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Seismic risk assessment of low rise RC frame structure

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ABSTRACT

Seismic evaluation and financial risk analysis of typical low rise reinforced concrete frames are performed. The financial risk assessment is determined on the basis of results of incremental dynamic analysis (IDA) of reinforced concrete frames analyzed using nonlinear time history analyses on IDARC platform with a suite of 20 ground motion records used by Vamvatsikos and Cornell (2002) for mid-rise buildings. Three frames were analyzed with capacity design concepts taking into account shear capacity, flexural capacity and contribution from floor reinforcement to beams. Maximum inter-story drift ratios obtained from time-history analyses are plotted against ground motion intensities. Results are statistically interpreted to develop cumulative distribution functions for frames. Fragility curves are plotted for HAZUS damage states of conventional structures. Fragility curves thus drawn are used to estimate the expected annual loss (EAL) of low rise RC frames using quadruple integral formula based on probabilistic financial risk assessment framework.

1. Introduction

Seismic risk assessment and loss estimation are very much essential to minimize the probabilities of seismic hazards. Risk and loss values are useful for budgetary planning, estimating need of manpower for disaster management, educating and making aware to general public and systemize the retrofit applications [3]. Thus loss estimation methodology is an essential tool for earthquake preparedness. This also helps to estimate the insurance premium of building stock for a particular event.

Probabilistic financial risk assessment methodologies have been very popular and are being used since last few decades. Recent studies based on seismic design philosophies and financial risk assessment methodologies suggest that repairing of minor to moderately damaged structures contributes significantly in seismic financial risk. Therefore philosophies like Damage Avoidance Design (DAD) have been proposed by Mander and Cheng [14] for bridge substructures which showed better performance without causing any structural damage.

The triple integration equation developed by Pacific Earthquake Engineering Research (PEER) Center for probabilistic seismic risk assessment is often used to approximate the probability of exceeding a performance requirement. Further Dhakal and Mander [5] have extended the PEER framework formula to a quadruple integral by including time, thereby enabling the quantification of seismic risk in terms of expected annual loss (EAL). This equation certainly is more useful which allows integration of all probable losses in terms of dollar value that would indicate the annual financial risk of the structure due to all possible seismic hazards.

Khare et al. [12] used the quadruple integral formula for financial risk assessment of two reinforced concrete wall systems which were designed for ductility and damage avoidance. They established models for probable financial loss for two walls by combining nonlinear incremental dynamic analysis (IDA) results and seismic hazard recurrence relationship along with damage models developed based on experimental investigations.

Bothara et al. [2] extended the fragility functions to seismic loss assessment of Unreinforced Masonry (URM) houses. The URM test models were dynamically tested on shake tables by applying various ground motions with peak ground accelerations up to 0.5 to 0.8 in longer and shorter directions respectively. The experimental results thus obtained were used for the development of fragility functions and expected annual loss for URM was calculated using quadruple integral equation given by Dhakal and Mander [5].

In present work, the seismic risk assessment and estimation of expected annual loss of a typical low rise three story reinforced concrete frame are performed. The RC frame used in this paper for the risk assessment was analyzed by Melani et al. [18] using nonlinear time history analyses on IDARC platform with a suite of 20 ground motion records used by Vamvatsikos and Cornell [23]. In previous study, maximum inter-story drift ratios obtained from the time-history analyses of three frames

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Table 1
Classification of HAZUS damage states.

Damage state	Damage descriptor	Post-earthquake utility of structure
1	None (pre-yield)	Normal
2	Minor/Slight	Slight damage
3	Moderate	Repairable damage
4	Major/extensive	Irreparable damage
5	Complete collapse	–

(Frame-1, Frame-2 and Frame-3) were plotted against ground motion intensities based on yield and collapse damage states. Frame-1 is designed as per Indian seismic code [10] with ductility provisions which incorporates provisions of capacity design for shear in beams, columns and beam–column joints as per [9] which is Indian standard code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces. This frame does not incorporate capacity design concept in flexure as it is not available in Indian seismic codes. The designed sizes of columns and beams are 400 mm × 400 mm (2.35% steel) and 300 mm × 500 mm (2.5% steel) respectively. Frame-2 includes capacity design concept of columns in flexure as per [1], therefore the designed size of columns gets revised with 500 mm × 500 mm (2.35% steel) whereas beam sizes remain the same. As per clause 21.6.2.2 of [1], the contribution of floor reinforcement to beams requires even stronger capacity of columns. Therefore in Frame-3, the redesigned size of columns for such contributions comes out to be 600 mm × 600 mm (2.2% steel) and beam sizes remain the same.

The IDARC includes two types of hysteretic models, the polygonal and smooth hysteretic models. The polygonal model corresponds to the actual behavioral stages of structure which again includes bilinear model, trilinear model and vortex oriented model. In this study, vertex oriented model with four-parameter hysteretic models is used. The frames were analyzed for three different degradation conditions viz. mild, moderate and severe as described in IDARC 2D technical report by the National Center for Earthquake Engineering Research (NCEER). The different values of four parameters i.e. HC, HBD, HBE and HS decide the type of degradation condition. An increase in the value of HC retards stiffness degradation whereas HBD and HBE accelerate strength deterioration and an increase in value of HS reduces amount of slip. Among three degradation conditions, moderate degradation condition represents more realistic results when compared with past experimental studies, the values of four parameters i.e. HC, HBD, HBE and HS for this condition are taken as 10.0, 0.3, 0.15 and 0.25 respectively. In this paper using the same maximum inter-story drift ratios of RC frames, fragility curves are plotted based on HAZUS damage states for conventional structures. The fragility curves are used for the estimation of expected annual loss using Dhakal and Mander [5] quadruple integral equation.

2. Classification of damage states

As per the technical manual prepared by the National Institute of Building Sciences (Earthquake Loss Estimation Methodology) for Federal Emergency Management Agency [8], HAZUS classified the damage states as a numerical indicator from one to five with reference to increasing level of damages as shown in Table 1.

2.1. IDA plots of frames

The incremental dynamic analysis (IDA) results of frames as plots between peak ground acceleration (PGA) versus maximum inter-story drift ratios at 10th, 50th and 90th percentiles are shown in Fig. 1:

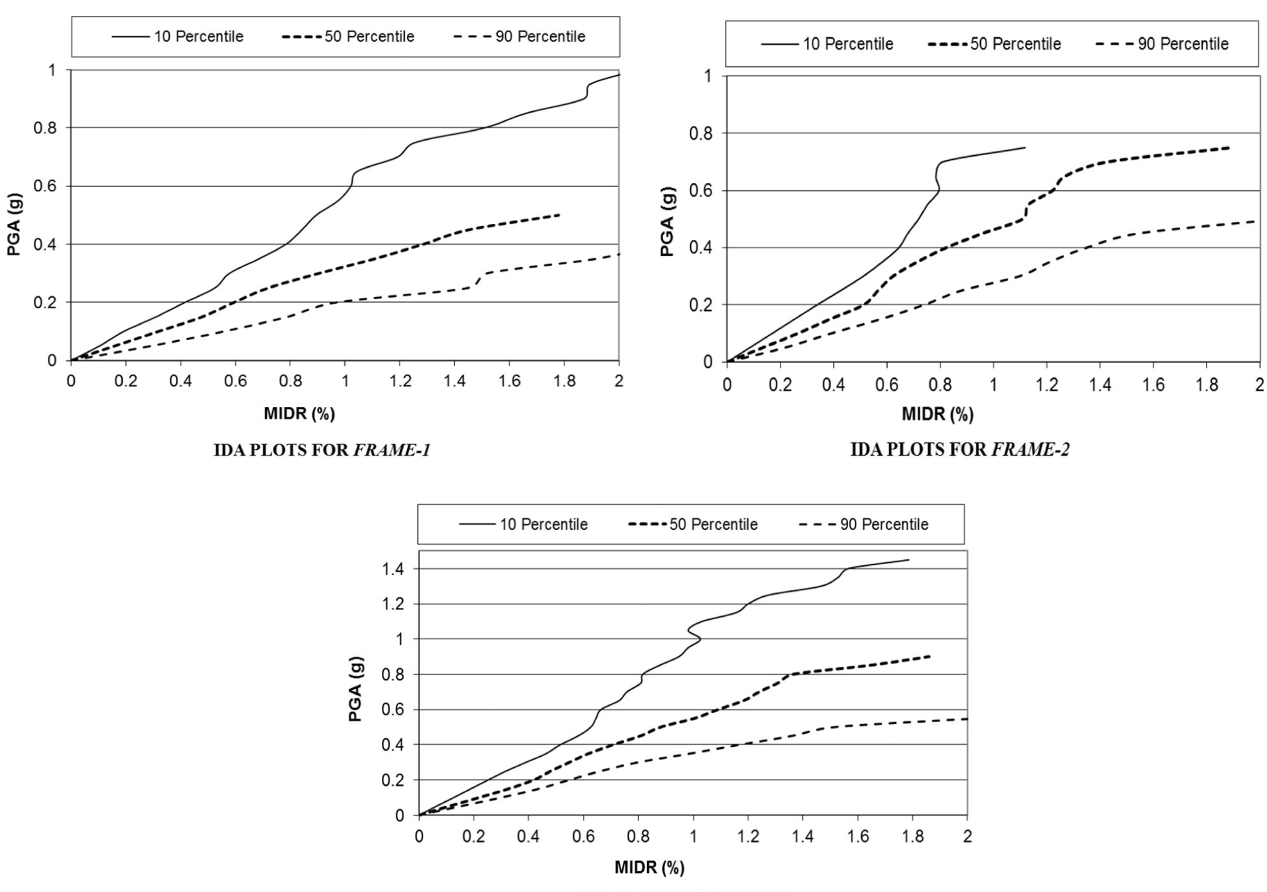

Fig. 1. IDA plots for 3-story RC frame.

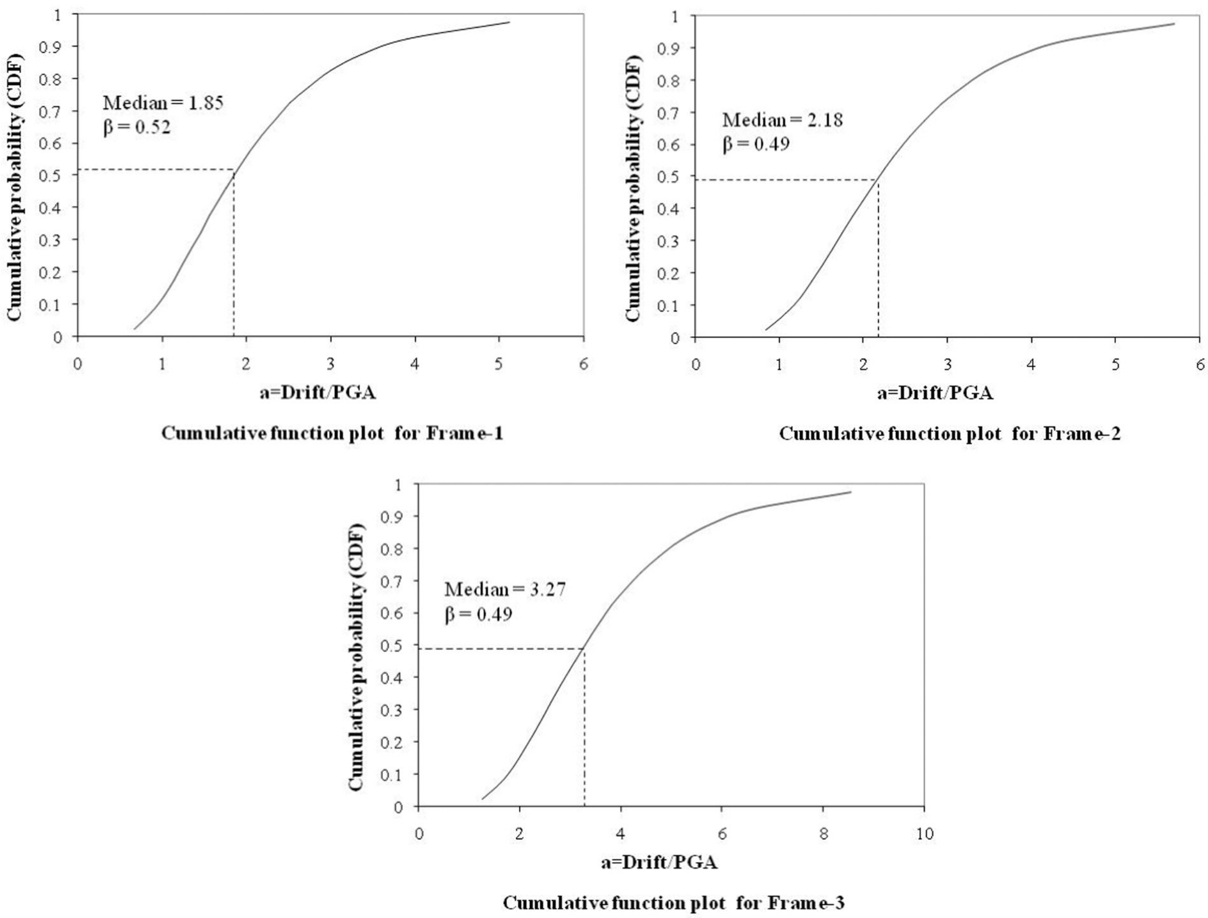

Fig. 2. Cumulative function plots for RC frames.

2.2. Assessment of cumulative distribution function & drift demand

Matthews [17] assessed the drift demands of seismically vulnerable multistory concrete buildings with precast hollow core floor units constructed in New Zealand. Matthews approach was initially developed by Cornell et al. [4] for steel structures and further extended by Lupoi et al. [13] for reinforced concrete structures. Dhakal et al. [6] used the similar approach during his

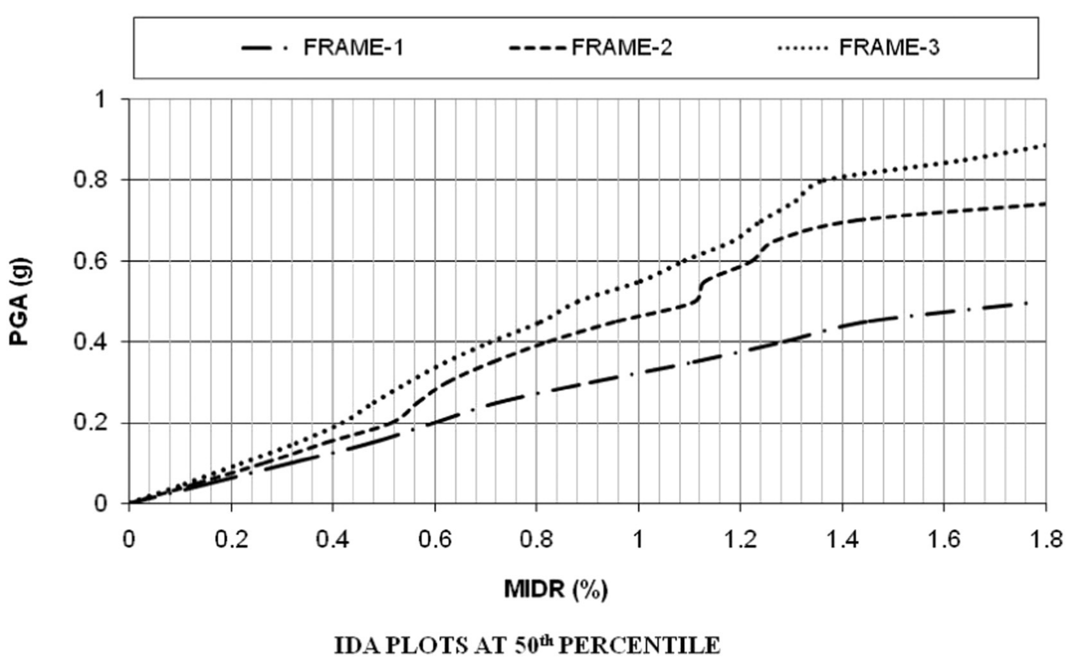

Fig. 3. IDA plots of frames at 50th percentile.

Table 2
Drift limits & PGA values based on IDA results.

Damage state	Frame-1		Frame-2		Frame-3	
	Drift (%)	PGA (g)	Drift (%)	PGA (g)	Drift (%)	PGA (g)
DS,1–2	0.5%	0.17	0.5%	0.2	0.5%	0.27
DS,2–3	1.2%	0.39	1.2%	0.6	1.2%	0.66
DS,3–4	2.5%	0.5	2.5%	0.74	2.5%	0.89
DS,4–5	2.5%	0.5	2.5%	0.74	2.5%	0.89

experimental research on prototype concrete buildings topped with precast concrete hollow core units. In order to normalize the time history analysis results and to have a common variable for various forms of earthquake motions, they plotted the time history analysis results in the form of cumulative distribution versus drift index proportionality parameter (a = drift/F_vS_1), where F_vS_1 being one second spectral acceleration. In this paper, similar approach is used by changing intensity measure from one second spectral acceleration to peak ground acceleration and drawn a relationship in the form of cumulative distribution versus peak ground acceleration as a drift index proportionality parameter (a = drift/PGA) separately for all three frames. See Fig. 2

To summarize drift values corresponding to different HAZUS damage states, 50th percentile values are taken in consideration as shown in Fig. 3. IDA curves developed are used to set drift limits at different levels of damages as defined by HAZUS. Further these curves are also used to develop linear equations to express relationship between median drift and peak ground acceleration (PGA). Drift limits and PGA values from 50th percentile IDA curves for different damage states as defined by HAZUS are tabulated in Table 2.

From Fig. 2, the cumulative lognormal probability distribution fits well with a median value of 1.85, 2.18, and 3.27 for Frame-1, Frame-2 and Frame-3 respectively. Similarly, the dispersion factor (lognormal coefficient of variation) conformed well at 0.52 for Frame-1 and at 0.49 for Frame-2 and Frame-3. Thus, the relationship between median drift and peak ground acceleration for all frames can be mathematically expressed as:

$$\text{For Frame}-1, \tilde{D}_D = 1.85(PGA)_D \tag{1}$$

$$\text{For Frame}-2, \tilde{D}_D = 2.18(PGA)_D \tag{2}$$

$$\text{For Frame}-3, \tilde{D}_D = 3.27(PGA)_D \tag{3}$$

where, $\tilde{D}_D$ = 50th percentile median drift demand as a percentage of story height, $(PGA)_D$ = peak ground acceleration. The median drift capacity can be expressed by inverting above equations as:

$$\text{For Frame}-1, (PGA)_C = 0.54\tilde{D}_C \tag{4}$$

$$\text{For Frame}-2, (PGA)_C = 0.49\tilde{D}_C \tag{5}$$

$$\text{For Frame}-3, (PGA)_C = 0.31\tilde{D}_C \tag{6}$$

where, $\tilde{D}_C$ = expected drift capacity of the structure. When both capacity and demand are involved in design, uncertainties bound to occur. The uncertainties involved in such situations may be due to drift capacity, drift demand, modeling uncertainties etc. Therefore, to overcome such uncertainties Kennedy et al. [11] suggested merging of lognormal distributions, thus the resultant lognormal coefficient of variation can be given as:

$$\beta_{C/D} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2} \tag{7}$$

where,

$\beta_{C/D}$ resultant lognormal coefficient of variation;

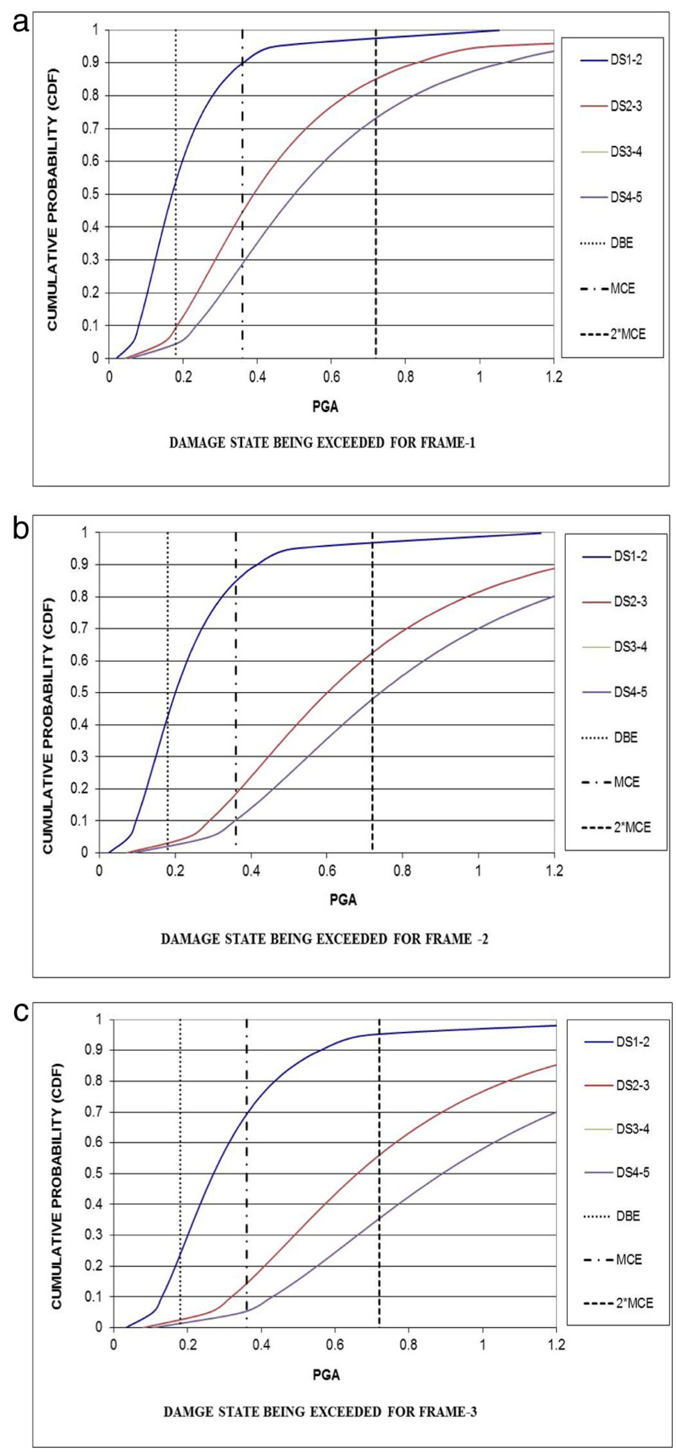

Fig. 4. Fragility curves for RC frames based on HAZUS damage states.

β_C uncertainty due to drift capacity taken here $\beta_C = 0.2$ as suggested by Dutta [7];

β_D uncertainty due to drift demand calculated as earlier 0.52 for Frame-1 and 0.49 for Frame-2 and Frame-3;

β_U dispersion parameter to account for modeling uncertainty taken here $\beta_U = 0.2$.

Thus, the distribution of ground motion demands required for a particular state of damage with $\beta_{C/D} = 0.59$ for Frame-1 and $\beta_{C/D} = 0.57$ for Frame-2 and Frame-3 can be given as:

$$\text{For Frame}-1, PGA = 0.54\tilde{D}_C(\text{DS})\xi_\beta \tag{8}$$

$$\text{For Frame}-2, PGA = 0.49\tilde{D}_C(\text{DS})\xi_\beta \tag{9}$$

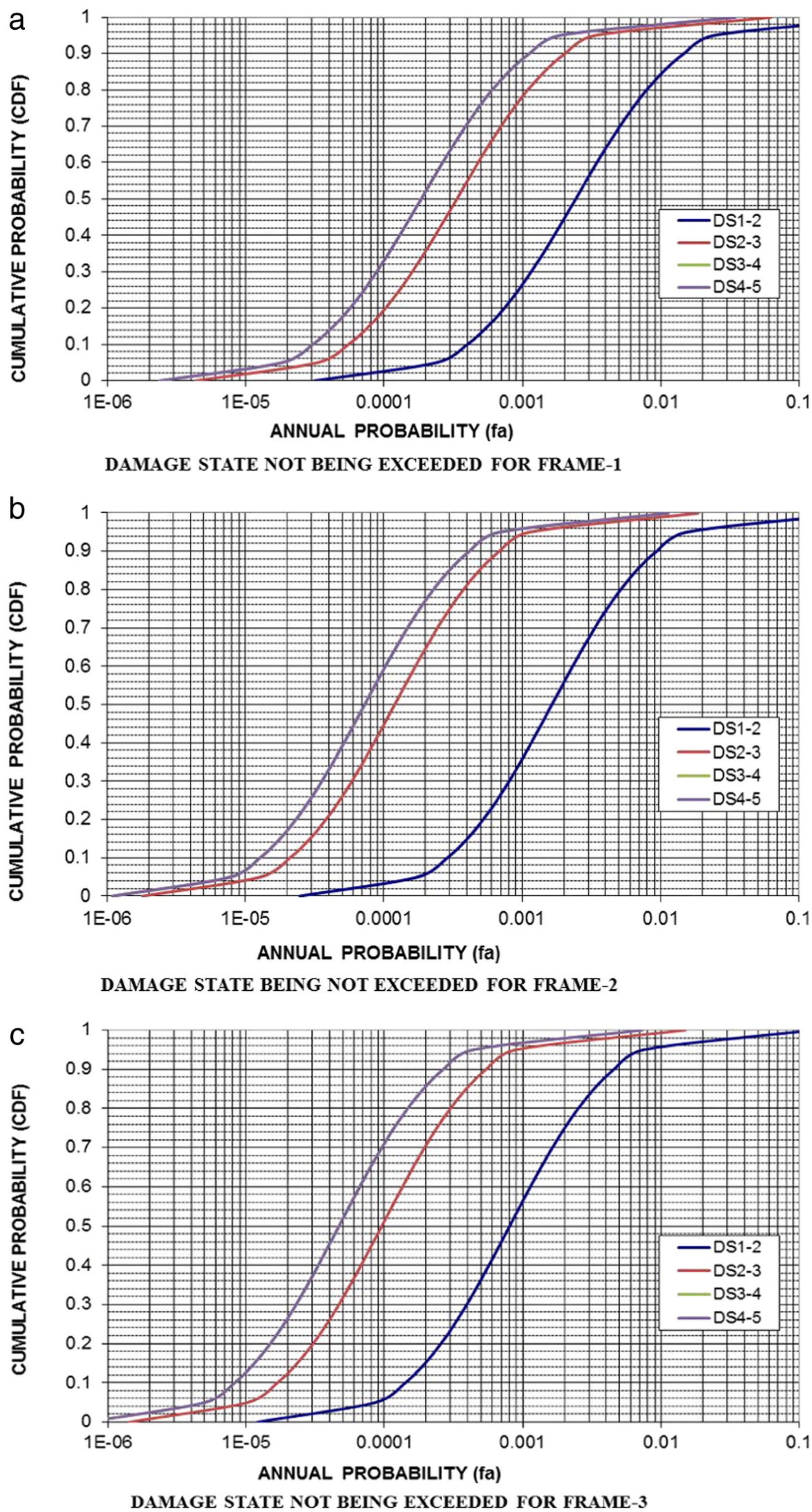

Fig. 5. Hazard survival curves for RC frames based on HAZUS damage states.

$$\text{For Frame}-3, PGA = 0.31\tilde{D}_C(\text{DS})\xi_\beta \quad (10)$$

where, $\tilde{D}_C(\text{DS})$ = observed expected drift corresponding to a given damage state (DS) as shown in Table 2 and ξ_β = lognormal variate.

2.3. Generation of fragility curves

From Table 2, when drift limits get replaced by damage states then fragility curves are generated. A fragility curve represents implicitly the probability of different damages being exceeded in a given earthquake event. The zone factors given in IS 1893 (Part 1): 2002 for a particular zone represents Peak Ground Acceleration of Maximum Considered Earthquake (MCE, 0.36 g for Zone-V) with 2475 years return period which is lowered to Design Basis Earthquake (DBE, 0.18 g for Zone-V) with 475 years return period. Apart from this it is evident from recent earthquakes in India and nearby countries; the earthquake shakings recorded were exceeding the code based limits. To assess vulnerability of analyzed frames in shakings beyond code based limits, one more hazard level is considered as twice of MCE at 0.72 g PGA in the fragility curves which can be treated as MCE value for future probable shakings with higher amplitudes.

It is observed from Fig. 4(a), only 10% of Frame-1 (detailed as per [9]) buildings will survive at MCE i.e. will sustain slight or no damage and remaining 90% would be expected to experience moderate to severe damage, out of these around 15% may sustain irreparable damages being above DS4 damage state. At 2*MCE level 85% such buildings would be experiencing irreparable damages, of these around 74% would be entirely collapsed. It is also evident for Frame-1 that at DBE around 50% of such buildings will escape damages and around 10% of these would be severely damaged which may lead to loss of life.

From Fig. 4(b) which includes capacity design concept of columns in flexure as per [1], shows better performance in terms of damage levels. At MCE 15% Frame-2 buildings would escape damages and 20% would experience irreparable damages or may need demolition of structure being completely collapsed. The Frame-2 results highlighting the fact that by incorporating capacity design concept design of columns in flexure, main frames perform well even at 2*MCE level with some improvement over Frame-1 which require 50% of buildings to be demolished as compared to Frame-1 with 74%.

Similarly, when contribution of floor reinforcement to beams as per Clause 21.6.2.2 of [1] is taken in consideration, frames perform even better with 35% of Frame-3 buildings would require to be demolished at 2*MCE, Fig. 4(c). These buildings are expected to have no damage with 80% and 30% survival at DBE and MCE respectively. As drift limits corresponding to damage state DS3-4 and DS4-5 boundaries are the same in Table 2, 50% probability of such buildings being in moderate damage state is assured at 2*MCE with repairable damages. It is observed from fragility curves that the performance of main frames even at expected future probable shakings can be improved by enhancing detailing techniques following international standards without affecting total project cost much.

3. Seismic risk assessment

Expected annual loss (EAL) has been proved an important tool to communicate seismic vulnerability incorporating all expected damages of structures in a median dollar value. It is very useful for decision makers for cost benefit analysis of new structures as well as for retrofitting alternatives of existing structures. Pacific Earthquake Engineering Research (PEER) Center has given an expression in the form of a triple integration equation for calculating EAL which is further extended by Dhakal and Mander [5] including time to a quadruple integral equation given as:

$$EAL = \int_0^1\int_0^1\int_0^1\int_0^1 L_R \cdot dP[L_R|DM] \cdot dP[DM|EDP] \cdot dP[EDP|IM] \cdot df_a[IM] \quad (11)$$

where,

IM	intensity measure
f_a[IM]	annual probability of an earthquake of a given IM.
EDP	engineering demand parameter
DM	damage measure.
L_R	loss ratio.

Table 3a
Probability of not exceeding different damage states for Frame-1.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	0.98	1	1	1	1
0.01	0.84	0.97	0.98	0.98	1
0.001	0.28	0.78	0.88	0.88	1
0.0001	0.02	0.2	0.34	0.34	1
0.00001	0	0.02	0.03	0.03	1
0.000001	0	0	0	0	1

Table 3b
Probability of not exceeding different damage states for Frame-2.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	1	1	1	1	1
0.01	0.96	0.99	1	1	1
0.001	0.36	0.94	0.96	0.96	1
0.0001	0.04	0.44	0.6	0.6	1
0.00001	0	0.04	0.07	0.07	1
0.000001	0	0	0	0	1

Table 3c
Probability of not exceeding different damage states for Frame-3.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	1	1	1	1	1
0.01	0.96	0.99	1	1	1
0.001	0.56	0.96	0.97	0.97	1
0.0001	0.06	0.52	0.72	0.72	1
0.00001	0	0.05	0.13	0.13	1
0.000001	0	0	0	0	1

Table 4a
Probability of being in a given damage state for Frame-1.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	0.98	0.02	0	0	0
0.01	0.84	0.13	0.01	0	0.02
0.001	0.28	0.5	0.1	0	0.12
0.0001	0.02	0.18	0.14	0	0.66
0.00001	0	0.02	0.01	0	0.97
0.000001	0	0	0	0	1

Table 4b
Probability of being in a given damage state for Frame-2.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	1	0	0	0	0
0.01	0.96	0.03	0.01	0	0
0.001	0.36	0.58	0.02	0	0.04
0.0001	0.04	0.4	0.16	0	0.4
0.00001	0	0.04	0.03	0	0.93
0.000001	0	0	0	0	1

Table 4c
Probability of being in a given damage state for Frame-3.

f_a	P[DS1]	P[DS2]	P[DS3]	P[DS4]	P[DS5]
0.1	1	0	0	0	0
0.01	0.96	0.03	0.01	0	0
0.001	0.56	0.4	0.01	0	0.03
0.0001	0.06	0.46	0.2	0	0.28
0.00001	0	0.05	0.08	0	0.87
0.000001	0	0	0	0	1

3.1. Earthquake recurrence relationship for hazard survival curves

In order to use fragility curves in terms of EAL i.e. Eq. (11), the horizontal axis of fragility curves needs to be replaced with annual probability (f_a). The well known relationship between peak ground acceleration of earthquakes (a_g) with their annual probability of occurrence (f_a) can be expressed as:

$$a_g = \frac{a_g^{DBE}}{(475 f_a)^q} \tag{12}$$

where, a_g^{DBE} is the PGA of the DBE (10% probability of occurrence in 50 years) and q is an empirical constant which is calibrated to be equal to 0.42 for seismic hazard as per Indian seismic code perspective. Therefore, hazard survival curves based on HAZUS damage states based on relationship expressed in Eq. (12) can be re-plotted by changing horizontal axis of fragility curves from IM to f_a as shown in Fig. 5 for all frames.

The intersections of any vertical line through a value of f_a with the hazard survival curves give the probability of these damage states not being exceeded in earthquakes of that annual probability of occurrence. Thus obtained damage state survival probabilities in earthquakes of different frequencies are shown in Tables 3a–3c. Similarly, Tables 4a–4c shows the probabilities of being in a given damage state for all frames.

4. Financial assessment of frames analyzed

4.1. Loss model

The financial implication of each damage state can be expressed in terms of loss ratio (L_R), which is defined as a ratio of the cost required to restore the structure to its full working condition to the replacement cost of the structure. The assumed values based on analysis results and likely ranges of loss ratios for different damage states are shown in Table 5.

Table 5
Loss ratios for different damage states.

	DS1	DS2	DS3	DS4	DS5
Likely range	0	0.05–0.15	0.2–0.4	1.0–1.2	1.0
Assumed L_R value	0	0.1	0.4	1.0	1.0

According to HAZUS damage state classification Table 1, as no damage is expected in DS1 damage state therefore no any financial loss is expected to be incurred, hence loss ratio is zero for DS1. To account for minor repairs loss ratio assumed for DS2 is $L_R = 0.1$. For moderate damages which are to be repaired for its functional use the likely range is 0.2–0.4, here $L_R = 0.4$ is adopted for DS3. Irreparable damages may prove to be uneconomical therefore it is better to go for the replacement of structure; hence for DS4 and DS5 assumed loss ratio is $L_R = 1.0$. Using the assigned loss ratios, the probable financial loss of all frames as a fraction of total replacement cost in different damage states is given in Tables 6a–6c. Also, Tables 6a–6c are represented in terms of bar charts in Fig. 6a–6c.

As predicted, no losses incurred due to DS1 damage state for any frame at any return period. Losses at DS2 damage state are also acceptable as being under minor damage category and contributing much to the total losses up to 0.001 annual probability earthquakes, however total losses for Frame-1 are 20% whereas it became almost half as 10% and 8% for Frame-2 and Frame-3 respectively. Repairing cost of all the frames is dominated by damage state DS3 than that of DS2 in the case of rare earthquakes of annual probability more than 0.001. At earthquake events of annual probability more than 0.001 the contribution in total losses of Frame-1 gets shifted from DS3 damage state (at 0.0001 fa) to DS5 damage state (i.e. >0.0001 fa). Whereas, for Frame-2 and Frame-3 the shifting is similar provided these frames will have minor damages instead of repairable damages at smaller earthquakes and total losses get reduced for improved column capacity frames even at rare earthquake events.

Table 6a
Probable financial loss of frame-1.

f_a	L_R [DS1]	L_R [DS2]	L_R [DS3]	L_R [DS4]	L_R [DS5]	Total L_R
0.1	0	0.002	0	0	0	0.002
0.01	0	0.013	0.004	0	0.02	0.037
0.001	0	0.05	0.04	0	0.12	0.21
0.0001	0	0.018	0.056	0	0.66	0.734
0.00001	0	0.002	0.004	0	0.97	0.976
0.000001	0	0	0	0	1	1

Table 6b
Probable financial loss of frame-2.

f_a	L_R [DS1]	L_R [DS2]	L_R [DS3]	L_R [DS4]	L_R [DS5]	Total L_R
0.1	0	0	0	0	0	0
0.01	0	0.003	0.004	0	0	0.007
0.001	0	0.058	0.008	0	0.04	0.106
0.0001	0	0.04	0.064	0	0.4	0.504
0.00001	0	0.004	0.012	0	0.93	0.946
0.000001	0	0	0	0	1	1

Table 6c
Probable financial loss of frame-3.

f_a	L_R [DS1]	L_R [DS2]	L_R [DS3]	L_R [DS4]	L_R [DS5]	Total L_R
0.1	0	0	0	0	0	0
0.01	0	0.003	0.004	0	0	0.007
0.001	0	0.04	0.004	0	0.03	0.074
0.0001	0	0.046	0.08	0	0.28	0.406
0.00001	0	0.005	0.032	0	0.87	0.907
0.000001	0	0	0	0	1	1

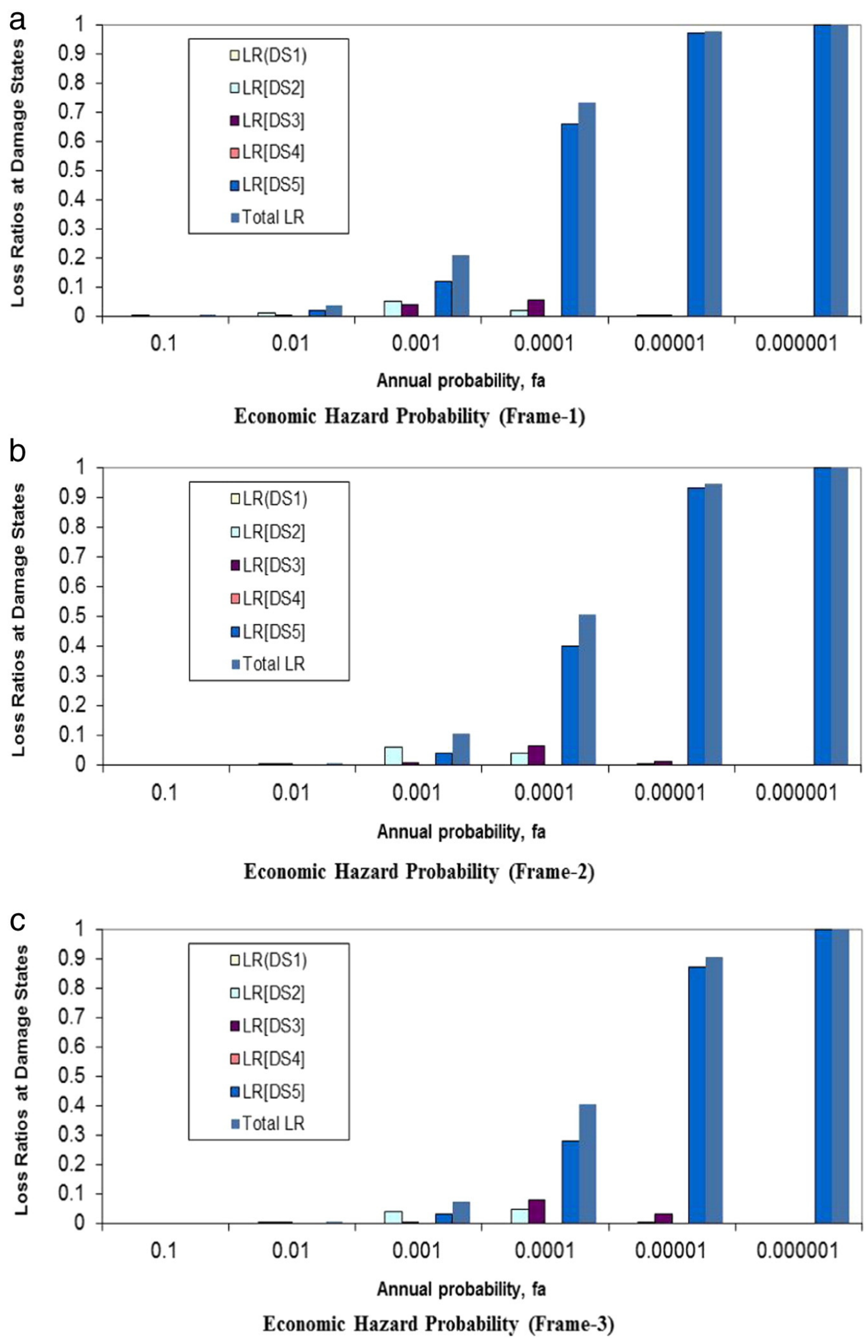

Fig. 6. Economic hazard probability curves as bar charts for RC frames based on HAZUS damage states.

4.2. Expected annual loss calculation

By integrating loss ratio over all possible annual frequencies of the seismic hazard, the expected annual loss can be expressed in continuous form as:

$$\mathrm{EAL} = \int_0^1 L_R df_a. \tag{13}$$

Similarly, in discrete form the expected annual loss can be expressed as:

$$\mathrm{EAL} = \sum_{all\ l_{r,i}} \left(\frac{l_{r,i} + l_{r,i+1}}{2}\right)\left(f_a\left[L_R = l_{r,i}\right] - f_a\left[L_R = l_{r,i+1}\right]\right) \tag{14}$$

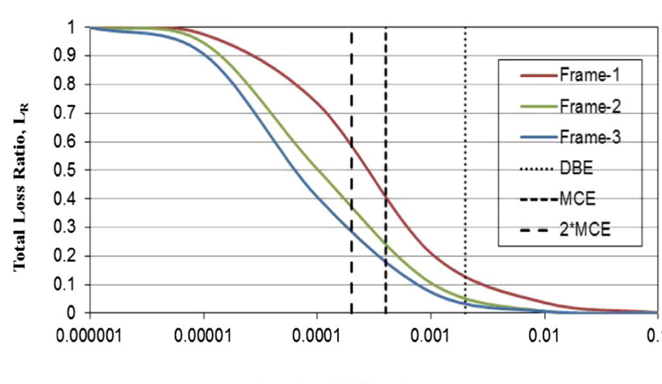

Fig. 7. Economic hazard probability curves for RC frames.

Table 7
Annual financial risk for frames.

f_a	EAL (per $1 million)					
	Frame-1		Frame-2		Frame-3	
	L_R	ΔEAL	L_R	ΔEAL	L_R	ΔEAL
0.1	0.002		0		0	
		1755		315		315
0.01	0.037		0.007		0.007	
		1111.5		508.5		364.5
0.001	0.21		0.106		0.074	
		424.5		274.5		216
0.0001	0.734		0.504		0.406	
		77		65.25		59
0.00001	0.976		0.946		0.907	
		9		8.75		8.5
0.000001	1		1		1	
Total EAL		3377		1172		963

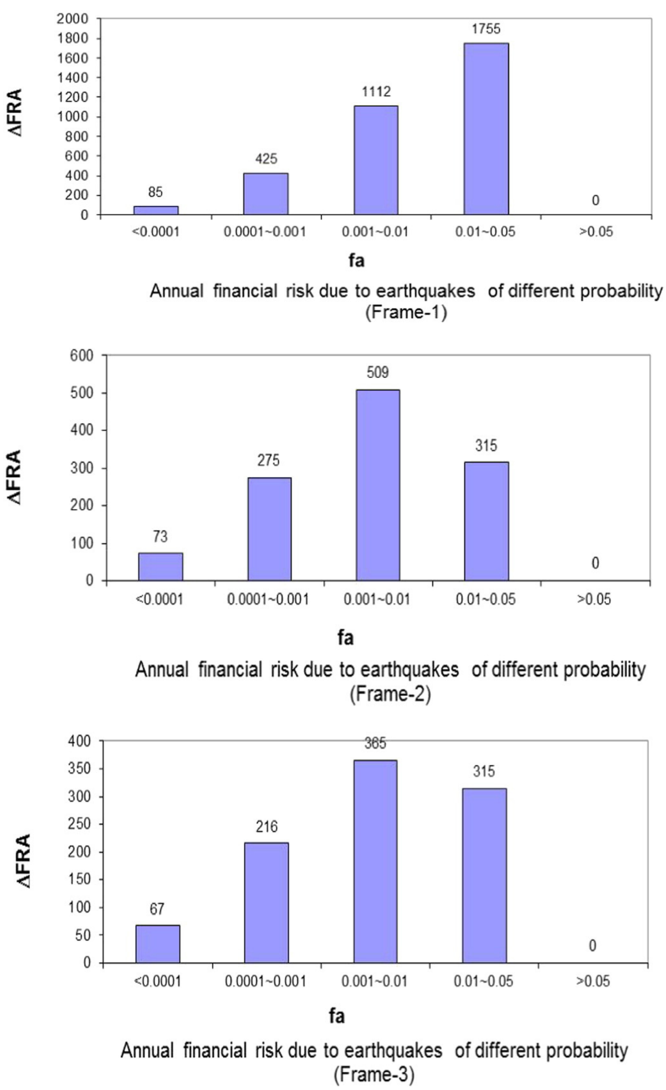

Fig. 8. Annual financial risk of frames due to earthquakes of different probabilities.

where, $f_a[L_R = l_r]$ is the annual probability of the loss ratio being equal to a given value l_r which can be obtained from the economic hazard probability curves (Fig. 7).

The losses contributed by the earthquakes with different ranges of probability are added together to obtain the total expected annual loss. The annual loss of all the three frames is shown in Table 7. From Fig. 7 it can be noticed that for higher amplitude earthquakes loss ratio has a significant effect on the overall performance of the structure. By adapting Frame-3 provisions and detailing techniques the losses can be minimized up to 50% viz. $L_R = 0.6$ for Frame-1 whereas $L_R = 0.29$ for Frame-3 at 2*MCE level of earthquake.

It can be seen from Table 7 that EAL for Frame-1 type buildings with IS-13920 provisions sustains majority of losses at frequent earthquakes of lower amplitudes. Whereas, annual losses become one third once capacity design concepts of columns in flexure are included in the main frames and even less (i.e. almost one-fourth) if contribution of floor reinforcement to beams is included.

It is evident from Fig. 8 and Table 8 that Frame-2 designed as per capacity design provisions for shear, flexure; and Frame-3 designed with capacity design provisions for contribution of floor reinforcement to beams will have lot of savings in repairing cost. The increase in base cost incurred for higher column sizes gets neutralized when savings in repairing cost is taken into account during an average lifespan of building. Besides this, improved capacity concepts in columns show phenomenal savings in annual repairing cost and these net savings can easily be utilized for paying premium of insurance policies for risk posed by stronger and rare earthquakes.

5. Conclusions

Seismic risk assessment of RC frames is done with the help of fragility curves, hazard survival curves, plotted based on results of incremental dynamic analysis of 20 time history records on IDARC platform. The expected annual loss of each frame is calculated which is based on probabilistic financial risk assessment methodology for buildings in NZ. The performance of three frames analyzed with capacity design concepts taking into account shear capacity, flexural capacity and contribution from floor reinforcement to beams. Under varied scaling of peak ground accelerations of 20 earthquake ground motions, the associated financial risk of buildings with these three frames is compared with each other.

5.1. Frame-1

This frame incorporates provisions of capacity design for shear in beams, columns and beam–column joints adopting ductility provisions as per [9]. It is found that these frames are likely to incur about 15%, 40% and 75% loss in a DBE (475 years return period), MCE (2475 years return period) and (5000 years return period) respectively. The expected annual loss (EAL) of this frame is found to be very high; in the order of $3377, per $1 million asset value of building. The frame sustain majority of losses at 0.01 return period earthquake events with minor damages with and/or no repairs. The probable reason of such damages in these frames may be due to the lack of flexural capacity to sustain cyclic loadings.

5.2. Frame-2

In addition to Frame-1 this frame includes capacity design concept of columns in flexure as per [1]. These frames incur about 5%, 25% and 38%

Table 8
Comparative cost analysis of frames.

Frame type	Base cost	% increase in base cost	Annual repairing cost	Total repairing cost (50 years)	Total repairing cost as % of base cost	Net savings 50 years
Frame-1	1.0 million	–	3377	168,850	16.88%	(−)16.88%
Frame-2	1.04 million	4%	1172	58,600	5.6%	(−)1.6%
Frame-3	1.12 million	12%	963	48,150	4.3%	(+)7.7%

loss in a DBE (475 years return period), MCE (2475 years return period) and (5000 years return period) respectively, which is relatively lesser from Frame-1. The expected annual loss (EAL) for such frames is in the order of $1172, per $1 million asset value of building, which is almost one-third of similar building detailed as per Frame-1. These frames share their majority of losses at earthquake events of annual probability 0.001 and 0.0001 with minor to moderate damages.

5.3. Frame-3

These frames are designed to incorporate contribution of floor reinforcement to beams (clause 21.6.2.2, [1]) in addition to Frame-2. These frames incur about 2%, 18% and 29% loss in a DBE (475 years return period), MCE (2475 years return period) and (5000 years return period) respectively, which is less than 50% losses incurred in Frame-1 at MCE and 2*MCE levels. The expected annual loss (EAL) for such frames is $963, per $1 million asset value of building, which is almost one-fourth of similar building detailed as per Frame-1. This frame sustain majority of its losses at earthquake events of annual probability 0.001 with minor damages without inducing much repair costs.

In this study, it can be concluded that all frames are likely to partially or completely collapse in large magnitude earthquakes (i.e. annual probability >0.0001), however financial risk will be minimal due to their low probability of occurrence. On the other hand, smaller and frequently occurring earthquakes will pose a big effect on financial risk of structures. Apart from this, it is observed that around 50–70% of annual financial risk is contributed by earthquakes with return period between 100–1000 years. Also, more than 80% contribution of annual financial risk of Frame-1 buildings at earthquakes with return period between 10–100 years reduced to 70% for Frame-2 and Frame-3 buildings and remaining risks shared by earthquakes with higher return period.

It has been seen that, smaller and more frequent events pose a small risk to buildings hence owners are prepared to bear repair costs by themselves. Whereas, for stronger earthquakes as can be seen in this study the repairing cost may exceed above 80% which necessitates replacement of building and proves to be uneconomical, hence owners would like to pass this risk to insurers. Therefore, it is recommended that frames designed with capacity design concepts in shear and flexure shall be insured for stronger earthquakes with higher return periods only which in turn give rise to low insurance premiums. Results of present work shall be valid only for structures which incorporate frames considered in present work.

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