Dynamic Response of Pile Cap and Distribution laws of Bending Moment about Vertical and Batter Pile Groups in Liquefiable Soil

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Abstract

This paper uses the ZJU400 geotechnical centrifuge of Zhejiang University to study the dynamic response of pile cap and distribution laws of bending moment about vertical and batter pile groups in liquefiable soil. Two groups of experiments, including vertical and batter pile groups, are prepared. The effects of pile type on the dynamic response of pile cap and bending moment of pile body in liquefiable soil were studied. The results show that: in the cases of 0.05 g and 0.1 g, the horizontal acceleration and displacement of the vertical pile cap are amplified, and the greater the vibration intensity is, the more obvious the dynamic response amplification, while in the case of 0.3 g, the horizontal acceleration and displacement are reduced. However, the horizontal acceleration and displacement of the batter pile cap do not exhibit obvious amplification under different vibration intensities. The bending moment distribution of the vertical and batter pile group changes greatly with increasing vibration intensity. The bending moments of the vertical and batter pile groups are large at the pile heads. However, in addition to the large bending moment at the top of the pile, the vertical piles have larger bending moments at different depths under different vibration intensities. In the cases of 0.05 g and 0.1 g, the bending moment envelope of the vertical pile is larger than that of the batter pile, but in the case of 0.3 g, due to the increase in the liquefaction depth, the bending moment envelope of the vertical pile is smaller than that of the batter pile.

Keywords: vertical and batter pile, liquefiable soil, dynamic response, bending moment

1. Introduction

As an important form of deep foundations, pile foundations have been widely used in various engineering constructions. In particular, in areas with poor geological conditions, the advantages of pile foundations are more prominent. At the same time, due to the wide application of pile foundations, especially in marine environments, pile foundation applications have begun to attract attention. However, the factors affecting the performance of a pile foundation in a marine environment are very complicated, and earthquake activity is a particularly significant factor affecting the pile foundation because earthquakes are often accompanied by sand liquefaction in the marine environment. Therefore, some scholars have made preliminary explorations on the seismic performance of pile foundations under the condition of liquefaction.

For the study of such problems, the methods currently used by some scholars include model tests and numerical simulations. The model test studies are mainly based on shaking table tests and centrifuge shake table tests. Miyajima et al. [1] conducted a shaking table test under the condition of saturated sand to investigate the influence of sand slope and thickness on soil deformation at an early stage. Choobbasti et al. [2] used the finite difference method to study the pile foundation response and pile-soil interaction after soil liquefaction under earthquake action. Samui et al. [3] studied the lateral instability buckling of piles in 26 pile foundations under applied axial loads, considering the liquefaction of soil around the pile. Su et al. [4] analyzed the damage due to liquefaction-induced lateral spreading of a single pile behind a retaining wall and studied the lateral pressure generated by the soil after liquefaction on the pile during a shaking table test. Li et al. [5] investigated a large-scale model test on the E-Defense shaking table facility in Japan and established a two-dimensional nonlinear dynamic finite element model to study the interaction of a pile-soil system considering two-dimensional effective stress. The results of the coupled finite element model analysis and the experimental results were compared by using OpenSees. Ling et al. [6] carried out a model test of a pile foundation by using a large shaking table and observed the sand liquefaction and its influence on the damage of the pile foundation through the input of different earthquake intensities. Huang et al. [7] studied the dynamic pile-soil-structure response under earthquake action through a small shaking table dynamic model test. With the change in excess pore pressure, the saturated sand gradually liquefied from top to bottom, and the settlement of the structure increased. At the same time, the pile-soil-structure interaction effect increased the seismic response of the upper structure. Panah et al. [8] studied the effects of different inclination angles on the seismic performance of micropiles by performing shaking table tests. Cengiza et al. [9] studied the response of geosynthetic composite gravel piles and ordinary gravel piles under dynamic loading by performing shaking table tests. Yang et al. [10] studied the pile-soil interaction mechanism under the condition of liquefied soil overlying a frozen soil layer during shaking table testing.

The above review introduces some progress on the research of pile foundations in liquefaction sites by using shaking table tests; shaking tables have commonly been used to research such problems. However, to solve the limitations of model size and other issues, scholars have gradually recognized the potential of centrifuge shaking table model tests in this research.

Su et al. [11] studied the variation in lateral soil resistance and the evolution of the horizontal pile-soil interaction of a single pile in saturated sand by using a centrifuge shaking table. Li et al. [12] studied the response of batter piles under different base shaking signals and superstructures in unsaturated soils during an earthquake. Liu et al. [13] studied the variation in the pile body response of a pile group foundation under the condition of earthquake liquefaction through centrifuge shaking table testing. Wang et al. [14] studied the influence of the permanent displacement of the sloping ground before and after liquefaction on the horizontal stress of a single pile by centrifuge shaking table testing. Hussien et al. [15] carried out a series of centrifuge experiments, including experiments to determine the dynamic response characteristics of single piles, 3×3 piles and superstructures in sand, and compared the bending moment distributions of piles at different locations. Wang et al. [16] studied the relationship between the displacement- load and displacement-stiffness of suction piles of offshore wind turbines under horizontal loads by applying reciprocating cyclic loads to the piles during the centrifuge test. Liang et al. [17] studied the influence of high and low cap form on the distribution of pile bending moment under the condition of unsaturated soil foundation through centrifuge shaking table testing. Tang et al. [18] proposed a new method for analyzing a pile-soil system using the ground motion parameter index by analyzing the centrifuge shaking table test of a single pile.

In summary, it has become more common to study the seismic behavior of pile foundations in liquefied soil by means of a shaking table test. At the same time, some research results have been achieved by studying the dynamic response of the pile foundation with a centrifuge shaking table. The study of single piles and pile groups under nonliquefied ground conditions and of vertical pile groups under liquefied soil have been preliminarily research. The related seismic damage mechanism requires further exploration. In particular, research on the seismic performance of batter piles under liquefied soil conditions has rarely been conducted in studies using constant 1-g shaking tables or centrifuge shaking tables. Therefore, this paper conducts a detailed study on the seismic liquefaction of saturated sand and the dynamic response a the high-rise platform pile foundation through two sets of centrifuge shaking table tests.

2. Centrifuge shaking table test setup and procedure

2.1 Testing equipment

The device used in this experiment is the Zhejiang University ZJU400 centrifuge [20], which is geotechnical centrifugal test equipment jointly developed by Zhejiang University and the China Academy of Engineering Physics (Fig. 1). The rated load of this device is 400 gt, and the maximum centrifugal acceleration is 150 g. The maximum effective radius of rotation is 4.5 m. In the dynamic test, due to the participation of the vibration table, considering the reliability of the test, the centrifugal acceleration can reach 100 g.

Fig. 1 – ZJU400 geotechnical centrifuge

The shaking table (Fig. 2) was jointly developed by Zhejiang University and Japan Solution Company. Its driving mode is electrohydraulic servo hydraulic drive, which can realize horizontal one-way vibration, with a vibration amplitude of ±6 mm and a frequency range of 10-200 Hz. The maximum vibration acceleration and speed are 40 g and 150 cm/s, respectively. The shaking table has a size of 900 mm×800 mm and a maximum load of 500 kg.

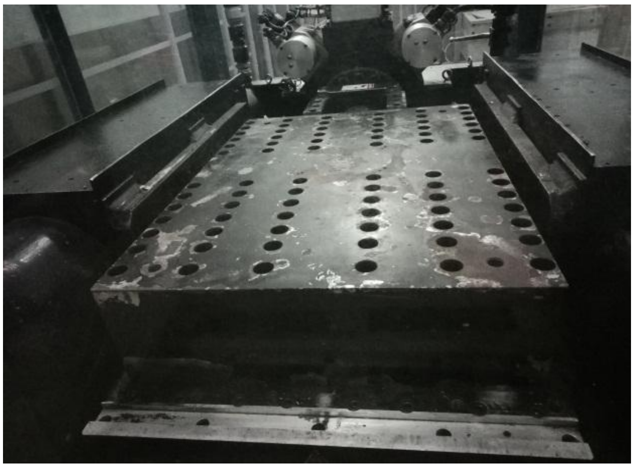

Fig. 2 – Shaking table

Fig. 3 – laminar container

In this test, the influencing factors were comprehensively considered in the selection of the container; ultimately, a laminar container (Fig. 3) was used as the container for the test. The container is made of a laminated aluminum alloy; between the layers of the frame, approximately 2-mm thick bearing couplers are installed. The design concept was proposed by Whitman et al. [20] in 1986. The advantage of this design is that the finite-sized model can be used to simulate the horizontal deformation of the infinite field of soil during vibration excitation to minimize the influence of the boundary effect. The specific parameters are shown in Table 1.

Table 1 – Layered shear container parameters

Parameter	Value
Size of container	730 mm×330 mm×420 mm
Layers in container	12
Thickness of a single layer	35 mm

2.2 Centrifugal model design

Considering the effects of prototype size, test materials and test conditions, the test similarity ratio was determined to be 1:50. The test piles foundation models mainly consider the similarity with bending stiffness. Ultimately, 6061 aluminum alloy is used as the model material, and the detailed dimensions of the model are determined according to the design similarity ratio (Figs. 4-5). The test soil is the common Fujian standard sand: the average particle diameter d_{50} is 0.00016 m, specific gravity is 2.645, maximum void ratio is 0.961, and minimum void ratio is 0.615. The model is prepared by an air pluviation technique. The relative density of the sand is controlled by adjusting the falling sand height to 50%. In the pluviation process, the acceleration and pore pressure transducers are placed in different sand layers according to the design position. After the preparation of the dry sand model is completed, due to the existence of supergravity in the test, 50 units of silicone oil is used instead of water for saturation to consider the influence of the liquid flow rate in the sand, and the entire model is placed in a vacuum box; the model is evacuated and then saturated under vacuum.

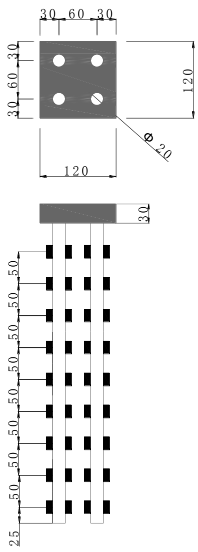

Fig. 4 — Geometric characteristics of the vertical pile (at model scale in mm)

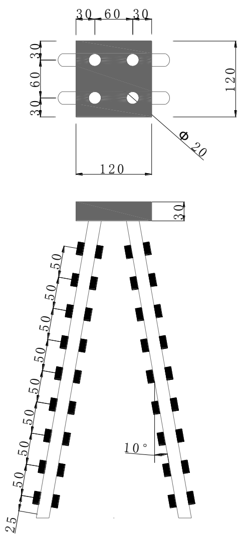

Fig. 5 — Geometric characteristics of the batter pile (at model scale in mm)

The testing was divided into two groups of tests. The test conditions were consistent except for the pile type. In each test, seven accelerometers were set at different depths in the sand, and two accelerometers were arranged on the shaking table surface and on the pile cap. A1 was set on the shaking table. A2 was set on the pile cap, and A2-A8 were embedded in the sand. Seven pore pressure transducers were installed to measure the variation in pore pressure at different depths and inside and outside the pile groups, numbered P1-P7. On the side of the pile cap, a laser displacement transducer, J1, is horizontally arranged along the vibration direction to measure the horizontal displacement of the pile cap during the vibration process. A target is placed on the surface of the sand near the pile, and a laser displacement transducer, J2, is arranged vertically above the target to measure the settlement of the sand during the experiment. Each transducer position and the size of the model are shown in Figs. 6-7. In each set of tests, there are two piles with strain gauges. The specific locations of the strain gauges are given in Figs. 4-5. In this test, the relationship between the output voltage value of the data acquisition system and the bending moment is determined by the bending moment calibration method. Then, the bending moment value of the prototype is obtained by the similarity ratio. For clarity and consistency, all the measurements mentioned below are given in the prototype scale.

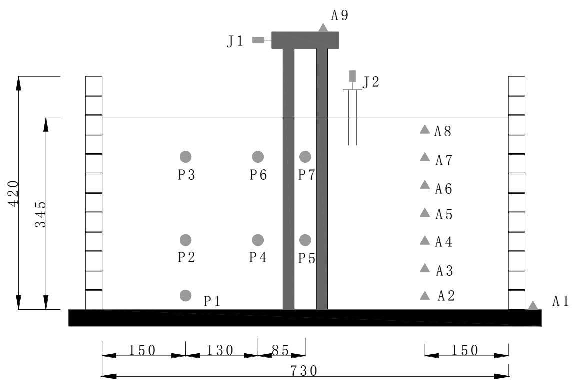

Fig. 6 — Configuration of the sensors in the vertical pile test (at model scale in mm)

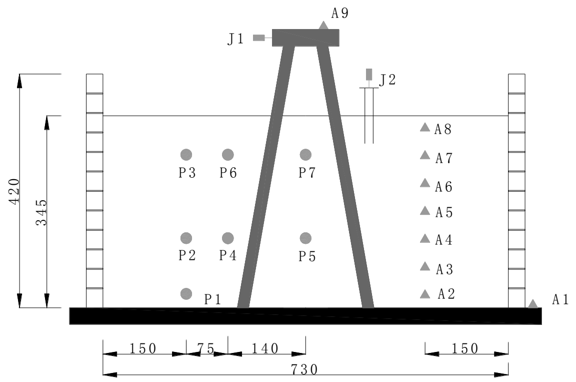

Fig. 7 — Configuration of the sensors in the batter pile test (at model scale in mm)

2.3 Base shaking signals

The main research content of this experiment is the pile body response of vertical piles and batter piles under the action of saturated sand. Therefore, a sine wave input with a relatively simple spectral component is selected. The test conditions are shown in Table 2.

Table 2 – Test conditions

	Type	PGA（g）
Test 1	Sine wave	0.05
Test 2	Sine wave	0.1
Test 3	Sine wave	0.3

The data collected by the shaking table accelerometer (A1) are taken as the real input waveform of the model. After the filtering process, the input waveform is obtained, as shown in Figs. 8. The wave prototype frequency is 1 Hz. Because of its low prototype frequency, the acceleration peak has a certain attenuation after filtering. However, the acceleration signal pattern of the table output is clearly consistent with the signal pattern input by the shaking table controller.

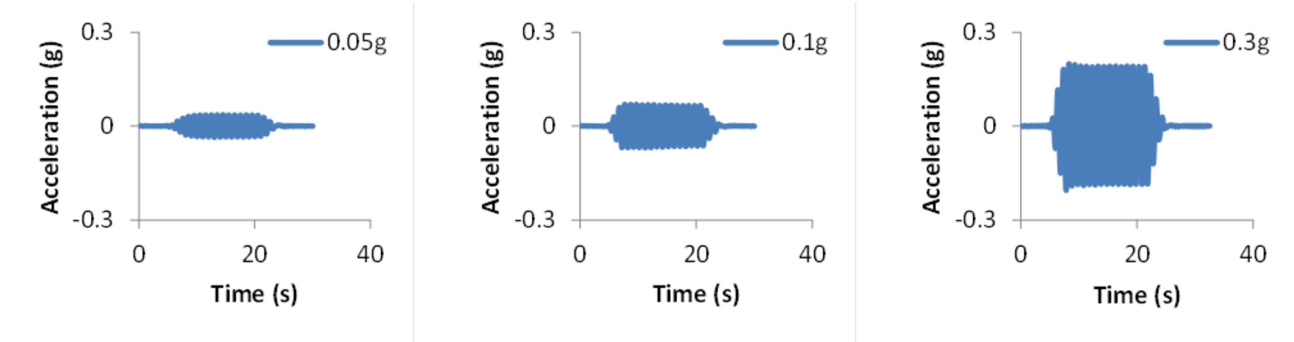

Fig. 8 — Acceleration time histories of the shaking table data with different intensities

3. Results and analysis

3.1 Dynamic response of pile cap

Figs. 9 and 10 show the acceleration time history curves of the caps of vertical and batter piles under different ground motion peaks. Fig. 9 shows that in the cases of 0.05 g and 0.1 g, the acceleration of the vertical pile cap has an obvious amplification phenomenon, and the larger the peak value is, the larger the amplification effect. After the test is finished, the acceleration of the cap will exhibit a certain hysteresis. Fig. 10 shows that the acceleration time history of the batter pile is significantly reduced compared with that of the vertical pile, especially in the case of 0.1 g. In the case of a 0.3 g peak acceleration, the acceleration of the vertical pile cap is basically the same as that of the shaking table in the initial stage, but it quickly decreases, and the amplitude of the cap acceleration is smaller than that of the shaking table until the vibration ends. There is no hysteresis after vibration, and the acceleration of the cap and table simultaneously return to approximately zero. However, the batter pile exhibits a phenomenon of decreasing acceleration at the beginning of the vibration but then rapidly increasing acceleration to the same state as the peak acceleration of the table, which is maintained until the vibration ends.

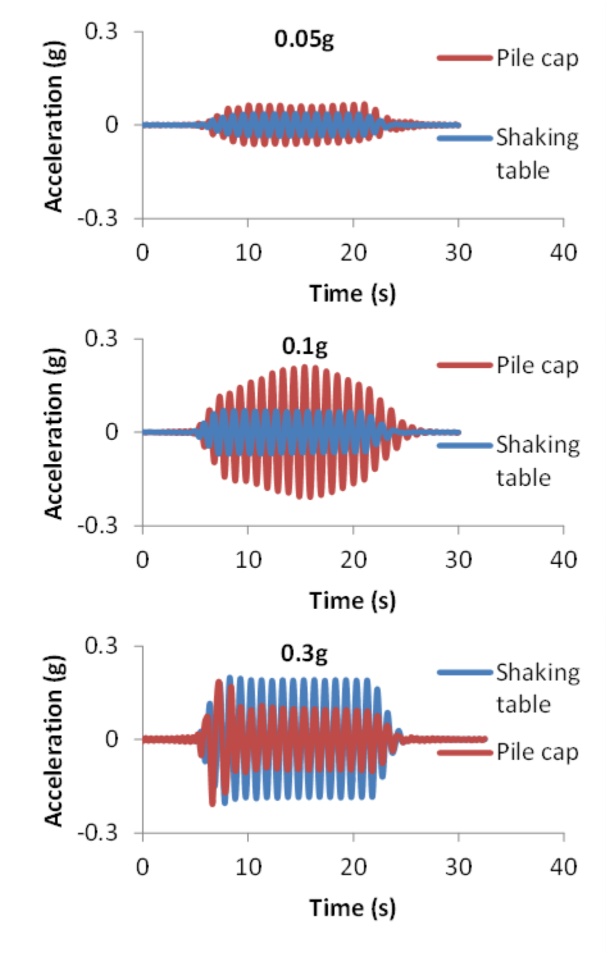

Fig. 9 — Acceleration time histories of different intensities for vertical pile caps

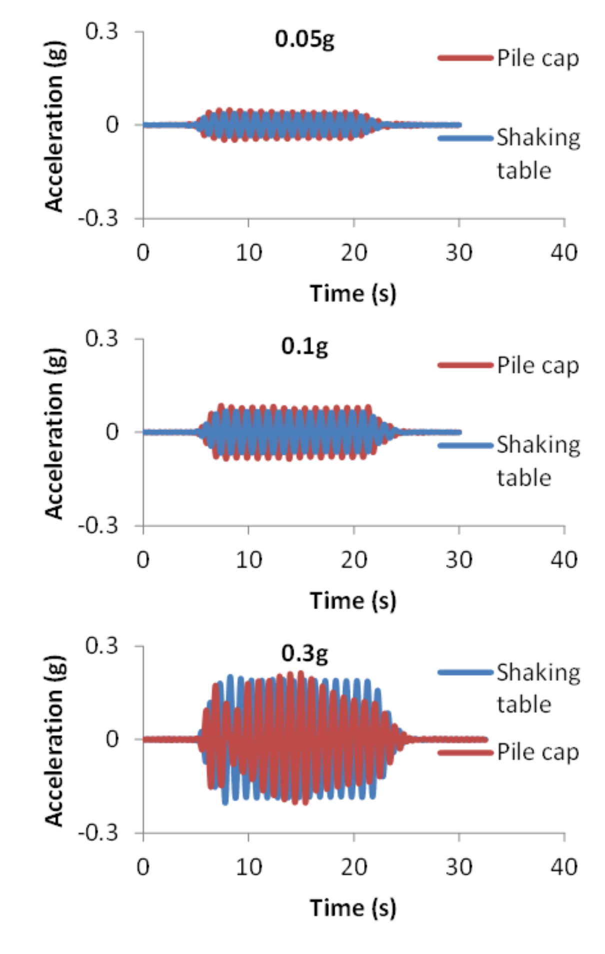

Fig. 10 — Acceleration time histories of different intensities for batter pile caps

The comparison of the above phenomena suggests that in the case of a small peak acceleration, the acceleration of the vertical pile cap will undergo amplification, but in the case of 0.1 g, the amplification is significantly higher than that of 0.05 g, considering the previous acceleration time history. This result may be because the thickness of the liquefied layer is greater in in the case of 0.05 g than that in the case of 0.1 g, which leads to a reduction in pile body restraint. However, the stiffness of the deeper sand around the pile does not significantly decrease for these two conditions, which leads to amplification of the cap acceleration under both two conditions. However, the amplification of cap acceleration with 0.1 g is greater than that with 0.05 g. Under the condition of 0.3 g, the acceleration of the vertical pile cap is clearly smaller than that of the shaking table. Considering the acceleration time history of each sand layer under 0.3 g, it can be found that the different acceleration peaks have been weakened throughout the sand. Thus, the stiffness of the deep sand has also been greatly weakened, and the restraint effect of the soil body is clearly weakened along the whole pile body. In addition, the pile is not rigidly connected with the bottom of the container, thus causing the peak acceleration of the pile cap to appear significantly weakened. However, the acceleration time history curve of the batter pile does not show an obvious amplification phenomenon regardless of the peak acceleration, possibly due to the large horizontal stiffness of the batter pile model; no matter whether the sand is liquefied, the acceleration of the pile cap will not increase.

Figs. 11-12 show that as the peak acceleration increases, the horizontal displacement of the vertical and batter piles tends to increase. However, in the 0.3 g condition, the horizontal displacement of the vertical pile cap rapidly increases at the beginning of the vibration, and then a phenomenon of rapid decline occurs. This variation is similar to the change in the acceleration of the vertical pile cap in the abovementioned 0.3 g case, which verifies the validity of the data. By comparing and analyzing the horizontal displacement of the vertical and batter pile cap under different acceleration peaks, it can be found that the horizontal displacement of the vertical pile cap is larger than that of the batter pile at 0.05 g and 0.1 g; the greater the peak acceleration is, the greater the difference. In the case of 0.3 g, the horizontal displacement of the vertical pile is also significantly larger than that of the inclined pile in the initial stage of vibration. However, in the subsequent process, the horizontal displacement of the batter pile cap is basically maintained in a stable state until a gradual decreasing trend begins in the late stage. This displacement phenomenon is also consistent with the acceleration time history described in the previous section. The reason for the above phenomenon is similar to the abovementioned reason for the change in the acceleration of the pile cap: due to the variation in sand liquefaction depth under different peak accelerations, the constraint on the pile body changes, resulting in the observed test phenomenon.

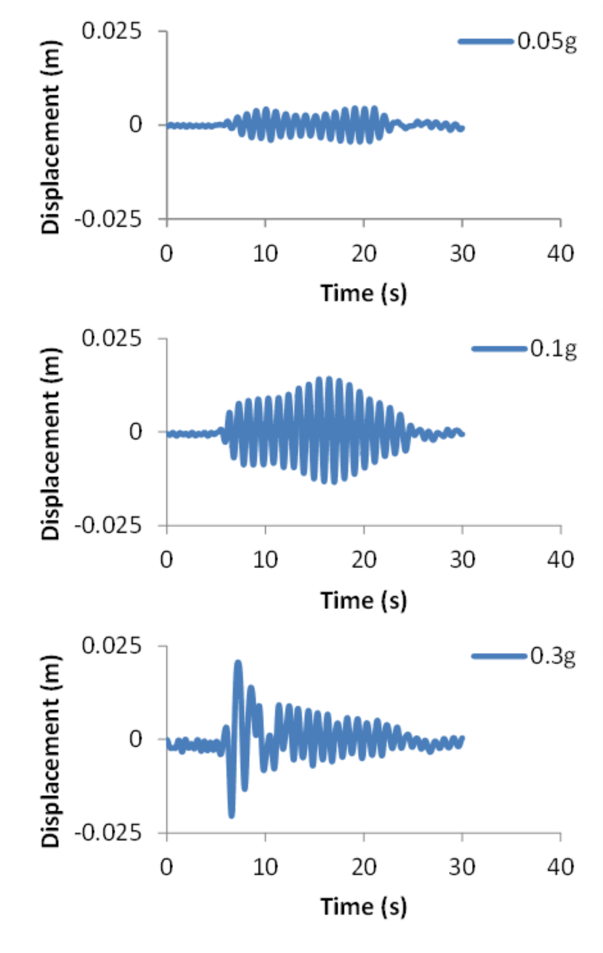

Fig. 11 — Displacement time histories of different intensities for vertical pile caps

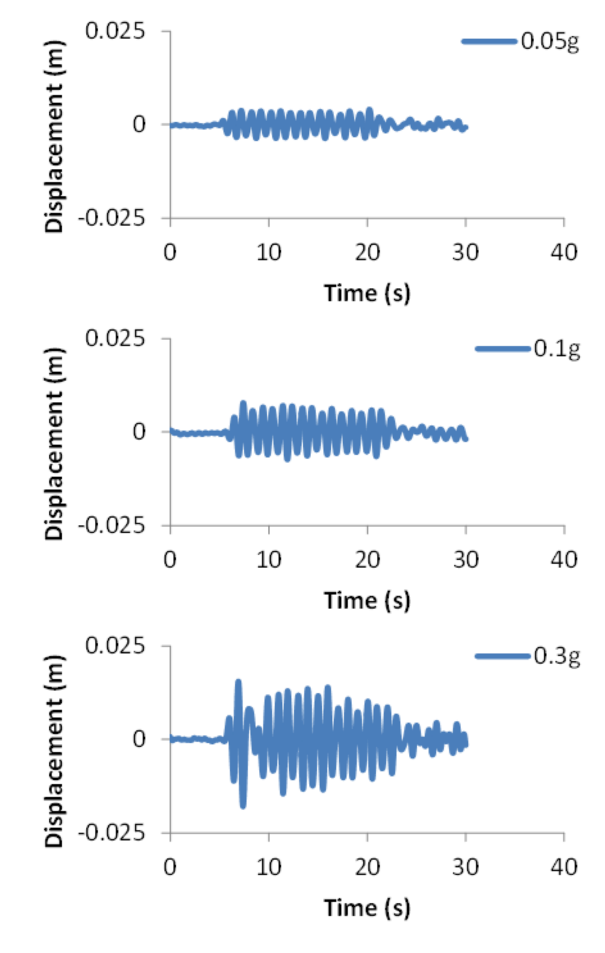

Fig. 12 — Displacement time histories of different intensities for batter pile caps

The comprehensive analysis of the acceleration and displacement changes in the cap suggests that the response strength of the high-rise pile cap under dynamic load has a very important relationship with the degree of sand liquefaction. Especially in the case of fully liquefied soil, the dynamic response of the superstructure is quite different from that of the soil that is not fully liquefied. However, the test results show that in most cases the dynamic response of the batter pile cap is not significantly amplified. Therefore, it can be concluded that the use of a high-rise batter pile cap exhibits obvious advantages for reducing the dynamic response of the superstructure.

3.2 Pile bending moment

In each set of tests, nine pairs of strain gauges (18 total) are symmetrically arranged along the adjacent two pile bodies, and the specific positions are given above. Figs. 13-15 show the bending moment envelope of the two piles under different vibration intensities. 'VS' stands for vertical pile and 'BS' stands for batter pile. VS1 (BS1) and VS2 (BS2) represent the bending moments of the two piles on the same side. No data were collected for the second pair of upper strain gauges of VS2 due to pile damage. The figures clearly show that the positive and negative bending moment envelopes of the pile foundations on both sides are basically the same, and the reliability of the experimental data is verified.

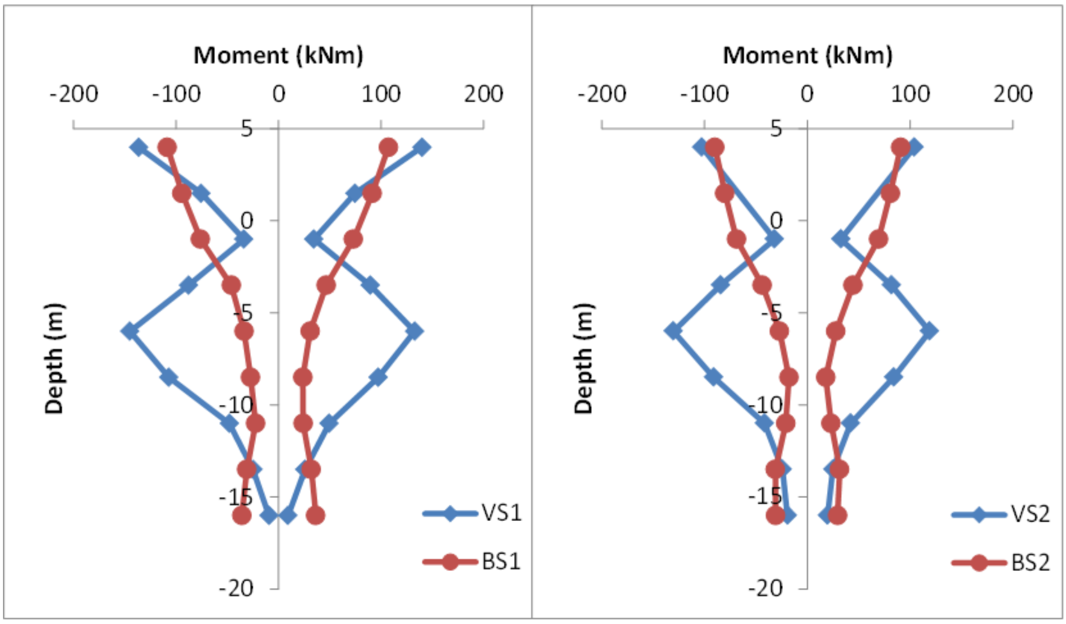

Fig. 13 — Maximum positive and negative bending moment envelopes of vertical and batter piles for 0.05 g

Fig. 13 shows that at 0.05 g, the bending moments of the vertical pile exhibit a minimum at the pile toe and continuously increase upward. The bending moment reaches a peak at the position of 1/3 of the depth of the foundation, and with decreasing depth, the bending moment decreases gradually, reaching a small bending moment at the soil surface; then, the bending moment increases toward the pile tip. The bending moments are basically the same at the pile head and 1/3 of the sand depth. The increase in the bending moment of the pile above the soil surface is due to the inertia of the pile cap in the vibration process. At the same time, the pile foundation is not bound by the soil, so the bending moment increases significantly at the pile head. This phenomenon is basically consistent with the conclusions of shaking table tests that have been completed in our research group [21]. Additionally, under the condition of 0.05 g, the batter pile exhibits the minimum bending moment at the position midway between the soil surface and foundation, and the bending moment of the pile increases continuously with the decrease in the burial depth. The bending moment envelope of the batter pile is quite different from that of the vertical pile in the case of 0.05 g, especially at the position of 1/3 of the burial depth. However, the bending moment of the batter pile near the joint between the pile and the cap also undergoes a large increase. In particular, in the vicinity of the soil surface, the bending moment of the batter pile is larger than that of the vertical pile; however, in other places, the bending moment of the batter pile is smaller than that of the vertical pile.

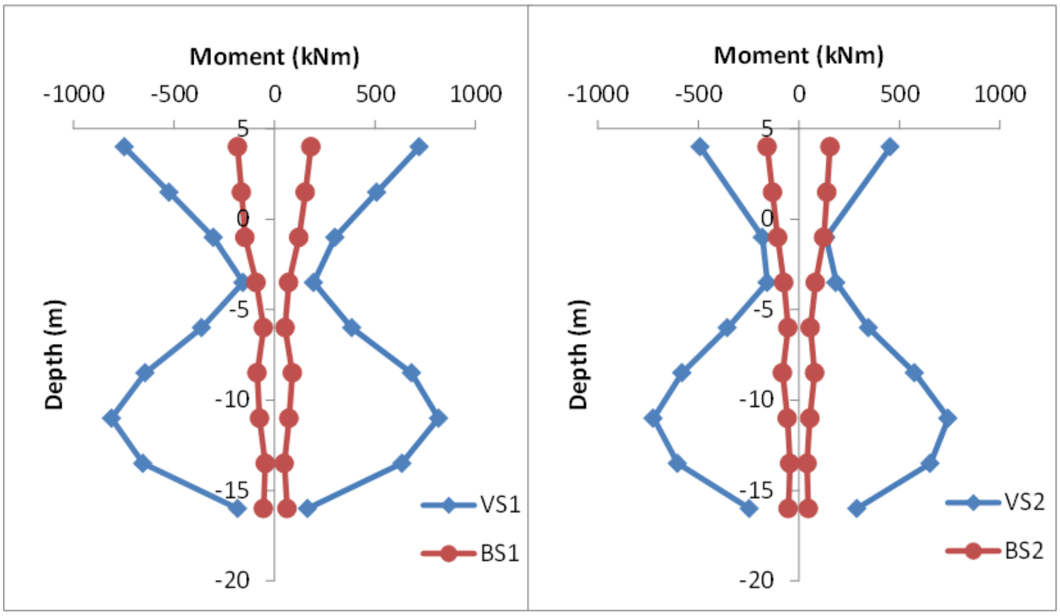

Fig. 14 — Maximum positive and negative bending moment envelopes of vertical and batter piles for 0.1 g

Fig. 14 shows that in the case of 0.1 g, the maximum bending moment of the vertical pile group is significantly increased. The bending moment of the vertical pile toe is clearly increased and is basically the same as that near the soil surface. The maximum positive and negative bending moments of the vertical pile still occur in the pile head and the foundation soil. Notably, the position of the maximum bending moment of the vertical pile in the soil shifts down relative to the abovementioned results, moving from 1/3 of the burial depth to 2/3 of the burial depth. There is no obvious change in the shape of the bending moment envelope of the batter pile for the 0.05 g case, but the bending moment of the whole pile body is increased. However, the difference between the 0.05 g test and 0.1 g test is that the bending moment of the whole batter pile body is smaller than that of the vertical pile in the 0.1 g test.

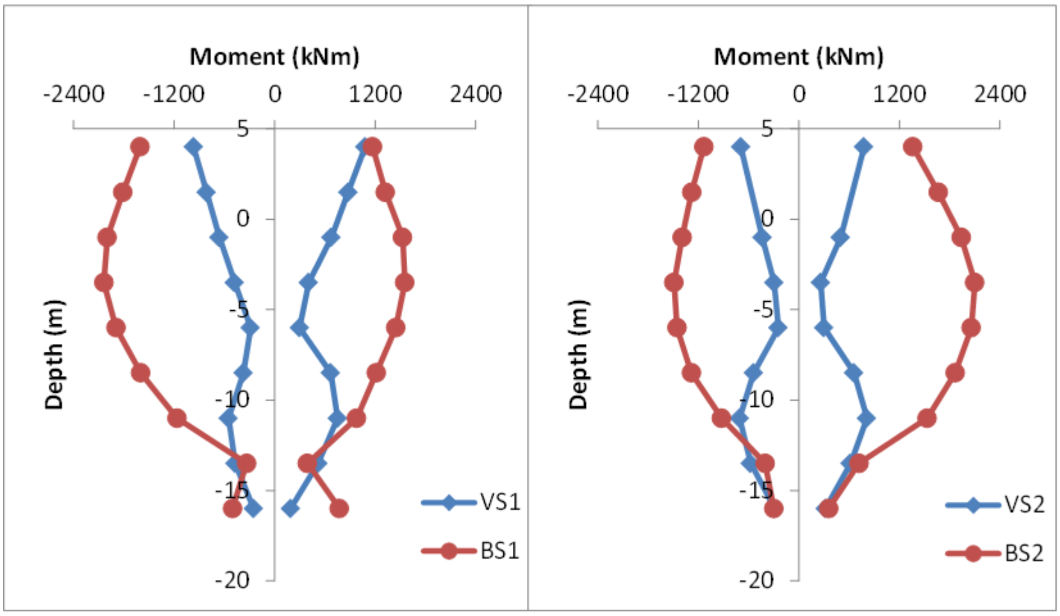

Fig. 15 — Maximum positive and negative bending moment envelopes of vertical and batter piles for 0.3 g

Fig. 15 shows the maximum positive and negative bending moment envelopes of the vertical and batter piles in the case of 0.3 g. The bending moments at each position of the pile body clearly increase, and the shape of the overall envelope of the vertical pile is basically the same as those for the first two working conditions. However, the bending moment envelope of the batter pile exhibits obvious differences due to its wine-glass shape. The bending moment of the pile toe is small, and the maximum bending moment increases with decreasing burial depth. After a "bulk belly" appears near the soil surface, the maximum bending moment gradually decreases. This results is different from those under the previous conditions, where the maximum bending moment occurs near the joint between the cap and the pile foundation. Notably, under the condition of 0.3 g, the bending moment envelope of the batter pile is larger than that of the vertical pile, except near the pile head and toe.

The above analysis shows that as the vibration intensity increases, the bending moment of the pile body of any pile type increases greatly. However, the distribution of the bending moment for the vertical and batter pile are quite different. The position of the maximum bending moment along the vertical pile shifts downward along the burial depth with the increase in the vibration intensity. However, the distribution of the bending moment of the batter pile does not changed substantially, and the lower end has a small bending moment, while the upper end has a large bending moment. In particular, in the case of 0.3 g, the bending moment envelope of the batter pile is clearly larger than that of the vertical pile. These phenomena may be due to the increases in vibration intensity, causing the pore pressure ratio at different depths in the soil to increase, especially in the deeper soils. Although the deep soil does not reach the liquefaction state, the apparent increase in the pore pressure ratio leads to a significant decrease in the stiffness of the soil, and the constraint that the soil applies to the pile body is clearly weakened. The influence of the inertia of the pile cap on the bending moment is clearly enhanced, which eventually leads to the above phenomenon. Because the horizontal stiffness of the batter pile is high, the influence of soil liquefaction on the bending moment is much smaller than that of the vertical pile.

4. Conclusions

In this paper, the dynamic response of pile cap under several vibration intensities are studied by using a centrifuge shaking table. Meanwhile, the analysis of the bending moment of the vertical and batter pile groups under soil liquefaction draws the following conclusions:

The influence of pile type on the acceleration and displacement of the upper cap is obvious. The acceleration and displacement of the batter pile cap do not show obvious amplification under all the tested working conditions. Therefore, for the pile group with a high pile cap, the form of a batter pile can reduce the acceleration and displacement amplification effect of the upper structure to some extent.

The bending moments of the vertical and batter pile groups are large at the pile heads. However, in addition to the large bending moment at the top of the pile, the vertical piles have larger bending moments at different depths under different vibration intensities. Therefore, when considering the seismic performance of vertical piles, it is necessary to design the pile foundation according to the distribution of the maximum bending moment of piles in different situations. Under certain circumstances, the design of a batter pile must consider only the bending moment near the pile head. At the same time, different peak accelerations lead to changes in the depth of liquefaction, greatly influencing the distribution of the bending moment along different pile types. In the case of 0.3 g, the depth of liquefaction increases, which results in the clearly larger bending moment envelope of the batter pile than that of the vertical pile. However, in the cases of 0.05 g and 0.1 g, the bending moment envelopes of the batter pile are clearly smaller than those of the vertical pile.

6. References

[1] Miyajima M, Kitaura M, Ando K (1991): Experiments on liquefaction-induced large ground deformation. *Proceedings of the Third Japan-U.S. Workshop on Earthquake Resistant Design of Lifeline Facilities and Counter measures for Soil Liquefaction*, Buffalo, New York. p. 269-278.

[2] Choobbasti A J, Saadati M, Tavakoli H R (2012): Seismic response of pile foundations in liquefiable soil: parametric study. *Arab J of Geosci*, 5(6), 1307-1315.

[3] Samui P, Bhattacharya S, Sitharam T G (2012): Support vector classifiers for prediction of pile foundation performance in liquefied ground during earthquakes. *Int J Geotech Earthq Eng*, 3(2), 42-59.
[4] Su L, Tang L, Ling XZ, Liu CH, Zhang XY (2016): Pile response to liquefaction-induced lateral spreading: a shake-table investigation. *Soil Dyn Earthq Eng*, 82, 196-204.
[5] Li GJ, Motamed R (2017): Finite element modeling of soil-pile response subjected to liquefaction- induced lateral spreading in a large-scale shake table experiment. *Soil Dyn Earthq Eng*, 92, 573-584.
[6] Ling XZ, Guo MZ, Wang DS, Wang C, Wang LX, Wang ZQ (2006): Large-scale shaking table model test of seismic response of bridge of pile foundation in ground of liquefaction. *Rock and Soil Mech*, 27(1), 7-11,22.
[7] Huang ZF, Wang XY, Wu ZA, Bai XH (2013): Shaking table test for single pile-soil dynamic interaction in liquefied foundation soil. *J Vib Shock*, 31(20), 189-192.
[8] Panah AK, Mashhoud HJ, Yin JH, Leung YF (2018): Shaking table investigation of effects of inclination angle on seismic performance of micropiles. *Int J Geomech*, 18(11), 04018 142
[9] Cengiz C, Guler E (2018): Shaking table tests on geosynathetic encased columns in soft clay. *Geotext Geomembranes*, 46, 748-758.
[10] Yang ZH, Zhang XY, Yang RL, Zhou XM, Niu FJ (2018): Shake table modeling of pile foundation performance in laterally spreading frozen ground crust overlying liquefiable soils. *J Cold Reg Eng*, 32(4), 04018012.
[11] Su D, Li XS (2006): Centrifuge investigation on seismic response of single pile in liquefiable soil. *Chin J Geotech Eng*, 28(4), 423-427.
[12] Li Z, Escoffier S, Kotronis P (2016): Centrifuge modeling of batter pile foundations under earthquake excitation. *Soil Dyn Earthq Eng*, 88, 176-190.
[13] Liu X, Wang R, Zhang JM (2018): Centrifuge shaking table tests on 4×4 pile groups in liquefiable ground. *Acta Geotech*, 13(6), 1405-1418.
[14] Wang R, Zhang JM, Zhang G (2012): Centrifuge shaking table test on single pile in lateral spreading soil. *Eng Mech*, 29(10), 98-105.
[15] Hussien MN, Tobita T, Iai S, Karray M (2016): Soil-pile-structure kinematic and inertial interaction observed in geotechnical centrifuge experiments. *Soil Dyn Earthq Eng*, 89, 75-84.
[16] Wang XF, Yang X, Zeng XW (2017): Centrifuge modeling of lateral bearing behavior of offshore wind turbine with suction bucket foundation in sand. *Ocean Eng*, 139, 140-151.
[17] Liang FY, Chen HB, Huang MS, Sun LM (2016): Model test on seismic response of superstructure and pile group. *J Build Struct*, 37(9), 135-141.
[18] Hui SQ, Tang L, Zhang XY, Wang YQ, Ling XZ, Xu BW (2018): An investigation of the influence of near-fault ground motion parameters on the pile's response in liquefiable soil. *Earthq Eng Eng Vib*, 17(4), 729-745.
[19] Chen YM, Han C, Ling DS, Kong LG, Zhou YG (2011): Development of geotechnical centrifuge ZJU 400 and performance assessment of its shaking table system. *Chin J Geotech Eng*, 33(12), 1887-1894.
[20] Whitman RV, Lambe PC (1986): Effect of boundary conditions upon centrifuge experiments using ground motion simulation. *Geotech Test J*, 9(2), 61-71.
[21] Li YR, Zhang YL, Chen ZS, Zhang YB (2018): Dynamic response and *p-y* curve of symmetric inclined piles in liquefied soil. *Chin J Rock Mech Eng*, 37(1), 239-250.

