SHAKE TABLE TESTING ON SEISMIC PERFORMANCE OF GRAVITY QUAY WALLS

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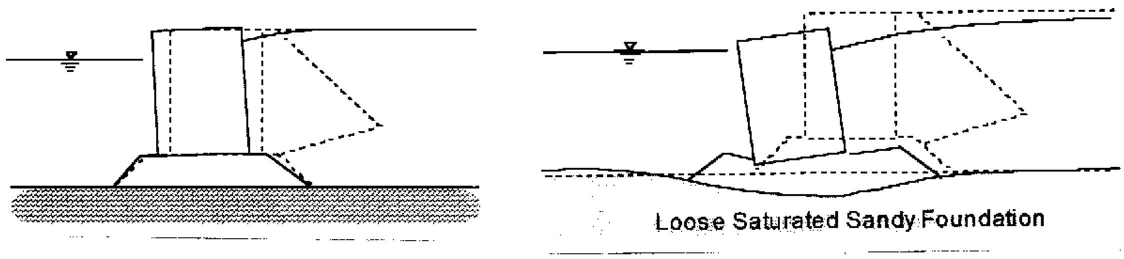
SUMMARY

This paper discusses the effects of excess pore water pressure increase in the backfill and foundation soils on the seismic performance of gravity quay walls based on the shake table tests. The first series of shake table tests are conducted to evaluate the capability of shake table tests in simulating the seismic performance of the prototype. The models scaled in 1/17 of the prototype are successful to simulate the prototype performance, which displaced 2.8 m toward the sea, settled 1.1 m and tilted about 4% during the Hyogoken-Nambu (Kobe) earthquake of 1995. The second series of shake table tests are conducted to evaluate the major parameters affecting the seismic performance of gravity quay walls. By varying the geotechnical conditions in backfill and foundation soils, the effects of the excess pore water pressures in the backfill and foundation soils increases the deformation about twice as large as that without effect of the excess pore water pressure increase. The effects of excess pore water pressure increases in the backfill and foundation soils on the deformation of the caisson wall were about 1:2.

INTRODUCTION

A gravity quay wall is made of a caisson or other rigid wall put on the seabed, and maintains its stability through friction at the bottom of the wall. When the wall is constructed on a firm foundation, deformation/failure during earthquakes is caused by a large inertia force on the body of the wall plus an increase in earth pressure from the backfill, and results in the seaward movement of the wall as shown in Fig. 1(a). If the width to height ratio of the wall is small, tilt may also be involved. Case histories for gravity quay walls subjected to earthquake shaking often belong to this category. When the soil deposit below the gravity wall is loose and excess pore water pressure increases in the subsoil, however, the movement of the wall is associated with significant deformation in the foundation soil, resulting in a large seaward movement involving tilt and settlement as shown in Fig. 1(b). The objective of this study is to discuss the latter mode of deformation/failure by reviewing the results of shake table tests [Sugano et al. 1996]. In particular, focus of attention will be directed to the effects of the excess pore water pressure increase in the backfill and /or foundation soil.



(a) On firm foundation (b) On loose saturated sandy foundation

Figure 1 Deformation/failure modes of gravity quay wall

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SHAKE TABLE TESTS

The results of the shake table tests reviewed will be categorized in two series. The first series simulated the actual geotechnical and structural conditions of caisson walls in Kobe Port during the Hyogoken-Nambu earthquake of 1995, and provided the detailed seismic performance data, including time histories of accelerations and excess pore water pressures in the various parts of the soil-structure system. The second series produced the relevant data for studying variation in the seismic performance of the caisson wall depending on the varitations in the excess pore water pressure increase in the backfill and/or foundation soils. Before going into the discussion on the mode of deformation/failure of caisson type quay walls, an outline of the model tests will be given below.

Many of the caisson walls in Kobe Port were constructed on a loose saturated backfill foundation of decomposed granite, which was used for replacing the soft clayey deposit in Kobe Port to attain the required bearing capacity of foundation. Shaken with a strong earthquake motion having the peak accelerations of 0.54g and 0.45g in the horizontal and vertical directions, these caisson walls were displaced an average of 3 m (maximum displacement = 5 m) toward the sea, settled 1 to 2 m and tilted about 4 degrees toward the sea. Figure 2 shows a typical example of the cross section and the deformation after the earthquake [Inagaki et al, 1996]. The geotechnical investigations were performed to evaluate the soil properties, including cyclic triaxial tests of undisturbed samples 60 cm long with a diameter of 30 cm obtained by an in-situ freezing technique The cyclic triaxial tests showed cyclic mobility type behavior and never exhibited the strain softening type behavior as shown in Fig. 3 [Ichii et al. 1998].

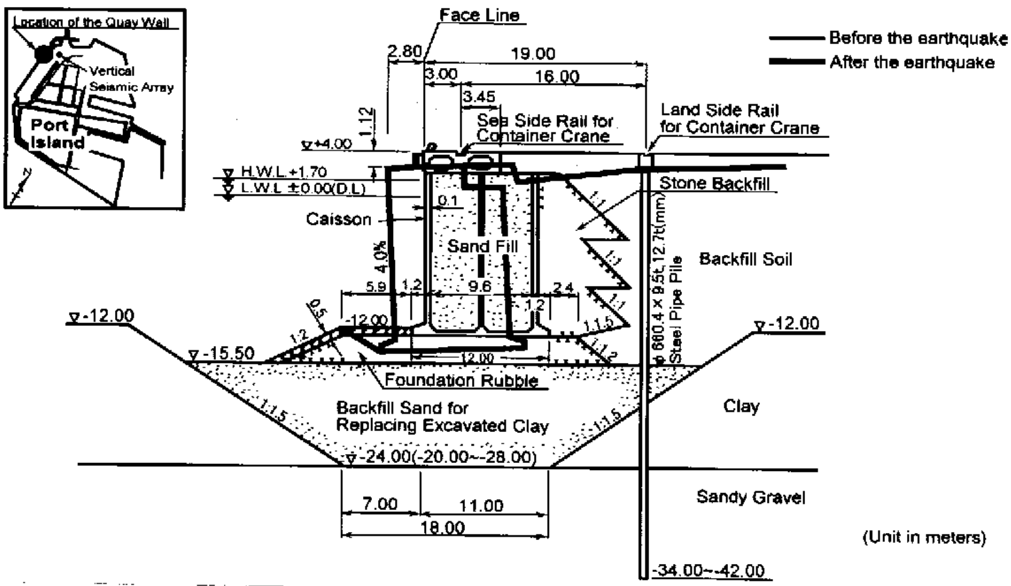


Figure 2 Location, cross section and residual deformation of a caisson quay wall at Port Island, Kobe Port during Hyogoken-Nambu earthquake of 1995

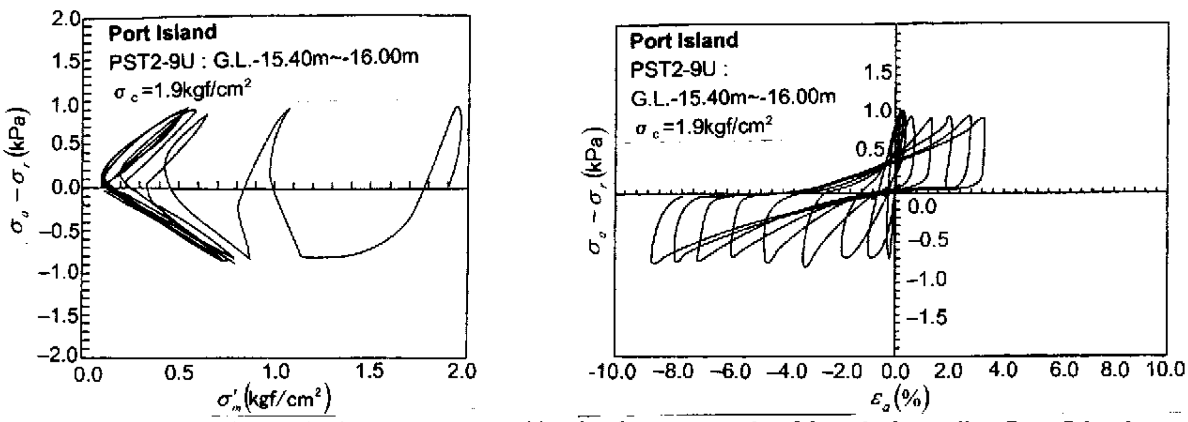


Figure 3 Undrained triaxial tests results of in-situ frozen sample of foundation soil at Port Island

In the shake table tests, the caisson type quay wall shown in Fig. 2 was modeled at a scale of 1/17 of the prototype [Sugano et al. 1996; Inagaki et al. 1996]. The quay wall model including foundation and backfill soils was made in a steel container 3.5 m long by 1.5 m wide and 1.5 m deep on a shaking table, which was set in the middle of a water pool 2 m deep and 15 m by 15 m wide to simulate the effect of sea water. The decomposed granite obtained from a site nearby the quay wall was pluviated into the water to simulate the dumping process of backfill soils at the quay wall construction.

The cross section of the model quay wall is shown in Fig. 4. The front end (i.e. left hand side in the figure) of the container above the model seabed level was open to the water pool whereas the back end of the container was sealed with unwoven textile reinforced with steel wire mesh to relieve adverse effects of rigid boundary conditions. Both sides of the container, however, were made of rigid steel plates to constrain the normal component of ground strain between these plates. Three model caisson were placed along the quay wall face line 1.5 m long and the caisson in the middle was used for monitoring accelerations and displacements.

The clay layer at the quay wall site was idealized in the model test by densely compacted layer of coarse grained Soma sand as shown in Fig. 4 to simulate the stable behavior of the clay layer during the earthquake. The surface of this layer was sealed with a thin bentonite layer to simulate the impermeable behavior of the clay layer during the earthquake. The layer above the water table was modeled by using gravel in stead of using the decomposed granite. This is to avoid effects of suction to be too strong relative to those of prototype.

The similitude in 1g field for soil-structure-fluid system [Iai 1989; Iai & Sugano 1999] was adopted for the model tests. Due to the fact that the shear modulus of sand in small strain level is proportional to the square root of the confining pressure, the scaling relation includes a scaling factor for strain as shown in Table 1. Three dimensional shaking was applied using the subsurface motion recorded at a depth of 32 m by the vertical seismic array at Port Island in Kobe Port, shown in the inset of Fig. 2, successfully recorded by Kobe City. The peak accelerations were 544, 461, and 200 cm/s^2 in NS, EW and UD direction. The input motion was applied in accordance with the direction of the quay wall, facing west.

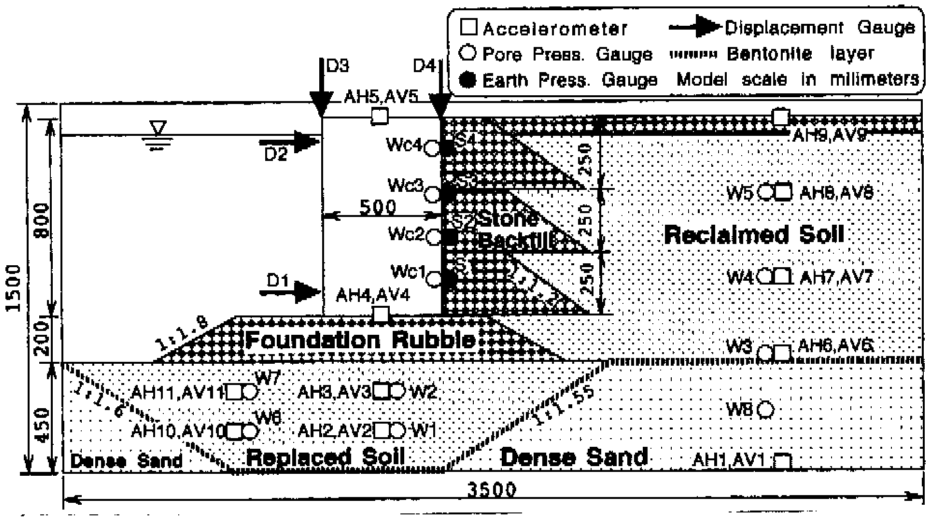


Figure 4 Cross section of a model quay wall for shaking table tests

Table 1 Scaling factors for shaking table tests for a gravity quay wall

Quantities	Scaling factors	Scaling factors for 1/17 model
Length	λ	17.0
Time	$\lambda^{0.75}$	8.4
Acceleration	1	1.0
Displacement	$\lambda^{1.5}$	70.1
Stress/pore water pressure	λ	17.0
Strain	$\lambda^{0.5}$	4.1

A total of nine tests have been performed with various conditions for the model tests. After each test, the model foundation soils and backfill soils were excavated for measuring the deformation, then completely removed from the container and new virgin soils transported from Port Island were pluviated into the water to form a new model foundation and backfill for the next case.

SIMULATION OF CASE HISTORY PERFORMANCE

The first series, Cases-2, 6 & 7, were conducted to repeatedly simulate the performance of the quay wall at Kobe Port during the earthquake. To facilitate the comparison to those measured in the field, the test results are presented in terms of the prototype scale. The initial densities and void ratios of the model foundation and backfill are shown in Table 2. The dry densities ranging from 1.6 to 1.9 g/cm^3 as shown in this table are very large, probably because the decomposed granite contains fair amount of large particles. The dry densities of the in-situ frozen samples of the decomposed granite range from 1.7 to 2.1 g/cm^3, basically consistent with those measured in the model foundation and backfill.

Residual displacements of the model caisson wall after shaking are shown in Fig. 5 together with those measured in the field as reproduced from Fig. 2. The model test results are basically consistent with those induced in the field during the earthquake. As shown in Fig. 5(b), the model caisson tilted into the foundation rubble and pushed the rubble mound out. This mode of deformation of the foundation rubble is also consistent with that investigated in the field by diving as shown in Fig. 6.

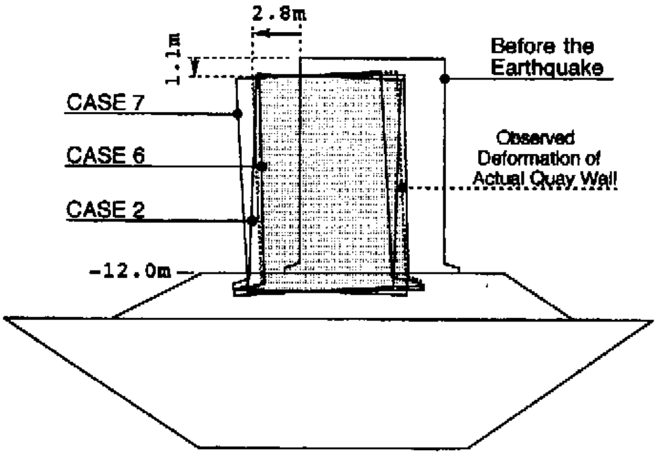
In order to obtain a comprehensive picture of the dynamic performance of the quay wall during the earthquake shaking, time histories of the response of the quay wall obtained by the model tests are shown in Fig. 7. Accelerations of the quay wall continued for about 20 seconds. In particular, the wave form at the ground surface is similar to that recorded at the ground surface in the Port Island shown in the upper right hand corner in the same figure. Displacements and excess pore water pressure were gradually induced with the shaking for about 10 seconds. These results indicated that displacements were induced not only by strong inertia force applied for a few seconds with one or two cycles of the strong shaking when the excess pore water pressure had not yet significantly increased in the soils but also in the later stage with a weaker motion and full excess pore water pressure increase in the soils. The order of magnitude of the rate of displacement of the model caisson wall (i.e. 0.2 to 0.3 m/s) was consistent with that of the prototype caisson evaluated based on the seismic response of a container crane (Scott, 1997).

In order to study the state of the foundation and the backfill soils, excess pore water pressures are plotted in Fig. 8 together with the initial effective overburden pressures. As shown in this figure, the maximum excess pore water pressures remained more or less at the level of about half of the initial effective overburden pressures both in the foundation and backfill soils. The in-situ observation of the quay walls suggested that there were generally a lack of sand boils in the backfill soils in the vicinity of the caisson walls whereas in the landfill soils further inland there was extensive evidence of liquefaction. The rather low excess pore water pressures in the backfill soils of the model tests might be exaggerated partly due to the effect of the boundary condition at the land side but they were basically consistent with the observations of lack of sand boils in the vicinity of the caisson wall. This may be caused by large shear deformation in the backfill associated with the movement of the wall.

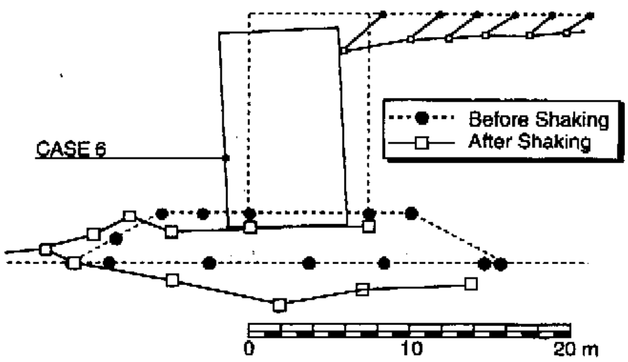
The low excess pore water pressures in the foundation soil may be caused by another mechanism; i.e. the existence of quasi-static deviator stress due to the dead weight of the caisson. Because stress state is brought to a state very close to the shear failure condition, slight increase in the excess pore water pressure will trigger large shear deformation in the foundation soil. The excess pore water pressures will no longer be able to increase beyond the state of the shear failure line.

Table 2 Initial dry densities and void ratios of model soils for a gravity quay wall (Cases-2, 6, 7)

Case No.	Soil deposits	Dry density (g/cm^3)	Void ratio
Case-2	Foundation soil	1.843	0.440
	Backfill soil	1.598	0.661
Case-6	Foundation soil	1.580	0.679
	Backfill soil	1.913	0.387
Case-7	Foundation soil	1.592	0.667
	Backfill soil	1.906	0.392



(a)



(b)

Figure 5 Residual displcement of model quay walls (scaled in terms of prototype)
(a) Displacements of caisson wall, (b) Deformation of foundation rubble and ground surface

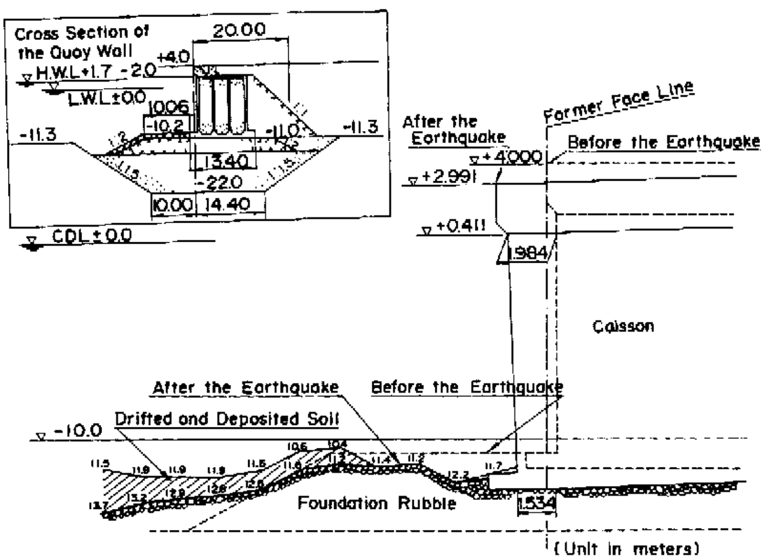


Figure 6 Deformation of rubble mound beneath the caisson (measured by diving)

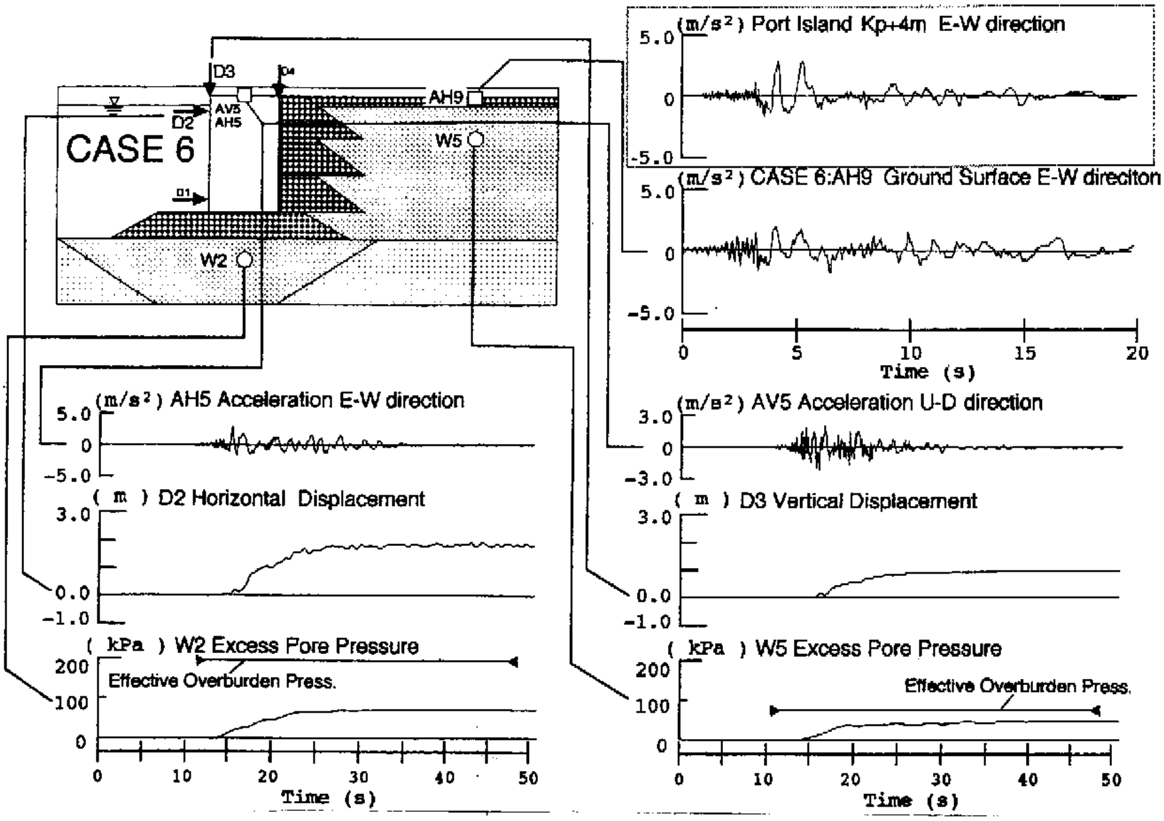


Figure 7 Time histories of model quay wall response (scaled in terms of prototype) and the recorded time history of acceleration at the ground surface at the vertical seismic array site in Port Island

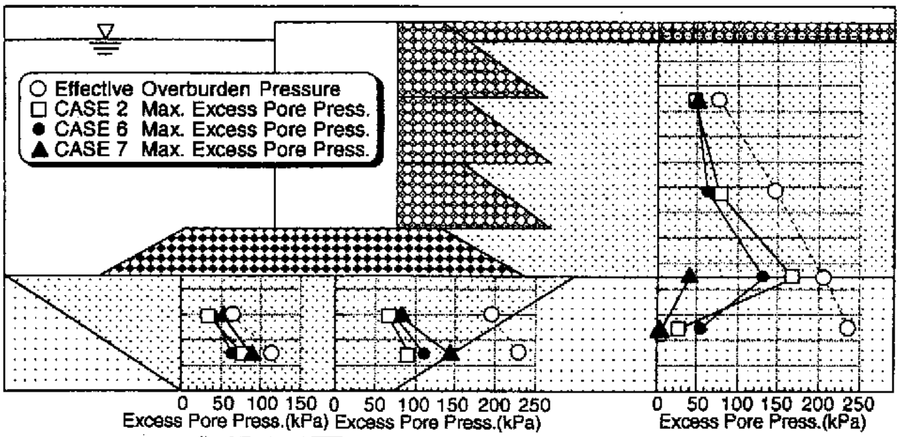


Figure 8 Distribution of maximum excess pore water pressures obtained from shake table tests (scaled in terms of prototype)

EFFECTS OF EXCESS PORE WATER PRESSURES

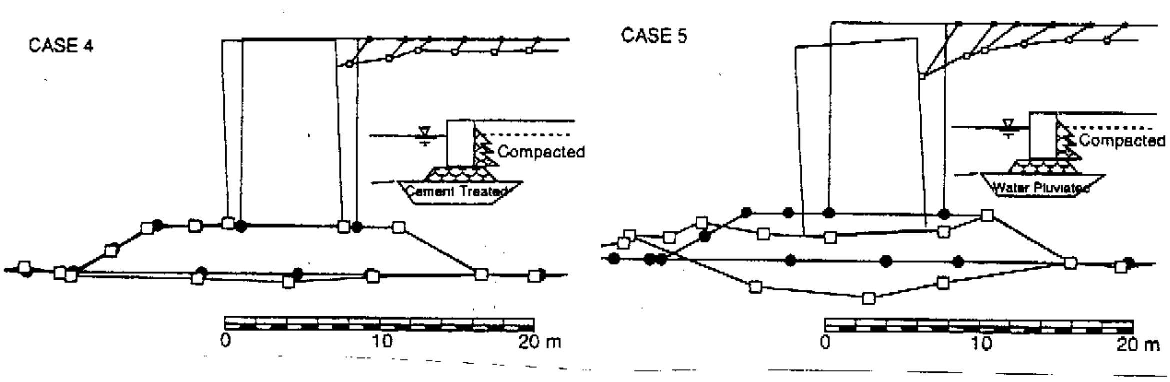
In order to quantify the effects of the excess pore water pressures in the foundation and backfill soils on the deformation of the caisson wall, another series of the model tests were conducted by varying the geotechnical conditions in the foundation and backfill soils as shown in Tables 3 and 4. The results of the model tests were summarized in the residual displacements shown in Fig. 9. The residual horizontal displacement of 1.4 m is induced without the effect of excess pore water pressures (Case-4). The increase in the excess pore water pressure in the backfill soil will increase the displacement in 50 % (Case-3). The increase in the excess pore water pressure in the foundation soil will increase the displacement in 100% (Case-5).

Table 3 Geotechnical conditions for Cases-3 through –5

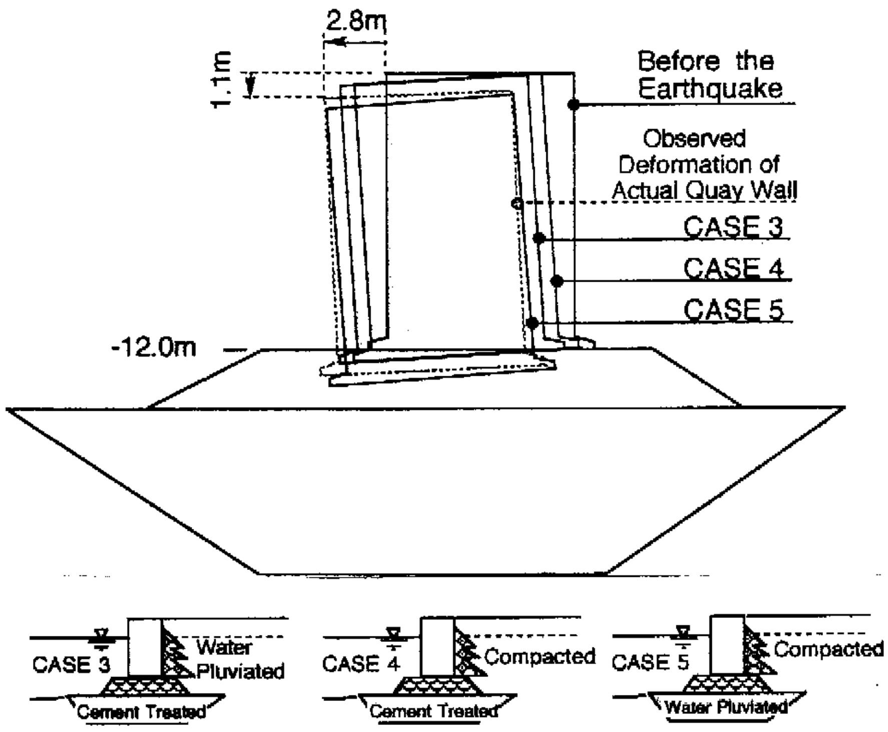
Cases	Foundation soil	Backfill soil
Case-3	cement treated	water pluviated
Case-4	cement treated	compacted
Case-5	water pluviated	compacted

Table 4 Initial dry densities and void ratios of model soils for a gravity quay wall (Cases-3 through 5)

Case No.	Soil deposits	Dry density (g/cm^3)	Void ratio
Case-3	Foundation soil	1.897	0.399
	Backfill soil	1.862	0.426
Case-4	Foundation soil	1.747	0.519
	Backfill soil	1.873	0.417
Case-5	Foundation soil	1.691	0.570
	Backfill soil	1.757	0.511



(a) Deformation of foundation rubble and ground surface (Cases-4 & 5)



(b) Displacements of caisson wall (Cases 3 through 5)

Figure 9 Residual displacement of model quay walls with various geotechnical conditions (scaled in terms of prototype)

Settlements of the wall is not significant when the foundation soil has no increase in the excess pore water pressure (Cases-3 and 4). When the excess pore water pressure is allowed to increase in the foundation soil, the caisson will be pushed into the rubble mound, which is then pushed into the foundation soil below, resulting in large settlement (Case-5).

To summarize, the excess pore water pressure increase in the foundation and backfill soils increased the displacement of the wall about twice as that induced without the excess pore water pressure increase. The ratio of the displacement increase is 2(foundation):1(backfill) for excess pore water pressure increase. Large settlement is also caused by the excess pore water pressure increase in the foundation soil.

CONCLUSIONS

Series of model tests were reviewed in order to discuss the seismic performance of gravity quay walls constructed on the loose foundation. Major conclusions obtained are as follows.

(1) Shake table tests of 1/17 scaled model successfully simulate the performance of prototype caisson walls during the Hyogoken-Nambu earthquake of 1995.
(2) Displacements of the caisson were not suddenly induced during the first few cycles of strong shaking but rather were gradually induced with increase of excess pore water pressure in the foundation and backfill soils.
(3) The mechanism of the deformation of the caisson wall constructed on a loose foundation is not the sliding of the caisson as often assumed in the conventional simplified analysis but an overall deformation of the foundation soils beneath the caisson.
(4) The excess pore water pressures in the foundation and backfill soils increases only about 50% of the initial effective vertical stress. This may be due to the effect of quasi-static deviator stress induced by gravity.
(5) The excess pore water pressure increase in the foundation and backfill soils increased the displacement of the wall about twice as that induced without the effect of the excess pore water pressures. The ratio of the displacement increase from the effects of the excess pore water pressure increase in the foundation and backfill soils is 2:1. Large settlements are also induced by the excess pore water pressure increase in the foundation soil.

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