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A NUMERICAL STUDY OF DYNAMIC BEHAVIOR OF A SELF-SUPPORTED SHEET PILE WALL

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ABSTRACT :

Self-supported sheet pile walls constructed near water-front environments in large cities are vulnerable to large earthquakes because of corrosion or aging of their components. Thus, maintenance and reinforcement of such a structure are of prime importance for robustness of infrastructures in large cities. In this study, physical model tests under centrifugal acceleration of 50 G are conducted and results are compared with the one obtained by a finite element analysis. A model sheet-pile wall is made of aluminum plate and the model ground is either dry or saturated with viscous water. In the physical model tests, to properly consolidate the model ground before shaking, the model is put under 50 G with pile head fixed. Then, sinusoidal waves are input with no constraint at the pile head. Numerical analysis is conducted with the same dimension as a prototype of the physical model. When the lateral movement of the sheet pile head is fixed during the self-weight analysis in simulation, that is the same condition as the model tests, computed bending moments due to consolidation agree with measured ones and also deflections after shaking are generally consistent with each other. While if the sheet pile head is free in the self-weight analysis, computed bending moments and deformations disagree with measured ones. Thus, to simulate an existing sheet pile wall behavior during large earthquakes with accuracy required in practice, it may be necessary to have in-situ bending moment profiles and use them as an initial condition.

KEYWORDS: self-supported sheet pile, seismic response, centrifuge model test, effective stress analysis

1. INTRODUCTION

It is known that lateral deflections of a self-supported sheet pile wall become much greater than those of the other sheet pile methods with pile anchorage. However, self-supported sheet pile walls have been widely used as earth retaining works in urban area due to its simplicity and cost for construction. Recently, it is reported that self-supported sheet pile walls constructed near water-front environments in large cities are vulnerable to large earthquakes because of corrosion or aging of their components. Thus, maintenance and reinforcement of such a structure are of prime importance for robustness of infrastructures in large cities.
In this study, physical model tests under centrifugal acceleration of 50 G are conducted and results are compared with the one obtained by the effective stress analysis by FLIP (Finite element analysis program for LIquefaction Process) (Iai et al., 1992a). The analytical program has been applied to design many waterfront structures in Japan. Principal objective of the study is to verify the applicability of the program and clarify how the analytical results are influenced by varying analysis conditions e.g., initial restraint conditions of the sheet pile head before shaking or the way of modeling between the sheet pile elements and the corresponding soil ones. In what follows, units are in prototype, if not otherwise specified.

2. CENTRIFUGE MODEL TEST

The experiments were carried out in a rigid wall container mounted on the 2.5m radius geotechnical centrifuge at the Disaster Prevention Research Institute (DPRI), Kyoto University. Overall dimension of the rigid container is 450×150×300 mm in length, width, and heights, respectively. Dynamic excitation was given in longitudinal direction. The applied centrifugal acceleration was 50 G. A shake table installed on a platform is unidirectionally driven by a servo hydraulic actuator and it is controlled through a laptop computer on the centrifuge arm. All the equipment necessary for shake table control is put together on the arm. The laptop PC is accessible from a PC in the control room through wireless LAN.

A model configuration shown in Fig. 1 is a cross section of a self-supported sheet pile wall. The model sheet pile wall is made of aluminum plate and its dimensions are 15 m in width by 22.5 m in length by 0.2 m in thickness. The model was instrumented with 5 accelerometers (SSK, A6H-50), 2 laser displacement transducers (Keyence, LBP-080), 3 pore water pressure transducers (SSK, P306A-2) and 10 strain gauges (see Fig. 1).

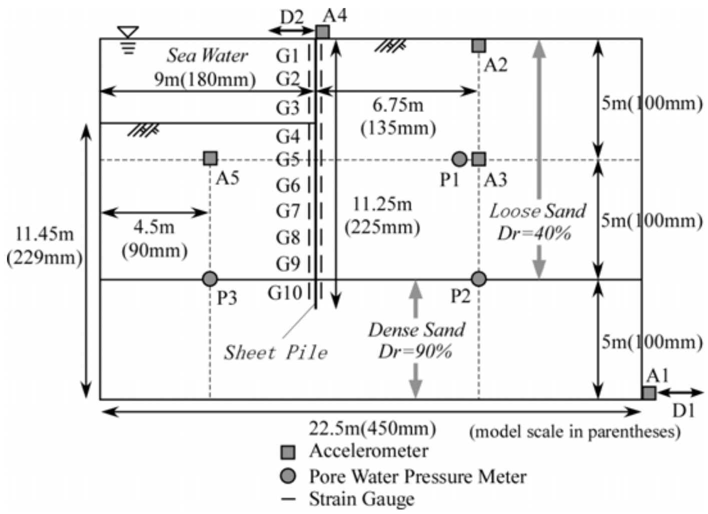

Figure 1 Model configuration in rigid container

Table 1 Summary of test cases

Case	Soil condition	Max. input displacement(mm)	Max. input acceleration (Gal)	Dr (%)
DC1	Dry sand	0.5	56.7	90
DC2		2.0	191.0	90
DC3		3.0	258.6	90
SC1	Saturated sand	2.0	141.2	40/90
SC2		2.0	101.4	40/90
SC3		2.0	210.9	40/90

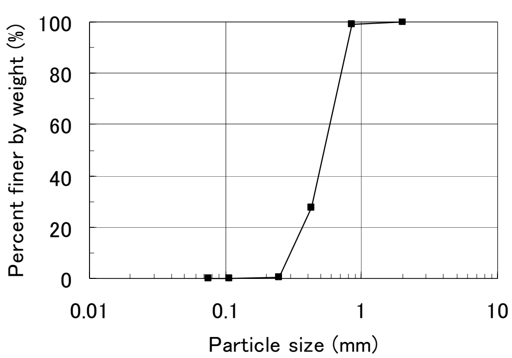

Figure 2 Grain size distribution curve for Silica sand No.5

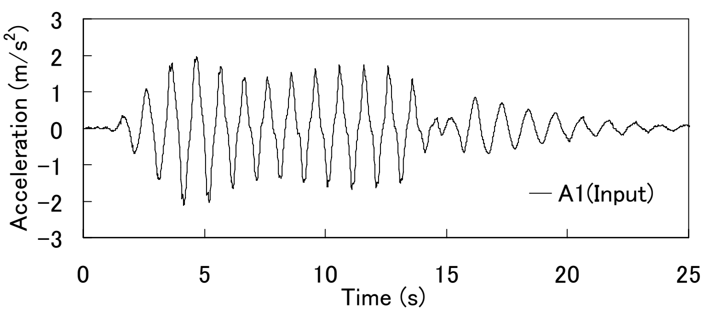

Figure 3 Input acceleration for SC3

Strain gauges were installed on the center of the sheet pile, at which strain and bending moment were measured. All the electric data was recorded by digital data recorders (TML, DC-104R) mounted on the centrifuge arm.

Fig. 2 shows the particle size distribution curve of the silica sand No. 5 (e_{max}=1.11, e_{min}=0.69, D_{50}=0.56 mm) used for deposit and backfill soil. For considering the presence of pore water, the sand deposit and the backfill sand were saturated with viscous fluid (Metolose, SM-25 Shin-Etsu Chemical Co.) whose viscosity was adjusted to 50 times of water (50cSt).

In the model tests, before filling sands, rehydratable noodles, which served as inclinometers for saturated sand deposit, were attached to the inner side of Plexiglas-like wall installed in the soil container. We employed either dried (air-pluviated) or saturated (water-pluviated) ground conditions. Relative density of the air-pluviated sand was about 70 %. On the other hand, two types of sand deposit were prepared by water-pluviation to relative densities of approximately 40 % and 90 % for, respectively, upper loose and lower dense deposit. During making the model ground, the movement of the sheet pile due to sand feeding was restrained with the jigs installed on the sheet pile head. To properly consolidate the model ground before shaking, the model was put under 50 G with pile head fixed for 5 minutes. Then, sinusoidal waves were input with no constraint at the pile head. Total six tests were employed as shown in Table 1; for dried condition, the input waves with the displacement amplitude of 0.5/2.0/3.0 mm in model scale were given.

Input motion was intended to be sinusoidal with 1 Hz and 15 waves. However, as shown in Fig. 3, for example, the amplitude was not constant. This might be due to the effect of mechanical resonance with the centrifuge arm during shaking. The peak amplitude of input acceleration in each case is shown in Table 1.

3. NUMERICAL ANALYSIS

To see the applicability of the effective stress analysis by FLIP (Iai et al., 1992a), the analytical results are compared with the experimental counterparts. The effective stress model used in FLIP is based on a multiple shear mechanism defined in strain space. The model consists of the constitutive law for average component and the one for deviatric component, called multiple shear mechanism [Towhata and Ishihara (1985)]. The mechanism has the capability to represent essential features in the cyclic behavior of sand such as the effect of rotation of principal stress axis directions. The effect is known to play an important role in the cyclic behavior of the anisotropically consolidated sand (Iai et al., 1992b)

Pore pressure model in FLIP is based on the concept of liquefaction front, in which the state variable S (conceptually equivalent to a mean effective stress) is defined by the combination of liquefaction front parameter S_0, which is a function of the shear work, and shear stress ratio r. Then modulus is adjusted by a function of mean effective stress ratio. Detail can be found in Iai et al. (1992a).

Let us explain the procedure of numerical analysis by FLIP. Firstly, a self-weight analysis is conducted under drained condition for calculating initial stress and strain of the model before shaking. The self-weight analysis is physically corresponding to consolidation of the ground. After the self-weight analysis, a dynamic analysis is conducted under undrained condition with the input motion specified along the base of the model, which was recorded at A1 in the centrifuge model test (see Fig. 1).

In the self-weight analysis, the lateral movement of the sheet pile was fixed to simulate the same condition as the centrifuge model tests, which is henceforth called “fixed pile head condition”. In addition, we conducted a self-weight analysis with the sheet pile head free in simulation, which is henceforth called “free pile head condition”, to study how the analytical results are affected due to the differences of initial restraint condition of the pile head. For each condition, the following two ways of applying gravity force were adopted. Namely “single step analysis” and “multi step analysis”; in single step analysis, gravity force was applied to the entire model at a time. In multi step analysis, gravity force was first applied only to the sand deposit, and next it was given only to the backfill soil.

The numerical model dimension was set as the prototype scale in the centrifuge model test. For solid phase, to have boundary conditions similar to the rigid container, displacement degrees of freedom at the base were fixed both horizontally and vertically, and only lateral displacements were fixed at two lateral boundaries.

Double nodes with the same coordinate but different node number were assigned to the sheet pile and neighboring soil node. By using the double node, the following restraint conditions can be given; “MPC_only” and “MPC_Joints.” Under “MPC_only” condition, we assume no friction between the sheet pile and the neighboring soils, and the amounts of lateral displacements of double nodes were set to be equal by using MPC command in FLIP. Under “MPC_Joints” condition, joint elements were employed between double nodes of the

Table 2 Model parameters for sheet pile

Young's modulus E(kPa)	Shear modulus G(kPa)	Poisson ratio ν	Mass density ρ (kg/m^3)
7.03×10^{7}	2.61×10^{7}	0.345	2.70×10^{3}

Table 3 Model parameters for deformation properties

Soil type		SPT blow count, N	Density	Porosity	Parameter for deformation characteristic							
					Elasitic tangent shear modulus	Elasitic tangent bulk modulus	Reference mean effective stress	Poisson ratio	Exponent of a power function for modulus	Internal friction angle	Cohesion	Max. damping ratio
			ρ	n	G_{ma}	K_{ma}	σ_{ma}'	ν	m	ϕ_f	c	h_{max}
		(N)	(kg/m^3)		(kPa)	(kPa)	(kPa)			(°)	(kPa)	
DC		21.6	1.44×10^{3}	0.45	1.248×10^{5}	3.256×10^{5}	98.0	0.33	0.50	41.15	0.0	0.24
SC	Loose	5.3	1.86×10^{3}	0.45	5.018×10^{4}	1.309×10^{5}	98.0	0.33	0.50	38.24	0.0	0.24
	Dense	31.2	1.92×10^{3}	0.45	1.596×10^{5}	4.162×10^{5}	98.0	0.33	0.50	42.33	0.0	0.24

Table 4 Model parameters for liquefaction properties

Soil type		SPT blow count, N	Parameter for liquefaction characteristic					
			Phase trans. Angle	Parameters for dilatancy				
		(N)	ϕ_p (°)	S_1	w_1	p_1	p_2	c_1
SC	Loose	5.3	28.0	0.005	0.760	0.50	1.131	1.524
	Dense	31.2	28.0	0.005	13.670	0.50	0.665	7.532

seabed in front of the sheet pile wall and pile nodes to consider the friction. However, between double nodes on the pile and backfill soil elements, the same condition as "MPC_only" was adopted because joint elements had the possibility of inducing negative excess pore pressure.

Modeling parameters are defined in Table 2, 3 and 4. The sheet pile parameters were adopted as standard values of aluminum (Chronological scientific tables, 2003). The soil and pore water model parameters were determined, firstly by fourier spectra of the input (A1) and surface (A2) acceleration waveforms recorded in the centrifuge model test. Then the spectra are smoothed with Parzen window, and predominant frequency is calculated by using frequency response function. Now, we deduce shear wave velocity from 1/4 wavelength rule, and shear modulus is obtained with the shear wave velocity and mass density of soil. Finally, with the shear modulus, the soil and pore water parameters are determined by using the simplified method for parameter determination (Morita et al., 1997). Only for the dense deposit under the saturated condition, by solving Eqn. 3.1 proposed by Meyerhof (1957) with relative density, we obtain SPT-N value.

$$Dr = 21\sqrt{N/\left(\sigma_v'/98+0.7\right)} \tag{3.1}$$

in which Dr = relative density; σ_v' = vertical effective stress, in kN/m^2. Then, we determine the soil and pore water model parameters from the empirical relations with SPT-N value (Morita et al., 1997).

4. MEASURED AND COMPUTED RESULTS

In this chapter, measured results are compared with computed ones to examine the numerical modeling. In addition, differences of the boundary condition are numerically examined. For the saturated condition, only the results of SC3 will be compared because of the luck of some measured data on SC1 and SC2.

4.1 Bending Moment profile

Bending moment profiles of the sheet pile due to gravity are shown in Fig. 4 and 5 for dried and saturated conditions, respectively. The profiles of numerical results shown in Fig. 4 and 5 are obtained after the self-weight analysis. In both figures (a) shows results of the single step analysis, while (b) shows those of the multi step analysis.

In the case of dried condition shown in Fig. 4, in terms of the peak values of the bending moments, the value about 4.5 m in depth under the fixed pile head condition shows good agreement with the experimental result, while under the free pile head condition the absolute maximum value of moment is about 2.5 times overestimated. In the range from 0 to 4 m in depth, there are significant differences between the measured and the computed profile of the fixed pile head condition. This may be due to the fact that in the analysis plane strain condition is assumed and the pile head is regarded as uniformly fixed in the direction perpendicular to the analytical mesh, while in the experiment sheet pile was supported only at the side of the sheet pile near the inner wall of the container, therefore the center of the sheet pile wall was somewhat close to the free condition.

Next, we compare the results under the saturated condition (Fig. 5). Peaks at about 5 m in depth under the fixed condition (both MPC_only and MPC_Joints) are generally consistent with the experimental result. Large discrepancy in the shallow range of these profiles may be due to the fixity condition at the pile head as mention-

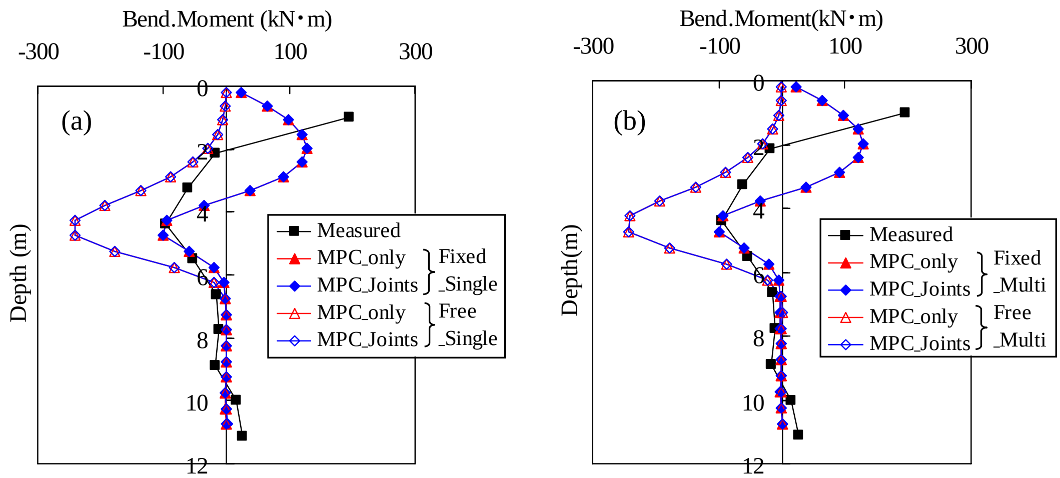

Figure 4 Bending moment profiles of the sheet pile before shaking under dried condition:
(a) Single-step analysis, (b) Multi-step analysis

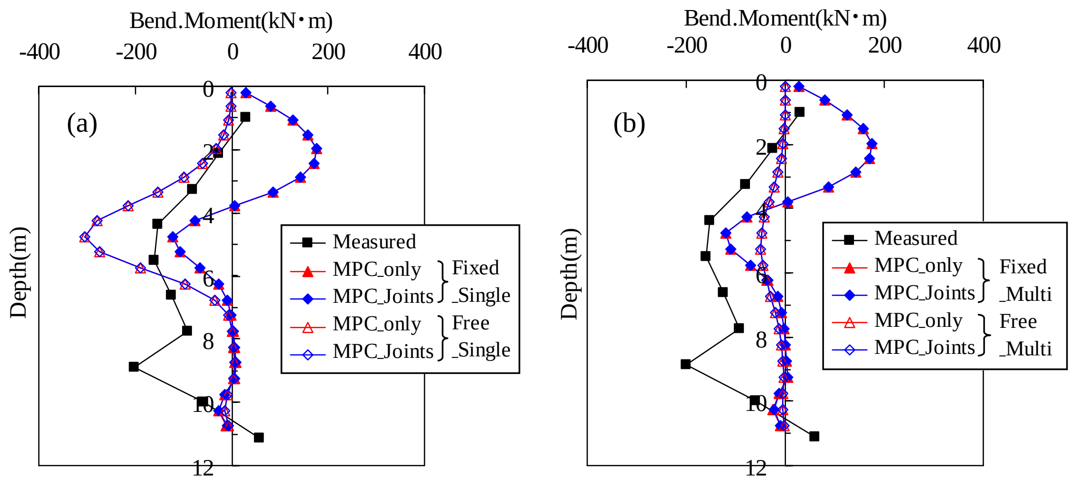

Figure 5 Bending moment profiles of the sheet pile before shaking under saturated condition:
(a) Single-step analysis, (n) Multi-step analysis

ed before. Under the free pile head condition, the absolute maximum value of the moment in the single step analysis is about 1.8 times overestimated, while the value in the multi step analysis is about 40 % underestimated. As shown in Fig. 4 and 5, in the self-weight analysis, fixity condition of the pile head has large influence on the bending moment profile, while existence of joint elements has little effects on it.

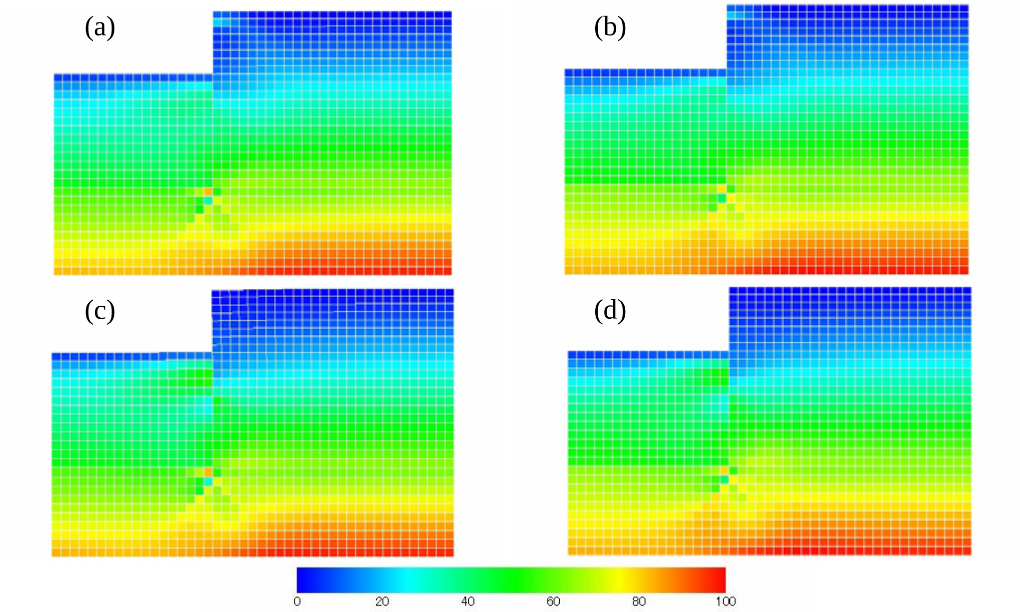

Figure 6 Simulated horizontal normal stress:
(a) Fixed_Single (b) Fixed_Multi (c) Free_Single (d) Free_Multi

Simulated horizontal normal stress distributions after the self-weight analysis under saturated conditions are shown in Fig. 6. In the fixed pile head condition [Fig. 6 (a) and (b)], horizontal earth pressure is distributed almost proportional to the depth, while in the free pile head condition [Fig. 6 (c) and (d)] slight variations in the soil near the sheet pile wall due to active and passive state caused by the deformation of the sheet pile wall are observed. Similar trend has been observed under dried conditions.

4.2 Lateral Displacement after dynamic loading

Lateral displacements of the sheet pile head after dynamic loading are compared in Figs. 7 (dried condition) and 8 (saturated condition). In Fig. 8, no result is obtained in the Free_Single analysis due to no convergence in simulation.

Fig. 7 indicates that the computed total displacements under the fixed pile head condition are about from 50 to 80 % of the measurement. Under the free pile head condition, we can see that the total displacements without joint element overestimate the measurement by up to about 6 times, while displacements with joint element decrease to about 30 % of the case of "MPC only". The displacement in the single step analysis is approximately equal to the one in the corresponding multi step analysis. Since total displacements of the dried condition shown in Fig. 7 are small,, i.e., less than 1 % of the sheet pile length (11.25 m), from an engineering point of view, a self-supported sheet pile wall in the dry sand deposit with relative density of 70 % may be stable for seismic shaking with the amplitude of input acceleration up to 300 Gal.

Next, we compare the results under the saturated condition. Fig. 8 shows that under the fixed pile head condition, lateral displacements with single step analysis are about 1.7 times overestimated, while the displacements with multi step analysis are comparatively close to the measurement. In particular, the computed result of multi step analysis with joint element is in good agreement with the measured one. Under the fixed pile head condition, lateral displacements in the single step analysis generally show greater than the one in the multi step analysis. In the free pile head condition with multi step analysis, we can see that the computed displacement with no joint element (MPC_only) are in close agreement with the measured one, while the displacement computed with joint element is overestimated 2.3 times.

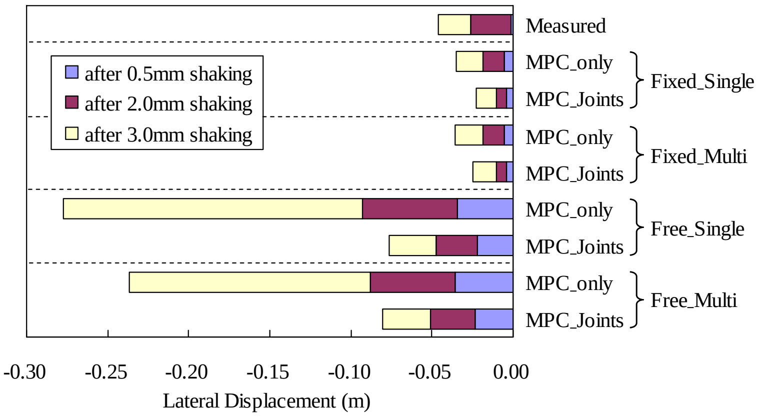

Figure 7 Lateral displacements of the sheet pile head after shaking under dried condition

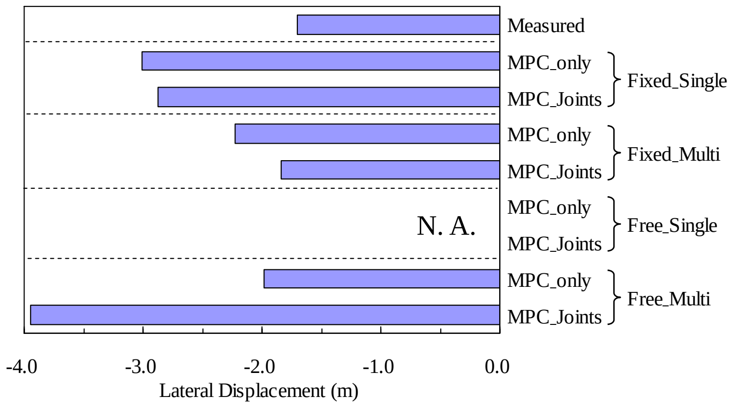

Figure 8 Lateral displacements of the sheet pile head after shaking under saturated condition

4.3 Deformation

Ground deformation of saturated condition after dynamic loading is shown in Fig. 9 and compared to the analytical results under the fixed pile head condition shown in Fig. 10. Simulated shear strain distributions of soil elements are also shown in Fig. 10 as filled contours.

In Fig. 9, the backfill soil deform toward sea due to the liquefaction, which induces the inclination of the sheet pile and the uplift of the seabed. We can see the shear zone like a trapezoidal shape (illustrated by the shaded area in Fig. 9) at each side of the sheet pile. In the single step analysis [Fig. 10 (a) and (b)], the deformation of the backfill soil and the inclination of the sheet pile are overestimated compared to the measurement, while in the multi step analysis [Fig. 10 (c) and (d)], deformation in the model test, particularly the inclination of the sheet pile and the shear zone in the seabed, are properly simulated

CONCLUSION

A two dimensional effective stress analysis (called FLIP) was conducted to simulate the dynamic behavior of a self-supported sheet pile wall in centrifuge experiments. The analysis leads to the following conclusions.

(1) Providing the finite element model with the equivalent restraint condition of the centrifuge model test, appropriate numerical results were obtained. Accuracy of the analysis was improved by conducting multi step analysis and using joint elements employed between double nodes of the seabed and pile nodes.

(2) How to apply gravity force to the finite element model in the self weight analysis can affect not only bending moment profiles after self weight analysis but also amount of displacement and deformation confi-

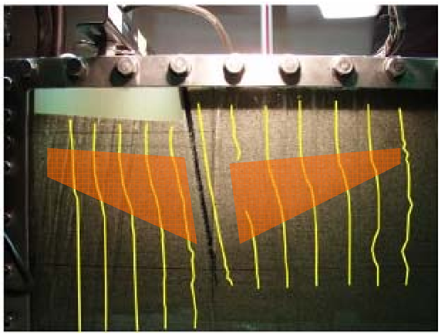

Figure 9 Deformation after shaking in the centrifuge experiment

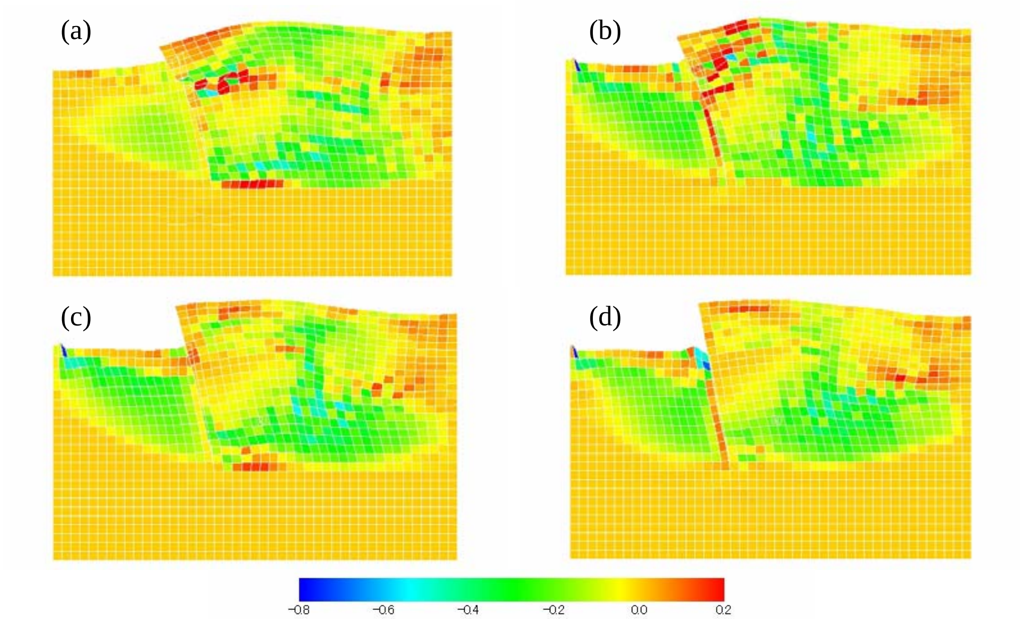

Figure 10 Simulated deformation and shear strain distribution after shaking under the fixed pile head condition:
(a) single step analysis with no joint element (b) single step analysis with joint element
(c) multi step analysis with no joint element (d) multi step analysis with joint element

guration after dynamic loading.

(3) To simulate an existing self-supported sheet pile wall behavior during large earthquakes with accuracy required in practice, it is necessary to obtain in-situ stress states e.g., bending moment profiles, and input them as an initial condition.

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