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Dynamic centrifuge model tests on pile foundation of plate-shape building in soft clayey ground and liquefiable sandy ground

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Abstract

The seismic behavior of pile foundations in soft ground is extremely complicated since the soft ground is intensively nonlinear in mechanical properties and gives large relative displacements to the pile foundations during earthquake. Plate-shape buildings cause large lateral force and varying axial force in their pile foundations and make their seismic behavior more complicated because they each have a high aspect ratio and single span composed two columns and several beams.

This paper conducted two dynamic centrifuge model tests on a pile foundation of plate-shape building in 50 g field. The first ground model was composed of soft clayey layer and load bearing layer. The second was composed of liquefiable sand layer, soft clayey layer, and load bearing layer. The soft clayey layer was simulated using the loam that was volcanic sandy silt and clay. The loam has an advantage in controlling shear stiffness and strength because it had a wide range of water content due to a wet tamping method. We used water as pore fluid in this layer, instead of viscous fluid for saving time in the consolidation process of a centrifuge field. The liquefiable sand layer was simulated using the silica sand made by wet tamping method. In this layer, the viscous fluid of 50 cSt was used as pore fluid. To reproduce the seismic behavior of actual pile foundation, a 13-story superstructure with a height of 40m was supposed, and cast-in-concrete piles with a diameter 1.5 m and 2.2 m in the shaft and bell respectively were reproduced. The specifications of centrifuge model were determined by reconciling the axial stress of the piles and natural frequency of the superstructure in the centrifuge model with those in the prototype.

From the several shaking tests with increasing the input motion form 0.5 m/s^2 to 6.0 m/s^2, the inertial force of superstructure on the clayey ground was increased with increasing input intensities. On the other hand, that on the ground with liquefiable layer was restricted certain value. We consider the reason for this is the natural period of the ground was increased because of the sand liquefaction and the larger subgrade reaction from sandy soil. The sand liquefaction also caused that the bending moment of the entire pile in sandy ground was larger than that in clayey ground, although the inertial force of superstructure was attenuated in the same intensity of input motion.

Keywords: Centrifuge model test; Clayey ground; Sand liquefaction; Structure-pile-ground system; Plate-shape building

1. Introduction

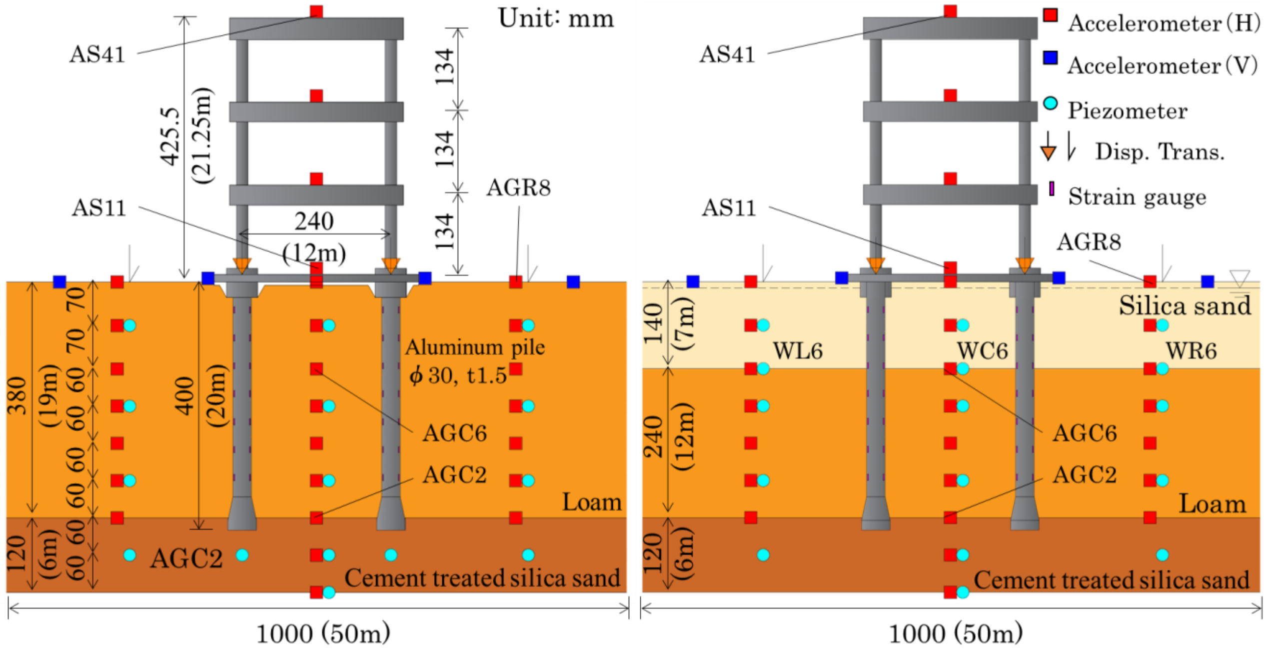
The seismic behavior of pile foundations in the soft ground is complicated since the soft ground is intensively nonlinear in mechanical properties and gives large relative displacements to the pile foundations during an earthquake. Also, plate-shape buildings cause large lateral force and varying axial force in their pile foundations and make their seismic behavior more complicated because they each have a high aspect ratio and a single span composed of two columns and several beams. However, only a few model tests have been carried out on pile foundations in the clayey ground because it was troublesome and time-consuming to prepare model grounds from slurry clay. A few numbers of centrifugal experimental studies have been carried out to investigate the behavior of pile foundation (including friction pile and piled-raft foundation) in soft clayey ground using artificial clay [1, 2, 3]. On the other hand, we developed useful method to enable low stiffness and strength soil in centrifugal experiment by using loam and water. The loam that is volcanic sandy silt and clay has an advantage in controlling shear stiffness and strength because it has a wide range of water content due to a wet tamping method. We used this method and conducted the dynamic centrifugal model tests to investigate the seismic behavior of a plate-shape building supported by pile foundations in the soft clayey ground [4]. This paper conducted additional series of dynamic centrifuge model tests on a pile foundation of plate-shape building in 50g field. The ground model was composed of liquefiable sand layer, soft clayey layer, and load bearing layer. It seems the ground to be more severe condition for pile foundation. The behavior of superstructure and pile foundation were discussed based on comparing the result of the two centrifugal test series.

2. Test program

2.1 Model set up

Fig.1 illustrates a schematic diagram of a plate-shape building and a pile foundation in the soft clayey ground and liquefiable sandy ground. The model A was composed of soft clayey layer and load bearing layer. The model B was composed of liquefiable sand layer, soft clayey layer, and load bearing layer. The soft clayey layer was simulated using the loam. We used water as pore fluid in this layer, instead of viscous fluid for saving time in the consolidation process of a centrifugal field that this substitution would not affect the permeability during shaking tests because of the loam's very low permeability (9.0 E^{-07} m/s). The physical and mechanical properties of the loam for the centrifuge models were reported in the previous study [4]. The liquefiable sand layer was simulated using the silica sand made by wet tamping method with the relative density of Dr = 70 %. In this layer, the viscous fluid of 50 cSt was used as pore fluid. Cement-treated silica sand with an unconfined strength of about 1,400 kN/m^2 was used as the load bearing stratum for the pile tips. A 13-story superstructure with 40 m in height and a pile foundation using cast-in-concrete piles, the shaft and bell diameters of which were 1.5 m and 2.2 m respectively, were reproduced in a centrifuge field of 50 g [5]. The building model was made from steel parts to reproduce axial force on the piles (9,330 kN) and natural periods of the superstructure (1st: 0.412 s, 2nd: 0.138 s, and 3rd: 0.084 s) in a prototype. Four piles made from an aluminum pipe with a thickness of 1.5 mm were connected to a slab plate through pile caps. Fig.1 also shows the arrangement of transducers. Accelerometers and pore water pressure gauges with ceramic filters were used. Displacement transducers were installed to measure the vertical displacements at the footing and ground surface respectively.

Model A and B was swung up to 50 g and stabilized by setting in centrifugal field for 6 and 4 hours respectively. Fig.2 shows the distribution of the pore water pressure after stabilizing the clayey layer. The blue rapture line in Fig.2 indicated the theoretical hydrostatic pressure if the groundwater level were 7 m and 3 m in model A and B, respectively. From this figure, the groundwater level in model B was higher than that in model A, and the silica sand was saturated from 3 m to 7 m depth in model B.



a) Model A b) Model B

Fig. 1 – Schematic diagrams of centrifugal model using plate-shape building and pile foundation

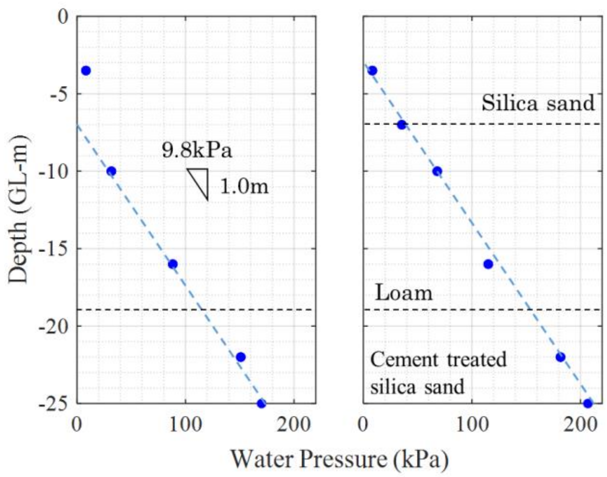


Fig. 2 – The distribution of the pore water pressure observed after stabilizing the clayey layer in 50g field.

After the stabilization of the clayey layer, several shaking tests were carried out with increasing the input motion step by step. Fig.3 shows an example of input motion defined as 2E. However, the input motion composed of the upward and downward waves (E+F) is necessary to control the shaking table. Therefore, one-dimensional seismic response analysis was executed using SHAKE. Table 1 shows the relationship between 2E and E+F. Fig.4 shows the pore water pressure observed at 7 m depth from the ground surface in the cases of B-3 and B-5. Also, the rapture line in Fig.4 shows the total stress (σ_z) in 7 m depth. At the time of 30 s from the time when the shaking table test started, the pore water pressure exceeded, and reached to the total stress in the case B-5 and didn't reach to that in the case B-2. From this result, it is considered that the silica sand layer was not liquefied in the case of B-3. On the other hand, the partial sand layer in the depth from 3 m to 7 m was liquefied in the case of B-5.

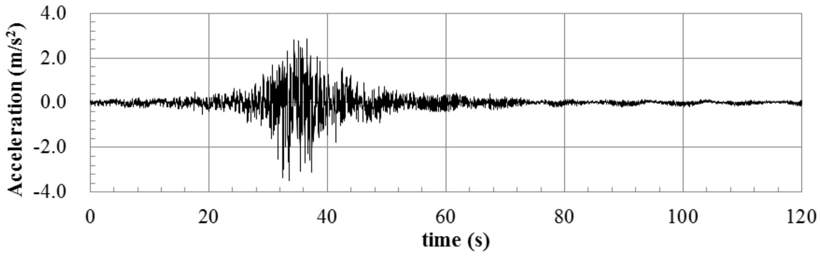


Fig. 3 – Example of input motion with a peak acceleration of 3.5m/s^2 defined as 2E.

Table 1 – Shaking Test Program considered in this study (at prototype scale)

Case Number	Peak acceleration (m/s^2)		Case index	
	Outcrop (2E)	Within (E+F)	Model A	Model B
1	0.50	0.35	A-1	B-1
2	2.00	1.23	A-2	B-2
3	3.50	2.14	A-3	B-3
4	4.50	2.96	A-4	—
5	6.00	3.85	A-5	B-5

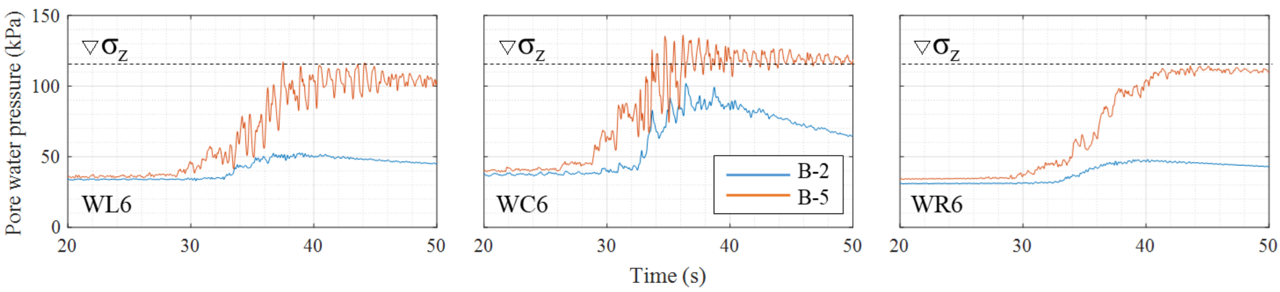
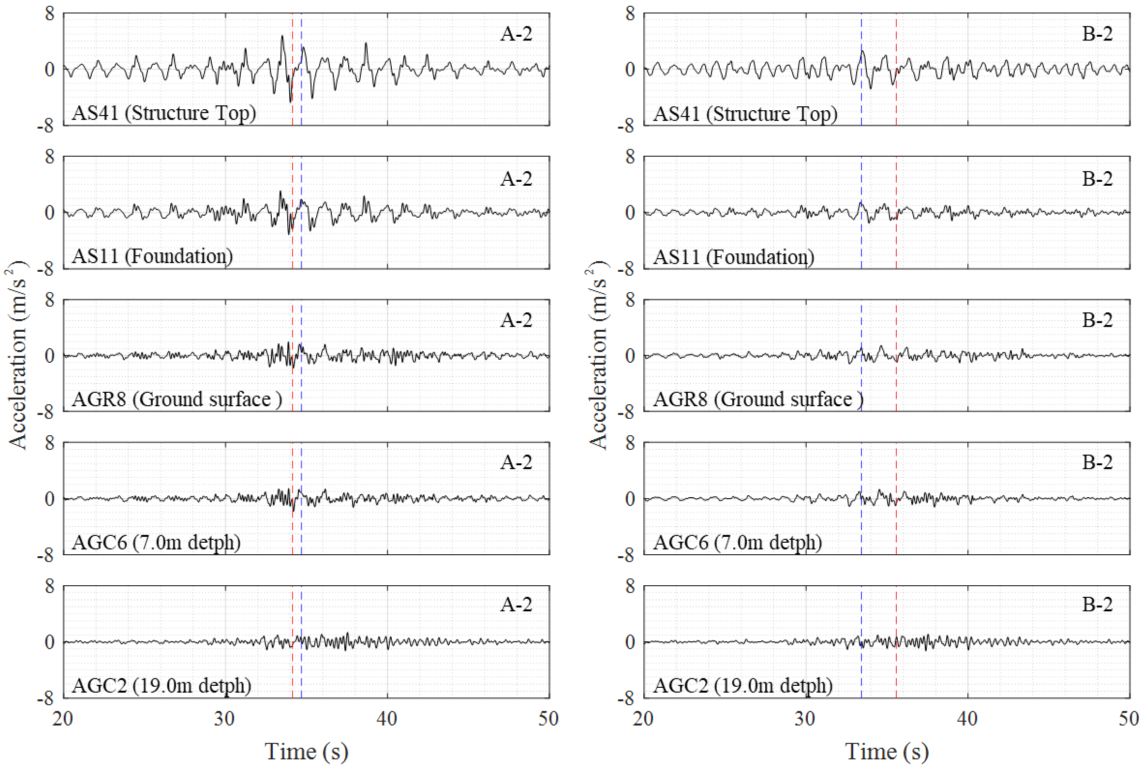


Fig. 4 – Pore water pressure observed at 7 m depth from the ground surface in model B

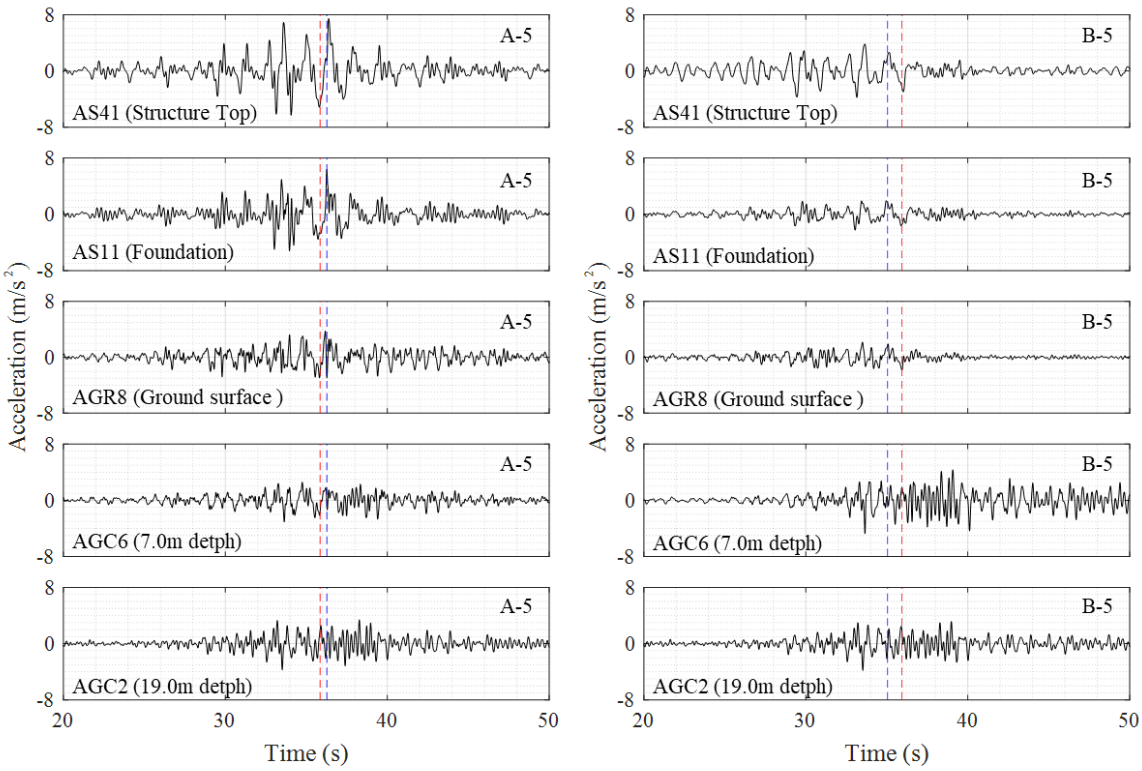
3. Test result

3.1 Seismic response

Fig.5 shows the response acceleration observed in the medium and large input case. From this figure, the acceleration observed on the ground surface (AGR8) was almost same size in A-2 and B-2. On the other hand, the acceleration observed on the ground surface in A-5 was larger than that in B-5. In addition, the accelerations observed on the structure top (AS41) and the footing (AS11) in model A (A-2, A-5) were lager than those in model B (B-2, B-5). To confirm this point, Fig.6 shows the summaries of the maximum inertial force and overturning moment of structure in each case. From this figure, the inertial force and overturning moment in model A increased with increasing input acceleration. On the other hand, the inertial force and overturning moment of structure in model B increased with increasing input acceleration from B-1 to B-3, but the inertial force and overturning moment were restricted certain value from B-3 to B-5.

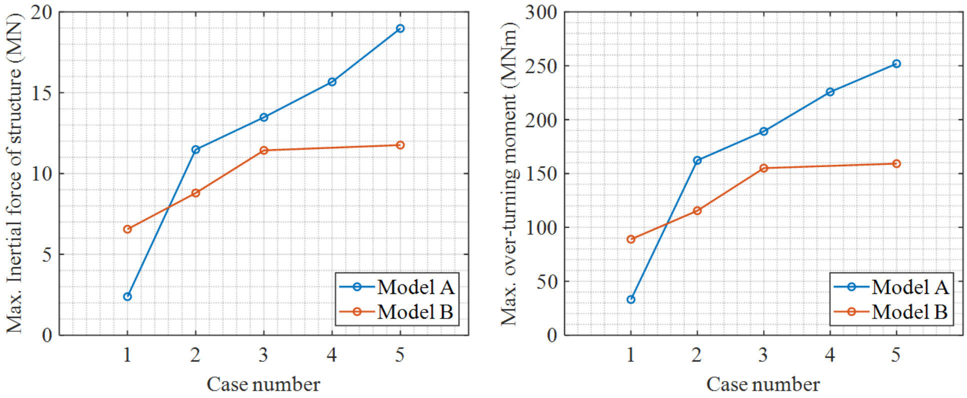


a) Medium size input



b) Large size input

Fig. 5 – Response accelerations observed in medium input (case A-2, B-2) and large input (case A-5, B-5)



a) Maximum inertial force b) Maximum overturning moment

Fig. 6 – Summaries of the maximum inertial force and overturning moment of the superstructure

3.2 Pile stress

We found that the incremental axial force of the pile foundation in plate-shape building were well-estimated by Eq. (1) from previous study [6]. Fig.7 shows the relationship of the observed incremental axial force and the incremental axial force estimated from Eq. (1). From this figure, axial force was well-estimated from the overturning moment and bending moment at the pile heads, and the amplitude of observed incremental axial force were almost proportional to the maximum overturning moment (see in Fig.6 (b)).

$$P_{est} = (OTM + \Sigma M_{top}) / 2L \tag{1}$$

Where P_{est} it the estimated axial force, OTM is the overturning moment of structure, ΣM_{top} is the summation of the bending moment at the pile heads, and L is the distance (center-to-center) of the piles in the shaking direction.

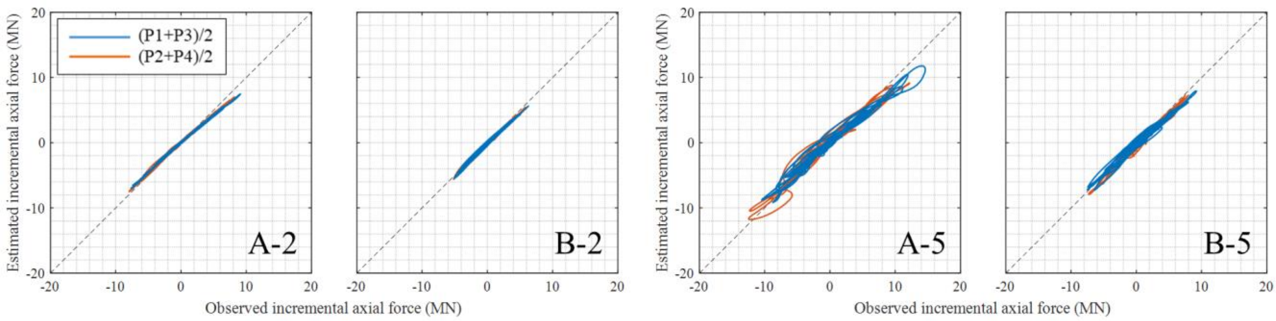


Fig. 7– The relation of observed incremental axial force in 6.75m depth and estimated incremental axial force from overturning moment and the bending moment at pile heads.

Fig.8 shows the bending moment observed at the pile heads (GL-2.25 m depth) in each case. In addition, the red and blue rapture lines indicate the time when the peak bending moment recorded. From this figure, the bending moment at the pile heads in A-2 and B-2 had almost same amplitude. On the other hand, the peak bending moment in B-5 was larger than that in A-5, although the inertial force of structure was smaller than A-5. The difference of bending moment in the pile heads was influenced by the ground conditions in the strong motion. To confirm this point, Fig.9 shows the distributions of the bending moment at the time when the peak moment at the pile heads recorded. From this figure, the distribution of the bending moment in the cases of A-2 and B-2 were almost same. On the other hand, the distribution in the cases of A-5 and B-5 were quite different. The bending moment observed not only at the pile heads but also at 11.25m and 13.75m depth in B-5 was larger than that in A-5. From this result, the deflection of the piles in B-5 was larger than that in A-5 in the entire piles although the inertial force was attenuated in the same intensity of input motion.

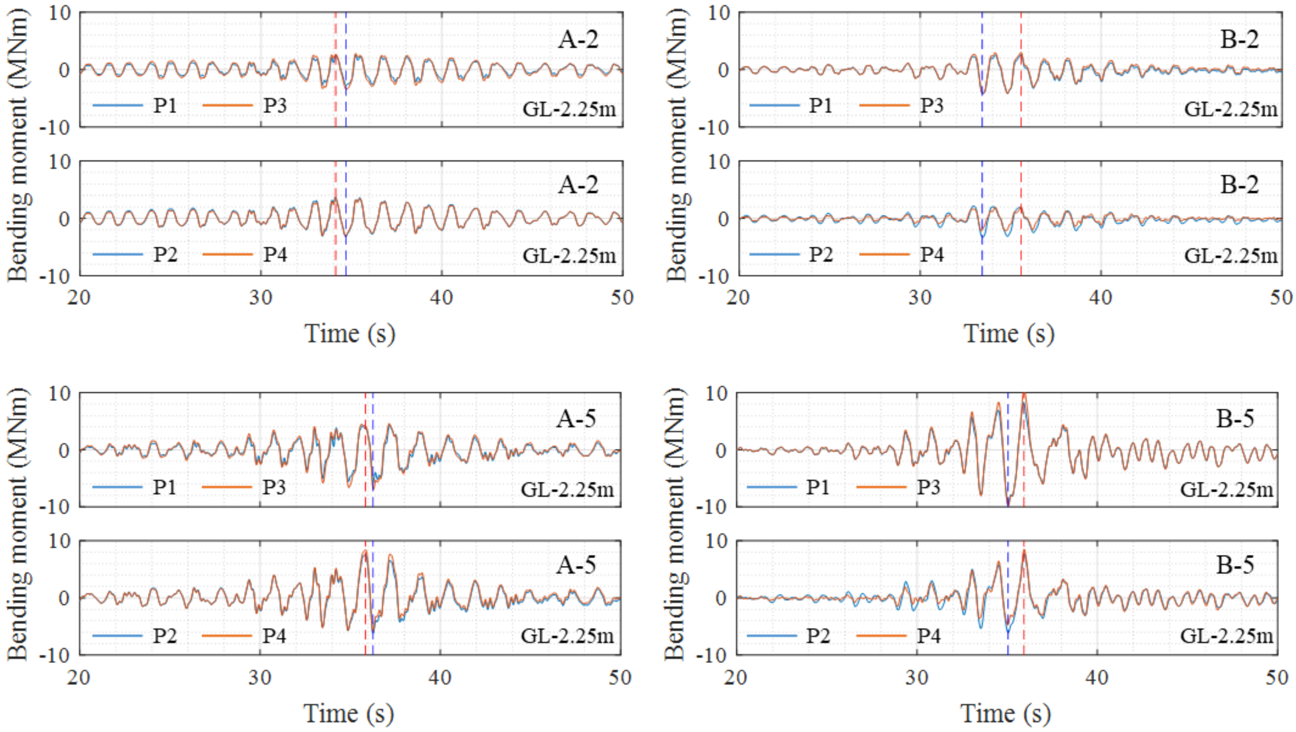


Fig. 8 – The bending moments observed at the pile heads (2.25 m depth)

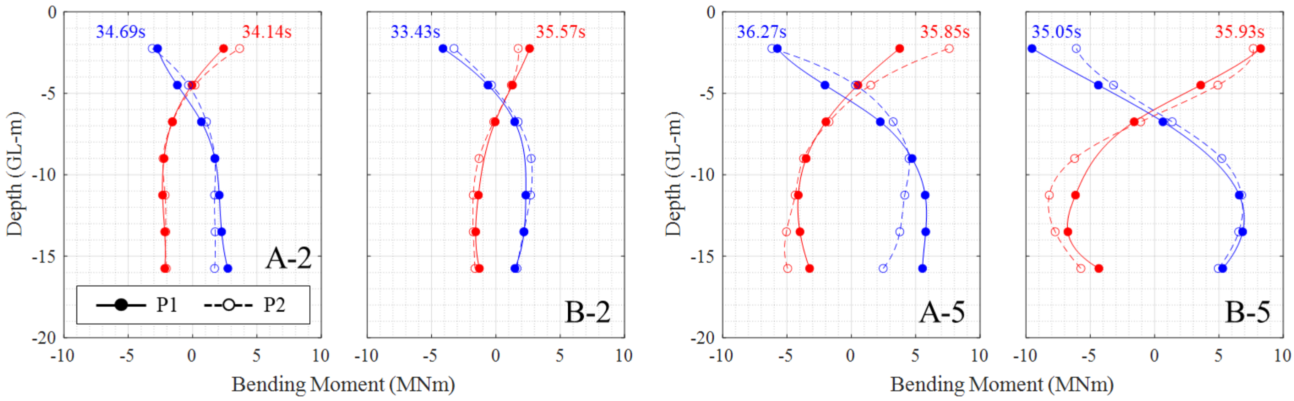


Fig. 9 – The distributions of the bending moment at the time of the peak bending moment at the pile heads was recorded.

4. Discussion

To consider the reason why the bending moment of piles in B-5 was larger than that in A-5, we calculated the ground and pile displacement. The ground displacement was calculated by the double integration with Trifunac's method [7]. On the other hand, pile displacement was calculated by the method illustrated in Fig. 10. Firstly, the inclination θ_f was calculated from the displacement of footing D_h and load bearing surface D_t assumed to be the displacement of pile head and tip respectively. The displacement of the footing D_h and load bearing stratum surface D_t were calculated from the double integration of their acceleration (AS11 and AGC2). Secondly, to calculate the inclination θ_p and the local distribution of the pile displacement from the double integration of deflection angle observed by strain gauge. In this integration, the distortion angle at the pile head was zero as the boundary condition. Finally, to calculate the global distribution of pile displacement, the local distribution of the pile displacement was translated by the displacement of the pile head and rotated by the difference of θ_p from θ_f.

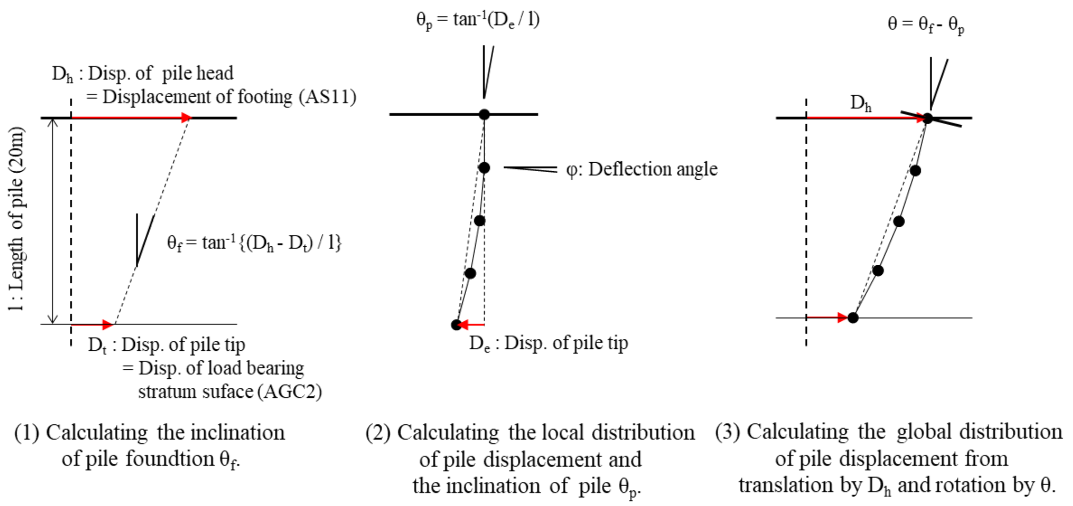


Fig. 10 – The schematic diagram of the method of calculating the global distribution of the pile displacement.

Fig. 11 shows the calculated piles and ground displacement at the time when the peak moment at the pile heads recorded. From this figure, the relative displacement of pile and ground at the ground surface in the model A was larger than that in the model B. We considered the reason for this is that the subgrade reaction from clayey soil was less than that from sandy soil in the nearly ground surface. In addition, in the case of B-5, the ground displacement was increased significantly in the depth from 3.5 m to 7.0 m. It is the reason for this that the large shear strain was occurred by the soil liquefaction in this layer (see in Fig. 4). From this result, the large subgrade reaction of piles from the clayey soil under the sandy soil was occurred. We considered that this is the reason why the deflection of piles in the case of B-5 was larger than that in the case of A-5.

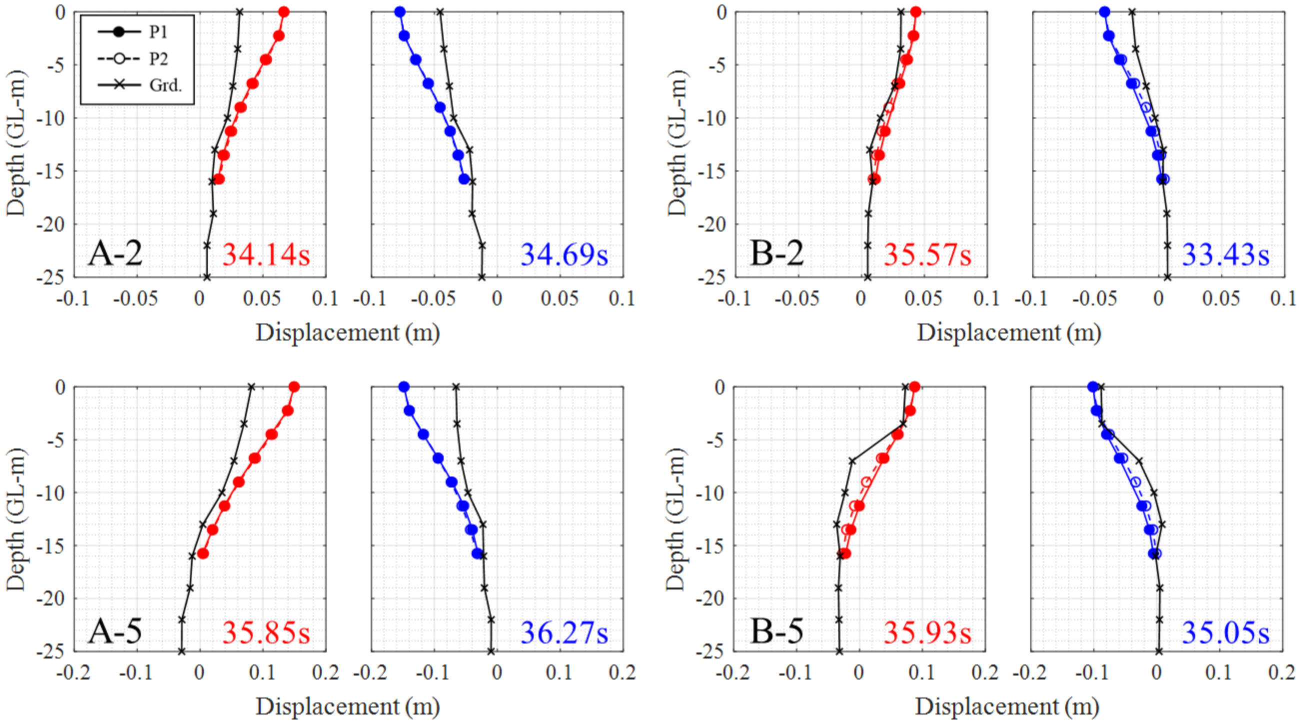


Fig. 11 – The distributions of piles and ground displacement at the time when the peak bending moment at the pile heads were recorded.

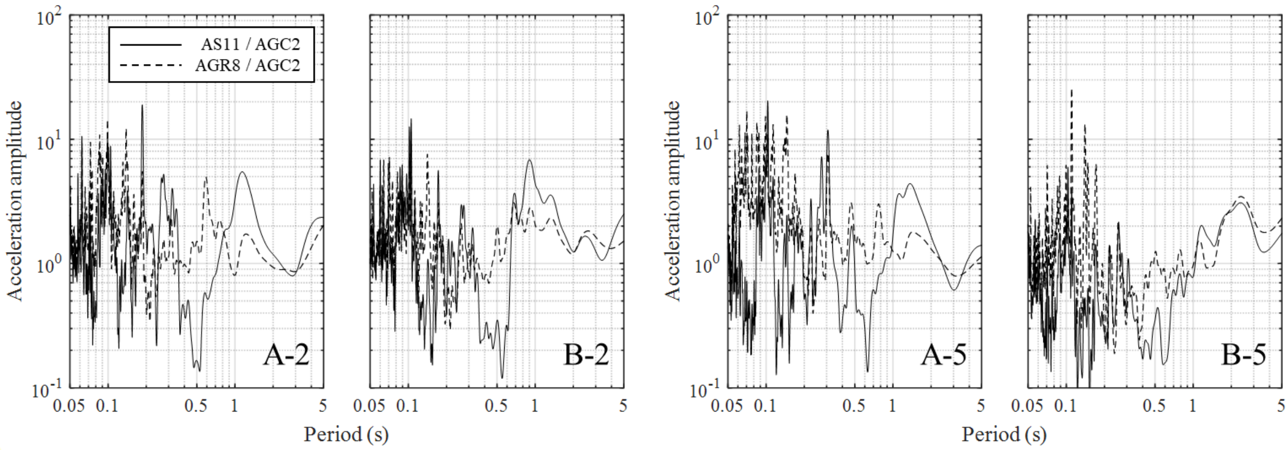


Fig. 12 – Fourier amplitude ratios of the acceleration observed on the footing (AS11) and the right-side ground surface (AGR8) against the acceleration observed on the load bearing stratum (AGC2)

To consider the seismic response of superstructure, Fig. 12 shows Fourier amplitude ratios of the acceleration observed on the footing (AS11) and the right-side ground surface (AGR8) against the acceleration observed on the load bearing stratum (AGC2). From this figure, in the cases of clayey ground (A-2, A-5), the peak amplitude of footing exceeded that of ground surface around the period of 1 s. We considered that the narural period of superstructure-pile-ground system existed around this period of 1s. On the other hand, in the cases of sandy ground (B-2, B-5), the amplitude of footing was almost similar to that of ground surface around the peak. However, the difference in B-2 was larger than that in B-5. From this result, in the case of clayey ground, the superstructure vibrated in the natural period of the superstructure-pile-ground system around the period of 1 s and the displacement of foundation exceeded that of the ground because the subgrade reaction from the clayey soil was little. On the other hand, in the case of sandy ground, the superstructure vibrated with the ground becase of the larger subgrade reaction from the surface sandy soil. In addtion, in the case of large intensity of input motion (B-5), the soil liquefaction made the natural period of ground longer. As well as, it occured the attenuation of the superstructure's response acceleration as well as the inertial force although the stronger intensity of input motion.

5. Conclusions

We carried out two series of centrifugal model tests to investigate the behaviour of a plate-shaped building supported by pile foundations in soft clayey ground and liquefiable sandy ground during the earthquakes. The first ground model was composed of soft clayey layer and load bearing layer and the second was composed of liquefiable sand layer, soft clayey layer, and load bearing layer. They were subjected to designed earthquakes of smaller and stronger intensities in 50 g centrifugal tests. The major conclusions from this research:

1. The response accelerations of superstructure were increased with increasing the intensity of input motions, except in the case when the sandy layer was liquefied by stronger intensity of input motion. Although the inertial force of structure was attenuated in that case, the bending moment at the pile heads were larger than that of the clayey ground test in the same intensity of input motion.

2. The sand liquefaction occurred the large shear strain in the partial sandy ground. The shear strain caused the larger deflection of the piles in the sandy ground than that in the clayey ground. From this result, the bending moment of piles in the sandy ground was larger than that in the clayey ground.

3. The relative displacement of the foundation and the clayey ground was larger than that of the foundation and the sandy ground because the subgrade reaction of piles from clayey soil was less than that from sandy soil. From this result, the superstructure vibrated around the natural period of superstructure-pile-ground system in the clayey ground. On the other hand, the superstructure vibrated around the natural period of the sandy ground. In addition, the inertial force of structure in stronger intensity of input motion was attenuated because the sand liquefaction made the natural period of ground longer.

6. References

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