Coefficient of earth pressure at rest for normally and overconsolidated peat ground in Hokkaido area

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Abstract

Peat, which is widely distributed in Hokkaido, is a very soft and problematic soil. To perform an elasto-plastic Finite Element (FE) analysis, it is important to accurately determine the initial stress conditions, and among them, the value of the coefficient of earth pressure at rest ($K_0$ value) is particularly important. A $K_0$-consolidation test using triaxial testing apparatus and a flat dilatometer were performed to investigate the $K_0$ value for peat ground in Hokkaido, Japan. It was found that the $K_0$ value for normally consolidated peat and organic clay ($K_0$NC) decreases with an increase in the ignition loss. The $K_0$ value for overconsolidated peat and organic clay ($K_0$OC) is more strongly dependent on the over consolidation ratio (OCR) than that of usual inorganic clay. That is, it is known that $K_0$OC is empirically related to $K_0$NC, as expressed by $K_0$OC = $K_0$NC $OCR^m$, and in peat and organic clay the power of $m$ increases with their ignition loss. An experimental equation to estimate $K_0$ using a flat dilatometer for peat ground is also proposed.

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Keywords: $K_0$ value; Flat dilatometer; $K_0$-consolidation test; Normal consolidation; OCR; Organic clay; Over consolidation; Peat (IGC: D5/D6)

1. Introduction

Peat ground is widely distributed throughout the Hokkaido region of Japan. Peat contains a large amount of organic matter and has a very high natural water content. Its unsuitability as a foundation material has brought about serious problems for construction works. In Hokkaido, the peat layer often overlies thick layers of organic clay and Holocene inorganic clay. Embankment constructions tend to deform the peat ground, not only due to consolidation but also shear. Hayashi et al. (1994) have demonstrated that a Finite Element (FE) analysis based on Cam-clay model (Schofield and Wroth, 1968) is quite effective in predicting the deformation of peat ground, since this model is capable of taking deformation caused by consolidation as well as shear into account. In order to perform an elasto-plastic FE analysis including the Cam-clay model, accurate initial stress conditions need to be determined. The value of the coefficient of earth pressure at rest ($K_0$ value) is particularly important. In this paper, we propose a method for determining the $K_0$ value for peat ground.

Remarkably little research has been done on the $K_0$ value for peat and organic clay compared to usual inorganic clay, so very little data is available in the literature. Miyakawa et al.
(1974), Edil and Dhowian (1981), Fujikawa et al. (1983), Kawano et al. (1986), and Mesri and Ajlouni (2007) reported the \( K_0 \) values for peat ground measured with one-dimensional oedometer test, with stress conditions restricted to the normally consolidated state. As will be explained later, peat ground is usually overconsolidated to some degree due to the in situ stresses which vary with seasonal changes in the groundwater table. This factor is significant since, combined with the extremely low density of peat, it results in high OCR values. As such, it is very important for determining the \( K_0 \) value of overconsolidated peat ground for numerical solutions.

It should be kept in mind that the engineering properties of peat ground are extremely heterogeneous. Therefore, the soil parameters for practical designs should be obtained from a large number of simple and robust tests, rather than from a few complicated and sophisticated tests. It is also useful to determine the soil parameters using simple in-situ tests that can measure soil parameters continuously to the required depth. In this study, methods for determining the \( K_0 \) of peat ground are established by performing laboratory \( K_0 \)-consolidation tests using triaxial testing apparatus on undisturbed peat and organic clay, as well as the use of in-situ tests with a flat dilatometer (DMT).

2. Importance of \( K_0 \) value at various OCR

2.1. OCR for peat ground

Peat is usually found directly from the ground surface. Since the wet density of peat is very low and the groundwater level in the peat ground is very close to the ground surface, the effective overburden pressure is very low unless artificial loading is applied. Furthermore, the groundwater level in the peat ground changes seasonally, so peat ground tends to be overconsolidated, even with no increase in an artificial stress. Futatsukawa and Kikuchi (1988) performed a detailed survey of changes in the groundwater levels of peat ground from 1985 to 1987 in the suburbs of Sapporo, Hokkaido. They found that the groundwater level was the highest from March to April and at its lowest during summer. The change in the groundwater level between these two seasons varies from 50 cm to 1 m. Let us consider the OCR for an imaginary ground consisting of a top usual soil layer with 0.5 m thickness and a peat layer with 4.5 m thickness, whose \( \gamma_1 \) are 15.0 kN/m\(^3\) and 10.3 kN/m\(^3\), respectively (Fig. 1). For this ground condition, the change in the OCR will be calculated when the groundwater level drops by 1 m. These adopted numbers are typical values for the peat ground in Hokkaido (CERI, 2002). Two cases of the initial groundwater level are examined: the ground surface and the boundary between the top soil and the peat layer.

In addition to these seasonal fluctuation, the groundwater levels in the peat ground are also widely influenced by drainage, ranging in an order estimated between several dozen centimeters to about 1 m (Hokkaido branch of JGS, 2002). As will be shown later in Figs. 7−10, which describe the soil profiles of the four investigation sites, peat soils are slightly overconsolidated. Therefore, it is necessary to carefully examine the relationship between the OCR and \( K_0 \) under in-situ stress to determine the \( K_0 \) value for peat ground.

2.2. Numerical evaluation for effects of \( K_0 \) value on lateral deformation

To evaluate the influence of the \( K_0 \) value on calculated deformation, an FE analysis was conducted on a test embankment constructed in Toubetsu, Hokkaido. Fig. 2 shows a cross-section of the test embankment and soil layers. The permeability of the sand layer located under the peat layer. The permeability of the sand layer located under the
Aco layer is high enough to provide drainage. The height of the embankment was 5 m, and its construction process is illustrated in Fig. 4.

A soil/water-coupled elasto-plastic FE analysis was conducted based on the finite deformation theory, which is considered a suitable method for peat ground with extremely large deformation (Yamazoe and Mitachi, 2007). In this analysis, the modified Cam-clay model was used to describe the dilatancy characteristics of peat, organic clay and clay layers. The relationship between the specific volume ($v$) and effective mean stress ($p'$) was assumed to be linear in ln $v$–ln $p'$ plot. The purpose of the FE analysis in this study is to clarify the importance of $K_0$ value for peat by demonstrating the influence of differences in the $K_0$ value on analytical results.


As shown in Table 1, the $K_{OOG}$ value of the peat layer was changed from 0.2 to 1.0 to examine its influence. Table 2 lists other soil parameters used for the analysis, based on the results from oedometer, triaxial compression ($CU$) and physical index tests. However, the effective angle of shear resistance ($\phi'$) of the peat (Ap) layer was found using the estimation equation (Eq. (1)) presented by Hayashi et al. (2007); where $L_i$ is ignition loss (%):

$$\phi' = 0.19L_i + 32 \quad (1)$$

The soil parameters were determined in line with the procedure proposed by Hayashi et al. (2007) as follows. The compression index $C_c$ in relationships between ln $e$ and ln $p'$ was determined by the oedometer test. The swell index $k_s$ was assumed to be 0.1 times of the $C_c$. The initial volume ratio $v_0$ was calculated using the natural water content and the soil particle density. The critical state parameter $M$ was determined using the $\phi'$ given above. The OCR was the ratio of the consolidation yield stress obtained from the oedometer test to the effective overburden stress calculated by the wet density of each soil layer.

Hayashi et al. (2008) reported that the coefficient of permeability ($k_0$) in peat layers measured by the field test is greater than that measured by the oedometer test ($k_1$), and $k_1$ is 10–30 times as high as $k_0$. Based on these results, $k$ for the FE analysis was assumed to be 10 times larger than that measured by the oedometer test. The change in the coefficient of permeability $\lambda_k$ was determined from the relationship between the void ratio and the coefficient in a logarithmic expression (the $e$-$\log k$ relationship) under
normal consolidation conditions obtained from the oedometer test.

Fig. 4 shows the process of the embankment construction and the observed settlement on the ground surface at the center of the embankment. The embankment construction commenced 86 day after sand mat construction. Large settlement (more than 3 m) due to the embankment load was observed. The analysis slightly overestimated the settlement due to the sand mat construction (elapsed time between 0 and 86 days), but the settlement caused by the embankment construction (86 days later) can be well predicted. Although calculated settlements decreased slightly with increase in the $K_{0OC}$ value, no significant difference was observed, considering that practical estimates in peat ground tend to be out by 10 cm or more.

Fig. 5 shows the observed lateral displacement of the ground under the toe of the embankment at the completion of the embankment (236 days later). It was as large as 60 cm at a depth of 4 m, which is about 25% of the settlement measured at the center of the embankment. As shown in the figure, calculated lateral displacements are very sensitive to the order of the $K_{0OC}$ value, and decrease with increase in the $K_{0OC}$ value. The difference in the maximum lateral displacements with $K_{0OC}$ values from 0.2 to 1.0 is 22 cm. It should be noted that, in practice, the required accuracy is sometimes as small as 1 cm for existing facilities near the embankment, so that the importance of the $K_{0OC}$ value in the analysis needs to be realized.

### 3. Investigation sites

Investigations of the $K_0$ values of peat ground were performed at four sites: Mihara and Shinotsu in Ebetsu City; Tsuruno in Kushiro City; and Riyamunai in Kyouwa town (Fig. 6). In the laboratory, the following tests were conducted: a $K_0$-consolidation test using a triaxial testing apparatus, a consolidated undrained triaxial compression test, an oedometer test, and several other tests for physical properties. The in-situ tests performed were a dilatometer (DMT) and an electric cone penetration test (CPT).

Figs. 7–10 show the geotechnical properties of the investigated site in this study. The yield stress $p_c$ in the figures was determined using the $e$-$\log p$ relationship obtained from the oedometer test. As the $p_c$ value for peat was extremely small, the consolidation pressure for the first step in the oedometer test was set at 5 kN/m$^2$.

The top layer at the Mihara site consists of peat with 400–500% natural water content. The underlying soils are organic clay, sandy silt and clay, which is a typical sequence of the ground covered by peat at the ground surface found in Hokkaido. The $OCR$ of the peat is in the range of 1–2, indicating it is slightly overconsolidated.

The Shinotsu site is located near the Mihara site. Therefore, the ground composition at the Shinotsu is similar to that at Mihara, except for the relatively large $OCR$ for the peat ground.

The Tsuruno site is the thinnest at the Tsuruno site. The soft deposits consisting of alternating layers of peat, organic clay and clay layers at the Tsuruno site are thinner than that at the other two sites. The $OCR$ of the peat layer is less than 2, which is lower than the $OCR$s of the organic clay and clay layers.

The natural water content of the peat at the Riyamunai site is 500–850% and its $OCR$ ranges between 2 and 3.

<table>
<thead>
<tr>
<th>Case</th>
<th>$K_{0OC}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2</td>
</tr>
<tr>
<td>2</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
</tr>
<tr>
<td>4</td>
<td>0.8</td>
</tr>
<tr>
<td>5</td>
<td>1.0</td>
</tr>
</tbody>
</table>
4. Testing procedures

4.1. $K_0$-consolidation test

A $K_0$-consolidation test using triaxial testing apparatus (Fig. 11) was conducted in accordance with Japanese Geotechnical Society standards (JGS 0525-2000) on undisturbed samples that were collected using a thin-walled tube sampler with a fixed piston and an inside diameter of 75 mm. To avoid unnecessary disturbance, the diameter of the peat specimen was the same as the sample retrieved from the sampling tube, i.e., 75 mm, and the height of the specimen was 75 mm. Otherwise, specimens of the other soil types were trimmed to 50 mm in diameter and 100 mm in height. To accelerate the dissipation of the excess pore water pressure, four pieces of filter paper were placed on the circumference of the test specimens. Furthermore, a nylon mesh was placed on both ends of the test specimens.

At the initial state of the consolidation, isotropic consolidation pressure was applied. The intensity was low enough not to cause an excess lateral displacement, i.e., 2 kN/m$^2$ for peat and 5 kN/m$^2$ for the organic clay test specimens. Then, the lateral pressure in the test was automatically controlled so that the lateral strain of the specimen should be zero, calculated from the axial displacement and the volume change.

To avoid any decrease in effective load by the generation of excess pore water pressure, axial loading was applied at the slow rate of 0.2 kN/m$^2$/min. In this study, the pore water pressure was not measured because the top and bottom ends of the test specimens were kept drained. However, the generated excess pore water pressure in the specimen was considered small enough to accurately measure the $K_0$ value during consolidation for the following reason. Oda and Mitachi (1992) performed a $K_0$-consolidation test in a triaxial cell on clay samples with a coefficient of consolidation ($c_v$) of $10^{-13}$ cm$^2$/day. They reported that the measured excess pore water pressure at the bottom of the specimen was less than 10% of the vertical stress under an axial loading rate of 0.9 kN/m$^2$/min or lower. The axial loading rate adopted in this study is slower than this rate and the $c_v$ of peat and organic clay is similar to or approximately 10 times the values of the clay they used. Therefore, it would follow that almost no excess pore water pressure was generated under the axial loading rate of 0.2 kN/m$^2$/min in this experiment.

It was observed that the lateral strain of the test specimens during the $K_0$-consolidation was about 0.01–0.03%, which is sufficiently below the value of 0.05% specified by the Japanese Geotechnical Society (JGS 0525-2000). A comparison of the specimen before and after the

Table 2

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Model</th>
<th>$\rho_i$ (g/cm$^3$)</th>
<th>$E$ (kN/m$^2$)</th>
<th>$\nu$</th>
<th>$\lambda$</th>
<th>$\kappa$</th>
<th>$v_0$</th>
<th>$M$</th>
<th>OCR</th>
<th>$K_{0NC}$</th>
<th>$K_{0OC}$</th>
<th>$k_0$ (m/d)</th>
<th>$\lambda'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bk 5.00–0.75</td>
<td>B</td>
<td>1.70</td>
<td>5600</td>
<td>0.33</td>
<td>0.305</td>
<td>0.031</td>
<td>8.8</td>
<td>1.90</td>
<td>3.20</td>
<td>0.28</td>
<td>0.73</td>
<td>2.0E–02</td>
<td>0.201</td>
</tr>
<tr>
<td>Sm 0.75–0.00</td>
<td>B</td>
<td>1.70</td>
<td>5600</td>
<td>0.33</td>
<td>0.305</td>
<td>0.031</td>
<td>8.8</td>
<td>1.90</td>
<td>3.20</td>
<td>0.28</td>
<td>0.73</td>
<td>2.0E–02</td>
<td>0.201</td>
</tr>
<tr>
<td>Ap-u 0.00 to −2.40</td>
<td>A</td>
<td>1.09</td>
<td>–</td>
<td>0.22</td>
<td>0.356</td>
<td>0.036</td>
<td>14.8</td>
<td>1.90</td>
<td>2.27</td>
<td>0.28</td>
<td>0.55</td>
<td>3.0E–02</td>
<td>0.196</td>
</tr>
<tr>
<td>Ap-l −2.40 to −4.80</td>
<td>A</td>
<td>1.06</td>
<td>–</td>
<td>0.22</td>
<td>0.356</td>
<td>0.036</td>
<td>14.8</td>
<td>1.90</td>
<td>2.27</td>
<td>0.28</td>
<td>0.55</td>
<td>3.0E–02</td>
<td>0.196</td>
</tr>
<tr>
<td>Aco −4.80 to −7.95</td>
<td>A</td>
<td>1.41</td>
<td>–</td>
<td>0.30</td>
<td>0.130</td>
<td>0.013</td>
<td>3.5</td>
<td>1.86</td>
<td>1.26</td>
<td>0.44</td>
<td>0.50</td>
<td>3.0E–03</td>
<td>0.109</td>
</tr>
<tr>
<td>As −7.95 to −12.00</td>
<td>B</td>
<td>1.80</td>
<td>9100</td>
<td>0.34</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Ac-u −12.00 to −17.40</td>
<td>A</td>
<td>1.73</td>
<td>–</td>
<td>0.32</td>
<td>0.099</td>
<td>0.010</td>
<td>2.18</td>
<td>1.30</td>
<td>1.31</td>
<td>0.47</td>
<td>0.54</td>
<td>4.0E–04</td>
<td>0.103</td>
</tr>
<tr>
<td>Ac-l −17.40 to −22.80</td>
<td>A</td>
<td>1.66</td>
<td>–</td>
<td>0.36</td>
<td>0.120</td>
<td>0.012</td>
<td>2.43</td>
<td>1.03</td>
<td>1.00</td>
<td>0.56</td>
<td>0.56</td>
<td>3.0E–04</td>
<td>0.129</td>
</tr>
</tbody>
</table>

Model-A: Modified Cam-clay model, Model-B: Elastic material, $\rho_i$: wet density, $E$: modulus of elasticity, $\nu$: Poisson's ratio, $\lambda$, $\kappa$: compression index, swell index in relationships between $\ln \nu$ and $\ln \nu'$, $\nu_0$: initial volume ratio, $M$: critical state parameter, OCR: over consolidation ratio, $K_{0NC}$: coefficient of earth pressure at rest in normally consolidated condition, $K_{0OC}$: coefficient of earth pressure at rest in overconsolidated condition, $k_0$: initial coefficient of permeability, $\lambda'$: change in permeability coefficient in relationships between $\ln \nu$ and $\ln \nu'$.
Fig. 6. Location of the investigation sites in this study (distribution of peaty soft ground by Noto, 1991).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil type</th>
<th>Groundwater level GL</th>
<th>$q_t$ (CPT) (MN/m²)</th>
<th>Natural water content $W_n$ (%)</th>
<th>Compression index $C_c$</th>
<th>Yield stress $P_e$ (kN/m²)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.2</td>
<td>Peat</td>
<td>GL-1.2m</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>4.2</td>
<td>Organic clay</td>
<td></td>
<td>5</td>
<td>300</td>
<td>3.0</td>
<td>100</td>
<td>0.0</td>
</tr>
<tr>
<td>10.8</td>
<td>Sandy silt</td>
<td></td>
<td>10</td>
<td>600</td>
<td>6.0</td>
<td>200</td>
<td>1.0</td>
</tr>
<tr>
<td>20.5</td>
<td>Clay</td>
<td></td>
<td>15</td>
<td></td>
<td></td>
<td></td>
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</tr>
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</table>

Fig. 7. Geotechnical profiles at the Mihara site.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Soil type</th>
<th>Groundwater level GL</th>
<th>$q_t$ (CPT) (MN/m²)</th>
<th>Natural water content $W_n$ (%)</th>
<th>Compression index $C_c$</th>
<th>Yield stress $P_e$ (kN/m²)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7</td>
<td>Peat</td>
<td>GL-0.2m</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>4.2</td>
<td>Organic clay</td>
<td></td>
<td>5</td>
<td>300</td>
<td>3.0</td>
<td>100</td>
<td>0.0</td>
</tr>
<tr>
<td>5.3</td>
<td>Peat</td>
<td></td>
<td>10</td>
<td>600</td>
<td>6.0</td>
<td>200</td>
<td>1.0</td>
</tr>
<tr>
<td>8.8</td>
<td>Fine sand</td>
<td></td>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20.3</td>
<td>Silt</td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>22.0</td>
<td>Silt/Fine sand</td>
<td></td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 8. Geotechnical profiles at the Shinotsu site.
test (see Fig. 12) indicated no noticeable large lateral deformations.

4.2. Consolidated undrained triaxial compression test

A series of isotropically consolidated undrained triaxial compression test was performed for sampled specimens, following the Japanese Geotechnical Society specifications (JGS 0523-2000). The peat specimens were 75 mm in diameter and 150 mm in height, while the organic clay and clay specimens were 50 mm in diameter and 100 mm in height.

To obtain strength parameters under the normal consolidated state, the order of the consolidation pressure was large enough to exceed the consolidation yield stress. Consolidation was terminated when the volume change versus logarithm of time ($t$) curve reaches $3t$-line, as specified by the JGS. After that, the specimen was subjected to undrained shear at an axial strain rate of 0.1%/min. Although most of the peat exhibited no peak in axial strain, the effective angle of shear resistance ($\phi'$) was defined at an axial strain of 15%.

4.3. Ignition loss test

The specimen was heated at 700–800 °C according to Japanese Industrial Standards (JIS A 1226) with the exception of the ignition duration: i.e., 4 h, which is 3 h longer than specified. This is because Nakayama et al.
(1989) reported that when ignition time is short, the organic matter in peat does not completely ignite and an hour testing underestimates 30% of the real ignition loss for highly organic soils. In addition to the longer ignition time, the specimen was prepared for the test at 90°C rather than 110°C since some of the organic matter may have been burned and be lost at 110°C. Considering the heterogeneity of the peat ground, three specimens were tested and $L_i$ in this paper is the average of the measured values for these specimens.

4.4. Flat Dilatometer Test (DMT)

DMT is an in-situ horizontal loading test, penetrating a thin blade (93 mm wide × 16 mm thick) into the ground and expanding the center of a metallic membrane in the blade by gas pressure. From the DMT, three indices are calculated. Only the horizontal stress index, $K_D$, was discussed in this study for evaluating the $K_0$. $K_D$ can be expressed by Eq. (2), where $P_0$ is the measured gas pressure at 0.1 mm expansion of the membrane, $u_0$ is the hydrostatic pressure, and $\sigma'_v$ is the effective overburden pressure.

$$K_D = (P_0 - u_0)/\sigma'_v \quad (2)$$

5. $K_0$ value from laboratory and field tests

5.1. $K_0$ at normal consolidated state

Fig. 13 plots a typical relationship between the axial consolidation pressure ($\sigma'_a$) and the $K_0$ value for peat retrieved from the Tsuruno site at depths of 1.39–1.56 m. When the axial consolidation pressure exceeds the consolidation yield stress and enters the normal consolidation stage, the $K_0$ value becomes almost constant. The constant $K_0$ value under the normally consolidated state has been also confirmed by several researchers, such as Miyakawa et al. (1974), Fujikawa et al. (1983), Kawano et al. (1986), Edil and Dhowian (1981), and Mesri and Ajlouni (2007). In this paper, the $K_0$ value in the normal consolidation is denoted as $K_{0\text{NC}}$.

Fig. 14 shows the relationship between $L_i$ and $K_{0\text{NC}}$ at the four investigation sites. The $K_{0\text{NC}}$ decreases linearly with increases in $L_i$ (i.e., increases in organic matter pressure).
content). For organic clay and peat with relatively low $L_i$, the $K_{0NC}$ is approximately 0.4, which is around 0.1 smaller than that of inorganic clay. Samples with high $L_i$ ($L_i = 85\%$ and 91\%) indicate $K_{0NC}$ values of 0.21 and 0.32, which is very low compared to that of inorganic clay.

The relationship between $L_i$ and $K_{0NC}$ can be approximated as the linear line shown in Fig. 14, including inorganic clay as well as organic clay. This relation is approximated as follows, where $L_i$ is ignition loss in %.

$$K_{0NC} = 0.47 - 0.0025 L_i$$

Mitachi and Fujiwara (1986) also measured the $K_{0NC}$ of undisturbed peat collected in Hokkaido, by a similar $K_0$-consolidation test using the triaxial test apparatus. Their measured point is also well fitted by this equation.

For the relationship between $K_{0NC}$ and $\phi'$, the following equation by Jaky (1948) is well known:

$$K_{0NC} = 1 - \sin \phi'$$

Watabe et al. (2003) confirmed that Eq. (4) gives accurate results for mineral clay. The relationship between $K_{0NC}$ and $\sin \phi'$ obtained from the isotropically consolidated undrained triaxial compression test is shown in Fig. 15. It can be seen that Jaky relation can be also applied to peat; in other words, the small $K_0$ value for peat is due to large $\phi'$ of peat.

5.2. $K_0$ at the overconsolidated state

As discussed in the previous chapter, the peat layer is in general somewhat overconsolidated and the $K_{0OC}$ value for peat is strongly affected by the calculated lateral displacements by FE analysis. After applying an axial consolidation pressure higher than the consolidation yield stress, unloading was performed in the $K_0$-consolidation test, as shown in Fig. 13. It can be seen that the $K_0$ value during the unloading process is greater than that at the normal consolidated state. Fig. 16 shows a typical relationship between $OCR$ and $K_{0OC}$ obtained from the results shown in Fig. 13. At the depth

$$m = 0.005 L_i (\%) + 0.45$$

Fig. 16. The typical relationship between $OCR$ and $K_{0OC}$ (Tsuruno site at depths of 1.39–1.56 m).
where this sample was retrieved, the $OCR$ was 1.37, and the $K_{0OC}$ at the in situ was estimated as 0.41.

It is well known that the $K_{0OC}$ for inorganic soils increases linearly with the $OCR$ in the log–log plot. Schmidt (1966) proposed the following equation for the relationship between $OCR$ and $K_{0OC}$:

$$K_{0OC} = K_{0NC}OCR^m$$  \hspace{1cm} (5)

where $m$ is the gradient when the $K_{0OC}$ and $OCR$ are plotted in log–log scale.

In this study, the relationship between $OCR$ and $K_{0OC}$ for peat as well as organic clay exhibited the same relation as inorganic soils, as shown in Fig. 16. That is, the $K_0$ of peat and organic clay are dependent on stress history and can be expressed by a function of $OCR$.

Fig. 17 shows the relationship between the $m$ value and $L_i$. The $m$ value for inorganic clay obtained by this study ranged between 0.46 and 0.52. Watabe et al. (2003) reported that $m$ values measured from a large number of clayey sites around the world have similar $m$ values. Hamouche et al. (1995) reported that clays from sites in North America present the $m$ value of 0.4–0.5 and some sensitive clays show relatively high $m$ values. These researchers suggest that most inorganic clays have the $m$ value of approximately 0.5, except for some very sensitive clays. In contrast, organic clay and peat have very high $m$ values, particularly peat. For the sample with high

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth(m)</th>
<th>Soil type</th>
<th>$w_0$ (%)</th>
<th>$L_i$ (%)</th>
<th>$OCR$</th>
<th>$\phi$(deg)</th>
<th>$K_{0NC}$</th>
<th>$m$</th>
<th>$K_{0OC}$</th>
</tr>
</thead>
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<tr>
<td>Mihara</td>
<td>3.00–3.86</td>
<td>Organic clay</td>
<td>82</td>
<td>17</td>
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<td>45.3</td>
<td>0.57</td>
<td>0.49</td>
<td>0.77</td>
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<td></td>
<td>12.50–13.36</td>
<td>Clay</td>
<td>64</td>
<td>–</td>
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<td>30.4</td>
<td>0.52</td>
<td>0.53</td>
<td>0.52</td>
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<tr>
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<td>16.30–17.36</td>
<td>Clay</td>
<td>57</td>
<td>–</td>
<td>1.8</td>
<td>31.9</td>
<td>0.53</td>
<td>0.52</td>
<td>0.75</td>
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<td>Shinotsu</td>
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<td>Peat</td>
<td>407</td>
<td>45</td>
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<td>36.9</td>
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<td>0.62</td>
<td>0.69</td>
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<tr>
<td></td>
<td>2.99–3.17</td>
<td>Organic clay</td>
<td>325</td>
<td>27</td>
<td>3.3</td>
<td>36.5</td>
<td>0.30</td>
<td>0.57</td>
<td>0.56</td>
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<td></td>
<td>3.83–4.00</td>
<td>Organic clay</td>
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<td>17</td>
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<td>36.5</td>
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<td>0.61</td>
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<td>4.18–4.35</td>
<td>Peat</td>
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<td>51</td>
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<td>5.02–5.19</td>
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<td>Clay</td>
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<td>0.61</td>
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<tr>
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<td>16.45–16.59</td>
<td>Clay</td>
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<td>Clay</td>
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<td>32.1</td>
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<td>0.49</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>4.77–4.95</td>
<td>Organic clay</td>
<td>91</td>
<td>17</td>
<td>1.5</td>
<td>–</td>
<td>0.43</td>
<td>0.48</td>
<td>0.50</td>
</tr>
<tr>
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<td>5.29–5.47</td>
<td>Peat</td>
<td>540</td>
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<td>1.3</td>
<td>46.4</td>
<td>0.21</td>
<td>0.83</td>
<td>0.23</td>
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<tr>
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<td>6.47–6.65</td>
<td>Organic clay</td>
<td>176</td>
<td>14</td>
<td>2.0</td>
<td>–</td>
<td>0.43</td>
<td>0.53</td>
<td>0.58</td>
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<tr>
<td>Riyamunai</td>
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<td>Peat</td>
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<td>91</td>
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<td>48.1</td>
<td>0.32</td>
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<tr>
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<td>4.57–4.74</td>
<td>Peat</td>
<td>628</td>
<td>59</td>
<td>2.3</td>
<td>50.7</td>
<td>0.35</td>
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<td>0.55</td>
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<tr>
<td></td>
<td>7.47–7.60</td>
<td>Clay</td>
<td>85</td>
<td>11</td>
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<td>30.0</td>
<td>0.48</td>
<td>0.48</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Fig. 18. The relationship between $K_D$ and $K_0$ obtained from $K_0$-consolidation tests.

Fig. 19. The relationship between $K_{0DMT}$ estimated by the proposed equation and $K_0$ value from $K_0$-consolidation tests.
ignition loss ($L_\text{i} = 91\%$), the $m$ value was as large as 0.90. This indicates that the $K_{\text{0OC}}$ of peat is more strongly affected by $OCR$ than that of inorganic clay. Furthermore, it was found that the $m$ value increased linearly with increases in $L_\text{i}$ (see Fig. 17), and this relationship can be approximated as follows:

$$m = 0.005L_\text{i} + 0.45$$

(6)

Combined with Eqs. (3), (5) and (6), $K_{\text{0OC}}$ can be finally expressed as follows, in terms of $L_\text{i}$ and $OCR$:

$$K_{\text{0OC}} = (0.47-0.0025L_\text{i})OCR^{0.005L_\text{i}+0.45}$$

(7)

Considering heterogeneity in the peat layer, Eq. (7) is very useful in practice because it consists of relatively simple and robust parameters of $L_\text{i}$ and $OCR$. The test results in this study are summarized in Table 3. The $W_n$, $L_\text{i}$ and $\phi'$ values for peat and organic clay shown in Table 3 are within the range of previous data obtained in Hokkaido region (JGS, 1990).

5.3. $K_0$ obtained from DMT

For heterogeneous materials such as peat, a simple in situ test would be the most preferable method since it does not require soil samples whose properties are significantly changed vertically and horizontally. The DMT seems an effective method for estimating the $K_0$ value of peat ground because of a simple and convenient in situ test method. In this study, a method for estimating the $K_0$ value from DMT was examined. As mentioned earlier,
$K_D$ was an index obtained by DMT, and using $K_D$, several equations have been proposed to estimate $K_0$, as follows (Eq. (8)—Marchetti, 1980; Eq. (9)—Iwasaki, 1999):

$$K_{0DMT} = (K_D/1.5)^{0.47} - 0.6$$

(8)

$$K_{0DMT} = 0.29K_D^{0.57}$$

(9)

where $K_{0DMT}$ is estimated $K_0$ value using DMT.

Fig. 18 shows the relationship between $K_D$ and $K_0$ obtained from the $K_0$-consolidation tests in a log-log plot. The measured trend in the relationship between $K_D$ and $K_0$ for peat layers is very different from the lines obtained using Marchetti’s and Iwasaki’s equations, especially in small $K_D$. The best fitted line for the peat layer is the following equation:

$$K_{0DMT} = 0.5K_D^{0.22}$$

(10)

To verify the applicability of Eq. (10), the relationship between the $K_{0DMT}$ estimated by Eq. (10) and the $K_0$ values from $K_0$-consolidation tests is shown in Fig. 19. The ratios of $K_0$ to $K_{0DMT}$ are approximately in the range of 0.8–1.3. Hayashi and Nishimoto (2006) reported that the ratios of $K_0$ to $K_{0DMT}$ estimated by Marchetti’s and Iwasaki’s equations for clay layers in Hokkaido are approximately 0.6–1.2. These results show that the applicability of Eq. (10) for peat and organic clay layers is similar to that of Marchetti’s and Iwasaki’s equations for clay.

Fig. 20 shows a comparison of the estimated $K_0$ values by DMT using different equations including the proposed one in this paper, with obtained $K_{0NC}$ values from $K_0$-consolidation tests, at the Shinotsu and the Tsuruno sites. The applicability in $K_0$ estimated by Marchetti’s and Iwasaki’s equations for inorganic clay is relatively good. However, large scatters are observed in the relationship between estimated $K_{0DMT}$ and $K_0$ for the peat and organic clay layers. Especially when the $K_D$ is very low ($K_D=0.07–0.18$), the estimates become increasingly inaccurate. This is particularly true where the points show the data at the Tsuruno site: the $K_{0DMT}$ estimated by Marchetti’s equation presents a negative value. For peat and organic clay layers, Eq. (10) provides much more accurate $K_0$ values.

6. Conclusions

The value of $K_0$ for peat and organic clay was obtained using triaxial testing apparatus and a method for estimating $K_0$ was proposed in terms of simple soil parameters: OCR and the ignition loss. A flat dilatometer test was also performed and a new correlation of $K_0$ with $K_D$ was established. The main results of this study are summarized below:

1. The $K_{0NC}$ for peat and organic clay under the normally consolidated state are smaller than that for usual inorganic soils, and the $K_{0NC}$ decreases linearly with increases in the ignition loss ($L_i$). This relationship is described by $K_{0NC}=0.47–0.0025L_i$ (%).

2. The peat layer has a tendency to be overconsolidated due to the seasonal changes in groundwater level. Furthermore, the $K_{0OC}$ value for peat is strongly affected by the calculated lateral displacements by elasto-plastic FE analysis. Therefore, it is necessary to carefully examine the relationship between OCR and $K_0$ value under in-situ stress to determine the $K_0$ value for peat ground.

3. The $K_0$ of peat and organic clay are dependent on stress history and can be expressed by a function of OCR, as indicated in $K_{0OC}=K_{0NC}OCR^n$.

4. The $K_0$ value for peat and organic clay layers depends more strongly on OCR than that of inorganic clay. The $m$ value, which is the power on OCR, increases linearly with increases in $L_i$. This relationship is described by $m=0.005L_i$ (%) + 0.45.

5. When using DMT to estimate the $K_0$ of peat and organic clay layers, $K_{0DMT}=0.5K_D^{0.22}$ shows higher applicability than the conventional equations.

References


