SEISMIC ANALYSI S OF G+7 BUILDING BY PERFORMANCE BASED DESIGN

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF DEGREE

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Submitted by:

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CANDIDATE’S DECLARATION

I, Saksham Gupta (2K18/STE/13), student of M.Tech (Structural Engineering), hereby declare that the project Dissertation titled ‘**SEISMIC ANALYSI S OF G+7 BUILDING BY PERFORMANCE BASED DESIGN’** which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of and Degree, Diploma Associateship, Fellowship or other similar title or recognition.

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Date : 01.08.2020

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CERTIFICATE

I hereby certify that the Project Dissertation '**SEISMIC ANALYSIS OF G+7 BUILDING BY PERFORMANCE BASED DESIGN**' which is submitted by Saksham Gupta (2K18/STE/13) to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the students under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University or elsewhere.

Place: Delhi — Mr. Hrishikesh Dubey

Date: 30/08/2020 — (Assistant Professor)

ABSTRACT

PBSD is an approach of designing of any complexity of building. A building constructed in this way is required to meet certain measurable or predictable performance requirements, such as energy efficiency or seismic load, without a specific prescribed method by which to attain those requirements. Building performance is an indicator of how well a structure supports the defined needs of its users. The performance-based design approach is not proposed as an immediate substitute for design to traditional codes. Rather, it can be viewed as an opportunity to enhance and tailor the design to match the objectives of the community. It basically evaluates how building systems are like to respond under a variety of conditions associated with potential hazardous events. A G+7 Residential building has been analyzed by ETABS at various values of PGA as defined by IS 1893:2016, reports by National Disaster Management agency (NDMA, 2011), World Conference on Earthquake Engineering (WCEE, 2012), 0.1g and 0.2g and various post-analysis results are shown like PP, roof displacements in X and Y-directions, storey drifts, hinges result, roof displacements and compared it with FEMA document and ATC-40.

ACKNOWLEDGEMET

The success of a Major 2 project requires help and contribution from numerous individuals and the organization. Writing the report of this project work gives me an opportunity to express my gratitude to everyone who has helped in shaping up the outcome of the project.

I express my heartfelt gratitude to my project guide **Mr. Hrishikesh Dubey** for giving me an opportunity to do my Major 2 project work under his guidance. His constant support and encouragement has made me realize that it is the process of learning which weighs more than the end result. I am highly indebted to the panel faculties during all the progress evaluations for their guidance, constant supervision and for motivating me to complete my work. They helped me throughout by giving new ideas, providing necessary information and pushing me forward to complete the work.

I also reveal my thanks to all my classmates and my family for constant support.

NOTATIONS AND ABBREVIATIONS

PBSE	Performance based Seismic Engineering
PBSD	Performance based Seismic Design
PL	Performance Level
OP	Operational
IO	Immediate Occupancy
LS	Life Safety
CP	Collapse Prevention
DS	Damage State
ATC	Applied Technology Council
FEMA	Federal Emergency Management Agency
PA	Pushover Analysis
Sa	Spectral Acceleration
Sd	Spectral Displacement
MDOF	Multi-degree of freedom
SDOF	Single-degree of freedom
IS	Indian Standards
PGA	Peak Ground Acceleration
PP	Performance Point
CS	Capacity Spectrum
RS	Response Spectrum

IRCC	Inter-Jurisdicational Regulatory Collaboration
ICC	International Code Council
ADRS	Acceleration-Displacement Response Spectra

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CHAPTER 1

INTRODUCTION

1.1 BRIEF

The guarantee of PBSE is to deliver structures with predictable seismic performance. A PBSD is the method of designing a building which must satisfy the requirements of the owner of the building or say, demand of the structure. In this, the capacity i.e. the performance of the building should be equal to or more than the demand of the structure. This study does investigation of multistory RC surrounded structures exposing them to monotonically expanding sidelong powers until the preset target displacement is reached.

As the consequences of the most deadly earthquakes that occurred in past are very well known, it is a need of an hour to design the structures in a manner that the persons residing in the building should remain safe. The Earthquake, being a natural phenomenon cannot be prevented but the thing we can do is to make the structures so reliable that it can resist the ground acceleration caused by the same.

The two keys terms used in this study are the ‘Hazard’ and ‘DS’. The output seismic response of a structure is basically designated by the maximum permitted PL for the input seismic ground motions.

1.2 BUILDING PERFORMANCE LEVELS

Basically, the response of any structure can be defined as a mix of structural and non-structural harm. The Performance destinations are identified with the normal harm that a structure may encounter comparing to a tremor ground movement and the impacts of that harm. The different Performance goals that are generally considered are OP, IO, LS and CP and various DS are illustrated in Fig 1.1.

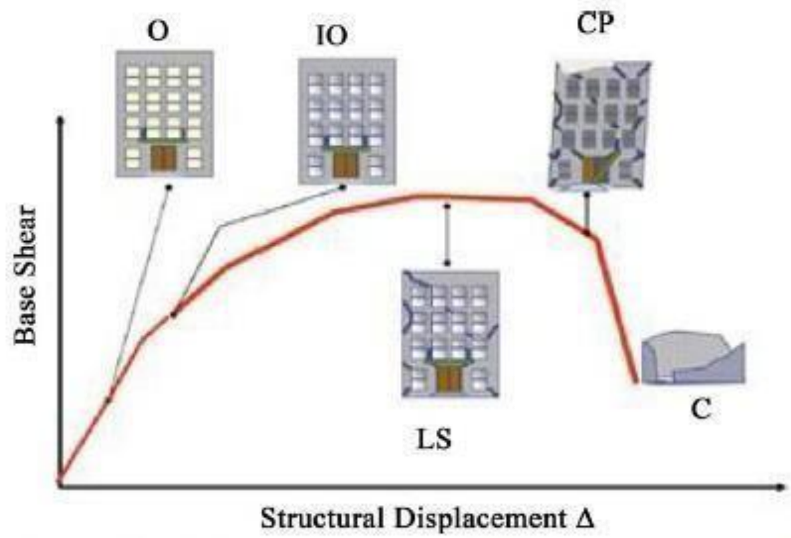


Fig 1.1 Building Performance level

The various PL of the building with corresponding target roof displacement are tabulated in Table 1.1.

Table 1.1 Structural performance and Target roof displacement [1]

Performance level	Structural Performance	Target Roof Displacement
Operational (OP)	Very light damage	0.37%
Immediate Occupancy (IO)	Light damage Minor cracking	0.70%
Life safety (LS)	Moderate damage Building may be beyond economical repair	2.5%
Collapse Prevention (CP)	Severe damage Large permanent drifts	5.0%

1.3 OBJECTIVES AND SCOPE

The Kutch earthquake of 26 January 2001, also known as Bhuj earthquake that cause huge destruction and taken away many lives questioned about the buildings by-laws, professional practices, construction materials, building design. Hence, it is required to upgrade our construction practices such that life of persons can be preserved as much as possible. Today, various documents are available for e.g. ATC-40 and FEMA356 that are created, modified by the well- known organizations that provide various guidelines and methodologies to be adopted to make the structure safe and reliable when it is subjected to ground motions.

The main objective is to study & analyze the RC framed building for finding out the seismic burden conveying limit of structures. Here, we model a 8-level Reinforced Concrete Framed Building and register the Seismic Response of the structure regarding Base Shear, Floor Drift, Sa, Sd and Story Displacements etc. then compare these Displacements with the Target Displacements given by FEMA documents.

The extent of the current examination predominantly focuses on the plan (according to IS 456:2000) and evaluation of the building using the IS 1893-2016 and ATC 40 and analyzing. In this analysis various procedures Such as Equivalent Static method (Linear Static Procedure), RS Analysis (Linear Dynamic Analysis) and PA (Non-Linear Static Analysis) are performed using ETABS 2018.

The above methodology is utilized to design a G+7 storied Reinforced Concrete Building located in zone IV (Zone Factor = 0.24) as per IS 1893-2016.

1.4 METHODOLOGY

The methodology adopted was to first gain the knowledge and understanding the PBSD philosophy by reviewing the research papers. Then, the building has to be modeled in ETABS 2018 and the loads will be applied as per Indian Codes followed by analyzing the structure by defining the non-linear hinges at appropriate locations in the frame. After analysis, the various results can be seen in the form of curve and tables and can be compared.

1.5 ORGANISATION OF THESIS

Chapter 1 ‘INTRODUCTION’ presented the brief of PBSD, various PL and its respective structural performance and target roof displacement, objectives and extent of the work, building subtleties and the technique adopted.

Chapter 2 ‘LITERATURE REVIEW’ shows the background studies of PBSD by reviewing the research papers presented by various students and professors.

Chapter 3 ‘PERFORMANCE BASED DESIGN’ shows the theory of this approach including its need, history, design process, PL etc.

Chapter 4 ‘METHODOLGY’ illustrates the various methods of analysis.

Chapter 5 ‘CALCULATIONS’ shows the manual calculations carried out in various methods.

Chapter 6 ‘RESULTS’ shows the post-analysis results which are illustrated in tabulated form.

Chapter 7 ‘SUMMARY AND CONCLUSION’ basically shows the summary of this study and its future degree as well.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The literature survey of several PBSD philosophy adopted was gathered and shown in upcoming section 2.2.

2.2 LITERATURE REVIEW

Rehan Md. and Azam Faiyaz, 'Performance based design of RCC Structure'[2], (2018) referenced that the expectation of inelastic seismic reactions and assessment of seismic performance of a structure are significant viewpoints in Performance based seismic plan. In this examination, the seismic presentation of a Reinforced Cement Concrete Building (20 story) situated in Zone V (as per IS 1893) assessed by Pushover investigation and Non-Linear Time History investigation are looked at. The outcomes show that PA is precise enough for functional applications in seismic performance assessment when contrasted and the non-direct powerful examination of MDOF framework.

In their research, El Centro, Kobe and Northridge was used as input earthquake and base shear (kN), top roof displacement (mm), Sa (g) and Sd (mm) are compared for all these three input earthquake. Response Spectra Base Shear (kN) and Time-History Base Shear (kN) are also compared in this research.

The conclusion was that the 20 storey Reinforced Concrete building deforms into the inelastic range which leads to yielding of some of the beams and columns for the seismic intensity of 0.36 PGA.

Dr. Rehan A. Khan, 'Performance based seismic design of RC Building'[3], (2014) finished assessment on a five story RCC building present in Seismic zone IV is organized using SAP 2000 and the technique breaks down the constraint of the structure (as a PC) of a MDOF system with different demands on the structure. The methodology is characterized in acceleration displacement ordinates. The proposed strategy is presented by finding the seismic PP for the same building arranged in Zone-IV, adjusted in plan (designed as per IS 456:2002) presented to three particular PGA levels. A wide parametric assessment is coordinated to investigate the effect of various noteworthy limits on the PP. The limits consolidate effect of input ground movement on the PP, varying degree of reinforcements in various segments. The aftereffects of investigation are expressed as base shear and story drifts.

The conclusion was summarized below:

- As PGA increases, base shear and roof displacement increases.
- As the reinforcement of the section increases, base shear increments and rooftop displacement diminishes.
- PP is marginally influenced by response reduction factor.

Dilip J. Chaudhary, Gopal O. Dhoot, 'Performance based seismic design of RC building'[4], (2016) concluded that the pushover analysis (i.e. non-linear static procedures) is used to investigate the presentation of structure under steadily expanding sidelong loads. The plastic pivot arrangements in basic individuals are gotten after the examination alongside other basic boundaries which legitimately show the presentation of part after a tremor. In this paper, a four-story Reinforced Concrete structure is displayed and planned according to IS 456:2000 and analyzed for LS PL in SAP2000 v17.

From the analysis, it may be concluded that the PL of the building is as per the assumption. The conclusion was that design building lies in between IO and LS range.

Priestley MJN, 'Performance based Seismic Design of RC Buillding'[5], (2000) in his work have outlined and compared three techniques which are (a) the CS approach, (b) the N2 method and (c) direct displacement-based design for PBSD method for the assessment of seismic forces in an existing structure or for the new construction. These three methods are compared with conventional force based seismic design. He emphasized mainly on the need, residual displacement, and incorporation of soil structure interaction in PBSD. He concluded that the main merit of this design method is its simplicity. According to him, there are ramifications of performance limit states to seismic plan of the structure. He built up another methodology that depended on plan so that to accomplish a predefined strain or drift PL under a predetermined seismic ground movement. It was easy to utilize and furthermore brought about uniform degrees of Seismic hazard.

Fajfar et al, 'Capacity Spectrum Method based on inelastic demand spectra'[6], (1999) presented a straightforward nonlinear methodology for the seismic analysis of structures (the N2 method). It combines the investigation of a multi-level of freedom (MDOF) model with the reaction range examination of an equal single-level of-opportunity (SDOF) framework. The system is created in the increasing speed dislodging position which allows the visual translation of the technique, rather than versatile spectra with comparable damping and period, was applied. Generally, the results of the N2 method are correct enough, as long as the structure oscillates predominantly within the first natural mode. In this study, the method is represented and mentioned, and its basic derivatives are given, the similarities and differences between the proposed methodology and FEMA 273 and ATC 40 nonlinear static analysis procedures are also mentioned.

Mahaney, Freeman et al, 'Review of the development of Capacity Spectrum Method'[7] **, (1993)** utilized the CSM in four examinations of structures to survey the seismic response after the Loma Prieta Earthquake. He has shown a couple of structures including one-story and two-story wood-diagram living courses of action, an eleven-story Reinforced concrete (RC) shear wall building and a couple of confined structures with block in-filled dividers. In this examination, the Acceleration-Displacement Response Spectra (ADRS) plan was initially introduced. The results showed that the damped flexible quake displacements demands need not be proportionate to the real inelastic displacements demands as he had anticipated. It was similarly communicated that the harm foreseen by the CSM was extremely near with the watched harm for the eleven-story Reinforced Concrete shear divider building.

Ghobarah Ahmed, 'Performance based Design on Earthquake Engineering'[11] **(2001)** presented a state of development of PBSD in earthquake hazards. As indicated by his investigation, the design objectives are to address LS and to control harm in minor and moderate seismic tremors whereas in severe earthquakes, collapse should not take place. He studied that the buildings designed according to prescribed codes performed well from LS point of view but the damage cost and cost of repair is too high. To reduce these high costs, various degrees of performance targets should be thought of which requires traditional design with relevant up-gradation.

Distinctive PL and their relating basic structural qualities, for example, quality, stiffness and capacity overwhelm the performance of the structure, however if any middle of the performance objective levels are chosen, at that point it turns out to be very hard to characterize that what characteristic command the performance.

Performance of a structure is mainly ordered into 5 levels, OP, IO, Damage Control, LS and CP.

These above mentioned PL are related with seismic tremor danger levels with return period as shown in table below.

Table 2.1 Earthquake Hazard Level

S.NO.	EARTHQUAKE FREQUENCY	RETURN PERIOD IN YEARS	PROBABILITY OF EXCEEDANCE
1	Frequent	43	50% in 30 Years
2	Occasional	73	50% in 50 Years
3	Rare	475	10% in 50 Years
4	Very Rare	970	5% in 50 Years
5	Extremely Rare	2475	2% in 50 Years

Pang Weichiang et al, 'Performance based Seismic Design of Six-Storied Wooden frame Structure'[12], (2008) have done the study based on the seismic performance of a Six Story wooden frame building by adopting direct displacement procedures developed for medium rise buildings which uses modal analysis and linearization techniques along with shear walls. He has selected multiple objectives and has not does done non-linear time history analysis of their structure. It was seen that wooden frames were effective in protecting the human life under seismic forces but they were not reliable in limiting damage to the structure. The design done was much better than traditional force-based approach and also doesn't require determination of force reduction factor.

Karapetrou, Pitilakis et al, 'Seismic Vulnerability of RC Building under the effect of Ageing' [13], (2017) have studied the performance degradation of RC buildings over time due to ageing effects. Probabilistic modeling of chloride induced corrosion is considered by performing 2D incremental dynamic analysis (IDA) to calculate seismic performance of un-corroded RC frame i.e. at t=0 and corroded for a life of 25, 50 and 75 years respectively. It was seen that for given corrosion situation pillars were more influenced than the sections of the structure as beams include reinforcements of relatively lower diameters. All the results were produced with the assumption that uniform corrosion effects must be considered.

Mander J B, 'Future directions in Seismic Design and Performance based Engineering'[14], (2001) concluded that in order to advance in the field of seismic resilient structures, innovative work exercises should be focused on the PBSD method which gives adaptability to the architect in-control to tell customers/proprietors of the probabilistic level of harm to empower a more hearty administration of seismic hazard to achieve anticipated PL, thus it becomes to adopt displacement-based design rather than traditional force-based design standards.

This improved plan approach alone would not bring about a prevalent degree of seismic versatile structure, anyway rather lead to a superior norm of the characterized PL when the post-earthquake outcome is going to be acknowledged with a particular degree of confidence.

Mauro Nino, A. Gustavo Ayala, Rafael Torres, 'Uniform Hazard Spectra for the Performance based Design of Structures'[15], (2004) builds up a methodology in this paper that decides the uniform danger spectra material in the PBSD of the structures. An eight-celebrated RC building situated in Mexico City has been examined. The central time frames for the structure in the particular versatile and inelastic states are T1=0.89 and T2=1.826 sec. The subsequent spectra relate to seismic plan targets made by pair of specific performance and design levels. It might be seen that the structure powers of the components acquired with the plan spectra for Damage Index DI_{BB} are larger than those obtained with the ductility design spectra. This is because the dissipated hysteretic energy in the DI_{BB} spectra, therefore the structure needs to have a larger strength than that required by maximum displacement ductility alone.

Goel RK and A.K. Chopra, 'Evaluation of Modal and FEMA Pushover analysis'[16], compares the Modal Pushover Analysis system with the nonlinear reaction history investigation (RHA). Seismic requests are assessed for six structures, each dissected for 20 ground movements. It was reasoned that as the quantity of modes expands, the stature savvy appropriation of story floats and plastic pivots assessed by MPA is very like outcomes acquired from nonlinear RHA. The exactness of the MPA approach was end up being adequate for the majority of the structure plan and retrofit applications. Notwithstanding, the case isn't valid for structures that twist well into the inelastic range with significant corruption in sidelong limit: for example 20-celebrated structure situated in Los Angeles exposed to extreme ground movements (2% likelihood of exceedance in 50 years). For such cases, MPA can't be relied upon to give good aftereffects of seismic requests, and ought to be abandoned, in this manner nonlinear RHA gets mandatory.

B.Ghosh, J.W.Pappin, K.M.O Hicyilmaz, 'Seismic Hazard Assessment in India' [17], (2012) made a study to re-consider the hazard zoning and to revise the boundaries of zoning areas in India. A preliminary site-explicit probabilistic seismic hazard assessment (PSHA) has been completed to survey the expected danger of seismic zones considering the most recent accessible tremor data in India. The PSHA technique fundamentally consolidates the information on seismic zones and their related tremor recurrence with appropriate attenuation connection to deliver hazards curves as far as level of seismic ground movement and their related likelihood of being exceeded in a year.

The outcomes are then contrasted with the traditional Indian seismic code and those from the ongoing investigation done by the Indian National Disaster Management Authority (NDMA). In any case, some Indian buildings are situated in the high seismicity zones of the dynamic Himalayan plate limit yet are grouped to be in Zone IV. This work shows that the DBE level seismic hazard is disparaged in these regions. They inferred that further collective examination is a need of an hour to refresh the seismic hazard map in the India tremor code.

Anil K. Chopra and Rakesh K. Goel, 'A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings'[18], (2001) The principle goal of this examination is to make an improved method for pushover investigation which depends on hypothesis of basic elements which incorporates invariant force conveyance and gives better exactness in evaluating seismic demands on structures. Right off the bat, the investigation of one-story framework demonstrated that pushover examination portrays consummately top seismic demands followed by building up a Modal Pushover Analysis (MPA) method for straightly flexible structures and show that it is proportionate to the linear dynamic methodology for example RS analysis method. At long last, the seismic requests on the structure controlled by PA utilizing three force distributions in FEMA-273 are contrasted and the MPA and nonlinear RHA approaches. The 9-story structure was likewise planned by Brandow and Johnston Associates for the SAC2 Phase II Steel Project. The vibration time frames for the initial three modes are 2.27, 0.85, and 0.49 sec, individually.

Pushover examination of a one-story inelastic framework portrays flawless pinnacle seismic requests: hinges plastic rotation, joint revolutions and so on. Nonetheless, the prime disadvantage of pushover investigation is that it can't give any aggregate proportion of reaction. This MPA method for flexible structures is demonstrated to be comparable to the Response Spectrum Analysis. In light of results introduced for El Centro ground movement scaled by factors shifting from 0.25 to 3.0, the mistakes in the MPA methodology are demonstrated to be just weekly reliant on ground movement force. This implies MPA can pass judgment on the reaction of structures reacting great into the inelastic range to a comparative level of precision when contrasted it with standard Response Spectra Analysis for assessing the pinnacle response of the elastic frameworks. In this way the precision of MPA technique is very high for commonsense application in building assessment and plan.

Farzad Naeim, Hussain Bhatia and Roy M. Lobo, 'Performance based Seismic Engineering'[19], have done a study on seven-storey Reinforced Concrete building and its total weight is 10,540 kips. The examination has been done in N-S and E-W bearings. The structure was pushed to a dislodging of 2.8 inches in the negative E-W heading and 4.34 inches in the negative N-S heading. The outcomes acquired after investigation shows that various sections which are supporting above dividers have turn past breakdown. Number of dividers and bars additionally had plastic revolutions past the Life Safety prerequisite at target uprooting. Obviously, this structure doesn't meet the acknowledgment measures of the fundamental wellbeing objective.

CHAPTER 3

PERFORMANCE BASED DESIGN

3.1 BRIEF

PA is a non-linear static examination finished under conditions of the reliable gravity loads and persistently increasing horizontal burdens until the structure becomes unstable (i.e. sufficient numbers of plastic hinges are formed) or a predefined limit is reached.

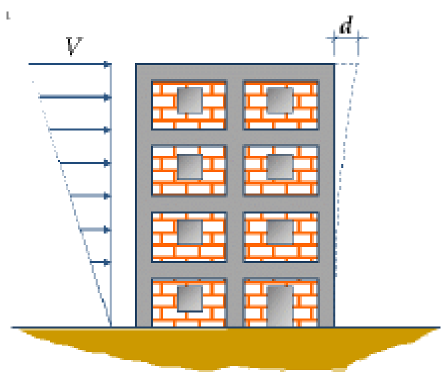


Fig. 3.1 Building subjected to lateral displacement

Force Based design is fundamentally a customary technique for doing the seismic examination of the structure. Using the RS procedure, the structure parallel forces on the Building are settled and the elements are arranged so as to withstand those forces. In this procedure, there is no extent of the twisting capacity of a section or of the structure with everything taken into account.

In PBSE, the distortions of the element and the structure as a whole taken into account are assessed under the horizontal forces of a quake that is required to occur at the territory of the structure. The strains better adds up to assess damage rather than stresses. A presentation based assessment requires a nonlinear sidelong burden versus distortion curve as the harm is depended upon to go past the flexible curve.

The flow chart showing the process of PBSD has been shown in Fig. 3.2.

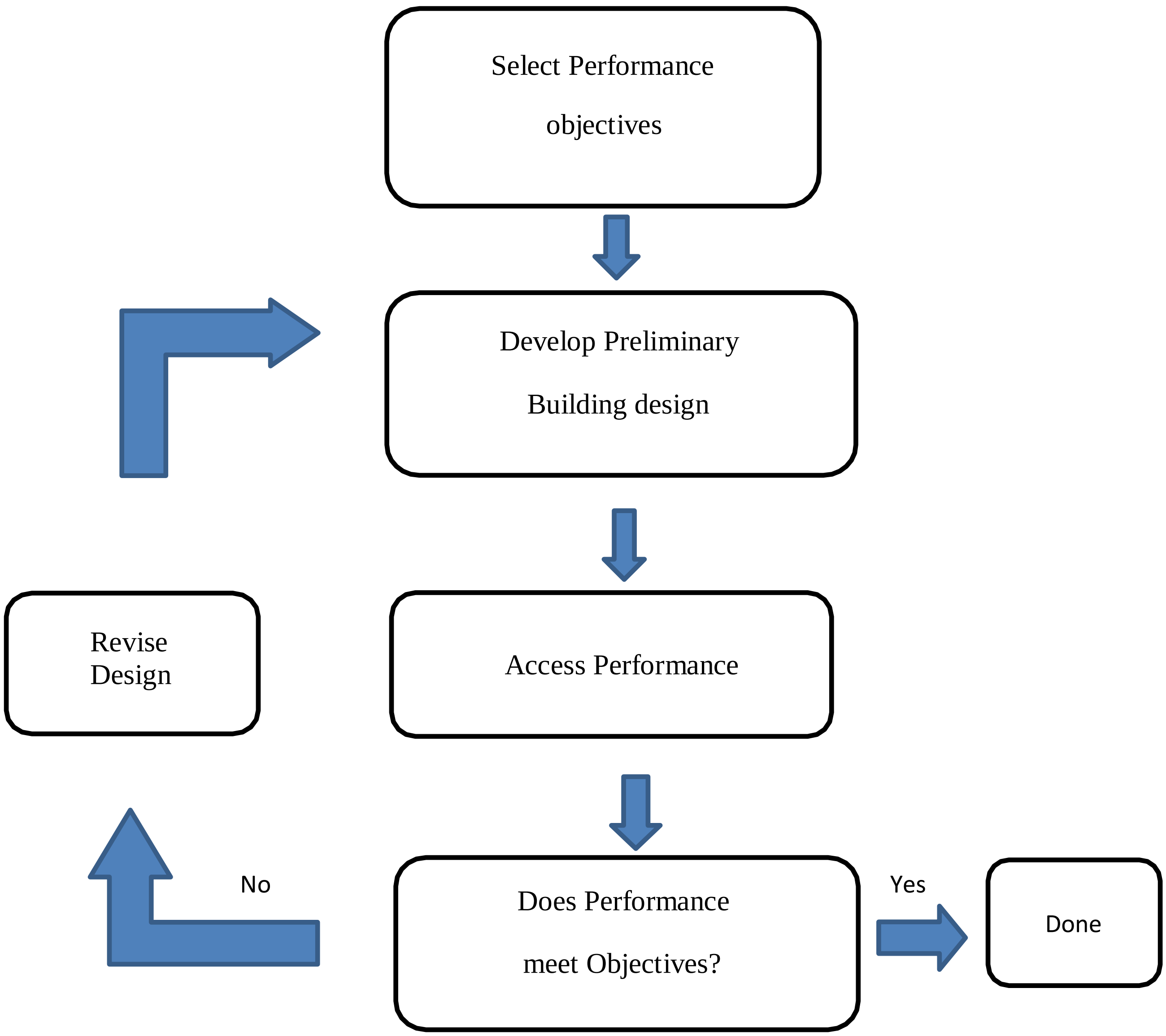


Fig. 3.2 Flow Chart[8]

3.2 NEED OF PERFORMANCE BASED DESIGN

The conventional design codes allow the normal use buildings (i.e. Residential, Commercial Buildings, not Dams, Nuclear Plants) to be designed to reach in-elastic deformations during any seismic activity.

The in-elastic deformations in the building caused by the earthquake leads to the energy dissipation and the design are more economical. Even if we design the building to remain elastic at a certain level of demand, the demand can be larger: We need to make sure that after the elastic capacity has been reached, the building can deform in-elastically without collapsing.

A PBSE is central theme for many research activities and modeling of structural and non-structural building systems, transportation networks and developing the computational approaches needed to predict performance. Most of the PBSD philosophy focuses on earthquake hazards, as they are random and most destructive in nature but it can also be applied to wind and ocean wave hazards.

3.3 HISTORY

PBSD of structures is been executed since mid-twentieth Century. Britain, Australia and New Zealand and many other nations had Performance- Based building codes available for decades. More than a century ago, when 1908 Urban Centre Earthquake occurred in European Nation, the intelligence activity committee appointed by the government of that country gave recommendations to design the structures that are earthquake resistant. The Geophysics Field Survey, America Coast and Geodetic Survey, USA have installed first strong motion seismograph in 1932, which recorded ground motions of Long beach earthquake,1933.

The linear dynamic analysis was made mandatory in Los Angeles for structures above 50 m height after the San Fernando Earthquake in USA. The IRCC is an international building administrative association of 10 countries framed to encourage global conversations of performance based administrative plan frameworks with a prime spotlight on distinguishing administrative foundation, instruction, and innovation issues identified with actualizing and dealing with these frameworks. The ICC in the United States had a presentation code accessible since 2001 (ICC, 2001). During 1989, a project was executed to develop basic engineering guidelines to retrofit the existing building whereas the main recommendations of the project were that the standards and rules to be adequately adaptable in order to oblige an assortment of structure-explicit seismic hazard decrease strategies for new structure development.

The underlying structure record, The National Earthquake Hazards Reduction Program (NEHRP) gave the standards for the Seismic Rehabilitation of Existing Buildings. FEMA 273 thus contained an extent of formal performance objectives that identify with the decided degrees of seismic ground development. The show levels were requested with the titles of OP, IO, LS, and CP. After the event of Northridge, Structural Engineers Association of California (SEAOC), 1995 put forth a PBSD strategy (also known as Vision 2000).

3.4 DESIGN PROCESS

As talked previously, PBSD method is an iterative philosophy that begins with the assurance of the Performance Objectives (as indicated by client), followed by the improvement of a Preliminary Design, surveying whether the arranging meets the targets or not and inevitably updating it, if necessary, till the necessary PL is accomplished.

The various steps of the designing process are explained in the upcoming sections.

3.4.1 SELECT PERFOMANCE OBJECTIVES

This method begins with the decision of structure rules imparted inside the sort of in any event one performance goals. Performance objectives are basically the depiction of the worthy threat of getting and thusly the earth shattering mishaps that may happen in view of this mischief, contrasting with a specific level of seismic hazard. Since misfortunes are regularly identified with either structural damage, nonstructural harm or both, therefore the performance objectives ought to be communicated thinking about the normal performance of each structural and nonstructural component. These performance objectives are often expressed in 3 completely different risk formats which are briefed below:

An ***Intensity-Based Performance*** objective can be characterized as an evaluation of the adequate degree of loss that might be normal, given that a power of ground shaking having a multi-year return period occurs, the expense of fix ought not surpass 20% of the structure's substitution value, no death toll or essential injury should occur, and re-inhabitance time ought not surpass 30 days.

A ***Scenario-Based Performance*** objective could be a characterized as an evaluation of the worthy degree of loss which might be normal, given a specific tremor of size 7.0 quake happens, the expense of fix ought not surpass 5% of the structure substitution cost, no death toll or imperative injury should occur, and inhabitance of the structure ought not be hindered for about seven days.

A ***Time-Based Performance*** objective is an assessment of the sufficient probability inside a particular time length that a given level of adversity will occur or outperformed, considering all the quakes that may influence the structure in this time span and therefore the probability of every occasion.

Fig. 3.3 depicts the different levels of the hazards, its vulnerability and the related cost of the damage in the building.

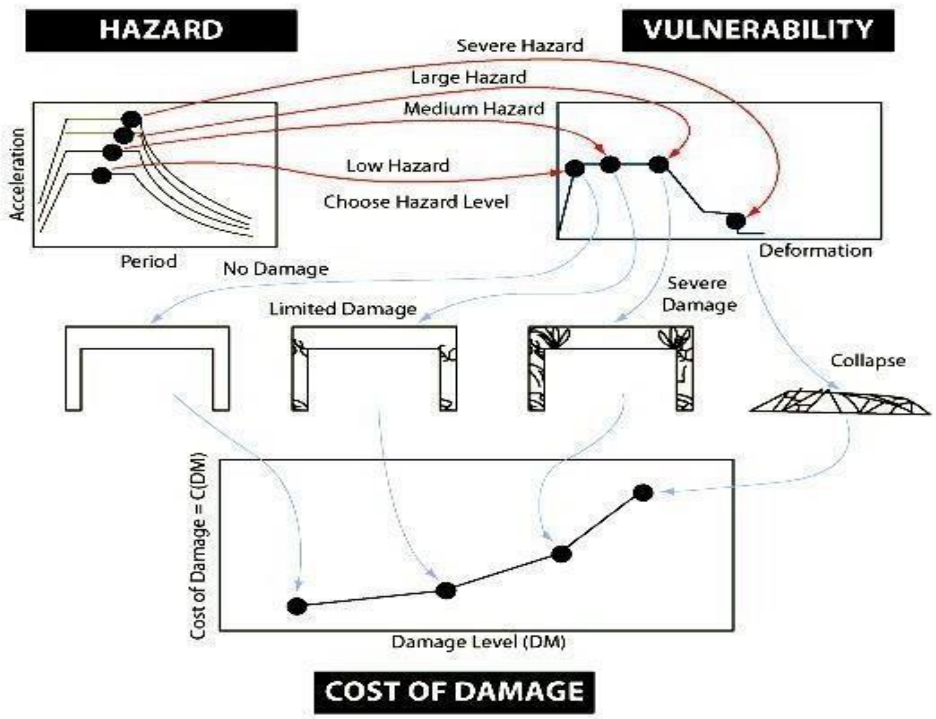


Fig. 3.3 Different Levels of Hazards

3.4.2 DEVELOP PRELIMINARY DESIGN

The advancement of preliminary plan for any structure incorporates characterizing the assortment of different key structure characteristics which may significantly influence the performance limit of the structure. These qualities are:

- Nature and location of the site.
- Building configuration, story height, number of stories, presence of irregularities and floor plate arrangement at every story etc.
- Basic structural system, for instance, steel frame, moment resisting frame or masonry bearing walls.
- Presence of any protecting technologies, for instance, seismic isolators, energy dissipation devices for e.g. Dampers, or any damage-resistant components.

- Approximate sizes of various structural and nonstructural elements and their placement.

Determination of an adequate starter structure idea is significant for viably and productively executing the performance based plan technique in our approach. Improper starter structures may wind up in broad emphasis before we reach to a satisfactory answer, or may likewise bring about arrangements that don't productively meet our presentation targets.

At present, engineers have scarcely any benefits on which to base a starter structure for meeting a specific introduction objective. Some check with the accessible current code arrangements, others may perhaps allude to original performance based plan systems, and some may utilize an extra instinctive methodology.

3.4.3 Assess Performance

After the improvement of preliminary plan, a movement of recreations (assessments of building response to seismic stacking) is performed to review the performance of the structure.

Performance assessment includes the subsequent steps:

- Characterization of the ground shaking hazard.
- Analyzing the structure so as to determine its probable response and hence the intensity of shaking transmitted to supported nonstructural elements as a function of ground shaking intensity within the case of maximum loading, which may be imparted by a severe earthquake, simulations could also be performed making the use of nonlinear analysis techniques.
- Determination of the normal harm that may happen to the building.
- Determination of the harm that may occur to the nonstructural components.
- Determination of the capital misfortune, any causality and inhabitance misfortunes.

- Calculation of the normal future misfortunes as an element of power, auxiliary and nonstructural reaction.

To make a presentation evaluation of the structure, factual connections between quake hazard, building reaction, harm and expected misfortune are required. Basically, the strategy incorporates the improvement of 4 kinds of probability limits identified with functions of hazard, response, damage and loss and afterwards controlling these capacities to assess conceivable misfortunes.

Hazard functions are fundamentally the numerical articulations of the likelihood that a structure is exposed to ground shaking of different force levels, though power could likewise be communicated as far as PGA, Spectral Response Acceleration and so forth.

Response functions are the scientific articulations of the likelihood of bringing about shifted levels of building reaction, given that building is exposed to very surprising degrees of ground shaking power. Building reaction is communicated inside the various boundaries which are gotten from basic investigation, along with member forces, story floats, joint plastic turn demand, and floor increasing velocities and so on. The most recent strategy for relating the quake harm to sources of info and registering the harm vulnerabilities is utilizing the Fragility Curves.

Damage functions are the scientific articulations of the likelihood that the individual structural and nonstructural components, or the structure overall, will be harmed to totally various levels, as long as totally various degrees of ground shaking happen.

Loss functions are the numerical articulations of the restrictive likelihood of the differed misfortunes, along with repair and replacement costs, casualties, and re-occupancy time period, in light of condition that the specific harm happens.

3.4.4 REVISE DESIGN

In case the performance meets or outperforms its objectives, the arrangement is done anyway if not, the structure ought to be upgraded in an iterative method until the objectives are met. Mostly, it may not be feasible to meet the communicated goals at reasonable cost; taking everything into account, some unwinding up of the first perfromance goals may likewise be given.

3.5 PERFORMANCE LEVELS

The choice of performance goals for a structure should be possible by a gathering of chiefs, executives which incorporates the structure proprietor (customer), plan experts, and building authorities and so forth.

3.5.1 Definition

The plausible post-tremor state of a structure a very much characterized point on a scale that measure the extent of misfortune brought about by the seismic ground shaking. Notwithstanding setbacks, misfortune may likewise be as far as property and re-operational capacity.

3.5.2 Building Performance Level

The overall structure PL is a blend of basic structural and a non-structural parts PL. The four Building PL are CP, LS, IO, and OP. Each Building PL is made out of Structural PL which portrays the compelling mischief state of the fundamental systems of a structure and a nonstructural PL that delineates the binding harm condition of the nonstructural fragments in the structure. Three Structural PL and four Nonstructural PL are used to outline the four fundamental Building PL discussed previously.

The PL of various structural and non-structural segments in the building are graphically represented in Table 3.1.

Table 3.1 Building Performance Levels

Nonstructural Performance Levels	Structural Performance Levels/Ranges					
	Immediate Occupancy S-1	Damage Control Range S-2	Life Safety S-3	Limited Safety Range S-4	Collapse Prevention S-5	Not Considered S-6
Operational N-A	**Operational** 1-A	2-A	Not Recomm.	Not Recomm.	Not Recomm.	Not Recomm.
Immediate Occupancy N=B	**Immediate Occupancy** 1-B	2-B	3-B	Not Recomm.	Not Recomm.	Not Recomm.
Life Safety N-C	1-C	2-C	**Life Safety** 3-C	4-C	5-C	6-C
Hazards Reduced N-D	Not Recomm.	2-D	3-D	4-D	5-D	6-D
Not Considered N-E	Not Recomm.	Not Recomm.	3-E	4-E	**Collapse Prevention** 5-E	Not Recomm.

The Building PL are assigned alphanumerically with number (at former place) which speaks to the Structural PL and a letter set speaking to the non-auxiliary PL (for example 1-B, 3-C).

- **OPERATIONAL LEVEL (1-A)**

This can be defined as a combination of the Structural IO Level and the Non-Structural OP Level.

Structures meeting this standard are required to encounter irrelevant or express no mischief to their essential and non-fundamental segments. The structure is suitable for the inhabitance and use, anyway maybe to some degree obstructed mode like water, power and various utilities and apparently with some unnecessary utilities not working. Structures satisfying the need of this PL may cause basically no hazard to life wellbeing.

- **IMMEDIATE OCCUPANCY LEVEL (1-B)**

This can be named as a mix of the Structural and Non-Structural IO levels. Structures meeting this rule are foreseen to experience immaterial (or no) harm to their auxiliary parts yet just non-structural components may endure minor harm. Along these lines, however quick re-inhabitance of the structure is practical, some fix and cleanup ought to be fundamentally performed and hold on for the modifying of utility help.

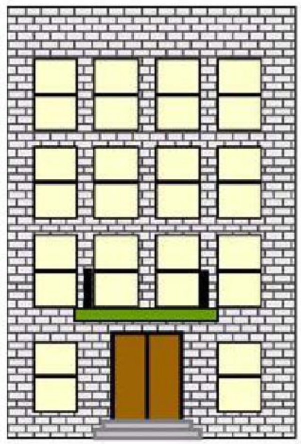
- **LIFE SAFETY (3-C)**

This consists of LS levels to both the Structural and Non-Structural components. Structures meeting this model may encounter noteworthy harm to both basic and nonstructural components. Fix work is an unquestionable requirement before the re-inhabitance of the structure occurs. The hazard to life in these structures is low.

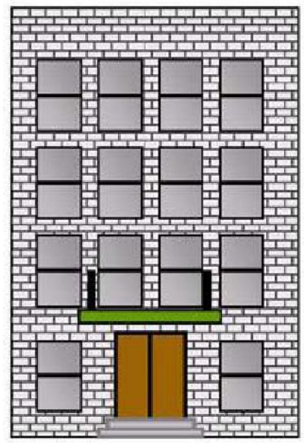
- **COLLAPSE PREVENTION (5-E)**

Structures meeting this model may make a major hazard to LS because of disappointment of nonstructural components. Be that as it may, because of this the structure itself doesn't fall however net death toll should be kept away from. The structures meeting this presentation level are finished monetary misfortunes.

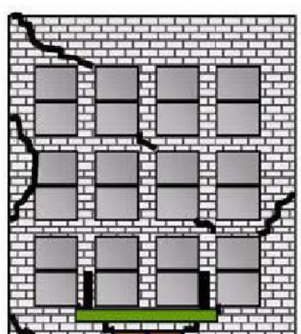
All the 4 PL of the building i.e. OP, IO level, LS level and CP level discussed earlier are represented in pictorial form in Fig. 3.4.



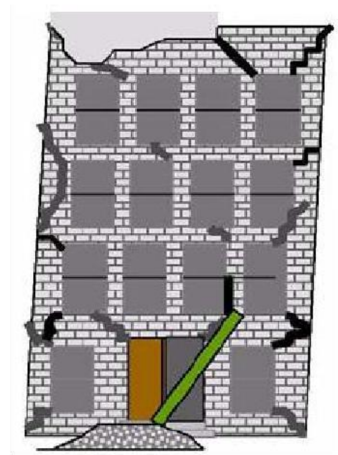
Operational



Immediate Occupancy



Life Safety



Collapse Prevention

Fig 3.4 Pictorial form of Performance Levels

CHAPTER 4
METHODOLOGY

To work out the forces evoked seismically inside the structures, there are a wide range of systems of investigation which offer various degrees of precision relying on a few variables. The technique for analysis might be ordered on the possibility of 3 factors: the sort of the remotely applied loads, the conduct of materials or state structure, in general, and furthermore the sort of model picked.

4.1 METHOD OF ANALYSIS

There are four methods of analysis, namely:

- Linear Static Analysis
- Linear Dynamic Analysis
- Non-linear Static Analysis
- Non-linear Dynamic Analysis

Linear Static Analysis (for example Equivalent Static Analysis) will be utilized for standard structures with obliged stature. Linear Dynamic Analysis might be acted in 2 particular habits either by mode superposition methodology or RS procedure. This assessment may convey the effect of the higher techniques for vibration in the structure and moreover the genuine spread of powers inside the adaptable range in an incredibly improved way.

The amazing differentiation between the linear static and dynamic assessment is that the level of force and their allotment on the tallness of the structure. Non-straight static methodology is an improvement over the linear static or dynamic technique with the inclination that it allows the inelastic conduct of the structure. The strategies despite everything accept monotonically increasing sidelong loads over the stature of structure.

A non-linear unique investigation or inelastic time history examination is the main philosophy to clarify the genuine conduct of the structure all through a seismic ground movements. The methodology relies upon the direct numerical blend of the movement differential conditions by considering the elasto-plastic misshapening of the structure parts. This way of thinking gets the impact of increase because of reverberation, the assortment of removals at various degrees of a structure.

4.1.1 EQUIVALENT STATIC METHOD

It is perhaps the easiest strategy for investigation which may require less computational endeavors because of the reliance of powers on fundamental time of the structure. In this strategy, the plan base shear for the structure is figured all in all and afterward circulated to various stories at their comparing focus of masses. At last, the plan seismic forces will be dispersed at different stories.

The calculation of forces at various story levels in this method is described below:

- The design base shear V_b can be computed by the formula:

$$\boxed{V_b = A_h.W}$$

Where,

A_h = design horizontal acceleration coefficient (see Section 4.1.1.1)

W = seismic weight of the building.

- Fundamental natural period (Ta)

$$\boxed{T_a = 0.075h^{0.75}}$$

- The configuration base shear determined above will be circulated along the tallness of the structure as given underneath:

$$\boxed{Q_i = \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \cdot V_b}$$

Where,

Q_i = design lateral force at i^{th} floor,

W_i = seismic weight of floor i,

h_i = height of the floor I when measured from base,

n = number of stories in the building.

4.1.1.1 DESIGN ACCELERATION SPECTRUM

It alludes to an ordinary smoothened diagram of most extreme acceleration as a part of characteristic recurrence or normal timeframe of faltering for a predetermined harm state for the normal quake excitation at the base of a unity level of freedom framework.

For computing design seismic force, our country is divided into four seismic zones which are:

- Zone II
- Zone III
- Zone IV
- Zone V

The Indian map showing the different seismic zones by different color codes is given in Fig. 4.1.

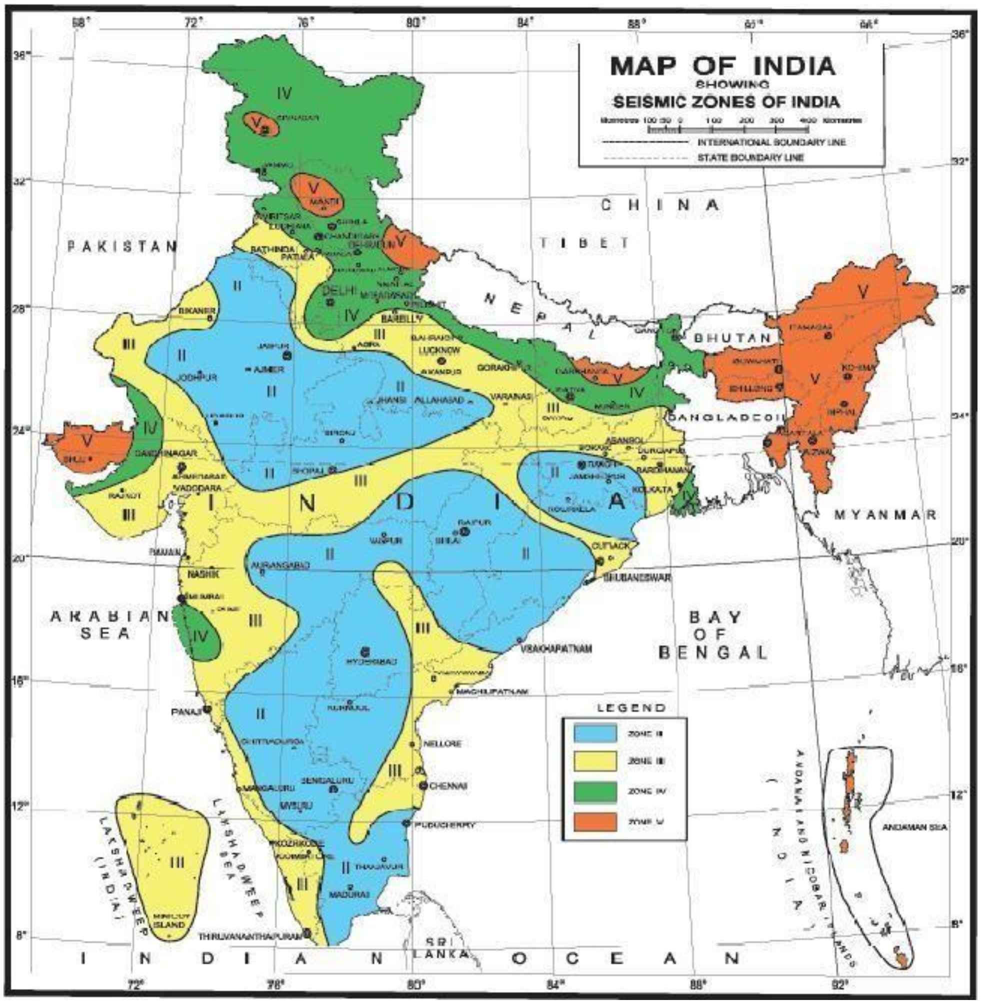


Fig. 4.1 Seismic zones of India [9]

The design horizontal seismic coefficient (A_h) is given by:

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

Where,

Z = Seismic zone factor (see Table 4.1)

I = Importance Factor for the corresponding structures (see Table 4.2)

R = Response Reduction Factor (see Table 4.3)

Table 4.1 Zone Factor [9]

Zones	II	III	IV	V
Factor	0.10	0.16	0.24	0.36

Table 4.2 Importance Factor [9]

S. No	**Structure**	**I**
1	Important services and community buildings or structures, signature buildings, monument buildings, lifeline and emergency buildings	1.5
2	Residential and commercial buildings with occupancy more than 200 persons	1.2
3	All other buildings	1

The Response Reduction Factors are given for various structural systems like Moment Resisting Frame system, Braced Frame system, Structural Wall system, Flat Slabs etc. in IS 1893:2016 but only the values for Moment resisting system are needed for this study and they are shown in Table 4.3.

Table 4.3 Response Reduction Factors

S.no	**Moment Frame System**	**R**
1.	RC Building with OMRF	3.0
2.	RC Building with SMRF	5.0
3.	Steel Building with OMRF	3.0
4.	Steel Building with SMRF	5.0

$\frac{S_a}{g}$ = design acceleration coefficient for different soil types, featured with peak ground acceleration, analogous to natural time period of the structure (with 5 percent damping) is given for both the methods separately i.e. ESM and RSM.

1. **For Equivalent Static Method**

$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 2.5 & 0 < T < 0.40\text{ s} \\ \dfrac{1}{T} & 0.40\text{ s} < T < 4.00\text{ s} \\ 0.25 & T > 4.00\text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 2.5 & 0 < T < 0.55\text{ s} \\ \dfrac{1.36}{T} & 0.55\text{ s} < T < 4.00\text{ s} \\ 0.34 & T > 4.00\text{ s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 2.5 & 0 < T < 0.67\text{ s} \\ \dfrac{1.67}{T} & 0.67\text{ s} < T < 4.00\text{ s} \\ 0.42 & T > 4.00\text{ s} \end{cases} \end{cases}$$

2. **For Response Spectrum Method**

$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.40\text{ s} \\ \dfrac{1}{T} & 0.40\text{ s} < T < 4.00\text{ s} \\ 0.25 & T > 4.00\text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.55\text{ s} \\ \dfrac{1.36}{T} & 0.55\text{ s} < T < 4.00\text{ s} \\ 0.34 & T > 4.00\text{ s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.67\text{ s} \\ \dfrac{1.67}{T} & 0.67\text{ s} < T < 4.00\text{ s} \\ 0.42 & T > 4.00\text{ s} \end{cases} \end{cases}$$

The different soil types can be categorized as given:

a) Soil Type I – Rock or hard soils;

b) Soil Type II – Medium or stiff soils;

c) Soil Type III – Soft soils.

4.1.2 RESPONSE SPECTRUM ANALYSIS

It is the portrayal of greatest reactions of a range of Single Degree of Freedom (SDOF) arrangement of various regular periods yet having steady damping during the activity of a Earthquake ground excitation at their bases. The reaction considered here, for the most part incorporates Maximum Absolute Acceleration, Maximum relative speed or Maximum relative distance.

This technique is generally pertinent to those structures whose modes note commendably influence the response of the structure. In this method, the response of Multi-Degree of Freedom (MDOF) system is appeared as the emplacement of the measured responses, each particular response being settled from the spectral examination of single-level of-freedom (SDOF) structure, which is finally merged to work out the whole response. Measured examination achieves the response history of the structure to a foreordained ground development; yet, the strategy is consistently used identified with a response run.

4.1.3 PUSHOVER ANALYSIS

In this, a profile demonstrating static level force on a structure, normally corresponding to the plan force profiles according to the codes, is applied to the structure. The force profile is then steadily expanded in slow advances and the structure is investigated at each progression in light of the fact that the loads are amplified, certain areas of the structure will experience yielding; at whatever point such yielding happens, the properties will change in order to show the impact of yielding. The investigation is done work the structure breakdown.

4.1.3.1 DESCRIPTION

The non-straight static pushover examination was initially evolved by two organizations to be specific, FEMA and ATC, under their rules.

As Per FEMA 273

The fundamental target of FEMA-273 code is to give worthy rules for the restoration of structures after the seismic activity. This will help plan experts to outfit the structure's plan and investigation, a reference archive for building managing authorities, and an establishment for the since quite a while ago run advancement and execution of construction law rules and principles.

As Per ATC 40

This record covers the Seismic investigation and Retrofitting of Concrete Buildings supported by California Safety Commission, United States of America; however the methodology directed during this document are for solid structures, they're appropriate to other structures also.

The following steps are recommended for the complete analysis and retrofit:

1. Starting the Project: Set the goal and possible scope of the project.

2. Selection of Experts: Choose experts with expertise in investigation, planning and retrofitting of structures in seismically dangerous areas, PBSE and non-linear strategies.

3. Selecting Performance Objective: Select a performance objective from the accessible decisions for a particular level of seismic danger.

4. Reviewing the Building: Site visits and assessing of drawings will be finished.

5. Alternative Options: Check to discover if the non-linear framework is good for the structure or not.

6. Review and Approval Procedure: Consult the structure authorities, basic specialists and review elective quality control estimates acceptable to seismic investigation and retrofit.

7. Careful Investigations: Perform a nonlinear static examination (if adequate).

8. Seismic Capacity: Verify the non-linear capacity graph with the PC and afterward changing over it to CS curve.

9. Seismic Hazard: The site-specific RS curve will be converted into Spectral Coordinates system.

10. Verification of Performance: Obtain PP as the combination of the diminished seismic interest and the capacity curve in spectral ordinates structure.

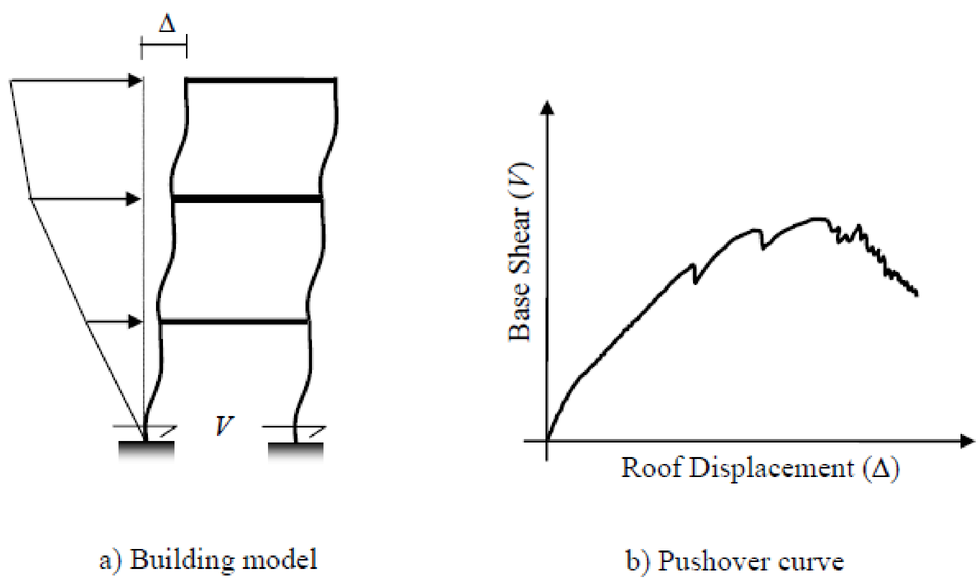


Fig. 4.2 Pushover Curve

4.1.3.2 PLASTIC HINGES

It is defined as a point of inelastic actions of the structural member. In this state, the members start losing their strength to come back to original position. Plastic hinges are normally appointed to watch the successive loss of solidarity in various PL of the structure. There are two distinct methods of giving out the pivot properties: Distributed pliancy model and Point Plasticity model. In this assessment, we have used point plasticity model in which the zone of yielding is believed to be collected at a specific point in the part of the structure.

Flexural hinges are basically defined by the moment-curvature curves which are obtained for each element based on reinforcement detailing and cross-sectional area of that element. For Beams, flexural hinges are modeled with M3 hinges. For columns, there is interaction of axial forces and bi-axial bending moment, thus P-M2-M3 hinges will be assigned to them.

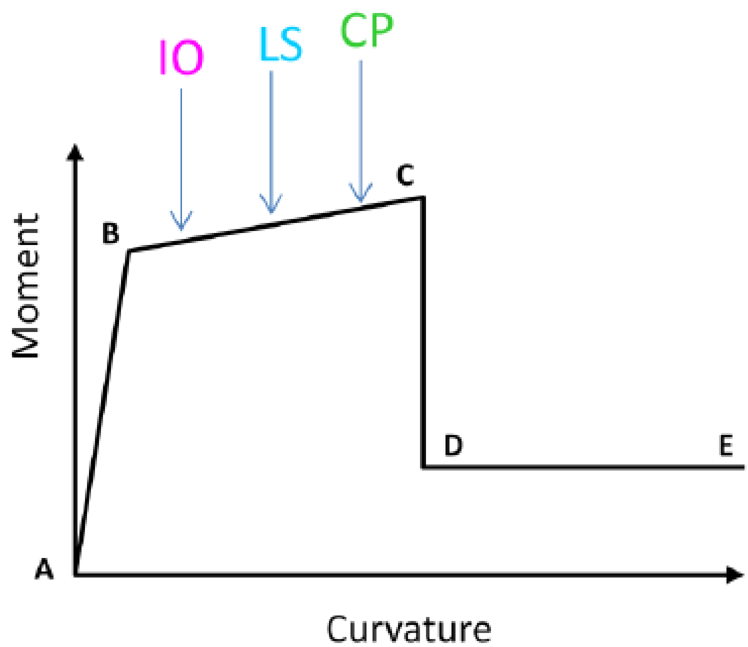


Fig. 4.3 A typical Moment-Curvature relationship

Fig. 4.3 shows a Moment-Curvature relationship where it may be plainly observed that till point B, the connection is straight and after that point, the curvature has hugely expanded with just minor increment in second obstruction. The Yielding from B to C is persistent however we have particularly separated this area into three Levels specifically IO, LS, and CP. Point C shows a definitive limit of the PC while point D delineates the residual strength level in the part.

In this study, the Moment-Curvature relationship is automatically developed by ETABS using the cross-sectional dimensions and reinforcement properties.

4.1.4 CAPACITY SPECTRUM METHOD

One of the manners which is used to get the performance purpose of the structure is the CSM, also implied as ADRS approach. To apply this procedure, both the limit twist and the demand curve are plotted in the spectral ordinates instead of the customary ordinates.

The value of S_d for every points of S_a-T curve is to be computed to convert conventional Spectrum into ADRS.

This can be calculated by using the following equation:

$$\boxed{S_d = \frac{1}{4\pi^2} S_a T^2}$$

At period Ti, the values of Sa and Sd are as follows:

$$Sa_i = \frac{2\pi}{T_i} S_v$$

$$Sd = \frac{T_i}{2\pi} S_v$$

The point by point conversion of traditional coordinates into spectral coordinates leads to the formation of PC. Every point on the curve will be converted into the corresponding points Sa_i, Sd_i on the CS by making use of values of participation factors and modal mass coefficient with the help of following equations [8]:

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

$$Sd_i = \frac{\Delta_{roof}}{PF_1 \times \emptyset_{1,}}$$

Where,

α_1= Modal Mass coefficient,

PFi = Participation factors,

$\emptyset_{1,}$= Roof Level amplitude for the first mode of the structure.

The values of modal mass coefficient and participation factors for different modes will be calculated by the given formulae [8]:

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (W_i \emptyset_{i1})/g}{\sum_{i=1}^{N} (W_i \emptyset_{i1}^2)/g}\right]$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (W_i \emptyset_{i1})/g\right]^2}{\left[\sum_{i=1}^{N} W_i/g\right]\left[\sum_{i=1}^{N} (W_i \emptyset_{i1}^2)/g\right]}$$

Where,

W_i = Weight at level i.

As the structure evacuating builds, the timeframe of the structure additionally increments. This is reliably found in the limit run. The CS procedure lessens the interest to look out an intersection point with the capacity range, at the spot the removal is in a state of harmony with the deduced damping.

Fig. 4.4 shows the change of PC to capacity range curve. PC shows the assortment of Base Shear with the Displacement with the past on ordinate and later on abscissa. The CS Curve is the outline of Sa v/s Sd with the past on ordinate axis and the later on absicca.

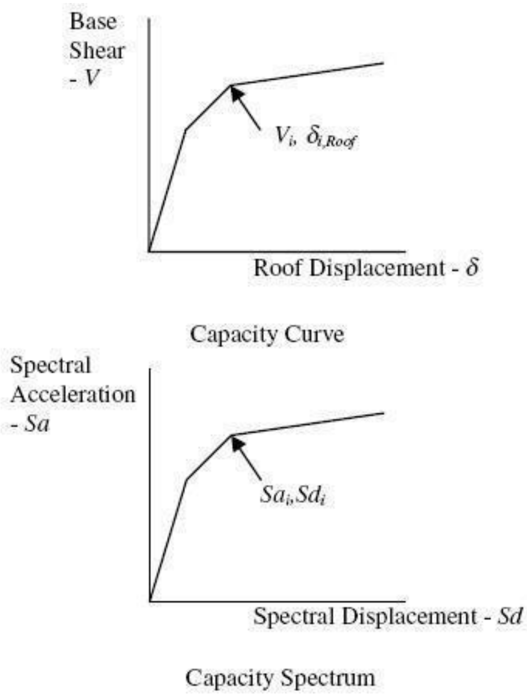


Fig. 4.4 Capacity Spectrum Curve

4.1.5 PERFORMANCE POINT

It can be determined by making use of Acceleration Demand Response Spectra (ADRS) format curves obtained for the given building. It is usually a point where capacity of the structure equals its demand. Therefore, it is named as a proportion of economy of our framework. For this situation, the PP is dictated by the software itself utilizing methodology C (referenced in ATC 40) which is a graphical method.

4.1.5.1 DETERMINATION OF PERFORMANCE POINT

There are basically three procedures of determining a PP of the structure, according to ATC-40 as given below:

Procedure A: This is the most simplest and transparent methodology used for programming in which a set of equations are used as described in ATC-40.

Procedure B: This technique expects that the yield point correspondingly as the post yield tendency of the bilinear depiction stays predictable. It is an iterative procedure to discover the PP. This system is generally accurate; at any rate every now and then this suspicion may not be shown exact.

Procedure C: This is convenient for both software and hand calculations as well. It is graphical in nature and ETABS make use of this procedure for the computation of PP.

4.1.6 BUILDING MODEL

An 8-storied Residential Building of plan 20x12m located in seismic zone IV has been analyzed in this study and various parameters including the sectional and material properties of various structural members including beam, column and slab has been discussed in the upcoming sections.

The extruded view of the structure has been shown in Fig. 4.5.

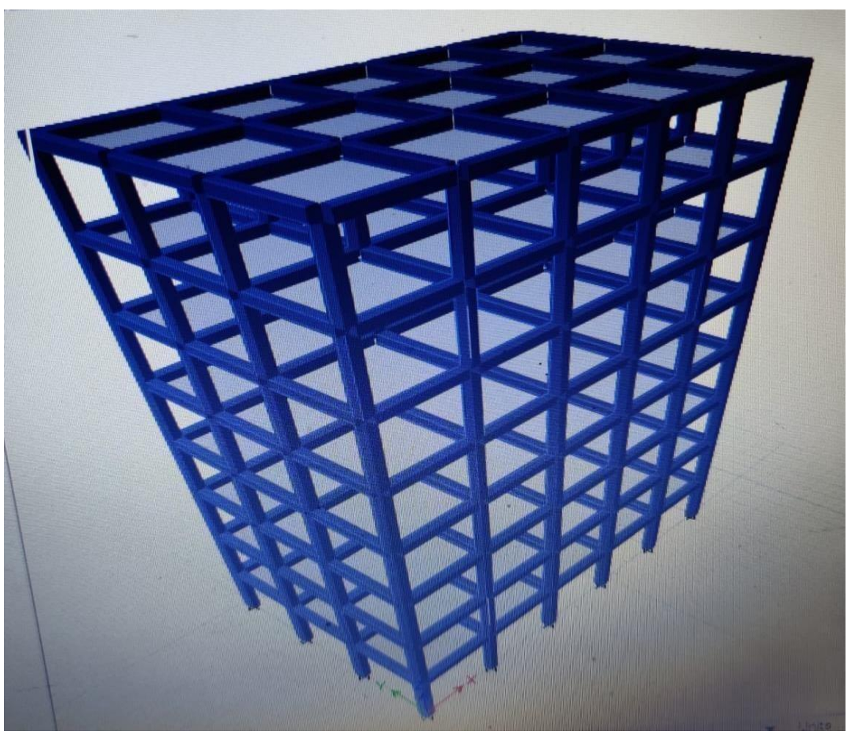


Fig. 4.5 Extruded View of building

4.1.6.1 SECTION PROPERTIES

Table 4.4 shows the sectional properties of various members used i.e. Beam, Column and Slab.

Table 4.4 Sectional Properties

S.no	Description	Dimensions
1.	Beam	300x450mm
2.	Column	400x400mm
3.	Slab	Thickness = 150mm

4.1.6.2 MATERIAL PROPERTIES

(a) Concrete:

M 30 (i.e. fck = 30 MPa)

Mass Density = 2549 kg/m^3

Modulus of Elasticity (E) = 27386.13 MPa

Poisson's Ratio (u) = 0.2

Coefficient of Thermal Expansion = $10x10^{-6}/^oC$

(b) Rebar:

Fe 500 (i.e fy = 500MPa)

Mass Density = 7850 kg/m3

Modulus of Elasticity (E) = $2x10^5$MPa

$\alpha = 7.2x10^{-6}/^oC$

4.1.6.3 ASSIGNING PROPERTIES

The respective properties of all the members i.e. column, beam and slab have to be assigned to the members so that all the loads that they are supposed to carry can be applied after that.

Assigning the square section of 400x400mm to all the columns present in the building, a shell element of thickness 150mm has be assigned to a slab and the beam will be assigned with a rectangular section of width 300mm and depth 450mm.

At last, the fixed support (i.e. restraint in all the 3 rotational and transitions as well) will be assigned to all the base supports.

4.1.6.4 LOADS

ETABS Software considers the dead load of the structure automatically, unlike the Stadd Pro, so no need to assign dead load coming solely by the building components. The loads that we have to assign additionally are super imposed load (also known as live load) of 2 KN/m^2 [10] on floors, Seismic loads in abscissa and ordibate i.e. EQ-X and EQ-Y for zone IV.

Finally, the model has been analyzed after inputting various functions like RS (Linear Dynamic method) and Pushover function (Non-Linear Static method) in abscissa and ordinate.

All the calculations done are mentioned in Chapter 5 while post-analysis results are discussed in Chapter 6.

4.1.7 RESPONSE SPECTRUM ANALYSIS BY ETABS

The following steps are involved in the RS analysis:

- Seismic loading is given as per IS 1893-2016 along abscissa and ordinate axis.
- Mass Source is defined as 100% building load and 25% live loads.
- Defining Response Spectrum Function as per IS 1893-2016 considering our structure in Zone IV (Z=0.24), I=1.2, R=5 for Soil Type II (i.e. medium to stiff soils) and having 5% damping.
- Run the analysis.

4.1.8 PUSHOVER ANALYSIS USING ETABS

The following steps are included to carry out pushover analysis in ETABS:

- Define and assign the structural members as per IS 456 in ETABS v 18.
- Convert the recently allotted straight Static dead load Case into Nonlinear Static with the goal that the product can utilize this case as the beginning stage of the Pushover examination.
- Now define Pushover Load Cases in X and Y-direction which will continue from the end of Nonlinear dead load case by considering previously added Mass Source and Acceleration load type in UX and UY direction.
- Assigning M3 hinges to beams with a relative distance of 0.05 and 0.95 and P-M2-M3 hinges to all the columns with same distance.
- Now run all Non-linear load cases to observe Structure’s response to see deformed shape for Push in both directions.
- PC can be observed to see how base shear drops as a number of hinges are formed.

CHAPTER 5

CALCULATIONS

In this chapter, the various manual calculations used in this study are mentioned with all the required formulae.

5.1 EQUIVALENT STATIC METHOD

In this method, lateral forces on all the stories are to be calculated. It thus requires calculation of weight of roof as well as the floors first, and then by applying relevant formulae, various values will be determined.

Weight of Roof = [(0.25x1.5) x 240] **+** [(25 x.400x.400x0.5 x3.5) x 24] + [(25x.300x.450x4) x 38] + [25x.150x240]

= 1617 KN

Weight of Floors = [(0.25 x 2) x 240] + [(25x.400x.400x3.5) x 24] + [(25x.3x.450x4) x 38] + [25x.150x240]

= 1869 KN

Table 5.1 shows the storey-wise calculation of lateral forces.

Natural period (Ta) = $0.075h^{0.75}$

$$= 0.075x28^{0.75} = 0.913 \text{ s}$$

Z = 0.24 (Table 3 of IS 1893:2016)

I = 1.2 (Table 8 of IS 1893:2016)

R = 5 (Table 9 of IS 1893:2016)

Sa/g = design acceleration coefficient = 1.36/T = 1.489 (Clause 6.4.2 of IS 1893:2016)

Table 5.1 Equivalent Static Method

Floor	**hi (m)**	**Wi (KN)**	**Wihi2 (KNm^2)**	**Q/Vb**	**Q**
Roof	28	1617	1267728	0.2834	179.1371
7	24.5	1869	1121867.25	0.2508	158.5307
6	21	1869	824229	0.1843	116.496
5	17.5	1869	572381.25	0.1279	80.84559
4	14	1869	366324	0.0819	51.76899
3	10.5	1869	206057.25	0.0460	29.0766
2	7	1869	91581	0.0205	12.95805
1	3.5	1869	22895.25	0.0051	3.22371

Design horizontal seismic coefficient (A_h) = (Z/2)x(I/R)x(S_a/g)

= (0.24/2)x(1.2/5)x(1.489)

= 0.043

Design Seismic Base Shear (Vb) = Ah x W = 0.043 x 14700 = 632.1 KN

Table 5.2 shows the calculated lateral forces and base shear at various storey levels as well.

Table 5.2 Lateral forces and Base Shear

Storey	Lateral Force (KN)	Base Shear (KN)
Roof	179.1371	179.1371
7	158.5307	337.6678
6	116.4961	454.1639
5	80.8456	535.0095
4	51.7689	586.7784
3	29.0766	615.8551
2	12.9580	628.8131
1	3.2237	632.0367

5.2 CAPACITY SPECTRUM MEHTOD

In this section, change of Base shear v/s Displacement curve to Sa v/s Sd curve by applying appropriate formulae is illustrated. The modal mass coefficient and modal mass participation factors for first three modes need to be calculated and afterwards, various points on PC are converted into spectral coordinates.

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (W_i \emptyset_{i1})/g}{\sum_{i=1}^{N} (W_i \emptyset_{i1}^2)/g}\right]$$

$$\alpha_1 = \frac{[\sum_{i=1}^{N} (W_i \emptyset_{i1})/g]^2}{[\sum_{i=1}^{N} W_i/g][\sum_{i=1}^{N} (W_i \emptyset_{i1}^2)/g]}$$

PFi = Participation factor for mode i

α = Modal Mass coefficient

ϕ = amplitude (mm)

Wi/g = Mass of various storey levels (Kg)

Table 5.3 (a), 5.3 (b), 5.3 (c) shows the calculations for participation factors and modal mass coefficient for mode 1, 2, and 3 respectively.

Table 5.3 (a) Mode 1

Story	Wi/g(kg)	PHY I1	(PHY I1)^2	W*PHY/g	W*PHY2/g	(Wi/g*PHY)^2
Story8	164831.804	0.037	0.001	6098.777	225.655	37195077.822
Story7	190519.877	0.036	0.001	6858.716	246.914	47041979.298
Story6	190519.877	0.033	0.001	6287.156	207.476	39528329.826
Story5	190519.877	0.029	0.001	5525.076	160.227	30526469.590
Story4	190519.877	0.023	0.001	4381.957	100.785	19201548.648
Story3	190519.877	0.018	0.000	3429.358	61.728	11760494.824
Story2	190519.877	0.011	0.000	2095.719	23.053	4392036.647
Story1	190519.877	0.005	0.000	952.599	4.763	907445.588
Sum	1498470.943			35629.358	1030.601	

Table 5.3 (b) Mode 2

Story	Wi/g(Kg)	PHY I1	(PHY I1)^2	W*PHY/g	W*PHY2/g	(Wi/g*PHY)^2
Story8	164831.804	0.049	0.002	8076.758	395.761	65234026.187
Story7	190519.877	0.047	0.002	8954.434	420.858	80181892.182
Story6	190519.877	0.044	0.002	8382.875	368.846	70272586.358
Story5	190519.877	0.039	0.002	7430.275	289.781	55208989.592
Story4	190519.877	0.032	0.001	6096.636	195.092	37168971.297
Story3	190519.877	0.024	0.001	4572.477	109.739	20907546.354
Story2	190519.877	0.015	0.000	2857.798	42.867	8167010.295
Story1	190519.877	0.006	0.000	1143.119	6.859	1306721.647
Sum	1498470.943			47514.373	1829.804	

Table 5.3 (c) Mode 3

Story	Wi/g(Kg)	PHY I1	(PHY I1)^2	W*PHY/g	W*PHY2/g	(Wi/g*PHY)^2
Story8	164831.804	0.036	0.001	5933.945	213.622	35211702.598
Story7	190519.877	0.024	0.001	4572.477	109.739	20907546.354
Story6	190519.877	0.005	0.000	952.599	4.763	907445.588
Story5	190519.877	0.016	0.000	3048.318	48.773	9292242.824
Story4	190519.877	0.031	0.001	5906.116	183.090	34882208.414
Story3	190519.877	0.036	0.001	6858.716	246.914	47041979.298
Story2	190519.877	0.030	0.001	5715.596	171.468	32668041.179
Story1	190519.877	0.014	0.000	2667.278	37.342	7114373.412
Sum	1498470.943			35655.046	1015.711	

By using the above equations, the values of participation factors and modal mass coefficients for these modes will be calculated which is illustrated in Table 5.4.

Table 5.4 Participation factors and Modal Mass Coefficient

Modes	PF_i	α
Mode 1	34.40	0.823
Mode 2	25.84	0.8245
Mode 3	34.94	0.8361

By using the following equations, the points on PC will be converted into Sa and Sd which are illustrated in Table 5.5.

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

$$Sd_i = \frac{\Delta_{roof}}{PF_1 \times \emptyset_{1,roof}}$$

Table 5.5 Capacity Spectrum Method

		Mode 1		Mode 2		Mode 3	
Monitored Displ(mm)	Base Force (KN)	Sa	Sd (mm)	Sa	Sd (mm)	Sa	Sd (mm)
26.079	1041.4175	0.064	20.488	0.064	20.597	0.063	20.733
78.363	2870.2615	0.178	61.562	0.177	61.890	0.175	62.299
119.973	3805.8808	0.235	94.251	0.235	94.752	0.232	95.380
164.961	4246.7138	0.263	129.593	0.262	130.283	0.259	131.146
165.02	4247.002	0.263	129.640	0.262	130.329	0.259	131.193
165.321	4249.6888	0.263	129.876	0.262	130.567	0.259	131.432
165.336	4249.8923	0.263	129.888	0.262	130.579	0.259	131.444
165.494	4251.2137	0.263	130.012	0.263	130.704	0.259	131.569
165.511	4251.3541	0.263	130.025	0.263	130.717	0.259	131.583
165.594	4252.1537	0.263	130.091	0.263	130.783	0.259	131.649
165.599	4252.1738	0.263	130.095	0.263	130.787	0.259	131.653
165.685	4253.766	0.263	130.162	0.263	130.855	0.259	131.721

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CHAPTER 6

RESULTS

In this section, results of RS Method and PA are given.

6.1 RESPONSE SPECTRUM ANALYSIS

This section has the various table showing different values like modal load participation ratios, time periods of different modes, their frequencies, modal participating mass ratios.

6.1.1 MODAL LOAD PARTICIPATION RATIOS

Table 6.1 shows modal load participations ratios.

Table 6.1 Modal Load Participation Ratios

Case	ItemType	Item	Static	Dynamic
			%	%
Modal	Acceleration	UX	99.98	97.82
Modal	Acceleration	UY	99.98	97.91

6.1.2 TIME PERIODS AND FREQUENCIES

Table 6.2 shows the time span and the frequencies of different modes.

Table 6.2 Time Period and Frequencies

Case	Mode	Period	Frequency
		sec	cyc/sec
Modal	1	1.154	0.867
Modal	2	1.108	0.903
Modal	3	1.006	0.994
Modal	4	0.377	2.651
Modal	5	0.364	2.751
Modal	6	0.33	3.026
Modal	7	0.217	4.605
Modal	8	0.211	4.732
Modal	9	0.193	5.184
Modal	10	0.15	6.654
Modal	11	0.147	6.799
Modal	12	0.134	7.453

6.1.3 MODAL MASS PARTICIPATION RATIOS

Table 6.3 shows the modal mass participation ratios of different modes and IS code says analysis should be done till the time, this ratio is greater than 90% and it can be seen that only 4 and 5 modes are required for UX and UY direction respectively.

Table. 6.3 Modal mass Participation ratios

Case	Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		sec						
Modal	1	1.154	0.8205	0	0	0.8205	0	0
Modal	2	1.108	0	0.8254	0	0.8205	0.8254	0
Modal	3	1.006	0	0	0	0.8205	0.8254	0
Modal	4	0.377	0.1021	0	0	0.9226	0.8254	0
Modal	5	0.364	0	0.0991	0	0.9226	0.9245	0
Modal	6	0.33	0	0	0	0.9226	0.9245	0
Modal	7	0.217	0.0364	0	0	0.959	0.9245	0
Modal	8	0.211	0	0.0359	0	0.959	0.9604	0
Modal	9	0.193	0	0	0	0.959	0.9604	0
Modal	10	0.15	0.0191	0	0	0.9782	0.9604	0
Modal	11	0.147	0	0.0187	0	0.9782	0.9791	0
Modal	12	0.134	0	0	0	0.9782	0.9791	0

6.2 PUSHOVER ANALYSIS

In this segment, different outcomes acquired in the wake of playing out the pushover investigation are examined for example pivots results, PP for X and Y bearings and so on.

6.2.1 PUSHOVER CURVE

Fig. 6.1 shows the pushover curve.

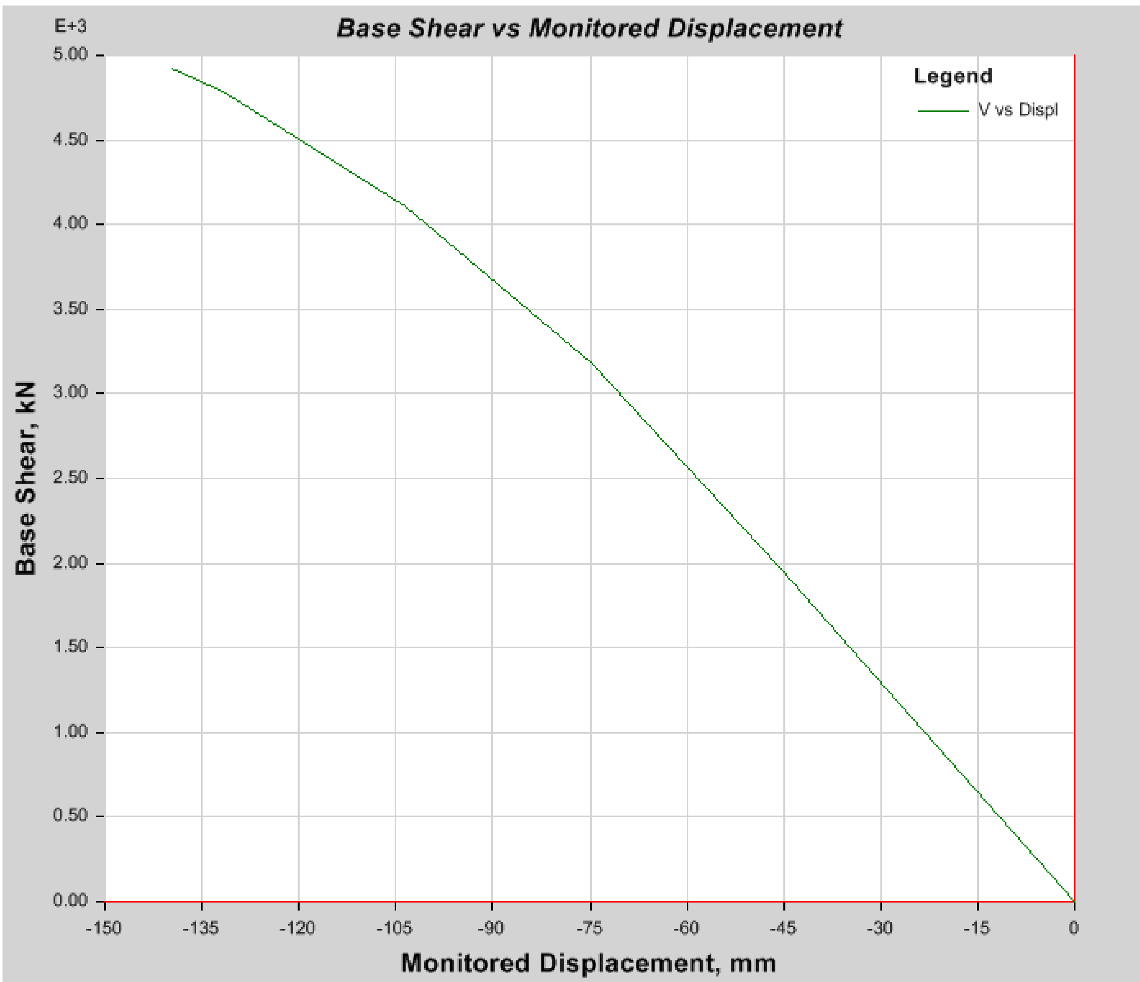

Fig. 6.1 Pushover Curve

6.2.2 HINGE RESULTS

Table 6.4 shows the hinges results as obtained by the software.

Table 6.4 Hinge Results

Step	Monitored Displ	Base Force	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0	0	1984	0	0	0	1984
1	-27.114	1173.5549	1984	0	0	0	1984
2	-45.676	1976.9828	1984	0	0	0	1984
3	-74.561	3181.4252	1984	0	0	0	1984
4	-103.509	4108.9804	1976	0	0	8	1984
5	-131.643	4782.1464	1976	0	0	8	1984
6	-139.093	4914.1355	1976	0	0	8	1984
7	-139.094	4914.0269	1976	0	0	8	1984
8	-139.179	4915.3597	1976	0	0	8	1984
9	-139.182	4915.3288	1976	0	0	8	1984
10	-139.246	4916.3241	1976	0	0	8	1984
11	-139.246	4916.2732	1976	0	0	8	1984
12	-139.276	4916.8226	1976	0	0	8	1984

6.2.3 PERFORMANCE POINT

In this section, the PP has been found for PGA of 0.043, 0.0214, 0.065, 0.08 as defined for MCE (Maximum Considered Earthquake), DBE (Design Basic earthquake), '*Seismic hazard assessment in India' (WCEE,2012)*[17] and *National Disaster Management Authority (NDMA,2011).*

The values of the PP in X direction are mentioned in Table 6.5 and the graph is illustrated for MCE and DBE in Fig. 6.2 and 6.3 respectively.

Table 6.5 Performance Point

S.No	PGA (g)	Performance Point (mm)	Damage State
1.	0.0214	6.941	Operational
2.	0.043	13.641	Operational
3.	0.065	20.579	Operational
4.	0.08	25.328	Immediate Occupancy
5.	0.1	31.661	Life Safety
6.	0.2	63.319	Collapse Prevention

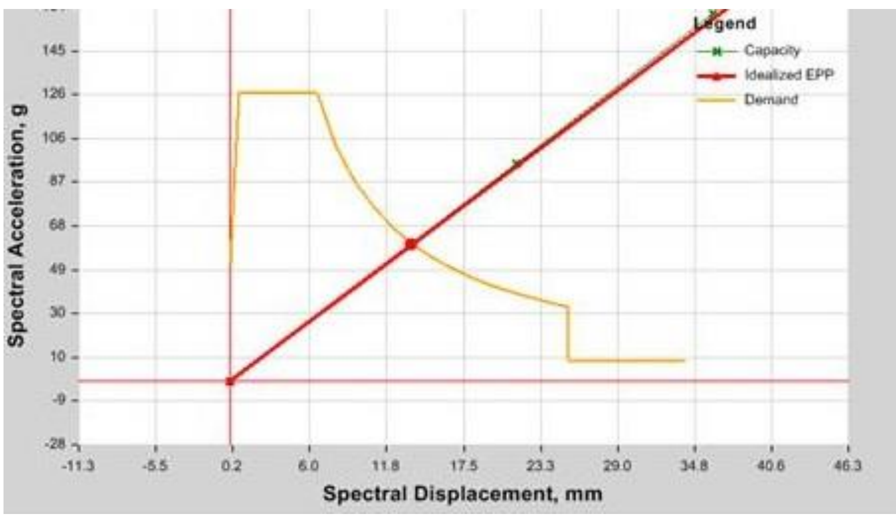

Fig. 6.2 MCE in X-direction

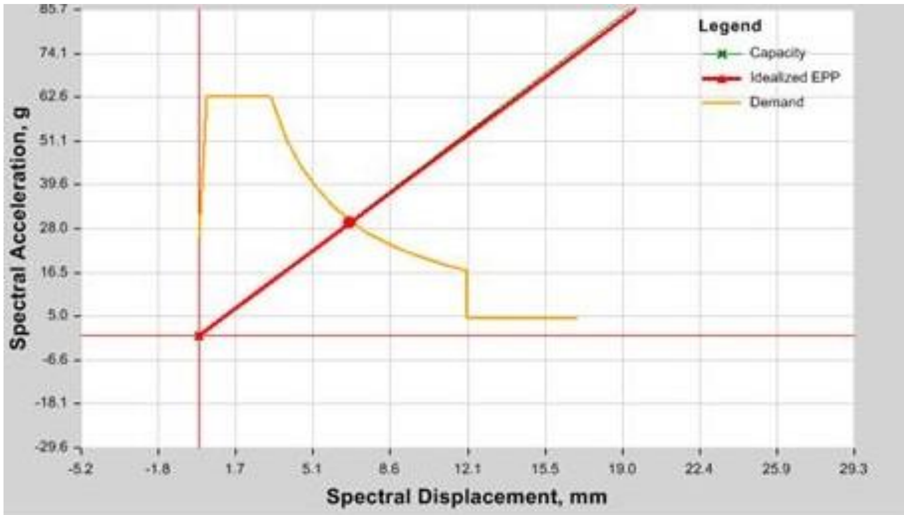

Fig. 6.3 DBE in X-direction

6.2.4 ROOF DISPLACEMENTS

Fig. 6.4 illustrates the top storey displacements for PUSH X and the values are given in tabulated form in Table 6.6.

Table 6.6 Roof Displacements

S.No	Direction	Roof Displacement (mm)
1.	X	139.899
2.	Y	226.261

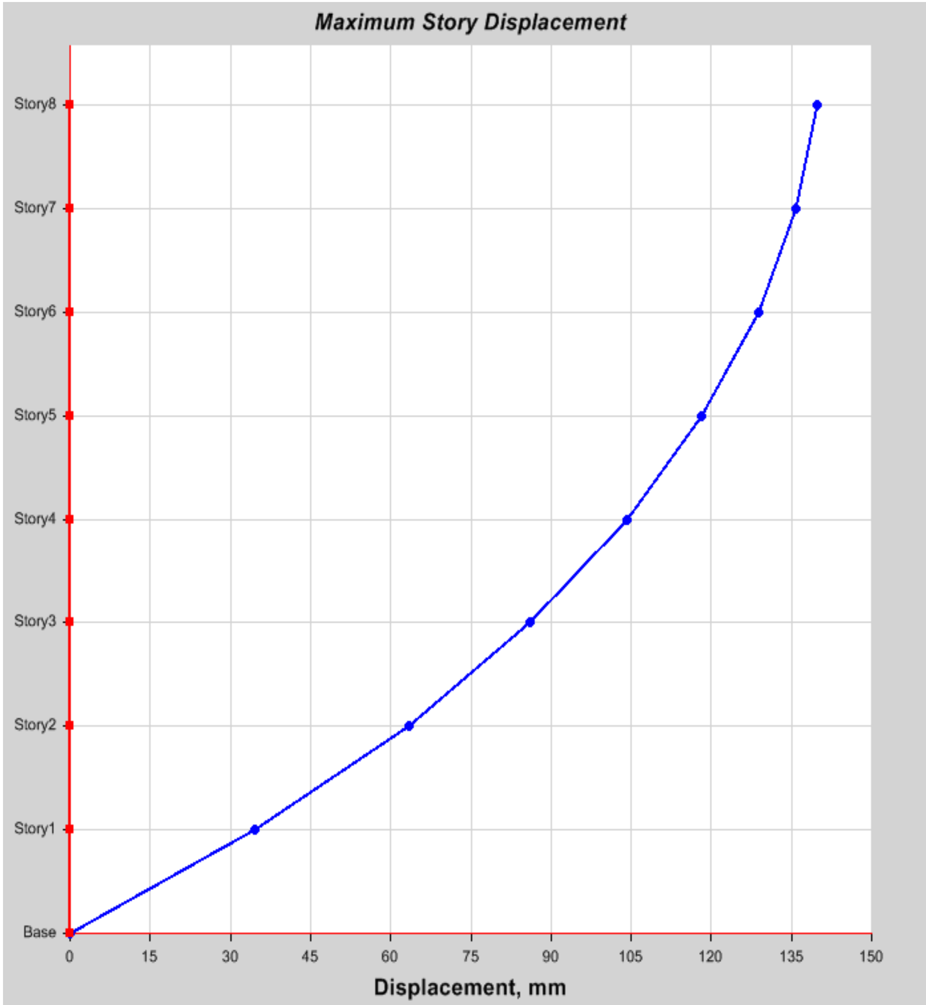

Fig.6.4 Displacement in X-direction

The roof displacement is 139.899 mm in X-direction which is about 0.49% of the stature of the building.

Fig. 6.5 shows the top story displacement due to PUSH Y in ordinate axis.

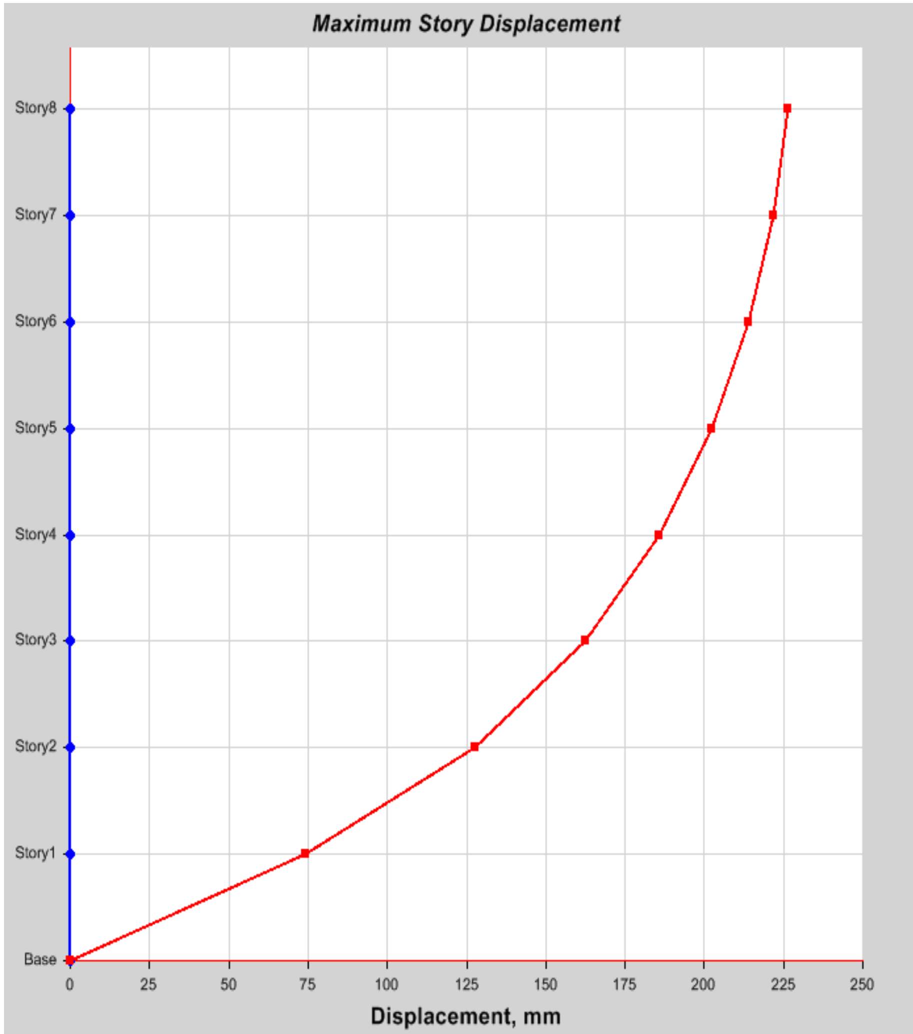

Fig. 6.5 Displacement in Y-direction

The top storey displacement in Y-direction is 226.261 mm which is around 0.81% of the height of the building (i.e. 28 m)

6.2.5 STOREY DRIFTS

The values of storey drift for X and Y direction are illustrated below in Fig. 6.6 and 6.7 respectively.

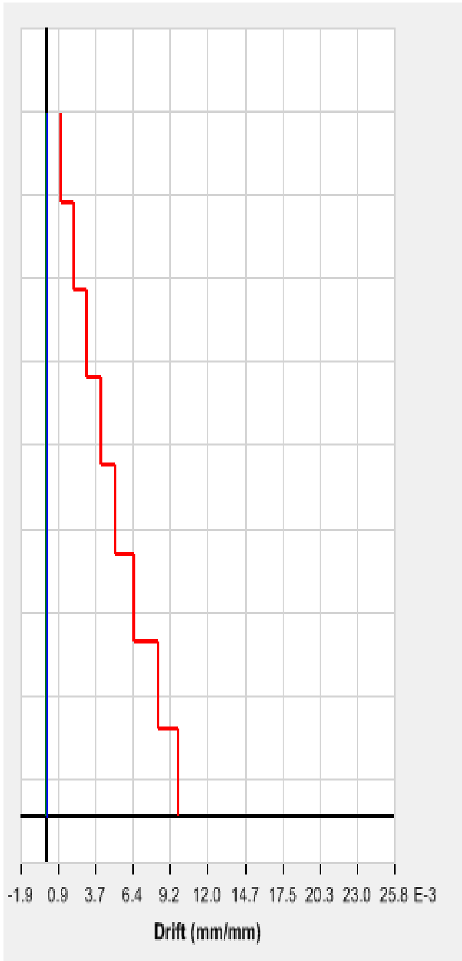

Fig. 6.6 Drift In X-direction

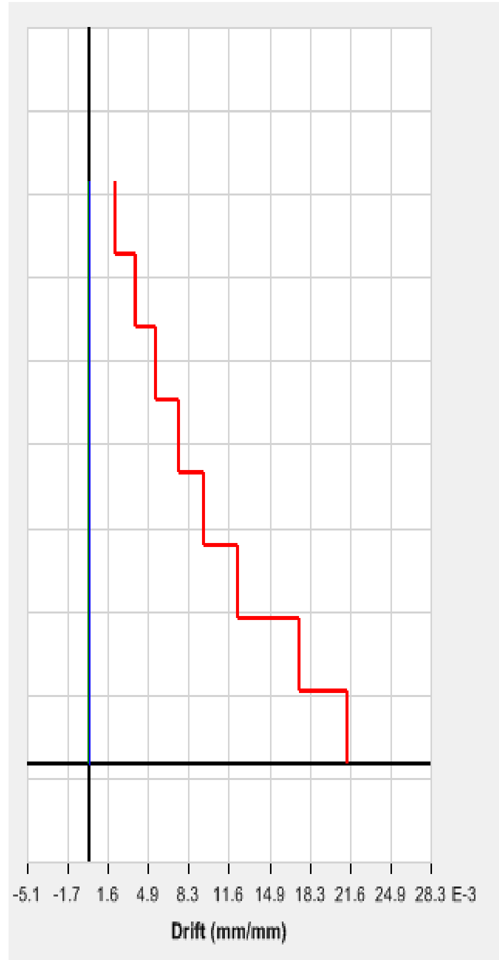

Fig. 6.7 Drift in Y-direction

6.2.6 HINGE RESPONSE IN X-DIRECTION

The hinge reaction (i.e. a relationship between Moment and Plastic Rotation or say Moment-curvature relationship) of beam present at storey 1 has been shown in Fig. 6.8 and 6.9 respectively for X and Y-direction.

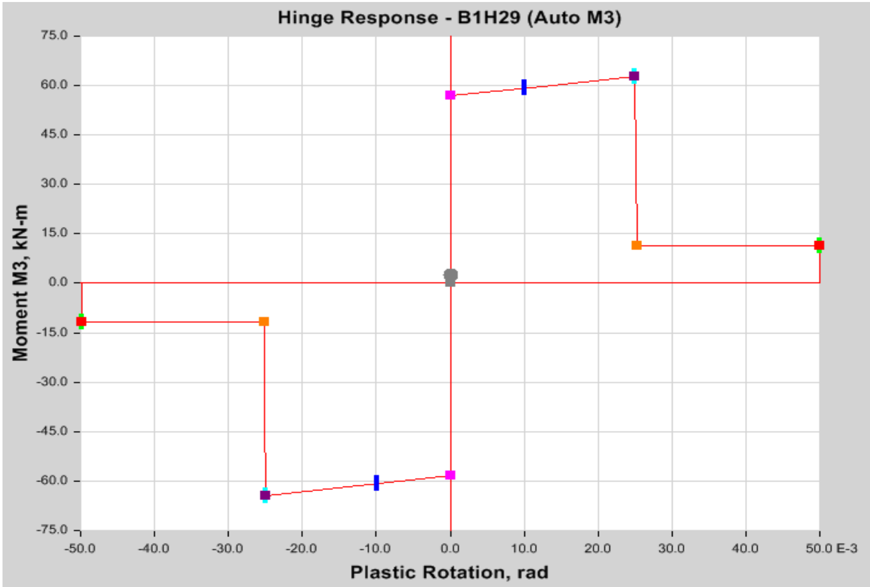

Fig. 6.8 Hinge Response in X-direction

6.2.7 HINGE RESPONSE IN Y DIRECTION

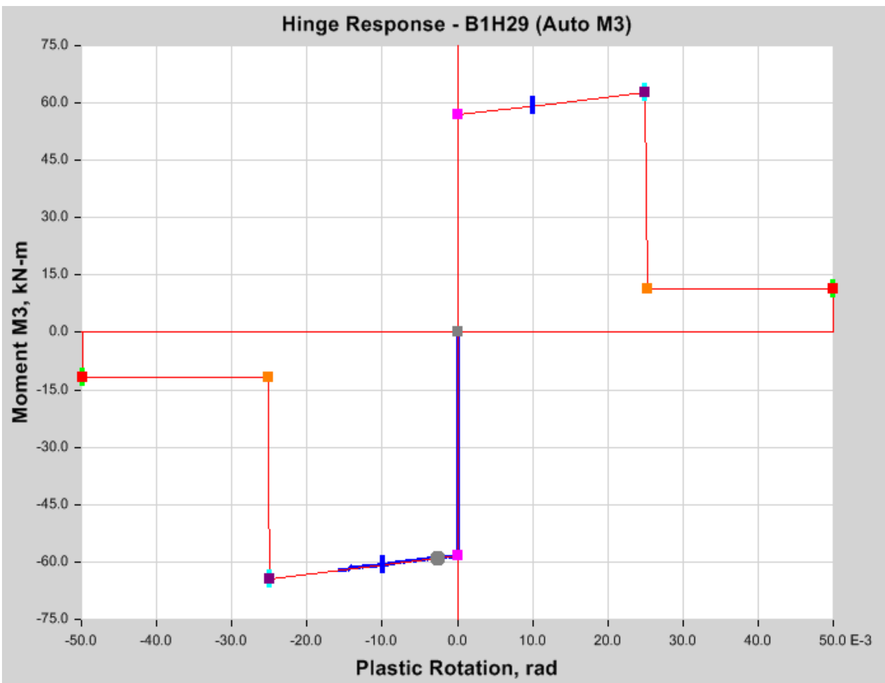

Fig. 6.9 Hinge Response in Y-direction

CHAPTER 7

SUMMARY AND CONCLUSION

7.1 SUMMARY

In the present study, a 8-storied Residential building is analyzed (located in Delhi, i.e. Zone IV) as per IS i.e. IS 456:2000 and IS 1893:2016 using a Software ETABS. The fundamental target of this work is to check by what level of performance, a structure react when planned according to IS codes. Literature review was carried after the modeling of the proposed residential structure about the concepts of PBSD which is most recent tool in current market scenario and is being used by western nations where an Owner can pick the sort of performance he/she needs from the structure. It might likewise help the Government in setting up the guidelines with the goal that it will be ordered to structure to follow a specific needed PL. In this software, the characterizing and displaying part was done which was trailed by RS method and PA and finally the Analysis was finished.

7.2 CONCLUSION

PA is a refined instrument to imagine the PL of a structure under a given quake.

It may be concluded that as we are increasing the demand of the structure which includes increasing the value of PGA on the structure, DS moves from OP towards CP. It may also be concluded that by increasing the value of PGA from 0.0214g to 0.043g for DBE and MCE respectively, the value of PP increased by about 96.52%.

It may also be concluded that roof displacement in Y-direction is about 61.73% more than in X-direction due to more stiffness in later case.

We may also conclude that PBSD gives a structure better seismic load conveying capacity, in this manner accomplishing the goal of Performance just as economy and there is still space for some further modifications.

7.3 FUTURE SCOPE

With the limited scope of the present work, many further studies can also be carried out in this field:

- In this study, a 8 storied residential building has been analyzed but can be further done for tall buildings.
- A comparative study can also be carried out by altering the structural member sizes, and comparing its performance.
- Structure equipped with shear walls can also be analyzed by this method which will increase the stiffness.

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