EFFECTS OF PLASTICITY AND SAMPLE DISTURBANCE ON STATISTICAL PROPERTIES OF UNDRAINED SHEAR STRENGTH

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ABSTRACT

Statistical properties of undrained shear strength c_u are investigated through laboratory and in-situ tests. In laboratory, a series of unconfined compression tests and triaxial UU tests are performed using mixed soil samples. Concerning in-situ soil samples, comparative study is made between disturbed and undisturbed soil specimens. Following findings and method are obtained: (1) When plasticity index I_p is greater than 15%, the difference of c_u between unconfined compression test and triaxial UU test is less than 2% and is almost negligible. However, when I_p is smaller than 15%, unconfined compression test always provides smaller strength than triaxial UU test. c_u of unconfined compression test is about 15% smaller than that of triaxial UU test when $I_p \approx 10$. (2) Concerning the coefficient of variation of strength $V(c_u)$, however, there is no significant difference between these two testing methods. Moreover, $V(c_u)$ is found almost independent of both plasticity index and consolidation pressure. (3) When soil specimens are disturbed, the mean value of q_u becomes small while the coefficient of variation becomes large. In the case of E_{50} , however, the coefficient of variation holds constant during disturbance procedure. (4) The method of predicting statistical values of strength of undisturbed specimens from those of disturbed samples is newly presented.

Key words : cohesive soil, plasticity, sample disturbance, sampling, soft ground, statistical analysis, shear strength, triaxial compression test, unconfined compression test (IGC : D 6/C 6)

INTRODUCTION

The unconfined compressive strength (q_u) is widely used for the stability analysis of clay foundation under undrained condition. This is mainly because the average value of $q_u/2$ well describes the undrained shear strength on a failure surface in a ground in

total, and in addition to this, the simplicity of the testing procedure always meets the demand in engineering practice. In the cases of stiff clay and sandy clay, however, q_u usually underestimates the actual shear strength in-situ, since the influence of the small cracks and the low plasticity appear strongly by release of overburden loads. Although this

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fact itself has been well recognized in profession, the quantitative investigation is considered to be still insufficient. It is important in engineering practice to develop the general procedure of modifying the q_u -test results of such a brittle soil. In the present study, the unconfined compression test results for sandy soils is discussed for the application in practice by making the comparison with data obtained by the unconsolidated undrained triaxial test.

The other view point is concerned with the reliability-based analysis of stability. In the case, it is needed to know the variance and distribution of the shear strength as well as the mean value. Influence of sample disturbance on the variation should be especially noted here, because the useful data for design have hardly been published. The effective information on this problem is also given in the present study based on some actual case studies in the fields.

There are many factors influencing on the statistical properties of undrained shear strength, in which only the plasticity and sample disturbance are discussed in this paper. The reason for this is not only the lack of space but also the fact that influence of these factors is specifically large in practical engineering design. The high plastic clay are used as the samples in investigation of the effects of sample disturbance, whereas the soils of the range of ($I_p=10\sim15$) are often assumed as the $\phi_u=0$ soil in the practical design. In such a case, q_u can be modified based on the experimental results associated with effects of plasticity and $\bar{q}_u$, and $V(q_u)$ for design can be evaluated by using the results concerning the effects of sample disturbance. The statistical independence between these two factors on the shear strength and the deformation characteristics of soils are confirmed in other literature (Matsuo and Shogaki, 1986).

EFFECTS OF PLASTICITY

Soil Samples and Test Procedure

Soil samples were made by mixing two

Table 1. Index properties of soil samples

I_p (%)	10.1	12.3	15.8	22.8	43.7
G_s	2.69	2.63	2.67	2.70	2.67
w_L (%)	23.5	28.9	31.9	45.3	70.2
sand (%)	51	43	36	16	0
silt (%)	28	30	30	48	43
clay (%)	21	27	34	36	57

Fig. 1. Grain size distribution curves of samples

kinds of clay and crushed sand. Mixing ratios were arranged as getting the aimed plasticity index (I_p) and the grain size distribution. The index properties and the grading curves of these mixed soil samples are shown in Table 1 and Fig.1, respectively. The I_p -values are in the range of 10 to 44. The grading curves have the similar profiles of natural sedimentary soils. The soil sample was mixed more than one day in a soil mixer of 0.4 m³ capacity and the soil slurry was consolidated during two months in a mould of 26 cm inner diameter and 30 cm height. The consolidation pressure (p) was determined as 294 and 490 kN/m² which were considered to make the clays to be sampled by a stationary piston sampler. By using the block sampling method, forty two specimens in all of 3.5 cm in diameter and 8 cm in height were provided. The mean and the coefficient of variation of the undrained strength (c_u) and the secant modulus (E_{50}) discussed later are calculated based on the test results for these samples. The samples were sheared by 1%/min strain rate under the undrained condition. In the case of triaxial tests, $\sigma_3=2p/3$ was given as the confining pressure. Axial displacement and load appli-

ed were measured and stored by a microcomputer by each 5 second. The value of c_u is calculated from the maximum stress corresponding to the axial strain ϵ less than 15%. The secant modulus E_{50} is defined as c_u/ϵ_{50} , where ϵ_{50} is the strain corresponding to $q_u/2$ in the case of unconfined test and $(\sigma_1 - \sigma_3)_f/2$ in unconsolidated undrained triaxial test.

Results of Experiments

1) Evaluation of quality of soil specimens

In the case of $I_p \doteq 23$ and $p = 294 \text{ kN/m}^2$, for example, fluctuations of the water content (w_0) and unit weight (γ_t) of specimens show the very small ranges of (29.64~30.08)% and (18.88~19.13) kN/m^3 , respectively. Other cases of I_p and p have the similar fluctuations and therefore, specimens in the same group are guaranteed to have the same index properties. The stress-strain curves for the specimens with $I_p \doteq 15$ and $p = 490 \text{ kN/m}^2$ are shown in Fig. 2. The initial tangent modulus and failure strain ϵ_f ($\doteq 2\%$) are the same in each specimen. Thus, these samples can be regarded to be nearly perfect samples, which do not have the mechanical disturbance. In other words, the original mechanical properties are the same for all samples.

2) Effects of consolidation pressure and plasticity index on the mean value of undrained shear strength

Concerning the mean value $\bar{c}_u$ and $\bar{E}_{50}$, the ratios of those values of unconfined compression tests and the values of unconsolidated undrained triaxial tests are plotted against I_p in Fig. 3. All data for $p = 294$ and 490 kN/m^2 are plotted, since the independence of the $\bar{c}_u$ and the $\bar{E}_{50}$ ratios from the consolidation pressure p are previously confirmed (Matsuo and Shogaki, 1986). The solid and the broken lines show the regression curves which can be expressed as follows:

$$\left. \begin{aligned} \text{for } I_p \geq 15 \quad \bar{c}_u \text{ ratio} &\geq 0.98 \\ \text{for } 10 \leq I_p < 15 \quad \bar{c}_u \text{ ratio} &= 1 - 0.08/(I_p - 9.42) \end{aligned} \right\} \quad (1)$$

$$\begin{aligned} \text{for } 10 \leq I_p \leq 44 \quad \bar{E}_{50} \text{ ratio} &= 0.94 \\ &- 3.07/(I_p - 2.72) \end{aligned} \quad (2)$$

It should be noted that the $\bar{c}_u$ ratio is larger

Fig. 2. Stress-strain curves ($p = 490 \text{ kN/m}^2$, $I_p \doteq 15$)

Fig. 3. Relationship between $\bar{c}_u$, $\bar{E}_{50}$ ratio and I_p

than 0.98 for $I_p \geq 15$. This means that there is no difference of the shear strength in engineering sense, between unconfined test and triaxial test for $I_p \geq 15$, while for $I_p \doteq 10$, $\bar{c}_u$ by the unconfined test is about 15% smaller than that by the triaxial test. The I_p value is apt to be affected by an individual testing skill of each technician and therefore it is needed to investigate the engineering effectiveness of Fig. 3 from the different aspect. For the purpose, both $\bar{c}_u$ and $\bar{E}_{50}$ ratios are re-arranged against the fine grain content (F) of less than $74 \mu\text{m}$ and clay component (C) which can be determined by a really mechanical manner. The results are shown in Fig. 4 and Eqs. (3)~(6):

Fig. 4. Relationship between $\bar{c}_u$, $\bar{E}_{50}$ ratio and clay component (C), fine grains content (F)

$$\left. \begin{array}{l} \text{for } F \geq 64 \quad \bar{c}_u \text{ ratio} \geq 0.98 \\ \text{for } 50 \leq F < 64 \quad \bar{c}_u \text{ ratio} = 1 - 0.21/(F - 48.5) \end{array} \right\} \quad (3)$$

$$\left. \begin{array}{l} \text{for } C \geq 34 \quad \bar{c}_u \text{ ratio} \geq 0.98 \\ \text{for } 21 \leq C < 34 \quad \bar{c}_u \text{ ratio} = 1 - 0.18/(C - 19.7) \end{array} \right\} \quad (4)$$

$$\left. \begin{array}{l} \text{for } 50 \leq F \leq 100 \quad \bar{E}_{50} \text{ ratio} \\ = 1.12 - 23.4/(F - 11.7) \end{array} \right\} \quad (5)$$

$$\left. \begin{array}{l} \text{for } 21 \leq C \leq 57 \quad \bar{E}_{50} \text{ ratio} \\ = 1.15 - 18.8/(C + 8.8) \end{array} \right\} \quad (6)$$

For the isotropically consolidated undrained compression and extension tests carried out by using a mixed soil of Kawasaki clay and Toyoura sand (Nakase and Kamei, 1983), a rapid change of c_u ratio occurs at $I_p \doteq 15$. The results in Fig. 3 have the same tendency and it was checked up that the points of $F = 64\%$ and $C = 34\%$ correspond to those of $I_p \doteq 15\%$ in Fig. 3. In both figures, it should be especially noted that the release of the confining pressure influences on the deformation more strongly than the strength itself.

3) Effects of plasticity index on the coefficient of variation of the undrained shear strength

Fig. 5 shows the relation between the mean $\bar{c}_u$ and the coefficient of variation $V(c_u)$ of undrained shear strength. The values of $V(c_u)$ are in the range from 0.04 to 0.12 which can be regarded to be nearly constant. Okumura (1971) examined the residual effective stress of an "undisturbed" soil specimen

Fig. 5. Relationship between $V(c_u)$ and $\bar{c}_u$

to that of disturbed one and the ratio of the undrained strength of a disturbed specimen to that of a perfect sample. Fig. 5 is nearly equivalent to the results obtained by rearranging the Okumura's data.

The value of $V(c_u)$ obtained here is smaller than that for naturally deposited cohesive soils which ranges from 0.2 to 0.4 (Matsuo and Kuroda, 1974). This is because the samples used here are the remolded normally

a) Unconfined compression test

b) Unconsolidated undrained triaxial test

Fig. 6. Relationship between $V(E_{50})$ and $\bar{c}_u$

consolidated clay which were carefully made in a laboratory. For the purpose of the present study to know the clear tendency quantitatively, a large number of samples with equal quality is needed, but it is impossible to get them from natural deposits. The value of $\bar{c}_u = 156.8 \text{ kN/m}^2$ in Fig. 5 is the limit strength of the clay which can be sampled by a stationary piston sampler and therefore this figure includes the data of a sufficient range of $\bar{c}_u$ for the unconfined and triaxial tests.

The coefficient of variation $V(E_{50})$ is plotted against $\bar{c}_u$ in Fig. 6. In the case of the triaxial test, there is the relatively strong negative correlation between $V(E_{50})$ and $\bar{c}_u$ as can be seen in Fig. 6(b), while in the unconfined test of Fig. 6(a), the correlation is not found and $V(E_{50})$ is located in the range from 0.05 to 0.25. This is considered to be caused by release of the confining pressure. It is consistent to the facts in Fig. 2 that the change of initial tangent modulus

for the triaxial test is very much small compared with that of the unconfined test and this tendency was the same in the different plasticity index I_p and consolidation pressure p . A plot of the plasticity index I_p versus the coefficient of variation $V(c_u)$ is given in Fig. 7 which shows the small variation of $V(c_u)$ and the uncorelation between $V(c_u)$ and I_p . The results of Figs. 5 and 7 are specifically important, since they guarantee the effectiveness of application of the unconfined test results to the reliability-based design for soft ground in a wide range of I_p and p . In addition, even for the soils of $I_p < 15$, the triaxial test is not needed in general by using the results of Fig. 3, although it is better to get several data of the triaxial test for soils or soft rocks with complex stress path which usually involve micro cracks.

EFFECTS OF SAMPLE DISTURBANCE ON UNDRAINED SHEAR STRENGTH

Effects on the Average Value

The variation of the shear strength is caused by the intrinsic inhomogeneity of a ground and the degree of technician's working skill through the in-situ sampling to testing in a laboratory. Many research works have been done for the former problems and some excellent results have been published for the various kinds of soils. Associated with the human-caused errors, however, the investigations are not sufficient although its influence is well known to be considerably large. This is dependent on the following two facts: (1) there are many individual factors caused by the degree of skill of the technicians carrying out the soil explorations and tests, and (2) since the number of combinations of natural conditions including the kinds of soil deposits is huge, it is actually impossible to experimentally investigate the influencing degree of each factor. In the case, the effective approach to the problem is the analysis of results of questionnaires for the senior engineers and researchers with a wealth of practical experiences. The re-

a) Unconfined compression test

b) Unconsolidated undrained triaxial test

Fig. 7. Relationship between $V(c_u)$ and I_p

Table 2. Notations of factors influencing q_u value given in questionnaires (after Matsuo and Shogaki, 1986)

A	1	Excess compression of soils due to penetration of sampler without drilling hole
	2	Swelling of soils due to insufficient of pressure of drilling mud
	3	Rapid rise of pumping pressure during drilling
B	1	Unexpected movement of soil sampling along the wall of sampling tube
	2	Deformation of soil samples due to insufficient rigidity of sampling tube
	3	Excess compression of soil samples 10% more than usual due to choked up drain hole
C	1	Compression of soils at the bottom of borehole due to unfixed drilling rod
	2	Drilling mud between stationary piston and soil samples
	3	Deformation of cutting edge
D	1	Excess compression of soil samples 5% more than usual due to insufficient locking of stationary piston
	2	Sampling with the use of free piston sampler
E	1	Irregular penetration of sampling tube
	2	Sample disturbance due to the existence of randomly distributed shells
	3	Excess penetration of sampler 10% more than usual
F	1	Rotation of sampler before withdrawal
	2	Drilling of casing before withdrawal of sampler
	3	Dropping of the sampler with soil samples to the bottom of borehole
G	1	Deformation of sampling tube during drawing of piston
	2	Suction at the top of soil sample due to drawing of piston
	3	Vibration and shocks of soil samples due to engine
H	1	Soil testing after three days with insufficient of sealing of sample
	2	Movement of soil samples under the work of sealing soil sampler
	3	Fall of soil sampler during operation of sealing
I	1	Deformation of sampling tube due to variations of temperature
	2	Deformation of sampling tube caused by high temperature
	3	Vibration during transportation
J	1	Friction between soil and sampler during extrusion
	2	Horizontal extrusion of soil samples
	3	Excess fastening of sampling tube during extrusion
K	1	Soil testing after seven to ten days since sampling
	2	Two specimens of 35 mm diameter from the sample of 75 mm diameter
	3	Trimming of soil samples by unskillful worker
	4	5%/min strain rate during compression

sponses from seventy-two such the persons were gathered and the results were analyzed by the trade-off method for the total model and the Probit and Rojit methods for the non total model. The factors given in questionnaires are shown in Table 2 (Matsuo and Shogaki, 1986). The average value of the degree of contribution on decrease of $\bar{q}_u$ is

given by the full line in Fig.8. At the same time, in order to examine the results by questionnaires quantitatively, the laboratory tests and field investigations as much as possible were carried out under the simplified conditions of each factor in Table 2. The results are shown as the decreasing ratio of $\bar{q}_u$ by the broken line in Fig.8. According

Fig. 8. Contribution on decrease and decreasing ratio of $\bar{q}_u$ for factors

to these results, the factor of giving the large degree of contribution produces the large decreasing ratio of $\bar{q}_u$ and they are consistent. Fig. 8 gives the information associated with the mean value of $\bar{q}_u$. The effect of soil disturbance on the variation will be discussed in the later sections.

Field Investigations of Effects of Sample Disturbance

There is a large number of combinations of the conditions yielding the sample disturbance through the process from in-situ sampling to testing in a laboratory. It is ideal to investigate the effect of each combination of factors, but actually impossible because such many experimental data cannot be provided. In this section, the effects are analyzed by three actual field cases and the results must give the useful information at the present.

i) Disturbance of clay foundation by sand piling

Sand piles were driven for construction of a shore structure revetment at Yokkaichi

Fig. 9. Plan of the site of soil sampling

port. In order to investigate the disturbance zone by sand piling works, three borings to sample the undisturbed soils were carried out before sand piling and at two weeks and two months after the construction. In Fig. 9, the sampling points are shown. At the place of 50 m away from this site, the similar soil investigations were performed and their results are also used in analysis. The field water content w_0 and the unit weight γ_t at each depth are plotted in Fig. 10. The silt layer of about 4 m thick located at 19 m below the surface shows the smaller water content than those of the upper and lower layers. Therefore, the soil profile of this site is divided into three zones, which are called as layer A, B and C, respectively, as shown in the figure. The frequency distributions of w_0 and γ_t of layer B which can be expressed by the normal distribution curves are shown in Fig. 11. The water content of about 1% decreases after piling, whereas the unit weight of about 0.1 kN/m³ increases. This is caused by consolidation and the influence of sand piling on w_0 and γ_t is very much small. The cumulative frequency curves of q_u , ϵ and E_{50} for each layer are plotted in Fig. 12 which shows the strong effect of disturbance. The effects of sample disturbance are typically found in layer A and B. That is, the mean of ϵ at two weeks after piling is about 1% larger than that before piling and as a result, the mean values of q_u and E_{50} become smaller by about 15 kN/m² and 1500 kN/m², respectively. Two months later, however, the effect of disturbance is a little made up by consolidation and $\bar{q}_u$ and $\bar{E}_{50}$ recover as much as about 3 kN/m² and

Fig. 10. w_o and γ_t with depth

80 kN/m².

ii) *Effects of types of samplers*

Field investigations were performed in order to verify the qualities of specimens sampled by the free piston sampler and the stationary piston sampler (Matsuo and Shogaki,

a) Frequency distribution of w_o b) Frequency distribution of γ_t Fig. 11. Frequency distribution curves of w_o and γ_t (layer B)

1984). The site is located off-shore in Yokkaichi port where the homogeneous alluvial marine clay is horizontally deposited at $- (8 \sim 42)$ m depth below the sea surface. From this layer, the soils were sampled by above stated two kinds of samplers. Both tubes were distanced 2 m from each other in order to avoid the disturbance by boring. Furthermore, for the purpose of distinguishing the characteristics of both sampling methods, only one technician operated the boring equipments and unconfined testing apparatus. Fig. 13 illustrates the profiles of w_o , γ_t , q_u , ϵ and E_{50} . The solid and broken lines denote the mean values calculated by the least square method corresponding to each sampling method. It can be seen that the q_u values obtained by the equipment of the free piston sampler are about 40% smaller than those by the stationary piston sampler equipment except for the data located between -26 m and -30 m.

iii) *Effects of individual technician*

Effects of the skill of individual technicians on the undrained shear strength are evaluated based on the field investigations. Technicians are tentatively named here D, E and F. Along a sea-side line of 500 m long, the technician D bored at 6 points, the technician E at 7 points and the technician F at 3 points. The clay layer can be regarded as

Fig. 12. Cumulative frequency curves of q_u , ϵ , E_{50} for each layer

Fig. 13. Profiles of w_0 , γ_t , q_u , ϵ , and E_{50} (after Matsuo and Shogaki, 1984)

the same in engineering sense. The difference of the average values of γ_t and w_0 at the same depth obtained by each technician are less than 6 kN/m^3 for γ_t and 2% for w_0 , respectively. For the statistical calculation, about fifty data of q_u -values included in every 2 m depth are used. This is based on the assumption that the data obtained in each 2 m depth are statistically homogeneous, because there are not so many data at the same

depth in actual condition. The values of $\bar{q}_u$ and $V(q_u)$ obtained by D are compared with those by E and F as shown in Fig. 14. The $\bar{q}_u$ values by E and F are about 10% and 2% smaller than those by D, respectively and the $V(q_u)$ are about 20% larger. The frequency distributions of ϵ are shown in Fig. 15. The mean values $\bar{\epsilon}$ corresponding to each technician become as $D < F < E$ in order and $\bar{\epsilon}$ by E is approximately 1% larger

 Fig. 14. $\bar{q}_u$ and $V(q_u)$ ratio at each depth

 Fig. 15. Frequency distributions of ϵ

than that by D. That is, the specimens sampled by E are much disturbed.

Change of Statistical Properties due to Sample Disturbance

The reduction of the shear strength caused by sample disturbance is shown in the relation to $V(q_u)$ in Fig. 16. In this figure, in addition to the data in Fig. 14, the results from Figs. 12 and 13 which are calculated by the similar manner to Fig. 14 concerning the statistical data are also plotted. Specimens which exhibit the increase of q_u in spite of the external disturbance should be classified into undisturbed ones because the effects of their disturbance factors are considered to be very small and the data for them are excluded from Fig. 16. The $V(q_u)$ values of specimens without disturbance vary from 0.021 to 0.169 which is the similar range of the perfect specimens of mixed soils

 Fig. 16. Relationship between $V(q_u)$ and the ratio of $\bar{q}_u$ of the disturbed samples to those of the undisturbed sample

shown in Fig. 5. On the other hand, the $V(q_u)$ values for the disturbed specimens are in the range of 0.057 to 0.526. That is, the $V(q_u)$ values become larger when soils are disturbed.

The decrease of q_u and increase of $V(q_u)$ caused by the external disturbances are shown in Fig. 17 in the form of the decreasing and the increasing ratios, respectively. The ratio of $\bar{q}_u$ has the considerably strong negative correlation to the ratio of $V(q_u)$. Fig. 17 includes the data under the different disturbance factors at three different sites and therefore the relationship of this figure can be said to be independent from I_p , p and q_u values. Since the decreasing ratio of $\bar{q}_u$ in Fig. 8 has the same meaning to the ratio of $\bar{q}_u$ in Fig. 17, if the disturbance factors applied to the specimens are known, the

 Fig. 17. Relationship between the ratio of $\bar{q}_u$ and the $V(q_u)$ of the disturbed samples to those of the undisturbed sample

Fig. 18. Relationship between the ratio of $\bar{E}_{50}$ and $V(E_{50})$ of the disturbed samples to those of the undisturbed sample

decrease of $\bar{q}_u$ can be obtained by Fig. 8 and the corresponding increase of $V(q_u)$ can be read by using Fig. 17. Thus the mean and the coefficient of variation of the undrained shear strength caused by sample disturbance are correctly estimated. In Fig. 18, The ratio of $\bar{E}_{50}$ is related to $V(E_{50})$ in the form of the ratio of the disturbed to undisturbed cases. The $V(E_{50})$ is free from the degree of the sample disturbance and it may be constant as estimated 1.28.

CONCLUSIONS

The conclusions obtained from the present study are summarized as follows :

1) The average value of $\bar{c}_u$ by the unconfined compression test is larger than 0.98 times of the value by the undrained triaxial compression test for $I_p \geq 15$. In other words, there is no difference in the undrained shear strength estimated by both tests in this case. But for the case of $I_p < 15$, the unconfined test yields the smaller shear strength than the triaxial test.

2) The $V(c_u)$ value is independent from the unconfined and confined testing methods, the plasticity index I_p and the consolidation pressure p . This fact ensures the effec-

tiveness and usefulness of the unconfined test in engineering practice, since it is easy to carry out.

3) The ratio of $\bar{E}_{50}$ of the unconfined test to that of the triaxial test decreases as I_p decreases, and it approaches to 0.514 for $I_p = 10$. This means that the release of confining pressure yields the stronger effects on deformation characteristics than on strength characteristics.

4) If a soil is disturbed, $V(q_u)$ becomes larger with decrease of q_u , but the $V(E_{50})$ ratio is not affected by the disturbance.

5) A comprehensive procedure to modify the measured $\bar{q}_u$ and $V(q_u)$ values of the disturbed samples is proposed.

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