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Probabilistic Approach for Sustainable Seismic Design for Performance Evaluation of Existing Buildings

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Abstract

Seismic event is most uncertain with respect to its occurrence, effect on structure and therefore it is always challenging to design engineers to provide safety in the design. Probabilistic approach is the recent trend adopted by the designers from developing countries which accounts various uncertainties involved in the process of seismic evaluation. This approach is an alternative to design practices which not only provides the life safety but also able to achieve the desired performance objectives. The present study is focused on this approach for seismic performance evaluation of existing reinforced concrete building situated in India. Seismic performance evaluation of existing RC building and retrofitting strategies has been assessed using this methodology. For the demonstration,an old reinforced concrete building situated in Indian seismic zone IV has been considered. Seven storied building with open ground story for parking has been retrofitted using concrete shear walls to improve global performance. Effectiveness of retrofitting strategyhas been assessed by comparing analytical based fragility curves. Incremental dynamic analysis based fragility curves have been developed using sixteen ground motions.Three damage states defined in FEMA356(2000) have been considered. Damage probability matrix for two levels of seismic hazard has been developed. Study highlights the effectiveness of probabilistic method for seismic performance evaluation than traditional strength base approach.

Key words – probabilistic approach, retrofitting, concrete jacketing, incremental dynamic analysis, fragility analysis

Introduction

In India, the seismic design philosophy mentioned in revised seismic code (IS1893-2016), is based on - strength approach, which is almost outdated philosophy as compare with the developing countries design philosophy. Probabilistic based seismic performance evaluation methodologyis the recent methodology adopted for performance evaluation of existing buildings in many developing countries(Jalayer and Cornell 2003; Gianvittorio and Immacolata 2009) This methodology account the various uncertainties (aleatory and epistemic) involved in the process of performance evaluation.The outcome of the methodology is to develop fragility curves for the buildings.

Fragility curve is the probabilistic curve expressing the probability of exceeding the damage state for given seismic intensity.
Present study is focused on developing a methodology for seismic performance evaluation of existing concrete building based on probabilistic approach.Fig. 1 shows the developed methodology.

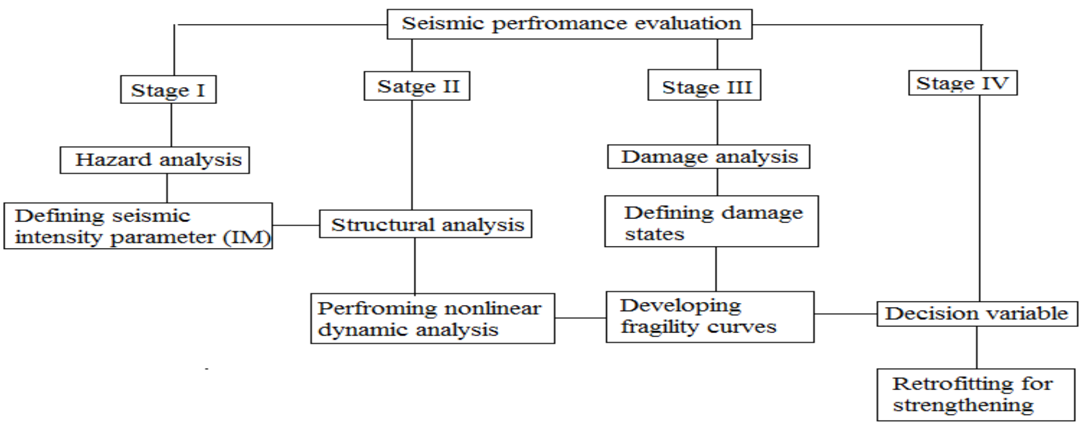

Fig. 1 Methodology for seismic performance evaluation

1 Features of case study building

For the illustration of the methodology, a representative building selected is seven storied apartment building located in Delhi (E), (India), built in 1984. Itis resting on medium clayand with plasticity index less than 20%, safe bearing capacity 200 kN/m^2, mass density 20 Mg/m^3,modulus of elasticity (E_S) 25 N/mm^{2}andPoisson's ratio (μ) 0.35. In plan,

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the structure's footprint is 19 m x 10m andin elevation it is 22.25 m tall as shown in Fig.2.Cross section details of frame elements are shown in table 2 and table 3.Non- destructive tests have also been carried out to obtain in-situ strength of concrete. Accordingly, average characteristic strength of concrete and rebaras14 MPa and 175 MPa have been considered.Live loadof 3 kN/m^{2}and dead load of 6.5 kN/m^{2}have beenconsidered.Linear response spectrum analysis is performed to check the building performance for seismic loading. It has been observed thatcolumns at parking floor are weak in shear due to deterioratedmaterial strength.Failure pattern shows weak column and strong beam behaviour and thus building need to retrofit to improve the local as well asglobal seismic performance.

1.1Retrofitting strategies

Indian code (IS 15988 2003)isrecommending global strengthening techniques to the building having softstory.In the present study shear wall and concrete jacketing for columns have been implemented. In particular followingthree models have been studied.

Model 1: The original un-retrofitted structure having open parking story at ground floor.The contribution of brick masonry infill (BMI)towards lateral resistance and stiffness has been ignored.

Model 2: Same as model-1,but the contribution of BMI towards lateral resistance and stiffness has been considered.

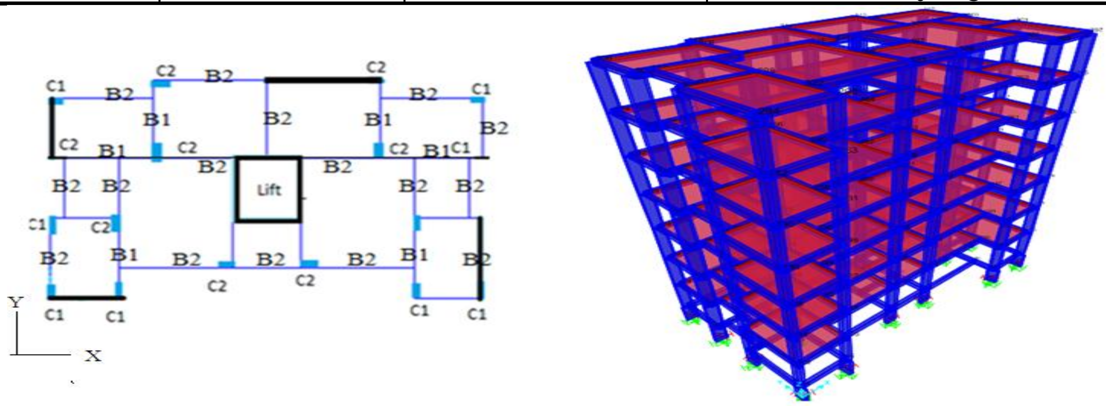
Model 3: Retrofit-I;to strengthen the building in model-1, it is retrofitted by the implementation of shear walls for full heightof model in the configuration shown in Fig.2(a).The columns adjacent to shear wall and all the columns in the parking floor have been concrete jacketed. Jacketed section details are shown in table 2 and in Fig.4b).Concrete jacketed column has been modeled in SAP20000v15as a composite section assuming the jacketed element behaves monolithically, with full composite action betweenold and new concrete,and the concrete properties of the jacket are to apply over the full section of the element. Concrete and rebar used for retrofit are of strength 20 MPa and 415 Mpa respectively have been considered.

Table 1 – Reinforcement details in columns in example buildings

Un-retrofitted column				Retrofitted column			
Title	Size in mm	Main steel (Fe175)	Links (Fe175)	Title	Size after retrofitting in mm	Main steel in Jacket (Fe415)	Links in Jacket (Fe415)
C1	230X600	12-20 Φ	6Φ@175c/c	FSCE -1	430X800	12-20 Φ	10Φ@90c/c
C2	230X750	12-20 Φ	6Φ@175c/c	FSCE -2	430X950	12-20 Φ	10Φ@90c/c

Table 2 – Reinforcement details in beams in example buildings

Title	Size in mm	Main steel (Fe175)		Shear reinforcement (Fe175)
		At top near end	At bottom at center	near end
B1	230X450	6-16 Φ	2-16 Φ + 2-12 Φ	2 –lgd 6Φ@150c/c
B2	230X375	6-12 Φ	2-16 Φ + 2-12 Φ	2 –lgd 6Φ@150c/c



a) Plan b) 3-D building

Fig.2 Example building

2. Nonlinear modelling

Beams and columns are modeled as a link element. Associated nonlinearity is defined by assigning plastic hinges at the ends. Two stiff zones have been considered at the ends of the element. Between two hinges at the ends, the element portion has been considered as elastic.For columns, PMM hinge and for the beams M3 hinge interaction is assigned. At the same location, two hinges were assigned; one is flexural hinge and other is shear hingeAs per FEMA356 (2000), flexural hinge is deformation controlled hinge and shear hinge is force controlled hinge. Fig.3

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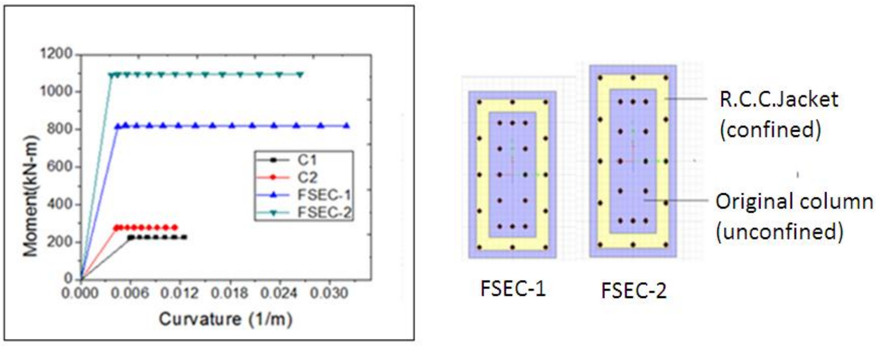
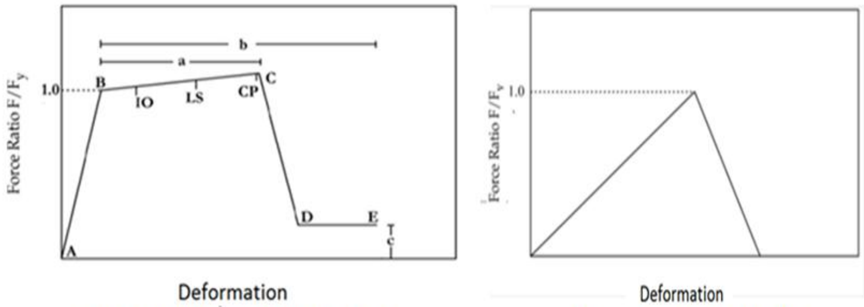
shows *M-ϕ* plot for retrofitted and un-retrofitted column sections. Fig.4 shows the idealized force deformation behaviour of hinge as per FEMA356 (2000).Shear wall is modeled as a mid-pier element. Mid-Pier is modeled as a frame element with the shear wall cross sectional parameters (Fahjanet al.2010;Rahmanet al.2012). Fig.5 a)represents the mid-pier frame model. Footing is modeled as a spring model shown in Fig.5 b) as per ASCE 41(2007).



a) Idealized moment curvature curves from SAP2000 b) cross section of retrofitted columns

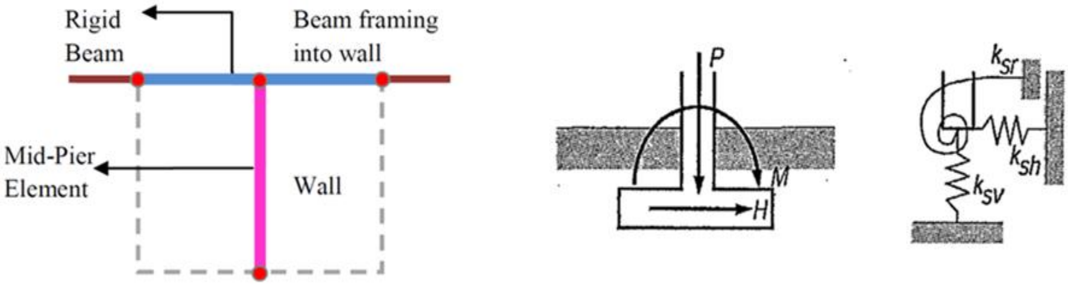
Fig.3 Showing *M-ϕ* curves for columns



a) Deformation controlled action b) Force controlled action

Fig.4 Force deformation behavior, FEMA 356(2000)

2.2.5 Modeling of shear walls



a) b)

a) Mid pier model for shear wall (Fahjan et al.2010) b) uncoupled spring modelfor rigid footings (ASCE/SEI 41- 6 2007)

Fig. 5 Nonlinear modeling of shear wall and rigid footing

4 Incremental Dynamic Analysis

To evaluate dynamic capacities of three models, Incremental dynamic analysis (IDA) has been performed (Vamvatsikos and Cornell 2002).The system capacity is evaluated by dynamic response analyses of the system under a suite of ground motion time histories, which are increased in intensity causing the structural response to increase from linear elastic range into the nonlinear inelastic range and finally to the point where the structure finally becomes unstable. For the present study, spectral acceleration, *Sa* (*T1*,5%) is selected as IM because it represents ground as well as structure response (Vamvatsikos and Cornell 2002). Maximum interstorey drift has been selected as damage measure (DM) since in most of codes damage states are expressed in terms of interstorey drift.

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Sixteen records from eightseismic stations (pairs in two horizontal component directions) have been selected from the PEER strong motion database for IDA.The selected ground motion (GM) records are shown in Table 3. Fig. 6 shows the median IDA curves for three models.

Table 3 Ground motion database for IDA

GM	Event	Station	* $\O^0$	V_{s30}** m/s	M_w	R_{jb}*** (km)	PGA (g)	PGV cm/s
1	Imperial Valley – 02 ,19/5/1940	El Centro Array #9	180	213.44	6.95	6.09	0.28	34.0
2	Imperial Valley – 02 ,19/5/1940	El Centro Array #9	270	213.44	6.95	6.09	0.20	32.0
3	San Fernando,9/2,1971	Pacoima Dam (upper left abut)"	164	2016.1	6.61	0.0	1.20	125.0
4	San Fernando,9/2,1971	Pacoima Dam (upper left abut)"	254	2016.1	6.61	0.0	1.20	55.0
5	Imperial Valley-06,15/10/1979	Bonds Corner	140	223.03	6.53	0.44	0.58	21.0
6	Imperial Valley-06,15/10/1979	Bonds Corner	230	223.03	6.53	0.44	0.80	40.0
7	Northridge-01,17,01,1994	Arleta - Nordhoff Fire Station	90	297.71	6.69	3.3	0.35	40.0
8	Northridge-01,17,01,1994	Arleta - Nordhoff Fire Station	360	297.71	6.69	3.3	0.32	22.0
9	Kobe_ Japan,16/01/1995	KJMA	0	312.0	6.9	0.94	0.70	86.0
10	Kobe_ Japan,16/01/1995	KJMA	90	312.0	6.9	0.94	0.60	85.0
11	Chi-Chi_ Taiwan,20/9/1999	TCU136	N	462.1	7.6	8.27	0.18	50.0
12	Chi-Chi_ Taiwan,20/9/1999	TCU136	W	462.1	7.6	8.27	0.2	45.0
13	Loma Prieta,18/10/1989	Los Gatos - Lexington Dam	0	1070.	6.9	3.22	0.42	86.0
14	Loma Prieta,18/10/1989	Los Gatos - Lexington Dam	90	1070.	6.9	3.22	0.4	98.0
15	Niigata_ Japan,23/10/2004	NIG020	EW	331.6	6.6	7.45	0.5	30.0
16	Niigata_ Japan,23/10/2004	NIG020	NS	331.6	6.6	7.45	0.43	25.0

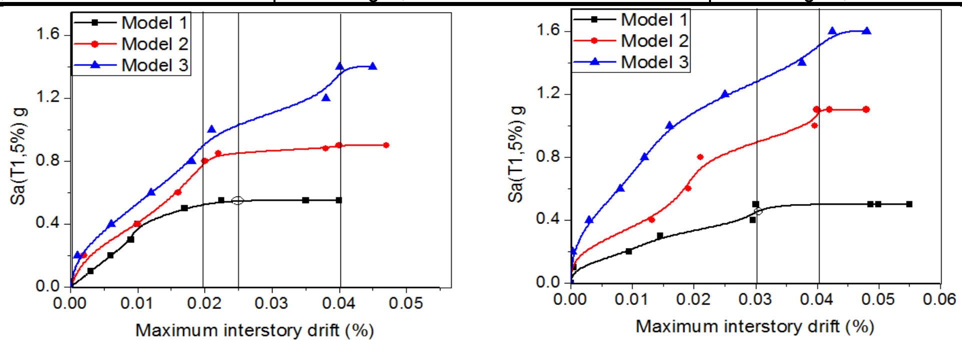
*- Component **- Average shear wave velocity in upper 30 m of soil ,***- Joyner - Boore horizontal distance to surface projection of the rupture.

4.1 Damage states

Under the seismic excitation,the structural model undergoes various damage states (DS). In the present study,three DS have been considered as Immediate occupancy (IO),Life safety (LS) and Collapse prevention (CP) suggested in FEMA356(2000).Table 4 gives the drift limits for three damage states. Since it is very difficult to predict the collapse state (CP) of building under seismic excitation ,therefore the median of IDA curves are used to determine the corresponding maximum inter-story drift ratio at which the median curve started to become a flat line (Vamvatsikos and Cornell 2002).

Table 4- Inter-story drifts capacity for different damage states

Model	Maximum Inter-Story Drift Ratio ($\theta_{max.}$) %		
	Immediate occupancy (IO) from FEMA 356	Life safety (LS) from FEMA 356	Collapse prevention (CP) from IDA
1	1% along *X,Y* direction	2% along *X,Y*direction	2 %along *X*,3% along *Y*
2	1% along *X,Y*direction	2% along *X,Y*direction	2.5 %along *X*,4% along *Y*
3	1% along *X,Y*direction	2% along *X,Y*direction	4 %along *X,Y* direction



a) X-direction b) Y-direction

Fig.6 Median IDA curves for three models

(Vertical lines are showing interstorey drift limit at CP damage state)

5 Probabilistic seismic demand model and Damage probability matrix

A probability distribution for the demand conditioned on the intensity measure (IM) is known as a probabilistic seismic demand model (PSDM). The demand on the structure is quantified using some chosen metric(s) (e.g. inter storey drift, ductility). A regression analysis of the responses related to the limit state of interest as a function of the

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excitation intensity measure is then performed. Wen et al.(2004) has developed a methodology for determining probabilities of different damage states for reinforced-concrete buildings through IDA. In this methodology, two parameter power law has been fitted on the median EDP for a given IM. This power law represents a straight line in log-log space and can be expressed as in equation (1),

$$\ln\theta_{max} = C_1 \ln S_a + C_2 \qquad (1)$$

Where, $\theta_{max}$is the EDP for a given IM S_a; and $C_1$and $C_2$are constants which are determinedfrom regression analysis.A proper distributionfunction (generally lognormal is a good fit) is then selected for EDP and based on which thefragility curve can be determined.The fragility function for limit states (LS) of damage, using IDA can be expressed as in equation 2(Wen et al.2004):

$$P(\mathrm{LS}/\mathrm{S_a}) = 1 - \Phi\left[\frac{\lambda_c - \lambda_{\frac{D}{sa}}}{\sqrt{\beta^2{}_{\frac{D}{sa}} + \beta^2{}_M + \beta^2{}_M}}\right] \qquad (2)$$

where, P(LS/S_a) = Probability of exceeding a limit state given the spectral acceleration at the fundamental period of the building ;Φ = standard normal distribution; $\lambda_c$is the natural logarithm of median inter-story drift capacity at given limit states, $\lambda_{D/Sa}$is the natural logarithm of median inter-story drift demand for a given spectral acceleration. $\beta_{D/Sa}$and β_C, are the variability parameters associated with demand and capacity, respectively, and are given as:

$$\beta_{\mathrm{D/Sa}} = \sqrt{\ln(1+S^2)} \qquad (3)$$

$$S^2 = \text{Standard error} = \frac{\Sigma\left(\ln Y_i - Y_p\right)^2}{N-2} \qquad (4)$$

$$\beta c = \sqrt{\ln(1+\mathrm{COV})^2} \qquad (5)$$

Where, $Y_i$and $Y_p$are the observed and power law predicted median inter-story drift rotation values, respectively, for a given spectral acceleration S_a, N is the number of sample demand data points, and COV is the coefficient of variation of estimated inter-story drift capacity.β_M is the modeling uncertainty which is generally assumed equal to 0.40 as suggested in HAZUS (2004).

5.1 Damage probability matrix

In order to underline the influence of the masonry infill walls and retrofitting strategy,particular damage probability matrices havebeen obtained for maximum considered earthquake (MCE) and design basis earthquake (DBE) seismic demand.MCE and DBE are the two seismic hazard levels defined in IS1893(2016).Three fragility curves corresponding to IO, LS and CP damage states have been developedfor each model in two directions and therefore four damage stages are considered as No damage (ND),Slight damage (SD),Moderate damage(MD) and Collapse damage (CD) stage .Table 10 present discrete damage probability matrices results.Discrete damage probabilities can be calculated as follows:

Probability of no damage, P [ND] = 1-P [IO | S_a]
Probability of slight damage (SD), P [SD] = P [IO | S_a] - P [LS | S_a]
Probability of moderate damage (MD), P [MD] = P [LS | S_a] - P [CP | S_a]
Probability of complete damage (CD), P [CD] = P [CP | S_a] - 0

5.2 Discussion on Fragility curves

❖ Table 5 presents the obtained fragility curve parameters. It has been observed that demandand capacity uncertainties are on lower side for model-3 as compared to model- 1 and model-2.

❖ Table 6 presents the obtained probability results. It has been observed that model- 2 and model- 3 have shown higher seismic resistance and hence lower probability in CD statesfor DBE as well as MCE level of seismic hazardcompare to model-1.

❖ Fig.7a) and b) shows fragility curves for model- 1 along *X*-direction (long direction,) and along *Y*-direction (short direction).Curves are closer (Fig.10b) along short direction showing immediate damages one by one .Curves are well spaced (Fig.7a) along longer direction showing higher resistance between the various damage states .Thus original building is showing higher resistance along long direction compare with short

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direction. This effect is due to poor aspect ratio of building showing substantial differences in results along long and short directions.

❖ Fig.7(d)are the fragility curves for the frame with BMI (model-2) along *Y* direction. It shows higher initial stiffness along short direction due to presence of infill, curves are well spread showing improved resistance compare with curves in Fig.7(b). Along long direction (Fig.7c) damage probability seen to be higher at DBE leveland seen to be lower at MCE level of seismic hazard indicatingno effect of infill at lower intensity.Along longer direction soft storymechanism hasdeveloped and columns in parking flowers have been failed first which results in higher probability of damage.But as seismic intensity increases, BMIeffect has been seen and exhibits improved resistance and hence lower probability of damage.

❖ Model- 3 is retrofitted building ,curves are well spread (Fig.7f) along short direction still showing better improvement in seismic resistance between different damage states compare with curves for model- 2 (Fig.7d). Along long direction also the significant improvement have been observed showing increase in resistance and hence lower probabilities for different damage stages (Fig.7e) as compare with curves for model- 2 (Fig.7c). The soft storey mechanism effect along long direction at lower seismic intensities has been reduced. Thus effectiveness due to retrofitting is seen.

❖ Fig.8 and Fig.9are damage probability matrices showing the performances in different damage statesin the building along*X* and *Y* direction. Table 11 show details of performances shown by three models. Collapse (CD) probability for model-2 with BMI ,as discussed above, along short direction is lower compare with model- 1at MCE(Fig.8 b) and DBE(Fig.9b) level of seismic intensity and along long direction it is more at lower seismic intensity (Fig.9a)and less at higher (Fig.8a)seismic intensity.For retrofitted model-3,collapse probability at CD stage is lower along both directionsthan probabilities for model- 2 as shown in Fig.8 (a and b), 9(a and b).Thus improvement in performance due to retrofitting over model- 2.

❖ Probability of ND is lower for original building model- 1along long direction (Fig.8a andFig.9a) compare with short direction (Fig.8b and 9b) at both level of seismic intensity. But there is an improvement in probabilities for ND due to BIM (model- 2)along both direction as shown in Fig.8 and Fig.9.This highlights the presences of BMI.

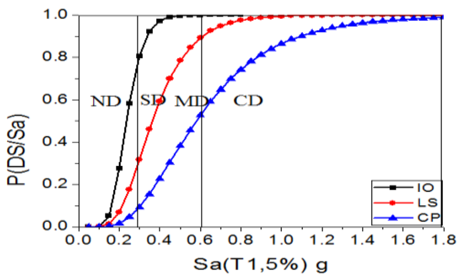
Table 5 Estimated fragility parameters

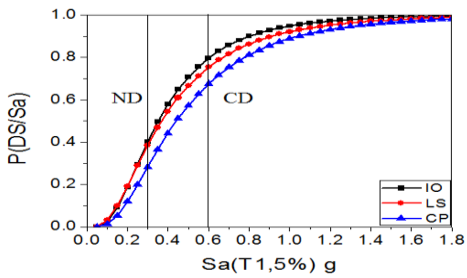
Model	Damage states	βc		$\beta D/Sa$		βM	C_1		C_2		R^2	
		X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*
1	IO	0.52	0.47	0.29	0.34	0.4	1.77	1.8	1.33	1.47	0.86	0.85
	LS	0.54	0.51	0.49	0.4	0.4	1.86	1.83	1.88	1.82	0.85	0.84
	CP	0.5	0.52	0.31	0.32	0.4	1.88	1.92	1.92	2.0	0.86	0.86
2	IO	0.44	0.44	0.33	0.33	0.4	1.98	1.93	0.43	0.4	0.85	0.85
	LS	0.44	0.43	0.39	0.39	0.4	1.90	1.90	0.62	0.62	0.91	0.9
	CP	0.53	0.54	0.37	0.36	0.4	1.95	1.98	0.68	0.73	0.83	0.84
3	IO	0.41	0.42	0.22	0.25	0.4	4.5	1.57	6.53	0.028	0.86	0.86
	LS	0.33	0.32	0.34	0.35	0.4	1.85	1.89	0.44	0.48	0.88	0.89
	CP	0.42	0.53	0.37	0.47	0.4	2.04	1.0	0.68	0.92	0.94	0.94

(R^2-Represents goodness of fit on median power law)

Table 6 Probabilities corresponding to DBE and MCE hazards along both directions

Mod el	DBE						MCE					
	IO		LS		CP		IO		LS		CP	
	X	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*	*X*	*Y*
1	80%	40%	30%	38%	8%	30%	97%	78%	90%	75%	50%	65%
2	55%	60%	40%	25%	30%	8%	70%	78%	50%	60%	42%	40%
3	35%	75%	8%	38%	5%	5%	70%	85%	30%	75%	20%	10%





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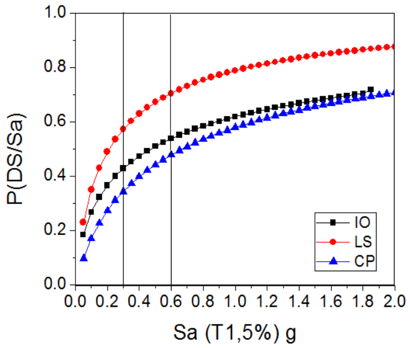
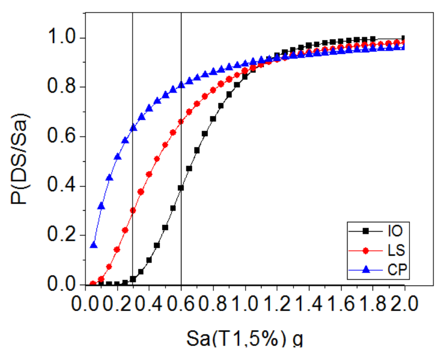
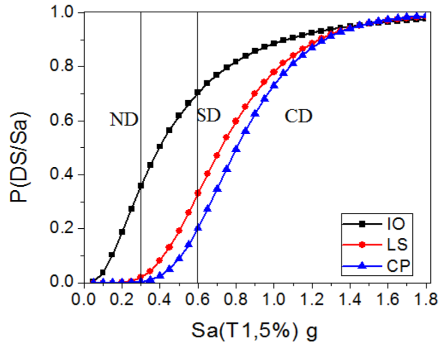
a) Model 1-X direction



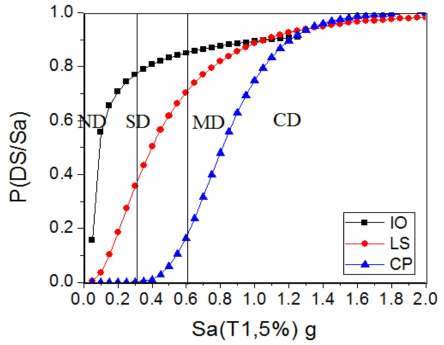
b) Model 1-Y direction



c) Model 2- X direction

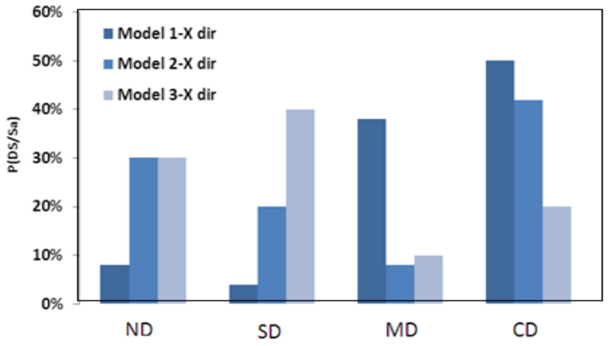


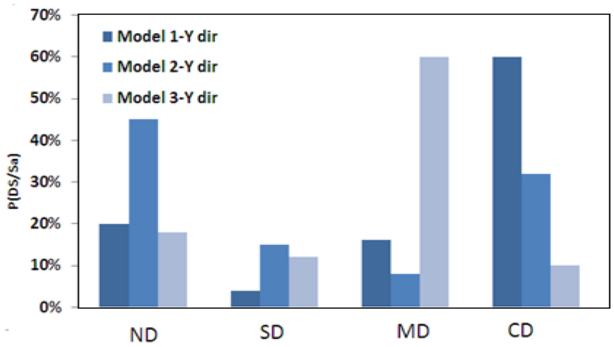
d)Model 2- Y direction



e) Model 3- X direction f)Model 3-Y direction

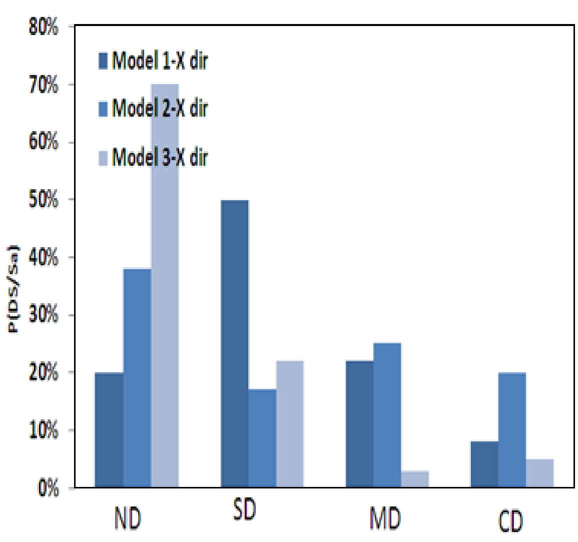
Fig.7 Fragility curves for three limit states
(The two vertical lines represent the seismic demand corresponding to DBE and MCE hazards).

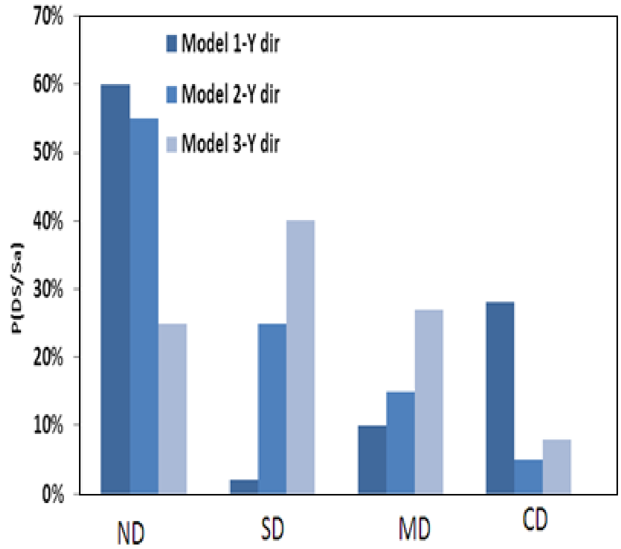




a) X- direction b) Y-direction

Fig.8 Probability matrices for three for MCE seismic hazard





a) X- direction b) Y-direction

Fig.9 Probability matrices for three for DBE seismic hazard

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Table 7 Damage Probability Matrices for MCE and DBE level

Damage State	Model 1		Model 2		Model 3	
	For MCE seismic demand level					
	X-dir.	*Y*-dir.	*X*-dir.	*Y*-dir.	*X*-dir.	*Y*-dir.
No Damage (ND)	8%	20%	30%	45%	30%	18%
Slight Damage (SD)	4%	4%	20%	15%	40%	12%
Moderate Damage (MD)	38%	16%	8%	8%	10%	60%
Complete Damage (CD)	50%	60%	42%	32%	20%	10%
	For DBE seismic demand level					
No Damage (ND)	20%	60%	38%	55%	70%	25%
Slight Damage (SD)	50%	2%	17%	25%	22%	40%
Moderate Damage (MD)	22%	10%	25%	15%	3%	27%
Complete Damage (CD)	8%	28%	20%	5%	5%	8%

6. Conclusions

The study deals with probabilistic seismic evaluation of an existing non-ductile R/C building retrofitted by concrete column jacketing and shear walls. From the study, it is concluded that probabilistic approach demonstrates the seismic performance in probabilistic sense considering the uncertainties and randomness in the evaluation processand also able to predict the performance to future earthquakes.Deterministic approach which is based on strength approach stillis in practice in India but is already outdated in developing countries. So the study recommends probabilistic based design philosophy in Indian seismic code for sustainability.

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