SEISMIC FRAGILITY ESTIMATION UNDER ORTHOGONAL EARTHQUAKE EXCITATIONS

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ABSTRACT

Estimation of fragility functions using dynamic structural analysis is an important step in a number of seismic assessment procedures. This paper discusses the multicomponent seismic fragility curves based on beta distribution by considering the Iran–specific characteristics to manage the earthquake risk in the region. The seismic design of low- and mid-rise RC-MRFs are carried out according to the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800), and the analytical models are formed accordingly in ZEUS-NL platform. A new optimal intensity measure (IM) called ($S_{a,\ avg}$) is used to obtain reliable fragility–based database for earthquake damage and loss estimation of RC buildings stock. It is observed that the presence of vertical component of the strong earthquake excitation significantly affects the response of RC-MRFs and including this component simultaneous to horizontal components in the analysis is highly recommended for reliable seismic assessment of RC structures.

INTRODUCTION

Earthquake hazard identification and structural vulnerability evaluation are the main components of earthquake risk assessment. Earthquake hazard identification is out of the scope of this study, but structural vulnerability evaluation is the subject of civil engineering and city planning disciplines and aims to determine, classify, and assess the fragility of existing building stock and other structures (dams, bridges, power plants, etc.) and are being focused in the current study.

For disaster management purposes, a fragility based assessment that considers local structural properties is required. However, local conditions are usually ignored and vulnerability based assessment studies for structures in different countries are adapted to earthquake hazard estimation and disaster mitigation. Unfortunately, differences in structural characteristics cause significant deviations on damage and loss estimation by influencing the resulting fragility curves. The aim of this study is to provide fragility information to inquire effects of ground motion parameters and Iranian construction practice state on structural vulnerability in the presence of vertical component of earthquake. After the devastating earthquakes that occurred within the last decades, well-organized and comprehensive seismic design code (Standard No. 2800, (1988, 1999 and 2005)) are published. Making use of this comprehensive design codes, this study is deemed to be a benchmark for future studies on earthquake damage and loss estimation in urban areas of Iran.

As mentioned, the main concern of the researchers in this study has been the fragility levels of the current RC-MRFs in Iran. Fragility functions are in general derived using a variety of approaches such as field observations of damage, static structural analyses, or judgment (e.g., Calvi et al. 2006, Porter et al. 2007, Shafei et al. 2011), but here the focus is on so-called analytical fragility functions developed from dynamic structural analysis. Unlike some other methods, in the case of analytical fragility functions the analyst has control over the data collected, by means of choosing the IM levels at which analysis is performed and the number of analyses performed at each level. The fragility curves generated at the final phase of this study provide a reliable database for earthquake damage and loss estimation of RC buildings stock in Iran. The RC buildings which are considered in the current study are classified into three subclasses defined below:

Class1: This group includes the RC buidldings designed based on the 1st edition of Iranian code of practice for seismic resistant design of buildings (Standard No. 2800, 1988), which the ductility levels are not clarified in a good way. Besides that, a few percentage of the RC buildings in Iran which are constructed in the last decades and most of them are suffering from construction, detailing and design deficiencies are as well included in this subclass because of the similarities in fragility results.

Class2: It represents a large numbers of the buildings stock concerning the RC residential buildings in Iran. They are generally engineered structures but may violate some fundamental requirements of earthquake resistant design and construction. The buildings in this subclass are designed based on 2nd edition of Iranian code of practice for seismic resistant design of buildings (Standard No. 2800, 1999) and earlier version of Iranian standards for design and construction of RC structures (ABA).

Class3: The buildings in this subclass are designed according to the latest codes (Standard No. 2800 3rd edition (2005) and Iranian standards for design and construction of RC structures, (Code #9) and have adequate structural capacity in terms of strength and ductility in a severe earthquake. Good material quality, earthquake resistant design, and good supervision in the construction stage result in reliable performance levels.

CURRENT SEISMIC DESIGN PHILOSOPHY

Many codes suggest scaling a single spectral shape, originally derived for horizontal components to deal with vertical earthquake motion. This implies that both components of motion have the same frequency content, which is clearly not the case.

A procedure was originally proposed by Newmark et al. (1973) and has since been widely used in the seismic codes. It was suggested that the average peak vertical-to-horizontal spectral ratio could be taken as 2/3. This implies that the vertical-to-horizontal ratio is also 2/3 assuming constant amplification. Recent study by Ambraseys and Simpson (1995) confirms that the 2/3 rule is unreasonable. Evidence from the, Loma Prieta earthquake of 1989, the Northridge earthquake of 1994 and the Kobe earthquake of 1995 confirm this, all with V/H ratios well in exceedence of 1.0 in the near-field. It is clear that this ratio is magnitude, distance and frequency dependent and should be a variable in code design.

VERTICAL COMPONENT OF GROUND MOTIONS

Vertical component of the strong ground motion is mainly associated with body waves: vertically propagating compressional waves (i.e., P-waves) and horizontally propagating dilatational waves (i.e., S-waves). Compared to the horizontal component, vertical motion may be richer in high-frequency content in the near field of an earthquake fault. As the distance from the source increases, difference in the frequency content between horizontal and vertical components becomes much smaller as a result of faster attenuation of high frequencies with distance, and mixing of horizontal and vertical motions due to non-homogeneities along the wave path. A common perception in engineering practice is that intensity of vertical ground motion is lower than that of the horizontal; thereby V/H ratio (i.e., the ratio of vertical to horizontal peak ground acceleration) is assumed to remain less than unity. (Bozorginia and Campbell 2004)

In Fig. 1(a) the authors presented the plot of the V/H ratio against the magnitude of the events for a large database of 80 earthquake records in the range of 5-7.6 Mw. It may be seen that the median V/H ratio is equal to 1.02, being much higher than the commonly accepted value of 0.67. The nearfault dataset used here implies that higher vertical acceleration tends to create larger V/H ratio as shown in Fig. 1(b). Similar correlation, however, does not exist between the V/H ratio and peak horizontal acceleration.

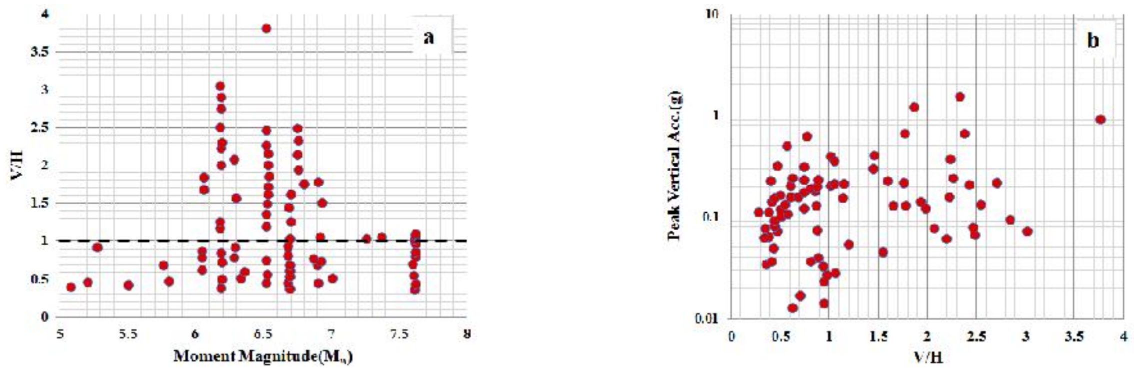

Figure 1. Ratios of peak vertical to horizontal acceleration (V/H) plotted against (a) moment magnitude, and (b) peak vertical acceleration (dashed line indicate the sample median)

SELECTION OF STRONG GROUND MOTIONS

Twenty forward-directivity ground motions from six earthquakes having a vertical PGA greater than 0.3g and V/H ratio larger than 0.6 with moment magnitude (M_w) greater than 6.5 were compiled into a database (Table 1). All records were taken from stations within 15 km of the fault rupture. Records were obtained from the Pacific Earthquake Engineering Research Centre Database.

Table 1. Earthquake Records used in this study

Earthquake Record	**Date**	**Moment Magnitude(M_w)**
Loma Prieta(LP)	1989	7.0
Northridge(N)	1994	6.7
Erzincan, Turkey(EZ)	1992	6.7
Kocaeli(K)	1999	7.4
Chi-Chi(CH)	1999	7.6
Duzce(D)	1999	7.1

FRAGILITY ESTIMATE AND BETA DISTRIBUTION

Fragility is the conditional probability of a building reaching or exceeding a certain performance level for a given ground motion parameter. Following the conventional notation in structural reliability theory (Ditlevsen and Madsen 1996), the limit state function for the building is written as Eq. (1).

$$g\left(C, IM ; «\right) = C - D(IM ; «) \tag{1}$$

Where *IM* is the seismic intensity parameter, « represents the vector of unknown parameters of the demand model, and *C* and *D* represent the capacity and demand of the building, respectively. Using Eq. (1), the fragility for the building is written as Eq. (2).

$$F\left(IM ; «\right) = P[\{g(C, IM ; «) \leq 0\} \mid IM] \tag{2}$$

The uncertainty in the event for given IM arises from the inherent randomness in the capacity *C*, the inexact nature of the limit state function, and the uncertainty inherent in the parameters of the demand models (Ramamoorthy et al. 2006a). In order to model for the epistemic uncertainty associated with fragility at a given shaking intensity, a beta probability distribution is used.

TYPOLOGIES OF THE BUILDINGS CONSIDERED

This study refers to two national codes for the design of RC buildings. These are the various versions of the Iranian Standards for Design and Construction of RC Structures (ABA and Code#9) and Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800, (1988, 1999 and 2005)) that state the minimum requirements for structures and structural components to be built in seismic prone areas.

Since number of stories is deemed to be important regarding the seismic response of RC frame structures, it is considered as a major parameter in this study. Hence 3, 5, 7, 9 and 12 story moment resisting frame models are constructed. The analytical models considered cover the low–and mid–rise RC frame structures population, which represents the majority of residential buildings in Iran. Story height of 3.2 meters and bay width of 5 and 7 meters are assumed in accordance with the common practice. Typical sketches of the analytical models are given In Fig. 2.

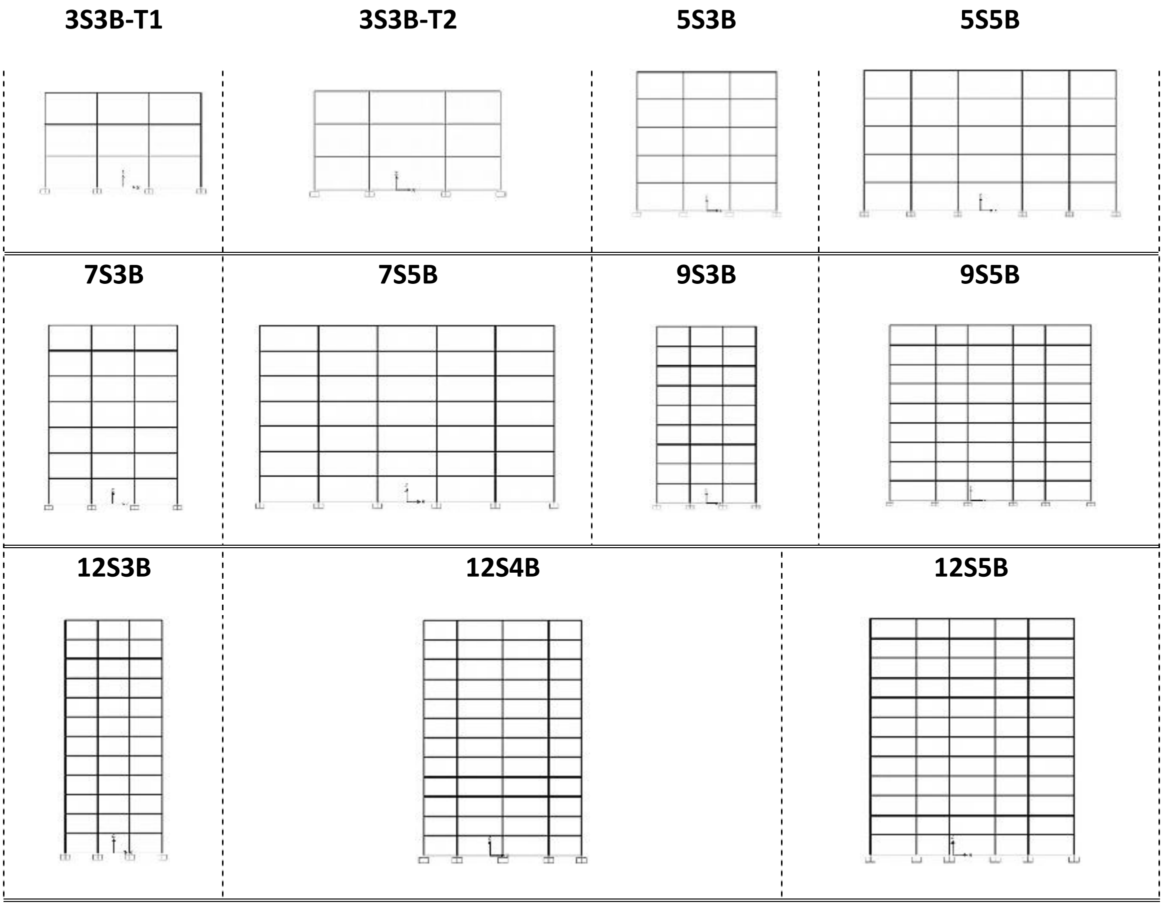

Figure 2. Schematic representations of the buildings considered in this study

HYSTERETIC MODEL USED

The hysteretic model used in this study was developed by Ibarra et al. (2005). Fig. 3 shows the trilinear monotonic backbone curve and associated hysteretic rules of the model, which provide for versatile modelling of cyclic behaviour. An important aspect of this model is the negative stiffness branch of post-peak response, which enables modelling of strain-softening behaviour associated with concrete crushing, rebar buckling and fracture, and bond failure. The model also captures four basic modes of cyclic deterioration: strength deterioration of the inelastic strain-hardening branch, strength deterioration of the post-peak strain-softening branch, accelerated reloading stiffness deterioration, and unloading stiffness deterioration. Additional reloading stiffness deterioration is automatically incorporated through the peak-oriented cyclic response rules.

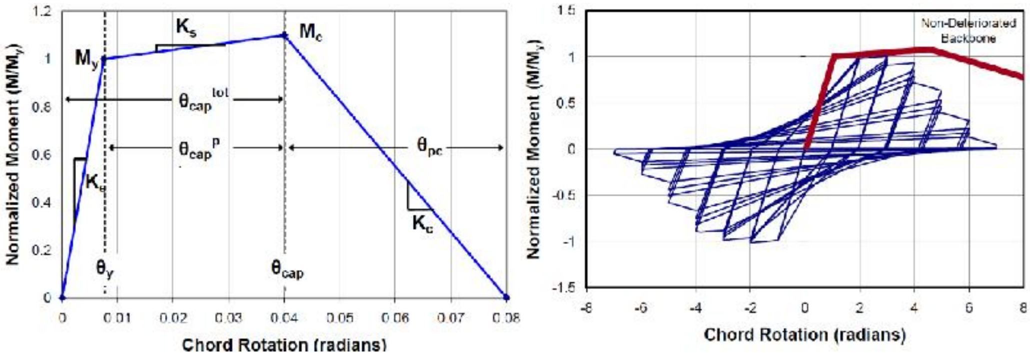

Figure 3. Monotonic and cyclic behavior of component model used in study. (Developed by Ibarra et al. (2005))

STRUCTURAL MODELING AND RESPONSE MEASURES

The Mid-America Earthquake Center program Zeus-NL was utilized to perform the analyses for the selected structures. Zeus-NL platform is an advancement of the earlier analysis packages ADAPTIC and INDYAS developed by MAE which was specifically developed for earthquake engineering applications (Elnashai *et al.* 2004).

Structural failure may occur due to the attainment of member or system level limit states. Thus, in this study the structural response was investigated at both the global and the local levels. Interstorey drift was considered as a global failure criterion, while the steel and concrete stress/strains of structural members were monitored to assess failure on a local level. The effect of vertical ground motion on axial force was also investigated. To account for shear deformation, the elements were modeled with a shear spring (Lee and Elnashai 2001) in parallel with an inelastic beam element as shown in Fig. 4(a). The primary curve of the shear spring is defined by a multilinear symmetric relationship that accounts for the cracking, yielding, and ultimate states, as shown in Fig. 4(b).

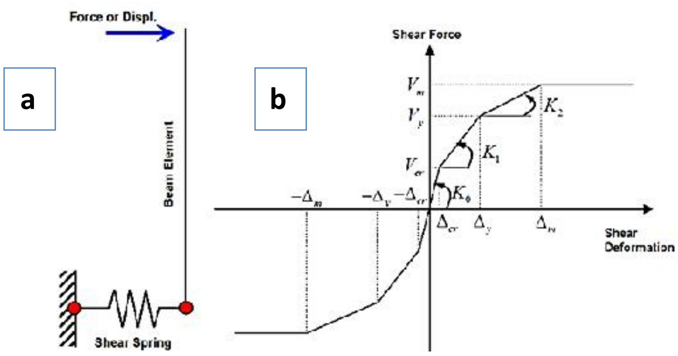

Figure 4. Shear spring modeling in Zeus-NL

INTENSITY MEASURE (IM) SELECTION FOR THE NON-LINEAR ANALYSIS

The damage potential of earthquake ground motion is usually characterized by a ground motion parameter called the intensity measure (IM) in seismic vulnerability assessment. A good IM should meet the requirement of efficiency and sufficiency (Lucco and Cornell 2007).

In the past, peak ground acceleration (PGA) was commonly used as an IM. Simplicity is the main advantage of PGA, but it results in great dispersion of structural response. More recently, the spectral acceleration at the first mode vibration of the structure, Sa(T_1), has been thoroughly studied and became very popular. This IM contains ground motion spectral information as well as dynamic character of structure, so it's more efficient and sufficient than PGA (Hwang and Huo 1997). However, earthquake disaster experience and strong ground motion data show that the structural seismic response depends on ground motion amplitude, spectrum, and duration characteristics simultaneously, and the different combinations of these three elements determine the degree of safety of the structure. In this study, in order to accurately characterize the ground motion potential, a new proposed IM by (Bianchini et al. 2009) which is the geometric mean of pseudo-spectral acceleration ordinates over a certain range of periods, $S_{a,avg}(T_1,\ldots,T_n)$, or briefly $S_{a,avg}$, is used as an optimal scalar IM to predict inelastic structural response of buildings subjected to recorded ground motions. The formulation for this IM is given in Eq. (3).

$$S_{a,avg}\left(T_1,\ldots,T_n\right)=\sqrt[n]{\left(\prod_{i=1}^{n}S_a(T_i)\right)} \tag{3}$$

It can be proven that, for multi-degree-of-freedom systems, $S_{a,avg}$ can be calculated using ten points logarithmic spaced in the interval $T_1,\ldots,T_n$. Furthermore, they supposed that T_1 and T_n are unknown, but tied to the fundamental period of the structure, $T^{(1)}$. So, the average of spectral accelerations is calculated such that $T_1 = k_1T^{(1)}$ and $T_n = k_uT^{(1)}$, where k_1 and k_u are constants specifying lower and upper bounds, respectively, relative to $T^{(1)}$. With these assumptions, Eq. (4) can be written as:

$$S_{a,avg}=\sqrt[10]{\left[S_a\left(k_1T^{(1)}\right)\times\ldots\times S_a\left(k_uT^{(1)}\right)\right]} \tag{4}$$

The constant k_1 is chosen to vary between $T_{low}/T^{(1)}$ and 1, whereas k_u between 1 and $T_{upp}/T^{(1)}$, where T_{low} and T_{upp} are, respectively, the lower and the upper period of the elastic spectrum (Fig. 5).

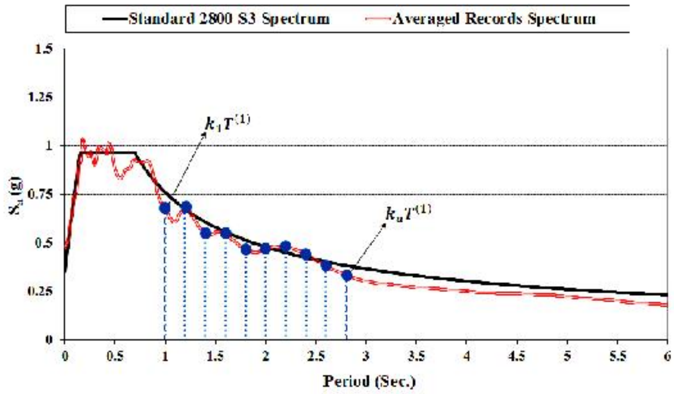

Figure 5. Schematic definition of $S_{a,avg}$ based on the selected strong motion records

RESULTS AND DISCUSSION

The dynamic response time history analyses were performed with selected records described previously. The extracted results are discussed accordingly.

THE EFFECT OF VERTICAL COMPONENT ON AXIAL LOADS OF COLUMNS

For the seven storey models, the averaged maximum values of the axial loads of the columns on each floor under three earthquake records with different V/H ratio are as shown in Fig. 6. Compare the axial loads of the columns under inputting horizontal ground motion only with that under combined horizontal and vertical ground motion. As can be seen, when inputting horizontal ground motion only, the axial loads of the interior columns had little change compare with the axial loads under gravity load, but the exterior columns change much. When inputting combined horizontal and vertical ground motion, the peak value of exterior columns increased evidently, while the variation of the peak value of interior columns was particularly significant. This significant fluctuation of axial force increased the possibility of the shear failure in the columns.

The tension force did not occur in all the exterior and interior columns under horizontal ground motion. But the action of vertical ground motion made the tension force occur in the columns and the maximum tension force mostly occurred in the middle and upper columns of the structure.

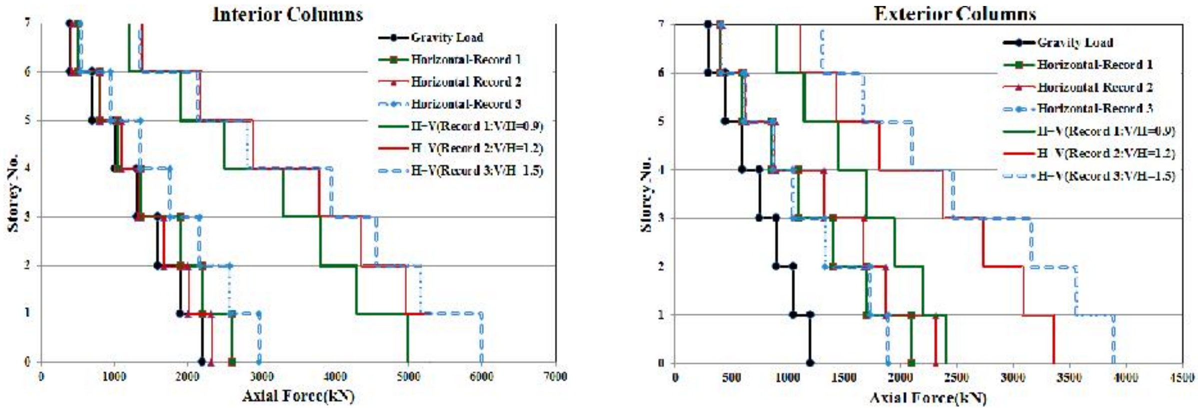

Figure 6. The peak value of the axial loads of the columns on each floor

PROBABILISTIC STRUCTURAL CAPACITY

To estimate the seismic fragility the capacity values must be specified in a probabilistic sense. The deterministic seismic structural capacity value corresponding to the damage levels from IDA are considered as the median capacity value. Uncertainty in estimation of the structural capacity arises from uncertain material properties, geometry, quality of construction, and assumptions in structural models of buildings.

Fig. 7 shows the probability density distribution of structural damage ratio under various levels of IM and beta fitting distributions for 5 storey typical models. From Fig. 7, it can be seen that choosing beta density distribution function to fit the probability density distribution of vulnerability matrix can basically express the distribution characteristics of structural damage ratio under determined seismic intensity.

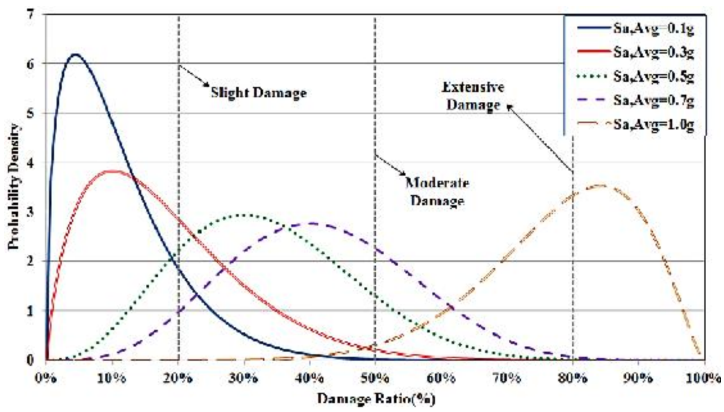

Figure 7. The probability density distribution of structural damage ratio with different intensities

FRAGILITY ESTIMATES CONSIDERING CONFIDENCE BOUNDS

Since for practical applications a continuous fragility estimate is preferred, a beta cumulative distribution function is selected to obtain continuous fragility estimates over the entire range of spectral accelerations. It is desirable to determine the epistemic uncertainty inherent in the fragility estimate, which is reflected in the probability distribution of relative to the parameter. Exact evaluation of this distribution requires nested reliability calculations (Der Kiureghian 1989). Following Gardoni et al. (2002a), approximate confidence bounds are obtained using a first-order analysis. These bounds approximately correspond to 5% and 95% confidence level on the fragility estimates. Fig. 8 show the fragility curves with confidence bounds for all buildings for collapse and moderate damage modes.

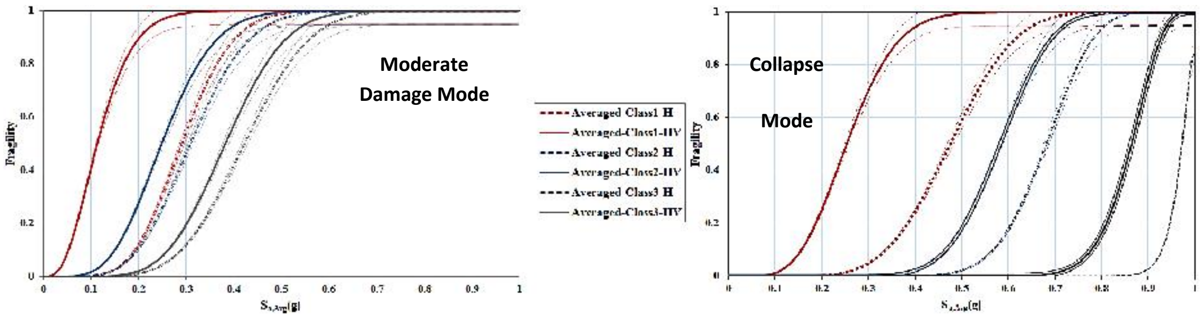

Figure 8. Averaged fragility curves for various subclasses in Moderate-damage and collapse modes

CONCLUSIONS

In this study, the effect of multicomponent seismic excitation including vertical ground motion on the RC-MRFs with 3 subclasses, which represents the majority of residential buildings in Iran, is presented. To generalize the results, we designed and assessed multiple RC-MRFs ranging in height from three to twelve stories and various span length, all based on various versions of Iranian Seismic Code (Standard No. 2800) provisions. Taking into account the given observations, RC-MRFs subjected to the concurrent horizontal and vertical seismic excitations could be more vulnerable than those subjected to horizontal ground motion only. Therefore, including vertical ground motion in the analysis is highly recommended for reliable seismic assessment of RC buildings. The topic of acceptable collapse risk and desired safety goals is worthy of substantial further study. Studies such as this can provide better understanding of the collapse safety of current buildings and can inform a decision making process to mitigate risk through the calibration of seismic codes for the design of new buildings.

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