EXPLICIT MODELING OF GROUND MOTION VARIABILITY IN THE DEVELOPMENT OF FRAGILITY MODELS

Ufuk YAZGAN[1]

ABSTRACT

An accurate evaluation of seismic risk requires accurate representation of the uncertainties associated with the factors that are related to the risk. The fragility of constructed facilities is a critical component in all seismic risk assessment studies. Fragility models are established for modeling the likelihood of the structure or a group of structures to exceed a damage level given that the structures are subjected to specific shaking intensity level. One of the commonly utilized approaches for establishing fragility models is to investigate the damage distributions observed after damaging earthquakes. In these investigations often the uncertainty associated with the ground excitation that had damaged the building, is not explicitly taken into account. A new approach is proposed here for building fragility models. The most important novelty of the proposed method is its capability of explicitly considering geospatial ground motion variability in the analysis. The statistical correlation structure of the peak ground motion intensities at the considered damage observation sites are explicitly modelled in the proposed approach. As an example application of the proposed method, fragility models for multi-story reinforced concrete moment resisting frame buildings are presented. In this application, the damages observed in 516 buildings inspected after the November 17th, 1999 M7.1 Düzce (Turkey) and May 1st, 2003 M6.4 Bingöl (Turkey) earthquakes are considered. The results from the example application approve the capabilities of the method in terms of effectively establishing fragility models.

INTRODUCTION

Fragility models are integral components of risk assessment frameworks. A fragility model is a relationship between the probability of a structure to sustain or exceed a specific damage grade and the ground motion intensity at the site of the structure. In the estimation of seismic risk, the likely damage states of the structures can be reliably estimated only if the adopted fragility models can accurately represent the actual vulnerability characteristics of the considered structural stock. Intensity of shaking at the site is commonly measured in terms of peak ground acceleration, spectral acceleration, peak ground velocity or Modified Mercalli Intensity. Various fragility and vulnerability models that are developed using these intensity measures can be found in the literature. The damage grade is typically defined based on the post-earthquake usability or cost-effective reparability of the damage, or in terms of integrity of the structural system.

Typically, fragility models are developed for certain classes of buildings and specific damage grades or threshold damage levels. The classes of buildings are defined in terms of key structural

[1] Asst.Prof.Dr., Istanbul Technical University, Istanbul, ufukyazgan@itu.edu.tr

properties such as: structural system, number of stories, design year, etc. The aim is to differentiate between different types of structures that have differing seismic vulnerability characteristics.

There four common types of approaches for establishing fragility models. These approaches are known as: (1) analytical approaches (e.g. Singhal and Kiremidjian, 1996), (2) empirical approaches (e.g. Rossetto and Elnashai, 2003), (3) expert-opinion based methods (e.g. ATC, 1985) and (4) hybrid methods (e.g. Singhal and Kiremidjian, 1998). The key principles underlying these approaches are summarized by Porter (2003). Method proposed in this proceeding is based on the empirical approach. In the empirical approach, the fragility models are developed based on the past earthquake damage observations. The empirical fragility modeling approaches are particularly suitable for non-engineered or non-code conforming structures. Such structures are often not designed for controlled ductile yielding mechanisms and fail under complex modes. Capturing the deformation mechanisms of non-code conforming structures accurately is not a simple task. A major benefit of the empirical methods is that there is no need for establishing an analytical model capturing the structural response. On the other hand, one limitation of empirical approaches is the requirement of large sets of earthquake damage observations related to the considered structural class. When the fragilities of relatively uncommon types of structures are of interest, this limitation becomes is an important issue. However, for the multistory reinforced concrete (RC) frame buildings located in seismically active regions, this issue is rarely a drawback.

Proper representation of the uncertainties associated with the peak motions experienced at the damage observation sites is a critical matter in empirical development of fragility curves. Seismic ground motions are known to exhibit significant spatial variability (Boore et al. 2003, Zerva, 2009). This variability is often attributed to the complexity of the reflections, refractions and scattering of the waves in the near-source region as well as the variability of local site properties (Abrahamson et al. 1987). Due to the variability of seismic motions, actual ground motion intensities at the sites of building damages can be estimated only probabilistically if there is no strong motion station at these sites.

In the existing fragility modeling approaches, the ground motion intensities at the considered sites are represented by deterministic estimates (e.g. Singhal and Kiremidjian, 1998; Rossetto and Elnashai, 2003). In the case when there are strong motion records related to the considered seismic event, peak motions at sites without any measurement are estimated based on these limited number of observations made in the region. Otherwise when there are no such records, the peak motions are estimated using ground motion prediction equations. Since the peak motions are taken into account in the analysis as deterministic values, the uncertainty associated with the level of peak motion is not represented in the resulting fragility models. This is often considered to be an important limitation of the empirical approaches for development of fragility models (Crowley et al. 2008).

Most important novelty of the proposed method is the explicit representation of the uncertainty related to the peak motions that have caused the observed structural damages. The geospatial correlation of the peak motions at the sites of damage observations are explicitly modeled in the analysis.

The method presented here is based on evaluation of parametrically generated models by considering actual damage observations. When using post-earthquake damage data for a site, it is critical to account for the intra-event uncertainty associated with the ground motions experienced at that site. This intra-event uncertainty is due to the high geospatial variability of ground motion intensity and the low density of the strong ground motion recording stations at the damaged region. In the methodology presented here, the uncertainty associated with the level of ground motion intensity is treated explicitly. As a result, there is no need to introduce any deterministic assumptions related to the distribution of ground accelerations in the affected region. Such assumptions have a strong influence on the vulnerability models developed by using damage data collected at sites located far away from the strong ground motion recording stations.

Previously, a similar method has been developed for establishing vulnerability models for structures was developed by Yazgan (2012). These models were aimed at providing estimates of damage ratios of structures at various levels of shaking. The damage ratios –when defined in terms of cost measures– provide the estimated cost of repairing a damaged structure divided by the total cost of decommissioning and rebuilding. Here this approach is further extended to be utilized in fragility model development. In the fragility models, the likelihoods of exceeding specific damage grades are estimated as a function of shaking intensity. Furthermore in the application of the method presented here, a further enhanced damage observation set that cover both Düzce'99 and Bingöl'03 earthquakes are presented.

PROPOSED METHOD

The empirical fragility modeling method proposed here aims at establishing reliable fragility models for groups structures. In the context of this study, a fragility model is a mathematical model for estimating the conditional likelihood of the damage to a group of buildings exceeding a specific damage state given that they are subject to a specific shaking intensity level. The fundamental strategy is to first generate a set of alternative fragility models and subsequently to analyze the accuracy of each fragility model in capturing the observed damage distribution. The Bayesian analysis approach is utilized for this purpose (Ang and Tang, 1984). In the following, the approach is presented for the case of developing fragility model for a damage state *ds* for the sake of simplicity. However, the approach can be utilized for developing fragility models for multiple damage states as well. For this purpose, the same procedure needs to be repeated over for each damage state. The general layout of the proposed method is presented in Fig.1b.

2.1. Prior fragility models

The first step of the method is to establish a set of prior fragility models. In order to represent the fragility, any monotonically increasing mathematical function may be used. The log-normal distribution model is utilized in the following as an example case. The log-normal probability distribution model is a commonly used function to represent the seismic fragility of buildings. When this model is utilized with a set of parameters represented by $\overline{\theta}_i$, the conditional probability of $P\left[DS_j \geq ds \middle| SA_j = x, \Theta = \overline{\theta}_i\right]$ of the damage state DS_j of the structure located at site- j being equal to or exceeding the damage state *ds* conditioned on the pseudo-spectral acceleration (i.e. $S_a(T)$), SA_j being equal to x, is evaluated as follows:

$$P\left[DS_j \geq ds \middle| SA_j = x, \Theta = \overline{\theta}_i\right] = \Phi\left(\frac{\ln(x) - \lambda_i}{\zeta_i}\right) \tag{1}$$

where $\Phi(.)$ is the standard normal distribution function, Θ is the random variable representing model parameters, $\overline{\theta_i} = \{\lambda_i, \zeta_i\}$ is the vector of distribution parameters, λ_i and ζ_i are the parameters of fragility model-*i*. A fragility model obtained using this relationship is plotted in Fig.1a. The $S_a(T)$ considered in the fragility modeling typically corresponds to the spectral acceleration of an equivalent single-degree-of-freedom system with a natural period T of vibration equal to the expected fundamental period of vibration of the structures and a viscous damping that is equal to 5% of critical. However, it is important to note that the application of the proposed method is not restricted to any specific ground motion intensity parameter. Any ground motion intensity measure of interest (e.g. *PGA*, *PGV*, $S_d(T)$) can be adopted into the framework proposed here.

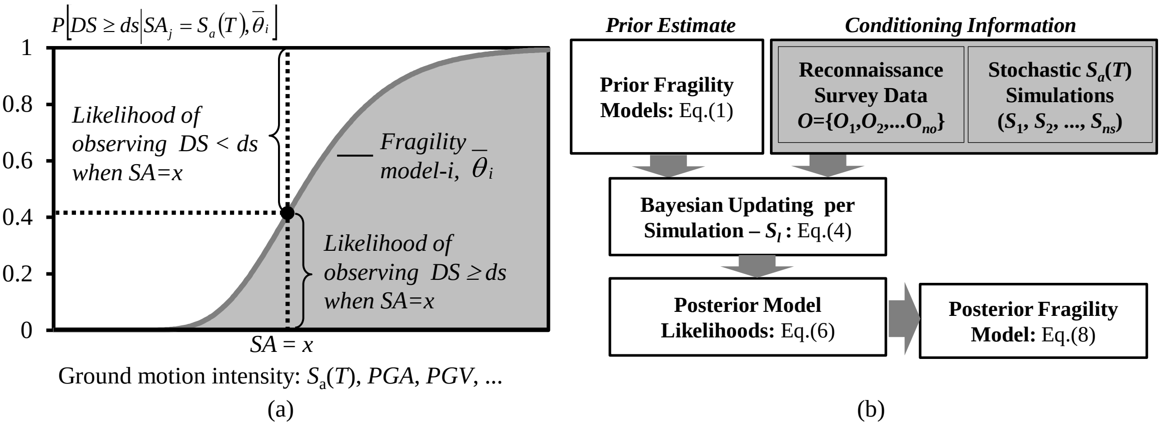


Figure 1. Definitions of the fundamentals: (a) a fragility model and (b) outline of the proposed method

2.2. Fragility model likelihoods conditioned on damage

Essence of the proposed method is to evaluate the likelihoods for alternative fragility models based on a given set of damage observations Fig.1b. For this purpose, first a set of alternative models are established using a wide range of plausible model parameter sets ($\bar{\theta}_1, \bar{\theta}_2$, ..., $\bar{\theta}_{nm}$). These are referred as prior models in the context of Bayesian analysis utilized here. Each of these parameters correspond to a model that portrays different fragility characteristics. For each prior model, a prior probability $P\left[\bar{\theta}_i\right]$ is assumed. A typical strategy is to assume equal likelihoods for all the plausible models. Subsequently, these prior probabilities are updated by considering damage observations from the post-earthquake reconnaissance surveys. If large sets of damage observations are available, the resulting updated likelihoods become insensitive to the assumed prior likelihoods.

In the framework proposed here, a damage observation at site is a binary event with two outcomes. One possible outcome is that –after the earthquake–the damage state DS_j of the building at site-*j* is equal to or greater than the damage state *ds*. The other outcome is observing that DS_j is lower than *ds*. The joint event *O* of all damage observations (i.e. O_1, O_2,...,O_{no}) made at *no* different sites, is mathematically defined as $O = \{O_1 \cap O_2 \cap \cdots \cap O_{no}\}$ where O_j is the damage observation event for the site-*j*. In the proposed method, the likelihoods $P\left[\bar{\theta}_i \middle| O\right]$ for the considered model parameters $\bar{\theta}_i$ conditioned on this joint event *O* are evaluated to establish a posterior model.

For the damage observation O_1 at site-1, likelihood $P\left[O_1 \middle| \bar{\theta}_i\right]$ can be estimated using Eq.(1), only if the $S_a(T)$ exhibited at site-1 (i.e. SA_1) is available. However, this $S_a(T)$ is known only if the accelerations are instrumentally recorded at the site during the earthquake. Very often, the closest site where the $S_a(T)$s are recorded instrumentally (e.g. strong motion station) lies within a distance of kilometers from the site where the damage observation is made. Investigation of the $S_a(T)$s recoded across geographic regions has indicated that the $S_a(T)$s that are recorded at two sites that are separated by distances in the range of a few kilometers can be significantly different with each other (Boore et al. 2003). As a result, the actual $S_a(T)$ exhibited the damage observation site cannot be estimated precisely unless there is a strong motion station within the close vicinity (i.e. closer than about 0.5 km). This leads to significant uncertainty related to the actual $S_a(T)$s that had affected the damaged buildings which are distributed over different sites in the region. The level of this uncertainty related to exhibited $S_a(T)$s is directly related to the coarseness of the strong motion station network in the affected region (Douglas, 2013).

An important novelty of the proposed method is the explicit consideration of the uncertainties associated with the $S_a(T)$s exhibited at the damage observtion sites. For this purpose, a stochastic simulation approach is utilized. The details of this approach is presented by Park et al. (2007). In the stochastic simulation of $S_a(T)$s, the instrumentally recorded accelerations –if there are any— are directly taken into account in the calculation of simulation parameters. Thus, the dispersion of the $S_a(T)$s at damage observation sites located close to strong motion stations are lower than that of the sites located far away from these stations. Moreover, the statistical correlations among the $S_a(T)$s for the pairs of sites are explicitly modeled as a function of the separation distances of the site pairs $\Delta_{i,j}$ (Fig.2a). For this purpose, the geospatial ground motion variability models (e.g. Boore et al. 2003, Goda and Hong, 2008) are utilized. This correlation model forms the basis of the stochastic simulation of *SA*s (Fig.2b).

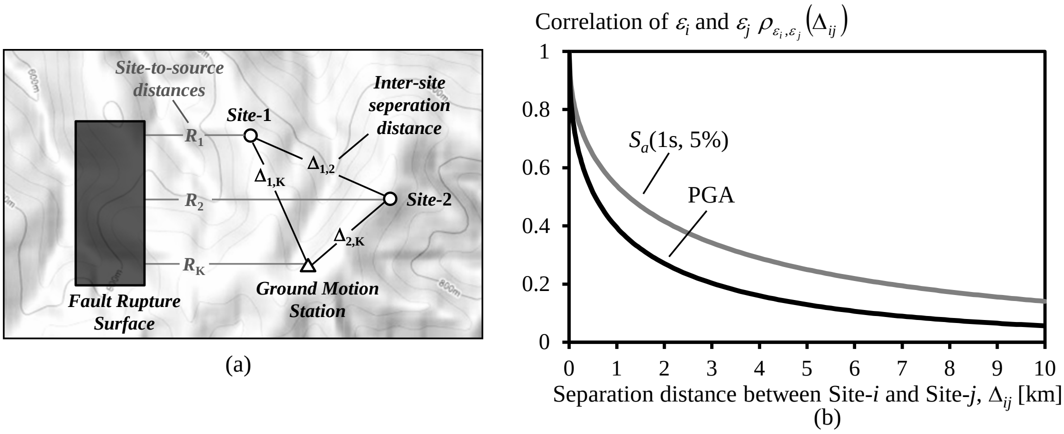


Figure 2. Definiton of the: (a) site-to-source distances and the inter-site separation distances for the considered sites and (b) the ε_i-ε_j correlation model as a function of the separation distance (Goda and Hong, 2008)

The results of the stochastic simulations are collected into matrix $\overline{SA}$. The element $sa(j;l)$ located at the row-j and column-l of this matrix is the $S_a(T)$ simulated for site-j in the stochastic realization-l. Each simulation can be considered as a randomly generated map of $S_a(T)$ values across the damaged sites like a stochastically simulated shake map. Accordingly, jth column, $\mathrm{col}_j(SA)$ of the matrix contains the vector of randomly simulated $S_a(T)$ values for site-j. For each pair of sites, the corresponding vectors of simulated $S_a(T)$s are correlated according to their separation distance $\Delta_{i,j}$. Following this notation and considering the damage observation O_1 at site-1 in simulation-l, the conditional likelihood of model parameter $\overline{\theta}_i$ can be evaluated as:

$$P\left[\overline{\theta}_i \middle| O_1, SA_1 = sa(1,l)\right] = \frac{P\left[O_1 \middle| \overline{\theta}_i, SA_1 = sa(1,l)\right] P\left[\overline{\theta}_i\right]}{\sum_{k=1}^{nm} P\left[O_1 \middle| \overline{\theta}_k, SA_1 = sa(1,l)\right] P\left[\overline{\theta}_k\right]} \tag{2}$$

In the equation above, $P\left[O_1 \middle| \overline{\theta}_i, SA_1 = sa(1,l)\right]$ is the conditional likelihood of the damage observation O_1 assuming parameters $P\left[\overline{\theta}_i\right]$ and assuming $SA_1 = sa(1,l)$ where $sa(1,l)$ is the $S_a(T)$ for site-1 obtained as a result of simulation-l. For a given site-j, the conditional likelihood of the corresponding damage observation O_j can be evaluated conditioned on $SA_j = sa(j,l)$ may be evaluated depending on the observed damage state as follows:

$$P\left[O_j \middle| \overline{\theta}_i, SA_j = sa(j,l)\right] = \begin{cases} P\left[DS_j \geq ds \middle| \overline{\theta}_i, SA = sa(j,l)\right] & \text{if } O_j : \left\{ds_j^* \geq ds\right\} \\ 1 - P\left[DS_j \geq ds \middle| \overline{\theta}_i, SA = sa(j,l)\right] & \text{otherwise} \end{cases} \tag{3}$$

where ds_j^* is the actual observed damage state of the building at site-j and ds is the damage state of interest in the fragility analysis. This notation basically suggests a grading being established for the order of damage states (i.e. None, Light, Moderate, etc.) that starts with lower grades and gradually reaches to higher grades. The conditional likelihoods $P\left[DS_j \geq ds \middle| \bar{\theta}_i, SA = sa(j,l)\right]$ in Eq.(3) are be evaluated by substituting $x = sa(j,l)$ in Eq.(1).

The conditional likelihoods of fragility model parameters $\bar{\theta}_i$ obtained using Eq.(2) are further updated in a recursive manner to obtain the likelihoods that are conditioned on the entire set of damage observations *O*. In the calculation of this likelihood, it is crucial to take into account the cross correlations among the *SA*s at the damage observation sites affected by the same earthquake. Here, the presence of correlation is taken into account by evaluating the fragility model likelihoods conditioned on individual simulations of *SA*s across all sites. Each simulation is thought as a realization of $S_a(T)$ distribution map in the affected area. Based on this concept and considering simulation-*l* S_l the conditional likelihood of fragility model parameter $\bar{\theta}_i$ can evaluated using the following recursive equation:

$$P\left[\bar{\theta}_i \middle| O_{1,2,\cdots j,j+1}, S_l\right] = \frac{P\left[O_{j+1} \middle| \bar{\theta}_i, S_l\right] P\left[\bar{\theta}_i \middle| O_{1,2,\cdots j}, S_l\right]}{P\left[O_{j+1} \middle| O_{1,2,\cdots j}, S_l\right]} \tag{4}$$

$$\text{where } P\left[O_{j+1} \middle| O_{1,2,\cdots j}, S_l\right] = \sum_{k=1}^{nm} P\left[O_{j+1} \middle| \bar{\theta}_k, S_l\right] P\left[\bar{\theta}_k \middle| O_{1,2,\cdots j}, S_l\right] \tag{5}$$

The conditional probability $P\left[\bar{\theta}_i \middle| O_{1,2,\cdots j}, S_l\right]$ at the right side of the equality is the posterior likelihood for fragility model $\bar{\theta}_i$ given the set of damage observations for the sites from site-*1* to site-*j* while the $P\left[\bar{\theta}_i \middle| O_{1,2,\cdots j,j+1}, S_l\right]$ is the conditional probability obtained after the inclusion of observation for site-(j+1) into the conditioning observation set. As it can be seen, this equation can be applied repeatitively to increment the set of conditioning damage observations by one in each application.

In order to evaluate the posterior likelihoods that are conditioned to the entire set of observations, Eqs.(4-5) are evaluated recursively as j=1, 2, ..., *no* until the total number of *no* damage observations are included in the damage observation set. Note that the conditional likelihood $P\left[O_{j+1} \middle| \bar{\theta}_i, S_l\right]$ in Eq.(4) is evaluated by substituting $x=sa(j+1,l)$ into Eq.(1). As a result of the recursive evaluation of Eq. (4), the posterior likelihood $P\left[\bar{\theta}_i \middle| O, S_l\right]$ for $\bar{\theta}_i$ that is conditioned on the joint event *O* of all damage observations and the set of stochastically simulated *SA*s from simulation-*l* (i.e. S_l).

In order to obtain likelihoods that are conditioned on the entire set of simulations, the conditional probabilities $P\left[\bar{\theta}_i \middle| O, S_l\right]$ related to each simulation S_l are marginalized over the entire set of simulations (i.e. S_1, S_2, ..., S_{ns}) as follows:

$$P\left[\bar{\theta}_i \middle| O\right] = \sum_{l=1}^{ns} P\left[O_{j+1} \middle| \bar{\theta}_i, S_l\right] P[S_l] \tag{6}$$

$$\text{where } P[S_l] \approx 1/ns \tag{7}$$

where $P[S_l]$ is the likelihood assumed for the simulated set of *SA*s related to simulation-*l* and *ns* is the total number of stochastic simulations. In otherwords, it is the total number of columns of the matrix $\overline{SA}$ explained in the preceeding seciton. The conditional probabilities $P\left[\bar{\theta}_i \middle| O\right]$ obtained using Eqs.(6-7) are then used for establishing the final posterior fragility model as shown next.

2.3. Posterior fragility model conditioned on the available damage data

The posterior fragility model is established by convolution of the prior fragility models with the corresponding conditional likelihoods $P\left[\overline{\theta}_i \middle| O\right]$ evaluated using Eq.(6). For the consistent combination of fragility curves in accordance with their likelihoods, the approach proposed by Shinozuka et al. (2000) is utilized for this purpose. Based on this approach, the posterior fragility model conditioned on a specific set of damage observations may be obtained as:

$$P\left[DS \geq ds \middle| SA = x, O\right] = \sum_{l=1}^{ns} P\left[DS \geq ds \middle| SA = x, \overline{\theta}_i\right] P\left[\overline{\theta}_i \middle| O\right] \tag{8}$$

where $P\left[DS \geq ds \middle| SA = x, \overline{\theta}_i\right]$ and $P\left[\overline{\theta}_i \middle| O\right]$ are the likelihoods obtained using Eqs.(1) and (6), respectively. It is important to note that, the relationship provided in the above relationship can be utilized with any monotonically increasing function that is substituted into Eq.(1) as the fragility function. Thus, it is not only applicable for the case of log-normal distribution function based fragility models.

The posterior fragility model that is obtained using Eq.(6) is the optimal model that is conditioned on the a set of damage observations by explicitly taking into account the uncertainty associated with the $S_a(T)$s at the damage observation sites. This resulting posterior fragility model is expected to provide accurate estimates of the losses related to scenario seismic events of interest. Moreover if used together with hazard estimates for the region, the posterior fragility model obtained using the proposed procedure is expected to provide accurate estimates of the actual risk associated with the considered portfolio of structures. In the next section, an example application of the proposed procedure is presented.

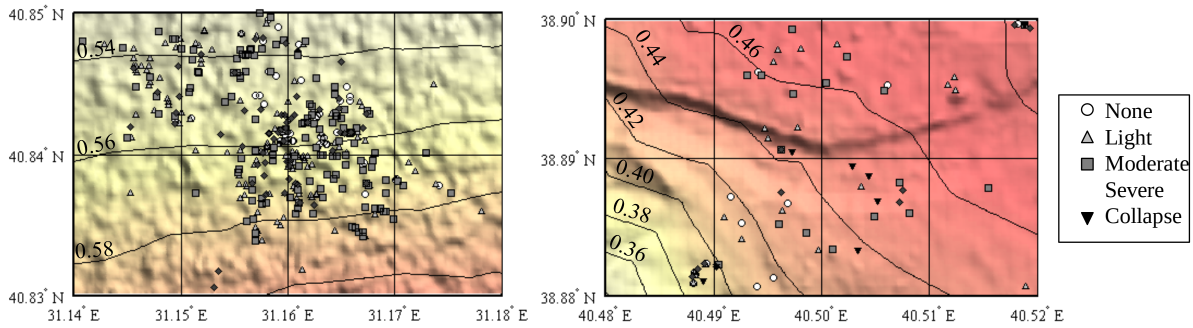
EXAMPLE APPLICATION

The proposed method for establishing fragility models is utilized here to establish fragility models for the existing multi-story reinforced concrete moment-resisting frame building stock in Turkey. Fragility models are established for three structural classes: *RC12*, *RC34*, and *RC56*. These classes correspond to buildings having total number of stories in the intervals: 1-2, 3-4, and 5-6, respectively. For each structural class, the fragility curves are developed for three damage states: *Light*, *Moderate*, and *Severe/Collapse*.

The damage observations made after two major earthquakes are considered in this example application. These are: (1) 1999 *M*7.1 Düzce (Turkey) and (2) 2003 *M*6.4 Bingöl (Turkey) earthquakes. Specifically, the damage observations reported in SERU-METU databases are utilized (SERU 2003a,b). In these databases, the damage grades and the main structural properties (e.g. number of stories, structural system, and latitude-longitude coordinates) are listed for the inspected buildings. The distribution of damage grades and numbers of stories are presented in Table.1. From the total set of 516 buildings, 425 buildings are those which were damaged during Duzce’99 and 91 buildings are the ones damaged during Bingol’03 earthquake. Geographic distributions of these building across the affected regions are plotted in Figs.3a and 3b for Düzce’99 and Bingöl’03 databases, respectively. The considered buildings where the damages were inspected had all been constructed before 1999. Therefore, they were not designed and constructed according to the improved ductility provisions introduced after the 1999 revision of the seismic design code of Turkey. Other details related to the utilized damage database and the definitions of the considered damage grades can be found elsewhere (Sucuoğlu et al., 2007). Currently, there is an ongoing effort to further enhance the considered database of damage observations by reviewing available reconnaissance reports related to past earthquakes. Future enhancement of the considered database is expected to lead to more reliable fragility estimates:

Table 1. Number of stories and the observed damage states ds^* of the buildings

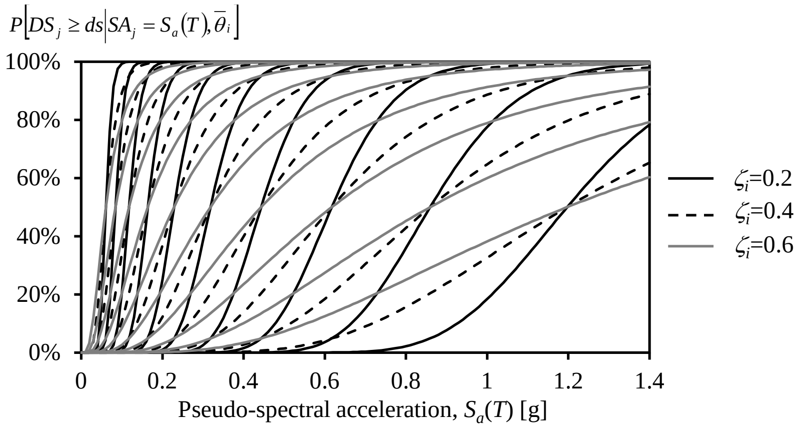
Total number of stories	Damage State, ds^* None	Light	Moderate	Severe	*Total*
1-2	9	15	5	3	*32*
3-4	46	122	107	58	*333*
5-6	21	40	62	28	*151*
Total	*76*	*177*	*174*	*78*	*516*



(a) (b)

Figure 3. Map of damage observation sites and median pseudo-spectral accelerations S_a(0.35s; 5%) [g] estimated using GMPE:(a)Duzce 1999 M7.1 and (b)Bingol 2003 M6.4 earthquakes

The parameters of the prior fragility models are established by reviewing the existing fragility models proposed for reinforced concrete moment resisting frame buildings. As a result of this review, the prior fragility curves presented in Fig.4a were established. The set of fragility curve parameters λ_i, ζ_i utilized in the construction of these curves are listed in Fig.4b. Each curve is established by substituting the related parameters into Eq.(1). The 30 curves resulting from this evaluated are considered as the prior models in the Bayesian analysis. Each fragility model was assigned to have equal prior likelihoods (i.e. $P\left[\bar{\theta}_i\right]$= 1/30) as shown in Fig.5b. Subsequently, these likelihoods were updated based on damage observations using the proposed procedure.



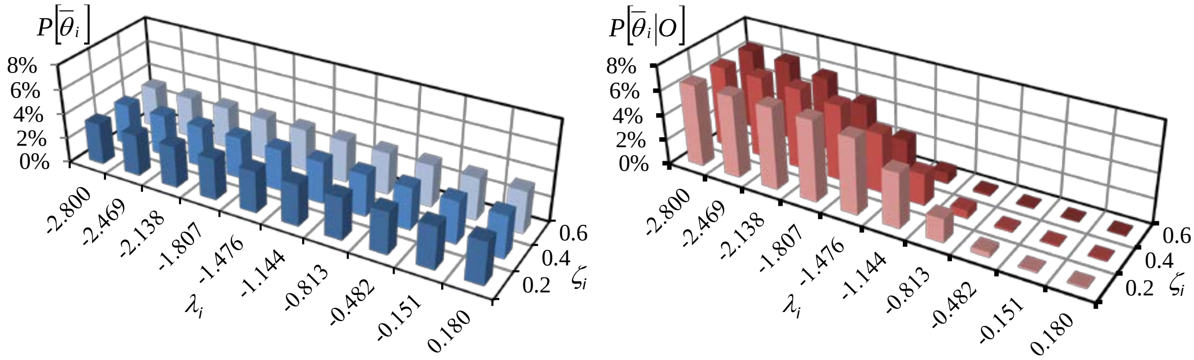
(a)

#	λ_i	ζ_i
1	-2.800	0.2
2	-2.469	0.4
3	-2.138	0.6
4	-1.807	
5	-1.476	
6	-1.144	
7	-0.813	
8	-0.482	
9	-0.151	
10	0.180	

(b)

Figure 4. Prior fragility models: (a) curves obtained for the considered set of λ_i, ζ_i pairs, and (b) set of λ_i, ζ_i values considered in the analysis

The pseudo-spectral accelerations $S_a(T)$ was adopted as the ground motion intensity parameter. The period of vibration was defined for each structural class based on the expected periods of vibration. For the structural classes: *RC12*, *RC34*, and *RC56* vibration periods were defined as T=0.15s, T=0.35s and T=0.55s, respectively. The pseudo-spectral accelerations corresponding to these periods were adopted as the measures of shaking intensity. The median $S_a(T)$s for 0.35s (i.e. $S_a(T = 0.35\text{s})$) were estimated using the ground motion prediction equation (GMPE) by Campbell and Bozorgnia (2008). In order to estimate the median $S_a(T)$s for Duzce'99 and Bingol'03 earthquakes the rupture plane models by Delouis et al. (2004) and Akkar et al. (2005) were utilized. The V_{s30} values at the sites were estimated using the topography based method by Allen and Wald (2009). The median S_a(0.35s) estimates obtained using the GMPE model are presented as contour lines in Fig.3a.



(a) (b)

Figure 5. Fragility model likelihoods for *Light* damage state fragility model of *RC34* class: (a) prior likelihoods $P\left[\bar{\theta}_i\right]$ assumed for the models, and (b) posterior likelihoods $P\left[\bar{\theta}_i \middle| O\right]$ obtained using Eq.(6)

The ground motions exhibited during the Düzce'99 and Bingöl'03 earthquakes were recorded by 3 strong ground motion stations located in the affected regions. These stations were operated by the National Strong Ground Motion Network of Turkey had the Station-ID numbers: 8101, 1201 and 2406 (AFAD, 2014). These ground motion records were explicitly taken into account in the stochastic simulation of the *SA*s that have affected the sites where the damage was observed. The method presented by Park et al. (2007) is used for this purpose. In order to model the spatial correlation of $S_a(T)$s at the damage observation sites, the correlation model by Goda and Hong (2008) is utilized in this example. Using this correlation model, a total of 50 $S_a(T)$ distributions are stochastically simulated at 516 sites where the damage observations were made. For each $S_a(T)$ distribution and fragility model $\bar{\theta}_i$, the likelihood $P\left[\bar{\theta}_i \middle| O, S_l\right]$ is evaluated using Eq.(4). Subsequently, the posterior likelihoods $P\left[\bar{\theta}_i \middle| O\right]$ are evaluated using Eq.(3). In this evaluation, each simulation is assigned equal probability (i.e. $P[S_l]$ = 1/50 = 2%). The posterior fragility models are obtained using the approach presented in the previous section.

Posterior fragility models obtained from the evaluation of Eq.(6) are presented in Fig.5. It is important to note that each fragility curve is based on a different measure of ground motion intensity (i.e. corresponding to different periods of vibration). Therefore, it is not possible to compare the curves among each other directly. However, it can be seen that the light damage is sustained at lower $S_a(T)$s compared to moderate and severe damages, as expected. The fragility curves presented in Fig.5 are obtained by considering 516 damaged buildings from two earthquakes. Hence, they should be considered as preliminary findings. In the future, more reliable fragility curves can be established using the proposed method by gathering larger sets of damage observations than the set utilized here.

CONCLUSIONS

A method for building fragility models that explicitly takes into account the spatial variability of ground motion intensity is proposed. Using the proposed method, fragility models for groups of structures can be developed based on the damage observations made after significant earthquakes. The method relies on the evaluation of likelihoods for a set of prior fragility models based on these post-earthquake damage observations.

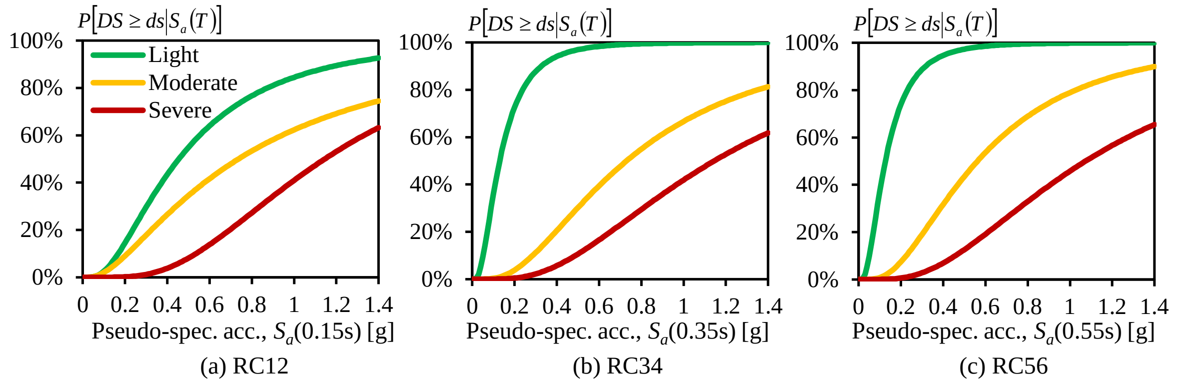


Figure 6. Fragility curves obtained for the: (a) RC12: 1-2 story RC frames, (b) RC34: 3-4 story RC frames, and (c) RC56: 5-6 story RC frames.

In order to calculate the likelihoods for alternative models, Bayesian analysis approach is utilized. First, a set of prior likelihoods are assumed. After that, these prior likelihoods are updated conditional on the damage observations. The uncertainty in the peak motions experienced at the observation sites are taken into account through stochastic simulation. In this simulation, the geospatial variability of intensity values is explicitly taken into account in the analysis. In order to present an example application of the proposed method, fragility models are established for RC moment resisting frame building in Turkey. In this application, the damage observations reported for the 1999 *M*7.1 Düzce (Turkey) and 2003 *M*6.4 Bingöl earthquakes are considered.

Following set of conclusions can be listed regarding the proposed fragility modeling method and its example application:

- The uncertainty associated with the peak ground motions experienced at the sites of damage observations is explicitly taken into account the proposed fragility modelling approach. As a result, there is no need for introducing deterministic estimates of intensity values. This is especially important for the damage observation sites located distantly from the sites of strong motion stations.

- The explicit modelling of geospatial variability and correlation of peak motion levels enable reducing the possible biases that may result from damage observation sites being closely spaced. Therefore, the additional uncertainty due to this sampling bias can be directly taken into account in the analysis.

- Proposed fragility modeling approach is highly flexible. It can be used for establishing fragility models for different types of structures given that post-earthquake damage observations are available. The method is not limited to any specific functional form.

- Application of the method to 516 buildings damaged during 1999 M7.1 Düzce (Turkey) and 2003 M6.4 Bingöl (Turkey) Earthquakes provided promising results. It was seen that the likelihoods (i.e. degrees of belief) associated with alternative models were updated when the damage observations are taken into account in the analysis. As a result of this example application, the set of posterior fragility curves presented in Fig.6 were obtained.

The fragility models developed using the proposed method can be directly implemented in to existing seismic loss estimation frameworks and risk management platforms. The potential losses and the level of risk estimated using these fragility models would directly reflect the fragility characteristics of the considered type of buildings as inferred from previous damage observations. Therefore, they are expected to provide reliable estimates of potential losses.

Acknowledgements

The author would like to acknowledge the Structural Engineering Research Unit (SERU) of Middle East Technical University (METU) for providing the Düzce'99 and Bingöl'03 damage databases.

REFERENCES

AFAD (2014) Strong ground motion database of Turkey, Earthquake Department, Turkish Disaster and Emergency Management Presidency (AFAD), Ankara, Turkey

Ang AHS, Tang WH (1984) Probability Concepts in Engineering Planning and Design: Vol. II Decision, Risk, and Reliability, John Wiley & Sons, New York

ATC (1985) Earthquake damage evaluation data for California, ATC-13, *Technical report*, Applied Technology Council (ATC), Redwood City, California

Abrahamson NA, Bolt BA, Darragh RB, Penzien J, Tsai YB, (1987) "The SMART I accelerograph array (1980-1987): A review, *Earthquake Spectra* 3(2):263–287

Akkar S, Boore D, Gülkan P (2005) "An evaluation of the strong ground motion recorded during the May 1, 2003 Bing¨ol, Turkey earthquake," *Journal of Earthquake Engineering* 9(2):173–197

Allen TI,. Wald DJ (2009) "On the use of high-resolution topographic data as a proxy for seismic site conditions (Vs30)," *Bulletin of the Seismological Society of America* 99(2A):935–943

Boore DM, Gibbs JF, Joyner WB, Tinsley JC, Ponti DJ (2003) "Estimated ground motion from the 1994 Northridge, California earthquake at the site of the Interstate 10 and La Cienega Boulevard Bridge collapse,West Los Angeles, California", *Bulletin of the Seismological Society of America* 93(6):2737–2751

Campbell KW, Bozorgnia Y (2008) "NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s", *Earthquake Spectra* 24(1):139–171

Crowley H, Stafford JP, Bommer JJ (2008) "Can earthquake loss models be validated using field observations?," *Journal of Earthquake Engineering,* 12(7):1078–1104

Delouis B, Lundgren P, Giardini D (2004) Slip distributions of the 1999 Düzce (Mw 7.2) and Izmit (Mw 7.6) earthquakes on the North Anatolian Fault (Turkey): A combined analysis, *Technical report*, ETH Zurich, Zurich, Switzerland.

Douglas J (2013) "Seismic network design to detect felt ground motions from induced seismicity", *Soil Dynamics and Earthquake Engineering* 48:193–197

Goda K, Hong H (2008) "Spatial correlation of peak ground motions and response spectra", *Bulletin of the Seismological Society of America* 98(1):354–365

Park J, Bazzurro P, Baker JW (2007) "Modeling spatial correlation of ground motion intensity measures for regional seismic hazard and portfolio loss estimation", *Proceedings of the International Conference on Applications of Statistics and Probability in Civil Engineering,ICASP10*, Tokyo, Japan

Porter KA (2003). "Seismic Vulnerability", Earthquake Engineering Handbook (Chen W.-F. and Scawthorn C. Eds), CRC Press, Boca Raton, Florida

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

SERU (2003a), Düzce database, Tech. rep., Structural Engineering Research Unit (SERU), Middle East Technical University (METU), Ankara, Turkey.

SERU (2003b), Bingöl database, Tech. rep., Structural Engineering Research Unit (SERU), Middle East Technical University (METU), Ankara, Turkey.

Shinozuka M, Feng MQ, Lee J, Naganuma T (2000) "Statistical analysis of fragility curves", *Journal of Engineering Mechanics*, ASCE 126 (12):1224–1231

Singhal A, Kiremidjian AS (1996) "Method for probabilistic evaluation of seismic structural damage," *Journal of Structural Engineering*, *ASCE* 122(12):1459–1467

Singhal A, Kiremidjian AS (1998) "Bayesian updating of fragilities with application to RC frames," *Journal of Structural Engineering* 124 (8): 922–929

Sucuoğlu H, Yazgan U, Yakut A (2007) "A screening procedure for seismic risk assessment in urban building stocks," *Earthquake Spectra* 23(2): 441–458

Yazgan U (2012) "A new approach to building stock vulnerability modeling based on Bayesian analysis", *15th World Conference on Earthquake Engineering*, Lisbon, Portugal

Zerva A (2009) Spatial Variation of Ground Motions: Modeling and Engineering Applications, CRC Press, New York.