PERFORMANCE BASED SEISMIC DESIGN OF G+6 STOREY BUILDING

A PROJECT REPORT

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF THE DEGREE OF

MASTER OF TECHNOLOGY

IN

STRUCTRURAL ENGINEERING

SUBMITTED BY

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<u>CANDIDATE'S DECLARATION</u>

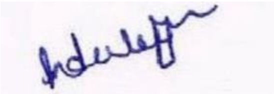
I hereby declare that the project entitled **"Performance Based Seismic Design Of G+6 Storey Building"** submitted by me to Delhi Technological University for partial fulfilment of the requirement for the award of the degree of Master of Technology in Structural Engineering is a record of bonafide project work carried out by me under the supervision of **Mr. Hrishikesh Dubey**. I further declare that the work reported in this project has not been submitted either in part or in full, for the award of any other degree or certificate in any other institute or university.

Place: New Delhi Prashant

Date: 12 Nov 2021 2K17/STE/013

<u>CERTIFICATE</u>

I hereby certify that the Project Dissertation titled **"Performance Based Seismic Design Of G+6 Storey Building"** which is submitted by **PRASHANT (2K17/STE/013)** of Civil Engineering Department, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of degree in Master of Technology, is a record of the project work done by the student under my supervision. To the best of my knowledge this work has not been submitted in part or in full for any Degree to this university or elsewhere.



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ABSTRACT

In recent years, emphasis has changed from strength to performance for design of seismic resistance structures. Strength and performance were considered as synonyms and recently it is realized that increasing strength may not enhance safety or necessarily reduce damage. So, Performance-based design is needed for change in the design process because although buildings designed using codes had performed well during the earthquakes from a life safety point of view but the level of damage of structures and economic losses due to loss of use and cost of repair were extremely high. A Performance based design is based upon accurate estimations of different response parameters. It is emphasized on limit state design, which is termed as Performance Based Engineering. Designing of a structure in a way to reduce damage during an earthquake makes the structure uneconomical, as the earthquake is a rare natural phenomenon which might or might not be occurring during the building's lifetime. The main reason for the occurrence of the earthquakes is movement of Tectonic Plates noticed as ground motion. These ground motion at any site depends upon magnitude, focal depth, epicentral distance, characteristics of the path of seismic waves and soil strata on which the structure is build.

Structures are designed using current seismic design codes which are mostly based on Force-Based Design approach. The initial aim of the current codes is the public safety. However, no clear information is provided regarding economic losses and business interruptions or downtime. Some information about damage states of structural components is provided, but very limited information is given for the damage states of non-structural members and content systems. Performance-Based Seismic Design (PBSD), which is a new concept in seismic design of structures, is a reliable approach capable of providing more detailed information on the performance levels of both structural and non-structural elements.

In this report performance-based design of G+6 Building is studied in an active seismic zone. The building is designed in software SAP2000 & its various parameters such as Displacement, Stresses, Drift due to lateral forces are estimated as per IS-1893-2016.

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CHAPTER 1 INTRODUCTION

1.1 GENERAL

Amidst all the natural jeopardy earthquake causes austere damages, as the earthquake forces are indiscriminate in nature and are often capricious so for reckoning of such forces engineering gizmos' needs improvement for scrutinizing the structures under the liveliness of these earthquake forces. Performance Based Seismic Design is a conception to engineering with proficiency and tools to design structures to have anticipated and decisive performance during earthquakes. Performance Based Seismic Design is conjoined with elastic design methodology done on the presumable performance of the structures underneath utterly different ground motions. Performance Based Seismic Design is a progression that permits design of new structures and upgradation of existing buildings with understanding of risk of life, occupancy and economic losses that occur during any future earthquakes. Earthquake loads are carefully modelled so as to access the real behaviour of structure and the damage that is likely to occur which should be regulated. Performance Based Seismic Design begins with the selection of criteria from design in form of different performance objectives. Each performance objective is a statement of the acceptable risk of specific levels of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. The Performance-Based engineering is not new airplanes, ships, automobiles, turbines and pumps are designed using this approach from many decades. In such cases one full scale prototype is modelled and subjected to immense testing and design is revised and manufacturing processes are incorporated to the lesson learned during testing. Once design, testing and redesign is completed, the product is manufactured on large scale.

The aim of structural engineering is to design structures to sustain various types of loads imposed by their service requirements and natural hazards. Currently, design of structures is guided through codes and standards. Structures designed with current seismic design codes and standards, should be able to satisfy specific performance level, defined as life safety performance level, for a specific intensity of ground motion (design earthquake with mean return period of 475 years). However, economic losses and occupancy interruptions are not provided (i.e., human lives are protected, but the damages are not limited which may not be economical to repair, the period for re-occupancy is not given). In addition, although life safety performance level is obtained for different structures, the concept of uniform risk is not satisfied (i.e., the response of various structures is different in terms of damages for the same earthquake hazard levels) [2].

Every building designed by this procedure is unique and experience obtained by this cannot be transferable to buildings of other sizes, types and performance objectives.

Performance objectives are the combination of performance levels and hazard levels, and performance levels can be determined by damage states of the structural and non-structural components and content systems [1]. Due to recent advances in seismic hazard assessment, PBSE methodologies, computer facilities and experimental facilities PBSE has become more useful to engineers for building structures in seismic zones. PBSE has become a standard, effective and intelligent method of design of earthquake resistant structures and to do that one should be aware of the uncertainties involved in both structural performance and seismic hazard estimations.

1.2 OBJECTIVES OF THE PRESENT STUDY

The Kutch Earthquake of January 26, 2001 in Gujarat, India, caused the destruction of a large number of buildings. This earthquake questioned about the buildings by-laws, professional practices, construction materials, building codes & education for civil engineers & architects. It led to revision of the seismic code and initiation of a National Program on Earthquake Engineering Education (NPEEE). The current Seismic Standards of India vouches for Seismically vulnerable construction in high seismic intensity areas of our 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.

The main objective of this dissertation is to study & analysis of the performance-based design of RC framed building's for ascertaining the seismic load carrying capacity of structures. Here, we design a Six-Storied RC frame Building and compute the Seismic Response Of the building in terms of Base Shear, Floor Drift, Spectral Acceleration, Spectral Displacements and Storey Displacements. Then compare these Displacements with the Target Displacements given in ATC 40 and FEMA documents.

1.3 SCOPE OF THE PRESENT STUDY

The scope of the present study mainly aims at the design (according to IS 456:2000) and evaluation of building using IS 1893-2016 and ATC 40 and analysing. In this analysis various procedures Such as Response Spectrum Analysis, Pushover Analysis and Time History Analysis are performed using SAP2000, a product of Computers and Structures.

In the analysis, Damage must be limited to Grade 2 (slight structural, moderate non-structural damage) to enable Immediate occupancy Performance level under DBE.

The above methodology is utilised to design a G+6 storey reinforced concrete building in zone IV as given in IS 1893-2016.

1.4 THESIS ORGANIZATION

Chapter 1- A brief introduction of the Performance based Seismic Design approach is given and objective of the study is summarized. Its historical background, need and advantages over the FBD are also discussed.

Chapter 2- Background studies of PBSD approach have been reviewed under literature review.

Chapter 3- Comprehensive description of Performance Based Seismic Design Methodology is discussed by considering design process, various performance levels and seismic hazard.

Chapter 4- Different types of analysis Procedures are considered such as response Spectrum Analysis, Pushover Analysis and a brief introduction about the Capacity Based Design for Performance Based Design has been Provided.

Chapter 5- The analysis procedures explained in chapter five are performed on a G+6 Storey Reinforced Concrete Building to compute the actual forces, displacements and compare them with target displacements provided in Seismic Design codes.

Chapter 6- In this chapter summary of the study with conclusions have been provided and some recommendations for the future studies are given.

Qiang Xue, Chia-Wei Wu et al (2008) gave a summarized seismic design draft for consideration in Taiwan Building code. In his draft he tried to incorporate the earlier concept described in the code with the new performance based seismic design approach. After going through the earlier process and PBSD a final draft is given where clear seismic objectives are mentioned and established considering site needs, safety criteria and conceptual design options.

The PBSD approach introduced the first time a way to explore what level of safety and security the owner needs by considering all different levels of earthquakes. These performance levels included structural strength, stiffness and ductility along with costs, safety and repairing cost. Conceptual design allowed owners and engineers to decide what level of storey drift they require or find safe.

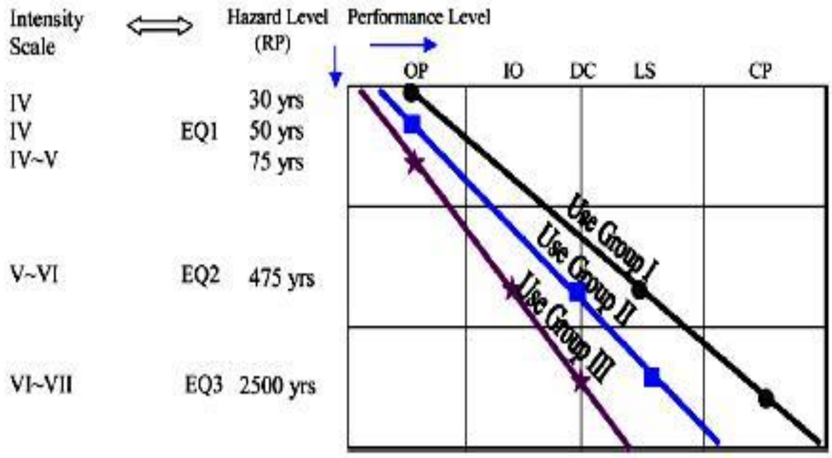


Fig. 2.1 Performance Objectives

In the draft direct displacement approach is used without considering the iteration on moment resisting frames. The method developed in the draft is for moment resisting frames only and are not valid or suitable for any other type of structures or frames.

As shown in Fig. 2.1, return period, probability of exceedance, or corresponding site intensity scale are the three seismic hazard levels considered. Also, performance of buildings is classified into five seismic performance levels as per the needs. These levels are Operational (OP), Immediate Occupancy (IO), damage control (DC), Life Safety (LS) and Collapse prevention (CP).

Ghobarah Ahmed 2001 had put forward some new important development to performance-based design**.** According to him the important objectives in performance-based design of low and moderate earthquakes are safety of life and control damage and in high intensity earthquake the prime objective is to control the total collapse of structures. As per his study the codes at the time focussed more on life safety but the method defined in the codes increase damage and repair cost. To counter the high-cost factor different performance objectives are considering while doing performance-based design.

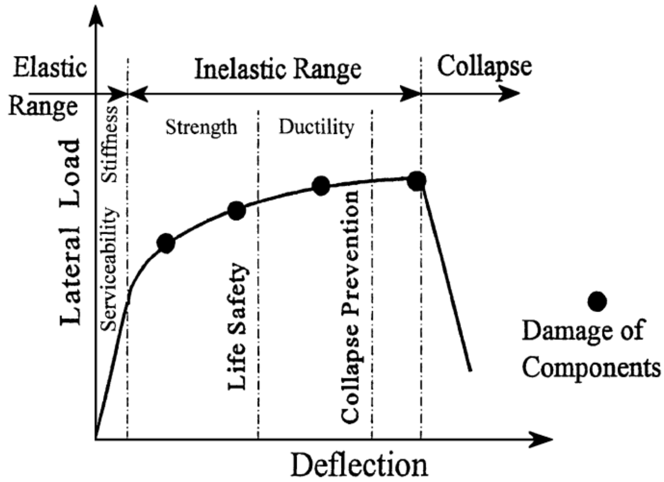


Fig. 2.2 Performance Curve of a Structure.

Performance levels like damage control, serviceability and life safety dominate the design along with structural characteristics like damage control and life safety. Even after concluding that much, it was difficult to predict the dominating factor in intermediate performances levels.

Operational (OP), Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS) and Collapse Prevention (CP) are the five levels in which performance of a building are classified. As shown in the following table earthquake hazard levels are associated with performance level.

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 or 10% in 100 Years
5	Extremely Rare	2475	2% in 50 Years

As per his research deformation-controlled design approach is the best approach for performance-based design.

Priestley MJN 2000 studied three methods (a) the N2 method (b) the capacity spectrum method and (c) direct-displacement method to find out the seismic force in a structure. The result is compared with force based seismic design. In his studied the main focus were soil problems, displacement and role of soil structure in seismic response of the structure. As per his study the key focus of a design should be simplicity and rationality. As per his understanding the better design requires focus on displacement and damage check. According to him there are implications of performance limit states to seismic design of structure. Another way is to design a structure on the basis of specified strain and stress under specific intensity. This approach turns out to be simpler and easier to apply to study seismic risk.

Mahaney, Freeman et al (1993), used the capacity spectrum method to study the response of four different type of structure to Loma Prieta Earthquake. The four structure includes one and two-storey wood frame houses, a 11 storey RCC building with shear walls and a 11 storey RCC building with Infill walls. In that study the ADRS spectra format was firstly introduced. The results shows that actual inelastic demands are way higher than damped inelastic displacement demands. Anyway, it was found out that damaged predicted by capacity spectrum method for RCC building is almost same as the reality.

Pang Weichiang et al In this study wooden structure of six storey were studied with shear walls using linear analysis method. It was seen in this study that wooden structure are good at protecting human life but they are not very effective in responding to earthquakes. This design turned out to more effective in designing than force-based design and does not need force reduction factor. Pre-fabricated shear walls are designed and tested in this paper.

Karapetrou, Pitilakis et al 2017 aimed at studying the building response to the earthquake based on age of the building. Modelling is done using chloride induced corrosion to study the response at age zero to age 25, 50 and 75 years. It was found the response of beams are more effected by corrosion than columns. And because the beams got weaker with time its stiffness decreases which increased the time period of the building. Uniform corrosion is considered to carry out this study.

Fragiadakis and Papadrakakis et al 2008 used a nonlinear response history analysis using structural optimization algorithm instead of the trial-and-error approach to find the most efficient design as per their cost and performance. Two approaches are considered in this paper, firstly, deterministic design and secondly, reliability-based optimization. It was concluded in the paper that unit material cost of steel and concrete directly influence the project cost. They also concluded that reliability optimization is more economical than deterministic approach.

X.-K. Zou et al (2005) put forward a better computer-based technique to study the seismic response using push-over analysis. As it is an iterative process the calculations are impossible to do by humans because of number of variables involves in a structure the computer-aided design seen to give a really good set of results.

In this study, the variable considered is steel reinforcement ratios as it is the most cost-effective thing in the building. The building is analysed using the non-linear inelastic method approach with steel reinforcement as variables. The design results are compared with respect to reinforcement in the paper.

R. K. Goel and A. K. Chopra put forward a simple and improved method for performance-based design using direct-displacement method. This method gives a simple procedure to determine the seismic response of a structure using single DOF.

In this process the deformation and rotation come a little lesser than non-linear analysis. Also, plastic rotation calculated using this method demands higher stiffness than appropriate value.

Qiang Xue, et al (2003) performed a performance –based design procedure by using displacement-based approach. The procedure that he explores in his study is reduced response spectrum using inelastic behaviour represented by reduction factor of the location. This idea gives a good brief understanding of performance-based design if the same examples are solved with non-linear time history analysis, it shows that the method used by him gives quite accurate results with simple calculations.

Mander J B (2001) started studying historical development in seismic design in New Zealand and studied the current practices, as per his study performance-based design helps the client to understand the degree of damage they can expect if an earthquake hits. The two philosophical approach he discussed in his paper are Control and Repairability of Damage (CARD), and Damage Avoidance Design (DAD).

CHAPTER 3 PERFORMANCE BASED DESIGN

3.1 PERFORMANCE BASED SEISMIC DESIGN

Performance-based seismic design gives a good idea about how a structure is going to perform. Figure 1.1 shows basic steps to follow to perform Performance- based seismic design.

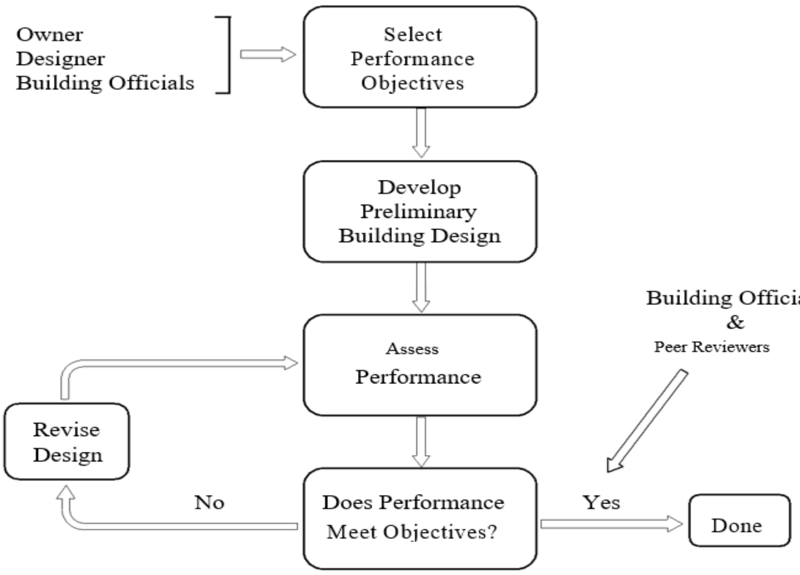


Fig. 3.1 Performance Based Flow Diagram

After all the objectives are done simulations are performed in succession to understand the actual behavior of the structure. In the case of extreme earthquake, non- linear approach is also performed for simulations. The design of structure is complete when the performance exceeds the objective, but the design is revised if the results are other way around.

As per the frame of reference of Performance-Based Design (SEAOC 2000 [13]), single or multiple objectives are taken into account and demand of the clients are taken into the consideration. To reach target performance and client's demand top story displacement, story drift, total displacement is considered with respect to displacement analysis.

After the conceptual design, structural design is made with all the detailing before the execution.

Preliminary design is performed by two different approached:

(1) Force based design is done to obtain the objectives

(2) Direct design methodology.

The later one gives the better idea and closer to real performance. Non- linear pushover and non-linear time history analysis approach are used to verify the result later.

Since this procedure has very complicated calculations only few objectives should be selected to perform the design.

3.2 NEED OF PERFORMANCE BASED DESIGN:

Performance-based is not used specifically to design earthquakes resistant design but it can also be used wind, ocean or fire system design. In such cases one full scale prototype is modelled and subjected to immense testing and design is revised and manufacturing processes are incorporated to the lesson learned during testing. Once design, testing and redesign is completed, the product is manufactured on large scale.

3.3 PERFORMANCE BASED DESIGN PROCESS

As Explained before, it is an iterative process where performance objectives are selected to meet the client's and structural objectives. The process can be better understood with following diagram:

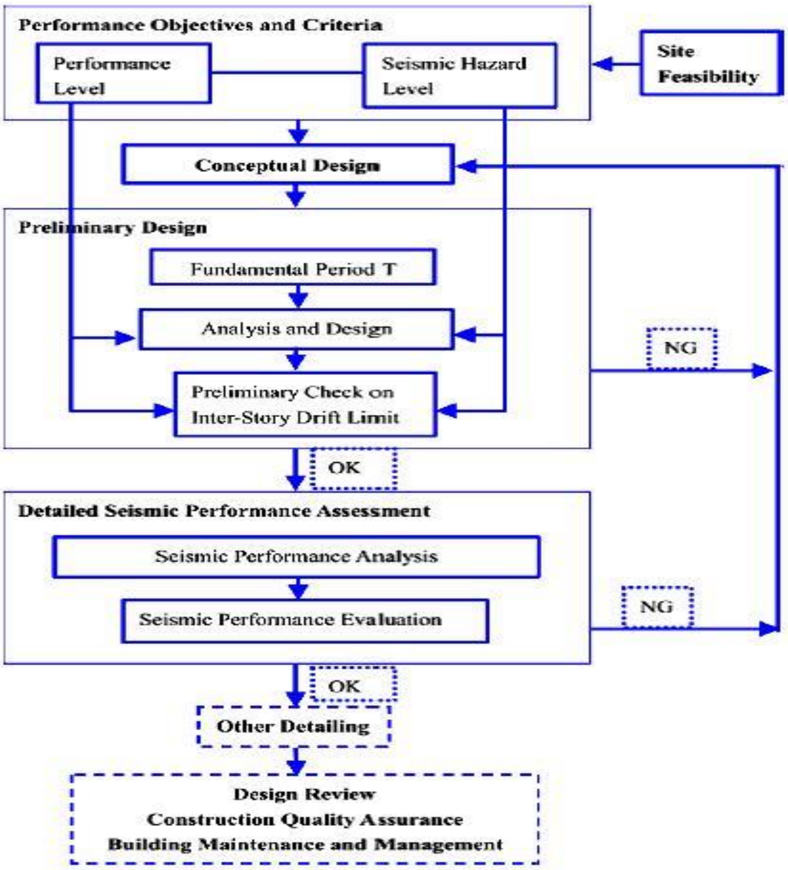


Fig. 3.2 Performance Based Design of New Building

3.3.1 SELECT PERFORMANCE OBJECTIVES

The design criteria start with defining acceptable performance objectives. Performance objectives are defined in the form of acceptable risk that a designer is allowed to take. These acceptable risks indicate the amount of loss whether structural or non-structural. These risks are mostly defined in the form of three formats mentioned below:

An ***Intensity-Based Performance*** accepts a decent loss after hitting by an earthquake, given that it is designed for the intensity of 475-year-mean-recurrence, cost of repair shouldn't exceed 20% of the current value of property, no life loss or serious injury is acceptable, and the building should be functional after 30 days of earthquake.

A ***Scenario-Based Performance*** accepts a decent loss after hitting by an earthquake, given that it is designed for the intensity of 7.0 earthquake, cost of repair shouldn't exceed 5% of the current value of property, no life loss or serious injury is acceptable, and the building should be functional after 7 days of earthquake.

A ***Time-Based Performance*** objective accepts a certain percentage of loss over a period of earthquake.

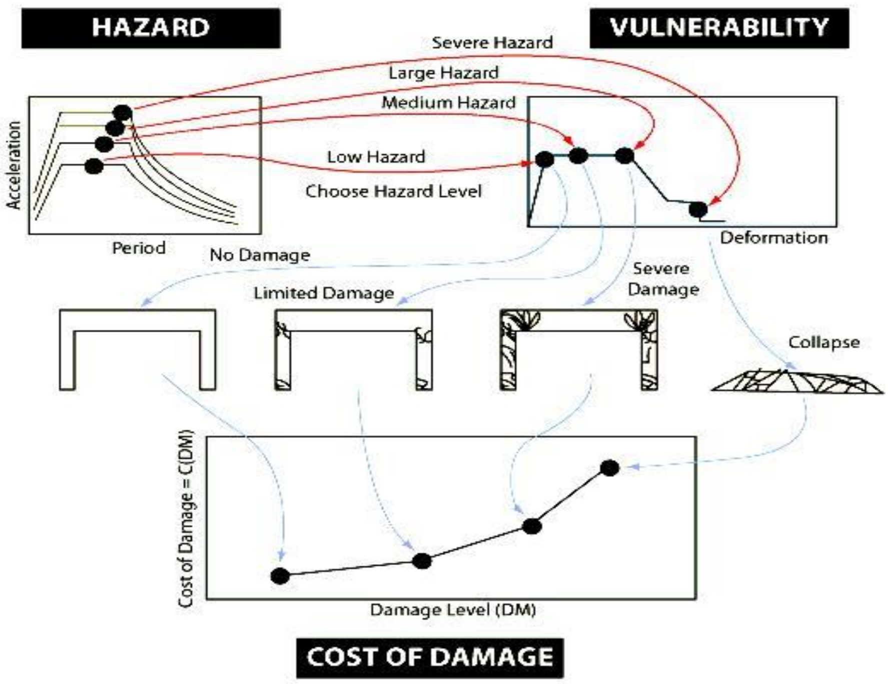


Fig 3.3 Performance Based Design Steps

3.3.2 DEVELOP PRELIMINARY DESIGN

The preliminary design for a structure depends on the following attributes:

- Site Location and territory.
- Building configuration, floor height, number of floors, irregularities and symmetry.
- Basic structural system.

- Presence of seismic Isolators, or any other energy dissipation system.
- Size and position of many structural and non-structural elements.

3.3.3 Assess Performance

After the preliminary design, series of simulations are done to know the probable response of the structure, which are:-

- Ground shaking hazard characterization.
- After that analysis of structure is performed to know the possible response when subjected to an earthquake. The behaviour of non-structural element is also tested as a function of intensity of ground shaking. These simulations can also be done by non-linear analysis method
- To find the damages that may happen to the structure because of these earthquakes.
- To anticipate the losses of life, wealth and value.

Earthquake hazard, building response, damage functions and loss are the four kinds of probability functions that we need to consider to complete the assessment performance.

Hazard functions are the mathematical calculation of earthquake intensities in different ways such as peak ground acceleration, spectral response acceleration etc.

Response functions are the mathematical calculations to study the response of a structure to different intensities of earthquake. These responses are mostly studied by calculating storey drift, inter-storey drift, top storey displacement, rotation etc. The technique which is used to understand the damage uncertainties to corresponding inputs is called fragility curves. **Figure 3.3** shows how we incorporate fragilities in performance-based design.

Damage functions are mathematical calculations which uses probability functions to make us understand the different levels of damage which includes structural, non-structural or any other damages. These probabilities are usually identified in laboratory testing or analytical simulations.

Loss functions are mathematical calculations regarding conditional probability considering various losses like life, wealth, cost for repair and time to make the structure functional again.

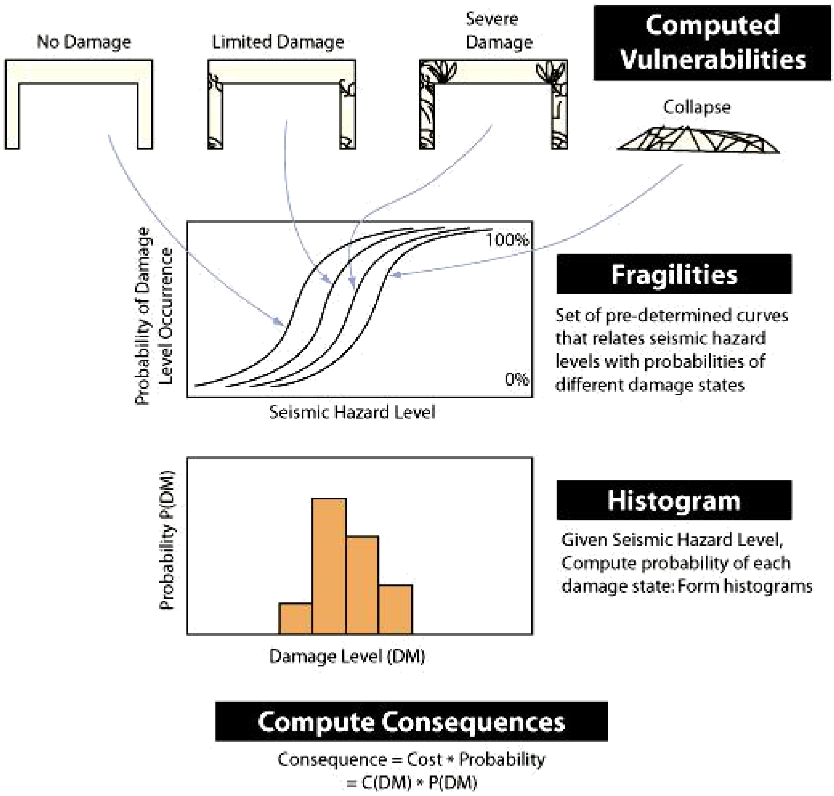


Fig. 3.4 Computation of Risk

3.3.4 REVISE DESIGN

If simulated performance does not meet the performance objectives, the design is revised again. If all the objectives could not be met then some relaxations are made on the performance objectives.

3.4 SEISMIC PERFORMANCE LEVELS

Mostly, the performance objectives are selected by all stakeholders which are a team of building owners, professional engineer and designers [2].

Performance Level: Performance level indicates the condition after an earthquake. This is measured as a point on a scale which tells us not just the life loss, but also in terms of property and its functions.

Building Performance Level: A combination of structural and non-structural damages are made to understand full nature of building damage. This damage indicates the overall damage level and cost.

3.4.1 STRUCTURAL PERFORMANCE LEVELS AND RANGES:

To study the behaviour and losses of structure three structural performance level and two structural performance ranges are defined. The Structural Performance Levels are the Immediate Occupancy Level (S-1), the Life Safety Level (S-3), and the Collapse Prevention Level (S-5).

The Structural Performance ranges are the Damage Control Range (S-2) and also the Limited Safety Range (S-4). There are no particular acceptance criteria for designing for intermediate performances ranges. The engineer must verify acceptance criteria before the designing for such performance.

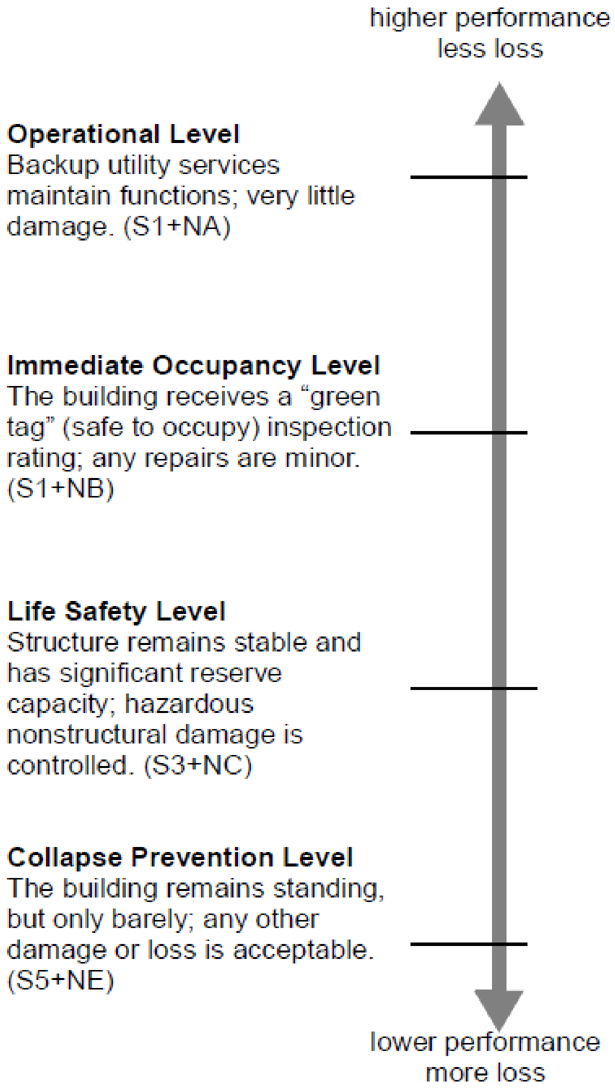


Fig. 3.5 Building Performance Levels and Ranges

Damage control Range (S-2) is obtained by interpolating the acceptance criteria for S-1 and S-3. Limited safety range (S-4) is obtained by the same way but interpolating S-3 and S-5.

3.4.1.1 Immediate Occupancy Performance Level (S-1)

Structural Performance Level S-1, Immediate Occupancy, signifies that the post-earthquake damage state does not have any damage to the vertical and lateral- force-resisting systems, in other words the building retains the nearly all the strength and stiffness after the earthquake.

3.4.1.2 Life Safety Performance Level (S-3)

Structural Performance Level S-3, Life Safety, means that some damages are acceptable like serious damage to non-structural elements, outside of the structure. Some small injuries are also acceptable but not any serious injuries.

3.4.1.3 Collapse Prevention Performance Level (S-5)

Structural Performance Level S-5, Collapse Prevention, means that the building is on the blink of almost total collapse. The structure may not fall immediately after the earthquake but will lose all the stiffness and strength.

3.4.1.4 Damage Control Performance Range (S-2)

Structural Performance Range S-2, Damage Control, means that the damage level is between immediate occupancy level (S-1) and Life safety Performance Level (S-3).

3.4.1.5 Limited Safety Performance Range (S-4)

Structural Performance Range S-4, Limited Safety, means that the damage level is between Safety performances Level (S-3) and Collapse Prevention Level(S-5).

3.4.2 NONSTRUCTURAL PERFORMANCE LEVELS:

Non-structural parts consist of wellbeing of things like partitions, HVAC systems, mechanical and electrical parts and lighting.

3.4.2.1 Operational Performance Level (N-A)

Operational Performance Level A, means that there is no serious damage to any non-structural elements like lightening and plumbing. There could be some small damage to these elements nut they will be in operational condition.

3.4.2.2 Immediate Occupancy Level (N-B)

Non-structural Performance Level B. In this level there could be some damage to the electric system, AC system or partition but the building will be easy to occupy after small reparation. Some damages to the windows and doors are acceptable as well.

3.4.2.3 Life Safety Level (N-C)

Non-structural Performance Level C, Life Safety, is the level where noticeable damages occur but no harm to the life inside is acceptable. However, it might cost a lot to repair those damages.

3.4.2.4 Hazards Reduced Level (N-D)

Non-structural Performance Level D, Hazards Reduced, is the level represents intensive damage to the non-structural elements and maybe life. However, serious injuries are avoided by avoiding failure of parapets, plaster ceilings or storage tanks.

3.4.2.5 Non-structural Performance Not Considered (N-E)

In some cases, Non-structural elements safety are not considered at all.

3.4.3 BUILDING PERFORMANCE LEVELS:

Structural and non-structural Performance levels are combined to obtain building performance level. Every Building Performance Level is defined by alphanumerical for example 1-B, 3-C. Some of the levels are defined below.

3.4.3.1 Operational Level (1-A)

Operational level (1-A) combines structural Immediate occupancy level (S-1) and non-structural operational level (N-A). Buildings with this level experience almost no damage to structural and non-structural elements. Building meeting this criterion cause very low risk to the life and property.

3.4.3.2 Immediate Occupancy Level (1-B)

This level is a combination of structural immediate occupancy level(S-1) and non-structural immediate occupancy level (N-B). Buildings designed considering this level cause almost no damage to their structural elements and only minimal damage to the non-structural damage. The building post-earthquake might need some repairs before preoccupancy.

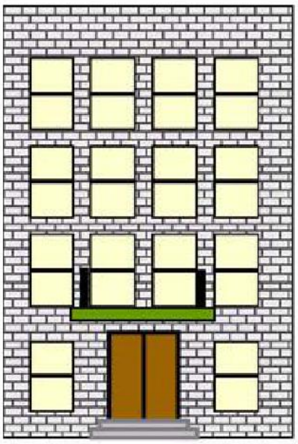
3.4.3.3 Life Safety Level (3-C)

This level is a combination of structural life safety occupancy (S-3) and non-structural lifesafety occupancy (N-C). Building designed with this level may have serious damage to structural as well as non-structural elements. The repair cost before preoccupancy can be large as well but it projects low risk to life safety.

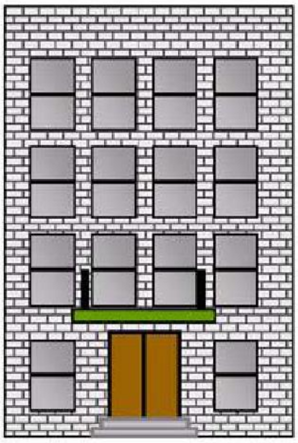
3.4.3.4 Collapse Prevention Level (5-E)

This level takes the structure to collapse but no serious concern for non-structural elements. This level can do huge damage to life safety from failure of non-structural elements.

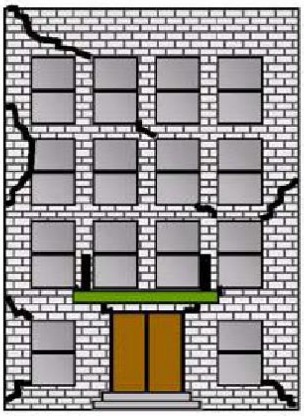
Although it may cause serious damage to the non-structural elements, but huge loss of life can be prevented as the structure will not collapse.



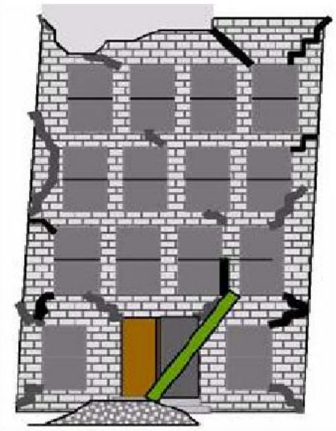
Operational



Immediate Occupancy



Life Safety



Collapse Prevention

Fig. 3.6 Graphical Representation of Performance Levels

Table 3.1 Building Performance Levels and Ranges

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 Recommended	Not Recommended	Not Recommended	Not Recommended
Immediate Occupancy N-B	**Immediate Occupancy 1-B**	2-B	3-B	Not Recommended	Not Recommended	Not Recommended
Life Safety N-C	1-C	2-C	**Life Safety 3-C**	4-C	5-C	6-C
Hazards Reduced N-D	Not Recommended	2-D	3-D	4-D	5-D	6-D
Not Considered N-E	Not Recommended	Not Recommended	3-E	4-E	**Structural Stability** 5-E	Not Recommended

3.5 SEISMIC HAZARD

Performance objectives are formed with combination of Hazard levels and the performance levels. As an example, effects of earthquakes like land sliding, liquefaction and settlements are parts of the earthquake hazards. These hazards are approached with response spectrum method or time history with deterministic or sometimes probabilistic analysis.

When maximum ground shaking is observed for a particular region using past earthquakes, it's called deterministic approach but when this deterministic approach is coupled with probability of occurrence it's called probabilistic approach.

ATC – 40 provide three earthquakes' hazards and they are quantified by the scale intensity, return period and probability of occurrence. These hazards are briefly explained below:

3.5.1 Serviceability Earthquake Hazard Level

This hazard level considers the chance of exceeding the frequent level of earthquake is 500th probability in 50 years with return period of 72 years. In some cases, the chance of exceeding the occasional seismic hazard level is considered 200th probability in 50 years with return period of 225years along with probability of frequent hazard level.

3.5.2 Design Earthquake Hazard Level

This earthquake hazard level is also sometimes mentioned as Basic Safety Earthquakes 1 (BSE-1). This hazard level considers 100 percent probability of occurring the hazard in 50 years with 475 years return period. This level comes into play mostly in rehabilitation function to obtain desired safety needs.

3.5.3 Maximum Earthquake Hazard Level

This earthquake hazard level is also sometimes mentioned as Basic Safety Earthquakes 2 (BSE-2). This hazard level consider 2 percent probability of occurring the hazard in 50 years with 2475 years return period. In some guidelines, this hazard level considers 5 percent probability of occurring the hazard in 50 years with 970 years return period.

CHAPTER 4 ANALYSIS AND DESIGN

To achieve different kind of accuracy we require different approach. The type of loads applied, the kind of structure material used and the structure geometry help us to decide what method we use to analyse the structure in hand.

4.1 SEISMIC DESIGN PHILOSOPHY:

The seismic design philosophy of structures can be put together as:

- The fundamental philosophy behind any seismic design is to make sure that a structure can take small earthquake without any damage, can take medium earthquake without any structural damage but a small non-structural damage is acceptable and can take big earthquakes without total collapse of the structure.
- It's observed that the true earthquakes forces on a structure are always higher than the loads recommended by standard code. Although with good knowledge and experience it is possible that a structure can be designed to resist very large earthquake but most of the time it is economically impossible to provide that much strength to a structure. So, to handle these dynamics of strength and economy we try to invest in providing lateral strength and ductility. Ductility allows a structure to deflect a little and maybe accept some damage but not complete collapse. The ductility can be provided using proper inelastic material and ductile reinforcement which can help us to avoid any big brittle failure in the structure. This approach can also provide extra reserve strength to our structure which may allow it to resist bigger earthquake than it is designed for.
- The structure response depends on some other factors apart from its lateral strength such as foundation soil, size of foundation, construction material and characteristics of ground movement.

4.2 METHODS OF ANALYSIS:

The analysis may be classified as:

- Linear Static Analysis
- Linear Dynamic analysis
- Nonlinear Static analysis
- Nonlinear Dynamic Analysis

Linear static analysis is only effective in the case of very small structure. Linear dynamic analysis helps us to understand not only the structure but also earthquake. In this type of analysis our basis to study the behaviour of a structure depends on history of earthquakes and behaviour of structure with different frequency. A history of earthquakes is studied and the response of structures to those earthquakes are measured, then this data is used to calculate the response (Displacement, acceleration and velocity) of our structure. Two methods are available to analyse the structure using this approach i.e., Response spectrum method and time history method.

Though the results obtained using linear static analysis are quite fruitful and accurate but the major disadvantage of using this method is that it only uses elastic property of a structure which ends up giving very big size of structure elements. This mostly result in highly uneconomical structure. To deal with this problem we use non-linear static analysis approach. In this method we can take advantage of inelastic behaviour of the structure which help us to have better understanding of a structure behaviour and earthquake load. The non-linear dynamic analysis helps us to get the best and most accurate results for response of a structure to an earthquake. It helps us to know that response of a structure at every moment of earthquake, the deformations of every element and what parts of the structure are more vulnerable to an earthquake.

4.3 DESIGN SEISMIC LATERAL LOADS

The earthquake forces are distributed uniformly over the floors to calculate the response of a structure. The two procedures used to calculate the distribution of forces across the height of the building are equivalent lateral static force method and modal analysis method. The first method uses a simple formula to distributed lateral forces across the floors and the later method uses properties of natural vibration modes of a building which further based on stiffness and mass distribution across the floors. The first and the second method tend to give the same results for small symmetrical structure but the modal analysis method gives better results for big and complex structures.

4.4 EQUIVALENT LATERAL FORCE PROCEDURE

Equivalent lateral force method is an elastic method of analysis which affords stable but highly uneconomical results. In this method a modest total lateral force at the base of the structure is premeditated and dispersed across the floors based on their stiffness. The procedure to estimate the forces is briefly discussed using subsequent points.

1. The base shear is calculated as:

$$V_{B=} A_h \times W$$

where,

A_h = design horizontal acceleration coefficient

W = seismic weight of the building

2. Fundamental natural period Ta

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

3. The design base shear is distributed across the floors as:

$$Q_i = \frac{W_i h_i^2}{\sum_{j=1}^{n} W_i h_i^2} \text{ X VB}$$

where,

Q_i = design lateral force at floor i,

W_i = seismic weight at floor i,

h_i = height of the floor i measured from base,

n = number of stories in the building,

4.5 DESIGN ACCELERATION SPECTRUM

Design acceleration spectrum method is a way to predict the acceleration or velocity of a structure when the data regarding the history of earthquakes at a place is not available. In the modal analysis method, a single structure is divided into multiple single degree of freedom system and then natural frequency of the structure corresponding to each degree of freedom is calculated. After we have natural frequencies we calculate different modes i.e., shape of the structure corresponding to each frequency. After we have the modes we combine them to get the understanding of behaviour of the structure. The design spectrum method defined in the code follows the same way but it helps to provide the response of a structure i.e., it's velocity, acceleration and displacement corresponding to a particular frequency. The design spectrum method defined in the IS 1893 is briefly discussed below:

- As per the position of tectonic plates the country is divided into four zones. These zones have different seismic factors used to calculate acceleration in case the history of earthquakes is not available.
- The design horizontal acceleration coefficient for a structure is calculated as:

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

where,

Z = Earthquake zone factor

I = This factor depends on the type of structure, it's history, and future importance.

R = As the design spectrum method is elastic method analysis, this factor allows to use inelastic properties of the structure by decreasing the acceleration and hence forces.

$\frac{S_a}{g}$ = design acceleration coefficient which depends on the soil properties.

4.6 PUSHOVER ANALYSIS

In pushover analysis the lateral load is applied in tiny steps and then behaviour of the structure is noticed. The response of structure is noted at every increment of the forces and the part of the structure that will yield first is identified. After finding the yielding points the structure elements are modified to be able to take that force and the lateral force is increased further to make the structure reaches the acceptable level of deflection.

4.6.1 Description of Pushover Analysis

The two organization that developed this method and recognize the potential of this method are Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC). These organization release following documents to help engineer and academician with their latest discoveries and research.

As Per FEMA 273

This organization helps professionals and professor to understand this method better and apply that in their professional and academicals careers.

As Per ATC 40

Unlike FEMA this publication deals with concrete structures only. Some points are mentioned below to understand the way this research organization work.

1. To understand the aim of the project and requirements of the clients.
2. Once we understand our requirement we need to contact and find a qualified professional with experience in designing and retrofitting the building in highly seismic area so that they can stand even if they are hit by an earthquake.
3. Then the hired Engineer needs to visit the site to get the better idea of the location and the geometry of the structure.
4. We need to find that if non-linear Analysis is compatible for the structure or not.
5. After that the engineer needs to sit with the team to understand whether the required design needs a complicated non-linear design or not.
6. After consulting carefully non-linear analysis is performed.
7. History of the response of the structures subjected to past earthquakes are taken and used to identify the response of the new structure that needed to be made.

8. After that performance of the structure is noted and the decision needs to be taken for the deflection and the force that the structure can bear.

4.7 INELASTIC COMPONENT BEHAVIOUR

The most important step for analysis of a structure that the elements which will resist the lateral load needs to be identified. It's Important to identify the points first that are vulnerable to lateral forces and needs to be redesigned to make them able to take the lateral load without failure.

By applying this approach, the relationship between displacement and lateral load is developed which is called the **Capacity Curve.** After developing the curve ATC 40 helps to decide the acceptability limits for an element of a structure.

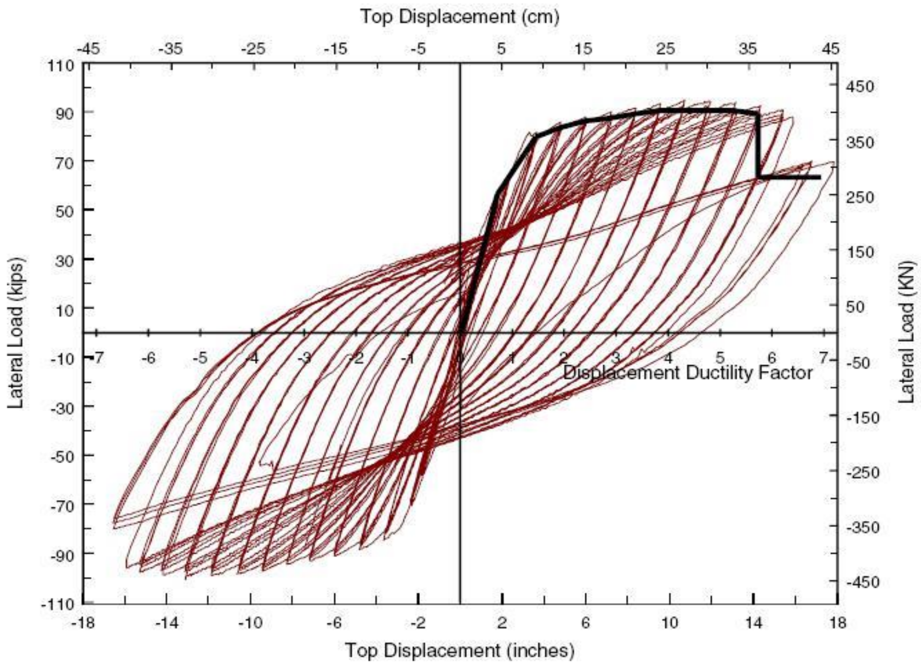


Fig. 4.1 Backbone Curve from Actual Hysteretic Behaviour

4.8 PLASTIC HINGES

Plastic hinges or Inelastic hinges are the points or locations of a structure that surpasses their elastic limit first and lose their strength to resist the forces and start transferring these forces to other elements of the structure.

Location of hinges are first identified to understand how much lateral load a structure can take without losing a single element of it.

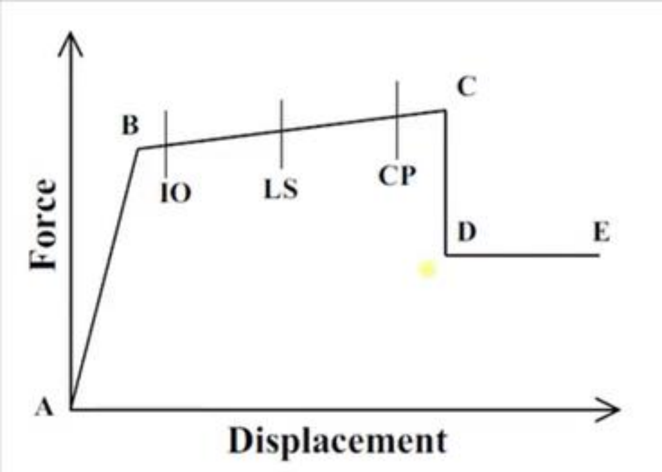


Fig.4.2 Force – Displacement Curve of a Hinge

The Backbone curve of hinges can be seen in fig. 5.3, where

a. Point A represents the first state of the structure often called Original State (OS)
b. Yielding is represented by point B.
c. The ultimate capacity is represented by point C.
d. Residual strength i.e., the strength after which the element or the structure cannot take further load is represented by point D.
e. After point D the structure starts or the element starts to fail and at point E the structure or the element fails.

4.9 CAPACITY SPECTRUM METHOD

This method required the capacity i.e., the lateral load the structure can take and the demand i.e., for the degree of deflection it is designed for. The point at which both the curves meet is called performance point i.e., when the demand and the capacity of a structure are equal.

4.9.1 Conversion of Pushover Curve to Capacity Curve

For converting the Spectral Acceleration i.e., S_a vs time graph it's necessary to find the Spectral Displacement for each point on the curve.

This is calculated as:

$$Sd_i = \frac{T_i^2}{4\pi^2} Sa_i g$$

After finding the displacement spectral acceleration and spectral velocity are calculated as

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

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

From the capacity curve pushover curve is formed through conversion of spectral coordinates. Base shear (V_i) and the top storey displacement are measured with the help of S_{ai} and S_{di} using the following equation.

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

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

where,

the Modal Mass coefficient, participation factors and roof level amplitude are represented by α_1, PF_1 and $\emptyset_{1,roof}$ for the first mode of structure and can be calculated as

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

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

where,

W_i is the weight of the ith floor.

The period of a structure increases as the displacement increases. Demand of a structure cuts back when inelastic displacement increases. The following figure shows how pushover curve coverts to capacity spectrum.

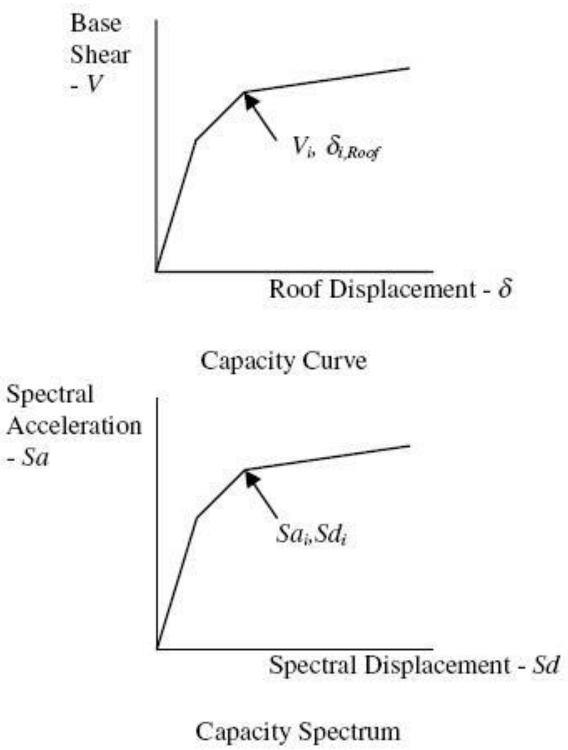


Fig. 4.3 Capacity Spectrum Conversion

In the inelastic range the damping is a combination of viscous and hysteretic damping. So, the total effective damping can be estimated as:

$$\beta_{eff} = K\beta_0 + 0.05$$

where,

the hysteric damping is represented by β_0 and 0.05 represents default 5 % viscous damping of the structure.

The K factor depends on the seismic resisting system and the structure's time period. The term β_0 can be computed as

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_{S0}}$$

where,

E_D represents the energy released by damping force.

E_{S0} represents the structure's strain energy.

The following picture gives the better clarity regarding the relationship discussed above.

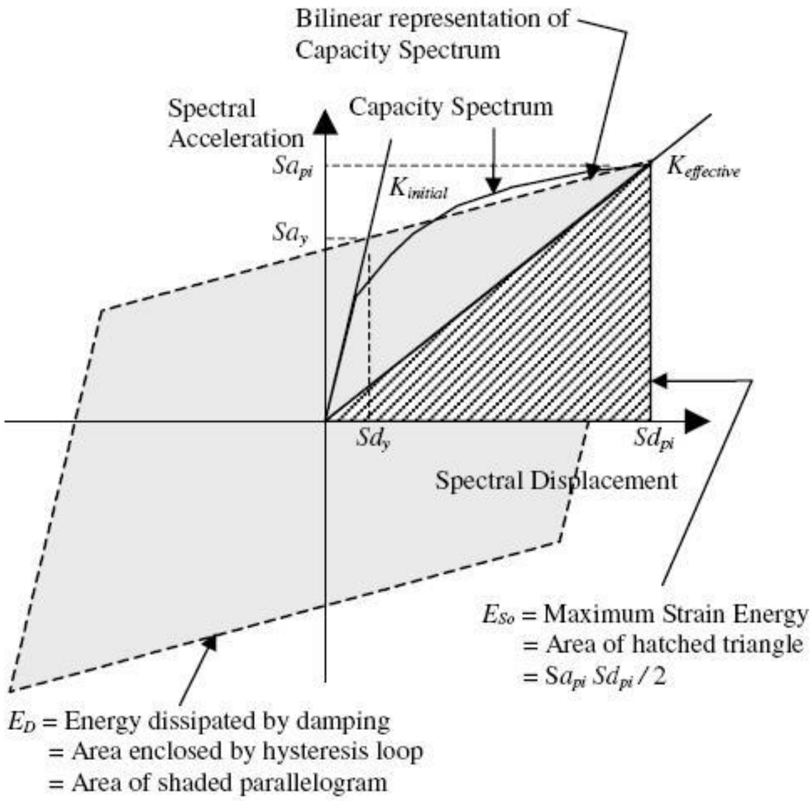


Fig. 4.4 Derivation of Energy Dissipated by Damping

As damping becomes more and more effective as the structure goes beyond elastic point the response of a structure is reduced by

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

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

Table 4.1 Minimum Allowable SR_A and SR_V Values

Structural Behaviour Type	SR_A	SR_V
Type A	0.33	0.50
Type B	0.44	0.56
Type C	0.56	0.67

S_{Ra} *and* S_{Rv} values should always be greater than the allowable value shown in the above figure. Response spectrum is achieved by reducing Elastic Response Spectrum as shown below.

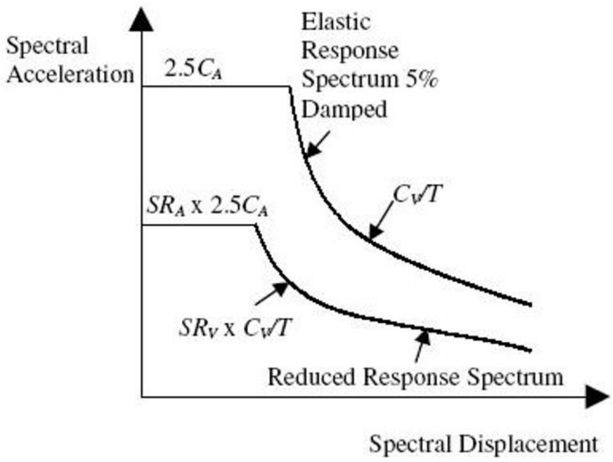


Fig. 4.5 Reduced Response Spectra

4.9.2 Determination of Performance Point

ATC-40 suggests three methods to identify the performance point and those are:

- **A)** The set of equation as mentioned in ATC-40 are used for easy and simple programming.
- **B)** The assumption is made that bilinear representation remains constant for yield point and post yield slope. As it's an iterative procedure it is easy to find interactive points. This procedure is valid most of the times/
- **C)** This procedure is easy to use and the established software like SAP2000 uses the same procedure.

4.9.3 Pushover Curve

Base shear is plotted against displacement of the structure in the pushover curve. The curves help us to understand the overall response of the structure. Incremental seismic loading is used to calculate the displacement of the structure and hence its response at any point of time.

4.9.4 Performance Point

The Performance point is the point where capacity equals demand of a structure which means the capacity and the demand are equally met. Therefore, this point gives the most cost-effective stable structure. Displacement effects the time period a lot, as displacements increases time period increases. This can be seen in capacity spectrum method. So, here for less displacement we have high capacity.

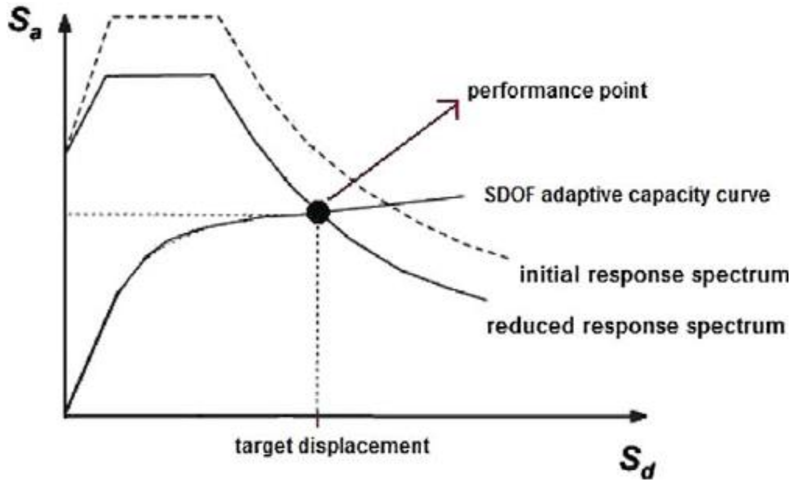


Fig 4.6 Performance Point for Pushover Analysis

CHAPTER 5 CALCULATIONS AND RESULTS

The cardinal objective of Performance Based Seismic Design of building is to abstain total catastrophic damage and to restrain the structural damage provoked to the Performance limit of the structure. For this aspiration analysis is performed to reckon the actual strength of the structure.

5.1 PERFORMANCE OBJECTIVE

The subsequent level Performance Objective is proposed for new structures.

- Under Serviceability Earthquake Level, slight structural damage in order to empower immediate occupancy Performance Level after Serviceability Earthquake.
- Under Design Basis Earthquake Level, moderate structural damage in order to empower Life Safety Performance Level after DBE.
- Under Maximum Considered Earthquake, high structural damage in order to prevent the structure from Collapse to empower Collapse Prevention Performance Level after MCE.

5.2 DESCRIPTION OF BUILDING

In this study, a G+6 storey RC SMRF building situated in Zone V is considered. The plan area of the building is 20m X 30m with 4m as typical storey height. The building has 4 bays of 5m each in X-direction and 6 bays of 5m each in y-direction. The total height of the structure is 28m. The plan and 3D view of the building is as shown in fig. 5.1 & 5.2 respectively.

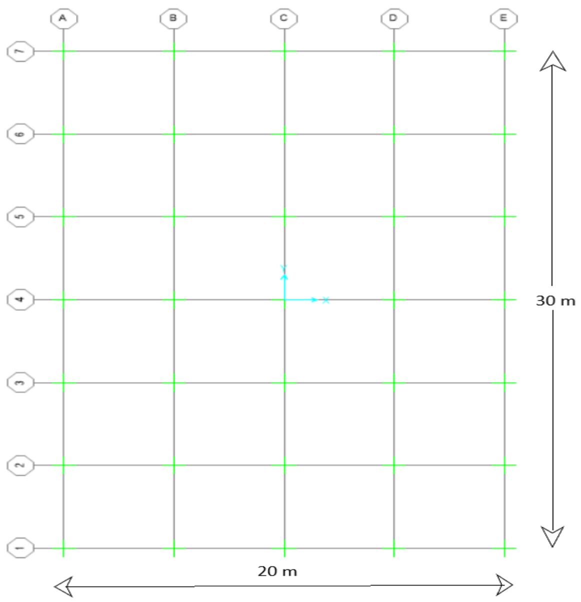


Fig. 5.1 Plan of the building

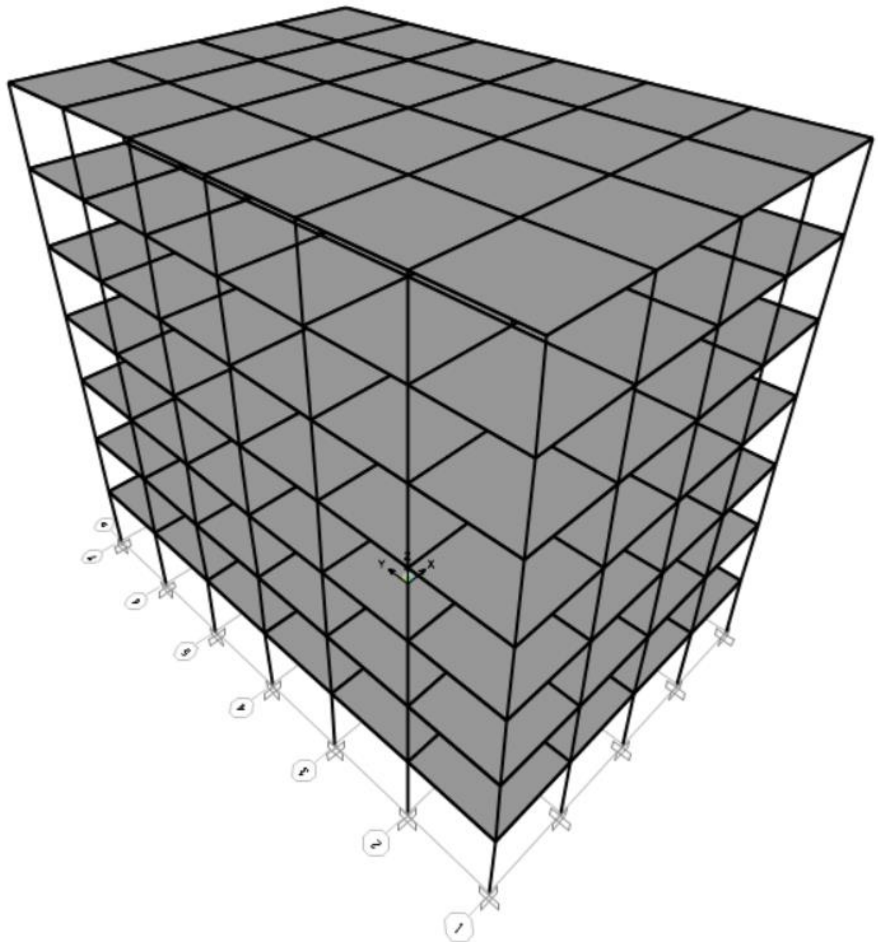


Fig. 5.2 3D view of the building

5.2.1 Sectional Properties of Elements

The sectional properties are as follows:

Size of Column	600mm × 600mm
Size of Beam	500mm × 300mm
Thickness of Slab	140mm

5.2.2 Loads Considered

The following loads were considered for the analysis as per S: 875.

Gravity Loads: The intensity of dead and live load at various floors and roof levels considered are listed as Dead load

Roof Level

Weight of Slab	0.140 × 25	3.5 kN/m^2
Roof finishes		1.0 kN/m^2
Total Dead load		4.5 kN/m^2

Floor Level

Weight of Slab	0.140 × 25	3.5 kN/m^2
Floor Finishes		1.0 kN/m^2
Total Dead load		4.5 kN/m^2

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

5.3 SEISMIC LOADS

The design lateral forces due to earthquake excitation s calculated as follows:

- Fundamental Natural Period, $T_a = 0.075h^{0.75}$= 0.913 sec
- Design Horizontal Seismic Coefficient, A_h

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} = 0.0536$$

$Z = 0.24$ (for zone V)

R = 5.0 (SMRF)

I = 1.5 (Important Structure)

$\left(\frac{S_a}{g}\right) = 1.4895$ (Medium stiff soil sites having $T_a = 0.913\ sec$)

- Seismic weight, W at every floor level is contribution of full dead load and percentage of imposed load as mentioned in S 1893-2016 Clause 7.3.1

  At roof level, W_i= 3637.5 kN/m^2

  At floor level, W_i= 5497.5 kN/m^2

  Total seismic weight for the building = 36622.5 kN/m^2
- Design Seismic Base Shear, V_B

$$V_B = A_h \times W = 1962.966 \text{ kN}/m^2$$

- Distribution of design Base Shear, Q_i

$$Q_i = V_B \times \frac{W_i h_i^2}{\sum_{j=1}^{n} W_i h_i^2}$$

$Q_1 = 15.905$ kN/m^2

$Q_2 = 63.618$ kN/m^2

$Q_3 = 143.141$ kN/m^2

$Q_4 = 254.473$ kN/m^2

$Q_5 = 397.614$ kN/m^2

$Q_6 = 572.564$ kN/m^2

$Q_7 = 515.650$ kN/m^2

5.4 RESPONSE SPECTRUM ANALYSIS USING SAP 2000

The following steps are included in the Response Spectrum analysis in sap 2000.

- Seismic loading as per S 1893-2016 along X direction and Y direction is taken as shown in fig. 5.3 and fig. 5.4.

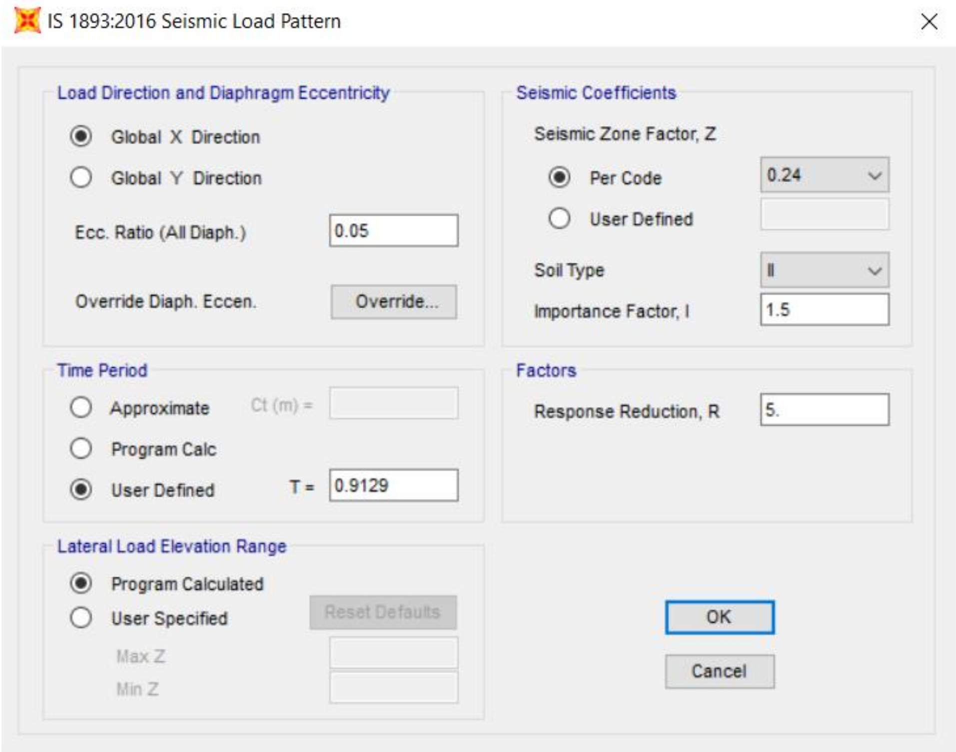


Fig. 5.3 Seismic Loading along X direction

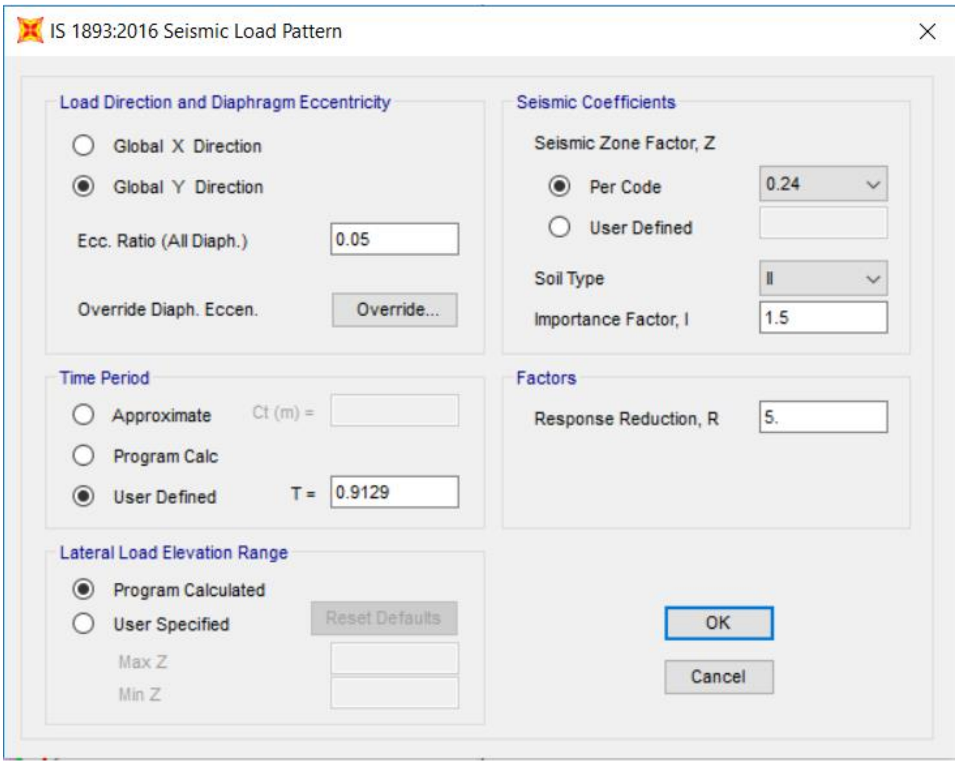


Fig. 5.4 Seismic loading along Y direction

- Mass Source is defined as 100% for Dead Loads and 50% for imposed Loads as used in calculation of base Shear and will definitely effect the displacement of the building in Seismic Modes as shown in fig. 5.5.

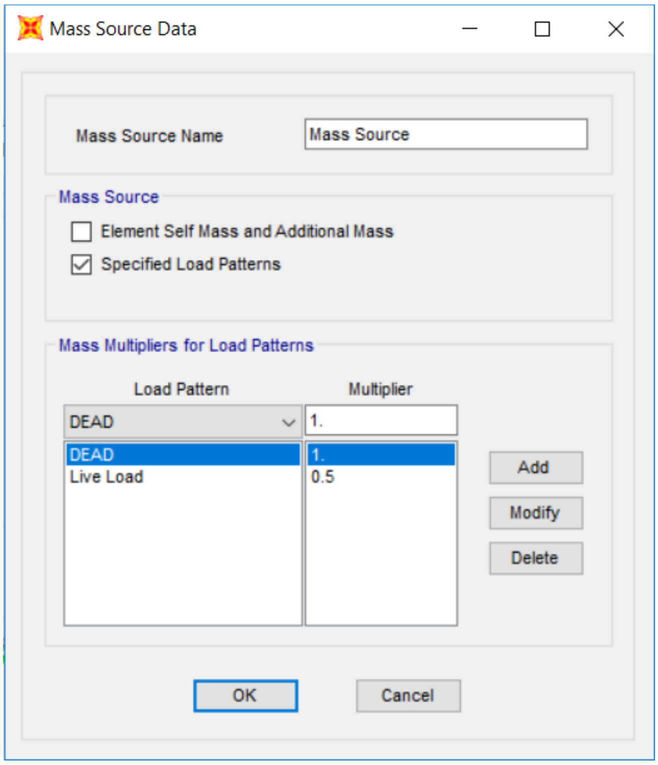


Fig. 5.5 Mass Source

- Modal cases are defined using Modal subtype as Eigen vectors based on Mass. Maximum no of modal cases to be considered in our analysis are 21 as for each floor level, we have 2 translational and 1 rotational degrees of freedom, so in our study we have considered G+6 building. But initially we will provide 8 number of modes as more than 90% of mass is participating in 8 modes only.
- Defining Response Spectrum Function As per S 1893-2016 considering our building in Seismic Zone V having Seismic Zone Factor of 0.24, importance factor of 1.5, Response Reduction factor of 5 on Soil Type I and having Functional Damping of 5% s shown in fig. 5.6.
- After defining response Spectrum function a load case is defined having case type as response Spectrum Along X direction and Y direction with modal combination method as CQC (Complete Quadrilateral Combination) and directional combination type as SRSS (Square Root Sum of Squares) for computing critical direction of the seismic ground motion with respect to principal axis of the structure. For our analysis we select 5% constant damping as shown in fig. 5.7 and fig. 5.8.

- Now we will Run Analysis and in results section we will see
  a) Time period and frequency for Response Spectrum Analysis having different modes with eigen values are represented in Table 5.1.
  b) Base reaction in X direction And Y direction when compared with Maximum Response Spectrum value in the same direction should come 85% or more. Table 5.2.
  c) Modal Participation Mass Ratio which when combined should always be greater than or equal to 85% of the actual mass in each orthogonal direction considered. If this ratio is found to be less than 90% then again no of modes are increased. Table 5.3.
  d) Modal Load Participation ratio whose percentage for static and dynamic behaviour must be greater than 90% in both directions. Table 5.4

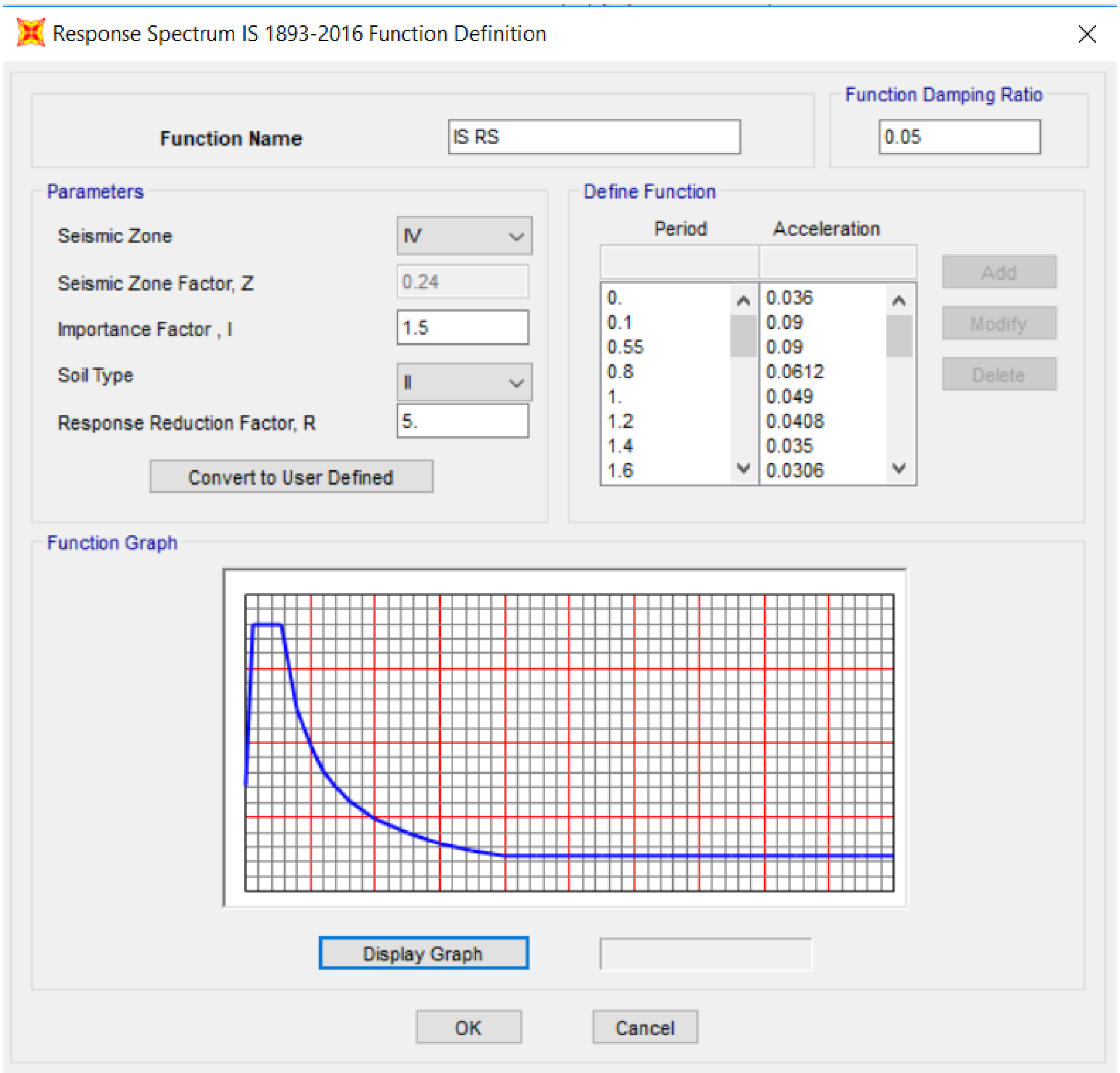


Fig. 5.6 Response Spectrum Function

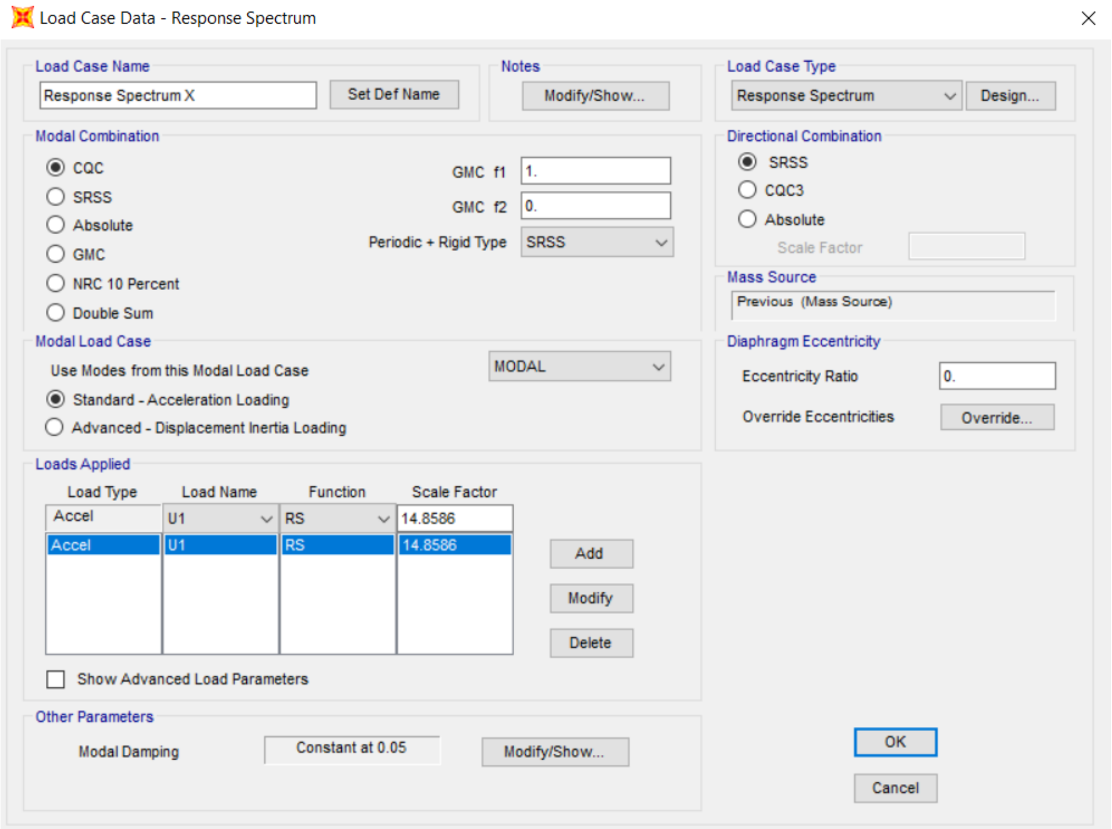


Fig. 5.7 Response Spectrum Load Case in X Direction

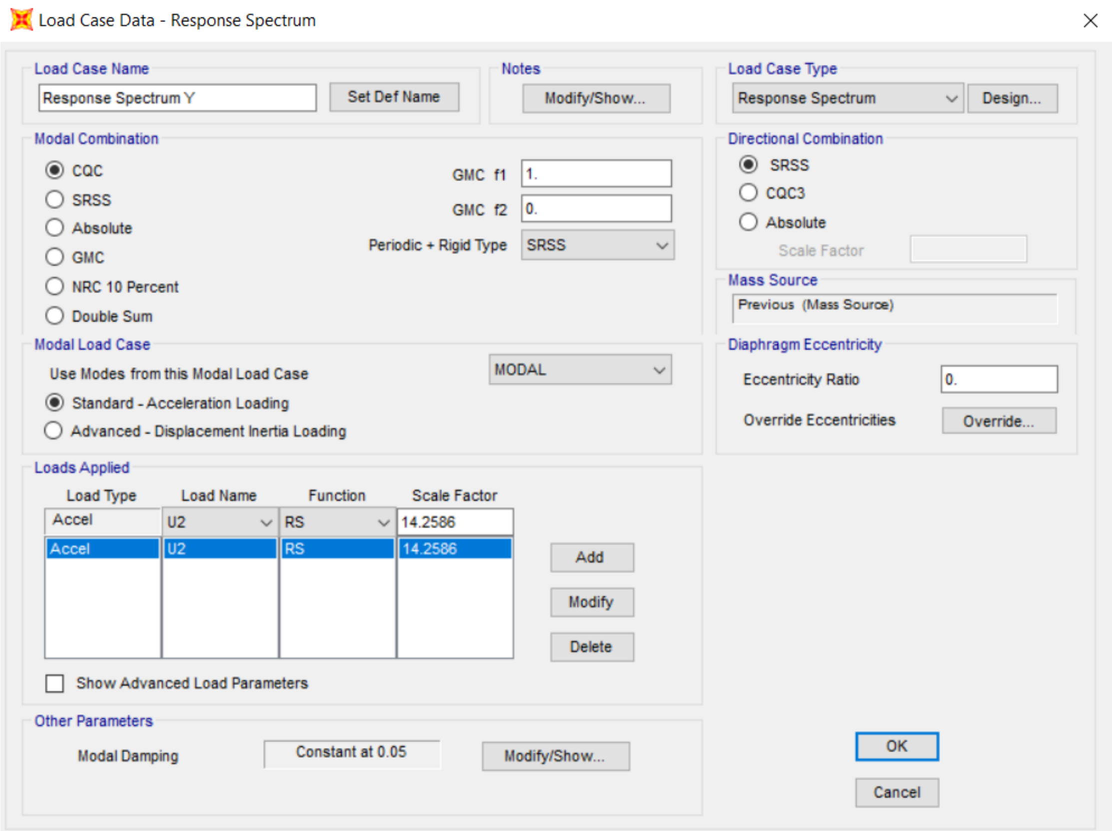


Fig. 5.8 Response Spectrum in Y Direction

Table 5.1 Periods and Frequency for Response Spectrum

Output Case	Step Type	Step Num	Period	Frequency	CircFreq	Eigenvalue
			Sec	Cyc/sec	rad/sec	rad2/sec2
MODAL	Mode	1	1.1575	0.8639	5.42822	29.4656
MODAL	Mode	2	1.1310	0.8841	5.5552	30.8603
MODAL	Mode	3	1.0504	0.9519	5.9814	35.7783
MODAL	Mode	4	0.3646	2.7425	17.2317	296.9315
MODAL	Mode	5	0.3574	2.7977	17.5786	309.0065
MODAL	Mode	6	0.3315	3.0162	18.9514	359.1547
MODAL	Mode	7	0.1990	5.0240	31.5668	996.4642
MODAL	Mode	8	0.1962	5.0971	32.0261	1025.6737

Table 5.2 Base Reaction for Response Spectrum

Output Case	Case Type	Step Type	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
			KN	KN	KN	KN-m	KN-m	KN-m
EQ X Load	Lin Static		-2254.85	1.24E-09	1.48E-11	-2.46E-08	-50034.8	2782.389
EQ Y Load	Lin Static		1.98E-09	-2254.85	8.41E-12	50034.84	2.97E-08	-1854.93
RS X	Linear RS	Max	2281.671	0.000282	0.018	0.0401	43050.44	0.0021
RS Y	Linear RS	Max	4.52E-05	2244.448	0.000216	42414.09	0.0067	0.0061

Table 5.3 Modal Participating Mass Ratio

OutputCase	StepType	StepNum	Period	SumUX	SumUY	SumRZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless
MODAL	Mode	1	1.157503	0.8059	3.25E-19	5.552E-16
MODAL	Mode	2	1.131045	0.8059	0.80815	2.256E-15
MODAL	Mode	3	1.050437	0.8059	0.80815	0.80756
MODAL	Mode	4	0.364629	0.90985	0.80815	0.80756
MODAL	Mode	5	0.357434	0.90985	0.91092	0.80756
MODAL	Mode	6	0.331542	0.90985	0.91092	0.9099
MODAL	Mode	7	0.199044	0.95219	0.91092	0.9099
MODAL	Mode	8	0.196189	0.95219	0.95288	0.9099

Table 5.4 Modal Load Participation Ratios

OutputCase	ItemType	Item	Static	Dynamic
Text	Text	Text	Percent	Percent
MODAL	Acceleration	UX	99.9489	95.2194
MODAL	Acceleration	UY	99.9482	95.2877
MODAL	Acceleration	UZ	1.901E-07	2.621E-08

5.5 PUSHOVER ANALYSIS USING SAP 2000

The following procedure is performed for the Pushover analysis n SAP 2000:

- Design structural members as per S 456 n SAP 2000.
- Convert the previously assigned Linear Static DEAD Load Case to Nonlinear Static that SAP2000 can use this load case as the starting point of the Pushover Load case. Fig. 5.9.

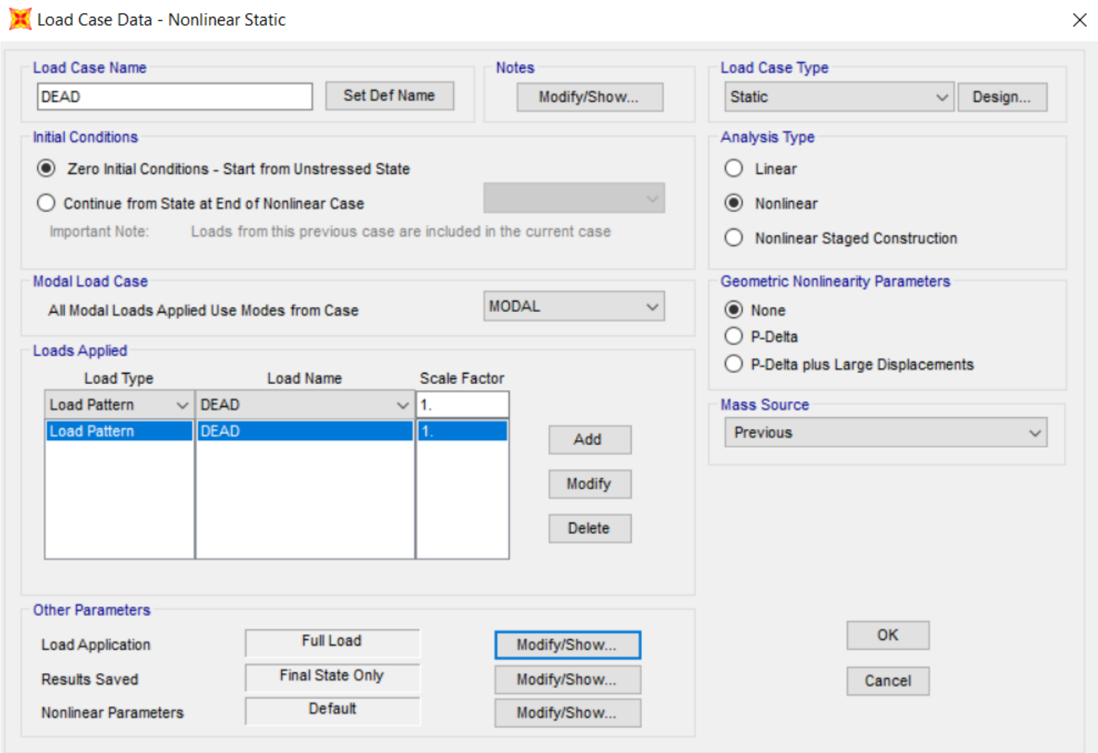


Fig 5.9 Non-Linear Static Dead Load

- Now define Pushover Load Cases in X-direction and Y-direction which will continue from the end of Nonlinear load case Dead considering Mass Source as previously assigned by loading as Acceleration load type in UX and UY direction with scale factor of 1 and load application as displacement control using monitored displacement of 230mm as

Target Displacement computed from FEMA 273 procedure on joint no. 8 in X direction and on joint no 8 in Y direction for immediate Occupancy Performance Level. Fig. 5.10 and fig. 5.11.

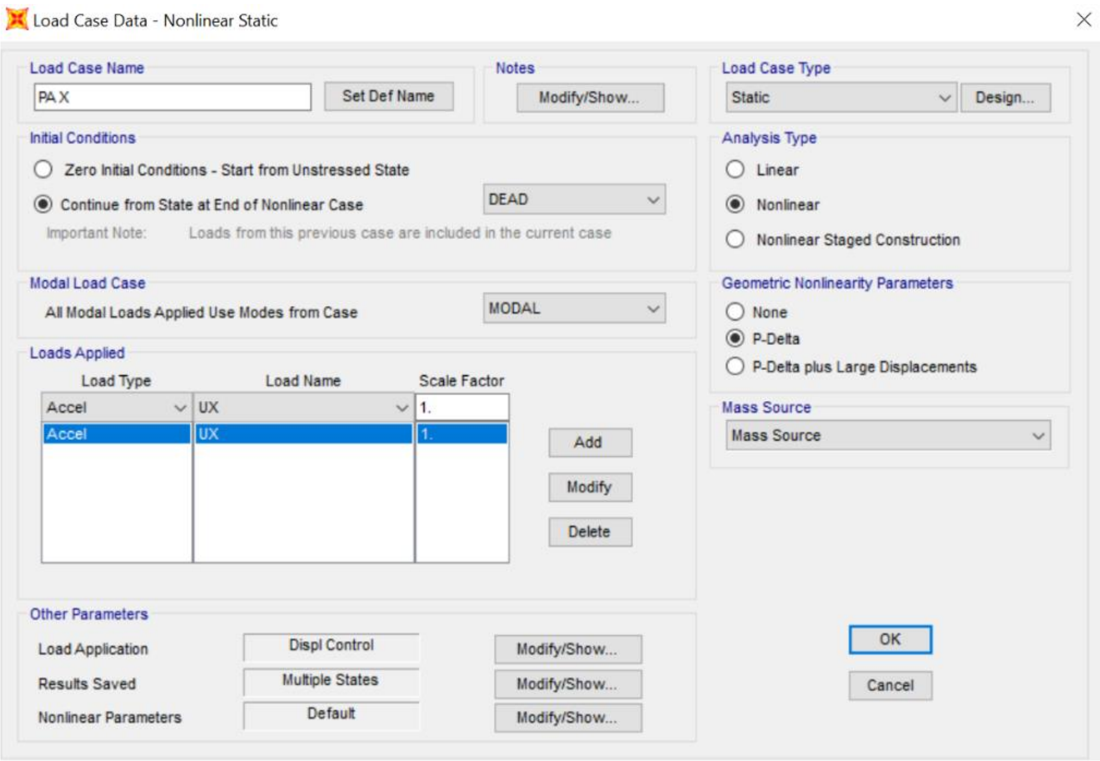


Fig. 5.10 Pushover Load Case Along X Direction

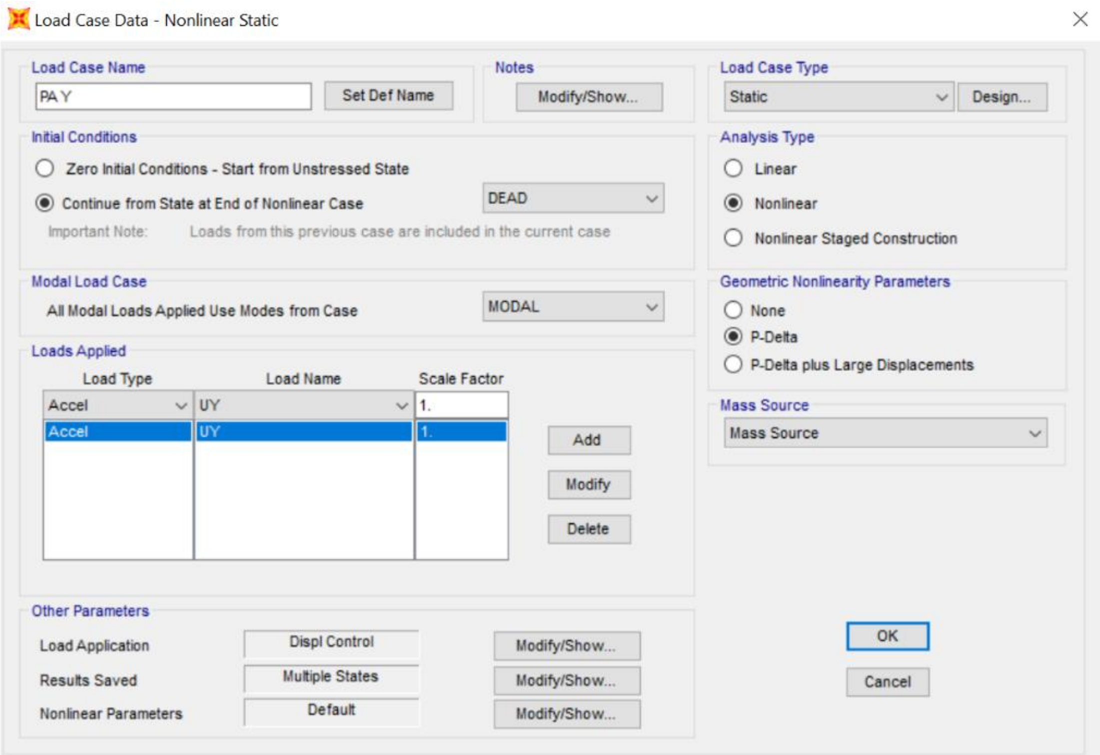


Fig. 5.11 Pushover Load Case Along Y Direction

- Define Hinges to Beam in flexure with a relative distance of 0.05, 0.5 and 0.95 by considering M3 hinges for beams using reinforcement details shown in Annexure 1 computed from Response Spectrum Analysis as shown in fig 5.12 & 5.13.

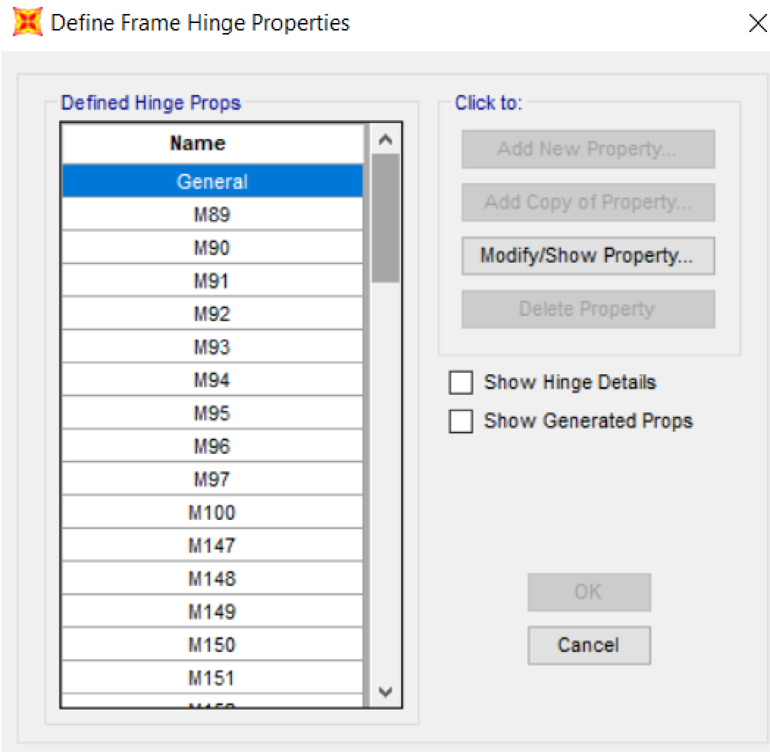


Fig. 5.12 Hinge Definition

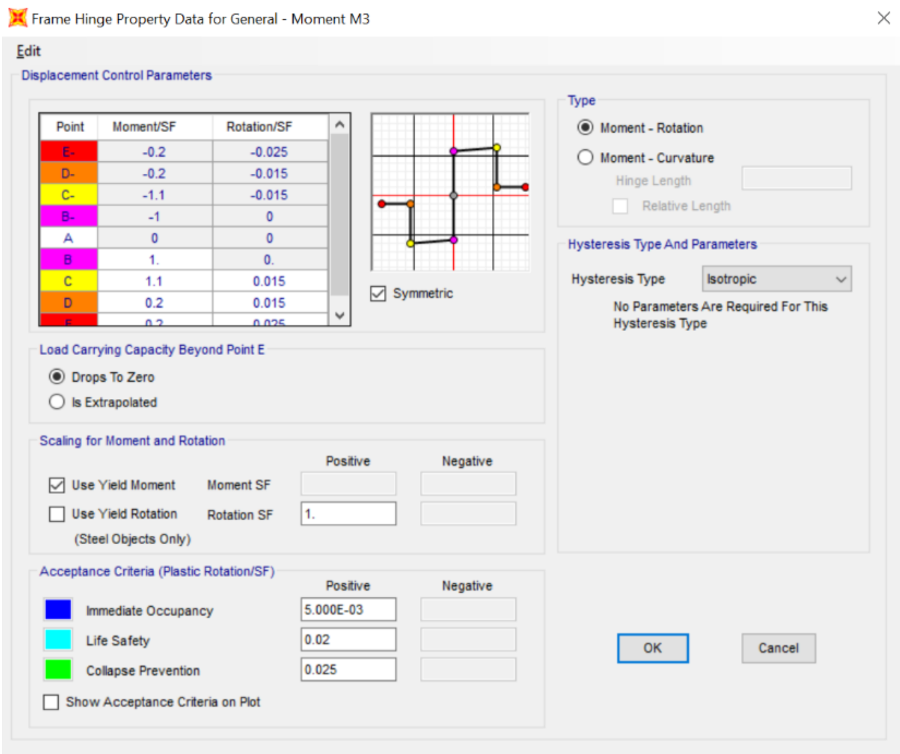


Fig. 5.13 Hinge Property Data

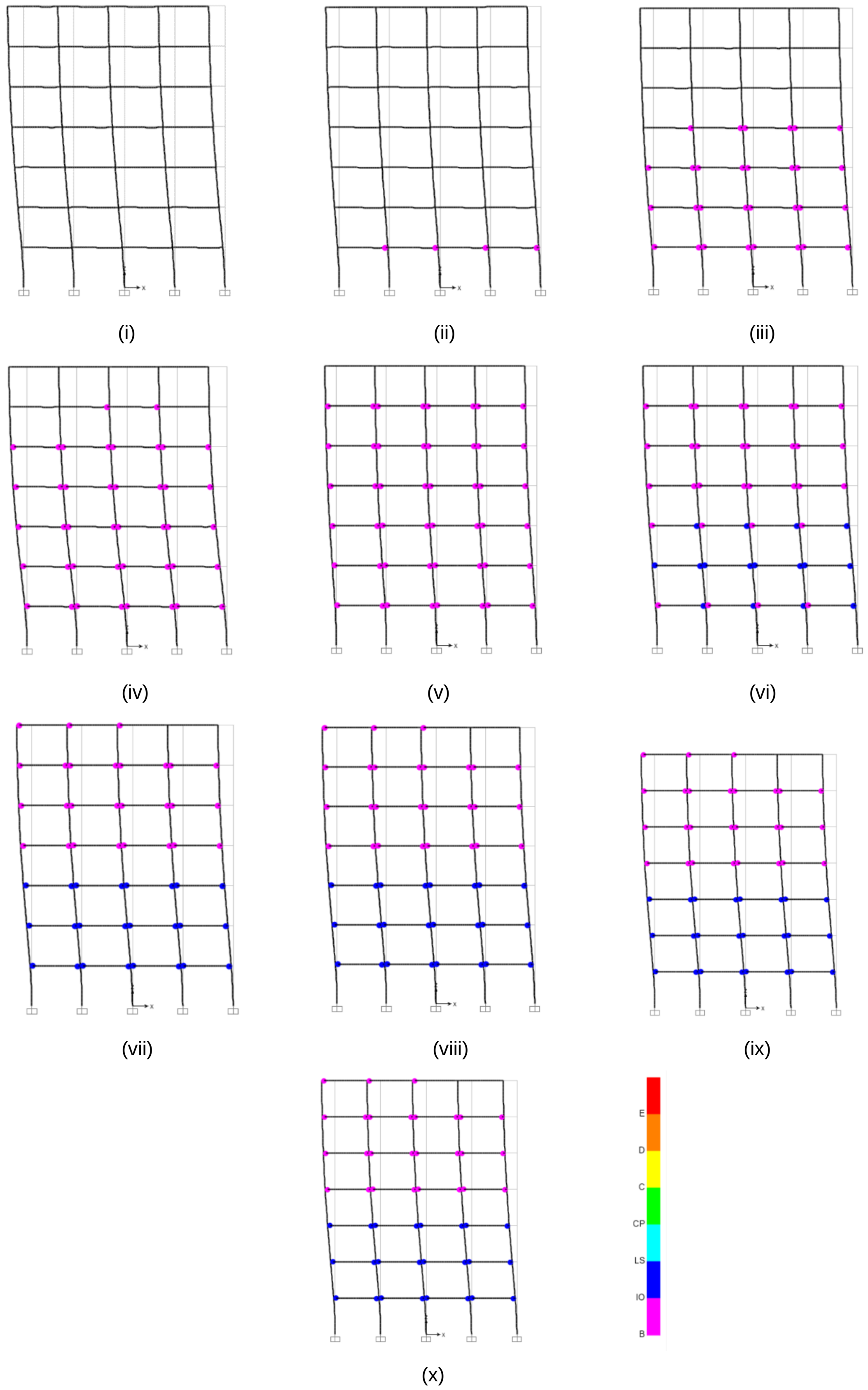


Fig 5.14 Deformed Shape & hinges formed due to Push X

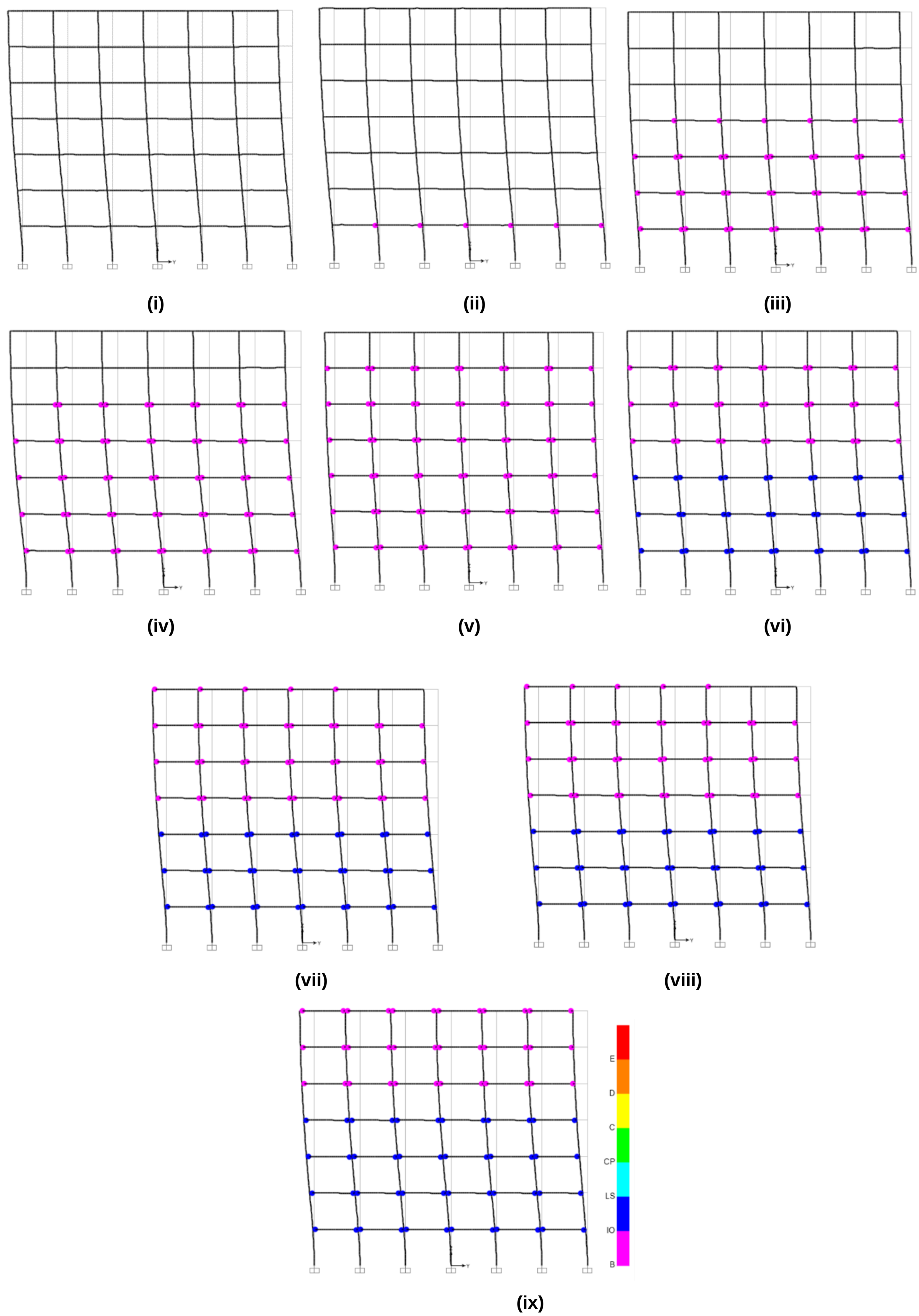


Fig 5.15 Deformed Shape & hinges formed due to Push Y

- Now pushover curve can be seen between base shear and displacement to see how base shear drops as a no. of hinges yield and they reach different stages. Fig. 5.16 and 5.17.
- After assigning hinges to beams select all beams then assign Hinge overwrites to discretize the members to give better results.
- Now run all Nonlinear load cases to observe Structures' behaviour for the defined push displacement. (Linear analysis is used to design the section sizes and reinforcement of the members) to see deformed shape for Push Along X direction & Y direction and then observe which Hinges are formed. Fig. 5.14 & Fig 5.15.

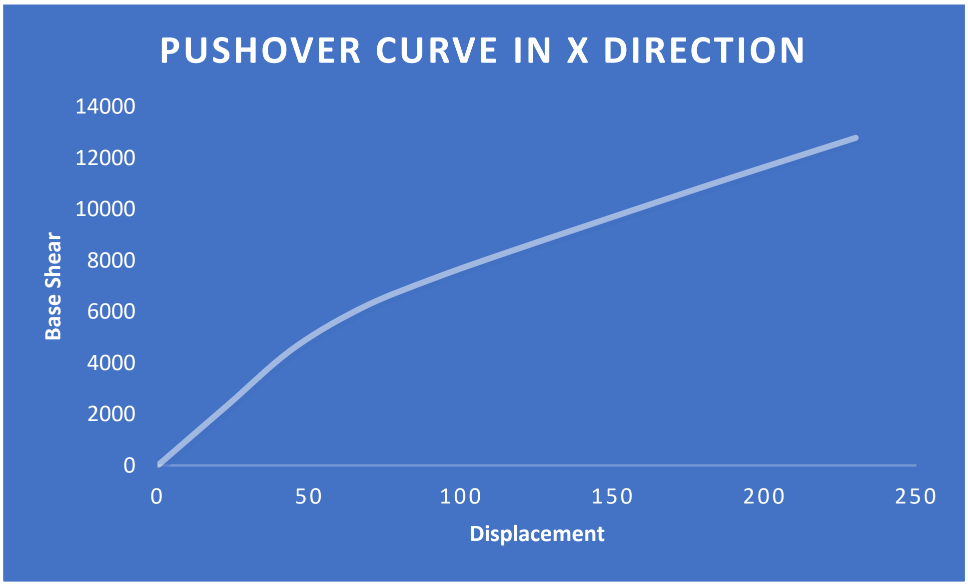


Fig. 5.16 Pushover Curve in X Direction

Table 5.5 Pushover Capacity Curve in X Direction

Pushover Capacity Curve							
LoadCase	**Step**	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**Total**
Text	Unitless	mm	KN	Unitless	Unitless	Unitless	Unitless
PA X	0	6.69E-15	0	1217	0	0	1217
PA X	1	23.000001	2352.705	1217	0	0	1217
PA X	2	44.574487	4559.591	1205	12	0	1217
PA X	3	67.631312	6172.312	1060	157	0	1217
PA X	4	91.618012	7328.531	974	243	0	1217
PA X	5	115.459039	8325.902	916	301	0	1217
PA X	6	142.627355	9407.604	884	296	37	1217
PA X	7	167.229881	10380.034	873	195	149	1217
PA X	8	191.074334	11308.479	853	181	183	1217
PA X	9	221.409181	12460.073	824	170	223	1217
PA X	10	230	12783.646	824	170	223	1217

A total of 1217 hinges yield during pushover analysis along X direction out of which 994 hinges are formed within mmediate Occupancy Performance level and 223 hinges lies between mmediate Occupancy Performance Level & Life Safety Performance Level.

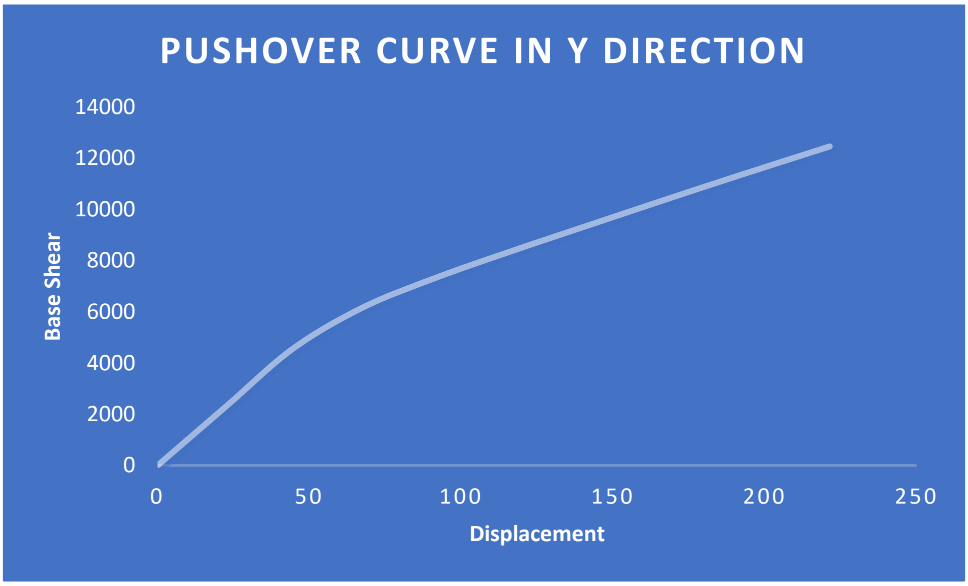


Fig. 5.17 Pushover Curve Y Direction

Table 5.6 Pushover Capacity Curve in Y Direction

Pushover Capacity Curve							
LoadCase	**Step**	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**Total**
Text	Unitless	mm	KN	Unitless	Unitless	Unitless	Unitless
PA Y	0	1.651E-14	0	1217	0	0	1217
PA Y	1	23	2469.405	1217	0	0	1217
PA Y	2	41.73	4479.836	1211	6	0	1217
PA Y	3	-65.900878	6317.762	1037	180	0	1217
PA Y	4	-88.925272	7466.688	947	270	0	1217
PA Y	5	-113.794629	8521.073	872	345	0	1217
PA Y	6	-149.270441	9934.781	857	230	130	1217
PA Y	7	-173.178876	10882.469	841	196	180	1217
PA Y	8	-197.905508	11845.237	814	163	240	1217
PA Y	9	-230	13062.473	797	180	240	1217

A total of 1217 hinges yield during pushover analysis along Y direction out of which 977 hinges are formed within immediate Occupancy Performance level and 240 hinges lies between immediate Occupancy Performance Level & Life Safety Performance Level which lead to Grade 2 Damage as per S 1893-2016.

- The Capacity Spectrum is performed using SAP2000 along X direction & Y direction for various Earthquake Shaking intensity are shown in fig. 5.18 & Fig. 5.19. The Performance point for the given values is obtained by intersection of the Capacity Curve and the Demand Curve.

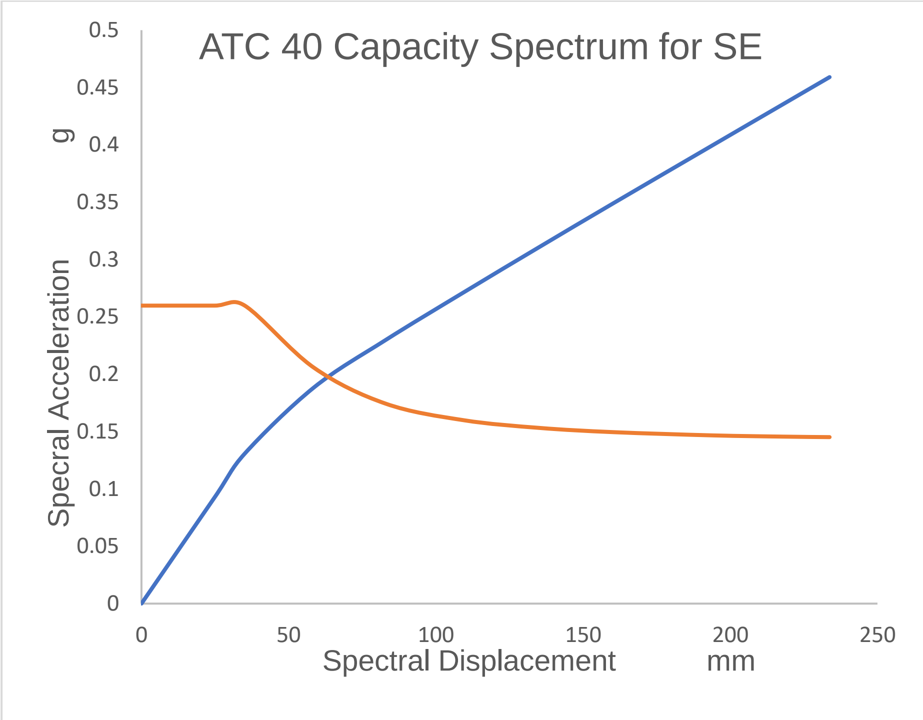


Fig 5.18 Capacity Spectrum in X Direction as Per ATC 40 for Serviceability Earthquake

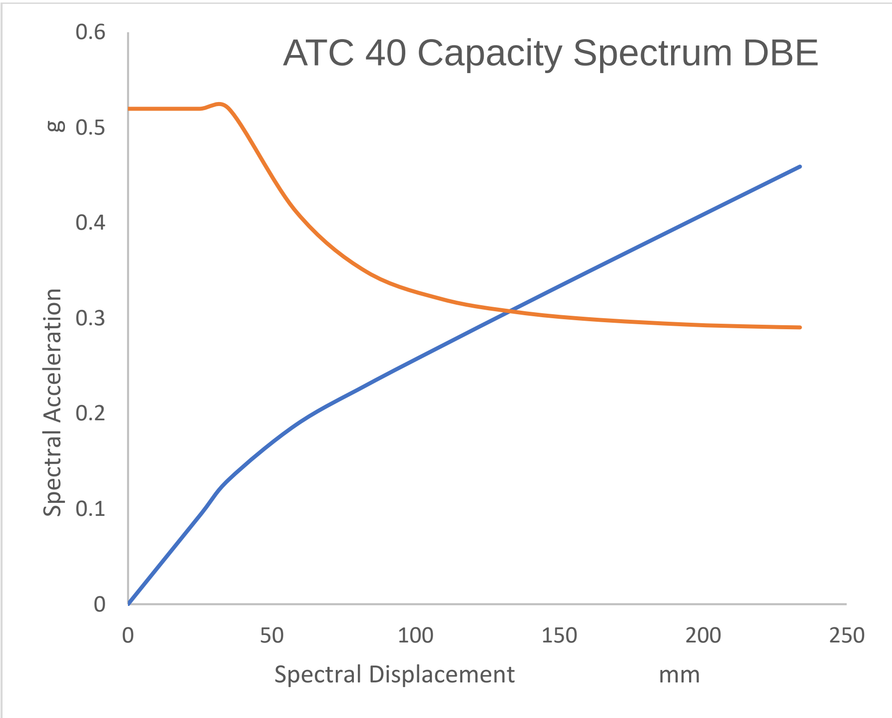


Fig 5.19 Capacity Spectrum in X Direction as Per ATC 40 for Design Earthquake

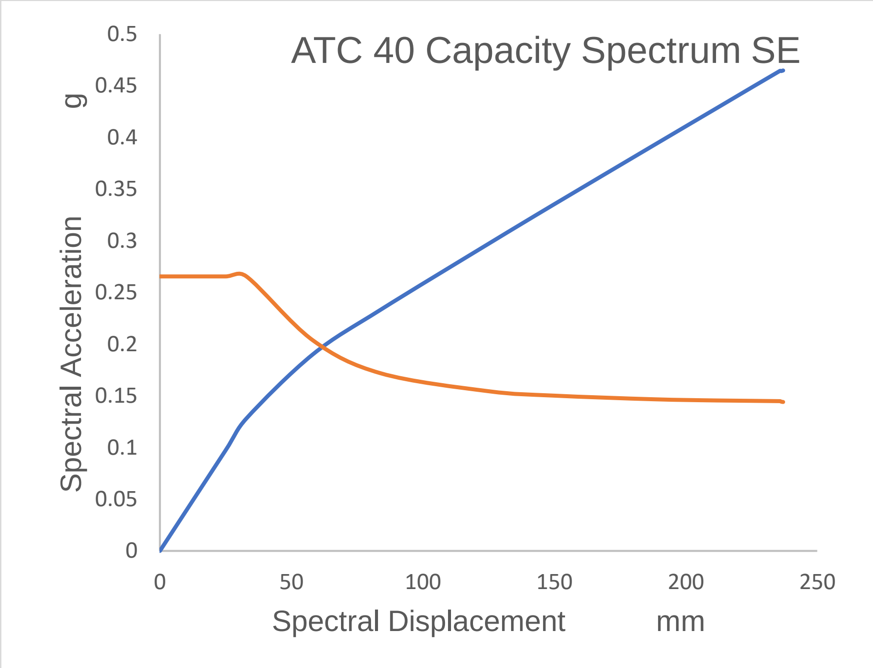


Fig 5.20 Capacity Spectrum in Y Direction as Per ATC 40 for Serviceability Earthquake

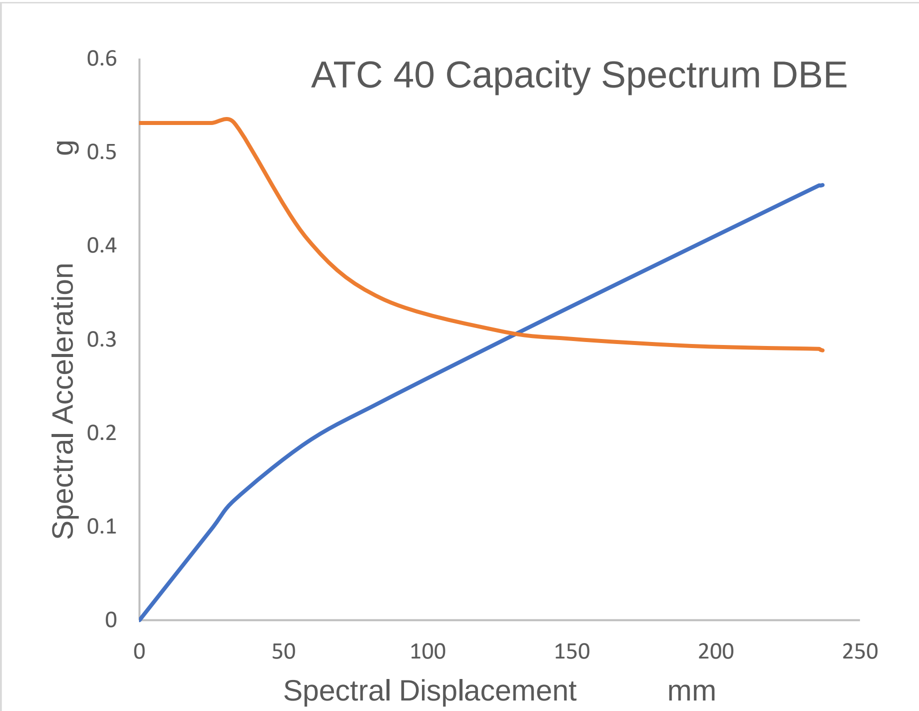


Fig 5.21 Capacity Spectrum in Y Direction as Per ATC 40 for Design Earthquake

The value of performance point as computed from Capacity Spectrum Method in which both the capacity and demand are portrayed in response spectral coordinates. The value of Performance Point as computed from capacity spectrum method for Serviceability Earthquake excitation is Performance Point (Sa-0.196, Sd-0.063) and for Design Basis Earthquake Excitation is Performance Point (Sa-0.307, Sd-0.132).

- Now same structure is again analysed in X-direction and Y-direction and this time monitored displacement of 325mm is used to push the structure to attain collapse in structure. The deformed shape for the push along X direction & Y direction is given in Fig 5.22 & Fig 5.23

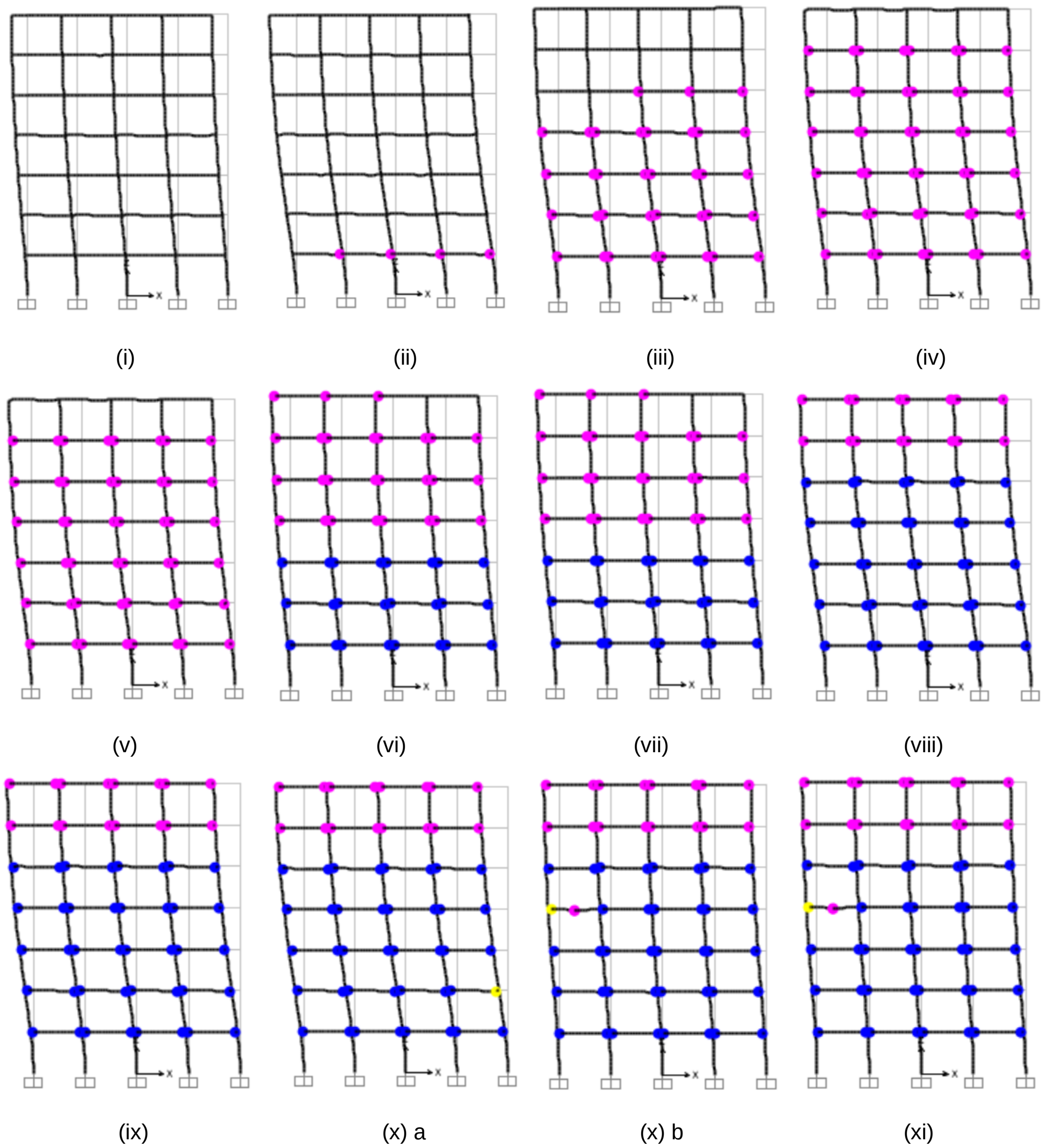
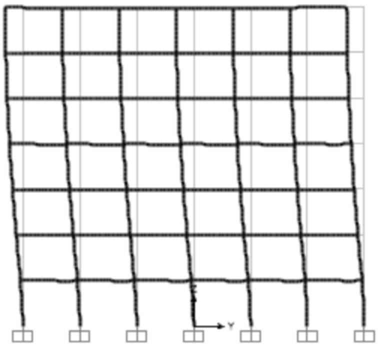
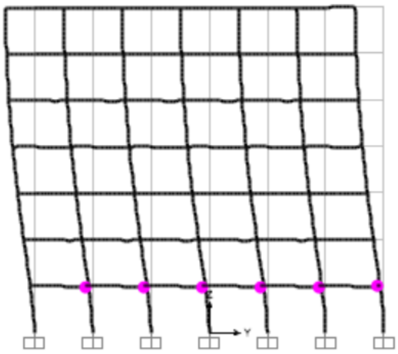


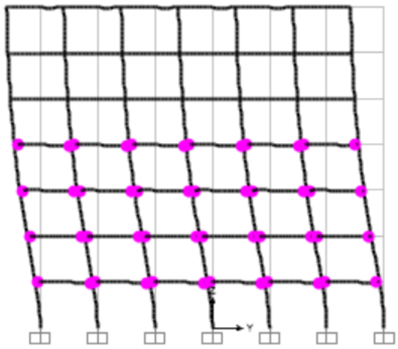
Fig 5.22 Deformed Shape & hinges formed due to Push X



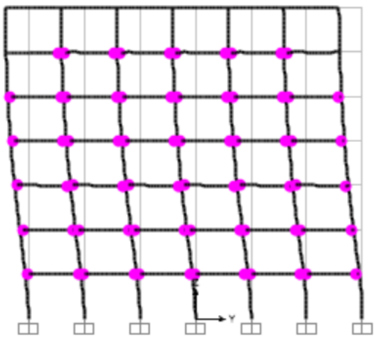
(i)



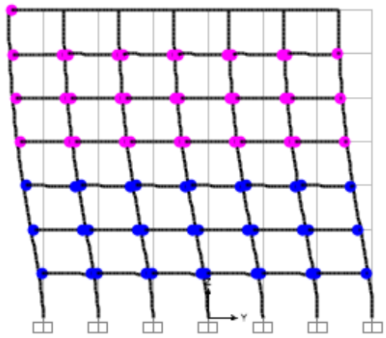
(ii)



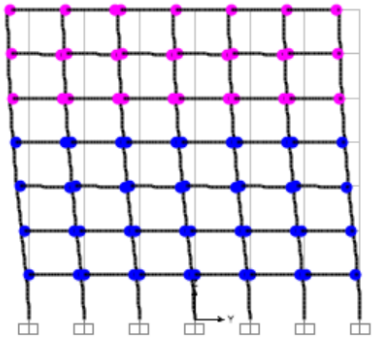
(iii)



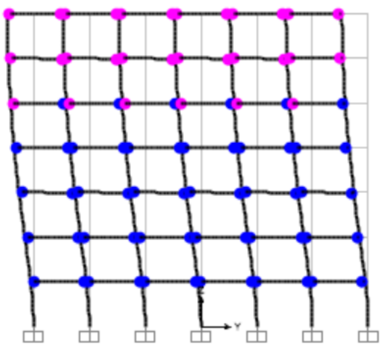
(iv)



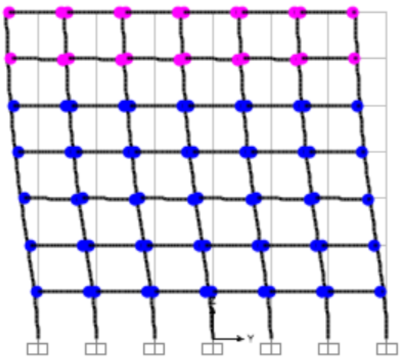
(v)



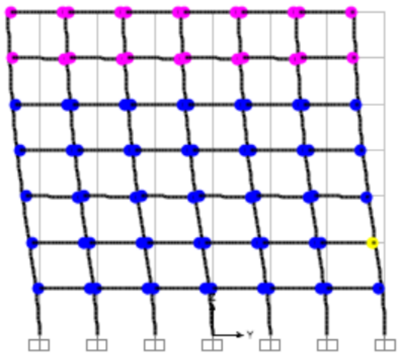
(vi)



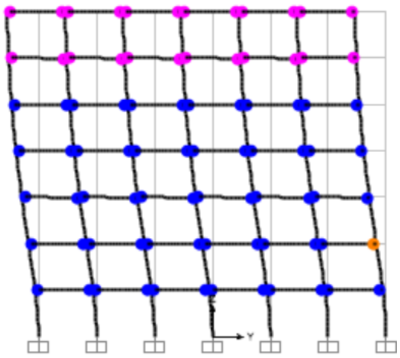
(vii)



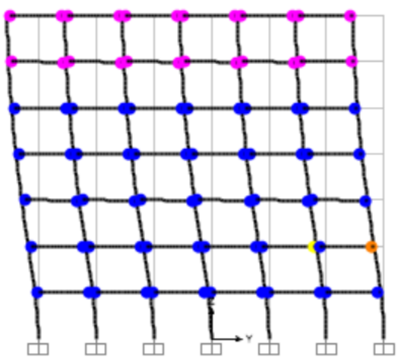
(viii)



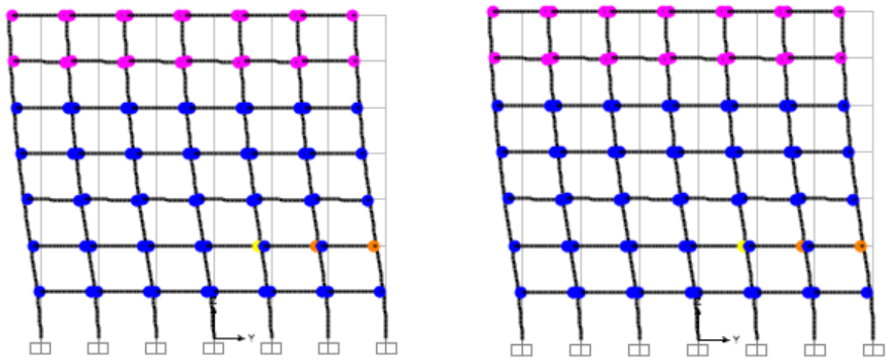
(ix)



(x)



(xi)



(xii) (xiii)

Fig 5.23 Deformed Shape & hinges formed due to Push Y

- Now pushover curve can be generated between base shear and displacement to see how base shear drops as a no. of hinges yield and they reach different stages. Fig. 5.24 and 5.25.

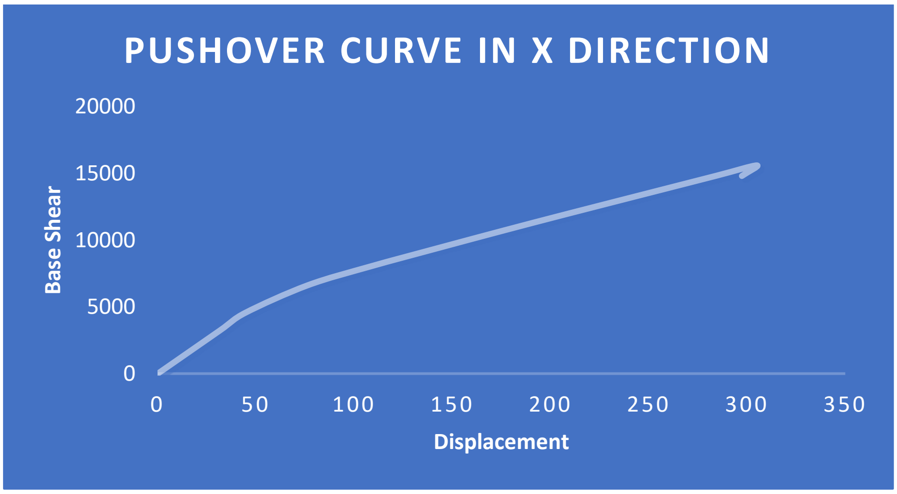


Fig. 5.24 Pushover Curve X Direction

Table 5.7 Pushover Capacity Curve in X Direction

Pushover Capacity Curve										
LoadCase	Step	Displacement	BaseForce	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	Total
		mm	KN							
PA X	0	6.69E-15	0	1217	0	0	0	0	0	1217
PA X	1	31.50	3222.18	1217	0	0	0	0	0	1217
PA X	2	44.58	4559.67	1205	12	0	0	0	0	1217
PA X	3	76.23	6615.76	1025	192	0	0	0	0	1217
PA X	4	108.19	8030.23	931	286	0	0	0	0	1217
PA X	5	142.63	9407.60	884	296	37	0	0	0	1217
PA X	6	175.86	10717.46	862	187	168	0	0	0	1217
PA X	7	208.72	11981.22	825	178	214	0	0	0	1217
PA X	8	252.91	13646.52	824	149	244	0	0	0	1217
PA X	9	284.41	14832.97	824	113	280	0	0	0	1217
PA X	10	304.95	15606.50	824	113	278	0	0	2	1217
PA X	11	297.36	14812.38	824	113	278	0	0	2	1217

A total of 1217 hinges yield during pushover analysis along X direction out of which 937 hinges are formed within immediate Occupancy Performance level, 278 hinges lies between immediate Occupancy Performance Level & Life Safety Performance Level and 2 hinges surpasses collapse Performance Level which lead to Grade 4 Damage as per S 1893-2016.

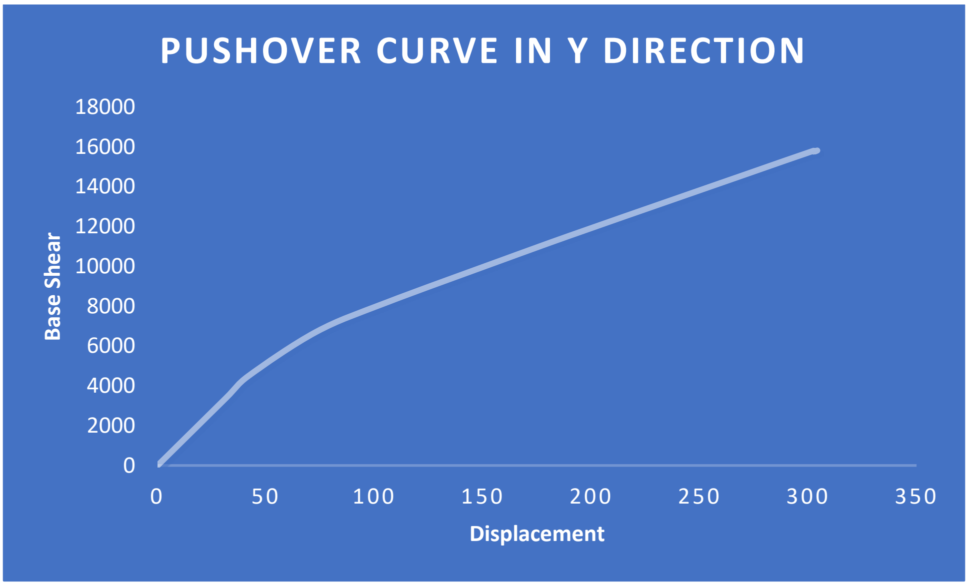


Fig. 5.25 Pushover Curve Y Direction

Table 5.8 Pushover Capacity Curve in Y Direction

Pushover Capacity Curve											
Load Case	**Step**	**Displacement**	**BaseForce**	**AtoB**	**BtoIO**	**IOtoLS**	**LStoCP**	**CPtoC**	**CtoD**	**DtoE**	**Total**
		mm	KN								
PA Y	0	-1.651E-14	0	1217	0	0	0	0	0	0	1217
PA Y	1	-31.50	3382.01	1217	0	0	0	0	0	0	1217
PA Y	2	-41.73	4479.83	1211	6	0	0	0	0	0	1217
PA Y	3	-73.26	6715.73	1002	215	0	0	0	0	0	1217
PA Y	4	-105.41	8177.58	894	323	0	0	0	0	0	1217
PA Y	5	-157.77	10272.69	856	191	170	0	0	0	0	1217
PA Y	6	-190.16	11545.45	828	174	215	0	0	0	0	1217
PA Y	7	-239.45	13419.95	797	174	246	0	0	0	0	1217
PA Y	8	-270.95	14611.57	797	120	300	0	0	0	0	1217
PA Y	9	-302.45	15803.18	797	120	299	0	0	1	0	1217
PA Y	10	-302.46	15762.02	797	120	299	0	0	0	1	1217
PA Y	11	-303.15	15797.57	797	120	298	0	0	1	1	1217
PA Y	12	-303.16	15770.11	797	120	297	0	0	1	2	1217
PA Y	13	-304.23	15817.02	797	120	297	0	0	1	2	1217

A total of 1217 hinges yield during pushover analysis along Y direction out of which 917 hinges are formed within immediate Occupancy Performance level, 297 hinges lies between immediate Occupancy Performance Level & Life Safety Performance Level and 3 hinges surpasses collapse Performance Level which lead to Grade 4 Damage as per S 1893-2016. Hinge details of all hinges which surpasses Collapse point is shown Fig 5.26.

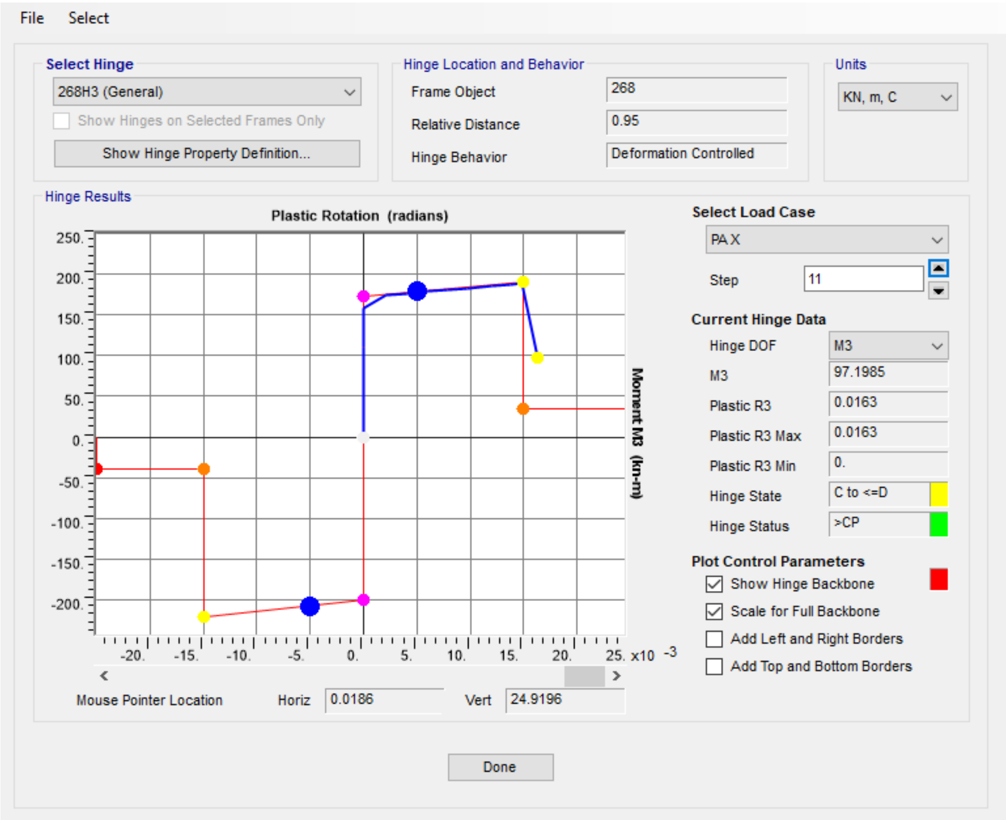


Fig 5.26 (a) Hinge details

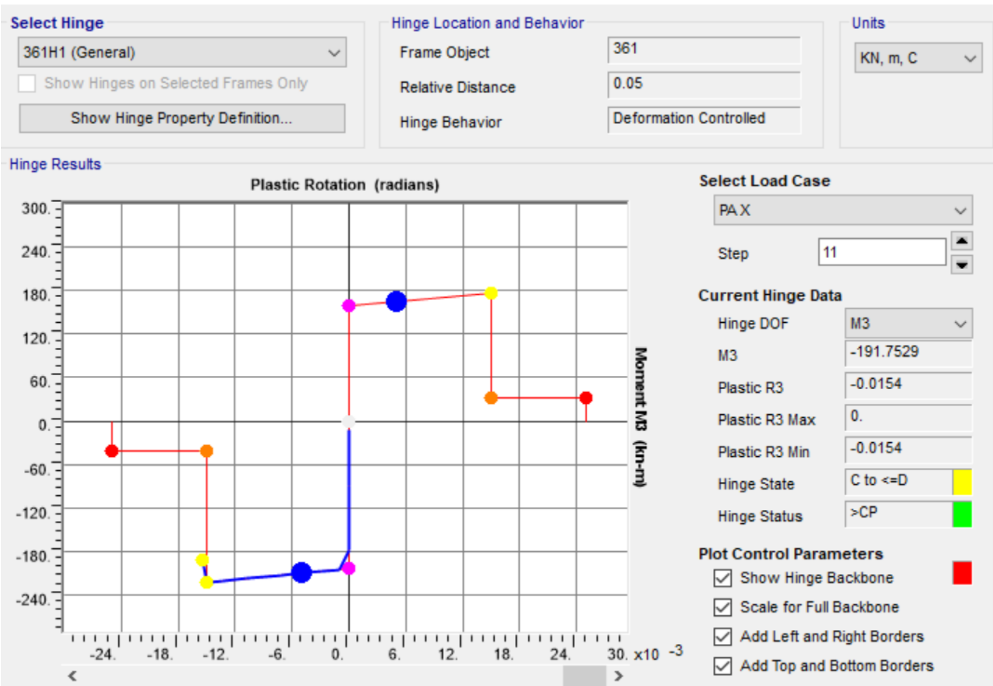


Fig 5.26 (b) Hinge details

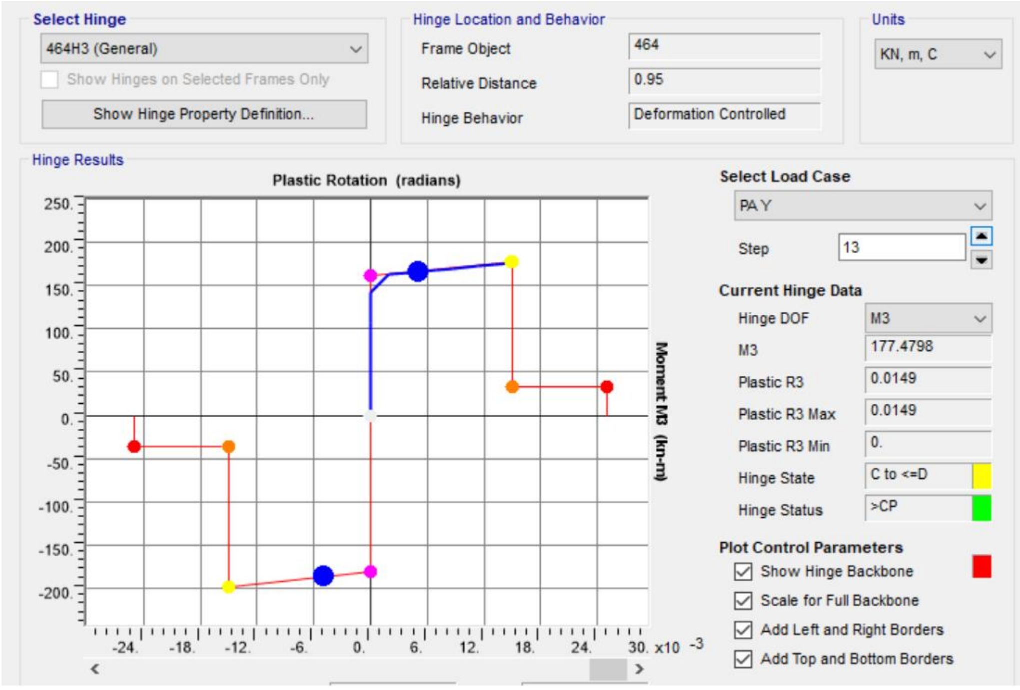


Fig 5.26 (c) Hinge details

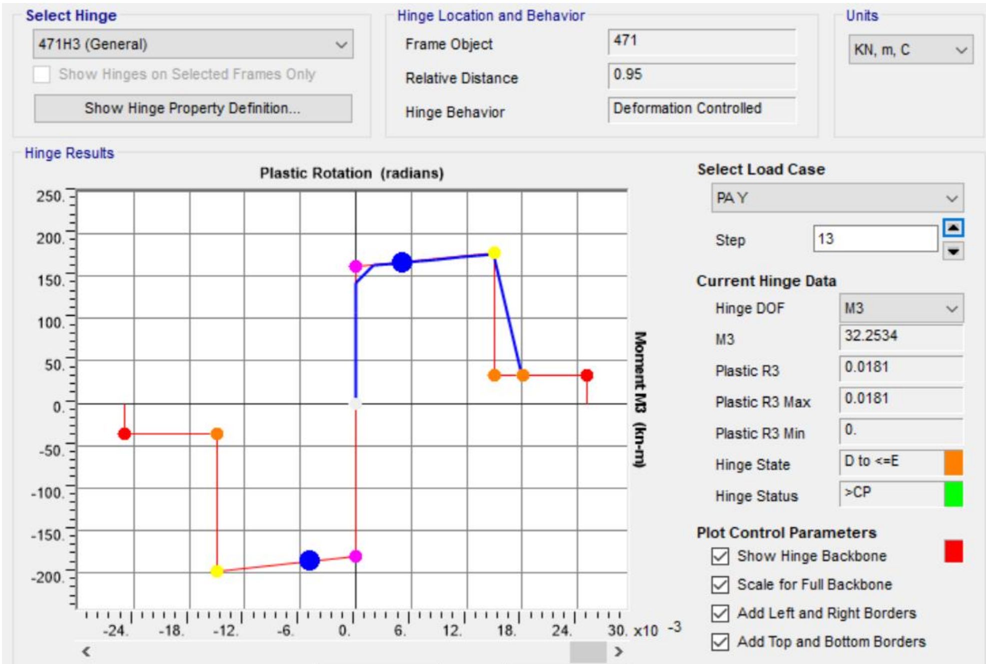


Fig 5.26 (d) Hinge details

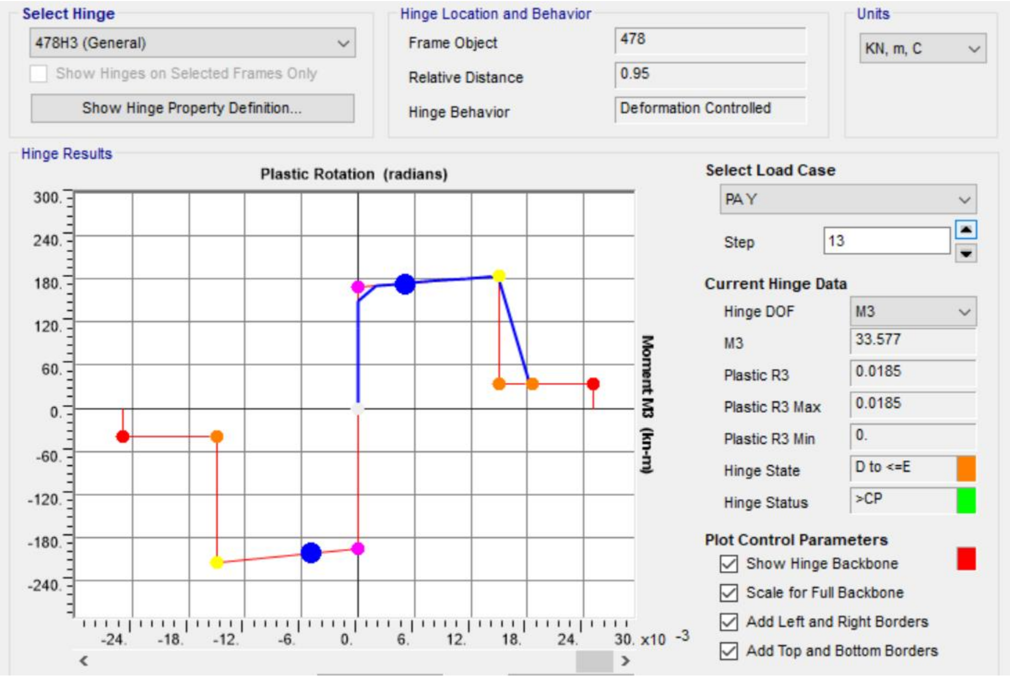


Fig 5.26 (e) Hinge details

- Again, Capacity Spectrum is performed using SAP2000 along X direction & Y direction for various Earthquake Shaking intensity are shown in fig. 5.18, 5.19, 5.20, 5.21, 5.27 & 5.28. The Performance point for the given values is obtained by intersection of the Capacity Curve and the Demand Curve.

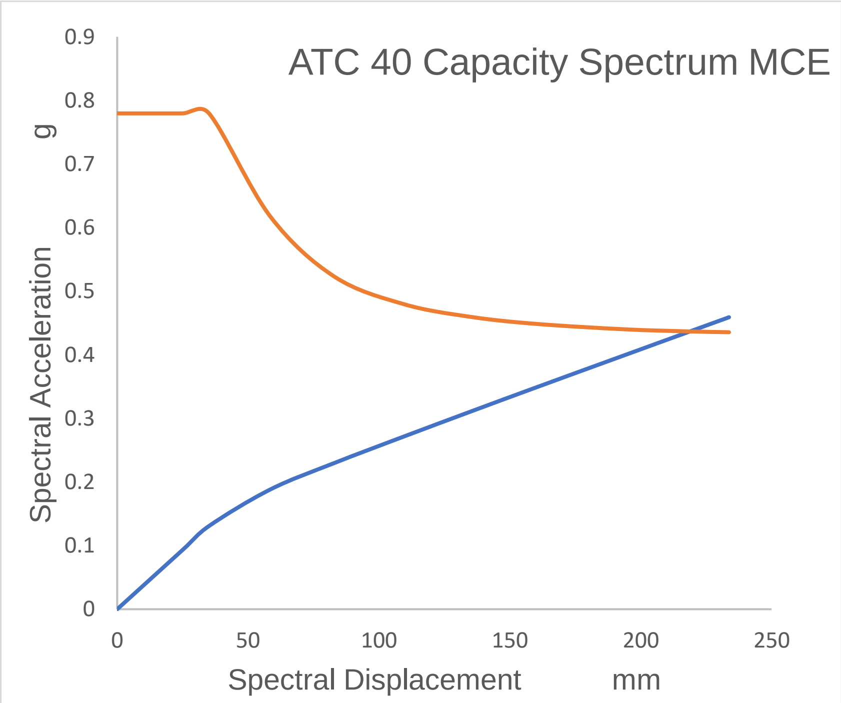


Fig 5.27 Capacity Spectrum in X Direction as Per ATC 40 for Maximum Considered Earthquake

The value of performance point as computed from Capacity Spectrum Method in which both the capacity and demand are portrayed in response spectral coordinates. The value of Performance Point as computed from capacity spectrum method for Serviceability Earthquake excitation is Performance Point (Sa-0.196, Sd-0.063), for Design Basis Earthquake Excitation is Performance Point (Sa-0.307, Sd-0.132) and for Maximum Considered Earthquake is Performance Point (Sa-0.436, Sd-0.217) as shown in Table 5.9.

Table 5.9 Performance Point for Different Shaking intensities

PERFORMANCE POINT						
Shaking intensity	**Performance Point**					
	V	D	Sa	Sd	Teff	Beff
Serviceability Earthquake	6948.234	0.078	0.197	0.062	1.124	0.101
Design Earthquake	10585.7	0.166	0.306	0.131	1.311	0.139
Maximum Considered Earthquake	14867.13	0.278	0.436	0.217	1.415	0.13

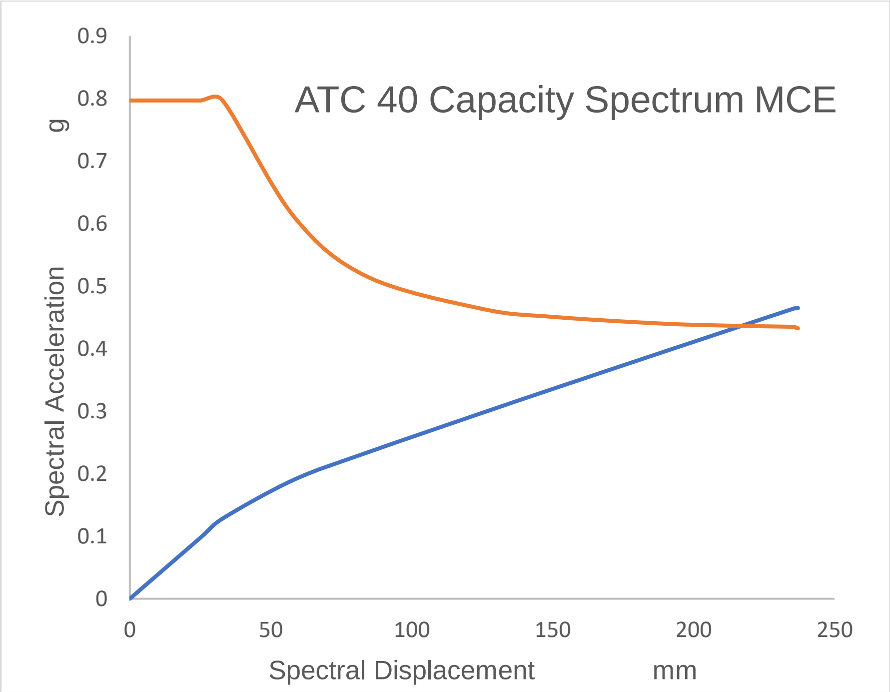


Fig 5.28 Capacity Spectrum in Y Direction as Per ATC 40 for Maximum Considered Earthquake

CHAPTER 6 CONCLUSION

6.1 Introduction

In this work, Performance Based Design of G+6 Storey building has been done by computing the performance with the help of Pushover Analysis. The Design and Analysis is carried out using Computers and Structures Software SAP 2000 using Nonlinear tools.

- Performance of the building is checked with the help of Static Nonlinear Analysis technique known as Pushover Analysis. Performance Based Seismic Design obtained by the above procedure shows a Life Safety Performance achieved when structure is deformed with Target Displacement of 230 mm calculated from FEMA 273 procedure and will reach collapse when this displacement is increased up to 315mm. The design is carried out using S 456:2000 using SAP 2000 for which Response Spectrum Analysis is carried out on the structure to compute the reinforcement details after which hinges are designed using ATC 40 Chapter 9 Modelling Procedures.
- From the given procedure of Pushover Analysis Roof Displacement of 304.95 along X direction and 304.23mm along Y direction is observed when structure is loaded as displacement controlled for 315mm magnitude and Roof Displacement of 230mm along both X & Y direction as shown in Table 6.1.

Table 6.1 Displacement corresponding to given Push along X & Y Direction

TOP STOREY DISPLACEMENT			
S.No	Load Applied	Displacement Along X	Displacement Along Y
1	230	230	230
2	315	304.95	304.23

- From the following Pushover Analysis, a total number of 1217 hinges are formed in X & Y direction out of which 2 hinges in X direction & 3 hinges in Y direction surpasses Collapse Point, rest 937 Hinges along X direction & 917 Hinges along Y direction are under immediate Occupancy Performance Level and 278 Hinges along X direction & 297 Hinges along Y direction under Life Safety performance Level.
- For computing the Performance of the structure under various intensities of Ground Shaking inelastic Response Spectrum is calculated by computing Seismic coefficients Ca which represents effective peak acceleration & Cv which represents average value of peak response of a 5% damped short period system from ATC 40. The Performance of the structure for various earthquake levels are successfully calculated as shown in Table 6.3.

Table 6.2 Hinge status of the hinges Surpassing Collapse Point

Frame Hinge States										
Frame	Output Case	Case Type	Step Type	Assign Hinge	Gen Hinge	Rel Dist	AbsDist	M3	Hinge State	Hinge Status
							m	KN-m		
268	PA X	NonStatic	Max	General	268H3	0.95	4.75	188.74	C to D	>CP
361	PA X	NonStatic	Max	General	361H1	0.05	0.25	-13.083	C to D	>CP
464	PA Y	NonStatic	Max	General	464H3	0.95	4.75	177.48	C to D	>CP
471	PA Y	NonStatic	Max	General	471H3	0.95	4.75	177.39	D to E	>CP
478	PA Y	NonStatic	Max	General	478H3	0.95	4.75	184.53	D to E	>CP

Table 6.3 Performance Point for Different shaking Intensities

PERFORMANCE POINT						
Shaking ntensity	Performance Point					
	V	D	Sa	Sd	Teff	Beff
Serviceability Earthquake	6948.234	0.078	0.197	0.062	1.124	0.101
Design Earthquake	10585.7	0.166	0.306	0.131	1.311	0.139
Maximum Considered Earthquake	14867.13	0.278	0.436	0.217	1.415	0.13

As a closing Remark, one can say that Performance Based Design gives a structure with better seismic load carrying capacity, thereby achieving the desired strength as well as economy and there is still room for some further improvements in the aforementioned Studies.

6.2 Scope for Future Work

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

- In the present Study, Pushover Analysis has been carried out on G+6 Storey building loading with different intensity of loads. This study can further be extended for tall buildings.
- In the present study, the surface on which structure is planned is uniform. The study can be further extended to hilly slopes.
- In the present study, the size of the members is unchanged which can further extended by changing the reinforcement or even the size of the members using Pushover Analysis.
- In the present study, shear walls, bracings and other load bearing members are not provided which can again be very useful to increase the stiffness of the structure in lateral direction.
- In the present study, Base solation techniques is not used which can also be adopted to resist lateral force generated in seismic events.

REFERENCES

1. ATC, 1997, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA 273 Report, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, D.C.
2. ATC, 2006, Next-Generation Performance-Based Seismic Design Guidelines: Program Plan for New and Existing Buildings, FEMA 445, Federal Emergency Management Agency, Washington, D.C.
3. ATC-40, 1996. *Seismic Evaluation and Retrofit of Concrete Buildings*: Vol. 1. Applied Technology Council, Redwood City, California. Report No. SSC 96-01. November 1996.
4. Chopra AK. 1995, Dynamics of structures—theory and applications to earthquake engineering. New Jersey: Prentice-Hall.
5. Fajfar P. 1999. Capacity spectrum method based on inelastic demand spectra. Earthquake Engineering and Structural Dynamics; pages 979–993.
6. Fragiadakis M & Papadrakakis M. Performance Based Optimum Seismic Design of Reinforced Concrete Structures. Earthquake Engineering & Structural Dynamics (2008), Wiley Interscience.
7. Ghobrah A. Performance-based design in Earthquake Engineering: State of Development. Elsevier Engineering Structures 23 (2001) 878-884.
8. Karapetrou S T, Fotopoulou S D, Pitilakis K D. Seismic Vulnerability of RC Buildings Under the Effect of Ageing. Elsevier Procedia Environmental Sciences 38(2017) 461-468.
9. Mander J.B., 2001, Future directions in seismic design and performance-based engineering, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, NZSEE 2001 Conference
10. Nourzadeh D, Humar J & Braimah A. Comparison of Response of Building Structure to Blast Loading and Seismic Excitations. Elsevier Procedia Engineering 210(2017) 320-325.
11. Pang W. & Rosowsky D. Performance-based Seismic Design of Six Story Woodframe Structure. Structural Engineering international 2/2008, 179-185.
12. Priestley, M.J.N. 2000. Performance Based Seismic Design, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 33, No. 3, pp. 325-346.
13. SEAOC, 1995, Vision 2000: Performance-Based Seismic Engineering of Buildings, Structural Engineers Association of California, Sacramento, California.
14. Xue Q. 2001. A direct displacement-based seismic design procedure of inelastic structures. Engineering Structures; 23(11):1453–1460.

15. Xue Q, Chia-Wei Wu, Cheng-Chung Chen, Kuo-Ching Chen, 2007, Civil and Hydraulic Engineering Research Centre, Sinotech Engineering Consultants inc., Taiwan, The draft code for performance-based seismic design of buildings in Taiwan. Engineering Structures 30 (2008) 1535-1547.
16. Zou XK. 2002. Optimal seismic performance-based design of reinforced concrete buildings. Hong Kong (PR China): Hong Kong University of Science and Technology, Engineering Structures 27 (2005) 1289-1302.
17. Zou XK & Chan CM. 2005. An optimal resizing technique for seismic drift design of concrete buildings subjected to response spectrum and time history loadings. Computers & Structures Volume 83, issues 19-20, July 2005, Pages 1689-1704.