PREDICTION METHOD ON DEFORMATION BEHAVIOR OF CAISSON-TYPE SEAWALLS COVERED WITH ARMORED EMBANKMENT ON MAN-MADE ISLANDS DURING EARTHQUAKES

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ABSTRACT

In this paper, centrifuge shaking table tests were conducted in order to understand the performance of seawalls during a seismic event. The model tests showed that the displacement of the caisson was much affected by the seaward shear deformation of the sand seabed beneath it during shaking. It was also confirmed that an armored embankment played an important role in the displacement of the caisson during shaking. Based on these test results, a two-dimensional DEM-FEM coupled analysis method was newly developed to numerically predict the deformation of seawalls covered with armored embankments during earthquakes. The movements of the armor units were calculated by DE analysis and deformations of the caisson, rubble mound, sand seabed and backfill were calculated by FE analysis considering the non-linearity of the soil materials based on the effective stress. Dynamic interaction was taken into account by delivering the nodal displacements of the finite elements or the nodal forces converted from the contact forces through the imaginary distinct elements defined at the boundaries between the DE and FE domain. The applicability of this method to the prediction of the deformation of seawalls was verified through numerical simulations of the centrifuge model test.

Key words: centrifuge, coupling analysis, distinct element method, finite element method, seawall, seismic deformation (IGC: D7/E8/E14)

INTRODUCTION

In the man-made island siting technology of nuclear power plants in an offshore space, assessing the stability of the seawalls against large ocean waves and earthquakes is indispensable. A main function demanded of the seawalls is to protect the island itself and the important structures behind them from serious submergence by excessive overtopping water during the occurrence of large ocean waves. A design flow of the seawalls derived from the performance verification design concept is indicated in Fig. 1 (Japan Society of Civil Engineer, 1996). In this flow, it is important to evaluate whether the seawalls deformed by a large earthquake retain the demanded function against large ocean waves after the earthquake or not. And, in this case, the prediction of the deformation of the seawalls during an earthquake, especially the prediction of the change of the elevation at the top of the caisson which most relates to the change of the overtopping water from the sea, with an accuracy as high as possible will become one of the important examination items on the performance verifications.

As for the studies on the seismic behavior of the cais-

son-type quay walls, since the 1995 Hyogoken-nambu earthquake where a lot of port facilities were seriously damaged, various model tests and numerical investigations have been conducted in order to make clear the mechanism of the deformation. Most of these investigations pointed out the importance of not only the inertia force but also the accumulation of the shear deformation in the soft soil foundation beneath the caisson against the displacement of the caisson (Inagaki et al., 1996; Ghalandarzadeh et al., 1998; Iai et al., 1998; Kanatani and Yoshida, 1998; Kanatani et al., 2000). These demonstrated that at least the deformations of the soil foundation should be appropriately estimated in order to predict the deformation of the quay walls during a large earthquake.

The biggest difference in the structural form of the seawalls in the present study compared with the above mentioned quay walls is the existence of an armored embankment in front of the caisson, as shown in Fig. 2. This cross section is designed based on the seismic coefficient method of horizontal and vertical seismic intensity $K_H=0.2$ and $K_V=0.0$, respectively. And the total mass of the armored embankment is determined from the wave

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ⁱⁱⁱ⁾ Manuscript was received for review on May 18, 2000.

Written discussions on this paper should be submitted before July 1, 2002 to the Japanese Geotechnical Society, Sugayama Bldg. 4F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101-0063, Japan. Upon request the closing date may be extended one month.

height with a return period of 100 years. The area occupied by the armored embankment among the seawalls is huge and it is expected that the deformation of the seawalls is not a little affected by this armored embankment. But there have been few experimental studies on the seismic behavior of this type of seawall. When attention is paid to the numerical studies, many of them seem to be on the seismic behavior of quay walls without armored embankments. For example, Toki et al. (1982) and Uwabe et al. (1987) investigated seismic response with the finite element method (FEM) with joint elements and Iai et al. (1998) performed a numerical investigation on caisson-type seawalls and breakwaters by FEM, not only

with joint elements but also considering the non-linearity of the soil materials based on the effective stress. On the other hand, Kurita et al. (1995) developed a numerical technique to predict the large deformation of the quay walls without water and armored embankment by the distinct element method (DEM) and Nakase et al. (1996) developed the three-dimensional analysis method of the assembly of the armor units by DEM.

In this study, from the above points of view, centrifuge shaking table tests were conducted in order to understand the fundamental behavior of seawalls during shaking and a new, highly accurate deformation analysis method of seawalls covered with armored embankments during earthquakes was developed and verified through the numerical simulation of the centrifuge model test.

CENTRIFUGE SHAKING TABLE TESTS

Test Program

Two series of centrifuge shaking table tests of seawalls were conducted. The features of each series are presented in Table.1. Three sets of tests in the SERIES-1 employed a centrifuge facility with a radius of 9.1 m at the University of California at Davis (Kutter et al., 1994) were planned in order to make clear the fundamental performance of a seawall during excitation and mainly investigate the effect of the seabed conditions on the displacement of a caisson. On the other hand, two sets of tests in the SERIES-2 employed a centrifuge facility with a radius of 6.5 m at the Takenaka Research and Development Institute (Suzuki et al., 1994) were performed in order to investigate the effect of the existence of an armored embankment on the displacement of a caisson during excitation.

Model preparation of SERIES-1

A typical cross section of the seawall model set up in an aluminum rigid soil container ($L(\text{length}) \times W(\text{width}) \times H(\text{height})$: $1760 \times 905 \times 500$ [mm]) in the SERIES-1 is illustrated in Fig. 3(a). The scale of this model was 1/120 of the prototype (see Fig. 2) and this corresponded to the 1/4 scale of the prototype at the centrifugal acceleration of 30G. Nevada sand whose mean grain size was $D_{50} = 0.15$ mm was used in the sand portions such as the

Fig. 1. Design flow of seawalls

Fig. 2. Fundamental cross section of actual scale seawalls determined by seismic coefficient method ($K_H = 0.2$, $K_V = 0.0$)

Table 1. Test cases and conditions of centrifuge shaking table tests of seawall

SERIES	CASE	Sand seabed			Backfill	Armored embankment
		Relative density* (%)	Thickness (m)	Improvement	Relative density* (%)	
1	CASE 1	—	0	—	88	○
	CASE 2	62	2.5	×	96	○
	CASE 3	64 (96)	2.5	○	101	○
2	CASE 4	65 (90)	2.5	○	65 (90)	×
	CASE 5	65 (90)	2.5	○	65 (90)	○

* ()Relative density of improved area

Fig. 3. Schematic figure and arrangement of representative measuring devices in two-series of centrifuge shaking table tests of seawalls: (a) SERIES-1, (b) SERIES-2

sand seabed and backfill, and these were prepared by the air pluviation method intending to achieve a relative density of about $D_r=60\%$ for the medium dense area in the sand seabed and of about $D_r=95\%$ for the backfill and the improved area. Crushed stone, whose mean grain size was $D_{50}=6.6$ mm and maximum grain size was $D_{\max}=9.5$ mm, was used as a rubble material such as the rubble mound and rubble backing. The caisson and superstructure were fabricated of concrete and the armor

unit model was made of mortar. In an effort to more accurately adjust the time scaling of the pore fluid pressure dissipation within the sand and the rubble material, a mixture of water and hydroxypropyl methylcellulose (HPMC) having a viscosity ten times that of water was used as the pore fluid (Kawai et al., 1998). The arrangement of the representative measuring devices related to the following numerical investigation is also shown in Fig. 3(a).

Table 2. Centrifugal accelerations and input motions

SERIES	Scale factor*	Centrifugal acceleration (G)	Input motion**			Nominal amp. of input acc. (G)
			Wave type	Wave no.	Frequency (Hz)	
1	1/120	30	sine	12	90	3, 10, 15
2	1/200	50	sine	10	160	10, 20, 30

* Scale factor for the full scale prototype seawall in Fig. 2.

** Horizontal direction

Fig. 4. Examples of time history of input acceleration: (a) SERIES-1, (b) SERIES-2

Model preparation of SERIES-2

A typical cross section of a seawall model with an armored embankment set up in an aluminum rigid soil container, ($L \times W \times H$: 1100 × 400 × 300 [mm]) in SERIES-2 is illustrated in Fig. 3(b). The scale of this model was 1/200 of the prototype (see Fig. 2), corresponding to the 1/4 scale of the prototype at the centrifugal acceleration of 50G. Toyoura sand whose mean grain size was $D_{50}=0.16$ mm was used in the sand portions and the relative density of each portion was adjusted to about $D_r=65\%$ for the medium dense area and to about $D_r=90\%$ for the improved area. Owigawa sand, whose mean grain size was $D_{50}=1.69$ mm and maximum grain size was $D_{max}=4.8$ mm, was used as the rubble material. The caisson and superstructure were fabricated of concrete and the armor units model was made of mortar. Dimethyl siloxane having a viscosity fifty times that of water was used as the pore fluid.

Shaking tests

After spinning up to an expected centrifugal acceleration and applying a series of small base shaking events in order to assess the small strain dynamic response of the model, a series of larger base shaking events was com-

menced to record the response of the model under strong motion. The profile of the input acceleration was a sinusoidal wave for both series and its frequency, wave number and nominal amplitude of the input acceleration in each series are summarized in Table 2. Nominal amplitude of the input acceleration was a target value for the shaking, but the acceleration amplitude actually measured was somewhat different. Representative time histories of the input acceleration for both series are shown in Fig. 4(a) and (b). The amplitude of the input acceleration of main shock part was almost constant for all shaking events.

The following test results on the displacement and the acceleration were presented in the model scale.

Test Results

The residual displacements (DH1 and DV1) at the top of the caisson after the shaking event of the nominal amplitude of the input acceleration of 10G are plotted in Fig. 5. Residual displacements were incremental ones induced by this shaking event. They were little induced in the case of the rigid foundation without a sand seabed (CASE1). Comparing CASE2 with CASE1, it was found that the existence of the sand seabed led to the relatively large displacements of the caisson and suggested that the displacements of the caisson were probably caused by the deformation of the foundation ground. Furthermore, it was also shown that the densification of the sand seabed led to significant reduction of the displacements of the caisson comparing CASE3 with CASE2, even though the densified area was limited beneath the rubble mound.

A typical example of the overall deformation modes of a seawall standing on the sand seabed during strong shaking is schematically illustrated in Fig. 6. Armor units moved to the direction of the sea along the slope of the embankment by excitation. The caisson was laterally moved towards the sea with seaward tilting and settlement, even if a large amount of the armor units existed in front of it. The shear deformation to the seaward direction was induced in the sand seabed and it was clearly observed that the displacements of the caisson were not a little affected by such deformation in the sand seabed. These results on the deformation of the seawall indicated that the prediction of the shear deformation in the sand seabed was very important for predicting the residual displacements of the caisson during an earthquake.

The distributions of the excess pore fluid pressure ratio

at the end of the shaking obtained by the test of CASE3 are shown in Fig. 7. Excess pore fluid pressure ratio was defined by the value into which the excess pore fluid pressure was divided by the initial effective vertical stress at the location of the transducer obtained from initial stress

Fig. 5. Effect of seabed condition on residual displacements of caisson: (a) Horizontal displacement, (b) Vertical displacement

analysis by an elastic FEM. When the input acceleration was small, such as the nominal amplitude of the input acceleration of 3G, a liquefaction condition was never observed in the sand stratum except for the upper part of the backfill and sand seabed without the armored embankment on it. On the other hand, accordingly, as the input acceleration was increased to the nominal amplitude of the input acceleration of 15G, the liquefaction areas were enlarged in the sand stratum. But the excess pore fluid pressure ratios just beneath the caisson never achieved the 1.0 level, whereas those in other areas did almost reach it. These results are consistent with those from the model tests of the quay walls without the armored embankment (Inagaki et al., 1996; Ghalandarzadeh et al., 1998).

In Fig. 8, comparisons of the residual displacements at the top of the caisson obtained from the tests of CASE4 and CASE5 in SERIES-2 are plotted against the measured amplitude of the input accelerations. Both of the horizontal and vertical displacements of the caisson with the armored embankment were still smaller than those of the caisson without the armored embankment. This proved that the armored embankment provided considerable support to the caisson and suppressed the displacements of the caisson to some extent. And it demonstrated the fact that the evaluation of the effect of the armored embankment played an important role in predicting the displacements of the caisson during earthquake.

Based on the above series of centrifuge model tests, the main features demanded of the analysis method on the prediction of the deformation of the seawalls with the armored embankment standing on the sand seabed during earthquakes with a high accuracy were arranged as follows:

- It is able to evaluate not only the sliding of the caisson but also the plastic deformations of the sand seabed, rubble mound and backfill.
- It is able to consider the effect of water such as the liquefaction phenomenon of the sand stratum.
- It is able to appropriately consider the effect of the armored embankment.

Fig. 6. Typical example of deformation of seawall after shaking

Fig. 7. Distributions of excess pore fluid pressure ratio at end of shaking

DEM-FEM COUPLED ANALYSIS METHOD OF SEAWALLS

Framework of Analysis Method

A two-dimensional dynamic deformation analysis method of caisson-type seawalls covered with an armored embankment (SEAWALL-2D) was composed by two main analysis codes as shown in Fig. 9. Tochigi (1995) had developed a two-dimensional dynamic response analysis code of the assembly of the armor units by DEM. Furthermore, Tochigi (1995) and Tochigi et al. (1998) developed it into a DEM-FEM coupled analysis method (TSLIDE-2D) in order to carry out a dynamic response analysis of the caisson-type seawalls covered with the armored embankment. But in this method, material properties in the finite element domain (FE domain) such as a sand seabed, rubble mound and back-fill were limited only to linear elastic ones. Then the plastic deformations of the soil materials could not be estimated by this method, even though the shear deformation of the sand seabed beneath the caisson played an important role in the displacements of the caisson, as described in the centrifuge model tests. On the other hand, Nishi and Kanatani (1990) proposed the constitutive relationships of soils under cyclic loading based on the elasto-plasticity theory and Kanatani et al. (1994) developed a two-dimensional nonlinear response analysis method of the ground based on the effective stress (NAFSS-2D) by FEM, introducing these constitutive relationships and Biot's porous elastic material theory (u - w scheme). SEAWALL-2D is a new analysis method, coupling these two analysis codes and utilizing each characteristic in order to more realistically predict the deformation of the seawalls considering the effect of the armored embankment and the accumulation of the shear deformation in the sand seabed beneath the caisson during earthquakes.

Analysis Flow and Dynamic Interaction

An analysis flow of SEAWALL-2D is shown in Fig. 10. In this method, the distinct element analysis (DE analysis) and the finite element analysis (FE analysis) are carried out independently. But each domain actually experiences a dynamic interaction with the other at the boundaries, rather than independently moving. Such dy-

Fig. 8. Effect of armored embankment on residual displacements of caisson: (a) Horizontal displacement, (b) Vertical displacement

namic interaction effects are taken into account simply by the following procedure.

Schematic figures of the dynamic interaction at the boundaries between the DE and FE analysis domain are shown in Fig. 11 and Fig. 12. In the DE analysis, imagi-

Fig. 9. Composition of DEM-FEM coupled analysis method SEAWALL-2D

Fig. 10. Flow chart of SEAWALL-2D

Fig. 11. Schematic figure of evaluation of contact forces at boundary surfaces in DE analysis after FE analysis

nary distinct elements are set up at the boundaries between the DE and FE domains such as the surfaces of the caisson, rubble mound and sand seabed with which the armor units contact. These imaginary distinct elements have the same size as the corresponding finite elements in the FE domain and the relative movements are fixed at the apexes connected with each other. After the initial calculation of the FE analysis, nodal displacements of the finite elements corresponding to the imaginary distinct elements are determined. This information on the nodal displacements is delivered to the next step of the DE analysis in order to determine the new coordinates of the apexes of the imaginary distinct elements as indicated in Fig. 10. Judgment of contact or non-contact between each element and the calculation of the contact forces including both the armor unit elements and the imaginary distinct elements which have the new apex co-ordinates are carried out. In this process, contact forces between the armor unit elements and imaginary distinct elements at the boundaries are estimated. After this process, the equation of motion, in which the contact forces, gravitational force and inertia force are treated as the external forces acting on each element, are solved to obtain the responses of all the armor unit elements. On the other hand, contact forces acting on the imaginary distinct elements are converted into the nodal forces by the simple proportional calculations as shown in Fig. 12, and these nodal forces are delivered to the next step of the FE analysis as reaction forces from the armor units at the boundary surfaces. In the next step of the FE analysis, the equation of motion is solved by treating these reaction forces as external forces and responses in the FE domain are obtained.

By repeating these operations in a short time interval, a coupled analysis considering the dynamic interaction be-

tween the DE and FE domains is achieved. The same approach is adopted for the reaction forces from the armor units in the initial stress analysis of the FE domain, as indicated in Fig. 10.

NUMERICAL SIMULATION OF CENTRIFUGE MODEL TEST

Analysis Model of Seawall

Numerical simulations of the representative centrifuge model test were performed in order to verify the applicability of SEAWALL-2D. The analysis model of the seawall is shown in Fig. 13. This model corresponds to the centrifuge shaking table test of CASE3. The relative densities of the sand seabed and backfill were 64% and 101%, respectively, and a part of the sand seabed beneath the armored embankment and the rubble mound was improved by densification into a relative density of 96%.

The configuration and dimensions of the armor unit employed in the centrifuge model tests and in the distinct element analysis are shown in Fig. 14(a) and (b), respectively. The armor unit which had a geometrically three-dimensional configuration was converted into a plane projection configuration which was as characteristic of the armor unit as possible, because the analysis were performed in the two-dimensional plane strain field.

Joint elements was set up at the interfaces between the caisson and rubble backing and rubble mound. The authors conducted a simple shear test and shaking table test of concrete caisson-rubble mound system employing a similar crushed stone whose maximum grain size was 25 mm in order to measure the friction coefficient between concrete and crushed stone, and obtained the friction coefficient of about from 0.47 to 0.54 (Japan Society

Fig. 12. Schematic figure of evaluation of nodal forces in FE analysis equivalent to contact forces at boundary surfaces after DE analysis

Fig. 13. Analysis model of seawall

of Civil Engineer, 1996). Referring to these test results, a friction coefficient of 0.5, which was about the average value, was used in the analysis. Furthermore, joint elements were also set up at both of the side boundaries of the analysis model, because the side walls of the soil container at the centrifuge model tests were rigid aluminum plate. A friction coefficient of 0.5 was adopted based on the ring-type torsion shear test on the friction resistance between aluminum and sand (friction coefficient 0.47~0.58) carried out by Kishida and Yoshimi (1977).

The gravitational acceleration during the analyses was 30G which was the same value as the centrifugal acceleration in the model tests and both the packing analysis of the armor units and the initial stress analysis of the FE domain (elastic analysis) were carried out under this gravitational acceleration. Although the nominal amplitudes of the horizontal input acceleration (sine wave, frequency 90Hz, wave number 12) were 3G, 10G and 15G, the actual recorded accelerations on the shaking table were input from the bottom of the model.

Determinations of Model Parameters

Model parameters of armor unit

In the DE analysis, the translation and rotation of each element were analyzed. Then the mass (M)_{ANL} and moment of inertia (I)_{ANL} of an element were necessary to solve the equations of motion (Tochigi, 1995). As mentioned above, because the DE analysis is performed in the two-dimensional plane strain field, it is not possible to square the density and porosity with the values in the model tests. Therefore, in the analysis, the mass and the moment of inertia of the element were modified so that the total mass per unit width (1 m) after the packing analysis of the armor units corresponded to the value in the experiment as indicated in the following relationships;

$$(M)_{ANL} = (\text{total mass of armor units in the experiment; per unit width 1 m}) / (\text{total number of armor unit elements in the analysis})$$

$$(I)_{ANL} = (\text{specific gravity of element}) \times (\text{polar moment of inertia of element})$$

Table 3. Combinations of stiffness of spring K_{ij} at contact point based on loading tests of armor unit model

	$K_1/2^*$	K_2	K_3
$K_1/2^*$	8500	5030	4390
K_2	5030	3570	3240
K_3	4390	3240	2960

(kN/m)

* Supported by two sets of spring in the case of one point supporting condition

Table 4. Input model parameters of armor unit for distinct element analysis

	Unit (per 1 m)	Input value
Mass (M) _{ANL}	N	16.6
Inertia moment (I) _{ANL}	N·m ²	4.10×10^{-3}
Stiffness of spring (K) _{ANL}	kN/m	1.68×10^5
Damping coefficient (C) _{ANL}	N·s/m	3360

Where the specific gravity of the element is expressed by;

$$\text{Specific gravity} = (\text{mass of element}) / (\text{volume of element; per unit width 1 m}) / (\text{density of water})$$

The mass and the moment of inertia of the element for the numerical simulations obtained from the above relationships are presented in Table 4.

Mechanical models at the contact point of the armor unit element in the DE analysis are shown in Fig. 15 (Tochigi, 1995). Spring and dashpot are set up at the contact point to both the normal and the shear directions. Even though there is no physical meaning against setting the dashpot, it is necessary to obtain stable solutions during the analysis. A divider in the normal direction means the separation between two elements. A slider in the shear direction expresses the sliding phenomena according to the friction coefficients, and 0.6 and 0.5 were used in the analysis for the coefficients of the static and dynamic friction, respectively. It is difficult to easily determine

Fig. 14. Configuration and dimensions of armor unit for centrifuge model test and distinct element analysis: (a) Centrifuge model test, (b) Two-dimensional distinct element analysis

the stiffness of the spring at the contact point. Regarding the stiffness of the spring, a relationship between the contact force and relative displacement at the contact point of two elastic bodies was theoretically introduced as follows;

$$\Delta u = \frac{\Delta f}{K_{ij}} = \left(\frac{1}{K_i} + \frac{1}{K_j} \right) \Delta f \quad (1)$$

in which Δu =relative displacement; K_{ij} =stiffness at the contact point; Δf =contact force; K_i and K_j =stiffness of (body)_i and (body)_j. This means that the stiffness of the spring at the contact point can be expressed by connect-

ing each stiffness in series. In the present study, the stiffness at the contact point was simply evaluated from the loading tests of the armor unit model employed in the experiment. Schematic figures of loading tests are illustrated in Fig. 16. TEST -1 was a loading test with the armor unit standing straight on the base, and TEST-2 was a test with the armor unit inclined. In this paper, TEST-1 and TEST-2 were named a one-point loading/three-point supporting system and two-point loading/two-point supporting system, respectively. The relationships between force and displacement obtained from the above two types of tests are shown in Fig. 17. The averaged line of each test

Fig. 15. Mechanical models at contact point

Fig. 16. Schematic figure of loading tests of armor unit model

is also drawn in the figure. Stiffnesses K_{EX1} and K_{EX2} , corresponding to the slopes of the averaged lines, were expressed as follows from Equation (1);

$$K_{EX1} = \frac{1}{\frac{1}{K_1} + \frac{1}{3K_3}} \quad (2)$$

$$K_{EX2} = K_2 \quad (3)$$

in which K_1 =stiffness of the projection in the one-point supporting condition, K_2 =stiffness of the projection in the two-point supporting condition, and K_3 =stiffness of the projection in the three-point supporting condition. K_2 was easily determined from the results of TEST-2, but K_1 and K_3 could not be determined from these data alone. Therefore, in this analysis, each value was estimated from the stiffness ratio (K_3/K_1) introduced by the elastic beam theory (see APENDIX-A). Strictly speaking, the stiffness would be changed according to the location of the contact point on the projection of the armor unit. But in order to simplify the problem, the averaged stiffness derived from the combinations of K_1 , K_2 and K_3 obtained by the above loading tests was adopted to the analysis. The number of combinations of K_1 , K_2 and K_3 was nine and the value of stiffness (K_{ij}) at each combination is listed in Table 3. The averaged stiffness (K_{ij})_{AVE} from these values became 4480 (kN/m). Furthermore, the

Fig. 17. Relationships between force and displacement by loading tests of armor unit model: (a) TEST-1, (b) TEST-2

value of the stiffness (K)_{ANL} for the analysis was also modified in the following way considering the number of the armor unit elements in the experiment;

$$(K)_{ANL} = (K)_{AVE} \times (\text{total mass of armor units in the experiment; per unit width 1 m}) / (\text{total number of armor unit elements in the analysis})$$

Regarding the damping necessary to obtain stable solutions, the damping coefficient (C)_{ANL} in the analysis was presumed as the damping ratio h became 10% by the fol-

Fig. 18. Liquefaction strength of sand: (a) Sand seabed, (b) Backfill

lowing equation.

$$(C)_{ANL} = 2h \sqrt{(M)_{ANL}(K)_{ANL}/g} \quad (4)$$

in which g = gravitational acceleration.

The values of $(K)_{ANL}$ and $(C)_{ANL}$ determined from the above mentioned procedures are also shown in Table 4. The same values of $(K)_{ANL}$ and $(C)_{ANL}$ were used to both the normal and the shear directions at the contact point in the analysis. The buoyancy force was considered against the armor unit elements whose center of gravity existed under the fluid during the analysis.

Model parameters of sand and rubble

The areas such as the sand seabed, backfill, rubble mound and rubble backing consisting of the sand and rubble materials in the FE domain were analyzed as the effective stress analysis domain introducing the constitutive relationships based on the elasto-plasticity theory. The model parameters of the sand seabed and backfill sand were determined by simulating the liquefaction strength tests of the Nevada sand used in the centrifuge model tests. In the tests, a hollow cylindrical torsion shear testing apparatus (inner diameter = 60 mm, outer diameter = 100 mm and height = 120 mm of the specimen) was employed and the relative densities were adjusted to approximately 60% and 90%, which corresponded as closely as possible to the values of the centrifuge model tests. A cyclic shear stress was applied to the specimen after the isotropic consolidation. Consolidation pressures in the tests and the simulation analyses are described in Fig. 18(a) and (b). In particular, considering that the liquefaction strength of dense sand was dependent on the confining pressure (Kanatani et al., 1994) and confining pressures of saturated dense sand layer at the centrifuge model test were approximately within the range of 40 kPa ~ 110 kPa, two kinds of confining pressure of 49 kPa and 98 kPa were selected during the tests of dense sand. The frequency of cyclic loading was 0.1 Hz. Comparisons of the liquefaction strength of the sand seabed and the backfill between test and simulated results are shown in Fig. 18(a) and (b), respectively.

On the other hand, model parameters of the rubble mound and the rubble backing which consisted of crushed stone were determined by simulating the dynamic deformation tests of the material used in the model

Fig. 19. Dynamic deformation characteristics of rubble material: (a) Rubble mound, (b) Rubble backing

tests. In the tests, a cylindrical cyclic triaxial testing apparatus (diameter = 100 mm, height = 120 mm of the specimen) was used and the relative density was adjusted to from 80% to 90% of the value of the model tests. Cyclic axial load was applied to the specimen after the isotropic consolidation. Confining pressures during the tests were changed within the range of 49 kPa ~ 98 kPa and the test results at the confining pressure of 49 kPa were selected as the target against the simulation analysis, as is shown in Fig. 19(a) and (b), considering the representative confining pressure at the centrifuge model test. The frequency of cyclic loading was 0.1 Hz. In addition, in order to avoid the effect of bedding error included in the measurement value of the axial displace-

Table 5. Input model parameters for dynamic response analysis in FEM domain

	γ (kN/m ³)	G_o (MPa)	k (m/s)	e	ϕ_f ($^\circ$)	ϕ_m ($^\circ$)	m^* (1/kPa)	κ	β_1	β_2	G_{ot}^*
Backfill	20.159	63.0	1.53×10^{-5}	0.57	40	30	1.02×10^{-4}	0.00078	970	0	50
Rubble mound	20.492	83.5	1.00×10^{-3}	0.56	40	30	1.02×10^{-4}	0.0005	400	100	50
Rubble backing	19.463	40.6	1.00×10^{-3}	0.730	40	30	1.84×10^{-4}	0.0015	450	200	50
Sandbed (under sea)	19.326	77.5	1.53×10^{-5}	0.71	40	30	1.84×10^{-2}	0.00046	900	0	50
Sandbed (beneath armored embankment)	19.326	50.0	1.53×10^{-5}	0.71	40	30	8.67×10^{-3}	0.00072	900	0	50
Sandbed (beneath mound)	20.041	107.9	1.53×10^{-5}	0.59	40	30	1.33×10^{-4}	0.00135	910	0	50
Sandbed (beneath backfill)	19.326	69.7	1.53×10^{-5}	0.71	40	30	6.12×10^{-3}	0.00102	950	0	50

γ : wet unit weight, G_o : initial shear modulus, κ : permeability coefficient, e : void ratio, ϕ_f : friction angle at failure, ϕ_m : friction angle at phase transformation, κ : gradient at swelling in e -log($\sigma m'$) relationship, m^* : constant related to maximum contractive volumetric strain, β_1 : constant for expressing cyclic mobility phenomenon

ment, targets of the displacement transducers were attached to the lateral surface of the specimen and relative axial displacement between two targets was measured by using the non-touched displacement transducers installed in the pressure cell (Tanaka et al., 1994). Comparisons of the dynamic deformation characteristics of the rubble mound and the rubble backing are shown in Fig. 19(a) and (b), respectively.

Through the above procedures, the model parameters of sand and rubble areas in the model (see Fig. 13) were determined as presented in Table 5. Parameters ϕ_f and ϕ_m are the material constants associated with the internal friction and phase transformation angle, respectively. These values were presumed by referring to Iai et al. (1993) and Iai et al. (1998). Of course, the internal friction angle of sand depends on the density and it is thought that there is some degree of difference in the internal friction angle between medium dense and dense sand in the model test. But as the liquefaction strength curve obtained from the calculation by the present constitutive relationships was empirically insensitive to the degree of difference of the internal friction angle compared with other parameters associated with the volume change and stiffness, the same value of the internal friction angle was presumed for all sand layers. The meanings of each of the model parameters are briefly explained beneath the table. Refer to Nishi and Kanatani (1990) and Kanatani et al. (1994) for detailed descriptions of the constitutive relationships and model parameters. Model parameters of caisson, superstructure

The caisson and the superstructure on the caisson made of concrete were treated as elastic bodies in the analysis; the masses of each structure were 20.75(kN/m³) and 20.67(kN/m³), respectively. And the shear modulus and Poisson's ratio were the same values of 9000(MPa) and 0.167 for both structures.

Numerical Time Integration

The numerical time integration method for the DE analysis was the central differential method and its time step was 5×10^{-7} second. On the other hand, Newmark's

β method ($\beta=0.25$, $\gamma=0.5$) in which the time step was 5×10^{-6} second, was adopted for the FE analysis. In this condition, FE analysis was carried out once in comparison to 10 times for the DE analysis, and this led to the shortening of the total analysis time. Of course, it was confirmed by several pre-study analyses that this operation did not lead to serious differences in the response of the model at all. In the FE analysis, iteration was not carried out in order to avoid huge calculation time; so the time step was sufficiently shortened and unbalanced forces induced in each step were taken into account as external forces of the next step of the analysis.

Comparisons of Experiment and Analysis Results

Time histories of acceleration and displacements

Figure 20 shows the comparisons of the time histories of the accelerations and the displacements at the top of the caisson and the surface of the backfill behind the caisson in the case of the nominal amplitude of the input acceleration 15G. Each location can be seen in Fig. 3(a). The sign convention for acceleration and displacement is defined positive for the seaward and downward direction.

Amplitudes of the response accelerations obtained from both the experiment and the analysis are reduced after the initial one half or two waves because of the softening of soils. Horizontal displacement (DH1) at the top of the caisson is gradually accumulated to the seaward direction in the experiment. The analysis results also exhibit the same tendency, although the dynamic amplitude is a little small compared with the experimental one. On the other hand, the vertical displacement (DV1) at the top of the caisson gradually progresses to the downward direction (i.e. settlement) in both the experiment and analysis. For the vertical displacement at the surface of the backfill behind the caisson induced by the movement of the caisson to the seaward direction and cyclic shearing, the analysis result is also coincident with the experimental one. Distributions of excess pore fluid pressure ratio

Distributions of the excess pore fluid pressure ratio at the end of shaking, in which the excess pore fluid pres-

Fig. 20. Comparisons of time histories of acceleration and displacement: (a) Acceleration, (b) Displacement

sure ratio is defined by the pore fluid pressure divided by the initial effective vertical stress, are shown in Fig. 21. In the case of the nominal amplitude of the input acceleration 15G, the experimental excess pore fluid pressure ratios in the sand seabed beneath the caisson reach only about 50% whereas the excess pore fluid pressure ratios in other areas of the sand seabed and the backfill achieve the 100% level as designated in Fig. 7. These main charac-

teristics on the distributions of the excess pore fluid pressure ratios derived from the model tests are clearly exhibited in the analysis result, as especially shown in Fig. 21(b).

Movements of armor units and deformation of seawall

Figures 22(a) and (b) show the computed movements of the armor units at the end of the shaking under the nominal amplitudes of the input acceleration of 3G and

Fig. 21. Contours of excess pore fluid pressure ratio at end of shaking by analysis: (a) Amplitude of input acceleration = 3G, (b) Amplitude of input acceleration = 15G

Fig. 22. Movements of armor units at end of shaking by analysis: (a) Amplitude of input acceleration = 3G, (b) Amplitude of input acceleration = 15G

15G, respectively. The movements of the armor units are larger in accordance with the larger input acceleration and the armor units are moved to the direction of the sea along the slope of the embankment in the case of the large input acceleration of 15G. This performance is basically consistent with the experimental one illustrated in Fig. 6. In the same style, computed deformations of the FE domain at the end of shaking are shown in Figs. 23(a) and (b), showing the calculated displacements 15 times.

The caisson is tilted and moved horizontally towards the sea with settlement having been found in the model tests (*see* Fig. 6); nevertheless a large number of armor units exist in front of the caisson. In particular, from Fig. 23(b) it is clear that the horizontal movement of the caisson is strongly affected by the shear deformation towards the sea in the sand seabed beneath the rubble mound. This is also consistent with the observed deformation mode at the model tests. These results support the idea

Fig. 23. Deformations of sand seabed, backfill, rubble mound and caisson at end of shaking by analysis: (a) Amplitude of input acceleration = 3G, (b) Amplitude of input acceleration = 15G

that the proposed analysis method is able to qualitatively reproduce the overall deformation behavior of the sea-walls during shaking well. Looking at Fig. 23(b) in detail, on the right side of the model, backfill near the ground surface by the analysis penetrated somewhat into the side boundary. In this analysis, joint elements were set up at the side boundaries in order to consider the boundary condition of the rigid walls of the soil container. At an initial stage of the simulation analysis, a higher stiffness of spring of the order of 10^7 kN/m³ for shear and normal directions was used, but the stable solution could not be obtained under the condition of large input acceleration. This was probably due to the higher contrast between the stiffness of spring of joint elements and the stiffness of the ground elements. In particular, the stiffness of the ground near the surface was originally low and it decreased further due to the softening by shearing during excitation. Therefore, in order to avoid such unstable conditions, the stiffness of spring of several joint elements contacted near the ground surface of the backfill and sand seabed was reduced by one order from the above value. By this treatment, a stable solution was obtained against the large input acceleration condition, though the backfill near the ground surface penetrated somewhat into the side boundary. However it was thought that this did not have a much influence on the deformation of caisson.

Residual displacements of caisson

Estimation of the displacement of the caisson during an earthquake is very important for the performance

Fig. 24. Comparisons of residual displacements at top of caisson

verification design of the seawalls as mentioned in the introduction. Figure 24 shows the comparisons of the residual displacements DH1, DV1 and DV2 of the caisson. DH1 is the residual horizontal displacement at the sea side top of the caisson and DV1 and DV2 are the residual vertical displacements at the sea side and land side top of the caisson, respectively. All of the computed residual displacements coincide well with the experimental results. These also demonstrate that the proposed method is able

to become an effective technique to predict the residual displacements of the caisson from the quantitative point of view.

CONCLUSIONS

In order to predict the deformation of the seawalls covered with an armored embankment, centrifuge shaking table tests were conducted and the deformation analysis method was newly developed. The major conclusions drawn from this study can be summarized as follows:

- (1) From the centrifuge shaking table tests of the seawall, it was observed that the residual displacements of the caisson during shaking were dependent on the condition of the foundation ground. Little displacement is induced in the case of a hard foundation ground such as a rock foundation and the improvement by the densification of the sand seabed only beneath the rubble mound led to quite a reduction of the residual displacements. And it was also shown that the estimations of plastic shear deformation in the sand seabed and the effect of the armored embankment in front of the caisson were very important to accurately predict the displacement of the caisson during a large earthquake.
- (2) The two-dimensional DEM-FEM coupled analysis method was newly developed to numerically predict the deformation of seawalls covered with an armored embankment during earthquakes. In this method, movement of the armor units was calculated by the DE analysis and deformation of the caisson, rubble mound, sand seabed and backfill were calculated by the FE analysis considering the non-linearity of the soil materials based on the effective stress. Dynamic interaction was taken into account by delivering the nodal displacements of the finite elements or the nodal forces converted from the contact forces of the distinct elements through the imaginary distinct elements at the boundaries between the DE and FE domain.
- (3) Numerical simulations were performed to verify the applicability of this method to the prediction of the deformation of the seawalls. Overall deformation behaviors including the armored embankment during shaking were qualitatively similar to the experimental ones. Furthermore, the residual displacements of the caisson and the distributions of the excess pore fluid pressure ratio in the sand seabed and backfill obtained from the analyses showed good agreement with the results of the experiments. These supported the idea that the proposed method could become an effective technique to predict the deformations of seawalls covered with an armored embankment.

ACKNOWLEDGEMENTS

A part of this research has been supported by nine Japanese Power Companies and Japan Atomic Power

Co. through the grant for Joint Research Program "Studies on development of siting technology of man-made island type NPP". The authors wish to acknowledge all the members related to this research program. They also express their highest appreciation to Dr. Koichi Nishi, Dr. Yukihiisa Tanaka and Dr. Toshiro Okamoto of the Central Research Institute of Electric Power Industry for useful suggestions and encouragements. The authors acknowledge the cooperation of the staff of the University of California at Davis under the instructions of Professor B. L. Kutter and the cooperation of the staff of the Takenaka Research and Development Institute for conducting the centrifuge model tests and, also acknowledge Mr. Masahiro Akimoto of Kozo Keikaku Engineering for assistance in making the computer program.

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APPENDIX-A

Simple Evaluation on Stiffness of Projection of Armor Unit by Elastic Beam Theory

Displacements of the projection of the armor unit induced by the unit force (1.0) parallel to the x - and y -axis at the edge of the top of the projection as schematically illustrated in Fig. A-1. are expressed as follows based on the elastic beam theory;

$$\delta_{xx} = \frac{1}{\pi E} \left\{ \frac{b}{r_1 r_t} + \frac{4b}{3r_1} \left(\frac{1}{r_t} + \frac{1}{r_1} + \frac{r_t}{r_1^2} \right) \right\} \quad (\text{A-1})$$

$$\delta_{yy} = \frac{4}{3\pi E} \left(\frac{b}{r_1} \right) \frac{1}{r_t} + \frac{2(1+\nu)\kappa}{\pi E} \frac{b}{r_1 r_t} \quad (\text{A-2})$$

$$\delta_{xy} = \delta_{yx} = -\frac{2}{3\pi E} \left(\frac{b}{r_1} \right)^2 \left(\frac{2}{r_1} + \frac{1}{r_t} \right) \quad (\text{A-3})$$

in which E =Young's modulus; ν =Poisson's ratio; κ =section modulus of circle (10/9). Furthermore, the displacement induced by the unit axial force in the one-point supporting condition of the projection is expressed as follows;

$$\delta_{xn} = \frac{1}{\pi E} \frac{b}{r_1 r_t} \quad (\text{A-4})$$

In this case, the stiffness corresponding to K_1 is evaluated

Fig. A-1. Schematic figure of projection of armor unit and loading conditions to evaluate displacements of projection by elastic beam theory

Fig. A-2. Schematic figure to evaluate stiffness of projection of armor unit under three-point supporting condition

by $K_1 = 1/\delta_{xn}$.

On the other hand, the displacement perpendicular to the floor induced by the unit force perpendicular to the floor in the three-point supporting condition of the projection as shown in Fig. A-2. (see TEST-1 in Fig. 15) is expressed as follows;

$$\delta_{tt} = \delta_{xx} \sin^2(\eta) + \delta_{yy} \cos^2(\eta) - 2\delta_{xy} \sin(\eta) \cos(\eta) \quad (\text{A-5})$$

in which η =angle between x -axis and floor ($\tan(\eta) = 1/2\sqrt{2}$). In this case, the stiffness corresponding to K_3 is evaluated by $K_3 = 1/\delta_{tt}$.

From the above relationships and Poisson's ratio of mortar, the stiffness ratio K_3/K_1 is theoretically obtained and K_3/K_1 becomes 0.174 supposing 0.167 as Poisson's ratio of the mortar.