PERFORMANCE BASED DESIGN AND OPTIMIZATION OF MULTISTORIED STRUCTURE

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

MASTER OF ENGINEERING

(STRUCTURAL ENGINEERING)

Submitted By

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CERTIFICATE

This is to be certify that the project entitle "**PERFORMANCE BASED DESIGN AND OPTIMIZATION OF MULTISTORIED STRUCTURE** " 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 RCC 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 RCC 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 RCC 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, the non-linear response of RCC frame using SAP2000 under the loading has been carried out with the intention to investigate the relative importance of several factors in the non-linear analysis of RCC frames. This in load include the variation displacement graph.

LIST OF ABBREVIATION

ENGLISH SYMBOLS

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

ED- the energy dissipated in the inelastic

ES- maximum strain energy

μ- displacement ductility ratio

α - Ratio of average post-elastic stiffness of capacity curve to effective elastic stiffness of the capacity curve

T_e- effective fundamental period

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

N - Total number of stories

Δ - Additional earthquake load added

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1.1 GENERAL

Earthquakes (among all natural Hazard) have the potential for inflicting the maximum damages. Since earthquake forces are unsystematic in nature & irregular, the engineering tools needs to be most proficient for analyzing structures underneath the action of these forces. Performance based design is gaining a replacement dimension within the seismic design idea wherein the near field ground motion (usually acceleration) is to be considered. Earthquake forces are to be carefully analyzed so as to assess the real behavior of structure with a apparent understanding that harm is expected but it should be synchronized. In this context pushover analysis which is an iterative procedure shall be looked upon as an alternative for the orthodox analysis procedures. This study focuses on pushover analysis of multistory RC framed buildings subjecting them to monotonically mounting lateral forces with an invariant height wise allocation until the predetermined performance level (target displacement) is reached. The promise of performance-based seismic engineering (PBEE) is to produce structures with predictable seismic performance. To turn these undertake into a authenticity, a broad and well-coordinated effort by professionals from various disciplines is required.

Performance based engineering is not new. Vehicles, airplanes, and turbines have been designed and manufactured using this method for many past years. Generally in such applications one or more full-scale prototypes of the structure are built and subjected to extensive testing. The design and manufacturing process is then updated to incorporate the lessons learned from the experimental evaluations. Once the sequence of design, prototype manufacturing, testing and redesign is successfully completed, the product is manufactured in a massive scale. In the automotive industry, for example, millions of Vehicles which are virtually identical in their mechanical features are produced following each performance-based design exercise.

What makes performance-based earthquake engineering (PBEE) special and more complicated is that in general this enormous payoff of performance-based design is not available. That is, except for large-scale developments of similar buildings, each building designed by this process is virtually unique and the experience obtained is not directly transferable to building structures of other types, sizes, and performance objectives. Therefore, up to now PBEE has not been an economically feasible option to traditional prescriptive code design practices. Due to the recent enhancements in seismic hazard evaluation, PBSE approaches, experimental facilities, and computer applications, PBEE has become increasing more striking to developers and engineers of buildings and structures in seismic regions. It is safe to say that within just a few coming years PBEE will become the standard process for design and delivery of earthquake resistant structures. In order to utilize PBEE effectively and smartly, one need to be attentive of the uncertainties implicated in both structural performance and seismic hazard calculations.

The recent initiation of performance based design has brought the nonlinear static pushover analysis method to the forefront. Pushover analysis is a static, nonlinear process in which the extent of the structural loading is incrementally increased in accordance with a certain preset pattern. With the increase in the amount of the loading, weak zones and failure modes of the structure are identified. The loading is monotonic with the effects of the cyclic activities and loading reversals being calculated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an effort by the structural engineering profession to assess the real strength of the structure and it promises to be a useful and effective tool for performance based earthquake design.

1.2 NECESSITATE OF PERFORMANCE BASED SEISMIC DESIGN

From the effects of significant earthquakes (since the early 1980s) it is assessed that the seismic risks in metropolitan areas are increasing and are far from socio-economically satisfactory levels. There is an urgent need to reverse this situation and it is believed that one of the most effective ways of doing this is through: the development of more reliable seismic standards and code provisions than those currently available and their stringent implementation for the complete engineering of new engineering facilities.

A performance-based design is intended at controlling the structural damage or disaster based on precise estimations of proper response parameters. This is possible if more accurate analyses are carried out, including all potential important factors involved in the structural behavior .With an importance on providing stakeholders the information required to make rational business or safety-related decisions, practice has moved toward predictive methods for assessing potential seismic behavior and has led to the development of performance based engineering methods for seismic design.

1.3 HISTORY

Performance-based design of buildings has been adopted since early in the twentieth century, England, New Zealand, and Australia had performance-based building guidelines/codes in place since decades. The International Code Council (ICC) in the U.S had a performance code available for voluntary adoption since 2001 (ICC, 2001). The Inter-Jurisdictional Regulatory Collaboration Committee (IRCC) is an international group representing the lead building regulatory organizations of 10 countries created to facilitate international conversation of performance-based regulatory systems with a focus on identifying public policies, regulatory network, education, and technology issues related to implementing and managing these systems.

In 1989, the FEMA-funded project was launched to develop official engineering guidelines for retrofit of existing buildings began (ATC, 1989), it was recommended that the rules and guidelines be sufficiently flexible to accommodate a much wider variety of local or even building-specific seismic risk reduction policies than has been traditional for new building construction. The initial design document, NEHRP *Guidelines for the Seismic Rehabilitation of Existing Buildings*, FEMA 273, therefore contained a range of formal performance objectives that corresponded to specified levels of seismic shaking. The performance levels were idealized with descriptions of overall damage states with titles of Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. These levels were projected to identify limiting performance states important to a broad range of stakeholders by measuring: the ability to use the building after the event; the traditional protection of life safety provided by building codes; and, in the worst case, the prevention of collapse. Following the Northridge event, the Structural Engineers Association of California (SEAOC, 1995) developed a PBSD process, known as Vision 2000 , which was more generalized than that contained in FEMA 273 but used similarly defined performance objectives.

Over the period after publication of FEMA 273, its procedures were reviewed and refined and eventually printed in 2006 as an American Society of Civil Engineers (ASCE) national standard - *Seismic Rehabilitation of Existing Buildings*, ASCE 41. Although intended for rehabilitation of existing buildings, the performance objectives and accompanying technical data in ASCE 41 responded to the general interest in PBSD and have been used for the design of new buildings to achieve higher or more reliable performance objectives than perceived available from prescriptive code provisions. ASCE 41 is taken into account to represent the initial generation of Performance-based Seismic design procedures.

1.4 PERFORMANCE BASED SESMIC ENGINEERING (PBSE)

Performance based seismic engineering implies design, evaluation, construction, monitoring the function and maintenance of engineered facilities whose performance under earthquake forces responds to the various needs and objectives of owners users and society. It is based on the basis that performance can be predicted and evaluated with quantifiable confidence to make, together with the client, intelligent and informed trade-offs based on life-cycle considerations rather than construction cost alone .

PBSE is a popular concept whose implementation has a long way to go. There are legal and professional barriers but there are also many questions whether PBSE will be able to full-fill its promises. It assures engineered structures whose performance can be estimated and confirmed to the owner‘s desires. PBSE implies, for example, accepting damage in seismic events, if that proves the most economic solution. This requires, however, that structural engineers be able to calculate these damages and there likelihood so as to make informed decisions. Implementation of such a design decision process necessitates a shift away from the dependence on empirical and experience-based conventions, and toward a design and assessment process more firmly rooted in the realistic prediction of structural behavior under a realistic description of the spectrum of loading environments that the structure will experience in the future. This implies a shift toward a more scientifically oriented design and evaluation approach with emphasis on more accurate classification and predictions, often based on a higher level of technology than has been used in the past.

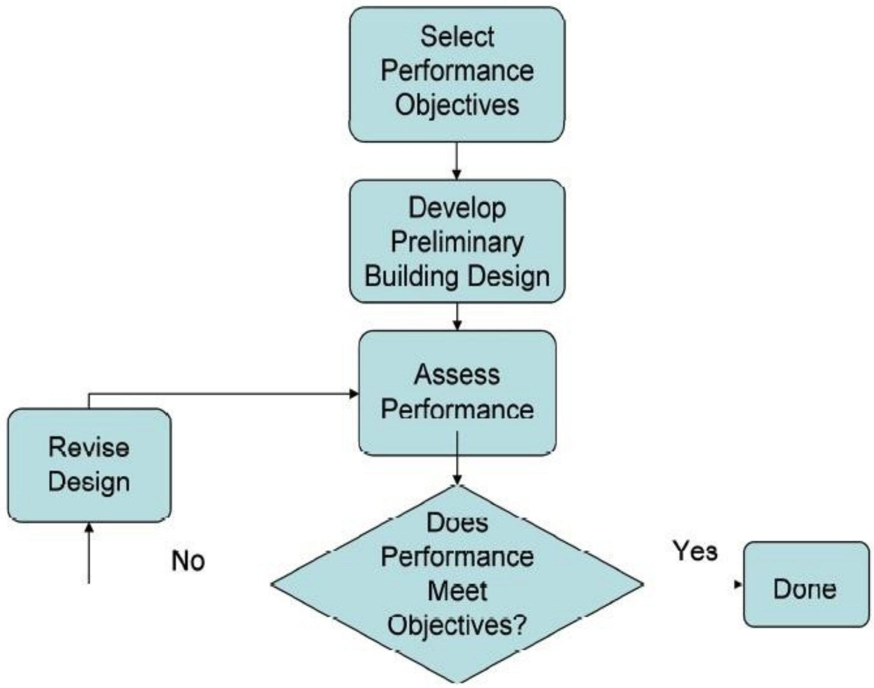


Figure 1.1 Performance-based design flow diagram

PBSE permits design of new buildings / structures or upgrade of existing structures (buildings) with a realistic understanding of the risk of losses, occupancy disturbance, and economic loss that may occur as a result of future earthquakes.

The goal of Performance-based Earthquake design is to ensure that performance objectives are satisfied. That is, the structure will perform in a specific manner under various intensity of earthquake loading. According to the structure of performance-based design (SEAOC 2000, single or multiple performance objectives are selected at first according to seismic design code and the requirement of the owner in the conceptual design stage. When adopting a direct displacement-based method, displacement parameters such as the top displacement/sway or inter-story drift ratio of a building, the plastic rotation of the hinge at the base of a column, displacement ductility ratio etc. can be employed to describe the target performance. Acceptable limits of these parameters regarding each level of seismic hazard corresponding to each performance objective are quantified or estimated. In the conceptual design stage, layout of the structure is then determined without numerical analysis. Conceptual design guide and energy balanced equation may be useful for engineering assessment. A successful conceptual design could hopefully reduce the impact of uncertainties on the real structural behavior of any building or structures. After the conceptual design phase is completed, the numerical design phase is proceeded to determine the structural detailing, which satisfy the pre-quantified performance objectives. Preliminary design can be conducted through two different approaches:

(1)Traditional force-based design approach followed by the check of performance objectives and

(2)Direct design approach starting from the pre-quantified performance objectives.

The result assessed by the latter is believed to be closer to the final design and requires less computational attempt. Validation of performance objectives employing non-linear pushover or non-linear time-history analysis is finally conducted out to reach the final design. The performance objectives are satisfied if the calculated performance parameters do not exceed the acceptance limits.

Since the numerical stage of performance-based design is an continuous procedure between design and validation, in order to save data processing effort, it is suggested to select fewer performance objectives in the preliminary design and check all performance objectives in the final design. The decision as to how many and which performance goal need to be selected depends on if that performance goal is the main concern of the users and owners and if quantification of the performance acceptable limit is consistent.

1.5 ADVANTAGES OF PERFORMANCE-BASED SEISMIC / EARTHQUAKE DESIGN

In contrast to prescriptive design approaches and stages, performance-based design provides a methodical methodology for assessing the performance capability of a building structure. It can be used to verify the equivalent performance of alternatives and options, deliver standard performance at a reduced economical cost, or confirm higher performance needed for critical facilities.

It also establishes a vocabulary that facilitates meaningful discussion between stakeholders and design professionals on the development and selection of design options. It provides a framework for determining what level of safety and what level of property protection, at what cost, are acceptable to stakeholders based upon the specific needs of a project.

Performance-based Earthquake design can be used to.

• Design individual buildings and relevant structure with a higher level of confidence that the performance intended by present building codes will be achieved.

• Design individual building structures that are capable of meeting the performance intended by present building codes and guidelines, but with lower construction costs.

• Design individual building structures to achieve higher performance (and lower potential losses) than intended by present building codes and guidelines.

• Assess the potential seismic performance of existing buildings and structures and estimate potential losses in the event of a seismic conditions.

• Assess the potential performance of current prescriptive codal requirements for new building structures, and serve as the basis for improvements to code-based seismic design criteria so that future buildings can perform more consistently and reliably.

Performance-based Earthquake design and engineering offers society the potential to be both more efficient and effective in the investment of financial resources to avoid future earthquake losses and damages. Further, the technology used to implement Performance-based Earthquake design and engineering is transferable, and can be adapted for use in performance-based design for other extreme hazards including fire, wind, flood, snow, blast, and terrorist attack.

The advantages of PBED or PBEE over the methodologies used in the current seismic design code are summarized as the following six key issues :

1. Multi-level seismic hazards are considered with an emphasis on the transparency of performance objectives.

2. Building and structure performance is guaranteed through limited inelastic deformation in addition to strength and ductility.

3. Seismic design criteria is oriented by performance objectives interpreted by engineering parameters as performance criteria.

4. An analytical computational method through which the structural behavior, particularly the nonlinear behavior is rationally obtained and assessed.

5. The building structure will meet the prescribed performance objectives reliably with accepted confidence.

6. The design will ensure the minimum life-cycle cost (economical design).

1.6 NEED AND OBJECTIVES OF THE PRESENT STUDY

1.6.1 Need

The Kutch Earthquake of January 26, 2001 in Gujarat, India, caused the massive destruction and of a large number of modern 4 to 10-storied buildings. After this earthquake, doubts raised about our professional practices, building by-laws and guidelines, construction materials, building codes and education for civil engineers and architects. It led to revision of the seismic code and their relevant guidelines and initiation of a National Programme on Earthquake Engineering Education (NPEEE).

The present seismic standards in India promote the construction of seismically most vulnerable constructions in highly seismic areas of the country. Better seismic standards are urgently needed in the new global economic setup and a working draft can be easily prepared by learning from ATC and FEMA documents developed in USA.

1.6.2 Objectives

The primary objective of this work is to study the seismic response of RC framed building using performance based seismic engineering. The effect of earthquake force on multi level building, having height (G+19) with the help of Non-linear pushover analysis, for various different sets of reinforcement at different levels has been investigated.

The main objectives of undertaking the present study are as follows:

1. To design a multi-storied RC wall framed building using ETABS and analyzing the same using Non linear pushover analysis procedure, using SAP2000, for ascertaining the seismic load carrying capacity of that structure.
2. To study the effect of change of reinforcement in Columns , walls and Beams of RC wall framed building at different level levels (in elevation), using nonlinear pushover analysis.
3. To study the effect of change of reinforcement in different Columns of RC wall framed building (in plan), using pushover analysis.
4. To study the effect of providing shear walls, in RC framed building, using pushover analysis.

5. To compare the seismic response of building in terms of base shear, level drift, spectral acceleration, spectral displacement and level displacements.
6. Determination of performance point of building.
7. To determine the best possible combination of reinforcement that would be both economical and effective. The resultant roof displacement is then compared with target displacement. If it is lower then, the design is known as performance based design.
8. To compare the resultant design with code based design.

1.7 SCOPE OF THE PRESENT STUDY

The scope of present study aims at Performance evaluation of R.C buildings (designed according to IS 456:2000) using Pushover Analysis and redesigning by changing the main reinforcement of various frame elements and again analyzing. The performance based seismic engineering technique known has Non-Linear Static Pushover analysis procedure has been effectively used in this regard. The pushover analysis has been carried out using SAP2000, a product of Computers and Structures International. Various cases for a particular four level building located in Zone-IV have been analyzed, changing reinforcement of different structural elements, i.e. Beams and Columns, in different combinations as well as at different level levels.

The results of analysis are compared in terms of base shear, level drift, spectral acceleration, spectral displacement and level displacements. Determine the best possible combination of reinforcement that would be both economical, effective and damage must be limited to Grade 2 (slight structural damage, moderate nonstructural damage) in order to enable Immediate Occupancy.

Optimal design is analyzed and damage must be limited to Grade 3 (moderate structural damage, heavy nonstructural damage) in order to ensure Life Safety under MCE. Finally, it is compared with code based seimic-resistant design The effect of providing shear walls, on the performance of RC framed building, is also studied using pushover analysis.

In chapter 4 of this study, the above formed methodology is used to design an four storied reinforced concrete frame building situated in Zone IV.

2.1 LITERATURE REVIEW

Qiang Xue, Chia-Wei Wu et al (2007) summarized the development of the seismic design draft code for buildings in Taiwan using Performance-based Earthquake design methodology and case studied following the guidelines in the paper. They presented the design of a reinforced concrete building by using the draft code [37].

In their study first, the current seismic design code provisions are examined according to the theoretical basis of PBED to identify which methodologies of PBED need to be incorporated into the current seismic design code. Then, a PBED flowchart is presented. Finally, a draft of the proposed code is described.

According to the case study, if the same column size has been adopted for the first several floors, a higher reinforcement ratio assigned to the first 2 stories is helpful for uniform distribution of system ductility. Adopting the performance criteria in the draft code, direct displacement-based design procedures have been applied successfully for moment resisting frames without iteration. The performance criteria associated with stiffness or displacement as suggested in the draft code should not be used either as optimized design criteria or in a direct displacement-based design procedure for structural systems other than moment resisting frames.

In this draft code, the design of nonstructural components is done to accommodate either acceleration or displacement. No specific criterion regarding economic loss is provided. The nonstructural damage is limited by the structural drift limit.

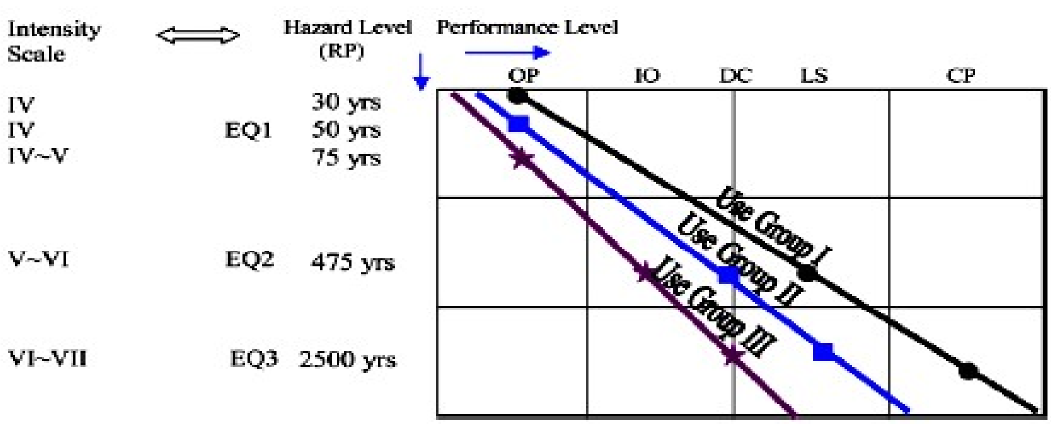


Figure 2.1 Performance objectives

As shown in Fig. 2.1, three seismic hazard levels were considered and can be distinguished by return period, probability of exceedance, or corresponding site intensity scale. Performance of a building has been classified into 5 levels, Operational (OP), Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS) and Collapse Prevention (CP).

Andreas J. Kappos et al (2004) proposed a performance-based design procedure for realistic 3D reinforced concrete (R/C) buildings, which involves the use of advanced analytical tools. The proposed method was then applied to a regular multistory reinforced concrete 3D frame building and was found to lead to better seismic performance than the standard code (Eurocode 8) procedure, and in addition led to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

The building was first designed to a standard code procedure, and then redesigned to the proposed method. Due to its high regularity, the building was designed using both versions of the method (based on either inelastic dynamic or inelastic static analysis). In addition, several alternative designs to the new method were carried out. All designs were subsequently assessed for a number of performance objectives, using both local and global criteria.

A six-level R/C, doubly symmetric structure (three 3 m spans in *y*-direction, three spans of 6-4-6 m in *x*-direction) was selected as a test of the proposed procedure. The building was first designed to the provisions of the current Greek Seismic Code, which is very similar to Eurocode 8 (CEN, 1995) [9] – ductility class ―M‖ (medium), for a design ground acceleration of 0.25*g*, assuming class A soil conditions (stiff deposits). Earthquake loading was combined with gravity loading G + 0.3 LL. The materials used in the structure are C20/25 (characteristic cylinder strength of 20 MPa) concrete, and S500 steel (characteristic yield strength of 500 MPa). Square column cross-sections (from 350 to 350 mm) were used, with reinforcement ratios not exceeding about 2% (the minimum reinforcement ratio for columns was 1%). Beam sections varied from 300×400 to 400×400 (mm^2).

For the pushover analysis, the ―triangular‖, code-type, distribution of lateral loading and a ‗modal' pattern, defined by the forces acting on the mass centres of each floor when the building is subjected to the response spectrum acting along each main axis, were tried. Modal forces were calculated taking into account the first three modes in each principal direction, whose modal masses contribute about 95% of the total.

In order to explore the various aspects of the proposed method and test the effect of some key design parameters, it was decided to carry out alternative designs of the same structure, resulting not only from different type of analysis (static or dynamic), but also from different ‗strength' of plastic hinge zones. The flexural design of plastic hinge zones was carried out accepting either ―usual‖ or ―high‖ serviceability requirements; in the first case the v_0 factor was taken as 2/3 and the serviceability earthquake as 1/2.5 the code spectrum (the lower value suggested in the previous section), while in the second case, the v_0 factor was taken as 3/4 and the serviceability earthquake as 1/2 the code spectrum.

The proposed procedure resulted, in an increase in ‗longitudinal' reinforcement of columns, at the lower levels. This increase was more significant (about 20% in the usual serviceability case and 40% in the high serviceability case, compared to Code design) when the design was carried out using time-history analyses; increases of only 8% to 25% were found when inelastic static analysis was used. On the contrary, the ‗transverse' reinforcement was significantly reduced (from 17% to 23%). A complete picture of the reinforcement requirements in each alternative design can be obtained from Fig. 2.2.

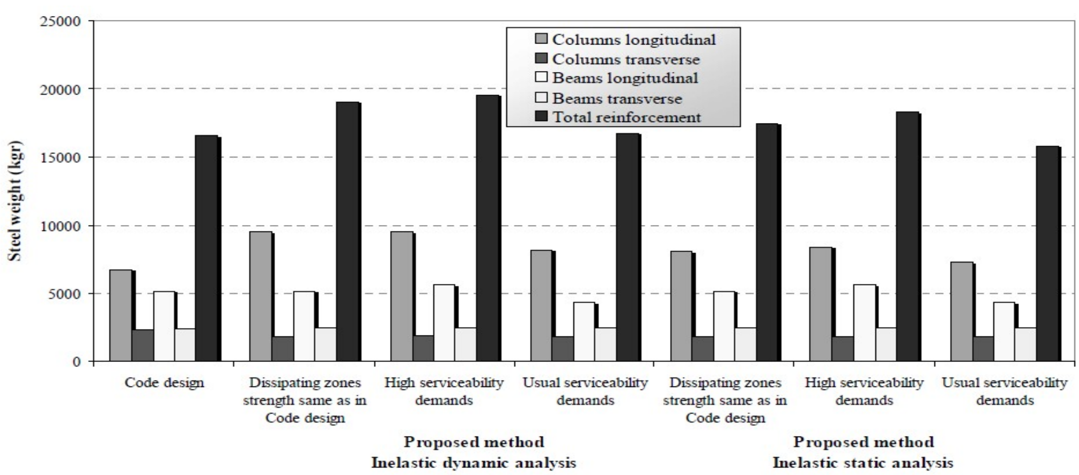


Figure 2.2 Required amount of steel in beams and columns, for all designs

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

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

1. Establish an initial design with optimal member dimensions, which can be obtained from the elastic seismic design optimization by minimizing the concrete cost of an RC structure subjected to a minor earthquake loading using the elastic response spectrum analysis method.
2. Determine the design spectra, corresponding to different earthquake demand levels, which will be used in the nonlinear pushover analysis.
3. Conduct a static virtual load analysis to obtain the member internal forces that will be used in formulating inelastic drift responses by employing the principle of virtual work.
4. On the basis of the optimal member size, determine the minimum and maximum size bounds of the steel reinforcement ratios, *pi* and *pi'*, in accordance with the strength-based code requirements.

5. Apply the initial preprocessor on the basis of a representative single drift constraint to establish a reasonable starting set of steel reinforcement design variables for the multiple drift constrained optimization.
6. Carry out the nonlinear pushover analysis using commercially available software such as the SAP2000 software to determine the performance point of the structure and the associated inelastic drift responses of the structure at the performance point.
7. Track down the locations of the plastic hinges, establish the instantaneous lower and upper bound move limits of *pi* for those members with plastic hinges and determine the values of the first-order and second order derivatives of the drift responses.
8. Establish the explicit inter story drift constraints using a second-order Taylor series approximation and formulate the explicit design problem.
9. Apply the recursive Optimality Criteria optimization algorithm to resize all steel reinforcement design variables and to identify the active inelastic drift constraints.
10. Check convergence of the steel cost and the inelastic drift performance of the structure. Terminate with the optimum design if the solution convergence is found; otherwise, return to Step 6.

It has been demonstrated that steel reinforcement plays a significant role in controlling the lateral drift beyond first yielding and in providing ductility to an RC building framework. Using the principle of virtual work and the Taylor series approximation, the inelastic Performance-based Earthquake design problem has been explicitly expressed in terms of the steel reinforcement design variables. Axial moment hinges and moment hinges should be considered in the nonlinear pushover analysis of a frame structure so that the behavior of columns and beams can be effectively modelled. Also, this Optimality Criteria design method developed is able to automatically shift any initial performance point to achieve the final optimal performance point. It is also believed that this optimization methodology provides a powerful computer-based technique for performance-based design of multistory RC building structures

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

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

J. B. Mander (2001) reviewed from an historical perspective past and current developments in earthquake engineered structures. Based on the present state-of-the-practice in New Zealand, and a world-view of the state-of-the-art, he argued that in order to make progress towards the building of seismic resilient communities, research and development activities should focus on performance-based design which gives the engineer the ability to inform clients/owners of the expected degree of damage to enable a better management of seismic risk. To achieve expected performance outcomes it will be necessary to supplement, current force-based design standards with displacement-based design methodologies.

Improved design methodologies alone will not lead to a significantly superior level of seismic resilient communities, but rather lead to a superior standard of performance-based engineered structures where the post-earthquake outcome will be known with a certain degree of confidence. This paper gives two philosophical approaches that are referred to as Control and Repairability of Damage (CARD), and Damage Avoidance Design (DAD).

Peter Fajfar et al (2000) presented a relatively simple nonlinear method for the seismic analysis of structures (the N2 method). It combines the pushover analysis of a multi-degree-of-freedom (MDOF) model with the response spectrum analysis of an equivalent single-degree-of-freedom (SDOF) system. The method is formulated in the acceleration- displacement format, which enables the visual interpretation of the procedure and of the relations between the basic quantities controlling the seismic response. Inelastic spectra, rather than elastic spectra with equivalent damping and period, were applied. This feature represents the major difference with respect to the capacity spectrum method. Moreover, demand quantities can be obtained without iteration. Generally, the results of the N2 method are reasonably accurate, provided that the structure oscillates predominantly in the first mode. In the work, the method is described and discussed, its basic derivatives are given. The similarities and differences between the proposed method and the FEMA 273 and ATC 40 nonlinear static analysis procedures are discussed. Application of the method is illustrated by means of an example.

In general, the results obtained using the N2 method are reasonably accurate, provided that the structure oscillates predominantly in the first mode. Applications of the method are, for the time being, restricted to the planer analysis of structures.

Vipul Prakash (2004) gives the prospects for Performance Based Engineering (PBE) in India. He lists the pre-requisites that made the emergence of PBE possible in California, compares the situation in India and discusses the tasks and difficulties for implementing PBE in India.

In India, the criteria for earthquake resistant design of structures are given in IS 1893, published by the Bureau of Indian Standards (BIS). IS 1893-2002 reduced the number of seismic zones to four by merging zone I with zone II and adopted a modified CIS-64 scale for seismic zoning and dropped references to the MMI scale. The mapping of zones to intensities in IS 1893-2002 is given in Table 2.1.

Table 2.1 Mapping Seismic Zones to Intensities in IS 1893-2002

In IS 1893-2002	
Seismic Zone	**Mapped to a Modified CIS-64 Scale**
II	VI and below
III	VII
IV	VIII
V	IX and above

In US, building performance levels are divided into structural performance levels (SP-1 to SP6) and nonstructural performance levels (NP-A to NP-E), and then a combination of structural and nonstructural performance levels is set as the performance objective to be met at a given level of earthquake. These combinations can be approximately mapped to the damage grades specified in EMS-98 as follows:

Table 2.2 Comparison of Damage Grades as per EMS-98 and Building Performance Levels

Damage Grade as per EMS-98	Approximate Building Performance Combination in PBE
Grade 1 (no structural damage, slight nonstructural damage)	SP-1 (immediate occupancy) + NP-A (operational) = 1-A (operational)
Grade 2 (slight structural damage, moderate nonstructural damage)	SP-1 (immediate occupancy) + NP-B (immediate occupancy) = 1-B (immediate occupancy)
Grade 3 (moderate structural damage, heavy nonstructural damage)	SP-3 (life safety) + NP-C (life safety) = 3-C (life safety)
Grade 4 (heavy structural damage, very heavy nonstructural damage)	SP-5 (structural stability) + NP-E (not considered) = 5-E (structural stability)
Grade 5 (very heavy structural damage)	SP-6 (not considered) + NP-E (not considered) = 6-E (not considered)

IS 1893- 2002 specifies two levels of earthquakes – Maximum Considered Earthquake (MCE) and Design Basis Earthquake (DBE). In Clause 6.1.3, it states the performance objective as follows: ―The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (< DBE), which occur frequently, without damage; resist moderate earthquake (DBE) without significant structural damage though some nonstructural damage may occur; and aims that structures withstand a major earthquake (MCE) without collapse.‖

In PBE, merely stating a performance objective is not sufficient; it has to be followed up by analyses or a methodology for ensuring that the stated performance objectives will indeed be met by the evaluated structures. PBE thus requires much tighter language and cross-referencing to be used in the specifications.

The following two-level performance objective is suggested for new ordinary structures.

· Under DBE, damage must be limited to Grade 2 (slight structural damage, moderate nonstructural damage) in order to enable Immediate Occupancy after DBE.

Under MCE, damage must be limited to Grade 3 (moderate structural damage, heavy nonstructural damage) in order to ensure Life Safety after MCE.

3.1 PERFORMANCE-BASED SEISMIC DESIGN PROCESS

As described earlier, performance-based design is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

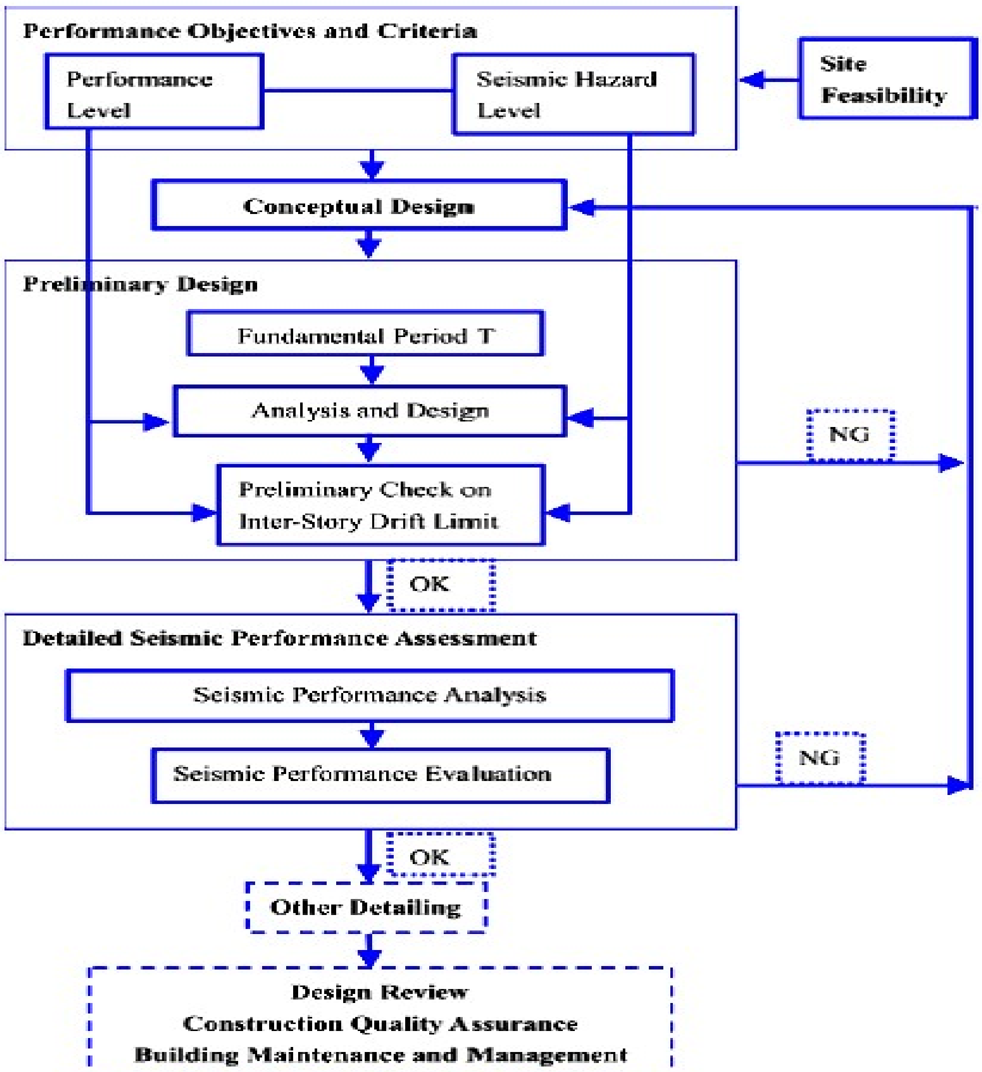


Figure 3.1 Performance Based Seismic Design for New Buildings [37]

3.1.1 Select Performance Objectives

The process begins with the selection of design criteria stated in the form of one or more performance objectives. Performance objectives are statements of the acceptable risk of incurring different levels of damage and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. Since losses can be associated with structural damage, nonstructural damage, or both, performance objectives must be expressed considering the potential performance of both structural and nonstructural systems.

These are based largely on the building stakeholders, namely, the building owner. It is these stakeholders that will determine the initial cost investment in design and construction, and this will drive the level of performance and the associated consequences. PBD requires more effort in the early phases of design.

In the next-generation performance-based design procedures, performance objectives are statements of the acceptable risk of incurring casualties, direct economic loss (repair costs), and occupancy interruption time (downtime) associated with repair or replacement of damaged structural and nonstructural building elements, at a specified level of seismic hazard. These performance objectives can be stated in three different risk formats:

An ***intensity-based performance objective*** is a quantification of the acceptable level of loss, given that a specific intensity of ground shaking is experienced. An example of an *intensity-based performance objective* is a statement that if ground shaking with a 475-year-mean-recurrence intensity occurs, repair cost should not exceed 20 percent of the building's replacement value, there should be no life loss or significant injury, and occupancy interruption should not exceed 30 days.

A ***scenario-based performance objective*** is a quantification of the acceptable level of loss, given that a specific earthquake event occurs. An example of a scenario-based performance objective is a statement that if a magnitude-7.0 earthquake occurs, repair costs should not exceed 5% of the building replacement cost, there should be no life loss or significant injury, and occupancy of the building should not be interrupted for more than a week.

A ***time-based performance objective*** is a quantification of the acceptable probability over a period of time that a given level of loss will be experienced or exceeded, considering all of the earthquakes that might affect the building in that time period and the probability of occurrence of each. An example of a time-based performance objective is a statement that there should be less than a 2 percent chance in 50 years that life loss will occur in the building due to earthquake damage, on the average the annual earthquake damage repair costs for the building should not exceed 1% of the replacement cost, and the mean return period for occupancy interruption exceeding one day should be 100 years.

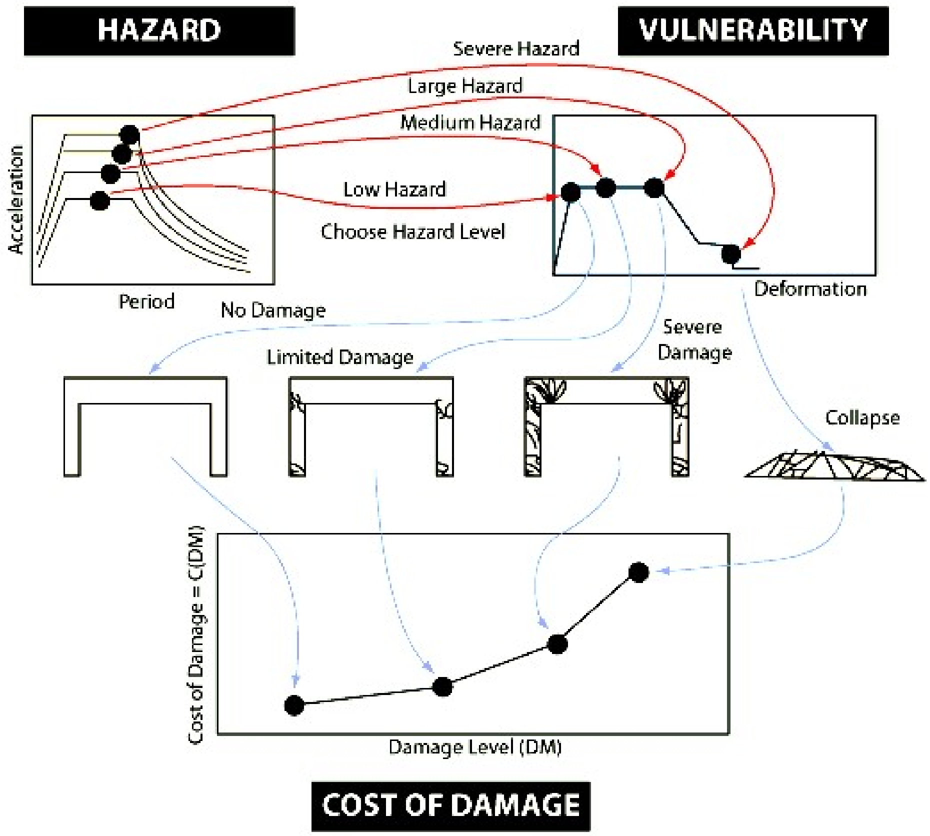


Figure 3.2 Performance Based Design Steps [34]

3.1.2 Develop Preliminary Building Design

The preliminary design for a structure includes definition of a number of important building attributes that can significantly affect the performance capability of the building. These attributes include:

• Location and nature of the site.

• Building configuration, including the number of stories, story height, floor plate arrangement at each story, and the presence of irregularities.

• Basic structural system, for example, steel moment frame or masonry bearing walls.

• Presence of any protective technologies, for example, seismic isolators, energy dissipation devices, or damage-resistant elements.

• Approximate size and location of various structural and nonstructural components and systems, and specification of the manner in which they are installed.

Selection of an appropriate preliminary design concept is important for effectively and efficiently implementing the performance-based design process. Inappropriate preliminary designs could result in extensive iteration before an acceptable solution is found, or could result in solutions that do not efficiently meet the performance objectives.

At present, engineers have few resources on which to base a preliminary design for meeting a specified performance objective. Some may refer to current building code provisions, others might refer to first-generation performance-based design procedures, and still others might use a more intuitive approach.

3.1.3 Assess Performance

After the preliminary design has been developed, a series of simulations (analyses of building response to loading) are performed to assess the probable performance of the building. Performance assessment includes the following steps:

• Characterization of the ground shaking hazard.

• Analysis of the structure to determine its probable response and the intensity of shaking transmitted to supported nonstructural components as a function of ground shaking intensity. In the case of extreme loading, as would be imparted by a severe earthquake, simulations may be performed using nonlinear analysis techniques.

• Determination of the probable damage to the structure at various levels of response.

• Determination of the probable damage to nonstructural components as a function of structural and nonstructural response.

• Determination of the potential for casualty, capital and occupancy losses as a function of structural and nonstructural damage.

• Computation of the expected future losses as a function of intensity, structural and nonstructural response, and related damage.

Performance assessment is based on assumptions of a number of highly uncertain factors. These factors include:

• Quality of building construction and building condition at the time of the earthquake.

• Actual strength of the various materials, members, and their connections incorporated in the building.

• Nature of building occupancy at the time of the earthquake, the types of tenant improvements that will be present, how sensitive these tenant improvements might be to the effects of ground shaking, and the tolerance of the occupancy to operating in less than ideal conditions.

• Availability of designers and contractors to conduct repairs following the earthquake.

• Owner's efficiency in obtaining the necessary assistance to assess and repair damage.

To complete a performance assessment, statistical relationships between earthquake hazard, building response, damage, and then loss are required. In a general sense, the process involves the formation of four types of probability functions, respectively termed: hazard functions, response functions, damage functions, and loss functions, and mathematically manipulating these functions to assess probable losses.

Hazard functions are mathematical expressions of the probability that a building will experience ground shaking of different intensity levels, where intensity may be expressed in terms of peak ground acceleration, spectral response acceleration or similar parameters. Hazard functions can be derived from the U.S. Geological Survey (USGS) ground shaking hazard maps, or may be developed based on a site-specific study that considers the seismicity of various faults in the region and the response characteristics of the building site. This can range in complexity from choosing only the hazard level and the shape of the design spectra to a more involved process, such as generating an ensemble of seismic acceleration time histories. In most situations, the designer needs to address issues such as return period (the duration of a seismic event at a given level) and maximum ground acceleration. In the second generation seismic PBD effort, the probability of the chosen seismic hazard is an integral part of the design input needs. This is necessary to compute the anticipated consequences of the design, as shown in Figure 3.3. Another feature of second generation seismic PBD is that it can be based either on a single scenario, such as a unique earthquake level, or on multiple earthquake levels with varied return periods.

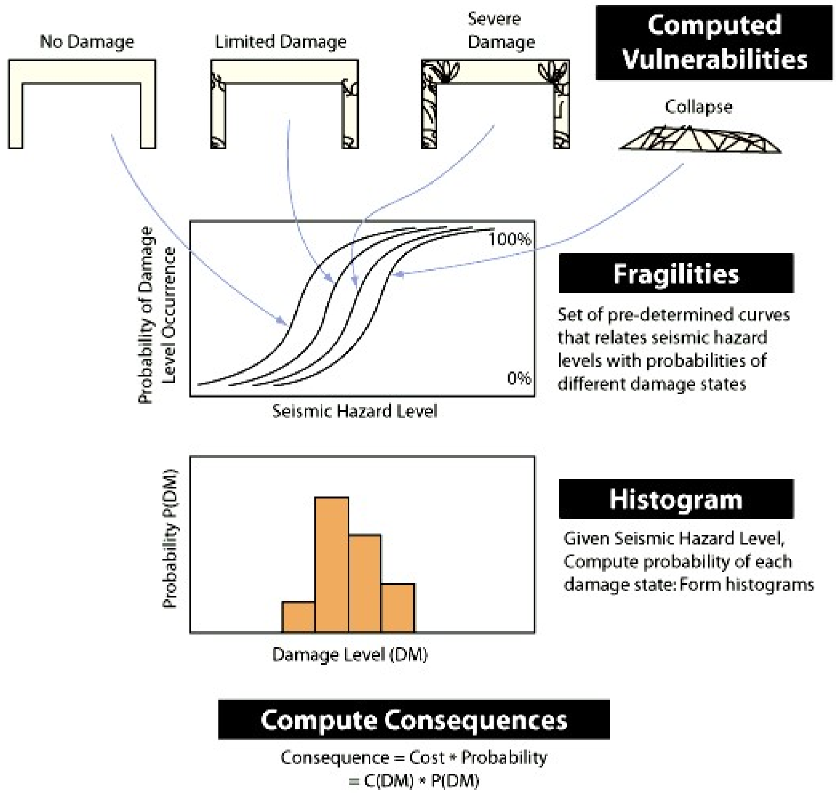


Figure 3.3 Computation of Risk [34]

Response functions are mathematical expressions of the conditional probability of incurring various levels of building response, given that different levels of ground shaking intensity are experienced. Building response is expressed in the form of parameters that are obtained from structural analysis, including story drifts, member forces, joint plastic rotation demands, floor accelerations and similar parameters. They are obtained by performing structural analysis of a building for different intensities of ground shaking.

Computing types, levels, and probabilities of structural or non-structural damage due to an earthquake are not easy tasks. This is one area which is currently undergoing extensive research and development. An emerging technique for relating earthquake damage to uncertain inputs and computing the damage uncertainties is the use of fragility curves. Figure 3.3 shows how fragilities are used in a PBD context. Component seismic fragilities have been under development for some time. Efficient, practical and general methods for system level fragility, on the other hand, are just starting to develop.

Damage functions are mathematical expressions of the conditional probability that the building as a whole, or individual structural and nonstructural components, will be damaged to different levels, given that different levels of building response occur. Damage functions are generally established by laboratory testing, analytical simulation or a combination of these approaches.

Loss functions are mathematical expressions of the conditional probability of incurring various losses, including casualties, repair and replacement costs, and occupancy interruption times, given that certain damage occurs. They are determined by postulating that different levels of building damage have occurred and estimating the potential for injury persons who may be present as well as the probable repair /restoration effort involved.

The mathematical manipulation of these functions may take on several different forms. For some types of performance assessments, closed-form solutions can be developed that will enable direct calculation of loss.

3.1.4 Revise Design

If the simulated performance meets or exceeds the performance objectives, the design is completed. If not, the design must be revised in an iterative process until the performance objectives are met. In some instances it may not be possible to meet the stated objectives at reasonable cost, in which case, some relaxation of the original performance objectives may be appropriate.

3.2 SEISMIC PERFORMANCE LEVELS

Generally, a team of decision makers, including the building owner, design professionals, and building officials, will participate in the selection of performance objectives for a building [6]. Stakeholders must evaluate the risk of a hazard event occurring, and must obtain consensus on the acceptable level of performance. The basic questions that should be asked are:

• What events are anticipated?

• What level of loss/damage/casualties is acceptable?

• How often might this happen?

While specific performance objectives can vary for each project, the notion of acceptable performance follows a trend generally corresponding to:

• Little or no damage for small, frequently occurring events

• Moderate damage for medium-size, less frequent events

• Significant damage for very large, very rare events

Building Performance Levels and Ranges

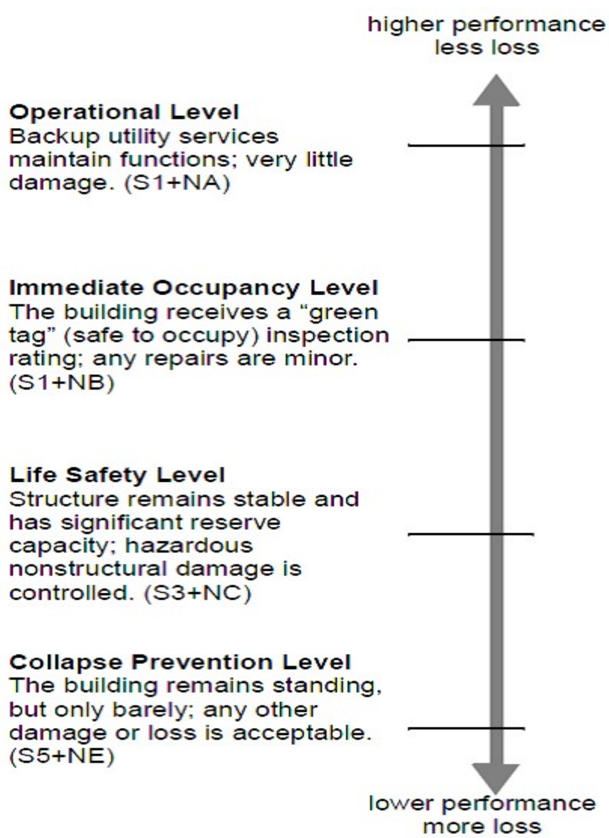
Performance Level: the intended post-earthquake condition of a building; a well-defined point on a scale measuring how much loss is caused by earthquake damage. In addition to casualties, loss may be in terms of property and operational capability.

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.

Figure 3.4 Building Performance Levels [4]



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. Table 3.1 describes the overall levels of structural and nonstructural damage. For comparative purposes, the estimated performance of a new building subjected to the DBE level of shaking is indicated. These performance descriptions are estimates rather than precise predictions, and variation among buildings of the same Performance Level must be expected. Independent performance definitions are provided for structural and nonstructural components. Structural performance levels are identified by both a name and numerical designator (following S-) in Section 3.2.1. Nonstructural performance levels are identified by a name and alphabetical designator (following N-) in Section 3.2.2.

3.2.1 Structural Performance Levels and Ranges

Three discrete Structural Performance Levels and two intermediate Structural Performance Ranges are defined. Acceptance criteria, which relate to the permissible earthquake-induced forces and deformations for the various elements of the building, both existing and new, are tied directly to these Structural Performance Ranges and Levels. A wide range of structural performance requirements could be desired by individual building owners. The three Structural Performance Levels have been selected to correlate with the most commonly specified structural performance requirements. The two Structural Performance Ranges permit users with other requirements to customize their building Objectives.
The Structural Performance Levels are the Immediate Occupancy Level (S-1), the Life Safety Level (S-3), and the Collapse Prevention Level (S-5). Table 3.2 relates these Structural Performance Levels to the limiting damage states for common vertical and horizontal elements of lateral force- resisting systems. The drift values given in Table 3.2 are typical values provided to illustrate the overall structural response associated with various performance levels. The Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety

Range (S-4). Specific acceptance criteria are not provided for design to these intermediate performance ranges. The engineer wishing to design for such performance needs to determine appropriate acceptance criteria.

Acceptance criteria for performance within the Damage Control Range may be obtained by interpolating the acceptance criteria provided for the Immediate Occupancy and Life Safety Performance Levels. Acceptance criteria for performance within the Limited Safety Range may be obtained by interpolating the acceptance criteria for performance within the Life Safety and Collapse Prevention Performance Levels.

3.2.1.1 Immediate Occupancy Performance Level (S-1)

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

3.2.1.2 Life Safety Performance Level (S-3)

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

3.2.1.3 Collapse Prevention Performance Level (S-5)

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

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

3.2.1.4 Damage Control Performance Range (S-2)

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

Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.

3.2.1.5 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.

3.2.1.6 Structural Performance Not Considered (S-6)

Some owners may desire to address certain nonstructural vulnerabilities for example, bracing parapets, or anchoring hazardous materials storage containers—without addressing the performance of the structure itself. The actual performance of the structure is not known and could range from a potential collapse hazard to a structure capable of meeting the Immediate Occupancy Performance Level.

3.2.2 Nonstructural Performance Levels

Nonstructural components addressed in performance levels include architectural components, such as partitions, exterior cladding, and ceilings; and mechanical and electrical components, including HVAC systems, plumbing, fire suppression systems, and lighting.

3.2.2.1 Operational Performance Level (N-A)

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

3.2.2.2 Immediate Occupancy Level (N-B)

Nonstructural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited nonstructural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable. There could be minor window breakage and slight damage to some components.

Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be non-operable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to nonstructural damage is very low.

3.2.2.3 Life Safety Level (N-C)

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

3.2.2.4 Hazards Reduced Level (N-D)

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

3.2.2.5 Nonstructural Performance Not Considered (N-E)

In some cases, the decision may be made to not to address the vulnerabilities of nonstructural components, since many of the most severe hazards to life safety occur as a result of structural vulnerabilities.

3.2.3 Building Performance Levels

Building Performance Levels are obtained by combining Structural and Nonstructural Performance Levels. A large number of combinations are possible. Each Building Performance Level is designated alphanumerically with a numeral representing the Structural Performance Level and a letter representing the Nonstructural Performance Level (e.g. 1-B, 3-C). Table 3.3 indicates the possible combinations and provides names for those that are most likely to be selected as a basis for design. Several of the more common Building Performance Levels are described below.

3.2.3.1 Operational Level (1-A)

This Building Performance Level is a combination of the Structural Immediate Occupancy Level and the Nonstructural Operational Level. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural and nonstructural components. The building is suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources, and possibly with some nonessential systems not functioning. Buildings meeting this performance level pose an extremely low risk to life safety.

Under very low levels of earthquake ground motion, most buildings should be able to meet or exceed this performance level. Typically, however, it will not be economically practical to design for this performance under severe levels of ground shaking, except for buildings that house essential services.

3.2.3.2 Immediate Occupancy Level (1-B)

This Building Performance Level is a combination of the Structural and Nonstructural Immediate Occupancy levels. Buildings meeting this performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. While it would be safe to reoccupy a building meeting this performance level immediately following a major earthquake, nonstructural systems may not function due to either a lack of electrical power or internal damage to equipment. Therefore, although immediate reoccupancy of the building is possible, it may be necessary to perform some cleanup and repair, and await the restoration of utility service, before the building could function in a normal mode. The risk to life safety at this performance level is very low.

Many building owners may wish to achieve this level of performance when the building is subjected to moderate levels of earthquake ground motion. In addition, some owners may desire such performance for very important buildings, under severe levels of earthquake ground shaking. This level provides most of the protection obtained under the Operational Level, without the cost of providing standby utilities and performing rigorous seismic qualification of equipment performance.

3.2.3.3 Life Safety Level (3-C)

This Building Performance Level is a combination of the Structural and Nonstructural Life Safety levels. Buildings meeting this level may experience extensive damage to structural and nonstructural components. Repairs may be required before reoccupancy of the building occurs, and repair may be deemed economically impractical. The risk to life in buildings meeting this performance level is low. Many building owners will desire to meet this performance level for a severe level of ground shaking.

3.2.3.4 Collapse Prevention Level (5-E)

This Building Performance Level consists of the Structural Collapse Prevention Level with no consideration of nonstructural vulnerabilities. Buildings meeting this performance level may pose a significant hazard to life safety resulting from failure of nonstructural components. However, because the building itself does not collapse, gross loss of life should be avoided. Many buildings meeting this level will be complete economic losses.

Table 3.1 Damage Control and Building Performance Levels

	Building Performance Levels			
	Collapse Prevention Level	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall Damage	Severe	Moderate	Light	Very Light
General	Little residual stiffness and strength, but loadbearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	No permanent drift; structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All Systems important to normal operation are functional.
Nonstructural Components	Extensive damage.	Falling hazards mitigated but many architectural, mechanical, and electrical systems	Equipment and contents are generally secure, but may not operate due to mechanical	Negligible damage occurs. Power and other utilities are available, possibly from
		are damaged.	failure or lack of utilities.	standby sources.

Comparison with performance intended for buildings designed, under the NEHRP Provisions, for the Design Earthquake	Significantly more damage and greater risk.	Somewhat more damage and slightly higher risk.	Somewhat more damage and slightly higher risk.	Much less damage and lower risk.

Table 3.2 Structural Performance Levels and Damage -Vertical and Horizontal Elements

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16" width.

	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Unreinforced Masonry Infill Walls	Primary	Extensive cracking and crushing; portions of face course shed.	Extensive cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	Minor (<1/8" width) cracking of masonry infills and veneers. Minor spalling in veneers at a few corner openings.
	Secondary	Extensive crushing and shattering; some walls dislodge.	Same as primary	Same as primary
		0.6% transient or permanent	0.5% transient; 0.3% permanent	0.1% transient; negligible permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element distress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, < 1/16" wide. Coupling beams experience cracking < 1/8" width.

	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience
			shattered and virtually disintegrated.	cracks < 1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent
Concrete Diaphragms		Extensive crushing and observable offset across many cracks.	Extensive cracking (< 1/4" width). Local crushing and spalling.	Distributed hairline cracking. Some minor cracks of larger size (< 1/8‖ width).

Table 3.3 Building Performance Levels/Ranges

Nonstructural Performance Levels	Structural Performance Levels/Ranges					
	S-1 Immediate Occupancy	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention	S-6 Not Considered
N-A Operational	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not Recommended
N-B Immediate Occupancy	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not Recommended
N-C Life Safety	1-C	2-C	3-C	4-C	5-C	6-C
N-D Hazards Reduced	Not Recommended	2-D	3-D	4-D	5-D	6-D
N-E Not Considered	Not Recommended	Not Recommended	Not recommended	4-E	5-E	No Rehabilitation

3.3 SEISMIC HAZARD

The way that ground shaking is characterized in the performance assessment process is dependent on the type of performance objective, (i.e., intensity based, scenario-based or time-based) that is being used. The simplest form of ground shaking characterization occurs when intensity-based performance objectives are used. In this case, it is only necessary to define a specific intensity of motion that the building will be designed to resist. The parameter used to describe ground motion intensity is termed an intensity measure. A number of different intensity measures have been used in the past, including Modified Mercalli Intensity (MMI), Rossi-Forrell Intensity, peak ground acceleration, and spectral response acceleration, among others. For more than 30 years, design procedures have used linear acceleration response spectra and parameters derived from these spectra as the basic intensity measures. Linear acceleration response spectra are useful and form the basis for both present national seismic hazard maps and building code procedures. However, there is presently a lack of consensus as to how to derive and scale ground motion records so that they appropriately match the intensity represented by a response spectrum. Further, most current procedures for ground motion record scaling produce significant variability in predicted response when nonlinear dynamic analyses are performed.

In order to assess the ability of a structure to meet a scenario-based or time based performance objective, it is necessary not only to define a single intensity of motion, but rather, a range of motion and intensities, and the probability of occurrence of each. This information is typically presented in the form of a hazard function. The hazard function for a site is simply an expression of the probability that ground shaking of different intensities may be experienced at the site. The hazard function can be formed on a scenario basis (considering only the occurrence of a specific magnitude earthquake on a specific fault) or on a time-period basis (considering all potential earthquakes on all known faults and the probability of occurrence of each within a defined period).

When time-based performance objectives are used, ground shaking intensity is represented by hazard functions that are developed considering all potential earthquake scenarios, and the probability of occurrence of each scenario within a given period of time. Time-based hazard functions appear similar to scenario-based hazard functions and are used in the same way.

However, rather than indicating the conditional probability of experiencing different levels of shaking intensity given that a specific scenario earthquake occurs, probabilistic hazard functions indicate the total probability of exceeding different shaking intensity levels at a site over a defined period of time. Hazard function may express the probability in the form of an annual probability of exceedance (or nonexceedance), an average return period, or the probability of exceedance (or nonexceedance) in a defined period of years, usually taken as 50. It can be expressed as a mean probability, in which the uncertainty associated with the function is averaged, or confidence bounds associated with the uncertainties can be expressly indicated.

The most common and significant cause of earthquake damage to buildings is ground shaking, thus the effects of ground shaking form the basis for most building code requirements for seismic design. Two levels of earthquake shaking hazard are used: - Design Basic Earthquake and Maximum Considered Earthquake (MCE). MCE earthquake is taken as a ground motion having a 2% probability of exceedance in 50 years (2%/50 year). The DBE earthquake is defined as that ground shaking having a 10% probability of exceedance in 50 years (10%/50 year).

Response spectra are used to characterize earthquake shaking demand on buildings. In USA, ground shaking hazard is determined from available response spectrum acceleration contour maps. Maps showing 5%-damped response spectrum ordinates for short-period (0.2 second) and long-period (1 second) response can be used directly for developing design response spectra for either or both the DBE and MCE, or for earthquakes of any desired probability of exceedance.

In the Site-Specific Procedure, ground shaking hazard is determined using a specific study of the faults and seismic source zones that may affect the site, as well as evaluation of the regional and geologic conditions that affect the character of the site ground motion caused by events occurring on these faults and sources.

3.3.1 General Ground Shaking Hazard Procedure

The general procedures of this section may be used to determine acceleration response spectra for any of the following hazard levels:

• Design Basic Earthquake (DBE)

• Maximum Considered Earthquake (MCE)

• Earthquake with any defined probability of exceedance in 50 years

Deterministic estimates of earthquake hazard, in which an acceleration response spectrum is obtained for a specific magnitude earthquake occurring on a defined fault, shall be made using the Site-Specific Procedures of Section 3.3.2.

The basic steps for determining a response spectrum under this general procedure are:

1. Determine whether the desired hazard level corresponds to one of the levels contained in the ground shaking hazard maps. These hazard maps include maps for MCE ground shaking hazards as well as for hazards with 10%/50 year exceedance probabilities.
2. If the desired hazard level corresponds with one of the mapped hazard levels, obtain spectral response acceleration parameters directly from the maps, in accordance with Section 3.3.1.1.
3. If the desired hazard level is the DBE, then obtain the spectral response acceleration parameters from the maps, in accordance with Section 3.3.1.2.
4. If the desired hazard level does not correspond with the mapped levels of hazard, then obtain the spectral response acceleration parameters from the available maps, and modify them to the desired hazard level, either by logarithmic interpolation or extrapolation, in accordance with Section 3.3.1.3.
5. Obtain design spectral response acceleration parameters by adjusting the mapped, or modified mapped spectral response acceleration parameters for site class effects, in accordance with Section 3.3.1.4.
6. Using the design spectral response acceleration parameters that have been adjusted for site class effects, construct the response spectrum in accordance with Section 3.3.1.5.

3.3.1.1 MCE and 10%/50 Response Acceleration Parameters

The mapped short-period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for MCE ground motion hazards may be obtained directly from the maps. The mapped short period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for 10%/50 year ground motion hazards may also be obtained directly from the maps.

Parameters S_S and S_1 shall be obtained by interpolating between the values shown on the response acceleration contour lines on either side of the site, on the appropriate map, or by using the value shown on the map for the higher contour adjacent to the site.

3.3.1.2 DBE Response Acceleration Parameters

The mapped short-period response acceleration parameter, S_S, and mapped response acceleration parameter at a one-second period, S_1, for DBE ground shaking hazards shall be taken as the smaller of the following:

• The values of the parameters S_S and S_1, respectively, determined for 10%/50 year ground motion hazards, in accordance with Section 3.3.1.1.

• Two thirds of the values of the parameters S_S and S_1, respectively, determined for MCE ground motion hazards, in accordance with Section 3.3.1.1.

3.3.1.3 Adjustment of Mapped Response Acceleration Parameters for Other Probabilities of Exceedance

When the mapped MCE short period response acceleration parameter, S_S, is less than 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance between 2%/50 years and 10%/50 years may be determined from the equation:

$$\ln(S_i) = \ln(S_{i10/50}) + [\ln(S_{iMCE}) - \ln(S_{i10/50})][0.606\ln(P_R) - 3.73] \tag{3.1}$$

Where :

In(Si) = Natural logarithm of the spectral acceleration parameter (–i‖ = –s‖ for short period or –i‖ = 1 for 1 second period) at the desired probability of exceedance

In($Si_{10/50}$) = Natural logarithm of the spectral acceleration parameter (–i‖ = –s‖ for short period or –i‖ = 1 for 1 second period) at a 10%/50 year exceedance rate

1n(Si_{MCE}) = Natural logarithm of the spectral acceleration parameter (–i‖ = –s‖ for short period or –i‖ = 1 for 1 second period) for the MCE hazard level

In(P_R) = Natural logarithm of the mean return period corresponding to the exceedance probability of the desired hazard level and the mean return period P_R at the desired exceedance probability may be calculated from the equation:

$$P_R = \frac{1}{1 - e^{0.02\ln(1 - P_{E50})}} \tag{3.2}$$

where, P_{E50} is the probability of exceedance in 50 years of the desired hazard level.

When the mapped MCE short period response acceleration parameter, S_S, is greater than or equal to 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance between 2%/50 years and 10%/50 years may be determined from the equation:

$$S_i = S_{i10/50}\left(\frac{P_R}{475}\right)^n \tag{3.3}$$

where Si, $Si_{10/50}$, and P_R are as defined above and n is dependent on particular site.
When the mapped MCE short period response acceleration parameter, S_S, is less than 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance greater than 10%/50 years may be determined from Equation 3.3, where the exponent n is dependent on particular.
When the mapped MCE short period response acceleration parameter, S_S, is greater than or equal to 1.5g, the modified mapped short period response acceleration parameter, S_S, and modified mapped response acceleration parameter at a one-second period, S_1, for probabilities of exceedance greater than 10%/50 years may be determined from Equation 3.3.

3.3.1.4 Adjustment for Site Class

The design short-period spectral response acceleration parameter, S_{XS}, and the design spectral response acceleration parameter at one second, S_{X1}, shall be obtained respectively from Equations 3.4 and 3.5 as follows:

$$S_{XS} = F_a S_S \tag{3.4}$$

$$S_{X1} = F_v S_1 \tag{3.5}$$

where F_a and F_v are site coefficients determined respectively from Tables 3.4 and 3.5, based on the site class and the values of the response acceleration parameters S_S and S_1.
Site classes shall be defined as follows:

- **Class A:** Hard rock with measured shear wave velocity, $V_s > 5{,}000$ ft/sec
- **Class B:** Rock with 2,500 ft/sec < V_s< 5,000 ft/sec
- **Class C:** Very dense soil and soft rock with

1,200 ft/sec < V_s < 2,500 ft/sec or with either standard blow count N > 50 or undrained shear strength S_u > 2,000 psf

• **Class D:** Stiff soil with 600 ft/sec < V_s < 1,200 ft/sec or with 15 < N < 50 or 1,000 psf < S_u < 2,000psf

• **Class E:** Any profile with more than 10 feet of soft clay defined as soil with plasticity index *PI* >20, or water content w > 40 percent, and Su < 500 psf or a soil profile with Vs < 600 ft/sec. If insufficient data are available to classify a soil profile as type A through D, a type E profile should be assumed.

• **Class F:** Soils requiring site-specific evaluations:

– Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly-sensitive clays, collapsible weakly-cemented soils

– Peats and/or highly organic clays (H > 10 feet of peat and/or highly organic clay, where H = thickness of soil)

– Very high plasticity clays (H > 25 feet with PI > 75 percent)

– Very thick soft/medium stiff clays (H > 120 feet)

Table 3.4 Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Response Acceleration S_S

Site Class	Mapped Spectral Acceleration at Short Periods S_S				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

* Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Table 3.5 Values of F_v as a Function of Site Class and Mapped Spectral Response Acceleration at One- Second Period S_1

Site Class	Mapped Spectral Acceleration at One-Second Periods S_1				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3

D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

* Site-specific geotechnical investigation and dynamic site response analyses should be performed.

NOTE: Use straight-line interpolation for intermediate values.

The parameters v_s, N, and s_u are, respectively, the average values of the shear wave velocity, Standard Penetration Test (SPT) blow count, and undrained shear strength of the upper 100 feet of soils at the site. These values may be calculated from Equation 3.6, below:

$$\bar{v}_s, \bar{N}, \bar{s}_u = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}, \frac{d_i}{N_i}, \frac{d_i}{s_{ui}}} \tag{3.6}$$

Where:

Ni = SPT blow count in soil layer ―i‖

n = Number of layers of similar soil materials for which data is available

di = Depth of layer ―i‖

S_{ui} = Undrained shear strength in layer ―i‖

V_{si} = Shear wave velocity of the soil in layer ―i‖

and

$$\sum_{i=1}^{n} d_i = 100\ ft \tag{3.7}$$

Where reliable v_s data are available for the site, such data should be used to classify the site. If such data are not available, N data should preferably be used for cohesionless soil sites (sands, gravels), and s_u data for cohesive soil sites (clays). For rock in profile classes B and C, classification may be based either on measured or estimated values of v_s. Classification of a site as Class A rock should be based on measurements of v_s either for material at the site itself, or for similar rock materials in the vicinity; otherwise, Class B rock should be assumed. Class A or B profiles should not be assumed to be present if there is more than 10 feet of soil between the rock surface and the base of the building.

3.3.2 General Response Spectrum

A general, horizontal response spectrum may be constructed by plotting the following two functions in the spectral acceleration vs. structural period domain, as shown in Figure 3.5. Where a vertical response spectrum is required, it may be constructed by taking two-thirds of the spectral ordinates, at each period, obtained for the horizontal response spectrum.

$S_a = (S_{XS} / B_S)(0.4 + 3T/T_0)$ (3.8)

For $0 < T \leq 0.2T_0$

$S_a = (S_{X1} / (B_1 T))$ for $T > T_0$

(3.9)

where To is given by the equation

$T_0 = (S_{X1}B_S) / (S_{XS}B_1)$

(3.1

0)

where B_S and B_1 are taken from Table 3.6.

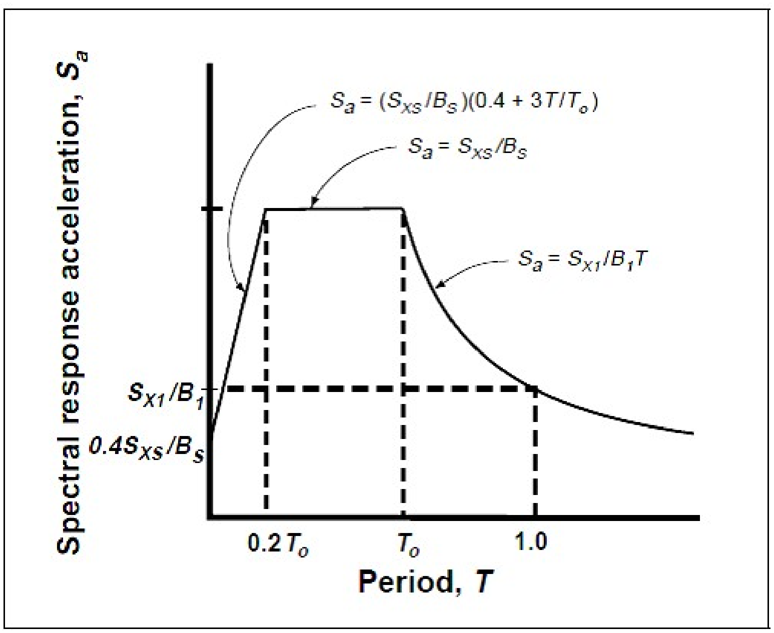


Figure 3.5 General response spectrum

Table 3.6 Damping Coefficients B_S and B_1 as a Function of Effective Damping β [4]

Effective Damping β (percentage of critical)[1]	B_s	B_1
< 2	0.8	0.8
5	1.0	1.0
10	1.3	1.2
20	1.8	1.5
30	2.3	1.7
40	2.7	1.9
> 50	3.0	2.0

1. The damping coefficient should be based on linear interpolation for effective damping values other than those given.

In general, it is recommended that a 5% damped response spectrum be used for the design of most buildings and structural systems. Exceptions are as follows:

• For structures without exterior cladding an effective viscous damping ration, β, of 2% should be assumed.

• For structures with wood diaphragms and a large number of interior partitions and cross walls that interconnect the diaphragm levels, an effective viscous damping ratio, β, of 10% may be assumed.

• For structures rehabilitated using seismic isolation technology or enhanced energy dissipation technology, an equivalent effective viscous damping ratio, β, should be calculated.

3.3.3 Site-Specific Ground Shaking Hazard

Where site-specific ground shaking characterization is used as the basis of design, the characterization shall be developed in accordance with this section.

3.3.3.1 Site-Specific Response Spectrum

Development of site-specific response spectra shall be based on the geologic, seismologic, and soil characteristics associated with the specific site.

Response spectra should be developed for an equivalent viscous damping ratio of 5%. Additional spectra should be developed for other damping ratios appropriate to the indicated structural behavior, as discussed in Section 3.3.1.5. When the 5% damped site-specific spectrum has spectral amplitudes in the period range of greatest significance to the structural response that are less than 70 percent of the spectral amplitudes of the General Response Spectrum, an independent third-party review of the spectrum should be made by an individual with expertise in the evaluation of ground motion.

When a site-specific response spectrum has been developed and other sections require values for the spectral response parameters, S_{XS}, S_{X1}, or T_0, they may be obtained in accordance with this section. The value of the design spectral response acceleration at short periods, S_{XS}, shall be taken as then response acceleration obtained from the site-specific spectrum at a period of 0.2 seconds, except that it should be taken as not less than 90% of the peak response acceleration at any period. In order to obtain a value for the design spectral response acceleration parameter S_{X1}, a curve of the form $Sa = S_{X1}/T$ should be graphically overlaid on the site-specific spectrum such that at any period, the value of Sa obtained from the curve is not less than 90% of that which would be obtained directly from the spectrum. The value of T_0 shall be determined in accordance with Equation 3.11. Alternatively, the values obtained in accordance with Section 3.3.1 may be used for all of these parameters.

$$T_0 = S_{X1} \,.\, S_{XS} \tag{3.11}$$

3.3.3.2 Acceleration Time Histories

Time-History Analysis shall be performed with no fewer than three data sets (two horizontal components or, if vertical motion is to be considered, two horizontal components and one vertical component) of appropriate ground motion time histories that shall be selected and scaled from no fewer than three recorded events.

Appropriate time histories shall have magnitude, fault distances, and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground motion time history data sets are not available, appropriate simulated time history data sets may be used to make up the total number required. For each data set, the square root of the sum of the squares (SRSS) of the 5%-damped site-specific spectrum of the scaled horizontal components shall be constructed. The data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5%-damped spectrum for the design earthquake for periods between 0.2T seconds and 1.5T seconds (where T is the fundamental period of the building). Where three time history data sets are used in the analysis of a structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time history data sets are employed, the average value of each response parameter may be used to determine design acceptability.

3.3.4 Seismicity Zones

Seismicity zones are defined as follows.

3.3.4.1 Zones of High Seismicity

Buildings located on sites for which the 10%/50 year, design short-period response acceleration, S_{XS}, is equal to or greater than 0.5g, or for which the 10%/50 year design one-second period response acceleration, S_{X1}, is equal to or greater than 0.2g shall be considered to be located within zones of high seismicity.

3.3.4.2 Zones of Moderate Seismicity

Buildings located on sites for which the 10%/50 year, design short-period response acceleration, S_{XS}, is equal to or greater than 0.167g but is less than 0.5g, or for which the 10%/50 year, design one-second period response acceleration, S_{X1}, is equal to or greater than 0.067g but less than 0.2g shall be considered to be located within zones of moderate seismicity.

3.3.4.3 Zones of Low Seismicity

Buildings located on sites that are not located within zones of high or moderate seismicity, as defined in Sections 3.3.3.1 and 3.3.3.2 shall be considered to be located within zones of low seismicity.

3.3.5 Other Seismic Hazards

In addition to ground shaking, seismic hazards can include ground failure caused by surface fault rupture, liquefaction, lateral spreading, differential settlement, and landsliding. Earthquake-induced flooding, due to tsunami, or failure of a water-retaining structure, can also pose a hazard to a building site.

3.4 PUSHOVER ANALYSIS

In Pushover analysis, a static horizontal force profile, usually proportional to the design force profiles specified in the codes, is applied to the structure. The force profile is then incremented in small steps and the structure is analyzed at each step. As the loads are increased, the building undergoes yielding at a few locations. Every time such yielding takes place, the structural properties are modified approximately to reflect the yielding. The analysis is continued till the structure collapses, or the building reaches certain level of lateral displacement.

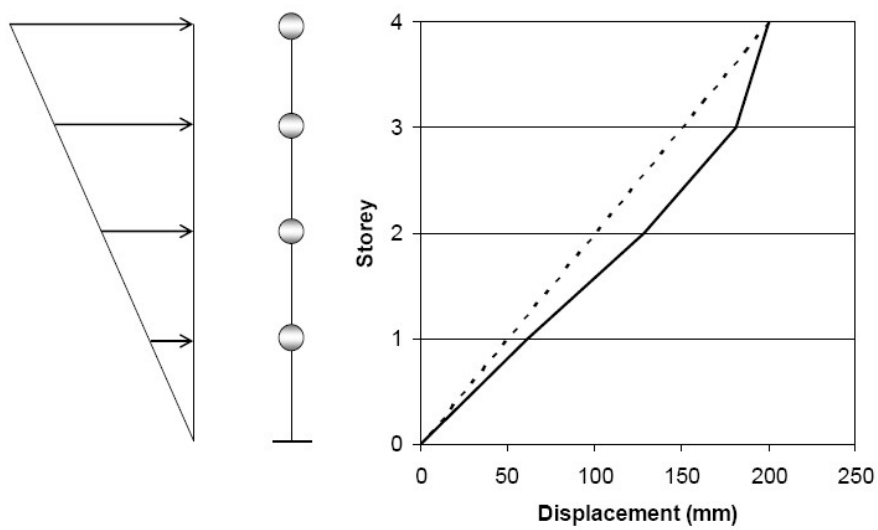


Figure 3.6 Inverted triangular Loading for Pushover Analysis

3.4.2 NEED FOR PUSHOVER ANALYSIS

Conventionally, seismic assessment and design has relied on linear or equivalent linear (with reduced stiffness) analysis of structural systems. In this approach, simple models are used for various components of the structure, which is subjected to seismic forces evaluated from elastic or design spectra, and reduced by force reduction (or behavior) factors. The ensuing displacements are amplified to account for the reduction of applied forces. This procedure, though simple and easy to apply in the design office environment, suffers from the following shortcomings:

1. **T**he force reduction factors recommended in codes of practice are approximate and do

not necessarily represent the specific structure under consideration.

2. When critical zones of a structure enter into the inelastic range, the force and deformation distribution change significantly. This change is not represented by a global reduction of forces.

3. The mechanism that will most likely perpetuate collapse is unlikely to be that represented by the elastic action and deformation distribution.

4. The global and particularly the local distribution of deformations in the inelastic range may bear no resemblance to those in the elastic range. The same applies to the values of deformations, not just the distribution.

As a consequence of the above, the reduced forces - amplified deformations linear elastic approach fails to fit within the principle of failure mode control, which is part of performance based assessment and design. This in turn has lead to an increase in the use of inelastic analysis as a more realistic means of assessing the deformational state in structures subjected to strong ground motion.

The pushover analysis is a significant step forward by giving consideration to those inelastic response characteristics that will distinguish between good and bad performance in severe earthquakes. The non-linear static pushover analysis is a partial and relatively simple intermediate solution to the complex problem of predicting force and deformation demands imposed on a structure and its elements due to ground motion.

Here, the important terms are static and analysis. Static implies that a static method is being employed to represent a dynamic phenomenon; a representation that is adequate in many cases but doomed to failure in some cases. Analysis implies that a system solution has been created already and the pushover is employed to evaluate the solution and modify it as needed.

The pushover is a part of an evaluation process and provides estimates of demands imposed on structures and elements. Hence, there is always a need of a method which is more rational and accurate and at the same time able to identify seismic deficiencies correctly and that too in correct order of vulnerability. Pushover analysis is able to satisfy these criteria satisfactorily and in a convenient way.

3.4.3 DESCRIPTION OF PUSHOVER ANALYSIS

The non-linear static pushover procedure was originally formulated and suggested by two agencies namely, federal emergency management agency (FEMA) and applied technical council (ATC), under their seismic rehabilitation programs and guidelines. This is included in the documents FEMA-273 [4], FEMA-356 [2] and ATC-40 [32].

3.4.4 Introduction to FEMA-273

The primary purpose of FEMA-273 [4] document is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. The Guidelines for the Seismic Rehabilitation of Buildings are intended to serve as a ready tool for design professionals for carrying out the design and analysis of buildings, a reference document for building regulatory officials, and a foundation for the future development and implementation of building code provisions and standards.

3.4.4.1 Introduction to ATC-40

Seismic Evaluation and Retrofit of Concrete Buildings commonly referred to as ATC-40 [32] was developed by the Applied Technology Council (ATC) with funding from the California Safety Commission. Although the procedures recommended in this document are for concrete buildings, they are applicable to most building types.

ATC-40 [32] recommends the following steps for the entire process of evaluation and retrofit:

1. Initiation of a Project: Determine the primary goal and potential scope of the project.
2. Selection of Qualified Professionals: Select engineering professionals with a demonstrated experience in the analysis, design and retrofit of buildings in seismically hazardous regions. Experience with PBSE and non-linear procedures are also needed.
3. Performance Objective: Choose a performance objective from the options provided for a specific level of seismic hazard.
4. Review of Building Conditions: Perform a site visit and review drawings.
5. Alternatives for Mitigation: Check to see if the non-linear procedure is appropriate or relevant for the building under consideration.
6. Peer Review and Approval Process: Check with building officials and consider other

quality control measures appropriate to seismic evaluation and retrofit.

7. Detailed Investigations: Perform a nonlinear static analysis if appropriate.
8. Seismic Capacity: Determine the inelastic capacity curve also known to pushover curve. Covert to capacity spectrum.
9. Seismic Hazard: Obtain a site specific response spectrum for the chosen hazard level and convert to spectral ordinates format.
10. Verify Performance: Obtain performance point as the intersection of the capacity spectrum and the reduced seismic demand in spectral ordinates (ADRS) format. Check all primary and secondary elements against acceptability limits based on the global performance goal.

3.4.5 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 Base Shear (V) Roof Displacement (Δ) 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. Fig18 shows the common lateral load pattern used in pushover analysis.

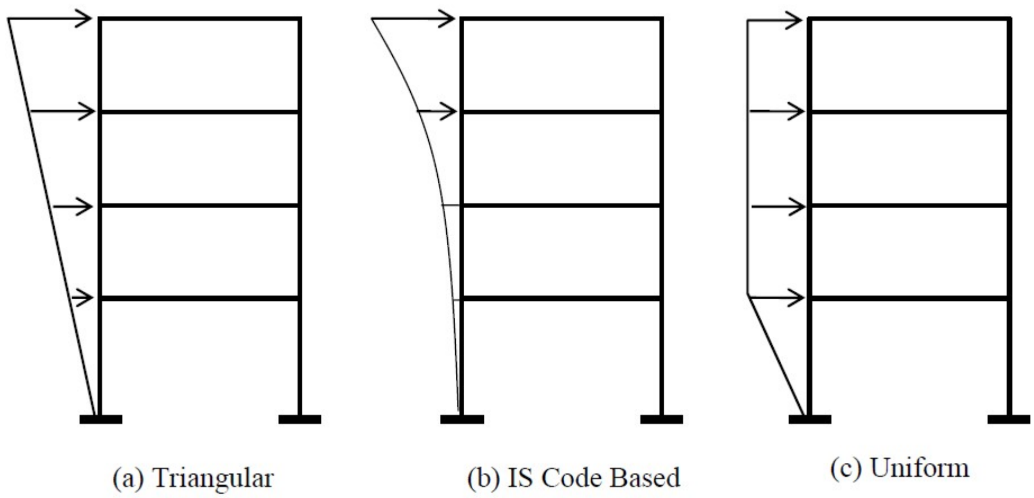


Fig3.7: Lateral load pattern for pushover analysis as per FEMA 356 (considering uniform mass distribution)

3.4.6 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.

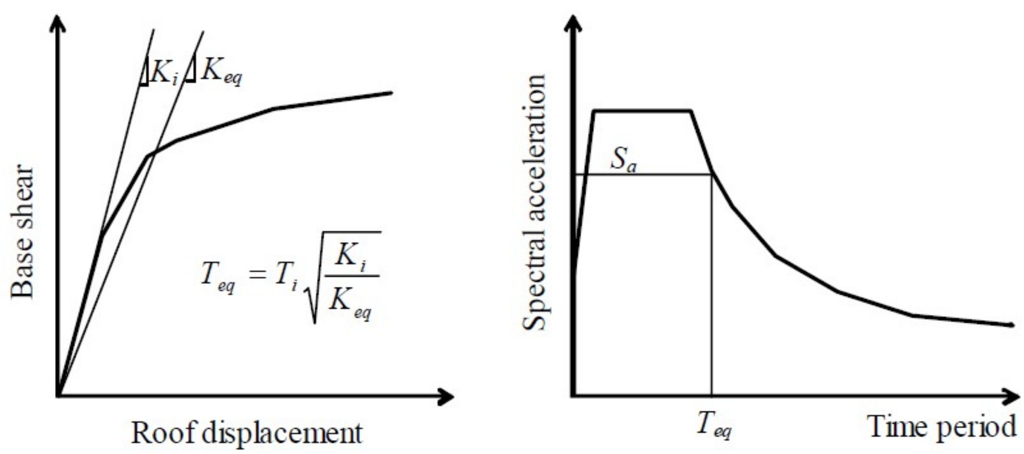
A. Displacement Coefficient Method (FEMA 356)

This method primarily estimates the elastic displacement of an equivalent SDOF system assuming initial linear properties and damping for the ground motion excitation under consideration. Then it estimates the total maximum inelastic displacement response for the building at roof by multiplying with a set of displacement coefficients.

The process begins with the base shear versus roof displacement curve (pushover curve) as shown in Fig19a. An equivalent period (*Teq*) is generated from initial period (*Ti*) by graphical procedure. This equivalent period represents the linear stiffness of the equivalent SDOF system.

The peak elastic spectral displacement corresponding to this period is calculated directly from the response spectrum representing the seismic ground motion under consideration (Fig19b).

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a$$



(a) Pushover Curve (b) Elastic Response Spectrum

Fig3.8: Schematic representation of Displacement Coefficient Method (FEMA 356)

Now, the expected maximum roof displacement of the building (target displacement) under the selected seismic ground motion can be expressed as:

$$\delta_t = C_0 C_1 C_2 C_3 S_d = C_0 C_1 C_2 C_3 \frac{T_{eq}^2}{4\pi^2} S_a$$

C_0= a shape factor (often taken as the first mode participation factor) to convert the spectral displacement of equivalent SDOF system to the displacement at the roof of the building.

C_1= the ratio of expected displacement (elastic plus inelastic) for an inelastic system to the displacement of a linear system.

C_2= a factor that accounts for the effect of pinching in load deformation relationship due to strength and stiffness degradation

C_3= a factor to adjust geometric nonlinearity (P-Δ) effects

These coefficients are derived empirically from statistical studies of the nonlinear response history analyses of SDOF systems of varying periods and strengths and given in FEMA 356.

From the above definitions of the coefficients, it is clear that the change in building geometry will affect C_0significantly whereas it is likely to have very little influence on the other factors.

As per FEMA 356, the values of C_0factor for shear buildings depend only on the number of levels and the lateral load pattern used in the pushover analysis. Table 8 presents the values of C_0provided by the FEMA 356 for shear buildings. In practice, Setback buildings have 5 or more levels and the C_0factor, as per FEMA 356, is constant for buildings with 5 or more levels (Table 8).

Number of storeys	Triangular Load Pattern	Uniform Load Pattern
1	1.0	1.00
2	1.2	1.15
3	1.2	1.20
5	1.3	1.20
10+	1.3	1.20

Table 3.7: Values of C_0factor for shear building as per FEMA 356

3.4.7 PUSHOVER ANALYSIS GUIDELINES AS PER ATC-40

In Nonlinear Static Procedure, the basic demand and capacity parameter for the analysis is the lateral displacement of the building. The generation of a capacity curve (base shear v/s roof displacement) defines the capacity of the building uniquely for an assumed force distribution and displacement pattern. It is independent of any specific seismic shaking demand and replaces the base shear capacity of conventional design procedures. If the building displaces laterally, its response must lie on this capacity curve. A point on the curve defines a specific damage state for the structure, since the deformation for all components can be related to the global displacement of the structure. By correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity, a point can be found on the capacity curve that estimates the maximum displacement of the building the earthquake will cause. This defines the performance point or target displacement. The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met.

Thus, for the Nonlinear Static Procedure, a static pushover analysis is performed using a nonlinear analysis program for an increasing monotonic lateral load pattern. An alternative is to perform a step by step analysis using a linear program. The base shear at each step is plotted again roof displacement. The performance point is found using the Capacity Spectrum Procedure. The individual structural components are checked against acceptability limits that depend on the global performance goals. The nature of the acceptability limits depends on specific components. Inelastic rotation is typically one of acceptability parameters for beam and column hinges. The limits on inelastic rotation are based on observation from tests and the collective judgment of the development team.

3.4.7.1 Inelastic Component Behavior

The key step for the entire analysis is identification of the primary structural elements, which should be completely modeled in the non-linear analysis. Secondary elements, which do not significantly contribute to the building‘s lateral force resisting system, do not need to be included in the analysis.

In concrete buildings, the effects of earthquake shaking are resisted by vertical frame elements or wall elements that are connected to horizontal elements (diaphragms) at the roof and floor levels. The structural elements may themselves comprise of an assembly of elements such as columns, beam, wall piers, wall spandrels etc. It is important to identify the failure mechanism for these primary structural elements and define their non-linear properties accordingly. The properties of interest of such elements are relationships between the forces (axial, bending and shear) and the corresponding inelastic displacements (displacements, rotations, drifts). Earthquakes usually load these elements in a cyclic manner as shown in Fig. 3.7. For modeling and analysis purposes, these relationships can be idealized as shown in Fig. 3.8 using a combination of empirical data, theoretical strength and strain compatibility.

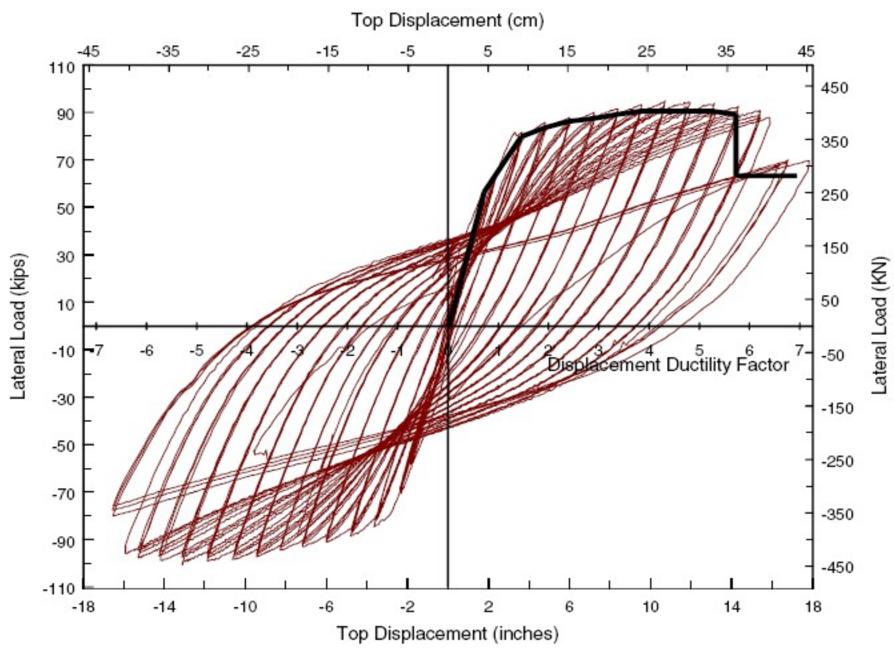


Figure 3.9 Backbone curve from actual hysteretic behavior

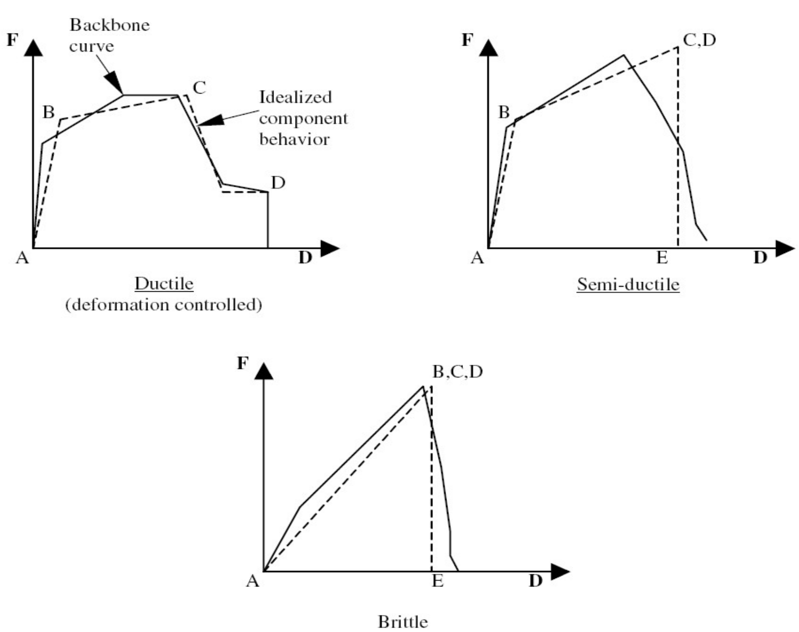


Figure 3.10 Idealized component behavior from backbone curves

Using the component load-deformation data and the geometric relationships among components and elements, a global model of the structure relates the total seismic forces on a building to it overall lateral displacement to generate the capacity curve. During the pushover process of developing the capacity curve as brittle elements degrade, ductile elements take over the resistance and the result is a saw tooth shape that helps visualize the performance. Once the global displacement demand is estimated for a specific seismic hazard, the model is used to predict the resulting deformation in each component. The ATC 40 document provides acceptability limits for component deformations depending on the specified performance level.

3.4.7.2 CAPACITY SPECTRUM METHOD

One of the methods used to determine the performance point is the Capacity Spectrum Method, also known as the Acceleration-Displacement Response Spectra method (ADRS). The Capacity Spectrum method requires that both the capacity curve and the demand curve be represented in response spectral ordinates.It characterizes the seismic demand initially using a 5% damped linear-elastic response spectrum and reduces the spectrum to reflect the effects of energy dissipation to estimate the inelastic displacement demand. The point at which the Capacity curve intersects the reduced demand curve represents the performance point at which capacity and demand are equal.

3.4.7.3 Conversion of Pushover curve to Capacity Spectrum Curve

To convert a spectrum from the standard S_a (Spectra Acceleration) vs T (Period) format found in the building codes to ADRS format, it is necessary to determine the value of Sd_i (Spectral Displacement) for each point on the curve, Sa_iT_i This can be done with the equation:

$$Sd_i = \frac{T^2}{4\pi^2} Sa_i g \tag{3.12}$$

Standard demand response spectra contain a range of constant spectral acceleration and a second range of constant spectral velocity; S_v. Spectral acceleration, Sa and displacement at period T_i are given by:

$$Sa_i g = \frac{2\pi}{T_i} Sv \qquad Sd_i = \frac{T_i}{2\pi} Sv \tag{3.13}$$

The capacity spectrum can be developed from the pushover curve by a point by point conversion to the first mode spectral coordinates. Any point *Vi* (Base Shear), δ*i* (Roof Displacement) on the capacity (pushover) curve is converted to the corresponding point Sa_i, Sd_i on the capacity spectrum using the equations:

$$Sa_i = \frac{V_i / W}{\alpha_1} \tag{3.14}$$

$$Sd_i = \frac{\delta_i}{PF_1 \times \phi_{1,Roof}} \tag{3.15}$$

Where α_1 and PF_1, are the modal mass coefficients and participation factors for the first natural mode of the structure respectively. ϕ_{1roof} is the roof level amplitude of the first mode.

The modal participation factors and modal coefficient are calculated as:

$$PF_1 = \frac{\sum_{i=1}^{n} (w_i \phi_{i1}) / g}{\sum_{i=1}^{n} (w_i \phi_{i1}^2) / g} \tag{3.16}$$

$$\alpha 1 = \frac{[\sum_{i=1}^{n} (w_1 \phi_{i1}) / g]^2}{[\sum_{i=1}^{n} (w_i / g)][\sum_{i=1}^{n} (w_1 \phi_{i1}^2) / g} \tag{3.17}$$

Where w_i is the weight at any level i.

As displacement increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Inelastic displacements increase damping and reduce demand. The Capacity Spectrum Method reduces the demand to find an intersection with the capacity spectrum, where the displacement is consistent with the implied damping. Figure 3.9 shows the conversion of Pushover curve to capacity spectrum curve.

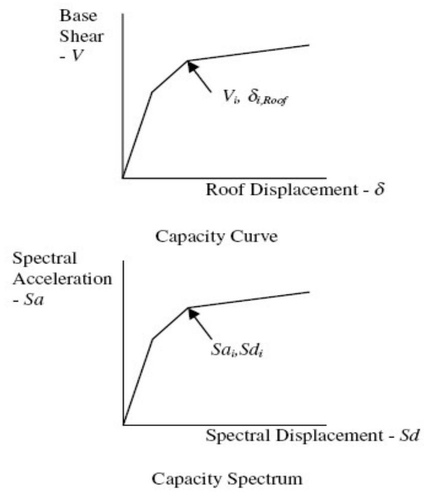


Figure 3.11 Capacity Spectrum Conversion

The damping that occurs when the structure is pushed into the inelastic range can be viewed as a combination of viscous and hysteretic damping. Hysteretic damping can be represented as equivalent viscous damping. Thus, the total effective damping can be estimated as:

$$\beta_{eff} = \lambda\beta_0 + 0.05 \qquad (3.18)$$

Where β_0 is the hysteretic damping and 0.05 is the assumed 5% viscous damping inherent in the structure. The λ-factor (called κ-factor in ATC-40) is a modification factor to account for the extent to which the actual building hysteresis is well represented by the bilinear representation of the capacity spectrum (See Table 3.7 & 3.8 and Figure 3.10).

The term β_o can be calculated using:

$$\beta_{o} = \frac{1}{4\pi}\frac{E_D}{E_{SO}} \qquad (3.19)$$

Where E_D is the energy dissipated by damping and E_{So} is the maximum strain energy. The physical significance is explained in Fig. 3.10.

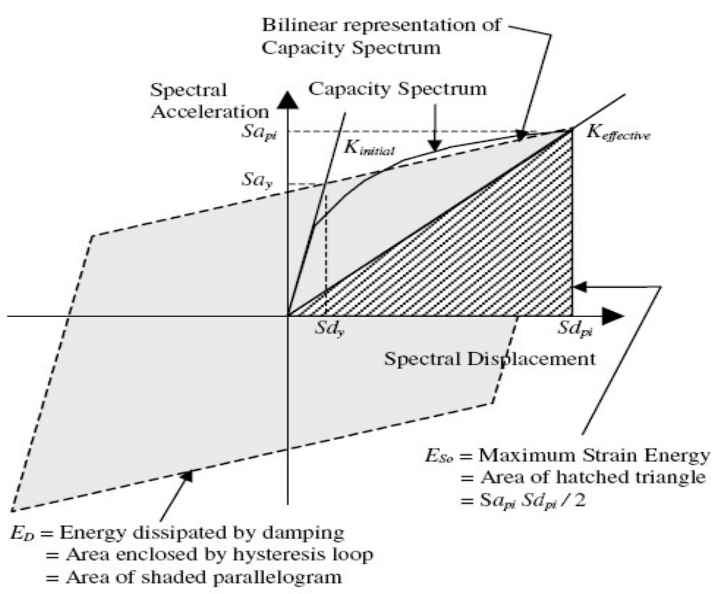


Figure 3.12 Derivation of Energy dissipated by Damping

Table 3.8: Structural Behavior Types

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

Table 3.8: Values for Damping Modification Values, λ

STRUCTURAL BEHAVIOUR TYPE	βo (PERCENT)	λ
TYPE A	≤ 16.25	1.0
	≥ 16.25	1.13 – 0.51
TYPE B	≤ 25	0.67
	≥ 25	0.845 - 0.446
TYPE C	ANY VALUE	0.33

To account for the damping, the response spectrum is reduced by reduction factors SR_A and SR_V which are given by:

$$SR_A = \frac{1}{B_s} = \frac{3.21 - 0.68\ln(\beta_{eff})}{2.12} \tag{3.20}$$

$$SR_V = \frac{1}{B_L} = \frac{2.31 - 0.41\ln(\beta_{eff})}{1.65} \tag{3.21}$$

Both SR_A and SR_V must be greater than or equal to allowable values. The elastic response spectrum (5% damped) is thus reduced to a response spectrum with damping values greater than 5% critically damped (See Figure 3.11).

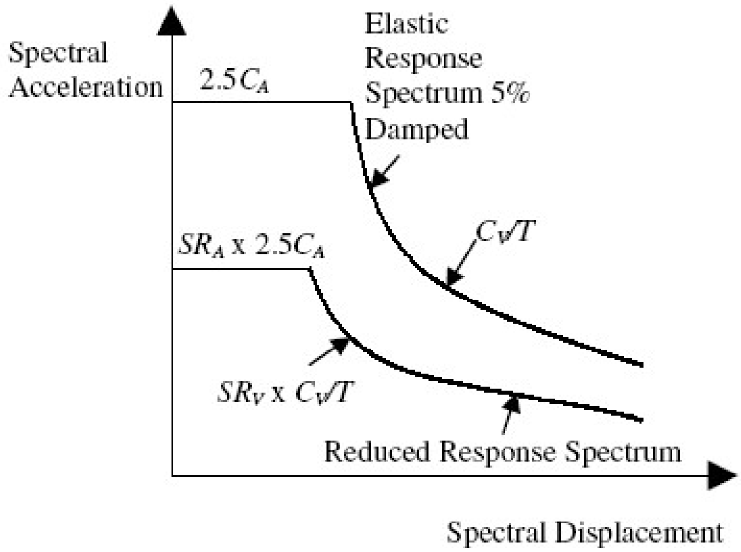


Figure 3.13 Reduced Response Spectrum

3.4.4.1 Determination of Performance Point

There are three procedures described in ATC-40 to find the performance point.

The most transparent and most convenient method for programming is

Procedure A, which uses a set of equations described in ATC-40.

Procedure B is also an iterative method to find the performance point, which uses the assumption that the yield point and the post yield slope of the bilinear representation, remains constant. This is adequate for most cases; however, in some cases this assumption may not be valid.

Procedure C is graphical method that is convenient for hand as well as software analysis. SAP2000 uses this method for the determination of performance point. To find the performance point using Procedure C the following steps are used:

First of all, the single demand spectrum (variable damping) curve is constructed by doing the following for each point on the Pushover Curve:

1. Draw a radial line through a point on the Pushover curve. This is a line of constant period.
2. Calculate the damping associated with the point on the curve, based on the area under the curve upto that point.
3. Construct the demand spectrum, plotting it for the same damping level as associated with the point on the pushover curve.
4. The intersection point for the radial line and associated demand spectrum represents a point on the Single Demand Spectrum (Variable Damping Curve).

A number of arbitrary points are taken on the Pushover curve and such points are obtained. A curve is then drawn by joining through these points. The intersection of this curve with the original pushover curve gives the Performance Point of the Structure as shown in fig. 3.12.

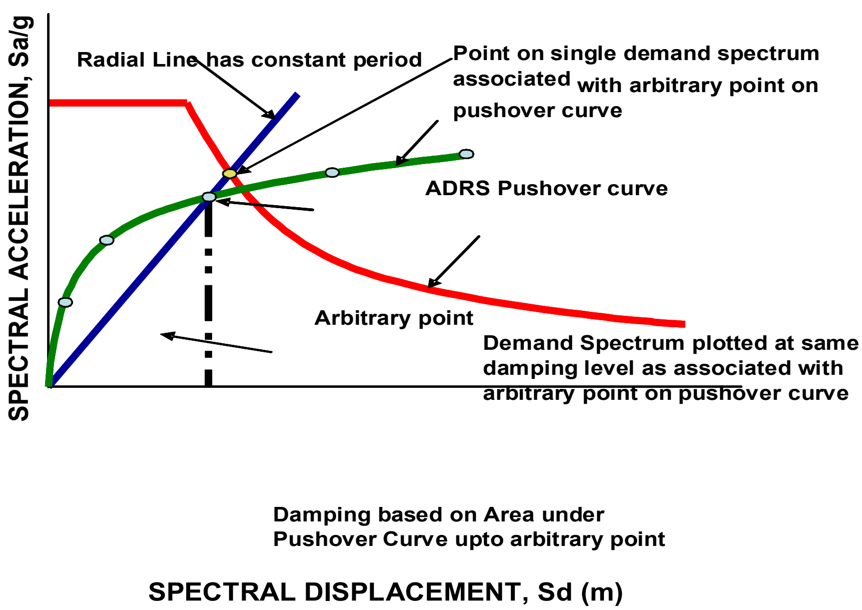


Figure 3.14 Capacity Spectrum Procedure C to Determine Performance Point.

4.1 GENERAL

The main objective of performance based seismic design of buildings is to avoid total catastrophic damage and to restrict the structural damages caused, to the performance limit of the building. For this purpose Static pushover analysis is used to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

4.2 PERFORMANCE OBJECTIVE

The following two-level performance objective is suggested for new ordinary structures.

· Under DBE, damage must be limited to Grade 2 (slight structural damage, moderate Nonstructural damage) in order to enable Immediate Occupancy after DBE.

· Under MCE, damage must be limited to Grade 3 (moderate structural damage, heavy Nonstructural damage) in order to ensure Life Safety after MCE.

4.3 DESCRIPTION OF BUILDING

In the present work, a four storied reinforced concrete frame building situated in Zone IV, is taken for the purpose of study. The plan area of building is 36 x 36 m with 4.2m as height of each typical level. It consists of 2 bays of 18m each in X-direction and 2 bays of 4m each in Y-direction. Hence, the building is symmetrical about both the axis. The total height of the building is 14m. The building is considered as a Specal Moment resisting frame. The plan of building is shown in fig. 4.1 and the front elevation is shown in fig. 4.2.

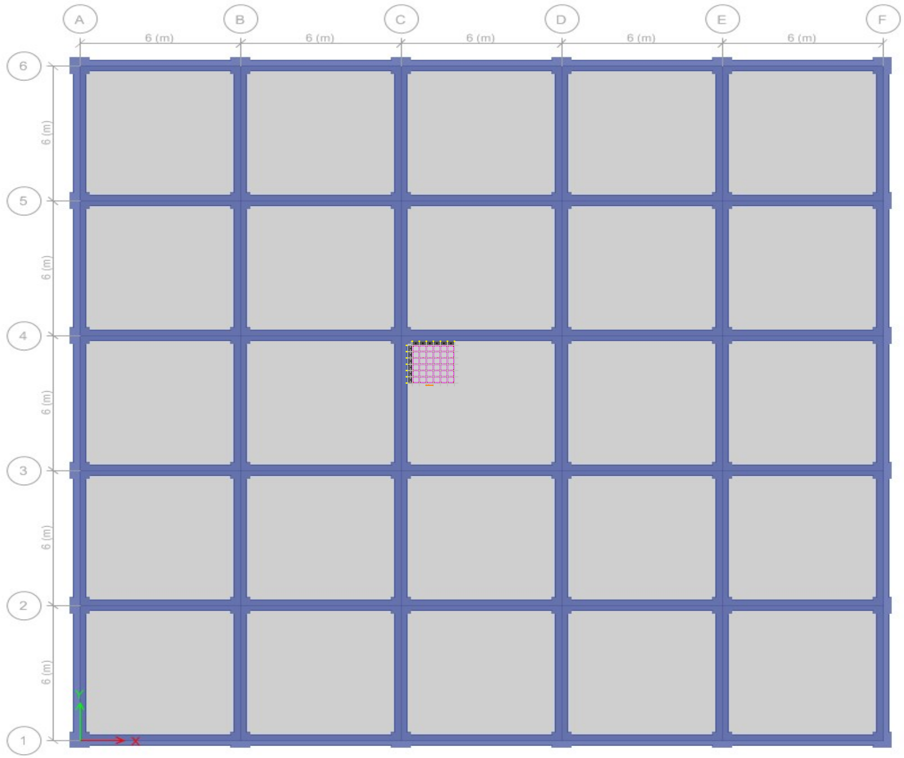


Figure 4.1 Plan at Typical Level of Building (Showing Arrangement of beam and columns)

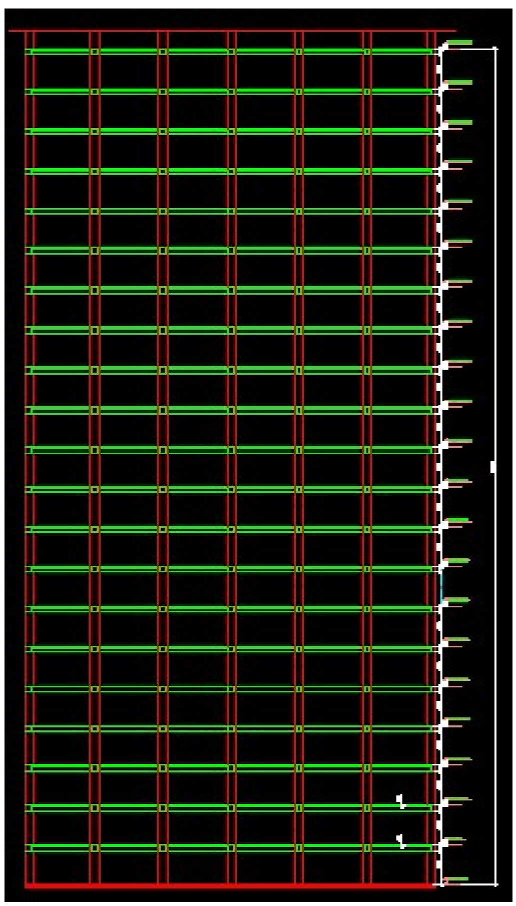


Figure 4.2 Elevation of Building

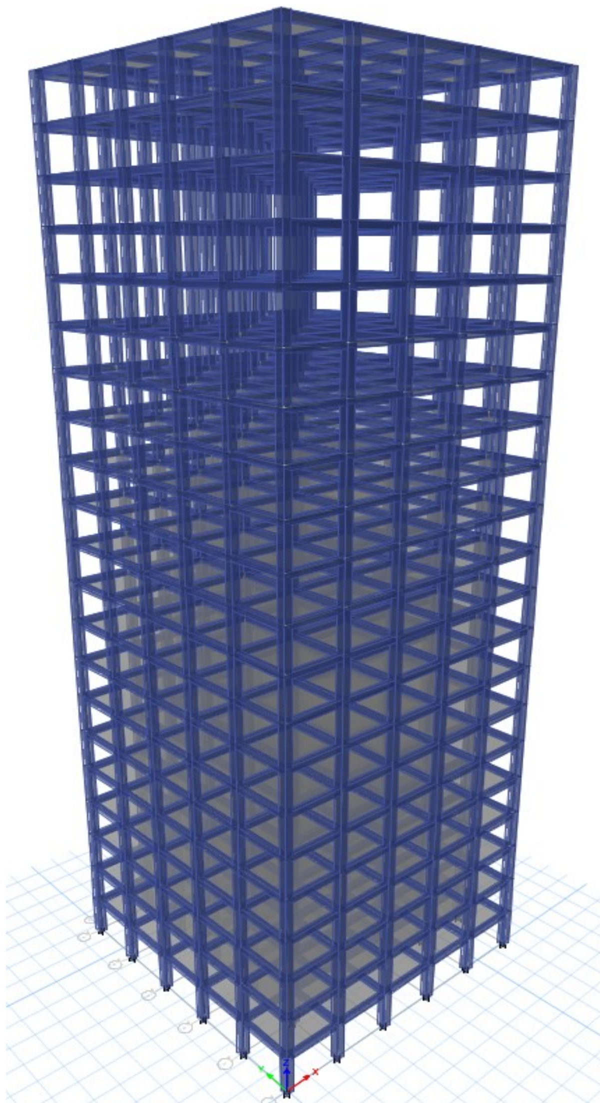


Figure 4.3 Analytical 3D View of Building

<u>(Note: Larger View can be elaborated from the annex-1)</u>

The choice of a regular and relatively simple structure as a first design example was mainly dictated by the need to identify any problems that may arise in applying the proposed procedure, other than those of the complexity of the structure, and obtain a first idea of the relative performance of the procedure in the case of regular frame buildings.

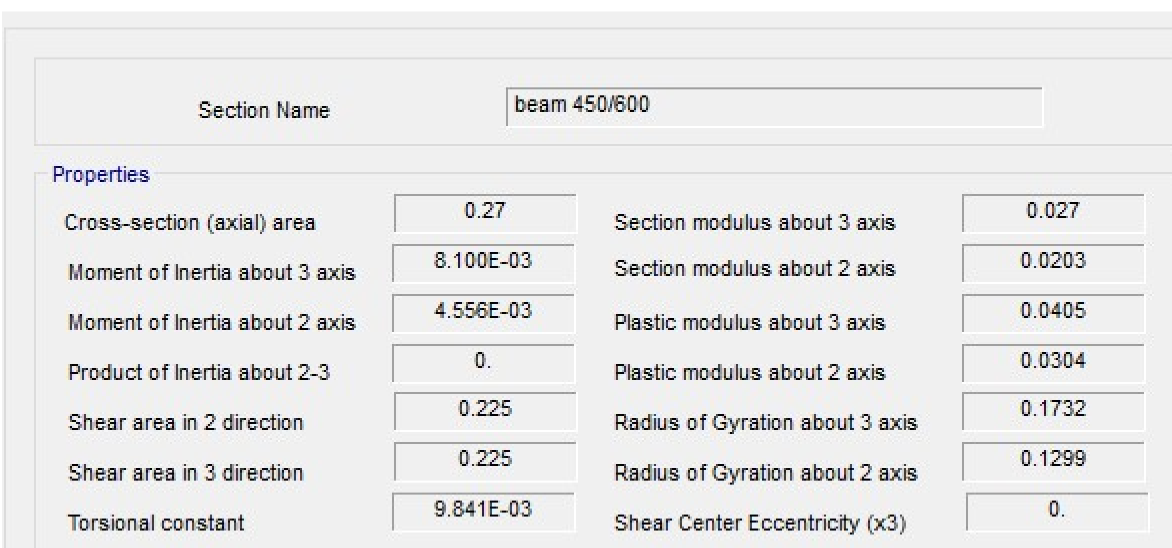
4.4 SECTIONAL PROPERTIES OF ELEMENTS AND ANOTHER RELEVANT DETAILS

Table 4.1. Sectional Properties

Plan type	RC Frame
Level height	4.2 m
Total height	84.0 m

Grade of concretes used	M25 for beams and slabs M35 for columns
Dimension of Beam	600x450 mm
Dimension of Columns	800x800 mm upto 2^{nd} floor and 750*50 up to terrace
Thickness of slab	200 mm
Support conditions	Fixed
Number of beams	1200
Number of 750x750 mm size columns(M40 grade)	612
Number of 800x800 mm size columns(M40 grade)	108
Number of slabs	500

Sectional properties for 450*600 deep beam



Section Name: beam 450/600

Properties

Cross-section (axial) area	0.27	Section modulus about 3 axis	0.027
Moment of Inertia about 3 axis	8.100E-03	Section modulus about 2 axis	0.0203
Moment of Inertia about 2 axis	4.556E-03	Plastic modulus about 3 axis	0.0405
Product of Inertia about 2-3	0.	Plastic modulus about 2 axis	0.0304
Shear area in 2 direction	0.225	Radius of Gyration about 3 axis	0.1732
Shear area in 3 direction	0.225	Radius of Gyration about 2 axis	0.1299
Torsional constant	9.841E-03	Shear Center Eccentricity (x3)	0.

Sectional properties for 800*800 Column

Section Name	C800/800		
Properties			
Cross-section (axial) area	0.64	Section modulus about 3 axis	0.0853
Moment of Inertia about 3 axis	0.0341	Section modulus about 2 axis	0.0853
Moment of Inertia about 2 axis	0.0341	Plastic modulus about 3 axis	0.128
Product of Inertia about 2-3	0.	Plastic modulus about 2 axis	0.128
Shear area in 2 direction	0.5333	Radius of Gyration about 3 axis	0.2309
Shear area in 3 direction	0.5333	Radius of Gyration about 2 axis	0.2309
Torsional constant	0.0577	Shear Center Eccentricity (x3)	0.

Sectional properties for 750*750 Column

Section Name	col750/750		
Properties			
Cross-section (axial) area	0.5625	Section modulus about 3 axis	0.0703
Moment of Inertia about 3 axis	0.0264	Section modulus about 2 axis	0.0703
Moment of Inertia about 2 axis	0.0264	Plastic modulus about 3 axis	0.1055
Product of Inertia about 2-3	0.	Plastic modulus about 2 axis	0.1055
Shear area in 2 direction	0.4688	Radius of Gyration about 3 axis	0.2165
Shear area in 3 direction	0.4688	Radius of Gyration about 2 axis	0.2165
Torsional constant	0.0446	Shear Center Eccentricity (x3)	0.

4.5 LOADS CONSIDERED

The following loads were considered for the analysis of the building. The loads were taken in accordance with IS: 875 Part 1, Part-2 and Part-3

4.5.1 Gravity Loads

The intensity of dead load and live load at various floor levels and roof levels considered in the study are listed below.

- **Dead Load**

Roof Level

Weight of Slab	0.25 x 25	5.00 kN/m^2
Weight of Mud Fuska	0.150 x 24	3.600 kN/m^2
Weight of Tiles	0.040 x 20	0.800 kN/m^2
Total Dead Load		**9.400 kN/m^2**

Floor Levels

Weight of Slab	0.2 x 20	5.00 kN/m^2
Weight of Falce Ceilling	-	1.0 kN/m^2
Weight of Floor Finish	0.050 x 24	1.20kN/m^2
Weight of partition Wall	-	1.000 kN/m^2
Total Dead Load		**9.400kN/m^2**

- **Live Load**

Live load at all floor levels = **4.0 kN/m²**

4.5.2 Seismic Loads

The design lateral force due to earthquake is calculated as follows:

- **Design horizontal seismic coefficient:**

The design horizontal seismic coefficient Ah for a structure shall be determined by the following expressions:-

$$A_h = \frac{Z\,I\,S_a}{2\,R\,g}$$

Provided that for any structure with T≤0.1 sec. the value of Ah will not be less than Z/2 whatever the value of R/I.

Z= Zone factor

I = Importance factor depending upon the functional use of the structure.

R = Response reduction factor, depending upon the perceived seismic damage performance of the structure.

Sa /g = Average response acceleration coefficient for rock or soil sites.

- **Seismic Weight**

The seismic weight of each floor is its full dead load. While computing the seismic weight of each floor, the weight of columns and walls in a level shall be equally distributed to the floors above and below the level. The seismic weight of the whole building is the sum of the seismic weights of all the floors.

- **Design Seismic Base Shear**

The total design lateral force or seismic base shear (V_h) along any principal direction is determined by the following expression:-

$$V_h = A_h W$$

Where W is the seismic weight of the building.

- **Fundamental Natural Time Period**

The approximate fundamental natural time period of vibration (T_s) in seconds of a moment resisting frame building without brick infill panels may be estimated by the following empirical expressions:

$T_S = 0.075\ h^{0.75}$ for RC framed building

$T_S = 0.085\ h^{0.75}$ for steel framed building

For all other buildings, it is given by:- $T_n = 0.09h/\sqrt{d}$

Where h=Height of the building in meters

d= base dimension of the building at the plinth level, in meters, along the considered direction of the lateral force.

- **Distribution of design force**

The design base shear (**V_h**) computed is distributed along the height of the building as below:

$$Q_i = \frac{V_h W_i h_i^2}{\sum W_i h_i^2}$$

Where,

Q_i = design lateral force at each floor level i

W_i = seismic weight of floor i.

i = height of floor i measured from the base.

- **Design lateral force**

The design lateral force shall first be computed for the building as a whole the design lateral force shall then be distributed to the various floor levels. The design seismic force thus obtained

at each floor level, shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

4.6 DETERMINATION OF LATERAL LOADS FOR PUSHOVER ANALYSIS

The maximum design lateral force, **Qi**, was computed for each level level and was distributed at each node. The calculation of this force is illustrated below:

Using IS 1893:2002 Base Shears for the designed building was calculated.
Percentage (%) of imposed load in Seismic Weight calculation was taken as 50 % .

- **Seismic Weight calculation**

  Top Floor:-

  1296*0.2*25+1296*4.4+0.45*6*25*72 + 0 = 19168.65 kN

  Rest Floors :-

  =((1296*0.2*25)+(1296*3)+(0.45*6*25*72)+(3
  6*36*2)+(36*0.75*0.75*4.2*25)+(72*8.28*3.6)
)*20= 441848.05 kN

Seismic Weight of the Building = 461016.7 kN

T_a= 0.075h^0.75, where

„d"= Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

„h"= Height of building in m

In X direction: T_a= 2.1586 s

In Y direction: T_a=2.1586s

Spectral acceleration (s_a/g) is 1.36/T for both of the two fundamental periods (i.e., in X- and Y-directions)

Design base shear = $\mathbf{V_B=A_h w}$

$$A_h = \left(\frac{Z}{2}\right) \times \left(\frac{I}{R}\right) \times \left(\frac{S_a}{g}\right) \left(\frac{0.24}{2}\right) \left(\frac{1}{5}\right)$$

Design base shear = .015 × 462910.7 = 7000.5347
kN

Vertical distribution of Base shear gave the following results

Table 4.2 (Showing Story Shear distribution) All forces are in TON

FLOOR	Wi (TONS)	hi (MTS)	Wi*hi^2	P_FACTOR	Qi (TONS)	NO. OF JOINTS	JOINT FORCE (TONS)	TOTAL FORCE CUMMU.	TOTAL BM AT FLOOR	TOTAL BM CUMMU.
=	=	=	=	=	=	=	=	=	=	
Roof	1916	88.2	14905024	0.1176	82.3	1	82.2869	82.3	0	0
19	2210	84	15593760	0.123	86.1	1	86.0893	168.4	345.6051	345.6
18	2210	79.8	14073368	0.111	77.7	1	77.6956	246.1	707.18	1052.8
17	2210	75.6	12630946	0.0996	69.7	1	69.7323	315.8	1033.501	2086.3
16	2210	71.4	11266492	0.0889	62.2	1	62.1995	378	1326.377	3412.7
15	2210	67.2	9980006	0.0787	55.1	1	55.0971	433.1	1587.615	5000.3
14	2210	63	8771490	0.0692	48.4	1	48.4252	481.5	1819.023	6819.3
13	2210	58.8	7640942	0.0603	42.2	1	42.1837	523.7	2022.409	8841.7
12	2210	54.6	6588364	0.052	36.4	1	36.3727	560.1	2199.58	11041.3
11	2210	50.4	5613754	0.0443	31	1	30.9921	591.1	2352.346	13393.6
10	2210	46.2	4717112	0.0372	26	1	26.042	617.1	2482.513	15876.1
9	2210	42	3898440	0.0307	21.5	1	21.5223	638.6	2591.889	18468
8	2210	37.8	3157736	0.0249	17.4	1	17.4331	656.1	2682.283	21150.3
7	2210	33.6	2495002	0.0197	13.8	1	13.7743	669.8	2755.502	23905.8
6	2210	29.4	1910236	0.0151	10.5	1	10.5459	680.4	2813.354	26719.2
5	2210	25.2	1403438	0.0111	7.7	1	7.748	688.1	2857.647	29576.8
4	2210	21	974610	0.0077	5.4	1	5.3806	693.5	2890.188	32467
3	2210	16.8	623750	0.0049	3.4	1	3.4436	697	2912.787	35379.8
2	2210	12.6	350860	0.0028	1.9	1	1.937	698.9	2927.25	38307
1	2210	8.4	155938	0.0012	0.9	1	0.8609	699.8	2935.385	41242.4

STILT	2210	4.2	38984	0.0003	0.2	1	0.2152	700	2939.001	44181.4
Base	175	0	0	0	0	1	0	700	2939.905	47121.3
=	=	=	=	=	=	=	=	=	=	=
TOTAL	**46291**		1.27E+08	1	**700**				**47121**	
	(W) MT				(Vb) MT				B.M. @ BASE	

This load was applied to the structure for pushover analysis. This load is similar to the inverted triangular loading suggested for pushover analysis by various codes such as ATC-40,FEMA-356

4.6.3 Wind Loads

Wind load is applied to the structure based on IS 875 part 3 :2010

Terrain Cateregory –3, Class 2 Vb-47 m/s

4.6 .4 ASSUMPTIONS FOR PERFORMING THEANALYSIS

1. The material is homogeneous, isotropic and linearly elastic.
2. All columns supports are considered as fixed at the foundation.
3. Tensile strength of concrete is ignored in sections subjected to bending.
4. The super structure is analyzed independently from foundation and soil medium, on the assumptions that foundations are fixed.
5. The floor acts as diaphragms, which are rigid in the horizontal plane.
6. Pushover hinges are assigned to all the member ends. In case of Columns PMM hinges (i.e. Axial Force and Biaxial Moment Hinge) are provided at both the ends, while in case of beams M3 hinges (i.e. Bending Moment hinge) are provided at both the ends.
7. The maximum target displacement of the structure is kept at 2.5% of the height of the building = (2.5/100) X 14= 0.35m = 350mm.

The building is designed by ETABS (according to I.S. 456:2000) for Dead Load and Live load case only for getting the reinforcement detail.

Table 4.3 Structural details (as per Analysis and Design on ETABS for Gravity and earthquake cases)

Element	Dimension	Reinforcement Area in	Reinforcement Area in
		(During Dead and live only)	(Based on IS 1893 and 13920)
		(IS456:2000)	
Corner Columns	0.80 x 0.80	5100	7700
Mid-face Columns	0.80 x 0.80	13200	14200
Interior Column	0.80 x 0.80	22400	24400
Beams 1st to 5th level	0.45 x 0.6	1900 (top) 1200 (bottom)	2500 (top) 1800 (bottom)
Beams 5th to 10th level	0.45 x 0.6	2100 (top) 1800 (bottom)	2700 (top) 2100 (bottom)
Beams 10th to 15th	0.45 x 0.6	2100 (top) 1800 (bottom)	2700 (top) 2100 (bottom)
Beams 15th to top level	0.45 x 0.6	2100 (top) 1800 (bottom)	2700 (top) 2100 (bottom)

4.7 Pushover analysis (Assessments using SAP2000)

Pushover analysis is performed for the above said plan in various steps, including the definition of hinges. Various steps can be listed out as follows..

- Analysis model is created using the SAP model creation option. The plan is choosen as shown in fig 4.1, the number of floors and elevational height is considered as shown in fig 4.2 and 4.3, the gravity loads and another sectional properties has been defined and assigned.
- Pushover hinges and their acceptance criteria are defined. 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-356 for steel members. In this analysis, P-M-M hinges have been defined at both the column ends and M-3 hinges have been defined at both the ends of all the beams.
- Hinges are assigned to the beam for M-3 and for columns P-M-M. (Hinges can be assigned only after the definition of gravity nonlinear load case)

- Define the pushover load cases. Also a Non-linear pushover load case can start from the final conditions of another Non-linear pushover load case that was previously run in the same analysis. Typically the first pushover load case was used to apply gravity load and then subsequent lateral pushover load cases were 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. In this case a Gravity load combination of DL+0.5LL has been used. This combination has been defined as "gravity" (Gravity load case is so important for the assignment for the hinges). The lateral loads, as calculated in 4.6.1, have been applied to a case called PUSX.
- Run the initial static analysis. Then run the static non-linear pushover analysis.
- The Pushover curve made for control nodes at each level level. This was done by defining a number of Non-linear pushover cases in the same analysis, and displacement was monitored for different node/joints in each case.
- The pushover curve was obtained as shown in Fig. 4.4. A table is also obtained which provides the coordinates of each step of the pushover curve and summarizes the number of hinges in each stage (for example, between IO and LS, or between D and E). This table is shown in Table 4.3.

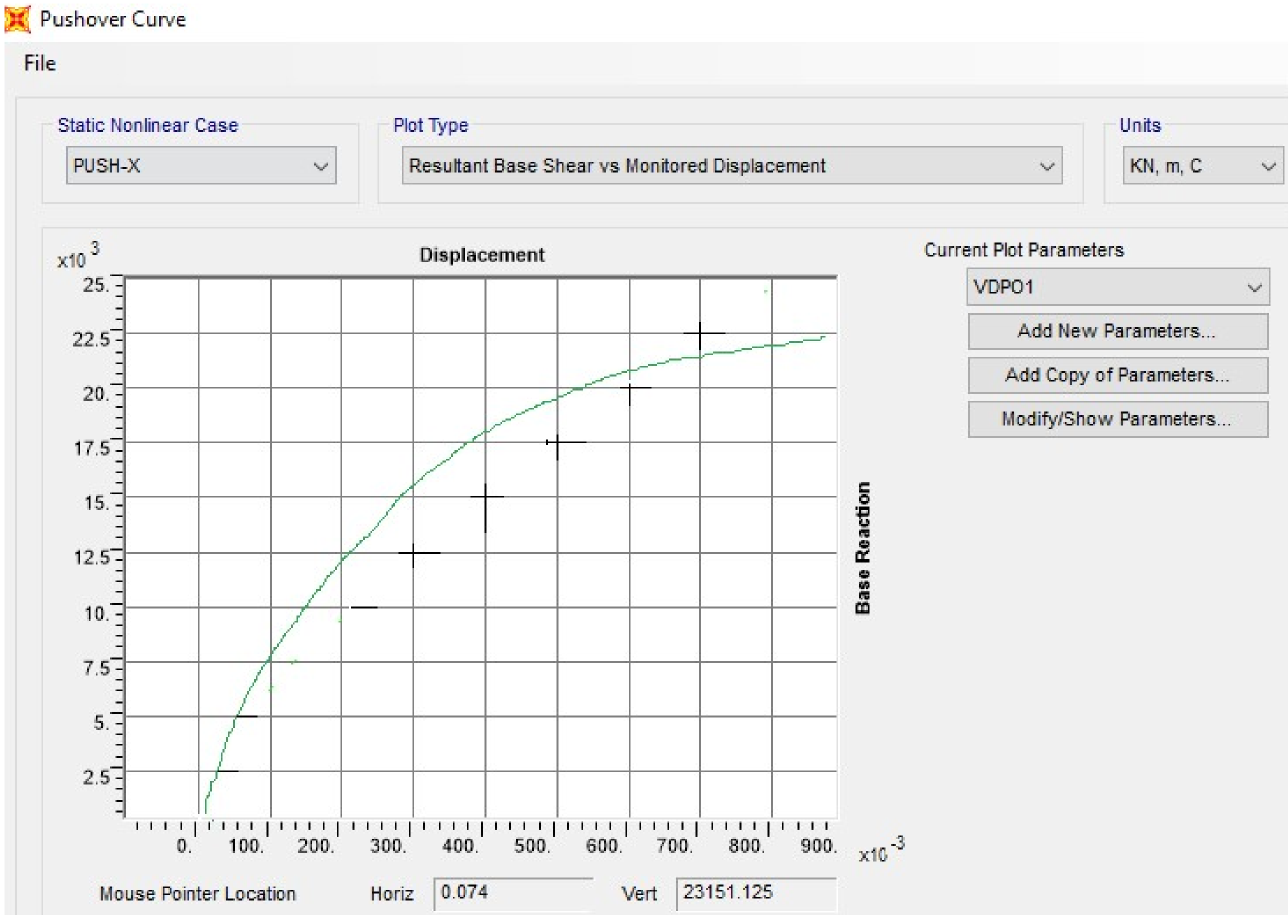


Figure 4.4 Pushover Curve (Base Shear Vs Displacement)

Table 4.4 Pushover Curve tabular Data for base shear and hinges type.

TABLE: Pushover Capacity Curve												
LoadCase	**Step**	**Displacement**	**Base Force**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**Beyond E**	**Total**
Text		m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH-X	0	0	0.0	3840	0	0	0	0	0	0	0	3840
PUSH-X	1	0.008	552.6	3840	0	0	0	0	0	0	0	3840
PUSH-X	2	0.016	1105.2	3840	0	0	0	0	0	0	0	3840
PUSH-X	3	0.024	1657.8	3840	0	0	0	0	0	0	0	3840
PUSH-X	4	0.032	2210.4	3840	0	0	0	0	0	0	0	3840
PUSH-X	5	0.04	2763.0	3840	0	0	0	0	0	0	0	3840
PUSH-X	6	0.048	3315.6	3840	0	0	0	0	0	0	0	3840
PUSH-X	7	0.052	3635.7	3827	13	0	0	0	0	0	0	3840
PUSH-X	8	0.061	4215.0	3800	40	0	0	0	0	0	0	3840
PUSH-X	9	0.069	4771.1	3735	105	0	0	0	0	0	0	3840

LoadCase	Step	Displacement	Base Force	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	Beyond E	Total
Text		m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH-X	10	0.078	5288.3	3657	183	0	0	0	0	0	0	3840
PUSH-X	11	0.087	5700.7	3584	256	0	0	0	0	0	0	3840
PUSH-X	12	0.096	6123.6	3496	344	0	0	0	0	0	0	3840
PUSH-X	13	0.105	6465.2	3426	414	0	0	0	0	0	0	3840
PUSH-X	14	0.113	6784.5	3378	462	0	0	0	0	0	0	3840
PUSH-X	15	0.122	7086.5	3343	497	0	0	0	0	0	0	3840
PUSH-X	16	0.13	7355.6	3300	540	0	0	0	0	0	0	3840
PUSH-X	17	0.139	7637.1	3262	578	0	0	0	0	0	0	3840
PUSH-X	18	0.147	7895.1	3250	590	0	0	0	0	0	0	3840
PUSH-X	19	0.158	8231.9	3218	622	0	0	0	0	0	0	3840
PUSH-X	20	0.167	8507.2	3195	645	0	0	0	0	0	0	3840
PUSH-X	21	0.177	8816.1	3180	660	0	0	0	0	0	0	3840
PUSH-X	22	0.186	9077.2	3164	676	0	0	0	0	0	0	3840
PUSH-X	23	0.198	9437.6	3142	698	0	0	0	0	0	0	3840
PUSH-X	24	0.209	9745.6	3121	719	0	0	0	0	0	0	3840
PUSH-X	25	0.221	10092.1	3106	734	0	0	0	0	0	0	3840
PUSH-X	26	0.23	10324.2	3088	752	0	0	0	0	0	0	3840
PUSH-X	27	0.239	10599.5	3079	761	0	0	0	0	0	0	3840
PUSH-X	28	0.25	10903.0	3060	780	0	0	0	0	0	0	3840
PUSH-X	29	0.259	11136.4	3049	791	0	0	0	0	0	0	3840
PUSH-X	30	0.271	11467.7	3035	805	0	0	0	0	0	0	3840
PUSH-X	31	0.281	11730.8	3027	813	0	0	0	0	0	0	3840
PUSH-X	32	0.289	11967.1	3013	827	0	0	0	0	0	0	3840
PUSH-X	33	0.3	12263.9	3000	840	0	0	0	0	0	0	3840
PUSH-X	34	0.312	12575.9	2989	851	0	0	0	0	0	0	3840
PUSH-X	35	0.32	12789.6	2976	864	0	0	0	0	0	0	3840
PUSH-X	36	0.33	13050.6	2970	870	0	0	0	0	0	0	3840
PUSH-X	37	0.338	13260.7	2964	876	0	0	0	0	0	0	3840
PUSH-X	38	0.348	13530.8	2954	886	0	0	0	0	0	0	3840
PUSH-X	39	0.358	13799.7	2944	896	0	0	0	0	0	0	3840
PUSH-X	40	0.368	14070.3	2941	899	0	0	0	0	0	0	3840
PUSH-X	41	0.382	14432.2	2938	902	0	0	0	0	0	0	3840
PUSH-X	42	0.396	14779.5	2929	911	0	0	0	0	0	0	3840
PUSH-X	43	0.407	15078.9	2921	919	0	0	0	0	0	0	3840
PUSH-X	44	0.415	15284.8	2916	924	0	0	0	0	0	0	3840

LoadCase	Step	Displacement	Base Force	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	Beyond E	Total
Text		m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH-X	45	0.429	15634.1	2902	938	0	0	0	0	0	0	3840
PUSH-X	46	0.441	15956.2	2893	947	0	0	0	0	0	0	3840
PUSH-X	47	0.451	16216.8	2886	954	0	0	0	0	0	0	3840
PUSH-X	48	0.465	16568.6	2881	953	6	0	0	0	0	0	3840
PUSH-X	49	0.478	16903.3	2875	950	15	0	0	0	0	0	3840
PUSH-X	50	0.486	17105.8	2872	935	33	0	0	0	0	0	3840
PUSH-X	51	0.5	17466.9	2867	916	57	0	0	0	0	0	3840
PUSH-X	52	0.508	17668.2	2865	906	69	0	0	0	0	0	3840
PUSH-X	53	0.516	17869.1	2865	899	76	0	0	0	0	0	3840
PUSH-X	54	0.53	18224.0	2858	888	94	0	0	0	0	0	3840
PUSH-X	55	0.538	18424.2	2852	885	103	0	0	0	0	0	3840
PUSH-X	56	0.551	18755.5	2844	847	149	0	0	0	0	0	3840
PUSH-X	57	0.559	18954.0	2842	839	159	0	0	0	0	0	3840
PUSH-X	58	0.571	19256.1	2830	839	171	0	0	0	0	0	3840
PUSH-X	59	0.579	19451.6	2829	829	182	0	0	0	0	0	3840
PUSH-X	60	0.587	19647.1	2828	795	217	0	0	0	0	0	3840
PUSH-X	61	0.595	19842.4	2823	792	225	0	0	0	0	0	3840
PUSH-X	62	0.605	20093.2	2818	769	253	0	0	0	0	0	3840
PUSH-X	63	0.613	20286.0	2814	757	269	0	0	0	0	0	3840
PUSH-X	64	0.621	20478.3	2812	751	277	0	0	0	0	0	3840
PUSH-X	65	0.632	20750.1	2799	758	283	0	0	0	0	0	3840
PUSH-X	66	0.64	20941.9	2799	757	284	0	0	0	0	0	3840
PUSH-X	67	0.648	21133.2	2798	740	302	0	0	0	0	0	3840
PUSH-X	68	0.662	21488.0	2794	717	329	0	0	0	0	0	3840
PUSH-X	69	0.67	21679.4	2792	701	347	0	0	0	0	0	3840
PUSH-X	70	0.678	21870.0	2790	700	350	0	0	0	0	0	3840
PUSH-X	71	0.686	22061.1	2790	692	358	0	0	0	0	0	3840
PUSH-X	72	0.694	22251.7	2788	690	362	0	0	0	0	0	3840
PUSH-X	73	0.702	22442.7	2783	694	363	0	0	0	0	0	3840
PUSH-X	74	0.716	22788.1	2774	695	371	0	0	0	0	0	3840
PUSH-X	75	0.724	22978.1	2771	692	377	0	0	0	0	0	3840
PUSH-X	76	0.732	23166.5	2770	691	379	0	0	0	0	0	3840
PUSH-X	77	0.744	23439.9	2763	676	401	0	0	0	0	0	3840
PUSH-X	78	0.754	23671.6	2753	661	426	0	0	0	0	0	3840

LoadCase	Step	Displacement	Base Force	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	Beyond E	Total
Text		m	KN	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
PUSH-X	79	0.762	23841.3	2748	653	439	0	0	0	0	0	3840
PUSH-X	80	0.77	24001.1	2745	652	443	0	0	0	0	0	3840
PUSH-X	81	0.778	24160.7	2744	647	449	0	0	0	0	0	3840
PUSH-X	82	0.786	24317.0	2742	647	451	0	0	0	0	0	3840
PUSH-X	83	0.792	24450.5	2739	649	452	0	0	0	0	0	3840

- The capacity spectrum curve (ATC-40)so obtained is shown in Fig. 4.5. The performance points for a given set of data's are defined by the intersection of the capacity curve and the single demand spectrum curve. Also, a table is generated which shows 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 (also known as ADRS format). See table 4.3 for complete details in tabular format.

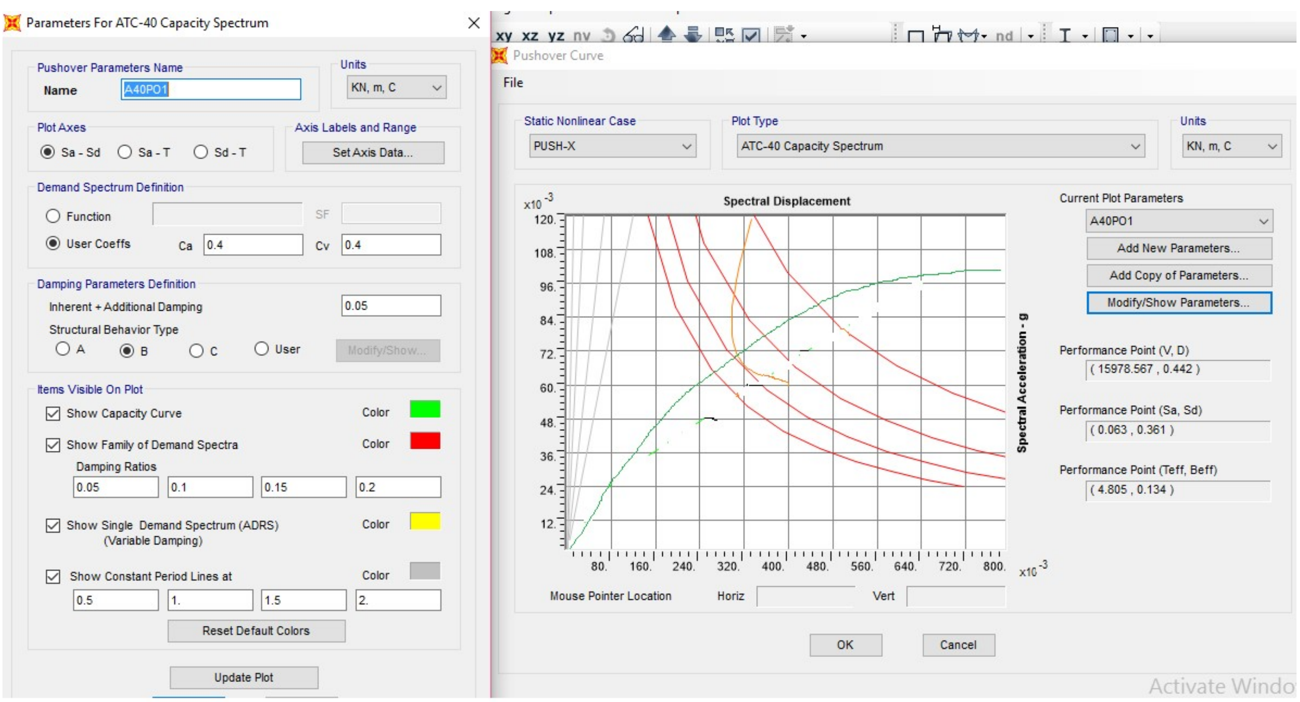


Figure 4.5 Capacity Spectrum Curve (ATC-40)

Table:4.5 Pushover Curve Demand Capacity - ATC40

LoadCase	Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha
		Sec		m		m		
PUSH-X	0	3.364325	0.05	0	0	0.334287	0.118895	1
PUSH-X	1	3.364325	0.05	0.006178	0.002197	0.334287	0.118895	0.809594
PUSH-X	2	3.364325	0.05	0.012356	0.004395	0.334287	0.118895	0.809594
PUSH-X	3	3.364325	0.05	0.018534	0.006592	0.334287	0.118895	0.809594
PUSH-X	4	3.364325	0.05	0.024712	0.008789	0.334287	0.118895	0.809594
PUSH-X	5	3.364329	0.050001	0.03089	0.010986	0.334312	0.118903	0.809593
PUSH-X	6	3.364328	0.05	0.037068	0.013184	0.334313	0.118904	0.809593
PUSH-X	7	3.364328	0.05	0.040646	0.014456	0.334313	0.118904	0.809593
PUSH-X	8	3.372821	0.051852	0.047349	0.016756	0.332128	0.117533	0.809803
PUSH-X	9	3.393336	0.055982	0.054185	0.018944	0.327727	0.114577	0.810766
PUSH-X	10	3.439227	0.065006	0.061511	0.020935	0.319469	0.108729	0.813184
PUSH-X	11	3.494078	0.074973	0.068204	0.02249	0.312258	0.102965	0.815993
PUSH-X	12	3.563104	0.086266	0.07591	0.02407	0.306084	0.097056	0.818974
PUSH-X	13	3.632428	0.096828	0.083037	0.025335	0.301681	0.092043	0.821509
PUSH-X	14	3.701845	0.106286	0.090284	0.026523	0.298928	0.087815	0.823467
PUSH-X	15	3.767521	0.11411	0.097524	0.027659	0.297624	0.08441	0.824775
PUSH-X	16	3.824627	0.120061	0.104211	0.02868	0.297334	0.081829	0.825639
PUSH-X	17	3.883959	0.125607	0.111483	0.029751	0.297617	0.079423	0.826374
PUSH-X	18	3.936673	0.129919	0.118341	0.030741	0.298376	0.077507	0.826772
PUSH-X	19	4.001191	0.134316	0.127418	0.03204	0.299978	0.075431	0.827085
PUSH-X	20	4.051285	0.137167	0.134989	0.033109	0.301632	0.073983	0.827139
PUSH-X	21	4.105054	0.139748	0.143648	0.034316	0.303746	0.072563	0.827026
PUSH-X	22	4.147773	0.141386	0.151025	0.035339	0.305714	0.071536	0.826874
PUSH-X	23	4.20344	0.14305	0.161324	0.036756	0.308602	0.070312	0.826566
PUSH-X	24	4.248419	0.144053	0.170245	0.037972	0.311172	0.069404	0.826213
PUSH-X	25	4.296096	0.144768	0.180375	0.039343	0.314138	0.068519	0.825762
PUSH-X	26	4.32627	0.145026	0.187195	0.040263	0.316155	0.068	0.825453
PUSH-X	27	4.360532	0.145163	0.195336	0.041356	0.318557	0.067445	0.825064
PUSH-X	28	4.396542	0.145141	0.204371	0.042563	0.321204	0.066896	0.824619
PUSH-X	29	4.423082	0.145024	0.211365	0.043493	0.323231	0.066512	0.824262
PUSH-X	30	4.45884	0.144693	0.221318	0.044814	0.326096	0.06603	0.823769
PUSH-X	31	4.485876	0.14433	0.229261	0.045864	0.328351	0.065688	0.823375
PUSH-X	32	4.509108	0.143934	0.236405	0.046807	0.330357	0.06541	0.823031
PUSH-X	33	4.537305	0.143396	0.245441	0.047994	0.332843	0.065085	0.822586
PUSH-X	34	4.565522	0.142753	0.254968	0.049243	0.33542	0.064781	0.822126
PUSH-X	35	4.584035	0.142272	0.261504	0.050098	0.337162	0.064592	0.821821

LoadCase	Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha
		Sec		m		m		
PUSH-X	36	4.605864	0.141654	0.269511	0.051144	0.339262	0.06438	0.821451
PUSH-X	37	4.622919	0.141153	0.275973	0.051984	0.340923	0.064219	0.821177
PUSH-X	38	4.644158	0.140495	0.284305	0.053065	0.343025	0.064025	0.820838
PUSH-X	39	4.66463	0.139833	0.292637	0.054142	0.345081	0.063845	0.820502
PUSH-X	40	4.684479	0.139148	0.301044	0.055226	0.347117	0.063678	0.820167
PUSH-X	41	4.709779	0.138194	0.312293	0.056676	0.349792	0.063482	0.819743
PUSH-X	42	4.732826	0.137249	0.323098	0.058067	0.352305	0.063316	0.819355
PUSH-X	43	4.751887	0.13643	0.332437	0.059267	0.354427	0.063188	0.819027
PUSH-X	44	4.764536	0.135858	0.338862	0.060093	0.355864	0.063108	0.818808
PUSH-X	45	4.785247	0.134886	0.349781	0.061493	0.358259	0.062984	0.818445
PUSH-X	46	4.803549	0.133988	0.359867	0.062785	0.360422	0.062882	0.818118
PUSH-X	47	4.81782	0.133262	0.368038	0.063831	0.362139	0.062808	0.817861
PUSH-X	48	4.836362	0.132284	0.379076	0.065242	0.364412	0.062718	0.817524
PUSH-X	49	4.853514	0.131396	0.389617	0.066583	0.366512	0.062635	0.817242
PUSH-X	50	4.863611	0.130867	0.396007	0.067395	0.367758	0.062587	0.817077
PUSH-X	51	4.881163	0.129949	0.407428	0.06884	0.369934	0.062505	0.816798
PUSH-X	52	4.890719	0.129451	0.413816	0.069647	0.371122	0.062461	0.816652
PUSH-X	53	4.900172	0.128979	0.420201	0.070449	0.372282	0.062415	0.816531
PUSH-X	54	4.916416	0.128154	0.431501	0.071866	0.374294	0.062338	0.816328
PUSH-X	55	4.925344	0.127697	0.437884	0.072665	0.375409	0.062297	0.816219
PUSH-X	56	4.939806	0.126957	0.448474	0.073987	0.377219	0.062232	0.816046
PUSH-X	57	4.948445	0.126548	0.454855	0.074778	0.378273	0.062188	0.815962
PUSH-X	58	4.961348	0.125943	0.464577	0.07598	0.379847	0.062122	0.815859
PUSH-X	59	4.969991	0.125623	0.470963	0.076756	0.38082	0.062065	0.815802
PUSH-X	60	4.978544	0.125323	0.47733	0.077527	0.38177	0.062006	0.81581
PUSH-X	61	4.986845	0.124998	0.483715	0.078303	0.382726	0.061955	0.815756
PUSH-X	62	4.997957	0.124691	0.491994	0.079289	0.383882	0.061866	0.815792
PUSH-X	63	5.006396	0.124465	0.498361	0.080045	0.384755	0.061798	0.815844
PUSH-X	64	5.014618	0.12422	0.504735	0.080803	0.385631	0.061736	0.815852
PUSH-X	65	5.026114	0.123893	0.513746	0.08187	0.386842	0.061646	0.815907
PUSH-X	66	5.033997	0.123646	0.520106	0.082624	0.387696	0.061589	0.815932
PUSH-X	67	5.041799	0.123409	0.526467	0.083376	0.388537	0.061532	0.815962
PUSH-X	68	5.055898	0.122956	0.538269	0.08477	0.390082	0.061433	0.816016
PUSH-X	69	5.063258	0.122699	0.544628	0.085522	0.390912	0.061384	0.816041
PUSH-X	70	5.070573	0.122454	0.550992	0.086272	0.391726	0.061335	0.816058
PUSH-X	71	5.077687	0.122196	0.557343	0.087022	0.39254	0.06129	0.816097
PUSH-X	72	5.084727	0.121942	0.563707	0.087772	0.393346	0.061246	0.816113
PUSH-X	73	5.091589	0.121678	0.570055	0.088522	0.394149	0.061206	0.816151

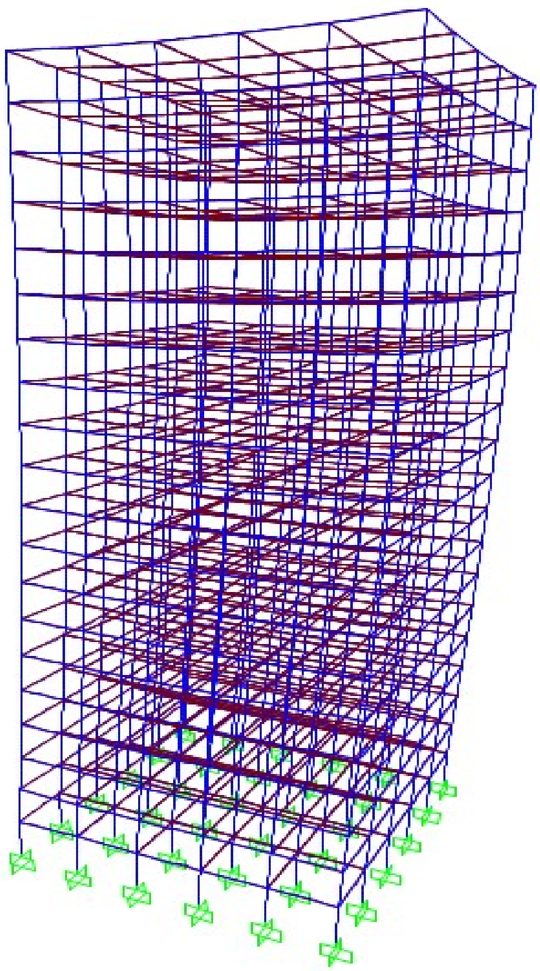
LoadCase	Step	Teff	Beff	SdCapacity	SaCapacity	SdDemand	SaDemand	Alpha
		Sec		m		m		
PUSH-X	74	5.103829	0.121211	0.581579	0.089878	0.395581	0.061134	0.816201
PUSH-X	75	5.110424	0.120951	0.587936	0.090627	0.396363	0.061097	0.81621
PUSH-X	76	5.117195	0.120749	0.594278	0.091362	0.3971	0.061048	0.81628
PUSH-X	77	5.127191	0.1205	0.603565	0.092428	0.398137	0.060969	0.816386
PUSH-X	78	5.136878	0.120494	0.61168	0.093318	0.398895	0.060855	0.816597
PUSH-X	79	5.146215	0.120831	0.618076	0.093952	0.399265	0.060691	0.816898
PUSH-X	80	5.157056	0.121467	0.624447	0.094522	0.399438	0.060462	0.817416
PUSH-X	81	5.167808	0.122084	0.630832	0.095091	0.399625	0.060239	0.817924
PUSH-X	82	5.178853	0.12276	0.637202	0.095642	0.399773	0.060005	0.818475
PUSH-X	83	5.186471	0.123058	0.642341	0.09613	0.40005	0.05987	0.818788

- The pushover displaced shape and sequence of hinge information on a step-by-step basis was obtained and is shown in the Figure 4.6(a) to 4.6(e).
- 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.

Figure 4.6(a) to 4.6(d).

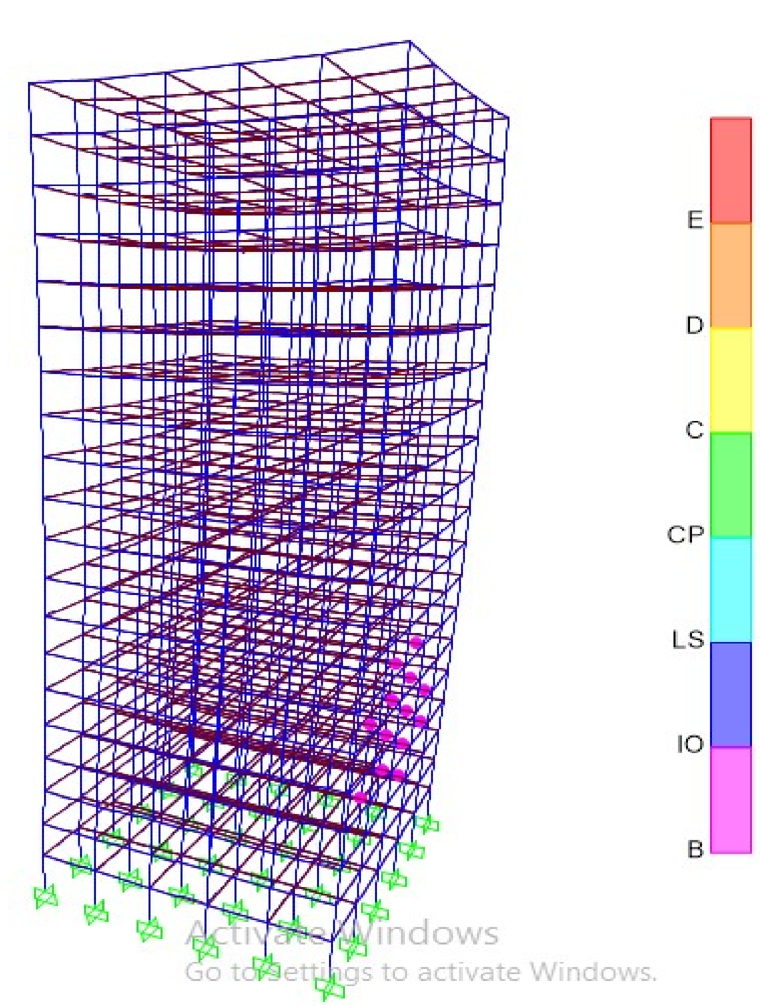
Step 1 to 6

Fig-4.6(a)



Step 7 (Hinges at selected location of IO level)

Fig-4.6(b)



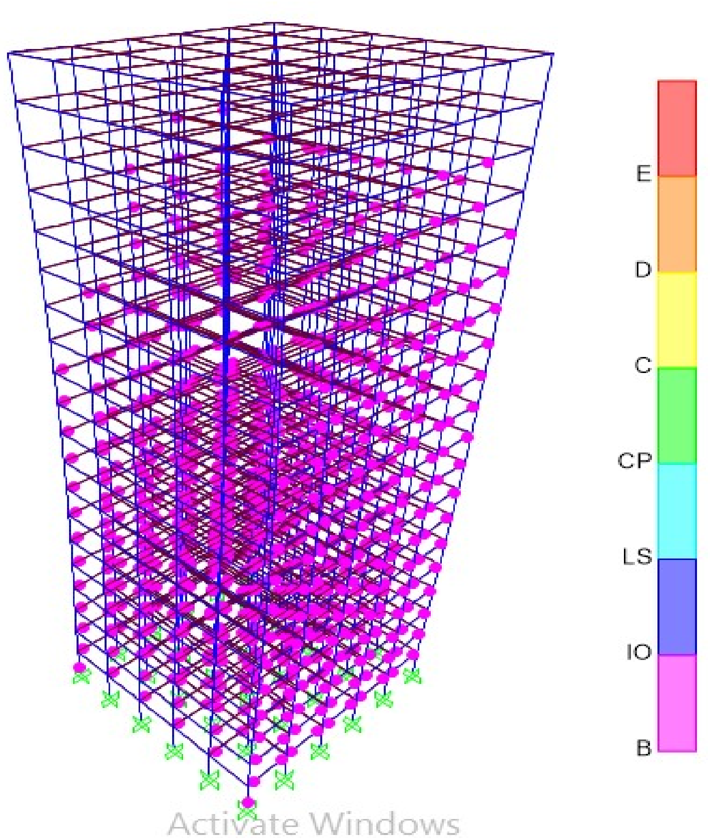


Fig-4.6(c) **Step 50 (All hinges are of B level)**

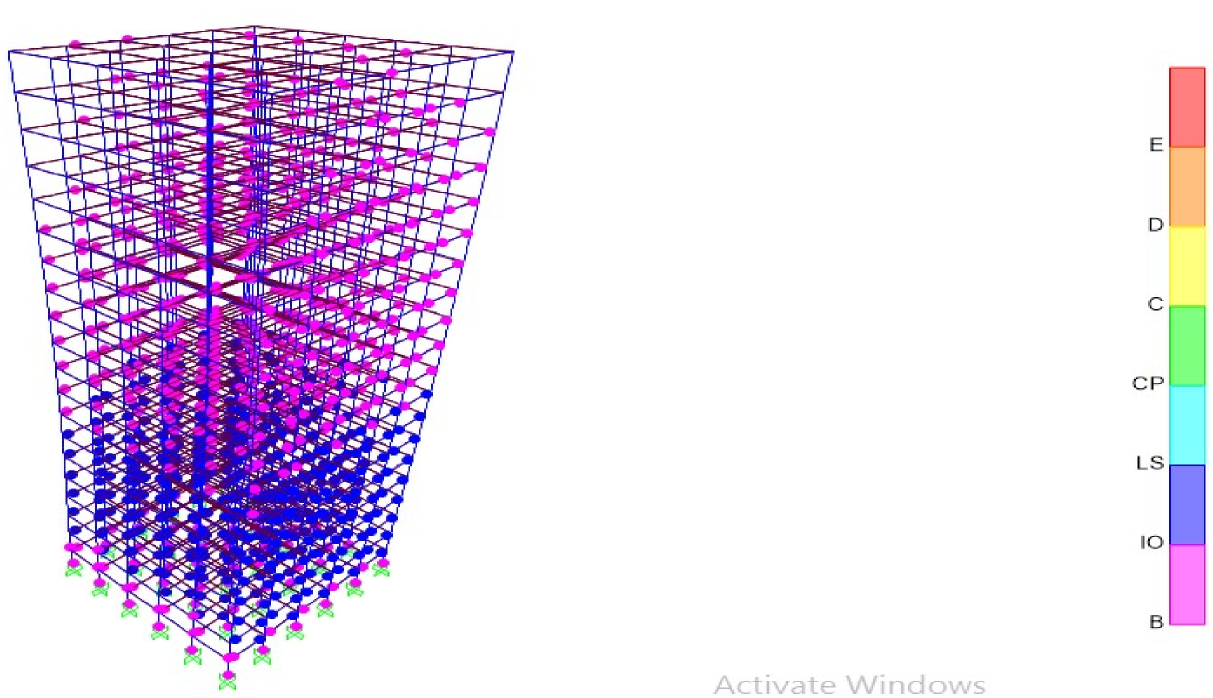


Fig-4.6(d) **Step 83 (All hinges are in between IO and LS level)**
Note: For detailed hinge results refer Annex-2

4.8 CASES INCLUDED IN STUDY

To study the effect of change of main reinforcement on the performance of the structure, various cases are made.

Cases are made to identify the location of formation of hinges during pushover analysis and its design requirement based on pushover analysis.

The thesis has been divided into two segment.

4.8.1 SEGMENT - 1

This segment is meant for the determination of the locations and numbers of hinges, in comparison of the actual equivalent static analysis.
Predetermined sizes has been dealt for the reinforcement (requirement of reinforcement has been calculated based on the ETABS analysis) as per the detailing code of SP-34 and IS 13920 of BIS standards. the plan and relevant details has been shown in fig. 4.8(a) to 4.8(e)

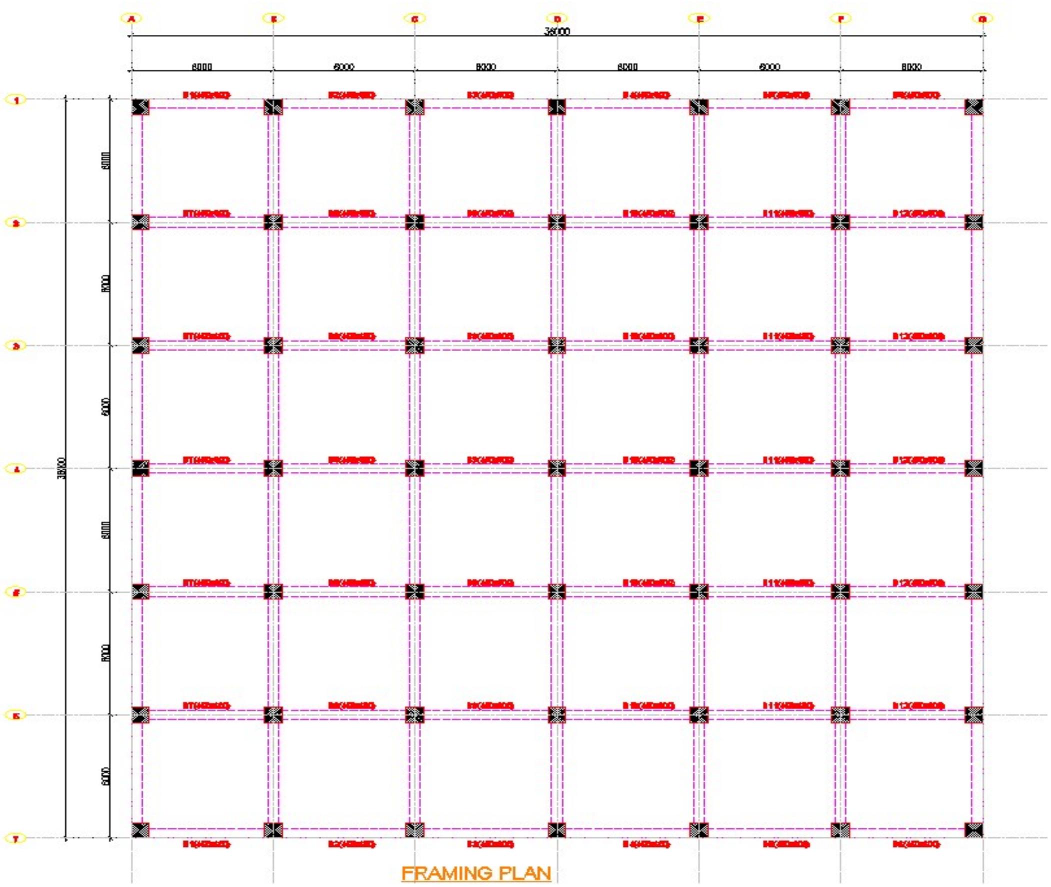


Fig-4.8(a) - **Arrangement Plan of structure (Showing sizes of beams and locations)**

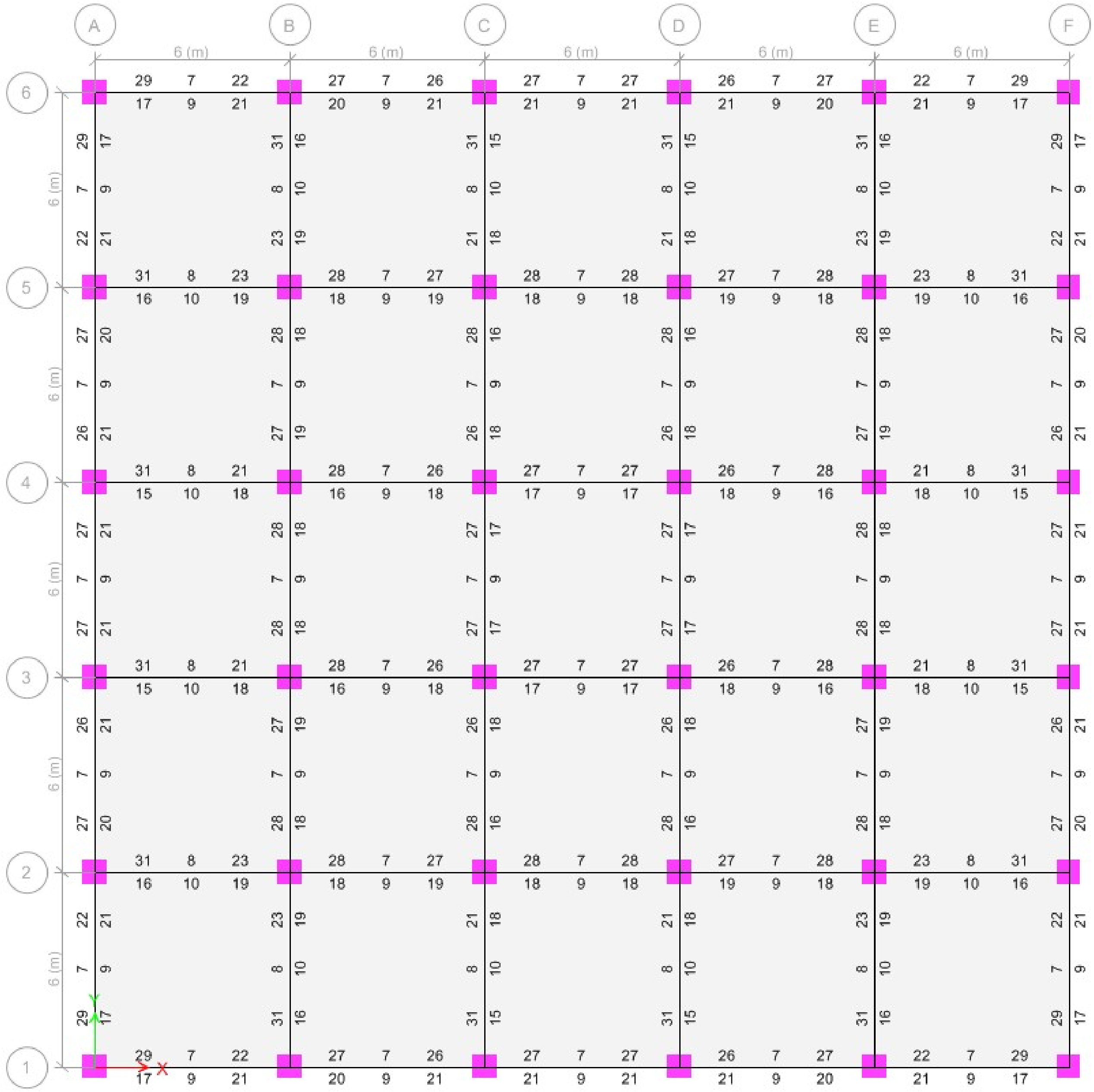


Fig-4.8(b) **- Requirement of reinforcement from analysis pakage (ETABS)**

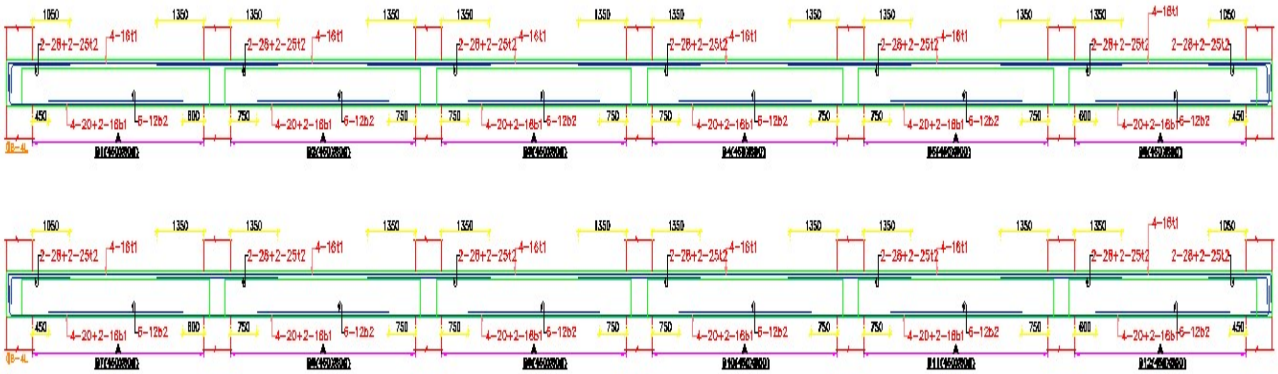


Fig-4.8(c) **- Details of Reinforcement based on design requirement (linear static analysis)**

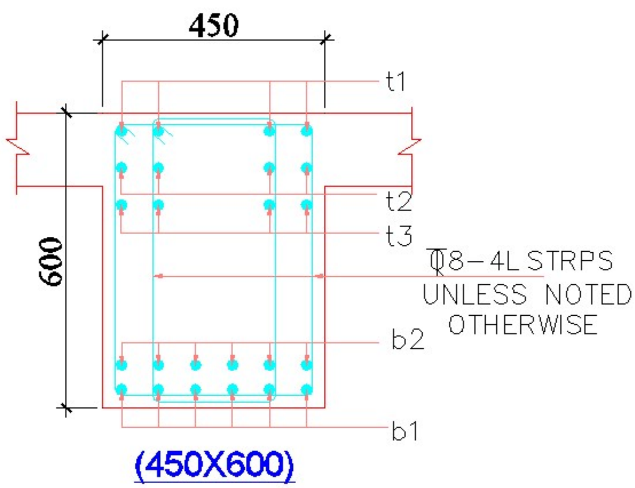


Fig-4.8(d) **- Typical cross section of beam showing location of bars and their notation**

Column has been marked as:-

C1 - All corner columns

C2- Apart from corner column and interior columns

C3 - All interior columns

Details of column is shown below

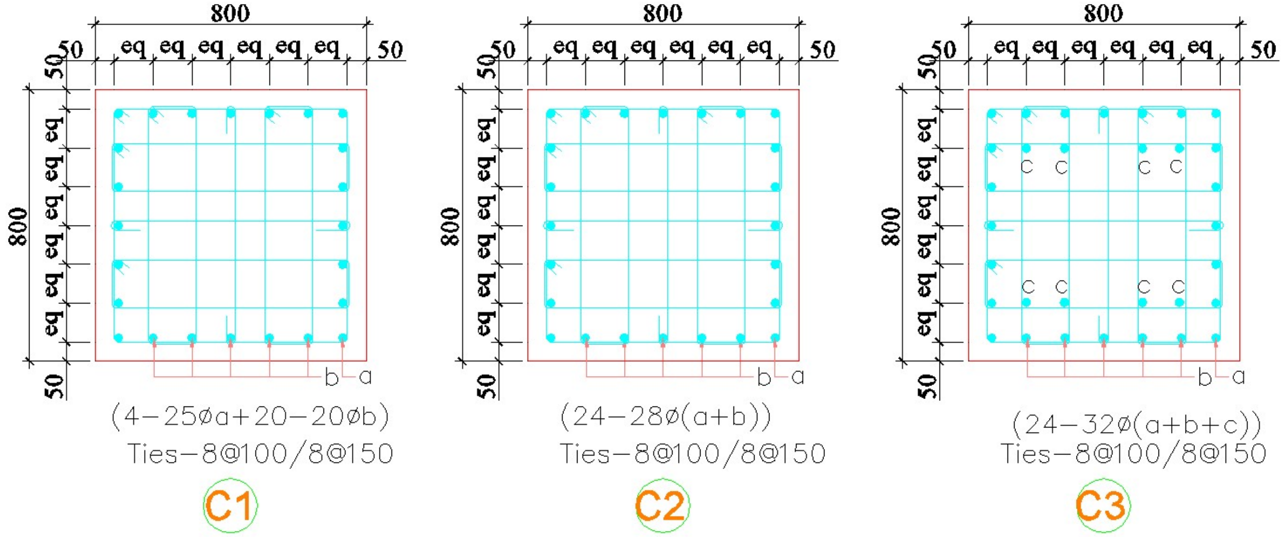


Fig-4.8(e) - **Typical details of columns at base level showing location of bars and their notation along with reinforcement details.**

4.8.1 .1 PUSHOVER analysis with above based inputs

Finally pushover analysis has been conducted using above reinforcement credentials, to assess the nonlinear behavior of structure and structural element.

Analysis is carried out to assess the formation of hinges, the steps has been followed as mentioned in section 4.7.

After analysis the list has been concluded for various stages of pushover analysis case, the list has been tabulated below in consecutive tables.

Sample list of **hinges at A to IO level** for P-M-M hinges

Table 4.6 :- Hinges at A to IO level for P-M-M hinge

TABLE: Frame Hinge States										
Frame	Output	Case	Step	Assign	Gen	P	M2	M3	Hinge	Hinge
	Case	Type	Type	Hinge	Hinge	KN	KN-m	KN-m	State	Status
2	PUSH-X	NonStatic	Max	Auto P-M2-M3	2H1	-5778.4	21.9	3627.78	B to C	A to IO
2	PUSH-X	NonStatic	Max	Auto P-M2-M3	2H2	-5711.2	-24.8	1042.21	A to B	A to IO
2	PUSH-X	NonStatic	Min	Auto P-M2-M3	2H1	-9987.5	-65	14.73	A to B	A to IO

Frame	Output Case	Case Type	Step Type	Assign Hinge	Gen Hinge	P	M2	M3	Hinge State	Hinge Status
						KN	KN-m	KN-m		
2	PUSH-X	NonStatic	Min	Auto P-M2-M3	2H2	-11009.8	-48.4	-31.73	A to B	A to IO
3	PUSH-X	NonStatic	Max	Auto P-M2-M3	3H1	-7504.6	20.2	3709.83	B to C	A to IO
3	PUSH-X	NonStatic	Max	Auto P-M2-M3	3H2	-7437.4	-42.6	776.8	A to B	A to IO
3	PUSH-X	NonStatic	Min	Auto P-M2-M3	3H1	-8110.4	19.8	2.16	A to B	A to IO
3	PUSH-X	NonStatic	Min	Auto P-M2-M3	3H2	-8043.2	-43.4	-66.67	A to B	A to IO
4	PUSH-X	NonStatic	Max	Auto P-M2-M3	4H1	-7874.4	20.1	3708.82	B to C	A to IO
4	PUSH-X	NonStatic	Max	Auto P-M2-M3	4H2	-7807.2	-43	779.55	A to B	A to IO
4	PUSH-X	NonStatic	Min	Auto P-M2-M3	4H1	-8030.3	20	0.42	A to B	A to IO
4	PUSH-X	NonStatic	Min	Auto P-M2-M3	4H2	-7963.1	-43.2	-61.74	A to B	A to IO
5	PUSH-X	NonStatic	Max	Auto P-M2-M3	5H1	-7857.7	20.1	3709.81	B to C	A to IO
5	PUSH-X	NonStatic	Max	Auto P-M2-M3	5H2	-7790.5	-43	778.43	A to B	A to IO
5	PUSH-X	NonStatic	Min	Auto P-M2-M3	5H1	-7940.4	20	-0.42	A to B	A to IO
5	PUSH-X	NonStatic	Min	Auto P-M2-M3	5H2	-7873.2	-43.3	-59.78	A to B	A to IO
6	PUSH-X	NonStatic	Max	Auto P-M2-M3	6H1	-7236.6	75.8	3685.61	B to C	A to IO
6	PUSH-X	NonStatic	Max	Auto P-M2-M3	6H2	-7213.8	-32.5	782.87	A to B	A to IO
6	PUSH-X	NonStatic	Min	Auto P-M2-M3	6H1	-7504.6	-18.1	-2.16	A to B	A to IO
6	PUSH-X	NonStatic	Min	Auto P-M2-M3	6H2	-8389.9	-56.4	-57.15	A to B	A to IO
7	PUSH-X	NonStatic	Max	Auto P-M2-M3	7H2	-1178.9	-2.9	1229.09	A to B	A to IO
7	PUSH-X	NonStatic	Min	Auto P-M2-M3	7H1	-5778.4	-42.8	-14.73	A to B	A to IO
7	PUSH-X	NonStatic	Min	Auto P-M2-M3	7H2	-5711.2	-31.7	31.73	A to B	A to IO
8	PUSH-X	NonStatic	Max	Auto P-M2-M3	8H1	-7504.6	3.9	3354.14	B to C	A to IO
8	PUSH-X	NonStatic	Max	Auto P-M2-M3	8H2	-7437.4	-4.7	898.91	A to B	A to IO
8	PUSH-X	NonStatic	Min	Auto P-M2-M3	8H1	-13557.5	2.2	19.81	A to B	A to IO

Frame	Output	Case	Step	Assign	Gen	P	M2	M3	Hinge	Hinge
	Case	Type	Type	Hinge	Hinge	KN	KN-m	KN-m	State	Status
8	PUSH-X	NonStatic	Min	Auto P-M2-M3	8H2	-13755.1	-8.4	-49.42	A to B	A to IO
9	PUSH-X	NonStatic	Max	Auto P-M2-M3	9H1	-9808.6	3.6	3597.95	B to C	A to IO
9	PUSH-X	NonStatic	Max	Auto P-M2-M3	9H2	-9741.4	-7	478.24	A to B	A to IO
9	PUSH-X	NonStatic	Min	Auto P-M2-M3	9H1	-10553.4	3.3	3.28	A to B	A to IO
9	PUSH-X	NonStatic	Min	Auto P-M2-M3	9H2	-10486.2	-7.8	-91.58	A to B	A to IO
10	PUSH-X	NonStatic	Max	Auto P-M2-M3	10H1	-10272.2	3.5	3596.16	B to C	A to IO
10	PUSH-X	NonStatic	Max	Auto P-M2-M3	10H2	-10205	-7.5	483.23	A to B	A to IO
10	PUSH-X	NonStatic	Min	Auto P-M2-M3	10H1	-10456.5	3.5	0.59	A to B	A to IO
10	PUSH-X	NonStatic	Min	Auto P-M2-M3	10H2	-10389.3	-7.6	-86.95	A to B	A to IO
11	PUSH-X	NonStatic	Max	Auto P-M2-M3	11H1	-10254.5	3.5	3597.53	B to C	A to IO
11	PUSH-X	NonStatic	Max	Auto P-M2-M3	11H2	-10187.3	-7.1	481.86	A to B	A to IO
11	PUSH-X	NonStatic	Min	Auto P-M2-M3	11H1	-10334.9	3.3	-0.59	A to B	A to IO
11	PUSH-X	NonStatic	Min	Auto P-M2-M3	11H2	-10267.7	-7.5	-86.29	A to B	A to IO

Table 4.7 :- Hinges at IO to LS level for P-M-M hinge

TABLE: Frame Hinge States										
Frame	Output	CaseType	Step	Assign	Gen	P	M2	M3	Hinge	Hinge
	Case	Type	Type	Hinge	Hinge	KN	KN-m	KN-m	Status	Status
21	PUSH-X	NonStatic	Max	Auto P-M2-M3	21H1	-10272.2	-0.59	3548.21	B to C	IO to LS
22	PUSH-X	NonStatic	Max	Auto P-M2-M3	22H1	-10770.5	-0.64	3546.13	B to C	IO to LS
23	PUSH-X	NonStatic	Max	Auto P-M2-M3	23H1	-10752.4	-0.33	3547.42	B to C	IO to LS
24	PUSH-X	NonStatic	Max	Auto P-M2-M3	24H1	-9904.09	3.71	3634.47	B to C	IO to LS
27	PUSH-X	NonStatic	Max	Auto P-M2-M3	27H1	-9808.63	-3.28	3597.95	B to C	IO to LS
28	PUSH-X	NonStatic	Max	Auto P-M2-M3	28H1	-10272.2	-3.48	3596.16	B to C	IO to LS
29	PUSH-X	NonStatic	Max	Auto P-M2-M3	29H1	-10254.5	-3.32	3597.53	B to C	IO to LS

Table 4.6 :- Hinges at A to IO level for M-3 hinge

TABLE: Frame Hinge States								
Frame	OutputCase	CaseType	StepType	AssignHinge	GenHinge	M3	HingeState	HingeStatus
Text	Text	Text	Text	Text	Text	KN-m	Text	Text
1	PUSH-X	NonStatic	Max	Auto M3	1H1	-120.018	A to B	A to IO
1	PUSH-X	NonStatic	Max	Auto M3	1H2	34.2879	A to B	A to IO
1	PUSH-X	NonStatic	Min	Auto M3	1H1	-168.309	A to B	A to IO
1	PUSH-X	NonStatic	Min	Auto M3	1H2	-13.6922	A to B	A to IO
38	PUSH-X	NonStatic	Max	Auto M3	38H1	-79.7223	A to B	A to IO
38	PUSH-X	NonStatic	Max	Auto M3	38H2	-40.4426	A to B	A to IO
38	PUSH-X	NonStatic	Min	Auto M3	38H1	-90.8212	A to B	A to IO
38	PUSH-X	NonStatic	Min	Auto M3	38H2	-53.2465	A to B	A to IO
39	PUSH-X	NonStatic	Max	Auto M3	39H1	-66.5545	A to B	A to IO
39	PUSH-X	NonStatic	Max	Auto M3	39H2	-66.5545	A to B	A to IO
39	PUSH-X	NonStatic	Min	Auto M3	39H1	-67.0215	A to B	A to IO
39	PUSH-X	NonStatic	Min	Auto M3	39H2	-67.0215	A to B	A to IO
40	PUSH-X	NonStatic	Max	Auto M3	40H1	-40.4426	A to B	A to IO
40	PUSH-X	NonStatic	Max	Auto M3	40H2	-79.7223	A to B	A to IO
40	PUSH-X	NonStatic	Min	Auto M3	40H1	-53.2466	A to B	A to IO
40	PUSH-X	NonStatic	Min	Auto M3	40H2	-90.8212	A to B	A to IO
41	PUSH-X	NonStatic	Max	Auto M3	41H1	34.2879	A to B	A to IO
41	PUSH-X	NonStatic	Max	Auto M3	41H2	-120.018	A to B	A to IO
41	PUSH-X	NonStatic	Min	Auto M3	41H1	-13.6922	A to B	A to IO
41	PUSH-X	NonStatic	Min	Auto M3	41H2	-168.309	A to B	A to IO
42	PUSH-X	NonStatic	Max	Auto M3	42H1	-201.701	A to B	A to IO
42	PUSH-X	NonStatic	Max	Auto M3	42H2	34.4294	A to B	A to IO
42	PUSH-X	NonStatic	Min	Auto M3	42H1	-213.234	A to B	A to IO
42	PUSH-X	NonStatic	Min	Auto M3	42H2	23.0629	A to B	A to IO
43	PUSH-X	NonStatic	Max	Auto M3	43H1	-109.619	A to B	A to IO
43	PUSH-X	NonStatic	Max	Auto M3	43H2	-57.7797	A to B	A to IO
43	PUSH-X	NonStatic	Min	Auto M3	43H1	-116.79	A to B	A to IO
43	PUSH-X	NonStatic	Min	Auto M3	43H2	-65.5259	A to B	A to IO
44	PUSH-X	NonStatic	Max	Auto M3	44H1	-88.6399	A to B	A to IO
44	PUSH-X	NonStatic	Max	Auto M3	44H2	-88.6399	A to B	A to IO
44	PUSH-X	NonStatic	Min	Auto M3	44H1	-88.9824	A to B	A to IO
44	PUSH-X	NonStatic	Min	Auto M3	44H2	-88.9824	A to B	A to IO
45	PUSH-X	NonStatic	Max	Auto M3	45H1	-57.7797	A to B	A to IO
45	PUSH-X	NonStatic	Max	Auto M3	45H2	-109.619	A to B	A to IO

45	PUSH-X	NonStatic	Min	Auto M3	45H1	-65.5259	A to B	A to IO
45	PUSH-X	NonStatic	Min	Auto M3	45H2	-116.79	A to B	A to IO
46	PUSH-X	NonStatic	Max	Auto M3	46H1	34.4294	A to B	A to IO

Table 4.9 :- Hinges at IO to LS level for M-3 hinge

Frame	OutputCase	CaseType	StepType	AssignHinge	GenHinge	M3	HingeState	HingeStatus
Text	Text	Text	Text	Text	Text	KN-m	Text	Text
128	PUSH-X	NonStatic	Max	Auto M3	128H1	250.721	B to C	IO to LS
129	PUSH-X	NonStatic	Max	Auto M3	129H1	251.8298	B to C	IO to LS
130	PUSH-X	NonStatic	Max	Auto M3	130H1	251.9215	B to C	IO to LS
131	PUSH-X	NonStatic	Max	Auto M3	131H1	249.9213	B to C	IO to LS
134	PUSH-X	NonStatic	Max	Auto M3	134H1	251.1121	B to C	IO to LS
149	PUSH-X	NonStatic	Max	Auto M3	149H1	251.1121	B to C	IO to LS
153	PUSH-X	NonStatic	Max	Auto M3	153H1	250.721	B to C	IO to LS
154	PUSH-X	NonStatic	Max	Auto M3	154H1	251.8298	B to C	IO to LS
155	PUSH-X	NonStatic	Max	Auto M3	155H1	251.9215	B to C	IO to LS
156	PUSH-X	NonStatic	Max	Auto M3	156H1	249.9213	B to C	IO to LS
188	PUSH-X	NonStatic	Max	Auto M3	188H1	254.2973	B to C	IO to LS
189	PUSH-X	NonStatic	Max	Auto M3	189H1	254.975	B to C	IO to LS
190	PUSH-X	NonStatic	Max	Auto M3	190H1	255.0941	B to C	IO to LS
191	PUSH-X	NonStatic	Max	Auto M3	191H1	253.3297	B to C	IO to LS
192	PUSH-X	NonStatic	Max	Auto M3	192H1	254.3492	B to C	IO to LS
193	PUSH-X	NonStatic	Max	Auto M3	193H1	253.5447	B to C	IO to LS
194	PUSH-X	NonStatic	Max	Auto M3	194H1	253.7041	B to C	IO to LS
195	PUSH-X	NonStatic	Max	Auto M3	195H1	252.9468	B to C	IO to LS
196	PUSH-X	NonStatic	Max	Auto M3	196H1	253.8255	B to C	IO to LS
197	PUSH-X	NonStatic	Max	Auto M3	197H1	254.0826	B to C	IO to LS
198	PUSH-X	NonStatic	Max	Auto M3	198H1	252.3396	B to C	IO to LS
199	PUSH-X	NonStatic	Max	Auto M3	199H1	253.4753	B to C	IO to LS
200	PUSH-X	NonStatic	Max	Auto M3	200H1	252.9735	B to C	IO to LS
201	PUSH-X	NonStatic	Max	Auto M3	201H1	254.1833	B to C	IO to LS
202	PUSH-X	NonStatic	Max	Auto M3	202H1	252.2586	B to C	IO to LS
203	PUSH-X	NonStatic	Max	Auto M3	203H1	252.3396	B to C	IO to LS
204	PUSH-X	NonStatic	Max	Auto M3	204H1	253.4753	B to C	IO to LS
205	PUSH-X	NonStatic	Max	Auto M3	205H1	252.9735	B to C	IO to LS
206	PUSH-X	NonStatic	Max	Auto M3	206H1	254.1833	B to C	IO to LS
207	PUSH-X	NonStatic	Max	Auto M3	207H1	252.2586	B to C	IO to LS
208	PUSH-X	NonStatic	Max	Auto M3	208H1	253.5447	B to C	IO to LS
209	PUSH-X	NonStatic	Max	Auto M3	209H1	253.7041	B to C	IO to LS
210	PUSH-X	NonStatic	Max	Auto M3	210H1	252.9468	B to C	IO to LS

211	PUSH-X	NonStatic	Max	Auto M3	211H1	253.8255	B to C	IO to LS
212	PUSH-X	NonStatic	Max	Auto M3	212H1	254.0826	B to C	IO to LS
213	PUSH-X	NonStatic	Max	Auto M3	213H1	254.2973	B to C	IO to LS
214	PUSH-X	NonStatic	Max	Auto M3	214H1	254.975	B to C	IO to LS
215	PUSH-X	NonStatic	Max	Auto M3	215H1	255.0941	B to C	IO to LS
216	PUSH-X	NonStatic	Max	Auto M3	216H1	253.3297	B to C	IO to LS
217	PUSH-X	NonStatic	Max	Auto M3	217H1	254.3492	B to C	IO to LS
248	PUSH-X	NonStatic	Max	Auto M3	248H1	254.1722	B to C	IO to LS
249	PUSH-X	NonStatic	Max	Auto M3	249H1	255.4164	B to C	IO to LS
250	PUSH-X	NonStatic	Max	Auto M3	250H1	256.0735	B to C	IO to LS
251	PUSH-X	NonStatic	Max	Auto M3	251H1	255.6389	B to C	IO to LS
252	PUSH-X	NonStatic	Max	Auto M3	252H1	255.8883	B to C	IO to LS
253	PUSH-X	NonStatic	Max	Auto M3	253H1	253.6263	B to C	IO to LS
254	PUSH-X	NonStatic	Max	Auto M3	254H1	254.6371	B to C	IO to LS
255	PUSH-X	NonStatic	Max	Auto M3	255H1	255.0704	B to C	IO to LS
256	PUSH-X	NonStatic	Max	Auto M3	256H1	254.4919	B to C	IO to LS
257	PUSH-X	NonStatic	Max	Auto M3	257H1	252.4725	B to C	IO to LS
258	PUSH-X	NonStatic	Max	Auto M3	258H1	255.1038	B to C	IO to LS
259	PUSH-X	NonStatic	Max	Auto M3	259H1	254.0105	B to C	IO to LS
260	PUSH-X	NonStatic	Max	Auto M3	260H1	255.1116	B to C	IO to LS
261	PUSH-X	NonStatic	Max	Auto M3	261H1	254.9784	B to C	IO to LS
262	PUSH-X	NonStatic	Max	Auto M3	262H1	254.3425	B to C	IO to LS
263	PUSH-X	NonStatic	Max	Auto M3	263H1	255.1038	B to C	IO to LS
264	PUSH-X	NonStatic	Max	Auto M3	264H1	254.0105	B to C	IO to LS
265	PUSH-X	NonStatic	Max	Auto M3	265H1	255.1116	B to C	IO to LS
266	PUSH-X	NonStatic	Max	Auto M3	266H1	254.9784	B to C	IO to LS
267	PUSH-X	NonStatic	Max	Auto M3	267H1	254.3425	B to C	IO to LS
268	PUSH-X	NonStatic	Max	Auto M3	268H1	253.6263	B to C	IO to LS
269	PUSH-X	NonStatic	Max	Auto M3	269H1	254.6371	B to C	IO to LS
270	PUSH-X	NonStatic	Max	Auto M3	270H1	255.0704	B to C	IO to LS
271	PUSH-X	NonStatic	Max	Auto M3	271H1	254.4919	B to C	IO to LS
272	PUSH-X	NonStatic	Max	Auto M3	272H1	252.4725	B to C	IO to LS
273	PUSH-X	NonStatic	Max	Auto M3	273H1	254.1722	B to C	IO to LS
274	PUSH-X	NonStatic	Max	Auto M3	274H1	255.4164	B to C	IO to LS

The detailed results of hinges has been idealizes in the following segments.

4.8.1 .2 Hinges Status for P-M-M hinge

The variation of hinge and their status can be analyzes from the following figures for the various stages.

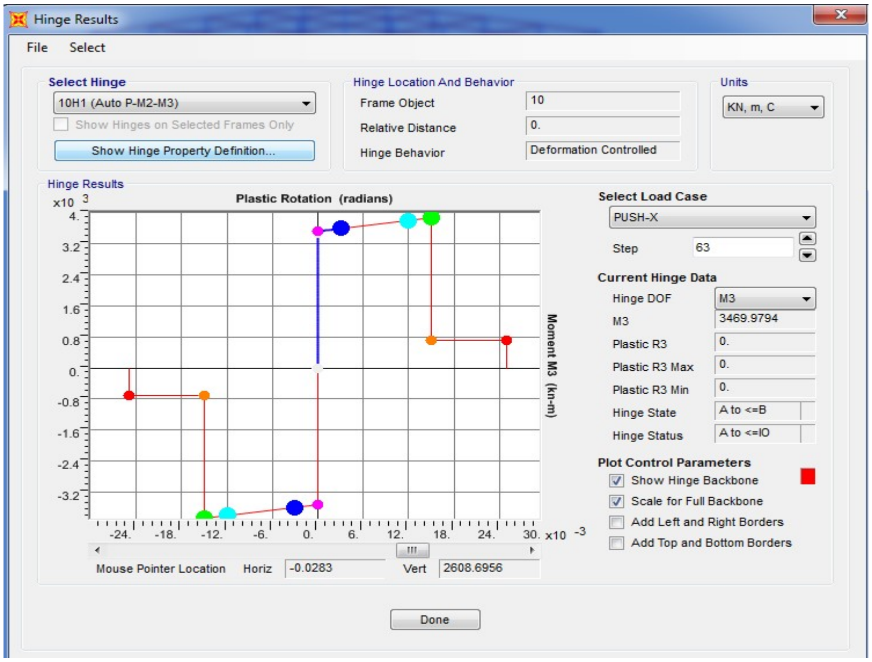


Fig-4.9(a) - **Hinge result for 10H1 hinge at step 63**

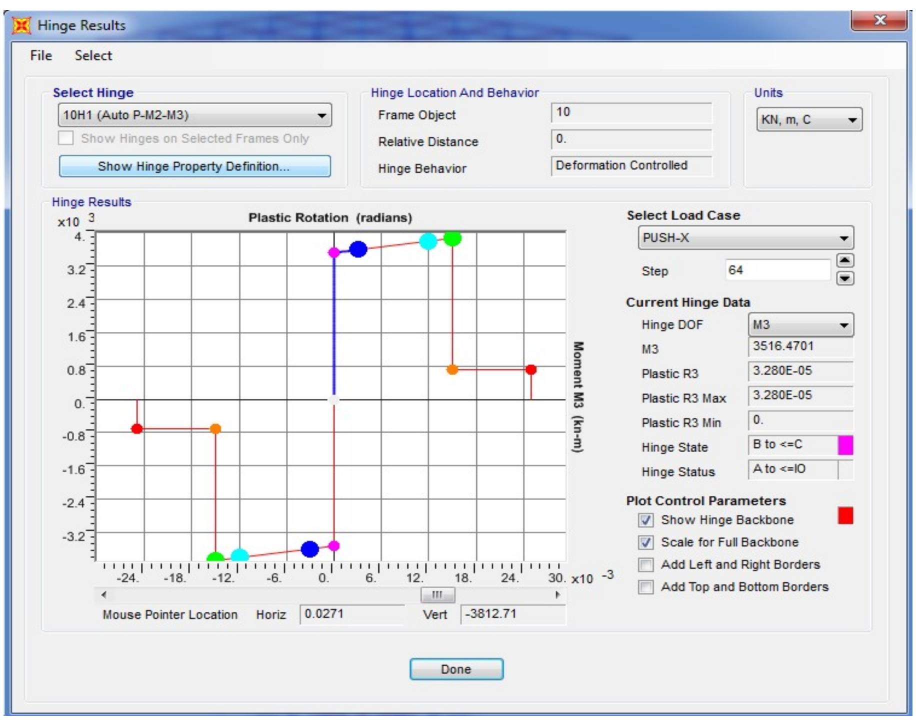


Fig-4.9(b) - **Hinge result for 10H1 hinge at step 64**

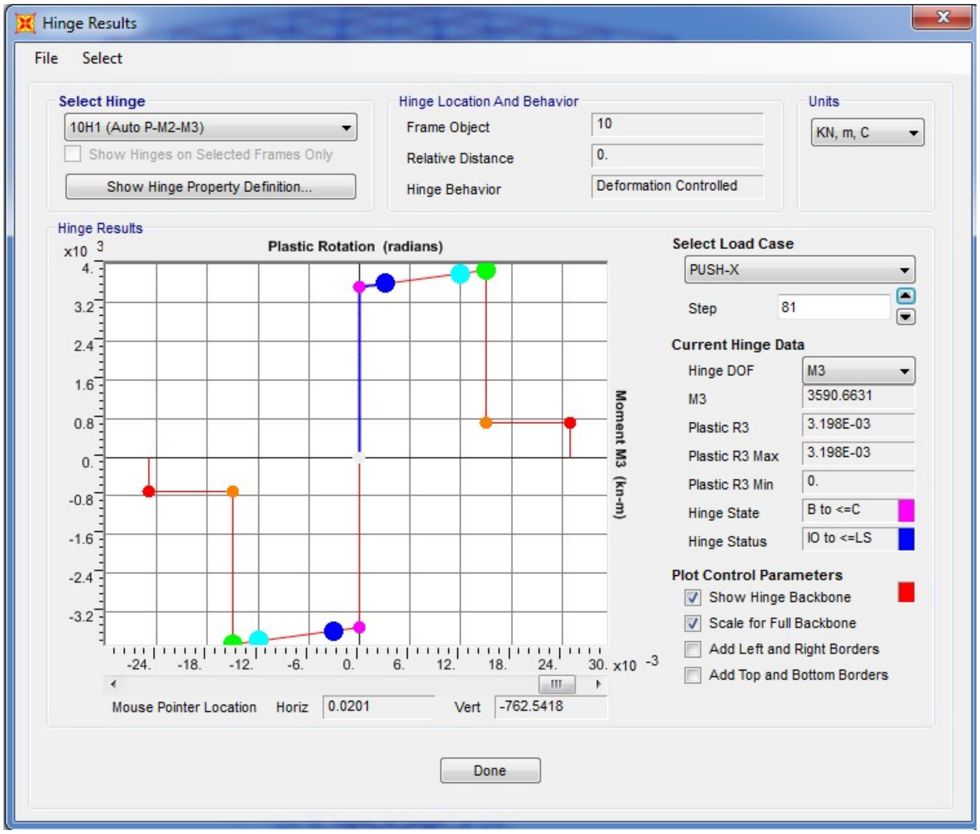
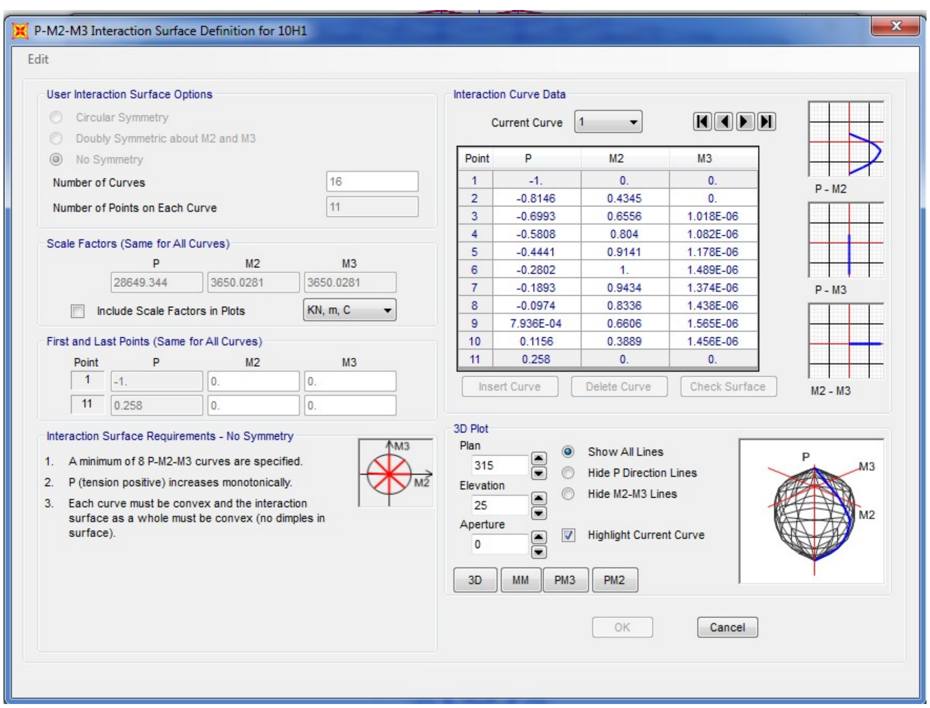


Fig-4.9(c) - **Hinge result for 10H1 hinge at step 81**



Point	P	M2	M3
1	-1.	0.	0.
2	-0.8146	0.4345	0.
3	-0.6993	0.6556	1.018E-06
4	-0.5808	0.804	1.082E-06
5	-0.4441	0.9141	1.178E-06
6	-0.2802	1.	1.489E-06
7	-0.1893	0.9434	1.374E-06
8	-0.0974	0.8336	1.438E-06
9	7.936E-04	0.6606	1.565E-06
10	0.1156	0.3889	1.456E-06
11	0.258	0.	0.

Fig-4.9(d) - **P-M-M interaction surface for 10H1 hinge** from above figure it is clear that up to step 63 the member hinge is in A to B state, at step 64 the hinge transferred to IO level and at step 81 it is transferred from IO to LS level. detailed and complete step wise result can be obtained from table 4.8

Table 4.10 :- Detailed hinge result for 10H1

TABLE: Hinge Results											
Hinge	**Load**	**Step**	**P**	**U1State**	**U1Status**	**M2**	**R2Status**	**M3**	**R3Status**	**HingeState**	**HingeStatus**
Name	Case		KN	Text	Text	KN-m	Text	KN-m	Text	Text	Text
10H1	PUSH-X	0	-10272	A to <=B	A to <=IO	3.4818	A to <=IO	0.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	1	-10275	A to <=B	A to <=IO	3.4825	A to <=IO	62.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	2	-10279	A to <=B	A to <=IO	3.4832	A to <=IO	123.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	3	-10282	A to <=B	A to <=IO	3.4839	A to <=IO	185.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	4	-10284	A to <=B	A to <=IO	3.4845	A to <=IO	234.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	5	-10292	A to <=B	A to <=IO	3.4916	A to <=IO	318.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	6	-10300	A to <=B	A to <=IO	3.4995	A to <=IO	379.1	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	7	-10311	A to <=B	A to <=IO	3.5016	A to <=IO	435.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	8	-10319	A to <=B	A to <=IO	3.5031	A to <=IO	494.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	9	-10326	A to <=B	A to <=IO	3.5039	A to <=IO	553.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	10	-10333	A to <=B	A to <=IO	3.5047	A to <=IO	624.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	11	-10338	A to <=B	A to <=IO	3.5055	A to <=IO	681.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	12	-10343	A to <=B	A to <=IO	3.5055	A to <=IO	744.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	13	-10347	A to <=B	A to <=IO	3.5062	A to <=IO	798.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	14	-10352	A to <=B	A to <=IO	3.5047	A to <=IO	856.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	15	-10355	A to <=B	A to <=IO	3.5047	A to <=IO	910.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	16	-10359	A to <=B	A to <=IO	3.5059	A to <=IO	961.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	17	-10363	A to <=B	A to <=IO	3.5076	A to <=IO	1010.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	18	-10365	A to <=B	A to <=IO	3.5075	A to <=IO	1056.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	19	-10369	A to <=B	A to <=IO	3.5086	A to <=IO	1106.9	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	20	-10371	A to <=B	A to <=IO	3.5079	A to <=IO	1155.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	21	-10374	A to <=B	A to <=IO	3.5096	A to <=IO	1202.4	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	22	-10377	A to <=B	A to <=IO	3.5097	A to <=IO	1263.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	23	-10380	A to <=B	A to <=IO	3.5112	A to <=IO	1313.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	24	-10382	A to <=B	A to <=IO	3.5115	A to <=IO	1363.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	25	-10384	A to <=B	A to <=IO	3.512	A to <=IO	1414.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	26	-10388	A to <=B	A to <=IO	3.5142	A to <=IO	1484.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	27	-10390	A to <=B	A to <=IO	3.5153	A to <=IO	1535.9	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	28	-10391	A to <=B	A to <=IO	3.5153	A to <=IO	1590.4	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	29	-10395	A to <=B	A to <=IO	3.5157	A to <=IO	1667.1	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	30	-10397	A to <=B	A to <=IO	3.5164	A to <=IO	1733.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	31	-10399	A to <=B	A to <=IO	3.5176	A to <=IO	1799.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	32	-10401	A to <=B	A to <=IO	3.5185	A to <=IO	1840.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	33	-10403	A to <=B	A to <=IO	3.5198	A to <=IO	1910.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	34	-10406	A to <=B	A to <=IO	3.5212	A to <=IO	1981.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	35	-10407	A to <=B	A to <=IO	3.5215	A to <=IO	2038.0	A to <=IO	A to <=B	A to <=IO

Hinge	Load	Step	P	U1State	U1Status	M2	R2Status	M3	R3Status	HingeState	HingeStatus
Name	Case		KN	Text	Text	KN-m	Text	KN-m	Text	Text	Text
10H1	PUSH-X	36	-10409	A to <=B	A to <=IO	3.5222	A to <=IO	2086.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	37	-10411	A to <=B	A to <=IO	3.5222	A to <=IO	2165.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	38	-10412	A to <=B	A to <=IO	3.5228	A to <=IO	2205.1	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	39	-10414	A to <=B	A to <=IO	3.5234	A to <=IO	2260.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	40	-10415	A to <=B	A to <=IO	3.524	A to <=IO	2299.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	41	-10416	A to <=B	A to <=IO	3.5245	A to <=IO	2343.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	42	-10417	A to <=B	A to <=IO	3.5254	A to <=IO	2389.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	43	-10419	A to <=B	A to <=IO	3.5267	A to <=IO	2467.1	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	44	-10421	A to <=B	A to <=IO	3.5276	A to <=IO	2532.4	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	45	-10423	A to <=B	A to <=IO	3.5289	A to <=IO	2606.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	46	-10424	A to <=B	A to <=IO	3.5295	A to <=IO	2644.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	47	-10425	A to <=B	A to <=IO	3.5299	A to <=IO	2683.2	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	48	-10426	A to <=B	A to <=IO	3.5304	A to <=IO	2721.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	49	-10427	A to <=B	A to <=IO	3.5307	A to <=IO	2786.4	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	50	-10429	A to <=B	A to <=IO	3.5314	A to <=IO	2839.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	51	-10430	A to <=B	A to <=IO	3.532	A to <=IO	2877.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	52	-10431	A to <=B	A to <=IO	3.5325	A to <=IO	2916.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	53	-10432	A to <=B	A to <=IO	3.5328	A to <=IO	2963.3	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	54	-10433	A to <=B	A to <=IO	3.5333	A to <=IO	3001.4	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	55	-10433	A to <=B	A to <=IO	3.5338	A to <=IO	3039.5	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	56	-10434	A to <=B	A to <=IO	3.5342	A to <=IO	3077.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	57	-10435	A to <=B	A to <=IO	3.5347	A to <=IO	3115.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	58	-10436	A to <=B	A to <=IO	3.5351	A to <=IO	3157.6	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	59	-10437	A to <=B	A to <=IO	3.5358	A to <=IO	3223.9	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	60	-10437	A to <=B	A to <=IO	3.5363	A to <=IO	3268.7	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	61	-10439	A to <=B	A to <=IO	3.5371	A to <=IO	3340.8	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	62	-10440	A to <=B	A to <=IO	3.5376	A to <=IO	3391.1	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	63	-10441	A to <=B	A to <=IO	3.5388	A to <=IO	3470.0	A to <=IO	A to <=B	A to <=IO
10H1	PUSH-X	64	-10442	B to <=C	A to <=IO	3.5397	A to <=IO	3516.5	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	65	-10444	B to <=C	A to <=IO	3.5407	A to <=IO	3519.4	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	66	-10445	B to <=C	A to <=IO	3.5411	A to <=IO	3522.9	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	67	-10446	B to <=C	A to <=IO	3.5415	A to <=IO	3527.3	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	68	-10446	B to <=C	A to <=IO	3.5416	A to <=IO	3531.3	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	69	-10447	B to <=C	A to <=IO	3.542	A to <=IO	3535.7	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	70	-10448	B to <=C	A to <=IO	3.542	A to <=IO	3539.7	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	71	-10449	B to <=C	A to <=IO	3.5424	A to <=IO	3544.1	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	72	-10449	B to <=C	A to <=IO	3.5424	A to <=IO	3548.1	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	73	-10450	B to <=C	A to <=IO	3.5426	A to <=IO	3552.6	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	74	-10451	B to <=C	A to <=IO	3.5424	A to <=IO	3556.4	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	75	-10451	B to <=C	A to <=IO	3.543	A to <=IO	3561.0	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	76	-10452	B to <=C	A to <=IO	3.543	A to <=IO	3565.0	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	77	-10453	B to <=C	A to <=IO	3.5436	A to <=IO	3569.5	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	78	-10453	B to <=C	A to <=IO	3.5436	A to <=IO	3573.4	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	79	-10454	B to <=C	A to <=IO	3.5442	A to <=IO	3578.0	A to <=IO	B to <=C	A to <=IO

Hinge	Load	Step	P	U1State	U1Status	M2	R2Status	M3	R3Status	HingeState	HingeStatus
Name	Case		KN	Text	Text	KN-m	Text	KN-m	Text	Text	Text
10H1	PUSH-X	80	-10455	B to <=C	A to <=IO	3.5443	A to <=IO	3582.1	A to <=IO	B to <=C	A to <=IO
10H1	PUSH-X	81	-10456	B to <=C	IO to <=LS	3.545	IO to <=LS	3590.7	IO to <=LS	B to <=C	IO to <=LS
10H1	PUSH-X	82	-10456	B to <=C	IO to <=LS	3.5457	IO to <=LS	3595.2	IO to <=LS	B to <=C	IO to <=LS
10H1	PUSH-X	83	-10456	B to <=C	IO to <=LS	3.5457	IO to <=LS	3596.2	IO to <=LS	B to <=C	IO to <=LS

4.8.1 .3 Hinges Status for M-3 hinge

The variation of hinge (hinge 209H1)and their status can be analyzes from the following figures for the various stages.

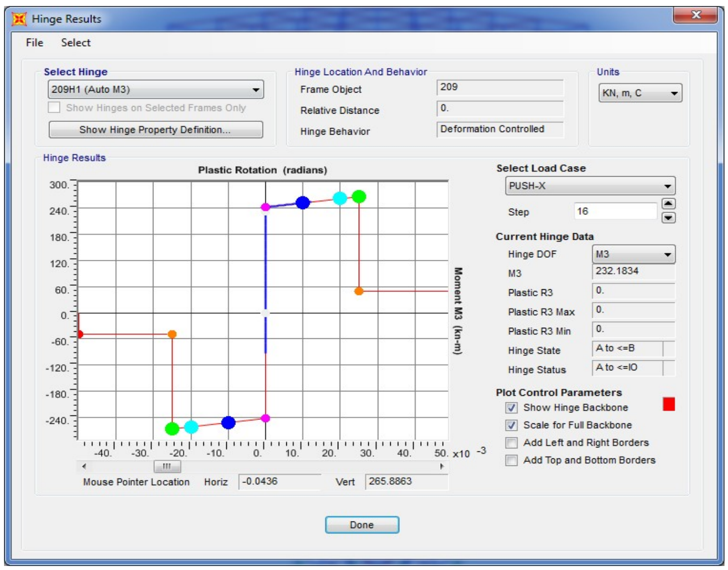


Fig-4.10(a) - **Hinge result for 209H1 hinge at step 16**

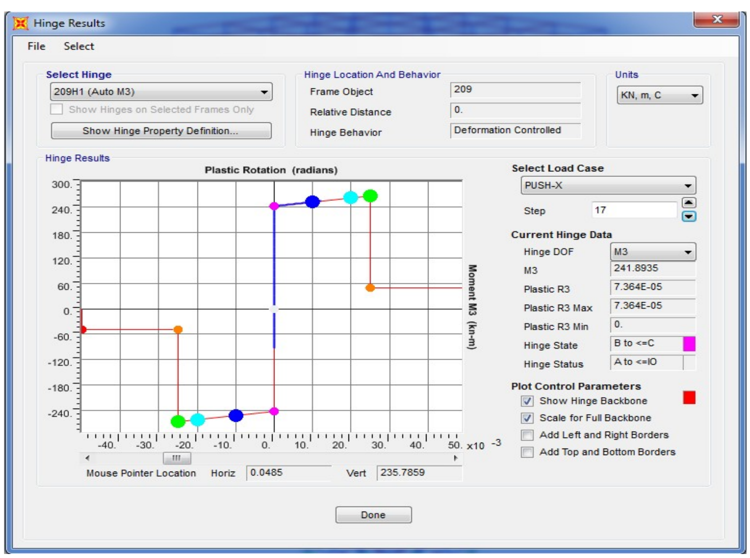


Fig-4.10(b) - **Hinge result for 209H1 hinge at step 17**

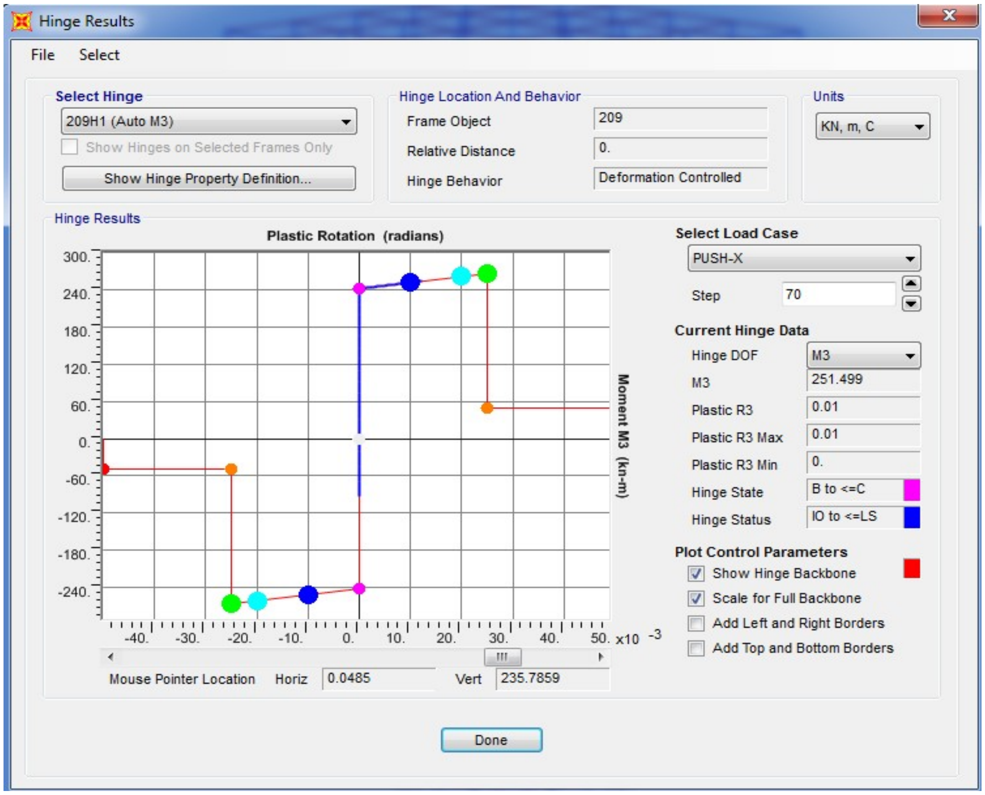


Fig-4.10(c) - **Hinge result for 209H1 hinge at step 70**

from above figure it is clear that up to step 16 the member hinge is in A to B state, at step 17 the hinge transferred to IO level and at step 70 it is transferred from IO to LS level. detailed and complete step wise result can be obtained from table 4.9

Table 4.11 :- Detailed hinge result for 209H1

TABLE: Hinge Results								
HingeName	LoadCase	Step	M3	R3Pl	R3PlMax	R3PlMin	R3State	R3Status
Text	Text	Unitless	KN-m	Radians	Radians	Radians	Text	Text
209H1	PUSH-X	0	-94.15	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	1	-70.09	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	2	-46.03	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	3	-21.96	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	4	-2.89	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	5	30.41	0.000	0.000	0	A to <=B	A to <=IO

HingeName	LoadCase	Step	M3	R3Pl	R3PlMax	R3PlMin	R3State	R3Status
Text	Text	Unitless	KN-m	Radians	Radians	Radians	Text	Text
209H1	PUSH-X	6	55.33	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	7	72.34	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	8	90.34	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	9	108.45	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	10	129.38	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	11	146.32	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	12	164.94	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	13	180.95	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	14	198.70	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	15	215.62	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	16	232.18	0.000	0.000	0	A to <=B	A to <=IO
209H1	PUSH-X	17	241.89	0.000	0.000	0	B to <=C	A to <=IO
209H1	PUSH-X	18	242.06	0.000	0.000	0	B to <=C	A to <=IO
209H1	PUSH-X	19	242.24	0.000	0.000	0	B to <=C	A to <=IO
209H1	PUSH-X	20	242.41	0.001	0.001	0	B to <=C	A to <=IO
209H1	PUSH-X	21	242.57	0.001	0.001	0	B to <=C	A to <=IO
209H1	PUSH-X	22	242.78	0.001	0.001	0	B to <=C	A to <=IO
209H1	PUSH-X	23	242.95	0.001	0.001	0	B to <=C	A to <=IO
209H1	PUSH-X	24	243.13	0.001	0.001	0	B to <=C	A to <=IO
209H1	PUSH-X	25	243.30	0.002	0.002	0	B to <=C	A to <=IO
209H1	PUSH-X	26	243.54	0.002	0.002	0	B to <=C	A to <=IO
209H1	PUSH-X	27	243.72	0.002	0.002	0	B to <=C	A to <=IO
209H1	PUSH-X	28	243.91	0.002	0.002	0	B to <=C	A to <=IO
209H1	PUSH-X	29	244.17	0.002	0.002	0	B to <=C	A to <=IO
209H1	PUSH-X	30	244.40	0.003	0.003	0	B to <=C	A to <=IO
209H1	PUSH-X	31	244.62	0.003	0.003	0	B to <=C	A to <=IO
209H1	PUSH-X	32	244.76	0.003	0.003	0	B to <=C	A to <=IO
209H1	PUSH-X	33	245.01	0.003	0.003	0	B to <=C	A to <=IO
209H1	PUSH-X	34	245.25	0.004	0.004	0	B to <=C	A to <=IO
209H1	PUSH-X	35	245.44	0.004	0.004	0	B to <=C	A to <=IO
209H1	PUSH-X	36	245.61	0.004	0.004	0	B to <=C	A to <=IO
209H1	PUSH-X	37	245.88	0.004	0.004	0	B to <=C	A to <=IO
209H1	PUSH-X	38	246.02	0.004	0.004	0	B to <=C	A to <=IO
209H1	PUSH-X	39	246.21	0.005	0.005	0	B to <=C	A to <=IO
209H1	PUSH-X	40	246.34	0.005	0.005	0	B to <=C	A to <=IO
209H1	PUSH-X	41	246.49	0.005	0.005	0	B to <=C	A to <=IO

HingeName	LoadCase	Step	M3	R3Pl	R3PlMax	R3PlMin	R3State	R3Status
Text	Text	Unitless	KN-m	Radians	Radians	Radians	Text	Text
209H1	PUSH-X	42	246.65	0.005	0.005	0	B to <=C	A to <=IO
209H1	PUSH-X	43	246.92	0.005	0.005	0	B to <=C	A to <=IO
209H1	PUSH-X	44	247.14	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	45	247.40	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	46	247.53	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	47	247.66	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	48	247.79	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	49	248.02	0.006	0.006	0	B to <=C	A to <=IO
209H1	PUSH-X	50	248.20	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	51	248.33	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	52	248.46	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	53	248.62	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	54	248.76	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	55	248.89	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	56	249.02	0.007	0.007	0	B to <=C	A to <=IO
209H1	PUSH-X	57	249.15	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	58	249.28	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	59	249.48	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	60	249.61	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	61	249.83	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	62	249.97	0.008	0.008	0	B to <=C	A to <=IO
209H1	PUSH-X	63	250.21	0.009	0.009	0	B to <=C	A to <=IO
209H1	PUSH-X	64	250.40	0.009	0.009	0	B to <=C	A to <=IO
209H1	PUSH-X	65	250.62	0.009	0.009	0	B to <=C	A to <=IO
209H1	PUSH-X	66	250.83	0.009	0.009	0	B to <=C	A to <=IO
209H1	PUSH-X	67	251.00	0.009	0.009	0	B to <=C	A to <=IO
209H1	PUSH-X	68	251.16	0.010	0.010	0	B to <=C	A to <=IO
209H1	PUSH-X	69	251.34	0.010	0.010	0	B to <=C	A to <=IO
209H1	PUSH-X	70	251.50	0.010	0.010	0	B to <=C	IO to <=LS
209H1	PUSH-X	71	251.67	0.010	0.010	0	B to <=C	IO to <=LS
209H1	PUSH-X	72	251.83	0.010	0.010	0	B to <=C	IO to <=LS
209H1	PUSH-X	73	252.01	0.011	0.011	0	B to <=C	IO to <=LS
209H1	PUSH-X	74	252.17	0.011	0.011	0	B to <=C	IO to <=LS
209H1	PUSH-X	75	252.34	0.011	0.011	0	B to <=C	IO to <=LS
209H1	PUSH-X	76	252.51	0.011	0.011	0	B to <=C	IO to <=LS
209H1	PUSH-X	77	252.68	0.011	0.011	0	B to <=C	IO to <=LS

HingeName	LoadCase	Step	M3	R3Pl	R3PlMax	R3PlMin	R3State	R3Status
Text	Text	Unitless	KN-m	Radians	Radians	Radians	Text	Text
209H1	PUSH-X	78	252.84	0.011	0.011	0	B to <=C	IO to <=LS
209H1	PUSH-X	79	253.01	0.012	0.012	0	B to <=C	IO to <=LS
209H1	PUSH-X	80	253.17	0.012	0.012	0	B to <=C	IO to <=LS
209H1	PUSH-X	81	253.50	0.012	0.012	0	B to <=C	IO to <=LS
209H1	PUSH-X	82	253.67	0.012	0.012	0	B to <=C	IO to <=LS
209H1	PUSH-X	83	253.70	0.012	0.012	0	B to <=C	IO to <=LS

4.8.2 SEGMENT - 2

The study of the effect of change of main reinforcement on the performance of the structure, various case studies are made. All beams and columns at a particular story are given same reinforcement. Reinforcement in columns is varies per two storiess.

Table 4.12Description of various case studies (in elevation)

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Base structure
2	1	Increasing reinforcement in beams of 1^{st} level only
3	2	Increasing reinforcement in beams of 2^{nd} level only
4	3	Increasing reinforcement in beams of 3^{rd} level only
5	4	Increasing reinforcement in beams of 4^{th} level only
6	5	Increasing reinforcement in columns of 1^{st} and 2^{nd} level only
7	6	Increasing reinforcement in columns of 3^{rd} and 4^{th} level only
8	7	Increasing reinforcement in beams & columns of 1^{st} and 2^{nd} level only
9	8	Increasing reinforcement in beams & columns of 3^{rd} and 4^{th} level only

To study the effect of change of main reinforcement of various columns on the performance of the structure, various case studies are made. For this the initial reinforcement of all the columns is kept almost same.

Columns are numbered from 1 to 3, starting from front left corner.

Table 4.13 Description of various cases (in plan)

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Base structure
2	1	By Increasing reinforcement in column 1 only
3	2	Increasing reinforcement in column 2 only
4	3	Increasing reinforcement in column 3 only

4.9 ANALYSIS OF RESULTS

4.9.1 Base Shear force

The base Shear force for the G+19 storied building with different combination of element reinforcement at various floor levels is presented in Table 4.11.

It is observed that with increase in reinforcement of beams only, there is a very nominal percentage change in the base shear varying from 1.22% to -2.97%, which the structure can withstand. However, with the increase in reinforcement of level columns, there is quite an appreciable change in the base force carrying capacity of the structure. The combination of change in reinforcement of beams and columns both show a small increment in base shear force capacity. the effect of change of reinforcement is studied only up-to 4th level. (above the 4th level variation of reinforcement is almost minimal)

Table: 4.14 Comparison of Base Shear Force

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR FORCE (KN)	PERCENTAGE CHANGE IN BASE SHEAR FORCE
Basic structure			**7000.55**	
Beams of 1st LEVEL	CASE 1	13.65	7490.59	1.07
	CASE 1	21	8540.67	1.22
Beams of 2nd LEVEL	CASE 2	14.88	6790.53	-2.97
	CASE 2	32.74	7168.56	0.24
Beams of 3rd LEVEL	CASE3	5.27	6965.54	-0.05
	CASE 3	9.03	6993.5	-0.01
Beams of 4th LEVEL	CASE4	16.48	7000.55	0.00
	CASE 4	33.54	7000.55	0.00
Columns of 1st & 2nd LEVEL	CASE 5	4.05	7016.65	0.23
	CASE 5	38.32	7047.45	0.67

Columns of 3^{rd} & 4^{th} LEVEL	CASE 6	4.08	6825.53	-2.41
	CASE 6	39.32	7022.95	0.32
Beams & Columns of 1^{st} & 2^{nd} LEVEL	CASE 7	11.28	8316.65	1.88
	CASE 7	29.66	8792.69	2.56
Beams & Columns of 3^{rd} & 4^{th} LEVEL	CASE 8	8.08	8036.63	1.48
	CASE 8	27.55	7022.95	0.32

4.9.2 Roof Displacement

The Roof displacement/drift for the G+19 storied building with different combination of element and reinforcement at various levels is presented in Table 4.5.

It is observed and studied that by increasing the reinforcement of beams only, there is a decrease in the roof displacement upto 3rd level and after 3rd level there is no change. The percentage change varies from 1.89% to 13.59%. However, the trends shown by increasing the reinforcement of columns only is a substantial decrease in the roof displacement which varies from 0.6% to 21.08%. The combination of increase of reinforcement of beams and columns both, show a little increase in the roof displacement upto 2nd level and after 3rd level it slightly decreases upto 4th level.

There is a predominant decrease (63.36%) in roof displacement when shear wall is provided in building.

Table: 4.14 Comparison of Roof Displacement

STRUCTURAL ELEMENTS	CASES	% INCREASE IN REINFORCEMENTS	ROOF DISPLACEMENT (mm)	% CHANGE IN ROOF DISPLACEMENTS
Basic structure			792	
Beams of 1st LEVEL	CASE 1	13.65	720.72	-9.81
	CASE 1	21	700.92	-11.61
Beams of 2nd LEVEL	CASE 2	14.88	778.2192	-1.74

	CASE 2	32.74	769.7448	-2.81
Beams of 3rd LEVEL	CASE 3	5.27	792	0
	CASE 3	9.03	676.4472	-14.59
Beams of 4th LEVEL	CASE 4	16.48	792	0
	CASE 4	33.54	792	0
Columns of 1st & 2nd LEVEL	CASE 5	4.05	785.7432	-0.69
	CASE 5	38.32	617.2056	-22.08
Columns of 3rd & 4th LEVEL	CASE 6	4.08	773.3088	-2.36
	CASE 6	39.32	751.0536	-5.17
Beams & Columns of 1st & 2nd LEVEL	CASE 7	11.28	783.5256	-1.07
	CASE 7	29.66	733.7088	-7.36
Beams & Columns of 3rd & 4th LEVEL	CASE 8	8.08	743.1336	-6.17
	CASE 8	27.55	696.4056	-12.07

4.9.3 Pushover Curve

The Pushover curve is the curve which is plotted between the Base force and Roof displacement. This curve shows the overall response of the structure in case of incremental seismic loading.

The structure is applied an inverted triangular loading. This loading is increased monotonically, in small increments, till there is a failure in the structure at any level. As the loading is increased, a curve between the base force and roof displacement is plotted. This curve is known as the pushover curve.

Table: 4.16Variation of Roof Displacement with Base Shear Force for all cases

STRUCTURAL ELEMENTS WITH LEVELS	CASES	% INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	ROOF DISPLACEMENTS (mm)
Basic structure			**7000.55**	**792**
Beams of 1st LEVEL	CASE 1	13.65	7490.59	720.72
	CASE 1	21	8540.67	700.92
Beams of	CASE 2	14.88	6790.53	778.2192
2nd LEVEL	CASE 2	32.74	7168.56	769.7448
Beams of 3rd LEVEL	CASE 3	5.27	6965.54	792
	CASE 3	9.03	6993.5	676.4472
Beams of 4th LEVEL	CASE 4	16.48	7000.55	792
	CASE 4	33.54	7000.55	792
Columns of 1st & 2nd LEVEL	CASE 5	4.05	7016.65	785.7432
	CASE 5	38.32	7047.45	617.2056
	CASE 6	4.08	6825.53	773.3088
	CASE 6	39.32	7022.95	751.0536
Beams & Columns of 1st & 2nd LEVEL	CASE 7	11.28	8316.65	783.5256
	CASE 7	29.66	8792.69	733.7088
Beams & Columns of 3rd & 4th LEVEL	CASE 8	8.08	8036.63	743.1336
	CASE 8	27.55	7022.95	696.4056

4.9.4 Performance Point

The performance point of the building structure can be b determined by using the ADRS pushover curves obtained. The performance point is the point where the capacity and demand of the structure are equal. Hence, it can be denoted as a measure of economy of the reinforcement system. The performance point is determined automatically by SAP2000, using the procedure C mentioned in ATC-40.

The point at which the capacity curve intersects the reduced demand curve represents the performance point of structure at which capacity and demand are equal. As displacement increase, the time period of the structure lengthens. This is reflected in the capacity spectrum. Displacements increase damping and reduce demand. Hence, the optimised point should have a higher capacity for a lesser displacement.

4.10 PERFORMANCE BASED DESIGN

Specified deformation states are often taken as a measure of building performance at corresponding load levels. For example, the US Federal Emergency Management Agency [4] identifies operational, immediate-occupancy, life-safety and collapse-prevention performance levels, and adopts roof-level lateral drift at the corresponding load levels as a measure of the associated behavior states of the building. The increasing degrees of damage that a building experiences at the various performance levels are associated with earthquakes having increasing intensities of horizontal ground motion.

Table 4.17 Target Roof Lateral Displacement ratios at various performance levels [4]

Performance level	Operational	Immediate Occupancy	Life-Safety	Collapse-Prevention
Lateral Drift ratio (δ/h) %	0.37	0.7	2.5	5

Where, δ is Lateral Roof Displacement and h is total height of building

Performance based design is obtained by increasing the main reinforcement of various frame elements by hit and trail method, so that the building performance level, (after performing Pushover Analysis) lies in Immediate Occupancy level i.e., roof displacement of building is 0.9% of total height of building.

Target Roof Displacement = 0.009 x 88.6m = 0.797m = 797mm

Design thus obtained is subjected to triangular loading corresponding to MCE, (Maximum Considered Earthquake) so that the structural damage is limited to Grade 3 (moderate structural damage, heavy nonstructural damage) in order to ensure Life Safety i.e., roof displacement of building is 2.5% of total height of building.

Target Roof Displacement = 0.025 x 88.6m = 2.215m = 2215mm

The design horizontal seismic coefficient A_h for a structure under MCE shall be determined by the following expressions:-

$\mathbf{A_h} = Z \times (I/R) \times (S_a/g)$

The reinforcement detail of the Performance based building design thus obtained is shown in Table 4.16. These are compared to the reinforcement obtained by seismic resistant design of building (according to IS 1893:2002) in ETABS

Table 4.18 Comparison of area of reinforcement in mm^2 in beams and columns for all designs up-to 4th floor level

Element	Dimensio	Reinforcement	Reinforcement Area	Reinforcement
		(During Dead and live only)	(Based on IS 1893 and 13920)	Performance based Design
		(IS456:2000)		
Corner Columns	0.80 x	5100	7700	6776
Mid-face	0.80 x	13200	14200	20448
Interior Column	0.80 x	22400	24400	37820
Beams 1st to 5th	0.45 x 0.6	1900 (top) 1200 (bottom)	2500 (top) 1800 (bottom)	2700 (top) 2100 (bottom)
Beams 5th to	0.45 x 0.6	2100 (top) 1800 (bottom)	3200 (top) 2100 (bottom)	3200 (top) 2100 (bottom)
Beams 10th to	0.45 x 0.6	2100 (top) 1800 (bottom)	3200 (top) 3200 (bottom)	3200 (top) 3200 (bottom)
Beams 15th to top	0.45 x 0.6	2100 (top) 1800 (bottom)	3200 (top) 2100 (bottom)	3200 (top) 2100 (bottom)

Following results are obtained for pushover analysis of Performance based design: Base Shear = 6631.73 kN

Roof Displacement = 658.0mm

Thus Roof displacement is less than target roof displacement.

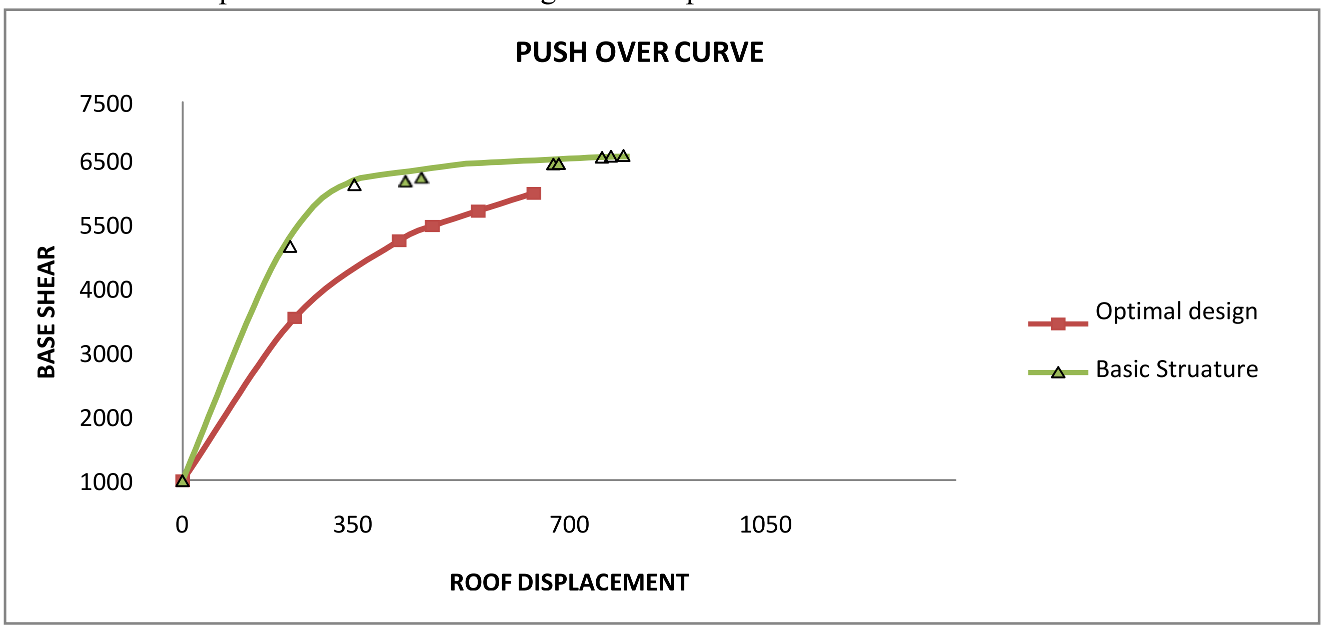


Figure 4.11 Pushover curve for Performance based design of four level building

5.1 GENERAL

In this work, Performance based seismic design of a G+19 storied symmetrical building structure has been done by evaluating their performance using Non linear pushover analysis. Reinforcement of various structural elements of the structure i.e. the beams and the columns increased with different combinations and their effect on the performance on the structure was studied. The design of reinforcement done in ETABS and further non-linear analysis was carried out using SAP2000 nonlinear software tool.

5.2 CONCLUSIONS

Based on the present study, the following conclusions can be drawn and pointed out:

1. Performance increases on increasing reinforcement of columns only resulting into an appreciable decrease in the maximum roof displacement. Decrement in roof displacement is maximum interior column and for corner and mid-face columns it is comparable.
2. The increment in reinforcement of columns only results into a nominal increase in base shear. It is observed that changing reinforcement of 1^{st} level affects base shear more than other levels.
3. Performance of the building decreases when the sectional sizes of beams and columns are reduced while keeping same reinforcement at various levels.
4. Increment in reinforcement of beams and columns both result in an appreciable decrement in roof displacement in building.

7. The performance based seismic design obtained by above procedure satisfies the acceptance criteria for immediate occupancy and life safety limit states for various intensities of earthquakes.
8. Performance based seismic design obtained leads to a small reduction in steel reinforcement when compared to code based seismic design (IS 1893:2002) obtained by ETABS
9. Sequential Formation of plastic hinges gives us the failure pattern of sequence of column/beams failure. This is the valuable information in the dynamic analysis and in

designing the structure. Hence we need to strengthen only selected member of the same story.

10. Formation of hinges starts from beam ends and then propagate to the upper stories and then in the lower stories column and then propagate to the upper stories. Most of the hinges developed in the beams and in the columns but it is limited into life safety limits.
11. The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behavior of structure.

As a closing remark, one can say that performance based seismic design gives a structure with better seismic load carrying capacity, thereby achieving the objective of ***PERFORMANCE*** as well as ***ECONOMY*** and there is certainly scope for further improvement in the mentioned method.

5.3 SCOPE OF FUTURE WORK

Within the limited scope of the present work, the broad conclusions can be drawn from this work have been reported. However, further study can be undertaken in the following areas and portions:

1. In the present study, the pushover analysis has been carried out for G+19 storied buildings. This study can further be extended for tall buildings.
2. In the present study, the conceptual design i.e., the sizes of beams and columns are kept same. Work can be done to optimization the sizes of various frame elements using Non-linear pushover analysis.
3. A comparative study can be done to see the effect of shear reinforcement on performance based seismic design using non-linear pushover analysis

REFERENCES

[1] ASCE, 1998, *Handbook for the Seismic Evaluation of Buildings, a Prestandard,* FEMA 310 Report, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

[2] ASCE, 2000, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings,* FEMA 356 Report, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C.

[3] ASCE, 2002, S*tandard Methodology for Seismic Evaluation of Buildings.* Standard No. ASCE-31. American Society of Civil Engineers, Reston, Virginia.

[4] ATC, 1997a, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273 Report, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.

[5] ATC, 1997b, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 274 Report, prepared by the Applied Technology Council, for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.

[6] ATC, 2006, *Next-Generation Performance-based Earthquake design Guidelines: Program Plan for New and Existing Buildings,* FEMA 445, Federal Emergency Management Agency, Washington, D.C.

[7] Bertero VV. 1997, Performance-based seismic engineering: a critical review of proposed guidelines. In: Proceedings of the International Workshop on Seismic Design Methodologies for the Next Generation of Codes. Bled/Slovenia.

[8] Biggs JM. 1964 Book:- Introduction to structural dynamics. USA, Publisher: McGraw-Hill.

[9] CEN 1995, *Eurocode 8: Earthquake Resistant Design of Structures - Part 1: General Rules*,

ENV1998-1, Parts 1-3, CEN, Brussels, Belgium.

[10] CEN 2003, *Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings*, prEN 1998-1, Doc CEN/TC250/SC8/N335, Comité Européen de Normalisation, Brussels, Belgium.

[11] Chan CM & Zou XK, 2002, *Optimal inelastic drift design of reinforced concrete structures under pushover loading*. In: The second China–Japan–Korea joint symposium on optimization of structural and mechanical systems.

[12] Chopra AK. 1995, *Dynamics of structures—theory and applications to earthquake engineering*. New Jersey: Prentice-Hall.

[13] Chopra AK & Chintanapakdee C, 2001 Comparing response of SDF systems to near-fault and far-fault earthquake motions in the context of spectral regions. Earthquake Engineering and Structural Dynamics; 30:1769–1789.

[14] Computers and Structures SAP2000: Three Dimensional Static and Dynamic Finite Element Analysis and Design of Structures‖, Computers and Structures Inc., Berkeley, California, U.S.A.

[15] Court AB & Kowalsky MJ, 1998, *Performance-based engineering of buildings – A displacement design approach*.

[16] CPAMI. 2002. Code requirements for structural concrete. Taiwan: Construction and Planning Agency, Ministry of the Interior, CPRMI publisher.

[17] Fajfar P. 1999. Capacity spectrum method based on inelastic demand spectra. Earthquake Engineering and Structural Dynamics; pages 979–993.

[18] Fajfar, P. and Krawinkler, H. 1997. *Seismic Design Methodologies for the Next Generation of Codes*, Proceedings of International Workshop held in Bled (Slovenia), Balkema, The Netherlands.

[19] ICC, 2001, *International Performance Code for Buildings and Facilities,* International Code Council, Whittier, California.

[20] ICBO 1997. *Uniform Building Code - 1997 Edition, Vol. 2: Structural Engineering Design Provisions*, International Conference of Building Officials, Whittier, California, U.S.A.

[21] Kowalsky MJ, Priestley MJN & MacRae GA. 1994. Displacement-based design, a methodology for seismic design applied to single degree of freedom reinforced concrete structures. Report no. SSRP-94/16. San Diego (La Jolla, CA): Structural Systems Research, University of California.

[22] Loh CH & Hwang YR. 2002. *Seismic hazard analysis of Taiwan area — considering multiple ground motion parameters*. Report no. NCREE-02-032. Taipei (Taiwan): National Center for Research on Earthquake Engineering;

[23] Mander J.B., 2001, *Future directions in seismic design and performance-based engineering,* Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, NZSEE 2001 Conference

[24] IS 1893: 2002

[25] IS 456: 2000

[26] SP:34

[27] IS 13920