6th International Conference on Earthquake Geotechnical Engineering
1-4 November 2015
Christchurch, New Zealand

Numerical Modeling of Pile Group Response subjected to Liquefaction-Induced Large Ground Deformations in E-Defense Shake Table Test

G. Li[1], R. Motamed[2]

ABSTRACT

This paper presents two-dimensional (2D) nonlinear dynamic finite element (FE) modeling of a large-scale shake table test conducted at the E-Defense shake table facility in Japan. The main objective of this numerical simulation is to evaluate the application and accuracy of 2D effective stress analyses in predicting soil-pile response subjected to liquefaction and lateral spreading using available constitutive models. The coupled soil-water FE model was developed in OpenSees and the analysis results are compared with measured data from the shake table experiment with the main emphasis on the response of liquefied soil and the demand applied to the piles. In addition, a sensitivity analysis is carried out to explore the effects of soil constitutive model parameters on the liquefied soil response.

Introduction

In recent earthquakes (e.g. 2010 Haiti, 2010 Chile, 2010-2011 Canterbury Sequence and 2011 Tohoku earthquakes), extensive damage on pile foundations has been observed due to liquefaction-induced lateral spreading. In this regard, many researchers have investigated the basic mechanisms of this phenomenon through physical modeling including shake table tests (e.g., Tokimatsu et al., 2007; Motamed et al., 2009) and centrifuge tests (e.g., Boulanger et al., 1999; Abdoun and Dobry, 2002). Meanwhile, a number of numerical simulations have been carried out based on physical experiments for validation and application using different modeling techniques (Chang et al., 2013).

In March 2006, a large-scale test on lateral spreading of liquefied sand behind a sheet-pile quay wall was successfully performed at the E-Defense facility in Japan. This was analyzed by Motamed et al. (2009), which deeply studied the soil-pile interaction in liquefied ground. The experiment included a simple structure model supported on a 2×3 pile group located adjacent to a sheet-pile quay wall. The model was heavily instrumented to measure the dynamic response of the soil-pile system. Liquefaction-induced lateral spreading was achieved and the soil moved laterally about 1.1m behind the quay wall. Based on the shake table test, a 2D numerical model is developed in this study using OpenSees (http://opensees.berkeley.edu) with existing constitutive models developed for other types of sands to assess the effectiveness of the FE analysis to predict the response of the soil-pile system when subjected to lateral spreading. The analytical results show similar deformation modes of the soil-pile-superstructure system and the results are presented hereafter.

[1]PhD student, Department of Civil and Environmental Engineering, University of Nevada Reno, Reno, USA, gli@unr.edu
[2]Assistant Professor, Department of Civil and Environmental Engineering, University of Nevada Reno, Reno, USA, motamed@unr.edu

Description of Shake Table Experiment

The shake table model of the soil-pile system was constructed in a large rigid box with the dimensions of 16m×5m×4m, and the soil configuration was a horizontal ground consisted of uniform liquefiable Albany Silica sand with the relative density of 60%. A LSP-2 type steel sheet pile quay wall was used, which deformed laterally and triggered the liquefaction-induced lateral spreading. Behind the quay wall, six hollow steel piles with outer diameters of 152.4mm and thicknesses of 2mm were pin connected to the base (i.e. zero displacement and moment) and used to support the 22 ton weight of the superstructure.

The large-scale model was subjected to two-directional ground motions (i.e. horizontal and vertical components). The records obtained at the JR Takatori station during the 1995 Kobe earthquake were scaled down by 20% and chosen as the input motion. The maximum amplitudes of the horizontal and vertical components were 0.6g and 0.23g, respectively. More details on the shake table experiment can be found in Motamed et al. (2009).

OpenSees Numerical Model

The 2D numerical model was built using OpenSees framework and post-processing was performed with GiD (http://www.gidhome.com). The discretization of the model is illustrated in Figure 1.

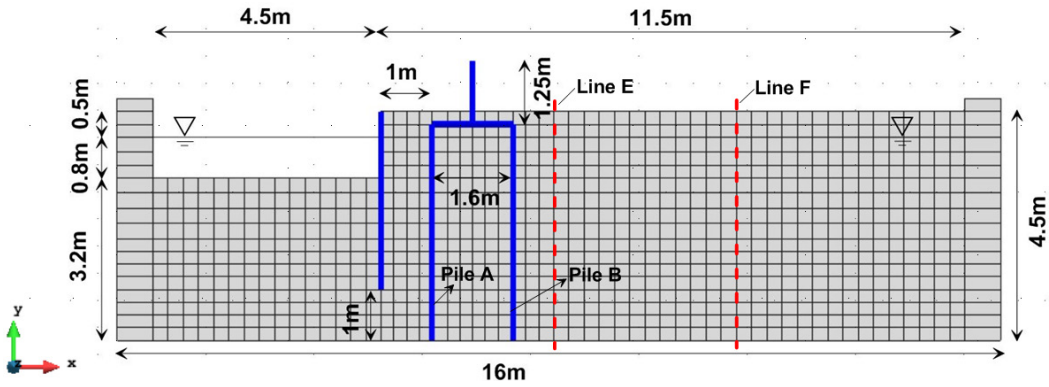

Figure 1. FEM model discretization of shake table experiment

Elements, Materials and Boundary Conditions

In the FEM model, the soil was simulated using QuadUP elements. The out-of-plane thickness of the soil elements was set to be the same as the width of the container in the shake table test: 4m. The constitutive behavior of the soil was captured by the PressureDependMultiYield02 (PDMY02) material available in OpenSees to simulate the response characteristics of sand. The input parameters for PDMY02 material were selected for Albany Silica Sand based on a combination of available E-Defense test information and the suggested values of the constitutive model developer. To investigate the effects of input variables on seismic response of the soil-pile system, three scenarios were studied. The first scenario was to utilize solely recommended values from the OpenSees manual for all parameters of soil with D_r=60%. The second scenario was to incorporate some of the measured parameters from the shake table experiment (maximum shear modulus, maximum bulk modulus, unit weight, void ratio and permeability) but while assuming a uniform shear wave velocity profile (Figure 2). This scenario has been used previously for blind prediction

efforts. The third scenario was similar to the second one, and the only difference was that the precisely measured shear wave velocity profile (Figure 2) was used to obtain the soil parameters.

Beam-column elements were utilized for the simulation of the sheet pile, piles, pile cap and superstructure. The properties of these components are listed in Table 1 below. The pile cap was modeled using rigid beam column elements with high flexural stiffness and masses lumped at the nodes. The superstructure was simplified as a concentrated mass of 12 tons at the center of gravity to take the place of the actual mass. The structure columns were simulated as a uniform flexural stiffness in order to produce theoretical fixed ends natural periods. The 2×3 group of piles was represented by two piles, each having three times the axial, bending, and cap connection stiffness of a single pile.

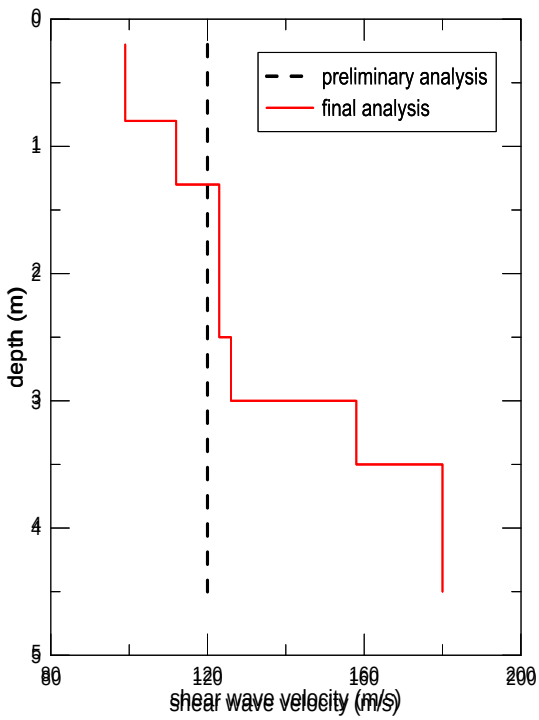

Figure 2. Shear wave velocity profile

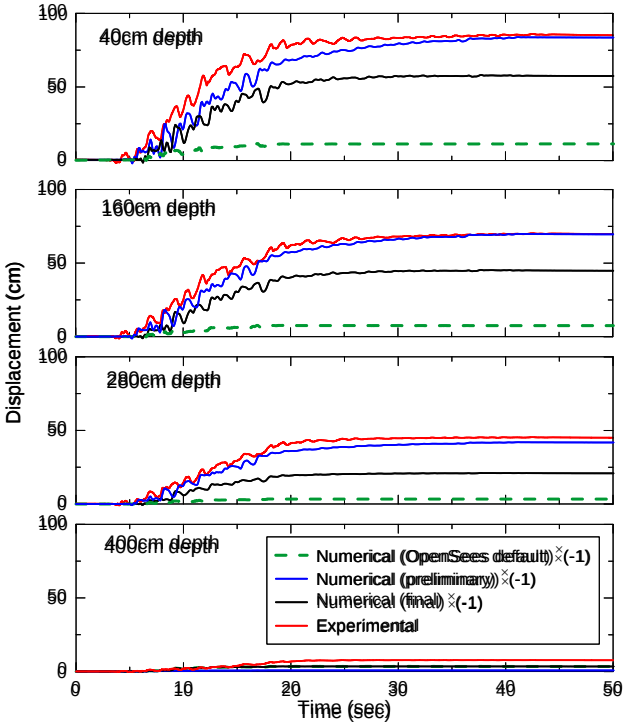

Figure 3. Lateral displacement time histories of soil at various depths behind the quay wall

Table 1. Properties of foundation and structural components.

Component	E (kPa)	I (m^4)	A (m^2)	Mass (ton)
Sheet pile	2.06E+08	4.28E-06	3.02E-02	/
Piles	2.06E+08	8.02E-06	2.84E-03	/
Pile cap	1.00E+09	1.00E+01	8.00E-01	1.00E+01
Superstructure	2.06E+08	6.53E-04	1.91E-02	1.20E+01

In the numerical model, the pile and soil elements were connected using zero-length elements with nonlinear p-y (lateral resistance) and t-z (shaft friction) springs represented by PYliq1 and TZliq1 materials in OpenSees. The pile base was pinned to the container in the same way as in the physical model. Consequently, q-z springs were not implemented in the numerical

model. Along all soil-sheet pile interfaces, a kinematic condition was specified that requires the sheet pile and adjacent soil to share identical displacements in both the horizontal and vertical directions. The container employed in the experiment had semi-rigid boundaries and was approximated in OpenSees using 2D linear elastic isotropic materials in quad elements. It also did not allow drainage from the soil similar to Chang et al. (2013). The bottom of the model was fixed in such a way that no movement or drainage was allowed in both the vertical and horizontal directions. The displacements of the model on both lateral sides were fixed with periodic boundary conditions, meaning each side had the same displacement and hence they moved together laterally.

Analysis Results and Discussions

The response of the soil and piles are presented in this section and compared with the recorded data of the shake table experiment. It is worth mentioning that the bottom connection of the front row piles were unexpectedly broken during the shaking in the experiment and the footing with the superstructure tilted 20 degrees toward the land. As a result, some sensors were hit and even disconnected by the tilting footing around 10.2 sec, which resulted in a small number of irregular recordings. However, in general, the computed responses are in reasonably good agreement with their recorded counterparts.

Soil Deformation

The computed lateral soil displacements at various depths behind the sheet pile for the three aforementioned scenarios are compared with experimental counterparts as presented in Figure 3. The predicted response is highly sensitive to the dynamic characteristics of the soil constitutive model used. Specifically, the calculated horizontal soil displacements in the case with the OpenSees default parameters were much lower than the recorded data. Contrarily, the simulation results of the preliminary numerical analysis utilizing uniform shear wave velocity profile were, surprisingly, highly comparable to the physical test results. The final numerical model with parameters derived from the actual shear wave velocity profile yielded results which fell in between the predictions of first two cases. Using measured values for some of the constitutive model parameters resulted in a 34% underestimation of the soil lateral response, though the overall behavior was reproduced fairly well. For brevity, only the numerical results from the third scenario (i.e. based on measured soil properties) are presented hereafter.

Figure 4 illustrates the overall lateral deformation and residual displacement pattern of the FE model and Figure 5 summarizes the soil lateral deformations as well as settlements observed and computed. According to Figure 5, the sheet pile quay wall was predicted to move laterally 0.85m toward the water, which underestimated the recorded 1.1m seaward displacement. Moreover, based on the FE simulation, the ground settled about 0.24m on the landside and heaved about 0.24m on the waterside while the experimental results showed 0.24m settlement and 0.34m heave.

Overall, the large ground deformations due to the extensive liquefaction in the soil during shaking were reproduced fairly well by the dynamic FE model (i.e. 0.85m in the FE model and 1.11m in the shake table experiment). However, it should be noted that the numerical analysis underestimated the maximum displacement recorded in the experiment by 34%.

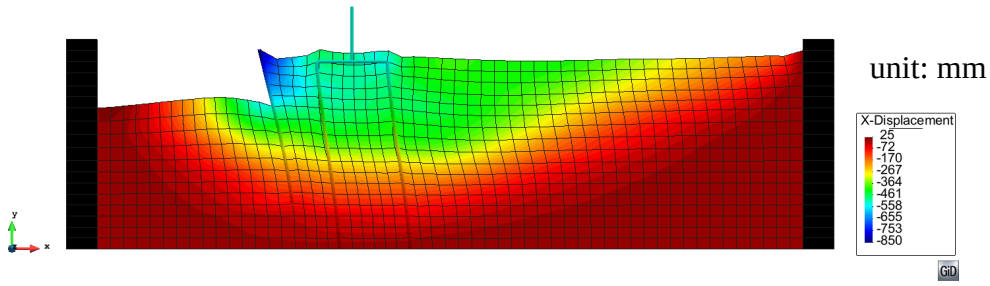

Figure 4. Deformed shape and horizontal residual displacements of the numerical model

Excess Pore Water Pressure Buildup

In order to monitor the generation, redistribution and dissipation of excess pore water pressure (PWP), the physical model was instrumented with PWP sensors inside the ground at different depths. For comparison, the recorded and predicted excess PWP data on the landside at two locations (see Lines E & F in Figure 1) and three different depths is presented in Figure 6. Numerically and experimentally, the liquefaction state in the ground was achieved soon after shaking started likely because of several pronounced peaks in the input motion. At a few locations, recordings of excess PWP were overall slightly higher than computed values since the sensors sunk into the ground during the liquefaction state. In addition, the excess PWP of the experimental model built up faster and dissipated more slowly than dynamic FE model. Moreover, the measured excess PWP exhibited profound fluctuations before 20sec during some loading cycles while the numerical model failed to reproduce such a strong cyclic mobility response.

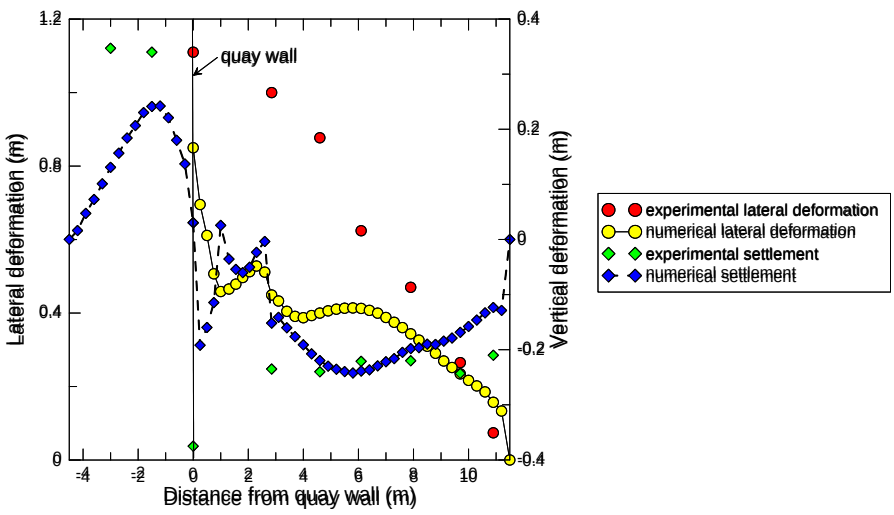

Figure 5. Comparison of residual lateral and vertical deformations of the ground surface

Acceleration

Figure 7 displays the computed and recorded acceleration time series of the horizontal soil acceleration at different locations and various depths. In order to present the comparison more clearly, only first 25sec time histories are presented. The computed accelerations reproduced the horizontal acceleration responses reasonably well, but missed some of the one-sided acceleration spikes associated with the observed instantaneous sharp drops of the excess PWP, which were resulted due to the soil dilatancy.

Bending Strain of Piles

Bending strain along the piles was recorded during the experiment by pasted pairs of strain gauges in the direction of liquefied soil lateral flow. The analytical bending moments of the piles were converted into bending strain assuming elastic linear behavior.

The estimated and measured time histories of bending strain of the piles (A at front row and B at rear row) are compared in Figure 8. The bending strain records and the analytical simulation model showed consistent bending strain at a depth of 3900mm although estimated values missed the large negative bending strains which were developed at the pile heads and large positive bending strain attained at the middle height of the piles. It is noteworthy here that due to failure mechanism and disconnection of some strain gauge cables, some records show a sudden increase in the strain amplitude which is not realistic. Overall, it seems the simulation results of piles response were in less agreement with the experimental data compared to the soil response. This discrepancy could be due to several reasons such as the three-dimensional nature of the experiment and disconnection of the base during the experiment.

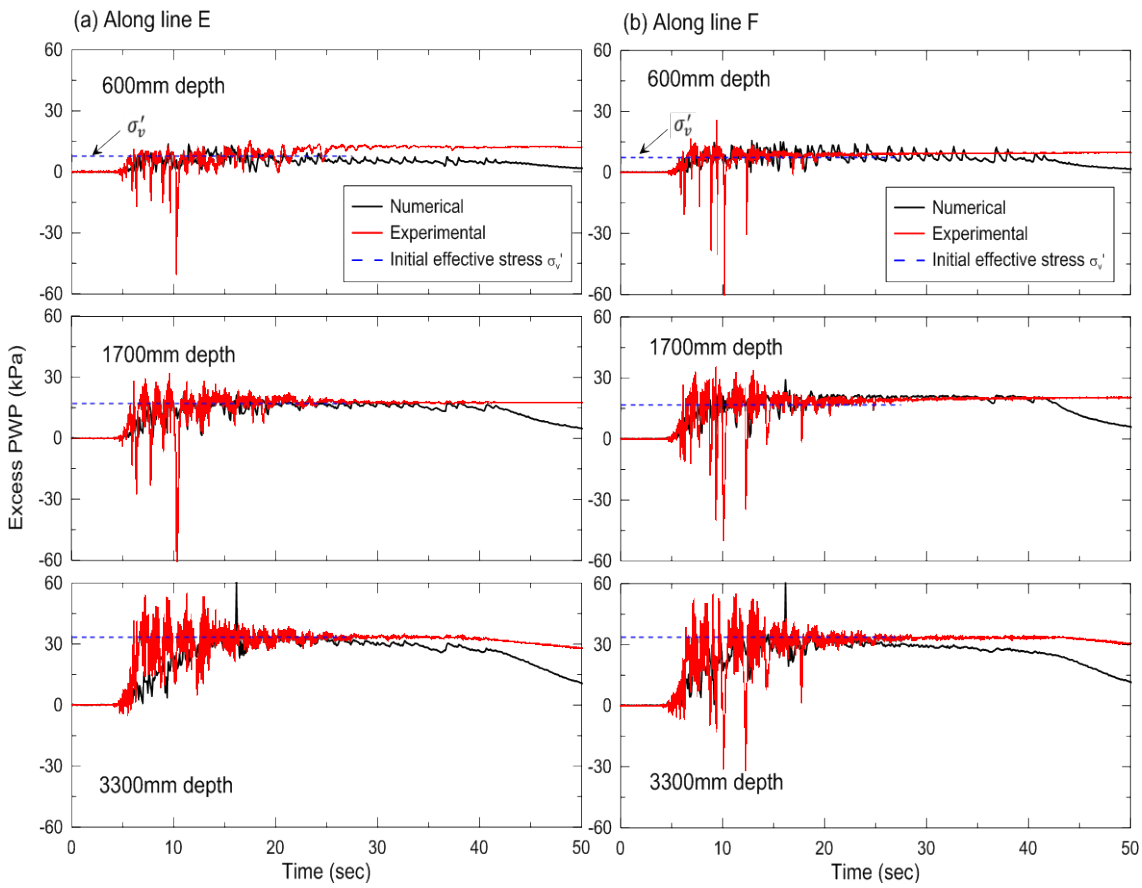

Figure 6. Excess pore water pressure time history response of soil

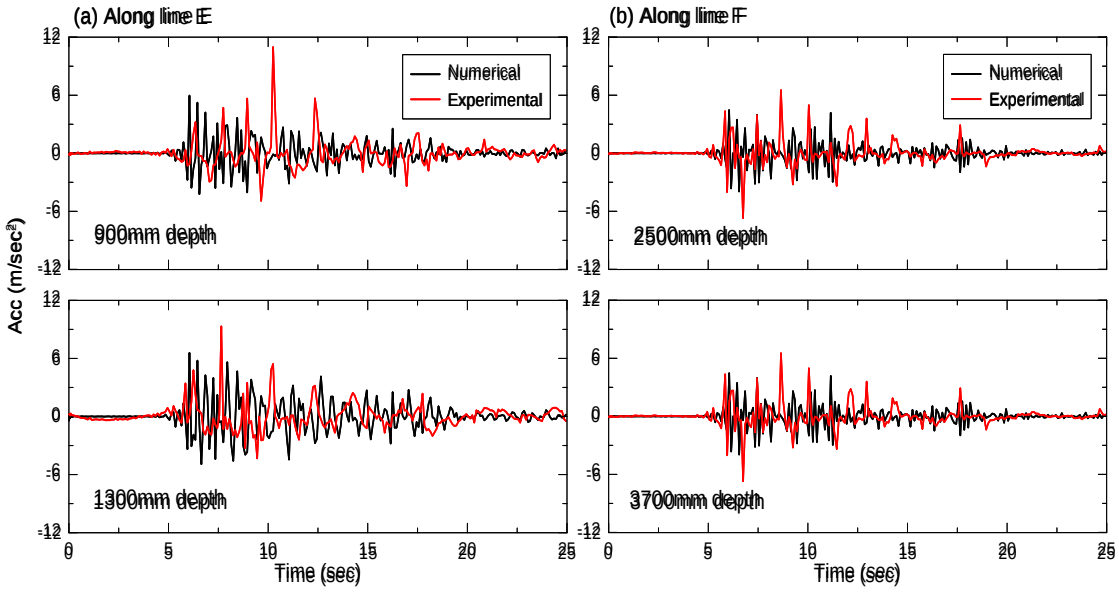

Figure 7. Horizontal acceleration time history response of soil

Sensitivity Analysis

A sensitivity analysis was carried out in order to understand the effects of the internal angle of friction and small strain shear modulus on the lateral displacement of liquefied soil. Based on the correlation between friction angle and relative density of silica sand (Kulhawy, et al., 1990), four friction angle values (31°, 33°, 37° and 39°) were selected to perform sensitivity analysis. The analysis results of lateral displacement of the soil at two locations (surface behind the sheet pile wall and surface at mid-length of the container) are demonstrated in Figure 9(a). It is shown that lateral soil deformation decreased with the increase of friction angle at all locations since soil becomes stiffer with larger friction angle. The excess PWP records were also investigated and it was found that liquefaction was achieved in all cases.

In light of the correlation between small strain shear modulus, relative density and void ratio of silica sand (Kokusho et al., 1980), four initial shear moduli, G_0, (40000kPa, 45000kPa, 60000kPa, 65000kPa) and associated respective reference bulk moduli, B, were chosen to perform a sensitivity analysis. The effect of the initial shear modulus is shown in Figure 9(b) and suggest that a decrease of G_0 cause an increase in horizontal soil displacement. The lateral deformation of the soil behind the quay wall was more sensitive to G_0 than the soil at mid-length of the container.

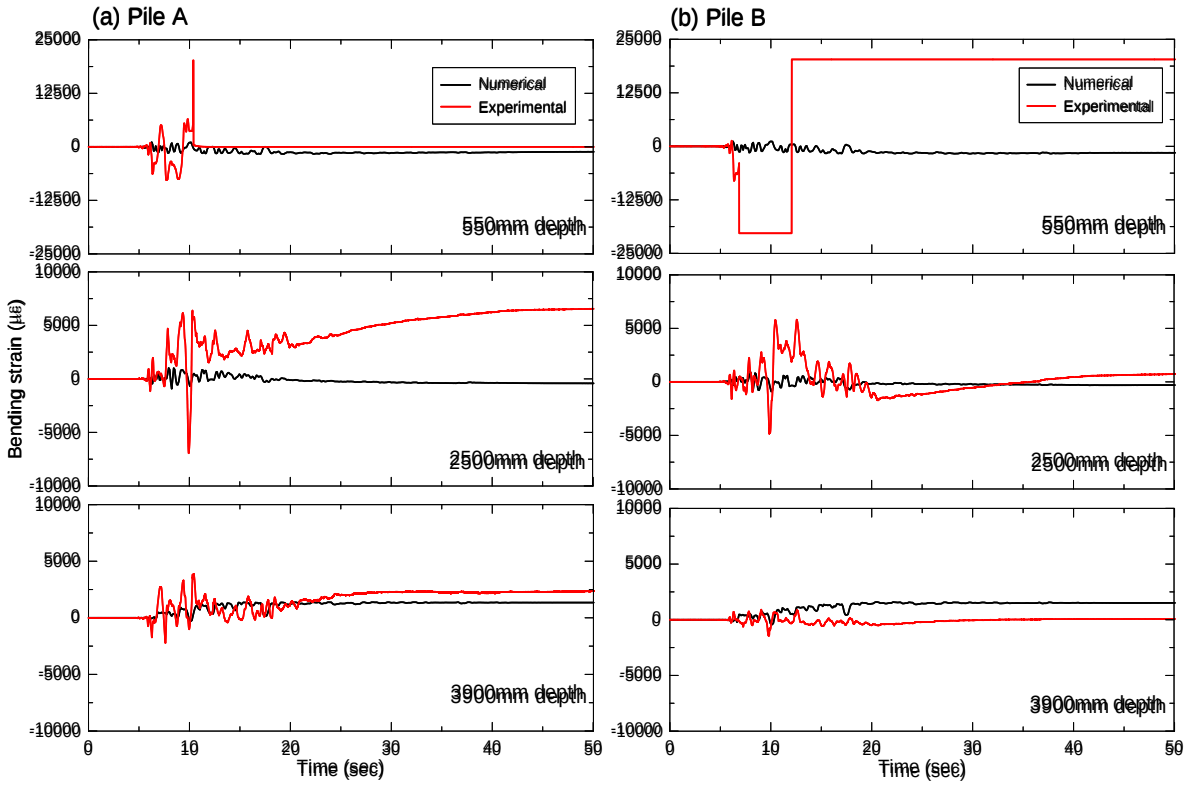

Figure 8. Bending strain time histories of piles

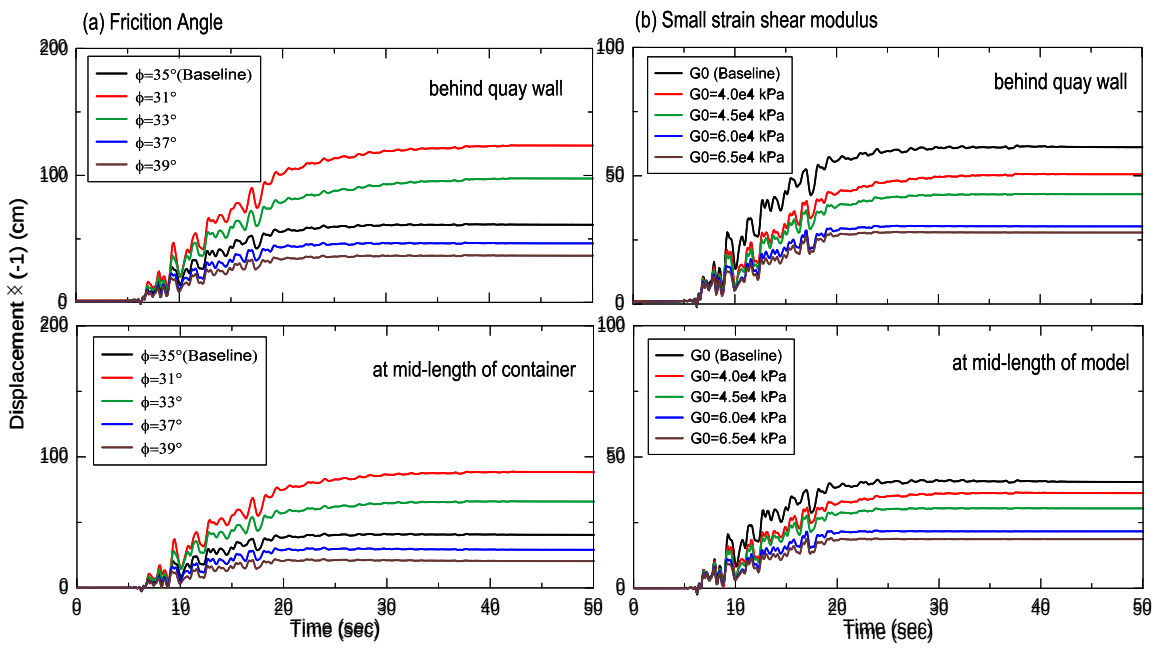

Figure 9. Influence of soil properties on lateral soil displacements at ground surface

Conclusions

The accuracy and effectiveness of using existing constitutive models in 2D effective stress FE analyses to reproduce the response characteristics of liquefaction-induced lateral spreading of pile foundation and superstructure have been evaluated. The computational results were compared with the large-scale shake table experimental results. Considering the involved complications, it can be said that an overall agreement has been reached concerning the dynamic response of the soil. However, the 2D FE model was unable to accurately predict the seismic demands on the piles, though the overall behavior was effectively captured. Because of the limitations and uncertainties involved in the numerical model, considerable differences were observed between the predicted and measured data. There are a number of possible reasons for this including, but not limited to, the uncertainty in the FE model parameters, differences between the 3D physical model and the 2D approximation, and the unexpected disconnection of front row piles and induced overturning action of the pile cap in the experiment. Sensitivity studies of the numerical model showed that the lateral soil displacements were sensitive to the friction angle and small strain shear modulus of the soil. Hence, these critical input constitutive parameters need to be characterized carefully in engineering practice if reasonable results are to be obtained.

Acknowledgement

The large-scale experiment described in this paper was carried out at the National Research Institute for Earth Science and Disaster Prevention (NIED), Hyogo Earthquake Engineering Research Center and was sponsored by the Japan Ministry of Education, Culture, Sports, Science, and Technology (MEXT). These supports are greatly appreciated.

References

Abdoun, T., and Dobry, R., 2002. Evaluation of pile foundation response to lateral spreading, *Soil Dynamics and Earthquake Engineering* **22**, 1051–1058.

Boulanger, R. W., Curras, C. J., Kutter, B. L., and Wilson, D. W., 1999. "Seismic soil-pile-structure interaction experiments and analyses." *J. Geotech. Geoenviron. Eng.* **125**, 750–759.

Chang, D., Boulanger, R., Brandenberg, S., and Kutter, B. (2013). "FEM analysis of dynamic soil-pile-structure interaction in liquefied and laterally spreading ground." *Earthquake Spectra*, Volume **29**, No.3, pages 733-755.

Kokusho, T. (1980). "Cyclic triaxial test of dynamic soil properties for wide strain range." *Soils and Foundations*, Vol. **20**, No. 2, pp.45-60.

Kulhawy, F.H. and Mayne, P.W. (1990). Manual on estimating soil properties for foundation design. Report EPRI EL-6800, *Electric Power Research Institute*, Palo Alto, p.131.

Motamed, R., Towhata, I., Honda, T., Yasuda, S., Tabata, K., and Nakazawa, H. (2009). "Behavior of pile group behind a sheet pile quay wall subjected to liquefaction-induced large ground deformation observed in shaking test in E-Defense project." *Soils and Foundations*, Vol. **49**, No. 3, 459-475.

Tokimatsu, K., Suzuki, H., Tabata, K. and Sato, M. (2007). Three-dimensional shaking table tests on soil-pile structure models using E-Defense facility. *Proc. of 4th International Conference on Earthquake Geotechnical Engineering.* Paper No. 1529.