FRAGILITY-BASED SEIMIC ASSESSMENT OF IRANIAN RC BUILDINGS

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ABSTRACT

Fragility curves are used to show that structural damage under various levels of seismic excitation is likely to surpass specified damage states by means of earthquake intensity-damage relations. In this study, seismic fragility curves are developed for reinforced concrete (RC) structures typically built in Iran. The structures considered in this study have RC Intermediate Moment Frame systems, and are designed according to Iranian Earthquake code termed standard No.2800 (2005). In order to probabilistic seismic assessment of these sub-classes, full 3-Dimensional finite element models are developed and analyzed with OpenSEES. The uncertainty of ground motion and material are considered in the fragility analysis. The engineering demand parameter in terms of maximum inter–story drift ratio are developed for 20 different far-source sets of ground motion records. The structure's vulnerability is determined according to HAZUS-MH damage states.

INTRODUCTION

During the last two decades, the earthquakes that have occurred in Iran have caused much tragic life and monetary losses. The high population density near or on fault zones is an indicator of potential future disasters. Therefore, it is necessary to estimate possible earthquake hazard and develop strategies to reduce related losses. A fragility-based assessment that considers local structural properties is required to prepare such disaster mitigation scenarios. The aim of this study is to provide fragility information to inquire about the effects of ground motion parameters on structural vulnerabilities.

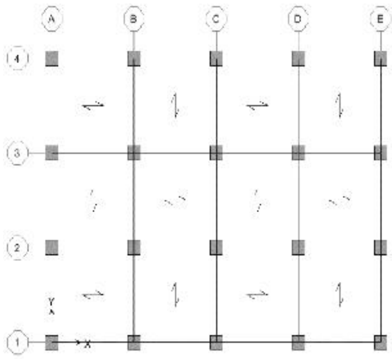
Fragility curves are found to be useful for estimating the seismic risk of urban infrastructures. These curves represent the probability of damage for different levels of earthquake intensity. They can determine the vulnerability rate of structures; therefore, fragility curves can be used to prioritize structures for seismic retrofitting. In addition, governmental management institutes and insurance companies, which are in charge of estimating post-earthquake damage, can make use -these curves (Akkar et al. 2005 ; Anagnos et al. 1994)
The history of generating fragility curves returns to nuclear installations due to their high risk in earthquakes. In 1980, fragility curves were developed for the first time for nuclear powerhouses (Kennedy et al. 1980).
Tanaka et al. (2000) used lognormal distribution to calibrate fragility curves. They classified 3683 bridges into five groups, and defined the damage rate for five levels. Then, they surveyed the parameters of

lognormal distribution.

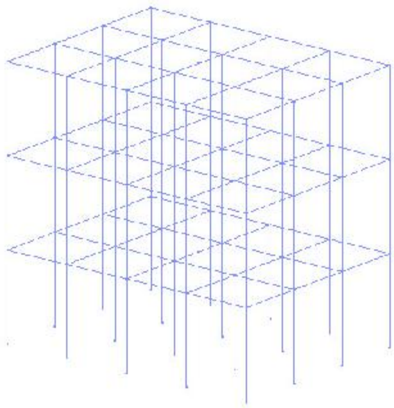
Aziminejad et al. (2007) developed fragility curves for RC special moment frame structures with shear wall. For this purpose, eight one-story models were analyzed under non-linear dynamic analysis using OpenSEES platform by considering the distribution effect of stiffness, strength, and torsion on fragility curves, and fragility curves were developed in terms of inter-story drift ratio, joints rotation and ductility, at different levels of PGA.

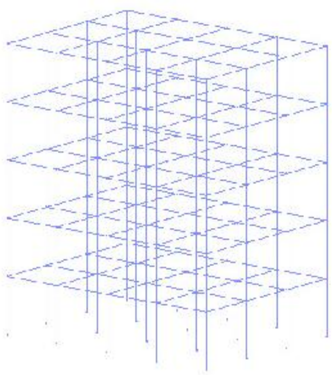
Barkhordary et al. (2011) evaluated the effect of bar splicing in columns and bars sliding on vulnerability of RC structures. They assessed the performance of four different structures (one-story, one-bay; one-story, three-bays; three-stories, one-bay; and three-stories, three-bays) with RC ordinary moment frame designed in accordance with ACI 318-08. Finally, fragility curves were developed based on Incremental Dynamic Analysis (IDA) of two-dimensional models using OpenSEES Software.

Adom-Asamoah (2012) investigated fragility curves of existing structures. The type of investigated structures in this were RC frame structures with low ductility. For this purpose, three types of buildings (three, four, and six stories) with symmetric plans, designed according to previous Standard BS 8110 (1985) and situated near a fault, were assumed. Thus, fragility curves were developed using IDRAC2D software, under Nonlinear Static (pushover) and Time History Dynamic Analysis.



Plan of three, five, and eight Story Buildings


Three-story Model


Five-story Model

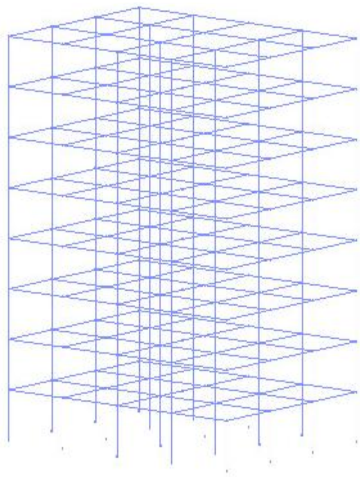

Eight-story Model

Figure 1. Model of three, five, and eight Story Buildings

It is assumed that the structures are located in a relatively high-risk region with Soil type III, according to the classification of Standard 2800. The buildings in both directions (two orthogonal directions) has an RC intermediate moment frame and typical Story height is 3.2 m. The compressive strength of concrete used in Beam and Column elements is assumed $fc = 220 \frac{kg}{cm^2}$. The type of longitudinal bars used in the linear elements is AIII, with yield stress of $fy = 4000 \frac{kg}{cm^2}$ and the type of Shearing bars (stirrups) is AII, with yield stress of $fy = 3000\frac{kg}{cm^2}$. The deck type of each story is ribbed slab and a rigid diaphragm is defined for each story.

CLASSIFICATIONS OF BUILDINGS BASED ON - HAZUS-MH SPECIFICATIONS

The Prescription of HAZUS-MH MR5. (2003), which has been utilized for defining performance levels of structures, divides the structures into three categories including Low-Rise, Mid-Rise, and High-Rise (Table 1). According to this classification, structures with three, five, and eight stories in this article are considered to be Low-Rise, Mid-Rise, and High-Rise, respectively.

Table 1. Types of buildings in terms of height (HAZUS-MH 2003)

Lable	Description	Range	
		Name	Stories
C1L	Concrete Moment Frame	Low-Rise	1 - 3
C1M		Mid-Rise	4 - 7
C1H		High-Rise	8+

MODELING SOFTWARE

The OpenSEES platform (McKenna et al. 2010) is used for time history dynamic analysis. Results of structural design are listed in tables 2 and 3.

Table 2. column specifications for the five-story Proposed building

Designed according to the third Edition of Standard No. 2800				
Number of Story	Column dimensions (cm)		Reinforcement columns	
	Length	Width	Number	Diagonal (mm)
Story 1	60	60	16	22Φ
Story 2	55	55	16	18Φ
Story 3	50	50	12	20Φ
Story 4	45	45	12	18Φ
Story 5	40	40	12	16Φ

Table 3. Beam specifications for the five-story Proposed building

Designed according to the Third Edition of Standard No. 2800					
Number of Story	Beam dimensions (cm)		Reinforcement Beams		
	Width	Height	Number of Bar		Diagonal (mm)
			Top	Bottom	
Story 1	60	45	6	5	18Φ
Story 2	55	45	8	6	18Φ
Story 3	50	40	7	5	18Φ
Story 4	45	40	8	5	16Φ
Story 5	40	35	7	3	14Φ

CHARACTERISTICS OF MATERIALS

CONCRETE BEHAVIOR

Concrete02 is used for defining concrete material in OpenSEES. This type of material considers the behavior of tensile stress of concrete (Mazzoni et al. 2007). The characteristics of this material is displayed in figure 2. The compressive strength of $220\,\frac{\mathrm{kg}}{\mathrm{cm}^2}$. at 28 days is defined for concrete material. Following the recommendations of Choi (2002), the compressive strength of concrete is modeled using a normal distribution with mean value, μfc, of $220\,\frac{\mathrm{kg}}{\mathrm{cm}^2}$ and standard deviation, σfc, of $40\,\frac{\mathrm{kg}}{\mathrm{cm}^2}$.

Figure 3 represents the Stress-Strain curve of the confined and unconfined concrete stated above, where strain corresponding to nominal compressive strength (f'_{c0}) is expressed by ε_{c0} , and ε_{sp} and ε_{cu} are defined as strain at the time of failure for unconfined and confined concrete, respectively. f'_{cc} is the compressive strength of concrete for confined concrete material, which is calculated according to Equation 1.

$$f'_{cc} = Kf'_{c0} \tag{1}$$

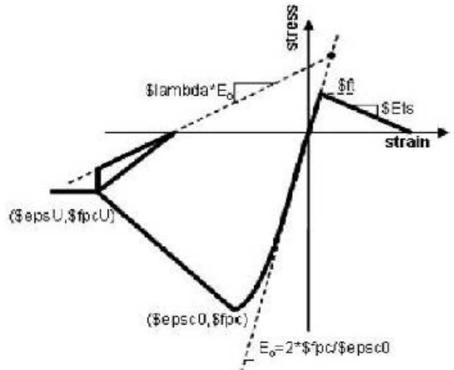


Figure 2. Concrete02 Material - parameters (Mazzoni et al. 2007)

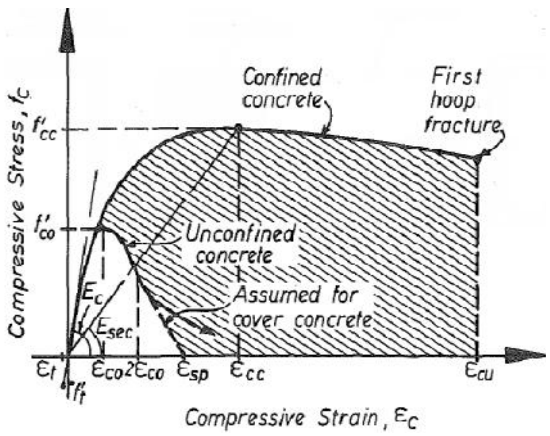


Figure 3. Stress-Strain Model Confined and Unconfined Concrete (Mander J. B. et al. 1988).

STEEL BEHAVIOR

Steel02 is used for defining Steel-Material in OpenSEES. The behavior of this material is presented in figure 4.

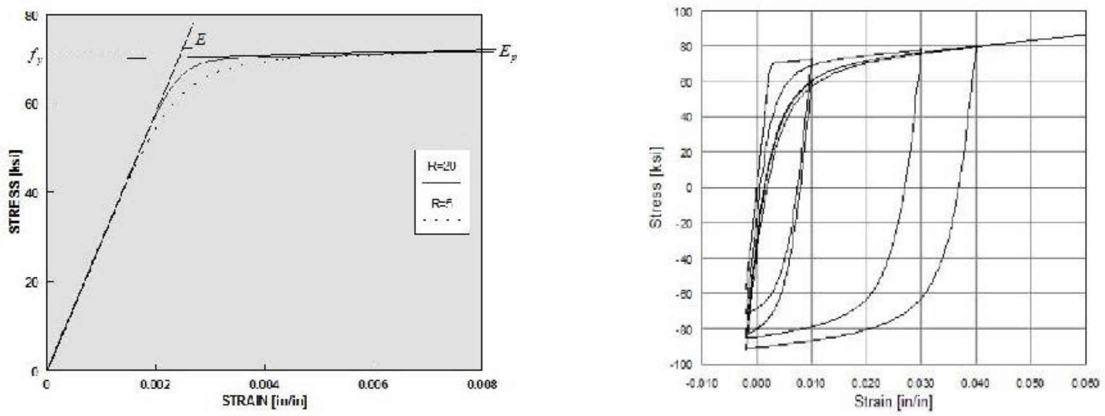


Figure 4. Steel02 Material - charectristics - (Mazzoni et al. 2007).

NATURAL PERIOD OF STRUCTURE CALCULATED IN OPENSEES SOFTWARE

The calculated Period in OpenSEES platform is listed in Table 4.

Table 4 – Natural period of the three, five, and eight-story structures

	Natural Period (sec)
three-Story	0.57
Five-Story	0.678
Eight-Story	0.80

GROUND MOTION CHARACTERIZATION

One of the most important factors in Incremental Non-linear Dynamic Analysis is to determine the ground motions, because the results obtained from analysis depend heavily on ground motions used for modeled structures. Therefore, selecting the right type of ground motion is extremely sensitive and will affect the results. The selection process should be carried out carefully until the obtained results include all types of structure behavior (elastic, plastic, and collapse).

According to recommendation of Shome and Cornell (1999), 10 to 20 ground motion records provide acceptable accuracy for estimating the vulnerability demand of structures. In addition, FEMA P695 (2009) has provided 22 recommended records of ground motions. In this study, far-fault ground motions recommended by FEMA P695 is used with some modifications.

DETERMINATION OF ENGINEERING DEMAND PARAMETERS (EDPS)

Determining Engineering Demand Parameters (EDPs) are necessary to generate the fragility curves. One of the most important EDPs that can represent the damage rate in buildings during an earthquake is Maximum Inter–story Drift. Vulnerability of buildings in different codes is defined from "slight level of damage" to "complete level of damage" (collapse). In this study, prescription of HAZUS-MH published by FEMA is used.

DETERMINATION OF DAMAGE LIMIT STATES

Damage limit state has direct relation with performance levels. Due to dissatisfaction of performance level regulations in case of passing a structural performance level, retrofitting would be necessary from the rehabilitation's standpoint. Thus, passing from each performance level of a structure is similar to failure of that performance level of the structure.

In HAZUS-MH, four levels of damage (slight, moderate, extensive, and complete) have been defined.

These levels for index of relative maximum inter–story drift are listed in Table 5. According to this instruction, the most appropriate point, which is the representative of demand at the performance level of collapse threshold, along the IDA curve is the point where the curve starts to soften leading the whole structure to dynamical instability. In fact, it should have the lowest damage rate among the other probable points. According to the definition, the first point that has about 20% of slope in the elastic area can be considered as earthquake demand in this performance level. A clear shortcoming in this method is that the slope of each IDA curve does not necessarily degrade to 20% of the primary linear area's slope before reaching complete instability of structure; therefore, using this method is not possible in some cases. Hence, maximum inter-story drift as a damage based index is presented in related codes according to the type of buildings. Each damage index that is passed sooner, would be considered as collapse point. Maximum inter–story drift for three, five, and eight-story structures is defined in Table 5 in the form of 6%, 4%, and 3% of complete level of damage, respectively (HAZUS-MH 2003)

Table 5. Average value of maximum inter–story drifts for different types of damage states according to HAZUS-MH (2003)

Type	Inter Story Drift at Threshold of Damage State			
	Slight	Moderate	Extensive	Complete
C1L	0.005	0.0087	0.0233	0.06
C1M	0.0033	0.0058	0.0156	0.04
C1H	0.0025	0.0043	0.0117	0.03

IDA ANALYSIS FOR STUDIED STRUCTURES

Incremental Dynamic Analysis is used for analyzing the structures. In this method, the applied Peak Ground Accelerations (PGA) has been scaled with steps of 0.1g from 0.1g to complete level of damage. Then, IDA Curves were plotted with analysis in each step. Figure 5 depict the behavior curves of the studied structures under a suite of 20 ground motions in the third edition of Standard No. 2800 via IDA.

THEOREM OF PROBABILITY IN FRAGILITY CURVES

To generate fragility curves, a probability distribution for engineering demand parameters obtained from nonlinear IDA should be considered. In this study, lognormal distribution, which is one of the most common probability distributions in this field, is used.

GENERATION OF FRAGILITY CURVES

Each structure is analyzed about 300 times under 20 records of ground motion from 0.1g to 1.5g. Then, the fracture probability of each structure is calculated in each level of earthquake intensity from 0.1g to 1.5g.

When structural capacity and seismic demand have two variables that follow lognormal distribution, by using the central limit rule, we can show that the obtained combined operation follows lognormal distribution. Hence, the fragility curve can be written as a cumulative lognormal distribution function according to Equation (2) (Cornell et al. 2002).

$$p(D > d \mid IM) = 1 - \Phi\left(\frac{\ln(d) - \ln(S_d)}{\beta_{D|IM}}\right) \tag{2}$$

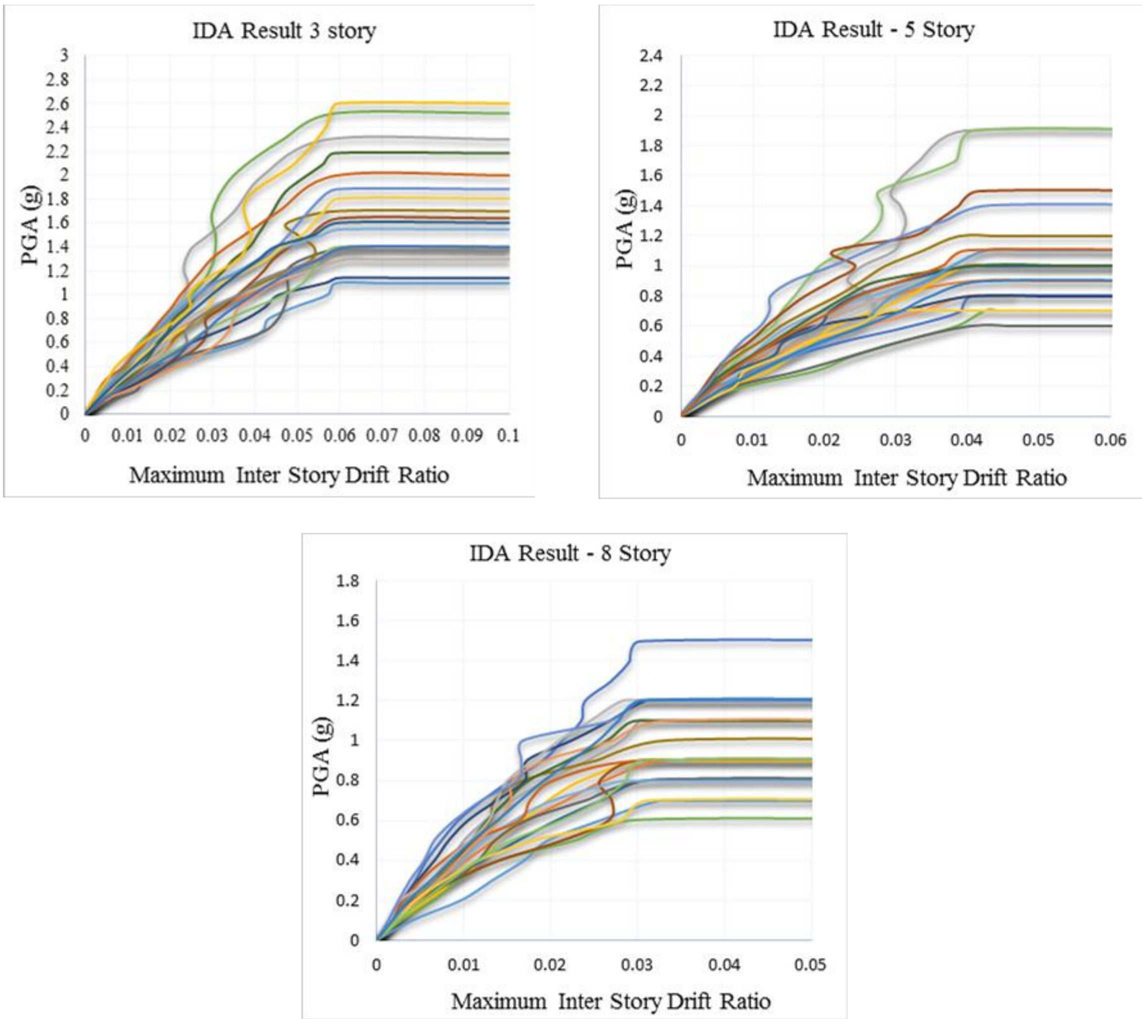


Figure 5. IDA study for twenty records

In the equation (2), p is the probability of getting to or passing from the damage state of D (in this study, maximum inter–story drift); $\Phi()$ is standard normal cumulative distribution function; $\beta_{D|IM}$ is the lognormal SD (dispersion) of maximum inter–story drift (Table 6), which is also estimated from the regression analysis. S_d is the median value of seismic demand that can be calculated from Equation (3).

$$\ln(S_d) = a \ln(IM) + b \tag{3}$$

Where a and b = coefficients obtained using regression analysis; and IM = intensity measure.

Table 6. Lognormal standard deviation of maximum inter–story drift (β_{sd})

	Lognormal standard deviation
3 Story	0.29
5 Story	0.326
8 Story	0.289

FRAGILITY CURVES

Figures 6 to 12 represent the fragility curves of structures in the four levels of damage (slight, moderate, extensive, and complete).

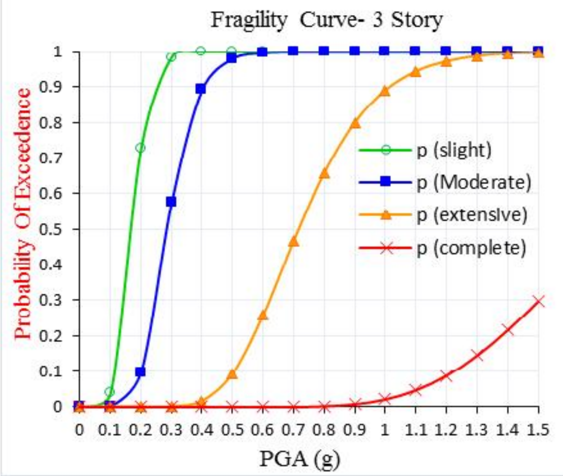


Figure 6. Seismic fragility curve of three-story structure

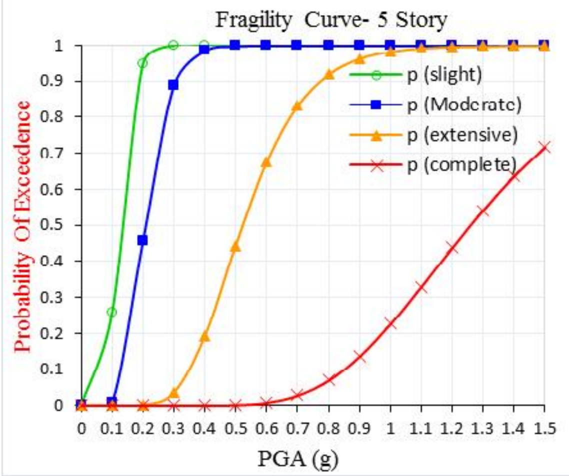


Figure 7. Seismic fragility curve of five-story structure

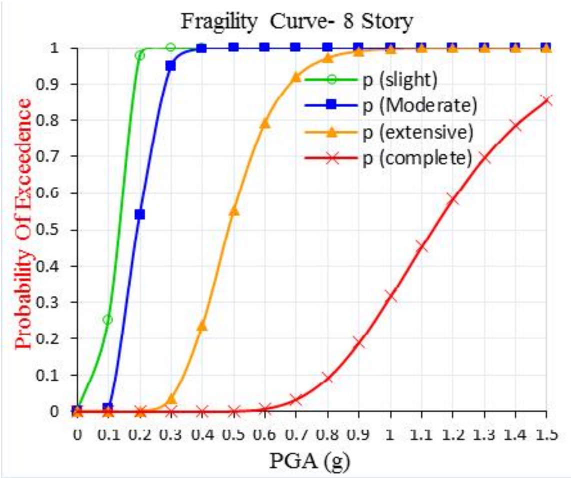


Figure 8. Seismic fragility curve of eight-story structure

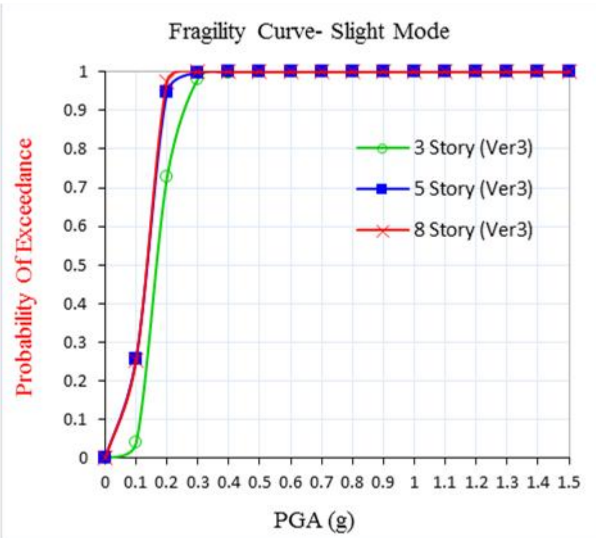


Figure 9. Seismic fragility curve of three, five, and eight-story structures in slight damage state

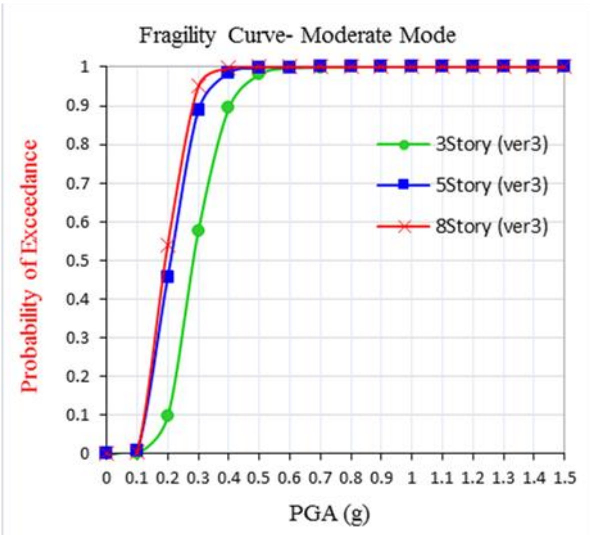


Figure 10. Seismic fragility curve of three, five, and eight-story structures in moderate damage state

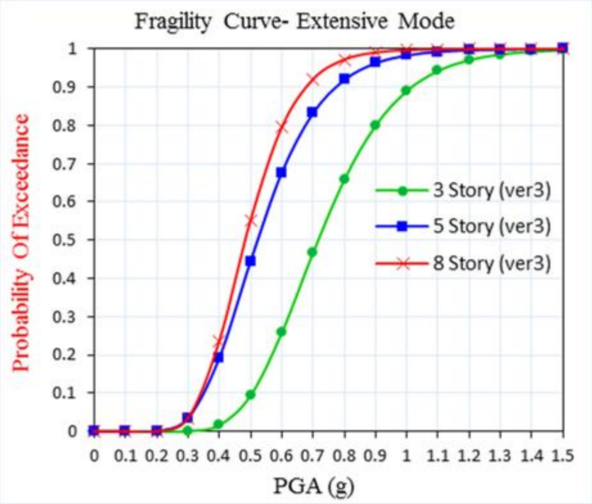


Figure 11. Seismic fragility curve of three, five, and eight-story structures in extensive damage state

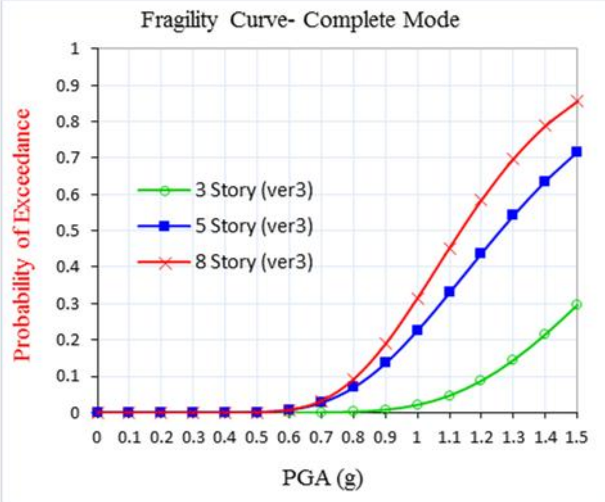


Figure 12. Seismic fragility curve of three, five, and eight-story structures in complete damage state

CONCLUSIONS

Based on the investigations carried out on the seismic vulnerability of three types of RC-moment-frame buildings according to the Third edition of standard No. 2800, the following results were achieved.

By observing the group of IDA curves, the general view of structures' behavior from complete elastic limit to complete damage can be acquired. Comparing the behavior of structures with different heights, it is understood that the increase in height, will make structure enter the non-linear area sooner, and hence, the structure capacity decreases. In general, with the increase in structure height, it's vulnerability increases in four specified levels of damage (slight, moderate, extensive and complete). For instance, the probability of complete damage in an eight-story building is more than a five-story building, and the probability of complete damage in a five-story building is more than a three-story building. However, if we want to survey more precisely, we observe that the fracture probability increase from the three-story building to five-story building is much more than a five-story building to an eight-story one. This means that with an increase in structure height, the trend of increase in damage decreases. The probability of vulnerability in five and eight-story buildings is close to each other, as the probability of vulnerability in five and eight-story buildings for slight level of damage is approximately the same. However, in higher levels of damage, they deviate from each other. As observed in figures, the slope of fracture curve in slight and medium level of damage is higher in lower values of PGA, and lower in higher values of PGA. This means that the rate increase of damage probability is more for lower values of PGA.

According to the plotted curves, for the low-rise RC intermediate moment frame structures that have been constructed in accordance with the Third Edition of the standard 2800 , the probability of extensive and complete damage in ground motions with PGAs lower than 0.4g and 0.1g is insignificant. In addition, for high-rise and mid-rise structures, the probability of extensive and complete damage in ground motions with PGAs lower than 0.3g and 0.7g is also insignificant.

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