ESTIMATION OF IN-SITU UNDRAINED STRENGTH OF SOFT SOIL DEPOSITS BY USE OF UNCONFINED COMPRESSION TEST WITH SUCTION MEASUREMENT

TOSHIYUKI MITACHI, YUTAKA KUDO and MASAKI TSUSHIMA

ABSTRACT

Unconfined compression tests have been widely used in Japan for the purpose of determining undrained strengths of clay samples, but the unconfined compressive strengths are usually scattered even if specimens tested seem to have been subjected to the same stress history. This is due to the characteristic of the test itself, which is performed under unstable confining stress given by capillary pressure of the sample itself. In order to make clear the cause of scattering of measured strengths, three series of tests simulating the process from sampling to unconfined and triaxial compression tests on saturated soil samples are performed with three remolded and three undisturbed clays together with an undisturbed peat, and the influence of stress release and mechanical disturbance of the test specimen on the undrained strength is examined. Based on the test results, a practical method for estimating the in-situ undrained strength of soft soils from the results of a routine unconfined compression (UC) test together with suction measurement is proposed. From comparisons of the undrained strength profiles estimated by the proposed method with the strength profiles obtained by in-situ sounding test, it is found that the strength obtained by UC test can be reasonably corrected to estimate the in-situ undrained strength of soft soil deposits.

Key words: clay, highly organic soil, overconsolidation, residual stress, sample disturbance, shear strength, suction, test procedure, unconfined compression test (IGC: D6/D5/C6)

INTRODUCTION

In estimating in-situ shear strengths based on laboratory shear tests, high quality "undisturbed" samples are required. In general, the shear strength of cohesive soil is greatly influenced by the stress change to which they have been subjected before shear. Even if the improvement of sampling technique meant that perfectly "undisturbed" samples could be obtained without mechanical disturbance, they would inevitably be subjected to a change in stress condition on removal from the ground, and the strength obtained might be more or less affected by this change. Moreover, soil samples suffer various mechanical disturbances due to vibration during transportation, straining during extrusion from the tube, trimming work and so on. Since the disturbances due to these causes include human factors, even samples retrieved from the ground by a high level sampling technique will be damaged if they are handled in a careless manner, and considerable scattering of unconfined compression strengths is usually observed. Therefore, it is very important to evaluate the sample quality just before laboratory testing.

Research on the influence of stress release and disturbances on the strength characteristics of cohesive soils has been extensively done since the 1960s (e.g. Skempton and Sowa, 1963; Ladd and Lambe, 1963; Okumura, 1963 and 1971; Noorany and Seed, 1965, among others), and its significance has been strongly recognized. Unconfined compressive strength ($q_u$) is especially sensitive to disturbance and consequently the data of $q_u$ are usually considerably scattered, even if the tested specimens seem to have been subjected to the same stress history. This is due to the fact that the unconfined compression test itself is performed under unstable confining pressure originating from the residual effective stress of the individual test specimen.

In view of these circumstances, the authors have investigated the influence of sample disturbance on $q_u$ (Mitachi et al., 1988; 1990) and proposed a method for estimating the in-situ undrained strength of clays from the results of a routine unconfined compression test with additional measurement of suction (Mitachi et al., 1994; 1996). By this method, in-situ undrained strength ($s_u$) corresponding to effective overburden stress ($\sigma_{oc}$) can be estimated based on the observed data of unconfined compressive strength $q$, and residual effective stress ($\sigma'$), together with compression and swelling indices $C_v$ and $C_s$ obtained...
by oedometer test. Even if $C_i$ and $C_v$ values are not available, in-situ undrained strength can also be estimated by plotting several sets of $q_u$ data with different values of quasi-overconsolidation ratio QCR ($=\sigma_{oc}/\sigma_i'$) for samples with different degrees of disturbance.

Examination of the effect of strain rate on the $q_u/\sigma_i'$ versus QCR relationship and confirmation of applicability of the proposed method to overconsolidated soils have already been carried out by conducting a series of simulation tests (Mitachi and Kudoh, 1996). The authors have also confirmed the validity of the proposed method by comparing undrained strength profiles estimated by the proposed method with strength profiles obtained by in-situ sounding tests at several domestic and foreign sites (Mitachi et al., 1998).

In this paper, a review of the theoretical background and a detailed explanation of the procedure of estimating in-situ undrained strength are given, and additional experimental results confirming the validity of one of the basic assumptions of the proposed method are also presented. A summary of the achievements mentioned above was presented at IS-Yokohama 2000 (Mitachi et al., 2000). Additional examination of the applicability of the proposed method to overconsolidated and aged soils by performing a series of simulation tests using undisturbed peat samples is also described in this paper.

**EFFECT OF DISTURBANCES ON UNDRAINED STRENGTHS**

In order to make clear the cause of scattering of measured strengths $q_u$, three series of tests simulating the process from sampling to unconfined and triaxial compression tests on saturated soil samples were performed, and the influence of stress release and mechanical disturbance of the test specimen on the undrained strength was examined.

**Samples Tested and Specimen Preparation**

The index properties of three remolded clays, MC kaolin, NSF and Kiyohoro clay, and three undisturbed clays, Shinoro and Iwamizawa clay sampled from a site near Sapporo City and Ariake clay from Saga Prefecture, are shown in Table 1, along with the data of consolidation and swelling indices of each sample. Remolded clays, which were in a powder state, were mixed with distilled water to obtain a water content twice their liquid limit. Before making the test specimen, the slurry was stirred in a soil mixer for about two hours and then transferred under a vacuum to a preconsolidation cell, the diameter and the height of which were 200 mm and 400 mm, respectively. The slurry was initially consolidated one dimensionally under a vertical consolidation pressure of 100 kPa. After consolidation, the samples were split into blocks of $90 \times 90 \times 180$ mm, which were coated with paraffin and wrapped with polyethylene film. Cylindrical specimens for shear tests, the diameter and the height of which were 50 mm and 120 mm or 75 mm and 150 mm respectively, were trimmed vertically from blocks of preconsolidated samples. In the case of the three undisturbed clays, samples were extruded from sampling tubes and were then carefully trimmed to make unconfined compression test specimens 50 mm in diameter and 120 mm in height.

Undisturbed peat samples used in the experiments were retrieved from a site in the vicinity of Akita City; its index properties are given in Table 2. Samples extruded from sampling tubes of 70 mm in diameter and 300 mm in length were carefully trimmed by cutting the top and bottom surfaces only to make specimens 70 mm in diameter and 160 mm in height.

**Table 1. Index properties of clay samples**

<table>
<thead>
<tr>
<th></th>
<th>MC</th>
<th>NSF</th>
<th>Kiyohoro</th>
<th>Shinoro</th>
<th>Iwamizawa</th>
<th>Ariake</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w_i$ (%)</td>
<td>87</td>
<td>55</td>
<td>41</td>
<td>77</td>
<td>54</td>
<td>135</td>
</tr>
<tr>
<td>$I_v$</td>
<td>49</td>
<td>26</td>
<td>16</td>
<td>43</td>
<td>19</td>
<td>75</td>
</tr>
<tr>
<td>$\rho_s$ (g/cm³)</td>
<td>2.77</td>
<td>2.78</td>
<td>2.73</td>
<td>2.72</td>
<td>2.67</td>
<td>2.61</td>
</tr>
<tr>
<td>$C_i$</td>
<td>0.730</td>
<td>0.320</td>
<td>0.171</td>
<td>0.668</td>
<td>0.490</td>
<td>-</td>
</tr>
<tr>
<td>$C_v$</td>
<td>0.200</td>
<td>0.051</td>
<td>0.017</td>
<td>0.066</td>
<td>0.055</td>
<td>-</td>
</tr>
</tbody>
</table>

$C_i$: Compression index, $C_v$: Swelling index

**Table 2. Index properties of peat sample**

<table>
<thead>
<tr>
<th>Initial water content $w_i$ (%)</th>
<th>Ignition loss $L_s$ (%)</th>
<th>Degree of decomposition $H$ (%)</th>
<th>Density of soil particle $\rho_s$ (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>560–680</td>
<td>60–70</td>
<td>80–90</td>
<td>1.69–1.75</td>
</tr>
</tbody>
</table>

**Test Apparatus and Testing Program**

In the test series simulating stress change during the process from sampling to unconfined compression test of samples initially consolidated under either the $K_0$ or $K$-constant anisotropic stress condition, triaxial compression apparatus with an automated $K_0$ consolidation system (Oda and Mitachi, 1992) was used. Experiments performed in this study are divided into three series of tests simulating the stress and strain history of the sample.

1) Experiments Simulating the Stress Change of Samples Taken from Normally Consolidated Soil Deposits

Three remolded and three undisturbed clay samples were used for the following experiments. Testing procedure and stress change during each simulation test program are shown in Fig. 1. Except in the UC test for undisturbed samples, $K_0$ consolidation with a back pressure ($u_b$) of 100 kPa was first performed on all specimens, and then the following experiments were conducted.

a) IS test ($K_0$ Consolidated Undrained Triaxial Compression Test): specimens were consolidated under $K_0$ condition or $K$-constant ($K=K_0$) anisotropic stress condition by an effective vertical stress 1.5 to 2 times the in-situ overburden stress for undisturbed clay samples, and 1.5 to 4 times the preconsolidation stress for remolded clay samples...
Fig. 1. Simulation of process from sampling to unconfined and triaxial compression test

respectively. Then they were sheared under undrained condition at a strain rate of 0.01%/min or 1%/min. These tests simulate the in-situ undrained strength (ideal strength) of clay at shallow and deep deposits.

b) PS test (Unconfined Compression Test on Perfect Sample): after $K_0$ consolidation, vertical and horizontal stresses were released under undrained condition until they became equal to back pressure $u_b$ during consolidation, and then the samples were sheared in the triaxial cell. As shown in Fig. 1, a cell pressure $\sigma_0 (=u_b)$ acts on the specimen so that it appears like a triaxial compression test, but considering effective stress acting on the specimen, the strength obtained by this test corresponds to the unconfined compressive strength of the sample subjected only to the effect of relieving in-situ stress.

c) UC test (Unconfined Compression Test with Suction Measurement):

Undisturbed Clay Samples:

Specimens extruded from a thin wall tube sampler were trimmed to become cylindrical specimens 50 mm in diameter, which were then cut into two parts: a long specimen (SL) of 100 mm height and a short specimen (SS) of 20 mm height, as shown in Photo 1. Unconfined compression tests were performed by using SL specimens with a strain rate of 0.01%/min or 1%/min in order to examine rate effect, and measurements of initial suction ($u_s$) were made simultaneously, as shown in Photo 2(c), by using SS specimens. The simple device shown in Photo 2(a) was used for suction measurement of specimens sampled from shallow deposits and a small chamber such as shown in Fig. 2, which is able to apply back air pressure $\sigma_a$, was used for specimens sampled from deep deposits. Ceramic discs, the air entry value (AEV) of which was 240 kPa, were assembled as shown in Photo 2(a) and Fig. 2 to measure the suction of SS specimens.

Remolded Clay Samples:

Simulation of the process from sampling to the unconfined compression test with suction measurement was performed as follows. After $K_0$ consolidation or $K$-constant ($K=K_0$) anisotropic stress consolidation with a specimen of 75 mm diameter and 150 mm height, vertical and horizontal stresses were released under undrained condition and some intentional disturbances (5 times repeated application of $\pm 5\%$ axial strain) were given to the specimen. Then the triaxial cell was disassembled and the filter paper strips on the specimen were removed. The specimens were trimmed to 50 mm diameter, and were then cut into two parts: a 100 mm high specimen (SL) and a 20 mm high specimen (SS). Unconfined compression tests were per-
formed with the SL specimen, and measurement of suction was performed simultaneously with the SS specimen.

d) CAU test (Anisotropically Consolidated Un-drained Triaxial Compression Test)

As in the previous case, the triaxial cell was dis-assembled after finishing consolidation, filter paper and membrane were newly wrapped around
the specimen, and after consolidating anisotropically again, the undrained shear test was performed. The purpose of this test was to investigate the effect of application of back pressure on the undrained strength obtained by triaxial compression tests after consolidation under in-situ overburden stress.

(2) Experiments Simulating the Stress Change of Samples Taken from Overconsolidated Soil Deposits

Remolded and preconsolidated MC Kaolin samples were used for the following experiments.

a) IS test ($K_0$ Consolidated and Rebounded Undrained Triaxial Compression Test): Specimens were first consolidated to a vertical effective stress of 400 kPa under $K_0$ condition, allowed to swell with reduced vertical stress to OCR=2 and 3 under $K_0$ condition, and finally subjected to undrained shear.

b) UC test (Unconfined compression test with suction measurement on overconsolidated clay)

Specimens, the initial diameter and height of which were 75 mm and 150 mm respectively, were consolidated and rebound under $K_0$ condition, and then the vertical and horizontal stresses were released and the specimens were trimmed to 50 mm diameter and 120 mm height. Subsequently unconfined compression tests with initial suction measurement were performed by the same procedure as mentioned in the previous section.

(3) Experiments Simulating the Process from Sampling to Unconfined Compression Test for Aged Soil Deposits

To investigate the influence of ageing on the unconfined compressive strength of soils, the following series of tests was performed with undisturbed peat soil by making use of its characteristic of high volume change development during ageing.

a) IS test: samples 70 mm in diameter and 160 mm in height were consolidated anisotropically ($K=0.6$) under different mean effective stress of 40, 60 and 100 kPa for each four series of consolidation duration 1, 4, 10 and 20 days, and then undrained triaxial compression tests were performed.

b) PS test: after anisotropic consolidation of 1, 4, 10 and 20 days, vertical and horizontal stresses were released under undrained condition and then the samples were sheared in the triaxial cell.

c) UC test: after anisotropic consolidation of 1, 4, 10 and 20 days, the triaxial cell was disassembled and filter paper strips on the specimen were removed. After resetting the sample in the triaxial apparatus again, an unconfined compression test with pore water pressure measurement was performed.

In the test series on peat soil mentioned above, triaxial test apparatus with a ceramic disc assembled in the pedestal was used and the rate of axial strain in the undrained shear stage of IS and UC test was 0.1% / min.

Figure 3(a) shows an example of the effective stress paths of IS, PS and UC tests on MC kaolin in terms of principal stress difference ($\sigma_1 - \sigma_2$) and mean effective stress $\sigma_{\text{eff}} (=\sigma_1 + 2\sigma_2)/3$ normalized by effective vertical consolidation pressure $\sigma_{\text{con}}$, where broken lines in the figure illustrate conceptual change of effective stress during the process from sampling to laboratory shear test. From this figure, it is found that the effective stress at the start of undrained shear, i.e. residual effective stress $\sigma_{\text{r}} (=\sigma_0 - u)$, where $\sigma_0$ and $u$ are back air pressure and pore water pressure, respectively decreases with the increase of disturbance. The undrained strength reduction for the sample experiencing stress release alone (PS test) is less than 10%. The strength decreases 20% for the carefully performed unconfined compression test (UC1), and 30% for the sample which is given an intentional disturbance of 5 cycles of ±5% axial strain (UC2). These strength decreases are controlled by the reduction of initial effective stress due to stress release and mechanical disturbances experienced before shear. It is also found that the magnitude of residual effective stress is important for an index of sample disturbance and a measure of sample quality evaluation.

Almost the same trend as in Fig. 3(a) is seen in Figs. 3(b) and (c) for Kiyohoro clay and NSF clay. In Fig. 3(b), CAU means undrained triaxial compression test for the specimen experiencing initial $K_0$ consolidation and stress.

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**Fig. 3.** Effective stress paths during undrained shear in simulation tests for: (a) MC kaolin, (b) Kiyohoro clay and (c) NSF clay
release, which is reconsolidated successively (with back pressure of 200 kPa for CAU1 and without back pressure for CAU2) under anisotropic stress condition. The effect of back pressure on the undrained shear strength is clearly seen, as has already been reported (e.g. Lowe and Johnson, 1960; Brand, 1975).

METHOD FOR ESTIMATION OF IN-SITU UNDRAINED STRENGTH

Frame of the Proposed Method

Figure 4 illustrates the conceptual change of effective stress from in-situ to laboratory. Point A shown in the figure represents the state after $K_0$ consolidation. The straight line with a slope of $-\lambda$ is the normal consolidation line in terms of mean effective stress $\sigma'_m$. The line with a slope of $-\kappa^*$ is the one combining the point A mentioned above with the point B which corresponds to the state after the change of void ratio due to effective stress change during the process from sampling to laboratory test.

Figures 5(a) and (b) show examples comparing the void ratio change during “undrained” stress release after isotropic consolidation, which is measured by the volume change of the inner cell in a double chamber type triaxial cell, with that during swelling, which is calculated from the volume of water sucked in the saturated specimen. It is considered that the volume increase during “undrained” stress release is caused by diffusion of dissolved air in the pore water (Okumura, 1974), and the assumption that a void ratio change corresponding to the difference between points A and B in Fig. 4 occurs even in “undrained” stress release seems to be supported.

Assuming that effective stress at the critical state of clay sheared under undrained condition starting from point D in Fig. 4 reaches the same point C that comes from point B, the following equations are derived.

As the void ratio change is the same for stress change along lines AB and AD, the following equation is obtained.

$$\sigma'_c / \sigma'_r = (\sigma'_c / \sigma'_r) \gamma^{*/\lambda}$$

where, $\sigma'_c$, $\sigma'_r$ and $\sigma'_e$ correspond to A, B and D in Fig. 4, respectively.

Assuming that the undrained strengths $s_u$ corresponding to point C are identical for both stress paths BC and DC, the ratio of $s_u$ to the residual effective stress $\sigma'_e$ is given as follows by using Eq. (1).

$$s_u / \sigma'_e = (s_u / \sigma'_e)(\sigma'_c / \sigma'_r)\gamma^{*/\lambda}$$

where,

$$\Lambda^* = 1 - \kappa^* / \lambda.$$ 

Mean effective stresses $\sigma'_c$ and $\sigma'_r$ are represented as follows by using vertical consolidation stress $\sigma'_vc$, equivalent consolidation stress $\sigma'_vc$ and $K_0$ value.

$$\sigma'_c = K\sigma'_vc$$

where,

$$\sigma'_c = K\sigma'_vc$$

$$K = (1 + 2K_0)/3.$$
an index representing the degree of disturbance. Since the parameters $K_0$, $\lambda$, $\kappa^*$ and $s_u/\sigma'_{uv}$ are considered to be constant for a given soil, Eq. (5) implies that the $s_u/\sigma'_i$ versus $QCR$ relationship shows a straight line in the log-log plot, and this relationship is confirmed for many kinds of clayey soils (Shimizu and Tabuchi, 1993; Mitachi et al., 1994; 1996) including highly organic soils (Tsushima and Mitachi, 1998).

The parameter $\kappa^*$ in Eq. (5) is the slope of the line linking the anisotropically consolidated state point with the point of isotropic effective stress state after the stress release accompanied by some disturbances. Assuming the value of $\kappa^*/\lambda \approx 0.1$, and calculating the coefficient $K^* = \lambda$ by changing the $K_0$ value at $0.5 - 0.6$, then we obtain $K^* = 0.96 - 0.97$. Therefore, approximating as $K^* = 1$, the equation for estimating in-situ undrained strength $s_u/\sigma'_i$ from the results of uncompressed compression test with suction measurement ($q_u/2\sigma'_i$) is obtained as follows since $s_u/\sigma'_{uv}$ in Eq. (5) is equal to $s_u/\sigma'_i$ and $s_u/\sigma'_i$ can be replaced by $q_u/2\sigma'_i$.

$$s_u/\sigma'_i \equiv (q_u/2\sigma'_i)QCR^{-\lambda^*}.$$  

Equation (6) is similar to the equation for estimating undrained shear strength of clay for any overconsolidated state using the strength test result for normally consolidated clay (Mitachi and Kitago, 1976).

Figure 6 illustrates examples of the $s_u/\sigma'_i$ versus log $QCR$ relationship obtained from the unconfined compression test with suction measurement, where $s_u$ is calculated as a half value of the unconfined compressive strength $q_u$ and $\sigma'_i$ is the residual effective stress obtained by suction measurement. Quasi-overconsolidation ratio $QCR$ is calculated by the ratio $\sigma'_{uv}/\sigma'_{ci}$, and $\sigma'_{uv}$ is the initial vertical consolidation stress. The coefficient $\lambda^*$ is calculated by the equation $\Lambda^* = 1 - C_s/C_i$, where $C_s$ and $C_i$ are compression and swelling indices obtained by oedometer test. The curves in the figure are obtained by using Eq. (6) together with the data obtained by UC test results. Fairly good coincidence of the calculated $s_u/\sigma'_i$ values, which are represented by the intersection of the curves and the ordinate axis ($QCR = 1$), with those measured from IS tests is shown in the figure.

A similar relationship to that shown in Fig. 6 can be seen in Fig. 7 for the case of initially overconsolidated clay. In Fig. 7, UC1, UC2 and UC3 are the unconfined compression test results for the sample $K_0$ overconsolidated to $QCR = 1, 2$ and 3. Curved lines for estimating $s_u/\sigma'_i$ which correspond to $QCR = 2$ and 3 are drawn based on the calculation using Eq. (6) and measured values of $q_u/2\sigma'_i$ (△ and ■) together with $\lambda^* = 1 - C_s/C_i$. Calculated values of in-situ $s_u/\sigma'_i$ for $QCR = 2$ and 3 (○) coincide quite well with the measured ones (△ and ■) by IS tests. Therefore, in-situ undrained strength, even for the overconsolidated case, can be estimated by unconfined compression test with suction measurement.

Figure 8 shows an example for a case when the value $C_s$ is not available. A series of unconfined compression tests with suction measurement was performed by the present authors on undisturbed Ariake clay samples obtained using a variety of sampling methods (Tanaka and Tanaka, 1995). Samples were retrieved by the pre-boring method and displacement method with a Japanese fixed piston sampler (JPN-P and JPN-D) and by the displacement method with an NGI sampler (NGI-D). In this figure of
logarithmic scale, the measured value of \( \frac{q_u}{\sigma'} \) is plotted against \( QCR \) which is calculated by dividing consolidation yield stress \( \sigma'' \) with \( \sigma' \). The point marked with (©) is the data obtained by undrained triaxial compression test performed after \( K_0 \) consolidation of the sample with effective overburden pressure (\( K_0 \) CU test). The slope of the straight line in the figure, which is fitted for the data points ( □, ○ and △ marks) of UC tests, corresponds to \( A* \) in Eq. (6). The intersection of the straight line and the ordinate axis coincides well with the data obtained by \( K_0 \) CU test (©).

Figure 9 shows an example of application of the proposed method to overconsolidated highly organic soil, in which \( \log \frac{s_u}{\sigma'} \) versus log \( QCR \) relationships obtained by IS and UC tests are plotted. The intersection of the straight line, which is fitted to the plotted points (● mark), and the ordinate axis (\( QCR = 1 \)) gives in-situ \( \frac{s_u}{\sigma''} \) for the normally consolidated state which corresponds to the measured value of the ○ mark. Calculated values of \( \frac{s_u}{\sigma''} \) by using the fitting line in Fig. 9 coincide well with measured values obtained by IS tests for overconsolidation ratios of 3, 6, 9 and 15 (© mark), respectively.

In addition to the experimental data shown above, IS and UC tests have been performed on various soils (Mitachi et al., 1998), the plasticity indexes, \( I_p \) of which are in the range of 16 to 112; it is confirmed that the proposed method can be successfully applied for these soils. Figure 10 shows the plot of \( A* \) values obtained from the slope of \( \log \frac{s_u}{\sigma'} \) versus log \( QCR \) relationship as shown in the examples of Figs. 8 and 9 against \( I_p \) values of the soils mentioned above. With one exception, \( A* \) varies from 0.7 to 1.0, and gives an average value of 0.85. Therefore, \( A* \approx 0.85 \) can be used as a reference value for most clayey soils with a wide range of \( I_p \).

**Procedure for Correction of Measured Unconfined Compressive Strengths**

Figure 11 is a schematic illustration of the procedure in which the method mentioned above is applied for cohesive soils in overconsolidated and normally consolidated aged states as well as the young normally consolidated state.

1. For overconsolidated and normally consolidated aged states;

   The state corresponding to point C in Fig. 11(a) is now considered for an example in which effective overburden stress and consolidation yield stress are defined as \( \sigma'_c \) and \( \sigma''_c \), respectively. For the sake of simplicity, it is assumed in Fig. 11(a) that the state point C aged from point A is equivalent to the state overconsolidated from point B.

   Procedures for correction of measured \( q_u \) are explained as follows;

   \[
   \frac{s_u}{\sigma'} = A* \frac{q_u}{2\sigma''} = \frac{q_u}{2\sigma'}
   \]

   Fig. 9. \( \log \frac{s_u}{\sigma'} \) vs. \( \log QCR \) relationship for overconsolidated peat soil

   Fig. 10. Trend of \( A* \) value for various cohesive soils of \( I_p = 16-112 \)

   Fig. 11. Proposed method for estimating in-situ undrained shear strength based on UC test with suction measurement: (a) state of specimen before UC test and (b) \( \log \frac{s_u}{\sigma'} \) vs. \( \log QCR \) relationship

   Fig. 12. Influence of variation of consolidation yield stress \( \sigma'' \) on estimated \( \frac{s_u}{\sigma'} \) value
1) Plot the data of measured unconfined compressive strengths divided by residual effective stress \( qu/2\sigma'_r \) with corresponding values of \( \sigma'_r / \sigma'_r \), which is called as quasi-overconsolidation ratio and is denoted by QCR in this paper, as shown in Fig. 11(b). Residual effective stress \( \sigma'_r \) is calculated from the data of suction measurement performed simultaneously with UC test on a specimen cut from the same sample used for the UC test.

2) Obtain a line having slope \( A^* \) which fits the data points given by the specimen having different values of QCR for samples with different degrees of disturbance, or for samples retrieved from different depths of the same soil deposits.

3) Calculate the in-situ overconsolidation ratio OCR (=\( \sigma'_r / \sigma'_c \)) for the given depth, which corresponds to point C in Fig. 11(a), by using consolidation yield stress \( \sigma'_r \) and effective overburden stress \( \sigma'_c \).

4) Applying the in-situ OCR value (=\( \sigma'_r / \sigma'_c \)) to the \( q_u/2\sigma'_r \) = QCR relationship as shown in Fig. 11(b), the value of \( s_u / \sigma'_c \) for the given depth can be obtained.

5) Multiplying the effective overburden stress \( \sigma'_c \) by the value of \( s_u / \sigma'_c \) obtained above, a corrected value of \( q_u/2 \) corresponding to point C is obtained.

(2) For normally consolidated state:

When the stress state is normally consolidated, corresponding to point B as shown in Fig. 11(a), the quasi-overconsolidation ratio QCR is calculated as the ratio of \( \sigma'_c / \sigma'_r \) since \( \sigma'_r = \sigma'_c \) in this case. In-situ undrained strength can be obtained by substituting OCR=1 in applying the procedure mentioned above.

Meanwhile, consolidation yield stress \( \sigma'_y \) may also be changed in the cases mentioned above depending on the degree of ageing and sample disturbances. It is known that \( \sigma'_y \) increases by the effect of ageing and decreases due to disturbances (e.g. Lerouell and Marques, 1996). In general, the oedometer test specimen for obtaining \( \sigma'_y \) differs from that for the unconfined compression test. Let us imagine a case in which the value of \( \sigma'_y \) is evaluated as small as \( \sigma'_y \) due to sample disturbance. Then the value of \( \sigma'_r / \sigma'_c \) shifts from the abscissa of point P to P* as shown in Fig. 12. Since in-situ overconsolidation ratio OCR is defined as the ratio of consolidation yield stress to effective overburden stress, the OCR value corresponding to the point C in Fig. 11(a) is calculated as \( \sigma'_y / \sigma'_c \) instead of \( \sigma'_r / \sigma'_c \) when the consolidation yield stress is evaluated as \( \sigma'_y \). Thus, the OCR values of \( \sigma'_r / \sigma'_c \) and \( \sigma'_y / \sigma'_c \) correspond to the abscissa of point Q and Q* respectively, in the \( \log s_u / \sigma'_c \) vs. \( \log QCR \) plot of Fig. 12. Therefore, the values of \( s_u / \sigma'_c \) corresponding to \( \sigma'_y / \sigma'_c \) and \( \sigma'_y / \sigma'_c \) are identical provided that the value of \( A^* \) is unchanged. The same result may be obtained for the case of an increased \( \sigma'_r \) due to the effect of ageing.

**Examples of Correction for Measured Unconfined Compressive Strengths**

Figure 13 shows the \( s_u / \sigma'_c \) versus log QCR relationship obtained by the simulation test series of IS, PS and UC tests for undisturbed peat mentioned earlier. The intersection of the straight line, which is fitted to the plotted points (○, ●, △, ▽ and □ marks), and the ordinate axis (QCR=1) gives in-situ \( s_u / \sigma'_c \). The value of \( s_u / \sigma'_c \) obtained above coincides well with the one measured by IS test (□ point) for consolidation duration \( T_d=1 \) day, as shown in the figure.

Measured values obtained by IS tests for consolidation durations of \( T_d=4, 10, 20 \) days, respectively are also shown in Fig. 13. Application of the proposed method shown in Fig. 11 to this case is presented as follows. Figure 14 shows the void ratio \( e \) versus consolidation pressure \( \sigma'_c \) relationship for samples consolidated under the condition of \( T_d=1, 4, 10, 20 \) days. From the plotted points (△, ▽, ● marks) obtained by different consolidation durations under the same effective vertical stress, parallel straight lines having the slope \( -C_e \) equal to that of swelling line, are drawn. The points of intersection of these straight lines and the straight line drawn through
the points for 1 day consolidation (□ mark) are presumed to be consolidation yield stress $\sigma_{cv}$. The ratio of $\sigma_{cv}$ for consolidation duration $T_c=4, 10, 20$ days to effective vertical consolidation stress $\sigma_{cv}$ gives quasi-overconsolidation ratio QCR. The value of QCR is plotted on the abscissa axis of Fig. 13, and the points of intersection with the straight line in this figure give in-situ strengths $s_i/\sigma_{cv}$ of the soil experienced ageing effect. As shown in this figure, $s_i/\sigma_{cv}$ values obtained above show fairly good coincidence with the ones measured by IS tests.

Figure 15 shows an example of correction of the measured $q_i/2$ values for consolidation duration $T_c=20$ days based on the log $s_i/\sigma_{cv}$ versus log QCR relationship obtained by applying the method shown in Fig. 13. The marks (●, ○) shown in the figure are the values of $q_i/2$ before and after correction, respectively. The marks (×) correspond to each in-situ undrained strength obtained by IS test. It is found that the strength obtained from unconfined compression test can be reasonably corrected to estimate in-situ undrained strength of the soil experiencing ageing effect by applying the method mentioned above.

Figure 16 shows an example of correction for measured unconfined compressive strengths of undisturbed Ariake clay based on the log $s_i/\sigma_{cv}$ versus log QCR relationship which is already shown in Fig. 8. Profiles of the measured values of unconfined compressive strengths along depth (○, ○ and △ marks) and corrected values of those by the proposed method (● mark) are shown in Fig. 16. Undrained strengths obtained by $K_0$ consolidated undrained triaxial test, cone penetration (CPT) and field vane (FV) test are also shown in Fig. 16 for the comparison. Test data obtained by CPT and FV were given by the Port and Harbor Research Institute (PHRI). The trend of undrained strength changes along depth obtained by the correction method mentioned above shows fairly good coincidence with those obtained by triaxial, CPT and FV test.

Figures 17(a) to (c) are examples of corrected $q_i/2$ versus depth relationships for clay deposits from Bothkennar (U.K.), Louisville (Eastern Canada) and Singapore. Unconfined compression tests with suction measurement were performed by PHRI (Tanaka et al., 1997). For the Bothkennar clay ($w_i=70\%$, $I_0=40$), scattering of the $q_i/2$ data is minimized by applying correction, and the increasing trend of the corrected $q_i/2$ data along depth becomes close to that of undrained strength $s_i$ by field vane shear test. Self boring pressure meter (SBPM) test results are also shown in the figure. Figure 17(b) illustrates the case for very sensitive cemented clay of Louisville ($w_i=60\%$, $I_0=30$). Corrected $q_i/2$ data, which were originally distributed almost constant irrespective of depth, show a clear tendency to increase along depth and to become very close to field vane shear test results. For the Singapore clay ($w_i=60-80\%$, $I_0=40-60$), the trend of corrected $q_i/2$ data varying along depth is close to that for CPT and FV test results.

![Fig. 16. Comparison of undrained strength profile corrected by proposed method with strength profile obtained by in-situ sounding test and $K_0$CU test results for Ariake clay](image1)

![Fig. 17. Comparison of undrained strength profile corrected by proposed method with strength profile obtained by in-situ sounding test results: (a) Bothkennar clay, U.K., (b) Louisville clay, Eastern Canada and (c) Singapore clay](image2)
CONCLUSIONS

Three series of tests simulating the process from sampling to unconfined and triaxial compression test on saturated soil samples were performed with three remolded and three undisturbed clays together with an undisturbed peat, and the influence of stress release and mechanical disturbance of test specimen on the undrained strength was examined. Based on the test results, a practical method for estimating the in-situ undrained strength of soft soils from the results of routine unconfined compression (UC) tests together with suction measurement was proposed. From the laboratory simulation test results and the comparisons of undrained strength profiles estimated by the proposed method with the strength profiles obtained by in-situ sounding test, it was found that:

1) Unconfined compressive strength depends strongly on the residual effective stress before undrained shear.

2) By measuring the suction value when performing unconfined compression test, the degree of sample disturbance can be evaluated by the magnitude of residual effective stress, and in-situ undrained strength can be estimated by using Eq. (6).

3) In-situ undrained strength can also be estimated without the data of compression and swelling indices by plotting at least two sets of unconfined compressive strength data having different values of quasi-overconsolidation ratio QCR, as shown in Figs. 8 and 9, for normally consolidated (aged) and overconsolidated samples with different degrees of disturbance. Therefore, unconfined compressive strength can be reasonably corrected by applying the simple method explained in section 3.2, provided that residual effective stress is measured simultaneously when a routine unconfined compression test is performed.

4) From the comparisons of undrained strength profiles estimated by the method mentioned above with the strength profiles obtained by in-situ sounding tests, such as field vane, cone penetration and pressure meter tests, it was found that the strength obtained by unconfined compression test can be reasonably corrected.

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