SEISMIC ANALYSIS AND DEVELOPMENT OF FRAGILTY CURVE FOR AN RCC FRAME

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

Master of Technology
In
Structural Engineering

By

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CANDIDATE'S DECLARATION

I hereby declare that the project work entitled **"Seismic analysis and development of fragility curve for an RCC frame"** submitted to Department of Civil Engineering, DTU is a record of an original work done by **Lalit Choudhury** under the guidance of **Mr G.P Awadhiya**, Associate Professor, Department of Civil Engineering, DTU, and this project work has not performed the basis for the award of any Degree or diploma/fellowship and similar project, if any.

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CERTIFICATE

It is certified that the thesis titled "**Seismic analysis and development of fragility curve for an Rcc frame"** Presented by **LALIT CHOUDHURY** in partial fulfillment of the requirement for the award Of the Master of technology in **Civil Engineering** with specialization in Structural Engineering At the **DELHI TECHNOLOGICAL UNIVERSITY, DELHI** is an authentic record of the Research work carried out under my supervision. The content of this thesis, in whole or in Part, has not been submitted to any other institute or university for the award of a degree or Diploma.

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ACKNOWLEDGEMENT

First and foremost, praises and thanks to the God, the Almighty, for His showers of blessings Throughout my work to complete the research successfully.

I would like to express my sincere gratitude to my guide **Prof. G.P AWADHIYA** for enlightening me with the first glance of research, and for his patience, motivation, enthusiasm, and immense knowledge. His guidance helped me in all the time of research and writing of this thesis. I could not have imagined having a better advisor and mentor for my project work. It was a great privilege and honour to work and study under his guidance. I am extremely grateful for what he has offered me.

Besides my advisor I extend my sincere thanks to all faculties in **Structural Engineering** Department, DTU, DELHI for their timely co-operations during the project work.

Last but not the least; I would like to thank my **family**, for supporting me spiritually throughout my life and for their unconditional love, moral support and encouragement. So many people have contributed to my research work, and it is with great pleasure to take the opportunity to thank them. I apologize, if I have forgotten anyone.

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ABSTRACT

In recent past, severe earthquakes have caused substantial physical losses and casualties. Parts of India are in high risk of facing devastating earthquakes. Since a majority of the population is living in earthquake prone areas, it is probable that such terrible events may take place again in the near future. Moreover, it is not easy to cope with the substantial direct and indirect economic losses after each devastating earthquake for a developing country like India. Because in this country many reinforced concrete buildings are not designed according to the current building code, seismic behaviour is not taken into consideration during selection of the structural system and in most cases supervision in the construction phase is not adequate which in turn induces deficiencies like poor concrete quality, inadequate detailing of reinforcement etc. It is, therefore, vital to quantify the earthquake risk and to develop strategies for disaster mitigation. In order to achieve this goal, an extensive and inter-disciplinary study is required.

This study describes the methods by which it is possible to determine the vulnerability of existing engineering structures and building stock. The tool that is employed to assess the seismic performance of reinforced concrete frame structures is the fragility curve, By definition, fragility curves provide estimates for the probabilities of reaching or exceeding various limit states at given levels of ground shaking intensity for an individual structure or population of structures. A limit state, which is in the same terms as the response, usually represents a damage condition or a limitation of usage. The seismic vulnerability of these structures for different earthquake can be interpreted from the developed fragility curves.

CONTENTS

Chapter 4: RESULTS

Chapter 5: CONCLUSIONS

LIST OF NOTATIONS

D = Random variable describing the intensity of the demand on the system

Θ_y = The yield global drift ratio

Θ_u = The ultimate global drift ratio

V_y = Yield base shear capacity

Θ_{cp} = Collapse prevention performance limit.

M_s = Surface magnitude of earthquake

R = Epicentre distance

h = Hypocentre depth of earthquake

K = Stiffness matrix

C = Damping matrix

KL = Stiffness matrix for the linear elastic elements

M = Diagonal mass matrix

f_{ck} = Compressive strength of concrete

f_y= Yield strength of steel

IJ = Yield base shear co-efficient

LIST OF ABBREVIATIONS

LSF = Life Safety Failure

IDR = Inter-Story Drift Ratio

PBE = Performance Based Engineering

PSA = Probabilistic Safety Assessment

FC = Fragility Curve

LS = Limit States

I0 = Immediate Occupancy

LS = Life Safety

CP = Collapse Prevention

DC = Damage Control

SR = Limited Safety Range

FY = First Yield

PMI = Plastic Mechanism Initiation

SBM = Specific Barrier Model

PSHA = Probabilistic Seismic Hazard Analysis

PGA = Peak Ground Acceleration

PGV = Peak Ground Velocity

FNA = Fast Nonlinear Analysis

RCC = Reinforced Cement Concrete

FEMA = Federal Emergency Management Agency

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Chapter 1

INTRODUCTION

1.1 GENERAL

Recent studies demonstrated that even moderate earthquakes could be fatal in populated, unplanned cities. General public and the engineering community are now becoming more and more aware of the situation. However, neither the possible extents of seismic damage of existing buildings are known nor there is any guideline for their strengthening measures. Even the performance of the engineered buildings under a seismic event is questionable, as enough work has not yet been done in this field. In this study the prime objective is to present an appropriate method to assess the seismic performance of RCC structures.

The primary focus of the present study is to determine the exceedance probability of different damage states of structures under seismic excitation through fragility curves. From the fragility curves seismic damages of the structures can be evaluated. This damage estimation is a vital part of the seismic performance evaluation of buildings and other structures with respect to multiple performance objectives. In turn, the proper evaluation of seismic performance is essential for decision making involved in managing the risk to building, bridges, and other infrastructure in seismically active areas. Today, the earthquake engineering community faces new challenges that are brought about by the latest needs of the real estate development industries. The safety of buildings and other structures used to be the main concern of designers, owners, and regulators. The development of modern building codes has provided society with guidelines that serve well for achieving the required safety levels. However, nowadays other issues are becoming significant for owners and risk managers. Providing that safety requirements are met, the questions being asked now are "how much does it cost to repair?", "how long it wil be shutdown in case of the earthquake?" etc. These questions relate to the economic aspect of the seismic performance of real estate. Given the multiple performance objectives, accurate damage estimation becomes more important than ever.

Fragility functions are the essential tools for seismic loss estimation in built environments. They represent the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation (Shinozuka et al., 1999). The damage limit states in fragilities may be defined as global drift ratio (maximum roof drift normalized by the building height), inter-story drift ratio (maximum lateral displacement between two consecutive stories normalized by the story height), maximum roof displacement or story shear force etc. The ground motion intensities in the fragility functions can be spectral quantities, peak ground motion values, modified Mercalli scale etc. In this respect, fragility curves involve uncertainties associated with structural capacity, damage **limit** state definition and variability of ground motion intensity. Thus from fragility functions the seismic performance of any structure can be examined and its level of serviceability during an earthquake can be evaluated.

1.2 OBJECTIVE OF THE PRESENT STUDY

The objectives of the research are as follows:

a) To analyse the structure for seismic performance.

b) To construct fragility curves for a particular type of RCC building.

1.3 METHODOLOGY OF THE STUDY

There are two methods to estimate the seismic fragility of a specific building type. In the first method which is known as empirical method, the damage reports are usually utilized to establish the relation between the ground motion intensity and the damage state of each building. The second approach which is known as analytical procedure is to conduct the fragility studies by performing structural analysis to estimate the structural response to a ground motion in terms of internal forces and deformations. The advantage of this method is that it is simple and economically feasible. In addition, the nontechnical decision makers prefer such simple and rapid estimates of anticipated losses to develop the proper judgment to execute their mitigation plans. The first method requires past earthquake damage data. The second approach is thus considered as an appropriate way to estimate seismic fragility of the building. ETABS 2015 is used in this study to estimate quantitative damage limit states by conducting **nonlinear static analysis** of the developed representative models of the building stocks and also to perform nonlinear **time history analysis**. The time history plots of earthquake motions are simulated utilizing MCEER project (2004). Peak Ground Acceleration (PGA) is taken as ground motion intensity in this study. Using quantitative damagezlimit states the exceedance probability of a particular damage state are computed from the PGA versus maximum global drift scatters. The global drift percentiles greater than a given damage threshold level are computed by using the normal distribution to estimate the exceedance probabilities of the fragility curve sand the jaggedly varying exceedance probability points are then smoothened to develop fragility curves for that specific damage state.

1.4 ORGANIZATION OF THE STUDY

The study is organized according to the stages followed for the development of fragility curves for RCC frame structures in Bangladesh. Thus, **Chapter 1** introduces a general statement of the fragility functions, objective and methodology of this research. **Chapter 2** reviews the available literature that is required to understand the background theories of various aspects of fragility functions and seismic analyse of frame. This chapter also includes a literature survey on the different techniques used for constructing fragility curves and some recent research in fragility studies. **Chapter 3** describes the development of guide lines to construct fragility curves for RCC frame structures in context of Delhi and the methods of seismic analysis of an RCC frame according to IS code. **Chapter 4** presents the analysis results and development of fragility curves for a particular type of RCC frame structure. Finally **Chapter 5** draws conclusion of the current work.

Chapter 2

LITERATURE REVIEW

Seismic performance of buildings and other structures is a vital characteristic for all agents that are involved in operations with real estate located in seismically affected areas. How well a particular building will perform during an earthquake at some point in the future is important because it affects the present value of the property. In particular, at any present time, a real estate owner can face a set of seismic risk management options to choose from: do nothing, sell the property, perform seismic retrofit or buy earthquake insurance. Likewise, a potential owner (a person who wants to buy a real estate property) faces similar choices: do not buy, buy and do nothing, buy and retrofit, buy and insure.

The process of making a choice between several alternatives can be analysed by decision theory. Here a simple procedure of formal decision making process is outlined. This analysis does not consider uncertainty in the outcomes or risk preferences of decision makers. The general approach of decision theory states that the best choice is the one that gives the highest utility among different options (for details about utility and decision theory, Resnik 1987). Calculation of utilities for different options depends on the decision maker's objectives and preferences. When applying this concept to the case of a real estate owner or a buyer, usually the most prevalent concern is safety. In terms of decision theory, this means that the higher the safety of some option, the higher is its utility, meaning that utility is the increasing function of safety. Normally, it suffices to use a very simplistic utility function to account for the matter of safety. It is convenient to utilize a step function like one shown in the Figure 2.1 Such function basically states that any option with the safety less than some acceptable level should be rejected. When the safety is higher than Sac, the utility is constant, implying that there is no marginal benefit from increasing safety beyond the acceptable level. This situation reflects an approach of real estate owners, where Sacre presents the safety level provided by modem building codes. Alternatively, for some owners, the acceptable level of safety is the one that meets minimum legal requirements. In both cases, once the safety requirement is satisfied, he or she does not care if the safety level is significantly higher than Sacre just barely exceeds the threshold value.

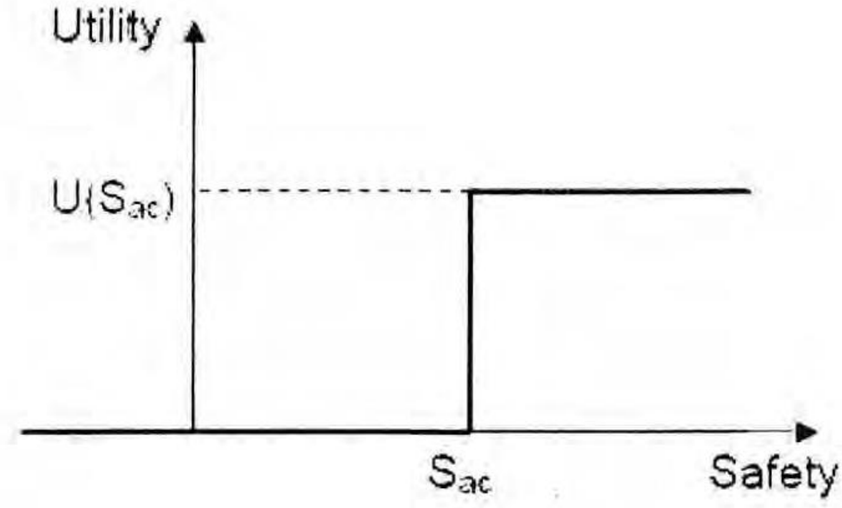


Figure 2.1: Example of utility function for decision making based on safety.

2.1 OVERVIEW OF STRUCTURAL RELIABILITY THEORY FOR SEISMIC SAFETY

Structural reliability theory for analysis of seismic safety usually does not directly consider such safety measures as expected number of lost lives. Instead, it deals with the events that can be directly related to the deaths caused by an earthquake. Such events are usually referred to as life safety failure (LSF). Two examples of LSF are total structural collapse and partial structural collapse. The problem of interest for practical applications is finding the probability of LSF. In general, this probability can be calculated according to the following probability integral,

$$\text{P(LSF)=} \quad \oint fQ, x^3(q, x^3)\, dqdx$$

where Q is a vector of random variables that fully define the seismic excitation (ground acceleration time history is commonly used); x^3 is a vector of random variables defining the values of all relevant structural properties; q and x^3 are particular values of the random vectors Q and x^3 ; respectively; $fQ, x^3(q, x^3)$ is the joint probability density function of random vectors Q and x^3 over failure region comprising all the values of Q and x^3 for which LSF occurs.

For convenience of calculation, the failure region is usually given a mathematical description as follows. Define a function $g(q, x^3)$ in such a way that it possesses the following property

$$g(q, x^3) < 0$$

meaning that region of negative values of $g(q, x^3)$ coincides with the failure region Function, $g(q, x^3)$ is called a limit-state function for the LSF.

Limit-state functions can be defined in a number of ways. One example is to define it in terms of maximum inter-story drift ratio (IDR)

$$\text{IDR}= d_1 - d_m (q, x^3)$$

where *dm* is the maximum IDR resulting from a particular earthquake excitation q applied to a structure with properties , x^3; d_1 is a chosen threshold value. This limit state function implies that life safety failure occurs once the threshold value is exceeded: d_1. > *dm* Therefore, this approach assumes that it is likely that the structure undergoes partial or complete collapse once the maximum IDR exceeds the threshold value. The choice of threshold value depends on a structure type and may be based on experimental or field observations.

Evaluation of integral is not a trivial task because vectors Q and x^3 can contain up to several thousand variables and calculation of the function $g(q, x^3)$ is often computationally expensive because it involves a nonlinear structural analysis.

2.2 PERFORMANCE BASED ENGINEERING (PBE)

Performance-based engineering (PBE) is a new paradigm for seismic risk reduction across regions or interconnected systems (Abrams, 2002). [n PBE, the risk to a distributed infrastructure systems is quantified, evaluated and managed through an assessment and selective intervention process aimed at selected components of that system. This process enables the benefits of alternate seismic risk mitigation strategies to be assessed in terms of their impact on the performance of the built environment during a spectrum of earthquake hazards and on the affected population. It is clear that components and systems that are dominant contributors to risk should receive the focus of attention in the assessment process underlying PBE. These dominant contributors can be identified through the formalism of a probabilistic safety assessment, or PSA.

A PSA is a structured framework for evaluating uncertainty, performance and reliability of an engineered system, and accordingly must play a central role in PBE. It is distinguished from traditional deterministic approaches to safety assurance by its focus on why and how the system might fail and by its explicit treatment of uncertainties, both in the phenomena and in the analytical tools used to model them. A PSA provides a basis for decision-making in the presence of uncertainty that can be scrutinized by the stakeholders of the project, audited independently by a building official or other regulatory authority, and updated periodically as circumstances warrant. The move toward quantitative risk assessment began in the nuclear industry in the mid-1970"s, and has accelerated in recent years as the benefits of quantitative risk analysis have become apparent in many fields (Ellingwood, 1999).

One begins the PSA process by identifying limit states (LS), or conditions in which the system ceases to perform its intended functions in some way. In a (narrow) structural engineering sense, such limit states for specific structural components and systems may be either strength or deformation-related (as discussed subsequently). In a broader socioeconomic context, the LS may be related to repair costs (e.g., expressed as a percentage of replacement value) that are in excess of a desired amount, opportunity losses, or morbidity/mortality. Limit state identification requires a thorough understanding of the behaviour of the safety-related systems within the plant and the role of structural components and systems in ensuring acceptable behaviour of such systems. With the limit states identified, the limit state probability can be expressed as,

$$\mathbf{P[LS] = \sum P[LS|D = d]\; P[D = d]}$$

In which D is a random variable (or random vector) describing the intensity of the demand on the system, and P[LS|D = d] is the conditional limit state probability, given that D = d, and the summation is taken over all possible values of D. The probability P[D = d] defines the hazard. The variable d is denoted the "control" or “interface" variable. The conditional probability, P[LS|D = d] = FR(x), is the **fragility**. The fragility of a component or system defines the conditional probability of its attaining a performance limit state, which may range from loss of function to incipient collapse, given the occurrence of a particular operational or environmental demand Shows that assessment of structural fragility is a key ingredient of any PSA. Furthermore, fragility function provides a probabilistic measure of safety margin with respect to design-basis or other events specified by a stakeholder. Such a margin can be used to evaluate system weaknesses or deficiencies identified during an inspection or condition assessment and can provide a means to assess if the observed weaknesses or deficiencies might be expected to have a significant impact on system risk. Modelling and engineering analysis provide a measure of response to a prescribed demand. For example, structural analysis of a building for an ensemble of ground motions, characterized by median peak ground acceleration, yields a corresponding set of deformations. Those deformations are uncertain, due to uncertainties in the ground motion as well as the dynamic properties describing the structure and the structural modelling process itself. In turn, those deformations give rise to various states of damage and potential economic loss to structural and non-structural components and systems. Those losses also are uncertain, due to uncertainties in the deformations, resulting damage, and the economic models used to model costs associated with different damage states.

2.3 FRAGILITY CURVE

As noted above, fragility (or vulnerability) can be described in terms of the conditional probability of a system reaching a prescribed limit state (LS) for a given system demand D = d, P(LS/D = d). Limit states related to structural behaviour range from un-serviceability to various degrees of damage including incipient collapse. Demands can be in the form of maximum force, displacement caused by earthquake ground motions, or more generally a prescribed intensity measure of the ground motion, over a given period of time. Expressed in this general manner, the fragility (or vulnerability) is a function of the system capacity against each limit state as well as the uncertainty in the capacity. The capacity controls the central location of the Fragility Curve (FC) and the uncertainty in the capacity controls the shape (or dispersion) of the FC. For a deterministic system with no capacity uncertainty, the FC is a step function. Strictly speaking, FC is primarily a property of the system dependent on the limit state. A fragility analysis is an essential ingredient of the fully coupled risk analysis embodied in It also can be used to determine probabilistic safety margins against specific identified events for decision purposes. Identification of probabilistic safety margins is central to modern engineered facility risk management. Although providing a less informative measure of safety than that obtained from the fully coupled risk analysis. Risk-informed decision-making based on the results of fragility assessment has several advantages:

(1) The probabilistic system analysis is effectively uncoupled from the hazard analysis. Thus, while knowledge of the hazard is useful in identifying appropriate events for risk assessment purposes (e.g., a 2,475yr mean recurrence interval earthquake), such knowledge is not essential. Absent credible data on such events, one might simply inquire as to the fragility were the design-basis event to be exceeded by some arbitrary margin, say 50 percent.

(2) The need to interpret and defend very small limit state is avoided. There are limited data to support probabilities of this level, and such estimates are highly dependent on the probabilistic models selected. At the current state-of-the-art,(conditional) fragilities are more robust than unconditional limit state probabilities.

(3) A properly conducted fragility analysis is less complex, less costly, and involves fewer disciplines than a fully coupled risk analysis. Accordingly, there is less likelihood of miscommunication among members of the risk analysis team and the results are more easily understood by a non-specialist stakeholder or decision-maker.

2.4 FRAGILITY CURVE

To tie the vulnerability of a given system to the seismicity of the region, the seismic hazard needs to be included in the consideration. The vulnerability needs to be described in terms of probability of a set of given limit states being reached of a system at a given location over a given period of time (0, t). Knowing the fragility curve, the limit state (LS) probability over the time period (0, t) can be evaluated.

2.4 IDENTIFICATION OF IMPORTANT LIMIT STATE

Performance levels or limit states for both structural and non-structural systems are defined in this document as the point in which the system is no longer capable of satisfying a desired function. There are many types of performance levels in the field of earthquake engineering. In addition, performance levels can be identified by qualitative and quantitative approaches. Both methods are summarized in the following sections.

2.5 TRADITIONAL QUALITATIVE APPROACHES

Qualitative approaches for identification of performance levels have traditionally been used in building codes. In particular, most building codes require designers to ensure life safety of the occupants during factored loading and serviceability or functionality during un-factored loading. FEMA 273, and its update FEMA 356, has the most comprehensive documentation on performance levels that are defined qualitatively and is briefly summarized below. FEMA 273/356 defines performance levels related to the structural system as:

2.5.1 Immediate Occupancy (IO)

Occupants are allowed immediate access into the structure following the earthquake and the pre-earthquake design strength and stiffness are retained.

2.5.2 Life Safety (LS)

Building occupants are protected from loss of life with a significant margin against the onset of partial or total structural collapse.

2.5.3 Collapse Prevention (CP)

Building continues to support gravity loading, but retains no margin against collapse load.

In addition to the discrete structural performance levels, FEMA 273/356 also defines structural performance ranges such as:

A. **Damage Control (DC)**

   Range of structural damage between immediate occupancy and life safety.

B. **Limited Safety Range (SR)**

   Range of structural damage between life safety and collapse prevention;

FEMA 273/356 also defines non-structural performance levels as:

(1) **Operational**

   Non-structural components are able to function as prior to the earthquake;

(2) **Immediate Occupancy**

   Building access and life safety systems generally remain available and operable.

(3) Life Safety

non-structural damage that is not life threatening.

(4) Hazard Reduced Range

Damage that includes potentially falling hazards, but high hazard components are secured and will not fall. Preservation of egress, fire suppression systems, and other life safety issues are not ensured;

In terms of identifying overall building performance levels, FEMA 273/356 utilizes both definitions of structural and non-structural performance levels. It is important to note that these traditional performance level definitions are based on qualitative definitions. For illustration purposes, FEMA 273/356 presents inter-story drift values that are typical for each structural performance level for the different types of structural systems in use. For example in reinforced concrete Tame structures, inter story deformations of 1%, 2%, and 4% of the story height may be acceptable for 10,LS, and CP, respectively. However, it is clear that deformation limits will depend on a variety of variables that include: degree of section confinement and detailing, level of axial column load (P-delta effect), non-structural participation, pre-existing damage etc.

2.6 Qualitative Approaches

Although current building codes and state-of-the-art publications have attempted to define the various performance levels for structural and non-structural systems, performance levels have only been identified qualitatively. Therefore, designers have to determine quantitative response limits that correspond to the qualitative code descriptions. Another approach for defining structural performance levels might be based on quantitative procedures using nonlinear pushover techniques. These quantitative performance levels can be utilized by the designer and judged to supersede the qualitative performance levels in current building codes. Wen Y.K., Ellingwood B.R., Bracci J., (2004) suggested following performance levels that can be identified analytically using nonlinear pushover procedures are:

(1) First Yield (FY) - Inter-story deformation at which a member of a story initiates yielding under imposed lateral loading.

(2) Plastic Mechanism Initiation (PMI) - Inter-story deformation at which a story mechanism initiates under imposed lateral loading.

For example, consider the portal frame in Figure 2.2. Under imposed lateral loading, the story shear force versus inter-story deformation can be calculated using pushover techniques and hypothetically shown in Figure 2.2. The FY performance level corresponds to an inter-story deformation at first member section yielding, shown at the base of the columns. The PMI performance level subsequently occurs after both ends of the beam yield. It is important to note that the sequence and form of member yielding during applied loading prior to the mechanism formation. Both can have significant effects on the levels of structural deformability (capacity) in building structures. A key input parameter required in identifying such quantitative performance levels is the imposed lateral loading or deformations. Since multi-story buildings are susceptible to high mode response and impulse-type loading during earthquakes, loading patterns that have been typically used for determining structural demands as in current building codes may not be appropriate for identifying performance levels, which are capacities. As such, the imposed lateral loading or deformation should be consistent with those that have the most critical consequence. Figure 2.3 shows the deformation pattern in a framed structure during inverted triangular lateral loading (similar to loading proportional to the fundamental mode shape of the structure) and during loading that might be critical for the second story of the building.

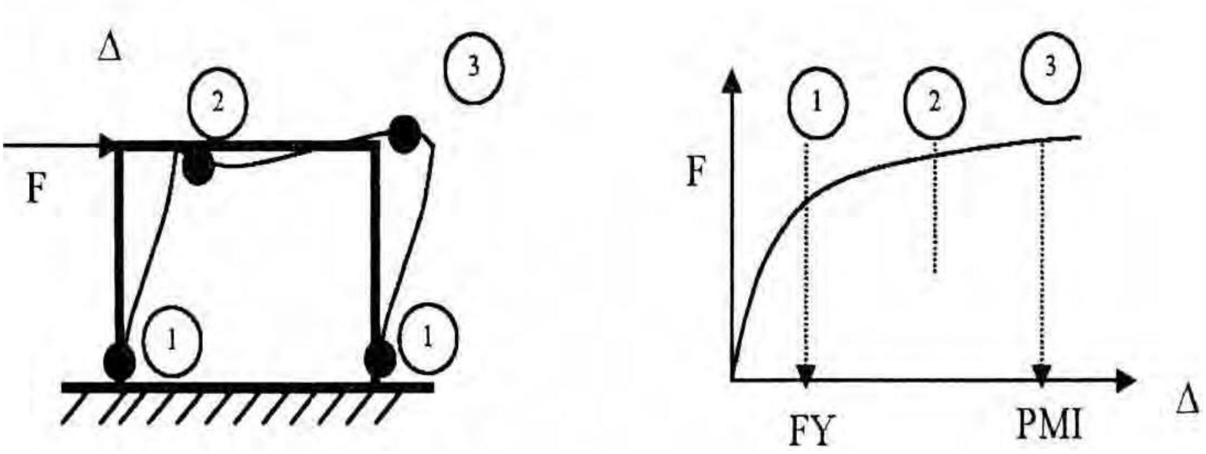


Figure 2.2: Pushover analysis and yield formation

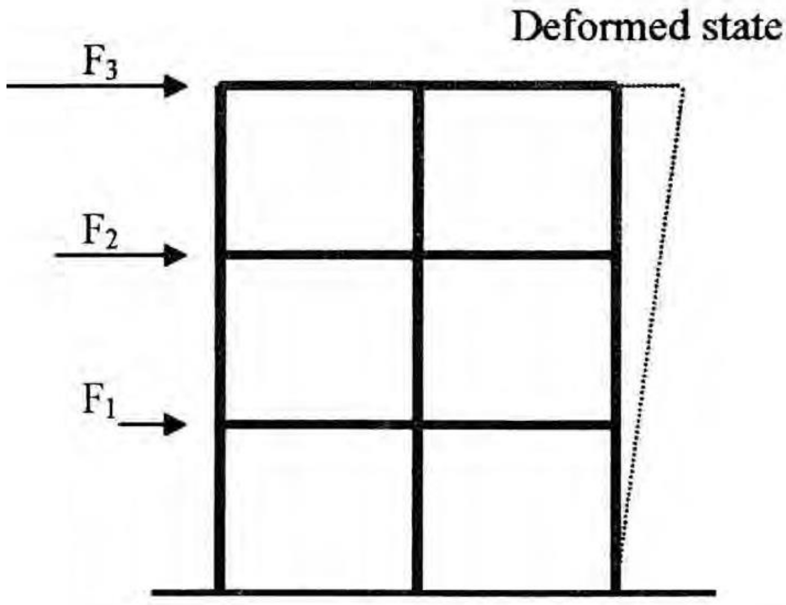


Figure 2.3: Loading patterns for pushover analysis

Akkar et aI., (2004) suggested another approach to identify quantitative damage state. According to this study the capacity curve from pushover analysis of each model can be approximated with a bilinear curve using the guidelines given in FEMA-356(ASCE, 2000). A typical idealization of a capacity curve is shown in Figure 2.4 It is required to specify the yield and ultimate strength capacities and their associated global drift values for constructing the approximate bilinear capacity curve. The global drifts can be used to represent the damage limit states of the buildings. The yield global drift ratio Θ_y represents significant yielding of the system when the yield base shear capacity (V_y) of the building is attained whereas the ultimate global drift ratio Θ_u corresponds to the state at which the building reaches its deformation capacity. The base shear coefficient n= V_y */W* in Figure 2.5 is the ratio of yield base shear capacity to the building weight.

It should be noted that there is no universal consensus on how to approximate a capacity curve with a bilinear force-deformation representation. An initial stiffness targeting at the state of significant global yielding may lead to considerable variations in V_y and Θ_y because there is no specific point all the capacity curve exactly describing significant yielding (Sullivan et at., 2004).

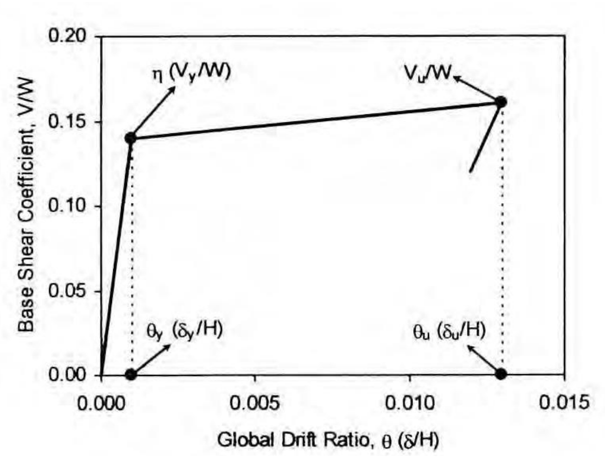


Figure 2.4: A typical bilinear capacity curve

Representative probability density functions of Θ_y and Θ_u can be determined in terms of mean, median and standard deviation. When global ductility capacities *($\Theta_u l \Theta_y$)* are calculated both Θ_y and Θ_u can be utilized to determine deformation capacities. It is more appropriate to employ Θ_u in assessing the deformation capacities of such buildings, which have infill walls or short span length (Akkar et aI., 2004).

Three performance limits, immediate occupancy, life safety and collapse prevention that are specified in several other international guidelines are usually adopted in fragility studies. Akkar et aI., (2004) suggested that the collapse prevention performance limit Θ_{cp} is taken as the 50 percent to 75 percent of the median Θ_u computed depending on the construction quality, level of confidence on proper design and detailing and uncertainty in modelling. The selected performance limits by Akkaret aI., (2004) are described in Table 2.1. These limits states are quantitative and conjectural and could be argued as subjective.

Performance Level	**Limit State**
Collapse Prevention (Severe Damage)	$\mathbf{\Theta = \Theta_{cp} = 0.5\Theta_u}$
Life safety (Moderate Damage)	$\mathbf{\Theta \leq 0.5\ \Theta_{cp}}$
Immediate occupancy (Light Damage)	$\mathbf{\Theta = 0.8\ \Theta_y}$

Table 2.1: Assumed drift ratio limits for performance levels

2.7 NONLINEAR STATIC ANALYSIS

Nonlinear static analysis is also known as pushover analysis. Although nonlinear static analysis has not previously been included in design provisions for new building construction, the procedure itself is not new and has been used for many years in both research and design applications. For example, nonlinear static analysis has been used for many years as a standard methodology in the design of offshore platform structures. It also has been adopted in several standard methodologies for the seismic evaluation and retrofit of building structures, including the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273) and Methodologies for Post-earthquake Evaluation and Repair of Concrete and Masonry Buildings (ATC 40).Nonlinear static analysis also forms the basis for earthquake loss estimation procedures contained in HAZUS, FEMA's nationally applicable earthquake loss estimation model. Finally, although it does not explicitly appear in the NEHRP Recommended Provisions, the nonlinear static analysis methodology forms the basis for the equivalent lateral force procedures contained in the provisions for base isolated structures and proposed for inclusion for energy-dissipated structures.

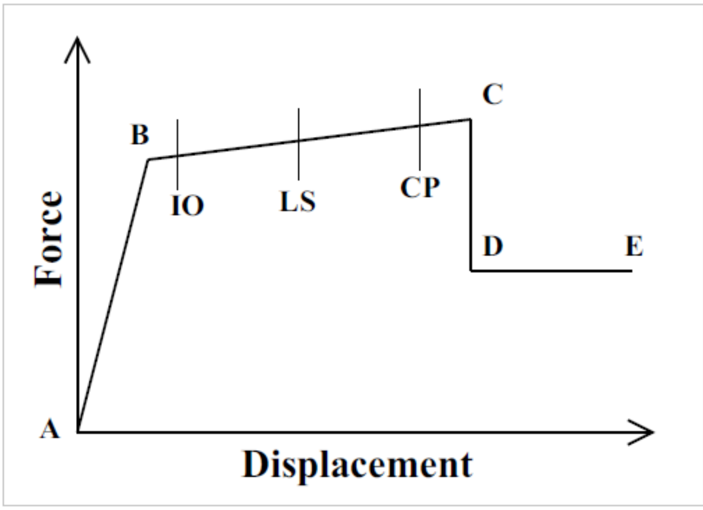
One of the key controversies surrounding the introduction of this methodology into the provisions relates to the determination of the limit deformation, sometimes also called a target displacement. Several methodologies for estimating the amount of deformation induced in a structure by the design earthquake have been proposed and are included in various adoptions of the procedure.

Nonlinear static analysis provides a simplified method of directly evaluating nonlinear response of structures to strong earthquake ground shaking that can be an attractive alternative to the more complex procedures of nonlinear response history analysis. It is hoped that exposure of this approach through inclusion in this Structural Design Criteria.

A nonlinear static analysis shall consist of an analysis of a mathematical model of the structure that directly accounts for the nonlinear behaviour of the structure's components under an incrementally increased pattern of lateral forces. In this procedure a certain mathematical model of the structure is incrementally displaced to a target displacement through application of a series of lateral forces or until the structure collapses and the resulting internal forces, QEj, and member deformations,(Yt), at each increment of loading are determined. At the target displacement for the structure, the resulting internal forces and deflections should be less than the capacity of each element calculated according to the applicable acceptance criteria in Sec. 2.7.3 of FEMA 273. The analysis shall be performed in accordance with this section.

The analysis procedure is intended to provide a simplified approach for directly determining the nonlinear response behaviour of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism and initiation of collapse. Response behaviour is gauged through measurement of the strength of the structure, at various increments of lateral displacement. The strength is measured by the shear forces resisted by a structure in the form of lateral forces, which cause the lateral deformations.

Usually the shear resisted by the system when the first element yields in the structure, although not always relevant for the entire structure, is defined as the "elastic strength." When traditional linear methods of design are used, together with R factors, the value of the design base shear sets the minimum strength at which this elastic strength point can occur. If a structure is subjected to lateral loads larger than represented by the elastic strength, then a number of elements will yield, eventually forming a mechanism. For most structures, multiple configurations of mechanisms are possible. The mechanism caused by the smallest set of forces is likely to appear before others do. That mechanism is considered to be the dominant mechanism. Standard methods of plastic or "limit" analysis can be used to determine the strength corresponding to such mechanisms.



If after the structure develops a mechanism it deforms an additional substantial amount, elements within the structure may fail, fracture, or buckle, etc., losing their strength contribution to the whole structural system. In such case, the strength of the structure will diminish with increasing deformation. If any essential element, or group of elements, fails, then the entire structure may loose capacity to carry the gravity loads, or any lateral load. This condition can also occur if the lateral deformation becomes so great that the P-delta effects exceed the residual lateral strength of the structure. Such conditions are defined as collapse and the deformation associated with collapse defined as the "ultimate deformation." This deformation can be determined by the nonlinear static procedure and also by plastic or limit analysis. Many structures exhibit a range of behaviour between the development of first yielding and development of a mechanism. When the structure deforms while elements are yielding sequentially (shown as progressive yielding), the relation between external forces and deformations cannot be determined by simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static analysis is to provide a simplified method of determining structural response behaviour at deformation levels intermediate to those which can be conveniently analysed using limit state methods.

2.8 Modal Time-History Analysis

Modal superposition provides a highly efficient and accurate procedure for performing time-history analysis. Closed-form integration of the modal equations issued to compute the response, as summing linear variation of the time functions, $f_i(t)$,between the input data time points. Therefore, numerical instability problems are never encountered, and the time increment may be any sampling value that is deemed fine enough to capture the maximum response values. One-tenth of the time period of the highest mode is usually recommended; however, a larger value may give an equally accurate sampling if the contribution of the higher modes is small.

The modes used are computed in a Modal Analysis Case that can be the un-damped free vibration Modes (Eigen vectors) or the load-dependent Ritz-vector Modes.

If all of the spatial load vectors pi, are used as starting load vectors for Ritz-vector analysis, then the Ritz vectors will always produce more accurate results than if the same number of eigenvectors is used. Since the Ritz-vector algorithm is faster than the Eigen vector algorithm, the former is recommended for time-history analyses.

It has to be determined that if the Modes calculated by the program are adequate to represent the time-history response to the applied load. It has to be checked:

• That enough Modes have been computed

• That the Modes cover an adequate frequency range

• That the dynamic load (mass) participation mass ratios are adequate for the load cases and/or Acceleration Loads being applied

• That the modes shapes adequately represent all desired deformations.

2.9 Nonlinearity

The following types of nonlinearity are available in ETABS 2015:

• Material nonlinearity

Various types of nonlinear properties in Link/Support elements

Tension and/or compression limits in Frame elements

Plastic hinges in Frame elements

• Geometric nonlinearity

P-delta effects

Large displacement effects

For nonlinear direct-integration time-history analysis, all of the available

Nonlinearities may be considered.

For nonlinear modal time-history analysis, only the nonlinear behaviour of the Link/Support elements is included. If the modes used for this analysis were computed using the stiffness from the end of a nonlinear analysis, all other types of nonlinearities are locked into the state that existed at the end of that nonlinear analysis.

2.10 NORMAL PROBABILITY DISTRIBUTION AND SAMPLING TECHNIQUES

Normal probability distribution is a very important continuous probability distribution. It consists of an infinite number of possible values within a specified range. The normal probability distribution and its accompanying normal curve have the following characteristics:

1. The normal curve is bell-shaped and has a single peak at the centre of the distribution. The arithmetic mean, median and mode of the distribution are equal and located at the peak. Thus, half the area under the curve is above this canter point and the other half is below it.

2. The normal probability distribution is symmetrical about its mean.

3. The normal curve falls off smoothly in either direction from the central value.

Probability sample is defined as a sample is selected in such a way that each item in the population has a known likelihood of being included in the sample. There are three methods of probability sampling techniques,

a. Simple Random Sampling
b. Systematic Random Sampling
c. Stratified Random Sampling

Simple Random Sampling

A sample selected so that each item or person in the population has the same chance of being included. For this purpose an identification number for each item in the population and a table of random numbers are used.

Systematic Random Sampling

In this process the items or individuals of the population are arranged in some way - alphabetically, in a file drawer by date received, or by some other method. A random sampling point is selected, and then every $\mathbf{k^{th}}$ member of the population is selected for the sample.

Stratified Random Sampling

A population is divided into subgroups, called strata, and a sample is selected from each stratum.

All these methods described above are the techniques for selecting unbiased samples from a given population. Unbiased sampling is essential for randomness of the collected samples. In the present study simple random sampling technique is used using table of random numbers.

2.10 RECENT WORKS ON FRAGILITY CURVE

The seismic fragility curves for RCC frame structures particularly for buildings and bridges have been studied and developed by a number of researchers. Some of the developed fragility curves are shown in the following part of this section.

Akkar et aI., (2004) in his study developed the fragility curves for four different types of RCC buildings in Turkey. Here light, moderate and severe limit states are I0, LS and CP respectively. These fragility curves are shown in Figure 2.5

Erberik et ai, (2005) in Turkey also developed fragility curves (Figure 2.5) for midrise in filled frames in terms of PGV and PGA both. Wen et aI, (2004) in MAE Canter Project DS-4 Report developed the fragility curve for a particular type of RCC frame building in terms of both FEMA and quantitative limit states. These are shown in Figure 2.6.

Shinozuka et aI, (2001) developed fragility curves (Figure 2.6) for multi-span RCC bridges. In this study five quantitative damage states are developed and utilized for defining limit states for fragility analysis. Damage states are shown in Table 2.5.

Fragility curves for different type of RCC buildings and bridges are also developed by a number of researchers. Ventuea et ai, (2001) estimated seismic loss in south western British Columbia based on fragility curves. Simiu et aI, (2002) developed fragility curves for RCC buildings for wind induced loss estimation. Loh et ai, (2002) conducted research on fragility of highway bridges in Taiwan. Shinozuka et ai, (2001) performed study on statistical analysis of fragility curves for RCC bridges and developed methodologies for constructing both empirical and analytical fragility curves for bridges

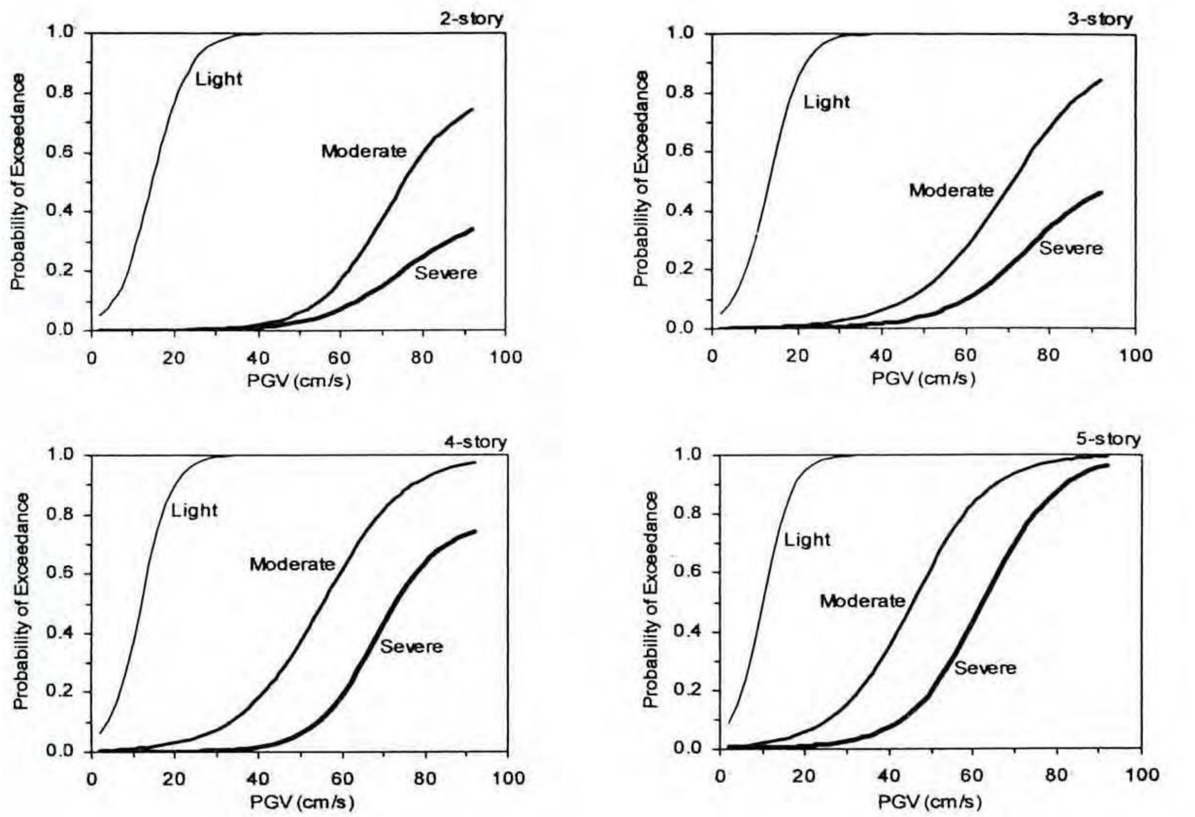


Figure 2.14: Fragility curves for (a) 2-, (b) 3-, (c) 4- and (d) 5-story buildings (after Akkar et al., 2004)

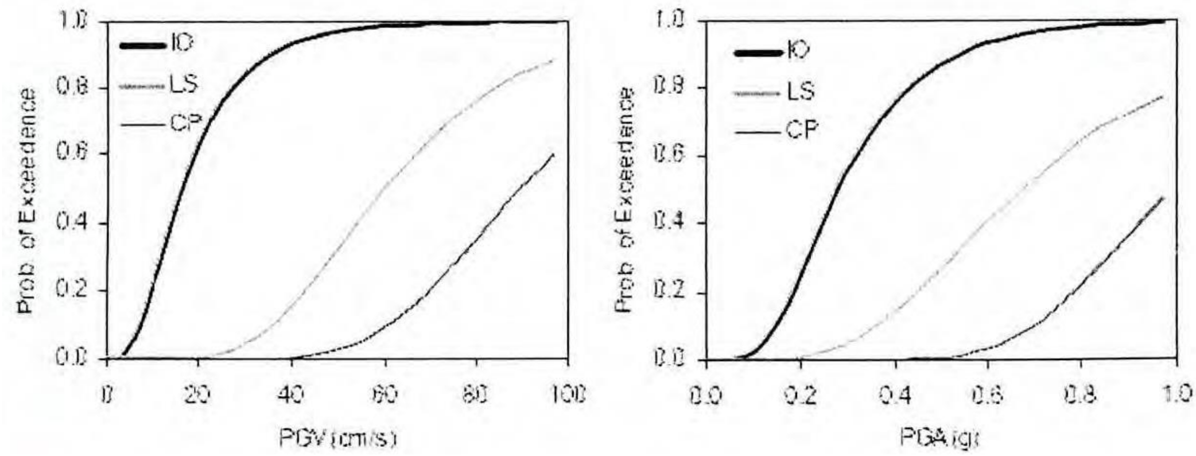


Figure 2.5: Sample fragility curves for mid-rise in filled frames in terms of a) PGV, b)PGA

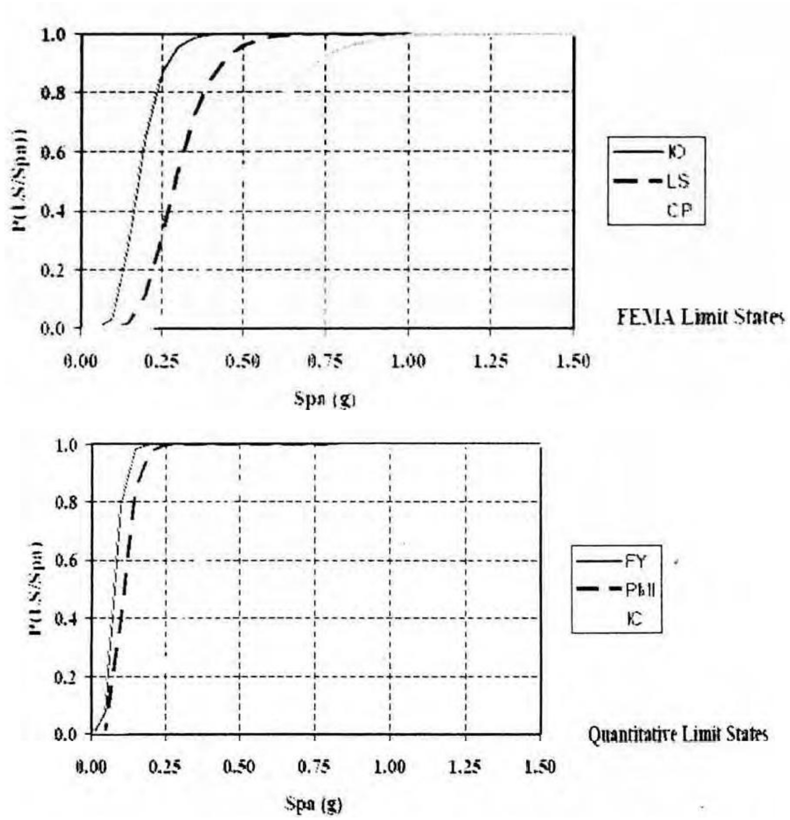


Figure 2.6: Sample fragility curves for both FEMA and Quantitative Limit States. (after Wen et aI., 2004)

Damage State	**Description**	**Ductility Demand**
1. No Damage	First Yield (θ_y)	1.00
2. Slight Damage	Cracking, Spalling	2.01
3. Moderate Damage	Loss of Anchorage	6.03
4. Extensive Damage	Incipient Pier Collapse	11.07
5. Complete Collapse	Pier Collapse	23.65

Table 2.4: Five quantitative damage state for multi-span RCC bridges (Shinozuka et al,2001)

2.12 **CONCLUDING REMARKS**

This chapter summarizes all the background theories in a brief but in explanatorily form that is required to construct fragility curves for RCC frame structures. It is well and widely recognized nowadays that fragility curves are extremely vital and useful tool for seismic loss estimation. So importance of fragility curves for the vulnerable RCC structures of our country is immense. This chapter also includes some examples of fragility curves that are constructed recently in various parts of the world for RCC buildings and bridges.

Chapter 3

METHODOLOGY AND PROCEDURE

One of the objectives of this work is to establish a guideline to quantify the vulnerability of reinforced concrete (RC) frame structures, specifically in Delhi due to potential earthquakes. The seismic vulnerability of such construction is described by means of fragility curves, which relate the probability of exceeding a particular limit state given an imposed seismic demand. In this work, seismic demand is defined as the peak ground acceleration of a particular earthquake.

3.1 REINFORCED CONCRETE FRAME STRUCTURES

Low to mid-rise RC frame buildings located in this region historically considered of low to moderate seismic risk were typically designed without consideration of lateral loading, since wind load seldom governed for low-rise construction. Therefore, such structures have been categorized as gravity load designed, or GLD structures (Bracciet at el., 1995a). In general, GLD RC frame structures have no special reinforcing details in the beam, column, and joint regions (El-Attar, 1997, Pessiki, 1990, Aycardi, 1994,and Bracci, 1995a). Another characteristic that distinguishes these structures from others designed in areas of higher seismic risk is the existence of strong beams and weak columns, which can lead to soft story failure mechanisms that are composed primarily of column hinging. The lack of sufficient column strength leads to column hinging at relatively low lateral loads, causing the formation of a story mechanism once all columns located on one story have hinged. Once the mechanism develops, the building's resistance is provided solely by the post-yield strength of the hinging column ends and inherent section ductility. Combining the lack of sufficient column strength with the lack of sufficient detailing in column sections for ductility, brittle soft story failure mechanisms may be prominent during strong earthquakes.

3.2 METHODS TO DETERMINE SEISMIC VULNERABILITY OF A STRUCTURE

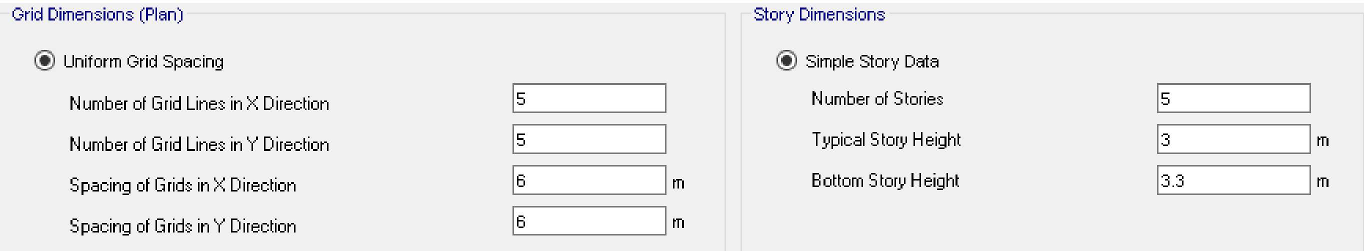
To estimate the seismic vulnerability of a specific building type, two different approaches can be considered. In the first approach, each building stock is examined individually and the vulnerability of the building stock is obtained by combining the fragility information associated with each building. Very detail modelling and analysis procedures are employed; hence the result will be highly accurate. On the other hand, this approach is practically and economically unfeasible. The second approach is to conduct the fragility studies by using the statistical properties of the building population. Simple models and methods are employed in this approach. The advantage of this method is that it is simple and economically feasible. In addition, the nontechnical decision makers prefer such simple and rapid estimates of anticipated losses to develop the proper judgment to execute their mitigation plans. However, the obtained results will be crude and the limitations of the models or the methods should be carefully understood.

3.3 CAPACITY UNCERTAINTY

The member and system capacity depend directly on the material strengths and stiffness, which are inherently random. The randomness can be modeled by random variable based on test data. It is common to use the first two moments, ie. the mean and standard deviation (or coefficient of variation), to describe the central value and the variability. Normal, lognormal or Weibull distributions are commonly used for convenience. The actual strength of the material of a given member generally differs, in some cases significantly, from the nominal values used in member capacity calculations during design. The relation between the nominal value and the actual value therefore needs to be established to estimate the real member capacity. The strength variability obviously depends on the material, manufacturing process, and sometimes the testing protocol. Material property variability and test data up to 1980can be found in the report by Ellingwood et al (1980). For example, the coefficient of variation of strength of timber varies in the range from 10 % to 30 % depending on species and in flexure or compression; and that of masonry walls from 10 % to 26 % depending on configuration and in compression or flexure. The coefficient of variation of compressive and tensile strength of concrete is around 18 % and that of the yielding strength of steel reinforcement and steel rolled shapes is around 10 % or less. Properties of construction material such as concrete and structural steel evolve over time. This variation in properties also country specific and varies in different countries and even in different region within the same country. Strength statistics of newer material such as high-strength steel and concrete may be found in more recent literature. For example, statistics on yield and ultimate strength of structural steel under various environmental conditions can be found in the recent FEMA/SAC report (2001).

3.4 DEVELOPMENT OR SELECTION OF REPRESENTATIVE MODELS

A five storey representative model has been created in ETABS 2015 and details for the same are as mention below:



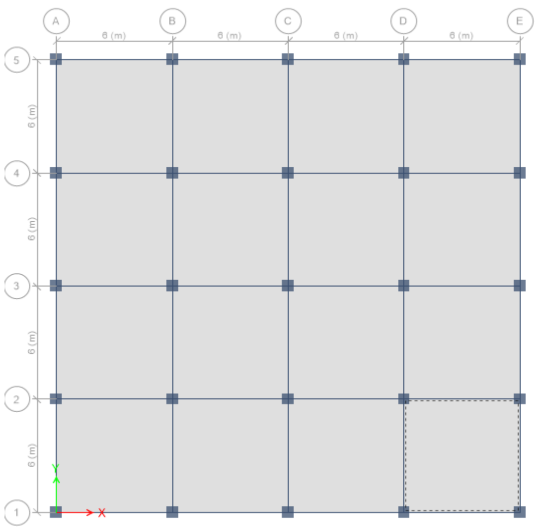


FIG 3.1 PLAN OF THE RCC FRAME

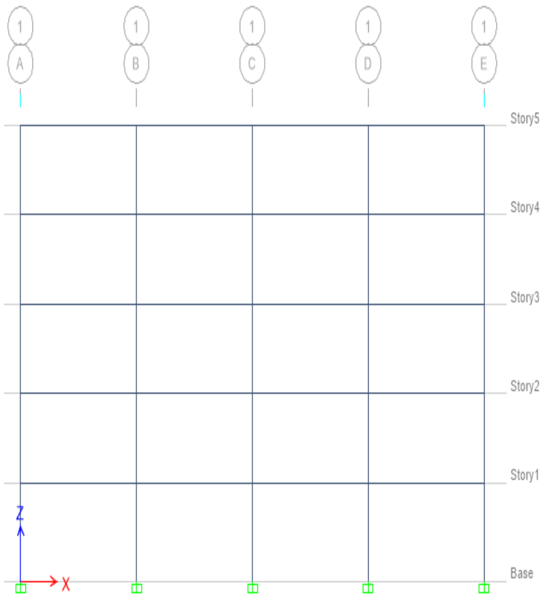


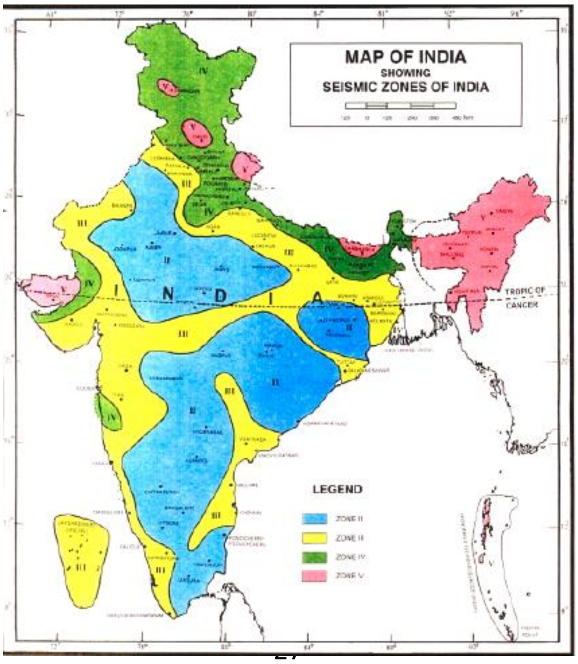
FIG 3.2 ELEVATION OF THE RCC FRAME

3.5 SPECIFICATIONS OF THE MODEL

Beam size(mm)	**300*600**
Column size(mm)	**600*600**
Slab thickness(mm)	**125**
Dead load(Kn/m^2)	**1**
Live load(Kn/m^2)	**2**
Wall load(Kn/m)	**5.25**
Density of Rcc(Kn/m^3)	**25**
Height of each floor(m)	**3**

3.6 IS 1893 CODAL PROVISION

Earthquake zone(Delhi)	**iv**
Damping ratio	**5%**
Importance factor (table 6)	**1**
Type of soil	**ii**
Response reduction factor(table 7)	**5**
Type of structure	**Special moment resisting frame**



3.6 IDENTIFICATION OF IMPORTANT LIMIT STATE

Performance levels or limit states for both structural and non-structural systems are defined as the point in which the system is no longer capable of satisfying a desired function. There are many types of performance levels in the field of earthquake engineering. In addition, performance levels can be identified by qualitative and quantitative approaches. Both methods are summarized below.

3.6.1 Traditional Qualitative Approaches

Qualitative approaches for identification of performance levels have traditionally been used in building codes. In particular, most building codes require designers to ensure life safety of the occupants during factored loading and serviceability or functionality during un-factored loading. FEMA 356 has the most comprehensive documentation on performance levels that are defined qualitatively and is briefly summarized below. FEMA 356 defines performance levels related to the structural system as:

1. **Immediate Occupancy (IO)**

Occupants are allowed immediate access into the structure following the earthquake and the pre-earthquake design strength and stiffness are retained.

2. Life Safety (LS)

Building occupants are protected from loss of life with a significant margin against the onset of partial or total structural collapse.

3. Collapse Prevention (CP)

Building continues to support gravity loading, but retains no margin against collapse load.

3.6.2 Quantitative Approaches

Although current building codes and state-of-the-art publications have attempted to define the various performance levels for structural and non-structural systems, performance levels have only been identified qualitatively. Therefore, designers' have to determine quantitative response limits that correspond to the qualitative code descriptions. Another approach for defining structural performance levels might be based on quantitative procedures using nonlinear pushover techniques (ATC-40, 1996 and FEMA 356). By this pushover technique customized values for different damage state such as Immediate Occupancy (I0), Life Safety (LS) and Collapse Prevention (CP) can be evaluated.

3.6 SEISMIC PERFORMANCE EVALUATION BY NONLINEAR STATIC PROCEDURE

Two or three-dimensional models of each sample building can be prepared in the ETABS 2015. Nonlinear static analysis is then conducted to determine the base shear versus roof displacement relationship (capacity curve). Flexural elements for beams, beam-column elements for columns, strut elements for infill walls and rigid diaphragms for floors can be employed for modelling the structural components of the buildings.

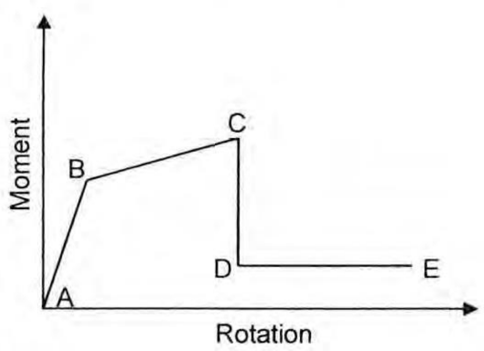


Figure 3.3: Idealized moment-rotation relationship of a frame member-end

Nonlinear flexural characteristics of the individual frame members are defined by moment-rotation relationships of plastic hinges assigned at the member ends. Flexural moment capacities are based on the section and material properties of members. Column capacities are calculated from the axial force-bending moment interaction diagrams. A typical moment-rotation relationship for frame members is shown in Figure 3.1. The segment AB. representing initial linear behaviour, is followed by the post-yield behaviour Be. Point C corresponds to the ultimate strength, where a sudden loss of strength occurs when the associated plastic rotation level is exceeded. This drop from C to O represents the initiation of failure in the member

3.7 DEVELOPMENT OF DIMENSIONLESS BILINEAR CAPACITY CURVE

The capacity curve of eachzmodel can be approximated with a bilinear curve using the guidelines given in FEMA-356 (ASCE, 2000). A typical idealization of a capacity curve is shown in Figure 2.5. It is required to specify the yield and ultimate strength capacities and their associated global drift values for constructing the approximate bilinear capacity curve. The global drifts can be used to represent the damage limit states of the buildings. The yield global drift ratio Θ_y represents significant yielding of the system when the yield base shear capacity (V_y) of the building is attained where as the ultimate global drift ratio Θ_u corresponds to the state at which the building reaches its deformation capacity. The base shear coefficient ŋ= V_y/W in Figure 2.5 is the ratio of yield base shear capacity to the building weight.

It should be noted that there is no universal consensus on how to approximate a capacity curve with a bilinear force-deformation representation. An initial stiffness targeting at the state of significant global yielding may lead to considerable variations in V_y and Θ_y because there is no specific point on the capacity curve exactly describing significant yielding (Sullivan et al., 2004).

3.8 IDENTIFICATION OF QUANTITATIVE LIMIT STATE

Representative probability density functions of Θ_y and Θ_u can be determined in terms of mean, median and standard deviation. When global ductility capacities *(*Θ_u/Θ_y*)* are calculated both Θ_y and Θ_u can be utilized to determine deformation capacities. It is more appropriate to employ Θ_u in assessing the deformation capacities of such buildings, which have infill walls or short span length (Akkar, 2004)

Three performance limits, immediate occupancy, life safety and collapse prevention that are specified in several other international guidelines are usually adopted in fragility studies. The collapse prevention performance limit Θ_{cp} is taken as is taken as the 50 percent to 75 percent of the median Θ_u computed depending on the construction quality, level of confidence on proper design and detailing, uncertainty in modelling and skewness of the ultimate drift probability function. The life safety performance is assigned as the 3 quartile or half of the suggested collapse prevention limit depending on the vulnerability of structure. The median Ɵy computed for each story-based building group is accepted to be the limiting value for the immediate occupancy performance level. It is assumed that light, moderate and severe damage states are experienced when the immediate occupancy, life safety and collapse prevention drift limits are exceeded, respectively. The selected performance limits that are described qualitatively in Table 3.1 are conjectural and could be argued as subjective. (Akkar, 2004).

Performance Level	Limit State
Collapse Prevention (Severe Damage)	**Ɵ≤Ɵcp**
Life safety (Moderate Damage)	**Ɵ≤3/4~1/2Ɵcp**
Immediate occupancy (Light Damage)	**Ɵ≤Ɵy**

Table 3.1: Assumed drift ratio limits for performance levels

3.9 DESCRIPTION TO PUSHOVER ANALYSIS

Federal Emergency Management Agency (FEMA) and Applied Technical Council (ATC) are the two agencies which formulated and suggested the Non-linear Static Analysis or Pushover Analysis under seismic rehabilitation programs and guidelines. This included documents FEMA-356, FEMA-273 and ATC-40.

3.9.1 Introduction to FEMA-356

The primary purpose of FEMA-356 document is to provide technically sound and nationally acceptable guidelines for the seismic rehabilitation of buildings. The guidelines for the seismic rehabilitation of the buildings are intended to serve as a ready tool for design professional for carrying out the design and analysis of the buildings, a reference document for the building regulatory officials and a foundation for the future development and implementation of the building code provisions and standards.

3.9.2 Introduction to ATC-40

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

3.10 TYPES OF PUSHOVER ANALYSIS

Presently, there are two non-linear static analysis procedures available, one termed as the Displacement Coefficient Method (DCM), documented FEMA-356 and other the Capacity Spectrum Method (CSM) documented in ATC-40. Both methods depend on lateral load-deformation variation obtained by non-linear static analysis under the gravity loading and idealized lateral loading due to the seismic action. This analysis is called Pushover Analysis.

3.10.1 Capacity Spectrum Method

Capacity Spectrum Method is a non-linear static analysis procedure which provides a graphical representation of the expected seismic performance of the structure by intersecting the structure's capacity spectrum with the response spectrum (demand spectrum) of the earthquake. The intersection point is called as the performance point, and the displacement coordinate d_p of the performance point is the estimated displacement demand on the structure for the specified level of seismic hazard.

3.10.2 Displacement Coefficient Method:

Displacement Coefficient Method is a non-linear static analysis procedure which provides a numerical process for estimating the displacement demand on the structure, by using a bilinear representation of the capacity curve and a series of modification factors or coefficients to calculate a target displacement. The point on the capacity curve at the target displacement is the equivalent of the performance point in the capacity spectrum method.

3.11 PERFORMANCE POINT

It is the point where the capacity spectrum intersects the appropriate demand spectrum. To have the desired performance in the structure it should be designed by considering these points of forces.

3.12 BUILDING PERFORMANCE LEVEL

Building performance is the combined performance of both structural and non-structural components of the building. Different performance levels are used to describe the building performance using the pushover analyses, which are described below.

3.12.1 Operational level (OL):

As per this performance level building are expected to sustain no permanent damages. Structure retains original strength and stiffness. Major cracking is seen in partition walls and ceilings as well as in the structural elements.

3.12.2 Immediate occupancy level (IO):

Buildings meting this performance level are expected to sustain no drift and structure retains original strength and stiffness. Minor cracking in partition walls and structural elements is observed. Elevators can be restarted. Fire protection is operable.

3.12.3 Life Safety Level (LS):

This level is indicated when some residual strength and stiffness is left available in the structure. Gravity load bearing elements function, no out of plane failure of walls and tripping of parapet is seen. Some drift can be observed with some failure to the partition walls and the building is beyond economical repair. Among the non-structural elements failing hazard mitigates but many architectural and mechanical and mechanical systems get damaged.

3.12.4 Collapse Prevention Level (CP):

Buildings meeting this performance level are expected to have little residual strength and stiffness, but the load bearing structural elements function such as load bearing walls and columns. Building is expected to sustain large permanent drifts, failure of partitions infill and parapets and extensive damage to non-structural elements. At this level the building remains in collapse level.

3.13 PLASTIC HINGE

Location of inelastic action of the structural member is called as plastic hinge.

3.13.1 Formation of Plastic Hinge:

The maximum moments caused by the earthquake occur near the ends of the beams and columns, the plastic hinges are likely to form there and most ductility requirements apply to section near the junction.

3.14 ASSIGNMENT OF HINGES FOR PUSHOVER ANALYSIS

For nonlinear static, and nonlinear direct-integration time-history analyses, users may simulate post-yield behaviour by assigning concentrated plastic hinges to frame and tendon objects. Elastic behaviour occurs over member length, and then deformation beyond the elastic limit occurs entirely within hinges, which are modelled in discrete locations.

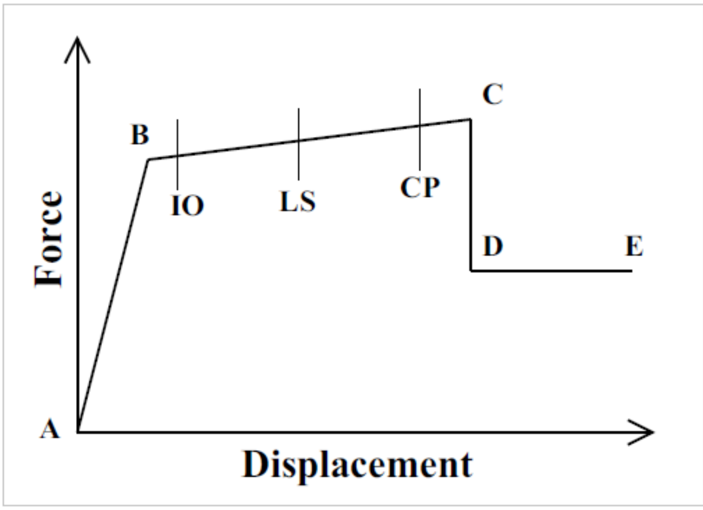


Figure 1: Force - Displacement curve of a Hinge.

Inelastic behavior is obtained through integration of the plastic strain and plastic curvature which occurs within a user-defined hinge length, typically on the order of member depth (FEMA-356). To capture plasticity distributed along member length, a series of hinges may be modeled. Multiple hinges may also coincide at the same location.

Plasticity may be associated with force-displacement behaviors (axial and shear) or moment-rotation (torsion and bending). Hinges may be assigned (uncoupled) to any of the six DOF. Post-yield behavior is described by the general backbone relationship shown to the right. The modeling of strength loss is discouraged, to mitigate load redistribution (which may lead to progressive collapse) and to ensure numerical convergence.

CSI Software automatically limits negative slope to 10% of elastic stiffness, though overwrite options are available. For informational purposes, additional limit states (IO, LS, CP) may be specified which are reported in analysis, but do not affect results. Unloading from the point of plastic deformation follows the slope of initial stiffness.

Both P-M2-M3 hinges and fiber hinges are available to capture coupled axial and biaxial-bending behavior. The P-M2-M3 hinge is best suited for nonlinear static pushover, whereas the fiber hinge is best for hysteretic dynamics.

3.15 Frame/Wall Nonlinear Hinge

Hinge properties are used to define nonlinear force-displacement or moment-rotation behavior that can be assigned to discrete locations along the length of frame (line) objects or to the mid-height of wall objects. These nonlinear hinges are used during static nonlinear analysis, fast nonlinear analysis (FNA) modal time history analysis, and nonlinear direct integration time history analysis. For all other types of analysis, the are rigid and have no effect on the behavior of the member. The number of hinges not only affects computation time, but also the ease in which model behavior and results may be interpreted. Therefore, it is strongly recommende that hinges be assigned only at locations where the occurrence of nonlinear behavior is highly probable.

Note: It is important that frame and wall objects be designed, e.g. reinforcement should be defined for concrete frames and walls, prior to running a nonlinear analysis utilizing hinges. Three kinds of hinge properties are available in ETABS:

3.16 Auto Hinge Properties.

Auto hinge properties are defined by the program. The program cannot fully define the auto properties until the section to which they apply has been identified. Thus, the auto property is assigned to a frame or wall object, and the resulting hinge property can then be reviewed.

3.17 User-Defined Hinge Properties.

User-defined hinge properties can be based on auto properties or they can be fully user defined.

3.18 Program Generated Hinge Properties.

The generated hinge properties are used in the analysis. They can be viewed, but they cannot be modified. Generated hinge properties have an automatic naming convention of LabelH#, where Label is the frame or wall object label, H stands for hinge, and # represents the hinge number. The program starts with hinge number 1 and increments the hinge number by one for each consecutive hinge applied to the frame or wall object. For example, if a frame object label is C4, the generated hinge property name for the second hinge applied to the frame object is C4H2.

The main reason for the differentiation between defined properties (in this context, defined means both auto and user-defined) and generated properties is that typically the hinge properties are section dependent. Thus, it is necessary to define a different set of hinge properties for each frame or wall section type in the model. This could potentially mean that you would need to define a very large number of hinge properties. To simplify this process, the concept of generated properties is used in ETABS. When generated properties are used, the program combines its built-in criteria with the defined section properties for each object to generate the final hinge properties. The net effect of this is that you do significantly less work defining the hinge properties because you do not need to define every hinge.

The user assigns auto hinge properties and user-defined hinge properties to a frame or wall object. The program then automatically creates a new generated hinge property for every assigned hinge.

Define user-defined hinge properties as follows:

1. Click the **Define menu > Section Properties > Frame/Wall Nonlinear Hinge** command to access the ***Define Frame/Wall Hinge Properties*** form.

2. Choose or input parameters for the following areas.

   - **Defined Hinge Props** area. A list of hinge properties, including any previously defined auto or user-defined hinge properties is displayed in this area. Check the *Show Generated Props* check box to include the generated hinge properties in this display list. Check the *Show Hinge Details* check box to display additional information about the hinges in the list (see *Show Hinge Details* check box write-up below).

   - **Add New Property** button. Click this button and the ***Default for Added Hinges*** form will display. Use that form to specify the type of default hinge definitions to be used as the basis of adding a new hinge definition. After selecting Steel, Concrete or User Defined, the ***Hinge Property Data*** form will display. Use that form to complete the definition of a new hinge property.

   - **Add Copy of Property** button.

1. Highlight a hinge property name in the *Defined Hinge Props* list box. Note that generated properties cannot be copied.

2. Click the **Add Copy of Property** button to display the ***Hinge Property Data*** form pre-loaded with the definition options of the selected hinge property.

3. Use that form to add a new definition based on the selected definition.

- **Modify/Show Property** button.

1. Highlight the hinge property name to be modified in the *Defined Hinge Props* list box.

2. Click the **Modify/Show Property** button to display the ***Hinge Property Data*** form.

3. Use that form to make the necessary changes to the definition.

Note: Generated hinge properties can be viewed, but cannot be modified.

Property button will be grayed out and inactive. A hinge property cannot be deleted until it has been removed from all objects. Remove a hinge by selecting the object(s) and deleting the assignment.

3.

- **Show Hinge Details** check box. When this check box is checked, the *Defined Hinge Props* area expands to a spreadsheet type area that has the following columns:

  - **Name**. The ID assigned to the hinge is displayed in this column.

- **Type**. The type of hinge (e.g., Axial P, Shear V, Moment M and so on) is displayed in this column.

- **Behavior**. This column identifies if the hinge is deformation or force controlled.

- **Generated**. If Yes is displayed, the hinge is a generated hinge. If No is displayed, the hinge is user defined or auto.

- **From**. If the hinge is a generated hinge (i.e., yes appears in the *Generated* column), this column displays the ID of the hinge upon which the generated hinge is based. If the hinge definition is program defined, auto displays in this column. If N.A. appears in this column, the hinge is a user-defined hinge that is based solely on the user's input.

Note: Make changes to any of these items by first highlighting the row of data to be changed. Then click the **Modify/Show Property** button to display the ***Hinge Property Data*** form and make the necessary adjustments. Note that generated properties cannot be modified.

4.

- **Show Generated Props** check box. By default, hinge properties that the program automatically generates at each hinge location are not listed in the *Defined Hinge Prop* area of the ***Define Frame/Wall Hinge Properties*** form. Check the *Show Generated Props* check box, and ETABS will display those properties in the *{Defined, all} Hinge Props* area along with any Auto hinge properties that have been assigned to the model.

- **Convert Auto to User Prop** button. This button appears on the form when an Auto hinge property has been assigned to a frame or wall object(s) in the model and the *Show Generated Props* check box is checked. When this button is clicked, the program converts the Auto property hinge to a user-defined hinge property. After an Auto hinge property, has been converted to a user-defined property, the resulting hinge property definition can be modified by clicking on it and then clicking the **Modify/Show Property** button to display the ***Hinge Property Data*** form.

3.19 CAPACITY

It is defined as the expected ultimate strength (in flexure, shear and axial loading) of the structural components excluding the reduction factors commonly used in the design of concrete members. The capacity generally refers to the strength at the yield point of the element or structure's capacity curve. For deformation controlled component's, capacity beyond the elastic limit generally includes the effect of strain hardening.

3.20 Capacity Curve:

The plot between base shear and roof displacement is referred as capacity curve. Also, mentioned as pushover curve.

3.21 Capacity Spectrum

The capacity curve transformed from base shear v/s roof displacement *(V v/s d)* to spectral acceleration v/s spectral displacement (S_a *v/s* S_d) is referred as capacity spectrum.

3.22 Capacity Spectrum Method:

A nonlinear static procedure that produce a graphical representation of the expected seismic performance of the building by intersecting the structure's capacity curve with a response spectrum representation of earthquake's displacement demand on the structure, the intersecting point is called performance point and the displacement coordinate d_p of the performance point is the estimated displacement demand on the structure for the specified level of hazard.

3.23 DEMAND

Demand is represented by an estimation of the displacement or deformation that the structure is expected to undergo. This is in contrast to conventional, linear elastic analysis procedures in which demand is represented by prescribed lateral forces applied to the structure.

3.24 Demand Spectrum

It is plot between average spectral acceleration versus time period. It represents the earthquake ground motion in capacity spectrum method.

3.25 Pushover analysis procedure

The use of the nonlinear static analysis pushover analysis came into practice in 1970's but the potential of pushover analysis has been recognised for last 10 to 15 years. This procedure is mainly used to estimate the strength and drift capacity of existing structure and the seismic demand for this structure subjected to selected earthquake this procedure can be used for checking the adequacy of new structural design as well pushover analysis is defined as an analysis wearing a mathematical model directly incorporating the normal load deformation characteristics of individual components and elements of the building shall be subjected to monotonically interesting lateral loads representing inertia forces in an earthquake until a target displacement is excised accident exceeded target displacement is the maximum displacement elastic plus in asterisk inelastic of the building address expected under selected earthquake ground motion pushover analysis assesses the structural performance by estimating the force and deformation capacity and seismic demand using a nonlinear static analysis algorithm the seas meet demand parameters are global displacement at roof or any other reference point story dressed story forces component deformation and component forces the analysis accounts for geometrical nonlinearity, material inelasticity and the redistribution of internal forces.

Pushover analysis can be performed as either force control or displacement controlled depending on the physical nature of the Lateral load and behaviour expected from the structure force. Controlled procedure is useful when the load is known such as gravity loading and the structure is expected to be able to support the load. Displacement controlled procedure should be used when a specified source such as in seismic loading where the magnitude of the applied load is not known in advance or when the structure can be expected to lose strength or become unstable. The nonlinear pushover analysis of a structure is an iterative procedure. It depends on the final displacement as the effective damping depends on the hysteretic energy loss due to inelastic deformation which in turn depends on the final displacement. This makes the analysis procedure iterative. Difficulty in the solution is faced near the ultimate load as the stiffness Matrix at this point becomes negative, definite due to instability of the structure becoming a mechanism.

3.25.1 The analysis of ETABS

1. Modelling

2. Static analysis

3. Design

4. Pushover analysis

3.25.2 Steps for Pushover Analysis in ETABS

1. The ETABS has inbuilt default ACI 318 material proportions ATC 40 and FEMA 273 hinge properties also it has capability for inputting any material or Hinges property ETABS deals with the buildings only where uncoupled moment M2 and M3, Torsion T, axial force p and V2 and V3 force displacement relations can be defined and the column axial load changes under lateral loading there is also a coupled P-M2-M3(PMM) hinge which yields based on the interaction of axial force and bending moment At The hinge location in a location also more than one type of hinge can be assigned at the same location of a frame element following are the steps in performing pushover analysis for a 3D frame building one creating the basic model without the pushover data in the usual manner.

2. Defining properties and acceptance criteria for the pushover hinges the program includes several built-in default hinge properties that are based on average values from ATC 40 for concrete members and average values from FEMA 273 for steel members these built-in properties can be useful for preliminary analysis but user defined properties are recommended for final analysis.

3. Locate the pushover Hinges on the model by selecting one or more frame members and assigning them one or more hinge properties and its locations.

4. Defining the pushover analysis load cases inner tabs more than one pushover load can be run in the same analysis also a pushover load case can start from the file and conditions of

another pushover Loads that was previously run in the same analysis typically a gravity load pushover is force control and lateral pushover displacement controlled.

5. Run the basic static analysis and if desired dynamic analysis then run the static nonlinear pushover analysis.

6. Display the pushover curve.

7. Review a pushover displaced shape and sequence of hinge formation on a step-by-step basis.

3.26 PUSHOVER ANALYSIS:

An overview of the procedure for pushover analysis is given as follows:

3.26.1 Create the computational model

- Create the computational model, without pushover data, using conventional modelling techniques.
- Define properties for pushover hinges using Define > Section Properties > Hinge Properties. Hinges may be defined manually or by using one of several default specifications which are available.
- Assign the pushover hinges to selected frame objects using Assign > Frame > Hinges.
- Select Define > Load Patterns to define load patterns which will contain the loads applied during pushover analysis.

3.26.2 Define a nonlinear static load case

- Select Define > Load Cases > Add New Load Case to define a nonlinear static load case which will apply the previously-defined load pattern. This load case may be force-controlled (pushed to a specified force level) or displacement-controlled (pushed to a specified displacement).
- Select Other Parameters > Results Saved to Multiple States such that various parameters may be plotted for each increment of applied loading.

3.26.3 Run the analysis

- Select Analyse > Run Analysis to run the static-pushover analysis.

3.26.4 Review results

- To plot base shear vs. monitored displacement, select Display > Show Static Pushover Curve. Additional variables are also available for plotting.
- To plot hinge deformation vs. applied loading, select Display > Show Hinge Results. Moment as a function of plastic rotation is one such option.
- To review displacement and the step-by-step sequence of hinge formation, select Display > Show Deformed Shape.
- To review member forces on a step-by-step basis, select Display > Show Forces/Stresses > Frames/Cables.
- Select Display > Show Plot Functions to plot response at each step of the pushover analysis, including joint displacement, frame member forces, etc.

3.27 SUMMARIZATION OF THE PROCEDURE FOR THE GENERATION OF FRAGILITY CURVES

Step 1: Development of representative models of the building stocks using assumed probability density function and general trend of construction parameters.

Step 2: Nonlinear static Pushover technique is then employed to develop bilinear capacity curve.

Step 3: From the bilinear capacity curves the yield base shear co-efficient (Vy/W), the yield global drift ratio (ϴy) and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope (α) are then selected as random variables and the statistical properties of these Three quantities (Vy/W , ϴy and α) are determine.

Step 4: From bilinear capacity curve ϴy and ϴu are estimated to identity quantitative limit state in terms of global drift ratio: I0, LS and CP.

Step 5: Nonlinear time history analyses are then carried out to determine the maximum global drift ratio of the developed models corresponding to each earthquake.

Step 6: Using the damage threshold levels defined in step 4, the exceedance probabilities of a particular damage state are computed from the PGA versus maximum global drift scatters.

Step 8: The global drift percentiles greater than a given damage threshold level are computed by using the normal distribution to estimate the exceedance probabilities of the fragility curves and the jaggedly varying exceedance probability points are then smoothened to develop fragility curves for that specific damage state.

Chapter 4

RESULTS

4.1 SFD and BMD diagram of frame

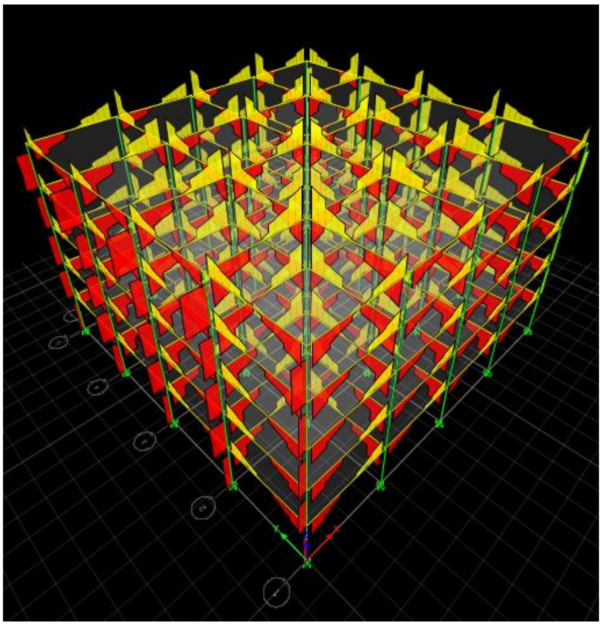


Fig 4.1 SFD of RCC frame

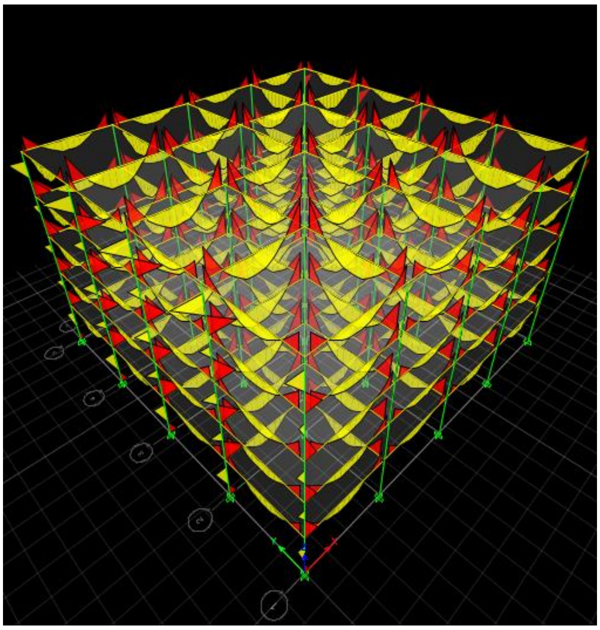


Fig 4.2 BMD of RCC frame

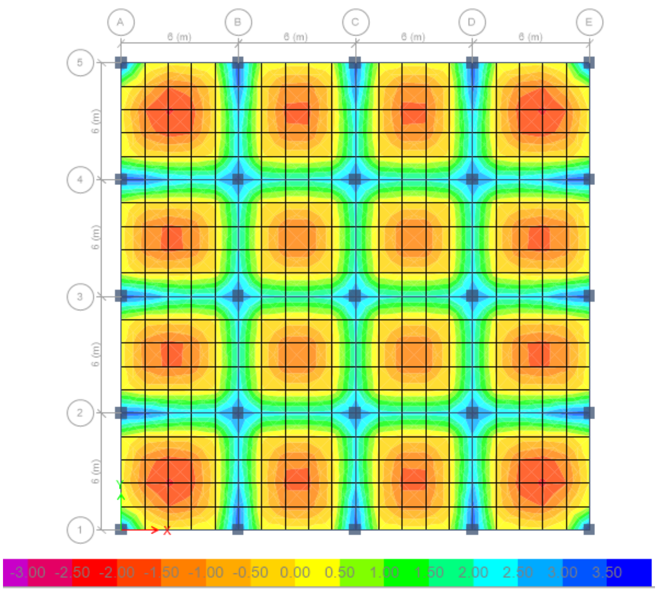


Fig 4.3 stress diagram of 5th floor

4.2 ETABS 2015 Concrete Frame Design

Beam Element Details Type: Ductile Frame (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	B21	beam 300*600	DCon2	5700	6000	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
300	600	300	0	25	25

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
-58.3128	0.9021	64.6485	0

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
-58.3128	1.5919	0	-59.9047

Design Moment and Flexural Reinforcement for Moment, M_{u3} & T_u

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm^2	+Moment Rebar mm^2	Minimum Rebar mm^2	Required Rebar mm^2
Top (+2 Axis)	-59.9047		432	0	247	432
Bottom (-2 Axis)		0	123	0	0	123

Shear Force and Reinforcement for Shear, V_{u2} & T_u

Shear V_e kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm^2/m
64.6485	62.139	69	0	332.53

Torsion Force and Torsion Reinforcement for Torsion, T_u & V_{U2}

T_u kN-m	V_u kN	Core b_1 mm	Core d_1 mm	Rebar A_{svt} /s mm^2/m
0.9021	64.6485	270	570	289.16

Beam Element Details Type: Ductile Frame (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	B21	beam 300*600	DCon2	5700	6000	1

Section Properties

b (mm)	h (mm)	b_f (mm)	d_s (mm)	d_{ct} (mm)	d_{cb} (mm)
300	600	300	0	25	25

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
25000	25	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
-58.3128	0.9021	64.6485	0

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
-58.3128	1.5919	0	-59.9047

Design Moment and Flexural Reinforcement for Moment, M_{u3} & T_u

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm²	+Moment Rebar mm²	Minimum Rebar mm²	Required Rebar mm²
Top (+2 Axis)	-59.9047		432	0	247	432
Bottom (-2 Axis)		0	123	0	0	123

Shear Force and Reinforcement for Shear, V_{u2} & T_u

Shear V_e kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
64.6485	62.139	69	0	332.53

Torsion Force and Torsion Reinforcement for Torsion, T_u & V_{U2}

T_u kN-m	V_u kN	Core b_1 mm	Core d_1 mm	Rebar A_{svt} /s mm²/m
0.9021	64.6485	270	570	289.16

Column Element Details Type: Ductile Frame (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story1	C5	col 600*600	DCon2	2700	3300	0.613

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
600	600	58	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
27386.13	30	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar Area mm²	Rebar % %

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar Area mm²	Rebar % %
949.0439	-26.3382	26.3382	24.1057	24.1057	2880	0.8

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	2.259774	2700	10.5353	0	24.1057
Minor Bend(M2)	2.259774	2700	-10.5353	0	24.1057

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
Major, V_{u2}	0	0	0	0	0
Minor, V_{u3}	16.0927	193.2437	130.0804	0	665.06

Joint Shear Check/Design

	Joint Shear Force kN	Shear V_{Top} kN	Shear $V_{u,Tot}$ kN	Shear V_c kN	Joint Area cm²	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(1.1) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

Additional Moment Reduction Factor k (IS 39.7.1.1)

A_g cm²	A_{sc} cm²	P_{uz} kN	P_b kN	P_u kN	k Unitless
3600	28.8	5940	2235.052	949.0439	1

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	No	0.818	600	10.169	12	No	0
Minor Bending (M_2)	No	0.818	600	10.169	12	No	0

Column Element Details Type: Ductile Frame (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story5	C5	col 600*600	DCon2	2400	3000	1

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
600	600	58	30

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
27386.13	30	1	500	500

Design Code Parameters

γ_C	γ_S
1.5	1.15

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M_2 kN-m	Minimum M_3 kN-m	Rebar Area mm²	Rebar % %
162.6437	-58.3362	58.3362	4.0336	4.0336	2880	0.8

Axial Force and Biaxial Moment Factors

	K Factor Unitless	Length mm	Initial Moment kN-m	Additional Moment kN-m	Minimum Moment kN-m
Major Bend(M3)	3.911747	2400	23.3345	0	4.0336
Minor Bend(M2)	3.911747	2400	-23.3345	0	4.0336

Shear Design for V_{u2} , V_{u3}

	Shear V_u kN	Shear V_c kN	Shear V_s kN	Shear V_p kN	Rebar A_{sv} /s mm²/m
Major, V_{u2}	0	0	0	0	0
Minor, V_{u3}	48.0878	159.8374	130.0804	0	665.06

Joint Shear Check/Design

	Joint Shear Force kN	Shear V_{Top} kN	Shear $V_{u,Tot}$ kN	Shear V_c kN	Joint Area cm²	Shear Ratio Unitless
Major Shear, V_{u2}	N/A	N/A	N/A	N/A	N/A	N/A
Minor Shear, V_{u3}	N/A	N/A	N/A	N/A	N/A	N/A

(1.1) Beam/Column Capacity Ratio

Major Ratio	Minor Ratio
N/A	N/A

Additional Moment Reduction Factor k (IS 39.7.1.1)

A_g cm²	A_{sc} cm²	P_{uz} kN	P_b kN	P_u kN	k Unitless
3600	28.8	5940	2235.052	162.6437	1

Additional Moment (IS 39.7.1)

	Consider M_a	Length Factor	Section Depth (mm)	KL/Depth Ratio	KL/Depth Limit	KL/Depth Exceeded	M_a Moment (kN-m)
Major Bending (M_3)	No	0.8	600	15.647	12	Yes	0
Minor Bending (M_2)	No	0.8	600	15.647	12	Yes	0

4.3 Story Response - Story Overturning Moment

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Overturning moments	Story Range	All Stories
Modal Case	Modal	Top Story	Story5
Mode Number	1	Bottom Story	Base

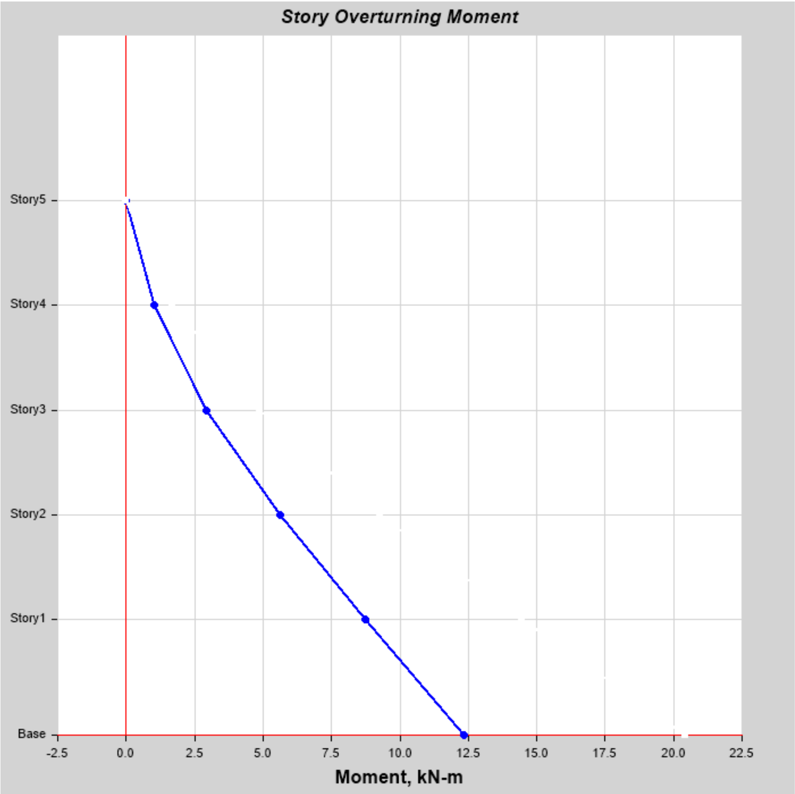


Fig 4.4 storey vs Moment

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		kN-m	kN-m
Story5	15.3	Top	0	0
Story4	12.3	Top	1.0099	1.6731
Story3	9.3	Top	2.9513	4.8895
Story2	6.3	Top	5.6101	9.2943
Story1	3.3	Top	8.7165	14.4408
Base	0	Top	12.325	20.419

4.1 Tabulated Plot Coordinates

4.4 Story Response - Maximum Story Displacement

Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp1		
Display Type	Max story disp	Story Range	All Stories
Load Case	RS-X	Top Story	Story5
Output Type	Not Applicable	Bottom Story	Base

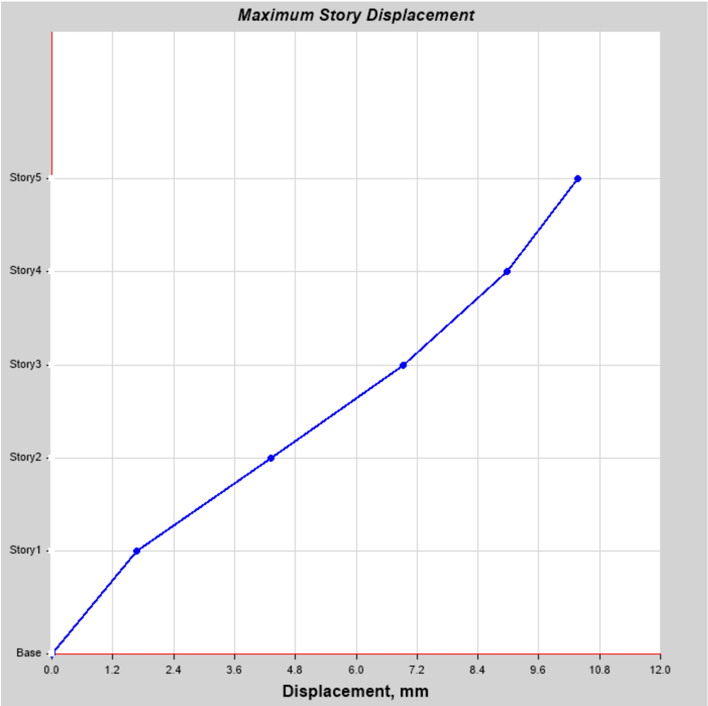


Fig 4.5 storey vs diplacement

Tabulated Plot Coordinates

Story Response Values

Story	Elevation	Location	X-Dir	Y-Dir
	m		Mm	mm
Story5	15.3	Top	10.4	5.966E-08
Story4	12.3	Top	9	9.422E-08
Story3	9.3	Top	6.9	4.223E-08
Story2	6.3	Top	4.3	4.215E-08
Story1	3.3	Top	1.7	1.825E-08
Base	0	Top	0	0

4.5 Response Spectrum from Time History

Summary Description

This shows a response spectrum plot obtained from time history results at a specified point for a specified time history load case.

Input Data

Name	RSFromTH1		
Load Case	TH-X	Coordinate System	Modal
Story	Story5	Response Direction	X
Point	1	Spectrum Widening	0 %

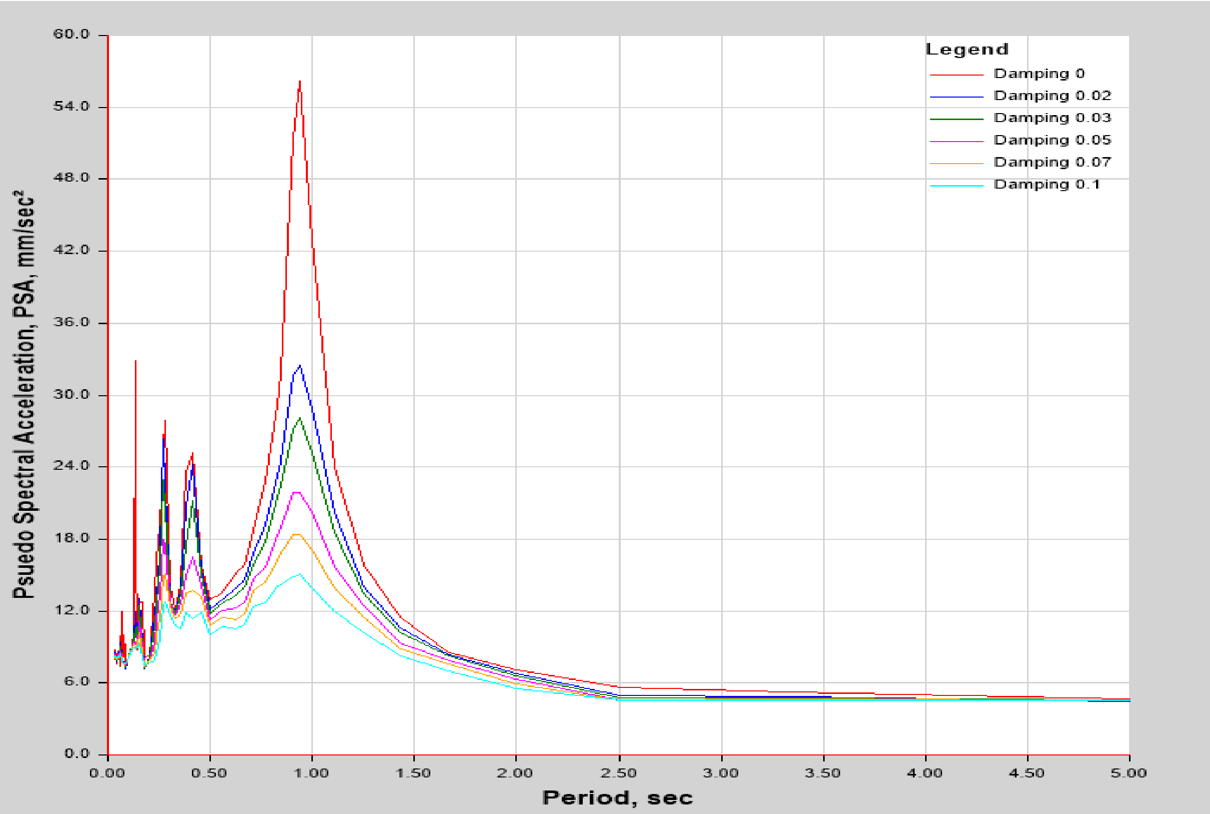


Fig 4.6 psa vs period

Tabulated Plot Coordinates

Response Spectrum Values

Period	Damping 0	Damping 0.02	Damping 0.03	Damping 0.05	Damping 0.07	Damping 0.1
	PSA	PSA	PSA	PSA	PSA	PSA
sec	mm/sec²	mm/sec²	mm/sec²	mm/sec²	mm/sec²	mm/sec²
0.2	4.66	4.5	4.52	4.56	4.58	4.59
0.3	5.26	4.9	4.74	4.65	4.62	4.6
0.4	5.63	4.95	4.77	4.62	4.61	4.61
0.5	7.09	6.77	6.61	6.28	5.96	5.51
0.6	8.53	8.36	8.22	7.89	7.54	7.02
0.7	11.45	10.64	10.21	9.38	8.81	8.25
0.8	15.84	13.98	13.42	12.4	11.46	10.2
0.9	24.01	20.19	18.58	15.78	13.86	11.93
1	42.97	28.86	25.21	20.27	17.09	13.89
1.061	56.26	32.53	28.09	21.88	18.39	15.11
1.1	51.35	31.62	27.12	21.85	18.41	14.88
1.181	31.82	24.55	22.43	18.94	16.84	14.22
1.2	29.14	23.36	21.48	18.28	16.27	14.12
1.3	22.88	19.13	17.76	15.7	14.39	12.76
1.4	18.76	16.78	15.89	14.71	13.68	12.34
1.5	15.87	14.63	13.98	12.77	11.79	10.89
1.6	15.14	13.9	13.32	12.25	11.3	10.46
1.8	13.5	12.98	12.67	12.05	11.48	10.7
2	12.92	12.11	11.76	11.28	10.79	10.06
2.2	15.74	15.5	15.11	14.2	13.24	11.85
2.4	25.22	24.25	21.17	16.51	13.72	11.35
2.6	23.68	20.67	17.46	14.87	13.5	11.88
2.8	14.22	13.68	13.38	12.61	11.73	10.48
3	12.16	11.97	11.9	11.69	11.38	10.8
3.3	13.99	12.77	12.75	12.63	12.34	11.72
3.6	27.87	22.36	20.71	17.46	15.05	12.75
3.644	25.8	26.35	22.97	17.95	15.02	12.57
4	18.75	16.72	15.65	13.26	11.21	9.71
4.063	18.05	14.85	14.19	12.58	11.01	9.11
4.4	13.73	11.56	10.81	9.5	8.55	7.85
4.7	9.86	9.32	8.97	8.41	8.16	7.78
5	8.23	8.13	8.08	7.96	7.84	7.64
5.5	8.17	7.2	7.21	7.32	7.42	7.49
6	12.78	9.2	8.9	8.93	8.55	8.16
6.5	12.78	13.01	12.44	11.16	10.21	9.33
7	8.76	9.06	9.05	9.07	9.11	8.99
7.44	18.14	11.53	10.51	9.43	8.9	8.84
7.5	32.85	11.94	10.62	9.43	8.96	8.85
8	10	9.85	9.73	9.45	9.2	8.92
8.32	9.33	9.26	9.23	9.13	9.01	8.82
8.5	9.07	9.05	9.04	8.98	8.89	8.74
9	8.76	8.73	8.71	8.67	8.62	8.53
10	8.23	8.23	8.23	8.22	8.2	8.17
11	7.8	7.82	7.84	7.85	7.87	7.88
12	7.19	7.21	7.32	7.49	7.59	7.68
12.417	9.27	7.4	7.41	7.54	7.6	7.66

Period	**Damping 0**	**Damping 0.02**	**Damping 0.03**	**Damping 0.05**	**Damping 0.07**	**Damping 0.1**
	PSA	**PSA**	**PSA**	**PSA**	**PSA**	**PSA**
sec	**mm/sec²**	**mm/sec²**	**mm/sec²**	**mm/sec²**	**mm/sec²**	**mm/sec²**
13	8.2	7.81	7.7	7.61	7.62	7.67
13.918	11.92	7.82	7.67	7.69	7.74	7.78
14	10.39	8.17	7.89	7.77	7.78	7.8
15	8.23	8.2	8.18	8.13	8.09	8.03
16.5	9.94	9.41	9.01	8.63	8.45	8.3
17.308	7.35	8.4	8.53	8.54	8.47	8.35
18	9.02	8.82	8.7	8.55	8.45	8.34
19.433	8.35	8.35	8.34	8.31	8.28	8.23
20	8.23	8.23	8.23	8.22	8.21	8.18
22	7.59	7.78	7.86	7.95	7.99	8.03
25	8.23	8.21	8.2	8.17	8.15	8.13
28	8.74	8.53	8.46	8.37	8.31	8.24
33	8.22	8	8.02	8.07	8.11	8.13

4.6 Fragility curve

A particular structure type is considered in this study, namely 5-storey concrete buildings, which generally do not comply with modern seismic resistant design and construction practice. Three dimensional models are created in ETABS 2015 environment to perform nonlinear static analysis (pushover) and nonlinear time history analysis. As proper design data are not available in our country, models have to be constructed using assumed probability density function and general trend of construction parameters. The random variables (yield base shear coefficient, yield global drift ratio and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope) are then selected and statistical properties of these random variables in terms of mean and standard deviation are then determined. These statistical properties represent the group of building stock which seismic vulnerability will be reflected by the generated fragility curves by analysing these models.

Simple three dimensional models are developed and for models the construction parameters are the f_{ck}, f_y, column size, beam size and bay length. In this work f_{ck} and column size are taken as variable parameters. Beam size, f_y of steel and bay length are kept constant. Table 4.1 shows details about various construction parameters of developed models.

Construction parameter	Type	Parameter
Concrete compressive strength(f_{ck})	**Variable**	**M30-M25**
Steel yield strength(f_y)	**constant**	**500 MPa**
Column size	**Variable**	**(500*500)mm–(600*600)mm**
Beam size	**Constant**	**(300*600)mm**
Bay length	**Constant**	**6 m**

Table 4.3 Details of construction parameter

4.7 PUSHOVER ANALYSIS AND FAILURE MECHANISM

Using the variable construction parameters shown in Table 4.3, 30 three dimensional models are developed in ETABS 2015 having different f_{ck} and column size. Then nonlinear static analysis (pushover) is carried out to develop pushover curves for these 30 models. The bilinear capacity curves are constructed for these 30 samples of structures. From these bilinear capacity curves the yield base shear co-efficient (V_y/W), the yield global drift ratio (Θ_y) and the ratio of the post elastic slope of the bilinear capacity curve to the elastic slope (α) are then selected as random variables and the statistical properties of these three quantities (V_y/W , Θ_y and α) are determined.

The pushover curve, defined hinge properties and various steps of pushover for structure having f_{ck} of M30 and column size 600 by 600 mm are shown in Figure 4.7, Figure 4.8 and Figure 4.9, Figure 4.10 and Fig 4.11 respectively. This mechanism of structural failure reflects the lack of sufficient column strength that is familiar in this region.

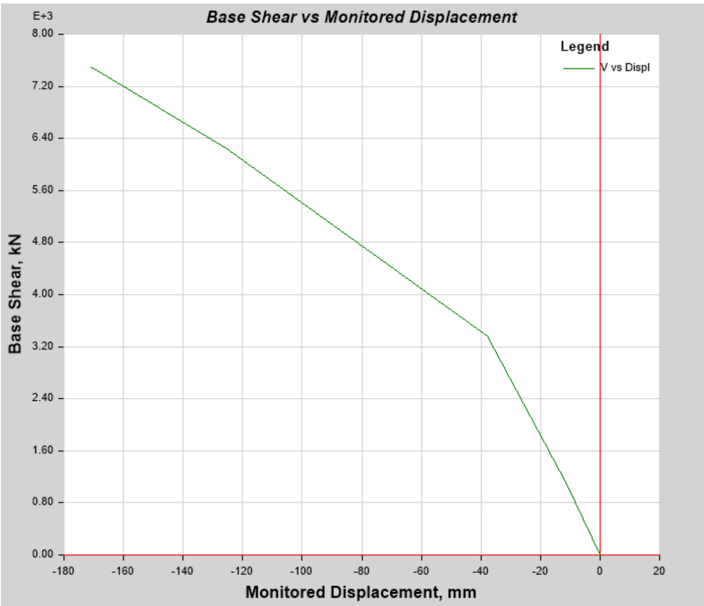


Fig 4.7 Pushover curve

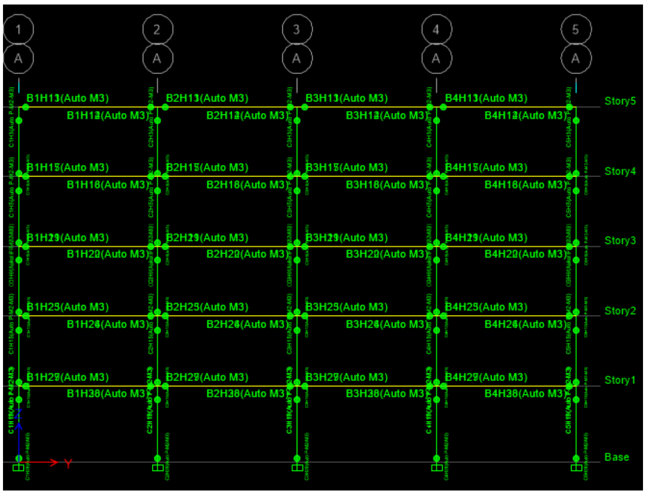


Fig 4.8 Column hinges

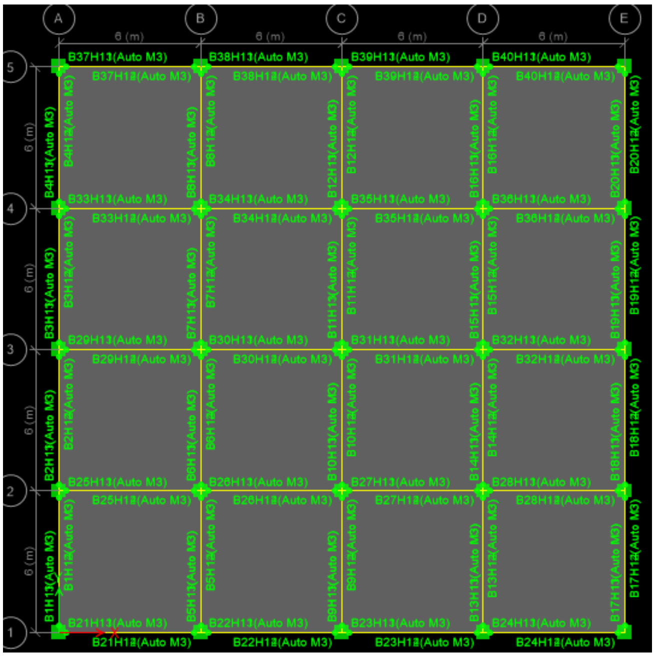


Fig 4.9 Beam hinges

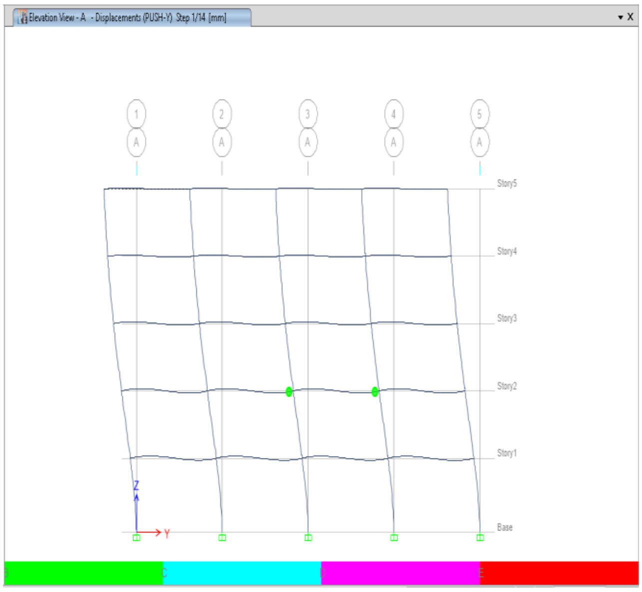


Fig 4.10 step 1 of POA

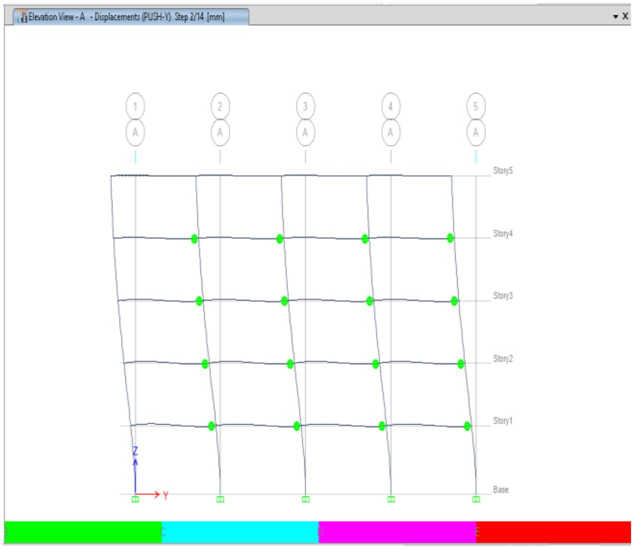


Fig 4.11 step 2 POA

4.8 IDENTIFICATION OF LIMIT STATES

From the 30 capacity curves probability density functions of Θ_y and Θ_u are determined in terms of mean, median and standard deviation. Three performance limits, immediate occupancy, life safety and collapse prevention that are specified in several other international guidelines are adopted in this fragility study.

From bilinear capacity curves Θ_y and Θ_u for thirty structures are determined. The collapse prevention performance limit Θ_{cp} is taken as the 50 percent of the median Θ_u computed considering the deficiency in construction quality in this region, lack of proper detailing and uncertainty in modeling. The life safety performance is assigned as the half of the suggested collapse prevention limit and immediate occupancy is assigned to the 80 percent of median By as most the structures in this region are not properly designed and detailed for seismicity. It is assumed that light, moderate and severe damage states are experienced when the immediate occupancy, life safety and collapse prevention drift limits are exceeded, respectively.

Parameter	**Mean**	**Median**	**Standard deviation**
$\mathbf{\Theta_y}$	**0.0014**	**0.0014**	**0.0092**
$\mathbf{\Theta_u}$	**0.0094**	**0.0095**	**0.0018**

Table 4.2 statistical properties of Ɵy and Ɵu

Limit state	**Value**
Immediate occupancy(light damage)	**0.0011**
Life safety(moderate damage)	**0.0024**
Collapse prevention (severe damage)	**0.0048**

Table 4.3 Damage threshold

4.9 NONLINEAR DYNAMIC TIME HISTORY ANALYSIS AND DEVELOPMENT OF FRAGILITY CURVES

The set of earthquake records 0.1g to 0.75 g utilized to compute the dynamic time-history response of the developed models. The ETABS 2015 in order to simulate the state of damage of each structure under ground acceleration time-history. The global drift ratios are calculated by dividing the maximum value of the roof displacement, &top by average building height. In this case the average building height is 15.3m. The maximum global drift values computed by the above procedure are then assumed to represent the seismic performance of the investigated concrete frames. Using the damage threshold levels defined in Table 4.3, the exceedance probabilities of that particular fragility curve were computed. The probability distribution function is the standard normal or lognormal distribution in most cases (Shinozuka et aI., 2000; Kircher et aI., 1997). From the central limit theorem it is known that if a random variable X is made of the sum of many small effects then X might be expected to be normally distributed. The global drift percentiles greater than a given damage threshold level are computed by using the normal distribution to estimate the exceedance probabilities of the fragility curves. Table 4.4 describes the statistical properties of probability density function of drift ratios in terms of mean, median and standard deviation and probability of exceedance of a given damage threshold for each of the fourteen generated earthquakes.

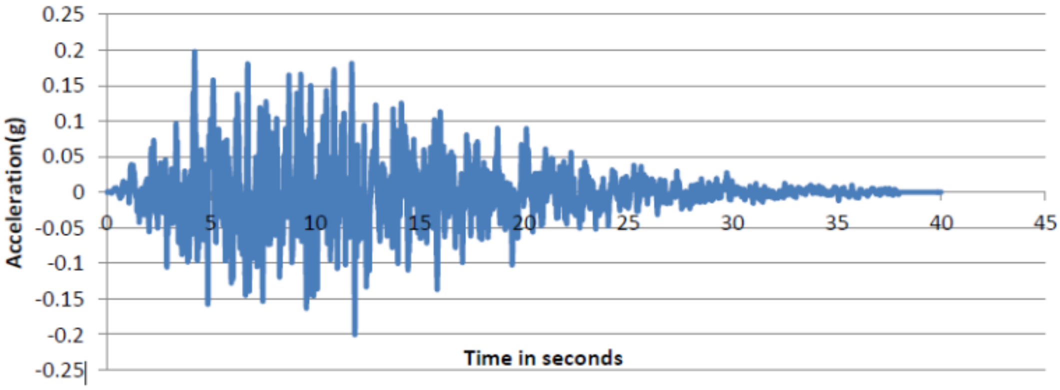


Fig 4.12 IS code ground motion with PGA 0.2 g

The probability of exceedance for each earthquake is calculated considering normal distribution of global drift ratios. The global drift percentiles greater than a given damage threshold level are computed by using the Z- Table (shown in Appendix)of standard normal distribution to estimate the exceedance probabilities of the fragility curves. Calculation of probability of exceedance for each damage state for earthquake of 0.65g are shown here. Statistical properties of probability distribution of global drift ratios for earthquake of PGA 0.65g.

Mean of global drift ratios for Earthquake of 0.65g (μ)	0.0047
Standard deviation of global drift ratios for Earthquake of 0.65g (σ)	0.00123
Median of global drift ratios for Earthquake of 0.65g	0.0049

Damage state	Y	Z **{Z=(Y- μ)/ σ }**	Probability of exccedance
Immiadiate occupancy	0.0011	-2.93	1
Life safety	0.0024	-1.87	0.97
Collapse preventation	0.0048	0.08	0.47

Table 4.4 Statistical properties of probability distribution of global drift ratios for earthquake of PGA 1.20g and corresponding Z value and probability of exceedance for each damage state.

PGA of earthquake	IO	LS	CP	MEAN	MEDIAN	STANDARD DEVIATION
0.10g	**0**	**0**	**0**	**0.0005**	**0.0005**	**0.00013**
0.15g	**0.08**	**0**	**0**	**0.0008**	**0.0008**	**0.00021**
0.20g	**0.63**	**0**	**0**	**0.0012**	**0.00125**	**0.00029**
0.25g	**0.88**	**0.03**	**0**	**0.0016**	**0.00163**	**0.00043**
0.30g	**0.96**	**0.21**	**0**	**0.002**	**0.0021**	**0.0005**
0.35g	**0.99**	**0.25**	**0**	**0.0024**	**0.0024**	**0.00058**
0.40g	**0.99**	**0.73**	**0.09**	**0.0028**	**0.00285**	**0.00067**
0.45g	**0.99**	**0.79**	**0.22**	**0.003**	**0.0029**	**0.00076**
0.50g	**1**	**0.91**	**0.35**	**0.0036**	**0.00365**	**0.00088**
0.55g	**1**	**0.94**	**0.47**	**0.004**	**0.0041**	**0.00102**
0.60g	**1**	**0.97**	**0.63**	**0.0044**	**0.00445**	**0.00106**
0.65g	**1**	**0.97**	**0.70**	**0.0047**	**0.0049**	**0.00123**
0.70g	**1**	**0.99**	**0.97**	**0.0052**	**0.00525**	**0.00125**
0.75g	**1**	**0.99**	**0.99**	**0.0055**	**0.0056**	**0.00135**

Table 4.8: Statistical properties of probability density function of drift ratios and probability of exceedance of a given damage threshold

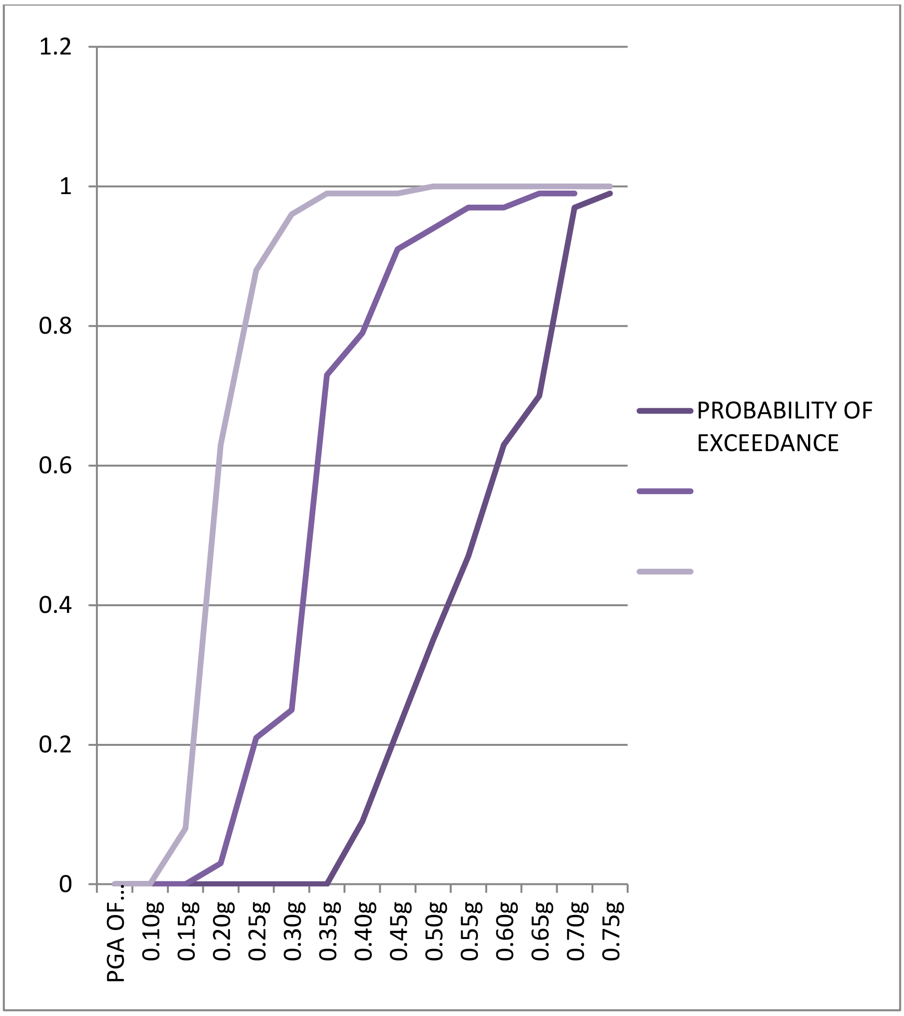


Fig 4.13 Fragility curves for IO LS and CP

Chapter 5

CONCLUSIONS

5.1 CONCLUSIONS

Building fragility curves describe the probability of reaching or exceeding, structural and non-structural damage states, given median estimates of spectral response, for example global drift ratios. These curves take into account the variability and uncertainty associated with capacity curve properties, damage states and ground shaking. The fragility curves distribute damage among slight, moderate and severe damage states. For any given value of spectral response, damage state probabilities are calculated as the difference of the cumulative probabilities of reaching, or exceeding, successive damage states. The probabilities of a building reaching or exceeding the various damage levels at a given response level sum to 100%. Each fragility curve is defined by a median value of the demand parameter that corresponds to the threshold of that damage state and by the variability associated with that damage state.

REFERENCES

Abrams, D.P., 2002. "Consequence-based engineering approaches for reducing lossin mid-America," Linbeck Distinguished Lecture Series in Earthquake Engineering, University of Notre Dame.

Akkar, S., Sucuoglu, H. And Yakut A., 2004. "Displacement-Based Fragility Functions for Low and Mid-rise Ordinary Concrete Buildings", Earthquake Spectra, in review.

American Society of Civil Engineers (ASCE), 2000. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Report No. FEMA-356, Washington,D.C.

ATC-40, 1996. "Seismic Evaluation and Retrofit of Concrete Buildings Volume I", Report No. SSC 96-01, Applied Technology Council.

Aycardi, L .E., Mander, J. B., and Rein horn, A. M., 1994. "Seismic Resistance of Reinforced Concrete Frame Structures Designed Only for Gravity Loads: Experimental Performance of Sub Assemblages," ACI Structural Journal, ACI, 91(5),552-563.

Bilham, R., V.K. Gaur and P. Molnar, 200 I. Himalayan seismic hazard. SCIENCE, 293.

Cardona, c., R. Davidson and C. Villacis, 1999. "Understanding urban scismic riskaround the world." Summary Report on the Comparative Study of the United nationsInternational Decade for Natural Disaster Reduction, RADIUS Initiative.

Computers and Structures, Inc., 2000. ETABS 2015 Nonlinear,, Structural Analysis Program, Berkeley CA.

Ellingwood, B.R., 1999. Probability-based Structural Design: Prospects forAcceptable Risk Bases, Keynote lecture, Proc. 8th International Conference onApplications oftatistics and Probability, Sydney, Austrailia, 1999.

Federal Emergency Management Agency, 2000. FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings. American Society of Civil Engineers, Washington DC.

FiIlibcn, J.J., Gurley, K.• Pinelli, J.P. and Simiu, E., 2002. "Fragility Curves, Damage Matrices and Wind Induced Loss Estimation." Proc. of the third International Conference on Computer Simulation in Risk Analysis and Hazard Mitigation, June 19-21, 2002, Sintra. Portugal, I J 9-126pp., 2002.

Gutenberg, B. and Richter, C. F., 1944. "Frequency of Earthquake in California." Bulletin of the Seismological Society of America. Vol. 34, No.4, pp. 1985-1988. Hwang, H.M. and Hun, J.R., 1996, "Simulation of Earthquake Acceleration Time Histories", Centre for Earthquake Research and Information, The University of Memphis, Technical Report.

Kafali, C. and Grignrin, M., 2004. "Ground Motion Simulator version 1.0", MCEER Program.

Kalkan, E., Eeri, S.M. and Kunnath, S.K., 2006. "Effects of fling step and forward directivity on seismic response of buildings." Earthquake Spectra, 22: 367-390.

Kircher, C.A., Nassar, A.A., Kustu, 0., and Holmes, W.T., 1997. "Development of building damage functions for earthquake loss estimation." Earthquake Spectra, 12: 663-682.

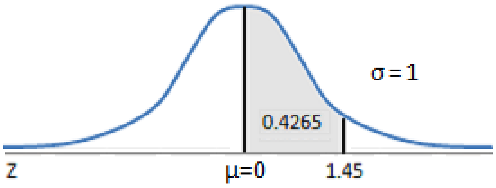
Liao, W.I. and Loh, C.H., 2002. "A Study on the Fragility of Highway Bridges in **Taiwan.".**

McGuire, R. K. 1978. "Seismic Ground Motion Parameter Relations." Journal of the Geotechnical Engineering Division, ASCE, Vol. 104. No. GT4, pp. 481-490.

APPENDIX A

Areas Under the One-Tailed Standard Normal Curve

This table provides the area between the mean and some Z score.
For example, when Z score = 1.45 the area = 0.4265.



Z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0040	0.0080	0.0120	0.0160	0.0199	0.0239	0.0279	0.0319	0.0359
0.1	0.0398	0.0438	0.0478	0.0517	0.0557	0.0596	0.0636	0.0675	0.0714	0.0753
0.2	0.0793	0.0832	0.0871	0.0910	0.0948	0.0987	0.1026	0.1064	0.1103	0.1141
0.3	0.1179	0.1217	0.1255	0.1293	0.1331	0.1368	0.1406	0.1443	0.1480	0.1517
0.4	0.1554	0.1591	0.1628	0.1664	0.1700	0.1736	0.1772	0.1808	0.1844	0.1879
0.5	0.1915	0.1950	0.1985	0.2019	0.2054	0.2088	0.2123	0.2157	0.2190	0.2224
0.6	0.2257	0.2291	0.2324	0.2357	0.2389	0.2422	0.2454	0.2486	0.2517	0.2549
0.7	0.2580	0.2611	0.2642	0.2673	0.2704	0.2734	0.2764	0.2794	0.2823	0.2852
0.8	0.2881	0.2910	0.2939	0.2967	0.2995	0.3023	0.3051	0.3078	0.3106	0.3133
0.9	0.3159	0.3186	0.3212	0.3238	0.3264	0.3289	0.3315	0.3340	0.3365	0.3389
1.0	0.3413	0.3438	0.3461	0.3485	0.3508	0.3531	0.3554	0.3577	0.3599	0.3621
1.1	0.3643	0.3665	0.3686	0.3708	0.3729	0.3749	0.3770	0.3790	0.3810	0.3830
1.2	0.3849	0.3869	0.3888	0.3907	0.3925	0.3944	0.3962	0.3980	0.3997	0.4015
1.3	0.4032	0.4049	0.4066	0.4082	0.4099	0.4115	0.4131	0.4147	0.4162	0.4177
1.4	0.4192	0.4207	0.4222	0.4236	0.4251	0.4265	0.4279	0.4292	0.4306	0.4319
1.5	0.4332	0.4345	0.4357	0.4370	0.4382	0.4394	0.4406	0.4418	0.4429	0.4441
1.6	0.4452	0.4463	0.4474	0.4484	0.4495	0.4505	0.4515	0.4525	0.4535	0.4545
1.7	0.4554	0.4564	0.4573	0.4582	0.4591	0.4599	0.4608	0.4616	0.4625	0.4633
1.8	0.4641	0.4649	0.4656	0.4664	0.4671	0.4678	0.4686	0.4693	0.4699	0.4706
1.9	0.4713	0.4719	0.4726	0.4732	0.4738	0.4744	0.4750	0.4756	0.4761	0.4767
2.0	0.4772	0.4778	0.4783	0.4788	0.4793	0.4798	0.4803	0.4808	0.4812	0.4817
2.1	0.4821	0.4826	0.4830	0.4834	0.4838	0.4842	0.4846	0.4850	0.4854	0.4857
2.2	0.4861	0.4864	0.4868	0.4871	0.4875	0.4878	0.4881	0.4884	0.4887	0.4890
2.3	0.4893	0.4896	0.4898	0.4901	0.4904	0.4906	0.4909	0.4911	0.4913	0.4916
2.4	0.4918	0.4920	0.4922	0.4925	0.4927	0.4929	0.4931	0.4932	0.4934	0.4936
2.5	0.4938	0.4940	0.4941	0.4943	0.4945	0.4946	0.4948	0.4949	0.4951	0.4952
2.6	0.4953	0.4955	0.4956	0.4957	0.4959	0.4960	0.4961	0.4962	0.4963	0.4964
2.7	0.4965	0.4966	0.4967	0.4968	0.4969	0.4970	0.4971	0.4972	0.4973	0.4974
2.8	0.4974	0.4975	0.4976	0.4977	0.4977	0.4978	0.4979	0.4979	0.4980	0.4981
2.9	0.4981	0.4982	0.4982	0.4983	0.4984	0.4984	0.4985	0.4985	0.4986	0.4986
3.0	0.4987	0.4987	0.4987	0.4988	0.4988	0.4989	0.4989	0.4989	0.4990	0.4990
3.1	0.4990	0.4991	0.4991	0.4991	0.4992	0.4992	0.4992	0.4992	0.4993	0.4993
3.2	0.4993	0.4993	0.4994	0.4994	0.4994	0.4994	0.4994	0.4995	0.4995	0.4995
3.3	0.4995	0.4995	0.4995	0.4996	0.4996	0.4996	0.4996	0.4996	0.4996	0.4997
3.4	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4997	0.4998
3.5	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998	0.4998
3.6	0.4998	0.4998	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.7	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.8	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999	0.4999
3.9	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000