A METHOD FOR CORRECTING UNDRAINED SHEAR STRENGTH FOR SAMPLE DISTURBANCE

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ABSTRACT

In order to find out an appropriate method for correcting the undrained shear strength for sample disturbance, a series of triaxial compression tests is performed on mixed soil samples having wide range of plasticity index. Conditions of the ideal and perfect samples are brought to by the K_0 consolidation and the shear stress release after the consolidation, then further disturbance is given to specimens by cyclic loadings. Model sampling is made for obtaining the "undisturbed" samples, and unconsolidated undrained triaxial compression tests are performed on these "undisturbed" samples. In these compression tests, the pore pressure is measured for assessing the residual effective stress in the ideal, perfect and "undisturbed" samples respectively.

Results of the compression tests show that the pore pressure can increase on shear stress release for soils of low plasticity index, and that the effect of sample disturbance is similar to the effect caused by cyclic loadings. By combining the concepts of strength reduction due to rebound and reduction in effective stress due to sample disturbance, a comprehensive way for correcting the measured value of the undrained shear strength for sample disturbance is proposed.

Key words: cohesive soil, overconsolidation, plasticity, residual stress, sample disturbance, shear strength, triaxial compression test (IGC: D 6/C 6)

INTRODUCTION

The undrained shear strength c_u of cohesive soils has been widely used in the $\phi=0$ analysis of stability. In most cases, the c_u is determined from the unconfined compression test in Japan. It is known that there are three major factors influencing the c_u , i.e.

strength anisotropy, strain rate and sample disturbance. The strength of vertical samples, in general, is higher than that of horizontal samples, and the strain rate employed in laboratory shear tests is usually much higher than that occurring in situ. Then these two factors contribute to increase the c_u . Sample disturbance, on the

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other hand, decreases the c_u . Apparently successful use of the $\phi=0$ analysis with the c_u , therefore, is considered to have relied on the compensating of these three factors, i. e. sample disturbance offsets strength anisotropy and strain rate effects. It has been pointed out that the large variability in sample disturbance causes major uncertainty in the reliability of the geotechnical data (Ladd et al, 1977).

In dealing with a problem of sample disturbance, three different kinds of soil samples are considered, i. e. the ideal sample, perfect sample and so-called "undisturbed" sample. The ideal sample refers to the in situ state of a soil element. The perfect sample is the sample to which no disturbance has been given other than that involved with the release of the in situ shear stresses. The so-called "undisturbed" sample is the soil sample actually taken out from the ground. This kind of sample has experienced either the effect of the release of the in situ shear stress and mechanical disturbance occurring during the course of field sampling, storage and handling in laboratory. It is usual to assume that the water content of the perfect and "undisturbed" sample is equal to that of the ideal sample. The perfect sample and the "undisturbed" sample are under a condition of all round confining pressure.

Skempton and Sowa (1963) investigated the change in the c_u from ideal to perfect samples. In their study, the ideal sample was brought to by the K_0 consolidation in a triaxial cell, and the perfect sample was obtained by releasing the principal stress difference after the K_0 consolidation. Their perfect samples were under all round pressure equal to the cell pressure, which was based on a concept that the change in all round pressure had no effect on the c_u of saturated cohesive soils. They reported that the strength reduction from the ideal to perfect samples of the Weald clay ($I_p=24$) was as small as 1.5%.

Noorany and Seed (1965) performed a similar type of study, however, they investigated the strength of the perfect sample

by completely releasing the confining pressure after the K_0 consolidation. Strength reduction observed on the San Francisco Bay mud ($I_p=45$) was reported to be about 6%.

The main interest in the study by Ladd and Lambe (1963) was the change in the c_u due to mechanical disturbance. They measured the residual effective stress in the perfect sample, $\bar{\sigma}_p$, and that in the "undisturbed" sample, $\bar{\sigma}_s$. They proposed to use the ratio $\bar{\sigma}_p/\bar{\sigma}_s$ for correcting the measured c_u value of the "undisturbed" sample in a similar concept to the overconsolidation ratio of rebounded soils.

Okumura (1971 b, 1974) studied the degree of disturbance by examining results of various types of shear tests on clays, and found that the reduction in the c_u due to mechanical disturbance is uniquely related to the disturbance ratio $R=\bar{\sigma}_p/\bar{\sigma}_s$. He suggested a possibility of correcting the measured c_u value for disturbance based on the relationship between the strength reduction ratio, the disturbance ratio and the ratio of the secant modulus E_{50} to the in situ effective overburden pressure. It has been reported that a reduction in the E_{50} due to sample disturbance is far larger than that in the c_u (Nakase et al, 1972).

These previous studies clearly show that the strength reduction from the perfect sample to the "undisturbed" sample is much larger than that from the ideal to perfect samples. Much importance, therefore, has been set on the correction of the c_u for the mechanical disturbance. In practice the design value of the c_u is often determined from plots of the c_u against depth (Nakase, 1967). This type of determination does not seem satisfactory, since each set of plots has been already influenced by incidental factors such as technician's skill and variety of micelleneous conditions. So it would be necessary to make an appropriate correction of the measured value of c_u before the determination of the design value, for increasing the reliability of the data for geotechnical designing, in particular in the probabilistic method for designing (Matsuo and Asaoka,

1977).

At present the most reliable method for correcting the c_u for sample disturbance would be the method based on the disturbance ratio, since it reflects the quality of each soil sample. This method, however, requires sophisticated testing techniques for measuring the residual pore pressure and it does not seem to be readily employed in engineering practice. For this reason, the concept of the SHANSEP (Ladd and Foott, 1974) seems advantageous for not requiring the measurement of residual pore pressure. The SHANSEP method, on the other hand, requires another kind of special testing and the case where the use of this method is justified would be limited. Okumura's method based on the E_{50} value does not require the measurement of residual pore pressure, however, his investigation has been limited to two kinds of clays. So if a different kind of clay is to be treated, one has to perform a similar sort of sophisticated testing as had been done by Okumura.

The authors' study aims at proposing a comprehensive method for correcting the c_u value, obtained by either unconfined compression tests or unconsolidated undrained triaxial compression tests, for sample disturbance based on soil parameters readily obtained from the result of routine soil testing for wide range of soil type from sandy clays to highly plastic clays. The main interest in the present study is to develop the correction method applicable for the intermediate soils (Nakase and Kamei, 1982), soil properties of which are between those of sands and clays. It has been known that the degree of disturbance increases as the plasticity of cohesive soils decreases (Berre and Bjerrum, 1973; Nakase et al, 1978).

It should be noted here that the authors' study is confined to the vertical sample of normally consolidated young cohesive soils. And a possible effect of decrease in saturation degree on stress release (Okumura, 1974) is not taken into account for simplicity.

SOIL SAMPLES

Soil samples used in this study were prepared by mixing the Kawasaki clay with the crushed Toyoura sand. Ratio of mixing was determined so as to obtain different values of plasticity index I_p and also smooth grading curves. Index properties of thus mixed soil samples are shown in Table 1, and the grading curves in Fig. 1. The soil samples were termed M-51, M-18 and M-10 after their I_p values. By using this type of soil samples, it would be possible to look into the general tendency of degree of disturbance without any special consideration on difference in mineralogical effect.

Test specimens were prepared through a process of preliminary consolidation. Small amount of soil slurry was put in a consolidation mould of inner diameter of 200 mm and 180 mm high. This small amount of the slurry was consolidated one-dimensionally under a pressure of 30 kN/m², then another small amount of the slurry was added in the mould and consolidated again under the same pressure of 30 kN/m². This process was repeated three to four times. One step of this preliminary consolidation took one

Table 1. Index properties of soil samples

	M-51	M-18	M-10
G_s	2.69	2.67	2.67
w_L (%)	84.5	36.8	28.0
w_p (%)	33.4	18.5	18.0
I_p	51.1	18.3	10.0
Sand (%)	4.1	55.2	66.2
Silt (%)	47.4	31.6	24.3
Clay (%)	48.5	13.2	9.5

Fig. 1. Grain size distribution curves of samples

day for each small amount of the slurry.

After sufficient amount of soil sample was consolidated, the consolidation pressure was increased to 50 kN/m² for the M-51 and 70 kN/m² for the M-18 and M-10, and the sample was consolidated for one week. After the consolidation under this final pressure was completed, the soil block was taken out and split vertically into five pieces. Each piece of soil sample was coated with resin mixed paraffin and tested few days later.

Another set of soil samples was prepared for obtaining the condition of the "undisturbed" sample. Slurry of the soil sample was consolidated one-dimensionally in a similar manner to the above mentioned preparation. Final consolidation pressure was 200 kN/m² in this particular case, and consolidated for one week under this pressure. After the consolidation was completed, the soil block was not taken out, but a model sampling was made by using stainless steel tubes of inner diameter of 50 mm, thickness of 2 mm and 150 mm long. Four tubes were pushed simultaneously into the soil block with a bellows cylinder, on a square pattern with centre to centre distance of 110 mm. Thus obtained "undisturbed" samples were trimmed and unconsolidated undrained triaxial compression tests were performed immediately.

COMPRESSION TESTS

Two series of K_0 consolidated undrained triaxial compression tests were performed for obtaining the conditions of the ideal and perfect samples respectively. Testing procedures are shown in Fig. 2. In the K_0 consolidation process, the lateral pressure was maintained at 140 kN/m² and the axial pressure was adjusted for obtaining the K_0 condition by the use of the Bishop type lateral strain gauge (Bishop and Henkel, 1962). Back pressure of 200 kN/m² was applied throughout the test, hence the total cell pressure was 340 kN/m². Throughout the test, pore pressure was measured at the lower end of the specimen.

Fig. 2. Testing procedures

In the test for the ideal sample, I-test, the axial pressure was increased from the final axial consolidation pressure as shown in Fig. 2. After observing the peak strength, the principal stress difference was released and the specimen was brought to the all round cell pressure, then the axial pressure was increased again. Three cycles of loading and unloading were made on the M-51 and M-18. In the case of the M-10, however, two cycles were made because of considerably large axial strains in the first and second cycles. The purpose of the compression after the second cycle was to give further disturbance to the sample.

In the test for the perfect sample, P-test, the axial consolidation pressure was decreased to the lateral consolidation pressure at the end of the K_0 consolidation. Then the loading tests were performed as shown in Fig. 2. In both the I-test and P-test, the loading and unloading were performed at the rate of axial strain of 0.08%/min.

In the model sampling test, S-test, the unconsolidated undrained triaxial compression tests were performed. The cell pressure in

this series of tests was 500 kN/m^2 , which was increased in five steps with an increment of 100 kN/m^2 . In each step the pore pressure was measured, and the initial pore pressure, i.e. residual pore pressure, was estimated from the relationship between the pore pressures and the cell pressures. It took about 20 minutes for the pore pressure to become steady. Pore pressure response at the final increment of the cell pressure showed that the Skempton's pore pressure coefficient B (Skempton, 1954) was in the range from 0.90 to 0.95. The compression tests were performed at the rate of axial strain of $1\%/min$.

TEST RESULTS

Residual Effective Stress in Perfect Samples

Total principal stresses acting on the ideal, perfect and "undisturbed" samples are schematically shown in Fig. 3. The residual effective stress $\bar{\sigma}_p$ in the perfect sample is given by the expression

$$\bar{\sigma}_p = \{K_0 + A_p(1 - K_0)\} \bar{\sigma}_{1c} \quad (1)$$

where K_0 is the coefficient of earth pressure at rest and equal to $(\sigma_3 - u_0)/(\sigma_1 - u_0)$, $\bar{\sigma}_{1c}$ is the effective vertical stress before the stress release and A_p is the pore pressure coefficient corresponding to the stress release (Ladd and Lambe, 1963).

A_p values obtained in the test are shown in Fig. 4, where data from other sources are also shown. It is clearly shown in the figure that the A_p value decreases with decrease in the I_p , and it becomes negative for the M-18 and M-10. The negative value of A_p means that the pore pressure increases

Fig. 3. Total principal stresses acting on soil samples

Fig. 4. A_p and I_p (Numerals in parentheses refer to reference number)

Table 2. Results of the P-Test

	A_p	K_0	$\bar{\sigma}_p/\bar{\sigma}_{1c}$	$\bar{\sigma}_p/\bar{\sigma}_{mc}^*$
M-51	0.14 ± 0.02	0.45 ± 0.02	0.53	0.83
M-18	-0.05 ± 0.02	0.39 ± 0.01	0.36	0.60
M-10	-0.12 ± 0.01	0.40 ± 0.01	0.33	0.54

$$^* \bar{\sigma}_{mc} = (1 + 2K_0)\bar{\sigma}_{1c}/3$$

when the shear stresses are released. A similar observation has been reported on cohesive soil of $I_p=10$ (Sangrey et al, 1969). This result gives rise to the fact that the decrease in effective stress on stress release, hence the degree of disturbance, is greater for soils of lower value of the I_p (Nakase et al, 1978).

Range of values of A_p and K_0 observed in the P-test is shown in Table 2. Degree of reduction in effective stress from the ideal sample to perfect sample is also shown in the table. Average values of $\bar{\sigma}_p/\bar{\sigma}_{1c}$ in the table is within the range of those suggested by Ladd and Lambe (1963) for clayey silt, lean clay and plastic clay.

Stress Paths

Stress paths obtained in the I-test and P-test are shown in Fig. 5. It will be seen in the figure that the stress paths of loading process after the second cycle loading in the I-test are not different from those of the P-test. This would imply that the cyclic loading caused an identical effect to both the ideal and perfect samples. All round effective stress at start of compression

Fig. 5. Stress paths of I-test and P-test

of the perfect sample corresponds to $\bar{\sigma}_p$. For the loading after the second cycle, the effective stresses at start of loading are the residual effective stresses $\bar{\sigma}_s$.

Normalized stress paths of the S-test are compared with those of the P-test in Fig. 6, where the stresses are divided by the major effective principal stress of corresponding ideal sample $\bar{\sigma}_{1c}$. Residual effective stress $\bar{\sigma}_p$ at the perfect sampling can not be obtained directly for the S-test samples, then

Fig. 6. Stress paths of P-test and S-test

the corresponding value of $\bar{\sigma}_p$ was worked out with an average value of A_p obtained in the P-test and also an average value of K_0 in the I-test and P-test.

It will be seen in Fig. 6 that the stress path of the S-test sample is not much

different from that of the P-test sample having a similar value of the disturbance ratio $R = \bar{\sigma}_p / \bar{\sigma}_s$. This result would imply that the sample disturbance given to the S-test sample is similar to that caused by cyclic loading for the samples of the I-test and P-test.

Strain in Cyclic Loadings

Fig. 7 shows the relationship between an axial strain ϵ_p at start of loading in the cyclic loading test and corresponding disturbance ratio. All the data of the P-test, S-test and the data of loading after the second cycle in the I-test are plotted in the figure. The plots at $R=1$ correspond to the perfect sample, and a negative value of the axial strain indicates a small amount of axial elongation at the release of shear stresses.

Fig. 7 shows that the disturbance ratio corresponding to a certain value of the axial strain ϵ_p is larger for soils with lower value of I_p . This result indicates that when a certain amount of axial compression is given to soil sample, the resulting reduction in the effective stress is greater for soils of lower plasticity.

Fig. 7. Residual axial strain and disturbance ratio

EVALUATION OF DISTURBANCE RATIO

Initial states of effective stress and the undrained shear strength of the ideal, perfect and "undisturbed" samples are schematically shown in Fig. 8. The ratio of initial effective stress in the perfect sample to that of the "undisturbed" sample $\bar{\sigma}_p / \bar{\sigma}_s$ is the disturbance ratio R . A reduction in the c_u due to sample disturbance has been known to be uniquely related to the R (Okumura, 1971, 1974).

For obtaining the R for a particular soil sample, it is necessary to assess the corresponding $\bar{\sigma}_p$ value and also to measure the residual effective stress $\bar{\sigma}_s$ before the shear, hence the initial pore pressure. Since the measurement of this initial pore pressure in soil sample requires sophisticated techniques, it would be desirable to evaluate the disturbance ratio without measuring the residual pore pressure.

Okumura (1974) has also found that there is a unique relationship between the R and a ratio $E_{50} / \bar{\sigma}_{1c}$ for a particular soil, where E_{50} is the secant modulus and $\bar{\sigma}_{1c}$ is the major effective principal stress in consolidation process of the perfect sample. Okumura has reported this unique R vs. $E_{50} / \bar{\sigma}_{1c}$ relationship for the Boston Blue clay and the Honmoku clay.

Relationship between R and $E_{50} / \bar{\sigma}_{1c}$ obtained from the I-test, P-test and S-test in the present study is shown in Fig. 9.

Fig. 8. Initial effective stress and undrained shear strength

Fig. 9. $E_{50}/\bar{\sigma}_{1c}$ and disturbance ratio

$E_{50}/\bar{\sigma}_{1c}$ value at $R=1$ corresponds to that of the perfect sample. In the case of the I-test, after the second cycle loading, the R value was worked out by using an average value of pore pressure coefficient A_p measured in the P-test. For the S-test, the R value was obtained by the use of the A_p value from the P-test and also an average value of K_0 in the I-test and P-test.

As shown in Fig. 9, relationship between $E_{50}/\bar{\sigma}_{1c}$ and R for each of the three soil samples could be expressed in a form

$$\log(E_{50}/\bar{\sigma}_{1c}) = a - b \log R \quad (2)$$

where a and b are constants depending on the I_p value of the soil. Since 10^a is equal to the $(E_{50}/\bar{\sigma}_{1c})_p$ value of the perfect sample, Eq. (2) becomes

$$\log(E_{50}/\bar{\sigma}_{1c})_s = \log(E_{50}/\bar{\sigma}_{1c})_p + \log R^{-b} \quad (3)$$

where $(E_{50}/\bar{\sigma}_{1c})_p$ and $(E_{50}/\bar{\sigma}_{1c})_s$ are corresponding to the perfect sample and the "undisturbed" sample respectively. Rewriting the above expression, the disturbance ratio R is given

$$R = [(E_{50}/\bar{\sigma}_{1c})_p / (E_{50}/\bar{\sigma}_{1c})_s]^{1/b} \quad (4)$$

In normally consolidated cohesive soil strata, $\bar{\sigma}_{1c}$ may be taken equal to the in situ effective overburden pressure, however, $(E_{50}/\bar{\sigma}_{1c})_p$ can not be readily known from the results of routine soil tests. $(E_{50}/\bar{\sigma}_{1c})_p$ values

Fig. 10. $E_{50}/\bar{\sigma}_{1c}$ of perfect samples and I_p

obtained in the present study are plotted against the I_p of the soil sample in Fig. 10. There are some scatter in the plot, however, the upper bound of these plots seems to be representative relationship in considering the nature of the $(E_{50}/\bar{\sigma}_{1c})_p$ value. This upper bound may be approximated by a hyperbolic form of

$$(E_{50}/\bar{\sigma}_{1c})_p = \left[\frac{41}{(0.019 I_p + 1.017)^2 - 1} + 91 \right] \quad (5)$$

Combining Eqs. (4) and (5) we have

$$R = \left[\left\{ \frac{41}{(0.019 I_p + 1.017)^2 - 1} + 91 \right\} / (E_{50}/\bar{\sigma}_{1c})_s \right]^{1/b} \quad (6)$$

Eq. (6) implies that it would be possible to know the disturbance ratio R of a particular soil sample from the I_p , E_{50} and the in situ effective overburden pressure, without measuring the residual pore pressure of the soil sample.

METHOD OF CORRECTING MEASURED C_u VALUE FOR SAMPLE DISTURBANCE

Ladd and Lambe (1963) suggested to correct the measured c_u value for sample disturbance by assuming that the disturbance ratio was equivalent to the overconsolidation ratio OCR. It will be tried here to correlate the disturbance ratio and the OCR for the purpose of finding out an appropriate method for correcting the c_u for sample disturbance.

Fig. 11. Key sketch to reduction in c_u due to rebound

Referring to Fig. 11, a ratio of the c_u after rebound, $c_{u,n}$, to that before the rebound, $c_{u,1}$ has been given (Nakase et al, 1979), by the expressions

$$\left. \begin{aligned} c_{u,n}/c_{u,1} &= (1-r)n^{-\lambda} + rn^{-\mu} \\ r &= \sin \phi_e / \sin \phi' \\ \lambda &= C_s/C_c, \quad \mu = C_f/C_c \end{aligned} \right\} \quad (7)$$

where ϕ_e is the Hvorslev's effective friction angle (Hvorslev, 1960), and ϕ' is the angle of shear resistance. C_f is the secant modulus in the e -log σ_f curve, where σ_f is the effective stress at failure, and is equivalent to the swelling index C_s in the compression curve. It has been also shown that $r \doteq 1$ for soils of $I_p < 25$, and $\lambda \doteq \mu$ for the case of $n < 20$, which gives the expression of $c_{u,n}/c_{u,1} = n^{-\lambda}$ or $(c_u/\sigma)_n = (c_u/\sigma)_1^{1-\lambda}$ as has been previously reported (Mitachi and Kitago, 1976; Ladd et al, 1977).

Let it be assumed here that

(a) the perfect sample is in a normally consolidated state, with a linear e -log σ re-

Fig. 12. Effective stresses before shear

lationship and a constant value of (c_u/σ) .

(b) the c_u value is uniquely defined by a state point in the e -log σ plane, i.e. referring to Fig. 12, the c_u of a soil sample which is subjected to sample disturbance from the points A to C is equal to the c_u value of soil sample which is rebounded from the points B to C.

The assumption (b) has been proposed implicitly (Ladd and Lambe, 1963; Okumura, 1974), however, no data have been presented for verifying it. Comparing the limited data of the rebound test on an intact marine clay (Nakase et al, 1969) and the data of the present test, the c_u of the rebounded soil, from B to C in Fig. 12, seems to be larger than that of the disturbed soil, from A to C, by about 10%. This implies that the assumption (b) would have an error of about 10% in conservative side in correcting the c_u for sample disturbance.

In considering the rebound process from the points B to C, in Fig. 12, the state point C is corresponding to the OCR of $n = \bar{\sigma}_B/\bar{\sigma}_C$. If the sample disturbance from A to C is considered, on the other hand, the state point C is corresponding to the disturbance ratio of $R = \bar{\sigma}_A/\bar{\sigma}_C$. A relationship of $C_c \log(\bar{\sigma}_B/\bar{\sigma}_A) = C_s \log(\bar{\sigma}_B/\bar{\sigma}_C)$ leads to

$$\bar{\sigma}_B/\bar{\sigma}_A = R^{\lambda/(1-\lambda)} \quad (8)$$

then the relationship between the overconsolidation ratio n and the disturbance ratio R is given by

$$n = (\bar{\sigma}_B/\bar{\sigma}_C) = (\bar{\sigma}_A/\bar{\sigma}_C)(\bar{\sigma}_B/\bar{\sigma}_A) = R^{1/(1-\lambda)} \quad (9)$$

Degree of the strength reduction due to sample disturbance is expressed

$$c_{uC}/c_{uA} = (c_{uC}/c_{uB}) \cdot (c_{uB}/c_{uA}) \quad (10)$$

The assumption (a) leads to $c_{uB}/c_{uA} = \bar{\sigma}_B/\bar{\sigma}_A$, and the strength ratio c_{uC}/c_{uB} through the rebound is given by Eq. (7), then we have

$$c_{uC}/c_{uA} = \{(1-r)R^{-\lambda} + rn^{-\mu}\} R^{\lambda/(1-\lambda)} \quad (11)$$

By substituting the relationship $n=R^{1/(1-\lambda)}$, we get

$$c_{uC}/c_{uA} = \{(1-r)R^{-\lambda/(1-\lambda)} + rR^{-\mu/(1-\lambda)}\} R^{\lambda/(1-\lambda)} \\ = (1-r) + rR^{(\lambda-\mu)/(1-\lambda)} \quad (12)$$

Values of the parameter $r = \sin \phi_e / \sin \phi'$ for reconstituted soils of I_p of 55, 30, 18 and 12 were found to be 0.85, 0.87, 0.99 and 0.98 respectively (Ohneda, 1977). If an approximation of $r=1$ is employed in Eq. (7), the c_u of the rebounded soil of I_p of more than 30 is to be underestimated by 4% at maximum for the range of OCR of up to 40. This implies that the use of this approximation gives an error in dangerous side when introduced to the correction for sample disturbance. Considering an extent of possible errors, coupling of the approximation of $r=1$ and the assumption (b) would have an error of a few per cents in conservative side in correcting for sample disturbance.

When an approximation of $r=1$ is introduced, the above expression becomes

$$c_{uC}/c_{uA} = c_{us}/c_{up} = R^{(\lambda-\mu)/(1-\lambda)} \quad (13)$$

where c_{up} and c_{us} are the c_u of the perfect and "undisturbed" samples respectively.

Ladd et al (1977) have shown a general expression of the ratio of (c_u/σ) of the over-consolidated soils (O.C.) to normally consolidated one (N.C.) as

$$(c_u/\sigma)_{O.C.}/(c_u/\sigma)_{N.C.} = OCR^m \quad (14)$$

They suggested that a typical value of m was 0.80 for soils of I_p in the range of 21-75. If the relationship between (c_u/σ) values of the perfect and "undisturbed" samples and the R is assumed to be expressed in an analogous form to Eq. (14), i.e.

$$(c_u/\sigma)_s/(c_u/\sigma)_p = R^m \quad (15)$$

we get a relationship of

$$c_{us}/c_{up} = R^{m-1} \quad (16)$$

Comparing Eqs. (13) and (16), we get a relationship of $m = (1-\mu)/(1-\lambda)$. As shown in Fig. 13, the result of the present test seems to support a relevance of Eq. (15).

For the purpose of correcting the measured value of c_u , i.e. c_{us} , for sample disturbance, the correction factor should be in a form of c_{up}/c_{us} , then we have

$$c_{up}/c_{us} = R^{1-m} \quad (17)$$

Substituting Eq. (4) into the above expression, we get the correction factor in the form of

$$c_{up}/c_{us} = [(E_{50}/\bar{\sigma}_{1c})_p / (E_{50}/\bar{\sigma}_{1c})_s]^{(1-m)/b} \quad (18)$$

As shown in Table 3, a parameter $(1-m)/b$ does not seem to depend much on the value of I_p of soils. Then if an average value of $(1-m)/b = 0.085$ is taken, and also the expression of $(E_{50}/\bar{\sigma}_{1c})_p$ by Eq. (5) is

Fig. 13. $(c_u/\sigma)_s/(c_u/\sigma)_p$ and disturbance ratio

Table 3. Values of m and b

	m^*	b^{**}	$(1-m)/b$
M-51	0.869	1.495	0.088
M-18	0.873	1.411	0.090
M-10	0.900	1.281	0.078

* from Fig. 13.

** from Fig. 9.

introduced, we obtain the expression

$$c_{up}/c_{us} = \left[\left\{ \frac{41}{(0.019I_p + 1.017)^2 - 1} + 91 \right\} / (E_{50}/\bar{\sigma}_{1c})_s \right]^{0.085} \quad (19)$$

DISCUSSIONS

The correction factor to be used for estimating the c_u of the perfect sample based on the measured values of c_u , I_p , E_{50} and in situ effective overburden pressure is proposed in the form of Eq. (18). Eq. (19) is derived from the result of the present study. The result of the use of Eq. (19) is shown in Fig. 14, where c_{us}/c_{up} values observed in the P-test are compared with those calculated by Eq. (19). This result shows that, as far as the result of the P-test is concerned, an accuracy in estimation is in the range of $\pm 10\%$.

The correction factor of Eq. (18) depends on the values of $(E_{50}/\bar{\sigma}_{1c})_p$ and $(1-m)/b$. As far as the authors are aware, three values of $(E_{50}/\bar{\sigma}_{1c})_p$ have been reported, i.e. 100 for the Honmoku clay of $I_p=51$ and 268 for the Boston Blue clay of $I_p=20$ (Okumura, 1974), and 150 for the Kawasaki clay of $I_p=34$ (Ladd, 1964). The values of Japanese clays, Honmoku and Kawasaki clays, are comparable with those given by Eq. (5), however, 268 of the Boston Blue clay is two times that of Eq. (5). If $(E_{50}/\bar{\sigma}_{1c})_p$ value becomes two times larger, the correction factor of Eq. (18) increases by 6%. Values of $(1-m)/b$ in Table 3 are in the range from 0.078 to 0.090, which corresponds to a change in the correction factor of about $\pm 1.5\%$.

It is known that the E_{50} is much more sensitive to the sample disturbance than the c_u value itself (Nakase et al, 1972). Eq. (19)

Fig. 14. Comparison of measured and calculated values of c_{us}/c_{up}

indicates that a 10% reduction in the c_u is associated with a 70% reduction in the E_{50} , and a 20% of the c_u reduction with a 93% reduction in the E_{50} . Empirical relationship between the c_u and E_{50} for the perfect sample of clays has been given as $E_{50} = (275 \sim 500)c_u$ (Ladd, 1964). For a range of I_p from 30 to 50, the $(E_{50}/\bar{\sigma}_{1c})_p$ values by Eq. (5) are about 110, and if the (c_u/σ) of 0.25~0.40 are assumed, $E_{50} = (275 \sim 440)c_u$ is obtained.

Fig. 15 shows a form of correction chart to be used in practice. It should be noted again that the correction factor of Eq. (19) is based solely on results of the present study. The above mentioned difference in the $(E_{50}/\bar{\sigma}_{1c})_p$ vs. I_p relationship between Japanese clays and the Boston Blue clay may suggest a possibility of dependence of this relationship on the clay type. If further information on the $(E_{50}/\bar{\sigma}_{1c})_p$ vs. I_p relationship is collected, a more comprehensive set of the correction chart will be obtained.

Measured c_u values are to increase by applying the correction for sample disturbance. This does not necessarily imply a simple increase in the safety factor in stability analysis. The c_u is influenced by three major factors of sample disturbance, strength anisotropy and strain rate effect. The cor-

Fig. 15. Correction factor to sample disturbance

rection for sample disturbance, therefore, is to be considered to increase a reliability of the data for applying the correction for the other two factors.

CONCLUSIONS

The following conclusions are obtained from the present study.

1) The pore pressure coefficient corresponding to shear stress release can be negative for soils of low plasticity index, which implies an increase in pore pressure on shear stress release.

2) Comparison of the effective stress paths indicates that the effect of sample disturbance is similar to the effect caused by cyclic loading on the ideal and perfect samples.

3) When a certain amount of axial com-

pression is given to soil sample, the resulting disturbance is greater for soils of lower plasticity index.

4) The disturbance ratio, which is a ratio of the residual effective stresses in the perfect and "undisturbed" samples, can be evaluated without measuring the residual pore pressure of soil samples.

5) A comprehensive way of correcting the measured c_u value for sample disturbance is proposed, which is based on soil parameters readily obtained from routine soil testings, and is applicable to soils of wide range from plastic clays to intermediate soils.

NOTATIONS

A_p = pore pressure coefficient at shear stress release

$c_{u.1} = c_u$ of clay with OCR of $n=1$

$c_{u.n} = c_u$ of clay with OCR of n

C_f = secant modulus in void ratio vs. stress at failure

E_{50} = secant modulus of stress-strain curve in compression test

n = overconsolidation ratio (OCR)

R = disturbance ratio = $\bar{\sigma}_p / \bar{\sigma}_s$

$r = \sin \phi_e / \sin \phi'$

u_o = residual pore pressure in perfect sample

ϵ_p = residual axial strain in cyclic loading test

$\lambda = C_s / C_c$

$\mu = C_f / C_c$

$\bar{\sigma}_{1c}$ = effective vertical consolidation pressure

$\bar{\sigma}_p$ = residual effective stress in perfect sample

$\bar{\sigma}_s$ = residual effective stress in "undisturbed" sample

ϕ_e = Hvorslev's effective friction angle

REFERENCES

- 1) Berre, T. and Bjerrum, L. (1973): "Shear strength of normally consolidated clays," Proc. 8th Int. Conf. SMFE, Vol. 1, pp. 39-49.
- 2) Bishop, A. W. and Henkel, D. J. (1953): "Pore pressure changes during shear in two undisturbed clays," Proc. 3rd Int. Conf. SMFE, Vol. 2, pp. 94-99.
- 3) Bishop, A. W. and Henkel, D. J. (1962): "The measurements of soil properties in the triaxial test," 2nd ed. London, Edward Arnold, 228 p.
- 4) Hvorslev, M. J. (1960): "Physical components of the shear strength of saturated clays," Proc.

- Research Conf. Shear Strength of Cohesive Soils, ASCE, pp.169-273.
- 5) Ladd, C. C. and Lambe, T. W. (1963): "The strength of 'undisturbed' clay determined from undrained tests," ASTM, STP 361, pp. 342-371.
 - 6) Ladd, C. C. (1964): "Stress-strain modulus of clay in undrained shear," Proc. ASCE, Vol. 90, No. SM 5, pp. 103-132.
 - 7) Ladd, C. C. and Foott, R. (1974): "New design procedure for stability of soft clays," Proc. ASCE, Vol. 100, No. GT 7, pp. 763-786.
 - 8) Ladd, C. C., Foott, R., Ishihara, K., Schlosser, F. and Poulos, H. G. (1977): "Stress-deformation and strength characteristics - State-of-the-art Report," Proc. 8th Int. Conf. SMFE, Vol. 2, pp. 421-494.
 - 9) Matsuo, M. and Asaoka, A. (1977): "Probability models of undrained strength of marine clay layer," Soils and Foundations, Vol. 17, No. 3, pp. 53-63.
 - 10) Mitachi, T. and Kitago, S. (1976): "Change in undrained shear strength of saturated remoulded clay due to swelling," Soils and Foundations, Vol. 16, No. 1, pp. 45-58.
 - 11) Nakase, A. (1967): "The $\phi=0$ analysis of stability and unconfined compression strength," Soils and Foundations, Vol. 7, No. 2, pp. 35-50.
 - 12) Nakase, A., Kobayashi, M. and Katsuno, M. (1969): "Change in undrained shear strength of saturated clays through consolidation and rebound," Report of Port and Harbour Research Institute, Vol. 8, No. 4, pp. 103-143 (in Japanese).
 - 13) Nakase, A., Kobayashi, M. and Kanechika, A. (1972): "Undrained shear strength and deformation modulus of clays," Rept. of Port and Harbour Research Institute, Vol. 11, No. 2, pp. 243-259 (in Japanese).
 - 14) Nakase, A., Kusakabe, O. and Nomura, H. (1978): "On the correlation between disturbance of cohesive soil and the grading," Proc. 1978 Sampling Symposium, Japanese Soc. SMFE, pp. 99-104 (in Japanese).
 - 15) Nakase, A., Kusakabe, O. and Ohneda, H. (1979): "Prediction of decrease in c_u due to rebound," Proc. 6th Asian Regional Conf. SMFE, Vol. 1, pp. 47-50.
 - 16) Nakase, A. and Kamei, T. (1983): "Undrained shear strength anisotropy of normally consolidated cohesive soils," Soils and Foundations, Vol. 23, No. 1, pp. 91-101.
 - 17) Noorany, I. and Seed, H. B. (1965): "In situ strength characteristics of soft clays," Proc. ASCE, Vol. 91, No. SM 2, pp. 49-80.
 - 18) Ohneda, H. (1977): "Effect of gradings on shear strength of cohesive soils," M. Engineering Thesis, Tokyo Institute of Technology (in Japanese).
 - 19) Okumura, T. (1969): "Studies on the disturbance of clay samples (1st Report)-Remolding tests by the repeated triaxial apparatus-," Rept. of Port and Harbour Research Institute, Vol. 8, No. 1, pp. 59-84 (in Japanese).
 - 20) Okumura, T., Umeda, H. and Narita, M. (1971 a): "Studies on the disturbance of clay samples (3rd Report)-Repeated loading tests and consolidation with a simple shear apparatus-," Rept. of Port and Harbour Research Institute, Vol. 10, No. 1, pp. 77-106 (in Japanese).
 - 21) Okumura, T. (1971 b): "The variation of mechanical properties of clay samples depending on its degree of disturbance," Proc. Special Session on Quality in Soil Sampling, 4th Asian Regional Conf. SMFE, pp. 73-81.
 - 22) Okumura, T. (1974): "Studies on the disturbance of clay soils and the improvement of their sampling techniques," Technical Note of Port and Harbour Research Institute, No. 193, 145 p. (in Japanese).
 - 23) Sangrey, D. A., Henkel, D. J. and Esrig, M. I. (1969): "The effective stress response of a saturated clay soil to repeated loading," Canadian Geotechnical Journal, Vol. 16, No. 3, pp. 241-252.
 - 24) Seed, H. B., Smith, I. and Noorany, I. (1964): "Effects of sampling and disturbance on the strength of soft clays," Rept. No. TE-14-1, Institute of Engineering Research, Univ. of California, Berkeley, 149 p.
 - 25) Skempton, A. W. (1954): "The pore-pressure coefficients A and B," Geotechnique, Vol. 4, No. 4, pp. 143-147.
 - 26) Skempton, A. W. and Sowa, V. A. (1963): "The behaviour of saturated clays during sampling and testing," Geotechnique, Vol. 13, No. 4, pp. 269-290.