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Dynamic Behavior of Group Pile Under Lateral Spreading

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Synopsis

Results of a series of centrifuge experiments to study the dynamic response of pile foundations under lateral spreading were compared to the results of numerical analysis. Experiments were carried out under the centrifugal acceleration of 40 G. Piles in the model foundation were lined up 3 by 3 pattern with a spacing of three pile diameter, and both pile head and bottom were rotation fixed. Piles were placed in the inclined ground of saturated sands and applied lateral loads due to the ground deformation. The effective stress finite element analysis was conducted to simulate these experiments. Computed time histories of pile head acceleration and displacement were consistent with those obtained from experiments. However, much smaller surface ground deformation in simulation might cause small amplitude of bending moments. Some calibration in the numerical modeling may be required to have more consistent results on bending moments.

Keywords: Pile foundation, Centrifuge experiments, Numerical simulation, Liquefaction

1. Introduction

Many waterfront structures are built on deep foundations which are vulnerable to lateral loads especially due to the ground deformation during and after large earthquakes. Liquefaction-induced lateral spreading is one of the major causes of that ground deformation. (e.g., Mizuno 1987; Matsui and Oda 1996; Hamada and O'Rourke 1992; O'Rourke and Hamada 1992; Tokimatsu and Asaka 1998). Pile foundations not only support the inertial loads of the superstructures but also suffer the lateral loads due to the ground deformation. Therefore, the dynamic behavior of soil-pile system have been intensively studied for the last decade. Dynamic behavior of pile foundations is highly nonlinear and influenced by numbers of parameters, such as material of a pile and soil, pile diameter and spacing, natural period of superstructures and soil-pile system. To study such a complicated phenomena, full scale experiment, although cases are limited, have been conducted (e.g., Brown et al. 1988; Peterson and Rollins 1996; Ashford and Rollins 2002). Brown et al. (1987) conducted large-scale tests for group pile subjected lateral load and proposed the p-multiplier concept to reshape the *p-y* curve of single pile to take into account the group effect. Rollins et al. (2005ab) conducted lateral loading tests with a full-scale pile group under blast-induced liquefaction and developed p-y curve for liquefied ground.

Instead of carrying out those full scale experiments, numerous small scale model tests have been employed using shaking tables or the geotechnical centrifuge (e.g. Tobita et al. 2004). Abdoun et al. (2003), for example, conducted centrifuge experiments with a slightly inclined laminar box to study pile response under lateral spreads of 0.7 to 0.9 m. One of their results is consistent with the observation that the maximum permanent bending moments occurred at the boundaries between liquefied and non-liquefied ground.

Numerical simulation is also widely used to simulate the dynamic response of pile foundations (e.g. Reese et al. 1996; Kitade et al. 2004). Most of two dimensional model uses p-y curve concept, or soil-pile interaction spring, that is required to simulate the three dimensional effects, such as slippage of soils near a pile surface. Therefore, it is important to properly determine the soil-pile interaction spring for a reasonable estimate of pile response. In the present study, the interaction spring obtained by the method proposed by Ozutsumi et al. (2003) is used to simulate those afore mentioned three dimensional effects observed in centrifuge experiments, and that is of prime objective of this paper.

2. Centrifuge modeling for group pile behavior under lateral spreading

Experiments were carried out with the geotechnical centrifuge at the Disaster Prevention Research Institute, Kyoto University (DPRI-KU). The centrifuge with rotation radius of 2.5 m has dual swing platforms at both ends of arms. The maximum capacity is 24 G-tons with a maximum centrifugal acceleration of 200 G. A shake table unidirectionally driven by a servo hydraulic actuator is attached to a platform and it is controlled through a personal computer (PC) on the centrifuge arm. All the equipment necessary for shake table control is put together on the arm. The PC is accessible during flight from a PC in the control room through wireless LAN and

Table 1 Soma-Silica sand No. 5.

e_{max}	e_{min}	D_{50}(mm)	U_c
1.105	0.685	0.38	1.5

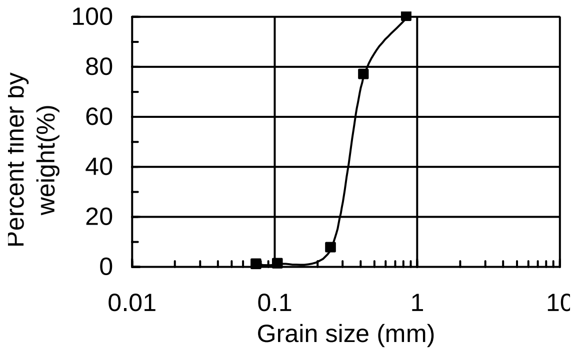

Fig. 1 Particle size distribution curve for Soma-Silica sand No.5.

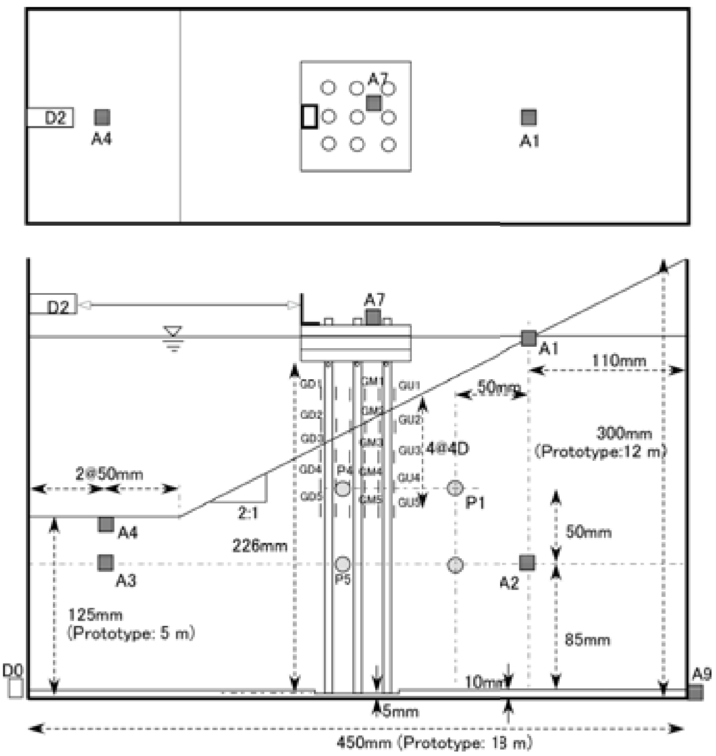

Fig. 2 Cross section of centrifuge model.

"Remote Desktop Environment" of WindowsXP (Microsoft, 2003). Capacity of the shake table is 15 kN, 10G and ±5 mm in maximum force, acceleration and displacement, respectively. Base excitation was given to a rigid soil box.

The uniform model ground was made of Soma-Silica sand No. 5 having the physical properties shown in Table 1 and the particle size distribution curve shown in Fig. 1. To study the behavior of pile foundations under lateral spreading, the model ground surface was inclined with the target inclination of 26.6° to the horizontal (vertical/horizontal=1/2) as shown in Fig. 2 and Photo 1, thus simulating rather steep slope which might be encountered in port facilities. The ground had flat part in downstream to simulate the sea floor. Sands were slowly sprinkled over the soil box filled with a viscous fluid. The target relative density of the ground was set to be 40 %. However, slightly lower values were obtained for all the cases as shown in Table 3. During the preparation of inclined ground, some fraction of surface soils were slowly moved downward due to the gravitational force, that might caused a development of the loose surface deposit.

A viscous fluid made of Metolose (Type: SM-25 Shin-Etsu Chemical Co.) was used to properly simulate pore water pressure dissipation during and after shaking. Metolose is water-soluble cellulose made of organic material. After preparing in the room temperature, the fluid was tested using a viscometer to achieve the specified viscosity (40cSt for 40g centrifugal acceleration) before pouring sands. There might be slight difference between the room temperature and the temperature in the centrifuge pit during flight but the effects of the difference was considered negligible because the duration of flight is about 20 to 30 min before the dynamic loads were applied.

A pile with a 7 mm diameter brass tube simulated a prototype diameter pile of 0.28 m at 40 G. Other dimensions and mechanical properties of a model pile are shown in Table 2. As shown in Fig. 2, group piles were lined up 3 by 3 with a spacing of 3 times a pile diameter. Both the pile top and bottom were set in rotation fixed condition. To achieve the fixity condition at the pile head, each pile in group was squeezed into a hole on an aluminum plate of 2 mm thickness, then a mass of 2.7 kg was placed on top and tied up with the plate. The weight of mass was determined so that the natural period of soil-pile system becomes about one

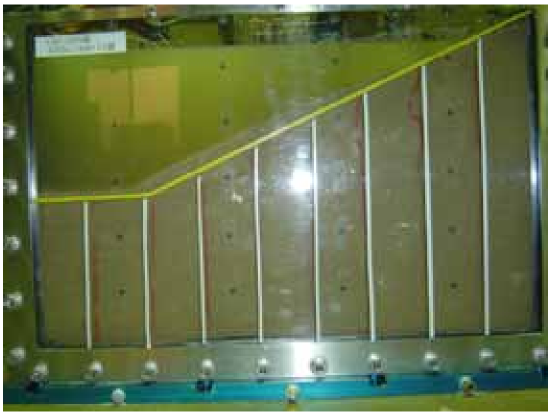

Photo 1 Side view of the model ground.

Table 2 Model pile properties.

	Model	Prototype	Uni
Length	0.25	10	m
Outer diamter (D)	7	280	mn
Wall thickness	0.9	36	mn
Young's modulus (E)	101	101	Gp.
Moment of inertia of area (I)	82	2.1×10^{8}	mr
Bending stiffness (EI)	8.2	2.1×10^{8}	MN-n

Table 3 Experimental condition.

	Input	Dr (%)
CS1	Sin	35.4
CS2	Near Field	36.0
CS3	Plate Boundary	35.5
CS8	Plate Boundary	40.5

second. The bottom of each pile was plugged into a hole on a bottom plate.

In a series of model tests, 6 accelerations, 2 displacements and 15 strain gage readings of a pile, and 4 pore water pressures were measured as shown in Fig. 2. A laser displacement sensor was used to measure pile head displacement. Accelerometers of strain gage type and pore pressure transducers of semi-conductor type were used. Total four cases are presented in this study as listed in Table 3, CS1: sinusoidal input with 40 Hz, CS2: synthesized near-field earthquake, CS3: synthesized plate boundary earthquake, and CS8: duplicated test of CS3. Input accelerations measured at the base of soil box are shown in Fig. 3. In what follows, units are all in prototype, unless otherwise noticed.

2.1 Ground Deformation After Shaking

Tracing the shape of the ground surface before and after shaking, and vertical markers attached on the inside of soil box, the ground deformation pattern is drawn as

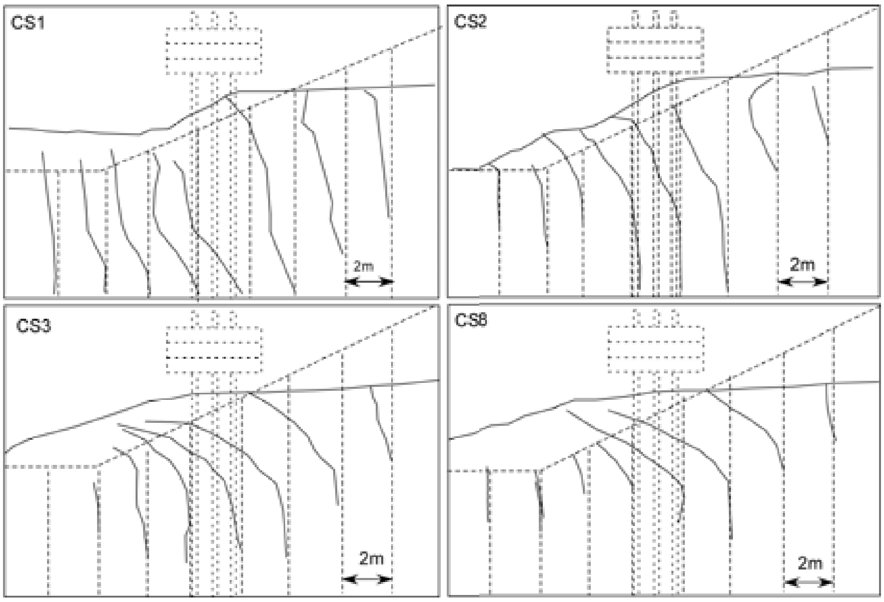

Fig. 4 Ground deformation after shaking.

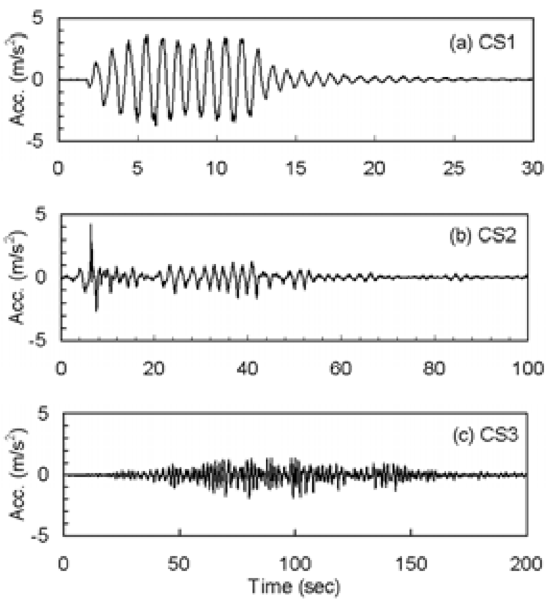

Fig. 3 Input acceleration in centrifuge experiments.

shown in Fig. 4. Large ground deformation of 2 to 4 m was occurred after shaking. It is clear that duration of shaking, i.e., 10, 60, and 100 seconds for CS1, CS2 and CS3/CS8, and the magnitude of surface displacement proportionally increase, and as duration becomes longer the ground surface tends to become flat. For CS1, compared with other cases the ground at deeper depth was mobilized and the surface displacement is about 3 m. While for CS2 and CS3/CS8, the surface displacements are, respectively, about 2 and 6 m, and the ground at shallower depth was mobilized. The pattern of the ground deformation of CS8 is consistent with the one of CS3.

2.2 Pile Head Acceleration, Pile Head Displacement and Excess Pore Water Pressure

Results of a series of centrifuge experiments, CS1, CS2, and CS3, are shown in Fig. 5. Piles had no permanent deformation after experiments and therefore were within an elastic range. Initial vertical effective stress drawn in Fig. 5 is computed based on the vertical depth of the sensors (P1) from the ground surface. Comments for each case are as follows:

- CS1: Compared with the input acceleration shown in Fig. 3, the peak of measured pile head acceleration in Fig. 5 seems to be reduced with the development of excess pore water pressure. The pile head is shifted downward about 400 mm after about 5 seconds and the residual displacement of 300 mm is measured.
- CS2: Compared with the input acceleration shown in Fig. 3 (b), pile head acceleration is amplified about 100 %, even after the soil is liquefied. The pile head is moved downward with the maximum displacement of 600mm, and the residual displacement of about 80 mm is observed.
- CS3: The pile head acceleration is amplified similarly to CS2. Maximum displacement is 600 mm in downstream, and residual displacement is about 80 mm. Compared to the ground deformation of about 6 m as shown in Fig. 4, the residual displacement is small. This is because liquefied soil is soft enough to pass through the space between the piles.

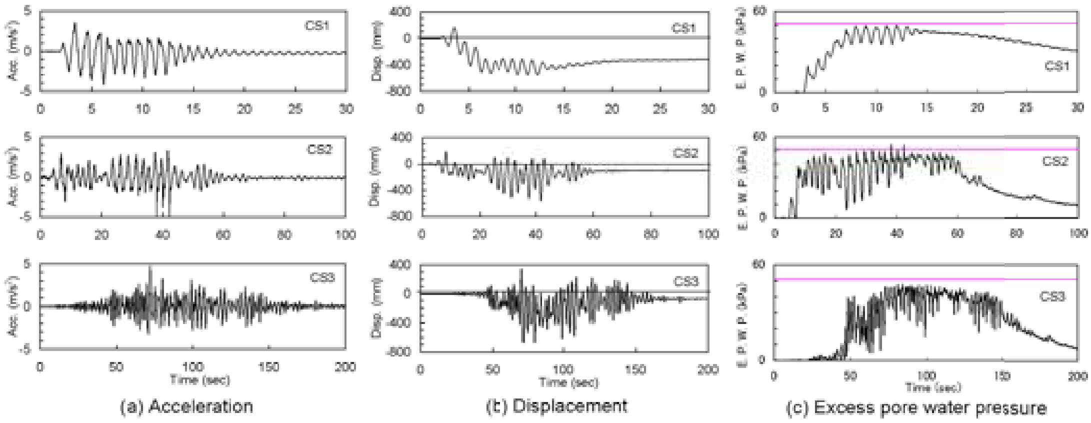

Fig. 5 Time histories of pile head acceleration (a) and pile head displacement (b) and excess pore water pressure (c).

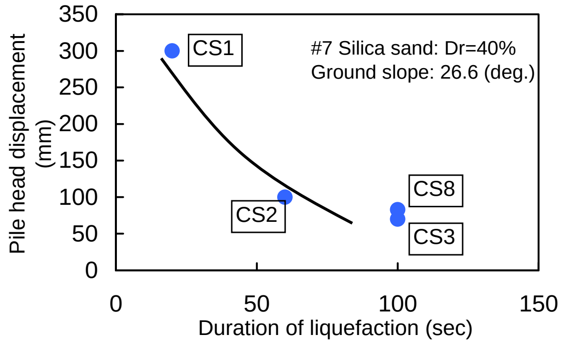

Fig. 6 Duration of shaking and pile head displacement

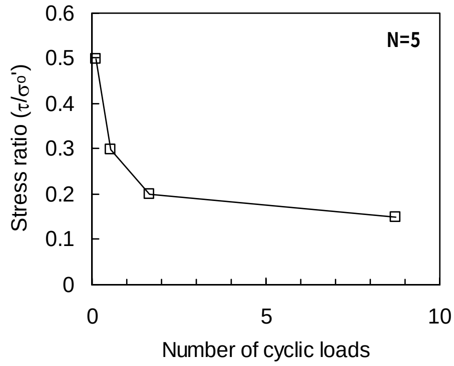

Fig. 7 Liquefaction strength curve.

The relation between the duration of liquefaction and the pile head displacement obtained from a series of the experiments is summarized in Fig. 6. It is clearly shown that the residual pile head displacement becomes smaller as duration of liquefaction becomes longer, although, as mentioned earlier, the ground deformation contrary becomes larger. This is because liquefied soils are soft enough to pass through the space between piles. From the residual displacement of CS2 and CS3, the resistance force of pile and lateral force of the deformed ground may be in equilibrium at the pile head displacement of 80 mm.

3. Numerical simulation

Two dimensional effective stress finite element analysis, FLIP (Iai et al. 1992), is employed. Research is still on going and therefore results of CS2 only are presented. Physical properties of soil corresponding to the SPT value of 5 were assumed because of the lack of experimental results of the soil used in the centrifuge tests. Soil is modeled as having the multi-shear mechanism and piles are modeled with elastic beam elements. No joint elements but soil-pile interaction springs are implemented between soil and pile elements. The liquefaction strength curve in simulation is shown in Fig. 7. The meshes before and after the shaking are shown in Fig. 8. Compared to the one shown in Fig. 2, the pattern of ground deformation is similar but the magnitude of deformation is computed small. Pile head acceleration and displacement are compared in Fig. 9. The shape and amplitudes of both acceleration and displacement are consistent with the results of the experiment, except high spikes on the measured acceleration, and the amplitude of displacement which is computed smaller than the measured when piles moved downward.

Time histories of excess pore water pressure of P1 and P2 shown in Fig. 10 agree with experiments, however, for those of P4 and P5, simulation gives build up of pressure about one to two seconds earlier. Simulated pore pressure adjacent to piles may depend strongly on the response of piles. The elements to plot P4 and P5 are adjacent to the pile and therefore they may be subject to

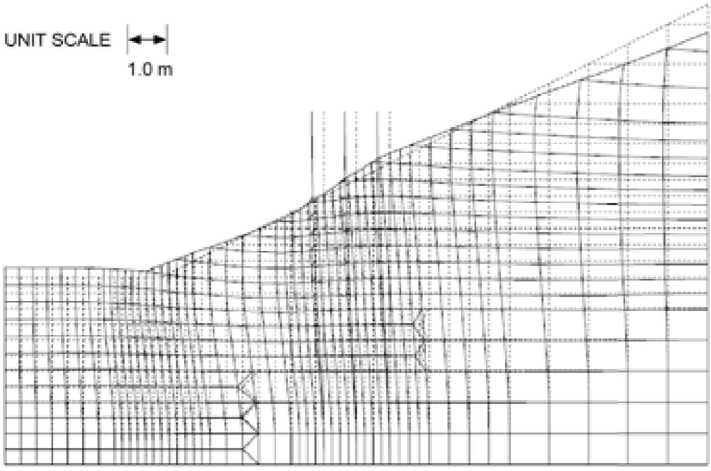

Fig. 8 Mesh and deformation after shaking: CS2

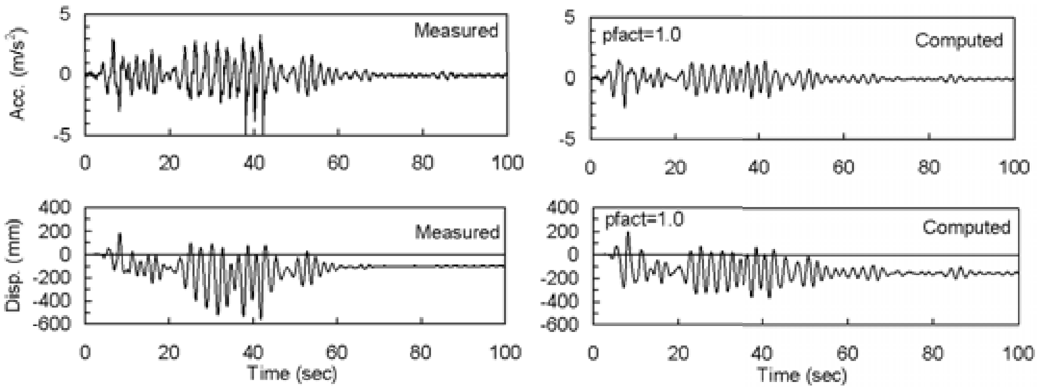

Fig. 9 Comparison of pile head acceleration and displacement: CS2.

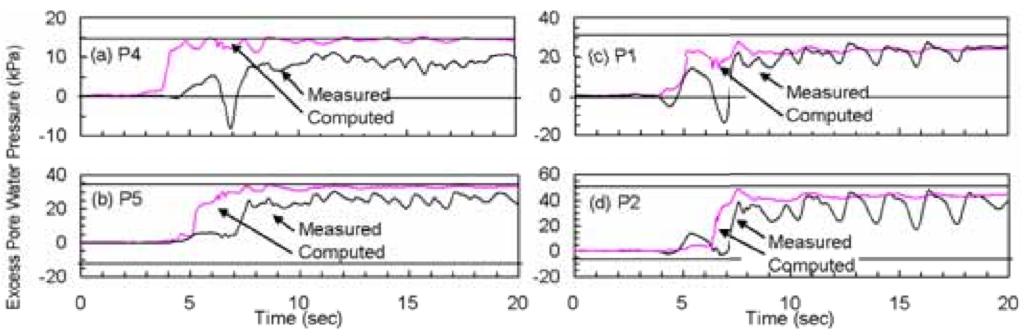

Fig. 10 Comparison of excess pore water pressure: CS2.

large shear compared to the elements of P1 and P4 surrounded by soil elements.

Time histories of bending moment for the downstream pile (GD1, GD3, and GD5) are plotted in Fig. 11. Computed bending moments from GD1 and GD3 are consistent with measured in terms of their amplitude and phase. As shown in Fig. 2, however, those are located above the ground surface. For the bending moments located under ground, GD5, computation gives smaller amplitude after 8 seconds than the ones measured in the experiment, This may be attributed to the small ground deformation in simulation, and require some calibration in the numerical model. Bending moments in depth is plotted in Fig. 12 for the time of 5.6 sec (a) and 41.8 sec (b). In Fig. 12(a) at t=5.6 sec, the trend of the curve is similar each other, however, at time 41.8 sec shown in Fig. 12(b) piles above the ground surface were mostly bent in computation.

4. Conclusions

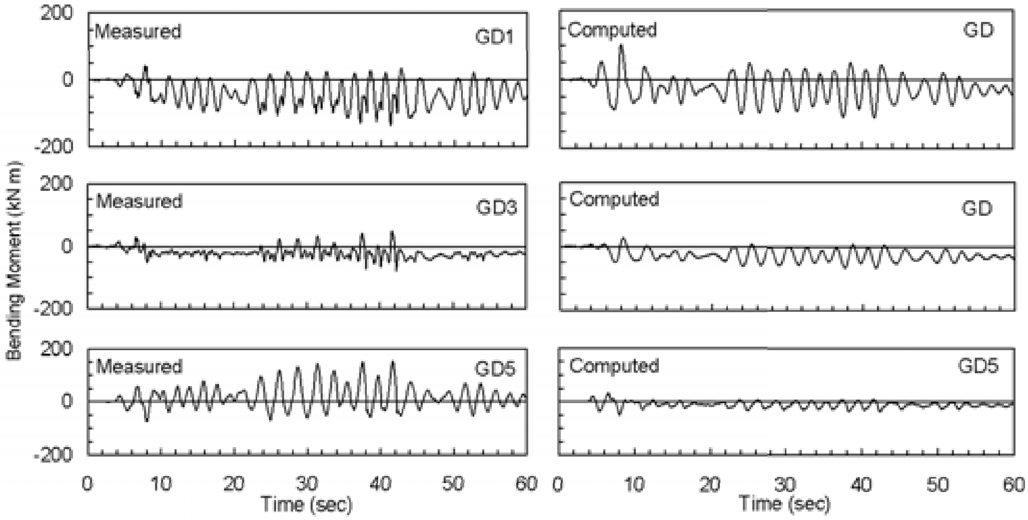

Fig. 11 Comparison of time history of bending moment of downstream pile: CS2.

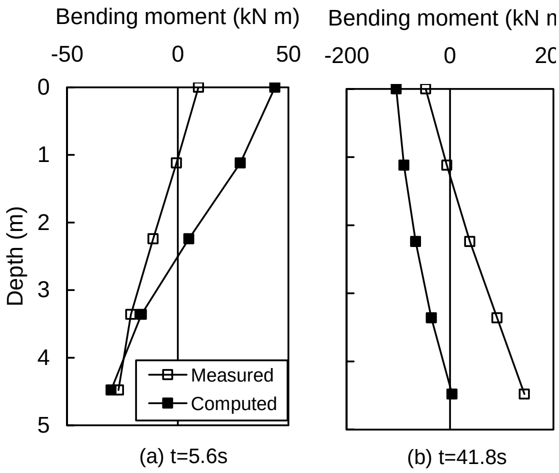

Fig. 12 Comparison of bending moment in depth: CS2.

To study the dynamic response of pile foundations under liquefaction-induced lateral spreading, centrifuge experiments are conducted with inclined ground surface. Experimental results show that the duration of liquefaction is inversely proportional to the residual pile head displacements because liquefied soils are soft enough to flow between piles and give less lateral force and enough time for piles to be unloaded. This may possibly be observed in pile supported waterfront structures. However, the piles have to survive large lateral force that gives the maximum pile head displacement.

Numerical analysis based on the effective stress analysis, FLIP, properly simulated pile head acceleration, pile head displacement, and bending moment above the ground surface with a reasonable degree of accuracy. However, the ground deformation and the bending moments of piles under ground were simulated to be small compared to the one in the experiments. Further calibration is required numerical model.

References

Abdoun, T., Dobry, R., O'Rourke, T.D., and Goh, S.H. (2003): Pile response to lateral spreads: Centrifuge modeling, Journal of Geotechnical and Environmental Engineering, , ACSE, Vol. 129, No. 10, pp. 869-878.

Ashford, S.A., and Rollins, K. M. (2002): TILT: The Treasure Island liquefaction test, Final Report No. SSRP 2001/17, Department of Structural Engineering, University of California, San Diego.

Brown, D.A., Reese, L.C., and O'Neill, M.W. (1987): Cyclic lateral loading of a large-scale pile group, Journal of Geotechnical Engineering, ASCE, Vol. 113 No. 11, pp. 1326-1343.

Brown, D.A., Morrison, C., and Reese, L.C. (1988): Lateral load behavior of pile group in sand, Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 11, pp. 1261-1276.

Hamada, M., and O'Rourke, T.D. (1992): Case histories of liquefaction and lifeline performance during past earthquakes, Technical Report NCEER-92-0001, National Center for Earthquake Engineering Research, State University of New York, Buffalo, 2 vols.

Iai, S., Matsunaga, Y., and Kameoka, T. (1992): Strain space plasticity model for cyclic mobility, Soils and Foundations, Japanese Society of Soil Mechanics and

Foundation Engineering, Vol. 32, No. 2, pp. 1- 15.
Kitade, T., Kawamata, K., Ichii, K., and Iai, S. (2004): Analysis of laterally loaded pile groups using 2-D FEM, The 11th International Conference on Soil Dynamics & Earthquake Engineering, The 3rd International Conference on Earthquake Geotechnical Engineering, pp. 850-856.
Matsui, T., and Oda, K. (1996): Foundation damage of structures, Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17 1995 Hyogoken-Nambu Earthquake, pp. 189-200.
Microsoft. (2003): Windows XP Professional, Microsoft Co., One Microsoft Way, Redmond, WA 98052-6399, USA.
Mizuno, H. (1987): Pile damage during earthquake in Japan (1923-1983), Proc. Session on Dynamic Response of Pile Foundations, T. Nogami ed., ASCE, New York, pp. 55-77.
O'Rourke, T.D., and Hamada, M. (1992): Case histories of liquefaction and lifeline performance during past earthquakes, Technical Report NCEER-92-0001, National Center for Earthquake Engineering Research, State University of New York, Buffalo, 2 vols.
Ozutsumi, O., Tamari, Y., Oka, Y., Ichii, K., Iai, S., and Umeki, Y. (2003): Modeling of soil-pile interaction subjected to soil liquefaction in plane strain analysis, Proceedings of the 38th Japan national conference on geotechnical engineering, Akita, Japan, pp. 1899-1990.
Peterson, K.T., and Rollins, K.M. (1996): Static and dynamic lateral load testing of a full-scale pile group in clay, Civil Engineering Department Research Report, CEG. 96-02, Brigham Young University, Provo, Utah.
Reese, L.C., Wang, S.T., Arrellaga, J.A., and Hendrix, J. (1996): Computer program GROUP for windows, User's Manual, Version 4.0, Ensoft, Inc., Austin, Texas.
Rollins, K.M., Gerber, T.M., Lane, D.J., and Ashford, S.A. (2005a): Lateral resistance of a full-scale pile group in liquefied sand, Journal of Geotechnical and Environmental Engineering, ASCE, Vol. 131, No. 1, pp. 115-125.
Rollins, K.M., Lane, D.J., and Gerber, T.M. (2005b): Measured and computed lateral response of a pile group in sand, Journal of Geotechnical and Environmental Engineering, ASCE, Vol. 131, No. 1, pp. 103-114.
Tobita, T., Iai, S., and Rollins, K.M. (2004): Group pile behavior under lateral loading in centrifuge model tests, International Journal of Physical Modelling in Geotechnics, Vol. 4 , No. 4, pp. 1-11.
Tokimatsu, K., and Asaka, Y. (1998): Effects of liquefaction-induced ground displacements on pile performance in the 1995 Hyogoken-Nambu earthquake, Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17 1995 Hyogoken-Nambu Earthquake, pp. 163-177.

側方流動を受ける群杭基礎の動的挙動

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要旨

液状化による側方流動地盤中の杭基礎の挙動について、遠心模型実験および数値解析により考察した。模型実験は40分の1の縮尺で、用いた杭の配置は3x3の9本群杭、杭間隔は杭径の3倍である。杭の上下端は固定とした。杭基礎を配置した飽和地盤表面に傾斜30度を設け、地震波を入力し液状化による流動を発生させた。実験より、流動初期には杭頭の変位は地表面の変位に追随しているが、最終的な杭頭の変位は地表面変位よりも小さくなった。これは杭の剛性が高く杭間を土がすり抜けるためである。この実験結果より2次元有効応力解析法の妥当性について検討したところ、地盤ばねを適切に評価することで、すり抜けの効果を再現することができた。

キーワード:杭基礎，遠心模型実験，数値解析，液状化