PERFORMANCE BASED SEISMIC ANALYSIS OF RC BUILDING

By

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PERFORMANCE BASED SEISMIC ANALYSIS OF RC BUILDING

Major Project

Submitted in partial fulfillment of the requirements for the Degree of

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IN

CIVIL ENGINEERING

(Computer Aided Structural Analysis And Design)

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Declaration

This is to certify that

i) The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Patel Stuti M

Certificate

This is to certify that the Project entitled "Performance based seismic analysis of RC building" submitted by Patel Stuti Mahinkumar (13MCLC11), towards the partial fulfillment of the requirements for the curriculum of Master of Technology in Civil Engineering of Nirma University, Ahmedabad is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project work, to the best of my knowledge, haven't been submitted to any other university or institution for the fulfilment of the academics.

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Abstract

Earthquake induced forces cause loss of life and properties. It is most important to keep in mind that earthquake don't kill people but buildings do. Building design is done by using factor of safety to account for uncertainity in building capacity and Earthquake demand, because of the lack of knowledge on earthquake demand. In seismic design, buildings can be designed by emplyoing shearwalls, braces and MR frames to increase its capacity. In past Bhuj earthquake, some buildings that seems to be strong are also found weak. To avoid damages, existing building can be retrofitted to increase their strength.

Performance based design(PBD) is the modern approach to earthquake resistant design. The thesis is an attempt to understand the basics of performance based design and to predict seismic performance. There are several static as well as dynamic non-linear analysis methods, which are used in PBD. Pushover and time history analysis is done on 2D frame and 4-storey bare frame for the basic understanding of this two methods and the performance of building in future earthquake is obtained. SAP2000 is used for Nonlinear static analysis as well as Nonlinear Dynamic analysis of 2D frame. ETABS is used for Nonlinear static analysis of 4-storey bare frame.

After this, fundamentals of Incremental Dynamic Analysis (Nonlinear Dynamic Analysis) is studied and Incremental Dynamic Analysis will be performed on the 4-storey bare frame. 7 different ground motions will be selected for this analysis and IDA curves will be generated and performance of the building is predicted.

Acknowledgement

It gives me great pleasure in expressing thanks and deep sense of gratitude to **Prof. J. M. Suthar**, Assistant Professor, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad, for helping me throughout the preparation of my thesis. I was fortunate to have such guides who were always there to solve my problems and difficulties during my thesis work. I like to give my special thanks to **Dr. P. V. Patel**, Head, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for his continual kind words of encouragement and motivation throughout the Major Project. I am also thankful to **Dr. K. Kotecha**, Director,Institute of Technology, Nirma University for his kind support in all respect during my study.

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Abbreviations, Notations and Nomenclature

IDA .... Incremental Dynamic Analysis
R.C. .... Reinforced Concrete
CMS .... Conditional Mean Spectrum
UHS .... Uniform Hazard Spectrum
S_a .... Spectral Acceleration
θ_{max} .... Maximum inter-storey drift ratio
Z .... Zone factor
I .... Importance Factor
R .... Response Reduction factor
T .... Time period
M .... Magnitude
r .... Distance from fault rupture
IO .... Immediate occupancy
CP .... Collapse Prevention
GI .... Global Instability
IM .... Intensity Measure
DM .... Damage Measure

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Chapter 1

Introduction

1.1 General

Earthquake shaking is the most severe effect than all other loading effects, e.g., wind loads, wave loads (excluding tsunami loads), blast loads, snow loads, imposed (live) loads and dead loads. Earthquake imposes displacement of the building, which is time varying. This demands lateral deformation of the building between its base and upper elevations. Higher is the seismic zone, larger is the severity of this imposed relative deformation. So the main challenge is to meet the demand of the building that should be able to withstand this imposed deformation with little or moderate damage under small intensity shaking, and with no collapse under high intensity shaking.

The building needs to possess large inelastic deformation capacity and needs to have the strength in all its members to sustain the forces and moments induced in them. The method of design of buildings should therefore take into account the deformation demand on the building and the deformation capacity of the building. While buildings are usually designed for seismic resistance using elastic analysis, most will experience significant inelastic deformations under large earthquakes.

Seismic analysis methods of the structures can be characterized as, Seismic coefficient method and Dynamic Analysis. Seismic coefficient method is an equivalent static analysis considering a design seismic coefficient. The design seismic coefficients include factors such as importance factor, soil-foundation factor, response reduction factor and zone factor. In order to simplify the methods of analysis for determining earthquake effects on structures, codes of practice recommend seismic-coefficient method. Currently two analysis tools are offered with different levels of complexity and required computational effort; nonlinear static analysis (push over) and nonlinear dynamic analysis (time-history). Nonlinear analysis procedures are important in identifying the patterns and levels of damage for a structures inelastic behavior and to understand the failure modes of the structure during severe seismic events. Performance based seismic design is modern approach to earthquake resistance design.

1.2 Performance based design

1.2.1 Background

Performance based design is largely used this days. Airplanes, automobiles and turbines have been designed using this approach for many decades. Modern performance-based design methods require number of ways to determine the realistic behavior of structures under such inelastic conditions. The basic concept of PBD is to provide the capability to design buildings that have a predictable and reliable performance in earthquakes. Performance-based earthquake engineering (PBEE) implies design, evaluation, and construction of engineered structures whose seismic performance meets the diverse economic and safety needs of owners and society. The Pacific Earthquake Engineering Research (PEER) Center has set the objective to develop procedures, knowledge, and tools that will provide the foundation on which to base the implementation of PBEE in engineering practice[d].

Performance-based seismic design is a process that permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of life, occupancy and economic loss that may occur as a result of future earthquakes.

1.2.2 Performance objectives

A performance objective has two main parts a damage state and a level of seismic hazard. Seismic performance is described by designating the maximum allowable damage state (performance level) for an identified seismic hazard (earthquake ground motion). A performance objective may include consideration of damage states for several levels of ground motion and would then be termed a dual or multiple-level performance objective. The methodology is performance based: the evaluation and retrofit design criteria are expressed as performance objectives, which define desired levels of seismic performance when the building is subjected to specified levels of seismic ground motion. Acceptable performance is measured by the level of structural and/or nonstructural damage expected from the earthquake shaking [a].

The target performance objective is divided into Structural Performance Level (SP-n, where n is the number) and Non-structural Performance Level (NP-n, where n is a letter). These may be specified independently. But the combination of the two determines the overall Building Performance leve[a]l.

Structural performance level:

a. Immediate occupancy (SP-1) : Structural damage is limited with the basic vertical and lateral force resisting system retaining most of their pre-earthquake characteristics and capacities.

b. Damage control (SP-2) : State of damage is between IO and LS.

c. Life safety (SP-3) : Significant damage with some margin against total or partial collapse. Injuries may occur with the risk of life-threatening injury being low. Repair may not be economically feasible.

d. Limited safety (SP-4) : Sate of damage lies between Life safety and Structural stability.

e. Structural stability (SP-5) : Structural damage in which structural system is on the verge of experiencing partial or total collapse.

f. Not considered (SP-6) : where only non-structural seismic evaluation or retrofit is performed.

Non-structural performance level:

a. Operational (NP-A) : Non-structural elements are generally in place and functional. Back-up systems for failure of external utilities, communications and transportation have been provided.

b. Immediate Occupancy (NP-B): Nonstructural elements are generally in place but may not be functional. No back-up systems for failure of external utilities are provided.

c. Life safety (NP-C) : Considerable damage to non structural elements and systems but no collapse of heavy items. Secondary hazards such as breaks in high-pressure, toxic and fire suppression piping should not be present.

d. Reduced hazard (NP-D) : Large damage to non-structural components but should not include collapse of large and heavy items that can cause significant injury to groups of people.

e. Not considered (NP-E) : Non-structural elements, other than those that have an effect on structural response, are not evaluated.

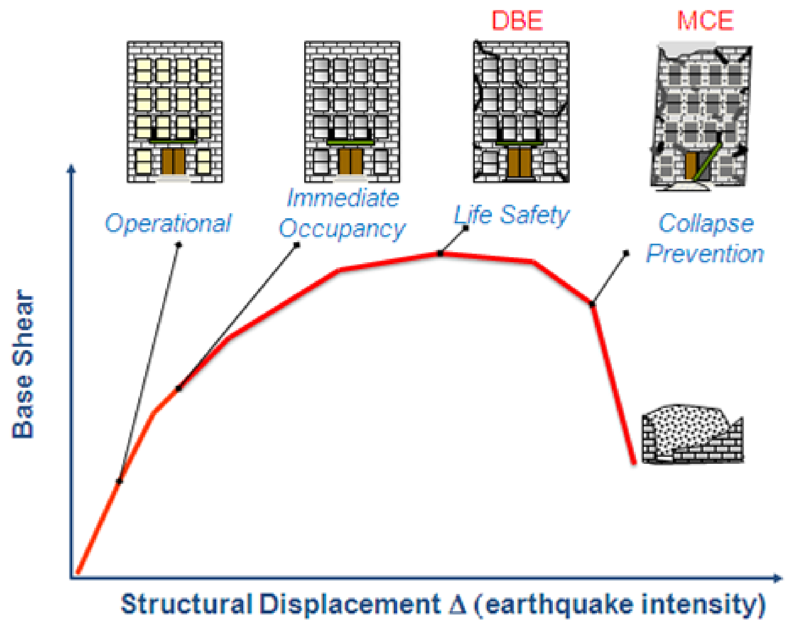

Figure 1.1: Standard performance level[d]

Building performance is obtained by considering the combination of structural and non-structural components.

1.2.3 Analytical procedures

There are four procedures that are used for evaluation of seismic performance of Structures:

- Linear static procedure

- Linear dynamic procedure
- Nonlinear static procedure
- Nonlinear dynamic procedure

Because the actual response of building to earthquake is not typically linear, the use of linear procedures is limited. If there are irregularities that occur in the building, prohibit linear procedures. These irregularities are the following:

- In-plane discontinuities
- Out-of-plane discontinuities
- Weak stories irregularities
- Torsional strength irregularities

Hence, nonlinear procedures are used for this irregular structures. In order to achieve the Basic Safety Objective, the building under analysis should achieve a Life Safety (LS) performance level and a Collapse Prevention (CP) performance level.

Nonlinear static procedure

It is commonly referred as a pushover analysis, which directly incorporates the nonlinear response of members in the structure. A model of the structure that incorporates the nonlinear load-deformation characteristics of individual components of a building is loaded with monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. This includes:

- Capacity Spectrum Method
- Displacement co-efficient Method
- Secant Method

Nonlinear dynamic procedure

The nonlinear dynamic procedure generally uses a similar computer model as the nonlinear static procedure. However, the loading and computational methods differ. Following methods are used in this procedure:

- Time History Analysis
- Incremental Dynamic Analysis
- N-2 method

The response of the structure is generated using time history analysis. This methods of analysis is highly sensitive to characteristics of individual ground motions and the assumptions made in computer modeling.

Following guidelines are used for PBD:

a. ATC 40

b. FEMA 273

c. FEMA 356

d. FEMA 440

e. FEMA 350

f. FEMA 351

1.2.4 Objective of work

Performance based analysis is essential for the building to understand its behavior and response during earthquake. It also helps in understanding collapse mechanism in case of extensive damage. Software like ETABS/SAP2000 are used for performance based analysis of building. The main objectives are as follows:

- Through Incremental Dynamic analysis (IDA), structural response under seismic loads will be studied.
- To study the overall behavior of structures, from their elastic response through yielding and nonlinear response and all the way to global dynamic instability.
- To understand procedure of Incremental Dynamic Analysis in PBD.

1.2.5 Scope of work

- Selection of appropriate layout of 4-storey.
- Selection of ground motions required for IDA analysis.
- Obtaining target ground motion properties of each Intensity Measures (IM).
- Developing IDA curves of the structural response, which is measured by Engineering Demand Parameter (EDP) versus ground motion intensity level.
- Limit states (Immediate Occupancy or Collapse prevention) will be defined on each IDA curve and corresponding capacities will be calculated.
- Using IDA data to understand the behaviour of the structure.

1.2.6 Organization of report

Chapter 2 includes Literature review. In this chapter, literatures regarding to pushover analysis and Incremental dynamic analysis(IDA) and its implement to the structures is discussed.
Chapter 3 includes some examples based on pushover and time history analysis. Their result are discussed in this chapter.
Chapter 4 discusses some points regarding to IDA and gives basic knowledge of IDA approach in performance based design
Chapter 5 contains postprocessing of IDA analysis.

Chapter 6 shows conclusion and future scope of work.

Chapter 2

Literature review

2.1 General

Literature survey is essential to review the work done in the area of performance based engineering. To take up the specific need to perform the analysis, the literature like technical papers, journals and books need to be referred. The prime important in the review was to understand the analysis and different concept of performance based engineering.

The publication of **Applied Technology Council (ATC 40)** [**1**] provides guidelines for performance based design of building. It includes objectives, retrofit strategies, linear as well as nonlinear analysis procedure, modeling rules, foundation effect on performance design and descriptive limits of expected performance. It also includes example of inelastic analysis of building.

The publication of **FEMA 273** [**2**] provides technical guidelines for the seismic rehabilitation of building. This document covers modeling and procedure of linear static analysis, linear dynamic analysis, nonlinear static analysis and nonlinear dynamic analysis. This document includes the performance based design for retrofit of an existing structure. This method can also be applicable for the new design. It also gives information regarding to foundations and geotechnical aspects.

2.2 Performance based design

Ahmed Ghobarah [3] discussed in his article about the need of performance based seismic design, methodology, issues of ground motion modeling, and demand and capacity evaluations. He also discussed the challenges to be addressed before the performance based design can be widely accepted. Some challenges that are mentioned : probabilistic characterization of capacity and performance; development of general design procedures for multi-performance and hazard levels; and analysis and modeling of the inelastic behavior of structures for the realistic determination of transient and residual deformations.

2.3 Analysis Method

M J N Priestley[5] has given two more methods which are N2 method and direct displacement-based design. In N2 method, an estimate of seismic displacement demand is found by response spectrum analysis of a single degree of freedom bilinear mode, representing the first elastic mode of the structure. This displacement demand is compared with the results of pushover analysis of a multi degree of freedom representation of the structure. The displacement-based design is the forced based approach with the addition of a displacement check to ensure that the acceptable performance is achieved in the design earthquake.

2.4 Nonlinear analysis

NIST GCR 14-917-27 publication by NEHRU [6] provides nonlinear analysis research and development program for performance based seismic engineering. It reviews the relationship between nonlinear analysis and seismic design, including the motivation and goals for performing nonlinear analysis. It provides a series of initiatives related to verification, validation, and calibration procedures to promote the development and implementation of more accurate nonlinear analyses and software codes. It also describes initiatives related to modeling capabilities that are intended to improve understanding of nonlinear behavior, improve mathematical modeling of materials and components, and inform explicit consideration of uncertainty. It provides initiatives for developing analysis guidelines and example applications to facilitate the use of nonlinear analysis in design practice, and for developing a framework for acceptance criteria for use with nonlinear dynamic analysis.

Rahul Rana, Limin Jin and Atila Zekioglu [7] performed pushover analysis of nineteen storey slender concrete building located in San Francisco with a gross area of 430,000 square feet. Building consists shear wall as lateral load resisting system. The building was designed as per 1997 Uniform Building Code, and pushover analysis was performed to verify code's underlying intent of Life Safety performance under design earthquake. Procedure for analysis and to obtain results was presented. How to obtain location of moment and shear hinges was presented.

Farzed Naem[4] considers Capacity Spectrum Method, a most popular method. The method is also known as Nonlinear Static Procedure, Nonlinear Pushover Analysis or simply Pushover analysis method. In a technical literature, Farzed Naem has described pushover analysis techniques in various points:

- Push-over analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., inverted triangular or uniform).
- The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various structural components is recorded.
- Push-over analysis can provide a significant insight into the weak links in

seismic performance of a structure.

- A series of iterations are usually required during which, the structural deficiencies observed in one iteration, are rectified and followed by another.

- This iterative analysis and design process continues until the design satisfies preestablished performance criteria.

- The performance criterion for push-over analysis is generally established as the desired state of the building given roof-top or spectral displacement amplitude.

Roberto Villaverde[8] gives comprehensive review of the analytical methods that are currently available to assess the capacity of building structures to resist an earthquake collapse, point out the limitations of these methods, describe past experimental work in which specimens are tested to collapse, and identify what is required for an accurate evaluation of the seismic collapse capacity of a structure and the safety margin against such a collapse. They also described basics of pushover analysis and Incremental dynamic analysis.

2.5 Incremental Dynamic Analysis

Vamvatsikos and Cornell[14] considered 9-storey steel moment resisting frame for IDA. The model incorporates ductile members, shear panels, and realistically fracturing reduced beam section connections, while it includes the influence of interior gravity columns and a first-order treatment of global geometric nonlinearities P-Delta effect. It was a first-mode-dominated structure that has its fundamental mode at a period of T1=2.3 s, accounting for 84.3% of the total mass, hence allowing for some significant sensitivity to higher modes. They took 20 ground motion recordss to represent scenario earthquake with the moment magnitude is within the range of 6.5-6.9 and all have been recorded on firm soil. They performed dynamic analysis and by interpolating between IM and EDP results, they plotted IDA curve on EDP-IM axes. Then they present 20 IDA curves into 16, 50 and 84% fractile curve as shown in fig 2.1. Limit states are then defined on the IDA curves. Immediate occupancy (IO) is violated when the building exceeds thetamax=2 percent and Collapse Prevention (CP) is reached when the local slope on the IDA curve is 20% of the elastic slope or $\theta max = 10\%$, whichever occurs first. Finally, the global dynamic instability (GI) was evident by the characteristic flattening on each IDA, termed the flatline, where the structure responds with practically infinite EDP to any IM increase. Such limit states were summarized into their 16, 50, and 84% values as shown in fig.2.1

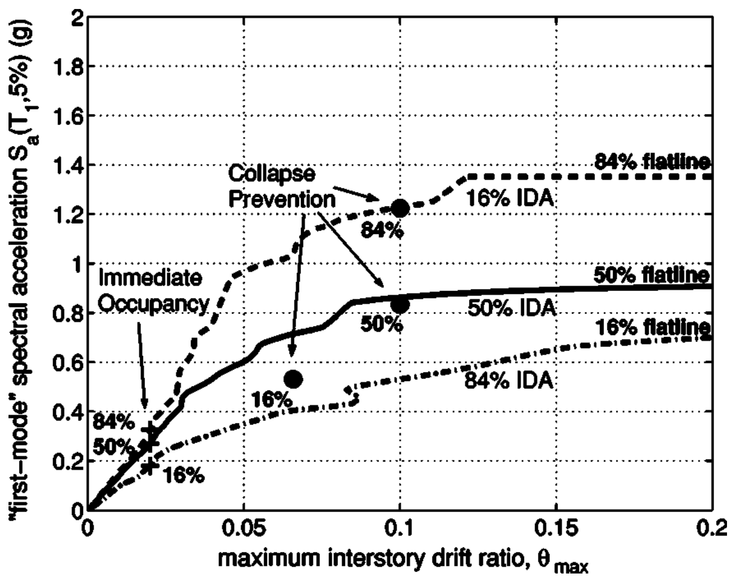

Figure 2.1: 16%, 50%, 84% fractile incremental dynamic analysis IDA! curves and limit-state capacities[14]

H.R. Vejdani Noghreiyan and A. Shooshtari[10] gives a comparative study to evaluate the reliability of MPA-based approximate method for regular and irregular RC plane frames. In the MPA-based IDA procedure, the MPA procedure is used to estimate the nonlinear response of the structure due to each ground motion intensity level.

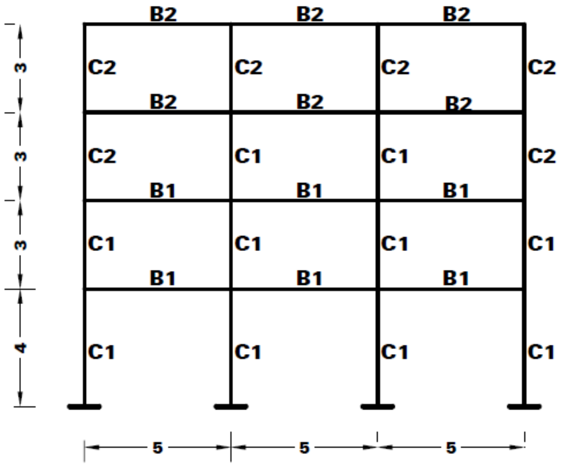

Figure 2.2: 4-storey regular frame[10]

Four and eight storey regular and irragular frames are considered and analysed them by both IDA and MPA based IDA method. Comparision of both the method are as shown in fig 2.2 and fig 2.4. This paper concludes that MPA-based IDA procedure is satisfactory for structural demands with a huge reduction in computational effort for regular buildings.

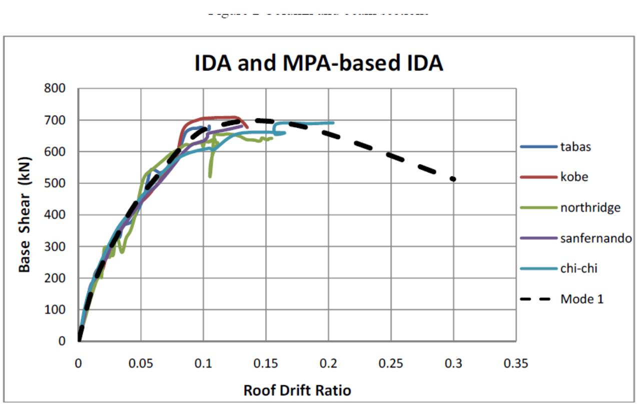

Figure 2.3: Comparison of IDA and MPA-based IDA[10[

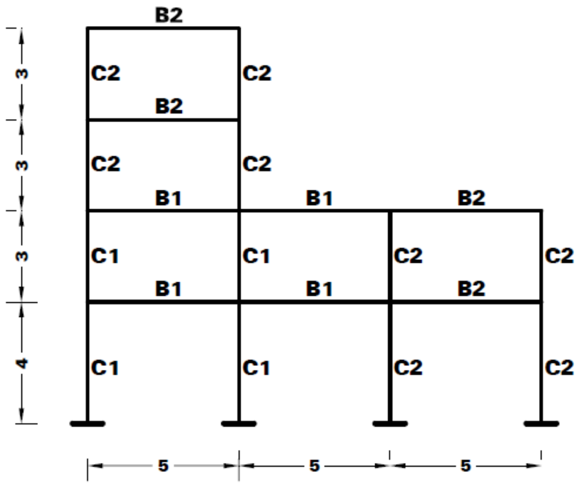

Figure 2.4: 4-storey irregular frame[10]

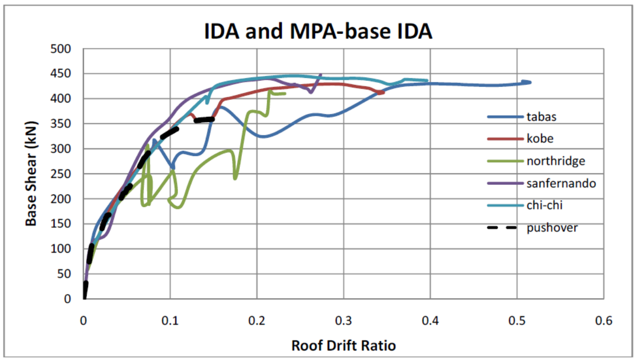

Figure 2.5: Comparison of IDA and MPA-based IDA[10]

Ting Lin and Jack W. Baker[11] considered challanges in changing records across IM levels. Different ground motion selection schemes were considered and compared with AIDA (Adaptive incremental dynamic analysis) to demonstrate the advantage of using AIDA. Benifits of IDA and PSHA (Probabilistic Seismic Hazard Analysis) without omitting data are used for this new AIDA approach. They concluded that AIDA is a promising new tool for linking ground motion selection and structural response assessment.

2.6 Summary

In this chapter, review of relevant literature is carried out. The review of literature includes basics of pushover analysis. It also includes the development of IDA curves and their implementation to the structure.

Chapter 3

Pushover and Time history Analysis

3.1 General

In this chapter, Pushover and time history analysis of 2D frame using SAP and pushover analysis of 4 storey bare frame is done using ETABS.

3.2 Pushover Analysis

As the building undergoes upto the plastic zone, the nonlinear static analysis should be carried out to understand its behavior. Pushover or capacity based analysis is more popular as a static nonlinear analysis. Two types of pushover analysis are as:

a. **Force controlled** Used when load is known and structure is desired to support this load. For gravity load on structure force controlled, push over analysis is used.

b. **Displacement controlled** Used when load is unknown but displacement is known and structure is desiredto lose their strength and become unstable. For lateral load on structure displacement controlled, pushover analysis is used.

Three main steps involved in this analysis procedure:

- Evaluation of Capacity of building i.e. Representation of the structures ability to resist a force
- Evaluation of Demand curve i.e. Representation of earthquake ground motion
- Determination of Performance point i.e. Intersection point of demand curve and capacity curve

3.2.1 Pushover analysis of 2D frame

A problem of 2D frame was taken for static nonlinear analysis. The beams and columns were of 230X230 mm. Supports were assumed to be fixed.

- **Nonlinear hinges**

  As only lateral load was applied the shear and axial forces were overruled, so only moment hinge was considered. Hinge properties are shown in fig.3.1.

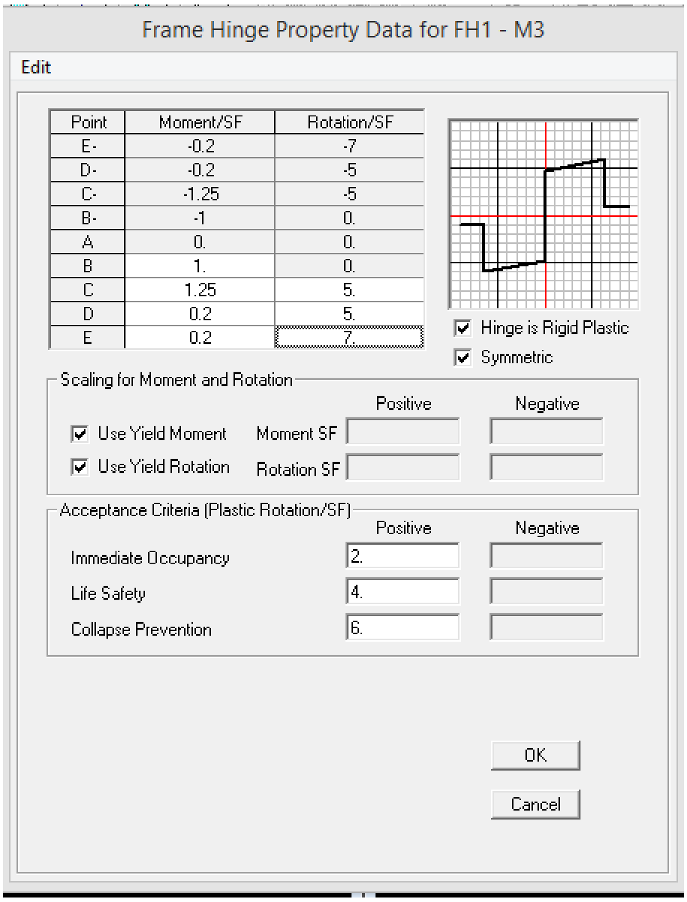

Figure 3.1: Moment hinge properties

At the time of adding hinges, relative distance of hinge location has to be specified. Relative distance zero indicates hinge location at starting point of the member while relative distance one indicates hinge location at the end of the member. Fig.3.2 shows hinges in 2D frame. Column hinges are located at relative distance zero while in beam , hinges are located at relative distance zero and one, i.e both end of the beam.

Seismic zone IV and hard soil is considered.

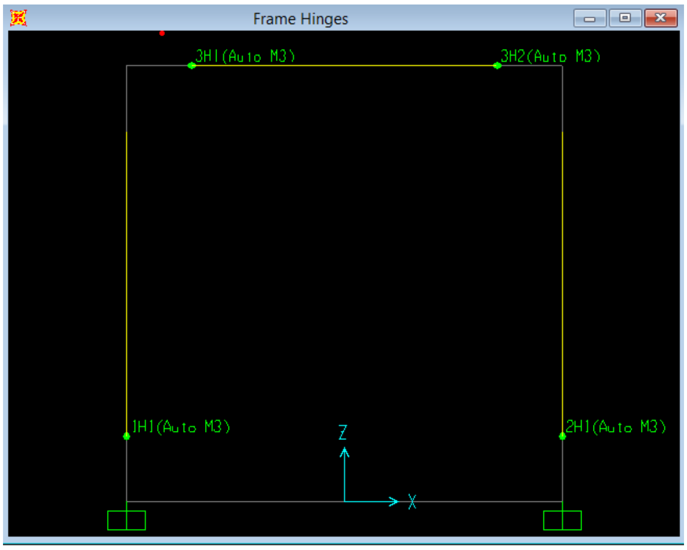

Figure 3.2: Nonlinear hinges in 2D frame

- **Static load cases**

  Two static load cases are defined: Dead load and Lateral load. Lateral load is applied in X-direction at the top of the frame. In the analysis, the value of lateral load can be taken as unity as the direction of applied load is important and not the magnitude. So lateral load of 1 KN is applied to the frame as shown in figure.3.3.

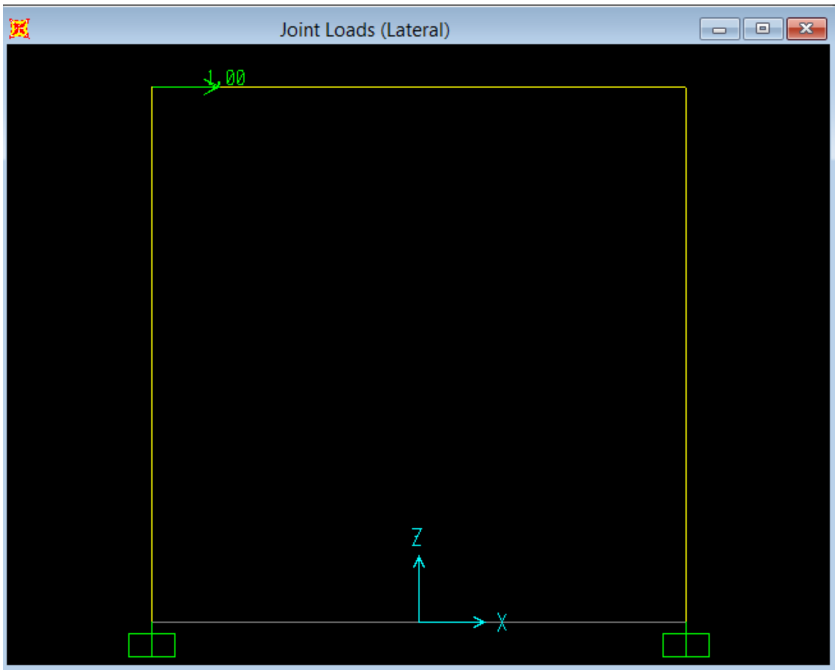

Figure 3.3: Lateral load on 2D frame

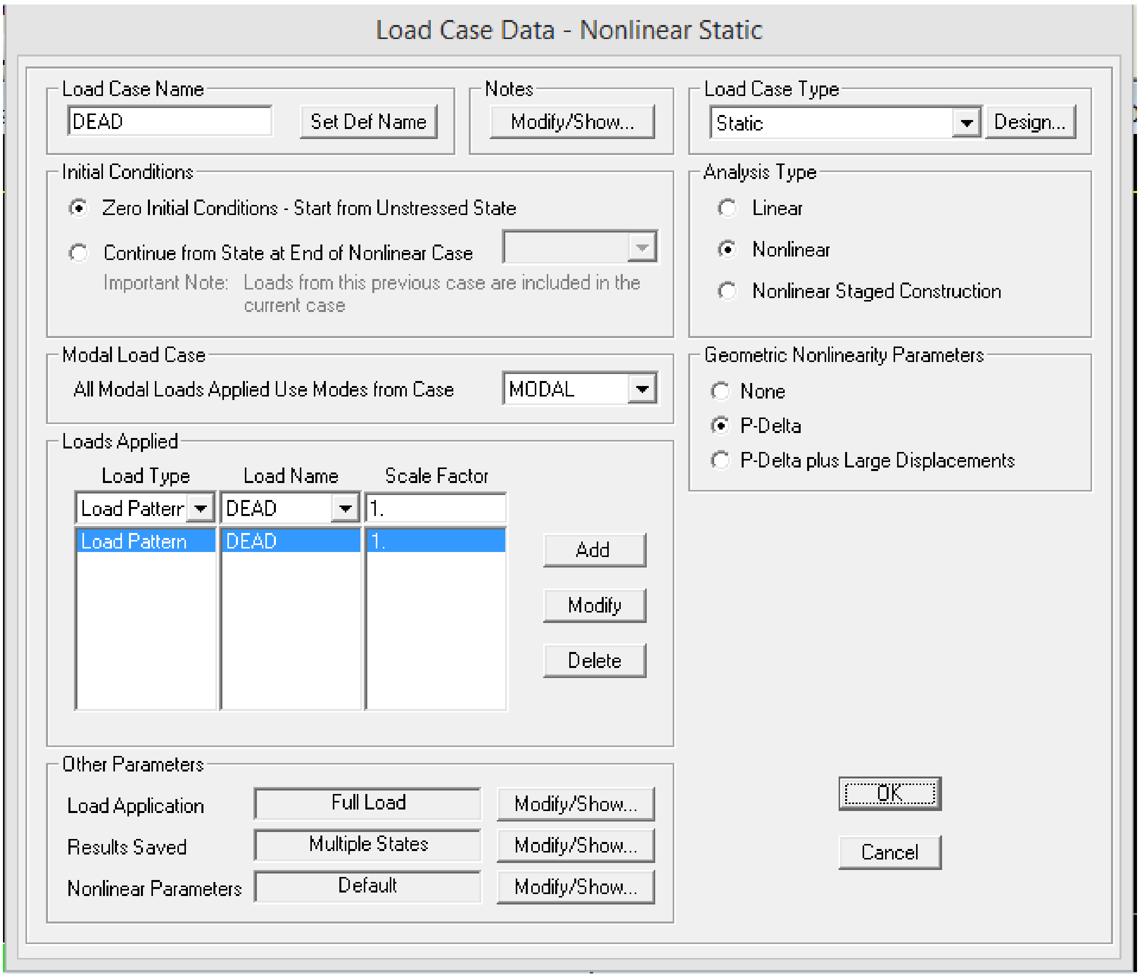

Figure 3.4: Defining DEAD load case

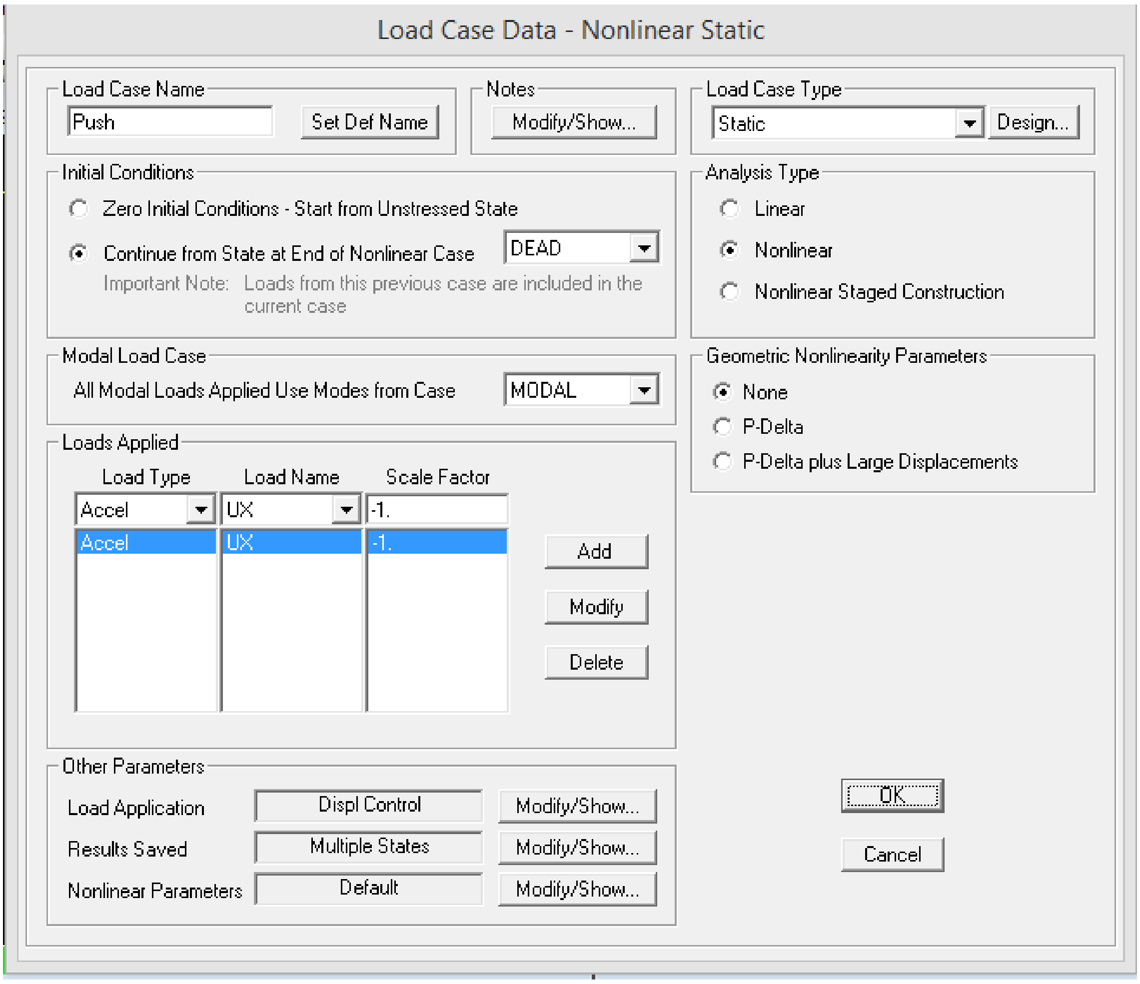

Figure 3.5: Defining PUSH case

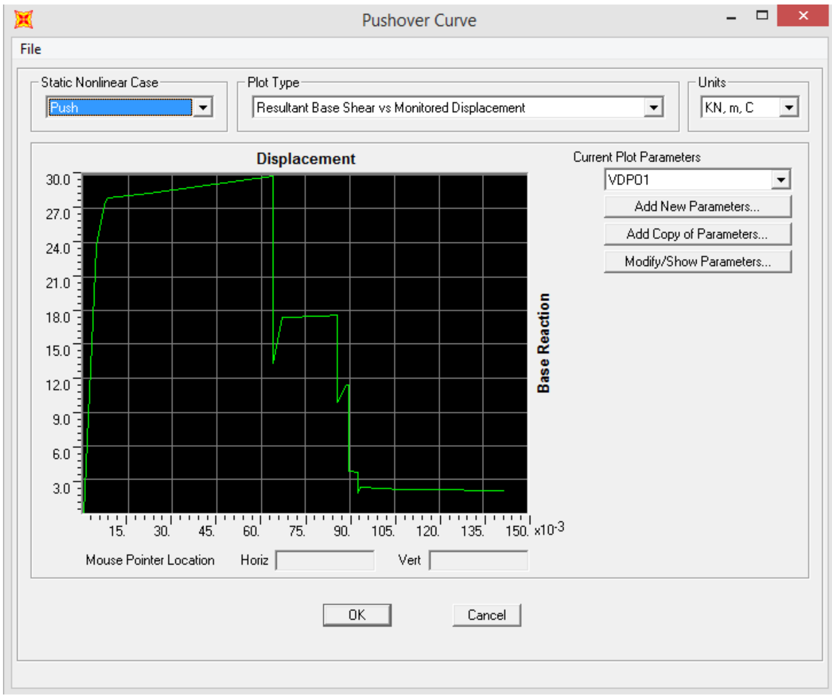

Figure 3.6: Pushover curve of 2D frame

- **Static nonlinear cases**

Two load cases are defined as shown in fig.3.4 and fig.3.5. In the DEAD case only dead load case was considered in the load pattern. As the load was known (gravity load) and the structure was expected to be able to support the load, the Force Control was used.

Member unloading method was unloading entire structure, geometrically non-linearity was included as P-delta and load was applied to the added elements. In PUSH case the load control was conjugate displacement control and only lateral load case was considered in load pattern.

Before carrying our pushover analysis, Static and Dynamic analysis had been carried out.

- **Pushover curve**

Pushover curve of 2D frame is as shown in fig.3.6.

Fig.3.7 shows tabular form of pushover curve.It also shows the step increment in the formation of hinges.

Table Display

File Edit

Pushover Curve - Push

Step	Displacemer m	BaseForce KN	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeyondE	Total
0	3.500E-07	0.000	4	0	0	0	0	0	0	0	4
1	0.004408	23.648	3	1	0	0	0	0	0	0	4
2	0.004614	24.320	2	2	0	0	0	0	0	0	4
3	0.006900	27.332	1	3	0	0	0	0	0	0	4
4	0.008046	27.935	0	4	0	0	0	0	0	0	4
5	0.022046	28.392	0	2	2	0	0	0	0	0	4
6	0.036046	28.894	0	2	2	0	0	0	0	0	4
7	0.049957	29.392	0	0	2	2	0	0	0	0	4
8	0.063957	29.894	0	0	2	0	0	2	0	0	4
9	0.063958	13.319	0	0	2	0	0	0	2	0	4
10	0.067134	17.377	0	0	2	0	0	0	2	0	4
11	0.081137	17.538	0	0	0	2	0	0	2	0	4
12	0.085559	17.619	0	0	0	1	0	1	2	0	4
13	0.085560	9.833	0	0	0	1	0	0	3	0	4
14	0.088725	11.361	0	0	0	1	0	0	3	0	4
15	0.089626	11.370	0	0	0	0	0	1	2	1	4
16	0.089627	3.723	0	0	0	0	0	0	3	1	4
17	0.092794	3.707	0	0	0	0	0	0	2	2	4
18	0.092796	1.866	0	0	0	0	0	0	2	2	4
19	0.092933	2.041	0	0	0	0	0	0	2	2	4
20	0.093500	2.315	0	0	0	0	0	0	2	2	4
21	0.107498	2.186	0	0	0	0	0	0	2	2	4
22	0.121498	2.110	0	0	0	0	0	0	2	2	4
23	0.135499	2.033	0	0	0	0	0	0	2	2	4
24	0.142299	2.034	0	0	0	0	0	0	2	2	4

Figure 3.7: Tabular form of Pushover curve of 2D frame

In the first step, all the four hinges were in A-B range. As the analysis progressed, the base shear increased and hence the hinges were going into the nonlinear range. In the subsequent steps, the hinges moved from linear range to nonlinear range, till the full development of the plastic hinge. Fig.3.8 shows hinge formation in 2D frame.

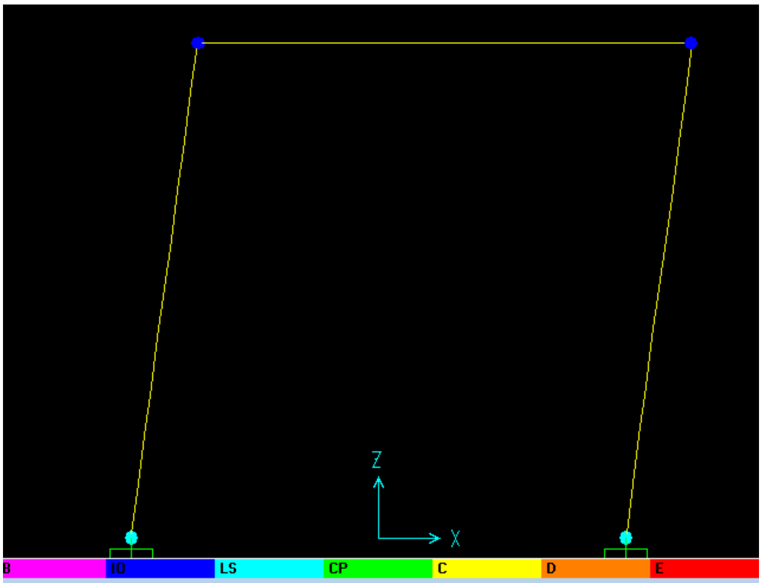

Figure 3.8: Hinge formation in 2D frame

3.2.2 Pushover analysis of 4-storey bare frame

Plan and elevation of 4-storey frame is shown in figure 3.9 and fig.3.10. It consists of 4 bay in X-direction and 3-bay in Y-direction. Each bay is of 5m X 5m size. Height of each story is 3m. Dimensions of beams and columns are as shown in table.1. Supports are fixed here.

Seismic zone = IV

Soil zone = III

Table 3.1: Data for 4-storey frame

Floor	**Column (mm)**	**Beam (mm)**	**LL (kN/m^2)**
GF	230x600	230x500	2
1st	230x600	230x500	2
2nd	230x500	230x450	1.5
3rd	230x500	230x450	1.5
4th	230x450	230x450	1.5

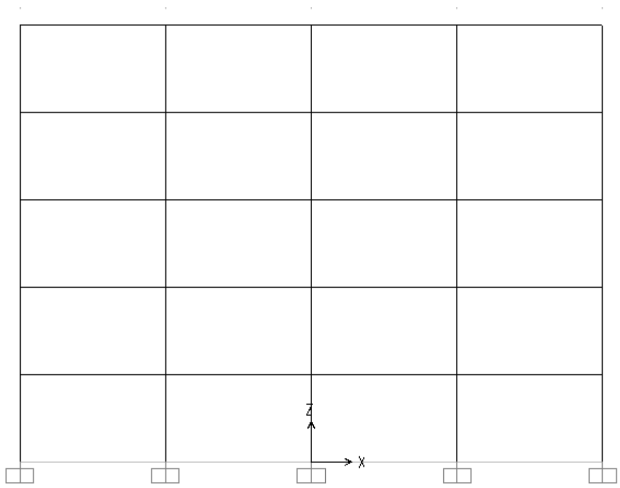

Figure 3.9: Elevation of 4-storey frame

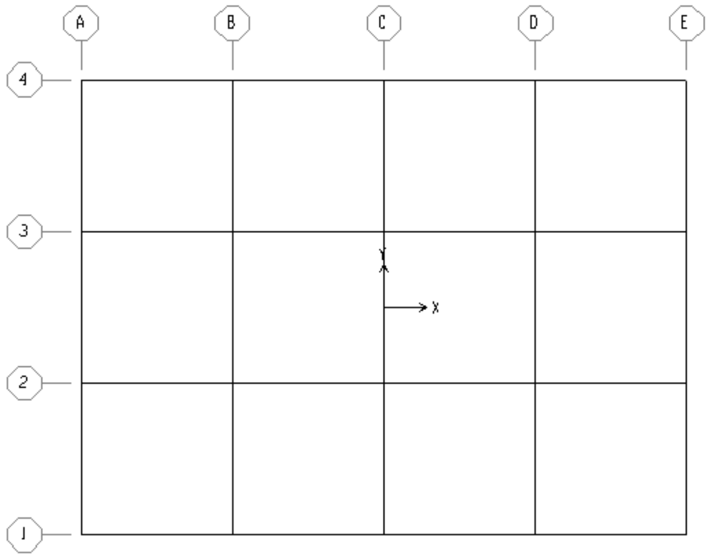

Figure 3.10: Plan of the 4-storey frame

Lateral load is applied in X-direction.The lateral load profile applied through out the height of the building is inverted triangular shape. The unit load was applied at the top of the column which was reduced to zero at the base.

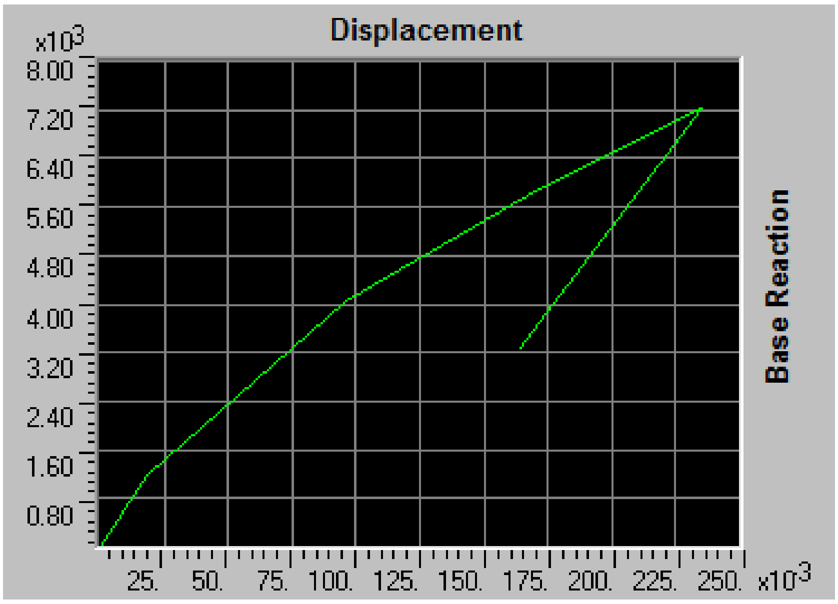

Figure 3.11: Pushover curve for 4-storey frame

Pushover curve for 4-storey frame is as shown in fig 3.11. The ultimate base shear the building can take before failure is around 7210 kN and the corresponding roof displacement is 235mm. The capacity spectrum curve of the same model is shown in Fig. 3.12.

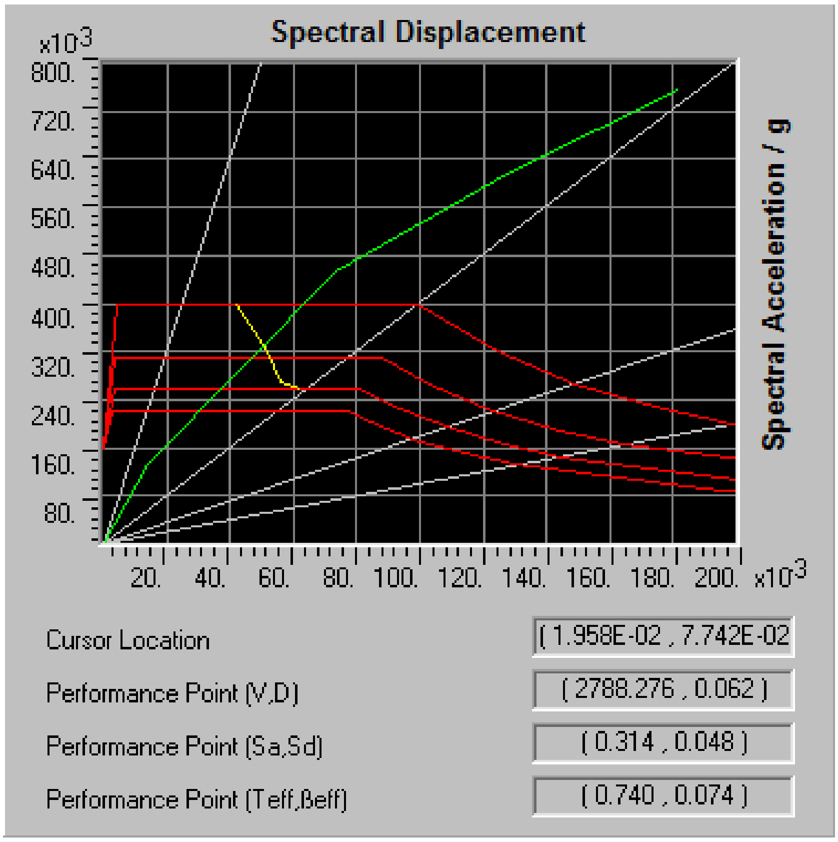

Figure 3.12: Capacity spectrum curve for 4-storey frame

Red curve in the Fig. 3.12 shows the response spectrum curve for various damping values. The Response Spectrum curves are governed by the values of Coefficient of Acceleration (Ca) and Coefficient of Velocity (Cv). For getting the response spectrum curve as per IS:1893-2002 (part I), the value of Ca and Cv were calculated and assigned to the software.

The values of Ca and Cv for all type of soils are given in table-2. For medium soil and Zone 3, Ca is 0.16 and Cv is 0.4. The base shear at performance point is 2788.27 kN and corresponding displacement is 62 mm.

The pushover analysis has included five steps. It has been observed that, on subsequent push to building, hinges started forming in beams first. Initially hinges were in B-IO stage and subsequently proceeding to IO-LS and LS-CP stages. Further pushing of building the hinges that formed initially, moved to higher stage of hinge property. At performance point, where the capacity and demand meets, out of 720 assigned hinges 540 were in AB stage, 44, 76 and 60 hinges are in B-IO, IO-LS and IO-LS stages, respectively.At performance point, hinges were in IO-LP range, therefore overall performance of building is said to be Life Safety to Collapse Prevention. Also it has been observed that, at ultimate capacity of building hinges formed were in columns. At ultimate load, columns capacity exhausted and analysis stopped. Hinge formation at performanced point is shown in fig.3.13

Table 3.2: Tabular form of pushover curve for 4-storey frame

Step	Displ(mm)	BS(kN)	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	¿E	Total
0	0	0	718	2	0	0	0	0	0	0	720
1	18.3	1165.8	586	62	72	0	0	0	0	0	720
2	95.8	4042.3	540	44	76	60	0	0	0	0	720
3	167.2	5783.2	534	38	36	108	0	4	0	0	720
4	234.7	7209.8	534	38	36	102	0	2	6	2	720
5	163.6	3255.1	720	0	0	0	0	0	0	0	720

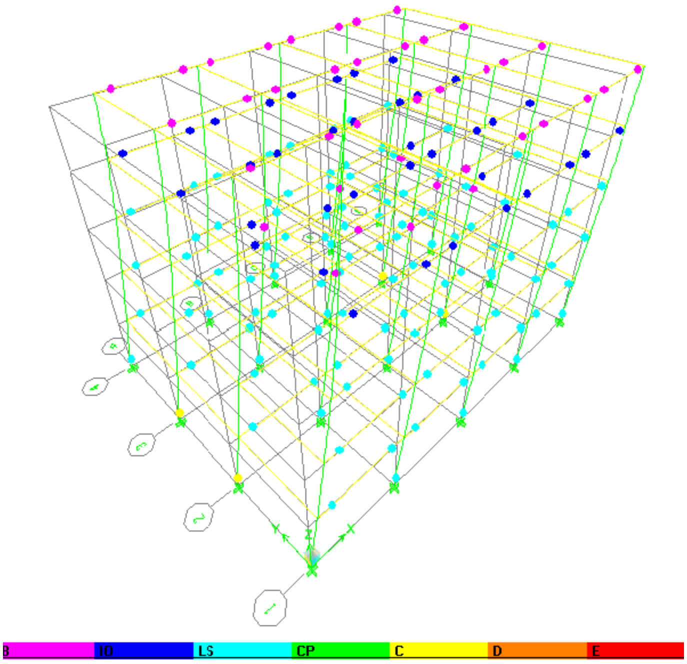

Figure 3.13: Hinge formation in 4-storey frame at performance point

3.3 Time History Analysis

Time history analysis is carried out to get response of the structure while subjected to ground motions.

3.3.1 Time History Analysis of 2D frame

Earthquake ground motion records are used to investigate the correlation between structural response and seismic intensity measures. A time history analysis should be performed to calculate a probabilistic demand curve to determine a relationship between the structural response demand and the intensity measure. After assigning static load cases and hinge properties same as pushover analysis, time history analysis is performed step by step in SAP.

Time history function

Time history function is defined for ELCENTRO ground motion with PGA of 0.35g. It is defined as shown in fig.3.13.

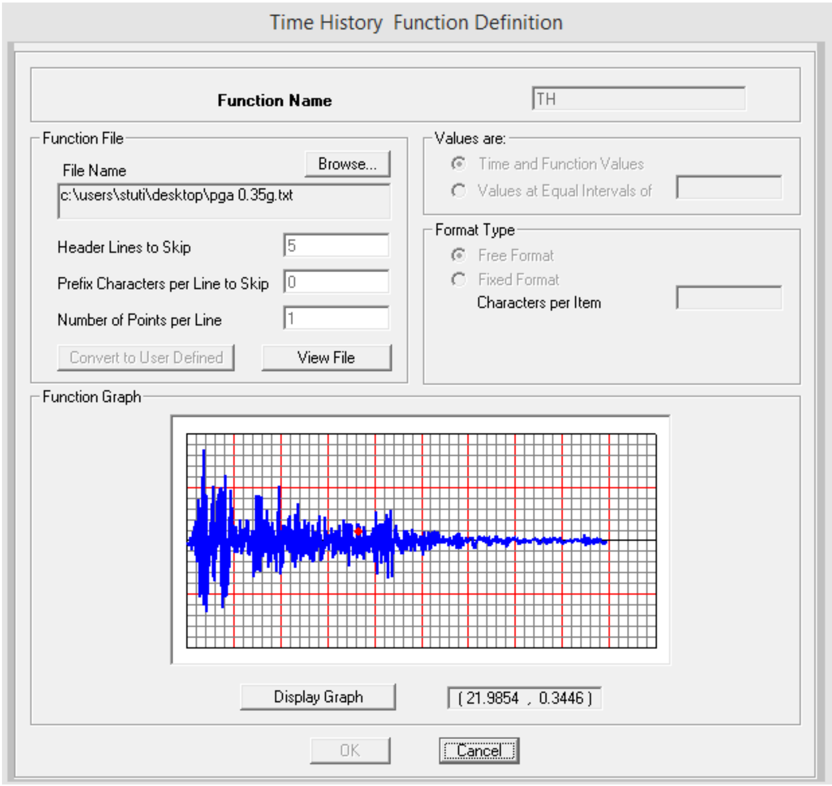

Figure 3.14: Time History function Definition

Time history load case

Time history load case is defined as shown in fig.3.14.

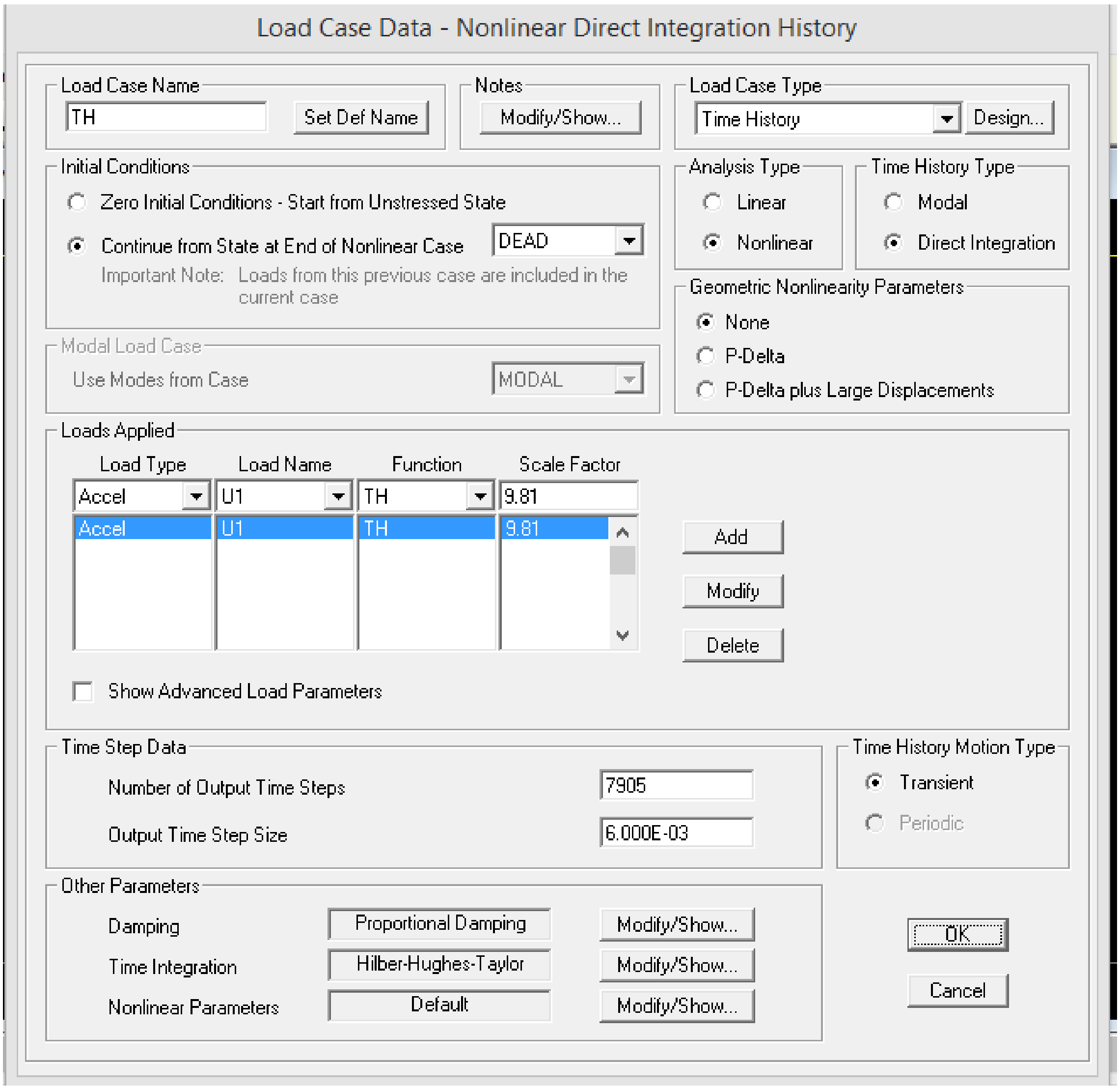

Figure 3.15: Time History load case data

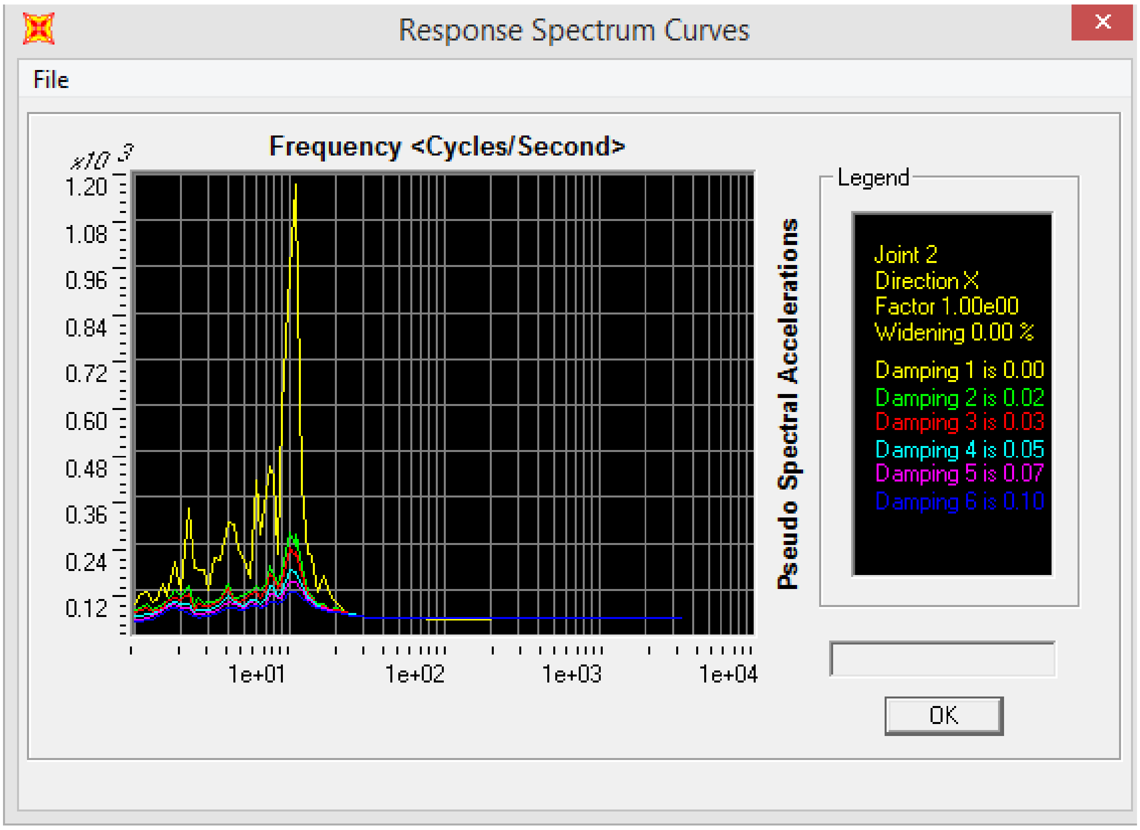

Figure 3.16: Response Spectrum Curve

Max displacement obtained is 0.0163 m at 5.016 sec.

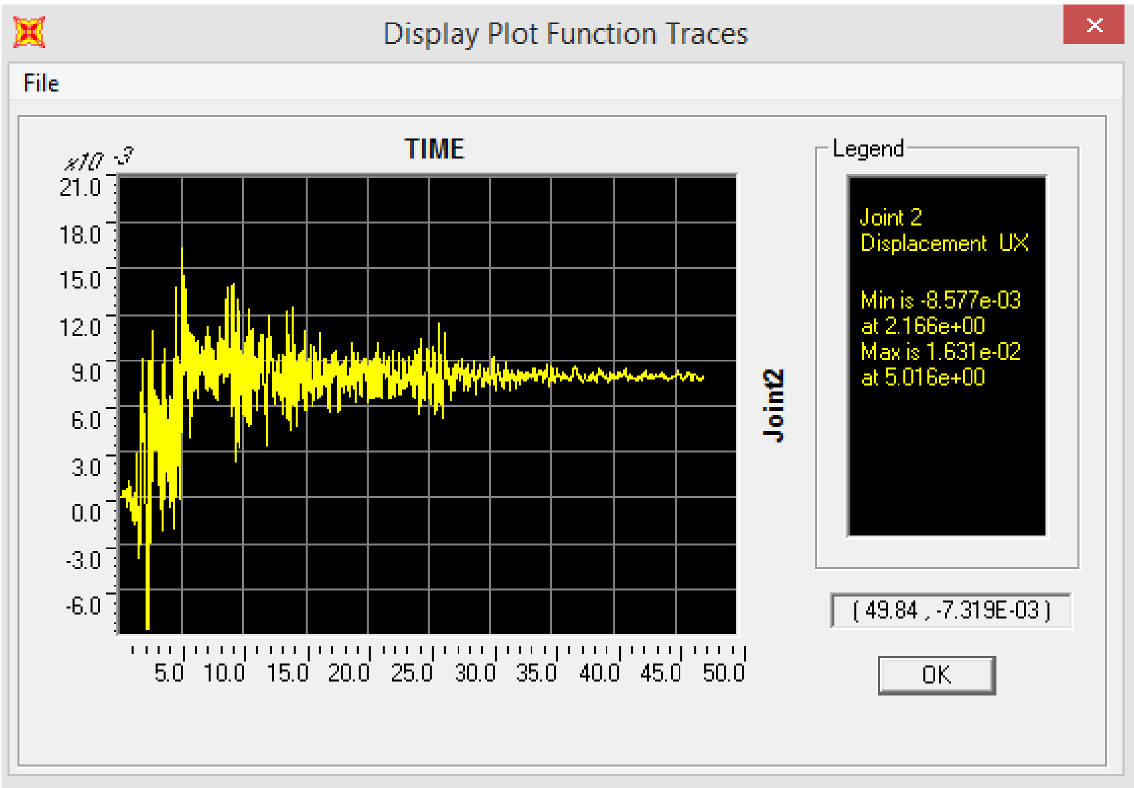

Figure 3.17: Displacement of joint 2

3.3.2 Time History analysis of 4-storey frame

Height of the building is 12 m with each storey height 3 m. Time history function is defined for ELCentro ground motion with PGA 0.35g. Displacement of each storey and storey drift calculations are shown in table below:

Table 3.3: Displacement and storey drift

Storey	Displacement	Drift
1	0.085	0.006
2	0.079	0.019
3	0.06	0.026
4	0.034	0.016
5	0.018	0.018

3.4 Summary

In this chapter, pushover analysis and time history analysis is studied for two different frames.

Chapter 4

Incremental Dynamic Analysis

4.1 General

Structural response assessment can be categorized as static or dynamic, linear or nonlinear. The complexity in the static regime increases from linear to nonlinear to pushover, where incremental static load is applied to the structure, leading to component by component failure and eventually system failure. Similarly, there is a parallel in the dynamic regime from linear to nonlinear, with a dynamic analysis termed Incremental Dynamic Analysis (IDA). In IDA incremental dynamic load is applied to the structure until it reaches to its dynamic instability.

Incremental Dynamic Analysis (IDA) is applied in a Performance-Based Earthquake Engineering context to investigate expected structural response and damage outcomes. This procedure consists of adopting a suitable suite of ground motions and performing IDA on a nonlinear model. IDA was specifically developed for seismic assessment: the dynamic load is earthquake ground motion, often scaled from lower to higher intensity; a suite of ground motions are typically applied to the structure, to obtain statistics about the structures performance, characterized by displacement and eventually collapse, under a range of earthquake excitation. The concept of IDA involves ground motions at multiple intensity levels. Ground motion selection provides the seismic input for structural response assessment. Ground motion intensity is often characterized by spectral acceleration (Sa) at the period of vibration of interest (T). We can generate IDA curves of the structural response, as measured by an Engineering Demand Parameter (EDP, e.g., the maximum peak interstory drift ratio max), versus the ground motion intensity level, measured by an Intensity Measure (IM, e.g., peak ground acceleration, PGA, or the 5%-damped first-mode spectral acceleration Sa(T1;5%)). Subsequently, limit-states (e.g., Immediate Occupancy or Collapse Prevention) can be defined on each IDA curve and the corresponding capacities can be calculated.

Basic steps in IDA procedures are as follows:

a. Construct model for the analysis.

b. Select an appropriate ground motion suit to represent the scenario of earthquake to result in collapse of the structure based on site seismic hazard.

c. Scale the ground motion suite such that the geomean spectral acceleration at the first mode period of the structure, Sa(T) , for each pair is at a value low enough for inelastic response to be negligible.

d. Analyse the model for each scaled ground motion pair and get maximum value of storey drift.

e. Increment the intensity of the ground motion at effective first mode spectral response acceleration.

f. Analyse the structure for each incrementally scaled ground motion pair and get maximum interstorey drift.

g. Repeat step e and f for each ground motion until analysis produces very large increase in drift for small increment in intensity value, which indicates global instability.

h. Determine the value of spectral acceleration, Sa(T) at which 50% of the ground motion pairs produce collapse and this value of Sa(T) is taken as median collapse capacity.

4.2 G+4 storey frame modelling

G+4 storey frame used in the IDA analysis is same as used in Pushover and time history analysis. Time period for each mode is calculated using SAP which is shown in table.

Table 4.1: Modal Period

Mode	Period
1	0.934514
2	0.647449
3	0.555006
4	0.327299
5	0.222721
6	0.201768
7	0.18886
8	0.148634
9	0.131696
10	0.126168
11	0.110052
12	0.090607
13	0.07066
15	0.055953

4.3 Selection and scaling of ground motions

Ground motions for IDA are selected based on magnitude and distance from the fault of an event. Soil class also affects selection of ground motions. Ground motion record sets include a set of ground motions recorded at sites located greater than or equal to 10 km from fault rupture, referred to as the Far-Field record set, and a set of ground motions recorded at sites less than 10 km from fault rupture, referred to as the Near-Field record set.

Following criteria are required for record selection:

- Source Magnitude: Large magnitude events pose greatest risk of building collapse due to longer duration of strong shaking and larger amounts of energy released. Smaller magnitude of events (M ł6.5) can cause damage to building (generally nonstructural nature), but are not likely to collapse new structures. Small magnitude events can generate strong ground motion but the duration of strong shaking is relatively short and the affected area is also relatively small. Similarly, large-magnitude events can generate strong, long duration shaking over large region and affects large population of building.

- Source Type: Record sets include ground motions from earthquakes with either strike-slip or reverse sources. These souces are typical of shallow crustal earthquakes in California and other Western United States locations.

- Site conditions: In record sets, ground motions recorded on either site class C (soft rock) or site class D (stiff soil). Records on Site Class E (soft soil) or site class F (sites susceptible to ground failure) are not used. Site class B (rock) generally includes relatively strong-motion.

- Site-Source: Site distance from fault is generally taken greater than 10 km.

- Number of records per Event: Strong-motion instruments are not evenly distributed across seismically active regions. Due to the number of instruments in place at the time of the earthquake, some large magnitude events have generated many records, while others have produced only a few. To avoid potential event-based bias in record sets, not more than two records are taken from any one earthquake for a record set.

Ground motions shall consist of pairs of appropriate horizontal ground motion components that shall be selected and scaled from individual recorded events. Each pair of motions shall be scaled such that for a period between 0.2T and 3T, the average of the SRSS spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the target response spectrum by more than 10 percent. Target response spectrum can be UHS(Uniform Hazard Spectrum) or CMS (Conditional Mean Spectrum).

Table 4.2: Ground motions without scaling

Sr. No	Event	Station	Magnitude	Site class	PGA(g)
1	Imperial Valley,1940	EL Centro Array 9	6.9	D	0.341
2	North-west Calif-02,1941	Femdale City	6.6	C	0.06
3	San Fernando, 1971	Lake Hughes	6.61	C,D	0.21
4	Imperial Valley-06,1976	Aeropuerto Mexicali	6.5	B,C	0.33
5	Imperial Valley,1979	EL Centro Array 1	6.53	D	0.19
6	Imperial Valley, 1979	Plaster city	6.53	C,D	0.057
7	Imperial Valley, 1979	Superstition Hills	6.53	D	0.13

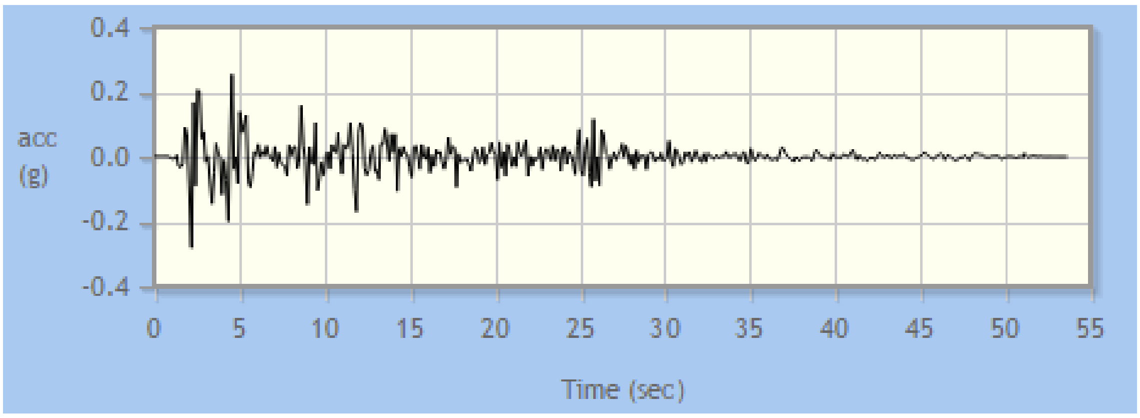

Figure 4.1: Acceleration Time History record 1

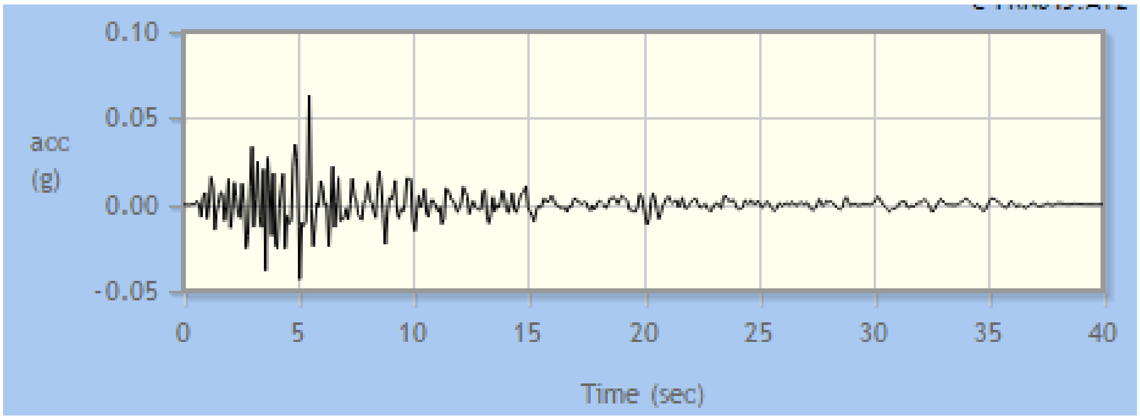

Figure 4.2: Acceleration Time History record 2

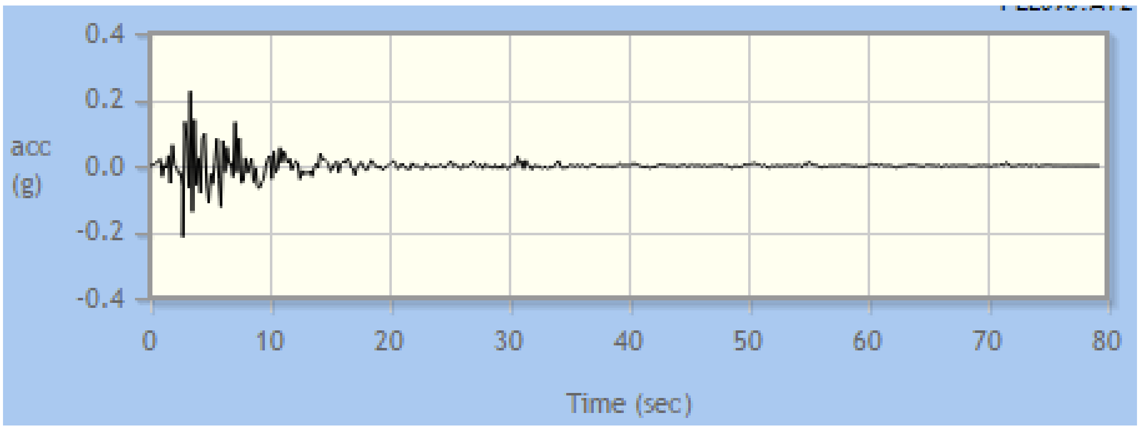

Figure 4.3: Acceleration Time History record 3

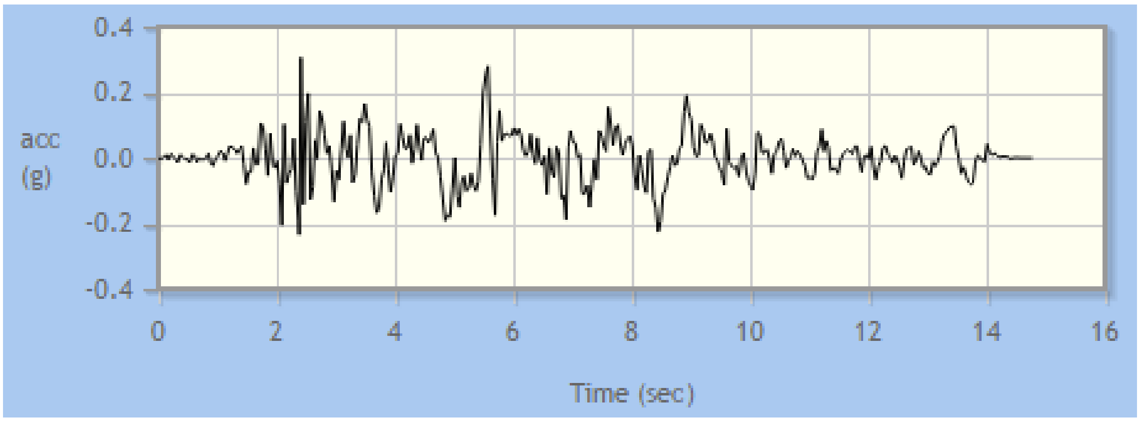

Figure 4.4: Acceleration Time History record 4

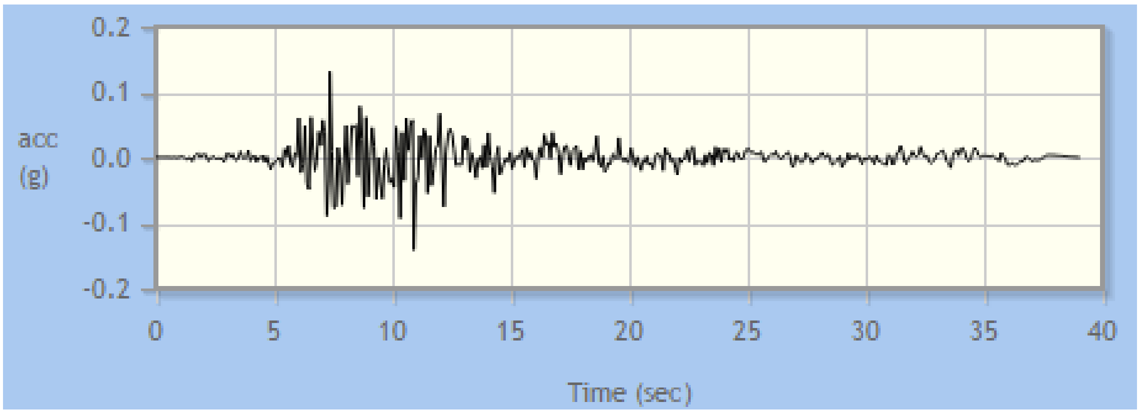

Figure 4.5: Acceleration Time History record 5

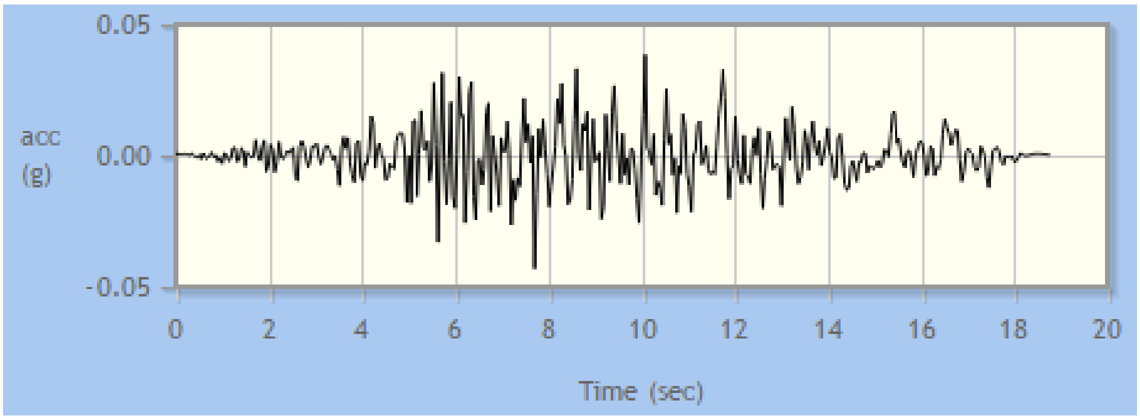

Figure 4.6: Acceleration Time History record 6

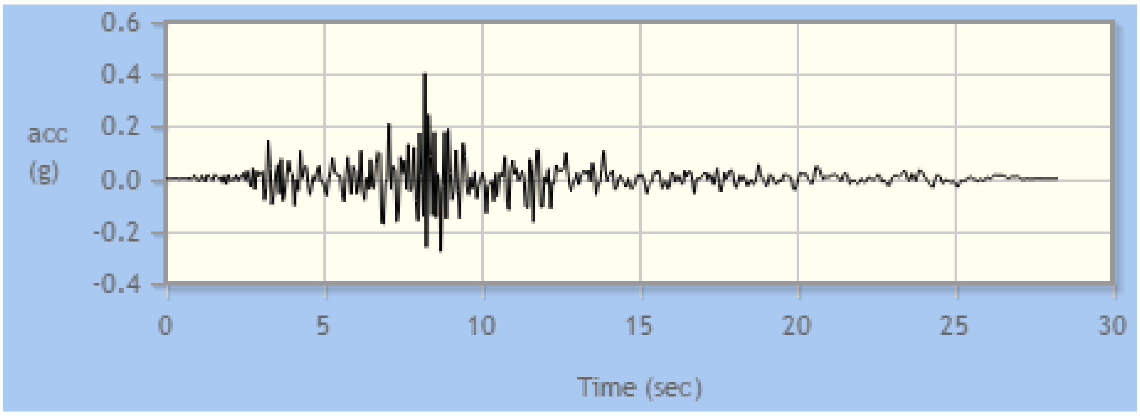

Figure 4.7: Acceleration Time History record 7

4.3.1 Target Spectrum and Spectral shape

Selection and scaling of ground motion can be done by uniform hazard spectra or conditional mean spectra. In both cases a number of ground motions should be used to calculate median collapse capacity. Generally use of 11 pair of ground motions, each rotated at 90 degrees to produce total 22 ground motion sets were found to be sufficient. But for some structures, fewer motions (minimum 7 motions) can be used for stable prectiction of median collapse capacity.

Ordinates of uniform hazard spectra are associated with the same annual frequency of exceedance. Scaling ground motions to match a target uniform hazard spectrum is recommanded by Building code and seismic design provisions for nonlinear response history analysis over a wide period range. Although this is convenient for design, and likely conservative, such scaling procedures have the following limitations:

- If the spectral ordinates of the UHS are governed by multiple scenario events, the spectral shape will not be representative of any of the governing events, regardless of the return period (Baker and Cornell, 2006).

- For earthquake shaking with long return periods, the spectral ordinates of uniform hazard spectra are typically associated with high values of epsilon (greater than 1) across a wide range of periods (Harmsen, 2001). For the case in which the geomean spectral ordinate of a ground motion pair matches the uniform hazard spectrum ordinate at a given period, the geomean spectral ordinates are likely to be less than the uniform hazard spectrum at other periods.

Conditional Mean Spectrum is introduced by Baker and Cornell, which considers the correlation of spectral demands at different periods, to address the limitations of UHS. Conditional Mean Spectra estimate the median geomean spectral acceleration response of a pair of ground motions given an [M, r] pair and a target spectral ordinate Sa(T1).

PEER NGA database Beta gives direct calculation method for target spectrum. Generally we use conditional mean spectrum as target spectrum to scale ground motion for nonlinear response history analysis. Target spectrum can be constructed by specifying average values of magnitude and fault rupture distance. In this case, ground motions with magnitude between 6.5 to 6.9 and fault rupture distance is between 10 to 30 km. Damping value is taken as 5% (as RC structure).

Target spectrum is constructed using following parameters:

Damping value= 5%

Average magnitude=6.7

Average rupture distance = 20 km

Fault type = strike slip or reverse slip

Figure 4.8 shows PEER NGA database, which includes all required parameters filled.

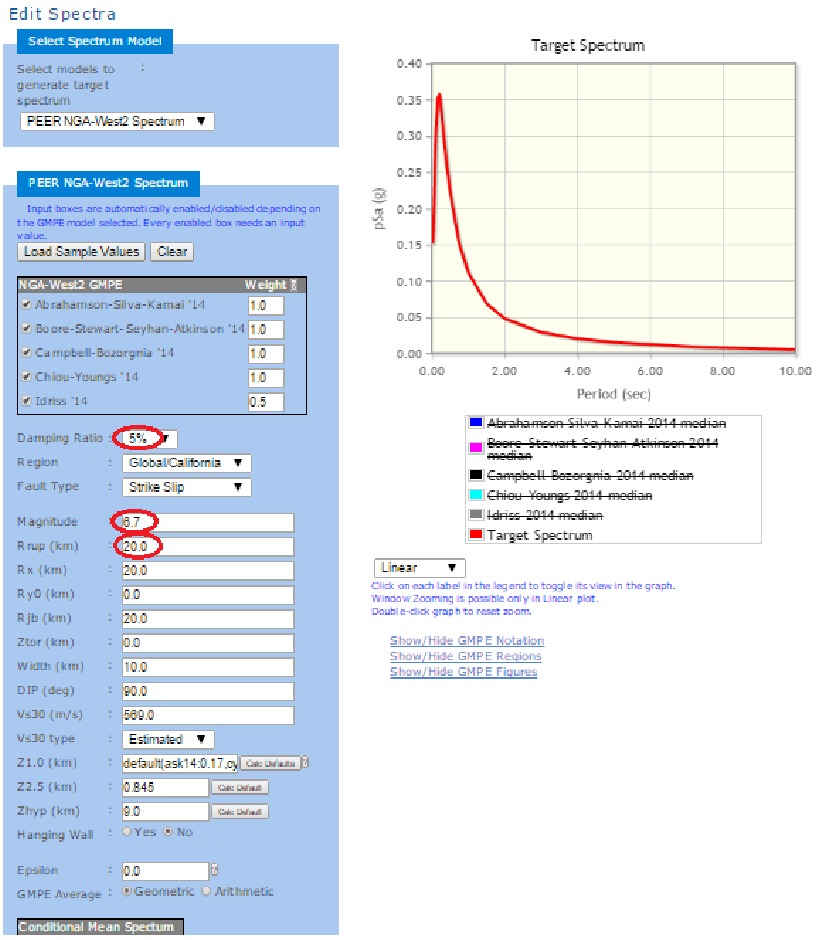

Figure 4.8: Target spectrum formed in PEER NGA database with average magnitude 6.7 and average rupture distance 20km with damping ratio 5%

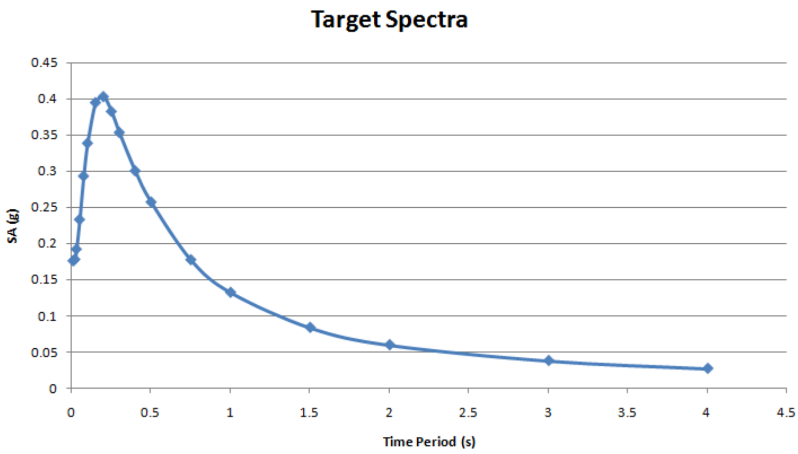

Figure 4.9: Target spectrum for scaling Ground motions

Comparision of spectral shape of selected suit of ground motion with target spectrum

PEER NGA database Beta is used to compare spectral shape of selected suit of ground motion with target spectrum. So fig.4.10 and fig.4.11 shows comparision. Fig.4.10 shows PEER NGA database form of comparing this ground motions.Also it includes suit mean+1 and mean-1 spectrum. Scale factores for spectral acceleration 0.15g is shown in table. Increasing the spectral acceleration value at every 0.15 interval, scale factors are calculated and analysis of structure is to be done.

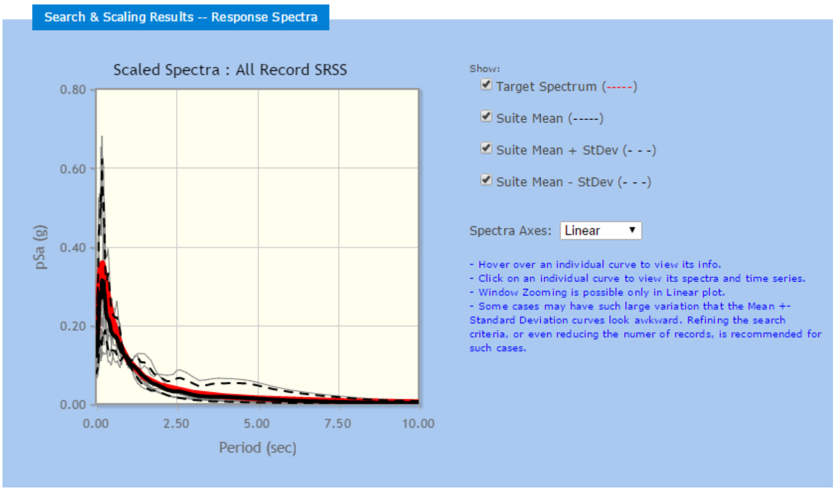

Figure 4.10: Comparision chart of spectral shape of selected suit of ground motion to target spectrum (PEER NGA Database)

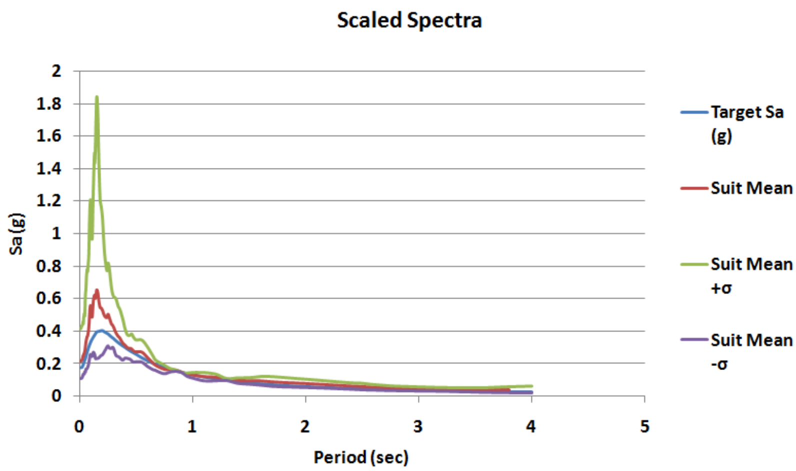

Figure 4.11: Comparision chart of spectral shape of selected suit of ground motion to target spectrum

Table 4.3: Ground motions with scale factor

Sr. No	Event	Station	Magnitude	Scale Factor	PGA(g)
1	Imperial Valley,1940	EL Centro Array 9	6.9	0.3579	0.341
2	North-west Calif-02,1941	Femdale City	6.6	3.0692	0.06
3	San Fernando, 1971	Lake Hughes	6.61	0.7058	0.21
4	Imperial Valley-06,1976	Aeropuerto Mexicali	6.5	0.4522	0.33
5	Imperial Valley,1979	EL Centro Array 1	6.53	1.8465	0.19
6	Imperial Valley, 1979	Plaster city	6.53	3.2424	0.057
7	Imperial Valley, 1979	Superstition Hills	6.53	3.6301	0.13

4.4 Summary

This chapter includes Incremental Dynamic Analysis procedure and scaling of ground motions at defined spectral acceleration value. For IDA G+4 storey bare frame is considered which is as same as considered in pushover analysis. Scale factores are calculated by scaling response spectrum of selected ground motions to the target spectrum which is conditional mean spectrum. PEER NGA database is used for scaling ground motions.

Chapter 5

Postprocessing

5.1 General

In this chapter, analysis of G+4 storey bare frame is done and IDA curves are generated, which is defined as a graph of spectral acceleration versus maximum storey drift ratio. Limit states are defined on this graph and behaviour of frame regarding to limit states is to be known.

5.2 Generation of IDA curves by interpolation

Once the desired IM (intensity measured) and DM (Demand measure) values are extracted from the dynamic analysis, we are left with a set of points for each record that reside in the IM-DM plane and lie on curve as shown in figure 5.3.

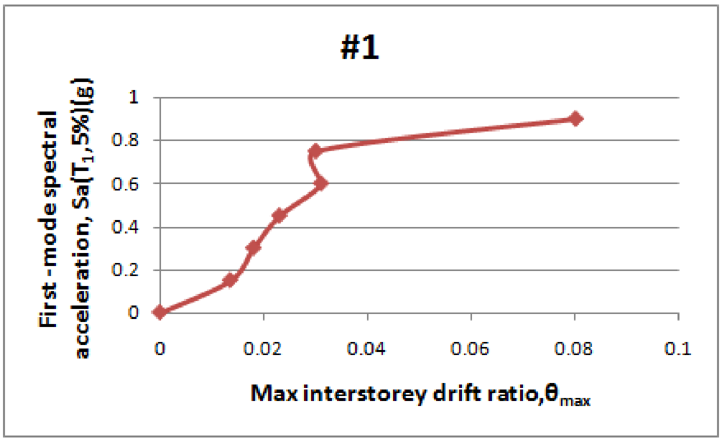

Figure 5.1: IDA curve for record #1

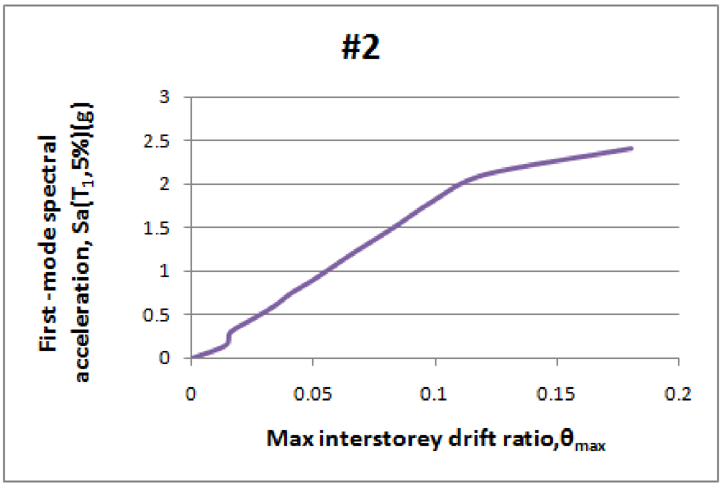

Figure 5.2: IDA curve for record #2

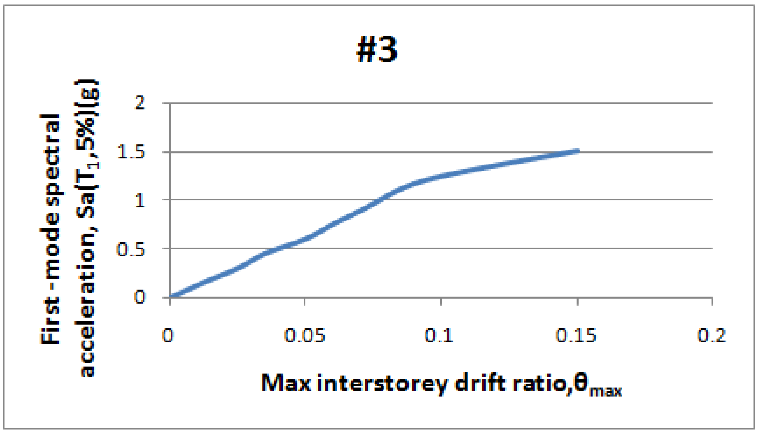

Figure 5.3: IDA curve for record #3

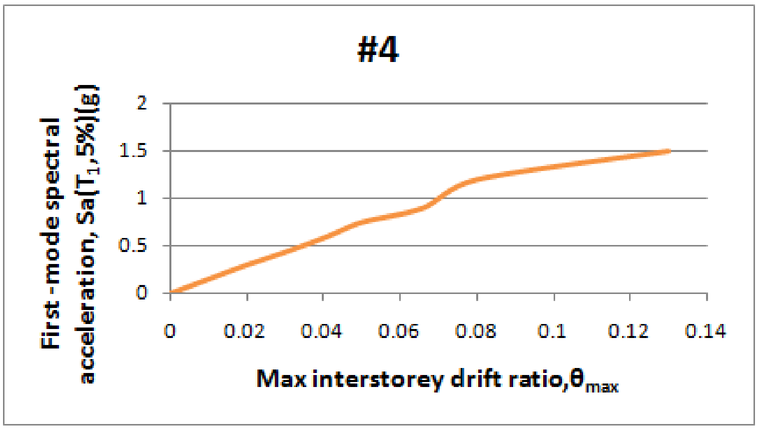

Figure 5.4: IDA curve for record #4

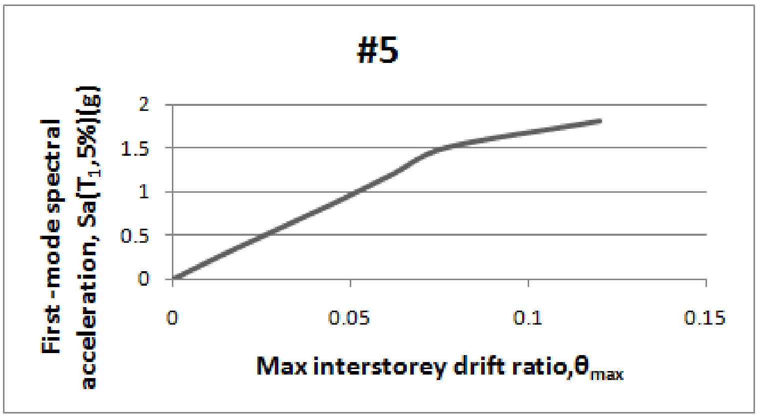

Figure 5.5: IDA curve for record #5

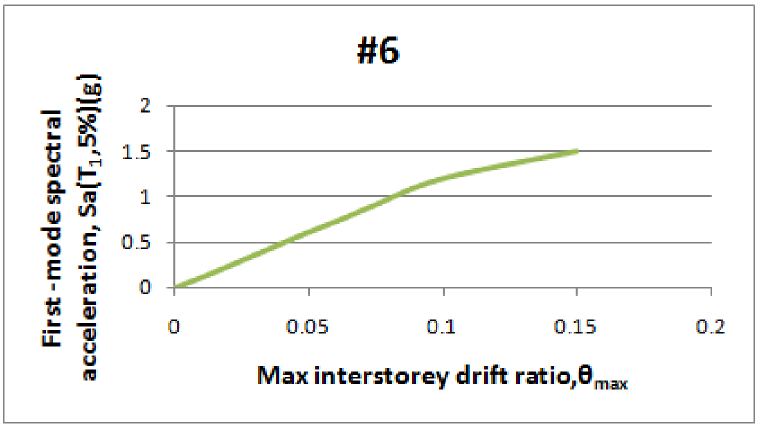

Figure 5.6: IDA curve for record #6

By interpolating these points, the entire IDA curve can be generated without performing additional dynamic analysis. For interpolating this points, we may use linear approximation or spline interaction. Here IDA curves are generated by linear interaction. Now having the complete curve, it is possible to calculate DM values at any IM level. IDA curve for all seven ground motions are shown in fig 5.1 to fig.5.7. For record #3, IDA curve is quite simple. It starts with a straight line in an elastic range and then shows the effect of early yielding and local damage by having some change in local tangent slope but generally it stays on elastic slope. At any given

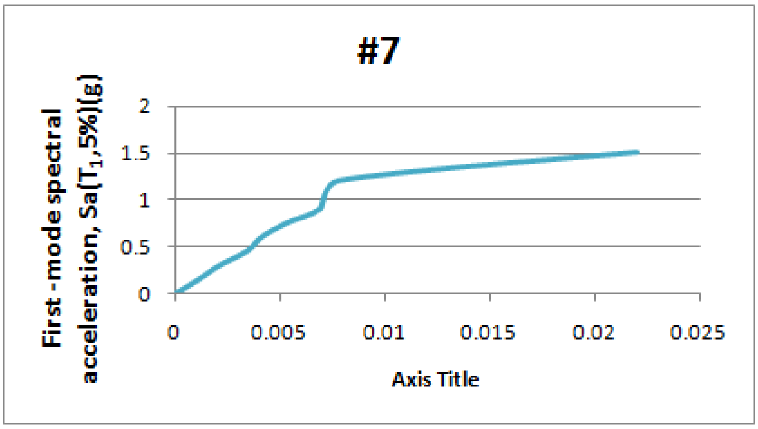

Figure 5.7: IDA curve for record #7

Sa level below 0.5g graph remains about the same displacement as an elastic system. Then after Sa value 0.5g it starts softening, showing increasing tangent slope, reaching the flatline slightly above Sa value 1.2g, where the strcture responds with practically "infinite' θmax values and numerical non-convergence has been encountered during the analysis. This is when the structure has reached to global dynamic instability, when a small increment in IM level results in unlimited increase in DM-response. IDA curves are always not simple, as curve for record #1 is quite different then others.

5.3 Defining limit-states on IDA curve

In order to get performance calculation for PBD, defining limit-states is necessary on the IDA curves. For these study, three limit states are chosen: Immediate Occupancy (IO), Collapse Prevention (CP) and Global dynamic Instability (GI). For this RC frame, IO limit-state is decided at $\theta max = 1\%$. FEMA-350 is used to define CP point, which is not exceeded on the IDA curve until the final point where local tangent reaches 20% of the elastic slope or $\theta max = 10\%$, whichever occures first in IM term. CP limit-state is defined at a point where IDA curve is softening towards flatline but at low enough values of θmax (less than 10%). Finally, GI limit-state happens when the flatline is reached and any increase in IM result is practically infinite DM response.

At IO limit state, it is easy to calculate IM-values. For these, IM values that produces $\theta max = 1\%$ is calculated and if more than one value of IM is there at $\theta max = 1\%$, lowest one will be selected. Considering record #3, IO is violated for $Sa(T1, 5\%) \geq 0.33g$ or $\theta max \geq 1\%$.

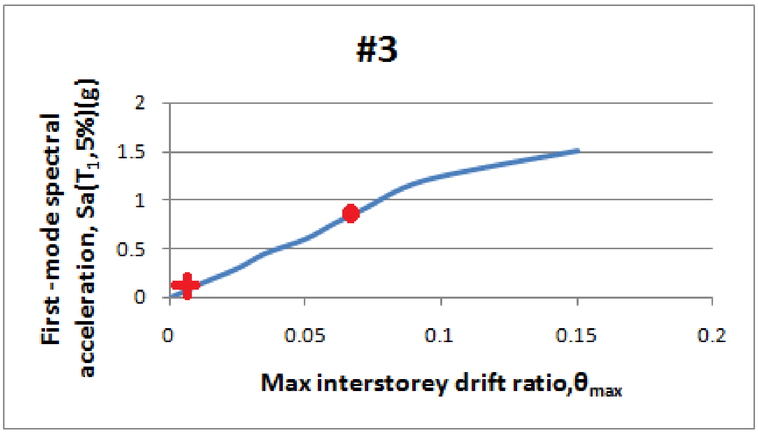

Figure 5.8: CP-limit state defined on IDA curve for record #3

As per CP limit-state defination, we have to find highest point where IDA tangent slope is equal to 20% of the elastic. These point usually lies on the softening segment that precedes the flatline. Another CP point is at $\theta max = 10\%$. So whichever comes first, decides CP capacity. Simple shape of IDA curve of record #3 makes this easy. Here 20% -slope rule governs and generates one point. So CP point is violated for$Sa(T1, 5\%) \geq 0.44g$ or $\theta max \geq 2\%$.

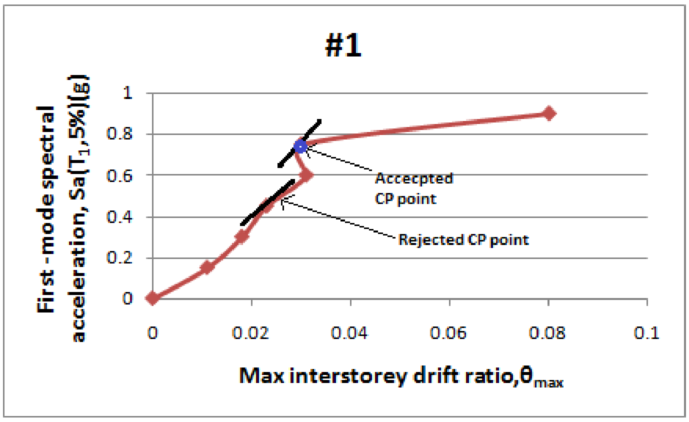

Figure 5.9: CP-limit state defined on IDA curve for record #7

Now considering record #1, it has complicated shape as shown in fig.5.8. The IDA curve starts softening at about 0.2g showing slope less than the elastic. After 0.2g it hardens by local slope higher than the elastic. Also the response of frame at Sa(T1,5%)= 0.6g is higher than Sa(T1,5%)= 0.75g. IDA curve also starts softening after 0.8g after it goes as flatline. These generates two locations where CP point can be defined. The lower CP point should be rejected, as it does not directly precedes to flateline. These shows that the frame is not as close to global collapse. So, other CP point definition should be required. So, CP point is exceeded for record #1 when $Sa(T1, 5\%) \geq 0.77g$. At these spectral acceleration value DM-value is 0.031.

IM and DM values of all records and each limit states are defined in table below:

Table 5.1: Table for limit-states data

	Sa(T1,5%)			θmax		
Sr	IO	CP	GI	IO	CP	GI
1	0.15	0.77	0.9	0.01	0.031	infinity
2	0.1	1.6	1.4	0.01	0.068	infinity
3	0.33	0.44	1.5	0.01	0.02	infinity
4	0.12	0.6	1.5	0.01	0.03	infinity
5	0.21	1.1	0.8	0.01	0.052	infinity
6	0.1	0.9	1.5	0.01	0.057	infinity
7	0.08	0.7	0.9	0.01	0.04	infinity

5.4 Summarizing IDA curves

All IDA curves are generated after analysis and limit-states are defined. Large number of data collected and fig. 5.10 shows combined IDAs on one graph. In that IDA curves shows different behaviour, showing large record-to-recird variability. So it is necessary to summarize all these data and show randomness introduce by the records. The limit-state capacities can be summarized by some mean value and measure of dispersion like standard deviation. Consequently we have choosen to calculate 16%, 50% and 84% fractile values of IM and DM capacities for each limit-state as shown in fig.5.11 and table 5.2.

From the table 5.2, it can be seen that at $Sa(T1, 5\%) = 0.9g$ or at $\theta max = 0.03$, 50% of the ground motion records have forced these G+4 storey frame to violate CP. The IO limit is shown at the intersection of each IDA curve with $\theta max = 1\%$ line, CP point is representated by dots, and GI occures at flatline.

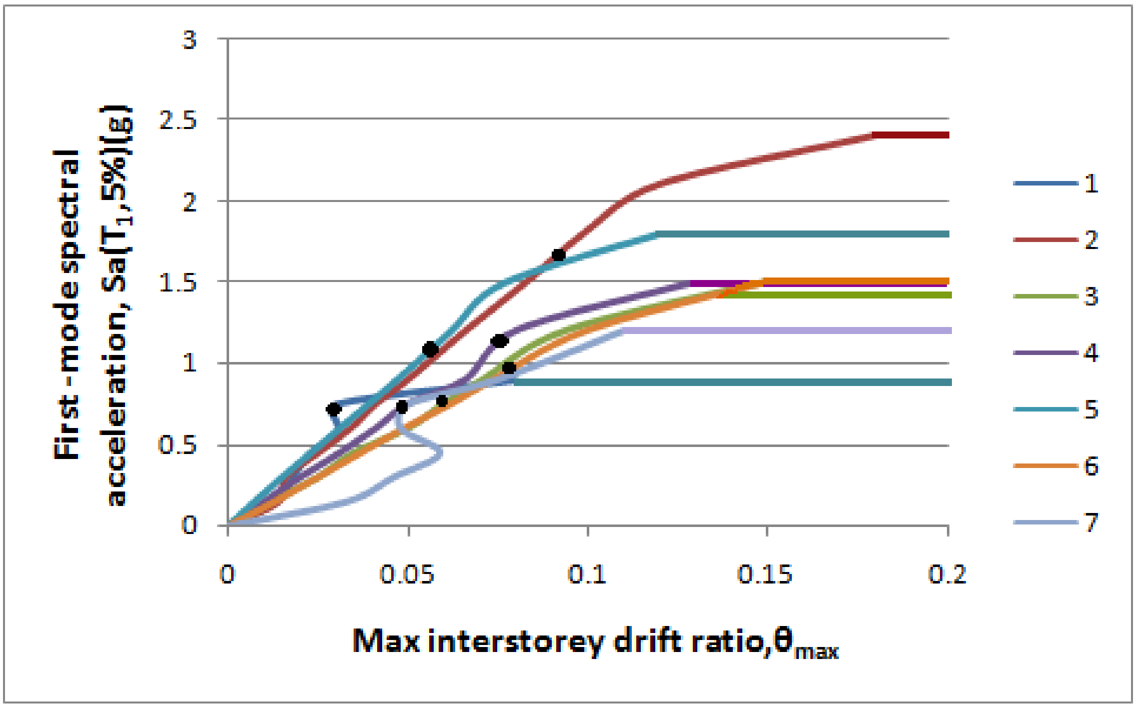

Figure 5.10: All seven IDA curves and associated limit-state capacities

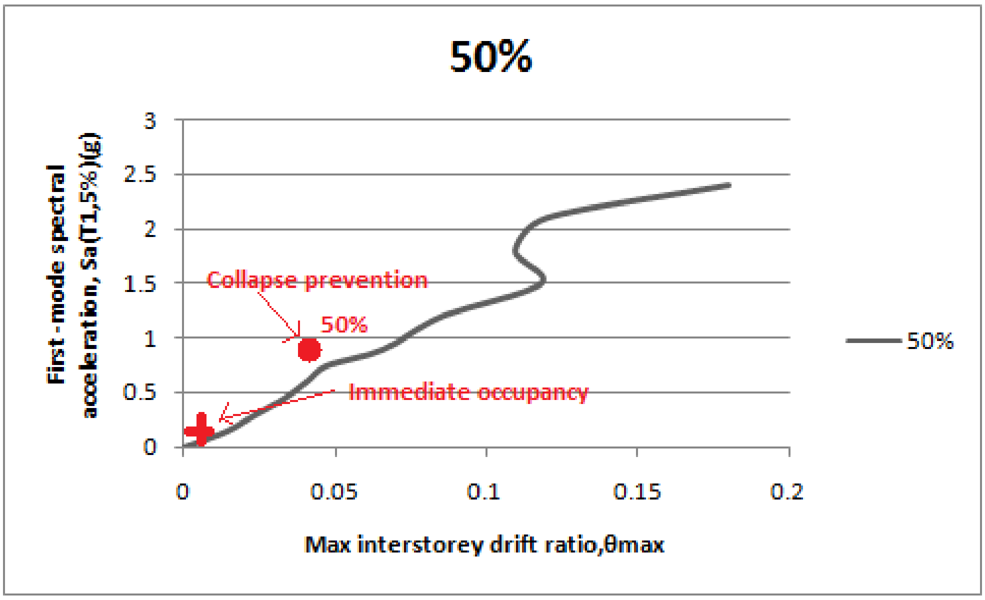

Figure 5.11: Summary of IDA curves and corresponding limit-state capacities into 50% fractile

As shown in fig.5.11, Immediate occupancy occures at Sa(T1,5%)= 0.16g and $\theta max = 1\%$, while Collapse prevention limit state occures at Sa(T1,5%)= 0.95g and $\theta max = 3.9\%$.

5.5 Summary

This chapter includes post-processing of IDA curves. Limit-states are defined on the individual curve and mean IDA curve is determined. Also limit-state is defined on mean IDA curve. Which indicates behaviour of structure according to that limit-state.

Chapter 6

Summary and Conclusions

6.1 Summary

The main objective of this work is to carry out performance based analysis of RC structure. Here first pushover and time history analysis is done as a basic of incremental dynamic analysis. Firstly pushover and time history analysis is done for 2D frame. After that G+4 storey bare frame is used for pushover analysis and time history analysis. Similar frame is used for IDA. IDA graphs are generated and limit-states are decided on IDA curve and building performance is predicted.

6.2 Conclusions

Based on the current study, following set of conclusion are made:

- Pushover analysis of G+4 storey frame shows building performance is limited between IO to CP. So retrofication is not required.
- IDA method takes long time for computation.
- IDA gives interesting aspect of structural behaviour, like large record to record variability, hardening softening of IDA curve and flatline.

- As all ground motion scaled for same spectral acceleration value do not show same behaviour, it is important to take number of ground motions for analysis and by deriving mean IDA curve building performance can be predicted.

- In order to generate demand of $\theta max = 4\%$, 50% records need to be scaled at level Sa(T_1,5%)= 0.95g.

6.3 Future Scope of Work

The future scope of work includes:

I. Incremental Dynamic Ananlysis of Steel structures like steel frame building, etc.

II. Comparison of incremental dynamic analysis with pushover analysis.

III. Using IDA, calculate mean annual frequencies of exceeding the specified limit-states.

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Appendix A

Calculation of C_a and C_v

Elastic Response Spectrum depends on site seismic coefficients C_a and C_v for each earthquake hazard level of interest at a site. Effective peak acceleration of the ground is represented by the seismic cofficient C_a. And average value of peak response of a 5% damped short period system is 2.5 times C_a in acceleration domain. C_v represents 5% damped response of a 1-sec system and when divided by period defines acceleration response in the velocity domain.

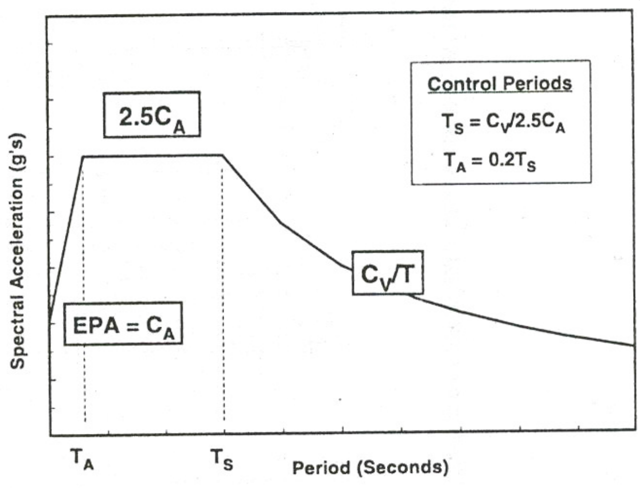

Figure A.1: Construction of a 5% damped Elastic Response Spectrum

The Response Spectrum for 5% damping given in IS:1893:2002(part I) is shown in Fig 2.

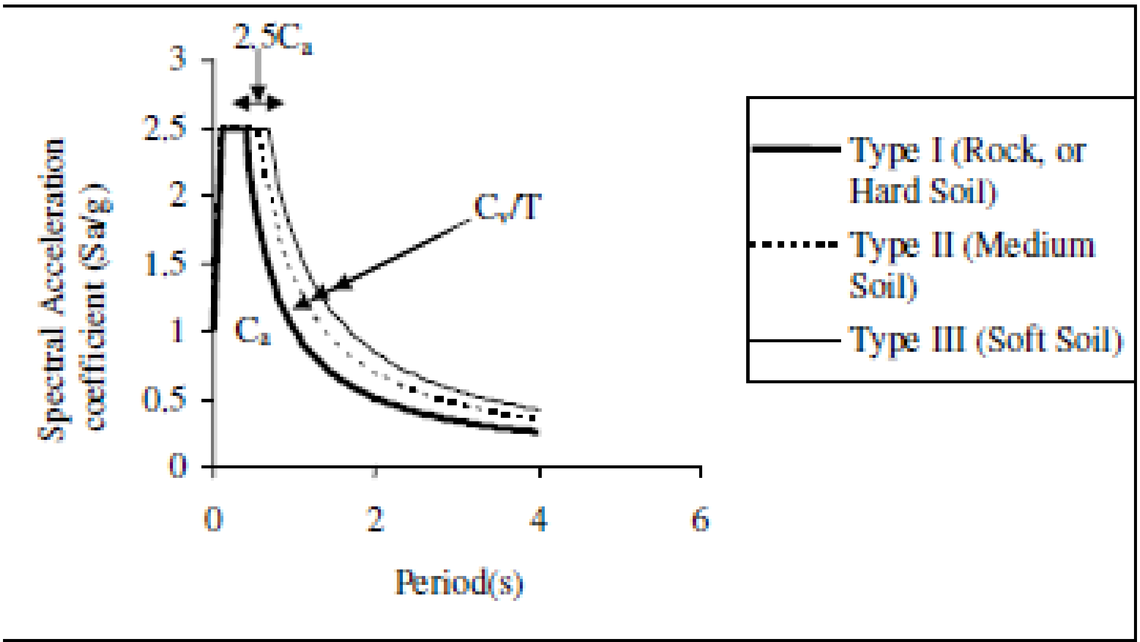

Figure A.2: Response Spectrum Curve of IS 1893 2002 (part I)

Coefficient of acceleration (C_a) = Z

Coefficient of velocity (C_v) = 2.5 * Ca * T_S

For Zone IV (Rock, or Hard Soil)

$C_a = 0.24$ T= 0.9 sec

$C_v = 2.5X0.24X0.9 = 0.54$

Appendix B

Excelsheets- IDA curve

Excelsheet used for generating IDA curves is attached here. It includes Spectral acceleration ($S_a(T_1{,}5\%)$), scale factor (SF) and max interstorey drift (θmax) for every increment.