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Vulnerability assessment of a reinforced concrete building

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ABSTRACT

In recent past, severe earthquakes have caused substantial physical losses, economic losses and casualties in the sub-continent. At present, it is in high risk of being attacked by earthquakes. Since a majority of the population is living in earthquake-prone areas, it is probable that such terrible events will take place again in the near future. It is, therefore, vital to quantify the earthquake risk and to develop strategies for disaster mitigation. In order to achieve this goal, an extensive and inter-disciplinary study is required. Such a study is composed of two parts: hazard determination and vulnerability assessment. This study describes the methods by which it is possible to determine the vulnerability of existing engineering structures and building stock. The tool that is employed to assess the seismic performance of reinforced concrete frame structures is the fragility curve. So, this work aims to present a quick and simplistic approach towards vulnerability assessment of Reinforced concrete building with the help of Fragility Curves. The primary focus of this study is to present a proper methodology that can be followed to construct fragility curves for R.C.C frame structures and to generate fragility curves for some specific type of RCC frame structures using this methodology.

Keywords: vulnerability; fragility curves; SAP2000; pushover analysis; time history analysis; normal distribution function; damage states

I.INTRODUCTION

Seismic Vulnerability of a structure is a measure of how vulnerable a structure is with respect to a given state of damage for a given level of ground shaking. How vulnerable a structure is can be quantified on the basis of levels of damage. The seismic vulnerability of a structure can be described as its susceptibility to damage by ground shaking of a given intensity. The aim of a vulnerability assessment is to obtain the probability of a given level of damage to a given building type due to a scenario earthquake. Assessing the seismic hazard (level of ground shaking at a site) and the seismic vulnerability of a structure at the site helps evaluate the seismic risk associated with that structure.

$$\text{Risk} = \text{Hazard} * \text{Vulnerability} * \text{Exposure} \qquad (1)$$

Fragility curves epitomise the conditional probability that a response of a particular structure may exceed the performance limit at a given ground motion intensity. These curves are valuable tools for the valuation of probability of structural damage due to earthquakes as a function of ground motion indices otherwise design parameters. Fragility curves depicts the probability of failure versus peak ground acceleration. A point in the curve represents the probability of exceedence of the damage parameter, which can be lateral drift, storey drift, base shear etc., over the limiting value mentioned, at a given ground motion intensity parameter.

$$\text{Fragility} = P\,[D \geq C|IM] \tag{2}$$

Sabetta et al. (1998) derived empirical fragility curves for three structural classes using six damage levels according to the Medvedev-Sponheuer-Karnik (MSK) macroseismic scale.

Empirical fragility curves for the Japanese building stock were developed by Murao and Yamazaki (2000). The studies involved the estimation of ground motion measures in terms of PGV. Fragility curves were derived by Rota et al. (2008) for several building typologies characterizing the Italian building stock using a dataset of about 150,000 buildings from post-earthquake surveys. Seismic severity was represented as PGA for each municipality evaluated from attenuation relationships.

An analytical procedure was carried out by Rossetto and Elnashai (2005) to derive displacement-based fragility curves for RC structures. Adaptive pushover analysis and capacity spectrum method when used in combination as done here, avoids repetition of analyses for increasing ground motions and reduces computational effort.

Omine et al. (2008) developed fragility curves to relate a single ground motion parameter to structural damage. It used PGA and PGV to develop fragility surfaces and found that PGV is better for expressing fragility for severe damage while both PGA and PGV are useful at lower damage levels.

Singhal (1996) developed fragility curves for low-, mid-, and high-rise RC frames based on non-linear dynamic analysis of structures. Five damage levels were characterized using Park-Ang’s global index.

A method to update fragility curves using damage data from earthquakes through a Bayesian statistical analysis method was presented by Singhal & Kiremidjian (1998). The Park and Ang damage index at specified ground motion intervals followed a lognormal probability distribution. In this study, a methodology for obtaining analytical fragility curves for an RC building is proposed. Such approach is based on nonlinear static and dynamic analyses of the whole building, taking advantage of the capabilities of the software SAP2000, a finite element based software developed by CSI which is able to perform nonlinear time history analyses on the buildings.

II.SEISMIC VULNERABILITY

2.1 Analytical Fragility Function

This probability is usually modelled as a cumulative normal distribution function as follows:

$$P_f = \emptyset\left(\frac{\ln(S_d/S_c)}{\sqrt{\beta_c^2+\beta_d^2}}\right) \tag{3}$$

Where S_d is the mean value of structural demand in terms of ground motion intensity parameter, S_c is the median value of structural capacity, β_c is the lognormal standard deviation of structural capacity, β_d is the lognormal standard deviation of structural demand and $\emptyset(\cdot)$ is the cumulative normal distribution function.

2.2 Intensity Measure

An Intensity Measure (IM) is the reference ground motion parameter against which the probability of exceedence of a given limit state is plotted. Many IMs have been developed till now. The preferred IMs for use in building loss assessment are Spectral acceleration (S_a), Spectral displacement (S_d), Peak ground acceleration (PGA), Peak ground velocity (PGV). In the current study the PGA is selected to be the IM.

2.3 Damage States

In seismic risk assessment, the performance levels of a building can be defined through damage thresholds called limit states. A limit state defines the threshold between different damage conditions, whereas the damage state defines the damage conditions themselves. The methods for deriving fragility curves generally model the damage on a discrete damage scale. In the current study, fragility functions are defined for five damage states as per [Park and Ang, 1985]. These damage states are as follows:

Table 1 Damage State definition as per Park and Ang, 1985

Building Damage State	**Damage Index Range**
No Damage	$DI \leq 0.14$
Slight Damage	$0.14 < DI \leq 0.4$
Moderate Damage	$0.4 < DI \leq 0.6$
Extensive Damage	$0.6 < DI < 1$
Collapse	$DI \geq 1$

2.4 Probabilistic Seismic Demand Model

Probabilistic Seismic Demand Model represents a definite mathematical relationship connecting the Engineering Demand Parameter (EDP), i.e. the parameter used o describe seismic demand on the structure and the Intensity Measure (IM).

$$\ln(EDP) = \ln(a) + b\,ln(IM) \qquad (4)$$

a and b are the coefficients evaluated using Regression analysis.

2.5 Dispersion Components

The dispersion component denoted by β, also called the lognormal standard deviation component, describes the total variability of fragility curve damage states (HAZUS, 1999). The dispersion component is calculated as root of sum of squares of the variabilities for demand and capacity.

$$\beta_{s,ds} = \frac{\sqrt{\beta_c^2 + \beta_{d|IM}^2}}{b} \quad (5)$$

The variability in demand for each damage state is calculated as the standard deviation of the demand quantity assuming a lognormal distribution. This factor accounts for variability in both demand and capacity, assuming a lognormal variation for both.

$$\beta_{d\,|\,IM} = \sqrt{\frac{\sum_{i=1}^{N} (\ln(IM) - \ln(\,IM_m))^2}{N-2}} \quad (6)$$

Where N is sample size of data. The final form of fragility function may be represented as follows:

$$P[ds|IM] = \emptyset\ (\left(\frac{\ln(IM) - \ln(IM_m)}{\beta_{s,ds}}\right) \quad (7)$$

III.BUILDING DESCRIPTION & MODEL

The modelling here is done using SAP2000. In this study, an RC frame having ten stories and four bays are considered. The frame is designed according to IS 456-2000 using M30 concrete and Fe415 steel. The frame is having a storey height of 3 m and bay width of 5 m in X-direction and 4 m in Y-direction. The base of the frame is considered as fixed. In addition to self-weights of beams and columns, the dead load (due to slabs and infill walls) and live loads prescribed for all beams are in accordance with IS 875. The structure is designed for Soil type-II and zone factor of 0.24

Table 2Brief details of the model

Storey Height	3000mm
Bay Width (X-dir)	5000 mm
Bay Width (Y-dir)	4000 mm
Size of Beams	250mm*450mm
Size of Columns	450mm*450mm
Thickness of Slab	150mm
Thickness of Exterior Wall	250mm
Thickness of Interior Wall	150mm
Height of Parapet Wall	1500mm

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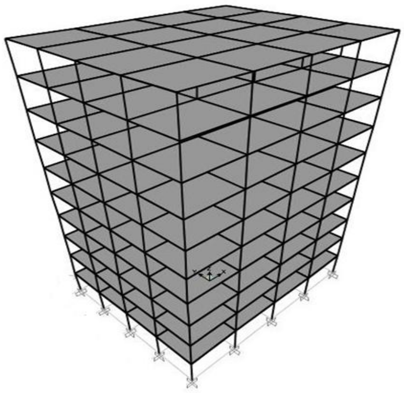
Figure 1 3-D view of the modelled building



IV.ANALYSIS & RESULTS

This model gives damage as a linear function of displacement ductility, energy ductility and cumulative hysteretic energy ductility. For this study, the damage incurred at the building can be reasonably assessed using this damage model.

Consistent with the dynamic behaviour, **Park and Ang** in 1985 expressed seismic structural damage as a linear combination of the damage caused by excessive deformation and that contributed by repeated cyclic loading effect.

$$DI = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \qquad (8)$$

Where δ_m is maximum deformation under earthquake, δ_u is ultimate deformation capacity under monotonic loading, Q_y is calculated yield strength, dE is incremental absorbed hysteretic energy, β is non-negative parameter representing the effect of cyclic loading on structural damage taken as 0.15 for systems with fairly stable hysteretic behaviour and δ_u = μd_y where μ is displacement ductility and d_y as yield displacement.

Or, $$DI = \frac{\mu_d}{\mu_u} + \frac{\beta}{\mu_u} \frac{\int dE}{Q_y \delta_y} \qquad (9)$$

where μ_d is the relative displacement ductility and μ_u is the ultimate ductility.

Now, for a perfectly elasto-plastic system subjected to a single plastic excursion (Teran Gilmore and Jirsa, 2004), the total energy is represented as:

$$\frac{\int dE}{Q_y \delta_y} = \frac{\delta_p Q_y}{Q_y \delta_y} = \frac{\delta_p}{\delta_y} = \mu_p \qquad (10)$$

where μ_p is the plastic ductility reached.

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Thus, the simplified expression for damage index can be written as

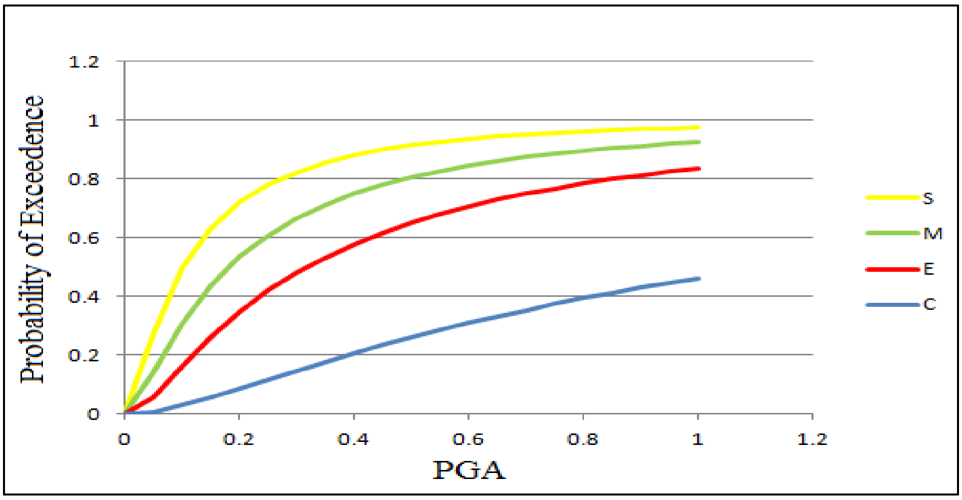
$$DI = \frac{\mu_d + \beta \mu_p}{\mu_u} \quad (11)$$

The computation of fragility curves requires execution of certain processes and analysis which are as follows:

- A linear Response Spectrum Analysis is the foremost analysis performed on the structure which is followed by designing the structure in accordance with IS 456-2000.
- Secondly, a non-linear static analysis, i.e. Pushover analysis is done on the model which gives us the Pushover curve. This curve is used for calculating the Performance point (target displacement) and the energy of the structure.
- Thirdly, select any earthquake ground motion records (El Centro data is taken in this study). Software known as Seismo match is used in order to make El Centro data compatible to response spectrum of zone 4. Now perform a non-linear Time History Analysis on the RC frame with PGA ranging from 0.05g to 1g with an interval of 0.05.
- Moving further, we compute the damage index of the RC frame under study which in turn is helpful in determining the damage states of the structure.
- Perform a regression analysis of simulated response data to establish the probabilistic characteristics of structural demand as a function of a ground shaking parameter, for example, spectral acceleration or peak ground acceleration.
- Then, a Probabilistic Seismic Demand Model as well as the Dispersion components of the model are calculated.
- Plot the fragility curves as a function of the selected ground shaking parameter.

The analysis of the above modelled building is carried out in two directions, i.e. along X axis and Y axis, addressed as Case-I and Case-II respectively.

Figure 2 Fragility curve for Case-I



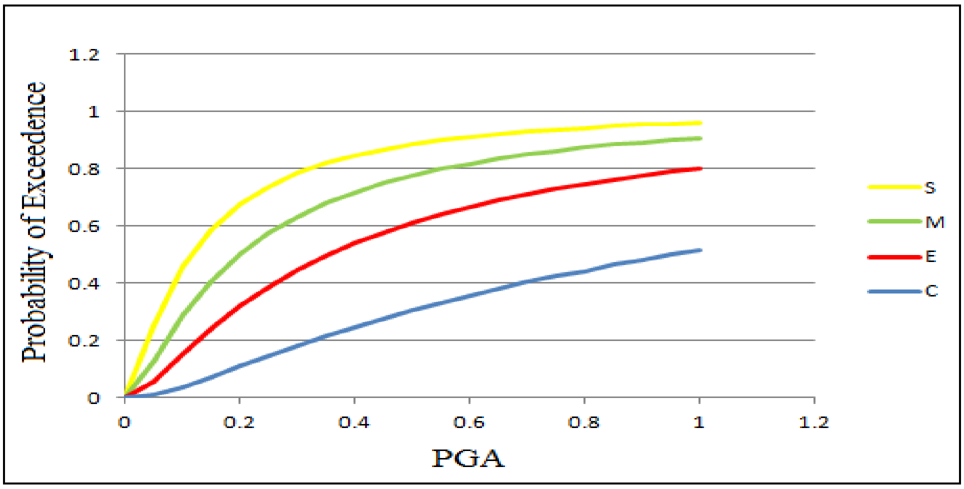
For a PGA value of .24g, the probabilities of exceeding the various damage states are listed as per the fragility curve shown in Figure 2 as follows:

Probability of exceeding Slight damage state	:	73.14%
Probability of exceeding Moderate damage state	:	54.88%
Probability of exceeding Extensive damage state	:	36.04%
Probability of exceeding Collapse damage state	:	9.24%

From this data, the discreet probabilities of achieving each of these damage states may be calculated as:

Discrete Probability of Collapse	:	9.24%
Discrete Probability of Extensive damage	:	26.80%
Discrete Probability of Moderate damage	:	18.84%
Discrete Probability of Slight damage	:	18.26%
Discrete Probability of No damage	:	26.86%

Figure 3 Fragility curve for Case-II



For a PGA value of .24g, the probabilities of exceeding the various damage states are listed as per the fragility curve shown in Figure 3 as follows:

Probability of exceeding Slight damage state	:	68.75%
Probability of exceeding Moderate damage state	:	51.58%
Probability of exceeding Extensive damage state	:	33.31%
Probability of exceeding Collapse damage state	:	11.60%

From this data, the discreet probabilities of achieving each of these damage states may be calculated as:

Discrete Probability of Collapse	:	11.60%
Discrete Probability of Extensive damage	:	21.71%
Discrete Probability of Moderate damage	:	18.27%
Discrete Probability of Slight damage	:	17.17%
Discrete Probability of No damage	:	31.25% mu.env1@gmail.com

V.CONCLUSIONS

This study presents an analytical methodology for developing seismic fragility curves for highway bridges. The choice of a parameter to represent the intensity of ground shaking is another consideration. Traditionally, peak ground acceleration (PGA) is used as the ground shaking parameter for the generation of fragility curves.

Based on the fragility curves, developed for RC buildings with various numbers of stories by using nonlinear time analyses, the following conclusions can be stated:

- Talking about Pushover analysis, the ultimate displacement comes out to be more in x direction than in y direction. Similarly, the energy value also follows the same trend.
- The maximum displacements obtained from Time History analysis is more in x direction as compared to y direction. Moreover, displacement follows an almost linear variation with the increasing values of PGA.

Table 3 Comparison of maximum displacements for both the cases using two different approaches

CASE-I		**CASE-II**	
Analysis	**Displacement (m)**	**Analysis**	**Displacement (m)**
Pushover	0.604	Pushover	0.531
Time History	0.116	Time History	0.101

- The discrete probabilities of No Damage and Collapse increases with the direction changing from x to y. This shows that the building is more vulnerable in y direction.
- In general for PGA values of 0.5g or more, the variation of fragility values is less than the variation corresponding to PGA values below 0.5g.

Table 4 Comparison of discreet probabilities of reaching various damage states

Damage State	**Case-I**	**Case-II**	**Remarks**
No Damage	26.86%	31.25%	Increase
Slight Damage	18.26%	17.17%	Decrease
Moderate Damage	18.84%	18.27%	Decrease
Extensive Damage	26.80%	21.71%	Decrease
Collapse	9.24%	11.60%	Increase

Finally, it should be noted that this study was on regular buildings. To get more general conclusions the irregular buildings should be studied as well.

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