Transactions, SMiRT-25
Charlotte, NC, USA, August 4-9, 2019
Division III

TIME DOMAIN SEISMIC RISK ANALYSIS FRAMEWORK FOR NUCLEAR INSTALLATIONS

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ABSTRACT

Presented is a time domain seismic risk analysis framework for soil structure system, focusing on nuclear installations (NIs). Both uncertain seismic excitations and uncertain structural parameters are considered. Uncertain ground motions are simulated from stochastic Fourier amplitude spectra and Fourier phase derivative. The inter-frequency correlation structure of Fourier amplitude spectra is taken into account. The non-stationarity of ground motion is quantified by statistical phase derivative model. Stochastic ground motions are modeled as random process, represented with Hermite polynomial chaos and propagated into uncertain structural system using stochastic finite element method (SFEM). SFEM analysis yields probabilistic dynamic structural response, from which full-spectrum seismic risk is computed.

Proposed framework avoids need to choose and use intensity measure as proxy for uncertain ground motions as is done in the conventional framework. All the uncertainties and important characteristics (e.g. spectrum acceleration Sa and peak ground acceleration PGA) of seismic motions are directly carried by the random process excitations in time domain. Stochastic dynamic structural response is solved in an intrusive way, circumventing non-intrusive time consuming Monte Carlo simulations. Propose methodology is implemented in the Real ESSI Simulator (Jeremić et al., 1989-2019) and illustrated by seismic risk analysis of a four-story building structure.

INTRODUCTION

Performance-based Earthquake Engineering (PBEE) (Cornell, 2000) has been a powerful framework that allows for objective and quantitative decision-making through seismic risk analyses. State of the art methodology of seismic risk analysis is shown as Equation (1):

$$\lambda(EDP > z) = \int \underbrace{\left|\frac{d\lambda(IM > x)}{dx}\right|}_{\text{PSHA}} \underbrace{G(EDP > z|IM = x)}_{\text{fragility}} dx \tag{1}$$

where $\lambda(EDP > z)$ is the annual rate of occurrence of engineering demand parameter (EDP, i.e. performance target) exceeding specified value z. EDP hazard is computed as the convolution of results from probabilistic seismic hazard analysis (PSHA) $\lambda(IM > x)$ and structural fragility $G(EDP > z|IM = x)$ with respect to intensity measure (IM) of ground shaking.

In conventional PBEE, IM needs to be selected as a proxy of damaging uncertain motions with respect to engineering structure. All the uncertainties in ground motion are expected to be represented by the variability of IM. However, the problem is that the scalar spectral acceleration cannot fully describe the influence of uncertain ground-motion upon engineering objects. Stafford & Bommer (2010) investigated different intensity measures and found that they are generally not strongly correlated. Uncertainties considered in the distribution of one IM is not sufficient to describe other ground-motion characteristics. Furthermore, in engineering practices it is very difficult to find a proper IM. For example, there is still no consensus on potential choice of IM from peak ground acceleration (PGA), peak ground velocity (PGV), Arias intensity (AI) and cumulative absolute velocity (CAV) for dam embankment deformation analysis (Davoodi et al., 2013). Many times, even if proper IMs, such as AI and CAV, are identified, additional efforts are still needed to develop GMPE for these IMs and their correlation.

Another issue needs to be addressed is the use of Monte Carlo method for uncertainty quantification in conventional PBEE. Traditionally structural fragility curve is developed by incremental dynamic analysis (IDA) (Vamvatsikos & Cornell, 2002). Thousands of structural response realizations need to be computed with possible samplings of uncertain material properties and uncertain ground excitations at incremental levels of IM. IDA is numerically demanding because of the slow convergence rate that is inherent in Monte Carlo (MC) approach. MC approach is non-intrusive in the sense that there is no modifications to the underlying deterministic code. The characterization of probabilistic space relies on statistically significant number of deterministic samplings of random parameters in the system. The non-intrusive approach becomes computationally difficult/intractable for developing fragility curve of large scale nonlinear structural systems.

To fundamentally resolve the aforementioned two issues, a time domain intrusive stochastic framework for seismic risk analysis is proposed. Without simplifying seismic motions into IM, stochastic ground motions are directly simulated in time domain and modeled as a non-stationary random process. The mean behavior of stochastic Fourier amplitude spectrum (FAS) has been well studied in engineering seismology over last several decades (Brune, 1970, Boore, 2003, Boore & Thompson, 2015). By combining that with recent findings in variability and inter-frequency correlation of FAS (Stafford, 2017, Bayless & Abrahamson, 2018*a*) and phase derivative modeling (Baglio, 2017), methodology for time domain simulations of uncertain motions is presented. With proposed framework, engineering seismologists do not need to interpret/simplify ground motion into IM(s). Correspondingly, structural engineers do not need to compute fragility curve and conduct RHA with spectrum-matched records based on IM. Instead, all the characteristics and uncertainties in seismic motions are captured through the random process and propagated into uncertain engineering system with direct "communication" between engineering seismologists and structural engineers.

Galerkin stochastic finite element method (SFEM) (Ghanem & Spanos, 1991, Matthies & Keese, 2005, Sett et al., 2011, Wang & Sett, 2016) is also incorporated into the proposed framework to avoid non-intrusive MC simulations. Galerkin SFEM is an intrusive approach, requiring new developments based on variational formulation of the underlying stochastic partial differential equations (SPDE). Galerkin SFEM guarantees optimal convergence rates, and is generally much more efficient than MC approach (Xiu, 2010, Elman et al., 2011). In SFEM, probabilistic dynamic structural response is characterized by Hermite polynomial chaos (PC) with unknown PC coefficients. Using Galerkin projection technique, deterministic linear system equations of these unknown temporal-spatial PC coefficients, equivalent to the original SPDE, are derived and solved. Seismic risk is then computed from the probabilistic dynamic response of structural system.

TIME DOMAIN INTRUSIVE SEISMIC RISK ANALYSIS

The proposed framework consists of four components, seismic source characterization (SSC), stochastic ground motion modeling, stochastic finite element analysis and seismic risk computation.

- Seismic source characterization (SSC): Many seismic hazard programs, for example, OpenSHA (Field et al., 2003), could perform SSC for a specific site. A list of potential earthquake scenarios M_i, R_i and corresponding scenario rate $\lambda_i(M_i, R_i)$ can be characterized.
- Stochastic ground motion modeling: For each seismic scenario (M_i, R_i), time domain uncertain ground motions are synthesized using inverse Fourier transform from stochastic FAS and Fourier phase spectrum (FPS).
- Stochastic FEM analysis: The random process seismic motions are spectrally represented with Polynomial Chaos (PC) - Karhunen-Loève (KL) expansion in probabilistic space. PC-represented uncertain motions are intrusively propagated into uncertain structure system using Galerkin stochastic FEM, which gives time-evolving probabilistic structural response.
- Seismic risk computation: Exceeding probability of EDP $P(EDP > z|M_i, R_i)$ conditioned on given scenario (M_i, R_i) can be obtained from probabilistic structural response. Seismic risk is then computed using Equation (2) without using any IMs and performing Monte Carlo fragility simulations.

$$\lambda(EDP > z) = \sum_i \lambda_i(M_i, R_i)P(EDP > z|M_i, R_i) \tag{2}$$

TIME DOMAIN STOCHASTIC GROUND MOTION MODELING

Time domain uncertain motions are simulated from stochastic FAS and FPS. Stochastic FAS is modeled as Log-normal distributed random field (Bora et al., 2015, Stafford, 2017) among different frequencies, whose marginal mean behavior is given by the stochastic method of Boore (2003), as Equation 3:

$$FAS(f) = A_0(M_0, f)Z(R)exp(-\pi fR/Q\beta)S(f)exp(-\pi\kappa_0 f) \tag{3}$$

where M_0 is the seismic moment; β is the source shear wave velocity; $Z(R)$ and $exp(-\pi fR/Q\beta)$ represent the contribution from path effects: $Z(R)$ is the geometrical spreading term as a function of distance R. $exp(-\pi f R/Q\beta)$ quantifies the anelastic attenuation as the inverse of the regional quality factor, Q. The site effects including site amplification through crustal velocity gradient and near surface attenuation are demonstrated by $S(f)$ (Boore & Joyner, 1997) and κ_0 filter $exp(-\pi\kappa_0 f)$ (Anderson & Hough, 1984), respectively. A_0 represents the radiated acceleration source spectrum, which could be characterized by single-corner-frequency model (Brune, 1970) as Equation (4:

$$A_0(M_0, f) = CM_0\Big[\frac{(2\pi f)^2}{1 + (f/f_0)^2}\Big] \tag{4}$$

The marginal total standard deviation $\sigma(f)$ of lognormal distributed FAS random field is taken as constant 0.8 according to recent statistical FAS studies of seismic records (Bora et al., 2015, Bayless & Abrahamson, 2018b). Recently, inter-frequency correlation models for stochastic $FAS(f)$ have been developed (Stafford, 2017, Bayless & Abrahamson, 2018a). Bayless & Abrahamson (2018c) pointed out that mis-representing the correlation structure, e.g. assuming inter-frequency independence, would lead to underestimation of seismic risk.

Stochastic Fourier phase model is another important part for seismogram synthesis. Adopted here is the Logistic model for Fourier phase derivative $\Delta\Phi/\Delta f$ established by Baglio (2017). To verify the above methodology for time domain stochastic ground motion modeling, 500 ground motion realizations for scenario $M = 6.5$ and $R_{rup} = 20km$ are simulated. Figure 1 shows three different acceleration realizations. The spectrum acceleration of synthesized realizations are compared with GMPE, as shown in Figure 2. Significant variability in both amplitude and temporal characteristic can be observed. It can be seen that simulated spectral accelerations are in very good agreement with GMPE in terms of both median behavior and variability.

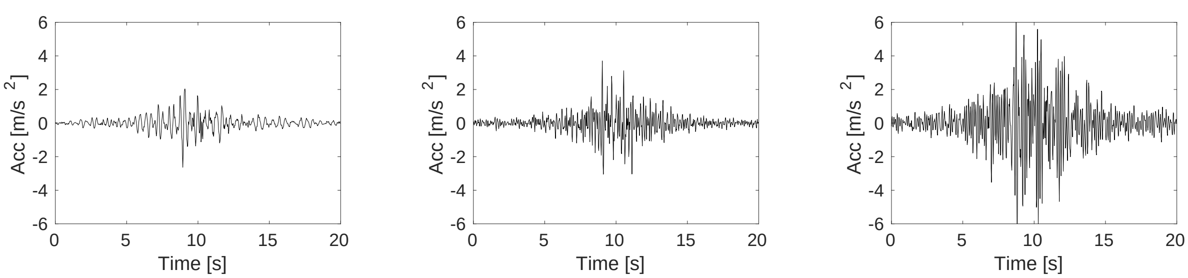


Figure 1: Realizations of uncertain acceleration time series population

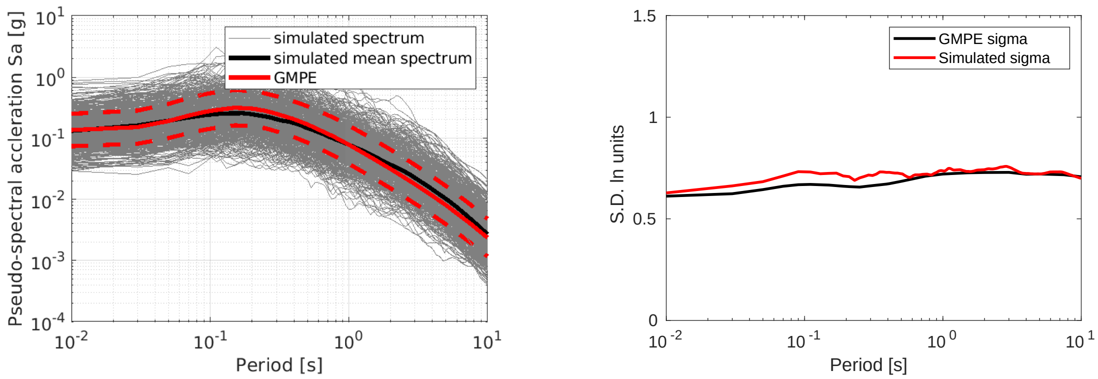


Figure 2: Verification of simulated uncertain seismic motions with GMPE

GALERKIN STOCHASTIC FINITE ELEMENT METHOD

Both uncertain motions and uncertain structural parameters can be modeled as general random process/field $h(\boldsymbol{x}, \theta)$ (Wang & Sett, 2016), where θ denotes the uncertainties. x is the general coordinate, which could be either temporal coordinate for random process or spatial coordinate for random field. Any marginal distribution of $h(\boldsymbol{x}, \theta)$ can be spectrally decomposed into Hermite polynomial chaos of Gaussian kernel $G(\boldsymbol{x}, \theta)$ as Equation 7, where $\{H_i\}$ are the Hermite polynomials.

$$h(\boldsymbol{x}, \theta) = \sum_{i=0}^{P} h_i(\boldsymbol{x}) H_i(G(\boldsymbol{x}, \theta)) \tag{5}$$

The correlation structure of $h(\boldsymbol{x}, \theta)$ is related to the Gaussian covariance kernel $Cov_G(\boldsymbol{x_1}, \boldsymbol{x_2})$ and can be quantified with Karhunen-Loève (KL) expansion:

$$G(\boldsymbol{x}, \theta) = \sum_{i=1}^{M} \sqrt{\lambda_i} f_i(\boldsymbol{x}) \xi_i(\theta) \tag{6}$$

Where λ_i and $f_i(\boldsymbol{x})$ are the eigen-value and eigen-vectors of the covariance kernel satisfying Fredholm's integral equation of the second kind:

$$\int Cov_G(\boldsymbol{x_1}, \boldsymbol{x_2}) f_i(\boldsymbol{x_1}) d\boldsymbol{x_1} = \lambda_i f_i(\boldsymbol{x_2}) \tag{7}$$

Performing the above PC-KL expansion for discretized uncertain stiffness $K_{IacJ}(\theta)$, nodal displacement $U_{Jc}(\theta)$ and nodal forcing $F_{Ia}(\theta)$ in FEM, we have:

$$\begin{aligned} K_{IacJ}(\theta) &= K_{IacJi} \Psi_i(\theta) \\ F_{Ia}(\theta) &= F_{Iaj} \psi_j(\theta) \\ U_{Jc}(\theta) &= U_{Jck} \phi_k(\theta) \end{aligned} \tag{8}$$

Where $\{\Psi_i\}$, $\{\psi_j\}$ and $\{\phi_k\}$ are multi-dimensional Hermite PC bases for uncertain stiffness, forcing and displacement. Using Galerkin projection technique, system of deterministic ordinary differential equations

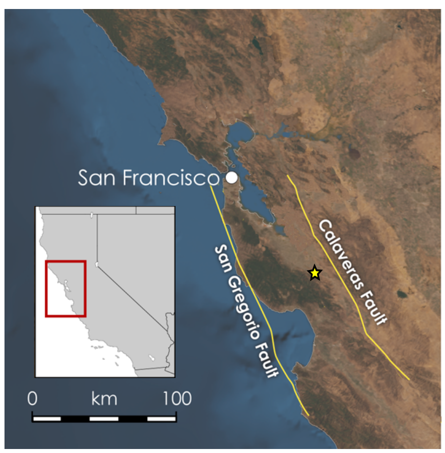
(ODEs) of unknown PC coefficients U_{Jck} are derived as equation 9, which can then be solved with any type of dynamic integrator scheme, e.g. Newmark method. Dynamic probabilistic structure response can be reconstructed from these solved PC coefficients U_{Jck} with ease.

$$M_{IacJ} \langle \phi_k \phi_m \rangle \ddot{U}_{Jck} + K_{IacJi} \langle \Psi_i \phi_k \phi_m \rangle U_{Jck} = F_{Iaj} \langle \psi_j \phi_m \rangle \tag{9}$$

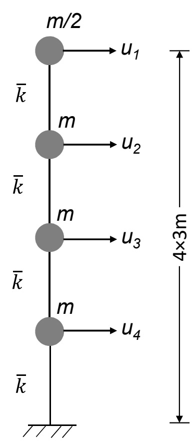
ILLUSTRATIVE EXAMPLE

To illustrate the proposed framework, seismic risk of a four-story building (multiple DOFs system) subjected to earthquake hazards from two strike slip faults (San Gregorio fault and Calaveras fault) is analyzed. The building structure is located at the engineering site (121.9146W, 37.2533N) with $Vs_{30} = 620m/s$. The floor height is $3m$ with deterministic floor mass $m = 100kips/g$. The uncertain floor stiffness is lognormal distributed random field with marginal median $\overline{k} = 168kip/in$ and marginal standard deviation $0.1ln$ units.

Seismic source characterization for the engineering site is performed with the hazard program HAZ45 (Hale et al., 2018). A list of 371 different seismic scenarios are identified with magnitude M 5.1 ∼ 8.3, distance R_{jb} $19km \sim 120km$ and occurrence rate $\lambda(M, R_{jb})$ $3.21 \times 10^{-7}/yr \sim 5.28 \times 10^{-3}/yr$.



(a) Faults and engineering site



(b) Building structure

Figure 3: Configuration of faults, engineering site and building structure: (a) Faults and engineering site (denoted by asterisk) (b) Four DOFs building structure

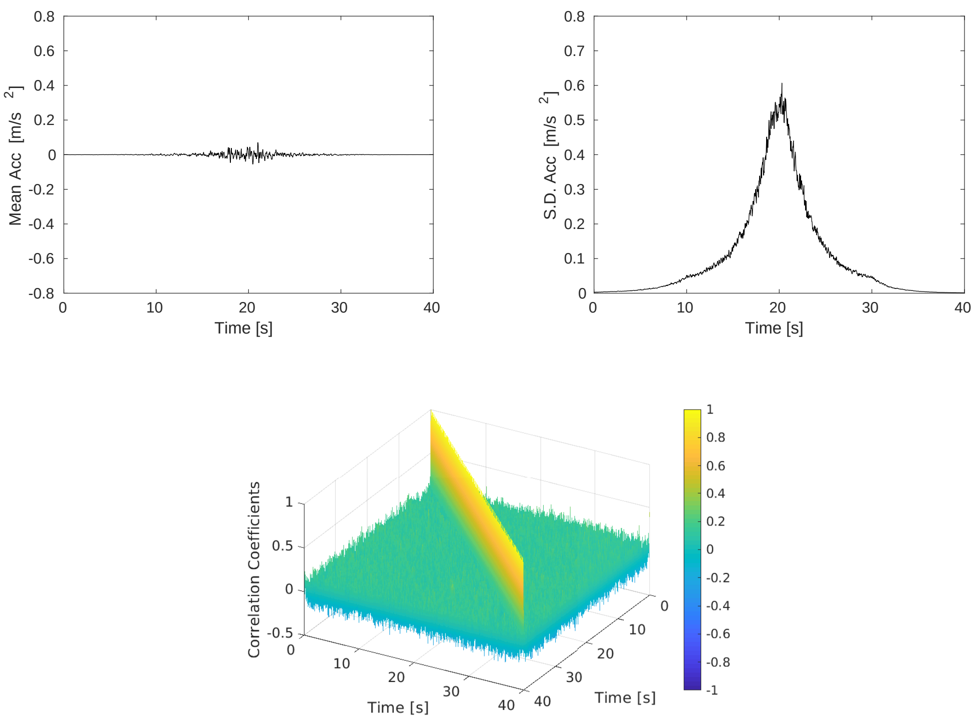


Figure 4: Marginal mean, standard deviation and correlation structure of random process motions

For each scenario, time domain stochastic motions are simulated and represented with Hermite PCs with PC-KL expansion. Hermite PCs with dimension 150, order 1 are adopted to quantify the non-stationary random process motions. Figure 4 shows the marginal mean, standard deviation and correlation structure of uncertain motions caused by scenario $M = 7$, $R_{jb} = 20km$.

On the other hand, the lognormal distributed stiffness random field is also quantified by Hermite PCs of dimensional 4, order 2. Exciting uncertain structural system with non-stationary random process motions by SFEM, probabilistic dynamic structural response is solved. Figure 5 presents the time-evolving mean and standard deviation of displacement, velocity and acceleration response of top floor. It can be seen that the standard deviation (i.e. uncertainty) of the top floor acceleration response increases along with the excitation of uncertain motions. The evolution of probabilistic density of top floor displacement is shown in figure 6. The probabilistic density function (PDF) of floor deformation is Gaussian like. Because of the large uncertainty in ground motion, probabilistic structural response is dominated by the Gaussian random excitations, though the marginal distribution of uncertain stiffness field is non-Gaussian (lognormal distributed). Compared with PDF at $t = 2s$, more dispersion (i.e. more uncertainty) can be observed for PDF of displacement response at $t = 16s$.

Here maximum inter-story drift ratio (MIDR) is chosen as engineering demand parameter. The PDF of MIDR can be easily calculated from the probabilistic dynamic structural response, as shown in Figure 7 (a). The EDP hazard (i.e. the annual exceedance rate of EDP) could also be computed as Figure 7 (b). Assuming damage measure (DM) as a step function of EDP, seismic risk for exceeding different levels of MIDR can be determined from EDP hazard curve. From Figure 7 (b), the risk is 2.3×10^{-3} for MIDR $> 1\%$ and $2.2\times$

10^{-4} for MIDR > 2%.

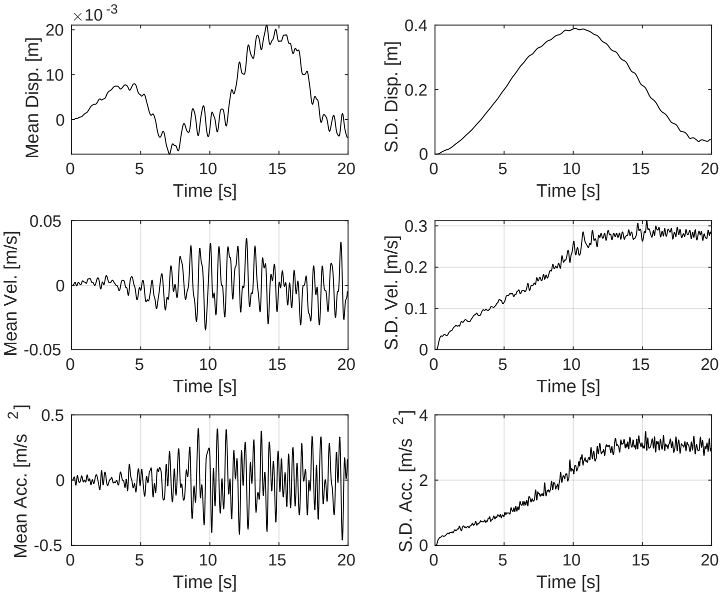
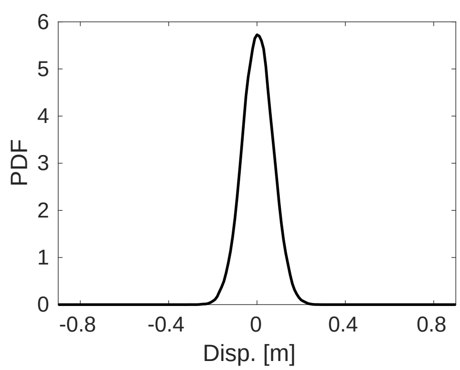
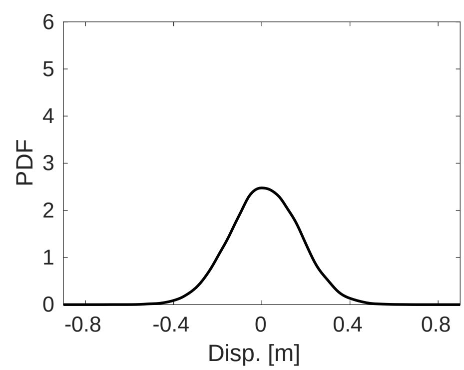


Figure 5: Time evolving mean, standard deviation of probabilistic response of top floor



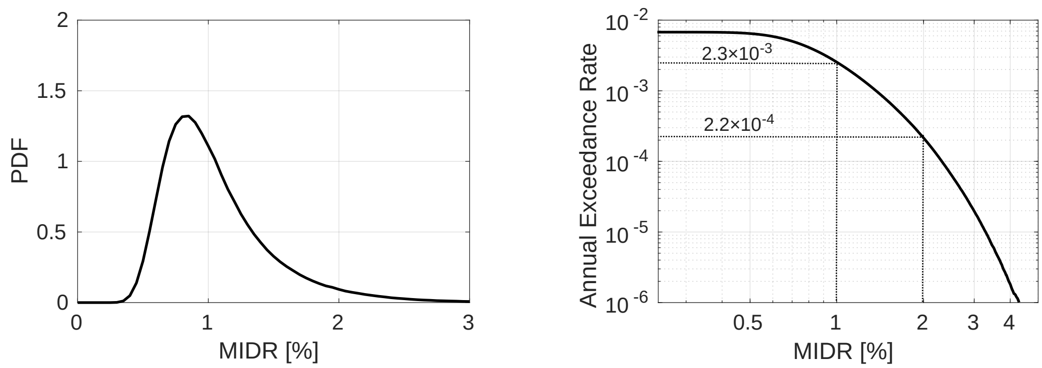
(a) t=2s



(b) t=16s

Figure 6: Evolution of probabilistic density function (PDF) of top floor displacement response: (a) PDF of displacement response at t= 2s (b) PDF of displacement response at t=16s

It is noted that proposed methodology could produce full spectrum EDP hazard and seismic risk for all levels of MIDR exceedance with only one-time stochastic ground motion modeling and SFEM analysis. This is much more efficient than the conventional framework of PBEE, where hundreds of MC-type incremental dynamic analyses are needed for developing a fragility curve for specific level of MIDR exceedance $G(MIDR > z|IM = x)$. By convolving the structural fragility with seismic hazard (Equation 1), risk for the selected level of MIDR exceedance is obtained, which is only a single risk point in Figure 7 (b). Repetitive development of fragility curves and convolution calculations are required if multiple levels of MIDR exceedance are of concern.



(a) PDF of MIDR (b) EDP hazard curve

Figure 7: The PDF of MIDR and EDP hazard curve

CONCLUSION

A time domain intrusive framework for seismic risk analysis is established. The established framework fundamentally solves the issue of IM as a (over-)simplified proxy of uncertain seismic motions in conventional PBEE. Without using any IM, stochastic ground motions are directly simulated in time domain and modeled as non-stationary random process that captures all the uncertainties and important characteristics of seismic motions.

Propagation of stochastic seismic motions into uncertain structural system using intrusive SFEM avoids time-consuming Monte Carlo simulations. Through one-time SFEM analysis, obtained is accurate probabilistic dynamic structural response and more importantly, stable tail distribution. A stable tail distribution of EDP is crucial for seismic risk analysis with notably low risk level and was computationally difficult using MC approach.

The capability and efficiency of the proposed framework is illustrated with risk analysis of a multi-DOF building structure. Both uncertainties in seismic motions and structural properties are well quantified and considered. Evolving characteristic of the probabilistic structural response is shown. The accurate and efficient seismic risk analysis could help improve the economy and safety of infrastructure designs. Future research will focus on application of the proposed framework to other large-scale, nonlinear infrastructure systems, for example, nuclear power plants and dams.

ACKNOWLEDGMENT

This work was supported in part by the US-DOE and by the UCD.

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