XIII ICSMFE, 1994, New Delhi, India / XIII CIMSTF, 1994, New Delhi, Inde

PROGRESSIVE FAILURE OF SOFT CLAY UNDER EMBANKMENT

RUPTURE PROGRESSIVE DE L'ARGILE TENDRE SOUS DU REMBLAI

Hideki Ohta[1] ***Atsushi Iizuka[1]*** ***Shin-ichi Monda[2]***
Masahiko Kuwabara[3] ***Toshifumi Muta[4]***

[1]Kanazawa University, Kanazawa, Japan
[2]Japan Highway Public Corporation, Tokyo, Japan
[3]Fudo Construction Co. Ltd., Tokyo, Japan
[4]Mori-Gumi Co. Ltd., Osaka, Japan

SYNOPSIS: Settlements and stability of a highway embankment placed on a very soft organic clay were monitored, since both the excessive settlements and probable instability were suggested by conventional analyses. The monitored performance of the fill-subsoil system was found to be in good accordance with the performance simulated by employing more refined analyses. The monitored performance being compared with the computed performance suggests that during construction progressive failure was initiated and that a small additional amount of fill might have led to general failure.

INTRODUCTION

A highway embankment on a very soft organic clay was under construction at a place called Ohsawago, Akita Prefecture, from July, 1987 to January, 1991. Since heavy falls of snow were expected from December to March, earth moving works had to be completed in a limited time. There were two crucial points seriously taken into consideration.

1. A large amount of settlement was estimated due to the compression of the soft organic clay 12 *m* thick loaded by 8.5 *m* high embankment (see Fig.1). The loss of fill height due to settlement had to be compensated by additional fill. Sand drains were planned beneath the main body of the fill to accelerate the settlement as shown in Fig.1. Conventional one-dimensional calculation predicted 5.8 *m* of the final settlement.

2. The factor of safety against failure was uncertain because of the unknown additional fill height and the strength of fill body. As seen in Fig.1, the lower part of the fill body and the berms at both sides were to be constructed of compacted soil mixed-in-place with lime of 5 % in weight. The lime mixed layers were to be reinforced by placing steel mesh. Conventional stability analysis made ignoring the strength of fill body and the ground surface crust gave the factor of safety of 0.88 – 0.95. This factor of safety being less than unity suggested stability problems for the embankment.

Since the time available was short, it was decided to follow an observational approach. Monitored performance of the compacted fill body, lime mixed soil, steel meshes, ground surface crust and soft organic clay both in sand-drained and natural areas were analysed by means of a soil/water coupling finite element programme called DACSAR (Deformation Analysis Considering Stress Anisotropy and Reorientation) developed by Iizuka and Ohta (1987). In the analysis, the fill and the soft subsoil were modelled as elasto-(visco)plastic materials, the surface crust as an elastic material, and the lime mixed soil as an elastic material having a stiffness which increased with time since mixing. Modelling of the materials is summarized in Fig.1.

CONSTITUTIVE MODEL AND SOIL PARAMETERS

The DACSAR programme employs an elasto-viscoplastic constitutive model proposed by Sekiguchi and Ohta (1977) which is an extension of a series of experimental works on the dilatancy of clays initiated by Shibata (1963). Since the laboratory test data available are limited as shown in the figure displayed in the upper row of Fig.2, material parameters of the soft subsoil are estimated mainly based on the data obtained from oedometer tests and unconfined

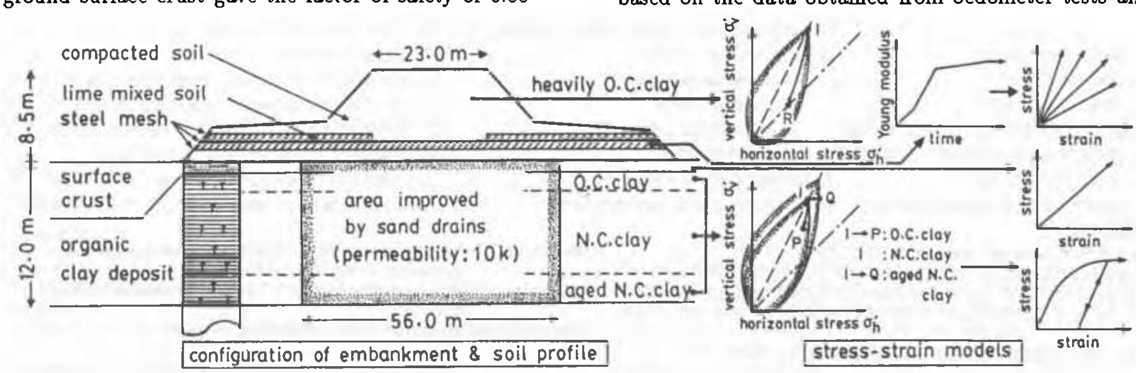
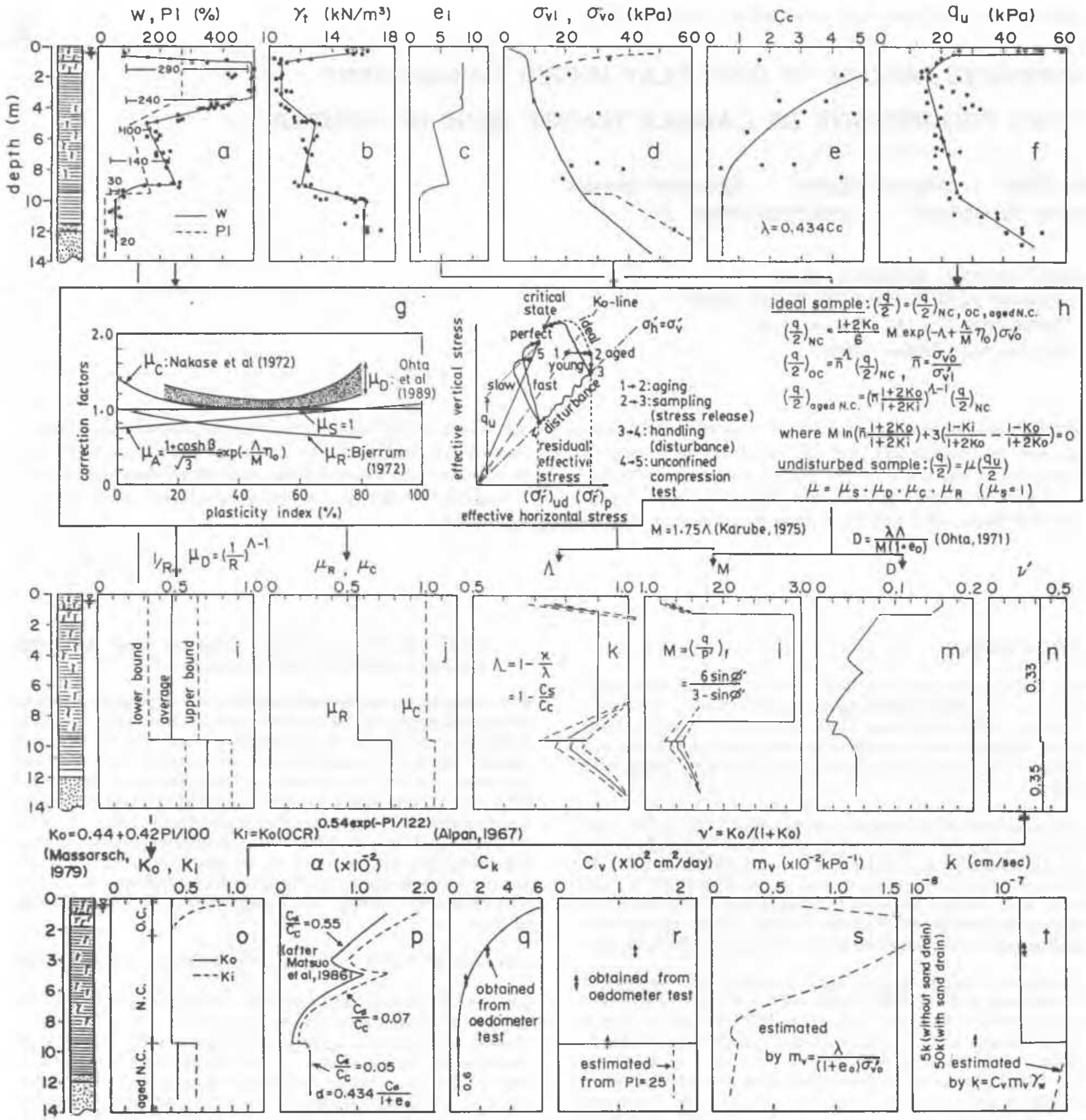


Fig.1 Modelling of materials in the fill-subsoil system



Notation

W : water content
PI : plasticity index
γ_t : unit weight
e_i : initial void ratio
σ'_{vo} : preconsolidation vertical stress
σ'_{vi} : effective overburden stress
Cc : compression index
k coefficient of permeability
q_u : unconfined compression strength
μ : correction factor
μ_s : stress release
μ_D : disturbance
μ_C : confining pressure
μ_R : strain rate
Λ : irreversibility ratio
M : critical state parameter
D : dilatancy coefficient
ν' : Poisson ratio
K_o : coefficient of earth pressure at rest
K_i : coefficient of earth pressure in situ
α : coefficient of secondary compression
C_k : gradient of e vs. log k relation
C_v : coefficient of consolidation
m_v : confficient of volumetric compressibility

Reference

Alpan(1967) Soils & Foundations,(7),1,31-40
Karube(1975) Proc. 20th Symp. on Geotech. Engrg. JSSMFE, 45-60
Massarsch(1979) Proc. 7th European Conf. SMFE, (2), 245-249
Nakase et al. (1972) Report of Port & Habour Research Inst.,(11), 4, 83-102
Ohta et al. (1989) Proc. 12th Int. Conf. SMFE,(1), 71-74
Bjerrum(1972) Performance of earth supported structures, ASCE Specialty Conf., (2), 1-54
Matsuo et al. (1986) Domestic Edition of Soils & Foundations, (26),1, 134-150
Ohta(1971) Dr. Eng. Thesis, Kyoto University, 114-116

Fig.2 Specification of the subsoil parameters

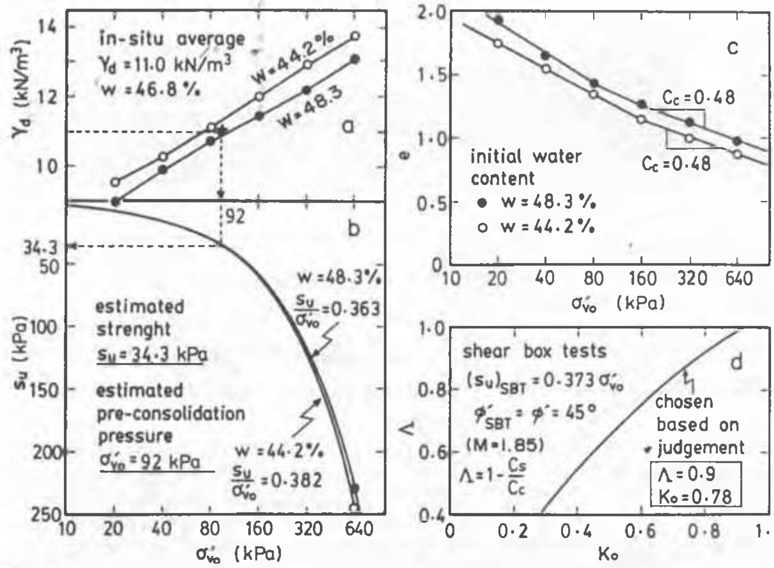


Fig.3 Estimate of parameters of compacted fill body based on the field and laboratory test data interpreted by the use of the theoretical equation for undrained strength obtainable from the shear box tests:

$$s_u(SBT) = \frac{(1+2Ko)M\exp(-\Lambda)}{3\sqrt{3}}\sigma'_{vo}$$

compression tests. Plasticity index is also used as an auxiliary index from which some of the parameters are specifiable through emperical equations. The unconfined compression strength is converted to the undrained strength obtainable from Ko-consolidated undrained compression tests. The conversion is made by applying the correction factors to raw unconfined compression data. Correction factors are shown in the figures in the second upper row of Fig.2. Equations theoretically derived by Ohta et al. (1989) are also shown. Depthwise distributions of material parameters specified using these equations are shown in the figures displayed in the lower half of Fig.2. Since the correction factor for disturbance is given as a band, see Fig.2 (g), some parameters are derived as bands, mid-values of which are used in analysis.

Material parameters for the compacted fill body were estimated from the dry unit weight and the water content determined during compaction. These values are plotted in Fig.3 (a), see (★). In the diagram, the dry unit weight of the fill material consolidated in a shear box is also plotted against the vertical consolidation pressure. As seen in Fig.3 (a), the dry unit weight increases linearly with the increase in logarithm of the consolidation pressure. Two lines are shown in Fig.3 giving test data for the fill material with water contents of 48.3 % and 44.2 %. The average water content of the fill material actually compacted in the field was 46.8 % which falls between 48.3 % and 44.2 %. Since the dry unit weight of the compacted fill body is $11.0\,kN/m^3$ ($1.12\,tf/m^3$), $92\,kPa$ ($9.4\,tf/m^2$) must be the consolidation pressure required to produce the statically compressed soil with a water content of 46.8 % and a dry unit weight of $11.0\,kN/m^3$ ($1.12\,tf/m^3$), see Fig.3 (a).

The fill material consolidated in a shear box with the pressure up to $628\,kPa$ ($64.0\,tf/m^2$) is then tested under constant-volume shear in which the volume of the soil is kept constant during shear by adjusting the vertical pressure. The change in vertical pressure accompanied by the increase in shear stress results in a curve which corresponds to a type of effective stress path giving the effective angle of internal friction. The shear strength obtained in this way is likely to correspond to the undrained strength, since the volume of specimens is kept constant during shear. The ratio of undrained strength to the consolidation pressure is 0.363 and 0.382 for the soils with the water content of 48.3 % and 44.2 % as shown in Fig.3 (b). This ratio is a constant independent of the consolidation pressure for most of the unsaturated soil. It was thought reasonable to assume that the ratio for the fill material compacted in the field should be 0.373 which is the average of 0.363 and 0.382.

The undrained strength obtainable from the constant-volume shear box tests is theoretically derived from the constitutive equation by Ohta, Nishihara and Morita (1985) in the form shown in the caption of Fig.3. Since the effective angle of internal friction obtained from the shear box tests is 45 degrees as indicated in Fig.3 (d), the critical state parameter M is 1.85. Two parameters, the irreversibility ratio Λ and Ko, which remain unknown, should therefore satisfy the relation represented by the curve in Fig.3 (d). The authors feel it appropriate to assume that $\Lambda = 0.9$. As seen in Fig.3 (c), Cc is 0.48, then $Cs = (1-\Lambda)Cc = 0.05$. Parameters of the compacted fill body are thus estimated. The preconsolidation pressure is assumed to be $92\,kPa$ ($9.4\,tf/m^2$) as shown in Fig.3 (a). This means that the overconsolidation ratio of the fill material depends on the location of the volume element relative to the configuration of the fill body.

The lime mixed soil is modelled as an elastic material having a stiffness which increases with the time elapsed since mixing. Data for unconfined compression tests on lime mixed soil are plotted in Fig.4 (a). Much scatter is evident, but the average value of E_{50} ($= 32.3\,MPa = 3,300\,tf/m^2$), which corresponds to the average strength, is chosen as representative since it is very close to the average of the scatter obtained from a number of in-situ tests. The timewise increase in the modulus is assumed as shown in Fig.4 (b). The elastic modulus of the ground surface crust is also estimated in the same fashion. The value chosen, $860\,kPa$ ($88\,tf/m^2$), corresponds to the average value of unconfined compression strength, see Fig.4 (c).

MONITORED AND COMPUTED PERFORMANCES OF THE FILL

Computed results are compared with the monitored performance of the fill-subsoil system as shown in Fig.5. Solid and open circles in Fig.5 (a) (top figure) indicate the locations of settlement gauges and piezometers. The computed values plotted as small dots shown in Figs.5 (b)–(m) are generally in good accordance with the monitored records shown as solid curves. The monitored and computed settlements are extremely close to each other under the main body of the fill, while the monitored settlements under the berm tend to diverge from the computed settlements at the loading stages indicated by numerals in a circle in Fig.5 (n). Such differences shown in Figs.5 (d)–(f) and (h) suggest that phenomena must have taken place in the field which cannot be simulated by the soil/water coupling finite element method, although the ground surface settlements monitored near the centre of the fill body (Figs.5 (b) and (c)) and settlements under the berm monitored before the events of divergence are extremely well simulated

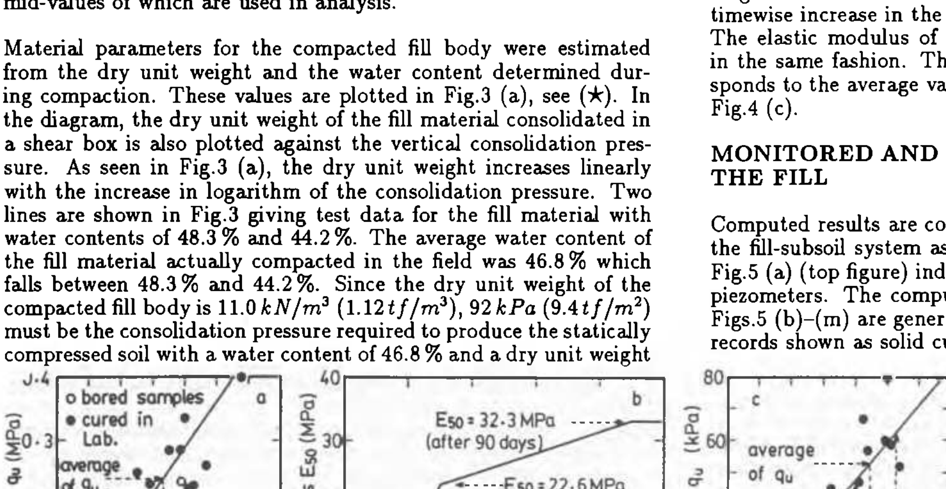


Fig.4 Elastic parameters of the lime mixed soil and the ground surface crust ($\nu' = 0.33$)

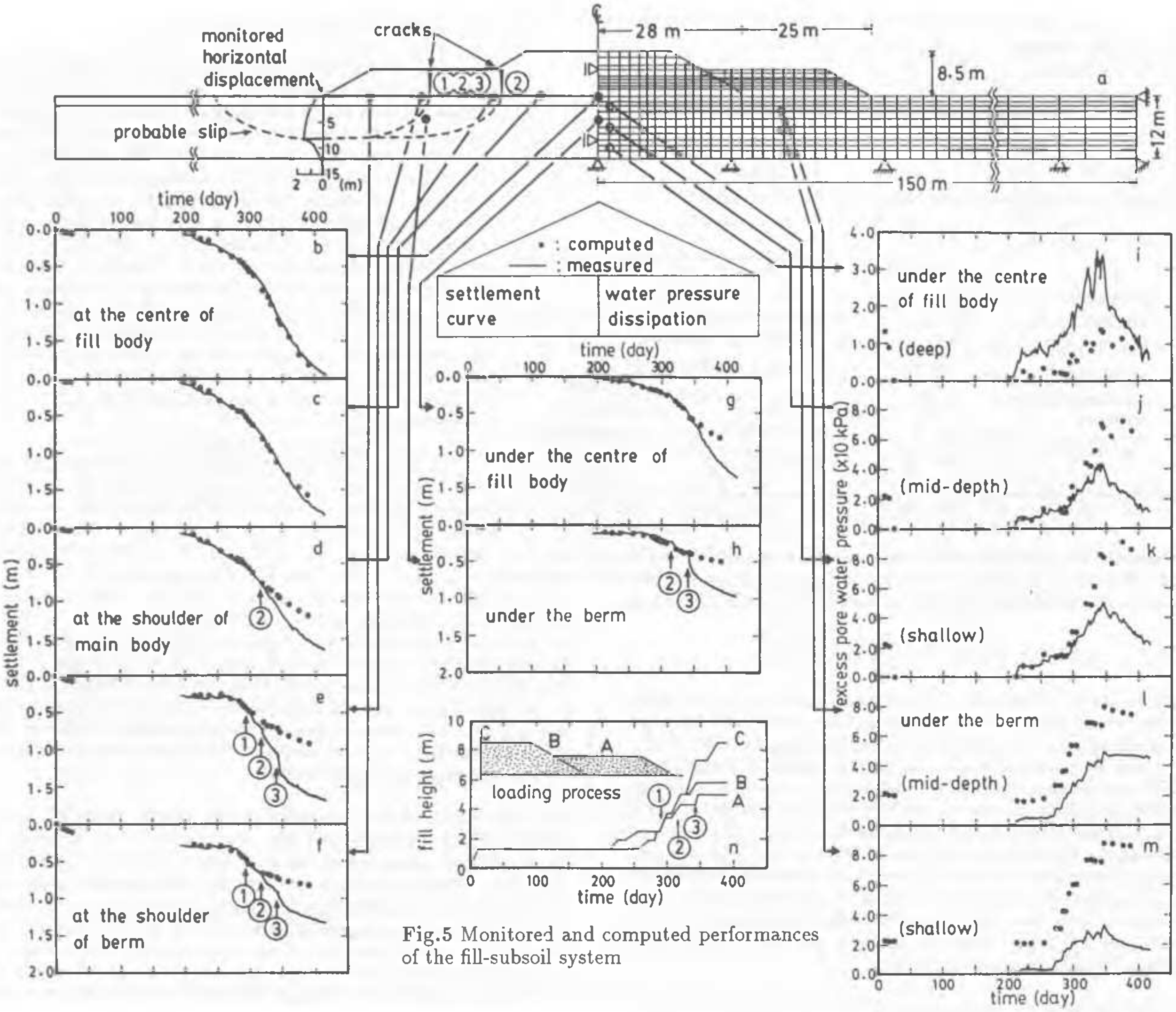


Fig.5 Monitored and computed performances of the fill-subsoil system

in the computation. The most likely cause of the divergence must be the occurrence of sequential slides beneath the berm. The finite element programme such as DACSAR is unable to simulate the phenomena with large strain such as slides. In fact, the inclinometer installed at the toe of the berm recorded a large horizontal displacement up to $1.8\,m$ at a depth of $8\,m$ as shown in Fig.5 (a). Tension cracks were also found on the surface of the berm. The probable locations of these slip surfaces are shown in Fig.5 (a) by dashed curves. The authors believe that this was the initiation of progressive failure which could be led to general failure by a small amount of additional loading.

The pore pressure responses monitored in the field are compared with the computed pore pressures in the right side of Fig.5. In the authors' past experience of analysing field performance of soil-structure interaction during construction works, the computed pore water pressure was always higher than the pore pressure actually monitored in the field by a factor ranging from 1.3 to 2.0, even when the deformation was simulated reasonably. The comparisons made in Figs.5 (j)–(m) seem reasonable to the authors. However, monitored pore pressures plotting higher than computed in Fig.5 (i) is not in accordance with the usual tendency; the reason for this is still open to question. It should be noted that no parameter adjustments were made in the course of the calculations discussed in this paper, aiming at achieving a better simulation of observed embankment behaviour.

CONCLUDING REMARKS

Observed settlements of the embankment suggested that the one-dimensional method of settlement prediction did not account for the complexity of the field situation. More refined analyses simulated most aspect of the field performance of the fill-subsoil system to a satisfactory degree. These analyses suggested that progressive failure was initiated during construction and that a small additional amount of fill might have led to general failure. It may be concluded that the embankment construction described in this paper was carried out in the most economical way within the given restriction of time.

REFERENCES

Iizuka, A. and Ohta, H. (1987).A determination procedure of input parameters in elasto-viscoplastic finite element analysis, *Soils and Foundations*, 27(3):71-87.

Ohta, H., Nishihara, A. and Morita, Y. (1985).Undrained stability of Ko-consolidated clays, *Proc. 11th Int. Conf. Soil Mech. & Found. Eng.*, San Francisco, 2:613-616.

Ohta, H., Nishihara, A., Iizuka, A., Morita, Y., Fukagawa, R. and Arai, K. (1989).Unconfined compression strength of soft aged clays, *Proc. 12th Int. Conf. Soil Mech. & Found. Eng.*, Rio de Janeiro, 1:71-74.

Sekiguchi, H. and Ohta, H. (1977).Induced anisotropy and time dependency in clays, *Proc. Specialty Session 9, 9th Int. Conf. Soil Mech. & Found. Eng.*, Tokyo, 475-484.

Shibata, T. (1963).On the volume change of normally consolidated clays, *DPRI Annuals, Kyoto Univ.*, 6:128-134, (in Japanese).