UNDRAINED STRENGTH CHARACTERISTICS OF AN ALLUVIAL MARINE CLAY IN THE TOKYO BAY

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ABSTRACT

Extensive field and laboratory tests were carried out on an alluvial marine clay in the Tokyo Bay to grasp its undrained strength characteristics. First, a procedure for obtaining the in-situ undrained strength of the clay in the laboratory tests is discussed in the light of the fundamental properties of the aged alluvial clay. Then undrained strength and anisotropy in the undrained strength of this clay both in the field and in the normally consolidated state are reported and discussed including the effect of the strain rate on the undrained strength. Further, overconsolidated state was considered and overconsolidation ratio resulting from secondary compression and cementation was quantitatively evaluated for this clay. Finally it was suggested from an examination on in-situ vane strength tests that K_0 value of this clay is constant through depth and is independent on overconsolidation ratio.

Key words: alluvial deposit, anisotropy, clay, consolidated undrained shear, laboratory test, overconsolidation

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INTRODUCTION

The major part of the Tokyo Bay is composed of soft alluvial clays which deposited about 3000 to 10000 years ago. The clays in the Bay are classified into high plastic clay ranging from 60% to 150% in water content (higher in the north and decreasing towards the south). Considerable amounts of field and laboratory tests have been carried out on the clays and undrained strength in the field has been generally determined from unconfined compression tests. The author recently carried out various kinds of strength tests, 1) unconfined compression, 2) in-situ vane, 3) triaxial compression and extension, and 4) direct shear tests on an alluvial marine clay at a south-west part of the Bay.

This paper first demonstrates the procedure for determining the in-situ undrained strength of the clay in the laboratory tests and then describes the undrained strength of this clay in the normally consolidated state. The paper further describes 1) various undrained strength in the field, 2) anisotropy in the undrained strength and 3) effect of the strain rate on the undrained strength. Finally considerations on the overconsolidated state and the stress conditions in the field of this clay predicted from various experimental studies will be given.

SITE CONDITIONS

The site investigated is nearly located at the Yokosuka Harbor which is situated at the

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Fig. 1. Guide map for the site

Fig. 2. Physical properties of Natsushima clay

south-west part of the Tokyo Bay as shown in Fig. 1. The bearing layer at this site is composed of mud stone (tertiary deposit) and its surface is characterized by the ups and downs resulting from erosion since its deposition. Thus the thickness of the alluvial clay overlying the mudstone is greatly changed from 0 m to 30 m. Two boreholes were performed at the place where the clay is thicker. In-situ vane tests and undisturbed sampling using stationary piston sampler were performed. Water content (w), liquid and plastic limits (LL and PL), and gradation components obtained from physical tests are shown in Fig. 2. The clay investigated will be hereafter called "Natsushima clay".

PROCEDURE FOR DETERMINING THE IN-SITU UNDRAINED STRENGTH OF THE CLAY IN THE LABORATORY TESTS

Unconfined compression and in-situ vane tests will be considered as the most representative procedure for obtaining the in-situ undrained strength of the clay. It is the advantage of the latter that the test carried out under the field stress conditions when compared with the former (in which release of stress during sampling process and consequently swelling always take place). On the other hand, a definite strength, "compression" strength is obtained in the former, however, the strength in in-situ vane test is unclear.

In obtaining the undrained strength of the low to the medium plastic clay, it has been emphasized that the sample must be reconsolidated at the same stress it carried in the field (Bjerrum and Landva, 1966; Berre, 1969; Raymond, 1972; Bjerrum, 1973; Berre and Bjerrum, 1973; Hanzawa, 1977a, 1977b). On the other hand, Ladd and Foott (1974) pointed out that this technique generally should not be used except for highly structured quick clay

Fig. 3. Idealized e - $\log p$ path of samples during sampling to reconsolidation process

Fig. 4. e - $\log p$ path of aged clay during sampling to reconsolidation process and the conditions adopted in the tests

and for naturally cemented clay because reconsolidation brings the clay to a state smaller in void ratio than the void ratio in the field.

Fig. 3 shows the process of 1) consolidation by overburden pressure, p_0 (sedimentation process), 2) swelling during the process from sampling to laboratory tests and 3) reconsolidation at the same stresses in the field, using the e - $\log p$ diagram. It is experimentally well known that the void ratio at point D, e_D is smaller than that at point B, e_B . Bjerrum (1973) emphasized that the difference, $e_B - e_D$ is negligible and that resulting detrimental effect of the reconsolidation on the clay structure is less serious than if the clay is sheared at a water content in excess of the original value. However, the difference, $e_B - e_D$ is greater the more plastic the clay and becomes much greater when the sample is disturbed during the process from sampling to laboratory tests as shown in Fig. 3 (Ladd and Foott, 1974).

To investigate this problem in detail, a series of consolidation-swelling-reconsolidation tests was carried out on a high plastic clay under the following conditions:—

- 1) The consolidation of the clay during deposition would be limited to the primary compression stage.
- 2) After the completion of deposition, the secondary or the delayed compression (Taylor and Merchant, 1940; Bjerrum, 1967) takes place because naturally deposited clays are aged for hundreds to thousands years.
- 3) The swelling during the process from sampling to laboratory tests would be limited to the primary swelling stage.
- 4) The reconsolidation at the same stresses in the field is terminated when the primary compression is ensured.

The e - $\log p$ path under the above conditions is as shown in Fig. 4.

The clay used for this test was high plastic Kawasaki clay (95% and 41.5% in LL and PL). The slurry conditioned sample was first consolidated in a large oedometer under a small consolidation pressure (20 kN/m² to 30 kN/m²) and then placed in a standard oedometer (6 cm in diameter and 2 cm high). Two kinds of tests were carried out as follows:—

- 1) The specimen was primarily consolidated under pressures, 10, 20, 40, 60, 80 and 100 kN/m² and finally consolidated under 100 kN/m² for 24 hours. Then the specimen was primarily swelled under pressures, 75, 50, 25 and 0 kN/m² and was primarily consolidated under the same pressures as the swelling pressures.

Fig. 5. e — $\log p$ path obtained from consolidation-swelling-reconsolidation tests on high plastic Kawasaki clay

2) The procedure for testing was the same as 1) but the final consolidation and swelling pressures were 200 and 100 kN/m^2 respectively.

Tests results are represented in Fig. 5. As shown in the figure, the difference in the void ratio between e_B and e_D ranged from 0 to 0.007 (average 0.0025) and it can be said that the effect of such difference on the undrained strength must be negligible. It should be pointed out, however, that the sample in this test was not subjected to disturbance. When the sample is disturbed (the sample is usually disturbed more or less even if very highly qualified sampling procedure is used), the reconsolidation at the original stresses leads to larger volume change (Fig. 3) and consequently larger strength might be obtained (Ladd and Foott, 1974).

In order to investigate this problem, an experimental study was carried out. In this test, two specimens were prepared from an undisturbed sample and placed on the triaxial apparatus. During placing the specimens on the triaxial apparatus, one specimen was slightly disturbed by overstressing under undrained conditions. The two specimens were then consolidated under the same stresses in the field and then subjected to shear by compression. The stress-strain curves of the both specimens are shown in Fig. 6. As shown in the figure, the volume change of the disturbed specimen during consolidation stage is greater than that of the undisturbed specimen ($\Delta V = 1.2 \text{ cc}$ and 1.7 cc for the undisturbed and disturbed specimen respectively). The strength of the disturbed specimen, however, is smaller than that of the undisturbed specimen. In addition, it should be noted that there is a great difference in axial strain at failure between two specimen.

From the tests results shown in Fig. 6, the following would be suggested as the funda-

mental strength characteristics of alluvial aged clay:—

- 1) Considerable part of the undrained strength of the aged clay is composed of the strength resulting from the aging effects such as secondary compression, cementation and so on developed since deposition (=structural strength). Because undrained strength as the ratio of the vertical consolidation stress, $(\sigma_1 - \sigma_3)_{\max}/2\sigma_{vc}$ values ranged 0.86 to 0.97 as shown in Fig. 6 which are considerably higher values for normally consolidated clay.
- 2) This structural strength will be destroyed to a certain extent by mechanical disturbance and can't be restored by reconsolidation. Thus the strength of the clay subjected to mechanical disturbance is still smaller than that in the field even if its void ratio after reconsolidation is smaller than the void ratio in the field. The effect of disturbance is sharply reflected in axial strain at failure.

From these experimental studies, it should be stated that the reconsolidation of the sample at the field stresses is also an effective procedure for obtaining the in-situ undrained strength of the high plastic clay. It must be pointed out, however, that the most important thing is to carry out very highly qualified procedure in sampling and extremely careful handling in laboratory tests.

Fig. 6. Comparison of stress-strain curves of the undisturbed and the lightly disturbed specimens consolidated under the same stresses they carried in the field

UNDRAINED STRENGTH TESTS PERFORMED

In-situ vane, unconfined compression and various kinds of consolidated undrained tests on undisturbed samples were performed as follows:—

Fig. 7. Shapes of the vane used in this investigation

1) In-situ vane tests

Three different shapes of vane were used in this investigation as shown in Fig. 7. The strength was measured using a tension ring under the rotating rate of $1^\circ/10s$. Drilling was carried out using casing pipe and three guides which prevent the vane from irregular rotation during strength measurement were used.

2) Unconfined compression tests

The specimen (3.5 cm in diameter and 8.5 cm high) was trimmed from the undisturbed sample (75 mm in diameter) and was subjected to shear by compression under the strain rate of $1\%/min$.

3) Triaxial tests

Both compression and extension tests were performed on K_0 consolidated specimens. From the stress conditions in the consolidation stage and the type of shear, triaxial tests performed in this investigation are classified to the four as follows:—

- (a) CK_0UC_1 tests in which the specimen was consolidated under a K_0 condition where the vertical consolidation stress, σ_{vc} was equal to the effective overburden pressure, p_0 and then brought to shear by compression.
- (b) CK_0UC_2 tests in which the specimen was consolidated under a K_0 condition where σ_{vc} was considerably greater than the preconsolidation pressure, p_c and then brought to shear by compression.
- (c) CK_0UE_1 tests in which the specimen was consolidated under a K_0 condition ($\sigma_{vc} = p_0$) and then brought to shear by extension.
- (d) CK_0UE_2 tests in which specimen was consolidated under a K_0 condition ($\sigma_{vc} > p_c$) and then brought to shear by extension.
- (e) In addition to these four types triaxial tests, a series of K_0 consolidated compression tests was carried out to investigate the effect of strain rate on undrained strength, so called CK_0UC_t tests in this paper.

The specimen used for triaxial tests was 3.5cm in diameter and 8.5cm high, and was subjected to shear under the strain rate of 0.6mm/min (=0.08%/min) except those in CK_0UC_t tests. Long strip shape and spiral shape of drain paper were used for compression and for extension tests respectively.

4) Direct shear tests

The specimen (6cm in diameter and 2cm high) was consolidated in a direct shear apparatus (in which K_0 consolidation is automatically ensured) and then was sheared under a constant volume condition in which the normal stress was controlled to maintain the constant volume

Table 1. Summary of the strength tests performed in this investigation

kinds of tests and symbol			consolidation conditions	shear conditions	strength	numbers
unconfined compres. q_u				strain control 1%/min	$q_u/2$	36
in-situ vane test				rotation rate 1°/10s	$S_{u(v)}$	25
triaxial tests	compression	CK_0UC_1	K_0 consolidation $\sigma_{vc} = p_0$	strain control 0.6mm/min	$S_{uf(c)}$	11
		CK_0UC_2	" $\sigma_{vc} > p_c$	"	$S_{un(c)}$	3
		CK_0UC_t	" $\sigma_{vc} > p_c$	strain control 3%/min-0.0005%/min		7
	extension	CK_0UE_1	" $\sigma_{vc} = p_0$	" 0.6mm/min	$S_{uf(e)}$	11
		CK_0UE_2	" $\sigma_{vc} > p_c$	"	$S_{un(c)}$	3
direct shear tests		CK_0US_1	" $\sigma_{vc} = p_0$	0.125mm/min	$S_{uf(s)}$	11
		CK_0US_2	" $\sigma_{vc} > p_c$	"	$S_{un(s)}$	2

(Taylor, 1952). Two type tests were performed as follows:—

- (a) CK_0US_1 tests in which the specimen was consolidated under a stress, $\sigma_{vc} = p_0$ and then was sheared under the strain rate of 0.125 mm/min.
- (b) CK_0US_2 tests in which the specimen was consolidated under a stress, $\sigma_{vc} > p_c$ and then was sheared under the same shear rate as in CK_0US_1 tests.

In triaxial and direct shear tests, consolidation was terminated when the excess pore pressure almost dissipated (for triaxial tests) and when the primary consolidation was ensured (for direct shear tests). As explained above, the test marked by 2 and 1 were carried out to obtain the strength in the normally consolidated state and in the field. All the strength tests carried out in this investigation are summarized in Table 1. In addition to these tests, considerable amounts of consolidation tests (34 specimens) were also performed.

UNDRAINED STRENGTH IN THE NORMALLY CONSOLIDATED STATE

It is well known fact that the strength characteristics of the clay is greatly dependent on whether the clay is in the normally consolidated state or in the overconsolidated state. Thus, to obtain and to compare the both strength in the normally consolidated state and in the field are very important in evaluating the strength characteristics of the clay in the field. In this section of the paper, undrained strength characteristics of Natsushima clay in the normally consolidated state will be described.

The samples from G.L(–)8m, (–)12m and (–)23m were used for the tests. Consolidation was carried out under multi stage conditions (4 to 5 steps) and was terminated when the excess pore pressure mostly dissipated (for triaxial tests) and when the primary consolidation was completed (for direct shear tests). The dissipation of the excess pore pressure was only checked at the final loading stage and the completion of the primary consolidation was checked at each loading stage. Thus the normally consolidated clay defined in this paper is the clay which has just completed the primary consolidation stage, that is, same as the normally consolidated young clay according to Bjerrum's classification (1973).

K_0 conditions was checked from the vertical deformation and the volumetric change during

Fig. 8. Undrained compression, extension and shear strengths in the normally consolidated state, $S_{un(c)}$, $S_{un(e)}$ and $S_{un(s)}$ obtained from CK_0UC_2 , CK_0UE_2 and CK_0US_2 tests plotted versus vertical consolidation stress, σ_{vc}

consolidation stage. In this method the following conditions must be kept during consolidation.

$$A_0 \times \delta_v = \Delta V \quad \text{Eq. (1)}$$

where A_0 = initial sectional area of the specimen

δ_v = vertical deformation of the specimen

ΔV = volumetric change during consolidation

The K_0 values obtained from this method ranged from 0.490 to 0.519 (average 0.506).

The undrained strengths obtained from CK_0UC_2 , CK_0UE_2 and CK_0US_2 tests are plotted versus vertical consolidation stress, σ_{vc} in Fig. 8. As shown in the figure, the undrained strength as the ratio of the vertical consolidation stress in normally consolidated state, S_{un}/σ_{vc} (S_{un} = undrained strength in the normally consolidated state) and the strength ratio between compression, extension and shear strengths were obtained as follows:—

$$\begin{aligned} S_{un(c)}/\sigma_{vc} &= 0.36 \\ S_{un(e)}/\sigma_{vc} &= 0.22 \\ S_{un(s)}/\sigma_{vc} &= 0.25 \end{aligned} \quad \text{Eq. (2)}$$

Strength ratio

$$\begin{aligned} S_{un(e)}/S_{un(c)} &= 0.61 \\ S_{un(s)}/S_{un(c)} &= 0.69 \end{aligned} \quad \text{Eq. (3)}$$

$S_{un(c)}$: undrained compression strength in the normally consolidated state.

$S_{un(e)}$: undrained extension strength in the normally consolidated state.

$S_{un(s)}$: undrained shear strength in the normally consolidated state.

The difference in undrained strength indicated in Eq. (2) and Eq. (3) is the anisotropy in undrained strength due to a stress induced anisotropy that results from rotation of principal stresses during shear and variation in the intermediate principal stress (Ladd and Foott, 1974). For practical purposes the effect of these components can be considered collectively by modeling the system and sample orientation that will exist in the field. Procedures how to use the anisotropy in undrained strength for practical problems have been demonstrated in detail by Bjerrum (1973) and Ladd and Foott (1974), and the values shown in Eq. (2) can be used for evaluating the strength increase of the clay resulting from consolidation under loading for corresponding directions.

IN-SITU UNDRAINED STRENGTH CHARACTERISTICS OF NATSUSHIMA CLAY

The strength obtained from unconfined compression and from in-situ vane tests, $q_u/2$ and $S_{u(v)}$ respectively and their distribution with depth are summarized in Fig. 9. As indicated in the figure, strength values from both tests mostly increase with depth. Such unconfined compression strength values that $q_u/2p_0$ are less than 0.36 ($=S_{un(c)}/\sigma_{vc}$) are enclosed by the dashed lines in the figure, which shows that the clay is underconsolidated state. The underconsolidated state of this clay which deposited thousands years ago is so unusual that such strength should be excepted when the undrained strength for practical use is determined from unconfined compression strength values.

On the other hand, in-situ undrained compression, $S_{uf(c)}$, extension, $S_{uf(e)}$ and shear, $S_{uf(s)}$ strengths obtained from CK_0UC_1 , CK_0UE_1 and CK_0US_1 tests are summarized in Fig. 10 in which the range of unconfined compression and in-situ vane strengths both shown in Fig. 9 are also shown. It can be observed that unconfined compression strength values are in the zone enclosed by in-situ compression and extension strengths.

It is interesting to compare the strength ratio of the clay in the field with those in the

Fig. 9. Undrained strength obtained from unconfined compression and in-situ vane tests, $q_u/2$ and $S_{u(v)}$ plotted versus depth

Fig. 10. In-situ undrained compression, extension and shear strengths, $S_{u(f(c))}$, $S_{u(f(e))}$ and $S_{u(f(s))}$ obtained from CK₀UC_①, CK₀UE_① and CK₀US_① tests plotted versus depth

normally consolidated state. The comparison of each in-situ strength as the ratio of in-situ compression strength are indicated in Fig.11 in which the above strength ratio in the normally consolidated state shown in Eq. (3) are also shown. From the comparison represented in the figure, the following notes and suggestions can be obtained:—

- 1) In-situ extension and shear strength values as the ratio of in-situ compression strength, $S_{u(f(e))}/S_{u(f(c))}$ and $S_{u(f(s))}/S_{u(f(c))}$ show good agreement with those in the normally consolidated state below G.L (—)10m.
- 2) Unconfined compression strength values as the ratio of in-situ compression strength, $q_u/2S_{u(f(c))}$ range from 0.5 to 1.2 and show relatively large scatter.
- 3) In-situ vane strength values as the ratio of in-situ compression strength, $S_{u(v)}/S_{u(f(c))}$ range from 0.8 to 1.2 and show smaller scatter compared with $q_u/2S_{u(f(c))}$ values.
- 4) In-situ extension and shear strength values as the ratio of in-situ compression strength, $S_{u(f(e))}/S_{u(f(c))}$ and $S_{u(f(s))}/S_{u(f(c))}$ above G.L (—)10m show considerably higher values than those in the normally consolidated state. The slight disturbance during the process from sampling to laboratory tests is so detrimental as previously pointed out that the specimens used for CK₀UC_① tests above G.L (—)10m (particularly G.L (—)5m to (—)10m) would have been disturbed during the process as previously pointed out. This prediction would be reasonable because in-situ vane and unconfined compression strength values as the ratio of in-situ compression strength also showed higher values in this zone.
- 5) Thus it may be stated that the strength ratio between compression, extension and

Fig. 11. In-situ undrained strength as the ratio of in-situ compression strength, $S_{uf}/S_{uf(c)}$ plotted versus depth

shear strengths of Natsushima clay are constant through depth and are same as the values in the normally consolidated state.

On the other hand, in-situ undrained strength as the ratio of the effective overburden pressure, S_{uf}/p_0 is very useful for evaluating the strength characteristics of the clay. Various in-situ undrained strength as the ratio of the effective overburden pressure and their distribution with depth are summarized in Fig.12. As shown in the figure, in-situ undrained strength values as the ratio of the effective overburden pressure are higher in upper part and decrease with increasing depth. The procedure how to determine the three lines, $S_{uf(c)}/p_0$, $S_{uf(e)}/p_0$ and $S_{uf(s)}/p_0$ represented by the dashed lines in the figure will be described later.

Various in-situ undrained strengths and their characteristics of Natsushima clay were discussed. As many researchers have reported, relevant strength values should be used for practical problems such as for stability analysis for sliding (Bjerrum, 1973; Ladd and Foott, 1974; Hanzawa et al., 1979). From this point of view, the following two can be pointed out for Natsushima clay:—

- 1) The fact that unconfined compression strength values, $q_u/2$ range between in-situ compression and extension strengths suggests the validity of using $q_u/2$ values for stability analysis.
- 2) Safety factor of unsafe side would be obtained when in-situ vane strength are used for stability analysis because the values showed relatively good agreement with in-situ compression strength values.

The consolidation stresses in CK_0UC_1 and CK_0UE_1 tests are so small that strange K_0 value would be obtained when K_0 consolidation is carried out ensuring the condition shown in Eq.

Fig. 12. Various undrained strengths as the ratio of the effective overburden pressure, S_{uf}/p_0 plotted versus depth

(1) because of the entrapped water between the specimen and the rubber membrane and so on. Actually when some preliminary tests were carried out ensuring the conditions shown in Eq. (1), K_0 values of 0.15 to 0.25 were obtained. Therefore K_0 value in CK_0UC_1 and CK_0UE_1 tests was fixed to 0.5 which is equal to K_0 value in the normally consolidated state.

EFFECT OF THE STRAIN RATE ON UNDRAINED STRENGTH

The effect of time on shear strength of clays was first pointed out by Terzaghi (1931). Since it, many reserchers have studied this problem using various procedures. Recent reserch on this problem, including an analysis of failure, has demonstrated that the undrained strength of soft clays depends on the rate at which the clay is brought to failure, the effect being greater the more plastic the clay (Bjerrum, 1973).

A series of K_0 consolidated compression tests, CK_0UC_t tests (previously described) in which the specimen were brought to shear by compression under different rate of strain was carried out to investigate this problem. In this test the specimens obtained from an undisturbed sample were consolidated under a K_0 condition ($\sigma_{vc}=200\text{ kN/m}^2$ and $\sigma_{hc}=100\text{ kN/m}^2$) and then brought to shear under different rate of strain ranging from 3%/min to 0.0005%/min. The tests results are shown in Fig. 13 in which the strength of each specimen as the ratio of the strength under the strain rate of 1%/min ($=(\sigma_1-\sigma_3)_{\max}/(\sigma_1-\sigma_3)_{\max}=1\%/min$) are plotted versus strain rate. As shown in the figure, the strength increases with increasing strain rate and the strength increase becomes considerable when the strain rate exceeds 0.5%/min to 0.1%/min. From practical point of view, it can be concluded that strength tests on Na-

Fig. 13. Relationship between the undrained strength and the strain rate obtained from CK_0UC_t tests

tsushima clay should be carried out under a strain rate equal to or less than 0.05%/min.

CONSIDERATIONS ON OVERCONSOLIDATED STATE OF NATSUSHIMA CLAY

As shown in Fig. 12, the in-situ undrained strength values as the ratio of the effective overburden pressure (S_{uf}/p_0 values) of Natsushima clay are higher in upper part and decrease

Fig. 14. Overconsolidation ratio obtained from various kinds of consolidated undrained strength tests and from consolidation test plotted versus depth

with increasing depth. In addition, in-situ compression, extension and shear strength values as the ratio of the effective overburden pressure, $S_{uf(c)}/p_0$, $S_{uf(e)}/p_0$ and $S_{uf(s)}/p_0$ are greater than the corresponding values in the normally consolidated state, which suggests that Natsushima clay is in overconsolidated state. In this section of the paper, the overconsolidated state of this clay will be discussed in some detail.

Overconsolidation ratio (O.C.R) of the clay will be obtained by the following operation using the in-situ undrained strength values as the ratio of the effective overburden pressure when the clay has brought to an overconsolidated state by factors other than release of the overburden (Hanzawa, 1977b).

$$\begin{aligned} \text{O.C.R} &= S_{uf(c)}/p_0 \div S_{un(c)}/\sigma_{vc} = S_{uf(c)}/p_0 \div 0.36 \\ &= S_{uf(e)}/p_0 \div S_{un(e)}/\sigma_{vc} = S_{uf(e)}/p_0 \div 0.22 \\ &= S_{uf(s)}/p_0 \div S_{un(s)}/\sigma_{vc} = S_{uf(s)}/p_0 \div 0.25 \end{aligned} \quad \text{Eq. (4)}$$

Overconsolidation ratio obtained from Ep. (4) and from the consolidation tests (from which O.C.R is given as p_c/p_0) are summarized in Fig. 14. As shown in the figure, overconsolidation ratio obtained from the two procedures show good agreement each other. The values are higher in upper part and decrease with increasing depth. When overconsolidation ratio with depth are represented by the dashed line shown in Fig. 14, in-situ compression, extension and shear strengths as the ratio of the effective overburden pressure can be obtained as follows:—

$$\begin{aligned} S_{uf(c)}/p_0 &= \text{O.C.R} \times 0.36 \\ S_{uf(e)}/p_0 &= \text{O.C.R} \times 0.22 \\ S_{uf(s)}/p_0 &= \text{O.C.R} \times 0.25 \end{aligned} \quad \text{Eq. (5)}$$

The three dashed lines, $S_{uf(c)}/p_0$, $S_{uf(e)}/p_0$ and $S_{uf(s)}/p_0$ lines shown in Fig. 12 were obtained by this operation.

The following factors may bring the clay to an overconsolidated state:— 1) stress release (release of overlying deposits and change in ground water level), 2) secondary or delayed compression, 3) drying or desiccation and 4) chemical reaction chiefly represented by cementation. Considering the geologic history of the Tokyo Bay, it may be stated that there have been few possibility for removal of the overburden. On the other hand, the sea level of the Tokyo Bay was the highest about 5 to 6 thousands years ago and has gradually been lowering to the present time (Kaizuka, 1964). The surface level at the site, however, is about 10m below the present sea level so that it should be also stated that Natsushima clay has not been subjected to drying or desiccation. Thus the two factors, 1) secondary compression and 2) cementation will be considered as the main factors to have brought this clay to an overconsolidated state.

To obtain the quantitative effect of the secondary compression on the overconsolidation ratio, an experimental study was carried out, in which slurry conditioned Natsushima clay was consolidated in a standard oedometer and secondary compression corresponding to about 6000 years (which is almost equal to the age of this clay since deposition) was measured. The tests results are shown in Fig. 15. As shown in the figure, the decrease in void ratio resulting from secondary compression is constant with the consolidation pressure and showed a value of about 1.3 in overconsolidation ratio. This value (O.C.R=1.3) is almost corresponding with overconsolidation ratio at around the depth of G.L(—) 25m as shown in Fig. 14.

It is very difficult to know the exact history of the clay and to reproduce the actual phenomena occurred since deposition in the laboratory test, however, it might be possible to divide the overconsolidation ratio of Natsushima clay into the two as follows:—

- 1) Overconsolidation ratio resulting from secondary compression = 1.3

Fig. 15. Consolidation tests results for evaluating the secondary compression of Natsushima clay corresponding to 6 000 years

Fig. 16. Distribution of three strength components resulting from 1) primary compression (S_{un}), 2) secondary compression (S_{u1}) and 3) cementation (S_{u2}) obtained from predicted O.C.R₁ and O.C.R₂

- 2) Overconsolidation ratio resulting from factors other than secondary compression such as from cementation = O.C.R (shown in Fig. 14) - 1.3

When the strength of Natsushima clay is divided into the following three resulting from 1) primary compression, 2) secondary compression and 3) chemical reaction such as cementation, the distribution of these three strength components with depth can be represented as shown in Fig. 16.

SUGGESTIONS OBTAINED FROM IN-SITU VANE TESTS RESULTS

As previously described, three different shapes of vane were used in this investigation. Thus it is possible to obtain the following three strengths on 1) a horizontal direction, 2) a vertical direction and 3) a 45° direction. The three strengths and their ratio obtained in this investigation are shown in Fig. 17. As shown in the figure, the strength ratio between horizontal and vertical directions range from 0.6 to 0.8 (average 0.7) and it may be stated that the ratio is constant through depth. Strengths on horizontal and vertical directions are suggested to be the function of the effective overburden and horizontal pressures, p_0 and $K_0 p_0$, and when each strength is expressed as αp_0 (for horizontal direction) and $\beta K_0 p_0$ (for vertical direction), the K_0 value will be given by the ratio between both strengths. The value α/β is not known for this clay at the present time but it may be suggested that K_0 value is constant when the strength ratio between horizontal and vertical directions is constant,

Fig. 17. In-situ vane strength on three directions and their strength ratio between different directions plotted versus depth

which suggests that the K_0 value of Natsushima clay is independent on the overconsolidation ratio. In addition, this suggestion will be supported by the tests results that the strength ratio between in-situ compression, extension and shear strengths were constant through depth as previously described. It is also consistent with some researchers' works that the K_0 value is independent on overconsolidation ratio when the clay was brought to an overconsolidated state by factors other than release of the overburden (Bishop, 1958; Burland, 1967; Hanzawa, 1977b).

SUMMARY AND CONCLUSIONS

Undrained strength characteristics of an alluvial marine clay in the Tokyo Bay were reported. Some subjects discussed in this paper are classic and others are new, and there are some problems still unsolved. Conclusions and suggestions obtained from this

study will be summarized as follows with some comments.

- 1) For classic problems such as how to deal with unconfined compression strength, anisotropy in undrained strength and effect of the strain rate, conclusions same as those in the past studies were obtained.
- 2) It may be stated that the reconsolidation of the sample at the same stresses in the field is an effective procedure for obtaining the in-situ undrained strength of the aged clay. However, it must be required first of all, to carry out very highly qualified technique in sampling and extremely careful handling in laboratory testing.
- 3) A procedure how to evaluate quantitatively the both strengths resulting from 1) secondary compression and 2) chemical reaction such as cementation, was shown. However it is still questionable in the author's opinion that the secondary compression is constant through depth for such a thick clay stratum. It will be desired to solve this problem clear.
- 4) It was suggested that the K_0 value of an overconsolidated clay caused by factors other than release of the overburden is independent on overconsolidation ratio. It should be also desired to establish the method for its direct measurement with enough accuracy.

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NOTATION

A_0 =initial sectional area of the specimen	$S_{un(c)}$ =undrained compression strength in normally consolidated state
e =void ratio	
K_0 =coefficient of earth pressure at rest	$S_{un(e)}$ =undrained extension strength in normally consolidated state
LL=liquid limit	
O.C.R=overconsolidation ratio	
p_c =preconsolidation pressure	$S_{un(s)}$ =undrained shear strength in normally consolidated state
PL=plastic limit	
p_0 =effective overburden pressure	$S_{u(v)}$ =in-situ vane strength
q_u =unconfined compression strength	w =water content
S_u =undrained strength	σ_{hc} =horizontal consolidation stress
S_{uf} =in-situ undrained strength	
$S_{uf(c)}$ =in-situ undrained compression strength	σ_{vc} =vertical consolidation stress
$S_{uf(e)}$ =in-situ undrained extension strength	$(\sigma_1 - \sigma_3)_{\max}$ =maximum deviator stress
$S_{uf(s)}$ =in-situ undrained shear strength	ΔV =volumetric change
S_{un} =undrained strength in normally consolidated state	δ_v =vertical deformation

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