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Stochastic seismic response analysis of engineering site considering correlations of critical soil dynamic parameters

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Abstract: This paper presents a method to generate random samples of soil dynamic shear modulus and dynamic damping curves with full consideration of the correlations of critical soil dynamic parameters to investigate the influence of their uncertainties on the engineering site seismic response in the implementation of equivalent linear method. A one-dimensional (1D) equivalent linear site seismic response analysis program, which serves stochastic dynamic response analysis of the engineering site, has been developed in MATLAB. A 1D free field model for the typical layered engineering sites of site class II is established in this study. The target response spectra, which are defined with spectra of the outcrop corresponding to different earthquake return periods based on the acceleration response, are employed to generate artificial seismic records. These records are scaled down by ½ and referred to as input motions at the engineering bedrock for the 1D free field model. The numerical results show that the uncertainties of the critical soil dynamic parameters has significant influence on seismic response of engineering sites, which are highly related to the amplitude and the frequency aspects of the input ground motions as well as the fundamental periods of the engineering sites. The variations of the peak shear strain and the peak ground acceleration of the site, which reach 10% and 14%, respectively, increase with the amplitudes of the input ground motions. Besides, the variations of acceleration response spectra corresponding to the plateaus of the target response spectra and the fundamental periods of engineering sites exceeds 20% with the consideration of the uncertainties of soil dynamic parameters.

Keywords: site seismic response analysis; soil dynamic parameters; uncertainty; correlation; equivalent linear method

1 Introduction

The existing earthquake damage studies show that the site effect has a significant impact on the earthquake damage distribution and damage degree of engineering structures, and the seismic response analysis plays an important role in the seismic design of engineering structures[1−4]. The variance of some factors causes uncertainty for the analysis and prediction of seismic response of actual engineering site. These factors, for example, can involve four aspects: analysis method, input ground motion, nonlinear dynamic soil parameters and site shear wave velocity profile (Idriss [5]). Bradley[6] summarized the uncertainty of site seismic response into site characteristics, parameters of soil constitutive model, soil constitutive model and analysis method. Therefore, the quantification of their corresponding influence on the site seismic response analysis would be significant in engineering practice.

In recent years, a large number of scholars in this field have carried out a series of studies around the uncertainty factors in site seismic response analysis. Tsai et al.[7] classified the site seismic response analysis methods into three categories: frequency-domain equivalent linear method, time-domain nonlinear method and time-domain equivalent linear method. The first two have been introduced in software DEEPSOIL, whilst the other was used in the QUAD4M software, to analyze the seismic response of three typical sites, respectively. Kaklamanos et al.[8] selected 6 typical sites and 191 seismic records in KIK-net, compared the site response analysis results under linear, equivalent linear and nonlinear methods, and established a mixed effect model to quantify the difference between each method and actual seismic responses. Andrade et al.[9] considered the uncertainty of shear modulus, modulus curve and damping curve through Monte Carlo method. They adopted boundary surface plastic model and equivalent linearization method respectively to conduct dynamic response analysis on the two sites. The sensitivity of the results on different parameters were measured through the Aristotle strength and acceleration response spectrum of the site surface. Chen et al.[10] studied the deep soft ground along Nanjing Metro. They investigated the influence of soil shear wave

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velocity variability, dynamic shear modulus ratio and damping ratio variability on the peak ground acceleration and response spectrum of the site through the point estimation method. Their research highlighted the significant influence of soil shear wave velocity variability on the ground acceleration and response spectrum, compared to that of dynamic shear modulus ratio and damping ratio variability. Guzel et al.[11] used the dynamic hardening elastoplastic soil constitutive model to characterize the nonlinearity of the soil, and used the Monte Carlo method to describe the uncertainties of the shear modulus, modulus curve and damping curve of the soil. By comparing the acceleration response spectrum, the standard deviation of the peak ground surface acceleration and peak shear strain distribution of the site surface under two different intensities of ground motions, they analyzed the sensitive factors of the site nonlinear seismic response analysis. The factors have been concluded as initial shear modulus under small amplitude ground motion, whereas the modulus curve and damping curve under large amplitude ground motion. Boaga et al.[12] considered the influence of the uncertainty of the damping curve in the equivalent linear method under the condition of small epicenter earthquake (medium strain) on the site seismic response analysis. They took the transfer function of the ground surface relative to the bedrock as the measurement factor. The research results show that considering the uncertainty of the damping curve will enlarge or reduce the low-frequency component of the ground acceleration response close to the basic period of the site by 20%. The high-frequency component of the ground acceleration response changes by 60% under large wave impedance of the site. Griffiths et al.[13] considered the influence of the uncertainty of the site shear wave velocity profile on the site seismic response analysis. They generated the site shear wave velocity profile samples through three different methods, compared the surface acceleration response spectrum and its ratio under linear elasticity and equivalent linear models. The root mean square error of frequency dispersion curve and acceleration response spectrum of each site were calculated, and the relationship between the uncertainty of shear wave velocity profile and frequency dispersion was explored. Darendeli[14] made a quantitative evaluation on the influence of various dynamic soil parameters. They built up the mean and envelope curves for different soil types. The site seismic response was calculated considering the variance of modulus curve and damping curve. Their research indicates the significant impact on the analysis caused by uncertainty of soil dynamic parameters.

The above literatures mainly focus on the uncertainty of ground motion and the impact of analysis method on site seismic response, while the uncertainty of soil dynamic parameters is not fully considered. Most cases simplify the soil dynamic parameters as random independent variables. The range of their values lacks convincing test basis. The correlation among dynamic parameters is not considered. The influence degree of soil dynamic parameter uncertainty on site seismic response is not systematically quantified and discussed. In summary, in view of the lack of research on site seismic response analysis of soil dynamic parameters uncertainty, this study has been carried out based on the frequency domain equivalent linear analysis method. The artificial earthquake motions compatible with the rock outcrop response spectra with different seismic intensities are used as input, and the uncertainty and correlation of dynamic shear modulus curve and damping curve are considered. This paper quantitatively evaluates the influence of uncertainty of soil nonlinear dynamic parameters on site seismic response analysis, and provide reference and guidance for reasonable selection of soil dynamic parameters and prediction of site seismic response.

2 Methodology and verification

2.1 Frequency domain equivalent linear analysis method

In this paper, the dynamic shear modulus model (MKZ model) of soil mass related to confining pressure revised by Matasovi[15], based on the hyperbolic model of Konder and Zelasko (1963), is adopted. The skeleton curve formula and dynamic shear modulus ratio are

$$\tau = \frac{G_{\max}\gamma}{1+\beta\left(\dfrac{\gamma}{\gamma_{\mathrm{r}}}\right)^{s}} \tag{1}$$

$$\frac{G}{G_{\max}} = \frac{1}{1+\beta\left(\dfrac{\gamma}{\gamma_{\mathrm{r}}}\right)^{s}} \tag{2}$$

where τ is the shear stress; β and s are fitting parameters for correcting the shape of the skeleton curve; γ is the shear strain; γ_{r} is the reference shear strain, which is a parameter related to confining pressure and represents the equivalent shear strain value when the modulus curve $G/G_{\max}$ is 0.5; G and $G_{\max}$ are the dynamic shear modulus and initial shear modulus.

According to the skeleton curve and Masing's law, the hysteretic curve of soil under dynamic load is constructed, and the equivalent damping ratio is calculated based on the principle of energy equivalence. In order to avoid the phenomenon of over damping under large strain when the equivalent damping ratio obtained according to Masing's law is compared with the test curve, the software DEEPSOIL adopts the modulus reduction and damping curve fitting (MRDF) proposed by Phillips et al.[16] to eliminate the phenomenon of over damping under large strain. In addition, this program selects the empirical formula of damping curve improved by Chen et al.[17–18] on the basis of Hardin et al.[19] to directly fit the given test curve, as shown in the following formula:

$$D = D_{\min} + D_{\max}\left(1 - G/G_{\max}\right)^{n} \tag{3}$$

where D is the dynamic damping ratio; $D_{\min}$ is the small strain damping; $D_{\max}$ and n are fitting parameters.

2.2 Site information and input ground motion

In this paper, a typical site profile in Tongzhou, Beijing is selected as an analysis case. The site is composed of artificial fill, silty clay, fine–medium sand, fine silty sand and pebble from top to bottom. The equivalent shear wave velocity calculated according to *Code for Seismic Design of Urban Rail Transit Structures* (GB50909 – 2014)[20] is 237 m/s, which belongs to class II site. The basic frequency of the site is calculated as 1.65 Hz according to the transfer function method[21–22], corresponding to a fundamental period of the site of 0.604 s, which is basically consistent with the result of the basic period of the horizontal layered site (0.623 s) using the method proposed by Qi et al.[22]. The physical parameters of the site calculated in this paper are shown in Table 1.

Table 1 Type Ⅱ site soil physical parameters

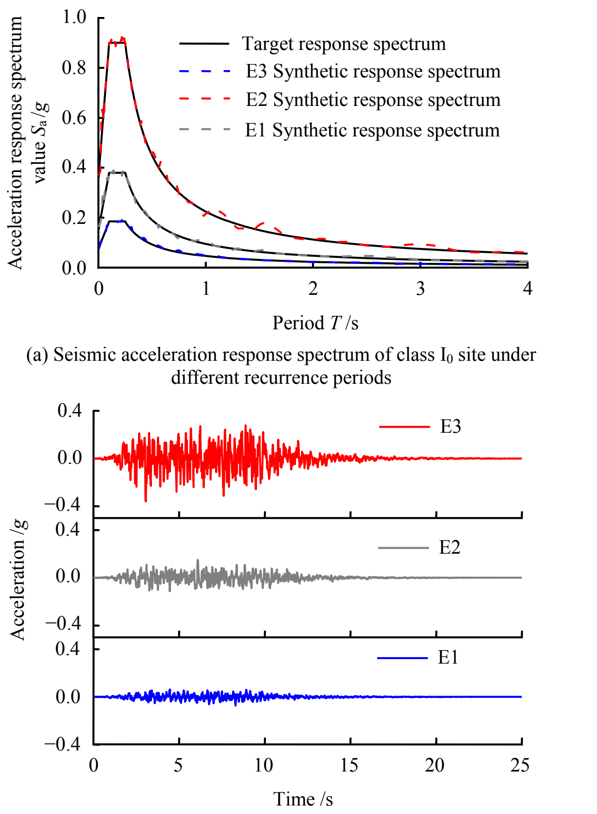
Soil layer	Soil type	Thickness /m	Density /(kg • m^{-3})	Shear wave velocity /(m • s^{-1})	Shear modulus /MPa
1	Backfill	5.0	1 750	180	57.0
2	Silty clay	10.0	1 900	250	118.8
3	Fine-medium sand	10.0	2 000	300	180.0
4	Fine silty sand	15.0	2 000	320	205.0
5	Pebble	20.0	2 280	500	525.0

In order to explore the influence of the uncertainty of soil dynamic parameters on the seismic response of the site under intensities of earthquake motions and also eliminate the interference of the uncertainty of input ground motions, the design response spectrum of class I$_0$ site is used as the target acceleration response spectrum in this paper. According to the provisions of *Code for Seismic Design of Urban Rail Transit Structures*[20], the equivalent shear wave velocity of class I$_0$ site is greater than 800 m/s, and the characteristic period T_g of acceleration response spectrum is 0.25 s, which can be approximated as the ground motion response spectrum of rock outcrop. In this paper, the design response spectra of class I$_0$ sites under three seismic intensity levels with recurrence periods of 100, 475 and 2 450 a are taken as the target acceleration response spectra, and the design peak ground acceleration $a_{\max}$ is 0.074g, 0.152g and 0.360g respectively. The conversion formula of response spectrum and success rate spectrum proposed by Kaul [23] is adopted:

$$S(\omega) = \frac{\zeta}{\pi\omega}\left[S_{\mathrm{a}}^{\mathrm{T}}\left(\zeta,\omega\right)\right]^{2} \frac{1}{\ln\left[-\dfrac{\pi}{\omega t}\ln\left(1-r\right)\right]} \tag{4}$$

where S_{a} is the target acceleration response spectrum; ω is the circular frequency; ζ is the damping ratio, taken as 0.05 according to the code[20]; t is the earthquake duration; r is the probability of exceedance, which is taken as 0.15 following Kaul's suggestion.

The stationary stochastic ground motions are synthesized using the superposition of trigonometric function, and then multiplied by the piecewise envelope function (Jennings model) to convert it into a non-stationary process. The iterative correction Fourier spectrum method is adopted to obtain the results with an average error of less than 5% with the target spectrum referring to document[24] for details of the specific process. The artificial ground motions (E1, E2 and E3) under different seismic fortification intensities are shown in Fig. 1.



(a) Seismic acceleration response spectrum of class I$_0$ site under different recurrence periods

(b) Artificial acceleration time history curves

Fig. 1 Acceleration response spectra corresponding to different recurrence periods in Ⅰ$_0$ site and synthetic acceleration time histories corresponding to target spectra

According to the elastic medium wave theory, it is assumed that the bedrock is a uniform elastic half space medium, the amplitude of the seismic acceleration of the rock outcrop is always twice that of the incident seismic acceleration of the underlying bedrock[1]. Therefore, the generated artificial ground motion records of the rock outcrop are converted into half as the input ground motion of the underlying bedrock of the engineering site.

2.3 Verification of one-dimensional site dynamic analysis model

In this study, a one-dimensional equivalent linear seismic site response analysis program is developed based on MATLAB. The site response under E1, E2 and E3 ground motions is compared with the equivalent linear analysis results in DEEPSOIL[7, 16, 25−26] developed by Hashash team for program testing. The equivalent linearized soil dynamic parameters in this paper and DEEPSOIL are shown in Table 2 and Table 3. Fig. 2 is the statistical mean curve of a large number of soil dynamic test results of Seed et al.[27−28] and Vucetic et al.[29] for sand and clay respectively. The dynamic damping curve parameters in this paper are obtained by data fitting using Eq. (3), and the corresponding parameters are shown in Table 3.

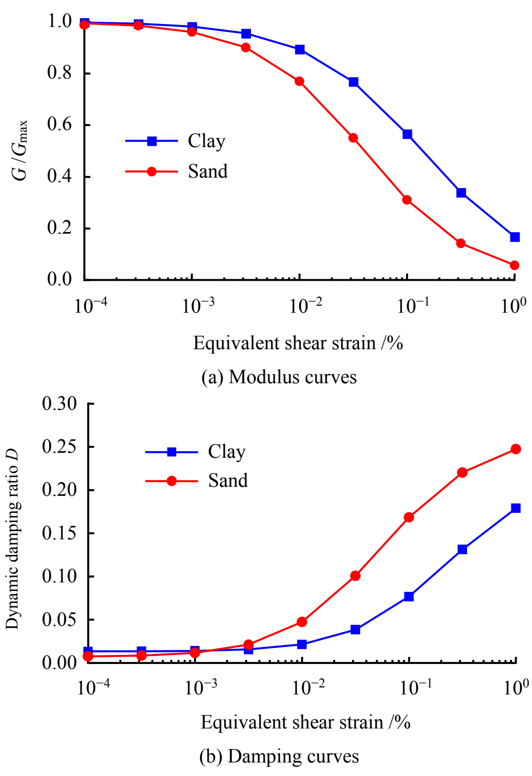
Table 2 Critical dynamic properties for sand and clay in DEEPSOIL[15]

Ref.	Soil type	Parameters						
		β	s	γ_r /%	ξ_{small} /%	p_1	p_2	p_3
[27]	Sand−mean value	1.000	0.870	0.040	0.381	0.980	0.380	1.850
[29]	Clay−mean value	1.000	0.810	0.139	0.953	0.960	0.360	0.500

Note: ξ_{small} is the small strain damping ratio; p_1−p_3 are the fitting parameters of damping reduction function in Deepsoil.

Table 3 Critical dynamic properties for sand and clay in this study

Ref.	Soil type	Parameters					
		β	s	γ_r /%	D_{min} /%	D_{max} /%	n
[27]	Sand−mean value	1.000	0.870	0.040	0.381	25.0	1.30
[29]	Clay−mean value	1.000	0.810	0.139	0.953	21.0	1.45

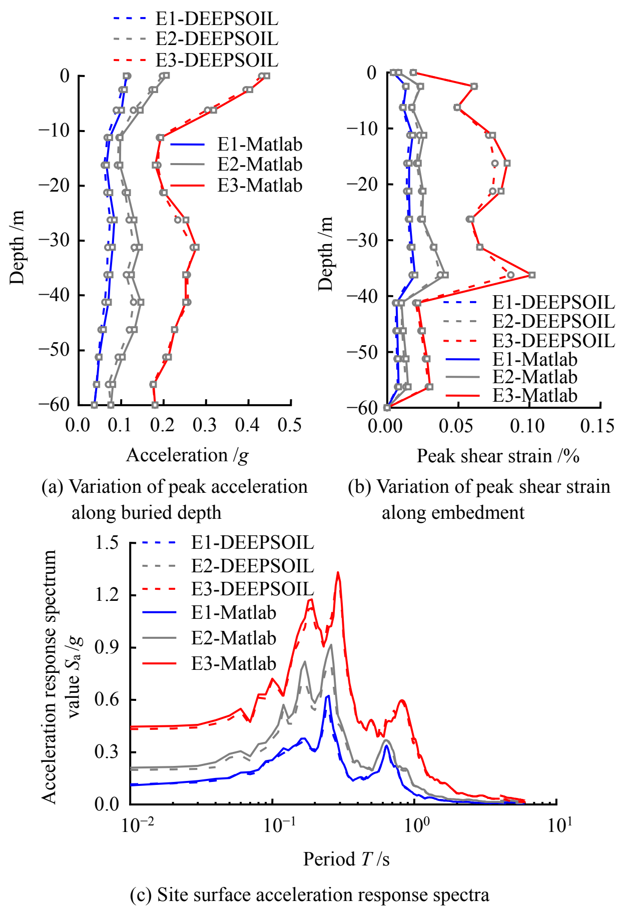


(a) Modulus curves

(b) Damping curves

Fig. 2 Dynamic shear modulus ratio and dynamic damping ratio of sand and clay

Figure 3 shows the comparison between the calculation results of this program and DEEPSOIL under E1, E2 and E3 ground motions.



(a) Variation of peak acceleration along buried depth

(b) Variation of peak shear strain along embedment

(c) Site surface acceleration response spectra

Fig. 3 Comparison of calculation results between the developed program and DEEPSOIL

As illustrated in Fig. 3, under the excitation of E1, E2 and E3 earthquakes, the peak acceleration and peak shear strain distribution curve along the buried depth and the site surface acceleration response spectrum calculated by the program developed in this paper have reached good agreement with the calculation results of DEEPSOIL, which verifies that the calculation case of this program is correct and reasonable. The model philosophy can be applied to the large-scale calculation and analysis of the uncertainty of the site soil parameters.

3 Uncertainty model of soil dynamic parameters

In order to consider the uncertainty of soil dynamic parameters and the seismic response of the site, based on the statistical analysis of a large number of soil dynamic test results of sand and clay by Seed et al.[27−28] and Vucetic et al.[29], this paper gives the numerical characteristics and probability distribution forms of various parameters of uncertainty modeling of sand and clay modulus curves, as shown in Table 4. Table 5 shows the correlation between parameters of sand and clay. The correlation coefficients of parameters s and γ_{r} are achieved by curve fitting of the shape of modulus curves of sand and clay. According to the negative correlation between dynamic shear modulus and damping ratio, the correlation coefficients of parameters $D_{\min}$ and $D_{\max}$ and γ_{r} are all taken as −0.95.

Table 4 Statistical characteristics and probability distributions of dynamic properties for sand and clay

Sand								Clay							
s		γ_{r}		$D_{\min}$		$D_{\max}$		s		γ_{r}		$D_{\min}$		$D_{\max}$	
Mean value	Coefficient of variation	Mean value /%	Coefficient of variation	Mean value /%	Coefficient of variation	Mean value /%	Coefficient of variation	Mean value	Coefficient of variation	Mean value /%	Coefficient of variation	Mean value /%	Coefficient of variation	Mean value /%	Coefficient of variation
0.87[a]	0.04[b]	0.04[a]	0.2[b]	0.381[b]	0.15[b]	25[b]	0.05[b]	0.81[a]	0.01[c]	0.139[a]	0.3[c]	0.953[c]	0.02[c]	21[c]	0.02[c]

Note: a represents reference [16], b represents data fitting in reference [27−28], and c represents data fitting in reference [29]. The parameters not listed in the table shall be taken as the mean value according to the dynamic parameters of sand and clay in Table 3. All parameters are lognormal distribution.

Table 5 Correlation between critical dynamic properties for sand and clay

Parameters	s	γ_{r}	$D_{\min}$	$D_{\max}$
Correlation between sand parameters and γ_{r}	0.95	1	−0.95	−0.95
Correlation between clay parameters and γ_{r}	0.00	1	−0.95	−0.95

Based on the research of existing papers, the key parameters of each soil mass in Table 4 obey lognormal distribution[11−12], and there are correlations between them, that is, the soil mass parameters s, $D_{\min}$ and $D_{\max}$ are correlated non normal random variables. Generating correlated non normal random samples is realized by the inverse Nataf transformation method [30−31], which can transform independent normal random variables into correlated non normal random variables.

Figure 4 shows the random sample generation process considering the correlation of soil dynamic parameters in this paper, which is mainly divided into the following steps:

(1) Monte Carlo method generates independent standard normal random variables $\boldsymbol{S}^{\mathrm{U}}, \boldsymbol{\gamma}_{\mathrm{r}}^{\mathrm{U}}$, $\boldsymbol{D}_{\min}^{\mathrm{U}}$ and $\boldsymbol{D}_{\max}^{\mathrm{U}}$.

(2) According to the correlation coefficients among the given parameters in Table 5, the equivalent correlation coefficients among the parameters are computed through the empirical formula $\boldsymbol{\rho}' = \boldsymbol{F}' \cdot \boldsymbol{\rho}$ proposed by Liu et al. [32].

(3) The lower triangular matrix $\boldsymbol{L}_0$ is obtained by Cholesky matrix decomposition of the equivalent correlation coefficient matrix $\boldsymbol{\rho}'$;

(4) The independent standard normal random variables are transformed into the correlated standard normal random variables $\boldsymbol{S}^{\mathrm{Y}}, \boldsymbol{\gamma}_{\mathrm{r}}^{\mathrm{Y}}, \boldsymbol{D}_{\min}^{\mathrm{Y}}$ and $\boldsymbol{D}_{\max}^{\mathrm{Y}}$ by linear transformation.

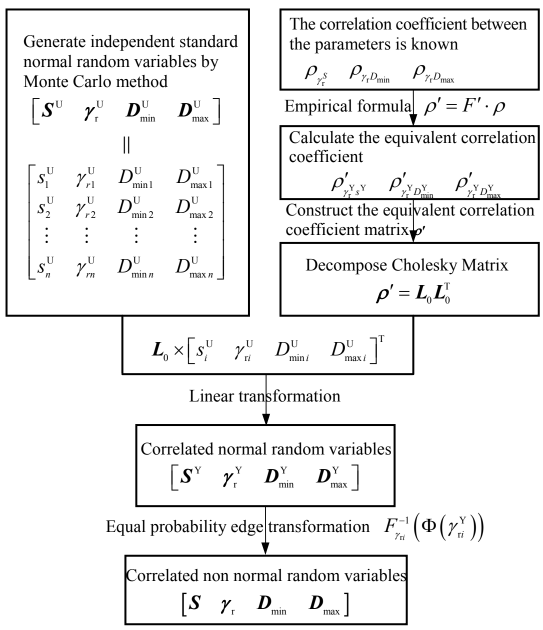
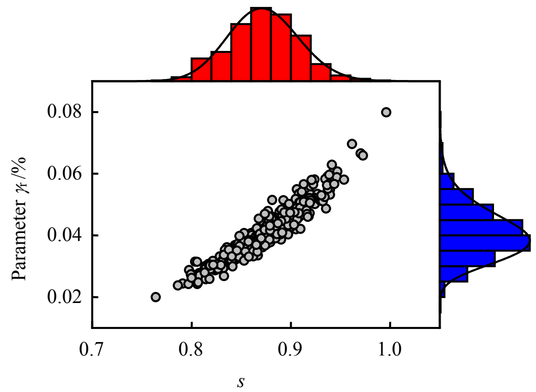


Fig. 4 Flowchart of random sample generation considering the correlation among parameters

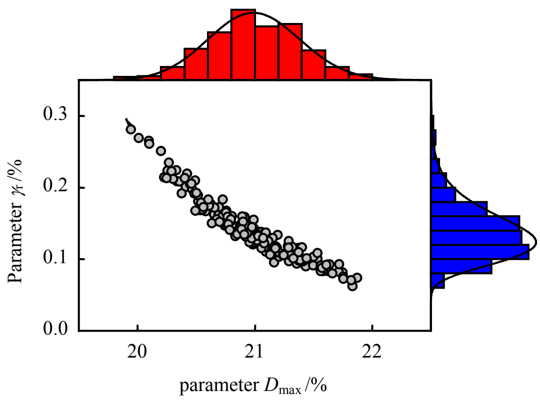
(5) According to the equal probability edge transformation law, the edge distribution of the correlated standard normal random variables is converted to the

corresponding log normal edge distribution, and the correlated non normal random variables $\boldsymbol{S}, \boldsymbol{\gamma}_{\mathrm{r}}$, $\boldsymbol{D}_{\min}$ and $\boldsymbol{D}_{\max}$ are obtained.

Under the condition that the probability distribution and correlation coefficient of key dynamic parameters of sand and clay are clarified, the frequency distribution histogram and samples of dynamic modulus curve parameters of sand γ_{r} and s generated according to the flowchart in Fig. 4 are shown in Fig. 5 (a). Fig. 5 (b) illustrates the frequency distribution histogram and samples of dynamic modulus curve parameters γ_{r} of clay and dynamic damping curve parameter $D_{\max}$. It can be seen that this method fully considers the correlation among parameters, and the generated values are consistent with the given probability distribution form and numerical statistical characteristics. The random curves of dynamic shear modulus ratio and dynamic damping ratio can be obtained by substituting the randomly generated soil parameter samples into Eqs. (2) and (3).

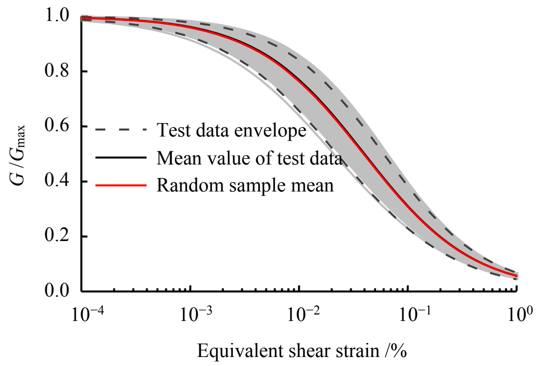


(a) Marginal map of parameters γ_{r} and s (sand)

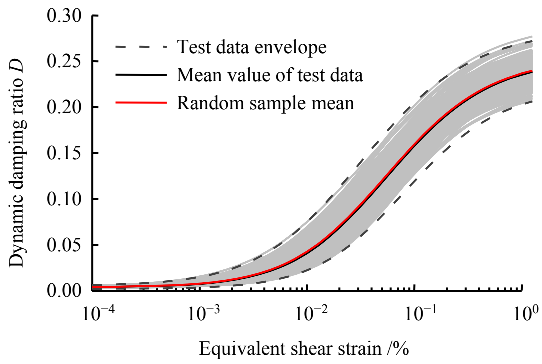


(b) Marginal map of parameters γ_{r} and s (clay)

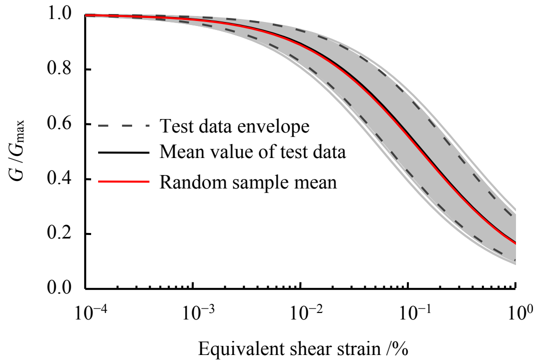
Fig. 5 Correlated lognormal distribution of dynamic parameters for sand



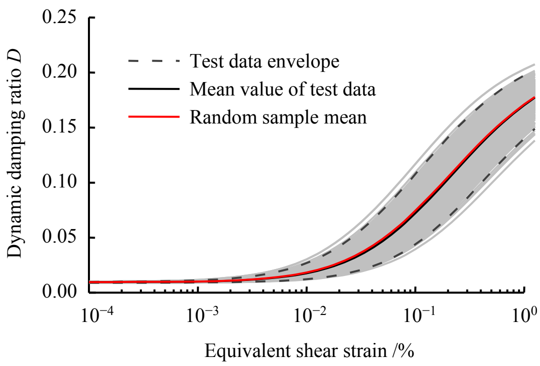
(a) Sand modulus curves



(b) Sand damping curve



(c) Clay modulus curves

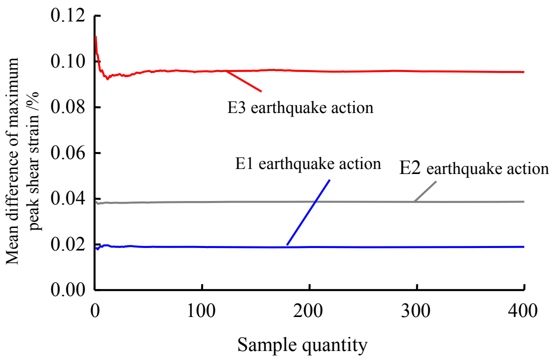


(d) Clay damping curves

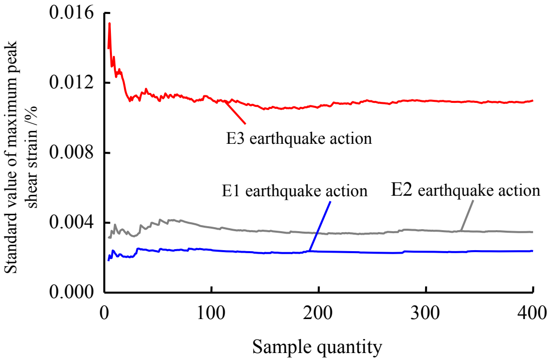
Fig. 6 Uncertainty of dynamic modulus curves and dynamic damping curves of sand and clay

Figure 6 gives the dynamic shear modulus ratio and dynamic damping ratio curves of sand and clay generated based on the 400 realizations Monte Carlo method. In this study, both corresponding curves of sand and clay are employed. The mean value and envelopes obtained from the tests of Seed et al.[27−28] and Vucetic et al.[29] are taken as reference, as shown by the dotted line in Fig. 6. It can be seen that the method in this paper takes the correlation between modulus and damping curve parameters into account, and the generated random sample mean curve is also in good agreement with the mean curve obtained by the existing test statistics. It can be seen that the generated random curve is basically within the given range of the existing test results.

The convergence analysis results of the mean and standard deviation of the maximum peak shear strain under different ground motion intensity levels is shown in Fig. 7 to verify that the number of random samples for Monte Carlo analysis in this study is sufficient.



(a) Mean convergence



(b) Standard deviation convergence

Fig. 7 Convergence of mean and standard deviation of maximum peak shear strain of engineering site obtained from Monte Carlo Simulation method

It can be seen in this study, when the number of samples reaches 300 times, the mean value and standard deviation of the maximum peak shear strain of the site under the action of different ground motions all converge. Therefore, under the ground motions of different strength levels, 400 Monte Carlo realizations is deemed sufficient to obtain reliable statistical characteristics of the site dynamic response.

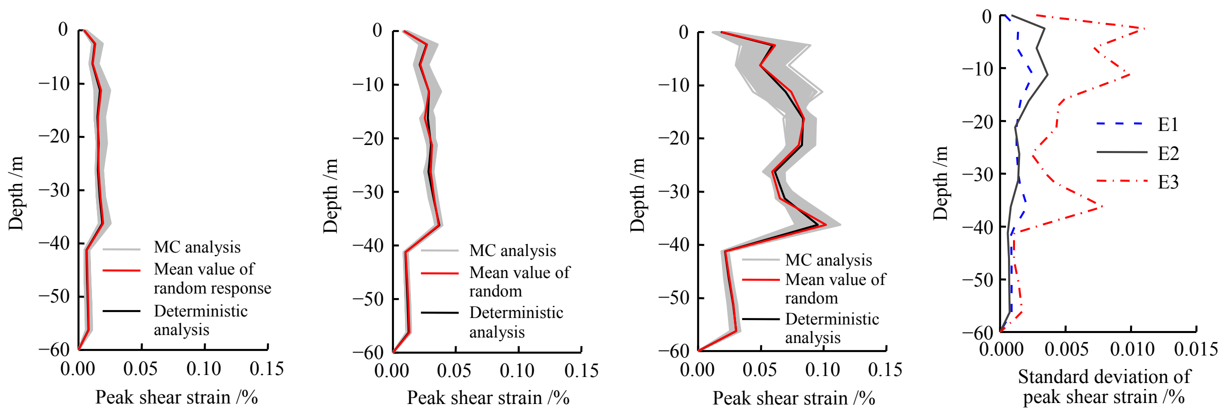
4 Site seismic response analysis results

In order to evaluate the influence of uncertainty of soil dynamic parameters on site seismic response under different seismic intensities. The synthetic artificial rock outcrop earthquake acceleration time histories corresponding to E1, E2 and E3 intensity levels are scaled to half and used as the input ground motion of the underlying bedrock of the engineering site. A large number of one-dimensional equivalent linear site response analysis are carried out, with a total of 1 200 analysis times. The distribution of the peak response along the buried depth and the response spectrum of ground acceleration under different intensity ground motions are analyzed and compared.

4.1 Distribution of site peak acceleration and peak shear strain along buried depth

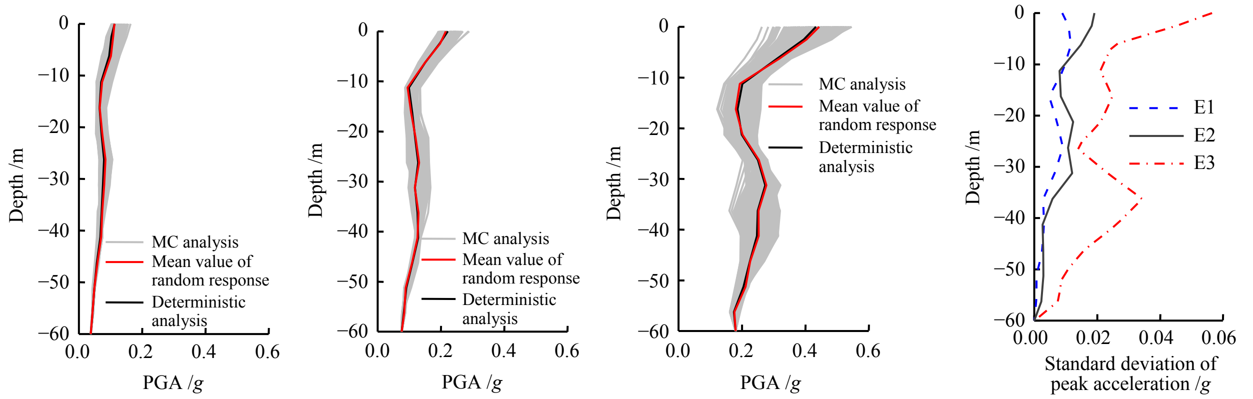
Figures 8 and 9 show the comparison of the response calculation results of the site peak acceleration profile and peak shear strain profile considering the uncertainty of soil dynamic parameters under the action of E1, E2 and E3 ground motions.

It can be observed from Fig. 8 that the mean curve from the Monte Carlo calculation basically coincides with the results of the deterministic analysis. The maximum peak shear strain of the site occurs at 22.5 m from the seismic action surface (bedrock). Under the action of E1, E2 and E3 earthquakes, the average value of the maximum peak shear strain is 0.02%, 0.04% and 0.09% respectively. Considering uncertainty of soil dynamic parameters, with the increase of ground motion intensity, the dispersion of site peak response under the same burial depth gradually increases (see Fig. 8 (d)).



(a) E1 ground motion (b) E2 ground motion (c) E3 ground motion (d) Standard deviation of peak shear strain

Fig. 8 Calculated results of site peak shear strain profile under E1, E2 and E3 ground motions



(a) E1 ground motion (b) E2 ground motion (c) E3 ground motion (d) Standard deviation of peak acceleration

Fig.9 Calculated results of site peak acceleration profile under ground motion of E1, E2 and E3

Figure 9 presents similar trend to Fig. 8. It indicates that the mean curve of Monte Carlo calculation results basically matches the results of deterministic analysis, and the maximum peak acceleration value appears on the ground surface. Excited by E1, E2 and E3 earthquakes, the average value of the maximum peak acceleration is 0.12g, 0.20g and 0.41g respectively. When considering the uncertainty of soil dynamic parameters, with the increase of ground motion intensity, the fluctuation range of peak response along the buried depth of the site will gradually increase, as shown in Fig. 9 (d).

The mean and standard deviation of the maximum peak shear strain and maximum peak acceleration response of the site under different ground motion intensity levels in Fig. 8 and Fig. 9 are statistically analyzed, as shown in Table 6.

Table 6 Mean and standard deviation of the maximum peak acceleration and the maximum peak shear strain of the site

Site response	Mean value of maximum peak shear strain /%	Standard deviation of maximum peak shear strain /%	Mean value of maximum peak acceleration /g	Standard deviation of maximum peak acceleration /g
E1 earthquake action	0.02	0.002 5	0.12	0.010
E2 earthquake action	0.04	0.004 0	0.20	0.020
E3 earthquake action	0.09	0.011 0	0.41	0.058

The variation coefficient of the maximum peak response can be calculated from the mean and standard deviation of the maximum peak shear strain and the maximum peak acceleration of the site under different intensity ground motion levels, as shown in Fig.10. It can be found that after considering the uncertainty of soil dynamic parameters, the fluctuation ranges of the peak shear strain of the site under the action of E1, E2 and E3 ground motions is about 10%, and the fluctuation range of the peak acceleration of the site increases with the ground motion intensity level, which is 8%, 10% and 14% respectively.

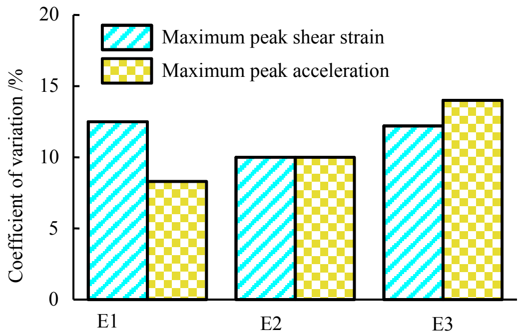


Fig. 10 Coefficients of variation of the maximum peak shear strain and the maximum peak acceleration

4.2 Site surface acceleration response spectrum

The calculation results of the ground acceleration response spectrum of the site are displayed in Fig.11 under the action of E1, E2 and E3 ground motions. The mean value of the ground acceleration response spectrum obtained by Monte Carlo calculation is in good agreement with the mean value of the deterministic analysis. The 0.10−0.25 s and 0.6−1.0 s sections of the ground acceleration response spectrum are more prominent than other periods, which correspond to the platform section of the artificial ground motion response spectrum and the basic period of the site. The increase of ground motion intensity leads to larger fluctuation range of ground acceleration response spectrum.

Figure 12 depicts the standard deviation of the ground acceleration response spectrum of the site under the action of E1, E2 and E3 earthquakes. With the increase of the ground motion intensity, the peak shear strain of the site soil layer increases. Greater uncertainty of the damping ratio curve leads to larger standard deviation of the acceleration response spectrum on the ground surface. In addition, the uncertainty of the ground acceleration response spectrum corresponding to the design response spectrum platform section (0.10−0.25 s) and the site basic period (0.6−1.0 s) is greater than that of other sections, with the fluctuation range 10%−20%. The corresponding value of basic period less than 0.1 s is 8%−15%.

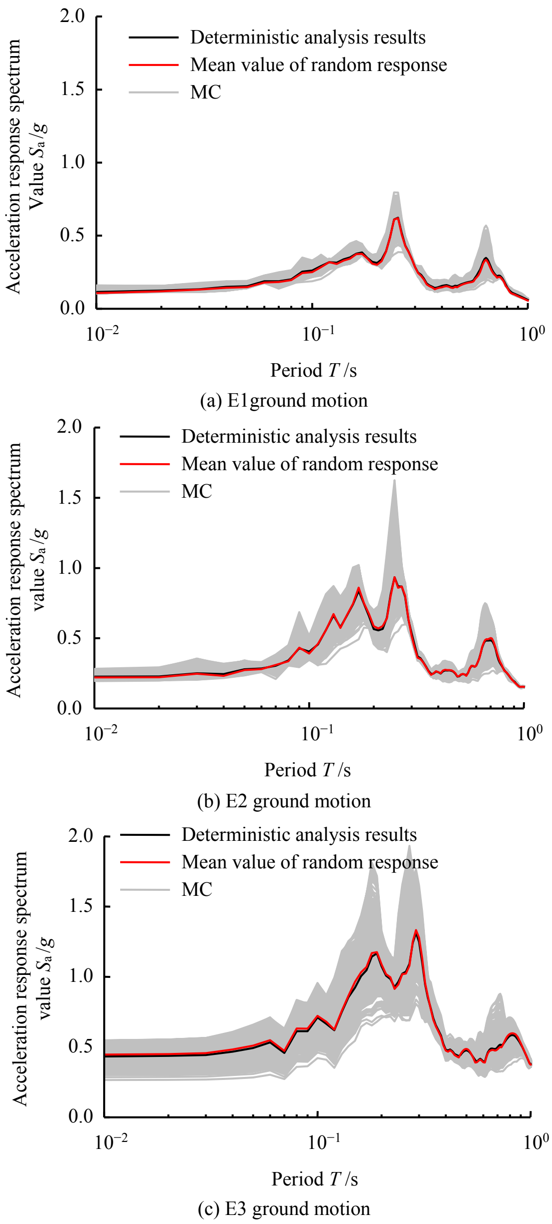


Fig. 11 Comparison of acceleration response spectra at ground surface under E1, E2 and E3 ground motions

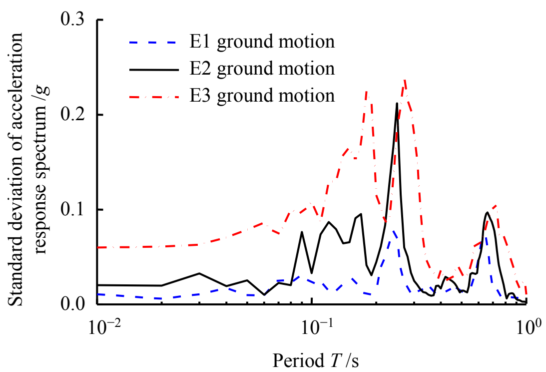


Fig. 12 Standard deviations of surface acceleration response spectra under E1, E2 and E3 earthquakes

5 Conclusions

(1) Based on the statistical results of the existing soil dynamic parameters test and the one-dimensional site dynamic analysis, the uncertainty of the site soil dynamic parameters has a great impact on the site seismic response. The degree of this impact is highly related to the ground motion intensity, spectrum components and the basic period of the site.

(2) Under the action of E1, E2 and E3 ground motions, the fluctuation range of the site peak shear strain is 10%, and the fluctuation range of the site peak acceleration increases with the ground motion intensity level, with 8%, 10% and 14% respectively.

(3) The uncertainty of the ground acceleration response spectrum of the site is determined on the ground motion intensity level, ground motion frequency components and the basic period of the site. The uncertainty of the ground acceleration response spectrum at the design response spectrum platform section (0.10−0.25 s) and the basic period of the site (0.6−1.0 s) is greater than that of other sections, and the fluctuation range is 10%−20%. The corresponding value of basic period less than 0.1 s is 8%−15%.

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