Responses of Liquefiable Soils in Pile Group Foundations of Tall Buildings from Shaking Table Tests

Peizhen Li¹, Jinping Yang², Zheng Lu*¹ and Xilin Lu³

¹Associate Professor, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China
²Doctoral Student, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China
³Professor, State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China

Abstract

The responses of liquefiable soils around pile group foundations of tall buildings to earthquakes were measured using acceleration arrays in shaking table tests. System identification techniques were used to analyze the measured accelerations and pore pressures and to calculate the shear stress and shear strain in the soil inside and outside the pile group of a 12-story building as well as the soil shear moduli and damping. The analysis shows that the calculated soil stiffness is similar to the stiffness determined using laboratory tests and that the soil damping is higher. It also shows that pore pressure buildup reduces the shear wave velocity and the soil stiffness.

Keywords: liquefaction; sandy soil; soil-structure interaction; shaking table test; system identification

1. Introduction

In spite of past research efforts, soil liquefaction remains a major cause of damage to buildings during earthquakes (e.g., Green et al. 2010, National Research Council 1985). The effects of soil liquefaction on structures have been investigated using various methods, including analyses of case histories (e.g., Idriss et al. 2008, Holzer et al. 1989, Chang et al. 1991), computer simulations, centrifuge tests (e.g., Elgamal et al. 2005, Wei et al. 2010), and shaking table tests (e.g., Kagawa et al. 2004, Koga et al. 1990, Funahara et al. 2000, Lin et al. 2006, Suzuki et al. 2008, Pitilakis et al. 2008, Haeri et al. 2012).

Soil liquefaction has been documented in soil deposits by measuring the time histories of the acceleration and the pore pressure in vertical downhole seismic arrays and analyzing using system identification techniques that identified the stress-strain responses of liquefiable soils (e.g., Abdel-Ghaffar et al. 1979, Zeghal et al. 1994, Zeghal et al. 2009, Oskay et al. 2011, Lozano-Galant et al. 2013). However, a recent review of geotechnical system identification methods (Oskay et al. 2011) indicates that system identification has not yet been applied to the responses of liquefiable soils around the pile foundations of tall buildings, which is an important problem in earthquake engineering practice that remains poorly documented by experimental data.

In addition to geotechnical centrifuges, shaking table tests are useful tools for examining the effects of liquefiable soils on embankments and pile foundations (Kagawa et al. 2004, Koga et al. 1990, Funahara et al. 2000, Lin et al. 2006, Suzuki et al. 2008, Pitilakis et al. 2008, Lee et al. 2012, Chen et al. 2013, Varghese et al. 2014). With advances in model similitude theory (e.g., Lu et al. 1999) and structural seismic testing technology, shaking table tests have become powerful tools for investigating soil-structure interactions. They are often used in China in the seismic design of tall buildings (12 or more floors; e.g., Lu et al. 1999). Until now, no shaking table tests had been attempted for tall buildings built on liquefiable soil due to concerns about permanent structural tilts developing after earthquakes.

This study summarizes and analyzes the results of shaking table tests used to identify the dynamic responses of liquefiable soils under tall buildings. The shaking table tests were conducted at the State Key Laboratory of Disaster Reduction in Civil Engineering, Tongji University, China (Li et al. 2008). The recorded data were analyzed using system identification techniques to examine the stress-strain responses and damping characteristics of liquefiable soils under tall buildings.

2. Introduction of the Shaking Table Tests

The experimental setup, the procedure and the results of the shaking table model tests are described in detail in (Li et al. 2008). Hereafter, only the main points are summarized.
2.1 Similitude Design of the Models

The test models were scaled down from full-scale buildings and foundations based on similitude principles and the practical considerations of seismic studies for the shaking table tests (Li et al. 2008). (1) Similar similitude relations were applied to the soil, the foundation and the superstructure. (2) The effects of the earth's gravity were not scaled; extra weight was not added in the present study. (3) The dynamic loads were limited to ensure that they remained within the maximum load capacity of the shaking table. (4) The model's geometry was designed to keep it within the maximum size allowed by the shaking table.

As shown in Figs. 1. and 2., the prototype superstructure (full-scale) is a 12-story cast-in-place concrete frame with a single bay and a single span that has a 3-by-3 pile group foundation that is embedded in deep sand deposits. All of the physical quantities were scaled using similitude formulas derived using the Buckingham π theorem (Li et al. 2008). The model's scale was 1/10. The similitude factor of the mass density was 1, the similitude factor of the structure's elastic modulus was 1/2.67, the similitude factor of the soil's elastic modulus was approximately 1/3.2 and the similitude factor of time was 0.163. Hereafter, all of the experimental results are reported in terms of the model's scale rather than the prototype's scale. Figs.1. and 2. show the experimental setup for the shaking table test and the pile foundation used in the test, respectively.

2.3 Design and Fabrication of the Models

The soil model was made of saturated sand overlain by a layer of silty clay. The top layer of silty clay was 0.2 m thick, and the underlying sand layer was 1.3 m deep. The foundation of the superstructure was made of a pile group containing nine piles, each of which was 1.2 m long. The pile foundation and the surrounding soil were both designed using the same scale, i.e., 1/10. The superstructure and foundation were made of micro-concrete and fine zinc-coated steel bars. The properties of all of the materials were measured in independent laboratory tests before the shaking table tests.

2.4 Arrangement of the Instrumentation

As shown in Fig. 4., the model was instrumented with five types of sensors. Accelerometers (with labels starting with the letter S when located in the soil), displacement meters, and strain gauges were used to measure the dynamic responses of the superstructure, foundation, and soil. Pore pressure gauges (with labels starting with the letter H) were embedded into the soil to measure changes in the pore pressure. Pressure gauges (with labels starting with the letter P) were used to measure the contact pressure between the piles and the surrounding soil. As shown in Fig. 4., acceleration sensors S1-S4 were located within the pile group, and acceleration sensors S5-S10 were located away from the pile group.

2.5 Test Loading Schedules

The ground was shaken in the x-direction of the shaking table. The record of the 1940 El Centro earthquake was selected for the input shaking; it is labeled EL hereafter. Fig. 5. shows the acceleration time-history and the corresponding Fourier spectrum.
Table 1. displays the complete loading sequences, which consisted of 5 levels of excitation that were labeled from EL1 to EL5. The peak values of the accelerations were 0.131 g, 0.375 g, 0.75 g, 1.125 g and 1.5 g during the loading steps from EL1 to EL5, respectively. Before and after these accelerations were applied, white noise (WN) with a small amplitude was applied to the model to determine the change in the dynamic characteristics of the system. The input acceleration for the model test was specified at a time interval equal to 0.00327 s, which corresponded to 0.02 s on the prototype's scale. Hereafter, only the results of tests EL1 and EL3 are presented and analyzed for the sake of conciseness.

3. The Recorded Acceleration and Pore Pressure Ratio

Fig.6. depicts the time history of the recorded acceleration at point S3 and the associated variations in the pore pressure ratio at point H2 for excitation EL3 (Table 1.). Fig.7. shows the time history of the pore pressure ratio measured at point H2 for excitation EL1. The pore pressure ratio is defined as the ratio between the water pressure generated during shaking and the initial effective vertical stress. In Fig.6., the time scale is reported in terms of the model scale rather than the prototype scale. The earthquake duration (53.7 s) in Fig.6. is scaled down to 8.8 s (by a factor of 0.1633) in Fig.5. The acceleration displays spikes associated with simultaneous drops in the pore pressure that have also been observed by other researchers (e.g., Holzer et al. 1989). These acceleration spikes and pore pressure drops are identified by the numbers (1) to (15) in Fig.6.

Table 1. Summary of the Test Program for the Present Study

| No. | Excitation Name | Peak Acceleration (g) | Prototype | Model |
|-----|----------------|-----------------------|-----------|-------|
| 1   | WN            | -                     | 0.07      |       |
| 2   | EL1          | 0.035                 | 0.131     |       |
| 3   | WN            | -                     | 0.07      |       |
| 4   | EL2          | 0.1                   | 0.375     |       |
| 5   | WN            | -                     | 0.07      |       |
| 6   | EL3          | 0.2                   | 0.75      |       |
| 7   | WN            | -                     | 0.07      |       |
| 8   | EL4          | 0.3                   | 1.125     |       |
| 9   | WN            | -                     | 0.07      |       |
| 10  | EL5          | 0.4                   | 1.5       |       |
| 11  | WN            | -                     | 0.07      |       |

Note: (1) White noise (unidirectional X)
(2) El Centro earthquake (unidirectional X)

As shown in Figs.5. and 6., the pore water pressure in the sand layer builds up gradually as the acceleration cycles for small (EL1) and large (EL3) earthquakes. The magnitude of the increase in the pore pressure depends on the location at which it is measured and the magnitude of the input acceleration. The pore pressure...
ratio at point H2 for excitation EL1 does not reach 10%, but it reaches 100% at point H2 for excitation EL3, which indicates that the sand liquefies.

4. Identification Procedure for Shear Stress and Shear Strain

The methods of data processing and analysis used in shaking table studies (Koga et al. 1990) can be improved using system identification techniques with downhole arrays (Oskay et al. 2011). In these latter analyses, the soil deposits are assumed to deform in accordance with a shear beam model during seismic shaking (see Fig.8.). This assumption leads to realistic first-order approximations of the low-frequency modes of shear deformation that dominate in the laminar shear container (Fig.3.). The shear stress, \( \tau(z,t) \), at depth \( z \) is

\[
\tau(z,t) = \int_0^\rho \rho u(z,t) \, dz.
\]

where \( \rho \) is the mass density and \( u(z,t) \) is the acceleration at depth \( z \). Using linear interpolation to find the acceleration at various depths, the shear stresses at depths \( z_i \) and \( (z_{i-1} + z_i)/2 \) are (Elgamal et al. 1994)

\[
\tau_i(t) = \tau_{i-1}(t) + \frac{\rho}{2} \left[ \frac{u_{i-1} + u_i}{2} \Delta z_{i-1}, \quad i = 2, 3, \ldots \right. (2)
\]

\[
\tau_{i-1/2}(t) = \tau_{i-1}(t) + \frac{3\rho}{8} \left[ 3u_{i-1} - u_i \Delta z_{i-1}, \quad i = 2, 3, \ldots \right. (3)
\]

where \( z_i \) is the depth of the \( i \)-th accelerometer, \( (z_{i-1} + z_i)/2 \) is the depth that is halfway between accelerometers \( i \) and \( i-1 \), \( \tau_i(t) = \tau(z_i,t) \) is the shear stress at depth \( z_i \), \( u_i(t) = u(z_i,t) \) is the acceleration at depth \( z_i \), and \( \Delta z \) is the interval between two accelerometers. The corresponding second-order shear strains at levels \( z_i \) and \( (z_{i-1} + z_i)/2 \) are (Elgamal et al. 1995)

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

\[
\gamma_{i-1/2}(t) = \frac{u_{i-1} - u_i}{\Delta z_{i-1}}, \quad i = 2, 3, \ldots
\]

where \( u_i(t) = u(z_i,t) \) is the absolute displacement evaluated by integrating the recorded acceleration history, \( u(z_i,t) \), twice. These estimates of the stress and strain estimates are accurate to second order (Zeghal et al. 1994). To process the measured acceleration data, a filter was employed to eliminate high-frequency noise and baseline drifts from the estimates of the displacement. This filtering removes the minor contributions of high-frequency stresses and low-frequency strain drifts.

5. Identification Results

5.1 Time Histories of the Shear Stress and the Shear Strain

Figs.9. and 10. show the shear stress-strain response curves at points S1-S9 that were calculated using Eqs. (2) – (5) for excitations EL1 and EL3. In Figs.9. and 10, the graphs at points S1 through S9 are displayed in a way that corresponds to the sensor locations shown in Fig.4. Points S1 through S4 are located within the pile group, and points S5 through S9 are located away from the pile group. Table 2. lists the sensors used to calculate the shear stress and the shear strain at different points.

| Point | Sensors used |
|-------|--------------|
| S1    | S1 & S2      |
| S2    | S2 & S3      |
| S3    | S3 & S4      |
| S4    | S4 & S11     |
| S5    | S5 & S6      |
| S6    | S6 & S7      |
| S7    | S7 & S8      |
| S8    | S8 & S9      |
| S9    | S9 & S10     |

For the smaller excitation, EL1 (Fig.9.), these stress-strain curves display elastic unloading as well as cyclic and hysteretic behavior identical to those usually observed in laboratory experiments (e.g., S2-S9). The peak shear strain occurs at approximately 0.075% at point S4 and decreases to approximately 0.03% at point S1. The corresponding peak shear stress varies from approximately 1.8 kPa at point S4 to approximately 3 kPa at point S1. The stress-strain curves are similar at identical depths inside and outside the pile group. The only noticeable difference is that the stress loops reverse sharply within the pile group (S1-S4) and are more rounded outside the pile group (S5-S8).

For the larger excitation, EL3 (Fig.10.), the stress-strain cycles become more rounded and exhibit more hysteretic damping than the ones shown in Fig.9. The peak shear strain is approximately 0.45% at point S4 and decreases to approximately 0.2% at point S1. The associated peak shear stress varies from approximately 3 kPa at point S4 to 7.5 kPa at point S1. In Fig.10., the stress-strain loops are rounded both inside and outside the pile group, while the stress-strain loops in Fig.9. exhibit sharp reversals inside the pile group that are absent outside the pile group. The soil stiffness, which is related to the slopes of the stress-strain curves, clearly increases with depth under both excitations, EL1 and EL3.
Figs. 11 and 12 show a few isolated cycles of shear stress-strain responses. During the initial phase between 0 and 0.3 s, the stress-strain loop at point S3 is almost linear and lacks any significant stiffness degradation, which corresponds to a negligible increase in the pore pressure at point H2 (Fig. 6). During the strong shaking phase that occurs after 0.3 s, the stress-strain curve becomes more hysteretic and exhibits a stiffness degradation that is associated with a sharp increase in the pore water pressure at point H2 (Fig. 6). A clear stiffness reduction takes place at 4.55 s, which corresponds almost perfectly to a pore pressure ratio that is close to 1 (i.e., to liquefaction).

5.2 The Shear Modulus and Damping Ratio Curves

In addition to the stress-strain response curves (e.g., Figs. 11-12), information can be extracted from the measured acceleration, e.g., variations in the soil stiffness and material damping as functions of the amplitude of the equivalent shear strain. Equivalent stiffness and damping are described well in the geotechnical engineering literature (e.g., 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, \(G\), and damping ratio, \(\xi\), during a shear stress-strain cycle are

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

where \(\gamma_m\) is the maximum amplitude of the shear strain, \(\tau(\gamma_m)\) is the corresponding shear stress, \(W_d\) is the amount of energy dissipated during a stress-strain cycle, and \(W_e\) is the amount of elastic energy stored by an equivalent stiffness \(G\). As described 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, $\xi_i$, for shear strain $\gamma_i$ is as follows:

$$
\xi_i = 0 \quad \text{and} \quad \xi_i = 2 \pi \left( \frac{2A_i}{G_{\gamma_i}} - 1 \right) \quad i = 2, \ldots, n, \quad (7)
$$

where $A_i$ is the area under the shear stress-strain curve.

Fig.11. The Shear Stress-Strain Response Curves at Point S3 During Selected Loading Cycles for Excitation EL3 (Time: 0.20-2.93 s)

Eq. 7 implies that $\xi$ can be calculated from the $G/G_{\text{max}} \sim \gamma$ curve and is independent of $G_{\text{max}}$. The damping ratio curves (i.e., the $D \sim \gamma$ curve) were calculated on the basis of the $G/G_{\text{max}} \sim \gamma$ curves using NERA (Bardet et al. 2001).

Figs.13. and 14. compare the moduli and damping ratios obtained from the shear stress-strain curves (data points) and the damping ratios calculated from the $G/G_{\text{max}} \sim \gamma$ curves (dashed lines). Figs.13. and 14. also show the shear moduli and equivalent damping ratios that were independently measured using resonant column tests and triaxial tests in the laboratory (solid lines). The variations in the shear modulus with the amplitude of the shear strain, which was established in the shaking table tests, are in good agreement with the laboratory results. The damping ratios determined from the shear stress-strain curves and the $G/G_{\text{max}} \sim \gamma$ curves are noticeably higher than the damping ratios measured in laboratory tests. The damping ratios calculated using the areas under the shear stress-strain curves are greater than the other damping ratios. The damping ratios for excitation EL3 are significantly larger than those observed for excitation EL1 because the soil liquefies under excitation EL3. In general, the scatter of the calculated shear moduli is less than that of the damping ratios (Figs.13. - 14.).

5.3 Average Shear Wave Velocity

Cross-correlation techniques, which are based on the recognition of specific waveforms within small time windows, were used to evaluate the average shear wave velocity between the acceleration sensors. This analysis is based on the notion that the cross-correlation function between two acceleration histories, $a_i(t)$ and $a_j(t)$, is maximized when the time delay is $t = t_{ij}$, where $t_{ij}$ is the time required for a seismic wave to travel from point $i$ to point $j$. An estimate of the apparent velocity, $v_{ij}$, of wave propagation between points $i$ and $j$ is

$$
v_{ij} = \frac{d}{t_{ij}}, \quad (8)
$$

where $d$ is the distance between points $i$ and $j$. Fig.15. shows the time variation of the average shear wave velocity, which is evaluated based on cross-correlation analyses of the time histories of the acceleration at points S4 and S1 for excitation EL3 (Chang et al. 1991). The decrease in the shear wave velocity from approximately 50 m/s to 30 m/s corresponds to a decrease in the soil stiffness, which is caused by an increase in the pore pressure during shaking. As shown in Fig.15.b, the effect of the pore pressure on the average shear velocity becomes significant when the pore pressure ratio is greater than 0.8. When the pore pressure ratio is smaller, the variations in the average shear wave velocity have higher amplitudes. A significant reduction in the shear wave velocity occurs...
during the strong excitation between 0.5 s and 2 s, which coincides with the onset of the increase in the pore pressure (Fig.6).

5.4 Effective Stress Path

Fig.16. shows the evolution of the vertical component of the effective stress and the shear stress at points S7 and S5, which were determined using measured accelerations and pore pressures. The vertical effective stress, \( \sigma_v' \), is calculated by subtracting the excess pore pressure, \( p \), from the initial vertical total stress, \( \sigma_v \), i.e., \( \sigma_v' = \sigma_v - p \). The pore pressure, \( p \), is measured at points H2 and H1, and the initial vertical stress is calculated at points S7 and S5 using the unit weight of the soil. The curves relating the shear stress and the effective vertical stress are interpreted as effective stress paths. As shown in Fig.16., the soil reaches zero effective stress at points S7 and S5 for excitation EL3, which corresponds to liquefaction.

6. Conclusions

System identification has been used to analyze the experimental results of shaking table tests and to determine the response of the soil inside and outside the pile group foundation of a twelve-story building. The technique produced time histories of the shear stress and strain within the soil as well as of the variations in the soil’s shear moduli and in the damping characteristics with changes in the amplitude of the shear strain. The analysis has shown that: (1) The pore pressure and the pore pressure ratio are related to the location and magnitude of the input acceleration. (2) The soil stiffness increases with depth, and the stress-strain loops are rounded and exhibit larger hysteretic damping for greater earthquake excitations. (3) The soil stiffness and shear strength both decrease as the pore pressure builds up. (4) The soil stiffness calculated using the measured acceleration is comparable to that obtained through independent laboratory tests, but the material damping calculated using the measured accelerations is much larger than that measured in laboratory tests, which is smaller than the equivalent damping associated with hysteretic damping.

This study has demonstrated that system identification is useful for retrieving valuable information on dynamic soil behaviors, even when complex soil-pile-structure interactions occur in liquefiable ground, from the results of shaking table tests. The system identification of measured accelerations provides researchers with useful insight into the relationship between the responses of structures and those of soils, including the effects of increased pore pressure and decreased soil stiffness.
and shear strength on structural responses. From the perspective of engineering practice, this technique enhances the methods of processing and analyzing the data from shaking table tests, which are widely used in the seismic design of tall buildings in China.

Fig.15. The Variations in the Average Shear Wave Velocity During Excitation EL3 as Functions of (a) Time and (b) the Pore Pressure Ratio

Fig.16. The Effective Vertical Stress–Shear Stress Curves for Excitation EL3 at Points S7 and S5

Acknowledgments

This project was supported as a project of the National Natural Science Foundation of China (Grant No. 51178349, 51478355). The study was also supported by the Ministry of Science and Technology of China (Grant No. SLDRCE14-B-22).

References

1) Abdel-Ghaffar, A. M. and Scott, R. F. (1979), "Shear moduli and damping factors of earth dam", J. Geotech. Eng., Div. ASCE, 105(12), pp.1405-1426.
2) Bardet, J. P. and Tobita, T. (2001), NERA - A computer program for nonlinear earthquake site response analyses of layered soil deposits, University of Southern California, http://gees.usc.edu/GEES/.
3) Chang, C. Y., Mok, C. M., Power, M. S. and Tang, Y. K. (1991), Analysis of ground response data at Lotung large-scale soil-structure interaction experiment site, Rep. NP-7306-SL, Electric Power Research Institute, Palo Alto, CA.
4) Chen, G. X., Wang, Z. H., Zuo, X., Du, X. L., Gao, H. M. (2013), "Shaking table test on the seismic failure characteristics of a subway station structure on liquefiable ground", Earthquake Eng. & Struct. Dyn., 42(10), pp.489-1507.
5) Elgamal, A. W., Yang, Z. H., Lai, T., Kutter, B. L., and Wilson D. W. (2005), "Dynamic response of saturated dense sand in laminated centrifuge container", J. Geotech. Geoenviron. Eng., 131(5), pp.598-609.
6) Funahara, H., Fuji, S., and Tamura S. (2000), "Numerical simulation of pile failure in liquefied soil observed in large-scale shaking table test", 12th World Conference on Earthquake Engineering, Auckland, New Zealand, No. 0927.
7) Green, R. A., and Cubrinovski, M. (2010), Geotechnical reconnaissance of the 2010 Darfield (New Zealand) Earthquake, GEER association, www.geerassociation.org, p.180.
8) Haeri, S. M., Kavanda, A., Rahmani, I. and Torabi, H. (2012), "Response of a group of piles to liquefaction-induced lateral spreading by large scale shake table testing", Soil Dyn. Earthquake Eng., 38, pp.25-45.
9) Holzer, T. L., Youd, T. L., and Hanks, T. C. (1989), "Dynamics of liquefaction during the 1987 Superstition Hills, California earthquake", Science, 244, pp.56-59.
10) Idriss, I. M. and Boulanger, R. W. (2008), Soil liquefaction during earthquakes, Earthquake Engineering Research Institute, Oakland, Calif., p.244.
11) Iwan, W. D. (1967), "On a class of models for the yielding behavior of continuous and composite systems", J. Appl. Mech., 34, pp.612-617.
12) Kagawa, T., Sato, M., Minowa, C., Abe, A., and Tazoh, T. (2004), "Centrifuge Simulations of Large-Scale Shaking Table Tests: Case Studies", J. Geotech. Geoenviron. Eng., 130(7), pp.663-672.
13) Koga, Y. and Matsuoka, O. (1990), "Shaking table tests of embankments resting on liquefiable sandy ground", Soils Found., 30, pp.162-174.
14) Lee, C. J., Wang, C. R., Wei, Y. C., Hung, W. Y. (2012), "Evolution of the shear wave velocity during shaking modeled in centrifuge shaking table tests", Bull Earthquake Eng., 10(2), pp.401-420.
15) Lozano-Galant, J. A., Mogal, M., Castillo, E., Turmo, J. (2013), "Application of Observability Techniques to Structural System Identification", Computer-Aided Civil and Infrastructure Eng., 28(6), pp.434-450.
16) Li, P. Z., Ren, H. M., Lu, X. L. and Cheng, L. (2008), "Shaking table tests of dynamic interaction of soil-structure considering soil liquefaction", 14th World Conference on Earthquake Engineering, Beijing, China.
17) Lim, M. L., and Wanga, K. L. (2006), "Seismic slope behavior in a large-scale shaking table model test", Eng. Geol, 86(2-3), pp.118-133.
18) Lu, X. L., Zhang, H. Y., Hu, Z. L. and Lu W. S. (1999), "Shaking table testing of a U-shaped plan building model", Can. J. Civ. Eng., 26(6), pp.746-759.
19) Lu, X. L., Li, P. Z., Chen, B. and Chen, Y. Q. (2005), "Computer simulation of the dynamic layered soil-particle-structure interaction system", Can. Geotech. J., 42(3), pp.742-751.
20) Mróz, Z. (1967), "On the description of anisotropic work hardening", J. Mech. Phys. Solids, 15, pp.163-175.
21) National Research Council. (1985), Liquefaction of Soils During Earthquakes, National Academy Press, Washington, D.C.
22) Oskay, C. and Zeghal, M. (2011), "A survey of geotechnical system identification techniques", Soil Dyn. Earthquake Eng., 31 (4), pp.568-582.
23) Pitilakis, D., Dietz, M., Wood, D. M., Clouteau, D., Modaressi, A. (2008), "Numerical simulation of dynamic soil-structure interaction in shaking table testing", Soil Dyn. Earthquake Eng., 28(6), pp.453-467.
24) Seed, H. B. and Idriss, I. M. (1970), Soil moduli and damping factors for dynamic response analyses. Rep. EERC 70-10, Earthquake Research Center, University of California, Berkeley, California.
25) Suzuki, H., Tokimitsu, K., Sato, M., and Tabata, K. (2008), "Soil-pile-structure interaction in liquefiable ground through multi-dimensional shaking table tests using E-defense facility", 14th World Conference on Earthquake Engineering, Beijing, China.
26) Varghese, R. M., Latha, G. M. (2014), "Shaking table tests to investigate the influence of various factors on the liquefaction resistance of sands", Natural Hazards, 73(3), pp.1337-1351.
27) Wei, Y. C., Lee, C. J., Hung, W. Y., and Chen H.-T. (2010), "Application of Hilbert-Huang transform to characterize soil liquefaction and quay wall seismic responses modeled in centrifuge shaking-table tests", Soil Dyn. Earthquake Eng., 30(7), pp.614-629.
28) Zeghal, M. and Elgamal, A. W. (1994), "Analysis of site liquefaction using earthquake records", J. Geotech. Eng. Div. ASCE, 120(6), pp.996-1017.
29) Zeghal, M. and Abdel-Ghaffar, A. M. (2009), "Evaluation of the nonlinear seismic response of an earthdam: nonparametric system identification", J. Earthquake Eng., 13(3), pp.384-405.

318 JAABE vol.15 no.2 May 2016 Peizhen Li