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# SIMPLIFIED DEFORMATION ANALYSIS FOR EMBANKMENT FOUNDATION USING ELASTO-PLASTIC MODEL

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## ABSTRACT

This paper presents a simplified deformation analysis for a clay foundation under an embankment using the soil parameters for the constitutive model estimated from plasticity index and the elasto-plastic model proposed by Sekiguchi and Ohta. Based on this method, and using the finite element method, the deformation characteristics of a well documented case history for the Kurashiki trial embankment on a soft ground was investigated.

A comparison with the measurements indicates reasonable agreement between the measured and computed values of the deformation characteristics for the multi-soil layers. The predicted lateral deformation, however, overestimates the field measurements. Based on these results, the present method can be concluded applicable to other fields and it may be stated with reasonable accuracy that other case histories with similar characteristics and parameters will probably behave in a similar manner.

**Key words:** cohesive soil, consistency limits, consolidation, deformation, earthfill, elasticity, finite element method, foundation, plasticity, soft ground (**IGC**: E2/D2)

## INTRODUCTION

The availability of computing facilities has recently created a boom in the application of numerical techniques, among which the finite element method has proven to be a most versatile and useful tool. Methods to determine the relevant soil parameters in the laboratory and in the field have also been improved, although the progress in this area has not kept pace with the analytical techniques. Ladd et al. (1977) stated that a prediction capability consists of three components: (i) a model to describe soil behavior, (ii) suitable methods to evaluate the required soil parameters, and (iii) computational procedures for applying the model to practical problems. The key to the success of these deformation analyses largely depends on the factors stated above.

It was also found that the computed deformations are governed by the selected soil parameters. It appears, however, that in practice, the selection of soil parameters from tests is not an easy task. Regarding this, Kamei (1985), Nakase and Kamei (1983, 1988a) and Nakase et al. (1988b) carried out an extensive testing program to study the mechanical behavior of cohesive soils over a range of plasticity index PI from 10 to 55. Twelve soils were used in two series of experiments: (i) a Kawasaki clay mixture series and (ii) a reconstituted natural marine clay series. The reconstituted natural marine clay series was obtained from various locations along the coastal areas of Japan. Based on these test data, Kamei (1985) and Nakase et al. (1988b) proposed linear correlations for undrained shear strength and some soil parameters for constitutive equations with plasticity index.

The plasticity index PI can be obtained from Standard Penetration Test samples which are used widely in Japan and it is one of the key components of Casagrande's plasticity chart, which still offers a very useful means of relating on a global basis the general physical behavior of soft and sedimentary cohesive soils. Geologically similar deposits plot in bands parallel to the A-line, which in turn separates typical clays from inorganic silts and organic soils, and important behavioral characteristics such as the potential for being highly sensitive, anisotropic or susceptible to creep are generally related to the values of the soil properties plotted on the plasticity chart. Thus physical behavior is therefore related to PI. In addition, the plasticity index PI is one of the key components of the activity chart, which offers a degree of electrical characteristics in constitutive mineralogy. A rough estimate of the PI values can be more easily made by evaluating physical parameters such as touch and vision. The method of obtaining PI values is simple and the value does not vary significantly with respect to personal fac-

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tors (Kamei, 1985). In addition, PI has been used not only as an index of soil classification but as an index of the mechanical properties of soil. Skempton (1957), for example, proposed that the ratio of undrained shear strength to initial vertical effective stress is related to the plasticity index of the soil based on field vane data.

It would be interesting to investigate a possible deformation analysis for an embankment foundation using an elasto-plastic model. Numerical analysis now makes it possible for complete solutions to be obtained for the complex problems that arise in geotechnical engineering. Ideally, computation should be carried out with one method of analysis and one soil model, with one consistent set of soil parameters.

The Sekiguchi-Ohta model has been presented incorporating dilatancy and rheological behavior (Sekiguchi and Ohta, 1977; Sekiguchi, 1977). A revised Sekiguchi-Ohta model has been presented in which the strain increments for an anisotropically consolidated clay subjected to a stress reorientation were determined (Ohta and Sekiguchi, 1979). The Sekiguchi-Ohta model, therefore, has been widely used in Japan in order to evaluate the mechanical behavior of a clay foundation under an embankment (Duncan, 1994).

To propose a simplified procedure for deformation analysis of clay foundation under an embankment, it would be interesting to seek possible application of the soil parameters estimated from the plasticity index using the Sekiguchi-Ohta model and the finite element method with small strain theory.

Iizuka and Ohta (1987) proposed a procedure to estimate the input parameters for the Sekiguchi-Ohta model and investigated its suitability by comparing the data with many published laboratory data. The procedure makes it possible to estimate the soil parameters using some simplified correlations on the basis of PI and oedometer test results. The authors' procedure is more simplified and can determine all the soil parameters considering PI only. In addition, all the correlations between each soil parameter and PI in the present study were investigated extensively under consistent test conditions, rather than amassing information consisting of isolated laboratory test results presented in individual papers.

Chai et al. (1994) and Chai and Bergado (1995) reported that a case study using finite element analysis considering large deformation explained the behavior of a clay foundation under an embankment better than by conventional finite element calculation with small strain theory. This implies that the results computed for deformation and excess pore water pressure based on the large deformation technique were only a little different from those based on conventional analytical procedure with small strain theory. The authors, however, employed a conventional analytical procedure based on small strain theory with reasonable modelling for boundary conditions in order to establish the most simplified analytical procedure.

## ELASTO-PLASTIC MODEL PROPOSED BY SEKIGUCHI AND OHTA

The constitutive equations used in this study were based on the model proposed by Ohta and Sekiguchi (Sekiguchi and Ohta, 1977; Sekiguchi, 1977; Ohta and Sekiguchi, 1979), which can explain the rheological and the anisotropic mechanical behavior of $K_0$-consolidated clay quantitatively as well as qualitatively. The elasto-plastic model derived by eliminating viscosity from this model, however, can also explain very well the mechanical behavior of a cohesive soil qualitatively with the soil parameters estimated by PI (Nakase et al., 1988b). The authors employed this elasto-plastic model herein in order to establish a simple procedure for deformation analysis. These elasto-plastic and elasto-viscoplastic models can be explained briefly as follows.

In the elasto-viscoplastic model, the relationship between volumetric strain $\varepsilon_V$ and volumetric strain rate $\dot{\varepsilon}_V$ can be expressed as the following equation:

$$\varepsilon_V=\frac{\lambda-\kappa}{1+e_0}\ln\frac{p'}{p'_0}+D\cdot\eta^*-\alpha\ln\left(\frac{\dot{\varepsilon}_V}{\dot{V}_0}\right) \tag{1}$$

where, $\dot{V}_0$ is the initial volumetric strain rate, $D$ is the coefficient of dilatancy (Shibata, 1963; Ohta and Sekiguchi, 1979), and $\eta^*$ is the shear stress ratio based on a consideration of $K_0$-consolidated stress condition at insitu expressed as the following equation:

$$\eta^*=\sqrt{3/2\cdot(\eta_{ij}-\eta_{ij0})(\eta_{ij}-\eta_{ij0})} \tag{2}$$

$\eta_{ij}$ can be expressed as follows:

$$\eta_{ij}=\sigma_{ij}/p'-\delta_{ij},\quad \eta_{ij0}=\sigma_{ij0}/p'_0-\delta_{ij} \tag{3}$$

By solving Eq. (1) using the initial conditions, the visco-plastic volumetric strain $\varepsilon_V^{vp}$ was obtained as the following equations (Sekiguchi and Ohta, 1977):

$$\varepsilon_V^{vp}=\alpha\cdot\ln\left\{1+\frac{\dot{V}_0\cdot t}{\alpha}\exp\left(\frac{f}{\alpha}\right)\right\}\equiv F \tag{4}$$

where, the $F$ is the function of visco-plastic potential (Shibata and Sekiguchi, 1980). Applying the normality rule to this potential, the following stress-strain relationship was derived:

$$\dot{\sigma}_{ij}=D_{ijkl}^{evp}\cdot\dot{\varepsilon}_{kl}-\dot{\sigma}_{rij} \tag{5}$$

where, $D_{ijkl}^{evp}$ and $\dot{\sigma}_{rij}$ are given by the following equations:

$$D_{ijkl}^{evp}=D_{ijkl}^{e}-D_{ijop}^{e}\frac{F_{op}\cdot F_{mn}\cdot D_{mnkl}^{e}}{(F_{mn}\cdot D_{mnqr}^{e}+\delta_{qr})\cdot F_{qr}} \tag{6}$$

$$\dot{\sigma}_{rij}=D_{ijkl}^{e}\frac{F_t\cdot F_{kl}}{(F_{mn}\cdot D_{mnqr}^{e}+\delta_{qr})\cdot F_{qr}} \tag{7}$$

Taking an approximation of Euler-Type forward difference on Eq. (5) for a time increment $\Delta t$, the following incremental elasto-viscoplastic constitutive equation can be obtained:

$$\Delta\hat{\sigma}=\hat{D}^{evp}|_t\cdot\Delta\hat{\varepsilon}-\Delta\hat{\sigma}_r|_t \tag{8}$$

where, $\Delta\hat{\sigma}$ is an incremental general stress vector, $\Delta\hat{\varepsilon}$ is an incremental general strain vector, $\Delta\hat{\sigma}_r|_t$ is an incremental stress relaxation vector due to viscosity, and $\hat{D}^{evp}|_t$ is a stress-strain matrix.

Equation (8) may be converted into an elasto-plastic model by neglecting all the terms of viscosity in the equation. In this case, the stress-strain matrix, $\hat{D}^{evp}|_t$ becomes an elasto-plastic matrix, $\hat{D}^{ep}$ and the term of stress relaxation, $\Delta\hat{\sigma}_r|_t$ is neglected. In this study, this elasto-plastic model was adopted as a simple model to establish a simplified procedure for evaluating deformation analysis. The details of the derivation of these constitutive models are discussed in the original papers (Sekiguchi and Ohta, 1977; Sekiguchi, 1977; Ohta and Sekiguchi, 1979; Shibata and Sekiguchi, 1980).

**Table 1. A set of soil parameters estimated by PI**

| Parameter (1) | $r$ (2) |
|---|---|
| $\lambda=0.02+0.0045\cdot$PI | 0.98 |
| $\kappa=0.00084\cdot$(PI-4.6) | 0.94 |
| $N=1.517+0.019\cdot$PI | 0.95 |
| $M_C=1.65$ | — |
| $M_E=1.385-0.00505\cdot$PI | 0.85 |
| $K_{0NC}=0.45$ | — |
| $K_{0OC}=K_{0NC}\cdot(\mathrm{OCR})^{0.45}$ | — |
| $\nu'=K_{0NC}/(1+K_{0NC})$ | — |
| $k_v=k_{v0}\cdot\exp\{(e-e_0)/\lambda_k\}$ | — |
| $\lambda_k=0.073+0.019\cdot$PI | 0.98 |

$r$: coefficient of correlation in linear regression analyses

## SOIL PARAMETERS FOR CONSTITUTIVE EQUATIONS

Kamei (1985) and Nakase et al. (1988b) proposed many correlations to determine the soil parameters for the constitutive equations (compression index $\lambda$, swelling index $\kappa$, specific volume $N$, slope of Critical State Line in triaxial compression loading side $M_C$, slope of Critical State Line in triaxial extension loading side $M_E$) by plasticity index. Herein $N(=1+e_0)$ is the specific volume corresponding to a mean effective principal stress of $p'=98$ kPa.

The coefficient of earth pressure at rest for a normally consolidated soil $K_{0NC}$ and the coefficient of earth pressure at rest for an over consolidated soil $K_{0OC}$ in Japan were proposed by Kamei and Sakajo (1993) as the following equations:

$$K_{0OC}=K_{0NC}(OCR)^{0.45} \tag{9}$$

The Poisson's ratio $\nu'$ in elastic region can be obtained by $K_{0NC}$ based on linear-elastic theory as follows:

$$\nu'=K_{0NC}/(1+K_{0NC}) \tag{10}$$

The coefficient of permeability $k_v$ is given by the following equation (Taylor, 1948):

$$k_v=k_{v0}\cdot\exp\{(e-e_0)/\lambda_k\} \tag{11}$$

where, $k_{v0}$ can be obtained as the coefficient of permeability at $e=e_0$ and $\lambda_k$ is the material constant governing the rate of change in permeability subjected to a change in the void ratio. The value of $k_{v0}$ can be obtained from the plasticity index (Kamei, 1985) and the $\lambda_k$ can be estimated by the following equation considering the plasticity index (Kamei and Sakajo, 1993):

$$\lambda_k=0.073+0.019\cdot\mathrm{PI} \quad (r=0.984) \tag{12}$$

These equations and correlations between soil parameters and PI are summarized in Table 1, where the $r$ means the correlation coefficient in linear regression analyses. These equations and correlations are based on extensive test data under consistent test conditions, therefore, these values of correlation coefficients in this table are remarkably high. The applicability of these equations and correlations to other data were confirmed by comparing them with other experimental results based on compression index and swelling index (Nakase et al., 1988b).

Although the consistent soil parameters mentioned above are sufficient enough for the elasto-plastic model proposed by Sekiguchi and Ohta, to calculate the coefficient of dilatancy, $D$, in Eq. (1), only $M_C$ is used. Kamei (1989) investigated the $D$-value of Sekiguchi-Ohta model based on the value of $M_C$ and shows good agreement with the experimental values for triaxial extension loading of cohesive soils with a different plasticity index. The use of $M_E$ therefore is limited as the shear stress ratio at critical state for extension loading in this study. The $D$-value is calculated from the following equation:

$$D=\frac{\lambda-\kappa}{1+e_0}\cdot\frac{1}{M} \quad (M=M_C) \tag{13}$$

where, $\lambda$ and $\kappa$ are the compression index and the swelling index respectively, $e_0$ is the void ratio, $M$ is the slope of the Critical State Line.

Using these parameters in an elasto-plastic analysis, it should be noted that these values correspond to a mean effective principal stress of 98 kPa. The $e_0$, for instance, seems to represent the void ratio value of the subsoils at a depth of 10 to 15 m. This value may therefore underestimate and overestimate the actual void ratio at both a shallower and at deeper depth, respectively. The computed ground deformation, however, may be characterized not by each value of $\lambda$, $\kappa$ and $e_0$ but by $\lambda/(1+e_0)$ or $\kappa/(1+e_0)$, which does not vary so much with the depth of the subsoils, because generally each value of $\lambda$, $\kappa$ and $e_0$ decreases in relation to the depth below ground surface. The prediction accuracy of these method therefore can be expected not to be worse at a shallower or at deeper depth than 10 to 15 m.

Kamei and Sakajo (1995), based on the set of soil parameters stated above conditions simulated the several strain controlled undrained behavior of $K_0$-consolidated cohesive soils using the elasto-viscoplastic model proposed by Sekiguchi and Ohta and obtained reasonable agreement between the observed and the computed results for $K_0$-consolidated cohesive soils.

## APPLICATION TO AN ACTUAL EMBANKMENT FOUNDATION

Although it has been shown that each individual soil parameter appearing in the constitutive equations can be expressed in terms of PI, it may be appropriate to demonstrate the overall accuracy of the relationships obtained from this series of tests by computing the deformation characteristics for an embankment foundation on a soft multi layer subsoil profile. The Sekiguchi-Ohta model is used for this illustration.

Herein, the numerical results obtained using the assumed model subsoil profile, Sekiguchi-Ohta model and the soil parameters estimated by plasticity index can be compared to the results of field measurements. A well documented case history for the deformation characteristics of clay foundation under embankment has been reported by Mochizuki et al. (1980).

### *Outline of Kurashiki Trial Embankment*

The Kurashiki trial embankment was constructed to evaluate safety against land slides and lateral ground deformation due to the limitation of the construction area (Mochizuki et al., 1980; Sekiguchi and Shibata, 1982). The embankment construction is described generally as follows.

The plan view and cross-section for the Kurashiki trial embankment foundation is shown in Fig. 1. The Kurashiki trial embankment is located in the Tamashima District of Kurashiki City in Japan. Facing the Seto-naikai Sea, the level of the ground surface is almost the same as the sea water level (T.P. +0.0 m) and the foundation is a typical weak marine clayey soil deposit, which was reclaimed to expand some agricultural fields.

A diluvial sandy deposit extends from the surface to the bedrock which is located at a depth of approximately 45 m. This deposit consists mainly of sand layers interspersed with some thin cohesive soil layers. Weak cohesive soil exists from the ground surface to a depth of 6.3 m below the ground surface. The lower layer of this deposit is a sandy soil with a Standard Penetration value of 15. The coefficient of permeability of the sandy layer is almost 0.08 (cm/sec), measured using the tube method where the Young's modulus was measured as $E_p=1470$ kPa from pressure meter tests. A 40 cm thick sand mat was placed on the original surface of the weak cohesive soil before embankment construction began. The consolidation due to the weight of the sand mat was almost complete. The ground water level was measured and found to be almost the same elevation as the bottom of the mat. The soil profile for this foundation is shown in Fig. 2. The values of natural water content of the weak cohesive soil layer are almost the same as the values of liquid limit, although the closer to the top and bottom sand layer, the natural water content becomes less than the liquid limit. This weak cohesive soil, therefore, may be roughly evaluated to be normally consolidated, although the stress history near the sandy layer must be precisely considered.

In the soil investigation for this site, some undisturbed samples were obtained for the foundation and a series of laboratory tests made to define the soil parameters (Sekiguchi and Shibata, 1982). In this research, however, the authors estimated these soil parameters from the equations only and correlations with plasticity index as shown in Fig. 2. Table 2 indicates the all soil parameters estimated in this manner, namely vertical overburden

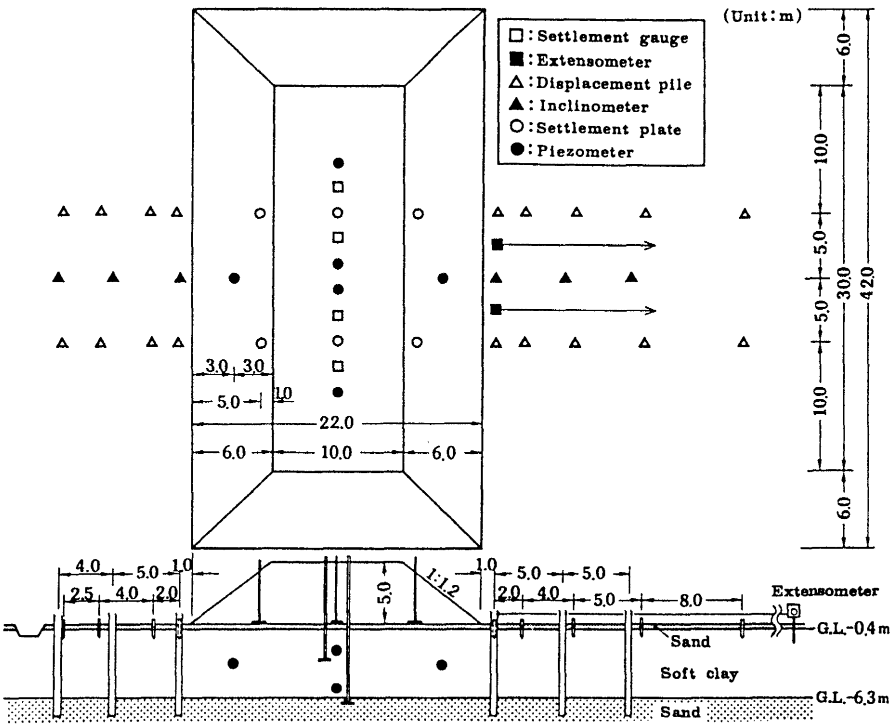



**Fig. 1. Plan and cross-section for Kurashiki Trial Embankment foundation**

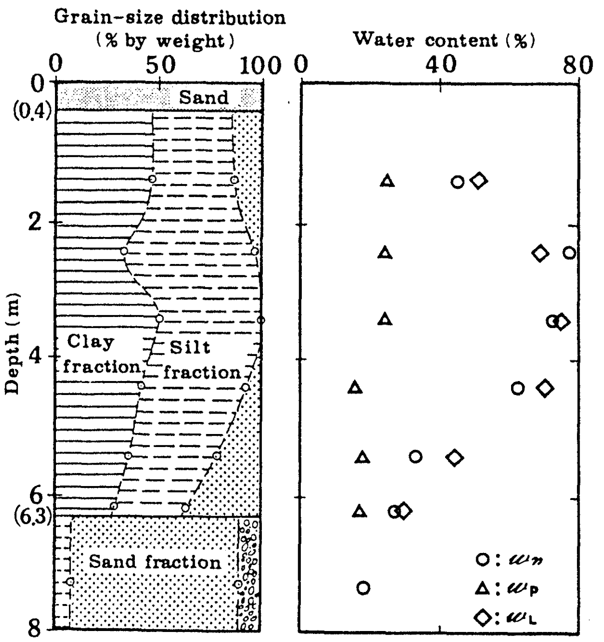



**Fig. 2. Soil profile of Kurashiki Trial Embankment foundation**

pressure $\sigma'_{v0}$ and pre-consolidation stress $\sigma'_{vc}$. In this table, only the L-1 layer is assumed to be an elastic material with $E=2400$ kPa and $\nu'=0.3$, which was estimated using oedometer test results by Sekiguchi and Shibata (1982). Comparing vertical overburden pressure with pre-consolidation stress, the over consolidated state can be determined at the portion of the cohesive soil close to the top and bottom sandy layer, being coincident with the fact that the natural water content is smaller than the liquid limit there.

A number of instruments, ground surface settlement piles, screw type soil layer settlement gauges, ground surface movement piles, extensometers, inclinometers and piezometers were installed to measure the ground deformation as shown in Fig. 1. According to the data observed, it was found that these instruments could very well measure the ground deformation except for excess pore water pressure behavior. The two of the three piezometers could not record the pore water pressure after the embankment end of construction because electric cables were broken. The comparisons between observations and computations, which will be discussed later in Section, are therefore limited to only ground surface settlements and lateral deformation.

Figure 3 shows the embankment construction schedule. The material for the embankment was Masa (a local sandy soil in Japan) and it was placed in a 30 cm layer thickness with bulldozers at 2 day intervals (the speed of the embankment construction was 9.5 cm/day on an average based on a consideration of the effect of embankment settlement due to its own weight.). An intermission for nine days took place once during the entire period of construction at the time when the embankment height reached 3.9 m. It was reported that it took 58 days to complete the embankment construction.

*Finite Element Analyses, Boundary Conditions and Their Modelling*

The finite element program for this study was coded according to the method proposed by Christian (1968). The

**Table 2. Soil parameters estimated by PI for Kurashiki Trial Embankment foundation**

| Soil | Depth | PI | $\lambda$ | $\kappa$ | $e_0$ | $M_C$ | $M_E$ | $K_{0NC}$ | $\nu'$ | $k_{v0}$ (cm/sec) | $\sigma'_{v0}$ (kPa) | $\sigma'_{vc}$ (kPa) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| L-1 | 0–0.8 | 11 | (Elastic material; $E=2400$ kPa, $\nu'=0.3$) | | | | | | | | 7.2 | — |
| L-2 | 0.8–1.6 | 26 | 0.137 | 0.018 | 1.01 | 1.65 | 1.25 | 0.45 | 0.31 | $6\times10^{-7}$ | 13.6 | 112 |
| L-3 | 1.6–2.4 | 45 | 0.223 | 0.034 | 1.37 | 1.65 | 1.16 | 0.45 | 0.31 | $7\times10^{-8}$ | 19.2 | 57 |
| L-4 | 2.4–3.2 | 45 | 0.223 | 0.034 | 1.37 | 1.65 | 1.16 | 0.45 | 0.31 | $7\times10^{-8}$ | 24.0 | 57 |
| L-5 | 3.2–4.0 | 50 | 0.245 | 0.038 | 1.46 | 1.65 | 1.13 | 0.45 | 0.31 | $7\times10^{-8}$ | 28.8 | 57 |
| L-6 | 4.0–4.8 | 54 | 0.263 | 0.042 | 1.54 | 1.65 | 1.11 | 0.45 | 0.31 | $7\times10^{-8}$ | 33.6 | 57 |
| L-7 | 4.8–5.6 | 26 | 0.137 | 0.018 | 1.01 | 1.65 | 1.25 | 0.45 | 0.31 | $7\times10^{-8}$ | 39.2 | 57 |
| L-8 | 5.6–6.4 | 13 | 0.079 | 0.007 | 0.76 | 1.65 | 1.32 | 0.45 | 0.31 | $2\times10^{-7}$ | 45.6 | — |

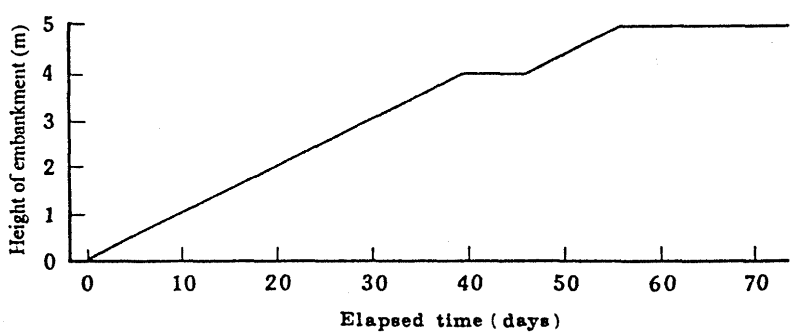



**Fig. 3. Trial embankment construction schedule**

type of solid element used in the present study is the first order iso-parametric plane-strain element with 4 nodal points. Using this type of element, the Kurashiki trial embankment foundation was modeled with 169 solid elements (201 nodal points) as shown in Fig. 4. The idealized geometry is symmetrical with respect to the centerline, so the mesh represents one half of the embankment foundation cross section.

For the boundary condition, horizontal displacements were determined along the vertical line at the center of the embankment and the right side of the foundation. In addition both horizontal and vertical displacements were determined along the bottom line of the foundation. The embankment foundation was completely drained along the ground surface (above the water level) and the bottom line of the foundation (Sekiguchi and Shibata, 1982). Inside of the embankment was also assumed to be completely drained.

Figure 5 shows the model foundation used for this research. As mentioned in Section above, considering the soil profile shown in Fig. 2 the foundation might be evaluated to be normally consolidated but it might also be occasionally that the top and bottom layers of the weak clay foundation were slightly over consolidated due to their drainage toward the outer sandy layers. In modelling the foundation, therefore, two cases were assumed: (i) normally consolidated and (ii) slightly over consolidated at the top and bottom layers of the foundation.

The simulation was carried out to follow the embankment construction schedule for 160 days after the construction began in a careful manner. Herein, the coefficient of permeability in-situ was assumed to be 3, 6 and 10 times larger than that obtained from the oedometer tests, as pointed out by Sekiguchi and Shibata (Sekiguchi and Shibata, 1982). They recommended the use of a value 6 times larger than the coefficient of permeability obtained from the oedometer tests to make better predictions because it is generally recognized that the in-situ permeability characteristic might be larger than the value obtained from the laboratory oedometer tests.

The laboratory-established soil parameters for constitutive equations of soils are expressed in terms of PI. The field modelling can be done using physical properties

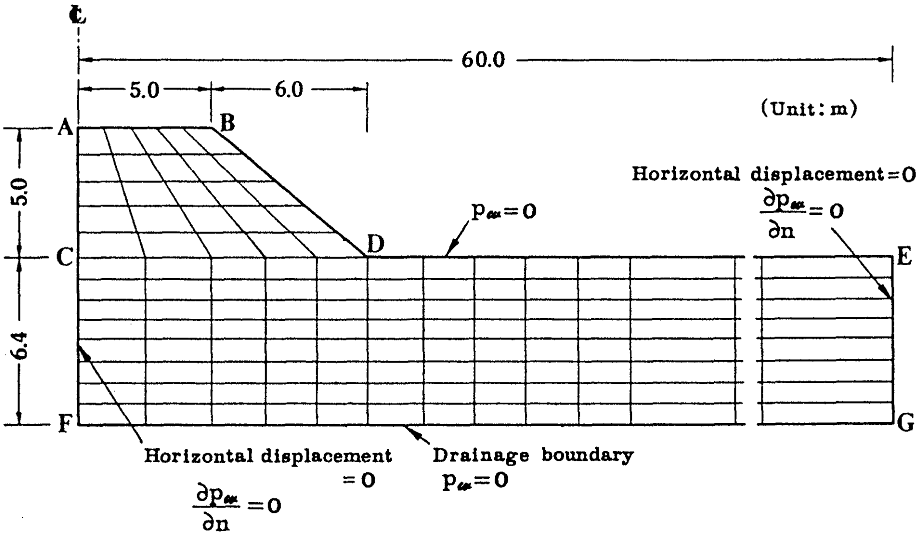



**Fig. 4. FEM mesh used in the numerical analysis**

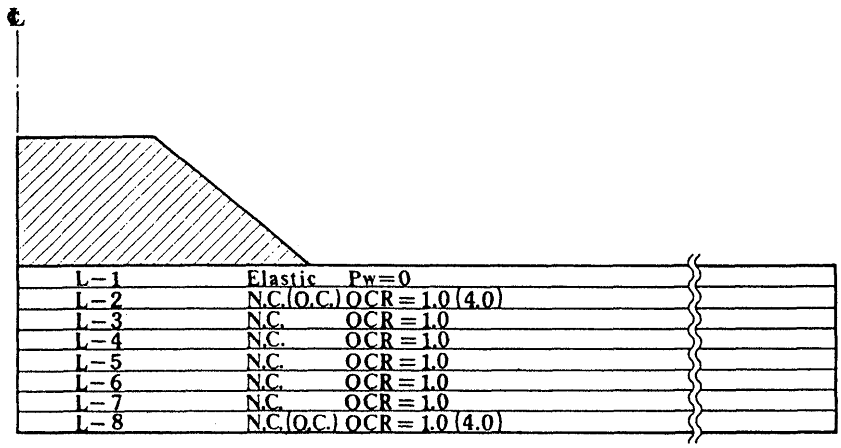



**Fig. 5. Model foundation used in the numerical analysis**

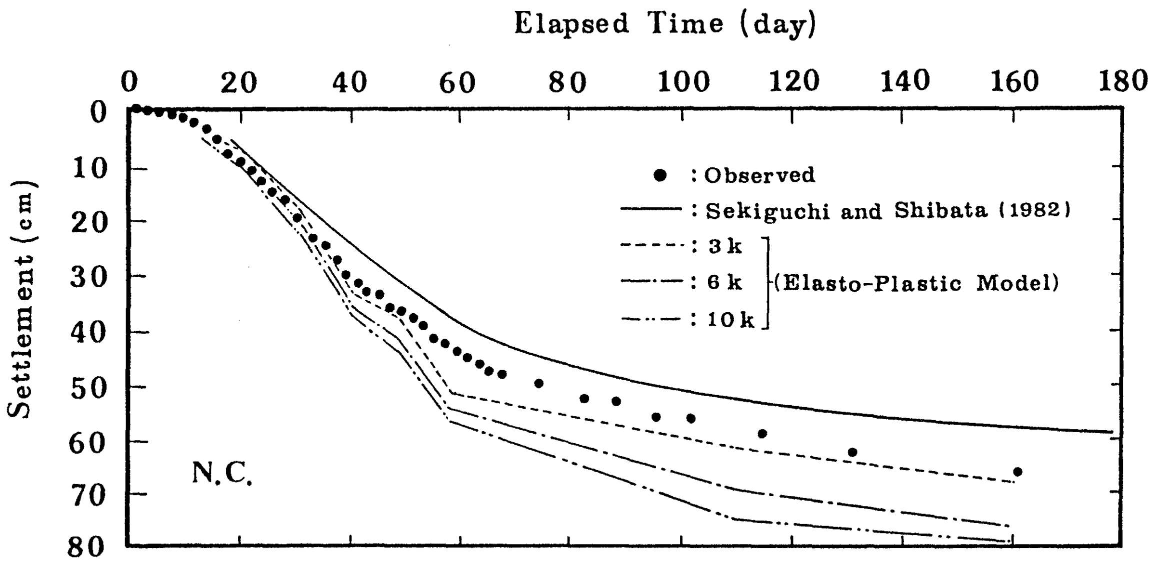



Fig. 6. Case (i): Relationship between ground surface settlements and elapsed time

such as plasticity index, liquid limit and natural water content obtained from the standard penetration test. The procedure used for deformation analysis based on this modelling and the applicability is discussed in next Section.

*Comparisons between the Computations and the Observations*

*Case (i): Considering Normally Consolidated State for the Foundation*

Initially, the first computation was carried out because it was naturally assumed that the entire area of the foundation was normally consolidated. The computed and observed ground surface settlement and elapsed time relationship at the center of the embankment is shown in Fig. 6, and the computed and observed lateral deformation near the embankment toe at the end of embankment construction is shown in Fig. 7. The ground surface settlements were actually measured with settlements plates at two points at the center of the embankment as shown in Fig. 1. Both settlements plates showed the same values until the end of the embankment construction. They differed, however, from each other and finally the difference was reached 2.6 cm. The observations plotted correspond to the data observed at the point which indicated the larger settlements. In Figs. 6 and 7, the 3 types of multiplying factors on the permeability than estimated by Eq. (11), 3, 6 and 10 times, were used to obtain a realistic permeability of the subsoil profile.

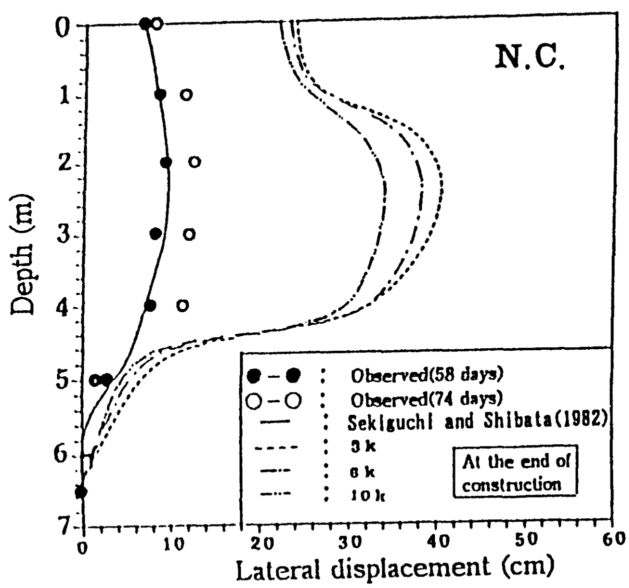



Fig. 7. Case (i): Distribution of lateral displacement at the embankment toe

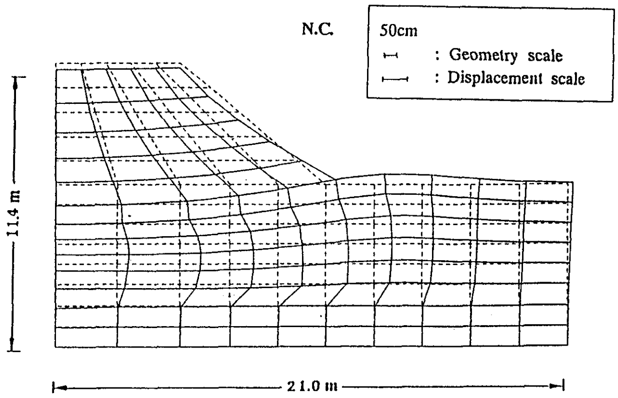
Comparing the observed results with the computed ones, the analysis using the coefficient of permeability 3 times larger than the one estimated, indicated the best agreement during the first 30 days after embankment construction began and afterwards the computations overestimated the observations only by the amount of 2 cm (3% overestimation compared to the last observed settlement) 160 days after the embankment construction began. The analyses using the coefficient of permeability 6 and 10 times larger than estimated one, showed the 2nd and 3rd best agreement during the entire period of the embankment and post construction. Each of computations overestimated the observations by 9 cm (14% overestimation against the last observed settlement) and 12 cm (18% overestimation compared to the last observed settlement) 160 days after the embankment construction began. It was found that the larger settlements were computed based on the larger multiplying factors for the coefficient of permeability.

The solid line shown in Fig. 6 is the result of the elasto-viscoplastic analysis obtained by Sekiguchi and Shibata (1982) with soil parameters and stress history of the subsoil determined on the basis of precise soil investigations. Although the analysis agreed with the observations for the first 30 days since the embankment construction, it underestimated them by the amount of 8 cm, 160 days after the embankment construction began (12% underestimation compared to the last observed settlement). This may be due mainly to a consideration of stress history in this analysis (Sekiguchi and Shibata, 1982). The authors, however, assumed the foundation to be normally consolidated.

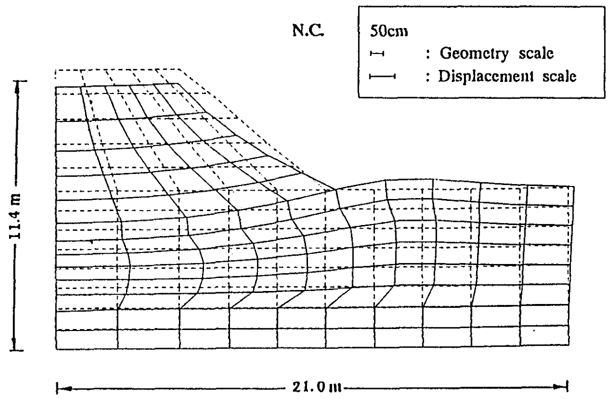
Although compared with the observations, the computed settlements overestimated the observed ones on such a simple model, it was found that the analytical procedure proposed was accurate to some degree to estimate settlement. The numerical simulations of the elasto-plastic model were therefore concluded to explain the observations very well.

In addition, with regard to lateral deformation, the computations by Sekiguchi and Shibata (1982) based on the specific soil investigations could simulate very well. The plotted observations, however, corresponding to the data indicated larger values (H-3 gauge) among the two measured. The authors' predictions exceed the observations very much, even though the lateral displacements were reduced based on the predictions which assumed a higher coefficient of permeability. According to the report by Mochizuki et al. (1980), the lateral displacement H-3 gauge of aluminum pipe did not measure because it was broken at a depth of 4.5 m 80 days after the embankment construction began. This indicated that this measured lateral deformation might contain some possible error. It must also be pointed out that the constitutive model used in the present study, which is categorized into a Cam clay model family might contain some prediction error for the lateral deformation (Nakai and Matsuoka, 1987).

Figure 8 shows the deformation computed by FEM 160 days after the embankment construction began, which gave the best agreement with the observations based on the assumption of a 3 times larger coefficient of permeability than that estimated by Eq. (11), which was derived based on the data obtained from oedometer tests.




(a) Completion of embankment construction




(b) 160 days after embankment construction began

**Fig. 8. Case (i): Computed ground deformation**

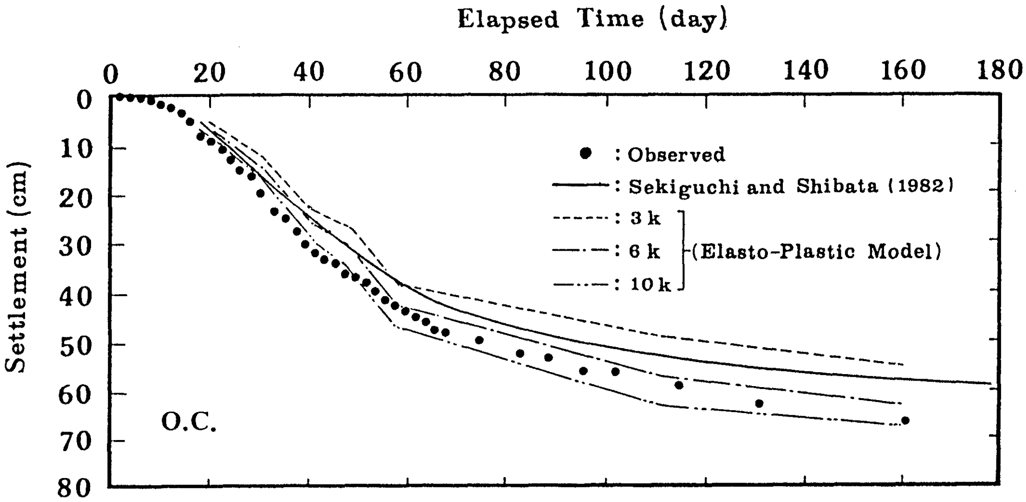



**Fig. 9. Case (ii): Relationship between ground surface settlements and elapsed time**

*Case (ii): Considering Slightly Over-Consolidated State at the Top and Bottom of the Foundation*

Succeeding Case (i), as it is widely recognized in the natural soil deposits, the authors carried out another case by assuming that the top and bottom of the foundation were slightly over-consolidated. The computed settlements at the center of the embankment are shown in Fig. 9. The lateral displacement near the embankment center at the end of the embankment construction is shown in Fig. 10. Comparing the observed results with the computed ones, the elasto-plastic analysis assuming a 10 times larger coefficient of permeability gave the best agreement with the observations for 160 days after the construction began, for the entire period of observation. 160 days after the construction began, this analysis overestimated the observation about 2 cm (3% overestimation compared to the last observed settlement). The analyses on the 6 and 3 times larger coefficient of permeability gave the 2nd and 3rd best agreement with the observations. Each computation underestimated the observation by the amount of 3 cm (5% underestimation against the last observed settlement) and 10 cm (16% underestimation against the last observed settlement) 160 days after the embankment construction began.

Comparing the computed results between Case (i) and Case (ii), it might be concluded that the computed results of Case (ii) were improved totally to be closer to the observations than those of Case (i), by reducing their settlements because of the consideration of the stress history on the top and bottom layers of the foundation. This indicates that the modelling was more realistic than Case (i) and at the same time the applicability of elasto-plastic analysis without consideration of the viscosity of cohesive soil might simulate sufficiently accurately the settlement due to consolidation. In addition, based on both numerical results for Case (i) and Case (ii), it was concluded

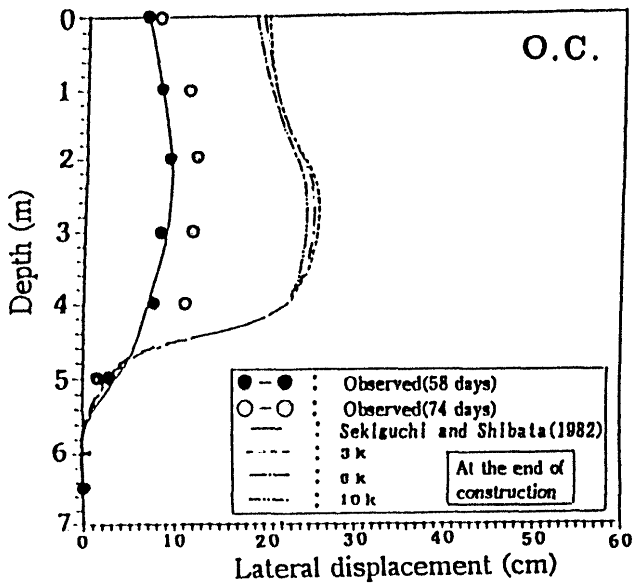



**Fig. 10. Case (ii): Distribution of lateral displacement at the embankment toe**

that in order to carry out a realistic deformation analysis, the use of a certain times larger coefficient of permeability than that obtained by the empirical correlations is very important, which may be largely dependent on the conditions of the foundations, existence of sand seams, thickness of foundation and so on. From the comparisons with another deformation analysis for a deep clay foundation with a thickness of about 20 m under embankment (Sakajo and Kamei, 1995), it was concluded that the thicker a clay foundation may be, the less multiple numbers of the coefficient of permeability estimated by PI will be adopted for this simple analytical procedure.

In addition, the in-situ coefficients of permeability can be not easily evaluated, for the following reasons: (i) the difficulty in evaluating the boundary drainage conditions, (ii) the uncertainties to determine particularly permeable strata to act as a drainage medium, and (iii) the lateral coefficient of permeability of strata is sometimes more uncertain than the vertical coefficient of permeability. In this regard (iii) above, it should be noted that the lateral coefficient of permeability was assumed equal to the vertical in the authors' modelling.

From Fig. 10, the lateral displacement, related to the assumption of the stress history of the cohesive soil, might also reduce the computed values more realistically by comparing with Case (i). In this figure, the solid line is the numerical results from the elasto-viscoplastic model by Sekiguchi and Shibata (1982). From this figure, it can also be seen that the computed lateral deformation was reduced based on the computations assumed for the higher coefficient of permeability in the same way as Case (i).

Figure 11 shows the deformation of the clay foundation under embankment computed by FEM 160 days after embankment construction began, which provided the best agreement with the observations based on the assumption of a coefficient of permeability 10 times larger. From this figure, the pattern of the deformation can be understood when the embankment is constructed.

As a result, reasonable agreements have been obtained between measured and computed values of settlement for multi soil layers. In addition, the ground surface movements below both the center and beneath the shoulder of the embankment can be shown quantitatively. The predicted lateral deformation, however, overestimates the field measurements. The present analytical method therefore still has some discrepancy with respect to the lateral deformation and in order to overcome this, careful attention is required with respect to model boundary conditions and field measurements. It should be emphasized that even if the constitutive parameters are only estimated from plasticity index, the finite element analysis gives acceptable results when compared with field performance.

A more practical analysis of non-linear elasticity or perfect elasto-plasticity than the present method based on an elasto-plastic model might also be used for the prediction of ground deformation if the soil properties are evaluated properly. The analytical results obtained using the

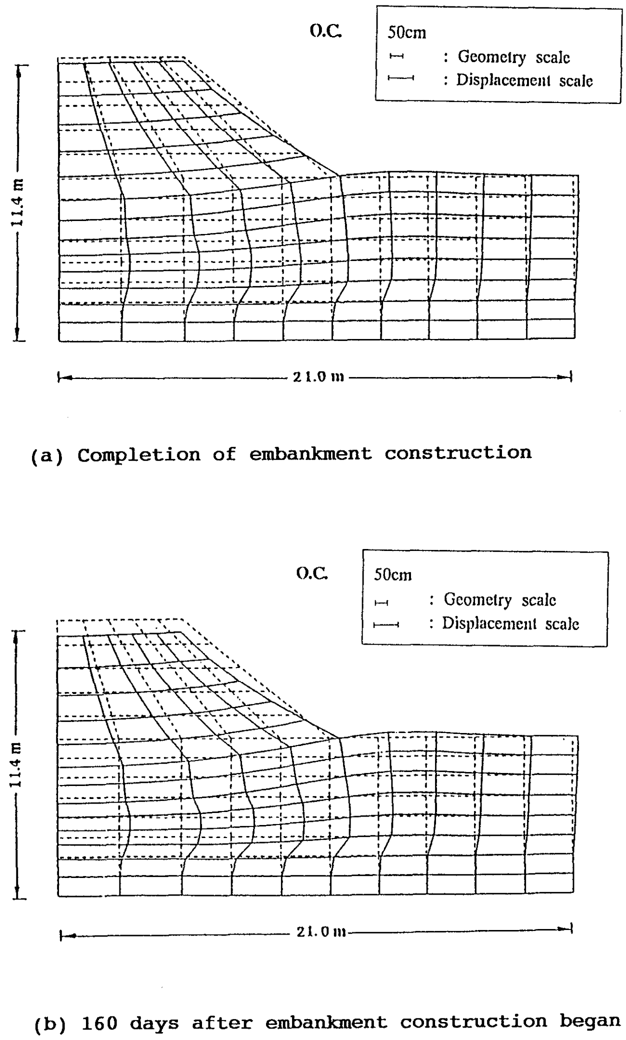



**Fig. 11. Case (ii): Computed ground deformation**

present method, however, were found to be reasonable and offer encouragement for the use of this method in engineering practice. This analysis may admittedly be suitable for preliminary work, due to idealized assumptions and uncertainty in the data. Additional study in geotechnical engineering would seem to be justified.

## CONCLUSIONS

The following conclusions were made:

1) The results of the numerical simulation were assessed both qualitatively and quantitatively by comparison with the measured field data. The numerical illustration confirms therefore the usefulness of the Sekiguchi-Ohta model and the correlations between the soil parameters for the constitutive equations of soils and the plasticity index proposed by Kamei.

2) The analytical results obtained by the present method therefore were found to be reasonable and indicate encouragement for the use of this method in engineering practice. Although the predicted lateral deformation in-situ overestimated the actual field measurements, the approach adopted was concluded to be capable of producing realistic predictions for deformation characteristics of embankment foundation.

3) This analysis can be considered suitable for preliminary work, due to idealized assumptions and uncertainty in the data. Additional study in geotechnical engineering seems to be justified, however.

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