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EXPERIMENTAL AND NUMERICAL MODELING OF THE LATERAL RESPONSE OF A PILE BURIED IN LIQUEFIED SAND

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SUMMARY

This paper presents experimental and numerical modeling of the lateral resistance of the pile subjected to liquefaction-induced lateral flow. The main objective of this study is to model the behavior of the soil surrounding the pile when a large relative displacement between the pile and the soil is induced.

A series of shaking table tests were undertaken to observe the soil surrounding the pile during liquefaction. The pile was modeled as a buried cylinder that corresponded to a sectional model of a prototype pile at a certain depth in the subsoil. In order to create a realistic stress condition in the model ground, the model was prepared in a sealed container and the overburden pressure was applied to the ground surface by a rubber pressure bag. The model pile was actuated back and forth through rods attached on each side by an electro-hydraulic actuator.

To verify the results gathered in the experimental modeling, a numerical modeling was prepared using a finite element program created exclusively for this purpose.

Test results show that a larger resistance is mobilized as the loading rate becomes higher. When the loading rate is higher, the cylinder displacement required for the lateral resistance becomes smaller. This behavior of the soil surrounding the pile is further verified using numerical modeling.

INTRODUCTION

Past major earthquakes have shown that liquefaction is one of the major geotechnical hazards posed by an earthquake. Piles which are supposedly employed to increase the performance of the foundations at potentially liquefiable sites are found out to be susceptible to liquefaction-induced damages. Damages to piles are due to the reduction of the stiffness of the surrounding liquefied soil as well as due to the lateral spreading of ground induced by soil liquefaction.

The widespread soil liquefaction that happened in Japan during the 1995 Hyogoken-Nambu earthquake (or the Kobe Earthquake) caused tremendous damage to various structures and lifeline facilities built in the reclaimed land along the Kobe coastline. As a result of liquefaction of the backfill materials, several quay walls moved toward the sea. Serious concern to pile foundation was then realized by researchers due to damages occurred to several buildings supported by pile foundations. These buildings settled and tilted without significant damage to the superstructure.

Detailed in-situ investigations were performed on various pile foundations immediately after the earthquake to check damages inflicted to the existing piles, Tokimatsu[1]. Various model tests, both small-scale and large-scale shaking table tests were also undertaken to simulate the effect of liquefaction to the piles, (e.g. Abdoun [2], Satoh[3], Takahashi[4]). Information derived from the shaking table tests is seen to be of value in demonstrating the actual behavior of piles and soils during earthquake. However, the actual behavior of piles is complicated and is affected by several factors. Investigating the effect of each factor from the complicated behavior is not a straight forward process.

This study focuses on observing the deformation of the liquefied soil surrounding the pile when a large relative displacement between the pile and the soil is induced. To easily observe the soil surrounding the pile, it was modeled as a buried cylinder that represents a sectional model of a pile at a certain depth in the subsoil (Fig. 1). Measurements of the drag force of the cylinder embedded in sand were based on the experiment done by Towhata [5]. In order to create a realistic stress condition in the model ground, the model was prepared in a sealed container and the overburden pressure was applied to the ground surface by a rubber pressure bag.

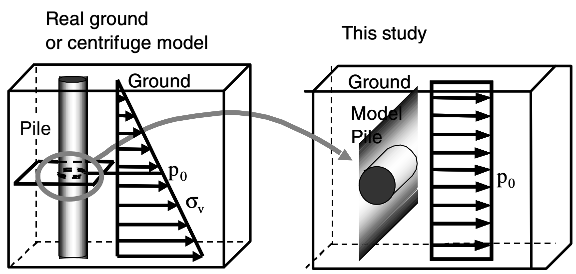


Fig. 1. Modeling of pile in this

The effect of different loading rates on the lateral resistance of the pile in the liquefied sand is also investigated. To verify the results gathered in the experimental modeling, a numerical modeling was prepared using a finite element program created exclusively for this purpose.

TEST PROCEDURES AND CONDITIONS

Test Procedure

The model setup used in this study is schematically illustrated in Fig. 2. An aluminum model container was used with inner sizes of 450 mm in width, 150 mm in breadth, and 250 mm in height. To observe the deformation of the model ground during the test, the front face of the box was made transparent. A rubber pressure bag was attached underneath the top lid of the container to apply an overburden pressure, P_A, on the surface of the soil. A fluid tank was connected at the bottom of the box to supply and drain out fluid and to apply a back pressure, P_B, to the pore fluid of the soil.

Figure 3 shows an aluminum-made-model pile equipped with pore and earth pressure transducers. The surface of the cylinder was smoothly fabricated. Rubber sheets were put on both ends of the cylinder for lubrication and to prevent sand particles from getting into the gap between the cylinder and the side walls of the container. Two rods were connected to the center of the model pile that will serve as a link to the actuator. Two load cells were inserted into the respective rods near the cylinder to avoid the influence of friction in measuring the net lateral force on the cylinder. The cylinder was actuated back and forth through the rods by an electro-hydraulic actuator which is mounted on the side wall of the model container.

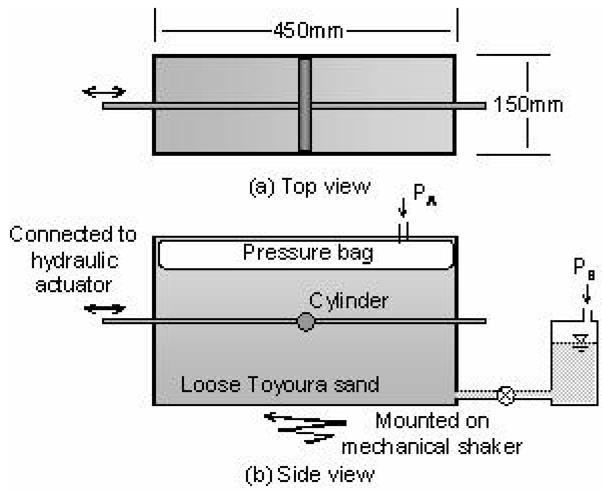


Fig. 2 Schematic Drawing of Model Container

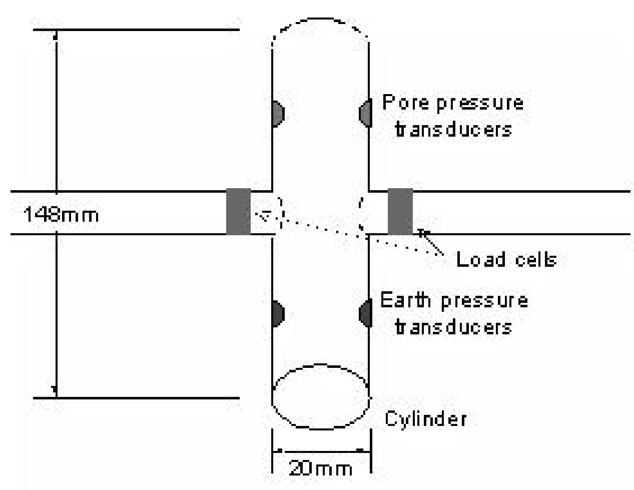


Fig. 3 Schematic Drawing of the Cylinder

The sand used in this study is uniformly graded sub-angular quartz sand (D_{50}=0.19mm), Toyoura sand. To achieve the relative density of 30 to 40%, the model ground was prepared by the air pluviation method. The model ground was saturated up to the ground surface with de-aired water or methyl cellulose solution under a negative pressure of 98kPa in a large tank by applying a vacuum. Japanese noodles “somen” were placed parallel with each other between the model ground and the transparent window to serve as markers to observe the deformation of the ground during the course of the experiment. After the saturation, the top lid of the box was attached and the over burden pressure was applied to the soil under the drained condition.

After the model ground preparation, the container was set on the mechanical shaker (Takemura[6]) and the electro-hydraulic actuator was attached to it. During tests, the horizontal shaking of the container started two seconds prior to the pile loading. This duration was enough to liquefy the model ground. Horizontal shaking was applied thereafter to the container by sinusoidal waves with a frequency of 50Hz and a maximum acceleration of approximately 5g. The period of shaking was 10 seconds. During the test, acceleration of the container, horizontal load and displacement of the cylinder, and earth pressure and pore fluid pressure around the cylinder were measured. The movement of the model pile and the ground were recorded by a digital video camera.

Test Conditions

Different test cases with varying parameters were conducted in this study. In all the cases, the applied over burden pressure was P_A=49kPa. As shown in Table 1, effects of ground vibration and different loading rates on the lateral resistance of the pile were investigated. The loading rate of the cylinder, V, was varied from 1mm/s to 100mm/s. In the cyclic loading tests, the symmetrical triangular waves were applied in order to achieve a constant loading rate. The relative density used in this test condition is 40%.

In cases SW1, SW10, SW100, SM1, SM10 and SM100, a horizontal sinusoidal motion was applied to the container to generate liquefaction in the model. Meanwhile, in cases SW1Q and SW10Q, pore pressure increase, P_B, of 49kPa was applied to the soil after the consolidation of the soil at P_A=49kPa without ground vibration. The effective confining stress of the model became almost zero by increasing the back pressure without ground vibration on the condition that the model was subjected to almost the same stress history that the models in the other cases experienced. This stress condition is defined as artificial soil "liquefaction without ground vibration" in this study. SM10N and SM100N were the tests in which the cylinder was moved back and forth with the initial condition of no liquefaction.

TABLE 1. TEST CONDITION

Case	Pore fluid material	Back pressure P_B (kPa)	Cylinder loading rate V (mm/s)		Shaking
SW1Q	water	49	1	(monotonic)	X
SW10Q	water	49	10	(monotonic)	X
SW1	water	0	1	(monotonic)	O
SW10	water	0	10	(monotonic)	O
SW100	water	0	100	(cyclic)	O
SM1	methyl cel. sol.	0	1	(cyclic)	O
SM10	methyl cel. sol.	0	10	(cyclic)	O
SM100	methyl cel. sol.	0	100	(cyclic)	O
SM10N	methyl cel. sol.	0	10	(cyclic)	X
SM100N	methyl cel. sol.	0	100	(cyclic)	X

At the beginning, de-aired water was used as the pore fluid. However, considering the partial drainage around the cylinder, the migration velocity of water was relatively large, as the diameter of the cylinder was very small compared with the actual pile. In order to consider this problem, in the latter half of the series of the tests, the scaling laws of the centrifuge modeling were adopted, i.e. a higher viscosity fluid (methyl cellulose solution) was used as the pore fluid to avoid conflict with the scaling laws for the time of dynamic events and seepage. The viscosity of the methyl cellulose solution is 50 times higher than that of fresh water. With this similitude rule, measured lateral resistance of the 20mm-diameter cylinder corresponds to the lateral resistance of the 1m-diameter pile at a depth of 5m. Loading rate of 1mm/s corresponds to the situation of the pile in a very slow flow of liquefied soil, while that of 100mm/s corresponds to the vibration of the pile during an earthquake.

NUMERICAL ANALYSIS

To verify the results gathered in the experimental modeling, a finite element analysis program has been developed. This program is designed to analyze soil-structure interaction, especially dynamic events. The finite element program solves the governing equations for dynamic behavior of saturated porous media by Biot [7][8][9] (for details of mathematical formulations, integration scheme along time, sensitivities of parameters for soil model used in this program, refer to Takahashi[10]. This program can calculate initial stress state by self-weight analysis, calculate first eigen value of the system by modal analysis, evaluate dynamic responses of soils and structures, evaluate soil-structure interactions in static events, and pick up relevant data from output file.

In order to numerically simulate the model test, two dimensional finite element analyses were conducted under the plain strain condition. Geomaterial parameters used in the analysis is based on the extended subloading surface model proposed by Hashiguchi [11], [12] since the loading criterion is simple and the model has the capability to describe realistic strain accumulation behavior during a cyclic loading though many material parameters are needed. Details of the constitutive model are described in Takahashi [10]. The pile was modeled as linear elastic element with Young's modulus of 1.0×10^9 N/m^2, Poisson's ratio of 0.33, and density of 2.69Mg/m^3). The rubber pressure bag on top of the model ground that will provide overburden pressure was also modeled as linear elastic element with Young's modulus of 1.0 N/m^2, Poisson's ratio of 0.33, and density of 500 Mg/m^3).

The size of the element was 10mm x 10mm. Fluid flow velocities were set to zero at all the boundaries except at the surface of the ground. In order to take into account the fact that the horizontal movement of the soil was not allowed at the side boundaries of the model container, the horizontal displacements of the nodes at the side boundaries are fixed. In general, as the waves should appropriately be permitted to transmit from/to analysis domain to/from the outside of the analysis domain in-situ, appropriate boundary conditions other than the fixed condition should be considered.

The applied earthquake motion was similar to the wave utilized in the shaking table test. In order to obtain a numerical solution, the differential equations are integrated in time. The integration scheme used in time was Newmark's β method, and the time step for the integration was Δt=0.0005sec. System damping was represented by Rayleigh damping, and the damping ratio was 1% in a first mode of free vibration of the system.

In this analysis, two stages of calculations had been undertaken. Dynamic response analysis was first undertaken to simulate the condition of the model ground during a seismic event and when the model ground reached the point of liquefaction, shaking is terminated and the second stage of calculation, which is the static analysis followed immediately. In the static response analysis, pile loading is induced to simulate the movement of the prototype pile immediately after the liquefaction.

EXPERIMENTAL AND NUMERICAL RESULTS AND DISCUSSIONS

Ground Deformation

The effect of ground vibration on the lateral resistance of the pile and the deformation of the soil surrounding the pile is evaluated by analyzing the results of cases SW1Q, SW10Q, SW1, and SW10. As have been mentioned in the preceding section, ground vibration has not been induced on SW1Q and SW10Q. To induce liquefaction on these cases, a back pressure (P_B = 49kPa) which is equivalent to the overburden pressure was applied instead.

As we can observe in Figure 4, the cases with artificial liquefaction (SW1Q and SW10Q) are remarkably larger than those with ground vibration (SW1 and SW10).

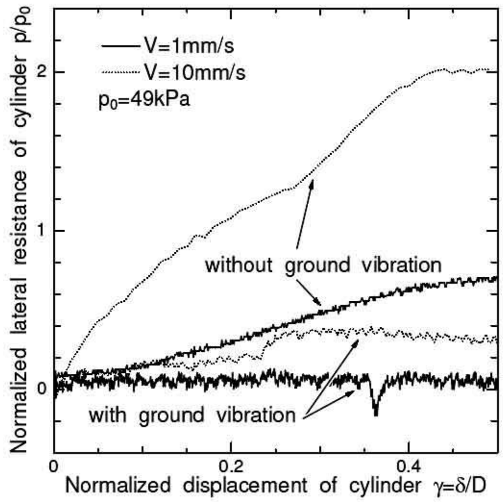
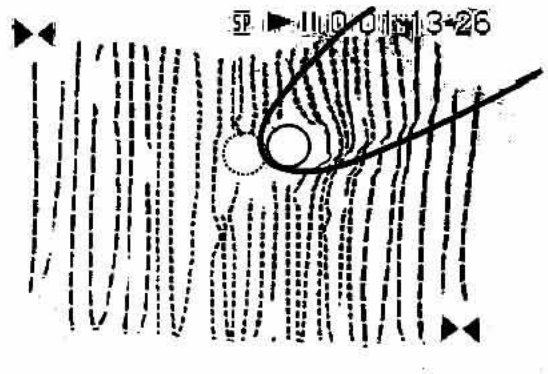
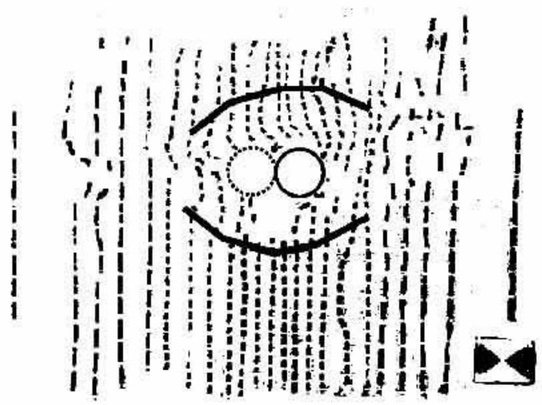


Figure 4. Normalized Lateral Resistance Against Normalized Lateral Displacement of Pile

The movement of the pile section with respect to the movement of the ground was also observed during the experiment by means of the Japanese noodles attached between the clear glass wall and the model ground. Figure 5 shows some of the pictures of the pile behavior on the liquefied ground. It can be observed in the picture without shaking, that a large amount of soil in front of the pile (right side of the pile) have moved forward resulting in the upheaval of the ground surface of the front side. Application of the "artificial liquefaction" or "no shaking" can not be simulated in the numerical analysis; therefore, no graphs can be compared with this result. On the other hand, in the case where ground vibration is applied, the deformation of the liquefied soil is quite limited adjacent side to the pile. Figure 6 shows the numerical output of the movement of the pile with respect to the movement of the liquefied ground. It is clearly depicted in this figure that the movement of the pile with respect to the liquefied ground is similar than that of the experiment.



(a) Without ground vibration (SW10Q)



(b) With ground vibration (SW10)

Fig. 5 Deformation of the Pile with Respect to the Liquefied Ground for the Cases with and without Ground Vibration (Experimental Result)

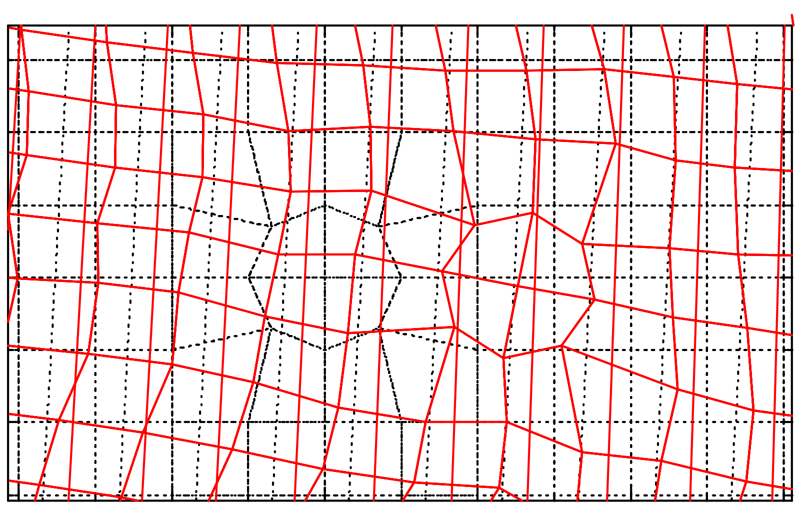


Fig. 6 Deformation of the Pile with Respect to the Liquefied Ground (Numerical Result)

The difference in the area influenced may directly affect the lateral resistance of the pile shown in Figure 4. The vibration of the ground may have caused instability in the soil particles contact, and thus, reduced the resistance of the surrounding soil against the movement of the pile.

Loading Rate Effects

Figure 10 shows the time histories of the lateral resistance, the displacement of the pile and the excess pore pressure around the pile for cases SM1 and SM10. If we look closely at the second to the last quarter of the first loading cycle, for both cases (SM1 and SM10), the pore pressure on the side of the movement direction (the dotted line) slightly increased by the sand contraction, then it showed rapid decrease due to sand dilation and suction force at the back of the pile, while the pressure at the back side (the solid line) monotonically decreased due to the suction force. The pore pressure decrease on both sides (movement direction and back side) of the pile was also observed in the numerical analysis output shown in Figure 11 and Figure 12 for both 1 mm/sec and 10 mm/sec loading rates, respectively. As a result, the pore pressure decreased on both sides when the maximum displacement of the pile was imposed though the pressure on the side of the direction of movement is larger than on the other side in both cases.

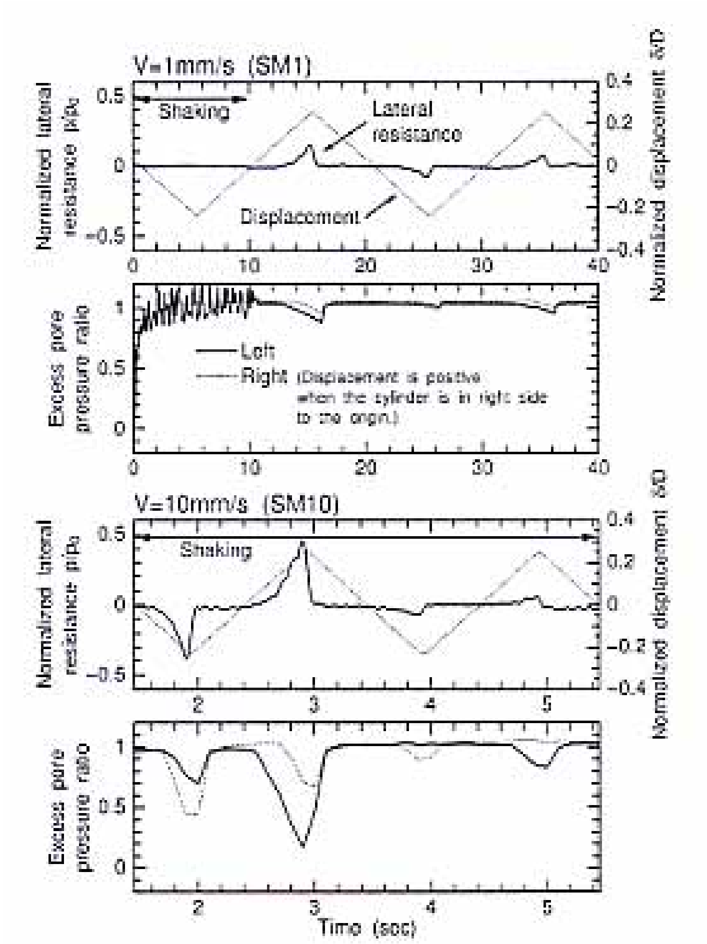


Figure 10 Time Histories of Lateral Resistance and Displacement of Pile and Excess Pore Pressure Around the Pile (SM1 and SM10)

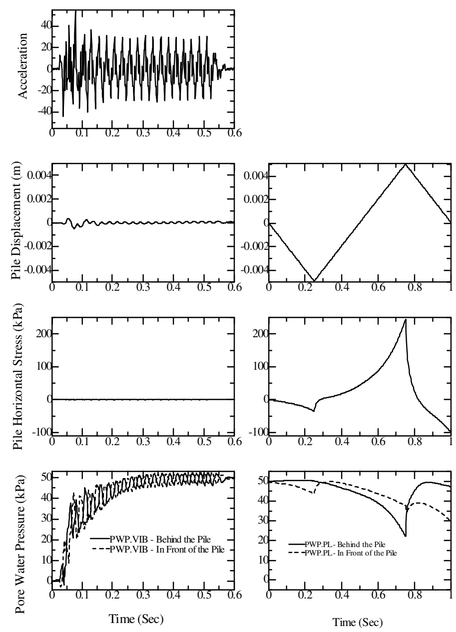


Figure 11. Numerical Analysis of Time Histories of Lateral Resistance and Displacement of Pile and Excess Pore Pressure around the Pile (Pile Loading Rate = 1 mm/sec)

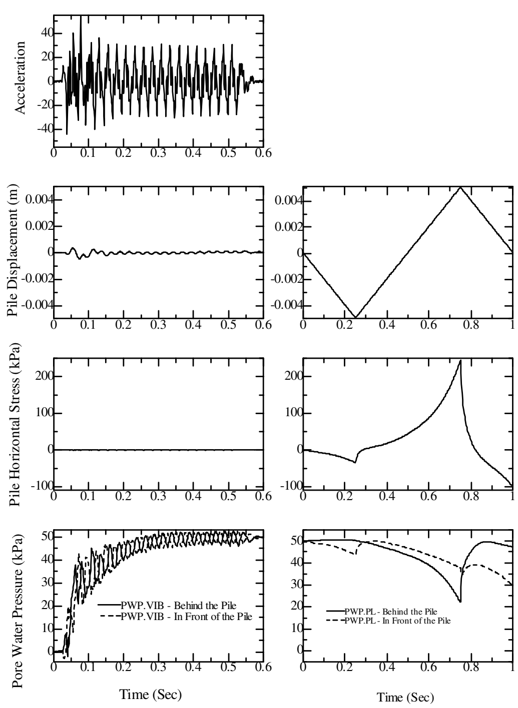


Figure 12. Numerical Analysis of Time Histories of Lateral Resistance and Displacement of Pile and Excess Pore Pressure around the Pile (Pile Loading Rate = 10 mm/sec)

CONCLUSIONS

Lateral loading tests on buried cylinder were conducted to study the lateral resistance of a pile in liquefied soil. The study focused on the observation of the deformation of the liquefied soil surrounding the pile when a large relative displacement is induced between the pile and the soil. Lateral resistance of the pile in liquefying sand is directly measured using a pile loading system apparatus that has the capability of applying horizontal cyclic vibrations to the pile during shaking and allowing the observation of the liquefying sand deformation. The loading rate effect on the lateral resistance of the pile in the liquefied soil was also investigated.

A numerical analysis has been undertaken using a finite element program created solely for this purpose to verify the results derived from the experiment. The following conclusions were obtained in this study:

- The deformation of the soil surrounding the pile has been successfully observed by video camera through the transparent window of the box. Without ground vibration, a large amount of the soil in front of the model pile move forward causing the upheaval of the ground in the surface. With ground vibration, both in the experimental and numerical analysis show that the deformation of the soil was quite limited in the vicinity of the model pile when the shaking was applied. The difference in the deformation mode of the soil directly affected the lateral resistance of the model pile.

- A larger lateral resistance is mobilized as the loading rate becomes higher. Furthermore, when the loading rate is higher, the pile displacement required for the lateral resistance mobilization becomes smaller. These tendencies are associated with not only the dilatancy characteristics of sand but also pore fluid migration around the cylinder.

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