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E-DEFENSE SHAKING TABLE TEST ON THE BEHAVIOR OF LIQUEFACTION-INDUCED LATERAL SPREADING OF LARGE-SCALE MODEL GROUND WITH A PILE-FOUNDATION STRUCTURE BEHIND QUAY WALL

M. Sato[1] and K. Tabata[2]

ABSTRACT

A shaking table test of a large-scale model was performed at the E-Defense three-dimensional shaking table facility in order to observe the behavior of lateral spreading of liquefiable ground and to evaluate the mechanism of its influence on the failure of structures. The model ground was prepared in a large rectangular container with a caisson-type quay wall, pile-supported structure and about nine hundred sensors. To this model, horizontal and vertical input motions based on one of the 1995 Hyogoken-Nambu earthquake records were applied. The motions induced liquefaction of the ground and horizontal displacement of the caisson to waterside, causing deformation of the structure due to the piles bent and collapsed. Observation of the test explains contributions of structural inertia and ground deformation to the behavior of the caisson and structure.

Introduction

Many large earthquakes have caused severe damage to various structures. Especially in the 1995 Hyogoken-Nambu earthquake, a lot of pile-supported structures behind quay walls in port areas collapsed due to widespread liquefaction and its resulting lateral spreading of ground. Such structures are necessary as facilities for rescue supplies to disaster areas as well as economical distribution systems during rehabilitation process after the disaster. Therefore, understanding the lateral spreading behavior of liquefied ground and its influence so as to mitigate earthquake disaster for port structures is one of the very important problems in geotechnical earthquake engineering discipline (Sato et al. 1998). In spite of its importance, reproduction of a lateral spreading phenomenon in the model ground on a centrifuge apparatus or shaking table is almost impossible because mainly of the small model size, so that the influences of lateral spreading on structures still have not been completely revealed. In other words, it is necessary to conduct shaking tests of a large-scale model ground with a structure subjected to actual magnitude earthquake motions to understand the behavior and influences of lateral spreading.

[1]Principal Senior Researcher, Disaster Prevention System Research Center,National Research Institute for Earth Science and Disaster Prevention, 3-1 Tennodai, Tsukuba, Ibaraki, 305-0006, Japan
[2]Senior Researcher, Hyogo Earthquake Engineering Research Center, National Research Institute for Earth Science and Disaster Prevention, 1501-21 Shijimicho Mitsuda Nishikameya, Miki, Hyogo, 673-0515, Japan

For this reason, the authors carried out a series of tests of a large-scale model ground with a quay wall and group-pile-supported structure on lateral spreading due to liquefaction by the E-Defense shaking table (MEXT 2007). The objective of the testing series is reproduction of liquefaction-induced lateral spreading to observe the phenomena of the model ground and structures in detail. This experimental study involves revealing the failure mechanism of a quay wall and pile-supported structure behind the wall. The paper here describes the results of one of the tests of the model ground with a caisson-type quay wall and pile-supported structure, and explains their behaviors.

Outline of the test

E-Defense shaking table

The test was carried out by the E-Defense shaking table. E-Defense is the name of a full-scale three-dimensional earthquake testing facility operated by the National Research Institute for Earth Science and Disaster Prevention (Ohtani et al. 2003, Sato et al. 2004a). After the start of its operation in 2005, a lot of shaking tests on soil-structure interaction and collapse behavior of various structures have been successfully performed (Kajiwara et al. 2007, Tabata et al. 2009). The key of the E-Defense is a shaking table, which is the world's largest three-dimensional table 20 m long and 15 m wide as shown in Figure 1. The remarkable feature of the table presented in Table 1 is the capability to reproduce ground motions recorded in the 1995 Kobe earthquake for a 12-MN structure by 10 horizontal and 14 vertical actuators.

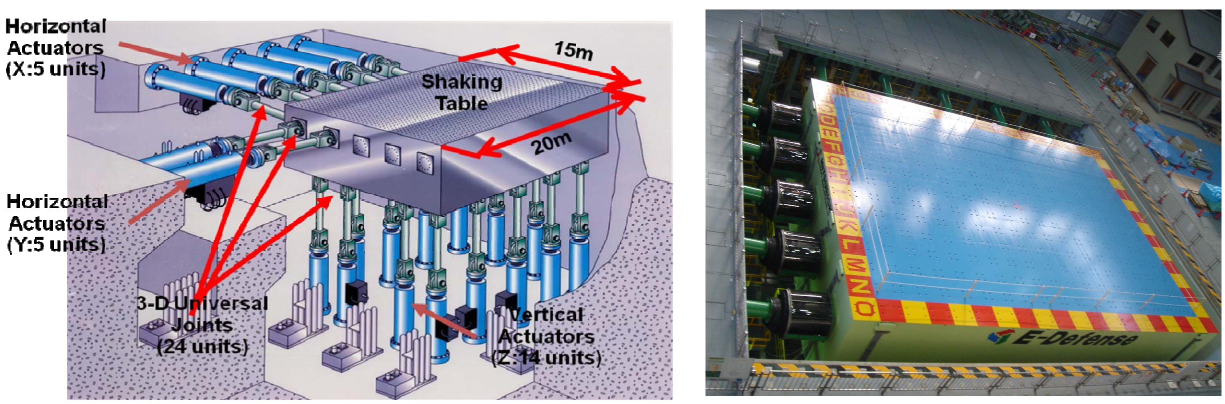

Figure 1. Illustration and photo of the E-Defense shaking table.

Table 1. Specifications of the E-Defense shaking table.

Table size	20m x 15m	
Loading capacity	12MN	
Maximum performance*	Horizontal (x and y)	Vertical (z)
acceleration	900gal	1500gal
velocity	2m/s	0.7m/s
displacement	±1m	±0.5m
allowable moment	150MNm	40MNm

* at the maximum load

Specimen of the model ground

Figure 2 and Photo 1 show the specimen of the model prepared in a rectangular container 16 m long, 4 m wide and 5 m high (Sato et al. 2004b). The specimen was a liquefiable ground with a caisson-type quay wall and structure supported by a 3-by-2 pile group. The liquefiable ground was made of Albany silica sand compacted to 60-percent relative density and saturated by de-aired water before testing. The properties and indices of the sand are presented in Table 2, and its deformation characteristics are described by Yasuda et al. (2006). The grain size distribution of the sand is also shown in Figure 3, indicating that the distribution is similar to that of Toyoura sand. The deposit was divided by the caisson into 2.5-meter-thick "waterside" and 4.5-meter-thick "landside" deposits. The water table was 0.5 m below the landside ground surface, i.e. 1.5 m above the waterside ground surface. In the landside deposit, the pile-supported structure was installed behind the caisson, consisting of six hollow steel piles, footing and the weight modeling a superstructure. The six piles were aligned with three parallel to the caisson in two rows and fixed to the footing and pinned at the container's bottom. The 10-ton footing penetrated into the landside deposit to a depth of 0.5 m. The weight was placed on the footing, inducing inertial force by a 12-ton weight.

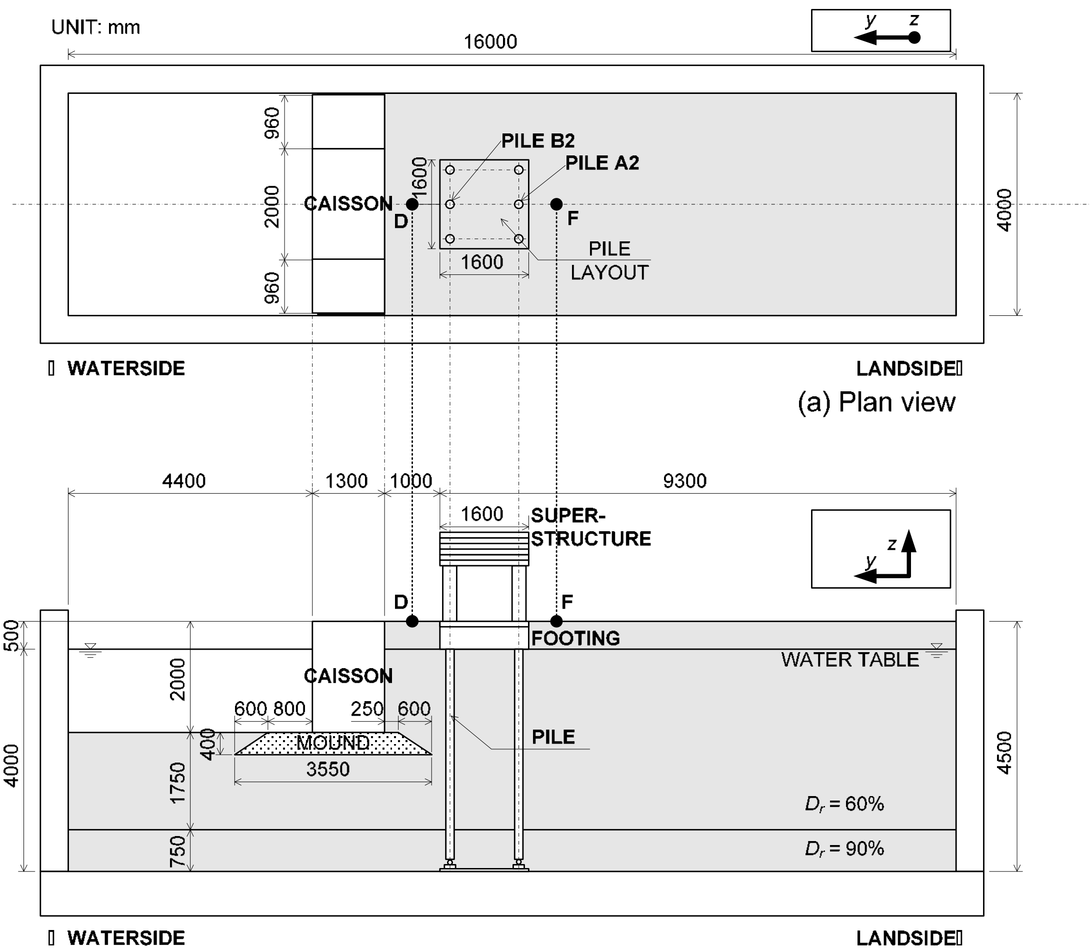

Figure 2. Illustrations of the specimen and the locations of the measurement points D and F : (a) Plan and (b) section views.

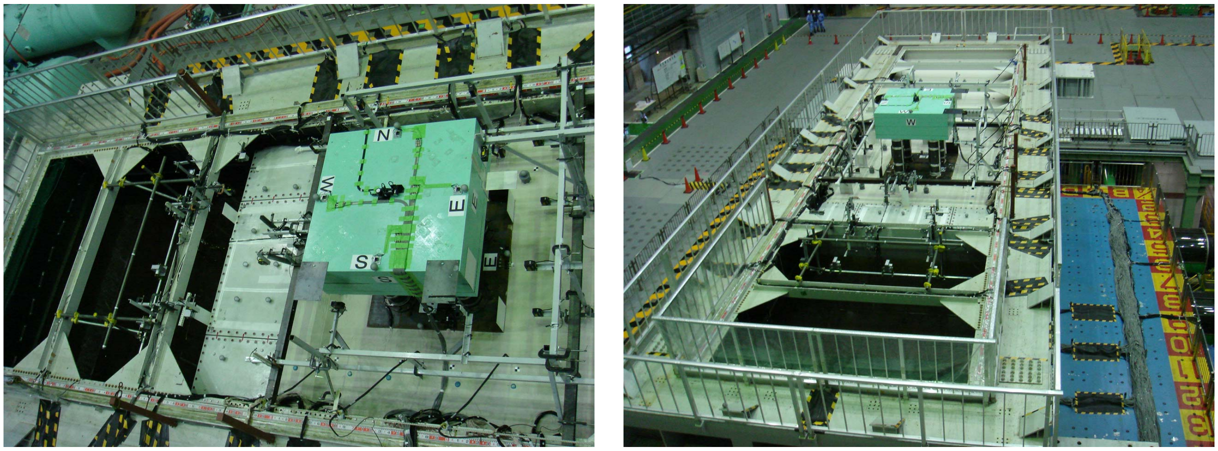

Photo 1. Specimen on the table before shaking.

Table 2. Properties and indices of Albany silica sand.

Density of soil particle	$\rho_s = 2.63$ g/cm^3
Maximum void ratio	$e_{max} = 0.783$
Minimum void ratio	$e_{min} = 0.513$
Mean grain size	$D_{50} = 0.20$ mm
Uniformity coefficient	$U_c = 1.64$
Coefficient of curvature	$U'_c = 1.13$

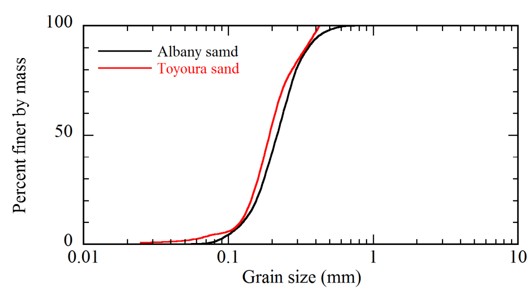

Figure 3. Grain size distributions of Albany silica sand and Toyoura sand.

Measurement installation

To achieve the objective , measurement sensors listed in Table 3 were installed with the specimen. The table presents a total of 836 sensors that were mounted to observe the behavior in detail. Additionally, a unique three-dimensional displacement measuring system was employed to investigate the motions of ground surface and structures (Tokuyama et al. 2007). In this system, displacement is determined by digital video cameras that observed reflective, spherical markers attached to an object.

Table 3. Sensors installed with the specimen.

Type of sensor	Ground	Caisson	Piles	Superstructure and footing	Container	Total
Strain gauge			234	40		274
Accelerometer	89	8	22	24	24	167
Veolcity transducer				3	3	6
Displacement transducer	11	14		11		36
Earth pressure transducer		17	104	16		137
Water pressure transducer	119	7	72			198
Load cell			18			18

Testing program

The specimen set on the table was shaken under two-dimensional, horizontal and vertical motions based on the north-south and up-down motions recorded at the JR Takatori station during the 1995 Kobe earthquake. In the test, the north-south component was applied to the specimen's long direction, and the up-down component to the vertical direction. The peak table accelerations were approximately 6.0 and 1.7 m/s^2 in the horizontal and vertical directions respectively, and the shaking duration was about 42 seconds. Figure 4a shows the acceleration time histories of the target input motion and actual, observed table motion from zero (data acquisition start time) to 30 seconds. Because the nature of the specimen was significantly changed due to liquefaction caused by strong motions, the table's control system could hardly follow such changes and reproduce table motions that were identical to the target. However, as shown in Figure 4b, both shapes of Fourier spectrum are very similar, especially in the domain of lower frequency.

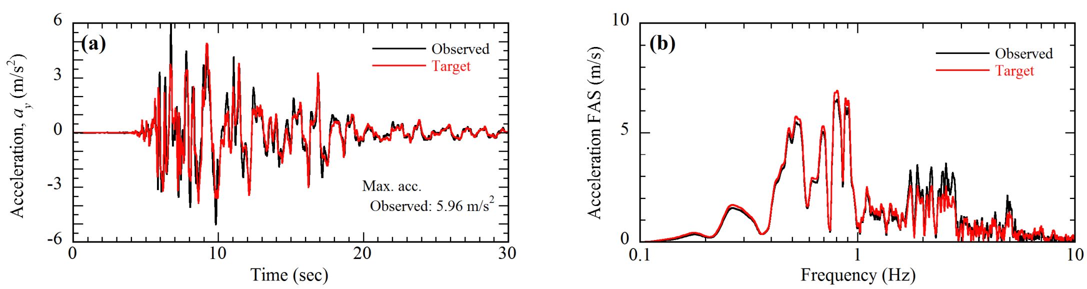

Figure 4. Comparison between the target input motion signaled to the table and actual, observed table motion: (a) Acceleration time histories and (b) acceleration Fourier amplitude spectra.

Test results

Observation of the specimen after test completion

Photo 2 shows the specimen before and after the test, and Figure 5 illustrates the section of the specimen that demonstrates its change due to shaking. The caisson overturned toward the waterside (left of the specimen in the photo) with the horizontal displacement at the top of about 2 m and 25-degree decline, accompanying its mound with horizontal deformation and small settlement. Three waterside piles bent at almost the same level of the mound. This, in turn, caused horizontal deformation of the pile-supported structure toward the caisson with 47-degree decline of the weight and footing.

As shown in Figure 5, horizontal displacement of the landside ground surface ranged up to 2 m and decreased with the distance from the caisson. Such trend is similar to some cases observed in the 1995 Kobe earthquake (Ishihara et al. 1996). The liquefaction and resulting lateral spreading also caused settlement of the landside deposit of about 20 cm, while relatively small settlement was observed at the waterside deposit.

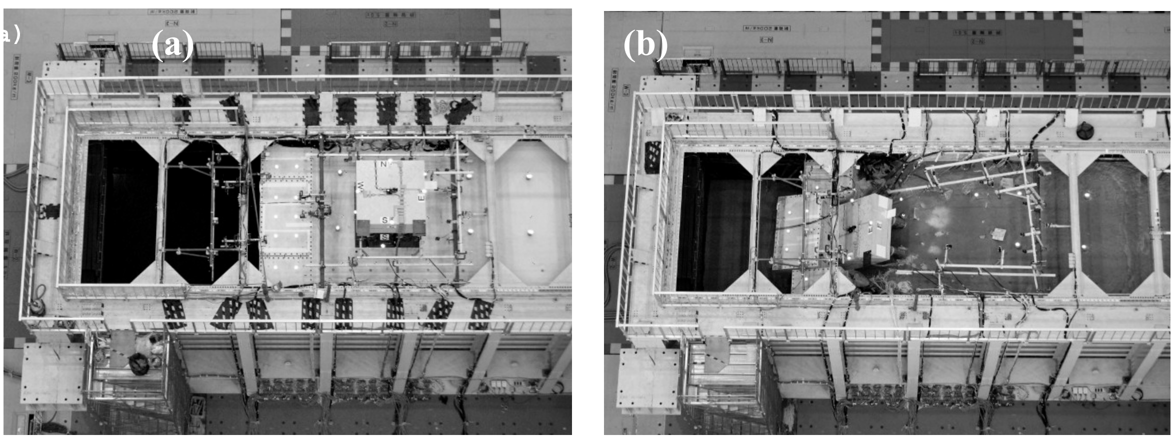

Photo 2. Caisson, pile-supported structure and their surroundings
(a) before shaking and (b) after test completion.

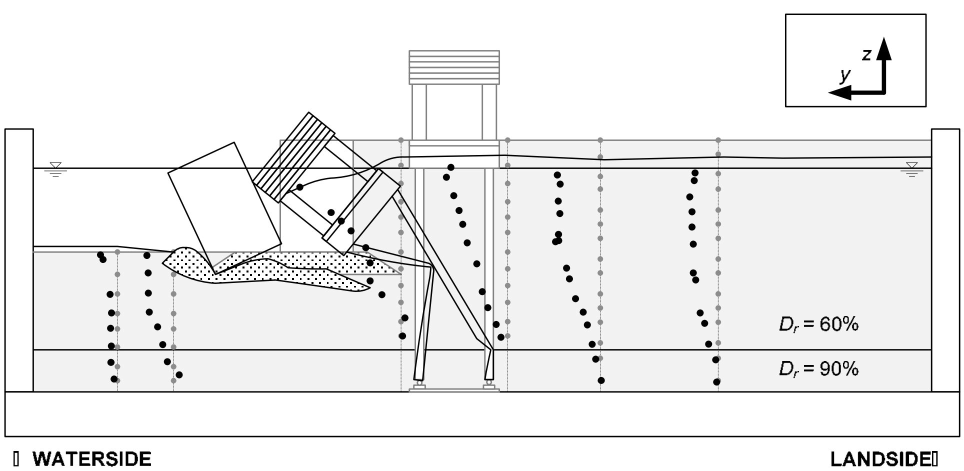

Figure 5. Section of the change of the specimen due to shaking. Gray and black colors indicate the structures and measurement points before and after the test respectively.

Excess pore water pressure change during and after shaking

Figure 6 shows the changes of excess pore water pressure, Δu, at four different levels (0.6, 1.7, 2.9 and 3.7 m from the landside surface) under the measurement point F. As shown in the figure, Δu build-up occurred at all levels when the excitation started, and then reached their overburden pressure in 7 to 10 seconds after the start. In consequence, it can be said that all layers of the deposit were liquefied due to the applied input motions. Following this Δu increasing process, Δu dissipation started from the bottom layer to the surface. This observation implies that liquefied deposit became dense from the bottom to the surface during the dissipation process of Δu. In addition, Δu began to dissipate when its pressure level reached that of the lower layer.

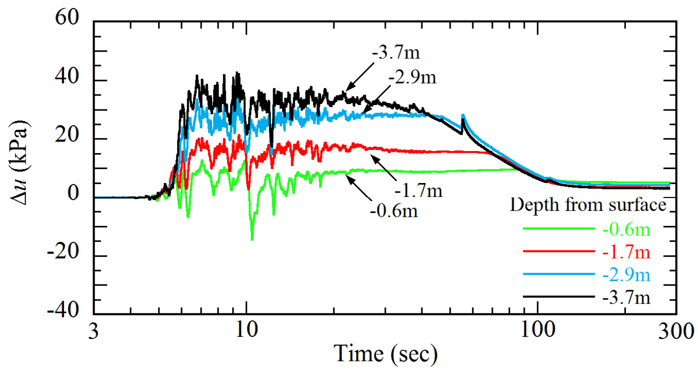

Figure 6. Excess pore water pressure changes at different levels under the point F.

Behavior of the caisson during shaking

Just after start of the shaking, the caisson moved to the waterside and landside alternately a few times, and finally overturned toward the waterside at 10.6 seconds after the test start. To evaluate the effects of its inertial force and backfill earth pressure change on the caisson's behavior, the time histories of the displacements and effective earth pressure changes are introduced as following. Figure 7 shows the vertical displacements, z, of the waterside and landside tops of the caisson, presenting decline of the caisson began at 6 seconds. Figure 8 shows the horizontal displacements, y, of the caisson, the measurement point D on its backfill, and the shaking table. In the figure, a positive y value means the displacement to the waterside. As shown in the figure, the horizontal displacement of the caisson is always larger than that of the point D with almost opposite phase of the table. Additionally, the effective earth pressure change on the caisson's landside, $\Delta p'$, at different levels from the surface shown in Figure 9 is constantly negative from 5 seconds to 10.6 seconds at which the caisson overturned toward the waterside. These facts suggest that the behavior of the caisson during shaking was principally dominated by its inertial force rather than the $\Delta p'$ change of the backfill. Indeed, many cracks on the ground surface and gaps between the caisson and its backfill are observed in Photo 3, which is the close view of the caisson and its surroundings including the footing and the point D around at 8 seconds. Therefore, it can be said that the influence of caisson's inertial forces is more dominant to the stability of the caisson under earthquake motions than that of the $\Delta p'$ change of its backfill.

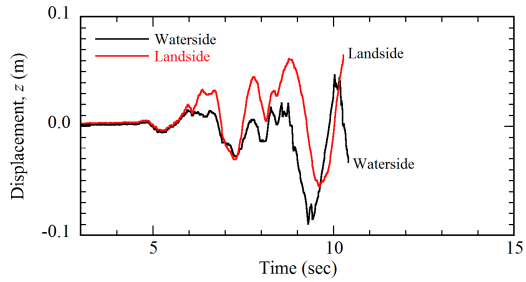

Figure 7. Vertical displacements determined at the waterside and landside tops of the caisson.

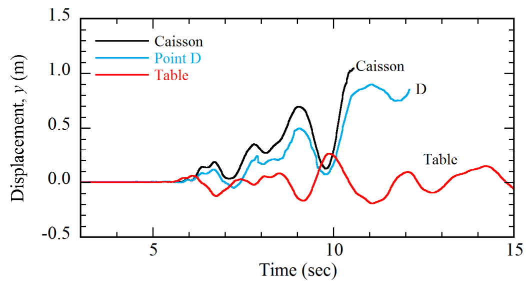

Figure 8. Horizontal displacements of the caisson, point D and shaking table.

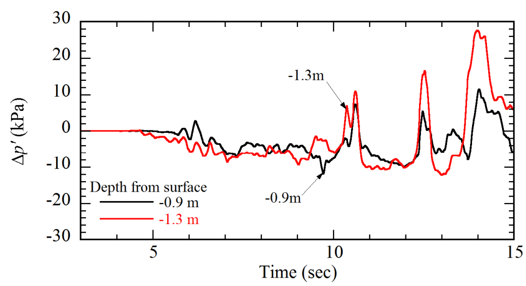

Figure 9. Effective earth pressure changes on the backfill side of the caisson.

Photo 3. Close view of the caisson, backfill surface and footing during shaking.

Behavior of the pile-supported structure during shaking

Figure 10 shows horizontal displacements of the caisson, weight of the pile-supported structure and their surroundings including the measurement points D and F. As already explained, the caisson overturned toward the waterside at 10.6 seconds after the test started. Until the caisson overturned, the horizontal displacement of the point D on the backfill of the caisson was always smaller than that of the caisson, while the displacement of the weight was smaller than those of the caisson and point D and almost same behavior of the point F on the ground behind the pile-supported structure. It is also observed that these displacements change with nearly same phase. Just after the caisson overturned at 10.6 seconds, the displacement of the weight reached the exactly same of the point D and increased, and finally declined to the waterside. In this process, the displacement of the point F was smaller than those of the weight and point D. This observation implies that one of the triggers to decline the pile-supported structure can be the large deformation of the ground between its footing and the caisson due to the caisson overturned.

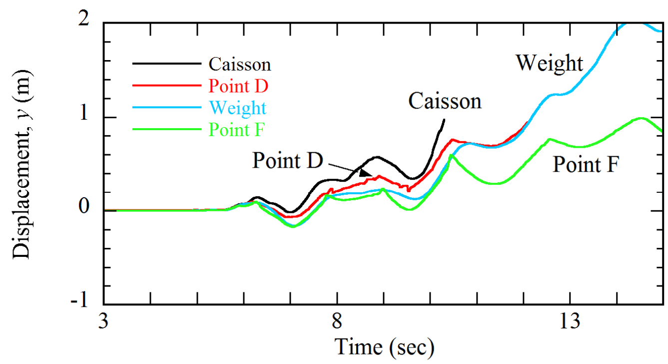

Figure 10. Horizontal displacements of the caisson, weight, and points D and F.

Conclusions

In order to investigate the behavior of liquefaction-induced lateral spreading and the mechanism of its influence on structures, a shaking table test of a large-scale model ground with a caisson-type quay wall and group-pile-supported structure assumed as a situation of port areas was conducted at the E-Defense shaking table facility. In the test, about nine hundred sensors monitored the behavior in detail and the three-dimensional displacement measuring system was employed to observe large displacements.

The specimen of the model ground was shaken under two dimensional, horizontal and vertical motions based on one of the 1995 Kobe earthquake records. Such motions induced liquefaction in all layers of the deposit, causing overturn of the caisson toward the waterside and horizontal deformation of the landside ground. According to the displacement and effective earth pressure change of the caisson, the overturn was mainly caused by its inertial force, not the influence of its backfill ground deformation. After the caisson overturned and the following deformation of its backfill ground caused, the pile-supported structure declined to the waterside. It can be considered that this phenomenon was triggered by large deformation of the ground behind the caisson. Note that these observations and considerations are based on the model testing results, meaning that the observed area is still limited compared to the actual field.

Hence, to apply the knowledge from this study to practice, other research procedures, such as parametric testing of small models, field investigations and computational analyses, as well as more large-scale model testing are needed.

Acknowledgments

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