Comparison of Damage Index and Fragility Curve of RC Structure Using Different Indian Standard Codes

Tathagata Roy and Pankaj Agarwal

Abstract In today's world, there is much talk about seismic vulnerability, its features and the necessity of study. The main problem that is faced in today's world is knowledge about proper quantification of damage. The response is obtained after the ground motion has been imparted on the structure. Despite of making significant progress in the field of seismic design codes for dynamic analysis, there is a lack of progress in the quantification of damage. The damage criterion should include large displacements as well as the effect of repeated cyclic loading. An energy-based damage model is used to assess the damage index for the structure taking into account the effect of repeated cycles. The present study has been done to carry out the vulnerability analysis of a four-storey reinforced concrete (RC) moment resisting frame by modified Park and Ang damage model which is then compared by different Indian standard code for earthquake resistant design. A new damage scale has been proposed for each of the analysis against an individual code. Each of the fragility parameter has been individually calculated in carrying out this analysis. Fragility curve is plotted by calculating the uncertainty and the median values.

Keywords Damage index · Fragility · Median · Pushover · Vulnerability

1 Introduction and Background

Seismic vulnerability is practically defined as "the proneness of some category of elements at risk to undergo adverse effects inflicted by potential earthquakes", i.e. the degree of damage to the structure or of a region when the ground motion has been imparted on the structure. Vulnerability is always directly related to risk definition. Every structure is vulnerable to some extent on application of external loading. The main problem that is faced in India is that there has been a significant contribution in the development of various design codes for dynamic analysis, but

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no proper method of quantification of damage. For the developing countries like India, the present code does not meet any requirements against the structural safety for an existing building that is very well exposed to the damage. There has been an urgent concern regarding the structural restoration of the existing building class, which comprises of important structures like historical buildings, lifeline structures, etc. Thus, the earthquake engineers are constantly focusing on the aspects of developing probabilistic risk models or the vulnerability models. Major earthquakes over the last couple of decades show that seismic risk is the only natural risk that affects the socio-economic prospect of the society. So, only rough estimates can be made regarding the damage. Thus, the effect on any existing building must be carried out to check the sustainability of the structure. In urban areas of high seismicity, the probability of collapse is very high, needing the concept of retrofit to a large extent. One of the major parameter by which the seismic risk can be described is on the fragility of the structural component. When the process of information is processed about fragility, the main component that needs to be considered is the uncertainty due to calculation method. Fragility curve is very essential to structural engineers, reliable specialists, and hospital and highway network. It is also used for post-earthquake damage assessment, evaluation and improvement of seismic performance of structural and non-structural systems.

Extensive studies were carried by Park and Ang [5], Powell and Allahabadi [4], DiPasquale et al. [1], and Ghobarah et al. [2] for evaluating the damage index of the reinforced concrete (RC) structure. They laid the contribution for the development of damage index in today's world for assessing the vulnerability of the structure.

The damage model that is used to quantify the damage is the Park and Ang [5] damage model. The damage model is used for expressing the potential damage of RC structure as a function of maximum deformation and dissipated energy that is given in Eq. (1). The study in this paper consists of development of vulnerability curve of a 4-storey RC moment resisting frame model without infill. The damage index is computed by using modified Park and Ang damage model (1992) by performing the non-linear response history analysis. Due to the impetus laid on the vulnerability assessment, the energy-based model is of best use to find out the damage index of the structure. The modified Park and Ang damage model is given as below,

$$I_D = \frac{d_m - d_y}{d_u - d_y} + \beta_e \frac{\int dE}{F_y d_u} \tag{1}$$

where d_m = maximum displacement obtained from non-linear response history analysis, d_u = ultimate displacement obtained from the pushover analysis, d_y = yield displacement, dE = incremental energy obtained from the time history analysis, β_e = parameter representing the cyclic loading-strength degradation factor, taken as 0.1.

2 Structure Model and Dynamic Properties

The analytical model that is used is 4-storey RC moment resisting frame. The building has a rectangular plan of 12 m × 16 m; 3 bays of 4 m each are present in the X-direction while in Y-direction it has 4 bays of 4 m each. The height of each floor is 3 m as shown in Fig. 1. The damage index is used to quantify the damage caused in beams and columns or due to the local collapses that occur in the structure. Space frame model is used. The characteristic compressive strength that is used in design is M25 and that for steel it is Fe415. Dynamic analysis of the structure is determined by free vibration. Seismic zone V and type II soil is used for calculating the demand.

Dynamic analysis of the structure is determined by free vibration. The first three frequencies obtained are 1.412, 4.831, and 9.556 Hz that is obtained in X-direction. The response is mainly governed by first mode that contributes about 82 %. Table 1 corresponds to the base shear obtained from the analysis by different Indian standard codes.

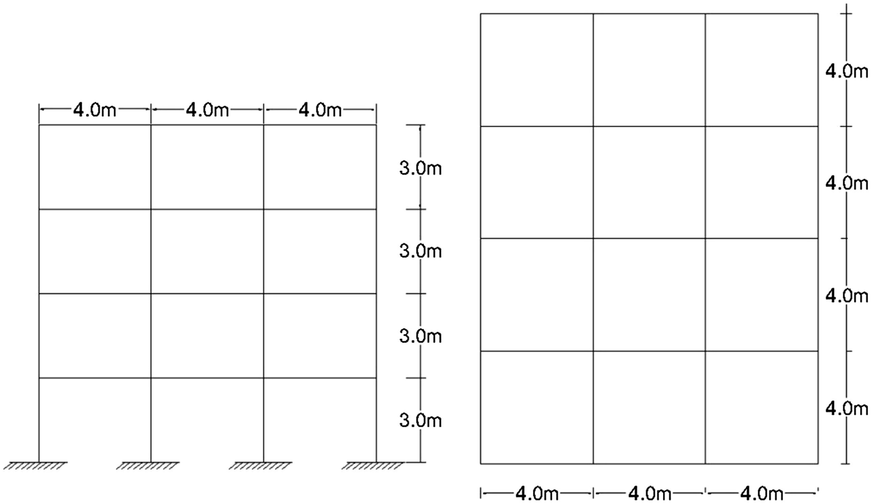

Fig. 1 Building plan and elevation

Table 1 Structural damage state definition

IS code	Base shear (kN)
IS-1893 (Part I):2002	392
IS-1893:1984	359
IS-1893:1970	300

Table 2 Characteristics of the ground motion

Event	Station	PGA (g)	Mechanism	Magnitude	V_s30 (m/s)
Bigbear 1992	North Shore	0.043	Strike-slip	6.46	338.5
Imperial Valley 1979	El Centro E10	0.176	Strike-slip	6.53	202.8
Kobe 1995	Kakogawa	0.058	Strike-slip	6.9	312.0
Landers 1992	Mission Creek Fault	0.122	Strike-slip	7.28	345.4
Northridge 1994	Arleta	0.237	Reverse	6.69	297.7

3 Ground Motion Selection

In the inelastic analysis of the structure, there is great variation of response with a slight change in ground motion. Ground motion plays a great role in assessing the response due to the non-linear analysis of the structure. Large set of ground motion can be used for the analysis. For realistic results, the ground motions are scaled with respect to the design spectrum for which the structure is modeled. In this paper, Indian standard code for earthquake resistant design is used. Zone V type II soil is taken. For comparison and to assess the ductility and damage index, IS-1893 (Part I):2002, IS-1893:1984 and IS-1893:1970 has been used. Five earthquakes are considered in this example model to perform the time history analysis. The dynamic characteristics are mentioned in Table 2. The analyses are carried out for the example building using accelerograms scaled to 0.108g, 0.216g, 0.324g, 0.432g, 0.54g (g denotes gravitational acceleration).

4 Assessment from the Pushover Analysis

A building capacity curve is known as pushover curve, which is a non-linear static approach. It is plotted between building's lateral load resistance component and the roof displacement. It is derived from a plot of static-equivalent base shear versus building displacement. Non-linear hinges are assigned to each beam and column for the observation of failure criteria and to assess the damage states observed after damage. It is basically a displacement-based approach. Thus, the capacity curves obtained for the building under 3 different seismic conditions and codes analyzed in SAP2000 v14.2 are plotted. From these plots, the yield and ultimate values of the response can be found out. Table 3 shows the result of the bilinear plot for different Indian standard codes.

Table 3 Bilinear result of pushover analysis

IS code	Base shear (kN)		Roof displacement (m)	
	Yield	Ultimate	Yield	Ultimate
IS-1893 (Part I):2002	855.875	855.875	0.03649	0.34298
IS-1893:1984	749.945	778.866	0.03196	0.28720
IS-1893:1970	690.4733	702.6597	0.02888	0.22130

5 Assessment from Time-History Method

Time history analysis or the modal analysis is used to determine the response of the structure due to the induced earthquake motion. The non-linear response history analysis is performed for the five earthquakes that are scaled up to visualize the structural response due to these scaled earthquake motions. The non-linear response, i.e. the hysteresis curve is of utmost importance, which is the plot between force and roof displacement. As the maximum displacement occurs in the top storey, so the response of the top storey is plotted. The parameters that are taken into account are: maximum displacement (m), roof drift (%) and inter-storey drift (%). The building is subjected to spectrum scaled ground motion of different peak ground acceleration (PGA). The response of the building is shown in Table 4 that is computed by SAP2000 v14.2. The mean results are tabulated as below. The plots among different responses are given in Fig. 2.

6 Damage States

In this paper, the main focus is on development of fragility curve that would consider the damage states obtained from quantification of the damage of the structure. Thus, the indices or basically the limits of the damage state, i.e. slight, moderate, extensive and collapse are required to set. There is no damage below

Table 4 Plot of responses obtained from time-history analysis

PGA (g)		0.108	0.216	0.324	0.432	0.54
Max. roof displacement (m)	IS-1893:2002	0.02952	0.05280	0.09672	0.11796	0.20976
	IS-1893:1984	0.02712	0.05208	0.08616	0.13284	0.22308
	IS-1893:1970	0.02628	0.04836	0.09572	0.13776	0.20064
Inter-storey drift (%)	IS-1893:2002	0.29	0.52	0.77	1.36	2.96
	IS-1893:1984	0.30	0.56	1.33	1.87	2.86
	IS-1893:1970	0.33	0.66	1.74	2.24	3.27
Damage index	IS-1893:2002	0	0.070	0.201	0.400	0.862
	IS-1893:1984	0.009	0.116	0.280	0.459	0.996
	IS-1893:1970	0.036	0.188	0.466	0.694	1.008

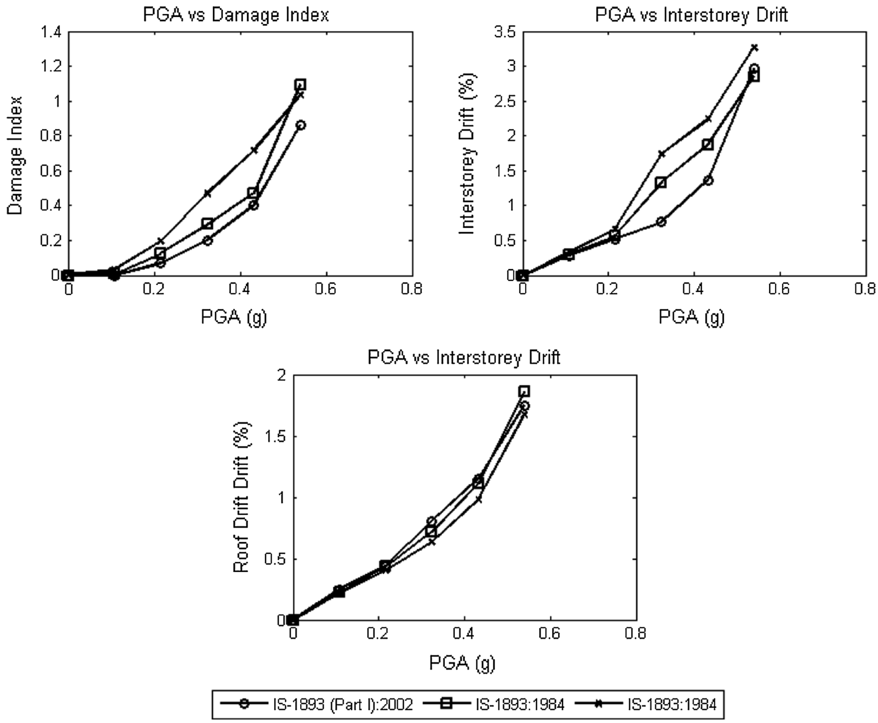

Fig. 2 Building plan and elevation

Table 5 Damage states showing limits

Damage states		Slight	Moderate	Extensive	Collapse
Peak ground acceleration (g)	IS-1893:2002	0.17	0.3	0.44	0.5
	IS-1893:1984	0.10	0.19	0.32	0.46
	IS-1893:1970	0.10	0.16	0.28	0.38
Damage index	IS-1893:2002	0.06	0.18	0.44	0.7
	IS-1893:1984	0.03	0.1	0.3	0.7
	IS-1893:1970	0.04	0.16	0.38	0.6
Inter-storey drift (%)	IS-1893:2002	0.4	0.7	1.5	2.5
	IS-1893:1984	0.3	0.5	1.4	2.2
	IS-1893:1970	0.3	0.5	1.3	2

slight damage. Based on the analytical approach and the damage observed in the structure due to the formation of plastic hinges, the Damage states are set in accordance to the response obtained for the structure. Table 5 plots the damage states, i.e. slight, moderate, extensive and collapse are set for the different seismic conditions provided by Indian standard codes shown in the table.

7 Assessment of Fragility Parameters

Fragility curve is defined as the probability of reaching or exceeding different damage states of a given peak building response. The conditional probability of being in, or exceeding, a particular damage state, ds, given the spectral displacement, S_d, is defined by the function:

$$P[ds|S_d] = \phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_d}{\overline{S_{d,ds}}}\right)\right] \tag{2}$$

where $\overline{S_{d,ds}}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds; β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state, ds; ϕ is the standard normal cumulative distribution function. So, the main two parameters involved in the fragility curve are $\overline{S_{d,ds}}$ which is the median value of spectral displacement at which the building reaches the threshold of damage state, ds and β_{ds} which is the standard deviation of the natural logarithm of spectral displacement for damage state, ds.

The total variability of each structural damage state, $\beta_{s,ds}$ is modeled by the combination of three contributors to structural damage variability, β_c, β_d and β_m.

$$\beta_{s,ds} = \sqrt{\beta_c^2 + \beta_d^2 + \beta_m^2} \tag{3}$$

where $\beta_{s,ds}$ is the lognormal standard deviation that describes the total variability for structural damage state, ds; β_c is the lognormal standard deviation parameter that describes the variability of the capacity curve; β_d is the lognormal standard deviation parameter that describes the variability of the demand spectrum; and β_m is the lognormal standard deviation parameter that describes the uncertainty in the estimate of the median value of the threshold of structural damage state, ds.

1. **Calculation of variability in capacity curve**

From the non-linear pushover analysis, the main part of concern is the plastic state that after the yield has significant contribution in the energy. The plastic state up to the ultimate position is divided into a number of regions depending upon the damage occurred in the structure. The different occupancy levels denoted are the immediate occupancy, life safety and the collapse prevention. Therefore, we have considered the different damage states as stated earlier to be divided as per the occupancy levels.

Yield to immediate occupancy (IO) = slight damage state, immediate occupancy to life safety = moderate damage state, life safety to collapse prevention = extensive damage state, beyond collapse prevention = collapse damage state. In calculating the variability of the capacity curve, the properties of the building are changed such that it affects the capacity of the structure. Therefore, by changing the material property there will always be a variation in the capacity curve. Thus, the material

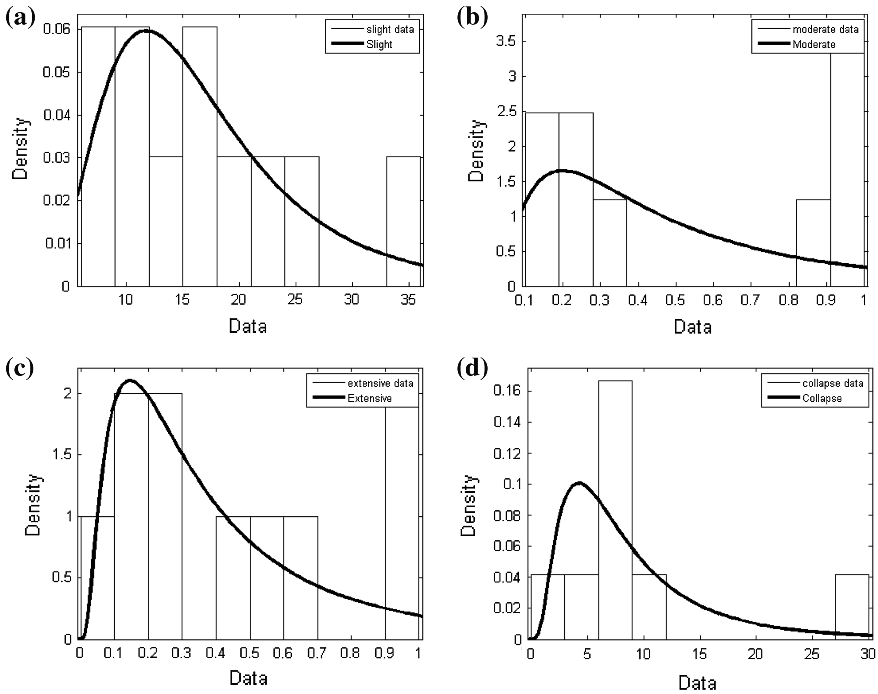

Fig. 3 Uncertainty in the capacity *curve* showing variation in slight, moderate, extensive and collapse damage state

properties, i.e. the grade of concrete and steel are changed. Here, ten models are taken without changing the other parameters except the grade of material. In this work, M20 Fe415, M30 Fe415, M25 Fe500 etc. is assigned to each model building and the pushover analysis is done to find out the capacity curve for each building due to the change in material property. The variation in the capacity curve will give the value of β_c. The variation is then coded in MATLAB program and the value is found out by simulation. Figure 3a–d show the variation in capacity curve for slight, moderate, extensive and collapse damage states, respectively. Table 6 gives the results for the variation in capacity for the different Indian standard codes.

Table 6 Variability of the capacity curve

IS code	$\beta_{c,s\|ds}$	$\beta_{c,m\|ds}$	$\beta_{c,e\|ds}$	$\beta_{c,c\|ds}$
IS-1893(Part I):2002	0.502	0.851	0.883	0.722
IS-1893:1984	0.440	0.731	0.554	0.610
IS-1893:1970	0.441	0.424	0.438	0.481

Table 7 Damage state variability of the model structure

IS code	Total variability			
	$\beta_{s,s\|ds}$	$\beta_{s,m\|ds}$	$\beta_{s,e\|ds}$	$\beta_{s,c\|ds}$
IS-1893:2002	0.669	0.959	0.989	0.847
IS-1893:1984	0.623	0.854	0.709	0.753
IS-1893:1970	0.624	0.613	0.622	0.653

2. **Calculation of variability in demand**

For 5 % damping, about 50 major earthquakes from all over the world are taken which consist of major Indian earthquakes. Response spectrum is found out for every earthquake and their variation is computed with the help of MATLAB. The variability of the demand curve, $\beta_{dem} = 0.387$. The variability in median is taken as 0.4 for this model type of building [3]. Thus, the total variability is given by (3). Table 7 refers to the damage state variability of the example building.

3. **Calculation of the median**

The inter-storey drift found out for each of the damage state is discussed above. Median values of structural component fragility are based on building drift ratios that describe the threshold of damage states. Damage-state drift ratios are converted to spectral displacement by,

$$\overline{S_{d,sds}} = \delta_{R,sds} \cdot \alpha_2 \cdot h \tag{4}$$

where $\overline{S_{d,ds}}$ is the median value of spectral displacement, in inches, of structural components for damage state, ds; $\delta_{R,sds}$ is the drift ratio at the threshold of structural damage state, ds; α_2 is the fraction of the building (roof) height at the location of push-over mode displacement, which is equal to 0.75; h is the typical roof height, in meter, of the model building type of interest. The median values calculated for different damage states are shown in Table 8.

Table 8 Calculation of Median

IS code	Inter-storey drift at threshold damage state (%)				Median value (m)			
	Slight	Moderate	Extensive	Collapse	Slight	Moderate	Extensive	Collapse
IS-1893:2002	0.4	0.7	1.5	2.5	0.036	0.063	0.135	0.225
IS-1893:1984	0.3	0.5	1.4	2.2	0.027	0.045	0.126	0.198
IS-1893:1970	0.3	0.5	1.3	2.0	0.027	0.045	0.117	0.180

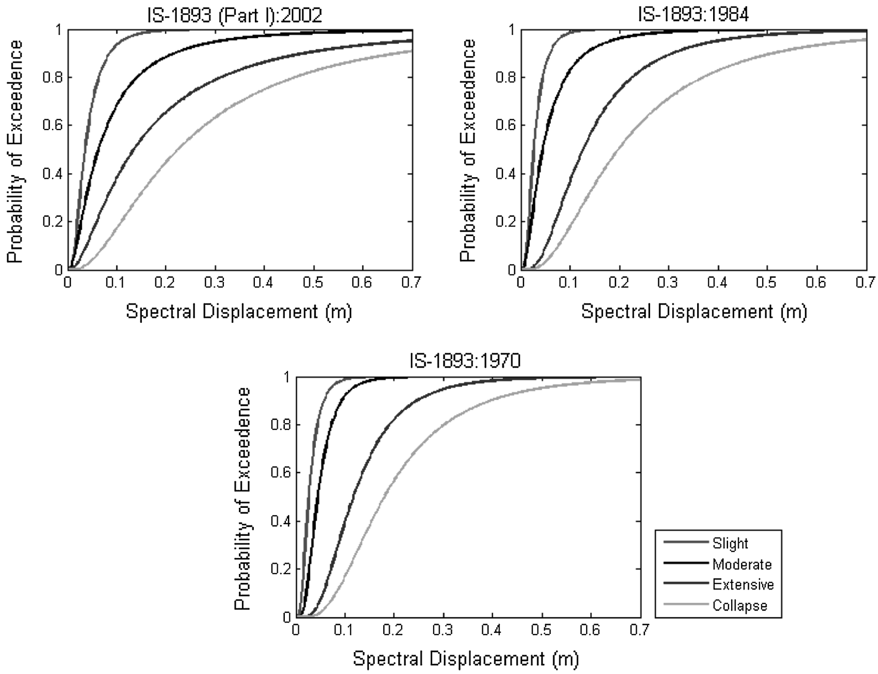

Fig. 4 Cumulative probability of being or exceeding the damage state and spectral displacement

8 Results

The fragility curve distributes among slight, moderate, extensive and collapse damage state. The probabilities of a building reaching or exceeding the various damage levels at a given response level sum to 100 %. From the fragility analysis, the spectral displacement at different damage states is compared against the Indian standard codes. Figure 4 plots the graph between cumulative probability of being or exceeding the damage state and spectral displacement for IS-1893:2002, IS-1893:1984 and IS-1893:1970 and Table 9 shows comparison of probability of exceedance at different damage states.

Table 9 Comparison of probability of exceedance (%) at different damage states

Damage states	IS-1893 (Part I):2002			IS-1893:1984			IS-1893:1970		
	100 mm	300 mm	500 mm	100 mm	300 mm	500 mm	100 mm	300 mm	500 mm
Slight	90	99	100	98	100	100	99	100	100
Moderate	67	93	98	83	98	100	92	100	100
Extensive	39	77	89	39	87	98	40	94	98
Collapse	17	62	80	18	70	90	18	79	94

9 Conclusions

A new damage scale has been developed based on the response parameters obtained from non-linear time history analysis and damage index. The base shear is compared. The following conclusions are arrived at from the study.

1. Design by recent code or high code ensures proper ductility as the capacity is maximum by this code. More the ductility more is the energy dissipation before collapse.
2. In case of damage obtained by pushover analysis, the damage for IS-1893:1970 is maximum for a fixed value of roof displacement compared to IS-1893:1984 and IS-1893:2002, in which the damage obtained by IS-1893:2002 has the least value.
3. Conventionally, it lightens up that the inter-storey drift would increase as capacity goes on reducing, but for PGA 0.54g the inter-storey drift obtained by IS-1893:2002 is more than that obtained by IS-1893:1984.
4. Due to high ductility obtained by the most recent code, the pattern would follow that the maximum roof displacement will occur for IS-1893:2002, but at 0.54g PGA the maximum roof displacement for IS-1893:1984 comes out to be higher than IS-1893:2002.
5. The variability of the capacity curve for slight to collapse decreases from high to low codes. This is in par with the HAZUS MR4 earthquake model.
6. From the fragility curve, it is clearly observed that the values for slight to collapse increases with change from high to low codes. This is due to the fact that the ductility is of primary concern for energy dissipation.

The further scope of study would be to assess the vulnerability for different type of soil conditions and for different type of building stocks.

References

1. DiPasquale E, Ju J-W, Askar A, Cakmak AS (1990) Relation between global damage indices and local stiffness degradation. ASCE J Struct Eng 116(5):1440–1456
2. Ghobarah A, Abou-Elfath H, Biddah A (1999) Response-based damage assessment of structures. Earthq Eng Struct Dynam 28(1):79–104
3. HAZUS (1999) Earthquake loss estimation methodology. HAZUS99 service release 2 (SR2) technical manual, Federal Emergency Management Agency (FEMA), Washington
4. Powell GH, Allahabadi R (1988) Seismic damage prediction by deterministic methods: concept and procedure. Earthquake Eng Struct Dynam 16(5):719–734
5. Park Y-J, Ang AHS (1985) Mechanistic seismic damage model for reinforced concrete. ASCE J Struct Eng 111:722–739
6. Haselton CB (2012) Selecting and scaling earthquake ground motion for performing response history analyses. In: 15th world conference on earthquake engineering, Lisboa
7. Kunnath SK, Reinhorn AM, Lobo RF (1992) IDARC version 3.0: a program for the inelastic damage analysis of reinforced concrete structures. National Center for Earthquake Engineering and Research, Report # NCEER-92-0022. Buffalo, NY, USA, SUNY

8. Mihai M A (2013) Theoretical review of the damage indices used to model the dynamic nonlinear behavior of reinforced concrete structures, pp 109–119
9. Sanagle K, Patil S, Thakur S (2012) Evaluation of damage index of high rise building using non-linear static seismic analysis. Int J Emerg Technol Adv Eng 2(10):595–600
10. Vimala A, Kumar RP (2012) Displacement based damage estimation of RC bare frame subjected to earthquake loads: a case study on 4 storey building. In: 15th world conference on earthquake engineering, Lisboa