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Simplified Deformation Analysis of Clay Foundation Under Embankment Using Elasto-Viscoplastic Model

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This paper describes a simple procedure for evaluating deformation behavior of clay foundation under embankment using the simplified method for determining the soil parameters estimated by plasticity index and elasto-viscoplastic model. In the present analysis, we used the in-situ data obtained by standard penetration test which is used worldwide in the economical points of view to estimate the soil parameters and the stress state of the clay foundation. Reasonable agreements have been obtained between the measured and the computed values of the ground surface movements beneath the center and the shoulder of the embankment for the multi-soil layers. The predicted lateral deformation, however, overestimates the field measurements. Finally, the analytical results obtained by the present methods are found to be reasonable in the preliminary design, owing to idealizing assumptions and uncertainty in the data, therefore, it is encouraged to use the presented methods in engineering practice of embankment on normally consolidated cohesive soil stratum. The applicability of the present method to other fields will require further research.

Introduction

Vertical and horizontal deformations for foundations under embankments are often large and potentially damaging to structures. The embankment fill applies vertical load to the foundation surface in combination with an outward shear stress caused by the horizontal stresses in the fill. Evaluating their magnitudes, and the rate at which they will occur, plays an important part in many geotechnical engineering projects. In addition, deformation analysis of the clay foundations under embankments is one of the necessary steps in embankment design. This deformation analysis becomes increasingly important when embankments are constructed over weak (normally consolidated cohesive stratum) significantly.

It has been a few decades since geotechnical researchers developed theoretical soil mechanics, sophisticated laboratory, in-situ testing devices and the finite element method (FEM) with the powerful computers. Most researchers who are familiar with them developed the complementary approaches to performance for predicting prototype mechanical behavior with any real confidence. 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 this sort of deformation analyses largely depends on the above factors. In addition, such analyses must be practical, not be too costly to employ, and have a theoretical basis.

Efforts have been made to correlate engineering properties of soils to simple soil indices. Such correlations play an important role in practice by providing evaluates of soil parameters for use in preliminary design and serve as a check on data obtained from laboratory and in-situ tests.

The standard penetration test (SPT), on the other hand, is used worldwide to a greater extent than any other in-situ test. The SPT has several significant advantages: (i) the equipment is relatively simple and rugged, (ii) the procedure is easy to carry out and permits frequent tests, (iii) a sample of the soil is usually obtained, (iv) tests can be carried out in most soil types, and (v) many useful correlations have been developed. No other in-situ test combines this range of flexibility (Robertson et al., 1992).

As mentioned above, the accurate prediction of the performance of embankments founded on soft soils requires the selection of the appropriate soil parameters for analysis. This is true regardless of the degree of sophistication of both the soil model and the method of analysis. In addition, the accurate prediction of the consolidation behavior of foundation clays is important not only for computing settlements but also for determining strength gain for staged loading and for preloading operations.

The purpose of this paper is to investigate the mechanical behavior of clay foundation under embankment using elasto-viscoplastic model and, constitutive parameters and field modelling for deformation will be using only the in-situ data obtained by standard penetration test in the preliminary design point of view. In addition, we also evaluate whether it is possible to estimate the undrained shear behavior of clay using elasto-viscoplastic model proposed by Sekiguchi and Ohta and constitutive parameters estimated by plasticity index (Kamei, 1985), if so, how accurate these predictions are. In the present analysis, we will be using only the in-situ data obtained by standard penetration test which is used worldwide in the economical points of view.

Elasto-Viscoplastic Model Proposed by Sekiguchi and Ohta

The constitutive equations used in this present study is the model which can explain the elemental mechanical behavior of soil considering the initial anisotropic stress condition and the rotation of principal stresses on the process of shearing, proposed by Sekiguchi and Ohta (Sekiguchi, 1977; Sekiguchi and Ohta, 1977; Ohta and Sekiguchi, 1979). Sekiguchi and Ohta model is the modified model to explain the effect of the natural clayey deposits at K_0-consolidated condition and the viscosity inherent to clayey soil. It has been widely used in Japan to simulate the deformation of weak clayey soil deposit under embankment. The model can be briefly explained as follows.

In the above mentioned model, the relationship between volumetric strain ϵ_V and volumetric strain rate $\dot{\epsilon}_V$ can be expressed as follows:

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

Where $\dot{V}_0$ is initial volumetric strain rate, D is the coefficient of dilatancy (Ohta and Sekiguchi, 1979), and η^* is shear stress ratio to consider K_0-consolidated stress condition at in-situ expressed as the following equation:

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

η_{ij} can be expressed as follows:

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

By solving Equation (1), visco-plastic volumetric strain ϵ_V^{vp} can be obtained as the following equations:

$$\epsilon_V^{vp} = \alpha\cdot\ln\left\{1+\frac{\dot{V}_0\cdot t}{\alpha}\exp\left(\frac{f}{\alpha}\right)\right\} \equiv F \quad \left(f \equiv \frac{\lambda-\kappa}{1+e_0}\ln\frac{p'}{p'_0} + D\cdot\eta^*\right) \tag{4}$$

where, the F is the visco-plastic potential. Applying the normality rule to this potential, the following stress-strain relationship can be derived:

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

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

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

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

Integrating Equation (5) on a time increment Δt and taking a matrix expression form, the following incremental elasto-viscoplastic constitutive equation can be obtained:

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

where, $\Delta\widehat{\sigma}$ is a general stress increment matrix, $\Delta\widehat{\epsilon}$ is a general strain increment matrix, $\Delta\widehat{\sigma}_r|_t$ is a stress relaxation matrix due to viscosity and $\widehat{D}^{evp}$ is a stress-strain matrix.

These matrices can be expressed in the triaxial stress-strain space as follows:

$$\Delta \widehat{\sigma} = \begin{bmatrix} \Delta\sigma_{xx} \\ \Delta\sigma_{yy} \\ \Delta\sigma_{xy} \\ \Delta\sigma_{zz} \end{bmatrix} \tag{9}$$

$$\Delta \widehat{\epsilon} = \begin{bmatrix} \Delta \epsilon_{xx} \\ \Delta \epsilon_{yy} \\ \Delta \epsilon_{xy} \\ \Delta \epsilon_{zz} \end{bmatrix} \tag{10}$$

$$\Delta \widehat{\sigma} \,|_t = \Delta t \cdot \frac{C_4}{C_2} \begin{bmatrix} A_{xx} \\ A_{yy} \\ A_{xy} \\ A_{zz} \end{bmatrix} \tag{11}$$

$$\widehat{D}^{evp} = \begin{bmatrix} L+2G & L & 0 \\ L & L+2G & 0 \\ 0 & 0 & G \\ L & L & 0 \end{bmatrix} - \frac{C_3}{C_2} \begin{bmatrix} A_{xx}^2 & A_{xx}A_{yy} & A_{xy}A_{xx} \\ A_{xx}A_{yy} & A_{yy}^2 & A_{xy}A_{yy} \\ A_{xx}A_{zz} & A_{yy}A_{xy} & A_{xy}^2 \\ A_{xx}A_{zz} & A_{yy}A_{zz} & A_{xy}A_{zz} \end{bmatrix} \tag{12}$$

where, C_2, C_3, C_4, A_{ij}, f_{kk} and f_{ij} are given as follows:

$$C_2 = [L \cdot f_{kk}^2 + 2 \cdot G\,(f_{xx}^2 + f_{yy}^2 + 2 \cdot f_{xy}^2 + f_{zz}^2)] \cdot C_3 + f_{kk} \tag{13}$$

$$C_3 = 1 - \exp\,(- \,\epsilon_V^{vp}/\alpha) \tag{14}$$

$$C_4 = \dot{V}_0 \cdot \exp\,\{(f - \,\epsilon_V^{vp})/\alpha\} \tag{15}$$

$$A_{ij} = L f_{kk} \delta_{ij} + 2 \cdot G f_{ij},\ (i, j = x, y, z) \tag{16}$$

$$f_{kk} = f_{xx} + f_{yy} + f_{zz} \tag{17}$$

$$f_{ij} = \frac{D}{3p'} \left\{ M - \frac{3}{2\eta^*} \eta_{kl} (\eta_{kl} - \eta_{klo}) \right\} \delta_{ij} + \frac{3D}{2\eta^* p'} (\eta_{ij} - \eta_{ijo}) \tag{18}$$

where, the L and the G are Lamé's constants.

Choice of Soil Parameters

It is 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. Kamei (1985), Nakase and Kamei (1988) and Nakase et al. (1988) carried out an extensive testing program to study the mechanical behavior of cohesive soils over a range of plasticity index from PI = 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 the test data, they proposed linear correlations between undrained shear stregnth and some soil parameters for constitutive equations and plasticity index. Although it has been shown that each individual soil parameter appearing in the constitutive equations can be expressed in terms of *PI* with reasonable accuracy. These correlations are summarized in Table 1, in which the *r* means the coefficient of correlation in linear regression analyses.

In the table, the soil parameters for the constitutive equations (compression index λ, swelling index κ, 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, coefficient of secondary compression C_α) from plasticity index *PI*, were obtained. Herein the specific volume $N\ (=1+e_0)$ is correpsonding to the mean effective principal stress $p'=98\,\text{kPa}$. C_α is defined as the ratio of decrement in e to log t during secondary compression ($C_\alpha = \Delta e/\Delta \log t$). The coefficient of earth pressure at rest for normally consolidated soil K_{ONC} and the coeffcient of earth pressure at rest for

Table 1. A set of soil paramenters estimated by PI (Kamei, 1985)

Parameter (1)	r (2)
$\lambda=0.02+0.0045\cdot\text{PI}$	0.98
$\kappa=0.00084\cdot(\text{PI}-4.6)$	0.94
$\text{N}=1.517+0.019\cdot\text{PI}$	0.95
$\text{M}_\text{C}=1.65$	—
$\text{M}_\text{E}=1.385-0.00505\cdot\text{PI}$	0.85
$C\alpha=0.00168+0.00033\cdot\text{PI}$	0.96
$\text{K}_\text{ONC}=0.45$	—
$\text{K}_\text{OOC}=\text{K}_\text{ONC}\cdot(\text{OCR})^{0.45}$	—
$\upsilon'=\text{K}_\text{ONC}/(1+\text{K}_\text{ONC})$	—
$\dot{\text{V}}_\text{O}=\alpha/\text{t}\,\vert\,\text{t}=\text{t}_\text{c}$	—
$\alpha=0.434\cdot C\alpha/(1+\text{e}_\text{o})$	—
$\text{k}_\text{v}=\text{k}_\text{vo}\cdot\exp\ \{(\text{e}-\text{e}_\text{o})/\lambda_\text{k}\}$	—
$\lambda_\text{k}=0.073+0.019\cdot\text{PI}$	0.98

r: coefficient of correlation in linear regression analyses overconsolidated soil K_{OOC} in Japan are proposed by Kamei and Sakajo (1993).

Furthermore the initial volumetric strain rate $\dot{V}_0$ is necessary as a input parameter for the elasto-viscoplastic model proposed by Sekiguchi and Ohta as expressed as follows:

$$\dot{V}_0 = \alpha / t \,|\, t = t_C \tag{19}$$

where, α is the secondary compression index and can be obtained as the following equation:

$$\alpha = 0.434 \cdot C\alpha / (1 + e_0) \tag{20}$$

The t_c is corresponding to the completion time of the primary consolidation, if secondary consolidation is assumed to start after the primary consolidation finished.

On the other hand, the vertical coefficient of permeability k_v can be obtained as follows:

$$k_v = k_{v0} \cdot \exp\{(e - e_0) / \lambda_k\} \tag{21}$$

where, the k_{v0} can be obtained as the coefficient of permeability corresponding to a mean principal stress of $p' = 98$ kPa (Kamei, 1985) and e_0 is the void ratio obtained from N.

The gradient of linear relationship between void ratio and logarithmic coefficient of permeability λ_k is defined by the following equation through plasticity index (Kamei and Sakajo, 1993) as follows:

$$\lambda_k = 0.073 + 0.019 \cdot PI \,(r = 0.984) \tag{22}$$

It would be interesting to investigate possible numerical analysis of undrained shear behavior of normally consolidated cohesive soil using elasto-viscoplastic model. Numerical analysis now make it possible for complete solutions to be obtained to the complex problems that arise in geotechnical engineering. Ideally, calculation should be carried out with one method of analysis and one soil model, with one consistent set of soil parameters.

Model Simulation of Undrained Shear Behavior of K_0-Consolidated Cohesive Soils

To investigate the time-dependence of undrained shear behavior for cohesive soils, undrained triaxial compression and extension loadings have been carried out after K_0-consolidation. The Sekiguchi-Ohta model combined with Kamei's parameters can be applied to the simulation of the time-dependent behavior of K_0-consolidated cohesive soils.

Herein, one elemental analysis is carried out using a finite element program coded as the method proposed by Christian (1968).

The predicted results are compared with the test results obtained for normally consolidated cohesive soil (Nakase and Kamei, 1983 and 1986) in the present paper. These experimental data was provided regarding Kawasaki clay (K-30), according to their values of the plasticity index (*PI*). The notation, K, stands for Kawasaki clay. The preparation method of the soil samples has been reported elsewhere (Kamei, 1985; Nakase and Kamei, 1988; Nakase et al., 1988). The physical properties are shown in Table 2. The Sekiguchi-Ohta model is used for this illustration. The input data of the soil parameters used in the analysis is given in Table 3.

Herein, the numerical results obtained using the Sekiguchi-Ohta model and the soil parameters estaimated by plasticity index will be compared to the results of K_0-consolidated undrained traixial compression and extension loadings of K-30. The strain controlled tests were performed and the prescribed strain rates per minute are three kinds of 0.7%/min., 0.07%/min. and 0.007%/min. (Nakase and Kamei, 1986).

Figs. 1 and 2 show the comparison between simulated and typical experimental results of stress-strain behavior and effective stress paths for the normally consolidated soil when the sample named K-30 was subjected to the three kinds of strain rates after the K_0-consolidation.

Fig. 1 shows typical stress-strain curves of the specimens in the triaxial tests, where the principal stress difference $q = \sigma_a - \sigma_r$ is normalized by dividing by the vertical effective consolidation pressure σ'_{vc}.

Typical effective stress paths are shown in Fig. 2, where the principal stress difference q and the mean effective stress $p' = (\sigma'_a + 2\sigma'_r)/3$ are normalized by dividing by the vertical effective consolidation pressure σ'_{vc}. As seen in these figures, reasonable agreements have been obtained between the measured and the computed results of undrained triaxial compression and extension loadings for remoulded Kawasaki clay. Although details of the simulated results are different from the experimental results, the model simulation is capable of predicting qualitatively the salient features of time-dependent behavior for the K_0-consolidated cohesive soil.

Table 2. Index properties of soils studied

SOIL	ρ_s(g/cm^3)	w_L(%)	w_P(%)	PI	Sand(%)	Silt(%)	Clay(%)
Kawasaki clay	2.69	55.3	25.9	29.4	16.1	61.6	22.3

Table 3. Estimated soil properties for undrained shear behavior predictions

SOIL	PI	λ	κ	e_0	K_{ONC}	M_C	M_E	$C\alpha$	α(%)
Kawasaki clay	29.4	0.16	0.021	1.07	0.45	1.65	1.24	0.011	0.24

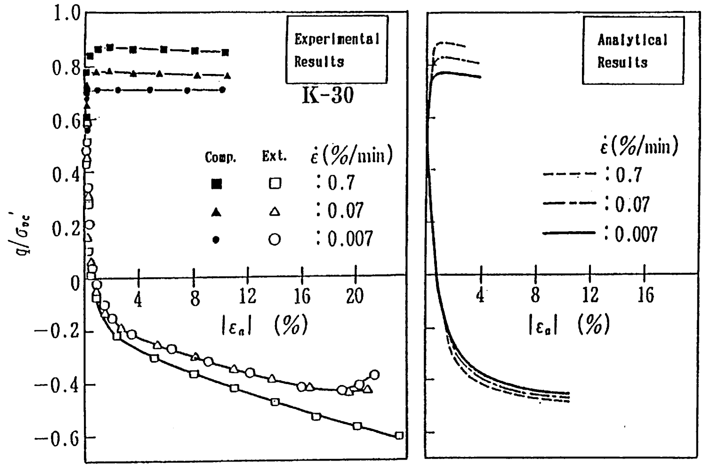


Fig. 1. Comparison of the observed and the computed stress-strain behavior for remolded Kawasaki clay

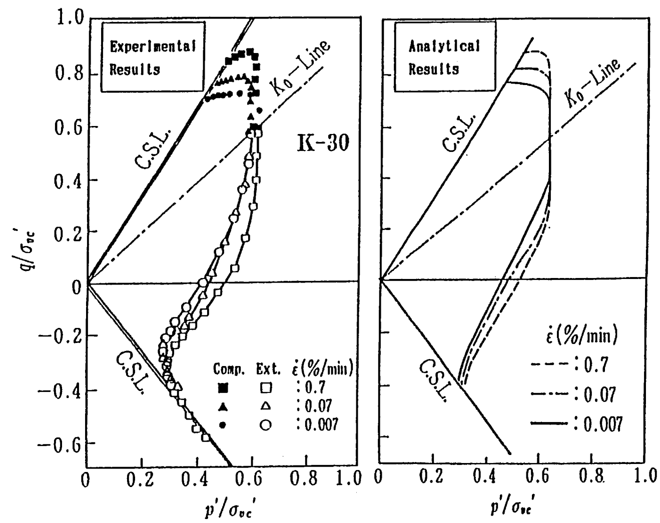


Fig. 2. Comparison of the observed and the computed effective stress paths for remolded Kawasaki clay

Numerical Analysis

Simplified procedures used for deformation analysis based on the results of in-situ liquid limit, plastic limit and natural water content and elasto-viscoplastic model proposed by Sekiguchi and Ohta are introduced, and the applicability of the present method for estimating associated ground deformations is discussed. The field modelling for deformation behavior is expressed in terms of field parameters such as plasticity index, liquid limit and natural water content from the standard penetration test.

Although it has been shown that each individual soil parameter appearing in the constitutive equations can be expressed in terms of *PI* with reasonable accuracy, it may be appropriate to demonstrate the overall accuracy of the relationships obtained from this series of tests by computing a deformation characteristics of embankment foundation on a soft multi layer ground.

Herein, the numerical results obtained using the assumed model ground, Sekiguchi-Ohta model and the soil parameters estimated by plasticity index will 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). A cross-section of the embankment is shown in Fig. 3 and settlement gage, extensometer, displacement pile, inclinometer, settlement plate and piezometer locations in Fig. 3.

(1) Outline of Kurashiki Trial Embankment

Kurashiki Trial Embankment was constructed to check the safety against the vertical and horizontal deformation because the limitation of the construction area. The embankment construction is explained generally as follows.

The plane view and cross-section of Kurashiki Trial Embankment foundation is shown in Fig. 3. Kurashiki Trial Embankment is located at Tamashima District of

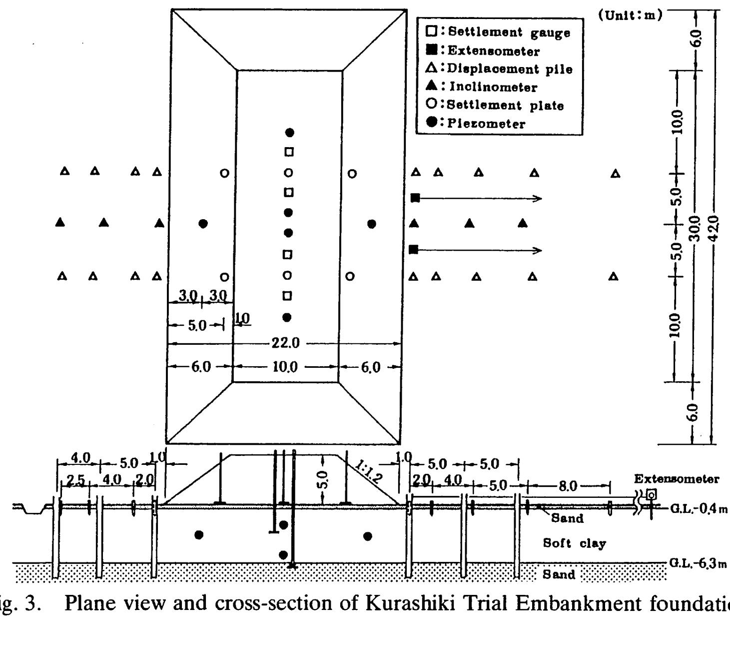


Fig. 3. Plane view and cross-section of Kurashiki Trial Embankment foundation

Kurashiki City in Japan. Facing the Seto-naikai Sea, the level of the ground surface is almost same with the sea water level (T. P. + 0.0 m) and the foundation is the typical marine clayey weak 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 is deposited from the ground surface to a depth of 6.3 m from the ground surface. The lower layer of this deposit is sandy soil with a Standard Penetration Number of 15. The coefficient of permeability of the sandy layer is almost 0.08 (cm/sec), measured using the tube method which the Young's modulus was measured as Ep = 1470 kPa with pressure meter tests. A 40 cm thick sand mat was placed on the original surface of the weak cohesive soil before embankment construction. The consolidation due to the weight of the sand mat was almost complete. The ground water level was measured and found to be at nearby the bottom of the mat. The soil profile of this foundation is shown in Fig. 4. The values of natural water content of the foundation are almost the same with the values of its liquid limit. This weak cohesive soil, therefore, may be evaluated to be normally consolidated state.

In the soil investigation of this site, some undisturbed samples were taken from the foundation for a series of laboratory tests to define soil parameters (Sekiguchi and Shibata, 1982). In this research, however, the authors estimated these soil parameters by the plasticity index only as shown in Fig. 4 from the engineering points of view. Table 4 shows the all soil parameters estimated in this manner.

A number of measuring apparatuses, ground surface settlement piles, screw type soil layer settlement gauges, ground surface movement piles, extensometers, inclinometers and piezometers were set to measure the ground deformation as shown in Fig. 3. According to the observational data, it was found that these appratuses could measure very well the ground deformation except the excess pore water pressure behavior. The

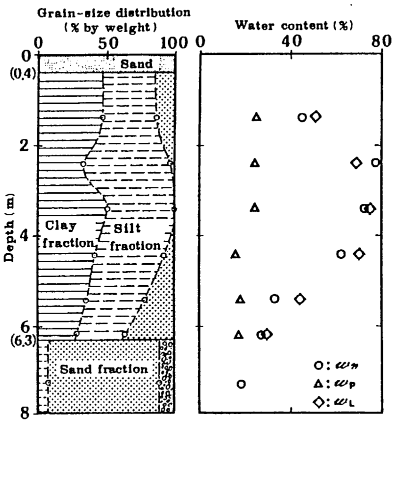


Fig. 4. Soil profile of Kurashiki Trial Embankment foundation

Table 4. Soil parameters estimated by PI for Kurashiki Trial Embankment foundation

SOIL	DEPTH (m)	PI	λ	κ	e_0	M_C	M_E	$C\alpha$	$\alpha(\%)$	K_{ONC}	υ'
L-1	0–0.8	11	(Elastic Material)								
L-2	0.8–1.6	26	0.137	0.018	1.01	1.65	1.25	0.0103	0.2216	0.45	0.31
L-3	1.6–2.4	45	0.223	0.034	1.37	1.65	1.16	0.0165	0.3027	0.45	0.31
L-4	2.4–3.2	45	0.223	0.034	1.37	1.65	1.16	0.0165	0.3027	0.45	0.31
L-5	3.2–4.0	50	0.245	0.038	1.46	1.65	1.13	0.0182	0.3200	0.45	0.31
L-6	4.0–4.8	54	0.263	0.042	1.54	1.65	1.11	0.0195	0.3330	0.45	0.31
L-7	4.8–5.6	26	0.137	0.018	1.01	1.65	1.25	0.0103	0.2216	0.45	0.31
L-8	5.6–6.4	13	0.079	0.007	0.76	1.65	1.32	0.0060	0.1470	0.45	0.31

two of three piezometers were not recorded the pore water pressure. The reason for this is due to breaking of the electric cables after the embankment construction end. The comparisons between the observations and the computations, which will be discussed later in Session (3), therefore are limited to only ground surface settlements and lateral deformation.

The material of embankment was Masa (a local sandy soil in Japan) and it was placed in 30 cm thickness at each scattering with bulldozers after 2 days intervals (the speed of the embankment construction was 9.5 cm/day on an average considering the effect of embankment self settlement due to its weight.). A break for nine days was taken once while the whole construction duration at the moment when the embankment height reached 3.9 m. It was reported that it had taken 58 days to complete this embankment construction.

(2) Finite Element Analyses, Boundary Conditions and Their Modelling

The finite element program in this present study was coded according to the method proposed by Christian (1968). The 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, Kurashiki Trial Embankment foundation was modeled with 169 solid elements (201 nodal points) as shown in Fig. 5. The finite element mesh and the boundary conditions are also shown in Fig. 5. The idealized geometry is symmetrical about the centerline, so the mesh represents are half of a cross-section through the cutting.

About the boundary condition, horizontal displacements were fixed along the vertical line at the center of the embankment and the right side of the foundation. The both horizontal and vertical displacements were fixed along the bottom line of the foundation. Concerning the drainage condition, on the other hand, it was completely drained along the ground surface (above the water level) and the bottom line of the foundation (Sekiguchi and Shibata, 1982). The inside of embankment was assumed completely drained.

Fig. 6 shows the model foundation for this research. As mentioned in Session (1), from the soil profile in Fig. 4 the foundation might be evaluated to be normally consolidated state, however, it might be sometimes found that the top and bottom layers of the weak clay foundation were slightly over consolidated state due to their drainage toward the outer sandy layers. In modelling the foundation, therefore, there were assumed two cases: (i) normally consolidated state and (ii) slightly over consolidated state at the top and bottom layers of the foundation.

Fig. 7 shows the embankment construction schedule. The simulation was carried out to follow the above embankment schedule for 160 days after the construction started

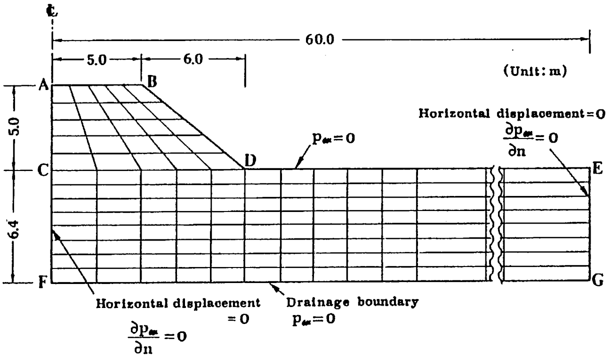


Fig. 5. FEM mesh used in this numerical analysis

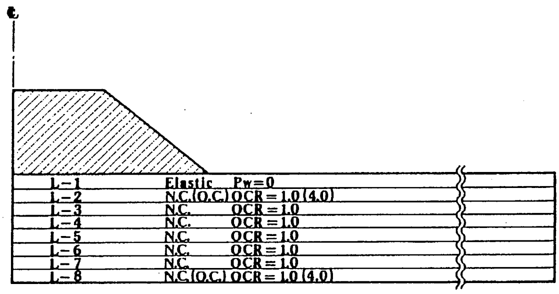


Fig. 6. Model foundation used in this numerical analysis

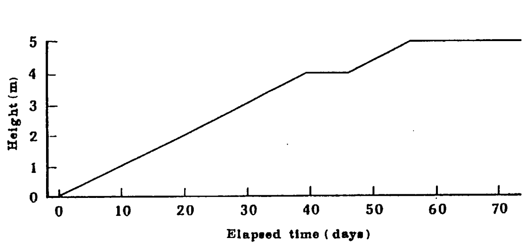


Fig. 7. Trial embankment schedule

in the careful manner. Herein, laboratory permeability tests frequently do not give results indicative of the actual in-situ permeability of many grounds. It is usually very difficult to predict with confidence the settlements. The drainage conditions are in general difficult to establish, since thin sand or silt seams, fissures and anisotropy present in the soil affect the consolidation rate (Leonards, 1962). In general, the in-situ coefficients of permeability are larger than the ones obtained by the laboratory tests. Measurements in normally consolidated clay in Swedish indicates that the permeability of the clay in the horizontal direction is two to five times larger than the permeability in the vertical direction (Broms and Hansbo, 1981). Sekiguchi and Shibata (1982) recommended to use the 6 times larger coefficient of permeability obtained by the oedometer tests to have the better predictions because it is generally recognized that the in-situ permeability characteristic might be larger than the value obtained by the laboratory oedometer tests. As mentioned above, the coefficient of permeability at in-situ was assumed 3, 6 and 10 times larger than that obtained from the oedometer test in the present study.

The laboratory-establihsed soil parameters for constitutive equations of soils are expressed in terms of *PI*. The field modelling for deformation behavior is expressed in terms of field parameters such as plasticity index, liquid limit and natural water content from the standard penetration test. This procedure used for deformation analysis based on the above modelling and the applicability will be discussed in Session (3).

(3) Comparisons Between the Computations and the Observations

In the above analyses, two cases of modelling were computed: (i) normally consolidated state in the foundation and (ii) slightly overconsolidated state at the top and bottom layers of the foundation.

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

At the beginning, the first case of computation was carried out because it was assumed naturally that the all area of the foundation was normally consolidated state. The computed and the observed ground surface settlement and elapsed time relationship at the center of embankment is shown in Fig. 8, and the computed and the observed lateral deformation near the embankment toe at the embankment construction end is shown in Fig. 9. The ground surface settlements were actually measured with settlements plates at two points at the center of embankment as shown in Fig. 3. The both of them showed the completely same values until the end of the embankment construction, however, they differed each other and finally reached 2.6 cm difference. The plotted observations correspond to the observed data at the point showed the larger settlements. In these Figs. 8 and 9, the 3 kinds of multiplying factors on the coefficient of permeability than the estimatd one by Equation (21), 3, 6 and 10 times, were

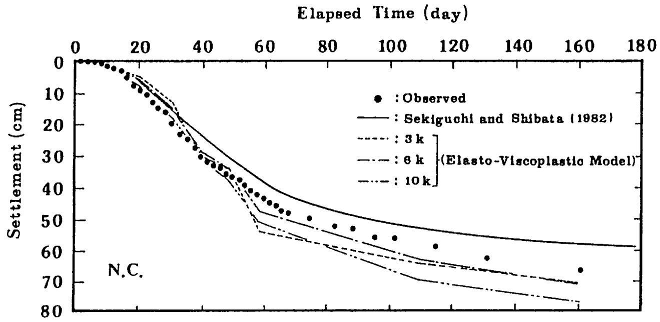


Fig. 8. Case (i): Relationship between the ground surface settlements and elapsed time

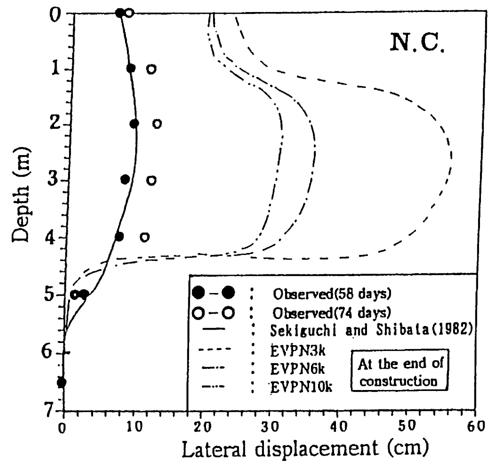


Fig. 9. Case (i): Distribution of the lateral displacement at embankment toe

employed to find a realistic permeability of the ground.

Comparing the observed results with the computed ones in Fig. 8, the analysis using the 6 times larger coefficient of permeability than the estimated one explained the observation qualitatively and quantitatively better than the other two cases and the elasto-viscoplastic analysis by Sekiguchi and Shibata (1982) with soil parameters and stress history of the ground determined on the basis of the precise soil investigations. This analysis underestimated the observation little during the first 50 days after embankment construction started and it overestimated the observation only by the amount of 5 cm (7% overestimation against the last observed settlement) at the moment 160 days passed since the embankment construction started.

The analysis using the 10 times larger coefficient of permeability than the estimated one, has the 2nd best agreement with the observation during all the period the embankment construction and the post construction. This computation has the best agreement with the observation among the three cases during the first 50 days since embankment construction started. The analysis using the 3 times larger coefficient of permeability than the estimated one yields the poorest simulation on the settlements among the three cases, in which there happened an instability in computation because of rheological clay model in the almost undrained condition caused by the very low permeability of foundation. Therefore, it was generally concluded that the larger settlements were computed with the larger multiplying factors for the coefficient of permeability.

From the above comparisons between the computations and the observations, the numerical analyses on such a simple model overestimated the observation, however, it was found that the procedure of analyses proposed could have some certain accuracy to estimate settlement from the engineering points of view.

In addition, about lateral deformation, the computations by Sekiguchi and Shibata (1982) based on the specific soil investigations can simulate it very well, however, the plotted observations correspond to the data showed the larger values (H-3 gauge) among two measuring. On the contrary, the authors' predictions exceed the observations very much, even though the lateral displacements were reduced on the predictions assumed on the higher coefficient of permeability. According to the report by Mochizuki et al. (1980), the lateral displacement H-3 gauge of aluminum pipe was not able to measure by breaking at a depth of 4.5 m after 80 days passed since the embankment construction started. This indicated that this measured lateral deformation might be underestimated and there was a possibility for this data to contain some error.

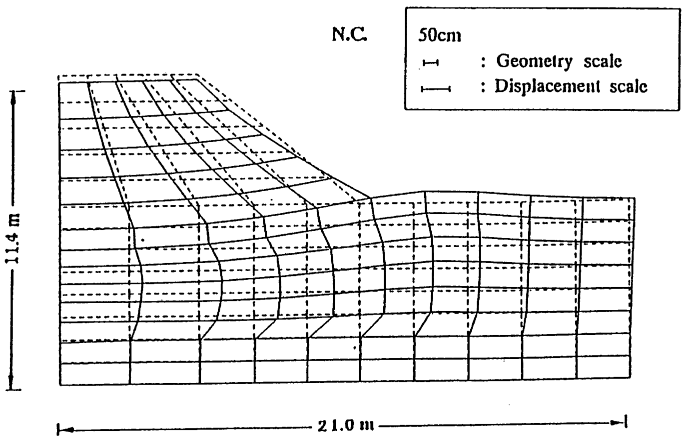
Fig. 10 shows the deformation computed by FEM at the moment of 160 days passed since the embankment construction started, which gave the best agreement with the observations on assuming the 3 times larger coefficient of permeability than that estimated by Equation (21), which was derived on the data obtained by oedomenter tests.

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

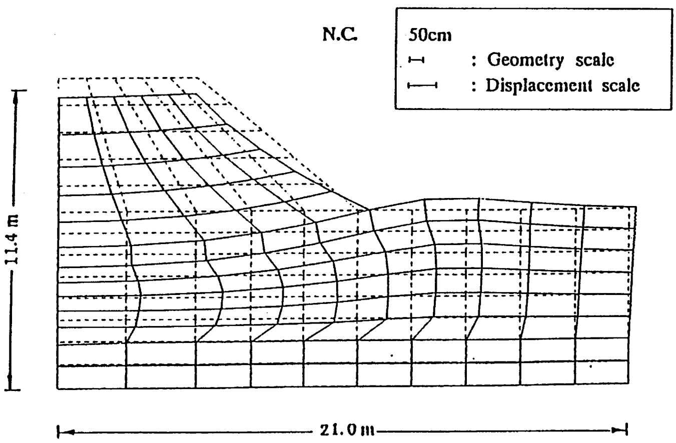
Succeeding Case (i), as it is widely recognized in the natural soil deposits, the authors carried out the another case by assuming that the top and bottom of the foundation were slightly overconsolidated state. The computed settlements at the center of embankment is shown in Fig. 11. The lateral displacement near the embankment center at the end of the embankment construction end is shown in Fig. 12.

Comparing the observed results with the computed ones in Fig. 11, the analysis assumed on the 10 times larger coefficient of permeability yielded the best simulation for 160 days after the construction started. At the moment of 160 days after the construction started, this analysis overestimated the observation about 1 cm only (1% overestimation against the last observed settlement). This analysis simulate very well qualitatively and quantitatively the observation. On the other hand, the analyses on the 6 and 3 times larger coeffient of permeability have the 2nd and 3rd best agreement with the observations.

Comparing the computed results with the observations for 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.



(a) At the completion of embankment construction



(b) 160 days since embankment construction commenced

Fig. 10. Case (i): The computed ground deformation
(a) At the completion of embankment construction
(b) 160 days since embankment construction commenced

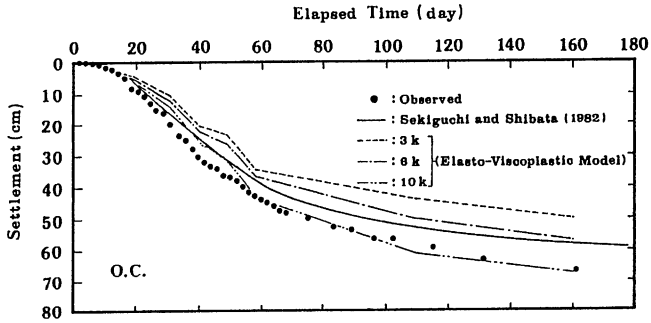


Fig. 11. Case (ii): Relationship between the ground surface settlements and elapsed time

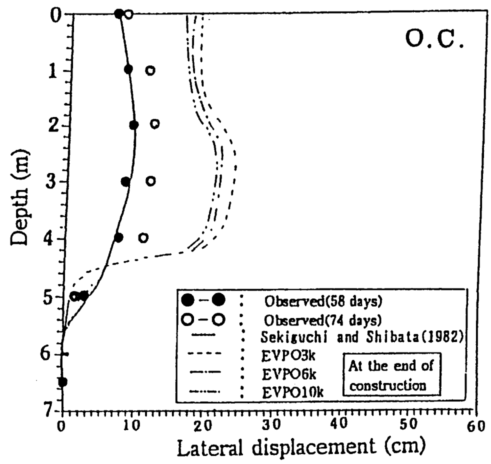


Fig. 12. Case (ii): Distribution of the lateral displacement at embankment toe

This shows the above modelling was more realistic than Case (i) and at the same time the applicability of elasto-plastic analysis not considering the viscosity of cohesive soil might simulate enough accurately the settlement due to consolidation on the engineering level. In addition, from the both numerical results of Case (i) and Case (ii), it is concluded 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 oedometer tests is very important, which may be largely dependent on the conditions of the foundations, existence of sand seams, thickness of foundation and so on.

Furthermore, on in-situ coefficients of permeability, it is not easy to evaluate them, because of the following reasons: (i) the difficulty in evaluating the boundary drainage conditions, (ii) the uncertainties to find particularly permeable strata to act as drainage strata, and (iii) the lateral coeffcient of permeability of strata is sometimes more obscure than the vertical coefficient of permeability. Regarding above (iii), it must be noticed that the lateral coeffcient of permeability was treated equal to the vertical one on the authors' modelling, even so, their numerical results can be evaluated very worthy to modelling on the in-situ coefficient of permeability on engineering practice.

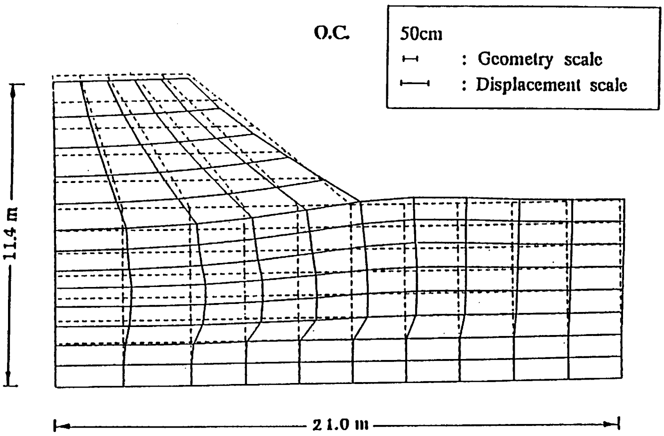
On the other hand, from Fig. 12 showing the lateral displacement, this assumption on the above modelling might also work to reduce the computed values more realistically by comparing with Case (i). In this figure solid line is the numerical results on the elasto-viscoplastic model by Sekiguchi and Shibata (1982). From this figure, it can be also found that the computed lateral deformation was reduced on the computations assumed on the higher coefficient of permeability in the same way with Case (i).

Fig. 13 shows the deformation computed by FEM at the moment 160 days passed since the embankment construction started, which gave the best agreement with the observations on assuming the 10 times larger coefficient of permeability. From the figure, the pattern of the deformation might be understood when the embankment would be constructed.

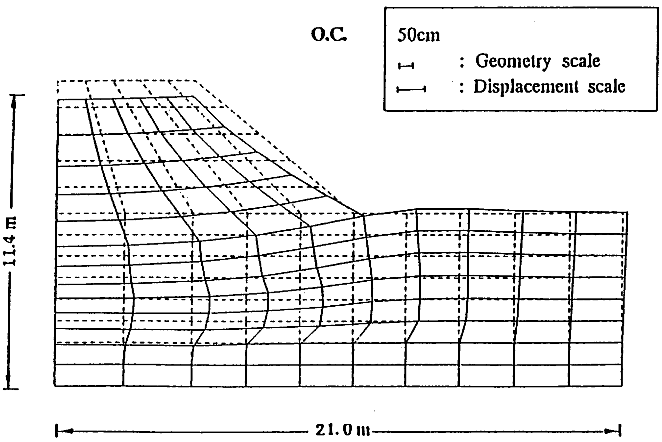
The coefficient of permeability as determined from laboratory tests was about three to six times smaller compared to the results in the field from analytical results. The lateral deformation results obtained from the finite element method were found to be larger than the corresponding values obtained from the field measurements. This paper gives a detailed evaluation of the predictive capabilities and limitations of the proposed simplified deformation analyses. Input soil parameters for the constitutive model are obtained from plasticity index.

As a result, reasonable agreements have been obtained between the measured and the computed values of settlement for multi soil layers. In addition, the ground surface movements beneath the center and beneath the shoulder of the embankments can be shown quantitatively. The predicted lateral deformation, however, overestimates the field measurements. Therefore, the present analytical method has still little discrepancy as far as the lateral deformation is concerned, so, to overcome this, careful attention is required considering model boundary condition and field measurements. It should be emphasized that even if the constitutive parameters are estimated only by plasticity index, the finite element analysis gives acceptable results when compared with the field performance. To this end, the analytical results obtained by the present method is found to be reasonable and encouraging for the use of the presented method in engineering practice. This analysis may be admittedly suitable for preliminary work, owing to idealizing assumptions and uncertainty in the data, however, it would seem to warrant additional study in geotechnical engineering.

The applicaiblity of these results and conclusions to other fields is not entirely clear. It can be stated with reasonable accuracy that other soils possessing similar qualities will likely behave in a similar manner. The authors believe the results and conclusions presented in the present paper regarding deformations for foundations and embankments are applicable in some degree to other foundations under embankments, although some minor modifications are needed to apply some of them. Further study regarding the present method is definitely needed, therefore, the research is currently being



(a) At the completion of embankment construction



(b) 160 days since embankment construction commenced

Fig. 13. Case (ii): The computed ground deformation
(a) At the completion of embankment construction
(b) 160 days since embankment construction commenced

conducted by the authors.

Conclusions

The following conclusions were developed:

1) The results of the numerical simulation are assessed qualitatively and quantitatively

by comparison with field-measured data. Therefore, the numerical illustration confirms the usefulness of Sekiguchi-Ohta model and the correlations between the soil parameters for the constitutive equations of soils and the palsticity index proposed by Kamei in engineering points of view.

2) The approach adopted is concluded to be capable of producing realistic predictions of deformation characteristics of embankment foundation. To this end, the analytical results obtained by the present method is found to be reasonable and encouraging for the use of the presented method in engineering practice.
3) This analysis may be admittedly suitable for preliminary work, owing to idealizing assumptions and uncertainty in the data at this stage. The applicability of the present method to other fields, however, currently being conducted by the authors, although some minor modifications are needed to apply some of them.

Appendix. I References

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Appendix. II Notations

The following symbols are used in this paper:

ϵ_V = volumetric strain;

$\dot{\epsilon}_V$ = volumetric strain rate;

D = coefficient of dilatancy;

$\dot{V}_0$ = initial volumetric strain rate;

η^* = shear stress ratio;

σ_{ij} = general stress;

ϵ_{ij} = general strain;

ϵ_V^{vp} = visco-plastic volumetric strain;

F = visco-plastic potential ($F \equiv \epsilon_V^{vp}$);

$\dot{\sigma}_{ij}$ = general stress rate;

$\dot{\epsilon}_{ij}$ = general strain rate;

D_{ijkl}^{evp} = elasto-viscoplatic stress strain matrix;

$\dot{\sigma}_{rij}$ = residual stress rate;

Δt = time increment;

$\Delta \hat{\sigma}$ = general stress increment matrix;

$\Delta \hat{\epsilon}$ = general strain increment matrix;

$\Delta \hat{\sigma}_r|_t$ = a stress relaxation matrix due to viscosity;

$\hat{D}^{evp}$ = stress-strain matrix;

L, G = Lamé's constants;

PI = platicity index;

λ = compression index;

κ = swelling index;

M_C = slope of Critical State Line in triaxial compression loading side;

M_E = slope of Critical State Line in triaxial extension loading side;

N = specific volume isotopically normally consolidated at $p' = 98$ kPa ($N = 1 + e_0$);

e_0 = initial void ratio;

e = void ratio;

K_0 = coefficient of earth pressure at rest;

K_{ONC} = coefficient of earth pressure at rest for normally consolidated soil;

K_{OOC} = coefficient of earth pressure at rest for overconsolidated soil;

C_α = coefficient of secondary compression ($C_\alpha = \Delta e / \Delta \log t$);

α = secondary comparession index ($\alpha = 0.434 \cdot C_\alpha / (1 + e_0)$);

t_c = completion time of the primary consolidation;

k_v = vertical coefficient of permeability;
k_{v0} = vertical coefficient of permeability at $p' = 98$ kPa;
λ_k = gradient of linear relationship between void ratio and logarithmic coefficient of permeability;
σ'_{vc} = vertical effective stress;
σ'_a = axial effective stress;
σ'_r = radial effective stress;
p' = mean effective stress ($p' = (\sigma'_a + 2\sigma'_r)/3$);
p'_0 = pre-consolidation mean effective stress and
q = principal stress difference ($q = \sigma'_a - \sigma'_r$).