Longtime-behavior of big rail yard embankment on soft ground

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ABSTRACT: Japan Railway Construction, Transport and Technology Agency constructed a railcar depot as named (Kumamoto-depot) at a southern area in Japan for Kyushu Shinkansen. The depot was planned to serve as a maintenance yard for a new Shinkansen line operated between Shin-Yatsushiro and Hakata in Kyushu area. It is a large embankment accompanied reinforced soil retaining wall of 200,000m^2. In addition, it is under construction on a very soft clayey layer, it was necessary to adopt an accelerated consolidation method for its construction. Therefore, considering various aspects i.e. construction schedule, cost factor, and possible adverse effects susceptible to adjacent structures in this area, adoption of a pre-loading method and vacuum consolidation method for the construction anticipated. Moreover, an experimental embankment applying the accelerated consolidation method was constructed, and then we evaluated the effects and adverse effects under the subject environment according to the said method. In this study, we evaluated long-term behavior of the yard applying the measured data and the F.E. analysis.

1 INTRODUCTION

Kumamoto railcar depot is under construction on Kumamoto plain consisted very soft ground. This plain situates above the sea 5m or less. Therefore, an adverse effect on the structure in the surrounding according to the consolidation settlement and the horizontal displacement is feared. Additionally, it is planned that a site concerned constructs the box culvert structure for drainage in the fill, and is likely to subside as well as the embankment. In the design, it is necessary to consider the assumed immediately and long term residual subsidence to maintain the performance of these structures for a long term.

In this paper, it modeled based on the result of the laboratory tests and the test embankment on site, and the simulation analysis by soil/water coupled visco-plastic deformation analysis of two dimensions was executed.

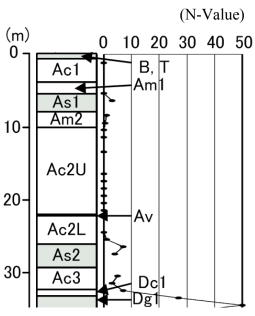


Figure 1 Ground strata

Moreover, we made the analysis based on the measurement data obtained with the progress of the construction, the parameter and analysis model used by the simulation analysis was evaluated.

2 SITE CONDITION

Kumamoto plain was chiefly used as a rice field. Figure 1 shows the outline of the ground strata in the this site. This ground strata has a soft clay layer (Ac1, Am1, Am2, Ac2) to the depth of about 25m. In addition sandy soil layer (As1, Av) exists between soft clay layers. Especially, the Ac2 layer that is a thick soft clay layer is divided into two layers (Ac2(U, L)) where the characteristic is different according to the lying Av layer. Under the alluvium layer, excellent diluvium (drift: Dc1, Dg1) exists. Moreover, the groundwater level is very high at the position of G.L(Ground Level)-0.8m.

The consolidation settlement by the embankment is conceivable that an extremely soft Ac1 and Ac2 (U, L) layer greatly affect.

3 ANALYSIS MODEL

Prior to executing the analysis of the site behavior, to examine the constitutive model of the ground and modeling the consolidation promotion method, the simulation analysis of tri-axial compression tests of soft clays (Ac1, Ac2) and we analyzed the trial embankment.

3.1 *Simulation analysis of tri-axial compression test*

To decide soil constitutive models, that adjusted to soft clay layers, the simulation analysis of the tri-axial consolidation un-drained compression test with no disturbance sample (Ac1 and Ac2(U,L)) was executed. It is soil/water coupled deformation analysis code DACSAR was used. It was assumable that soil constitutive model was Sekiguchi and Ohta model. Yield functions of this model are shown Eq.1 and 2. Eq.1 is the elasto-plastic model and Eq.2 is visco-elastoplastic model.

Table 1 shows soil parameters used here. These parameters have been decided based on a physical test and the consolidation test. The secondary consolidation index α has been decided based on the relation between the secondary consolidation speed (c_{α}) and the consolidation index (C_c) by Mesri and Goldewski (The cohesive soil: Cα/Cc=0.05 and assume it with Cα/Cc=0.05±0.02).

Figure 2 shows the stress pass by the experiment and the analysis. The solid line is an outcome of an experiment, and the point shows the analytical result. The elasto-plastic model shows a tendency that is nearer the outcome of an experiment than the visco-elastoplastic model in the Ac1 layer. Oppositely, the visco-elastoplastic model was able to reproduce the outcome of an experiment in accuracy in the

Table 1 Soil parameter

Ground strata / Parameter	Ac1	Am1	Am2	Ac2U	Ac2L	Ac3
Void ratio(e_0)	2.535	1.555	1.386	1.758	2.524	1.444
Critical state parameter(M)	1.313	1.382	1.523	1.191	1.453	1.532
Dilatancy coefficient (D)	0.083	0.040	0.037	0.086	0.103	0.111
Compression index(λ)	0.428	0.163	0.151	0.306	0.567	0.450
Irreversibility ratio(Λ)	0.906	0.887	0.885	0.919	0.926	0.927
Coefficient of earth pressure(K0)	0.559	0.539	0.513	0.589	0.534	0.519
Poisson ratio(ν)	0.359	0.350	0.339	0.371	0.348	0.342

Table 2 Soil parameter (Elastic material)

	Em-bank-ment	BT	As1	Av	As2	Dc1	Dg1
Elastic index (Mpa)	25.0	7.5	8.2	10.4	12.5	12.7	48.7
Poisson ratio	0.3	0.3	0.3	0.3	0.3	0.4	0.3

Table 3 Sheet pile parameter

Modulus of deformation (kPa)	Sectional area (m^2)	Moment (m^4)
2.1×10^8	0.01732	0.000324

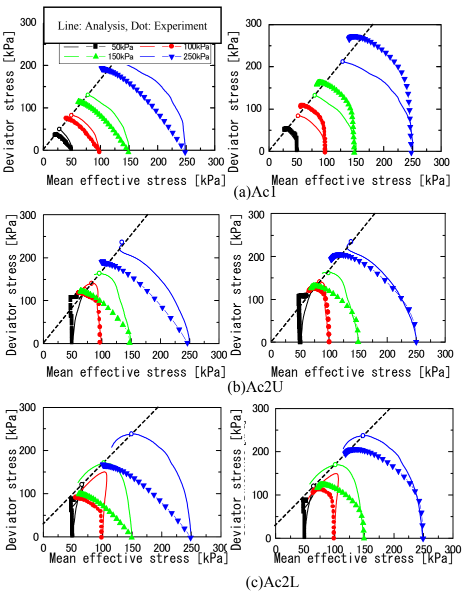
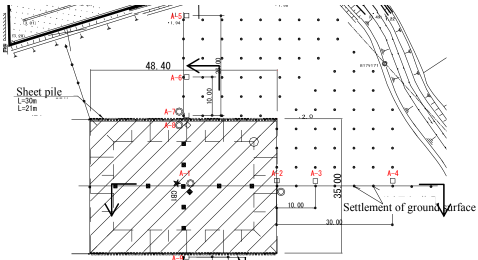


Figure 2 Stress pass
(Left: Elasto-plastic model, Right: Visco-elastoplastic model)



Symbol	Measurement	Point
◎	Differential settlement gauge	4
◆	Water Pressure mater	1
□	Inclinometer	9
■	Settlement	9
●	Stake	105
◇	Strain gauge	1
★	Boring	1

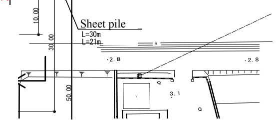


Figure 3 Outline of test embankment A

$$f = \frac{\lambda - \kappa}{} \ln \frac{p'_m}{} + D\frac{q}{} - \varepsilon_v^p \quad \text{(Eq.1)}$$

$$f = \frac{\lambda - \kappa}{1 + e_0} \ln \frac{p'_m}{p'_{m0}} + D\frac{q}{p'} - \alpha \ln\left(\frac{t_0}{t}\left(\exp\left(\frac{\varepsilon_v^p}{\alpha}\right) - 1\right)\right) \quad \text{(Eq.2)}$$

κ :swelling index, ε_v^P: plastic volume strain, p': effective principal stress, q: principal stress deviator, p_m': effective mean principal stress, t, t_0:time

Ac2(U,L) layer. A constitutive model and a parameter in each clay layer were set by using this result.

3.2 *Simulation analysis of test embankment*

We constructed a test embankment, and evaluated the soil properties and the workability, the behavior, and the countermeasure there for.

3.2.1 *Test embankment A*

Figure 3 shows the outline of test embankment A. This embankment is a size of 48.4m×35.0m and the inclination is 1:1.5, the height of the embankment is about 4.0m. The purpose of the embankment is an evaluation of the soil properties. Another purpose is an examination of the effect of sheet piles as the displacement interception method.

Sheet piles are continuously placed in the hip of embankment in long length direction (A-A' section: 48.4m), and any countermeasure is not executed in short length direction (B-B' section: 35.0m). The behavior of the ground when the embankment is constructed with the displacement of the long length direction is understood. The effect of the interception of displacement by sheet piles is understandable from the measurement result in the short length direction.

Sheet piles set up one piece every five pieces to hard strata considering an economical factor, workability, and groundwater (the length of sheet piles is 30.0m). The other four pieces were assumed length to the soft strata (the length of sheet piles is 20.0m).It was assumable that Ac2 was a visco-elastoplastic model and Ac3 was an elastoplastic model from the simulation result of tri-axial compression test. The linear elasticity model modeled sand, the gravel bed, and the diluvium. Table 1 and Table 2 show the soil parameters used with each constitutive model. Sheet piles were modeled by the beam element (Table 3). Slipping of the ground and sheet piles were made by installing the thin and weak strength element as the boundary element between the ground and sheet piles.

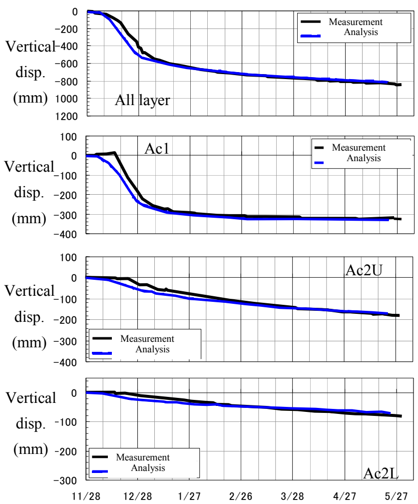
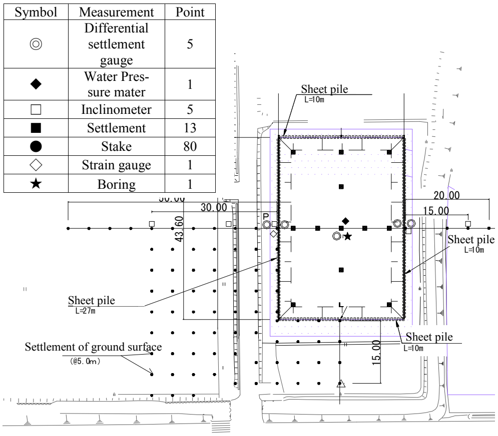


Figure 4 Analysis result of test embankment



Symbol	Measurement	Point
◎	Differential settlement gauge	5
◆	Water Pressure mater	1
□	Inclinometer	5
■	Settlement	13
●	Stake	80
◇	Strain gauge	1
★	Boring	1

Figure 5 Outline of test embankment B

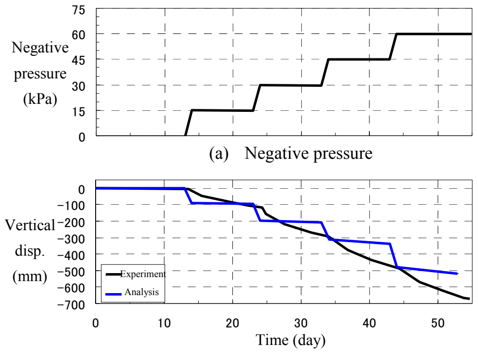
Figure 4 shows the analytical result and the measurement value of the ground level and each strata of the embankment. An analytical result can reproduce a measurement value in behavior in each layer though the initial behavior is slightly different. We confirmed the validity of soil parameters and a constitutive model according to this result.

3.2.2 *Test embankment B*

Fig. 5 shows the outline of test embankment B. The plastic board drain is set up in the range of the vacuum consolidation construction. The drain length is 22m, and the drain interval is 1m.

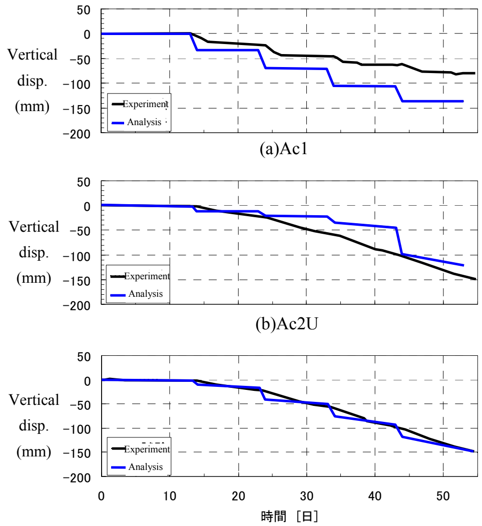
Sheet piles for underground water still water from the surrounding soil, sheet piles to disown it, and the part layer of sand on were placed in surroundings within the range of the improvement. Surroundings within the range of the vacuum consolidation construction set up sheet piles for the interception of settlement and groundwater from the sand strata. The soil parameters and the constitutive model as used for the analysis are similar to test embankment A. The vacuum consolidation method was modeled by setting a negative pore water pressure within the range of construction. The coefficient of permeability in the vacuum consolidation area was set based on

$$k' = \frac{(A_e - A_w)k_s + A_w k_w}{A_e} \qquad \text{(Eq.3)}$$



(a) Negative pressure
(b) Amount of settlement

Figure 6 Negative pressure and amount of settlement



(a)Ac1
(b)Ac2U
(c)Ac2L

Figure 7 Amount of settlement

the next equation considering the drainage effect by plastic drains.

Here, Ae is area (=π･de2/4) of an equivalent effective circle, Aw is sectional area (=π･dw2/4) of the drain, d is installation interval (=1m) of the drain, De is a diameter of an equivalent effective circle (1.13d and square = arrangement), ks is a coefficient of permeability of the ground and kW is coefficient of permeability (=1.0×10^{-2}cm/s) of the drain. Thee coefficient of permeability of the construction area of the vacuum consolidation was assumable to be 2.86×10^{-5}cm/s.

Fig. 6 shows the step of negative pressure and the vertical displacement in the ground level at the center of the vacuum area and the vertical displacement in each stratum are as shown in Fig. 7. The analysis and the experiment of the absolute value of the amount of the settlement are roughly equal. However, the process of the settlement became a contrasting behavior. The subsidence progresses also at the stage where negative pressure is maintained in the experiment. When negative pressure increased in the analysis, the subsidence is rapidly caused.

However, the subsidence is apparently unprogressive when negative pressure is maintained. From the measurement result of the pore water pressure, the pore water pressure decreases immediately when negative pressure rose at the upper side more than Am2. However, when negative pressure is constant-

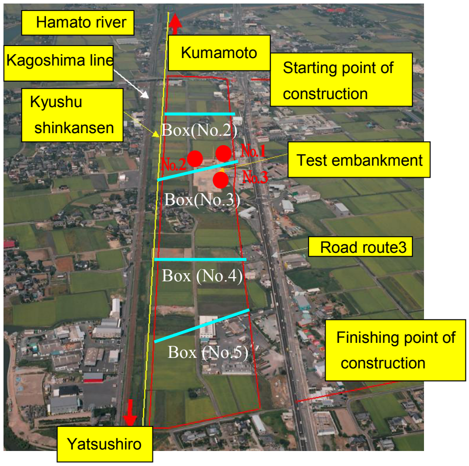
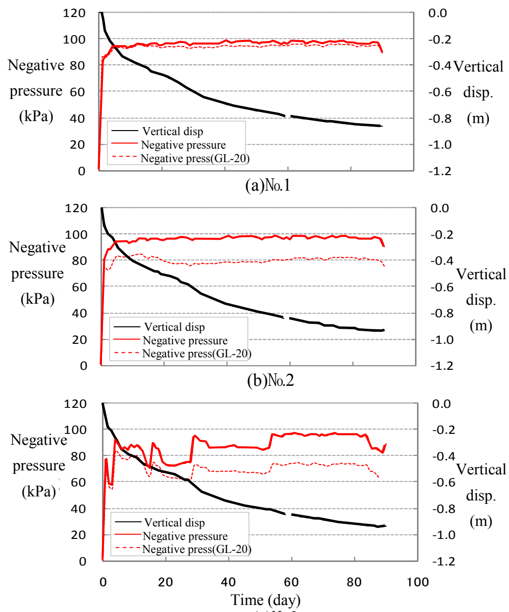
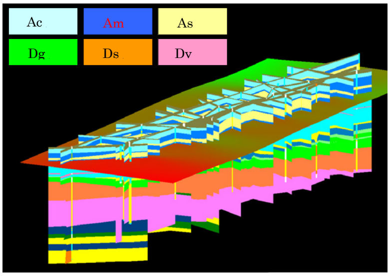


Figure 8 View of Kumamoto railcar depot

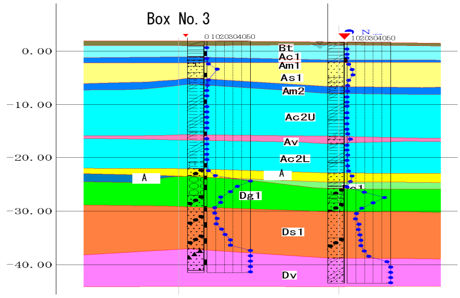


(a)No.1
(b)No.2
(c)No.3

Figure 9 Monitoring data



(a) 3-dimensional model



(b) 2-dimensional model

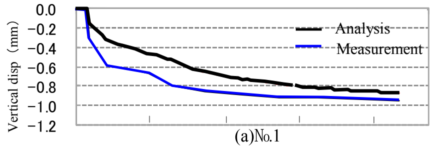
Figure 10 ground model

ly maintained, the change of the pore water pressure is hardly caused. On the other hand, the pore water pressure at the time of when negative pressure was raised has not decreased still enough in Ac2U and the Ac2L layer.

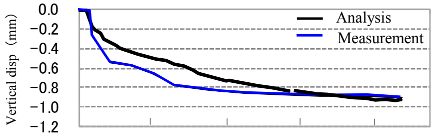
Moreover, it was understandable that the pore water pressure decreased at the time when negative pressure was constantly maintained. It is conceivable that this becomes a cause and the subsidence tendency in Ac2U and the Ac2L layer could not be reproduced sufficiently. Therefore, it is important to give the movement of the pore water pressure constructing the vacuum consolidation appropriately, when actual behavior is evaluated.

4 ANALYSIS OF RAILCAR DEPODT

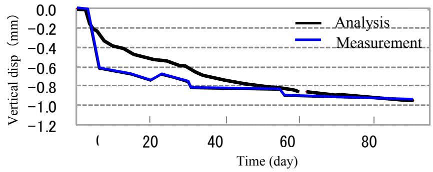
The measurement data of an actual structure was used and analyzed. Here, the measurement data of the vacuum consolidation construction area adjacent to the test embankment area was used.



(a)No.1



(b)No.2



(c)No.3

Figure 12 Result of analysis

4.1 *Measured data*

The measurement data used the data of three points (No.1, 2, 3) shown in Fig. 8. Fig. 9 shows the measurement data of an amount of the subsidence and negative pressure (G.L.20m). The negative pressure on the point of No.1 is both approximately 95kPa of G.L and G.L-20 m and a pressure at the same level as G.L is given to a deep layer. On the point of No.2 and 3 negative pressure of G.L-20m has lowered by approximately 20% compared with G.L. However, a comparatively high, negative pressure is secured in a deep layer. The amount of the subsidence at the time of completion of loading the negative pressure was 0.85-0.95m. This amount is very large because the effect of the consolidation promotion was given to a deep layer.

4.2 *Analysis condition*

Fig.10 shows the ground model used the analysis. We made 3-dimensional ground model from boring data calculated the voxel and draw the contour. Fig.11 shows the finite element model. The displacement boundary condition was assumed all node fix at the bottom of the model. Moreover, the side is assumable that horizontal direction is fixed, and a

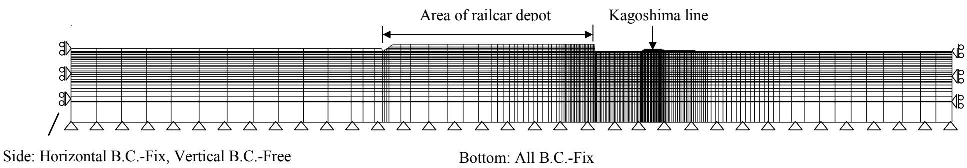


Figure 11 Finite element model

vertical direction is assumed a roller. GL-0.8m was a groundwater level side, and this level was assumed a drainage boundary. The soil parameters and the constitutive model were same of the test embankment. The embankment weight is 20kN/m^3 from the laboratory test. We considered it by the equivalent nodal load converted from the embankment layer thickness. Fig.12 shows the result of analyses. In the case of No.1, the analytical result was 0.95m while an actual amount of the subsidence was 0.85m at the time of finish of vacuum consolidation. In the case of No.2 and 3, the actual measurement value was 0.9m, and an almost equal as for analytical result was obtained. It has been understandable that the amount of the subsidence of the result of the analysis and the measurement data is roughly corresponding. Therefore, the validity of the ground model and the ground parameter used was able to be confirmed. However, the amount of the immediate subsidence of giving a negative pressure as well as the analysis of the test embankment is growing than the measurement data. As a factor, it is conceivable that the evaluation of the coefficient of permeability of the ground in the area setting up plastic drains. The coefficient of permeability is assumable to be a large the difference, approximately ten times different. It is necessary to remark it sufficiently in the evaluation in the future.

5 CONCLUSIONS

We decided soil parameters and models from the simulation of tri-axial compression tests and trial embankment. By using these parameters and models and measured data on the site, the long term behavior of the deposit embankment could be evaluated. The conclusions are as follow:

(1) The application of soil constitutive model against individual clayey layers needs for a high accuracy from the simulation of tri-axial compression tests. From the analyses of test embankment, we confirmed that it was correct these analysis conditions.

(2) The behavior of the embankment during the construction could simulate from F.E.M. analyses. Therefore, it was possible to confirm that the soil parameter and the constitutive model used were appropriate. Additionally, it is important to set boundary conditions along depth based on the measured data of pore pressure.

We executed the behavior analysis in the vacuum consolidation method, and real behavior was able to be reproduced roughly. Modeling negative pressure besides the soil parameters is essential to evaluate the behavior of the ground in the vacuum consolidation method. It is preferable to examine the improvement of analysis model by using a newly measured data in the future.

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