Compression and consolidation characteristics of structured natural clay

J.-C. Chai, N. Miura, H.-H. Zhu, and Yudhbir

Abstract: The compression and consolidation behavior of some structured natural clays are discussed. It is shown that for some structured natural clays, the relation between void ratio ($e$) and mean effective stress ($p'$) is more linear in a $\ln(e + e_c) – \ln(p')$ plot (where $e_c$ is a soil parameter) than in an $e – \ln(p')$ plot. It is proposed that for structured natural clay with a sensitivity value greater than 4, a linear $\ln(e + e_c) – \ln(p')$ relation can be used in settlement and consolidation calculation. The effect of introducing a linear $\ln(e + e_c) – \ln(p')$ relation on the calculated load–settlement curve and consolidation behavior of structured clays is discussed. The linear $\ln(e + e_c) – \ln(p')$ relation was incorporated into the modified Cam–clay model by modifying the hardening law of the model. It is shown that using the linear $\ln(e + e_c) – \ln(p')$ relation simulated the consolidation behavior of the structured natural clays better than using the linear $e – \ln(p')$ relation.

Key words: structured natural clay, compression, consolidation, constitutive model, numerical analysis.

Résumé : On discute le comportement en compression et en consolidation de certaines argiles naturelles structurées. On montre que pour certaines argiles naturelles structurées, la relation entre l’indice des vides ($e$) et la contrainte moyenne effective ($p'$) est plus linéaire dans un graphique $\ln(e + e_c) – \ln(p')$, où $e_c$ est un paramètre du sol, que dans un graphique $e – \ln(p')$. On suggère que pour une argile naturelle structurée avec une valeur de sensibilité plus grande que 4, une relation linéaire $\ln(e + e_c) – \ln(p')$ peut être utilisée dans les calculs de tassement et de consolidation. On discute l’effet d’introduire une relation linéaire $\ln(e + e_c) – \ln(p')$ sur la courbe charge-tassement calculée et sur le comportement de la consolidation dans les argiles structurées. La relation linéaire $\ln(e + e_c) – \ln(p')$ a été incorporée dans le modèle Cam–clay modifié en modifiant la loi d’écrasissage du modèle. Il est montré que l’utilisation de la relation linéaire $\ln(e + e_c) – \ln(p')$ simulait mieux le comportement en consolidation des argiles structurées naturelles que l’utilisation de la relation linéaire $e – \ln(p')$.

Mots clés : argile naturelle saturée, compression, consolidation, modèle constitutif, analyse numérique.

Introduction

For reconstituted clay, the virgin compression curve in a plot of $e – \ln(p')$ (where $e$ is the void ratio, and $p'$ is the mean effective stress) is very close to a straight line. The linear $e – \ln(p')$ relation is used in conventional one-dimensional (1D) settlement calculation and incorporated in the hardening law of most elastoplastic models for clay. For structured (because of aging, cementation effect, etc.) natural clays, however, the compression curve is nonlinear in a plot of $e – \ln(p')$, and the slope of the curve, $\lambda_s$, is not a constant in the entire stress range. Several investigators (e.g., Burland 1990) reported this nonlinear phenomenon.

Haan (1992) proposed a general power formula for virgin compression curves of soil, but it has three parameters that need to be determined by regression analysis. It has been observed that for some natural clays, the $\ln(e) – \ln(p')$ relation is close to linear (e.g., Butterfield 1979). A linear $\ln(e + e_c) – \ln(p')$ relation only has two parameters (the soil parameter $e_c$ and the slope of the curve $\lambda_s$) and is analogous to a linear $e – \ln(p')$ relation (Chai 2001). This paper studied the compression and consolidation characteristics of some structured natural clays. The condition for using a linear $\ln(e + e_c) – \ln(p')$ relation, the corresponding equation for 1D settlement calculation, and the hardening law for an elastoplastic model were derived, and the variation of the calculated coefficient of consolidation with mean effective stress is investigated. The linear $\ln(e + e_c) – \ln(p')$ relation was introduced into the modified Cam–clay model (Roscoe and
Virgin compression curves of some structured natural clays

Figures 1 and 2 show the compression curves of five natural clays. The index properties of the clays are listed in Table 1. It can be seen that for all these clays the compression curves in the \( \ln(e + e_c) - \ln(p') \) plot have much better linearity than those in the \( e - \ln(p') \) plot. The value of \( e_c \) is varied between –1 and 1. Taking the logarithm of the void ratio \((e + e_c)\) means reducing the distance between the larger \( e \) values (in the lower compression stress range) and increasing the distance between the smaller \( e \) values (in the higher compression stress range), which makes a nonlinear curve in a plot of \( e - \ln(p') \) close to linear in a plot of \( \ln(e + e_c) - \ln(p') \). For example, the void ratio reduction \((\Delta e)\) for \( e \) from 3 to 2 and from 2 to 1 is the same \((\Delta e = 1)\), but \( \Delta \ln(e) \) is different, \( \ln(3) - \ln(2) \equiv 0.41 \) and \( \ln(2) - \ln(1) \equiv 0.69 \) (>0.41), with the difference being about 0.28. Further, a shift in \( e \) value by \( e_c \) will modify this difference. The difference increases when \( e_c \) is less than 0, and decreases when \( e_c \) is greater than 0. When \( e_c = -0.5, \ln(3 - 0.5) - \ln(2 - 0.5) \equiv 0.51 \) and \( \ln(2 - 0.5) - \ln(1 - 0.5) \equiv 1.1 \), with the difference being about 0.59 > 0.28 (in the case of \( e_c = 0)\). When \( e_c = 0.5, \ln(3 + 0.5) - \ln(2 + 0.5) \equiv 0.34 \) and \( \ln(2 + 0.5) - \ln(1 + 0.5) \equiv 0.51 \), with the difference being about 0.17 < 0.28.

Nonlinearity in an \( e - \ln(p') \) plot results from the rapid failure of soil structures (destructuring) when compression stress exceeds the yielding stress of the soil. Sensitivity \((S_t)\) is one of the indices to indicate the degree of structuring of clay. Table 1 shows that all the soils are highly sensitive clay. The linear \( \ln(e + e_c) - \ln(p') \) relation is applicable to high-sensitivity clays. Bangkok clay has a sensitivity value of 2–6 (Brand and Brenner 1981). The compression curve is less nonlinear in the \( e - \ln(p') \) plot but almost linear in the \( \ln(e) - \ln(p') \) plot (Fig. 3; data from Yokoyama et al. 1999).

Another factor is that for reconstituted Ariake clay, the laboratory vane shear test (a vane with a diameter of 20 mm and height of 40 mm) gave a sensitivity value of about 3 (due to the thixotropy effect, etc.). The compression curve of reconstituted Ariake clay is close to linear in an \( e - \ln(p') \) plot. According to the classification of Rosenqvist (1953), clays with an \( S_t \) value greater than 4 are classed as very sensitive. Based on the aforementioned limited information, it is proposed that for natural clay, when \( S_t \) is greater than 4, a linear \( \ln(e + e_c) - \ln(p') \) relation will better represent the compression and consolidation behaviors than a linear \( e - \ln(p') \) relation.

The values of \( e_c \) and \( \lambda_l \) can be obtained by the following method. Under the condition that the \( \ln(e + e_c) - \ln(p') \) plot is linear (a constant slope of \( \lambda_l \)) and the \( e - \ln(p') \) plot is nonlinear with a varied \( \lambda_l \) value, the following relationship between \( \lambda \) and \( \lambda_l \) exists:

\[
\lambda = (e + e_c)\lambda_l
\]

For a stress range \( p'_1 \) to \( p'_2 \), with corresponding void ratios \( e_1 \) to \( e_2 \), in the \( e - \ln(p') \) plot, if the tangent slopes at

Fig. 1. Consolidation curves for (a) Ariake (Kawasoe) clay and (b) Mexico City clay (data from Mesri et al. 1975).
Fig. 2. Consolidation curves for (a) Leda clay (data from Quigley and Thompson 1966), (b) Batiscan clay (data from Leroueil et al. 1985), and (c) Louiseville clay (data from Tanaka et al. 2001).

Table 1. Index properties of the clays considered.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Clay</th>
<th>Depth (m)</th>
<th>W_n (%)</th>
<th>W_L (%)</th>
<th>I_p</th>
<th>I_L</th>
<th>Clay fraction (%)&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Sensitivity, S&lt;sub&gt;i&lt;/sub&gt;</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ariake clay (Kawasoe)</td>
<td>7.5</td>
<td>130</td>
<td>86</td>
<td>41</td>
<td>2.07</td>
<td>46.5</td>
<td>20–26</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Ariake clay (Fukutomi)</td>
<td>2.0–2.5</td>
<td>136</td>
<td>112</td>
<td>63</td>
<td>1.38</td>
<td>59.1</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Mexico City clay</td>
<td>−15.0</td>
<td>559</td>
<td>500</td>
<td>350</td>
<td>1.17</td>
<td>Mesri et al. 1975</td>
<td>20–30</td>
<td>8–16&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>4</td>
<td>Leda clay</td>
<td>—</td>
<td>78</td>
<td>56</td>
<td>27</td>
<td>1.80</td>
<td>86</td>
<td>10–12</td>
<td>Quigley and Thompson 1966</td>
</tr>
<tr>
<td>5</td>
<td>Batiscan clay</td>
<td>7.3</td>
<td>80</td>
<td>43</td>
<td>21</td>
<td>2.70</td>
<td>81</td>
<td>125</td>
<td>Leroueil et al. 1985</td>
</tr>
<tr>
<td>6</td>
<td>Louiseville clay</td>
<td>7.3</td>
<td>65–75</td>
<td>70–80</td>
<td>45–60</td>
<td>0.80–0.95</td>
<td>50–60</td>
<td>28</td>
<td>Tanaka et al. 2001</td>
</tr>
</tbody>
</table>

Note: I<sub>L</sub>, liquidity index; I<sub>p</sub>, plasticity index.

<sup>a</sup>For the Ariake clays, the data for the clay fraction are for soil particles <5 μm, and for all other clays the data are for soil particles <2 μm.

<sup>b</sup>From Terzaghi and Peck (1967).
...and (p'_2, e'_2) are λ_1 and λ_2, respectively, e_c can be determined as

\[ e_c = (e_1λ_2 - e_2λ_1)/(λ_1 - λ_2) \]

Generally, e_c is within the range -1 to 1. Another restriction on e_c is that (e + e_c) > 0. The value of λ_i can then be obtained by substituting (e_c and (e_1, λ_1), or (e_2, λ_2)) into eq. [1]. For selecting two points in an ln(p) – ln(p') plot, it is suggested that the point (p'_1, e_1) be selected just after yielding, and point (p'_2, e_2) at the maximum stress considered. Since the natural clay may not strictly follow a linear ln(e + e_c) – ln(p') relation, using different points may yield slightly different e_c and λ_1 values.

Generally, when compressing the structured clay to its plastic limit, the structure will be destroyed or will not have a significant effect on its compression and consolidation behavior. Therefore, the difference between natural water content and plastic limit (W_n - W_p) provides a measure of volume change before the structure is destroyed. Figure 4 shows plots of e_c versus [1/(W_n - W_p)]100 for the clays considered here (units for W_n and W_p are in %). The e_c values increase almost linearly with a decrease in the value of [1/(W_n - W_p)]100. As mentioned previously, the e_c and λ_i values will change slightly with the change of stress range considered, and thus the e_c versus [1/(W_n - W_p)]100 relationship may not be unique. We consider, however, that it may provide a reference for determining the value of e_c.

Using a linear ln(e + e_c) – ln(p') relation, the virgin compression curve can be represented as

\[ \ln(e + e_c) = \ln(e_0 + e_c) - λ_1 \ln(p'/p'_0) \]

where e_0 is the initial void ratio, and p'_0 is the initial mean effective stress in the virgin compression range.

The effect of soil structure on the unloading–reloading curve should be less than that on the virgin compression curve, and for structured natural clay the unloading–reloading curves should be close to that of the reconstituted case. For mathematical simplicity, however, a linear ln(e + e_c) – ln(p') relation is adopted for the unloading–reloading range and the slope is defined as κ_i. Assuming κ_i/λ_i = κ/λ, the assumption of a linear ln(e + e_c) – ln(p') relation will predict more elastic deformation in the lower stress range and less in the higher stress range compared with those from a linear e – ln(p') relation. To avoid larger elastic deformation in the lower stress range, the use of κ_i/λ_i < κ/λ is recommended.

**Coefficient of consolidation**

**Expression for the coefficient of consolidation (C_v)**

Introducing a linear ln(e + e_c) – ln(p') relation for virgin compression influences not only the calculated load–settlement curve, but also the consolidation process. The coefficient of consolidation, C_v, is a function of hydraulic conductivity (k) and the coefficient of volumetric compression (m_v) of soil:

\[ C_v = k/m_vγ_w \]

where γ_w is the unit weight of water.

It is generally agreed that k varies with the void ratio (e). Taylor’s equation (Taylor 1948) can be used to express this variation:

\[ k = k_010^{(e-e_0)/C_k} \]

where k_0 and k are hydraulic conductivities corresponding to e_0 and e, respectively; and C_k is a constant, which can be estimated as 0.5e_0 (Tavenas et al. 1983). The value of m_v varies with stress, p', and void ratio, e. In elastoplastic analysis, for a linear e – ln(p') relation, the expression of m_v for virgin compression is

\[ m_v = λ/[1 + (1 + e)p'] \]

and for a linear ln(e + e_c) – ln(p') relation...
For both relations, \(m_c\) will decrease with an increase in \(p'\). For a given value of \(p'\), however, with a linear \(e - \ln(p')\) relation, \(m_c\) will increase with a decrease in \(e\), whereas with a linear \(\ln(e + e_c) - \ln(p')\) relation \((e_c < 1.0)\), \(m_c\) will decrease with a decrease in \(e\). A decrease in the value of \(m_c\) will contribute to the increase in the coefficient of consolidation.

### Comparing the calculated and measured values

For the clays discussed in this paper, the measured coefficients of consolidation are only available for Ariake clay (Kawasoe, Saga, Japan) and Louiseville clay. The conventional odometer was used to test Ariake clay (Kawasoe), and the constant rate of strain (CRS) odometer to test Louiseville clay with a strain rate of 3.3 \(\times 10^{-6}\)/s (Tanaka et al. 2001).

The calculated and measured \(C_v\) values for Ariake clay (Kawasoe) are compared in Fig. 5. In calculation, a hydraulic conductivity of 7.5 \(\times 10^{-9}\) m/s corresponding to a yielding stress condition and \(k_1 = 0.02\) were assumed. Other parameters are given in Fig. 1a. The following can be observed from Fig. 5:

1. Before yielding, calculation resulted in a lower \(C_v\) value, which may indicate that the calculation underestimated the stiffness of the soil in the overconsolidated state.
2. Both the test data and the calculated results showed a rapid decrease in \(C_v\) after yielding. The linear \(\ln(e + e_c) - \ln(p')\) relation \((e_c = 0)\) gave a greater decrease in \(C_v\) than the linear \(e - \ln(p')\) relation just after yielding \((\lambda_1(e + e_c) > \lambda_2)\).
3. Although at a higher stress level, using the linear \(\ln(e + e_c) - \ln(p')\) relation, the calculation yielded a faster increase in \(C_v\) with consolidation stress than that measured in the test, the linear \(\ln(e + e_c) - \ln(p')\) relation generally yielded a better simulation of the test results.

It is demonstrated in a later section that a faster increase in the value of \(C_v\) in a higher stress range is desirable for fitting the measured excess pore pressure variation during consolidation tests. Figure 5 also indicates that using a constant value of hydraulic conductivity in elastoplastic analysis may overpredict the value of \(C_v\) and result in a faster rate of consolidation.

For Louiseville clay, the comparison is given in Fig. 6. During calculation, a hydraulic conductivity of \(6.0 \times 10^{-9}\) m/s corresponding to yield state and \(k_1 = 0.04\) were assumed (to fit the measured value at that point). The values of \(e_o\), \(\lambda\), and \(\lambda_2\) are indicated in Fig. 2c. It can be seen that the linear \(\ln(e + e_c) - \ln(p')\) relation yielded a much better simulation of the test results. In the normal consolidation range, when considering the hydraulic conductivity variation with void ratio, the linear \(e - \ln(p')\) relation resulted in a continuous decrease in the value of \(C_v\). The initial void ratio \((e_o)\) of Louiseville clay is smaller than that of Ariake clay (Kawasoe), and thus the value of \(C_v\) \((0.5 e_o)\) is also smaller. Therefore, with the same amount of void ratio reduction \((\Delta e)\), there will be a larger percent reduction in hydraulic conductivity for Louiseville clay than for Ariake clay. In other words, with the parameters used, for Louiseville clay, in the normal consolidation range the contribution of the reduction in hydraulic conductivity to the value of \(C_v\) is greater than that of the increase in stiffness. Another point is that all calculations overestimated the measured lowest value of \(C_v\) (just after yielding). As seen in the \(\ln(e - 0.8) - \ln(p')\) plot in Fig. 2c, the curve is not straight just after yielding. This indicates that the linear \(\ln(e - 0.8) - \ln(p')\) assumption failed to express the phenomenon of a sharp reduction in void ratio with little or almost no increase in mean effective stress (reduction in stiffness).

### Application of the linear \(\ln(e + e_c) - \ln(p')\) relation

#### One-dimensional settlement calculation

In the virgin compression range, using eq. [3], the total volumetric strain, or vertical total strain for the 1D case, \(\varepsilon_v\), is as follows:

\[
\varepsilon_v = \varepsilon_0 + \frac{e_c}{1 + e_0} \left[1 - \left(\frac{p'}{p_0}\right)^{-\lambda_1}\right]
\]

Equation [8] can be used for 1D settlement calculation. To illustrate the difference in using \(\lambda_1\) versus \(\lambda\) in settlement calculation, the settlement–load curves for Ariake clay.
Incorporating the linear $\ln(e + e_c) - \ln(p')$ relation in the modified Cam–clay model

Generally, for an elastoplastic model, three functions need to be defined: (i) yielding function ($F$), (ii) plastic potential function ($G$), and (iii) hardening (or softening) law (or function). To incorporate the linear $\ln(e + e_c) - \ln(p')$ relation for a soil model with volumetric strain hardening (or softening), a modification of the hardening (or softening) law is needed. Mathematically, replacing the linear $e - \ln(p')$ relation with the linear $\ln(e + e_c) - \ln(p')$ relation means replacing $\kappa$ with $(e + e_c)\lambda_1$ and $\lambda$ with $(e + e_c)\lambda_2$. This indicates that introducing the linear $\ln(e + e_c) - \ln(p')$ relation is equivalent to using a varied slope, $\lambda = (e + e_c)\lambda_1$, for virgin compression and $\kappa = (e + e_c)\lambda_2$ for unloading–reloading in the $e - \ln(p')$ plot. The linear $\ln(e + e_c) - \ln(p')$ relation is introduced into the modified Cam–clay (MCC) model (Roscoe and Burland 1968).

In the MCC model, the isotropic compression line (ICL), normal compression line (NCL), and critical state line (CSL) in the $e - \ln(p')$ plot are parallel. Keeping the structure of the MCC model and introducing the linear $\ln(e + e_c) - \ln(p')$ relation imply that the CSL is also linear in the $\ln(e + e_c) - \ln(p')$ plot (Fig. 8). Butterfield (1979) showed that the CSL is close to linear in the $\ln(e) - \ln(p')$ plot. From the definition, “critical state” is an ultimate state at which there is no change in deviator stress and soil volume with an increase in shear strain. For a structured natural clay, there is significant strain softening before the critical state is reached, and the MCC model has limitations in simulating this phenomenon. After introducing a linear $\ln(e + e_c) - \ln(p')$ relation, the model still inherits this shortcoming from the MCC. Therefore, it is considered that critical state here corresponds to the peak strength state of a soil in the virgin compression range. The steady state boundary surface (SSBS) is then as follows:

\[
e + e_c = e_{\text{ICL}} + e_c = e_{\text{CSL}} + e_c = e_{\text{NCL}} + e_c = e_{\text{iso}} + e_c = e_c + e_c = e_c + e_c
\]

where $e_{\text{CSL}}$ is the void ratio at the critical state line at $p' = 1$ unit, $\eta = q/p'$ (where $q$ is the deviator stress), and $M$ is the slope of the critical state line in the $q-p'$ plot. The undrained shear stress path is the same as that of the MCC if $\kappa_1/\lambda_1 = \kappa/\lambda$, which indicates that introducing a linear $\ln(e + e_c) - \ln(p')$ relation into the MCC does not change the predicted undrained shear strength if $\kappa_1/\lambda_1 = \kappa/\lambda$:

\[
p'_{\text{iso}} = \left(1 + \frac{\eta^2}{M^2}\right)^{-1 - \frac{\kappa}{\lambda}}
\]

where $p'_{\text{iso}}$ is the stress on the current yield surface and on the isotropic compression line. As discussed previously, if adopting $\kappa_1/\lambda_1 = \kappa/\lambda$, the linear $\ln(e + e_c) - \ln(p')$ relation will predict more elastic deformation at a lower stress range and less elastic deformation at a higher stress range than the linear $e - \ln(p')$ relation. To avoid a larger elastic deformation in the lower stress range, $\kappa_1/\lambda_1 < \kappa/\lambda$ is recommended. In this case, the linear $\ln(e + e_c) - \ln(p')$ relation will predict a lower undrained shear strength than the MCC model.

For structured natural clay, the undrained shear strength depends on the soil structure. It is considered that a different soil structure will have a different yielding stress (or size of yielding locus). With a different yielding stress, the proposed model can predict different undrained shear stress.

The linear $\ln(e + e_c) - \ln(p')$ relation and the variation of hydraulic conductivity with the void ratio relation (eq. [5]) were incorporated into the CRISP program (Britto and Gunn 1987), and the program was used to conduct finite element simulation.

Consolidation simulation

The laboratory consolidation tests of stepwise loading for Ariake clay (Fukutomi, Saga, Japan) and constant rate of strain (CRS) for Batiscan clay (Leroueil et al. 1985) were simulated and the results of the simulation were compared with the test data. Both the linear $e - \ln(p')$ and the linear

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Fig. 7. Comparison of the load–settlement curves for Ariake clay at Kawasoe. Sample thickness 20 mm.

Fig. 8. Linear $\ln(e + e_c) - \ln(p')$ relationship.
\( \ln(e + e_c) - \ln(p') \) relations were used in the simulation, and their results are also compared and discussed. There is no measured excess pore pressure variation reported for Louisville clay (Tanaka et al. 2001), and therefore it is not used for the consolidation simulation. Eight-node quadrilateral elements with nine integration points were adopted in the finite element analysis.

**Consolidation test under stepwise loading**

The test was conducted using a Maruto multiple odometer apparatus, which has a pore-water pressure measurement system. This kind of test simulates the stage construction of an embankment on soft ground. The size of the sample was the same as that for the conventional consolidation test (i.e., 60 mm in diameter and 20 mm in height). A one-way drainage condition was adopted and the pore-water pressure was measured at the bottom of the sample. The equal load increment was used with an increment of 10 kPa and consolidation period of 15 min for each step. For the final loading increment, the load was maintained for 24 h and the excess pore pressure was completely dissipated.

For each load increment, before applying the next load increment, the excess pore pressure was not completely dissipated (except for the final increment). The corresponding average consolidation pressure \( p_{av}' \) within the sample was calculated by the following expression:

\[ p_{av}' = p - 2u_b/3 \]

where \( p \) is the applied total pressure, and \( u_b \) is the excess pore-water pressure at the bottom of the sample.

The Ariake clay sample was obtained from Fukutomi, Saga, Japan, at a depth below the ground surface of 2.0–2.5 m. The results in Fig. 1a are for the sample of Ariake clay from Kawasoe, Saga, Japan, and the soil parameters are not the same (Table 1). The parameters used for the simulation are as follows: \( \lambda = 0.85, k = 0.10, \lambda_1 = 0.308, k_1 = 0.025, e_0 = 3.73, \) and \( M = 1.2, \) vertical yielding stress of \( \sigma_{v0}' = 50 \) kPa, and an initial hydraulic conductivity of \( k_0 = 2.5 \times 10^{-9} \) m/s.
For simulating the odometer tests, the finite element mesh used is shown in Fig. 9. The thickness of the element varied from 1 to 4 mm.

Figure 10 compares the measured and simulated vertical effective stress versus settlement curves. Although within the stress range tested the difference is small, the linear $\ln(e) - \ln(p')$ relation yielded a better simulation. The measured and simulated excess pore-water pressures at the bottom of the sample are depicted in Fig. 11. Using the linear $\ln(e) - \ln(p')$ relation, the predicted values are higher than the measured values, but the simulation yielded a very good prediction of the pattern of excess pore-water pressure variation. Using the linear $e - \ln(p')$ relation, the calculation failed to simulate the excess pore pressure decrease tendency when the vertical effective stress was greater than about 100 kPa. As discussed previously, when considering the variation of hydraulic conductivity with void ratio, the linear $e - \ln(p')$ relation results in a low increment rate or even a decrease in the coefficient of consolidation with the increase in effective stress in the virgin consolidation range (Figs. 5, 6). Generally, the linear $\ln(e) - \ln(p')$ relation gives a better prediction of the stepwise loading consolidation test of Ariake clay (Fukutomi).

**Consolidation test under constant rate of strain (CRS)**

The test results reported by Leroueil et al. (1985) for Batiscan clay were considered. The case simulated had a strain rate of $1.43 \times 10^{-7}$/s, and there was excess pore pressure generated at the bottom of the sample, which can be used to compare with the simulated results. In the analysis, a yielding vertical effective stress of about 110 kPa obtained from the test result was adopted. The parameters adopted were as follows: (i) for the linear $e - \ln(p')$ relation case, $\lambda = 1.03$ (Fig. 2b), $\kappa = 0.03$ (Poisson’s ratio), $\nu = 0.3$, $k_0 = 1.8 \times 10^{-10}$ m/s, $M = 1.2$, and $e_0 = 2.12$ (estimated from water content); and (ii) for the linear $\ln(e - 1) - \ln(p')$ case, $\lambda_1 = 1.56$, $\kappa_1 = 0.03$, $e_1 = -1.0$ (Fig. 2b), and the other parameters were the same as those for the linear $e - \ln(p')$ case. The simulation was started with an initial effective vertical stress of 50 kPa, and the results are compared in Fig. 12. Using the linear $\ln(e - 1) - \ln(p')$ relation resulted in a much better simulation than that using the linear $e - \ln(p')$ relation. With the linear $\ln(e - 1) - \ln(p')$ relation, the simulated load–settlement curve is very good, and the load – excess pore pressure curve is fair. There are two points that need to be explained. First, there was a reduction in simulated excess pore-water pressure just before yielding of the whole sample. This is because the yielding started from the top of the sample, and before the whole sample yielded the strain increment concentrated at the top, and the bottom was less loaded and part of the previously generated excess pore-water pressure ($\Delta u_b$) dissipated (the same tendency can also
be observed in Fig. 11). Second, for the linear $e - \ln(p')$ case, the simulated $\Delta u_0$ showed an increased increment rate during the later part of the loading process. Under a constant strain rate condition, the stress rate exponentially increases, and it tends to increase the excess pore-water pressure generation rate. For the linear $e - \ln(p')$ case, at a later stage the effect of stress rate increase was more than that of the increase in the $C_v$ value and resulted in an increased rate of excess pore pressure generation. For the linear $\ln(e - 1) - \ln(p')$ case, during the consolidation process, the increase in the $C_v$ value is faster than that of the linear $e - \ln(p')$ case (Figs. 5, 6), and the excess pore-water pressure showed a reduced increment rate and is closer to the measurements.

The simulated strain and strain-increment distributions within the sample are shown in Fig. 13. At an early stage (about 5% strain), the strain at the top of the sample was two to three times that at the bottom of the sample, and the strain increment was also larger at the upper part of the sample. This tendency was stronger for the linear $\ln(e - 1) - \ln(p')$ case. At a later stage (about 20% strain), the strain distribution became less non-uniform. For the linear $e - \ln(p')$ case, the strain increment in the upper part of the sample was still larger than that in the lower part, but for the linear $\ln(e - 1) - \ln(p')$ case, the strain increment in the lower part of the sample became larger. This indicates that during the test, for the linear $e - \ln(p')$ case, the strain rate within the sample did not vary much, whereas for the linear $\ln(e - 1) - \ln(p')$ case, the higher strain rate zone was located first in the upper part of the sample and then moved to the lower part of the sample.

**Conclusions**

The compression and consolidation behavior of some structured natural clays is studied. It is proposed that for structured natural clay with a sensitivity value larger than 4, a linear $\ln(e + e_c) - \ln(p')$ ($e_c$ is a soil parameter) relation can be used instead of a linear $e - \ln(p')$ relation. The effects of introducing a linear $\ln(e + e_c) - \ln(p')$ relation on the calculated load–settlement curve and the coefficient of consolidation are discussed. It is shown that for structured natural clays, the linear $\ln(e + e_c) - \ln(p')$ relation can simulate the load–settlement curve much better than the linear $e - \ln(p')$ relation.

The linear $\ln(e + e_c) - \ln(p')$ relation is incorporated in the modified Cam–clay model by modifying the hardening law and the critical state line in the $e-p'$ plot. The stepwise loading and constant rate of strain (CRS) consolidation tests were simulated and the results were compared with the test data. It is demonstrated that the linear $\ln(e + e_c) - \ln(p')$ relation simulated the consolidation behavior (load–settlement and (or) load – excess pore pressure curves) of Ariake clay (Fukutomi, Saga, Japan) and Batiscan clay much better than the linear $e - \ln(p')$ relation.

**References**


