Settlement Prediction of Soft Clay Ground under Sustained and Transient Loading

by

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Two case studies of settlement prediction of Ariake clay ground which is counted as one of the soft clays in Japan are described. The one of them is to report a long-term settlement which have been observed over 25 years since construction of embankment for breakwater on the coastal Ariake deposit. The another case study is concerned with the settlement of low embankment highway on Ariake clay whose shallow surface was improved by quicklime-clay mixture as a countermeasure for the settlement. It is featured by the fact that the predominant secondary settlement is common with two case studies.

The finite element method using an elasto-plastic model was adopted to analyze the settlement of Ariake clay observed in above-mentioned two case studies under sustained and transient loading, respectively. It is concluded from comparison of analytical results with observed settlement that the proposed model with consideration of secondary compression is advantageous for settlement prediction of soft clay ground.

1. Introduction

A soft marine alluvial deposit called the Ariake clay is sedimented along the Ariake Sea in Japan. The Ariake clay is well-known as one of the most problematic soils in Japan, because of its high sensitivity, high compressibility and low bearing capacity. Earth and building structures founded on the Ariake clay ground have frequently lost their functions because of the differential settlement which have had harmful influences on the environs such as the residential area.

It has called attention of engineers that there are some case studies in which low embankment highways suffer from unpredictably abnormal settlement which may be induced by traffic loading (Yamanouchi and Yasuhara : 1975). Hence, in order to maintain the evenness of pavement on the Ariake clay ground, the overlay of pavement surface has been done repeatedly every year. A predication method for settlement is therefore necessary as well as a countermeasure for the settlement to make every structure fulfill its sufficient function.

The present paper first describes the geotechnical properties of the Ariake clay. Then, two case studies were introduced on the settlement of embankment on the Ariake clay ground : one refers to the long-term settlement over 25 years after completion of embankment and construction of a breakwater on the Ariake coastal area, and the another case study is concerned with the influence of traffic loads

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on the settlement of low embankment highway on the Ariake clay ground.

It is concluded from the field investigation that long-term settlements observed in both case studies are caused by secondary compression. The analytical method for predicting these long-term settlements of the Ariake clay ground was proved to be available under both sustained loading and cyclic loading. The method consists of incorporating the effects of secondary compression, time-dependency and loading conditions into an elasto-plastic model proposed by the first author.

2. Geotechnical Properties of the Ariake Clay

The Ariake clay sedimented mainly in Saga Plain which lies north of the Ariake Sea located in central Kyushu (Fig. 1), is to be as one of the most soft clay in Japan. The Ariake clay layer is sedimented in general 15~20 m thick. The natural water content of the clay is mostly higher than its liquid limit and N value is usually zero. The mechanical proporties of the Ariake clay is summarized in Table 1 (Onitsuka ; 1983). The sensitivity ratio is above 16, and sometimes exceed 100.

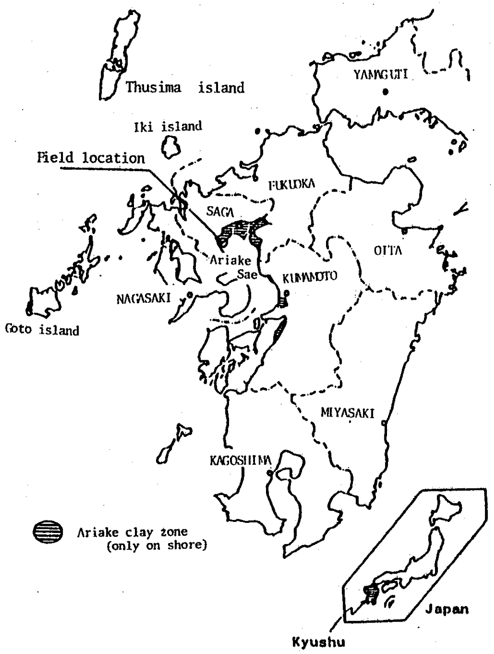


Fig. 1 Distribution of the Ariake Clay Deposits

Table 1 Mechanical Properties of the Ariake Clay

Compression index C_c	$C_c = 0.49\ (e_n - 0.41)$ $C_c = 0.013\ (w_L - 10)$		
Coefficient of volume compressibility m_v (1/kPa)	Consolidation pressure		
	$p<200$kPa	$p>200$kPa	$p=Pc$
	10^{-3}	10^{-4}	10^{-3}
Coefficient of consolidation C_v (cm²/d)	$3.5\times10^{1}-1.5\times10^{3}$		
Unconfined compressive strength q_u (kPa)	Upper layer	Lower layer	
	3—30	30—100	
Strain at failure in unconfined compression test ε_f (%)	2—4		
Constants a, b ($q_u = a + bz$)	a : 40—160 b : 14— 36		
Rate of strength increase Cu/p	$\frac{1}{3}$		
Sensitivity ratio S_t	> 8, >16 (most of Ariake clay)		

(Onitsuka. K ; 1983)

3. Elasto-Plastic Constitutive Model with Time-Dependency

3.1 Constitutive Model

An elasto-plastic constitutive model was derived by the first author (Tanabashi : 1984. b and 1985) on the basis of the postulate that a soil is a strain hardening material for consolidation and shear. This model considers the following mechanical properties of clay.

i) The compression index, $C_c\ (= 2.3\lambda)$, and swelling index, $C_s\ (= 2.3\kappa)$, observed in e-log p' curve are assumed to be constant independently of the isotropic and anisotropic consolidation.

$$dv_c = \frac{\lambda}{1+e}\frac{1}{p'}dp', \quad dv_c^{e} = \frac{\kappa}{1+e}\frac{1}{p'}dp' \qquad (1)$$

where, v_c : volumetric strain due to compression and superscript (e) means elastic component.

ii) Volumetric strain due to dilatancy is linearly related to the effective stress ratio ($\eta = q/p'$)

(Shibata ; 1963).

$$dv_d = dv_d^p = \mu d\eta \qquad (2)$$

where, p' : mean effective stress, q : octahedral shear stress, η : octahedral stress ratio πv_d : volumetric strain due to dilatancy and superscript (p) means plastic component.

iii) Incremental plastic strain rate is given by the linear equation of effective stress ratio as

$$q/p' = M_o - N_o \frac{dv_d^p}{d\gamma_d^p} \qquad (3)$$

where, γ_d : octahedral shear strain

iv) The volumetric strain versus elapsed time relaton is approximated by the logistic curve.

$$1/dv_c = R + a \cdot b^{\log t/\log 2} \qquad (4)$$

v) Both time-dependent plastic shear and volumetric strains are expressed by the modified Singh-Mitchell's general equation (1968) which was proposed by Yasuhara (1979).

It is characterized that the elasto-plastic model used for two-dimensional deformation analysis may take the effect of time-dependent deformation and the scale effect of clay layer into account (Tanabashi et al., 1984. a). The model was formulated as

$$\begin{bmatrix} dv \\ d\gamma \end{bmatrix} = \begin{bmatrix} S_c^e & S_d^e \\ 0 & S_s^e \end{bmatrix} \begin{bmatrix} dp \\ dq \end{bmatrix} + \begin{bmatrix} S_c^{vp} & S_d^{vp} \\ 0 & S_s^{vp} \end{bmatrix} \begin{bmatrix} dp \\ dq \end{bmatrix} = \begin{bmatrix} S_c & S_d \\ 0 & S_s \end{bmatrix} \begin{bmatrix} dp \\ dq \end{bmatrix} \qquad (5)$$

where S_c, S_d and S_s are given by

$$S_c = S_c^e + S_c^{vp} = \frac{1}{1+e} \cdot \frac{1}{p} \left[\kappa + (\lambda - \kappa) \frac{k + a \cdot b^{\log t_{fc}/\log 2}}{k + a \cdot b^{\log t/\log 2}} \right] \qquad (6\text{-a})$$

$$S_d = S_d^e + S_d^{vp} = \frac{1}{1+e} \cdot \frac{1}{p} \{0 + \mu (t/t_{fd})^{1-m_d}\} \qquad (6\text{-b})$$

$$S_s = S_s^e + S_s^{vp} = \frac{1}{p} \left\{ \nu + \frac{\mu}{1+e} \cdot \frac{N_0}{M_0 - \eta} (t/t_{fs})^{1-m_s} \right\} \qquad (6\text{-c})$$

where λ, κ, μ, ν, M_0 and N_0 are elasto-plastic parameters, R, a, b, m_d and m_s are time effect parameters. t_{fc}, t_{fd} and t_{fs} in eq. (6) are defined by

$$t_{fc} = (H_e/H^\star)^{n_c} \cdot t^\star \qquad (7\text{-a})$$

$$t_{fd} = (H_e/H^\star)^{n_d} \cdot t^\star \qquad (7\text{-b})$$

$$t_{fs} = (H_e/H^\star)^{n_s} \cdot t^\star \qquad (7\text{-c})$$

where n_c, n_d and n_s are scale effect parameters and $H^\star$ is effective drainage distance of clay sample, H_e is effective drainage distance of each element, $t^\star$ is time measured at each load increment in laboratory tests.

3.2 Determination of Parameters

The parameters involved in the constitutive model are elasto-plastic, time-dependent and scale effect parameters. Those were determined by the results from isotropic consolidation tests and p'-constant drained triaxial tests on undisturbed samples. The undisturbed Ariake clay taken from the site of embankment was used for both tests. Index properties of the clay are: $G_s = 2.60$, $W_L = 100\%$, $I_p = 55$, $e_i = 3.50$ and $W_i = 140\%$. The parameters determined are listed in Table 2.

Table 2 Parameters Determined by Two Triaxial Comp. Tests

elasto-plastic parameters		time parameters	
C_c	1.15	R	8.99
C_s	0.126	a	111.
		b	0.614
μ	0.298	m_d	0.769
		m_s	0.792
ν	0.011	scale effect parameters	
M_0	0.710	n_c	1.00
		n_d	0.50
N_0	0.295	n_s	0.50

4. Case Study on Long-Term Settlement under Sustained Loading

4.1 Profile of the Location Site

The case study adopted in this paper is an example of the embankment for breakwater constructed on the Ariake clay ground in Kashima-cho, Saga prefecture (shown in Fig. 1). The cross sectional view of the embankment is shown in Fig. 2. The primary embankment was constructed in 1960, and secondary raised embankment was constructed in 1971 under slow construction about 1 year. The observation of settlement was continued from 1971 to 1985 following about 14 years. And the settlement by the primary embankment was confirmed about 1.5m in 1971 before construction of the secondary raised embankment.

Geotechnical properties of this site at the Ariake clay ground are summarized in Fig. 3 and Table 3. These show in comparison on the original condition before execution with after construction of the primary embankment.

4.2 Analytical Procedure

For simplicity of numerical analysis, the Ariake clay ground was subdivided into five layers according to variation of characteristics of geotechnical properties with depth as is given in Fig. 3. The in-put index parameters determined by the results from laboratory and field investigation are listed in Table 3. The banking loads were simulated by subdivided filling with 9 stages in accordance with the execution works for the last 25 years.

4.3 Comparison between Observed and Calculated Settlements

(1) Conventional one-dimensional settlement analysis

Fig. 4 shows the comparison between the calculated settlement by using the conventional one-dimensional method and the observed settlement of soft ground. A family of calculated settlement versus time curves were drawn by changing into 1 through 10 times as the coefficient of consolidation, C_v, obtained by standard oedometer tests on undisturbed Ariake clays. The in-situ observed rate of settlement is inclined to be kept higher than that predicted by the results from oedometer tests. Besides, settlement still continues as secondary compression.

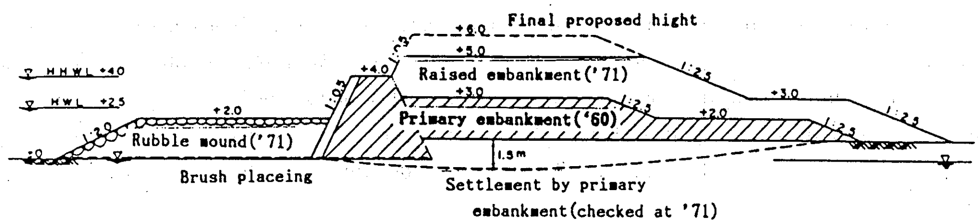


Fig. 2 Profile and Earthwork of the Embankment for Breakwater on the Ariake Clay Ground

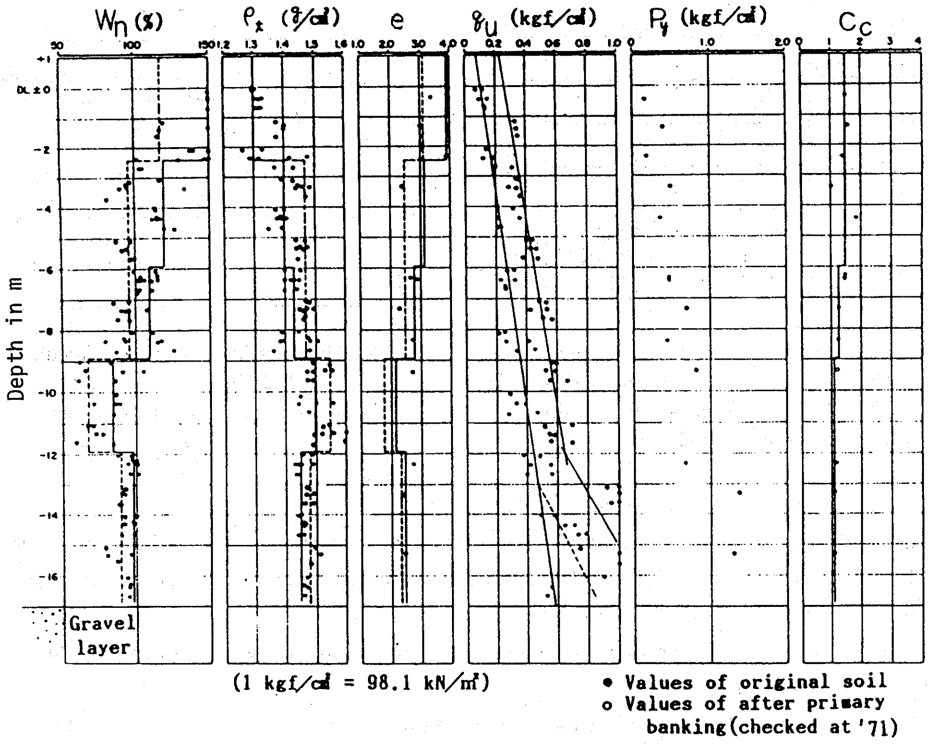


Fig. 3 Geotechnical Properties of the Ariake Clay Ground

Table 3 Soil Properties at the Site (Averaged)

Condition	Original soil	Before raised banking ('71)
Water content w_n (%)	112	94
Wet density ρ_t (g/cm³)	1.42	1.47
Void ratio e	2.89	2.60
Unconfind compressive strength q_u (kgf/cm²)	$0.028 \cdot Z + 0.10$	$0.032 \cdot Z + 0.26$
Consolidation yield stress p_y (tf/m²)	$0.45 \cdot Z + 1.5$	$0.52 \cdot Z + 3.0$
Compression index C_c	1.28	1.28
Coefficient of consolidation c_v (cm²/day)	100	100

(1 kgf/cm² = 98.1kN/m²) Z ; Depth in m

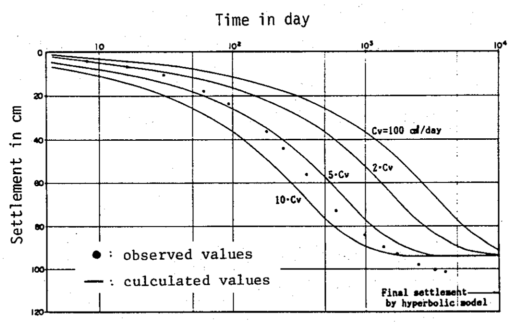


Fig. 4 Time-Settlement Curves Obtained by Conventional 1-Dimensional Method with Observed Values (for Raised Embankment after 1971)

(2) Two-dimensional settlement analysis by elasto-plastic model

Fig. 5 shows the time-settlement curves obtained by the observed values and the calculated values by two-dimensional finite element analysis, for raised embankment after 1971 as same as in Fig. 4.

Although the predicted settlement-time curve seems to slightly over-estimate the secondary compression, better agreement is recognized in comparison between the calculated and the observed settlement than that predicted by the conventional analysis.

Fig. 6 shows the comparison of calculated settlement versus time relation at the center of embankment with the observed variation of settlement with time for primary embankment after 1960. Good agreement was recognized in comparison between computation and observation of settlement, 150cm,

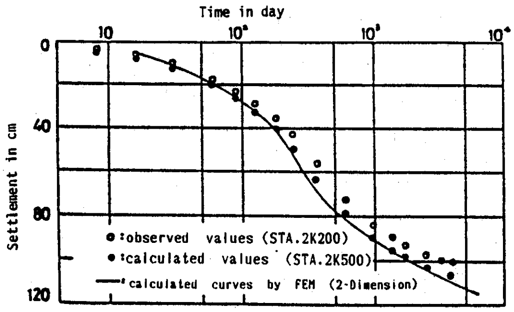


Fig. 5 Time-Settlement Curves Calculated by 2-Dimensional Finite Element Analyses with Observed Values (for Raised Embankment after 1971)

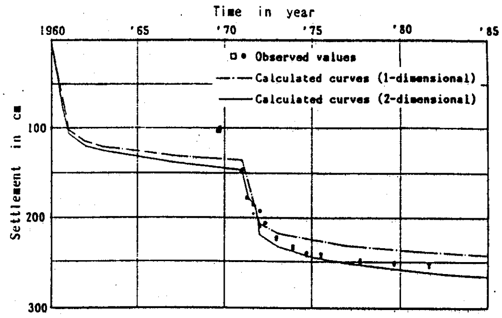


Fig. 6 Time-Settlement Curves Calculated by 1 and 2-Dimensional Finite Element Methods with Observed Values

at 1971 before the execution of the raised embankment.

The results of the finite element analysis were illustrated in Figs. 7 (a), (b) and (c) which were in accordance with behavior of Ariake clay ground in 1961, 1971 and 1985, respectively. We may recognize the indications from Fig. 7 as follows:

i) The settlement due to the raised embankment after 1971 is more predominant than that due to the primary embankment after 1960.

ii) A maximum settlement occurs at the concrete retaining wall due to the primary embankment after 1961, meanwhile it gradually spreads towards the center of the embankment due to the raised one after 1971.

(3) Final Settlement

Final settlements are 94cm and 111cm which are in accordance with the calculated values by a conventional C_c-method and a hyperbolic model, respectively. On the other hand, the predicted final settlement until 2005 D. C. up to 137cm in case of adopting the two-dimensional finite element analysis.

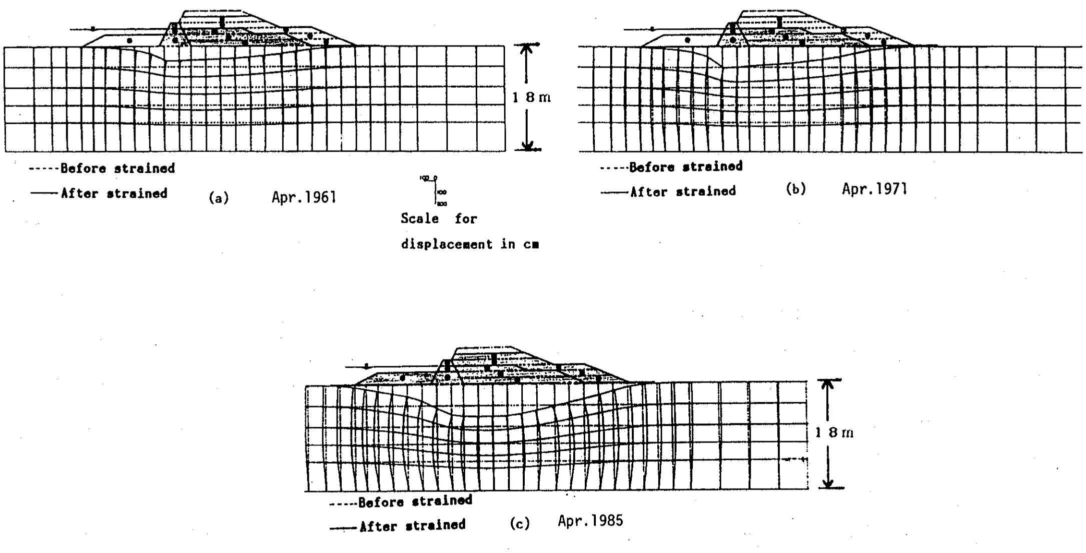


Fig. 7 Cross Sectional Deformation Calculated by the Finite Element Analysis

5. Case Study on Traffic-Induced Settlements

5.1 Profile of the Location Site

The Kohoku-bypass 34 is situated in Kohoku-town of Saga in Japan which intersects the National highway 202. Both sides of the bypass are spreaded by the field of which subsoils consist of soft alluvial and sometime sensitive clay. In a part of the Kohoku-bypass with 1300m in distance the 1m depth of clay grounds was improved by stabilization layer with quicklime whose width is narrower by 2m than the width of embankment. The height of the embankment in 1.95m with inclusion of 20cm pavement layer which was constructed after the settlements of the embankment was almost finished. The embankment and the ground were modeled for finite element analysis as shown in Fig. 8.

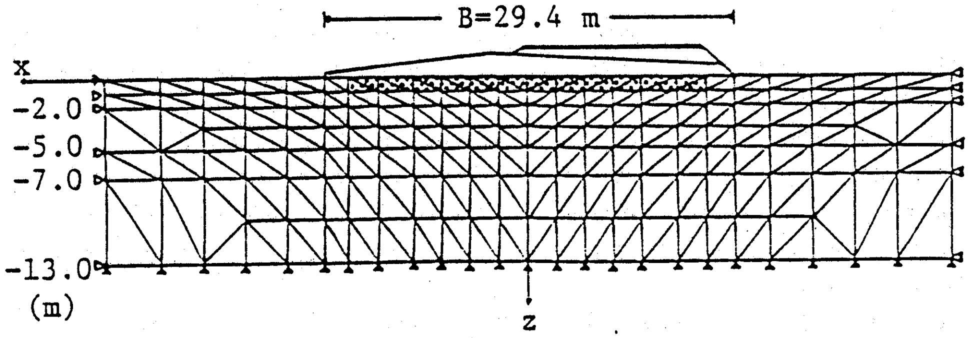


Fig. 8 Analytical Model for Traffic-Induced Settlement

5.2 Analytical Procedure

In application of the elasto-plastic model used for deformation analysis of clay under ordinary static loading to clay behaviour under traffic-induced cyclic loading, it is essential to estimate the traffic load acting on the low embankment and ground. It is assumed in this paper that the settlements of low embankment highway on clay after opening to traffic should be induced by traffic loads as a result of secondary time effects. The cyclic effect due to traffic loading was considered in the terms to give the time-dependent volumetric strain in the constitutive model.

Fig. 9 is the key sketch to illustrate the cyclic effect in the observed settlements plotted to elapsed time. Let us consider the case that the clay is consolidated by the embankment until an arbitrary time and then is open to traffic loads. At first the settlement-time curve under embankment is represented by curve I. Settlement after t_1 increases with time due to traffic-induced additional load. Thus the

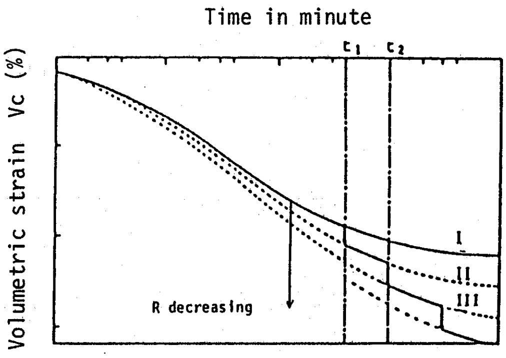


Fig. 9 The Key Sketch to Illustrate the Cyclic Effect

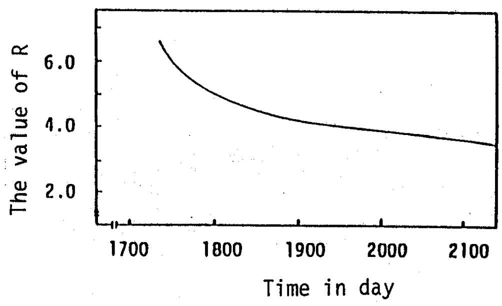


Fig. 10 Relationship between Time and the Value of R for the Finite Element Analysis

time-settlement relation moves from curve I to curve II at t_1. Further, as cyclic loading conditions, the settlement may shift into curve III, for instance. These relations of time-settlement relations such as curves I, II and III under different loading circumstances can be represented by varying the value of R in Fig. 10. The process where the time-settlement relations vary from curve I to curve III as was shown in Fig. 9 is simulated by making the value of R decrease with time.

5.3 Analytical Results

The calculated settlements versus elapsed time relations are compared in Fig. 11 with the observed ones at the center of embankment. In the field, the first banking started in 1977 and the 2nd and 3rd banking succeeded to in April 1980 and December 1981, respectively. Since then, the highway was open to traffic. The predominant settlement in this stage may be caused by the secondary compression due to traffic loads.

The effect of traffic loads was considered in deformation analysis by changing the value of R given by Fig. 10. The predicted settlements by means of the elasto-plastic model seem to under-estimate the observed ones as shown in Fig. 11.

Further investigation on the effect of traffic loads on settlements were carried out by comparison between calculated and observed settlements after opening to traffic in 1981. It can be seen from Fig. 12 that both are in considerably good agreement with each other. The calculated settlements of clay due to static banking loads is illustrated as well in Fig. 12 which indicates the predominance of cyclic settlements to static settlements.

Fig. 13 shows the analytical results of distribution of principal stresses and maximum shear stress in the clay ground under traffic loading whose surface is improved by quicklime stabilization of 25.4m

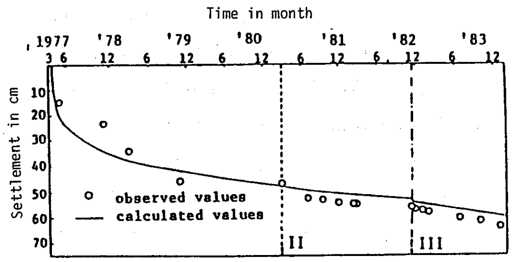


Fig. 11 Comparison between Observed and Calculated Time-Settlement Curves after Completion of the Embankment

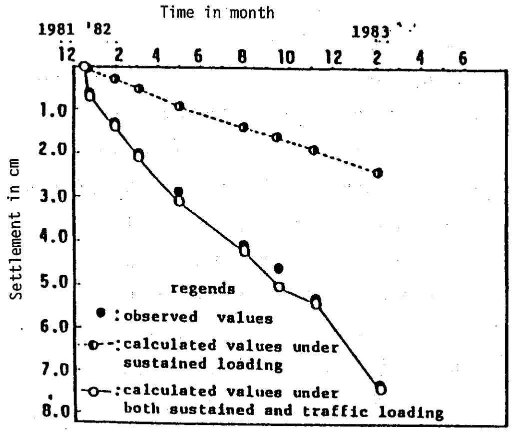
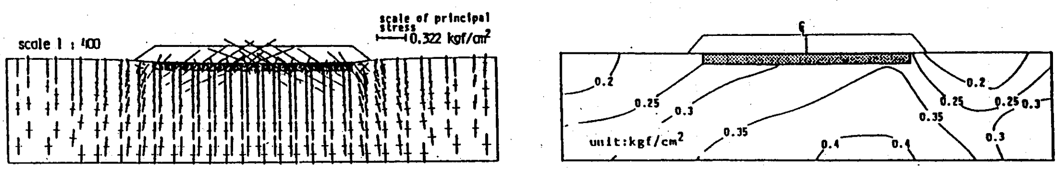


Fig. 12 Comparison between Observed and Calculated Time-Settlement Curves after Opening to Traffic



(a) Principal Stress Distribution (b) Maximum Shear Stress Distribution

Fig. 13 The Results of the Finite Element Analysis in width and 1.0m in depth. The only one side lane (left side of Fig. 13 (b)) of this highway is open to traffic at this moment.

Fig. 13 (a) points out that principal stress distribution remains homogeneous beneath the stability layer because it supports the applied load.

6. Conclusions

The Ariake clay deposit has been considered as a highly plastic clay with a high water content and high sensitivity. This feature of property causes long-term settlement in the field so-called secondary compression under sustained loading as well as under cyclic loading.

An elasto-plastic constitutive model with consideration of the time-dependent deformation was adopted to explain the time-settlement records observed in case studies at the Ariake clay in Japan. The observed settlement versus elapsed time relation in the embankment for breakwater founded on the Ariake clay ground was compared with the calculated results, using this model. Better agreement was recognized in comparison between computed and observed settlements.

In extension of the constitutive model for prediction of settlements of clay under repeated loading, the effect of traffic induced cyclic loading was replaced by the equivalent static load. Besides, it was assumed that the traffic-induced settlement of soft ground is primarily governed by consolidation under repeated loading. By taking these cyclic effects into consideration, the proposed model successfully explained the variation of settlement with time observed settlement of low embankment highway on the Ariake clay layer.

From the analysis of two case studies, it is proved that the proposed model is useful for predicting the time-dependent settlements of soft grounds which potentially exhibits long-term secondary compression.

Acknowledgments

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