Int. J. of GEOMATE, June, 2015, Vol. 8, No. 2 (Sl. No. 16), pp. 1218-1225
Geotech., Const. Mat. and Env., ISSN:2186-2982(P), 2186-2990(O), Japan

SEISMIC RESPONSES OF SHORT GROUPED-PILES EMBEDDED IN LIQUEFIABLE SANDY SOILS DURING EARTHQUAKES

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ABSTRACT: A series of centrifuge shaking table tests was conducted to simulate the seismic responses of grouped-piles that was embedded in liquefiable sandy soils and subjected to earthquake loading. The grouped-piles connected with a pile cap were used to support 4 sets of model dry storage spent fuel casks. Different test conditions including the elevations of pile cap, and the grouped-piles embedded in the dry and saturated sand beds with the different levels of groundwater table are reported. Using pre-shaking the profile of shear wave velocity in the tested sand bed and the natural frequencies of both the sand bed and the grouped-piles were determined. The lower shear wave velocities and the lower natural frequencies were measured for the saturated sand bed, but no obvious difference in the natural frequency of the grouped-piles system with the pile cap embedded in the sand deposit were observed. The grouped-piles with the pile cap exposed to the ground surface and embedded in the saturated sand bed have the lowest natural frequency. The magnitudes of bending moment along pile depths would increase with the increases of base shaking. The lowest bending moment were measured for the grouped–piles with the pile cap embedded in the dry sand bed while the largest bending moments were observed for the grouped–piles with the pile cap embedded in the saturated sand bed and the groundwater table at the surface.

Keywords: Seismic Response, Grouped-Piles, Liquefaction, Shaking Table Test, Centrifuge Modeling

1. INTRODUCTION

Dramatic failure of structures founded on pile foundations after a strong earthquake has led many researchers to investigating the behavior of grouped- piles supported foundations in liquefiable deposits [1; 2]. The liquefaction-induced pile foundation failures not only occurred in sloped grounds (lateral spreading) but were also observed in level grounds. The failure of pile foundation, resulting from that the bending moments or shear forces on the piles exceed the bending or shear carrying capacity of the pile section, were often accompanied by settlements and tilting of superstructures. The damages rendered the supported-structures becoming useless or costing expensive to rehabilitate after strong earthquakes. Therefore soil-structure interaction effects must be properly considered in the grouped-piles design especially embedded in the liquefiable sandy soils. Currently, piles in liquefiable soils are designed as beams to resist bending failure from lateral loads due to the inertia of the structure and/or slope movement (lateral spreading)[3].

Dry cask storage is an alternative method of temporary storing spent fuels that have already been cooled in a spent fuel pool. There are various designs of spent fuel dry storage cask system. With some designs, the casks are placed vertically on a concrete slab that is founded with a grouped-piles foundation. The seismic responses of the grouped-piles system located at the liquefiable sites are critically concerned. In the study a series of model grouped-piles (2×2) centrifuge shaking table tests at an acceleration of 80 *g* was conducted to simulate seismic responses of a grouped-piles embedded in liquefiable sandy soil subjected to different magnitudes of earthquake loading.

2. CENTRIFUGE MODELING AND TESTING SETUP AND PROCEDURES

2.1 Testing Equipment

The experimental work presented here was undertaken in the Centrifuge at the National Central University (NCU), Taiwan. The NCU Centrifuge has a nominal radius of 3 m and is equipped with a 1-D servo-hydraulically controlled shaker integrated into a swing basket. The shaker has a maximum nominal shaking force of 53.4 kN, a maximum table displacement of ±6.4 mm, and is operated at an centrifugal acceleration of up to 80 *g*. The nominal operating frequency range was 0–250 Hz. A laminar container with inside dimensions of 711 mm x 356 mm x 353 mm was constructed from 38 light-weight aluminum alloy rings arranged in a stack. Each ring was 8.9 mm in height and was separated from the adjacent rings by roller bearings that were specially designed to permit translation in the longitudinal direction with minimal frictional resistance. The laminar container was designed for dry or saturated soil models and permits the development of stresses and strains associated with 1-D wave propagation [4]. A flexible 0.3 mm thick latex membrane bag was used to retain the soil and the pore fluid within the laminar container.

2.2 Design and Fabrication of Grouped-Piles Model

Fig. 1 is a typical layout of spent fuel dry storage system. The dry storage casks are placed on a concrete slab founded with a row of grouped-piles which are embedded in a medium dense to dense sand deposit and penetrated into the bed rock. Each concrete pile having a diameter of 1.8 m and EI=12772754 kN/m^2 are used to support a dry storage cask by means of the concrete slab. In the study 4 sets of spent fuel cask rest on a simplified 4 grouped-piles model ($2\times\mathbf{2}$) is used in the centrifuge modeling.

The grouped-piles model ($2\times\mathbf{2}$) as shown in Fig. 2 was fabricated using 4 copper tubes with a 19 mm outer diameter and a 17 mm internal diameter. The length of pile was 160 mm and the embedment depth of the model pile in the sand bed was 125 mm both in model scale. Strain gauges were attached externally at nine positions on the pile shaft surface for two of four piles to measure the bending moment distributions along both the front pile and the back pile (MA and MB). These two model piles are named as the bending piles. The surface of model pile shaft was then resin coated for waterproofing. The coated model piles (22.5 mm in diameter) are designed to replicate a circular pile with a diameter of 1.8 m and having an embedment depth of 10 m when it was tested at an acceleration of 80 *g*. The ratio of pile length and diameter (*L/D*) is around 5.5. The model pile exhibited flexural rigidity (*EI*) of 10350668 kN-m^2 in prototype scale. Here *E* and *I* are the Young's modulus and the inertia moment of the model instrumented pile, respectively. In addition the strain gauges are also attached on the pile shafts of the other two piles to measure the axial forces along the pile depths. These two piles are named as the axial piles. All the instrumented piles (two bending pile and two axial piles) were calibrated for establishing the relationships between the output voltages and bending moments (or axial forces) prior to the model tests. Therefore, the bending moment (or axial force) profiles on the front and back piles can be measured during the tests. The total weight of four model dry storage casks and the pile cap as shown in Fig. 2 are 24.2 N in model scale. The height of gravity center of dry storage casks and the pile cap is scaled from the prototype.

2.3 Sand Bed Preparation and Testing Layout

A fine quartz sand (No. 306) was used to prepare the uniform dense sand deposit. The characteristics of the quartz sand are summarized in Table 1. The four grouped-piles were first screwed at the bottom of laminar container and then the quartz sand was pluviated with a regular path into the container from a hopper at a constant falling height and at a constant flow rate for preparing fairly uniform sand deposits having relative density of about 85%. Finally the saturation process was conducted for the saturated sand bed. In the case of saturation an acrylic plate was used to tightly cover the container for the saturation process of sand bed. Air was then simultaneously and continuously vacuumed out from both the inside and the outside of container. At the same time a de-ionized, de-air water-metulos solution having a viscosity of 80 times the viscosity of water was carefully dripped into the container to saturate the sand bed until the fluid level rose to the predetermined elevations. One day was required to saturate a sand bed.

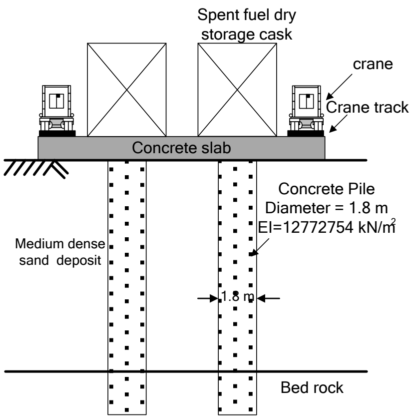


Fig. 1 Layout of spent fuel dry storage system.

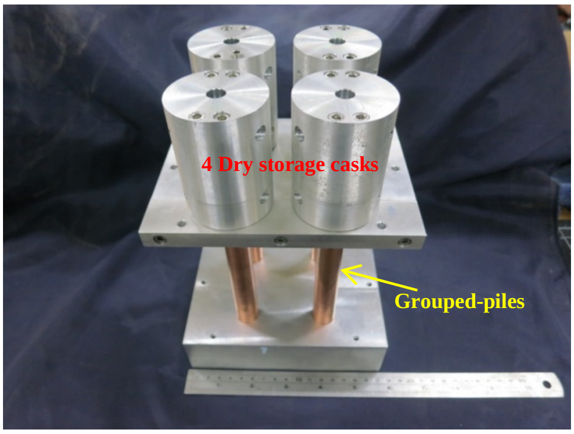


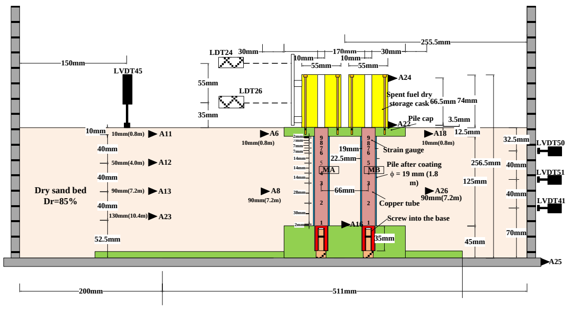
Fig. 2 Four model spent fuel storage casks rested on the concrete founded with grouped piles

Five grouped-piles centrifuge model tests were conducted. Figures 3(a), 3(b), 3(c), 3(d) and 3(e) show the soil profiles and the instrumentation layouts used in the centrifuge modeling. Table 2 lists the testing conditions. The total depth of the sand bed for both the grouped-piles and the pile cap embedded in the sand bed is 14.6 m and that for the pile cap outcropping on the ground surface is 13.6 m both in prototypes. Two accelerometers were attached at two elevations on the spent fuel cask to measure the seismic responses. Each sand bed was equipped with 3 rows of vertically spaced accelerometers (A#) to record the propagation of shear waves from the base to the ground surface, as shown in Figure 3. At the elevations near those of the accelerometers, eight pore-water pressure transducers (P#) were installed for the saturated sand beds. A linear variable differential transformers (LVDTs) were installed on the ground surface to measure the time histories of surface settlement. The transducers were installed at similar locations in each test. Two laser displacement transducers (LDTs) were used to measure the horizontal displacements on the spent fuel casks. Three LVDTs were attached on the side wall of laminar container to measure the lateral wall displacements. All measurement points were located away from the boundaries of the laminar container to minimize boundary effects [4].

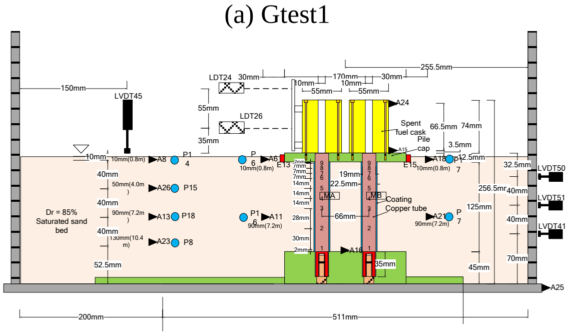
Table 1 Characteristics of fine quartz sand.

	G_s	D_{50} (mm)	D_{10} (mm)	[1]ρ_{max} (g/cm^3)	[1]ρ_{min} (g/cm^3)
Quartz sand	2.65	0.193	0.147	1.66	1.44

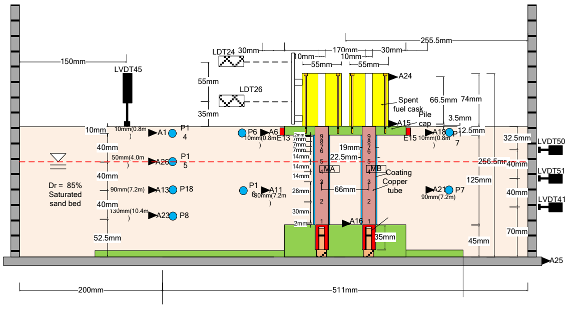
[1] The maximum and minimum densities of the sand were measured in the dry state, according to the method (JSF T 161-1990) specified by the Japanese Geotechnical Society.



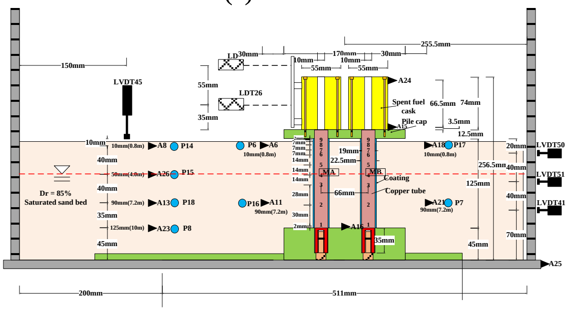
(a) Gtest1



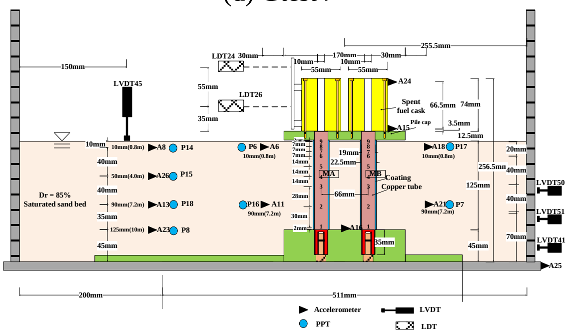
(b) Gtest2



(c) Gtest3



(d) Gtest4



(e) Gtest5

Fig. 3 Test setup and instrumentations: (a) Gtest1; (b) Gtest2; (c) Gtest3; (d) Gtest4; (e) Gtest5; the model dimensions are in centimeter and prototype dimensions (in parentheses) are in meters.

Table 2 Testing condition of grouped-piles

Test No.	Test conditions
Gtest1	Both the grouped-piles and the pile cap embedded in the dry sand bed
Gtest2	Both the grouped-piles and the pile cap embedded in the saturated sand bed having groundwater table on the ground surface
Gtest3	Both the grouped-piles and the pile cap embedded in the sand bed having the groundwater table 4 m below the ground surface
Gtest4	The grouped-piles embedded in the sand bed having the groundwater table 4 m below the ground surface but the pile cap outcropping on the top of ground surface
Gtest5	The grouped-piles embedded in the sand bed having the groundwater table on the ground surface but the pile cap outcropping on the top of ground surface

Fig. 4 shows a test package resting on the NCU shaker is ready for testing. Comprehensive instrumentation and detailed measurements at various elevations and different positions are necessary to determine the seismic responses of the grouped-piles and the associated excess pore-water pressure generation and dissipation in the sand deposits during shaking. After completing the centrifuge flight safety checks, the centrifuge was accelerated at an acceleration of 10 g per step until it reached 80 g. The model was maintained at each g-level for 5 min. to ensure that the sand bed reached full consolidation at each overburden stress. Finally the model was subjected to different base shaking events with various amplitudes of base acceleration at an acceleration of 80 g. The time histories of acceleration, the pore-water pressure at various elevations, the bending moments and the axial forces on the piles, and surface settlement were recorded simultaneously. The measurements in the following sections are presented in prototype units unless specifically noted otherwise.

Table 3 lists the typical shaking events for the five tests (Gtest1 – Gtest5) in the study. The S1 event is a pre-shaking technique for measuring the V_s profile of sand deposits and identifying the natural frequencies of sand bed and soil structure system at the acceleration of 80 g. The pre-shaking technique is a non-destructive test. It uses a shaker as a wave generation source and a vertical array of accelerometers embedded in the sand bed and the accelerometers attached to the pile head as receivers [5]. After the pre-shaking events each model was subjected to multiple shaking events with amplitudes of base acceleration from small to large.

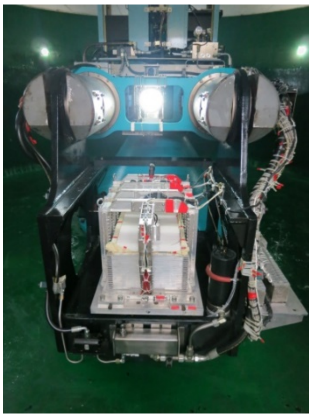


Fig. 4 Test package resting on the NCU shaker

3. TEST RESULT AND INTERPRETATIONS

3.1 Fundamental Frequencies of Sand Deposit and Grouped -Piles

The pre-shaking method (S1 Event as listed in Table 3) was conducted to measure the profile of shear wave velocity along the depths in terms of the vertical accelerometer array as shown in Figure 3. The shaker fired one cycle of a low, controlled amplitude sinusoidal wave (approximately 0.01 g in amplitude and 1 Hz, 2 Hz, and 3 Hz in frequency, both on the prototype scale) to provide a vibration source. A higher sampling rate of 30 000 samples/s was used to collect the acceleration time histories from the accelerometers those which were embedded in the sand bed and were attached on the spent fuel cask. Therefore the average shear wave velocity, V_s, of the sand deposit between the accelerometer pairs could be calculated using the following equation:

$$\boldsymbol{V_s} = \Delta \boldsymbol{S}/\Delta \boldsymbol{t} \tag{1}$$

where ΔS and Δt are the distance between adjacent accelerometers and the registered time difference between the first peaks of the accelerometer pairs, respectively. Figure 5 displays the typical test result of shear wave velocity profile for Gtest1. Figure 6 displays the amplification factors of the acceleration measured at various depths and at two different elevations on the spent fuel cask as shown in Figure 3(a). The peaks in the curves represent the nature frequencies of the sand bed and the entire soil-pile-casks system. Table 4 lists the measured average shear wave velocities and the measured nature frequencies of sand deposit and grouped-piles system. The dry sand bed has the highest nature frequency (Gtest1), the saturated sand bed has the lowest nature frequency. There are two nature frequencies measured from the grouped-piles, as listed in Table 4 and shown in Figure 6(b). The first one is the nature frequency of sand bed in which the grouped-piles are embedded and the second one is the nature frequency of grouped-piles system. The grouped-piles with pile cap embedded in the sand deposit have the highest natural frequency (4.1 Hz for Gtest1). By contrast the grouped-piles embedded in the sand bed having the ground water table on the ground surface and the pile cap outcropping on the top of ground surface have the lowest nature frequency (3.1 Hz for Gtest5).

Table 3 Shaking events in the study

Test events	$a_{max(base)}$/freq./No. of cycle g / Hz / cycles	Dr(%)
S1-1Hz	0.0083g / 1Hz / 1	88.6
S1-2Hz	0.0091g / 2Hz / 1	
S1-3Hz	0.0087g / 3Hz / 1	
S2	0.0118g / 1Hz / 15	86.6
S3	0.0373g / 1Hz / 15	86.7
S4	0.0858g / 1Hz / 15	87.3
S5	0.1491g / 1Hz / 15	87.9
S6	0.2112g / 1Hz / 15	88.0

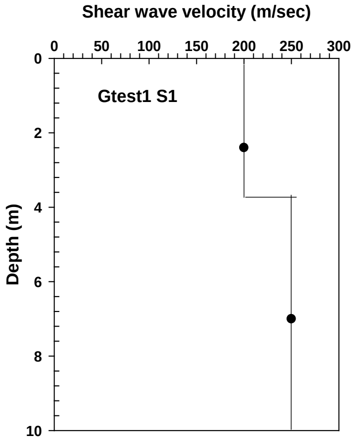
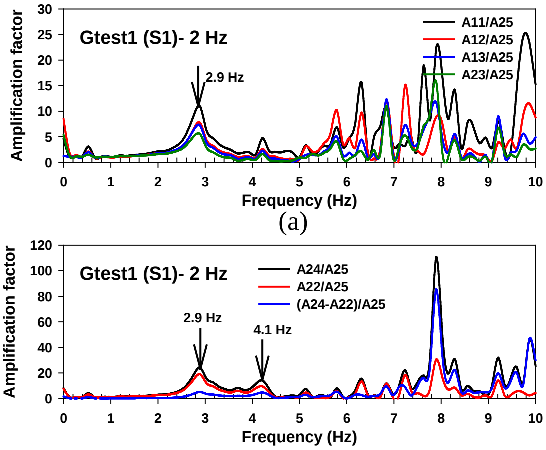


Fig. 5 Measured shear wave velocity profile of sand deposit for Gtest1



(b)

Fig. 6 Amplification factor vs. frequency for Gtest1: (a) measured at the sand bed; (b) measured at grouped-piles system

Table 4 Average shear wave velocities and natural frequencies of sand deposit and grouped-piles system

Test No.	V_s [1] (m/sec)	f_n [2] (Hz)	1st f_n [3] (Hz)	2nd f_n [4] (Hz)
Gtest1	225	2.9	2.9	4.1
Gtest2	170	2.4	2.4	4.1
Gtest3	170	2.9	2.9	3.97
Gtest4	170	2.4	2.4	3.6
Gtest5	160	2.1	2.1	3.1

[1]Measured average shear wave velocity of sand deposit;
[2]Measured natural frequency of sand deposit;
[3]Measured first natural frequency of grouped-piles system;
[4]Measured 2nd natural frequency of grouped-pile system

3.2 Comparison of Time Histories of Acceleration and of Excess Pore Water Pressure at Different Depths

Figure 7(a), 7(b) and Figure 8(a), 8(b) show the time histories of acceleration of Gtest1 (S5) and Gtest5 (S5), those which were measured at the corresponding depths in the sand bed and at the spent fuel cask, respectively. The high frequency and high amplitude spikes in the histories of acceleration as shown in Figure 8(a) are observed. Figure 9 shows the measured time histories of excess pore water ratio of Gtest5 (S5) measured at different depths. Both the observed results show the top 7 m depth of the sand deposit becoming liquefying during big base shaking (the base acceleration ≅ 0.3 g).

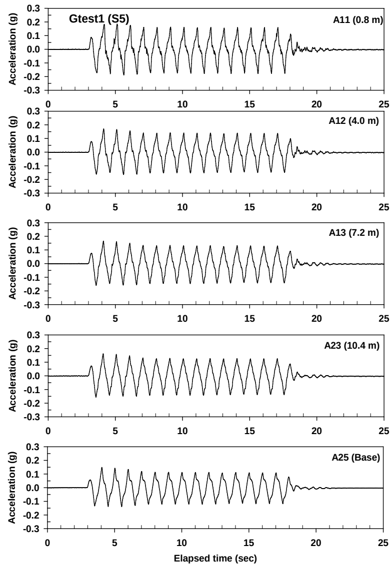
After comparing the acceleration time histories the soil responses of these two models are quite different although nearly the same amplitude of input base acceleration. A single parameter that includes the effects of the amplitude and frequency content of the acceleration time history is the root mean square acceleration (RMS acceleration), defined as:

$$\boldsymbol{a_{rms}} = \sqrt{\frac{1}{T_d}\int_0^{T_d}[\boldsymbol{a}(\boldsymbol{t})]^2\boldsymbol{dt}} \qquad (2)$$

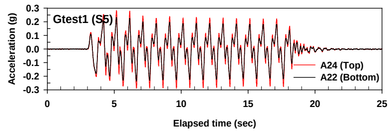
where T_d is the duration of the acceleration and dt is the time interval of the acceleration time history. The larger the RMS acceleration value the higher the energy input to the sand deposit. An acceleration amplification ratio is defined as the ratio of the RMS acceleration value measured at different depths to the base input RMS acceleration value.

Figure 10 presents the RMS acceleration amplification ratios, those which were calculated from the accelerometer array embedded in the sand deposit and on the fuel spent cask; provide a comparison of amplification ratio profiles along the depths and on the spent fuel cask for Gtest1 to Gtest5 those which were subjected to different levels of base shaking. There was considerable amplifying effect on the acceleration measured on the top soil following shear waves propagating from the base to the surface on Gtest1 - Gtest4, but no amplification of ground response on the top 7 m of the sand deposit (Gtest5) were observed because occurrence of soil liquefying blocked the shear waves upward propagating. However the calculated amplification ratios on the spent fuel cask of Gtest5 had the largest magnitude. The spent fuel cask rested on the grouped-piles with a pile cap outcropping on the ground surface and that was embedded in the liquefied sand would experience the highest acceleration. The pile cap outcropping on the ground surface would greatly reduce the soil confinement; as a result, the spent fuel cask experienced the largest vibrations as shown in

Figure 10(e).

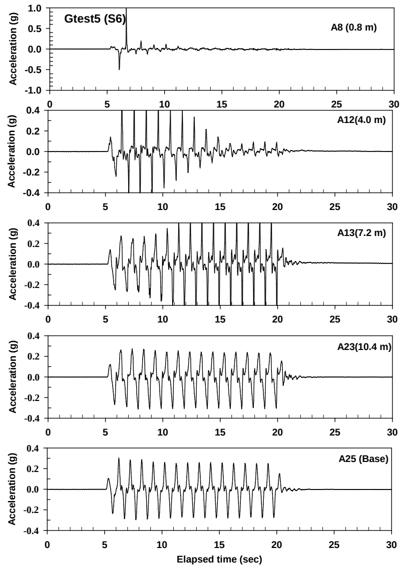


(a)

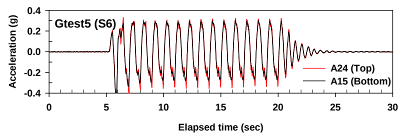


(b)

Fig. 7 Time histories of acceleration for Gtest1: (a) measured at different depths in the sand deposit; (b) measured at two elevations on the spent fuel cask



(a)



(b)

Fig. 8 Time histories of acceleration for Gtest5: (a) measured at different depths in the sand deposit; (b) measured at two elevations on the spent fuel cask

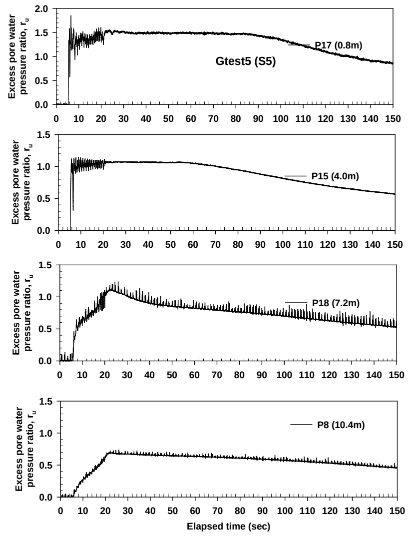
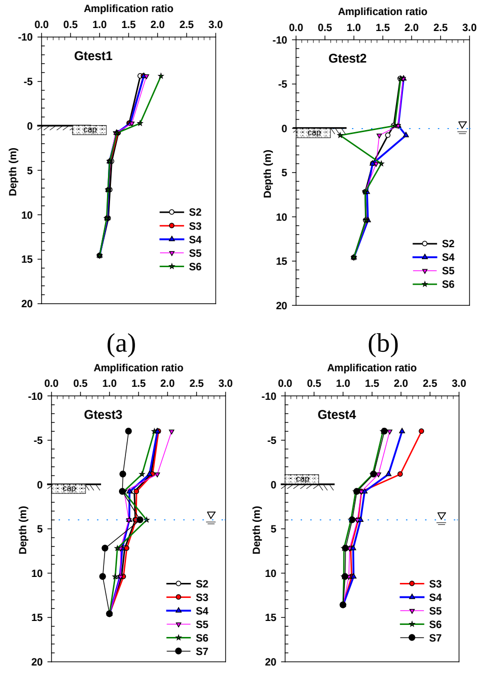
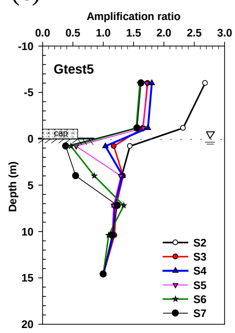


Fig. 9 Time histories of ratio of excess pore water pressure for Gtest5 (S5)



(a) (b)

(c) (d)

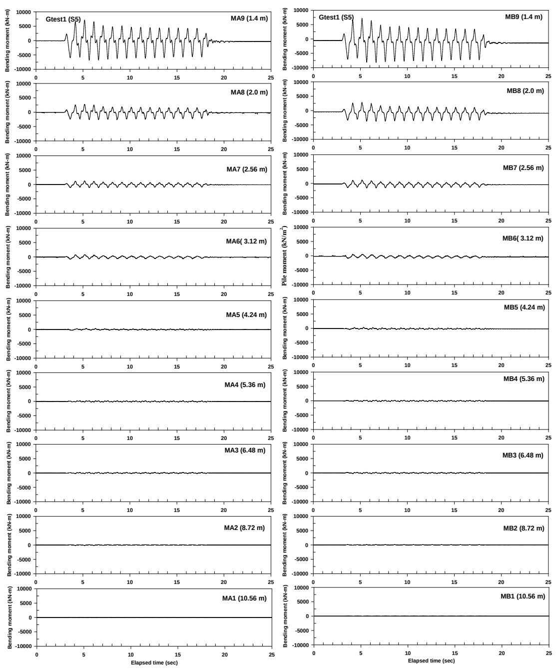


(e)

Fig. 10 Profiles of RMS acceleration amplification ratios along depths and on the spent fuel cask: (a) Gtest1; (b) Gtest2; (c) Gtest3; (d) Gtest4; (e) Gtest5

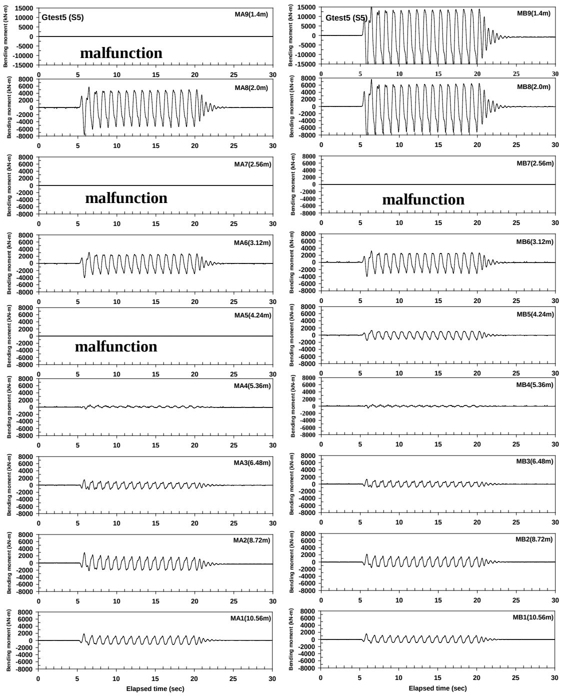
3.3 Time Histories of Bending Moment along Depths on the Grouped-Piles

Figs 11(a) and (b) and Figs 12(a) and 12(b) show the time histories of bending moment measured from the pile head to the pile tip for Gtest1 (S5) and Gtest5 (S5). The grouped-piles and the pile cap both which were embedded in dry sand deposit (Gtest1) experienced the less magnitude of bending moments. Furthermore only top parts of piles (around 3.12 m in depth) experienced the bending moments. By contrast the grouped-piles embedded in the saturated sand bed and the pile cap outcropped to the ground surface experienced the largest bending moments. Nearly all the depths of pile shaft experienced the larger bending moment (Gtest5) compared to Fig 11. In addition the lower parts of pile also experienced the bending moments that different from the measured results of Gtest1. These resulted from the facts that the saturated sand became liquefying in the top of sand deposit and greatly losing the surrounding confinements to the piles if the cap of the grouped-piles exposed on the ground surface. Figs 13(a)-13(d) display the relations of measured maximum bending moment on the pile at MA9 (measured point is just below the pile cap) and the base acceleration or the acceleration measured on spent fuel cask for Gtest1 – Gtest5. The grouped-piles would experience the larger maximum bending moments on the pile in the case of exposing the pile cap on the ground surface than the case of the pile cap embedded in the sand deposit.



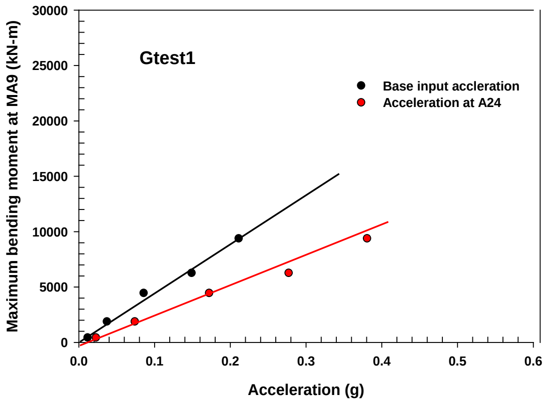
(a) (b)

Fig. 11 Time histories of bending moment along the depths for Gtest1 (S5): (a) A pile; (b) B pile.

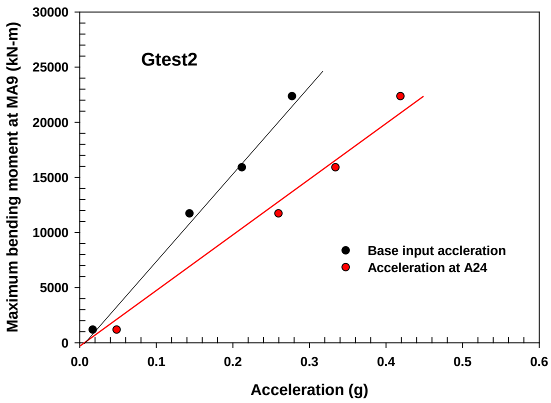


(a) (b)

Fig, 12 Time histories of bending moment along the depths for Gtest5 (S5): (a) A pile; (b) B-pile



(a)



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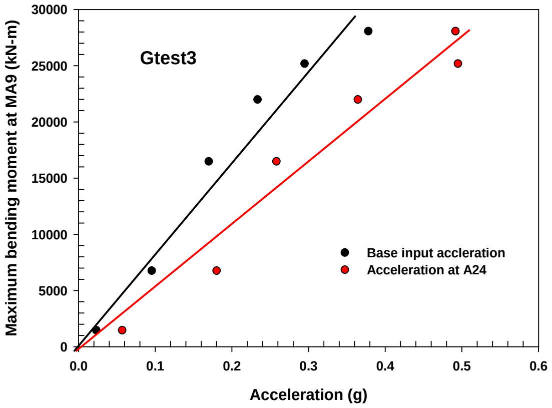

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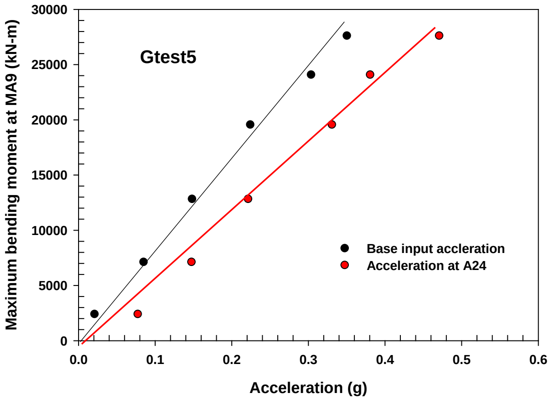

(d)

Fig. 13 Maximum bending moment measured at MA9 vs. base input acceleration and acceleration at spent fuel cask (A24): (a) Gtest1; (b) Gtest2; (c) Gtest3; (d) Gtest5

4. CONCLUSION

A series of centrifuge shaking table tests was conducted to simulate the seismic responses of short grouped-piles that was embedded in liquefiable sandy soils and subjected to earthquake loading at an acceleration of 80 *g*. The grouped-piles connected with a pile cap were used to support 4 sets of model dry storage spent fuel casks. Different test conditions including the elevations of pile cap, and the grouped piles embedded in the dry and saturated sand beds with the different levels of groundwater table are reported in the study. Using pre-shaking technique the profile of shear wave velocity in the tested sand bed and the natural frequencies of both the sand bed and the grouped-piles were determined simultaneously. The lower shear wave velocities and the lower natural frequencies were measured for the saturated sand bed, but no obvious difference in the natural frequency of the grouped-piles system with the pile cap embedded in the sand deposit were observed. The grouped-piles with the pile cap those which were exposed to the ground surface and embedded in the saturated sand bed have the lowest natural frequency. The spent fuel cask rested on the grouped-piles with a pile cap outcropping on the ground surface and that was embedded in the liquefied sand would experience the highest acceleration.

The pile cap outcropping on the ground surface would greatly reduce the soil confinement; as a result, the spent fuel cask experienced the largest vibrations. The magnitudes of bending moment along the pile depths would increase with the increases of the base shaking. The lowest bending moments were measured for the grouped–piles with the pile cap embedded in the dry sand bed while the largest bending moments were evidenced for the grouped–piles with the pile cap embedded in the saturated sand bed and the groundwater table located at the surface.

4. ACKNOWLEDGEMENTS

The authors would like to express their gratitude for the financial supports (NL1020467) from the Institute of Nuclear Energy Research (INER), Atomic Energy Council, Taiwan and the National Science Council (NSC 100-2221-E-008-099-MY3), Taiwan. Without their financial supports, the study would not become possible.

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Int. J. of GEOMATE, June, 2015, Vol. 8, No. 2 (Sl. No. 16), pp. 1218-1225.
MS No. 4141 received on May 19, 2014 and reviewed under GEOMATE publication policies.

Pertinent discussion including authors’ closure, if any, will be published in June 2016 if the discussion is received by Dec., 2015.
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