16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1506 (Abstract ID)

Registration Code: S-C1463115046

MODEL SHAKE TESTS AND NUMERICAL ANALYSIS ON SEISMIC BEHAVIOR OF GRAVITY TYPE QUAY WALL USING A COARSE-GRAINED ROCK WASTE

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Abstract

The 2011 of the Pacific coast of Tohoku Earthquake caused the damages of mooring facilities, such as seaward displacement of quay walls and subsidence of backfill ground. It is supposed that ground surface subsidence at quays reclaimed with coarse-grained rock waste were influenced by its volumetric shrinkage characteristic during the earthquake. In this paper, we conducted shaking table tests with laminar shear box, indicating the relationship between the volumetric strain and relative density of coarse-grained rock waste. Further, conducting shaking table tests of a gravity type quay wall, it is clarified that horizontal displacement of the gravity type quay is less dependent on density of the coarse-grained rock waste. Effective stress analysis was carried out to simulate the results of the laminar shear box test and the model quay wall test, using the same model parametars for the coarse-grained rock waste.

Keywords: coarse-grained rock waste, shake table test, effective stress analysis

1. Introduction

Coarse-grained rock waste (hereafter rock debris) is a byproduct that is produced in stone processing, and is a non specification material. Accordingly, various kinds of rock debris are produced depending on host rocks, grainy textures and particle configurations. They generally consist of gravel-sized or larger stone particles, and has high permeability, without large accumulation of excess pore water pressure on seismic conditions; therefore it is often applied as a land reclamation material to prevent liquefaction.

Many harbor facilities were damaged by the 2011 off the Pacific coast of Tohoku Earthquake. For quay walls, damages such as distorted front side and settlement of ground (land subsidence) behind the walls were reported. [1] According to a research result by Takahashi et al [2], on a caisson type quay wall using rock debris as land reclamation material in Hitachinaka Port Area of Port of Ibaraki, the caisson was pushed out toward the ocean and approximately 1.3 m of land subsidence was observed. Since there was no sand boils observed on the surface of the reclaimed ground, liquefaction probably did not occur on rock debris. There is a possibility that in addition to the horizontal movement of the quay wall, the volumetric change of the rock debris during the earthquake caused the land to subside.

Lots of research had been carried out on the dynamic characteristics of rock debris focus on the generation of excess pore water pressure by vibration test using a shear box [3, 4], but few of them analyze the volumetric shrinkage by repeated shearing during seismic movement.

In this research, laminar shear box tests using shaking table were firstly performed in order to evaluate the characteristic of volumetric shrinkage of rock debris caused by cyclic loading. Also, model vibration test on the caisson type quay wall was performed to analyze the influence of using rock debris as the backfill material on the structural stability of the quay wall. In addition, based on the results of these tests, behaviors of the rock debris and the quay wall were simulated by two-dimensional effective stress analysis.

2. Shaking table test

2.1 Basic properties of rock debris

Table 1 shows the basic properties of the rock debris that was used for the testing. This rock debris was collected from the quarry where the rock debris used for the quay walls referred to in the report by Takahashi et al, and the maximum grain size ϕ of the rock debris is 106mm. Fig.1 shows the grain size distribution curve. This curve indicates that fine particle fraction and sand fraction are very little, the fraction of gravel and larger grains composes most of the whole, and thus the permeability is high. Also based on the design standards on port structures [5], the possibility of liquefaction with this material is considered to be low. Particles of the rock debris are flat shaped and angular.

Table 1 – Basic properties of the rock debris

Particle density ρ_s (g/cm^3)	Maximum dry density of soil ρ_{dmax} (g/cm^3)	Minimum void ratio e_{min}	Maximum void ratio e_{max}
2.734	2.003	0.385	0.752

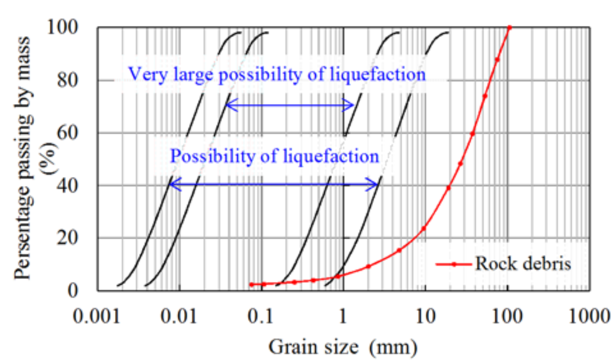


Figure 1 — Grain size distribution curve of rock debris

2.2 Shaking test using laminar shear box

2.2.1 Description

The purpose of the vibration test with laminar shear box is to evaluate the characteristics of subsidence of rock debris at the time of an earthquake. The test was performed with a laminar shear box with height of 1.0m, width of 2.0m and depth of 1.0m, installed on a large shaker table. For measurements, accelerometers and pore water pressure gauges were placed in the ground. Laser displacement gauges were placed on the steel frame and on the ground surface, wire displacement sensor were connected to the subsidence plates to measure the vertical displacement in the ground during the vibration, and targets were placed on the ground to measure subsidence after the vibration. Fig.2 shows the model dimensions and the placements of gauges. As the laminar shear box is flexibly movable in the horizontal directions, and can be deformed along the ground response. As Table 2 shows, two low density ground cases and one high density ground case, three cases in total were carried out. Low density ground was prepared by gently placing air dried rock debris with shovels. High density ground was prepared by stomping the low density ground with man power. After forming the rock debris ground to the designated height, water was injected through bottom of the box slowly to equalize the underground water level to the ground surface level.The scale ratio of the model λ was determined to be 20 (actual scale/model scale) considering the 20m thick rock debris layer on the actual damaged quay wall. For the similarity rule, the similarity for the shaking table test at a gravitational field [6] proposed by Iai was applied. This similarity is calculated based on equilibrium equations of forces and continuity equation of two-layer saturated material (pore water and soil skeleton) of the ground. Table 3 shows the similarity and the scale ratio applied for the test. From now on, the test results will be indicated by the actual scale based on the applied similarity. Fig.3 shows the input seismic wave used for the test. For the seismic wave, the estimated seismic motion at observation station Hitachinaka-U by the harbor area seismological observatory [7] was converted to E+F wave on the engineering bedrock for use. Similarity wave applied with the time scale $\lambda^{0.75}$ and actual size wave with the original scale were used. The prepared grounds were vibrated three times with the similarity wave and the actual size wave respectively.

Table 2 ― Initial relative density of tests case

case No.	Density	Initial relative density *Dr* (%)
case1	low density	25
case2	low density	34
case3	high density	55

Table 3 ― Similarity and scale ratio

Parameter	Real/model	Scale
Length	λ	20.00
Density	1	1.00
Time	$\lambda^{0.75}$	9.46
Stress	λ	20.00
Pore water pressure	λ	20.00
Displacement	$\lambda^{1.5}$	89.44
Acceleration	1	1.00
Strain	$\lambda^{0.5}$	4.47
Stiffness	$\lambda^{0.5}$	4.47
Hydraulic conductivity	$\lambda^{0.75}$	9.46

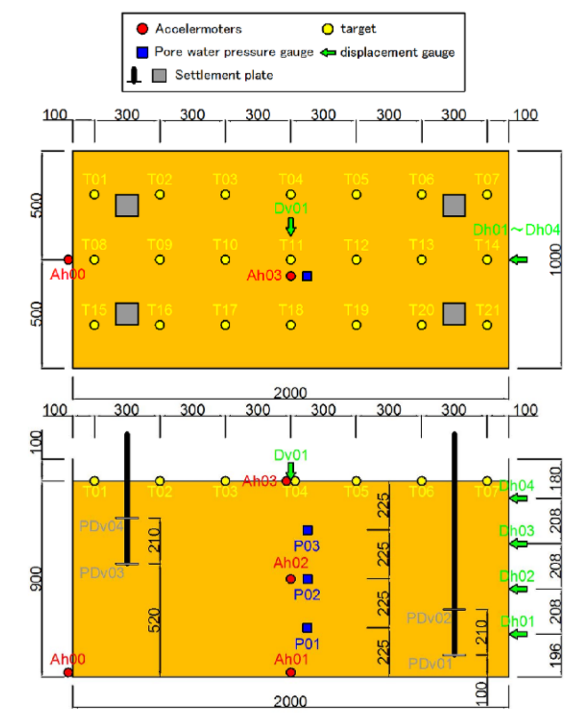
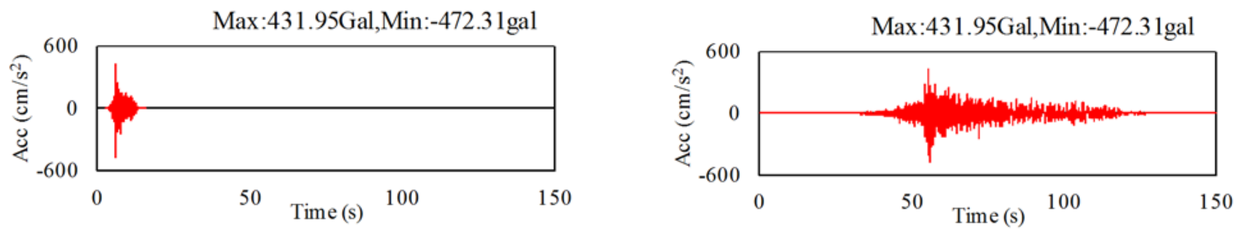


Figure 2 ― Model dimensions and placements of gauges (scale: mm)



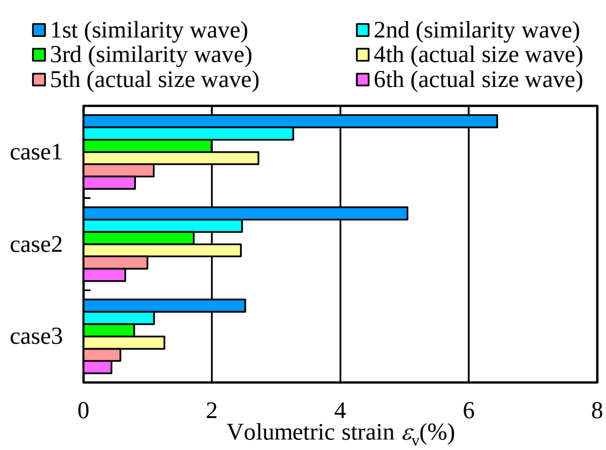
(a) Similarity wave

(b) Original scale wave

Figure 3 — Input seismic waves

2.2.2 Results and observations

Fig.4 shows the volumetric strain caused by each vibration tests. Volumetric strain was calculated based on the layer thickness before each vibration. By first vibration with the similarity wave, 6.5% and 2.5% of volumetric strain occurred for case 1 and case 3 respectively and the amount of strain decreased as the vibration was repeated. As this is assumed to be due to the increased rock debris ground density, the relationship between relative density and the volumetric strain are shown in Fig.5. The white circles (○) show the results with vibration by the similarity wave, and the solid colored circles (●) show the results with the actual size wave. This indicates that the volumetric strain decreases as the relative density increases, and when the relative density reaches 70%, volumetric strain barely occurs even with vibrated by the actual size wave that has large displacement amplitude with long duration. Overall, volumetric strains over the three times vibration by similarity waves for case 1, case 2 and case 3 were 11.6%, 9.2%, and 4.4% respectively. These strains were calculated dividing the accumulated subsidence due to the repeated similarity wave excitations by the initial ground thickness.



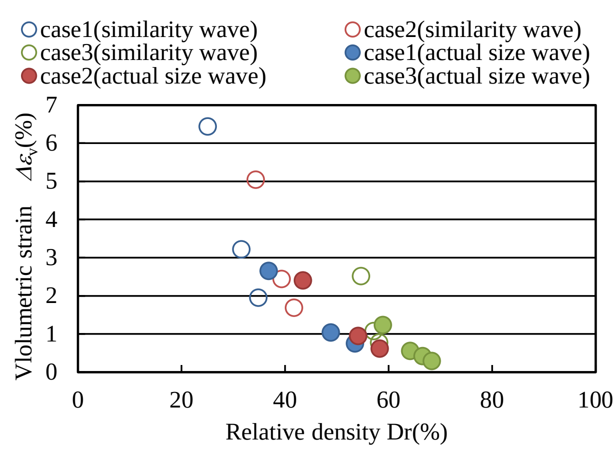


Figure 4 — Volumetric strain caused by each stepped vibration

Figure 5 — Relationship between relative density and volumetric strain

2.3 Caisson model vibration test

2.3.1 Description

A caisson model vibration test was performed with a steel rigid box with a height of 1.5m, width of 4.0m and depth of 2.8m, installed on a large underwater shaking table. As the test was conducted on two cases, namely, one with low and the other with high density of the rock debris ground for the back filling, a partition plate was placed in the box to divide the depth into 2 parts (1.4m for each), making simultaneous vibration on the two cross sections possible. A quay model with similarity index 1/20 was made based on the gravity type quay wall described in the report by Takahashi et al. The bottom base layer was prepared using Iide silica sand #6 with 6% of jet cement in it and made into solidified soil assuming an engineering bedrock layer. A box of caisson was placed for each cross section. After preparing the ground, tap water was injected to fill the ground pores and also to create the sea water area. It should be noted that the relative density of the rock debris layer was 42% for the low density ground and 93% for the high density ground. Accelerometers and pore water pressure gauges were placed in the ground. The displacement gages were placed as follows: one on the top of the caisson; on the front face of caisson0.15m and 0.93m above the rubble mound, respectively; and one on the surface of rock debris layer that is 0.2m behind the caisson (The positions refer to Fig.9). Also, targets were placed on the top of the caisson and the surface of rock debris ground in order to measure the subsidence after vibration. In the same way as the laminar shear box test, the similarity rule in the shaking table test at a gravitational field [6] proposed by Iai was applied. The test conditions and results will be indicated by the model scale in the followings. The same input seismic wave was used as in the laminar shear box vibration test, which is shown in Fig.3 (similarity wave).

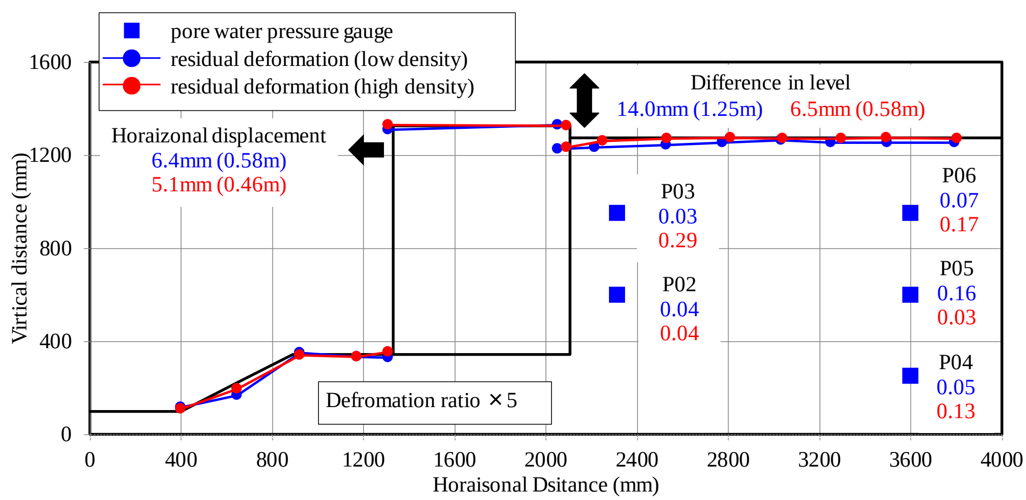


Figure 6 ― Residual deformation after shaking (blue line : low density, red line : high density)

2.3.2 Results and observations

Fig.6 shows the residual deformation after vibration test. Results for the low density cross section are indicated in blue line and high density in red line in the figure. The deformation is magnified by five-fold in the figure. The values in parentheses for horizontal displacement of the top of the caisson and the level difference between the caisson and the rear ground were calculated to the actual scale based on the similarity rule. The blue squares (■) stand for the pore water pressure gauge's locations and the appended numbers indicate the maximum excess pore water pressure ratio. Excess pore water pressure barely increased on the both cross sections, indicating that liquefaction did not occur. Horizontal displacements of the top of the caisson were 6.4mm and 5.1mm for the low density cross section and the high density cross section respectively on the model scale, having little difference with each other. However, the level differences between the caisson and the rear ground were 14.0mm and 6.5mm for the low density cross section and the high density cross section respectively, where the former is almost twice of the latter. This is presumably because the subsidence of the rock debris portion caused by an earthquake varies depending on the ground density. Also, according to the test results converted to actual scales, the horizontal displacement of the caisson with the low density ground was 0.57m and the level differences between the caisson and the rear ground was 1.25m. Therefore, the deformation of the case of low density rock debris is close to the reported damage of the actual ground, and it is inferred on the basis of the test results that the rock debris used at the site was of low density.

For the high density ground, as the relative density of the rock debris layer was 93% and was high enough, it is inferred that volumetric shrinkage did not occur during the vibration. Therefore, it is inferred that the 0.58m of the level difference between the caisson and the rear ground was not because of volumetric shrinkage, but because of the 0.46m of the horizontal displacement of the caisson. Meanwhile, for the low density ground, the level difference between the caisson and the rear ground was 1.25m with the horizontal displacement of caisson of 0.58m, indicating that the level difference attributable to the volumetric shrinkage would be approximately 0.6m, which is calculated by excluding the amount of level difference caused by the movement of the caisson based on the case of the high density ground.

For the low density ground, the relative density of the rock debris layer was 42% before vibration, indicating that the relative density after vibration would be approximately 51%, based on the change in volume of the rock debris layer caused by the movement of the caisson and the subsidence of the ground surface. Here, considering the results of the laminar shear box test (Fig.5), volumetric strain with relative density 42% and 51% would be 3% and half of it respectively. With this in consideration, approximately 0.3m of subsidence due to volumetric shrinkage of the rock debris would occur if the quay experiencing the earthquake further received the same earthquake.

3. Numerical analysis by effective stress analysis

3.1 Behavior of rock debris in laminar shear box vibration test

Numerical analysis by two-dimensional effective stress analysis code FLIP (Ver.7.2.2) against the test performed in Chapter 2[8 ,9] was carried out. Numerical analysis of the laminar shear box vibration test was performed in the first phase, and then analysis for caisson model vibration test was conducted using the parameters of the rock debris successfully simulating the test results in the first phase. It should be noted that the laminar shear box test case to be analyzed was selected in such a manner that the difference in relative density is minimized between the laminar shear box test case and the caisson model test case. For the laminar shear box vibration test, one-dimensional analysis was applied as the influence of the lateral boundary condition was assumed to be small. Fig.7 shows the analysis model.

For the modeling of the rock debris, cocktail glass model [10 ,11] was applied to take the dilatancy into consideration under the drainage condition so that the subsidence by shaking can be expressed. Table 4 shows the parameters used for the rock debris modeling. The parameters of dynamic characteristics of rock debris shown in Table 4 were calculated from shear wave velocity obtained by pulsed wave excitation during the density measurements. The permeability coefficient k was estimated based on Creager's relationship between the general 20% grain size and permeability, which is mentioned in earlier part. The parameters of the cocktail glass model of the rock debris were determined based on the parametric study through the analysis of laminar shear box vibration test. ε_d^{cm}, which is the coefficient of the increase of negative dilatancy, was defined as 0.13

supposing that the volume would shrink from initial void ratio to the minimum void ratio e_{min}. Also, the parameter $\gamma\varepsilon_d^c$ that controls negative dilatancy and the parameter $\gamma\varepsilon_d$ that controls both positive and negative dilatancies were adjusted to be consistent with the test results. The other parameters of the cocktail glass model were defined as general values used for sand. For the input seismic wave, the value measured by accelerometer (Ah01), which was placed on the bottom of the shear box, was used.

Fig.8 shows the time histories of acceleration, excess pore water pressure, horizontal and vertical displacements, volumetric strain and shear strain in the laminar shear box vibration test. It should be noted that noise reduction was applied on the displacements and each strain in the test values using a 50Hz frequency low pass filter. Vertical displacement was made consistent for both the increase tendency during vibration and residual displacement amount. The horizontal displacement was difficult to define because the main movement was almost buried in the large electrical noises caused by measuring instruments and recording equipment. Nevertheless, time at around from 3 sec. to 12 sec. with large acceleration amplitude, it showed similar behavior to test values (Dh02) and (Dh03), and the amplitude was mostly consistent as well. Excess pore water pressure was lower than initial effective overburden pressure at all the measurement location, indicating that liquefaction did not occur. Excess pore water pressure at the location (P03) varies during the vibration, which is assumed to be due to the large influence of dynamic water pressure in the test. In addition, the time histories of volumetric strain and shear strain were compared based on the displacement measured by the laser displacement meters and the wire displacement sensors. The time histories of shear strain at (Dh04) and (Dh03), and of the volumetric strain at (PDv03) and (PDv04) are shown in Fig.8, which were collected at specifically close locations. While the time histories of volumetric strain of the test and the analysis were mostly consistent, the time history of shear strain of the analysis showed slightly larger in amplitude than the test. Based on these results, this analysis was interpreted to be proper, and the parameters of cocktail glass model were determined.

Table 4 ― Parameters of dynamic characteristics of rock debris

Parameter		Rock debris
Density	ρ_{sat}(t/m^3)	1.74
Reference confining pressure	σ_{ma}'(kN/m^2)	3.530
Elastic shear modulus at a confining pressure	G_{ma}(kN/m^2)	2,980
Bulk modulus at a confining pressure	$K_{U/La}$(kN/m^2)	7,772
Poisson's ratio	ν	0.33
Porosity	n	0.38
Shear resistance angle	ϕ_f(deg)	39
Cohesion	c(kN/m^2)	0
Maximum damping coefficient	h_{max}	0.24
Bulk modulus (pore water element)	K_f(kN/m^2)	2,200,000
Limit of contractive component	$-\varepsilon_d^{cm}$	0.13
Parameter controlling contractive component	$\gamma\varepsilon_d^c$	0.5
Parameter controlling dilative and contractive components	$\gamma\varepsilon_d$	0.08
Parameter controlling initial phase of contractive component	q_1	1.0
Parameter controlling final phase of contractive component	q_2	0.0
Power index of bulk modulus for liquefaction analysis	l_k	2.0
Reduction factor of bulk modulus for liquefaction analysis	γ_k	0.5
Small positive number to avoid zero confining pressure	s_1	0.005

Parameter controlling elastic range for contractive component	c_1	1.0

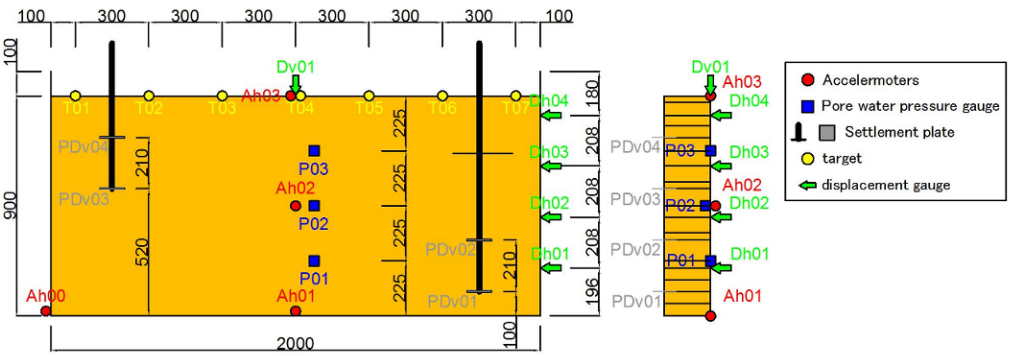


Figure 7 — Laminar shear box and one-dimension analysis model

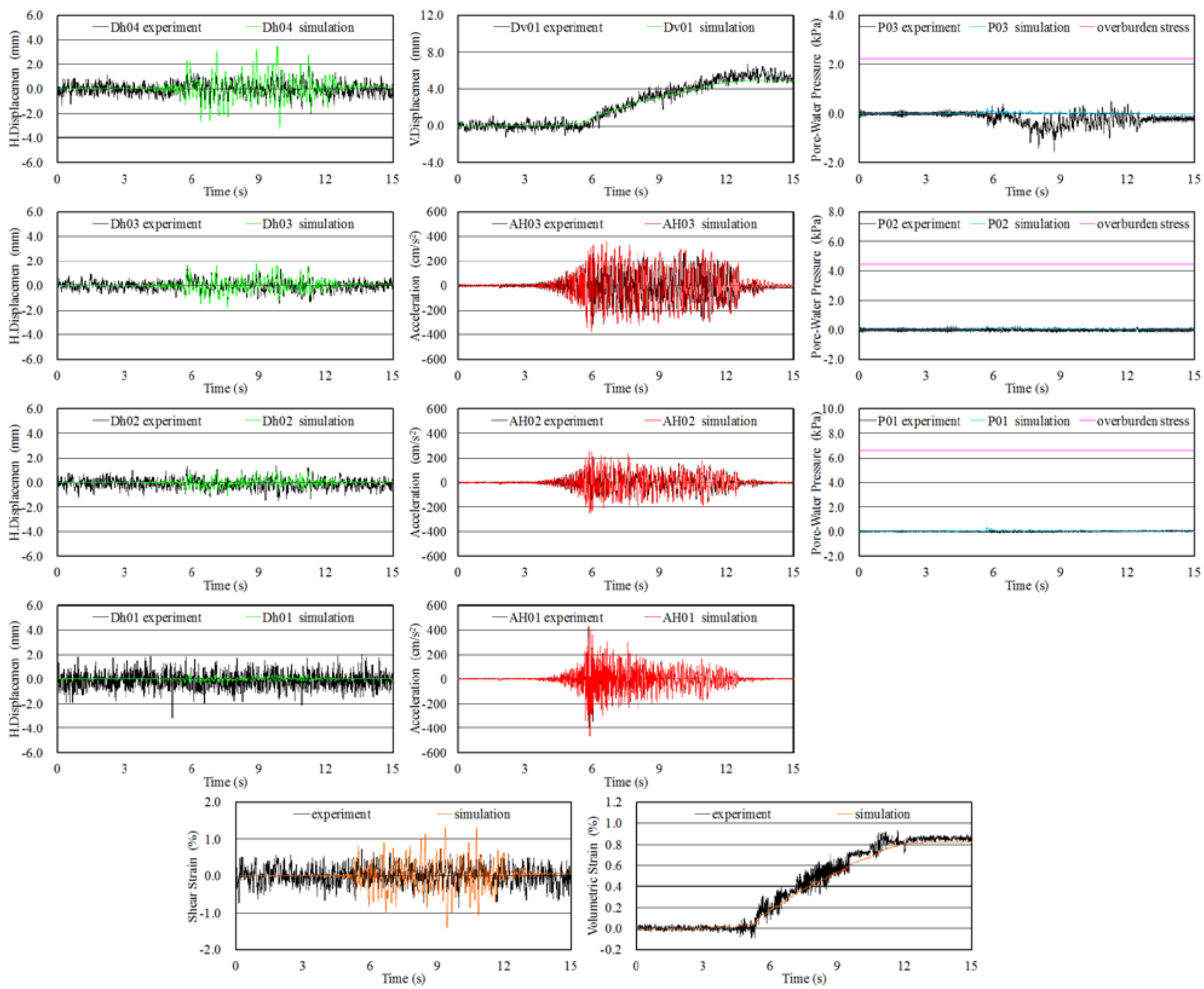


Figure 8 － Time history of acceleration, excess pore water pressure, horizontal and vertical displacements, shear strain and volumetric strain during laminar shear box vibration test and effective stress analysis result

3.2 Behavior of quay wall model

Continuously, numerical analysis with caisson model vibration test was performed using the dynamic characteristic parameters and cocktail glass model parameters of the rock debris determined from the analysis of laminar shear box test. The cocktail glass model was applied to the rock debris in the same way as the analysis for the laminar shear box test, and model parameters of other materials were determined as follows.

The caisson was modeled with linear and plane elements. The rubble mound was modeled with multi-spring elements. Table 5 shows the parameters of their dynamic characteristics. The parameters of dynamic characteristics were calculated based on the mass, water content and shear wave velocity measured during the test process. The rubble mound used for this test was made with Crushed stone #5. The angle of shear resistance ϕ_f was determined based on triaxial CD test which was performed separately. As the rubble mound was modeled with multi-spring elements of undrained condition while modeling based on the drainage conditions would be desirable, the bulk modulus of pore water K_f was reduced to give an apparent permeability [12 ,13]. Fig.9 shows analyzed cross section, acceleration, displacement and the output position of excess pore water pressure. The comparison of the time histories of test and analyzed results at each output position is shown in Fig.10. Subsidence of the rock debris (DV02) was well agreed with the increase tendency during vibration and the residual subsidence. The horizontal displacement of the front side of the caisson (DH02) was also accurately simulated, regarding both the residual displacement and the time history. For the vertical displacement (DV01) of the front side of the caisson, the subsidence that was shown during the test was not agreed. The excess pore water pressure (P04, P05, P06) did not reach the initial effective overburden pressure which is indicated in pink line in the figure, with little accumulation of excess pore water pressure in the same way as the test results. Although the acceleration response was smaller at (AH04-04) placed at a shallow location, the other acceleration were well agreed.

Fig.11 shows the comparison of residual displacements between the test and the analysis. Although the analyzed ground surface subsidence tends to show slightly smaller than that of test result as the distance from caisson increases, the level difference between the rock debris portion and the caisson, and the overall ground surface subsidence are mostly consistent with the test results.

Table 5 － Parameters of dynamic characteristics（Multi-spring model element）

Parameter		Solidification Improve soil	Rubble mound
Density	ρ(t/m^3)	2.00	2.00
Reference confining pressure	σ_{ma}'(kN/m^2)	5.625	5.625
Elastic shear modulus at a confining pressure	G_{ma}(kN/m^2)	245,000	10,370
Bulk modulus at a confining pressure	K_{ma}'(kN/m^2)	638,900	27,040
Poisson's ratio	ν	0.33	0.33
Porosity	n	0.45	0.47
Shear resistance angle	ϕ_f(deg)	0	39
Cohesion	c(kN/m^2)	720	20
Maximum damping coefficient	h_{max}	0.24	0.24
Bulk modulus (pore water element)	K_f(kN/m^2)	2,200,000	22,000

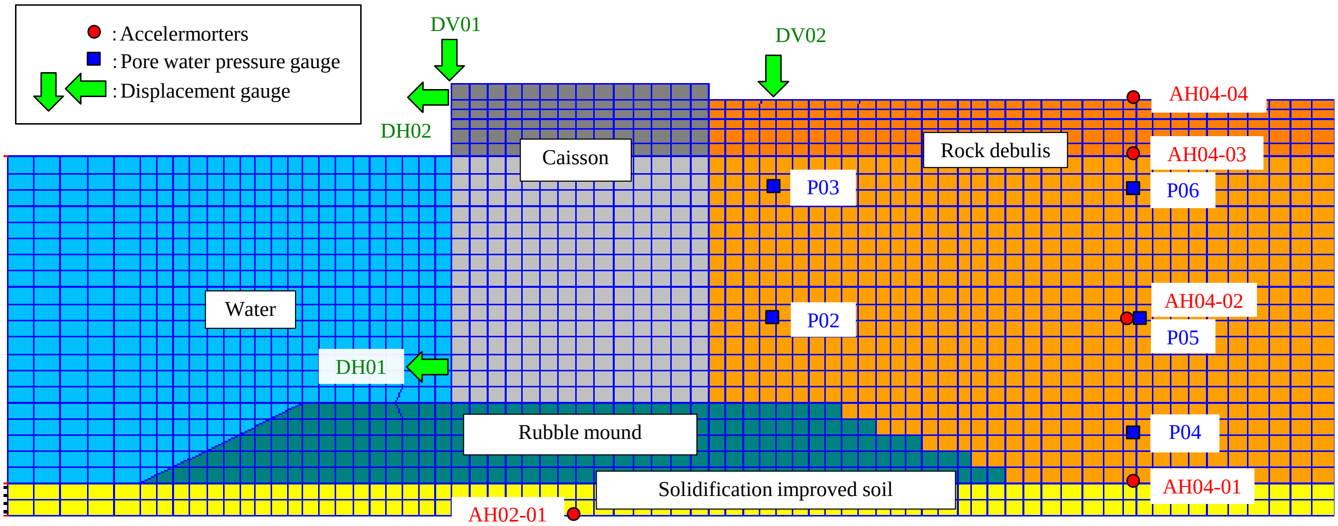


Figure 9 － Analysis model for caisson model vibration test

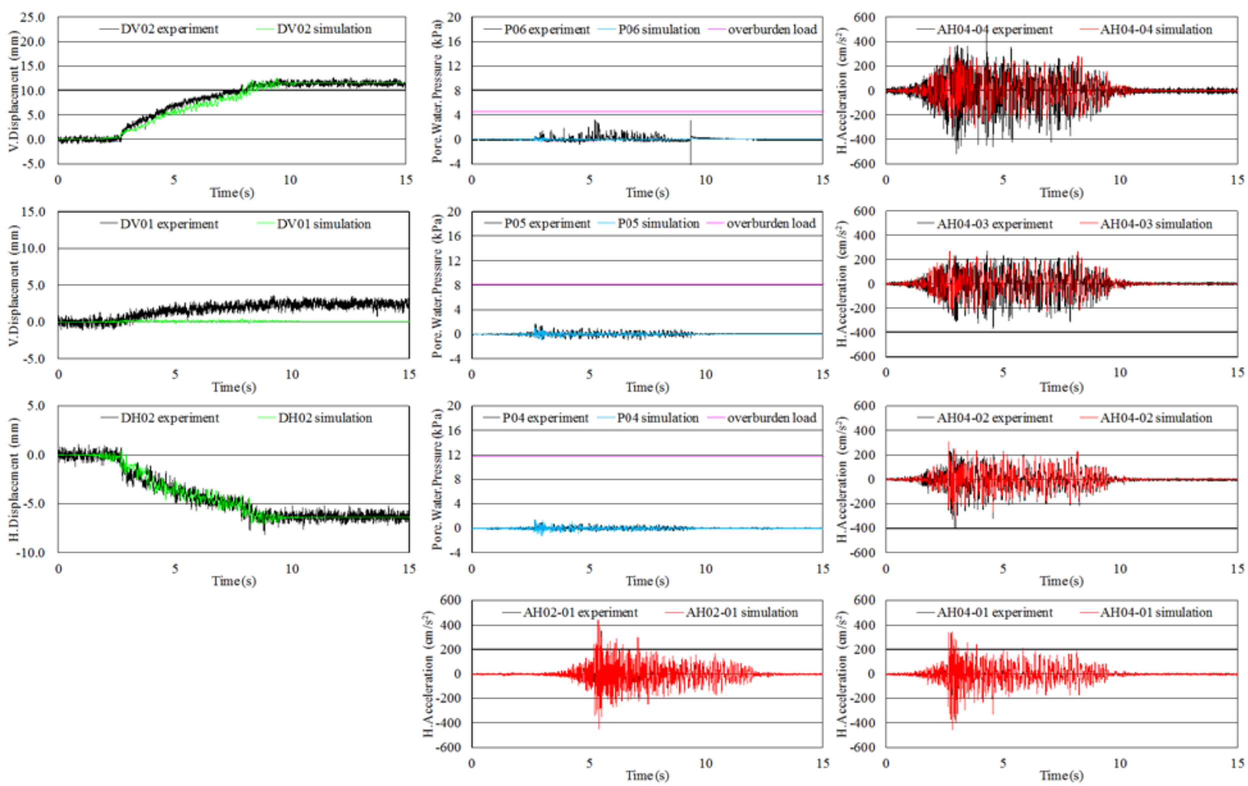


Figure 10 － Time history of acceleration, excess pore water pressure, horizontal and vertical displacements during caisson model vibration test and effective stress analysis

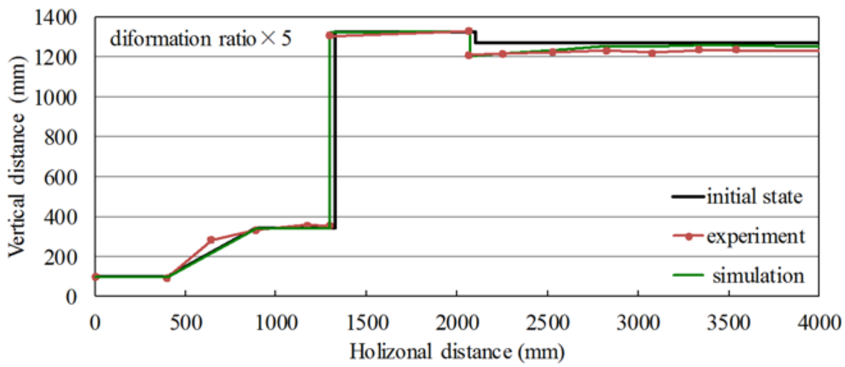


Figure 11 — Residual displacements between test and analysis result

4. Conclusion

The laminar shear box vibration test and the large-sale caisson model vibration test were performed to evaluate the volumetric shrinkage characteristics of rock debris and to verify the influence of rock debris to the overall behavior of the quay wall during an earthquake. Also, based on the results of these tests, numerical analysis by two-dimensional effective stress analysis code FLIP was performed.

The following is the knowledge acquired from the studies above.

- In the laminar shear box vibration test, the volumetric strain decreased as the relative density increased. For the rock debris used, it was verified that volumetric strain is barely caused even by vibration from actual size seismic waves which has larger amplitude than similarity wave, when the relative density reaches 70%.

- With the vibration test by a large-scale caisson model, it was verified that when rock debris material was used behind the structure, the subsidence of rock debris during an earthquake differs depending on its density. Also, the influence of rock debris density to the horizontal displacement of the caisson was verified to be small. Comparison of the test results with the actual situation in the reports on ground damage at the actual site shows that the displacement of low density ground obtained in the test was close to the actual one; the test values obtained from the low density ground converted based on the actual scale were 0.57m for horizontal displacement of the caisson and 1.25m for the level difference between the caisson and the backfill ground respectively.

- In the two-dimensional effective stress analysis code FLIP, rock debris was modeled with cocktail glass model and its parameters with good reproducibility were determined on the basis of laminar shear box vibration test. Using these parameters, analysis for large-scale caisson model vibration test was performed, in which especially the horizontal displacement of front side of the quay wall and the subsidence of rock debris ground behind the caisson resulted to be consistent.

5. Acknowledgements:

For this treatise, we acquired support from Yokohama Port and Airport Technology Investigation Office of Kanto Regional Development Bureau, the Ministry of Land, Infrastructure, Transport and Tourism for the laminar shear box test and the large model test. Also, we acquired huge cooperation from Dr. Makoto Yoshida from Penta-Ocean Construction Co., Ltd. We hereby express our gratitude.

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