PUSHOVER ANALYSIS OF REINFORCED CONCRETE FRAME

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Submitted by

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CERTIFICATE

This is to certify that the Thesis titles,"Pushover analysis of Reinforced Concrete Frame", being submitted by **Rohit Bansal**, in partial fulfillment of the requirements for the award of degree of MASTER OF ENGINEERING (STRUCTURES) at THAPAR UNIVERSITY, PATIALA is a bonafide work carried out by him under my guidance and supervision and no part of this thesis submitted for the award of any other degree.

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ABSTRACT

To model the complex behaviour of reinforced concrete analytically in its non-linear zone is difficult. 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 include the variation in load displacement graph.

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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 was 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 seismo-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. It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved. **(ATC, 1997a)**

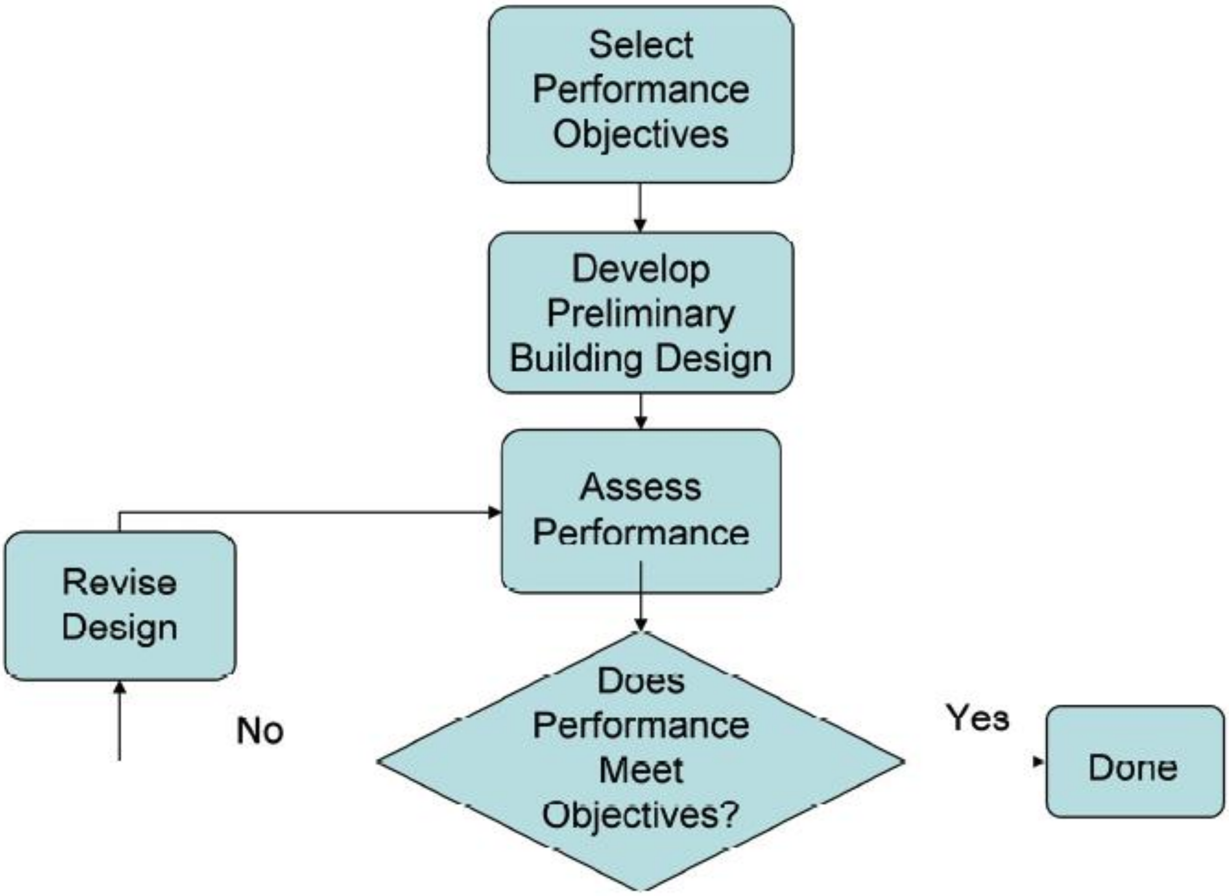

Fig. 1.1 Performance-Based Design Flow Diagram (ATC, 1997a)

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 trilinear 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). Also, in force-controlled pushover procedure some numerical problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P-delta effects.

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., 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, force demands on brace connections, 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 inelastically 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 interstory 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 systems, all the connections, and stiff non-structural elements of significant strength, and the foundation system. (**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 behavior. 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 are 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 modeling nonlinear member behavior, 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 IN PUSHOVER ANALYSIS ON 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 LabelH#, 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 OBJECTIVE

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. Firstly, the superiority of pushover analysis over elastic procedures in evaluating the seismic performance of a structure is discussed by identifying the advantages and limitations of the procedure. Then, pushover analyses are performed on case study frames using SAP2000.

Also, the effects and the accuracy of various invariant lateral load patterns 'Uniform', 'Elastic First Mode', 'Code', 'FEMA-273' and 'Multi-Modal utilized in traditional pushover analysis to predict the behavior imposed on the structure due to randomly selected individual ground motions causing elastic and various levels of nonlinear response are evaluated. 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. Story displacements, inter-story drift ratios and plastic hinge locations are also estimated by performing an improved pushover procedure named Modal Pushover Analysis (MPA) on case study frames. Pushover predictions are compared with the 'exact' values of response parameters obtained from the experimental results to assess the accuracy of software.

1.7 SCOPE OF THE PRESENT STUDY

In the present study, modelling of the RCC frame under the loads has been analyzed using SAP2000 software and the results so obtained have been compared with available experimental results from the Push Over test conducted at CPRI Banglore.

The frame is analyzed using SAP2000 software up to the failure and the load deformation curves. In this study user defined hinges are used in beams and columns. The frame has been analyzed and results have been compared with the experimental results.

1.8 ORGANIZATION OF THE THESIS

The thesis is organized as per detail given below:

Chapter 1: Introduces to the topic of thesis in brief.

Chapter 2: Discusses the literature review i.e. the work done by various researchers in the field of modelling of structural members by pushover analysis.

Chapter 3: In this chapter pushover analysis has been discussed in detail. The theory related to pushover analysis also discussed in brief.

Chapter 4: Deals with the details of the structure modelled in SAP2000.

Chapter 5: The results from the analysis, comparison between the analytical and the experimental results, all are discussed in this chapter.

Chapter 6: Finally, salient conclusions and recommendations of the present study are given in this chapter followed by the references.

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

Dhileep. M et al., (2011) explained the practical difficulties associated with the non linear direct numerical integration of the equations of motion leads to the use of non linear static pushover analysis of structures. Pushover analysis is getting popular due to its simplicity. High frequency modes and non linear effects may play an important role in stiff and irregular structures. The contribution of higher modes in pushover analysis is not fully developed. The behavior of high frequency model responses in non linear seismic analysis of structures is not known. In this paper an attempt is made to study the behavior of high frequency model responses in non linear seismic analysis of structures.

Non linear static pushover analysis used as an approximation to non linear time history analysis is becoming a standard tool among the engineers, researches and professionals worldwide. High frequency modes may contribute significantly in the seismic analysis of irregular and stiff structures. In order to take the contribution of higher modes structural engineers may include high frequency modes in the non linear static pushover analysis. The behavior of high frequency modes in non linear static pushover analysis of irregular structures is studied. At high frequencies, the responses of non linear dynamic analysis converge to the non linear static pushover analysis. Therefore non linear response of high frequency modes can be evaluated using a non linear static push over analysis with an implemental force pattern given by their modal mass contribution times zero period acceleration. The higher modes with rigid content as a major contributing factor exhibit a better accuracy in non linear pushover analysis of structures when compared to the damped periodic modes.

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

1. Performance-based design in earthquake engineering implies consideration of the uncertainties in the structura
2. l 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.

2\. 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.

3\. 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.

4\. 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.

5\. 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.

6\. 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 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. is 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 modeled 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 deforma

tions 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 0.8*lw* 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 skeptically 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.

A. Shuraim et al., (2007) summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame, in order to examine its applicability. Potential structural deficiencies in RC frame, when subjected to a moderate seismic loading, were estimated by the code seismic-resistant design and pushover approaches. In the first method the design was evaluated by redesigning under one selected seismic combination in order to show which members would require additional reinforcement. It was shown that most columns required significant additional reinforcement, indicating their vulnerability if subjected to seismic forces. On the other hand, the nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column. Vulnerability locations from the two procedures are significantly different. The paper has discussed the reasons behind the apparent discrepancy which is mainly due to the default assumptions of the method as implemented by the software versus the code assumptions regarding reduction factors and maximum permissible limits. In new building design, the code always maintains certain factor of safety that comes from load factors, materials reduction factors, and ignoring some post yielding characteristics (hardening). In the modeling assumptions of ATC-40, reduction factor is assumed to be one, and hardening is to be taken into consideration. Hence, the paper suggests that engineering judgment should be exercised prudently when using the pushover analysis and that engineer should follow the code limits when designing new buildings and impose certain reductions and limits in case of existing buildings depending on their conditions. In short software should not substitute for code provisions and engineering judgment.

A. Whittaker , Y. N. Huang et al (2007) summarize the next (second) generation tools and procedures for performance-based earthquake engineering in the United States. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings.

The procedures and methodologies described in these guidelines include an explicit treatment of the large uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering and a significant departure from the first generation deterministic procedures.

Fig.2.1 identifies the five basic steps proposed for a next-generation seismic performance assessment. Unlike prior assessment procedures that addressed either structural damage or repair cost, three measures of seismic performance are proposed in the guidelines: 1) direct economic loss (repair cost), 2) indirect economic loss (downtime or business interruption), and 3) casualties (including injuries and death). Each of three performance measures is treated as a potential loss. Section 2 of the paper introduces the three types of performance assessment that can be performed using the draft Guidelines and identifies the basic procedure for each. Section 3 describes the five steps for seismic performance assessment that are identified in Fig 2.1.

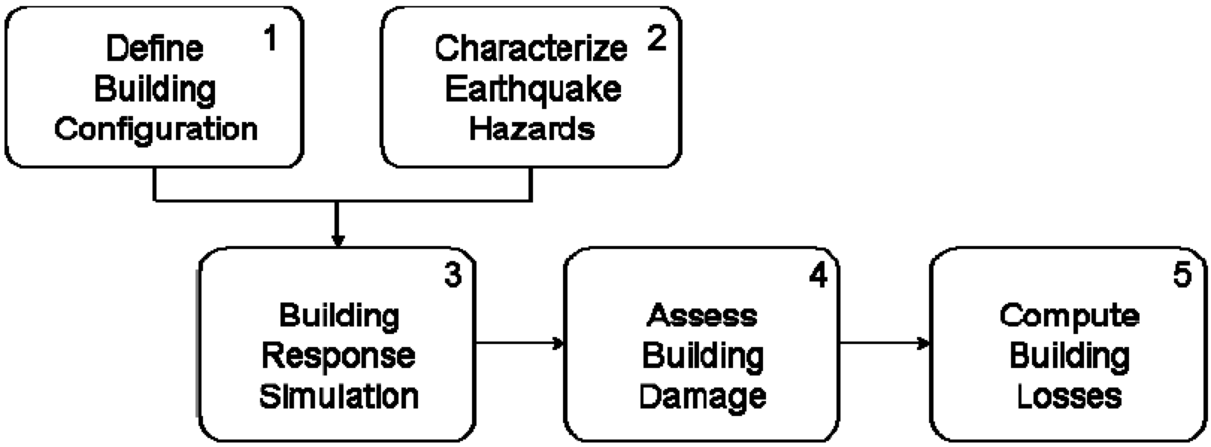

Figure 2.1 Procedure for Performance Assessment (Whittaker et al., 2007)

The procedures set forth in these guidelines represent a substantial departure from the deterministic tools and procedures used at this time because uncertainty and randomness is captured explicitly in every step of the proposed procedures. Fragility functions, damage states and building-level consequence functions, are used in the proposed procedures to compute losses.

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 non-linear 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 **(Cosenza, E. et al., 1998).**

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^{1} is due to the plastic rotation of the column and Lp^{11} to the fixed end rotation at the footing zone of the column. In order to extrapolate a formulation for Lp^{1} and Lp^{11}, 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 fc=30 MPa and the volumetric percentage of stirrups is 0.1%. The ratio ft/fy does vary 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/Nu considered are 0, 0.1, 0.2, 0.3, and 0.4.

$$L_p^{\ I} = 6.1 \cdot \left(\frac{L}{H}\right)^{0.43} \cdot \left(\frac{f_t}{f_y}-1\right)^{0.65} \cdot \varepsilon^{-0.32} \cdot \left(1+\frac{N}{N_o}\right)^{-1.83}$$

$$L_p^{\ II} = 5 \cdot d_b \cdot \left(\frac{f_t}{f_y}-1\right)^{0.2}$$

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:

$$Lp = 0.08L + 6d_b$$

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^1 and Lp^{11} 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:

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

Recently in **(Fib Bulletin. 2003)** formulations similar to last eqn. for monotonic and cyclic loads have been suggested, as follows:

for monotonic loads: $$\mathbf{Lp = 0.18Ls + 0.025f_y \,.\, d_b}$$

for cyclic loads: $$\mathbf{Lp = 0.08Ls + 0.017f_y \,.\, d_b}$$

where Ls is the shear span

At last he concluded in his formulation that the availability of a reliable formulation for the plastic hinge length is a key issue for any analysis of r.c. element ductility, i.e. to non-linear behaviour of r.c. frames under seismic actions. The proposed formulation is based on a wide numerical analysis developed through a detailed mechanical model which takes into account the non-linear constitutive relationship of material and the steel-concrete bond law. It allows considering the effect of yielding penetration between cracks of the structures but also at the steel anchorage in the foundation.

In particular, the two contributions to the plastic deformability of a column can be separately evaluated multiplying the respective plastic length by the curvature of the section at the element base; thus the element ductility can be easily evaluated knowing the section behaviour. The formulation in terms of plastic rotation takes into account many parameters and shows a low scatter respect to the numerical results; furthermore the influence of parameters appears in agreement with the mechanical behaviour. The range of some parameters considered to assess the proposal is wider than the ones used in experimental tests at the base of other available formulations, but it is limited to cold formed steel and elements without shear-flexure interaction; therefore the analysis has to be extended developing an experimental comparison too.

Chung-Yue Wang et al., (2007) in this paper he presented a method for the determination of the parameters of plastic hinge properties (PHP) for structure containing RC wall in the pushover analysis is proposed. Nonlinear relationship between the lateral shear force and lateral deformation of RC wall is calculated first by the Response-2000 and Membrane-2000 code. The PHP (plastic hinge properties) value of each parameter for the pushover analysis function of SAP2000 or ETABS is defined as the product of two parameters α and β. Values of α at states of cracking, ultimate strength and failure of the concrete wall under shear loading can be determined respectively from the calculations by Response-2000. While the corresponding β value of each PHP parameter is obtained from the regression equations calibrated from the experimental results of pushover tests of RC frame-wall specimens. The accuracy of this newly proposed method is verified by other experimental results. It shows that the presented method can effectively assist engineers to conduct the performance design of structure containing RC shear wall using the SAP2000.
SAP2000 is a well known and widely accepted, general-purpose, three-dimensional structural analysis program. The pushover analysis module has been installed into the SAP2000. In the procedure of the pushover analysis, the assignment of the values of plastic hinge properties (PHP) strongly affects the prediction of the capacity curve of RC structure.
SAP2000 program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members. These built-in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses **(Habibullah and Pyle, 1998)**. Yielding and post-yielding behavior can be modeled using discrete user-defined hinges. Currently SAP2000 allows hinges can only be introduced into frame elements; the PHP properties can be assigned to a frame element at any location along it. The authors have been developed a dual parameters method to define the PHP properties of RC frame structure for the pushover analysis **(Ho and Wang, 2006)**. The purpose of this paper is to extend the application of this method to the RC structures containing RC shear wall. In order to use the functions provided by the SAP2000 code, the RC shear wall is treated as a wide, flat column. Modeling a RC wall as a wide and flat column (frame

elements) not only can consider the steel reinforcements in RC elements exactly, but also can assign the PHP of RC walls according to its plastic behavior. In SAP2000, the default properties are available for hinges in the following degrees of freedom:

1. Axial (P)
2. Major shear (V2)
3. Major moment (M3)
4. Coupled P-M2-M3 (PMM)

He concluded that a dual parameters method is introduced to define the plastic hinge properties (PHP) of RC wall in the pushover analysis of RC structure. The effectiveness of this simple method is verified by the agreement of the prediction curves with some additional test data. This newly proposed method is quite simple and is easy for engineers to link with commercial structural analysis code to conduct the performance design of structure under seismic loading.

Konuralp Girgin et al., (2007) explained that structural frames are often filled with infilled walls serving as partitions. Although the infills usually are not considered in the structural analysis and design, their influence on the seismic behaviour of the infilled frame structures is considerable. In this study, a parametric study of certain infilled frames, using the strut model to capture the global effects of the infills was carried out. Three concrete planar frames of five-stories and three-bays are considered which have been designed in accordance with Turkish Codes. Pushover analysis is adopted for the evaluation of the seismic response of the frames. Each frame is subjected to four different loading cases. The results of the cases are briefly presented and compared. The effect of infill walls on seismic behaviour of two sample frames with different infill arrangements was investigated. The results yield that it is essential to consider the effect of masonry infills for the seismic evaluation of moment-resisting RC frames, especially for the prediction of its ultimate state, infills having no irregularity in elevation have beneficial effect on buildings and infills appear to have a significant effect on the reduction of global lateral displacements.

Infills have been generally considered as non-structural elements, although there are codes such as the Eurocode-8 that include rather detailed procedures for designing infilled R/C frames, presence of infills has been ignored in most of the current seismic codes except their weight. However, even though they are considered non-structural elements the presence of infills in the reinforced concrete frames can substantially change the seismic response of buildings in certain cases producing undesirable effects (torsional effects, dangerous collapse mechanisms, soft storey, variations in the vibration period, etc.) or favourable effects of increasing the seismic resistance capacity of the building.

The pushover analysis can be considered as a series of incremental static analyses carried out to examine the non-linear behaviour of structure, including the deformation and damage pattern. The procedure consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation. Then, a pushover analysis is carried out on the structure until the displacement at the top of the building reaches the target displacement. The extent of damage experienced by the building at the target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking. A judgment is formed as to the acceptability of the structural behavior for the design of the new building, or the level of damage of an existing building for evaluation purposes.

In the conclusion he states that the effect of infill walls on seismic behavior of a two sample frames with different infill arrangements was investigated. The results yields the following conclusions.

1. It is essential to consider the effect of masonry infills for the seismic evaluation of moment resisting RC frames, especially for the prediction of its ultimate state.

2. Infills having no irregularity in elevation have beneficial effect on buildings. In infilled frames with irregularities, such as soft story, damage was found to concentrate in the levels where the discontinuity occurs.

3. Since infills increases lateral resistance and initial stiffness of the frames they appear to have a significant effect on the reduction of the global lateral displacement.

4. Arrangement of infills may effect the post yield behavior and has an influence on distribution and sequence of damage formation. To generalize this, more infill arrangements should be investigated.

5. A carefully performed pushover analysis can provide insight into structural aspects that control performance of the structure during a severe earthquake.

6. The choice of the static load distribution used in pushover analysis can affect the accuracy of the response estimates.

Mehmet et al., (2006) explained that due to its simplicity, the structural engineering profession has been using the nonlinear static procedure (NSP) or pushover analysis. Modeling for such analysis requires the determination of the nonlinear properties of each component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions. Pushover analysis is carried out for either user-defined nonlinear hinge properties or default-hinge properties, available in some programs based on the FEMA-356 and ATC-40 guidelines. While such documents provide the hinge properties for several ranges of detailing, programs may implement averaged values. The user needs to be careful; the misuse of default-hinge properties may lead to unreasonable displacement capacities for existing structures. This paper studies the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. Four- and seven-story buildings are considered to represent low- and medium- rise buildings for this study. Plastic hinge length and transverse reinforcement spacing are assumed to be effective parameters in the user-defined hinge properties. Observations show that plastic hinge length and transverse reinforcement spacing have no influence on the base shear capacity, while these parameters have considerable effects on the displacement capacity of the frames. Comparisons point out that an increase in the amount of transverse reinforcement improves the displacement capacity. Although the capacity curve for the default-hinge model is reasonable for modern code compliant buildings, it may not be suitable for others. Considering that most existing buildings in Turkey and in some other countries do not conform to requirements of modern code detailing, the use of default hinges needs special care. The observations clearly show that the user-defined hinge model is better than the default-hinge model in reflecting nonlinear behavior compatible with the element properties. However, if the default-hinge model is preferred due to simplicity, the user should be aware of what is provided in the program and should avoid the misuse of default-hinge properties.

He concluded that the interior frames of 4- and 7-story buildings were considered in pushover analyses to represent low- and medium rise reinforced concrete (RC) buildings for study. Beam and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. The frames were modeled with default and user-defined hinge properties to study possible differences in the results of pushover analyses. The following findings were observed:

1. The base shear capacity of models with the default hinges and with the user-defined hinges for different plastic hinge length and transverse reinforcement spacing are similar; the

variation in the base shear capacity is less than 5%. Thus, the base shear capacity does not depend on whether the default or user-defined hinge properties are used.

2. Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. Comparisons show that there is a variation of about 30% in displacement capacities due to Lp.

3. Displacement capacity depends on the amount of transverse reinforcement at the potential hinge regions. Comparisons clearly point out that an increase in the amount of transverse reinforcement improves the displacement capacity. The improvement is more effective for smaller spacing. For example, reducing the spacing from 200 mm to 100 mm provides an increase of up to 40% in the displacement capacity, while reducing the spacing from 200 mm to 150 mm provides an increase of only 12% for the 4-story frame.

4. Comparison of hinging patterns indicates that both models with default hinges (Case A) and the user-defined hinges (Case B3) estimate plastic hinge formation at the yielding state quite well. However, there are significant differences in the hinging patterns at the ultimate state. Although the hinge locations seem to be consistent, the model with default hinges emphasizes a ductile beam mechanism in which the columns are stronger than the beams; damage or failure occurs at the beams. However, this mechanism is not explicitly guaranteed for the structures designed according to the 1975 Turkish Earthquake Code or pre-modern codes in other countries.

5. Time-history results point out that pushover analysis is reasonably successful in capturing hinging patterns for low and medium-rise buildings, except that the plastic hinge formation in the upper levels is not estimated adequately by pushover analysis, as observed by other researchers.

6. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties. Based on the observations in the hinging patterns, it is apparent that the user-defined hinge model is more successful in capturing the hinging mechanism compared to the model with default hinges.

7. Although the capacity curve for the default-hinge model is reasonable for modern code compliant buildings, it may not be suitable for others. Considering that most existing buildings in Turkey and some other countries do not conform to requirements of modern code detailing, the use of default hinges needs special care.

Some programs (i.e. SAP2000) provide default-hinge properties based on the ATC - 40 or FEMA-356 documents to make modeling practical for nonlinear analysis. If they are used cautiously, they relive modeling work considerably. The misuse of default-hinge properties may result in relatively high displacement capacities. Based on the observations in this study, it is clear that, although default-hinge properties provided in SAP2000 are suitable for modern code compliant buildings, the displacement capacities are quite high for other buildings. Pushover analysis of the default-hinge modal emphasizes a ductile beam mechanism for buildings constructed according to pre-modern codes, while Pushover analysis of the user-defined hinge model and time-history analysis of both models indicate strong beams and weak columns. This study is carried out to investigate the possible differences between Pushover analysis of the default-hinge and user-defined hinge models. The observations clearly show that the user-defined hinge model is better than the default-hinge model in reflecting nonlinear behavior compatible with element properties. However, if the default-hinge model is preferred due to simplicity, the user should be aware of what is provided in the programme and should definitely avoid the misuse of default-hinge properties.

X.-K. Zou et al., (2005) presented an effective computer- based technique that incorporates Pushover Analysis together with numerical optimisation procedures to automate the Pushover drift performance design of reinforced concrete buildings. Performance-based design using nonlinear pushover analysis, which generally involves tedious and intensive computational effort, is a highly iterative process needed to meet designer-specified and code requirements. This paper presents an effective computer-based technique that incorporates pushover analysis together with numerical optimization procedures to automate the pushover drift performance design of reinforced concrete (RC) buildings. Steel reinforcement, as compared with concrete materials, appears to be the more cost-effective material, that can be effectively used to control drift beyond the occurrence of first yielding and to provide the required ductility of RC building frameworks. In this study, steel reinforcement ratios are taken as design variables during the design optimization process. Using the principle of virtual work, the nonlinear inelastic seismic drift responses generated by the pushover analysis can be explicitly expressed in terms of element design variables. An optimality criteria technique is presented in this paper for solving the explicit performance-based seismic design optimization problem for RC buildings.

It has been recognized that the interstory drift performance of a multistory building is an important measure of structural and non-structural damage of the building under various levels of earthquake motion **(Moehle JP, Mahin SA., 1991)**. In performance based design, interstory drift performance has become a principal design consideration **(SEAOC. 1995, ATC-40. 1996)**. The system performance levels of a multistory building are evaluated on the basis of the interstory drift values along the height of the building under different levels of earthquake motion **(Ghobarah A. et al., 1997)**. The control of interstory drift can also be considered as a means to provide uniform ductility over all stories of the building. A large story drift may result in the occurrence of a weak story that may cause catastrophic building collapse in a seismic event. Therefore, a uniform story ductility over all stories for a multistory building is usually desired in seismic design **(Chopra AK., 1995)**. Numerous studies on structural optimization in the seismic design of structures have been published in the past two decades, including **(Cheng and Botkin, 1976)**, **(Feng et al., 1977)**, **(Bhatti and Pister., 1981)**, **(Balling et al., 1983)**, **(Cheng and Truman., 1983)**, **(Arora J.S., 1999)**. However, most of these previous research efforts were concerned with optimization through prescriptive-based design concepts. Recently, **(Beck JL. et al., 1998)** developed an optimization methodology for performance-based design of structural systems operating in an uncertain dynamic environment. **(Ganzerli S. et al., 2000)** presented an optimal performance-based design method for a reinforced concrete (RC) frame, in which performance based constraints were implemented in terms of plastic rotations of the beams and columns of the frame. **(Foley CM. 2002)** provided a review of current state-of-the-art seismic performance-based design procedures and presented the vision for the development of performance-based design optimization. It has been recognized that there is a pressing need for developing optimized performance-based design procedures for seismic engineering of structures.

In seismic design, it is commonly assumed that a building behaves linear-elastically under minor earthquakes and may respond nonlinear-inelastically when subjected to moderate and severe earthquakes. Under such an assumption, the entire design optimization process can therefore be decomposed into two phases **(Zou XK,. 2001, Chan CM, 2002)**. In the first phase, the structural concrete cost is minimized subject to elastic drift responses under minor earthquake loading using elastic response spectrum analysis. In this phase, concrete member sizes are considered as the only design variables since the concrete material plays a more dominant role in improving the elastic drift performance of the building. Once the optimal structural member sizes are determined at the end of the first phase of the optimization, the steel reinforcement quantities can then be considered as design variables in the second phase. In controlling the inelastic drift responses, steel reinforcement is the only effective material that provides ductility to an RC building structure beyond first yielding. In this second design phase, the member sizes are kept unchanged and the cost of the steel reinforcement is minimized subject to design constraints on inelastic interstory drift produced by the nonlinear pushover analysis.

The design optimization procedure for limiting performance-based seismic drifts of an RC building structure is listed as follows:

1. Establish an initial design with optimal member dimensions, which can be obtained from the elastic seismic design optimization by minimizing the concrete cost of an RC structure subjected to a minor earthquake loading using the elastic response spectrum analysis method **(Zou XK. 2002)**.
2. Determine the design spectra, corresponding to different earthquake demand levels, that will be used in the nonlinear pushover analysis.
3. Conduct a static virtual load analysis to obtain the member internal forces that will be used in formulating inelastic drift responses by employing the principle of virtual work.
4. On the basis of the optimal member size, determine the minimum and maximum size bounds of the steel reinforcement ratios, ρi and $\rho^1 i$, in accordance with the strength-based code requirements.
5. Apply the initial pre processor on the basis of a representative single drift constraint to establish a reasonable starting set of steel reinforcement design variables for the multiple drift constrained optimization.
6. Carry out the nonlinear pushover analysis using commercially available software such as the SAP2000 software **(CSI. 2000)** to determine the performance point of the structure and the associated inelastic drift responses of the structure at the performance point.
7. Track down the locations of the plastic hinges, establish the instantaneous lower and upper bound move limits of ρi for those members with plastic hinges and determine the values of the first-order and second order derivatives of the drift response.
8. Establish the explicit interstory drift constraints using a second-order Taylor series approximation and formulate the explicit design problem.
9. Apply the recursive OC optimization algorithm using to resize all steel reinforcement design variables and to identify the active inelastic drift constraints.
10. Check convergence of the steel cost and the inelastic drift performance of the structure. Terminate with the optimum design if the solution convergence is found; otherwise, return to Step 6.

It has been demonstrared that steel reinforcement plays a significant role in controlling the lateral drift beyond first yielding and in providing ductility to an RC building framework.

Using the principle of virtual work and the Taylor series approximation, the inelastic performance-based seismic design problem has been explicitly expressed in terms of the steel reinforcement design variables. Axial moment hinges and moment hinges should be considered in the nonlinear pushover analysis of a frame structure so that the behavior of columns and beams can be effectively modelled. Also this optimality criteria design method developed is able to automatically shift any initial performance point to achieve the final optimal performance point. It is also believed that this optimisation methodology provides a powerful computer- based technique for performance-based design of multistorey RC building structures.

A. K. Chopra (2001) extracted an improved Direct Displacemnt-Based Design Procedure for Performance-Based seismic design of structures. Direct displacement-based design requires a simplified procedure to estimate the seismic deformation of an inelastic SDF system, representing the first (elastic) mode of vibration of the structure. This step is usually accomplished by analysis of an "equivalent" linear system using elastic design spectra. In their work, an equally simple procedure is developed that is based on the well-known concepts of inelastic design spectra. This procedure provides: (1) accurate values of displacement and ductility demands, and (2) a structural design that satisfies the design criteria for allowable plastic rotation. In contrast, the existing procedure using elastic design spectra for equivalent linear systems is shown to underestimate significantly the displacement and ductility demands.

In this work, it is demonstrated that the deformation and ductility factor that are estimated in designing the structure by this procedure are much smaller than the deformation and ductility demands determined by nonlinear analysis of the system using inelastic design spectra. Furthermore, it has been shown that the plastic rotation demand on structures designed by this procedure may exceed the acceptable value of the plastic rotation.

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. Invariant load patterns can provide adequate predictions if the structural response is not severely affected by higher modes and the structure has only a single load yielding mechanism that can be captured by an invariant load pattern.

FEMA-273 recommends utilising at least two fixed load patterns that form upper and lower bounds for inertia force distributions to predict likely variations on overall structural behavior and local demands. The first pattern should be uniform load distribution and the other should be "code" profile or multi-modal load pattern. The 'Code' lateral load pattern is allowed if more than 75% of the total mass participates in the fundamental load. The invariant load patterns cannot account for the redistribution of inertia forces due to progressive yielding and resulting changes in dynamic properties of the structure. Also, fixed load patterns have limited capability to predict higher mode effects in post-elastic range. 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 VARIOUS HINGE MODELS OF PUSHOVER ANALYSIS

These are the various hinge models used in pushover analysis:

According to the **Ceroni et., al, (2007)** the **rotational capacity of the element** can be defined as the plastic fraction θp of the rotation θu at failure. It can be evaluated as the difference between the rotation at the maximum moment and the rotation at the steel yielding θy:

$$\theta p = \theta u - \theta y \qquad 3.1$$

The plastic rotation must include the contribution of the fixed end rotation $\theta p,fix$,

$$\theta p = \theta p,c - \theta p,fix \qquad 3.2$$

The fixed end rotation θp,fix, is evaluated as the ratio between the slip of the tensile bars at the column base and the neutral axis depth of the base section. The value of θp,fix depends on all the parameters introduced, but above all the steel characteristics and the bond-slip relation are important; moreover the bar diameter has to be considered, for its influence on bond. The term θp,c represents the contribute to plastic rotation of column deformability.

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 previously introduced.

The plastic hinge length can be obtained dividing the plastic rotation θp to the plastic curvature Øp:

$$\mathrm{Lp} = \frac{\theta \mathrm{p}}{\emptyset \mathrm{p}} \tag{3.3}$$

$$\emptyset \mathrm{p} = \emptyset \mathrm{u} - \emptyset \mathrm{y} \tag{3.4}$$

$$\theta \mathrm{p} = \theta \mathrm{u} - \theta \mathrm{y} = (\emptyset \mathrm{u} - \emptyset \mathrm{y}) \cdot \mathrm{Lp} \tag{3.5}$$

Due to the fixed end rotation, the Lp value can be divided into two contributions:

$$\mathrm{Lp} = \mathrm{Lp}^{\mathrm{I}} + \mathrm{Lp}^{\mathrm{II}} \tag{3.6}$$

where Lp^{I} is due to the plastic rotation of the column and $\mathrm{Lp}^{\mathrm{II}}$ to the fixed end rotation at the footing zone of the column.

The following expressions for Lp^{I} and $\mathrm{Lp}^{\mathrm{II}}$ have been obtained:

$$L_p{}^I = 6.1 \cdot \left(\frac{L}{H}\right)^{0.43} \cdot \left(\frac{f_t}{f_y} - 1\right)^{0.65} \cdot \varepsilon^{-0.32} \cdot \left(1 + \frac{N}{N_o}\right)^{-1.83} \tag{3.7}$$

$$L_p{}^{II} = 5 \cdot d_b \cdot \left(\frac{f_t}{f_y} - 1\right)^{0.2} \tag{3.8}$$

According to **Priestley et., al**, **(1987)** the plastic hinge length formula is:

$$\mathrm{Lp} = 0.08\mathrm{L} + 6\mathrm{db} \tag{3.9}$$

where L is the distance from the point of contraflexure of the column to the section of maximum moment and db the bars diameter;

According to **B.I.A. 1996,** the plastic hinge length formula is:

$$Lp = 0.08L + 0.022\ fy\ db \qquad 3.10$$

According to **Bulletin of TG7.2**, **(2003)** the formula of plastic hinge length:

$$\text{for monotonic loads: } Lp = 0.18 \cdot Ls + 0.025 \cdot f\,y \cdot db \qquad 3.11$$

$$\text{for cyclic loads: } Lp = 0.08 \cdot Ls + 0.017 \cdot f\,y \cdot db \qquad 3.12$$

where Ls is the shear span.

According to **Bulletin of TG7.2**, **(2003)** the ultimate rotation θu calculated according to the following equation:

$$\theta u = \theta y + (\emptyset u - \emptyset y). Lp. \{1 - \frac{0.5.Lp}{Ls}\} \qquad 3.13$$

The ultimate and yielding curvatures were calculated using the section equilibrium equations and considering a constitutive relationship for the confined concrete. Rotation at steel yielding, θy, was calculated through an empirical expression statistically fitted to the experimental results on beams, columns and walls.

According to **Priestley et., al**, **(1996)** the ultimate concrete compressive strain can be calculated by:

$$\epsilon cu = 0.004 + \frac{1.4\ \rho s\ \text{fyh}\ \epsilon \text{su}}{\text{fcc}} \qquad 3.14$$

where εcu is the ultimate concrete compressive strain, εsu is the steel strain at the maximum tensile stress, ρs is the volumetric ratio of confining steel, fyh is the yield strength of transverse reinforcement, and fcc is the peak confined concrete compressive strength.

3.4 ELEMENT DESCRIPTION OF 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.

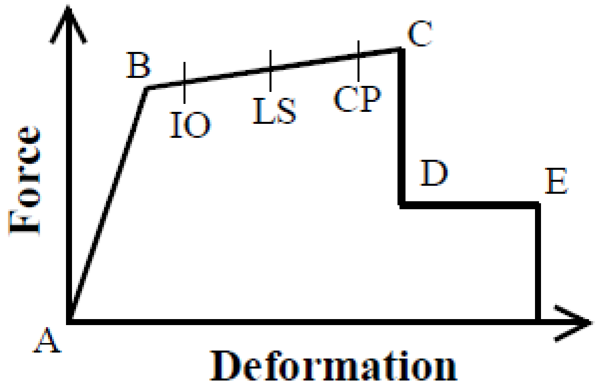

Fig. 3.1 Force-Deformation for Pushover Hinge (Habibullah. et al., 1998)

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 coupledP-M2-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 user-defined. 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)**

3.5 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)

3.5.1 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)**

3.5.2 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)

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

3.6.1 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.

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

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

3.6.4 BUILDING PERFORMANCE LEVEL: The combination of a Structural Performance Level and a Nonstructural Performance Level to form a complete description of an overall damage level.

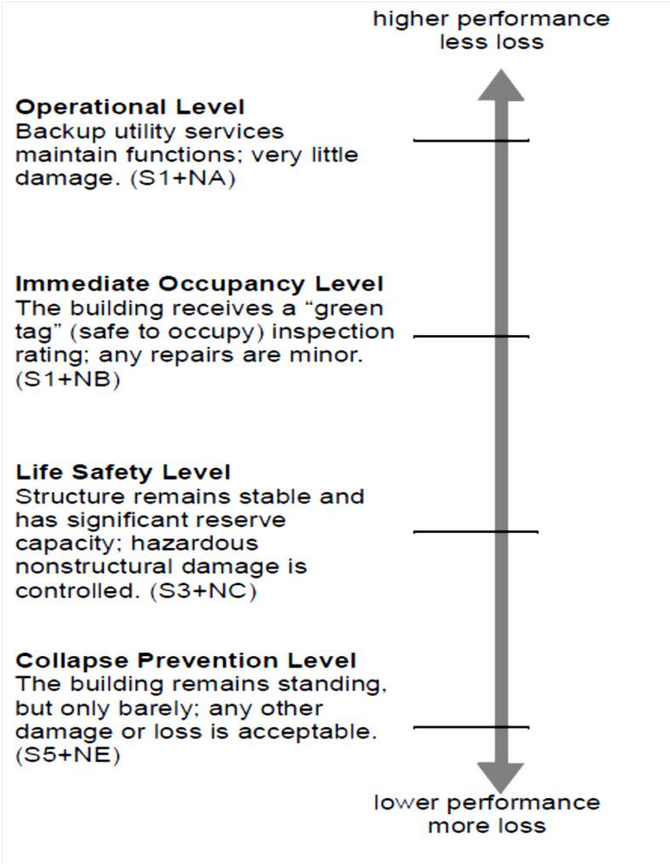

Fig. 3.2 Building Performance Levels (ATC, 1997a)

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 Nonstructural Performance Level that describes the limiting damage state of the nonstructural systems. Three Structural Performance Levels and four Nonstructural Performance Levels are used to form the four basic Building Performance Levels listed above.

Other structural and nonstructural 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 S-6, Structural Performance Not Considered, to cover the situation where only nonstructural improvements are made.

The four Nonstructural 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, Nonstructural 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.

3.7 STRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

3.7.1 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.

3.7.2 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.

3.7.3 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.

3.8 STRUCTURAL PERFORMANCE RANGES (ATC, 1997a)

3.8.1 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.

3.8.2 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.

3.9 NONSTRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

3.9.1 OPERATIONAL PERFORMANCE LEVEL (N-A)

Nonstructural Performance Level A, Operational, means the post-earthquake damage state of the building in which the nonstructural components are able to support the building's intended function. At this level, most nonstructural 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.

3.9.2 IMMEDIATE OCCUPANCY LEVEL (N-B)

Nonstructural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited nonstructural 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 non-operable. 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.

3.9.3 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.

3.9.4 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 GENERAL DESCRIPTION OF STRUCTURE

One of the major objectives of this work was to test a real- life structure under pushover loads. In order to keep the structure as close to reality as possible, no special design for the structure as such was performed and instead a portion of a real life existing office building was selected. Thus the structure tested in this work was a replica of a part of an existing office building. The portion was deliberately selected so that it had certain eccentricities and was un-symmetric in plan (Fig. 4.1). Also the column sizes and sections were varied along the storey as in the case of original real life structure. **(reddy. et., 2010)**

Although the geometry of the structure tested in this work was kept same as the portion of the original structure, there were few major differences in the reinforcement detailing as mentioned below:

1. Although the original structure was detailed according to new conforming seismic detailing practice as per IS 13920 (BIS, 1993), the structure for the experiment followed the non-seismic detailing practice as per IS 456 (BIS 2000). The reason for this is the fact that pushover analysis is mostly used for retrofit of old structures, which have not followed the seismic detailing practice. Consequently, special confining reinforcement as recommended by IS 13920(BIS, 1993) was not provided. Also no shear reinforcement in the beam- column joints was provided.
2. Since the structure tested is replica of a small portion of the large original structure, the continuous reinforcement in the slab and beams were suitably modified to fit as per the requirement.
3. Another major difference is in the foundation system. In order to avoid any nonlinear behavior of the foundation, a raft foundation with a number of rock anchors were provided.

4.1.1 MATERIAL PROPERTIES

The material used for construction is Reinforced concrete with M-20 grade concrete and fe-415 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, Es = 21,0000 MPa

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

Characteristic strength of concrete, fck = 20 MPa

Yield stress for steel, fy = 415 MPa

Ultimate strain in bending, ε_{cu} =0.0035

4.1.2 MODEL GEOMETRY

The structure analyzed is a four-storied, one 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 = 4

Number of bays along X-direction =1

Number of bays along Y-direction = 1

Storey height = 4.0 meters

Bay width along X-direction = 5.0 meters

Bay width along Y-direction = 5.0 meters

4.1.3. PLAN OF BUILDING

The plan of the building is shown in the Fig. 4.1 on next page. The bay width, column positions and beams positions can be seen on next page:

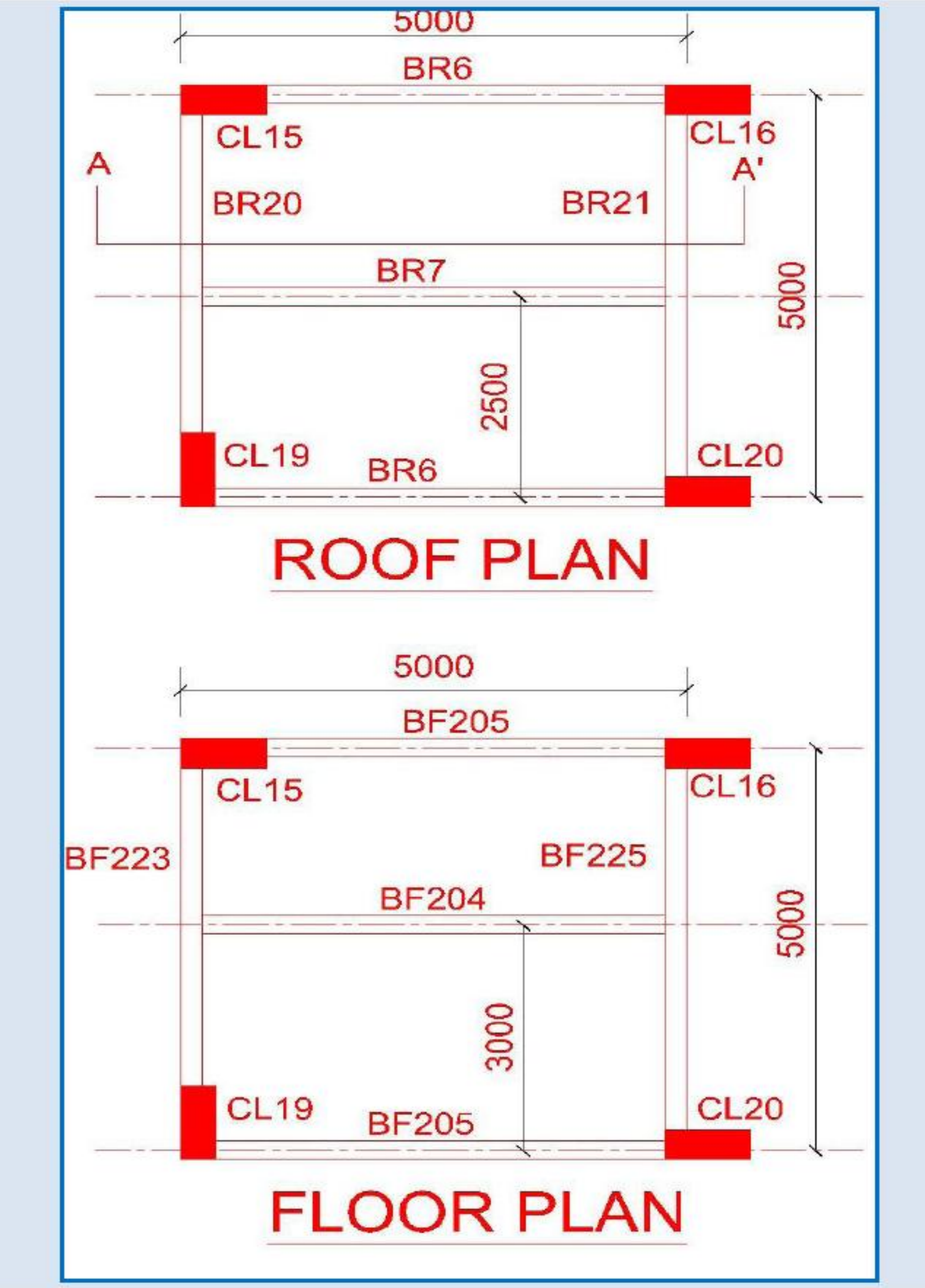

Fig. 4.1 Roof Plan and Floor Plan of Structure (Reddy. et., al, 2010)

4.1.4 ELEVATION OF BUILDING

The Figure 4.2 shows the sectional elevation of the structure. The storey heights, column lines, description of slabs etc. can be seen in this picture.

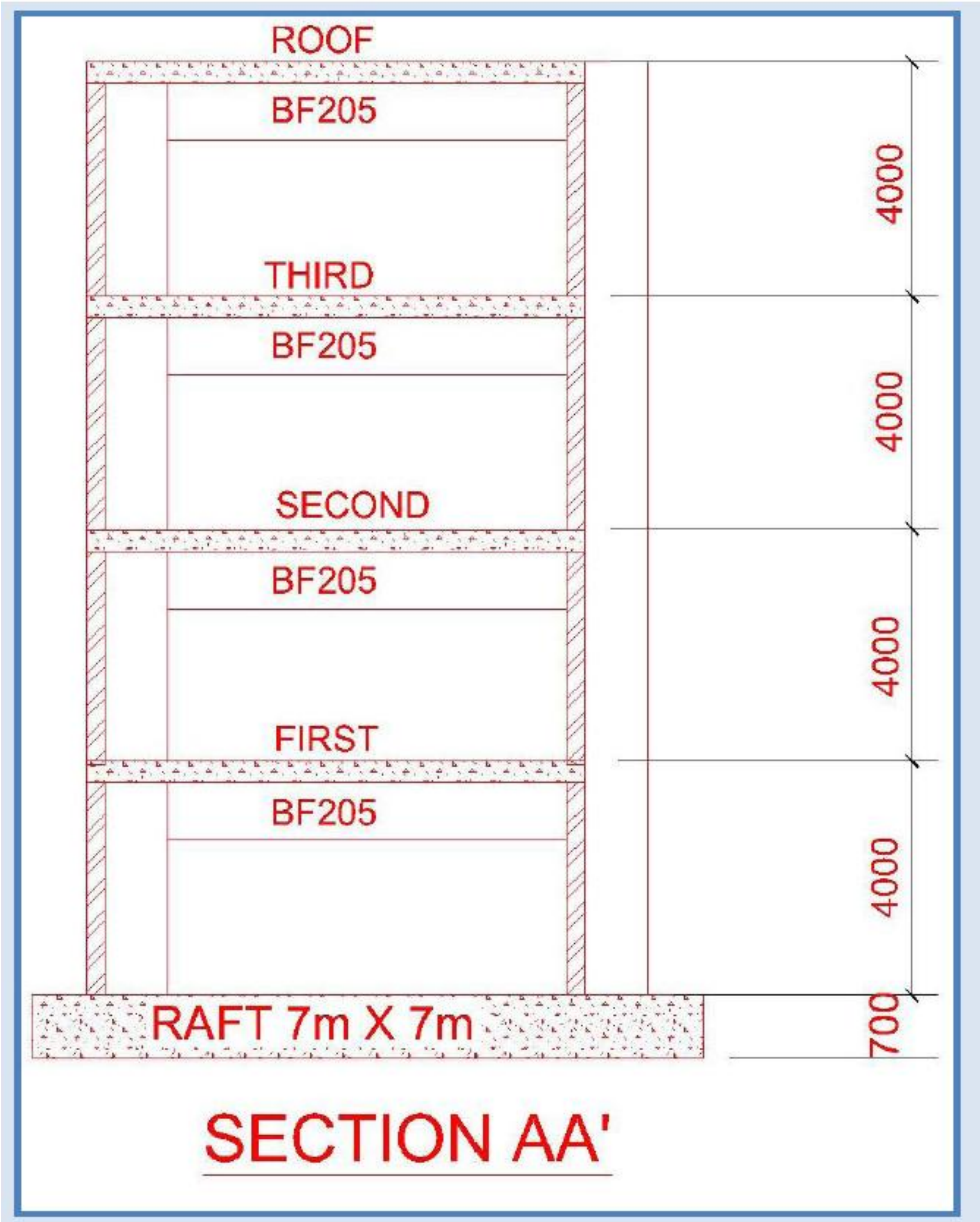

Fig. 4.2 Sectional Elevation of the Structure (Reddy. et., al, 2010)

4.1.5 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:

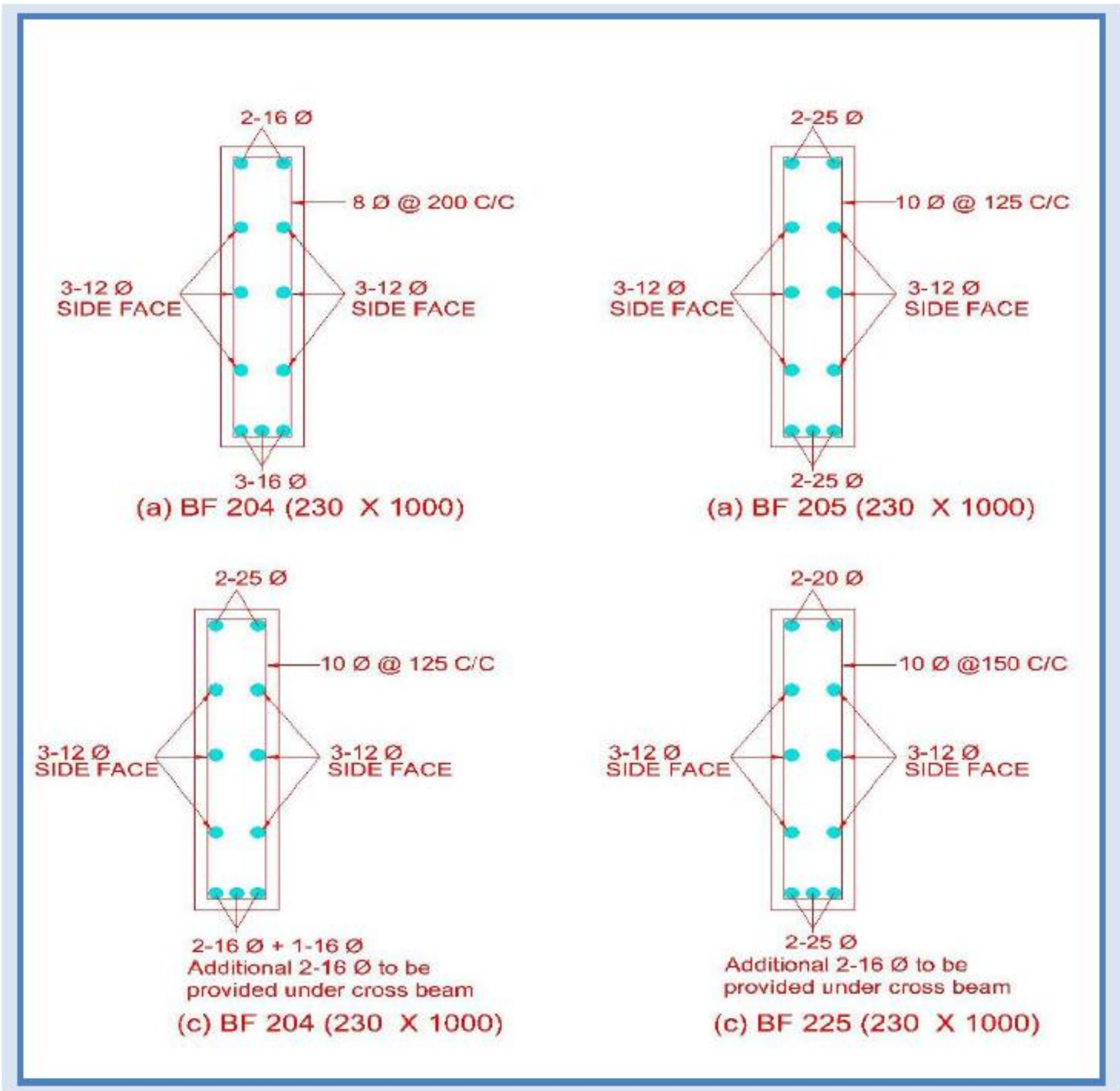

Fig. 4.3 Detail of Floor Beams (Reddy. et., al, 2010)

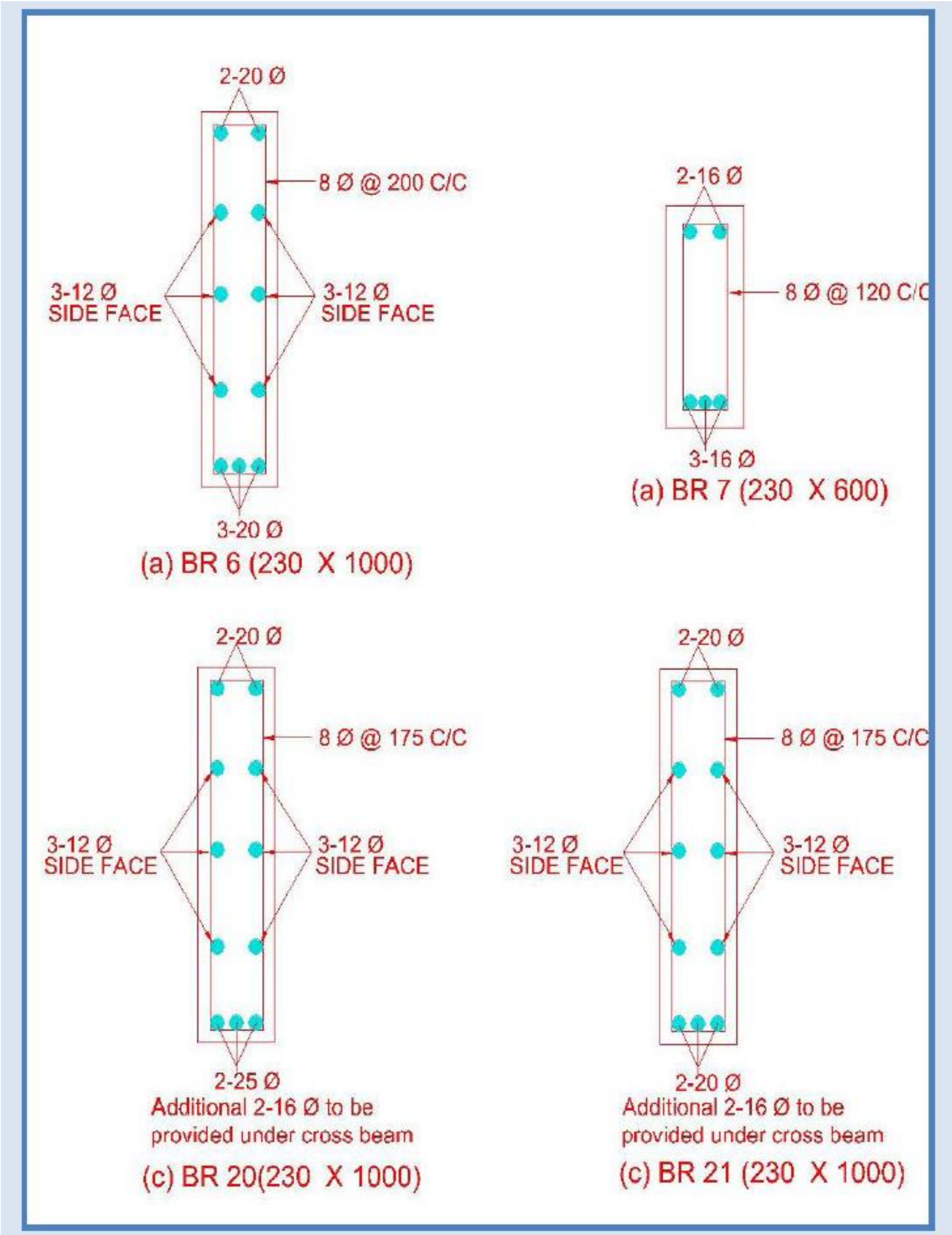

Fig. 4.4 Detail of Roof Beams (Reddy. et., al, 2010)

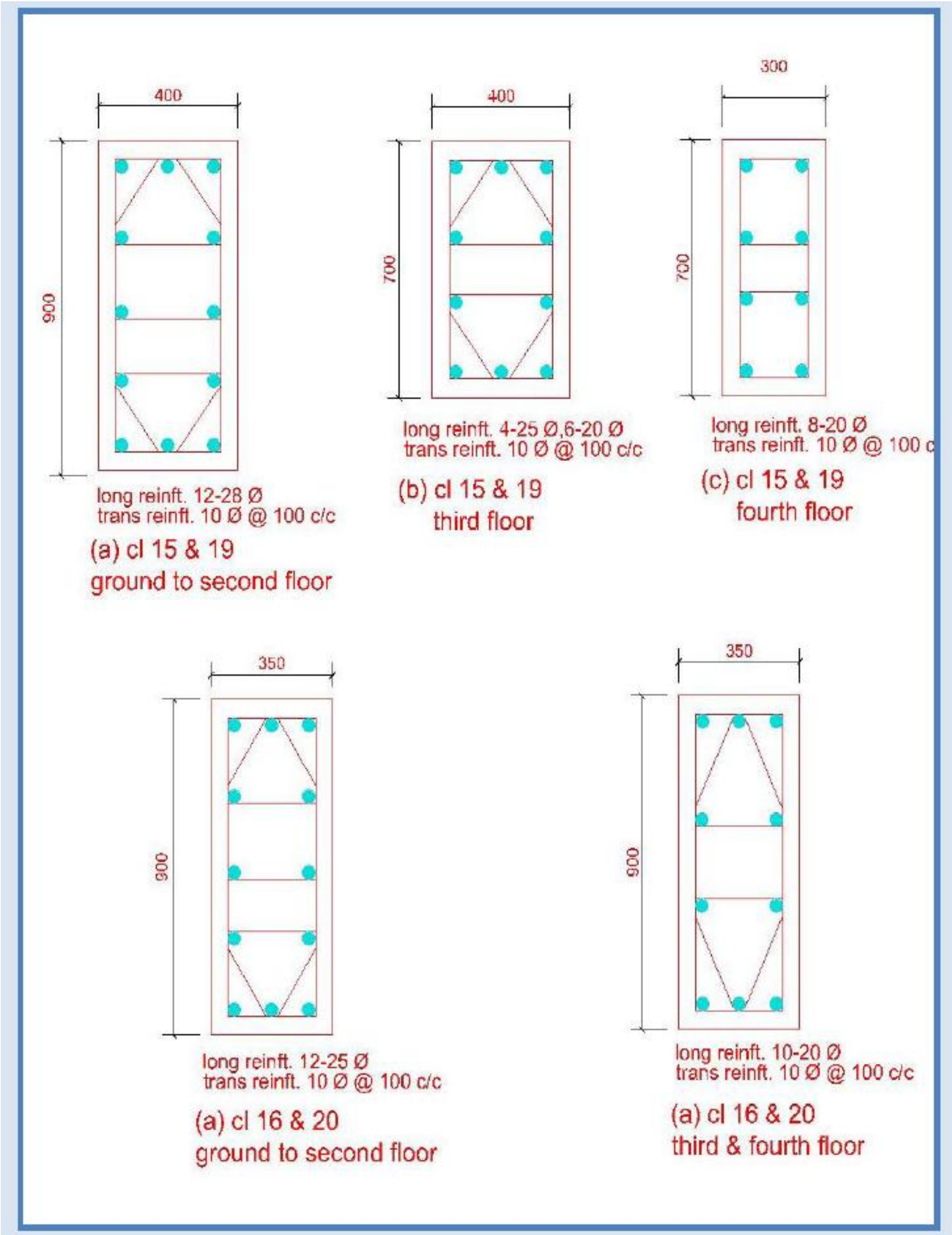

Fig. 4.5 Detail of Columns (Reddy. et., al, 2010)

4.1.6 REINFORCEMENT DETAILS

Detail reinforcement provided is as given in Table 4.1

Identification	B	D	Top Cover	Bottom Cover	Top steel nob-dia	Bottom steel stirrups	Dia of stirrups	Spacing of stirrups
BR6M	230	1000	35	35	2-20	3-20	8	200
BR6L	230	1000	35	35	3-20	3-20	8	200
BR6R	230	100	35	35	5-20	3-20	8	200
BR7MR	230	600	33	33	2-16	3-16	8	120
BR7L	230	600	33	33	2-16+1-20	3-16	8	120
BR20M	230	1000	35	37.5	2-20	2-25	8	175
BR20LR	230	1000	35	37.5	4-20	2-25	8	175
BR21M	230	1000	35	35	2-20	2-20	8	175
BR21LR	230	1000	35	35	3-20	2-20	8	175
BF204MR	230	1000	33	33	2-16	3-16	8	200
BF204L	230	1000	33	33	1-16+1-20	3-16	8	200
BF205M	230	1000	37.5	37.5	2-25	2-25	10	125
BF205L	230	1000	37.5	37.5	2-15+2-12	2-25	10	125
BF205R	230	1000	37.5	37.5	4-25	2-25	10	125
BF223M	230	1000	37.5	37.5	2-25	2-25+1-16	10	125
BF223LF	230	1000	37.5	37.5	4-25	2-25+1-16	10	125
BF225M	230	1000	35	37.5	2-20	2-25	10	150
BF225LR	230	1000	35	37.5	4-20	2-25	10	150
C15-G-2	400	900	54	54	12-28		10	100
C15-G-3	400	700	52.5	52.5	4-25+6-20		10	100
C15-G-R	300	700	50	50	8-20		10	100
C1620-G-2	350	900	52.5	52.5	12-25		10	100
C1620-G-R	350	900	50	50	10-20		10	100
C19-G-2	900	400	54	54	12-28		10	100
C19-G-3	700	400	52.5	52.5	4-25+6-20		10	100
C19-G-R	700	300	50	50	8-20		10	100

Table 4.1 Detail of Reinforcement (Reddy. et., al, 2010)

4.2 STEP BY STEP PROCEDURE OF ANALYSIS IN SAP2000

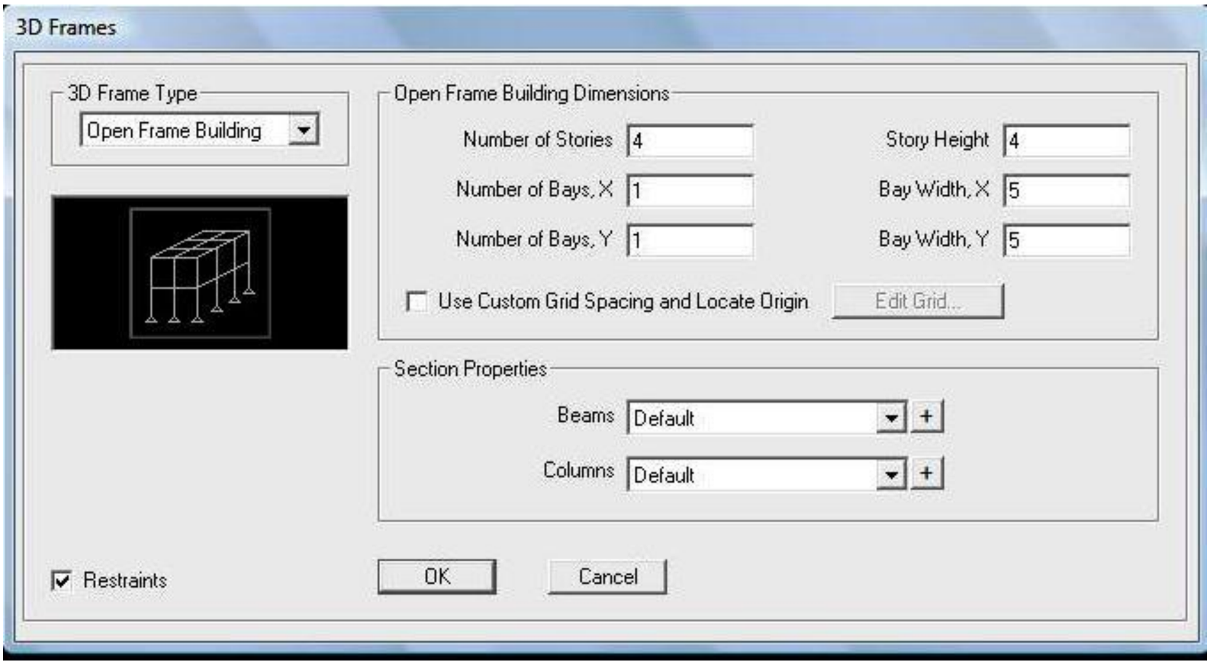

Fig. 4.6 Basic Dimensions of a Building

In the above figure, providing the basic dimensions of a building in SAP2000.

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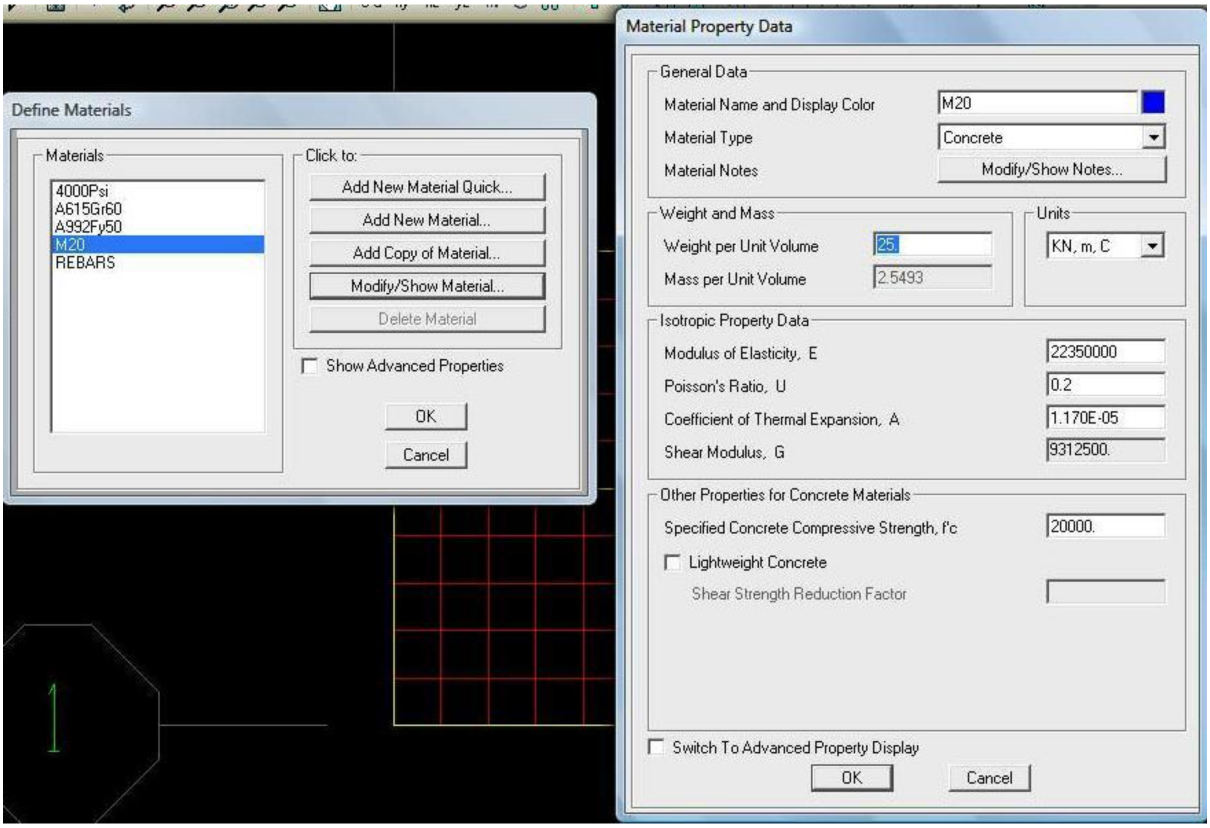

Fig. 4.7 Material Properties of a Building

Here, providing the material properties of a building by using the M20 grade of concrete so inserting the concrete weight, modulus of elasticity and other important things are also inserted in this figure.

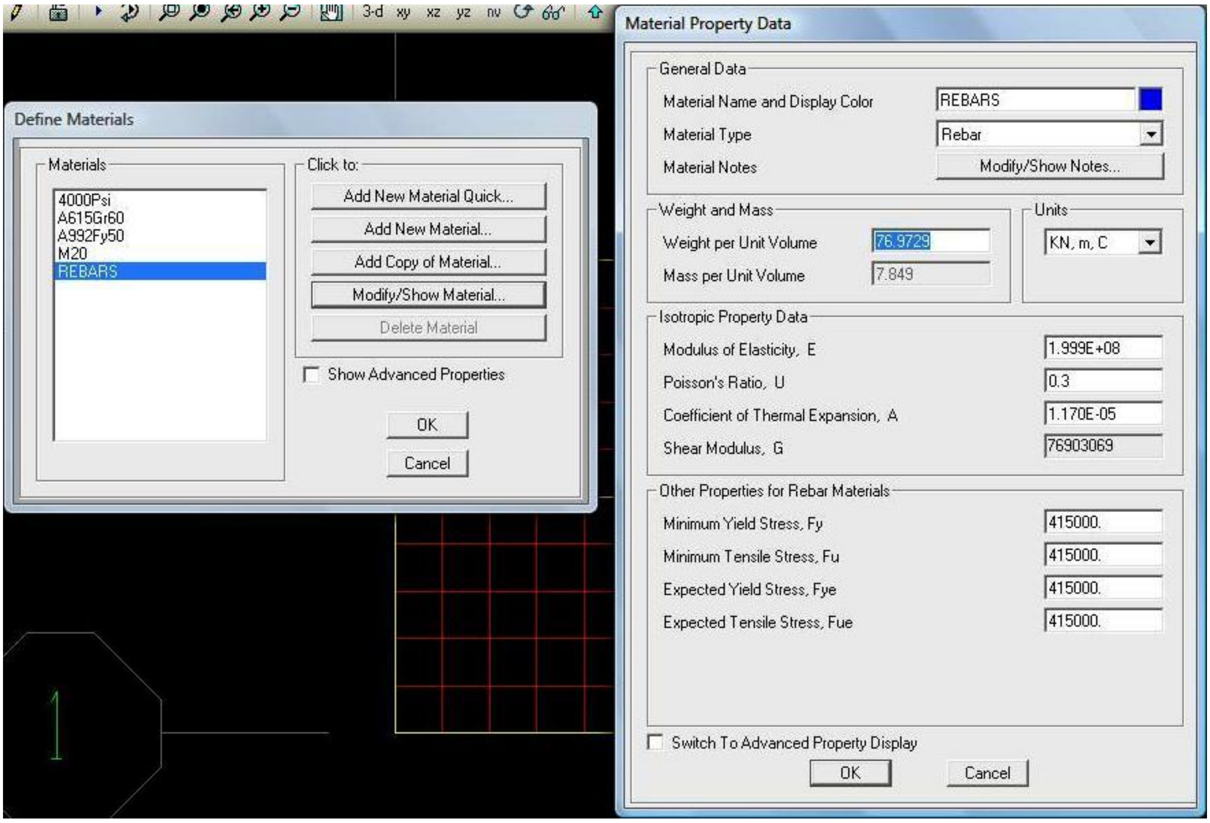

Fig. 4.8 Material Properties of a Building

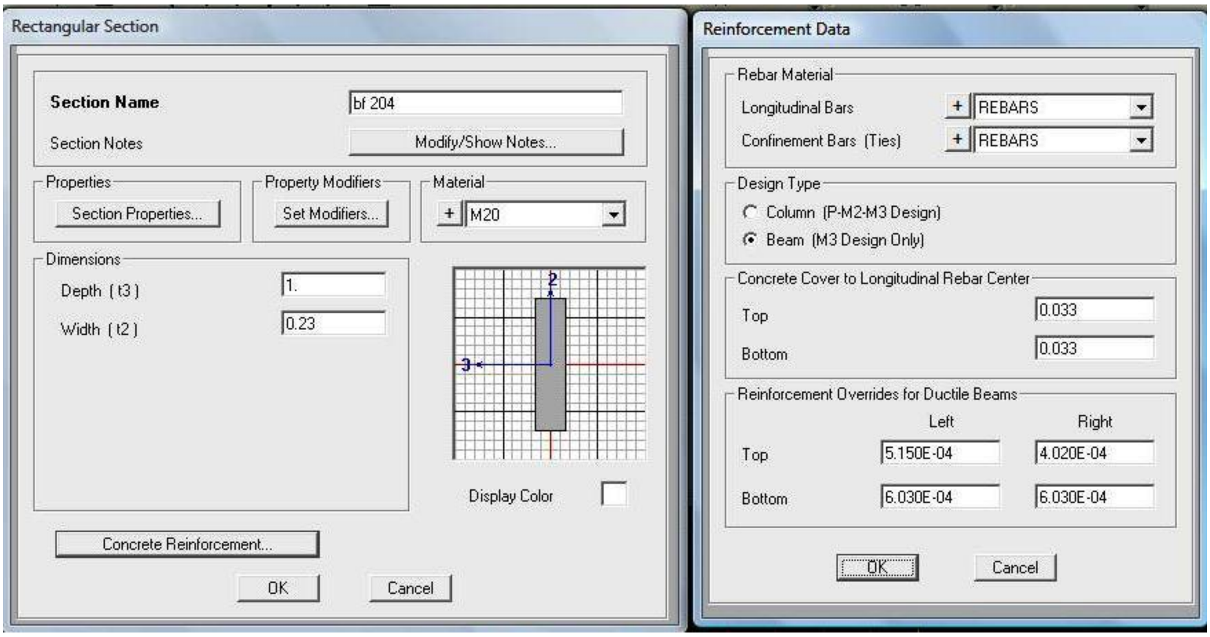

Fig. 4.9 Basic Dimensions of a Beam

In this figure, defining a beam and its dimensions, clear cover and grade of concrete. After defining this beam will define all the beams similarly using the same step.

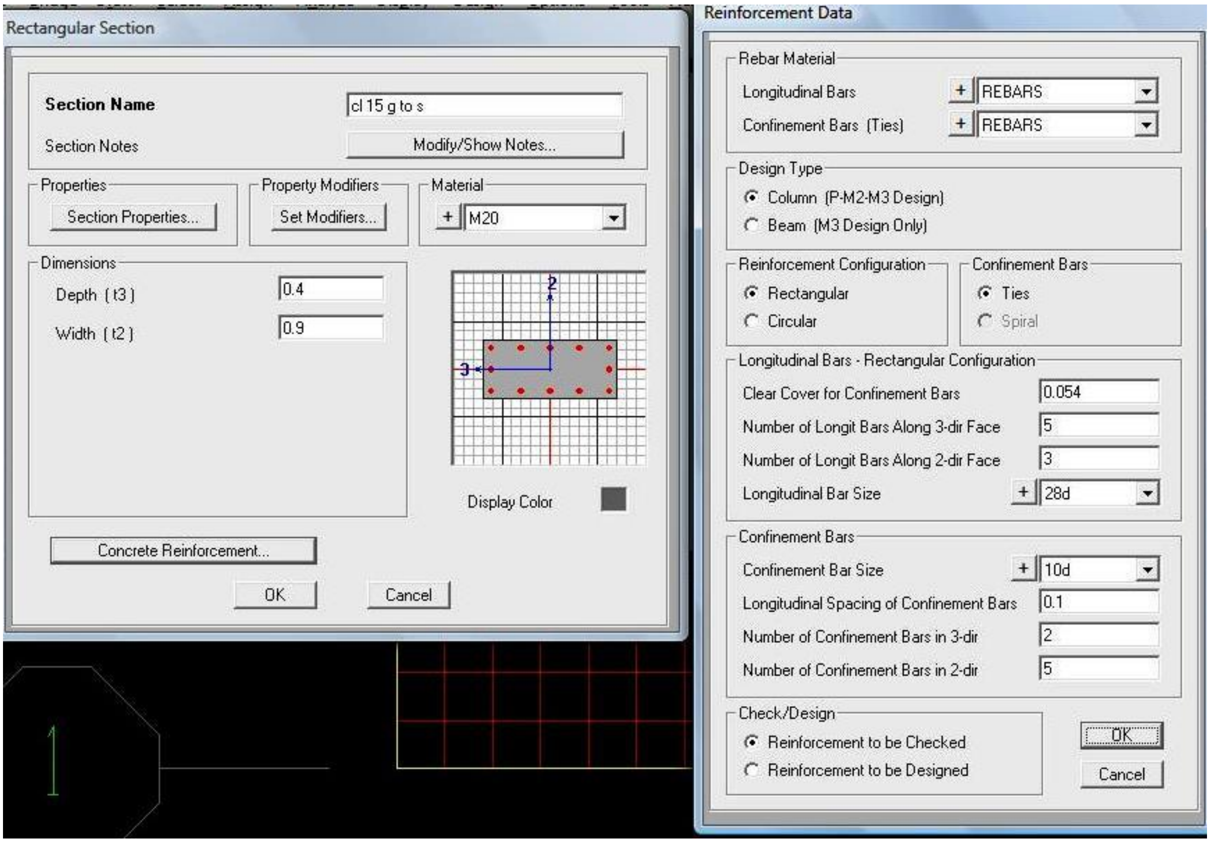

Fig. 4.10 Basic Dimensions of a Column

After defining the dimensions of a beam now define the basic dimensions of a Column. Firstly, give the section name then select the material used. After this, provide the dimensions (depth & width). Then click on concrete reinforcement option, in this, insert the dia. of longitudinal & confinement bars.

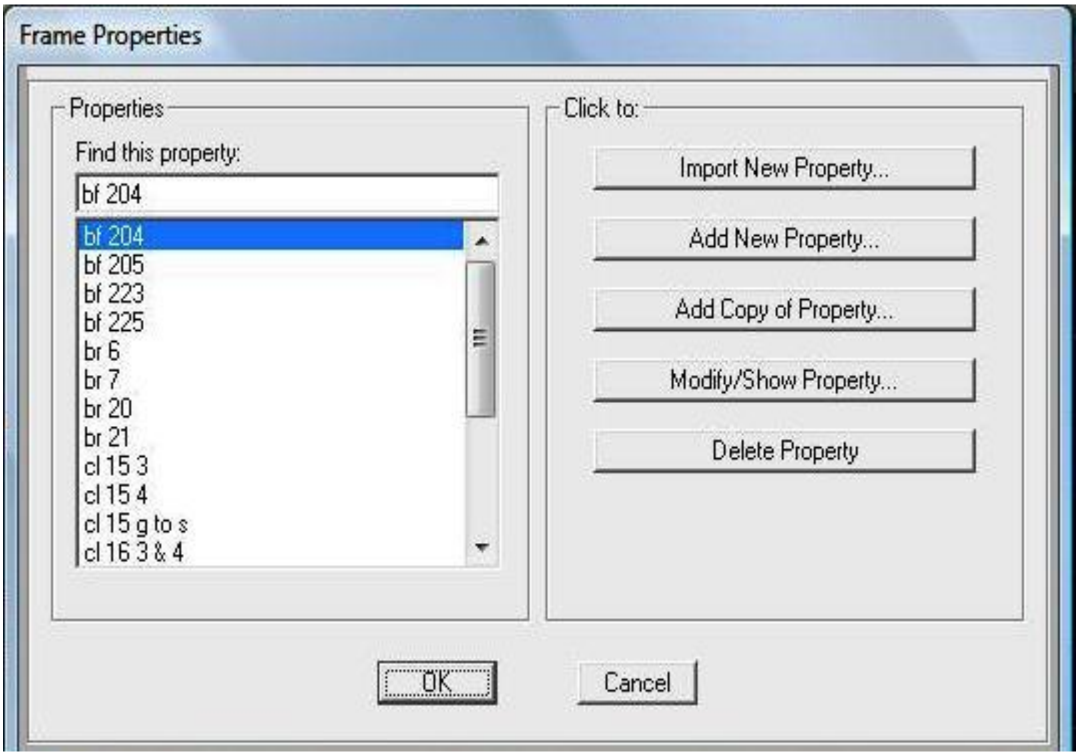

Fig. 4.11 Total no. of Columns & Beams

After defining dimensions of beams & columns in the previous figures individually. In this figure inserting all the beams & columns dimensions. And in last, assign all the beams & columns in the structure.

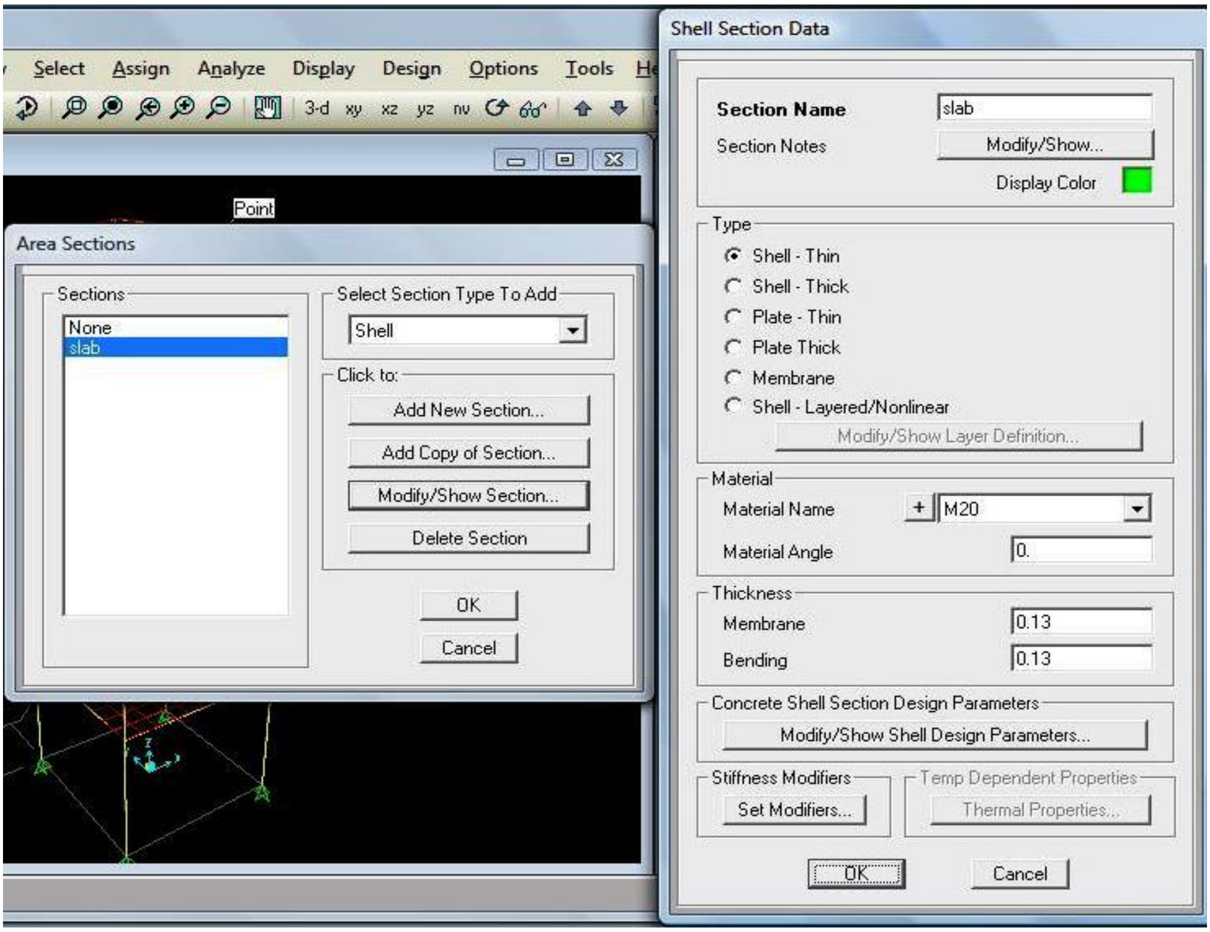

Fig. 4.12 Basic Dimensions of a Slab

Here, defining the dimensions of a slab. Firstly, select the slab type then select the material properties and then provide the thickness of the slab.

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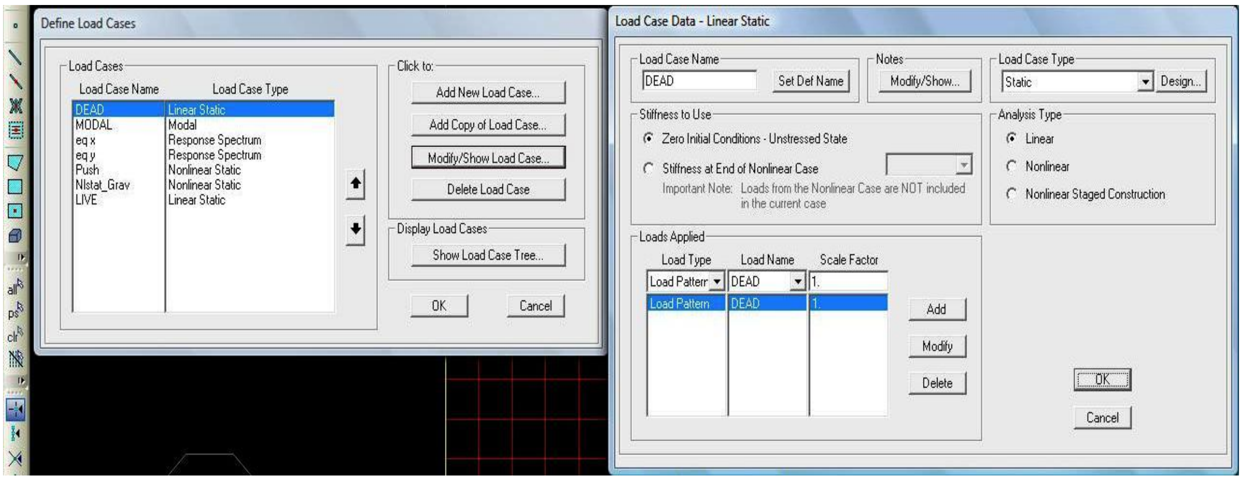

Fig. 4.13 Dimensions of Dead Load

Now here providing the dead load on a building. In the starting click on the new load case button and then change the load case name after this select load type and other parameters like load application.

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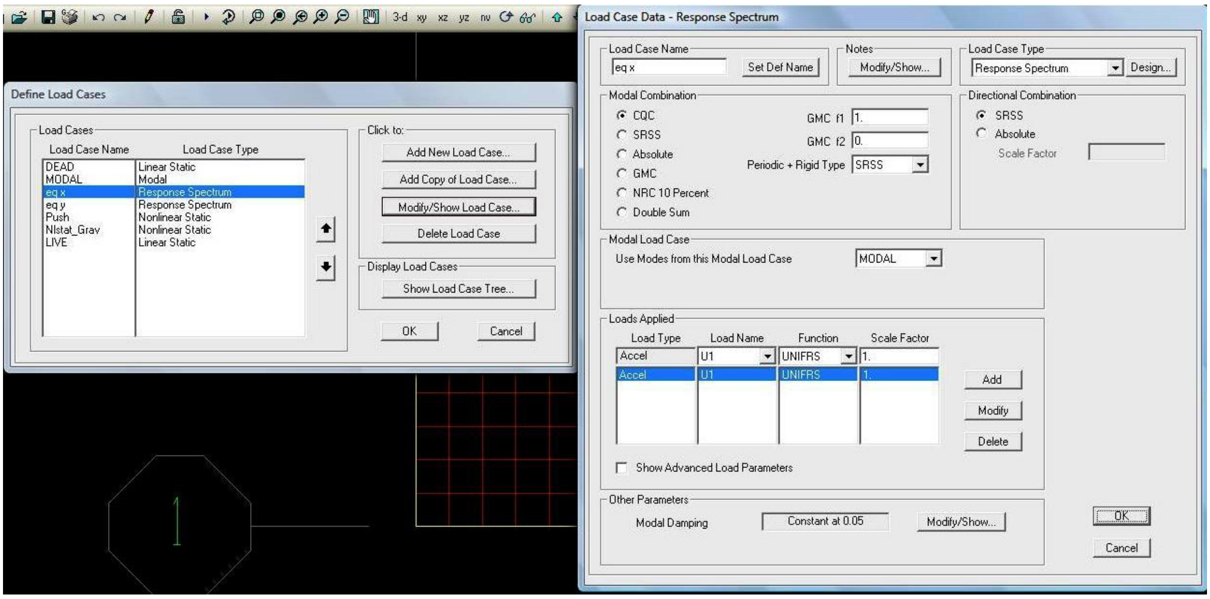


Fig. 4.14 Dimensions of Earthquake Load

Once upload the dead & live load on a structure, now apply the earthquake load. In this the first step, to select the add new load case option then give all the parameters.

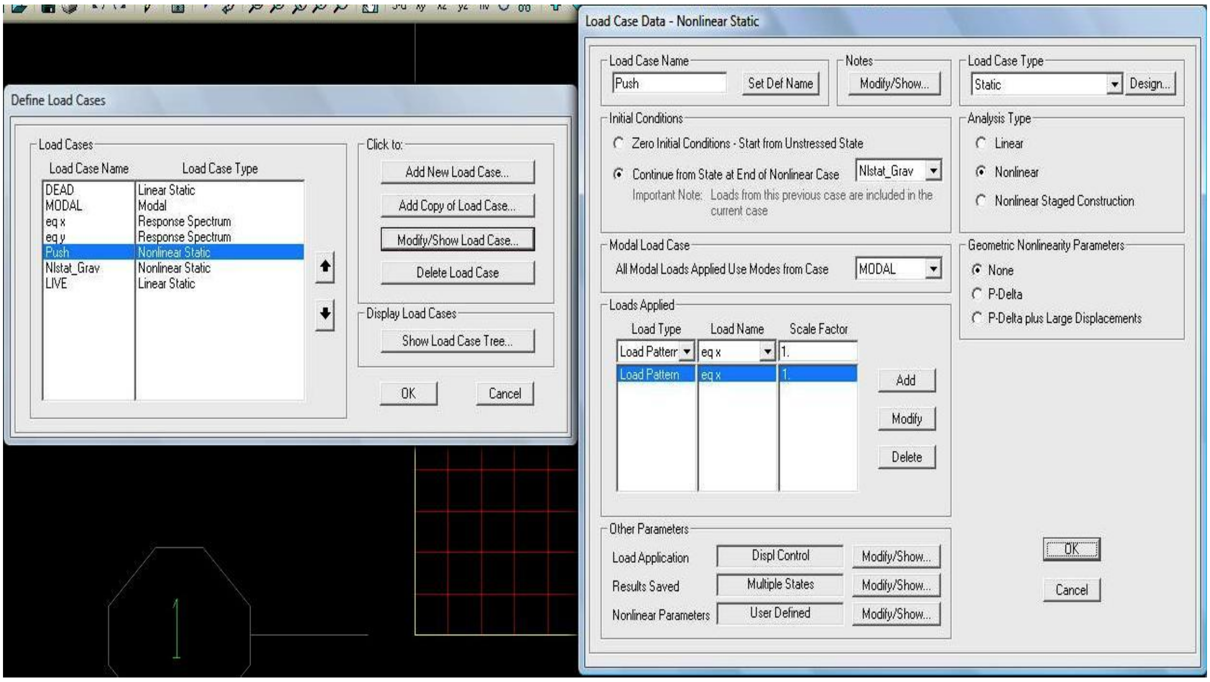


Fig. 4.15 Dimension of Pushover Load

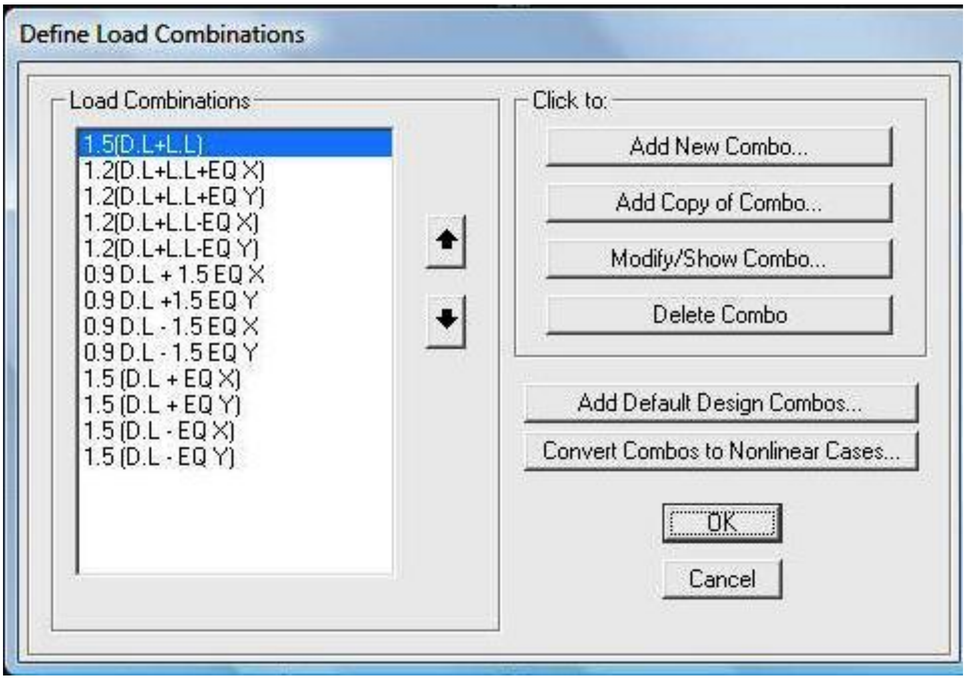


Fig : 4.16 Load Combinations

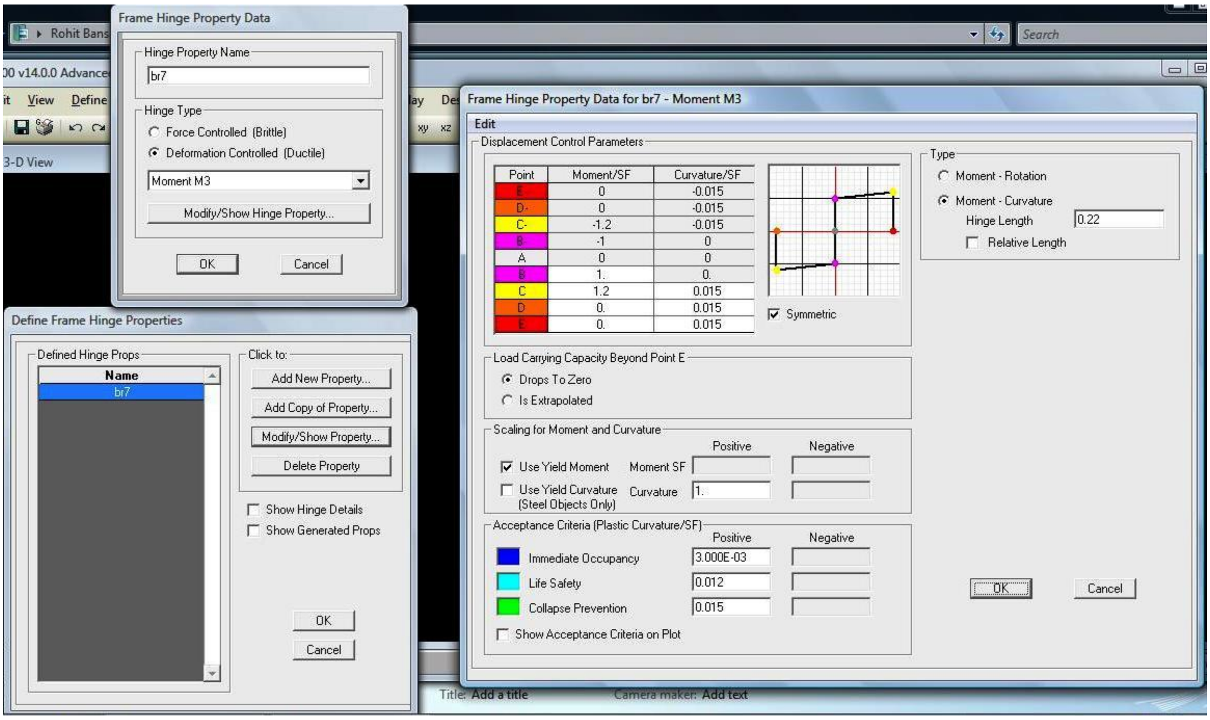


Fig. 4.17 Plastic Hinges Properties on a Beam

In the above providing the user-defined hinge properties on a beam. In the first select the beam number by giving Moment-M3. In the next step provide the hinge length, moment & curvature.

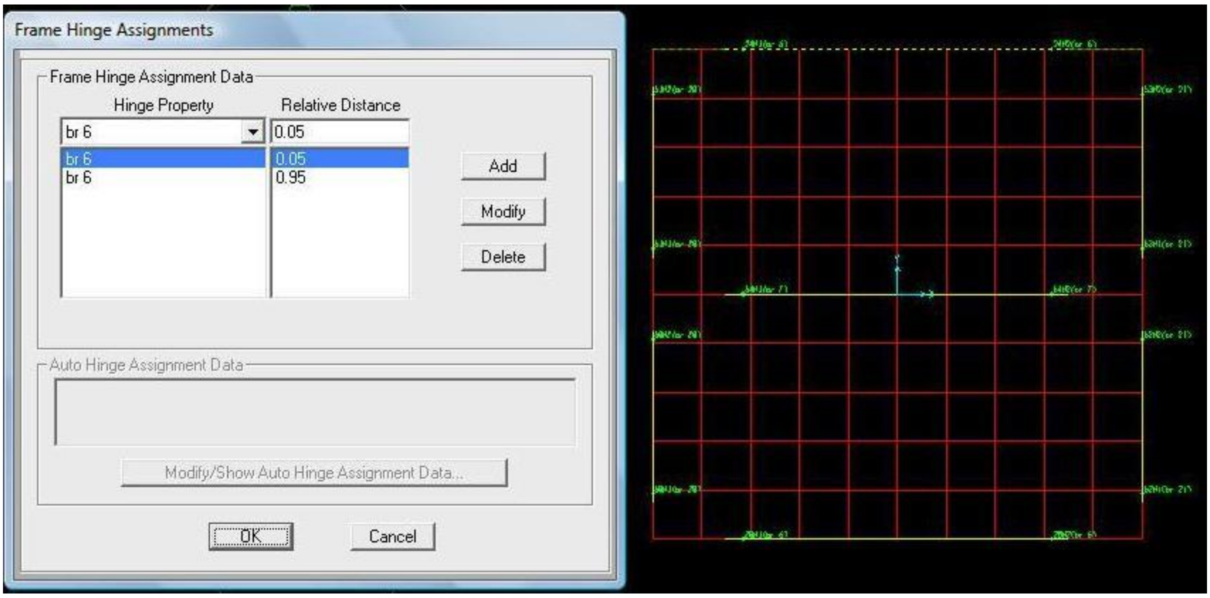


Fig. 4.18 Assign Hinges on a Beam

Here assign the hinge properties on a beam with the different relative distances and then apply on the beam.

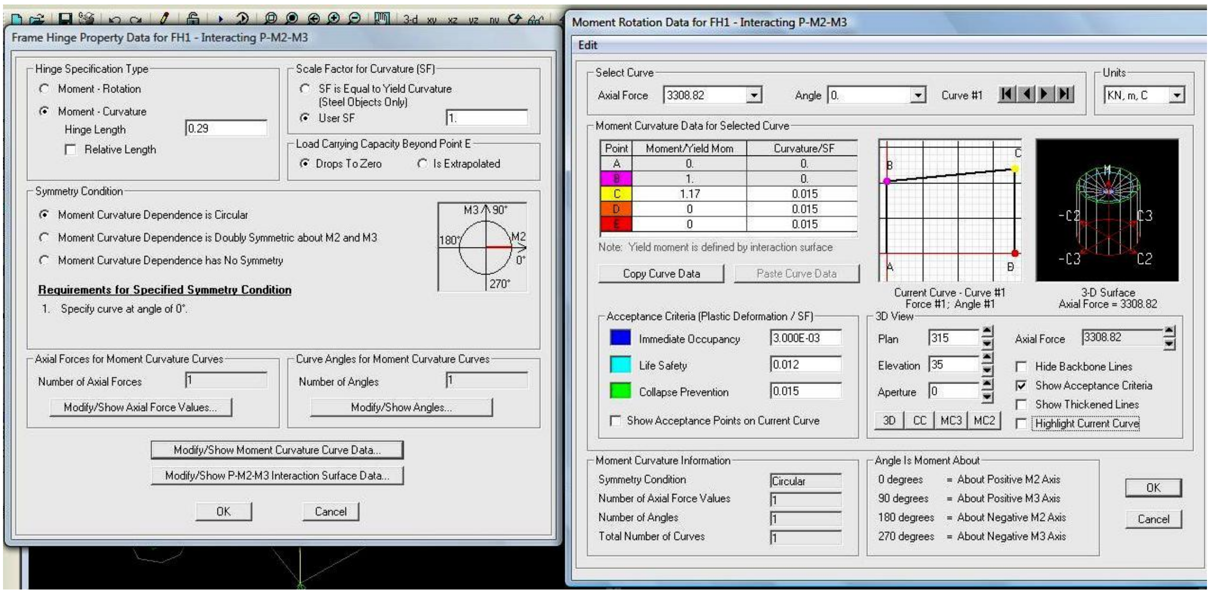


Fig. 4.19 Plastic Hinges Properties on a Column

In this figure, providing the user-defined hinge properties on a column. In the first step select the column number by giving interacting Axial Load (P), Moment-M2 & Moment-M3. In next step provide the hinge length, moment & curvature.

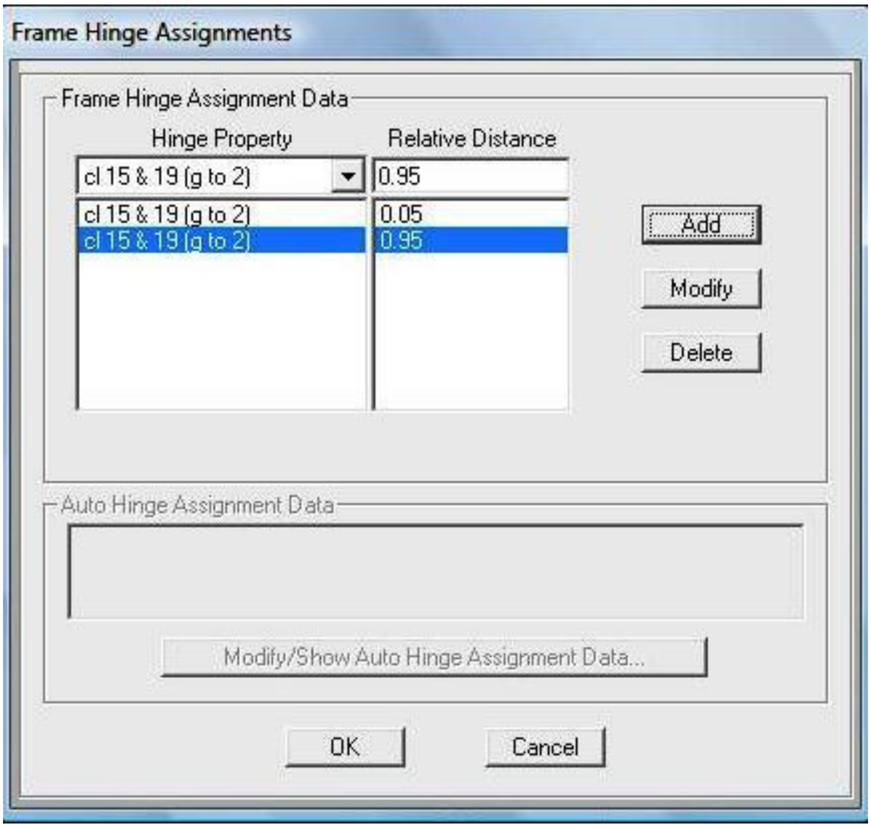


Fig. 4.20 Assign Hinges on a Column

Here assign the hinge properties on a column with the different relative distances and then apply on the column.

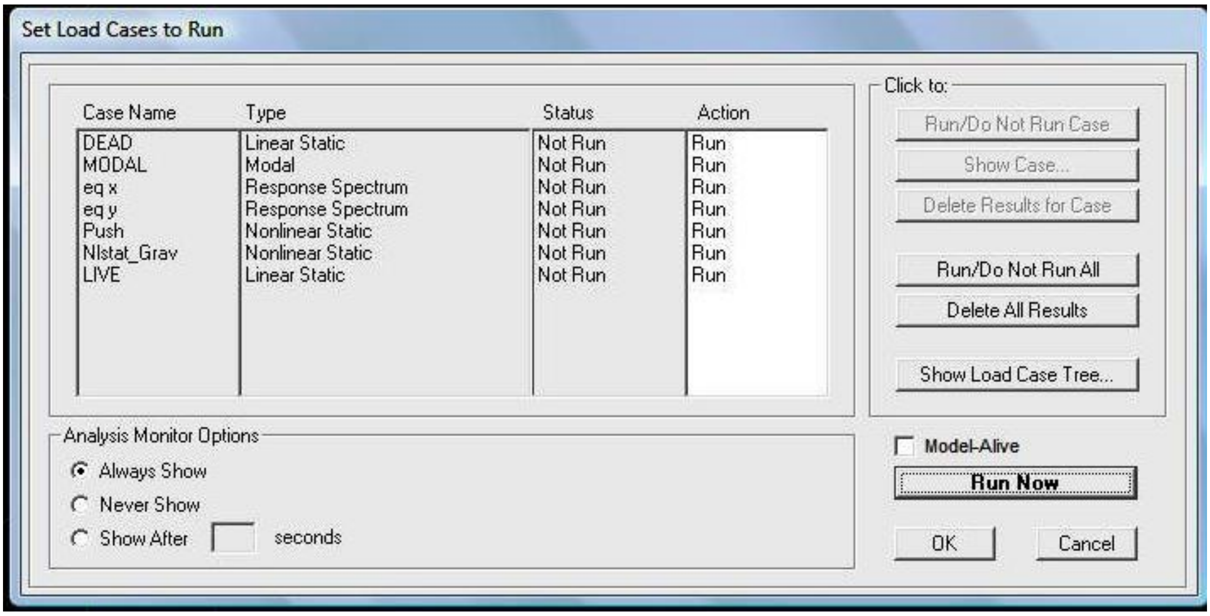


Fig. 4.21 Run Analysis

After putting all the values, at last run the analysis.

CHAPTER – 5

RESULTS AND DISCUSSIONS

5.1 GENERAL

This chapter presents the results of Analysis of RCC frame. Analysis of RCC frame under the static loads has been performed using SAP2000 software. Subsequently these results are compared with experimental results of "Round Robin Exercise on Experiment and Analysis of Four Storey Full Scale Reinforced concrete Structure under Monotonic Push-over Loads‟. This is followed by load deflection curve.

5.2 ANALYSIS RESULTS OF R.C.C 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 running the analysis in last chapter, Now the pushover curve is obtain as shown in Fig. 5.1. 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. 5.1.

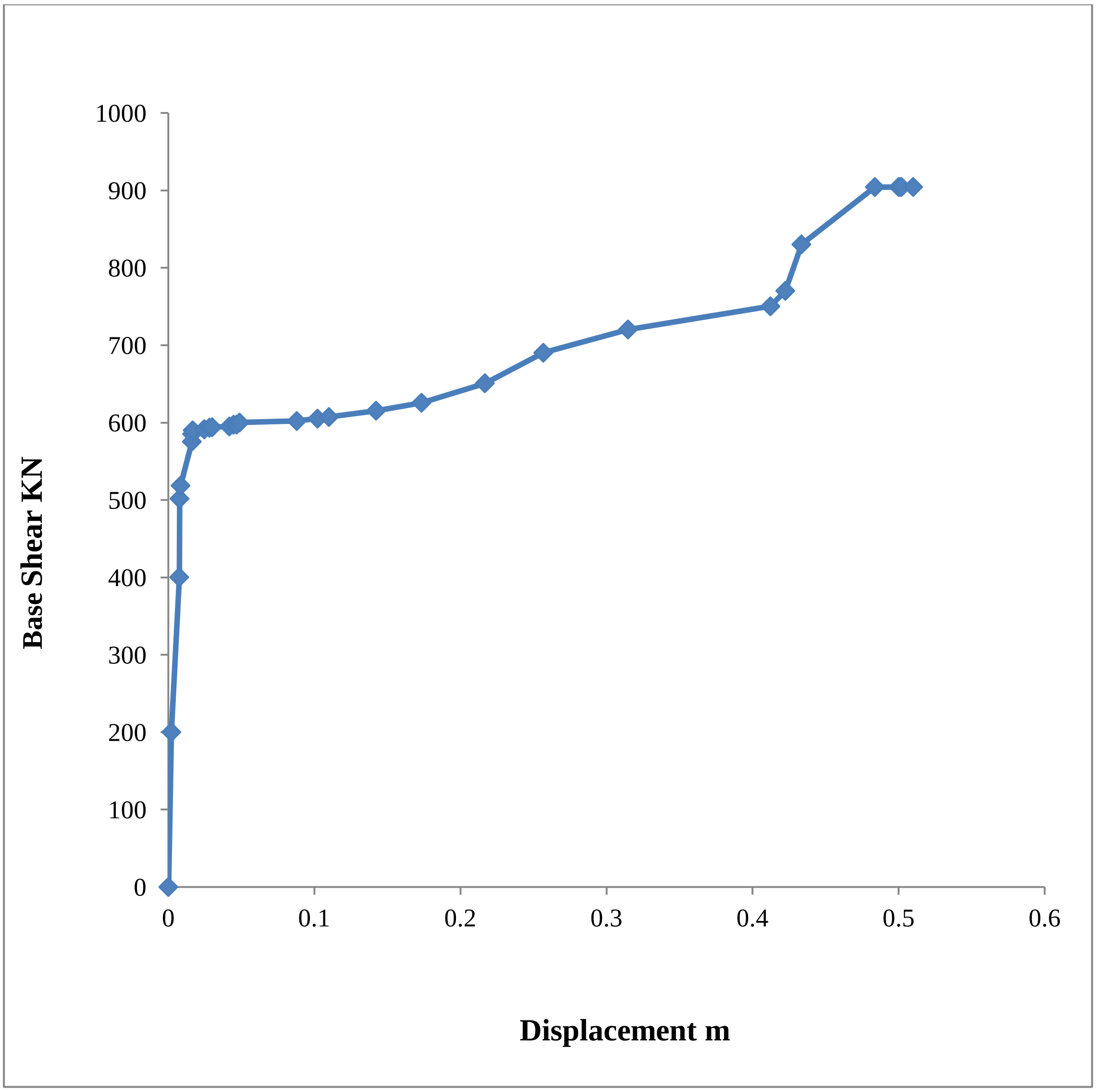


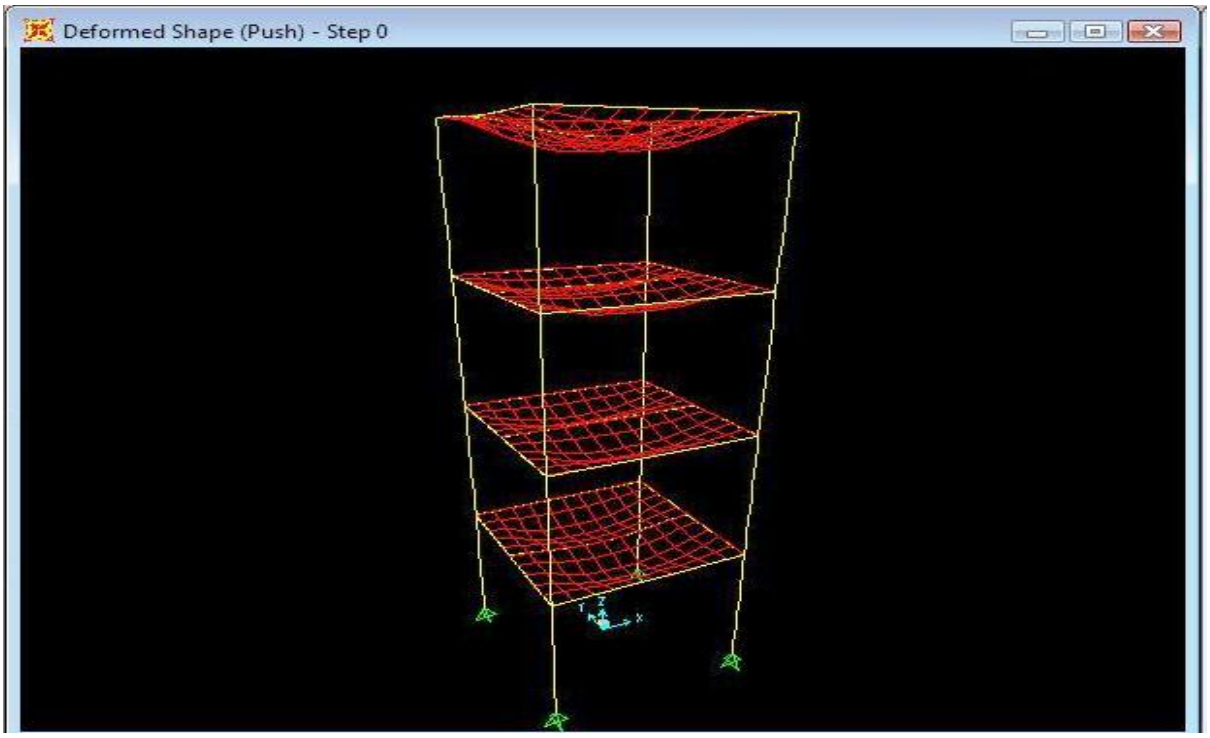
Fig. 5.1 Pushover Curve of a Building

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

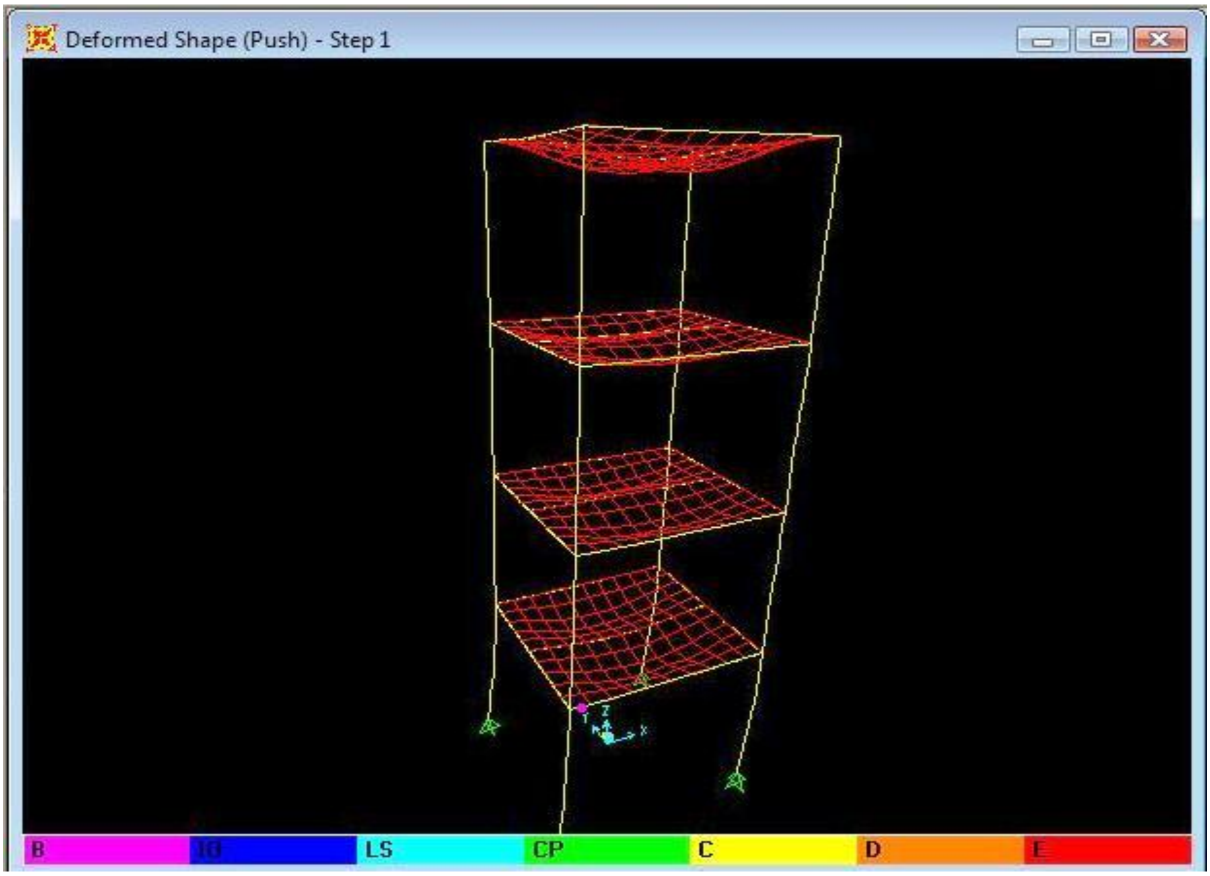
Table 5.1 Tabular Data for Pushover Curve

The pushover displaced shape and sequence of hinge information on a step-by-step basis has been obtained and shown in the Figure 5.3(a) to 5.3(p).

Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The types of output available in this form include joint displacements at each step of the pushover, frame member forces at each step of the pushover, and hinge force, displacement and state at each step of the pushover.

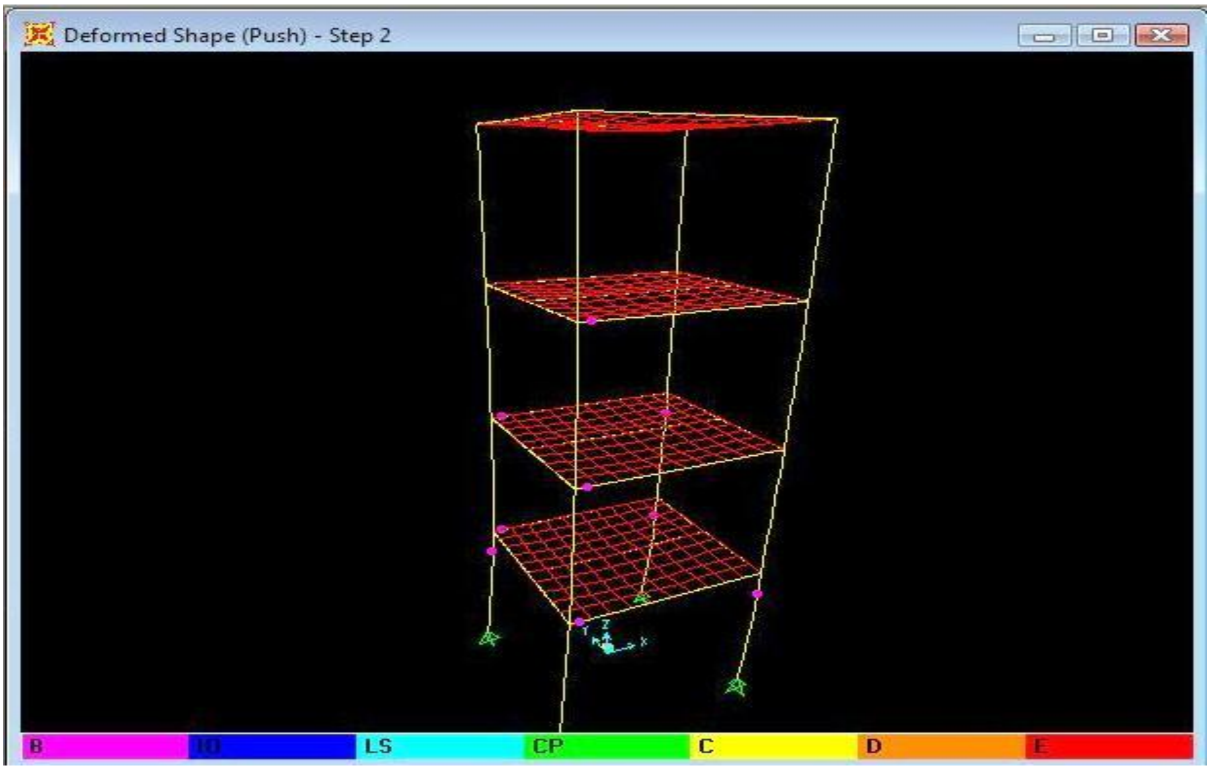


STEP 0

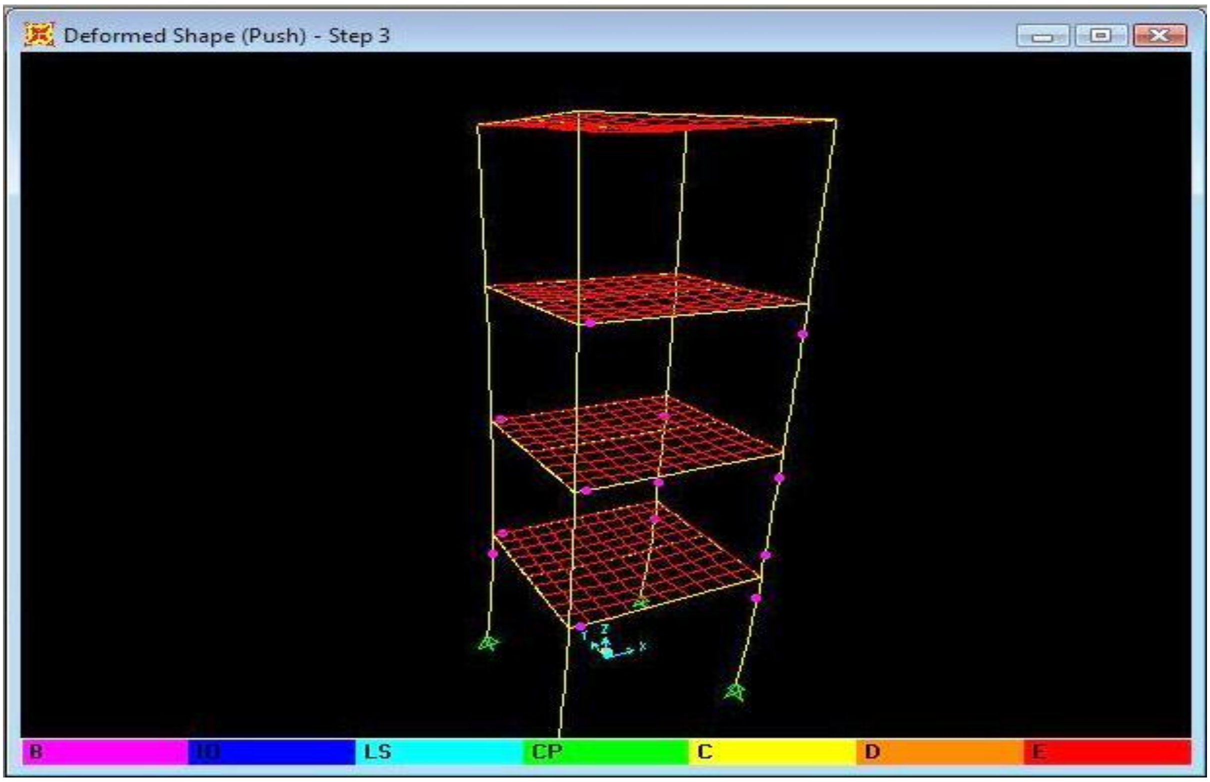


STEP 1

Fig 5.2(a): Step By Step Deformations for Pushover

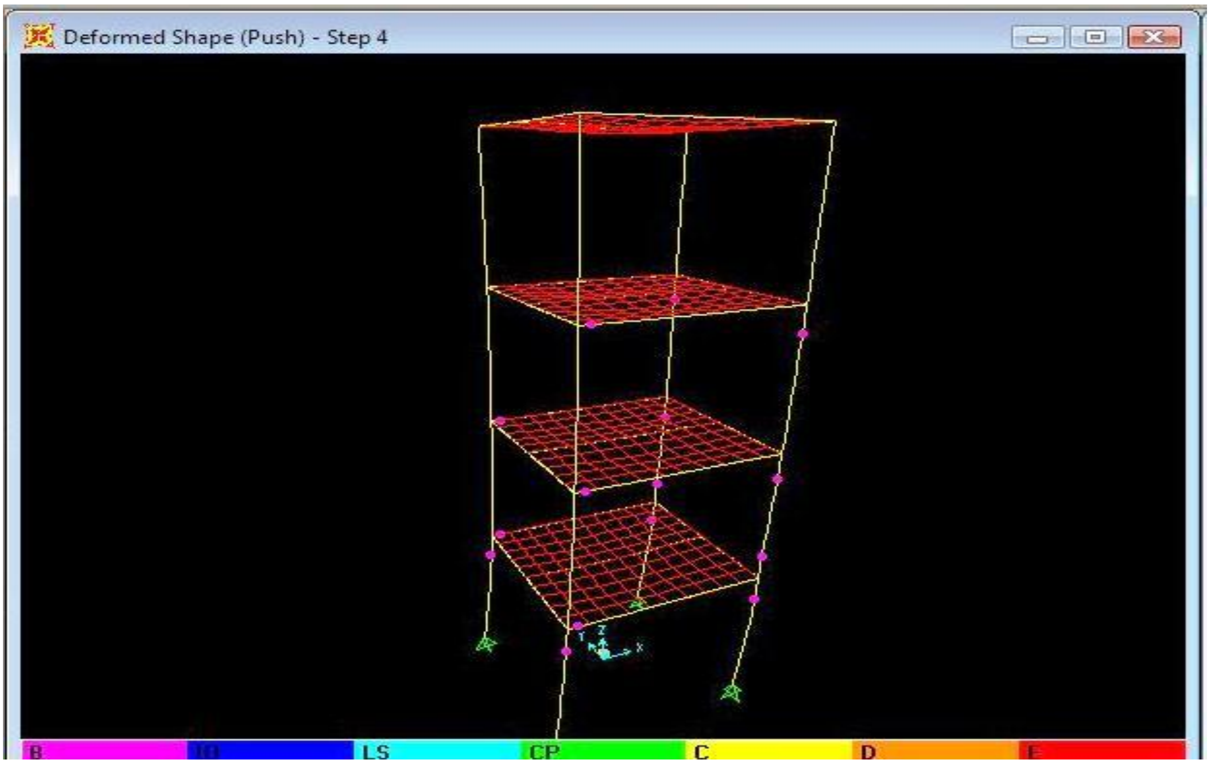


STEP 2

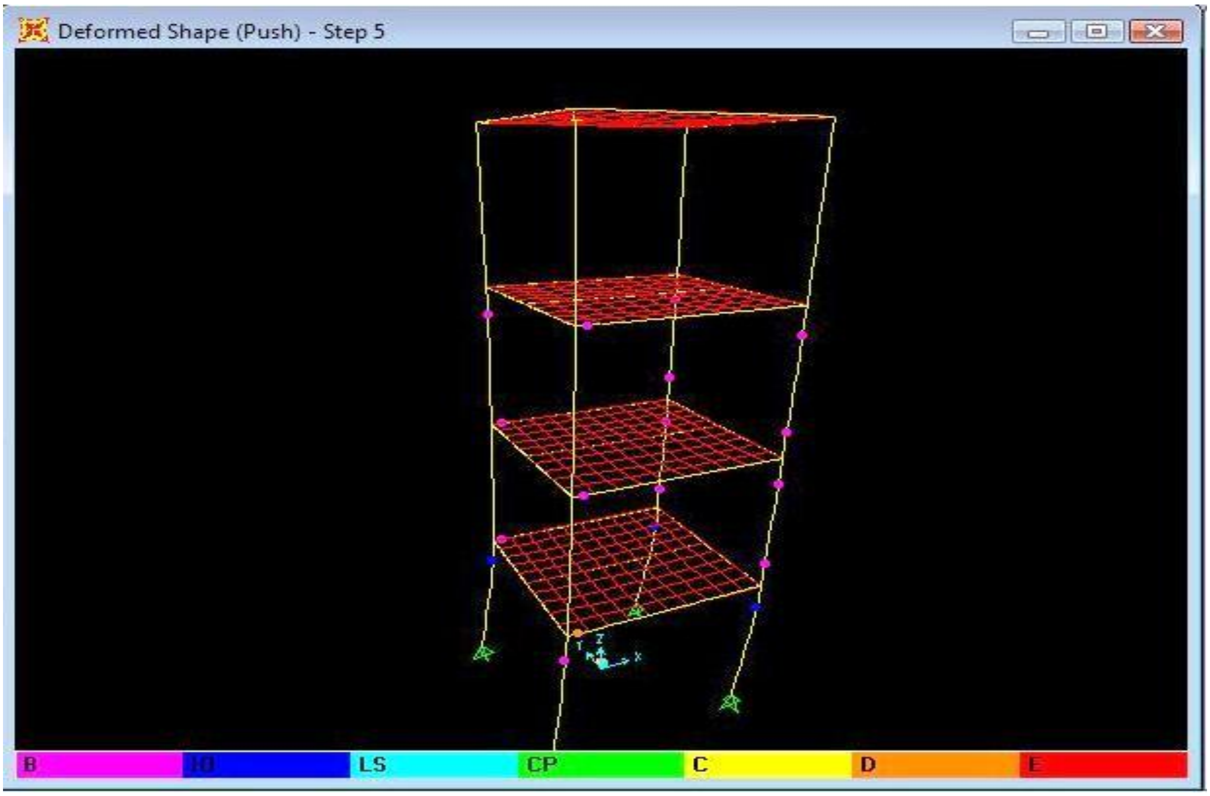


STEP 3

Fig 5.2(b): Step By Step Deformations for Pushover

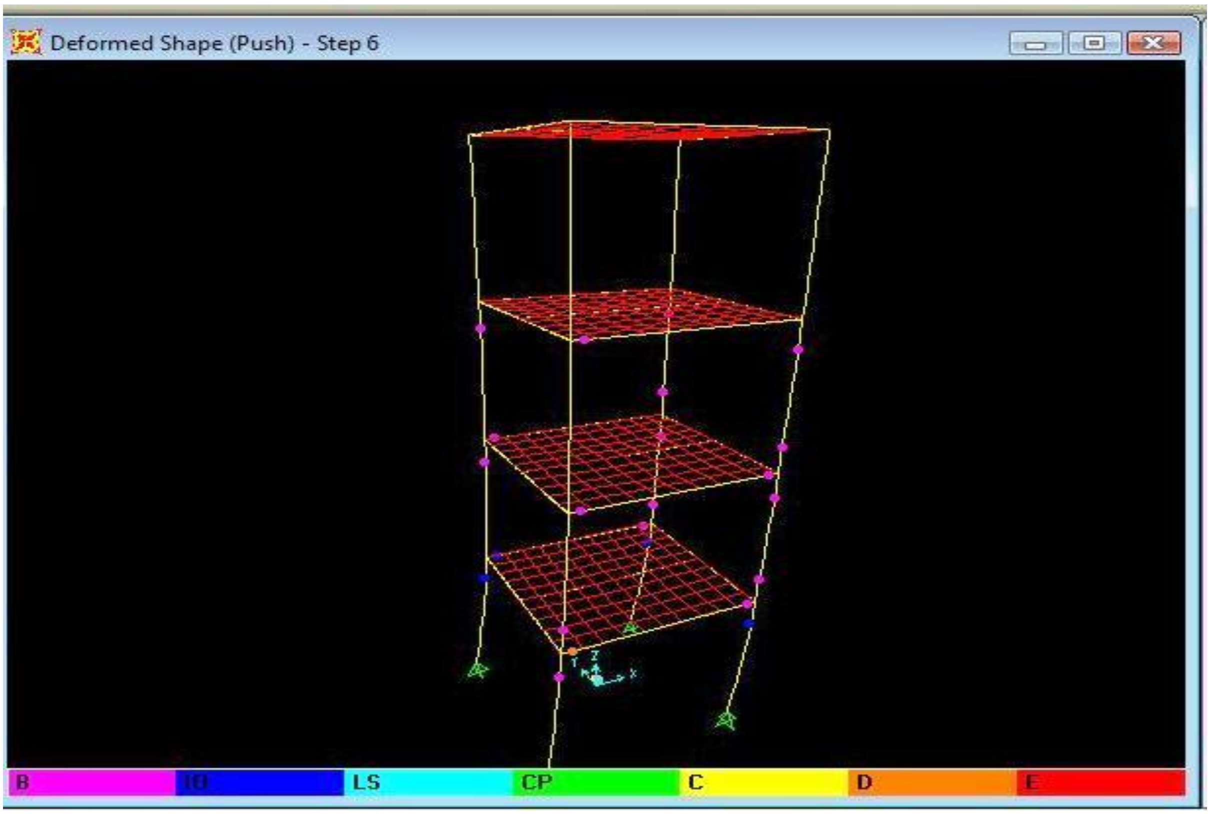


STEP 4

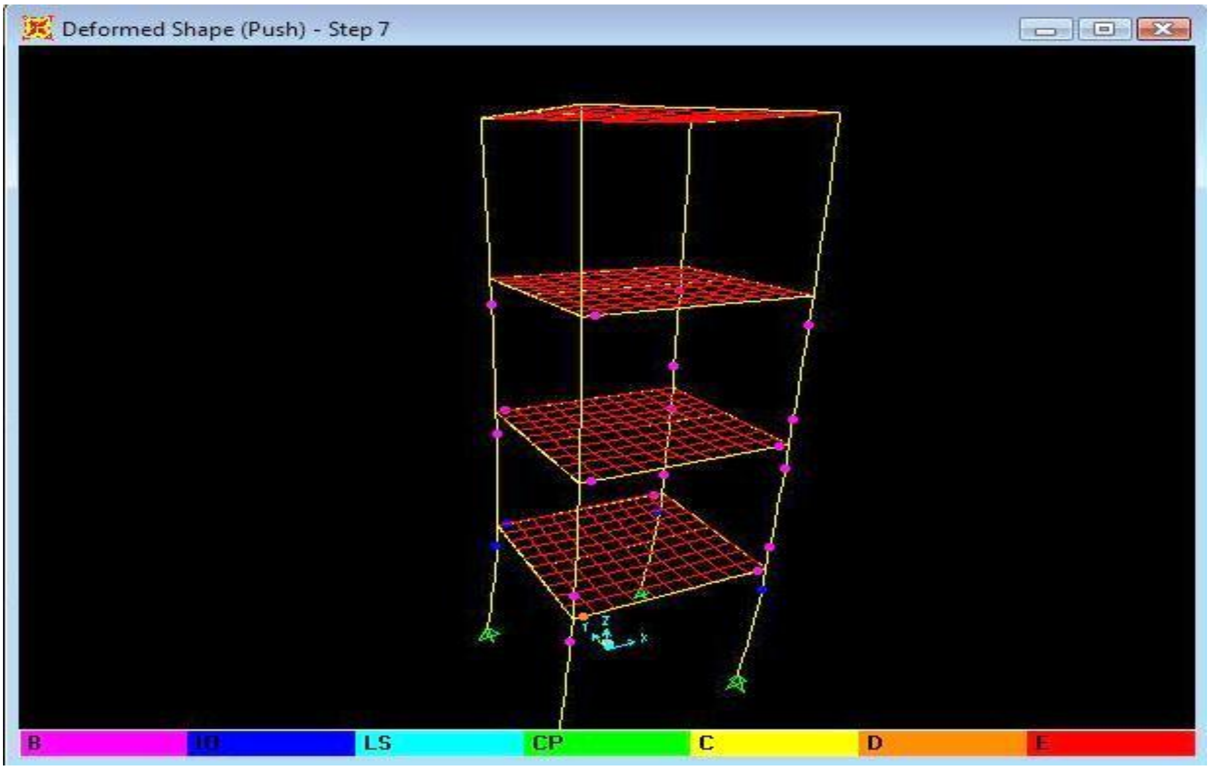


STEP 5

Fig 5.2(c): Step By Step Deformations for Pushover

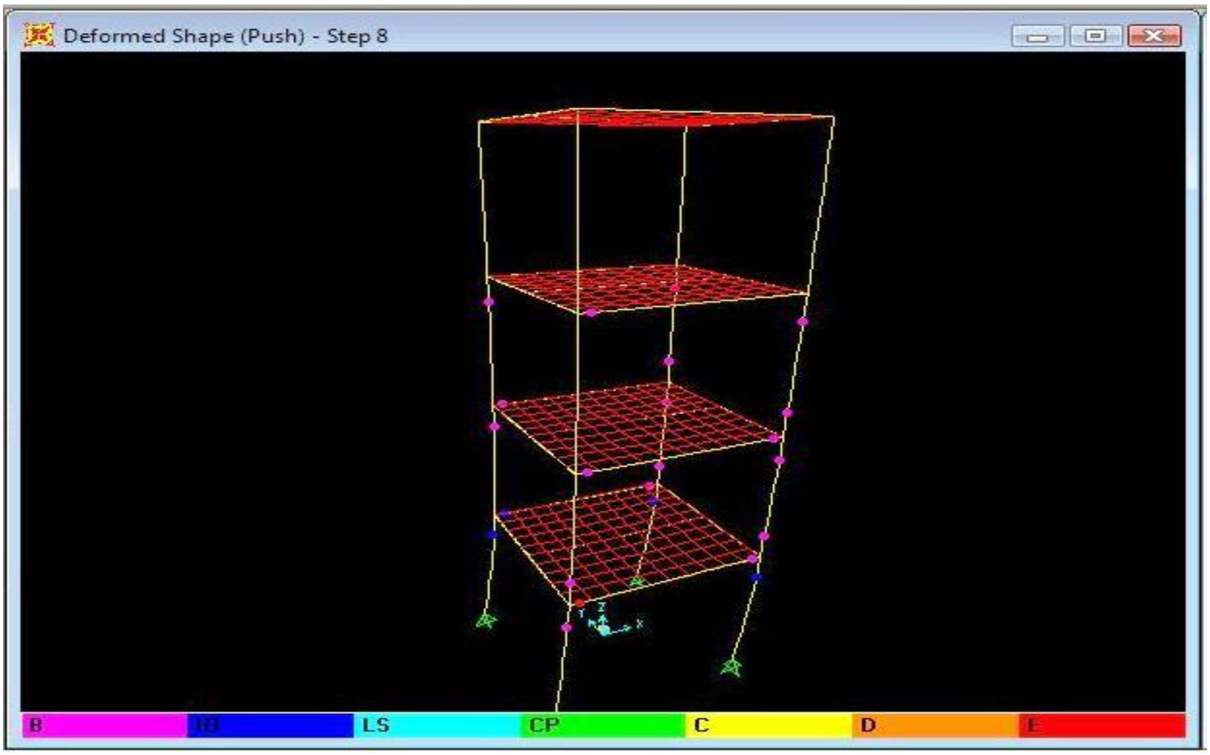


STEP 6

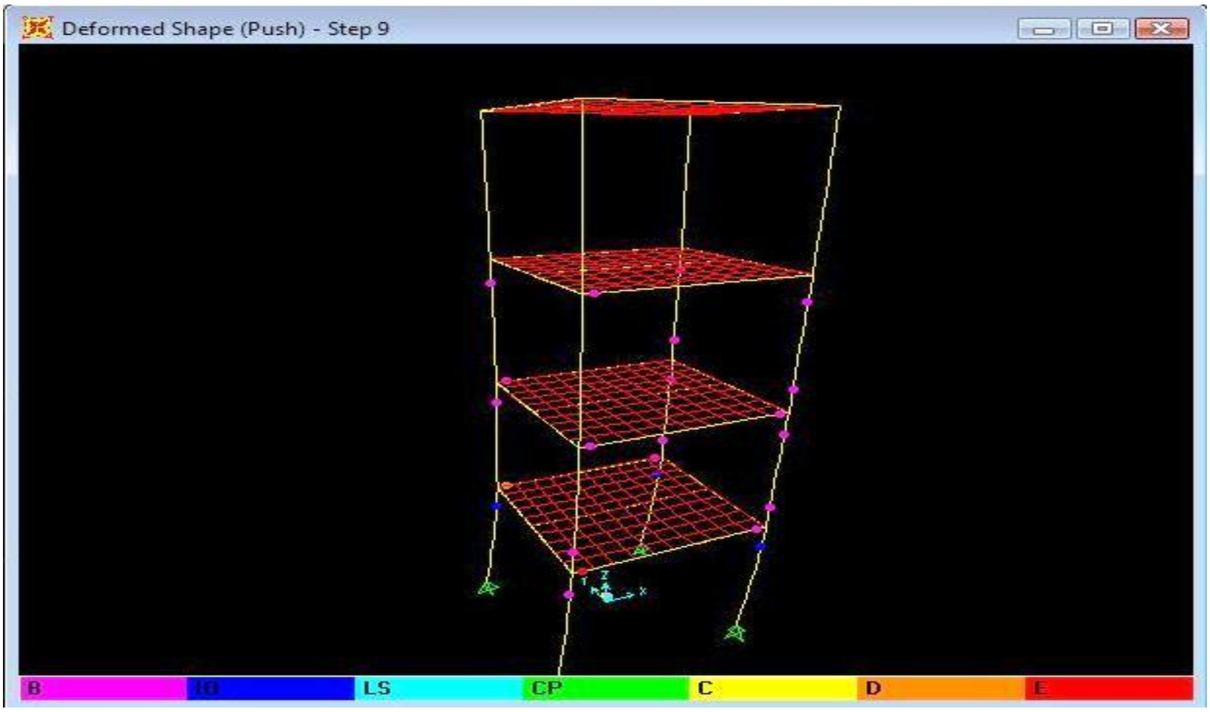


STEP 7

Fig 5.2(d): Step By Step Deformations for Pushover

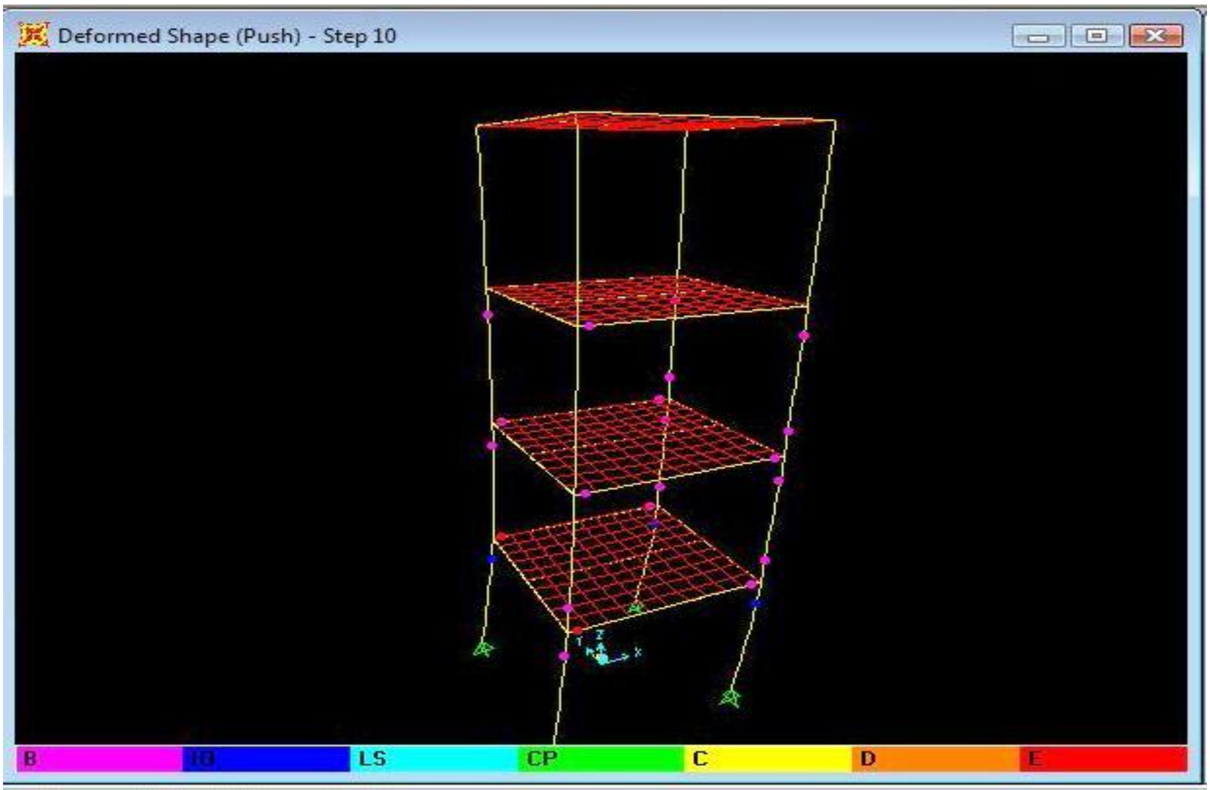


STEP 8

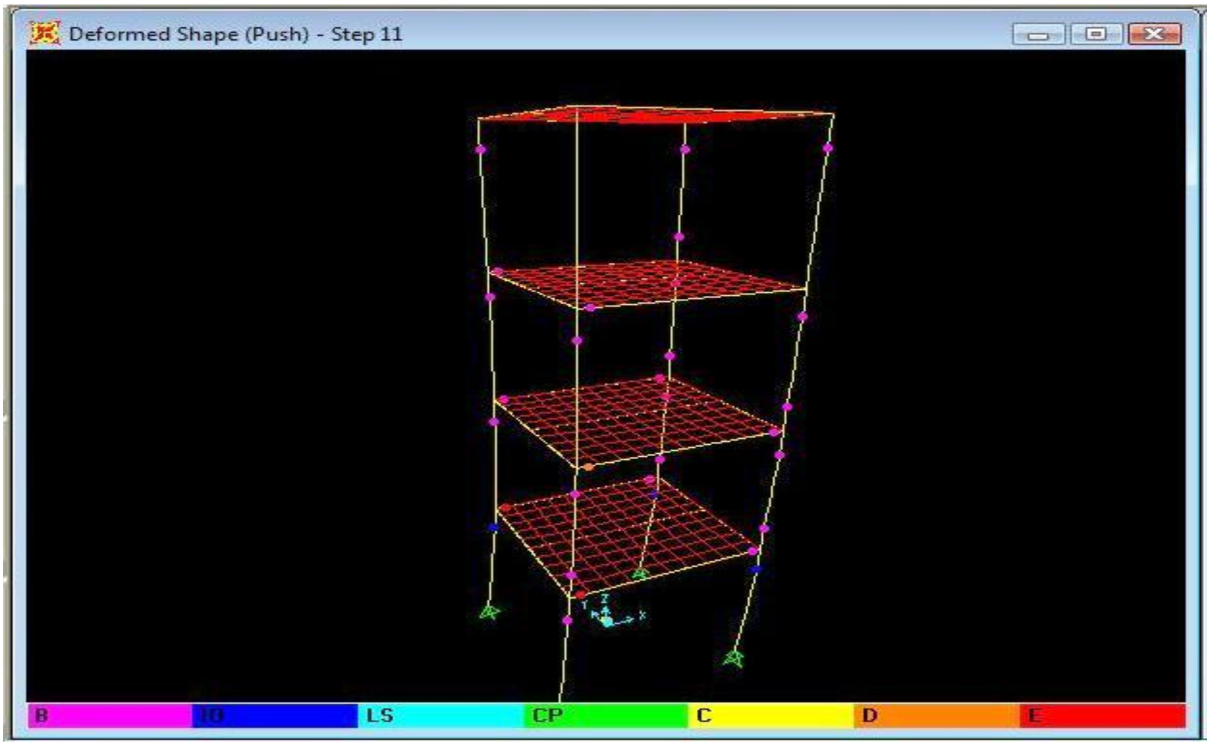


STEP 9

Fig 5.2(e): Step By Step Deformations for Pushover

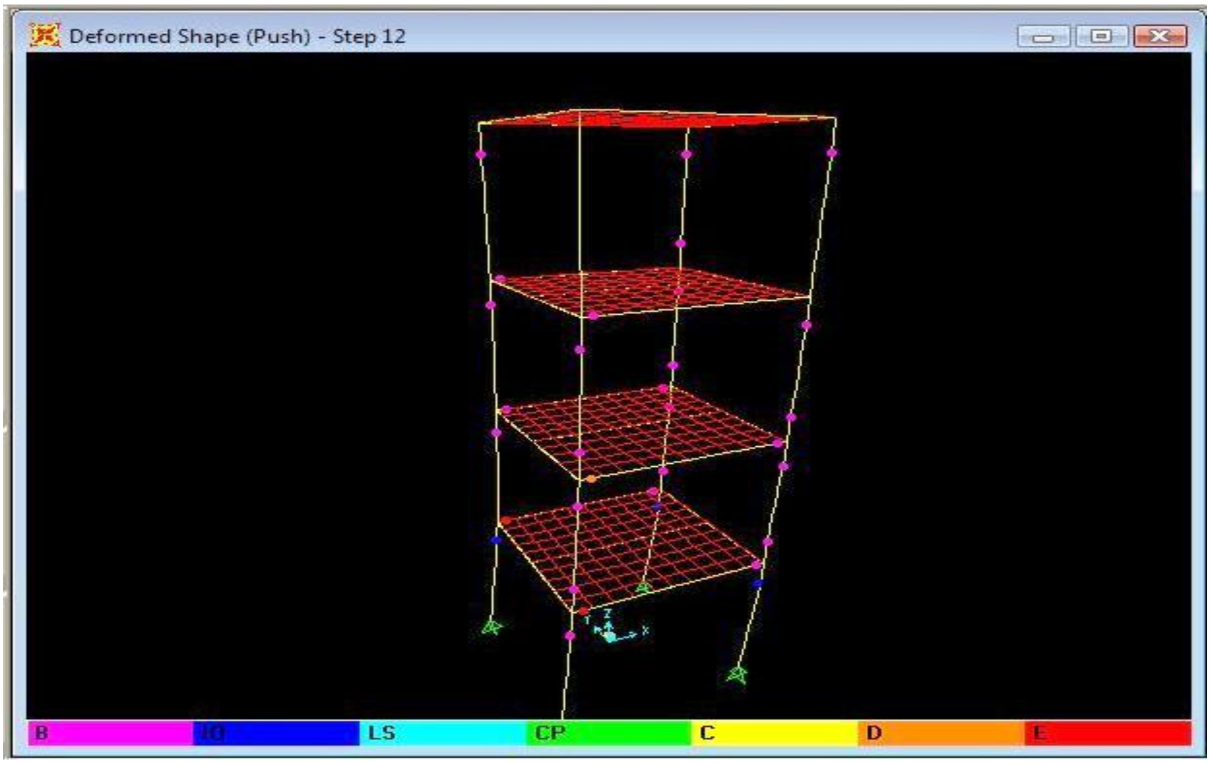


STEP 10

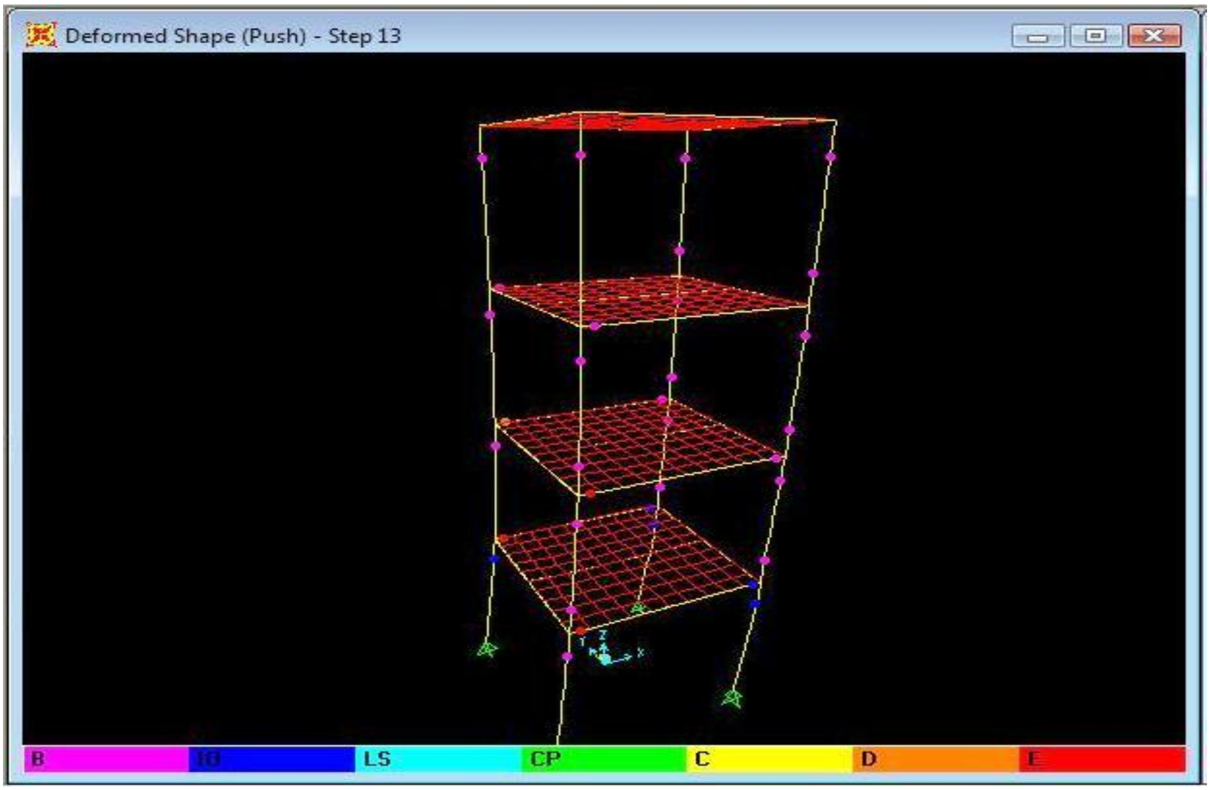


STEP 11

Fig 5.2(f): Step By Step Deformations for Pushover

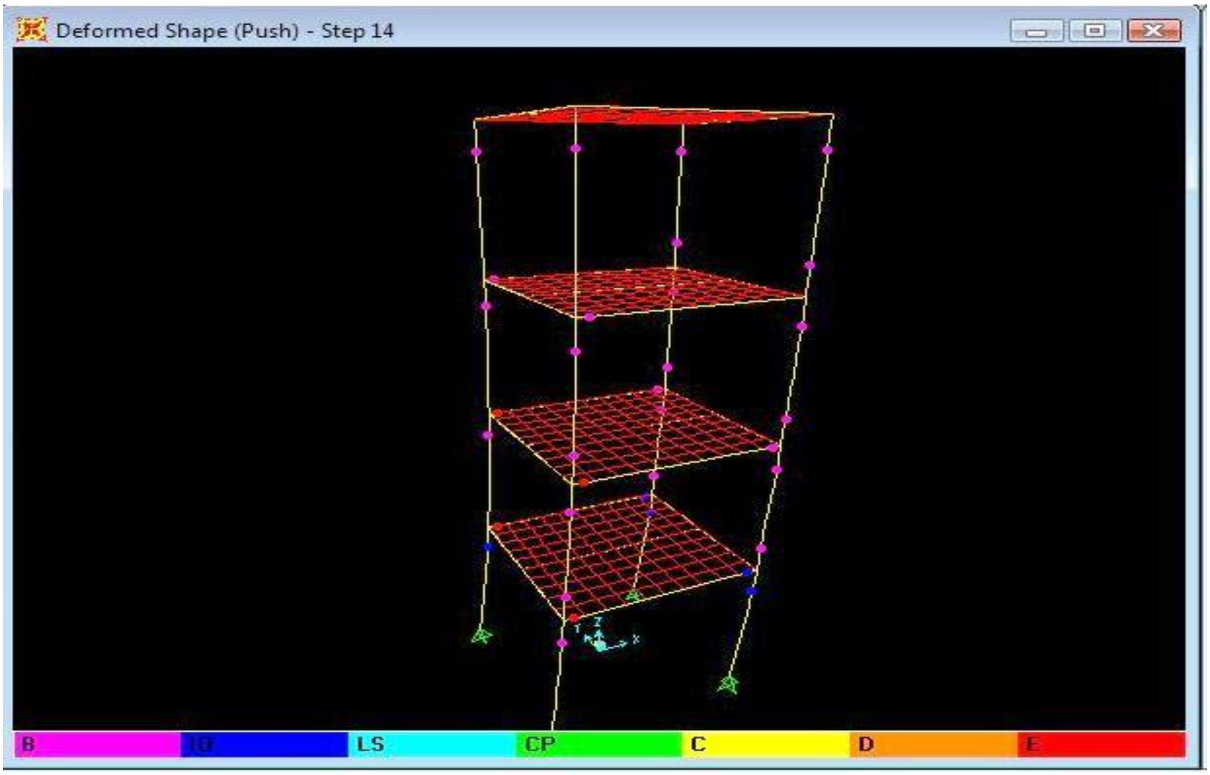


STEP 12

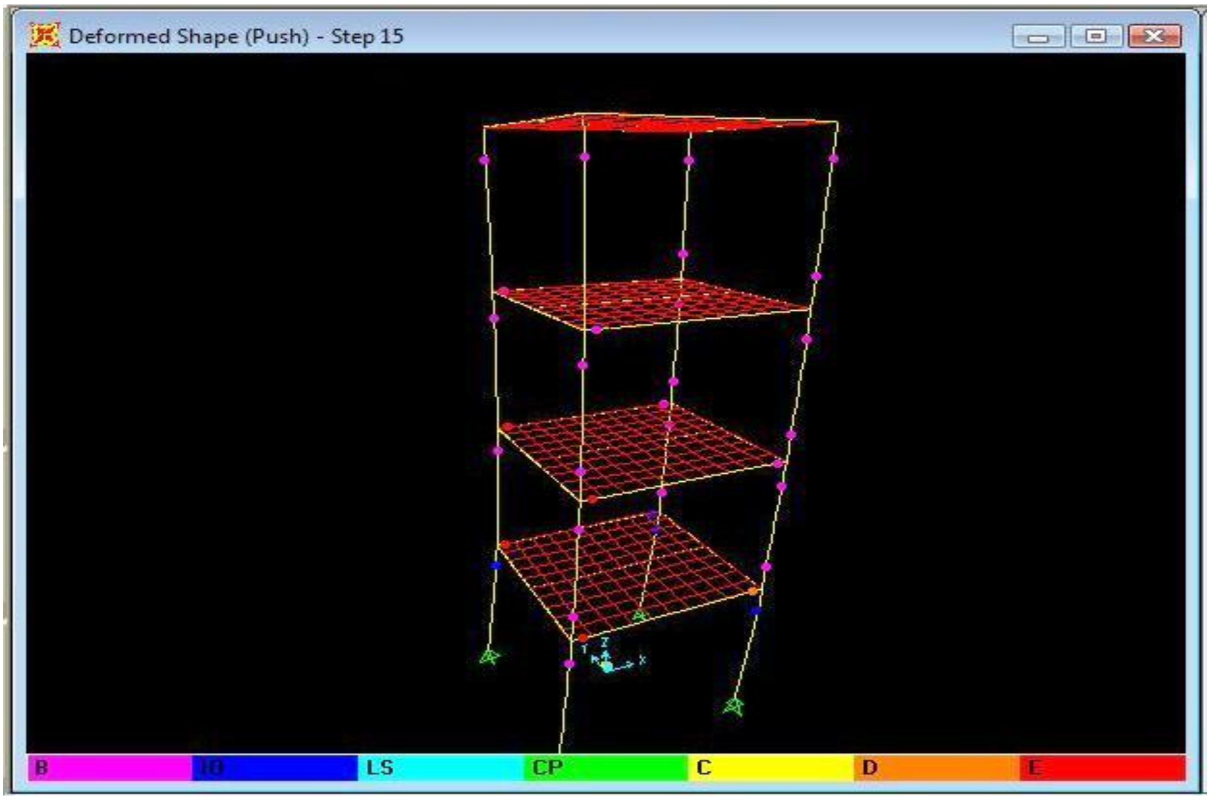


STEP 13

Fig 5.2(g): Step By Step Deformations for Pushover

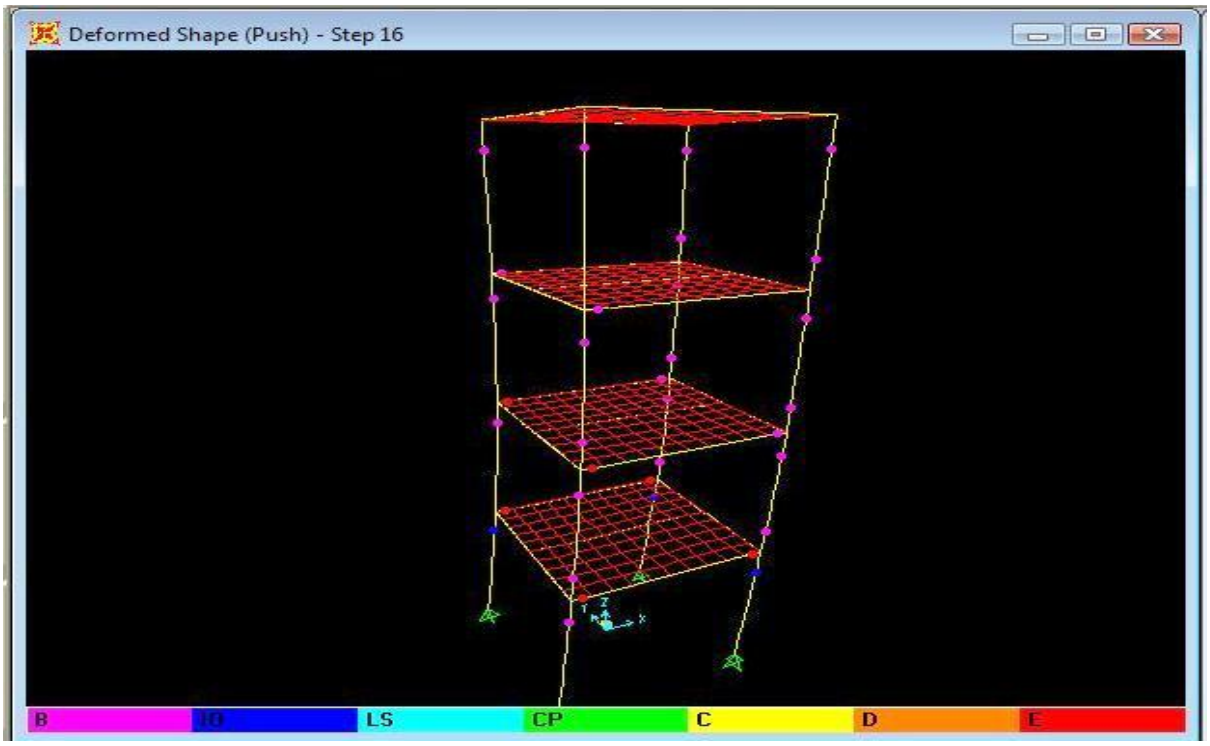


STEP 14

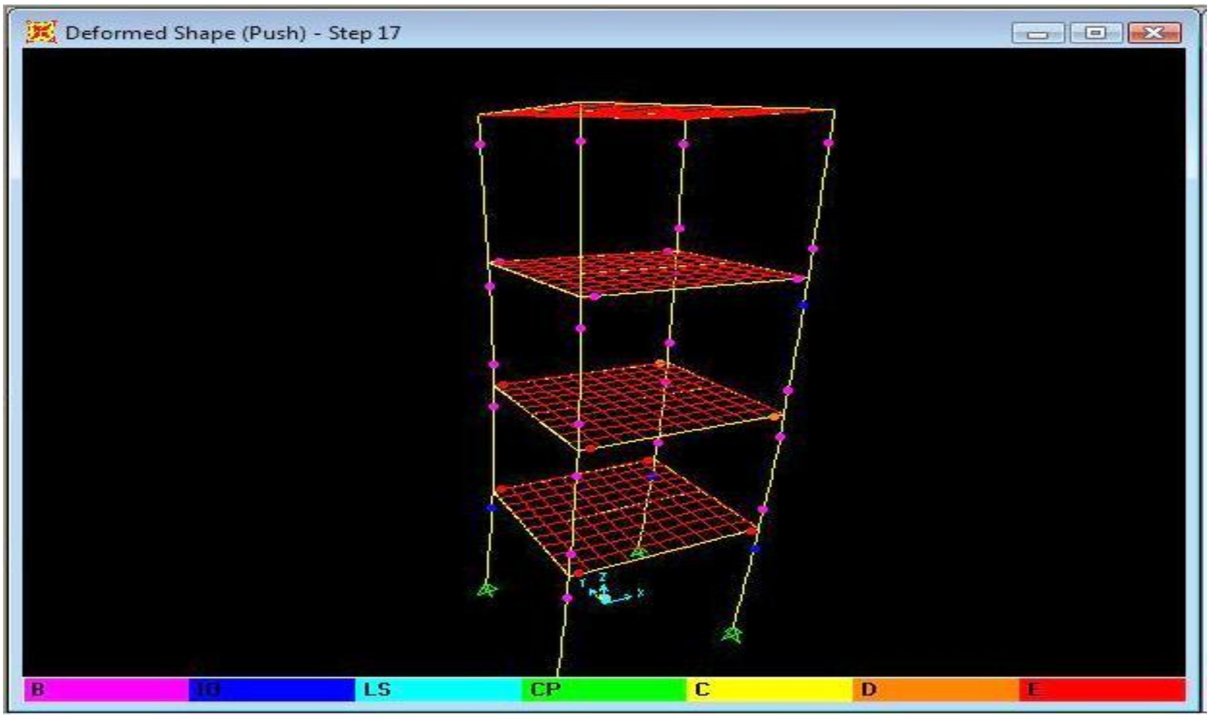


STEP 15

Fig 5.2(h): Step By Step Deformations for Pushover

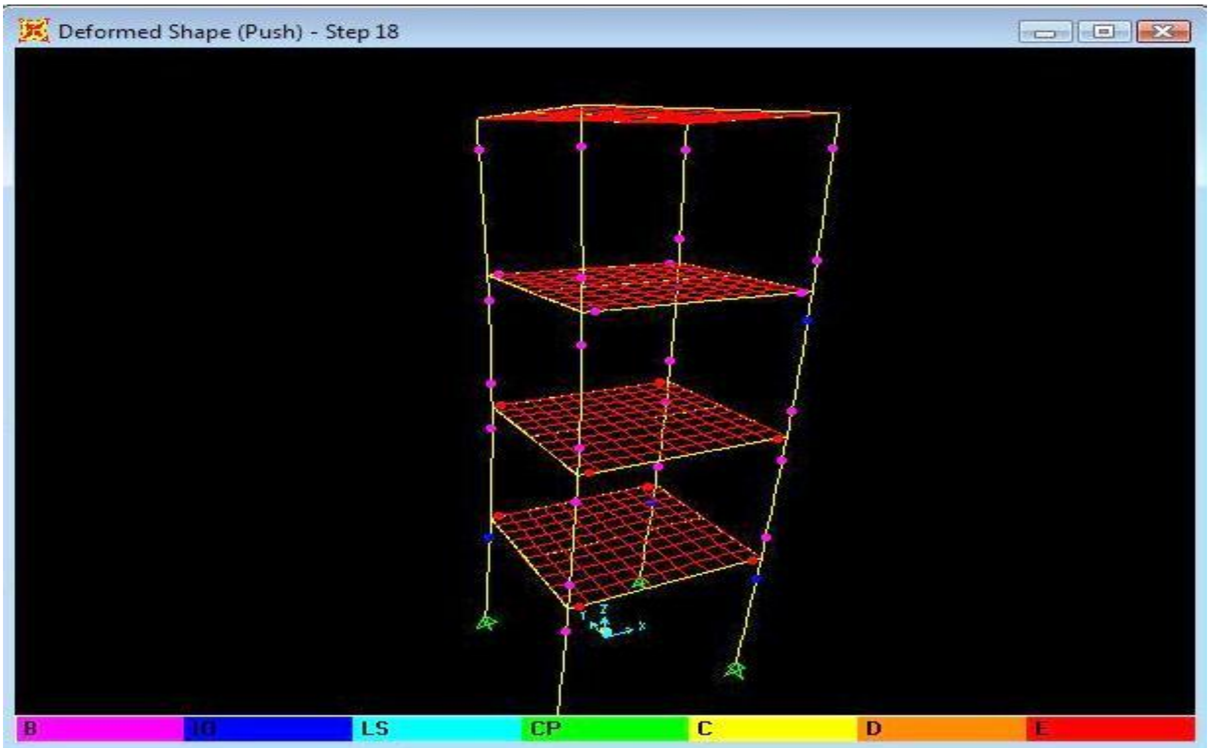


STEP 16

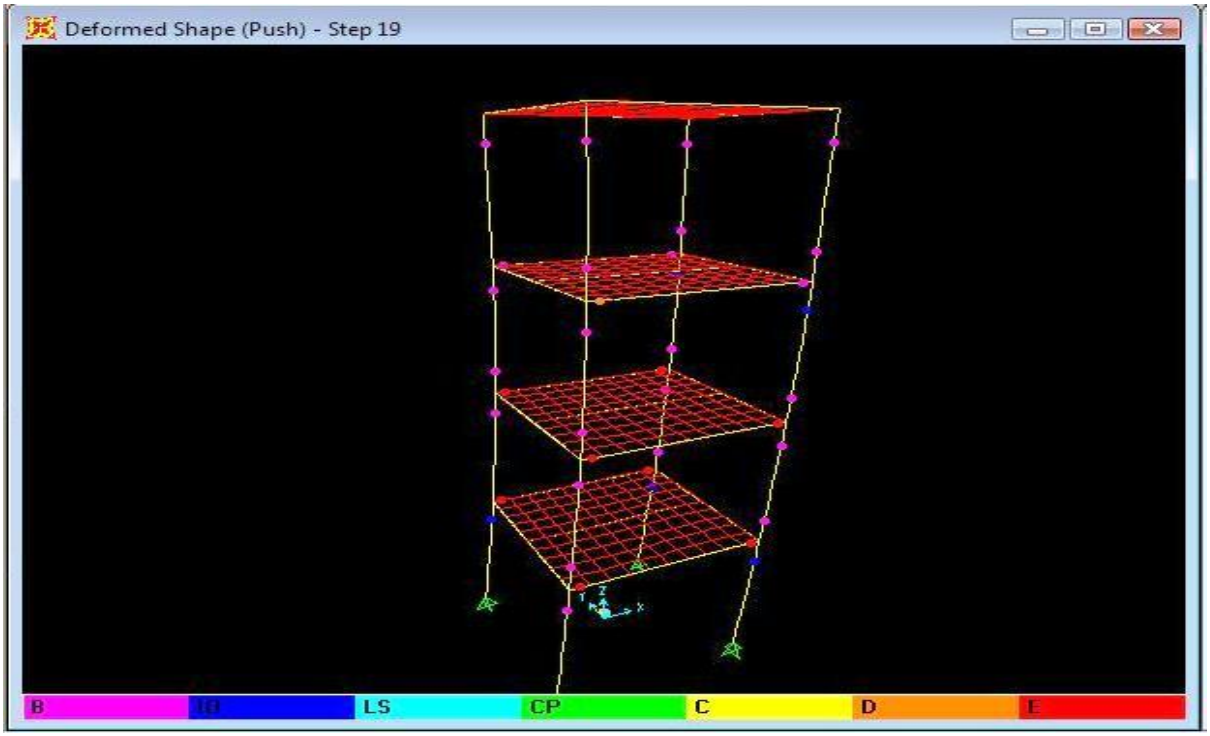


STEP 17

Fig 5.2(i): Step By Step Deformations for Pushover

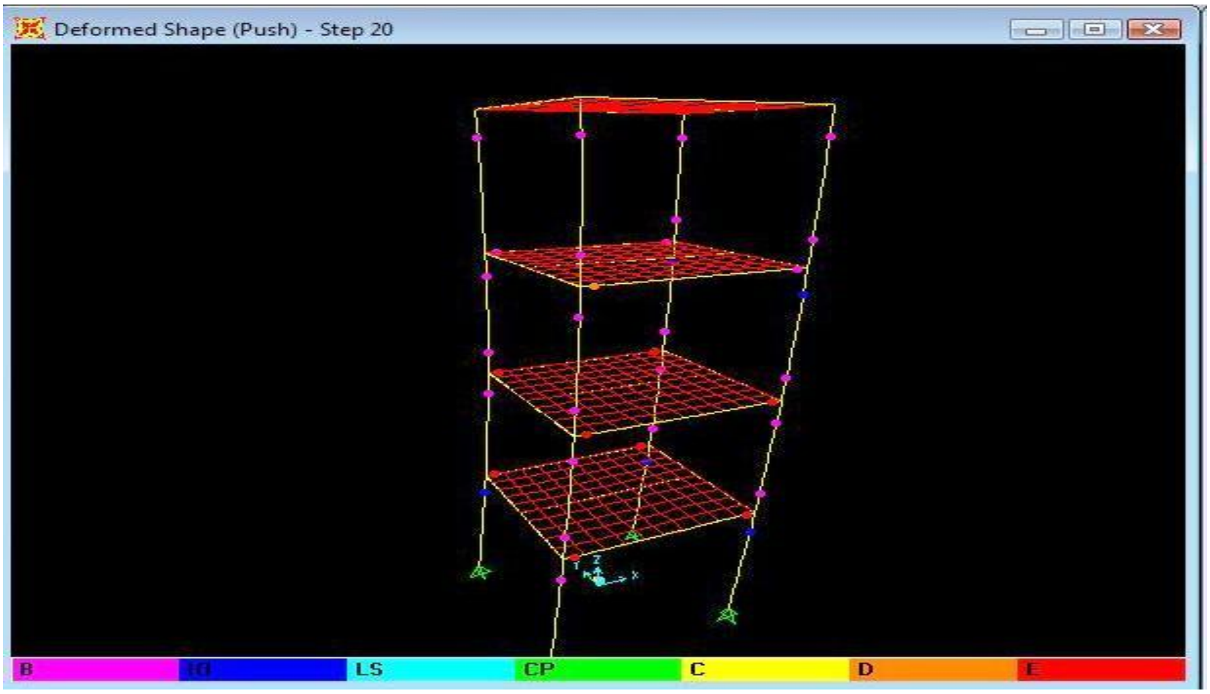


STEP 18

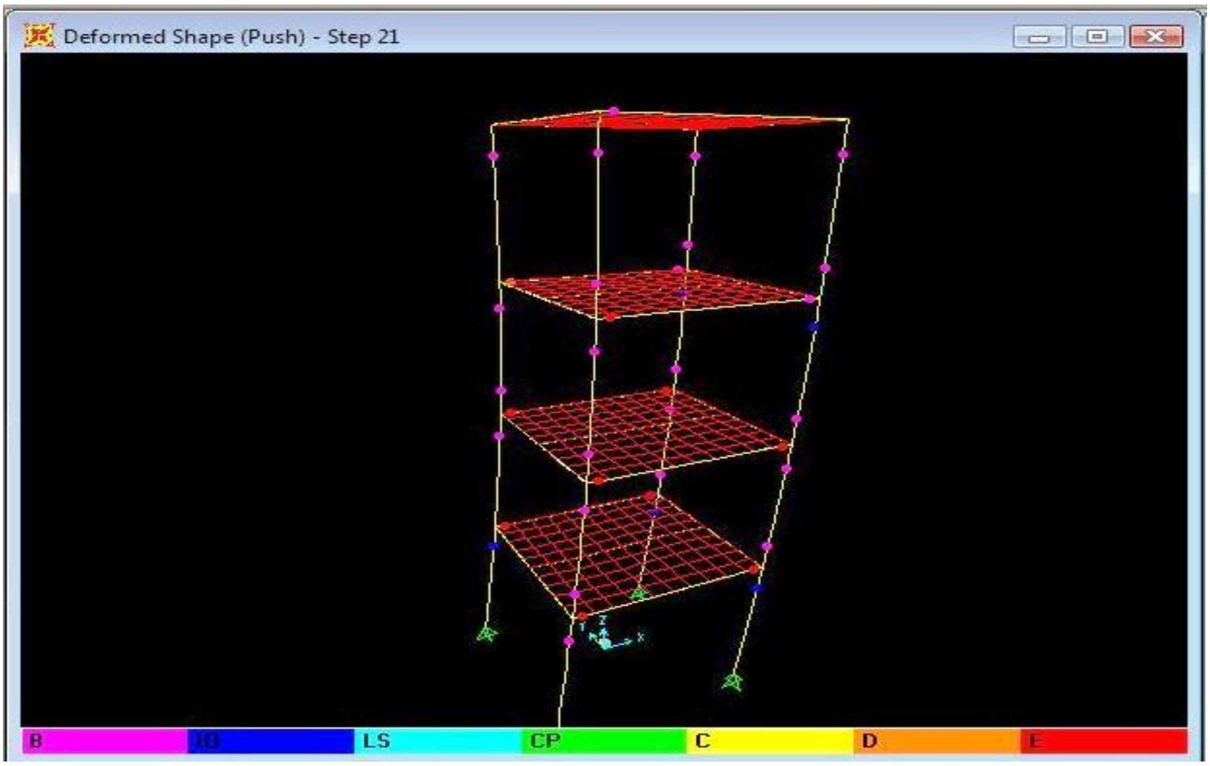


STEP 19

Fig 5.2(j): Step By Step Deformations for Pushover

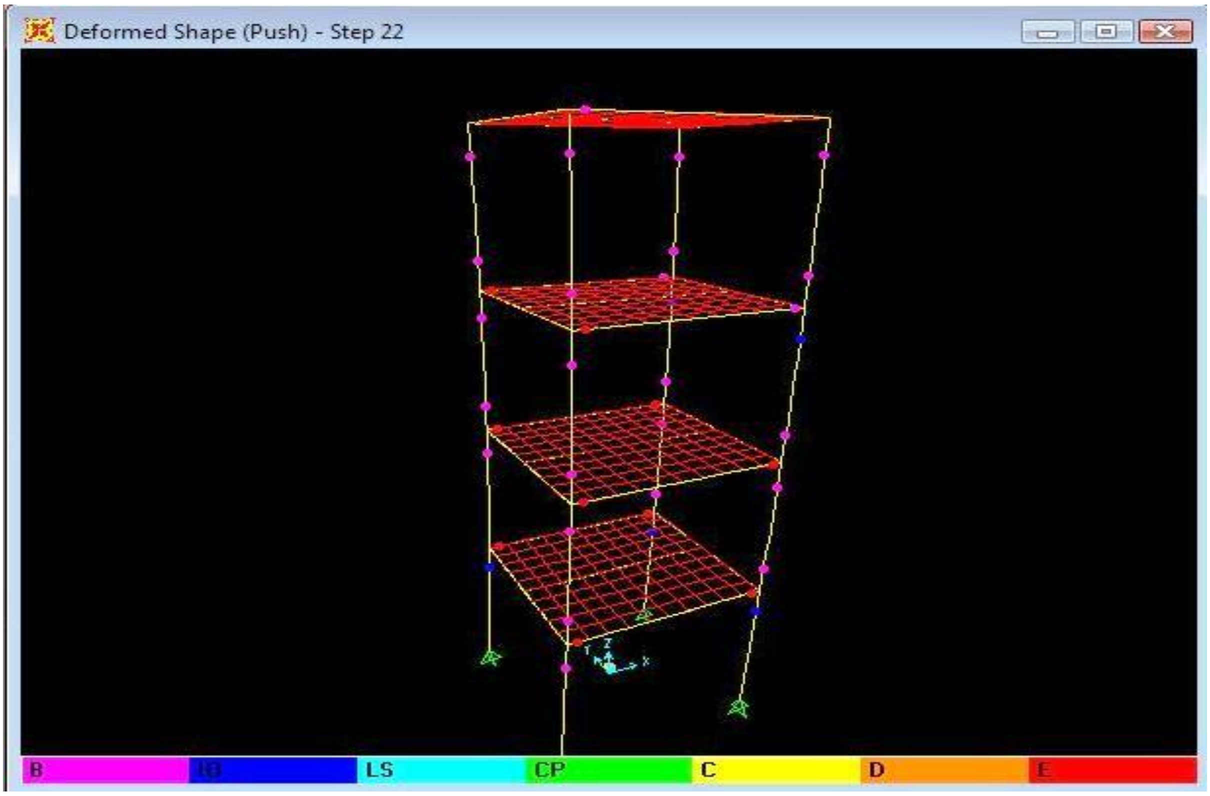


STEP 20

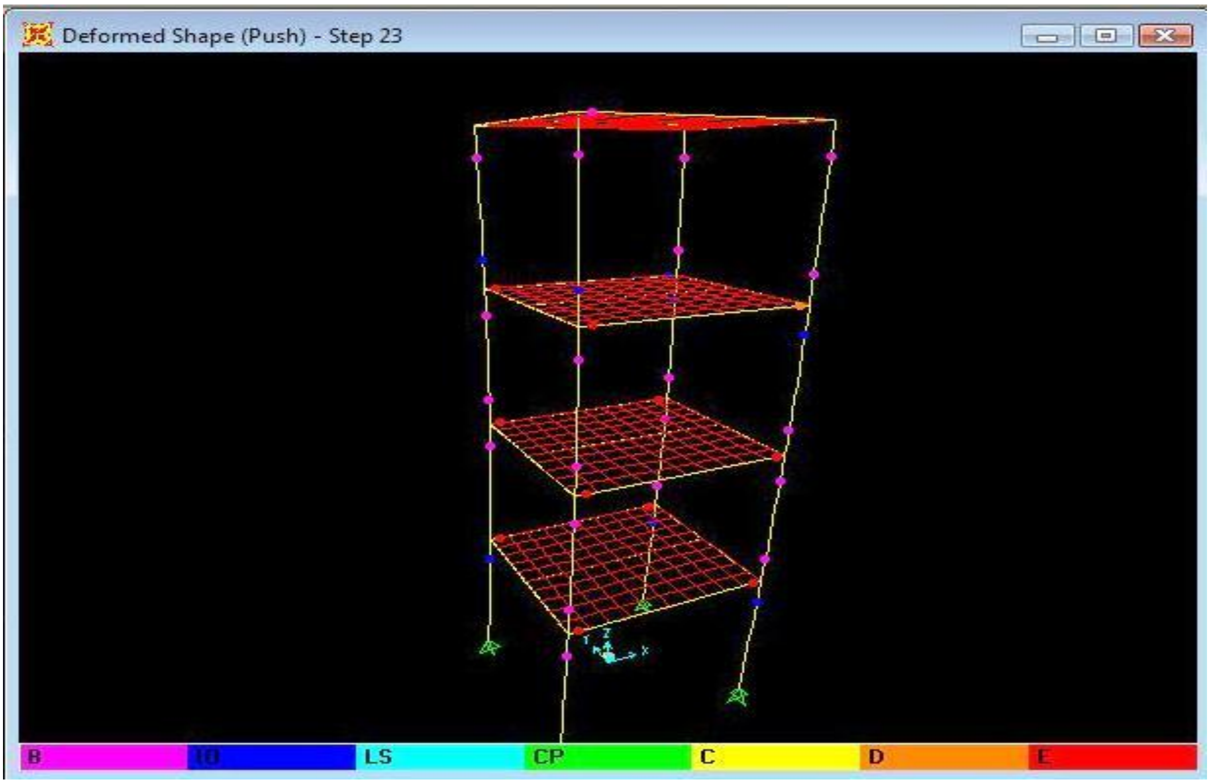


STEP 21

Fig 5.2(k): Step By Step Deformations for Pushove

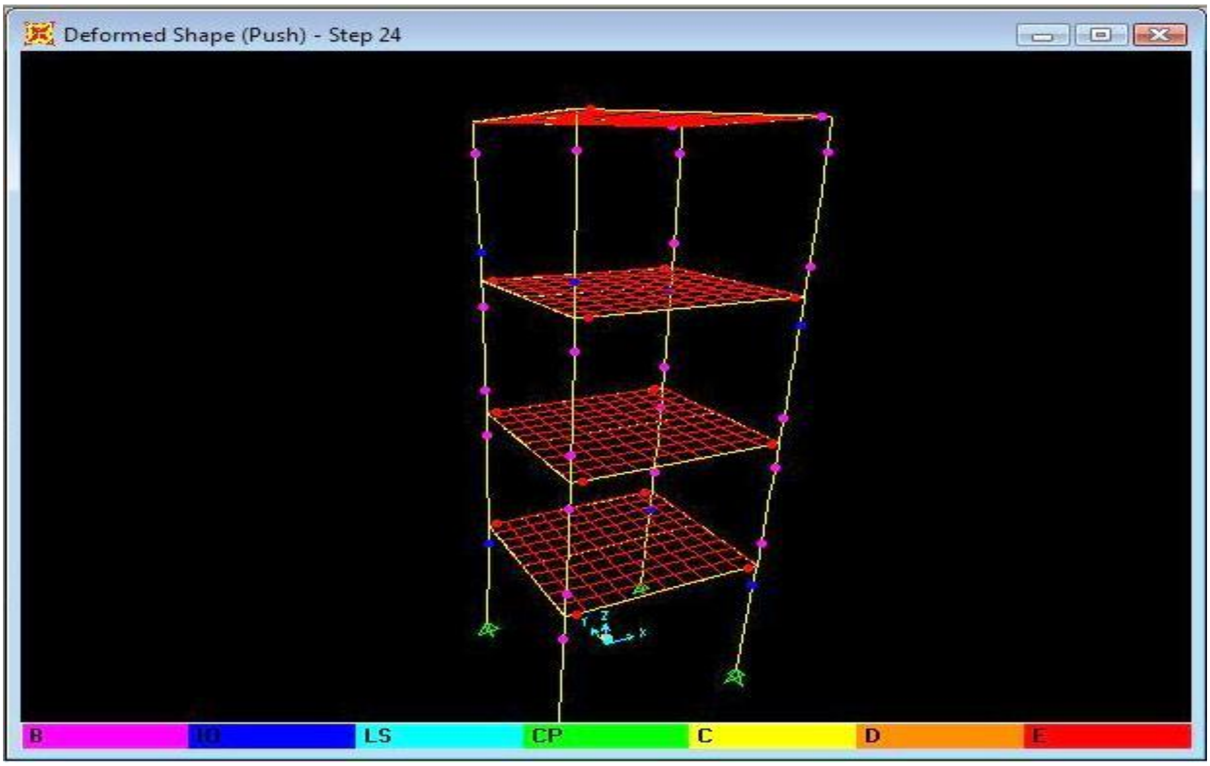


STEP 22

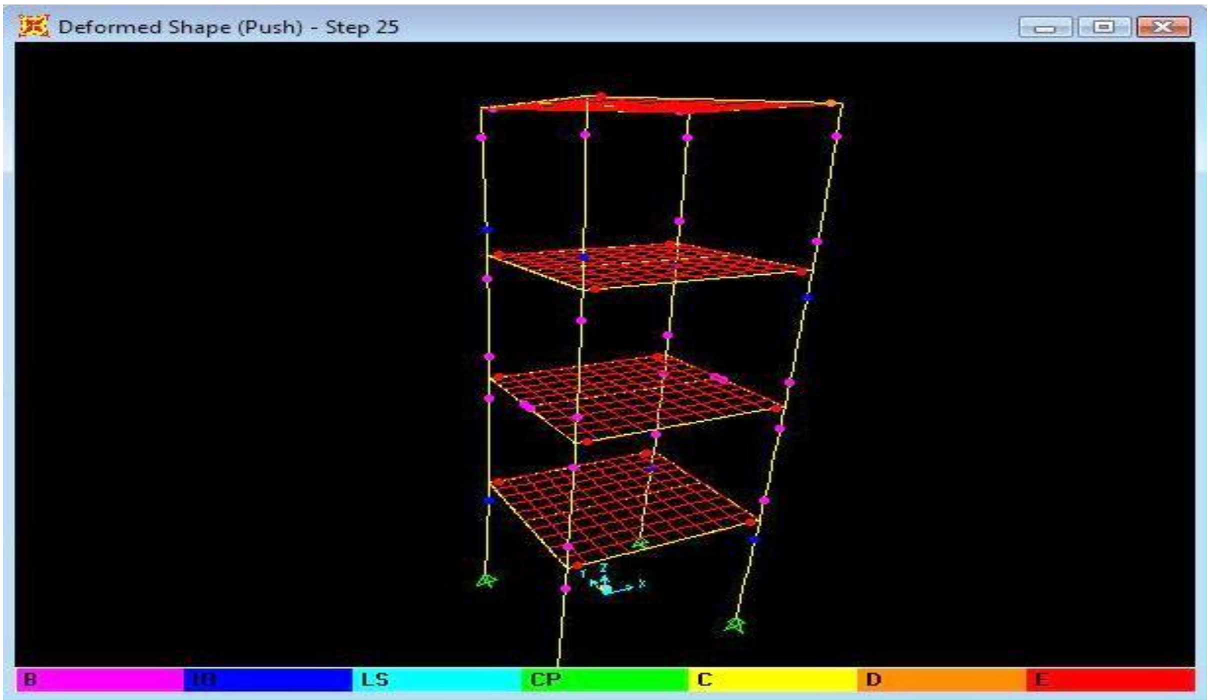


STEP 23

Fig 5.2(l): Step By Step Deformations for Pushover

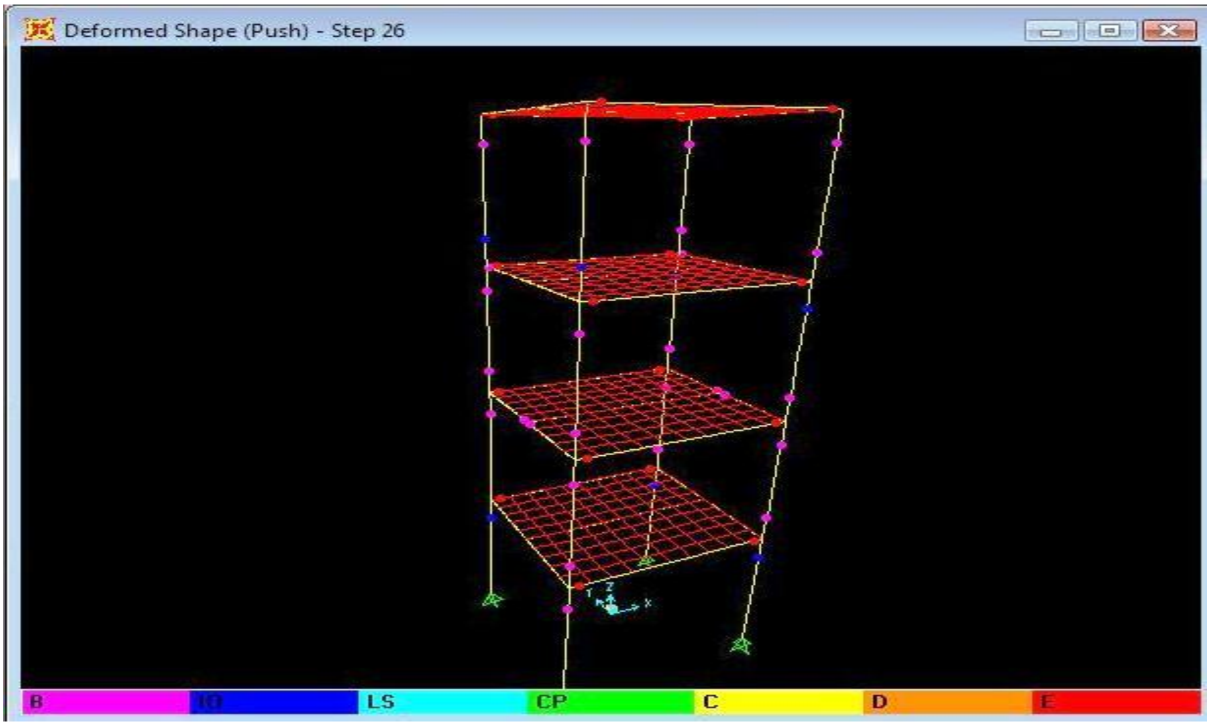


STEP 24

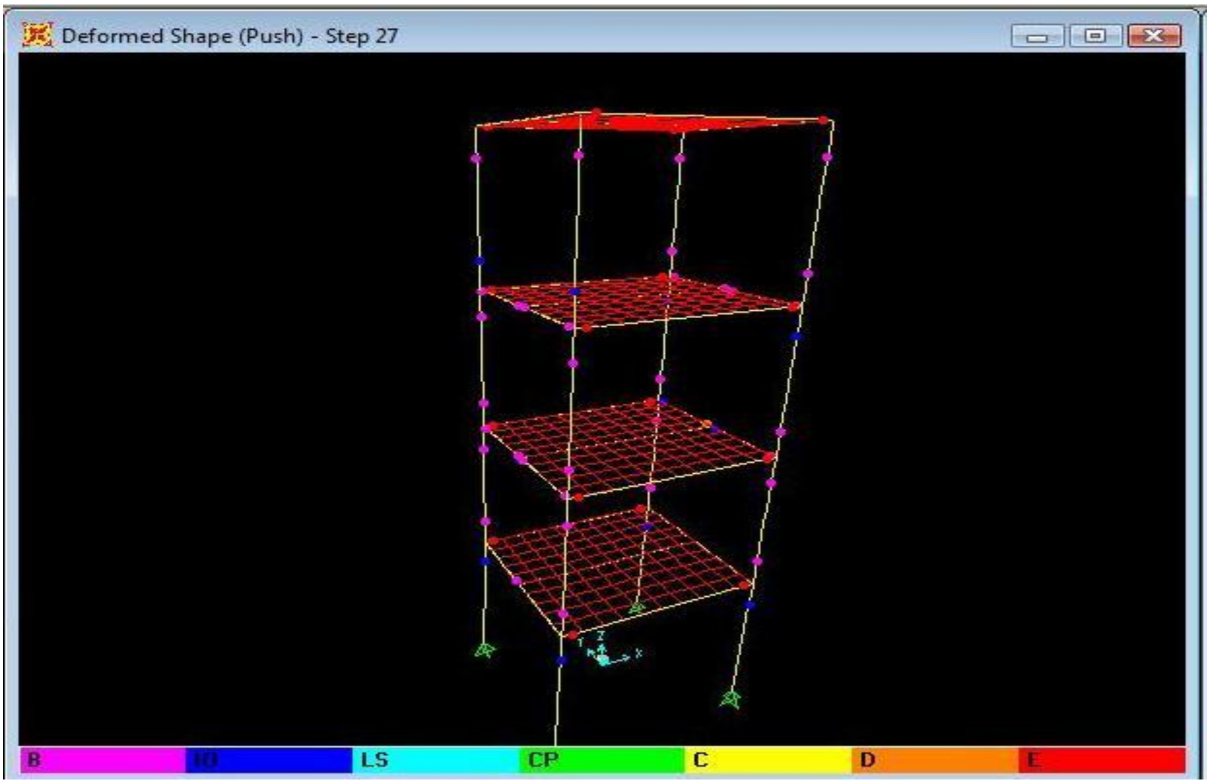


STEP 25

Fig 5.2(m): Step By Step Deformations for Pushover

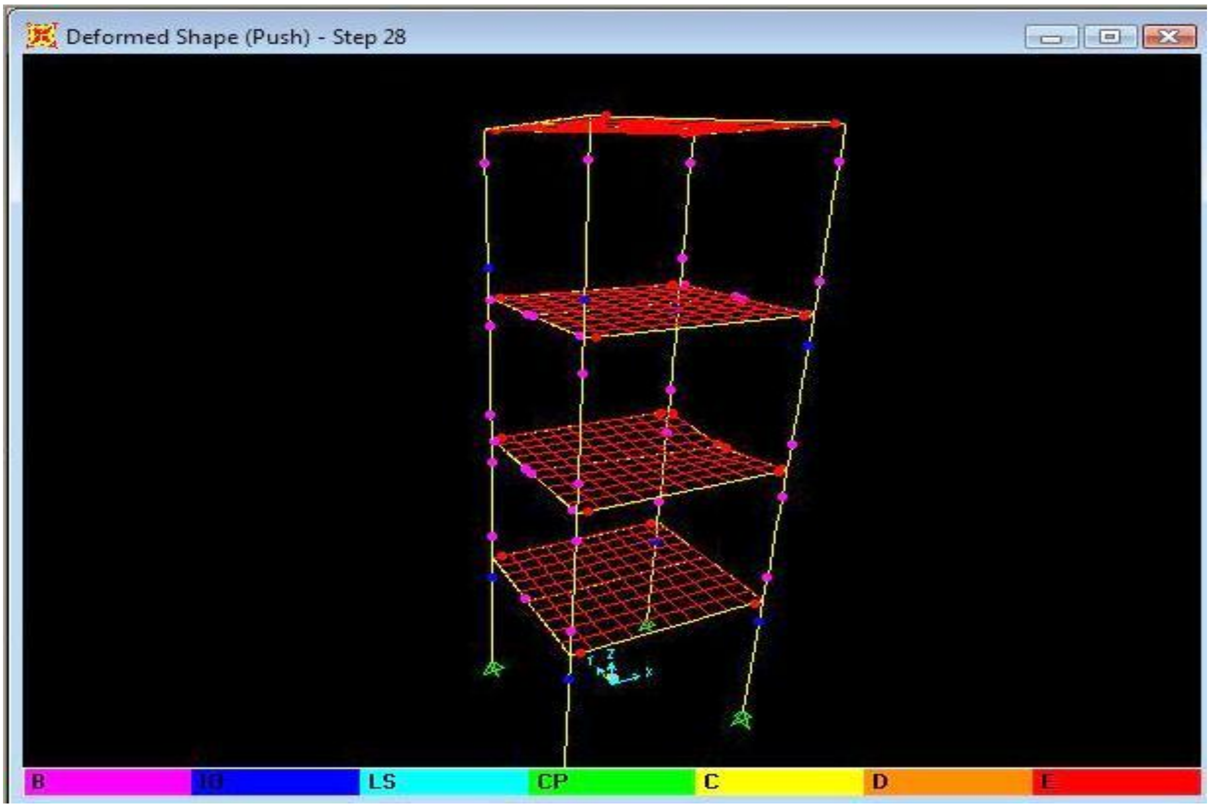


STEP 26

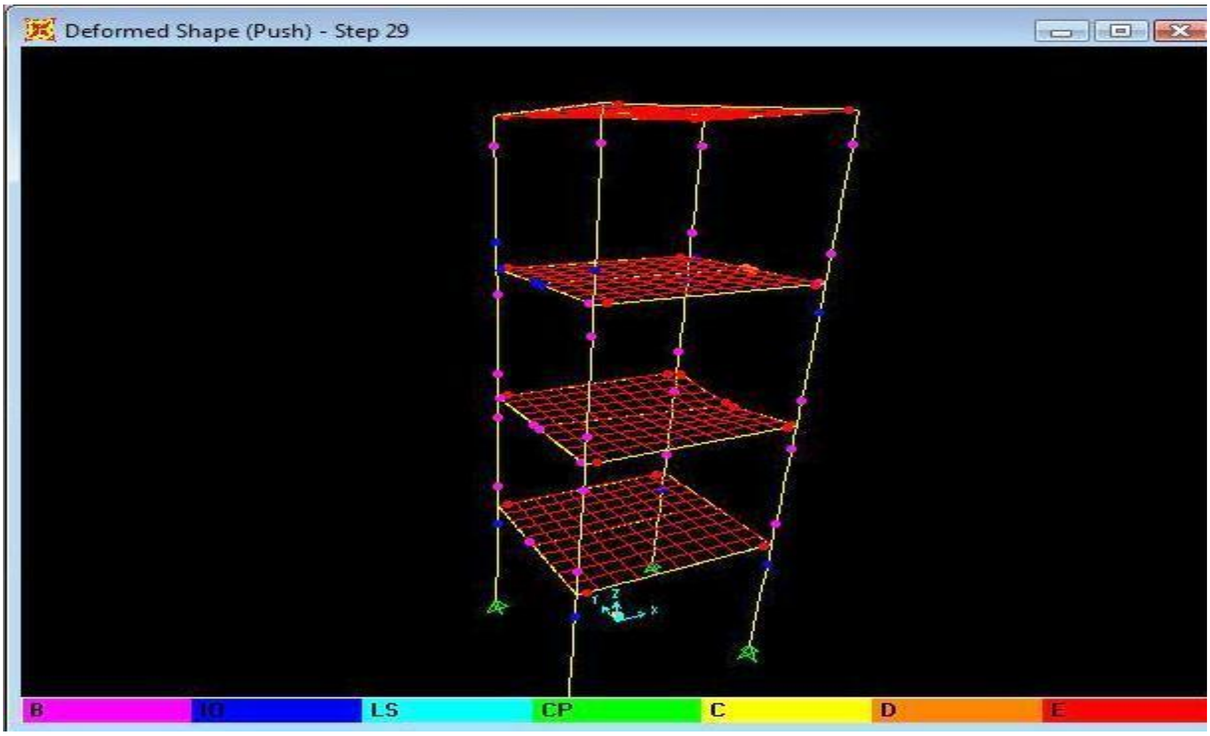


STEP 27

Fig 5.2(n): Step By Step Deformations for Pushover

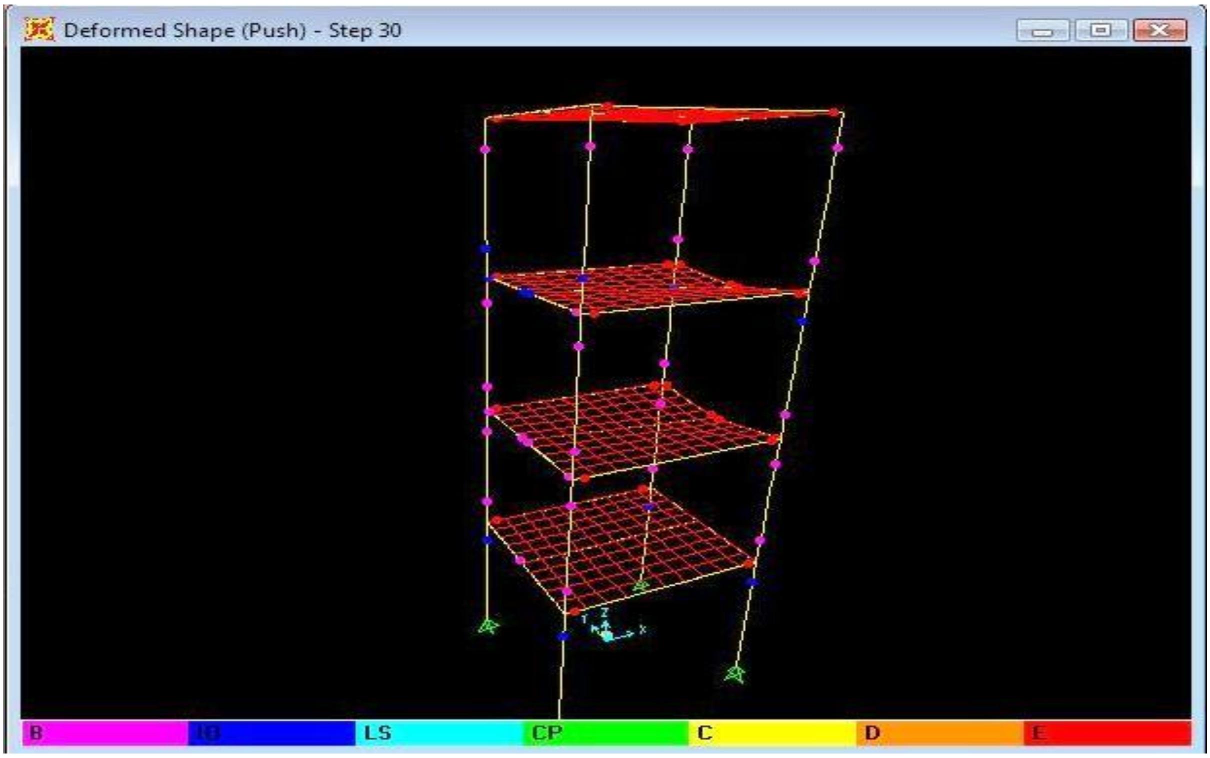


STEP 28

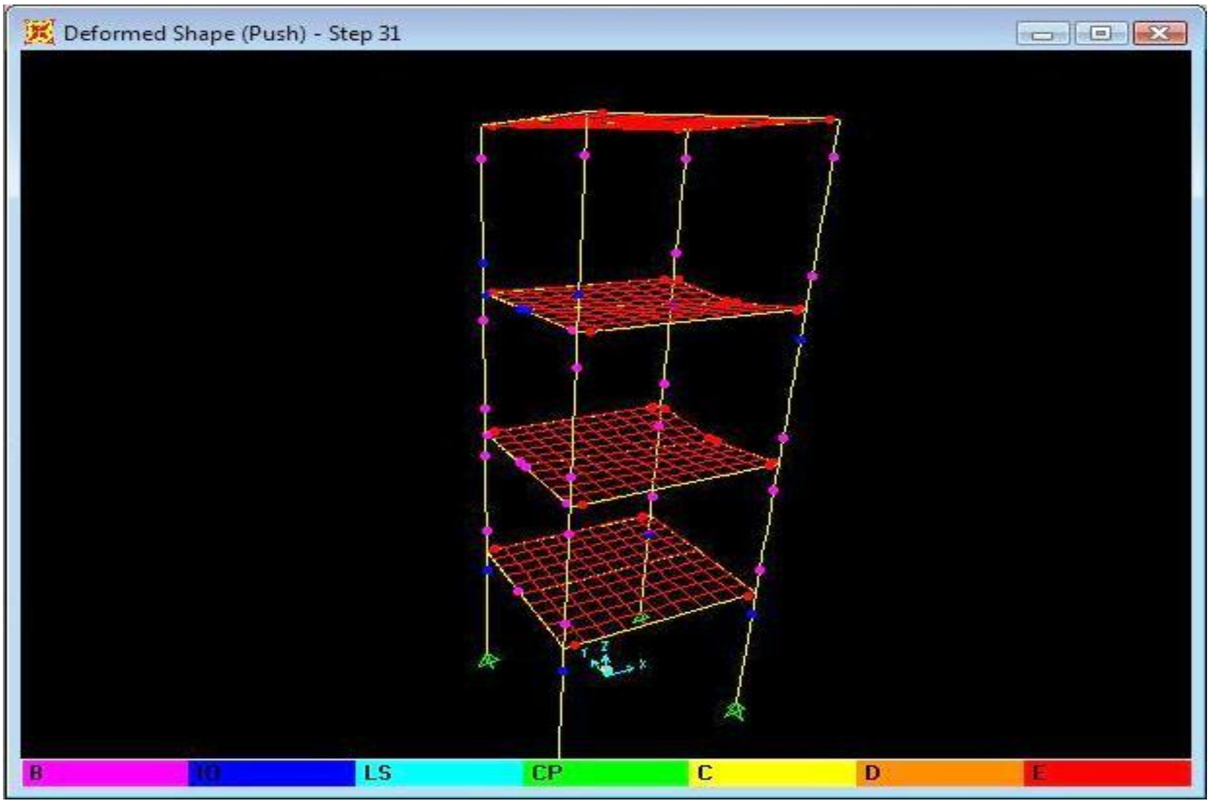


STEP 29

Fig 5.2(o): Step By Step Deformations for Pushover



STEP 30



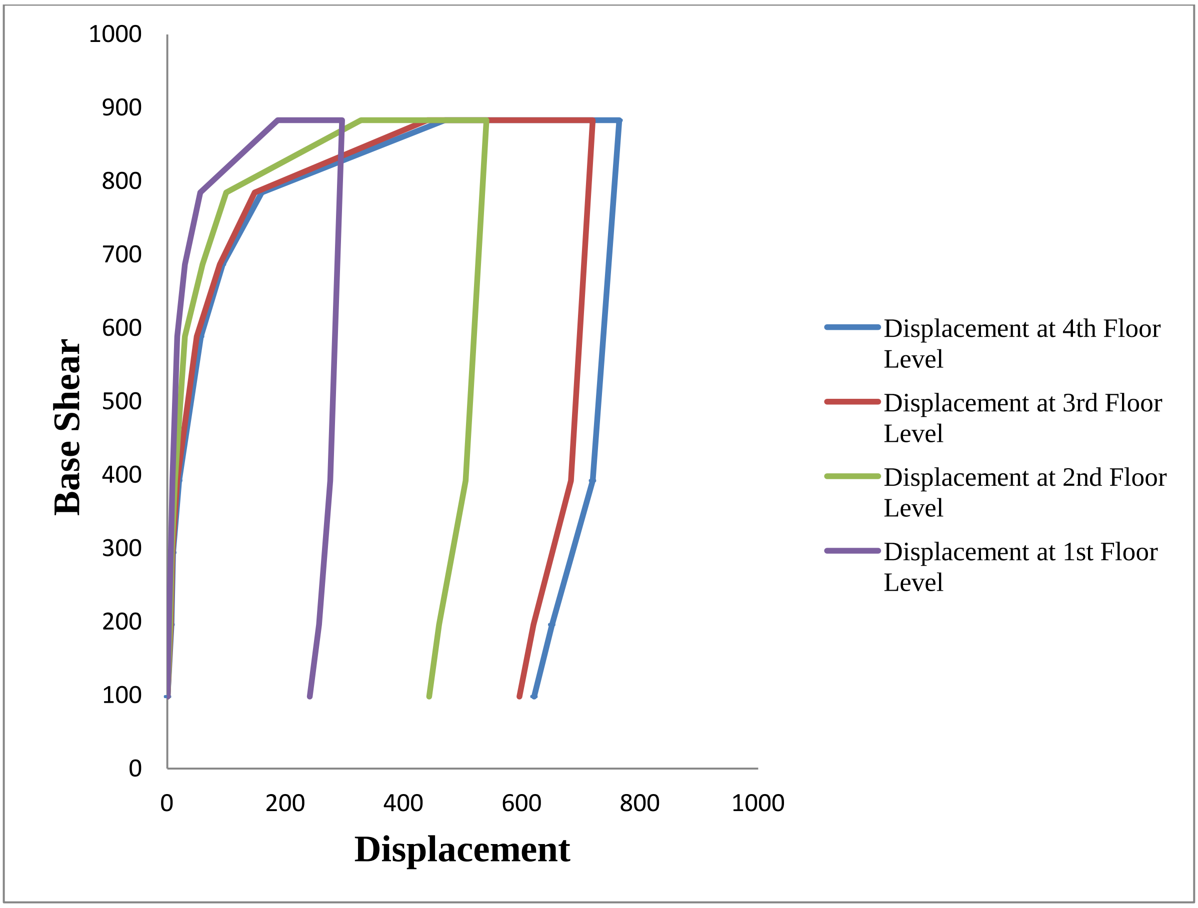
STEP 31

Fig 5.2(p): Step By Step Deformations for Pushover

5.3 COMPARISON BETWEEN THE SAP2000 MODEL AND THE EXPERIMENTAL RESULTS OF THE FRAME

5.3.1 EXPERIMENTAL RESULTS OF FRAME (Reddy. et.,al,2010)

Base shear v/s Floor Displacement (Pushover Curves) the pushover curves as obtained for extreme right side are plotted in **Fig 5.4**. As can be seen from the **Fig 5.4**, the maximum displacement has been obtained as 765mm. The average top drift is therefore equal to round 4% of the total height of the building.



5.3 Combined Pushover Curve from Experimental Data (Reddy. et., al,2010)

5.3.2 COMPARISON BETWEEN THE ANALYSIS AND THE EXPERIMENTAL RESULTS OF THE FRAME

The behaviour of the frame has been observed to be linear up to the value of base shear around 235 KN, whereas the structure has been found to be linear up to the value of base shear 300 KN in case of experiment. At this point the flexural tension cracks at the base of the columns depicting reduction in stiffness have been observed.

After reaching a base shear value of approximately 430 KN, the cracks at the base of the columns have been found to open wider and failures at other location like beams and beam – column joints started. Whereas in case of experiment at a base shear value of approximately 500 KN, the cracks at the base of the columns have been observed and failures at beam – column joints start to show up. As a result the stiffness of the frame further goes down, as can be seen from the pushover curve.

After reaching the base shear values of 670 KN, displacements have found to be increasing at fast rate whereas in case of experiment at the base shear values of 700 KN, the joints of the frame have found to be displaying rapid degradation and the inter storey drift increasing rapidly.

Maximum deflection has been found to be more than 510mm at 905 KN. Maximum deflection is more than 770mm at 880KN experimentally.

Variety of failures like beam-column joint failure, flexural failures and shear failures have been observed almost in the same way as seen in the case of experiment.

Prominent failures shown by both models have been the joint failures. Also the severe damages have been observed at joints of lower floors whereas moderate damages have been observed in first and second floors. Minor damage is seen at floor level in both the cases.

CHAPTER – 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 GENERAL

In the present study, the non-linear response of RCC frame using SAP2000 under the loading has been carried out with the intention to study the relative importance of several factors in the non-linear analysis of RC frames.

6.2 CONCLUSIONS

The main observations and conclusions drawn are summarized below:
The frame behaved linearly elastic up to a base shear value of around 235 KN. At the value of base-shear 670KN, it depicted non-linearity in its behaviour. Increase in deflection has been observed to be more with load increments at base-shear of 670 KN showing the elasto-plastic behaviour.

The joints of the structure have displayed rapid degradation and the inter storey deflections have increased rapidly in non- linear zone. Severe damages have occurred at joints at lower floors whereas moderate damages have been observed in the first and second floors. Minor damage has been observed at roof level.

The frame has shown variety of failures like beam-column joint failure, flexural failures and shear failures. Prominent failures are joint failures. Flexural failures have been seen in beams due to X-directional loading.
It has been observed that the top storey experienced major damages in this case opposite to the case of frame.

Micro cracks have been observed to appear even when the frame is in its elastic zone. The cracks have been found increasing with the increase in deflections

6.3 RECOMMENDATIONS

The literature review and analysis procedure utilized in this thesis has provided useful insight for future application of SAP2000 for analysis. It helps in comparing the results with experimental results data. Modelling the RCC frame in SAP2000 software gives good results which can be included in future research.

6.4 FUTURE SCOPE

In the present study frame has been studied under monotonic loads. The frame can be studied under cyclic-loading to monitor the variation in load-deflection curves at given time history.

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