PROBABILSTIC SEISMIC RISK EVALUATION OF RC BUILDINGS

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Abstract

As more and more emphasis is being laid on non-linear analysis of RC framed structures subjected to earthquake excitation, the research and development on both non-linear static (pushover) analysis as well as nonlinear dynamic (time history)analysis is in the forefront. Due to prohibitive computational time and efforts required to perform a complete nonlinear dynamic analysis, researchers and designers all over the world are showing keen interest in non-linear static pushover analysis. The paper considers two statistical random variables namely characteristic strength of concrete (f_{ck}) and yield strength of steel (f_y) as uncertainties in strength. Using Monte Carlo simulation 100 samples of each of random variable were generated to quantify effect of uncertainties on prediction of capacity of structure. Based on these generated samples different models were created and static pushover analysis was performed on RC (Reinforced Concrete) Building using SAP2000. Lastly, the main objective of this article is to propose a simplified methodology to assess the expected seismic damage in reinforced concrete buildings from a probabilistic point of view by using Monte Carlo simulation and probability of various damage states were evaluated.

Index Terms: *Seismic Vulnerability, Probabilistic Seismic Risk Evaluation, Fragility Analysis and Pushover Analysis*

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1. INTRODUCTION

Earthquakes are one of the most destructive calamities and cause a lot of casualties, injuries and economic losses leaving behind a trail of panic. It is a known fact that the Globe is facing a threat of natural disasters from time to time. Hence, earthquakes are like a wake-up call to enforce building and seismic codes, making building insurance compulsory along with the use of quality material and skilled workmanship. The occurrence of an earthquake cannot be predicted and prevented but the preparedness of the structures to resist earthquake forces become more important.

India has experienced destructive earthquakes throughout its history. Most notable events of major earthquakes in India since 1819 to 2001, in 1819 the epicenter was Kutch, Gujurat and later in 2001 it was at Bhuj, Gujarat. In many respects, including seismological and geotechnical point of view, the January 26, 2001 earthquake was a case of history repeating itself 182 years later and has made the engineering community in India aware of the need of seismic evaluation and retrofitting of existing structures. Bhuj earthquake of 26 January 2001 and Tsunami of south-east coast of India of 26 December 2004, have given more insights to performance of RC frame constructions.

Based on the technology advancement and knowledge gained after earthquake occurrences, the seismic code is usually revised. Last revision of IS 1893 (Criteria for earthquake resistant design of structures) was done in 2002 after a long gap of about 18 years. Some new clauses were included and some old provisions were updated. A primary goal of seismic provisions in building codes is to protect life safety through prevention of structural collapse. To evaluate the extent to which these specifications meet the collapse prevention objective, assuming that the concerned authorities will take enough steps for code compliance and the structures that are being constructed are earthquake resistant or else intended to conduct detailed assessments of the collapse performance of reinforced concrete structures.
The process of assessing structural seismic performance at the collapse limit state through nonlinear simulation is highly uncertain. Many aspects of the assessment process, including the treatment of uncertainties, can have a significant impact on the evaluated collapse performance. In view of this, an earthquake risk assessment is needed for disaster mitigation, disaster management, and emergency preparedness. In order to do so, vulnerability of building is one of the major factors contributing to earthquake risk.

2. LITERATURE REVIEW

The following review is concerned with studies of the development and application of pushover analysis (POA) and probability risk assessment of RC buildings. It is provided in order to offer an insight into the attempts that have been made to verify the potential, shortcomings and limitations of these methods.

Shinozuka et al., presented a method for the seismic risk analysis of structures using a concept of damage probability matrix in which probability of occurrence of damage stress is defined by combining the seismic risk with the probability of exceedence of certain response level [2]. Bolotin presented a systematic study of random factors involved in risk assessment of structures subjected to strong seismic action using Monte Carlo Simulation procedure [3]. A compressive study of vulnerability of buildings and structures to various earthquake intensities has not been conducted in a systemic way in the country (India) so far [4]. Chowdhary et al. carried out the reliability assessment of reinforced concrete frames under seismic loading using response spectrum method [5].

So far as Probabilistic Risk Analysis (PRA) is concerned, it has not been so widely used for building frames. The reason for this is the large number of failure mechanisms that are to be investigated for performing the non-linear analysis. No attempt has been made to simplify this complexity of the problem and provide a methodology for finding a preliminary estimate of the probability of failure of frame structures.

The review on POA has shown that for structures that vibrate primarily in the fundamental mode the method will provide good information on many of the response characteristics, which includes [6]:

• Identification of critical regions in which the deformation demands are expected to be high and hence which lead to careful detailing.

• Identification of strength discontinuities in plan or elevation that will lead to changes in dynamic characteristics in the inelastic range.

• Estimation of inter-storey drifts accounting for strength or stiffness discontinuities which may be used to control or gauge damage.

Finally, it has been suggested that pushover procedures imply a separation of structural capacity and earthquake demand, whereas in practice these two quantities appear to be interconnected.

Although relatively large work was done by researchers to improve the predictions of demand on the structure [7, 8, 9, 10, 11, 12& 13]; the evaluation of capacity was next to demand and taken a back seat. It is mainly due to the fact that due to the lack of experimental data, the results of analysis are relied up on and considered adequate. It is true that the calculation procedures to predict the capacity curve for the structures are well understood and documented [14, 15, 16 & 17], the evaluation of capacity curve is highly sensitive to the models and procedures followed to evaluate the characteristics of the members and therefore validation with experimental results is the only way to establish the most suitable modeling techniques

3. THE SALIENT OBSERVATIONS OF IS: 1893(PART 1)-2002

Keeping the view of constant revision of the seismic zones in India, lack of proper design and detailing of structures against earthquake. Earthquake performance of RC bare frame has been well documented in the past. Also, damage patterns in reinforced concrete frames during the past earthquakes have been extensively studied. The salient observations of IS 1893(Part 1)-2002 are indicated in Table 1.

Table- 1: The Salient Observations

Risk level	Not specified
Number of seismic zones	Four
Design Spectra	Single normalized response spectra
Soil types	Classification is based on SPT N value and soil description
Fundamental time period	Empirical
Design Basis Earthquake	Half of maximum considered earthquake
Ductility factors	Response reduction factor
Scale factor for lateral forces	Ratio of base shear from equivalent static analysis to base shear from dynamic analysis
Vertical component of earthquake	2/3 of design horizontal earthquake
Design eccentricity (e_d)	e_{di}=1.5 e_{si} +0.05 b_i or e_{di}=1.5 e_{si} – 0.05 b_i, e_{si}-static eccentricity at floor i defined as the distance between the centre of mass and centre of rigidity
P-delta effect	Nothing has been mentioned about for which type of building this effect needs to be considered

4. FORMAT FOR PROBABLISTIC RISK ANALYSIS

4.1 Components of Seismic Vulnerability Simulation

Analytical derivation of a vulnerability relationship includes hazard definition, reference structure, limit state definition, analysis method, uncertainty quantification, and probabilistic simulation method, as shown in Figure 1. In probabilistic performance assessment the relationship between the seismic demand and the seismic intensity has to be determined for different values of the seismic intensity measure. Usually, the top displacement is used as the engineering demand parameter and the spectral acceleration, i.e. the value in the elastic acceleration spectrum at the period of the idealized system, represents the intensity measure. Sometimes, it is convenient to use the peak ground acceleration as the seismic intensity measure.

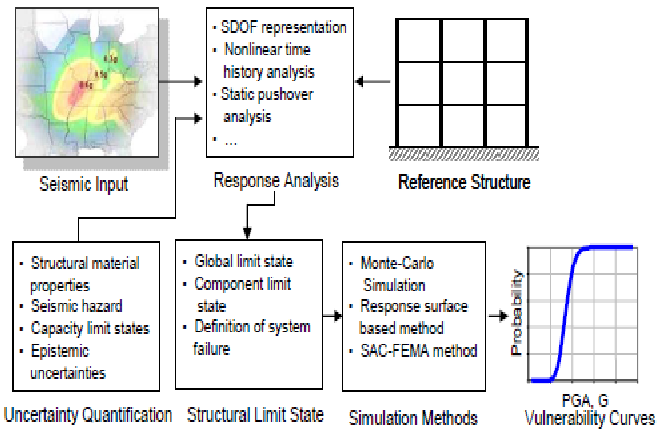

Fig – 1: Components of Seismic Vulnerability Simulation

Risk assessment is the process of obtaining a distribution of probabilities over potential outcomes. This is typically accomplished through some form of systems-level modelling. Fragility curves can also be developed to represent the probability of failure for given multiple failure modes and multiple loads.

4.2 Methodology of Probabilistic Risk Analysis

The paper provide an analytical methodology to quantify hazard through system reliability for the probabilistic risk analysis of reference building as depicted in Figure 2 and Numerical simulation of 4-story reinforced concrete building is summarized as follows,

Step 1: Analytical Building Model

In the model, the nonlinear behavior is represented using the concentrated plasticity concept with rotational springs or distributed plasticity concept where the plastic behavior occurs over a finite length. The rotational behavior of the plastic regions in both cases follows a bilinear hysteretic response based on the Deterioration Model proposed by many researchers. All modes of cyclic deterioration are neglected. A leaning column carrying gravity loads is linked to the frame to simulate P-Delta effects [23].

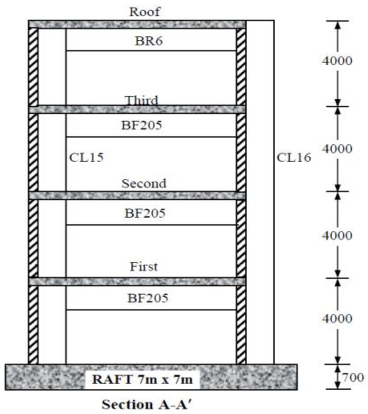

Fig- 2: Overall Geometry of the Structure

Step 2: Pushover Analysis (POA)/Incremental Dynamic Analysis (IDA)

Conventional pushover analysis is carried out to determine the ground motion intensity the building must be subjected to for it to displace to a specified inter-story drift ratio using SAP/E-TABS software's of latest version. The general procedure for the implementation of the probabilistic Capacity Spectrum Method (CSM) [24] is as shown in Figure 3. Methods like POA/IDA are preferred depending on the uncertainty.

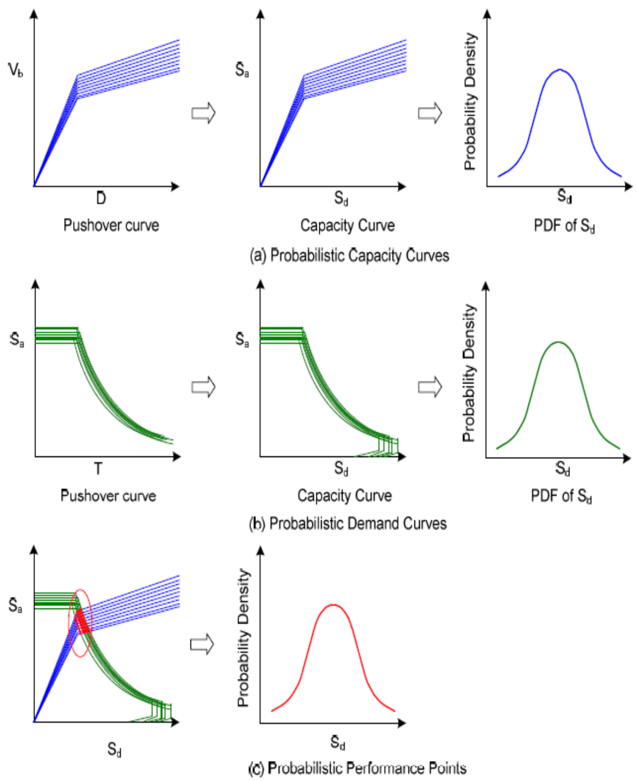

Fig- 3: General procedure of the probabilistic CSM.

Step 3: Define Damage State Indicator Levels (Failure Criteria and Performance Limit States)

The top storey displacement is often used by many researchers as a failure criterion because of the simplicity and convenience associated with its estimation. The limit states (immediate occupancy, life safety, and collapse prevention) associated with various performance levels of reinforced concrete frames as mentioned in FEMA 356[17] and the damage state indicator levels are defined depending on progressive collapse starting from yielding and rotation to instability, which has been tabulated in Table 2[25].
One of the most challenging steps in probabilistic risk analysis is the determination of damage parameters and their corresponding limit states. These parameters are very essential for defining damage state as well as determining the performance of RC building under a seismic event. Therefore, realistic damage limit states are required in the development of reliable fragility curves, which are employed in the seismic risk assessment packages for mitigation purposes.

Table- 2: Damage State Indicator Levels

Slight Damage	Hinge yielding at one floor
Moderate Damage	Yielding of beams or joints at more than one floor
Extensive Damage	Hinge rotation exceeds plastic rotation capacity
Collapse	Structural Instability

Step 4: Incorporate the Uncertainty

Conduct a vulnerability analysis of reference RC building located in Zone-IV/Zone V of IS: 1893-2002 with uncertainty. However, a considerable level of uncertainty (epistemic uncertainty) and randomness (aleatory uncertainty) cannot be avoided in the analysis of structures subjected to seismic action

Step 5: Building Fragility Curves

Develop an analytical fragility estimates to quantify the seismic vulnerability of RC frame building

5. EXPERIMENTAL BUILDING DESCRIPTION

The building is a four storey office building assumed to be in seismic zone IV as depicted in Figure 4 (a) & (b). A brief summary of the building is presented in Table 3.

Table- 3: Summary of Building

Type of structure	Ordinary moment resisting RC frame
Grade of concrete	M20
Grade of reinforcing steel	Fe415
Plan size	5 m X 5 m
Number of stories	G+3 Storey
Building height	12 m above ground storey
Type of foundation	Raft foundation which is supported on rock bed using rock grouting

5.1 Structural System and Members

The building is an RC framed structure. The floor plan is same for all floors. The beam arrangement is different for the roof. It is symmetric in both the direction. The concrete slab is 120 mm thick at each floor level. Overall geometry of the structure including the beam layout of all the floors is as shown in Figure 4(b). Details roof beams, floor beams and columns are as been accomplished in Figure 5, 6 and 7.

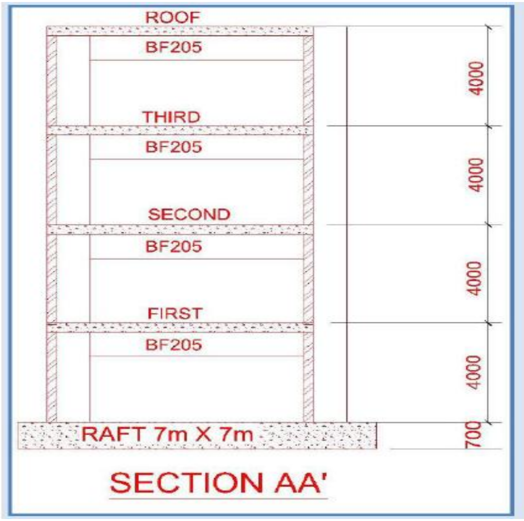

Fig-4 (b): Sectional Elevation

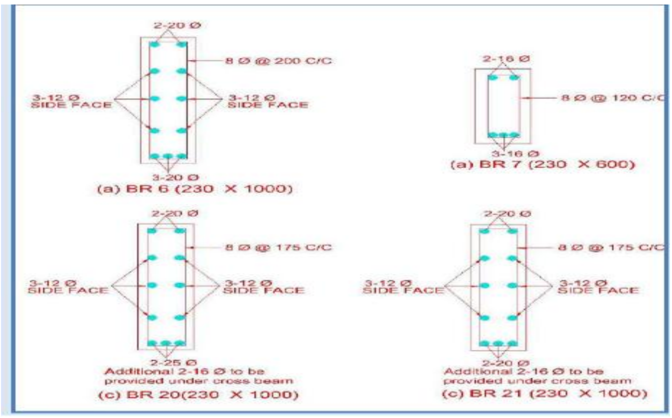

Fig- 5: Details of Roof Beams

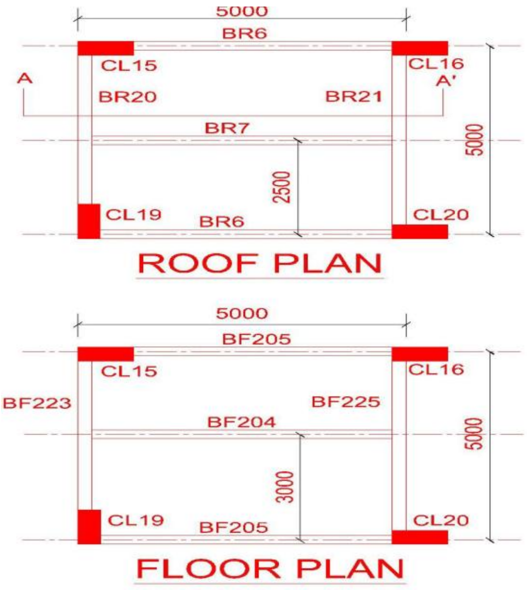

Fig- 4 (a): Overall Geometry of Structure

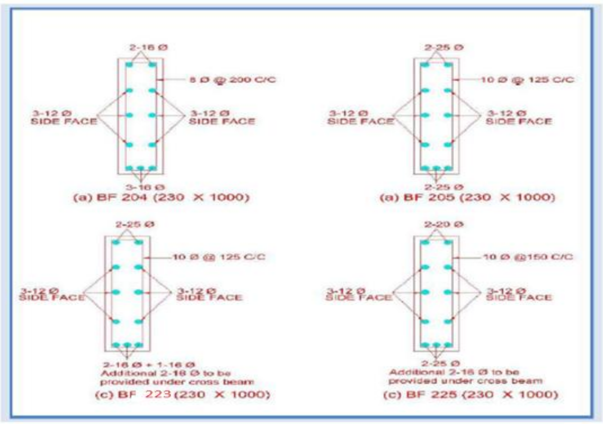

Fig-6: Details of Floor Beams

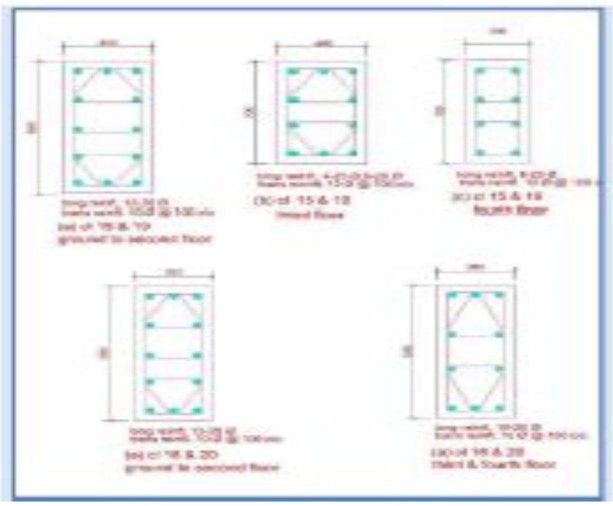

Fig- 7: Detail of Columns

5.2 Characteristics of Reinforced Concrete Sections

Analytical modelling of reinforced concrete members has gained the attention of many researchers in the past and present. Consequently, many models have been proposed to model reinforced concrete structures, considering various effects. However, most of the models are either too simple to predict the response accurately, or accurate but overly complex to incorporate in the analysis. Few models offer a good balance between simplicity and accuracy.

5.3 Material Properties

The material properties considered for the analysis are given in Table 4.

Table 4: Material Properties

Material	Characteristic Strength(MPa)	Modulus of Elasticity (MPa)
Concrete(M20)	$f_{ck} = 20$	$E_c = 22360$
Reinforcing steel (Torsteel)	$f_y = 415$	$E_s = 2\ E + 5$

6. MONTE CARLO SIMULATION

Monte Carlo simulations determine the effect of modelling uncertainties on the structural response predictions. The Monte Carlo procedure generates realizations of each random variable, which are input in a simulation model, and the model is then analysed to determine the collapse capacity. When the process is repeated for thousands of sets of realizations a distribution on collapse capacity results associated with the input random variables is obtained. The simplest sampling technique is based on random sampling using the distributions defined for the input random variables, though other techniques, known as variance reduction, can decrease the number of simulations needed. The Monte Carlo procedures can become computationally very intensive if the time required to evaluate each simulation is non-negligible.

Two random variables (RV's) considered in this study are f_{ck} (x) and f_y (y), which are the mechanical properties of structural elements. Their probability density function is taken as Gaussian normal distribution. For the study 100 samples of two random variables are generated taking codal provisions as coefficient of variation, from these samples we get mean. As we increase number of samples we will get results very near to exact results. Results obtained from this analysis are as indicated in Table 5.

Table 5: Results Obtained From Monte Carlo Simulation

Variables	Mean(MPa)	Distribution	Coefficient of Variance
x = f_{ck}=20	27.75	Normal	0.15
y = f_y=415	502.5	Normal	0.10

7. STRUCTURAL MODELLING

The analytical model was created in such a way that the different structural components represent as accurately as possible the characteristics like mass, strength, stiffness and deformability of the structure. Non-structural components were not modelled. The various primary structural components that were modelled are as follows,

7.1Beams and columns

Beams and columns were modelled as 3D frame elements. The characteristics like strength, stiffness and deformability of the members were represented through the assignment of properties like cross sectional area, reinforcement details and the type of material used. The following values are adopted for effective flexural stiffness of cross-section: *Ief* = 0.5 *Ig* for beams, and *Ief* = 0.70 *Ig* for columns (*Ig* is the moment of inertia of the gross concrete section). In this way, the effects of stiffness reduction due to concrete cracking and bar yielding are taken into consideration. The modelled effective moment of inertia for the beams and columns are as in Table 6.

Table 6- The Modelled Effective Moment of Inertia

Sections	Effective Moment of Inertia (I_{eff})
Rectangular Beam	0.5 I_g
Columns	0.7 I_g

7.2 Beam-column joints

The beam-column joints were assumed to be rigid modelled. A rigid zone factor of 1 was considered to ensure rigid connections of the beams and columns.

7.3 Slab

The slabs were not modelled physically, since modelling as plate elements would have induced complexity in the model. However the structural effects of the slabs i.e., the high in-plane stiffness giving a diaphragm action and the weight due to dead load were modelled separately.

7.4 Foundation Modelling

The foundation was modelled based on the degree of fixity which is provided. The effect of soil structure interaction was ignored in the analysis. In the model, fixed support was assumed at the column ends at the end of the footing. The structure is resting on a 700 mm thick raft resting on rock below, with rock anchors provided.

7.5 Stress-Strain Models for Concrete

The stress-strain model for unconfined concrete under uniaxial stress is as shown in Figure 8. Usually experimental stress-strain curves are obtained from concrete cylinders loaded in uniaxial compression. The ascending part of the curves is almost linear up to about one-half the compressive strength. The peak of the curve for high strength concrete is relatively sharp, but for low strength concrete the curve has flat top. The strain at the maximum stress is approximately 0.002.

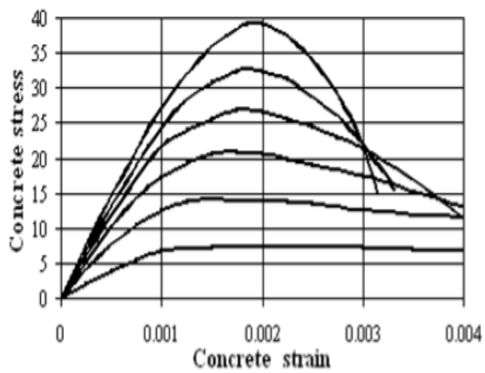

Fig- 8: Typical Stress-Strain Curve for Concrete

Many models for the stress-strain curve of concrete under uniaxial compression have been proposed in past years. Probably the most popular and widely accepted curve is that proposed by Hognestad as shown in Figure 9, which consists of a second order parabola up to the maximum stress f_c" at a strain ε_0 and then a linear falling branch. The extent of falling branch behaviour adopted depends on the limit of useful concrete strain assumed as 0.0038. The corresponding stress was proposed to be 0.85 f_c". Hognestad's curve was obtained from tests on short eccentrically loaded columns and for these specimens, f_c" = 0.85f_c'. Indian Standard (IS) recommends a stress-strain curve very similar to the Hognestad's curve.

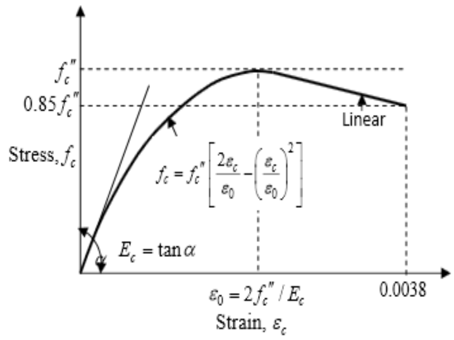

Fig- 9: Hognestad's Curve for Concrete

In IS recommended curve, the maximum stress, f_c" of concrete is assumed as 0.67 times the characteristic cube strength of concrete (fck). Assuming that cylinder strength is 0.8 times the characteristic cube strength, i.e. $f_c' = 0.8f_{ck}$, this becomes same as Hognestad's value of f_c". Since, $f_c'' = 0.85f_c'$, we get $f_c'' = 0.85 \times 0.8f_{ck} = 0.67f_{ck}$. The ascending curve is exactly similar to that of Hognestad's model assuming $\varepsilon_0 = 0.002$. The major difference between the two curves is in the post peak behaviour. IS recommends no degradation and hence no falling branch in the stress after a strain of 0.002. The ultimate strain is also limited to 0.0035 instead of 0.0038 as recommended by Hognestad as shown in Figure10.

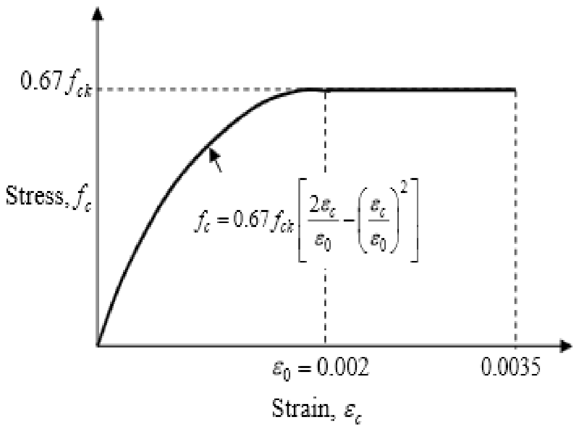

Fig-10: Stress Strain Curve recommended by IS Code

5.6 Stress-Strain Models for Reinforcing Steel

Typical stress-strain curve for steel bars used in reinforced concrete construction is shown in Figure 11. The curves exhibit an initial linear elastic portion, a yield plateau (i.e., a yield point beyond which the strain increases with little or no increase in stress), a strain-hardening range in which stress again increases with strain (with much slower rate as compared to linear elastic region), and finally a range in which the stress drops off until fracture occurs. The modulus of elasticity of the steel is given by the slope of the linear elastic portion of the curve. For steel lacking a well-defined plateau, the yield strength is taken as the stress corresponding to a particular strain, generally corresponding to 0.2% proof strain. Length of the yield plateau depends on the strength of steel.

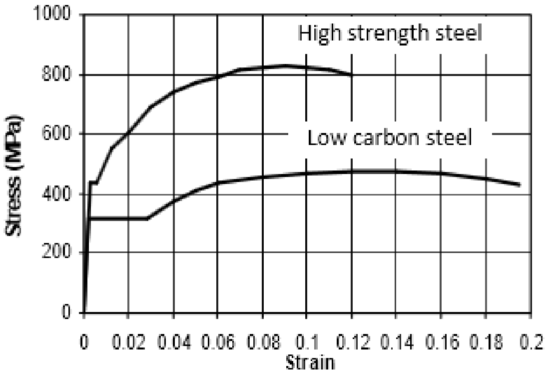

Fig-11: Typical stress-strain curves for steel reinforcement

7.7 High Strength and Low Carbon Steel

High strength high-carbon steels generally have a much shorter yield plateau than low strength low-carbon steels. Similarly, the cold working of steel can cause the shortening of the yield plateau to the extent that strain hardening commences immediately after the onset of yielding. High strength steels also have a smaller elongation before fracture than low strength steels. Generally the stress-strain curve for steel is simplified by idealizing it as elastic-perfectly plastic curve (having a definite yield point) ignoring the increase in stress due to strain hardening as shown in Figure 12 (a). This simplification is particularly accurate for steel having a low strength. The idealization recommended by IS code for HYSD bars is shown in Figure 12 (b). The curve shows no definite yield point and the yield stress are assumed corresponding to a proof strain of 0.2%. If the steel strain hardens soon after the onset of yielding, this assumed curve will underestimate the steel stress at high strains. A more accurate idealization is shown in Figure 12 (c). Values for the stresses and strains at the onset of yield, strain hardening, and tensile strength are necessary for use of such idealizations. These points can be located from stress-strain curves obtained from tests.

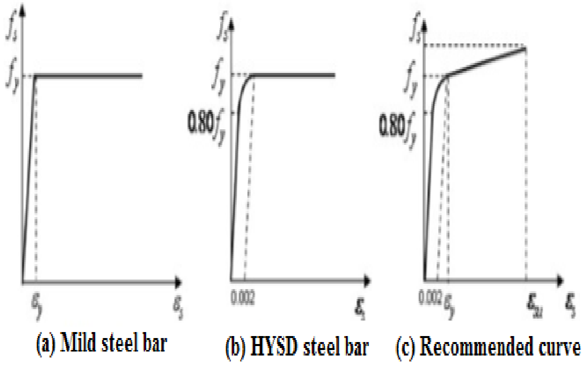

Fig-12: Stress Strain Curve for Steel

7.8 Moment Rotation Relationship

Moment-rotation curve for a member is a plot showing the strength and deformation for the member in terms of moment and corresponding rotation that the member will undergo. Moment curvature characteristics for a typical beam and column as accomplished in Fig. 13 and Fig.14. These are derived from the moment-curvature characteristics of its section, which is a representation of strength and deformation of the section in terms of moment and corresponding curvature of the section.

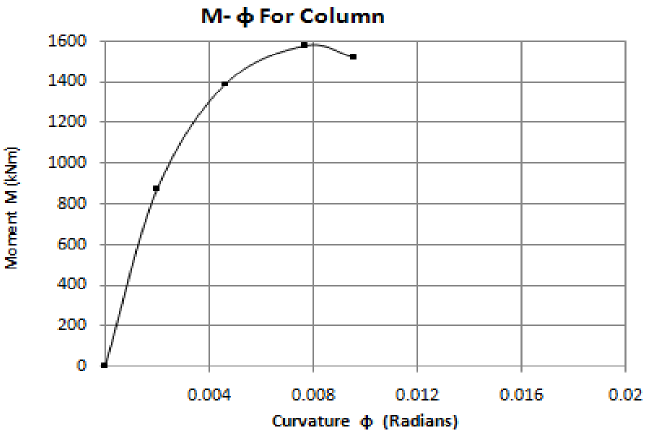

Fig-13: Typical Plot of Moment versus Curvature for Column

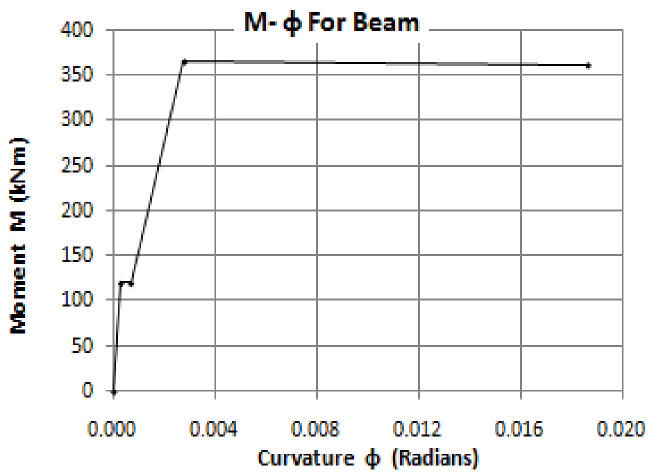

Fig-14: Typical Plot of Moment-Curvature curve for beam

7.9 Dynamic Properties of Building

Structure used for analysis is a four storied RCC structure with single bay 5m x 5m dimension. Height of the storey is 4m. The structure is modeled in SAP2000 and the dynamic properties of the building is calculated and presented in Table 7, based on that the lateral loads are calculated and the structure is then analyzed by applying the lateral loads.

Time period and mode shapes are two of the most important dynamic properties of building. These are the pre-requisite parameters for the analysis and design of buildings for random type load like earthquakes. Response of a building to dynamic loads depends primarily on the characteristics of both the excitation force and the natural dynamic properties of the building. These properties can be computed both analytically and experimentally. Figure15 shows the normalized mode shape of the building.

Table 7: Dynamic Properties of the Building.

Modal Properties	Mode	
	1	2
Period (sec)	0.2971	0.2625
Modal Participation Factor	229.91	150.62
Modal Mass	55.94	24.01

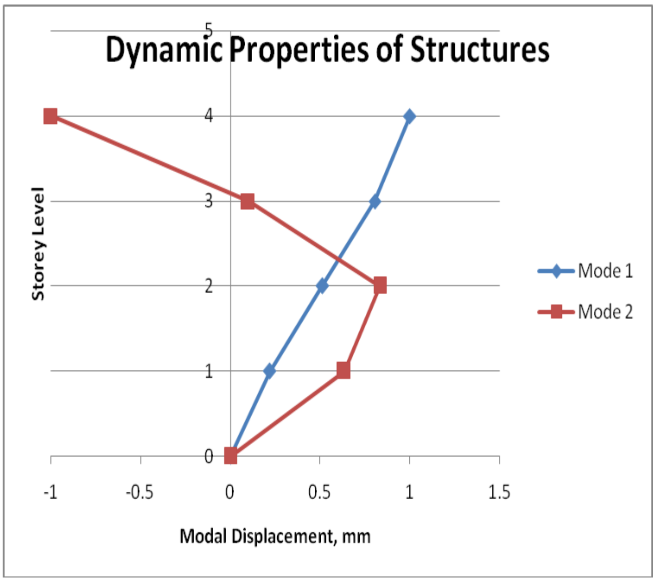

Fig-15: Normalized Mode Shape of the Structure.

7.10 IS-1893(2002) Response Spectrum Load

For the linear static analysis of structures IS- 1893(2002) recommends two methods; the seismic coefficient method and the response spectrum method. Here the response spectrum analysis of the structure is done and the lateral load distribution on the structure is obtained. This load is applied as a lateral load pattern in pushover analysis as tabulated in Table 8.

Table 8: Lateral Load Distribution as per IS-1893(2002)

Storey	Lateral force distribution (kN)
4th floor	16.3
3rd floor	13.1
2nd floor	6.1
1st floor	1.5

7.11 Loading Pattern

Pushover loads can acceptably be applied in an inverse triangular profile, parabolic profile or in the ratio of the first mode shape etc. In view of the existing tower test facility as depicted in figure16, it was found that the best possible control of loading would be through the inverse triangular loading. Therefore, the load on the structure was applied in an inverted triangular profile. The ratio of force at “1st floor: 2nd floor: 3rd floor: 4th floor” was kept as “1: 2: 3: 4” as shown in Figure 17.

Fig- 16: Tower Testing Facility at CPRI, Bangalore

7.12 Loading sequence

Due to the loading pattern, if P is the load on the 1st floor then the base shear would be equal to P+2P+3P+4P = 10P. The load on the structure was gradually increased in the steps of 1t at 1st floor, which resulted in a corresponding load step of 20 t at 2nd floor, 30 t at 3rd floor and 40 t at 4th floor resulting in a load step of 10 t in Base shear. The base shear in the first step was 10 t, in the second step 20t and so on till failure.

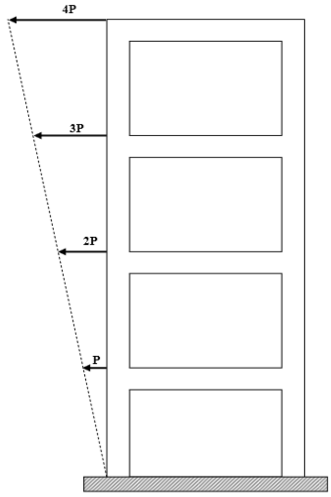

Fig-17 Schematic of Loading Pattern along the Height of Building

8. PUSHOVER ANALYSIS

The compressive strength of concrete and the yield strength of steel are treated as the random variables. A normal probability distribution for concrete strength and a lognormal probability distribution for steel strength might be used. The outcome of the pushover analyses is a family of capacity curves, which can be described as mean or mean plus/minus one/ two/three times standard deviation capacity curves, along with experimental results as shown in Figure 18.

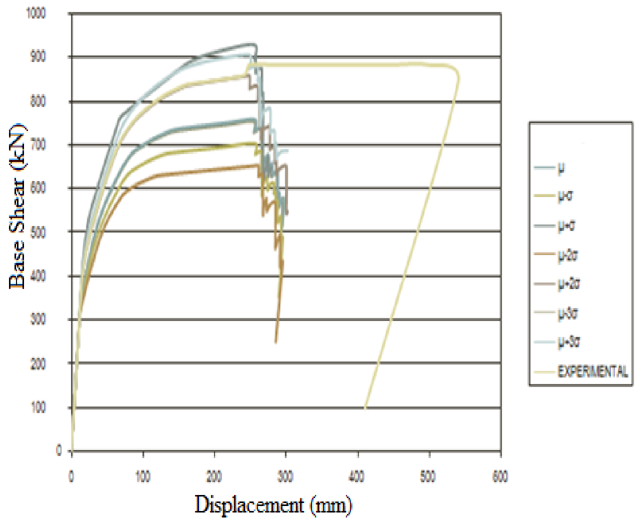

Fig-18: Capacity curve, Monte-Carlo simulation

8.1 Probability of Different Damage States

The discrete damage states are obtained from fragility curve of particular damage sate. The lower damage state is obtained from subtracting higher damage state in fragility curve. The discrete damage state probability for design basis earthquake is evaluated and values are given in Table 9.

Table 9: Calculation of Probability of Various Damage States

Label	Probability				
	Slight	Moderate	Extensive	Complete	No Damage
Analytical	0.01	0.21	0.51	0.26	0.005
Mean	0.02	0.020	0.52	0.25	0.006
Mean+Sigma	0.03	0.22	0.49	0.25	0.008
Mean+2Sigma	0.03	0.22	0.5	0.24	0.008
Mean+3Sigma	0.03	0.22	0.5	0.24	0.008
Mean-Sigma	0.02	0.17	0.53	0.27	0.004
Mean-2Sigma	0.01	0.16	0.54	0.28	0.005
Mean-3Sigma	0.02	0.16	0.53	0.28	0.005

CONCLUSIONS

The methodology proposed/outlined in this paper for probabilistic seismic risk analysis of RC building will be used as a guideline for seismic vulnerability assessment of building structure based on nonlinear static analysis (pushover analysis) using any sophisticated software. Taking uncertainty into consideration, the probability of failure to quantify the seismic vulnerability of RC building may be achieved, provided failure criteria and performance limit states are known for different types of earthquakes. For the risk analysis of building structure, normally either permissible top-storey drift values based on different structural performance levels or different damage states depending on various damage indicator levels are the main failure criteria to obtain the building fragility estimates (probability of failure) in the case of probabilistic risk analysis. The salient features of IS: 1893 (2002) code were also discussed, keeping probabilistic format in view. It is very clear from the study that Monte Carlo Simulation can be effectively used instead of conducting experiments when available data of structure is limited. The value of base shear is well within the limit for the statistics of (μ-3σ) to (μ+3σ).

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