PUSHOVER ANALYSIS OF 15 STOREY STEEL BUILDING USING SAP2000

Dissertation submitted in the partial fulfillment of the requirement for the award of

MASTER OF ENGINEERING

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CERTIFICATE

This is to be certify that the project entitle "**PUSHOVER ANALYSIS OF A MULTI STORY STEEL BUILDING** " being submitted by me , is a bonafied record of my own work carried by me under the guidance & supervision of Associate Professor, Mr. G.P. AWADHIYA in partial fulfillment of the requirement for the award of the Degree of Master of Technology (Structural Engineering) in Civil Engineering, from Delhi Technological University, Delhi.

The matter embodied in this project has not been submitted for the award of the any other degree.

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ABSTRACT

To model the complex behavior of steel building 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 steel structures. For structural design and assessment of steel members, the non-linear analysis has become an important tool. The method can be used to study the behavior of steel structures including force redistribution. This analysis of the nonlinear response of steel structures to be carried out in a routine fashion. It helps in the investigation of the behavior of the structure under different loading conditions, its load deflection behavior and the cracks pattern. In the present study, Pushover analysis to obtain non-linear response of steel frame using SAP2000 under the specify loading as per given in code has been carried out with the intention to investigate the relative importance of several factors in the non-linear analysis of steel frames.

LIST OF ABBREVIATION

C – Classical damping
C_0 – Factor for MDOF displacement
C_1 - Factor for inelastic displacement
C_2 – Factors for strength and stiffness degradation
C_3 – Factor for geometric nonlinearity
E_c – short term modulus of elasticity of concrete
E_d – Energy dissipating by damping
E_s – Modulus of elasticity of steel rebar
W- total weight of building (kN)
U_R- roof displacement (m)
α_1 - modal mass
Γ_1- modal participation factor
Ø_1,r- amplitude of first mode at roof level
S_a- spectral acceleration
S_d - spectral displacement (m)
M- mass of the building (t)
M*- effective mass of SDOF (t)
K*- effective stiffness of SDOF (kN/m)
ω_{eff}- effective frequency of SDOF
W*- effective weight of the building
F_y- yield force (kN)
V_y- yield base shear (kN)
ξ_{eq} - equivalent damping ratio 0
κ - damping modification factor
ζo - hysteretic damping ratio
μ- displacement ductility ratio
T_i- elastic fundamental period
K_i- elastic lateral stiffness
K_e- effective lateral stiffness
F_b - base shear
T -elastic period of the idealized SDOF system
S_a - spectral acceleration corresponding to T
F_i- lateral force at i-th story
M_i- mass of i^{th} story
V_b- Base shear
h - Height of i^{th} story above the base
Δ - Additional earthquake load added

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CHAPTER – 1

INTRODUCTION

1.GENERAL

Nonlinear Static Procedures (NSPs) can be integrated in a Performance Based Seismic Design philosophy. It is generally recognized that structures designed within these deformation-based criteria, using Performance-Based Design Procedures, are more likely to behave sensibly in seismic scenarios than the structures designed according to the classic force-based philosophy. It is also widely accepted that evaluating the deformations in the structure, both at global and component levels, can better control performance criteria.

Nonlinear Static Procedures are deemed to be very practical tools to assess the nonlinear seismic performance of structures. On the other hand, nonlinear dynamic Time-History analyses are very time-consuming, which is a relevant drawback in design offices, where the deadlines are restrictive.

The NSPs introduced in this context are a powerful tool for performance evaluation. Seismic design codes like the FEMA273, FEMA356, FEMA440 and the ATC40, have recommended the use of this type of procedures.

However, some issues still need to be clarified regarding the format with which the pushover analysis has to be performed, thus requiring further research and development. The positive outcome from recent research seems to indicate that it is certainly worthwhile to continue to pursue the further development or verification of NSPs taking a further step with the 3-D Pushover problem with the objective of arriving at an eventual introduction in seismic design codes and regulations of improved procedures capable of dealing with plan irregular structures.

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve of the overall structure. A two or three-dimensional model which includes bilinear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern, which is distributed along the building height, is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is plotted with base shear to get the global capacity curve.

Pushover analysis can be performed as force-controlled or displacement-controlled. In force-controlled pushover procedure, full load combination is applied as specified, that is force-controlled procedure should be used when the load is known. 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 is the preferred tool 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 as per Girgin. et., 2007.In last decades Steel structure plays an important role in the construction industry. It is necessary to design a structure to perform well under seismic loads. Introducing Steel bracings in the structural system as well as bracings can be used as retrofit can increase shear capacity of the structure. There are N numbers of possibilities are there to arrange Steel bracings. Such as D, K, and V type eccentric bracings. Design of such structure should have good ductility property to perform well under seismic loads. To estimate ductility and other properties for each eccentric bracing Pushover analysis is performed.
A simple computer based pushover analysis is a technique for performance-based design of building frameworks subject to earthquake loading. Pushover analysis attains much importance in the past decades due to its simplicity and the effectiveness of the results. The present study develops a pushover analysis for different eccentric steel frames designed according to IS-800(2007) and ductility behavior of each frame.

1.2 DEFINITION

Pushover analysis is 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. In technique 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. The performance criteria for pushover analysis are generally established as the desired state of the building given rooftop or spectral displacement amplitude. The seismic response of RC building frame in terms of performance point and the effect of earthquake forces on multi story building frame with the help of pushover analysis are carried out in this paper. In the present study a building frame is designed as per Indian standard i.e. IS 456:2000 and IS 1893:2002. The main objective of this study is to check the kind of performance a building can give when designed as per Indian Standards. Using structural analysis and using design software SAP 2000 carry out the pushover analysis of the building frame.

1.3 PURPOSE OF NON-LINEAR STATIC PUSH-OVER ANALYSIS

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

• The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam to column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
• Estimates of the deformations demands for elements that have to form in elastically in order to dissipate the energy imparted to the structure.
•Consequences of the strength deterioration is individual elements on behavior of structural system.
• Identification of the critical regions in which the deformation demands is expected to be high and that have to become the focus through detailing.
• Identification of the strength discontinuities in plan elevation that will lead to changes in the dynamic characteristics in elastic range.
• Estimates of the inter story drifts that account for strength or stiffness discontinuities and that may be used to control the damages and to evaluate P-Delta effects.

1.4 OBJECTIVE

Following are the main objectives of the present study:

a) To investigate the seismic performance of a multi-story steel frame building with different bracing arrangements using Nonlinear Static Pushover analysis method.

b) To evaluate the performance factors for steel frames with various bracing arrangements designed according to Indian Code.

CHAPTER – 2

LITERATURE REVIEW

The following review is concerned with studies of the development and application of pushover analysis. It is provided in order to offer an insight into the attempts that have been made to verify the potential, shortcomings and limitations of the method. The findings of previous researchers are given in chronological order. Pushover analysis was introduced by Freeman *et al.* (1975)[2] as the CSM (Capacity Spectrum Method). The main purpose of this empirical approach was to use a simplified and quick method to assess the seismic performance of a series of 80 buildings located in a shipyard in the USA. The study combined the use of analytical methods with site-response spectra to estimate values of peak structural response, peak ductility demands, equivalent period of vibration, equivalent percentages of critical damping, and residual capacities. It was concluded that it could perform, in most of the cases, a worthwhile evaluation of existing structures in a reasonable time-scale and cost.

Freeman (1978)[3] presented the Capacity Spectrum method in a clearer manner together with its application to two instrumented 7-storey reinforced concrete structures. The data obtained from the recorded motions were compared with the analysis results showing reasonable agreement. Freeman cautioned engineers that the elastic modeling assumptions, e.g. the choice between cracked or uncracked sections, the inelastic stiffness degradation, e.g. appropriate reduction of structural elements' stiffnesses in the post-elastic region, and the percentage of critical damping used to construct the demand spectra, and determination of the inelastic capacity needed careful judgment and some experience to be adequately defined and assessed. It was suggested that two levels of equivalent viscous damping should be assumed relating to the initial undamaged state and to the ultimate limit state in order to account for the effect of period lengthening that is usual when the structure enters the nonlinear region. Furthermore, it was concluded that more structures needed to be assessed to validate the method.

The published reports ATC 40 (1996)[42] and FEMA 273 (1997)[40] highlighted the non-linear static pushover analysis. It is an efficient method for the performance evaluation of a structure subjected to seismic loads. The step-by-step procedure of the pushover analysis is to determine the capacity curve, demand curve and performance point. These reports deal with modeling aspects of the hinge behavior, acceptance criteria and procedures to locate the performance point. The seismic performance of non-ductile reinforced concrete framed buildings, in regions of low to moderate seismic forces was evaluated by Kunnath et al (1999)[23]. The detailing configurations included in the analysis were discontinuous positive flexural reinforcement, lack of joint shear reinforcement and inadequate transverse reinforcement for column core confinement. When the buildings were subjected to a moderate level earthquake, the buildings suffered significant but not severe damages. The beams were more damages than the columns, except in the lower storey levels of the nine-storey structure.

Saidi and Sozen (1981)[6] produced a 'low-cost' analytical model which was named the Q-Model for calculating displacement histories of multi-storey reinforced concrete structures subjected to ground motions. The Q-model, which was based on the idea of Gulkan et al. (1974)[1], involved two simplifications, the reduction of a MDOF model of a structure to a SDOF oscillator and the approximation of the variation of the stiffness properties of the entire structure by a single spring to take account of the nonlinear force-displacement relationships that characterise its properties. Earthquake-simulation experiments of eight small-scale structures were performed and the displacement histories were compared with the results from nonlinear static analyses based on the Q-model. It was shown that the performance of the Q- Model in the simulation of high- and low- amplitude responses was satisfactory for most of the test structures. It was stated that the model would need to be further validated by more experimental and theoretical analyses.

Kabeyasawa et al. (1983), Okamoto et al. (1984), Bertero et al. (1984), and Fajfar et al. (1984)[8]. The authors used the uniform and inverted triangular load distributions to perform nonlinear static analyses of the structure. The pushover curves were compared to the dynamic experimental and analytical results showing considerable differences in their shapes. It was noted that the inverted triangular distribution was unconservative in estimating base shear demands due to the effect of higher modes. It was observed that the uniform distribution seemed more rational when shear strength demand was to be assessed. It was also observed that the nonlinear dynamic analysis of the equivalent SDOF system yielded in general non-conservative shear forces compared with the experimental and theoretical results. However the target displacement at the ultimate limit state and the rotations of the floors were approximated satisfactorily compared with the experimental and theoretical results.

Baik, Lee, and Krawinkler (1988)[9] proposed a simplified analysis model for the seismic response prediction of steel frames that was based on the pushover analysis concept but included cumulative damage parameters using the Park-Ang damage model (Park et al. 1985). These parameters accounted for the effects of all inelastic excursions and not only for the maximum excursion. The model was tested on 10- and 20- single bay steel structures and was considered to be acceptable for preliminary design purposes. It was noted, that the prediction of damage using the equivalent SDOF model 'deteriorated' with increasing structure height, and in the presence of irregularities. The authors suggested though that the ESDOF nonlinear model could provide better estimation of damage parameters than an elastic multi-storey model.

Deierlein and Hsieh (1990) utilized the Capacity Spectrum method to compare the experimental and theoretical results for the seismic response of a single storey single bay steel frame with the analytical results of a 2D pushover analysis. The frame was modeled with semi-rigid connections. The results showed differences of the order of 10% to 20% between the compared quantities such as the period of vibration, maximum displacement and maximum acceleration. It was concluded that the Capacity Spectrum method could provide reasonably accurate lower and upper bounds on the inelastic response of a structure subjected to strong ground motion.

Gaspersic, Fajfar, and Fischinger (1992)[11], extended the N2 method by attempting to include cumulative damage; a characteristic resulting from numerous inelastic excursions. The test structure was the seven-storey reinforced concrete building tested in the U.S. – Japan research project. The seismic demands for each element were computed in terms of the dissipated hysteretic energy using the Park-Ang model (Park et al. 1985). The conclusions drawn were that the dissipated hysteretic energy increased with increasing duration of ground motion, and it was significantly affected by the reduction of strength of the structural elements. They also concluded that when the fundamental period of the structure was much larger than the dominant period of the ground motion, the higher mode effects became an important issue. In this case the input energy and dissipated hysteretic energy of a MDOF system were generally larger than the corresponding quantities in the equivalent SDOF system. The authors suggested that the N2 method was likely to underestimate quantities, which governed damage in the upper part of a structure.

Lawson, Vance and Krawinkler (1994)[12] carried out a general assessment of pushover analysis on 2-, 5-, 10-, and 15- storey steel moment resisting frames. The pushover analysis results were compared to nonlinear dynamic analyses results using seven ground motions. Storey deflections calculated from the pushover analyses correlated well with those derived from nonlinear dynamic analyses for the short structures. Additionally, pushover analysis could identify weak stories that led to concentration of inelastic deformations. For the tall structures large differences between nonlinear static and nonlinear dynamic deflections across the storey levels were observed and the results became sensitive to the applied load pattern indicating that higher mode effects became important. Good correlation of inter-storey drifts from the pushover and nonlinear dynamic analyses results was observed for the short structures while poor correlation was observed in the upper storeys of the tall structures. The accuracy in the evaluation of inter-storey ductility ratios and plastic hinge rotations, decreased with the increasing height of structures especially at the higher storeys. Furthermore, the area under the static load-displacement curve correlated poorly with the dynamic hysteretic energy dissipation and therefore was a poor measure of the cumulative damage demand.

Krawinkler (1996)[15] carried out a general appraisal of pushover analysis. The physical meaning of the modification factors used in the Displacement Coefficient Method was explained in some detail. It was noted that generally the displacement of an inelastic SDOF system would differ from the one of the respective elastic SDOF system. This difference will depend on the extent of yielding and the period of the system. Additionally degradation of the unloading or reloading stiffness could have an effect on the target displacement, though this was found to be only significant for very short-period systems. Strength deterioration was noted to have more adverse effects on the inelastic displacement demands. The magnitude of this effect was said to depend on the strong ground motion duration. P-delta effects were also noted to affect significantly the target displacement of a structure. These effects are dependent on the ratio of the post-yield stiffness to the effective elastic stiffness, the fundamental period of the structure, the strength reduction factor, the hysteretic load deformation characteristics of each storey, and frequency characteristics and duration of the ground motion. Other modification factors would need to be developed to account for different levels of damping, foundation uplift, torsional effects, and semi-rigid floor diaphragms. The effect of load pattern on the sensitivity of the results was acknowledged. The general conclusion about pushover analysis was that the different aspects of structural response that could affect the displacement response should be considered explicitly.

Helmut Krawinkler and Seneviratna (1998)[18] discussed that, the pushover analysis would be a great improvement over presently employed elastic evaluation procedures and they also pointed out that a carefully performed pushover analysis would provide insight into structural aspects that control performances during severe earthquakes. Further it was concluded that, for structures that vibrate primarily in the fundamental mode, the pushover analysis would provide good estimates of global as well as local inelastic, deformation demands. These analyses also expose design weaknesses that may remain hidden in an elastic analysis.

Naeim and Lobo (1998)[19] attempted to identify some potential pitfalls when carrying out a pushover analysis and summarized ten important aspects, which should be considered preceding the analysis. These were:

The importance of the loading shape function should not be underestimated. The effect of three load patterns, Uniform, Triangular, FEMA, was checked on a two- storey reinforced concrete frame. The pushover curves showed differences in the global base shear- roof displacement response of about 10%, which do not seem particularly significant

1. Performance objectives should be known before the building is 'pushed'.

2.If the building is not designed, it cannot be pushed.

3.Gravity loads should not be ignored. The structure should not be pushed beyond failure unless the engineer can model failure. This has to do mainly with the available computer codes being incapable of modeling post-failure behavior of structural components. It was suggested that the pushover analysis should be stopped at the onset of the first failure mechanism.

4.Attention to rebar development and lap lengths should be given.

5.Shear failure mechanisms should not be ignored.

6.P-Delta effects should be adequately included; otherwise the results obtained could be inconservative. An example of the base shear-roof displacement responses of a ten-storey frame model with and without P-Delta effects showed a difference of the order of 10% for the base shear.

7.The pushover loading should not be confused with the real earthquake loading.

8.Three-dimensional buildings may require more than a planar 'push'.

Ashraf Habibullah and Stephen (1998)**[22]** described the use of SAP2000[44] for the performing a pushover analysis of a simple three-dimensional building. SAP2000[44] is a state-of-the-art, general purpose, and three dimensional structural analysis programs. SAP2000[44] has static pushover analysis capabilities which were fully integrated into the program allow quick and easy implementation of the pushover procedures for both two and three-dimensional frames.

Mwafy and Elanashai (2000)[28] owing to the simplicity of inelastic static pushover analysis, a comparison study was made between inelastic dynamic analysis and inelastic static pushover analysis for 12 reinforced concrete buildings of different characteristics. The analysis was carried out using natural and artificial earthquake records. It was found that the static pushover analysis was more appropriate for low rise and short period framed structures. For well-designed buildings but with structural irregularities, the result of the procedure also shows good correlation with the dynamic analysis.

Elnashai (2001)[31] analyzed the dynamic response of structures using static pushover analysis. The significance of pushover analysis as an alternative to inelastic dynamic analysis in seismic design and assessment were discussed. New developments towards a fully adaptive pushover method accounting for spread of inelasticity, geometric non-linearity, full multimodal, spectral amplification and period elongation within a framework of fiber modeling of materials were discussed and preliminary results were given. These developments lead to static analysis results that were closer than ever to inelastic time-history analysis.

A modal pushover analysis procedure for estimating seismic demands for buildings was developed by Chopra and Goel (2004)[36]**.** The modal pushover analysis was applied to a nine-storey steel building to determine the peak inelastic response and it was compared with rigorous non-linear response history analysis. It was concluded that the modal pushover analysis was accurate enough for practical application in building evaluation and design.

CONCLUSIONS

Many guidelines are reviewed for linear, non-linear analysis and the seismic evaluations of the structures are also discussed. Most of the researchers have reviewed that the buildings were assumed to be placed in various zones of India and carried out the investigation on the non-linear analysis (pushover analysis) and compared the performance of the building components, maximum base shear capacity of the structures located in the various zones. Many papers considered different amount of masonry infill walls to investigate the effect of infill walls on earthquake in response to the structures. SAP2000[44], ETABS and IDARC-2D software's were mainly used to find out the seismic evaluation and performance of the structures. All these studies require further research not based on assumptions, but in real terms it is essential to consider existing reinforced concrete structures under seismic evaluation.

Chapter 3
Pushover analysis methods

3.1 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.

3.1.1 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 has automatically included in time history analysis. In demand/capacity ratio procedure the force actions are compared to corresponding capacities as demand/capacity ratios (DCR). 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 two structures could not be identified by an elastic analysis. However, post elastic behavior 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 do not 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 cannot 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.

3.1.2 Inelastic Methods of Analysis

The 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 required for input preparation and 3 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. The theoretical background, reliability and the accuracy of inelastic static analysis procedure is discussed in detail in the following sections.
The uncertainties involved in accurate determination of material properties, element and structure capacities the limited prediction of ground motions that the structure is going to experience and the limitations in accurate modeling of structural behavior make the seismic performance evaluation of structures a complex and difficult process. Displacement-based procedures provide a more rational approach to these issues compared to force-based procedures by considering inelastic deformations rather than elastic forces. The analytical tool for evaluation process should also be relatively simple which can capture critical response parameters that significantly affect the evaluation process.

3.2 DESCRIPTION OF PUSHOVER ANALYSIS

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analyses, superimposed to approximate a force-displacement curve of the overall structure. A two or three-dimensional model which includes bilinear or tri-linear load-deformation diagrams of all lateral force resisting elements is first created and gravity loads are applied initially. A predefined lateral load pattern, which is distributed along the building height, is then applied. The lateral forces are increased until some members yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members yield. The process is continued until a control 4 Roof Displacement, δ Base Shear, V δ V 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 (Figure 2.1). Figure 2.1: Global Capacity (Pushover) Curve of a Structure 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. Generally, pushover analysis is performed as displacement-controlled proposed by Allahabad to overcome these problems. In displacement-controlled procedure, specified drifts are sought (as in seismic loading) where the magnitude of applied load is not known in advance. The magnitude of load combination is increased or decreased as necessary until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations computed at the target displacement are used as estimates of inelastic strength and deformation demands that have to be compared with available capacities for a performance check.

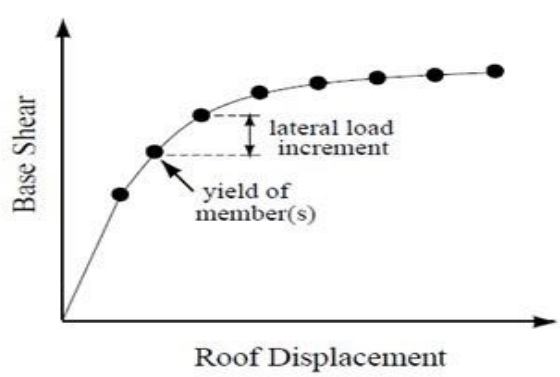


FIGURE 3.1: GLOBAL CAPACITY (PUSHOVER) CURVE OF STRUCTURE

3.3 Force Based Design Method

Force based design method practiced in India, which focus on the seismic force over the structure. In this method, the design procedure is carried out for the seismic force acting on the system where stiffness, time period and strength are the initial properties of the design. FBD method is performed based on IS 1893(Part 1):2002[43].The existing conventional code based procedures are normative in nature. This code needs to cover a wide range of structures and this method usually cannot be considered as the expected performance level and seismic risk levels are not generalized. Linear elastic analysis of the structure is performed for the lateral forces calculated from the procedure.

3.4 Displacement Based Design Method

Displacement-Based Design (DBD), first proposed by Priestley (1993) is a performance design approach in which Performance levels, indeed, are described in terms of displacements, as damage is better correlated to displacements rather than forces. The fundamental goal of DDBD is to obtain a structure which will reach a target displacement profile when subjected to earthquakes consistent with a given reference response spectrum. The performance levels of the structure are governed through the selection of suitable values of the maximum displacement and maximum inter storey drift. In our study, we follow nonlinear dynamic seismic analysis procedure. The Nonlinear dynamic analysis procedure utilizes a combination of ground motion records with a detailed structural model, and therefore is capable of producing results with relatively low uncertainty. The detailed structural model subjected to a ground motion record produces estimates of component deformations for each degree of freedom in the model and the model responses are combined using schemes such as square-root-sum of squares. The method captures the effect of amplification due to resonance, the variation of displacements at diverse levels of a frame, an increase of motion duration, and a tendency of regularization of movements.

3.5 Analysis Method

Pushover analysis creates a capacity spectrum expressed in terms of a lateral load-displacement relationship by incrementally increases static forces to the point of the ultimate performance. The capacity spectrum is then compared with the demand spectrum, which is expressed in the form of a response spectrum to seismic loads to examine if the structure is capable of achieving the target performance. However pushover is often referred to as second stage analysis.
Pushover analysis can provide the following advantages:

1. It allows to evaluate overall structural behaviors and performance characteristics.
2. It enables us to investigate the sequential formation of plastic hinges in the individual structural elements constituting the entire structure.
3. When the structure to be strengthened through a rehabilitation process it allows us to selectively reinforce only the required members, thereby maximizing the cost efficiency. Estimate of force and displacement capacities of the structure, Sequence of the member yielding and the progress of the overall capacity curve.

Capacity: The overall capacity of a structure depends on the deformation capacity of the structures components and the strength that it has, and determining the structural capacity beyond the elastic limits. A nonlinear analysis such as the pushover analysis is to be performed. Sequential elastic analysis is used in pushover analysis procedure in a series, superimposed to approximate a force displacement capacity diagram of the overall structure. A lateral force distribution is again applied until additional components yield. This process is continued until the structure become unstable or until a predetermined limit is reached. In short capacity is seismic demand resisting ability of structure.

Demand: During an earthquake, the ground motion produces complex horizontal displacement patterns in the structures. To determine the structural design parameters it is not practical to trace this lateral displacement at each time step. And once the capacity curve for the structure & the demand displacement are defined the performance check for the structure can be done. In short demand is, the structure is subjected to a ground shaking or an earthquake ground motion an estimation of displacements or deformations in which the structure is expected to undergo. The performance of the structure depends on this two key elements, whether the capacity of the structure is enough to resist the demand or the structure should have adequate capacity to resist the demands of the earthquake ground motions so that performance and objective of design are compatible with each other.

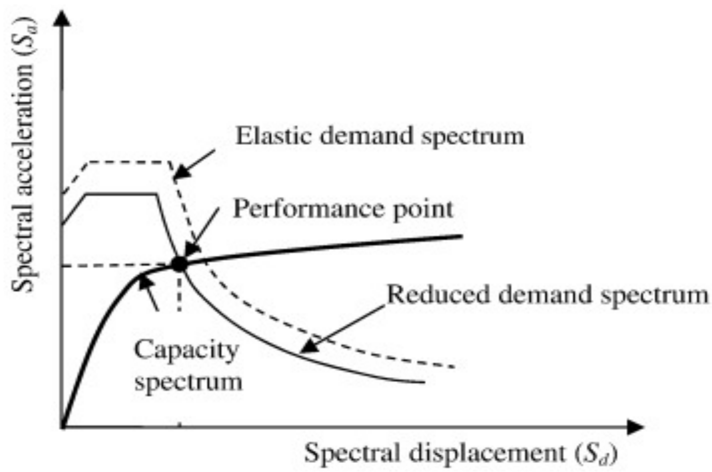


FIGURE 3.2

Performance: It is dependent on the manner that the capacity is able to handle the demand. In other words the structure has the capacity to resist demand of the earthquake such that the performance of the structure is compatible with the objectives of the design.

Once a capacity curve and demand displacement is defined a performance check can be done. A performance check verifies that structural and non-structural components are not damaged beyond the acceptable limits of the performance objective for the force and displacement demand. The inter section of the capacity and demand spectrum in the capacity spectrum method (the displacement at the performance point i.e. equivalent to the target displacement in the coefficient method.

Chapter-4

Performance level and capacity curve

4.1 PERFORMANCE LEVEL

A performance level describes a limiting damage condition: which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage and the post-earthquake serviceability of the building.

4.2 PERFORMANCE LEVEL OF A STRUCTURE

The structural and non- structural components of the buildings together comprise the building performance. The performance levels are the discrete damage states identified from a continuous spectrum of possible damage states. The structural performance levels based on the roof drifts are as follows: Five points labeled A, B, C, D and E are used to define the force deflection behavior of the hinge and these points labeled as

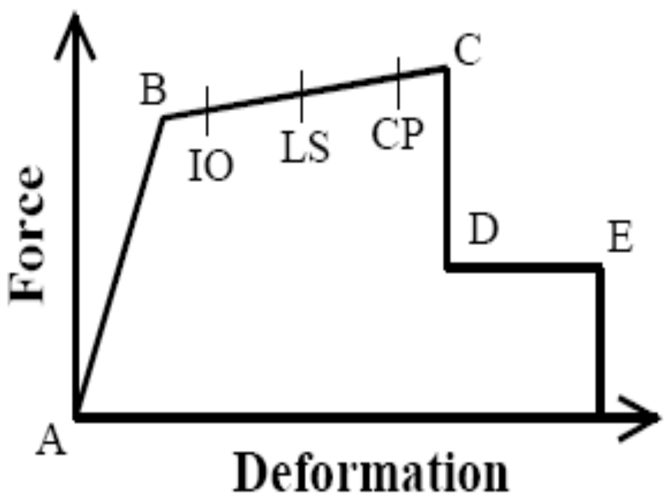


FIGURE 4.1: FORCE-DEFORMATION FOR PUSHOVER HINGE

The performance levels of a structural element are represented in the load versus deformation curve as shown below: -

1. **A to B**- Elastic state

A) Point A- Corresponds to the unloaded condition.

B) Point B- Corresponds to the onset of yielding.

2. **B to IO**- Below immediate occupancy.

3. **IO to LS**- Between immediate occupancy and life safety

4. **LS to CP**- Between life safety to collapse prevention

5. **CP to C**- Between collapse prevention and ultimate capacity, i) Point ‘C’ corresponds to the ultimate strength

6. **C to D**- Between C and residual strength, i) Point ‘D’ corresponds to the residual strength

7**. D to E**- Between D and collapse

I) **Point E-** Corresponds to the collapse

4.3 Performance Objective

Performance objectives are statements of acceptable performance of the structure. The performance target can be specified limits on any response parameter such as stresses, strains, displacements, accelerations, etc. It is appealing to express the performance objective in terms of a specific damage state or the probability of failure against a prescribed probability demand level. Various documents promote the same concepts but differ in detail and specify different performance levels. Some of the suggested performance levels can be grouped in equivalent categories as listed in Table 1. It is recognized that drift levels associated with specific damage categories may vary considerably with the structural system and Construction material. An attempt was made to define drift levels for different structural systems and materials. However, more research is needed, particularly in the development of realistic and quantitative estimates of drift damage relationships. In addition, design criteria that apply to various parameters may be required by different performance objectives.

Table 4.1- Performance levels, corresponding damage state and drift limits

Performance Level	Damage State	Drift
Fully operational, Immediate occupancy	No damage	<0.2%
Operational, Damage control, Moderate	Repairable	<0.5%
Life safe , Damage state	Irreparable	<1.5%
Near collapse, Limited safety, Hazard reduced	Severe	<2.5%
Collapse		>2.5%

To implement performance-based design, there is a need for consensus on the number and definition of performance levels, associated damage Structural system performance can also be quantified using a reliable damage index such as that based on displacement ductility and hysteretic energy. The performance of the contents of the structure and secondary systems may be quantified using damage indices based on different parameters such as floor acceleration levels. Performance levels are associated with earthquake hazard and design levels. Some of the proposed earthquake hazard levels are listed in Table 2. There are unresolved issues concerning the need to improve our quantitative understanding of site-specific ground motion characteristics, their likely effects on structures, and some aspects of near-field effects. This research will lead to reduced uncertainties and the development of improved procedures for prediction of seismic demands.

Table 4.2 - Proposed earthquake hazard levels

Earthquake Frequency	Return Period in years	Probability of Exceedance
Frequent	43	50 % in 30 years
Occasional	72	50 % in 50 years
Rare	475	10 % in 50 years
Very Rare	970	5% in 50 years or 10 % in 100 years
Extremely Rare	2475 2% in 50	2 % in 50 years

The various aspects of pushover analysis and the accuracy of pushover analysis in predicting seismic demands are 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.

4.4 Earthquake Resistant Design Technique

The design Seismic forces acting on a structure as a result of ground shaking are usually determined by one of the following methods:
1.Static Analysis
2.Dynamic Analysis

4.4.1 Static analysis

Although earthquake forces are of dynamic nature, for majority of building equivalent static analysis procedure can be used. These have been developed on the basis of considerable amount of research conducted on the structural behavior of structures subjected to base movements. They determine the shear acting due to earthquake as equivalent base shear. It depends on the weight of the structure, the dynamic characteristics of the building as expressed in the form of natural period or frequency.

4.4.1.1 Linear Static Procedure (LSP)

The LSP is the most basic of the four procedures. A static seismic load is determined using the seismic weight of the structure, an appropriate response spectrum acceleration based on the structure period and damping. In order to more accurately approximate the maximum displacement achieved during a seismic event the LSP utilizes a lateral load that is generally much greater than the capacity of the structure. The load is then distributed vertically based on the seismic weight, height of each story, and building period. The LSP model represents the building with a linear-elastic stiffness that corresponds to the building's stiffness before yield occurs. Although the procedure is described as linear, geometric nonlinearity such as P-delta effects are considered. Because the lateral load used is generally greater than the capacity of the structure ASCE 41-06 provides factors, called M factors, to reduce demands on the individual components of the structure. M factors are used as a measure of the nonlinear deformation capacity of the components in the structure and indirectly incorporate the nonlinear response of a building to the linear analysis procedures.

If the building contains one of the following characteristics, the LSP is not an acceptable analysis procedure:

• The fundamental period of the building is greater than or equal to 3.5 times Ts, where Ts is defined by ASCE 41-06.

Performance Based Analysis of Steel Buildings

• The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story is greater than 1.4.

• The building has a torsional stiffness irregularity in any story

• The building has a vertical stiffness irregularity

• The building has a non-orthogonal lateral-force-resisting system

The building used for the project does not contain any of the previous characteristics therefore the LSP was an acceptable procedure to be used for the project.

4.4.1.2 Linear Dynamic Procedure (LDP)

The LDP is based on loading generated using modal response spectrum analysis or linear time history analysis. The damped general response spectrum provided by ASCE 41-06 for use in the LDP is the same as the one used in the LSP and can be seen in Figure A. Because this project utilizes the general response spectrum, a 5% damped response spectrum is assumed. Because the effect of higher modes is considered in the LDP procedure, the base shear produced is generally smaller than the base shear from the LSP, as illustrated in FEMA 274

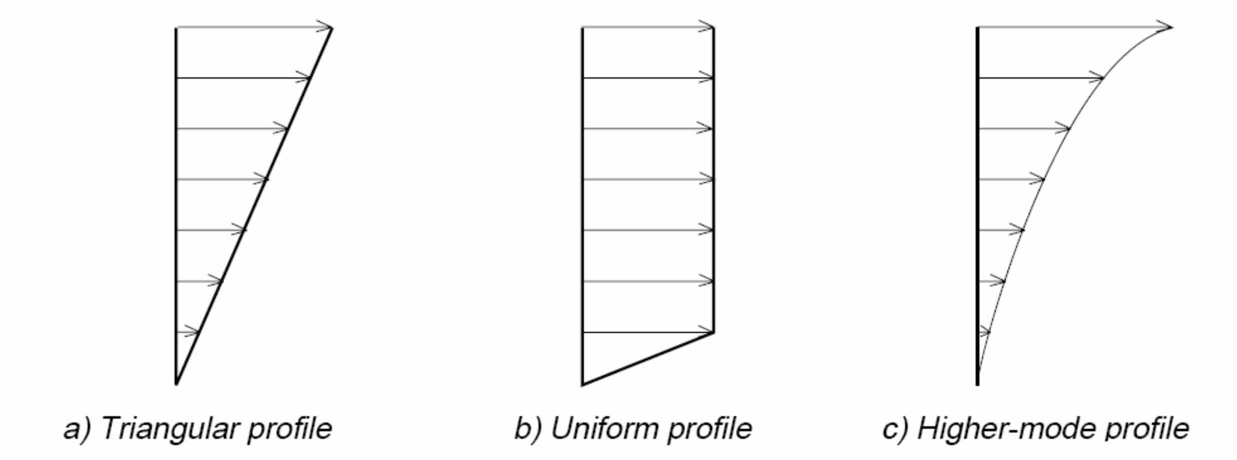


FIGURE 4.2: SAMPLE INERTIA FORCE DISTRIBUTIONS

ASCE 41-06 requires that sufficient modes be considered to capture ninety percent of the building mass in the building's two principal orthogonal directions. As in the LSP, the building is modeled with a linearly elastic stiffness and analyzed with a base shear that is generally greater than the capacity of the structure. *M*-factors and acceptance criteria for the components are the same as in the LSP.

4.4.1.3 Nonlinear Static Procedure (NSP)

The NSP, commonly referred to as a pushover analysis, directly incorporates the nonlinear response of members in the structure. A model of the structure that incorporates the "nonlinear load-deformation characteristics of individual components of a building" is loaded with "monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. The target displacement is developed using procedures provided by ASCE 41-06 and is intended to represent the maximum displacement likely to be experienced during the design level earthquake. The target displacement represents the mean displacement for the design level earthquake for the building at a given location; because considerable scatter can exist about the mean, ASCE 41-06 requires the building be pushed to 150% of the target displacement. Although the strength and deformation levels are checked at the target displacement, the requirement to push the building to 150% of the target displacement encourages the design engineer to investigate likely building performance of the model under extreme load and deformation conditions that exceed the design values (ASCE 41-06). The 150% requirement also allows the engineer to ensure there is not a sudden loss of strength if the target displacement is exceeded.

4.4.1.4 Nonlinear Dynamic Procedure (NDP)

The nonlinear dynamic procedure generally uses a similar computer model as the nonlinear static procedure; however, the loading and computational methods differ. The response of the structure is generated using nonlinear time-history analysis. This Method of analysis is highly sensitive to characteristics of individual ground motions and to assumptions made in computer modeling. Because of the NDP's high sensitivity, to ground motions and modeling assumptions, ASCE 41-06 requires the NDP be Performed.

4.5 DESIGN EVALUATION

Acceptable procedures for design evaluation include:
(1) Elastic analysis.
(2) Component-based elastic analysis procedure.
(3) Simplified nonlinear analysis methods.
(4) Dynamic nonlinear time history analysis.

For the analysis to be reliable and credible, it is necessary to ensure that:
• Appropriate site-specific ground motion with specified hazard level can be generated with confidence.
• The structural model is realistic.

• The cyclic load-deformation model for each element is representative of the behavior.
• Analysis procedures and interpretation tools are reliable.
• Identification of modes and sequence of element and component failure are also realistic.

4.6 DESIGN CRETERIA

A fundamental question in performance-based design is to validate the appropriateness of the selected performance levels, the specific parameters used to define their minimum performance, and the seismic hazard definitions. For the case of three performance levels (serviceability, damage control and life safety or collapse prevention), three corresponding structural characteristics (stiffness, strength and deformation capacity) dominate the performance as illustrated in Fig.4.3.

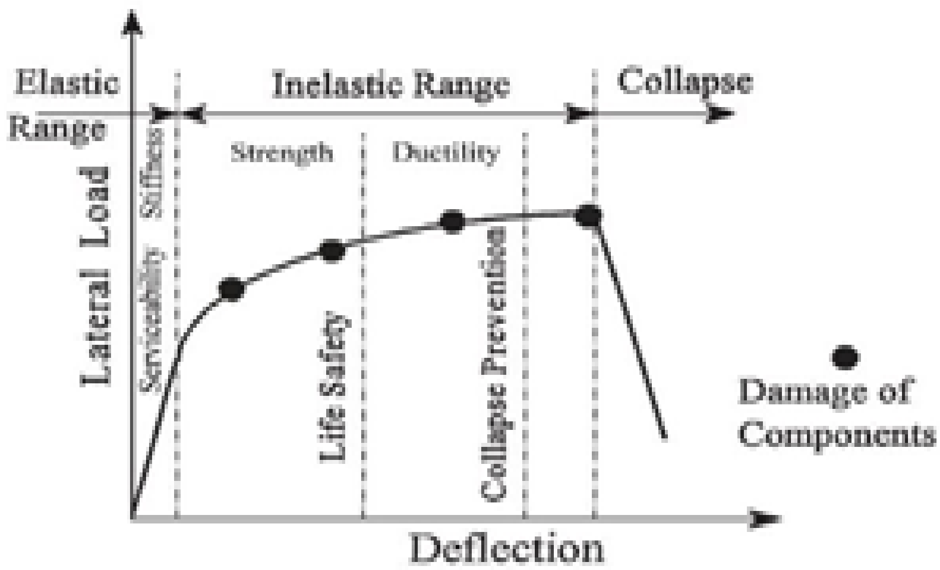


Fig .4.3: Typical performance curve for the structure

If more intermediate performance levels are selected, then it becomes difficult to define which structural characteristics dominate the performance. It can be argued that different performance objectives may impose conflicting demands on strength and stiffness [13]. Much research is needed to associate the displacement or drift limits with the damage states and the stated general performance objectives. The displacements or drift limits are also functions of the structural system and its ability to deform (ductility). Design criteria may be established on the basis of observation and experimental data of deformation capacity. For example, near the collapse point, the drift limits of structural walls are different from a moment-resisting frame, which suggest that different structural systems will undergo unequal displacements. Other issues related to the damage evaluation are the quantification of the relationship between building restoration time/costs and earthquake hazard level. It is of interest to identify the damage level at which building restoration becomes impractical, which represents the state of irreparable damage.

4.7 DEFORMATION-CONTROLLED DESIGN

The most suitable approach to achieve the objectives of performance- based seismic design with displacement based performance objectives appears to be the deformation-controlled design approach. It is anticipated that deformation-controlled design will be implemented in future codes, both by enhancing force-based design through verification of deformation targets and by the development of direct deformation-based design procedures. Computer tools are needed to predict the inelastic dynamic response of complex structures. Extensive efforts are believed to be necessary to develop versatile and robust, yet efficient, numerical standard programs to simulate seismic response of three-dimensional structures taking into account various nonlinearities. It is necessary that these tools be design oriented rather than Research oriented. The general design methodology may have to go beyond the methods that assume a single-degree of freedom representation of the structure. This assumption results in severe restrictions on the reliability of the estimated performance. At the risk of sacrificing simplicity, it is important to obtain a good estimate of the local displacements within the structure, take higher mode effects into consideration, and account for the sequence of element damage. Nonlinear static pushover analysis coupled with new methods (other than SDOF-based spectra) to determine demand, or nonlinear inelastic dynamic analysis, may provide a more reliable prediction of the performance.

Chapter – 5

Pushover curve development and capacity spectrum method

5.1 Capacity spectrum method

The Nonlinear Static Procedure in these documents is based on the capacity spectrum method originally developed by Freeman et al. (1975) and Freeman (1978). It consists of the following steps:

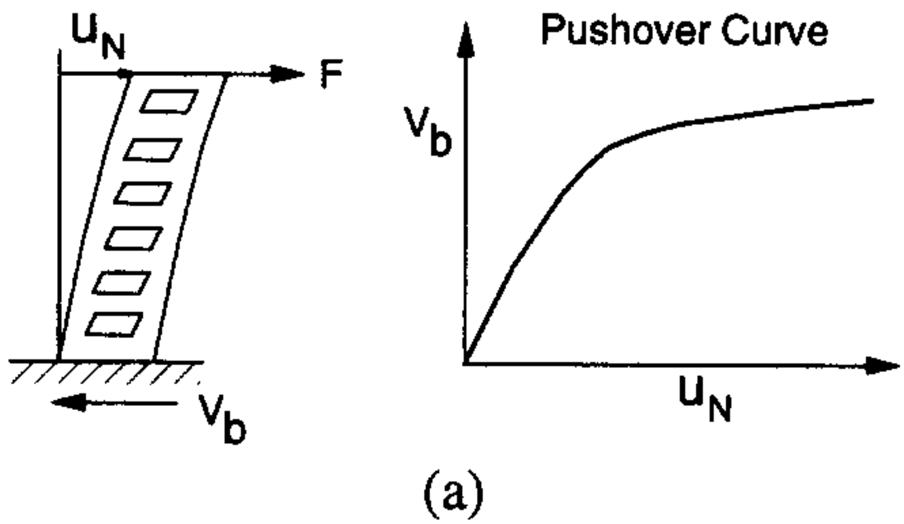


FIG.5.1 DEVELOPMENT OF PUSHOVER CURVE

1. Develop the relationship between base shear, V_b, and roof (N^{th} floor) displacement, u_n (Fig. a), commonly known as the pushover curve.

2. Convert the pushover curve to a capacity diagram as shown in Fig. b
3. Convert the elastic response (or design) spectrum from the standard pseudo-acceleration A versus natural period, T_n, format to the A- D format, where D is the deformation spectrum ordinate (Fig. c).

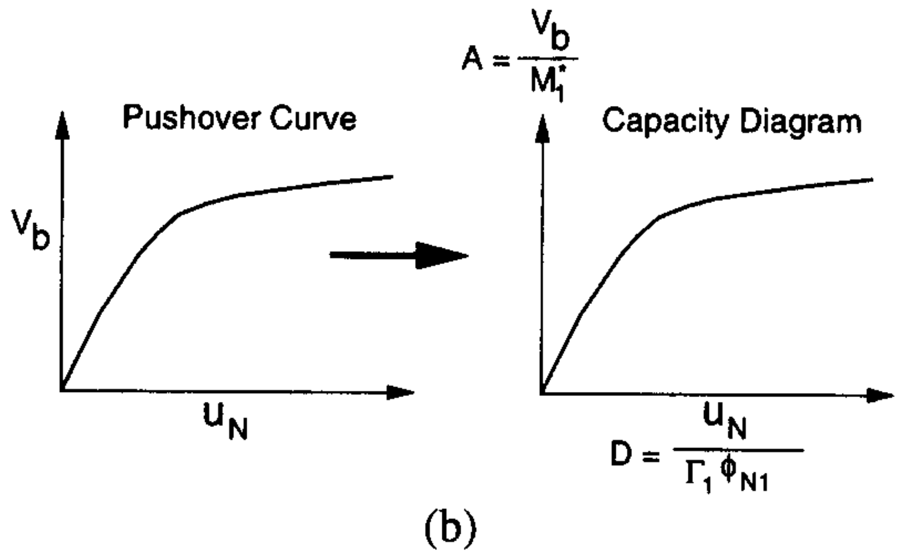


FIG.5.2 CONVERSION OF PUSHOVER CURVE TO CAPACITY DIAGRAM

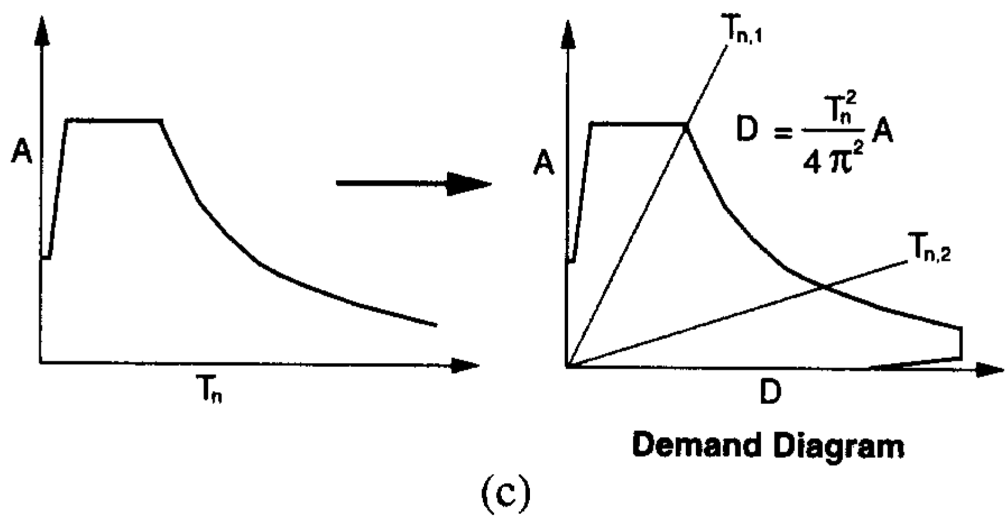
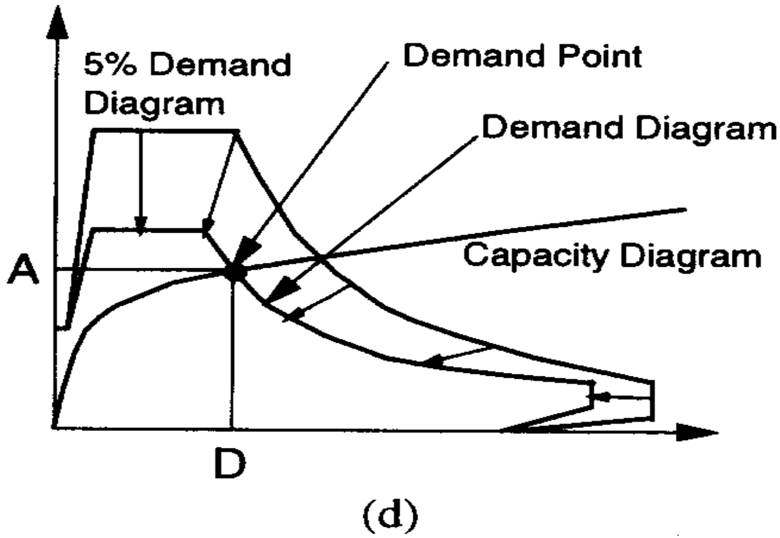


FIG.5.3 CONVERSION OF ELASTIC RESPONSE SPECTRUM FROM STANDARD FORMAT TO A-D FORMAT



(d) Determination of displacement demand

FIGURE 5.4: CAPACITY SPECTRUM METHOD

4. Plot the demand diagram and capacity diagram together and determine the displacement demand (Fig. 1d). Involved in this step are dynamic analyses of a sequence of equivalent linear systems with successively updated values of the natural vibration period, *Teq* , and equivalent viscous damping, *Zeq* (to be defined later).

5. Convert the displacement demand determined in Step 4 to global (roof) displacement and individual component deformation and compare them to the limiting values for the specified performance goals.

5.2 PUSHOVER ANALYSIS PROCEDURE

Pushover analysis can be performed as either force-controlled or displacement controlled depending on the physical nature of the load and the behavior expected from the structure. Force-controlled option is useful when the load is known (such as gravity loading) and the structure is expected to be able to support the load. Displacement controlled procedure should be used when specified drifts are sought where the magnitude of the applied load is not known in advance, or where the structure can be expected to lose strength or become unstable.

1. A two or three-dimensional model that represents the overall structural behavior is created.

2. Bilinear or tri-linear load-deformation diagrams of all important members that affect lateral response are defined.

3. Gravity loads composed of dead loads and a specified portion of live loads are applied to the structural model initially.
4. A pre -defined lateral load pattern, which is distributed along the building height, is then applied.
5. Lateral loads are increased until some member(s) yield under the combined effects of gravity and lateral loads.
6. Base shear and roof displacement are recorded at first yielding.
7. The structural model is modified to account for the reduced stiffness of yielded member(s).
8. Gravity loads are removed and a new lateral load increment is applied to the modified structural model such that additional member(s) yield. Note that a separate analysis with zero initial conditions is performed on modified structural model under each incremental lateral load. Thus, member forces at the end of an incremental lateral load analysis are obtained by adding the forces from the current analysis to the sum of those from the previous increments. In other words, the results of each incremental lateral load analysis are superimposed.
9. Similarly, the lateral load increment and the roof displacement increment are added to the corresponding previous total values to obtain the accumulated values of the base shear and the roof displacement.
10. Steps 7, 8 and 9 are repeated until the roof displacement reaches a certain level of deformation or the structure becomes unstable.
11. The roof displacement is plotted with the base shear to get the global capacity (pushover) curve of the structure (Figure 4.2).

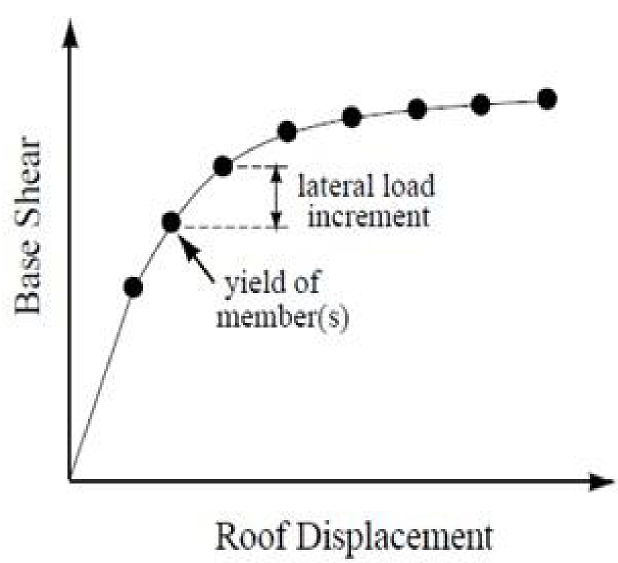


FIGURE 5.5: GLOBAL CAPACITY (PUSHOVER) CURVE OF STRUCTURE

5.3 PUSHOVER METHODS

Pushover analysis is a technique in which any structure is subjected to incremental lateral loads, which represent the inertia forces in an earthquake. The sequence of cracks, yielding, plastic hinge formation and failure of structural components are noted. For this procedure, a relation between bases shears and control node displacement plotted.

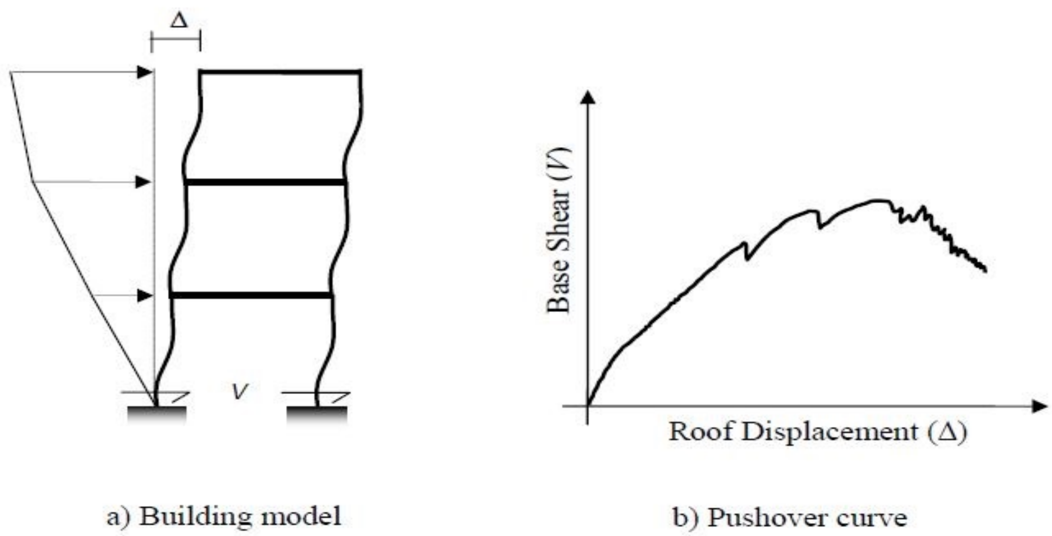


FIG. 5.6: SCHEMATIC REPRESENTATION OF PUSHOVER ANALYSIS PROCEDURE

Target displacement is the overall global displacement of a structure subjected to the design earthquake. This plays a key role in the pushover analysis. The methods to evaluate target displacement are Displacement Coefficient Method (DCM) of FEMA 356 and Capacity Spectrum Method (CSM) of ATC 40.

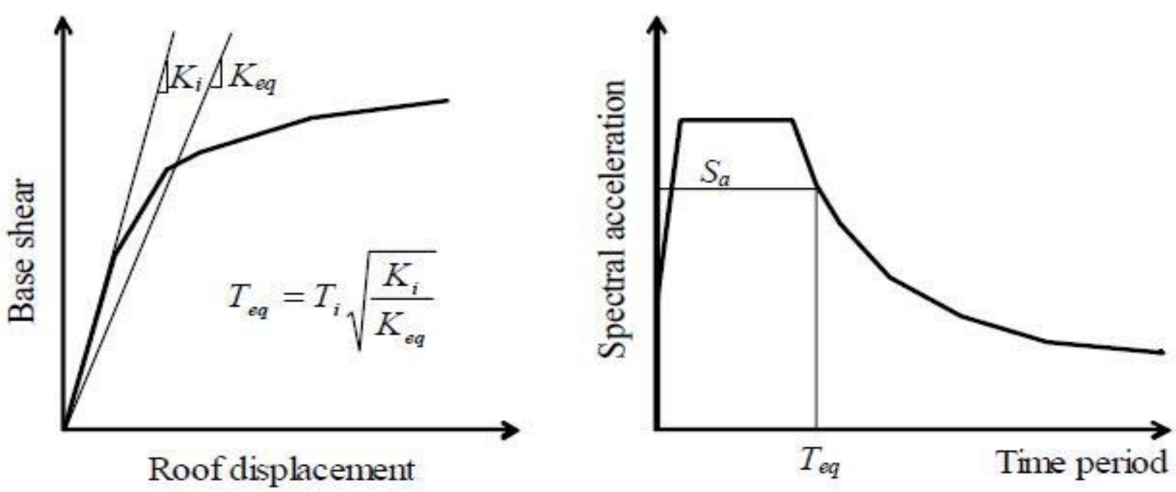


FIG. 5.7: SCHEMATIC REPRESENTATION OF DISPLACEMENT COEFFICIENT METHOD (FEMA 356)

5.4 Procedure to Determine Demand

Two Methodologies are presented…
A. Capacity Spectrum Method (CSM**)**
B. Displacement Coefficient Method (DCM)

In CSM method, the pushover curve is used in a displacement response spectrum (ADRS) format and the effective period and damping is calculated with the use of estimated ductility. For the above procedure, the pushover curve is used in an acceleration displacement response spectrum (ADRS) format, which could be obtained using the dynamic properties of the system. The pushover curve in an ADRS format is termed as capacity spectrum for the structure. The seismic ground motion is represented by a response spectrum in the same ADRS format and it is termed as demand spectrum. The Pushover analysis procedure is used to determine the seismic demands on any structure. But in case of high rise buildings, it is sometimes it is difficult to apply the pushover analysis the higher modes are not accounted in such case. So a modal pushover analysis (MPA) procedure was used which considers the redistribution of inertia forces after the structure yields proposed by Chopra *et al*. (2001). The total seismic demand can be estimated by the combination of the first two or three terms of expansion. This provides a much more accurate estimation of seismic demands. Despite of the accuracy, it still doesn't avoid the consideration of lateral force distributions.

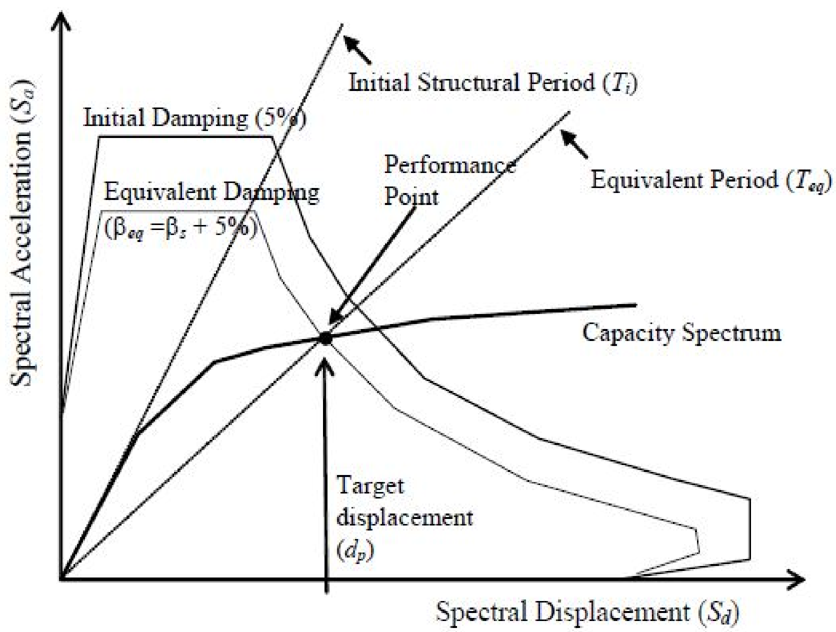


FIGURE 5.8 RELATIONS BETWEEN ACCELERATION AND DISPLACEMENT

5.4.1 CAPACITY SPECTRUM METHOD (ATC-40 PROCEDURE A) (CSM)

ATC-40 presents the recent three versions of the Capacity Spectrum Method to estimate the earthquake induced displacement demand of inelastic systems. All three procedures are based on the same underlying principles that these procedures are approximate since they avoid the dynamic analysis of inelastic system. Instead, the displacement demand of inelastic system is estimated by dynamic analysis of a series of equivalent linear systems with successively updated values of T_{eq} and ζ_{eq}. Procedures A and B are analytical and suitable to computer implementation while Procedure C is graphical and more suitable for hand analysis. In this study, the procedure, which is equivalent to Procedure A in ATC-40 except that it is specialized for bilinear systems, was utilized. The procedure consists of the following steps:

1. Perform same Steps 1-3 described in the approach proposed by ATC-40 in Section 5.2

2. Convert 5% elastic response (demand) spectrum from standard S_a vs T format to S_a Vs S_d (ADRS) format. For this purpose, the spectral displacement, S_d, can be computed using Eqn. 5.13 for any point on standard response spectrum. (See Figure 4.10).

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

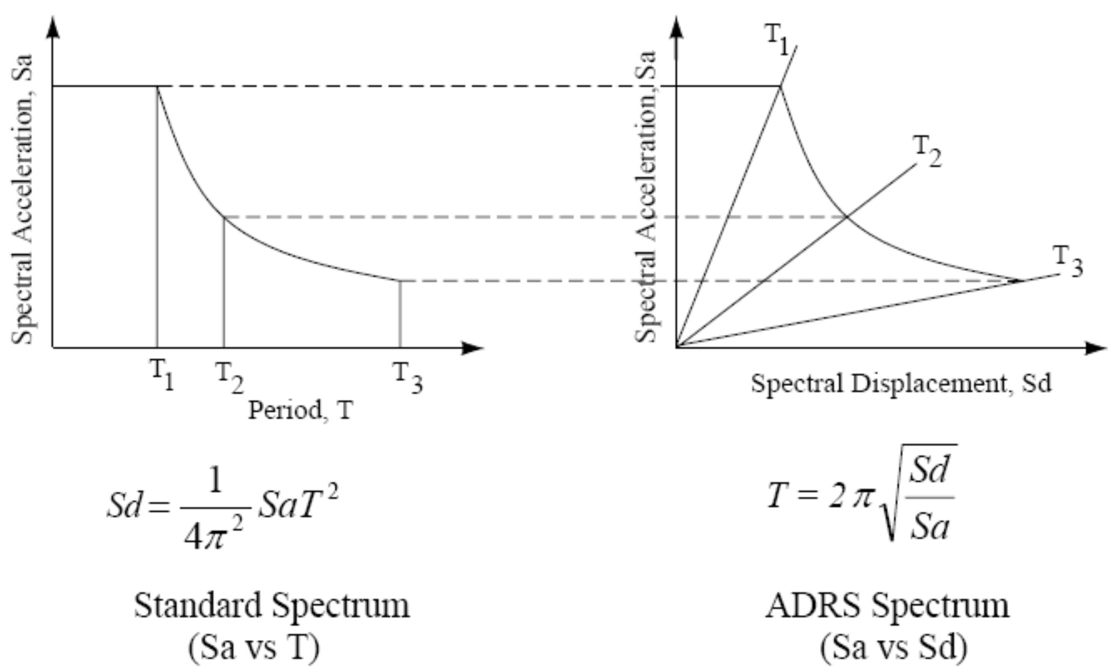


FIGURE 5.9: RESPONSE SPECTRUM IN STANDARD AND ADRS FORMATS

3. Initially, assume a peak spectral displacement demand $S_{di} = S_d$ (T1, ξ = 5%) determined for period T_1 from the elastic response spectrum.

4. Compute displacement ductility ratio $\mu = S_{di} / S_{dy}$

5. Compute the equivalent damping ratio ξeq from the following equation:
eq o ξ = 0.05 +κ ·ξ (5.14)

The most common method for defining equivalent viscous damping ratio is to equate the energy dissipated in a vibration cycle of the inelastic system and of the equivalent linear system. Based on this concept, Chopra defines equivalent viscous damping ratio as

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

ED: the energy dissipated in the inelastic system given by the area enclosed by the hysteresis loop
ES: maximum strain energy

Substituting ED and ES in Equation leads to

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

Where:
μ: displacement ductility ratio
α: Ratio of average post-elastic stiffness of capacity curve to effective elastic stiffness of the capacity curve

The κ-factor depends on the structural behavior of the building which in turn depends on the quality of seismic resisting system and the duration of ground shaking. ATC-40 defines three different structural behavior types. Type A represents hysteretic behavior with stable, reasonably full hysteresis loops while Type C represents poor hysteretic behavior with severely pinched and/or degraded loops. Type B denotes hysteresis behavior intermediate between Type A and Type C (Table 4.1)

Table 5.1 : Structural Behavior Types (ATC-40)

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

6. Plot elastic demand spectrum for ζ_{eq} determined in Step 7 and bilinear capacity spectrum on same chart and obtains the spectral displacement demand S_{dj} at the intersection (Figure 5.8)

Check for convergence. If $(S_{dj}-S_{di})/S_{dj}$ <= tolerance (=0.05) then earthquake induced spectral displacement demand is $S_d = S_{dj}$. Otherwise, set $S_{di} = S_{dj}$ (or another estimated value) and repeat Steps 6-9.

8. Convert the spectral displacement demand determined in Step 9 to global (roof) displacement by multiplying estimated spectral displacement demand of equivalent SDOF system with first modal participation factor at the roof level.

5.4.2 DISPLACEMENT COEFFICIENT METHOD (FEMA-356) (DCM)

The Displacement Coefficient Method described in FEMA-356 is an approximate method which provides a direct numerical calculation of maximum global displacement demand of structures. Inelastic displacement demand, δ_t is calculated by modifying elastic displacement demand with a series of displacement modification factors. Bilinear representation of capacity curve is required to be used in the procedure. The procedure described in Section 5.2 is recommended for bilinear representation. After the construction of bilinear curve, effective fundamental period (Te) of the structure is calculated using Equation.

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

Where:
T_e: effective fundamental period (in seconds)
T_i: elastic fundamental period (in seconds) in the direction under consideration
K_i: elastic lateral stiffness of the structure in the direction under consideration
K_e: effective lateral stiffness of structure in the direction under consideration

The target displacement, δ_t, is computed by modifying the spectral displacement of an equivalent SDOF system using the coefficients as shown below.

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

Where:

C_0: Modification factor to relate spectral displacement and likely roof displacement of the structure. The first modal participation factor at the roof level is used.

C_1: Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2: Modification factor to represent the effect of hysteresis shape on the maximum displacement response. In this study, C2 was taken as 1.1 for both elastic and inelastic deformation levels. As the estimates of Displacement Coefficient Method (FEMA-356) depend on the coefficient C2, the coefficient C2 should be taken as unity in the elastic range and should take the specified value for the considered performance level in the inelastic range for seismic performance evaluation purposes.

C_3: Modification factor to represent increased displacements due to second-order effects.

S_a: Response spectrum acceleration at the effective fundamental period of the structure.

T_e: Effective fundamental period of the structure.

In this method, different target displacements can be obtained for different seismic performance levels. In this study, target displacements for each ground motion record were calculated for life safety performance level.

5.5 CONSTANT DUCTILITY PROCEDURE (CHOPRA&GOEL)

Chopra and Goel proposed an improvement to Capacity Spectrum Method described in ATC-40. The improved capacity-demand diagram method uses constant ductility demand spectrum to estimate seismic deformation of equivalent SDOF system representation of MDOF structure.

There are three versions of the proposed improved procedure; Procedure A, Procedure B and Numerical procedure. Procedures A and B are graphically similar to ATC-40 Procedures A and B. In this study, Procedure A was used to estimate seismic displacement demand of inelastic SDOF systems. The procedure consists of the following steps:

1. Perform same Steps 1-3 described in the approach proposed by ATC-40.
2. Obtain elastic 5% damped response spectrum and a set of inelastic response spectra for various ductility levels.
3. Plot the bilinear capacity spectrum and demand spectra together.
4. Determine the displacement demand as follows: Compute the ductility value at the intersection of capacity spectrum and each demand spectrum (u_m / u_y). When the computed ductility matches the ductility of intersecting demand spectrum, that intersection point is selected as inelastic displacement demand of SDOF system.
5. Convert the spectral displacement demand determined in Step 4 to global (roof) displacement by multiplying estimated spectral displacement demand of equivalent SDOF system with first modal participation factor at the roof level.

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

5.6 Pushover Analysis Methods

Displacement-based design methods make use of non-linear static, or pushover, analysis (Fajfar and Fischinger, 1988; Lawson et al. 1994; Krawinkler and Seneviratna, 1998). Appropriate lateral load patterns are applied to a numerical model of the structure and their amplitude is increased in a stepwise fashion. A non-linear static analysis is performed at each step, until the building fails. A pushover curve (base shear against top displacement) can then be plotted. This is then used together with the design response spectrum to determine the top displacement under the design earthquake – termed the target displacement or performance point. The non-linear static analysis is then revisited to determine member forces and deformations at this point. These methods are considered a step forward from the use of linear analysis and ductility modified response spectra, because they are based on a more accurate estimate of the distributed yielding within a structure, rather than an assumed, uniform ductility. The generation of the pushover curve also provides the engineer with a good feel for the nonlinear behavior of the structure under lateral load. However, it is important to remember that pushover methods have no rigorous theoretical basis, and may be inaccurate if the assumed load distribution is incorrect. For example, the use of a load pattern based on the fundamental mode shape may be inaccurate if higher modes are significant, and the use of a any fixed load pattern may be unrealistic if yielding is not uniformly distributed, so that the stiffness profile changes as the structure yields. The main differences between the various proposed methods are (i) the choices of load patterns to be applied and (ii) the method of simplifying the pushover curve for design use. The methods used in this study are summarized below.

1. Pushover analysis – apply the following two load patterns:
Modal – the acceleration distribution is assumed proportional to the fundamental mode shape. The inertia force F_i on mass i is then:

$$F_i = \frac{m_i \phi_i}{\sum_{j-1}^{n} m_j \phi_j} F_b$$

Where: F_b is the base shear, m_i the ith storey mass and φ_i the mode shape coefficient for the i^{th} floor. If the fundamental mode shape is assumed linear then φ_i is proportional to storey height h_i and Equation can be written as:

$$F_i = \frac{m_i h_i}{\sum_{j-1}^{n} m_j h_j} F_b$$

Uniform – the acceleration is assumed constant with height. The inertia forces are then given by:

$$F_i = \frac{m_i}{\sum_{jj-1}^{n} m} F_b$$

Plot pushover curve F_b vs d, with maximum displacement d_m.

2. Convert pushover curves to equivalent SDOF system using:

$$\Gamma = \frac{\sum_{j=1}^{n} m_j \phi_j}{\sum_{j=1}^{n} m_j \phi_j^2} \qquad F^* = \frac{F_b}{\Gamma} \qquad d^* = \frac{d}{\Gamma}$$

3. Simplify to elastic-perfectly plastic as shown in Figure 5.3. Set F_y equal to maximum load, choose D_y to give equal areas under actual and idealized curves.

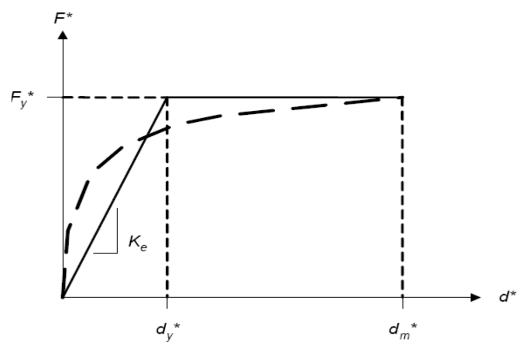


FIGURE 5.10 IDEALIZATION OF PUSHOVER CURVE IN EC8

4. Calculate target displacement of SDOF system under design earthquake:

$$d_t^* = S_a \frac{T^2}{4\pi^2} \qquad T \geq T_c \qquad q = \frac{S_a}{(F_y^* / m^*)}$$

$$d_t^* = S_a \frac{T^2}{4\pi^2} \frac{1}{q}\left[1 + (q-1)\frac{T_c}{T}\right] \qquad T < T_c \qquad m^* = \sum_{j=1}^{n} m_j \phi_j$$

T is the elastic period of the idealized SDOF system, S_a is the spectral acceleration corresponding to T, and T_c is the corner period of the design response spectrum, i.e. the period at the transition between the constant acceleration and constant velocity parts of the curve.

5. Transform target displacement back to that of the original MDOF system using Equation.

6. Check that $d_t \leq d_m/1.5$. Check member strengths and storey drifts are acceptable at this value of d_t.

CHAPTER - 6

Pushover analysis using SAP2000 of G+14 multistorey building

6.1 General

The study in the thesis is based on nonlinear analysis of steel frames. This chapter presents a summary of various parameters defining the computational models, the basic assumptions and geometry considered for the study.

6.2 Details of the Model

Three dimensional model of steel moment resisting frame of 2 bay in X and 3 bay in Y direction with total 15 number of stories are taken for the analysis. In this frame the storey height is 3.84 m high, 4m wide and bays length is 6 meters

Table 6.1 Model details

1	No. of bays in X-Dir.	2 bay
2	No. of bays in Y-Dir.	3 bay
3	No. of stories	15 nos
4	Total Height of building	57.60 meter
5	Each Storey Height	3.84 meter
6	Length of each bay in X-dir.	4.00 meter
7	Length of each bay in Y-dir.	4.00 meter
8	Steel Used in Frame	Fe 345
9	Size of Beam	ISMB 350
10	Size of Column	600x600x0.25 mm
11	Thickness of slab	150 mm
12	Concrete Used	M-25
13	Size of Bracings	65x65x8 mm
16	μ- steel	0.3
17	μ- Concrete	0.2

6.2.1 Defining the material properties, structural components and modeling the structure

The required material properties like mass, weight density, modulus of elasticity, shear modulus and design values of the material used can be modified as per requirements or default values can be accepted. The columns have been fixed in all six degrees of freedom at the base. Slabs are defines as area elements having the properties of shell elements with the required thickness.

6.2.2 Assigning Loads

After having modeled the structural components, all possible load cases are assigned.

6.2.2.1 Gravity loads

Gravity loads on the structure include the self weight of beams, columns, slabs, walls, and other permanent members. Self weight of beams and columns and slabs is automatically considered by the program itself.

Wall load= unit weight of brick work x thickness of wall x height of wall.
Unit weight of brick work = 20 KN/m^3
Thickness of wall = 0.115 m
Height of wall = 3.84 m
Height of parapet Wall = 1.20 m

Wall load on all floors except roof = 20 x 0.115 x (3.84-0.35-0.15) = 7.682KN/m

Wall load on Roof level(Parapet wall) = 20 x 0.115 x 1.20 = 2.76 KN/m

Live loads have been assigned as uniform area loads on the slab elements as per IS 1893(part-I) 2002
Live load on roof is 2 KN/m^2
Live load on all other floors is 4 KN/m^2
Floor Finish on roof is 3 KN
Floor Finish on all other floors is 1.5 KN

The program itself automatically considers earthquake loads on the structure.

According to IS 1893(part 1) 2002 for the limit state design, the following combinations have been defined

1.5(DL+LL)
1.2(DL+LL+EL)
1.2(DL+LL-EL)
1.2(DL+LL+WL)
1.2(DL+LL-WL)
1.5(DL+EL)
1.5(DL-EL)
0.9DL+1.5EL
0.9DL-1.5EL

DL- Dead Load
LL- Live Load
EL- Earthquake Load
WL- Wind Load

6.2.3 Analysis of the structure

Namely three types of analysis procedures have been carried out for determining the various structural parameters of the model. Here we are mainly concerned with the behavior of the structure under the effect of ground motion and dynamic excitations such as earthquakes and the displacement of the structure in the elastic range.
The analysis carried out is as follows:
Response spectrum analysis
Pushover analysis

6.2.3.1 Response Spectrum Method

Modal analysis is carried out for obtaining the natural frequencies, modal mass participation ratios and other modal parameters of the structure. Response spectrum analysis of this modal are done in Zone – IV where

$$Z= 0.24 \qquad R= 5.00$$

$$I= 1.00 \qquad S_a/g = 2.5$$

CHAPTER – 7

RESULTS FOR PUSHOVER ANALYSIS

The inelastic analysis of the structure under static and dynamic loading is performed by the nonlinear analysis using SAP2000.

7.1 Model properties

Modal properties of the structure model were obtained from the linear dynamic model analysis which is the primary step used in pushover analysis.

Table 7.1: ELASTIC DYNAMIC PROPERTY OF THE MODEL.

Modes	**Model Period (in Second)**
Mode 1	4.313
Mode 2	3.402
Mode 3	2.338
Mode 4	1.312
Mode 5	1.009
Mode 6	0.733
Mode 7	0.678
Mode 8	0.517
Mode 9	0.413
Mode 10	0.396
Mode 11	0.333
Mode 12	0.270

Modal period increases as the height of the building increases.

7.2 Base Shear (KN) vs Displacement (m):

In our analysis my modal give a linear curve up to a level of base shear after that its slightly irregular curve is get. Fig. Given below shows approx. linear curve in between base shear and displacement.

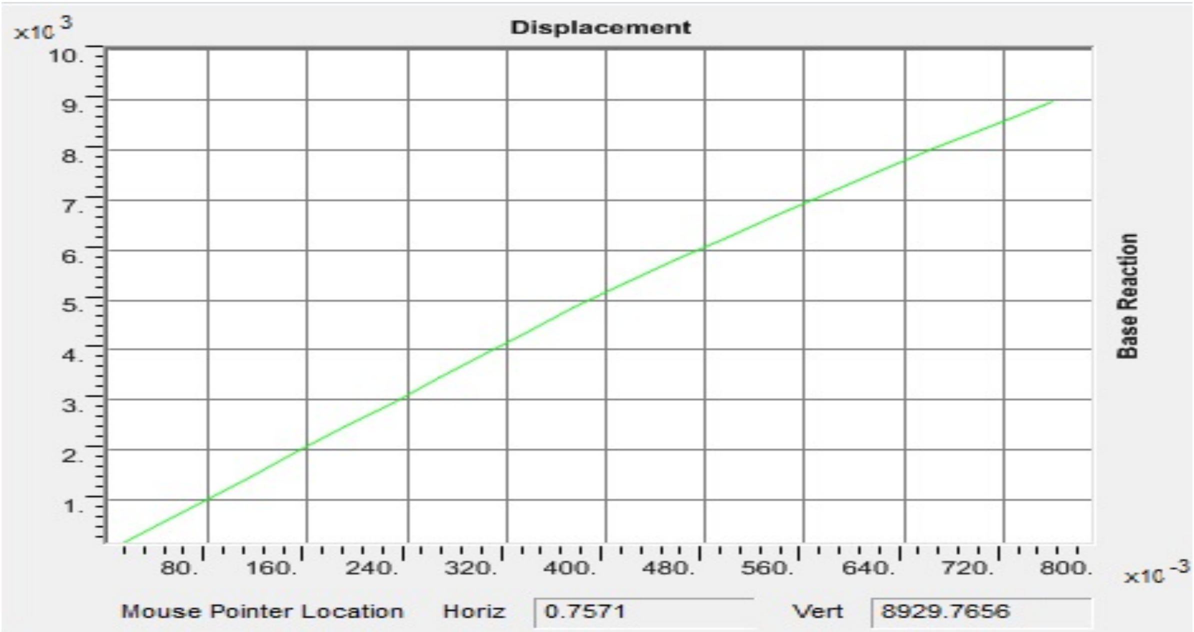


Fig.7.1 Graph of base shear vs Displacement

7.3 Storey Height Vs Displacement

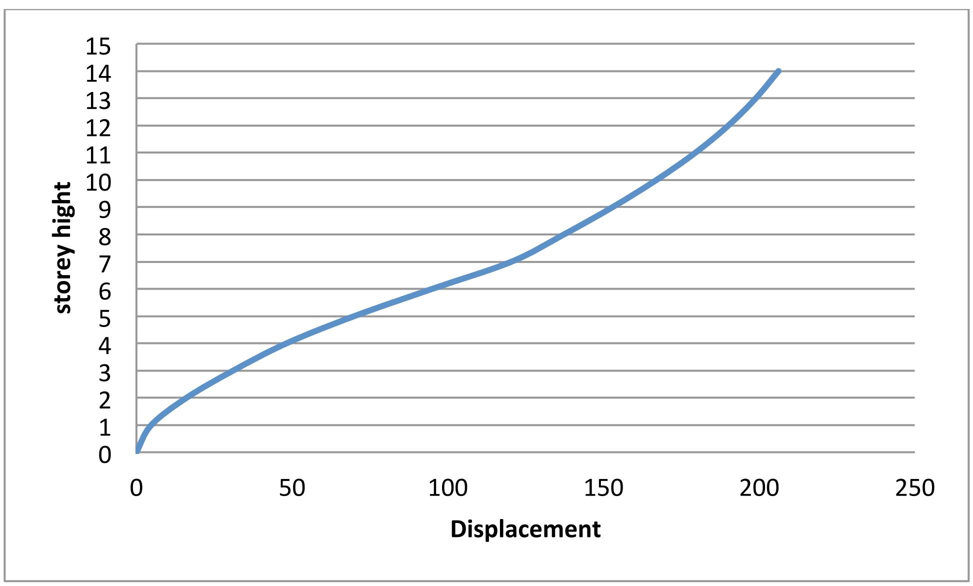


Fig.7.2 Graph of Storey Height vs Displacement

7.4 Capacity Spectrum

The curve shows the variation of spectral acceleration vs spectral displacement for the considered frame.

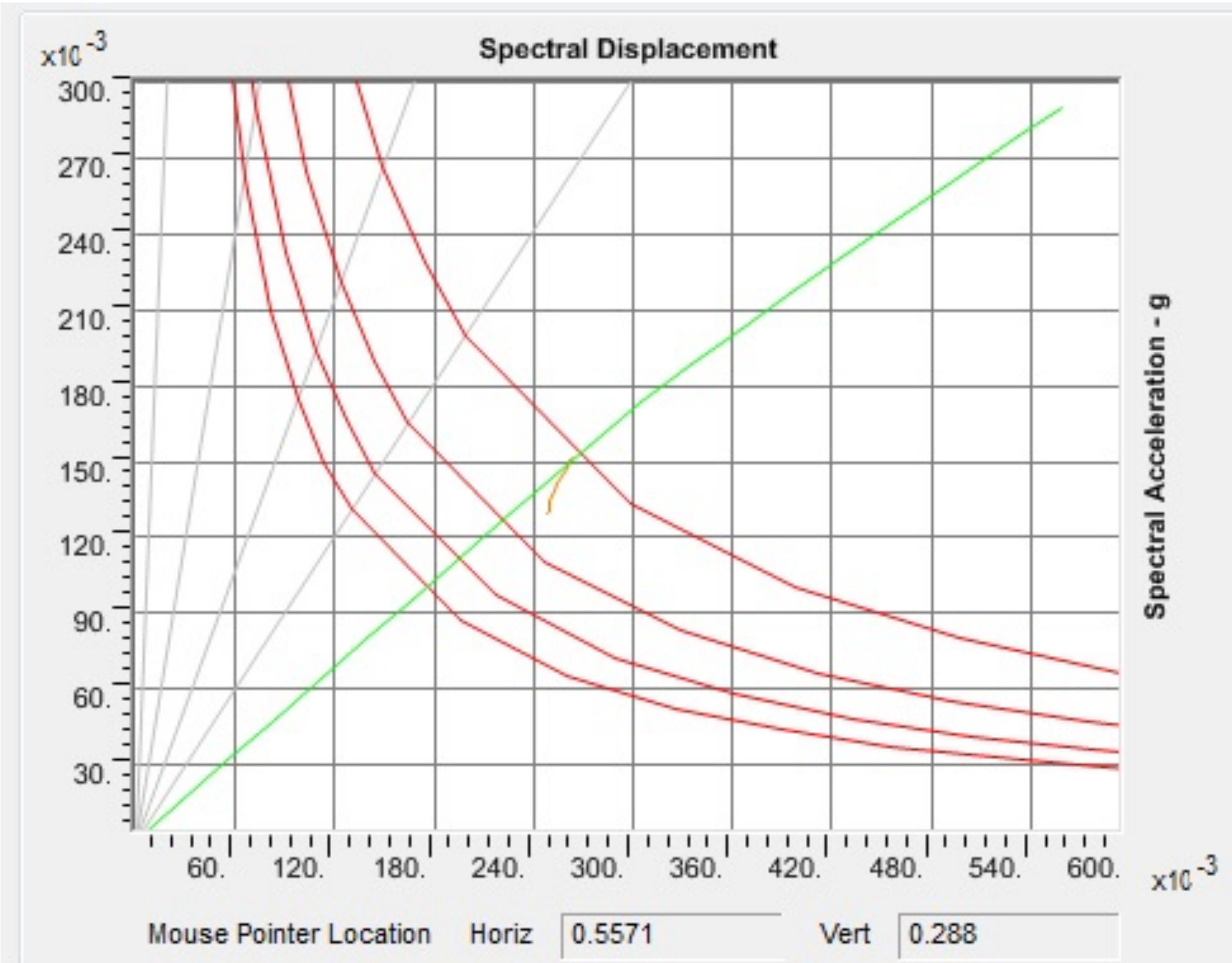


Fig 7.3 Graph of capacity spectrum (ATC40)

Performance Point (V, D)
(4639.667 , 0.359)

Performance Point (Sa, Sd)
(0.15 , 0.263)

Performance Point (Teff, Beff)
(2.651 , 0.051)

The main output of a pushover analysis is in terms of response demand versus capacity. The structure has a good resistance because the demand curve intersects the capacity envelop near the elastic range. From Fig it can be concluded that the structure will behave safely during the imposed seismic excitation and not needed for retrofitting.

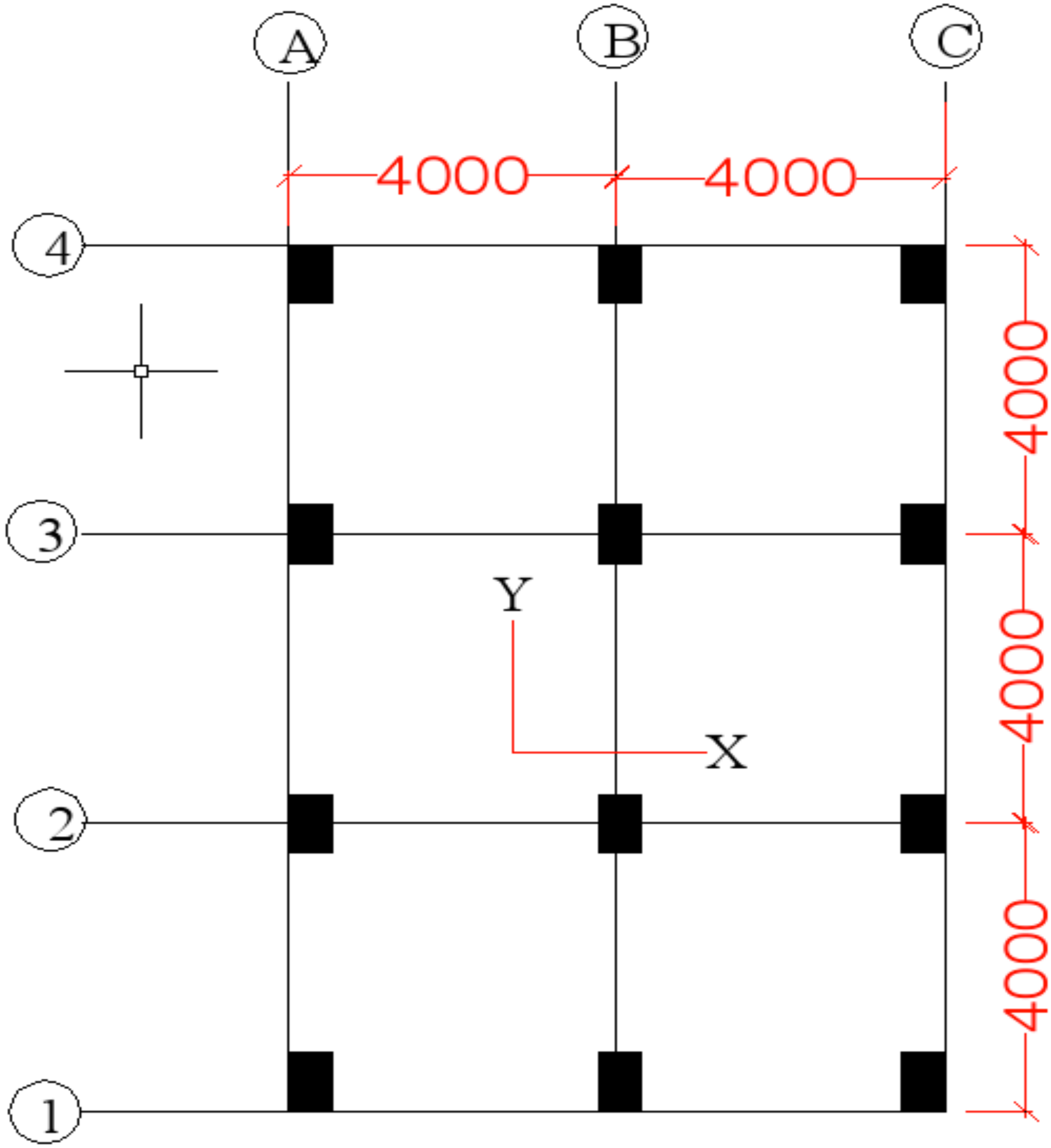


FIG 7.4 PLAN SHOWING LOCATION OF COLUMN FOR SHEAR FORCE AND BENDING MOMENT (ALL DIMENSIONS IN MM)

7.2 Comparison of Shear Force and Bending Moment in linear static and non linear analysis

	SHEAR FORCE				
Column No.	**Storey**	**Linear static analysis**		**Non linear static analysis**	
		V2	**V3**	**V2**	**V3**
C3 - 4storey		0.966	51.726	2.928	244.94
C3 8 storey		0.687	43.51	4.371	90.415
C3 12 storey		0.372	28.024	5.135	16.374
C3 15 storey		0.797	0.481	7.307	0.25

	BENDING MOMENT				
Column No.	**Storey**	**Linear static analysis**		**Non linear static analysis**	
		M3	M2	M-3	M-2
C3	4	2.91	163.49	8.353	1315.1495
C3	8	1.87	115.45	9.888	185.88
C3	12	2.664	132.29	10.5037	175.375
C3	15	2.717	48.33	17.39	80.86

7.6 Formation of Plastic Hinges

Below fig shown point of formation of plastic hinges in our structure. We observe that formation of hinges start from beam-ends and then in lower storey column and then propagate to the upper stories. This formation is random.

Sequential Formation of plastic hinge gives us failure pattern of column/beam failure. This is valuable information in the dynamic analysis.

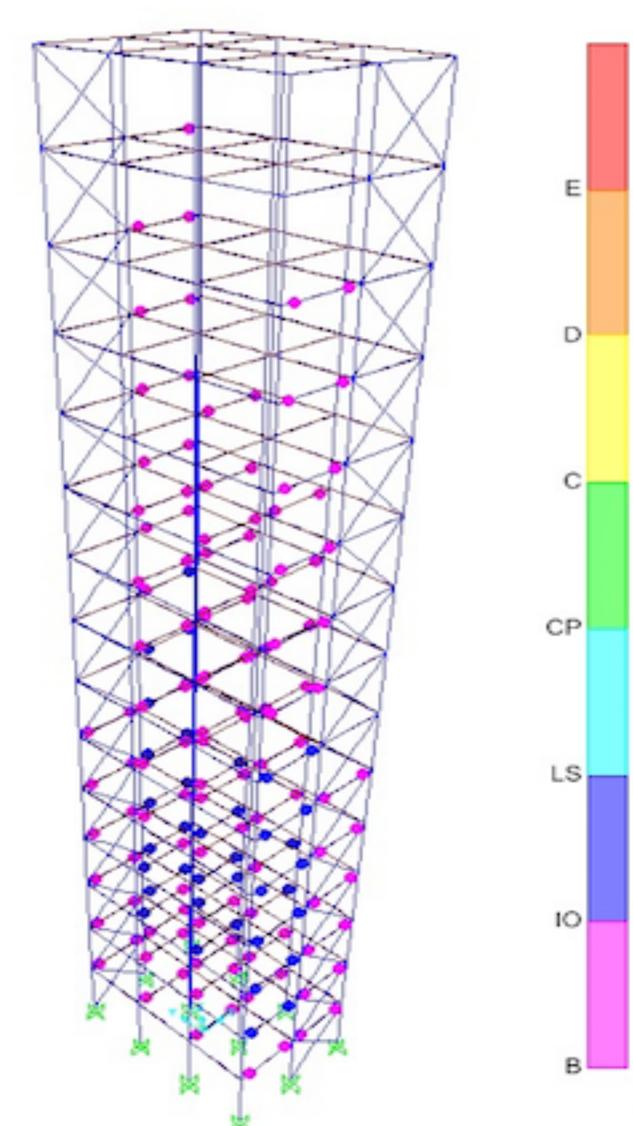


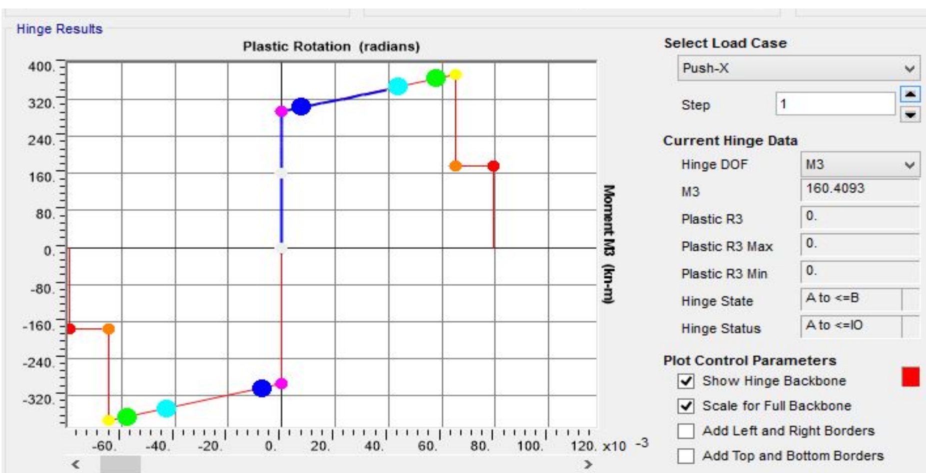
FIGURE 7.5: HINGES FORMATION PATTERN FOR 15 STOREY BUILDING

7.6.1 Beam Hinge 408 H1 (auto HS)- computer generated hinge by our design in beam.

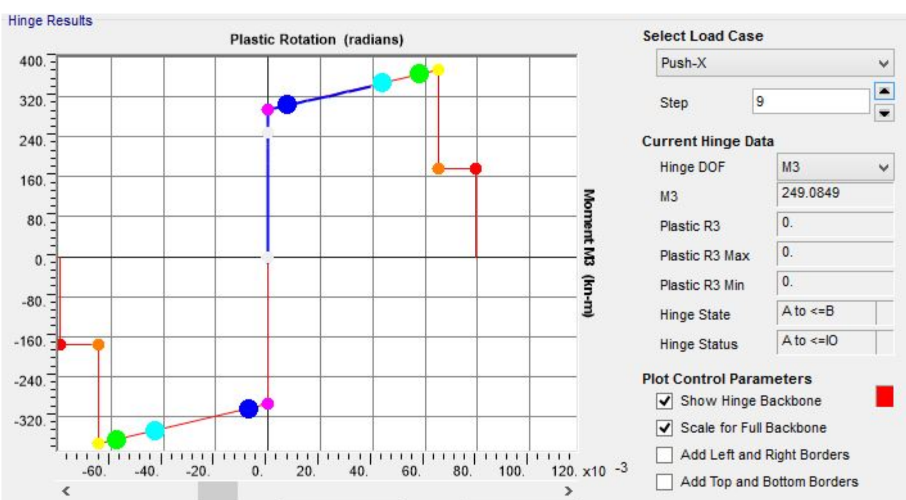
Table 7.3 Beam Hinge 408 H1 (auto HS)- computer generated hinge by our design in beam

Step	Moment	Rotation
1	160.40	0
9	249.08	0
16	295.81	1.75
25	304.94	9.23
33	341.98	0.03

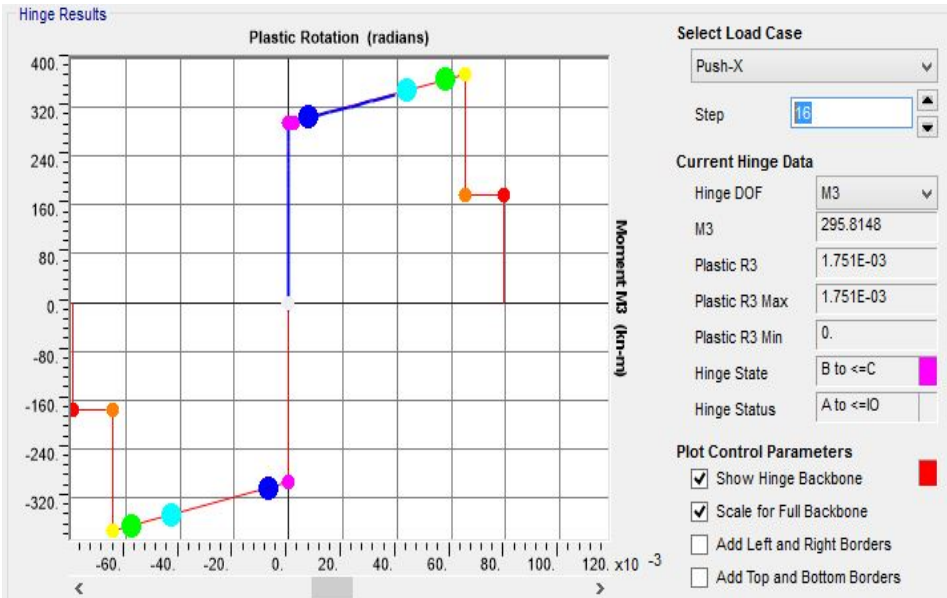
At Step - 1



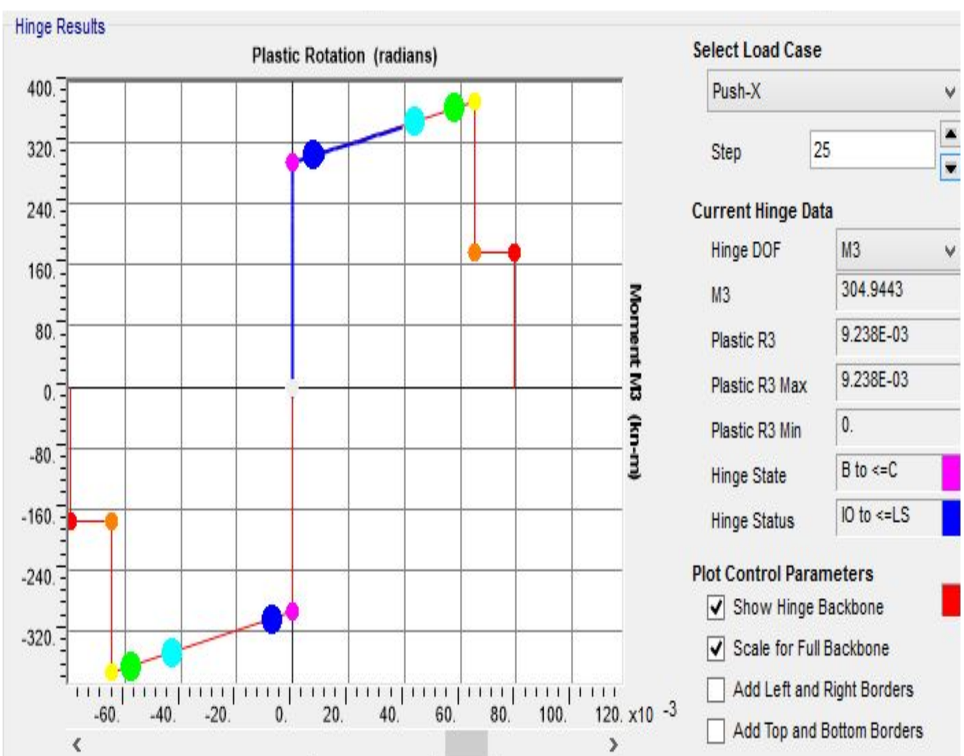
At Step - 9



At Step -16



At Step - 25



At Step – 33 (Position of Collapse)

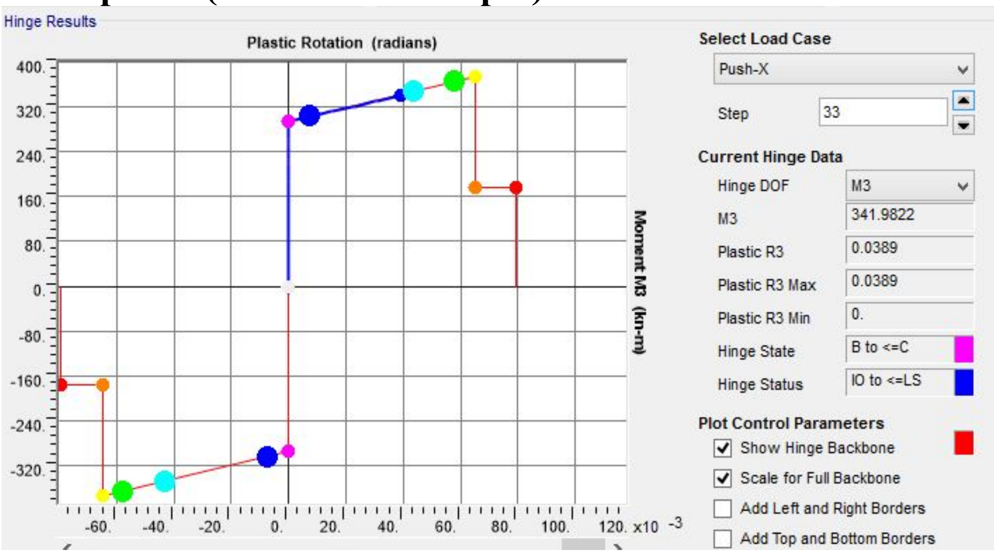


FIG 7.6 PATTERN OF HINGES AT DIFFERENT STAGES

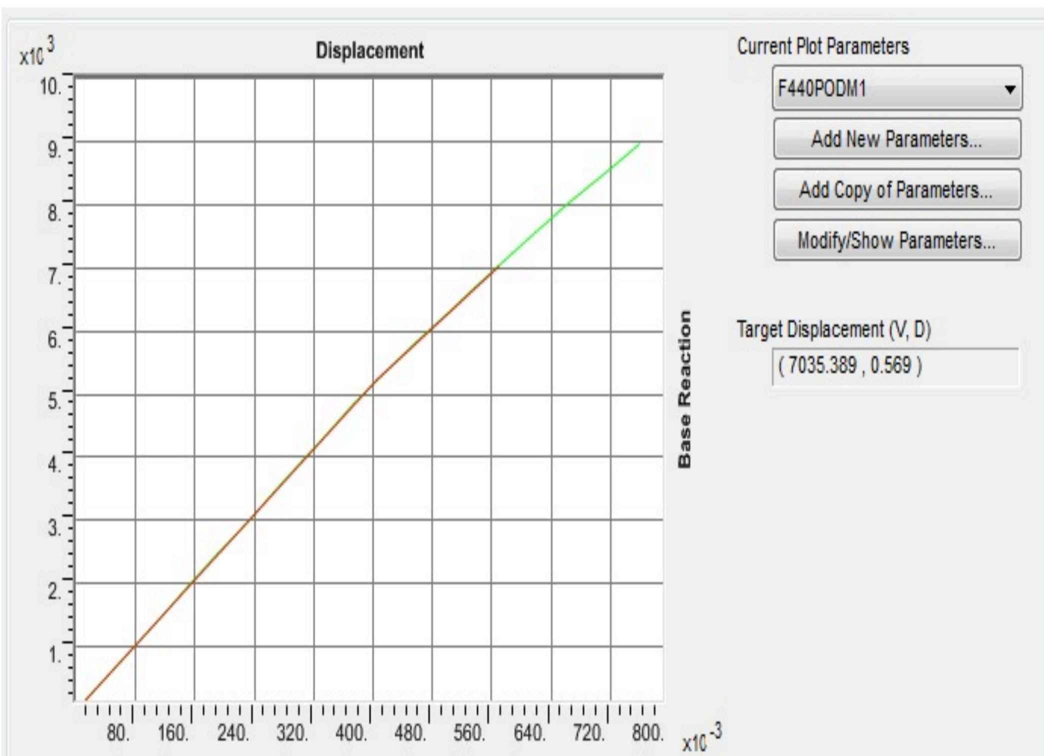


FIG .7.7 BASE SHARE VS DISPLACEMENT CURVE AT TARGET DISPLACEMENT 7035.389, 0.569

7.7 Ductility Demand

The ductility demands imposed on the frames at the various performance levels are found tabulated in table.

Table 7.4 Ductility Demand

S.No	**Displacement Level**	**Roof Displacement (m)**	**Ductility Demand**
1	δ_{yield}	0.396	1
2	Δ_{IO}	0.607	1.54
3	Δ_{LS}	1.24	3.13

7.8 Comparison of shear force and bending moment in linear static and non-linear static pushover analysis

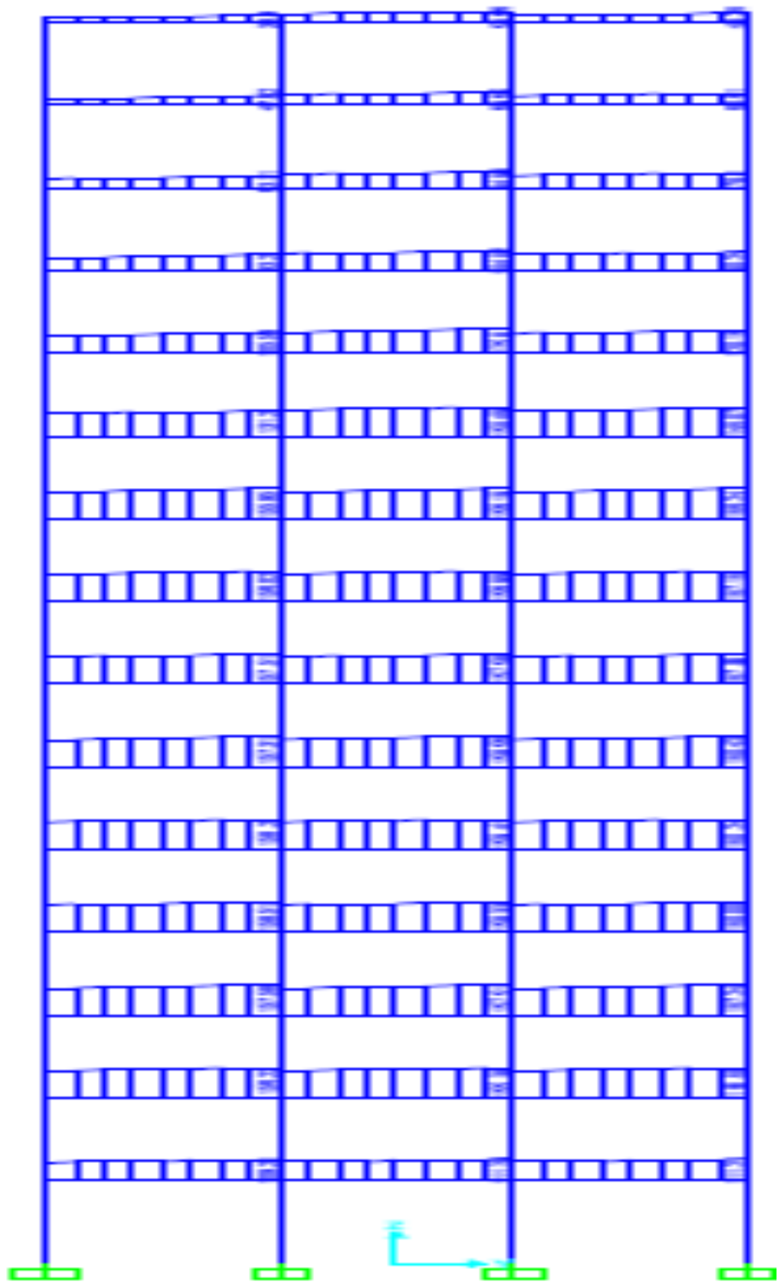


FIGURE 7.8: SHEAR FORCE FOR THE 15 STOREY STEEL FRAME BUILDING.

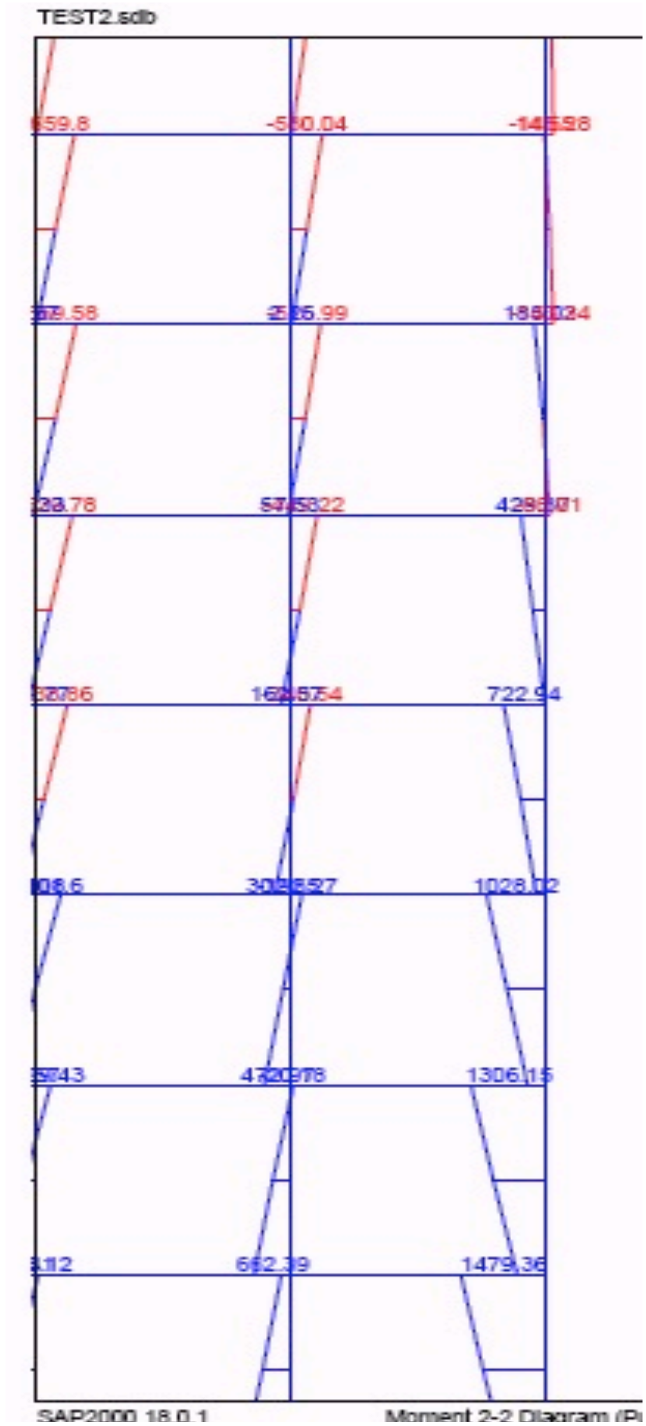


FIGURE7.9: BENDING MOMENT DIAGRAM FOR THE 15 STOREY STEEL FRAME BUILDING COLUMNS

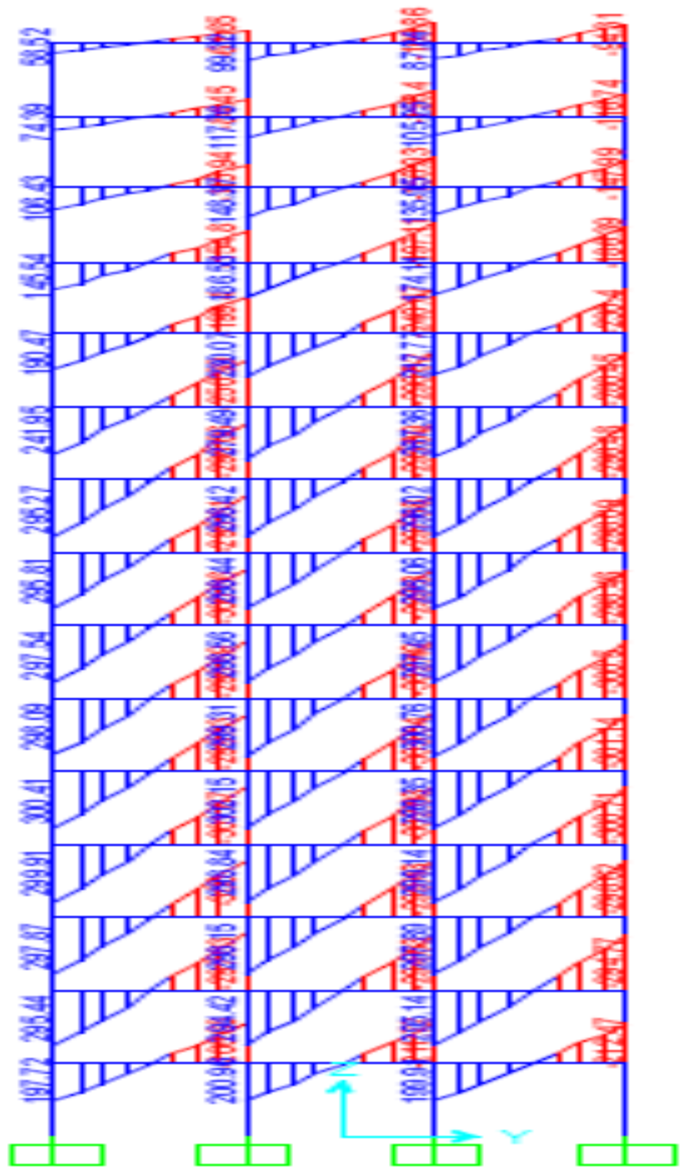


FIGURE7.10: BENDING MOMENT DIAGRAM FOR THE 15 STOREY STEEL FRAME BUILDING BEAMS

CHAPTER – 8

DISCUSSION OF RESULTS AND CONCLUSION

8.1 PUSHOVER ANALYSIS

The present work has been carried out to study pushover analysis on high rise steel building using SAP2000.

8.1.1 Effect on Base Shear

Comparing the base shear for the static linear and static nonlinear .

In 15 storey steel frame the base shear for the pushover analysis is 1.54 times the elastic base shear.

8.1.2 Effect on capacity curve

Results obtained for the present work is carried out using SAP2000.
Fig shows the combined results for the capacity curve in which the base shear increases significantly in the elastic range for the very small increase in displacement and then displacement increases significantly as compared to base shear. This shows the structure has good resistance against expected imposed seismic loads and the curve for the cases follow the same pattern.

8.1.3 Variation of roof top displacement

Table : 8.1 Variation of roof top displacement

Frame	Displacement (m)	
	Base	Top
15 Storey	0	0.488

8.1.4 Effect on Demand Curve

Fig shows the variation of capacity and demand curve the three cases.

The figure shows the structure has a good resistance against imposed earthquake as the demand curve intersects the capacity envelope near the elastic range which shows good structural resistance.

From figure it can concluded that the structure will behave safely during the imposed seismic excitation and need not to be retrofitted.

8.1.5 Sequential formation of plastic hinges

Sequential formation of plastic hinges gives us the failure pattern or sequence of column/beam failure. This is valuable information in the dynamic analysis and designing the structure. Hence we need to strengthen only selected member and not all the members of the same storey.

8.1.6 Effect on shear Force

Comparing the shear force for the static linear and static nonlinear analysis

In 15 storey frame there is 156.76% increase in shear force in non linear analysis as compared to linear static.

8.1.7 Effect on bending moment

Comparing the shear force for the static linear and static nonlinear analysis

In 15 storeys frame there is 256.64% increase in Bending Moment in non-linear analysis as compared to linear static.

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