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SYNOPSIS: Two case histories on long-term settlements of Ariake clay which is counted as one of the soft clays in Japan are described. The one of them is to report the settlement which have been observed over 25 years since construction of embankment for breakwater on the coastal Ariake deposit. The another case history is concerned with the settlement of low embankment highway on Ariake clay whose shallow surface was improved by quickline-clay mixture as a countermeasure for the settlement. The current paper is featured by the fact that the predominant secondary settlement is common with two case histories.
The finite element method using an elasto-plastic model was adopted to analyze the settlement of the Ariake clay observed in the above-mentioned two case histories 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 long-term settlement prediction of soft clay.

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 histories 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 histories were introduced on the long-term 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 history is concerned with the influence of traffic loads 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 histories are caused by secondary compression. The analytical method predicting for these long-term settlements of the Ariake clay ground was proved to be available under both sustained loading and cyclic loading. The

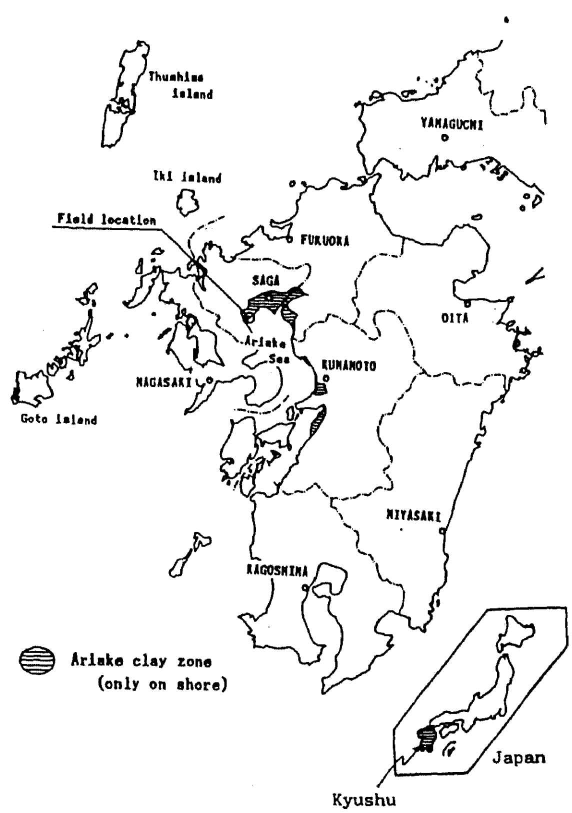


Fig.1 Distribution of thi Ariake Clay Deposits

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 properties of the Ariake clay is summarized in Table 1 (Onitsuka; 1983). The sensitivity ratio is above 16, and sometimes exceed 100.

Table 1 Mechanical Properties of the Ariake Clay

Compression index C_c	$C_c=0.49(e_o-0.41)$ $C_c=0.013(w_n-10)$		
Coefficient of volume compressibility m_v (1/kPa)	Consolidation pressure		
	p<200kPa	p>200kPa	p=Pc
	10[illegible]	10[illegible]	10[illegible]
Coefficient of consolidation c_v (cm²/d)	3.5 x 10[illegible] - 1.5 x 10[illegible]		
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)

ELASTO-PLASTIC CONSTITUTIVE MODEL WITH TIME-DEPENDENCY

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 s oil is a strain hardening material for consolidation and shear.
This model considers the following mechanical properties of clay.
i) The compression index, C_c(=2.3λ), and swelling index,C_s(=2.3κ),observed in e-log p' curve are assumed to be constant independently of the isotropic and anisotropic consolidation.

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

where,Vc : volumetric strain due to compression
ii) Volumetric strain due to dilatancy is linearly related to the effective stress ratio (η = q/p') (Shibata;1963).

$$dV_d = dV_d^P = \mu d\eta \quad (2)$$

where,p': mean effective stress
q : octahedral shear stress
η : octahedral stress ratio
V_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

$$q/p' = M_o - N_o \frac{dV_d^P}{d\gamma_d^P} \quad (3)$$

where, γ_d : octahedral shear strain

iv) The volumetric strain versus elapsed tim relation is approximated by the logistic curv

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

v) Both time-dependent plastic shear an volumetric strains are expressed by th modified Singh-Mitchell's general equatior (1968) which was proposed by Yasuhara (1978).

It is characterized that the elasto-plastic model used for two-dimensional deformatior analysis may take the effect of time-dependent deformation and the scale effect of clay layer into account (Tanabashi et al., 1983) The model was formulated as

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

where S_c,S_d and S_s are given by

$$S_c = \frac{1}{1+e}\cdot\frac{1}{p'}\left[\kappa + (\lambda - \kappa)\frac{k + a \cdot b^{\log t_{fc}/\log 2}}{k + a \cdot b^{\log t/\log 2}}\right] \quad 6\text{-a)}$$

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

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

where λ, κ, μ, ν, M_o and N_o : elasto-plastic parameters, R,a,B,m_d and m_s :time effect parameters are defined by

$$t_{fc} = (H_e/H^*)^{n_c}\, t^* \quad (7\text{-a})$$

$$t_{fd} = (H_e/H^*)^{n_d}\, t^* \quad (7\text{-b})$$

$$t_{fs} = (H_e/H^*)^{n_s}\, t^* \quad (7\text{-c})$$

where n_c, n_d and n_s are scale effect parameters and H*: effective drainage distanc of clay sample, H_e : effective drainage dista of each element, t* : time measure at each loa increment in laboratory tests.

Determination of Parameters

The parameters involved in the constitutive model are elasto-plastic, time-dependent and scale effect parameters were determined by arranging the results from isotropic consolidation tests and p'-constant drained triaxial tests on undisturbed samples. The undisturbed Ariake clay taken from the site for 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	1.145	R	8.985
C_s	0.126	a	111.325
		b	0.6135
μ	0.298	m_d	0.7685
ν	0.011	m_s	0.7920
		scale effect parameters	
M_0	0.710	n_c	1.00
N_0	0.295	n_d	0.50
		n_s	0.50

CASE STUDY ON LONG-TERM SETTLEMENT UNDER SUSTAINED LOADING

Profile of the Location Site

The case adopted in this paper is example of banking, the embankment for breakwater with 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 2. These show in comparison on the original condition before execution with after construction of the primary embankment.

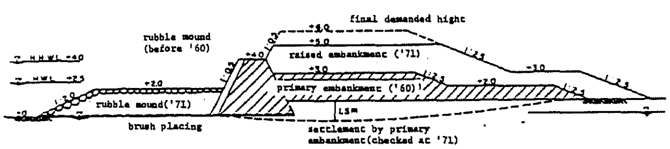
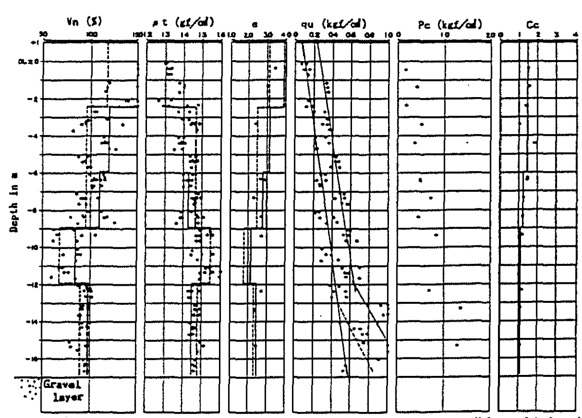


Fig.2 Profile and Earthwork of the Embankment for Breakwater on the Ariake Clay Ground



(1 kgf/cm^2 = 98.1 Mpa)

• Values of before banking
∘ Values of after primary banking(checked at '71)

Fig.3 Geotechnical Properties of the Ariake clay Ground

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.

Table 3 Soil Properties at the Site (Averaged)

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

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

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

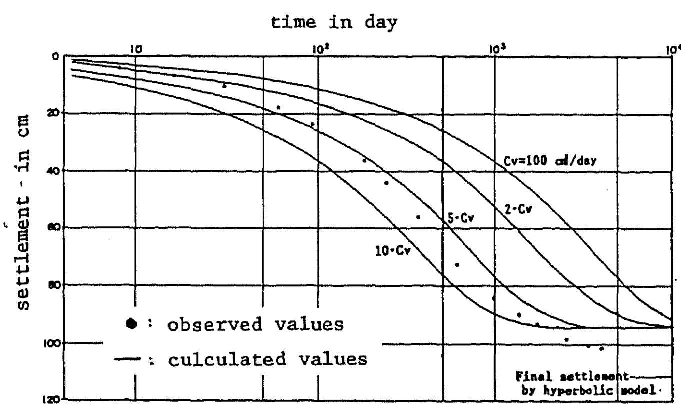


Fig.4 Time-Settlement Curves Obtained by Conventional 1-Dimensional Method with Observed Values (for Raised Embankment after '71)

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.

(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 F.E. analysis, for raised embankment after 1971 as same as 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, 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 :

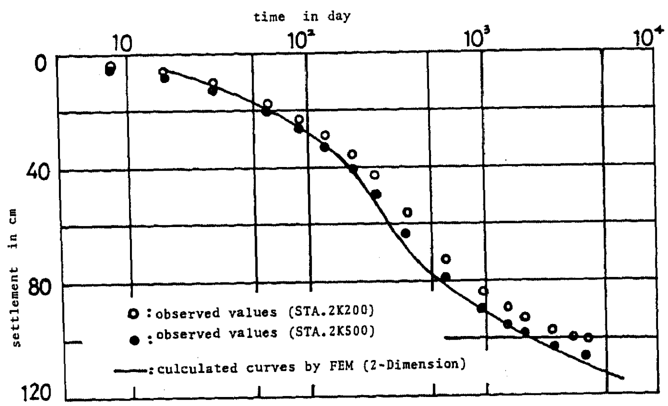


Fig.5 Time-Settlement Curves Calculated by 2-Dimensional F.E. Analyses with Observed Values (for Raised Embankment after '71)

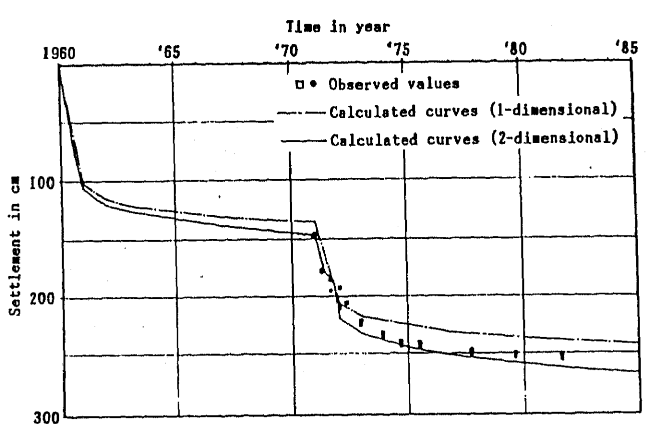


Fig.6 Time-Settlement Curves Calculated by 1 and 2-Dimensional F.E. Analyses with Observed Values

i) The settlement due to the raised embankmen after 1971 is more predominant than that due t the primary embankment after 1960.
ii) A maximum settlement occurs at the concret retaining wall due to the primary embankmen after 1961, meanwhile it gradually spread towards the center of the embankment due to th raised one after 1971.

3) Final Settlement
Final settlements are 94cm and 111cm which ar 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.

CASE STUDY ON TRAFFIC-INDUCED SETTLEMENTS

In-Situ Test Highway

In Japan for various reasons it is necessary tc build roads, when on soft clay ground, with a type of low bank. However after opening tc traffic over a few years,such a type of road or soft clay ground causes an abnormally large residual settlement though the rate is gradually converged. This kind of settlement has been proven to result mostly from secondary compression. This phenomenon has been characterized in repeated consolidation tests. Fundamental studies have been carried out under triaxial stress conditions (Yamanouchi and Yasuhara, 1977).

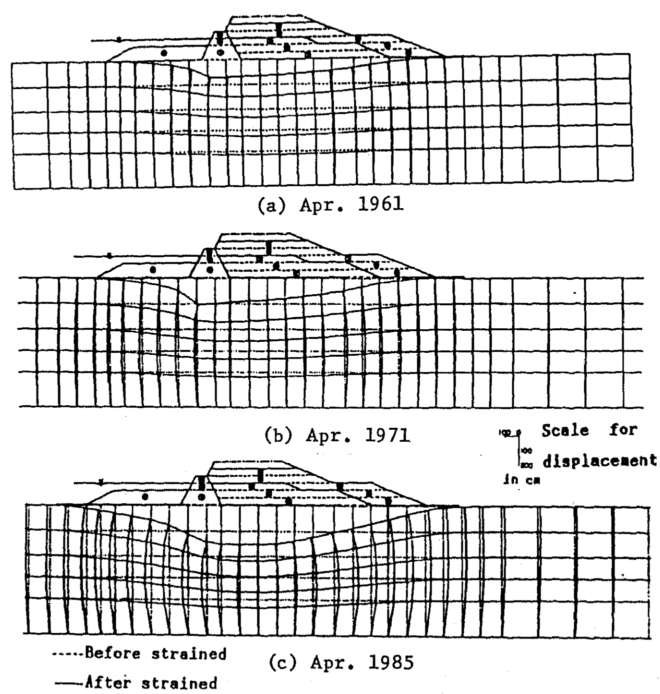


Fig.7 Cross Sectional Deformation Calculated b F.E. Analysis

The bank was built from decomposed granite soil common with the mountains in this area. The test road was 375 m long connecting six kinds of sections as shown in Fig.8. Aiming at searching the best section of these stabilized clay layers were treated with quicklime at the addition of 18.5% (89.7 kgf/m^2 quicklime spreading) and 12.0% (33.3 kgf/m^2) respectively to the sections of 1.0 m and 0.5 m deep stabilization to be able to attain CBR value of more than 3 after 28 days on the basis of a laboratory mixture test.

(1) Construction of the Test Road

A test road was built on a typical alluvial soft clay ground (11 m to 13 m deep of the clay) which was a part of a planned bypass route by the Saga Road Construction Office, Ministry of Construction, in 1975, with the object of investigating the usefulness of a new type of low bank highway. The subgrade was stabilized with quicklime using the new machine.

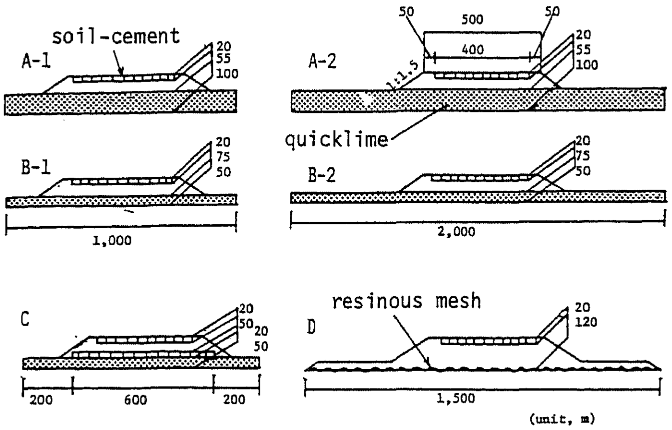


Fig.8 Types of the Subgrades stabilized with quicklime

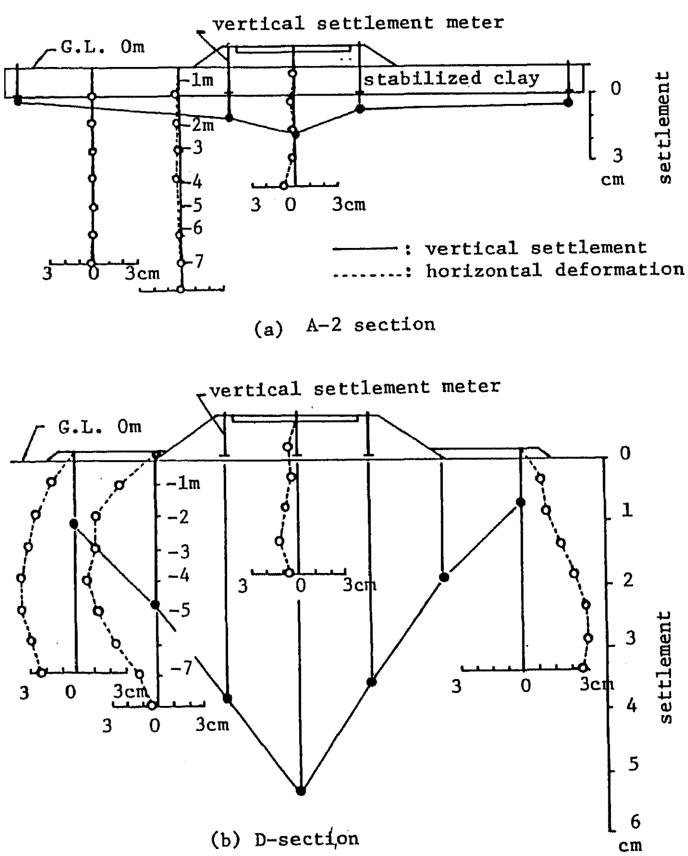


Fig.9 Deformations after a Track Running of 30,000 Times

The D-section was banked after placing a resinous mesh, but it will be counted, at the present study, as the non-stabilized layer section when compared with investigated on its structural effect by the C-section in which the bank was structurally sandwiched (Yamanouchi, 1965) saving the thickness of quicklime stabilized layer. Soil-cement was placed with the addition of 6.5% Portland cement to give a strength of 39 kgf/cm^2 after 7 days.

(2) Vehicle Running Test and the Results

A 15 ton truck was run at an average speed of 20 km/h during daytime until reaching 30,000 load repetitions were achieved (300 times a day for 100 days after seal-coating the road surface with asphalt emulsion). Load applied by the truck were 2.4 tf (4.0 kgf/cm^2) and 5.0 tf (4.5 kgf/cm^2) on the front and rear wheels, respectively.

Instruments were installed in all sections beneath the stabilized layers. The instruments were settlement meters, inclinometers, earth pressure cells and pore water pressure meters. In this paper, only the vertical and lateral deformations are considered in details because the changes in earth and pore pressure were small. This was possibly due to installing the instruments at excessive depths.

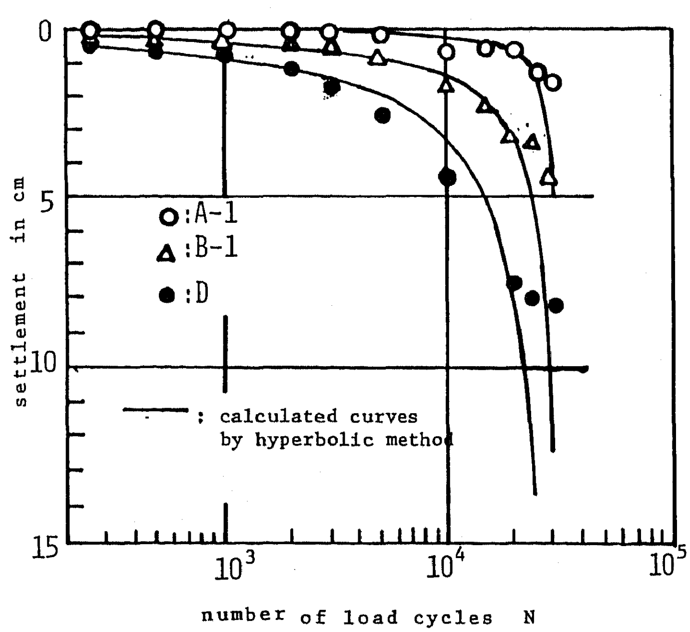


Fig.10 Results of Vehicle Running Tests on Soft Clay Ground (Yamanouchi et al.;1978)

Fig.9 represents the observed residual vertical and lateral deformations only the A-2 and D sections. The results of vehicle running tests on soft clay ground are shown in Fig.10.
The following differences were indicated between sections:
(i) Relative to the non-stabilized layer(D-section), all sections showed smaller residual vertical settlements and lateral displacements. Comparing sections it is clear that the A-section was the most effective on the basis of these deformations. The vertical settlement, in this section, reached about five times of that in the A-1 section or the A-2 section.
(ii) Lateral extension of the width of stabilized layer was effective in controlling the both kinds of deformations.
(iii) Placement of the sandwich-type soil-cement layers in the bank was also effective in controlling the both kinds of deformations.
(iv) There had been a tendency of lateral deformation at the upper part of clay ground in any section. By contrast the non-stabilized layer (D-section) rose very largely at a relatively deep part of the ground. The maximum lateral deformation occurred at the toe of the bank slope in any section composed of a stabilized layer.

Case Study on Traffic-Induced Settlement of Low Embankment

(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 peddy field of which subsoils consist of soft alluvial and sometimes sensitive clay.
In a part of the Kohoku bypass with 1300 m in distance the 1 m depth of clay grounds was improved by stabilization layer with quicklime whose width is narrower by 2 m than the width of embankment. The height of the embankment in 1.95 m with inclusion of 20 cm pavement layer which was constructed after the settlements of the embankment was almost finished. The embankment and the ground were modeled for F.E. analysis as shown in Fig.11.

(2) Anaritical 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 are induced by traffic loads as a results 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.12 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 called by the embankment until an arbitrary time and then is open to traffic loads. At first the settlement-time curve under embankment load is represented by curve I. Settlement after t_1 increases with time due to traffic-induced additional load. Thus the 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-settlemen relations such as curves I,II and III under different loading circumstances can be represented by varying the value of R in Fig.12
An example of the calculated results is shown in Fig.13.The process where the time-settlement relations vary from curve I to curve III as was shown in Fig.12 is simulated by making the valu of R decrease with time.

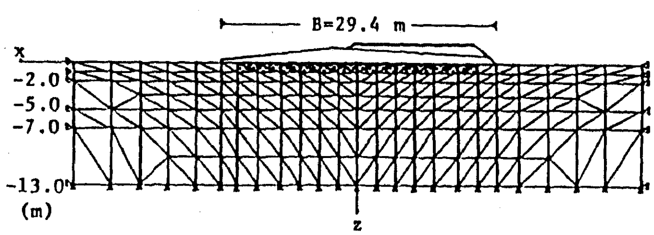


Fig.11 Analytical Model for Traffic-Induced Settlement

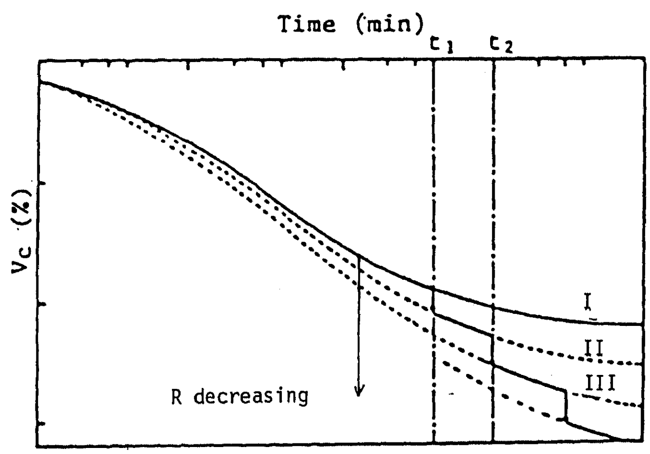


Fig.12 The Key Sketch to Illustrate the Cyclic Effect

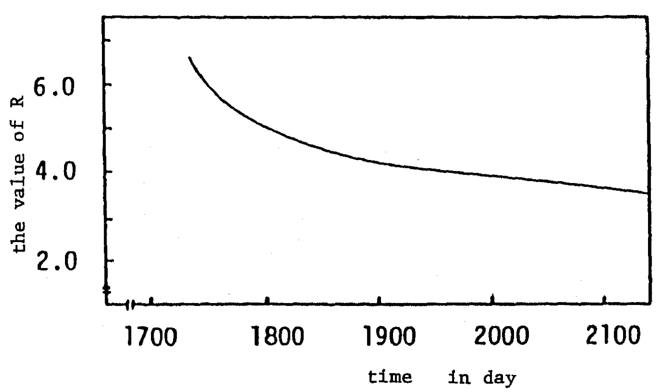


Fig.13 Relationship between Time and the Value of R for F.E. Analysis

(3) Analytical Results

The calculated settlements versus elapsed time relations are compared in Fig.14 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.13. The predicted settlements by means of the elasto-plastic model seem to under-estimate the observed ones as shown in Fig.14.

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.15 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.15 which indicates the predominance of cyclic settlements to static settlements.

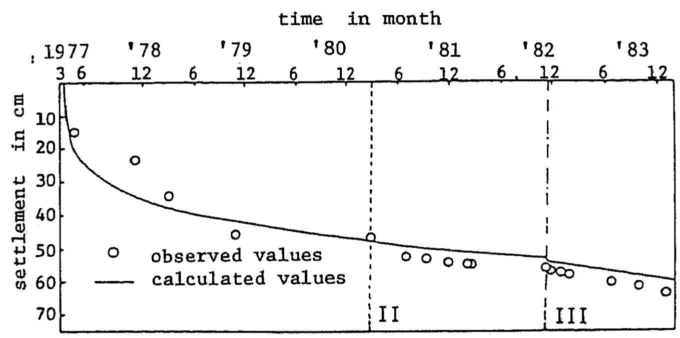


Fig.14 Comparison between Observed and Calculated Time-Settlement Curves

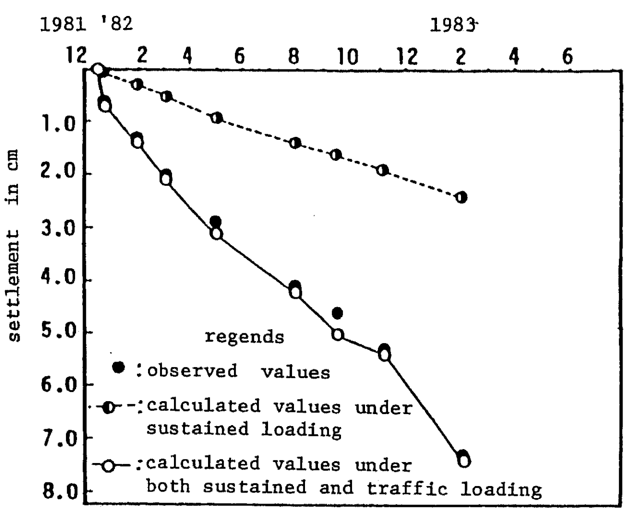


Fig.15 Comparison between Observed and Calculated Time-Settlement Curves after Opening to traffic

Fig.16 shows the analytical results of distribution of principal stresses and maximum shear stress in the clay ground under traffic loading whose surface is , as was mentioned before, improved by quicklime stabilization of 25.4 m in width and 1.0 m in depth. The only one side lane (left side of Fig.16) of this highway is open to traffic at this moment. Hence Fig.16(b) designated that concentration of shear stress is inclined toward the right side of the ground. Fig.16(a) points out that principal stress distribution remains homogeneous beneath the stability layer because it supports the applied load.

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 he time-dependency deformation was adopted to the time-settlement records was adopted to the time-settlement records observed in case histories 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 equipment 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 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.

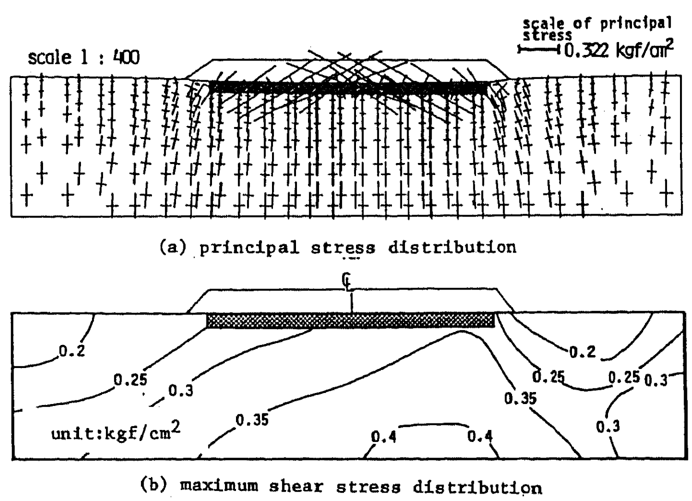


Fig.16 The results of F.E.Analysis

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