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CHANGE IN UNDRAINED SHEAR STRENGTH CHARACTERISTICS OF SATURATED REMOLDED CLAY DUE TO SWELLING

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ABSTRACT

Change in undrained shear strengths s_u due to swelling under K_0 and isotropic stress conditions were investigated for three saturated remolded clays. Test results indicate that the relationship between the ratio s_u/p, where p is the vertical effective stress at the end of swelling, and the overconsolidation ratio n is represented by a straight line in logarithmic plot. Moreover, the slope of this line is almost the same for both K_0 and isotropic stress conditions, and its value is closely approximate to that of $(1-C_s/C_c)$, where C_c and C_s are compression and swelling indices, respectively.

Based on the test results, the authors proposed a simple method of estimating the in situ undrained shear strength s_u for overconsolidated clays from a series of conventional laboratory test.

Furthermore, the coefficient of earth pressure at rest K_0 and the pore pressure coefficient A at failure were discussed in relation to overconsolidation ratio n.

Key words: at rest pressure, cohesive soil, consolidated undrained shear, overconsolidation, pore pressure, shear strength, triaxial compression test

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INTRODUCTION

Excavations in saturated clay deposits lead to gradual decrease in shearing resistance of the clay due to swelling. So-called "long term" type problems correspond to such cases and effective strength parameters c' and ϕ' have been proposed to apply to this type of problems (Bishop and Bjerrum, 1960). In such cases, the failure of the clay layer is assumed to occur at a sufficiently slow rate so that little or no excess pore pressures are developed, and the pore pressures used to calculate the effective normal stresses acting on the potential failure surface are those corresponding to the ground water table or steady state seepage.

However, there is no evidence that the failure of the clay layer occurs at a sufficiently slow rate as in a drained test. Moreover, it is very difficult, in general, to measure accurately small values of cohesion intercept c' (Ladd, 1971). Therefore, taking the position that the stability problems should always be treated conservatively, it might be more practical to apply the $\phi_u=0$ analysis to the long term type problem, based on the

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estimation of the rate of decrease in undrained shear strength s_u in relation to overconsolidation ratio n.

In this paper, a new test procedure to estimate the rate of decrease in s_u due to swelling is investigated and results of a series of triaxial compression test on normally and overconsolidated clay are presented.

s_u/p IN OVERCONSOLIDATED CLAY

In order to find the relationship between the ratio s_u/p in overconsolidated clay and that in normally consolidated clay, assumptions are made as follows (refer to Fig. 1), wherein p is preshear effective stress.

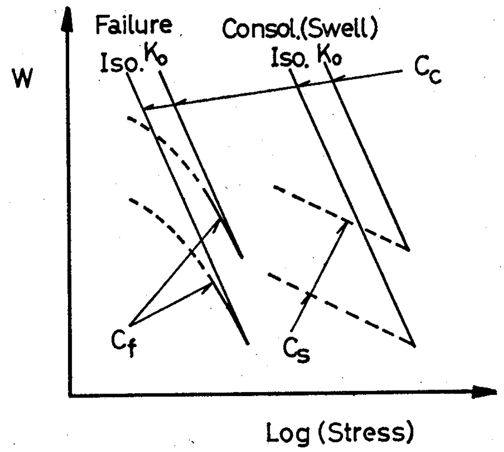

Fig. 1. Assumptions about water content vs. effective stresses

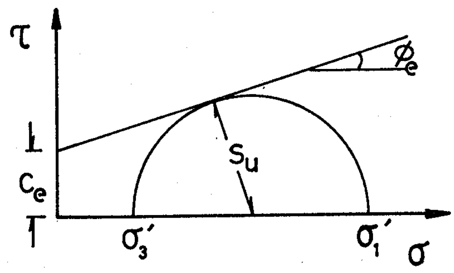

Fig. 2. Hvorslev failure criterion

(1) In normal consolidation state, the relation between water content w after consolidation and effective vertical consolidation pressure p_c is represented by a straight line in semi-logarithmic plot. And the slope of this line, i.e. compression index C_c, is a constant for a given clay, irrespective of the stress systems during consolidation.

(2) In overconsolidation state, the relation between water content w after swelling and effective vertical swelling pressure p_s is also represented approximately by a straight line in semi-logarithmic plot. And the inclination of this line, i.e. swelling index C_s, is a constant for a given clay, irrespective of the stress systems in swelling process.

(3) In normally consolidated clay, w versus $\log p_f$ line is parallel to the w versus $\log p_c$ line mentioned above, where p_f is $(\sigma_1'+\sigma_3')/2$, and σ_1' and σ_3' are the major and minor principal effective stresses at failure, respectively.

(4) In overconsolidated clay, w versus $\log p_f$ relationship is approximated by a straight line within the range of $n \leqq 10$. In this paper, a symbol C_f is applied to the slope of this line.

Based on these assumptions, relationship between s_{u1}/p_1 versus s_{un}/p_n can be derived as follows, where subscripts 1 and n represent the state of normal and overconsolidation, respectively.

According to Hvorslev failure criterion, we get following equation refering to Mohr's stress circle in Fig. 2.

$$s_u = \frac{\sigma_1 - \sigma_3}{2} = c_e \cos \phi_e + \frac{\sigma_1' + \sigma_3'}{2} \sin \phi_e$$

or

$$s_u=\kappa\cdot\sigma_e'\cos\phi_e+p_f\sin\phi_e \tag{1}$$

where, c_e and ϕ_e are effective cohesion and effective angle of internal friction, respectively. κ is the coefficient of cohesion, and σ_e' is the equivalent consolidation pressure. In this paper, effective vertical stress is adopted as equivalent consolidation pressure. Idealized relationships for w versus $\log p$ and w versus $\log p_f$ are illustrated in Fig. 3. From this figure, following equations are obtained.

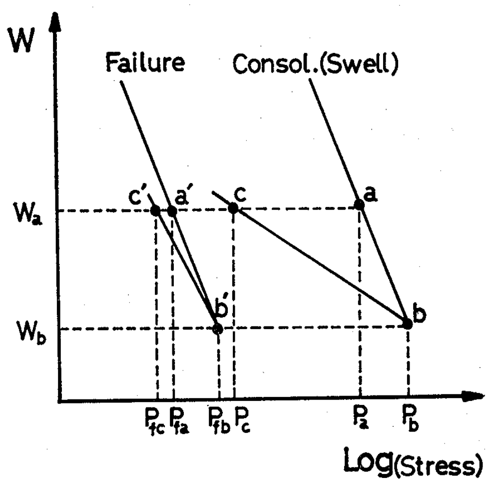

Fig. 3. Idealised relationships between water contents and effective stresses

$$w_a-w_b=C_c\log p_b/p_a=C_s\log p_b/p_c=C_f\log p_{fb}/p_{fc} \tag{2}$$

Combining $\lambda=C_s/C_c$ and $n=p_b/p_c$ with Eq. (2), we get

$$p_b/p_a=n^{\lambda} \tag{3}$$

And putting $\mu=C_s/C_f$ into Eq. (2) we obtain

$$p_{fb}/p_{fc}=n^{\mu} \tag{4}$$

The ratio of undrained shear strength s_{uc} to effective swelling pressure p_c at point c in Fig. 3 is expressed as follows. Since the equivalent consolidation pressure at point c is p_a, following equation is obtained on refering to Eq. (1).

$$s_{uc}/p_c=\kappa p_a/p_c\cos\phi_e+p_{fc}/p_c\sin\phi_e \tag{5}$$

Meanwhile, from Eqs. (3), (4) and $n=p_b/p_c$, we obtain

$$p_a/p_c=n^{1-\lambda},\quad p_{fc}/p_c=p_{fb}/p_b n^{1-\mu} \tag{6}$$

Substituting these into Eq. (5), following equation is derived.

$$s_{uc}/p_c=\kappa n^{1-\lambda}\cos\phi_e+p_{fb}/p_b n^{1-\mu}\sin\phi_e \tag{7}$$

On the other hand, s_u/p at point b, which is in the state of normal consolidation, is represented as follows.

$$s_{ub}/p_b=\kappa\cos\phi_e+p_{fb}/p_b\sin\phi_e \tag{8}$$

In Eq. (7), if we assume as $\lambda\doteqdot\mu$ within the range of $n\leqq 10$, we obtain,

$$s_{uc}/p_c\doteqdot(\kappa\cos\phi_e+p_{fb}/p_b\sin\phi_e)\,n^{1-\lambda} \tag{9}$$

Therefore, from Eqs. (8) and (9) we get following approximate equation,

$$s_{uc}/p_c\doteqdot s_{ub}/p_b n^{1-\lambda}$$

In general,

$$s_{un}/p_n\doteqdot s_{u1}/p_1 n^{1-\lambda} \tag{10}$$

This equation is valid for any clay satisfying the assumptions (1) to (4) and the condition $\lambda\doteqdot\mu$.

EXPERIMENTS

Clays Tested

Index properties of clay samples used are listed in Table 1. These samples were thoroughly mixed with distilled water, sieved by a 420μ size sieve and stored in the state of slurry. Before making test specimen, the slurry were stirred again in a soil mixer for about an hour and then transfered under a vacuum to a preconsolidation cell, the diameter and the height of which are 165mm and 350mm, respectively. Preconsolidation pressure

Table 1. Index properties of samples

Sample number	Liquid limit (%)	Plasticity index (%)	Specific gravity	Clay fraction (<2μ) %
1	52	21	2.70	21
2	51	21	2.72	19
3	72	32	2.69	26

was 0.8kg/cm² and consolidation period was about 4 days. Cylindrical specimens for triaxial tests, 50mm in diameter and 120mm in height, were trimmed from this preconsolidated sample.

Testing Procedure

Four series of consolidated undrained triaxial test were performed on three saturated remolded clays mentioned above. Test conditions are as follows.

(1) *CIU* Test; Specimens are consolidated isotropically and then sheared under undrained condition.

(2) *CIRIU* Test; Specimens are first consolidated under all-round pressure, allowed to swell under reduced isotropic pressure and finally subjected to undrained shear.

(3) CK_0U Test; Specimens are consolidated under K_0 condition and then subjected to undrained shear.

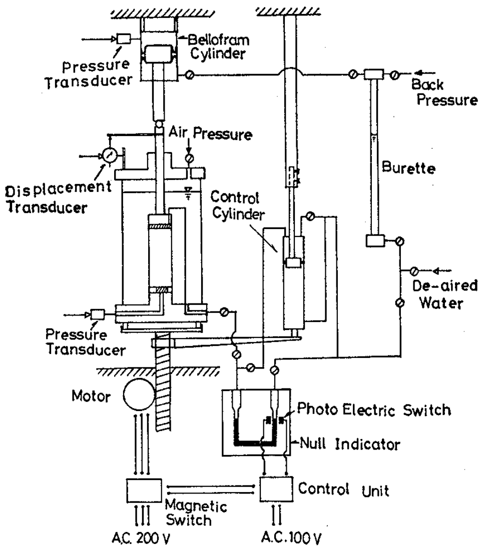

Fig. 4. Schematic diagram of K_0-control system

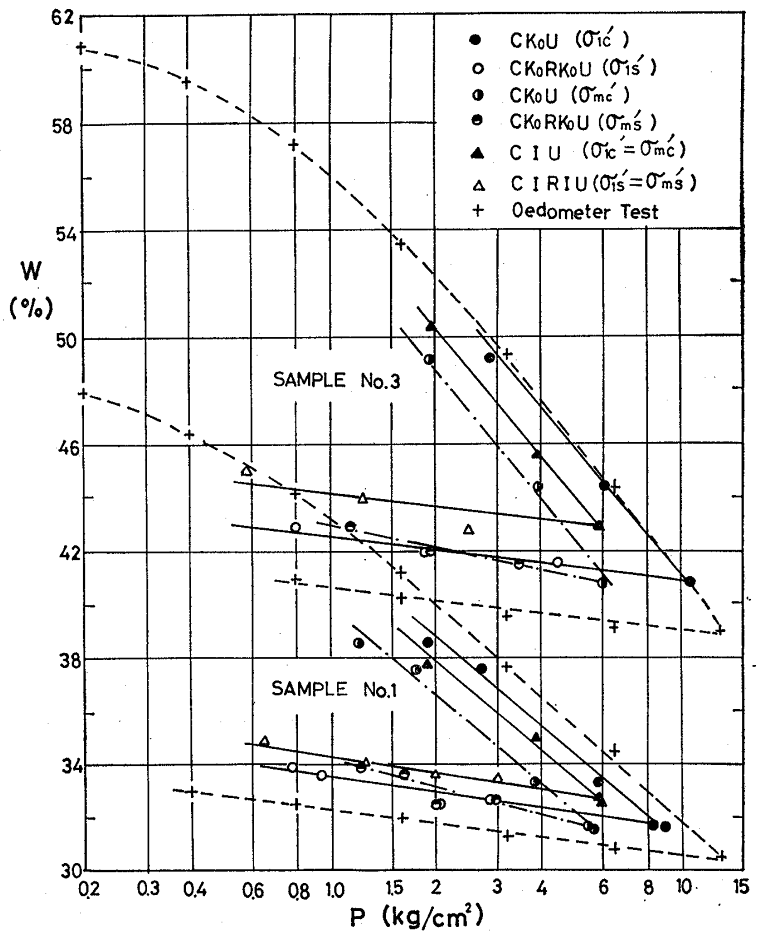

Fig. 5. Effective consolidation (swelling) pressure vs. water content

(4) CK_0RK_0U Test; Before undrained shear, the consolidation and succeeding swelling are carried out under K_0 condition.

CIU and $CIRIU$ tests were carried out with conventional triaxial apparatus. While, in consolidation and swelling processes of CK_0U and CK_0RK_0U tests, following K_0 consolidation equipment was used to establish the condition of no lateral strain.

Fig. 4 schematically illustrates an equipment designed for controlling automatically the K_0 condition during consolidation, on refering to Lewin's method (Lewin, 1971). Drainage water from specimen is led to burette via null indicator and control cylinder. This route is filled with de-aired water. Sectional area of the upper room of control cylinder (sectional area of piston rod is subtracted) is made to be equal to the sectional area A_0 of the specimen. This equipment establishes the K_0 condition in such a manner that the change in volume ΔV of specimen is made equal to the axial displacement ΔH multiplied by the original cross sectional area A_0 of the sample, i.e. $\Delta V = A_0 \cdot \Delta H$.

Mechanism of this equipment is as follows. The specimen is set up in a conventional triaxial cell in an usual manner and the K_0 control system is connected to the specimen just prior to consolidation. The test procedure involves increasing the cell pressure in steps. Subsequent volume change due to consolidation under all-round pressure makes an movement to the mercury in the null indicator. Photo-electric switch mounted on the null indicator detects this movememt and switches on, through delay relay and magnetic switch, the motor which drives the loading shaft to initiate the vertical compression of the specimen. At the same time, the movement of the shaft carries up with it the control cylinder. This displacement of the control cylinder sucks back the mercury and so switches off the motor. In the consolidation stage, the control unit of photo-electric switch is set to "dark on" and the change-over switch of the motor to "forward" so that the interception of the light of the photo-electric switch by the mercury switches on and rotates the motor in the direction compressing the specimen. We can perform K_0 swelling test in similar manner as the K_0 consolidation by setting the control unit to "light on" and the motor to "reverse".

In this way, depending on the magnitude of the final pressure, the consolidation took 3 to 6 days in CK_0U and CK_0RK_0U tests. In the process of consolidation in CIU and $CIRIU$ tests, all-round pressure was increased in steps so that the rate of increase was to be nearly equal to that in K_0 consolidation. Consolidation duration after the cell pressure reached final pressure was specified to be 24hr. in both K_0 and isotropic conditions.

Cell pressure in isotropic swelling was decreased to final pressure in one step, while, in K_0 swelling, it was decreased in 3 to 4 steps. However, the time allowed for swelling was 24hr. irrespective of steps in cell pressure decrease. Initial back pressure of 1kg/cm^2 was applied to all specimens.

The rate of axial strain in undrained compression was 0.05%/min. for all specimens and pore pressure was measured at the bottom of the specimen.

TEST RESULTS AND DISCUSSION

Relationship between Water Content and Preshear Effective Stress

It was reported that the relationship between the water content w and the average consolidation presure σ'_{mc} in K_0 consolidation was almost identical with that in isotropic consolidation (Henkel and Sowa, 1963).

Fig. 5 illustrates the w versus $\log p$ relationship for 4 tests (Results of Oedometer test are also plotted in this figure). For K_0 test, w is plotted against both vertical and average effective stresses. In the consolidation range, w versus $\log p_c$ lines are almost parallel and K_0 consolidation gives lower water content when compared to that of isotropic consolidation at the same average effective stress. By the way, Akai and Adachi (1965) demonstrated that the volumetric strain in the K_0 test was larger than that in isotropic consolidation test at the same average effective stress. Khera and Krizek (1967) found that the water content decreases with the increase in stress ratio at the same average effective stress. These test results indicate that the water content depends not only on the average effective stress but also on the stress ratio.

Results such as shown in Fig. 5 are also obtained by others (e.g. Lewin and Burland 1970, Campanella and Vaid 1972). Moreover, Henkel and Sowa (1963) described their own experimental results as "the fact that the water content and p' lines for both types of consolidation almost coincide must be regarded as fortuitus".

In the light of these experimental results, it will be reasonable to consider as follows. Since in the K_0 consolidation there exists a volume change due to dilatancy exerted by the increase in deviator stress, the water content after consolidation should be lower than that in isotropic consolidation at the same average effective stress.

Apart from the above discussion, the fact that w versus $\log p_c$ lines are almost parallel in normal consolidation range, indicates that compression index C_c is almost constant for both K_0 and isotropic stress conditions. And this means the adequacy of the preceeding assumption (1).

Turning attention to w versus $\log p_s$ relationship in swelling range, they approximate to straight lines within the range of $n \leqq 10$. And the lines representing w versus $\log p_s$ in terms of effective vertical swelling pressure σ_{1s}' in K_0 and isotropic swelling tests are almost parallel. This means that the swelling index C_s is constant and satisfies the assumption (2). Moreover, λ, which is the ratio of compression index C_c to swelling index C_s in terms of vertical effective stress, is almost identical for both K_0 and isotropic stress conditions. The values λ are computed as 0.20, 0.16 and 0.11 for Sample No. 1, 2 and 3, respectively.

Coefficient of Earth Pressure at Rest

The coefficient of earth pressure at rest K_0 is defined as the ratio of the minor principal effective stress to the major one after consolidation or swelling under the condition of no lateral strain. Table 2 shows the average K_0 value in normal consolidation and the effective angle of shearing resistance ϕ' obtained from CIU test. Brooker and Ireland (1965) reported that the value K_0 in normally consolidated clay could be approximated by the empirical relationship

Table 2. Coefficient of earth pressure at rest and effective angle of shearing resistance in normally consolidated samples

Sample number	K_0			ϕ' (°)	
	Observ.	Eq. (12)	Eq. (13)	CIU	CK_0U
1	0.45	0.40	0.43	37.2	35.1
2	0.45	0.43	0.45	35.1	34.9
3	0.47	0.41	0.44	36.1	34.0

$$K_0 = 0.95 - \sin\phi' \tag{11}$$

, whereas the original relationship proposed by Jáky

$$K_0 = 1 - \sin\phi' \tag{12}$$

were more valid for cohesionless soils.

However, Akai and Adachi (1965) and Yamaguchi (1972) described that both equations were open to question because K_0 value after consolidation were combined to the angle of shearing resistance at failure. Yamaguchi (1972) derived the equation expressing the relation between K_0 value and interparticle friction angle ϕ_μ, and combining ϕ_μ and ϕ' by

Caquot's equation, he finally obtained the following equation.

$$K_0=\frac{1-0.404\tan\phi'}{1+\sin\phi'} \tag{13}$$

In table 2, there listed the values of K_0 calculated by Eqs. (12) and (13) using ϕ' obtained from *CIU* test. K_0 values calculated by Eq. (13) are closer to the observed ones.

Brooker and Ireland (1965) found that the K_0 values in overconsolidation state could be expressed by the function of overconsolidation ratio n and plasticity index I_p. In this paper, let us investigate the relationship between the earth pressure at rest in overconsolidation state K_{0n} and overconsolidation ratio n.

Fig. 6 schematically illustrates the general trend existing in between observed water content and consolidation pressure, which can be seen in Fig. 5. From this figure, following equation is obtained.

$$w_a-w_b=C_{s1}\log p_b/p_d=C_{sm}\log p_b'/p_d' \tag{14}$$

where, C_{s1} is the slope of the swelling line $\overline{bd}$ which is represented in terms of vertical effective stress and C_{sm} is that of the swelling line $\overline{b'd'}$ in terms of average effective stress. While, there exists the following relationship.

$$p_b'=\frac{p_b(1+2K_{01})}{3},\quad p_d'=\frac{p_d(1+2K_{0n})}{3},\quad p_b/p_d=n \tag{15}$$

Combining Eq. (15) with Eq. (14)

$$1+2K_{0n}=(1+2K_{01})\,n^{(1-C_{s1}/C_{sm})} \tag{16}$$

As $\overline{bd}$ and $\overline{b'd'}$ lines intersect each other at point c,

$$C_{s1}\log p_b/p_c=C_{sm}\log p_b'/p_c \tag{17}$$

, where $p_b/p_c=n_0$ is overconsolidation ratio at point c. Combining Eqs. (15), (17) and $p_b/p_c=n_0$, we get following equation.

$$C_{s1}/C_{sm}=1+\frac{\log\dfrac{1+2K_{01}}{3}}{\log n_0} \tag{18}$$

Putting above equation into Eq. (16), following equation is obtained.

$$1+2K_{0n}=(1+2K_{01})\,n^{\alpha} \tag{19}$$

, where

$$\alpha=-\frac{\log\dfrac{1+2K_{01}}{3}}{\log n_0} \tag{20}$$

Comparison of K_{0n} values calculated by Eq. (19) to those observed in the tests are shown in Fig. 7, where the values K_{01} and n_0 required to calculate the K_{0n} were read from Table 2 and Fig. 5, respectively. Based on the theoretical consideration, Yamanouchi and Yasuhara (1974) described that K_{0n} is defined as $K_{0n}=f(n, I_p)$. Further, they reviewed the data reported up to that time and found that the slope of $\log(1+2K_{0n})$ vs. $\log n$ line, i.e. λ_m varies with I_p, where, $\lambda_m=C_{sm}/C_c$.

As it is apparent from the above mentioned investigation and from the data shown in Fig. 7 by the present authors, K_{0n} can be expressed as a function of n. But, finding out the dominant factor, which govern the value λ_m of Yamanouchi and Yasuhara or n_0 in the authors' Eq. (20), is the subject for a future study.

Pore Water Pressure Coefficient at Failure

Fig. 8 shows the variation of the pore water pressure coefficient at failure A_f with the overconsolidation ratio n, where the value A_f at $n=1$ i.e. in normal consolidation state is

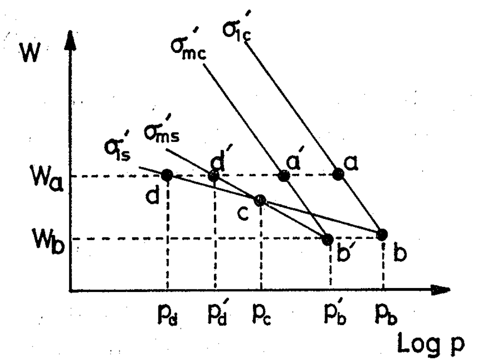

Fig. 6. Schematic representation of preshear effective stress vs. water content relationship

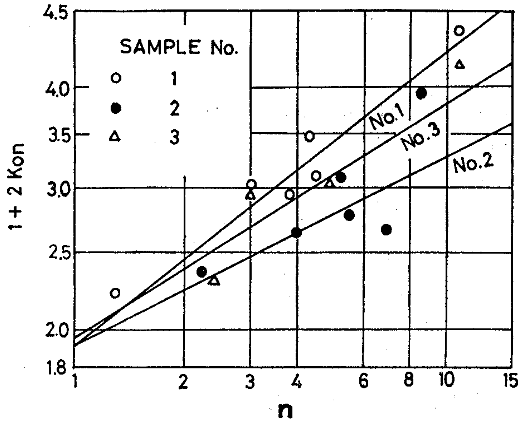

Fig. 7. Coefficient of earth pressure at rest in overconsolidation state vs. overconsolidation ratio

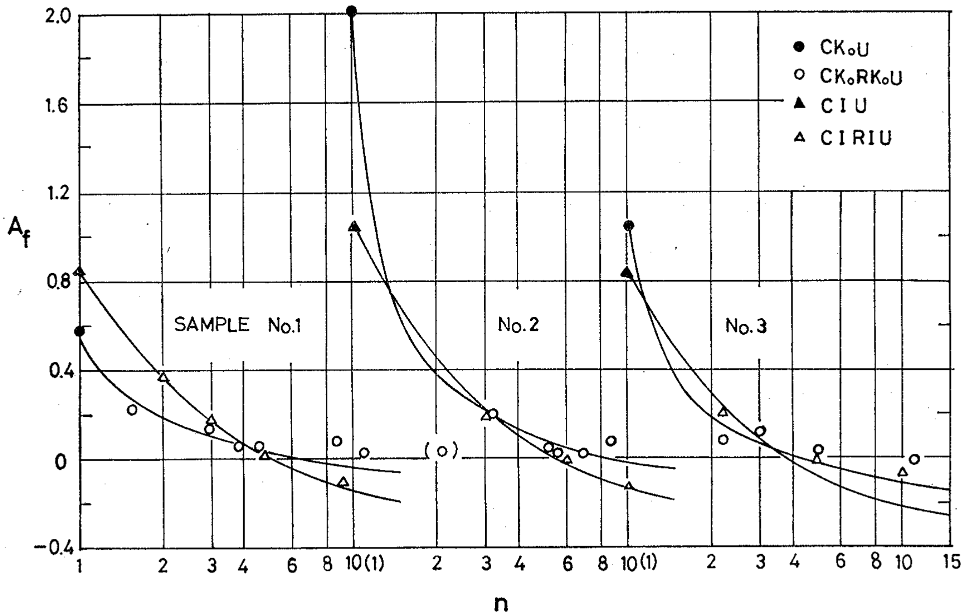

Fig. 8. Pore water pressure coefficient at failure vs. overconsolidation ratio

the average of values obtained from the tests in different consolidation pressures.

Curves in Fig. 8 were obtained by the following manner. As stated before, if the assumption $\lambda \doteqdot \mu$ is permitted, C_f is nearly equal to C_c, and hence p_{fa} is equal to p_{fc} in Fig. 3. For CK_0U and CK_0RK_0U test, p_{fa} and p_{fc} can be expressed in terms of undrained shear strength s_u, preshear effective stress p, coefficient of earth pressure at rest K_0 and the pore water pressure coefficient at failure A_f as follows.

$$\left.\begin{aligned} p_{fa}&=p_a\{K_{0a}+A_{fa}(1-K_{0a})\}+s_{ua}(1-2A_{fa}) \\ p_{fc}&=p_c\{K_{0c}+A_{fc}(1-K_{0c})\}+s_{uc}(1-2A_{fc}) \end{aligned}\right\} \tag{21}$$

According to Hvorslev failure criterion, undrained strength s_{ua} of normal consolidation will be equal to the undrained strength s_{uc} of overconsolidation, provided that the water content at point a is equal to that at c and $p_{fa}=p_{fc}$. Putting above condition and the relationship $s_{uc}/p_c=s_{ua}/p_a\ n^{1-\lambda}$ into Eq. (21) we get

$$A_{fc}=\frac{A_{fa}\{1-p_a/2s_{ua}(1-K_{0a})\}-p_a/2s_{ua}(K_{0a}-K_{0c}n^{\lambda-1})}{1-p_a/2s_{ua}(1-K_{0c})n^{\lambda-1}}$$

In general

$$A_{fn}=\frac{A_{f1}\{1-p_1/2s_{u1}(1-K_{01})\}-p_1/2s_{u1}(K_{01}-K_{0n}n^{\lambda-1})}{1-p_1/2s_{u1}(1-K_{0n})n^{\lambda-1}} \tag{22}$$

, where subscripts 1 and n represent the state of normal and overconsolidation, respectively. For *CIRIU* test, let $K_{01}=K_{0n}=1$ in Eq. (22), we obtain the following equation.

$$A_{fn}=A_{f1}-p_1/2s_{u1}(1-n^{\lambda-1}) \tag{23}$$

Eq. (23) is substantially the same as Eq. (22) of Roscoe et al. (1958), although the form of the equation is different to each other. Putting the values A_{f1} (average value A_f at $n=1$), s_{u1}/p_1 (shown later in Table 3), $1-\lambda$ (also shown later in Table 4), K_{01} (read from Table 2) and K_{0n} (calculated by Eq. 19) into Eqs. (22) and (23), we obtain the curves in Fig. 8. As seen in the figure, the calculated values of A_{fn} agree well with the observed ones, especially in the number 1 and 2 samples.

Effective Stress at Failure

Fig. 9 represents the relation between the water content w after consolidation or swelling and effective stress p_f, where p_f is equal to the average value of major and minor effective principal stresses at failure. From this figure, it is obvious that w versus $\log p_f$ lines in normally consolidated clay are straight and the slopes of these lines are almost equal to the compression index C_c. This fact satisfies the previous assumption (3). Further, the points corresponding to *CIRIU* and CK_0RK_0U tests are closely adjacent to the w versus $\log p_f$ lines for *CIU* and CK_0U test, respectively. Therefore, the previous assumption of approximate equality of λ and μ will be permitted within the range of $n\leqq 10$.

Change in Undrained Shear Strength Due to Consolidation and Swelling

The variation of the undrained strength s_u with the preshear effective stress p is shown

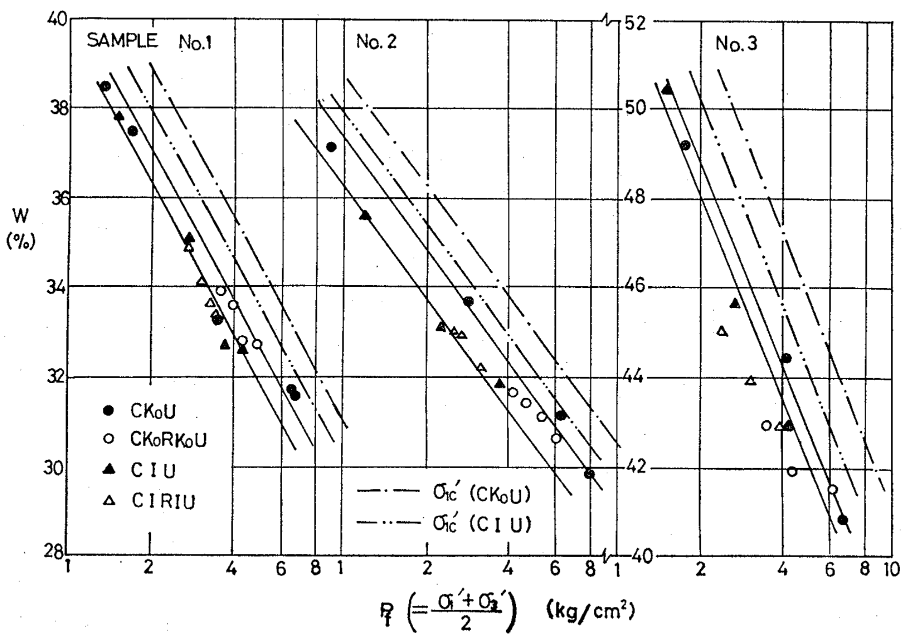

Fig. 9. Relationship between water content and effective stress at failure

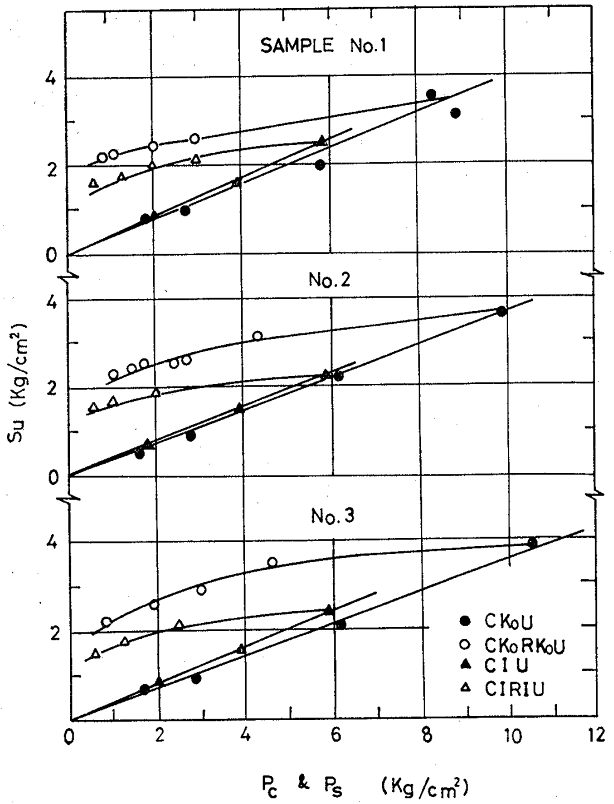

Fig. 10. Undrained shear strength vs. preshear effective stress

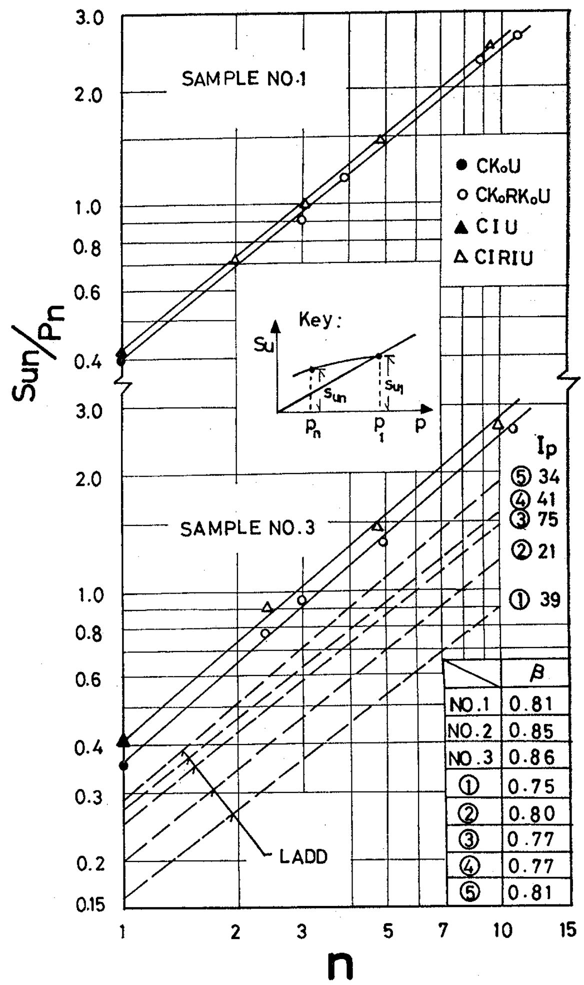

Fig. 11. Rate of change in undrained shear strength vs. overconsolidation ratio

in Fig. 10. The ratio s_u/p for normally consolidated clay in *CIU* tests are 5 to 12 per cent larger than those in CK_0U tests (see Table 3). These results coincide with that reported by others (e.g. Henkel and Sowa, 1963, Nakase et al. 1969).

Table 3. Comparison of the ratio s_u/p in *CIU* and CK_0U tests

Test	Sample number		
	1	2	3
CIU	0.42	0.36	0.41
CK_0U	0.40	0.34	0.36

Table 4. Comparison of the value $(1-\gamma)$ and β

Sample number	$1-\lambda$	β
1	0.80	0.81
2	0.84	0.85
3	0.89	0.86

Fig. 11 illustrates the relationships between the overconsolidation ratio n and the normalised ratio s_u/p, in which p is the preshear effective vertical stress. Broken lines are those replotted from the data given by Ladd and Foott (1974) in terms of s_u/p versus $\log n$. Their data were obtained by direct simple shear test, the test procedure of which corresponds to the CK_0RK_0U test by the present authors. As shown in Fig. 11, it is obvious that the relation between $\log s_u/p$ and $\log n$ is represented by a straight line (the data for Sample No. 2 were not shown in this figure because they fall in the same range in Sample No. 1), and the line for each CK_0RK_0U test is almost parallel to that for *CIRIU* test.

Consequently, s_u/p versus n relationship within the range of $n \leqq 10$ can be represented as follows.

$$(s_{un}/p_n)_{K_0} = (s_{u1}/p_1)_{K_0} n^{\beta} \tag{24}$$

$$(s_{un}/p_n)_I = (s_{u1}/p_1)_I n^{\beta} \tag{25}$$

where, the subscripts K_0 and I represent the K_0 and isotropic stress condition, respectively and the exponent β represents the slope of the $\log s_u/p$ versus $\log n$ line.

The form of Eqs. (24) and (25) are quite the same as Eq. (10) and the exponent β in Eqs. (24) and (25) corresponds to $(1-\lambda)$ in Eq. (10). By the way, the comparison of the values of $(1-\lambda)$ with those of β indicates fairly good agreement as shown in Table 4.

In this study, the experimental data are limited within the range of $n \leqq 10$. Experiments in isotropic condition for wide range of n values up to 375 were carried out by Yudhbir and Varadarajan (1974). Their data indicate that $(s_{un}/p_n)_I$ values increase abruptly for n values greater than about 100. In any case, the in situ undrained shear strength of a clay soil with any value of n within the range of $n \leqq 10$ can be estimated by using the value $(s_{u1}/p_1)_I$ obtained from the conventional CU triaxial compression test and λ obtained from the oedometer test, provided that the relation between $(s_{u1}/p_1)_I$ and $(s_{u1}/p_1)_{K_0}$ is established.

Previously, the authors (Mitachi and Kitago 1973) proposed a temporary method for estimating the $(s_{u1}/p_1)_{K_0}$ from $(s_{u1}/p_1)_I$. This method was based on the experimental evidence of Henkel and Sowa (1963) and that of the authors themselves (Mitachi and Ueda 1969 and Mitachi and Kitago 1973) that the water content in normal consolidation is uniquely defined by the average effective consolidation pressure irrespective of the stress systems during consolidation. The authors' present test data indicate that the relationship mentioned above is not always be satisfied. Therefore, the previous method of estimation lost its basis.

Accordingly, let us consider how to represent the relation between $(s_{u1}/p_1)_{K_0}$ and $(s_{u1}/p_1)_I$ in simple expression. From the experimental results in the past (e.g. Henkel and Sowa 1963, Ladd 1965, Nakase et al. 1969, Mitachi et al. 1970) the maximum difference between the values of $(s_{u1}/p_1)_I$ and $(s_{u1}/p_1)_{K_0}$ is about 20%. Now, it should be noted that the undrained shear strengths s_u in s_u/p mentioned above are not the shear stresses acting upon the failure plane but the ones exerted on the 45° plane. Mikasa (1969) proposed a convenient method to obtain the s_u on the failure plane from the conventional CIU test. His method is as follows. CIU test data are arranged as if they were conducted under the condition of constant average principal stress (in terms of total stress), and the plane on which this average principal stress acts is presumed to be the failure plane. The undrained shear strength obtained by this method is 6% less than that obtained by assuming $\phi_u=0$. Strictly speaking, the shear stress calculated by this method is not always equal to that on the failure plane, but the difference between the shear stress on the 45° plane and that on the failure plane may be in this order. On the other hand, the undrained shear strengths for stability computation have to be obtained from the tests in plane strain condition, since the conditions of many field problems closely approximate to those of plane strain. Investigations hitherto (e.g. Henkel and Wade, 1966, Kitago et al. 1973 and Vaid and Campanella, 1974) show that the s_u/p in plane strain condition is about 8 to 10 % larger than that in axi-symmetrical stress condition. Therefore, it can roughly be assumed that both effects mentioned above cancel each other.

Consequently, for conservative estimate, let $(s_{u1}/p_1)_{K_0} \fallingdotseq 0.8(s_{u1}/p_1)_I$, and following equation is obtained

$$(s_{un}/p_n)_{K_0} \fallingdotseq 0.8(s_{u1}/p_1)_I n^{1-\lambda} \tag{26}$$

Estimation of In Situ Undrained Shear Strength of Overconsolidated Clay

Using Eq. (26), the ratio of in situ undrained shear strength s_u to effective overburdon pressure p for overconsolidated clay can be estimated from conventional laboratory test. The procedures are as follows.

(1) From the conventional CU triaxial test which is performed under consolidated pressures greater than 1.5 to 2 times the in situ maximum past pressure (Ladd and Foott 1974), we obtain the value $(s_{u1}/p_1)_I$.

(2) Obtain the compression index C_c and swelling index C_s from a standard oedometer test, and calculate the value λ.

(3) Putting the values $(s_{u1}/p_1)_I$, λ and the overconsolidation ratio n for the ground into Eq. (26), then one can get the value $(s_{un}/p_n)_{K_0}$.

CONCLUSIONS

1) Based on the several assumptions, the ratio of undrained shear strength s_u to effective swelling pressure p for overconsolidated clay can be expressed approximately by Eq. (10) irrespective of the stress systems during consolidation and swelling.

2) From the experiments for three saturated remolded clays, it was found that the water content after K_0 consolidation is not always equal to that after isotropic consolidation under the same average effective consolidation pressure.

3) Coefficient of earth pressure at rest K_{0n} in overconsolidation state can be expressed as a function of overconsolidation ratio n. But finding out the dominant factor which govern the K_{0n} versus n relationship is the subject for a future study.

4) Pore water pressure coefficient at failure A_f in overconsolidation state within the range of $n \leqq 10$ is represented by Eq. (22) for K_0 condition and by Eq. (23) for isotropic stress condition.

5) Using Eq. (26), s_u/p in overconsolidated clay can be estimated from the data obtained by conventional laboratory test.

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NOTATION

A = Skempton's pore water pressure coefficient
A_f = Skempton's pore water pressure coefficient at failure
A_{f1}, A_{fn} = Skempton's pore water pressure coefficient at failure in normally and overconsolidated samples
A_0 = initial cross sectional area of sample
C_c = compression index
C_f = slope of w versus $\log p_f$ line
C_s, C_{s1} = swelling index in terms of vertical effective swelling pressure
C_{sm} = swelling index in terms of average effective swelling pressure
c' = cohesion intercept in terms of effective stress
c_e = Hvorslev effective cohesion
CIU = isotropically consolidated undrained test

$CIRIU$ = isotropically consolidated and rebounded undrained test
$CK_0U = K_0$ consolidated undrained test
$CK_0RK_0U = K_0$ consolidated and K_0 rebounded undrained test
ΔV = change in volume of sample
ΔH = change in height of sample
I_p = plasticity index
K_0 = coefficient of earth pressure at rest
K_{01}, K_{0n} = coefficient of earth pressure at rest in normal and overconsolidation state
n = overconsolidation ratio
n_0 = overconsolidation ratio at $K_{0n}=1$
p = preshear vertical effective stress
p_c = vertical effective stress
p_f = effective stress at failure on the 45° plane $\left(=\frac{\sigma_1'+\sigma_3'}{2}\right)$
p_1, p_n = preshear vertical effective stress in normal and overconsolidation state
p_s = vertical effective swelling pressure
s_u = undrained shear strength
s_{u1}, s_{un} = undrained shear strength in normally and overconsolidated samples
w = water content

$$\alpha = \frac{\log \dfrac{1+2K_{01}}{3}}{\log n_0}$$

β = slope of $\log s_{un}/p_n$ versus $\log n$ line
κ = coefficient of cohesion
$\lambda = C_s/C_c$
$\lambda_m = C_{sm}/C_c$
$\mu = C_s/C_f$
σ_1', σ_3' = major and minor effective principal stress
$\sigma_{1c}', \sigma_{mc}'$ = vertical and average effective consolidation pressure
σ_{1s}, σ_{ms} = vertical and average effective swelling pressure
σ_e' = equivalent consolidation pressure
ϕ' = effective angle of shearing resistance
ϕ_e = Hvorslev effective angle of internal friction
ϕ_μ = interparticle friction angle

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