Shaking Table Test on Superstructure-foundation-Ground System in Liquefiable Soil and Its Numerical Verification

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ABSTRACT: In this paper, the effect of seismic enhancement on group-pile foundation with partial ground improvement method is investigated with shaking table tests and soil-water coupling dynamic elastoplastic finite element analyses. Model tests on a superstructure-group pile foundation-ground system were conducted with the shaking table test device in which the liquefaction of sandy ground is considered. The model pile is made from aluminium and the model ground is made from Toyoura Sand. The model ground is carefully prepared with in-water sedimentary method. Different patterns of the partial ground improvement for an existed group-pile foundation are investigated in the tests and numerical analyses. In the analyses, nonlinear behavior of the ground and the piles are described by the cyclic mobility model (Zhang et al, 2007&2010) and the axial force dependent (AFD) model proposed by Zhang and Kimura (2002) that can take into consideration the axial-force dependency in the nonlinear moment-curvature relation. The applicability of the numerical analysis using the program named as DBLEAVES (Ye et al, 2007) has already been confirmed in previous researches that can be referred to relevant references. It is found that the effectiveness of the partial ground improvement method has been proved by both the shaking table tests and the numerical analyses.

1. INTRODUCTION

In order to clarify the mechanical behaviors of pile foundations at ultimate state during strong earthquake, many researches about the tests on group-pile foundation subjected to earthquake loading have been conducted. For instance, Tokimatsu et al. (2007) conducted a shaking table test using E-defense, one of the largest shaking table test device in the world, to estimate the effects of dynamic interaction among soils, pile foundation and superstructure. Shirato et al (2008) also conducted a large-scale shake table experiment on the nonlinear behavior of pile-groups subjected to lateral loading during huge earthquake. Motamed et al (2009) conducted a shaking table test using the E-Defense to investigate the behavior of pile group behind a sheet pile quay wall subjected to liquefaction-induced large ground deformation. Shaking table tests using centrifuge machine were also frequently conducted and many results have been published in literature such as the works by Uno et al (2011), Ishizaki et al (2011) and Tazoh et al (2011).

It is known that during a strong earthquake, the dynamic behavior of a group-pile foundation is related not only to the inertial force coming from superstructures but also to the deformation of surrounding ground. Therefore, in seismic evaluation of group pile foundation, it is necessary to properly describe the nonlinear behavior of the group-pile foundations, the superstructures and the ground simultaneously during a major earthquake, especially in the case when the ground is a liquefiable soil.

In this paper, the efficiency of seismic enhancement by the partial-ground improvement method, firstly proposed by Adachi (2009), is investigated by 1G shaking table tests for different patterns. On the other hand, as has been pointed out in the works by Jin et al. (2010) and Bao et al. (2012), that numerical simulation also plays a very important role in clarifying the behavior of superstructure-pile foundation-ground system, the verification using numerical analyses for the partial-ground improvement method is also conducted. Ye (2007) developed a three-dimensional (3D) static and dynamic finite element method (FEM) with the code named as DBLEAVES based on finite deformation scheme. The applicability of the proposed numerical method has been confirmed in many case studies such as the works by Jin et al. (2010) and Bao et al. (2012a, b). In this paper, the numerical analyses, using DBLEAVES, is conducted to verify the effectiveness of the partial-ground improvement method, particular attention is paid to the strong nonlinear behavior of the liquefied sandy ground and the interaction among the superstructure, the group-pile foundation and the ground. Because the shaking table tests are conducted in liquefiable sandy ground and a strong nonlinear behavior of the ground is expected, the nonlinear behavior of the ground is described by the cyclic mobility model (Zhang et al, 2007&2010) and the piles is described by the AFD model (Zhang and Kimura, 2002).

2. SHAKING TABLE TEST ON A SUPERSTRUCTURE-PILE FOUNDATION-GROUND SYSTEM

The partial-ground-improvement method using cement-treated soil around an existing group-pile foundation is an applicable way to increase the seismic resistance of the pile foundation because it has some distinct advantages such as less cost, time saving and less space necessary for the construction of the reinforcement. Some researchers and the applications of this method can be found in the works by Maeda et al (2008) and Adachi (2009). In the following sections, the detailed description about the shaking table tests will be given.

1G shaking table device

Photo 1 shows the shaking table test device whose size is 120cm in width and 160cm in length. The maximum acceleration is 1g and the maximum displacement is 5cm. The maximum payload is 16 kN and highest frequency is 10Hz. The vibration load is applied with air-pressure actuators that are very simple and can be maintained very easily. A laminate box, whose size is 100cm in width and 120cm in length, lies on the shaking table. An oil jack is also installed on the shaking table to drive an up-down movable frame on which a sand-dropping device is attached, as shown in Photo 1.

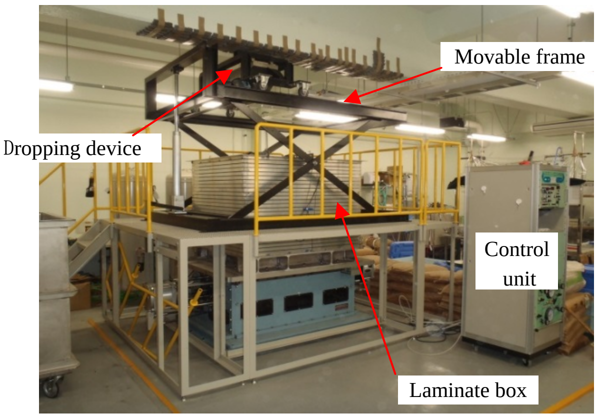


Photo 1 Shaking table test device

Laminate box

In order to make sure that the model ground during shaking can get rid of the influence of fixed boundary condition, a laminate shear box, shown in Photo 2, is deliberately manufactured with a size of 100cm in width, 120cm in length and 80cm in height. 27 layers of aluminum frame, each of which has a thickness of 3 cm, are connected one by one with smoothly movable bearings, so that the deformation of the ground during earthquake loading can be kept as layered ground, which is the same as the behavior of the ground in natural condition.

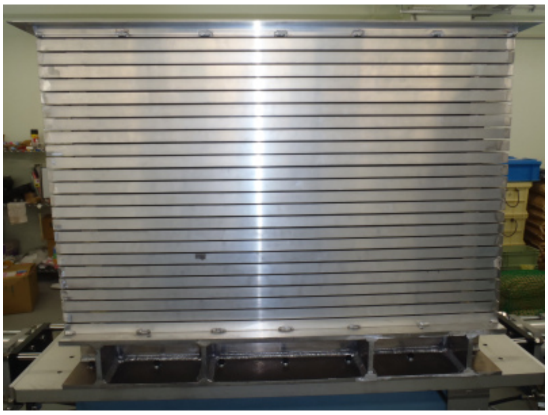
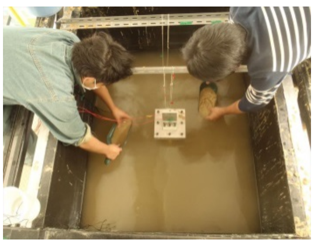
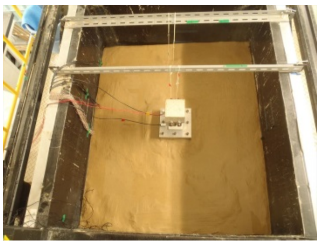


Photo 2 Laminate box

Preparation of model ground

Meanwhile, in order to prepare a uniformed saturated model ground with a prescribed density, an in-water sedimentary method is used to prepare carefully the sandy ground. In the preparation, the saturated Toyoura sand is dropped within the water with a depth of at least 10cm. After the ground level reaches a prescribed height, the water above the ground is then taken away, as shown in Photo 3. Because the sand is scattered evenly within the water, the density of the ground can be kept uniform rather easily.





(a) Preparation of saturated ground with in-water sedimentary method

(b) Saturated ground after water was taken away

Photo 3 Preparation of saturated model ground with Toyoura sand

Model group-pile foundation and superstructure

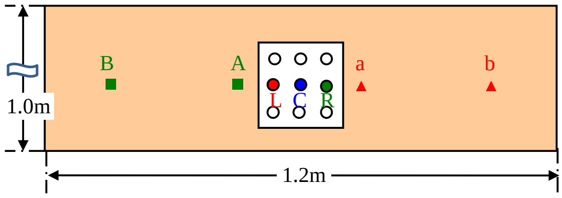
Photo 4 shows the model group-pile foundation and the superstructures. The length of the pile is 50 cm. Figure 1 shows the layout of the model group-pile foundation and the measuring devices in the tests. Table 1 lists the parameters of the piles, the pier and the footing in model scales.

Photo 5 shows the setup of the accelerometers and piezometers. In order to fix the position of the meters, a flexible ring of sockets was hanged on a stiff beam that lies on the laminate box and the accelerometers and the piezometers were then fixed on the sockets, as shown in Photo 5.

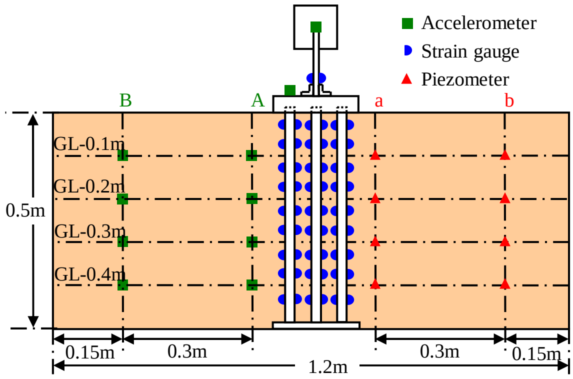
(a) Model 9-pile foundation

(b) Piles stuck with strain gauges and covered with water-proof tape

Photo 4 Model group-pile foundation and superstructure



(a) Plane view



(b) Vertical view

Figure 1 Layout of group-pile foundation and measuring devices in shaking table tests

Table 1 Parameters of model pile, pier and footing

Pile diameter (cm)	2.0
Thickness of pile (cm)	0.10
Pile length (cm)	50.
Pile spacing (cm)	6.0
Bending stiffness of pile ($N{\cdot}m^2$)	1.89E+02
Weight of upper structure (kN)	0.059
Height of pier (cm)	15.
Width of footing (cm)	18.
Thickness of footing (cm)	5.0
Elastic modulus of pile E_p (kPa)	7.0E+07
Density of pile ρ (g/cm^3)	2.7

(a) Flexible ring of sockets hosting accelerometers (b) Stiff beam hanged on laminate box

Photo 5 Setup of accelerometers and piezometers

Partial-improved ground made from cement-treated soil

In practical engineering, the ground improvement for seismic enhancement is usually conducted by mixing some cemented materials with the soft soils by high-pressure jet grouting or mechanical mixing method. In present study, a mixed soil from Toyoura sand and Fujimori clay, together with Portland blast-furnace slag cement B-type (in short, slag cement), were mixed with water to make the improved ground material. In the test, the materials with different ratios of each component of the mixed soil and the slag cement were tested with uniaxial compression tests to find a suitable improved ground material. Table 2 lists the physical properties of the cement-treated soil whose ratio of sand: clay: slag cement: water is 80:20:3:22.

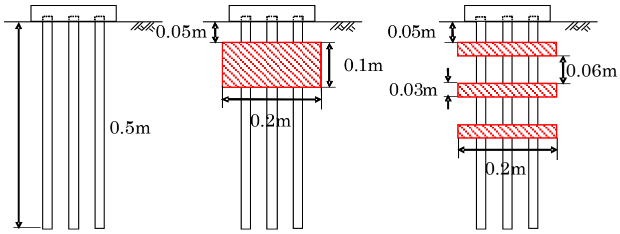
Table 2 Physical properties of cement-treated soil

Wet unit weight	γ_t （kN/m^3）	19.9
Water content	w（%）	21.6
Uniaxial strength	q_u （kPa）	600.
Poisson's ratio	ν	0.20
Deformation stiffness	E_{50} （MPa）	10.8

Photo 6 shows the model group-pile foundation with the partial-improvement blocks. In the tests, different type of the reinforcement blocks were used to compared the efficiency of the method. As shown in Photo 6 and Figure 2, three cases, that is, Case 1=No reinforcement; Case 2=Block reinforcement; Case 3=Multi-layer reinforcement, were tested in the shaking table tests.

(a) Case 2=Block reinforcement (b) Case 3=Multi-layer reinforcement

Photo 6 Model group-pile foundation with partial-ground-improvement blocks



(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 2 Cases of tests and corresponding analyses

Figure 3 shows the input wave used in the shaking table tests. The input wave is formerly aimed to be a cosine wave with a magnitude of 2.0 m/s^2 and a frequency of 4Hz. Due to the limitation of the air actuator used in the test device, however, there exists a relative large wave at the beginning of the vibration, which is the same for the three cases. The main earthquake vibration in the input wave lasts for about 10 seconds. This input wave is also used in the numerical calculation that will be discussed in the next chapter.

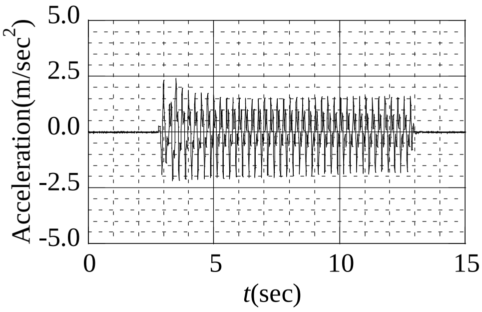


Figure 3 Input wave

3. NUMERICAL VERIFICATION

In simulate the shaking table tests, the nonlinear behavior of the Toyoura sand and the pile are described by the cyclic mobility model (Zhang et al, 2007, 2010 and 2011) and the AFD model (Zhang and Kimura, 2002) that can take into consideration of axial-force dependency in the nonlinear moment-curvature relations. It should be emphasized here that we call the calculation as numerical verification, not numerical simulation, only on the reason that the material parameters of Toyoura sand are fixed in all cases and are the same as those used in other theoretical simulations and boundary value problems that can be referred to in relevant researches (Zhang et al, 2010 and 2011; Jin et al., 2010; and Bao et al., 2012a,b). In other words, the parameters of the Toyoura sand are definitely determined, no matter what kind of the boundary value problem may be.

In the cyclic mobility model, some important concepts related to the mechanical behavior of soil, such as stress-induced anisotropy (Sekiguchi, 1977), subloading yield surface (Hashiguchi and Ueno, 1977), superloading yield surface (Asaoka et al., 1998) and transformed stress concept (Yao et al, 2008) were adopted and their intimate relations were firstly considered in an unified way. The model can properly take into consideration the influence of the stress-induced anisotropy, the density, the structure of soil and the intermediate stress. It can describe the mechanical behavior of soft soil subjected to different loadings, monotonic or cyclic, under different drained condition, drained or undrained, in a unified way with only eight fixed parameters. Detailed description about the model can be referred to the works by Zhang et al (2007, 2010 and 2011). The evidence of the successful applications of the model to the boundary value problem (BVP) of soil-water coupling analysis in geotechnical engineering can be referred to the works by Ye

(2007), Ye et al (2007), Xia et al (2010) , Jin et al (2010) , Bao et al., 2012a) and Bao et al., (2012b).

Figure 4 shows the 3D FEM mesh used in the dynamic analysis for the shaking table test of Case 1, considering the soil-water coupling problem. All the ground conditions, the size of the group-pile foundation, the footing and superstructure, are the same as those in the shaking table test. Because of the symmetric geometric and loading condition in the tests, only the half of the domain was considered in the calculation.

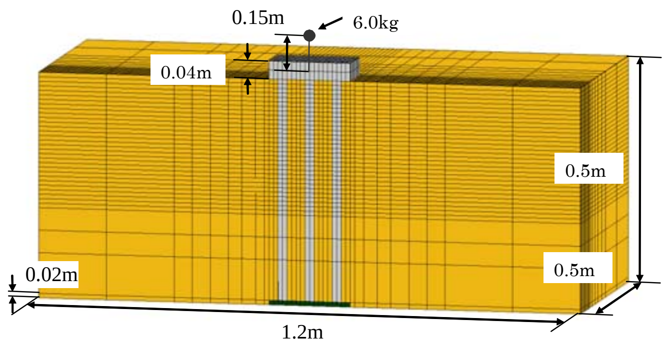


Figure 4 3D-FEM mesh

The boundary condition of the ground is that: (a) the bottom of the ground is fixed; (b) for two side boundaries, a periodic boundary is used, that is, the nodes at the two sides are restricted with equal-displacement condition, to simulate the one-dimensional layered behavior of free ground in far field. The boundary condition of the piles in the calculation is that the head of the pile is connected to the footing and the toe of the pile is free. The boundary condition of the pier is that the bottom of the pier is connected to the footing.

As for the hydraulic condition, except for the surface that is assigned as a drained boundary, all other boundaries are assigned as undrained. The underground water table is set to be the surface of the ground, as is the same as the test condition.

The parameters of the pile and the footing are listed in Table 1. The parameters of Toyoura sand are listed in Table 3. While the initial values of the state parameters of Toyoura sand and the cement-treated soil are listed in Table 4. The improved ground made of the cement-treated soil is also simulated with the cyclic mobility model. The way to determine the values of the material parameters can be referred to the works by Zhang et al. (2007, 2010 and 2011). In the calculation, the input wave is just the same as that used in the shaking table tests, as shown in Figure 3. The pier is modelled with tri-linear model.

Table 3 Material parameters of Toyoura sand & cement-treated soil

	Toyoura sand	Cement-treated ground
Compression index λ	0.050	0.010
Swelling index κ	0.0064	0.0030
Critical state parameter M	1.30	1.66
Void ratio **N** (p`=98 kPa on *N.C.L.*)	0.87	1.1
Poisson's ratio ν	0.30	0.20
Degradation parameter of overconsolidation state m	0.01	0.10
Degradation parameter of structure a	0.50	0.05
Evolution parameter of anisotropy b_r	1.50	0.25

A Rayleigh type of damping is adopted and the values of the ground and the structures (piles and pier) are assumed as 5% and 2%, respectively, in the dynamic analysis of the full system. A direct integration method of Newmark-β is used in the dynamic analysis and the time interval of the integration is 0.002 sec. It should be point out that in the case of analyses considering strong nonlinearity, only the initial stiffness of the materials is adopted for the Rayleigh damping.

Table 4 Initial conditions of Toyoura sand & cement-treated soil

	Toyoura sand	Cement-treated ground
Initial void ratio e_0	0.68	-
Permeability k (m/sec)	5.7e-4	1.2e-10
Initial degree of structure R_0^*	0.99	0.50
Initial degree of overconsolidation OCR ($1/R_0$)	30.	50.
Initial anisotropy ζ_0	0.0	0.0

Note: Wet unit weight of Toyoura sand= 15.5 kN/m^3

4. RESULTS AND DISCUSSION

Figure 5 shows the measured excessive pore water pressure (EPWP) and excessive pore water pressure ratio (EPWPR, the value of EPWP divided by the initial vertical effective stress) at different measuring points. It is known from the figure that all the ground at different depths liquefied immediately after the strong motion hit the sandy ground. It is also known that the dissipation of the EPWP after the strong motion happened more quickly in deep layer than in shallow layer, implying that the supply of pore water from the deep layer may delay the dissipation of the excessive pore water in the surface layer.

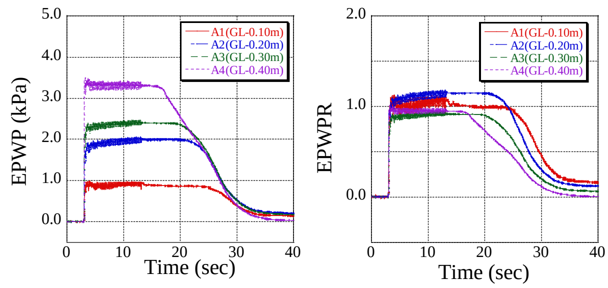
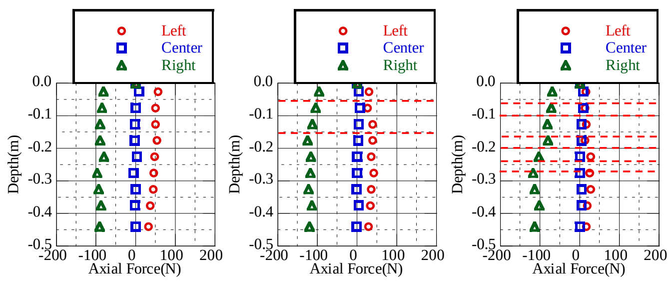


Figure 5 Measured EPWP at different depth in Case 1

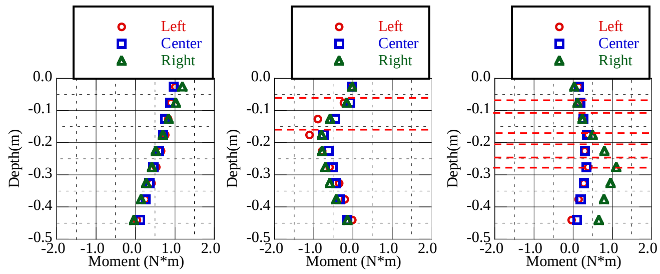
Figure 6 shows the measured distribution of axial forces of the piles at the time when the maximum bending moment occurs. There is no much difference among the three cases at the first look. It is, however, that the axial force within the area of partial-improvement decreases somehow, especially the left pile in Case 2 and Case 3, and the right pile in Case 3. The reason is quite clear that due to the increase of the frictional resistance of the cement-treated soil, the load ratio shared by the piles will consequently decrease.



(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 6 Measured distributions of axial forces at the time when maximum bending moment happened

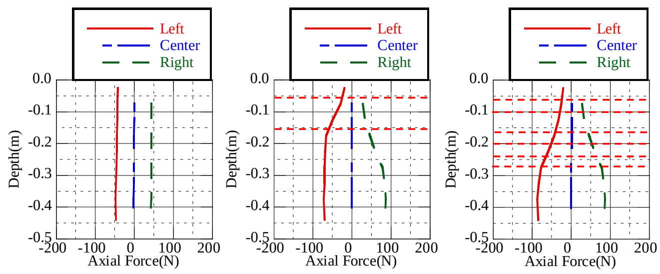
Figure 7 shows the test results of the distribution of bending moment at the time when the maximum bending moment occurs. It can also be seen that the moment at the top of the piles was reduced significantly due to the reinforcement effect of the partial-improvement ground. While the moment beneath the partial improved ground increased due to the abrupt change of the stiffness of the original sandy ground and the cement-treated soil. The moment at the pile head is usually thought to be caused by the inertia force from the superstructure while the moment far deep from the surface is caused by the deformation of the ground. The maximum bending moment does decrease in some piles when using the partial-improvement method. But this is not always the case, e.g., in Case 3 (Multi-layer reinforcement), the maximum bending moment of the right pile is almost the same as that in other two cases, which means that the efficiency of the partial-improvement method is not prominent. It is also known from the figure that the maximum bending moment occurs at different position due to the ground improvement. Moreover, the moment in the lower part of the piles has no prominent difference among the three cases, implying that the range affected by the ground improvement was limited to a local area.



(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 7 Measured distribution of bending moments at the time when maximum bending moment happened

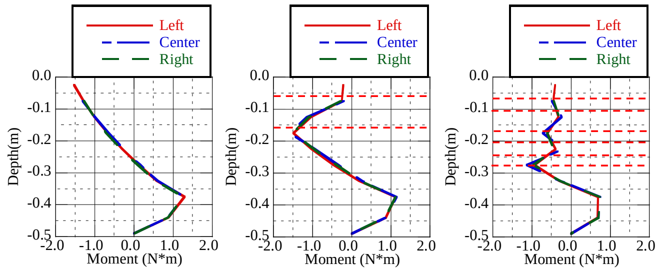
Figure 8 shows the calculated distribution of axial forces of the piles at the time when the maximum bending moment occurs. The results are quite similar to those of the tests. The only thing that should be emphasized here is that contrary to the test results, the tendency that the axial force within the area of partial-improvement decreases evidently, can be clearly observed in the calculation, which again supports the argument that due to the increase of the frictional resistance of the cement-treated soil, the load ratio shared by the piles will consequently decrease.



(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 8 Calculated distribution of axial forces at the time when maximum bending moment happened

Figure 9 shows the calculated distribution of the bending moments in different piles at the time when the maximum bending moment occurs. The calculated results are on the whole similar to the test results to some extent, but with more rigorous change in the distribution that cannot be measured in the tests. As shown in the figure, the phenomenon that the moment at the top of the piles was reduced significantly due to the partial-improvement ground while the moment beneath the partial improved ground increased, can be clearly reproduced. Meanwhile, the rigorous change of the bending moment at deep part of the ground that cannot be observed in the tests is clearly shown in the calculation.

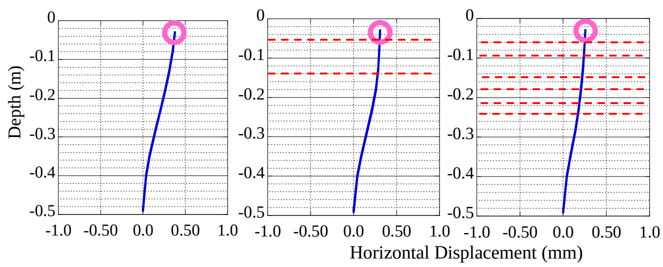


(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 9 Calculated distribution of bending moments at the time when maximum bending moment happened

To check the validity of these calculated results, the calculated distribution of the horizontal displacement of the pile at the time when the maximum bending moment occurs, is shown in Figure 10, in which the second mode of the horizontal deformation pattern shows up clearly, implying that a sharp change of the corresponding moment should occur because of the deformation mode.

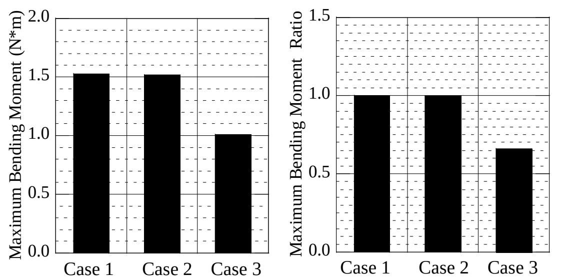
It is also known from the calculated results shown in Figure 10 that the partial-improvement method does repress the deformation of the pile foundation effectively, which cannot be measured in the test in liquefied ground due to the limitation of the measuring technique.



(a) No reinforcement (b) Block reinforcement (c) Multi-layer reinforcement

Figure 10 Calculated distribution of horizontal displacement at the time when maximum bending moment happened

Figure 11 shows the calculated results of the reinforcement effect of the partial-improvement method with different type of reinforcement blocks. It is known from the figure that the partial-ground improvement method with the multi-layer reinforcement is most efficient in reducing the maximum bending moment. The maxi -mum bending moment is reduced almost one third in the case of multi-layer reinforcement pattern.



(a) Maximum bending moment (b) Maximum bending moment ratio

Figure 11 Calculated results of the reinforcement effects in different types

5. CONCLUSION

In this paper, a shaking table test was conducted on a full system consisting of a superstructure, a nine-pile foundation and a sandy ground, considering the liquefaction of the ground with different reinforcement pattern using the partial-ground improvement method. Moreover, as a verification, a corresponding 3D dynamic soil-water coupling FEM analyses using the program named as DBLEAVES was also conducted. The following conclusions can be given:

By careful preparation of the saturated sandy ground and careful setting of the measuring devices in the shaking table test, it is possible to conduct a high-quality shaking test on a full system with superstructure, group-pile foundation and ground. The efficiency of seismic enhancement by the partial-ground improvement method was confirmed in the shaking table tests. It is found that on the whole, the enhancement effect to reduce the bending moment within the piles can be achieved, especially within the improved area. In present case, the bending moment at the top of the pile is reduced greatly. On the other hand, the moment beneath the partial improved ground increased due to the abrupt change of the stiffness of the original sandy ground and the cement-treated soil. An extreme case is that, the maximum bending moment of the right pile is almost the same as that in other two cases, which means that the efficiency of the partial-improvement method seams not prominent. On the whole, however, the bending moments of group piles decrease due to the reinforcement.

In the numerical analysis, the values of the material parameters of the model ground made from Toyoura sand, are all the same as those used in the past researches, in other words, no any calibration of the parameters of Toyoura sand was conducted, which is quite different from the normal numerical calculations in which the calibration of parameters is always necessary. The aim of the analyses in this paper, is not to simulate or fit for the test results, but to confirm whether it is possible to describe tests results based on the element geomaterial data obtained from laboratory element tests. Another purpose of the calculation is to find something that cannot be measured in the tests, such as the deformation within the liquefied model ground. The calculated result is quite convincing and encouraging, it reveals the fact that the partial-improvement method does repress the deformation of the pile foundation effectively. As the consequent result, the maximum bending moment is reduced almost one third in the case of multi-layer reinforcement pattern. It is, however, need to confess that in order to make the readers convincing the above-mentioned arguments, more cases of model tests are needed to increase the reproducibility, which is a very hard task indeed.

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