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ESTIMATES OF SEISMIC FRAGILITY WITH UNCERTAINITY FOR A RC STRUCTURE

Manoj G[1], Shilpa B S[2]

[1]*P.G Student, Civil Structural Engineering Department, East West Institute of Technology, Bangalore*
[2]*Assistant Professor, Faculty of Civil Engineering, East West Institute of Technology, Bangalore*

Abstract - *In seismic risk mitigation policies an essential role is played by fragility functions. A destructive effect is acted upon the reinforced concrete structures by earthquake and it is crucial to determine and analyze the characteristics of the structure, its resistance to the damage caused to its elements by the shaking of the ground. According to the guidelines laid by HAZUS Manual for estimating direct losses from earthquake in RC building, an approach is developed. A model of G+12 storey building of 3X3 bay is created to analyze for material uncertainty in concrete and steel. In expressions of spectral acceleration and spectral displacement, the derivation for capacity curves and Discrete Seismic fragility estimates in done. Based upon the hybrid approach, the vulnerability assessment methodology is being made and combines approximately processed results from nonlinear static analysis i.e., Pushover analysis with statistical data. A step by step process of generating and analyzing the pushover curve and arriving at the discrete probability of damage state is presented. As per the codes, the design of the selected building typologies was carried using displacement based approaches.*

***Key Words*: pushover analysis, fragility curve, damage state probability, damage state threshold, HAZUS model, capacity spectrum**

1. INTRODUCTION

In the last 4 to 5 decades, there has been a momentous development in research of Earthquake Engineering. Researchers have been extensively rigorous on the concept of "Performance Based Earthquake Engineering (PBEE)". The key principle of the PBEE is to examine the performance of a structure for the duration of the "expected earthquake" at a proposed location. Performance-based approaches provide the coherent usage of source which provides vital economical contribution. It investigates the performance level of a structure based on its reaction to the level of loadings. Hence, earthquake engineers investigate this approach on a vast scale. The main aim is to gain sufficient data for a appropriate structural design for a required objective. Non-linear analysis is used to determine non-linear displacements. Hence, due to the complexity of Non-linear Response History Analysis (NRHA), the prominence of Non-linear Static Procedures amplified as a practical determination tool of seismic response.Linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis procedures are used to determine the seismic performances of a structure. The dynamic response value of the building increases from initial to final. Hence, for a nonlinear response, nonlinear dynamic analysis is accepted as a precise source. However, stability or convergence problem take place regularly due to its complexity. A major run time and post processing effort is necessary for Non-linear Response History Analysis (NRHA).Numerous aspects of assessment process, as well as the handling of uncertainties, can have a significant influence on the performance of evaluated collapse. Hence, for mitigation of disaster, management of disaster and preparedness of emergency, there is a necessity of an assessment for the risk of earthquake.

2. PUSHOVER ANALYSIS

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. The bi-linear or tri-linear load deformation diagram of all forces of resisting elements a 2 or 3-dimentional model is first created and gravity loads are applied initially. The distribution of a predefined lateral loads along the building height is made. Until the yielding of certain member occurs, loads are increased. To account for the reduced stiffness of yielded members, modification of structural model is done and until the additional members yield, the lateral loads are again increased. Until the structure becomes unstable or a control displacement at the building's top reaches some deformation level, the procedure is continued. In order to obatin the global capacity curve roof displacement vs base shear is illustrated. Performance of the Pushover analysis can be done by the following way:

1) Force-controlled Method
2) Displacement controlled Method

Force-controlled pushover technique is carried when a load is identified such as gravity load.

To perform pushover analysis, Displacement-controlled is a generally used method. For inelastic strength of inelastic strength and deformation demands that have to be compared with available capacities for a performance check, the internal forces and deformation computed at the target displacement are used.

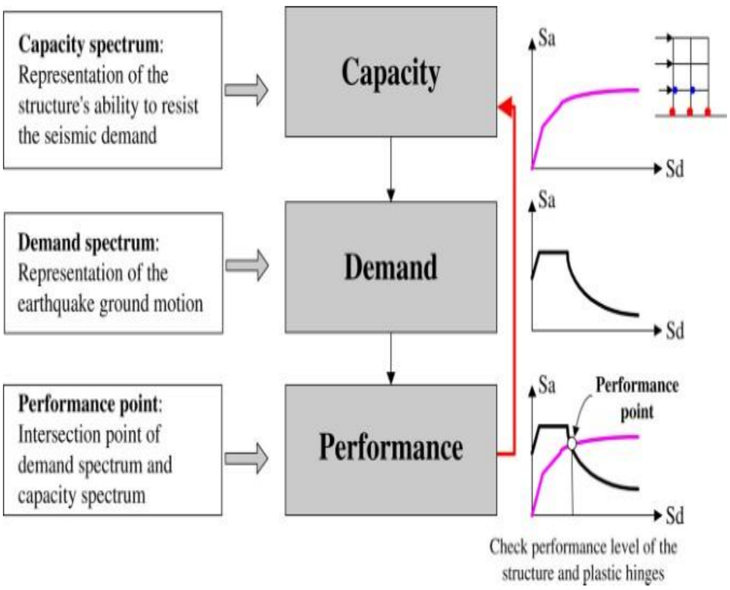


Fig. 2.1: Procedure for Nonlinear analysis

2.1.1 CAPACITY

The overall structure's capacity is defined by the strength as well as deformation limits of individual structural components. To find capacity further than elastic limits, non-linear pushover analysis is carried. For the determination of force-displacement curves of the reinforced concrete frame model, a sequential elastic series of analysis is used. Until the additional components yield, lateral loads are applied in sequential steps. The above steps are continued until the structure becomes unstable. Approximate idea about the structure's behaviors is given by the capacity curve, when the elastic limit is exceeded.

2.1.2 DEMAND

Deformations are produced in the structure whose pattern may vary with time, at the time of ground shaking. Non-linear methods include set of lateral displacements as a design condition. The maximum probable response of the structure at the time of ground shaking is defined as displacement demand of the reinforced concrete frame.

2.1.3 PERFROMANCE

It includes the extent of damage to structural as well as non-structural components beyond the satisfactory limits and is carried after determining the capacity and demand of the structure.

3.1 CONVENTIONAL PUSHOVER ANALYSIS

It includes, step by step solution of the equilibrium equation. During the analysis, a constant function including forces and displacement is created. The stiffness matrix is updated according to the calculated structural resistance, depending on the iterative procedure adopted. The three critical elements of the procedure is forcing nature of function, distribution and magnitude , that is, choosing a required value of applied action at each load step if they are not held constant.

3.2 TARGET DISPLACEMENT

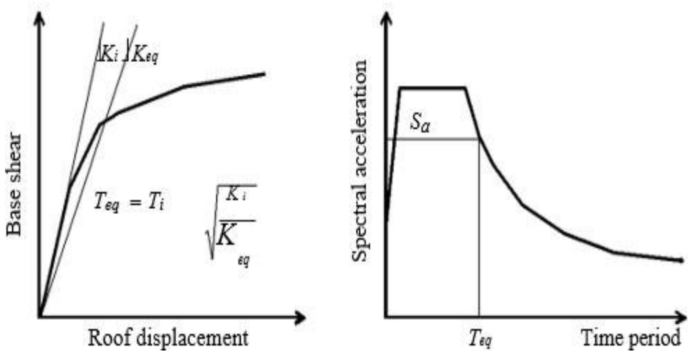
The demand of displacement for the building at the control mode put through the ground motion considered is called target displacement. As the global and responses of components of the structure at the target displacement are compared with the preferred limit state of performance to know the building performance, this is considered a significant parameter in pushover analysis. Target displacement can be calculated by the following methods:

1) Displacement Coefficient Method (DCM) of FEMA 356
2) Capacity Spectrum Method (CSM) of ATC 40

3.2.1 METHOD OF DISPLACEMENT COEFFICIENT (FEMA 356)

By assuming the properties initial linearity and damping for the ground motion excitation, this approach calculates the elastic displacement of an equivalent SDOF system. Multiplying with a set of displacement coefficient, the estimation of the total maximum inelastic displacement response at the root of the building is done. The base shear versus roof displacement curve is plotted. From initial period (T_i), an equivalent period (T_{eq}) is evaluated. The linear stiffness of the equivalent SDOF system is represented by equivalent period. From the response spectrum representing the considered seismic ground, calculation of the peak elastic spectral displacement consequent to this period is done.

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a$$



(a)Pushover Curve (b) Elastic Response Spectrum

Fig. 3.1: Graphical illustration of Method of Displacement Coefficient (FEMA 356)

The expression for the probable maximum roof displacement of the structure under the chosen seismic ground motion is as follows

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

3.2.2 METHOD OF CAPACITY SPECTRUM (ATC 40)

The maximum deformation of a linear elastic SDOF system with an equivalent period and damping is used to approximate the maximum inelastic deformation of a nonlinear SDOF system. The pushover curve in an acceleration-displacement system response spectrum (ADRS) format is used in this approach. By conversion by means of the dynamic properties of the system, this can be achieved. For the structure, the term “capacity spectrum” in an ADRS format is used for a pushover curve. The seismic ground motion is identified by a response spectrum in an identical ADRS format and is known as demand spectrum (Fig 3.2).

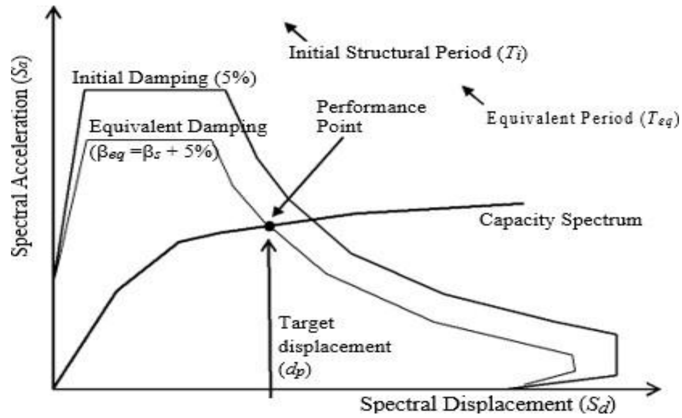


Fig. 3.2: Graphical illustration of Method of Capacity Spectrum (ATC 40)

$$T_{eq} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}}$$

$$\beta_{eq} = \beta_i + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} = 0.05 + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)}$$

The reduction factors to decrease spectral ordinate in the unvarying acceleration region and constant velocity region as a function of the effective damping ratio is provided by ATC 40. The expressions for spectral reduction are as follows:

3.3 STRUCTURAL LEVELS OF PERFORMANCE

$$SR_A = \frac{3.21 - 0.68\ln(100\beta_{eq})}{2.12}$$

$$SR_V = \frac{2.31 - 0.41\ln(100\beta_{eq})}{1.65}$$

AND RANGES:

It is defined by three discrete Structural Levels of Performance and two intermediate Ranges of Structural Performance. Owners of the building aspire an extensive range of structural performance requirements. To correlate with the most commonly specified structural performance requirement, selection of the structural performance levels is done. The requirement to customize building Rehabilitation Objectives is permitted by the two Ranges of Structural Performance.

Following are the Structural Performance Levels:

1) Level of Immediate Occupancy (S-1)
2) Level of Life Safety (S-3)
3) Level of Collapse prevention (S-5)

Following are the Structural Performance Ranges:

1) Range of Damage Control (S-2)
2) Range of Limited Safety (S-4)

To these intermediate ranges of performance, detailed design acceptance criteria are not provided. A suitable acceptance criteria needs to be resolute by the engineer expectation to design such performance. By interpolating the criteria of acceptance given for the Levels of Immediate Occupancy and Life Safety Performance, criteria of acceptance for performance within the Damage Control Range can be achieved. Also, by interpolating the criteria of acceptance for performance within the Life Safety and Collapse Prevention Performance Levels , is the acceptance criteria for the performance within the Range of Limited Safety can be achieved.

4.1 FRAGILITY ANALYSIS OF RC STRUCTURE

The relation among probabilities of reaching far beyond specific damage level verses earth intensity is a continuous curve and is called as a fragility curve or a vulnerability curve. The concept here is that for a given earthquake intensity, the comparable type of structures will have identical probability of a given damage state. For a recurring earthquake catastrophe planning and aftermath-earthquake resurgence and retrofitting works, the usage of the curves of fragility for the evaluation of seismic losses is in escalating order.

The likelihood of attaining or exceeding, structural and non-structural states of damage, given median estimates of spectral response, such as spectral displacements are explained by developing fragility curves which are functions of lognormal. The variability and uncertainty related with properties of capacity curve, damage states and ground shaking is taken into account by these curves. Fragility curves classified the sates of damages as Slight, moderate, extensive and complete damage sates. To calculate a variety of building losses, the discrete probabilities of damage- states are used as inputs. With a median value of the demand value of the parameter of demand that correlate with to the threshold of that state of damage and by the variability connected with that damage state defines each of the fragility curve. The curves of fragility are given in Fig 4.1

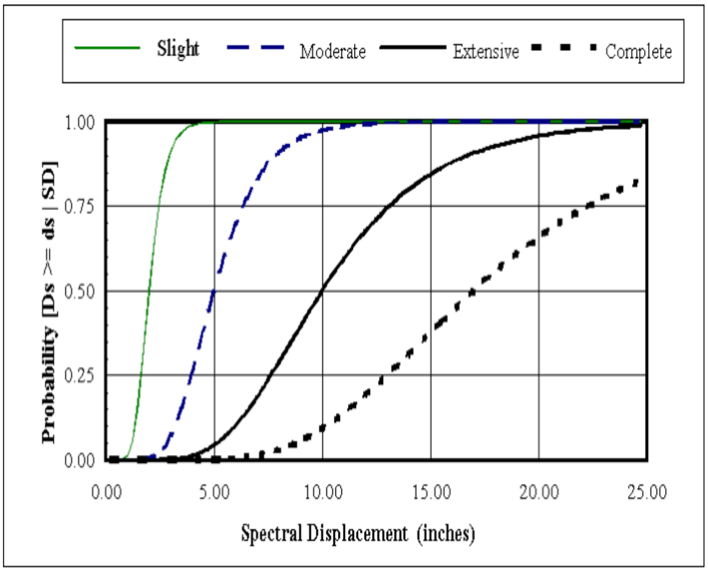


Fig. 4.1: Damage states fragility curves examples.

4.2 COMPUTATION OF DAMAGE PROBABILITY

The prospect of a probable global damage, d, of a building exceeding a specified state of damage, ds, as a function of a specification quantify the rigorousness of the seismic is defined by the fragility curve. Hence, the corresponding fragility curve for each damage state is defines by representing P[d ≥ ds] in the y-axis and the spectral displacement , Sd, in represented in the x-axis. Following lognormal probability density expression describes a curve of fragility for a specified state of damage [4.1].

$$P\left[\frac{ds}{ds}\right] = \phi\left[\frac{1}{\beta ds}\ln\left(\frac{Sd}{Sd,ds}\right)\right] \quad (4.1)$$

Where,

$\overline{S_{ds}}$ = Spectral displacement Median value where the structure arrive at the threshold sate of damage, *ds*

βds = Natural logarithm Standard deviation of spectral displacement for state of damage damage, ds

ϕ = Standard normal cumulative distributive function

Sd = Specified peak spectral displacement

4.3 Damage state medians development

For each of the states of damage and fro every one of components of structural and non-structural elements, median values of curves of fragility are developed. On the basis of ratios of building drift that express the threshold of states of damage, median values of structural components are defined. The following expression is used to convert drift of damage-state to spectral displacement:

$$\bar{S}_{d,ds} = \delta_{R,Sds}\, h\, \alpha_1$$

Where,

$\bar{S}_{d,}$ = spectral displacement median value (in terms of mm) of components of structure for states of damage, *ds*

$\delta_{R,}$ = ratio of drift at the structural damage state threshold, *ds.*

h = building height at the level of roof.

α = participation factor of modal mass for the natural mode of first order

4.4 Development of damage state variability β_{ds}

The variability of curves of fragility for every one of the states of damage is explained by the lognormal standard deviation (β). The amalgamation of the three influencers to damage variability of structure β_c, β_D, $\beta_{M(Sds)}$ models the total variability of all of the structural state of damage, β_{ds}.

$$\beta_{ds} = \sqrt{(conv[\beta c, \beta D \bar{S}_{d,ds}]^2 + (\beta M(Sds))}$$

Where,

β_{ds} = deviation of lognormal standard that express the variability for states of damage of a structure, *ds*

β_c = deviation of lognormal standard parameter that express the variability of the curves of capacity.

β_D = deviation of lognormal standard parameter that express the variability of the spectrum of demand.

(Sds)= deviation of lognormal standard parameter that express the uncertainty in the median value estimates of the structural damage state threshold, *ds.*

Demand and capacity are factors that the variability of building response depends on. A compound procedure of combing a series of distributions of probability of the demand spectrum, and capacity is indicated by a function “conv”. The properties of median and variability criteria β_D and β_c parameter respectively, describe the spectrum of demand and capacity and are defined probabilistically.

Following criteria is based on the HAZUS standard deviation values:

1) Height group of building.
2) Post-yield deprivation of the system of structural.
3) Threshold variability of states of damage.
4) Variability of curves of capacity.

4.5 STATES OF DAMAGE

For investigating the expected patterns of damage in a specified area for dissimilar circumstances of earthquakes, the building forecast of damage may be used. This helps in understanding the characteristics and magnitude of the damage to a structure type from the forecast of damage output so that life-safety, function of social and monetary losses, the outcome of the damage can be evaluated. As a continuous functions of deformations of building, damage of the structure vary from “none” to “complete”. The approach evaluates the states of structural damage in the following ranges of “states of damage”:

1) Slight state of damage
2) Moderate state of damage
3) Extensive state of damage
4) Complete state of damage

4.6 Damage state threshold

In this study, damage states of four are taken into account for a structure, obtaining the damage articulated as matrices of probability. The structural characteristics uncertainties and damage state thresholds uncertainties contain a major authority on the results, even after the usage of significantly improved approaches. To study the effect of uncertainties in the damage stated threshold of a RC structure is the major purpose of the paper. The main approach is obtaining curves of probabilistic vulnerability which contemplate the states of damage threshold as haphazard variable. In conduction risk analysis of urban area, these curves are useful. With this records of curves influencing all the offered typologies of structures can be understood. The definition of the displacement of median spectrum, for each of the damage is given (4.1). For defining threshold for each one of the states of damage, capacity spectrum is used in this approach. Pushover is converted into capacity spectrum (ADRS format). Fig.(4.2) displays the spectrum of bilinear capacity with ultimate as well as yield points. The synopsis of the parameters utilized for the states of damage threshold as a function of the displacement of yielding, dy, and the ultimate displacement, du, of the building is shown in Table 4.1.

Table 4.1: Damage state threshold

States of Damage	Median spectral displacement, $\overline{S}_{d,ds}$
Slight state of damage	$\overline{S}_{d,S} = 0.7 S_{d,y}$
Moderate state of damage	$\overline{S}_{d,M} = S_{d,y}$
Extensive state of damage	$\overline{S}_{d,E} = S_{d,y} + 0.25\,(S_{d,u} - S_{d,y})$
Complete state of damage	$\overline{S}_{d,C} = S_{d,u}$

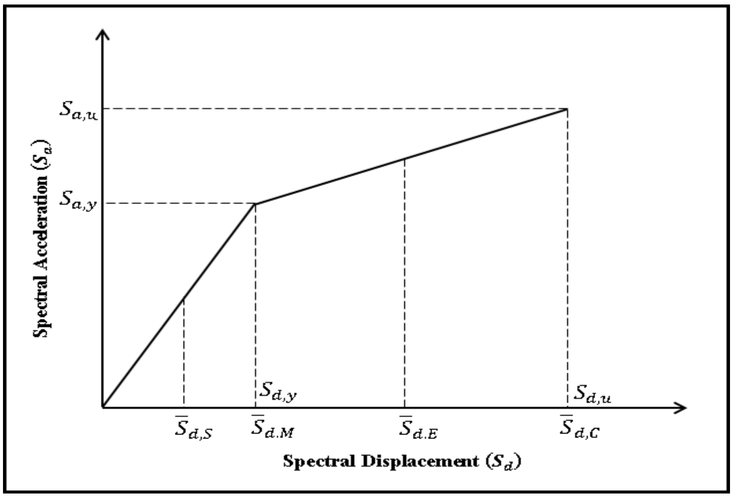


Fig. 4.2: Threshold of Damage state on spectrum of bilinear capacity

4.7 PERFORMANCE BASED PROBABILISTIC ANALYSIS

1) The pushover analysis is conducted using SAP2000 program for a constantly increasing load on the structure of roof top, considering non-linear properties for the materials.
2) In expressions of spectral acceleration and spectral displacement, a capacity curve is obtained. The point of performance is given by the conjunction point of the capacity spectrum and demand spectrum.
3) Taking into account the median spectral displacement shown in Table 4.1, curves of discrete probability are obtained with Eqn.4.1, to conduct the evaluation of risk of the considered frame.
4) To resolve the probabilities for all of state of the damage, the spectral displacement of the point of performance is taken into account for the performance-based design. The probability of damage for each state of damage subsequent to the point of performance is developed.

4.8 UNCERTAINITY

At all times there will be a degree of uncertainty in the result, as there is accurate approach of calculation, for whatever aspects of behavior. While considering the calculation and the results obtained, many sources of uncertainty should be borne in mind.

One is able to recognize the probable uncertainty source in considering the accuracy.

1) Occurring of uncertainty in the loading level and history of loading.
2) Occurrence of uncertainty in motion of the ground motion
3) Occurrence of uncertainty about the concrete properties in the actual building.
4) Occurrence of uncertainty about the modeling performance accuracy.
5) Occurrence of uncertainty about the member geometry.

The uncertainty about the aspects of concrete strength in the building as built is an uncertainty source that is worth bearing in mind in more detail. An amalgamation of strength and ductility that the rebars posses are far in surplus of the minimum limits and are taken as per IS: 1786. Even though the standard mentions tensile strength value as 415 N/mm^2, the typical value is high as 460 N/mm^2, in case of yield strength. Therefore, it can be identified that the tensile strength of the steel as an uncertainty source.

5.1 STRENGTH MATRIX PROBABILITY:

The current paper, a study on the behavior of the structure considering the uncertainty has been conducted. Here the tensile strength of steel (f_y) and concrete compressive strength (f_{ck}) is considered as haphazard variable. As per the guidelines provided by IS 456: 2000, the target strength of M25 grade concrete to be between 30 MPa to 31.6 MPa. Hence, considering the uncertainty of material and the factor of partial safety as 1.5, 25 MPa and 32 MPa was obtained as upper limit and lower limit and a sequence of characteristic strength between these values were selected with a variation of 1.5 MPa and 2 MPa. A value of 1.15 as partial factor and wide range of values were chosen for the tensile strength between 500 MPa to 540 MPa.

As a result, the number models created was 15, considering the combination of characteristic strength of concrete (f_{ck}) and tensile strength of steel (f_y) as mentioned below and analysis was conducted using SAP2000 program.

5.2 STRUCTURAL MODELLING

A model of G+12 storey of 3X3 bay was created so the various components of structure correspond to precisely as far as achievable the characteristics such as mass, strength, stiffness and deformability of both structure as well as non-structural components modeling were not done.

Following are the numerous primary components of structure were being modeled:

5.2.1 BEAMS AND COLUMNS

The structural elements such as beams and column as #D elements were modeled. By assigning of characteristics such as area of cross section, details of reinforcement and the utilized material type, the characteristics of members such as stiffness, strength and deformability were represented. Table 5.1 shows the modeled effectual inertial moment for the columns and beams. The concrete cracking and bar yielding influencing the effects of stiffness reduction are taken into consideration.

According to the code ACI 318M-05(Section 10.10.4.1) modification factor for cracked section is as follows:

Sections	Effective Moment of Inertia
Beams	0.35Ig
Columns	0.7Ig

Table 5.1: The modeled effective moment of inertia

5.2.2 FRAME SCTIONS

Table 5.2: Frame sections

Beam	450x600 mm
Column	500x750 mm

5.2.3 SLAB SECTIONS

A model of 2 way slab slab of dimension 120mm thick was created.

5.2.4 FOUNDATION

On the basis of the fixity degree that is given, the foundation was modeled. Soil structure effects communication was disregarded for the calculation. Assumption was made that the fixed support are at the column end at the terminal point of the footing, in the model.

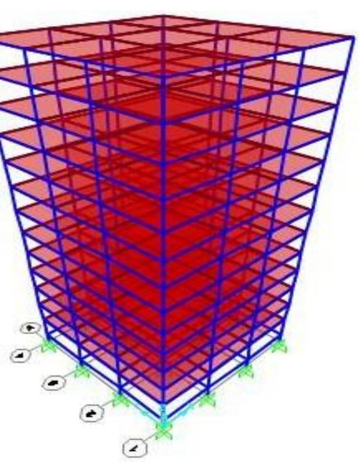


Fig. 5.1: Building model in SAP2000

5.3 LOADING

Loading types:

For computing reasons, the maximum stress in any member of a structure, the load and load effects should be considered as follows in accordance with their application:

1) Deal load according to IS 873 2015 (Part I)
2) Imposed load as per IS 873 2015 (Part II)
3) Wind load as per IS 873 2015 (Part III)
4) Earthquake load as per IS 1893 (Part I) -2002

5.3.1 DEAD LOAD

Self-weight of members and super dead loads form as dead loads. Floor finish of 1.5 kN/m^2 is applied on slabs and wall load of 732 kN/m beams and are to be identified as super dead loads.

5.3.2 LIVE LOADS

3kN/m^2 of live load was applied on all slabs.

5.3.3 WIND LOAD

According to the guidelines of IS: 875 (PART 3) calculation of wind-load was done.

Vb = 33 m/s was taken as basic wind speed of the location (Bengaluru)

5.3.4 SEISMIC LOAD

As per IS: 1893:2002, seismic design was performed. Seismic Zone II was set as location for the G+12 storey reinforced concrete building. Following are the parameters used for the analysis and design. (in accordance with Indian Standards : 1893 (Part I):2002)

5.4 DISCREATE PROBABILITY OF DAMAGE STATE:

As per HAZUS,

1) $B_c = 0.25$
2) $\beta_D = 0.45$
3) $\beta_{M(Sds)} = 0.4$

Post convolution suing MATLAB

$\beta_{M(Sds)}$

Table 5.3.1: $_{(Sds)}$ values for each Damage States.

0.009	Slight
0.018	Moderate
0.054	Extensive
0.144	Collapse

$\overline{S}_{d,ds}$

0.401985	Slight
0.407863	Moderate
0.466065	Extensive
0.752939	collapse

Table 5.3.2: $\bar{s}_d$, values for each Damage States.

6.1 PUSHOVER ANALYSIS RESULTS

The result and analysis of RCC frame is explained in his chapter. Using SAP2000, analysis of RCC under static load is conducted. Consequently, the discreet probability of damage states is found using the obtained results.

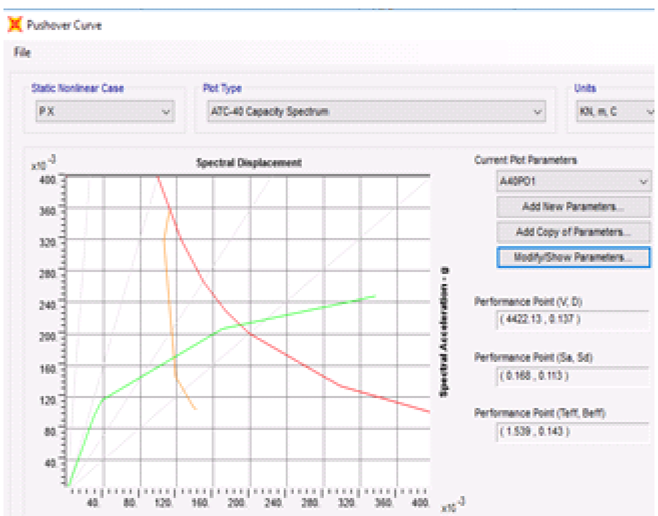


Fig.6.1.1 ADRS Plot -X direction for f_{ck} =25MPa, f_y =500MPa.

From the above ADRS plot, the point of performance happen within the section of the range of damage control performance, demonstrating that from this earthquake, this building would have a smaller amount of damage than permissible for the immediate occupancy level and more than would be permitted for the yielding of the element.

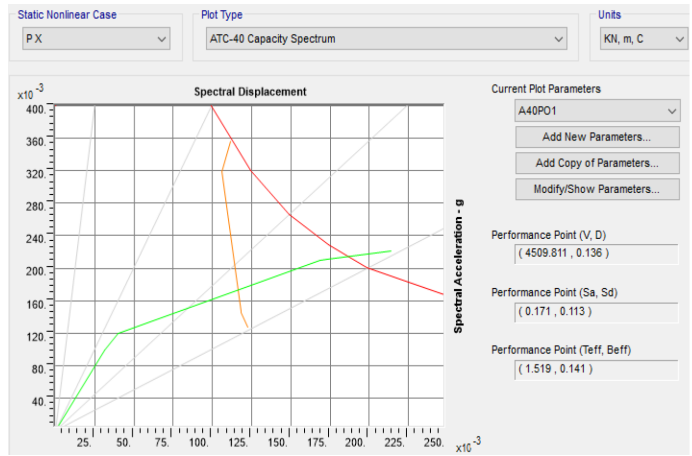


Fig.6.1.2 ADRS Plot -X direction for f_{ck} =25MPa, f_y =520MPa.

It is represented in the above ADRS plot that earthquake such as this, the building would undergo little damage than allowed for the level of immediate occupancy and the allowed would be more for the yielding of the element.

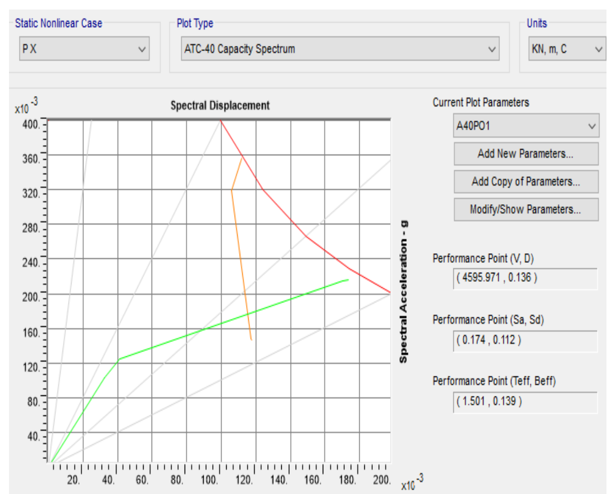


Fig.6.1.3 ADRS Plot -X direction for f_{ck} =25MPa, f_y =540MPa.

It is represented in the above ADRS plot that this building would have undergo little damage than allowed for the level of immediate occupancy and the allowed would be more for the yielding of the element.

6.2 PERFORMANCE LEVELS OF BUILDING

Table 6.1 Performance levels of building

	fy = 500	fy =520	fy = 540
M25	**B- IO**	**B- IO**	**B- IO**
M26.5	**B- IO**	**B- IO**	**B- IO**
M28	**B- IO**	**B- IO**	**B- IO**
M30	**B- IO**	**B- IO**	**B- IO**
M32	**B- IO**	**B- IO**	**B- IO**

All show performance level B-Immediate Occupancy.

6.3 DAMAGE STATES OF BUILDING

Following are the damage states of the building

Table 6.2 Damage states of the building

	DAMAGE STATES	fy = 500	fy =520	fy = 540
M25	Slight	0.023832	0.024757	0.025723
	Moderate	0.034045	0.035367	0.036748
	Extensive	0.109539	0.080028	0.070941
	Collapse	0.336022	0.214011	0.173522
M26.5	Slight	0.023205	0.024106	0.025053
	Moderate	0.033151	0.034438	0.03579
	Extensive	0.122593	0.118856	0.082835
	Collapse	0.390922	0.37211	0.223973
M28	Slight	0.022625	0.023506	0.024431
	Moderate	0.032322	0.033581	0.034902
	Extensive	0.071103	0.126411	0.114076
	Collapse	0.187447	0.404904	0.351598
M30	Slight	0.021916	0.022772	0.023669
	Moderate	0.031308	0.032531	0.033813
	Extensive	0.076314	0.130605	0.131072
	Collapse	0.211332	0.424826	0.422849
M32	Slight	0.021268	0.022101	0.022974
	Moderate	0.030384	0.031573	0.03282
	Extensive	0.133011	0.072553	0.135123
	Collapse	0.440895	0.195493	0.442033

6.4 DISCREATE PROBABILITY OF BUILDING DAMAGE STATES

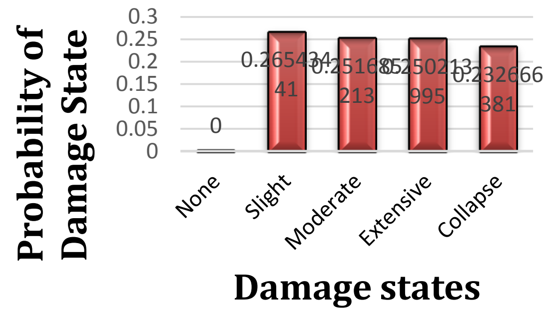


Fig 6.4.1 f_{ck} =25MPa, f_y =500MPa

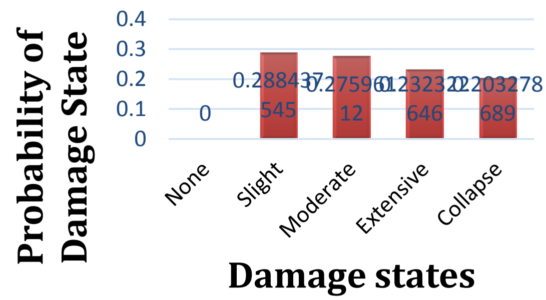


Fig 6.4.2 f_{ck} =25MPa, f_y =520MPa

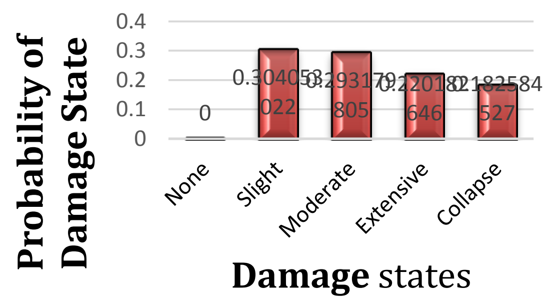


Fig 6.4.3 f_{ck} =25MPa, f_y =540MPa

From the above charts we can see that the probability of collapse damage state decreases by 21.55% for fy=540MPa in comparison with fy= 500MPa for fck=25MPa

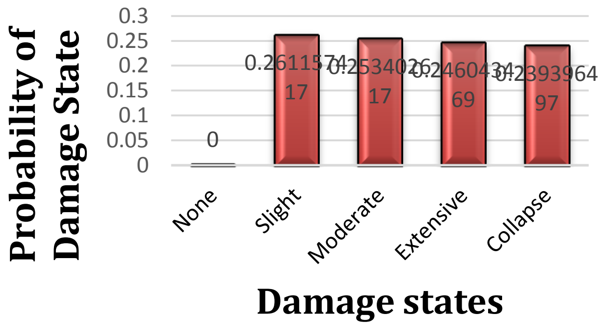


Fig 6.4.4 f_{ck} =26.5MPa, f_y =500MPa

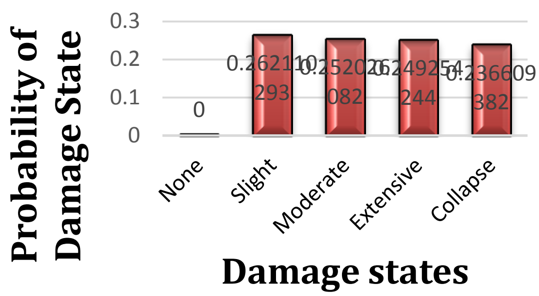


Fig 6.4.5 f_{ck} =26.5MPa, f_y =520MPa

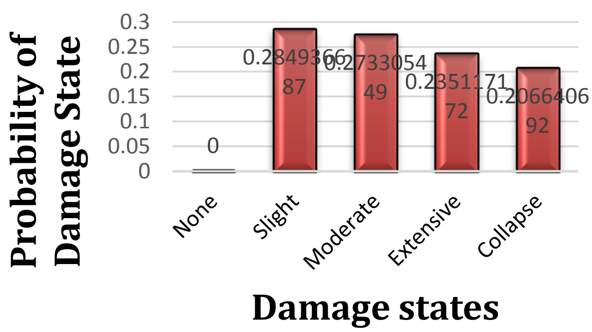


Fig 6.4.6 f_{ck} =26.5MPa, f_y =540MPa

From the above charts we can see that the probability of collapse damage state decreases by 14.16% for fy=540MPa in comparison with fy= 500MPa for fck=26.5MPa

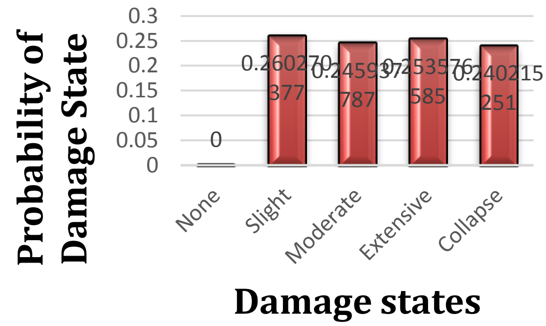


Fig 6.4.7 f_{ck} =28MPa, f_y =500MPa

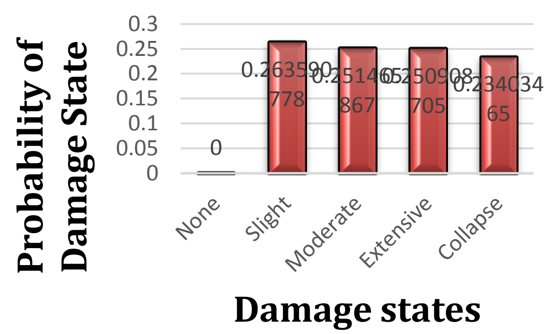


Fig 6.4.8 f_{ck} =28MPa, f_y =520MPa

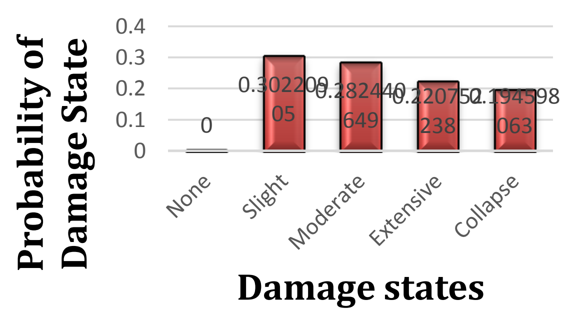


Fig 6.4.9 f_{ck} =28MPa, f_y =540MPa

From the above charts we can see that the probability of collapse damage state decreases by 20.833% for fy=540MPa in comparison with fy= 500MPa for fck=28MPa

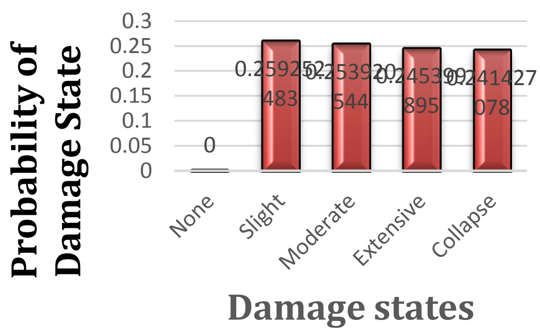


Fig 6.4.10 f_{ck} =30MPa, f_y =500MPa

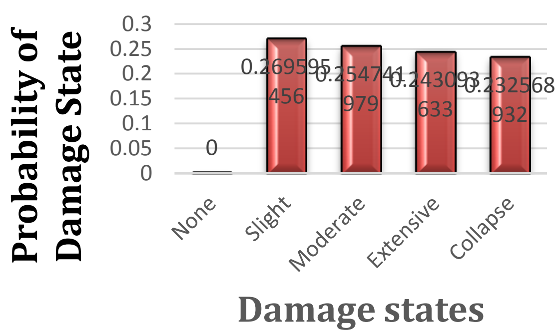


Fig 6.4.11 f_{ck} =30MPa, f_y =520MPa

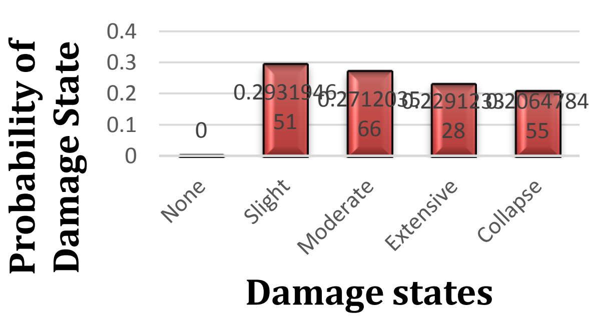


Fig 6.4.12 f_{ck} =30MPa, f_y =540MPa

From the above charts we can see that the probability of collapse damage state decreases by 14.52% for fy=540MPa in comparison with fy= 500MPa for fck=30MPa

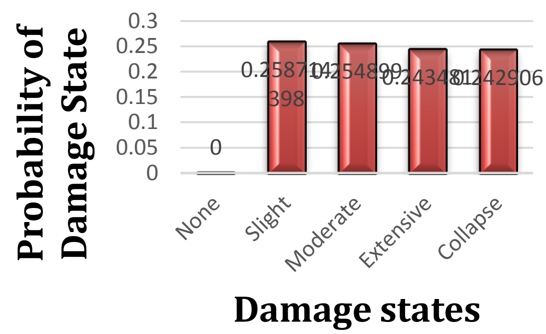


Fig 6.4.13 f_{ck} =32MPa, f_y =500MPa

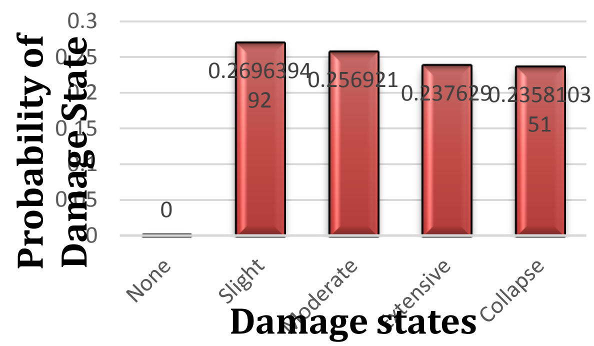


Fig 6.4.14 f_{ck} =32MPa, f_y =520MPa

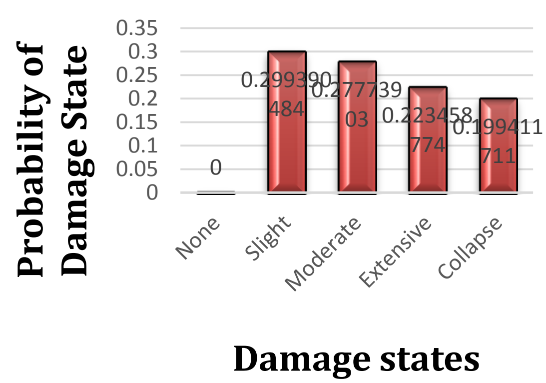


Fig 6.4.15 f_{ck} =32MPa, f_y =540MPa

From the above charts we can see that the probability of collapse damage state decreases by 18.107% for fy=540MPa in comparison with fy= 500MPa for fck=32MPa

7. CONCLUSIONS

1) As per the guidelines provided by the HAZUS method, the probability analysis of seismic risk of reinforced concrete structure was conducted for the seismic assessment of building structure using pushover analysis.
2) The prospect of failure to enumerate seismic susceptibility of reinforced concrete structure may be attained by considering the material uncertainties, if and only if the criteria of failure and limit states of performance are known for diverse types of earthquake.
3) The obtained results from pushover demand spectrum, the real behavior of the building is given by the spectrum of capacity and the formation of hinge as per specification of ATC and FEMA guidelines.
4) We can infer that form the Pushover analysis, better will be the performance of the building, if the level of the spectral acceleration is bigger and the immensity of displacement spectrum is smaller.
5) The method of analysis, idealization of structure, identification of seismic hazard and the models of damage influence the obtained Fragility curves. Therefore, in vulnerability predications, no definite conclusions could be arrived.
6) The result shown by fragility analysis is that there is a high prospect of damage of slight and moderate levels by considering building with varying fck and fy and for high fy, keeping fck constant, low probability of damage of extensive and collapse states is also perceived.

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