Shaking Table Tests on Model Pile in Saturated Sloping Ground

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SUMMARY:
For the study of the soil-structure interaction, including the effect of lateral spreading, in a saturated sloping ground during earthquake, shaking table tests on a model pile in an inclined large biaxial laminar shear box filled with saturated clean sand were conducted at the National Center for Research on Earthquake Engineering (NCREE), Taiwan. The pile tip was fixed at the bottom of the shear box to simulate the condition of a pile foundation embedded in a firm stratum. The pile top was mounted with 6 steel disks to simulate the superstructure. Input shakings including sinusoidal and recorded earthquake accelerations were imposed perpendicularly or parallel to the slope direction. Strain gauges and accelerometers were placed on the pile surface to obtain the response of the pile under shaking. The near- and far-field soil responses, including pore water pressure changes, accelerations, and settlements were also measured. Lateral spreading displacement of the soil and pile behavior were observed and evaluated while soil liquefaction was triggered under shakings in either direction.

Keywords: shaking table test, liquefaction, lateral spreading, pile

1. INTRODUCTION

Pile foundations have suffered extensive damages within liquefiable and laterally spreading ground in many large earthquakes such as the 1964 Niigata Earthquake, the 1989 Loma Prieta Earthquake, the 1995 Kobe Earthquake, the 1999 Chi-Chi Earthquake, the 2011 Christchurch Earthquake and the 2011 Great East Japan Earthquake. Previous studies on soil-pile interactions were conducted in order to understand the mechanism of the dynamic loading on the piles (soil-pile interaction) and their responses under earthquake loading. Lateral loading tests in the field or in the laboratory and shaking table tests on model piles within soil specimens, under either 1 g or centrifugal condition, have been used to investigate the pile behaviors and soil-pile interaction in saturated soil (e.g. Dobry et al. 2003, Tokimatsu et al., 2005, Ashford et al., 2006, Brandenberg et al., 2006, Cubrinovski et al., 2006, Madabhushi et al., 2010, Ueng and Chen, 2010). The results of these studies, including failure mechanisms, bending moments of pile in laterally spreading ground, pore water pressure variation around the piles, p-y relations for soil-pile interaction, and pile cap effect, can provide information on performance criteria for aseismic design of structures with pile foundations.

However, there are still uncertainties concerning the soil-pile interaction issues in laterally spreading ground, including (1) the kinematic loading on pile foundation due to lateral spreading; (2) the relation between the lateral pressure on pile foundation and the ground displacement; and (3) the transient responses of the surrounding soil and pile during soil liquefaction. In this study, a large biaxial laminar shear box developed at the National Center for Research on Earthquake Engineering (NCREE) was used as the soil container and an instrumented aluminum model pile was installed inside the shear box filled with saturated sand. The biaxial shear box with the model pile in a saturated sloping ground was placed on 1 g shaking table and the sinusoidal and recorded earthquake accelerations were applied perpendicularly or parallel to the slope direction. The soil and pile responses and their interaction, including the inertial and kinematic actions on the model pile, under these types of shakings were studied.

2. SAND SPECIMEN AND MODEL PILE

The biaxial laminar shear box is composed of 15 layers of sliding frames. Each layer consists of two nested frames, an inner frame (1880 mm × 1880 mm) and an outer frame (1940 mm × 2340 mm). Both frames are made of a special aluminum alloy with 30 mm in thickness and 80 mm in height, except the uppermost layer that has a height of 100 mm. These 15 layers of frames are separately supported on the surrounding rigid steel walls with a gap of 20 mm between adjacent layers. The 20-mm gap is provided to avoid rupture of the rubber membrane inside the box during a large relative deformation between layers. Thus, a sand specimen of 1880 mm × 1880 mm × 1520 mm can be placed inside the inner frames. A 2-mm thick silicone rubber membrane was placed inside the box. The silicone rubber membrane was fixed at the top and bottom of the shear box.

Linear guideways consisting of sliding rails and bearing blocks are used to allow an almost frictionless horizontal movement without vertical motions. Each outer frame is supported by the sliding rails built on two opposite sides of the outer rigid walls. The bearing blocks on the outer frame allow its movement in the X-direction with minimal friction. Similarly, sliding rails are also provided for each outer frame to support the inner frame of the same layer such that the inner frame can move in the Y-direction (perpendicular to X-direction) with respect to the outer frame. With these 15 nested layers of inner and outer frames supported independently on the rigid walls, the soil at each depth can move multi-directionally in the horizontal plane without torsion.

A clean fine silica sand (G_s = 2.65, D_{50} = 0.31 mm) from Vietnam was used in this study for the sand specimen inside the laminar shear box. This sand has been used in the shaking table tests for liquefaction studies at NCREE, Taiwan (Ueng et al., 2006). The representative grain size distribution of the sand in this test is shown in Figure 1. The maximum and minimum void ratios are 0.918 and 0.631, respectively, according to ASTM D4253 Method 1B (wet method) and ASTM D4254 Method A. The sand specimen was prepared using the wet sedimentation method after placement of the model pile and inside instruments in the shear box. The sand was rained down into the shear box filled with water to a pre-calculated depth. The size of the sand specimen is 1.880 m × 1.880 m in plane and about 1.31 m in height before shaking tests. The relative density of the sand is about 10 %. Details of biaxial laminar shear box and the sand specimen preparation were described in Ueng et al. (2006).

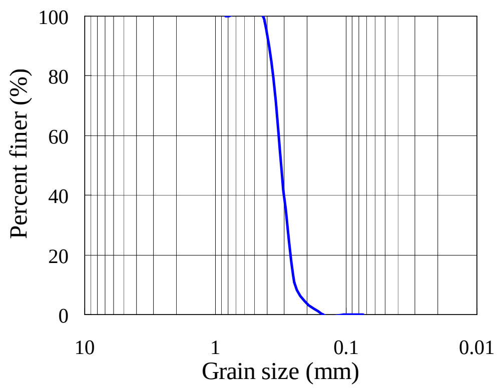

Figure 1. Grain size distribution of Vietnam sand

The model pile was made of an aluminum alloy pipe, with a length of 1600 mm, an outer diameter of 101.6 mm, a wall thickness of 3 mm and its flexural rigidity, EI = 75 kN·m^2, was obtained by flexural test. Strain gauges and mini-accelerometers were placed at different locations to respectively measure bending strains and accelerations along the model pile. The shear box was inclined 2° to the horizontal, simulating a mild infinite slope. The sloping direction of this model was X direction as shown in Figure 3. The pile was fixed vertically at the bottom of the shear box. Hence, this physical model can be used to simulate the condition of a vertical pile embedded in sloping rock or within a sloping firm soil stratum. In addition, 6 steel disks, 226.14 kg, were attached to the top of the model pile to simulate the superstructure. The model pile was instrumented and installed inside the shear box before preparation of the sand specimen, as shown in Figure 2.

Figure 2. The instrumented aluminum pipe was fixed at the bottom of shear box

3. INSTRUMENTATION

Two magnetostriction type linear displacement transducers (LDTs) were set up to measure X- and Y-displacements of the pile top. They were mounted to the reference frames outside the shaking table. The waterproof resistance-type strain gauges were placed on the pile surface to measure the bending strains of the model pile. There are 10 different depths with 15-cm spacing along the pile axis as shown in Figure 3. At each depth, two pairs of strain gauges were attached on opposite sides of the pile in X- and Y-directions. Vertical acceleration arrays along the pile were also installed on the model pile in X- and Y-directions for acceleration measurements. In addition, in order to observe the build-up and dissipation of the pore water pressures and accelerations in the sand specimen (near field and far field), mini-piezometers and mini-accelerometers were installed inside the box at different locations and depths. Additionally, for the measurement of lateral movement of soil profile during the shaking, two shape arrays were installed at both uphill and downhill locations in the specimen (Figure 3). The shape array (FAAs) consists of 8 rigid segments separated by special flexible joints. Each segment with a MEMS gravity sensor can measure tilt and acceleration along three axes so that shape array can measure dynamic profile during the test period.

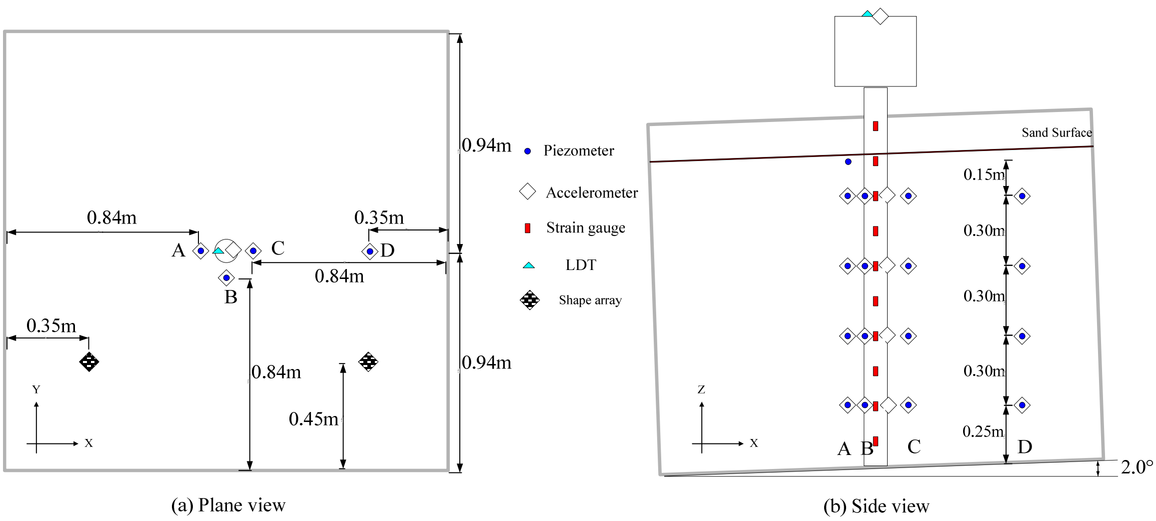

Figure 3. Instrumentation on the model pile and within the sand specimen

The responses of 15 layers of inner and outer frames at different depths of the laminar shear box were also recorded to evaluate the ground motion and liquefaction depth of the sand specimen using linear displacement transducers and accelerometers.

4. SHAKING TABLE TESTS

Shaking table tests were first conducted on the model pile without the sand specimen in order to evaluate the dynamic characteristics of the model pile itself. Sinusoidal and white noise accelerations with amplitudes from 0.03 to 0.05 g were applied in X and/or Y directions. In the two-dimensional (multidirectional) sinusoidal shaking, there is a 90° phase difference between the input acceleration in X and Y directions, i.e., a circular or ellipse motion was applied. The model pile in saturated sloping ground was then tested under one dimensional sinusoidal (1-8 Hz) and recorded accelerations during Chi-Chi Earthquake and Kobe Earthquake with amplitudes ranging from 0.03 to 0.15 g. Input motions were mainly imposed perpendicularly to the slope direction for the study of kinematic effect on the pile foundation (lateral spreading force) independently, and also tested in another direction parallel to the slope to investigate the resultant force on the pile foundation including the inertial and kinematic effect. White noise accelerations with amplitude of 0.03 g were also applied in both X- and Y-directions to evaluate the dynamic characteristics of the model pile within soil and the sand specimen. Figure 4 shows a picture of shaking table test on the model pile with 6 steel disks on its top. The height of the sand surface after each shaking test was measure to compute the settlement and density of the sand specimen. Soil samples were taken using short thin-walled cylinders at different depths and locations after completion of the shaking tests to obtain the densities of the sand specimen.

Figure 4. A model pile with 6 steel disks on its top in saturated sloping ground on the shaking table

5. TEST RESULT

5.1. Dynamic characteristics of the model pile

Shaking table tests on the model pile without sand specimen were conducted to evaluate the dynamic characteristics of the model pile itself. The behavior of model pile on the shaking table can be regarded as a single-degree viscously damped system. Hence, the amplification curve was obtained from the Fourier spectral ratio of the measured acceleration of the pile top to that of the input motion. The predominant frequency of the model pile with 6 steel disks on its top was identified at about 2.1 Hz as shown in Figure 5. Table 1 lists the predominant frequencies of the model pile according to the test data. The damping ratio of the model pile ranges from 0.7 % to 1.2 % obtained by the back-calculations of free vibration and forced vibration.

Table 1. Predominant frequencies of the model pile

Mass on pile top	Aluminum pile
	Freq., Hz
No mass	22.9
6 steel disks	2.1

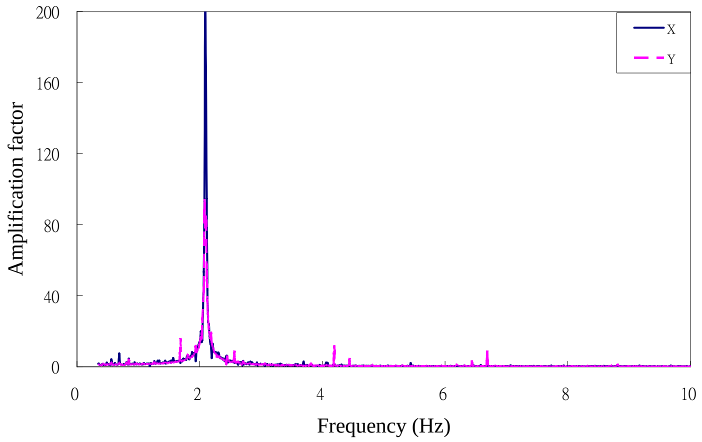

Figure 5. Amplification factor vs. frequency for the model pile

5.2. Dynamic characteristics of soil and soil-pile system under small amplitude of shakings

The dynamic characteristics of soil stratum and soil-pile system were evaluated by a series of shaking table tests on the model pile within the saturated sand specimen with small amplitude. Table 2 lists the predominant frequencies of the soil and the soil-pile system for the model pile (with 6 steel disks on its top) in soil of various relative densities. It can be seen that the predominant frequency of soil increases with the relative density, but that of pile increase slightly with the relative density. In addition, the predominant frequency of soil-pile system is significantly lower than that of the soil specimen. Comparing the predominant frequencies of the model pile without and within soil specimen (Table 1 and Table 2, respectively), one can find that, except for the case without mass on the pile top, the predominant frequencies of the model pile in the soil specimen were higher than those without soil due to the constraint of the soil on the pile. Hence, the response of the pile was mainly governed by the inertia force from the 6 steel disks.

Table 2. Predominant frequencies of the soil and the aluminum pile in soil of different relative densities

Soil density, Dr %	Soil Freq., Hz	Pile in soil Freq., Hz
11.9	10.92	4.61
26.0	11.7	4.64
42.4	12.7	4.65
70.1	13.8	4.67

5.3. Responses of model pile and soil under laterally spreading

A shaking table test under one-dimensional sinusoidal acceleration with frequency of 8 Hz and amplitude of 0.068 g in Y direction (i.e. the input motion was imposed perpendicularly to the slope direction) was conducted to study the kinematic effect on the pile foundation in a saturated sloping ground with relative density of 13.6 %. The depth of liquefaction was determined based on the measured pore water pressures in the sand specimen and accelerometers on the frames (Ueng et al., 2010). In this test, the liquefied depth of the sand specimen reached about 112.6 cm. Figure 6 shows a distinct lateral spreading displacement after the shaking. (Compared with Figure 4)

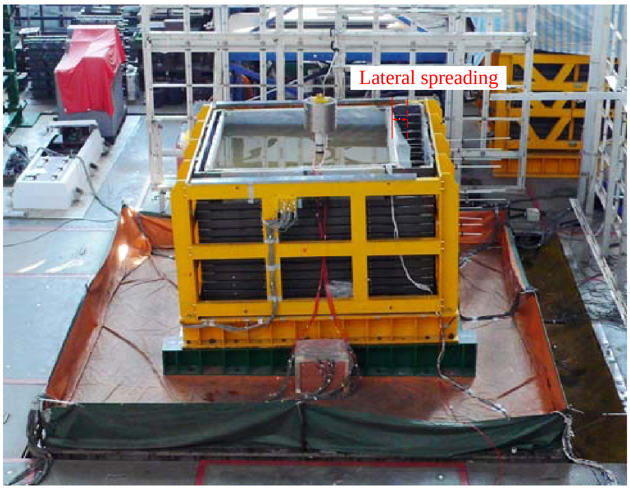

Figure 6. Liquefaction-induced lateral spreading displacement in X direction

Four piezometers arrays were placed in the sand specimen, around the pile and in the free field. (Figure 3) Figure 7 shows the time histories of excess pore water pressure ratios (r_u) near the model pile and in free-field location. It can be observed that the excess pore water pressures at a shallower depth were firstly liquefied at about 2.8 seconds, and then those at the greater depth were also liquefied at around 3.6 seconds. The observation can indicate that liquefaction is progressive from the shallower depth to the greater depth of the specimen in a short period during the shaking. In addition, the pore water pressures at a shallower depth exhibit an impermanent reduction phenomenon near the model pile, especially in the downslope side. At the greater depth, the pore water pressures do not present such kind of responses. This might be attributed to dilative behaviour of the soil due to a larger shear strain under low confinement. The trends of pore water pressure changes are similar to those obtained in the centrifuge tests by others, e.g., Haigh and Madabhushi. (2005).

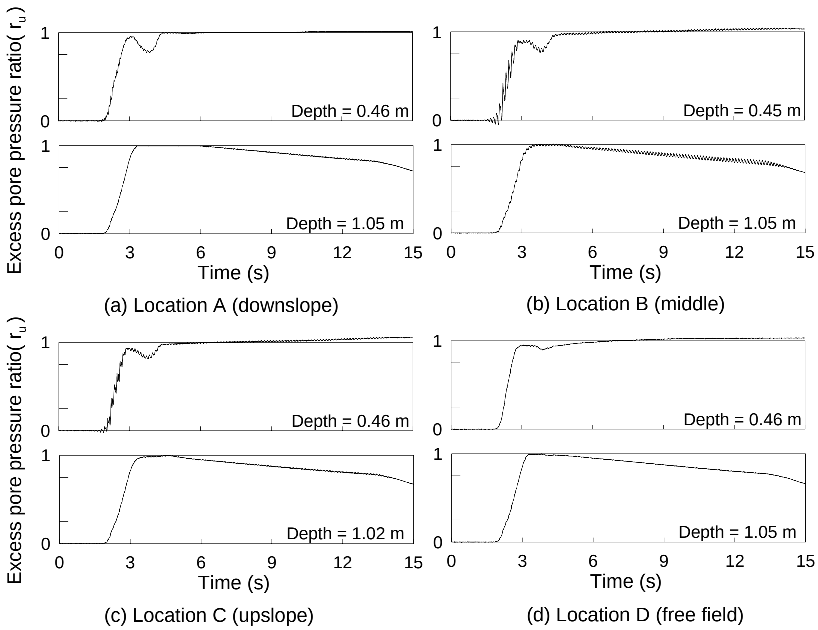

Figure 7. Excess pore water pressure time histories at various locations

Figure 8 shows the measured acceleration time histories on the model pile and those of free-field soil at various depths in the sand specimen. It can be seen that the accelerations of pile and soil increase slightly with height due to the upward shear wave propagation in Y direction, and the responses of pile and soil in X direction are quite small. It was also found that the maximum acceleration of the pile occurred just before liquefaction of the sand specimen. After liquefaction, the acceleration of the pile top reduced and remained steady while the acceleration of the soil diminished. This phenomenon can be interpreted as that the stiffness of the soil nearly vanished when the specimen was liquefied.

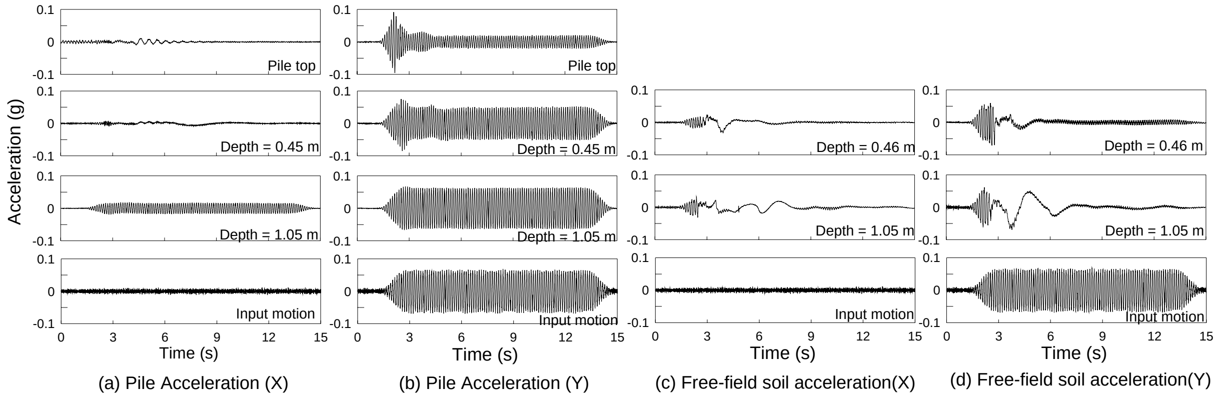

Figure 8. Acceleration time histories of the model pile and free-field soil at various depths

The time histories of relative displacement and trajectory of the pile top were shown in Figure 9. The X-displacement of the model pile is mainly caused by lateral spreading, and the other direction is induced by the shaking. Hence, the force exerted on the model pile due to lateral spreading (X direction) can be extracted from this kind of test. It is also observed that the pile response in X direction can be divided into three stages during the shaking. (i) The first stage is only small movement and rebound during 2.2 to 3 seconds. The response of pile is due to the small amount of movement and softening at the shallower depth of soil. (ii) The second stage, pile has the maximum displacement and rebound again during 3 to 4.2 seconds. In this stage, the model pile suffered the majority of liquefaction-induced lateral ground displacement (Figure 10), and it had a maximum displacement at about 3.618 seconds. After this time, the pile rebounded again because of the reduction of lateral force on the pile when the specimen was liquefied. (iii) Pile response at this stage demonstrated a free vibration motion during 4.2 to 8 seconds. The predominant frequency of the acceleration on the pile top in X direction is about 2 Hz. Comparing this result with the predominant frequency of the model pile without soil specimen (Table 1), one can find that the predominant frequency of the model pile within liquefied soil was almost the same as that of model pile without soil specimen. This inferred that the stiffness of the soil nearly vanished when soil liquefaction occurred.

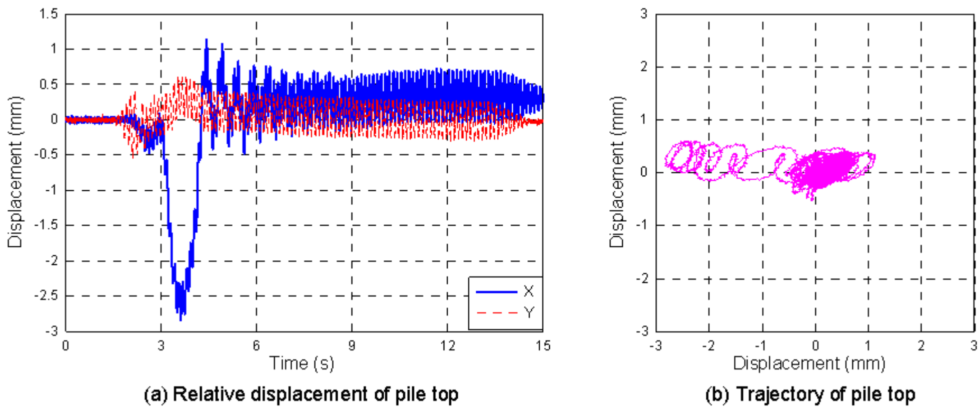

Figure 9. The time histories of relative displacement and trajectory of the pile top

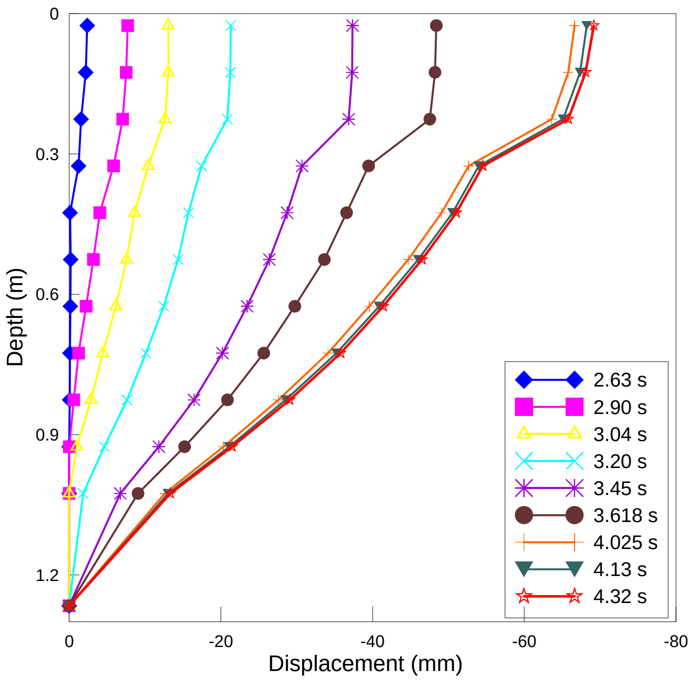

Figure 10. Profiles of free-field ground at various times in X direction

6. CONCLUSIONS

Shaking table tests were conducted on an aluminum model pile in saturated sloping ground by using the biaxial laminar shear box. Analyses of the dynamic behavior of the model pile and the soil stratum were conducted during the shaking tests according to observations. Lateral spreading displacements of the soil and pile behavior were observed while soil liquefaction was triggered under shakings. It was also found that the kinematic loading on the model pile due to lateral spreading during shaking can be separated individually in a suitable by using the biaxial shear box. Further analyses of the test data will be performed to obtain more information on the relation between the mobilized lateral pressure on the pile and the ground movement and the evaluation of bending moment of pile in design.

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