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Probabilistic seismic hazard analysis considering site-specific soil effects

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ABSTRACT

This study presents a framework to perform Probabilistic Seismic Hazard Analysis (PSHA) for soil sites, which yields accurate soil UHS and associated seismic hazard curves. The variabilities of soil parameters, the nonlinear responses of soils, and the vector-valued site responses analysis comprehensively integrate into the PSHA for soil sites. In this framework, site amplification is used to modify the Ground Motion Prediction Equations (GMPEs) to make them suitable for a soil site. Based on the modified GMPEs with updated uncertainties, PSHA for soil sites are performed accurately; thus, acceptable soil UHS and associated seismic hazard curves considering site-specific uncertainties could be achieved. Using an example soil site, influences of soil parameter variabilities and soil nonlinearity on UHS and associated seismic hazard curves are discussed in this study.

1. Introduction

1.1. Background

The design response spectrum is usually represented by site-specific ground motions [1], such as Uniform Hazard Spectra (UHS). It plays a crucial role in the design and analysis of nuclear facilities. For example, the design response spectrum is used to generate synthetic or artificial time histories for seismic analysis of complex structures [2], or generate floor response spectra [3,4] for Nuclear Power Plants (NPPs).

When incident bedrock motions propagate from bedrock to the soil surface, the soil deposit changes characteristics of the ground motions; the extent of this change largely depends on features of the incident bedrock motions and characteristics of the local soil deposit. Thus, differences between UHS at the soil surface (soil UHS) and UHS at the bedrock (rock UHS) are governed by this change.

To construct UHS from Probabilistic Seismic Hazard Analysis (PSHA), Ground Motion Prediction Equations (GMPEs) are required, which predict the level of ground shaking and its associated uncertainties at a given site based on the earthquake magnitude, the source-to-site distance, and the fault mechanism etc. Some empirical GMPEs [5] may be used to construct the soil UHS in the same way as constructing the rock UHS. However, the attenuation relationships in the empirical GMPEs are based on ground motions recorded at stiff and generally deep soil sites, and use generic soils to characterize various site-specific soils. These empirical GMPEs are also constrained by the ground motions which were used to develop these attenuation relationships. Thus, it is only appropriate to use these attenuation relationships to estimate ground motions at the soil surface above a similar soil deposit, considering differences between a practical site-specific soil profile and the generic soil profile [6]. This requirement actually greatly restricts the usage of empirical GMPEs to construct the soil UHS.

1.2. Literature review

To overcome this problem of empirical GMPEs in constructing soil UHS, McGuire et al. [7] suggested that site amplification be used to modify GMPEs into site-specific attenuation relations prior to performing PSHA for soil sites. Based on this idea, several methods have been proposed to perform PSHA for soil sites.

Tsai [8] proposed a method to calculate Peak Ground Acceleration (PGA) at the soil surface, and concluded that: (1) nonlinear site effects play a crucial role in the calculation of annual probability of exceedance for PGA at the soil surface, and failure to consider nonlinearity of soils may dramatically distort the soil-hazard curve and may not always lead to conservative estimates; (2) the annual probability of exceedance for PGA at the soil surface calculated by nonlinear site response analysis cannot be facilitated by GMPEs method, due to the loss of detailed site information in GMPEs method; and (3) the result of annual probability of exceedance for PGA greatly depends on the standard deviation of the site amplification.

Cramer [9] also proposed a method to calculate the soil-hazard curve following the suggestions of McGuire. By applying the proposed equation to two example soil sites, Cramer concluded that using the proposed method can make about a 10% difference or even larger in

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ground motion estimates over simply multiplying the bedrock probabilistic ground motion by the mean site amplification.

Bazzurro and Cornell [10,11] used Monte Carlo simulation to study effects of uncertainties from soil parameters and input bedrock motions on site amplification. Based on two different example soil sites, they developed site amplification models for these two example sites. They further modified GMPEs and proposed equations to perform PSHA for soil sites. Using the proposed equations, soil UHS for the two example sites are constructed.

1.3. Research objectives

This study provides a probabilistic framework to perform PSHA for a specific soil site, from which soil UHS and associated seismic hazard curves can be generated. In this framework, the uncertainties from soil parameters and input bedrock motions, the vector-valued site response analysis, and soil nonlinearity are considered to develop a site amplification model. Next, using the site amplification model, the modified GMPEs with updated uncertainties for the soil site are obtained. Based on the modified GMPEs, PSHA for soil sites are conducted, and soil UHS and associated seismic hazard curves considering site-specific uncertainties are generated.

2. Soil parameter uncertainties

In site response analysis, the shear-wave velocity, normalized shear modulus and damping ratio are the three most critical dynamic soil parameters that affect the analysis. Any investigation of dynamic soil parameters should be performed with the recognition of inevitable uncertainty during the process of soil parameter testing [12].

2.1. Shear-wave velocity

To consider variabilities of shear-wave velocity, a statistical model for shear-wave velocity provided by Electric Power Research Institute (EPRI) [13] is used in this study. The statistical model assumes that shear-wave velocity is lognormally distributed at any given depth. According to this model, the probability distribution of shear-wave velocity are expressed using the cumulative distribution of standardized variable:

$$D_i = \frac{\ln V_S(i) - \ln V_{S,m}(i)}{\sigma_{\ln V_S}} \tag{1}$$

where D_i is normal random variable for the ith layer with zero mean and unit standard deviation, $V_S(i)$, $V_{S,m}(i)$, and $\sigma_{\ln V_S}$ are the shear-wave velocity, the median shear-wave velocity, and the standard deviation of shear-wave velocity at the ith layer, respectively.

Using a first-order auto-regressive model, the lognormal distribution of V_S and the correlation of random V_S among adjacent layers are expressed as

$$\begin{aligned} &D_1 = \varepsilon_1, \\ &D_i = \rho \cdot D_{i-1} + \varepsilon_i \sqrt{1-\rho^2}, \quad i > 1, \end{aligned} \tag{2}$$

where ρ is the auto-correlation coefficient of D_i and D_{i+1}, and $\varepsilon_i (i \geq 1)$ is the independent normal random variable with zero mean and unit standard deviation.

In Eqs. (1) and (2), $\sigma_{\ln V_S}$ and ρ were estimated by linear regression as $\sigma_{\ln V_S} = 0.39$ (corresponding to a coefficient of variation of 0.41) and $\rho = 0.577$ [13]. A distribution truncation at $\pm\ 2\sigma^i_{\ln V_S}$ is adopted to avoid unrealistic parameter values.

2.2. Normalized shear modulus and damping ratio

Based on soil samples from various sites, Darendeli [14] assumed that the normalized shear modulus and damping ratio are normally distributed at any given shear-strain level. Then Darendeli proposed a model to represent the standard deviation of normalized shear modulus and damping ratio of soils at any given shear-strain level:

$$\begin{aligned} &\sigma_{NG} = \exp(\varphi_{13}) + \sqrt{\exp(-\varphi_{14}) \cdot [0.25 - (\overline{G/G}_{\max} - 0.5)^2]}, \\ &\sigma_{\xi} = \exp(\varphi_{15}) + \exp(\varphi_{16}) \cdot \bar{\xi}^{\,0.5}, \end{aligned} \tag{3}$$

where σ_{NG} and σ_{ξ} respectively denote the standard deviations of normalized shear modulus and damping ratio at a given shear-strain level, $\overline{G/G}_{\max}$ and $\bar{\xi}$ respectively denote the mean normalized shear modulus and the mean damping ratio at this shear-strain level, φ_{13}, φ_{14}, φ_{15}, and φ_{16} are the model parameters obtained from regression analysis. Values of these model parameters are: $\varphi_{13} = -4.23$, $\varphi_{14} = 3.62$, $\varphi_{15} = -5$, and $\varphi_{16} = -0.25$. To prevent unrealistic parameter values, a distribution truncation at $\pm\ 2\sigma_{NG}$ and $\pm\ 2\sigma_{\xi}$ is adopted to generate randomized normalized shear modulus and damping ratios, respectively.

For the example site in this study, the median shear-wave velocity, mean normalized shear modulus, and mean damping ratio are referred to the site-specific data in the study of Zhang and Andrus [15]. The standard deviation, $\sigma_{\ln V_S} = 0.39$, from the statistical model of EPRI [13] is used to characterize uncertainty of shear-wave velocity at different layers. The standard deviation, σ_{NG} and σ_{ξ}, in the Darendeli's model [14] are used to characterize uncertainties of normalized shear modulus and damping ratio.

It is noted that, since the material curves (i.e., the normalized shear modulus and damping ratio curves) and shear-wave velocity profiles vary concurrently in this study, it is assumed that those parameters are perfectly correlated in this study.

3. Site response analysis

3.1. DEEPSOIL and soil nonlinear models

DEEPSOIL [16] is a one-dimensional site response analysis computer program. It can perform both nonlinear time-domain wave propagation analysis and equivalent linear frequency-domain wave propagation analysis. The site response analysis in this study solves the one-dimensional vertical SH-wave propagation problem.

Soil nonlinear models relatively accurately characterize dynamic behavior of soil under low to high ground motion intensities. Several soil nonlinear models have been proposed in the past, among which the *Modified Konder and Zelasko* (MKZ) model is usually used. The MKZ model is also used in this study to characterize nonlinear stress-strain relationship of soil under seismic excitations.

3.2. Vector-valued site response analysis

Due to effects of soil deposits, ground motions propagating from bedrock to the soil surface are changed. Prediction of the change requires site response analysis, which is affected by many factors, such as soil parameters and incident bedrock motions; most of these factors are uncertain. Thus, probabilistic method is applied to site response analysis.

At a soil site, if a_k is taken as a response measure at a period T_k, its probability is given by

$$p(a_k) = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \cdots \int_{-\infty}^{\infty} p(a_k | i_1, i_2, \ldots, i_n) f_I(i_1, i_2, \ldots, i_n) \mathrm{d}i_1 \mathrm{d}i_2 \cdots \mathrm{d}i_n, \tag{4}$$

where $i_1, i_2, \ldots, i_n$ are intensity measures of input motions, and $f_I(i_1, i_2, \ldots, i_n)$ are their joint probability density function. Since multiple intensity measures (such as the peak ground acceleration and spectral accelerations of input motions) are used, it is called vector-valued site response analysis. These multiple intensity measures are the same with predictor variables for multiple regression analysis. Selection of these multiple intensity measures is discussed in Section 5.3.