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Centrifuge Model Tests and Finite Element Analyses on Seismic Behavior of Quay Walls Backfilled with Cement-Treated Granular Soils

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ABSTRACT: Dynamic centrifuge model tests and finite element analyses (FEA) were conducted to investigate the seismic behavior of quay walls backfilled with cement-treated granular soils (CTGS). In particular, the effects of the CTGS fill depth and fill range on seismic behavior were investigated. The centrifuge model tests showed that no liquefaction was generated in the CTGS backfills. The quay wall's horizontal displacement induced by the seismic loading decreased with increases in the CTGS fill depth because the earth pressure acting on the quay wall was reduced. In addition, a wedge-shaped CTGS fill was found to be effective at reducing the horizontal displacement of the quay wall. However, the results of the dynamic FEA indicate that the wedge angle of the CTGS fill should be carefully designed.

Keywords: Ground improvement, Cement-treated granular soil, Centrifuge model test, Finite element analyses

1. INTRODUCTION

Attempts have been made recently to reuse soft soils obtained from dredging conducted near ports and harbors. The methods developed so far include dehydration, segregation, and cement-treated techniques. Disadvantages as well as advantages have been found for each method. For example, dehydration and segregation methods usually take a long time to stabilize a large amount of dredged soils. Cement-treated methods take a shorter time compared to dehydration and segregation methods. However, cement-treated materials show brittle behavior; cracks are generated when the foundation layers show differential settlement owing to consolidation.

The focus of this study is on a granulating technique, which is used for overcoming these difficulties. The method converts the dredged soils with their high water content to granular materials by adding cement and polymer. To date, the authors' research group has conducted fundamental studies on the geotechnical characteristics of cement-treated granular soils (CTGS) [1], [2]. These studies established the mixture design and the production method to obtain CTGS at low cost. Based on these findings, Dong et al. (2011) performed a series of laboratory tests to investigate the physical and mechanical properties of CTGS[3]. It was concluded that CTGS is a lightweight material because the particles include many voids. Therefore, it can be expected that CTGS would reduce the earth pressure acting on retaining walls if used as backfill soils. However, the applicability is not fully understood at present. In particular, it is important to clarify the effects of the CTGS fill depth and fill range on the seismic behavior of a quay wall.

Therefore, in this study, a series of dynamic centrifuge model tests and finite element analyses (FEA) were conducted to investigate the seismic behavior of quay walls backfilled with CTGS.

Figure 1 Cement-treated granular soils (CTGS) prepared for centrifuge model tests

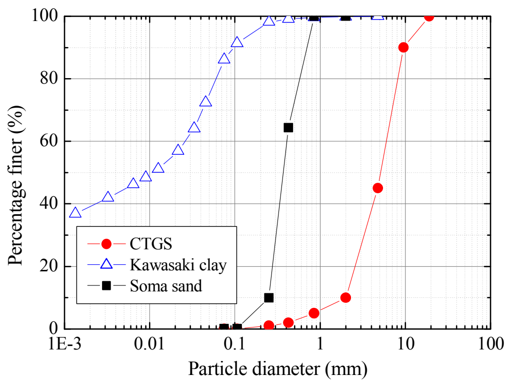


Figure 2 Particle size distributions of CTGS, Kawasaki clay, and Soma sand

2. CENTRIFUGE MODEL TESTING METHOD

2.1 Materials

CTGS were produced in the following manner for centrifuge model tests: First, the water content of a dredged soil called Kawasaki clay was adjusted to 60%. Then, the soil was mixed with a small amount of polymer with a weight of 0.1% of the clay. Owing to the effect of the polymer, the plasticity of the mixture became lower. After the plasticity change was observed, Ordinary Portland Cement was added and the mixture was stirred for about 10 min. Granular particles (shown in Fig. 1) gradually appeared because of the effect of cementation. The weight of the cement added to the mixture was 5% of the clay. Finally, the granular particles thus obtained were cured for more than 28 days.

Figure 2 shows the particle size distributions of the CTGS and Kawasaki clay, as well as that of Soma sand, which was also prepared for the centrifuge model tests. It is clear that Kawasaki clay was converted to a gravel-type soil (CTGS) by the granulation process.

2.2 Model preparation

Four models (denoted by Cases 1 through 4) were prepared to physically simulate a quay wall, as shown in Fig. 3. The quay wall consisted of a foundation layer, a caisson, and backfill soils. As seen in the figures, CTGS and Soma sand were used as the backfill soils. The foundation layer of each model was made with Soma sand of 90% relative density and had a 2.5-m thickness at prototype scale.

The sandbox used for Cases 1 and 2 had dimensions of 410 mm (height) by 556 mm (width) by 200 mm (length), whereas that used for Cases 3 and 4 had dimensions of 512 mm (height) by 710 mm (width) by 200 mm (length) at model scale.

As seen in Figs. 3(a) and (b), the caisson in Cases 1 and 2 was backfilled with two horizontally layered soils. In Case 1, the overlying layer was filled with CTGS that was 1.8 m thick (20% of the total thickness of the backfilled ground) at prototype scale. The underlying layer was Soma sand that was 7.2 m thick. Case 2 was similar, but the overlying layer was 6.75 m thick (75% of the total thickness of the backfilled ground) and the underlying layer was 2.25m thick.

In Cases 3 and 4, the CTGS was placed in a wedge shape with one edge of the wedge adjacent to the caisson. In Case 3, the right angle of the wedge was at the upper left corner of the backfill, as shown in Fig. 3(c). The remaining part outside the wedge was filled with Soma sand. In contrast, the right angle of the wedge in Case 4 was in the lower left corner of the backfill, as shown in Fig. 3(d). The remainder was filled with Soma sand.

In all cases, first, Soma sand was poured using the air pluviation method, resulting in a foundation layer of relative density of 90%. Next, in Cases 1, 2 and 3, Soma sand was filled using the air pluviation method to become about 50 % relative density. Then, a small shovel was used to place CTGS in the sandboxes at zero height, such that a loose state was achieved. In Case 4, CTGS was filled, followed by the fill of Soma sand of 50% relative density.

In each case, two displacement transducers (denoted by D1 and D2 in the figures) were set at the seaward side of the caisson to measure its horizontal displacement. The other two displacement transducers (denoted by DT1 and DT2) were set above the backfill soils to measure the ground surface settlement. Pore water pressure gauges and accelerometers were installed inside the backfill soils at prefixed positions, as shown in the figures.

To evaluate the earth pressure distribution acting on the caisson, four load cells (denoted by LC1 through LC4) were built in the caisson. A rigid rectangular plate that directly contacted the backfill soils was connected to each load cell.

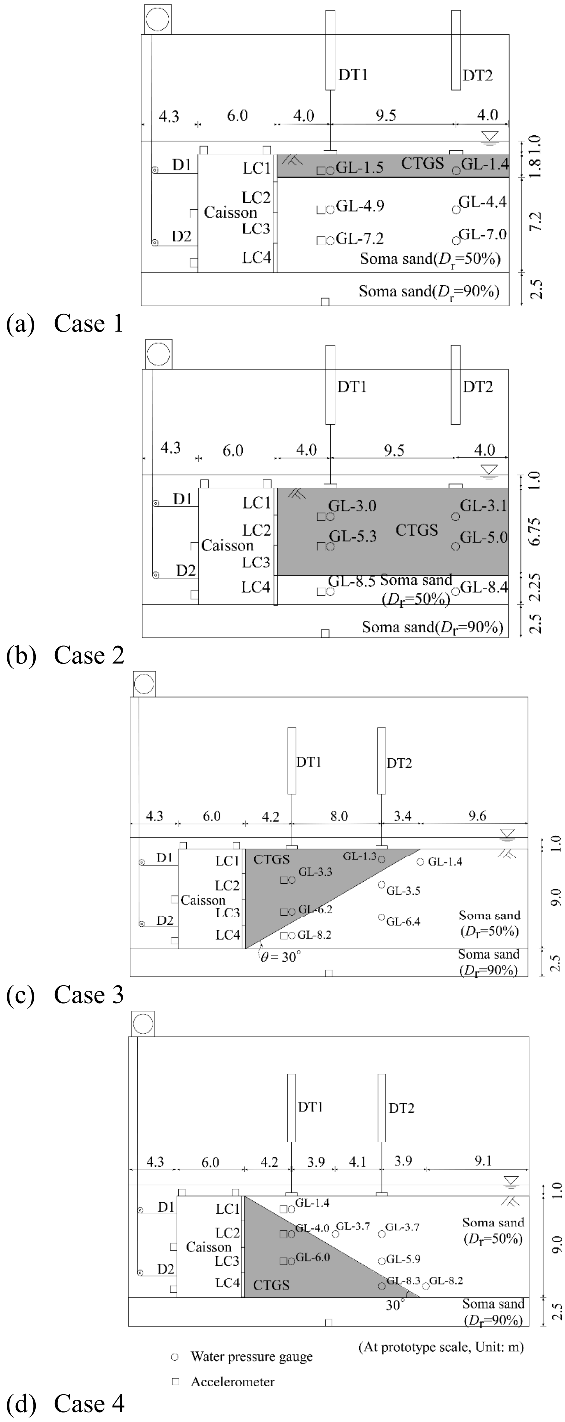


Figure 3 Schematic images of models prepared for centrifuge model tests

The plate surface was set to be smooth in this study. A small space was allowed between the plates, so that they would not interfere with each other.

In preparing the models, efforts were made to minimize the friction between the inside of the box and the model soils by inserting thin, smooth films.

2.3 Centrifugal acceleration and dynamic shaking

A geotechnical centrifuge facility known as Mark II at the Port and Airport Research Institute was used in this study. The beam radius of the centrifuge is about 3.8 m and the maximum loading mass is 2.76 ton. Detailed information can be found in Kitazume and Miyajima (1995) [4].

Centrifugal acceleration and dynamic shaking were conducted in the following manner: First, after each model was prepared, it was set on the shaking table installed on the centrifuge platform. The model was then accelerated up to 20 G by the centrifuge. At the 20-G acceleration field, viscous fluid was infiltrated into the ground from the bottom to saturate the soils. The high saturation degree of the soils was obtained as proposed by Okamura and Kitayama (2008) [5]. The fluid's viscosity was set to be 50 times that of water to satisfy the similitude of the pore water dissipation of the ground. Then, the centrifugal acceleration was increased up to 50 G, and dynamic shaking was applied six or seven times. In each shaking period, 20 cycles of the sinusoidal waves of 2-Hz frequency at prototype scale were applied. The maximum amplitude of acceleration applied to the ground was increased stepwise up to about 400 Gals.

close to that of LS, despite the fact that the upper part (20% of the total thickness) of the backfill ground consisted of CTGS. On the other hand, the caisson did not show a large horizontal displacement in Case 2. The horizontal displacement was close to that of DS. The caisson showed the smallest displacement in the CT case. These facts indicate that the caisson's horizontal displacement was reduced as the fill depth of the CTGS increased.

Looking at the results of Cases 3 and 4 in Fig. 4, the caissons show a horizontal displacement close to that of Case 2. This fact suggests that CTGS filled in the shape of a wedge or a reversed wedge can be effective at reducing the caisson's horizontal movement. The total CTGS volume required for Case 3 or 4 was lower than that required for Case 2.

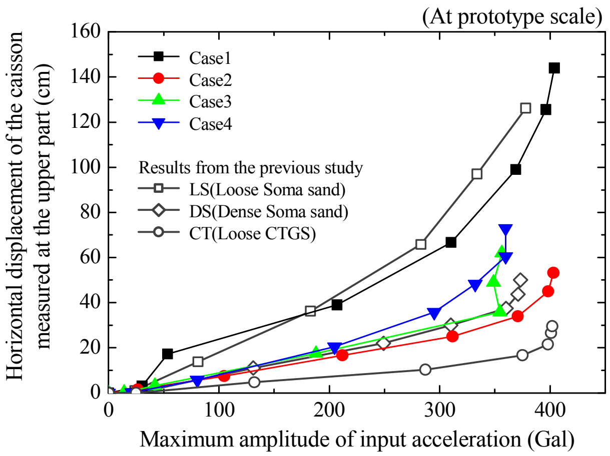


Figure 4 Relationship of input acceleration and caisson horizontal displacement

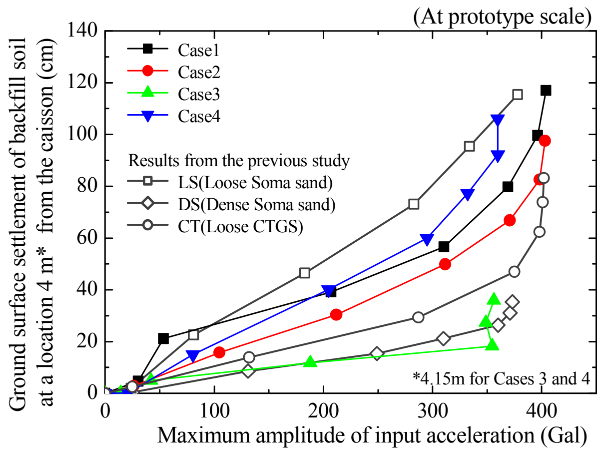


Figure 5 Relationship of input acceleration and backfill surface settlement

3. CENTRIFUGE TEST RESULTS

3.1 Horizontal displacement of the caisson and the ground surface settlement

Figure 4 shows the relationship between the maximum amplitude of the input acceleration and the cumulative horizontal displacement of the caisson for each case. The horizontal displacement was measured at the upper part of the seaward side of the caisson (see Fig. 3). The relationships obtained from the previous study [6], [7] were also shown in Fig. 4.

The previous study included three centrifuge model tests, denoted here by loose sand (LS), dense sand (DS) and CTGS (CT). The LS model was backfilled solely by Soma sand of 50% relative density, while the DS model was backfilled only by Soma sand of 95% relative density. The CT model was backfilled solely by loose CTGS. The caisson used in the LS, DS, and CT tests was the same as that used in this study.

Figure 4 shows that LS had the largest horizontal displacement at the higher accelerations. This was because the Soma sand was liquefied, and the earth pressure acting on the caisson increased as compared to other models. This will be discussed in detail in sections 3.3 and 3.4.

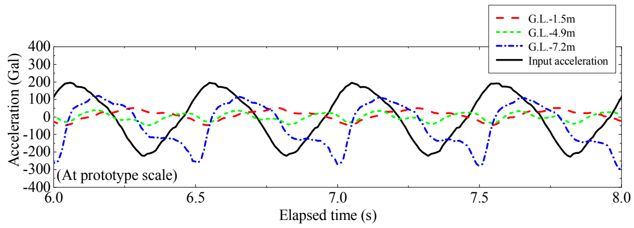
The caisson's horizontal displacement in Case 1 was

Figure 5 shows the relationships between the maximum amplitude of the input acceleration and the cumulative settlement of the ground surface. The surface settlement was measured at locations 4.0 or 4.2 m from the caisson edge (see Fig. 3). As shown in the figure, LS has the largest ground surface settlement, owing to the liquefaction of the Soma sand.

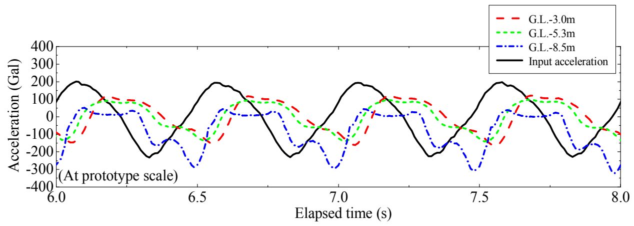
Wedge-shaped CTGS fills were a common characteristic between Cases 3 and 4; however, Case 3 shows a notably smaller ground surface settlement than Case 4. The ground surface settlement of Case 3 was close to that of DS. The reason why a small ground surface settlement was observed in Case 3 will be discussed in section 4.3.

3.2 Ground response acceleration

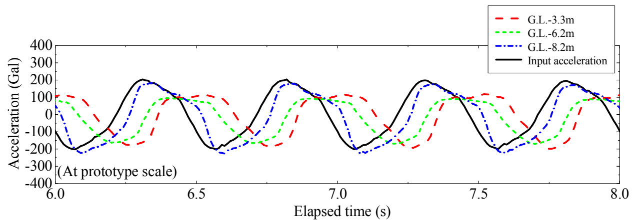
Figure 6 shows the time histories of the accelerations recorded in the backfill soil in Cases 1, 2, and 3. The data presented here were obtained in each case during the 6–8 s that elapsed after the start of the third shaking period. The ground acceleration was measured at locations 4.0 or 4.2 m from the caisson. Acceleration time histories were not obtained in Case 4 owing to connection errors with the accelerometers. The ground accelerations were attenuated in each model as seen in the figures. In particular, the maximum amplitude of accelerations obtained at ground depths of 1.5 and 4.9 m in Case 1 were smaller than those obtained in Cases 2 and 3.



(a) Case 1



(b) Case 2



(c) Case 3

Figure 6 Time histories of ground response and input accelerations

It is believed that the stiffness of Soma sand was considerably reduced by liquefaction and that the shear stress transmitted in the ground was attenuated in Case 1.

Phase delays were also observed in the ground acceleration waves. The wave phase differences between the acceleration obtained at a depth of 1.5 or 4.9 m and the input acceleration were about 0.2–0.3 s in both Cases 2 and 3.

3.3 Excessive pore water pressure

The ratio of the maximum excessive pore water pressure to the effective overburden pressure $\Delta u/\sigma_v'$ was evaluated based on measurements made with the pore water pressure gauges. Figure 7 shows the relationships between $\Delta u/\sigma_v'$ and the ground depths for Cases 1–4. The excessive pore water pressure was measured at locations 12.2 or 13.5 m from the caisson. The data presented here were obtained at the third, fourth, and fifth shaking periods (periods 3–5) in each case. The calculated effective overburden pressure was also shown in the figures.

As shown in Fig. 7(a), the values of $\Delta u/\sigma_v'$ obtained at ground depths of 4.4 and 7.0 m were close to 1.0, which indicated the liquefaction of loose Soma sand in Case 1. In addition, the values of $\Delta u/\sigma_v'$ obtained at the Soma sand layers in Cases 3 and 4 were close to 1.0, as seen in Figs. 7(c) and (d). This was also because of liquefaction.

On the other hand, $\Delta u/\sigma_v'$ obtained at a ground depth of 1.4 m in Case 1 was about 0.3. Small $\Delta u/\sigma_v'$ values were also observed at ground depths of 3.1 and 5.0 m in Case 2, 1.3 m in Case 3, and 8.3 m in Case 4. These values were all obtained with the pore water pressure gauges installed in the CTGS. These findings verified that liquefaction was not observed in CTGS, even though the CTGS fill was loose and at shallow ground depths. The low liquefaction potential of CTGS can be explained by the high permeability characteristic of CTGS, which is suggested by the grain-size distribution shown in Fig. 2.

3.4 Dynamic earth pressure distribution

Figure 8 shows the dynamic earth pressure distribution acting on the caisson in each case. The dynamic earth pressure presented here was the minimum–maximum range obtained during period 3. The dynamic earth pressure was calculated by subtracting hydrostatic pressures from the total earth pressures measured with the load cells.

The figures show that the dynamic earth pressure at ground depths of 3.2, 5.3, and 7.4 m in Case 1 was higher than those obtained in the other cases. Similarly, a higher dynamic earth pressure was observed at a ground depth of 7.4 m in Case 2. These higher pressures were induced by the effect of the liquefaction of the Soma sand.

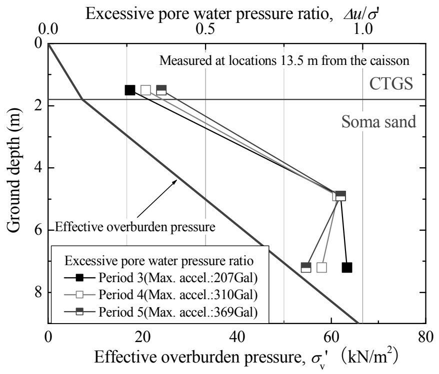
On the other hand, the dynamic earth pressure obtained at a ground depth of 1.1 m in Case 1 and at ground depths of 1.1, 3.2, and 5.3 m in Case 2 were not so high. In addition to the fact that no liquefaction was generated in CTGS, it is believed that the CTGS layers in Cases 1 and 2 induced a small dynamic earth pressure because of their light weight. This light weight characteristic is attributable to the fact that many small voids were included in the particles. Please see Dong et al. (2011) [3] for further information related to this issue.

The dynamic earth pressures obtained at ground depths of 3.2 and 5.3 m in Cases 3 and 4 were slightly higher than those in Case 2. This was probably induced by the liquefaction of the Soma sands filled behind the CTGS. This suggested that the effect of liquefaction of Soma sand filled behind the CTGS might be higher if the fill zone range of a wedge of CTGS became smaller. This matter will be investigated in detail in the following sections.

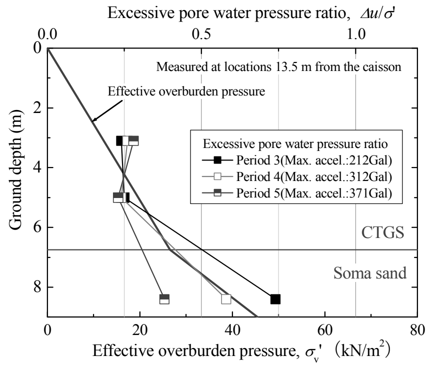
4 FINITE ELEMENT ANALYSES

4.1 Software and numerical analysis conditions

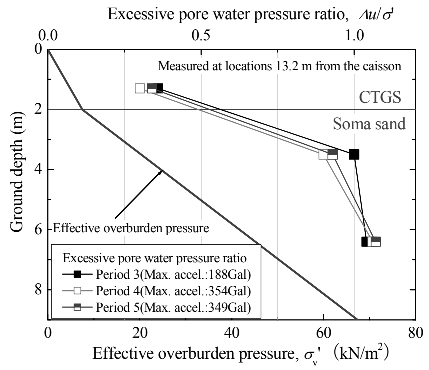
The centrifuge model test results indicated that the wedge-shaped fill of CTGS in Case 3 was effective at reducing the caisson's horizontal displacement and the backfill ground surface settlement when the quay wall was subjected to the dynamic loading. However, owing to the limited conditions of the centrifuge model tests, the appropriate range of the wedge-shaped zone was not well understood. Therefore, dynamic FEA were conducted to obtain the effect of the wedge-shaped zone range (in particular, the effect of the wedge angle) on the caisson's horizontal displacement. Dynamic FEA software named FLIP was used in this study. The software is widely used for seismic designs of port and harbor structures in Japan (e.g., Iai and Kameoka, 1993[8], Iai et al., 1998[9]). First, the analyses using FLIP were focused on simulating the effect of the fill depth of CTGS on the caisson's horizontal displacement.



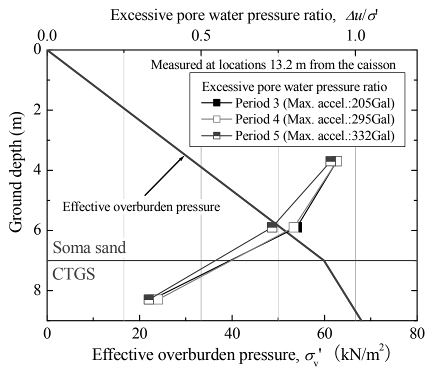
(a) Case 1



(b) Case 2



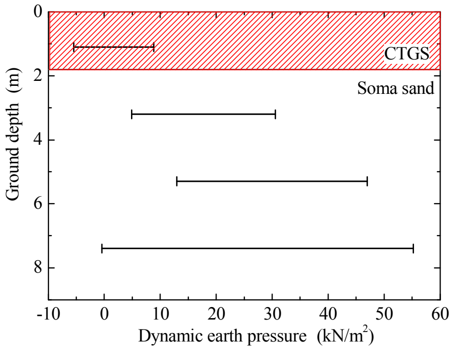
(c) Case 3



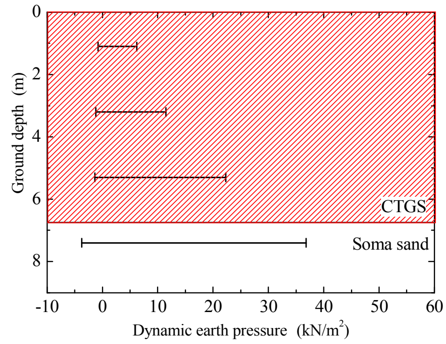
(d) Case 4

Figure 7 Depth distributions of excessive pore water pressure ratio and effective overburden pressure

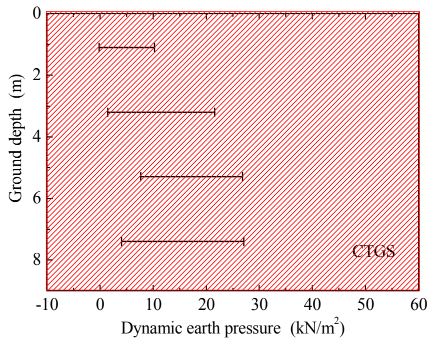
The centrifuge models (Cases 1 and 2 in this study and LS and CT in the previous study) were simulated. An analysis was also conducted to simulate the model in which the upper half of the backfill was CTGS.



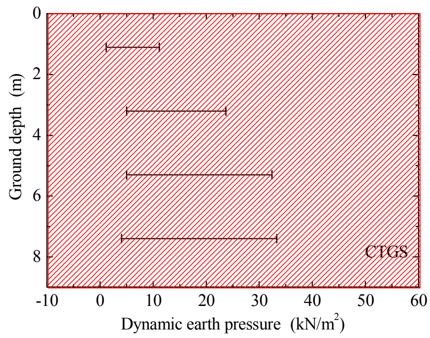
(a) Case 1



(b) Case 2



(c) Case 3



(d) Case 4

Figure 8 Depth distributions of dynamic earth pressure distributions at caisson wall

As an example, the finite element meshes used to simulate Case 1 are shown in Fig. 9. The scales of the foundation layer, the caisson, and the backfill layers in the figure were set to be the same as those of Case 1 at the prototype scale (see Fig. 3). For the dynamic acceleration, sinusoidal waves that were the same as those of the centrifuge model tests were introduced into the FEA. The maximum amplitude of the acceleration was adjusted to

200 Gal.

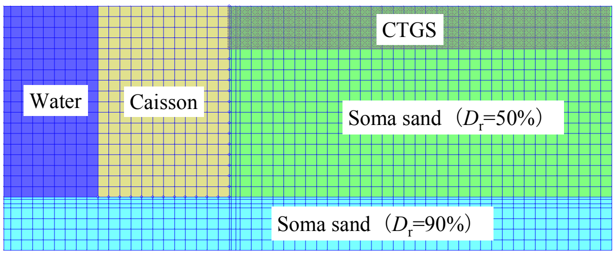


Figure 9 Finite element meshes adopted to simulate Case 1 in dynamic FEA

Table 1 Strength parameters used in dynamic FEA based on CD triaxial tests

Material	Submerged unit weight, γ' (kN/m^3)	c_d (kN/m^2)	ϕ_d (kN/m^2)
Soma sand (D_r = 50%)	8.5	0.6	36.1
Soma sand (D_r = 90%)	9.5	8.6	40.4
CTGS	4.3	0.0	26.6

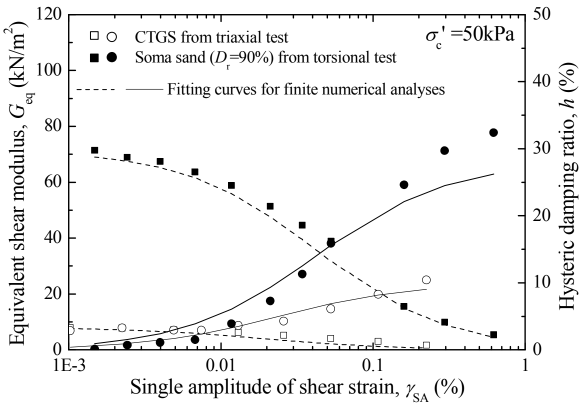


Figure 10 Comparison of laboratory and FEA results for equivalent shear modulus and hysteric damping ratio as a function of shear strain

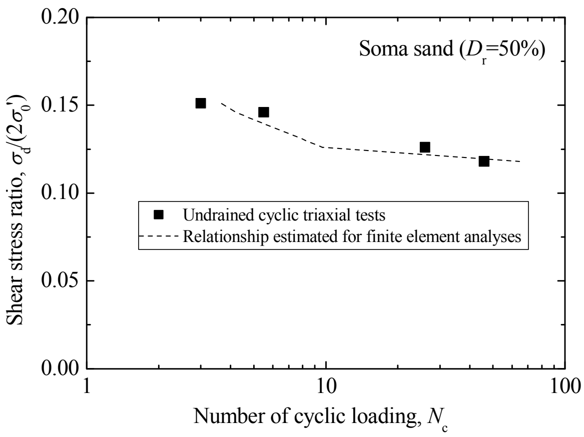


Figure 11 Comparison of liquefaction resistance curves obtained from laboratory tests with those estimated based on FEA

The material parameters related to the strength properties of CTGS and Soma sands were determined based on the consolidated drained triaxial tests are listed in Table 1, and those related to the deformation properties were determined based on cyclic triaxial tests or cyclic torsional tests, as shown in Fig 10. Laboratory experiments were conducted according to JGS 0524-2009, JGS 0542-2009 and JGS 0543-2009.

The liquefaction parameters necessary for simulating the liquefaction behavior of Soma sand were evaluated based on the cyclic undrained triaxial tests (JGS 0541-2009), as shown in Fig. 11. The CTGS were treated as nonliquefiable soils based on the centrifuge model test results. Detailed information on the input parameters required for FLIP and the constitutive material models can be found in Morita et al. (1997) [10].

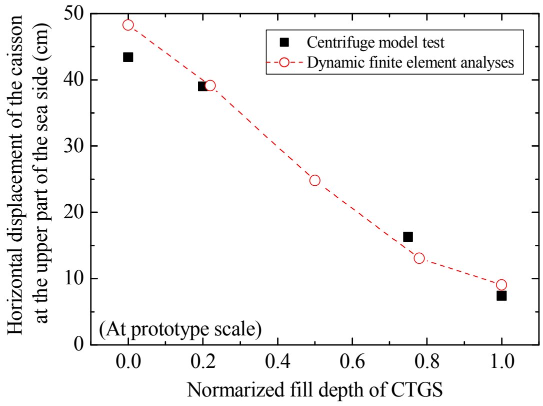


Figure 12 Comparison of centrifuge model tests and FEA results for caisson horizontal displacement as a function of fill depth

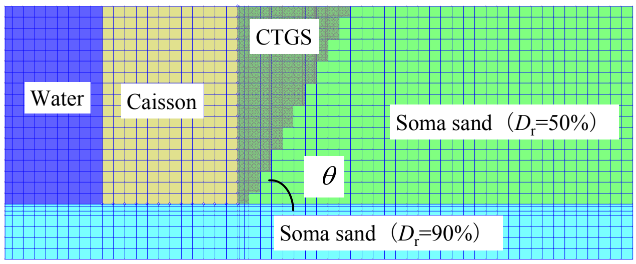


Figure 13 Example of finite element meshes adopted to represent wedge-shaped CTGS fill (θ = 60°)

4.2 Numerical analysis results on the effect of the filling depth of CTGS

Figure 12 shows the relationship between the fill depth of CTGS and the horizontal displacement of the caisson. The fill depths of CTGS were normalized by the total thickness of the backfill soil. The horizontal displacements of the caisson were evaluated at the upper part of the seaward side of the caisson, the same as for the centrifuge model tests (see Fig. 3). For comparison, the centrifuge model test results are also shown in the figure. The figure shows that the FEA results in horizontal displacements that are close to the centrifuge model test results. These results suggest that the horizontal displacement of the caisson was reduced proportionally to the normalized fill depth.

4.3 Numerical analysis results on the effect of the range of the wedge-shaped fill zone

As described in the previous section, the horizontal displacement of the caisson could be reasonably predicted by the finite element analyses by introducing the appropriate parameters into the numerical models. Then, using the same parameters and the constitutive models, analyses were conducted to investigate the effect of range of the wedge-shaped CTGS fill zone (in particular, the effect of the wedge angle) on the caisson's horizontal displacement. The wedge angle θ was defined as the inclination angle of the wedge plane from the horizontal plane. Typically, as shown in Fig. 13, the wedge angle θ was set to 0°, 30°, 45°, 60°, and 90° in each analysis. The scales except for the wedge angle were designed to be as same as those of the centrifuge model tests.

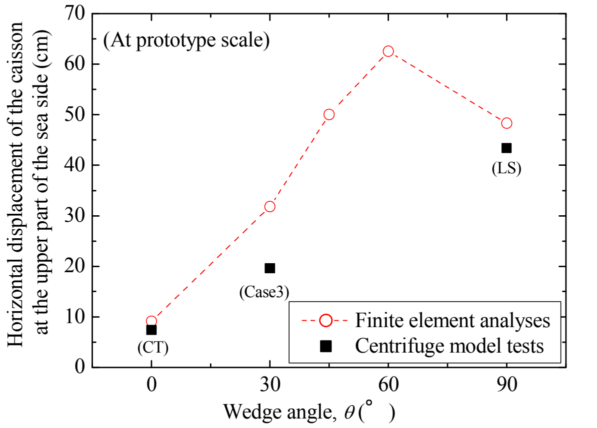
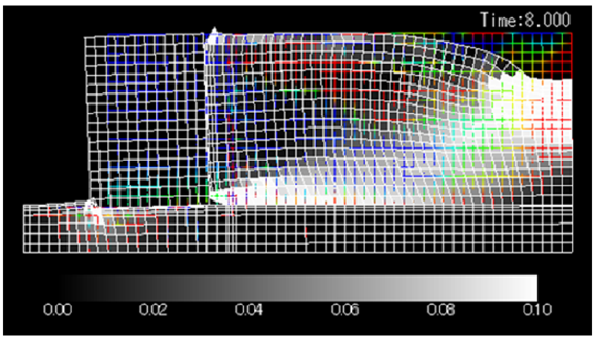
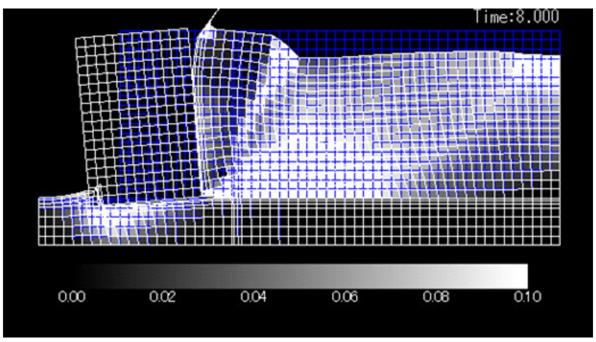


Figure 14 Comparison of centrifuge model tests and FEA results for wedge-angled CTGS fill and caisson horizontal displacement



(a) $\theta = 30°$



(b) $\theta = 60°$

Figure 15 Maximum shear strain distribution obtained from dynamic FEA

Figure 14 shows the relationship between the wedge angle θ and the horizontal displacement of the caisson. The centrifuge model test results were also shown in the figure. Here, the wedge with the 0° angle represented the backfill ground solely filled by CTGS (CT) and that of the 90° angle represented the backfill ground solely filled by Soma sand (LS). It can be seen in the figure that the wedge with a 60° angle showed the largest horizontal displacement.

Figure 15 shows the maximum shear strain distributions of the grounds for the wedges with 30° and 60° angles. The results indicate that higher shear strain was concentrated near the boundary between CTGS and Soma sand in the 60° wedge case compared to that in the 30° wedge case. This may suggest that the range of the wedge-shaped CTGS fill would be better designed to cover the active failure zone of backfill ground.

In addition, Figure 15 shows that Soma sand moved in the horizontal direction toward the caisson after it was liquefied. This sand appeared to push the CTGS up toward the ground surface during the movement because the density of the liquefied soil was much higher than that of CTGS. This mechanism can explain the reason why a small amount of ground surface settlement was observed in Case 3 but not in Case 4 in the centrifuge model tests.

5 CONCLUSIONS

Dynamic centrifuge model tests and finite element analyses (FEA) were conducted to investigate the seismic behavior of quay walls backfilled with cement-treated granular soils (CTGS). Effects of the fill depth and fill range of CTGS on the seismic behavior were investigated. The following results were obtained:

1) The centrifuge model tests showed that no liquefaction was generated in the CTGS backfills. The low liquefaction potential of CTGS was due to the highly permeable characteristic of CTGS resulting from the coarse grain-size distribution.

2) The quay wall horizontal displacement induced by the seismic loading was decreased as the CTGS fill depth increased because the earth pressure acting on the quay wall was reduced. The small earth pressure was attributed to the light weight characteristic of the CTGS.

3) The centrifuge model tests also showed that the wedge-shaped CTGS fill was effective at reducing the quay wall's horizontal displacement and the ground surface settlement.

4) The dynamic FEA using FLIP could reasonably give the caisson's horizontal displacements, which were close to those of the centrifuge model tests. The FEA also indicated that the wedge angle of the CTGS fill zone should be carefully designed to cover the active failure zone of the backfill ground.

5) The dynamic FEA for the wedge-shaped fill also showed that liquefied Soma sand filled behind the CTGS and pushed the CTGS up toward the ground surface. This contributes to the fact that a small ground surface settlement was observed for the wedge-shaped CTGS fill in the centrifuge model tests.

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