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THREE DIMENSIONAL ANALYSIS OF SOIL-PILE-STRUCTURE MODEL IN A SHAKE TABLE TEST

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SUMMARY

A series of two-dimensional shake table tests on soil-pile-structure models were performed in National Research Institute for Rural Engineering (NIRE) in Japan. A cylindrical laminar box with 1.8m diameter and 1.5m height was employed and a model of saturated sand to depth of 1.35 m, a group of four steel piles and a superstructure was subjected to two horizontal components of input acceleration (X and Y direction). The results of these shake tests, the first attempt in its type, were used for a numerical study. In this paper, the analysis results of the three-dimensional soil-pile-structure interaction by using three-dimensional nonlinear effective stress analysis method and discussion on applicability of this analysis method are presented.

The computed response in this analysis is in general agreement with that measured in the experiment. It was found that pore water pressure increments under two-dimensional input showed faster increase, compared to one-dimensional input. The acceleration responses of soil-pile-structure system were strongly affected by the pore pressure built-up. The computed accelerations of the pile and the structure under two-dimensional input agreed with the measured ones. It was found that this analysis method is applicable on the three-dimensional soil-pile-structure interaction problem considering liquefaction.

INTRODUCTION

Many kinds of structures were damaged in 1995 Kobe earthquake. The lessons learned from this earthquake suggest investigating failure mechanisms and collapse processes of various kinds of full-scale structures experimentally. For that purpose, National research Institute for Earth science and Disaster prevention in Japan (NIED) is constructing the world's largest three-dimensional (3-D) full-scale shake table in Miki city near Kobe, which is now nicknamed as "E-Defense" [1]. This facility will be used to

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reproduce dynamic behavior of full-scale structure models subjected to actual huge earthquakes. Consequently, it will greatly contribute in improving the seismic performance and design of structures. The construction of E-Defense will be completed by June 2005.

In order to study the most effective ways for using E-Defense, NIED conducted a new research project, tilted as “No.2 significant improvement of seismic performance of structures” [2]. One of the six research topics which are included in this project is “Test and analysis of soil-pile-structure systems”. The purpose of this research topic is to investigate a three-dimensional dynamic response and failure mechanism of soil-foundation system, and to obtain the data for development of 3-D numerical simulation method which can evaluate and predict the dynamic response of soil-foundation system by utilizing E-Defense.

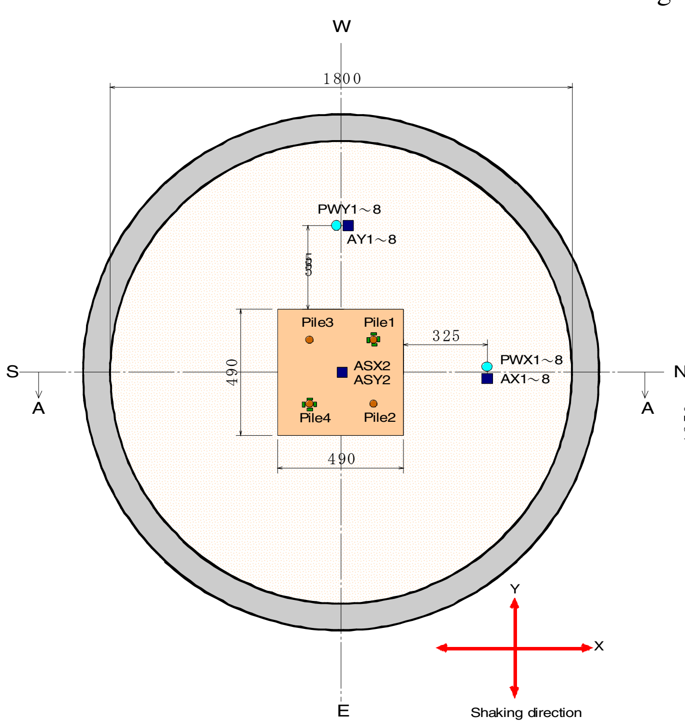
A series of shake table tests on dynamic response of soil-pile-structure system are being conducted by using the existing 3-D shake tables. We simulated one of these experimental models by using three-dimensional nonlinear effective stress analysis method. In this paper, the analysis results of the three-dimensional soil-pile-structure interaction and the discussion on the applicability of this analysis method are presented.

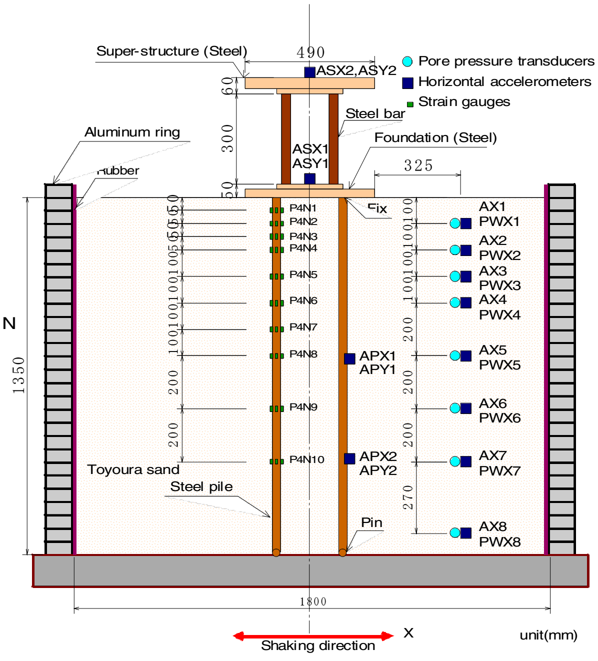
OUTLINE OF THE SHAKE TABLE TEST

Shake table and soil laminar box

A series of two-dimensional shake table tests on soil-pile-structure model were performed in National Research Institute for Rural Engineering (NIRE) in Japan [3]. Plan and cross-sectional view of the physical model of experimental soil-pile-structure system are schematically shown in Fig. 1.

The shake table is 4m in length and 6m in width and its usable payload is 490.3kN. An electro-hydraulic servo controlled system was used to provide three-dimensional base input motion. A cylindrical laminar box, 1.8m in diameter and 1.5m in height, was employed. This container was surrounded by a rubber membrane to protect leaking pore water of soil model and reinforced with 30 aluminum circular rings which could follow the shear deformation of the model ground.





(a) Plan section **(b) Cross section (A-A section)**

Figure 1 Experimental model of soil-pile-structure

Ground and structure model

A general view of the ground and structure model is illustrated in Fig.1. Toyoura sand was used in ground model. The thickness of the saturated sand layer was 1.35 m in the cylindrical laminar box. Sand layers were prepared by water sedimentation method. Then, the layer was compacted by vibration of a loading plate on the surface of the sand layer. The mean relative density of the layers reached to about 85%. The shear wave velocities were measured, as well. The water level was 30mm below the surface.

The structure model consisted of a 2×2 pile group, a foundation model and a super-structure model. All the piles were made of stainless steel pipes and had a diameter of 32mm with 1mm wall thickness. Their tips were pinned to the laminar box base with the pin joints, while their heads were rigidly fixed to the foundation model. Foundation model and super-structure model were steel mass with total weight of 47.1kg and 58.7kg, respectively. The steel columns with 36mm diameter and 300mm length rigidly connected foundation and super-structure model.

Measurement installation and input motions

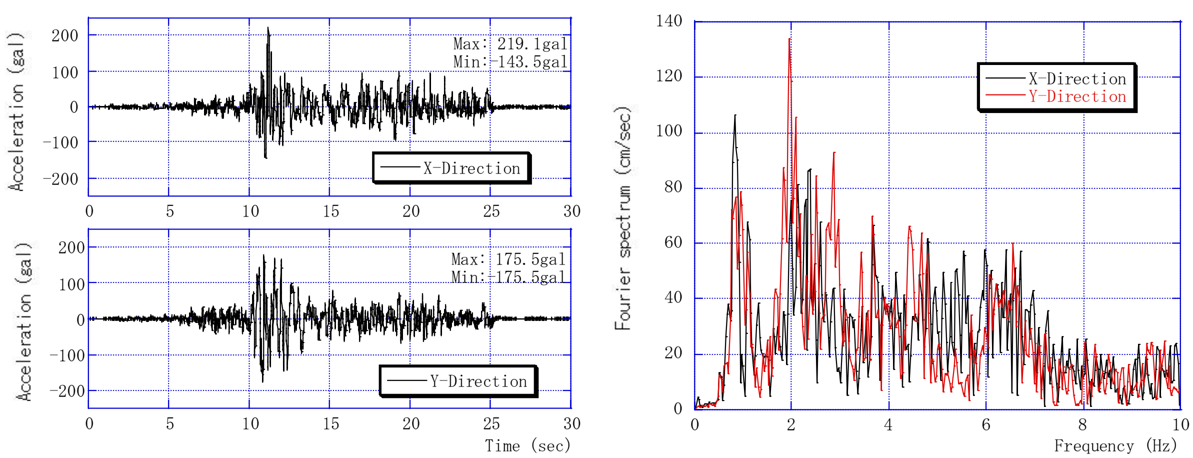
The location of the measurement transducers are shown in Fig.1. Accelerations of the sandy ground, piles, foundation and the super-structure, as well as pore water pressure in the sandy ground were measured. In order to measure the bending moments and displacements of the piles, several strain gauges were attached to two of the piles (Pile1 and Pile4).

For the shake table tests, the strong ground motions recorded at Hachinohe of Tokachi-oki Earthquake (May 16, 1968) were used as an input base acceleration. The duration of the input motion was reduced to $1/\sqrt{5}$ times of the real record, by considering the governing 1-G similitude [4]. Input motions measured on the shake table and their Fourier spectra are shown in Fig. 2; in which, X- and Y-Direction components are N-S and E-W direction of the recorded strong motion, respectively.

In the shake test, the following two test cases were conducted;

TEST-1: Shake table test subjected to one horizontal component of input acceleration (X direction)

TEST-2: Shake table test subjected to two horizontal components of input acceleration (X&Y direction)



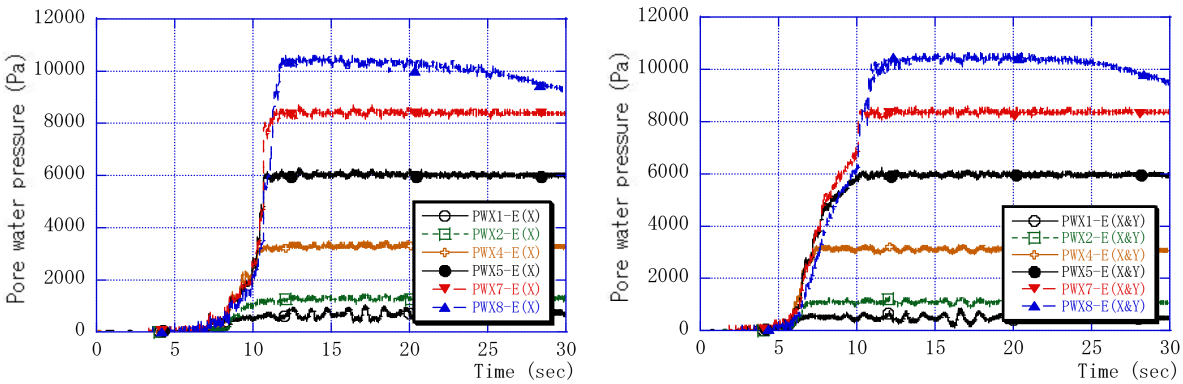
(a) Time history **(b) Fourier spectra**

Figure 2 Input motions measured

Test results

Fig. 3 shows pore water pressure of the ground at each measured point under one and two-dimensional inputs. It can be seen that water pressure increments under two-dimensional input started to increase faster than one-dimensional input case. Pore water pressures at each point under two-dimensional input began to rise in about 6sec, and increased gradually and approached the initial effective overburden pressure in about 12sec. While under one-dimensional input, pore water pressures started its rapid increase in about

10sec. However, the difference of water pressure built-up between one and two-dimensional inputs became smaller at the shallower depths.

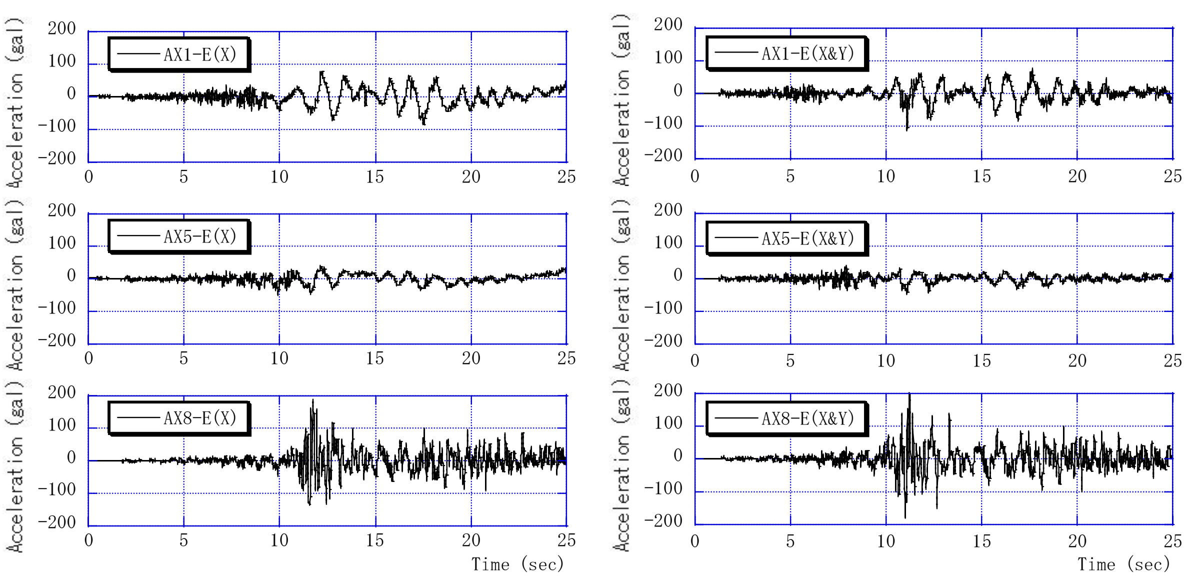


(a) One-dimensional input **(b) Two-dimensional input**

Figure 3 Time history of pore water pressure of ground at each measured point

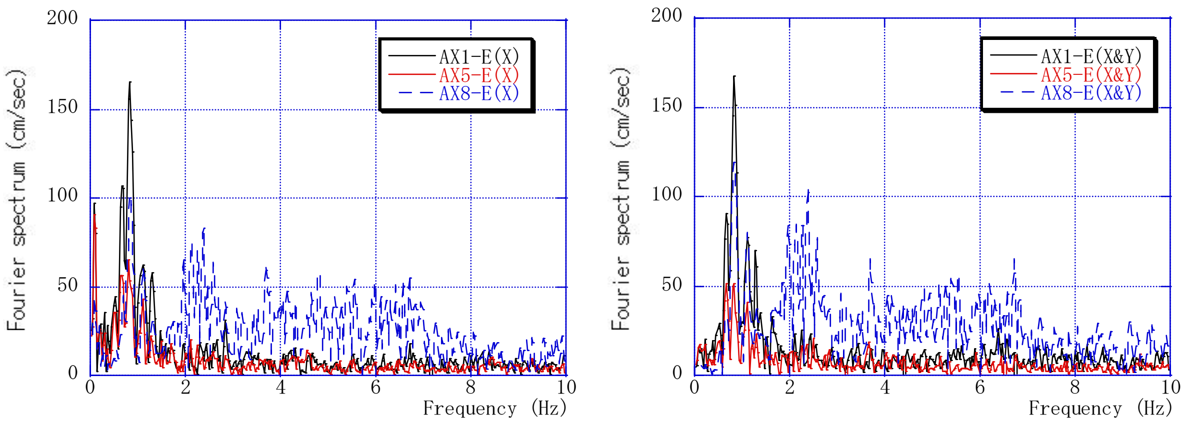
Figs. 4 and 5 show the time histories and Fourier spectra of acceleration of the ground at different depths, respectively. The response at the deepest measured point, AX8 (GL-1270mm), is similar to the input motion. At shallower measured point, the response in the high frequency range reduced in comparison to the point at greater depth (AX8). Furthermore, the component of the long period in the waveform is predominant after 9 sec which liquefaction occurred. This tendency is observed under two-dimensional input also.

Figs. 6 and 7 show the time histories and Fourier spectra of acceleration of the pile, foundation and super-structure, respectively. The response of pile, foundation and super-structure is almost the same and the component of the long period is predominant by occurrence of liquefaction similar to the ground. The response of the pile is larger than the surrounding soil, and effect of direct input from the pile tip and the inertial force of the super-structure can be considered as the reasons. There is almost no difference between the response of pile, foundation and super-structure in test cases with one and two-dimensional inputs.



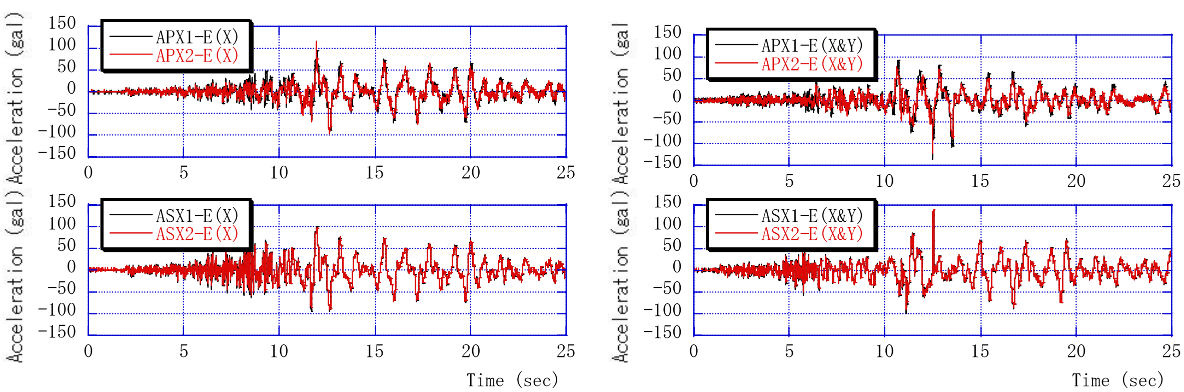
(a) One-dimensional input **(b) Two-dimensional input**

Figure 4 Time history of horizontal acceleration of ground at each measured point



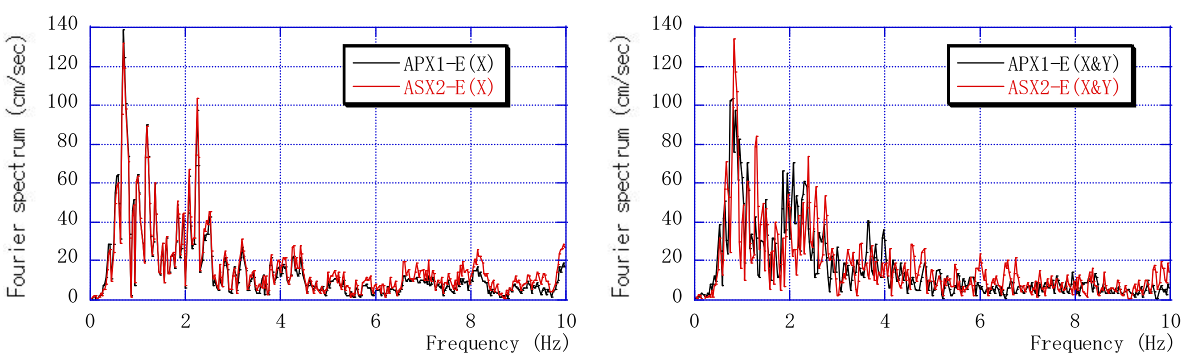
(a) One-dimensional input **(b) Two-dimensional input**

Figure 5 Fourier spectra of acceleration of ground at each measured point



(a) One-dimensional input **(b) Two-dimensional input**

Figure 6 Time history of horizontal acceleration of pile, foundation and super-structure



(a) One-dimensional input **(b) Two-dimensional input**

Figure 7 Fourier spectra of acceleration of pile and super-structure

NUMERICAL ANALYSIS OF SHAKE TABLE TEST

Numerical method and data for analysis

A finite element method (FEM) code "DINAS" [5] was used for numerical analysis in this study. In this code, three-dimensional nonlinear effective stress analysis method based on an explicit-implicit finite-element method is implemented. The code can model liquefaction of ground, including soil-structure interaction simulation. The constitutive model consists of the extended Ramberg-Osgood model (R-O model) and the Bowl model as a model of excess pore water pressure developed by Fukutake et al. [6].

Fig. 8 shows the three-dimensional finite element mesh of the soil-pile-structure system for the experimental model. The soil was modeled with 8-node solid elements and used similar constitutive models and parameters. Fixed and roller boundary conditions were considered in the bottom and lateral boundaries of the model, respectively. Steel piles and steel columns were modeled with beam elements and as linear elastic materials. Solid elements with rigid stiffness were used for the super-structure and the foundation. Total number of the nodes and elements were about 12,000 and 10,000, respectively.

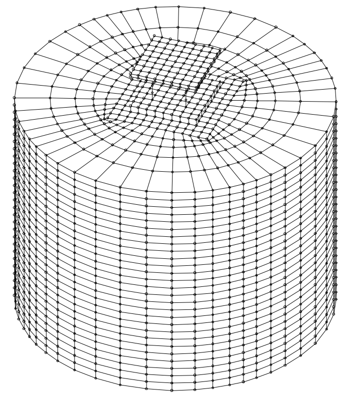


Figure 8 Finite element mesh for the experimental model

The parameters of the model were determined as shown in CASE-1 in Table 1 from the shake test condition. The shear moduli of soil were determined at center height of ground model and changed in the depth direction according to effective stress. The parameter of dependability on the effective stress was 0.5. Scaled-down 1-G shake table models of ground were characterized by very low confining stresses in the soil. For model shown in Fig.1, the effective overburden stress at the center height of the ground model was about 4.5 kPa. The parameters of the R-O model and the Bowl model under the low confining stresses were determined by theoretical simulations. Fig. 9 shows the variation of normalized shear modulus and damping with shear strain under the low confining stresses. Fig. 10 indicates the liquefaction resistance curves with the parameters shown in Table 1. In the figure, the measured values of the liquefaction resistance of Toyoura sand with effective stresses are also plotted [7, 8]. The material parameters of piles and columns are shown in Table 2.

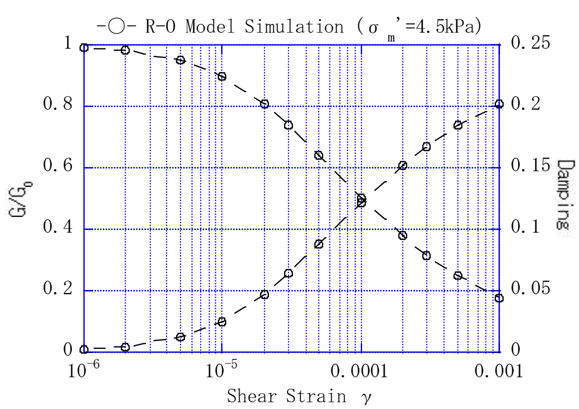


Figure 9 Variations of normalized shear modulus and damping with shear strain

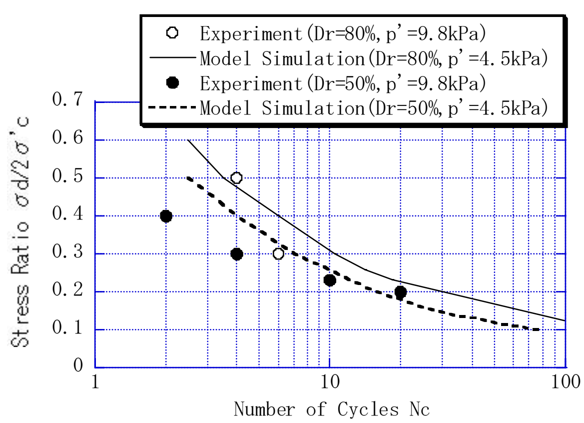


Figure 10 Liquefaction resistance curves

Numerical analyses were conducted using the shake table accelerations recorded in the experiment as input motion. Analysis was conducted for both one-dimensional input(X) and two-dimensional input(X&Y) motions. Duration of 0-25 seconds as shown in Fig.2, and a time step of 0.004sec were used in this numerical analysis. Total computing time in one case was about 130,000 seconds by using the personal computer with Pentium Ⅳprocessor(2.4GHz).

Table 1 Parameters for constitutive model of soil

CASE	Relative density (%)	R-O model			Bowl model					
		Initial shear modulus	Max. damping ratio	Reference shear strain	Positive dilatancy		Negative dilatancy		Swelling index	
		G_{0i}(kN/m^2)	h_{max}	$\gamma_{0.5i}$	A	B	C	D	$C_s/(1+e_0)$	X_l
CASE-1	80	16258	0.25	0.00005	-2.0	1.5	1.0	30	0.006	0.12
CASE-2	50	13853	0.25	0.00005	-1.5	1.5	1.0	20	0.006	0.08

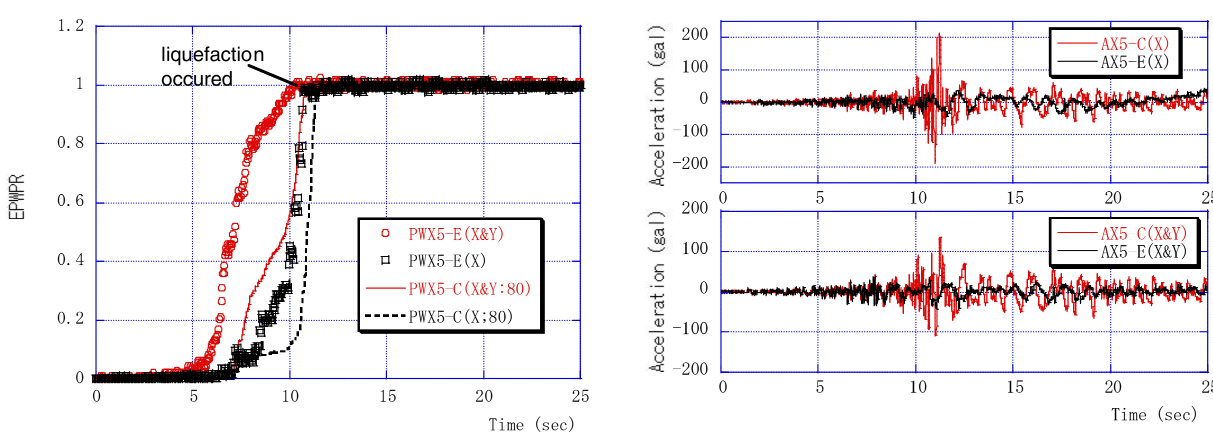
G_{0i}, $\gamma_{0.5i}$ at σ'_{mi}=1.0 kN/m^2

Table 2 Material parameters of piles and columns

	Cross section area (m^2)	I (m^4)	E (kN/m^2)	Unit weight (kN/m^3)
pile	9.74E-05	1.17E-08	1.93E+08	7.77E+01
column	1.02E-03	8.24E-08	2.06E+08	7.54E+01

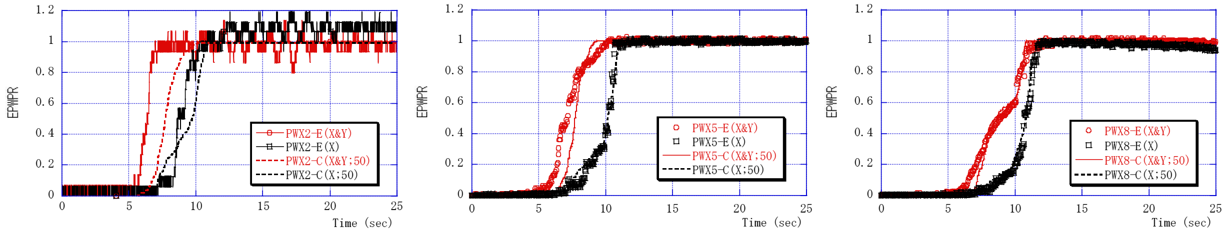
Results of the numerical analysis

Fig. 11 shows the computed (C) and measured (E) excess pore water pressure ratio (EPWPR) and the horizontal response of acceleration at the measured point PWX5 (GL-600mm) under one-dimensional input (X) and two-dimensional input (X&Y), respectively. The EPWPR means the ratio of excess pore water pressure to the initial effective overburden pressure. There is good agreement between the computed and measured time of occurrence of liquefaction. However, there are two notable differences between the computed and measured results, regardless of input component. One is the pore pressure built-up in EPWPR. The other is the maximum value and the waveform in 9-12sec of accelerations. This is probably related to overestimate the liquefaction resistance of saturated sand in shake tests. Then, the relative density, which greatly effects on the liquefaction resistance of saturated sand, is assumed smaller than the shake test condition. The parameters of the model were determined as shown in CASE-2 of Table 1.



(a) EPWPR **(b) Acceleration**

Figure 11 Comparison of computed and measured excess pore water pressure ratio (EPWPR) and acceleration at measured point PWX5 (GL-600m)

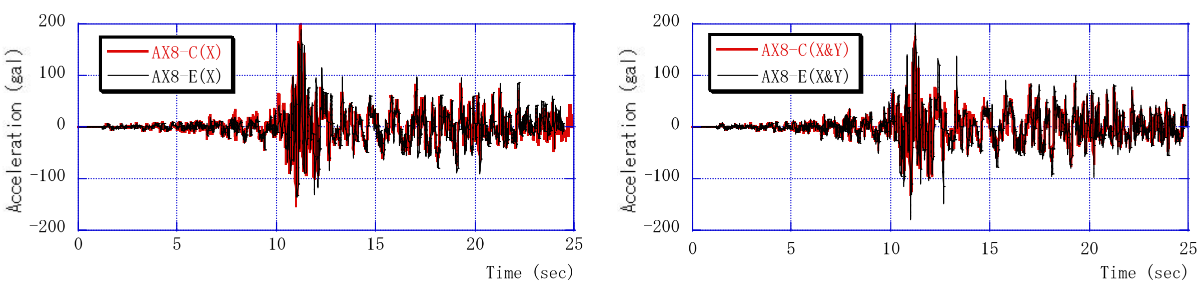


(a) PWX2 (GL-200m) **(b) PWX5 (GL-600m)** **(c) PWX8 (GL-1270mm)**

Figure 12 Comparison of computed and measured excess pore water pressure ratio (EPWPR) at each measured point

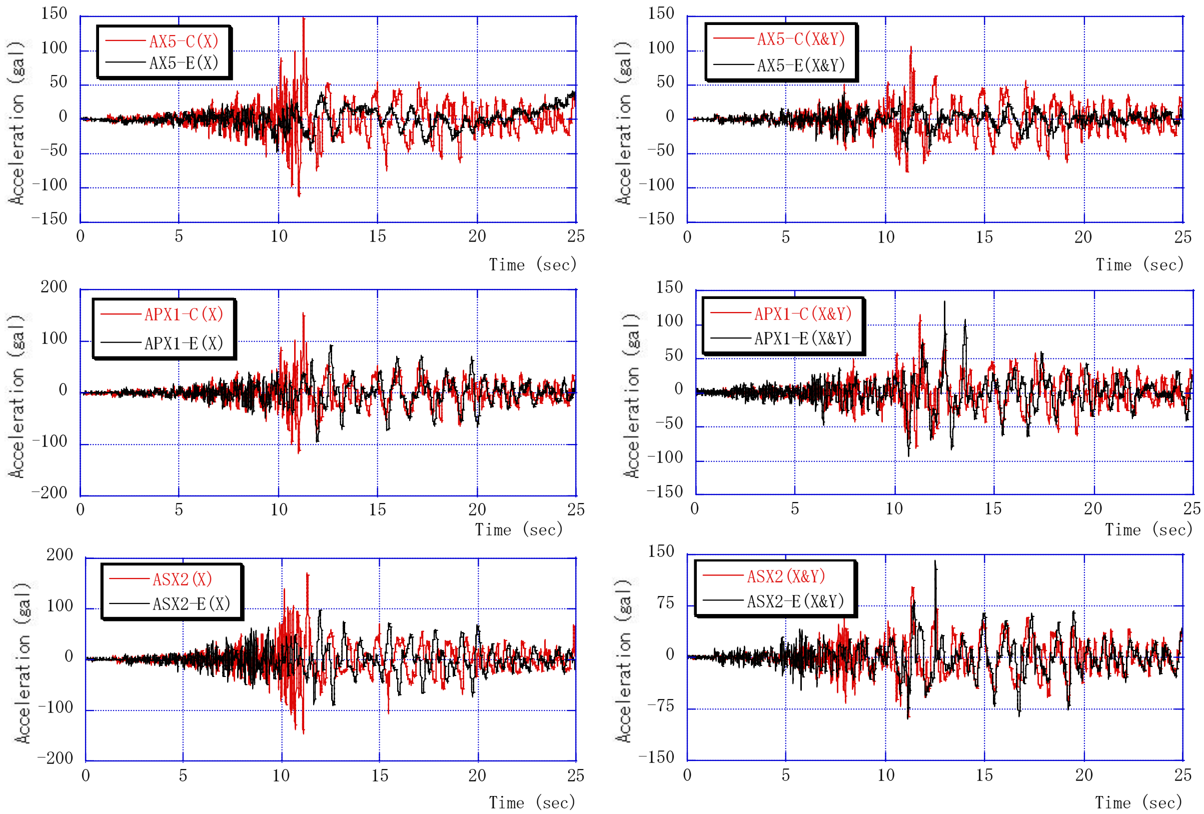
The computed EPWRP by using the parameters of CASE-2 in Table-1 are shown in Fig. 12 with the measured one. It can be seen that the water pressure increments under two-dimensional input start to increase faster than one-dimensional input case. The computed EPWPR at PWX5 and PWX8 (GL-1270mm) are good agreement with the measured ones, regardless of input component. In both of the computed and measured cases, the EPWPR under two-dimensional input began to rise from 6 sec and then, gradually increased and approached 1.0 in about 11 sec. While under one-dimensional input, the EPWPR increased rapidly in about 10sec. However, there are two notable differences between the computed and measured EPWPR at PWX2 (GL-200mm). One is the pore pressure built-up and the other is the time of occurrence of liquefaction. This difference under two-dimensional input is larger than that during one-dimensional input. It is thought that this point suggests the difficulty and the need for further investigation on the modeling under the low confining stress.

Fig. 13 shows the computed (C) and measured (E) acceleration of soil-pile-structure system at the measured points. Those are AX8 (GL-1270mm) and AX5 (GL-600mm) of ground, APX1 (GL-600mm) of pile and ASX2 (GL+410mm) of structure, under one- and two-dimensional inputs. Fig. 14 shows the computed and measured Fourier spectra of acceleration at ground (AX5) and super-structure (ASX2). It can be seen that the responses of acceleration are affected by the pore pressure built-up from 6 sec to 11 sec shown in Fig. 12. The computed acceleration at the deepest measured point of ground, PWX8, is in good agreement with the measured one. The component of the long period in the computed acceleration of AX5, where is in the middle depth of ground, is predominant after time of occurrence of liquefaction (12 sec), similar to the measured one. The amplitude of computed waveform becomes smaller than that shown in Fig. 11, but still larger than the measured one. The computed Fourier spectrum at AX5 has peak at the same frequency as that of the measured one, but the amplitudes in computed Fourier spectra are larger than the measured values. Further investigation is needed to increase the accuracy of the simulation. In both of the computed and measured response, there are almost no difference between one-and two-dimensional inputs.



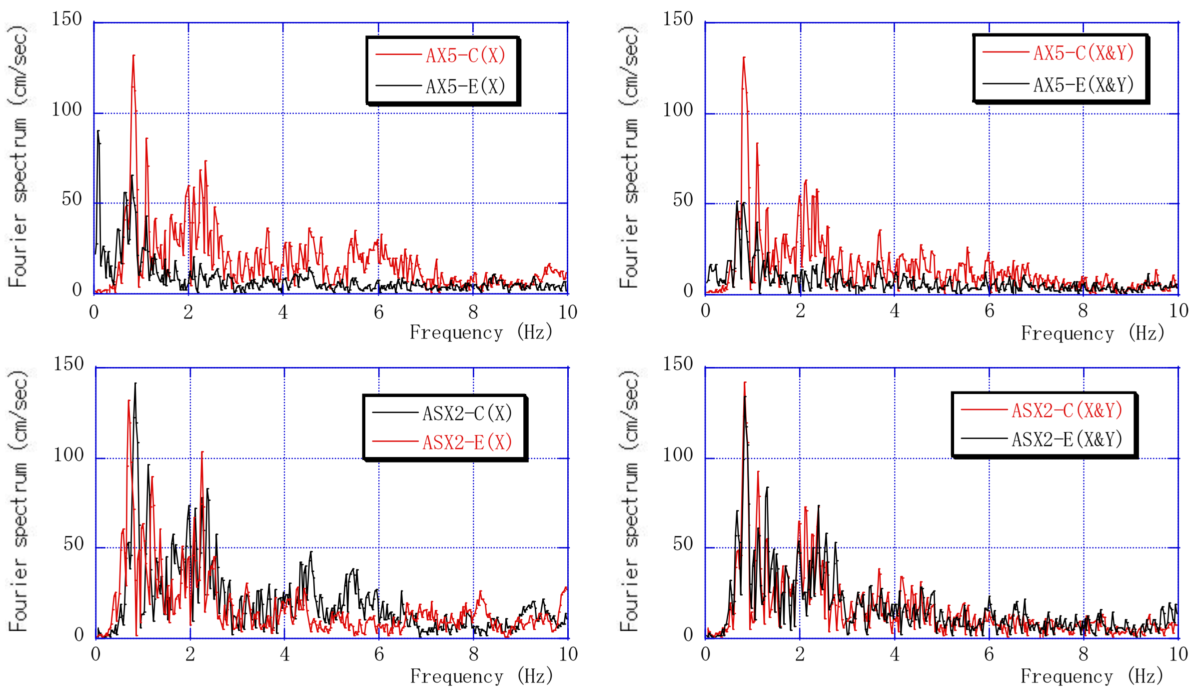
(a) One-dimensional input **(b) Two-dimensional input**

Figure 13-1 Comparison of computed and measured acceleration of soil-pile-structure system



(a) One-dimensional input (b) Two-dimensional input

Figure 13-2 Comparison of computed and measured acceleration of soil-pile-structure system



(a) One-dimensional input (b) Two-dimensional input

Figure 14 Comparison of computed and measured Fourier spectra of acceleration of soil and super-structure

The computed accelerations of the pile and the structure under two-dimensional input agreed better with the measured ones composed to one-dimensional input case. The computed Fourier spectrum at APX2 has peak at the same frequency as that of measured one, and their peak amplitude are similar to the measured values. The results indicate that, this analysis method could generally predict the three-dimensional soil-pile-structure interaction problem, while liquefaction occurs. However, further study and improvement seems to be necessary to accurate simulate of the test results.

CONCLUSIONS

A series of two-dimensional shake table tests on soil-pile-structure model were performed in NIRE. We simulated one of these experimental models by using three-dimensional nonlinear effective stress analysis method.

The responses computed in this analysis were in general agreement with those measured in the experiment. In particular, the pore pressure built-up and time of occurrence of liquefaction was well-simulated. Also, it could be found that the pore water pressure increments under two-dimensional input increased faster than that during one-dimensional input. The responses of acceleration of soil-pile-structure system are strongly affected by the pore pressure built-up. The computed accelerations of the pile and the structure under two-dimensional input agreed with the measured ones. However, exact simulation of the accelerations of the ground at shallow depths was not possible.

Although it is thought that this analysis method is applicable for the three-dimensional soil-pile-structure interaction problem considering liquefaction, further investigation is necessary by analysis for succeeding series of two-dimensional shake table tests on soil-pile-structure model performed in NIRE.

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