TIME HISTORY ANALYSIS AND PLOT OF FRAGILITY CURVE

A Thesis submitted in partial fulfilment of the requirement for the degree of

Master of Technology
In
Structural Engineering

By
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CANDIDATES DECLARATION

I hereby declare that the project work entitled "**Time History analysis and plot of Fragility curve**" submitted to Department of Civil Engineering DTU is a record of work done by **Ravi Anand** 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 " **Time History analysis and plot of Fragility curve**" presented by **RAVI ANAND** is partial fulfilment of the requirement for the award of Mater 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 for the award of a degree or diploma.

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ACKNOWLEDGEMENT

I would like to express my sincere gratitude to my guide **Mr. 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 project. I would not have imagined having a better advisor and mentor for my project. It was a great privilege and honour to work and study under his guidance.

Besides my advisor I extend my sincere thanks to all faculties in Structural Engineering Department, DTU, Delhi for their timely co-operation during the project work. I wish to thank **Mr. Lalit Choudhary**, MTech STE,DTU for providing able guidance and helping me during project. So many people have contributed to my research work, and it is with great pleasure to take this 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 at high risk of facing devastating earthquakes. Since majority of population is living in earthquake prone areas, it is probable that such terrible events may take place in near future. Moreover it is not easy to cope up with 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 codes, seismic behaviour is not taken into consideration during selection of structural system and in most cases supervision in construction phases is not adequate which in turn indicates deficiencies like poor concrete quality, inadequate detailing of reinforcement etc. It is therefore vital to qualify the earthquake risk and to develop strategies for disaster mitigation.

This study describes the method by which it is possible to determine the vulnerability of existing engineering structures and building stock. The tool employed to assess the seismic performance of reinforced concrete framed structure is fragility curve. By definition, fragility curve provide estimates for the probability of reaching or exceeding various limit states at given level of ground shaking intensity.

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

R = Epicentre distance

h = Hypocentre depth of earthquake

K = Stiffness matrix

C = Damping matrix

M = diagonal mass matrix

KL = Stiffness matrix for linear elastic elements

IJ = Yield base shear coefficient

Θ_y = The yield global drift ratio

Θ_u = The Ultimate global drift ratio

Θ_{cp} = Collapse prevention performance limit

f_y = Yield strength of steel

f_{ck} = Compressive strength of concrete

V_y = Yield base shear capacity

M_s = Surface magnitude of earthquake

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Recent study exhibits that even moderate seismic tremor could be deadly populated, unplanned urban communities. Overall population and building network are presently ending up increasingly mindful of the circumstance. Indeed, even the execution of designed working under seismic occasion is questionable as enough work has not yet been done in this field. General public and the engineering community are now becoming more and more aware of situation. In this study the main objective is to present an appropriate method to assess the seismic performance of RCC structures.

The essential focus of the present examination is to decide the exceedance likelihood of various damage conditions of structure under seismic excitation. In performing a seismic risk analysis of a structural system, it is essential to recognize the seismic vulnerability of segment structures related with different conditions of harm. It is a widely practiced approach to develop vulnerability information in the form of fragility curves. One of the rising fields in seismic plan of structures is the Performance Based Design. The subject is still in the domain of research and scholastics, and is just gradually rising out into the professional's field. Seismic outline is gradually changing from a phase where a straight versatile examination for a structure was adequate for the two its flexible and bendable plan, to a phase where an extraordinarily devoted non-direct method is to be done, which at long last impacts the seismic outline in general. In Reinforced Concrete (RC) structures, the individuals (ie., bars and segments) are point by point, for example, to ensure that the structure can take the full effect without crumble past its Limit State limit up to its flexible limit.

The improvement of present day construction standards have furnished society with rules that work well for accomplishing the required safety levels. Fragility capacities are basic instrument for seismic loss estimations in build conditions. As far as possible states in fragility might be characterized as global drift ratio, inter storey drift ratio and so forth the ground movement forces in the fragility function can be spectral amounts, crest ground movement esteems, changed Mercalli scale and so on. Fragility curve includes uncertainty related with structural capacity, damage limit state definition and variability of ground movement intensity. Subsequently from fragility work the seismic execution of any structure can be analysed and its level of serviceability during a tremor can be assessed.

1.2 OBJECTIVE OF THE STUDY

The objective of the research are as follows:

1. To analysis the structure for seismic performance.

2. To construct the fragility curve of a particular type RCC building.

1.3 METHODOLOGY

There are two strategy to estimate the seismic fragility of explicit structure type. In the primary procedure which is known as experimental system, the harm reports are commonly used to set up the association between the ground movement force and the damage state of each structure. The second philosophy known as the methodical procedure is to lead the fragility studies by playing out the basic examination to survey the essential response to a ground development in term of internal forces and deformations. The upside of this methodology is that it is basic and monetarily feasible. In addition, non-particular decision maker slant toward such essential and quick estimates of anticipated misfortunes to develop the right judgment to execute their moderation plan. The principal methodology requires past seismic tremor damage data. The second philosophy is thus considered as a fitting strategy to assess seismic fragility of the structure. Peak ground acceleration is taken as ground motion intensity in this examination. Using quantitative damage limit state express the exceedance probability of a particular harm state are enlisted from the PGA from most extreme global drift scatters. The global drift percentiles more essential than a given damage edge level are figured by using typical appropriation to evaluate the exceedance probabilities of the fragility curve send the generally varying exceedance probability demonstrates are than smoothened to make fragility curve for that express damage state. ETABS is used in this study to estimate quantitative damage limit state by conducting nonlinear static analysis of developed model of building stocks and also perform nonlinear time history analysis.

CHAPTER 2

LITERATURE REVIEW

Seismic performance of structures and different structures is a fundamental characteristics for all agents that are associated with activities with land situated in seismically influenced regions. How well a specific building will perform during an earthquake sooner or later is imperative since it influences the present estimation of the property. Specifically, at any present time, a land proprietor can confront an arrangement of seismic hazard the board choices to browse: do nothing, offer the property, perform seismic retrofit or purchase earthquake insurance. In like manner, a potential proprietor (a man who needs to purchase a land property) faces comparable decisions: don't purchase, purchase and do nothing, purchase and retrofit, purchase and guarantee.

The way toward settling on a decision between a few options can be investigated by choice hypothesis. Here a simple technique of formal basic leadership process is sketched out. This study does not consider vulnerability in the results or hazard inclinations of leaders. The general methodology of choice hypothesis expresses that the best decision is the one that gives the most noteworthy utility among various alternatives (for insights about utility and choice hypothesis, Resnik 1987). Count of utilities for various choices relies upon the leader's destinations and inclinations. While applying this idea to the instance of a land proprietor or a purchaser, ordinarily the most pervasive concern is security. As far as choice hypothesis, this implies the higher the security of some choice, the higher is its utility, implying that utility is the expanding capacity of security. Ordinarily, it gets the job done to utilize an exceptionally short sighted utility capacity to represent the matter of security. It is advantageous to use a stage work. Such capacity essentially expresses that any alternative with the security not exactly some worthy dimension ought to be rejected. At the point when the wellbeing is higher than Sac, the utility is consistent, suggesting that there is no minimal profit by expanding security past the worthy dimension. This circumstance mirrors a methodology of land proprietors, where Sacre presents the security level given by modem construction standards. Then again, for a few proprietors, the worthy dimension of security is the one that meets least lawful prerequisites. In the two cases, when the security prerequisite is fulfilled, he or she couldn't care less if the wellbeing level is fundamentally higher than Sacre scarcely surpasses the limit esteem.

2.1 PERFORMANCE BASED ENGINEERING (PBE)

Performance based engineering (PBE) is another worldview for seismic hazard decrease crosswise over areas or interconnected frameworks (Abrams, 2002). In PBE, the hazard to a conveyed foundation frameworks is measured, assessed and managed through an appraisal and particular intervention procedure went for chosen segments of that framework. This procedure empowers the advantages of alternate seismic hazard moderation techniques to be evaluated as far as their effect on the performance of the built condition during a range of tremor risks and on the influenced population. Unmistakably segments and frameworks that are prevailing supporters of hazard ought to get the focus of consideration in the evaluation procedure underlying PBE. These prevailing benefactors can be recognized through the formalism of a probabilistic safety assessment, or PSA.

A PSA is an organized structure for assessing vulnerability, execution and dependability of a built framework, and as needs be must assume a central role in PBE. It is recognized from conventional deterministic ways to deal with safety confirmation by its attention on why and how the framework may fail and by its express treatment of vulnerabilities, both in the phenomena and in the investigative instruments used to display them. A PSA gives a premise to basic decision making within the sight of vulnerability that can be investigated by the partners of the task, inspected autonomously by a structure official or other administrative specialist, and refreshed occasionally as conditions warrant. The advance toward quantitative risk assessment started in the atomic business in the mid-1970"s, and has quickened lately as the advantages of quantitative hazard investigation have turned out to be obvious in numerous fields (Ellingwood, 1999).

One starts the PSA procedure by distinguishing limit states (LS), or conditions in which the framework stops to play out its expected capacities in some way. In a (restricted) structural designing sense, such limit states for explicit basic segments and frameworks might be either quality or deformation related (as discussed subsequently). In a more extensive financial setting, the LS might be identified with repair costs (e.g., communicated as a level of substitution esteem) that are more than an ideal sum, opportunity misfortunes, or grimness/mortality. Limit state distinguishing proof requires a thorough comprehension of the conduct of the safety related frameworks inside the plant and the job of basic parts and frameworks in guaranteeing worthy conduct of such frameworks. With the limit states distinguished, the limit state probability can be expressed as,

$$P[LS] = \sum P[LS|D = d]\ P[D = d]$$

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**.

2.2 FRAGILITY

The fragility of a segment or framework characterizes the restrictive likelihood of its achieving a performance limit state, which may extend from loss of capacity to beginning breakdown, given the event of a specific operational or ecological interest Shows that evaluation of structural fragility is a key element of any PSA. Moreover, fragility capacity gives a probabilistic proportion of safety edge as for structure premise or different occasions indicated by a partner. Such a margin can be utilized to assess framework shortcomings or insufficiency recognized during a review or condition appraisal and can give a way to evaluate if the observed shortcomings or lacks may be required to significantly affect framework. Demonstrating and building investigation give a proportion of reaction to a recommended interest. For instance, structural analysis of a structure for an outfit of ground movements, described by median peak ground acceleration, yields a relating set of distortions. Those distortions are unsure, because of vulnerabilities in the ground movement just as the dynamic properties depicting the structure and the structural demonstrating process itself. In turn, those deformations offer ascent to different conditions of harm and potential financial loss to structural and non-structural segments and frameworks. Those losses additionally are dubious, because of vulnerabilities in the deformations, resulting damages, and the monetary models used to display expenses related with various harm states.

2.3 FRAGILITY CURVE

As noted, fragility (or weakness) can be depicted as far as the conditional probability of a system achieving a recommended limit state (LS) for a given framework request D = d, P(LS/D = d). Limit -states identified with structural conduct run from un-functionality to different degrees of harm including beginning breakdown. Requests can be as greatest power, uprooting caused by seismic tremor ground movements, or all the more by and large an endorsed force proportion of the ground movement, over a given timeframe. Expressed in this general way, the fragility (or vulnerability) is a component of the framework limit against each limit state and also the vulnerability in the limit. The limit controls the focal area of the Fragility Curve (FC) and the vulnerability in the limit controls the shape (or scattering) of the FC. For a deterministic framework with no limit vulnerability, the FC is a step work. Entirely, FC is basically a property of the framework subject as far as possible state. A fragility examination is a fundamental element of the completely coupled hazard investigation encapsulated in it, additionally can be utilized to decide probabilistic safety edges against explicit recognized occasions for choice purposes. Identification of probabilistic security edges is fundamental to modem engineered facility risk management. In spite of the fact that giving a less instructive proportion of safety than that got from the completely coupled hazard investigation. Hazard informed decision making dependent on the consequences of fragility evaluation has a few preferences:

(1) The probabilistic framework investigation is successfully uncoupled from the risk examination. Therefore, while learning of the danger is valuable in recognizing suitable occasions for hazard evaluation purposes (e.g., a 2,475yr mean repeat interim tremor), such information isn't basic. Missing believable information on such occasions, one may essentially ask with regards to the fragility was the plan premise occasion to be surpassed by some subjective edge, say 50 percent.

(2) The need to decipher and defend little limit state is kept away from. There are constrained information to help probabilities of this dimension, and such gauges are exceptionally reliant on the probabilistic models chose. At the current state-of-the-art,(conditional) fragilities are more strong than unrestricted limit state probabilities.

(3) An appropriately directed fragility analysis is less complex, less costly, and includes less teaches than a completely coupled hazard examination. As needs be, there is less probability of miscommunication among individuals from the hazard investigation group and the outcomes are all the more effectively comprehended by a non-specialist stakeholder or decision maker.

To tie the vulnerability of a given framework to the seismicity of the locale, the seismic risk should be incorporated into the consideration. The vulnerability should be portrayed regarding the probability of an arrangement of given limit states being come to of a framework at a given area over a given timeframe (0, t). Realizing the fragility curve, the limit state (LS) probability over the day and age (0, t) can be assessed.

2.4 RECENT WORKS ON FRAGILITY CURVE

The seismic fragility curve for RCC frame structures especially for structures and scaffolds have been studied and created by various specialists. A portion of the created fragility curve are appeared in the following part of this section. Akkar et aI., (2004) in his investigation built up the fragility curve for four distinct sorts of RCC structures in Turkey. Here light, moderate and serious limit states are IO, LS and CP separately. These fragility curves are shown in Figure 2. Erberik et ai, (2005) in Turkey additionally created fragility curve (Figure 3) for midrise in filled edges as far as PGV and PGA both. Wen et al, (2004) in MAE Canter Project DS-4 Report built up the fragility curve for a specific kind of RCC frame working as far as both FEMA and quantitative limit states. These are appeared in Figure 4. Shinozuka et aI, (2001) created fragility curve (Figure 4) for multi-range RCC spans. In this investigation, five quantitative harm states are created and used for characterizing limit states for fragility examination. Damage states appear in Table

Fragility curve for various sort of RCC structures and bridges are additionally created by various specialists. Ventuea et ai, (2001) assessed seismic loss in south western British

Columbia dependent on fragility curve. Simiu et aI, (2002) created fragility curve for RCC structures for wind-actuated loss estimation. Shinozuka et ai, (2001) performed study on statistical analysis for RCC bridges and created philosophies for constructing both experimental and systematic fragility curve for bridges.

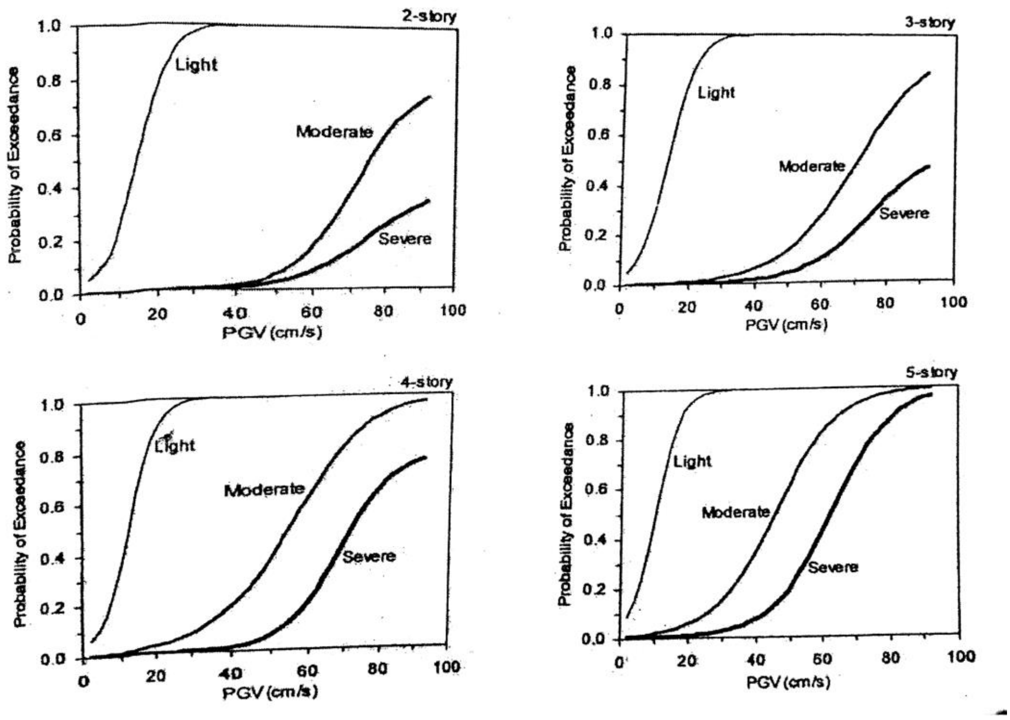


Fig 2.1.Fragility curve for (a) 2-, (b) 3-, (c) 4- and (d) 5- storey buildings (after Akkar et al.,2004)

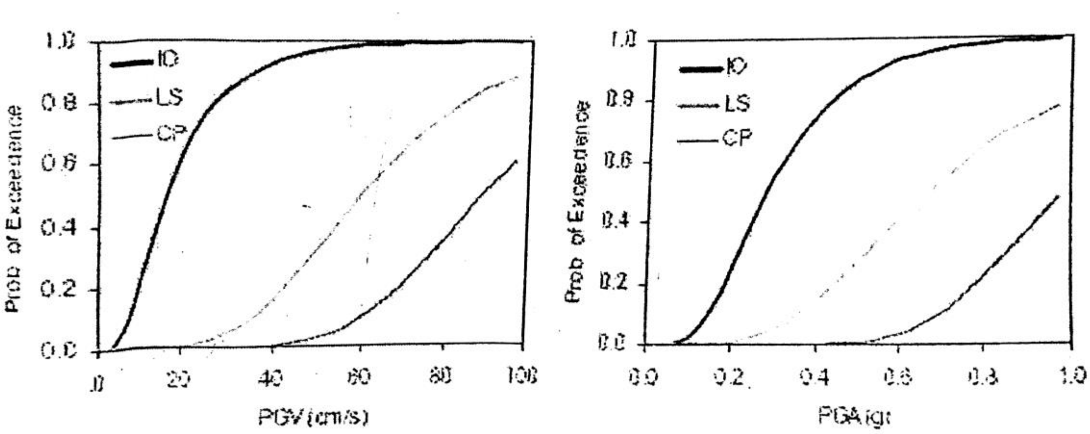


Fig 2.2. Sample fragility curve for mid rise in filled frames in terms of a) PGV, b)PGA

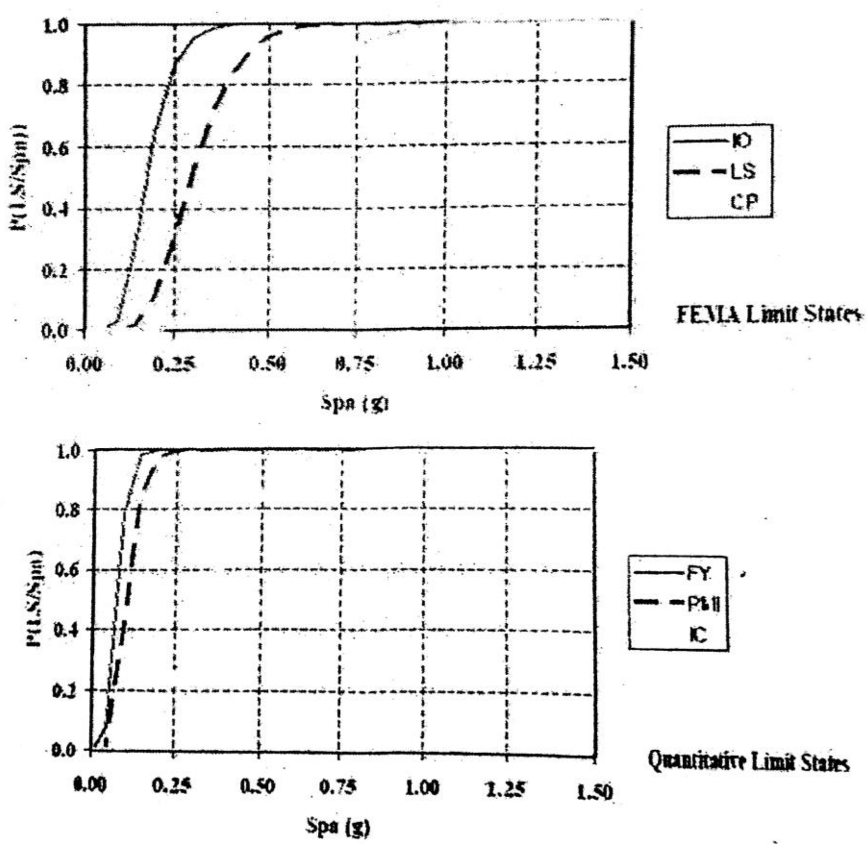


Fig 2.3.Sample fragility curves for both FEMA and Quantitative Limit State.
(After Wen et al.,2004)

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

2.5 QUALITATIVE APPROACHS

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

2.6 NON LINEAR STATIC ANALYSIS

Nonlinear static analysis is otherwise called pushover analysis. Although nonlinear static analysis has not recently been incorporated into configuration arrangements for new structure development, the methodology itself isn't new and has been utilized for a long time in both research and plan applications. For instance, nonlinear static analysis has been utilized for a long time as a standard approach in the plan of offshore stage structures. It additionally has been embraced in a few standard techniques for the seismic assessment and retrofit of structures, including the NEHRP Guidelines for the Seismic Rehabilitation of Buildings (FEMA 273) and Methodologies for Post-tremor Evaluation and Repair of Concrete and Masonry Buildings (ATC 40).

Despite the fact that it doesn't expressly show up in the NEHRP Recommended Provisions, the nonlinear static analysis philosophy shapes the reason for the comparable lateral force methodology contained in the arrangements for base separated structures and proposed for consideration for vitality scattered structures. Nonlinear static analysis gives an improved strategy for straightforwardly assessing nonlinear reaction of structures to strong seismic tremor ground shaking that can be an alternative option in contrast to the more unpredictable methodology of nonlinear reaction history annalysis. It is trusted that introduction of this methodology through consideration in this Structural Design Criteria.

One of the key discussions encompassing the presentation of this philosophy into the arrangements identifies with the determination of the limit disfigurement, some of the time likewise called as target displacement. A few approaches for evaluating the measure of distortion induced in a structure by the design seismic tremor have been proposed and are incorporated into different adoptions of the procedure. A nonlinear static analysis will comprise of an analysis of a numerical model of the structure that represents the nonlinear conduct of the structure's parts under a steadily expanded pattern of lateral loads. In this methodology a specific scientific model of the structure is steadily dislodged to a target displacement through use of a progression of lateral loads or until the structure breakdown and the subsequent internal forces, QEj, and member deformations,(Yt), at every augmentation of loading are resolved. At the target loading for the structure, the subsequent internal forces and deflection ought to be not exactly the limit of every component determined by the relevant acknowledgment criteria in Sec. 2.7.3 of FEMA 273. The investigation shall be performed as per this section.

Normally the shear opposed by the framework when the first element yields in the structure, in spite of the fact that not entirely for the whole structure, is characterized as the "elastic strength." When conventional direct techniques for configuration are utilized, together with R factors, the estimation of the design base shear sets the minimum quality at which this elastic strength point can happen. On the off chance that a structure is exposed to lateral loads bigger than represented by the elastic strength, at that point various components will yield, in the long run shaping a system. For most structures, multiple arrangements of components are possible. The system brought about by the little arrangement of loads is probably going to show up before others do. That instrument is viewed as the dominant component. Standard strategies for plastic or limit analysis can be utilized to decide the quality relating to such components.

The analysis method is planned to give a rearranged way to deal with directly deciding the nonlinear reaction conduct of a structure at various levels of lateral displacement, running from initial elastic response through improvement of a failure mechanism and commencement of collapse. If any basic component, or group of components, comes up short, at that point the whole structure may loose ability to convey the gravity loads, or any lateral loads. This condition can likewise happen if the lateral deformation turns out to be great to the point that the P-delta impacts surpass the residual lateral strength of the structure. Such conditions are characterized as breakdown and the twisting related with collapse characterized as ultimate deformation. This distortion can be determined by the nonlinear static strategy and furthermore by plastic or limit analysis. When the structure deform while components are yielding consecutively, the connection between external force and deformations can't be controlled by simple limit analysis. For such a case, different techniques for analysis are required. The reason for nonlinear static analysis is to give a simplified strategy for deciding structural reaction conduct at distortion levels intermediate to those which can be advantageously analysed utilizing limit state methods.

2.7 Time History Analysis

Modal superposition gives an exceedingly effective and precise methodology for performing time-history analysis. Close structure integration of the modal conditions issued to process the reaction, as summing linear variety of the time functions, between the input data time points. Thusly, numerical unsteadiness issues are never experienced, and the time addition might be any examining value that is esteemed fine enough to catch the greatest reaction values. One-tenth of the time period of the highest mode is recommended.The modes utilized are computed in a Modal Analysis Case that can be the undamped free vibration Modes (Eigen vectors) or the load dependent Ritz-vector Modes.

If majority of the spatial load vectors, are utilized as beginning load vectors for Ritz-vector analysis, at that point the Ritz vectors will dependably deliver more exact outcomes than if a similar number of eigen vectors is utilized. Since the Ritz-vector calculation is quicker than the Eigen vector calculation, the previous is suggested for time-history analysis.

It must be checked:

• That enough Modes have been figured

• That the Modes spread a satisfactory frequency range.

• That the dynamic load (mass) support mass proportions are sufficient for the load cases or potentially Acceleration Loads being applied

• That the modes shapes satisfactorily speak to every single wanted twisting.

2.8 NORMAL PROBABILITY DISTRIBUTION AND SAMPLING TECHNIQUE

Normal probability distribution is a significant continuous probability distribution. It comprises of an infinite number of potential qualities inside a predefined range. The normal probability distribution and its going with normal bend have the accompanying qualities:

1. The normal curve is bell molded and has a solitary top at the focal point of the circulation. The arithmetic mean, median and mode of the appropriation are equivalent and situated at the peak. Along these lines, a large portion of the zone under the curve is over this center point and the other half is beneath it.

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

3. The normal bend tumbles off easily in either direction from the focal value.

Probability test is characterized as an example is chosen so that everything in the populace has a known probability of being incorporated into the example. There are three strategies for probability sampling systems,

1. Simple Random Sampling

2. Stratified Random Sampling

3. Systematic Random Sampling

Simple Random Sampling

A sample chosen with the goal that every thing or individual in the population has a similar possibility of being incorporated. For this reason a distinguishing proof for every thing in the population and a table of arbitrary numbers are utilized.

Stratified Random Sampling

A population is isolated into subgroups, called strata, and an example is chosen from every stratum. Every one of these strategies depicted above are the systems for choosing fair examples from a given population. Impartial inspecting is basic for randomness of the gathered examples.

Systematic Random Sampling

In this procedure the things or people of the population are organized some way in order, in a document cabinet by date got, or by some other strategy. An irregular inspecting point is chosen, and after that each k^{th} individual from the population is chosen for the example.

CHAPTER 3

METHODOLOGY AND PROCEDURE

One of the targets of this work is to build up a rule to evaluate the weakness of Reinforced Concrete (RC) frame structures, explicitly in Delhi because of potential quakes. The seismic vulnerability of such development is depicted by methods for fragility curves, which relate the likelihood of surpassing a specific limit state given a forced seismic demand. In this work, seismic demand is characterized as the peak ground acceleration of a specific tremor.

3.1 REINFORCED CONCRETE FRAME STRUCTURES

Low to mid-rise RC frame structures situated in this region generally considered of low to moderate seismic hazard were commonly planned without thought of lateral loads, since wind load only from time to time administered for low-rise construction. In this way, such structures have been arranged as gravity load designed, or GLD structures. As a rule, GLD RC casing structures have no exceptional reinforcing details in the beam, segment, and joint regions. Another characteristics that recognizes these structures from others planned in territories of higher seismic hazard is the presence of solid shafts and weak sections, which can prompt delicate story failure systems that are made basically out of column hinging. The absence of adequate section quality prompts segment pivoting at moderately low lateral loads, causing the development of a story system once all segments situated on one story have pivoted. When the mechanism builds up, the structure's obstruction is given exclusively by the post-yield quality of the pivoting segment closures and characteristic segment flexibility. Consolidating the absence of adequate segment quality with the absence of adequate itemizing in segment areas for flexibility, fragile delicate story failure components might be noticeable during strong quakes.

3.2 TECHNIQUES TO DETERMINE SEISMIC VULNERABILITY

To appraise the seismic vulnerability of a particular structure type, two unique methodologies can be considered. In the first approach, each structure stock is analyzed independently and the weakness of the structure stock is gotten by joining the fragility data related with each structure. Very detail displaying and investigation methodology are utilized; consequently the outcome will be exceedingly exact. Then again, this methodology is for all intents and purposes and financially unfeasible. The second methodology is to lead the fragility studies by utilizing the statistical properties of the structure populace. Basic models and techniques are utilized in this methodology. The upside of this strategy is that it is straightforward and monetarily possible. What's more, the nontechnical leaders incline toward such basic and fast estimates of anticipated losses to build up the best possible judgment to execute their mitigation plans. However, the obtained outcomes will be rough and the impediments of the models or the techniques ought to be deliberately comprehended.

3.3 CAPACITY UNCERTAINTY

The part and framework capacity depend legitimately on the material qualities and firmness, which are characteristically arbitrary. The randomness can be demonstrated by arbitrary variable dependent on test information. It is entirely expected to utilize the initial two moments, ie. the mean and standard deviation (or coefficient of variety), to portray the central value and the fluctuation. Ordinary, log normal appropriations are usually utilized for accommodation. The real quality of the material of a given member varies, sometimes fundamentally, from the nominal qualities utilized in member estimations during structure. The connection between the nominal value and the actual value in this way should be built up to gauge the real member limit. The strength variability clearly relies upon the material, fabricating process, and now and again the testing convention. Material property fluctuation and test information can be found in the report by Ellingwood et al (1980). For instance, the coefficient of variety of strength of timber shifts in the range from 10 % to 30 % relying upon species and in flexure or pressure; and that of stone work from 10 % to 26 % relying upon arrangement and in pressure or flexure. The coefficient of variety of compressive and rigidity of cement is around 18 % and that of the yielding quality of steel reinforcement and steel rolled shapes is around 10 % or less. Properties of development material, for example, concrete and structural steel advance after some time. This variety in properties likewise nation explicit and changes in various nations and even in various locale inside a similar nation. Quality insights of more current material, for example, high-quality steel and cement might be found in more recent writing. For instance, measurements on yield and extreme quality of structural steel under different ecological conditions can be found in the ongoing FEMA/SAC report.

3.4 IDENTIFICATION OF IMPORTANT LIMIT STATE

Performance levels or limit states for both structural and non-structural frameworks are characterized as the point where the framework is never again equipped for fulfilling an ideal capacity. There are numerous sorts of performance levels in the field of tremor designing. Likewise, performance levels can be recognized by subjective and quantitative methodologies. The two techniques are summarized below.

3.4.1 Conventional Qualitative Approaches

Qualitative methodologies for distinguishing of performance levels have customarily been utilized in construction standards. Specifically, most construction codes expect designer to guarantee life security of the inhabitants during calculated loading and serviceability or functionality during un-factored loading. FEMA 356 has the most complete documentation on performance levels that are characterized subjectively. FEMA 356 characterizes execution levels identified with the structural framework as:

1. Immediate Occupancy (IO)

2. Life Safety (LS)

3. Collapse Prevention (CP)

3.4.2 Quantitative Approaches

Although current construction laws and best in class productions have endeavored to characterize the different performance levels for structural and non-structural frameworks, performance levels have just been recognized subjectively. In this way, creators' need to decide quantitative reaction constrains that compare to the qualitative code description. Another methodology for characterizing structural performance levels may be founded on quantitative strategies utilizing nonlinear pushover systems (ATC-40, 1996 and FEMA 356). By this pushover method modified qualities for various damage state, for example, Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) can be assessed.

3.5 DIMENSIONLESS BILINEAR CAPACITY CURVE

The capacity curve of each model can be approximated with a bilinear bend utilizing the rules given in FEMA-356 (ASCE, 2000). A typical idealisation of a capacity curve is appeared in Figure 2.5. It is required to determine the yield and ultimate strength limits and their related global drift ratio for developing the estimated bilinear cpacity curve. The global drift can be utilized to represent the damage limit state of the structures. The yield global drift ratio θ_y shows critical yielding of the framework when the yield base shear capacity (V_y) of the structure is achieved where as the ultimate global drift ratio θ_u relates to the state at which the structure achieves its twisting limit. The base shear coefficient $\eta = V_y/W$ in Figure 2.5 is the proportion of yield base shear ability to the structure weight.

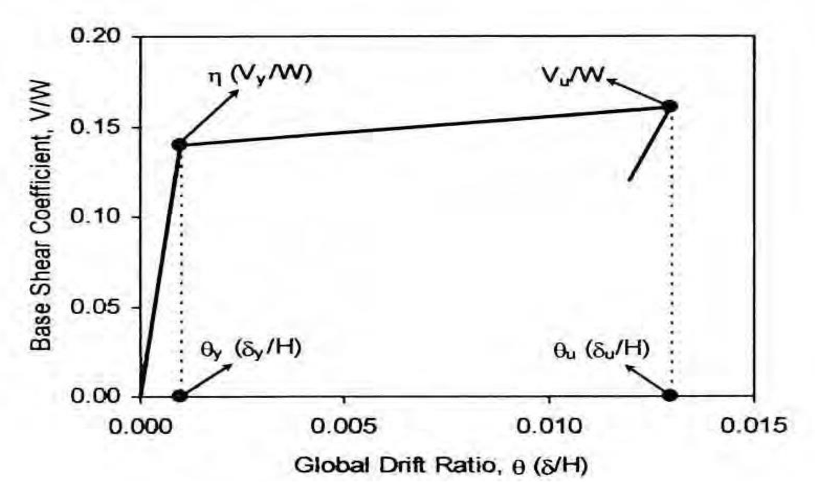


Fig 3.1 A typical bilinear capacity curve

It ought to be noticed that there is no universal agreement on the most proficient method to estimated capacity curve with a bilinear force distortion portrayal. An underlying firmness focusing at the condition of critical global yielding may prompt extensive varieties in Vy and ϴy in light of the fact that there is no particular point on the capacity curve precisely portraying significantly yielding (Sullivan et al., 2004).

3.6 IDENTIFICATION OF QUANTITATIVE LIMIT STATE

Probability density elements of Θ_y and Θ_u can be resolved in terms of mean, median and standard deviation. At the point when global ductility limits (Θ_u/Θ_y) are determined both Θ_y and Θ_u can be used to decide deformation limits. It is progressively fitting to utilize Θ_u in evaluating the twisting limits of such structures, which have infill walls or limited span length (Akkar, 2004)

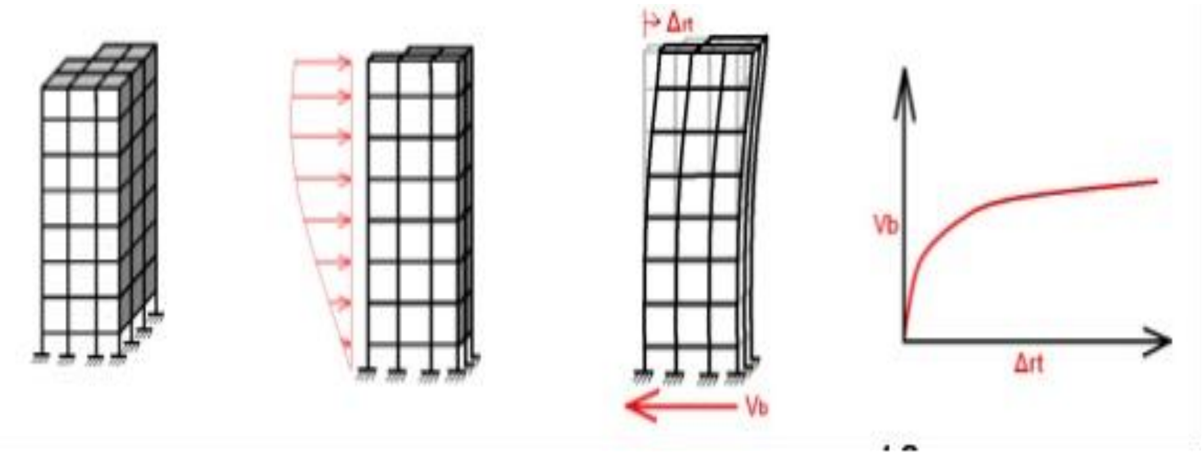
Three performance limits, immediate occupancy, life safety and collapse prevention that are indicated in a few other universal guidelines are generally received in fragility studies. The collapse prevention performance limit Θ_{cp} is taken as the 50% to 75% of the median Θ_u figured relying upon the development quality, dimension of certainty on appropriate plan and detailing, vulnerability in demonstrating and skewness of a definitive drift probability work. The life safety performance is assigned out as the 3 quartile or half of the proposed collapse prevention limit relying upon the vulnerability of structure. The median Θ_y registered for every story-based structure group is acknowledged to be the limiting value for the immediate occupancy performance level. It is expected that light, moderate and serious damage states are experienced when the immediate occupancy, life safety, collapse prevention drift points of confinement are surpassed, individually. The chose performance limits that are portrayed subjectively in Table 3.1 are approximate and could be contended as abstract. (Akkar,2004).

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

Table 3.1: Assumed drift ratio limits for performance levels

3.7 PUSHOVER ANALYSIS

Pushover analysis provide adequate information on seismic demands. Pushover analysis evaluate the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes. Evaluate the expected performance by means of a static inelastic 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

3.7.1 Introduction to FEMA-356

The main role of FEMA-356 archive is to give in fact sound and broadly adequate rules for the seismic recovery of structures. The rules for the seismic recovery of the structures are proposed to fill in as a prepared instrument for plan professional for doing the plan and examination of the structures, a reference report for the structure administrative authorities and an establishment for the future improvement and execution of the construction law arrangements and principles.

3.7.2 Introduction to ATC-40

Seismic assessment and retrofit of concrete structures regularly alluded to as ATC-40 was created by the Applied Technology Council (ATC) with financing from California Safety Commission. In spite of the fact that the systems prescribed in this report are for concrete structures, they are applicable to most structure types.

3.8 Different Pushover Approaches

Presently, there are two non-linear static analysis strategies accessible, one named as the Displacement Coefficient Method (DCM), recorded FEMA-356 and other the Capacity Spectrum Method (CSM) archived in ATC-40. The two strategies rely upon lateral load twisting variety acquired by non-linear static analysis under the gravity loading and lateral loading because of the seismic activity. This examination is called Pushover Analysis.

3.8.1 Capacity Spectrum Method

Capacity Spectrum Method is a non-linear static analysis system which gives a graphical portrayal of the normal seismic performance of the structure by crossing the structure's capacity spectrum with response spectrum of the quake. The intersection point is called as the performance point, and the dislodging coordinate dp of the performance point is the evaluated displacement demand on the structure for the predetermined dimension of seismic risk.

3.8.2 Displacement Coefficient Method

Displacement Coefficient Method is a non-linear static analysis technique which gives a numerical procedure to assessing the displacement demand on the structure, by utilizing a bilinear portrayal of the capacity curve and a progression of adjustment elements or coefficients to compute an objective displacement. The point on the capacity curve at the objective displacement is what could be compared to the performance point in the capacity spectrum method.

3.9 BUILDING PERFORMANCE LEVEL

Building execution is the joined presentation of both basic and non-basic parts of the structure. Distinctive performance levels are utilized to portray the structure execution utilizing the pushover analysis, which are depicted below.

3.9.1 Operational Level (OL)

According to this performance level structure are required to continue no permanant harms. Structure holds original strength and stiffness. Major breaking is found in partition walls and roofs just as in the structural components.

3.9.2 Immediate Occupancy level (IO)

Structures meeting this performance level are required to continue no drift and structure holds original strength and stiffness. Minor splitting in partition walls and basic components is observed. Lifts can be restarted. Fire protection is operable.

3.9.3 Life Safety Level (LS)

This dimension is demonstrated when some leftover strength and stiffness is left accessible in the structure. Gravity burden bearing components work, no out of plane failure of walls and tripping of parapet is seen. Some drift can be seen with some inability to the partition walls and the structure is past conservative fix. Among the non-basic components failing danger mitigates however numerous building and mechanical frameworks get harmed.

3.9.4 Collapse Prevention Level (CP)

Structures meeting this performance level are relied upon to have minimal remaining quality and firmness, however the load bearing basic components capacity, for example, load bearing walls and columns. Building is relied upon to support huge permanent drift, failure of partitions infill and parapets and broad harm to non-structural components. At this dimension the structure stays in collapse level.

CHAPTER 4

PLASTIC HINGES

4.1 GENERAL

A plastic hinge, in structural engineering, refers to the deformation of a part of a beam wherever plastic bending happens. Hinge means that having no capability to resist moment. Therefore, a plastic hinge behaves like a standard hinge - permitting free rotation. The concept of plastic hinge is important in understanding structural failure.

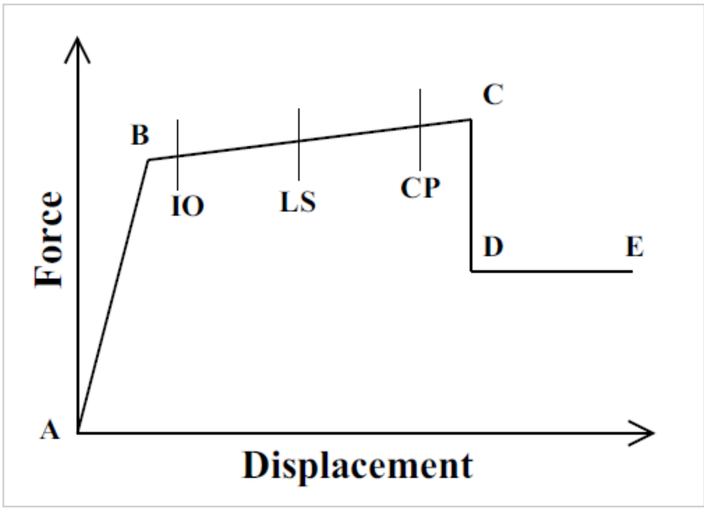


Fig 4.1 Force - Displacement curve of a Hinge.

4.2 FORMATION OF HINGES

The maximum moment brought about by the tremor happen close to the closures of the beams and columns, the plastic hinges are probably going to form there and most ductility necessities apply to segment close to the intersection.

4.3 TASK OF HINGES FOR PUSHOVER ANALYSIS

For nonlinear static, and nonlinear direct-integration time-history examinations, clients may reenact post-yield conduct by allotting concentrated plastic hinges to casing and tendon objects. Flexible behaviour happens over member length, and afterward distortion beyond the elastic limit occurs within hinges, which are displayed in discrete areas.

Inelastic conduct is acquired through mix of the plastic strain and plastic bend which happens inside a client characterized hinge length, regularly on the order of member depth (FEMA-356). To catch plasticity dispersed along member length, a progression of hinges might be displayed. Numerous hinges may likewise correspond at a similar area.

Plasticity might be associated with force displacement behaviours (axial and shear) or moment rotation (torsion and twisting). Hinges might be allocated (uncoupled) to any of the six DOF. Post-yield conduct is depicted by the general backbone relationship appeared to one side. The displaying of solidarity failure is discouraged, to relieve load redistribution (which may prompt dynamic breakdown) and to ensure numerical convergence.

CSI Software automatically negative slope to 10% of flexible stiffness, however overwrite alternatives are accessible. For enlightening purposes, additional limit states (IO, LS, CP) might be determined which are accounted for investigation, however don't influence results. Unloading from the purpose of plastic distortion pursues the slope of initial firmness.

Both P-M2-M3 hinges and fiber hinges are accessible to catch coupled axial and biaxial bending conduct. The P-M2-M3 hinge is most appropriate for nonlinear static pushover, though the fiber hinge is best for hysteretic elements.

4.4 Casing/Wall Nonlinear Hinge

Hinges properties are utilized to characterize nonlinear force-displacement or moment rotation conduct that can be assigned to discrete areas along the length of frame (line) objects or to the mid-height of wall objects. These nonlinear hinges are utilized during static nonlinear examination, fast nonlinear analysis (FNA) modal time history analysis, and nonlinear direct integration time history analysis. For every single other sort of examination, hinges are unbending and have no impact on the conduct of the member. The quantity of hinges influences calculation time, yet additionally the simplicity where model conduct and results might be deciphered. Consequently, it is firmly suggested that hinges be assigned uniquely at areas where the event of nonlinear conduct is exceedingly probable.

Three kinds of hinge properties are available in ETABS:

4.4.1 Auto Hinge Properties.

Auto hinge properties are characterized by the program. The program can't completely characterize the auto properties until the area to which they apply has been distinguished. Along these lines, the auto property is allocated to a frame or wall object, and the subsequent hinge property would then be able to be assessed.

4.4.2 User-Defined Hinge Properties

Client characterized hinge properties can be founded on auto properties or they can be completely client characterized.

4.4.3 Program Generated Hinge Properties

The created hinge properties are utilized in analysis. They can be seen, yet they can't be altered. Created hinge properties have a programmed naming of LabelH#, where Label is the frame and wall article name, H represents hinge, and # speaks to the hinge number. The program begins with hinge number 1 and additions the hinge number by one for each back to back hinge connected to the casing or wall object.

The principle explanation behind the separation between characterized properties (both auto and user defined) and created properties is that normally the hinge properties are area dependent. In this way, it is important to characterize an alternate set of hinge properties for each frame segment type in the model. This could conceivably imply that you would need to characterize countless hinge properties. To simplify this procedure, the idea of created properties is utilized in ETABS. At the point when produced properties are utilized, the program joins its implicit criteria with the characterized area properties for each article to create the last hinge properties. The net impact of this is you do fundamentally less work characterizing the hinge properties since you don't have to characterize each 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.

    - **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.
    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.

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.

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. 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.

4.5 CAPACITY

It is characterized as the normal extreme strength (in flexure, shear and axial loading) of the structural segments barring the reduction factors generally utilized in the structure of concrete member. The capacity for the most part alludes to the strength at the yield point of the component or structure's capacity curve. For twisting controlled component's, capacity past elastic limit large incorporates the impact of strain hardening.

4.5.1 Capacity Curve

The plot between base shear and rooftop removal is referred as capacity curve. Additionally, referenced as pushover curve.

4.6 Capacity Spectrum

The capacity curve changed from base shear v/s rooftop displacement (V v/s d) to spectral acceleration v/s spectral displacement (Sa v/s Sd) is referred as capacity spectrum.

4.6.1 Capacity Spectrum Method

A nonlinear static methodology that produce a graphical representation of the normal seismic performance of the structure by crossing the structure's capacity curve with a response spectrum representation of tremor's displacement demand on the structure, the meeting point is called performance point and the relocation coordinate dp of the performance point is the evaluated displacement demand on the structure for the predetermined level of hazard.

4.7 DEMAND

Demand is represented by an estimation of the displacement or distortion that the structure is relied upon to experience. This is in contrast to ordinary, flexible investigation strategies in which demand is represented by prescribed lateral forces applied to the structure.

4.7.1 Demand Spectrum

It is plot between average spectral acceleration versus time period. It represent the seismic tremor ground movement in capacity spectrum technique.

4.8 PUSHOVER ANALYSIS PROCEDURE

Pushover analysis methodology is principally used to assess the strength and drift limit of existing structure and the seismic interest for this structure exposed to chosen quake. This technique can be utilized for checking the sufficiency of new structural design also pushover analysis is characterized as an examination wearing a numerical model directly joining the normal load deformation attributes of individual segments and components of the structure will be exposed to monotonically interesting lateral burdens representing inertia forces in a quake until an objective relocation is excised.pushover examination evaluates the structural execution by assessing the forces and disfigurement capacity and seismic demand utilizing a nonlinear static analysis calculation.

Pushover analysis can be executed as either force control or displacement controlled relying upon the physical idea of the Lateral burden and conduct anticipated from the structure force. Force controlled system is helpful when the load is referred to, for example, gravity loading and the structure is required to have the option to support the load. Displacement controlled methodology ought to be utilized when a predetermined source, for example, in seismic loading where the magnitude of the applied loads isn't known ahead of time or when the structure can be required to lose strength or become unstable. The nonlinear pushover analysis of a structure is an iterative technique. It relies upon the last displacement as the effective damping relies upon the hysteretic energy loss because of inelastic deformation which thus relies upon the final deformation. This makes the analysis system iterative. Trouble in the arrangement is looked close to a definitive burden as the stiffness Matrix now winds up negative, clear because of structure turning into an instrument.

4.9 PUSHOVER ANALYSIS:

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

4.9.1 Create the 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.

4.9.2 Define a nonlinear static load

- 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.

4.9.3 Run analysis

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

4.9.4 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.

4.10 PROCEDURE FOR THE GENERATION OF FRAGILITY CURVES

Stage 1: Development of representative models of the structure stocks utilizing expected probability density capacity and general pattern of development parameters.

Stage 2: Nonlinear static Pushover system is then utilized to create bilinear capacity curve.

Stage 3: From the bilinear limit bends the yield base shear coefficient (Vy/W), the yield global drift ratio (ϴy) and the proportion of the post elastic slope of the bilinear capacity curve to the flexible slope (α) are then chosen as arbitrary factors and the measurable properties of these three quantities (Vy/W , ϴy and α) are decide.

Stage 4: From bilinear capacity curve ϴy and ϴu are assessed to identity quantitative limit state as far as global drift ratio: I0, LS and CP.

Stage 5: Nonlinear time history analysis are then completed to decide the maximum global drift ratio of the created models relating to every seismic tremor.

Stage 6: Using the harm threshold levels characterized in stage 4, the exceedance probabilities of a specific damage state are figured from the PGA versus maximum global drift dissipates.

Stage 8: The global drift percentiles more noteworthy than a given damage threshold level are processed by utilizing the typical circulation to evaluate the exceedance probabilities of the fragility curve and the roughly changing exceedance probability focuses are then smoothened to create fragility curve for that particular damage state.

CHAPTER 5
RESULT

5.1 SFD AND BMD OF FRAME

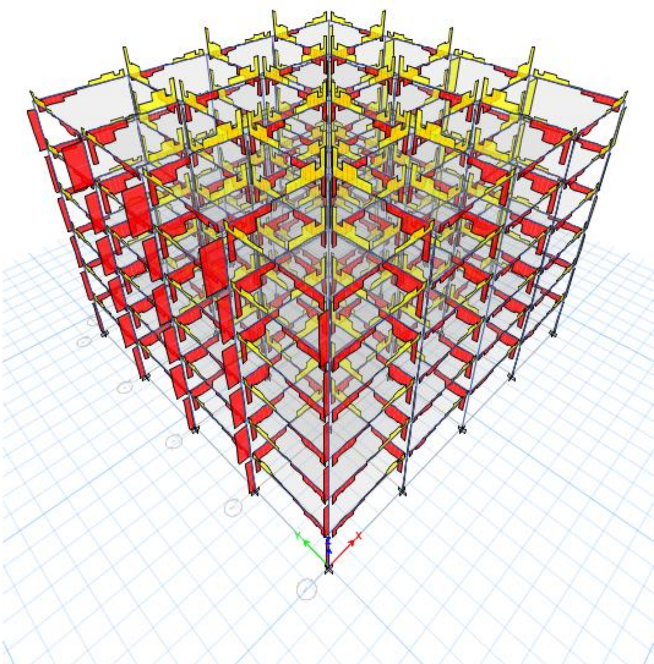


FIG 5.1 SFD

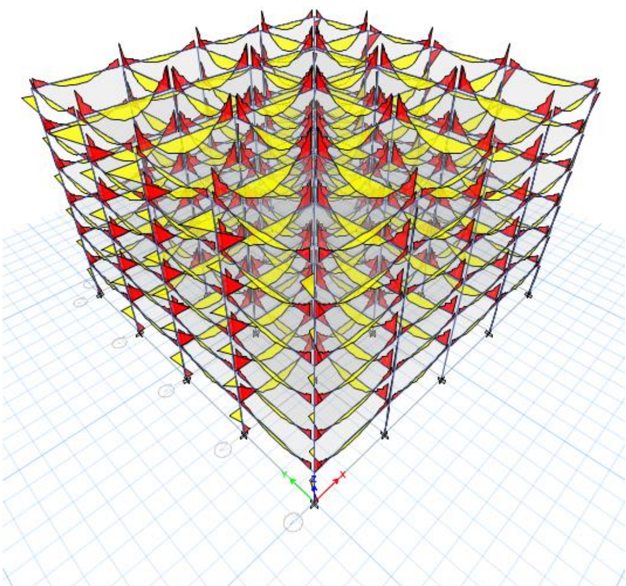


FIG 5.2 BMD

5.2 FRAME DESIGN

TABLE 5.1 SPECIFICATIONS OF THE MODEL

Beam size(mm)	**450*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)	**4**

TABLE 5.2 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**

TABLE 5.3 Story Data

Name	Height Mm	Elevation Mm	Master Story	Similar To	Splice Story
Story6	4000	24400	Yes	None	No
Story5	4000	20400	No	Story6	No
Story4	4000	16400	No	Story6	No
Story3	4000	12400	No	Story6	No
Story2	4000	8400	No	Story6	No
Story1	4400	4400	No	Story6	No
Base	0	0	No	None	No

TABLE 5.4 Material Properties

Name	Type	E MPa	ν	Unit Weight kN/m³	Design Strengths
HYSD500	Rebar	200000	0	76.9729	Fy=500 MPa, Fu=545 MPa
M25	Concrete	25000	0.2	24.9926	Fc=25 MPa
M30	Concrete	27386.13	0.2	24.9926	Fc=30 MPa

TABLE 5.5 Frame Sections

Name	Material	Shape
beam 450x600	M25	Concrete Rectangular
col 600x600	M30	Concrete Rectangular

TABLE 5.6 Shell Sections

Name	Design Type	Element Type	Material	Total Thickness Mm
Slab1	Slab	Shell-Thin	M25	150

TABLE 5.7 Reinforcement Sizes

Name	Diameter mm	Area mm²
18	18	255

5.3 Beam Element Details

Factored Forces and Moments

Factored M_{u3} kN-m	Factored T_u kN-m	Factored V_{u2} kN	Factored P_u kN
-58.3146	0.9023	64.6484	0

Design Moments, M_{u3} & M_t

Factored Moment kN-m	Factored M_t kN-m	Positive Moment kN-m	Negative Moment kN-m
-58.3146	1.5918	0	-59.9045

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)	-59.9049		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
64.6479	62.138	67	0	332.54

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
0.9023	64.6479	270	570	289.18

5.4 Column Element Details

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 % %
949.0459	-26.3482	26.3482	24.1047	24.1047	2880	0.8

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}	16.0937	193.2457	130.0824	0	667.06

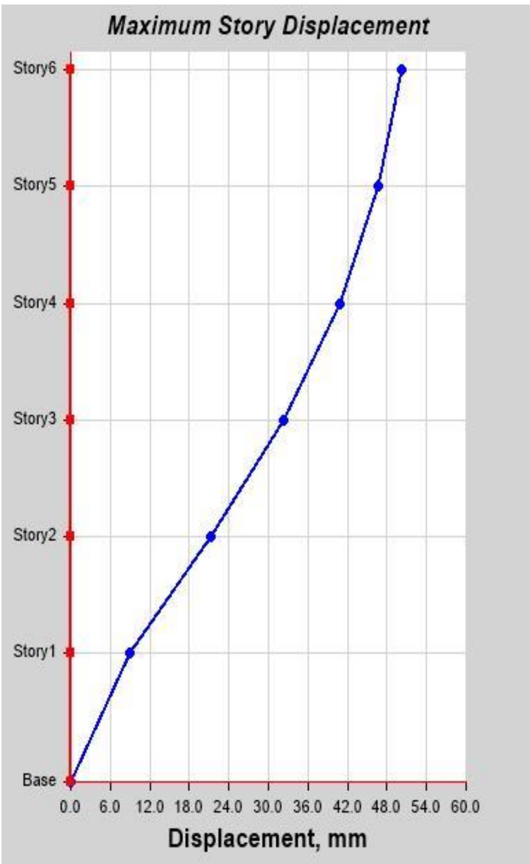


Fig 5.3 storey vs displacement

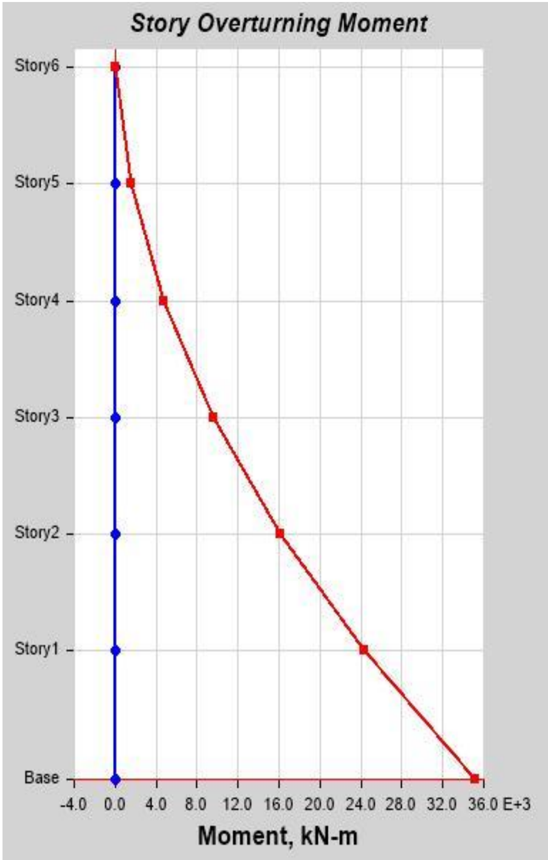


fig 5.4 storey vs moment

5.5 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	Story6	Response Direction	X
Point	1	Spectrum Widening	0 %

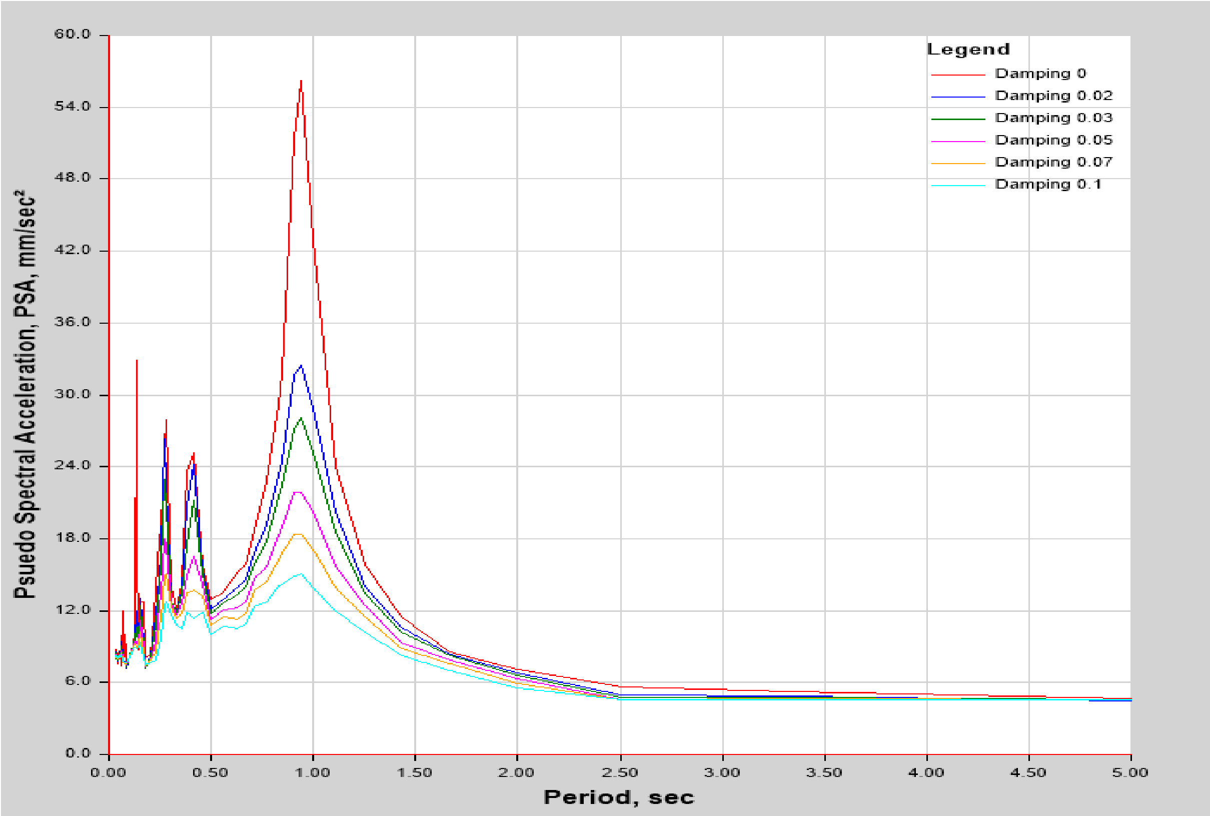


Fig 5.5 PSA VS PERIOD

TABLE 5.8 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	4.68	4.52	4.54	4.56	4.58	4.58
0.3	5.24	4.87	4.75	4.68	4.64	4.62
0.4	5.64	4.97	4.77	4.63	4.63	4.61
0.5	7.08	6.78	6.63	6.25	5.97	5.54
0.6	8.54	8.37	8.23	7.88	7.55	7.03
0.7	11.44	10.65	10.24	9.37	8.83	8.26
0.8	15.86	13.97	13.44	12.42	11.45	10.22
0.9	24.02	20.18	18.57	15.76	13.84	11.97
1	42.98	28.85	25.22	20.25	17.08	13.88
1.06	56.28	32.54	28.07	21.84	18.37	15.13
1.1	51.36	31.61	27.14	21.82	18.43	14.87
1.18	31.83	24.56	22.45	18.96	16.82	14.24
1.2	29.16	23.37	21.46	18.24	16.28	14.13
1.3	22.87	19.12	17.77	15.72	14.38	12.77
1.4	18.78	16.77	15.85	14.73	13.67	12.36

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/sec^2	mm/sec^2	mm/sec^2	mm/sec^2	mm/sec^2	mm/sec^2
1.5	15.84	14.64	13.97	12.75	11.78	10.87
1.6	15.16	13.91	13.34	12.26	11.34	10.45
1.8	13.52	12.96	12.66	12.06	11.47	10.71
2	12.90	12.13	11.75	11.27	10.76	10.08
2.2	15.76	15.51	15.12	14.24	13.25	11.83
2.4	25.22	24.26	21.15	16.52	13.73	11.36
2.6	23.67	20.65	17.47	14.86	13.53	11.87
2.8	14.24	13.67	13.36	12.63	11.75	10.45
3	12.15	11.96	11.92	11.68	11.37	10.86
3.3	13.98	12.78	12.76	12.64	12.33	11.74
3.6	27.86	22.35	20.73	17.47	15.06	12.73
3.64	25.82	26.36	22.96	17.96	15.01	12.55
4	18.76	16.73	15.66	13.25	11.23	9.73
4.06	18.04	14.84	14.18	12.56	11.03	9.13
4.4	13.72	11.58	10.83	9.51	8.54	7.84
4.7	9.85	9.34	8.98	8.42	8.17	7.76
5	8.26	8.16	8.07	7.97	7.83	7.66
5.5	8.18	7.22	7.23	7.33	7.44	7.47
6	12.74	9.21	8.91	8.94	8.56	8.12
6.5	12.74	13.02	12.42	11.17	10.22	9.30
7	8.78	9.08	9.07	9.06	9.13	8.98
7.46	18.16	11.56	10.52	9.44	8.92	8.83
7.5	32.87	11.95	10.61	9.44	8.97	8.82
8	10.02	9.84	9.74	9.47	9.23	8.93
8.30	9.35	9.27	9.24	9.14	9.05	8.83
8.5	9.08	9.06	9.06	8.97	8.87	8.75
9	8.77	8.75	8.72	8.66	8.63	8.54
10	8.25	8.24	8.24	8.24	8.21	8.16

5.4 FRAGILITY CURVE

A specific structure type is considered in this examination, in particular 6-story concrete structures, which for the most part don't consent to present day seismic safe plan and development practice. Three dimensional models are made in ETABS condition to perform nonlinear static analysis (pushover) and nonlinear time history analysis. As legitimate structure information are not accessible in our nation, models must be built utilizing accepted probability density capacity and general pattern of development parameters. The irregular factors (yield base shear coefficient, yield global drift proportion and the proportion of the post elastic slope of the bilinear capacity curve to elastic slope) are then chosen and measurable properties of these arbitrary factors as far as mean and standard deviation are then decided. These factual properties represent the group of structure stock which seismic vulnerability will be reflected by the produced fragility curve by investigating these models.

Three dimensional models are created and for models the development parameters are the fck, fy, column size, beam size and bay length. In this work fck and section size are taken as factor parameters. Beam size, fy of steel and bay length are kept consistent. Table 5.10 shows insights concerning different development parameters of created models.

Table 5.9 Details of construction parameter

Construction parameter	**Type**	**Dimension**
Concrete compressive strength(fck)	Variable	M30-M25
Steel yield strength(fy)	Constant	500 Mpa
Column size	Variable	(500*500)mm–(600*600)mm
Beam size	Constant	(450*600)mm
Bay length	Constant	8 m

Utilizing the variable development parameters appeared Table 5.9, 50 two dimensional models are created in ETABS having distinctive fck and column size. At that point nonlinear static analysis (pushover) is completed to create fragility curve for these 50 models. The bilinear capacity curve are developed for these 50 tests of structures. From these bilinear capacity curve the yield base shear co-coefficient(Vy/W), the yield global drift ratio (ϴy) and the proportion of the post elastic slope of the bilinear capacity curve to the elastic slope (α) are then chosen as irregular factors and the factual properties of these three amounts (Vy/W , ϴyand α) are resolved.

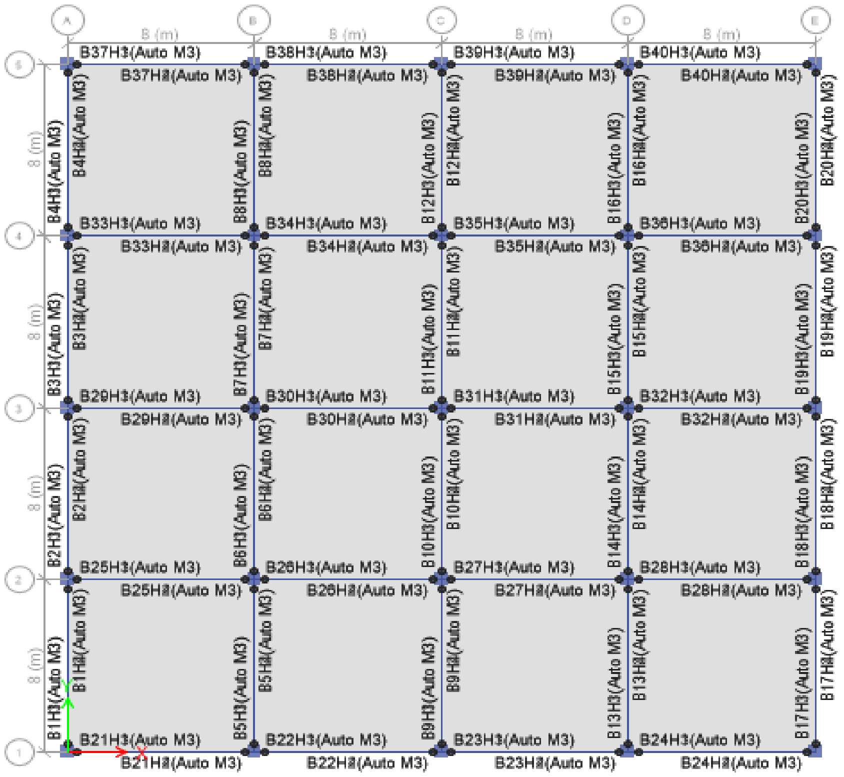


Fig 5.6 Beam hinges

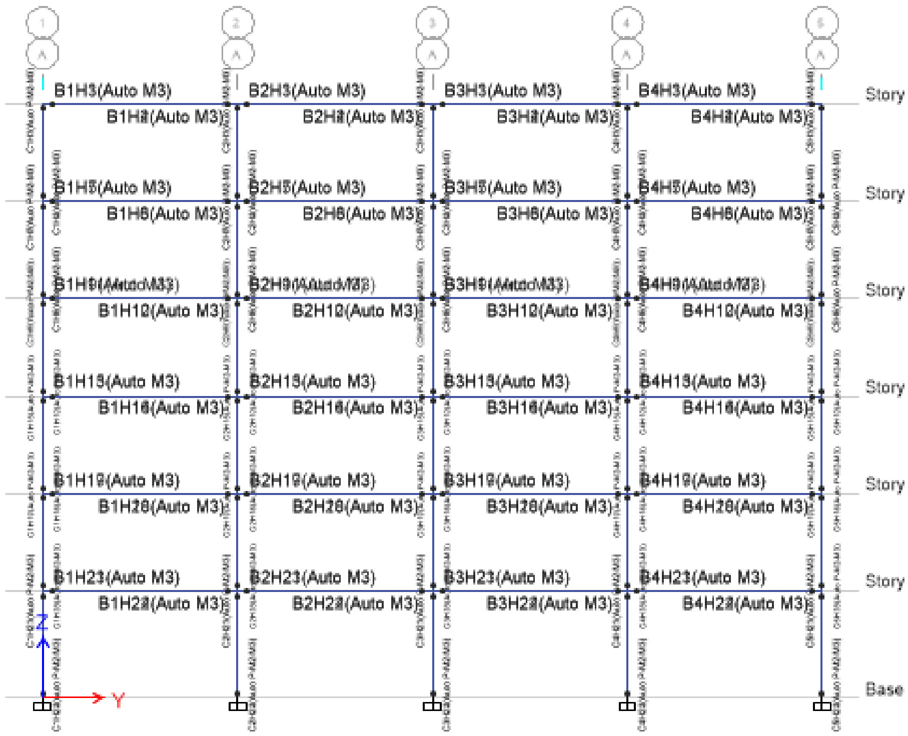


Fig 5.7 Column hinge

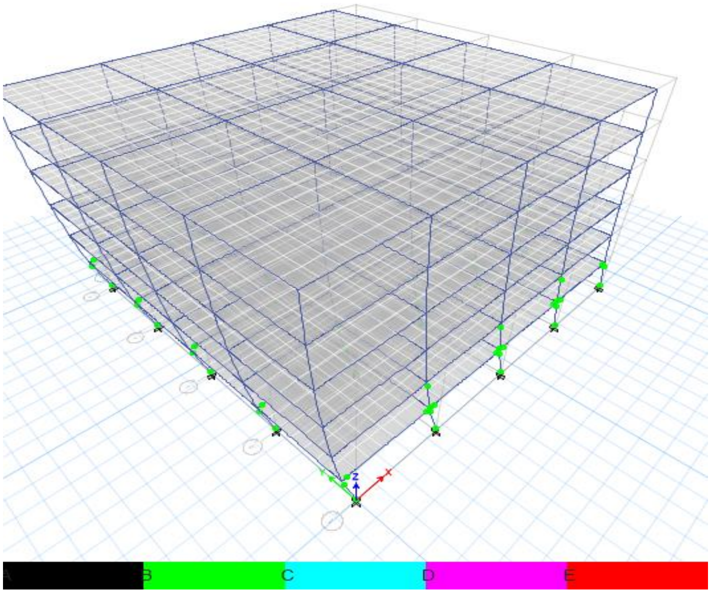


Fig 5.8 Step 12 of POA

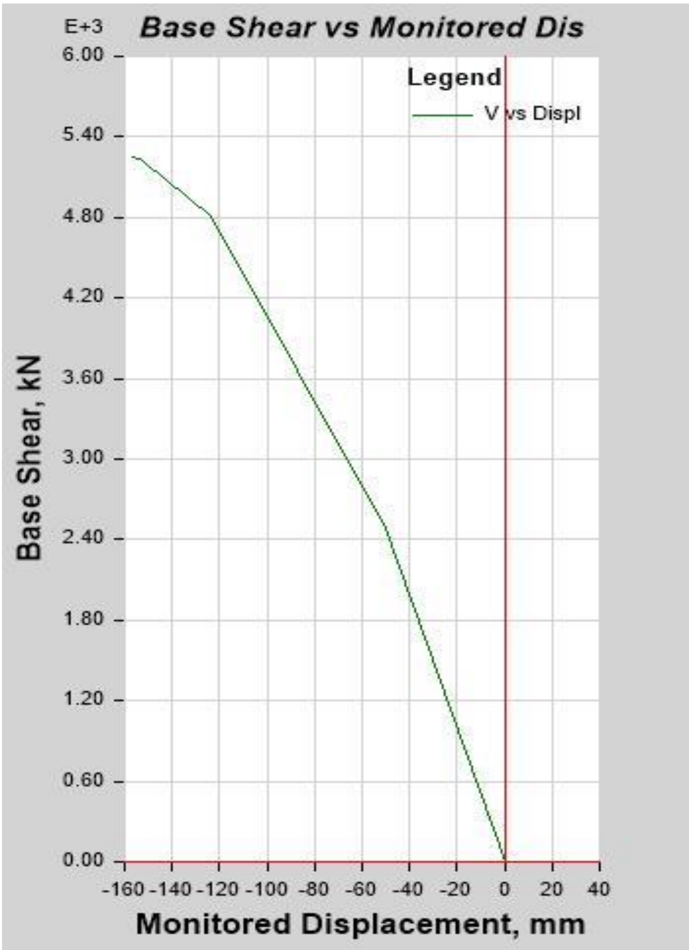


Fig 5.9 Pushover curve

5.7 IDENTIFICATION OF LIMIT STATES

From the 50 capacity curve probability density elements of Ɵy and Ɵu are resolved regarding mean, median and standard deviation. Three performance limits, immediate occupancy, life safety and collapse prevention that are determined in a few other worldwide rules are adopted in this fragility study.

From bilinear capacity curve Ɵy and Ɵu for thirty structures are resolved. The collapse prevention performance limit Ɵcp is taken as the 50 percent of the median Ɵu processed considering the inadequacy in development quality in this region,, absence of appropriate itemizing and vulnerability in demonstrating. The life safety performance is assigned as the half of the recommended collapse prevention limit and immediate occupancy is alloted to the 80 percent of median as most the structures in this area are not appropriately planned and definite for seismicity. It is expected that light, moderate and serious damage states are experienced when the immediate occupancy, life safety and collapse prevention drift limit of confinement are surpassed, individually.

Parameter	Mean	Median	Standard deviation
Ɵy	0.0014	0.0014	0.0092
Ɵu	0.0094	0.0095	0.0018

Table 5.10 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

5.8 NONLINEAR DYNAMIC TIME HISTORY ANALYSIS AND DEVELOPMENT OF FRAGILITY CURVES

The arrangement of quake records 0.10g to 0.65g used to figure the dynamic time-history reaction of the created models. The ETABS so as to recreate the condition of harm of each structure under ground increasing speed time-history. The global drift proportions are determined by dividing maximum value of the rooftop displacement, & top by normal structure height. For this situation the normal structure height is 24.4m. The maximum global drift values processed by the above method are then accepted to represent the seismic performance of the researched concrete casings. Utilizing the damage threshold levels characterized in Table 5.10, the exceedance probabilities of that specific fragility curve were registered. The probability distribution capacity is the standard ordinary or log-normal appropriation much of the time (Shinozuka et aI., 2000; Kircher et aI., 1997). From central limit hypothesis it is realized that if an arbitrary variable X is made of the total of numerous little impacts then X may be required to be normally distributed. The global drift percentiles more prominent than a given damage limit level are processed by utilizing the normal distribution to gauge the exceedance probabilities of the fragility curve. Table 5.11 portrays the measurable properties of probability density capacity of drift ratio regarding mean, median and standard deviation and probability of exceedance of a given damage limit for every one of the fourteen produced seismic tremors.

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 1.20g are shown here. Statistical properties of probability distribution of global drift ratios for earthquake of PGA 1.20g.

Mean of global drift ratios for Earthquake of 1.2g (X)	0.0047
Standard deviation of global drift ratios for Earthquake of 1.2g (α)	0.00123
Median of global drift ratios for Earthquake of 1.2g	0.0049

Damage state	Y	Z {Z=(Y-X)/α}	Probability of exccedance
Immediate occupancy	0.0011	-2.93	1
Life safety	0.0024	-1.87	0.97
Collapse prevention	0.0048	0.08	0.47

Table 5.11 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	0.0008	0.0008	0.00017
0.15g	0.07	0	0	0.0012	0.00012	0.00023
0.20g	0.68	0	0	0.0016	0.00162	0.00029
0.25g	0.89	0.05	0	0.0018	0.00183	0.00046
0.30g	0.95	0.26	0.02	0.0021	0.0021	0.00053
0.35g	0.99	0.45	0.07	0.0025	0.0025	0.00057
0.40g	0.99	0.75	0.09	0.0031	0.00315	0.00068
0.45g	1	0.82	0.27	0.0033	0.00324	0.00079
0.50g	1	0.96	0.35	0.0036	0.00365	0.00088
0.55g	1	0.98	0.49	0.004	0.0041	0.00103
0.60g	1.2	1	0.74	0.0044	0.00446	0.00107
0.65g	1.4	1.2	0.86	0.0049	0.00494	0.00128

Table 5.12 Statistical properties of probability density function of drift ratios and probability of exceedance of given damage threshold

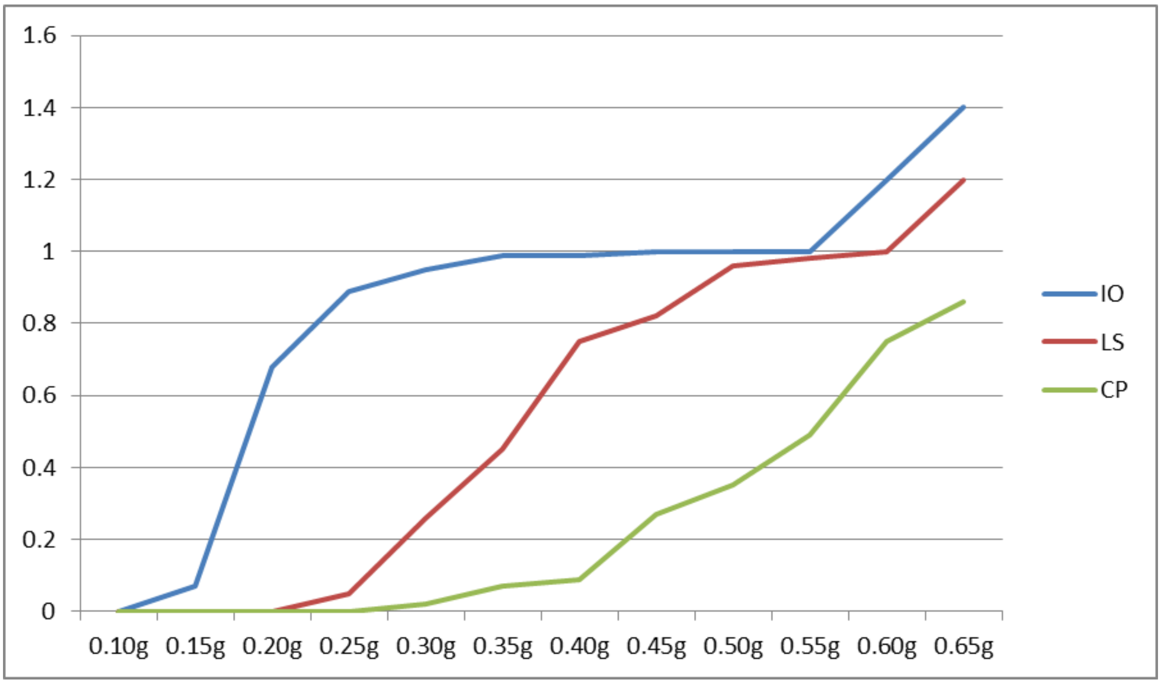


Fig 5.10 Fragility curve for IO, LS & CP

CHAPTER 6

CONCLUSION

Building fragility curve describes the probability of reaching or exceeding, structural and non-structural damage states. The method belongs to the category of analytical approaches to the fragility estimation and can account for all sources of variability, all possible modes of failure and their correlation. These curves takes into account the variability and uncertainty associated with capacity curve properties, damage state and ground shaking. The fragility curve distribute damage among slight, moderate, severe damage states. For any given value of spectral response, damage stste probabilities are calculated because the difference of the cumulative chances of achieving, or exceeding, successive harm states. The chances of a construct achieving or exceeding the numerous damage tiers at a given response stage sum to one hundred percent. Each fragility curve is described with the aid of an average value of the call for parameter that corresponds to the edge of that harm state and by using the variety associated with that damage state.

CHAPTER 7

REFERENCES

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.

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

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

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

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

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

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

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

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.

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.

Fajfar, P. and Fischinger, M., 1987, Non-linear seismic analysis of RC buildings: Implications of a case study, European Earthquake Engineering.

Lalit Choudhary, 2017, “Seismic Analysis and Development of fragility curve for RCC frame”

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.

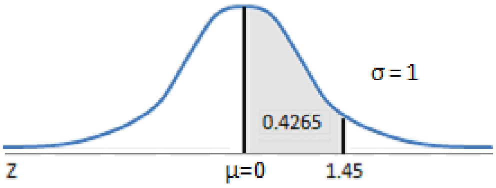
Shinozuka, M., Feng, M.Q., Lee, J. and Naganuma, T. (2000). Statistical analysis of fragility curves. Journal of Engineering Mechanics.

Erberik, M.A. “ Fragility based assessment of typical mid-rise and low-rise RC buildings in Turkey”.

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