Stochastic Seismic Analysis of Geosynthetic-Reinforced Soil Slopes Based on Probability Density Evolution Method

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Abstract: This paper performs stochastic seismic analysis of a geosynthetic-reinforced soil slope under random ground motions. The paper employs the probability density evolution method (PDEM) to carry out the probabilistic analysis. The stochastic seismic ground motions are generated using the random function based spectral representation method. The results show that for the same seismic deformation the middle to bottom reinforcement layers generally have higher probability of failure than other reinforcement layers. Thus, an increase in reinforcement strength in the middle to bottom part of the reinforced soil slope may be needed in order to keep the slope from large displacement of failure. The results indicate that the PDEM is an efficient method to carry out the stochastic seismic analysis of geosynthetic-reinforced soil slopes.

Keywords: Geosynthetics; soil slopes; probabilistic seismic analysis; probability density evolution method.

1 Introduction

The geosynthetic-reinforced soil structures performed well during strong earthquakes (such as Chi-Chi, El Salvador, Kobe et al.) as demonstrated by Ling et al. (2001) and Koseki et al. (2006). However, failure cases of reinforced soil structures have been reported under earthquake seismic action (Koseki 2012). Pseudo-static method and pseudo-dynamic method are the conventional method to carry out stability analysis of geosynthetic-reinforced soil structures under seismic actions. Khosravizadeh et al. (2016) introduced a procedure to determine the location and shape of failure surface using the Horizontal Slice Method within the framework of the pseudo-static method. Ahmad and Choudhury (2008) proposed a simple design methodology for reinforced soil wall using the pseudo-dynamic approach. However, many uncertainties exist in soil, reinforcements and earthquakes, it is necessary to carry out probabilistic analysis of reinforced soil structures.

Sayed (2008) performed reliability analysis of reinforced soil wall under static and seismic loads considering the uncertainty of soil and reinforcement properties using first-order second-moment method (FOSM), point estimate method (PEM) and first-order reliability method (FORM). Basha and Babu (2009; 2011) presented a method to evaluate the reliability for external and internal stability of reinforced soil structures subjected to earthquake loads within the framework of the pseudo-dynamic method. As the design of reinforced soil structures moves towards the performance-based approach, the serviceability assessment becomes more important (Yu and Bathurst 2017). Shinoda et al. (2006) investigated reliability for seismic deformation of geosynthetic-reinforced soil slopes subjected to strong earthquakes by a low-discrepancy sequence Monte Carlo (LDSMC) method and an importance sampling with low-discrepancy sequence Monte Carlo (ISLDSMC) method.

The studies on probabilistic analysis of reinforce soil structures under seismic conditions generally ignored the uncertainty of seismic ground motions. However, earthquake is a non-stationary random process with strong randomness, and has a significant impact on the reinforced soil structures subjected to seismic conditions. Thus, it is necessary to perform the probabilistic seismic analysis of reinforced soil structures considering the randomness of earthquake.

In order to fully understand the seismic performance of geosynthetic-reinforced slopes, this paper presents probabilistic seismic analysis for the deformation of geosynthetic-reinforced soil slopes subjected to stochastic earthquake excitation using recently developed probability density evolution method (PDEM). The stochastic earthquake excitation is first generated by random function based spectral representation method. Thereafter the deterministic analysis is carried out by the finite difference method (FDM) program FLAC. The probability density function of the seismic deformation is then obtained by using PDEM.

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2 PDEM

The probability density evolution method (PDEM) is proposed by Li and Chen (2008) based on the principle of preservation of probability. The accuracy and efficiency of the results obtained from this method have been validated by the Monte Carlo simulation (MCS) (Huang and Xiong 2017; Rui Pang et al. 2018). Huang and Xiong (2017) showed that the calculation speed of PDEM is approximately 14 times faster than the traditional MCS. According to the principle of preservation of probability (Li and Chen 2008), the generalized density evolution equation for dynamic performance of reinforced soil slopes can be expressed by the following equation:

$$\frac{\partial p_{D\mathbf{\Theta}}\left(D,\mathbf{\theta},t\right)}{\partial t}+\sum_{l=1}^{m}\dot{D}_l\left(\mathbf{\theta},t\right)\frac{\partial p_{D\mathbf{\Theta}}\left(D,\mathbf{\theta},t\right)}{\partial D_l}=0 \tag{1}$$

where D is the horizontal displacement of reinforced soil slopes; $\dot{D}_l$ is horizontal velocity of reinforced soil slopes; Θ is the random vector composed of basic random variables; $p_{D\mathbf{\Theta}}\left(D,\mathbf{\theta},t\right)$ is the probability density function (PDF) of the horizontal displacement which depends on the random vector (Θ).

To solve the GPDEM equation, a series of discrete points θ_q (q = 1, 2, … , n_{sel}, where n_{sel} is the total number of discrete points) in the distribution space Ω_Θ of the basic random vector (Θ) are selected. Thereafter the probability of each discrete point is determined. The randomness in this study derives from the peak acceleration value (PGA) and the frequency spectrum distribution.

This paper selected the discrete points using the Number-Theoretical method proposed by Hua and Wang (1981). The stochastic ground motion generated process will be presented in the next section. The horizontal displacement of reinforced soil slopes ($D(\theta_q,t)$) is first obtained by the deterministic analysis for each representative point θ_q . Then the horizontal velocity of reinforced soil slopes $\dot{D}(\theta_q,t)$ is calculated and substituted into the Eq. (1). Thereafter, Eq. (1) can be solved using the finite difference method. The initial condition of Eq. (1) is shown as:

$$p_{D\Theta}(D,\theta_q,t)\mid_{t=t_0}=\delta(D-D_0)P_q \tag{2}$$

where $\delta(D-D_0)$ is Dirac function; P_q is the probability of representative point (θ_q). By summing all of the above $p_{D\Theta}(D,\theta_q,t)$ (q = 1, 2, …, n_{sel}) the numerical solution of $p_D(D,t)$ can be obtained as:

$$p_D(D,t)-\sum_{q=1}^{n_{sel}}p_{D\Theta}(D,\theta_q,t) \tag{3}$$

3 Stochastic Seismic Ground Motions

The random variable in this paper is the seismic ground motion. The stochastic seismic ground motions need to be discretized in the distribution probability space based on the PDEM in order to obtain the earthquake samples. The random function based spectral representation method (Liu et al., 2016) is used to generate a series of acceleration time histories in this study. The stochastic seismic grounds are assumed to be a zero-mean and real non-stationary process, which can be simulated by the following equation:

$$\ddot{D}_g\left(t\right)=\sum_{k=1}^{N}\sqrt{2S_{\ddot{D}_g}(t,\omega_k)\Delta\omega}\left[\cos\left(\omega_k t\right)X_k+\sin\left(\omega_k t\right)Y_k\right] \tag{4}$$

where $\ddot{D}_g\left(t\right)$ is the simulated non-stationary stochastic seismic ground motion; $S_{\ddot{D}_g}$ is bilateral evolutionary power spectral density function of $\ddot{D}_g\left(t\right)$; ω is the frequency of power spectral density function; X_k and Y_k (k = 1, 2, …, N) are the standard orthogonal random variables. Other variables are defined as $\omega_k=k\Delta\omega$, $\Delta\omega=\omega/N$ with N = 1600 and $\Delta\omega$ = 0.15 rad/s resulting a small time interval of Δt = 0.01 s.

The orthogonal variables $\overline{X}_n$ and $\overline{Y}_n$ are assumed to be the function of the independent elementary random variables Θ_1 and Θ_2 , respectively as:

$$\overline{X}_n=cas(n\Theta_1) \tag{5}$$

$$\overline{Y}_n=cas(n\Theta_2) \tag{6}$$

where $cas(x)$ is the Hartley function ($cas(x) = cos(x) + sin(x)$), and two independent elementary random variables, Θ_1 and Θ_2, are assumed to follow the uniform distribution in the range of $[0, 2\pi]$.

The bilateral evolutionary power spectral density function used in this study is shown in the following equation based on Cacciola and Deodatis (2011):

$$S_{\ddot{U}_g}(t,\omega) = A^2(t)\cdot\frac{\omega_g^4(t)+4\xi_g^4(t)\omega_g^2(t)\omega^2}{\left[\omega^2-\omega_g^2(t)\right]^2+4\xi_g^4(t)\omega_g^2(t)\omega^2}\cdot\frac{\omega^4}{\left[\omega^2-\omega_f^2(t)\right]^2+4\xi_f^4(t)\omega_f^2(t)\omega^2} \tag{7}$$

where $A(t)$ is intensity modulation function and calculated as:

$$A(t) = \left[\frac{t}{c}\exp\left(1-\frac{t}{c}\right)\right]^d \tag{8}$$

where c is the average time of PGA emerging; d is the shape control index of $A(t)$; $c = 4$s; $d = 2$ in this study.

In evolutionary power spectral density function, the frequency modulation function can be determined by the following parameters:

$$\omega_g(t) = \omega_0 - a\frac{t}{T}, \xi_g(t) = \xi_0 - b\frac{t}{T} \tag{9}$$

$$\omega_f(t) = 0.1\omega_g(t), \xi_f(t) = \xi_g(t) \tag{10}$$

where ω_0 and ξ_0 are the angular frequency and damping ratio of the site soil, respectively; ω_0, ξ_0, a, b are determined based on the field classification and seismic design categories; T is the total duration of non-stationary seismic acceleration process. The parameter values used in this paper are: $\omega_0 = 25$ s^{-1}, $\xi_0 = 0.45$, $a = 3.5$, $b = 0.3$ and $T = 15$ s.

The parameters reflecting the seismic ground motion intensity are expressed as follows:

$$S_0(t) = \frac{\bar{a}_{\max}^2}{\gamma^2\pi\omega_g(t)\left[2\xi_g(t)+1/2\xi_g(t)\right]} \tag{11}$$

where $\bar{a}_{\max}$ is the mean value of PGA; γ is equivalent peak factor; other parameters are set as $\bar{a}_{\max} = 0.2$g, $\gamma = 2.6$ in this paper.

The representative discrete points are selected by Number-Theoretical method (Hua and Wang 1981). A total of 233 seismic acceleration time histories samples are selected in the probability space of the basic random variable distribution space Ω_Θ in this study. Figure 1 shows the typical stochastic seismic ground motion sample. Figure 2 shows the mean and standard deviation of seismic ground motion.

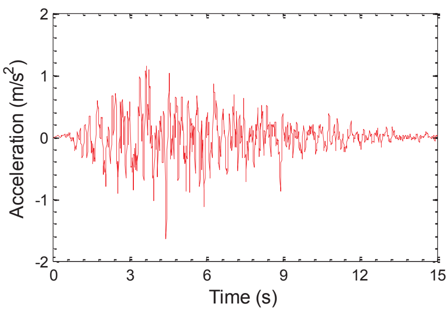

Figure 1. An example of the stochastic seismic ground motion.

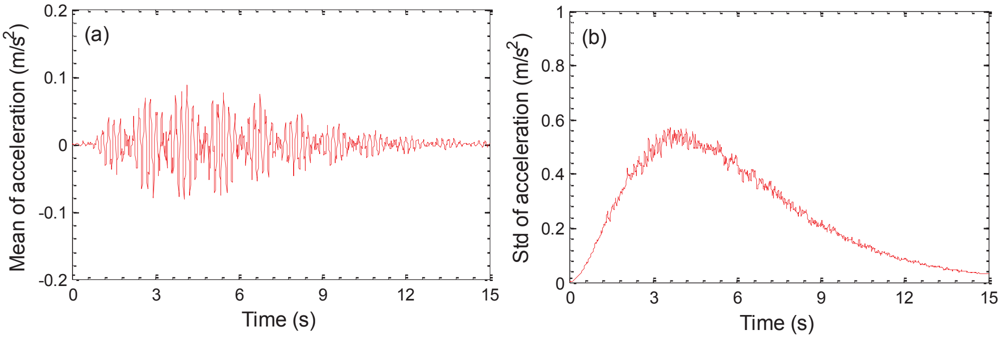

Figure 2. An example of mean (a) and standard deviation (b) of seismic ground motion.

4 Numerical Case

4.1 Numerical model

Figure 3 shows a reinforced soil slope model with a geometry similar to that from Luo et al. (2016). The slope has a height of 5 m and a width of 20 m at the crest of the slope. The foundation has a height of 5 m and a width of 35 m. The slope angle is 45°. The reinforcement has a length of 5 m and the vertical space of reinforcement layers is 1 m. The soil is assumed to be homogeneous and is modeled as ideal elastoplastic model followed the Mohr-Coulomb failure criterion. The reinforcement is modelled using cable elements in FLAC. The bottom boundary is fixed in both x- and y-direction. There is no interface between the reinforcement and the soil for simplicity (Luo et al. 2016). The parameters of soil and reinforcement are given as: the Young's modulus of soil E = 60MPa, Poisson's ratio of soil v = 0.3, friction angle of soil ϕ = 25°, cohesion of soil c = 2 kPa, unit weight of soil γ = 20 kN/m^3, axial stiffness of reinforcement J = 1000 kN/m, and ultimate strength of reinforcement T_{ult}=100 kN/m. After the representative discrete points are selected, the dynamic performance of reinforced soil slopes can be derived by the deterministic analysis from the process of PDEM. The deterministic analysis of reinforced soil slopes in this paper is carried out using FDM program FLAC. In dynamic analysis, the free-field boundary condition was applied at both left and right boundaries to eliminate the wave reflection. A 2% Rayleigh damping was chosen to model the dynamic behavior of the soil.

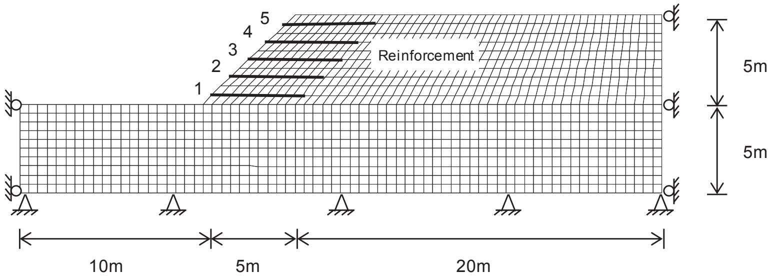

Figure 3. The geometry of reinforcement soil slope.

4.2 Stochastic seismic response of reinforced soil slopes

After a series of deterministic dynamic analyses of reinforced soil slopes, the GPDEM equation can be solved. Fig. 4 illustrates the mean and standard deviation time histories for the horizontal displacement of the slope face at different reinforcement evaluations under 0.2g seismic ground motion level. The mean values in Fig. 4a demonstrates that the second reinforcement layer has higher mean value of horizontal displacement of the slope face than other layers. The standard deviation value tends to increase with time because the variability of the horizontal displacement increases due to the increased cumulative displacement. Fig. 5a shows the PDFs of the horizontal displacement of the slope face at different time points. The results demonstrate the variability of the horizontal displacement at different times. Thus, it is not precise for probabilistic seismic analysis of reinforced soil slopes assuming the earthquake as the inertia force based on pseudo-static method, and it is necessary to take into account the randomness of the earthquakes. The PDF distributions at different time points are not normally distributed. The evolution of the PDF between 4-5s is displayed in Fig. 5b.

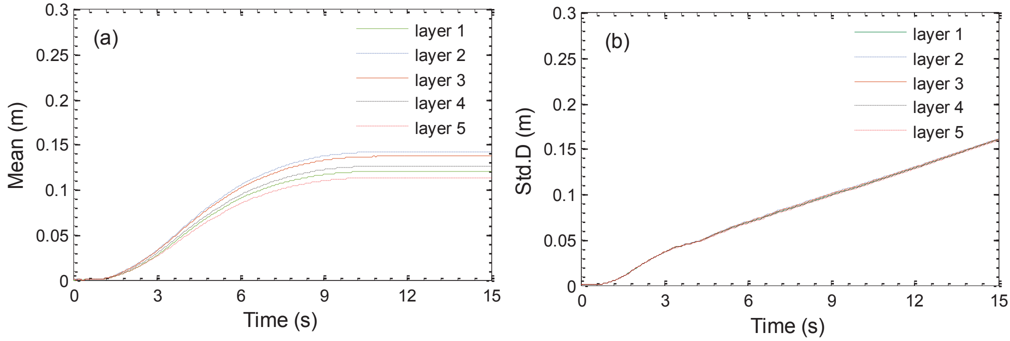

Figure 4. The mean value (a) and standard deviation (b) of the horizontal displacement of the slope face at different reinforcement elevations.

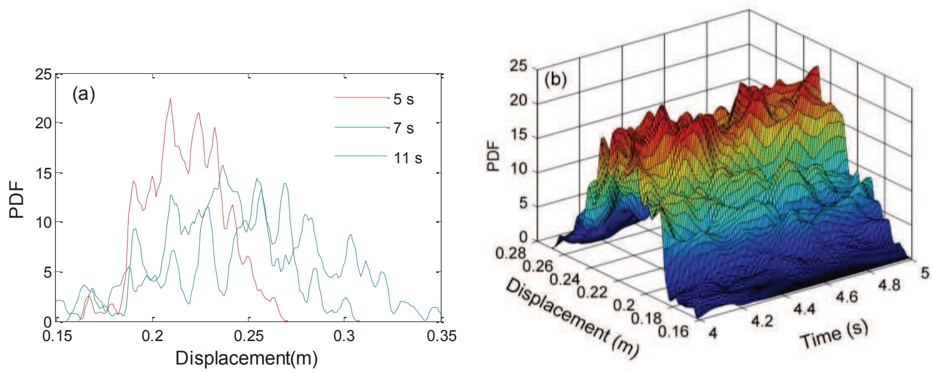

Figure 5. The PDF of the horizontal displacement of the slope face at the second reinforcement layer: (a) for three discrete times and (b) between 4-5s.

Figure 6 shows the cumulative distribution function (CDF) of maximum horizontal displacement at different reinforcement layers. From Fig. 6, the second reinforcement layer has a probability of 95.11% for the maximum horizontal displacement less than 0.4 m under 0.2g stochastic seismic ground motion, which is the lowest probability than other reinforcement layers. That is, for the same seismic deformation the middle to bottom reinforcement layers generally have higher probability of occurrence than other reinforcement layers. Thus, an increase in reinforcement strength in the middle to bottom part of the soil slope may be needed in order to increase the reliability of the horizontal displacement of the slope face to the allowable design level.

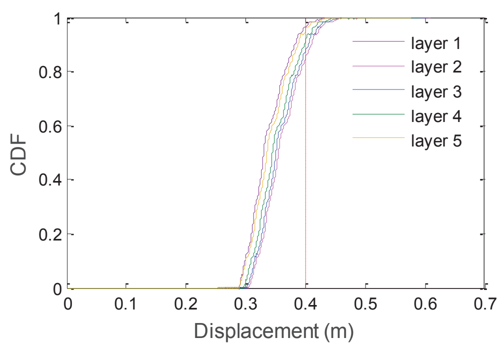

Figure 6. The CDF of the maximum horizontal displacement of the reinforcement soil slope at different reinforcement layers.

5 Conclusion

In order to fully understand the seismic performance of geosynthetic-reinforced slopes, this paper presented the probabilistic seismic analysis of a geosynthetic-reinforced soil slope under 0.2g stochastic seismic ground motion using the probability density evolution method (PDEM). The results show that for the same seismic deformation, the middle to bottom reinforcement layers are easier to exceed the limit probability level than other layers, therefore the reinforcement strength in the middle to bottom part of the reinforced soil slope may be needed to be enhanced so as to increase the reliability of the horizontal displacement of the slope face to the allowable design level. The study also indicated that the PDEM is an efficient method to carry out probabilistic seismic analysis of geosynthetic reinforced soil slopes. The dynamic deformation reliability analysis of geosynthetic reinforced soil slopes could serve as a reference of the design and assessment of geosynthetic reinforced soil slopes. The present study only considers the uncertainty of earthquake, the uncertainty of soil and reinforcement will be considered in the future work.

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