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Seismic risk assessment of existing RC structures using fragility-based approach

Cho Wai Phyo Kyaw* and Khin Aye Mon

Department of Civil Engineering, Yangon Technological University, **Myanmar**

*Corresponding Author: chowaiphyokyaw@gmail.com

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Abstract: This study investigates the seismic performance of two groups of existing reinforced concrete (RC) buildings: those designed and constructed according to older standards (pre-code RC buildings) and those designed and constructed in accordance with current seismic code requirements (moderate-code RC buildings). Recognizing the potential seismic vulnerability of these structures, this research aims to develop fragility curves to probabilistically assess their seismic performance. Non-linear time history analysis (NTHA) and incremental dynamic analysis (IDA) are employed, considering inter-story drift ratios (%ISDR) as key engineering demand parameters. These parameters are employed to link structural response to ground motion intensities (PGA) across various hazard levels, including Service Level Earthquake (SLE), Design Basic Earthquake (DBE), and Maximum Considered Earthquake (MCE). Eleven sets of ground motions, selected from the PEER database and matched to the Yangon target response spectrum, are used to simulate seismic loading. A representative 12-story RC frame with two plan aspect ratios is analyzed, considering material and geometric non-linearities. Five performance limit states (Fully operational, Immediate Occupancy, Damage Control, Life Safety, and Collapse Prevention) are defined based on FEMA 356. The developed fragility curves provide valuable insights into the seismic vulnerability of existing RC structures, informing the development of effective seismic risk-mitigating strategies and enhancing the resilience of urban areas.

Keywords: Non-linear time history analysis; Incremental dynamic analysis; Percent inter-story drift ratios; Peak ground acceleration; Fragility curves

1. Introduction

The older buildings in seismically active regions were constructed without adherence to modern seismic design codes. These buildings lack ductile detailing and do not follow capacity design principles, making them particularly vulnerable during major seismic events. Consequently, their structural elements are prone to brittle failure, which increases the risk of irreparable damage or even the collapse of the entire structure. Therefore, the seismic risk assessment of RC building stocks in earthquake-prone regions is crucial for post-earthquake inspections aimed at ensuring public safety. Quantifying the damage caused by these events mitigates risks to life and property by understanding building response and fragility. Conducting a probabilistic seismic damage analysis is to develop an analytical model capable of accurately predicting the inelastic response to seismic loading. This is particularly crucial for older reinforced concrete (RC) structures, where the lack of ductile detailing and capacity design increases the likelihood of shear and bond failures. To quantify the seismic sequence effects on RC structures, many researchers have performed many studies on the effect of seismic sequence on the multiple-degrees-of-freedom structures [1], [2], [3], [4].

Probabilistic seismic hazard assessments and seismotectonic investigations indicate that Myanmar lies in an earthquake-prone region with several active faults, including the Sagaing fault, one of the country's most significant and active faults. Yangon, the most populous city in Myanmar, is situated approximately 40 km west of the Sagaing fault [4], [5], [6]. According to the information of YCDC, the study area includes a significant amount of the existing inventory that was designed and constructed in accordance with different building codes. Moreover, the old buildings in the Yangon region that were not designed to resist seismic loads. Hence, the buildings inventory was categorized into two categories based on their construction date, namely before 2000 (pre-code) and after 2000 (moderate-code) [7]. The moderate code structures have adequate structural capacity in terms of strength and ductility since they are designed according to modern design codes. The pre-code buildings were only designed to resist gravity and wind loads, which often lack seismic design provisions, leading to potential structural deficiencies [7], [8], [9].

Non-linear structural analysis and probabilistic methodologies, as outlined in FEMA guidelines [9], [10], provide a framework for accurate and reliable seismic assessments, facilitating informed rehabilitation decisions. Seismic risk assessments are particularly vital to estimate the probability of building damage, evaluate the feasibility of continued occupancy, and determine the need for repairs following an earthquake. Recent studies have explored building vulnerability through nonlinear static analysis; however, this study focuses on seismic risk assessment using non-linear time history analysis (NTHA) for pre-code and moderate-code buildings [11].

The main aim of this study is to evaluate the seismic performance of pre-code and moderate-code RC building frames and to develop fragility curves that correlate inter-story drift ratios (ISDR) with peak ground accelerations (PGA). By integrating structural performance and seismic hazard levels, this approach offers a clearer and more reliable understanding of how RC structures behave under earthquake conditions. The findings enhance seismic risk assessments, support informed decisions for building design and retrofitting, and contribute to safer, more resilient structures. Beyond predicting structural responses, this methodology enables precise damage probability assessments across varying hazard levels, advancing the field of earthquake engineering and fostering resilient infrastructure development.

2. Material and methods

The methodology of this study is divided into four key parts. First, code-based designs are developed for all building frames. Next, the seismic performance levels of the frames are evaluated through non-linear time history analysis (NTHA). This is followed by conducting Incremental Dynamic Analysis (IDA) for both pre-code and moderate-code buildings to predict the probability of damage. This study analyzes the seismic performance of two groups of existing reinforced concrete (RC) buildings: those designed and constructed according to older standards (pre-code RC buildings) and those designed and constructed in accordance with current seismic code requirements (moderate-code). A representative 12-story residential RC frame with two plan aspect ratios of 2 and 1.6, considering material properties and structural configurations based on construction practices documented by the Yangon City Development Committee (YCDC). Nonlinear time history analysis (NTHA) and incremental dynamic analysis (IDA) are performed using eleven ground motions, with magnitudes ranging from 6.0 to 8.5 on the Richter scale.

Finally, fragility curves are developed in the later stages, considering five performance limit states as suggested by FEMA 356: Operational (OP), Immediate Occupancy (IO), Damage Control (DC), Life Safety (LS), and Collapse Prevention (CP). These states correspond to maximum inter-story drift ratios of 0.5%, 1.0%, 1.5%, 2.0%, and 2.5%, respectively. The fragility curves are used to assess the buildings under various earthquake hazard levels, including the target Yangon service level earthquake, design basic earthquake (DBE) level, and maximum considered earthquake (MCE) level. The level of damage is defined by an engineering demand parameter (EDP), with the maximum percentage of inter-story drift ratio (%ISDR) commonly used as an appropriate indicator [12], [13], [14]. This study does not rely on a single definitive method or strategy, as uncertainty exists at each stage, including ground motion characteristics, analytical modeling, materials, and limit states. The implementation process, outlining the various steps, is shown in Figure 1.

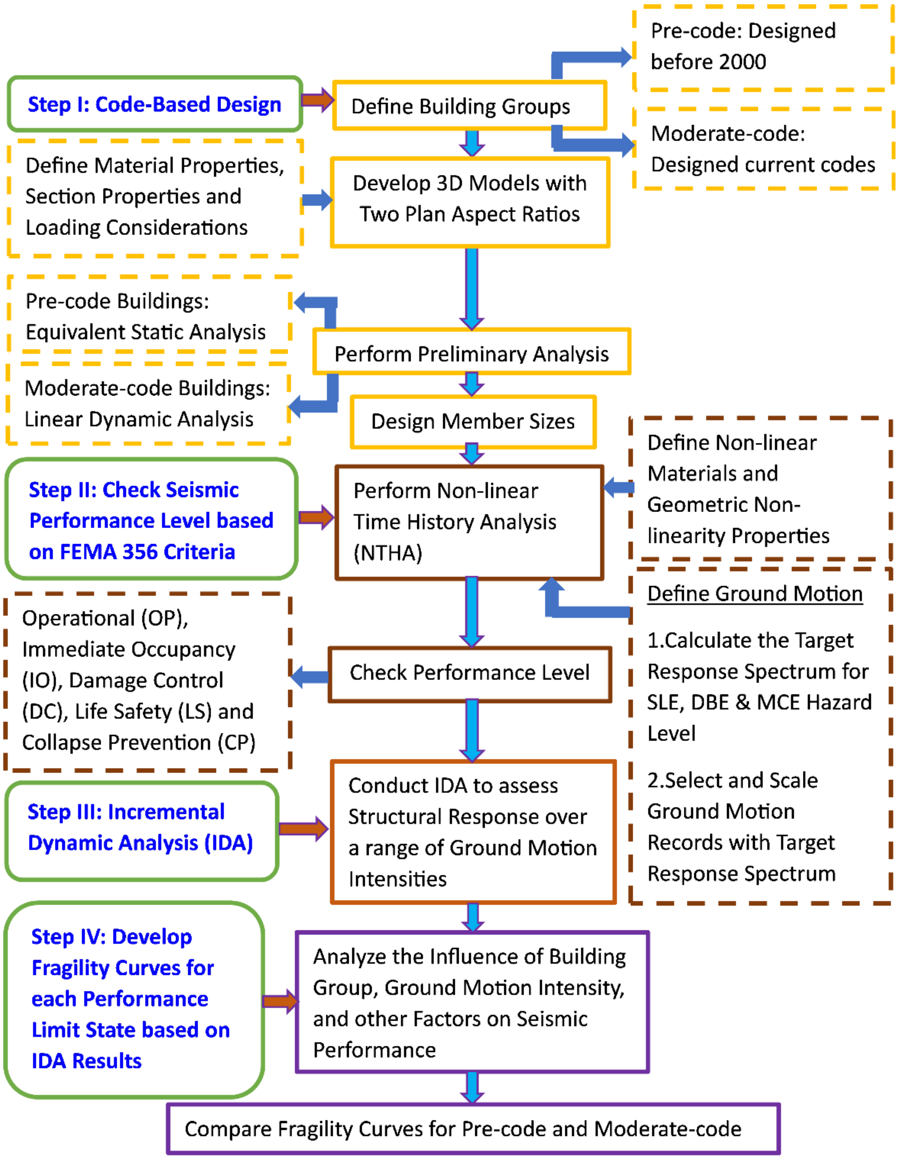
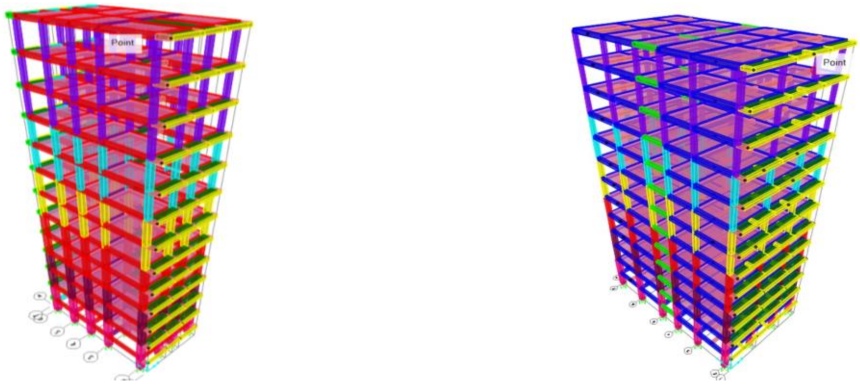


Figure 1. Implementation process for developing fragility curves

2.1 Code-based design

This study examines representative typical 12-story residential RC frames. The typical story height of these buildings is 10 feet, and they are rectangular in shape. Model 1 has dimensions of 49 feet in length and 25 feet in width, while Model 2 measures 70 feet in length and 42 feet in width. The site soil profile is classified as Site Class D (stiff soil). Seismic parameters, structural system selection, loading considerations, and load combinations for the case study buildings are based on the MNBC 2020 guidelines [7]. Figure 2 shows the 3D models of the reference building frames.



(a) (b)

Figure 2. 3D view of case study building frames; Figure (a) Model 1 and Figure (b) Model 2

Three-dimensional (3D) finite element models of the buildings were developed using integrated solution for structural analysis and design (SAP2000, Standalone license, version 20). The two pre-code buildings are specifically designed for this study, following the building codes in effect at the time of construction. These pre-code buildings are designed based on gravity loads only, with wind loads as the only lateral loads considered, reflecting the real conditions before 2000. The parameters required to define wind and seismic loads for both pre-code and moderate-code building frames are shown in Table 1.

Table 1. Seismic and wind parameters for pre-code and moderate code

Pre-code building (Consider gravity and wind load)	**Moderate-code building (Consider seismic loading)**
Windward coefficients = 0.8	Seismic importance factor = 1.25
Leeward coefficients = 0.5	Response modification factor = 8
Basic wind speed = 100 mph	System overstrength = 3
Method used = Diaphragm Method	Deflection amplification = 4.5
Pre-code building (Consider gravity and wind load)	**Moderate-code building (Consider seismic loading)**
Exposure Type = B	0.2 sec spectral acceleration = 0.7
Wind Important factor = 1	1 sec spectral acceleration = 0.2
Ordinary moment resisting frame system	Long-period transition periods = 6 sec
	C_t, x = 0.016, 0.9

The permanent loads used in the design of pre-code and moderate-code buildings include superimposed dead load (20 psf) and the self-weight. The live load (20 psf) except for staircases and corriders (40 psf). To accurately represent the pre-code structures, the material properties that were utilized at the time of construction were considered. The material properties for pre-code and moderate code design are shown in Table 2. A total of 15 design load combinations for pre-code building frames and 31 design load combinations for moderate code building frames are shown in Table 3.

Table 2. Material properties for pre-code and moderate code design

Case study	**Nominal material properties**	**Expected material properties**
Pre-code buildings	$f_c' = 2.5$ ksi $f_y = 40$ ksi	$f_c' = 1.5 \times 2.5$ ksi$= 3.75$ ksi $f_y = 1.25 \times 40$ ksi$= 50$ ksi
Moderate code buildings	$f_c' = 4$ ksi $f_y = 50$ ksi	$f_c' = 1.5 \times 4$ ksi$= 6$ ksi $f_y = 1.25 \times 50$ ksi$= 62.5$ ksi

Table 3. Selected load combinations for pre-code and moderate code design

No.	Load combinations (pre-code design)	Load combinations (moderate code design)
1	1.4 DL	1.4 DL
2	1.2 DL + 1.6 LL	1.2 DL + 1.6 LL
3	1.2 DL + 1 LL	1.2 DL + 1 LL
4	0.9 DL ± 1.6 WX	0.9 DL ± 1.6 WX
5	0.9 DL ± 1.6 WY	0.9 DL ± 1.6 WY
6	1.2 DL ± 1.6 WX + 1 LL	1.2 DL ± 1.6 WX + 1 LL
7	1.2 DL ± 1.6 WY + 1 LL	1.2 DL ± 1.6 WY + 1 LL
8	1.2 DL ± 0.8 WX	0.9 DL ± 1 EQX
9	1.2 DL ± 0.8 WY	0.9 DL ± 1 EQY
10		1.2 DL ± 1 EQX + 1 LL
11		1.2 DL ± 1 EQY + 1 LL
12		1.2 DL ± 0.8 WX
13		1.2 DL ± 0.8 WY
14		0.9 DL ± 1 SPECX
15		0.9 DL ± 1 SPECY
16		1.2 DL ± 1 SPECX + 1 LL
17		1.2 DL ± 1 SPECY + 1 LL

The design is carried out carefully for each building to obtain the optimum cross sections for different structural elements for pre-code buildings. Moreover, the moderate-code buildings are analyzed and designed by linear dynamic analysis. An iterative design process was carried out using SAP2000 software under the above load combinations. And then, the structural stability of the case study RC building frames is checked and designed for moderate code in accordance with ASCE 7-16. Table 4 summarizes the design results of beam and column sizes for pre-code and moderate-code buildings shown in the following.

Table 4. Member sizes for Model 1 and Model 2 (pre-code and moderate design)

Case study	Code	Type (column)	Floor level	Sizes (in x in)	Type (beam)	Sizes (in x in)
Model 1	Pre-code	C1	Base to 2F	18 x 18	FB1	9 x 9
		C2	2F to 5F	16 x 16	FB2	9 x 12
		C3	5F to 8F	14 x 14	FB3	9 x 14
		C4	8F to 11F	12 x 12	FB4	9 x 16
		C5	11F to 12F	9 x 9	FB5	10 x 18
	Moderate-code	C1	Base to 1F	24 x 24	FB1	12 x 12
		C2	1F to 3F	22 x 22	FB2	12 x 14
		C3	3F to 5F	20 x 20	FB3	12 x 18
		C4	5F to 7F	18 x 18	FB4	12 x 20
		C5	7F to 9F	16 x 16	FB5	14 x 16
		C6	9F to 12F	14 x 14	FB6	14 x 18
Model 2	Pre-code	C1	Base to 2F	18 x 18	FB1	9 x 9
		C2	2F to 5F	16 x 16	FB2	12 x 12
		C3	5F to 8F	14 x 14	FB3	12 x 14
		C4	8F to 11F	12 x 12	FB4	12 x 18
		C5	11F to 12F	9 x 9		
	Moderate-code	C1	Base to 2F	26 x 26	FB1	12 x 12
		C2	2F to 5F	24 x 24	FB2	12 x 14
		C3	5F to 8F	22 x 22	FB3	14 x 16
		C4	8F to 10F	20 x 20	FB4	14 x 18
		C5	10F to 11F	18 x 18		
		C6	11F to 12F	16 x 16		

2.2 Selection of ground motions

To perform Non-linear Time History Analysis (NTHA), a suite of minimum 11 ground motions is selected for each Yangon target response spectrum at the site, which are loaded from the Pacific Earthquake Engineering Research Centre's (PEER) Ground Motion Database by selecting fault type (strike slip), source distance, and magnitude that are consistent with the saturation that should be considered for the expectation of an earthquake in Yangon. The dominance of seismic loads depends on the target response spectrum at the site. It is obtained from probabilistic seismic hazard analysis of the Yangon region. In this study, site coefficients and target spectral response acceleration parameters for Service Level Earthquake (SLE), Design Basic Earthquake (DBE), and Maximum Considered Earthquake (MCE) hazard levels are estimated from MNBC 2020 [7]. These are shown in Figure 4, where the MCE response spectrum is estimated by multiplying the design response spectrum by 1.5. The SLE response spectrum is estimated by 0.5 times the level of ground shaking of the design response spectrum. After that, 11 ground motions were selected from the PEER ground motion database and scaled using SeismoMatch software according to the Yangon target response spectrum at the site, as shown in Figure 3-5 and Table 5.

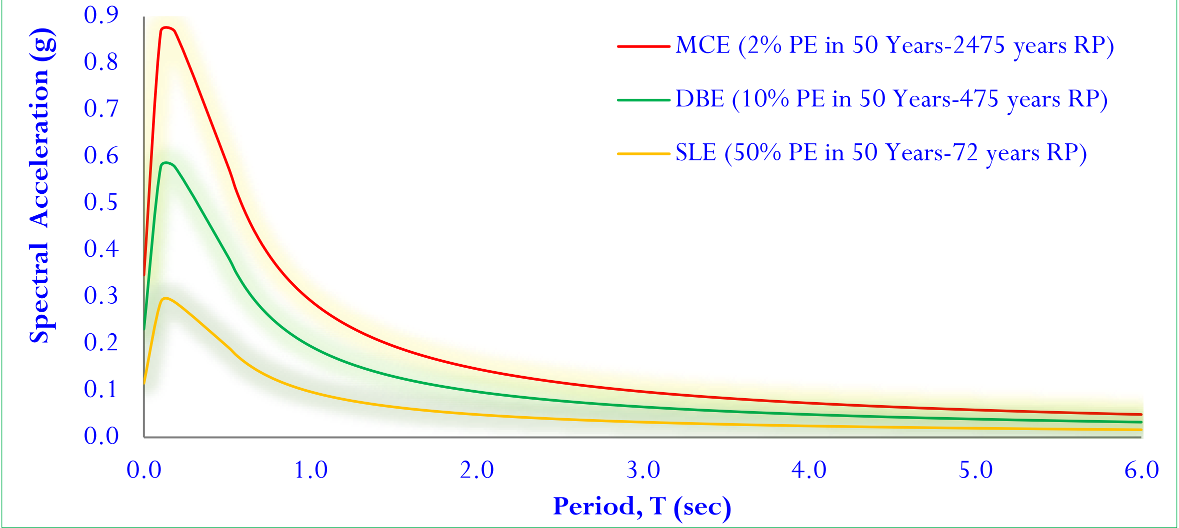


Figure 3. Estimated target response spectrum for Yangon

Table 5. Selected ground motions

Event	Magnitude	Duration (sec)	R_{JB} (km)	R_{rup} (km)	$V_{s,30}$ (m/sec)
Morgan hill	6.19	15.3	39.08	39.08	288.6
Chi-Chi Taiwan	6.2	21.5	87.07	87.07	292.61
Victoria, Mexico	6.33	19	18.53	18.53	242.05
Bigbear	6.46	12.1	39.52	39.52	359
Imperial valley	6.53	10.8	30.33	30.33	316.64
Superstition hills	6.54	14.3	17.03	17.03	208.71
Kobe Japan	6.9	19.4	11.34	11.34	256
Hector mine	7.13	23.1	64.08	64.08	324.62
Duzce, Turkey	7.14	9	12.02	12.02	293.57
Landers	7.28	18.9	23.62	23.62	353.63
Denali Alaska	7.9	23.1	42.99	42.99	341.56

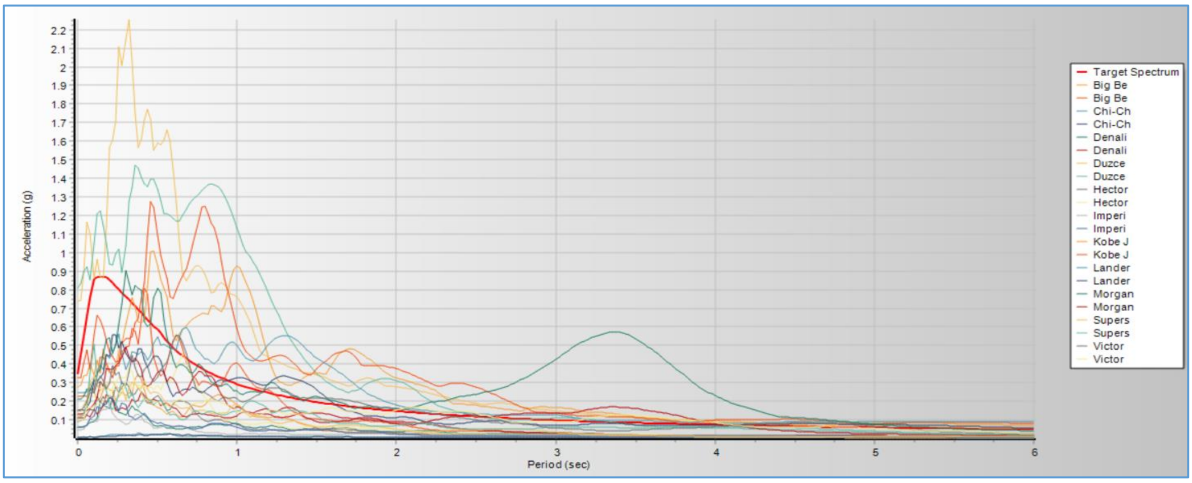


Figure 4. Original spectral response acceleration for 11 suites of ground motions

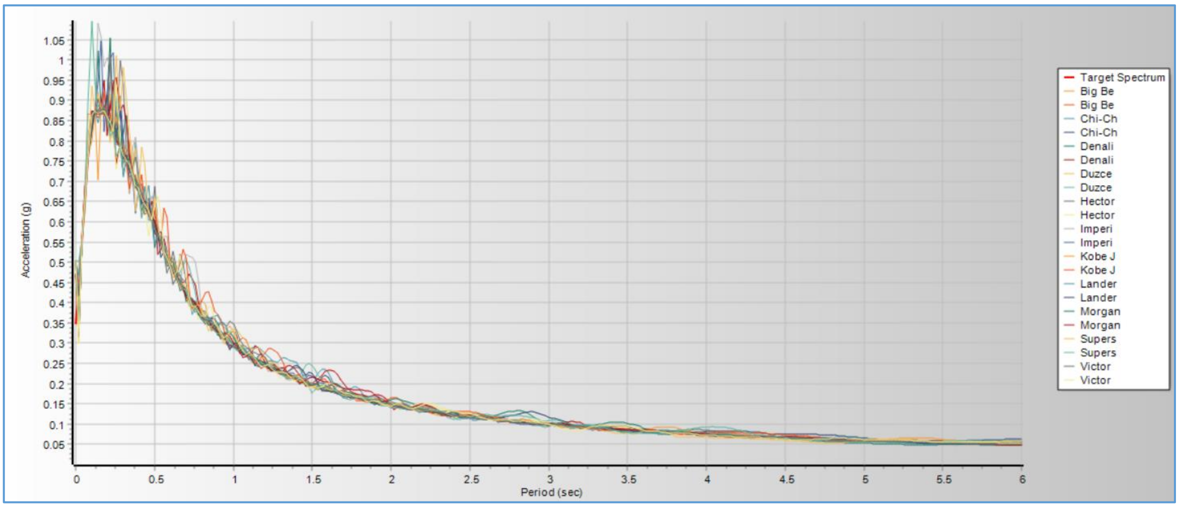


Figure 5. Matched spectral response acceleration with Yangon target response spectrum

2.3 Non-linear time history analysis (NTHA)

To check the structural performance level, non-linear time history analysis (NTHA) is carried out. The time history analysis determines the response of a structure due to ground motion intensities (Hashemi et al., n.d.; Singh, 2021). Modelling for NTHA requires the determination of the non-linear properties of each material, hinge properties, and component in the structure, quantified by strength and deformation capacities, which depend on the modeling assumptions according to FEMA 356 and ASCE 41-13 [15]. In this study, geometrical non-linearities (P-Delta effect) and material non-linearities (cracking in concrete and yielding in steel) are considered. The effective stiffness properties of concrete elements shall consider the effects of cracked sections. Non-linear hinge properties for the columns are modeled as (fiber P-M2-M3 hinge) an elastic element in the middle and two fiber section beam-column elements in the plastic hinge regions. While the beams are modelled as (Moment M3 hinge) is assigned at the ends of the beams that is composed of two plastic hinge zones (fiber section) at the ends of the element and a linear elastic region in the middle of the element. Default non-linear expected concrete compressive strengths and reinforcing steels are determined by multiplying the lower bound value by an appropriate factor selected from Table 10-1 (ASCE 41-13). The expected material properties of concrete and reinforcing steel are shown in the Table 2.

In order to measure the performance of the case study building frames against seismic loads, the inter-story drifts are used to observe the critical damages that will lead to structural collapse. Then, the maximum inter-story drift ratio is calculated by dividing the maximum inter-story drift with the story height of the building. Based on each ground motion, NTHA is performed by using SAP2000 software. And then %ISDR is monitored and analyzed as crucial engineering demand parameters under different hazard levels. The global structural performance level depends on the mean value of structural response under 11 sets of earthquake ground motions, according to ASCE 41-13 [15].

2.4 Incremental dynamic analysis (IDA)

Incremental dynamic analysis (IDA) is applied to evaluate the expected structural response under different ground motion intensities. This analysis includes executing multiple non-linear inelastic response history analyses of a structural model under a suite of selected ground motion records: each is scaled to Yangon target response spectrum. A set of wisely selected ground motion records to outfit the hazard spectra of the study area helps to provide a precise evaluation of the seismic performance of structures. The scaling levels are properly selected to force the structure through the entire range of behavior, from elastic to inelastic and lastly to global dynamic instability, where the structure experiences collapse. The IDA gives more realistic results about the performance of a particular type of structure under seismic excitations [2], [16], [17].

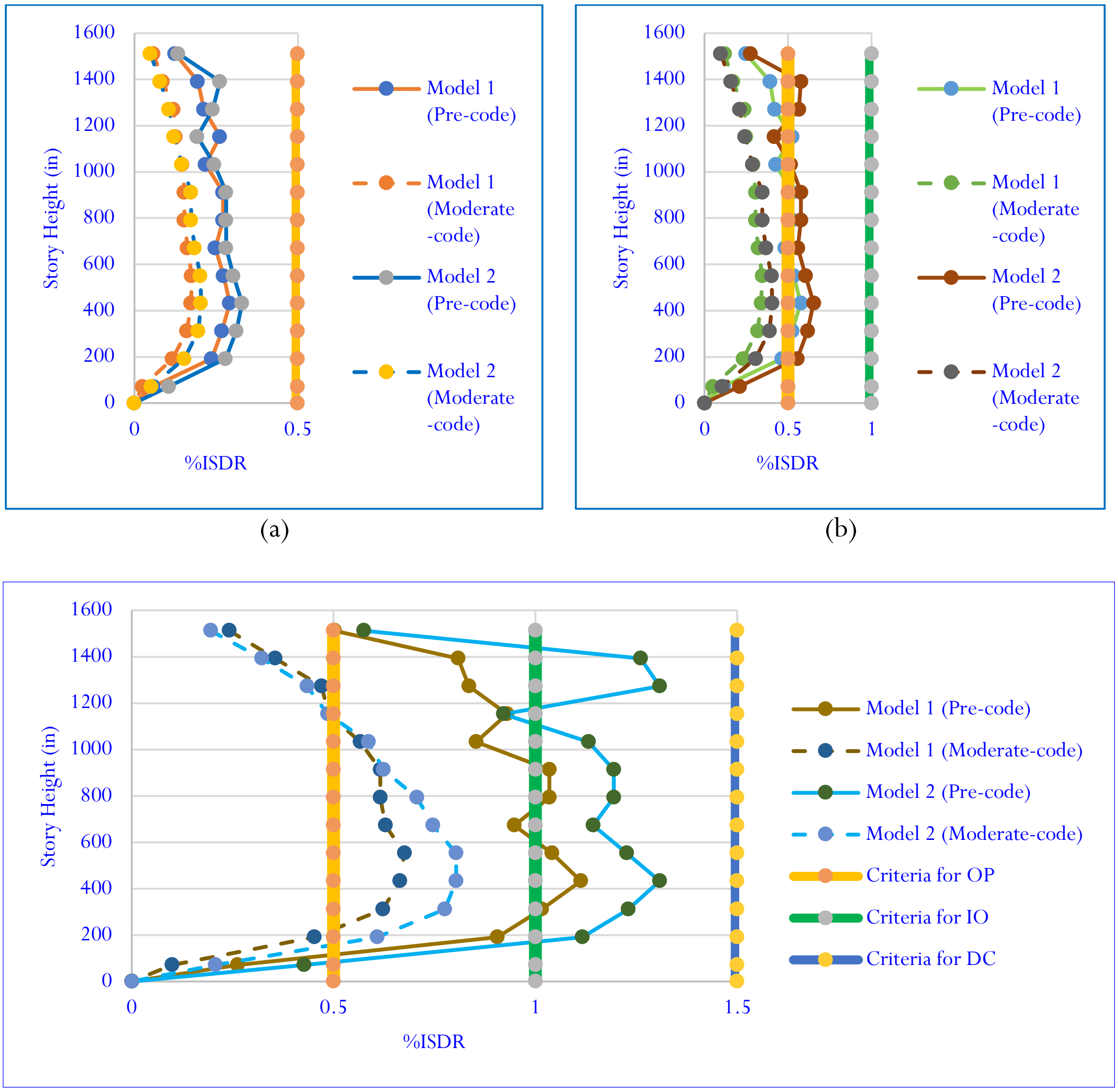
Numerous seismic parameters are responsible for developing fragility curves since the peak ground acceleration (PGA) parameter of the scaled time history spectral response acceleration and damage measure (%ISDR) was used in this study. Finally, the fragility curves are used analytically by using IDA to predict the risk of the seismic effect on the case study building frames. The two main parameters are needed to develop the fragility curves: mean (μ) and standard deviation (σ). Many equations were used to develop fragility curves [1], [16], [18], [19] ; in this study, the following equation [14] is used, where ∅ is the standard normal cumulative distribution function, μ and σ are the mean and standard deviation of logarithm PGA, and D is the damage state:

$$P[\mathrm{D/PGA}] = \emptyset\left[\frac{\ln(\mathrm{PGA}) - \mu}{\sigma}\right] \tag{1}$$

Based on the results of the fragility analysis, the collapse margin ratios are determined to show that the existing building has the possibility to reach fully half threatening damages was 50% at the relevant seismic intensity. In this study, the peak ground acceleration is used as an earthquake intensity measurement. The usage of collapse margin ratio as a seismic indicator can an important tool in the seismic assessment of the structures which assure the results generated from the IDA [14]. This indicator is characterizing the collapse safety of the structure by integrating the median spectral acceleration and MCE spectral acceleration in the fundamental period of the structure related to the site classifications. The collapse margin ratio is the ratio of the earthquake intensity corresponding to the 50 % probability of structural collapse and MCE intensity in PGA [14].

3. Results and discussion

Firstly, to assess the building frame performance, the non-linear time history analysis was performed with 11 suites of each ground motion acceleration by using SAP2000 software. And then, %ISDR was monitored and analyzed with ground motion intensities (PGA). The global structural performance level was investigated on the mean value of structural response (% ISDR) and peak ground acceleration (PGA) under 11 sets of earthquake excitations.



(a) (b)

(c)

Figure 6. Mean value of %ISDR for model 1 and 2; Figure (a) SLE hazard level, Figure (b) DBE hazard level, Figure (c) MCE hazard level

Figure 6 shows the mean value of %ISDR for models 1 (pre-code and moderate code) and 2 (pre-code and moderate code) at the SLE, DBE, and MCE hazards levels at PGA = 0.1 g, 0.2 g, and 0.3 g, respectively. According to the analysis results from Figure 6, under Yangon SLE hazard level at PGA = 0.1g, the mean values and maximum %ISDR of all models were less than (0.5%) criteria for OP level. It was found that all models were fully operational subjected to earthquake PGA = 0.1 g. Under DBE hazard level (PGA = 0.2g), moderate-code buildings remain fully operational, but pre-code buildings are expected to sustain minor damage to structural elements but remains safe to occupy and requires minimal repairs. Under MCE hazard level, moderate-code buildings are expected to require minor repairs, and pre-code buildings are expected to cause moderate damage to structural elements.

Secondly, to predict the probability of damages for the building frames, the incremental dynamic analysis was scaled until the point of structural collapse. A significant amount of time and effort were dedicated to conducting IDAs, which are performed for reference structures using the selected 11 far-field ground motions.

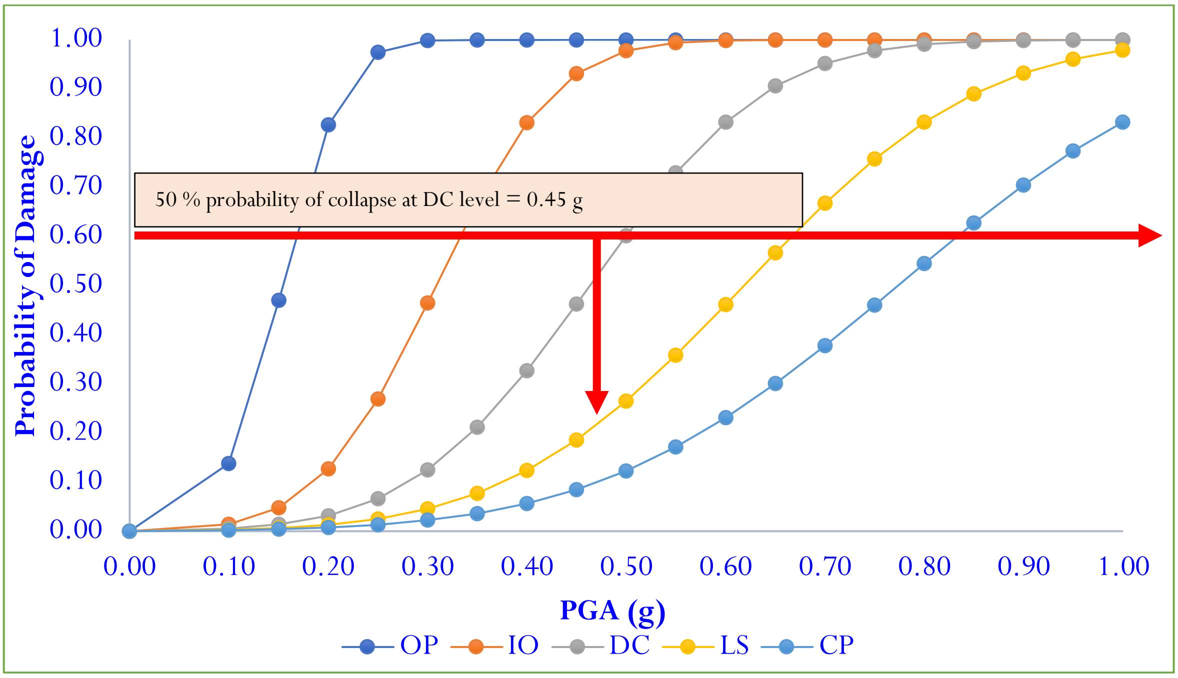


Figure 7. Fragility curve for model 1 (pre-code)

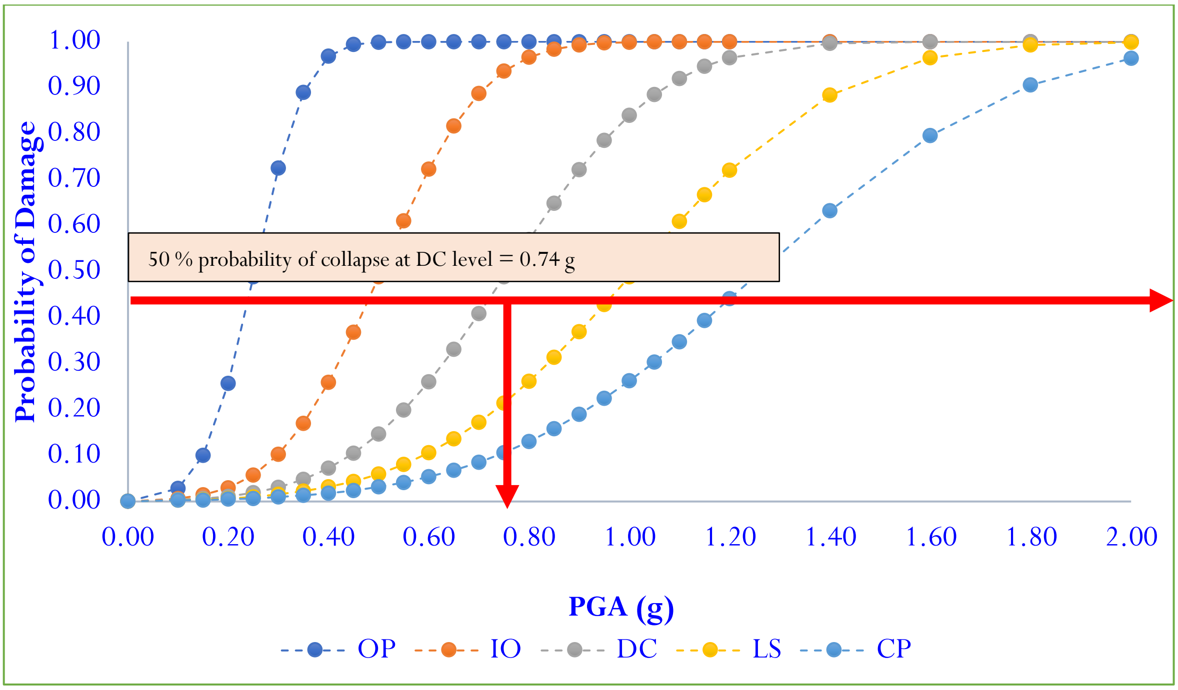


Figure 8. Fragility curve for model 1 (moderate-code)

For pre-code frame structures, a suite of 11 ground motion records was incrementally scaled from a PGA of 0.025 g to 1.2 g using a scaling factor of 0.025 g. For moderate-code frame structures, a suite of 11 ground motion records was incrementally scaled from a PGA of 0.025 g to 2 g using a scaling factor of 0.025 g. And then, the mean and standard deviation of ground motion intensities and structural response parameters in Equation 1 were used to develop fragility curves. Figure 7 and Figure 8 show fragility curves of model 1 (pre-code and moderate code).

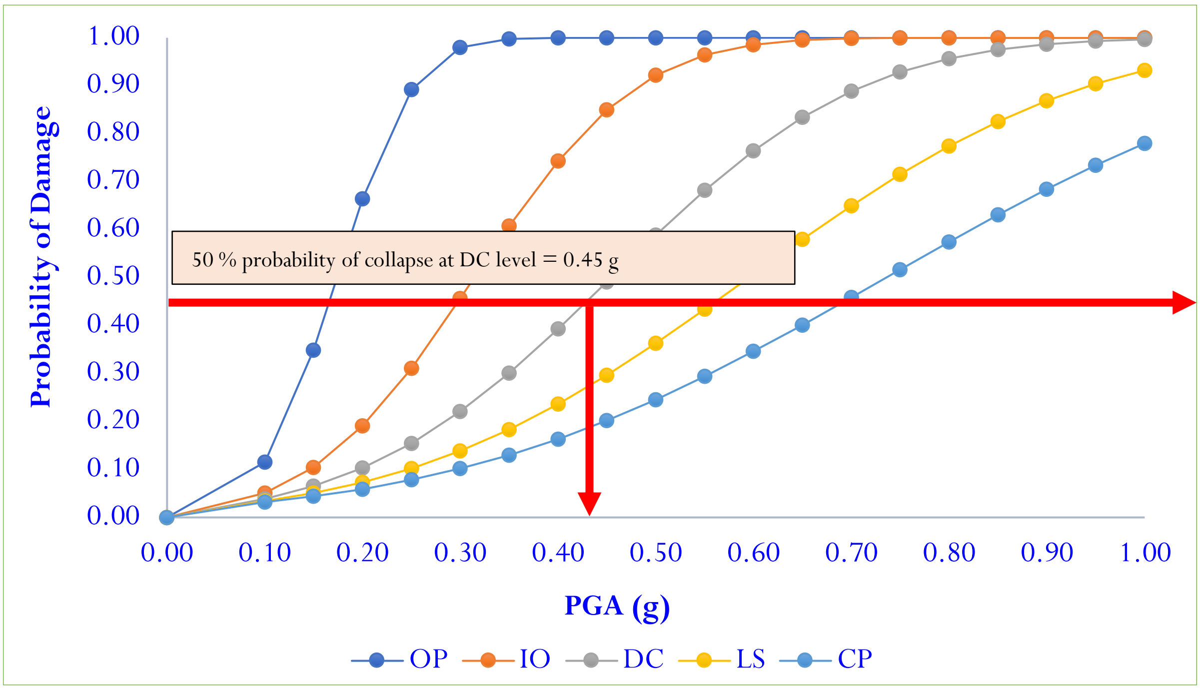


Figure 9. Fragility curve for model 2 (pre-code)

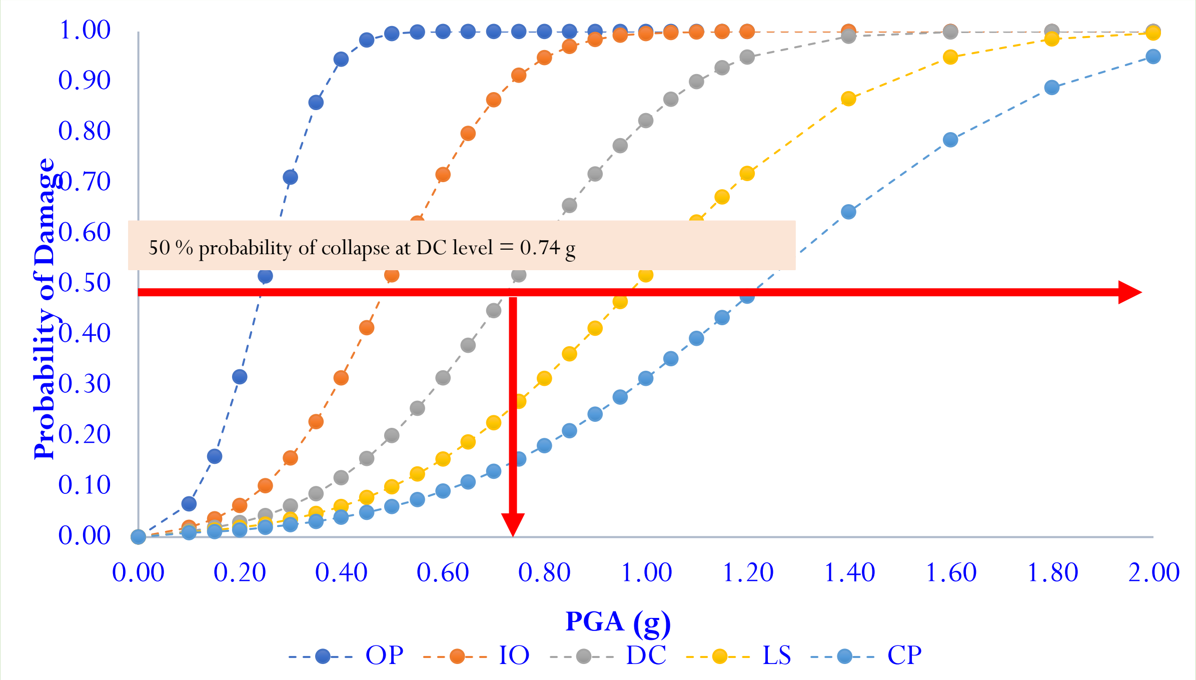


Figure 10. Fragility curve for model 2 (moderate-code)

The fragility estimation of simulated models is depicted in Figure 9 and Figure 10, which show fragility curves of model 2 (pre-code and moderate code). The results investigated that pre-code building frames will be subjected to a 50% probability of damages at the DC level under the high intensity and stronger earthquakes with PGA nearly 0.45 g. Thus, it is anticipated that the structure will suffer permanent deformation, noticeable cracking, and structural repair is needed. Although moderate code building frames will be subjected to a 10 to 15% probability of damages at the DC level under PGA nearly 0.45 g. Therefore, these structures will suffer minor damage but are safe to occupy. Moderate code building frames will suffer severe damage under the high, stronger PGA, approximately 0.74 g.

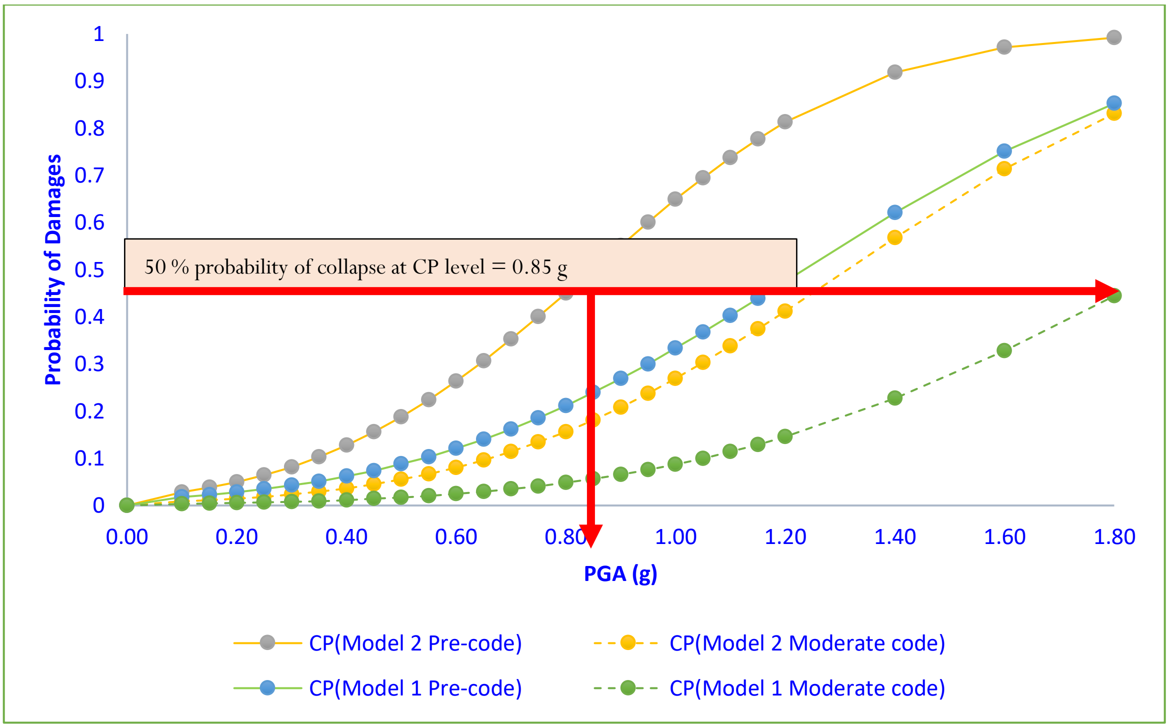


Figure 11. Comparison on fragility curves

Figure 11 shows comparison on the fragility curve analysis under pre-code and moderate code design, where pre-code building designs have a more than 50% higher probability of damages than moderate code building designs. Moderate-code buildings are typically designed to withstand seismic events with a probability of approximately 10% of experiencing damages over a design life of 50 years. This study suggests that Model 2 reaching the 50% probabilities of the CP performance levels under strong ground motions (PGA = 0.85 g) emphasizes the need for immediate retrofitting and strengthening measures. For buildings in seismic-prone areas, updating construction practices to comply with performance-based seismic design can significantly decrease the probability of damage and improve overall safety during and after an earthquake.

The simulation and analyses were performed using a non-linear platform. The model was designed based on ACI 318-14, and then non-linear time history analysis was carried out to evaluate the performance of existing structures under three sets of different ground motions (average PGA of 0.25 g) based on the Lebanese seismic zone [14]. According to the collapse fragility curves of relevant research studies, the values of the collapse margin ratio were 1.24 based on the results obtained from IDA and fragility assessments. According to the previous research by Kassem M. and Mohamed Nazri F. (Kassem et al., 2020), the existing building has a 50% probability of experiencing severe damage at a seismic intensity of 0.31 g.

In the current study, based on the results of the collapse fragility analysis, the reference case study pre-code building RC frames has a 50% probability of collapse at a seismic intensity of nearly 0.45 g related to the site classifications in Yangon. The average of MCE seismic intensity in PGA is 0.35 g under 11 suites of ground motions. The collapse margin ratio for pre-code building frames is 1.28 The moderate-code building frames have a 50% probability of collapse at a seismic intensity of nearly 0.74 g under 11 suites of ground motions. The collapse margin ratio for moderate-code building frames is 2.14. All of the results obtained from the current study are consistent with the results obtained from the relevant research results obtained from the reference study. Comparative analysis of pre-code and moderate-code frames revealed significant differences in seismic performance. Moderate-code buildings have higher collapse margin ratios, reflecting their superior resilience.

4. Conclusion

Based on the analytical analysis results of the fragility curve, the following conclusion has been drawn. Firstly, this paper examines the seismic performance of 12-story RC building frames for pre-code and moderate-code design under different hazard levels using SAP2000 software. Results show all models are operational under the Yangon SLE hazard level; moderate-code buildings remain operational under the DBE hazard level, and pre-code buildings cause light damage to structural elements. Under expected Yangon MCE hazard level, pre-code buildings caused moderate damage to structural elements. The existing structural frame system is inadequate to resist seismic loadings that could hit Yangon in the future. Therefore, pre-code buildings should be able to meet strengthening solution targets for an existing building frame system, such as shear walls, steel bracing systems, and other strengthening techniques.

Moreover, fragility curves were developed to predict the probability of damages under different PGAs. The incremental dynamic analysis for the four structural frames is compared based on the observations of the IDA and collapse margin ratios. The existing structural system is adequate to resist gravity and wind load assigned to the building. However, it is unable to resist any potential earthquake that will hit PGA of 0.45 g in the future. The analyses demonstrated that moderate-code buildings have better performances compared to the pre-code buildings because moderate-code buildings are able to resist the targeted performance of the Yangon MCE hazard level. The advantages from the creation of fragility curves by correlating engineering demand parameters with seismic intensities, offering probabilistic insights into potential damage levels. By leveraging science and technology, this paper underscores the potential for transformative impacts in seismic performance evaluation, supporting not only safer buildings but also resilient communities and sustainable development.

Author's declaration

Author contribution

Cho Wai Phyo Kyaw: Conceptualization, Methodology of the study, Simulation, Data analysis, Investigation, Software and Writing - original draft preparation, Reviewing and Editing. **Khin Aye Mon**: Conceptualization, Supervision, Investigation, Data curation, Reviewing and Editing.

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Conflict of interest

The authors state that there is no conflict of interest.

Ethical clearance

This research does not involve humans as subjects.

AI statements

The language structures used in this article were checked by using Grammarly and has been verified by an English Language expert. In addition, none of the sentences and figures in this article was AI tool-generated. All the data were obtained from the process of the study, and authors' and previous research review.

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Nomenclature

NTHA	Non-liner Time History Analysis
IDA	Incremental Dynamic Analysis
RP	Return Period
PE	Probability of Exceeded
SLE	Service Level Earthquake
DBE	Design Basic Earthquake
MCE	Maximum Considered Earthquake
OP	Fully Operational
IO	Immediate Occupancy
DC	Damage Control
LS	Life Safety
CP	Collapse Prevention
PGA	Peak Ground Acceleration
%ISDR	Percent Inter-story Drift Ratios