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Deformation of embankments on soft ground –Better computer simulation resulted from in-put data closer to the reality-

Déformation des entassements de terre sur terrain mou -Introduction de données informatiques plus proches de la réalité résultant en une meilleure performance des simulations informatiques -

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ABSTRACT
The mechanical behaviour of 5 pre-loading highway embankments placed on very uniform soft Ariake Clay is analysed by employing the soil/water coupled finite element programme DACSAR coded by Iizuka and Ohta (1987) incorporating an elasto-viscoplastic constitutive model proposed by Sekiguchi and Ohta (1977). A series of computer simulation confirms that a set of input data closer to the reality results in the better performance of the computer simulation especially in calculating the lateral movement of subsoil beneath the toe of the embankments.

RÉSUMÉ
Cinq pre-loading d'entassement de terre pour autoroute ont été placés sur le terrain argileux mou et uniforme d'Ariake pour examiner les déformations du terrain sur lequel seront construites les structures annexes de l'autoroute. Le mouvement mécanique de ces pre-loading a été analysé selon le programme DACSAR codé par Iizuka et Ohta(1987) auquel a été incorporé le modèle constitutif elasto-viscoplastic proposé par Sekiguchi et Ohta(1977). L'exécution d'une série de simulations informatiques a confirmé que l'introduction de données plus proches de la réalité entraine, dans le domaine de ces simulations, de meilleurs résultats qui correspondent avec plus de précision aux valeurs mesurées sur les lieux en question, particulièrement au niveau du calcul du mouvement latéral de la partie souterraine du sol situé au pied de l'entassement de terre.

1 INTRODUCTION

Computer simulation of the mechanical behaviour of soft subsoil during and after construction using soil/water coupled FEM is expected to reliably work when a set of input parameters close to the one representing the actual soil properties and initial and boundary conditions is chosen. In this paper, a trial series of computer simulation is made aiming at demonstrating that the choice of input parameters is amazingly influential on the computed results.

2 SITES SIMULATED (5 EMBANKMENTS ON A UNIFORM ARIAKE CLAY)

Analysed are the settlement of the ground surface, pore water pressure generated and lateral displacement during construction of 5 pre-load embankments placed at Takeo-Kitagata interchange of Nagasaki Highway shown in Figs.1 and 2. The clay layer is so uniform that the soil parameters must be identical in all the cases of 5 embankments. Therefore it is practically impossible to intentionally select the soil parameters of Ariake Clay in such a way that computed results match well with the mechanical quantities monitored at each of 5 embankments.

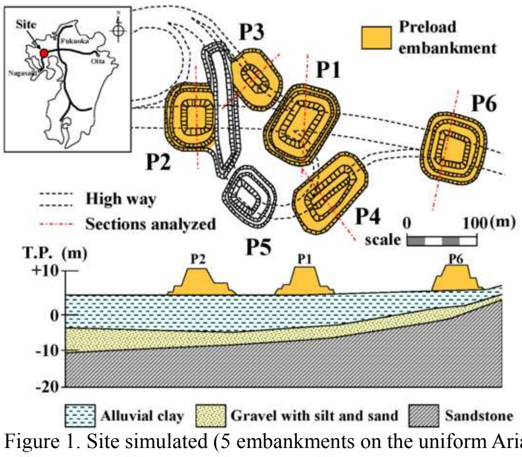


Figure 1. Site simulated (5 embankments on the uniform Ariake Clay)

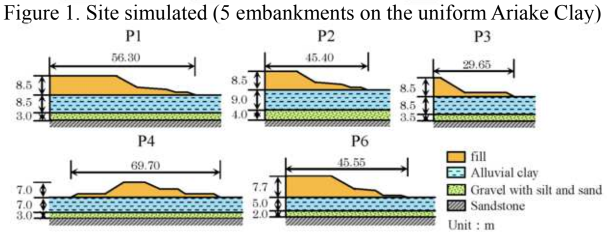


Figure 2. Cross-sections of pre-loading embankments (Embankment P5 did not settle due to installation of cement-mixed columns)

3 SOIL/WATER COUPLED FINITE ELEMENT PROGRAMME (DACSAR) INCORPORATING AN ELASTO-VISCOPLASTIC CONSTITUTIVE MODEL PROPOSED BY SEKIGUCHI AND OHTA

In this investigation, employed is a programme called DACSAR incorporating an elasto-viscoplastic constitutive model proposed by Sekiguchi and Ohta (1977). The outline of this constitutive model is shown in Fig.3.

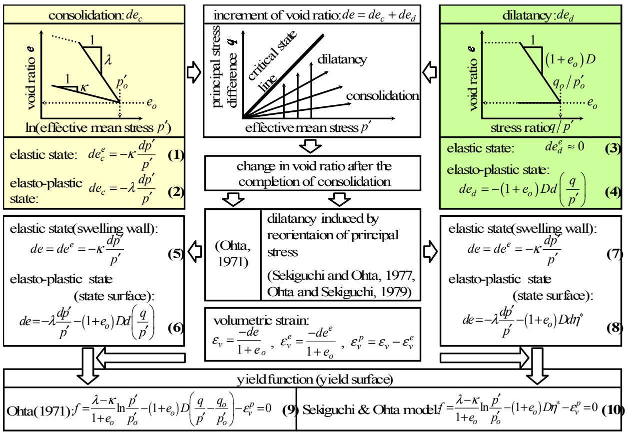
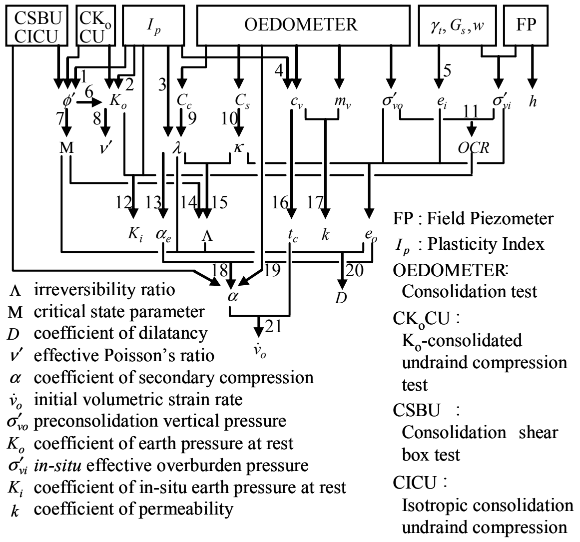


Figure 3. Outline of the model proposed by Sekiguchi and Ohta(1977)

As widely known, the change in void ratio due to loading under conditions of constant stress ratio q/p' is derived as Eqs.(1) and (2) in Fig.3 from the e - $\ln p'$ linear relationship. It is assumed that the change in void ratio represented by the normal consolidation line is elasto-plastic while the change in void ratio represented by swelling line is elastic. Change in void ratio de_c due to isotropic and/or anisotropic consolidation is described by $de_c = de_c^e + de_c^p$, where the subscript c denotes the component due to consolidation, the super scripts e and p denote elastic and plastic components. Change in void ratio de_d under conditions of constant p' is called dilatancy (or contractancy) and is assumed to be irreversible as shown by Eqs.(3) and (4), where D is coefficient of dilatancy proposed by Shibata (1963) and the subscript d denotes dilatancy. Ohta (1971) assumed that change in void ratio consists of components due to consolidation (Eqs.(1) and (2)) and dilatancy (Eqs.(3) and (4)) resulting in the total change in void ratio described by Eqs.(5) and (6). Eqs.(5) and (6) are integrated with the pre-consolidated state (e_0 , p'_0 , q_0) as the boundary conditions. The yield function f is obtained as Eq.(9) in which the plastic volumetric strain ε_v^p plays a role of hardening parameter while the volumetric strain ε_v is defined as $\frac{e_0 - e}{1+e_0}$. Eq.(9) is extended and generalized as Eq.(10) by Sekiguchi and Ohta (1977). The governing equations of a soil/water coupled initial/boundary value problem are (i) equations for soil skeleton (equilibrium equation, constitutive equation and equation describing strain-displacement relationship), (ii) equations for the pore water (continuity equation and Darcy's law) and (iii) an equation for both of soil and pore water (definition of effective stress). The constitutive model (Eq.(10)) is incorporated in DACSAR as the stiffness matrix derived by spatial and time discretization of the weak form of equations of equilibrium and continuity. The construction sequence is described in the computer simulation as the change of boundary conditions and hence is required to be specified in detail close to the actual construction sequence.

4 ESTIMATE OF INPUT PARAMETERS (ESTIMATE BASED ON PLASTICITY INDEX I_P)

The soil parameters needed in the constitutive model proposed by Sekiguchi and Ohta (1977) have to be principally obtained from a set of laboratory tests which are not practical enough to carry because of the time and cost required. Iizuka and Ohta (1987) proposed the procedure of estimating soil parameters based primarily on the conventional soil testing methods and the plasticity index I_p as summarized in Fig.4.



1. $\sin\phi' = 0.81 - 0.233\log I_p$ estimated from data collected by Kenney (1959)
2. $K_o = 0.44 + 0.42 I_p \times 10^{-2}$ Massarsch (1979)
3. $\lambda = 0.015 + 0.007 I_p$ Iizuka & Ohta (1987)
4. $\log c_v = -0.025 I_p - 0.25 \pm 1$ (cm²/min) Iizuka & Ohta (1987)
5. $e_i = wG_s$
6. $K_o = 1 - \sin\phi'$ Jaky (1944)
7. $\mathrm{M} = 6\sin\phi'/(3 - \sin\phi')$
8. $\nu' = K_o/(1+K_o)$
9.10. $\lambda = 0.434 C_c$, $\kappa = 0.434 C_s$
11. $OCR = \sigma'_{vo}/\sigma'_{vi}$
12. $K_i = K_o(OCR)^{0.54\exp(-I_p/122)}$ Alpan (1967)
13. $\alpha_e/\lambda = 0.05 \pm 0.02(clay)$ Mesri and Godlewski (1977)
13. $\quad = 0.07 \pm 0.02(peat)$
14. $\Lambda = \mathrm{M}/1.75$ Karube (1975)
15. $\Lambda = 1 - \kappa/\lambda$
16. $t_c = H^2 T_v(90\%)/c_v$ Sekiguchi
17. $k = m_v c_v \gamma_w$
18.19. $\alpha = \alpha_e/(1+e_o)$, $\alpha = dc_v/d(\ln t)$ Sekiguchi (1977)
20. $D = d\varepsilon_v/d(q/p')$ Shibata (1963) $D = \lambda\Lambda/\{\mathrm{M}(1+e_o)\}$ Ohta (1971)
21. $\dot{v}_o = \alpha/t_c$ Sekiguchi (1977)

Figure 4. Determination procedure of the input parameters

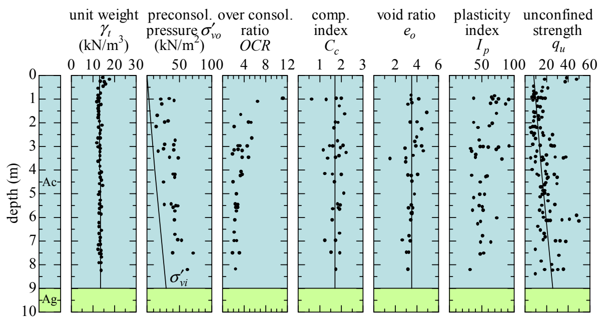


Figure 5. Subsoil characteristics used in input parameter determination

The procedure can be used, if necessary, as a quick parameter determination method based only on the plasticity index I_p.

As the first trial in this investigation the input parameters are estimated based on the data summarized in Fig.5 using the procedure shown in Fig.4. The parameters estimated from the information obtained by oedometer tests and plasticity index are listed in Table 1 to be used in the analyses for all of 5 embankments.

At the site investigated, the paper drains (square spacing, 0.8m pitch) were installed to accelerate the consolidation settlement due to embankment loading. The installation of paper drains is supposed to have resulted in 30% of reduction of coefficient of consolidation c_v due to disturbance. Adopting the method proposed by Yoshikuni (1979), the apparent increase in coefficient of consolidation due to the shortening of drainage length is estimated in such a way that 50% of consolidation time both of Barron's horizontal-radial water flow and Terzaghi's one dimensional vertical water flow become to coincide. Table 2 summarizes the factor F_p to be multiplied to the coefficient of permeability for each pre-loading embankment.

Table 1: Summary of input parameters

layer	depth (m)	γ_t (kN/m^3)	OCR	I_p	M	Λ	D	K_0
fill		20.0	E=25000kN/m^2					
Ac1	0.0-1.0	13.0	E=5000kN/m^2					
Ac2	1.0-2.5	13.0	4.0	60	0.91	0.52	0.094	0.69
Ac3	2.5-4.0	13.0	3.5	50	0.96	0.55	0.094	0.65
Ac4	4.0-5.0	13.5	3.0	50	0.96	0.55	0.094	0.65
Ac5	5.0-7.0	13.5	2.5	50	0.96	0.55	0.094	0.65
Ac6	7.0-9.0	13.5	2.5	50	0.96	0.55	0.094	0.65
Ag		16.0	E=60000kN/m^2					

layer	depth (m)	K_i	ν'	λ	e_0	α	v_0 (1/day)	k (m/s)
fill								1.16E-02
Ac1	0.0-1.0							1.67E-06
Ac2	1.0-2.5	1.31	0.41	0.74	3.51	8.18E-03	1.64E-05	1.67E-06
Ac3	2.5-4.0	1.02	0.39	0.74	3.51	8.18E-03	1.64E-05	1.67E-06
Ac4	4.0-5.0	0.91	0.39	0.74	3.51	8.18E-03	1.64E-05	1.67E-06
Ac5	5.0-7.0	0.88	0.39	0.74	3.51	8.18E-03	1.64E-05	1.67E-06
Ac6	7.0-9.0	0.85	0.39	0.74	3.51	8.18E-03	1.64E-05	1.16E-06
Ag								1.16E-03

Table 2: Factor to be multiplied to coefficient of permeability

pre-load	P1	P2	P3	P4	P6
thickness of clay layer (m)	8.0	9.0	8.5	7.0	5.0
factor F_p	**4.1**	**7.7**	**5.8**	**4.4**	**2.3**

5 RESULT OF ANALYSIS

A series of computer simulation is carried out in 4 steps by using 4 sets of input data stepwisely getting closer to the reality of the construction work.

5.1 *First step (Input parameters listed in Table 1)*

The results of computer simulation on 5 embankments performed by using a set of input parameters listed in Table 1 are shown in Figs.6 through 10 respectively for 5 embankments P1, P2, P3, P4 and P6.

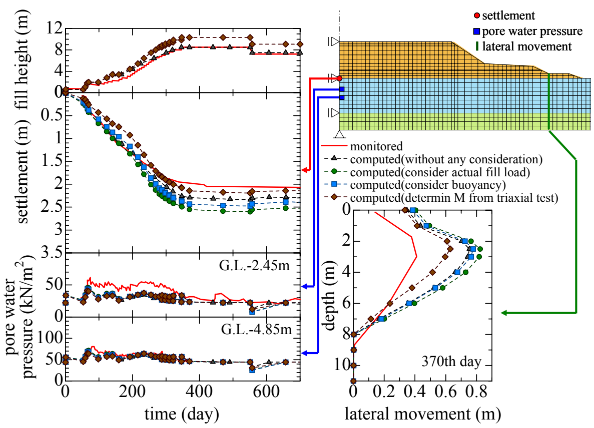


Figure 6. Monitored and computed performance of P1

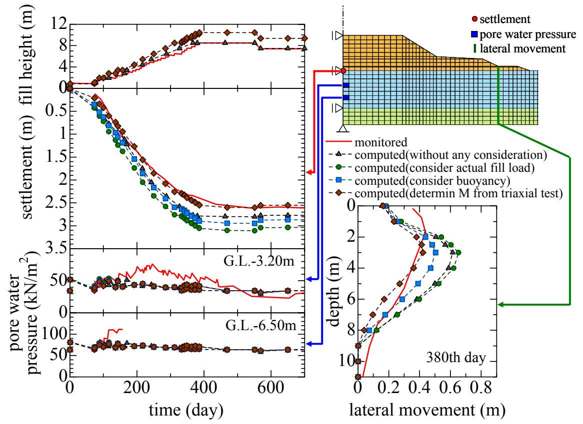


Figure 7. Monitored and computed performance of P2

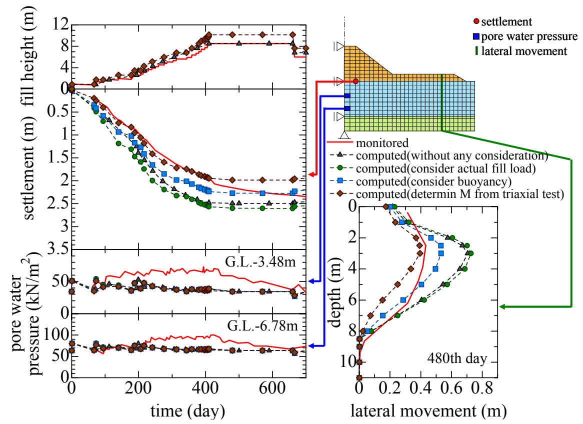


Figure 8. Monitored and computed performance of P3

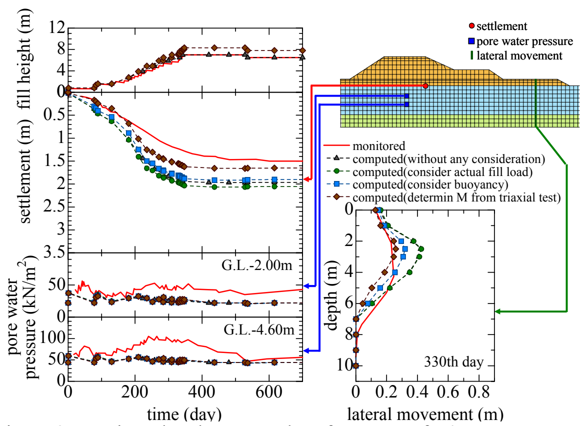


Figure 9. Monitored and computed performance of P4

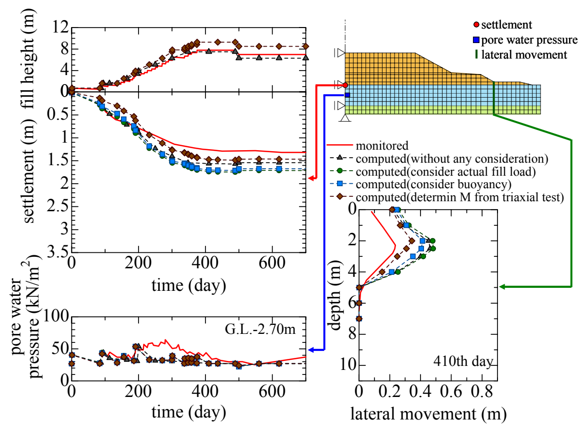


Figure 10. Monitored and computed performance of P6

In each of these figures, the mesh in vicinity of the embankment is shown at the upper right. The region of mesh formation actually used in the analysis is much wider than these figures in order to eliminate the influence of the boundary on computed results.
The graph at the upper left shows the input fill height of the embankment and the monitored fill height. The other two graphs in the lower part of each figure compare the computed values with monitored values of settlement, pore water pressure and lateral movement at the positions shown in figure of mesh. Results of the first step simulation are shown by ▲, indicating tolerable agreement between computed (▲) and monitored (represented by a curve without plots) values despite relatively simple procedure of the parameter estimation.

5.2 *Second step (Analysis considering actual fill load)*

The height of the fill is measured relative to the original ground surface and is not directly related to the thickness of the fill that is the height of the fill being added by the settlement of the fill. In the first step of the simulation, the fill load is calculated by simply multiplying the unit weight of the fill body and fill height together. In this second step the actual fill thickness is taken as the load rather than the fill height resulting in the computed values shown by ●. It is understandable that both of the computed settlement and the computed lateral movement are larger than the results ▲ obtained in the first step.

5.3 *Third step (Analysis considering buoyancy)*

The pore water expelled during consolidation was drained through the net work of the drainage channel being combined with shallow wells, however after the pumps in the wells stopped working, the ground water level recovered and came back to the original height of G.L.±0.0m resulting in the additional effect of buoyancy that reduces the fill load. Taking the effect of recovery of ground water level, the third step of the simulation is carried resulting in the computed values shown by ■ which are closer to the monitored values except in the case of P1 embankment.

5.4 *Fourth step (ϕ' -parameter estimated from triaxial test)*

In the previous steps ϕ'-value of Ariake Clay is estimated as ϕ' = 23.3 and 24.5 (M =0.91 and 0.96) using Eq.1 in Fig.4 based on the plasticity index I_p because triaxial tests were not carried out on undisturbed samples taken from the site.
Matsumura and Miura (1990) reviewed the critical state parameter M of Ariake Clay sampled from various sites and concluded that M =1.065 ～ 1.622. In the fourth step of simulation, the authors assume that M =1.4 for Ariake Clay at the site and obtain the results shown by ◆ which are much closer to the monitored values compared with the computed results in the previous steps for all cases of P1 to P6. The excessive estimate of the lateral movement of the soft foundation loaded by an embankment was widely experienced by many research workers triggering discussions about the proper estimate of material parameters especially those related to anisotropy of the clay as well as the discussions on the actual reliabillty of the lateral displacements monitored by inclinometers. In the fourth step of the simulation, the increase in M successfully but unexpectedly results in much better agreement between monitored and computed values. This suggests that the general trend of estimating excessive lateral movement might have been generated from modelate estimate of shear resistance of clays and encourages more frequent usage of triaxial tests.

6 CONCLUSIONS

It is confirmed that computer simulation using the finite element programme DACSAR is primarily reliable enough to be used in engineering practice because the choice of input parameters closer to the reality both of material parameters and boundary conditions results in the computed values closer to the monitored values of settlement, lateral movement and pore water pressure during and after construction works.

It should be noted here that the first step of simulation in this paper has already been reported by Ohta et al. (1991) demonstrating the usefulness of the plasticity index I_p in parameter determination. This paper provides further information how the parameter estimation can be made closer to the reality and what the consequence of better choice of parameters is.

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