Parameter Identification and Analysis of Shaking Table Tests on SSI System

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1. Introduction

Studies on Soil-Structure Interaction (SSI) are very important in predicting the exact seismic response of a structure and have become a hotspot in the fields of earthquake engineering research. In recent decades, many methods have been applied in the research of SSI, including theoretical studies, calculation analyses, and experimental studies. Comparisons between the results of calculation analysis and experimental data are important to such research. On one hand, calculation results can verify the feasibility of experiments; on the other hand, experimental results can also provide appropriate and necessary parameters for calculations and can confirm the validity of a calculation method.

Pitilakis D. conducted numerical simulations of dynamic SSI using shaking table testing in the linear viscoelastic domain following a substructure approach (Pitilakis et al., 2008). Eduardo K. provided a concise review of some of the leading developments of SSI (Eduardo et al., 2010). In 2007, several in situ tests and corresponding simulations using the finite element method were conducted for school buildings in Taiwan (Ko et al., 2010).

However, it is well known that the accuracy of any calculation result depends on the calculation parameters, and soil dynamic characteristics are very complicated because soil is a visco-elastic material that exhibits nonlinearity, hysteresis, and strain accumulation under dynamic loads. Many calculation results of SSI have indicated that, if the soil dynamic characteristics are well simulated, various kinds of models can obtain acceptable results, even simplified models; thus, the dynamic characteristics of soil are very important parameters for calculation. The procedures for measuring soil properties that have previously been applied to shake table tests studies (Koga et al., 1990) can be expanded using system identification studies in downhole arrays (Elgamal et al., 1995). To the authors' knowledge, soil parameters have not yet been determined using a shaking table test for a soil-pile-high-rise structure (12 or more floors) interaction system. The main objective of the present study was to apply shaking table tests to identify the dynamic responses of soils supporting the pile groups of tall structures (e.g., 12 floors).

Under the sponsorship of the National Natural Science Foundation of China, shaking table tests on dynamic SSI systems have been conducted at the State Key Laboratory of Disaster Reduction in Civil Engineering of Tongji University, and the tests have recently reached the fourth stage. The first stage of tests included shaking table tests on an interaction system consisting of a uniform soil, a pile foundation (box foundation), and a single column with mass blocks. The second stage of tests involved shaking table tests for an interaction system composed of layered soil,
a pile foundation (box foundation), and a reinforced concrete frame structure of a 12-story interaction system. The third stage of testing also adopted layered soil, while the thicknesses of the clay and the sand soil layers were increased and their water contents were reduced to make the soil harder than in the first two test stages. The upper structure was simulated by a 12-story reinforced concrete frame structure. The fourth stage of tests included shaking table tests of a liquefiable soil-pile-high-rise structure interaction system. In each test, different soils were employed to simulate different sites, and the authors have presented corresponding test results in previous papers (Lu et al., 2002a; Lu et al., 2002b; Lu et al., 2005; Li et al., 2006; Li et al., 2008; Ren et al., 2009). This paper discusses parameter identification and analysis of the third-stage test, PS10H.

2. Description of the Shaking Table Test

The experimental setup, procedure and results of the PS10H shaking table model tests are described in detail in (Li et al., 2006). Hereafter, only the main points relevant to the present analysis are summarized.

2.1 Similitude Model Design

The test model in the present study was scaled down from full-scale buildings and foundations using similitude principles and practical considerations commonly used in the seismic study of SSI systems in shaking table tests (Sabnis et al., 1983; Lu et al., 1999). The test model is shown in Fig.1.

As shown in Fig.1., the prototype superstructure (full scale) is a 12-story cast-in-place concrete frame with a single bay and a single span, which has a 3 by 3 pile group foundation embedded in soil. All physical quantities were scaled using similitude formulas from the Bockinghram π theorem. The model scale was 1/10. The similitude factor for the mass density was 1. The similitude factor of the elastic modulus for the structure was 1/3.870. The similitude factor for time was 0.1967.

The test adopted layered soil consisting of silty clay, clayey silt and sand from top to bottom, to simulate Shanghai soft soil. The depths of the layers were 0.5 m, 0.5 m, and 0.6 m, respectively. The superstructure and foundation were made of micro-concrete and fine zinc-coated steel bars. The properties of all materials were measured by independent laboratory tests prior to the shaking table test. In the tests, the shear wave velocity of each soil was determined. The shear wave velocities of the silty clay, clayey silt and sand were 113, 159 and 182 m/s, respectively.

2.2 Simulation of Soil Boundary Conditions

In shaking table tests, the soil has to be confined within a box of limited size. In the case of rigid containers, the wave reflects from the containers walls and produces boundary effects that induce errors in the test results. In the present shaking table test, a flexible container was used to reduce these undesirable boundary effects (Fig.2.).

The flexible container used herein was cylindrical and 3000 mm in diameter. Its lateral rubber membrane was 5 mm thick. The container was reinforced in the tangential direction with steel loops 4 mm in diameter, spaced by 60 mm. The ratio of the ground plane diameter D to the structural plane diameter d (i.e., D/d) was equal to 5.

2.3 Arrangement of Instrumentation

Fig.1. also shows the arrangement of the instrumentation for the test. The model test was conducted using four types of sensors. Accelerometers and strain gauges were used to measure the dynamic response of the superstructure, foundation, and soil. Pressure gauges were used to measure the contact pressure between the piles and the surrounding soil. As shown in Fig.1., acceleration sensors S1-S4 were located within the pile group, whereas acceleration sensors S5-S10 were located away from the pile group.

2.4 Test Loading Schedule

The El Centro wave, Shanghai artificial wave, Kobe wave and Shanghai bedrock wave were adopted as excitations. The El Centro wave selected for the study is the N-S component from the 1940 El Centro earthquake. The Kobe wave is the N-S component from the 1995 Kobe earthquake. The Shanghai artificial wave and Shanghai bedrock wave are artificial waves for the Shanghai area. The acceleration peak value
was based on the corresponding epicentral intensity according to the seismic code of China, and the peak value and time interval were adjusted according to the similitude relation. For the sake of conciseness, only the x-directional loading schedule is given in Table 1.

Table 1. Summary of x-direction Test Schedule for the PS10H Test

| No. | Excitation | Acceleration Peak Value (g) |
|-----|------------|-----------------------------|
|     |            |Prototype| Model |
| 1   | EL1, SH1, KB1, SJ1 | 0.035| 0.090 |
| 2   | EL2, SH2, KB2, SJ2 | 0.1| 0.258 |
| 3   | EL3, SH3, KB3, SJ3 | 0.15| 0.388 |
| 4   | EL4, SH4, KB4, SJ4 | 0.2| 0.517 |
| 5   | EL5, SH5, KB5, SJ6 | 0.25| 0.646 |
| 6   | EL6, SH6, KB6, SJ6 | 0.3| 0.775 |
| 7   | EL7, SH7, KB7, SJ7 | 0.35| 0.904 |

Note: (1) EL: El Centro wave; (2) SH: Shanghai artificial wave; (3) KB: Kobe wave; (4) SJ: Shanghai bedrock wave

Before and after applying these acceleration levels, white noise with a small amplitude was applied to the model to study the corresponding changes in the dynamic characteristics of the system. The input acceleration for the model test was specified at a time interval equal to 0.003934s, which corresponds to 0.02s according to the prototype scale.

3. Parameter Identification of the Model Soil
3.1 Identification Procedure for Shear Stress and Shear Strain

Procedures for determining soil properties have previously been applied to shaking table tests and downhole array studies (Koga et al., 1990; Elgamal et al., 1995). Similar to these analyses, in this work, the soil deposits are assumed to deform as a shear beam model during seismic shaking (see Fig.3.). This assumption leads to a realistic first-order approximation of the low-frequency shear deformation modes that are dominant in the laminar shear container (Fig.2.). The shear stress $\tau(z,t)$ at any depth $z$ is given by (Elgamal et al., 1995):

$$\tau_i(t) = \tau_{i-1}(t) + \rho \frac{\ddot{u}_{i-1} + \ddot{u}_i}{2} \Delta z_{i-1}, \quad i = 2, 3, \ldots$$  \hspace{1cm} (1)

$$\tau_{i-1/2}(t) = \tau_{i-1}(t) + \rho \frac{3\ddot{u}_{i-1} + \ddot{u}_i}{8} \Delta z_{i-1}, \quad i = 2, 3, \ldots$$  \hspace{1cm} (2)

where $\rho$ is the mass density; $z_i$ is the depth of the $i$th accelerometer; $(z_{i-1} + z_i)/2$ is the depth halfway between accelerometers $i$ and $i-1$; $\tau_i(t) = \tau(z_i,t)$ is the shear stress at depth $z_i$; $\ddot{u}(t) = \ddot{u}(z_i,t)$ is the acceleration at depth $z_i$; and $\Delta z_i$ is the spacing interval between accelerometers. The corresponding second-order accurate shear strains at levels $z_i$ and $(z_{i-1} + z_i)/2$ are [e.g., Elgamal et al., 1995]:

$$\gamma_i(t) = \frac{1}{\Delta z_{i-1} + \Delta z_i} \left( (u_{i+1} - u_i) \frac{\Delta z_{i-1}}{\Delta z_i} + (u_i - u_{i-1}) \frac{\Delta z_i}{\Delta z_{i-1}} \right) \quad i = 2, 3, \ldots$$  \hspace{1cm} (3)

$$\gamma_{i-1/2}(t) = \frac{u_i - u_{i-1}}{\Delta z_{i-1}} \quad i = 2, 3, \ldots$$  \hspace{1cm} (4)

in which $u_i(t) = u(z_i,t)$ is the absolute displacement evaluated through double integration of the recorded acceleration history $\ddot{u}(z_i,t)$. These stress and strain estimates are second-order accurate (Zeghal et al., 2006).

For data processing, a Butterworth low-pass filter (with a cutoff frequency $f_c = 15$ Hz) was employed to eliminate the minor contribution of high-frequency stresses (Fig.4.), and baseline drifts in the displacement estimates (obtained by double integration of the acceleration records) were eliminated by removing linear trends (Fig.5., Li et al., 2009).

3.2 Shear Stress-Strain Response Curves

Fig.6. and Fig.7. show the shear stress-strain response curves calculated at points S1-S9 and SD using Eqs. (1) – (4) under the EL2 and EL4 excitations. As shown in Fig.6. and Fig.7., the results for points S1-S9 and SD are displayed according to the sensor locations shown in Fig.1. Points S1 to S4 are located within the pile group, whereas points S5-S9 and SD are located away from the pile group.
For the smaller EL2 excitation (Fig.6.), these stress-strain curves display elastic unloading as well as cyclic and hysteretic behavior identical to that usually observed in laboratory experiments. The peak shear strain is about 0.096% at point S9 and decreases to about 0.066% at point S5. The corresponding peak shear stress varies from about 1.024 kPa at point S9 to 4.117 kPa at point S5. The stress-strain curves are similar at identical depths inside and outside the pile group.

For the larger EL4 excitation (Fig.7.), the stress-strain cycles become more rounded and exhibit greater hysteretic damping as compared to those in Fig.6. The peak shear strain is about 0.503% at point S9 and decreases to about 0.180% at point S5. The associated peak shear stress varies from about 3.232 kPa at point S9 to 8.086 kPa at Point S5. As shown in Fig.7., the stress-strain curves at identical depths are more similar inside and outside the pile group than those of Fig.6. The soil stiffness, which is related to the slope of these stress-strain curves, clearly increases with depth under both the EL1 and EL3 excitations, because the soil stiffness at the lower layer is greater than that at the upper layers and because the effective pressure increases with the soil depth.

Fig.8. shows the shear stress-strain response curves for point S7 during different loading cycles under the EL4 excitation. The stress-strain loop is almost linear, without any significant stiffness degradation during the smaller excitations. However, during the strong shaking phase, the stress-strain curve becomes more hysteretic and exhibits stiffness degradation.

### 3.3 Shear Modulus and Damping Ratio Curve

The variation in soil stiffness and material damping as a function of the equivalent shear strain amplitude can also be extracted from the measured accelerations. The equivalent stiffness and damping concepts have been well described in the geotechnical engineering literature (Seed et al., 1970). The hysteretic stress-strain loops can be fitted to reproduce the same energy dissipation and shear stress at the peak shear strain as a visco-elastic material. The equivalent shear modulus

\[ G = \frac{\tau(\gamma_m)}{\gamma_m}; \quad \xi = \frac{W_d(\gamma_m)}{2\pi W_e(\gamma_m)} \]  

\( G \) and damping ratio \( \xi \) during a shear stress-strain cycle are evaluated in which \( \gamma_m \) is the maximum shear strain amplitude; \( \tau(\gamma_m) \) is the corresponding shear stress; \( W_d \) is the energy dissipated during a stress-strain cycle; and \( W_e \) is the elastic energy stored by an equivalent stiffness \( G \). As reviewed by Bardet et al. (2001), nonlinear stress-strain curves can be modeled using a series of mechanical elements with different stiffnesses and sliding resistances. In this model, which was initially proposed by Iwan (1967) and Mróz (1967) and is referred to as the IM model, the critical damping ratio \( \xi \), at shear strain \( \gamma \), is expressed as follows:

![Fig.6. Shear Stress-Strain Response Curves Obtained during the EL2 Excitation](image1)

![Fig.7. Shear Stress-Strain Response Curves Obtained during the EL4 Excitation](image2)

![Fig.8. Shear Stress-Strain Response Curves for Point S7 during Different Loading Cycles under the EL4 Excitation](image3)
\[ \xi_i = 0 \quad \text{and} \quad \xi_i = \frac{2}{\pi} \left( \frac{2A_i}{G_{\gamma_i}} - 1 \right) \quad i = 2, \ldots, n \quad (6) \]

where \( A_i \) is the area under the shear stress-strain curve. Eq. 6 implies that \( \xi_i \) can be calculated from the \( G/G_{\max} \sim \gamma \) curve and is independent of \( G_{\max} \). These damping ratio curves (i.e., \( D \sim \gamma \) curves) were calculated from the \( G/G_{\max} \sim \gamma \) curves using NERA (Bardet et al., 2001). Fig.9. compares the moduli and damping ratios evaluated from the shear stress-strain curves along with the damping ratios calculated from the \( G/G_{\max} \sim \gamma \) curves. Fig.9. also shows the shear moduli and equivalent damping ratios independently measured from resonant column tests and triaxial tests conducted in the laboratory. The curves for points S9, S7 and S5 represent the soil characteristics of silty clay, clayey silt and sand, respectively. The variations in shear modulus with shear strain amplitude, as established from the shaking table tests, are in good agreement with the laboratory results. The damping ratios determined from the shear stress-strain curves are noticeably higher than the damping ratios measured in the laboratory tests. The damping ratios calculated from the areas under the shear stress-strain curves are smaller than those for the other damping ratios. In general, the scatters for the calculated shear moduli are smaller than those for the damping ratios (Fig.9.). The reason for this is as follows: The soils used in the shaking table tests are disturbed soils, which have different mechanical properties than the undisturbed soils used in downhole arrays or laboratory tests. Moreover, the damping identified from the shaking table test does not only the material damping of the soil but also the influence of damping energy dissipation induced by the nonlinear development of the soil. For the above reasons, the damping ratios identified from the shaking table test are higher than those from earthquake measurements in downhole arrays or laboratory test results.

4. Modeling Method of the Test

4.1 Simulation of the Flexible Container

The behavior of the flexible container in the shaking table test should be included in the modeling of the SSI system. In order to accomplish this, the lateral rubber membrane of the container was meshed with shell elements in the computer model.

The base plate of the container was rigidly bolted to the shaking table. Crushed rock was attached to the base plate by epoxy resin to achieve a rough interface between the soil and the base during the test. This ensured a negligible relative slip between the soil and the bottom surface of the container and justified the fixed-base assumption in the computer model.

Reinforcement loops outside the container were used to provide radial rigidity to the system and to allow the soil to deform as a series of horizontal shear layers during the test. In the computer model, the reinforcement loops were modeled as follows. The nodes along the container perimeter of the same height have the same displacement in the excitation direction (x direction of the shaking table), and this property can be realized by coupling degrees of freedom (DOFs) in the ANSYS program; thus, DOFs were coupled together and X DOFs were constrained to take on the same value.

4.2 Dynamic Constitutive Model of the Soil and Simulation of the Material Nonlinearity

The soil was meshed using three-dimensional solid elements, which have six degrees of freedom at each
node. An equivalent linear model of soil was adopted by taking the effective shear strain $\gamma_i$ to be equal to 65% of the maximum shear strain.

The influence of the effective confined pressure of the soil on the initial shear modulus was taken into consideration when the initial shear modulus of each layer of soil was chosen. Eq. 7 shows that the initial shear modulus is proportional to the square root of the ratio of the effective confined pressure, namely:

$$\frac{G_i}{G_{i+1}} = \sqrt{\frac{\sigma_{i+1}^\prime}{\sigma_i^\prime}}$$  

where $G_i$ is the initial shear modulus of the soil at the $i$ th layer; $G_{i+1}$ is the initial shear modulus of the soil at the $(i+1)$ th layer; $\sigma_i^\prime$ is the effective confined pressure of the soil at the $i$ th layer; and $\sigma_{i+1}^\prime$ is the effective confined pressure of the soil at the $(i+1)$ th layer. These four variables are given in Pascals. The equation is valid only if layers $i$ and $(i+1)$ are of the same type of soil.

The parameter identification results — the $G_i/G_0-\gamma_d$ curves for silty clay, clayey silt and sand in the range of $10^4$ to $10^5$ (Fig.9.) — were adopted in modeling, and the initial value of $G_i/G_0$ was taken as 0.82. The ANSYS equivalent linear model was automatically implemented with the APDL option. APDL stands for the ANSYS Parametric Design Language, a scripting language that can be used to automate common tasks or even build a model in terms of parameters. The APDL also encompasses a wide range of other features such as command repetition, macros, if-then-else branching, do-loops, and scalar, vector and matrix operations (ANSYS Inc. 2004).

4.3 Simulation of the Nonlinearity of the Soil-Structure Interface

Due to the different characteristics of the soil and concrete of the foundation, sliding and separation may occur at the soil-structure interface when the interface stress reaches a certain limit. Furthermore, the gapped interface between the soil and the structure (foundation) may separate and close under certain loads. In ANSYS, the contact between elements is realized by overlaying a thin layer of elements on the contact interface of the model. The soil surface is taken as the contact surface and that of the structure (foundation) as the target, since its rigidity is greater than that of the soil. Contact and target elements were formed on the contact and target surfaces, respectively, and they were modeled as contact pairs using the same real constant. In ANSYS, the target and contact elements that make up a contact pair are associated with each other via a shared real constant set (ANSYS Inc. 2004). In this simulation, the normal contact stiffness factor was taken as 1.0, and the maximum contact friction coefficient was taken as 0.2.

4.4 Damping Model

In SSI systems such as the one considered here, the soil damping ratio is usually larger than that of the concrete superstructure. In order to account for this difference, a material damping input method available in ANSYS was used. As is well known, the Rayleigh damping ratio can be calculated by Eq 8:

$$\xi_i = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2}$$

where $\xi_i$ is the ratio of actual damping to critical damping for a particular mode of vibration $i$, and $\omega_i$ is the natural circular frequency of mode $i$.

In many practical structural problems, alpha damping (or mass damping) can be ignored ($\alpha = 0$). In such cases, $\beta$ can be evaluated from known values of $\xi_i$ and $\omega_i$ as:

$$\beta = 2\xi_i/\omega_i$$ (9)

Material-dependent damping allows one to specify beta damping ($\beta$) as a material property. Different damping ratios can be input for different materials by this method (ANSYS Inc. 2004). The initial damping ratio of soil was taken as 0.05, which was then iteratively determined from the $D-\gamma_d$ curves measured by material tests. The damping ratio of the superstructure was taken as 0.05.

4.5 Consideration of Gravity

The initial stress produced by gravity has a significant influence on the contact state of the soil-structure interface. A significant error in the analysis will arise if gravity is not taken into account in the dynamic calculation. It is obvious that the contact pressure is larger when gravity is taken into account than when gravity is not included in the analysis.

In this study, gravity was taken as a dynamic load in the calculation. Before the seismic wave was input, the gravity load was applied to the system as a vertical acceleration field. Once the load was applied, and the transient response of the system disappeared and the response reached a constant value (a static-like condition), the seismic wave was added to the gravity load to continue computing the time history response. The dynamic response due to the earthquake loading alone was then obtained by subtracting the constant value from the total response.

4.6 Meshing

In the ANSYS program, the group pile foundation and soil were meshed using three-dimensional solid elements. The beams and columns of the superstructure were meshed with three-dimensional beam elements, and the slabs were meshed with shell elements. Both the beam and shell elements have six degrees of freedom at each node: translations in the x, y, and z directions and rotations about x, y, and z-axes. The modeling was based on the following principles.

(1) Wave motion constraint on meshing. It is difficult for the high-frequency component of the wave motion to be transmitted if the element is too large. A study reported by Gupta et al. (1982) showed that, in the case of a shear wave transmitted vertically, the height of the
element $h_{\text{max}}$ can be taken as $(\frac{1}{2} - \frac{1}{8})v_s/f_{\text{max}}$, where $v_s$ denotes the velocity of the shear wave and $f_{\text{max}}$ denotes the highest wave frequency intercepted. The limitation of mesh size in the plane is not as strict as that in the height direction, and the size in the plane was chosen as 3 to 5 times $h_{\text{max}}$. In this case, the maximum mesh size in the height was taken as 0.1 m, and the maximum mesh size in the plane was taken as 0.3 m.

2) Constraint upon meshing to allow for comparison between the calculation and the test results. In order to make the comparison easy, the nodes of the calculation model were chosen to correspond to the measuring points of the test.

3) Effect of meshing on precision. With finer meshing and more degrees of freedom, the precision is higher, but the time required for calculation is longer. Thus, a proper grid size should be used. Fig.10 shows the meshing of the PS10H test model, which satisfies the above modeling requirements.

5. Calculation Results for the PS10H Model

Based on the modeling method presented above, the El Centro wave was taken as the x-direction excitation of the PS10H model to study the SSI system; the results obtained are as follows.

Selected acceleration time histories of the computational and experimental results of the PS10H model are given in Fig.11. EL4 denotes excitation of the El Centro wave, with a peak acceleration of 0.517 g. Fig.11. shows that the acceleration time history curves match reasonably well. The maximum acceleration at A7 is 0.593 g according to the tests and 0.565 g according to the analysis. The maximum acceleration at S9 is 0.281 g according to the tests and 0.299 g according to the analysis. From this comparison, it can be stated that the computational model is rational and appropriate for further studies of SSI effects.

In order to analyze the effect of the nonlinearity of soil and that of the soil-structure interface on the results, the computational analysis was conducted (1) without nonlinearity; (2) only accounting for soil nonlinearity; and (3) accounting for both soil nonlinearity and the nonlinearity of the soil-structure interface.

A comparison of the acceleration response under conditions (1) and (2) is shown in Fig.12. Point A7 is the central point on the top of the frame (as shown in Fig.1.), and point S10 is on the surface of the soil, 0.6 m away from the boundary of the container. It can be observed that the acceleration response decreases significantly when the soil nonlinearity is considered.

A comparison of the acceleration responses obtained under conditions (2) and (3) is shown in Fig.13. From the figure, it can be seen that the responses (at point A7) are obviously different in these two cases. This indicates that the nonlinearity of the group pile and the soil has a significant effect on the structural response of the SSI system. However, the acceleration response at point S10, which is far from the foundation, shows little change. This means that the nonlinearity at the soil-structure interface has little effect on the response of the soil far from the foundation.

This result demonstrates that a significant error will occur when the nonlinearity of the soil and that of the soil-structure interface are not considered in the simulation of soft soil.
Fig. 12. Comparison of Computational Analysis for Linearity and Soil Nonlinearity (under EL4 Excitation)

Fig. 13. Comparison of Computational Analysis for Soil Nonlinearity and Contact Analysis (under EL4 Excitation)

6. Conclusions

Using measured accelerations from the PS10H shaking table test, the time histories of the shear stress and strain of the modeled soil were obtained, and variations in the soil shear moduli and damping characteristics with shear strain amplitude were also identified. By partly referencing the identification results and adopting a rational modeling method, the PS10H test was analyzed using the general finite program ANSYS. From these studies, the following conclusions were obtained.

(1) The soil stiffness increases with depth, and the stress-strain loops are rounded and exhibit more hysteretic damping for larger earthquake excitations. The soil stiffness as calculated from measured accelerations is comparable to that obtained through independent laboratory tests, but the material damping as calculated from measured accelerations is much larger than that measured in laboratory tests, which itself is smaller than the equivalent damping associated with hysteretic damping.

(2) By comparing the computational and experimental results, it was verified that the modeling method employed is rational. The developed model is suitable for the numerical analysis of an SSI system under weak and strong ground motions.

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References

1) ANSYS Inc., (2004), User's Manual for ANSYS 9.0.
2) Bardet, J. P., and Tobita, T. (2001) NERA--A computer program for nonlinear earthquake site response analyses of layered soil deposits. http://gees.usc.edu/GEES/.
3) Eduardo, K. (2010) Early history of soil-structure interaction. Soil Dynamics and Earthquake Engineering, 30(9), pp.822-832.
4) Elgamal, A. W., Zeghal, M., Tang, H. T., and Steep, J. C. (1995) Evaluation of low-strain site characteristics using the Lotung seismic array. Journal of Geotechnical Engineering Division, ASCE, 121(4), pp.350-362.
5) Gupta, S., Penzien, J., Lin, T. W. and Yeh, C. S. (1982) Three dimensional hybrid modelling of soil-structure interaction, Earthquake Engineering Structure Dynamic, 10, pp.69-87.
6) Iwan, W. D. (1967) On a class of models for the yielding behavior of continuous and composite systems. Journal of Applied Mechanics, ASME, 34, pp.612-617.
7) Ko, Y. Y. and Chen, C. H. (2010) Soil-structure interaction effects observed in the in situ forced vibration and pushover tests of school buildings in Taiwan and their modeling considering the foundation flexibility. Earthquake Engineering and Structural Dynamics, 39(9), pp.945-966.
8) Koga, Y., and Matsuo, O. (1990) Shaking table tests of embankments resting on liquefiable sandy ground. Soils Found 30, pp.162-174.
9) Li, H. N., Yi, T. H. Gu, M. and Huo, L. S. (2009) Evaluation of earthquake-induced structural damages by wavelet transform. Progress in Natural Science, 19(4), pp.461-470.
10) Li, P. Z., Chen, Y. Q., Lu, X. L., et al. (2006) Shaking table testing of hard layered soil-pile-structure interaction system. Journal of Tongji University, 34(3), 307-313 (in Chinese).
11) Li, P. Z., Ren, H. M., Lu, X. L., et al. (2008) Shaking table tests of dynamic interaction of soil-structure considering soil liquefaction. 14WCEE, Beijing, China, paper No. 04-01-0024.
12) Lu, X. L., Zhang, H. Y., Hu, Z. L. and Lu W. S. (1999) Shaking table testing of a U-shaped plan building model. Canada Journal of Civil Engineering, 26(6), pp.746-759.
13) Lu, X. L., Chen, Y. Q., Chen, B. and Li, P. Z. (2002a) Shaking table model test on dynamic soil-structure interaction system. Journal of Asian Architecture and Building Engineering, 1(1), pp.55-64.
14) Lu, X. L., Li, P. Z., Chen, B. and Chen, Y. Q. (2002b) Numerical analysis of dynamic soil-box foundation-structure interaction system. Journal of Asian Architecture and Building Engineering, 1(2), pp.9-14.
15) Lu, X. L., Li, P. Z., Chen, B. and Chen, Y. Q. (2005) Computer simulation of the dynamic layered soil–pile–structure interaction system. Canadian Geotechnical Journal, 42(3), pp.742-751.
16) Mróz, Z. (1967) On the description of anisotropic work hardening. Journal of Mechanics and Physics of Solids, 15, pp.163-175.
17) Pitolakis, D., Dietz, M., Wood, D. M., Clouteau, D. and Modaresi, A. (2008) Numerical simulation of dynamic soil-structure interaction in shaking table testing. Soil Dynamic and Earthquake Engineering, 28(6), pp.453-467.
18) Ren, H. M., Lu, X. L. and Li, P. Z. (2009) Numerical simulation of dynamic PSSI system considering liquefaction. Journal of Asian Architecture and Building Engineering, 8(1), pp.191-196.
19) Sabnis, G. M., Harris, H. G., White, R. N., and Mirza, M. S. (1983) Structural modeling and experimental techniques. Prentice-Hall, Inc., Englewood Cliffs, N.J.
20) Seed, H. B., and Idiss, I. M. (1970) Soil moduli and damping factors for dynamic response analyses, Rep. EERC 70-10, Earthquake Research Center, Univ. of California, Berkeley, California.
21) Zeghal, M., Kallou, P. V., Oskay, C., et al. (2006) Identification and imaging of soil and soil-pile deformation in the presence of liquefaction. Earthquake Engineering and Engineering Vibration, 5(5), pp.171-182.