A METHOD FOR CORRECTING CONSOLIDATION PARAMETERS FOR SAMPLE DISTURBANCE USING VOLUMETRIC STRAIN

TAKAHARU SHOGAKI¹⁾

ABSTRACT

Sample disturbance was investigated in terms of the volumetric strain of an oedometer specimen, ε_{vo} , at in-situ effective overburden pressure, σ'_{vo} , determined from the 24-hour $e - \log \sigma'_v$ curve. Unconfined compression and oedometer tests were performed on samples of natural deposits from ten different sites in Japan.

The ε_{vo} value has a unique correlation with the plasticity index, I_p , σ'_{vo} , overconsolidation ratio, OCR, and degree of sample disturbance. The ε_{vo} values of the in-situ sample before intentional disturbance in the laboratory were in the range of 0.85% to 7.02%, and the average was 3.80%. The ε_{vo} values of the completely remolded sample were in the range of 7.52% to 23.07% and the average was 15.75%. The ε_{vo} value, resulting from intentional sample disturbance in the laboratory, is given as a function of the ratio of unconfined compressive strength, q_u , of sample disturbed in the laboratory to that of the in-situ sample. A procedure for obtaining values of strength and consolidation parameters of an in-situ soil, and a method for estimating the preconsolidation pressure, σ'_p , compression index, C_c , $e - \log \sigma'_v$ curve, and coefficient of consolidation, c_v , values for the in-situ soil are proposed from the I_p and the measured values of ε_{vo} , σ'_p , C_c , and c_v for disturbed samples in the standard oedometer test.

Key words: cohesive soil, consolidation, oedometer test, sample disturbance, sampling, shear strength, statistical analysis, unconfined compression test, volumetric strain (IGC: D5/D6)

INTRODUCTION

Various characterizations of sample disturbance in terms of undrained shear strength, s_u , have been proposed by many researchers such as Skempton and Sowa (1963), Okumura (1974a), Nakase et al. (1985), Matsuo and Shogaki (1988), Shimizu and Tabuchi (1993) and Shogaki (1995a). These studies focused on the direct application of the s_u value to the short term stability problem of a clay foundation under undrained conditions. When the preconsolidation pressure, σ'_p , is adopted for estimating the mobilized undrained shear strength (e.g., Ladd and Foott, 1974; Mesri, 1989), the effects of sample disturbance on both undrained shear strength and consolidation parameters should be established.

Shogaki and Kaneko (1994) proposed a method for correcting the statistical data obtained from the laboratory test results, including the s_u value and the consolidation parameters such as the σ'_p , coefficient of consolidation c_v , compression index C_c , coefficient of volume compressibility m_v , and coefficient of permeability k_v . This method can not evaluate the consolidation parameters for sample disturbance without unconfined compressive strength, q_u , data because in this method the relationship between

consolidation parameters and sample disturbance is expressed as a function of the ratio of the q_u values of the laboratory disturbed sample to that of the undisturbed sample, $R(q_u)$. In order to correct the consolidation parameters for the sample disturbance by this method, therefore, the $R(q_u)$ value from unconfined compression test for the samples with different degrees of sample disturbance must be determined. A guideline for the quality of specimen or sample disturbance based on the results of consolidation tests was proposed by Andresen and Kolstad (1977), and Lacasse and Berre (1988). The effects of sample disturbance on consolidation parameters, however, can not be determined based on these guidelines.

The deformation of samples under the same consolidation pressure, σ'_v , in the standard oedometer test increases with sample disturbance since the degree of rigidity against compression decreases with sample disturbance. The void ratio, e , therefore, under the same σ'_v value decreases with sample disturbance. These characteristics have been shown by the experimental results of Schmertman (1955) and Shogaki and Kaneko (1994), etc. Utilizing these properties in which the relationship between the e and the logarithm of the σ'_v in the standard

¹⁾ Associate Professor, Department of Civil Engineering, National Defense Academy, Hashirimizu, Yokosuka 239.
Manuscript was received for review on May 25, 1995.

Written discussions on this paper should be submitted before April 1, 1997 to the Japanese Geotechnical Society, Sugayama Bldg. 4 F, Kanda Awaji-cho 2-23, Chiyoda-ku, Tokyo 101, Japan. Upon request the closing date may be extended one month.

oedometer test reflect the effect of sample disturbances, the relationship between the volumetric strain, ε_{vo} , under the effective overburden pressure, σ'_{vo} , in a standard oedometer test and the sample disturbance was investigated by laboratory testing. A series of unconfined compression tests and oedometer tests were performed on natural deposits from ten different site locations in Japan.

This study showed that the ε_{vo} value has a unique correlation with I_p , OCR ($=\sigma'_p/\sigma'_{vo}$), and degree of sample disturbance. The ε_{vo} value caused by sample disturbance is given as a function of $R(q_u)$. A method is proposed for estimating the σ'_p , C_c , $e-\log \sigma'_v$ curve, and c_v values for the in-situ soil from the I_p and the measured ε_{vo} , σ'_p , C_c , and c_v values of disturbed samples using the standard oedometer test.

SOIL SAMPLES

The undisturbed soil samples used in this study were obtained from holocene marine clays located offshore at ten different sites in Japan. Field sampling was performed with a stationary piston sampler with an inner diameter of 75 mm to enhance the quality of the samples. For the Japanese soft clays, this sampler gives a quality similar to that sampled by the Laval type sampler (Tanaka et al., 1995).

The sites and index properties of these soils are shown in Fig. 1 and Table 1, respectively. These are saturated soils located from the Hokkaido to the Chugoku region in Japan. The plasticity index, I_p , and q_u values are in the range of 10% to 102% and 8 kPa to 220 kPa, respectively.

TEST PROCEDURES

The equipment used to cause disturbance to the soil samples, together with the procedure, is shown in Fig. 2 (Shogaki and Kaneko, 1994). As shown in Fig. 2(a), the ratio, R_a , of the cross sectional area, A_E , of a device with an inner diameter of the sampling tube changed to values of 1.0, 0.95, 0.90, 0.80 and 0.70. These samples were

designated S₁, S₂, S₃, S₄, and S₅, respectively. Sample S₁ is the original in-situ sample undisturbed in the laboratory and sample S₆ is completely remolded. Samples with a length of 80 mm were extruded from the edge of the sampling tube, in the order S₁, S₂ etc. It had been previously confirmed that the strength and deformation characteristics of the sample disturbed by this method are similar to those of a sample disturbed by in-situ sampling and preparation for laboratory tests (Shogaki et al., 1996).

The specimens used for the oedometer test were 60 mm in diameter and 20 mm in height. S specimens for the un-

Fig. 1. Site locations where soil samples were obtained

Table 1. Index properties of all clays

Sites	w_n (%)	w_L (%)	w_P (%)	I_p (%)	CF^{**} ($-2\mu\%$)	σ'_{vo} (kPa)	σ'_p/σ'_{vo}	q_u (kPa)	*
Kuwana	34-71	51-95	25-38	26-57	3-30	99-205	1.17-2.94	92-220	S
Haneda	105-108	113-120	46-48	64-73	47-54	160-223	0.50-1.08	66-189	S
Kawata	48	36	26	10	15	77	1.95	59	S
Shizunai	50	62	29	33	41	292	1.20	107	S
Urayasu	81-85	104-114	44-49	60-65	50-52	232-457	1.23-1.19	127-177	S
Amagasaki	47-72	59-105	26-41	33-69	30-54	191-241	0.68-1.36	130-138	S
Tokuyama	68-130	82-150	29-48	48-102	36-42	8-57	0.95-3.09	15-54	S
Kobe	68-89	81-109	29-34	52-75	32-42	18-33	1.99-2.73	37-60	O
Sennan	89-109	96-108	32-34	64-75	25-43	6-46	0.93-2.71	8-47	O
Hiroshima	48-72	59-93	22-39	26-64	16-26	29-69	2.44-6.22	101-121	O

*: Unconfined compression test, S: S Specimen, O: O Specimen

** : Clay fraction, less than $2\mu\text{m}$

Sample	d_d (cm)	A_d (cm ²)	R_a
S_1	7.50	44.18	1.00
S_2	7.31	41.95	0.95
S_3	7.12	39.86	0.90
S_4	6.72	35.47	0.80
S_5	6.30	31.44	0.70
S_6	Remolded soil		

a) Disturbing equipment

b) Procedure of disturbing

Fig. 2. Equipment and procedure for disturbing the soil samples (after Shogaki and Kaneko, 1994)

confined compression test were 15 mm in diameter and 35 mm in height. The specimens for the unconfined compression tests of the Kobe, Sennan and Hiroshima clay used the O specimen, 80 mm in height and 35 mm in diameter (Table 1). The O specimen is used as the standard size for unconfined compression tests in Japan. The effects of sample disturbance on the strength and consolidation properties were determined for the S specimens. There is no difference in shear strength characteristics between the S and the O specimens, which were tested for soils of I_p and q_u of the range of 20% to 85% and 10 kPa to 220 kPa, respectively (Shogaki, 1991b; Shogaki et al., 1993).

There are four to seven specimens for the unconfined compression test and one for the oedometer test. The $\bar{q}_u$ value is the mean value of q_u for the unconfined compression test specimens. 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 c_v were determined according to Taylor's method (Taylor, 1948). The values of C_c and σ'_p were determined from the $e - \log \sigma'_v$ curves corresponding to 24-hour compression. Mikasa's method was used to identify preconsolidation pressure on the 24-hour $e - \log \sigma'_v$ curve. These methods have been employed in Japan as the Japanese industrial standard for determining one-dimensional con-

solidation properties of soils (JIS A 1217-1990). It was shown by Okumura (1974b) that there is no difference in the σ'_p values between the Mikasa method and the Casagrande method (1936).

In the unconfined compression tests, the specimens were sheared at a strain rate of 1%/min. The value of q_u was determined to be the maximum stress corresponding to axial strain, ϵ_a , of less than 15%.

SAMPLE DISTURBANCE INDEX

A quantitative index of sample disturbance is necessary for correcting the measured consolidation parameters in standard oedometer tests. Figure 3 shows a schematic representation of the relationship between the e value and the logarithm of σ'_v in the standard oedometer test. The notation for some of the parameters is also shown. The void ratio, e_1 , under the σ'_{vo} value, was defined from the $e - \log \sigma'_v$ curve as shown in Fig. 3. The volumetric strain, ϵ_{vo} , is defined as

$$\epsilon_{vo} = \frac{e_0 - e_1}{1 + e_0} \times 100 (\%) \quad (1)$$

where the e_0 is the initial void ratio of the sample. In the in-situ soil under the σ'_{vo} value, there was no sample disturbance, therefore, the ϵ_{vo} value is 0 since the e_1 value is equal to the e_0 value. The e_1 value decreases with sample disturbance because the e value decreases with sample disturbance under the same σ'_v value (Schmertman, 1955; Shogaki and Kaneko, 1994), and the ϵ_{vo} value becomes larger. Therefore, the ϵ_{vo} value can be employed as an index in order to express the effect of sample disturbance.

TEST RESULTS

Quality of S_1 Sample

The axial strain at failure, ϵ_f , of the unconfined compression tests on S_1 samples are plotted versus I_p and q_u

Fig. 3. Definition of volumetric strain in standard oedometer test

values in Figs. 4 and 5, respectively. The ε_f values, in the range of 1.7% to 7.0% and with a mean value of 3.6%, appear to be independent of the I_p and q_u values.

The relationships between the values of ε_{vo} and q_u for S_1 samples are shown in Fig. 6. The OCR value for the Haneda clay, with $\varepsilon_{vo}=13.8\%$, is about 0.5 because the consolidation of this clay under a reclamation load has not yet been completed. The ε_{vo} values of the other clays

are in the range of 0.85% to 7.02%, with a mean value of 3.80%. Data reported by Shogaki (1995b) indicate a typical $(\varepsilon_{vo})_{EOP}/(\varepsilon_{vo})_{24\text{ hr}}$ of 0.70 for a number of soft clays. Where $(\varepsilon_{vo})_{24\text{ hr}}$ is defined from the 24 hour $e-\log \sigma'_v$ curves for ε_{vo} . $(\varepsilon_{vo})_{EOP}$ is defined from the $e-\log \sigma'_v$ curves corresponding to the "end of primary consolidation" (EOP) by the Casagrande method for ε_{vo} . The values of $(\varepsilon_{vo})_{EOP}$, therefore, are in the range of 0.60% to 4.90%, with a mean value of 2.70%. According to Specimen Quality Designation (SQD) of Mesri et al. (1994) these values correspond to a range of A (ε_{vo} less than 1%) to D (ε_{vo} in the range of 4% to 10%) with mean SQD of C (ε_{vo} in the range of 2% to 4%). The values of ε_{vo} for clays other than Haneda are independent of $\bar{q}_u$. The results of Figs. 4, 5, and 6 suggest that S_1 samples have a similar quality regardless of the values of I_p , q_u and ε_f .

Volumetric Strain of S_1 Sample

The relationships between the values of ε_{vo} and I_p , σ'_v and the OCR for S_1 samples are shown in Figs. 7, 8 and 9, respectively. For the soils from the same site, the values of ε_{vo} tend to increase with the increase in the I_p and σ'_v values. In general, ε_{vo} tends to decrease with an increase in OCR. Sample disturbance increases with depth of sampling. Normally consolidated "young" clays are

Fig. 4. Relationship between ε_f and I_p (S_0 Samples)

Fig. 5. Relationship between ε_f and q_u (S_0 Samples)

Fig. 7. Relationship between ε_{vo} and I_p (S_0 Samples)

Fig. 6. Relationship between ε_{vo} and $\bar{q}_u$ (S_0 Samples)

Fig. 8. Relationship between ε_{vo} and σ'_{vo} (S_0 Samples)

Fig. 9. Relationship between ε_{vo} and OCR (S_0 Samples)

most susceptible to sample disturbance because they possess weak interparticle loads.

Lacasse and Berre (1988) showed that the ε_{vo} value according to the high quality clay samples are (2.1~4.0)% for $OCR=(1.0\sim1.2)$, (1.2~3.3)% for $OCR=(1.2\sim2.0)$, and (0.5~2.0)% for $OCR=(3.0\sim8.0)$ as shown in Fig. 9. The plots of ε_{vo} in Fig. 9 are larger than those suggested by Lacasse and Berre (1988), possibly because ε_{vo} values by Lacasse and Berre (1988) correspond to the end-of-primary consolidation.

Effects of Sample Disturbance on Strength and Consolidation Properties

The relationship between the $R(\bar{q}_u)$ and the $R(\varepsilon_{vo})$ is shown in Fig. 10. Where the $R(\bar{q}_u)$ and $R(\varepsilon_{vo})$ are the ratio of $\bar{q}_u$ and ε_{vo} of each sample to the corresponding values for the S_1 . The $R(\varepsilon_{vo})$ values decreases linearly with the $R(\bar{q}_u)$ values, and this tendency is independent of q_u , I_p , and sites. The regression line can be expressed as follows:

$$R(\varepsilon_{vo}) = 3.78 - 2.78R(\bar{q}_u) \quad (2)$$

The $R(\varepsilon_{vo})$ of S_6 sample are in the range of 2.38 to 6.59, with an average value of 3.64.

Fig. 10. Relationship between $R(\varepsilon_{vo})$ and $R(\bar{q}_u)$ Fig. 11. Relationship between ε_{vo} and OCR (All samples)

Figure 11 shows the relationship between the ε_{vo} value and the OCR value for the disturbed sample and the S_1 sample. In Fig. 11, the plots are categorized as the S_1 , the S_6 , and the four types of $R(\bar{q}_u)$. The degree of sample disturbance can be explained from these categories. When the OCR and ε_{vo} values are known from the standard oedometer test, the approximate value of $R(\bar{q}_u)$ for an S_1 sample can be estimated from Fig. 11. Shogaki and Kaneko (1994) showed the change of the statistical properties of the undrained shear strength and the consolidation parameters for S_1 sample as a function of the $R(\bar{q}_u)$. A combination of this method and Fig. 11 from the ε_{vo} and OCR values in the standard oedometer test, therefore, provides a practical method for correcting the sample disturbance for the statistical properties of strength and consolidation parameters.

A PROCEDURE FOR CORRECTING THE MEASURED CONSOLIDATION PARAMETERS OF DISTURBED SAMPLES

Figure 12 shows the relationship between the σ'_p values and ε_{vo} values for the Kuwana clay (sample K-13). When the ε_{vo} value increases with sample disturbance, the σ'_p value decreases because the degree of rigidity against compression decreases with sample disturbance. According

Fig. 12. Relationship between σ'_p and ε_{vo} (Kuwana clay, K-13)

to this extrapolation in Fig. 12, the σ'_p value of in-situ S_0 soil, $\sigma'_{p(I)}$, for the Kuwana clay (sample K-13) is 221 kPa since ε_{vo} value of the in-situ soil is 0 as described in the explanation of Eq. (1).

Figure 13 shows the relationship between $\eta_1 = \sigma'_{p(I)} / \sigma'_p$ and the ε_{vo} values for all samples of Kuwana clay. The $\sigma'_{p(I)}$ values for each of the samples in Fig. 13 were estimated using the same procedure as described in Fig. 12. The η_1 value can be used to correct the measured σ'_p of disturbed samples. The η_1 values for each sample in Fig. 13 have a unique correlation with the increase in ε_{vo} values. The I_p values for samples K-10, K-13, K-18, and K-21 (Table 2) are in the range of 43% to 57%, and the relationship between the η_1 and ε_{vo} values for these samples are similar. The η_1 values of the samples of K-3 and K-9, however, are larger than those of K-10, K-13, K-18 and K-21 based on the same ε_{vo} value because of the lower values of I_p for K-3 and K-9. The effect of sample disturbance on the σ'_p values increases with the decrease in I_p values, since the soil becomes more brittle with the decrease in I_p values. In the range of $I_p = (43 \sim 57)\%$, the effect of sample disturbance on the relationship between η_1 and the ε_{vo} value is similar, and this tendency is independent of the difference of range of $\sigma'_{vo} = (148 \sim 205)$ kPa, $q_u = (143.7 \sim 220.2)$ kPa, $OCR = (1.17 \sim 1.59)$, and degree of sample disturbance.

In order to discuss the effects of the different site locations on this relationship between the η_1 and the ε_{vo} value, the results of the samples for all sites are shown in Fig. 14. The relationship between η_1 and the ε_{vo} value is different for sites with the same I_p value. The plots for each site, however, have a unique correlation. This

Fig. 13. Relationship between η_1 and ε_{vo} (Kuwana clays)

Table 2. Index properties of Kuwana clays

Clays	σ'_{vo} (kPa)	I_p (%)	$\bar{q}_u$ (kPa)	σ'_p / σ'_{vo}
K-3	99	26	92.3	2.94
K-9	140	36	117.4	1.86
K-10	148	51	169.0	1.59
K-13	167	57	143.7	1.23
K-18	193	54	220.2	1.30
K-21	205	43	192.9	1.17

means that when the relationship between η_1 and ε_{vo} for soils of the scheme area is determined based on oedometer testing, the $\sigma'_{p(I)}$ value can be estimated by the test results for specimens with different degrees of sample disturbance.

Figure 15 shows the relationship between the C_c and ε_{vo} values for Kuwana clay, sample K-13. These values of C_c include a consolidation pressure range from σ'_p to $2\sigma'_p$. The C_c value of in-situ soil, $C_{c(I)}$, for the Kuwana clay, sample K-13 was found to be 1.02. Figure 16 shows the relationship between $\eta_2 = C_{c(I)} / C_c$ and the ε_{vo} values for

Fig. 14. Relationship between η_1 and ε_{vo} (All clays)

Fig. 15. Relationship between C_c and ε_{vo} (Kuwana clay, K-13)

Fig. 16. Relationship between η_2 and ε_{vo} (Kuwana clays)

all Kuwana clay samples. The η_2 value can be used to correct the measured C_c of disturbed samples. The η_2 values for each sample have a unique correlation with the ε_{vo} values. The relationship between the η_2 and ε_{vo} values for samples K-10, K-13, K-18, and K-21 have a similar tendency as the η_1 values. The η_2 values of the samples of the K-3 and K-9 are smaller, however, than those of samples K-10, K-13, K-18, and K-21 for the same ε_{vo} value. This behavior is apparently related to the lower value of I_p for K-3 and K-9. The main reason is that the change of void ratio caused by sample disturbance increases with the increase in I_p value, since the void ratio also increases with the increase in I_p value, and the effect of sample disturbance on the C_c values increases, since the e_0 value increases with the increase in I_p values. For the range of $I_p = (43 \sim 57)\%$, the effect of sample disturbance on the relationship between the η_2 and ε_{vo} values is similar. This relationship is independent of the difference of the range of $\sigma'_{vo} = (148 \sim 205)$ kPa, $q_u = (143.7 \sim 220.2)$ kPa, and $OCR = (1.17 \sim 1.59)$. When the relationship between η_2 and ε_{vo} for soils of the scheme area is obtained from oedometer testing, the $C_{c(I)}$ value can also be estimated from the test results for samples with different degrees of sample disturbance.

For the Kuwana clay, sample K-13, the void ratio is plotted against the logarithm of consolidation pressure in Fig. 17, and the σ'_p , C_c , and $\bar{q}_u$ values of each specimen are also given in the inset of Fig. 17. These values decrease when the soil is disturbed. Figure 17 also shows the $e - \log \sigma'_v$ curve of the in-situ condition using the $\sigma'_{p(I)}$ and the $C_{c(I)}$ value shown in Figs. 12 and 15, respectively. The e_0 value of the in-situ condition was assumed to be that of the sample S₁. This assumption is generally accepted for saturated clays. It can be judged by comparison with the six different measured $e - \log \sigma'_p$ curves that the estimated $e - \log \sigma'_v$ curve of the in-situ soil is a reasonable curve.

The relationship between the c_v and ε_{vo} values for Kuwana clay, sample K-13, are shown in Fig. 18. The c_v

Fig. 17. Relationship between e and σ'_v (Kuwana clay, K-13)

Fig. 18. Relationship between c_v and ε_{vo} (Kuwana clays)

Fig. 19. Relationship between η_3 and ε_{vo} (Kuwana clays)

value of in-situ soil, $c_{v(I)}$, for the Kuwana clay, sample K-13 was found to be $1663 \text{ cm}^2/\text{day}$. Figure 19 shows the relationship between $\eta_3 = c_{v(I)}/c_v$ values and the ε_{vo} values for all Kuwana clay samples. It is well known, that the c_v value increases with σ'_v in both the recompression and compression ranges. The c_v values in Figs. 18 and 19, therefore, are the mean values of c_v to σ'_v values greater than σ'_p for each sample. The c_v value decreases with an increase in ε_{vo} value, since the c_v value also decreases with an increase in sample disturbance (Shogaki and Kaneko, 1994). There is considerable scatter in Fig. 18, however, because of the rather complex influence of sample disturbance on m_v and k_v . The larger scatter can also be seen in the relationship between η_3 and ε_v shown in Fig. 19. The variation of η_3 values at the same ε_{vo} value, therefore, is larger than that of the η_1 and η_2 values in Figs. 13 and 16, respectively.

When the relationships between the η_1 , η_2 , and η_3 , and ε_{vo} values for the soil of the scheme are determined from the standard oedometer test, the $\sigma'_{p(I)}$, $C_{c(I)}$, $e - \log \sigma'_v$ curve, and $c_{v(I)}$ values for the soil of the in-situ can be estimated from the ε_{vo} , σ'_p , C_c , and c_v values of disturbed specimens.

Figures 20 and 21, respectively show the relationship between the η_1 and η_2 values and the I_p values for all of the sites. The ε_{vo} values were classified into seven different ranges. The η_1 and η_2 values for ε_{vo} values of the same

Fig. 20. Relationship between η_1 and I_p (All clays)

Fig. 21. Relationship between η_2 and I_p (All clays)

classification differ with the I_p values. When the I_p and ε_{vo} values for the soil of the scheme are determined related to the liquid and plastic limit tests and the standard oedometer test, respectively, the $\sigma'_{p(u)}$, $C_{a(u)}$ and $e - \log \sigma'_v$ curve of the in-situ soil can be estimated from the ε_{vo} and I_p values of a disturbed sample.

CONCLUSIONS

The conclusions obtained in this study are summarized as follows:

- 1) Under effective overburden pressure, σ'_{vo} , in the standard oedometer tests, the values of volumetric strain, ε_{vo} , were in the range of 0.85% to 7.02%, and the average was 3.80% for the original in-situ undisturbed sample in the laboratory, S_1 , in the range of $I_p = (10 \sim 102)\%$ and $q_u = (8 \sim 220)$ kPa. The ε_{vo} values of the completely remolded sample, S_6 , were in the range of 7.52% to 23.07% and the average was 15.75%.
- 2) For soils from the same site, the ε_{vo} values tended to increase with an increase in the I_p and the σ'_{vo} values. The ε_{vo} values tended to decrease with an increase in overconsolidation ratio, OCR.
- 3) The relationship between the ratio $R(q_u)$ and $R(\varepsilon_{vo})$, respectively, of q_u and ε_{vo} for each sample to the corre-

sponding values for the S_1 apparently showed a negative correlation, and this tendency was independent of q_u , I_p , and site location. The ε_{vo} value, caused by sample disturbance, was presented as a function of $R(q_u)$. The $R(\varepsilon_{vo})$ of S_6 sample were in the range of 2.38 to 6.59, with an average value of 3.64.

4) A method for correcting the statistical properties of strength and consolidation parameters for sample disturbance were proposed based on a method of estimating the $R(q_u)$ value from the ε_{vo} and OCR values.

5) A method for estimating the preconsolidation pressure, σ'_p , compression index, C_c , $e - \log \sigma'_v$ curve, and coefficient of consolidation, c_v , values for the in-situ soil were proposed from the I_p and the measured values of ε_{vo} , σ'_p , C_c , and c_v for disturbed samples determined in the standard oedometer test.

ACKNOWLEDGMENT

The work reported in this paper was done during the author's stay at the University of Illinois at Urbana-Champaign from 1993 to 1994. The many valuable discussions with Professor Gholamreza Mesri are acknowledged.

REFERENCES

- 1) Andresen, A. A. and Kolstad, P. (1977): "The NGI 54-mm sampler for undisturbed sampling of clays and representative sampling of coarser materials," Proceedings of the International Symposium of Soil Sampling, Singapore, pp. 13-21.
- 2) Casagrande, A. (1936): "The determination of the preconsolidation load and its practical significance," Proceedings of the 1st ICSMFE, Cambridge, Mass., Vol. 1, pp. 60-64.
- 3) Japanese Standards Association (1993): "Test method for one-dimensional consolidation properties of soils," JIS A1217-1990, pp. 1-13.
- 4) Ladd, C. C. and Foott, R. (1974): "New design procedure for stability of soft clays," Proceedings of ASCE, Vol. 100, No. GT7, pp. 763-786.
- 5) Lacasse, S. and Berre, T. (1988): "Triaxial testing methods for soils, Advanced triaxial testing of soil and rock," ASTM STP 977, pp. 264-289.
- 6) Matsuo, M. and Shogaki, T. (1984): "Analyses of several factors influencing q_u -value," Domestic Edition of Soils and Foundations, Vol. 24, No. 3, pp. 139-150 (in Japanese).
- 7) Matsuo, M. and Shogaki, T. (1986): "Experimental study on contribution of various factors influencing q_u ," Domestic Edition of Soils and Foundations, Vol. 26, No. 2, pp. 121-131 (in Japanese).
- 8) Matsuo, M. and Shogaki, T. (1988): "Effects of plasticity and disturbance on statistical properties of undrained shear strength," Soils and Foundations, Vol. 28, No. 2, pp. 14-24.
- 9) Mesri, G. (1989): "A reevaluation of $s_u(\text{mob}) = 0.22\sigma'_p$ using laboratory shear tests," Canadian Geotechnical Journal, Vol. 26, No. 1, pp. 162-164.
- 10) Mesri, G., Lo, D. O. K. and Feng, T. W. (1994): "Settlement of embankments on soft clays," Proceedings of Settlement '94, Vertical and Horizontal Deformations of Foundations and Embankments, Vol. 1, pp. 8-56.
- 11) Milovic, D. M. (1971): "Effect of sampling on some soil characteristics," Sampling of Soil and Rock, ASTM, STP 83, pp. 164-179.
- 12) Nakase, A., Kusakabe, O. and Nomura, H. (1985): "A method for correcting undrained shear strength for sample disturbance," Soils and Foundations, Vol. 25, No. 1, pp. 52-66.
- 13) Okumura, T. (1974a): "The variation of mechanical properties of

- clay samples depending on its degree of disturbance," Proceedings of Specialty Session, 4th Asian Conference, ISSMFE, pp. 73-81.
- 14) Okumura, T. (1974b): "Studies on the disturbance of clay soils and the improvement of their sampling techniques," Report of the Port and Harbor Research Institute, Vol. 10, No. 193, p. 45 (in Japanese).
 - 15) Rochelle, P. and Lefebvre, G. (1971): "Sampling disturbance in Champlain clays," Sampling of Soil and Rock, ASTM, STP 483, pp. 143-163.
 - 16) Schmertman, J. H. (1955): "The undisturbed consolidation behavior of clay," Transactions ASCE, Vol. 120, pp. 1201-1233.
 - 17) Shimizu, M. and Tabuchi, T. (1993): "Effective stress behavior of clays in unconfined compression tests," Soils and Foundations, Vol. 33, No. 3, pp. 28-39.
 - 18) Shogaki, T. (1991a): "Effects of sample disturbance on strength and consolidation," Proceedings of the 9th Asian Regional Conference on ISSMFE, Bangkok, pp. 67-70.
 - 19) Shogaki, T. (1991b): "Strength properties of clay by portable unconfined compression apparatus," Proceedings of the International Conference on Geotechnical Engineering for Coastal Development, Yokohama, pp. 85-88.
 - 20) Shogaki, T., Watanabe, K. and Sudho, T. (1993): "Effects of size and shape of specimen on undrained shear strength of diluvial clay," Proceedings of the International Symposium on Geotechnical Engineering of Hard Soils-Soft Rocks, Athens, pp. 803-807.
 - 21) Shogaki, T. and Kaneko, M. (1994): "Effects of sample disturbance on strength and consolidation parameters of soft clay," Soils and Foundations, Vol. 34, No. 3, pp. 1-10.
 - 22) Shogaki, T. (1995a): Discussion, "Effective stress behavior of clays in unconfined compression tests," Soils and Foundations, Vol. 35, No. 1, pp. 169-171.
 - 23) Shogaki, T. (1995b): "A method for estimating in-situ consolidation parameters using volumetric strain measured in standard oedometer tests," Proceedings of the 50th National Conference on JSCE, pp. 240-241 (in Japanese).
 - 24) Shogaki, T., Miwa, K. and Maruyama, Y. (1996): "A method for correcting consolidation parameters for sample disturbance using disturbed sample," Proceedings of the 31st National Conference on JGS, pp. 645-646 (in Japanese).
 - 25) Skempton, A. W. and Sowa, V. A. (1963): "The behavior of saturated clays during sampling and testing," Géotechnique, Vol. 13, No. 4, pp. 269-290.
 - 26) Tanaka, M., Tanaka, H., Yokoyama, Y. and Suzuki, K. (1995): "Quality of undisturbed clay samples obtained by the various type of samplers," Proceedings of the Symposium on Soil Sampling, Tokyo, pp. 31-36 (in Japanese).
 - 27) Taylor, D. W. (1948): Fundamentals of Soil Mechanics, John Wiley & Sons, pp. 208-249.