

# NUMERICAL STUDY ON LATERAL SPREADING OF LIQUEFIED GROUND BEHIND A SHEET PILE MODEL IN A LARGE SCALE SHAKE TABLE TEST

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### SUMMARY

Lateral spreading of saturated cohesionless soil behind a sheet pile wall is one of the typical ground failure phenomena resulting from strong ground earthquake shaking. Extensive damage related to backfill liquefaction and sheet pile failure has been observed in the past earthquakes. Seaward displacements of the sheet pile quay walls accompanied by lateral spreading of liquefied soil causes translation and inclination to neighboring pile foundations. Therefore, it is important to understand the mechanisms regarding seismically induced ground deformation behind sheet pile quay walls and to evaluate their effects on neighboring pile foundations. Recently, at National Research Institute for Earth Science and Disaster Prevention in Japan (NIED), a series of large scale shake table tests were conducted to study the seismic response of sheet pile wall system and the liquefaction and deformation characteristics of the saturated cohesionless backfill, as well as the response of the neighboring pile foundations. In these tests, the largest laminar box in the world was employed in order to obtain nearly full-scale testing results. The objective of the current study, which has been conducted in frame work of a collaborative research, is to make blind numerical prediction and simulation of the shake test results. At the first step, it was tried to model the results of cyclic triaxial tests to obtain the best material parameters to feed the constitutive model. Then, static self weight analysis was performed in order to obtain the initial state of stress distribution in the model ground before the shaking; and as the last stage, two-dimensional dynamic nonlinear effective stress analysis based on finite-element method was conducted. Simulation of liquefaction phenomena was relatively successful and analysis results showed that prediction of starting time of liquefaction in different parts of the soil was close to what had been observed during the shake test. The calculated values for the residual displacement on top of the sheet pile were smaller than the measured ones during the test. However, the incremental accumulation of displacements until the final cycle was quite satisfactory. The comparison showed that, although the prediction and/or simulation of the results do not exactly fit to the experimental measurements, the current two dimensional analyses could predict the incremental excess pore water pressure generation and residual pore water pressure at the end of the shaking. Moreover, these blind numerical predictions were performed without using test results, and outputs were compared only after the end of the analyses.

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## CURRENT COLLABORATIVE RESEARCH PROJECT

National research Institute for Earth Science and Disaster Prevention in Japan (NIED) is constructing the so-called "E-Defense" or the world's largest three dimensional full scale shake table near Kobe city in Japan. This unique facility will be used to reproduce dynamic behavior of full scale structure models subjected to actual records of huge earthquakes. Consequently, it will greatly contribute in improving the seismic performance and design of structures. The construction of the civil, mechanical and electrical facilities of "E-Defense" will be completed by April 2005. Readers are encouraged to visit the English website of "E-Defense" at: <a href="http://www.bosai.go.jp/sougou/sanjigen/3De/index.htm">http://www.bosai.go.jp/sougou/sanjigen/3De/index.htm</a>

Having a single dead line, which is April 2005, all the research groups are working round the clock to meet the targets. Three independent technical working groups are concentrated on preparation of practical plans for full scale shake table test models of reinforced concrete structures, soil-pile-structure systems and wooden structures. In the geotechnical engineering group alone, dozens of domestic research centers, universities, research institutions and private companies are collaborating to conduct numerical and experimental research.

As a parallel research group in geotechnical earthquake engineering division, NIED started a trilateral research with Nihon University and JIP Techno Science Corporation in Japan. Figure 1 illustrates general framework of the collaboration which aims at feeding the main E-Defense research project. General plan for this collaborative research, which is in its initial stage, is:

- 1- Run uniquely large scale shake table tests using the existing facilities at NIED (Tsukuba Branch), data processing and interpretation of the results [4] (256 channels) (2003 and 2004)
- 2- Simulation of the results with a two dimensional approach (2003 and 2004)
- 3- Simulation of the results with a three dimensional approach, checking and/or probably modifying the constitutive modeling (2004 and 2005)
- 4- Hopefully obtaining enough experience and skills to run blind analysis to roughly predict the soil behavior before running the huge shake table tests on E-Defense (2005 and 2006)

Paper No. 2516 in this proceeding [4] presents a part of the results of the first step of this research work and the current paper explains a part of the results of the 2-D simulations in step 2 of this framework.



Figure 1. General framework of the collaboration for the current study

#### **INTRODUCTION**

Recently, at National Research Institute for Earth Science and Disaster Prevention in Japan (NIED), a series of large scale shake table tests were conducted to study the seismic response of sheet pile wall system and the liquefaction and deformation characteristics of the saturated cohesionless backfill, as well as the response of the neighboring pile foundations [4]. In these experiments, both the liquefaction and post liquefaction stages were modeled and studied. The largest laminar box in the world was employed in order to obtain nearly full-scale testing results to ascertain the mechanisms of the lateral ground flow of the liquefied soil behind sheet pile quay walls and to evaluate the effects of the liquefied earth pressure acting on pile foundations both during the ground shaking and post liquefaction stage. In the current study, it was tried to run blind numerical prediction analyses of the results, wishing to obtain enough skills and experience to predict the E-defense shake table tests, in advance. This paper reports the numerical analysis results and compares the outputs with the results obtained from direct measurements during a shake test.

#### SHAKE TABLE TEST

A large laminar box (6.0m x12.0m x 3.5m) and a large shake table (15m x 14.5m) were used to perform the experiments (Photo 1). Clean sand from Kasumigaura area in Japan was sampled to make model ground in this experiment ( $D_{50}$ =0.21). Four reinforced concrete piles were installed in the center of the laminar box by pin connection to the base. Diameter of the piles was 15 cm and their length was 4.5 m (L/D=1/30). Center to center space between the piles was 0.9 m and a heavy steel top cap provided a rigid connection on top of the piles.



Photo 1 Large-scale laminar container, fixed on the shake table in NIED

Sheet piles were installed in the east side of the pile foundation with 30 cm space from the laminar box wall. These sheet piles were used to provide lateral soil pressure on the liquefied soil to keep it moving toward the pool side. Sample was prepared by pluviation of dry sand into water ( $Dr=40 \sim 50\%$ ). In the next step, the main sheet piles installed, and the filling procedure continued until the soil level in east and west side of the sheet pile reached to 4.0 and 3.2 m, respectively. The water level at this stage was 4 m from the base. Then, in the east side, a layer of unsaturated soil was placed to raise the ground level to 4.5 m. Soil, pile foundation and sheet piles were heavily instrumented with pore water pressure and displacement transducers, as well as accelerometers and strain gauges. Total number of 256 channels was used for data acquisition of the outputs of these instruments. Heavy plates were mounted and fixed on top cap until its weight reached to about 10 tons, as a model for a massive super structure.



Figure 2. Schematic illustration of the shake table test model

#### NUMERICAL ANALYSIS

#### General Description of the Analysis procedure

The objective of this step of the study was to simulate the shake table test which was introduced above. To this goal, a series of analyses were carried out. It is important to note that the analysis were conducted

as blind calculation state, and test results were compared with analytical results, only after the calculations were done. The 2-D dynamic analyses were conducted using the measured acceleration time history on the table as shown in Figure 3. Five cycles of sinusoidal wave with maximum acceleration of about 450 gal and frequency of 4 Hz was used as input motion. DIANA, which is a multi-purpose finite element code was employed in these analyses. A modified version of Towhata-Iai constitutive model [1, 2, 3, 5, 6] was linked to DIANA [7] (release 8.1.2, 28 Nov. 2003) as user subroutine and the calculations were conducted based on it. The analyses were performed in three steps. At the first step, it was tried to model the



Figure 3 Measured input motion time history on shake table

results of cyclic triaxial tests to obtain the best material parameters to feed the constitutive model. Then, static self weight analysis was performed (two-phased analysis), in order to obtain the initial state of stress distribution in the model ground before the shaking; and as the last stage, two-dimensional dynamic nonlinear effective stress analysis based on finite-element method was conducted.

#### Material Modeling

The Towhata-Iai Model is based on an undrained two-dimensional approach of the soil (application under drained condition or application in three-dimensional models is not possible). The constitutive model of Towhata-Iai is formulated by the following relationship for the stress [1, 2, 5, 6, 7]:

$$\begin{cases} \sigma'_{xx} \\ \sigma'_{yy} \\ \sigma'_{xy} \end{cases} = -B(\varepsilon_p - (\varepsilon_{xx} + \varepsilon_{yy}))^2 \begin{cases} 1 \\ 1 \\ 0 \end{cases} + \sum_{i=1}^{l} Q_i(\gamma_i) \Delta \theta_i \begin{cases} \cos \theta_i \\ -\cos \theta_i \\ \sin \theta_i \end{cases}$$
(1)

with stress  $\sigma$ , total strain  $\varepsilon$  and volumetric plastic strain  $\varepsilon_p$ . The stress components are described by two parts. The first term is isotropic component and the second, the deviatoric component. The isotropic stress is directly derived from a state variable which is defined as the ratio of the initial effective isotropic stress. The state variable (or liquefaction front parameter) is based on the shear stress variable which is the shear stress divided by the initial isotropic stress. These expressions are governed by the angle  $\phi_f$ , the friction angle at failure, and  $\phi_p$ , the friction angle determined by the phase transformation line. Four other parameters are also involved in determining the state variable as a function of the shear work. These parameters are material parameters characterizing the cyclic mobility of the cohesion-less soil. The deviatoric model is composed from several virtual shear mechanisms, called springs. Each of these mechanisms gives a contribution to the shear stress components. It was tried to model the results of cyclic triaxial tests to obtain the best material parameters to feed the constitutive model. The parameters of the model were determined as shown in Figure 4 and Table 1.



Figure 4 Model simulations of liquefaction resistance of Kasumigaura sand

Table 1: Material parameters for Towhata – Iai constitutive model of soil

K <sub>a</sub> (kPa)	G <sub>ma</sub> (kPa)	$p_1$	<b>p</b> <sub>2</sub>	$\mathbf{W}_1$	$S_1$	<b>C</b> <sub>1</sub>	sin¢' <sub>f</sub>	sin¢'p	$H_{v}$
97500	45000	0.45	0.80	1.35	0.0001	1.30	0.574	0.469	0.3

 $K_a$  and  $G_{ma}$  are given for  $(-\sigma'_{ma}) = 98$  kPa

Figure 5 shows stress –strain curve and effective stress path during cyclic loading as model performance with these parameters in one of the typical element test simulations.



Figure 5 Model performance in element test simulations: (a) stress strain curve; (b) effective stress path

Table 2 shows the material parameters for steel sheet piles and reinforced concrete pile foundations. The stiffness of the group pile was considered as total stiffness of the four piles in the center of the group, as a single pile.

	Moment Inertia I (m <sup>4</sup> )	Young's Modulus E (kPa)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Poisson's ratio v
Pile Foundation	9.089E-3	33,500,000	4.944	0.2
Sheet pile	85.1E-8	211,000,000	3.568	0.2

Table 2: Material parameters for sheet pile and pile foundation

#### Analysis procedure

Figure 6 shows the finite element mesh of the numerical model (1155 nodes and 1142 elements). The analyses were performed in several consecutive phases (phased analysis option). At first, the model ground with horizontal surface was made to obtain the vertical stress to be used for finding the horizontal stress in  $K_0$  condition, and then, gravity

acceleration was applied. In this step Mohr-Columb criteria was used to consider the plastic deformation. After getting the vertical displacements of the model ground, the sheet piles and pile foundations were embedded in the model as linear elastic beam elements. In this stage, soil elements kept the stress that was computed in the previous phase. Then, gravity acceleration was applied to the model, with Mohr-Columb criteria. Towhata – Iai model could not be used



Figure 6 Finite element mesh of the numerical model

in phase – 1 and phase -2 analyses, since it could not model the drained behavior of soil. At the beginning of the next phase, the soil elements of pool area were removed, and gravity acceleration was applied again. By doing so, the sheet pile has been exposed to an unbalanced earth pressure from the backfill soil and submerged sand resulting in movement of the sheet pile towards the water and change of stress in soil. After equilibrium, Towhata-iai model was activated, and equilibrium was checked again on the stress state

that was obtained from Mohr-Columb criteria. These two phases of analyses could estimate the initial stress state that existed in the model ground prior to the application of dynamic excitation. This is considered as an important step in the numerical evaluation, since it is known that the seismic response and particularly the cumulative ground deformation during lateral spreading are significantly affected by static shear stresses. After getting the equilibrium with Towhata - Iai model, the displacements obtained in previous phases were ignored, and then, time integration was started (Newmark betha method). Top cap was modeled as a concentrated mass on top of the pile foundation and the mass of the water in pool was applied on nodes on sheet pile. Left side and right side nodes of the model in the pool level were constrained to move together in horizontal direction. Two cases were analyzed. In first case (case-1, hereafter), the piles were modeled as out-of-plane walls with a flexural rigidity equivalent to the combined rigidity of all individual piles aligned in the out-of-plane direction. Consequently, the soil was entirely separated by this vertical, continuous wall. In the second case (case-2, hereafter), the stiffness of the pile foundation was set to zero (very small value to avoid numerical instability), though the beam elements were remained in the model.

#### Numerical Simulation Results

Pore water pressure and pore water pressure ratio time history in backfill soil is shown in Figure 7. In this figure, the results from both analyses cases are compared with the measurements during the shake table test. The position of elements No. 783 and 243 are marked in Figure 6 and the location of pore water pressure transducer labeled as "PWP-4B" and "PWP-8B" is shown in Figure 2. Similar comparison for soil under the pool is shown in Figure 8. The position of element No. 100 and the location of pore water pressure transducer labeled as "PWP-8P" are shown in Figures 6 and 2, respectively.



Figure 7 Comparison of the pore water pressure and pore water pressure ratio time history in backfill (case-1, case-2 and Shake test results)



Figure 8 Comparison of the pore water pressure and pore water pressure ratio time history in soil under the pool (case-1, case-2 and Shake test results)

These results revealed that incremental accumulation of pore water pressure in case-1 and case-2 analyses are almost the same, indicating that numerical simulation of the pore water pressure generation was not very sensitive to the stiffness of the pile foundations. Prediction of residual pore water pressure was relatively successful. Analysis results also showed that prediction of starting time of liquefaction was close to what had been observed during the shake test. Pore water pressure was slightly larger than zero in the beginning of time integration. It is important to note that at the starting point of time integration, the stress state, which was obtained by self weight analysis, was adopted. The stress state was given to the constitutive model, and iterative computation was conducted to reach the equilibrium state. On the other hand, the stress state is not isotropic, and there is initial shear stress. Therefore, during the iteration process, pore water pressure increased because of the plastic shear work that is generated from the initial shear stress and deformation by the initial shear stress. Figure 9 shows pore water pressure distribution after about 2 seconds of the analysis for both cases. The figure indicates that model of pile foundation acted as a wall to detach soil elements in two sides. Even in case-2, in which the stiffness of the pile foundation was set to zero, pore water pressure distribution in two sides showed different pattern. This reminds that the limitations of a two-dimensional analysis should be kept in mind during numerical simulation of lateral flow behind a retaining structure.



Figure 9 Pore water pressure distributions after about 2 seconds

The displacement time history in top of the sheet pile was measured by a conventional laser transducer during the shake table test (DS-1 in Figure 2). Similarly, the displacement time history of the top cap was measured by an LVDT (DS-2 in Figure 2). Figure 10 compares these measurements with the results obtained from the two analyses (case-1 and case-2). Analyses showed smaller values. One of the reasons for this underestimation was probably due to application of larger stiffness for the sheet piles during the analysis. It is also important to note that the constitutive model could only simulate an undrained condition. Again, it is clear from these results that the analyses were not sensitive to the stiffness of the pile foundation and both cases resulted in almost similar residual displacements. However, the prediction of the test results until about 2.5 seconds were quite satisfactory.



Figure 10 Displacement on top of the sheet pile and pile foundation

Figure 11 shows the deformed mesh at the end of the analysis, which can be considered residual deformation at the end of the shake. The mode of deformation for the sheet pile is very similar to what was observed during the shake test. However, the existence of the beam elements to model the pile foundations, even with zero stiffness, caused some unrealistic behavior. The excessive settlement of the sheet pile is also noteworthy. The sheet pile was not supported at bottom point and since in these analyses, the friction between the soil and wall was ignored, slightly larger settlements were predicted.



Figure 11 Residual displacements and deformed mesh

#### CONCLUSIONS

It was attempted to predict and simulate the results of a large scale shake table test. Simulation of a single element of cyclic triaxial test, self weight static analysis and dynamic analysis were conducted, in sequence. These results revealed that incremental accumulation of pore water pressure was not very sensitive to the stiffness of the pile foundations. Prediction of residual pore water pressure was relatively successful. Analysis results also showed that prediction of starting time of liquefaction was close to what had been observed during the shake test. The results proved that model of pile foundation acted as a wall to detach soil elements in two sides and pore water pressure distribution in two sides showed different pattern. This reminds that the limitations of a two-dimensional analysis should be kept in mind during numerical simulation of lateral flow behind a retaining structure. Calculated values for the residual displacement on top of the sheet pile were smaller than the measured ones during the test. However, the incremental accumulation of displacements until the final cycle was quite satisfactory. It is also important to note that the constitutive model could only simulate an undrained condition. The comparison showed that, although the prediction and/or simulation of the results did not exactly fit to the experimental measurements, the current two dimensional analyses could predict the general phenomena which happened in the shake test to an acceptable extent.

This study is an on-going research process and more study is being conducted in the frame work of the collaborative research plan, as explained in the paper.

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