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AN IMPROVED METHOD FOR ESTIMATING *IN-SITU* UNDRAINED SHEAR STRENGTH OF NATURAL DEPOSITS

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ABSTRACT

An equation for estimating *in-situ* undrained shear strength ($q_{u(I)}$) of natural deposits is derived as $q_{u(I)}/2c_{u(I)}=1.0\text{–}0.285\ \ln p_m/S_0$ through the unconfined compression test (UCT) and K_0 consolidated-undrained triaxial compression test (CK_0UC). The $q_{u(I)}$ of natural clay deposits can be estimated from the q_u value multiplied by the reciprocal number of $q_{u(I)}/2c_{u(I)}$ of the equation using the suction (S_0) and q_u obtained from UCT for a specimen, where $c_{u(I)}$ is *in-situ* shear strength measured from CK_0UC and p_m is two times the effective overburden pressure divided by three. The $q_{u(I)}/2c_{u(I)}$ values were unrelated to I_p, q_u and p_m/S_0 and the mean value of these ratios was 0.98 in the range of $I_p=26\sim110$ and $q_u=12\sim178$ kPa. The mean values of the ratios for q_u, $q^*_{u(I)}$ and $q_{u(I)}$ to $2c_{u(I)}$ were 0.629, 0.998 and 0.977 and the standard deviation of those ratios were 0.14, 0.10 and 0.16, respectively. Therefore, it can be seen that the improved method is appropriate as well as Shogaki's basic method ($q^*_{u(I)}$). The mean values of $q_{u(I)}/2c_{u(I)}$ were 0.94, 0.99 and 0.91 for Iwai organic and soft clay plus Kahokugata clay, respectively. The coefficient of variations of the q_u and $q_{u(I)}$ values were $(13\sim14)$% and unrelated to soils, q_u or $q_{u(I)}$ values. Therefore, the applicability of the improved method newly developed in this study can be confirmed for Kahokugata and Iwai clays as well as Iwai organic soils. The proposed method is a simple and easy one for practical engineering usage.

Key words: clay, *in-situ* undrained strength, organic soil, sample disturbance, suction, unconfined compression test (**IGC**: C6/D5)

INTRODUCTION

The undrained shear strength is the most important design parameter for short-term stability problems of clay foundations. The unconfined compressive strength (q_u) is widely used in Japan for stability analysis of clay foundations under undrained conditions. This is mainly because the mean value of $q_u/2$ clearly describes the undrained shear strength on the failure surface in a soil (Nakase, 1967; Matsuo and Asaoka, 1976; Shogaki et al., 1997) and in addition to this, the simplicity of the testing procedure always meets the requirements of engineering practices and is successful in investigating for design and in analyses of failure cases.

It is well known that the q_u value is changeable by the intrinsic inhomogeneity of a soil and the degree of a technician's skill from *in-situ* sampling to testing in a laboratory (Matsuo and Shogaki, 1984; Shogaki and Kaneko, 1994). From this and through international coordination on soil investigation, testing and foundation design methods, there are critics of using the q_u value in practical design (Hanzawa, 1996). However, the efforts involved in providing these imperfect UCT procedures are very important in order to take advantage of the $\phi_u=0$ method that uses the q_u value, which is an excellent design method systematized by a number of Japanese pioneers in this field of study. Limit state design and/or performanced based design methods have been applied to foundation design standards for railroads (Railway Technical Research Institute, 1997) and are included in Eurocodes 7, 8 and Geocode 21 (Honjo and Kusakabe, 2002), etc. The probability and statistical approach to geotechnical data is indispensable to these design methods. Therefore, the UCT can be used in these approaches based on the advantages of the q_u described. This study also considered the treatment of geotechnical data for these approaches.

For the approximately 60 factors influencing q_u values, the effects of sample disturbance were analyzed from the results of questionnaires given to senior engineers and researchers with a wealth of practical experience. Also, laboratory tests and field investigations were carried out under the simplified conditions of each factor (Matsuo and Shogaki, 1988). However, there was not enough quantitative interpretation of the effective stress behavior of disturbed samples in the study. Therefore, the degree of sample disturbance for the measured q_u values could not be quantitatively evaluated.

The undrained shear strength for the short-term stability problems of clay foundations does not take into consideration the *in-situ* undrained shear strength since it is influenced by strength isotropy, strain rate, etc.

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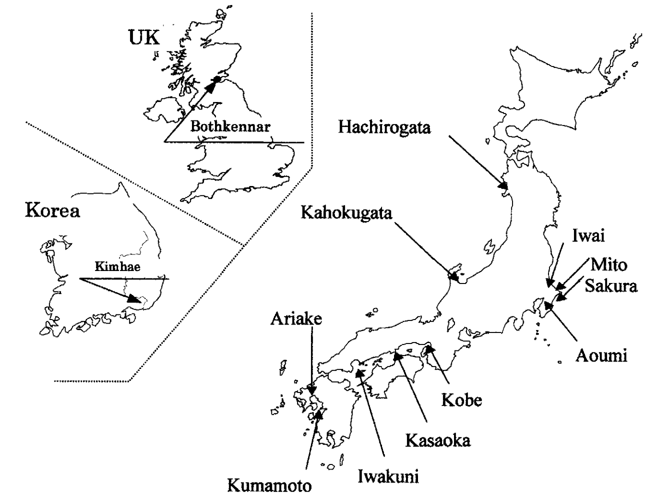

Fig. 1. Sampling sites of used soils

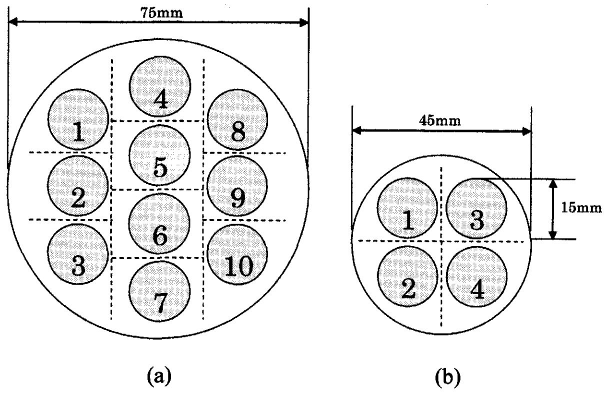

Fig. 2. Location of specimens for the plane of the sampling tube

However, it does not mean that the *in-situ* undrained shear strength is not estimated. Site investigation, laboratory testing and design method regulatory efforts need to place emphasis on performanced based design. Therefore, the studies on the estimation of the strength relevant to idealized shear conditions, evaluation of sample disturbance and the relationship between measured q_u value and estimated *in-situ* undrained shear strength, etc. are required.

In this paper, an equation for estimating *in-situ* undrained shear strength of natural deposits is derived as $q_{u(I)}/2c_{u(I)}=1.0\text{–}0.285 \ln p_m/S_0$ through the unconfined compression test (UCT) and K_0 consolidated-undrained triaxial compression test (CK_0UC). The *in-situ* undrained shear strength ($q_{u(I)}$) of natural clay deposits can be estimated from the q_u value multiplied by the reciprocal number of $q_{u(I)}/2c_{u(I)}$ of the equation using the suction (S_0) and q_u obtained from UCT for a specimen. The applicability of the improved method newly developed in this study can be confirmed for Kahokugata and Ibaraki clays as well as Ibaraki organic soils. In this study, only one or two samples, 74 mm or 45 mm in diameter and 100 mm in height, obtained from different sites, were used for the whole series of tests to provide small-size specimens.

SOIL SAMPLES AND TEST PROCEDURES

The undisturbed soils and their remolded samples used in this study were obtained from the Holocene clay deposits located in the Holocene Plains of Hachirougata, Kahokugata, Mito, Iwai, Sakura, Aoumi, Nagoya, Kobe, Kasaoka, Iwakuni, Ariake and Kumamoto in Japan, as well as Bothkennar clay in the United Kingdom and Kimhae clay in Korea, as shown in Fig. 1. Field sampling was performed with fixed-piston samplers having inner diameters of 75 mm (JGS 1221-2003) and 45-mm (Shogaki, 1997a). The 45-mm sampler gives a sample quality similar to or better than that of the 75-mm sampler (Shogaki and Sakamoto, 2004). The mean values (w_n) of natural water content (w_n), plasticity index (I_p), effective overburden pressure (σ'_{vo}), mean value (q_u and S_0) of q_u and specimen suction (S_0) are shown in Table 1. The I_p and q_u values are in the range of 26 to 370 and 15 kPa to 168 kPa respectively, which are very wide ranges.

S (Small) specimens 15 mm in diameter (d) and 35 mm in height (h) are used for the UCT and the triaxial compression test. Ten S specimens, 75 mm in d and 45 mm in h, were obtained by a 75-mm sampler and four S specimens, 45 mm in d and 45 mm in h were obtained by a 45-mm sampler, as shown in Fig. 2. Shogaki et al. (1995b) showed that the strength and deformation properties of ten S specimens obtained from a sample 75 mm in d and 45 mm in h were similar in an engineering sense. The specimen site, located a few mm away from the tube wall, as shown in Fig. 2, does not influence the sample disturbance caused by tube penetration and friction between soil and tube during sample extrusion. This was also confirmed by using a Scanning Electron Microscope (Shogaki and Matsuo, 1985) and a color laser three dimensional profile microscope (Shogaki, 2006).

The Portable Unconfined Compression Apparatus (PUCA) for measuring the S_0 and the q_u is shown in Fig. 3. In this apparatus, the load is applied by the linear head and is transmitted through an AC/DC powered motor. This equipment has a height of about 20 cm and a mass of about 70 kN. Therefore, since the equipment is portable, it is practical for field use. The UCT was performed on S specimens at a strain rate of 1%/min, after the specimen suction was measured using a ceramic disc plate. The air entry value of a ceramic disc is about 200 kPa. The PUCA, using a perspex cylinder, measures the suction over one atmospheric pressure and the q_u value for hard soil with latent hair cracks (Shogaki, 1997b).

The procedure for measuring the suction is the same as that reported in other literature (Shogaki et al., 1995a; Shogaki and Maruyama, 1998). It is important that the specimen suction be measured quickly, not only for the shear test using measured specimen suction, but also for shortening of the testing time and cost. The suction measurements of specimens taken in advance were confirmed as follows:

1) The S_0 of a specimen, in which the suction becomes

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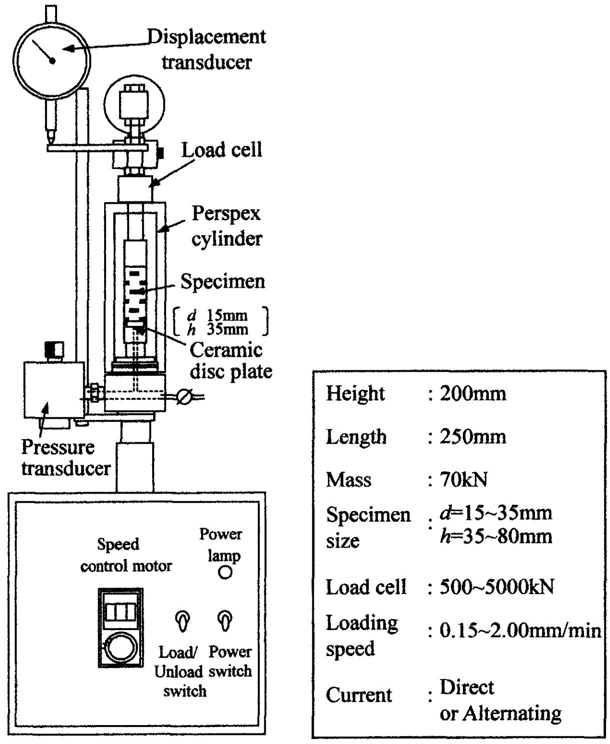

Fig. 3. Layout of the portable unconfined compression apparatus

constant, can be measured quickest when the piezometer measurement indicates the same value as the specimen suction.

2) However, the possibility exists that suction greater than that of the piezometer measurement cannot be accurately measured if all air is not removed from the pressure transducer pipe.
3) If the air in the pressure transducer pipe is removed and the piezometer sensitivity is high, the measured S_0 values are unrelated when the specimen is put on the ceramic disc plate to give piezometer measurements.
4) However, the time in which suction becomes constant is less when the suction drops from a larger value to specimen suction.

The maximum time for measuring suction was six minutes. For the effect of measuring suction before and after shear on strength and deformation properties of specimens, it was confirmed by Shogaki et al. (1995a) that there was no effect for natural deposits. Namely, the decreasing water content of specimens was less than 1% under suction measurement and the strength and deformation properties were independent of suction measurement. Each specimen was sheared at 1%/min after suction measurement using PUCT according to the Japanese Industrial Standard for unconfined compression tests of soils (JIS A1216–1993).

The procedures for preparing the remolded soil samples, using Shogaki's basic method (Shogaki and Maruyama, 1998), were as follows:

1) The undisturbed specimen was removed after the soil was put into a plastic pouch. The amount of soil used for remolding almost equaled the total amount of five S specimens.
2) The mouth of the pouch was closed after the air was removed by suction.
3) The soil was well kneaded by hand from outside the pouch. The q_u value, e.g., the Aoumi clay, did not change after a remolding time of more than 5 min.

The value of q_u was determined to be the maximum stress corresponding to an axial strain of less than 15%. There were no differences in suction and shear strength characteristics between the S and the ordinary size specimens (35 mm in diameter and 80 mm in height), which were examined for soils having I_p from 17 to 150 and q_u from 20 kPa to 1000 kPa and taken from twenty different sites (Sakamoto and Shogaki, 2003).

The CK_0UC were performed with the precision triaxial test apparatus (PTA) (Shogaki et al., 1999; Shogaki and Nochikawa, 2004) using S specimens. The CK_0UC tests for the undisturbed and disturbed soils were performed according to the standards of the Japanese Geotechnical Society (JGS) for consolidated-undrained triaxial compression tests (JGS 0525–1996) on soils with pore water pressure measurement. The specimen height decreases with K_0-consolidation, becoming about 35 mm at shear after K_0-consolidation. The initial isotropic consolidation pressure before K_0-consolidation was $5 \sim 10$ kPa. The pore pressure coefficient values were greater than 0.98. The specimens were sheared under a shear strain rate ($\dot{\varepsilon}_s$) of 1.0%/min after the K_0 consolidation at a strain rate of 0.005%/min. The undrained shear strength from the CK_0UC equals half of the principal stress difference.

The oedometer tests were performed using a load increment ratio of unity and the duration of loading for each load increment was one day. The values of the compression index (C_c) and the preconsolidation pressure (σ'_p) were determined from the e-log σ'_v curve based on the Japanese Industrial Standard for determining one-dimensional consolidation properties of soils (JIS A 1217–1993).

REVIEW OF STUDIES FOR ESTIMATING *IN-SITU* UNDRAINED SHEAR STRENGTH

Related Studies and Shogaki's Basic Method

There have been numerous methods proposed for the estimation of *in-situ* undrained shear strength. The recompression method, using the CK_0UC test (Bjerrum, 1973) and the SHANSEP method (Ladd and Foott, 1974), are well known. These methods are based on the idea that the undrained shear strength under the stress condition of *in-situ* corresponds to the *in-situ* undrained shear strength. Mitachi and Kudoh (1996) proposed an estimation method for *in-situ* undrained shear strength from the results of S_0 and standard oedometer tests. This method is based on the idea that strength reduction caused by stress release and sample disturbance during sampling is a behavior concerning the swelling process of the e-log σ'_v relationship. However, the effect of sample disturbance on the consolidation parameters, such as the

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σ'_p and C_c, is not considered in their method.

If the saturated clay is taken from *in-situ* pressure to atmospheric pressure, the negative pore pressure (u) within the sample is given by the following Eq. (1) (Noorany and Seed, 1965), in which σ'_{vo} is the effective overburden pressure, K_0 the coefficient of earth pressure at rest and A_s Skempton's pore pressure coefficient.

$$u = -K_0\sigma'_{vo} - A_s\,(\sigma'_{vo} - K_0\sigma'_{vo}) \tag{1}$$

The effective pressure (σ'_{ps}) of the perfect sample (Ladd and Lambe, 1963) subjected to the complete release of total stress is therefore equal to minus u and is determined by Eq. (2);

$$\sigma'_{ps} = \sigma'_{vo}\,(K_0 + A_s\,(1 - K_0)) \tag{2}$$

In Shogaki's basic method (Shogaki et al., 1995a; Shogaki and Maruyama, 1998) for estimating *in-situ* undrained shear strength, two parameters were considered. They are the ratio of effective pressure to the maximum value of S_0 as Eq. (3) and the ratio (Rq_u) of q_u of other samples to that ($q_{u(max)}$) of q_u of the high quality sample within the sampling tube.

$$\sigma'_{ps}/S_0 \tag{3}$$

The ratio of the effective pressure of a perfect sample to the S_0 is equal to one. The Rq_u of the sheared samples are plotted versus the ratio of effective pressure expressed by Eq. (3). The Rq_u of the perfect sample for a ratio of effective pressure of one can be extrapolated using the data points in such a plot. *In-situ* q_u value ($q_{u(I)}{}^*$) can be obtained from the $q_{u(max)}$ times Rq_u. For practical use, the mean consolidation pressure (p_m) is used instead of σ'_{ps}, as shown in Eq. (4), in which K_0 is assumed as 0.5 and A_s is defined by u_f/q_u. The u_f is suction under the axial strain corresponding to q_u.

$$p_m = \frac{\sigma'_{vo} + 2\sigma'_{vo}K_0}{3} = \frac{2}{3}\sigma'_{vo} \tag{4}$$

There are problems in comparing the relationship between the p_m and S_0 values since the S_0 value includes not only the residual effective stress but also the matrix suction and seepage pressure suction. However, it was determined through testing that the effects of matrix suction and seepage pressure suction on the S_0 values are negligible, since the S_0 values for remolded clay are very small and about 0.3 kPa, unrelated to soil types.

Shogaki's basic method as described above, in which the *in-situ* strength can be estimated by measuring q_u and S_0, is easier than Mitachi's from a practical point of view. However, these methods also require several specimens with different degrees of sample disturbance. It is important in engineering practice that the testing method for undrained shear strength of clayey soils be simple because of the design parameters set in foundation construction design procedures.

Table 1. Indexes values and mechanical properties of samples

Sample	$\overline{w}_n$ (%)	I_p	σ'_{vo} (kPa)	$\overline{q}_u$ (kPa)	$\overline{S}_0$ (kPa)
Aoumi 1	90	52	166	163	65
Aoumi 2	95	52	166	168	59
Ariake 3	138	68	39	22	5
Ariake 5	124	54	46	31	6
Ariake 7	112	53	52	38	8
Hachirogata 2	157	101	50	33	7
Hachirogata 4	168	110	56	33	7
Hachirogata 5	158	100	62	40	10
Hachirogata 7	140	84	67	38	10
Iwai 1-5	636	167	18	24	3
Iwai 1-7	128	74	17	19	4
Iwai 2-5	467	226	14	23	4
Iwai 2-6	589	370	14	32	5
Iwai 2-9	81	34	19	15	2
Iwai 5-3	398	306	14	23	5
Iwai 5-4	593	289	14	29	3
Iwai 5-5	117	61	18	18	3
Iwakuni 16	80	59	130	91	19
Kahokugata	104	88	174	102	32
Kasaoka 8	95	72	34	44	16
Kobe	40	28	196	119	44
Kumamoto 9	91	46	87	66	13
Kumamoto 15	96	57	143	97	35
Mito 2	63	39	133	128	31
Mito 13	52	33	163	140	44
Sakura 8	113	50	101	83	39
Sakura 12-1	108	55	101	85	24
Sakura 12-2	110	62	74	51	18
Bothkennar	61	50	102	119	40
Kimhae 4-7	40	26	95	92	27
Kimhae 4-15	55	34	151	104	25
Kimhae 7-15	64	40	154	74	28

In-situ Undrained Shear Strengths Estimated from Shogaki's Basic Method

The ratios of $q_{u(I)}{}^*$ to $2c_{u(I)}$ for 24 samples listed in Table 1, excluding Iwai deposits, are plotted against the I_p and q_u in Figs. 4 and 5 respectively, where the $c_{u(I)}$ is the *in-situ* undrained shear strength measured from CK_0UC under *in-situ* preconsolidation pressure ($\sigma'_{p(I)}$) estimated from Shogaki's method (Shogaki, 1996). The $c_{u(I)}$ value can be considered as an appropriate value for the *in-situ* shear strength from the examinations of the K_0 value under K_0-consolidation, c_u/p and effective stress paths (Shogaki and Nochikawa, 2004). The $q_{u(I)}{}^*/2c_{u(I)}$ values are almost constant in the range of $I_p = 26 \sim 110$ and $q_u = 12 \sim 178$ kPa and unrelated to I_p and q_u values. The mean

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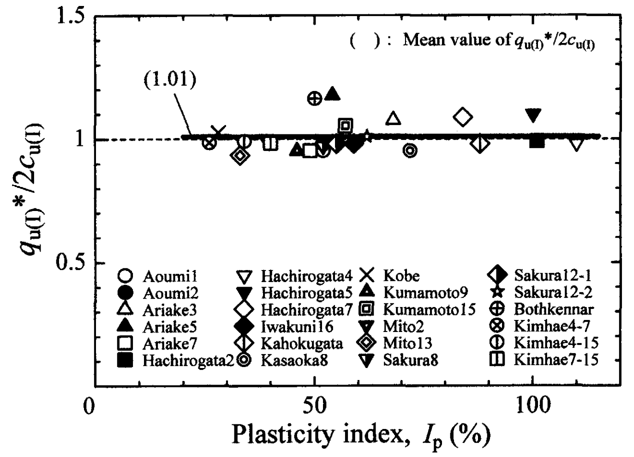

Fig. 4. Relationship between $q_{u(I)}{}^*/2c_{u(I)}$ and I_p

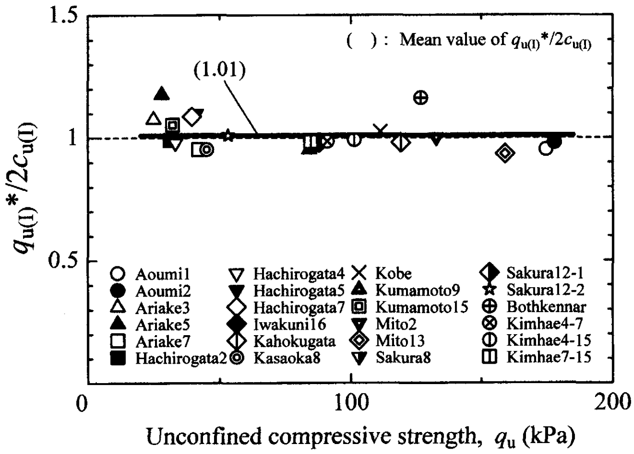

Fig. 5. Relationship between $q_{u(I)}{}^*/2c_{u(I)}$ and q_u

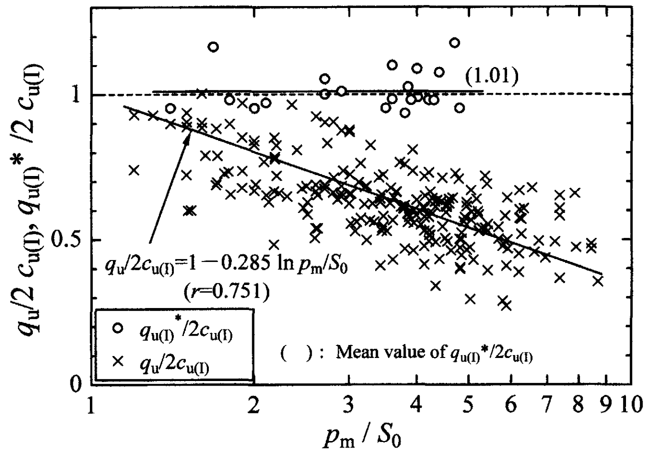

Fig. 6. Relationships between the ratios of q_u and $q_{u(I)}{}^*$ to $2c_{u(I)}$ and p_m/S_0

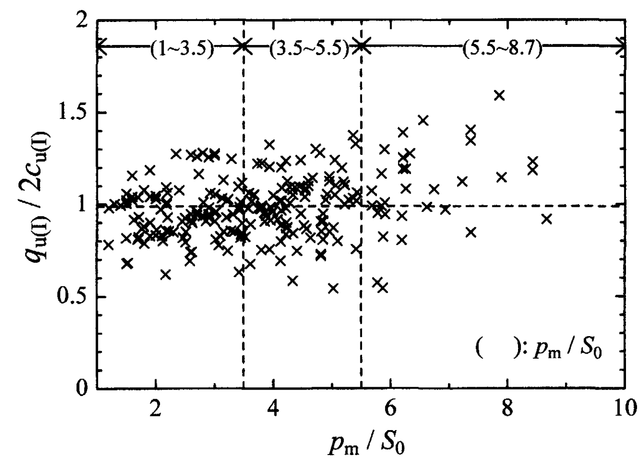

Fig. 7. Relationship between $q_{u(I)}/2c_{u(I)}$ and p_m/S_0

value of this ratio is 1.01 and $q_{u(I)}{}^*$ gives a similar undrained shear strength as $2c_{u(I)}$ in the wide I_p and q_u values. Therefore, it can be seen that Shogaki's basic method is appropriate for the sample used in Figs. 4 and 5. The relationship between the ratios of q_u and $q_{u(I)}{}^*$ to $2c_{u(I)}$ and the logarithm of p_m/S_0 is shown in Fig. 6. The mean value of the ratio of $q_{u(I)}{}^*$ to $2c_{u(I)}$ is 1.01, as shown in Figs. 4 and 5. Namely, the $q_{u(I)}{}^*$ are similar to $2c_{u(I)}$ and also unrelated to the p_m/S_0 values or sample disturbance. However, the $q_u/2c_{u(I)}$ values lineally decrease with increasing p_m/S_0 caused by sample disturbance. The q_u (×) values obtained from UCT are in the range of (27~99)% of $2c_{u(I)}$ and caused by stress release, sample disturbance, etc. from soil sampling to testing. To wit, the change from $c_{u(I)}$ is not unique for p_m/S_0 or sample disturbance. Figure 6 shows the lower reliability of q_u for design results and evaluating sample quality by measuring specimen suction since the samples used in Fig. 6 were taken for development of practical construction designs.

IMPROVED METHOD FOR ESTIMATING *IN-SITU* UNDRAINED SHEAR STRENGTH BY SHOGAKI

The regression line for the plots (×) of UCT in Fig. 6 is given as Eq. (5) and the correlation coefficient (r) is 0.751.

$$q_u/2c_{u(I)} = 1 - 0.285 \ln p_m/S_0 \tag{5}$$

This equation is obtained from 231 specimens at nineteen different Japanese sites as listed in Table 1, excluding Kahokugata and Iwai, Kimhae in Korea and Bothkenner in the United Kingdom. The relationship between $q_u/2c_{u(I)}$ and p_m/S_0 is unrelated to soil types and locations. Therefore, the *in-situ* undrained shear strength ($q_{u(I)}/2$) of natural clay deposits can be estimated from the q_u value multiplied by the reciprocal number of $q_u/2c_{u(I)}$ of the Eq. (5) using the S_0 and q_u values obtained from UCT for a specimen. This method for estimating *in-situ* undrained shear strength of natural deposits avoids the problem described previously for estimating $q_{u(I)}{}^*$.

The relationship between $q_{u(I)}/2c_{u(I)}$ and p_m/S_0 is shown in Fig. 7. The $q_{u(I)}$ values are estimated from Eq. (5) using the measured S_0 and q_u. The $q_{u(I)}/2c_{u(I)}$ values changed in

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Table 2. Statistical values of the ratios of $q_{u(I)}$ to $2c_{u(I)}$ shown in Fig. 8

p_m/S_0	n	Mean value	s
1.0 ~ 3.5	102	0.951	0.15
3.5 ~ 5.5	97	0.994	0.16
5.5 ~ 8.7	32	1.085	0.24
1.0 ~ 5.5	199	0.977	0.16
1.0 ~ 8.7	231	1.007	0.18

n: Number of specimens, s: Standard deviation

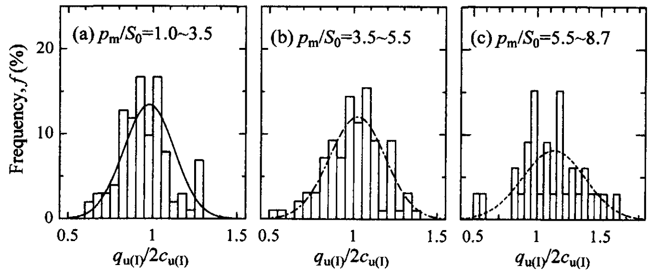

Fig. 8. Frequency and distribution curves of $q_{u(I)}/2c_{u(I)}$

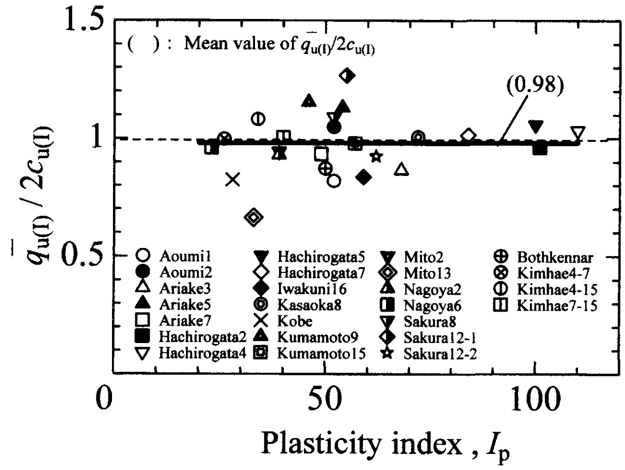

Fig. 9. Relationship between $q_{u(I)}/2c_{u(I)}$ and I_p

the range of 0.62 to 1.28, where p_m/S_0 was less than 3.5. However, the scatter of this ratio increases where p_m/S_0 is greater than 3.5. The frequency and distribution curves and the statistical values of the ratio of $q_{u(I)}$ to $2c_{u(I)}$ are shown in Fig. 8 and Table 2 for each plot classified with $p_m/S_0=(1.0\sim3.5)$, $(3.5\sim5.5)$ and $(5.5\sim8.7)$. The n and s in Table 2 indicate the number of specimens and standard deviation of $q_{u(I)}/2c_{u(I)}$, respectively. The s value for all specimens is 0.18 and for the data of $p_m/S_0=1.0\sim5.5$, is 0.16. It is determined that the distribution curves in Fig. 8 are regarded as normal through verification of the congruence. Namely, the $q_{u(I)}/2c_{u(I)}$ have a deviation of 0.98 ± 0.16 as a mean value where $p_m/S_0=1.5\sim5.5$. This means that the estimated $q_{u(I)}$ value, using Eq. (5), is in the range of $(-18\sim+14)$% of $2c_{u(I)}$, measured from CK_0UC under the consolidation pressure of $\sigma'_{p(I)}$. The coefficient of variation of $q_{u(I)}/2c_{u(I)}$ is similar to those of

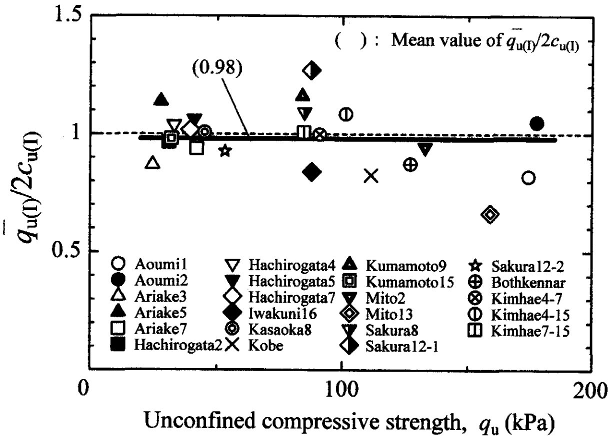

Fig. 10. Relationship between $q_{u(I)}/2c_{u(I)}$ and q_u

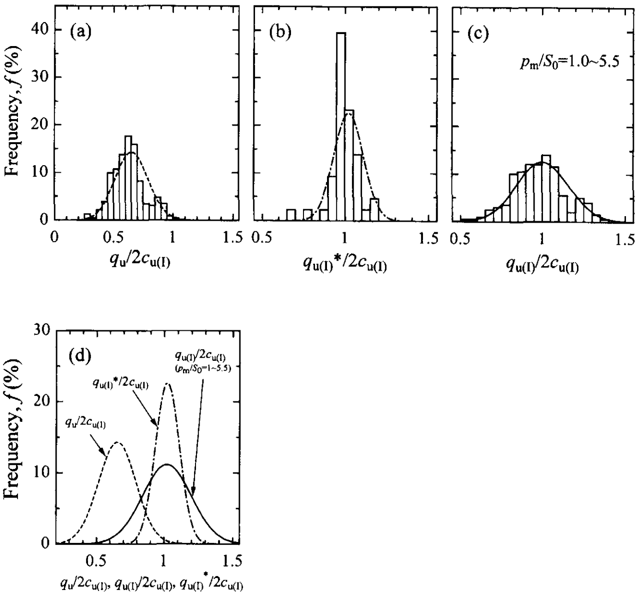

Fig. 11. Frequency and distribution curves of the ratios of q_u, $q_{u(I)}^*$ and $q_{u(I)}$ to $2c_{u(I)}$

glass and soft iron (Yokobori, 1974). It was confirmed by Matsuo and Shogaki (1988) and Shogaki et al. (1995b) that the coefficients of variation for undisturbed Holocene clay deposits and their remolded clay are (8 ~ 17)% and similar to those of $q_{u(I)}/2c_{u(I)}$. The small s value of $q_{u(I)}/2c_{u(I)}$ means that the reliability of $q_{u(I)}$ estimated from Eq. (5) is higher than the measured q_u. Therefore, it can be seen that the improved method for estimating $q_{u(I)}$ by using a regression equation, as shown in Eq. (5), is appropriate, based on the samples used in this study.

The ratios $(\bar{q}_{u(I)}/2c_{u(I)})$ of the mean value $(\bar{q}_{u(I)})$ of $q_{u(I)}$ to $2c_{u(I)}$ are plotted against the I_p and q_u in Figs. 9 and 10, respectively. The $q_{u(I)}/2c_{u(I)}$ values are unrelated to I_p, q_u and p_m/S_0 and the mean value of this ratio is 0.97, in the range of $I_p=26\sim110$ and $q_u=12\sim178$ kPa.

The distribution curves and statistical values of the ratios of q_u, $q_{u(I)}^*$ and $q_{u(I)}$ to $2c_{u(I)}$ are shown in Fig. 11 and Table 3. The mean values of the ratios for q_u, $q_{u(I)}^*$ and $q_{u(I)}$ are 0.629, 0.998 and 0.977 and the s values of

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Table 3. Statistical values of the ratios of q_u, $q_{u(I)}^*$ and $q_{u(I)}$ to $2c_{u(I)}$ for 24 clays excluding Iwai deposits

Ratio	n	Mean value	s
$q_u/2c_{u(I)}$	231	0.629	0.14
$q_{u(I)}^*/2c_{u(I)}$	23	0.998	0.10
$q_{u(I)}/2c_{u(I)}$	199	0.977	0.16

n: Number of specimens, s: Standard deviation

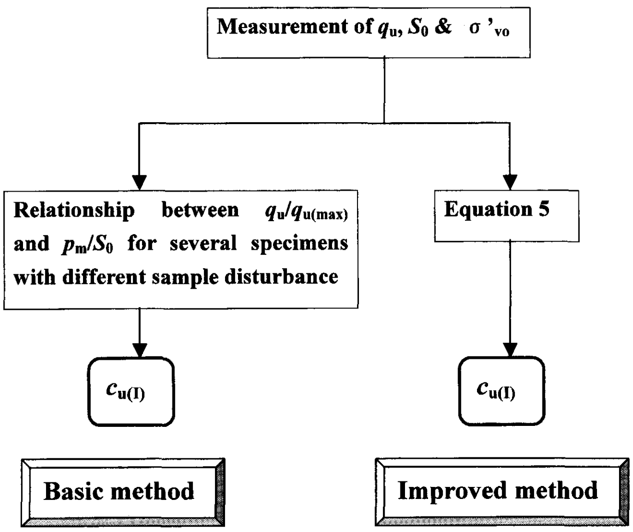

Fig. 12. Estimating flow for $c_{u(I)}$ from the basic and improved methods

those ratios are 0.14, 0.10 and 0.16, respectively. Therefore, it can be seen that the improved method is better from a time and economic standpoint than the basic method.

Figure 12 shows the flow estimation for $c_{u(I)}$ from the basic and improved methods. The improved method has an advantage in testing time and cost over the basic method since the improved method can estimate similar $c_{u(I)}$ values as the basic method for the mean value, using the q_u and S_0 values obtained from a specimen. However, the basic method requires several specimens for estimating $c_{u(I)}$, *but has an advantage on the estimated accuracy since the standard deviation of the estimated value is slightly smaller than that of the improved method.*

APPLICABILITY OF THE IMPROVED METHOD FOR IWAI ORGANIC AND SOFT CLAY

The undrained shear strengths for a sample obtained from the penetration tube of the Standard Penetration Test (SPT) are lower than those of tube samplers in general due to sample disturbance. Therefore, it is mainly used for determining basic content and index properties, etc. The SPT is widely used all over the world for site investigation and if the strength and consolidation properties of the sample obtained from the SPT can be measured, it is advantageous in a practical engineering sense.

In this section, the applicability of the improved

Table 4. Indexes and strength properties for Iwai deposits

Bore hole	Sampler	z (m)	w_n (%)	ρ_t (g/cm^3)	q_u (kPa)
1	75-mm	4.4～4.8	387～487	1.01～1.07	18～21
2	45-mm	4.4～5.1	374～492	1.03～1.09	17～33
5	50-mm	4.4～4.7	406～494	1.01～1.06	12～22
SPT		4.5～5.0	350～379	1.06～1.07	17～20
1	75-mm	7.4～7.8	126～132	1.35～1.38	13～22
2	45-mm	7.5～8.1	56～84	1.49～1.65	16～26
5	Cone	7.4～7.8	104～147	1.34～1.42	15～23
SPT		7.5～8.0	106～136	1.34～1.41	13～19

method on estimating *in-situ* undrained shear strength is examined for highly organic and clayey Holocene soils.

Soil Sampling, Soil Samples and Test Procedures

The undisturbed soil samples were obtained from the Holocene and Pleistocene clay deposits located in Iwai City using the cone (Shogaki et al., 2004a; Shogaki et al., 2004b), 45-mm (Shogaki, 1997a; Shogaki et al., 2002b; Shogaki and Sakamoto, 2004), 50-mm and 75-mm samplers (JGS-1221, 1995) and the disturbed soil samples were obtained from the SPT sleeve (Shogaki et al., 2004a). The sampling depths (z) were 4 m to 8 m below the ground surface. These samplers were used at about the same depths in different boreholes. The major machinery components were the same for these samplers. The sample recovery ratios were 100% for all sampling.

The indexes and strength properties of these soils are shown in Table 4 together with the samples used in this section. The w_n and q_u are in the range of (56～494)% and (12～33) kPa respectively and these are classified as highly organic and high plasticity clays for the depths shown. The ignition loss and pH values of Iwai organic soils are (42～73)% and (6.1～6.2), respectively.

The sleeves are brassware with a 35 mm inner diameter, 100 mm in height and 2 mm in thickness and five of them are contained in the split-barrel penetration tube. Two S specimens can be taken from a sleeve, as shown in Fig. 13.

Unconfined Compressive Strength Properties

The results of UCT on the samples obtained from the tube and SPT samplers for the z of (4.5～5) m and (7.5～8) m are shown in Figs. 14 and 15. The boreholes 1, 2 and 5 shown in Figs. 14 and 15 show the test results for the Holocene soil samples obtained from the 75-mm, 45-mm and cone samplers. It was confirmed that the sample quality obtained from all samplers, except the SPT, was similar (Shogaki et al., 2004a). The shaded plots, as shown in Figs. 14 and 15, have similar w_n and ρ_t values, therefore the strength properties are compared for the plots of these specimens.

Figures 16, 17 (a) and (b) show the relationships between the pore water pressure (u) measured at the base

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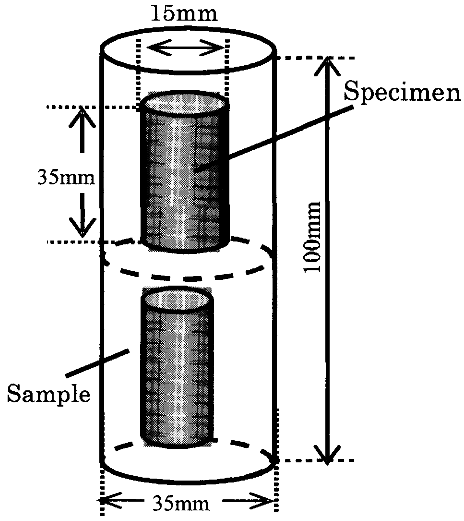

Fig. 13. Location of specimens for sample of SPT sleeve

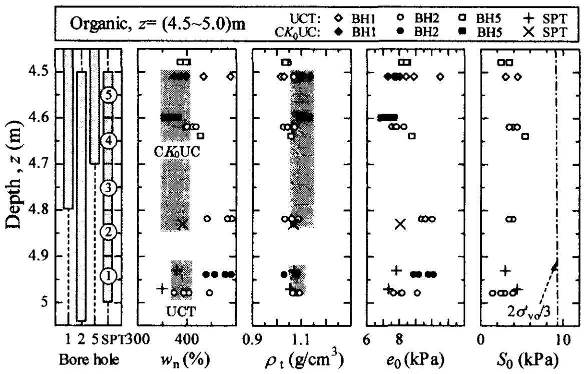

Fig. 14. Results of UCT (Iwai organic soil)

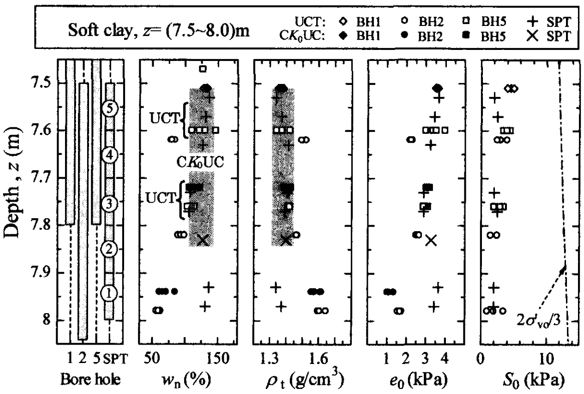

Fig. 15. Results of UCT (Iwai soft clay)

of the specimen, the stress (σ) and the axial strain (ε_a) for the same specimens and the test results are also listed in Tables 5 and 6. The suction under shear is represented as the u in Figs. 16, 17 (a) and (b) since the suction under shear becomes plus, with small S_0 values. Therefore, the u values at the $\varepsilon_a=0\%$ are S_0 values. The S_0, q_u and E_{50} values of each tube are given in Tables 5, 6(a) and (b). The test results of different sleeve samples in the same

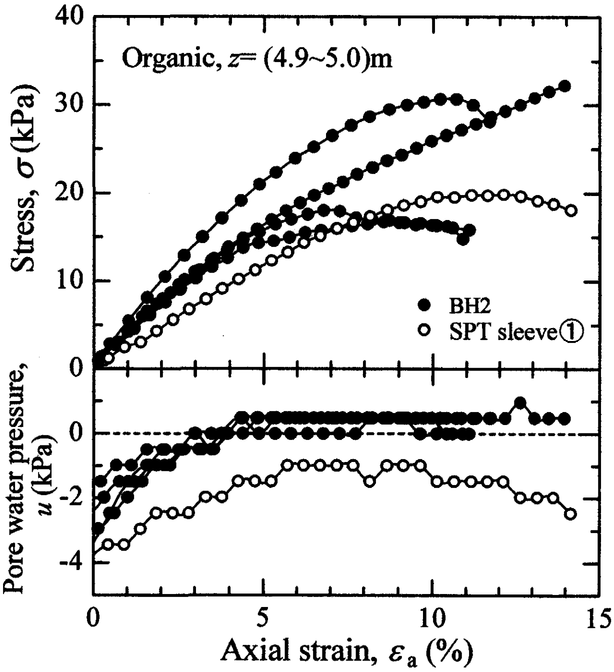

Fig. 16. Relationships between pore water pressure, stress and axial strain (Iwai organic soil)

split-barrel penetrator shown in Figs. 17 (a) and (b) are shown as sleeves (⑤) and (③) of the penetrator tube in Fig. 15. The relationships between σ and ε_a of the specimens obtained from the 45-mm sampler and the penetrator tube sleeves are similar for the specimens obtained from sleeve (③). However the S_0, q_u and E_{50} values from the sleeves are smaller than those of the 45-mm and 50-mm samplers for Figs. 16 and 17(a). These have large sample disturbance and are caused by sampling.

Triaxial Properties and Sample Quality

The K_0 values measured from K_0-consolidation and rate of strength increase (c_u/p) after K_0-consolidation for organic soil and soft clay are plotted against the σ'_a in Figs. 18 and 19 respectively, where p is σ'_a and $\sigma'_{p(I)}$ is estimated *in-situ* preconsolidation pressure from Shogaki's method (Shogaki, 1996). The K_0 values converged to a certain value in the area of greater than the $\sigma'_{p(I)}$ value. On the other hand, the K_0 values from the sleeve sample are smaller than those of the tube sample under the same σ'_a. These phenomena can be interpreted as stress change caused by a change of soil structure during drainage as follows (Shogaki and Nochikawa, 2004);

1) The specimen (◑), in which the soil particles are arranged isotropically by sample disturbance, is deformed in the direction of drainage.
2) If the soil rigidity increases when the void ratio decreases with drainage, the K_0 value increases with increasing σ'_a value since the K_0 condition can not be maintained without a corresponding increase in the lateral pressure, which in effect increases the σ'_a value.

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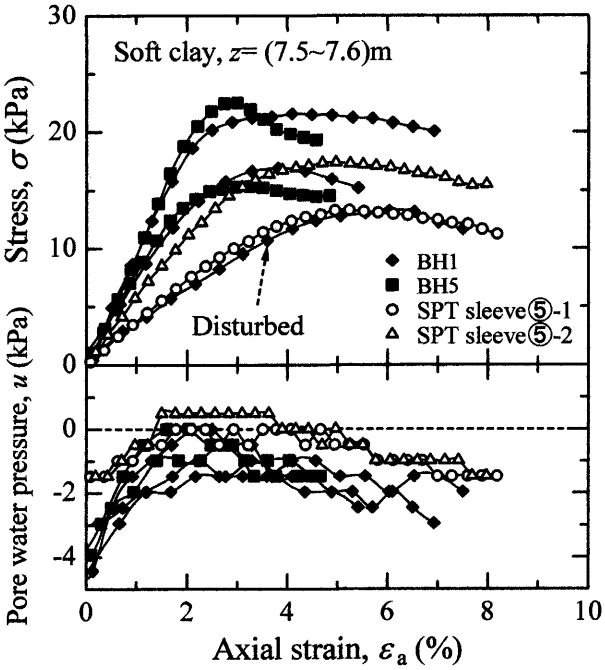
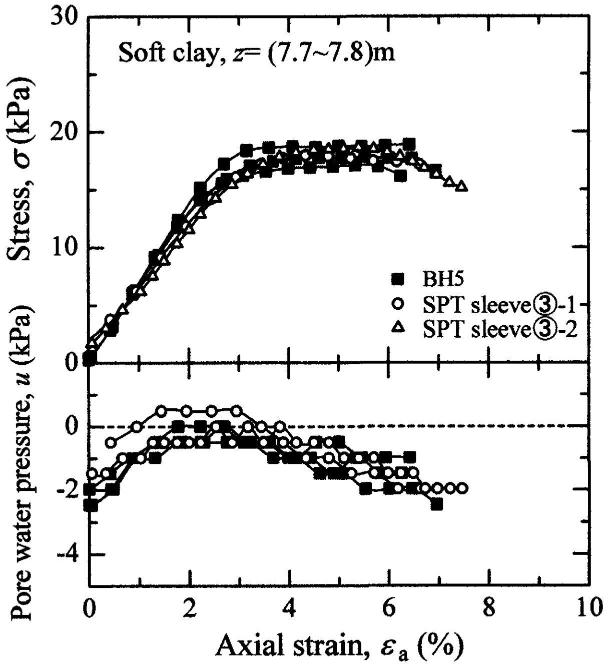


(a) $z=(7.5\sim7.6)$ m (sleeve ⑤)



(b) $z=(7.7\sim7.8)$ m (sleeve ③)

Fig. 17. Relationships between pore water pressure, stress and axial strain (Iwai soft clay)

3) For the undisturbed soil, the soil displays a plastic behavior when σ_a' becomes about the $\sigma_{p(I)}'$ value and the amount of void ratio change of each specimen becomes similar, unrelated to sample disturbance.

The c_u/p values in the area of small σ_a' values are larger and have an overconsolidated behavior. However, they decrease with increasing consolidation. The c_u/p values of the sleeve sample, in near or greater areas than $\sigma_{p(I)}'$, are similar to those from tube samplers under the same σ_a'. Namely, it can be seen from Figs. 18 and 19 that the undrained triaxial compression test after K_0-consolidation, around the $\sigma_{p(I)}'$ value, can eliminate sample disturbance caused by the SPT and penetration tube sampling, as described above.

The effective stress paths of CK_0UC and UCT under $\dot{\varepsilon}_a=1.0\%/\text{min}$ for organic soil and soft clay are shown in Figs. 20 and 21. The effective stress paths of the UCT show an overconsolidated behavior and changed to a normally consolidated behavior of CK_0UC with increasing σ_a' values. The stress level of $\sigma_{p(I)}'$ is located at the intersection of their rupture envelope lines, unrelated to the soils. Figures 20 and 21 show the validity of the estimated $\sigma_{p(I)}'$ value and the UCT method with suction measurement, using an S specimen.

Table 5. Results of UCT (Iwai organic soil)

Specimen		w_n (%)	ρ_t (g/cm^3)	q_u (kPa)	E_{50} (MPa)	ε_f (%)	S_0 (kPa)
BH#-No.	*						
2-1	●	374	1.09	30.8	0.46	10.6	3.9
2-2	●	404	1.09	33.2	0.33	15.0	3.0
2-3	●	447	1.07	18.1	0.40	7.1	1.5
2-4	●	394	1.07	16.8	0.35	8.9	2.5
**①	○	379	1.07	19.9	0.24	11.5	4.4

*: Symbols used in Fig.16, **: SPT sleeve

Table 6. Results of UCT (Iwai soft clay)

(a) $z=(7.5\sim7.6)$ m

Specimen		w_n (%)	ρ_t (g/cm^3)	q_u (kPa)	E_{50} (MPa)	ε_f (%)	S_0 (KPa)
BH#-No.	*						
1-1	◆	130	1.38	21.6	0.96	4.2	3.9
1-2	◆	127	1.35	16.9	0.74	3.8	4.9
1-3	◆	125	1.38	13.3	0.29	6.6	3.4
5-1	■	137	1.34	15.4	0.81	3.2	4.4
5-2	■	120	1.42	22.7	0.96	2.9	3.9
**⑤-1	○	136	1.34	13.3	0.37	5.2	2.0
**⑤-2	△	131	1.37	17.4	0.57	5.2	2.5

*: Symbols used in Fig.17(a), **: SPT sleeve

(b) $z=(7.7\sim7.8)$ m

Specimen		w_n (%)	ρ_t (g/cm^3)	q_u (kPa)	E_{50} (MPa)	ε_f (%)	S_0 (KPa)
BH#-No.	*						
5-1	■	110	1.42	18.9	0.70	5.6	2.0
5-2	■	107	1.42	17.1	0.69	5.6	3.4
5-3	■	104	1.42	17.8	0.67	5.7	3.0
**③-1	○	108	1.41	18.1	0.63	4.4	2.0
**③-2	△	106	1.39	18.7	0.60	5.0	2.5

*: Symbols used in Fig.17(b), **: SPT sleeve

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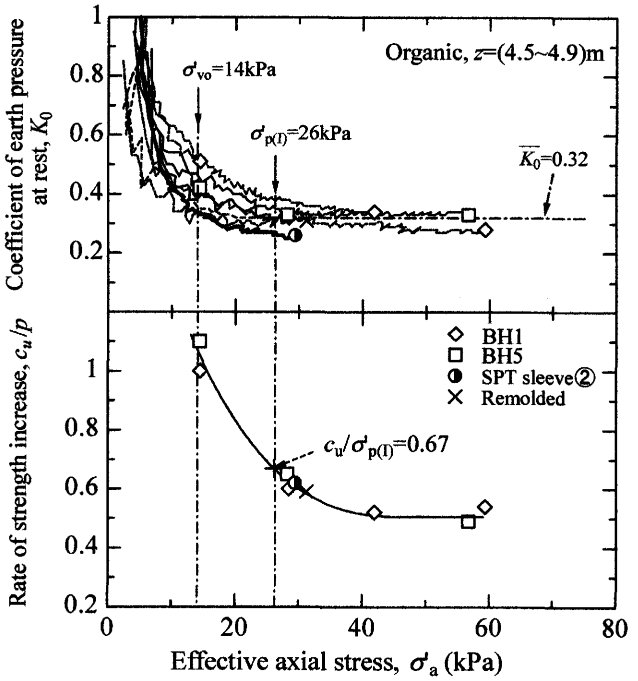

Fig. 18. Relationships between c_u/p, K_0 and σ'_a (Iwai organic soil)

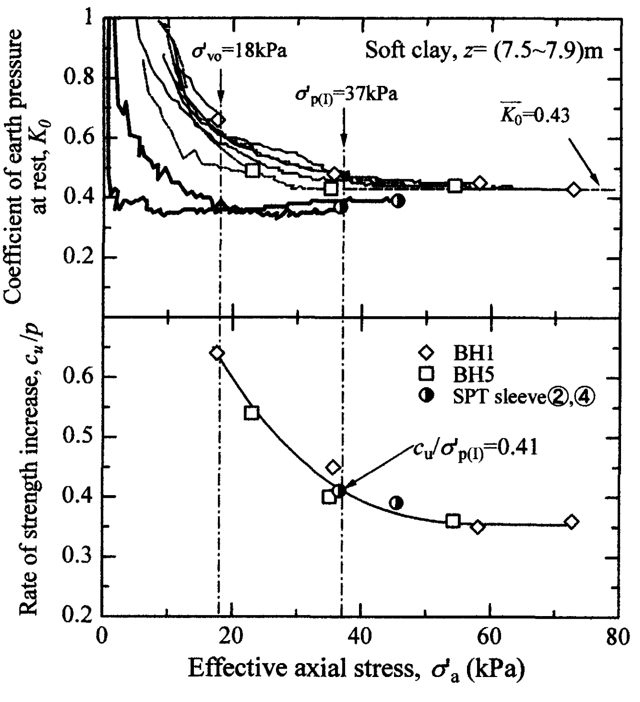

Fig. 19. Relationships between c_u/p, K_0 and σ'_a (Iwai soft clay)

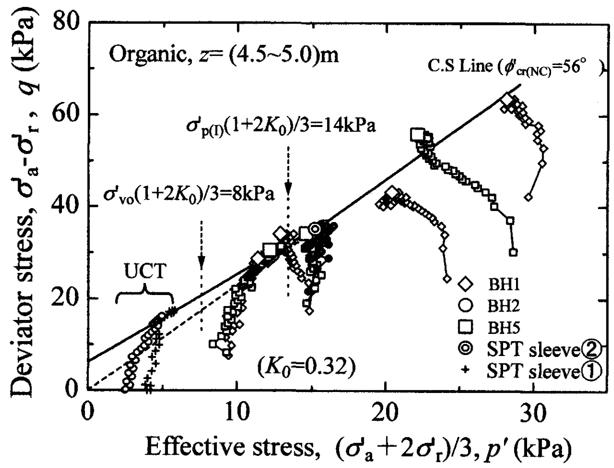

Fig. 20. Effective stress paths (Iwai organic soil)

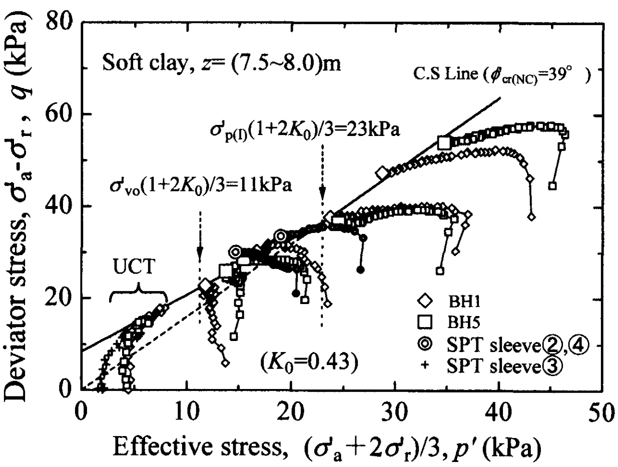

Fig. 21. Effective stress paths (Iwai soft clay)

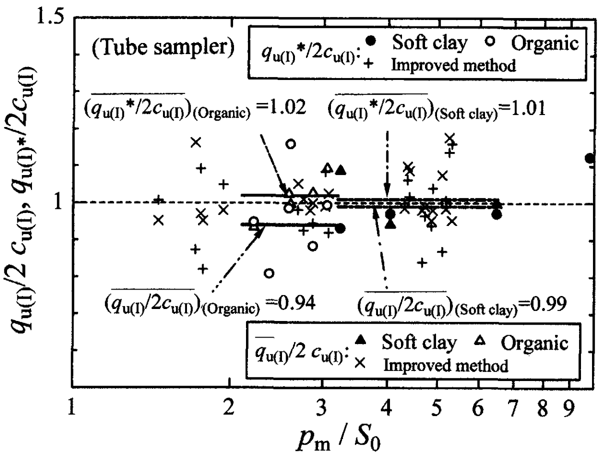

Fig. 22. Relationships between the ratios of $q_{u(I)}$ and $q_{u(I)}{}^*$ to $2c_{u(I)}$ (Iwai deposits)

The ratios of the mean value of the estimated $q_{u(I)}$ and $q_{u(I)}{}^*$ for the $2c_{u(I)}$ under the $\sigma'_{p(I)}$ value from CK_0UC are plotted against the p_m/S_0 and shown in Fig. 22. Figure 22 also shows the estimated results (+ and ×) obtained from the improved method. The mean values of ratios of $q_{u(I)}$ and $q_{u(I)}{}^*$ values to 2 $c_{u(I)}$ are 0.94 and 1.02 for the highly organic soils and 0.99 and 1.01 for the soft clay. To wit, the mean values of $q_{u(I)}$ and $q_{u(I)}{}^*$ are similar to $2c_{u(I)}$ for the soft clay and organic soil, the $q_{u(I)}$ values give a conservative result for short-term stability analysis for Iwai deposits and these ratios are unrelated to the p_m/S_0 values.

The ratios of the q_u, $q_{u(I)}$ and $q_{u(I)}{}^*$ to the $2c_{u(I)}$ values are plotted against z in Fig. 23. The mean values of $q_u/2c_{u(I)}$

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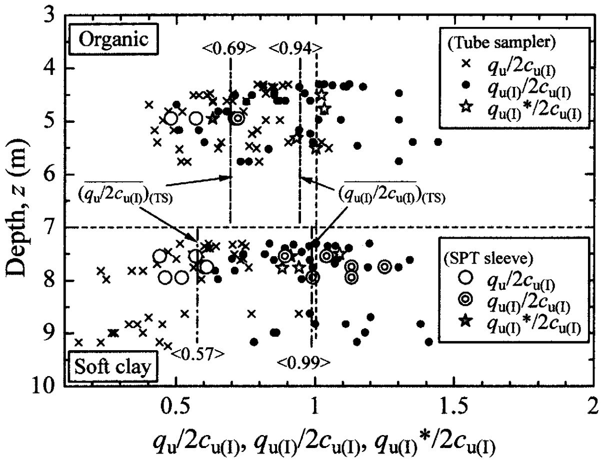

Fig. 23. Relationships between the ratios of q_u, $q_{u(I)}$ and $q_{u(I)}^*$ to $2c_{u(I)}$ and depth (Iwai deposits)

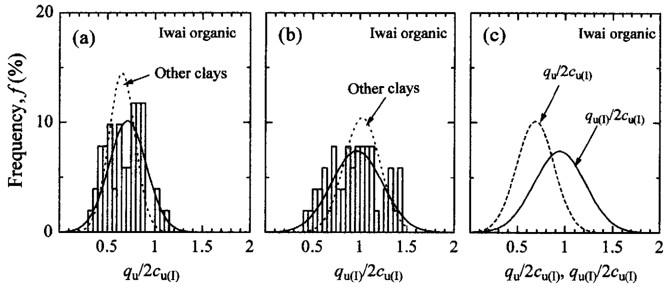

Fig. 24. Frequency and distribution curves of the q_u and $q_{u(I)}$ to $2c_{u(I)}$ (Iwai organic soil)

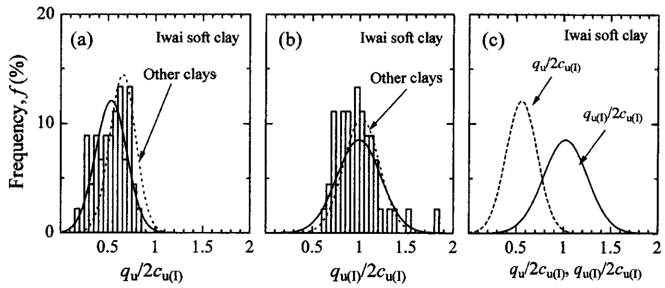

Fig. 25. Frequency and distribution curves of the q_u and $q_{u(I)}$ to $2c_{u(I)}$ (Iwai soft clay)

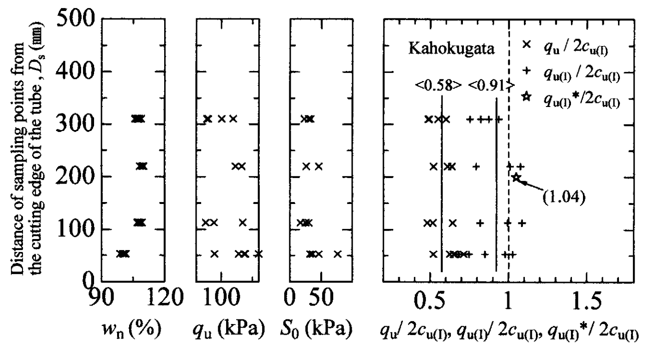

Fig. 26. Results of UCT and ratios of q_u, $q_{u(I)}$ and $q_{u(I)}^*$ to $2c_{u(I)}$ (Kohokugata clay)

Table 7. Statistical values of the ratios of q_u, $q_{u(I)}^*$ and $q_{u(I)}$ to $2c_{u(I)}$ (Iwai deposits)

(Iwai organic)

	n	Mean value	s
$q_u/2c_{u(I)}$	51	0.69	0.20
$q_{u(I)}^*/2c_{u(I)}$	5	1.02	—
$q_{u(I)}/2c_{u(I)}$	51	0.94	0.27

(Iwai soft clay)

	n	Mean value	s
$q_u/2c_{u(I)}$	45	0.57	0.16
$q_{u(I)}^*/2c_{u(I)}$	3	1.01	—
$q_{u(I)}/2c_{u(I)}$	45	0.99	0.23

(Other 23 clay)

	n	Mean value	s
$q_u/2c_{u(I)}$	231	0.63	0.14
$q_{u(I)}^*/2c_{u(I)}$	23	1.00	0.10
$q_{u(I)}/2c_{u(I)}$	199	0.98	0.16

n: Number of specimens, s: Standard deviation

are 0.69 and 0.57 for organic and soft clay respectively. On the other hand, the $q_{u(I)}^*$ and $q_{u(I)}$ values are also unrelated to z.

Table 7 shows the statistical values of the ratios of q_u, $q_{u(I)}^*$ and $q_{u(I)}$ to $2c_{u(I)}$ values for organic, soft clay and 23 different Japanese clays, including Bothkennar and Kimhae clays, as shown in Table 1. Figures 24 and 25 show the normal distribution curves of the ratios of q_u and $q_{u(I)}$ to $2c_{u(I)}$ values for organic and soft clay respectively, as shown in Table 7. However, Figs. (a) and (b) of those Figs. only show the normal distribution curves of 23 different Japanese soils with Iwai deposits to avoid complication. It can be seen from Table 7 and Figs. 23 and 24 that the q_u values are 31% and 43% of $c_{u(I)}$ values for Iwai organic and soft clay deposits respectively, as well as the other 23 clays. Therefore, the $q_{u(I)}$ and $q_{u(I)}^*$ values improve design reliability since the reliability of the mean value of q_u improves.

APPLICABILITY OF IMPROVED METHOD FOR KAHOKUGATA CLAY

Shogaki and Nochikawa (2004) examined the triaxial and consolidation properties of Kahokugata clay through the PTA, as well as this study, and the $c_{u(I)}$, $\sigma'_{p(I)}$, $C_{c(I)}$ and K_0 values are reported as 95 kPa, 230 kPa, 1.8 and 0.46, respectively. The results of UCT and the ratios of q_u, $q_{u(I)}$ and $q_{u(I)}^*$ to $2c_{u(I)}$ are plotted in Fig. 26. The mean values of these ratios are 0.58, 0.91 and 1.04, respectively and the $q_{u(I)}^*$ and $q_{u(I)}$ values are similar to $2c_{u(I)}$. However, the q_u value is 58% of $2c_{u(I)}$. The statistical values of the q_u and $q_{u(I)}$ are summarized in Table 8. The coefficient of

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Table 8. Statistical values of the ratios of q_u and $q_{u(I)}$ (Kohokugata clay)

	n	Mean value	* (%)
q_u	15	102	12.5
$q_{u(I)}$	15	176	13.6

n: Number of specimens, *: Coefficient of variation

variations of the q_u and $q_{u(I)}$ are 12.5% and 13.6% respectively, and are similar. Shogaki et al. (2002a) has reported the same result for Ariake clay. The coefficient of variation of the q_u value becomes greater with increasing sample disturbance (Matsuo and Shogaki, 1988). It was determined that the similar values of the coefficient of variation for the q_u and $q_{u(I)}$ values of Ariake and Iwai deposits are caused by the small number of specimens. Therefore, Shogaki's basic and improved methods for estimating *in-situ* undrained shear strength can be used for 28 Japanese clay deposits plus Bothkenner and Kimhae clays. The I_p and q_u values are in the range of 26 to 370 and 15 kPa to 168 kPa respectively, which are very wide ranges. However, the $q_{u(I)}{}^*$ or $q_{u(I)}$ values cannot be used directly for practical stability analysis since the sample disturbance is one of the factors influencing safety. Therefore, it is essential that effect of $q_{u(I)}$ or $q_{u(I)}{}^*$ on undrained shear strength in the formula for determining the safety factor, be reviewed by verifying the effect of sample disturbance on practical construction design.

CONCLUSIONS

The conclusions obtained in this study are summarized as follows:

1) An equation for estimating *in-situ* undrained shear strength ($q_{u(I)}$) of natural deposits was derived as $q_{u(I)}/2c_{u(I)}=1.0\text{–}0.285 \ln p_m/S_0$ through the unconfined compression test (UCT) and K_0 consolidated-undrained triaxial compression test (CK_0UC). The $q_{u(I)}$ of natural clay deposits can be estimated from the q_u value multiplied by the reciprocal number of $q_{u(I)}/2c_{u(I)}$ of the equation using the suction (S_0) and q_u obtained from UCT for a specimen, where $c_{u(I)}$ is *in-situ* shear strength measured from CK_0UC and p_m is two times the effective overburden pressure divided by three.
2) The $q_{u(I)}/2c_{u(I)}$ values were unrelated to I_p, q_u and p_m/S_0 and the mean value of these ratios was 0.98 in the range of $I_p=26\sim110$ and $q_u=12\sim178$ kPa. The mean values of the ratios for q_u, $q_{u(I)}{}^*$ and $q_{u(I)}$ to $2c_{u(I)}$ were 0.629, 0.998 and 0.977 and the standard deviation of those ratios was 0.14, 0.10 and 0.16, respectively. Therefore, it can be seen that the improved method is better from a time and economic standpoint than Shogaki's basic method.
3) The mean values of $q_{u(I)}/2c_{u(I)}$ were 0.94, 0.99 and 0.91 for Iwai organic, soft clay and Kahokugata clay, respectively. The coefficient of variations of the q_u and $q_{u(I)}$ values were $(13\sim14)$%, unrelated to soils, q_u and $q_{u(I)}$ values. Therefore, the applicability of the improved method newly developed in this study can be confirmed for Kahokugata and Iwai clays as well as Iwai organic soils.

It is essential that the effect of $q_{u(I)}$ or $q_{u(I)}{}^*$ on undrained shear strength, in the formula for determining the safety factor, be reviewed by verifying the effect of sample disturbance on practical construction design.

ACKNOWLEDGEMENTS

The applicability of an improved method for Iwai organic and soft clay was performed as a part of the Research Committee on Miniaturization, Accuracy and Design Reliability for Geotechnical Investigations and Lab Tests. The authors wish to express their sincere gratitude to the members of this research committee, to Mr. Yoshikazu Maruyama and Ryo Sakamoto, who were graduate students of the National Defense Academy, for their cooperation in experimental works and also to the Geotechnical Survey Laboratory of the Port and Airport Research Institute for the use of their Bothkennar clay sample in his research.

NOTATION

$c_{u(I)}$: *in-situ* undrained shear strength measured from the CK_0UC under $\sigma'_{p(I)}$
c_u/p: rate of strength increase
E_{50}: secant modulus
I_p: plasticity index
K_0: coefficient of earth pressure at rest
p_m: mean consolidation pressure defined as $2\sigma'_{vo}/3$
$q_{u(I)}$: *in-situ* q_u value estimated by Shogaki's improved method
$q_{u(I)}{}^*$: *in-situ* q_u value estimated by Shogaki's basic method
$q_{u(max)}$: maximum value of unconfined compressive strength
r: correlation coefficient
s: standard deviation
S_0: specimen suction
w_n: natural water content
$\phi'_{cr\,(NC)}$: effective internal friction angle under normary consolidation stage
ε_a: axial strain
ε_f: strain at failure
$\dot{\varepsilon}_s$: axial shear strain rate
ε_{vo}: volumetric strain
σ'_a: effective axial stress under CK_0UC test
σ'_p: preconsolidation pressure
$\sigma'_{p(I)}$: *in-situ* preconsolidation pressure estimated by Shogaki's method
σ'_v: effective vertical pressure under incremental loading oedometer test
σ'_{vo}: effective overburden pressure

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