Time Domain Probabilistic Seismic Risk Analysis for Earthquake Soil Structure Interacting Systems

H. Wang[(1)], H. Yang[(2)], B. Jeremic[(3)]

[(1)] *Graduate Student Researcher, University of California Davis, hexwang@ucdavis.edu*
[(2)] *Postdoctoral Researcher, University of California Davis, hhhyang@ucdavis.edu*
[(3)] *Professor, University of California Davis, jeremic@ucdavis.edu*

Abstract

Presented is a methodology for probabilistic seismic risk analysis (PSRA) that explicitly takes into account all uncertainties emanating from uncertain source, uncertain path, and uncertain soil-structure system behavior. Presented methodology is based on full probabilistic modeling and simulation of earthquake, soil and structure interacting (ESSI) system, by solving stochastic partial differential equations in analytic form, using Stochastic Elastic-Plastic Finite Element Method. Probabilistic Seismic Hazard Analysis (PSHA) is directly linked with structural fragility analysis and avoids the selection of Intensity Measures (IMs). Using presented comprehensive approach, seismic risk analysis is performed using the same underlying modeling assumption for all components of PSRA, namely, source, path, site and structure.

Keywords: Seismic risk analysis, Stochastic ground motion, Stochastic Elastic-Plastic Finite Element Method

1. Introduction

Numerous research efforts in past several decades have established a framework for Performance-based Earthquake Engineering (PBEE) [1–4]. Seismic design has gradually changed from deterministic strength-based design to probabilistic deformation/performance-based design that accounts for all sources of uncertainties in the system [2, 4]. In the probabilistic PBEE framework by Pacific Earthquake Engineering Research Center (PEER), there are four main components [3, 4]: (1) Hazard analysis for intensity measure (IM) of ground motions; (2) Structural fragility analysis for engineering demand parameter (EDP); (3) Damage analysis characterized by damage measure (DM); (4) Loss analysis for decision variable (DV).

Probabilistic seismic risk analysis is a crucial part of PBEE. Conventionally seismic risk is calculated as the integral of seismic hazard and structural fragility with respect to intensity measure(s). The underlying assumption is that damaging effects of ground motions upon structures can be represented by intensity measure (IM) as a proxy, which is summed over all seismic sources. The uncertainty in seismic motions is supposed to be quantified by the variability of IM. However, as revealed by many studies [5-7], it is difficult to find a proper intensity measure in engineering practices that can fully describe the influence of uncertain seismic motions upon engineering objects. For example, spectral acceleration at fundamental period $Sa(T_0)$ is commonly adopted as IM for building structure, which is based on frequency domain, linear elastic dynamic response of single degree of freedom (DOF) system. When nonlinear, inelastic and/or higher mode response is concerned, the use of $Sa(T_0)$ is questionable. Grigoriu [5] showed that generally $Sa(T_0)$ is weakly dependent with engineering demand parameters for realistic structures and fragilities defined as functions of $Sa(T_0)$ have large uncertainties and of limited practical use. Furthermore, for many other types of engineering objects (e.g., geosystem, dams, etc.), it is even more difficult to determine a suited IM for risk analysis. For example, it has been contentiously argued whether peak ground acceleration (PGA), peak ground velocity (PGV), Arias intensity (AI) or cumulative absolute velocity (CAV) should be selected for deformation analysis of dam embankment [6]. Stafford and Bommer [7] investigated different intensity measures and found that they are generally not strongly correlated, which indicates that knowledge of one IM distribution is not sufficient to describe any of the other ground-motion characteristics. It is noted that vector-valued hazard analysis [8] has been put forward to mitigate the aforementioned issue of IM. Nevertheless, it is rarely performed in practices for risk analysis. The difficulty lies in fragility computation. The fragility becomes a function of vector IMs (e.g., a fragility surface for two IMs), which requires a large number of structural analyses to be quantified. Properly choosing vector IMs is also a problem. Many times, even if proper IMs, such as AI and CAV, are identified, additional efforts are still needed to develop ground motion prediction equations (GMPEs) for these IMs and their correlation.

Another significant issue in conventional probabilistic seismic risk analysis is the use of Monte Carlo (MC) simulation. For instance, with incremental dynamic analysis (IDA) [9], fragility curve is developed through hundreds of structural analysis with deterministic sampling of uncertain material properties and uncertain ground excitations, scaled to different IM levels. MC approach, though theoretically straightforward, is computationally very expensive for fragility calculation of nonlinear systems with many uncertain parameters. Especially for those critical structures that demand very low risk level, even larger number of MC runs are required to obtain stable tail of probabilistic distribution of EDP due to the low convergence rate of MC approach. The same issue of MC approach also limits the application of physics-based seismic waveform modeling techniques [10-11] into hazard/risk analysis. Millions of MC earthquake scenarios over regional geology have to be simulated using deterministic wave propagation programs, such as CyberShake [10] considering uncertain kinematic sources, crustal geology and site conditions. Maechling et al. [11] estimated that “it would require 300 million CPU-hours and well over 100 years to complete all the simulations needed to calculate a PSHA hazard curve”.

To solve the above problems, a novel time domain intrusive framework for probabilistic seismic risk analysis has recently been established [12-14]. The framework is built upon stochastic modeling of Fourier amplitude spectra (FAS) of seismic motions [15]. Time series stochastic motions are simulated from stochastic FAS and Fourier phase derivative [16-17]. These uncertain motions are modeled as non-stationary random process and represented with Polynomial Chaos Karhunen-Loève (PC-KL) expansion [18]. In this way, all the uncertainties and important characteristics of seismic motions are captured through the random process without using any IM as simplified proxy. PC characterized random process motions are directly propagated into uncertain structural system. Dynamic probabilistic structural response is efficiently solved using Galerkin stochastic elastoplastic finite element method (SEPFEM) [19-21]. It is noted that MC method is a non-intrusive approach in the sense that no modification to the underlying deterministic solver is required. Though intrusive Galerkin SEPFEM requires new developments based on variational formulation of the underlying stochastic partial differential equations (SPDE), it guarantees optimal convergence rate and is generally much more efficient than Monte Carlo simulations [21].

Recently, engineering seismologists began to develop and promote GMPEs based on FAS [22-24]. Compared with traditional proxy, i.e., spectral acceleration Sa, FAS is a more direct representation of ground motion. The scaling of FAS is preferable and more clearly related to the underlying earth physics. As envisioned by Abrahamson [25], one of the major changes for seismic hazard/risk analyses in the near future will be the shift from ergodic Sa ground motion model to non-ergodic, site specific FAS model. Correspondingly, seismic risk analysis in PBEE can also change from frequency domain IM-based, Monte Carlo driven approach to time domain intrusive approach. To the best knowledge of the authors', there has not been any seismic risk analysis based on GMPE of FAS. Therefore, this paper incorporates several emerging GMPEs [22-24] of FAS into the developed time domain intrusive framework for seismic risk analysis. The stochastic modeling of uncertain motions are largely simplified with FAS GMPE, making the time domain framework readily applicable for practical seismic risk analysis.

2. Time domain probabilistic seismic risk analysis

Established framework for time domain intrusive probabilistic seismic risk analysis includes four steps: (1) Seismic source characterization (SSC), (2) Time domain stochastic ground motion modeling, (3) Stochastic finite element analysis, (4) Seismic risk computation.

Seismic source characterization quantifies the uncertainty in earthquake scenarios so that the probabilistic scenario space $\lambda(M, R, \Theta)$ for a given engineering site can be discretized into N mutually exclusive events as $\lambda_i (M_i, R_i, \Theta_i)$, where λ_i is the annual rate of the i^{th} scenario parameterized by magnitude M_i, distance R_i and other metrics Θ_i, e.g., style of fault, depth-to-top of rupture Ztor, and so forth. N is the total number of seismic scenarios. Many PSHA program can perform SSC. At the second step, time domain uncertain motions are simulated from stochastic FAS and Fourier phase derivative for each scenario $\{M_i, R_i, \Theta_i\}$ following the methodology shown in section 3. These uncertain motions are modeled as random process and characterized with Polynomial Chaos Karhunen-Loève (PC-KL) expansion. Stochastic FEM analysis is performed with PC characterized random excitations as seismic input, which provides complete probabilistic evolution of structural response. Failure probability $P(EDP > z|M_i, R_i, \Theta_i)$ of the selected performance indicator EDP is determined from the probabilistic dynamic structural response. Finally, seismic risk can be calculated following Equation 1:

$$\lambda(EDP > z) = \sum_{i=1}^{N} \lambda_i(M_i, R_i, \Theta_i) P(EDP > z|\mathbf{\Gamma}_i) \tag{1}$$

The above Equation 1 requires N PC-KL characterization of uncertain motions and follow-up stochastic FEM analyses. It is noted that uncertain seismic motions from N individual scenarios can also be combined into a single weighted ensemble population. Such ensemble population can be then characterized as a single random process with generally larger dimension of Hermite PCs. As a result, seismic risk can be calculated from single SFEM analysis. See Wang et al. [12] for more detailed formulation.

3. Time domain stochastic ground motion modeling

Time domain uncertain motions are inversely Fourier synthesized from uncertain FAS and Fourier phase spectrum. Stochastic FAS is modeled as Lognormal distributed random field in frequency domain, whose marginal median and standard deviation is given by GMPE of FAS [22-24]. It is noted that the inter-frequency correlation of the FAS random field is also investigated recently [26-27]. Adopted in this study are FAS GMPE by Bora et al. [22] (referred to as Bora15 model) and inter-frequency correlation model of Bayless and Abrahamson [27]. In Bora15 model, marginal median of FAS is correlated to moment magnitude *Mw*, stress drop $\Delta\sigma$, Joyner-Boore distance R_{JB}, time-averaged shear-wave velocity in upper 30m of site V_{S30} and site attenuation parameter k_0 as Equation 2:

$$lnFAS(f) = c_0 + c_1 M_w + c_2 M_w^2 + c_3 ln(\Delta\sigma) + (c_4 + c_5 M_w) ln(\sqrt{R_{JB}^2 + c_6^2}) \\ - c_7 \sqrt{R_{JB}^2 + c_6^2} + c_8 ln(V_{S30}) - c_9 \kappa_0 + \delta_{total}(f) \quad (2)$$

Fourier phase derivative is modeled as logistic distributed random field with exponential correlation structure. The GMPE of logistic distribution parameters are also statistically derived using NGA-West database [17]. The methodology of time domain stochastic ground motion modeling based on FAS GMPE is summarized in Figure 1:

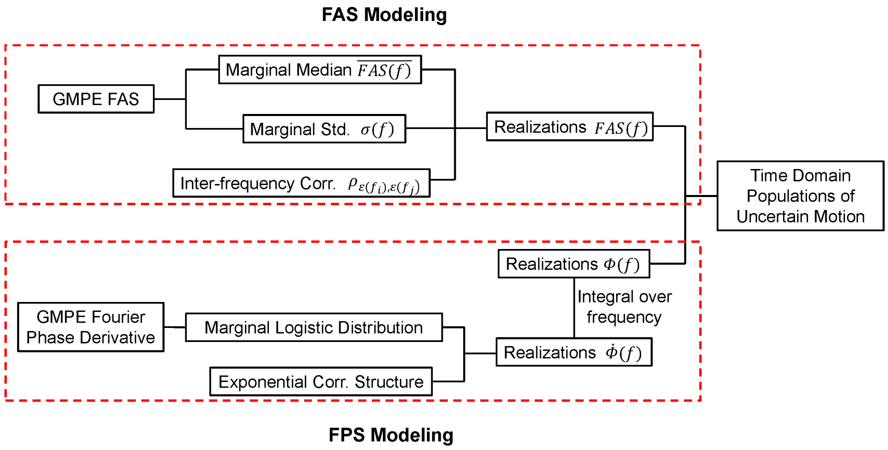


Fig. 1 Methodology for time domain stochastic ground motion modeling based on FAS GMPE adapted from Wang et al. [14]

4. Galerkin stochastic elastoplastic finite element method

In addition to spatial and temporal discretization in conventional deterministic FEM, Galerkin stochastic FEM introduces probabilistic discretization of random space using orthogonal polynomial chaos [19-20]. Specifically in this study SFEM is formulated based on Hermite polynomial chaos and implemented in high performance, high fidelity FEM simulator RealESSI [28-29]. Stochastic elastoplastic finite element method can account for not only random material property, i.e., left-hand-side stiffness term, but also random seismic excitation, i.e., right-hand-side forcing term, both of which are characterized as heterogeneous random field in spatial domain or non-stationary random process in time domain $R(\boldsymbol{x},\theta)$. Here $\boldsymbol{x}$ is the general coordinate and θ denotes the uncertainty.

Using the orthogonality of Hermite polynomial chaos (PCs), random field/process $R(\boldsymbol{x},\theta)$ with any type of marginal distribution can be represented as:

$$R(\boldsymbol{x},\theta)=\sum_{i=0}^{P}R_i(\boldsymbol{x})H_i(\gamma(\boldsymbol{x},\theta)) \tag{3}$$

where $R_i(\boldsymbol{x})$ is the deterministic PC coefficients field and $H_i(\gamma(\boldsymbol{x},\theta))$ is the Hermite PC bases constructed from kernel Gaussian random field $\gamma(\boldsymbol{x},\theta)$. The correlation structure of the original random field is then mapped into the correlation of kernel Gaussian random field $\gamma(\boldsymbol{x},\theta)$, which is further decomposed through Karhunen Loève theorem:

$$\gamma(\boldsymbol{x},\theta)=\sum_{i=1}^{M}\sqrt{\lambda_i}f_i(\boldsymbol{x})\xi_i(\theta) \tag{4}$$

where λ_i and $f_i(\boldsymbol{x})$ are the eigen values and eigen functions of the Gaussian covariance kernel of $\gamma(\boldsymbol{x},\theta)$ satisfying Fredholm integral equation of the second kind [18]. By combining Eq. (3) and (4), multidimensional Hermite PCs expression of random field/process $\gamma(\boldsymbol{x},\theta)$ can be synthesized.

Applying the above PC-KL expansion with respect to uncertain stiffness field $E(\boldsymbol{x},\theta)$ and uncertain force $f_m(t,\theta)$:

$$E(\boldsymbol{x},\theta)=\sum_{k}E_k(\boldsymbol{x})\Psi_k(\{\xi_r(\theta)\}) \tag{5}$$

$$f_m(t,\theta)=\sum_{l}f_{ml}(t)\psi_l(\{\xi_r(\theta)\}) \tag{6}$$

The unknown uncertain displacement random field $u(\boldsymbol{x},t,\theta)$ can also be represented as:

$$u(\boldsymbol{x},t,\theta)=\sum_{i}\sum_{j}N_i(\boldsymbol{x})u_{ij}(t)\phi_j(\{\xi_r(\theta)\}) \tag{7}$$

By plugging Equations (5), (6) and (7) into conventional deterministic FEM discretized formulation and applying Galerkin projection technique, system of ordinary differential equations (ODEs) of unknown displacement PC coefficients u_{nj} can be derived as:

$$M_{minj}\ddot{u}_{nj}+K_{minj}u_{nj}=F_{mi} \tag{8}$$

where

$$M_{minj}=\sum_{e}\int_{D_e}N_m(\boldsymbol{x})\rho(\boldsymbol{x})N_n(\boldsymbol{x})dV\ \langle\phi_i\phi_j\rangle \tag{9}$$

$$K_{minj}=\sum_{k}\sum_{e}\int_{D_e}B_m(\boldsymbol{x})E_k(\boldsymbol{x})B_n(\boldsymbol{x})dV\ \langle\Psi_k\phi_i\phi_j\rangle \tag{10}$$

$$F_{mi}=\sum_{l}f_{ml}\langle\psi_l\phi_i\rangle \tag{11}$$

It is noted that in the above equations $\langle\phi_i\phi_j\rangle$ and $\langle\Psi_k\phi_i\phi_j\rangle$ are double and triple products of Hermite PCs, which are pre-computed and used to construct stochastic expanded stiffness and mass matrix. The above formulation is complete for linear elastic problem with constant stiffness. However, for nonlinear inelastic problem, additional probabilistic constitutive formulations are required to close the loop. Here elastoplastic material model with vanishing elastic region is adopted in conjunction with Armstrong-Fredrick kinematic hardening [30]. In incremental form, Armstrong-Fredrick relation between inter-story restoring force F^R and inter-story drift η can be expressed as:

$$dF^R=H_a\,d\eta-C_rF^R|d\eta| \tag{12}$$

where H_a and C_r are model parameters with evolving tangential stiffness $E(F^R)$ as a function of inter-story restoring force F^R :

$$E(F^R) = H_a \pm C_r F^R \tag{13}$$

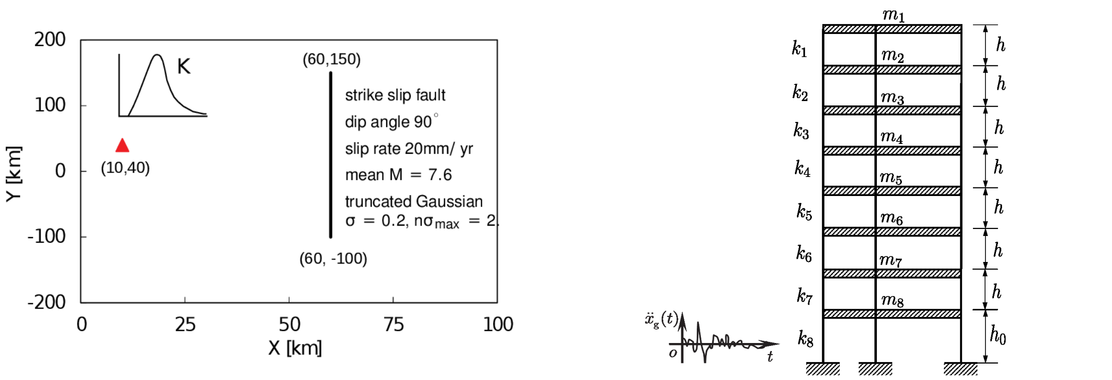
The + sign is taken for negative inter-story drift increment $d\eta$ while – sign is taken for positive inter-story drift increment $d\eta$. In the case of uncertain parameters $H_a(\boldsymbol{x},\theta)$, $C_r(\boldsymbol{x},\theta)$ and uncertain drift increment $d\eta(\boldsymbol{x},\theta)$, PC-KL expansion of these random fields are applied with Hermite PC bases $\varphi_i(\{\xi_r(\theta)\})$. Plugging these PC quantified random fields into Equation (13) and performing Galerkin projection, the evolving PC coefficients of probabilistic stiffness can be derived as follows:

$$E_i(\boldsymbol{x}) = H_{ai}(\boldsymbol{x}) \pm \frac{1}{\mathrm{Var}[\varphi_i]} C_{rl}(\boldsymbol{x}) F_n^R(\boldsymbol{x}) \left\langle \varphi_l \varphi_n \varphi_i \right\rangle \tag{14}$$

By combing equations (8) and (14), time evolving PC coefficients can be solved with any type of dynamic integrator scheme, e.g., Newmark method. Then dynamic probabilistic structure response can be reconstructed from these PC coefficients with ease.

5. Illustrative example

The above time domain framework is illustrated by probabilistic seismic risk analysis of an eight-story nonlinear shear frame structure subjected to potential earthquakes from a single strike slip fault. The location of the shear frame structure relative to the fault and seismic parameters of the fault are shown in Figure 2(a).



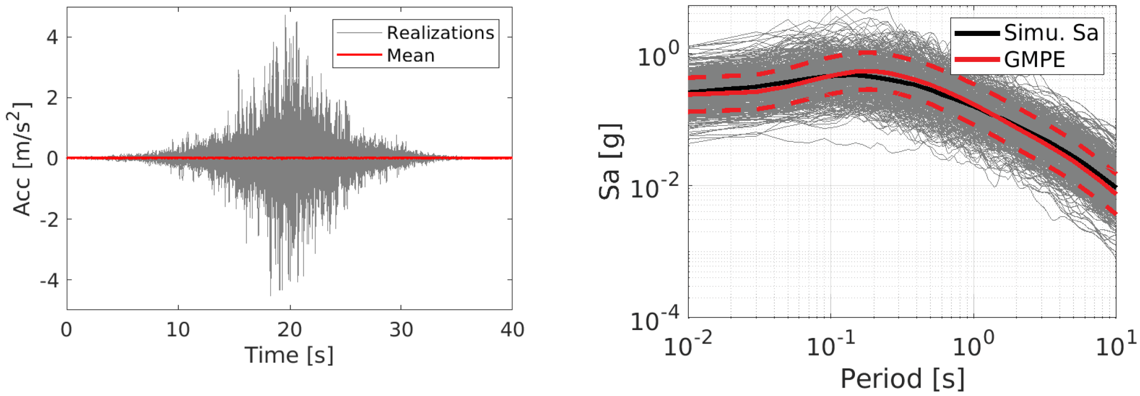
(a) Configuration of the fault and shear frame structure (b) Eight story shear frame structure

Fig. 2 Probabilistic seismic risk analysis: (a) Configuration of seismic fault (black line) and shear frame structure (red triangle) (b) Eight story shear frame structure.

The nonlinear shear frame structure is shown in Figure 2(b) with hysteretic restoring forcing versus inter-story drift described by Armstrong-Fredrick model. The structural parameters are the following: $h_0 = 4m$, $h = 3m$, $m_1 \sim m_2 = 2 \times 10^5 kg$, $m_3 \sim m_4 = 2.2 \times 10^5 kg$, $m_5 \sim m_6 = 2.4 \times 10^5 kg$, $m_7 \sim m_8 = 2.5 \times 10^5 kg$. Material parameter H_a is uncertain and modeled as Gaussian distributed random field with 15% coefficient of variation. The marginal mean of H_a for different floors are: $Ha_1 \sim Ha_2 = 1.59 \times 10^7 N/m$, $H_{a3} \sim H_{a6} = 1.66 \times 10^7 N/m$ and $H_{a7} \sim H_{a8} = 1.76 \times 10^7 N/m$. The correlation structure of parameter H_a is assumed to be exponential between different floors, with correlation length $l_c = 10$ floors. Material parameter C_r is 17.6*/m.*

Seismic source characterization is performed with hazard program HAZ45 [31]. Four seismic scenarios are identified for the strike slip fault: $M_1 = 7.3, R_{rup1} = 56km, \lambda(M_1, R_{rup1}) = 9.54 \times 10^{-4}$; $M_2 = 7.5$, $R_{rup2} = 56km, \lambda(M_2, R_{rup2}) = 2.4 \times 10^{-3}$; $M_3 = 7.7, R_{rup3} = 56km, \lambda(M_3, R_{rup3}) = 2.4 \times 10^{-3}$; $M_4 = 7.9$, $R_{rup4} = 56km, \lambda(M_4, R_{rup4}) = 9.54 \times 10^{-4}$.

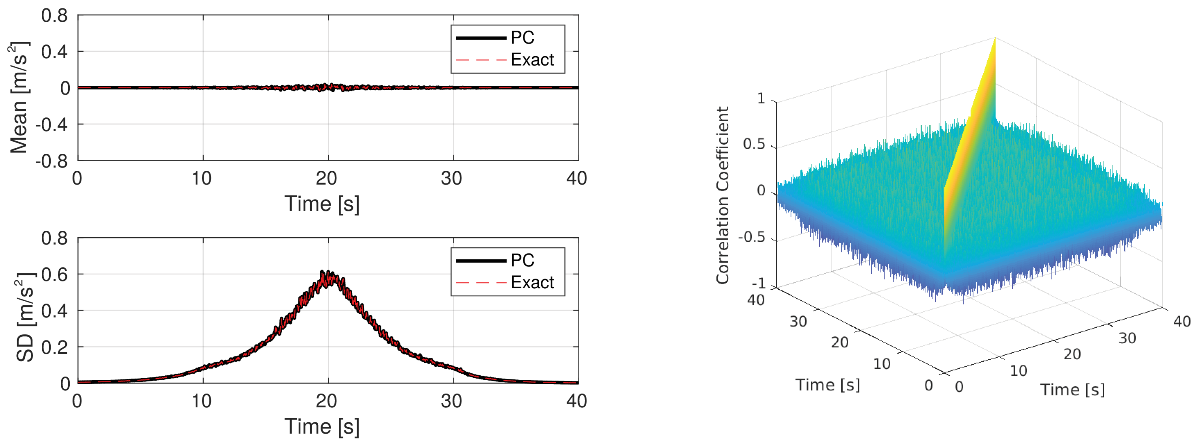
For each seismic scenario, time domain uncertain motions are simulated following the methodology shown in section 3. Here input parameters to GMPE are stress drop *$\Delta\sigma$ = 5MPa* and site attenuation *k_0=0.02s*. Figure 3 shows 200 realizations of seismic motions for earthquake scenario $M = 7.3, R_{rup} = 56km$ and verification of spectrum acceleration of synthesized motions with GMPE. It can be seen that simulated motions have desirable spectra acceleration ordinates that are consistent with GMPE.



(a) Realizations of seismic motions (b) Verification of simulated uncertain motions with GMPE

Fig. 3 Stochastic seismic motions for scenario *M= 7.3, Rrup = 56km:* (a) Realizations of uncertain motions (b) Verification of stochastic motions with GMPE

Stochastic seismic motions from different scenarios are combined into a single population according to their weights and modeled as a single non-stationary random process. The marginal mean, marginal standard deviation and correlation structure of such non-stationary random process is shown in Figure 4. Both uncertain seismic motions and uncertain structural parameters are represented with multi-dimensional Hermite polynomial chaos. In this case, it takes 150 dimensional PCs to characterize stochastic motions and 4 dimensional PCs to characterize uncertain structural parameter $H_a(\boldsymbol{x}, \theta)$.



(a) Marginal mean and standard deviation (b) Correlation structure

Fig. 4 Non-stationary random process seismic motions: (a) Marginal mean and standard deviation (b) Correlation structure

Intrusively propagating PC represented seismic motions into the uncertain structural system using SEPFEM, complete probabilistic dynamic structural response can be solved. Figure 5 shows the evolution of probability density function (PDF) of the top floor displacement. It is noted that at the beginning (t=0s) the structure is deterministically at rest, whose PDF is a Dirac delta function (peaks at infinity) and is not included in the figure. Along with the uncertain seismic excitations, the dispersion of PDF of top floor displacement is observed, which indicates the increasing degree of uncertainty in the structural response. The PDF shows high kurtosis again toward the end of the loading due to reduced variation in input excitation.

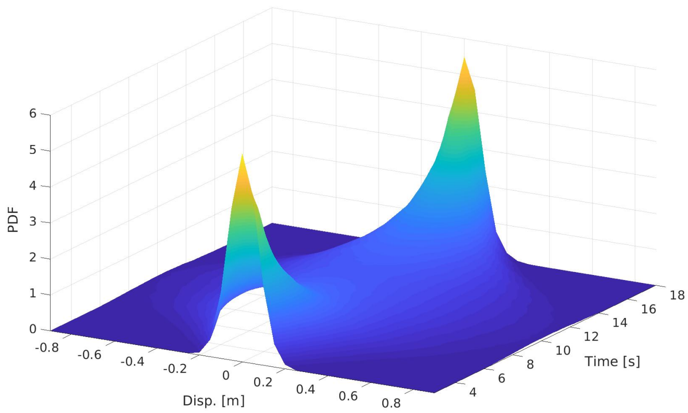
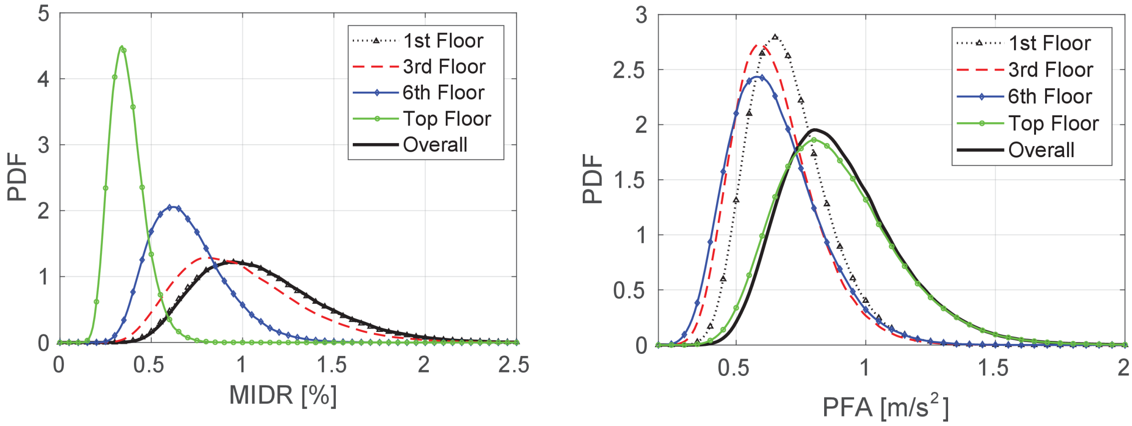


Fig. 5 Evolution of probabilistic density function (PDF) of top floor displacement.

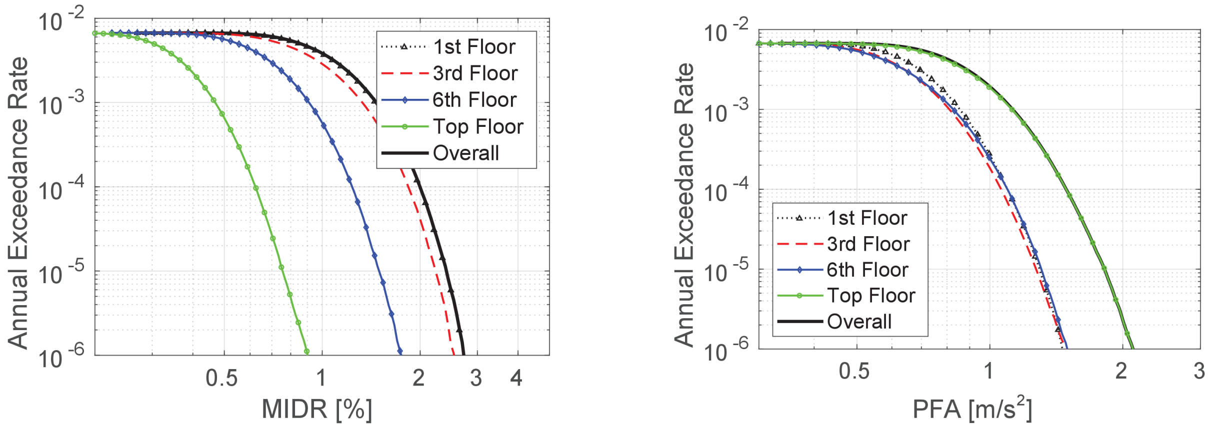
Maximum inter-story drift ratio (MIDR) and peak floor acceleration (PFA) are chosen as engineering demand parameters (EDPs) to quantify the seismic risk. The PDF of MIDR and PFA can be traced from probabilistic dynamic structural response and are shown in Figure 6. It can be seen that from the top floor to the first floor the mean value of MIDR increases and the PDF of MIDR also shows larger dispersion. This is expected considering the increase of interstory shear force from the top floor to the first floor. The PDF of MIDR of the whole structure is almost identical with that of the first floor. On the other hand, the top floor shows the largest mean value and variability of PFA. The distribution of PFA of the whole structure is also very close to the PFA distribution of the top floor.



(a) PDF of maximum inter-story drift ratio (MIDR) (b) PDF of peak floor acceleration (PFA)

Fig. 6 PDF of MIDR and PFA of the eight-story nonlinear shear frame structure: (a) PDF of MIDR (b) PDF of PFA

Exceeding probability of different levels of MIDR and PFA can be computed from the PDFs shown in Figure 6. By combining seismic scenario rate with exceeding probability according to equation 1, EDP hazards of MIDR and PFA are calculated and shown in Figure 7. Defining damage measure (DM) as a step function of EDP, seismic risk for different levels of MIDR exceedance can be determined from Figure 7(a). The risk of the whole structure is 3.83×10^{-3} for MIDR>1% and 9.97×10^{-5} for MIDR>2%. Similarly, it is determined from Figure 7(b) that the risk is 1.92×10^{-3} for PFA>1m/s^2 and 9.45×10^{-5} for PFA>1.5m/s^2.



(a) EDP hazard of MIDR (b) EDP hazard of PFA

Fig. 7 EDP hazards of MIDR and PFA: (a) Maximum inter-story drift ratio (MIDR) (b) Peak floor acceleration (PFA)

6. Conclusion

Presented is a time domain intrusive framework for probabilistic seismic risk analysis in performance based earthquake engineering. Methodology to simulate non-stationary seismic motions based on GMPE of FAS was developed and linked to state-of-the-art seismic source characterization. Both uncertain seismic motions and uncertain structural parameters are considered as random process/field and represented with Hermite polynomial chaos (PC) Karhunen-Loève (KL) expansion. Galerkin stochastic FEM is formulated and used to propagate the uncertain seismic motions into uncertain structural system, which yields complete probabilistic dynamic structural response. From that, seismic risk for any damage measure defined on engineering demand parameter(s) can be traced. The framework avoids choosing and using any simplified IM. All the desired ground motion characteristics and their uncertainties, e.g., uncertain peak ground acceleration (PGA), spectrum acceleration (Sa) and others, are directly captured by random process motions. The framework relies on stochastic modeling of two fundamental characteristics of seismic motions, i.e., FAS and Fourier phase derivative. The development of ground motion prediction equations (GMPEs) for potentially new intensity measures, e.g., Arias intensity or cumulative absolute velocity are also avoided. The intrusive uncertainty propagation through single stochastic FEM analysis significantly releases the burden of repetitive, time consuming Monte Carlo fragility simulations. The seismic risk analysis framework is illustrated within an eight-story shear frame structure excited by uncertain strike-slip fault earthquakes. However, the framework is general and can also be applied to earthquake soil structure interacting systems, for example, by simply adding more shear beams with soil probabilistic elastoplasticity constitutive model to the bottom of the frame structure.

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