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CASE STUDIES OF CAISSON TYPE QUAY WALL DAMAGE BY 1995 HYOGOKEN-NANBU EARTHQUAKE

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ABSTRACT

Over 250 caisson type quay walls were damaged in Kobe port during the 1995 Hyogoken-Nanbu Earthquake. The typical damage was observed, seaward horizontal displacements of the caisson walls were around 5m at maximum and 3m on average and around 1m settled, the caisson walls also suffered tilting. However, the caisson walls did not collapsed or overturned and deformed quite uniformly maintaining the face lines of the walls almost straight. To investigate the mechanism of damage, we carried out geotechnical investigations and field measurements including *in situ* freezing sampling, shaking table tests and effective stress analyses.

The results of shaking table tests and effective stress analyses suggested that the decrease in shear resistance due to excess pore water pressure increase in the foundation soil under the caissons and in the backfill soil increased the deformation of the caisson walls. The evaluation of accuracy of scaling law between 1/17 model scale shaking test and the prototype deformation was conducted. The scaling law chosen for the shaking tests indicated that the applicability of 1*g* gravitational field shaking test in high accuracy.

KEY WARDS

Earthquake damage; Liquefaction; Shaking table test; Effective stress analysis; Scaling law

INTRODUCTION

Kobe port is located about 17km south east of the epicenter of the 1995 Hyogoken-Nanbu Earthquake. Most of caisson walls displaced toward sea 3~5m, inclined toward sea and sank around 1m. About 3~4m subsidence were observed at the back site due to lateral deformation of quay wall as shown in Fig.1 The construction procedure is as following, 1) excavate alluvial clay layer and replace well graded soil, 2) put foundation rubble, 3) install caisson wall, 4) put stone backfill, 5) reclamation.

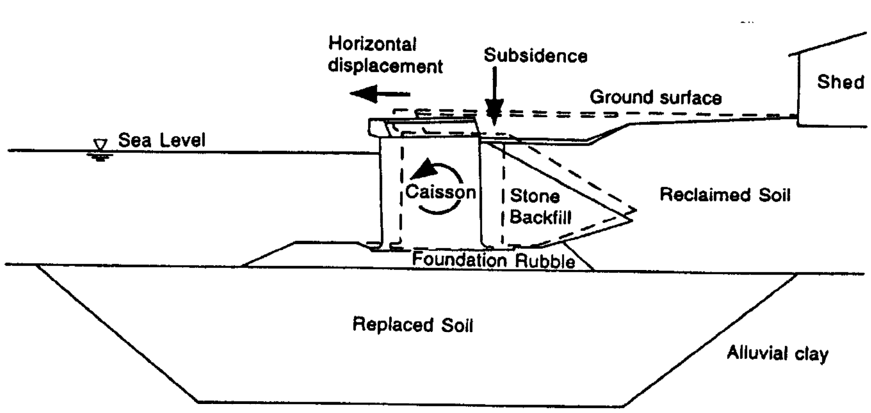


Fig.1 Typical damage of caisson type quay wall

To investigate the mechanism of the damage, a series of case studies was carried out by shaking table tests and effective stress analyses. Since a vertical seismic array records were obtained in Kobe Port Island, we focused on the typical damaged quay wall located nearby the seismic array as shown in Fig.2 Liquefaction took place at most part of Kobe Port Island which was reclaimed by well graded decomposed granite soil.

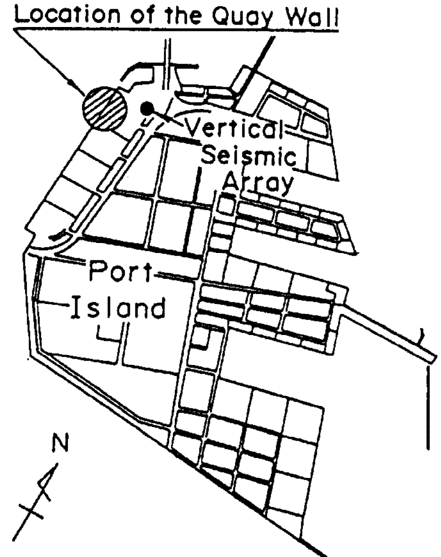


Fig.2 Kobe Port Island

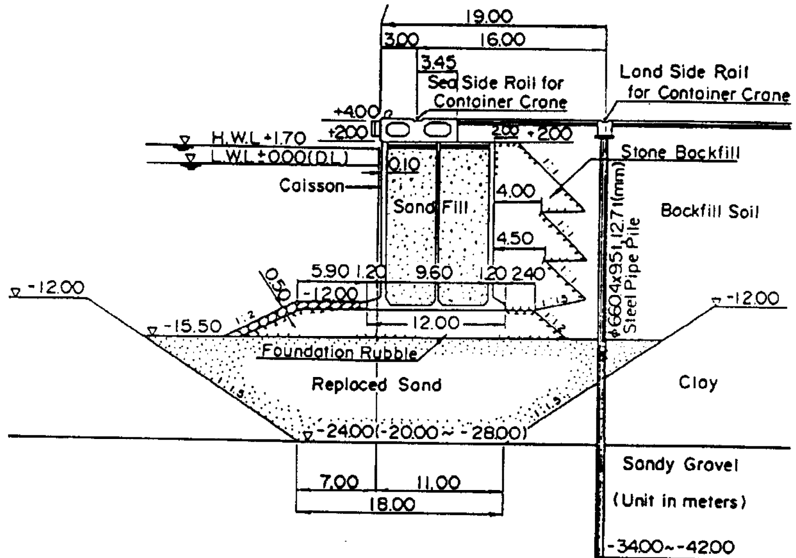


Fig.3 Cross section of a quay wall at Kobe Port Island

SHAKING TABLE TESTS

Shaking table

The specifications of the underwater shaking table used for the tests are listed in Table 1. The shape of shaking table is 5.65m diameter circle and which is installed at the bottom of 15m square, 2m depth water pool.

Table 1. Specification of the underwater shaking table

Table size	4m square w/ 5.65m diameter circle shape flange
Load	20tf rating, 60tf maximum
Vibration direction	3 axes (N-S, E-W, U-D) 6 degree of freedom
Water pool size	15m square 2m depth
Capacities	X; 200mm, 75cm/s, 2G
	Y; 300mm, 150cm/s, 1G
	Z(up-down); 100mm, 50cm/s, 1.5G
Frequency range	DC to 70Hz

Model construction procedure

In the shaking table tests, the caisson type quay wall shown in Fig.3 was modeled in a scale of 1/17 of the prototype. The quay wall model was constructed in a steel container 3.5m long by 1.5m wide and 1.5m deep on a shaking table. The soil used for the model was decomposed granite soil called "Masado". Masado was excavated from a site nearby the prototype quay wall and which experienced 1995 Hyogoken-Nanbu Earthquake. The grain size distribution curves are shown in Fig.4, the open square and open triangle symbol expressing, excavated Masado and prepared Masado sample for the tests using 30mm sieve respectively.

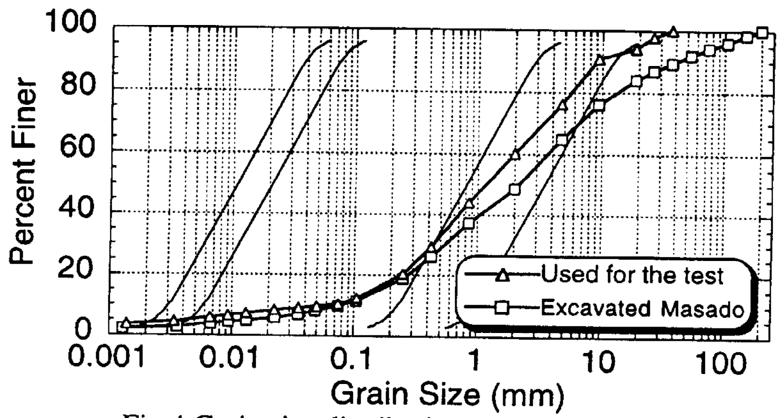


Fig.4 Grain size distribution curves of Masado

The cross section of the model quay wall is shown in Fig.5. After build up the steel container, installed several measurement devices. The alluvial clay layer at the quay wall site is idealized in the model test by a compacted sand layer. The surface of this layer was covered with a thin bentonite layer to simulates the impermeable behavior of the alluvial clay layer. After fill the pool with water, the replaced soil layer was constructed by means of dry Masado soil threw into the excavated concave region through water as shown in Fig.5. Put measurement targets on the surface of replaced soil layer and surface of rubble mound to investigate the relative deformations. Three model caissons were installed along a quay wall face line and the caisson in the middle was used for monitoring accelerations, displacements, earth pressures and pore water pressures. After construct each backfill stone, reclaimed soil layer was build up by water pluviated method.

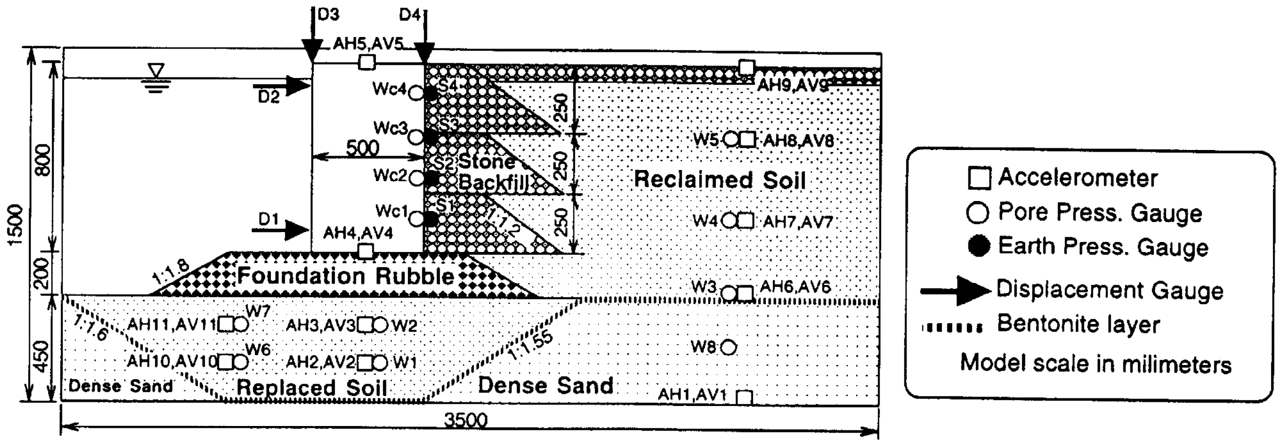


Fig.5 Cross section of a model quay wall for shaking table tests

The steel container was open to the water pool(left hand side of the figure), the boundary of land fill area(right hand side of the figure) was sealed with unwoven textile reinforced with wire mesh to relief the effect of rigid boundary condition. Both side of the container are made of rigid steel plates to maintain plain strain condition. Finally, measure the initial conditions of measurement devices and initial model ground surface.

Input motion

A set of input motions used for shaking table tests was recorded by a vertical seismic array in Kobe Port Island(Fig.2) at the ground surface(GL0m=Kp+4m), Kp-12m,Kp-28m and Kp-82m by the Development Bureau of the Kobe City. The Kp-28m records were chosen for the shaking table tests and effective stress analyses since the bottom level of the model was Kp-24m. The horizontal components of seismic motion were coincident with the direction of face line of the prototype quay wall. In case of shaking table tests, N-S,E-W and U-D components were applied to the shacking table.

For the effective stress analyses, E-W and U-D components were applied. The peak accelerations were 5.44m/s^2, 4.61m/s^2 and 2.00m/s^2 in N-S, E-W and U-D directions.

Scaling law

Dynamic problems such as shaking table test applying seismic motion require special consideration in order to define appropriate scaling law between prototype and model. The similitude in 1g gravitational field for soil-structure-fluid system(Iai, S. 1989) was adopted for the present study. The scaling law adopted for the tests was based on the basic equations which govern the behaviour of the saturated soil-structure-fluid systems under dynamic loading. The scaling relations between model and prototype include 1/17 scaling factor are shown in Table 2.

Table 2. Scaling factors for 1g shaking tests

	Items	Scaling factors in general expression (prototype / model)	Scaling factors for the tests (prototype / model)
x	length	λ	17.0
t	time	$\lambda^{0.75}$	8.4
ε	strain	$\lambda^{0.5}$	4.1
σ	stress	λ	17.0
p	water pressure	λ	17.0
u	displacement	$\lambda^{1.5}$	70.1
$\ddot{u}$	acceleration	1.0	1.0

Test conditions

To investigate the mechanism of damage, several model tests were performed as following;

Case 2; The model was constructed the same method as prototype. i.e. Replaced and reclaimed soil were filled by water pluviated method. (Sugano *et al.*, 1995)

Case 3; Replaced soil was cement treated / Reclaimed soil was filled by water pluviated method

Case 4 ; Replaced soil was cement treated / Reclaimed soil was compacted

Case 5; Replaced soil was filled by water pluviated method / Reclaimed soil was compacted

Case 6 and Case 7; To evaluate the accuracy of experiment, carried out Case-2 condition tests

In the case of cement treated, dry weight ratio of 2% cement was added to the replaced soil.

Results of the shaking table tests

Let us look at the accuracy of the shaking table tests, three cases Case 2, Case 6 and Case 7 were carried out to simulate the performance of the quay wall during the earthquake. To compare the test results in the model and the prototype damage, the test results will be interpreted into prototype scale calculated by the scaling law mentioned above. Residual deformations of the model caisson wall after shaking were shown in Fig.6 together with those measured in the field. The model test results were consistent with those obtained by the field measurements. The acceleration time histories of Case 6 at bottom and ground surface were shown in Fig.7. The wave form of E-W component of ground surface shows that the envelope of acceleration gradually declines and filtered out hight frequency motions due to the softening of reclaimed soil.

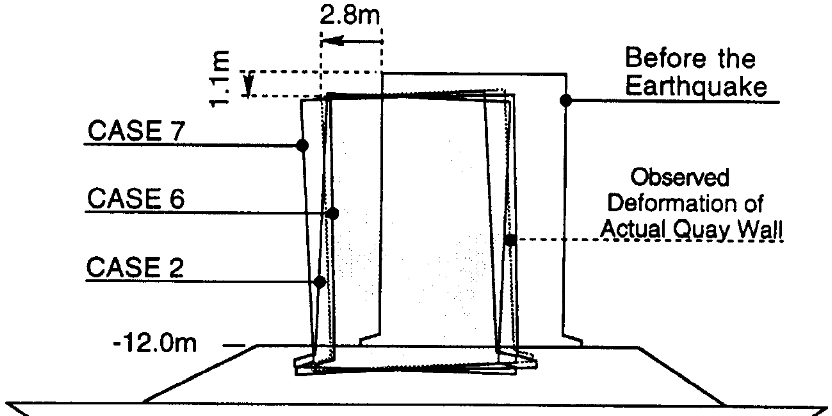


Fig. 6 Residual deformations of the model quay wall (Case 2, Case 6 and Case 7)

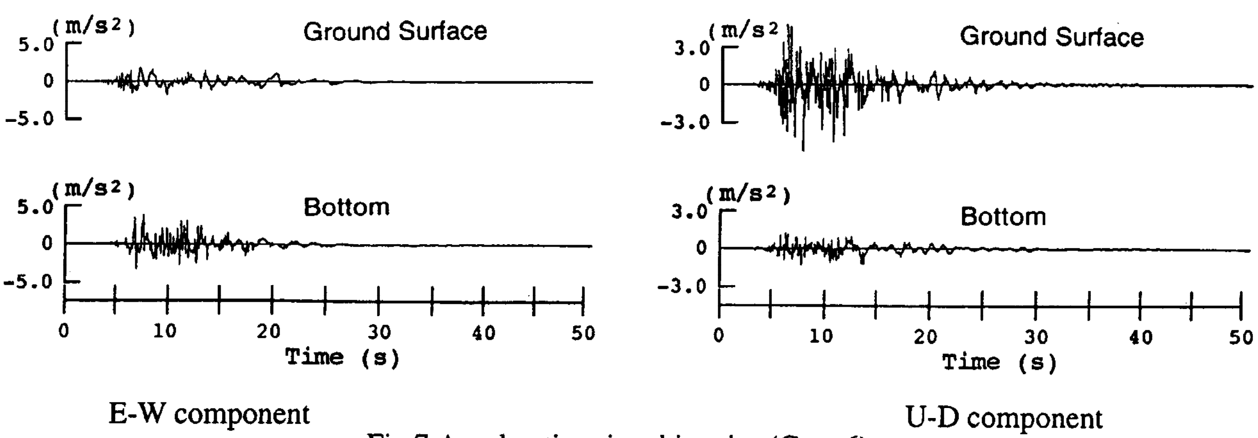


Fig.7 Acceleration time histories (Case 6)

Time history of excess pore water pressure in the reclaimed soil region indicate that the excess pore water pressure was generated with the shaking for about 10 seconds as shown in Fig.8. In this figure the horizontal line marked above the peak region indicates the effective overburden pressure at initial state. The same manner, displacements were gradually induced with the shaking for about 10 seconds as shown in Fig.8. These results indicate that displacements were induced not only by strong inertia force applied at a few seconds with one or two cycles of the storing shaking when the excess pore water pressure had not significantly generated in the reclaimed soil region but also in the later stage by weaker motion with full excess pore water pressure increase in the reclaimed soil region.

Maximum excess pore water pressures were plotted in Fig.9 together with the effective overburden pressures. Maximum excess pore water pressures generated more or less in about the level of half of the initial overburden pressures the reclaimed region and the underneath of caisson replaced soil region. The maximum excess pore water generation curves of the underneath of caisson replaced soil region reached about 50% of excess pore water ratio.

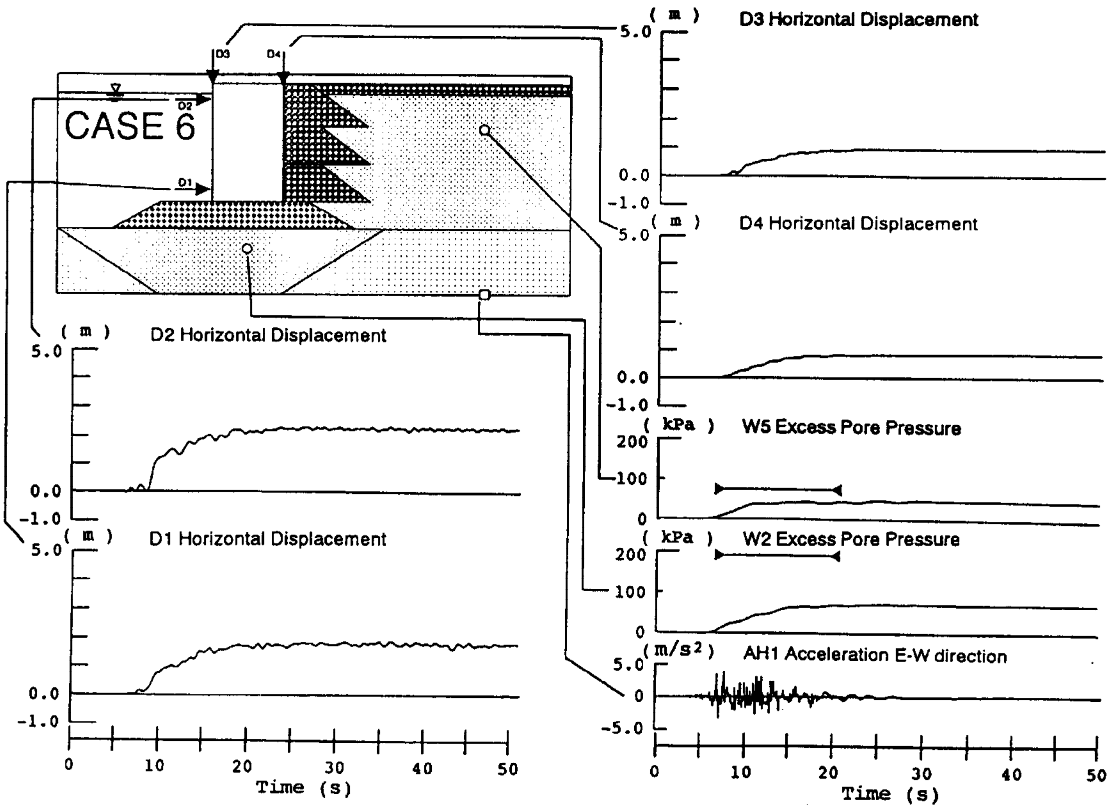


Fig. 8 Time histories of displacement and excess pore water pressure

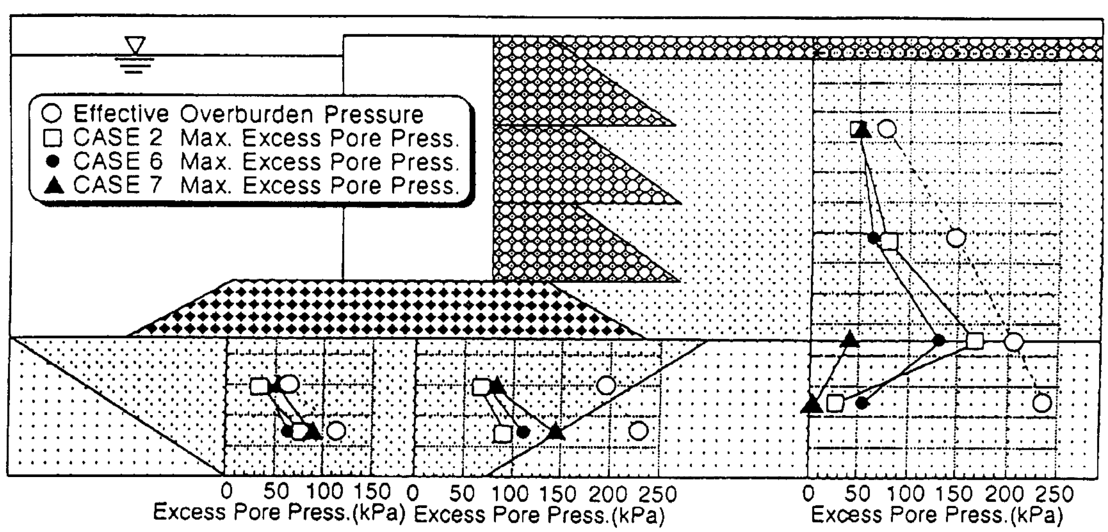
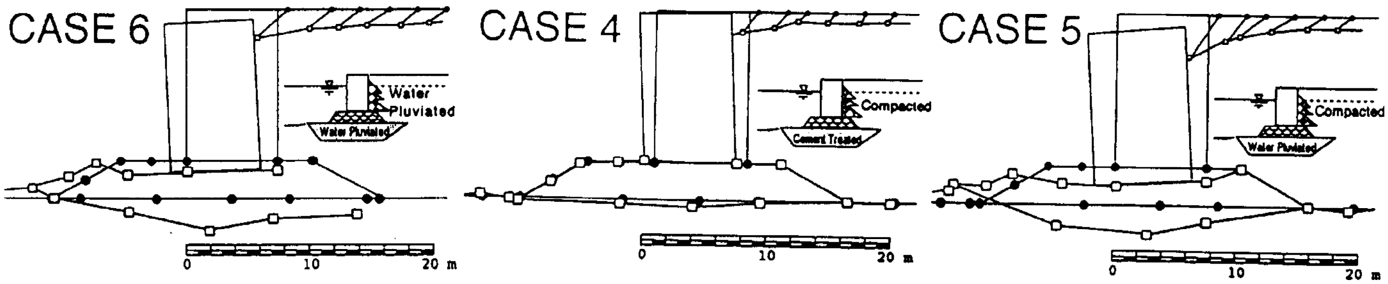


Fig.9 Maximum excess pore water pressure

As shown in Fig.10(a), the model caisson tilted into the foundation rubble and pushed out the rubble mound. This mode of deformation of foundation rubble is also consistent with that investigated in the prototype.

In view of soil conditions in the replaced region and the reclaimed region, let us then consider the performance of the caisson type quay walls. Residual deformations of caisson model are summarized in Fig.11. It can be Fig.11 and Fig.10 tell us that the condition of replaced soil region play an important role in earthquake damage especially deformation of caisson wall .



(a) Case 6 (b) Case 4 (c) Case 5

Fig. 10 Residual deformation of foundation rubble and ground surface

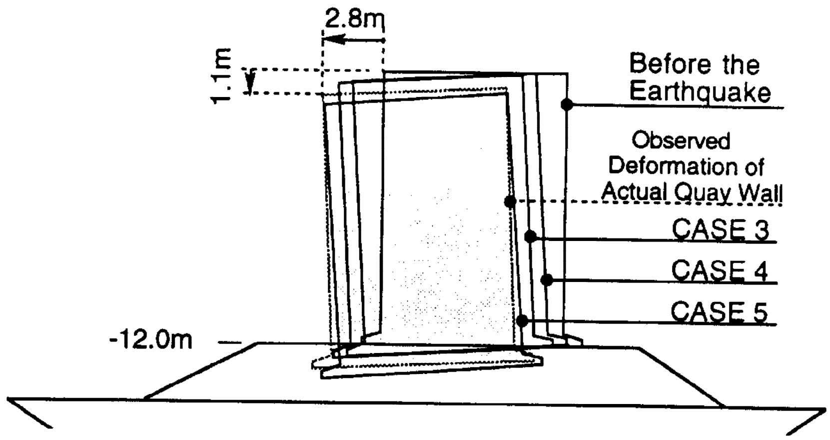


Fig. 11 Summarized residual deformations

EFFECTIVE STRESS ANALYSIS

The aim of the effective analysis was similar to that of the shaking table tests; to give some insight as to exactly what happened to the caisson type quay walls during the earthquake. Generally, these two approaches, i.e. shaking table test and effective stress analysis, were simulated the performance of prototype. However, in this study, the performance of the model quay wall during shaking table test was analyzed because of the scaling law adopted to the shaking table tests gave good agreement with prototype damages.

Constitutive law of soils and model parameters

Multiple mechanism model defined in strain space(Iai *et al.*,1992) was adopted for the analyses. The constitutive model represents the effect of rotation or principal stress axis directions such as the stress conditions of ground during earthquake. The constitutive model parameters were determined by referring to the elastic wave velocity of model and the cyclic triaxial test results of the *in situ* frozen samples and previous study on dynamic properties of foundation rubble test data. The parameters used for the analysis are shown in Table 3.

Table 3. Constitutive model parameters

Soil type	ρ_{sat} (t/m^3)	G_{ma} (kPa)	σ_{ma}' (kPa)	ϕ_f (deg)	ϕ_p (deg)	Parameters for dilatancy
Alluvial clay	1.7	74970.	6.80	30.		
Replaced soil	1.8	58320.	6.95	37.	28.	w_1=5.5, p_1=0.6, p_2=0.6 c_1=2.3, S_1=0.005
Reclaimed soil	1.8	79380.	3.35	36.	28.	w_1=6.0, p_1=0.5, p_2=0.8 c_1=2.43, S_1=0.005
Foundation rubble Backfill stone	2.0	9900.	5.23	40.		

Friction angle at bottom of aluminum caisson model δ=31 (deg)
Friction angle at back of aluminum caisson model δ=31 (deg)

ρ_{sat}: saturated density, G_{ma} : inital shear modulus at $\sigma_0'=\sigma_{ma}'$,
σ_{ma}' : reference confining pressure, ϕ_f : internal friction angle , ϕ_p : phase transformation angle

Earthquake response analysis
Before the earthquake analysis, a static consolidation analysis was conducted by gravity in order to simulate the initial stress condition of model. The E-W and U-D component of recorded seismic motions were used as the input motions. In case of earthquake analysis, the undrained conditions were assumed in order to simplify the analysis. The canalized cases are the same cases as shaking table tests. To simplify the analysis, the cement treated soil and compacted soil are set up ideally non liquefiable soil.

Results of the analyses
There is space here only for one case of analysis, take the case which is equivalent to the Case 2, Case 6 and Case 7 in the shaking table tests.

The earthquake response analysis showed that the displacements of the caisson wall were gradually induced for about 10 seconds as shown in Fig.12. The top left graph shows the time history of horizontal(E-W) displacement, negative direction expresses seaward direction. The top right graph shows U-D displacement, negative direction means settlement. The residual displacements were 2.1m in horizontal, 0.6m in vertical.The order of magnitude of these results were consistent with the observed deformation of the caisson wall in field and also the shaking test results mentioned above.

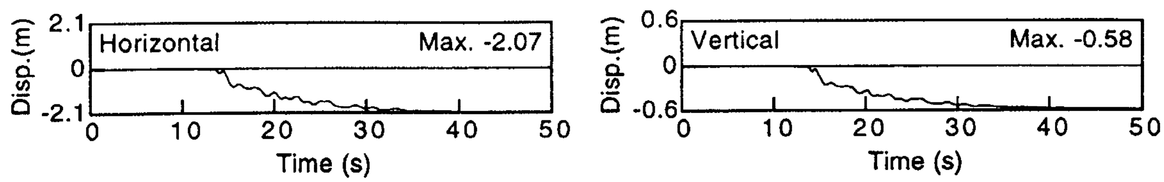


Fig. 12 Computed time histories of displacements of the top left point of caisson

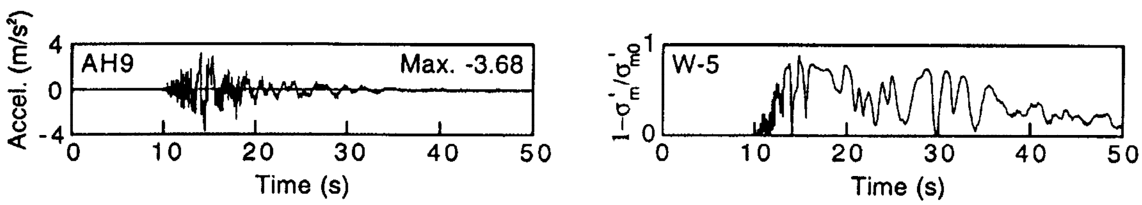


Fig. 13 Computed time histories of horizontal(E-W) acceleration and excess pore water pressure ratio

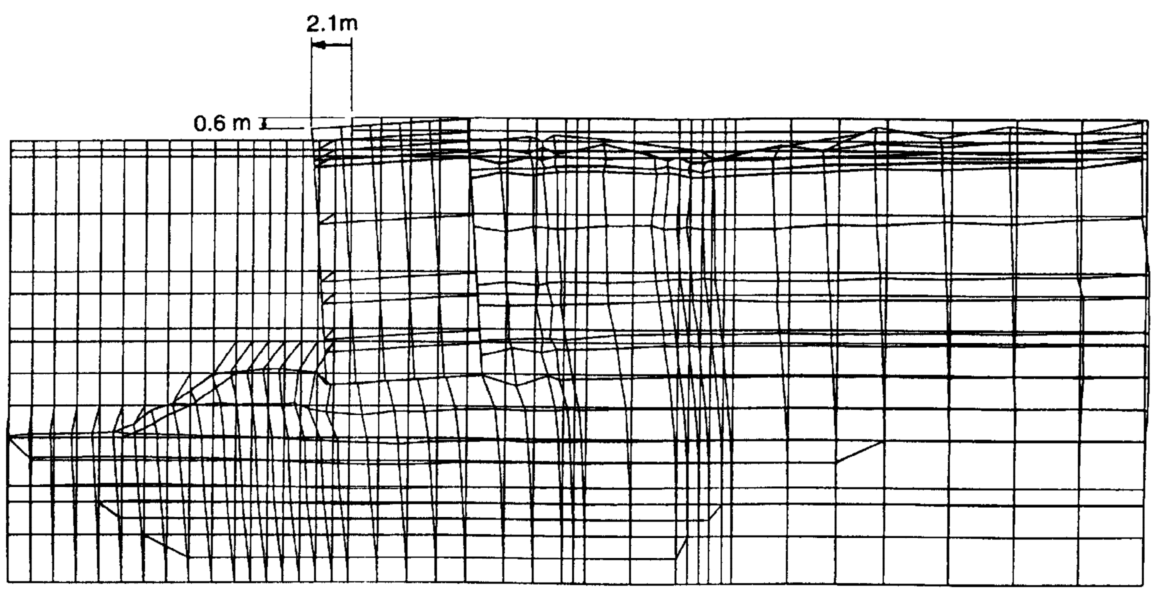


Fig.14 Computed residual displacements of the quay wall

The time histories of horizontal acceleration at the ground surface AH5 and excess pore water pressure ratio in the reclaimed soil point W5 were shown in Fig.13. These results show good agreement with the shaking test results. The replaced soil and the part of the backfill reclaimed soil in the vicinity of the caisson wall did not achieve the excess pore water pressure ratios beyond 0.8 but their excess pore water pressures have certainly increased during the earthquake.

The mode of deformation of the caisson quay wall as shown in Fig. 14 indicate that significant deformations were induced in the replaced soil region. Due to the significant deformations in the replaced soil region, the foundation rubble was also deformed. Then the caisson tilted into the foundation rubble and pushed out the rubble mound. Similarly to the results of the shaking table tests.

CONCLUDING REMARKS

The important finding from the shaking table tests and the effective stress analyses on the performance of the caisson type quay walls can be summarized as follows.

The main cause of the deformation of caisson quay wall was not the sliding between the foundation rubble mound and the bottom of caisson but a significant deformation of the foundation rubble mound and the replaced soil region. In the conventional stability analysis of caisson type quay wall, we consider the sliding between the foundation rubble mound and the bottom of caisson. However, in case of the damages due to 1995 Hyogoken-Nanbu Earthquake, the mode of deformation were quite different from the sliding mechanism generally assumed for the conventional stability analysis.
Excess pore water pressure increase in the replaced soil region underneath the caisson and in the reclaimed soil region did not reach 80% of the initial confining pressures. The stress conditions in the replaced soil region underneath the caisson are not uniform due to the initial shear loading by quay wall which sustain the earth pressure of back fill. Namely, the deformations of quay wall were induced not the effective stress reaches zero i.e. completely liquefied but the decrease of shear resistance of replaced soil.

Since the comparison between the results of model shaking table tests and the deformations of prototype, the scaling law adopted for the shaking table tests can be use in high accuracy.

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