EVALUTING THE PERFORMANCE LEVEL OF RCC FRAME STRUCTURE BY PERFORMANCE BASED ANALYSIS USING SAP 2000

A project report
In partial fulfilment of the requirements
for the award of

MASTER OF TECHNOLOGY
IN
STRUCTURE ENGINEERING

Submitted by:
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Submitted to:
DEPARTMENT OF CIVIL ENGINEERING
DELHI TECHNOLOGICAL UNIVERSITY
BAWANA ROAD, DELHI
2016

<u>DECLARATION</u>

I do here certify that

This project report entitled **"EVALUTING THE PERFORMANCE LEVEL OF RCC FRAME STRUCTURE BY PERFORMANCE BASED ANALYSIS USING SAP 2000"** submitted to Delhi Technological University, is a bonafide record of work done by me under the supervision of **Dr. Nirandra Dev.** The work contained in this report is original and has not been submitted to any other institute for any degree or diploma.

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To the best of my knowledge, the matter embodied in this Project review report has not been submitted to any other university/ institute for award of any Degree or Diploma.

Dr. Nirendra Dev
(Head of the Department)

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ABSTARCT

In every part of the world earthquakes are very common. Geographical figures of India show that almost 54% of the land is at risk to earthquakes. A report by the World Bank & United Nations estimates that about 200 million city inhabitants in India will be exposed in few years to earthquake. Due to the earthquakes, excessive destruction of infrastructure and buildings can be caused.

Increasingly, the non-linear analysis are a popular and relatively new and powerful way for seismic performance evaluation of new and existing building structures. Persistent hard work to resolve the variances between the actual observed performance and the expected performance of building structures is needed. It is expected that on the structural system and its components, the pushover analysis will provide sufficient data on seismic demands imposed by the design ground motion. The main objective of present study is to find the performance of building structure under earthquake using performance based seismic design analysis.

Nonlinear (Pushover) static analysis is method to evaluate the performance level of building. In this report, pushover analysis is carried out for a 9 stories building situated in ZONE IV to check the seismicity effect and performance level of a building by SAP2000. Pushover Analysis produces a Pushover curve consists of capacity spectrum, demand spectrum and performance point.

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CHAPTER- 1

INTRODUCTION

1.1 GENERAL

Due to increasing urbanization and spiraling population, there is high demand for building tall structures all over world. Earthquakes can cause excessive damages to those structures. Since earthquake forces are arbitrary in nature and unpredictable, the engineering tools need to be polished for analyzing structures under the action of these forces. Earthquake loads are essentially required to be carefully modeled so as to evaluate the real behavior of structure with a clear consideration of the expected damage but it should be regulated.

Earthquake can produce different shaking intensities at different locations in buildings and cause damage. Damage induced at these locations is also different. Thus, it is necessary to build structures which are earthquake resistance at a particular level of intensity of shaking a structure. Same magnitudes of earthquakes due to its varying intensity, results into dissimilar damaging effects in different regions. Therefore, for different seismic intensities, it is necessary to study variations in seismic behavior of multistoried RC framed structures in terms of various responses such as lateral displacements and base shear. It is also important to understand the seismic behavior of building structures with similar layout under earthquake of different intensities.

Capacity

The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. The mathematical model of the structure is modified to account for reduced resistance of yielding components. A lateral force distribution is again applied until predetermined limit is reached. Pushover capacity curves approximate how structure behaves after exceeding the elastic limits.

Demand

Ground motions during an earthquake produce complex horizontal displacement patterns in structure that may vary with time. For nonlinear method it is easier and more direct to use a set of lateral displacement as a design condition for a given structure and ground motion, the displacement is an estimate of the maximum expected response of the building during ground motion.

Performance level

The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range (Fig 1(a)), then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, Figure 1(b), then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.

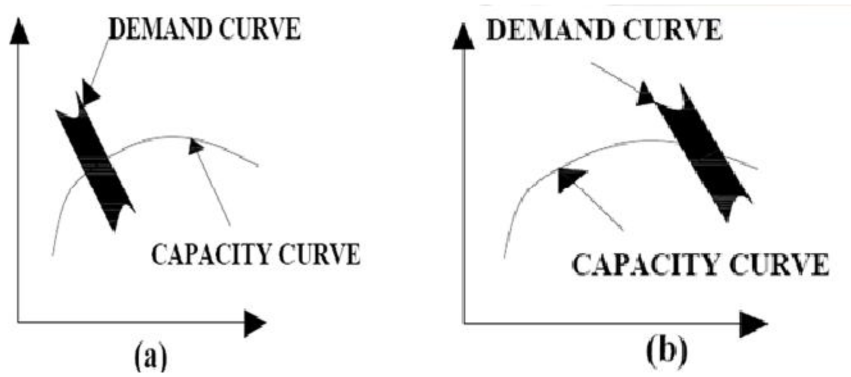

Fig.1 Typical seismic Demand vs. Capacity
(a) Safe design (b) Unsafe design

1.2 OBJECTIVES AND METHODOLOGY OF RESEARCH

My research project aims at doing seismic evaluation for the building using nonlinear static analysis method.

Taking the above results into consideration, our objective is to:

(i) Analyze the seismic performance of the existing structure with more degree of accuracy by using Non-linear Static Analysis Method.

(ii) Simulate the structure in accordance to the design generated by SAP2000 v18 and run Pushover analysis for the limiting case of the structure to generate a pushover curve.

(iii) Find the target displacement of the structure by using Idealized Force-Displacement Curve and Displacement Coefficient Method in accordance with ASCE 41-06.

(iv) Studying the behavior of the structure when subjected to the Pushover Analysis by limiting the maximum displacement of the top node to the calculated target displacement.

1.3 Layout of project

This report presents the method of analysis for evaluating the performance level of rcc frame structure by non linear pushover analysis with help of sap software.The report dividing in following chapters:

Chapter 1: Presents an introduction of report topic, objective and methodology.

Chapter 2: Various literatures reffered for the study are briefly presented. Past works and current developments in the area of project by various researchers are summerised.

Chapter 3: In this chapter different types of analysis and their limitations are discussed focusing on Pushover analysis. Also, Different guidelines for performance based analysis are mentioned.

Chapter 4: Design basis and analysis methodology focusing on the underlaying theory for 9story building through SAP 2000. Result obtained from study are presented.

Final conclusion is discussed.

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CHAPTER-2

LITERATURE REVIEW

2.1 GENERAL

To provide a detailed review of the literature related to modeling 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

Dakshes J. Pambhar et al.,(2012) explained the Performance Based Seismic Engineering is the modern approach to earthquake resistant design. It is limit-states design extended to cover complex range of issues faced by earthquake engineers. Two typical new R.C.C. buildings were taken for analysis: G+4 and G+10 to cover the broader spectrum of low rise & high rise building construction. Different modeling issues were incorporated through nine model for G+4 building and G+10 building were; bare frame (without infill), having infill as membrane, replacing infill as a equivalent strut in previous model. All three conditions for 2×2, 3×3, 4×4 bays. Comparative study made for bare frame (without infill), having infill as membrane, replacing infill as an equivalent strut. From the results for G+4 and G+ 10 storeys in bare frame without infill having lesser lateral load capacity (Performance point value) compare to bare frame with infill as membrane and bare frame with infill having lesser lateral load capacity compare to bar frame with equivalent strut. He conclude that as the no of bays increases lateral load carrying capacity increases but with the increase in bays corresponding displacement is not increases. Also conclude that as the no of storey increases lateral load carrying capacity does not increase but corresponding displacement increases.

Pwint Thandar Kyaw Kyaw et al., (2010) explained the performance based design, nonlinear lateral resistances of the building frame system, combination of ductility and overstrength of the system, are offering major share of lateral load resisting capacity. It comes out from ductility of constituent materials and components, plastic hinging capability of the frame system and uncertainty in probable strength of materials and overstrength of components. Therefore, nonlinear resistance natures of the designed buildings may be different from one to another. In this study, nonlinear resisting behavior of selected building designed and to be constructed according to local practice is evaluated using pushover analysis. For seismic design, it is also important to predict inelastic displacement (maximum lateral displacement) of the structures due to severe earthquakes. This paper aims to study the nature of inelastic deformation of RC framed buildings by carrying out Pushover analysis, modeling three-dimensional frames building located in seismic zone 2A. Total of seven different case studies are performed. It is found that displacement amplification factor Cd depends on ductility and over strength factors. The available ductility μ values are lower than expected and it is showing need of modification in design practice to synchronize between selected R values and nonlinear displacement capability of the system. It is also found that reduced base shear from elastic analysis is much lower than the actual frame's elastic limit. And building structural system is showing linear behaviors in lateral resisting although secant moduli are different about 0.7%. It means designs of non-plastic region using demand forces corresponding to specified over strength at plastic regions are uncertain in safety performance of selected building.

In this study, Pushover Analysis (Static Non-linear Analysis) was carried out, modeling three-dimensional frame buildings located in seismic zone 2A. Seven types of case studies were considered depending on construction practice and detailing. The different percentages of building height of displacement magnitude are used as target displacement at each case study. The result shows that displacement amplification factor Cd varied mostly with the changes in system ductility factor (i.e, the extent of yield displacement and maximum inelastic deformation). The values of Cd / R are generally within the code prescribed limit for building frame system according to UBC 1997, ASCE 7-05, Euro code 8, Mexico, New Zealand and NBC of Canada 2005 values.Although non-linear static analysis carrying on the end of linear elastic analysis, by the formation of yielding mechanisms in structural members to form inelastic deformation, available ductility factors μ are inconsistent with the response reduction factor R which was used in linear static analysis. Therefore, linear elastic analyses take into account only for the design base shear level.

A. Kadid and A. Boumrkik et al., (2008) summarized the Boumerdes 2003 earthquake which has devastated a large part of the north of Algeria has raised questions about the adequacy of framed structures to resist strong motions, since many buildings suffered great damage or collapsed. To evaluate the performance of framed buildings under future expected earthquakes, a non linear static pushover analysis has been conducted. To achieve this objective, three framed buildings with 5, 8 and 12 stories respectively were analyzed. The results obtained from this study show that properly

Designed frames will perform well under seismic loads.

The performance of reinforced concrete frames was investigated using the pushover Analysis. These are the conclusions drawn from the analyses:

- ❖ The pushover analysis is a relatively simple way to explore the non linear behavior of buildings
- ❖ The behavior of properly detailed reinforced concrete frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns. Most of the hinges developed in the beams and few in the columns but with limited damage
- ❖ The causes of failure of reinforced concrete during the Boumerdes earthquake may be attributed to the quality of the materials of the used and also to the fact that most of Buildings constructed in Algeria are of strong beam and weak column type and not to the intrinsic behavior of framed structures.
- ❖ The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behavior of structures.
- ❖ It would be desirable to study more cases before reaching definite conclusions about the behavior of reinforced concrete frame buildings.
- ❖ **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:

- ❖ 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.
- ❖ 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.
- ❖ 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.
- ❖ 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.

- ❖ 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.
- ❖ 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.
- ❖ 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.

R. Hasan, L. Xu, D.E. Grierson et al.,(2002) explained the simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. The technique is based on the conventional displacement method of elastic analysis. Through the use of a plasticity-factor that measures the degree of plastification, the standard elastic and geometric stiffness matrices for frame elements (beams, columns, etc.) are progressively modified to account for nonlinear elastic–plastic behavior under constant gravity loads and incrementally increasing lateral loads. The behavior model accounts for material inelasticity due to both single and combined stress states, and provides the ability to monitor the progressive plastification of frame elements and structural systems under increasing intensity of earthquake ground motion.

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:

- accurate values of displacement and ductility demands, and
- 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.

Ashraf Habibullah, S.E. and Stephen Pyle, S.E. et al.,(1998) presented the Practical Three Dimensional Nonlinear Static Pushover Analysis The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

CHAPTER-3

PERFORMANCE BASED ANALYSIS

3. SEISMIC ANALYSIS

It is necessary to carry out seismic analysis of the structure for the determination of seismic responses of the structure using different available methods.

The analysis can be performed on the basis of external action, the behavior of structure or structural materials, and the type of structural model selected. The analysis can be further classified as:

a) Linear Static Analysis

b) Nonlinear Static Analysis

c) Linear Dynamic Analysis

d) Nonlinear Dynamic Analysis

For regular structure with limited height, linear static analysis or equivalent static method can be used. Linear dynamic analysis can be performed by response spectrum method.

The significant difference between linear static and linear dynamic analysis is the,

- level of the forces and
- Distribution along the height of structure

Nonlinear static analysis is an enhancement over linear static or dynamic analysis in the manner that it allows inelastic behavior of structure.

A nonlinear dynamic analysis is the only method to define the actual behavior of a structure during an earthquake. This method is based on the direct numerical integration of the differential equations of motion, considering the elasto-plastic deformation of the structural element.

3.1 TYPE OF ANALYSIS

3.1.1 Equivalent Static Analysis

This method does not require dynamic analysis, but it account for the dynamics of building in an approximate manner. The static method is the simple method which requires less computational efforts. It is based on formulate given in the code of practice. Initially, the design base shear is computed for the whole building, and then it is distributed along the height of the building. The lateral forces at each floor level obtained are then distributed to individual’s lateral load resisting elements.

3.1.2 Nonlinear Static Analysis

In this approach, analysis is carried out under permanent vertical loads and gradually increasing lateral loads which determines the deformation and damage pattern of the structure.It is the method of seismic analysis in which behavior of the structure is characterized by capacity curve which represents the relation between base shear force and the displacement of the roof. This method is also known as Pushover Analysis.

3.1.3 Linear Dynamic Analysis

Linear dynamic analysis method is the Response spectrum method. In this method the peak response of structure during an earthquake is determined directly from the earthquake response. This method is quite accurate for structural design applications.

3.1.4 Nonlinear Dynamic Analysis

It is also known as Time history analysis. It is an important procedure for structural seismic analysis especially when the evaluated structural response is nonlinear. To perform such an analysis, a representative earthquake time history is required for a structure being evaluated. Time history analysis is a step-by step analysis of the dynamic response of a structure to a specified loading that may vary with time. Time history analysis is used to determine the seismic response of a structure under dynamic loading of representative earthquake.

3.2 DIFFERENCE BETWEEN NONLINEAR DYNAMIC ANALYSIS AND STATIC ANALYSIS

Nonlinear dynamic analysis methods generally provide more realistic models of structural response to strong ground shaking and, thereby, provide more reliable assessment of earthquake performance than nonlinear static analysis. Nonlinear static analysis is limited in its ability to capture transient dynamic behavior with cyclic loading and degradation. Nevertheless, the nonlinear static procedure provides a convenient and fairly reliable method for structures whose dynamic response is governed by first-mode sway motions. One way to check this is by comparing the deformed geometry from a pushover analysis to the elastic first-mode vibration shape. In general, the nonlinear static procedure works well for low-rise buildings (less than about five stories) with symmetrical regular configurations. FEMA 440, FEMA 440A, and NIST (2010) provide further details on the simplifying assumptions and limitations on nonlinear static analysis. However, even when the nonlinear static procedure is not appropriate for a complete performance evaluation, nonlinear static analysis can be an effective design tool to investigate aspects of the analysis model and the nonlinear response that are difficult to do by nonlinear dynamic analysis. For example, nonlinear static analysis can be useful to (1) check and debug the nonlinear analysis model, (2) augment understanding of the yielding mechanisms and deformation demands, and (3) investigate alternative design parameters and how variations in the component properties may affect response.

3.3 THE ROLE AND USE OF NONLINEAR ANALYSIS IN SEISMIC DESIGN

While buildings are usually designed for seismic resistance using elastic analysis, most will experience significant inelastic deformations under large earthquakes. Modern performance based design methods require ways to determine the realistic behavior of structures under such conditions. Enabled by advancements in computing technologies and available test data, nonlinear analyses provide the means for calculating structural response beyond the elastic range, including strength and stiffness deterioration associated with inelastic material behavior and large displacements. As such, nonlinear analysis can play an important role in the design of new and existing buildings.

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Typical instances where nonlinear analysis is applied in structural earthquake engineering practice are to:

(1) Assess and design seismic retrofit solutions for existing buildings;

(2) Design new buildings that employ structural materials, systems, or other features that do not conform to current building code requirements;

(3) Assess the performance of buildings for specific owner/stakeholder requirements. If the intent of using a nonlinear analysis is to justify a design that would not satisfy the prescriptive building code requirements, it is essential to develop the basis for acceptance with the building code authority at the outset of a project. The design basis should be clearly defined and agreed upon, outlining in specific terms all significant performance levels and how they will be evaluated.

Once the goals of the nonlinear analysis and design basis are defined, the next step is to identify specific demand parameters and appropriate acceptance criteria to quantitatively evaluate the performance levels. The demand parameters typically include peak forces and deformations in structural and nonstructural components, story drifts, and floor accelerations. Other demand parameters, such as cumulative deformations or dissipated energy, may be checked to help confirm the accuracy of the analysis and/or to assess cumulative damage effects.

In contrast to linear elastic analysis and design methods that are well established, nonlinear inelastic analysis techniques and their application to design are still evolving and may require engineers to develop new skills. Nonlinear analyses require thinking about inelastic behavior and limit states that depend on deformations as well as forces. They also require definition of component models that capture the force-deformation response of components and systems based on expected strength and stiffness properties and large deformations. Depending on the structural configuration, the results of nonlinear analyses can be sensitive to assumed input parameters and the types of models used.

It is advisable to have clear expectations about those portions of the structure that are expected to undergo inelastic deformations and to use the analyses to

1. **Confirm the locations of inelastic deformations and**
2. **Characterize the deformation demands of yielding elements and force demands in non-yielding elements.**

In This regard, capacity design concepts are encouraged to help ensure reliable performance while nonlinear analyses can, in concept, be used to trace structural behavior up to the onset of collapse, this requires sophisticated models that are validated against physical tests to capture the highly nonlinear response approaching collapse.

Since the uncertainties in calculating the demand parameters increase as the structure becomes more nonlinear, for design purposes, the acceptance criteria should limit deformations to regions of predictable behavior where sudden strength and stiffness degradation does not occur.

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.The ATC-40 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for pushover analysis. These documents define force-deformation criteria for hinges used in pushover analysis. As shown in Figure 3(a), five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge and three points labeled IO, LS and CP are used to define the acceptance criteria for the hinge. (IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention respectively.) The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents.

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

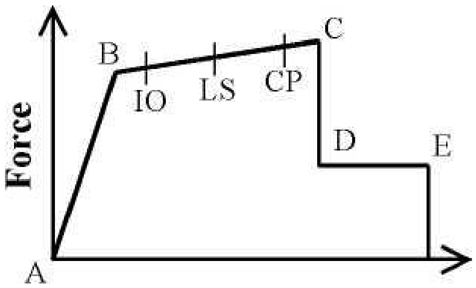


Fig. 3(a) Deformation
Force-Deformation for Pushover Hinge

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 modeling is involved in seismic hazard estimation. A computational scheme involves the following steps: delineation of seismic source zones and their characterization, selection of an appropriate ground motion attenuation relation and a predictive model of seismic hazard. Although these steps are region specific, certain standardization 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.

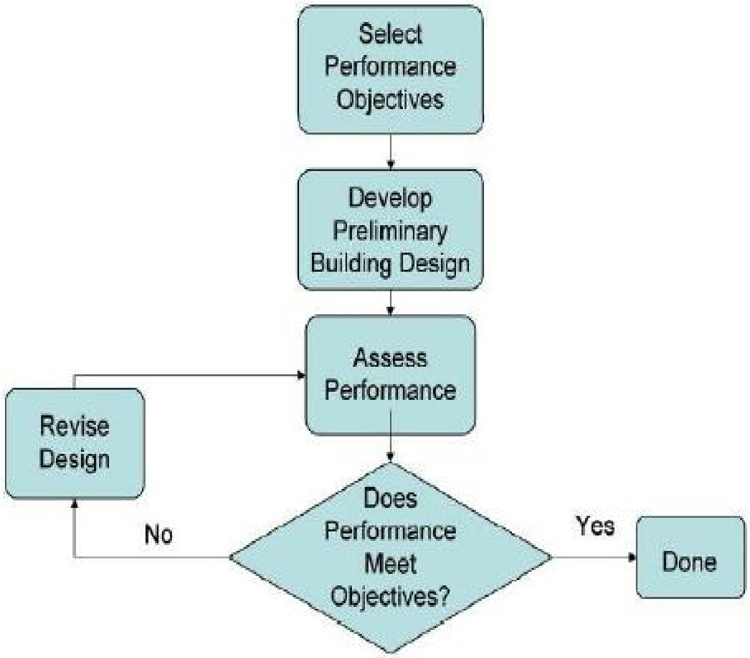


Fig. 3(b) Performance-Based Design Flow Diagram (ATC, 1997a)

3.4 PUSHOVER ANALYSIS

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 as shown in fig.3©

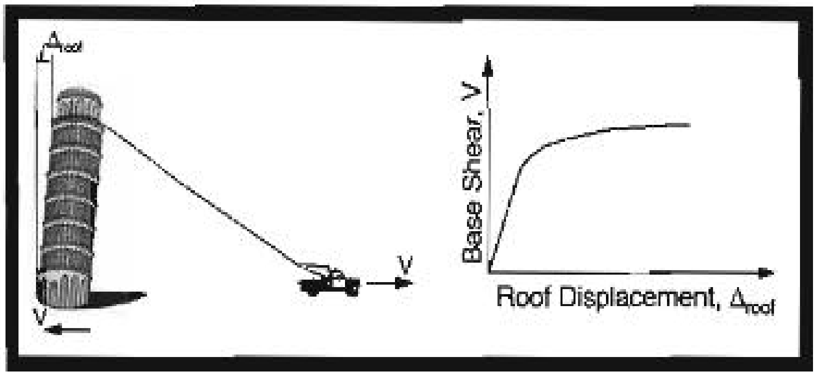


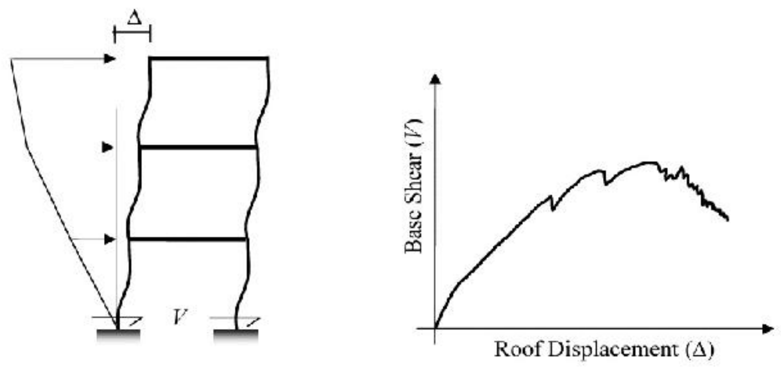
Fig. 3(c) force-displacement curve

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 observe in order to obtain useful results from such analysis.

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design. Pushover analysis is a performance based analysis. According to ATC 40, there are two key elements of a performance-based design procedure - demand and capacity. Demand is the representation of earthquake ground motion or shaking that the building is subjected to. In nonlinear static analysis procedures, demand is represented by an estimation of the displacements or a deformation that the structure is expected to undergo.Capacity is a representation of the structure's ability to resist the seismic demand. The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. Pushover analysis is performed by Displacement coefficient method/Capacity spectrum method.The Capacity Spectrum Method (CSM), a performance-based seismic analysis technique, can be used for a variety of purposes such as rapid evaluation of a large inventory of buildings, design verification for new construction of individual buildings, evaluation of an existing structure to identify damage states, and correlation of damage states of buildings to various amplitudes of ground motion. The procedure correlation of damage states of buildings to various amplitudes of ground motion. The procedure compares the capacity of the structure (in the form of a pushover curve) with the demands on the structure....Objective of Displacement coefficient method is to find target displacement which is the maximum displacement that the structure is likely to be experienced during the design earthquake.

3.5 PUSHOVER ANALYSIS PROCEDURE

Pushover analysis is a static nonlinear procedure in which the magnitude of the lateral load is increased monotonically maintaining a predefined distribution pattern along the height of the building (Fig. 3(d.1)). Building is displaced till the 'control node' reaches 'target displacement' or building collapses. The sequence of cracking, plastic hinging and failure of the structural components throughout the procedure is observed. The relation between base shear and control node displacement is plotted for all the pushover analysis (Fig. 3(d.2)). Generation of base shear – control node displacement curve is single most important part of pushover analysis. This curve is conventionally called as pushover curve or capacity curve. The capacity curve is the basis of 'target displacement' estimation.



(1) Building model (2) Pushover Curve

Fig. 3 (d) Schematic representation of pushover analysis procedure

So the pushover analysis may be carried out twice: (1) first time till the collapse of the building to estimate target displacement and (2) next time till the target displacement to estimate the seismic demand. The seismic demands for the selected earthquake (storey drifts, storey forces, and component deformation and forces) are calculated at the target displacement level. The seismic demand is then compared with the corresponding structural capacity or predefined performance limit state to know what performance the structure will exhibit. Independent analysis along each of the two orthogonal principal axes of the building is permitted unless concurrent evaluation of bi-directional effects is required.

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general, the centre of mass location at the roof of the building is considered as control node. For selecting lateral load pattern in pushover analysis, a set of guidelines as per FEMA 356 is explained in Section A.1.2. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behaviour.

Lateral Load Profile

In pushover analysis the building is pushed with a specific load distribution pattern along the height of the building. The magnitude of the total force is increased but the pattern of the loading remains same till the end of the process. Pushover analysis results (i.e., pushover curve, sequence of member yielding, building capacity and seismic demand) are very sensitive to the load pattern. The lateral load patterns should approximate the inertial forces expected in the building during an earthquake. The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as the members yield and stiffness characteristics change. It also depends on the type and magnitude of earthquake ground motion. Although the inertia force distributions vary with the severity of the earthquake and with time, FEMA 356 recommends primarily invariant load pattern for pushover analysis of framed buildings.

Several investigations (Mwafy and Elnashai, 2000; Gupta and Kunnath, 2000) have found that a triangular or trapezoidal shape of lateral load provide a better fit to dynamic analysis results at the elastic range but at large deformations the dynamic envelopes are closer to the uniformly distributed force pattern. Since the constant distribution methods are

incapable of capturing such variations in characteristics of the structural behaviour under earthquake loading, FEMA 356 suggests the use of at least two different patterns for all pushover analysis. Use of two lateral load patterns is intended to bind the range that may occur during actual dynamic response. FEMA
356 recommends selecting one load pattern from each of the following two groups:

1. Group – I:

i) Code-based vertical distribution of lateral forces used in equivalent static analysis (permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration).

ii) A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration (permitted only when more than 75% of the total mass participates in this mode).

iii) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (sufficient number of modes to capture at least 90% of the total building mass required to be considered). This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

2. Group – II:

i) A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.

ii) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

Instead of using the uniform distribution to bind the solution, FEMA 356 also allows adaptive lateral load patterns to be used but it does not elaborate the procedure. Although adaptive procedure may yield results that are more consistent with the characteristics of the building under consideration it requires considerably more analysis effort. Fig. 3(e) shows the common lateral load pattern used in pushover analysis.

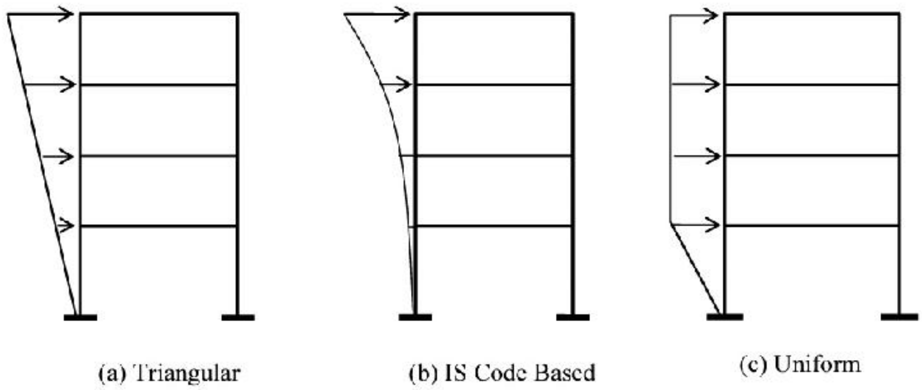


Fig. 3 (e) Lateral load pattern for pushover analysis as per FEMA 356 (Considering uniform mass distribution)

3.5.1 Target Displacement

Target displacement is the displacement demand for the building at the control node subjected to the ground motion under consideration. This is a very important parameter in pushover analysis because the global and component responses (forces and displacement) of the building at the target displacement are compared with the desired performance limit state to know the building performance. So the success of a pushover analysis largely depends on the accuracy of target displacement. There are two approaches to calculate target displacement:

(a) Displacement Coefficient Method (DCM) of FEMA 356 and

(b) Capacity Spectrum Method (CSM) of ATC 40.

Both of these approaches use pushover curve to calculate global displacement demand on the building from the response of an equivalent single-degree-of-freedom (SDOF) system. The only difference in these two methods is the technique used.

3.6 ADVANTAGES OF PUSHOVER ANALYSIS

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. The expectation from pushover analysis is to estimate critical response parameters imposed on structural system and its components as close as possible to those predicted by nonlinear dynamic analysis. Pushover analyses provide information on many response characteristics that cannot be obtained from an elastic static or elastic dynamic analysis..These are:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the 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 behavior of structural system.
- Consequences of the strength detoriation of the individual elements on the 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 discontinuities 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 system, all the connections, the stiff nonstructural elements of significant strength, and the foundation system.
- Estimates of inter-story drifts and its distribution along the height.

Pushover analysis also exposes design weaknesses that may remain hidden in an Elastic analysis. These are story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle members.

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, interstory drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations), The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behavior.

The last item is the most relevant one as the analytical model incorporates all elements, whether structural or non structural, that contribute significantly to the lateral load distribution. Load transfer through across the connections through the ductile elements can be checked with realistic forces; the effects of stiff partial-height infill walls on shear forces in columns can be evaluated; and the maximum overturning moment in walls, which is often limited by the uplift capacity of foundation elements can be estimated.

3.7 LIMITATIONS OF PUSHOVER ANALYSIS

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

1. Incorporation of torsional effects (due to mass, stiffness and strength irregularities).
2. 3-D problems (orthogonality effects, direction of loading, semi-rigid diaphragms, etc)
3. Use of site specific spectra.
4. Cumulative damage issues.
5. Most importantly, the consideration of higher mode effects once a local mechanism has formed.
6. There are good reasons for advocating the use of the inelastic pushover analysis for demand prediction, since in many cases it will provide much more relevant information

that an elastic static or even dynamic analysis, but it would be counterproductive to advocate this method as a general solution technique for all cases;

7. The pushover analysis is a useful, but not in fallible, tool for accessing inelastic strength and deformation demands and for exposing design weaknesses.
8. Its foremost advantage is that it encourages the design engineer to recognize important seismic response quantities and to use sound judgment concerning the force and deformation demands and capacities that control the seismic response close to failure, but it needs to be recognized that in some cases it may be provide a false feeling of security if its shortcomings and pitfalls are not recognized.
9. It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others. Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important.
10. Thus performance of pushover analysis primarily depends upon choice of material models included in the study.

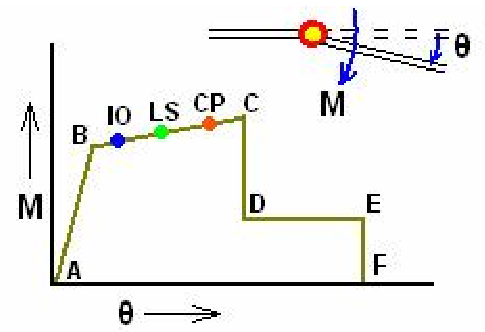
Since the pushover analysis is approximate in nature and is based on static loading, as such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that occur in a structure subjected to severe earthquakes, and it may significantly from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

3.8 THE HINGES

Hinges are points on a structure where one expects cracking and yielding to occur in relatively higher intensity so that they show high flexural (or shear) displacement, as it approaches its ultimate strength under cyclic loading.

These are locations where one expects to see cross diagonal cracks in an actual building structure after a seismic mayhem, and they are found to be at the either ends of beams and columns, the 'cross' of the cracks being at a small distance from the joint – that is where one is expected to insert the hinges in the beams and columns of the corresponding computer analysis model. Hinges are of various types – namely, flexural hinges, shear hinges and axial hinges. The first two are inserted into the ends of beams and columns. Since the presence of masonry infills have significant influence on the seismic behaviour of the structure, modelling them using equivalent diagonal struts is common in PA, unlike in the conventional analysis, where its inclusion is a rarity. The axial hinges are inserted at either ends of the diagonal struts thus modelled, to simulate cracking of infills during analysis.

Basically a hinge represents localised force-displacement relation of a member through its elastic and inelastic phases under seismic loads. For example, a flexural hinge represents the moment-rotation relation of a beam of which a typical one is as represented in Fig.3(f). AB represents the linear elastic range from unloaded state A to its effective yield B, followed by an inelastic but linear response of reduced (ductile) stiffness from B to C. CD shows a sudden reduction in load resistance, followed by a reduced resistance from D to E, and finally a total loss of resistance from E to F. Hinges are inserted in the structural members of a framed structure typically as shown in Fig.3(g). These hinges have non-linear states defined as ‘Immediate Occupancy’ (IO), ‘Life Safety’ (LS) and ‘Collapse Prevention’ (CP) within its ductile range. This is usually done by dividing B-C into four parts and denoting IO, LS and CP, which are states of each individual hinges (in spite of the fact that the structure as a whole too have these states defined by drift limits). There are different criteria for dividing the segment BC. For instance, one such specification is at 10%, 60%, and 90% of the segment BC for IO, LS and CP respectively (Inel & Ozmen, 2006).



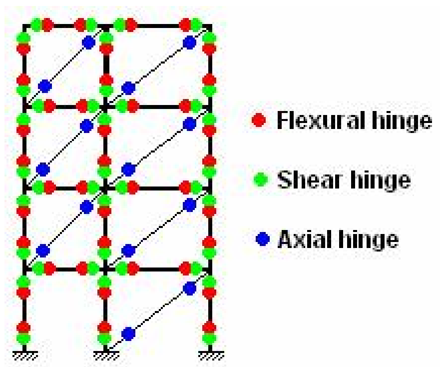


Fig.3 (f) A Typical Flexural Hinge Property, showing IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention)

Fig.3 (g) Typical Locations of Hinges in a Structural Model

3.8.1 VARIOUS HINGE MODELS OF PUSHOVER ANALYSIS

These are the various hinge models used in pushover analysis:

According to the Ceroni, (2007) the rotational capacity of the element can be defined as the plastic fraction èp of the rotation èu at failure. It can be evaluated as the difference between the rotation at the maximum moment and the rotation at the steel yielding Θy:

$\Theta p = \Theta u - \Theta y$ 3.1

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

$\Theta p = \Theta p,c - \Theta p,fix$ 3.2

The fixed end rotation $\Theta p,fix$, 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 $\Theta p,c$ 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:

$Lp = \Theta p/øp$

3.3

Øp=øu-øy

3.4

$\Theta p = \Theta u - \Theta y = (øu-øy).Lp$

3.5

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

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

3.6

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

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

$Lp^{1}=6.1(L/H)^{0.43}(f_t/f_y-1)^{0.65}.\mathcal{E}^{-0.32}(1+N/N_O)^{-1.83}$
3.7

$Lp^{II}=5.d_b.(f_t/f_y-1)^{0.2}$
3.8

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

Lp = 0.08L + 6db
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
3.10

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

for monotonic loads: Lp = 0.18. Ls + 0.025. f y . db
3.11

for cyclic loads: Lp = 0.08. Ls + 0.017. f y . db
3.12

where Ls is the shear span.

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

ϴU=ϴY + (Øu-øy)Lp.{1-0.5Lp/Ls}
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, èy, 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:

$$\varepsilon_{cu}=0.004+1.4\rho_s f_{yh} \varepsilon_{su}/f_{cc} \qquad 3.14$$

where ε_{cu} is the ultimate concrete compressive strain, ε_{su} is the steel strain at the maximum tensile stress, ρ_s is the volumetric ratio of confining steel, f_{yh} is the yield strength of transverse reinforcement, and f_{cc} is the peak confined concrete compressive strength.

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

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.

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

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.

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.

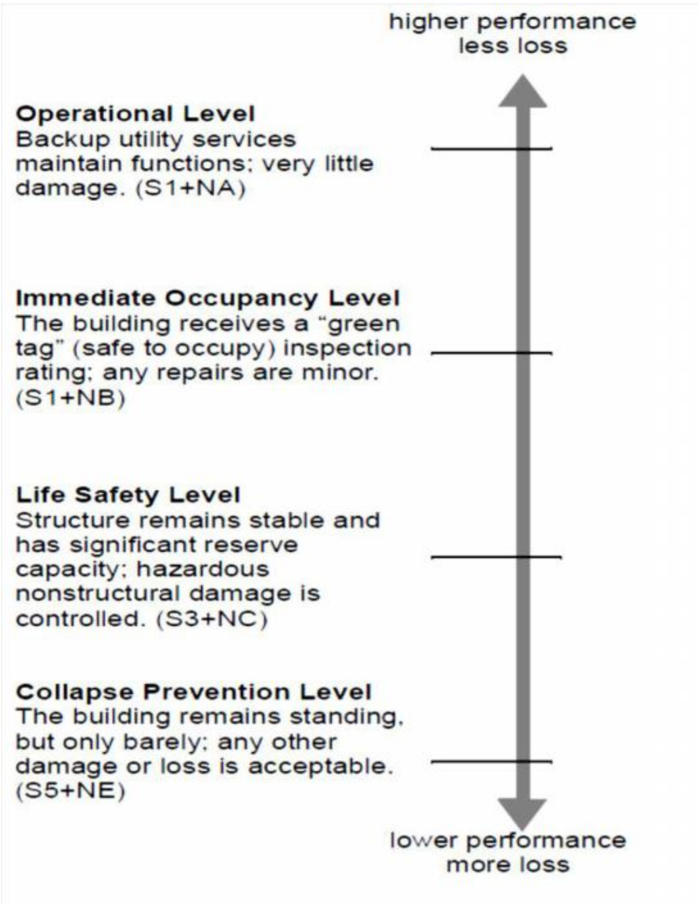


Fig. 3(h) Building Performance Levels (ATC, 1997a)

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.

3.9.1 STRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

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.

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.

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.

3.9.2 STRUCTURAL PERFORMANCE RANGES (ATC, 1997a)

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.

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.

3.9.3 NONSTRUCTURAL PERFORMANCE LEVELS (ATC, 1997a)

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.

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.

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.

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

Problem formulation and Methodology

4.1 GENERAL

Now a days different types of structural systems are available, from last many decades the most common structure system was beam column system with few shear walls located in circulation areas. The system is good in terms of performance in gravity and lateral loading for medium story height buildings.

In the present study, I have chosen the special moment resisting RCC frame with G+8 stories office building, has been modeled in sap2000 to undertaken nonlinear analysis. Beams and columns are modeled as nonlinear frame element with lumped plasticity at start and end of each RC elements. Sap2000 provide default-hinge properties and recommended P-M2-M3 hinge for columns and hinge M3 for beams as described in FEMA 356.

4.2 BUILDING DISCRIPTION

In this report, 9 story office building with floor to floor height of 3.9 m is taken for analysis.The size of building is 18 m in width and 36 m in length. The structural system used in building is special moment resisting frame (beam column system).
The building is situated in earth quake zone IV.The grade of concrete is M25 for structural components. The 230 mm brick wall at outer peripheral beams only. All inner area has no permanent partition walls, there is only movable partition proposed. The dead load of structure is calculated by software with 2500kg/cu.m (density of reinforced concrete). The superimposed dead load for floor finishing and false ceiling is 200kg/sq.m as floor load and for 230 mm brick work 500kg/sq.m/r.m.as member load considered.
The live load 400 kg/sq m as floor load considered.

4.2.1 Building geometry

Length of building (plan)	36 m
Width of building (plan)	18 m
Plan shape	Rectangular
Column to column spacing in X - dir.	6 m
Nos. of bays in X - dir.	6
Column to column spacing in Y- dir.	6 m
Nos. of bays in Y - dir.	3

Nos. of floors	9
Floor to floor height	3.9

4.2.2 Material specifications

Concrete

M25	M25
Beam and slab	Columns

Reinforcement

Fe 415	Fe 415
For longitudinal bars of beams, columns and slabs	For transverse bars of beams, columns.

Proposed member sizes (mm)

Column sizes	450x900
Beams	450x600
Slab thickness	185 mm

4.2.3 Gravity Loads:

Dead loads

Self-weight of structure	By software
50 mm thick Floor finishing + False ceiling	2.0 KN / sq.m
230 mm thick brick wall (3m height below beam)	5 KN /sq.m/r.m

Live loads

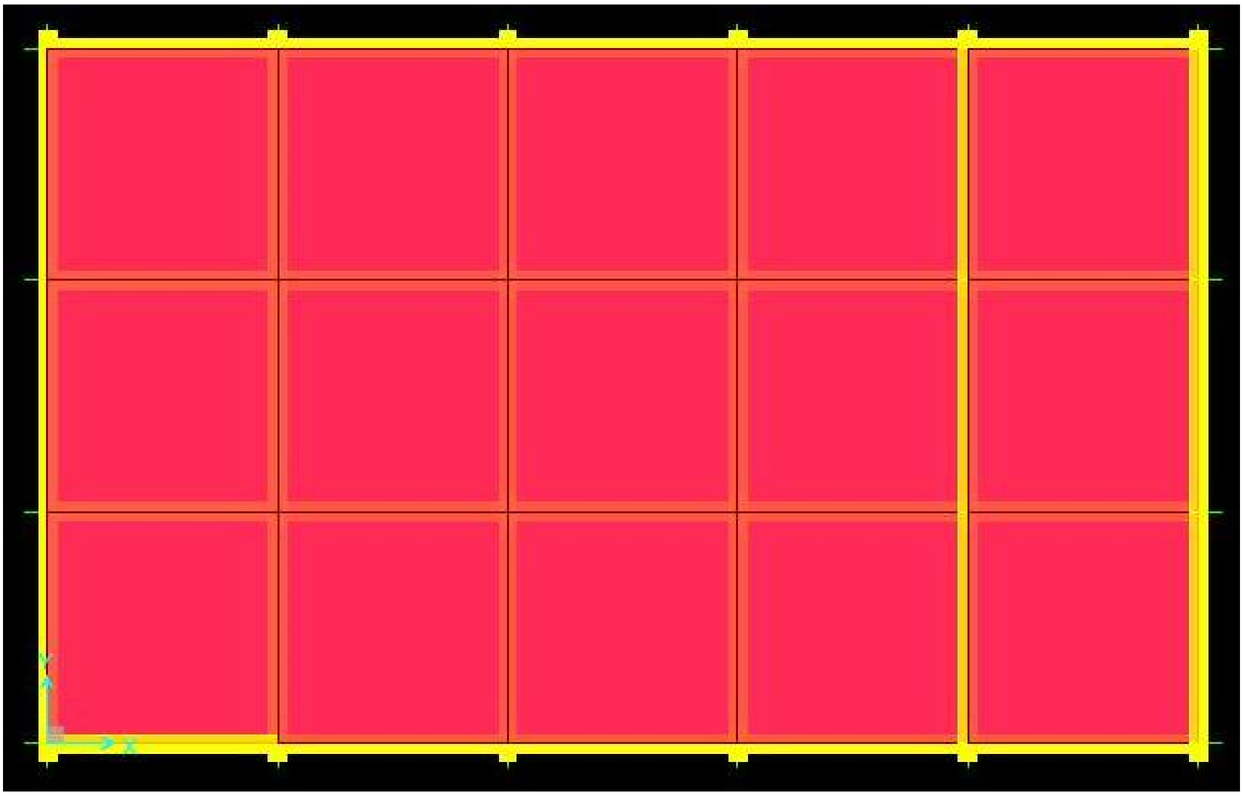
Live load on floors(office area)	4 KN / sq.m

4.2.4 Lateral Loads

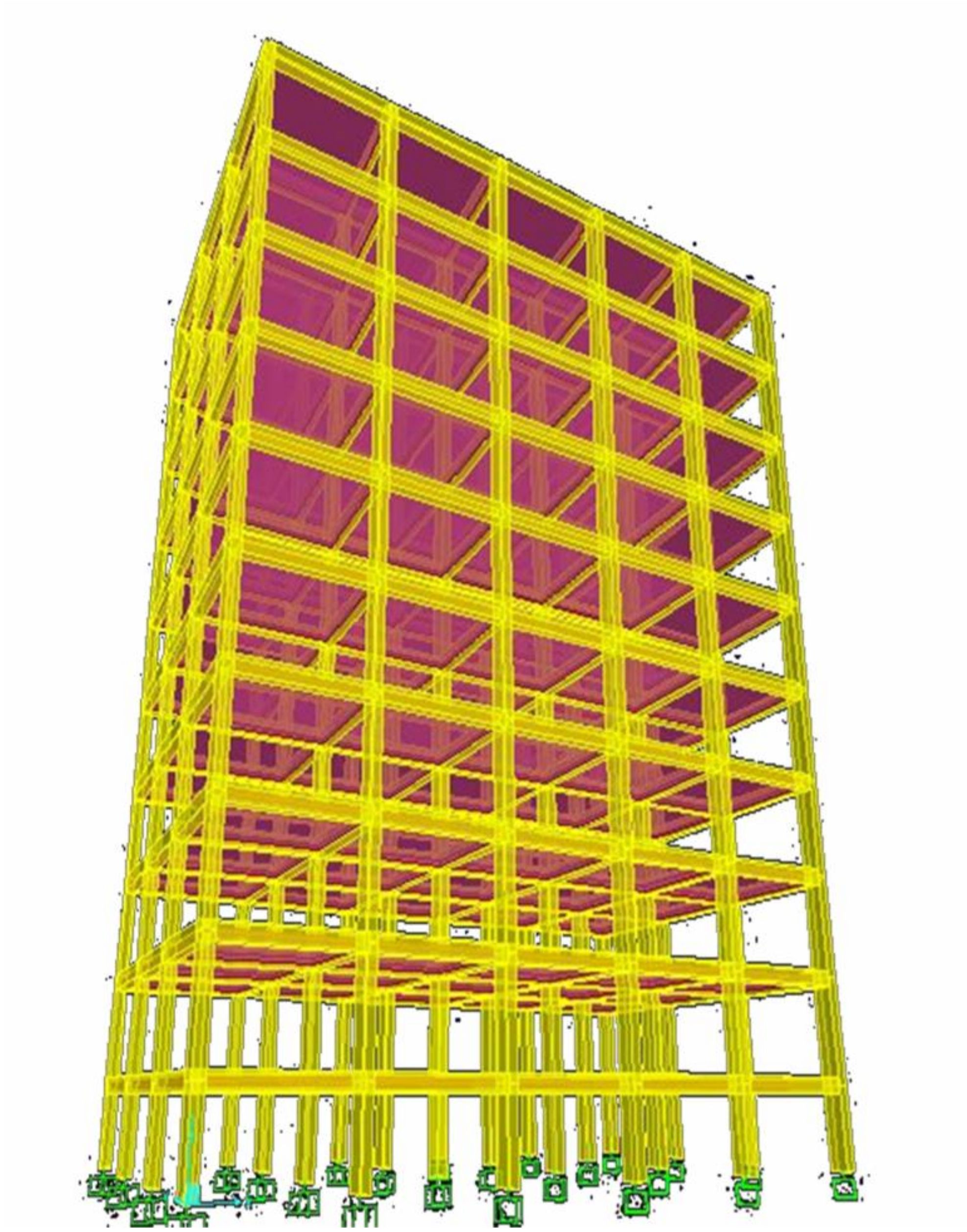
Seismic Load Parameters (As per IS: 1893 2002)

Use of building	Office purpose
Earthquake Zone	VI

Zone factor (Z)	0.24
Response reduction factor (R)	5
Importance factor (I)	1
Soil type	Hard
Time period (0.075 x $H^{0.75}$)	1.150 seconds



Plan



Model 3D view

4.3 BASIC CHECK FOR MODEL

STATIC BASE SHEAR CHECK

Design Seismic Base Shear =			**A_hxW**				
Design Horizontal Seismic cofficient (A_h) =				$\frac{Z\ I\ S_a}{2\ R\ g}$			
W =	Seismic Weight (DL + kLL) =				77343.00	KN	
	k = 0.5 for live load more than 3 KN/sq.m						
h =	height of building (m)				38.1	m	
T =	Time period		$0.075\ h^{0.75}$		1.150	sec	
	Average response acceleration coefficient						
S_a/g =	1/T						
S_a/g =					0.869		
Z =	Zone factor				0.24		
I =	Importance factor				1		
R =	Response reduction factor				5		
A_h =	Design Horizontal Seismic coefficient				0.0209		
		Design Seismic Base Shear by manual calculation =			**1614**	KN	
		Design Seismic Base Shear calculated by sap software =			**1605**	KN	0k

LOADING CHECK

	(DL+0.5LL)	**LOAD**	
LOAD	**Area = 49623 Sq.m**	**Per sq.m.**	
DEAD	49623.00	9.19	ok
FF	10800.00	2.00	ok
WALL	6120.00	1.13	ok
LIVE	10800.00	4.00	ok
	77343.00	16.32	

The total applied load should transfer on foundation means assigned base restraint. The total base reaction for basic load case like: dead & live load etc. should matched with total applied gravity load. Or the individual column base reaction should matched manually calculated load, based on tributary area of column to confirm the accuracy of modelling and loading as well.

Same as the static base shear calculated by software should be matched with the base shear calculated manually. This check again is to confirm the accuracy of model in terms of defined seismic parameters.

Based on the above checks we can validate the software results.

4.4 SAP NON LINEAR ANALYSIS STEPS:

The following steps are included in the pushover analysis. Steps 1 through 4 discuss creating the computer model, step 5 runs the analysis, and steps 6 through 10 review the pushover analysis results.

(i) Create the basic computer model (without the pushover data) in the usual manner using the graphical interface of SAP2000 makes this a quick and easy task.

(ii) Define properties and acceptance criteria for the pushover hinges.

(iii) The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. These built in properties can be useful for preliminary analyses, but user-defined properties are recommended for final analyses. This example uses default properties.

(iv) Locate the pushover hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and hinge locations.

(v) Define the pushover load cases. In SAP2000 more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis. Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or they can be displacement controlled, that is, pushed to a specified displacement. Typically a gravity load pushover is force controlled and lateral pushovers are displacement controlled. SAP2000 allows the distribution of lateral force used in the pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case.Here how the displacement controlled lateral pushover case that is based on a user-defined static lateral load pattern named PUSH is defined for this example.

(vi) Run the basic static analysis and, if desired, dynamic analysis. Then run the static nonlinear pushover analysis.

(vii) Display the pushover curve. The File menu shown in this display window allows you to view and if desired, print to either a printer or an ASCII file, a table which gives the coordinates of each step of the pushover curve and summarizes the number of hinges in each state as defined in Figure1 (for example, between IO and LS, or between D and E).

(viii) Display the capacity spectrum curve. Note that you can interactively modify the magnitude of the earthquake and the damping information on this form and immediately see the new capacity spectrum plot. The performance point for a given set of values is defined by the intersection of the capacity curve (green) and the single demand spectrum curve (yellow). Also, the file menu in this display allows you to print the coordinates of the capacity curve and the demand curve as well as other information used to convert the pushover curve to Acceleration-Displacement Response Spectrum format.

(ix) Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis. The arrows in the bottom right-hand corner of the screen allow you to move through the pushover step-by- step. Hinges appear when they yield and are color coded based on their state (see legend at bottom of screen).

(x) Review member forces on a step-by-step basis. Often it is useful to view the model in two side-by-side windows with the step-by-step displaced shape in one window and the step-by-step member forces in the other. These windows can be synchronized to the same step, and can thus greatly enhance the understanding of the pushover results.

(xi) 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.

For buildings that are being rehabilitated it is easy to investigate the effect of different strengthening schemes. The effect of added damping can be immediately seen on the capacity spectrum form. You can easily stiffen or strengthen the building by changing member properties and rerunning the analysis. Finally you can easily change the assumed detailing of the building by modifying the hinge acceptance criteria and rerunning the analysis.

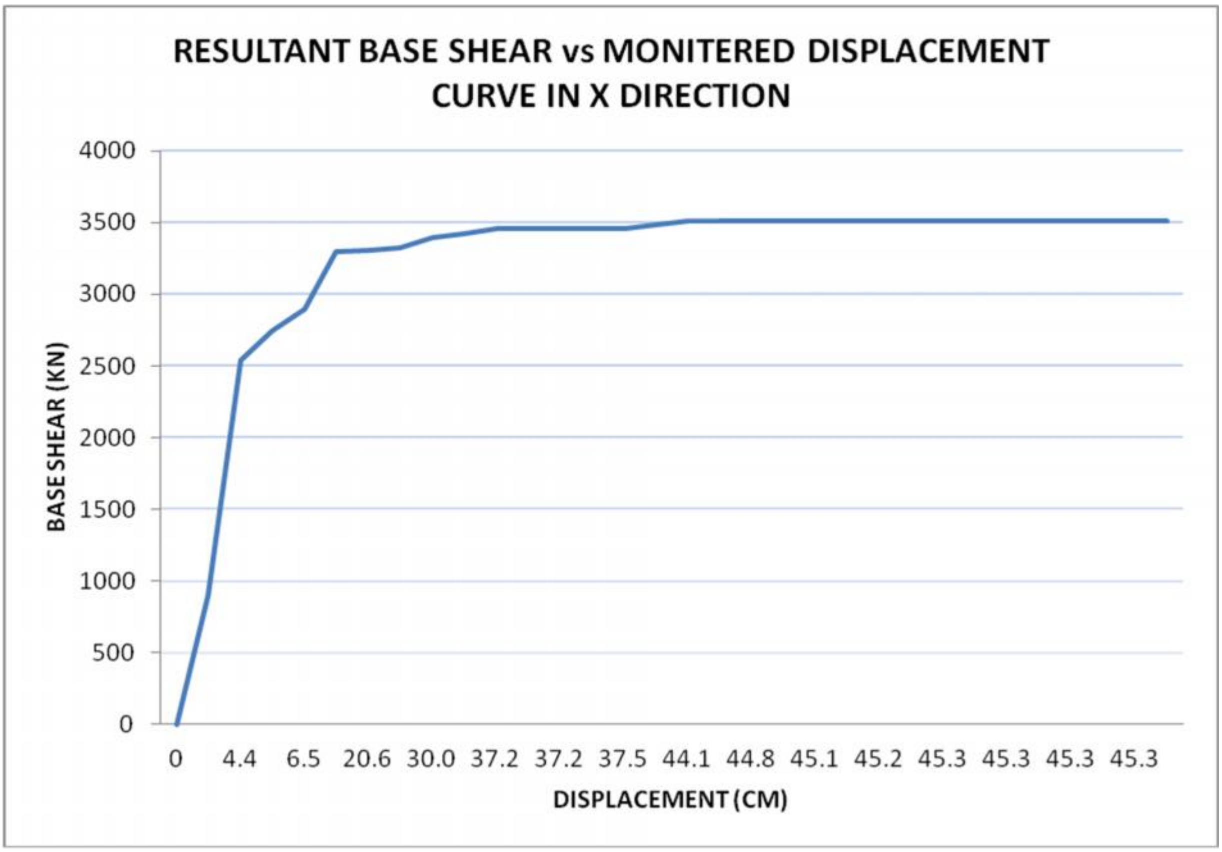
4.5 RESULTS

Linear static and dynamic analysis results:

TABLE: Modal Participating Mass Ratios							
OutputCase	**StepType**	**StepNum**	**Period**	**UX**	**UY**	**SumUX**	**SumUY**
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	1.971563	0.788	0	0.788	0
MODAL	Mode	2	1.696236	0	0.745	0.788	0.745
MODAL	Mode	3	1.549179	0	0.019	0.788	0.764
MODAL	Mode	4	0.655761	0.099	3.84E-19	0.888	0.764
MODAL	Mode	5	0.544151	3.54E-17	0.1	0.888	0.865
MODAL	Mode	6	0.501796	4.55E-17	0.006517	0.888	0.871
MODAL	Mode	7	0.381214	0.036	0	0.924	0.871
MODAL	Mode	8	0.297357	2.76E-16	0.036	0.924	0.908
MODAL	Mode	9	0.279452	7.66E-17	0.004746	0.924	0.912
MODAL	Mode	10	0.263933	0.021	1.76E-16	0.945	0.912
MODAL	Mode	11	0.199518	0.014	5.84E-15	0.958	0.912
MODAL	Mode	12	0.192371	1.47E-14	0.02	0.958	0.933

TABLE: Base shear				
OutputCase	**CaseType**	**StepType**	**GlobalFX**	**GlobalFY**
Text	Text	Text	KN	KN
EQX	LinStatic		-1606.514	1.532E-09
EQY	LinStatic		1.164E-09	-1606.514
SPECX	LinRespSpec	Max	1606.32	0.0007031
SPECY	LinRespSpec	Max	0.0007486	1610.085

Results of the Push-Over analysis are presented in Figures (push-over curves, in each of the 2 main directions). The performance point at the intersection of the capacity spectrum with the single demand spectrum has been obtained. Figures show the floor displacement. Plastic hinge formation for the building mechanisms has been obtained at different levels.


RESULTANT BASE SHEAR vs MONITERED DISPLACEMENT CURVE IN X DIRECTION
BASE SHEAR (KN)
4000
3500
3000
2500
2000
1500
1000
500
0
0 4.4 6.5 20.6 30.0 37.2 37.2 37.5 44.1 44.8 45.1 45.2 45.3 45.3 45.3 45.3
DISPLACEMENT (CM)

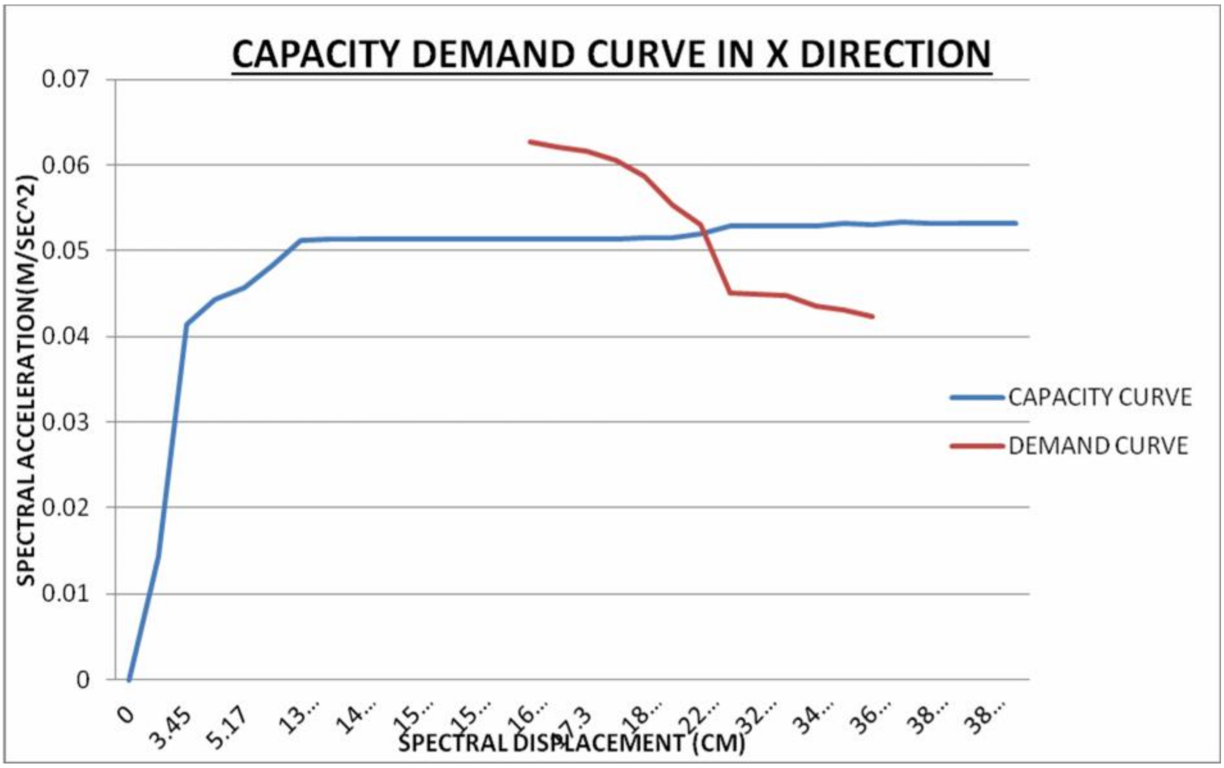

CAPACITY DEMAND CURVE IN X DIRECTION
SPECTRAL ACCELERATION(M/SEC^2)
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0
0 3.45 5.17 13... 14... 15... 15... 16... 17.3 18... 22... 32... 34... 36... 38... 38...
SPECTRAL DISPLACEMENT (CM)
CAPACITY CURVE
DEMAND CURVE

Fig.4 (a) & (b)

AT PERFORMANCE POINT			
Base Shear (KN)	Displacement (m)	Sa (m/s^2)	Sd (m)
3310	0.208	0.051	0.175

STEP WISE PLASTIC ROTATION IN PUSHOVER CASE IN X-DIRECTION

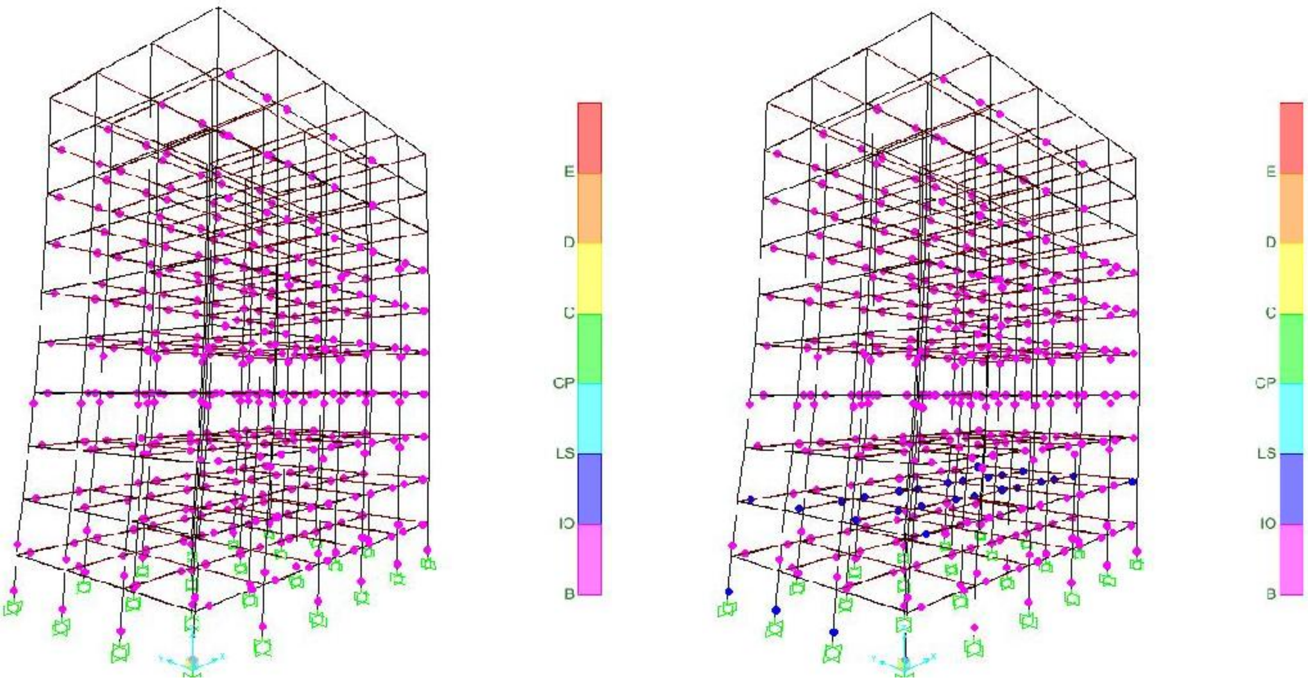


Fig. 4 (c)

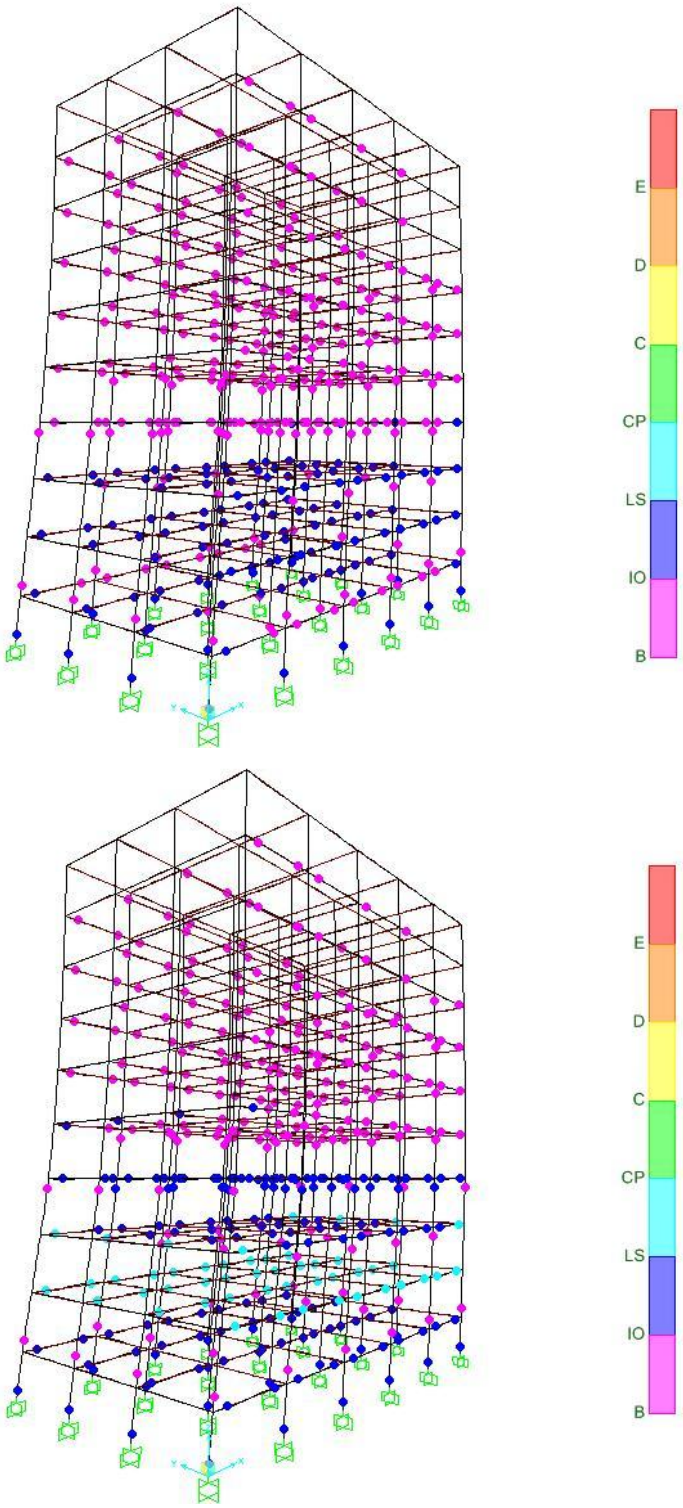


Fig. 4 (c)

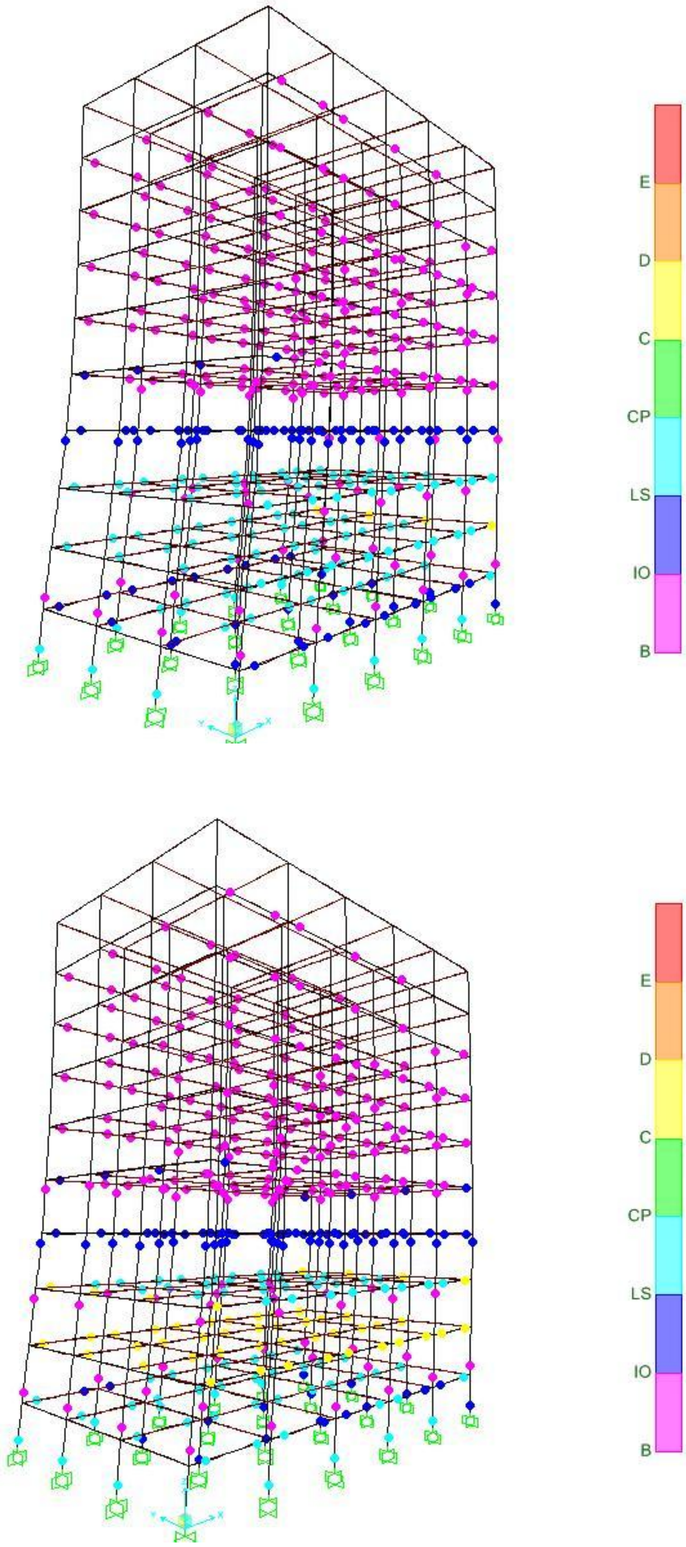
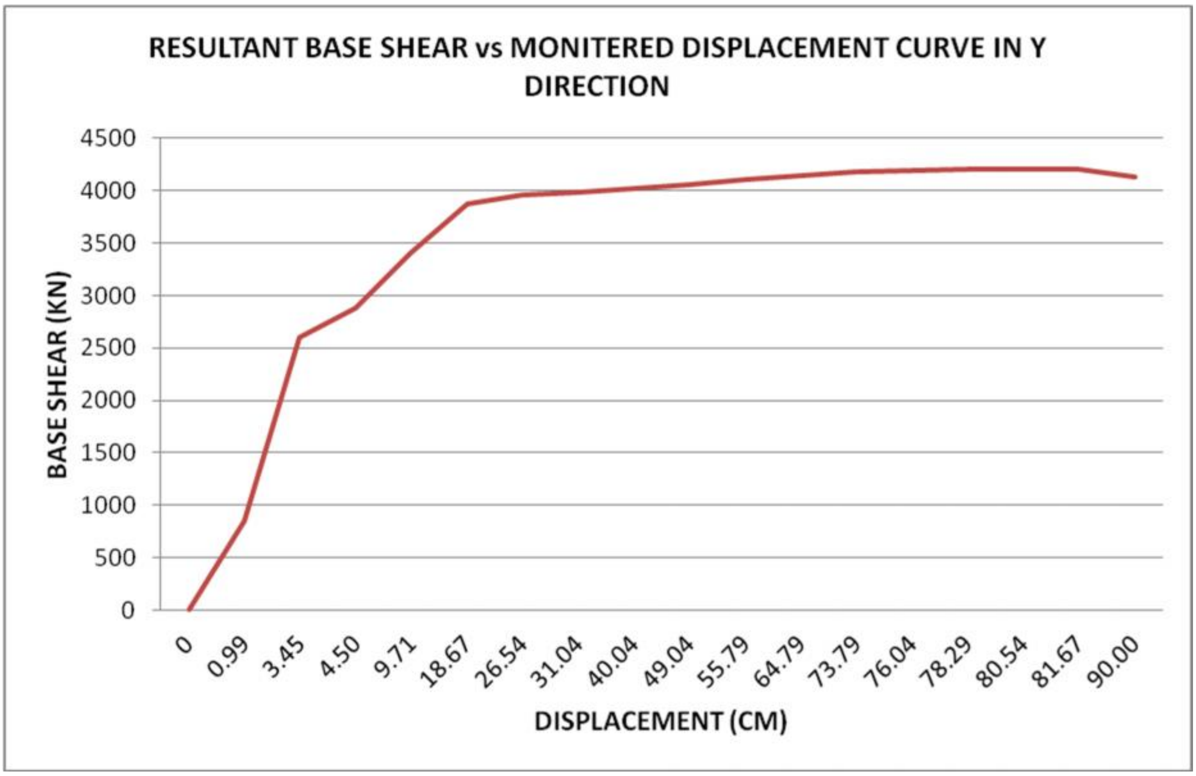
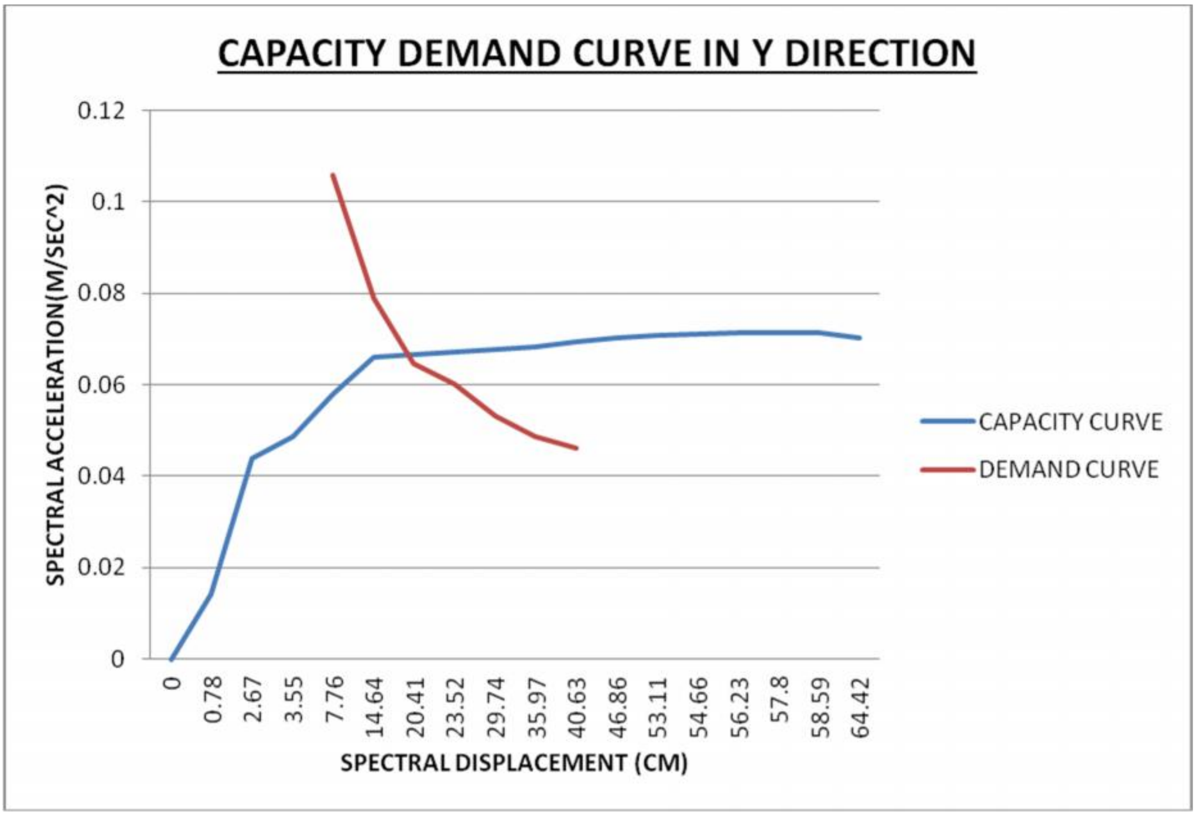


Fig. 4 (c)



RESULTANT BASE SHEAR vs MONITERED DISPLACEMENT CURVE IN Y DIRECTION
4500
4000
3500
3000
2500
2000
1500
1000
500
0
BASE SHEAR (KN)
0
0.99
3.45
4.50
9.71
18.67
26.54
31.04
40.04
49.04
55.79
64.79
73.79
76.04
78.29
80.54
81.67
90.00
DISPLACEMENT (CM)



CAPACITY DEMAND CURVE IN Y DIRECTION
0.12
0.1
0.08
0.06
0.04
0.02
0
SPECTRAL ACCELERATION(M/SEC^2)
0
0.78
2.67
3.55
7.76
14.64
20.41
23.52
29.74
35.97
40.63
46.86
53.11
54.66
56.23
57.8
58.59
64.42
SPECTRAL DISPLACEMENT (CM)
CAPACITY CURVE
DEMAND CURVE

Fig. 4 (d) & (e)

PUSH IN Y DIRECTION

AT PERFORMANCE POINT			
Base Shear (KN)	Displacement (m)	Sa (m/s^2)	Sd (m)
3904	0.215	0.066	0.167

STEP WISE PLASTIC ROTATION IN PUSHOVER CASE IN X-DIRECTION

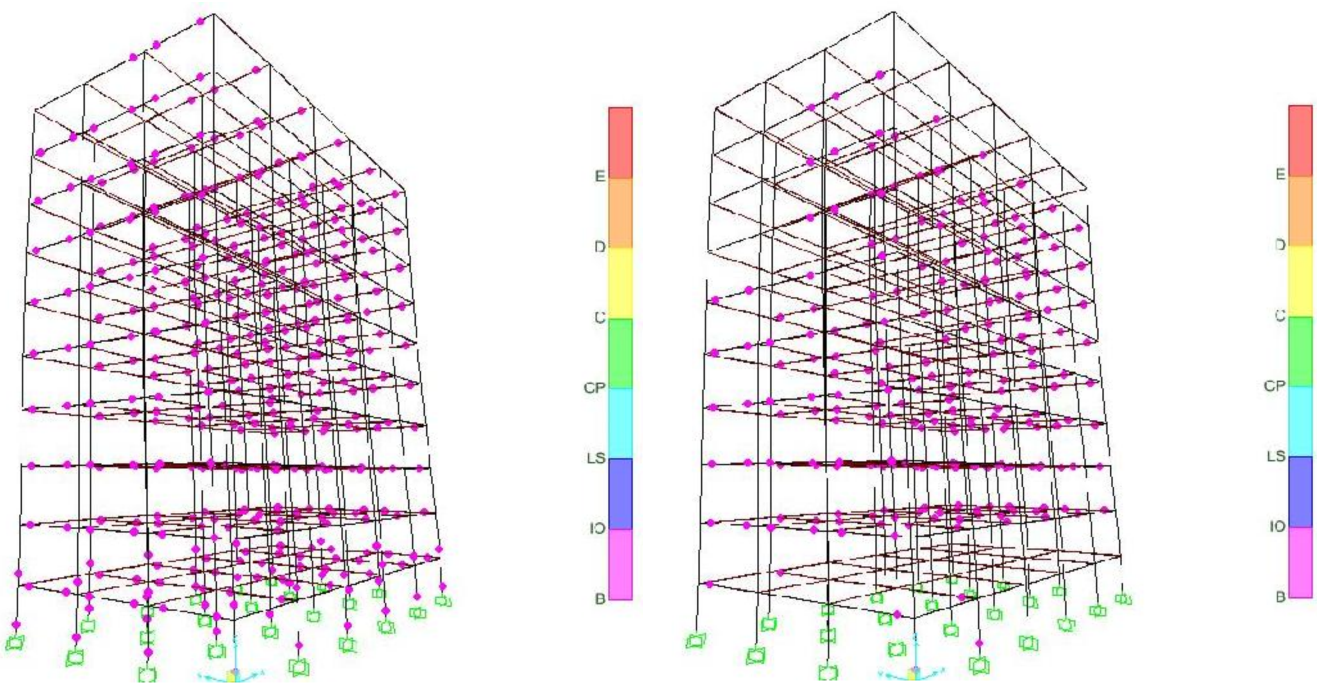


Fig. 4 (f)

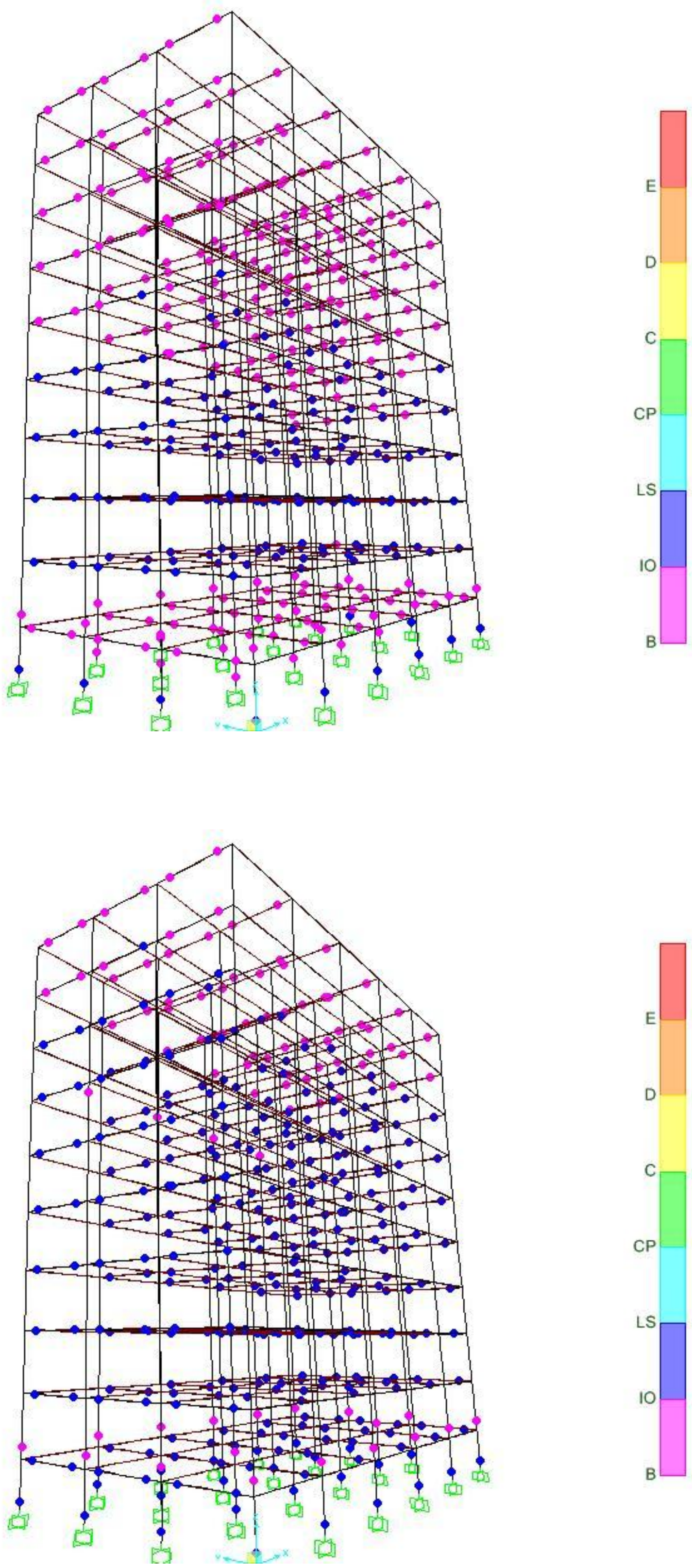
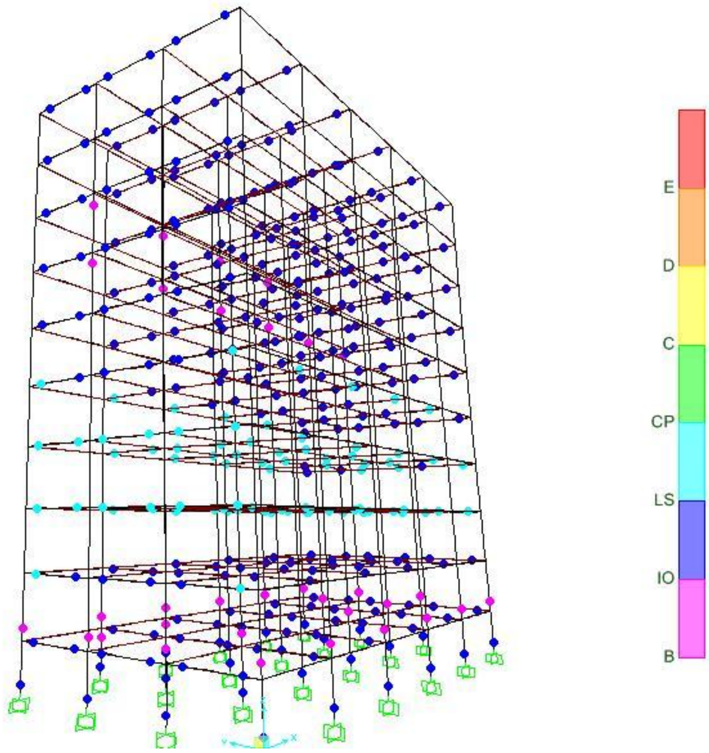


Fig. 4 (f)



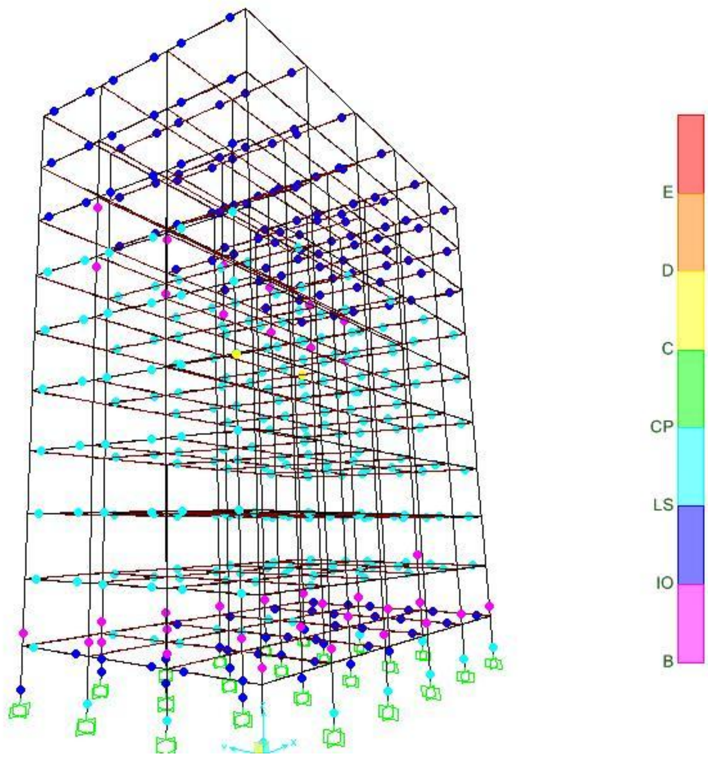
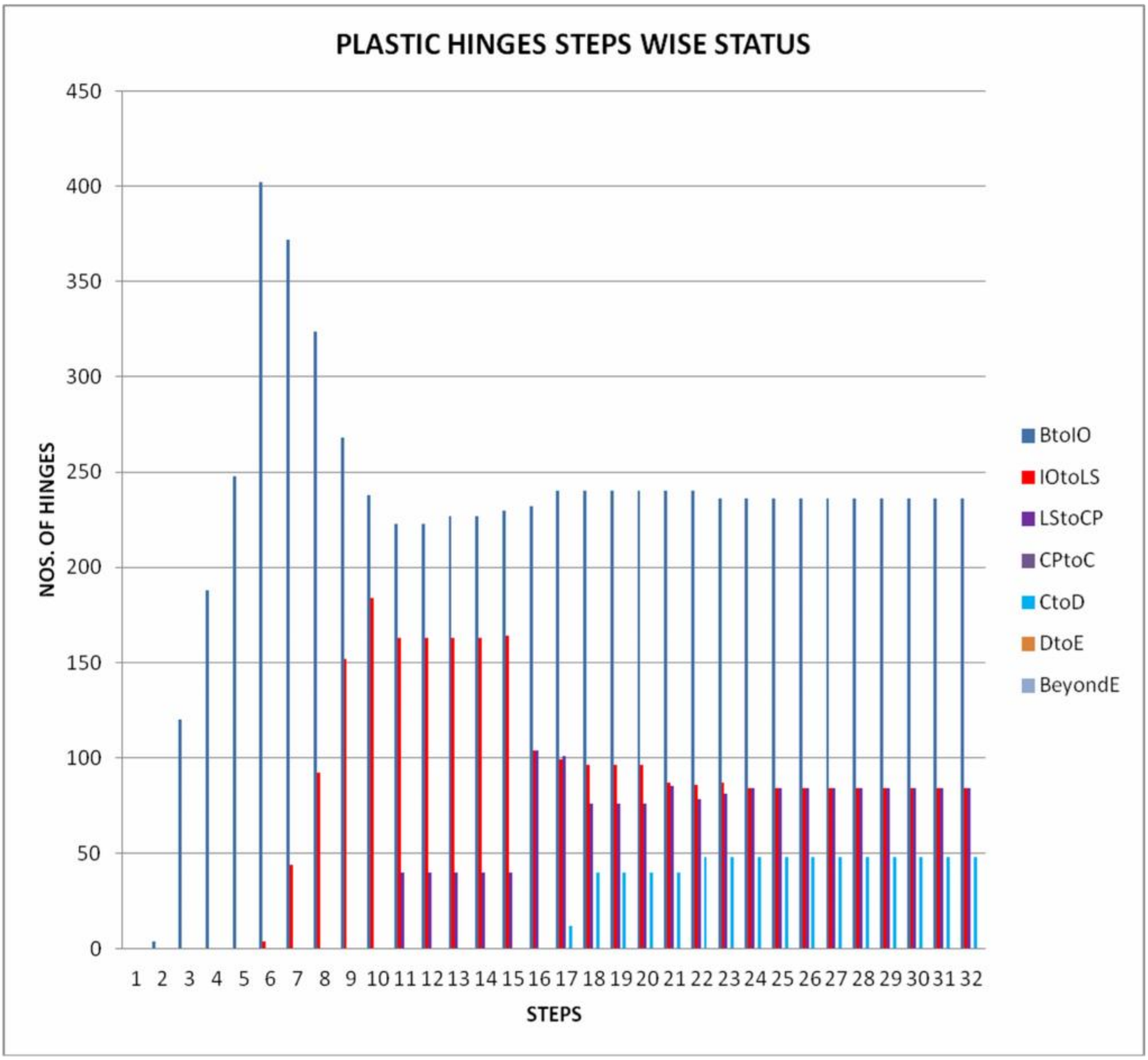


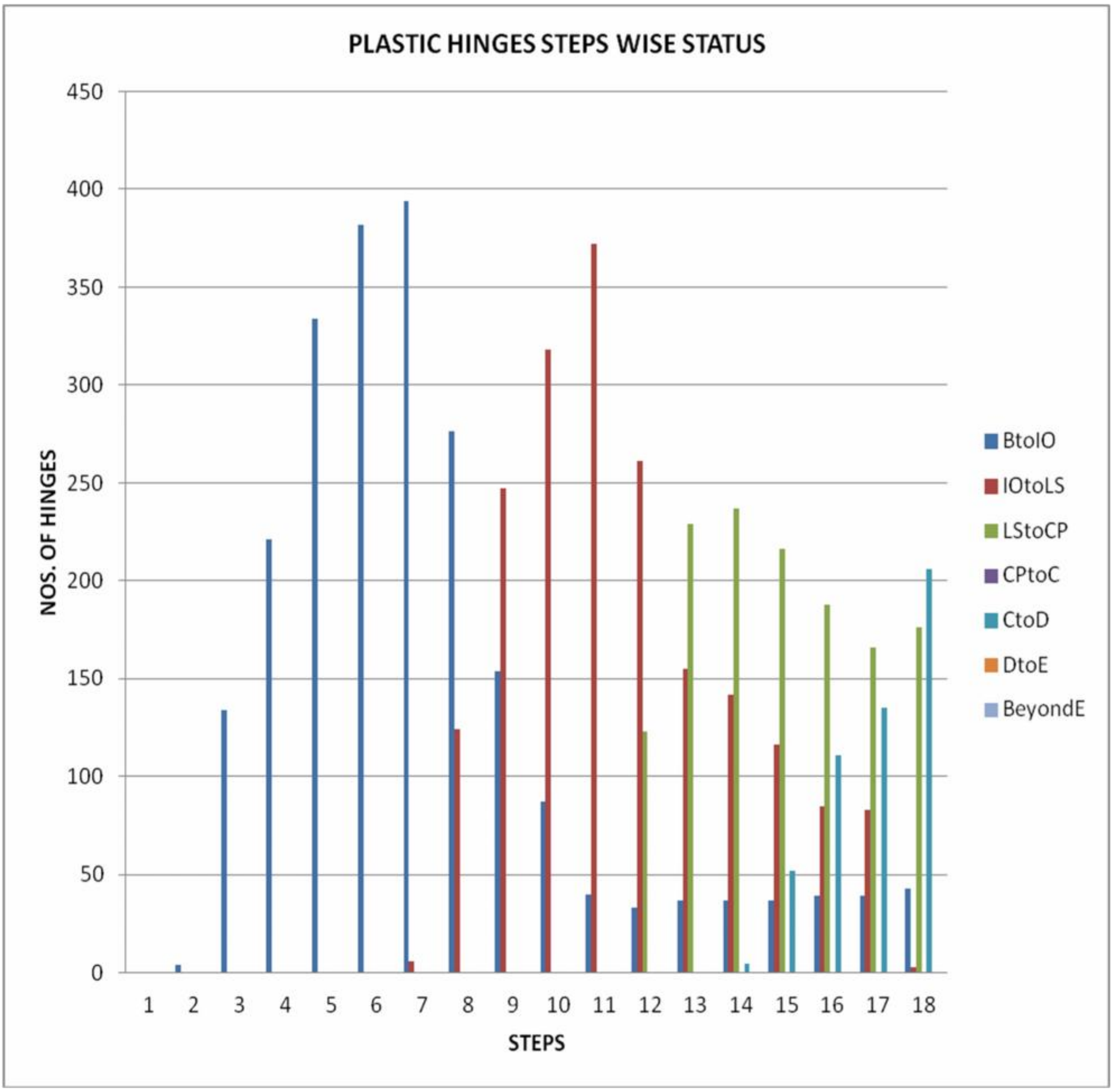
Fig. 4 (f)


PLASTIC HINGES STEPS WISE STATUS
450
400
350
300
250
200
150
100
50
0
NOS. OF HINGES
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32
STEPS
BtoIO
IOtoLS
LStoCP
CPtoC
CtoD
DtoE
BeyondE

This analysis was completed in 40 steps and performance point was set between steps 14 and 15 of the analysis.The performance point Sd is equal to 0.175 m.
Table 2. Shows some of steps of the analysis for X direction and for each step shows the details for the capacity and demand curve. Figures 12 and 13 presents the overall yielding pattern of the structure at the performance point for X direction.

TABLE: Step wise performance level of Hinges in X- direction												
LoadCase	**Step**	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**BeyondE**	**Total**
PUSH	0	0	0	1240	0	0	0	0	0	0	0	1240
PUSH	1	1.4	900.7	1236	4	0	0	0	0	0	0	1240
PUSH	2	4.4	2539.7	1120	120	0	0	0	0	0	0	1240
PUSH	3	5.2	2746.2	1052	188	0	0	0	0	0	0	1240
PUSH	4	6.5	2898.6	992	248	0	0	0	0	0	0	1240
PUSH	13	19.5	3295.2	834	402	4	0	0	0	0	0	1240
PUSH	14	20.6	3309.6	824	372	44	0	0	0	0	0	1240
PUSH	15	23.2	3321.9	824	324	92	0	0	0	0	0	1240
PUSH	16	30.0	3394.7	820	268	152	0	0	0	0	0	1240
PUSH	17	34.5	3422.6	818	238	184	0	0	0	0	0	1240
PUSH	18	37.2	3455.2	814	223	163	40	0	0	0	0	1240
PUSH	19	37.2	3455.2	814	223	163	40	0	0	0	0	1240
PUSH	20	37.2	3453.0	810	227	163	40	0	0	0	0	1240
PUSH	21	37.2	3453.2	810	227	163	40	0	0	0	0	1240
PUSH	22	37.5	3456.4	806	230	164	40	0	0	0	0	1240

Table 1



PLASTIC HINGES STEPS WISE STATUS
NOS. OF HINGES
STEPS
0
50
100
150
200
250
300
350
400
450
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
BtoIO
IOtoLS
LStoCP
CPtoC
CtoD
DtoE
BeyondE

This analysis was completed in 18 steps and performance point was set between steps 6 and 7 of the analysis.The performance point Sd is equal to 0.167 m.
Table 2. Shows some of steps of the analysis for Y direction and for each step shows the details for the capacity and demand curve. Figures 12 and 13 presents the overall yielding pattern of the structure at the performance point for Y direction.

TABLE: Step wise performance level of Hinges in Y- direction

LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
PUSH	0	0	0.0	1240	0	0	0	0	0	0	0	1240
PUSH	1	0.99	854.9	1236	4	0	0	0	0	0	0	1240
PUSH	2	3.45	2605.7	1106	134	0	0	0	0	0	0	1240
PUSH	3	4.50	2887.6	1019	221	0	0	0	0	0	0	1240
PUSH	4	9.71	3407.8	906	334	0	0	0	0	0	0	1240
PUSH	5	18.67	3869.9	858	382	0	0	0	0	0	0	1240
PUSH	6	26.54	3966.3	840	394	6	0	0	0	0	0	1240
PUSH	7	31.04	3990.4	840	276	124	0	0	0	0	0	1240
PUSH	8	40.04	4022.2	839	154	247	0	0	0	0	0	1240
PUSH	9	49.04	4060.3	835	87	318	0	0	0	0	0	1240
PUSH	10	55.79	4104.7	828	40	372	0	0	0	0	0	1240
PUSH	11	64.79	4145.5	823	33	261	123	0	0	0	0	1240

Table 2

CHAPTER 5

DISCUSSION AND CONCLUSION

- The pushover analysis is an efficient tool to assess the seismic performance of buildings.
- Pushover analysis was carried out separately in the X and Y directions. The resulting pushover curves, in terms of Base Shear – Roof Displacement (V-Δ), are given in Figures 4 (a) & (b) for X and Y directions respectively.
- The slope of the pushover curves is gradually changed with increase of the lateral displacement of the building.This is due to the progressive formation of plastic hinges in beams and columns throughout the structure.
- From the results obtained in X-direction there are 50 and Y-direction,6 elements exceeding the limit level between immediate occupancy (IO)and life safety (LS), as shown in Table 1 & 2. This means that the building requires retrofitting.
- The maximum displacements of the buildings obtained from pushover analysis are higher than the results obtained from linear analysis.
- All pushover methods will generally provide good estimates of base shear, but care should be taken because the estimate might be unconservative. This implies that it is difficult to justify the use of pushover analysis without complementing it with a nonlinear dynamic analysis.

The performance of reinforced concrete building was investigated using the pushoverAnalysisfrom which the following conclusions can be drawn:
The main output of a pushover analysis is in terms of response demand versus capacity. The demand curve intersects the capacity envelope near the elastic range in Y-direction, then the structure has a good resistance in y-direction. But the demand curve intersects the capacity curve close to life safety range in X-direction, then it can be concluded that the structure will not perform effectivly with given stiffness during the imposed seismic excitation and need to be retrofitted to avoid future substantial damage.

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- ATC 40 "Applied Technology Council, Seismic Evaluation and Retrofit of Concrete Buildings", Volume 1 Report, , Redwood City, California, 1996.
- FEMA 356 "Federal Emergency Management Agency", Pre standard and commentary for the seismic rehabilitation of buildings. (2000).
- CSI "Analysis Reference Manual for SAP2000, ETABS and SAFE – Computers and Structures, Inc", Berkeley, California, USA, October 2005.
- "Pushover Analysis of Existing 4 Storey RC Flat Slab Building" by A. E. Hassaballa, M. A. Ismaeil,A. N. Alzead,Fathelrahman M. Adam.