SHAKING TABLE TESTS WITH SHORT UNDERGROUND WALLS ON REDUCING SETTLEMENT OF SPREAD FOUNDATION IN LIQUEFIED SOIL

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SUMMARY

Shaking table test and FEM liquefaction analysis were conducted making use of the model installed short underground walls in order to develop a countermeasure for reducing settlement of a spread foundation in liquefied sand soil. From the results of the first half of the tests, the mechanism of decrease in settlements of the model with walls compared to those without walls was speculated, and the walls were improved in two points. One was to make the lower end of the walls thicker and the other was to surround the walls with gravel. Both methods were effective independently; besides, by using both in combination, the settlement of the model relative to that of the model ground was perfectly prevented within a certain weight of the model. As a result of experiments and analyses, it became clear that liquefaction did not occur near the gravel and the area surrounding the model foundation still had shear strength, to prevent settlement.

INTRODUCTION

Some kinds of countermeasures against liquefaction have been developed and their efficiency also has been improved through past earthquakes. However, we suggest a simpler countermeasure for reducing settlements, since they cannot be applied to small-scale personal structures because of their cost. In such cases, simpler measures that allow settlement within a certain allowable value or avoid uneven settlement may be suitable, while, little research in that field exists. The purpose of this study is to develop a simple countermeasure to control settlement of the foundation by relatively low cost. This paper deals with shaking table tests and FEM liquefaction analysis, using models with short underground walls as countermeasure for reducing settlement. Since behavior of the model with countermeasure in comparison with that without countermeasure was studied here, at the first stage, similarity to the real foundation and ground was not concerned at present.

FACTORS REDUCING SETTLEMENT WHEN WALLS ARE INSTALLED

In the shaking table tests, the spread foundation models were made of 14mm thickness wooden plates (E=120tf/cm^2=11.8×106kN/m^2). The base model section of the footing was 300mm×300mm, and four wooden plates were attached to the base as walls to surround the saturated sand soil under the base, with supplemental plates for reinforcement and setting the models. Average particle size D_{50} of sand was 0.4mm and uniformity coefficient Uc was 1.7. The model grounds were 60cm and 40cm depths, 60cm width and 1,800cm in length, with the water level at the ground surface. The average unit weight r_t was 1.86gf/cm^2. The details of other dimensions of every part are shown in Figure 1. Tests were conducted with an acceleration of 250cm/s^2 at a frequency of 5Hz, in the longitudinal direction of the soil container. The weight of the model was set to 21kgf(206N), involving the additional weight, which eliminates buoyancy force acting on the walls under the water surface. By those weight and acceleration, liquefaction at a sufficient distance from the foundation model was completely induced. Test results and discussion for improvement of the countermeasure are as follows.

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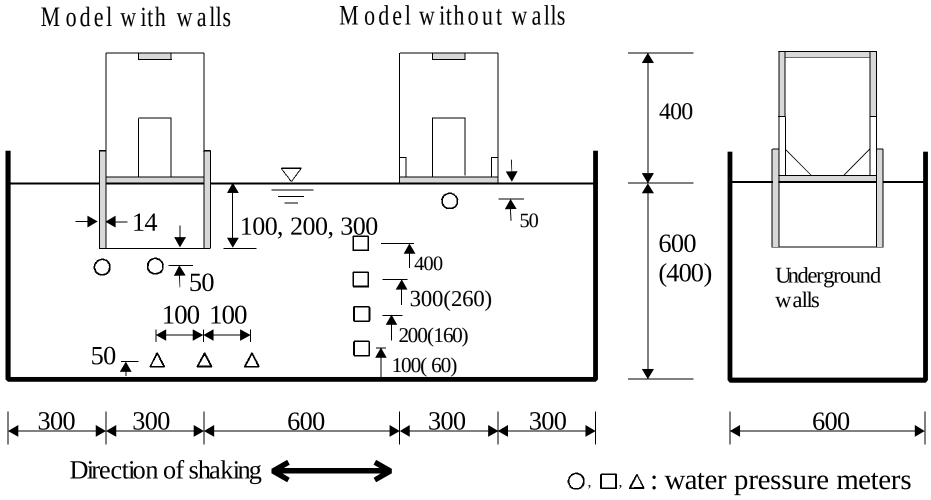


Figure 1: Experimental equipment to compare settlements of models (mm)

Verifying the effect of simple underground walls

Two models, of which settlements would be compared, were shaken at the same time to evaluate their relative settlements: one with walls and the other without walls; effect of a countermeasure was evaluated by Reduction Rate(RR) defined as;

$$RR = \frac{\delta_0 - \delta}{\delta_0} \quad (1)$$

in which

δ_0 is the settlement of the model without countermeasure;
δ is the settelment of the model with countermeasure.

These two amounts are relative values to subsidence of the model ground. The weight of the model decreases with increasing buoyancy force as the model sinks into the liquefied ground. However, the evaluation is conservative because the settlement of the model with countermeasure is less than that of the model without it and the model with countermeasure is in a disadvantage position in this respect.

The amounts of settlement of the models are listed in Table 1 and reduction rate (RR) is shown in Figure 2. It was natural that RR increased with longer walls because the bottom of the model with walls was closer to the bottom of the soil container. RR increased as the depth of ground decreased.

Table 1: Settlements (cm)

Depth (cm)		Ground	non walls	Length of walls (cm)		
				10	20	30
40	Max.	1.2	15.5	10.6	5.0	----
	Min.	0.6	11.6	7.1	3.5	----
	Mean	0.9	13.8	8.6	4.5	----
60	Max.	1.7	28.0	18.9	13.1	7.6
	Min.	0.6	22.1	17.9	11.0	5.6
	Mean	1.3	25.3	18.6	12.0	7.0

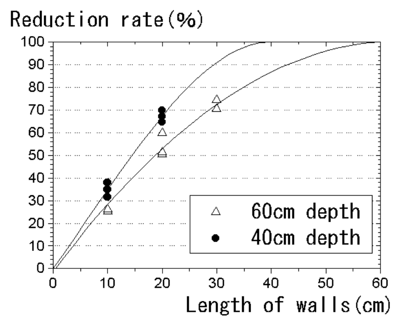


Figure 2: Reduction rate

Time histories of settlement, and liquefied ground flow

After videotaping the sinking model with an attached scales, the behavior of the model was read by every second. Liquefied ground flow was also observed through the acrylic side of the container, using colored sand arranged with 5cm separation along the side. Results are shown in the case of 60cm depth ground, in Figure 3 and Figure 4 respectively. The final values of settlements include those of the model ground, which were about 1.5cm. Figure 3 shows that models sank fast at constant speeds immediately after shaking. Initial velocities of settlement continued for about 6～13 seconds depending on the wall length, and they decreased with wall length. Similar tendency was shown in the case of 40cm depth. The same was result was reported in another research [Sasaki et, al., 1998]. It seems that the difference of initial velocity between models depends on the length of walls, because soil at the outer side of the wall marked by fell with the wall for a while in Figure 4, and stress acting along the surface of walls would controll settlement in early stage.

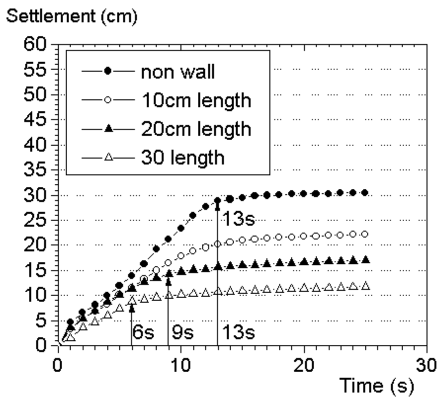


Figure 3: Time-Settlement curves

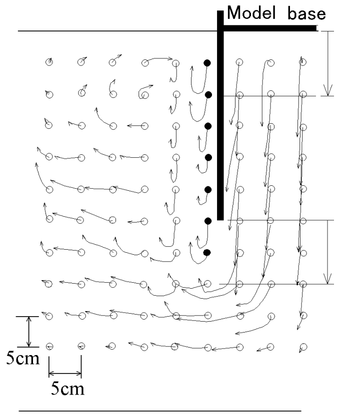


Figure 4: Ground flow
(60cm depth, 30cm length wall)

Excess pore water pressure and the area having shearing force

In the experiments for 20cm walls and 60cm depth ground, excess pore water pressures (EPWP) were measured at positions under the models shown by circles in Figure 1. Figure 5 shows a typical shape, which was obtained below the edge of the wall at the center of the model. Since it took peak when sinking was close to an end (9s in Fig. 5), it was suggested that the area under the model was not liquefied and the area was hard to flow during rapid sinking. While, at the stratum where sand particles have fallen in liquefied ground, the soil recovers bearing capacity and liquefaction is ending gradually from the bottom of the ground to its surface. From time histories of EPWP in Figure 6, at the position measured vertically at regular intervals (squares in Fig. 1), time histories of depths of recompressed strata could be approximated as fitting curves in Figure 7 [Kitada et, al., 1998]. The amount of settlement of the model depends on the length between the bottom of the model and the surface of the stratum S_2 in Figure 7. It could be said that slow settlement following rapid sinking was caused when the area having shear strength reached the stratum S_2 that had enough bearing capacity to support the model.

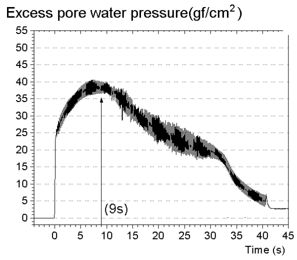


Figure 5: EPWP 5cm below the end of the wall at the center of the model.

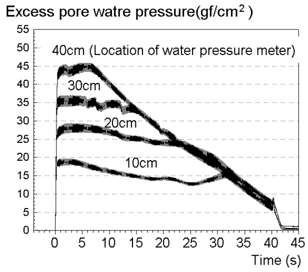


Figure 6: EPWP at four positions square marked in Figure 1.

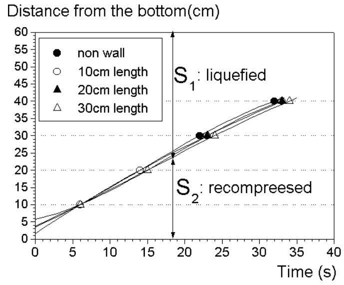


Figure 7: Boundary curve between the stratum liquefied and the stratum where liquefaction had been over.

IMPROVEMENT OF COUNTERME

On discussions mentioned above, the effective improvement independent by of depth of liquefied stratum, is to make initial velocity of sinking decrease. Two plans to reduce settlement further could be made. They were:
1) to increase vertically projected area of the model in order to increase the area subjected to upward pressure during settlement: for this purpose, the lower end of the wall was thickened outward.
2) to increase shearing force or viscosity acting along the outside of the wall: for this purpose, walls were surrounded by gravel in expectation of its effect on preventing liquefaction of the soil around the model [Saito, 1991]. It would be possible to substitute gravel for wall, if a part of the wall would be cut out and filled up with gravel. Previous tests were conducted by using the wall with various square holes in order to determine the optimum area to cut out. Results show that decrease in reduction rate was within 10% unless the whole length was cut off. When the whole length was cut off, the horizontal section of the lower end of the wall was not box-section, as a result, the area that was hard to flow under the wall might decrease substantially. Thus the length cut out was set as long as possible shown in Figure 8.

Outline of tests

In order to cope with various countermeasures, size of the model ground and the foundation model was reduced. The base plate section of the model was 200mm × 200mm, and total weight of the model was set to 9.3kgf(91N) nearly equal to the former model in weight per unit area. Maximum grain size of gravel was 10mm. Permeable coefficient of gravel k was 2.9cm/s and sand 0.14cm/s.
There were 11 combinations in tests as shown in Table 2. When gravel was used, excess pore water pressures were measured.

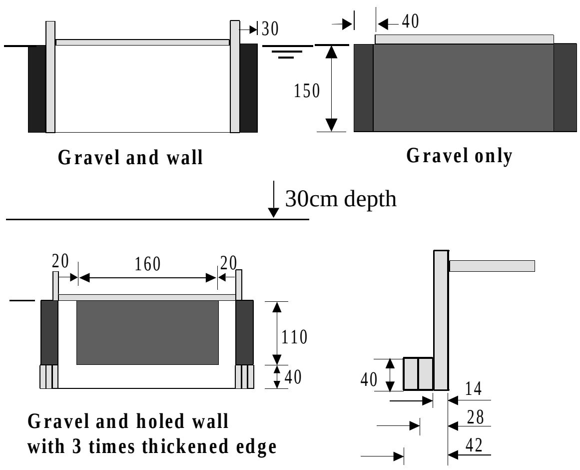


Figure 8: Improved countermeasures (mm)

Table 2: Combination of additional methods

Case	Normal wall	Holed wall	Thickness (times)	Gravel installed
W1	O		1	
W2	O		2	
W3	O		3	
H1		O	1	
H2		O	2	
H3		O	3	
WG1	O		1	O
WG3	O		3	O
HG1		O	1	O
HG3		O	3	O
G0	---	---	---	O

Results and Discussion

Figure 9 indicate reduction rate, RR defined in Equation (1), for improved method. As thickness of the edge increased, RR increased linearly. When gravel and wall were used together, RR reached almost 100%, and RR exceeded 100% in some cases, WG3 and HG3, because the surface of the ground settled a few millimeters below the surface of the gravel. In case only gravel was used (G0), the gravel was rather scattered under the model and the RR decreased, therefore, stability of the gravel depended on existence of the wall. When the weight was increased by 1.5 times, the model sank in the ground and RR decreased to about 90%. In this case, the shape of the surface of the model ground near the gravel was as shown in Figure 10 until initial velocity of settlement decreased. When the settlement began to decrease, surrounding soil at the ground surface flew in the depression. This phenomenon shows that the area around the gravel had shear strength.
Figure 11 indicates time histories of excess pore water pressure (EPWP) on the same level at 25cm depth as shown in Figure 1 (triangular marks), and Figure 12 is that on the horizontal positions 5cm away from the gravel at a depth of 10cm in the ground. It seemed that liquefaction almost occurred at the position sufficiently away under the model, while liquefaction did not occur at the distance 5cm from the gravel. If the area under the model was liquefied, the weight of the model would be transmitted to surrounding area, and this is why EPWP in Figure 11 exceeded initial effective stress. Therefore, it could be said that shear strength remained in the area that was surrounding and close to the gravel, and shearing force acted along the surface of the gravel. Since there was no slipping between the wall and the gravel, the shearing force was transmitted to walls to support the model.

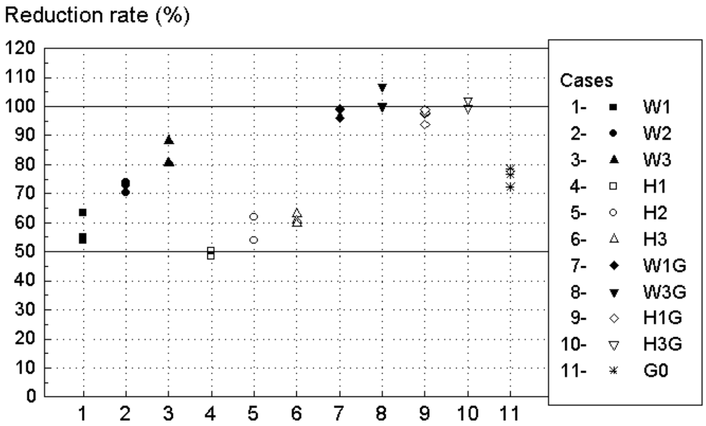


Figure 9: RR for improved countermeasure

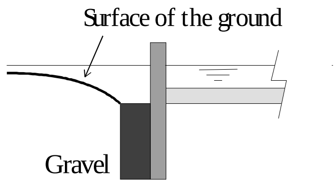


Figure 10: The shape of the surface of the ground while the model was sinking fast

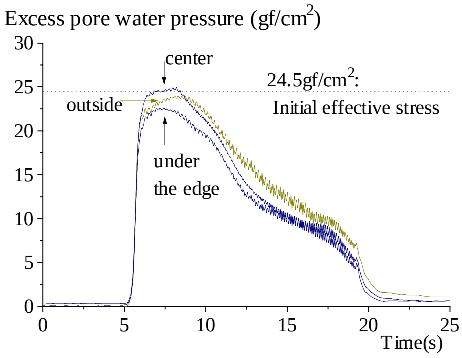


Figure 11: EPWP at 10cm depth 5cm away from the gravel

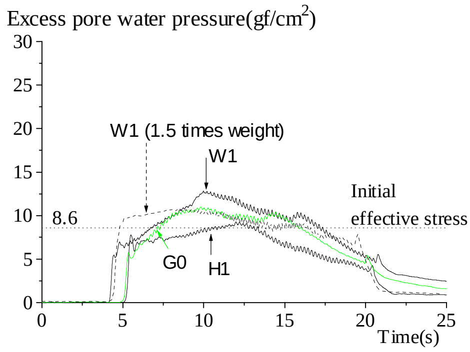


Figure 12: EPP at 25cm depth

2D-FEM ANALYSYS

For surveying the efficiency of gravel drain system and degree of liquefaction of whole model ground, 2D-FEM analysis was performed. In concerned with effective stress, though the analysis was plastic 2D method [Mori, H., 1992], tensile stress and shearing stress exceeding the sandy element's shear strength were re-distributed and the calculation was repeated until those stress came to allowable values. In liquefaction analysis, the coefficients listed in Table 3 were quoted from another study using same materials we used [Kondo, 1998]. Since liquefaction was induced within a second in the test, simulating time duration was set to 1.5 sec. with time step of 0.001. The applied acceleration is shown in Figure 13.

Figure 14 shows excess pore water pressure ratio (PPR) in three cases at 1.5 sec. Whiter the color is, less the PPR is and more liquefiable the element is, and Figure 15 indicates the results of liquefaction analysis at the

Table 3: Coefficients for liquefaction analysis

	Sand	Gravel	Model
Young's modulus (kgf/cm^2)	----	----	120,000
Poisson's ratio	0.33	0.33	0.4
Unit weight (gf/cm^2)	1.86	1.87	0.53
Damping factor	0.3	0.3	----
Coefficient of volume compressibility	0.02	0.001	----
Relative density (%)	32.5	80	----
Coefficient of permeability	0.0138	2.03	----
Initial Shear modulus (kgf/cm^2)	270 $(\sigma_0)^{0.5}$	807 $(\sigma_0)^{0.58}$	----
Standard strain	4.8^{-2} $(\sigma_0)^{0.5}$	5.1^{-2} $(\sigma_0)^{0.48}$	----

σ_0 : initial effective stress

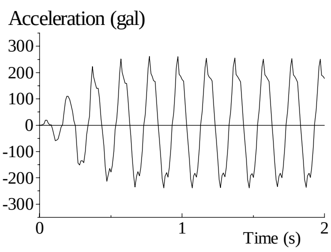
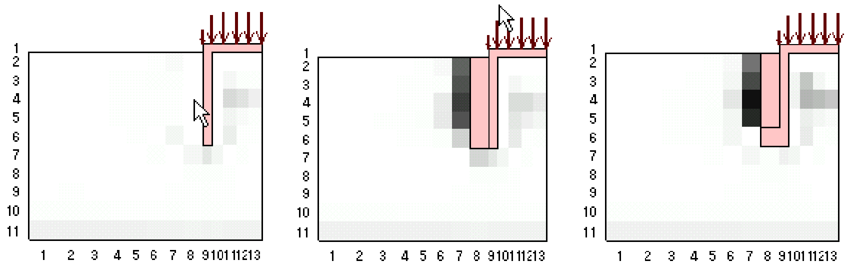


Figure 13: Acceleration applied to the test and the FEM analysis

element 5 (vertical axis)-6 (horizontal axis). When the gravel was installed, PPR at the outside of the model was restrained. From these figures, it is clear that pore water was drained through gravel and the area around the model has still shear strength.

Using the shear modulus reduced by the equation of Hardin-Drnevich model, the deflection of the three cases was obtained as shown in Figure 16. When gravel was installed, the deflection at the elements near the outside of the wall is small, while, without gravel, the deflection is large as if the settlement has occurred. In case the wall's edge was 3 times thickened, the deflection is smaller than those for other two, because shearing strain was smaller. Since the spread of non-liquefied area between three cases in Figure 14 is not so different, the boundary condition of whether settlement occurs or not would depend on shear strength around the model



a) without gravel

b) with gravel

c) with gravel and 3 times thickened edge

Figure 14: Distribution of PPR in three cases at 1.5 sec.

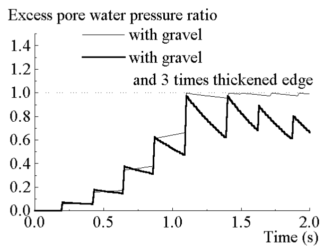
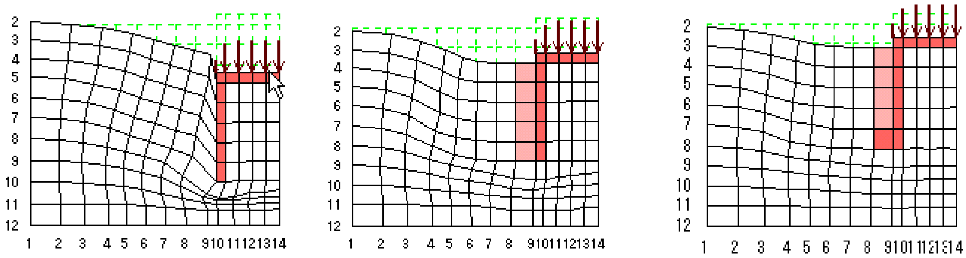


Figure 15: time history of PPR at the element 5(y)-6(x)



a) deflection for Figure 14-a)

b) deflection for Figure 14-b)

c) deflection for Figure 14-c)

Figure 16: Results of FEM analysis calculation

CONCLUSIONS

As a result of experiments and analyses, it became clear that liquefaction did not occur near the gravel, and the area that surrounded the foundation model still had shear strength to prevent settlement. Therefore, it was considered that the model did not sink until the weight of the model did not exceed the shear strength of that area. In case of exceeding the shear strength, the settlements depended on not only spread of non-liquefied area but also thickness of the end of walls

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