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ABSTRACT

In this study, centrifuge model tests and numerical analysis were carried out to investigate effects of improvement by deep mixing method (DMM). Model grounds used in model tests were (1) unimproved ground (case1), (2) improved ground with vertical soil columns under toe of slopes (case2), (3) improved ground with vertical soil columns under shoulder of slopes (case3), (4) improved ground with inclined soil columns under toe of slopes (case4), and (5) improved ground with inclined soil columns under shoulder of slopes (case5). The improvement ratio was about 10% for all test cases except case1. The embankment was constructed by sand dropped from the sand hopper under 56g centrifugal field.

Following conclusions were obtained from the model tests and the numerical analysis.

1) The deformation of ground was considerably controlled in spite of adopting the low improvement ratio of 10%.
2) For preventing the failure or the large deformation of ground, ground improvement just bellow the shoulder of the slope was more effective than that bellow toe of the slope.
3) Numerical analysis by means of FEM could be utilized to simulate the tendency of deformation behaviors of soft grounds.

INTRODUCTION

Failures or undesirable large deformations sometimes occur during construction of embankments. Recently, Deep Mixing Method (DMM) has been widely used as a ground improvement technique to prevent these large deformations. However, there still remains some problems to be resolved, for instance, the non-uniformity of the strength of the improved soil column, how to evaluate the strength of the composite ground and so on. Especially, in the case of adopting a low improvement ratio, it is considered that the strength of the composite ground would not be much higher than that of original ground, if the low improvement ratio below a significant value was adopted. In this study, centrifuge model tests and numerical analysis were carried out to investigate the effect of improvement by DMM at the low improvement ratio.

TEST CONFIGURATION

Centrifuge model tests were carried out in this study to clarify the effect of the location and the inclination of soil column improved by DMM at the low improvement ratio.

Geotechnical Centrifuge

Geotechnical centrifuge used in this experiments is shown in Fig. 1. Technical specifications of our centrifuge are listed in Table 1.

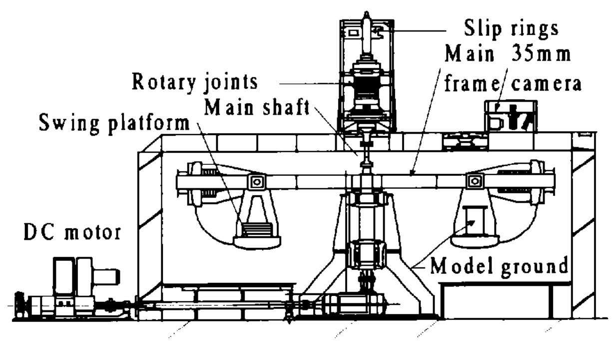

Fig.1. Geotechnical centrifuge.

Table 1. Technical specifications of the centrifuge.

Key item	Specification
Maximum acceleration	200 g
Maximum Payload	500 kgf (4.9kN)
Payload capacity	100 g-tons
Effective radius	2.1 m
Payload size	0.7 x 0.9 m

Test Models

Five kinds of model tests were carried out and are listed in Table.2. Improvement ratio used in this study is defined as the ratio of the total cross sectional area of the soil columns to the improved area. The improvement ratio was about 10% for all test cases except case1. Improved soil columns of case4 and case5 are inclined with the angle of 20 degrees from the vertical and clockwise direction. However, the level of column's top for these cases is same as case2, 3.

Table 2. List of test models.

	case1	case2	case3	case4	case5
Improved area (below of embankment)	-----	toe	shoulder	toe	shoulder
Improvement ratio(%)	0	10	10	10	10
Column's angle (degree)	-----	0	0	20	20

Sand Hopper

A sand hopper and a model ground are shown in Fig.2. Model embankment was constructed by pluviating the Toyoura sand from a sand hopper. This sand hopper equipped of two functions. One function is the rotational bar with eight ditches of same shape attached at lower end of a sand container. The constant volume sand can be dropped from a sand container by rotating this bar. Another function is the moving table, which is used to move a sand container horizontal direction. Model embankment is constructed by rotating the bar and by moving the sand container horizontally. A personal computer can control the rotational speed of the bar and the moving speed of the sand container, so that, the model embankment can be constructed automatically. By changing these parameters, a varied shape of embankment could be formed.

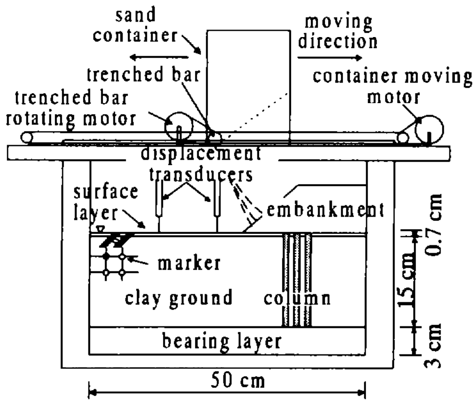

Fig.2. Sand hopper and model ground using centrifuge model tests.

Model Ground Preparation

A model ground is shown in Fig.2. Toyoura sand was used for the bearing layer, surface layer and embankment. NSF clay (w_l=66.8%, w_p=26.4%, I_p=40.3) was used for the soft ground. Model grounds were prepared as following.

1) The bearing layer was made with air pluviation method and relative density of the bearing layer was about 75%. The thickness of this layer was 3 cm.
2) The saturated clay slurry, with water content of 120%, was poured into the strong box to the thickness of 25cm. This slurry was mixed using a soil mixer for three hours under vacuum condition.
3) This model ground was pre-consolidated under 1g for about 60 hours with the load of equivalent self-weight of the surface sand layer under 56g centrifuge field.
4) After the pre-consolidation, the surface sand layer was formed with the similar air pluviation method of the bearing layer.
5) The model ground was consolidated under self-weight with double drainage conditions under a centrifugal acceleration of 56g for about 16 hours. The final thickness of the soft ground was around 15cm.
6) Model grounds were normally consolidated ground. From the cone penetration test at 56 g field, Strength of ground increased with depth. Cone index was q_c=118.7kN/m^2 in the middle of clay layer.

The stress condition in this soft ground under 56g was equivalent to the prototype soft ground of 8.4m heights.

Improved Soil Columns

Improved soil columns were made as following. The materials of improved soil columns were NSF clay, silica sand (No.8), high early strength cement and water. These materials were mixed with the weight ratio of 1:1:1.6:3.6.

The mixture was poured into the vinyl chloride mold with the inner diameter of 16mm and cured for 28 days in water. The average unconfined compression strength of this soil column at 28 days was 1.18MN/m^2.

Embankment and Construction Procedure

The model embankment, constructed by the sand hopper under 56g, is shown in Fig.3. The unit weight of the embankment was 19.18kN/m^3. The height of prototype embankment was 4.48m. In order to simulate rapid construction process where the clay layer were not consolidated by the weight of embankment during construction, construction time was determined to 79 seconds. The Converted prototype time was about 69 hours.

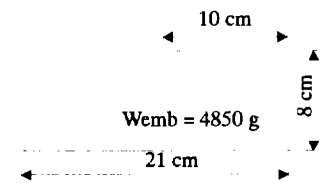

Fig.3. Model embankment

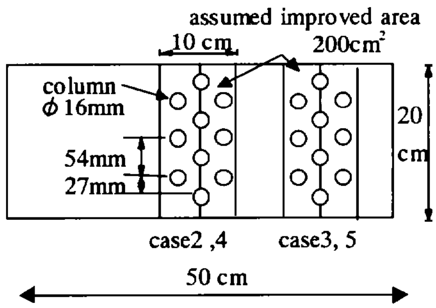

Fig.4. The location of improved soil columns.

Experimental Procedure

After the self-weight consolidation, optical targets were placed on the surface of the consolidated soft clay with a pitch of 2 cm. Clay ground was drilled vertically or diagonally, the miniature improved soil columns were installed into the soft clay at the predetermined position as shown in Fig.4. After the sand hopper was placed on the strong box, the strong box was mounted on the centrifuge. At first, a centrifugal acceleration was kept about 56g for 120 minutes to dissipate of excess pore pressure in clay ground. Then the model embankment was constructed by toyoura sand dropped from the sand hopper at 56g centrifuge field.

EXPERIMENTAL RESULTS

Deformations within the Ground

The deformation behavior of the clay ground is shown in Fig.5-9. The deformations like shapes of arc were observed in clay ground in all grounds except case 3. In case3, the horizontal displacement surpassed vertical one and any clear circular sliding surfaces was not observed in clay layer. The clear circular sliding surface located from below of the shoulder of slope to below of the toe of slope in case1, 2 and 4, however, circular sliding surface was observed from the

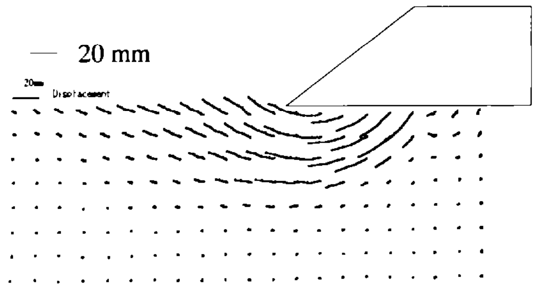

Fig.5. Displacement vectors for case1(unimproved ground)

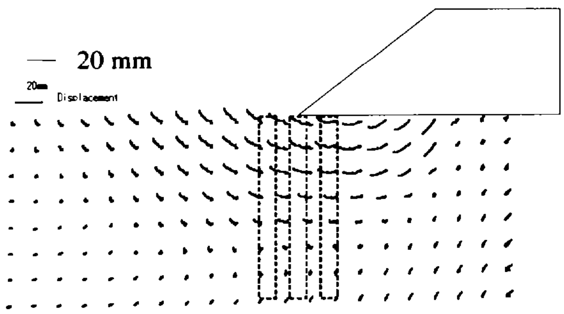

Fig.6. Displacement vectors for case2(toe improvement)

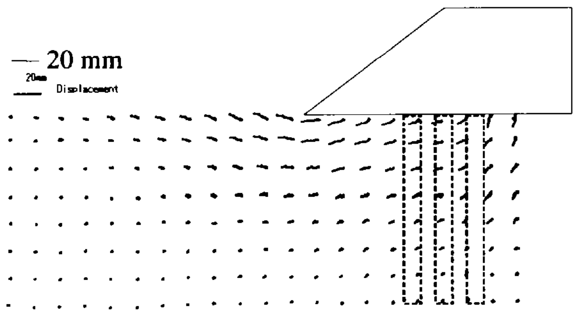

Fig.7. Displacement vectors for case3(shoulder improvement)

front of improved area to below of toe of slope in case5. The deformed area in case 2 and case4 propagated downward deeper than case1. Judging from displacement behaviors, ground improvement in case3 was more effective than other cases. It might be considered that improved soil columns effectively transferred embankment load to lower bearing layer so that sliding force due to weight of embankment might be decreased.

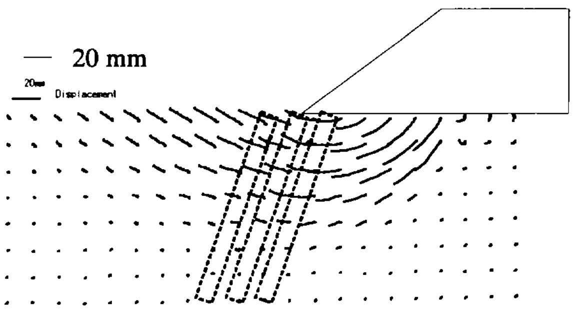

Fig.8. Displacement vectors for Case4 (toe improvement, inclination angle is 20 degree)

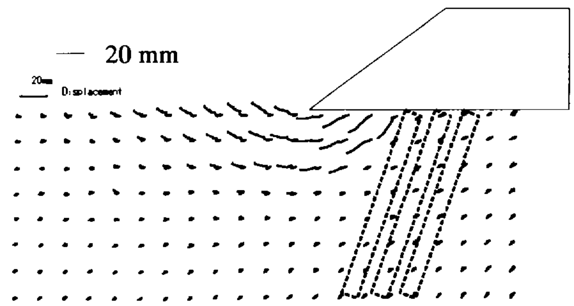

Fig.9. Displacement vectors for Case5(shoulder improvement,inclination angle is 20 degree)

Displacement of Ground Surface

The vertical and horizontal displacement of ground surface are shown in Fig.10 and Fig.11. The negative value means the settlement and the movement to the left hand side. All data were converted into prototype scale. In Fig.10,11, X- axis represents the horizontal distance from the toe of slope, and H means the height of the model embankment after construction. In Fig.10, any settlement or heaving could not be observed at the position of the toe of slope in case3 and case5. In case1(unimproved ground), case2 and case4 (improved grounds at the toe area of slope), maximum settlements were observed at the position below of the shoulder of slope. The position of maximum heaving of ground were observed at the location of about 0.25H left from the toe of slope for all cases. Heaving and settlement in case3 was considerably small compared to other cases.

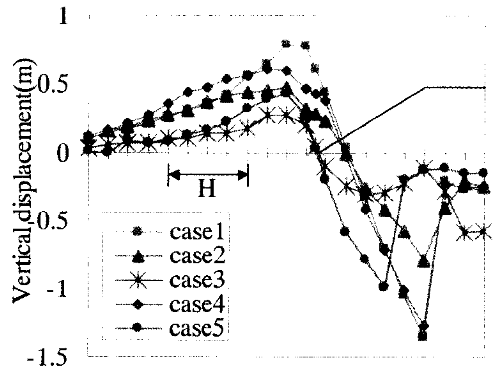

Fig.10. Vertical displacement of ground surface.

The maximum horizontal movement was observed at the position below of the slope in case1. In another cases of improved ground, the positions of maximum horizontal movements located around the toe of slope. Horizontal displacement in case3 was considerably small compared to other cases.

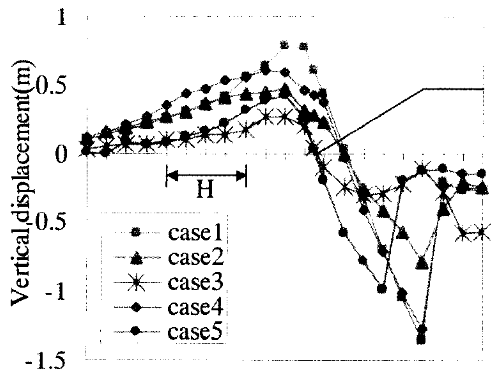

Fig.11. Horizontal displacement of ground surface.

NUMERICAL ANALYSIS

FEM Method

The test results were also analyzed by two-dimensional elasto-plastic FEM code 'PLAXIS'. Undrained condition and Mohr-Coulomb's failure criterion were assumed in the numerical analysis. Material properties of soils used in the calculation are shown in Table. 3.1 and table 3.2. FEM mesh was shown in Fig.12. Improved soil columns were modeled as elasto-plastic beam elements, and material properties of the column, EI, EA, was calculated as following for converting the value of unit depth. Maximum bending moment of

Table. 3.1. Input parameters for FEM(No.1).

	γ_t (kN/m^3)	N_u^*	G^* (kN/m^2)
Embankment	19.18	0.3	3771
Surface layer	19.25	0.3	2640
Clay 2	16.24	0.495	169
Clay 1	16.24	0.495	256
Bearing Layer	19.25	0.3	6034

(*: Presumed value)

Where; Nu: poisson's ratio

Table. 3.2. Input parameters for FEM(No.2).

	E* (kN/m^2)	ϕ^*	ψ^*	K_0
Embankment	9805	30	5	0.37
Surface layer	6864	30	5	0.37
Clay 2	506	0	0	0.57
Clay 1	765	0	0	0.57
BearingLayer	15688	30	5	0.37

(*: Presumed value)

Where; ϕ: internal friction angle, ψ: dilatancy angle

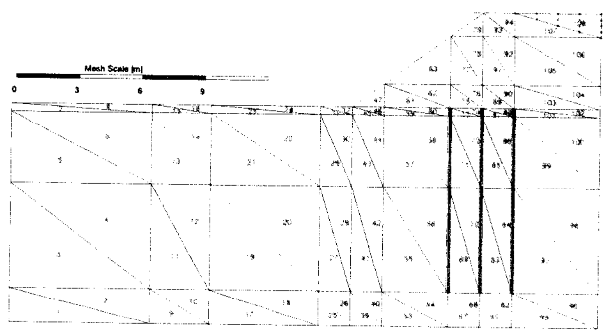

Fig.12. Mesh used in FEM analysis

column, Mp, was estimated the value which acted maximum shear stress on the middle of column. Maximum shear stress was presumed 10% of unconfined compression strength.

$$EI = E_0 \cdot I_0 \cdot n = 1837 \text{ (kN m}^2\text{/m)} \quad (1)$$

$$EA = E_0 \cdot A_0 \cdot n = 37716 \text{ (kN/m)} \quad (2)$$

$$Mp = pl/4 = 494.22 \text{ (kN}\cdot\text{m/m)} \quad (3)$$

Where:

E_0: elastic modulus of column (kN/m^2),
n: number of columns in unit depth,
I_0: moment of inertia (m^4),
Mp: maximum bending moment,
P: shear stress,
l: length of column (m),
A_0: cross sectional area of the column (m^2)

Analytical Results

The results of FEM are shown in Fig.13 and Fig.14. Judging from vertical displacements, the displacement of the improvement area was relatively controlled in case 3 and in case 5 where grounds were improved at shoulder area. However, on the horizontal displacement, the displacement curves were similar to all cases except case5. The case5 was the best improvement way to control the ground deformation among all test cases in analytical aspect. The position of the maximum heaving and maximum horizontal displacement was located around bellow of the toe of slope that agreed comparatively with experimental results. However, the position of the maximum settlement didn't agree with experimental results. The absolute value of computed displacements didn't coincident to experimental results. The main reasons were considered as follow. 1) The values of EI and EA might be overestimated when the three-dimensional column arrangement was converted to two-dimension field.
2) As the improved soil columns were modeled by elasto-plastic beam elements, the bending moment of the column could not be estimated correctly.

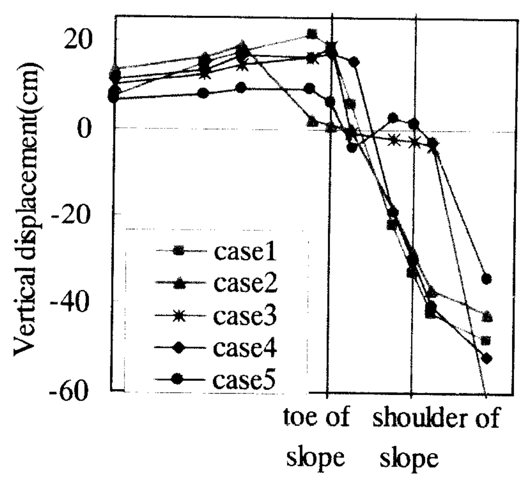

Fig.13. Vertical displacement of ground surface(FEM).

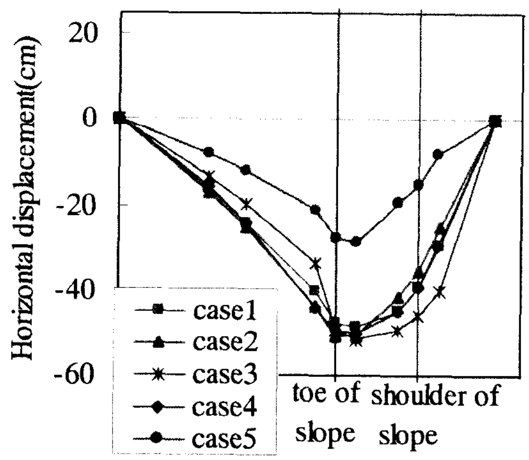

Fig.14. Horizontal displacement of ground surface(FEM).

CONCLUSION

Centrifuge model tests were carried out to investigate the effect of the location and the angle of improved soil columns. The deep soil mixing method (DMM) was adopted for the method of ground improvement in this paper. Also, the test results were simulated by a numerical analysis by mean of FEM. Following conclusions were obtained from this research.

1. The deformation of ground was considerably controlled in spite of adopting the low improvement ratio of 10%.
2. For preventing the failure or the large deformation of ground, ground improvement just bellow the shoulder of the slope was more effective than that bellow toe of the slope
3. Numerical analysis by means of FEM could be utilized to simulate the tendency of deformation behaviors of soft grounds

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