PERFORMANCE BASED SEISMIC DESIGN OF BUILDINGS

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Submitted by

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CERTIFICATE

This is to certify that thesis entitled, **"Performance Based Seismic Design of Buildings"**, being submitted by Mr. **Sudhanshu Sood,** Roll No 800822009 in partial fulfillment for the award of degree of **Master of Engineering in Civil (Structure) Engineering** in **Civil Engineering Department** of **Thapar University, Patiala**, is a bonafide work carried out by him under the work carried out by him under my guidance and supervision and that no part of this thesis has been submitted for the award of any other degree.

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ABSTRACT

A performance-based design is aimed at controlling the structural damage based on precise estimations of proper response parameters. Performance-based seismic design explicitly evaluates how a building is likely to perform; given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

In this present study two R.C buildings, one symmetrical and one unsymmetrical in plan (designed according to IS 456:2000) are analysed using Pushover Analysis and redesigning by changing the main reinforcement of various frame elements and again analyzing. The pushover analysis has been carried out using SAP2000, a product of Computers and Structures International. A total of 24 cases for a particular four storey 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 storey levels.

The results of analysis are compared in terms of base shear, storey drift, spectral acceleration, spectral displacement and storey displacements. The best possible combination of reinforcement that is economical, effective and whose damage is limited to Grade 2 (slight structural damage, moderate nonstructural damage) in order to enable Immediate Occupancy is determined and is termed as Performance Based Design.

The effect of providing shear walls, on the performance of RC framed building, is also studied using pushover analysis.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

Page No.

1.1 GENERAL

Amongst the natural hazards, earthquakes have the potential for causing the greatest damages. Since earthquake forces are random in nature & unpredictable, the engineering tools needs to be sharpened for analyzing structures under the action of these forces. Performance based design is gaining a new dimension in the seismic design philosophy wherein the near field ground motion (usually acceleration) is to be considered. Earthquake loads are to be carefully modeled so as to assess the real behavior of structure with a clear understanding that damage is expected but it should be regulated. 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 increasing lateral forces with an invariant height wise distribution until the preset performance level (target displacement) is reached. The promise of performance-based seismic engineering (PBSE) is to produce structures with predictable seismic performance. To turn this promise into a reality, a comprehensive and well-coordinated effort by professionals from several disciplines is required.

Performance based engineering is not new. Automobiles, airplanes, and turbines have been designed and manufactured using this approach for many decades. 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 revised to incorporate the lessons learned from the experimental evaluations. Once the cycle 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 automobiles which are virtually identical in their mechanical characteristics are produced following each performance-based design exercise.

What makes performance-based seismic engineering (PBSE) different and more complicated is that in general this massive payoff of performance-based design is not available. That is, except for large-scale developments of identical buildings, each building designed by this process is virtually unique and the experience obtained is not directly transferable to buildings of other types, sizes, and performance objectives. Therefore, up to now PBSE has not been an economically feasible alternative to conventional prescriptive code design practices. Due to the recent advances in seismic hazard assessment, PBSE methodologies, experimental facilities, and computer applications, PBSE has become increasing more attractive to developers and engineers of buildings in seismic regions. It is safe to say that within just a few years PBSE will become the standard method for design and delivery of earthquake resistant structures. In order to utilize PBSE effectively and intelligently, one need to be aware of the uncertainties involved in both structural performance and seismic hazard estimations.

The recent advent of performance based design has brought the nonlinear static pushover analysis procedure to the forefront. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are identified. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

1.2 NEED OF PERFORMANCE BASED SEISMIC DESIGN

From the effects of significant earthquakes (since the early 1980s) it is concluded that the seismic risks in urban areas are increasing and are far from socio-economically acceptable 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: (1) the development of more reliable seismic standards and code provisions than those currently available and (2) their stringent implementation for the complete engineering of new engineering facilities [9].

A performance-based design is aimed at controlling the structural damage 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 [6].

With an emphasis on providing stakeholders the information needed to make rational business or safety-related decisions, practice has moved toward predictive methods for assessing potential seismic performance and has led to the development of performance based engineering methods for seismic design.

1.3 HISTORY

Performance-based design of buildings has been practiced since early in the twentieth century, England, New Zealand, and Australia had performance-based building codes in place for decades [25]. The International Code Council (ICC) [19] in the United States 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 formed to facilitate international discussion of performance-based regulatory systems with a focus on identifying public policies, regulatory infrastructure, education, and technology issues related to implementing and managing these systems.

In 1989, the FEMA-funded project was launched to develop formal 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, [4] 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 generalized with descriptions of overall damage states with titles of Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. These levels were intended 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 avoidance of collapse. Following the Northridge event, the Structural Engineers Association of California (SEAOC, 1995) developed a PBSD process, known as Vision 2000 [32], which was more generalized than that contained in FEMA 273 but used similarly defined performance objectives.

Over the 10-year period after publication of FEMA 273, its procedures were reviewed and refined and eventually published 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 considered to represent the first generation of performance-based seismic design procedures.

1.4 PERFORMANCE BASED EARTHQUAKE ENGINEERING (PBEE)

Performance based earthquake engineering implies design, evaluation, construction, monitoring the function and maintenance of engineered facilities whose performance under seismic loads responds to the diverse needs and objectives of owners users and society. It is based on the premise 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 [7].

PBEE is a desirable concept whose implementation has a long way to go. There are legal and professional barriers but there are also many questions whether PBEE will be able to deliver its promises. It promises engineered structures whose performance can be quantified and confirmed to the owner’s desires.PBEE implies, for example, accepting damage in seismic events, if that proves the most economic solution. This requires, however, that structural engineers be able to predict 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 characterization and predictions, often based on a higher level of technology than has been used in the past.

1.5 PERFORMANCE-BASED SEISMIC DESIGN

Performance-based seismic design explicitly evaluates how a building is likely to perform; given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response [4],[6]. In performance-based design, identifying and assessing the performance capability of a building is an integral part of the design process, and guides the many design decisions that must be made. Figure 1.1 shows a flow chart that presents the key steps in the performance-based design process. It is an iterative process that begins with the selection of performance objectives, followed by the development of a preliminary design, an assessment as to whether or not the design meets the performance objectives, and finally redesign and reassessment, if required, until the desired performance level is achieved.

Performance-based design begins with the selection of design criteria stated in the form of one or more performance objectives. Each performance objective is a statement of the acceptable risk of incurring specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. Losses can be associated with structural damage, nonstructural damage, or both. They can be expressed in the form of casualties, direct economic costs, and downtime (time out of service), resulting from damage. Methods for estimating losses and communicating these losses to stakeholders are at the heart of the evolution of performance-based design.

Once the performance objectives are set, a series of simulations (analyses of building response to loading) are performed to estimate the probable performance of the building under various design scenario events. In the case of extreme loading, as would be imparted by a severe earthquake, simulations may be performed using nonlinear analysis techniques. If the simulated performance meets or exceeds the performance objectives, the design is complete. If not, the design is revised in an iterative process until the performance objectives are met. In some cases it may not be possible to meet the stated objective at reasonable cost, in which case, some relaxation of the original objectives may be appropriate.

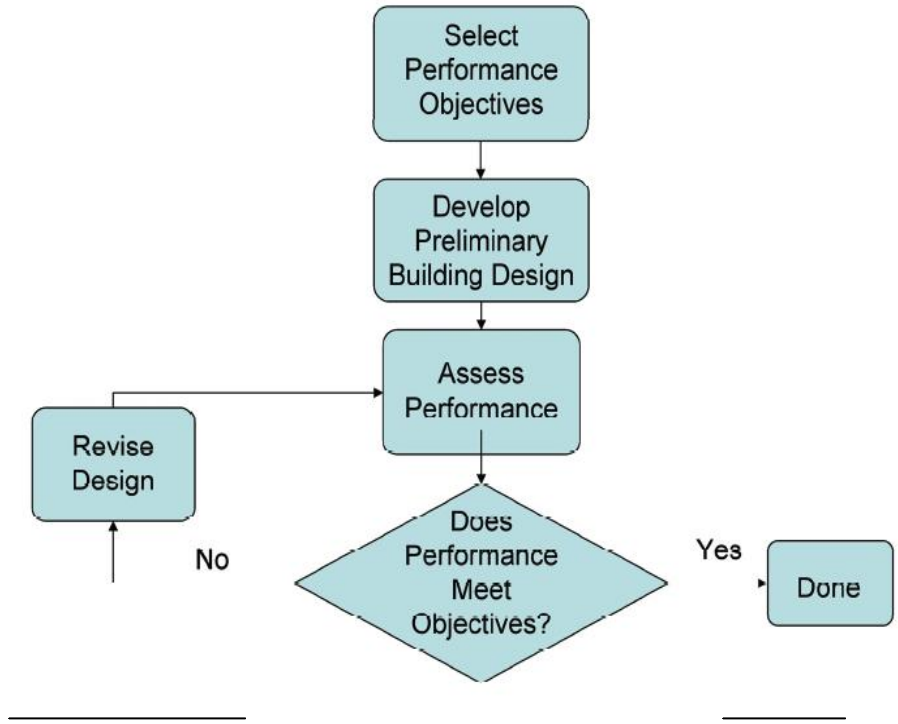

Figure 1.1 Performance-based design flow diagram [4]

PBSD permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes.

The goal of performance-based seismic design is to ensure that performance objectives are satisfied. That is, the structure will perform in a desired manner under various intensity of earthquake loading. According to the framework of performance-based design (SEAOC 2000 [32]), single or multiple performance objectives are selected at first according to seismic design code and the requirement of the owner in the conceptual design phase. When implementing a direct displacement-based approach, displacement parameters such as the top displacement 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. In the conceptual design step, layout of the structure is then determined without numerical analysis. Conceptual design guide [7] and energy balanced equation may be useful for engineering judgment. A successful conceptual design could hopefully reduce the impact of uncertainties on the real structural behavior.

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 method followed by the check of performance objectives and

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

The result obtained by the latter is believed to be closer to the final design and requires less computational effort. Verification of performance objectives employing non-linear pushover or non-linear time-history analysis is finally carried 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 phase of performance-based design is an iterative procedure between design and verification, in order to save computational 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 objectives need to be selected depends on if that performance objective is the main concern of the users and owners and if quantification of the performance acceptable limit is reliable.

1.6 ADVANTAGES OF PERFORMANCE-BASED SEISMIC DESIGN

In contrast to prescriptive design approaches, performance-based design provides a systematic methodology for assessing the performance capability of a building. It can be used to verify the equivalent performance of alternatives, deliver standard performance at a reduced cost, or confirm higher performance needed for critical facilities [6].

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 seismic design can be used to:

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

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

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

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

• Assess the potential performance of current prescriptive code requirements for new buildings, 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 seismic design offers society the potential to be both more efficient and effective in the investment of financial resources to avoid future earthquake losses. Further, the technology used to implement performance-based seismic design 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 PBSD over the methodologies used in the current seismic design code are summarized as the following six key issues [37]:

1. Multi-level seismic hazards are considered with an emphasis on the transparency of performance objectives.
2. Building performance is guaranteed through limited inelastic deformation in addition to strength and ductility.
3. Seismic design is oriented by performance objectives interpreted by engineering parameters as performance criteria.
4. An analytical method through which the structural behavior, particularly the nonlinear behavior is rationally obtained.
5. The building will meet the prescribed performance objectives reliably with accepted confidence.
6. The design will ensure the minimum life-cycle cost.

1.7 NEED AND OBJECTIVES OF THE PRESENT STUDY

1.7.1 Need

The Kutch Earthquake of January 26, 2001 in Gujarat, India, caused the destruction of a large number of modern 4 to 10-storied buildings. After this earthquake, doubts arose about our professional practices, building by-laws, construction materials, building codes and education for civil engineers and architects. It led to revision of the seismic code 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.

1.7.2 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 four-storey building, with the help of 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 four-storied RC framed building using Staad.Pro and analyzing the same using 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 and Beams of RC framed building at different storey levels (in elevation), using pushover analysis.
3. To study the effect of change of reinforcement in different Columns of RC 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, storey drift, spectral acceleration, spectral displacement and storey 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.

1.8 SCOPE OF THE PRESENT STUDY

The scope of present study aims at 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. A total of 24 cases for a particular four storey 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 storey levels.

The results of analysis are compared in terms of base shear, storey drift, spectral acceleration, spectral displacement and storey 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 unsymmetrical (L-shape) 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 seismic 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 PBSD to identify which methodologies of PBSD need to be incorporated into the current seismic design code. Then, a PBSD flowchart is presented. Finally, a draft of the proposed code is described. Transparent seismic design objectives for buildings of different use groups have been established qualitatively and interpreted quantitatively as performance criteria including drift limits. Site feasibility requirements, conceptual design scopes and basic rules have been proposed. Performance objective-oriented procedures for preliminary design and seismic performance evaluation have been presented. Suggestions on seismic performance criteria and the evaluation of existing buildings have been made. In order to provide clear, easy to follow guidelines, comparisons and case studies have also been conducted.

The performance-based seismic design code introduces a transparent platform in which the owners and designers can exchange their views on the expected seismic performance of the buildings under different levels of earthquakes. For buildings of different seismic use groups, specific performance goals are established without employing an importance factor. Performance levels are quantified through parameters associated with structural strength, stiffness and ductility. Conceptual design rules with focuses on redundancy and uniform continuity of strength, stiffness and ductility are specified. A performance objective-oriented preliminary design procedure is presented with consideration of flexibility. Preliminary checks on the interstory drift limit may help in finding the stiffness deficiencies earlier in the preliminary design stage and save some computational effort, particularly for steel structures.

The differences between seismic performance assessments of new buildings and those of existing buildings are pointed out. In engineering practice, member size standards and construction convenience are usually considered. A structure designed in this way usually has a lower ductility capacity than that specified in the code because structural ductility is not uniformly distributed. However, the structural strength and stiffness are usually higher than the demand. Therefore, the prescribed performance objective is usually satisfied.

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.

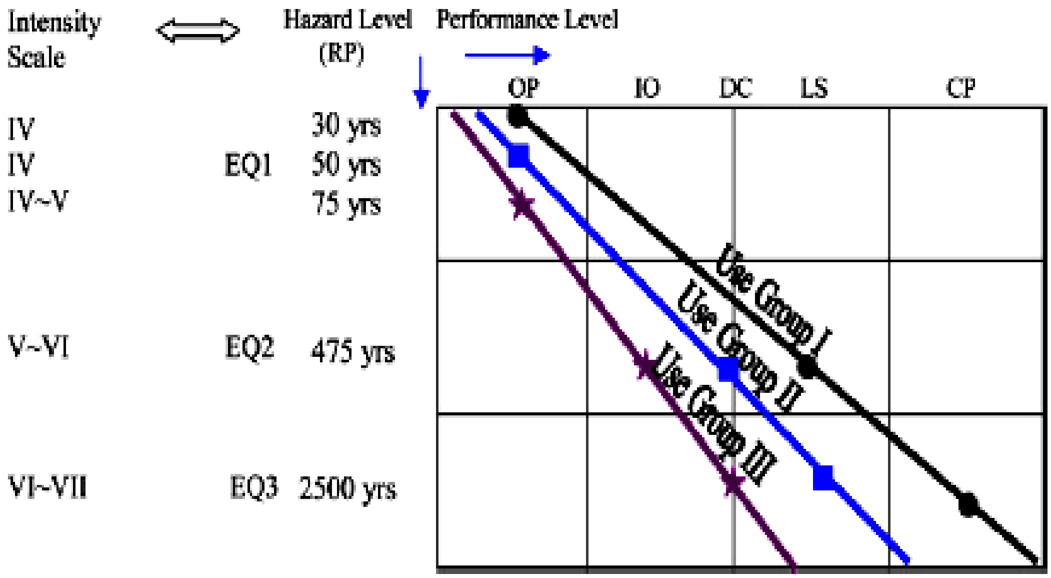

Figure 2.1 Performance objectives [37]

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-storey 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 $0.25g$, 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 S400 steel (characteristic yield strength of 400 MPa). Square column cross-sections (from 300 to 450 mm) were used, with reinforcement ratios not exceeding about 2% (the minimum reinforcement ratio for columns was 1%). Beam sections varied from 200×400 to 300×650 (mm^2).

Both elastic and inelastic (dynamic and static) analyses of the structure were carried out using "SAP 2000 Nonlinear" (Computers and Structures, 2000) [14], adopting a member-by-member modelling approach. Inelastic beam (and column) members were modelled as elastic elements with inelastic springs (plastic hinges) at their ends; the effective rigidity (EI_{ef}) of T-beams was taken equal to 40% the gross section rigidity (EI_g), while for columns 80% of EI_g was assumed. The moment curvature characteristics of the plastic hinges were estimated from section analysis using appropriate non-linear constitutive laws for concrete and steel (Penelis and Kappos, 1997) [27]; member strength and ductility were estimated on the basis of the nonlinear section analysis results.

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 storeys. 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.

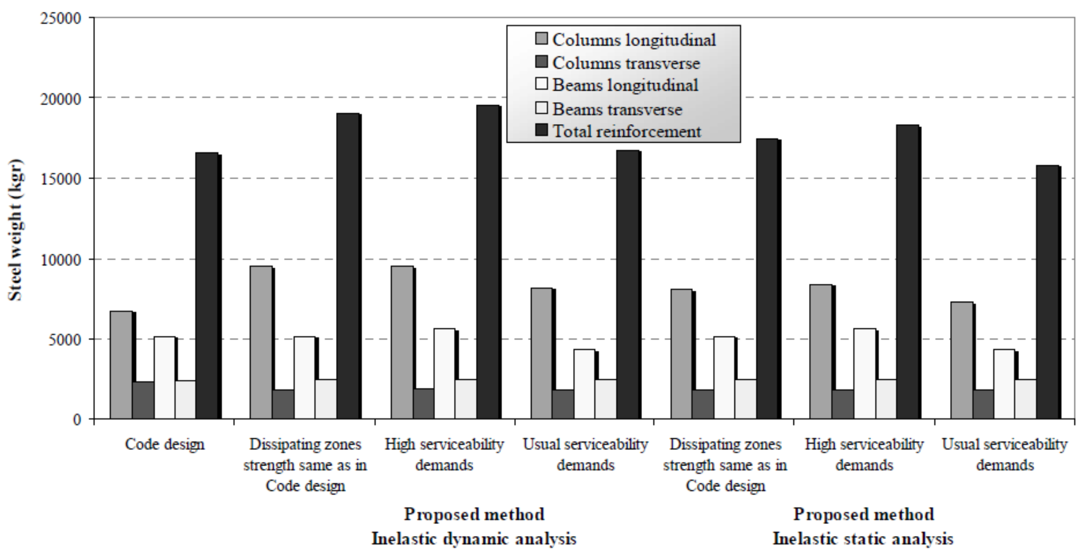

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Figure 2.2 Required amount of steel in beams and columns, for all designs

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 seismic 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 [37].
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 [14] 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 interstory 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 seismic design problem has been explicitly expressed in terms of the steel reinforcement design variables. Axial moment hinges and moment hinges should be considered in the nonlinear pushover analysis of a frame structure so that the behavior of columns and beams can be effectively modelled. Also, this Optimality Criteria design method developed is able to automatically shift any initial performance point to achieve the final optimal performance point. It is also believed that this 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 seismic design of structures. Direct displacement-based design requires a simplified procedure to estimate the seismic deformation of an inelastic SDF system, representing the first (elastic) mode of vibration of the structure. This step is usually accomplished by analysis of an “equivalent” linear system using elastic design spectra. In their work, an equally simple procedure is developed that is based on the well-known concepts of inelastic design spectra. This procedure provides: (1) accurate values of displacement and ductility demands, and (2) a structural design that satisfies the design criteria for allowable plastic rotation. In contrast, the existing procedure using elastic design spectra for equivalent linear systems is shown to underestimate significantly the displacement and ductility demands.
In this work, it is demonstrated that the deformation and ductility factor that are estimated in designing the structure by this procedure are much smaller than the deformation and ductility demands determined by nonlinear analysis of the system using inelastic design spectra. Furthermore, it has been shown that the plastic rotation demand on structures designed by this procedure may exceed the acceptable value of the plastic rotation.

Qiang Xue, et al (2003) presented a performance-based seismic design procedure, which is directly associated with pre-quantified performance criteria, by employing a displacement-based approach. A lower bound of yielding displacement of the structure to satisfy these performance criteria was proposed. This approach is general and applicable for any type of reduced response spectrum that taking into account of the inelastic behavior provided the spectrum reduction factor regarding each spectral region is given. The procedure can be extended to fulfill multiple performance objectives and to consider special effects such as the near-fault and accumulative damage. In the presented design procedure of the building, the k factor plays an important role in controlling the design strength. The simplicity and applicability of the proposed procedure is demonstrated through numerical examples. The proposed design procedure, which starts from the pre-quantified performance objectives, is transparent and straightforward to present the underlying concept of “performance-based design”. Non-linear time history analysis verified that this approach is applicable to control the target displacement to the performance acceptable limit. Its flexibility in considering special effect such as near-fault or strong motion duration and simplicity in a proposed multiple performance objectives design are demonstrated.

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).

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

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

Fig.2.3 identifies the five basic steps proposed for a next-generation seismic performance assessment. Unlike prior assessment procedures that addressed either structural damage or repair cost, three measures of seismic performance are proposed in the guidelines: 1) direct economic loss (repair cost), 2) indirect economic loss (downtime or business interruption), and 3) casualties (including injuries and death). Each of three performance measures is treated as a potential loss.

Section 2 of the paper introduces the three types of performance assessment that can be performed using the draft Guidelines and identifies the basic procedure for each.

Section 3 describes the five steps for seismic performance assessment that are identified in Fig 2.3.

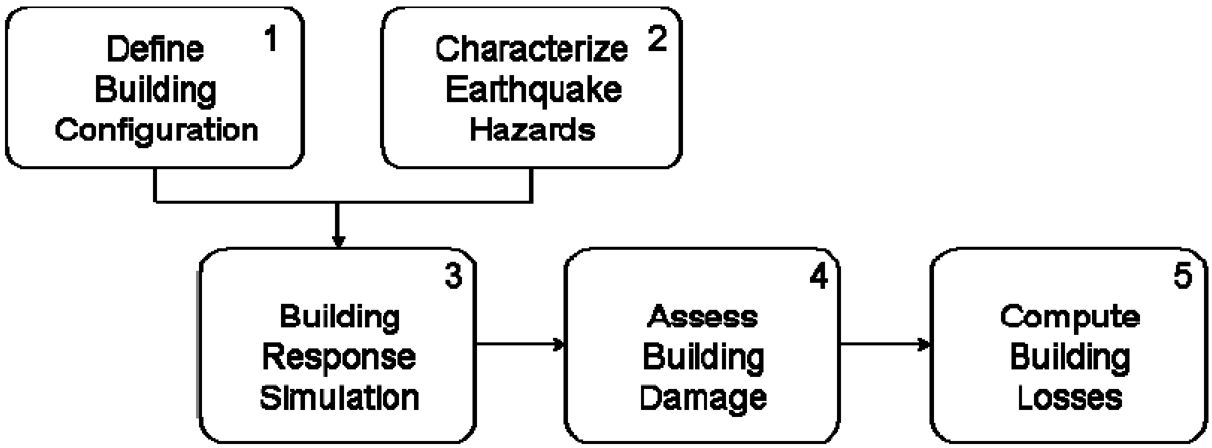

Figure 2.3 Procedure for Performance Assessment

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

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,

and 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.

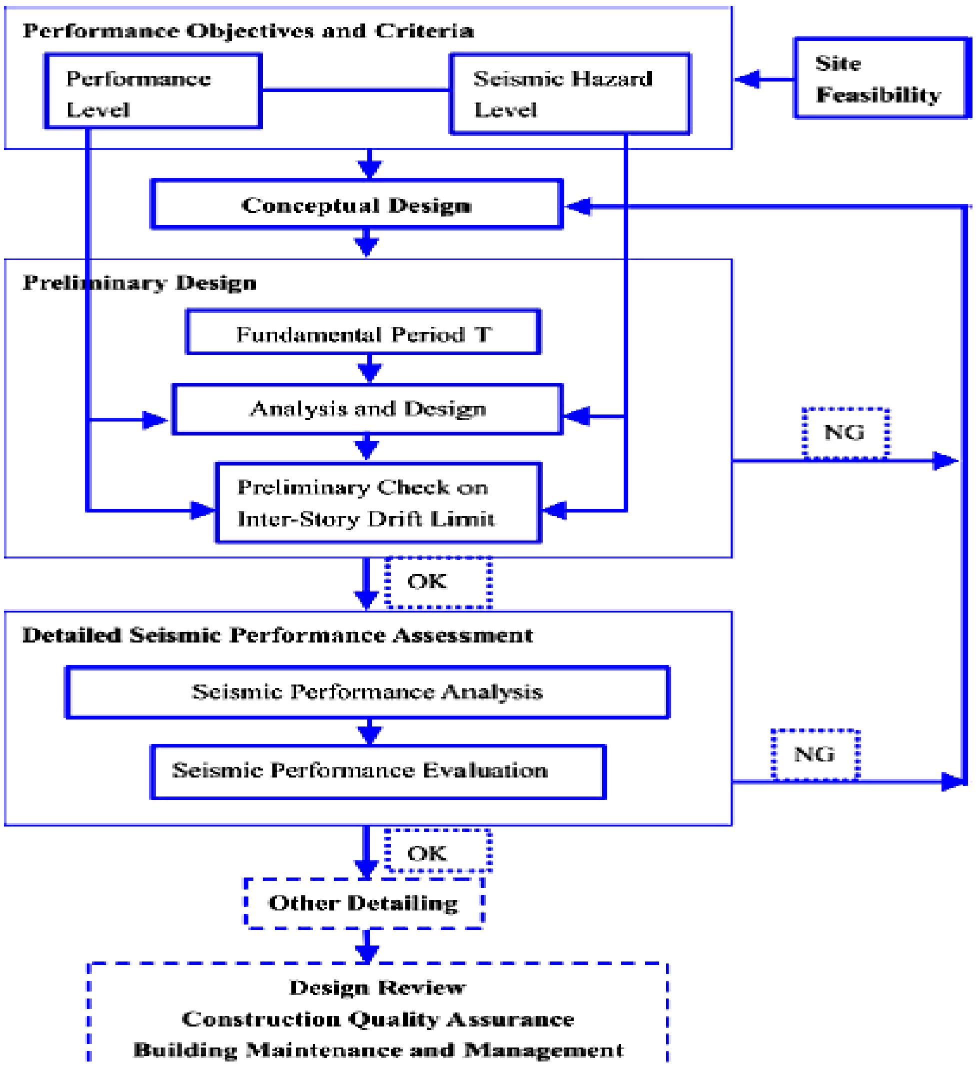

Figure 3.1 Performance Based Seismic Design for New Buildings [37]

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.

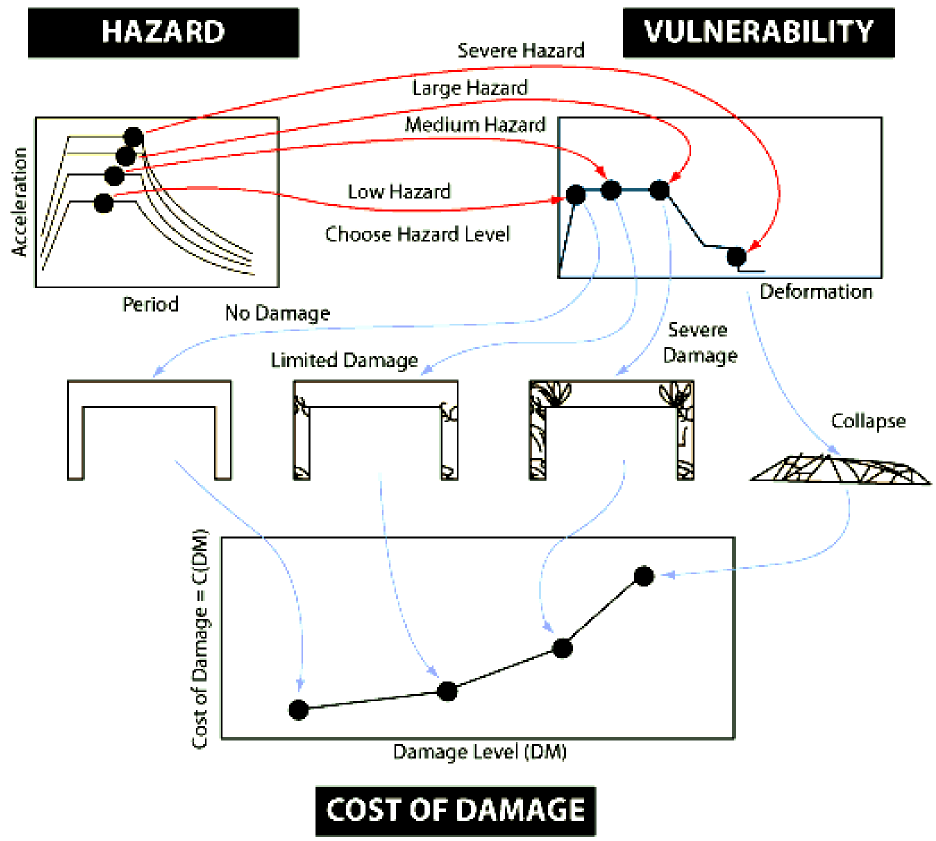

Figure 3.2 Performance Based Design Steps [34]

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.

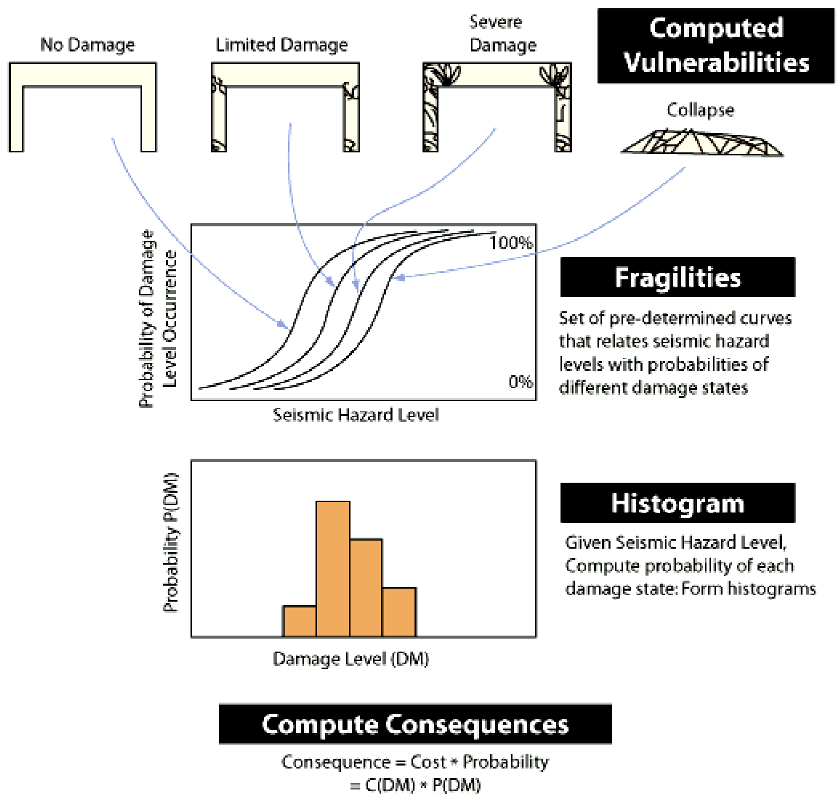

Figure 3.3 Computation of Risk [34]

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.

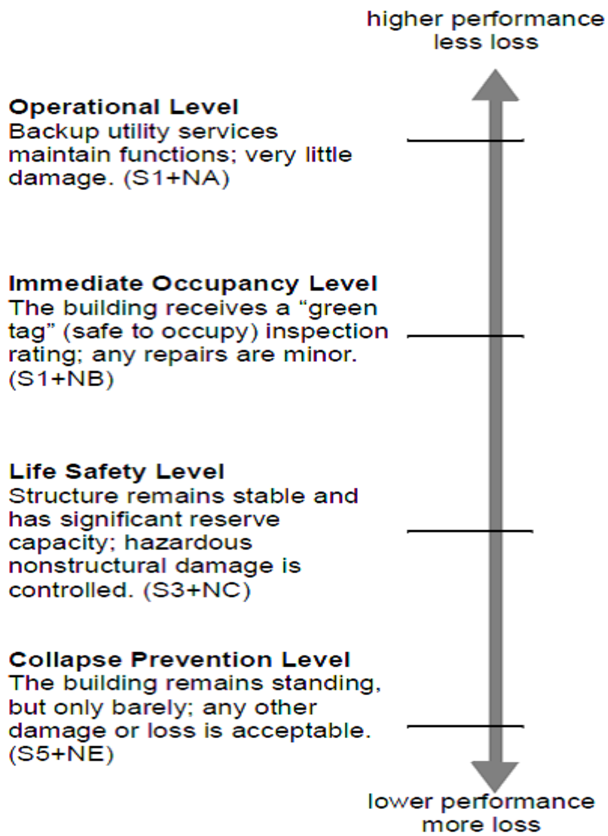

Figure 3.4 Building Performance Levels [4]

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 [4]

	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 [4]

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	Primary	Extensive	Extensive	Minor (<1/8"

Masonry Infill Walls		cracking and crushing; portions of face course shed.	cracking and some crushing but wall remains in place. No falling units. Extensive crushing and spalling of veneers at corners of openings.	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 [4]

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:

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

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:

$$P_R = \frac{1}{1 - e^{0.02\ 1\text{n}(1 - P_{E50})}} \qquad (3.2)$$

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:

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

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 [4]

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 [4]

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.

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

For $0 < T \leq 0.2T_0$

$$S_a = (S_{X1} / (B_1 T)) \text{ for } T > T_0 \tag{3.9}$$

where *To* is given by the equation

$$T_0 = (S_{X1} B_S) / (S_{XS} B_1) \tag{3.10}$$

where B_S and B_1 are taken from Table 3.6.

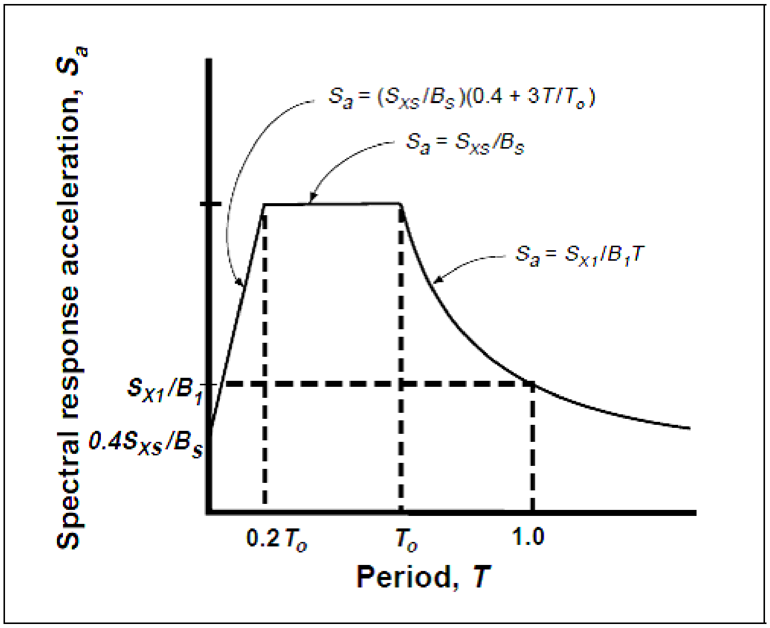

Figure 3.5 General response spectrum [4]

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.

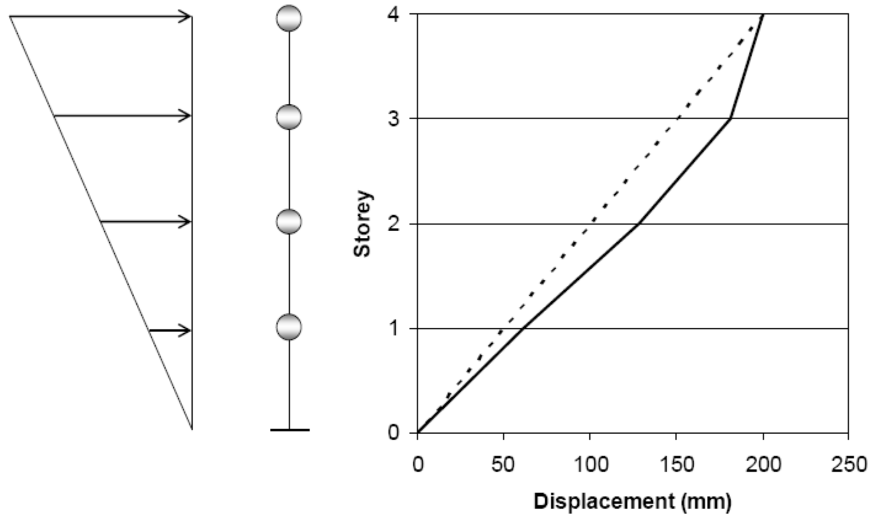

Figure 3.6 Inverted triangular Loading for Pushover Analysis

3.4.1 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:

- The force reduction factors recommended in codes of practice are approximate and do not necessarily represent the specific structure under consideration.
- 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.
- The mechanism that will most likely perpetuate collapse is unlikely to be that represented by the elastic action and deformation distribution.

- 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.

3.4.2 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].

3.4.2.1 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.

3.4.2.2 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.

3.4.3 PUSHOVER ANALYSIS GUIDELINES AS PER ATC-40

3.4.3.1 Basis of the Procedure

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.

3.4.3.2 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.

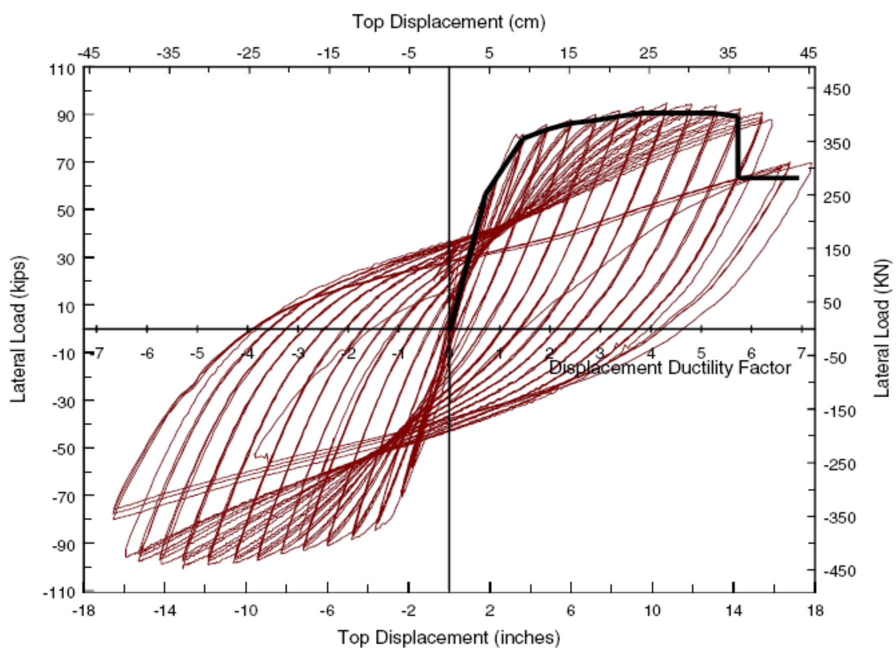

Figure 3.7 Backbone curve from actual hysteretic behavior

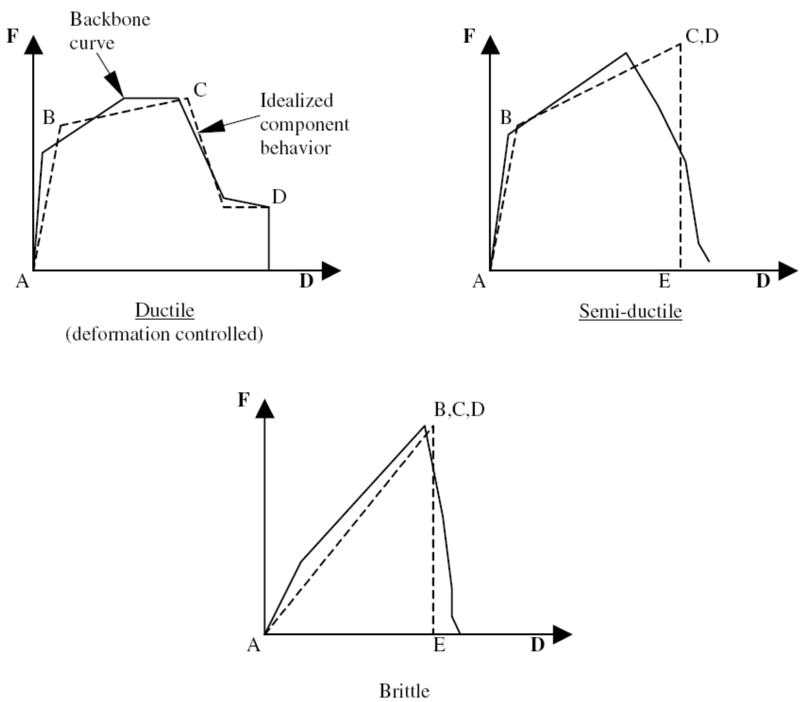

Figure 3.8 Idealized component behavior from backbone curves

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.

3.4.4 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.

3.4.4.1 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:

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

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:

$$\mathrm{Sa_ig} = \frac{2\pi}{T_i} Sv \qquad \mathrm{Sd_i} = \frac{T_i}{2\pi} Sv \tag{3.13}$$

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), δi (Roof Displacement) on the capacity (pushover) curve is converted to the corresponding point Sa_i, Sd_i on the capacity spectrum using the equations:

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

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

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:

$$\mathrm{PF}_1 = \frac{\sum_{i=1}^{n}(w_i\phi_{i1})/g}{\sum_{i=1}^{n}(w_i\phi_{i1}^2)/g} \tag{3.16}$$

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

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.

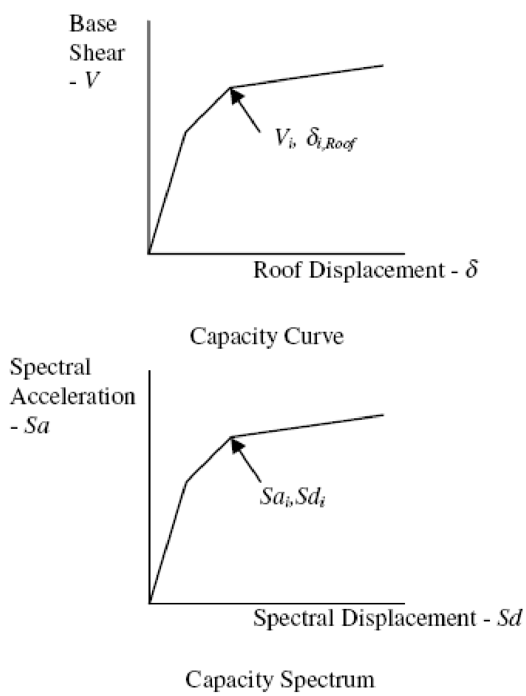

Figure 3.9 Capacity Spectrum Conversion

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:

$$\beta_{eff} = \lambda\beta_0 + 0.05 \tag{3.18}$$

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:

$$\beta_o = \frac{1}{4\pi}\frac{E_D}{E_{SO}} \tag{3.19}$$

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.

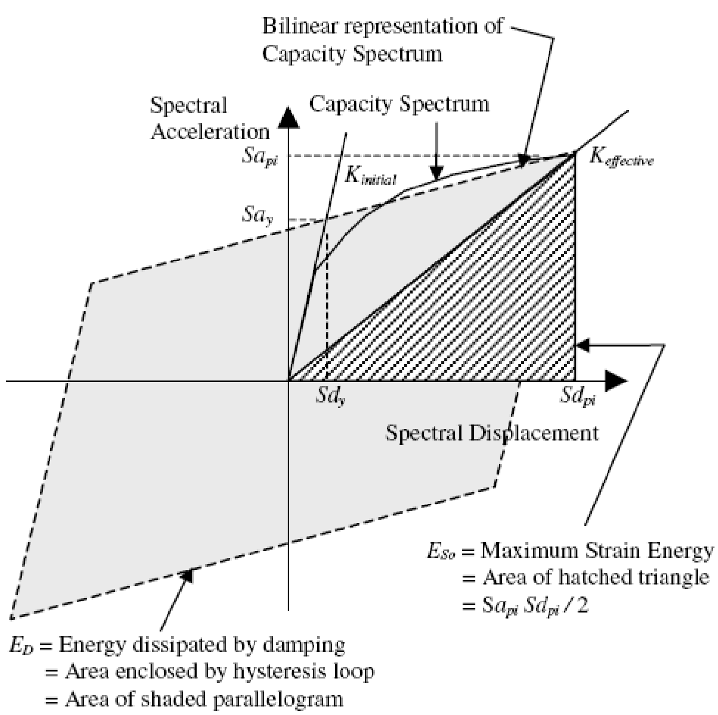

Figure 3.10 Derivation of Energy dissipated by Damping

Table 3.7: 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).

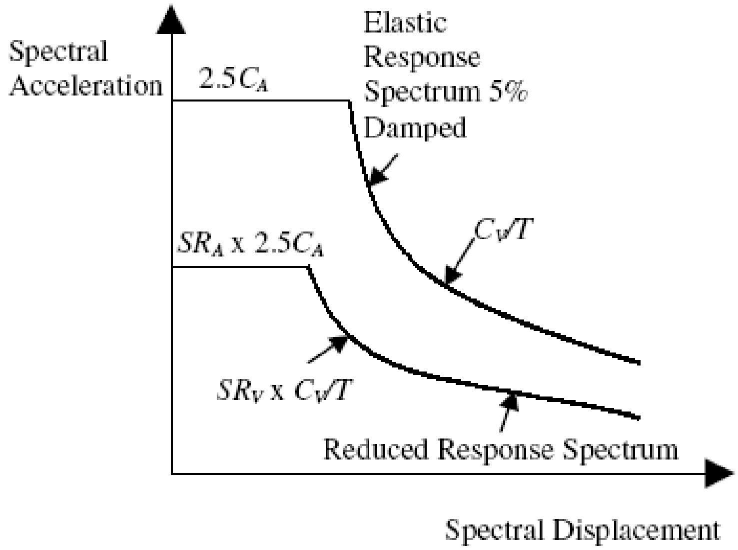

Figure 3.11 Reduced Response Spectrum

3.4.4.2 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.

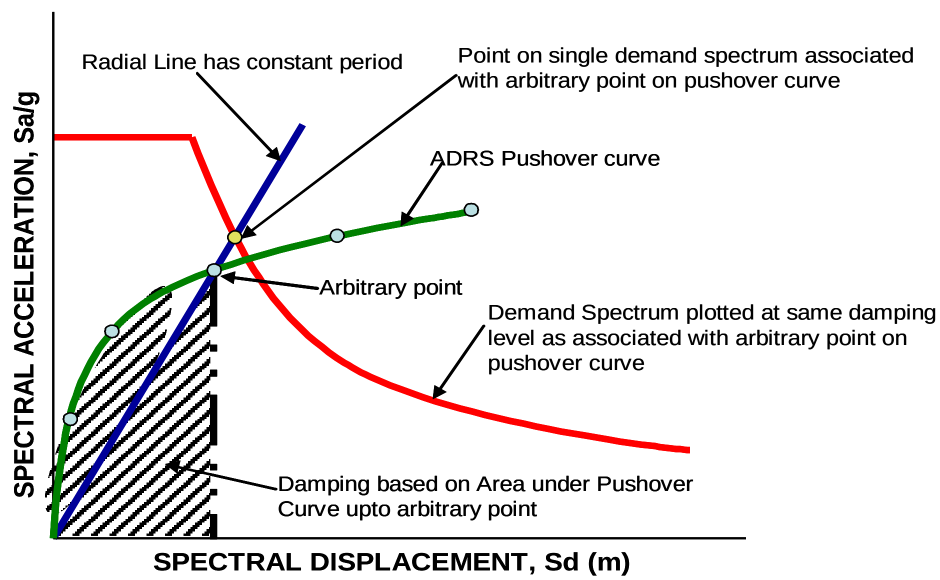

Figure 3.12 Capacity Spectrum Procedure C to Determine Performance Point

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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 10 x 8 m with 3.5m as height of each typical storey. It consists of 2 bays of 5m 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 Special Moment resisting frame. The plan of building is shown in fig. 4.1 and the front elevation is shown in fig. 4.2.

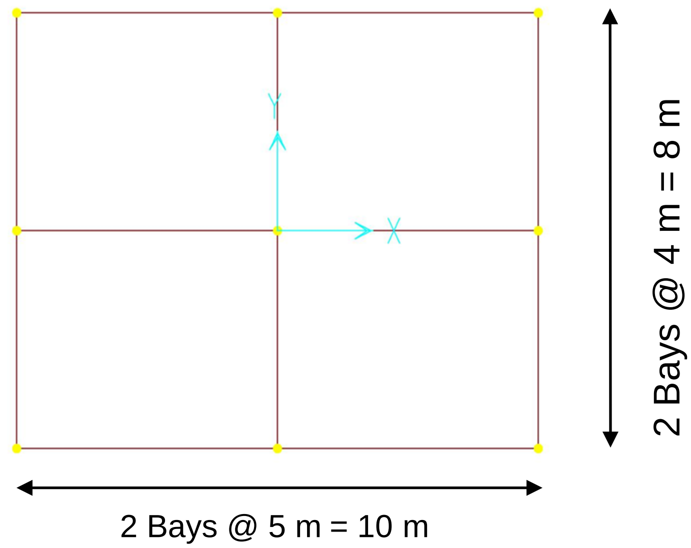


Figure 4.1 Plan of Building

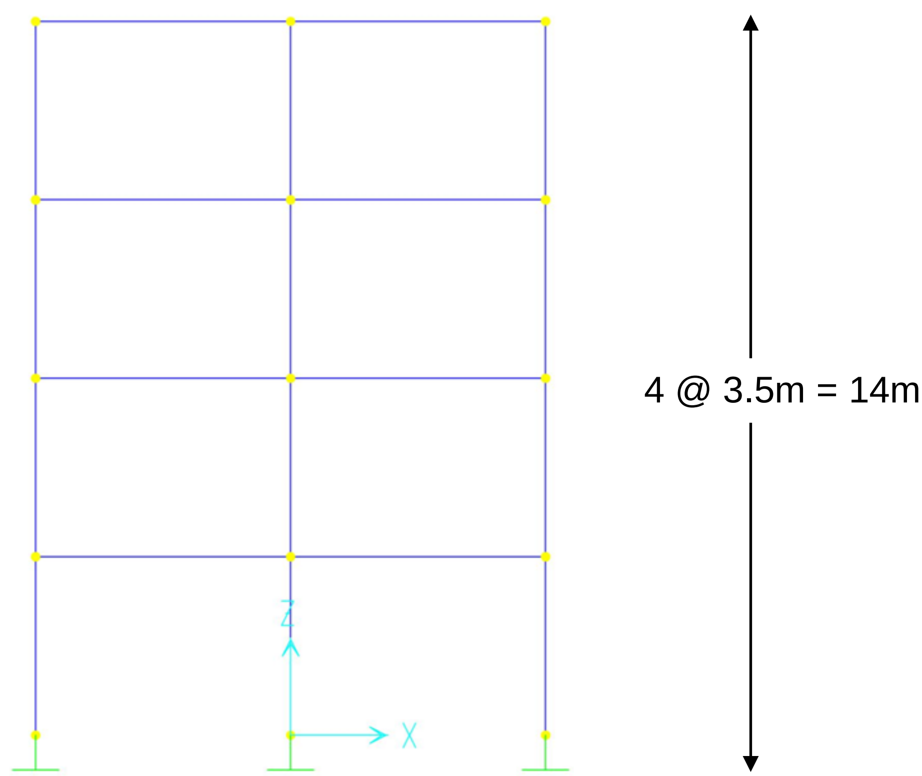


Figure 4.2 Elevation of Building

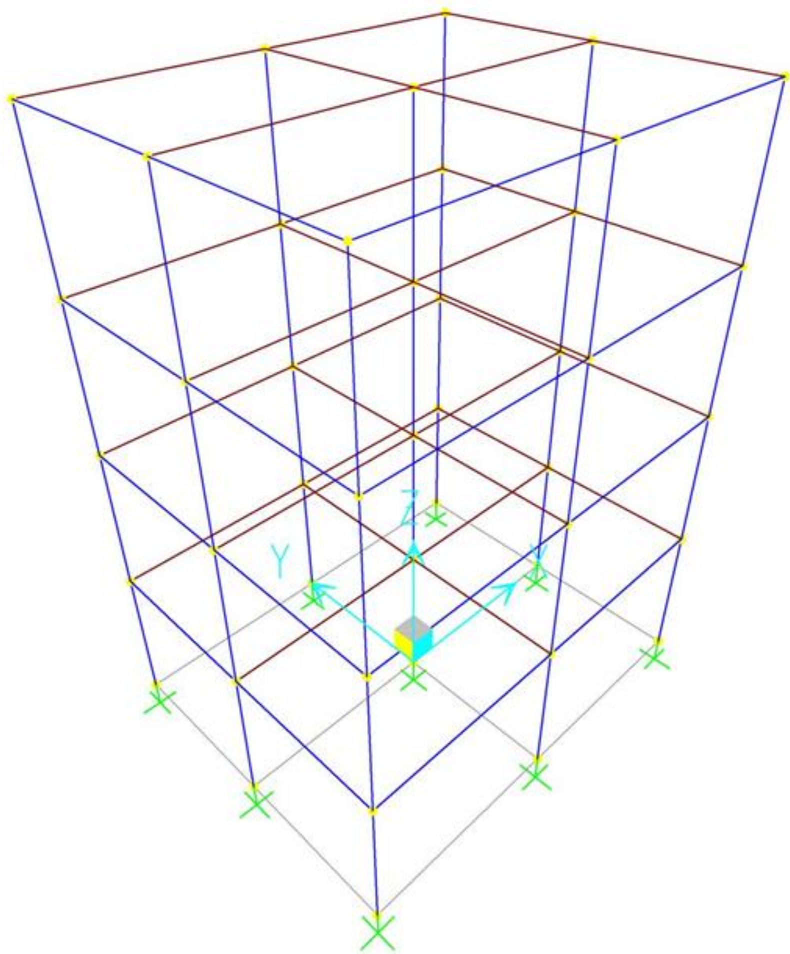


Figure 4.3 3D View of Building

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

The sectional properties of elements in case of the Basic structure are taken as follows:

Size of Column = 345 x 345mm

Size of Beam = 345 x 500 mm

Thickness of Slab = 125mm thick

4.5 LOADS CONSIDERED

The following loads were considered for the analysis of the building. The loads were taken in accordance with IS: 875.

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	0.125 x 25	3.125 kN/m^2
Weight of Mud Fuska	0.150 x 16	2.400 kN/m^2
Weight of Tiles	0.040 x 20	0.800 kN/m^2
Total Dead Load		**6.500 kN/m^2**

Floor Levels

Weight of Slab	0.125 x 25	3.125 kN/m^2
Weight of Screed	0.065 x 20	1.300 kN/m^2
Weight of Floor Finish	0.040 x 24	0.960 kN/m^2
Weight of partition Wall	-	1.500 kN/m^2
Total Dead Load		**7.000 kN/m^2**

- **Live Load**

Live load at all floor levels = **3.5 kN/m^2**

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:-

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

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 storey shall be equally distributed to the floors above and below the storey. 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:-

$$\mathbf{V_h} = A_h W$$

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:

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

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

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

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 ($\mathbf{V_h}$) computed is distributed along the height of the building as below:

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

Where,

$\mathbf{Q_i}$ = design lateral force at each floor level i

$\mathbf{W_i}$ = 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 storey level and was distributed at each node. The calculation of this force is illustrated below:

4.6.1 Calculation of seismic Weight of Structure

Seismic weight of roof is calculated as under:

Slab = 0.125 x 4 x 5 x 25 x 4 = 250 kN

Beams = 54 x 0.345 x 0.5 x 25 = 232.87 kN

Columns = 0.345 x 0.345 x 1.75 x 25 x 9 = 46.86 kN

Total = 529.73 kN

Seismic weight of one floor is calculated as under:

Slab = 0.125 x 4 x 5 x 25 x 4 = 250 kN

Beams = 54 x 0.345 x 0.5 x 25 = 232.87 kN

Columns = 0.345 x 0.345 x 3.5 x 25 x 9 = 93.73 kN

Total = 576.60 kN

Hence Total Seismic weight = 2259.5 kN

4.6.2 Calculation of base shear

The following parameters were taken:

Zone Factor, Z = 0.24

Importance Factor, I = 1.0

Response Reduction Factor = 5.0

Time Period is calculated from:

$TS = 0.09\ h/\sqrt{d} = .09x14/\sqrt{10} = 0.4$ seconds

Hence, $Sa/g = 2.5$ (For Medium Soil Conditions)

Hence, $A_h = (.24/2) \times (1/5) \times 2.5 = .06$

Thus $V_b = .06 \times 2259.5 = 131$ kN

$\sum W_j h_j^2 = 202714$

Now, $Qi = \dfrac{V_b\ Wi\ hi^2}{\sum Wi\ hi^2}$

Hence, $Q4 = (131 \times 529.73 \times 14^2)/202714 = 67.10$ kN

Similarly, $Q3 = 41.08$ kN

$Q2 = 18.26$ kN $\quad$ $Q1 = 4.56$ k

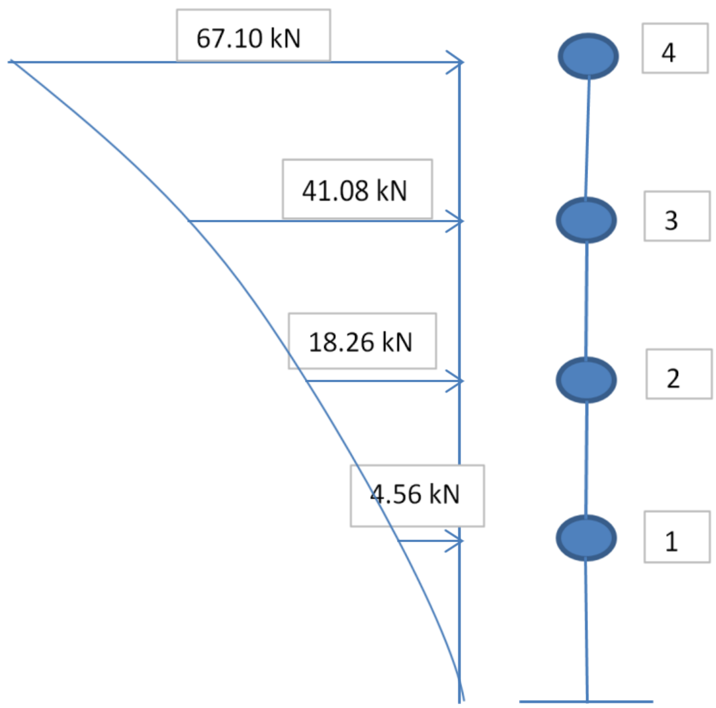


Figure 4.4 Applied Inverted Triangular Loading

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, etc.

4.7 ASSUMPTIONS

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 STAAD.Pro (according to I.S. 456:2000) for Dead Load and Live load case only for getting the reinforcement detail.

Table 4.1 Structural details (as per Analysis and Design on Staad.Pro)

Element	Dimension (m)	Reinforcement Area in mm^2
Corner Columns	0.345 x 0.345	452
Mid-face Columns	0.345 x 0.345	804
Interior Column	0.345 x 0.345	1260
Beams 1st storey	0.345 x 0.5	785 (top) 550 (bottom)
Beams 2nd storey	0.345 x 0.5	678 (top) 550 (bottom)
Beams 3rd storey	0.345 x 0.5	942 (top) 550 (bottom)
Beams 4th storey	0.345 x 0.5	678 (top) 550 (bottom)

4.8 PUSHOVER ANALYSIS USING SAP2000

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

1. Create the basic computer model (without the pushover data) as shown in Fig. 4.5. The graphical interface of SAP2000 makes this quick and easy task. Assigned sectional properties & applies all the gravity loads i.e. Dead load and Live load on the structure. For changing reinforcement, define frame section from the Define menu as shown in Fig. 4.6.

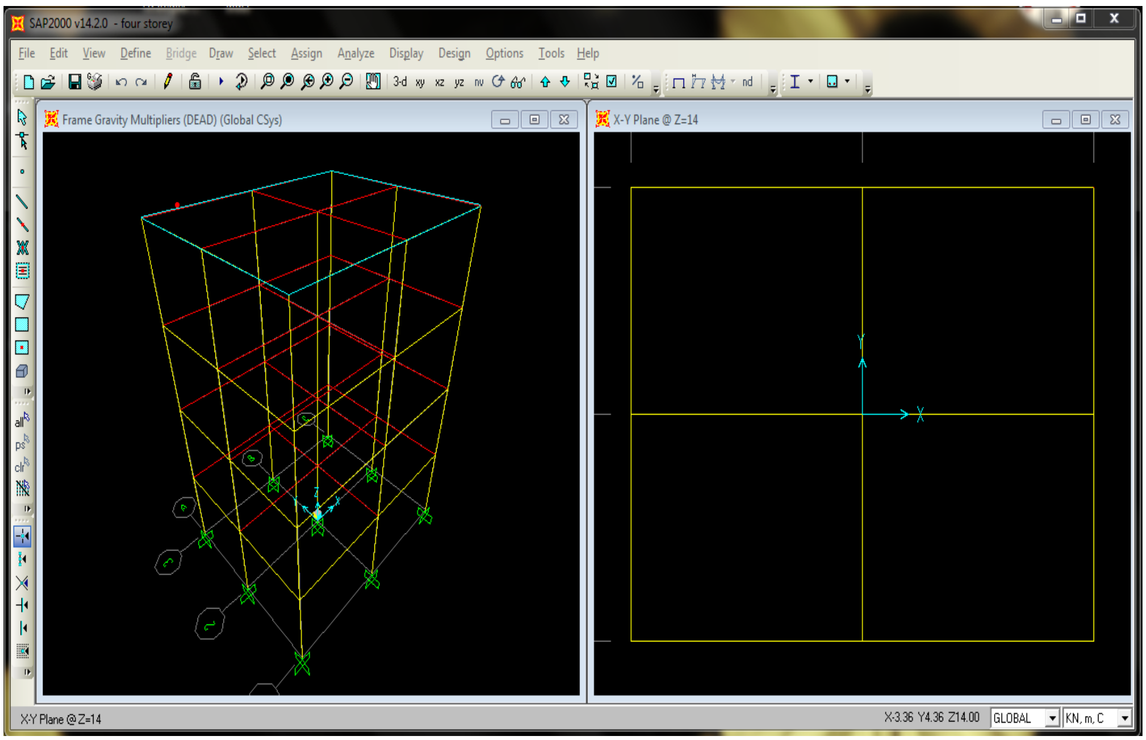


Figure 4.5 Basic Model in SAP2000

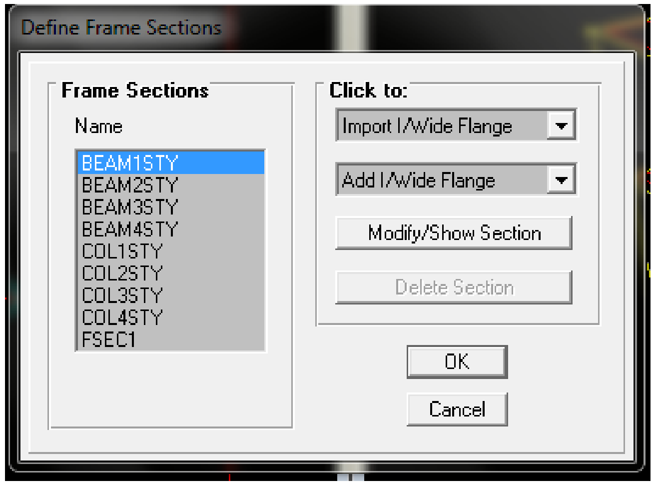


Figure 4.6 Defining Frame Sections

2. Define properties and acceptance criteria for the pushover hinges as shown in Figure 4.7. The program includes several built-in default hinge properties that are based on average values from ATC-40 for concrete members and average values from FEMA-273 for steel members. In this analysis, PMM hinges have been defined at both the column ends and M3 hinges have been defined at both the ends of all the beams.

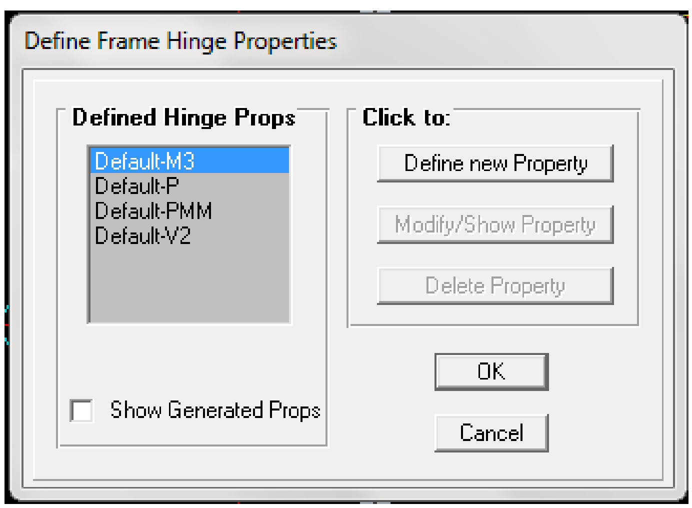


Figure 4.7 Defining Hinge Properties

3. Locate the pushover hinges on the model by selecting all the frame members and assigning them one or more hinge properties and hinge locations as shown in Figure 6.8.

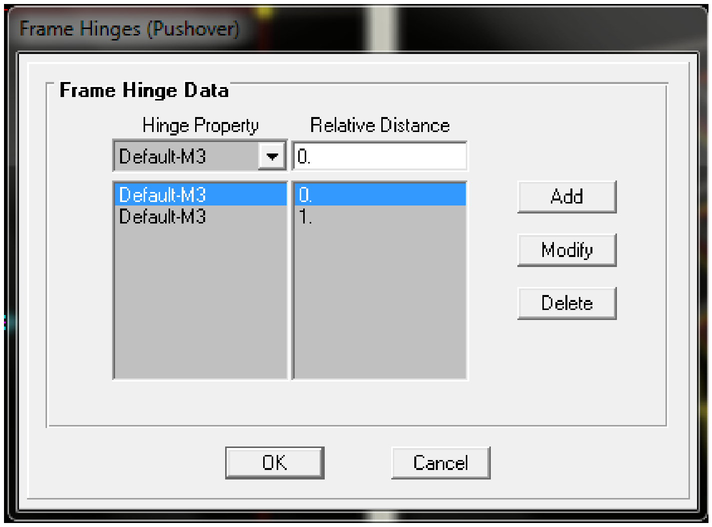


Figure 4.8 Assignment of Hinges

4. Define the pushover load cases, figure 4.9(a) and (b). In SAP2000 more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis. Typically the first pushover load case 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+0.25LL has been used. This combination has been defined as GRAV. The lateral loads, as calculated in 4.6.1, have been applied to a case called PUSHPAT.

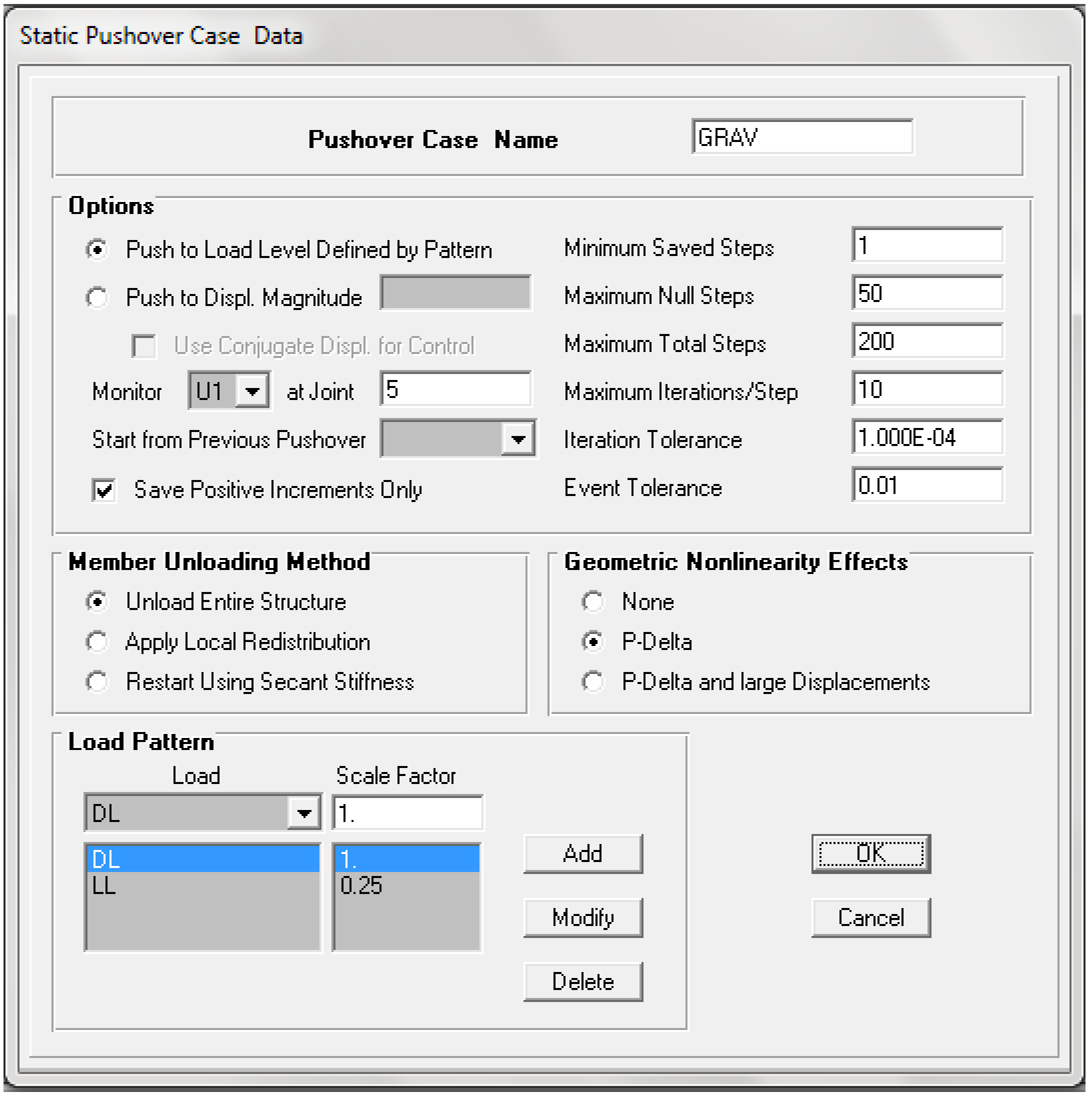


Figure 4.9(a) Defining Pushover Cases

Figure 4.9(b) Defining Pushover Cases

5. Run the basic static analysis. Then run the static nonlinear pushover analysis.
6. The Pushover curve was made for control nodes at each storey level. This was done by defining a number of pushover cases in the same analysis, and displacement was monitored for a different node in each case.
7. The pushover curve was obtained as shown in Fig. 4.10. A table was also obtained which gives the coordinates of each step of the pushover curve and summarizes the number of hinges in each state (for example, between IO and LS, or between D and E). This table is shown in Fig. 4.11.

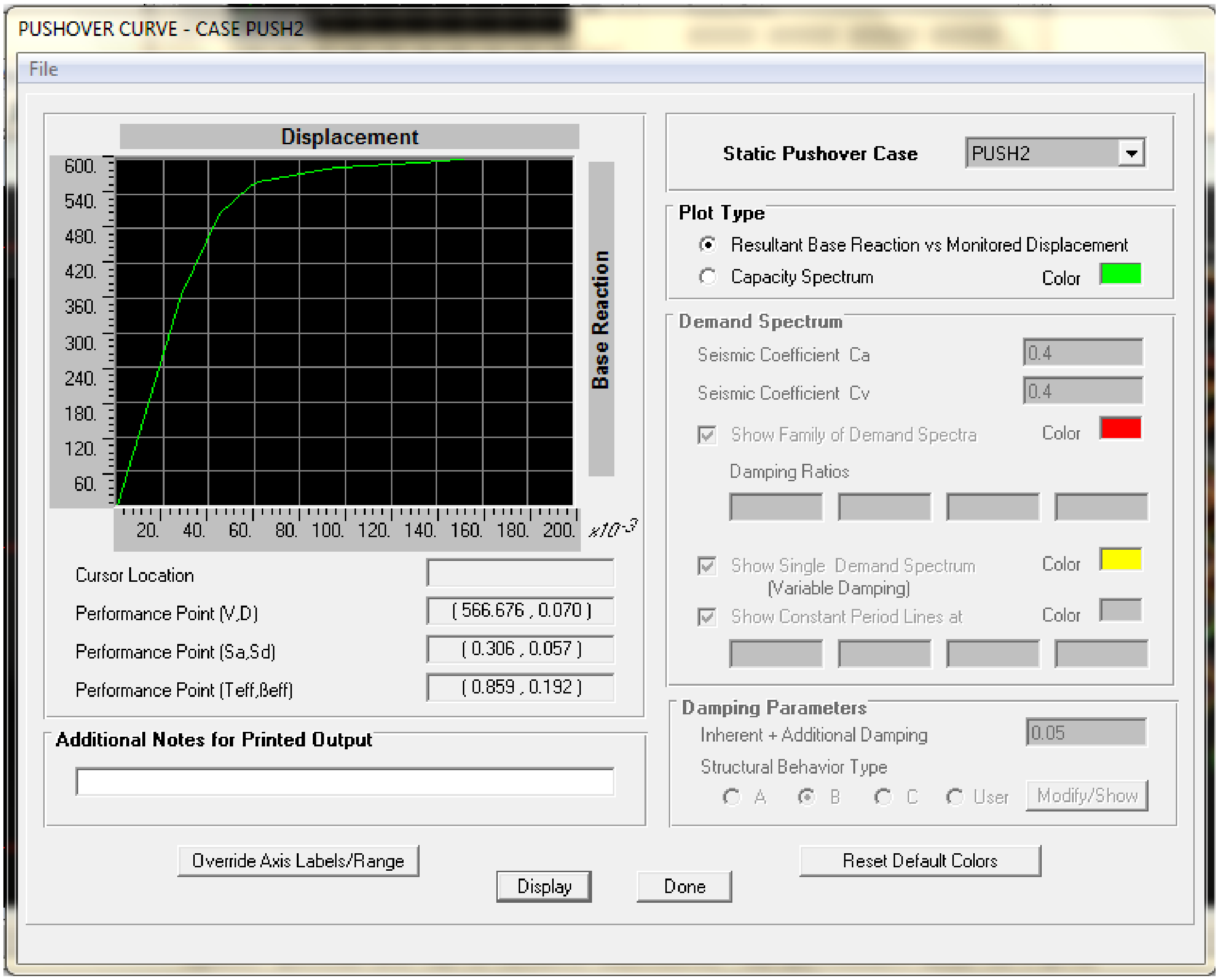
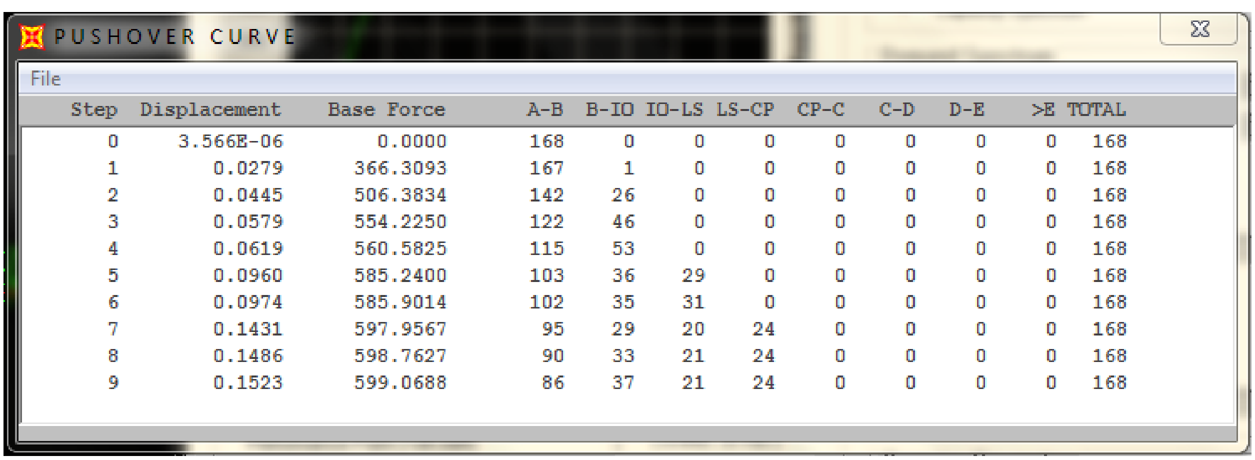


Figure 4.10 Pushover Curve



PUSHOVER CURVE

File

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	3.566E-06	0.0000	168	0	0	0	0	0	0	0	168
1	0.0279	366.3093	167	1	0	0	0	0	0	0	168
2	0.0445	506.3834	142	26	0	0	0	0	0	0	168
3	0.0579	554.2250	122	46	0	0	0	0	0	0	168
4	0.0619	560.5825	115	53	0	0	0	0	0	0	168
5	0.0960	585.2400	103	36	29	0	0	0	0	0	168
6	0.0974	585.9014	102	35	31	0	0	0	0	0	168
7	0.1431	597.9567	95	29	20	24	0	0	0	0	168
8	0.1486	598.7627	90	33	21	24	0	0	0	0	168
9	0.1523	599.0688	86	37	21	24	0	0	0	0	168

Figure 4.11 Tabular Data for Pushover Curve

8. The capacity spectrum curve obtained is shown in Fig. 4.12. The magnitude of the earthquake and the damping information on this form can be modified and the new capacity spectrum plot can be obtained immediately. The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve. Also, a table was 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 Fig. 4.13.

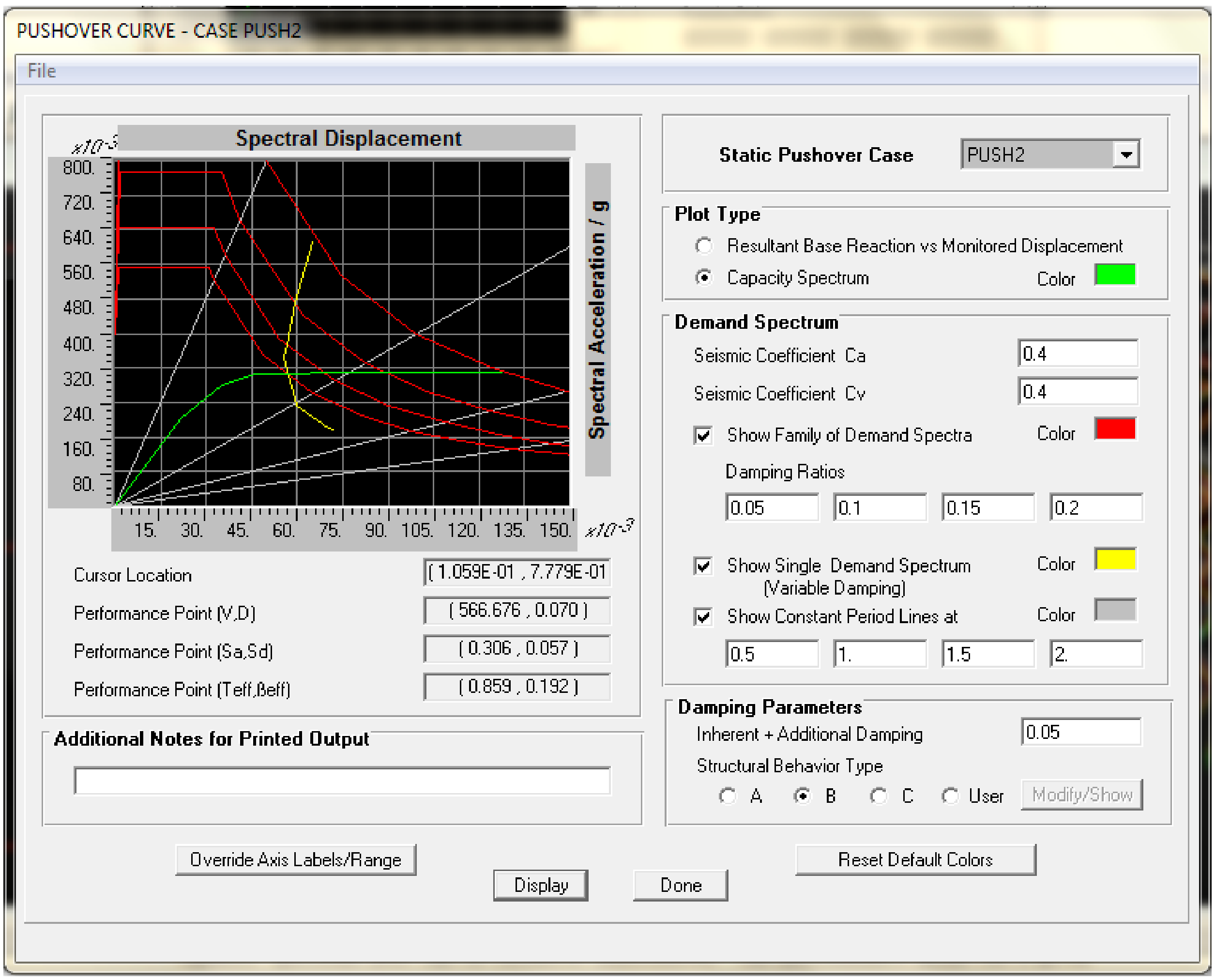


Figure 4.12 Capacity Spectrum Curve

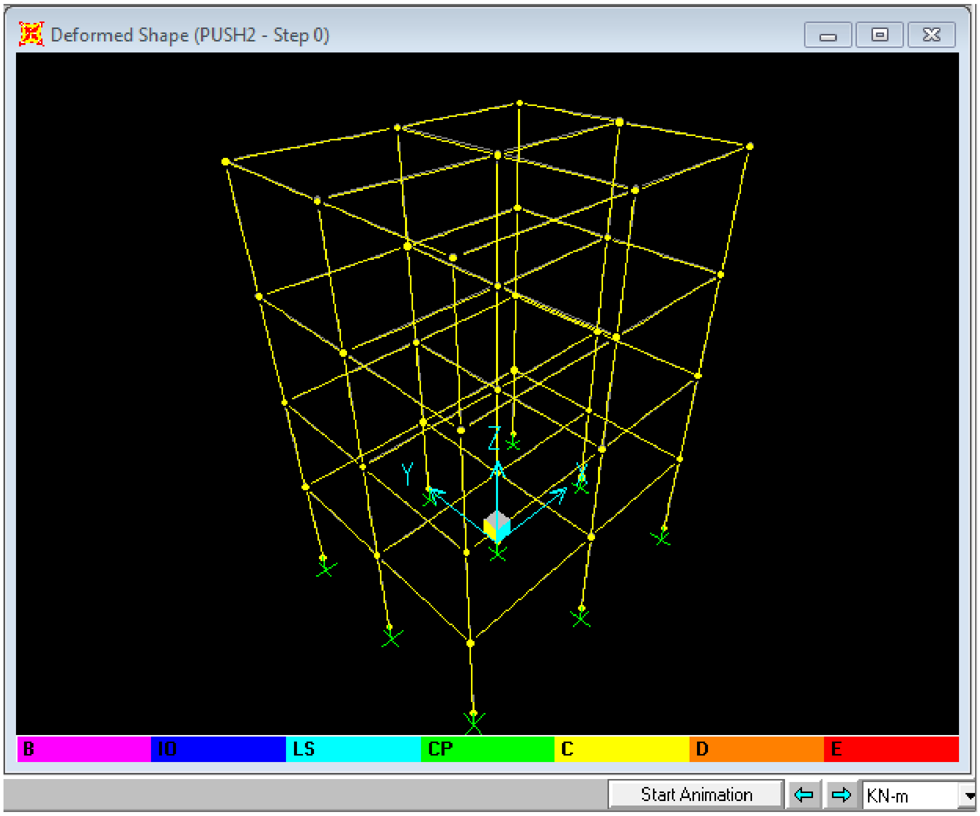
PUSHOVER CAPACITY/DEMAND COMPARISON

File

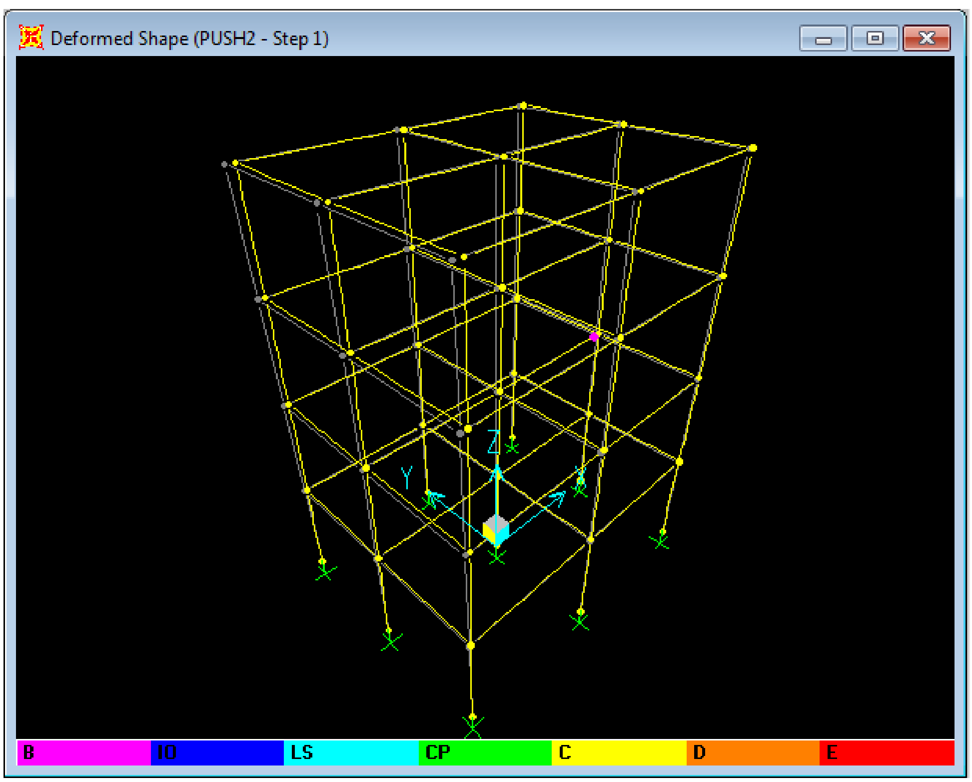
Step	Teff	ßeff	Sd (C)	Sa (C)	Sd (D)	Sa (D)	ALPHA	PF*Ø
0	0.656	0.050	0.000	0.000	0.065	0.610	1.000	1.000
1	0.656	0.050	0.022	0.203	0.065	0.610	0.830	1.286
2	0.710	0.096	0.035	0.280	0.059	0.473	0.833	1.269
3	0.782	0.151	0.046	0.304	0.056	0.371	0.838	1.251
4	0.808	0.171	0.050	0.306	0.056	0.344	0.844	1.247
5	1.012	0.257	0.079	0.309	0.060	0.234	0.873	1.222
6	1.020	0.259	0.080	0.309	0.060	0.232	0.874	1.221
7	1.250	0.287	0.120	0.309	0.070	0.181	0.890	1.193
8	1.275	0.288	0.125	0.309	0.072	0.177	0.891	1.190
9	1.292	0.291	0.128	0.309	0.072	0.174	0.892	1.188

Figure 4.13 Tabular Data for Capacity Spectrum Curve

9. The pushover displaced shape and sequence of hinge information on a step-by-step basis was obtained and is shown in the Figure 4.14(a) to 4.14(e).
10. Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The types of output available in this form include joint displacements at each step of the pushover, frame member forces at each step of the pushover, and hinge force, displacement and state at each step of the pushover.

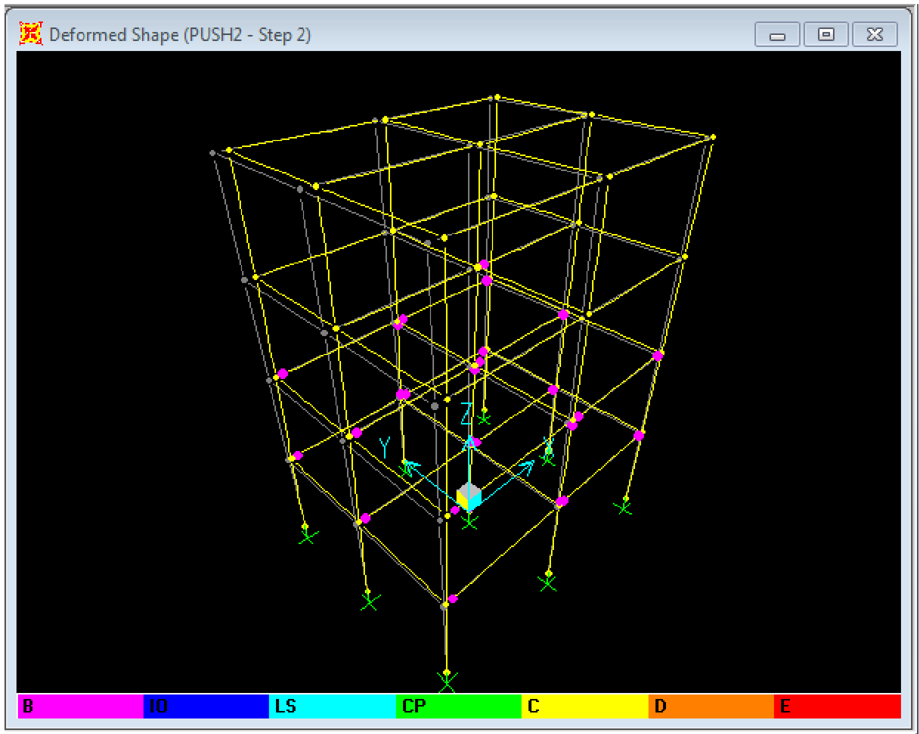


STEP 0

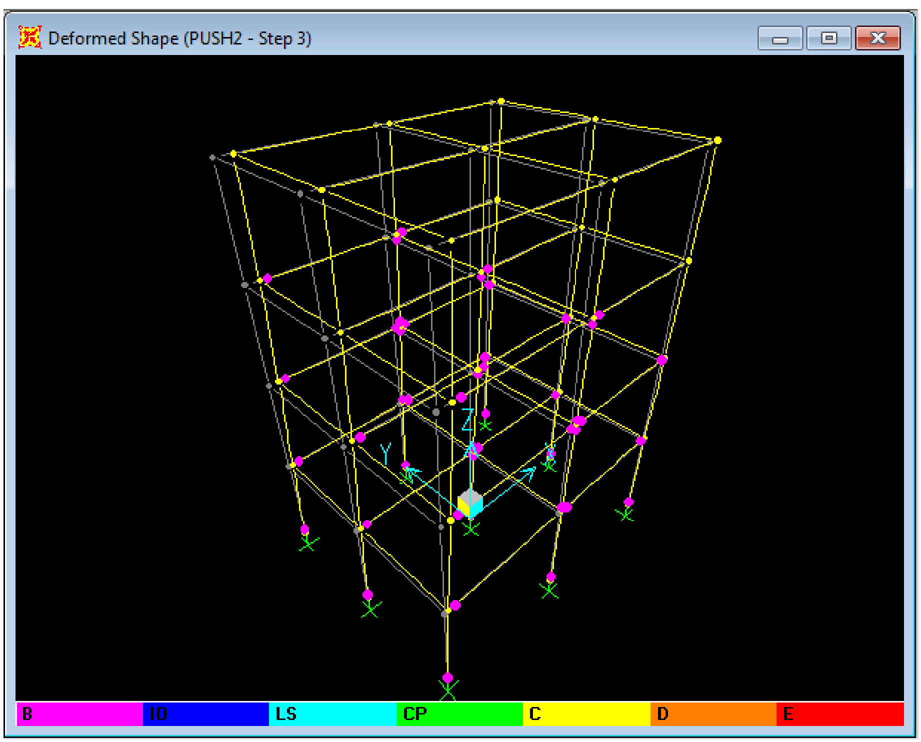


STEP 1

Fig 4.14(a): Step By Step Deformations for Pushover

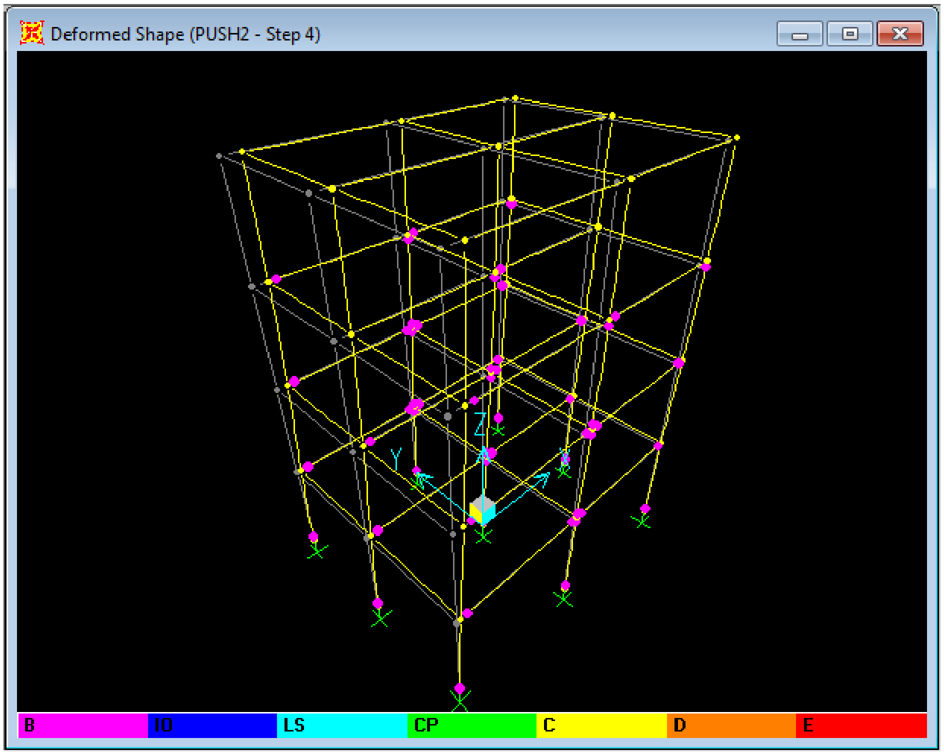


STEP 2

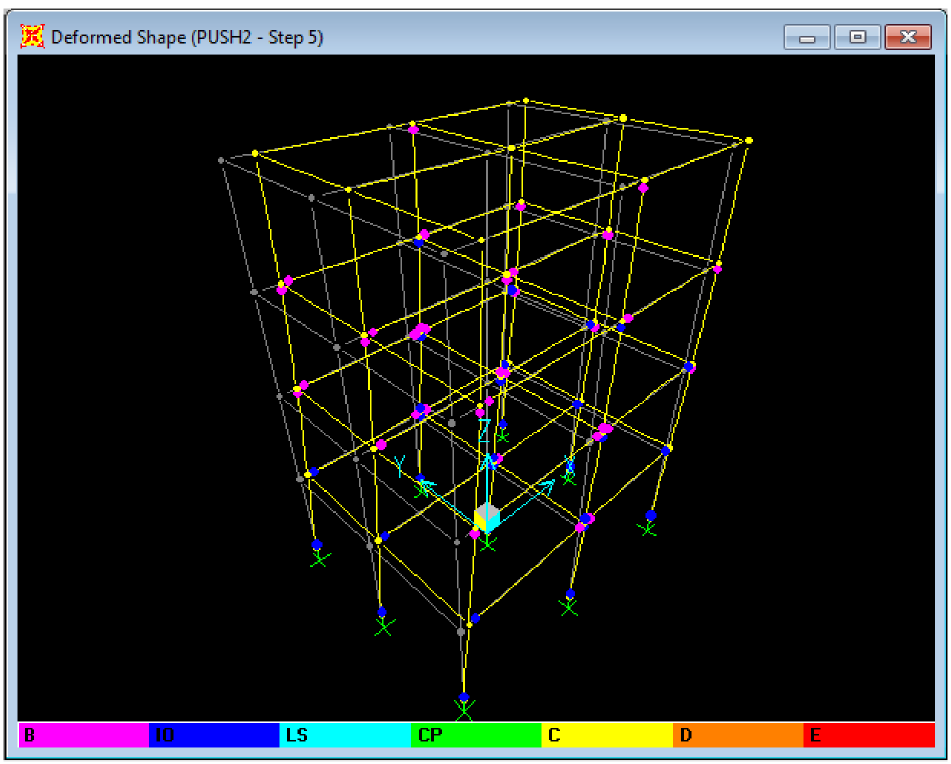


STEP 3

Fig 4.14(b): Step By Step Deformations for Pushover

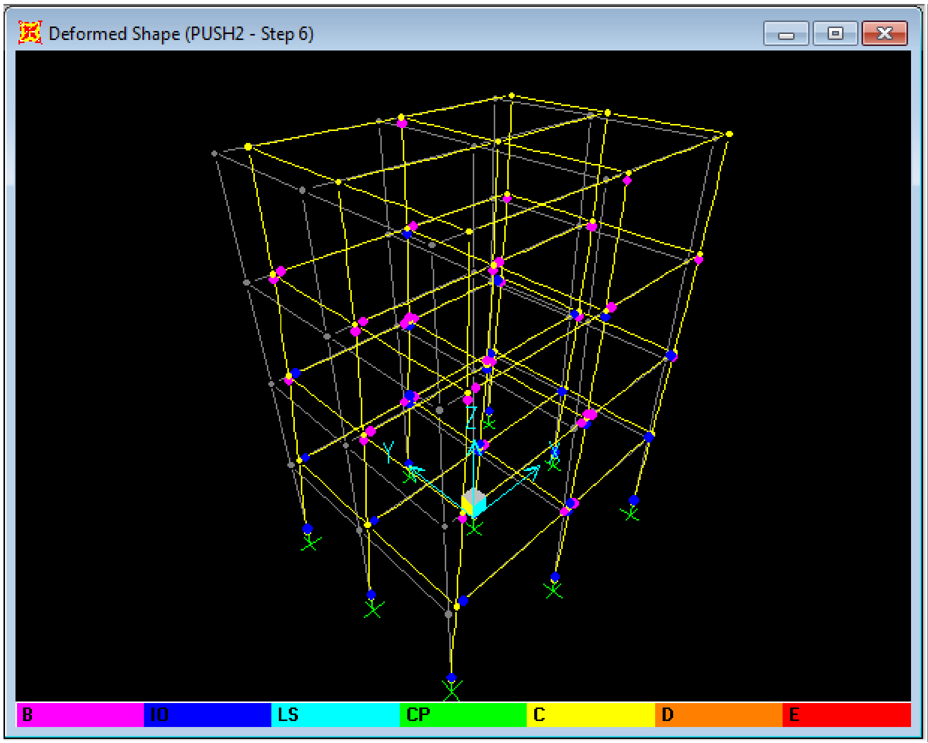


STEP 4

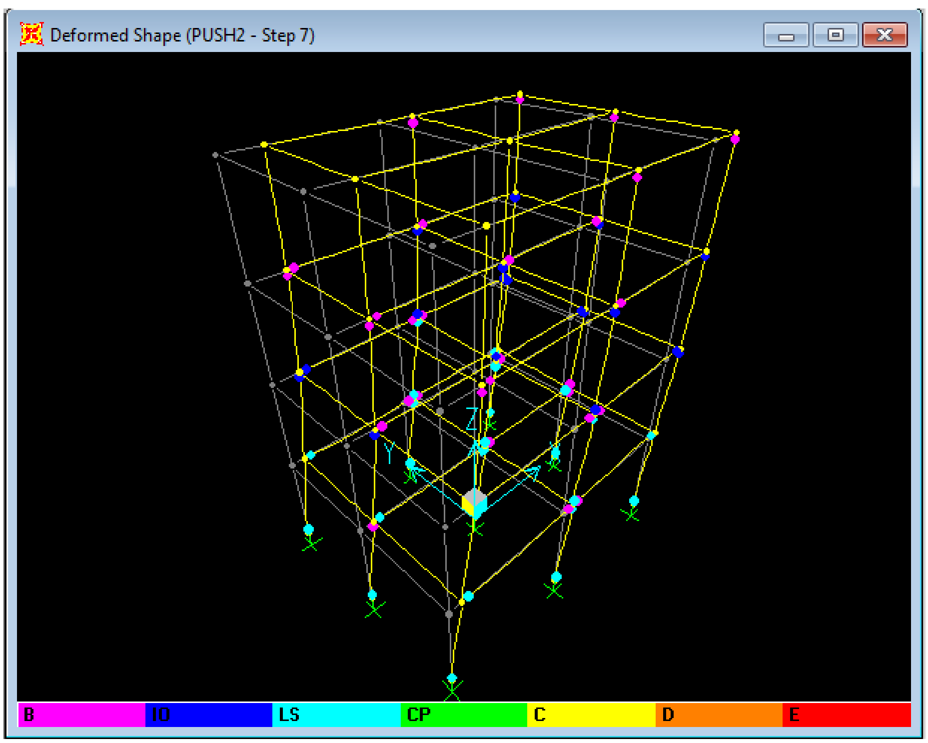


STEP 5

Fig 4.14(c): Step By Step Deformations for Pushover

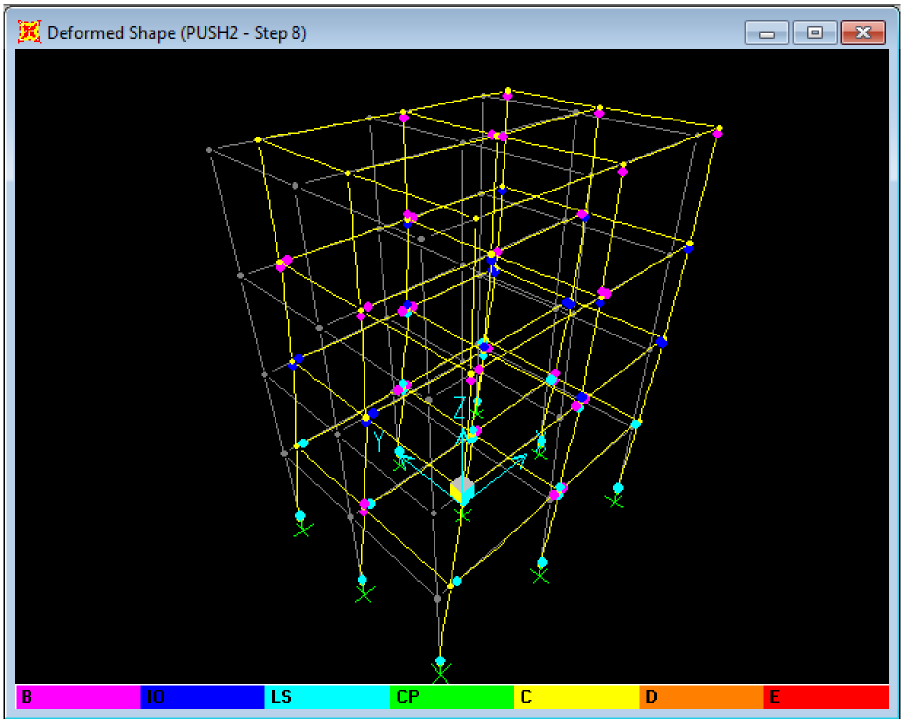


STEP 6

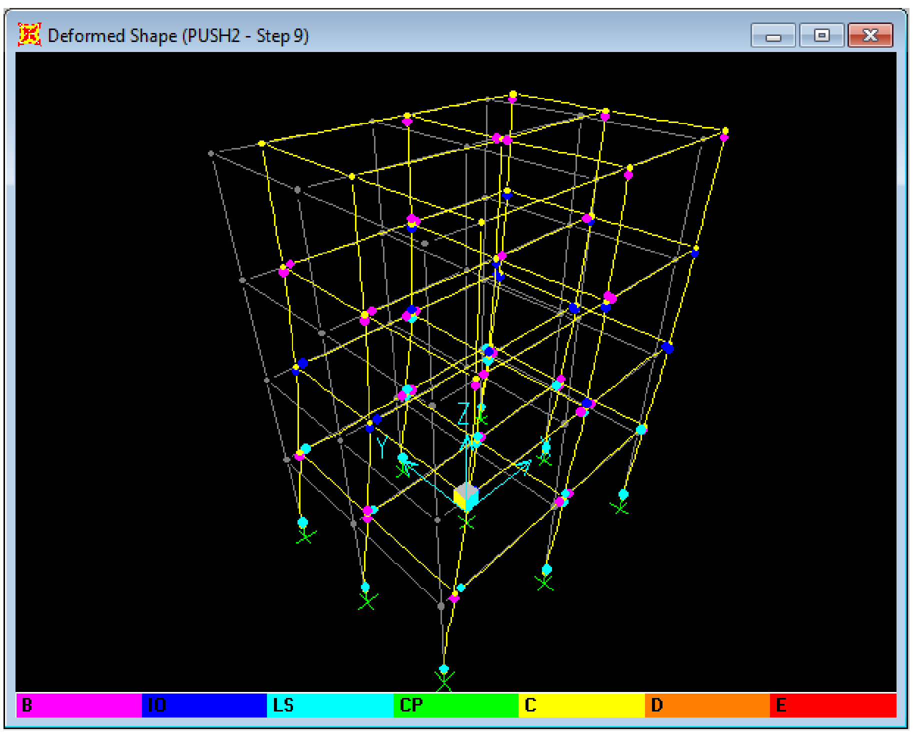


STEP 7

Fig 4.14(d): Step By Step Deformations for Pushover



STEP 8



STEP 9

Figure 4.14(e): Step By Step Deformations for Pushover

4.9 VARIOUS CASES INCORPORATED IN STUDY

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

All beams and columns at a particular story are given same reinforcement. Reinforcement in columns is varied per two storeys.

Finally to study the effect of shear walls in structure, shear wall is provided in the basic structure. As the building is symmetric shear wall is provided in one bay of building frame.

The following cases (Table 4.2) have been incorporated in the study:

Table 4.2 Description of various cases (in elevation)

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Basic structure
2	1,2	Increasing reinforcement in beams of 1st storey only
3	3,4	Increasing reinforcement in beams of 2nd storey only
4	5,6	Increasing reinforcement in beams of 3rd storey only
5	7,8	Increasing reinforcement in beams of 4th storey only
6	9,10	Increasing reinforcement in columns of 1st and 2nd storey only
7	11,12	Increasing reinforcement in columns of 3rd and 4th storey only
8	13,14	Increasing reinforcement in beams & columns of 1st and 2nd storey only
9	15,16	Increasing reinforcement in beams & columns of 3rd and 4th storey only
10	17	Basic structure with shear wall

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

Columns are numbered from 1 to 9, starting from front.

Table 4.3 Description of various cases (in plan)

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Basic structure
2	1,2	Increasing reinforcement in column 1 only
3	3,4	Increasing reinforcement in column 2 only
4	5,6	Increasing reinforcement in column 3 only
5	7,8	Increasing reinforcement in column 4 only
6	9,10	Increasing reinforcement in column 5 only
7	11,12	Increasing reinforcement in column 6 only
8	13,14	Increasing reinforcement in column 7 only
9	15,16	Increasing reinforcement column 8 only
10	17,18	Increasing reinforcement column 9 only

4.10 BASIC STRUCTURE WITH SHEAR WALL

Shear wall is modeled as shell element. Thickness of shear wall is taken equal to 130mm. As the building is symmetric shear wall is provided in one bay of building frame as shown in fig.4.15.

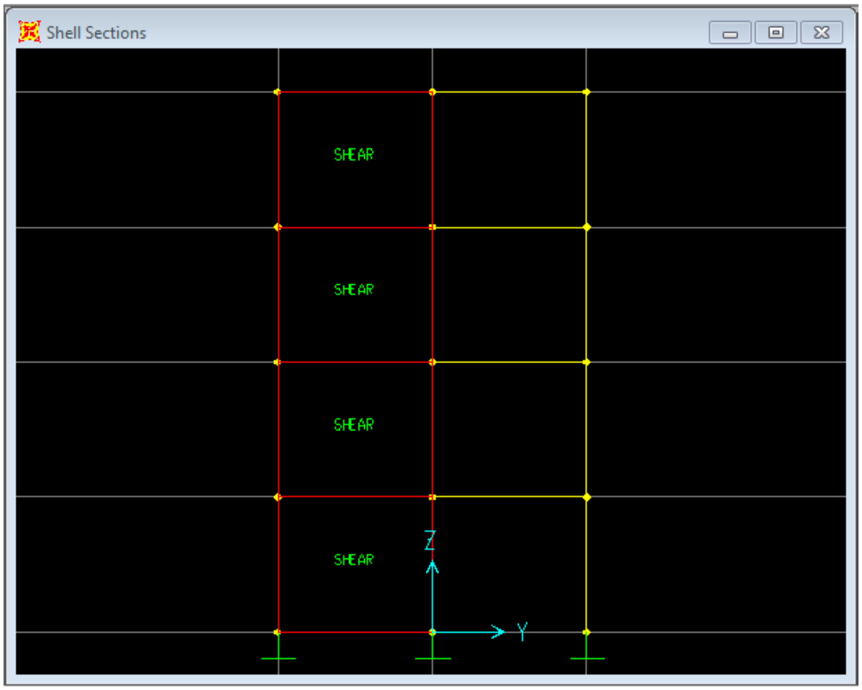


Figure 4.15 Shear wall as shell section

4.11 ANALYSIS OF RESULTS

4.11.1 Base force

The base force for the four-storey building with different combination of element reinforcement at various floor levels is presented in Table 4.4.

It is observed that with increase in reinforcement of beams only, there is a very minimal percentage change in the base force varying from 1.28% to -3.27%, which the structure can carry. However, with the increase in reinforcement of storey columns, there is quite an appreciable change in the base force carrying capacity of the structure. Further there is a decline of 4.63% in the base force capacity, when shear wall is provided in one bay of building frame. The combination of change of reinforcement in beams and columns both show a small increase in base force capacity.

Base shear decreases by 7.55% when shear wall is provided in one bay of structure.

Table: 4.4 Comparison of Base Shear

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	PERCENTAGE CHANGE IN BASE SHEAR
Basic structure			**599.07**	
Beams of 1st STOREY	CASE 1	14.65	605.49	1.07
	CASE 2	20	606.76	1.28
Beams of 2nd STOREY	CASE 3	15.78	579.46	-3.27
	CASE 4	32.74	600.53	0.24
Beams of 3rd STOREY	CASE 5	4.17	598.84	-0.04
	CASE 6	9.03	599.02	-0.01
Beams of 4th STOREY	CASE 7	15.78	599.07	0.00
	CASE 8	32.74	599.07	0.00
Columns of 1st & 2nd STOREY	CASE 9	4.09	600.50	0.24
	CASE 10	39.23	603.20	0.69
Columns of 3rd & 4th STOREY	CASE 11	4.09	584.63	-2.41
	CASE 12	39.23	600.97	0.32
Beams & Columns of 1st & 2nd STOREY	CASE 13	11.51	610.35	1.88
	CASE 14	30.66	614.45	2.56
Beams & Columns of 3rd & 4th STOREY	CASE 15	8.01	602.87	1.48
	CASE 16	27	600.97	0.32
Basic structure with shear wall	CASE 17		553.83	-7.55

4.11.2 Roof Displacement

The Roof displacement for the four-storey building with different combination of element reinforcement at various floor levels is presented in Table 4.5.

It is observed that by increasing the reinforcement of beams only, there is a decrease in the roof displacement upto 3rd storey and after 3rd storey 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 storey and after 3rd storey it slightly decreases upto 4th storey.

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

Table: 4.5 Comparison of Roof Displacement

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENTS (mm)	PERCENTAGE CHANGE IN ROOF DISPLACEMENT
Basic structure			152.30	
Beams of 1st STOREY	CASE 1	14.65	137.20	-9.91
	CASE 2	20	133.10	-12.61
Beams of 2nd STOREY	CASE 3	15.78	149.50	-1.84
	CASE 4	32.74	146.50	-3.81
Beams of 3rd STOREY	CASE 5	4.17	152.30	0.00
	CASE 6	9.03	131.60	-13.59
Beams of 4th STOREY	CASE 7	15.78	152.30	0.00
	CASE 8	32.74	152.30	0.00

Columns of 1st & 2nd STOREY	CASE 9	4.09	151.40	-0.59
	CASE 10	39.23	120.20	-21.08
Columns of 3rd & 4th STOREY	CASE 11	4.09	148.70	-2.36
	CASE 12	39.23	142.90	-6.17
Beams & Columns of 1st & 2nd STOREY	CASE 13	11.51	150.70	-1.05
	CASE 14	30.66	141.70	-6.96
Beams & Columns of 3rd & 4th STOREY	CASE 15	8.01	142.90	-6.17
	CASE 16	27	132.40	-13.07
Basic structure with Shear wall	CASE 17		55.80	-63.36

4.11.3 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: 4.6 Variation of Roof Displacement with Base Force for all cases

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	ROOF DISPLACEMENTS (mm)
Basic structure			**599.07**	**152.30**
Beams of 1st STOREY	CASE 1	14.65	605.49	137.20
	CASE 2	20	606.76	133.10
Beams of	CASE 3	15.78	579.46	149.50

2nd STOREY	CASE 4	32.74	600.53	146.50
Beams of 3rd STOREY	CASE 5	4.17	598.84	152.30
	CASE 6	9.03	599.02	131.60
Beams of 4th STOREY	CASE 7	15.78	599.07	152.30
	CASE 8	32.74	599.07	152.30
Columns of 1st & 2nd STOREY	CASE 9	4.09	600.50	151.40
	CASE 10	39.23	603.20	120.20
Columns of 3rd & 4th STOREY	CASE 11	4.09	584.63	148.70
	CASE 12	39.23	600.97	142.90
Beams & Columns of 1st & 2nd STOREY	CASE 13	11.51	610.35	150.70
	CASE 14	30.66	614.45	141.70
Beams & Columns of 3rd & 4th STOREY	CASE 15	8.01	602.87	142.90
	CASE 16	27	600.97	132.40
Basic structure with Shear wall	CASE 17		553.83	55.80

4.11.4 Performance Point

The performance point of the structure can be now 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 termed 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 at which capacity and demand are equal. As displacement increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Displacements increase damping and reduce demand. Hence, the optimum point should have a higher capacity for a lesser displacement.

4.12 EFFECT OF CHANGE OF REINFORCEMENT IN VARIOUS COLUMNS

Initially the reinforcement of all the columns is kept same (452mm^2) and then it is increased for each column, while the beam reinforcement is kept same as given by IS456:2000. The effect is studied by pushover analysis. The change in roof displacement for every case is given in Table 4.7.

Table: 4.7 Variation of Roof Displacement with column reinforcement for all cases

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENT (mm)	PERCENTAGE DECREASE IN ROOF DISPLACEMENT
Basic structure			**155**	
Column 1	CASE 1	77.87	144	7.1
	CASE 2	177.87	129.6	16.39
Column 2	CASE 3	77.87	138.9	10.39
	CASE 4	177.87	119.1	23.16
Column 3	CASE 5	77.87	138.1	10.90
	CASE 6	177.87	133.9	13.61
Column 4	CASE 7	77.87	144.5	6.77
	CASE 8	177.87	131.0	15.48
Column 5	CASE 9	77.87	0	15.87
	CASE 10	177.87	0	25.48
Column 6	CASE 11	77.87	144.2	6.97
	CASE 12	177.87	134.5	13.23
Column 7	CASE 13	77.87	144.3	6.90
	CASE 14	177.87	132.8	14.32
Column 8	CASE 15	77.87	143.6	7.35
	CASE 16	177.87	120.5	22.26

Column 9	CASE 17	77.87	135.9	12.32
	CASE 18	177.87	130.0	16.13

4.13 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 4.8 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 0.7% of total height of building.

Target Roof Displacement = 0.007 x 14m = 0.098m = 98mm

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 14m = 0.35m = 350mm

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

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

which is two times as that for DBE. Hence the triangular loading obtained is shown in fig. 4.16

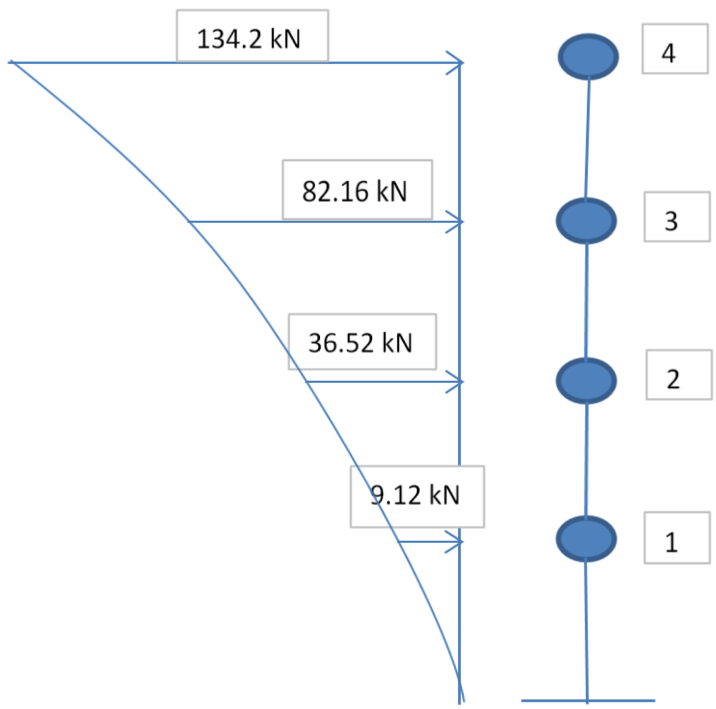


Figure 4.16 Triangular Loading for MCE

The reinforcement detail of the Performance based building design thus obtained is shown in Table 4.9. These are compared to the reinforcement obtained by seismic resistant design of building (according to IS 1893:2002) in STAAD.Pro.

Table 4.9 Comparison of area of reinforcement in mm^2 in beams and columns for all designs

Element	IS 456:2000	Performance based Design	IS 1893:2002
Corner Columns 1st and 2nd storey	452	1880	1260
Corner Columns 3rd and 4th storey	452	905	1260
Mid-Frame Columns 1st and 2nd storey	804	1880	1260
Mid-Frame Columns 3rd and 4th storey	804	905	1260
Interior Column 1st and 2nd storey	1260	1880	1260
Interior Column 3rd and 4th storey	1260	905	1260
Beams 1st storey	785 (top) 550 (bottom)	1020 (top) 550 (bottom)	1020 (top) 550 (bottom)
Beams 2nd storey	680 (top) 550 (bottom)	942 (top) 550 (bottom)	942 (top) 550 (bottom)
Beams 3rd storey	942 (top) 550 (bottom)	864 (top) 550 (bottom)	864 (top) 550 (bottom)
Beams 4th storey	680 (top) 550 (bottom)	680 (top) 550 (bottom)	785 (top) 550 (bottom)

Following results are obtained for pushover analysis of Performance based design:

Base Shear = 531.63 kN

Roof Displacement = 91.0mm

Thus Roof displacement is less than target roof displacement.

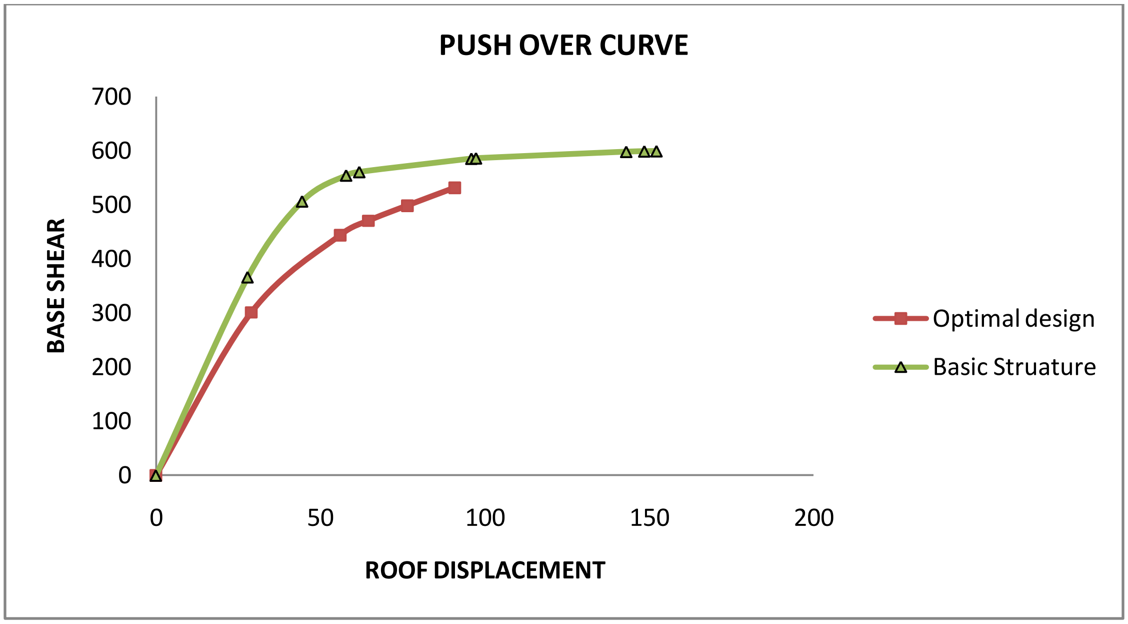


Figure 4.17 Pushover curve for Performance based design of four storey building

4.14 PERFORMANCE OF BUILDING WITH REDUCED SECTION OF COLUMNS AND BEAMS

The sectional properties of elements in case of the Basic structure are taken as follows:

Size of Column = 300 x 300mm

Size of Beam = 300 x 425 mm

Thickness of Slab = 125mm thick

The maximum design lateral force was computed for each storey level and was distributed at each node as in section 4.6. The inverted triangular loading thus obtained was applied to the structure for pushover analysis.

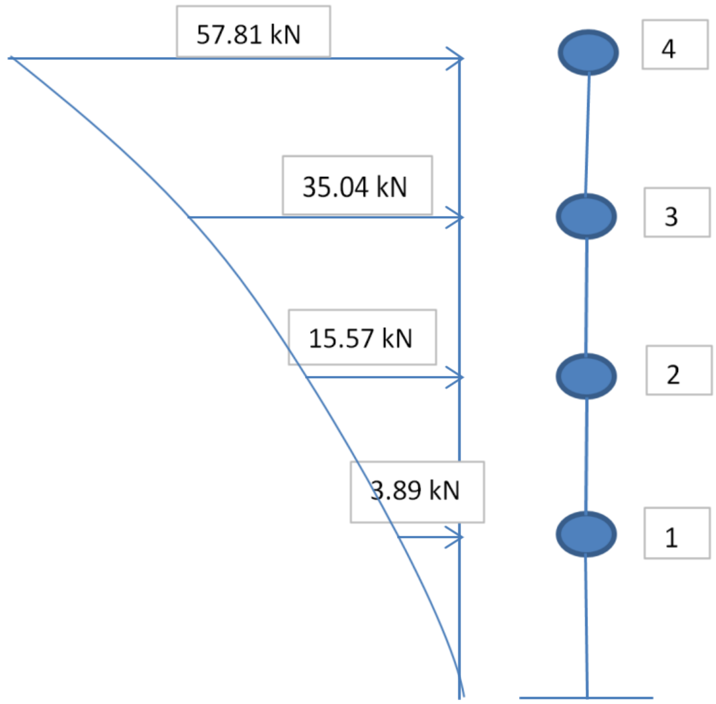
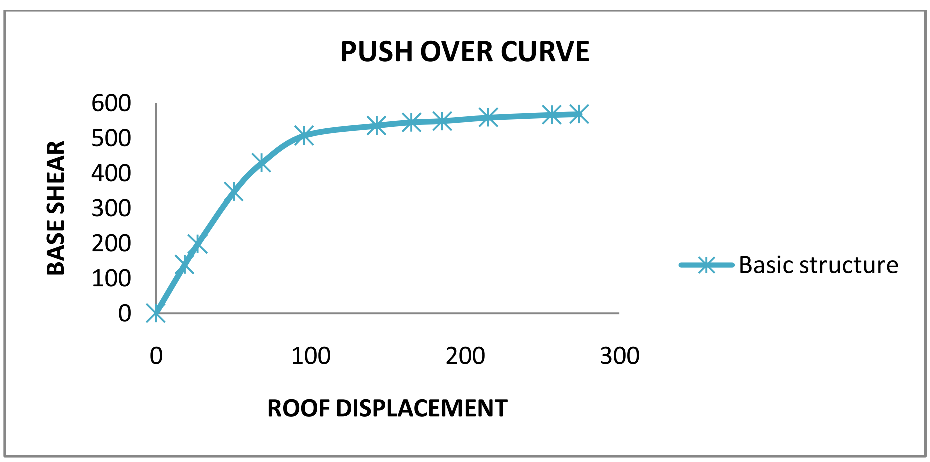


Figure 4.18 Applied Inverted Triangular Loading

Figure 4.19 Pushover curve of 4-storey building for reduced sectional sizes of columns & beams



Following results are obtained from pushover analysis:

Roof Displacement = 274.4mm

Thus Roof displacement is more than roof displacement for Immediate Occupancy and less than displacement for Life Safety.

4.15 UNSYMMETRICAL BUILDING

In this section, an unsymmetrical (L-shape) four storied reinforced concrete frame building situated in Zone IV is taken for the purpose of study. The plan of building is shown in fig. 4.18 and the front elevation is shown in fig. 4.19. Along x-axis, the bay span is 5m, and along z-axis the bay span is 4m. Height of the building is 14m with each storey height of 3.5m. The building is considered as a Special Moment resisting frame.

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

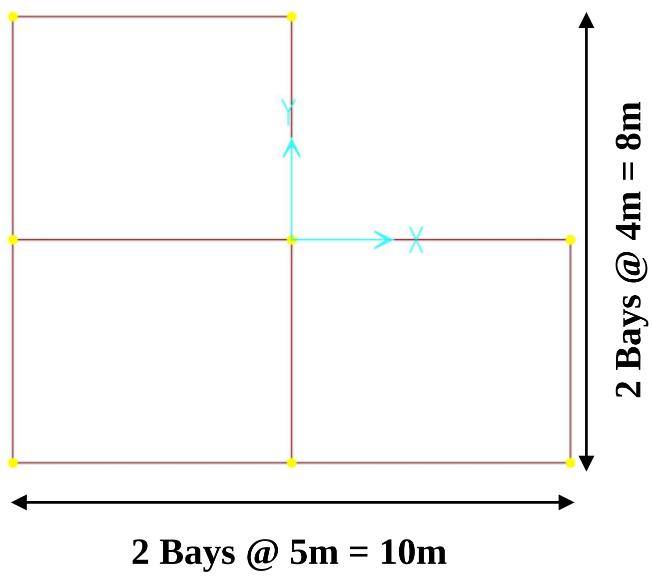


Figure 4.20 Plan of Building

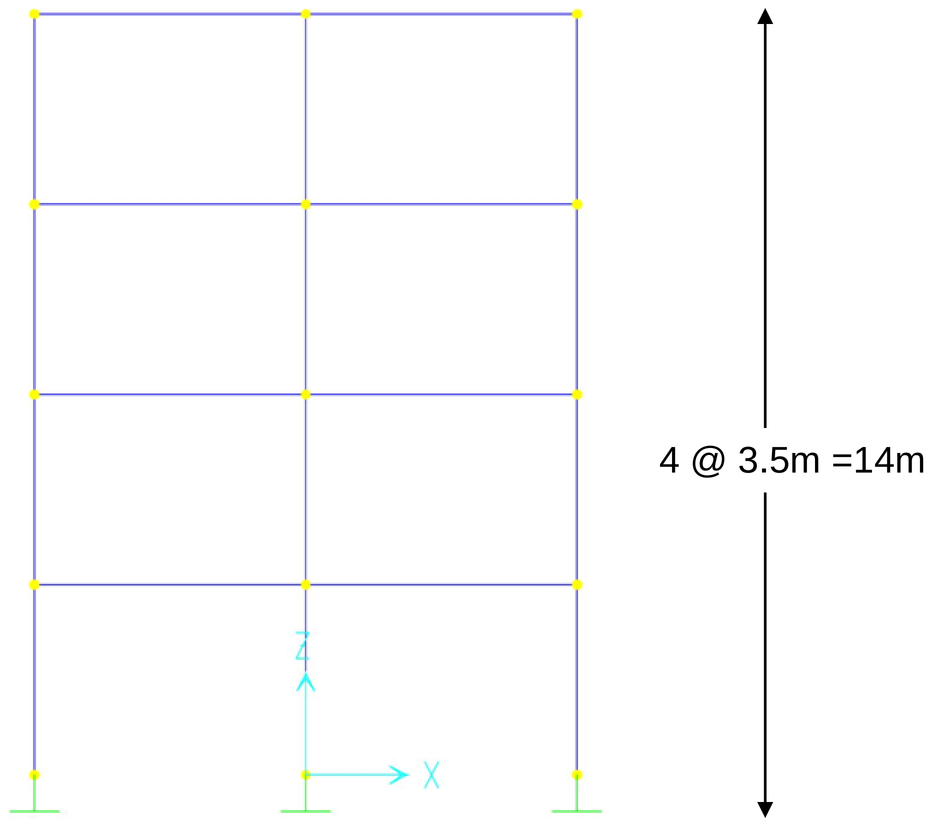


Figure 4.21 Elevation of Building

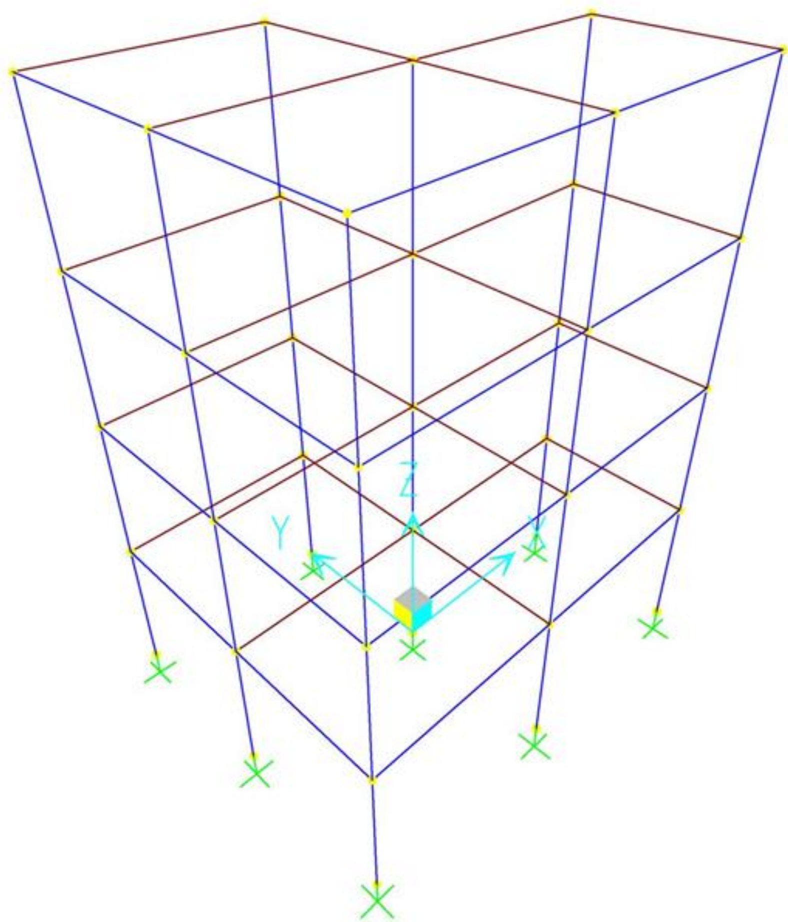


Figure 4.22 3D View of Building

4.16 SECTIONAL PROPERTIES OF ELEMENTS

The sectional properties of elements in case of the Basic structure are taken as follows:

Size of Column = 345 x 345mm

Size of Beam = 345 x 500 mm

Thickness of Slab = 125mm thick

4.17 LOADS CONSIDERED

Same loading as in section 4.5 is taken for the purpose of comparative study.

4.18 DETERMINATION OF LATERAL LOADS FOR PUSHOVER ANALYSIS

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

4.18.1 Calculation of seismic Weight of Structure

Seismic weight of roof is calculated as under:

Slab = 0.125 x 4 x 5 x 25 x 3 = 187.5 kN

Beams = 45 x 0.345 x 0.5 x 25 = 194.06 kN

Columns = 0.345 x 0.345 x 1.75 x 25 x 8 = 41.66 kN

Total = 423.22 kN

Seismic weight of one floor is calculated as under:

Slab = 0.125 x 4 x 5 x 25 x 3 = 187.5 kN

Beams = 45 x 0.345 x 0.5 x 25 = 194.06 kN

Columns = 0.345 x 0.345 x 3.5 x 25 x 8 = 83.32 kN

Total = 464.88 kN

4.18.2 Calculation of base shear

The following parameters were taken:

Zone Factor, Z=0.24

Importance Factor, I=1.0

Response Reduction Factor=5.0

Time Period is calculated from:

TS = $0.09\, h/\sqrt{d} = .09 x 14/\sqrt{10} = 0.4$ seconds

Hence, Sa/g = 2.5 (For Medium Soil Conditions)

Hence, A_h = (.24/2) x (1/5) x 2.5 = .06

Thus V_b = .06 x 1817.86 = 105.43 kN

$W_j h_j^2$ = 423.22 [14^2] + 464.88 [$3.5^2 + 7^2 + 10.5^2$] = 162678

$$\text{Now, } Qi = \frac{V_b\, Wi\, hi^2}{\sum Wi\, hi^2}$$

Hence, Q4 = (423.22 x 105.43 x 14^2) / 162678 = 53.76 kN

Similarly, Q3 = 33.22 kN

Q2 = 14.76 kN

Q1 = 3.69 kN

This load was applied to the structure for pushover analysis. This load is similar to the inverted trianglar loading suggested for pushover analysis by ATC-40.

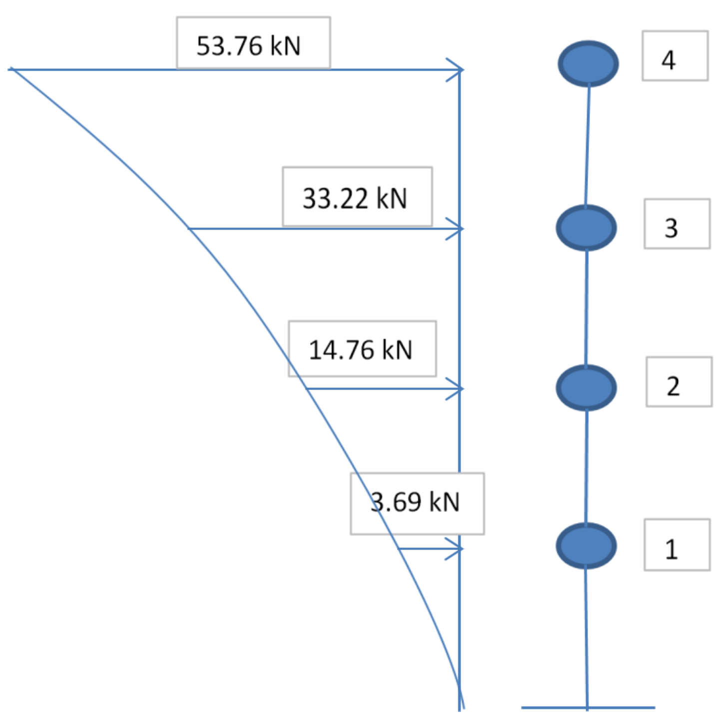


Figure 4.23 Applied Inverted Triangular Loading

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

Table 4.10 Structural details (as per Analysis and Design on Staad.Pro)

Element	Dimension (m)	Reinforcement Area in mm^2
Corner Columns	0.345 x 0.345	452
Mid face Columns	0.345 x 0.345	804
Interior Column	0.345 x 0.345	804
Beams 1st storey	0.345 x 0.5	700 (top) 550 (bottom)
Beams 2nd storey	0.345 x 0.5	680 (top) 550 (bottom)
Beams 3rd storey	0.345 x 0.5	600 Φ (top) 550 (bottom)
Beams 4th storey	0.345 x 0.5	600 Φ (top) 550 (bottom)

4.19 PUSHOVER ANALYSIS USING SAP2000

Steps included in the pushover analysis are shown below (similar as section 4.6).

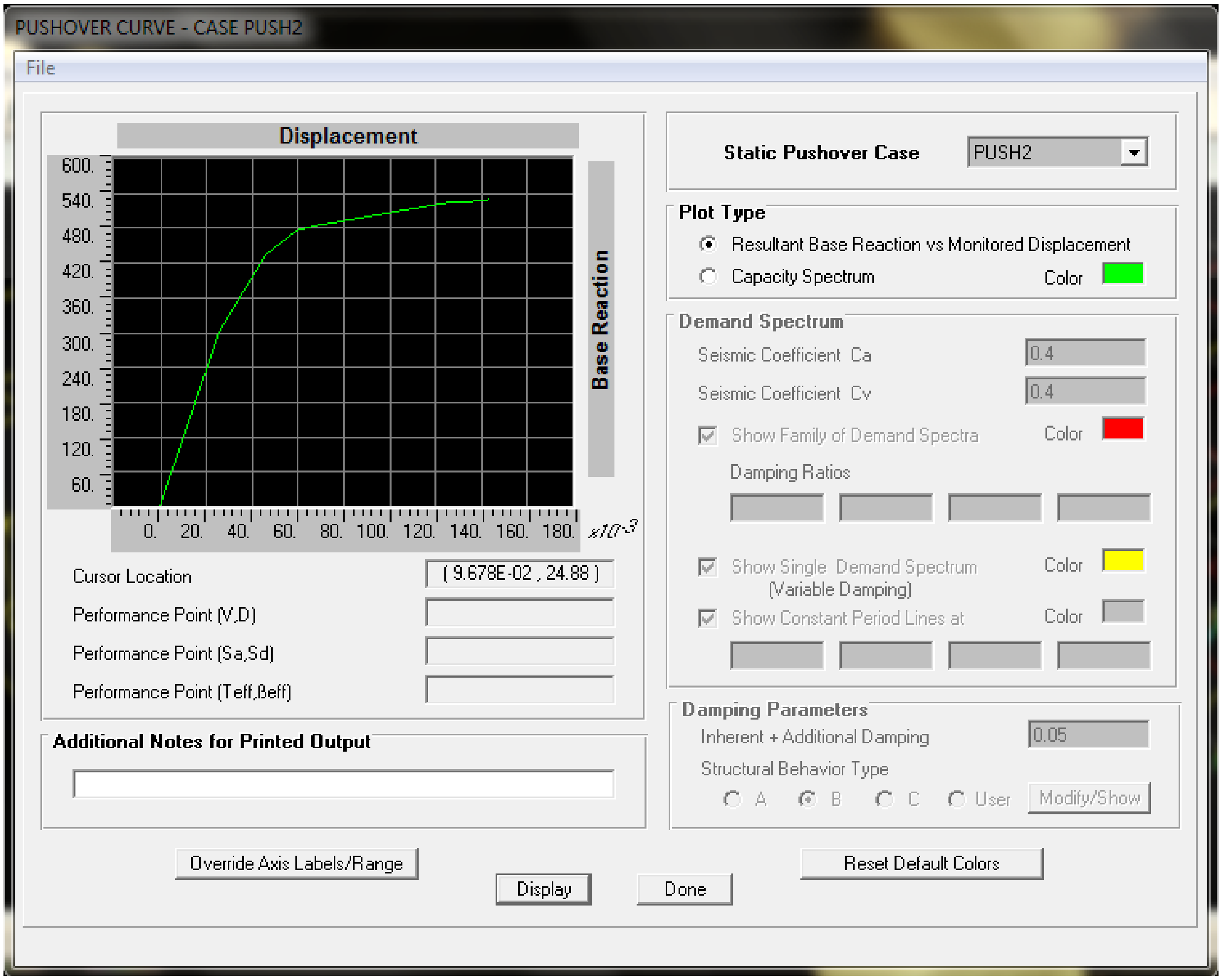


Figure 4.24 Pushover Curve

PUSHOVER CURVE

File

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	-4.107E-04	0.0000	144	0	0	0	0	0	0	0	144
1	0.0255	301.1623	143	1	0	0	0	0	0	0	144
2	0.0457	433.1904	116	28	0	0	0	0	0	0	144
3	0.0593	474.2477	101	43	0	0	0	0	0	0	144
4	0.0665	484.6605	95	49	0	0	0	0	0	0	144
5	0.1262	524.6257	81	27	30	6	0	0	0	0	144
6	0.1302	526.3769	79	28	28	9	0	0	0	0	144
7	0.1419	529.4417	75	31	23	15	0	0	0	0	144
8	0.1436	529.6473	73	32	24	15	0	0	0	0	144

Figure 4.25 Tabular Data for Pushover Curve

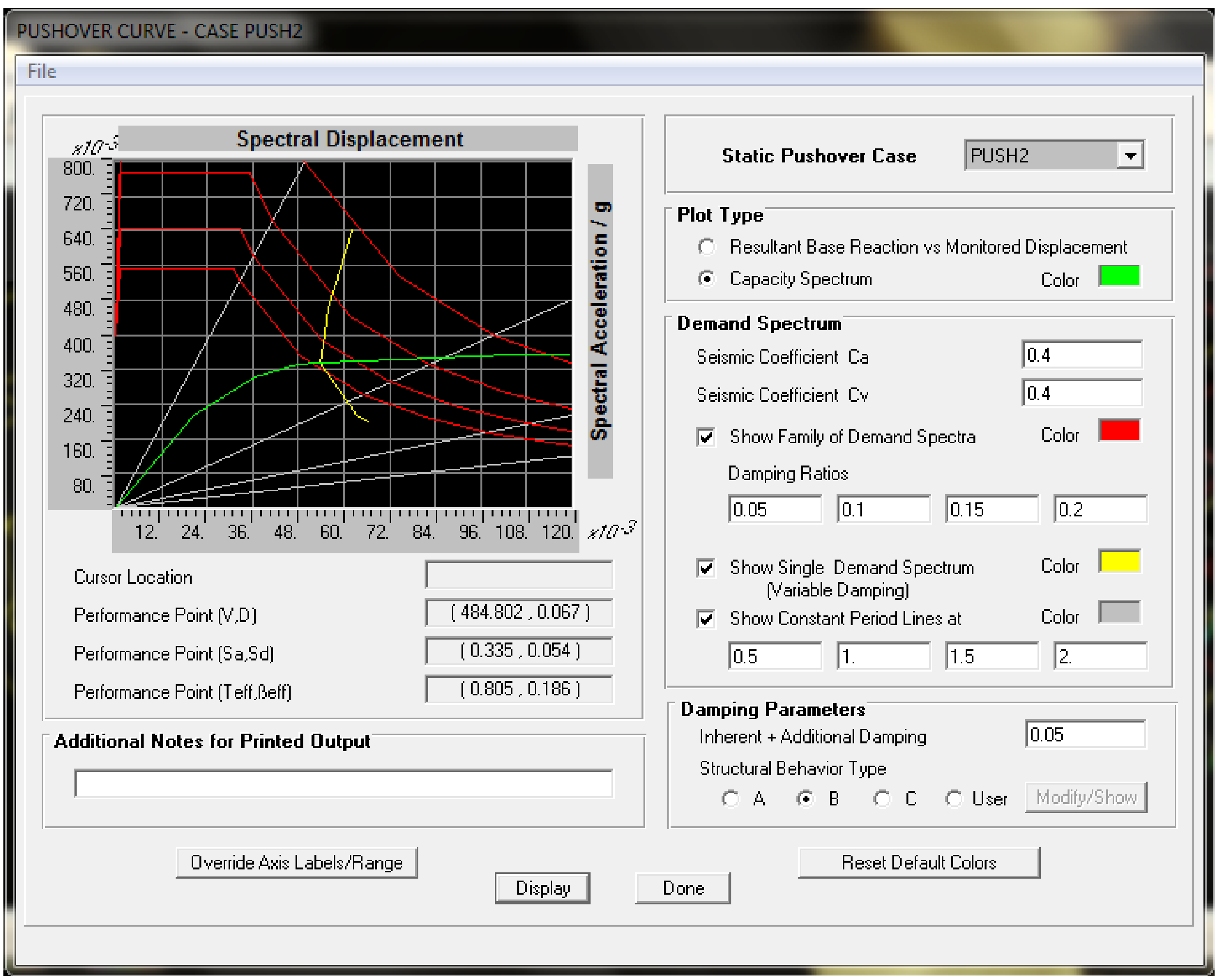


Figure 4.26 Capacity Spectrum Curve

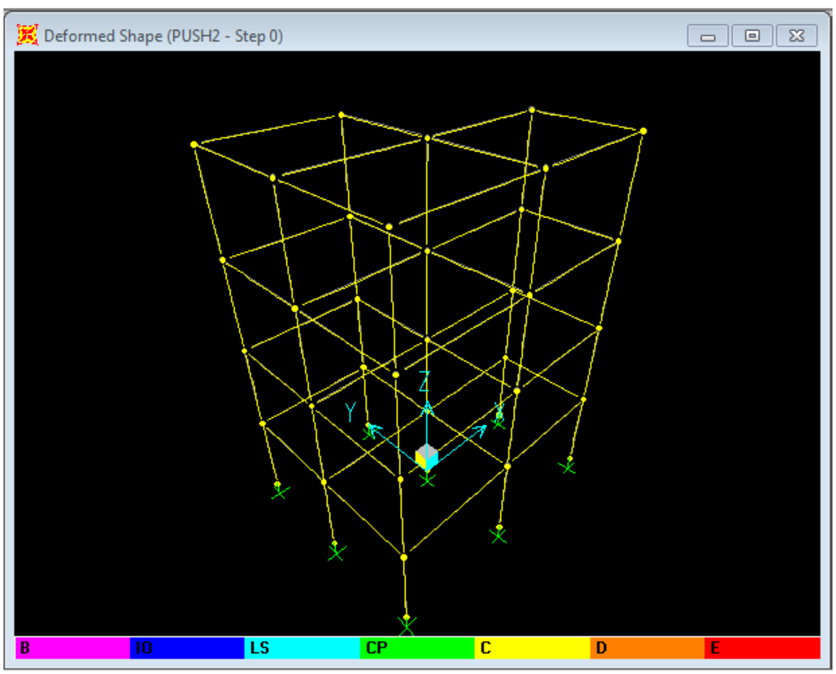
PUSHOVER CAPACITY/DEMAND COMPARISON

File

Step	Teff	βeff	Sd (C)	Sa (C)	Sd (D)	Sa (D)	ALPHA	PF*Ø
0	0.625	0.050	0.000	0.000	0.062	0.640	1.000	1.000
1	0.625	0.050	0.021	0.214	0.062	0.640	0.805	1.249
2	0.697	0.109	0.037	0.303	0.056	0.462	0.816	1.261
3	0.763	0.157	0.048	0.330	0.054	0.376	0.820	1.250
4	0.804	0.186	0.054	0.334	0.054	0.335	0.828	1.245
5	1.093	0.269	0.105	0.353	0.063	0.213	0.850	1.211
6	1.109	0.270	0.108	0.354	0.064	0.210	0.850	1.209
7	1.159	0.276	0.118	0.355	0.066	0.199	0.852	1.203
8	1.166	0.277	0.120	0.355	0.067	0.197	0.853	1.202

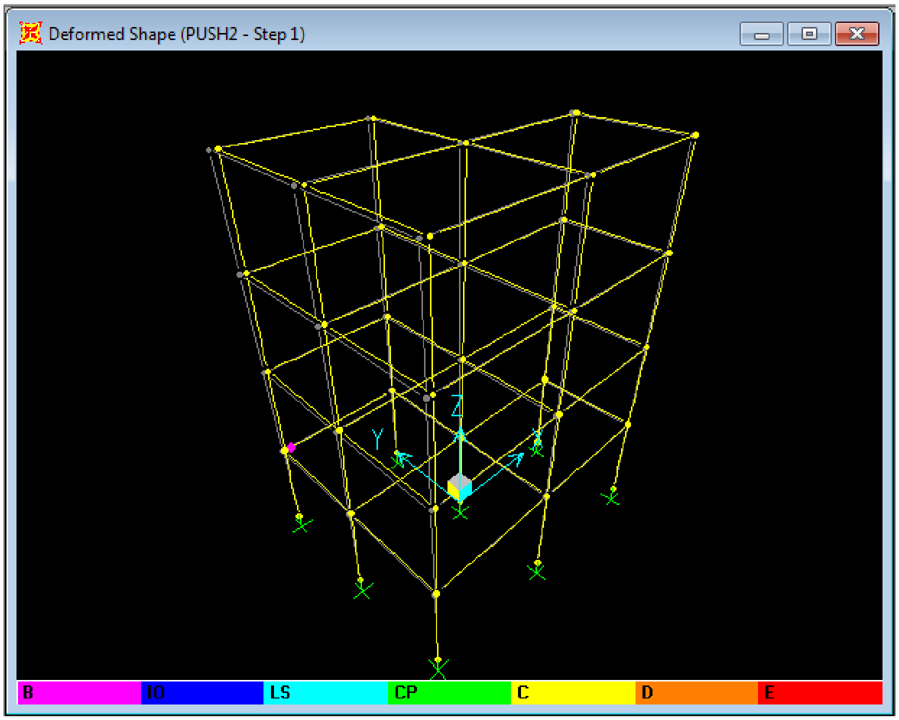
Figure 4.27 Tabular Data for Capacity Spectrum Curve

The pushover displaced shape and sequence of hinge information on a step-by-step basis was obtained and is shown in the Figure 4.28(a) to 4.28(e).

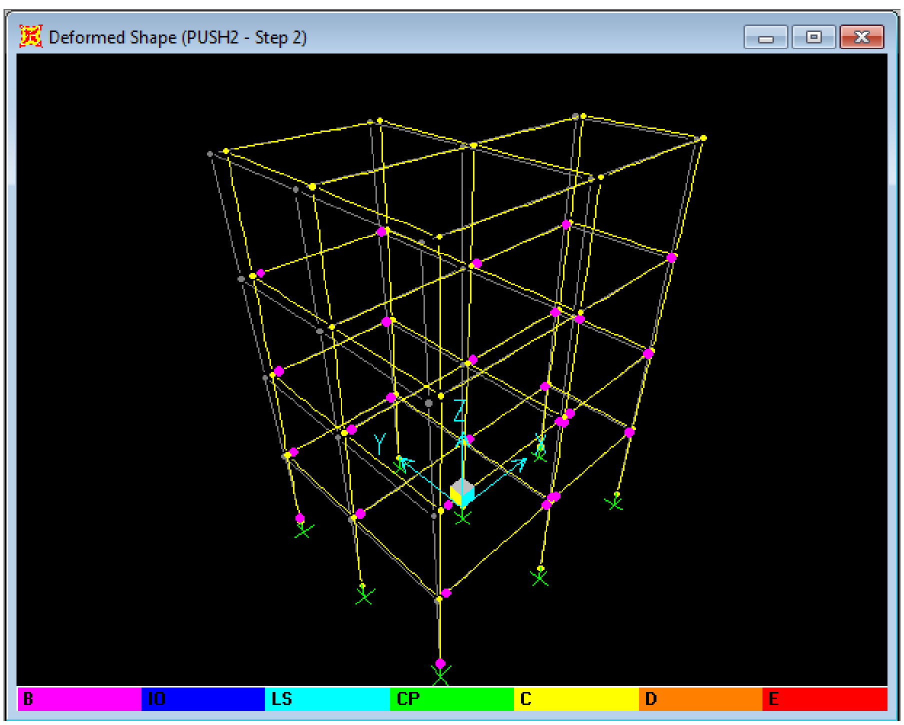


STEP 0

Figure 4.28(a): Step By Step Deformations for Pushover

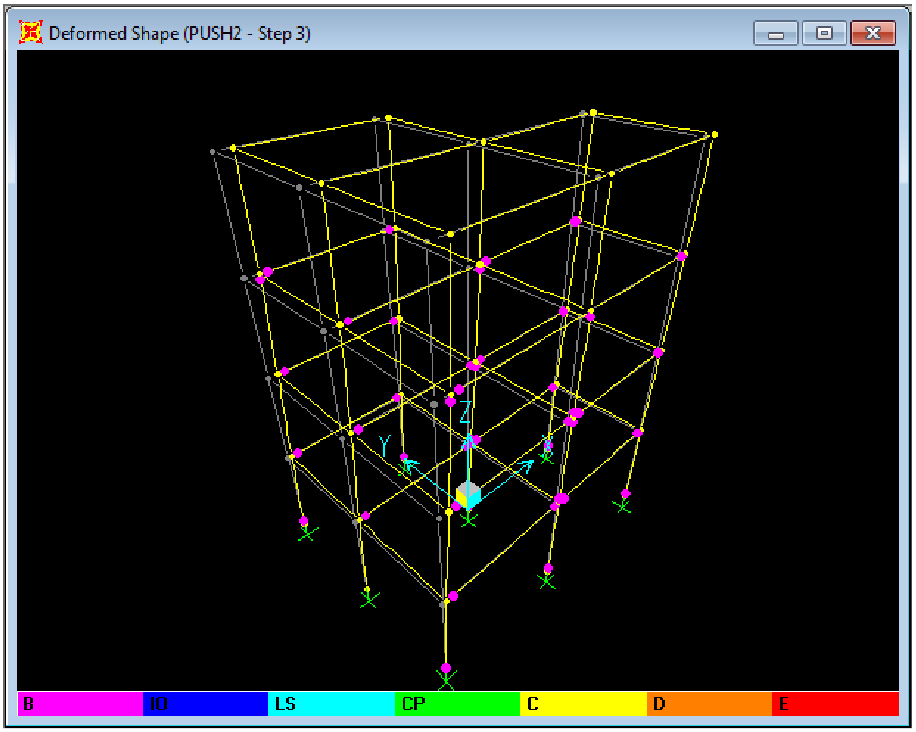


STEP 1

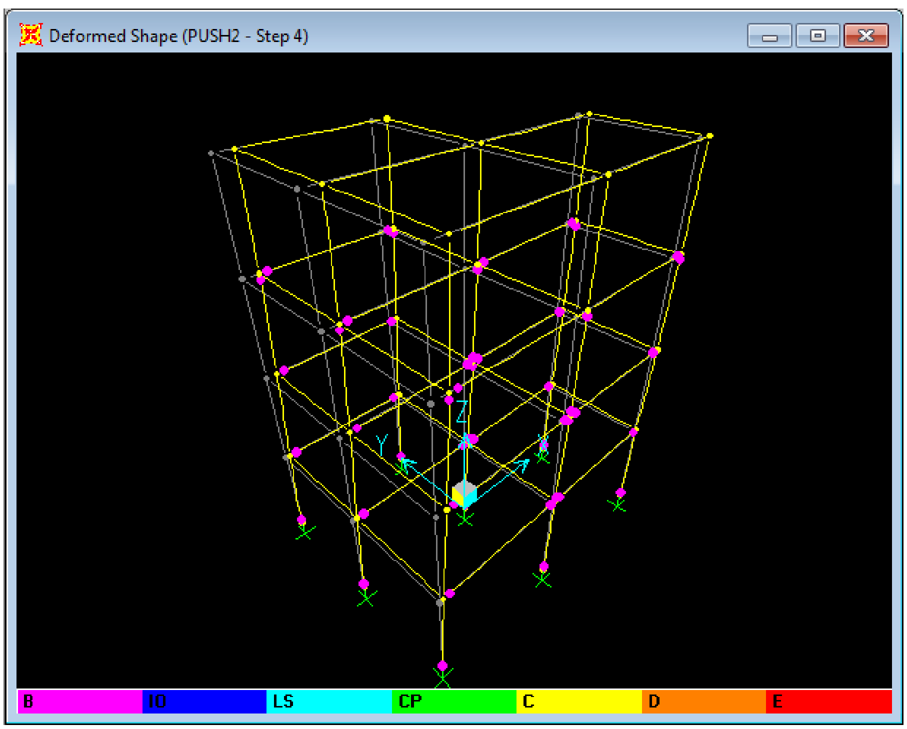


STEP 2

Figure 4.28(b): Step By Step Deformations for Pushover

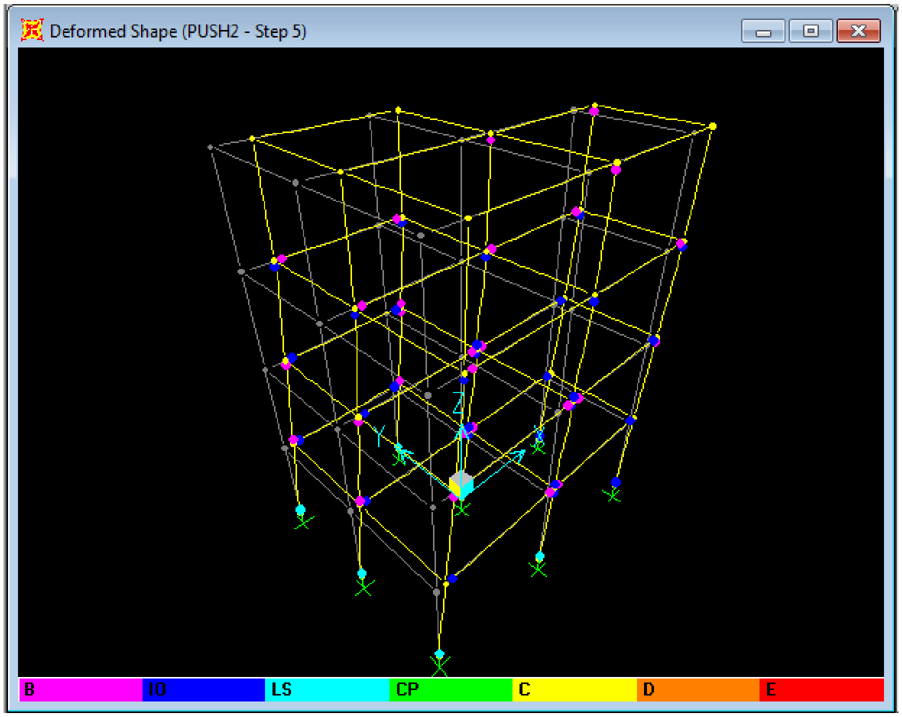


STEP 3

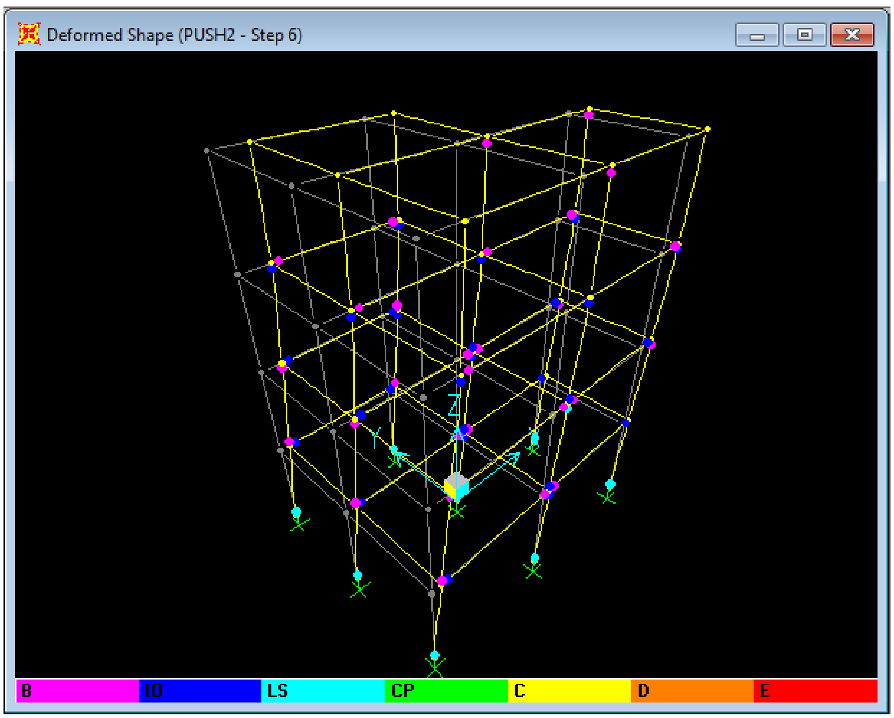


STEP 4

Figure 4.28(c): Step By Step Deformations for Pushover

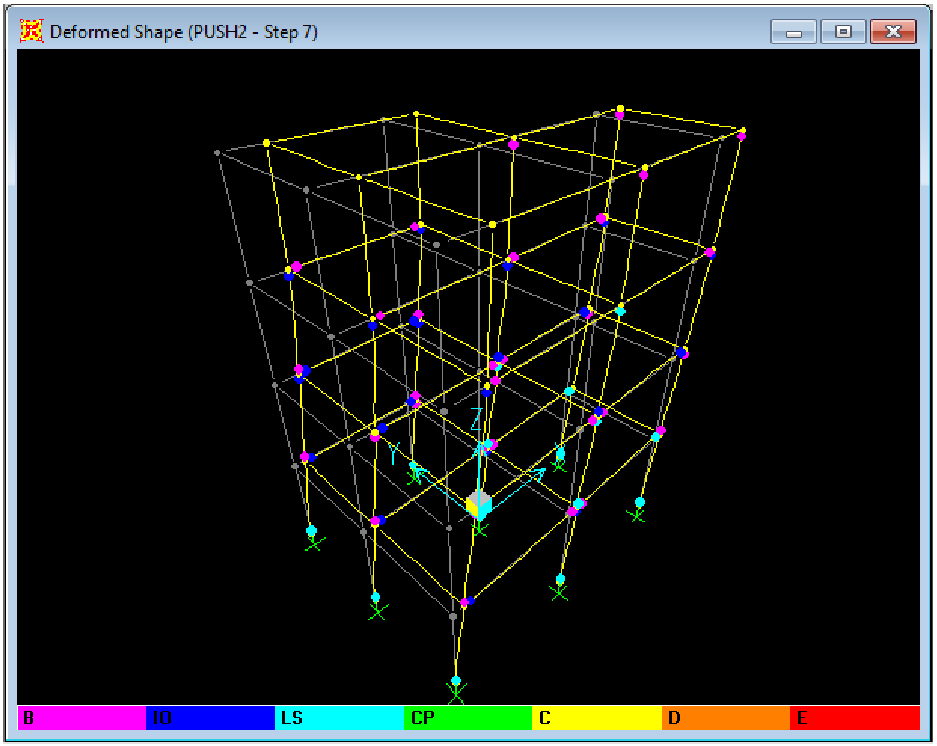


STEP 5

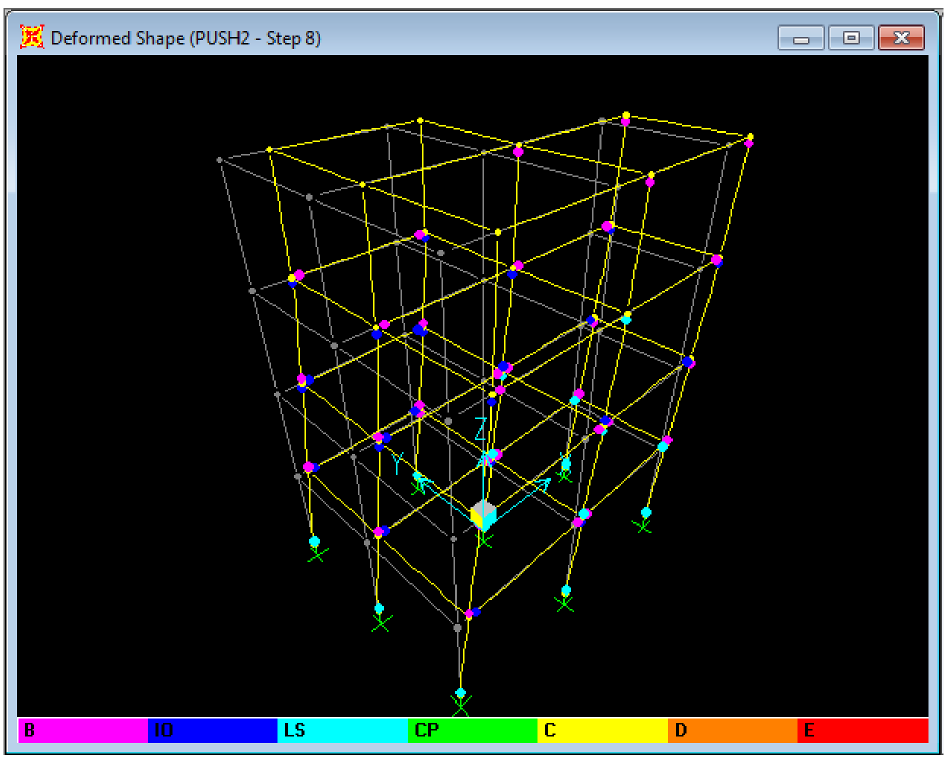


STEP 6

Figure 4.28(d): Step By Step Deformations for Pushover



STEP 7



STEP 8

Fig 4.28(e): Step By Step Deformations for Pushover

4.20 VARIOUS CASES INCORPORATED IN STUDY

To study the effect of change of main reinforcement on the performance of the structure, various cases are made. All beams and columns at a particular story are given same reinforcement. Reinforcement in columns is varied per two storeys.

Finally to study the effect of shear walls in structure, shear wall is provided in the basic structure. As the building is unsymmetric, there are 2 cases of providing shear wall. In first case shear wall is provided in one bay on the larger side of building and in 2nd case it is provided on the shorter side.

The following cases (Table 4.11) have been incorporated in the study:

Table 4.11 Description of various cases

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Basic structure
2	1,2	Increasing reinforcement in beams of 1st storey only
3	3,4	Increasing reinforcement in beams of 2nd storey only
4	5,6	Increasing reinforcement in beams of 3rd storey only
5	7,8	Increasing reinforcement in beams of 4th storey only
6	9,10	Increasing reinforcement in columns of 1st and 2nd storey only
7	11,12	Increasing reinforcement in columns of 3rd and 4th storey only
8	13,14	Increasing reinforcement in beams & columns of 1st and 2nd storey only
9	15,16	Increasing reinforcement in beams & columns of 3rd and 4th storey only
10	17	Basic structure with shear wall on larger side
11	18	Basic structure with shear wall on smaller side

To study the effect of change of main reinforcement of various columns on the performance of the structure, various cases are made. For this the initial reinforcement of all the columns is kept same. Columns are numbered from 1 to 8, starting from front elevation.

Table 4.12 Description of various cases (in plan)

S. NO.	CASE NO.	DESCRIPTION OF CASES
1		Basic structure
2	1,2	Increasing reinforcement in column 1 only
3	3,4	Increasing reinforcement in column 2 only
4	5,6	Increasing reinforcement in column 3 only
5	7,8	Increasing reinforcement in column 4 only
6	9,10	Increasing reinforcement in column 5 only
7	11,12	Increasing reinforcement in column 6 only
8	13,14	Increasing reinforcement in column 7 only
9	15,16	Increasing reinforcement column 8 only

4.21 BASIC STRUCTURE WITH SHEAR WALL

Shear wall is modeled as shell element. Thickness of shear wall is taken equal to 130mm. The building is unsymmetric and there are two case of providing shear wall as shown in fig. 4.29(a) and 4.29(b).

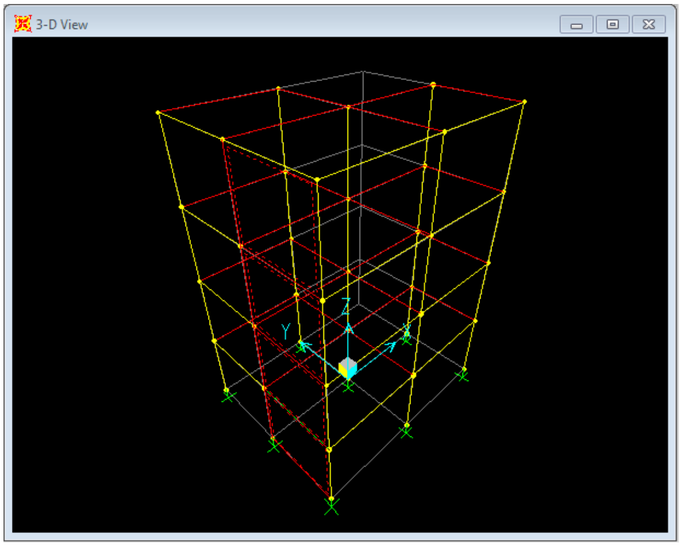


Figure 4.29(a) Shear wall as shell section on larger side

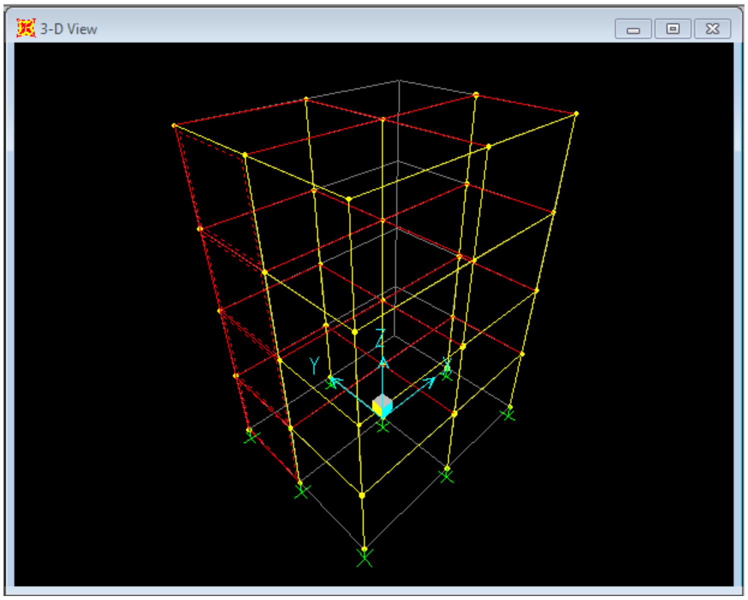


Figure 4.29(b) Shear wall as shell section on smaller side

4.22 ANALYSIS OF RESULTS

4.22.1 Base force

The base force for the four-storey building with different combination of element reinforcement at various floor levels is presented in Table 4.13.

It is observed that with increase in reinforcement of beams only, there is a nominal percentage change in the base force varying from 0.07% to -8.46%, which the structure can carry. However, with the increase in reinforcement of storey columns, there is quite an appreciable change in the base force carrying capacity of the structure. The combination of change of reinforcement in beams and columns both show a consistent increase in base force capacity.

Further there is a decline of 4.3% in the base force capacity, when shear wall is provided on the larger side of the building frame, whereas when it is provided on the smaller side there is a substantial decrease of 7.97%.

Table: 4.13 Comparison of Base Shear

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	PERCENTAGE CHANGE IN BASE SHEAR
Basic structure			**529.65**	
Beams of 1st STOREY	CASE 1	10.56	517.65	-2.26
	CASE 2	26.76	526.80	-0.54
Beams of 2nd STOREY	CASE 3	4.72	494.78	-6.58
	CASE 4	15.78	500.30	-5.54
Beams of 3rd STOREY	CASE 5	12.43	529.95	0.07
	CASE 6	17.75	521.53	-1.52
Beams of 4th STOREY	CASE 7	12.43	529.65	0.00
	CASE 8	17.75	529.65	0.00
Columns of 1st & 2nd STOREY	CASE 9	11.94	529.83	0.04
	CASE 10	17.16	530.36	0.14
Columns of 3rd & 4th STOREY	CASE 11	11.94	515.90	-2.56
	CASE 12	17.16	530.36	0.14
Beams & Columns of 1st & 2nd STOREY	CASE 13	9.07	534.06	0.84
	CASE 14	19.9	545.62	3.02
Beams & Columns of 3rd & 4th STOREY	CASE 15	12.27	528.61	-0.18
	CASE 16	17.55	528.94	-0.12
Basic structure with shear wall	CASE 17	-	506.84	-4.3
	CASE 18	-	487.43	-7.97

4.22.2 Roof Displacement

The Roof displacement for the four-storey building with different combination of element reinforcement at various floor levels is presented in Table 4.14.

It is observed that by increasing the reinforcement of beams only, there is a decrease in the roof displacement upto 3rd storey and after 3rd storey there is no change. The percentage change varies from 11.39% to 42.66%. However, the trends shown by increasing the reinforcement of columns only is a substantial increase in the roof displacement which varies from 9.09% to 30.01%. The combination of increase of reinforcement of beams and columns both show a little increase in the roof displacement upto 2nd storey and after 3rd storey it slightly decreases upto 4th storey.

There is a predominant decrease of 59.09% in roof displacement when shear wall is provided on the larger side of the building frame, whereas when it is provided on the smaller side the decrease is 58.92%.

Table: 4.14 Comparison of Roof Displacement

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENTS (mm)	PERCENTAGE CHANGE IN ROOF DISPLACEMENT
Basic structure			**143.6**	
Beams of 1st STOREY	CASE 1	10.56	127.1	-11.49
	CASE 2	26.76	123.4	-14.07
Beams of 2nd STOREY	CASE 3	4.72	138.4	-3.62
	CASE 4	15.78	130.5	-9.12
Beams of 3rd STOREY	CASE 5	12.43	142.9	-0.49
	CASE 6	17.75	141.2	-1.67
Beams of 4th STOREY	CASE 7	12.43	143.6	0.00
	CASE 8	17.75	143.6	0.00

Columns of 1^{st} & 2^{nd} STOREY	CASE 9	11.94	143.2	-0.28
	CASE 10	17.16	129.9	-9.54
Columns of 3^{rd} & 4^{th} STOREY	CASE 11	11.94	138.8	-3.34
	CASE 12	17.16	137.2	-4.46
Beams & Columns of 1^{st} & 2^{nd} STOREY	CASE 13	9.07	131.6	-8.36
	CASE 14	19.9	123.5	-14.0
Beams & Columns of 3^{rd} & 4^{th} STOREY	CASE 15	12.27	131.9	-8.15
	CASE 16	17.55	129.5	-9.82
Basic structure with shear wall	CASE 17	-	60.1	-58.15
	CASE 18	-	64.0	-55.43

4.22.3 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: 4.15 Variation of Roof Displacement with Base Force for all cases

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	BASE SHEAR (KN)	ROOF DISPLACEMENTS (mm)
Basic structure			**529.65**	**143.6**
Beams of 1st STOREY	CASE 1	14.65	517.65	107.1
	CASE 2	20	526.80	123.4
Beams of 2nd STOREY	CASE 3	15.78	494.78	138.4
	CASE 4	32.74	500.30	130.5
Beams of 3rd STOREY	CASE 5	4.17	529.95	142.9
	CASE 6	9.03	521.53	141.2
Beams of 4th STOREY	CASE 7	15.78	529.65	143.6
	CASE 8	32.74	529.65	143.6
Columns of 1st & 2nd STOREY	CASE 9	4.09	529.83	143.2
	CASE 10	39.23	530.36	129.9
Columns of 3rd & 4th STOREY	CASE 11	4.09	485.90	138.8
	CASE 12	39.23	530.36	137.2
Beams & Columns of 1st & 2nd STOREY	CASE 13	11.51	534.06	131.6
	CASE 14	30.66	545.62	123.5
Beams & Columns of 3rd & 4th STOREY	CASE 15	8.01	528.61	131.9
	CASE 16	27	528.94	129.5
Basic structure with shear wall	CASE 17	-	506.84	60.1
	CASE 18	-	487.43	64.0

4.23 EFFECT OF CHANGE OF REINFORCEMENT IN VARIOUS COLUMNS

Initially the reinforcement of all the columns is kept same (452mm^2) and then it is increased for each column, while the beam reinforcement is kept same as given by IS456:2000. The effect is studied by pushover analysis. The change in roof displacement for every case is given in Table 4.16.

Table: 4.16 Variation of Roof Displacement with column reinforcement for all cases

STRUCTURAL ELEMENTS	CASES	PERCENTAGE INCREASE IN REINFORCEMENT	ROOF DISPLACEMENT (mm)	PERCENTAGE DECREASE IN ROOF DISPLACEMENT
Basic structure			**145**	
Column 1	CASE 1	77.87	134.0	7.58
	CASE 2	177.87	112.9	22.14
Column 2	CASE 3	77.87	126.9	12.48
	CASE 4	177.87	105.6	27.17
Column 3	CASE 5	77.87	129.7	10.55
	CASE 6	177.87	108.6	25.10
Column 4	CASE 7	77.87	143.7	0.89
	CASE 8	177.87	132.8	8.41
Column 5	CASE 9	77.87	120.1	17.17
	CASE 10	177.87	102.2	29.4
Column 6	CASE 11	77.87	134.5	7.24
	CASE 12	177.87	115.3	20.48
Column 7	CASE 13	77.87	122.8	15.31
	CASE 14	177.87	106.1	26.83
Column 8	CASE 15	77.87	131.7	9.17
	CASE 16	177.87	109.2	24.69

4.24 PERFORMANCE BASED DESIGN

Optimal design is obtained by increasing the main reinforcement of various frame elements as discussed in section 4.13.

Target Roof Displacement for Immediate occupancy = 0.007 x 14m = 0.098m = 98mm
Target Roof Displacement for Life safety = 0.025 x 14m = 0.35m = 350mm

The triangular loading obtained for MCE is shown in fig. 4.30.

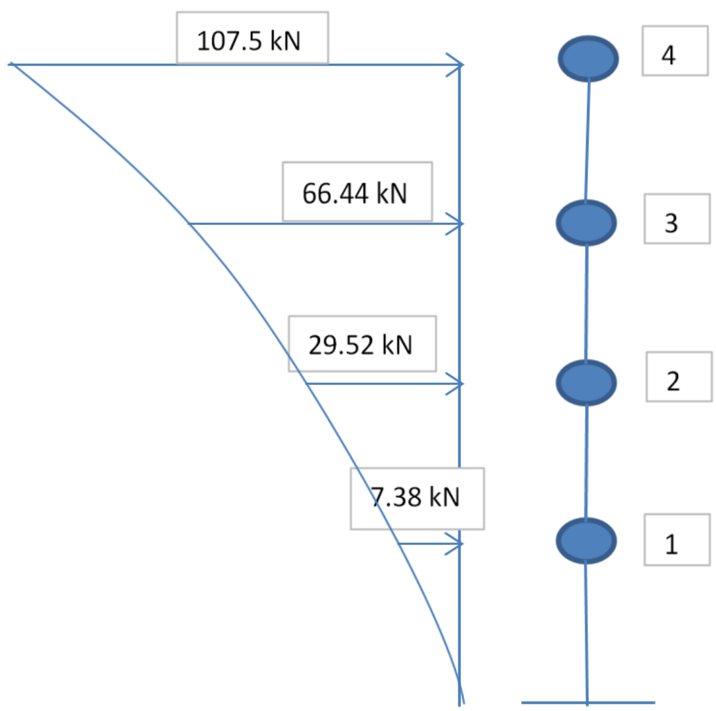


Figure 4.30 Triangular Loading for MCE

The reinforcement detail of the Performance based building design thus obtained is shown in Table 4.17. These are compared to the reinforcement obtained by seismic resistant design of building (according to IS 1893:2002) in STAAD.Pro.

Table 4.17 Comparison of reinforcement area in beams and columns for all designs

Element	IS 456:2000 Area (mm^2)	Performance based design	IS 1893:2002
Corner Columns 1st and 2nd storey	452	1260	1260
Corner Columns 3rd and 4th storey	452	905	1260
Mid-Frame Columns 1st and 2nd storey	804	1260	1260
Mid-Frame Columns 3rd and 4th storey	804	905	1260
Interior Column 1st and 2nd storey	804	1260	1260
Interior Column 3rd and 4th storey	804	905	1260
Beams 1st storey	707 (top) 550 (bottom)	940 (top) 550 (bottom)	940 (top) 550 (bottom)
Beams 2nd storey	680 (top) 550 (bottom)	865 Φ (top) 550 (bottom)	865 Φ (top) 550 (bottom)
Beams 3rd storey	600 (top) 550 (bottom)	785 (top) 550 (bottom)	785 (top) 550 (bottom)
Beams 4th storey	600 (top) 550 (bottom)	600 (top) 550 (bottom)	680 (top) 550 (bottom)

Following results are obtained for pushover analysis of performance-based design:

Base Shear = 609.013 kN

Roof Displacement = 82.2mm

Thus Roof displacement is less than target roof displacement.

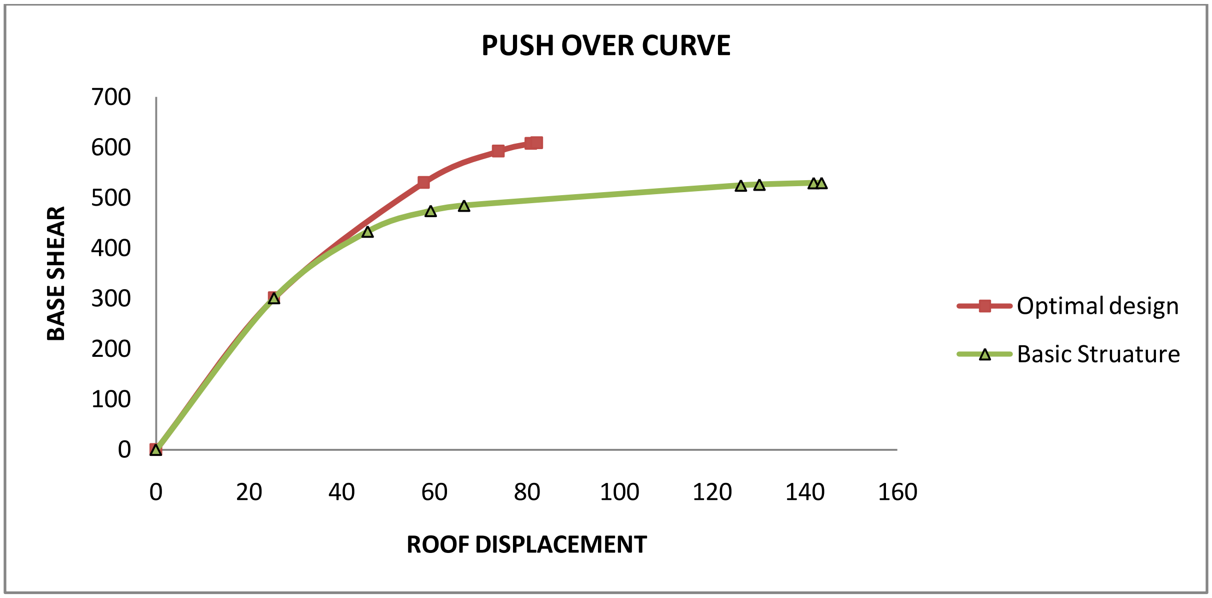


Figure 4.31 Pushover curve for Performance based design of four storey L-shaped building

5.1 GENERAL

In this work, Performance based seismic design of a four-storey symmetrical and a four storey unsymmetrical building has been done by evaluating their performance using pushover analysis. Reinforcement of various elements of the structure i.e. the beams and the columns was increased in different combinations and their effect on the performance of the structure was studied. The design of reinforcement was done in STAAD.Pro and analysis was carried out using SAP2000 nonlinear software tool. The effect of shear wall on the performance of the structure is also studied in this work.

5.2 CONCLUSIONS

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

1. Performance increases on increasing reinforcement of columns only resulting into an appreciable decrease in the maximum roof displacement both symmetrical as well as unsymmetrical building. Decrease in roof displacement is maximum interior column and for corner and mid-face columns it is comparable.
2. The increase in reinforcement of columns only results into a nominal increase in base shear, for both symmetrical building and un-symmetrical building. It is observed that changing reinforcement of 1st storey affects base shear more than other storeys.
3. Performance of the building decreases when the sectional sizes of beams and columns are reduced while keeping same reinforcement.
4. Increasing reinforcement of beams and columns both result in an appreciable decrease in roof displacement, for both symmetrical building and un-symmetrical building.
5. Provision of shear wall results in a huge decrease in base shear and roof displacement both symmetrical building and un-symmetrical building.
6. In L-shaped building when shear wall is provided on the larger side of the building results in a decrease of 4.3% in base shear and 58.15% in roof displacement and when provided on smaller side results in a decrease of 7.97% in base shear and 55.43% in roof displacement. Hence in unsymmetrical buildings shear wall must be provided on smaller side of 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 STAAD.Pro.

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 room for further improvement in the aforementioned method.

5.3 SCOPE OF FUTURE WORK

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

1. In the present study, the pushover analysis has been carried out for four storey 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 optimize the sizes of various frame elements using pushover analysis.
3. A comparative study can be done to see the effect of shear reinforcement on performance based seismic design.

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