30 Mar 2001, 1:30 pm - 3:30 pm

Shaking Table Tests on Seismic Deformation of Composite Breakwaters

Hideo Sekiguchi
Kyoto University, Japan

Hayoung Kim
Toyo Construction Co., Ltd., Japan

Katsutoshi Kita
Tokai University, Japan

SHAKING TABLE TESTS ON SEISMIC DEFORMATION OF COMPOSITE BREAKWATERS

Hideo Sekiguchi
Kyoto University
Uji, Kyoto-Japan-611-0011

Hayoung Kim
Toyo Construction Co. Ltd.
Nishinomiya, Hyogo-Japan-663-8142

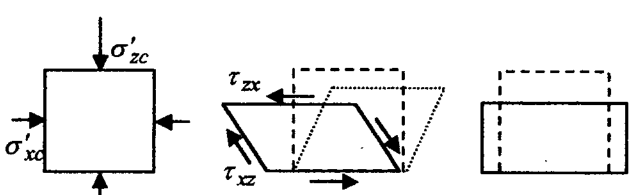
Katsutoshi Kita
Tokai University
Shimizu, Shizuoka-Japan-424-8610

ABSTRACT

A series of shaking table tests were conducted on composite breakwater systems under 30g centrifugal conditions. The emphasis was placed on investigating the mechanisms of seismic settlement of foundation ground, in particular of the contribution from the dispersion of mound rubble into the foundation soil. In-flight visual observation of the deformation process by a high-speed CCD camera showed the significance of cumulative vertical compression of foundation soils under cyclic loading. Pre- and post-shaking comparisons of weight of the rubble mound revealed that dispersion into the soil had occurred and contributed to caisson settlement.

INTRODUCTION

Composite breakwaters in the port of Kobe, Japan suffered from large settlements over 2.5m due to strong ground shaking during the January 1995 Hyogoken-Nambu Earthquake. These breakwaters were constructed on sandy fill that replaced the original clay seabed. Sekiguchi et al. [1996, a] conducted underwater acoustical surveying around the breakwaters, and observed some characteristic features of sand boils on the seabed surface in the free field nearby the rubble mound. This indicated the occurrence of liquefaction in the sandy fill of the free field. However, based on the results of shaking table tests of structures placed on liquefiable sand conducted by Yoshimi and Tokimatsu [1977], Sekiguchi et al. inferred that liquefaction had not occurred under the caisson itself due to suppression by the effective weight of the caisson and rubble mound. So the important question arises: how the breakwaters underwent large settlements of up to 10 % of the thickness of foundation sand fill without the occurrence of liquefaction.



(a) Initial state (b) Undrained cyclic pure shear (c) State of $\gamma_{zx} = 0$ ($\tau_{zx} \approx 0$)

Fig. 1 Deformation process of a soil element subjected to undrained cyclic pure shear (adapted from Sekiguchi et al. [1996, b])

One possible mechanism is schematized in Fig. 1. This would apply to the behavior of the sand under the rubble mound. Sekiguchi et al. [1996, b] carried out a series of undrained cyclic torsional shear tests on saturated silica sand, and attributed the large settlement of breakwaters to cumulative axial deformation during cyclic pure shear loading as shown in Fig. 1. Anisotropically consolidated specimens were subjected to cyclic torsional shearing under undrained conditions, and the specimens showed significant axial compression in the vertical direction (i.e. the initial direction of major principal stress) accompanied by lateral expansion, after the effective stresses reached the phase transformation state. This mechanism of deformation has been consistent with the finite element analyses of composite breakwater behavior performed by Iai et al. [1998].

A second possible mechanism relates to mass transport through the interface between the rubble mound and the foundation soil. Peiris et al. [1998], in their centrifugal shaking table tests on gravel embankments resting on saturated loose sand deposits, observed significant penetration of gravel particles into the fine-grained foundation soils. This increased the settlement of embankments compared with the sand embankment cases, where little penetration of embankment materials was observed.

This paper discusses the deformation behavior of composite breakwater-foundation systems during earthquake shaking in view of the mechanisms mentioned above. A series of centrifugal shaking table tests were conducted and pore pressures measured, with the deformation processes traced by means of a high-speed CCD camera.

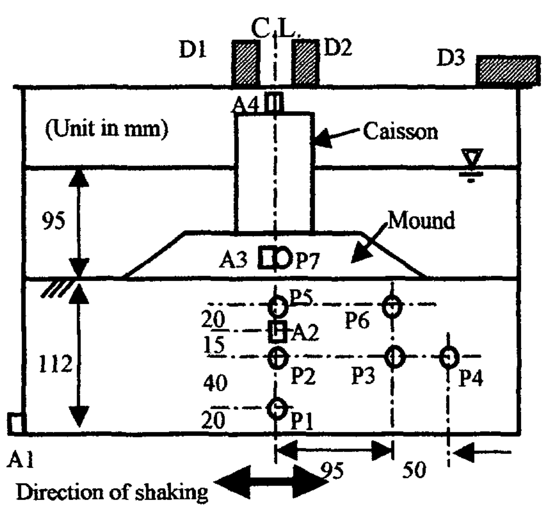
EXPERIMENTATION

All of shaking table tests described in this paper were conducted under 30g centrifugal conditions in a 2.5m-radius centrifuge at Disaster Prevention Research Institute of Kyoto University. These included a test program conducted by Kim et al. [1999]. More recently, a new test program has been carried out, with the five cases listed in Table 1.

Configuration of the model composite breakwaters and location of sensors are typically those as shown in Fig. 2. A horizontal layer of saturated foundation ground (112 mm – 124 mm thick) was formed in a strong box (400mm long, 100mm wide and 280mm deep) by pouring fine silica sand into a sea of de-aired silicone oil. The index properties of the silica sand are listed in Table 2. Two kinds of fine silica sand, denoted types A and B in Table 2, were used. These had the same mean grain size but different specific gravities and minimum/maximum void ratios. Silicone oil with a viscosity of 30 cSt was used as a substitute fluid for the pore fluid as well as for the fluid around the breakwater, in order to match the time scaling of consolidation to that of soil vibration. The foundation ground was preconsolidated under a centrifugal acceleration of 30g for 30 minutes. A trapezoidal mound was then constructed by pouring medium- or coarse- grained material onto the foundation ground. The index properties of the mound materials are also listed in Table 2. The top width, base width and height of the mound ranged 120 mm – 125 mm, 230 mm – 240 mm, and 30 mm – 37 mm respectively. A model caisson 60 mm wide, 99 mm long and 100 mm high, with a mass of 1.42 kg, was placed on top of the mound. Then the fluid level was raised to a height of 95 mm above the surface of the foundation ground.

Table 1 Experimental cases recently performed

Case No.	Dr of Ground	Mound Material	Caisson settlement	Settlement by dispersion
1	46%	Medium sand	17mm	1.3mm
2	50%	Gravel	15mm	3.4mm
3	15%	Gravel	55mm	8.7mm
4	29%	Medium sand	35mm	1.2mm
5	37%	Gravel	34mm	6.4mm



A1~A4 : Accelerometers (Horizontal)
P1~P7 : Pore pressure transducers
D1, D2 : Displacement transducers (Vertical)
D3 : Displacement transducer (Horizontal)

Fig.2 Typical model configuration

Table 2 Index properties of the soils used

Material	Grain size(mm)	Gs	e_{max}	e_{min}	Remarks
Fine sand Type A	0.1 (D_{50})	2.65	1.15	0.69	Ground Cases 1, 2, 3
Fine sand Type B	0.1 (D_{50})	2.68	1.28	0.76	Ground Cases 4, 5
Medium sand	0.85-2.0	2.64	-	-	Mound Cases 1, 4
Gravel	4.76-9.5	2.71	-	-	Mound Cases 2, 3, 5

Accelerometers and pore pressure transducers were installed during the model formation. Three laser displacement transducers were attached to the model container, in order to measure the deformation of the caisson. All the accelerometers were instrumented to detect horizontal accelerations.

Analog signals from the sensors described above were amplified and converted to digital signals in a signal conditioner that was mounted on a rotating arm. The digital signals were transferred through slip rings to a personal computer in an observation room. The sampling frequency selected was 1000Hz, and the sampling duration was 4 seconds.

The front side of the model container was made of transparent glass. For visual observation purposes, line and spot markers of colored sand were placed in the ground.

A high-speed CCD camera system was used for observing the deformation process during shaking (Fig.3). The CCD camera was attached to a rotating arm, and captured the images on the glass side of the model that was reflected by a mirror. The images were recorded in electrical memories of a CCD controller, and transferred to a personal computer after the experiment for detailed data processing. The recording rate used was 250 frames per second, with a recording duration of 3.256 s. The CCD had 640 by 240 pixels with 24 bit color resolution. Pictures were finally processed to 640 by 480 pixels.

After ensuring the electrical connections of measurement and observation systems, each of the model composite breakwaters was spun to arrive at the 30 g centrifugal acceleration and then subjected to sinusoidal excitation in a horizontal direction by using an electro-hydraulic shaker. The shaking amplitude and frequency used were 5 g and 30 Hz respectively. These

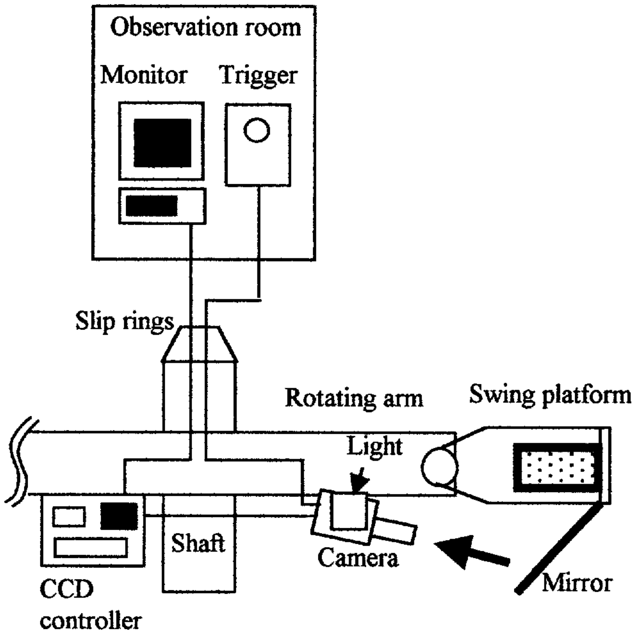


Fig. 3 High-speed CCD camera system

correspond to the acceleration amplitude of 1.63 m/s^2 and the frequency of 1.0 Hz on prototype scale. The number of the sinusoidal waves imposed was 20.

EXPERIMENTAL RESULTS

Of the five shaking table tests carried out (Table 1), cases 1 and 2 dealt with the medium dense foundations (relative density Dr = 46% - 50%). Cases 3 through 5 were conducted on loose foundations (Dr = 15% - 37%). The mounds were constructed with medium-grained material in cases 1 and 4, and with coarse-grained material in other cases.

Caisson settlement, acceleration and pore pressure responses

Let us first look at the performance of case 1. The measured time histories of caisson settlement, accelerations at four different locations as well as of excess pore pressures at five different soil locations are shown in Fig. 4. The locations of the transducers are shown in Fig. 2. The caisson settlement plotted was the average of the settlements that were measured with the two laser displacement transducers (D1 and D2). The input accelerogram (A1) is shown in the left lowermost diagram of Fig. 4.

It is seen from Fig. 4 that the caisson settlement increased continuously from the beginning of cyclic loading and reached 17 mm at the end of the main earthquake shaking. Post-shaking surveying showed that the caisson top settled by 17 mm and confirmed that a large portion of the settlement took place during shaking. The acceleration at the top of the caisson (A4) amplified relative to the input acceleration (A1) in the first several cycles of shaking. The acceleration amplitude at the caisson top decreased with time thereafter.

It can be seen that the acceleration amplitude measured in the foundation ground (A2) decreased gradually with time and reduced to about 50 % of the input acceleration (A1) at the end of the main shaking (t = 900ms). This particular waveform indicated that the soil location did not undergo liquefaction. The acceleration response of the mound (A3) shows that it was slightly amplified relative to the ground response (A2). This suggested that the upper part of the foundation ground remained stiff compared to the middle or lower part of the ground.

The excess pore pressure measured with transducer P4, which was located in the free field, developed markedly from the beginning of the earthquake shaking. The residual pore pressures reached 15kPa within the first few cycles. Note that the initial effective vertical stress at that location estimated from the elasticity theory was 15kPa. Thus, the upper part of the foundation ground of the free field underwent liquefaction. The residual part of the excess pore pressure measured with transducer P6, which was located at the shallow soil depth below the mound slope, increased with time, indicating gradual deterioration of the soil stiffness there. This deterioration should relate to reduction in the effective lateral confinement of the soil just beneath the caisson and the rubble mound.

The pore pressure responses measured at three different depths along the centerline of the model (P1, P2 and P5) are compared below. The excess pore pressure at the shallowest location (P5) took negative values from the beginning of shaking and stayed negative until the end of the shaking. The excess pore pressure then increased rapidly to positive values. Because the pore pressure transducer at that location settled with the ground deformation, the hydrostatic pore pressure should have increased during shaking. These considerations led us to conclude that the soil at that location underwent strong dilative behavior during shaking. The excess pore pressure observed at the mid depth (P2) initially decreased to negative values, however turned to increase with time after t = 400 ms. It again increased rapidly after the excitation was stopped. The excess pore pressure developed positive at the deepest location (P1). However, the maximum residual pore pressure did not exceed the initial vertical effective stress estimated for the point (46kPa).

Let us discuss the performance of case 4 where the medium-grained mound was constructed on the loose sand deposit. The pattern of the observed caisson settlement was essentially the same as in case 1. It should be emphasized here that the positive excess pore pressures developed during shaking at the mid depth on the centerline of the model ground.

Deformation of foundation ground

Time-histories of vertical and horizontal strains of the soil beneath the caisson were obtained from pictures of a high speed

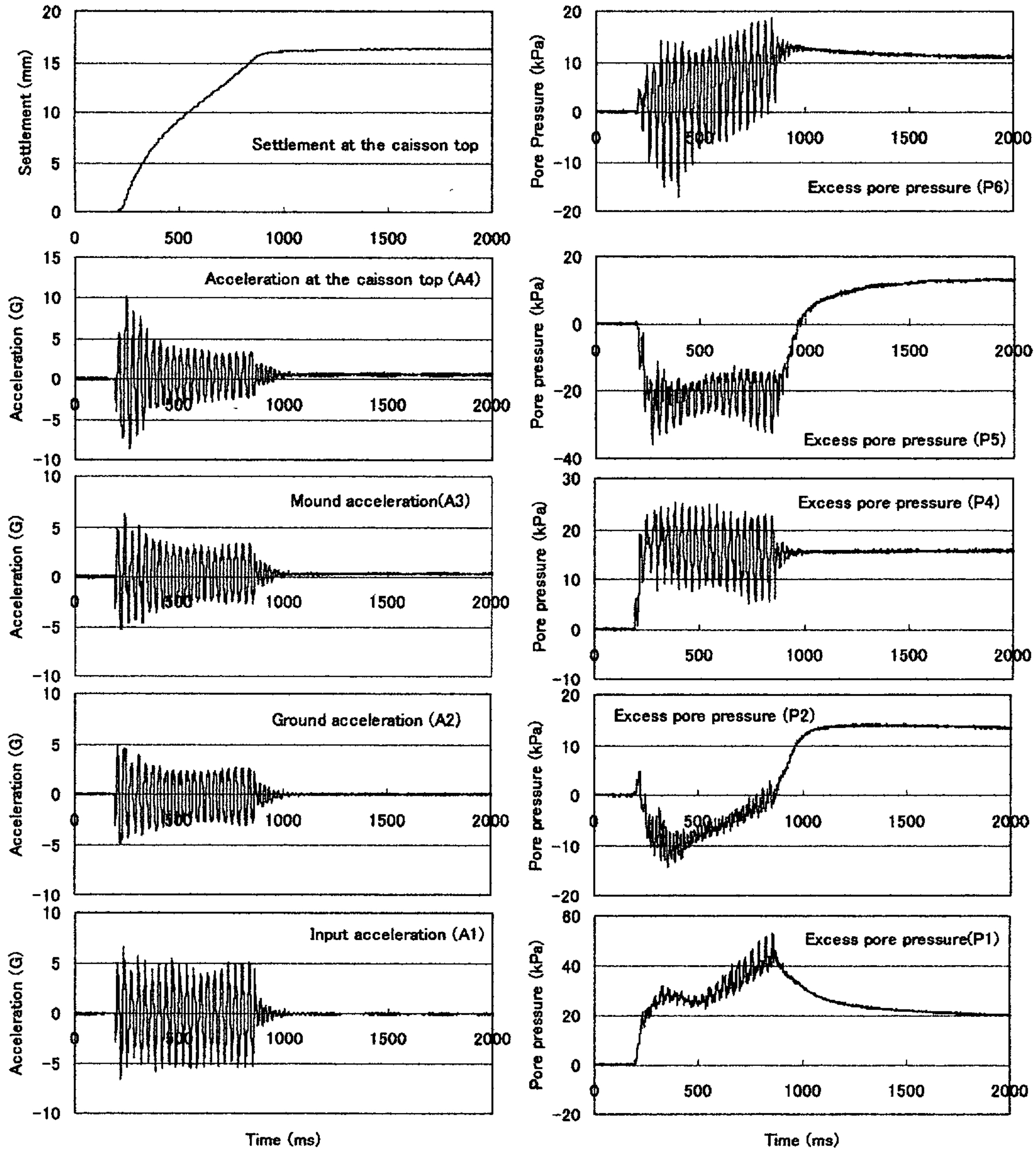


Fig.4 Time histories of settlement, accelerations and excess pore pressures (Case 1)

CCD camera by reading the temporal changes in the distances between the line and spot markers. A set of typical results are plotted in Fig. 5 for case 4. Compressive strains were taken positive in the figure. It can be seen that the strains within the foundation ground began to develop at t = 200ms when the excitation was started and leveled off around t = 900ms when the shaking was stopped. Vertical compressive strains at t = 900ms were 45% for the middle layer and 25% for the upper layer. The vertical compressions were accompanied by significant horizontal extensional strains. The middle layer showed somewhat rapid development in horizontal extension in the early stage of shaking (up to 500ms), compared with that in the upper layer.

Figure 6 shows two pairs of photographs that were taken before and after shaking in each of cases 1 and 4. The mounds were formed with medium grained sand, and thus the dispersion of mound materials was insignificant. One may note the clear boundary lines between the mound and the foundation ground even after shaking. It is noteworthy that the soil beneath the caisson was vertically compressed and was horizontally extended considerably. The vertical compressive strains of the foundation ground in case 1 were measured as 7%, 15 % and 21% for the upper, the middle and the lower layers respectively. In essence the vertical strain increased with depth in this case. By contrast, in case 4, the middle layer was most intensively compressed.

Close views of the mound in case 5 before and after shaking are depicted in Fig. 7. Note here that the mound was made from gravel particles and rested on the loose sand deposit. Gravel particles penetrated into the foundation sand (e.g. see the gravel particle marked by arrows). The settlement caused by dispersion was found to be equal to 6.4 mm in this particular case. It

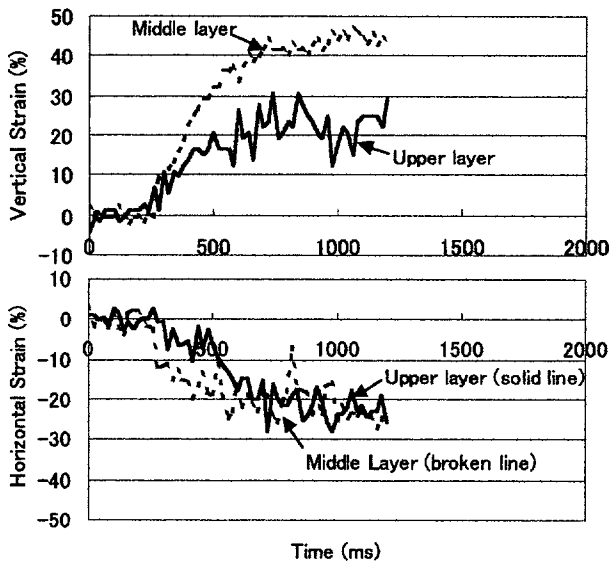


Fig.5 Measured strain-time histories during shaking

accounted for 20 % of the total final settlement of the caisson that was equal to 34 mm. This performance is well contrasted to case 4 in which the mound was constructed from the medium sand.

DISCUSSION

The foregoing sections have emphasized that the soils beneath the main part of the composite breakwater undergo large vertical compression together with significant horizontal extension. These deformations were accompanied by the negative excess pore pressure development at shallow soil depths. This development of negative pore pressures may be indicative of dilative soil behavior under conditions of high effective stress ratio. Note here that the soils just beneath the main part of the composite breakwater should initially be in states of anisotropic confining stresses. Thus, if strongly sheared, they would readily reach the phase transformation state. Once the effective stress path crossed the phase transformation line, further straining would induce negative excess pore pressures markedly in a manner schematized in Fig. 8 (a). Such development of negative excess pore pressures may be accentuated by the horizontal soil extension due to mound penetration (Fig. 8 (b)).

The final settlements of the caisson, S, normalized by the foundation thickness, D, are plotted in Fig. 9 against relative densities, Dr, of the foundation sand. In this figure the results from the centrifuge experiments by Kim et al. [1999] together with two additional tests are also plotted with solid triangles. It should be noted that Kim et al. employed sinusoidal input with the seismic intensity of 0.11 (shaking amplitude/centrifugal

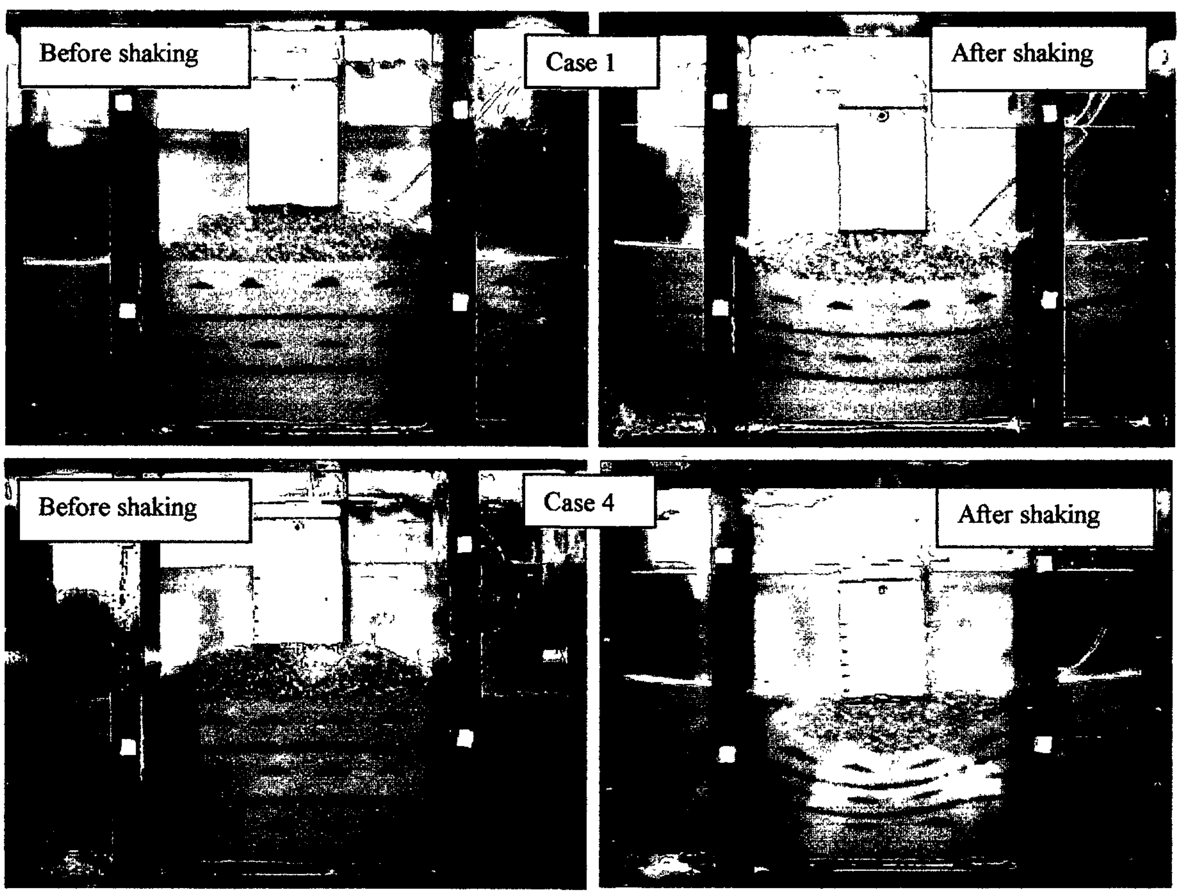


Fig.6 Photographs showing ground deformation

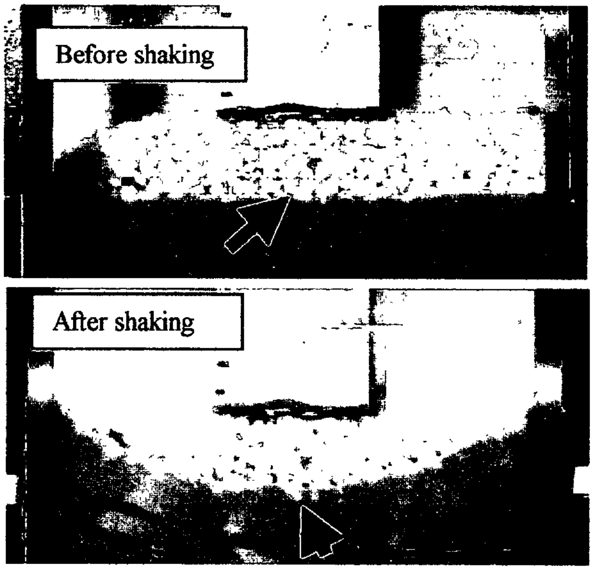


Fig. 7 A pair of photographs showing penetration of gravel particles into the foundation sand (case 5)

acceleration), whereas the seismic intensity for the present study was 0.17. It is seen that the normalized settlement tends to decrease with increasing relative density. A closer look at these data points will show that the data obtained in the present study show larger values of normalized settlement because of the larger input accelerations.

CONCLUDING REMARKS

1. The in-flight observations by means of a high-speed CCD camera as well as the pre- and post-shaking photography showed that the foundation sand below the composite breakwaters were markedly vertically compressed and horizontally extended. This deformation pattern is considered to be consistent with the mechanism schematized in Fig. 1.

2. Gravel particles of mound penetrated into the fine-grained foundation soil during ground shaking. The settlement caused by dispersion accounted for 20 % of the total final settlement of the caisson in this particular case.

REFERENCES

Iai, S., K. Ichii, H. Liu and T. Morita [1998] Effective stress analyses of port structures, Soils and Foundations, Special Issue No.2 on Getechnical Aspects of the January 1995 Hyogoken-Nambu Earthquake, pp. 97-114.

Kim, H., H.Sekiguchi, and A. Takebe [1999] Deformation mechanism of composite breakwaters under earthquake shaking, Proc. 11th Asian Regional Conf. Soil Mech. and Geotech. Eng., Vol. 1, pp.499-502.

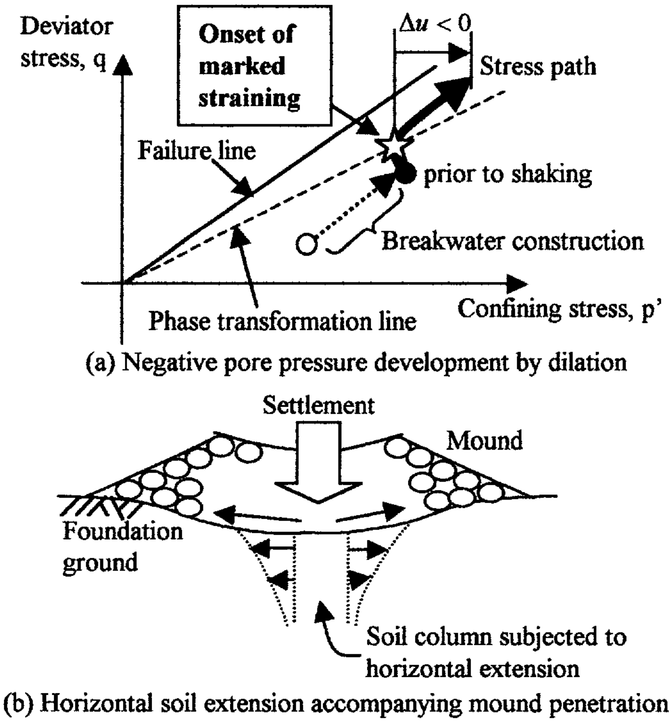


Fig.8 Mechanisms of negative pore pressure development

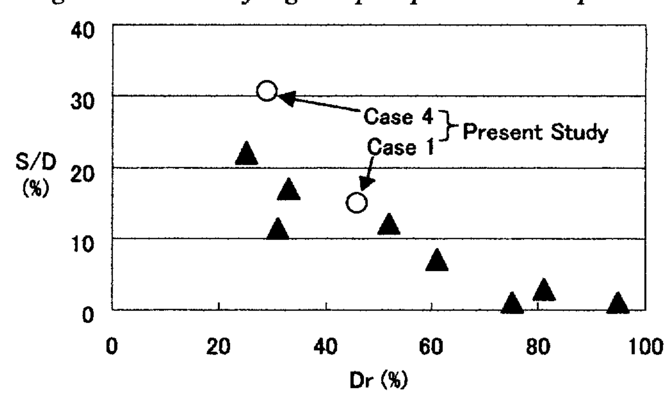


Fig.9 Relationship between the normalized settlement and relative density

Peiris, L. M. N., S. P. G. Madabhushi and A. N. Schofield[1998] Dynamic behaviour of gravel embankment on loose saturated sand, Proc. Int. Conf. Centrifuge 98, Vol. 1, pp.263-270.

Sekiguchi, H., K. Kita, K. Hashimoto and H. Katsui [1996, a] Deformation of composite breakwaters due to ground shaking, Soils and Foundations, Special Issue on Getechnical Aspects of the January 1995 Hyogoken-Nambu Earthquake, pp. 169-177.

Sekiguchi, H., K. Tanaka, S. Sassa and K. Kita [1996, b] Performance of undrained cyclic torsional shearing of saturated sands, Proc. First JSPS-NTU Seminar on Integrated Engineering, Kyoto, pp.50-59.

Yoshimi, Y. and K. Tokimatsu [1977] Settlement of buildings on saturated sand during earthquakes, Soils and Foundations, Vol.17, No.1, pp.23-38.