Earthq Eng & Eng Vib (2017) 16: 487-498 **DOI**:10.1007/s11803-017-0401-1

Accuracy of three-dimensional seismic ground response analysis in time domain using nonlinear numerical simulations

Liang Fayun†, Chen Haibing‡ and Huang Maosong†

State Key Laboratory of Disaster Reduction in Civil Engineering, International Joint Research Laboratory of Earthquake Engineering, Tongji University, Shanghai 200092, China

Abstract: To provide appropriate uses of nonlinear ground response analysis for engineering practice, a three-dimensional soil column with a distributed mass system and a time domain numerical analysis were implemented on the OpenSees simulation platform. The standard mesh of a three-dimensional soil column was suggested to be satisfied with the specified maximum frequency. The layered soil column was divided into multiple sub-soils with a different viscous damping matrix according to the shear velocities as the soil properties were significantly different. It was necessary to use a combination of other one-dimensional or three-dimensional nonlinear seismic ground analysis programs to confirm the applicability of nonlinear seismic ground motion response analysis procedures in soft soil or for strong earthquakes. The accuracy of the three-dimensional soil column finite element method was verified by dynamic centrifuge model testing under different peak accelerations of the earthquake. As a result, nonlinear seismic ground motion response analysis procedures were improved in this study. The accuracy and efficiency of the three-dimensional seismic ground response analysis can be adapted to the requirements of engineering practice.

Keywords: three dimensional soil column; seismic ground response; centrifugal model test; nonlinear analysis; accuracy verification

1 Introduction

The research in the past half century has shown that, when seismic waves propagate from deep soil or bedrock to the soil surface, it is necessary to implement nonlinear ground response analysis to account for soil dynamic properties with a high degree of nonlinearity (Hashash *et al*., 2010). At present, one-dimensional seismic ground response analysis is most commonly used in practice and research. These analyses are usually divided into two kinds of simplified methods: frequency domain analysis and time domain analysis. The frequency domain method is represented by Shake (Schnabel *et al*., 1972), which is based on the equivalent linear method, and is often used as a precise solution for one-dimensional seismic response of horizontal layered soils (Kwok *et al*. 2007; Hashash *et al*. 2010). However, for deep soft soil sites or sites under strong earthquakes, the equivalent linear method is not ideal (Hashash *et al*., 2010). Representative nonlinear seismic ground motion response analyses codes such as DEEPSOIL (Hashash *et al*., 2012), Cyclic1D (Elgamal *et al*., 2012) and TESS (Pyke, 2000) are preferred. As the time domain nonlinear method reflects nonlinear deformation characteristics of the soil, the analysis is more reliable than the frequency domain equivalent linear method (Fiegel, 1995) because the stress-strain relation and the equilibrium of layered soils have been considered under dynamic loading. However, the time domain nonlinear analysis is strongly limited by artificial factors, such as the damping calculation parameters and the dynamic constitutive model of soil parameters (Kwok *et al*., 2007; Stewart *et al*., 2008).

At present, the results from one-dimensional free field analysis codes are mostly the acceleration and the stress-strain relationship (Hashash *et al*., 2010). Compared with these programs, using the finite element method can realize the free field analysis in three-dimensional conditions to obtain the seismic displacement response without the integral process on acceleration. Displacement of the seismic ground response analyses has the same importance in geotechnical engineering, and the free displacement response can be widely applied to the analysis of pile-soil structure interaction (Wang *et al*., 1998, Boulanger *et al*., 1999; Cubrinovski *et al*., 2009; Elnashai and Di Sarno, 2015) or to the evaluation

Correspondence to: Liang Fayun, State Key Laboratory of Disaster Reduction in Civil Engineering, International Joint Research Laboratory of Earthquake Engineering, Tongji University, Shanghai 200092, China
Tel: +86 21 6598 6072; Fax: +86 21 6598 5210
E-mail: fyliang@tongji.edu.cn
†Professor; ‡PhD Candidate

Supported by: National Natural Science Foundation of China under Grant No. 41672266
Received January 11, 2016; **Accepted** November 4, 2016

of the soil box boundary effect in shaking table test models (Wilson *et al*., 1998; Duran *et al*., 2016). In order to reduce the reflection of seismic waves within a finite soil layered space, the seismic ground response analyses using the finite element method needs to define the artificial viscoelastic boundary conditions. Although considering the influence of the far field stiffness is important to ensure accuracy, the parametric variability of the modeling method cannot be implemented in engineering practice (Maheshwari *et al*., 2004).

Based on the framework of multi-surface plasticity, Yang *et al*. (2008) implemented a procedure with a three-dimensional soil element under sinusoidal base shaking on the OpenSees simulation platform (OpenSees, 2016). However, the precision of the seismic ground motion nonlinear analysis related to modeling of the free field, selection of soil dynamic constitutive parameters and soil damping were indeterminate. It is therefore necessary to determine the factors that influence the accuracy of the three-dimensional nonlinear seismic ground motion response analysis using finite element methods in engineering practice.

To implement nonlinear numerical simulation of horizontal soil response analyses under earthquake on the OpenSees simulation platform, this study aims to clarify the three-dimensional soil column modeling of free field with the specification of a dynamic soil nonlinear constitutive model [UCSD soil model (Elgamal *et al*., 2003; Yang *et al*., 2008)] and damping parameters. Compared with centrifugal dynamic experiments in different peak ground accelerations, the calculated results from the frequency domain equivalent linear and time domain nonlinear analyses are also used for comparative analysis. The ground response analysis of the three-dimensional (3-D) soil column finite element model accurately presents soil acceleration and displacement time histories, thus resolving the issues above and providing an appropriate reference for use in engineering practice.

2 Model implementation

2.1 Modeling method and element type

Distributed mass system of the continuum soil column simulates the horizontal layered site as shown in Fig. 1. The soil column only contains one unit in the horizontal x and y directions, and is set to a length of 1.0 m. The computational efficiency and precision are associated with the vertical partition thickness H of the soil element and the frequency, which can go through the element thickness. The maximum frequency should be greater than $f_{max}=V_s/4H$, where V_s is the shear wave velocity. Generally speaking, the seismic wave frequency of acceleration is mainly concentrated in the 15 Hz range. When the high frequency response of the seismic wave in the different elements is stable, the vertical partition thickness of the soil column can satisfy the calculation precision. On the OpenSees simulation platform compared with the solid-fluid coupled elements proposed by Yang *et al*. (2008), the eight node SSP brickUP element realized by McGann *et al*. (2012) is a *u-p* element suitable for dynamic analysis of fluid saturated porous media deformation and stress, and can simulate the solid-fluid coupled effect in the seismic action (Zienkiewicz and Shiomi, 1984). Each single node of the soil element has four degrees of freedom, including three displacement components and a component of pore pressure. The nodes are free to drain at and above the groundwater table and zero drainage is enforced on the remaining boundaries.

2.2 Boundary conditions

In the process of improving the computational efficiency and precision, this study considers the three-dimensional soil column modeling method, and the relationship of the element nodes need to be constrained to simulate the far-field stiffness of the free field. As shown in Fig. 1, the node's plane is connected to two elements of soil to constrain the relevant movement of each point in the plane, where the displacement applied to the degrees of freedom in each node of 2, 3, and 4 is the same as that of Node 1. The soil element can only realize the horizontal shear deformation in the direction of x and y and the effective gravity deformation in z. The phenomenon where the boundary damping effect of the free field is reflected in the dynamic soil constitutive model is discussed in Section 2.4.

2.3 Earthquake input

The ground motion input is related to the boundary conditions at the bottom of the model. Two kinds of input methods are adopted. (1) When the bottom of the soil is bedrock, the seismic acceleration integration is

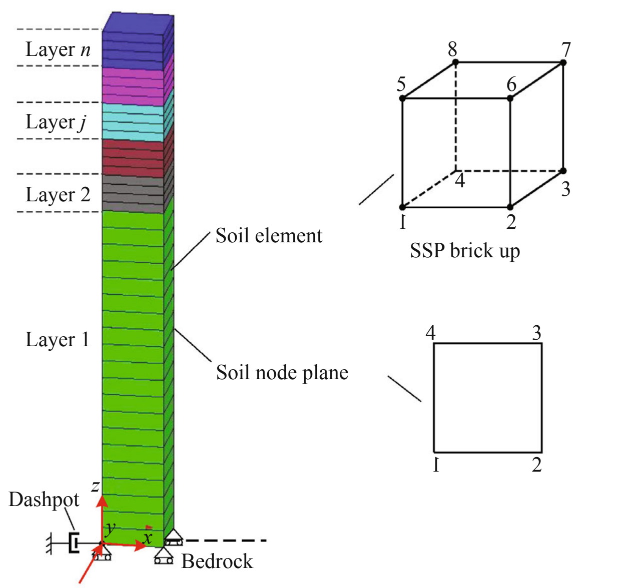

Fig. 1 Distributed mass system of horizontal layered soil column

converted into the velocity time history, multiplied by the boundary viscous damping coefficient, and which obtains the input form of the equivalent force that is then applied to the node at the bottom of the soil. The bottom boundary of the model is fixed vertically, and the Lysmer-Kuhlemeyer viscous boundary (Lysmer and Kuhlemeyer, 1969) is used in the horizontal direction of x and y to absorb the reflected wave due to setting artificial boundary to simulate the under bedrock. This treatment method is also used in DEEPSOIL (Hashash *et al*., 2012) and SUMDES (Li *et al*., 1992). The viscous damping coefficient of the contact surface between the bedrock and the soil is $\rho_E V_{sE} A_E$, where ρ_E is rock density, V_{sE} is the shear wave velocity of the bedrock, and A_E is the contact area between the bedrock and the soil column. (2) The other method used by Yang *et al*. (2008) is that the seismic acceleration schedule is added to the bottom node of the soil by the uniform excitation input mode, and the bottom boundary is fixed vertically and horizontally. However, this treatment method assumes that the bedrock is a rigid body.

2.4 Soil constitutive model

To perform seismic response analysis, the general three-dimensional finite element analysis needs to specify a range of soil space to simulate the real semi-infinite space, or set viscoelastic artificial boundary conditions to reduce the boundary reflection to seismic waves. This modeling method is mainly restrained by the dynamic soil constitutive model, and is not applicable for the soil static elastoplastic constitutive model to reflect the soil stress-strain relationship under earthquake loading. The UCSD soil model can reflect the complex stress path under earthquake loads and the effective stress response of the soil under irregular loads. The UCSD soil model can simulate the undrained features of cohesive soil under the rapid loading and can also be used to simulate the shear properties of sand to determine its reaction to the softening of strain and the pore water pressure changes. The response of octahedral shear stress-strain in the form of hyperbolic backbone curves is shown in Fig. 2, the soil nonlinear shear-strain behavior is approximately expressed in a piecewise linear relationship curve, and each linear segment represents a yield surface of f_m. The stress points move on multiple-surfaces as the the deviatoric stress continuously increases; correspondingly, the yield surface m = 1, 2,...,NYS, where NYS is the total number of surface yield, shear modulus H_m and the maximum yield surface $M_{NYS,}$ are defined by Stewart *et al*. (2008),

$$H_m = \frac{2\left(\tau_{m+1} - \tau_m\right)}{\gamma_{m+1} - \gamma_m} \tag{1}$$

$$M_{NYS} = \frac{6\sin\varphi}{3 - \sin\varphi} \tag{2}$$

where τ_{m+1}, τ_m and γ_{m+1}, γ_{m+1} are the τ~γ relationship of the adjacent yield surface to the hyperbolic backbone curve, φ is the friction angle, and H_{NYS} = 0 is on the outermost surface.

Both yield surface types of clay and sand are incorporated into the multi-surface framework on the OpenSees simulation platform. The clay of the UCSD soil model selects PressureIndependMultiYield materials (Yang *et al*., 2008), the associated rule is adopted, the yield function follows Von Mises shape which is only related to the undrained shear strength, and the yield surface is cylindrical in the effective stress space. PressureDependMultiYield02 materials are chosen for the sand model (Yang *et al*., 2008), the non-associated flow rule is used, the yield function follows Drucker-Prager shape, the yield surface is conical in the effective stress space as a function of friction angle and cohesion. Meanwhile, the hyperbolic backbone curve can be automatically generated from the UCSD soil model to determine the soil yield surface parameters, and the shear modulus reduction curves can also be used to generate yield surface; other relevant related parameters for the soil constitutive model are recommended by (Yang *et al*., 2003, 2008).

2.5 Modeling of damping

Kwok *et al*. (2007) summarized that general viscous damping or hysteretic damping was selected in nonlinear one-dimensional seismic ground response

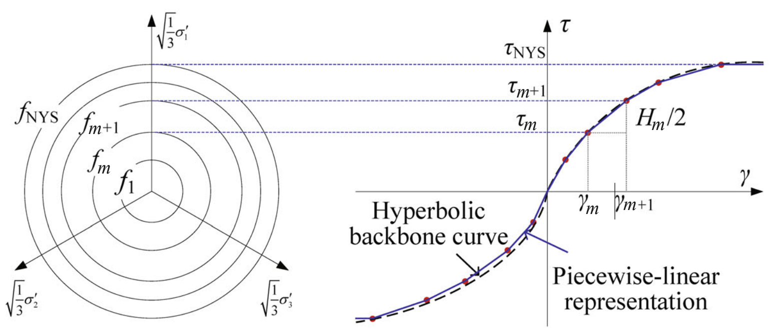

Fig. 2 Piecewise-linear representation in multi-surface plasticity (after Prevost, 1985; Stewart *et al*., 2008)

analyses. As the soil is viewed as a viscoelastic material, viscous damping can be used. When the high-frequency components of the seismic wave propagate within the soil deposit, the energy attenuation is faster than for low frequency components, and the propagation of the seismic wave to the surface is given priority over low frequency, which has an important influence on the displacement response. The Rayleigh damping is chosen as the available, approximate viscous damping and can effectively eliminate the vibration response with high frequency. Rayleigh damping assumes that the viscous damping matrix $\boldsymbol{C}$ is proportional to the mass matrix $\boldsymbol{M}$ and stiffness matrix $\boldsymbol{K}$,

$$\boldsymbol{C} = a_0\boldsymbol{M} + a_1\boldsymbol{K} \tag{3}$$

where a_0, a_1 is the coefficient of Rayleigh damping, respectively. The Rayleigh damping formulation is frequency dependent; for two specific frequencies, ω_m and ω_n, assume that the corresponding damping ratio ξ is the same, and there are proportions a_0 and a_1,

$$a_0 = \frac{2\xi\omega_m\omega_n}{\omega_m + \omega_n} \tag{4}$$

$$a_1 = \frac{2\xi}{\omega_m + \omega_n} \tag{5}$$

When the damping form related to the mass and stiffness is chosen, the damping ratio ξ, and frequency ω_m and ω_n are the key factors affecting the response results (Kwok *et al*., 2007). In the general nonlinear seismic ground response analysis, ω_m usually represents the fundamental frequency of the overall site (Park and Hashash, 2004;Kwok *et al*., 2007; Phillips and Hashash, 2009). It is suggested that ω_n should adopt $5\omega_m$ (Kwok *et al*., 2007) or a higher mode that corresponds to the predominant frequency of the input seismic wave (Phillips and Hashash, 2009). However, the corresponding frequency does not consider the effect of different properties of layered soil on the absorption and the filtering effect of different seismic frequency components. For the layered site, the same damping may artificially underestimate the capacity of the soil deposit to dissipate energy in some frequency bands and overestimate it in others. Natural soils exhibit greater frequency dependence and even relatively low frequencies have a noticeable effect on dynamic soil properties (Carvajal *et al*.,2002; Meng, 2007). Therefore, the layered site of the different soil properties has significantly different damping. It can be divided into several sub-soils depending on the shear velocities, and each subsystem chooses the corresponding Rayleigh damping parameters. A single sub-soil can be treated as a homogeneous soil, and the corresponding damping ratio ξ_i and the natural frequency ω_{mi}. ω_{ni} still select $5\omega_{mi}$ as recommended by Kwok *et al*. (2007), and the viscous damping matrix of a single sub-soil is formed by Eq.(3), where the natural frequency ω_{mi} is,

$$\omega_{mi} = \frac{\pi}{2H_i}V_{si} \tag{6}$$

where V_{si} and H_i are the corresponding shear wave velocities and thickness of each sub-soil.

3 Model validation

3.1 Overview of centrifugal model test

The dynamic centrifuge model results (Wilson *et al*., 1997) are used to verify the accuracy of the nonlinear seismic ground analysis using the 3-D soil column model under earthquakes. All sizes used the prototype to converse. The experiment consisted of two layers of soil. At the bottom was the dense sand with the relative density of 75%–80%, D_{50} = 0.15 mm, C_u = 1.5, and the sand density was 1.66 Mg/m^3. The upper layer was reconstituted soft clay with a liquid limit ≈ 88, and plastic index ≈ 48, the undrained shear strength was about 2.8–14.9 kPa, and the density of the soft clay was 1.53 Mg/m^3. To consider the constraint effect of a shear box on the soil, the soil unit weight was increased by 25% (Boulanger *et al*., 1999). The base plate was treated as a rigid substrate. As shown in Fig. 3, the maximum frequency f_{max} showed the soil model with an appropriate mesh and shear wave velocity obtained by Eq.(7),

$$V_s = \sqrt{\frac{G_{max}}{\rho}} \tag{7}$$

where ρ is soil density, for the sand, shear modulus G_{max} proposed by Seed *et al*. (1986) 's Eq. (8), and for the clay G_{max} was calculated by Eq. (9) (Boulanger *et al*., 1999),

$$G_{max} = 2.17K_{2,max}\sqrt{\frac{\sigma'_m}{P_{atm}}} \tag{8}$$

$$G_{max} = 380 \cdot c_u \tag{9}$$

where $\sigma'_m = (1 + 2K_0)\sigma'_v/3$, σ'_v is the vertical effective stress, K_0 = 0.6, P_{atm} is the atmospheric pressure, $K_{2,max}$= 65, and c_u is the undrained shear strength.

The input seismic wave of centrifugal vibration table was from the 1995 Kobe earthquake event, peak acceleration of input base motion a_{max} was 0.055 g, and the recorded acceleration data in the eastern basement was used to calculate the ground response, and measured a_{max} as 0.054 g. To identify the high frequency interference signal and the long-period trend, termin the acceleration signal is an important preprocess of the seismic data. The acceleration data collected in the shaking table test are susceptible to interference from external factors

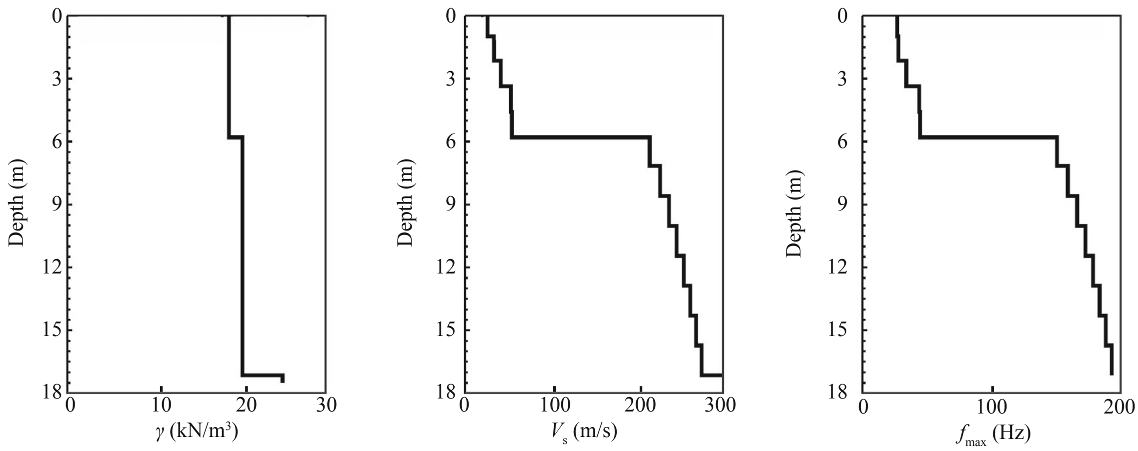

Fig. 3 Unit weight, shear wave velocity and maximum frequency versus depth

and need the digital signal processing. The processing methods eliminated the conventional digital signal and high frequency interference signals are filtered and corrected by the baseline (Boore and Bommer, 2005). For recorded acceleration digital signals, the amplitude of the acceleration spectrum had a significant downward trend before 0.2 Hz as shown in Fig. 7. Therefore, the band-pass filter in the frequency range of 0.2–25Hz was used to filter the digital signal as the input base motion.

Both PressureDependMultiYield02 material for sand and PressureIndependMultiYield material for clay are used in this study. The elastic modulus of soil is $E = 2G_{max}(1+v_s)$ and the bulk modulus is $K=E/3(1-2v_s)$, where v_s is Poisson's ratio of soil, taking 0.25 for sand and 0.45 for soft clay. The frequency dependent damping ratio is 0.2%, which is set to small strain material damping with respect to the viscous damping issue. Further details on these models can be found in Table 1. Contrast studies use the DEEPSOIL v5.1 (Hashash *et al*., 2012) program, which can perform both frequency domain one-dimensional (1-D) equivalent linear solution and time domain one-dimensional (1-D) nonlinear analysis. In Fig. 4, 1-D equivalent analysis uses curves for the modulus reduction G/G_{max} and damping in the sand and the soft clay as the functions of shear strain suggested by Boulanger *et al*. (1999). The curves also provide the fitting parameters for the MRDF pressure-dependent hyperbolic model in the time domain 1-D nonlinear analysis used in DEEPSOIL. Meanwhile, small strain damping (Phillips and Hashash, 2009) in the 1-D nonlinear analysis is frequency independent and calculated by fast fourier transform (FFT).

Table 1 Soil model parameters for 3-D soil column finite element method

Model parameters	Clay	Sand
Material type	PressureIndependMultiYield	PressureDependMultiYield02
Reativedesity, D_r	-	75%–80%
Soil effective unit weight, γ (kN/m^3)	18.75	20.36
Reference pressure, p_r' (kPa)	80	101
Reference shear wave velocity (m/s)	28.8–59.9	214.9–275.4
Shear modulus, G_r (kPa)	1585.1–6866.8	95863.3–157378.7
Bulk modulus, B_r (kPa)	9378.5–40628.3	159772.2–262297.8
Cohesi, c (kPa)	2.8–14.9	0.1
Peak shear strain, $\gamma_{max, r}$	0.1	0.1
Friction angle, ϕ (°)	0	38
Phase transformation angle, ϕ_{PT}(°)	-	26.5
Pressure dependent coefficient, d	0	0.5
Contraction coefficient, c_1	-	0.013
Contraction coefficient, c_3	-	0
Dilation coefficient, d_1	-	0.3
Dilation coefficient, d_3	-	0
Initial void ratio, e	-	0.6
Number of yield surfaces, NYS	18	20

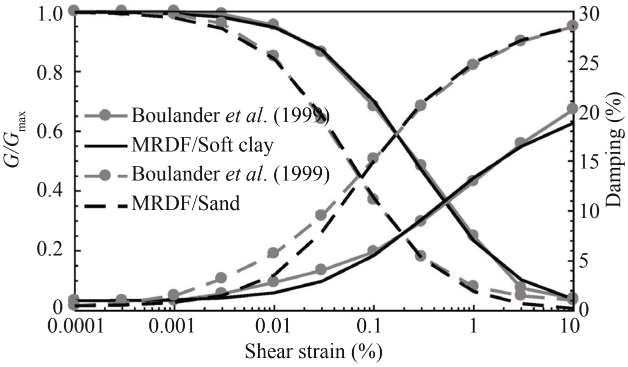

Fig. 4 Normalized modulus (G/G_{max}) and damping with shear strain

3.2 Comparison of results

Figure 5 presents a comparison of the calculated and recorded acceleration response spectrums, and seismic ground response analyses used different procedures including 1-D equivalent linear, 1-D nonlinear analyses and 3-D soil column finite element method. The results from the three procedures can basically reflect the measured response of the soil deposit, and the soil acceleration amplitude gradually enlarges from the bedrock to the surface. The calculated results in the sand layer match with the recorded results in the whole period, while in the clay layer they are consistent, with the top position. Although the recorded and calculated results of clay have few differences in the 1.0s period, when compared with the 1-D nonlinear analyses, the results of the 3-D soil column finite element method are more consistent with the calculated results using the1-D equivalent linear method.

4 Model accuracy

In theory, the 1-D equivalent linear analysis based on the wave propagation theory can be used as a precise solution for assessing the rationality of nonlinear ground responses analyses, (Hashash *et al*., 2010; Kwok *et al*., 2007), but in practice, cannot verify the accuracy of nonlinear ground response under earthquakes. This section combines the existing centrifugal model test (Wilson *et al*., 1997), and considers the self-weight stress, to investigate the accuracy of the analysis results from thr nonlinear ground response under earthquake loads from the smallest shaking level to the strongest shaking level. Both the frequency domain 1-D equivalent linear solution and the time domain 1-D nonlinear analysis are compared to 3-D soil column finite element analysis.

Soil layers and parameters are in accordance with the case verification and are presented in Table 1. During the calculation process, all sizes used the prototype to converse. The base input motion for each event used the 1995 Kobe earthquake as the prototype in the experiments. The calculated earthquake input took the actual acceleration time history data measured from the eastern side of the base. Table 2 shows the peak acceleration information from the experiment input and basic measurements, where peak acceleration changed from 0.016g to 0.577g. Figure 6 shows the uncorrected acceleration amplitude spectrum measured from the eastern side of the base, and the uncorrected acceleration amplitude spectrum has a significant downward trend before 0.2 Hz. Therefore, the frequency from 0.2 Hz to 25 Hz is chosen as the cut-off frequency in the filter. The acceleration data are used as base input to the 3-D soil column finite element after the frequency processing.

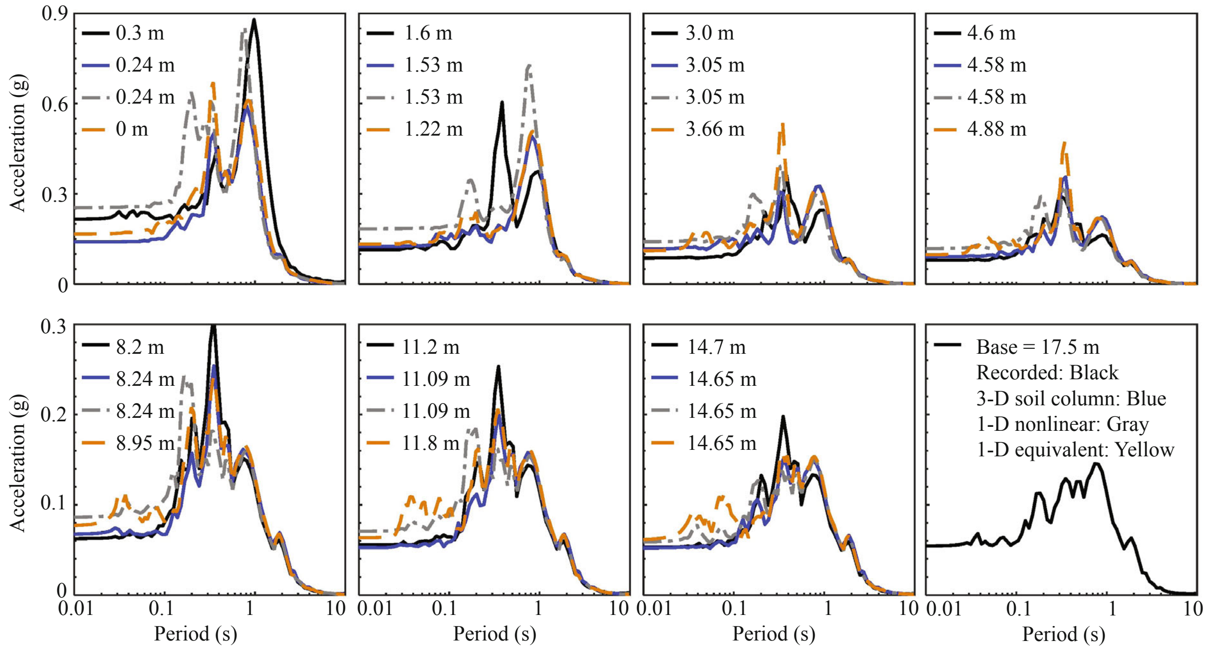

Fig. 5 Comparison of recorded and calculated acceleration response spectra (5% damping) versus soil depth in 0.054 g peak acceleration

Table 2 Earthquake events of shear box base

Event	Earthquake	Experiment input a_{max} (g)	Base measured a_{max} (g)	Band-pass filterrange (Hz)
Csp4B	Kobe	0.055	0.054	0.2–25.0
Csp4C	Kobe	0.016	0.016	0.2–25.0
Csp4D	Kobe	0.200	0.204	0.2–25.0
Csp4E	Kobe	0.580	0.577	0.2–25.0

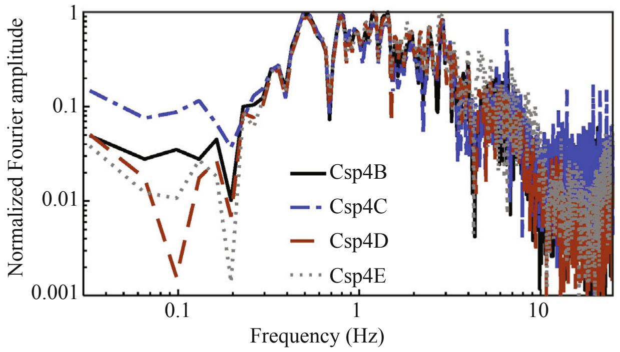

Fig. 6 Comparison of uncorrected Fourier acceleration spectra

4.1 Acceleration response spectrum

The peak acceleration input is 0.016 g, and the acceleration response spectrum comparison of the recorded and the calculated results are shown in Fig. 7. The soil models adopt 1-D equivalent linear, 1-D nonlinear analyses and 3-D soil column finite element methods in this comparison. The acceleration amplitude of the soil layers is gradually enlarged from the base to the soil surface. All three procedures are generally consistent with the recorded results in sand, but the results recorded in clay with a period of 0.1–1.0 s are larger than the results predict by the three ground response analysis procedures.

Figure 5 shows a comparison of the acceleration response spectrum (damping 5%) from the three ground response solutions and the results recorded when the peak acceleration input is 0.054 g. The results from the three methods can generally reflect the actual response shape of the soil and that the acceleration amplitude is gradually enlarged from the base to the surface, but are different from the calculation at the top of the clay layer, as shown in Section 2.

The comparison of the acceleration response spectrum of the recorded and calculated results, including the 1-D equivalent linear, 1-D nonlinear analyses and 3-D soil column finite element methods, is shown in Fig.8 when the peak acceleration input is 0.204 g. Compared with the results from the 1-D equivalent linear and 1-D nonlinear analyses, the results from the 3-D soil column finite element analysis are more consistent with the recorded results, and more accurately reflect the acceleration amplitude changes in the depth direction of the soil layers. Three methods on the surface are consistent with the results recorded in periods greater than 1.0 s, but the frequency component results from the 1-D equivalent linear method is higher

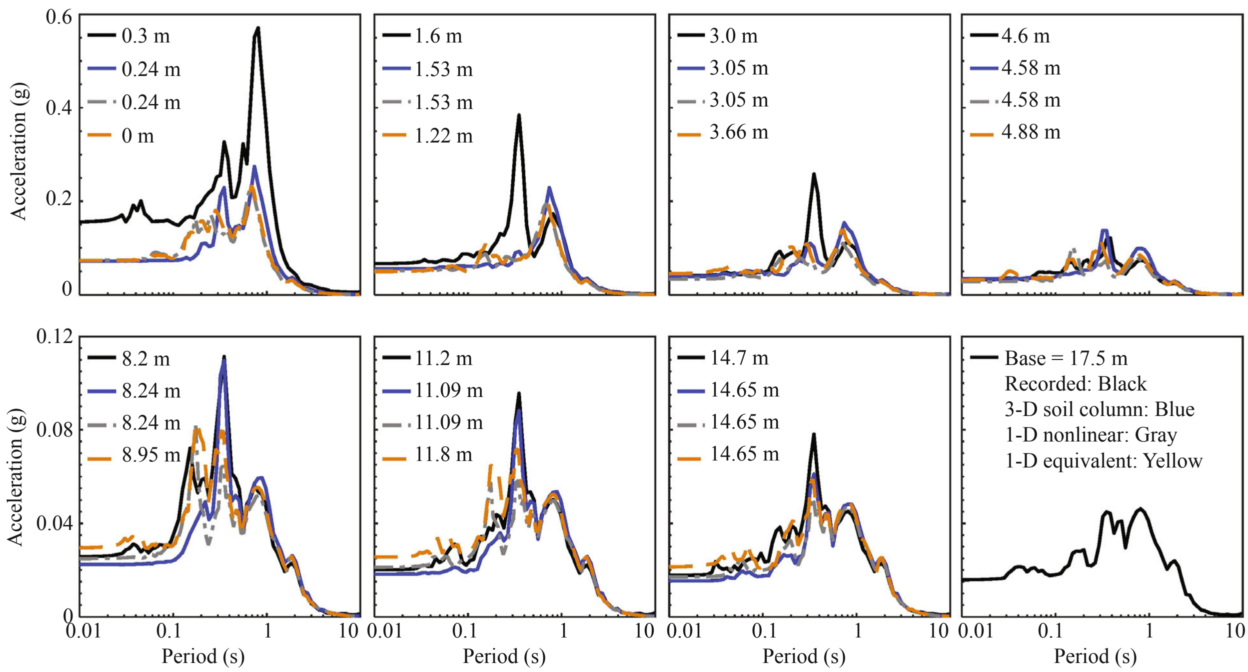

Fig. 7 Comparison of recorded and calculated acceleration response spectra (5% damping) versus soil depth in Csp4C

than that of the others when in the period of less than 0.1s, and the acceleration amplitudes of both the 1-D equivalent linear and 1-D nonlinear analyses are too high at a depth of about 3.0 m.

As shown in Fig. 9, compared with the recorded results, the ground response analysis uses different soil models including 1-D equivalent linear, 1-D nonlinear analyses and 3-D soil column finite element method when the peak acceleration input is 0.577g. The acceleration amplitude is gradually enlarged from the bottom to the sand-clay interface, and also gradually enlarged from the surface to the sand-clay interface. Except when the 1-D nonlinear analysis is in the period of 0.1–0.6 s, the acceleration is very high at depths of 3.05, 8.24 and 11.09 m. In addition, the calculated results from the three methods on the surface are consistent with the results recorded in periods of more than 1.0 s. In periods of less than 1.0 s, in addition to the surface, results from the three methods in the lower clay layer area are generally higher than the recorded results, but the acceleration patterns of the 3-D soil column finite element and experiment results show good agreement.

In the above comparison of the acceleration response spectrum, when the upper part is soft clay or

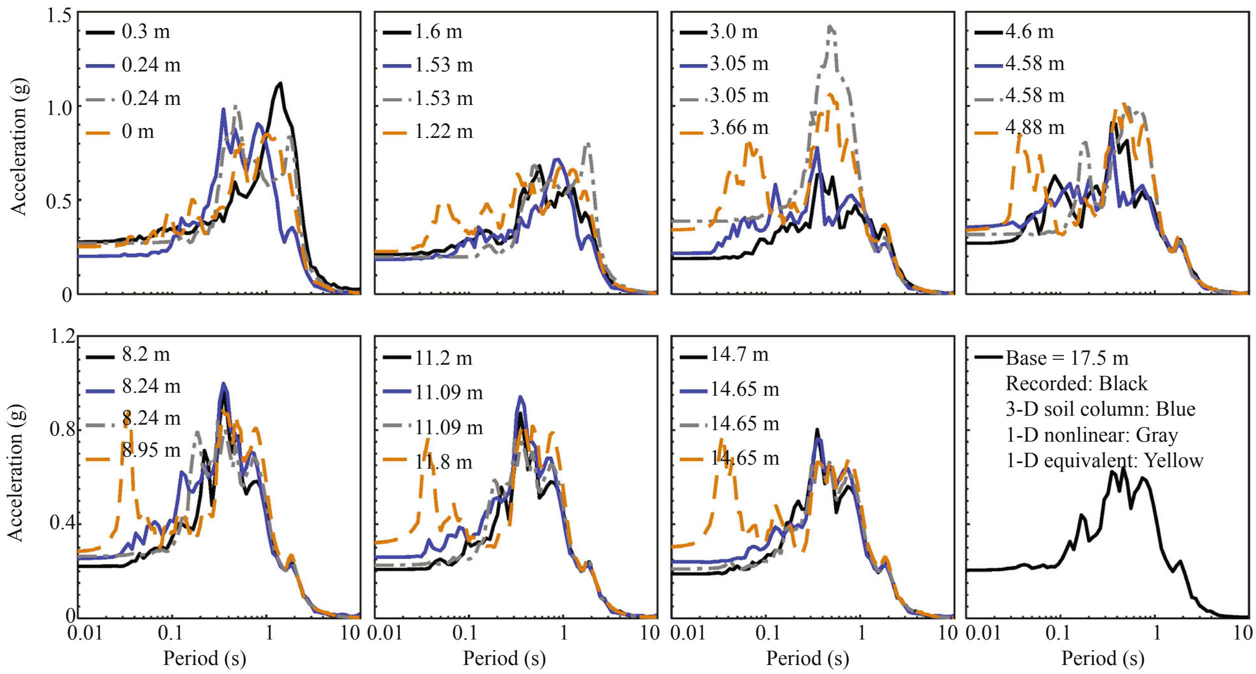

Fig. 8 Comparison of recorded and calculated acceleration response spectra (5% damping) versus soil depth in Csp4D

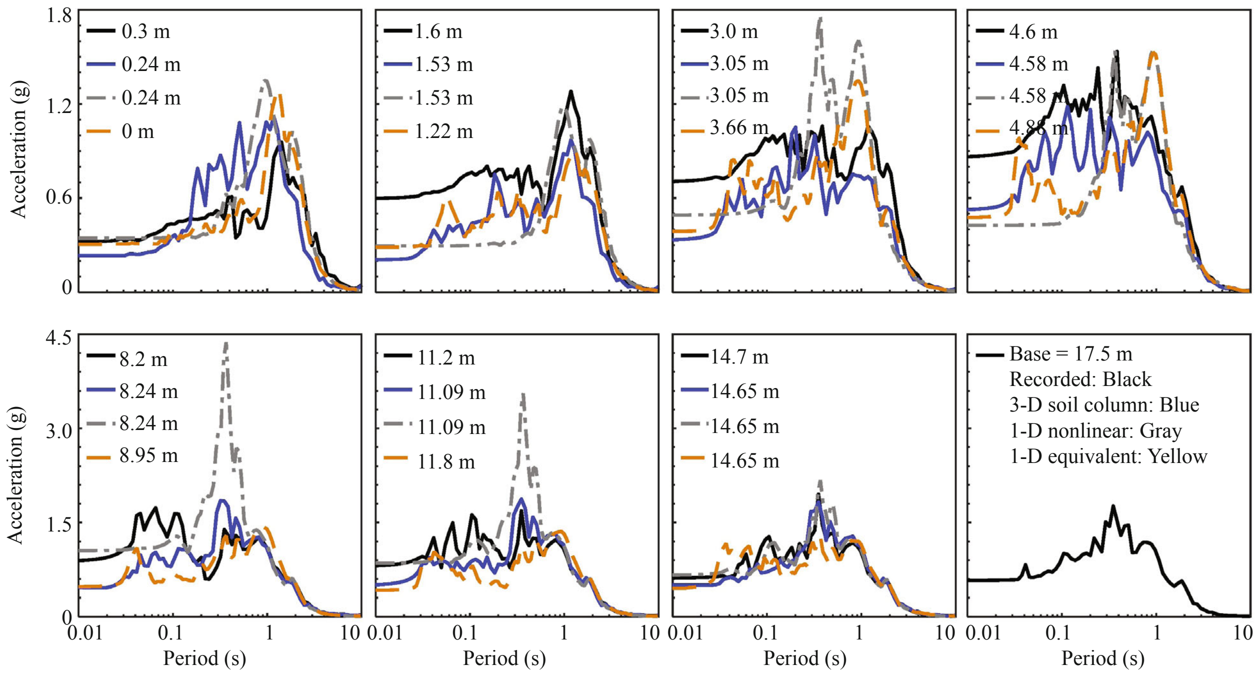

Fig. 9 Comparison of recorded and calculated acceleration response spectra (5% damping) versus soil depth in Csp4E

the peak acceleration input is under the strong level (e.g. 0.577g), the prediction results from the three methods are not entirely consistent with the soil response in the experiment. The results from the 1-D equivalent linear method is consistent with the reviewed conclusion (Hashash *et al.*, 2010) that, at present, the equivalent linear method cannot accurately produce the seismic ground response under the conditions of soft soil or sites subjected to strong motion events. Compared with the analysis results from the 1-D equivalent linear method, the 1-D nonlinear analysis restricted by simplified soil constitutive models may overestimate the acceleration response under strong motions. The 3-D soil column finite element method using advanced soil constitutive models has more consistent test results that capture the acceleration change in the depth direction. In particular at the small or medium shaking level, the prediction of ground response is more consistent. When the peak acceleration input is stronger, the 3-D soil column finite element method can better reflect the response trend of acceleration in the depth direction.

4.2 Displacement

Most of the conventional output in frequency domain programs such as Shake and one-dimensional time domain nonlinear codes such as DEEPSOIL are the acceleration response. The corresponding displacement response needs the integral process on acceleration signal. The 3-D soil column finite element model can not only satisfy the requirements of engineering applications in efficiency and precision, but also needs no integral process in acceleration; displacement can be obtained directly from the free field response.

Figure 10 shows the response of the free field verification from the dynamic centrifuge model test (Wilson *et al.*, 1997) in the peak acceleration from 0.016g to 0.577g. The black solid lines are the displacement curves of the acceleration experimental data through the frequency domain filtering range from 0.2 Hz to 25 Hz and frequency domain integral proceeding. The red dotted lines are the direct calculation results from the 3-D soil column finite element method. The displacement curves in Figs. 10(a), 10(b) and 10(c) are almost identical with the experimental curves, but the significant difference is that the distance between the surface soil displacements is large, which may partly be due to the embedded accelerometer on surface soil being more vulnerable to outside interference. As shown in Fig. 10(d), the peak acceleration input is 0.577g, and there is a significant difference between the calculated and the recorded results in the soft clay layer, but the

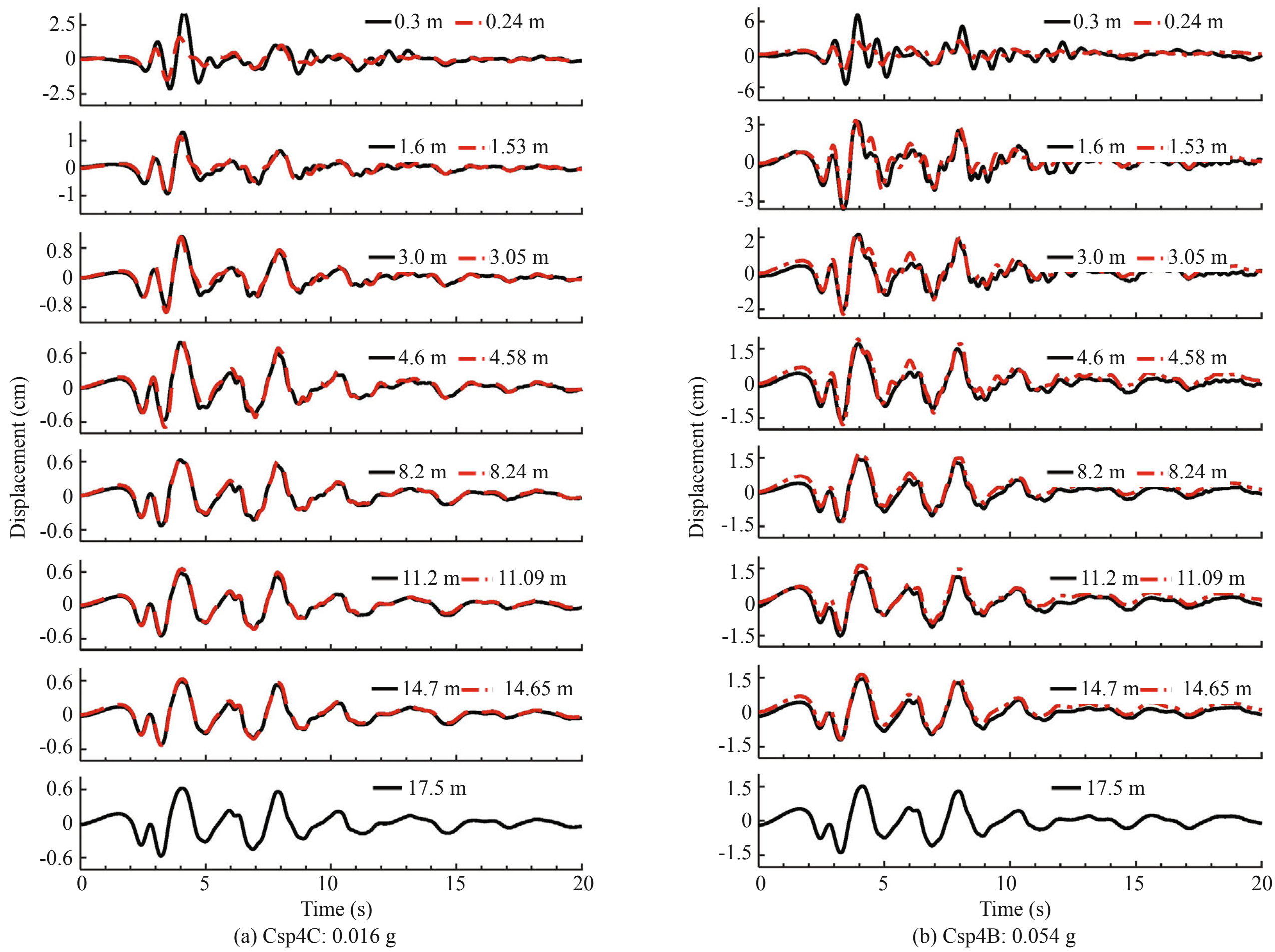

Fig. 10 Comparison of integral recorded and calculated displacement

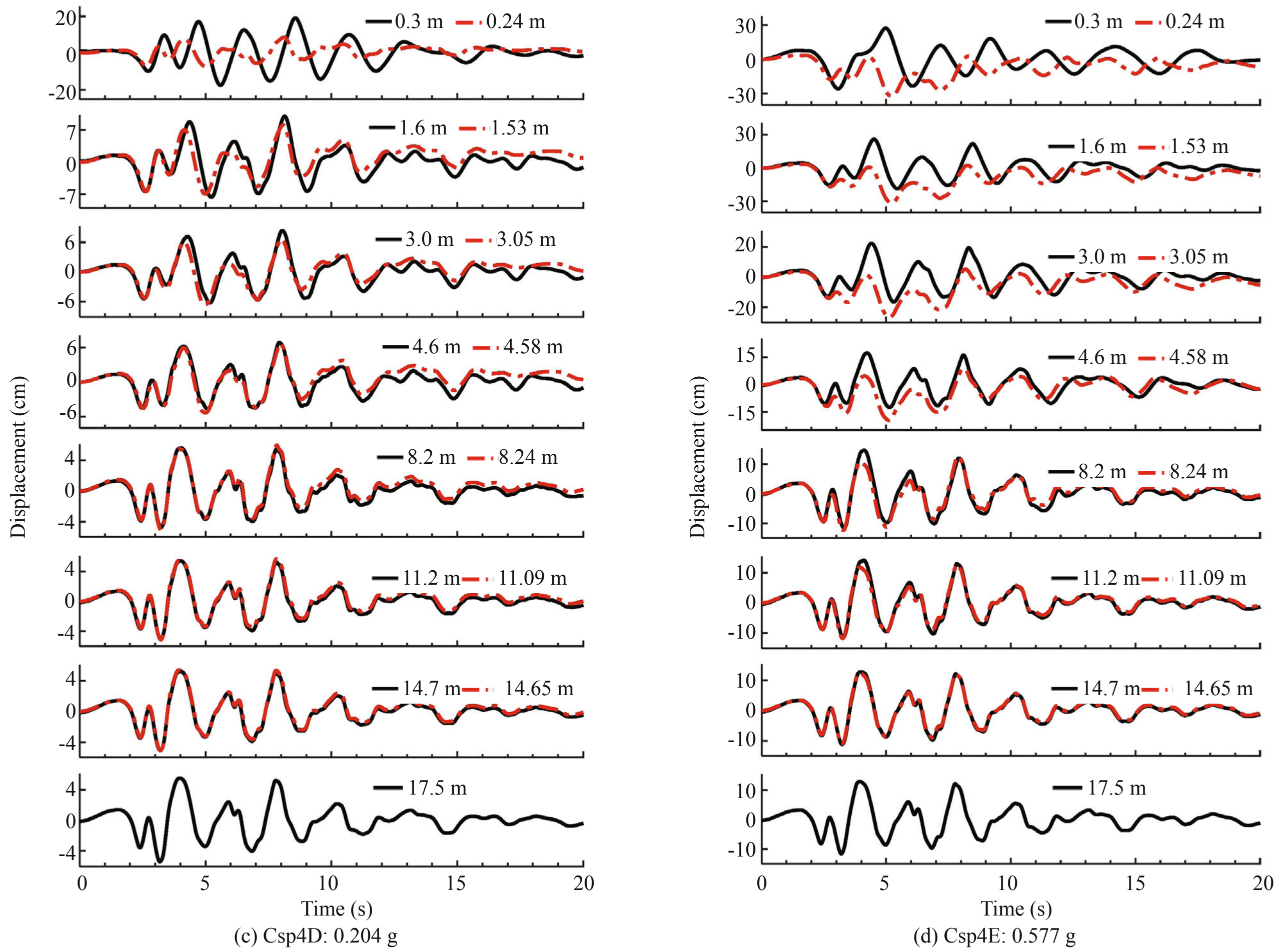

Fig. 10 Continued

displacement response of the sand layer is the same in both procedures.

5 Conclusions

This study considered a three-dimensional soil column distribution mass system model built on the OpenSees simulation platform, to provide appropriate applications of nonlinear ground response analysis in engineering practice. The modeling approach for the soil layers, the selection of soil dynamic constitutive model and the formulation of Rayleigh damping, are introduced. The results are as follows:

(1) Nonlinear seismic ground procedures allow the use of Rayleigh damping. Due to the effect of different properties of layered soils on the absorption of different wave frequency components, it can be divided into several sub-soils depending on the shear velocities, and each subsystem can use the small strain material damping ratio, select the site frequency of the sub-soils and five times the site frequency.

(2) Compared with the analyses, the 1-D equivalent linear and 1-D nonlinear procedures are restricted by simplified soil constitutive models that may overestimate acceleration response under strong motions, while the 3-D soil column finite element method using advanced soil constitutive models has a more consistent response trend of acceleration in accounting for the acceleration change in the depth direction. For small or medium shaking levels, the prediction of ground response is more consistent.

(3) Combined with the recorded data from the centrifuge experiments under different peak accelerations in the basement, this study focused on the accuracy of the soil acceleration and displacement time history output and resolved the above issues to provide an appropriate reference for use in engineering practice. Combined with research by Kwok *et al*. (2007) and Stewart *et al*. (2008), this study may further improve parameter selection and code usage protocols for nonlinear seismic ground motion response.

Acknowledgement

This work was supported by the National Natural Science Foundation of China (Grant No. 41672266). Financial support from these organizations is gratefully acknowledged.

References

Boore DM and Bommer JJ (2005), "Processing of Strong-motion Accelerograms: Needs, Options and Consequences," *Soil Dynamics and Earthquake Engineering*, **25**(2): 93–115.

Boulanger RW, Curras CJ, Kutter BL, Wilson DW and Abghari A (1999), "Seismic Soil-pile-structure Interaction Experiments and Analyses," *Journal of Geotechnical and Geoenvironmental Engineering*, **125**(9): 750–759.

Carvajal JC, Taboada-Urtuzuastegui VM and Romo M P (2002), "Influence of Earthquake Frequency Content on Soil Dynamic Properties at CAO Site," *Soil Dynamics and Earthquake Engineering*, **22**(4): 297–308.

Cubrinovski M, Ishihara K and Poulos H. (2009), "Pseudo-static Analysis of Piles Subjected to Lateral Spreading," *Bulletin of the New Zealand Society for Earthquake Engineering*, **42**(1): 28–38.

Durante MG, Di Sarno L, Mylonakis G, Taylor, CA and Simonelli AL (2016), "Soil-pile-structure Interaction: Experimental Outcomes from Shaking Table Tests," *Earthquake Engineering and Structural Dynamics,* **45**(7): 1041–1061.

Elgamal A, Yang Z and Lu J (2012), *Cyclic1D 1.3, Seismic Ground Response User's Manual. Department of Structural Engineering*, University of California, San Diego.

Elgamal A, Yang ZH, Parra E and Ragheb A (2003), "Modeling of Cyclic Mobility in Saturated Cohesionless Soils," *International Journal of Plasticity*, **19**(6): 883–905.

Elnashai AS and Di Sarno L (2015), *Fundamentals of Earthquake Engineering: from Source to Society*, Wiley, UK.

Fiegel GL (1995), *Centrifugal and Analytical Modeling of Soft Soil Subjected to Strong Seismic Shaking*. Department of Civil and Environmental Engineering. University of California, Davis PhD Dissertation.

Hashash YMA, Groholski DR, Phillips CA, Park D and Musgrove M (2012), *DEEPSOIL 5.1, User Manual and Tutorial*.

Hashash YMA, Phillips C and Groholski DR (2010), "Recent advances in non-linear site response analysis." *Paper presented at the Fifth International Conference in Recent Advances in Geotechnical Eartqhuake Engineering and Soil Dynamics*.

Kwok A, Stewart J, Hashash Y , Matasovic N, Pyke R, Wang Z and Yang Z (2007), "Use of Exact Solutions of Wave Propagation Problems to Guide Implementation of Nonlinear Seismic Ground Response Analysis Procedures," *Journal of Geotechnical and Geoenvironmental Engineering*, **133**(11): 1385–1398.

Li XS, Wang ZL and Shen CK (1992), "SUMDES: A Nonlinear Procedure for Response Analysis of Horizontally-layered Sites Subjected to Multidirectional Earthquake Loading," *Department of Civil Engineering, University of California*, Davis.

Lysmer J and Kuhlemeyer RL (1969), "Finite Dynamic Model for Infinite Media," *Journal of Engineering Mechanics Division*, ASCE, **95**(4): 859–877.

Maheshwari BK, Truman KZ, El Naggar MH and Gould PL (2004), "Three-dimensional Finite Element Nonlinear Dynamic Analysis of Pile Groups for Lateral Transient and Seismic Excitations," *Canadian Geotechnical Journal*, **41**(1): 118–133.

McGann CR, Arduino P and Mackenzie-Helnwein P (2012), "Stabilized Single-point 4-node Quadrilateral Element for Dynamic Analysis of Fluid Saturated Porous Media," *Acta Geotechnica*, **7**(4): 297–311.

Meng JW (2007), "Earthquake Ground Motion Simulation with Frequency-dependent Soil Properties," *Soil Dynamics and Earthquake Engineering*, **27**(3): 234–241.

OpenSees (2016), "Open System for Earthquake Engineering Simulation. Pacific Earthquake Engineering Research Center," University of California, Berkeley, CA. http://opensees.berkeley.edu.

Park D and Hashash YMA (2004), "Soil Damping Formulation in Nonlinear Time Domain Site Response Analysis," *Journal of Earthquake Engineering*, **8**(02): 249–274.

Phillips C and Hashash YMA (2009), "Damping Formulation for Nonlinear 1D Site Response Analyses," *Soil Dynamics and Earthquake Engineering*, **29**(7): 1143–1158.

Prevost JH (1985), "A Simple Plasticity Theory for Frictional Cohesionless Soils," *International Journal of Soil Dynamics and Earthquake Engineering*, **4**(1): 9–17.

Pyke R (2000), *TESS: A Computer Program for Nonlinear Ground Response Analyses: TAGA Engineering Systems and Software*, Lafayette, California.

Schnabel PB, LysmerJ and Seed HB (1972), *SHAKE: a Computer Program for Earthquake Response Analysisi of Horizontally Layered Sites*, *NISEE e-Library:* Earthquake Engineering Research Center, University of California, Berkeley.

Seed HB, WongRT, Idriss I. and Tokimatsu K (1986), "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," *Journal of Geotechnical Engineering*, **112**(11): 1016–1032.

Stewart JP, Kwok A O-L, Hashash YMA, Matasovic N, Pyke R, Wang Z and Yang Z (2008), *Benchmarking of Nonlinear Geotechnical Ground Response Analysis Procedures*, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley.

Wang S, Kutter BL, Chacko MJ, Wilson DW, Boulanger, RW and Abghari A (1998), "Nonlinear Seismic Soil-pile structure Interaction," *Earthquake Spectra*, **14**(2):

377–396.

Wilson DW, Boulanger RW and Kutter BL (1997), *Soil-pile-superstructure Interaction at Soft or Liquefiable Soil Sites - Centrifuge Data Report for Csp4*, Deptartment of Civil & Environmental Engineering, University of California at Davis, California.

Wilson DW, Boulanger RW and Kutter BL (1998), *Signal Processing for and Analyses of Dynamic Soil-pile Interaction Experiments*, Paper Presented at the Centrifuge 98, Kumura, Kusakabe, and Takemura, Eds. , Balkema, Rotterdam.

Yang Z, Lu J and Elgamal A (2008), *User's Manual-OpenSees Soil Models and Solid-fluid Fully Coupled Elements* (*Version 2008 ver1.0*), Department of Structural Engineering, University of California, San Diego.

Yang ZH, Elgamal A and Parra E (2003), "Computational Model for Cyclic Mobility and Associated Shear Deformation," *Journal of Geotechnical and Geoenvironmental Engineering*, **129**(12): 1119–1127.

Zienkiewicz OC and Shiomi T (1984), "Dynamic Behaviour of Saturated Porous Media: the Generalized Biot Formulation and Its Numerical Solution," *International Journal for Numerical and Analytical Methods in Geomechanics*, **8**(1): 71–96.