure named Modal Pushover Analysis (MPA) on case study frames.

For this purpose, six deformation levels represented a speak roof displacements the capacity curve of the frames are firstly predetermined and the response parameters such as story displacements, inter-story drift ratios, story shears and plastic hinge locations are then estimated from the results of pushover analyses for any lateral load pattern at the considered deformation level.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

To provide a detailed review of the literature related to modelling of structures in its entirety would be difficult to address in this chapter. A brief review of previous studies on the application of the pushover analysis of structures is presented is this section. This literature review focuses on recent contributions related to pushover analysis of structures and past efforts most closely related to the needs of the present work.

2.2 LITERATURE REVIEW ON PUSHOVER ANALYSIS

Oscar Moller et al. (2009) explained the following conclusions that can be offered as suggestions for further research:

a. Performance-based design in earthquake engineering implies consideration of the uncertainties in the structure.
b. Demands and capacities, in order to evaluate the reliability associated with each of the required performance levels. These reliabilities must satisfy minimum target values for each level.

1. Calculation of the structural responses for the formulation of the limit states equations requires a nonlinear dynamic analysis, and these responses cannot be given in an explicit relationship in terms of the intervening random variables. Discrete data can be obtained for chosen combinations of these variables, and the results can be expressed in terms of response surfaces or neural networks. In this work the latter approach has been followed, providing flexibility and adaptability.
2. The major computational demand in this approach is the construction of the discrete database, executing the nonlinear dynamic analysis for a number of variable combinations representative of the variable ranges. For a fixed combination within a sub-set of the variables, the analysis is carried out for another sub-set which groups variables including different ground motions. For each combination, and over the set of grouped variables, the mean and the standard deviation of each response of interest

are obtained. These statistics are then represented by neural networks, and are utilized in representing the responses in a probabilistic manner.

3. The utilization of neural networks' representation for the response demands makes feasible the calculation of the probability of non-performance via standard Monte Carlo simulation.
4. The reliability associated with each performance level can thus be estimated for different combinations of design parameters, and these reliabilities can themselves be represented by neural networks.
5. The optimization in performance-based design implies the minimization of an objective function (here the total structural cost was used) subject to the achievement of minimum
6. Target reliabilities at each performance level. This work has shown the implementation of an optimization scheme based on a search without calculation of gradients. This scheme is efficient, whether the intermediate reliability constraints are evaluated by simulation at each step, or they are implemented using the reliability neural networks.
7. The optimization scheme for minimum total cost has been applied to a multi-storey, multi-bay reinforced concrete frame, with the design parameters being the depths of beams and columns, and three steel reinforcement ratios. The results show good agreement between the two ways of implementing the calculation of the reliability constraints, and that somewhat different optimum design parameters may correspond to minor differences in the total cost. In particular, the results have shown that it is important the consideration of damage repair costs, as they influence the optimum solution.
8. This work has shown that neural networks offer a very useful tool to represent the relationship between structural responses and the intervening random variables, and between achieved reliabilities and the design parameters. The first application make feasible the use of Monte Carlo simulation to estimate reliabilities or probabilities of non-performance, while the second improves the efficiency of the optimization algorithm when intermediate reliabilities need to be evaluated.
9. The approach presented introduced a general scheme for reliability estimation and performance-based design optimization in earthquake engineering. It introduced required concepts like a relationship between damage level and repair cost – a

relationship that still needs further general development and should be the objective of continuing research.

10. Continuing research should also be focused on damage parameters and their relationship to calculated quantities like strains and displacements. Here a well-known damage index was used for the purpose of the application, but further research should be focused on how damage accumulates over time as a result of the applied strains or displacement history.

J.P. Moehle (2008) presented a performance based seismic design of tall buildings in the U.S. He presented that the building codes in the United States contain prescriptive requirements for seismic design as well as an option for use of alternative provisions. Increasingly these alternative provisions are being applied for the performance-based seismic design of tall buildings. Application of performance-based procedures requires:

An understanding of the relation between performance and nonlinear response; selection and manipulation of ground motions appropriate to the seismic hazard; selection of appropriate nonlinear models and analysis procedures; interpretation of results to determine design quantities based on nonlinear dynamic analysis procedures; appropriate structural details; and peer review by independent qualified experts to help assure the building official that the proposed materials and system are acceptable. Both practice- and research-oriented aspects of performance-based seismic design of tall buildings are presented. He said that the west coast of the United States, a highly seismic region, is seeing a resurgence in the design and construction of tall buildings (defined here as buildings 240 feet (73 meters) or taller). Many of these buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildings codes. In many cases, prescriptive provisions of governing building codes are found to be overly restrictive, leading to designs that are outside the limits of the code prescriptive provisions.

This is allowable through the alternative provisions clause of building codes. When the alternative provisions clause is invoked, this normally leads to a performance-based design involving development of a design-specific criteria, site specific seismic hazard analysis, selection and modification of ground motions, development of a nonlinear computer analysis model of the building, performance verification analyses, development of building-specific details, and peer review by tall buildings design experts. His views about the new generation of tall buildings in the western U.S. are that urban regions along the west coast of the United

States are seeing a boom in tall building construction. To meet functional and economic requirements, many of the new buildings are using specialized materials and lateral-force-resisting systems that do not meet the prescriptive definitions and requirements of current building codes.

According to Moehle's a design criteria document generally is developed by the designer to clearly and concisely communicate to the design team, the building official, and the peer reviewers the intent and the process of the building structural design. A well prepared document will likely include data and discussion regarding the building and its location; the seismic and wind force-resisting systems; sample conceptual drawings; codes and references that the design incorporates in part or full; exceptions to aforementioned code prescriptive provisions; performance objectives; gravity, seismic, and wind loading criteria; load combinations; materials; methods of analysis including software and modeling procedures; acceptance criteria; and test data to support use of new components.

The document is prepared early for approval by the building official and peer reviewers, and may be modified as the design advances and the building is better understood. The design criteria document must define how the design is intended to meet or exceed the performance expectations inherent in the building code. Performance-based seismic analysis of tall buildings in the U.S. increasingly uses nonlinear analysis of a three-dimensional model of the building. Lateral force-resisting components of the building are modelled as discrete elements with lumped plasticity or fiber models that represent material nonlinearity and integrate it across the component section and length. Gravity framing elements increasingly are being included in the nonlinear models so that effects of building deformations on the gravity framing as well as effects of the gravity framing on the seismic system Because the behavior is nonlinear, behavior at one hazard level cannot be scaled from nonlinear results at another hazard level.

Furthermore, conventional capacity design approaches can underestimate internal forces in some structural systems (and overestimate them in others) because lateral force profiles and deformation patterns change as the intensity of ground shaking increases (**Kabeyasawa, Eberhard et al., 1993**). Results of non-linear dynamic analysis are sensitive to modelling assumptions. A significant percentage of recent high-rise building construction in the western U.S. has been for residential and mixed-use occupancies.

Thus, much of it has been of reinforced concrete, and the majority of those have used reinforced concrete core walls. Some concrete and steel framing, and some steel walls, also are used. Under design-level earthquake ground motions, the core wall may undergo inelastic deformations near the base (and elsewhere) in the presence of high shear.

Ductile performance requires an effectively continuous tension chord, adequately confined compression zone, and adequate proportions and details for shear resistance. In locations where yielding is anticipated, splices (either mechanical or lapped) must be capable of developing forces approaching the bar strength. Furthermore, longitudinal reinforcement is to be extended a distance $0.8l_w$ past the point where it is no longer required for flexure based on conventional section flexural analysis, where lw is the (horizontal) wall length. Walls generally are fully confined at the base and extending into subterranean levels. Confinement above the base may be reduced (perhaps by half) where analysis shows reduced strains, though strains calculated by nonlinear analysis software generally should be viewed sceptically as they are strongly dependent on modeling assumptions (modeling procedures should be validated by the engineer of record against strains measured in laboratory tests).
The reduced confinement usually continues up the wall height until calculated demands under maximum expected loadings are well below spalling levels. Transverse reinforcement for wall shear generally is developed to the far face of the confined boundary zone; otherwise, the full length of the wall is not effective in resisting shear.

Coupled core walls require ductile link beams that can undergo large inelastic rotations. Away from the core walls, gravity loads commonly are supported by post-tensioned floor slabs supported by columns. Slab-column connections are designed considering the effect of lateral drifts on the shear punching tendency of the connection.

For post-tensioned slabs, which are most common, at least two of the strands in each direction must pass through the column cage to provide post-punching resistance. He concluded that Performance-based earthquake engineering increasingly is being used as an approach to the design of tall buildings in the U.S. Available software, research results, and experience gained through real building applications are providing a basis for effective application of nonlinear analysis procedures. Important considerations include definition of performance objectives, selection of input ground motions, construction of an appropriate nonlinear analysis model, and judicious interpretation of the results.

Implemented properly, nonlinear dynamic analysis specific to the structural system and seismic environment is the best way to identify nonlinear dynamic response characteristics, including yielding mechanisms, associated internal forces, deformation demands, and detailing requirements. Proportions and details superior to those obtained using the prescriptive requirements of the building code can be determined by such analysis, leading to greater confidence in building performance characteristics including serviceability and safety. Although performance-based designs already are under way and are leading to improved designs, several research needs have been identified, the study of which can further improve design practices.

Ceroni et al., (2007) formulated that ductility of R.C. elements has been widely studied either experimentally and theoretical since its evaluation is basic to carry out a reliable nonlinear analysis of structures; post-elastic deformability is a resource for redistributing stresses in a structure to increase the ultimate load but, above all, to absorb and dissipate energy during major earthquakes. However, the problem remains open and models still need an improvement in two directions. On one side, mechanical models can be implemented to take into account constructive details, shear-flexure interaction, size effects as well as non-linear constitutive relationship of materials and steel-concrete bond. On the other side, simplified approaches have to be assessed in order to allow an easy but reliable ductility evaluation without using any sophisticated analytical model, generally not very designers friendly. In this paper a wide parametric analysis with a refined model is carried out in order to build on a reliable formulation for the plastic hinge length of R.C. columns subjected to axial and flexural load. The model used to analyse the non-linear behaviour of the element and to estimate the plastic rotation is a point by point model, including an explicit formulation of the bond slip relationship and capable to take into account the effect of the distributed and concentrated non-linearity, as the spread of plasticity along the member and the fixed end rotation. Its efficiency has been already successfully applied to experimental comparison.

The rotational capacity evaluated by the model varying some parameters allows a clear understanding of the futures influence involved in the structural problem. Ductility of r.c. elements depends on behaviour of the cracked section, which is well represented by moment-curvature relationship; the ratio of ultimate curvature to the one at first yielding is called section ductility. If the rotational capacity has to be calculated in actual cases, models based on the evaluation of a plastic hinge length are very useful thanks to their procedure simplicity. It is therefore surely interesting to review the evaluation of the plastic hinge length Lp using the detailed model.

$$Lp = Lp^{I} + Lp^{II}$$

Where, Lp^{I} is due to the plastic rotation of the column and Lp^{II} to the fixed end rotation at the footing zone of the column.

In order to extrapolate a formulation for Lp^{I} and Lp^{II}, a wide parametric analysis has been developed in the same hypothesis explained in the previous paragraph. The column considered has length L equal to 1.5 m, 2 m, 2.5 m, 3 m and a square cross section with side H equal to 30 and 60 cm symmetrically reinforced; the combination of values of L and H gives back, for the ratio L/H, the values of 3.33, 5, 6.67, 8.33 and 10. The concrete strength in compression is f_c = 30 MPa and the volumetric percentage of stirrups is 0.1%. The ratio f_t/f_y varies in the range 1.05-1.45; the ultimate strain of steel εu does vary in the range 0.04-0.16. Three diameters of steel bar, db, (10, 16 and 20 mm) are considered. The values of the ratio N/N_u considered are 0, 0.1, 0.2, 0.3, and 0.4.

The influence on the plastic hinge of the ratio between an element typical length (distance of critical section to the point of contraflexure, shear span…) and the section height has been already pointed out by **(Baker. et., al, 1965)**, who also explicitly introduced the influence of the ratio N/Nu, while the steel properties and concrete strength were considered as factors for mild and cold-worked steel. Since then, laying on experimental results and empirical considerations, other expressions have been proposed aimed to simplify the formulation of Lp reducing the number of parameters and considering only the influence of geometrical properties of an element (length, height of section). The influence of steel bar diameter was taken into account by **(Priestley and Park. 1987)**, based on the analysis of experimental tests on 20 columns:

$$Lp = 0.08L + 6db$$

where L is the distance from the point of contraflexure of the column to the section of maximum moment and db the bars diameter; the first and second terms of the formulation represent the Lp^{I} and Lp^{II} contributions, both independent from the steel characteristics. The variables examined in the experimental tests were the section shape (square, rectangular and circular cross section), the longitudinal and lateral reinforcement content and the loading rate. The effect of axial load and steel properties was not analysed. Later on, in **(B.I.A. 1996)** a modification of the previous expression was proposed introducing the effect of the steel yielding stress:

$$Lp = 0.08L + 0.022f_y\, db$$

2.3. GAPS IN RESEARCH AREA

Many experimental and analytical works has been done by many researchers in the area of the pushover analysis of the structural members. The concept of pushover analysis is rapidly growing nowadays. This research is concerned with the pushover analysis of the RCC building. The use of pushover analysis of the structure have been studied extensively in previous studies. However, many researchers performed experimentally and analytically on the pushover analysis but limited work is done on the study of pushover analysis using SAP2000 with user defined hinges.

2.4 CLOSURE

The literature review has suggested that use of a pushover analysis of the RCC frame is feasible. So it has been decided to use SAP2000 for the modelling. With the help of this software study of RC frame has been done. It gives the load deflection curve of the building.

CHAPTER 3

PUSHOVER ANALYSIS

3.1 GENERAL

Pushover Analysis option will allow engineers to perform pushover analysis as per FEMA 356 and ATC-40. Pushover analysis is a static, nonlinear procedure using simplified nonlinear technique to estimate seismic structural deformations. It is an incremental static analysis used to determine the force-displacement relationship, or the capacity curve, for a structure or structural element. The analysis involves applying horizontal loads, in a prescribed pattern, to the structure incrementally, i.e. pushing the structure and plotting the total applied shear force and associated lateral displacement at each increment, until the structure or collapse condition. (**Sermin, 2005**).

Pushover analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., inverted triangular or uniform). The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formation, and failure of various structural components is recorded. Pushover analysis can provide a significant insight into the weak links in seismic performance of a structure.

A series of iterations are usually required during which, the structural deficiencies observed in one iteration, are rectified and followed by another.

This iterative analysis and design process continues until the design satisfies a pre-established performance criteria. The performance criteria for pushover analysis is generally established as the desired state of the building given a roof-top or spectral displacement amplitude.

Static Nonlinear Analysis technique, also known as sequential yield analysis, or simply “pushover” analysis has gained significant popularity during the past few years. It is the one of the three analysis techniques recommended by FEMA-273/274 and a main component of the Spectrum Capacity Analysis method (ATC-40). Proper application can provide valuable insights into the expected performance of structural systems and components. Misuse can lead to an erroneous understanding of the performance characteristics. Unfortunately, many engineers are unaware of the details that have to observed in order to obtain useful results from such analysis. **(Zou. et., al. 2005).**

3.2 LIMITATIONS OF PUSHOVER ANALYSIS

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

Target displacement is the global displacement expected in a design earthquake. The roof displacement at mass centre of the structure is used as target displacement. The accurate estimation of target displacement associated with specific performance objective affect the accuracy of seismic demand predictions of pushover analysis. However, in pushover analysis, generally an invariant lateral load pattern is used that the distribution of inertia forces is assumed to be constant during earthquake and the deformed configuration of structure under the action of invariant lateral load pattern is expected to be similar to that experienced in design earthquake. As the response of structure, thus the capacity curve is very sensitive to the choice of lateral load distribution, selection of lateral load pattern is more critical than the accurate estimation of target displacement. The lateral load patterns used in pushover analysis are proportional to product of story mass and displacement associated with a shape vector at the story under consideration. Commonly used lateral force patterns are uniform, elastic first mode, "code" distributions and a single concentrated horizontal force at the top of structure. Multi-modal load pattern derived from Square Root of Sum of Squares (SRSS) story shears is also used to consider at least elastic higher mode effects for long period structures. These loading patterns usually favour certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground motion and inelastic dynamic response characteristics of the structure. Moreover, invariant lateral load patterns could not predict potential failure modes due to middle or upper story mechanisms caused by higher mode effects. These limitations have led many researchers to propose adaptive load patterns which consider the changes in inertia forces with the level of inelasticity. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. Although some improved predictions have been obtained from adaptive load patterns, they make pushover analysis computationally demanding and conceptually complicated.

The scale of improvement has been a subject of discussion that simple invariant load patterns are widely preferred at the expense of accuracy. Whether lateral loading is invariant or adaptive, it is applied to the structure statically that a static loading cannot represent inelastic dynamic response with a large degree of accuracy. (**Wikipedia)**

3.3 FRAME ELEMENT IN SAP2000

In SAP2000, a frame element is modeled as a line element having linearly elastic properties and nonlinear force-displacement characteristics of individual frame elements are modeled as hinges represented by a series of straight line segments. A generalized force-displacement characteristic of a non-degrading frame element (or hinge properties) in SAP2000.

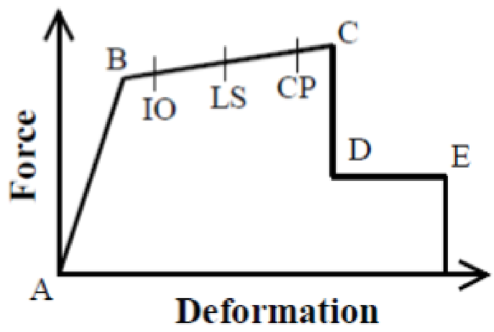


Fig. 3.1 Force-Deformation for Pushover Hinge (Habibullah. et al., 1998)

Point A corresponds to unloaded condition and point B represents yielding of the element. The ordinate at C corresponds to nominal strength and abscissa at C corresponds to the deformation at which significant strength degradation begins. The drop from C to D represents the initial failure of the element and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Beyond point E, the maximum deformation capacity, gravity load can no longer be sustained. Hinges can be assigned at any number of locations (potential yielding points) along the span of the frame element as well as element ends. Uncoupled moment (M2 andM3), torsion (T), axial force (P) and shear (V2 and V3) force-displacement relations can be defined. As the column axial load changes under lateral loading, there is also a coupledPM2-M3 (PMM) hinge which yields based on the interaction of axial force and bending moments at the hinge location.

Also, more than one type of hinge can be assigned at the same location of a frame element. There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements.

When these hinge properties (default and user-defined) are assigned to a frame element, the program automatically creates a new generated hinge property for each and every hinge.

Default hinge properties could not be modified and they are section dependent. When default hinge properties are used, the program combines its built-in default criteria with the defined section properties for each element to generate the final hinge properties.

The built-in default hinge properties for steel and concrete members are based on ATC-40 and FEMA-273 criteria.

User-defined hinge properties can be based on default properties or they can be fully userdefined. When user-defined properties are not based on default properties, then the properties can be viewed and modified. The generated hinge properties are used in the analysis. They could be viewed, but they could not be modified. (**Habibullah. et al., 1998)**

3.4 METHODS OF ANALYSIS

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to determine force and displacement demands in various components of the structure. Several analysis methods, both elastic and inelastic, are available to predict the seismic performance of the structures. (**Sermin, 2005**)

3.4.1 ELASTIC METHODS OF ANALYSIS

The force demand on each component of the structure is obtained and compared with available capacities by performing an elastic analysis. Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods are also known as force-based procedures which assume that structures respond elastically to earthquakes. In code static lateral force procedure, a static analysis is performed by subjecting the structure to lateral forces obtained by scaling down the smoothened soil-dependent elastic response spectrum by a structural system dependent force reduction factor, "R".

In this approach, it is assumed that the actual strength of structure is higher than the design strength and the structure is able to dissipate energy through yielding. In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis.

The dynamic analysis may be either a response spectrum analysis or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis. Any effect of higher modes are automatically included in time history analysis.

In demand/capacity ratio (DCR) procedure, the force actions are compared to corresponding capacities as demand/capacity ratios. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the full earthquake demand by an R-factor, the DCR approach takes the full earthquake demand without reduction and adds it to the gravity demands. DCRs approaching 1.0 (or higher) may indicate potential deficiencies.

Although force-based procedures are well known by engineering profession and easy to apply, they have certain drawbacks. Structural components are evaluated for serviceability in the elastic range of strength and deformation. Post-elastic behavior of structures could not be identified by an elastic analysis. However, post-elastic behaviour should be considered as almost all structures are expected to deform in inelastic range during a strong earthquake.

The seismic force reduction factor "R" is utilized to account for inelastic behavior indirectly by reducing elastic forces to inelastic. Force reduction factor, "R", is assigned considering only the type of lateral system in most codes, but it has been shown that this factor is a function of the period and ductility ratio of the structure as well.

Elastic methods can predict elastic capacity of structure and indicate where the first yielding will occur, however they don’t predict failure mechanisms and account for the redistribution of forces that will take place as the yielding progresses.

Real deficiencies present in the structure could be missed. Moreover, force-based methods primarily provide life safety but they can’t provide damage limitation and easy repair. The drawbacks of force-based procedures and the dependence of damage on deformation have led the researches to develop displacement-based procedures for seismic performance evaluation. Displacement-based procedures are mainly based on inelastic deformations rather than elastic forces and use nonlinear analysis procedures considering seismic demands and available capacities explicitly. (**Sermin, 2005)**

3.4.2 INELASTIC METHODS OF ANALYSIS

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis.

The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics. It requires proper modeling of cyclic load-deformation characteristics considering deterioration properties of all important components. Also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics.

Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method. **(Sermin, 2005)**

3.5 BUILDING PERFORMANCE LEVELS AND RANGES (ATC, 1997a)

3.5.1 PERFORMANCE LEVEL*:* the intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

3.5.2 PERFORMANCE RANGE: a range or band of performance, rather than a discrete level.

3.5.3 DESIGNATIONS OF PERFORMANCE LEVELS AND RANGES: Performance is separated into descriptions of damage of structural and non-structural systems; structural designations are S-1 through S-5 and non-structural designations are N-A through N-D.

3.5.4 BUILDING PERFORMANCE LEVEL: The combination of a Structural Performance Level and a Non-structural Performance Level to form a complete description of an overall damage level.

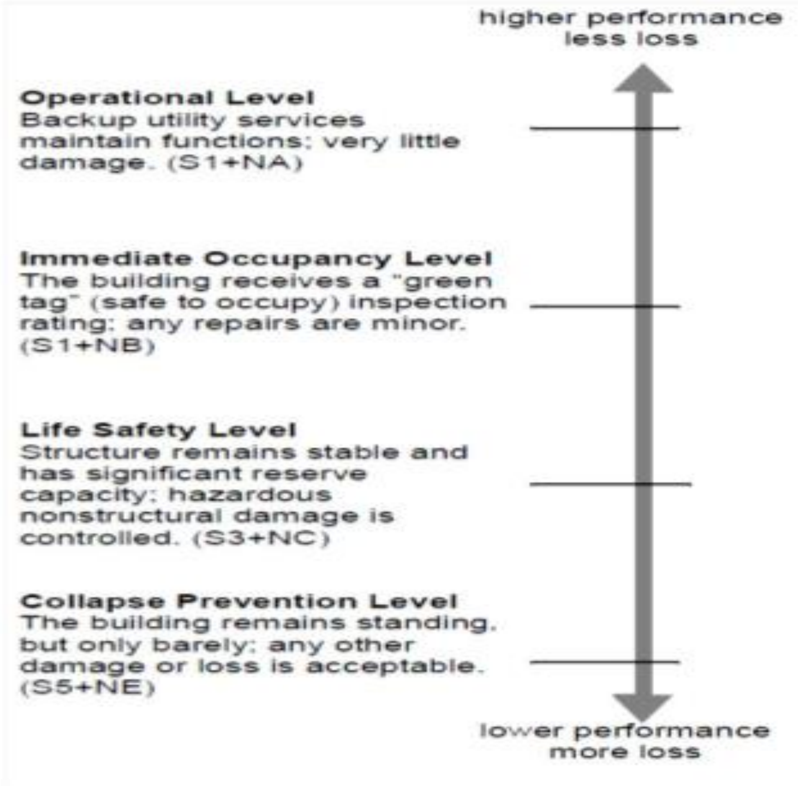


Fig 3.2 Building Performance Levels (ATC, 1997a)

Methods and design criteria to achieve several different levels and ranges of seismic performance are defined. The four Building Performance Levels are Collapse Prevention, Life Safety, Immediate Occupancy, and Operational. These levels are discrete points on a continuous scale describing the building's expected performance, or alternatively, how much damage, economic loss, and disruption may occur. Each Building Performance Level is made up of a Structural Performance Level that describes the limiting damage state of the structural systems and a Non-structural Performance Level that describes the limiting damage state of the non-structural systems. Three Structural Performance Levels and four Non-structural Performance Levels are used to form the four basic Building Performance Levels listed above.

Other structural and non-structural categories are included to describe a wide range of seismic rehabilitation intentions. The three Structural Performance Levels and two Structural Performance Ranges consist of:

•S-1: Immediate Occupancy Performance Level

•S-2: Damage Control Performance Range (extends between Life Safety and Immediate Occupancy Performance Levels)

•S-3: Life Safety Performance Level

• S-4: Limited Safety Performance Range (extends between Life Safety and Collapse Prevention Performance Levels)

• **S-5: Collapse Prevention Performance Level**

In addition, there is the designation of Structural Performance Not Considered, to cover the situation where only non-structural improvements are made. The four Non-structural Performance Levels are:

• N-A: Operational Performance Level

• N-B: Immediate Occupancy Performance Level

• N-C: Life Safety Performance Level

• N-D: Hazards Reduced Performance Level

In addition, there is the designation of N-E, Non-structural Performance Not Considered, to cover the situation where only structural improvements are made. A description of "what the building will look like after the earthquake" raises the questions: Which earthquake?

A small one or a large one? A minor-to-moderate degree of ground shaking severity at the site where the building is located, or severe ground motion? Ground shaking criteria must be selected, along with a desired Performance Level or Range, this can be done either by reference to standardized regional or national ground shaking hazard maps, or by site-specific studies.

Building performance is a combination of the performance of both structural and nonstructural components. Independent performance definitions are provided for structural and nonstructural components. Structural performance levels are identified by both a name and numerical designator. Nonstructural performance levels are identified by a name and alphabetical designator.

3.6 STRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

3.6.1 IMMEDIATE OCCUPANCY PERFORMANCE LEVEL (S-1) **:** Structural Performance Level S-1, Immediate Occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

***3.6.2 LIFE SAFETY PERFORMANCE LEVEL (S-3)* :** Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical.

***3.6.3 COLLAPSE PREVENTION PERFORMANCE LEVEL (S-5)* :** Structural Performance Level S-5, Collapse Prevention, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure and to more limited extent degradation in vertical-load-carrying capacity.

However, all significant components of the gravity load resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse.

3.7 STRUCTURAL PERFORMANCE RANGES (ATC, 1997a)

3.7.1 DAMAGE CONTROL PERFORMANCE RANGE (S-2)

Structural Performance Range S-2, Damage Control, means the continuous range of damage states that entail less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. Design for Damage Control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for Immediate Occupancy is excessive. Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.

3.7.2 LIMITED SAFETY PERFORMANCE RANGE (S-4)

Structural Performance Range S-4, Limited Safety, means the continuous range of damage states between the Life Safety and Collapse Prevention levels. Design parameters for this range may be obtained by interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels.

3.8 NONSTRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

3.8.1 OPERATIONAL PERFORMANCE LEVEL (N-A)

Non-structural Performance Level A, Operational, means the post-earthquake damage state of the building in which the non-structural components are able to support the building's intended function. At this level, most non-structural systems required for normal use of the building including lighting, plumbing, etc.; are functional, although minor repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer.

3.8.2 IMMEDIATE OCCUPANCY LEVEL (N-B)

Non-structural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited non-structural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable.

There could be minor window breakage and slight damage to some components.Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be nonoperable.

Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to nonstructural damage is very low.

3.8.3 LIFE SAFETY LEVEL (N-C)

Nonstructural Performance Level C, Life Safety, is the post-earthquake damage state in which potentially significant and costly damage has occurred to nonstructural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked. While injuries may occur during the earthquake from the failure of nonstructural components, it is expected that, overall, the risk of life-threatening injury is very low. Restoration of the nonstructural components may take extensive effort.

3.8.4 HAZARDS REDUCED LEVEL (N-D)

Nonstructural Performance Level D, Hazards Reduced, represents a post-earthquake damage state level in which extensive damage has occurred to nonstructural components, but large or heavy items that pose a falling hazard to a number of people such as parapets, cladding panels, heavy plaster ceilings, or storage racks are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons either inside or outside the structure should be avoided. Exits, fire suppression systems, and similar life-safety issues are not addressed in this performance level.

CHAPTER 4

MODELLING ON SAP2000

4.1 MATERIAL PROPERTIES

The material used for construction is Reinforced concrete with M-30 grade concrete and fe415 grade reinforcing steel. The Stress-Strain relationship used is as per I.S.456:2000. The basic material properties used are as follows:

Modulus of Elasticity of steel, E_s = 21,0000MPa

Modulus of Elasticity of concrete, E_c = 22,360.68 MPa

Characteristic strength of concrete, f_{ck} = 30 MPa

Yield stress for steel, f_y = 415 MPa

Ultimate strain in bending, ε_{cu} = 0.0035

4.1.2 MODEL GEOMETRY

Model 1

The structure analyzed is a five-storied, three bay along X-direction and two bays along Y-direction moment-resisting frame of reinforced concrete with properties as specified above. The concrete floors are modeled as rigid. The details of the model are given as:

Number of stories = 5

Number of bays along X-direction = 3

Number of bays along Y-direction = 2

Storey height = 4.0 meters

Bay width along X-direction = 6.0 meters

Bay width along Y-direction = 6.0 meters

4.2 STEP BY STEP PROCEDURE OF ANALYSIS IN SAP2000 (Model 1)

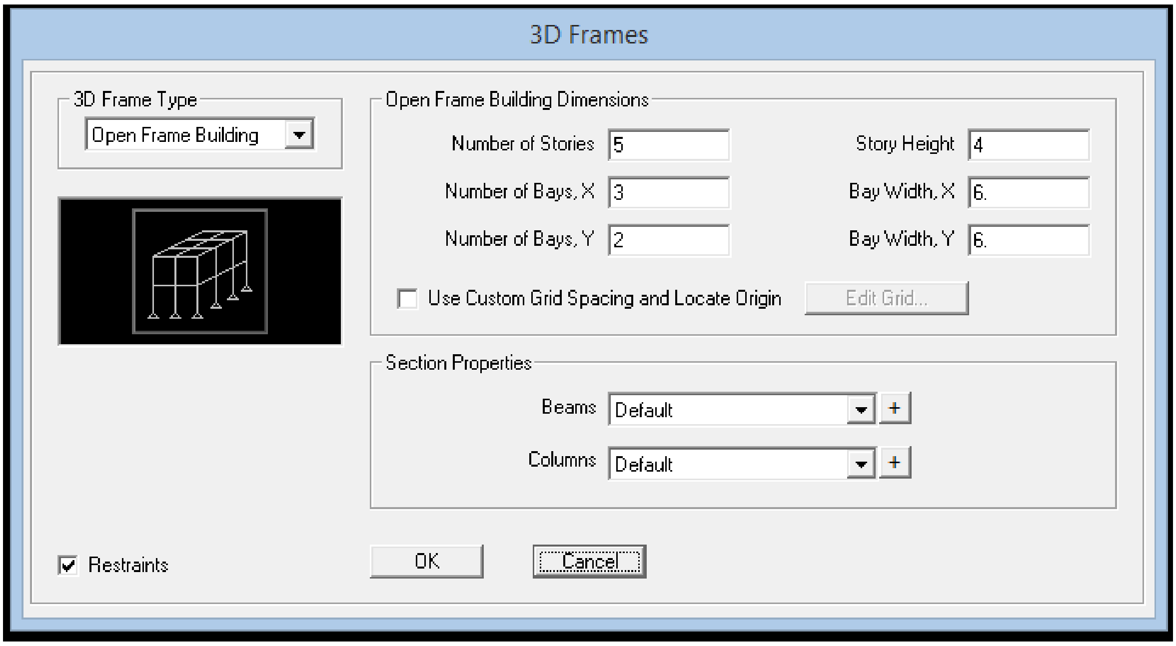


Fig 4.1 Model dimensions

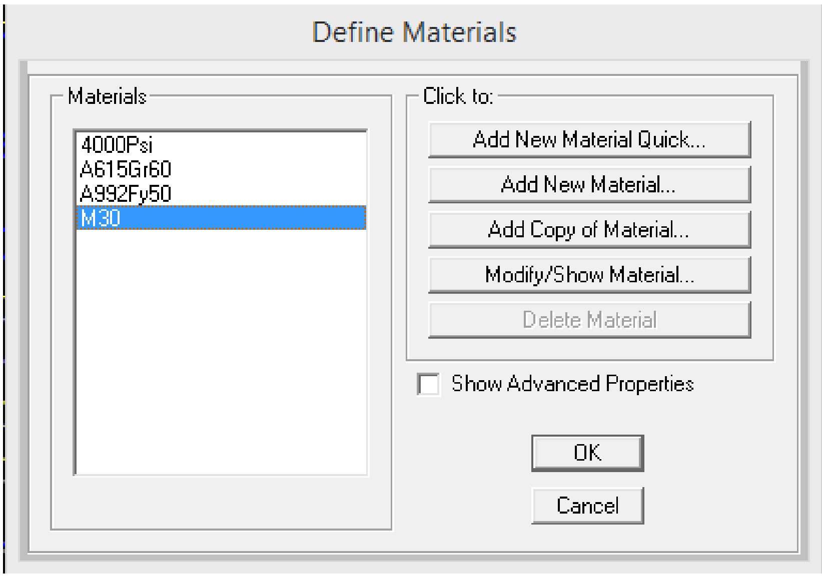
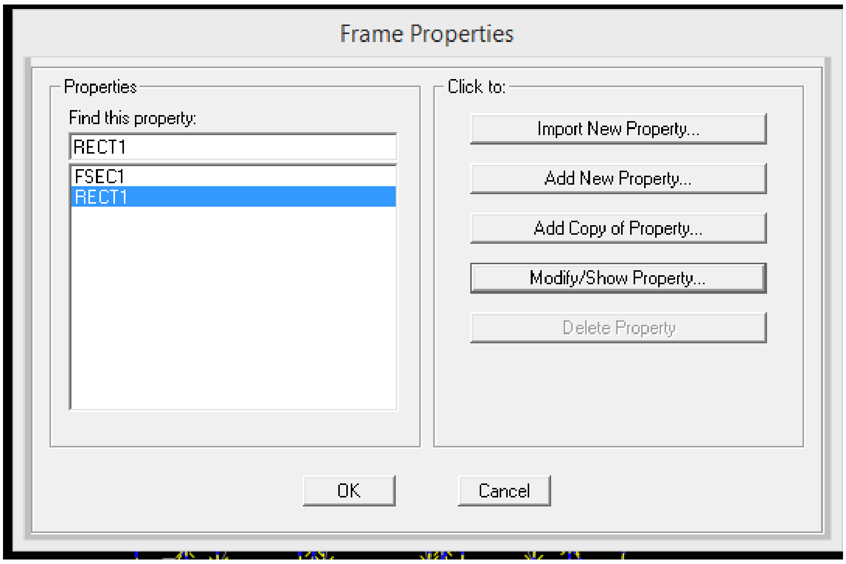


Fig 4.2 Material Definition



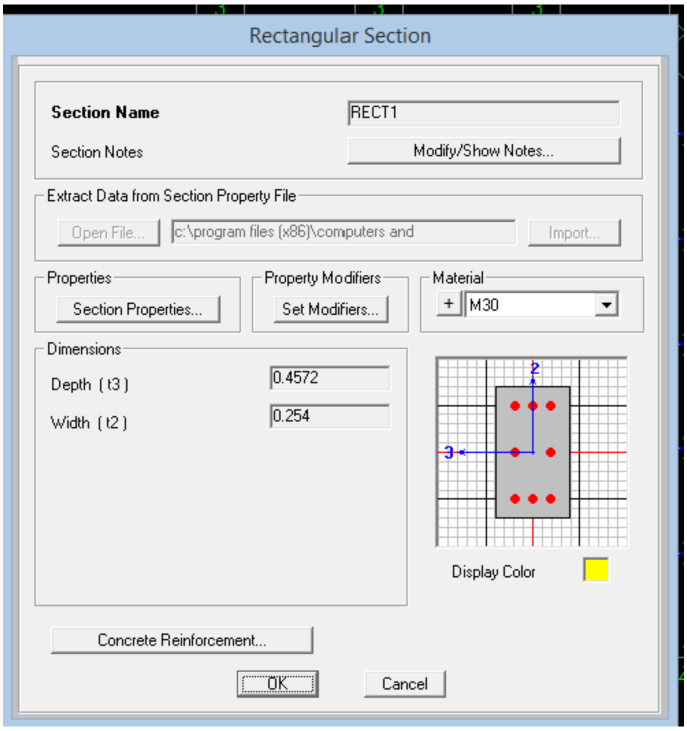


Fig 4.3 a) Frame and b) Section Properties

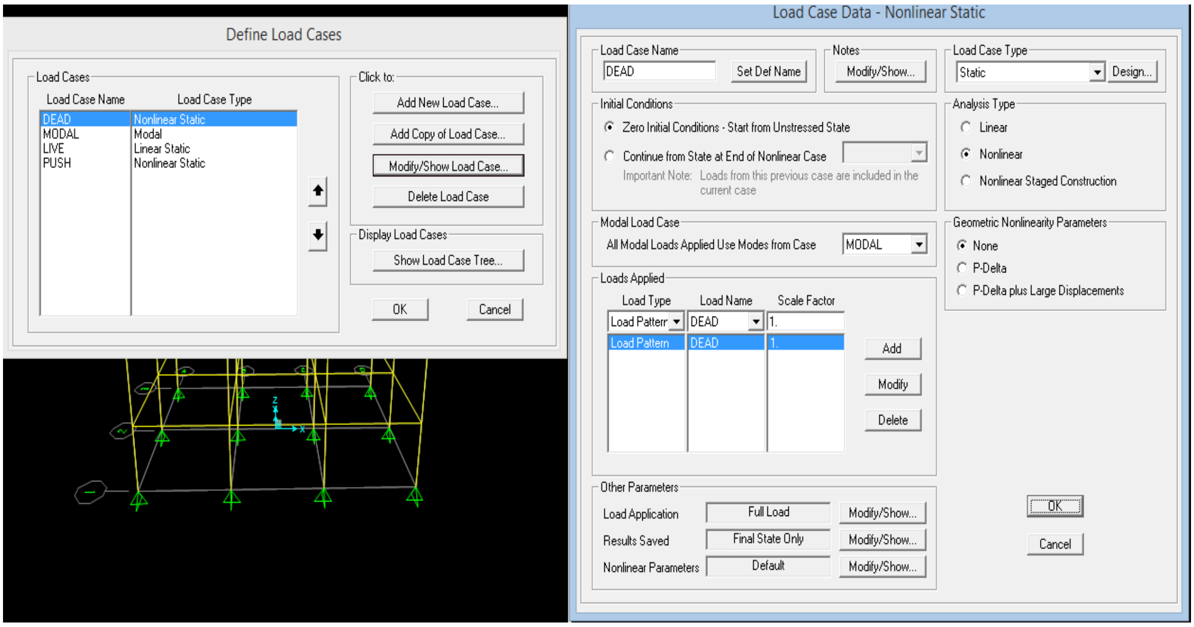


Fig 4.4 Load Definitions for dead load

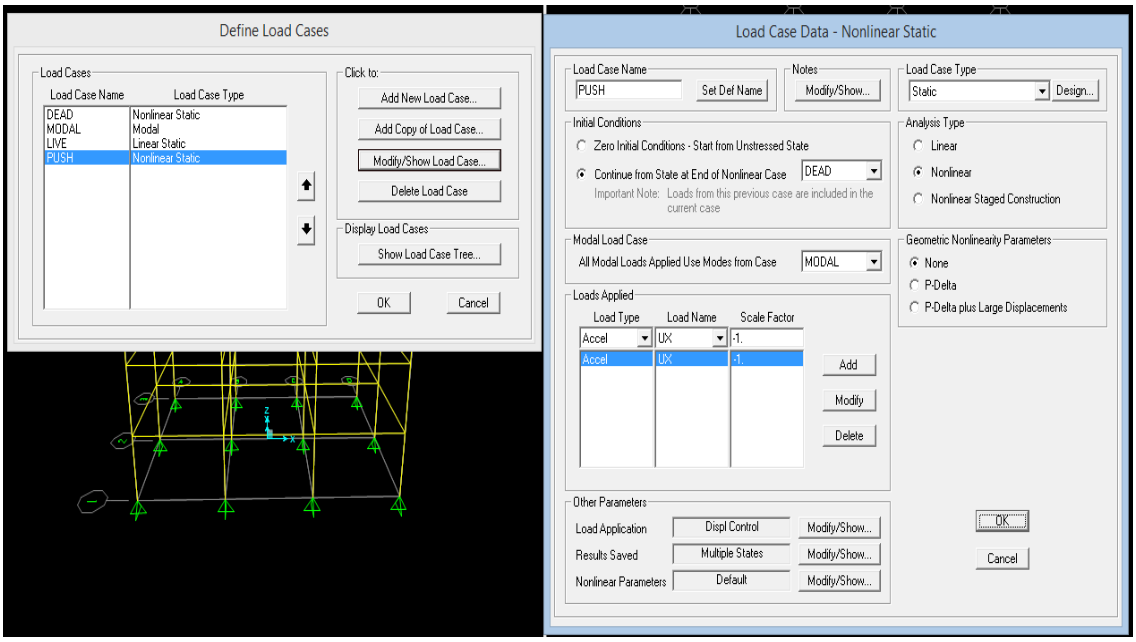


Fig 4.5 Load definition for Push-over

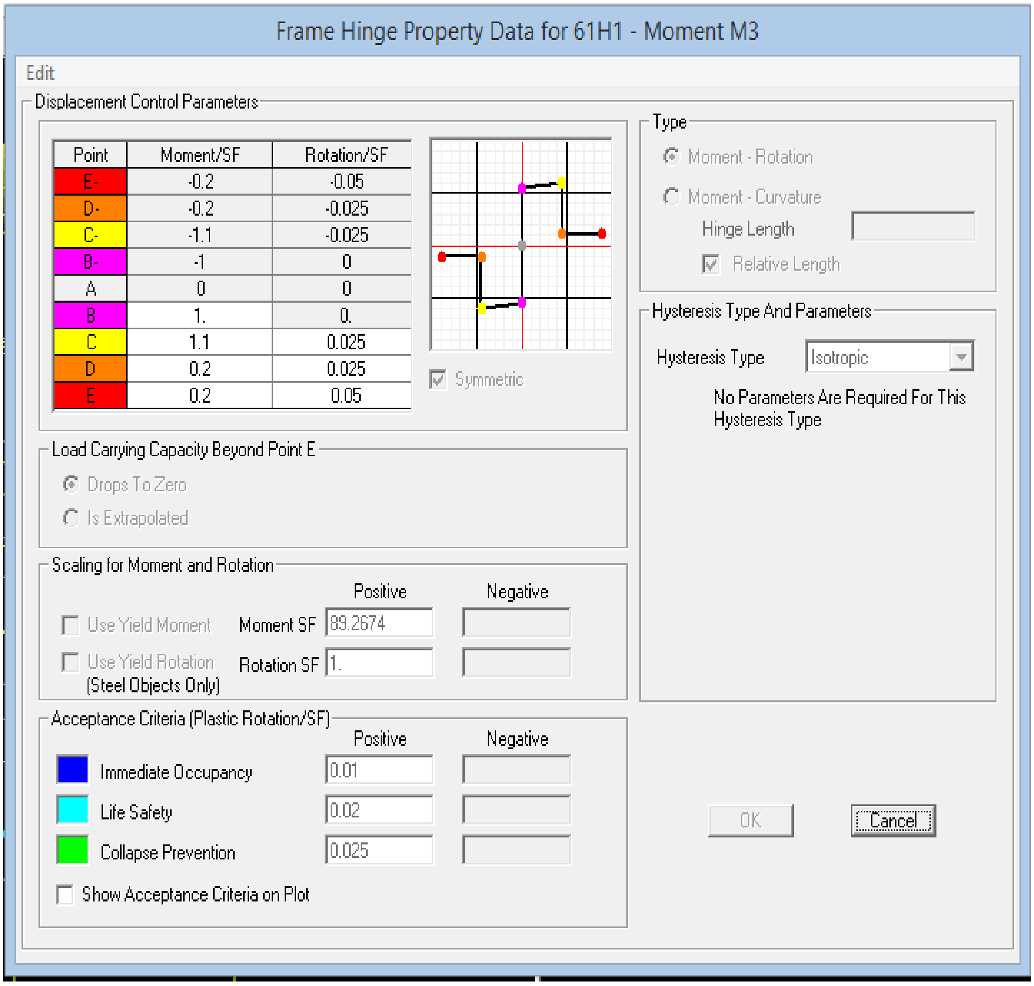


Fig4.6 Frame Hinge Property Data for 61H1

4.3 Model 2

The structure designed is a 4-story, one bay along X-direction and one bay along Y-direction. The material used for construction is Reinforced Concrete with M-20 grade concrete and Fe 415 grade reinforcing steel.

- Modulus of Elasticity of steel, Es = 21,000 MPa.
- Modulus of Elasticity of concrete ,Ec = 22,350000 MPa.
- Characteristic strength of concrete, fck = 20 MPa.
- Yield stress for steel, fy = 415 MPa.
- No. of stories = 4
- No. of bays along X-direction = 1
- No. of bays along Y-direction = 1
- Storey height = 4.0 metres
- Bay width along X-direction = 5.0 metres
- Bay width along Y-direction = 5.0 metres

4.3.1 *PLAN OF BUILDING*

The plan of the building is shown in the Fig. 4.6 .The bay width, column positions and beams positions can be read from the plan.

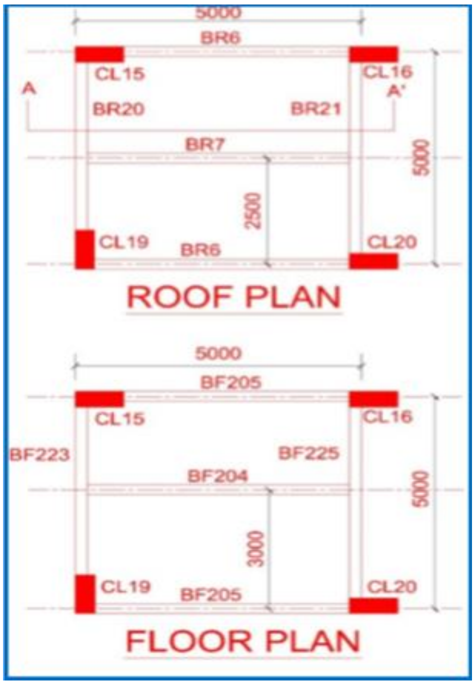


Fig 4.7 Roof and Floor Plan of the structure (Reddy. et al, 2010)

4.3.2 ELEVATION OF THE STRUCTURE

The Figure 4.7 shows the sectional elevation of the structure. The storey heights, column lines, description of slabs etc. can be seen in this picture.

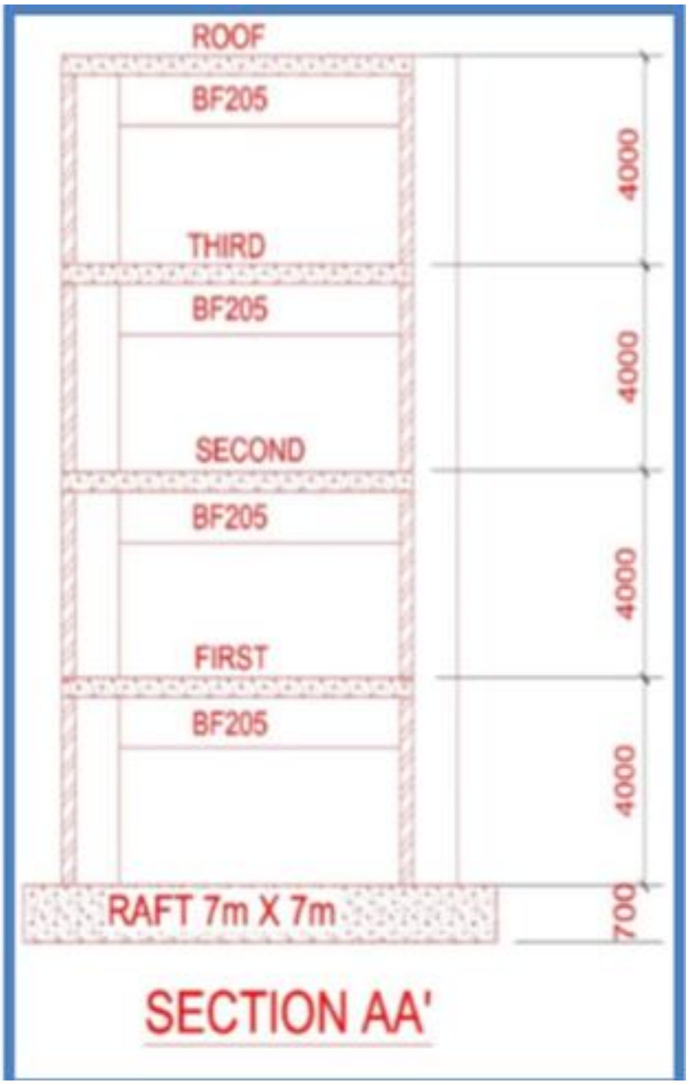


Fig 4.8 Section Elevation of the structure (Reddy. et.,al,2010)

4.3.3 SECTION DIMENSIONS

The structure is made of various sections whose dimensions are enlisted in table 4.1 below. In the identification column, ‘B‘ stands for beam, ‘F‘ stands for floor, and ‘R‘ stands for roof. The first numeral after ‘F’ in ‘BF’ stands for the floor number and the rest two are used to identify the beam at the floor. Therefore, all ‘BF’ designations stand for the floor beams while ‘BR‘ stands for roof beams. Similarly, ‘CL’ represents column while the first numeral after it stands for the floor to which that column is extending. The section dimensions are enlisted below:

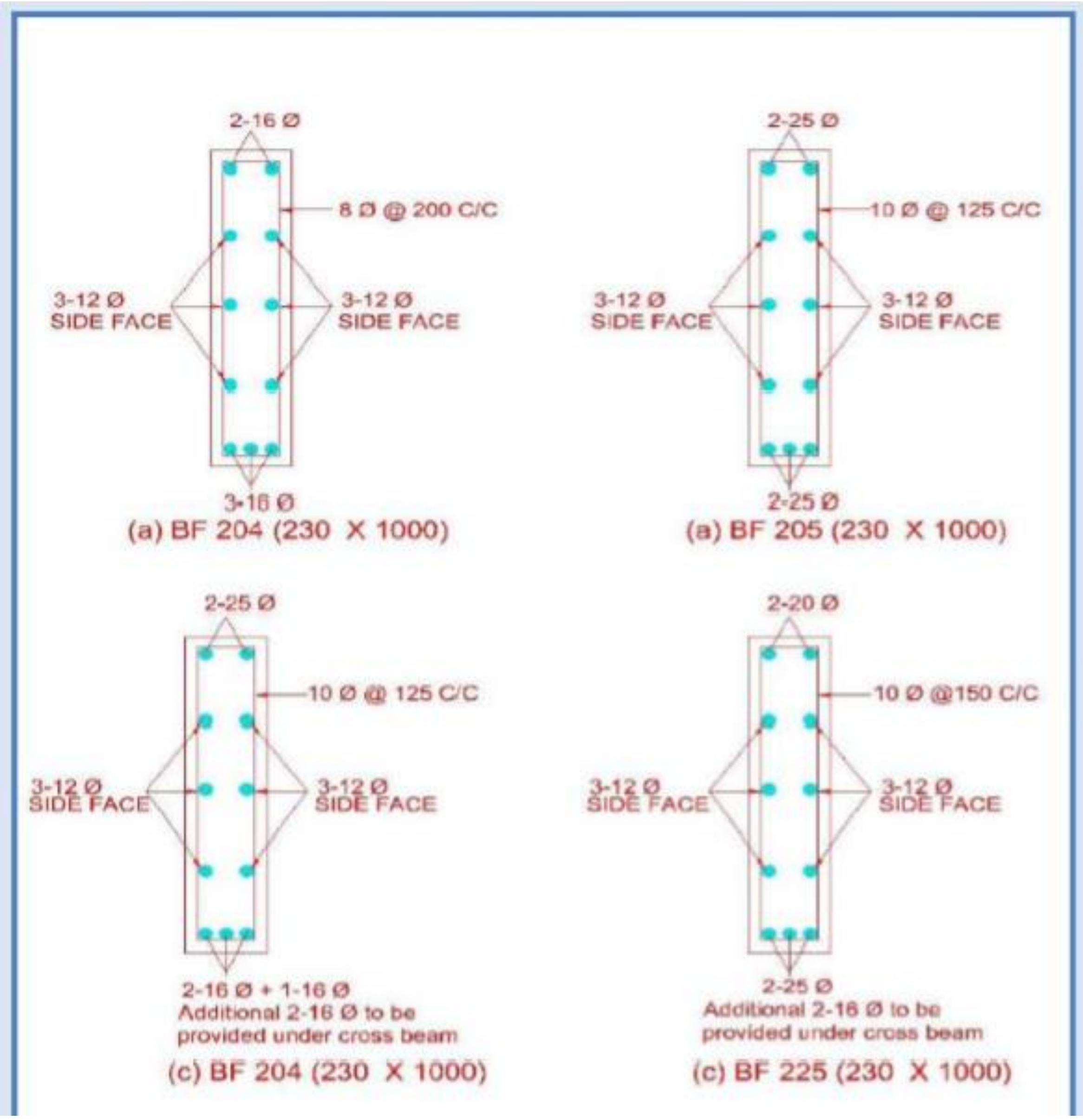


Fig 4.9 Detail of Floor Beams (Reddy. et., al, 2010)

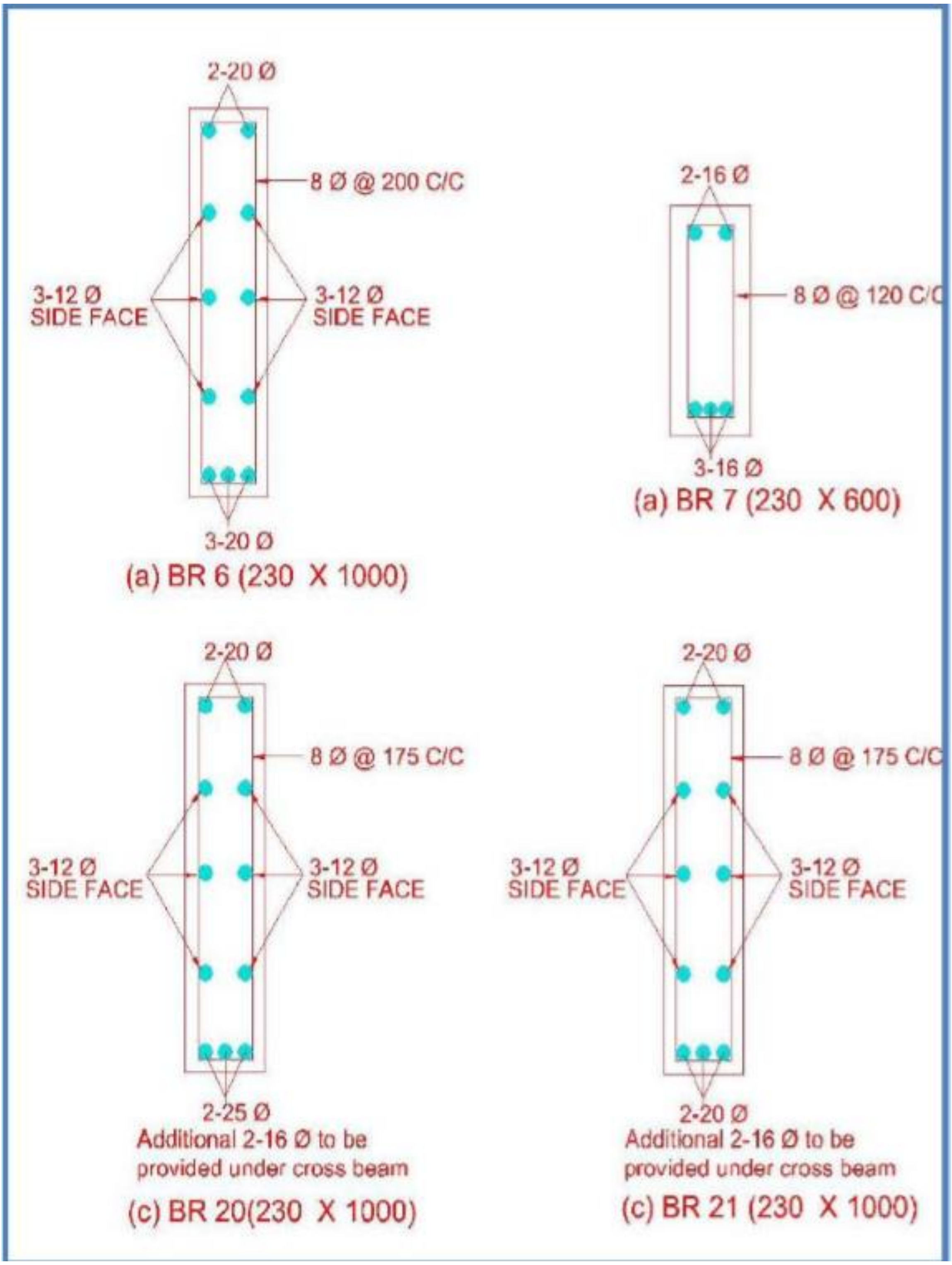


Fig 4.10 Detail of Roof Beams (Reddy. et., al, 2010)

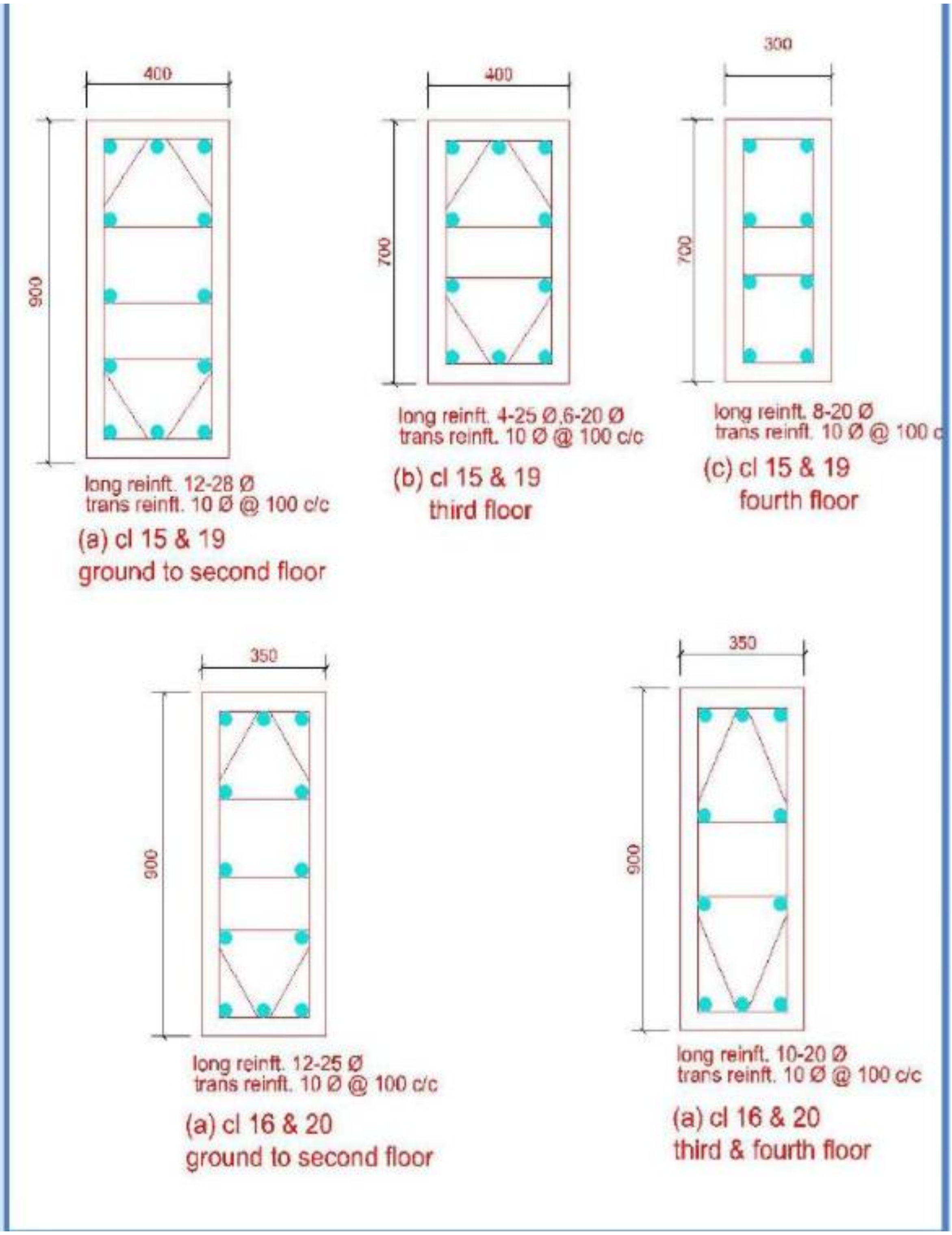


Fig 4.11 Detail of Columns (Reddy. et., al, 2010)

4.4 STEP BY STEP PROCEDURE OF ANALYSIS IN SAP 2000

Following picture shows us the basic dimension of the building namely the storey height which is 4m, the width of the bay along X and Y axis is equal to 5m. The structure is a 4 storey building having 2 bays along Y axis and 1 bay along X axis.

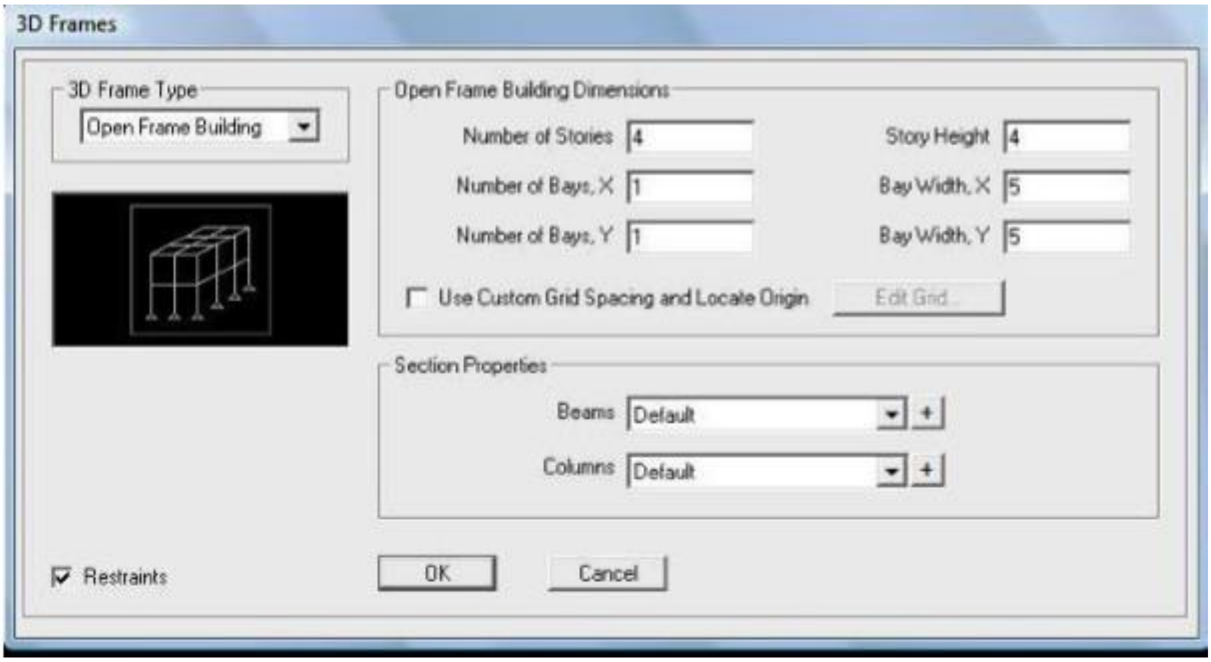
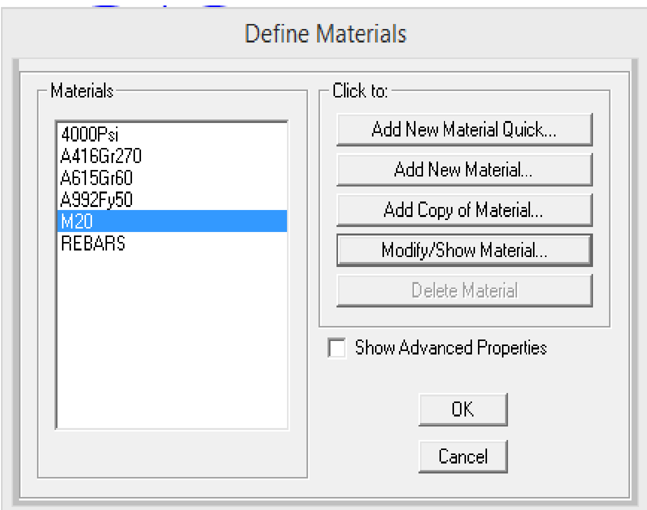


Fig 4.12 Basic Dimension of the Structure



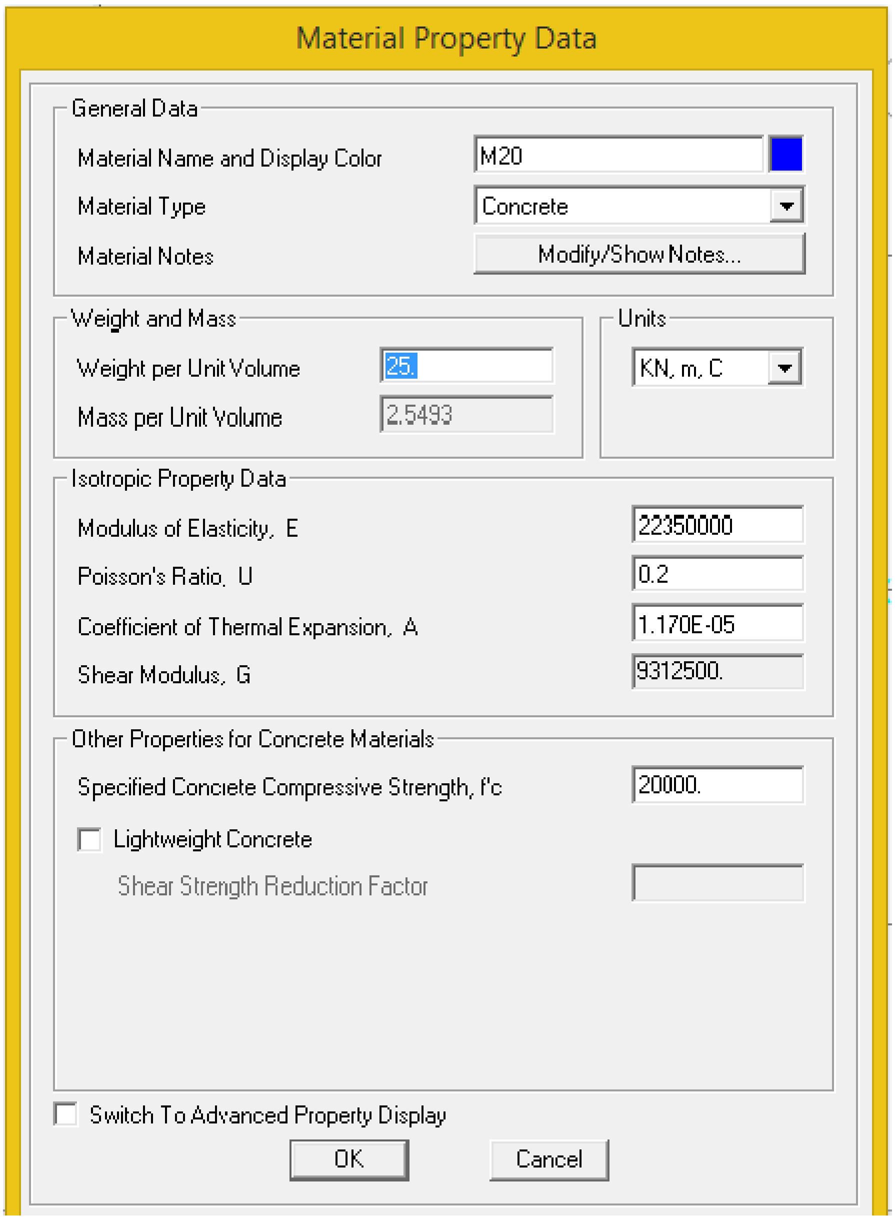
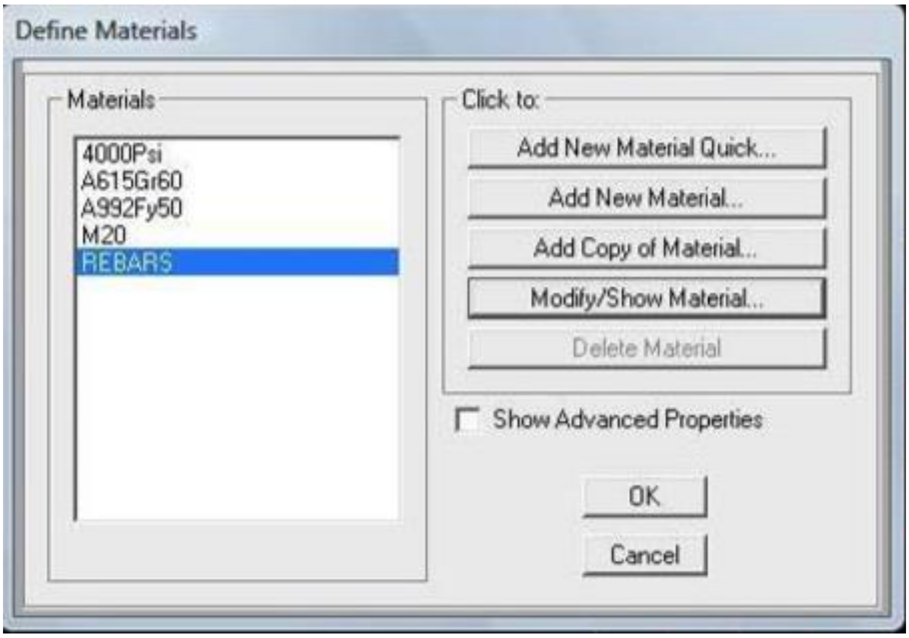


Fig 4.13 Material Properties definition (M20 conrete)



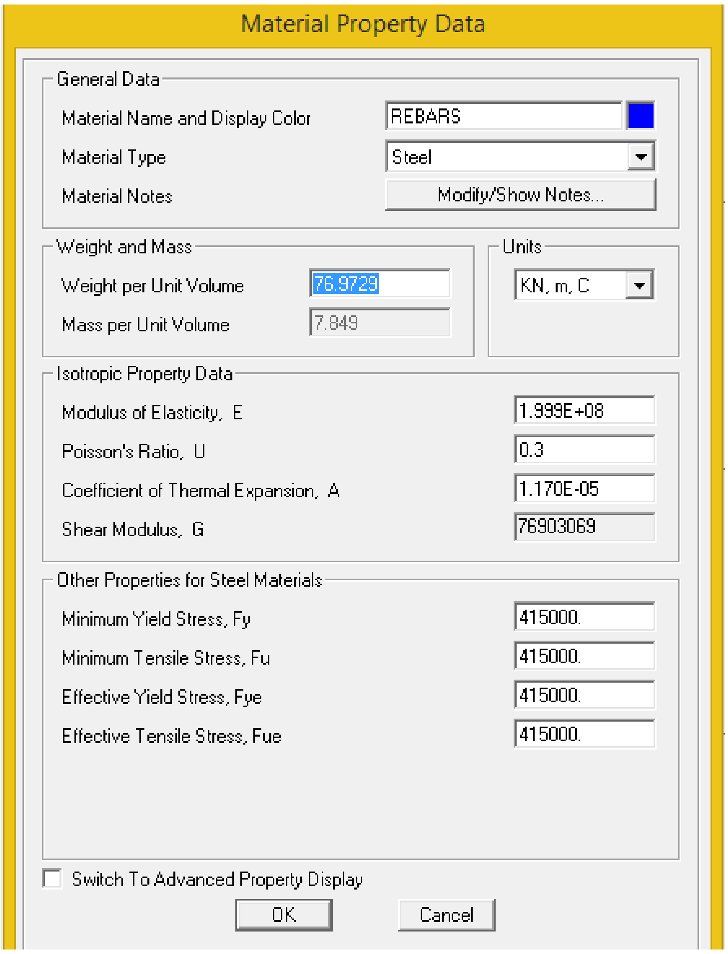


Fig 4.14 Material Properties for Rebars

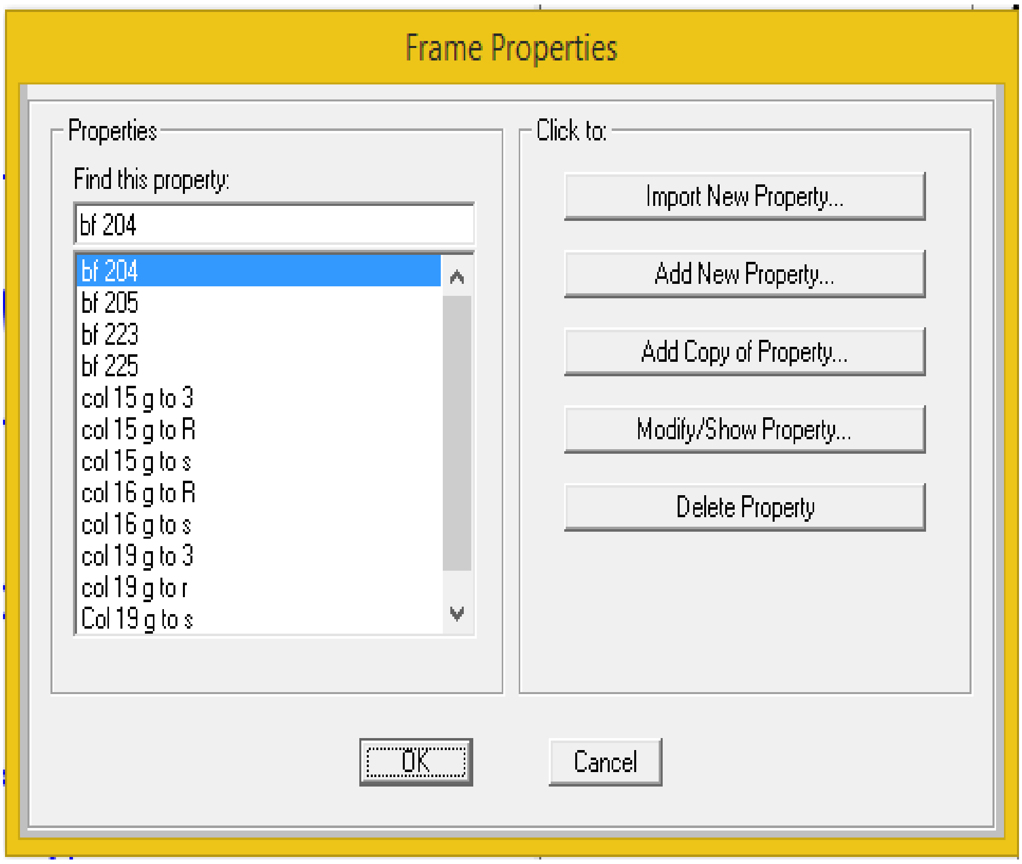
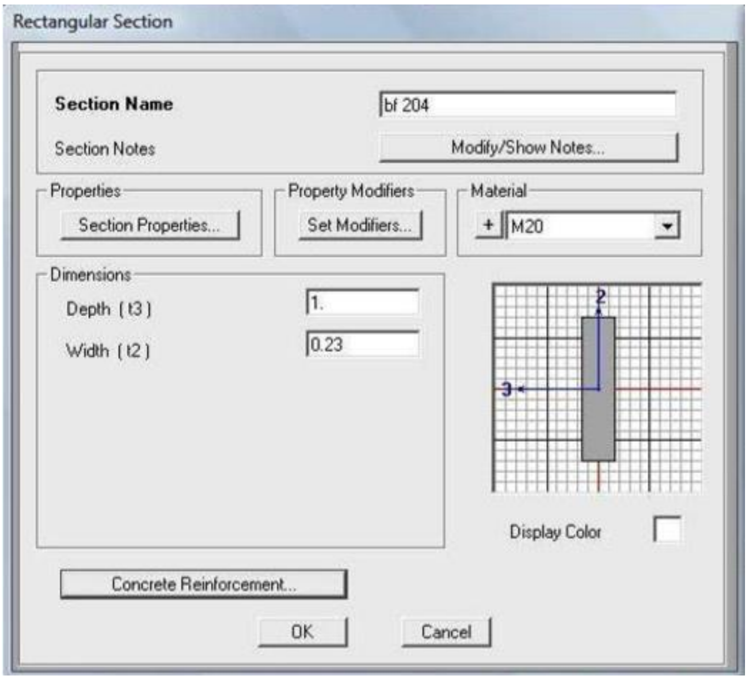


Fig 4.15 Defining beams and columns



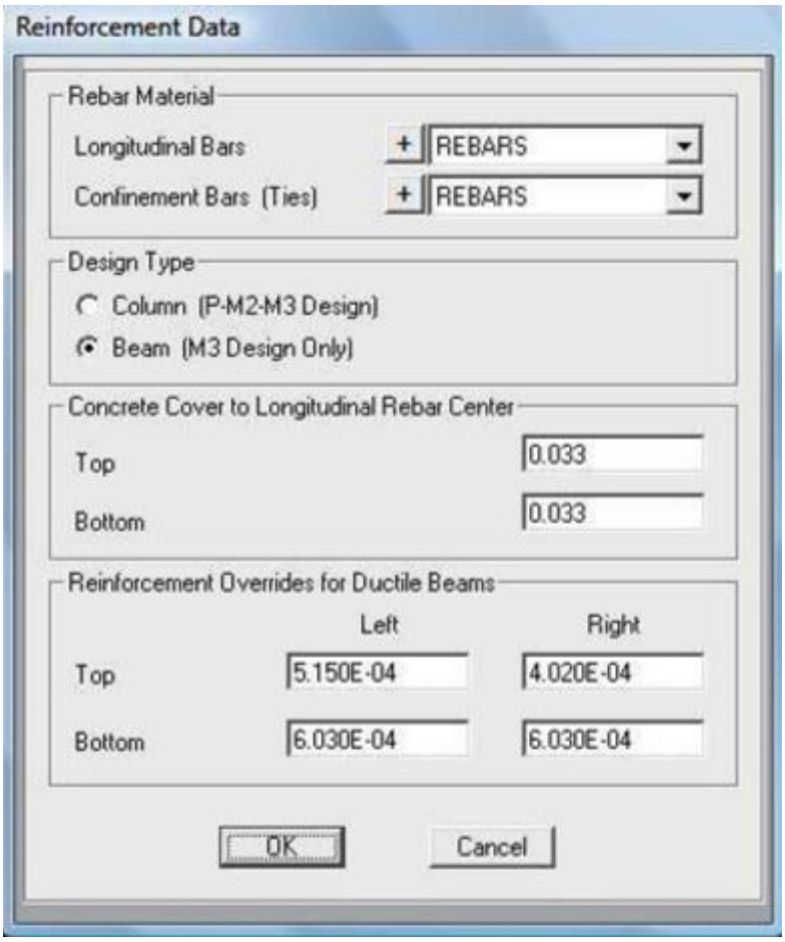
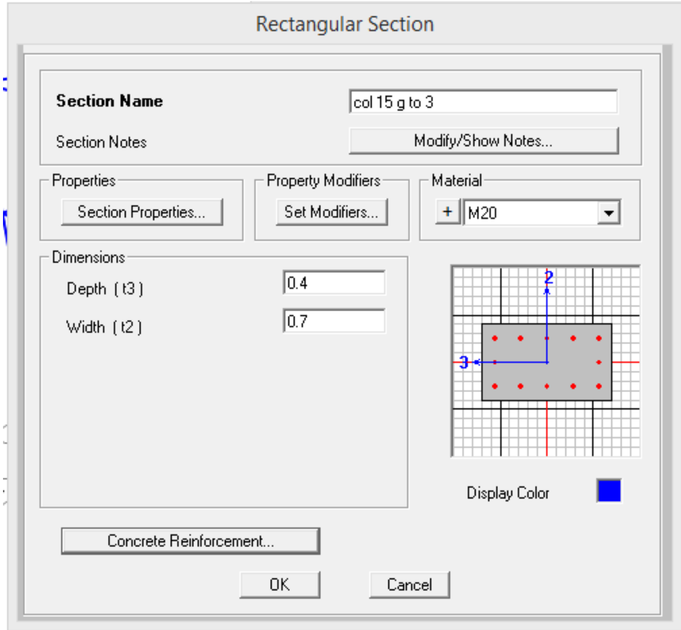


Fig 4.16 Basic dimension of a beam



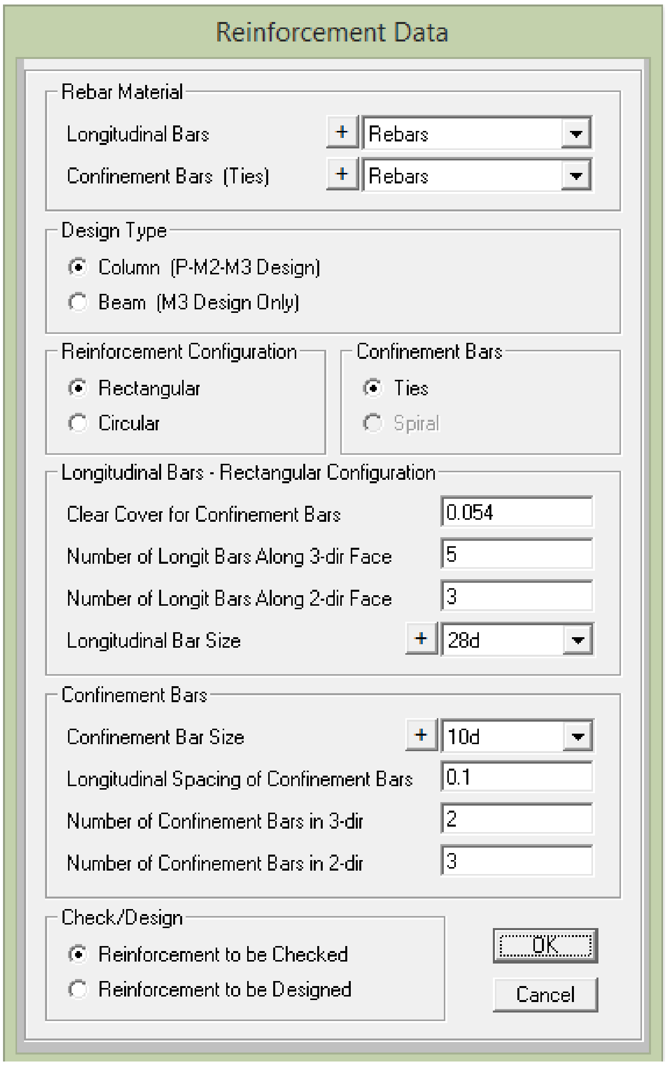
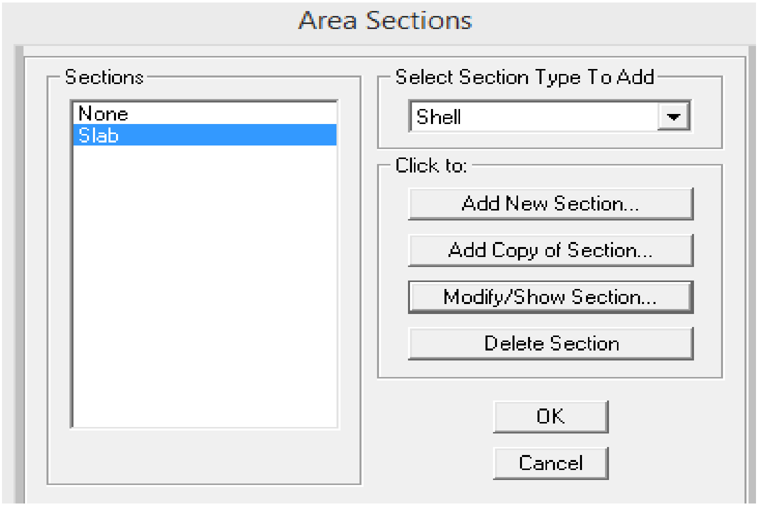


Fig 4.17 Basic dimension of column



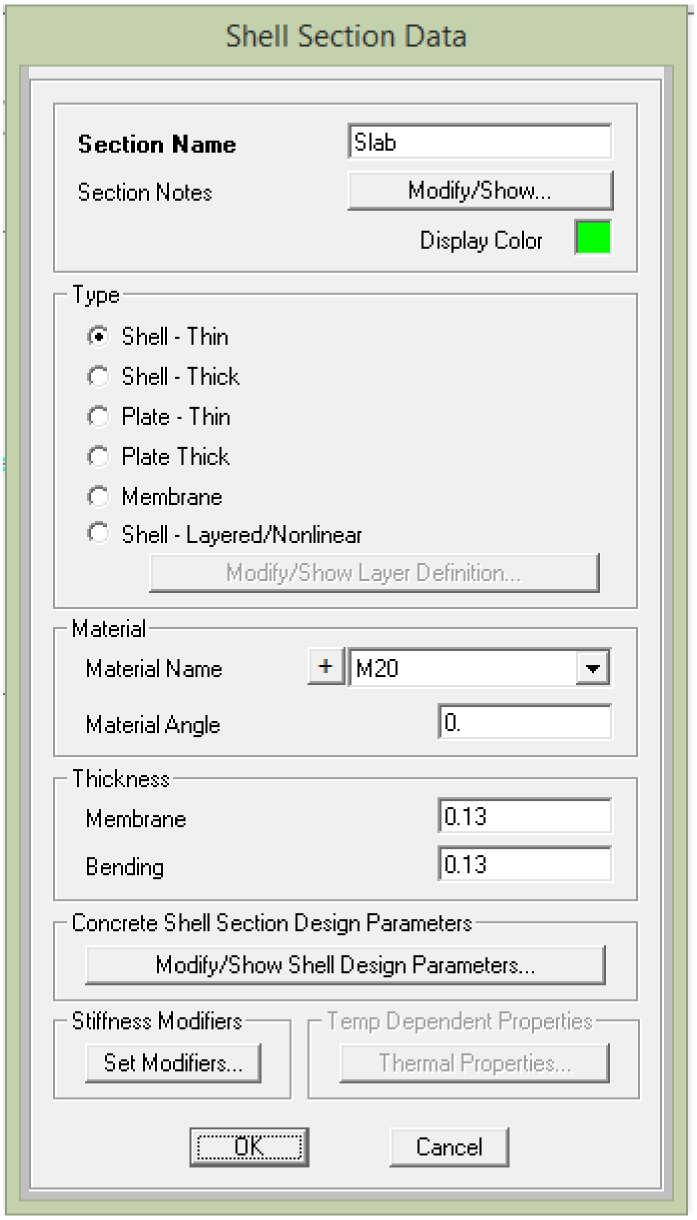


Fig 4.18 Basic Dimension of slab

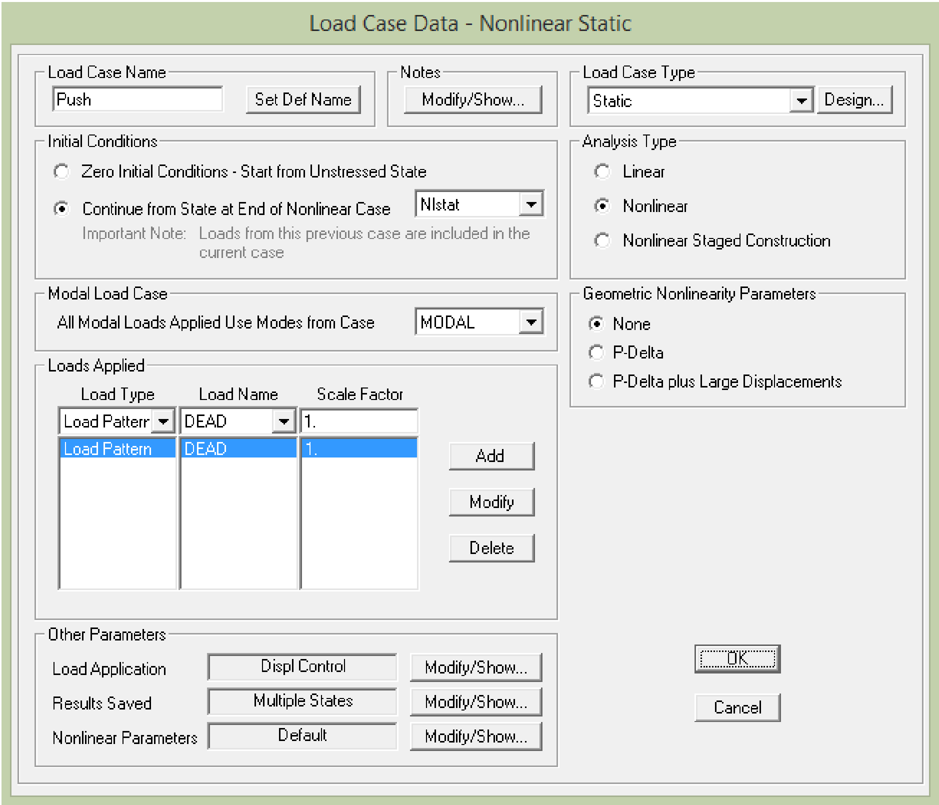


Fig 4.19 Defining Pushover load case

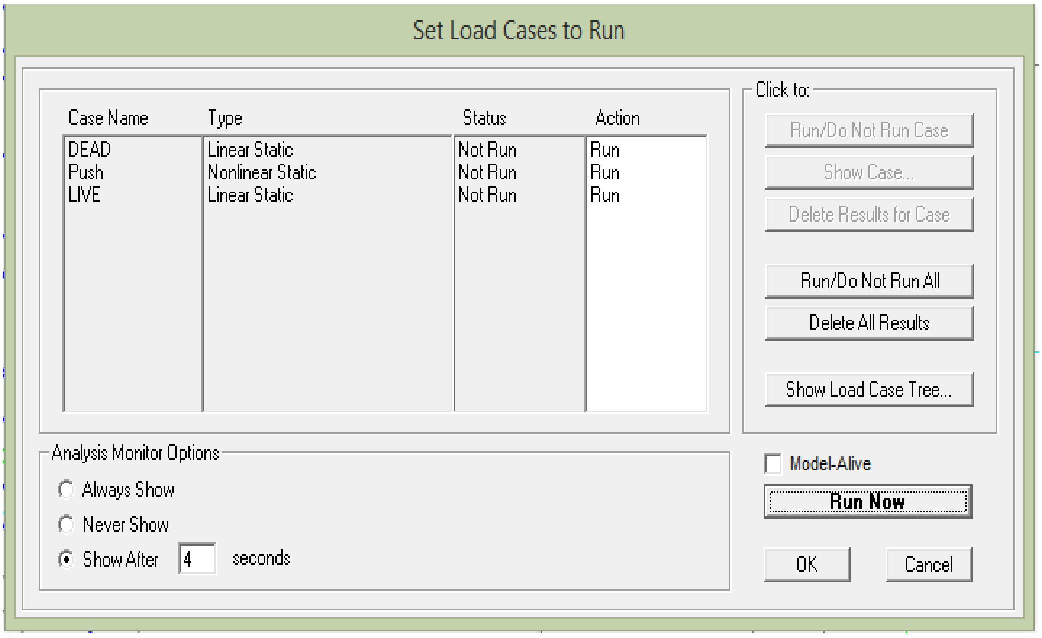


Fig 4.20 Run Analysis – Final Step

CHAPTER 5

RESULTS AND DISCUSSIONS

This chapter presents the results of Analysis of RCC frames considered in the present work.. Here formation of plastic hinges at the joints and graphs for capacity spectrum as well as base shear vs. roof displacement are recorded and studied.

5.1 ANALYSIS RESULTS OF RCC FRAME

In the present study, non-linear response of RCC frame modelled as per details discussed in Chapter 4 (4.1 General Description of Structure) using modelling under the loading has been carried out. The objective of this study is to see the variation of load- displacement graph and check the maximum base shear and displacement of the frame.

After the analysis is done (as explained in the previous chapter), the pushover curve is obtained which is shown in Fig. 5.1. A table is also obtained which gives the coordinates of each step of the pushover curve and summarizes the number of hinges in each state (for example, between IO, LS, CP or between D and E). These data are shown in Table. 5.1. The results for both the cases are presented below.

CASE I

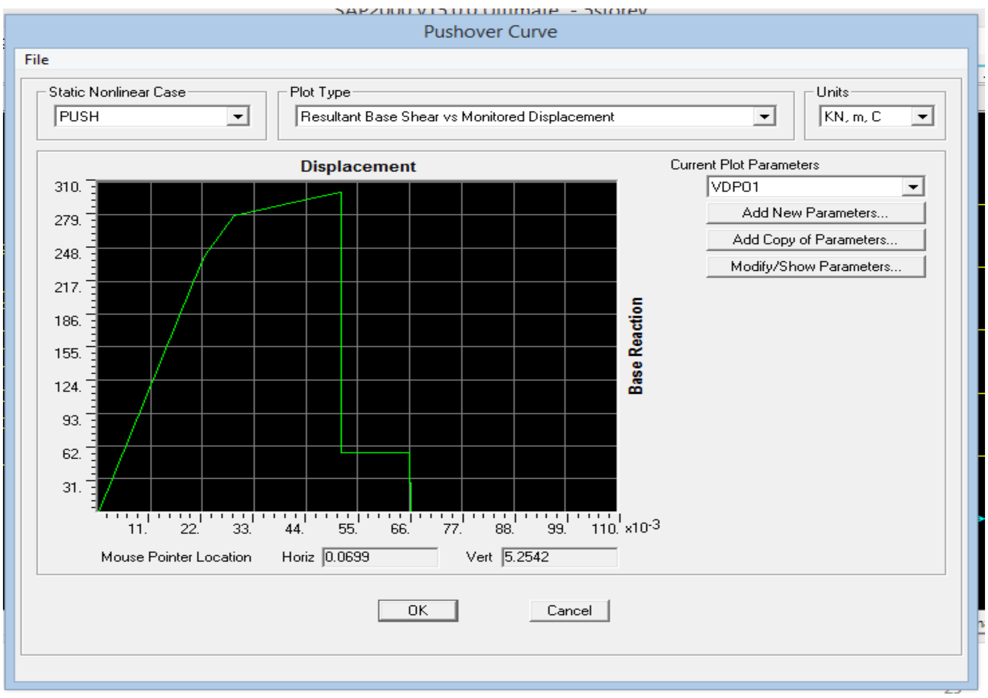


Fig 5.1 Pushover curve of the building

Step	Displacemer	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
	m	KN									
0	8.970E-08	0.000	290	0	0	0	0	0	0	0	290
1	0.010000	107.297	290	0	0	0	0	0	0	0	290
2	0.020000	214.594	290	0	0	0	0	0	0	0	290
3	0.022416	240.521	288	2	0	0	0	0	0	0	290
4	0.028648	278.416	275	15	0	0	0	0	0	0	290
5	0.043577	293.212	269	21	0	0	0	0	0	0	290
6	0.051391	300.859	269	15	0	4	0	2	0	0	290
7	0.051392	55.506	269	9	0	0	0	0	12	0	290
8	0.061392	55.517	269	9	0	0	0	0	12	0	290
9	0.065912	55.522	269	9	0	0	0	0	6	6	290
10	0.065913	27.696	269	9	0	0	0	0	6	6	290
11	0.066302	27.697	269	9	0	0	0	0	0	12	290
12	0.066303	0.024	269	9	0	0	0	0	0	12	290
13	0.076303	0.035	269	9	0	0	0	0	0	12	290
14	0.086303	0.046	269	9	0	0	0	0	0	12	290
15	0.096303	0.057	269	9	0	0	0	0	0	12	290
16	0.100000	0.061	269	9	0	0	0	0	0	12	290

Table 5.1 Tabular Data for Pushover Curve

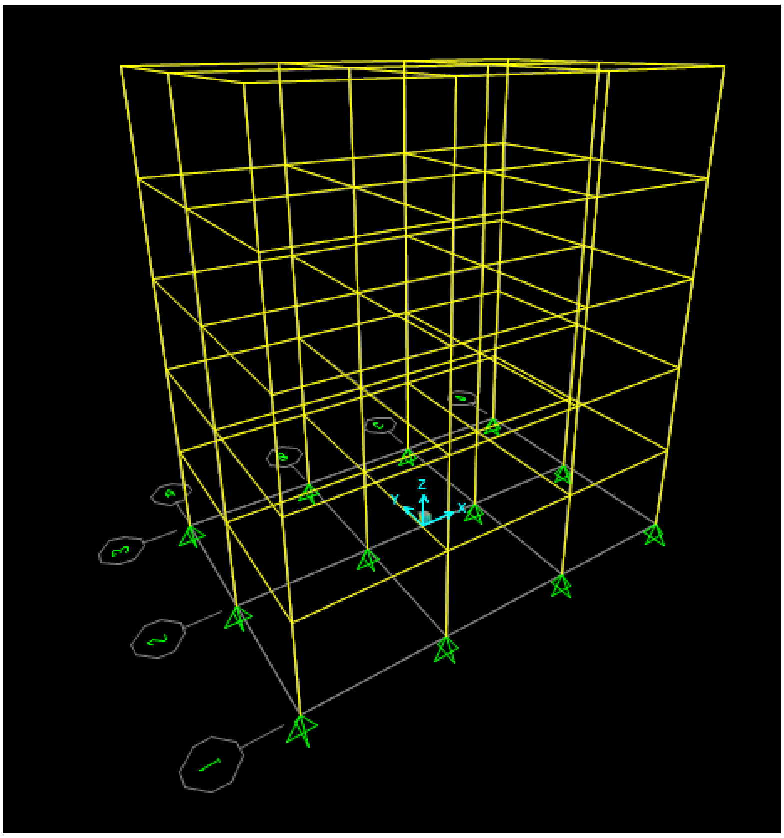


Fig 5.3 Initial Stage of the Structure

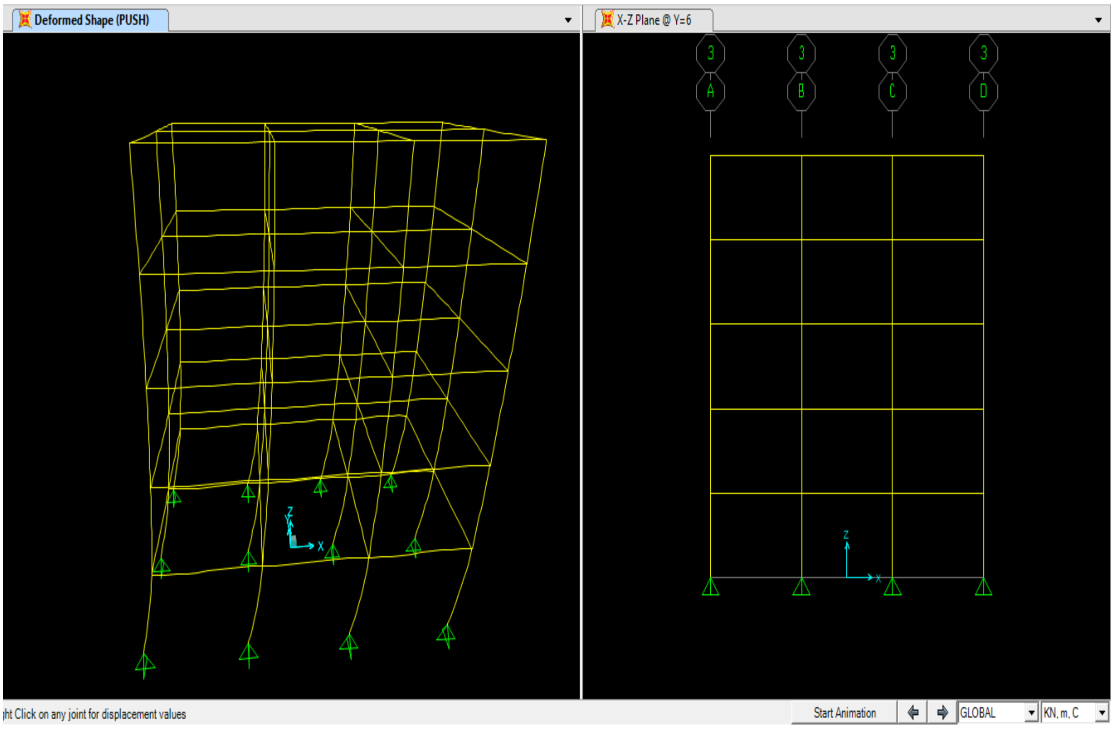


Fig 5.4 Final Shape after deformation

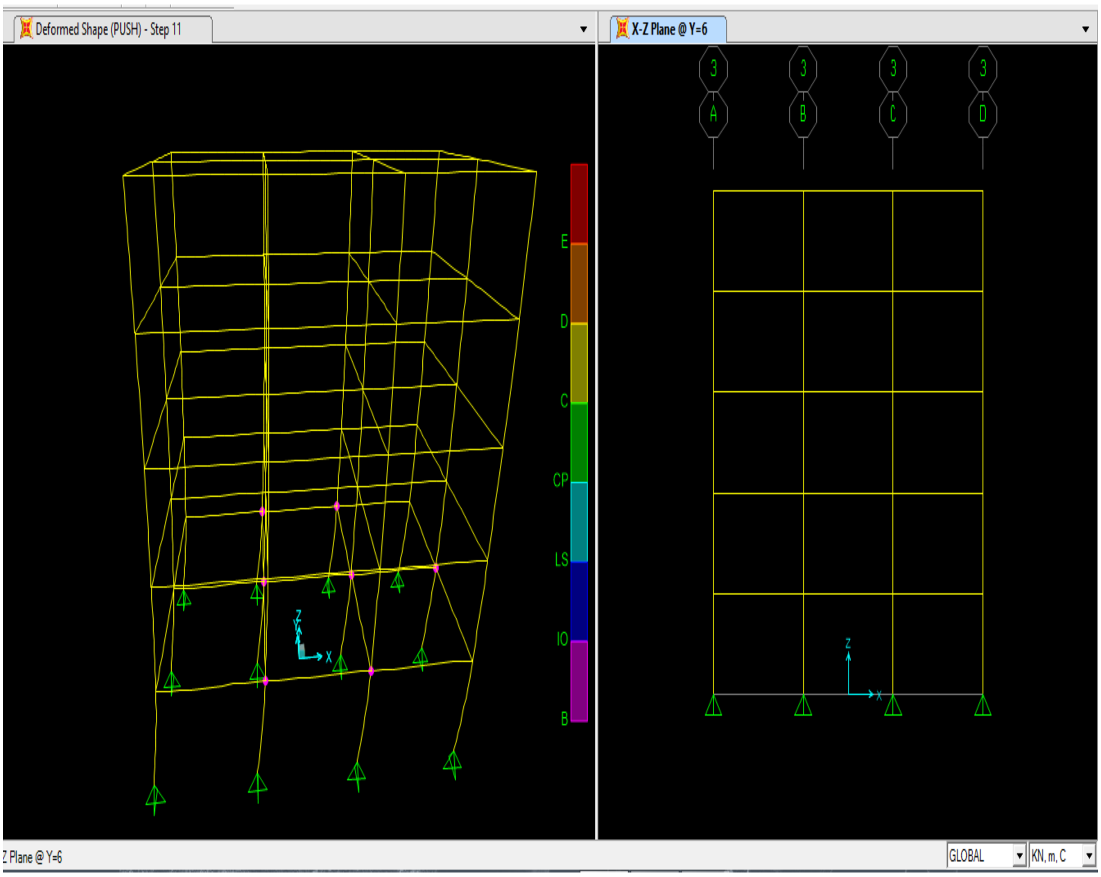


Fig 5.5 Formation of plastic hinges

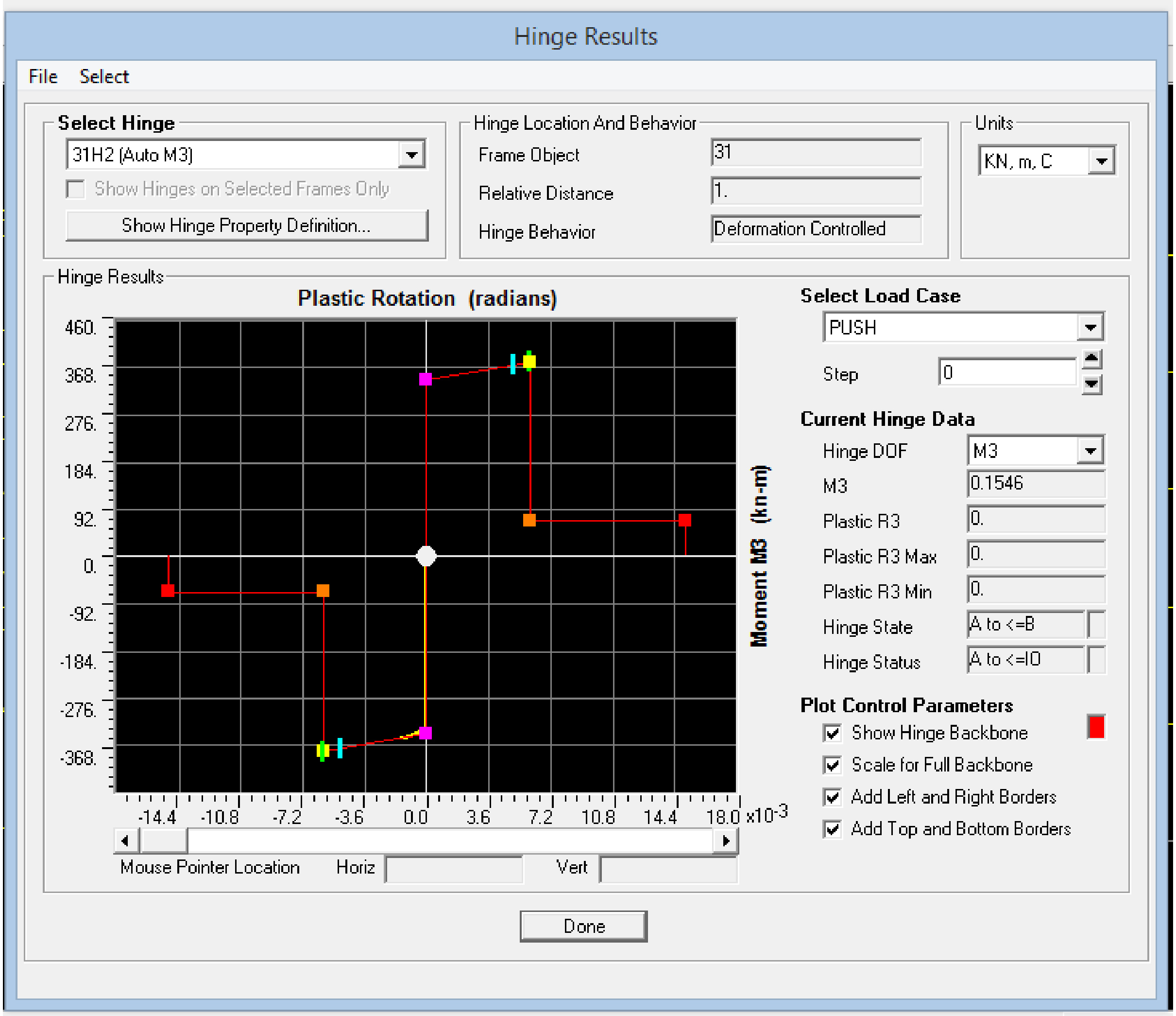


Fig 5.6 Moment vs. Rotation at hinge 31H2 during the 1st step

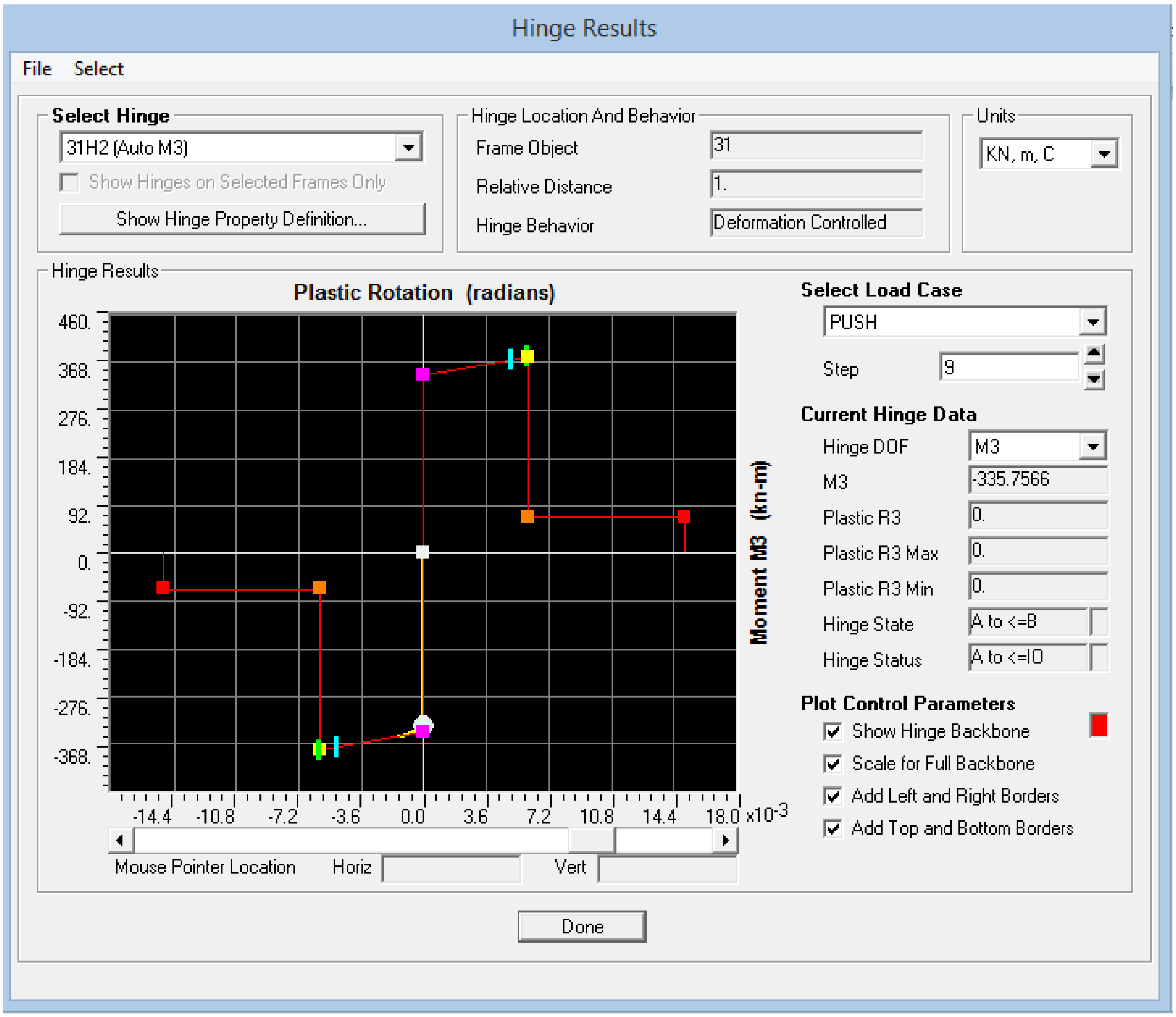


Fig 5.7 Moment vs. Rotation at hinge 31H2 at step 9.

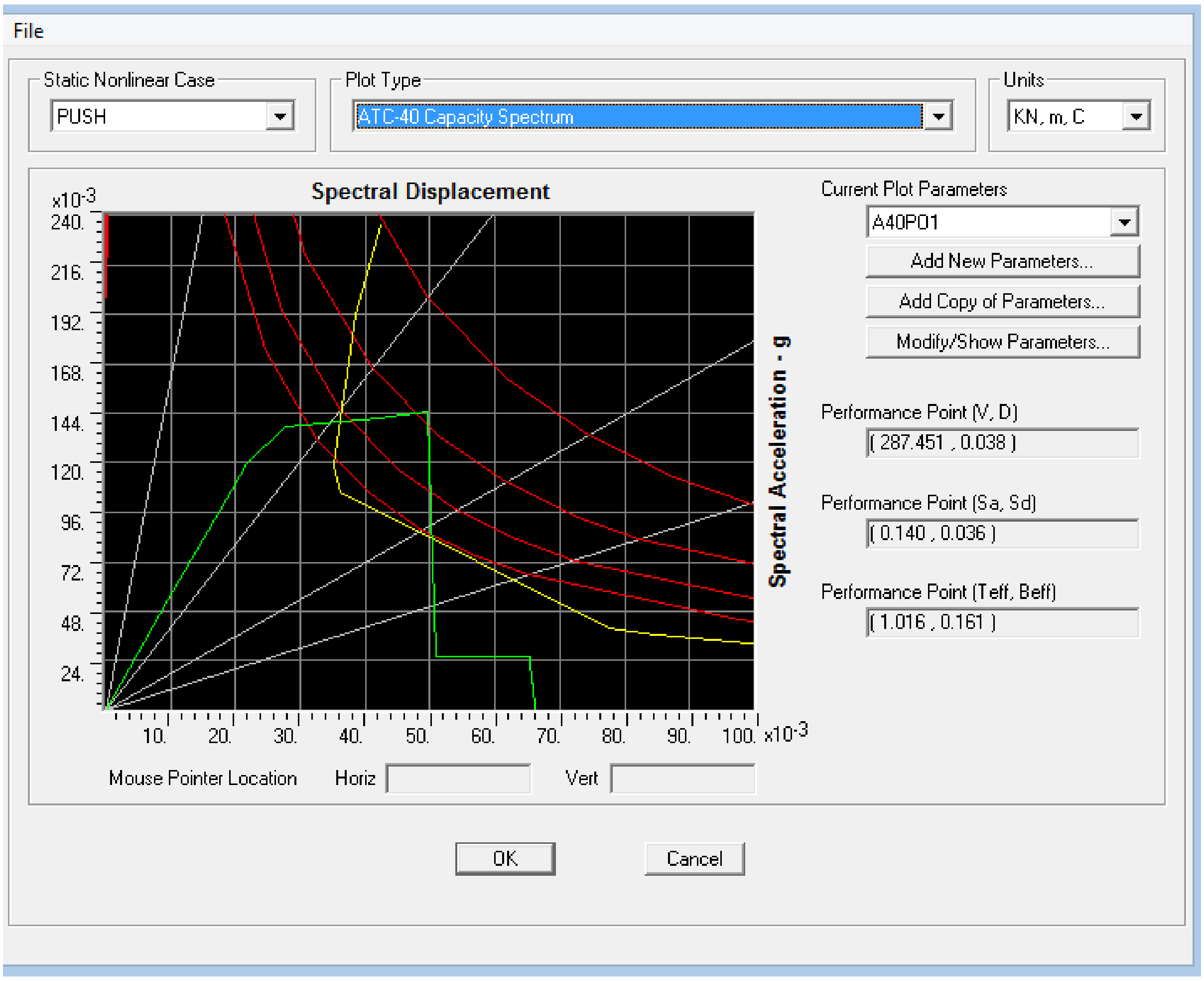


Fig 5.8 Capacity Spectrum Curve

Case II

For the second problem we obtained similar results. Fig 5.9 gives us a plot of Base shear vs. Displacement. A table also obtain which gives the coordinates of each step of the pushover curve and summarizes the number of hinges in each state (for example, between IO, LS, CP or between D and E). This data is shown in Table. 5.2 .

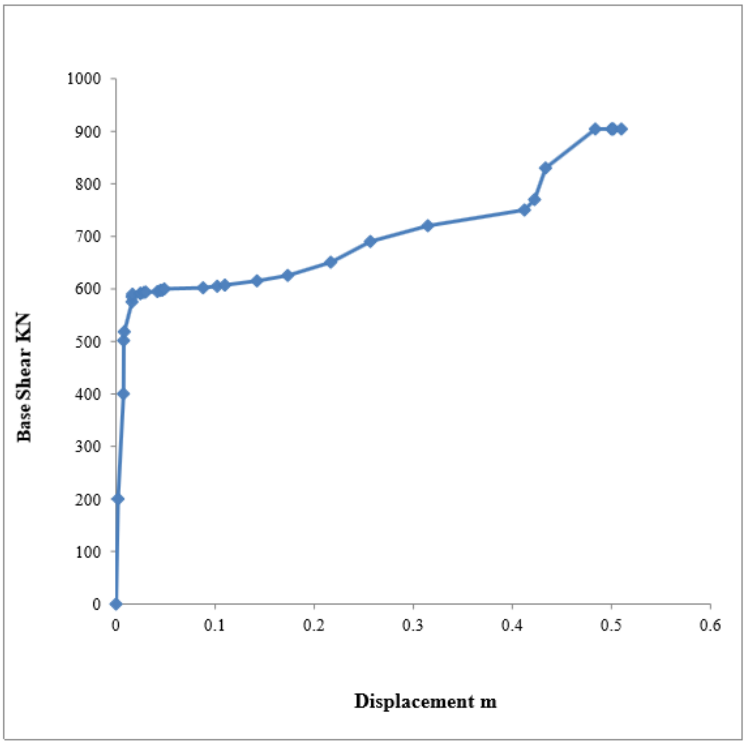
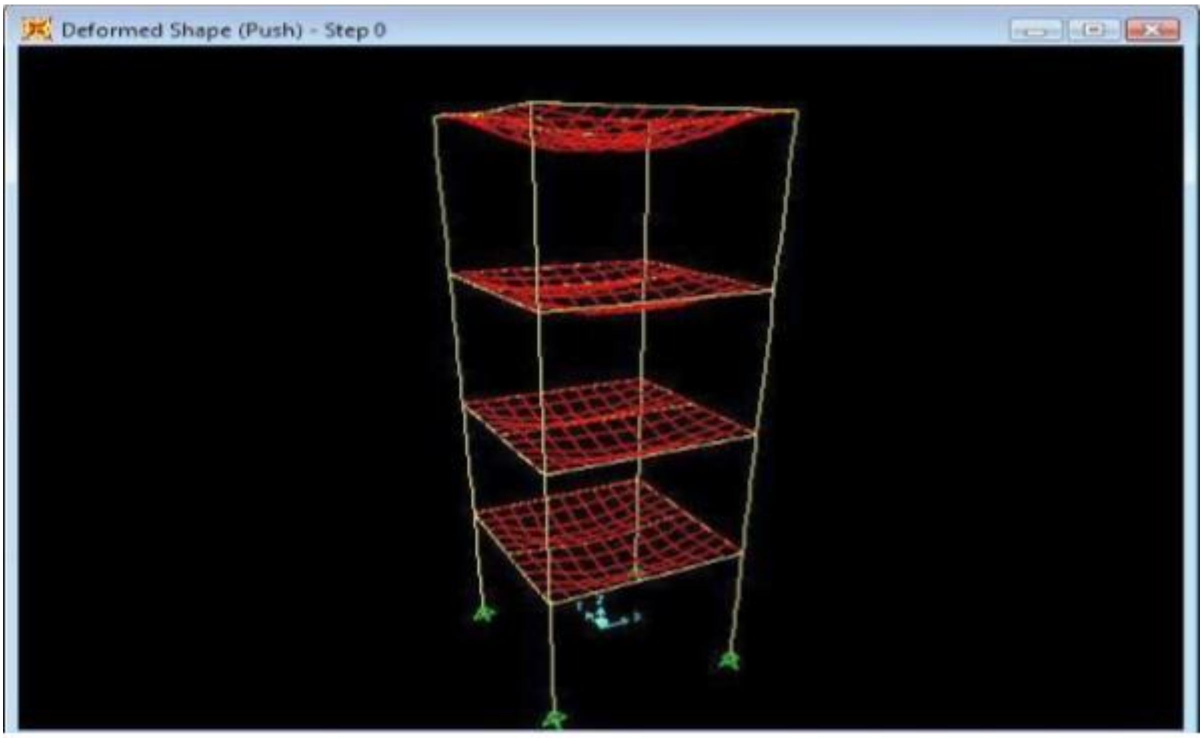


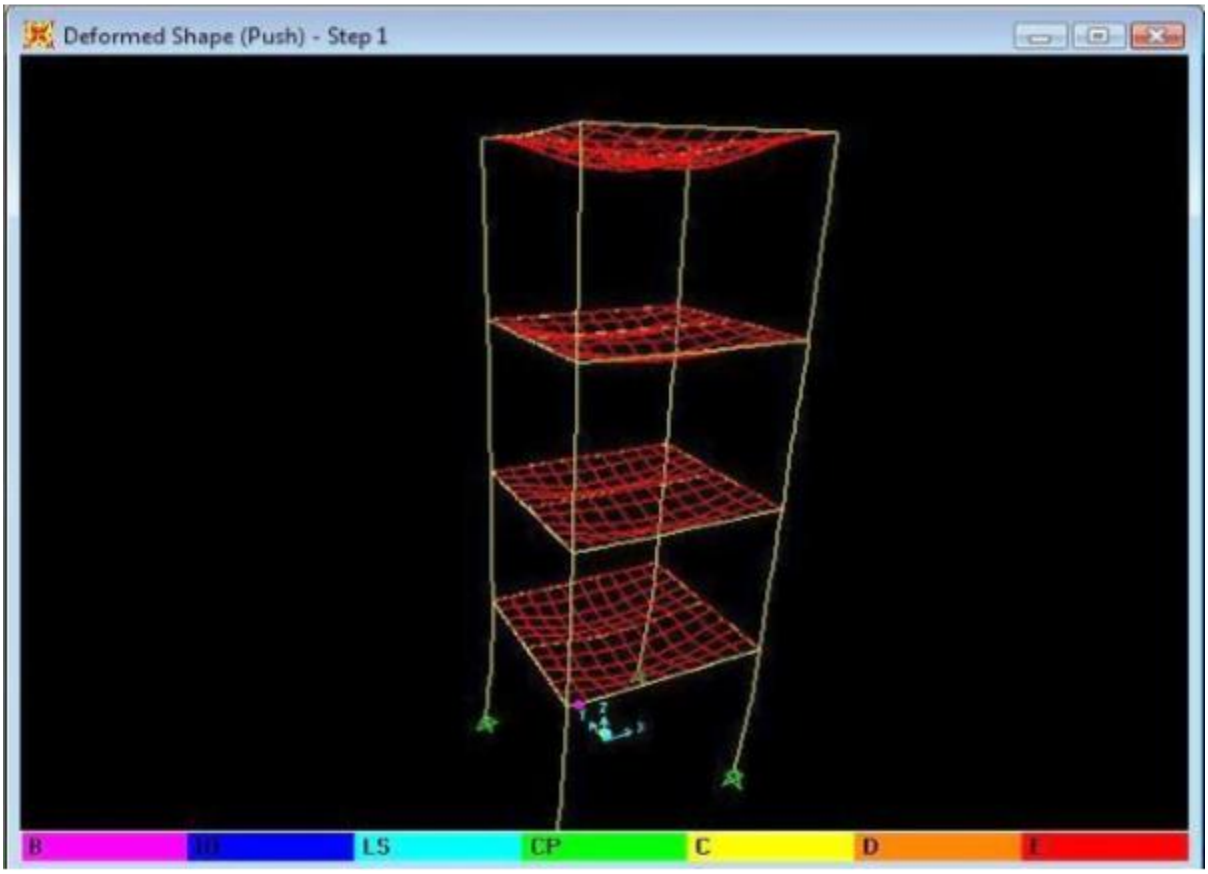
Fig 5.9 Plot between Base shear vs. Displacement

Step	Displacement	Base Force	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
	(M)	(KN)									
0	0.000085	0	92	0	0	0	0	0	0	0	92
1	0.002128	200.2	91	1	0	0	0	0	0	0	92
2	0.007542	400.3	83	9	0	0	0	0	0	0	92
3	0.007741	501.7	79	13	0	0	0	0	0	0	92
4	0.008436	518.6	77	15	0	0	0	0	0	0	92
5	0.016194	575.5	74	14	4	0	0	0	0	0	92
6	0.01634	585.3	69	18	5	0	0	0	0	0	92
7	0.016734	590.1	69	18	5	0	0	0	0	0	92
8	0.024679	591.3	69	18	5	0	0	0	0	0	92
9	0.024681	591.5	69	18	5	0	0	0	0	0	92
10	0.02818	593.4	68	19	5	0	0	0	0	0	92
11	0.03001	594.1	61	25	6	0	0	0	0	0	92
12	0.041833	595.05	60	25	7	0	0	0	0	0	92
13	0.04473	597.3	58	25	5	4	0	0	0	0	92
14	0.04683	597.5	58	25	5	4	0	0	0	0	92
15	0.048805	600.1	58	25	4	5	0	0	0	0	92
16	0.088	602.1	58	25	3	6	0	0	0	0	92
17	0.1022	605.2	55	24	5	8	0	0	0	0	92
18	0.11	607.3	54	25	5	8	0	0	0	0	92
19	0.1423	615.4	54	24	5	9	0	0	0	0	92
20	0.1734	625.7	54	24	5	9	0	0	0	0	92
21	0.2168	650.9	53	24	5	10	0	0	0	0	92
22	0.2568	690.3	52	25	5	10	0	0	0	0	92
23	0.3148	720.4	52	21	8	11	0	0	0	0	92
24	0.4123	750.3	50	22	7	13	0	0	0	0	92
25	0.4225	770.3	45	26	7	14	0	0	0	0	92
26	0.4335	830.3	43	26	7	16	0	0	0	0	92
27	0.4838	904.5	31	34	10	17	0	0	0	0	92
28	0.500085	904.5	31	33	8	20	0	0	0	0	92
29	0.5011	904.5	31	27	12	22	0	0	0	0	92
30	0.502	904.5	31	26	11	24	0	0	0	0	92
31	0.51	904.5	31	26	11	24	0	0	0	0	92

Table 5.2 Tabular Data for Pushover Curve

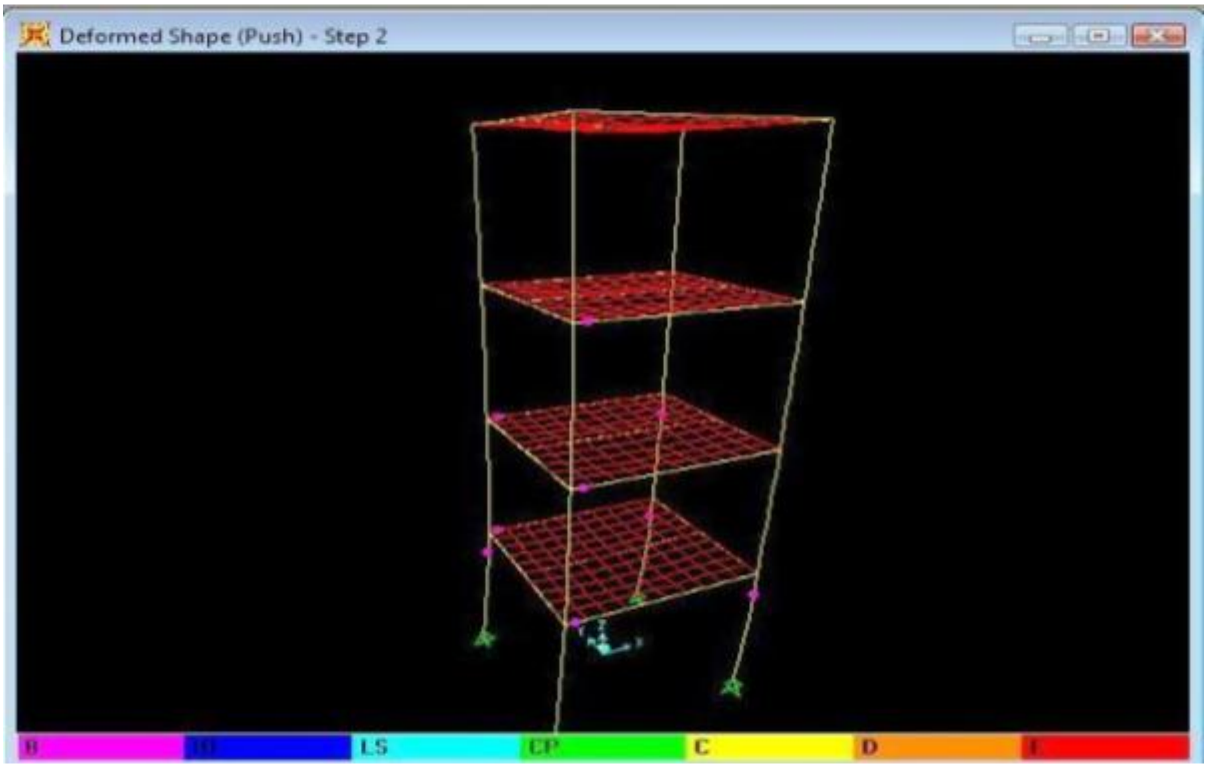


Step 0

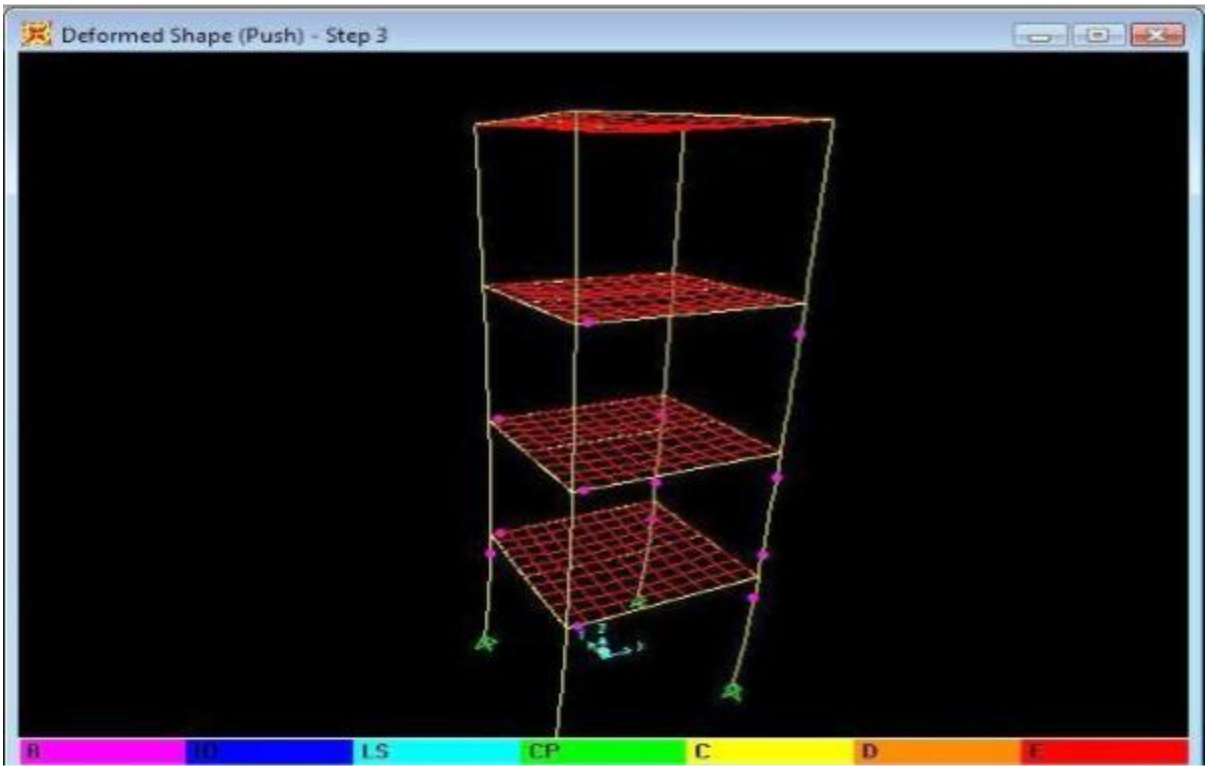


Step 1

Fig 5.10(a): Step by step deformation for Pushover

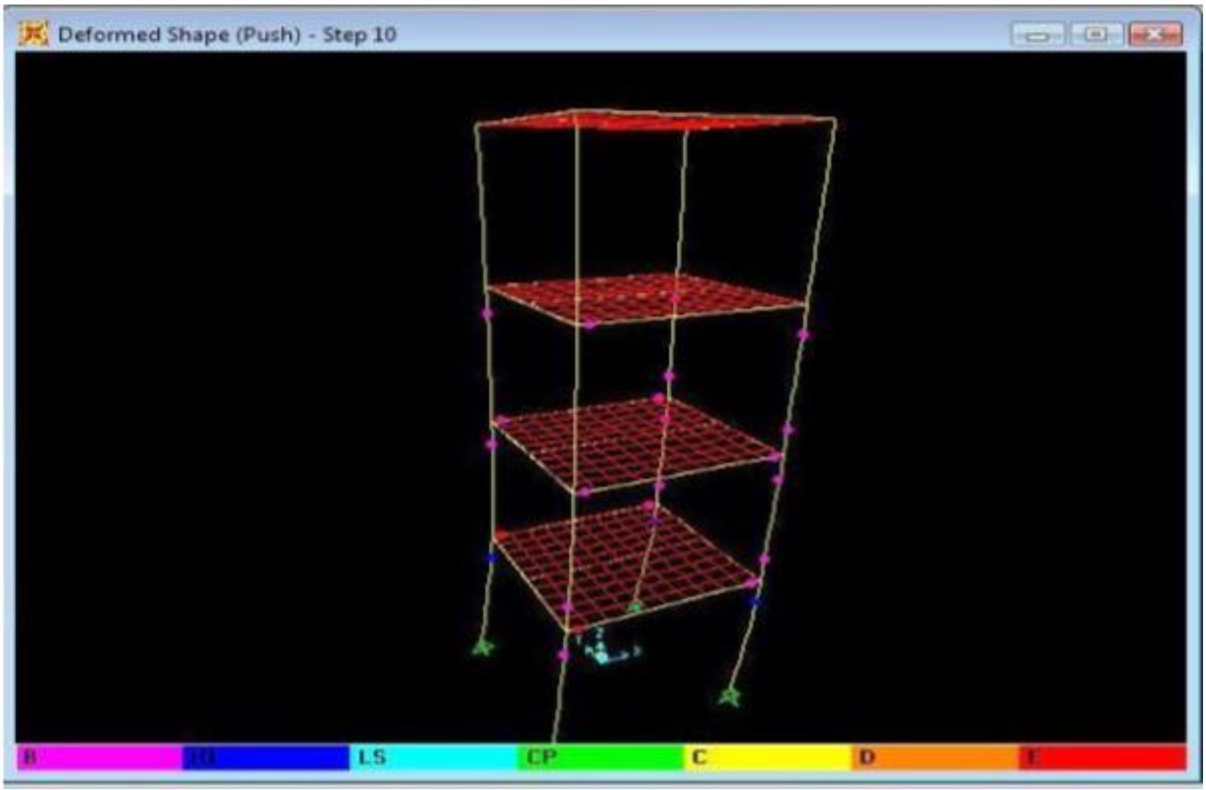


Step 2

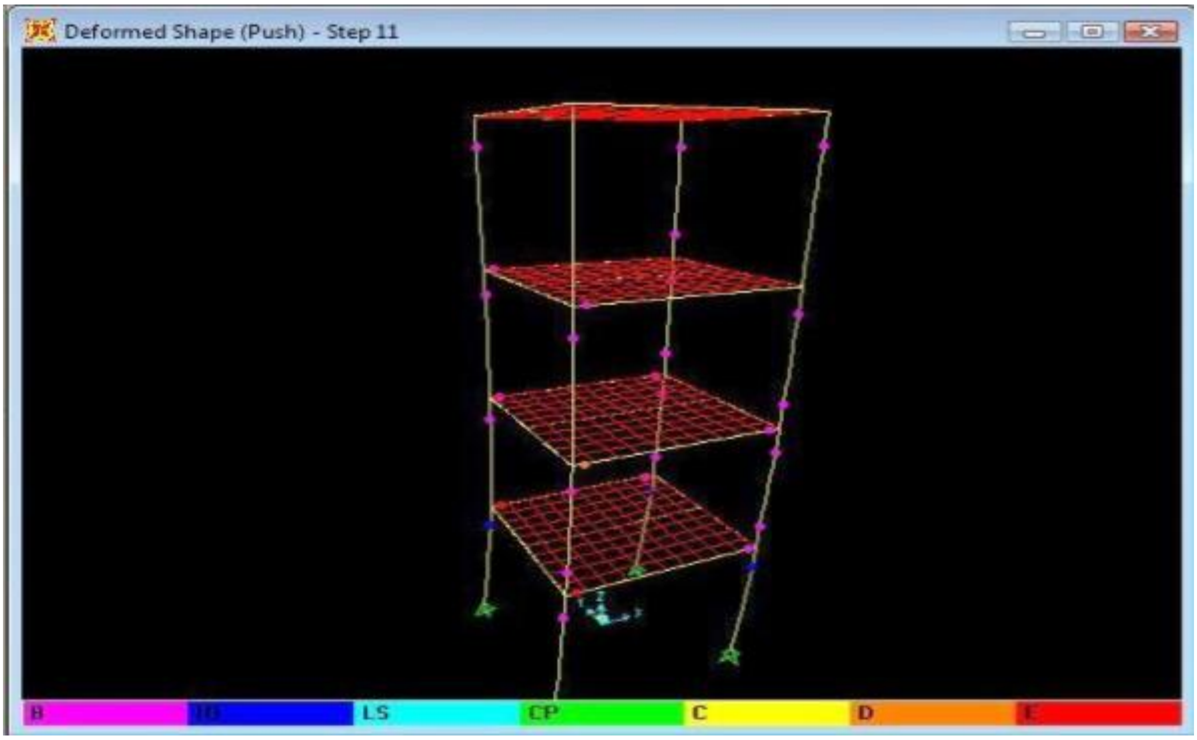


Step 3

Fig 5.10(b) : Step by step deformation for Pushover

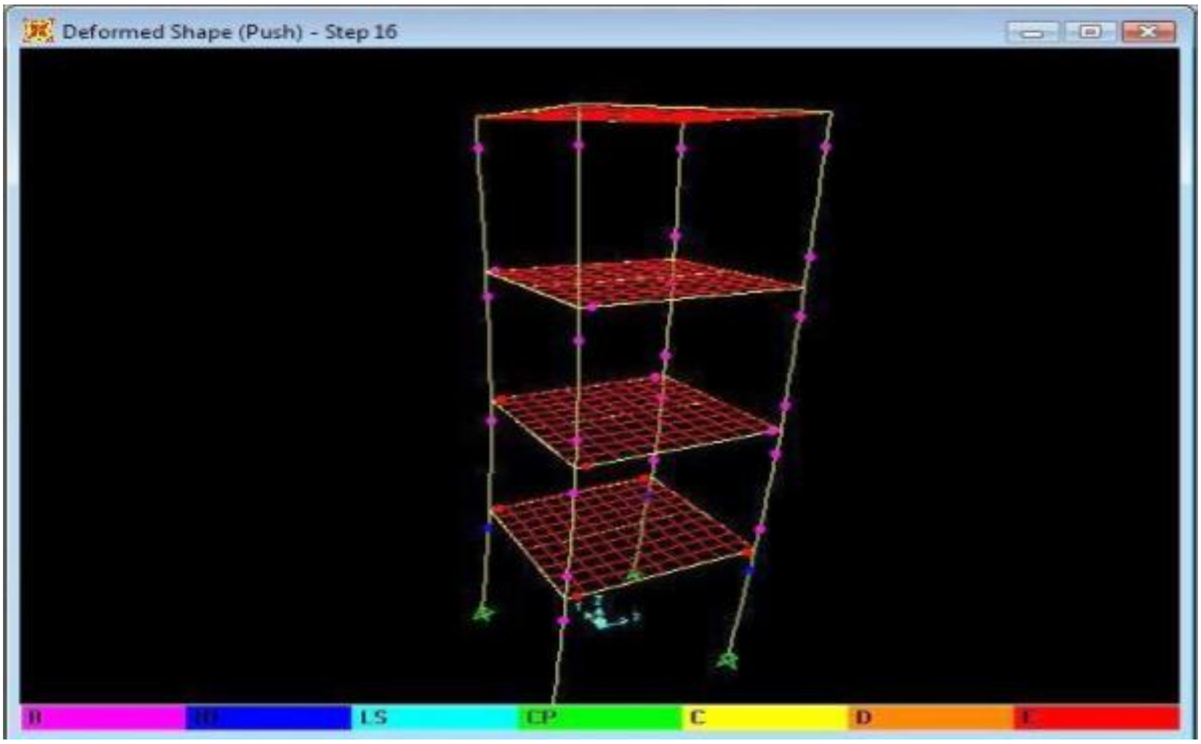


Step 10

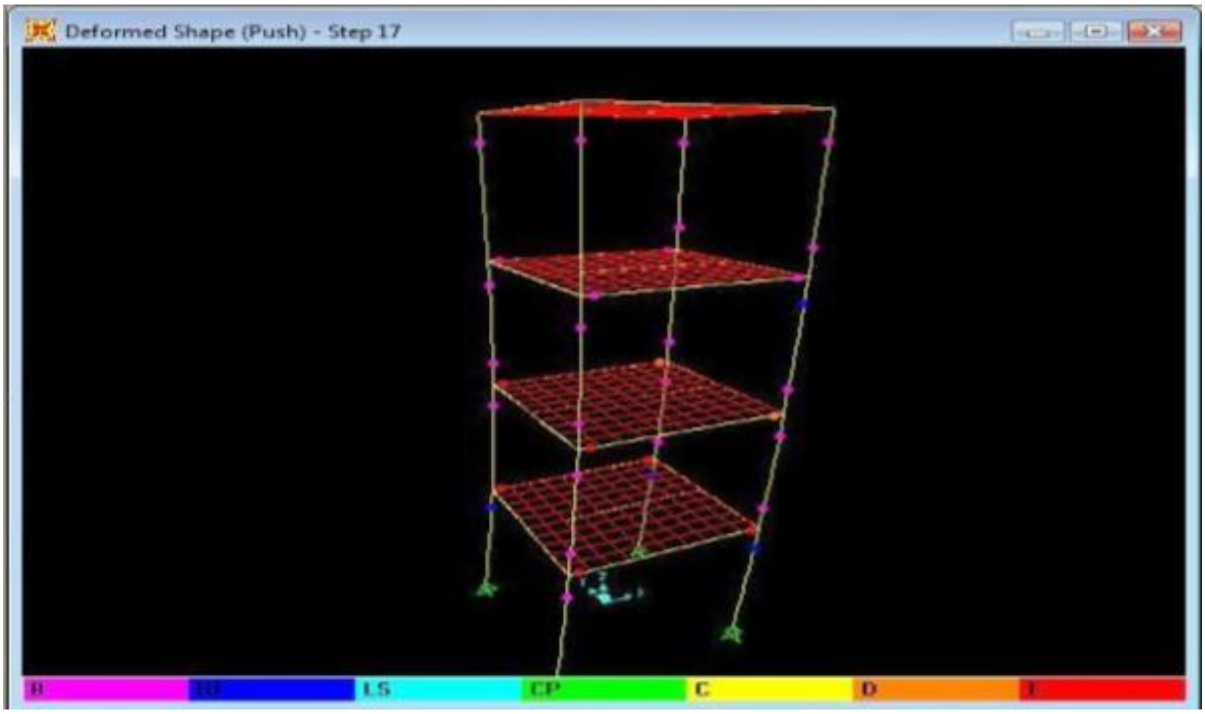


Step 11

Fig 5.10(c) : Step by step deformation for Pushover

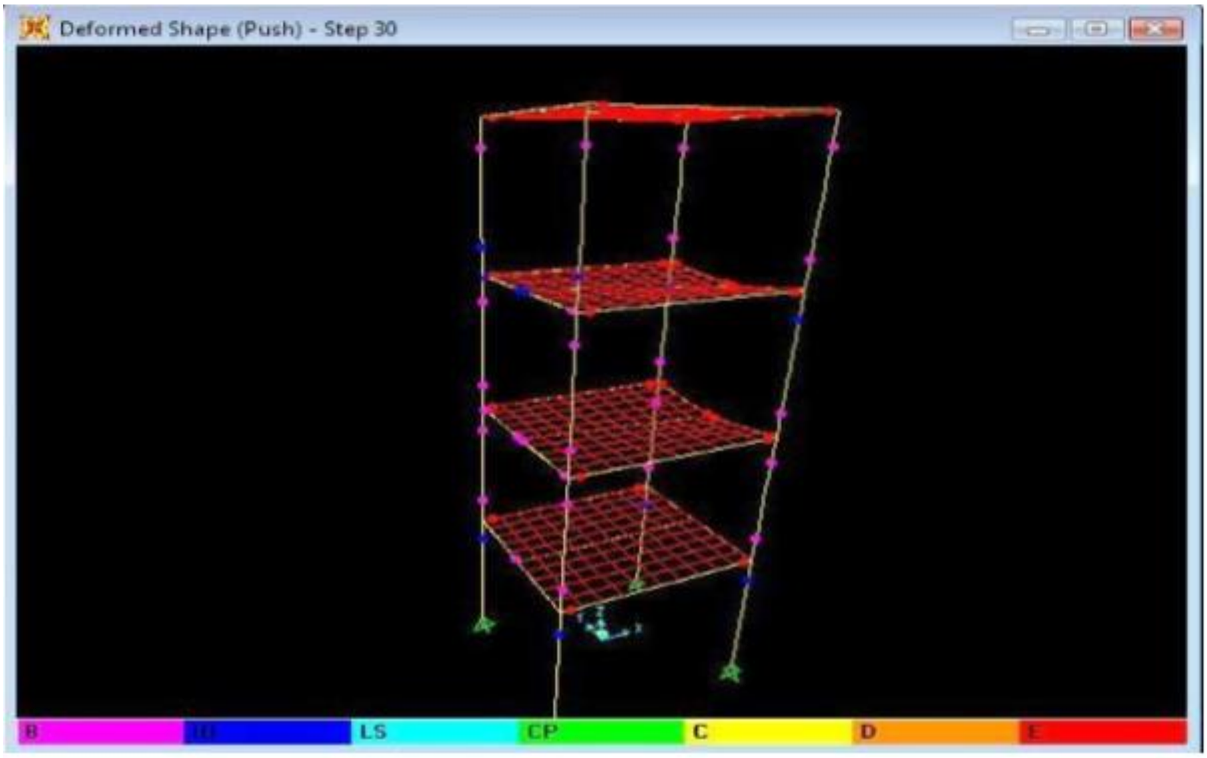


Step 16

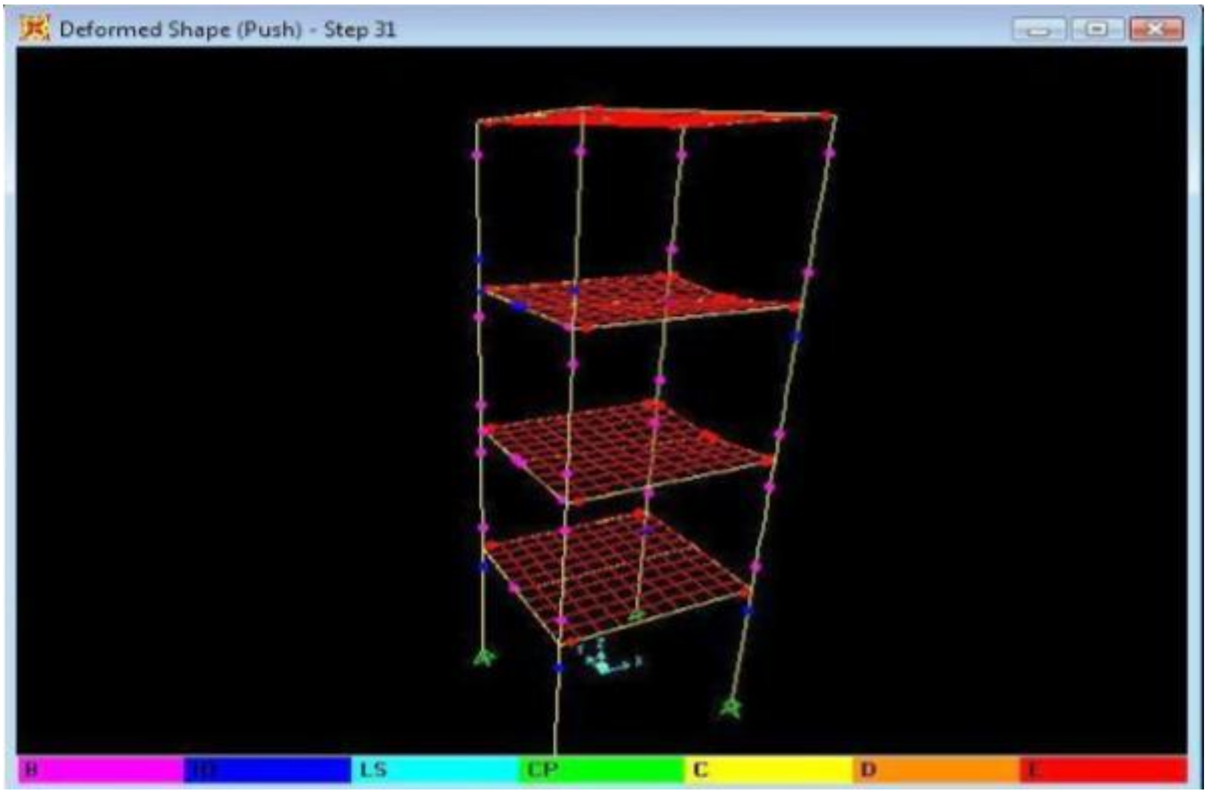


Step 17

Fig 5.10(d): Step by step deformation for Pushover



Step 30



Step 31

Fig 5.10(e): Step by step deformation for Pushover

CHAPTER 6

CONCLUSIONS

6.1 CONCLUSIONS

The performance of reinforced concrete frames were investigated using the pushover analysis. The conclusions drawn from the analysis are as follows.

- The pushover analysis is a relatively simpler way to explore the non-linear behaviour of buildings.
- The behaviour of properly detailed reinforced concrete frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns.
- Most of the hinges are developed in the beams and few in the columns but with limited damage.
- The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures.
- The causes of failure of reinforced concrete during the earthquake may be attributed to the quality of the materials and ductility of the members primarily at the joints.

REFERENCES

- *Datta T K*, “Seismic Analysis of Structures”, John Wiley & Sons, 2010.
- *Hasan R., XuL.,Grierson D.E*, “Pushover analysis for performance based seismic design”, Dept. of Civil Engineering, University of Waterloo, July 2002.
- Federal Emergency Federal Agency, FEMA-356. Pre-standard and Commentary for Seismic Rehabilitation of Buildings. Washington DC, 2000.
- *Habibullah A. and Pyle S.*, “Practical three dimensional nonlinear static analysis”, Structure Magazine , Winter, 1998.
- *M.Seifi, “*Nonlinear Static Pushover Analysis in Earthquake Engineering”.
- *KadidA.andBoumrkik A. ,*“Pushover Analysis of RCC Structures”.
- *Poluraju P. and NageswarRao*, “Pushover analysis of reinforced concrete frame structure using SAP 2000” *CSI-SAP2000 Training Manual, 2010.*
- ATC.Seismic evaluation and retrofit of concrete buildings—volume 1 (ATC-40). Report No. SSC 96-01. Redwood City (CA): Applied Technology Council; 1996.
- ATC 55. Evaluation and improvement of inelastic seismic analysis procedures. (2001).
- ATC, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273 Report, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C. 1997a.
- *Moehle JP, Mahin SA*, “Observations on the behavior of reinforced concrete buildings during earthquakes” in Earthquake-resistant concrete structures—inelastic response and design. American Concrete Institute SP-127; 1991.
- *Oscar Möller, Ricardo O. Foschi, Laura M. Quiroz, Marcelo Rubinstein*,”Structural optimization for performance-based design in earthquake engineering” in Applications of neural networks Structural Safety 31, 490–499. (2009).
- *Ho, S.Y. and Wang, C. Y. ,”*Determination of the Plastic Hinge Properties of RC structure containing Brick Walls,” in Structural Engineering of Taiwan, 2006.