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Two-dimensional Effective Stress Analysis on Consolidation of Clay under Highway Embankment and Its Seismic Response after Consolidation

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ABSTRACT

A new constitutive model (cookie model) for clay was proposed by Iai, et al. (2015). This model can simulate both monotonic and cyclic loading and express various characteristics of clay. This model has been developed as an extension of a model for liquefiable sandy soil under drained condition. Thus this model can simulate multiple disasters. For example, using this model, we continuously perform the self-weight analysis, the consolidation analysis and the seismic response analysis for the complex ground. We will perform, as an example, these continuous simulations using a highway embankment constructed on the foundation ground including thick clayey layers.

Introduction

Many important facilities, such as oil storage stations or electric power plants, have often been constructed in coastal zones. At coastal zones, we can frequently find clayey layers, and such important facilities are often constructed on the reclaimed land above original ground including clayey layers.

After landfilling, the settlement of the ground caused by consolidation and secondary consolidation of the clayey layers occurs, and then the forces acting on the shore protection of the reclaimed land changes. If an earthquake should occur at that time, the soundness of the facilities will have been affected not only by conditions of the shore protection but also liquefaction of the sandy soil used for landfilling and sand layers at the site. So, we should take into account possible effects of both consolidation of clayey layers and liquefaction of sandy layers so as to evaluate the soundness of the facilities during earthquakes.

From this point of view, we need tools that can simulate the processes of consolidation, evaluate the change of forces acting on the shore protection caused by the consolidation, and evaluate the degree of increase in pore water pressure caused by earthquakes during and after the consolidation. Then we could estimate the degree of damage by the earthquakes and evaluate the effectiveness of the improvements in the soils and the structures.

A new constitutive model (cookie model) for clay was proposed by Iai, et al. (2015) based on the framework of the strain space multiple mechanism model of granular materials. This model has been developed as an extension of a model for liquefiable sandy soil under drained condition,

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also proposed by Iai, et.al. (2011). These models have been installed in a two-dimensional dynamic effective stress analysis program “FLIP”. This code could continuously simulate the consolidation processes and evaluate damage to the structures caused by earthquakes after consolidation.

We will show that the model could simulate the consolidation process and continuing earthquake process of a highway embankment as an example problem for this method.

About the Highway Embankment

Cross-section of the Embankment and the Soil Profile

The highway embankment located in Hitachi city, Japan, was first constructed as a test embankment using dredge sand on the ground including thick soft marine sediment clay layer “Ac2” as shown in Figure 1. This test embankment was removed after 1.4 years, then one year later, construction of the permanent embankment began, using tunnel spoil. The unit weight of tunnel spoil was larger than dredge sand.

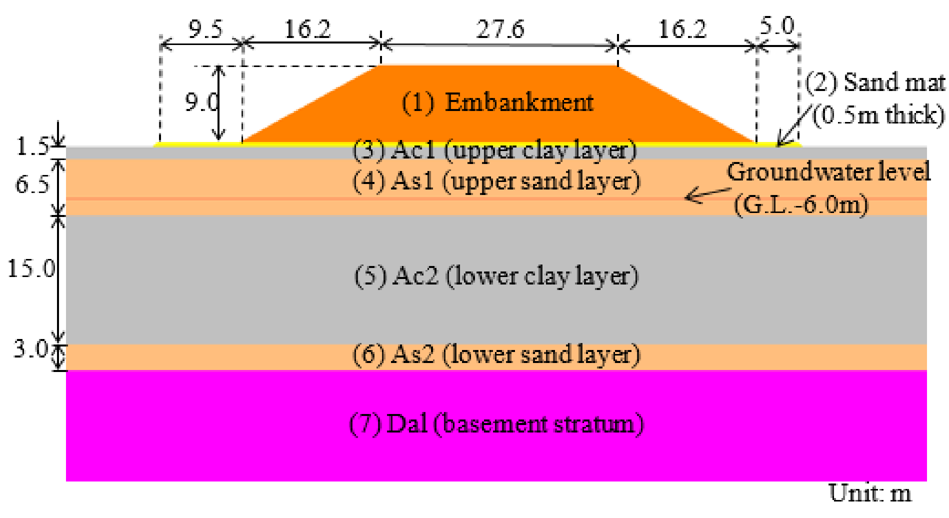
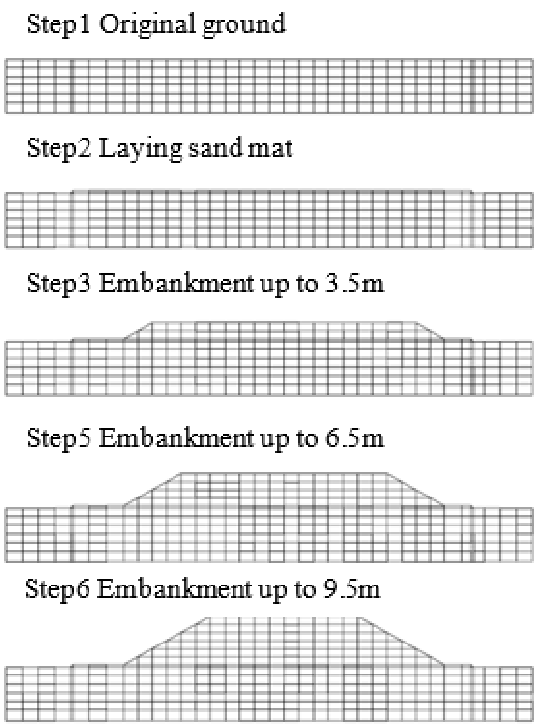


Figure 1. Cross-section of the embankment and the soil profile (Ohta, et al. (2005))

Construction Time Schedule of the Test Embankment

We show the construction time schedule of the test embankment in Figure 2. We simulated the consolidation process according to this schedule in two ways. We applied unit weight of the dredge sand to the embankment, while on the other hand we applied unit weight of tunnel spoil. We used the results of the former to evaluate the behavior of the test embankment and its foundation ground, i.e. short range simulation. The latter, we used to evaluate the behavior of the permanent embankment and its foundation ground i.e. long range simulation. In the latter case we ignored the effect of pre-loading because the time interval between removing and re-construction of the embankment is not so long and notable settlement not observed during this interval. We adjusted the time axis to ensure consistency between simulation results and observed data when showing long-range results.



Step	Comment	Height of embankment (m)	Time elapsed (day)
1	Original ground	0.0	0
2	Laying sand mat	0.5	90
3	Embankment up to 3.5m	3.5	123
4	Remaining unchanged	3.5	148
5	Embankment up to 6.5m	6.5	183
6	Embankment up to 9.5m	9.5	208

*Time was set to 0 (day) at the beginning of step2 in our numerical simulations.

Figure 2. Construction time schedule of the test embankment (Ohta, et al. (2005))

Settlement of the Embankment

The settlement of the embankment continued from the construction of the test embankment, except for the time interval between removing and re-construction of the embankment, resulting in about 4m of settlement during 21 years. So the overlay of the pavement had been repeated 5times, resulting in an increase of the thickness of the pavement by 125.9cm. Also they widened the road. We ignored the effects of the overlay and the widening for the sake of simplicity in the numerical simulations.

Analytical Models

Procedures of Numerical Simulation

We performed numerical simulations in order to evaluate the behavior of the embankment and the foundation ground. First we performed initial self-weight analysis on original ground setting boundaries at the groundwater level where pore water pressure is set to zero. Then we simulated the consolidation process in two ways, short range and long range simulations. The excess pore water pressure is set to zero at the ground surface. After long range simulations, we performed continuously earthquake response analyses.

The Constitutive Model and Computer Program

We applied a new constitutive model for clay proposed by Iai, et al. (2015) to “Ac2” layer. The constitutive model is installed in a computer program “FLIP” which is based on the finite element method. “Ac1” layer is also a clay layer, but its over-consolidation ratio is 42.92 and the thickness of the layer is only 1.5m, so we decided that “Ac1” layer could be modeled by linear plane elements the same as other layers.

We also simulated the consolidation process using the Sekiguchi-Ohta model which is installed in a computer program “DACSAR” (Iizuka and Ohta (1987)) for the purpose of reference. This constitutive model was proposed by Sekiguchi and Ohta (1977) which has a yield surface and an associated flow rule. The “DACSAR” program has been widely used in engineering practice in Japan.

Finite Element Model

Finite element mesh that is used commonly in this simulation is shown in Figure 3. Output points and elements also are shown in Figure 3.

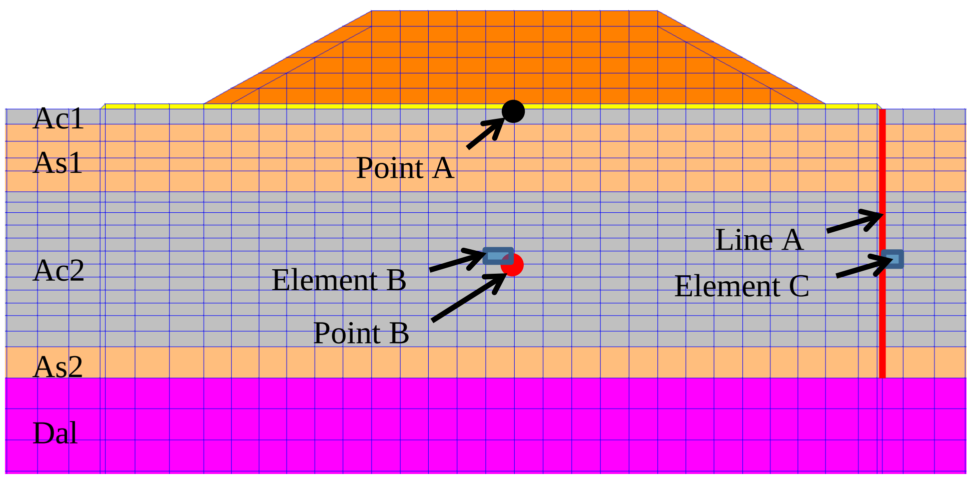


Figure 3. Finite element mesh (major portion) and output points and elements

Model Parameters of Ac2 Layer for Simulation

Model parameters of Ac2 for DACSAR are shown in Table 1. These parameters are given for concurrent analyses held by the Japanese Geotechnical Society. We set parameters for FLIP in Case 1 and Case 2 based on the parameters for DACSAR as shown in Table 2. Referring the swelling index κ, void ratio e_0 and effective overburden pressure σ_{vi}' in Table 1, we estimated the bulk modulus K_{Ua} as 1644kPa and corresponding shear modulus as 352kPa using a relationship between bulk modulus and shear modulus of isotropic linear elastic material. In order to represent the coefficient of earth pressure at rest K_i shown in Table 1 in the initial self-weight analyses by FLIP, it was required to adjust the bulk modulus K_{La}, resulting in 2155 kPa. However, in the consolidation analyses, we have to use bulk modulus corresponding to the compression index λ. Reduction factor of bulk modulus for consolidation analysis r_k was used for this purpose. Converting shear modulus (352 kPa) into shear velocity, we get shear velocity as 15m/s. On the other hand, from results of PS logging, we found shear velocity to be 100m/s at Ac2 layer. Then we could get shear modulus as 15100kPa (see Case 3 in Table 2). After getting shear modulus, we traced the similar procedure as Case 1 or Case 2, except we referred more detailed information about soil layer (Tatta, et al. (2003)).

Table 1. Model parameters of Ac2 for DACSAR (Ohta, et al. (2005))

Parameters	Symbol	Specified value
Void ratio	e_0	2.299
Compression index	λ	0.596
Swelling index	κ	0.272
Effective overburden pressure	σ_{vi}'	135.5 kPa
Over-consolidation ratio	OCR	1.00
Coefficient of earth pressure at rest	K_{i}	0.66
Coefficient of permeability	k	7.33E-10 m/s
Effective Poisson's ratio	ν'	0.40
Secondary consolidation coefficient	α	9.03E-03
Initial volumetric creep strain rate	$\dot{v}_0$	6.97E-10 1/s

Lacking actual parameters for marine sediment clay, we applied the TESRA / Isotach parameters for kaolin clay (Tatsuoka, et al. (2002)) as shown in Table 2 to the earthquake response analyses.

Table 2. Model parameters of Ac2 for FLIP (cookie model)

Parameters	Symbol	Specified value		
		Case 1	Case 2	Case 3
Reference effective stress	P_{a}	135.5 kPa		137.9 kPa
Bulk modulus at reference state	K_{La}	2155 kPa		68430 kPa
Power index for bulk/shear moduli at static gravity analysis	m	0.5		
Reduction factor of bulk modulus for consolidation analysis	r_{k}	0.348		0.0112
Over-consolidation state	$r_{p_{\mathrm{n0}}}$	1.00		
Shear modulus at reference state	G_{ma}	352 kPa		15100 kPa
Coefficient of permeability	k	7.33E-10m/s		
Parameter controlling contractive component	$r_{\varepsilon_{\mathrm{d}}^{\mathrm{c}}}$	0.85		5.0(Consolidation) 0.85(Dynamic)
Parameter controlling dilatancy at the steady state $r_{K\mathrm{us}} = \kappa / \lambda$	$r_{K\mathrm{us}}$	0.456		
Secondary consolidation coefficient	α_{c}	0.0	9.03E-03	
Initial volumetric creep strain rate	$\dot{v}_0$	0.0 1/s	6.97E-10 1/s	
TESRA parameter representing decay rate $r_{\mathrm{TSR}} = r_1$	r_{TSR}	0.0	0.1	
Isotach parameter representing max amplitude $r_{\mathrm{Iso}} = \alpha$	r_{Iso}	0.0	0.5	
Isotach parameter for normalizing strain rate $r_{\dot{\gamma}} = 1/\dot{\varepsilon}_{\mathrm{r}}^{\mathrm{ir}}$	$r_{\dot{\gamma}}$	0.0 s	1.0E+06 s	
Isotach parameter for representing strain rate dependency	q_6	0.0	0.04	

In all, we performed simulations of 3 cases using the FLIP program.

1) Case 1: Considering no effect of strain velocity.
2) Case 2: Considering effects of strain velocity, that is secondary consolidation and dumping of TESRA type.
3) Case 3: Same as Case 2, but using shear modulus, etc., calculated from results of PS logging.

Analytical Results

Results of Numerical Simulation of Short Range Consolidation

Vertical displacement history at the top of original ground (Point A; see Figure 3) and Excess pore water pressure history at the center of Ac2 layer (Point B) from beginning of construction of test embankment (t=90day) are shown in Figure 4. Horizontal displacement distributions along Line A at completion of test embankment (t=208day) are also shown in Figure 4. These figures show that the results of FLIP (Case 3) are roughly in accordance with the measured values (Tatta, et al. (2003)) rather than the other cases. The rigidity evaluated in the DACSAR program and the FLIP program (Case 1 and Case 2), that parameters are consistent with the parameters for DACSAR with regard to rigidity, is smaller than that of FLIP (Case 3). So the displacement by FLIP (Case 3) is smaller than the others, and agrees with the measured values because the parameters with regard to rigidity are set based on the results of PS logging performed at the site.

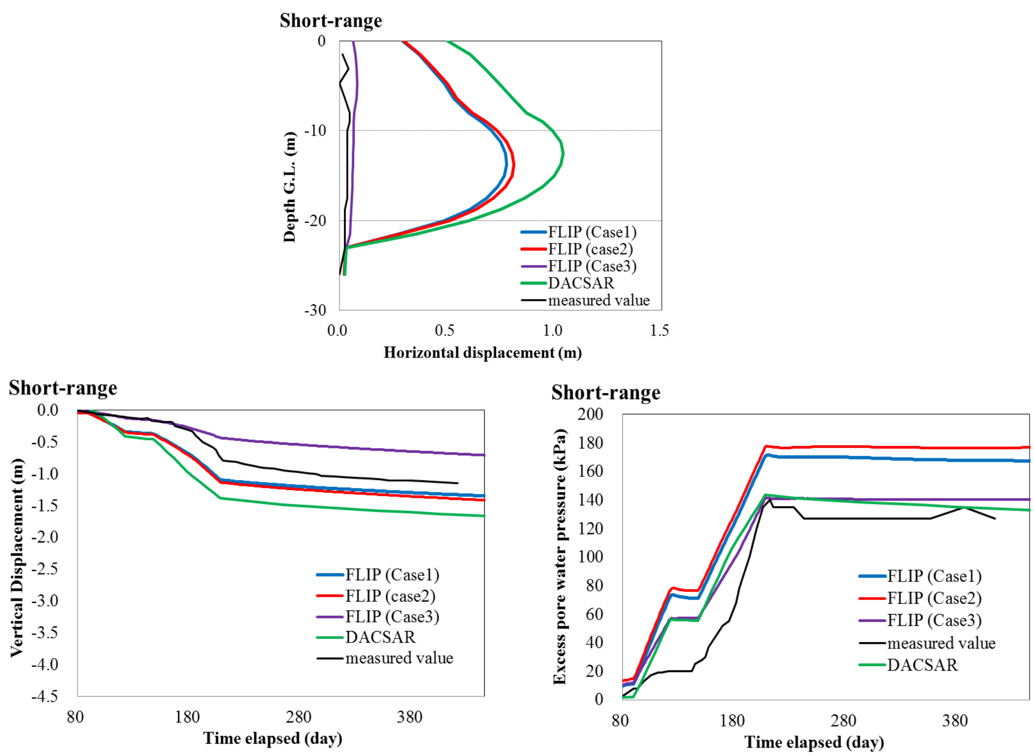


Figure 4. Horizontal displacement distribution along Line A at completion of test embankment (upper figure), vertical displacement history at Point A (lower left figure), excess pore water pressure history at Point B (lower right figure).

Results of Numerical Simulation of Long Range Consolidation

We ignored the effect of pre-loading for the sake of simplicity. So we directly started construction of permanent embankment according to the schedule shown in Figure 2. We skipped the procedure from removing to re-construction of the embankment. The construction of the permanent embankment began after 2.5years from starting construction of the test embankment. Before starting construction of the permanent embankment, we set a 90-day interval so as to lay a sand mat. Then our simulations start from day 823 after starting construction of the test embankment. This adjustment is needed to ensure consistency between simulation results and observed data.

Vertical displacement history at the top of the original ground (Point A; see Figure 3) and Excess pore water pressure history at the center of Ac2 layer (Point B) are shown in Figure 5. Excess pore water pressure distribution at completion of the permanent embankment is shown in Figure 6. The measured vertical displacement time history is steeper than the results of any simulation. We think the reason is the maximum overlay performed at day 4800. Before and after this overlay, the inclination pitch was similar to that of FLIP (Case 2). Comparing results of FLIP (Case 1) and FLIP (Case 2), the effect of secondary consolidation is apparent on the results of FLIP (Case 2).

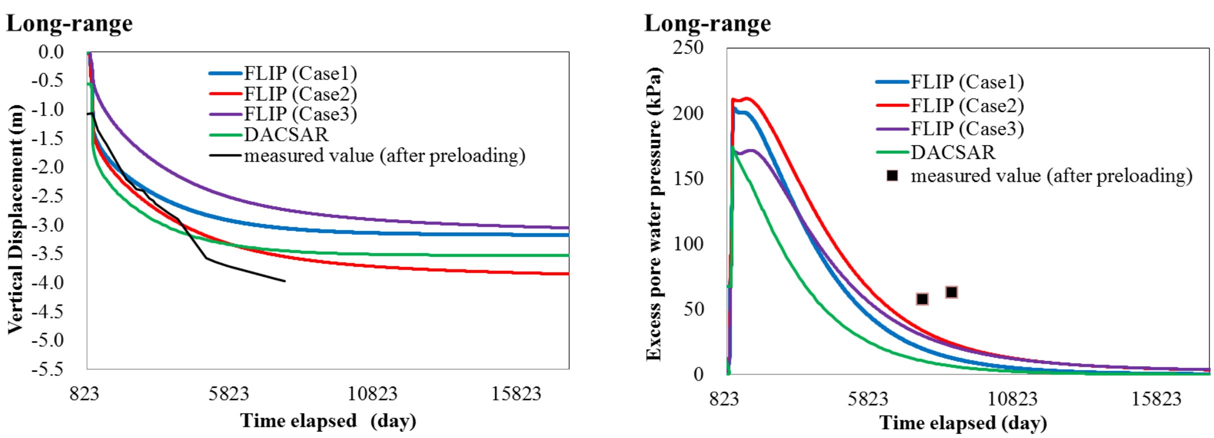


Figure 5. Vertical displacement history at Point A (left figure), excess pore water pressure history at Point B (right figure)

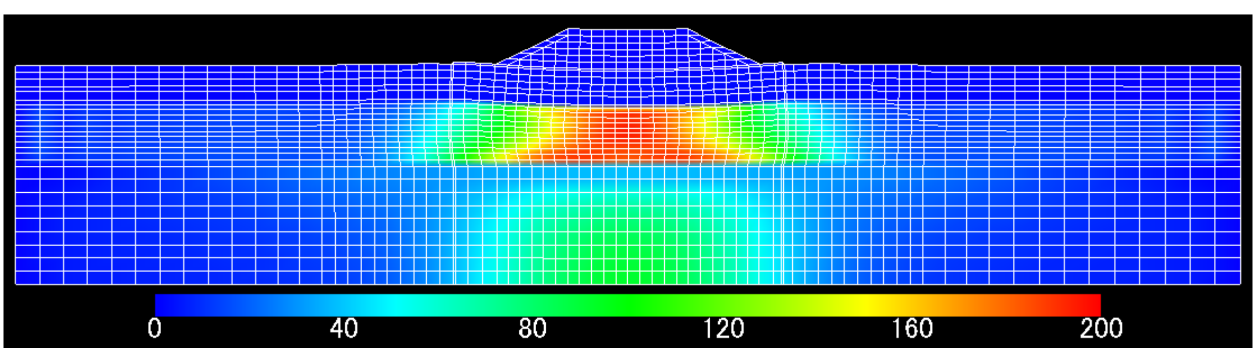


Figure 6. Excess pore water pressure (kPa) distribution at completion of permanent embankment

Results of Seismic Response Analyses after Consolidation

We performed seismic response analyses after consolidation applying the cookie model to Ac2 layer. The input motion and an example of the results of the seismic response analyses are shown in Figure 6. In this figure, we show the results obtained by FLIP (Case 2) considering the TESRA type dumping effect. Even if after consolidation, both the increase in excess pore water pressure at Point B and the settlement at Point A appear in the figures.

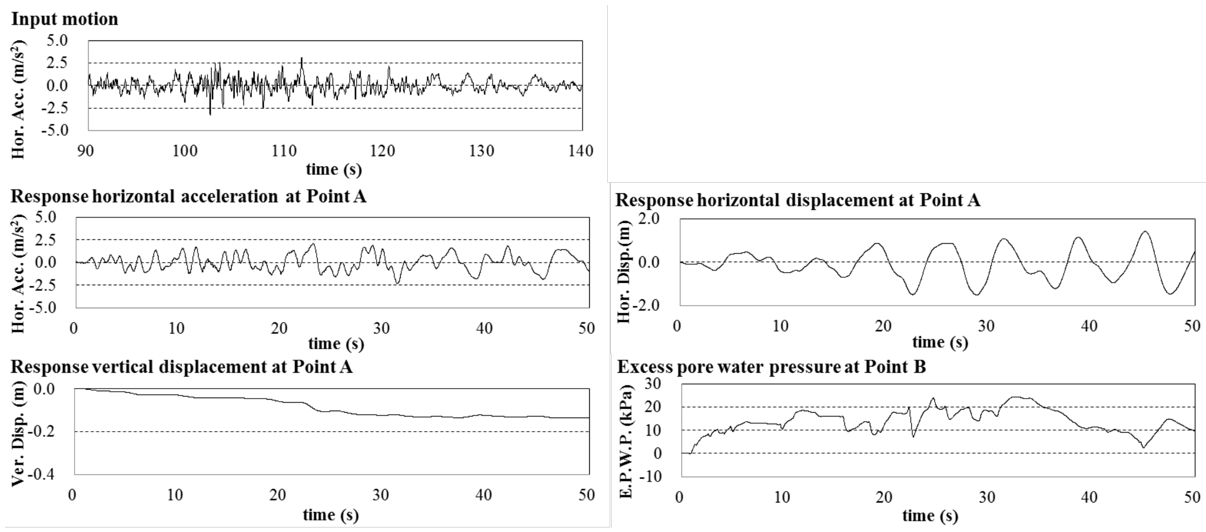


Figure 6. Input motion and results of earthquake response analysis on the Case 2

Conclusions

Applying the constitutive model (cookie model) proposed by Iai, et al. (2015) to the clay layer under the highway embankment, we could perform continuously the self-weight analysis, the consolidation analysis and the seismic response analysis. By the consolidation analyses we could obtain the results that are roughly in accordance with the measured values, especially in case of the short range simulation. It is important to assign the appropriate values to parameters on rigidity. By the seismic response analyses after the consolidation analyses, the possibility that additional settlement and increase in pore water pressure occurred was shown.

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