SEISMIC PERFORMANCE OF LNG STORAGE TANK FOUNDATIONS DURING THE VERY LARGE EARTHQUAKE

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SUMMARY

This paper describes the results of shaking table tests on LNG facilities on the reclaimed land using the centrifuge apparatus. A large centrifuge earthquake simulator was used to carry out a series of shaking table tests under 50*g* (1*g*=981cm/sec^2) centrifugal acceleration on the 1/50 scaled tank structure with piled foundation constructed on the model multi-layered ground.

During a series of centrifuge tests, seismic responses of multi-layered grounds and piled foundations of LNG facilities were investigated. Strong nonlinear response of the ground was observed during the strong earthquake event as well as post-yield response of piles of the piled foundation. It was found that residual deformation of the pile cap was small although damages of piles were observed during a strong event. Thus, it is concluded that the piled foundation used for LNG tank structure could be used or remains functional after the very large earthquake.

INTRODUCTION

Background

Following the Great Hanshin Earthquake of January 17 of 1995, many research projects in Japan are being focused on the safety of essential infrastructures during very large earthquakes. Essential infrastructure is required to remain functional after the earthquake. To ensure the seismic safety of the LNG facilities of the electric power industries, the Ministry of International Trade and Industry (MITI) of Japan has initiated a project named Seismic Proving Test of Equipment and Structures in Thermal Conventional Power Plants (SPT). SPT includes a series of surveys and experiments for proving the resistance of thermal power plant installations during severe earthquakes. Centrifuge model tests described in this paper were planned as a part of SPT project.

Purposes of centrifuge tests are to investigate the seismic performance of pile foundations of LNG facilities as well as verifying the seismic safety.

Four (4) papers are presented at 12WCEE as the results of SPT project. These titles are (excluding this paper) ;

(1) Proving Test of Analysis Method on Nonlinear Response of Cylindrical Storage Tank under Sever Earthquakes.
(2) Test of Nozzles at Wall of Cylindrical Tank for Several Loads under Earthquake.
(3) Proving Tests of Energy Absorbing Seismic Ties for Aseismic Design of Boiler Plant Structures.

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Purpose of Centrifuge Tests

The purposes of centrifuge tests are to investigate:

*the seismic response and the performance of piled foundations for pipe support structures and their residual displacement after a seismic event;

*the seismic performance of piled foundations for LNG tanks, their residual displacement and relative displacement between the tank and pipe support structures; and

*the seismic response of steel pipe piles in multiple soil layers of different strength, with particular reference to the post-yielding response.

EXPERIMENTAL PROCEDURE

Centrifuge Apparatus

The centrifuge apparatus at UC Davis (Kutter, et al., 1994), which equipped the strong hydraulic shaking table system was used for centrifuge tests. This centrifuge has 9.1m radius and 2,700kg payload mass capacity.

A Flexible Beam Container (FSB) was newly constructed for this project. The concept of FSB is to possess small mass and minimal shear stiffness compared to the soil mass in it. This new flexible container has 5 aluminum beam rings with inner dimension of 1651mm (Length) x 787mm (Width) x 585mm(Depth).

Testing Program

Centrifuge tests consist of 3 series of experiments. Purpose for the first series is to investigate the ground response. Purpose for the second series is for the pipe support structure response, and purpose for the third series is for the tank response during strong earthquake event. Centrifuge tests were mainly performed under 50*g* (1*g*=981cm/sec^2) centrifugal acceleration. Thus, the scaling factor of 1/50 was chosen. Scaling relationships for centrifuge model tests have been published by a number of researchers, Kazama, et al. (1988), for example are summarized as Table 1.

Table 1. Scaling relationships for centrifuge model tests

Parameter	Scaling factor (model/prototype)
Length	1/n
Stress	1
Strain	1
Force	$1/n^2$
Mass	$1/n^3$
Acceleration	n
Velosity	1
Density	1
Time	1/n (dynamic), $1/n^2$ (diffusion)
Frequency	n
Stiffness of pile	$1/n^4$

Model Profiles

Typical LNG storage tank constructed in Japan has a volume of 80,000kl (80,000m^3), a diameter of 60m and a height of 40m. Because of the limitation of the size of the centrifuge and the model container, 1,000kl (1,000m^3) LNG storage tank shown in Figure 1 was chosen as a prototype. The model tank was built with a scaling factor of 1/50 in length, which agrees to the centrifugal acceleration of 50*g* due to the scaling relationships.

The model tank has been designed based upon the overall dimensions of the prototype structure. The important facets of the tank response that need to be modeled are the overall mass and center of gravity. Although the fluid-tank wall interaction is very important for the dynamic response of the tank structure, it was not taken into account for the tank design. This is because the frequency range at which the phenomenon dominates is different from the frequency range of the ground response, on which this test focused. The tank shell and the pile cap are made of aluminum. Mass of the model tank is 7.63kg, which matches to the scaling factor of 1/50^3.

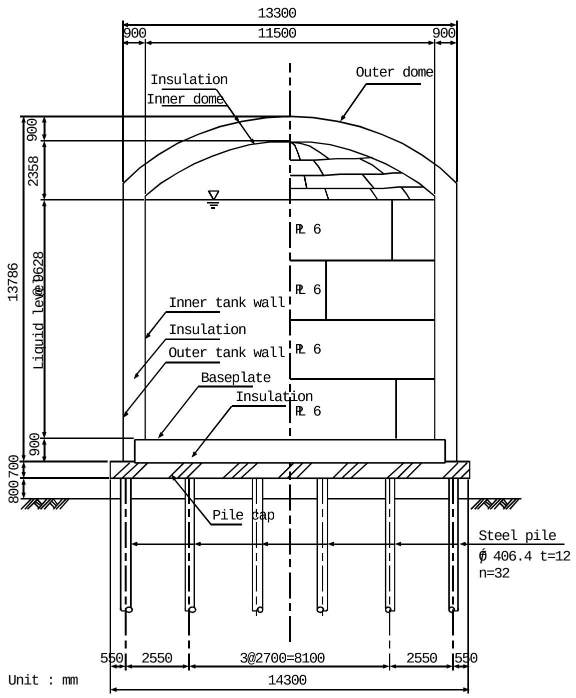


Figure 1. 1,000kl LNG storage tank (Prototype)

Piles used for the prototype tank foundation are made of steel and graded SKK400, having an ultimate strength of 400MPa. Due to the purpose of centrifuge tests, it is desired to model the post-yield response of the pile and therefore it is proposed to fabricate steel model pile. At the scaling factor of 50, model pile should replicate the bending stiffness of the prototype pile according to the following relationship derived from scaling laws;

$E_pI_p=50^4E_mI_m$ (1)

where E and I are the Young's modulus and second moment of inertia of the pile, and subscripts p and m refer to prototype and model respectively. To determine physical properties of model pile, loading tests were carried out. Results of model pile loading tests are shown in Table 3 comparing with prototype values.

Table 2. Physical properties of model pile

	Prototype	Model
Diameter (mm)	400	7.98
Thickness (mm)	12	0.29
I (mm^4)	$2.76x10^8$	$4.63x10^2$
E (MPa)	205,800	224,000
σ_y (MPa)	235	360
ε_y (μ)	1,142	1,600

To investigate the dynamic response of the reclaimed land, ground profiles illustrated in Figure 2 were proposed for the first series of experiments. Lower dense sand layer imitates the seabed, soft clay layer imitates the sediment on the seabed and upper sand layer imitates the reclaimed soil deposit. Note that remarkable difference of the shear stiffness exists between sand layers and the clay layer. One of these profiles will be chosen for the second series (Pipe support model) and the third series (Tank model) of tests.

Figure 3 shows the layout and instrumentation for the LNG tank structure model (Third series). One of the fully instrumented piles has 9 gauge-bridges, which were installed for measuring the distribution of bending moment generated by the soil deformation on the pile during shake events. Accelerometers were installed in the soil deposit and the tank structure. Displacements of the tank structure and ground surface were also recorded.

Due to the limitation of the paper length, ground responses and tank structure responses will be discussed in this paper.

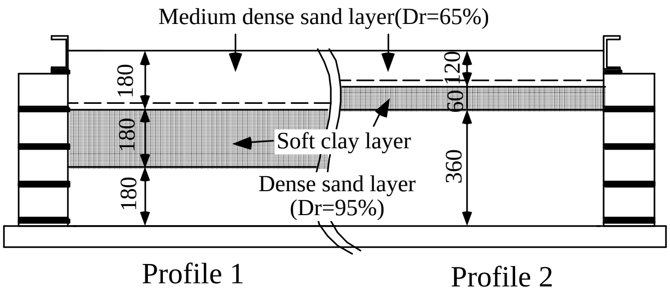
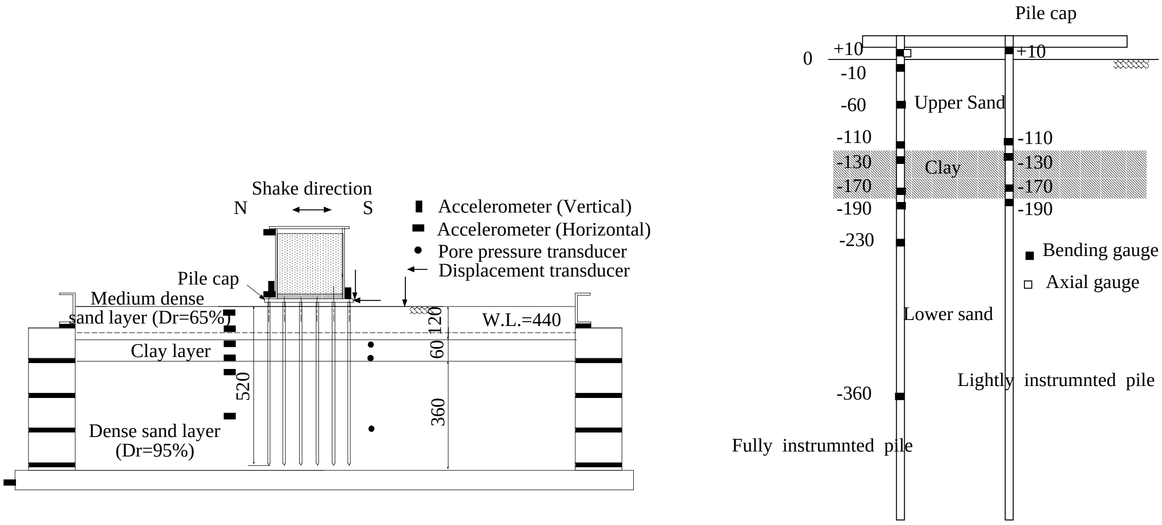


Figure 2. Proposed multi-layered ground profile



(a) Section view and instrumentation **(b) Strain gauge arrangement for piles**

Figure 3. Layout for tank structure model

Testing Procedure

Shaking tests were performed with variety of events such as small (peak acceleration < 0.05*g*), L1 (Level-1: strong earthquake motion for seismic design; peak acceleration = 0.15*g*) and L2 (Level-2: very strong earthquake motion; peak acceleration > 0.4*g*) motions. Artificial strong earthquake motions were newly synthesized basing on stochastic synthesis method (Kamae, et al., 1991) in this project. One of the time histories of L2 motions used for the centrifuge test is shown in Figure 4. The maximum acceleration is larger than 0.4*g* at the prototype. The original input motion was modified for shaking table basing on the scaling factor as shown in Table 2.1. The dynamic characteristics of the shaking table system were concerned when modifying.

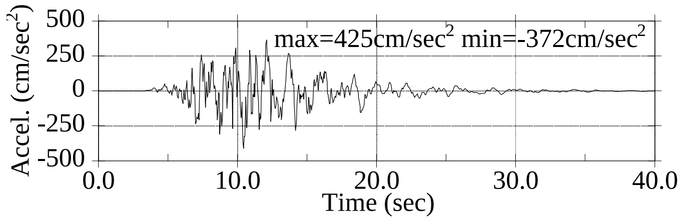


Figure 4. Artificial strong earthquake motion (L2) basing on stochastic synthesis method

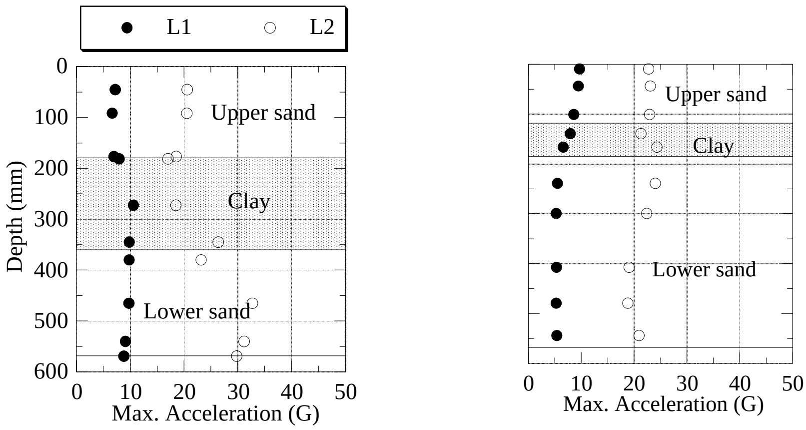
On completion of the model building, the model container is transported and mounted on the centrifuge bucket, then spin up to the centrifugal acceleration of 50*g*. After several hours spinning for consolidation of the clay layer, shaking tests were started with small event.

Note that further discussion will be made basing on the model scale (n=50) without any notice. Table 2 can be used for prototype conversion.

RESULTS AND DISCUSSIONS

Ground Responses

Figure 5 (a) and (b) show the maximum acceleration distribution in the soil deposit observed at L1 and L2 event. It is seen that remarkable acceleration reduction is observed in the thick clay layer at both L1 and L2 events in the model "Profile 1". In the model "Profile 2", on the other hand, acceleration increased when propagated through the clay layer at L1 event, but acceleration reduced at L2 event.



(a) Profile 1 **(b) Profile 2**

Figure 5. Maximum acceleration distribution in the soil deposit

Table 3 shows the peak acceleration ratio (ratio of maximum value of acceleration at the ground surface and the input motion.) of the ground observed at various input motions. It is seen that larger the input acceleration amplitude, smaller the peak acceleration ratio. Frequencies of peak amplitude of the frequency responses derived from transfer functions between the input motion and the ground surface acceleration of model grounds are also shown in Table 3. This indicates overall stiffness of the model "Profile 1" is larger than the model "Profile 2". It is seen that larger the input level, lower the resonant frequencies (ex. f=65 Hz at small event and f=45 Hz at L1 event, respectively in Profile 1). These phenomena reflect the strain-dependent stiffness of ground materials.

Table 3. Peak acceleration ratio and peak frequency of transfer functions of ground models observed at various input motions

	Ground Profile			
	Profile1		Profile2	
Input Motion	Acc. Ratio	Peak Freq.	Acc. Ratio	Peak Freq.
small	2.74	65	3.84	85
L1	0.84	45	1.84	65
L2	0.69	20	1.09	35

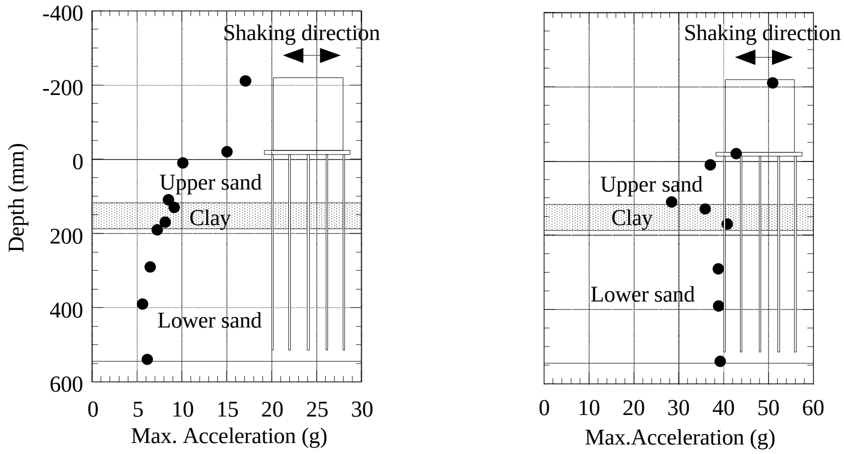
Maximum shear strains of the soil deposit during the L2 event were calculated from acceleration records of accelerometers located in the soil deposit. It is found out that very large shear strain are generated inside the clay layer, and these magnitudes are more than 10%, although shear strain of sand layer is less than 1%. This suggests that larger deflection might be carried out on the piles inside the clay layer. Similar phenomenon was reported through the studies on damages during the Great Hanshin Earthquake (Adachi et al., 1996). It was reported that damages of foundations were dominant on reclaimed lands, and pointed out these damages were caused by the large shear strain generation due to the contrast of the shear stiffness between the seabed clay layer and the reclaimed ground.

Large shear strain is also seen at the upper part of the lower sand layer. This is due to the large pore pressure generation in the sand and lost the shear stiffness.

Tank structure and piled foundation responses

Based on the result of the first series experiments, "Profile 2" was chosen for the next 2 series. One of the L2 events used in this series has the maximum acceleration of 0.8 *g* at prototype scale (40 *g* at model scale).

Figure 6 (a) and (b) show the maximum acceleration distribution throughout the soil deposit observed at L1 and L2 event. For L1 event, acceleration distribution is similar to that of "Profile 2" of the first series. This indicates the repeatability of the model preparation method used in this project. The maximum acceleration at the tank pile cap is about 1.5 times as large as that of observed at the free field. At L2 event, reduction of the maximum acceleration is seen in the clay layer, and amplified at the upper sand layer. Amplification factors throughout the ground are 1.63 for L1 event and 0.94 for L2 event.

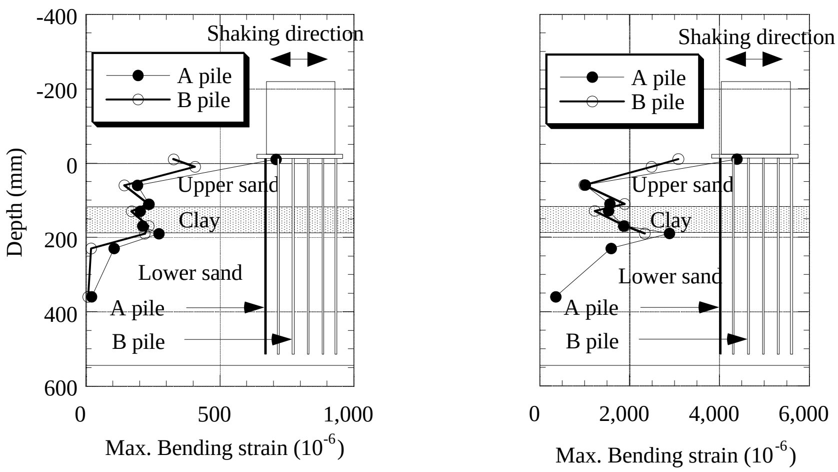


(a) L1 event **(b) L2 event**

Figure 6. Maximum acceleration distribution throughout the soil deposit and the tank structure

Figure 7(a) shows the maximum bending strain distribution of model piles during the L1 event. Relatively large bending strain is seen at the top of the pile of Pile A where the pile connected to the pile cap, and that is about 700×10^{-6}. That is a third of the yield limit of the model pile, which was obtained by the pile's loading test shown in Table 2. Relatively large strains are also seen at the interfaces of the different soil layers, but these are smaller than the yield limit.

Figure 7(b) shows the maximum bending strain distribution of model piles during the L2 event. More than $4{,}000 \times 10^{-6}$ of strain is seen at the pile cap of pile A, which is more than a double of the yield limit of the model pile. This is due to the large acceleration response of the tank and suggests the pile may be damaged at the pile cap connection during the very large earthquake. A strain of $2{,}900 \times 10^{-6}$ is also seen at the interface of the clay



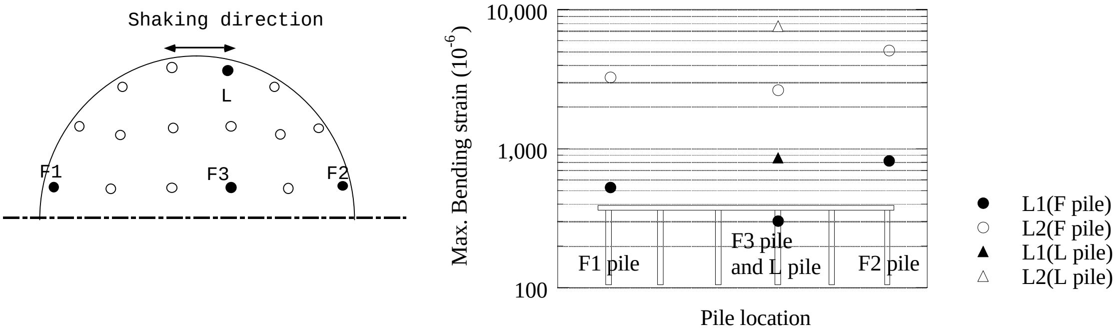
(a) L1 event **(b) L2 event**

Figure 7. Maximum bending strain distribution on piles of the tank foundation

layer and the lower sand layer. As it was mentioned in section 3.1, this is due to large deformation in the clay layer caused by the contrast of shear stuffiness between clay layer and sand layers. The maximum shear strain on the clay layer observed in this case was more than 10%. It should be noted from this result that piles possibly be damaged at the interface of the soil layers in the soil deposit typical for reclaimed land during the very large earthquake.

Remarkable difference of magnitude bending strain is seen between A pile and B pile. Location of A pile is at the edge of the pile cap, and B pile is at the center from the direction of shaking. From Figure 7, magnitude of strain of A pile is 1.5 times to twice as large as that of B pile at the pile cap. This is due to the effect of soil-structure interaction. When piles are deflected by lateral inertia force of the super structure generated by the earthquake motion, piles are subjected the reaction force from soil deposit. Under this condition, the force acting on the inside pile (B pile) of the pile group is smaller than the force acting on the outside pile (A pile) due to the existence of piles in front and back. This is so called "group effect", and have been published by a number of researchers.

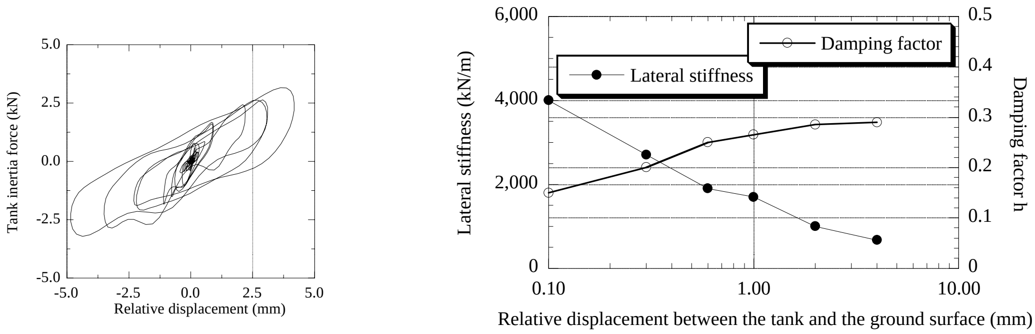
Figure 8 shows the maximum bending strain observed on the pile at the pile cap intersection with respect to the pile position. As it was mentioned above, magnitudes of bending strains of outside piles (F1 and F2) are larger than that of inside pile (F3). In addition, it is found the bending strain of pile L is larger than that of F1 or F2 pile. Same discussion as mentioned above can be made for this phenomenon. With respect to the shaking direction, F piles are lined up 6 piles. On the other hand, L pile has only one pile in this direction. So, it is concluded that group effect is less effective for the L pile, and this could be taken into account on the seismic design of group pile foundations.



(a) Pile location **(b) Max. bending strain**

Figure 8. Maximum bending strain on the pile at the pile cap with respect to the pile location

Nonlinear tank-foundation-soil displacement response is also found in this experiment. Figure 9 (a) shows the relation between the tank inertia force and the relative displacement of the ground surface to the pile cap. Remarkable difference of secant stiffness of force-displacement curve was seen in terms of the acceleration amplitude of input motion in various events. It is seen that the secant stiffness and the area of the hysteresis curve changes during the single event.



(a) Force-displacement curve (L2 event) **(b) Displacement dependent stiffness and damping**

Figure 9. Tank inertia force – relative displacement between the tank and the ground surface relations

Figure 9 (b) shows the secant stiffness and damping factor derived from force-displacement curve with respect to displacement. This secant stiffness corresponds to the lateral stiffness of the pile cap, and the damping corresponds to the damping of the pile group. It is seen that larger the relative displacement, smaller the secant stiffness, and larger the damping factor. This reflects both non-linear response of the ground and nonlinear response of piles.

Table 4 shows residual deformations of the pile cap and ground surface resulted from both L1 and L2 events. Although the ground surface settled 2.0 mm (100mm at prototype) and piles yielded during L2 event, the pile cap only settled 0.18mm (9 mm at prototype). This suggests the piled foundation for prototype LNG tank structure has enough strength and may remain functional during and after the very large earthquake.

Table 4. Residual deformations observed after L1 and L2 events

Event	Displacement (mm)		
	Ground surface V	Pile cap H	Pile cap V
L1	0.20	0.05	0.08
L2	2.00	1.20	0.18

CONCLUSIONS

Followings are concluded from the results of centrifuge tests.

(a) In case of reclaimed land, soft seabed clay layer could be largely deformed during the very strong earthquake event. The maximum shear strain may reach about 10% at this time.

(b) During the L2 event, bending strain of the pile at the pile cap was more than twice as large as yield limit. Group pile effect was observed at the strain response of piles at the pile cap as well.

(c) Almost twice as large as the yield strains of piles were found on piles during the L2 event due to deformation of the soil deposit. This suggests that the pile could be damaged in the soil deposit, especially at the interface of the soil layers in case the shear stiffness of layers are largely different.

(d) Nonlinear tank-foundation-soil displacement response was found. The phenomenon reflects nonlinear characteristics of both ground and piles. Displacement dependant of pile cap lateral stiffness and damping were seen.

(e) Seismic safety of the piled foundation for the prototype LNG tank structure is verified. Residual deformation of the pile cap is small although damages of piles are observed during a very large shaking event. Thus, the piled foundation could be used or remain functional after a very large earthquake.

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