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ABSTRACT

Fragility curves are the tools for evaluating the damage of structural or non-structural components under different levels of ground excitation. It is also the necessary information for calculating the life-cycle cost of buildings during their design period. In this paper, development of fragility curves for buildings designed based on HK standard is presented. Two prototype models, which include three-storey and ten-storey reinforced concrete rigid frames, were considered. Incremental dynamic analysis was conducted under nine sets of past earthquake records with scaled peak ground accelerations. Maximum inter-storey drift ratio was selected as the damage indices and three performance levels were considered. Based on the numerical results, two fragility curves were developed to quantify the damage probability of RC buildings with different building heights.

1. INTRODUCTION

Hong Kong is a small place with high population, and thus numerous high-rise commercial and residential buildings were built to meet the demand. According to GB50011-2010, Hong Kong is under low-to-moderate seismically active group. On the other hand, most of the existing buildings in HK are designed without considering the effects of seismic actions. Under the attacks of earthquake, the structural performance of existing buildings becomes uncertain. Damage or collapse of buildings may eventually happen, resulting in severe social and economic impacts to the society. More investigations have to be conducted in order to quantify the risk of buildings being damage from future earthquakes.

For this purpose, fragility curve, which is defined as a conditional probability of a building having or exceeding a specific damage level for a given ground motion level, could be employed to quantify the probability of damage of structural or non-structural members. This information could help for loss estimation and retrofitting decisions after the earthquake. Fragility curves can be developed based on actual data or computational simulation. The former method relies on the observation data during past

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earthquake events. For example, Rossetto (2003) used historical records to constructed empirical fragility curves for European-type RC building. On the other hand, the latter method adopts incremental dynamic analysis (IDA), which was proposed by Vamvatsikos (2002), to predict the performance of building under earthquakes with scaled magnitude. Recently, this approach is widely adopted for performance-based earthquake engineering (PBEE) and seismic vulnerability assessment of buildings.

In past few decades, many researches have been conducted to develop fragility curves of buildings designed with various standards. Akka (2005) constructed displacement-based fragility curves for reinforced concrete (RC) buildings designed based on Iranian seismic design code. Rossetto (2005) developed displacement-based vulnerability curves for RC structures in Italy. Kircil (2006) produced fragility curves for mid-rise RC frame buildings in Istanbul. Majd (2012) constructed fragility curves for steel structures with X-braces in Iran. Wu (2012) evaluated the seismic performance of RC buildings designed based on Chinese code. Tsionis (2014) developed fragility for RC wall-frame buildings in Greece. Saruddin (2015) developed fragility curves for low- and medium-rise buildings in Malaysia.

In this study, fragility curves for two RC buildings designed based on Hong Kong code of practice are developed based on IDA using nine past earthquake records. Material and geometric non-linearity are considered in the analyses to simulate the failure patterns of buildings. The results can reflect the damage probability of the buildings under earthquakes, and so the expected loss and the corresponding rehabitation works can be planned earlier.

2. METHODOLOGY

Two reinforced concrete buildings were used to simulate the structural responses under different levels of earthquake. Details about the prototype models, method of analysis and data collection are presented in the following sections.

2.1 Prototype Models

Prototype Model 1 is a 3-storey RC rigid frame structure, while prototype Model 2 is a 10-storey RC rigid frame structure, as shown in Fig. 1. The storey height and bay width are 3.5 m and 6.0 m, respectively. The models were designed based on RC codes of practice in HK (2013) with the consideration of gravity loads. Super-imposed dead load and imposed load were equal to 15 kN/m and 12 kN/m, respectively. The cross-sectional dimensions of beams and columns and the corresponding reinforcement detail are summarised in Table 1. Concrete grade C40 with f_{cu} = 40 N/mm^2 and E_c = 25.1 kN/mm^2 and high-yield steel reinforcement with f_y = 500 N/mm^2 were used in member design. All columns were modelled as fixed supports at the base level. The fundamental periods of the prototype Model 1 and Model 2 are 0.71 s and 1.06 s, respectively.

Material non-linearity was represented using lumped plasticity model, which involves assignment of plastic hinges at the member ends. The backbone curves for plastic hinges in beams and columns were specified in according to Table 10-7 and Table 10-8 in ASCE41-14. Fig. 2 illustrates the definitions of performance points (A to

E) and limit states (IO, LS and CP) on the backbone curve of plastic hinge at beam ends. Similar definitions were adopted for plastic hinge at column ends, with the consideration of column axial force levels. Takeda hysteresis model was adopted to represent the cyclic behaviour of plastic hinges.

Table 1 Cross-section of beam and column

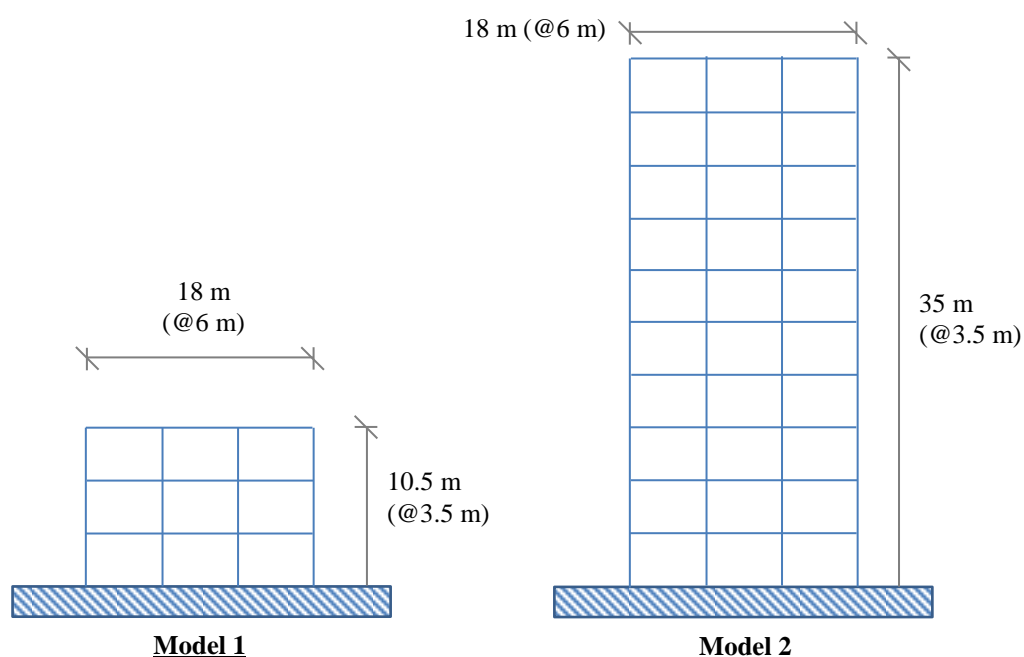|	Model 1		Model 2	
	Beam	Column	Beam	Column
Size	300x450	350x350	300x450	350x350
Reinforcement	3T20	4T20	4T20	6T32
Shear link	T12@250	T12@250	T12@250	T12@250

Fig. 1 Prototype models

2.2 Method of Analysis

Incremental dynamic analysis (IDA) is adopted to conduct seismic vulnerability assessment of buildings. This method involves the use of numerous non-linear time-history analyses (NLTHA) under a particular ground motion with increasing intensity measure (IM). The outcome is to produce IDA curve, which is a plot of IM versus damage measure (DM). In general, earthquake characteristics, such as peak ground acceleration (PGA), peak ground velocity, peak ground displacement, spectral acceleration, etc. could be adopted as the IM. In this study, PGA was chosen as the IM to represent the magnitude of earthquake. Nine sets of past earthquake records, where some were obtained from ATC-63 far field ground motion set, were used in NLTHA. Table 2 summarises the selected earthquake records. For each earthquake record, the accelerogram was scaled to PGA of 0.1g to 1.0g for evaluating the seismic behaviour of building. The NLTHA was conducted based on direct integration method

with HHT integration scheme using SAP2000 software.   Second order P-Δ effect was considered in the analyses.

On the other hand, inter-storey drift ratio (DR) was chosen as the engineering demand parameter (EDP) to quantify the performance levels of building.   This variable is widely adopted to describe the damages appeared in structural or non-structural elements such as partition walls and windows.   Other commonly used parameters include maximum top drift, inelastic deformation, ductility and damage index.   Three levels of damage or limit states, including Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP), are represented by drift ratios of 1%, 2% and 3%, respectively, with reference to the values given in Table C1-3 in FEMA 356 (2000).
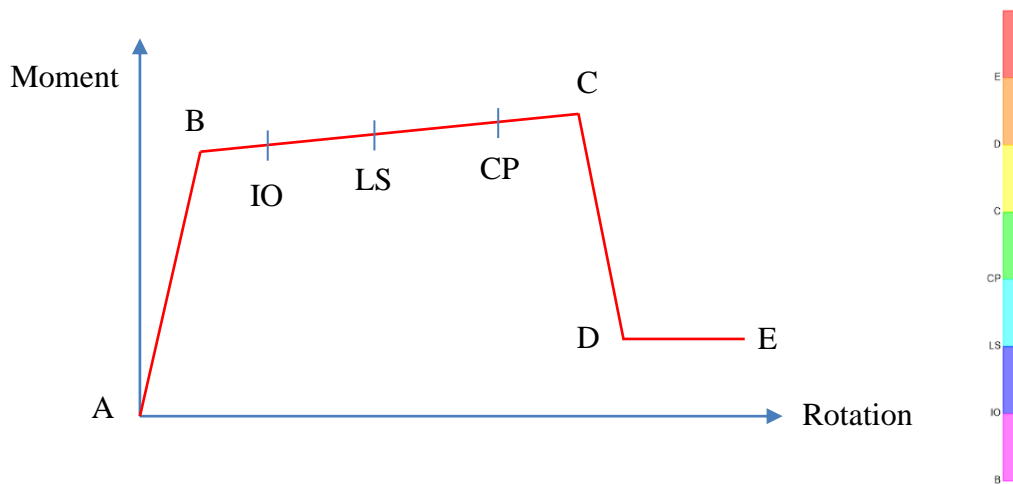

Fig. 2 Backbone curve of plastic hinge at beam ends

Table 2 Nine past earthquake records used in IDA

| No.	Name of earthquake	Year
1	Elcentro Earthquake	1940
2	Kobe Earthquake	1995
3	Chi-Chi Earthquake	1999
4	Northridge Earthquake	1994
5	Campano-Lucano Earthquake	1980
6	Imperial Valley Earthquake	1979
7	Kocaeli Earthquake	1999
8	San Fernando	1971
9	Loma Prieta	1989

3. IDA Curves and Fragility Curves

After numerous NLTHA, the overall performance of buildings under selected earthquakes can be represented using IDA curves, which is a plot between PGA and DR in this study.   Fig. 3a and 3b shows the IDA curves for Model 1 and 2, respectively. Each point on the IDA curves was obtained by a NLTHA under an earthquake record with a scaled PGA, and totally 180 analyses were conducted.   The maximum value of

DR was obtained by determining the DR at different time instants in the response histories.
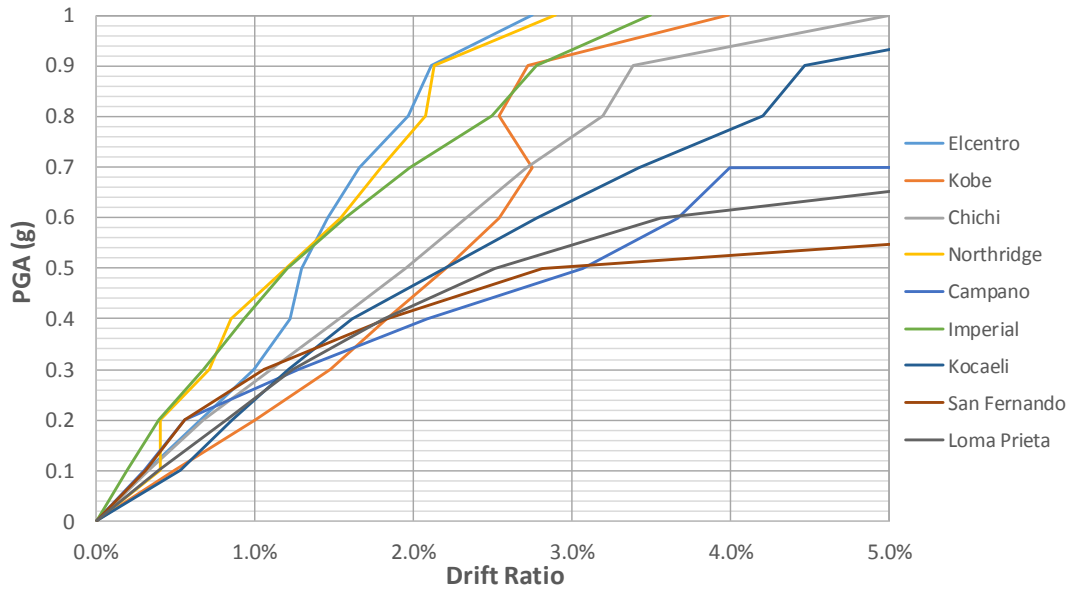
(a)
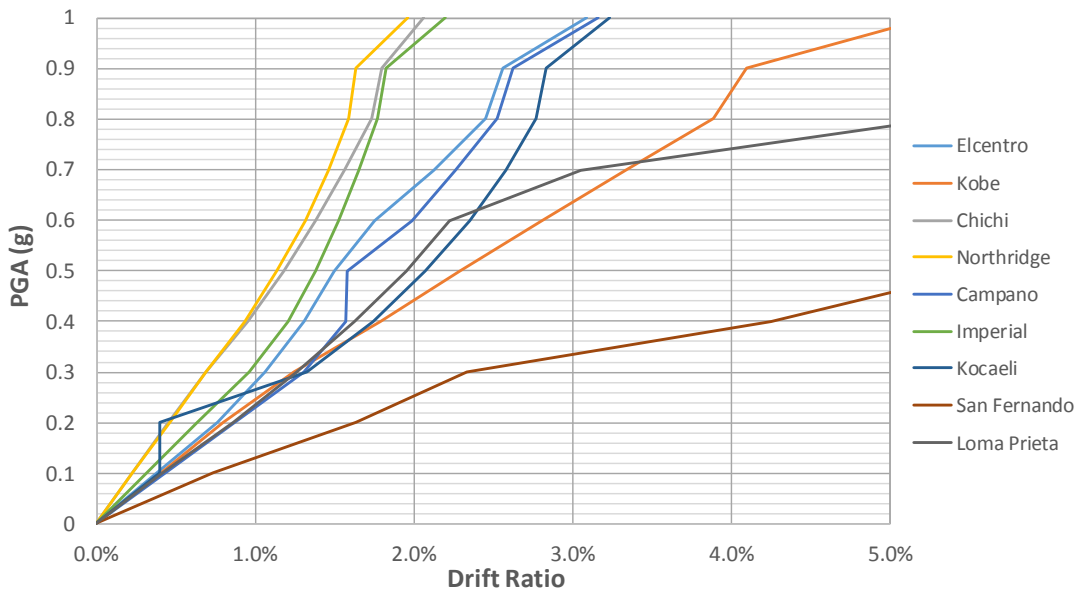
(b)

Fig. 3 IDA curves of (a) Model 1, and (b) Model 2

The distribution of damage (i.e. plastic hinge distribution) can be investigated through IDA. Fig. 4a and 4b show the patterns of plastic hinge formation and the corresponding performance levels after the attack of Kobe earthquake with PGA of 0.1g, 0.3g and 0.5g, respectively. For Model 1, plastic hinges were formed at the beam ends and the bases of column at PGA of 0.1g. The plastic hinges remained at performance point B (after yielding). At PGA of 0.3g, more plastic hinges were

observed at column ends and reached the LS state.   At higher PGA levels, most of the plastic hinges at the first storey reached the LS state.   Performance level beyond point C was not observed in the mentioned PGA levels.   For Model 2, no plastic hinge was found in the building at PGA of 0.1g.   At PGA of 0.3g, plastic hinges were formed at beam ends at storey 1 to 7.   No plastic hinge was found at column ends.   At PGA of 0.5g, plastic hinges formed at the base level.   Besides, most all the plastic hinges reached the LS state.
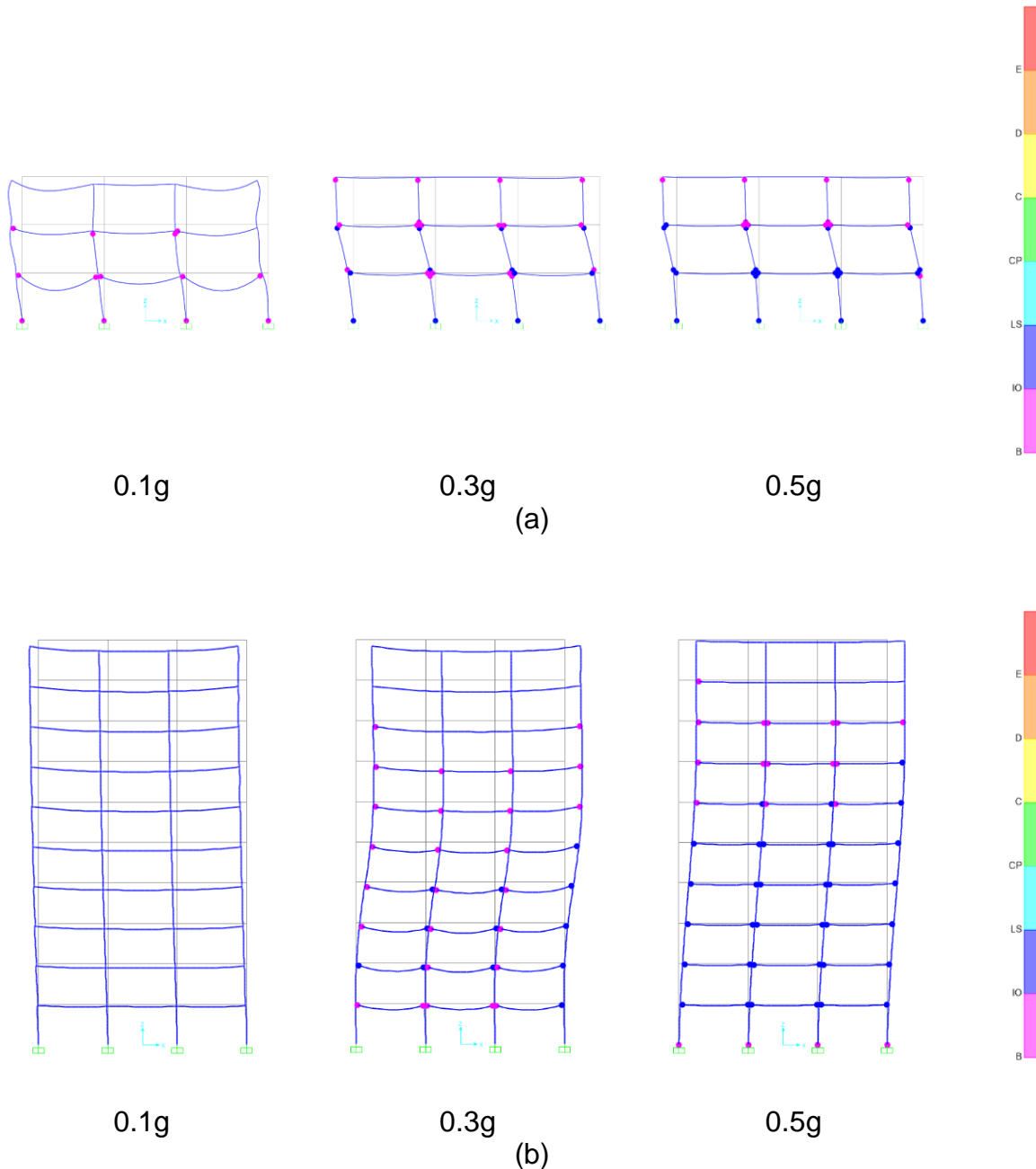
0.1g	0.3g	0.5g

(a)

0.1g	0.3g	0.5g

(b)

Fig. 4 Damage patterns of (a) Model 1, and (b) Model 2 under Kobe Earthquake

4. Fragility Curves

The next step is to develop the fragility curves for Model 1 and 2. In general, fragility curve can be represented by lognormal distribution, and thus the cumulative probability of occurrence of damage, with value equal or exceed a damage level *D*, is given as

$$P(D|\text{PGA}) = \Phi[(\ln(\text{PGA}) - \mu)/\sigma] \qquad (1)$$

where μ and σ are the mean and standard deviation of ln(PGA); and $\Phi(\dots)$ is the standard normal cumulative distribution. The values of μ and σ for each level of damage measure can be evaluated by plotting ln(PGA) and the corresponding standard normal variable on lognormal probability plots. The results are summarised in Table 3 for reference.

Table 3 Parameters of log-normal distribution

	IO (DR = 1.0%)		LS (DR = 2.0%)		CP (DR = 3.0%)	
	μ	σ	μ	σ	μ	σ
Model 1	-1.246	0.252	-0.638	0.272	-0.312	0.293
Model 2	-1.315	0.346	-0.507	0.432	-0.333	0.377

Fig. 4 and Fig. 5 present the fragility curves developed for Model 1 and 2, respectively, using Eq. (1). From these figures, the probability of reaching certain limit state under a particular PGA level can be estimated by mapping. For Model 1, the building model has about 20% probability to reach IO state at PGA of 0.2g. The chance to reach IO increases rapidly up to around 100% at PGA of 0.25g. The model has around 10% probability to reach LS state at PGA of 0.4g and rapidly increases to 95% at PGA of 0.5g. The model has around 10% probability to reach CP state at 0.6g and more than 95% chance at 0.8g. For Model 2, limit state is not yet achieved until PGA is equal to or exceeds 0.12g. The probability of reaching or exceeding OP is approximately equal to 90% at PGA of 0.2g. At PGA of 0.4g, there is 15% chance to reach LS state. At PGA of 0.6g, the building has 86% change to reach LS state and 35% change to reach CP state.

4. CONCLUSIONS

In this study, two fragility curves were developed for 3-storey and 10-storey RC buildings designed based on HK standards. Extensive incremental dynamic analyses were conducted to determine the relationships between maximum inter-storey drift ratio and peak ground acceleration. Nine past earthquakes, each was scaled from PGA of 0.1g to 1.0g, were considered in IDA. The seismic performance of selected buildings were presented in IDA curves. Three damage levels were selected based on maximum inter-storey drift ratio to quantify the performance of buildings. Fragility curves were then constructed with the assumption that PGA follows natural logarithm normal distribution. Based on the results, the performance levels of building under a

given PGA level could be obtained for the purposes of risk assessment, loss estimation or rehabilitation planning.
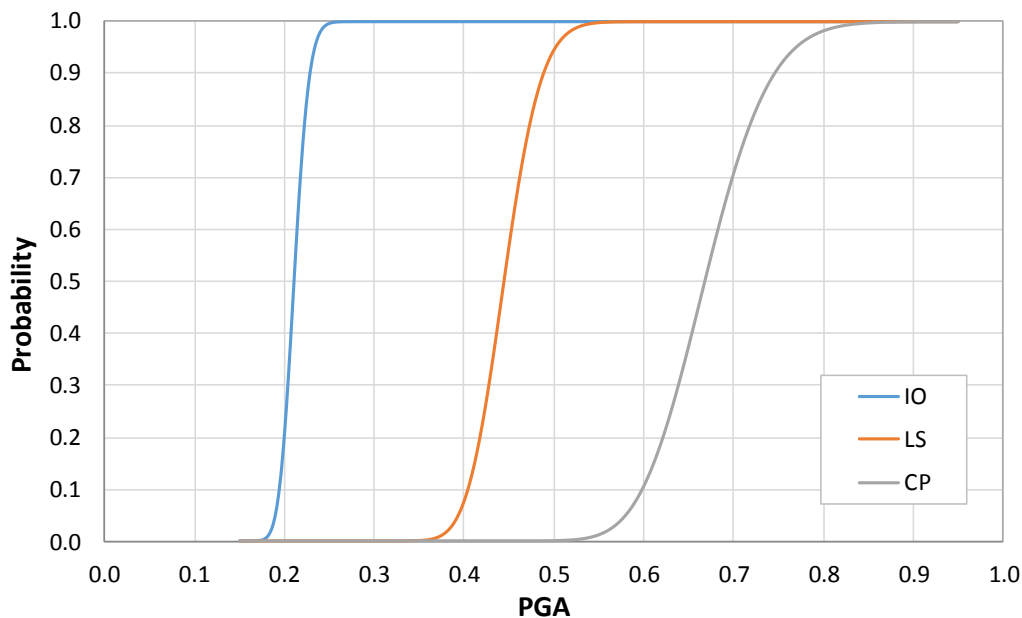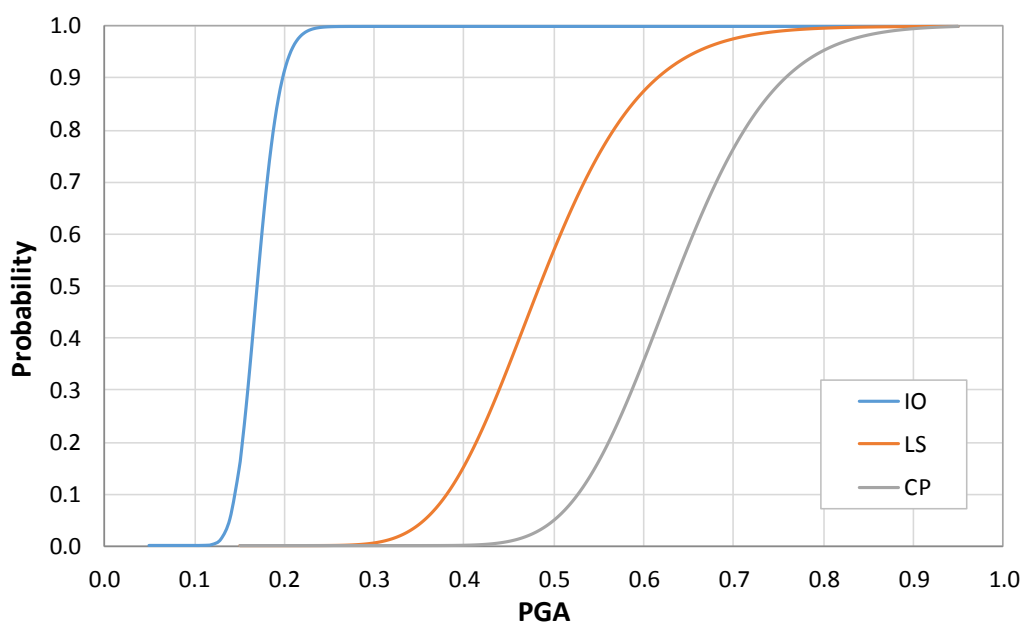

Fig. 4 Fragility curves for Model 1

Fig. 5 Fragility curves for Model 2

REFERENCES

Akkar S., Sucuoglu H. and Yakut A. (2005), "Displacement-based fragility functions for low- and mid-rise ordinary concrete buildings", *Earthquake Spectra*, **21**(4), 901-927.

ASCE (2000), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings – FEMA 356*, American Society of Civil Engineers (ASCE), Reston, VA.

Hong Kong, Building Department (2013), *Code of Practice for Structural Use of Concrete*.

Karim, K.R. and Yamazaki, F. (2003), "A simplified method of constructing fragility curves for highway bridges", *Earthquake Eng. Struct. Dyn.*, **32**, 1603-1626.

Kircil, M.S. and Polat, Z. (2006), "Fragility analysis of mid-rise R/C frame buildings", *Engineering Structures*, **28**(9), 1335-1345.

Majd, M., Hosseini, M. and Amini, A.M. (2012), "Developing fragility curves for steel buildings with X-bracing by nonlinear time history analyses", *15th WCEE*, Lisboa.

National Standard of the People's Republic of China (2010), *Chinese Code for Seismic Design of Building* (GB50011-2010), Ministry of Construction of Peoples Republic of China, Beijing, China.

Porter K., Kennedy, R. and Bachman, R. (2007), "Creating fragility functions for performance-based earthquake engineering", *Earthquake Spectra*, **23**(2), 471-489.

Rossetto, T. and Elnashai, A. (2003), "Derivation of vulnerability functions for European-type RC structures based on observational data", *Engineering Structures*, **25**(10), 1241-1263.

Rossetto, T. and Elnashai, A. (2005), "A new analytical procedure for the derivation of displacement-based vulnerability curves for populations", *Engineering Structures*, **27**(3), 397-409.

Saruddin, S.N.A. and Nazri, F.M. (2015), "Fragility curves for low- and mid-rise buildings in Malaysia", *EACEF-5*, *Procedia Engineering*, **125**, 873-878.

Tsionis, G. and Fardis, M.N. (2014), "Seismic fragility curves for reinforced concrete buildings and bridges in Thessaloniki", *2nd European Conference on Earthquake Engineering and Seismology*, Istanbul.

Vamvatsikos, D. and Cornell, C.A. (2002), "Incremental dynamic analysis", *Earthquake Eng. Struct. Dyn.*, **31**(3), 491-514.

Wu, D., Tesfamariam, S., Stiemer, S.F. and Qin, D. (2012), "Seismic fragility assessment of RC frame structure designed according to modern Chinese code for seismic design of buildings", *Earthq Eng & Eng Vib*, **11**(3), 331-342.