COMPARATIVE STUDY ON UNDRAINED SHEAR STRENGTH OF OSAKA BAY PLEISTOCENE CLAY DETERMINED BY SEVERAL KINDS OF LABORATORY TEST

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ABSTRACT

This paper describes a geotechnical survey evaluating the shear strength of Pleistocene clay in Osaka Bay. In the stability evaluation for a seawall structure of Kansai International Airport, as the slip circle giving the minimum safety factor possibly passes through the Pleistocene clay layer, the stability could be greatly affected by the shear strength profile of the Pleistocene clay. The result of the stability analysis is very sensitively affected by the undrained shear strength profile, which is roughly estimated based on a ratio of undrained shear strength to consolidation yield stress (s_u/p_y) of natural clay deposit (Mesri, 1975; Tanaka and Tanaka, 1994) and the average apparent overconsolidation ratio ($OCR = p_y/\sigma'_{v0}$) at this site (Horie et al., 1984). In this study, in order to evaluate the undrained shear strength obtained by typical testing methods, UC (unconfined compression) test, UU, CU, K_0 CU compression and extension triaxial tests and DS (direct shear) test were carried out for undisturbed samples of the Pleistocene clays collected for the second phase construction of Kansai International Airport. Mechanical behaviors and shear strength profiles obtained by each test were compared and discussed. The undrained shear strength and its scattering depend on the testing methods. The recompression method, in which a specimen is consolidated under the in-situ stress, gives many reliable test results. The UC and UU tests, in which a specimen is not consolidated, give very scattered and unreliable test results. From the comparison between undrained shear strengths for design determined by each test with correction factors, it is found that the strength of the DS test is smaller than those of K_0 CU and CU tests. This fact is derived from the relatively larger strength anisotropy of the Pleistocene clay. The undrained shear strength increase ratio with depth $\Delta s_u/\Delta z$ for design is obtained as 2.09 kPa/m, considering strength anisotropy.

Key words: direct shear test, Pleistocene clay, stability analysis, triaxial test, unconfined compression test, undisturbed sample (IGC: D6)

INTRODUCTION

This paper describes a geotechnical survey evaluating the shear strength of Pleistocene clay in Osaka Bay. In most coastal regions of Japan, Pleistocene clay layer is beneath a Pleistocene sand or gravel layer, which lies under a 10 to 30 m thick Holocene soft clay deposit. Because the Pleistocene clay in Osaka Bay is consolidated under a larger overburden pressure, additional strength has been developed by the so-called ageing effect. In most construction projects in the past, Pleistocene layers have been considered as stable layers supporting superstructures. In recent years, however, the size of structures in coastal regions has been increasing, and more construction work is being carried out in deeper sea areas. Also, when geotechnical engineers consider a Holocene clay layer at a shallower depth, they can stabilize the layer by physical or chemical treatment; however, when they con-

sider a Pleistocene clay layer at a deeper depth, they cannot stabilize it because it is too deep to be treated. Thus, evaluation of the engineering characteristics of a Pleistocene clay layer has become an important issue for design and construction in coastal areas.

A typical example is the Kansai International Airport construction project, which was constructed on an artificial island in Osaka Bay, 5 km off the Senshu area southwest of the Osaka City, as shown in Fig. 1. The airport was inaugurated in September 1994 when the first phase construction was completed. Since the airport is currently operating with only one runway, the second phase construction for an additional runway parallel to the existing one was started in summer 1999. The reclamation area in the second phase has an average water depth of 19.5 m, in contrast to 18.0 m in the first phase, and an average thickness of the Holocene clay layer of 23~25 m, in contrast to 18 m in the first phase. These factors

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Fig. 1. (a) Location of Kansai International Airport and (b) borehole points in present study

Fig. 2. Typical section of seawall structure in second phase Kansai International Airport construction project

will increase the consolidation pressure due to the reclamation by 40% compared to the first phase.

In the first phase construction, the undrained shear strength was not evaluated in detail because the stability analyses of the seawall structures indicated that the safety factors against deep-seated slip circles passing through the Pleistocene clay layer were much larger than the required design safety factor of 1.3. In the second phase construction, as the slip circle giving the minimum safety factor possibly passes through the Pleistocene clay layer, the shear strength of the Pleistocene clay layer could greatly affect the stability of seawall structures.

Stability analyses were carried out for a typical seawall structure in the second phase construction. The assumed cross section in this study is shown in Fig. 2. The safety factor for the base slide was calculated by slip circle analysis with modified Fellenius method. Table 1 shows a summary of material properties used in the stability analyses.

In the calculations, layer number 11 represents the Pleistocene clay stratum in which the undrained shear strength increase ratio with depth $\Delta s_u / \Delta z$ varies from 1.5 to 2.5 kPa/m, and $s_u = 0$ at the seabed ($z = -19.2$ m). Figure 3 shows variations of the calculated bottom depth

of slip circle and safety factor F_s with strength increase ratio $\Delta s_u / \Delta z$. In this figure, F_s reaches a constant value at about 1.29, where $\Delta s_u / \Delta z$ is larger than 2.2 kPa/m, because the critical slip circle is within the Holocene clay layer, and does not pass through the Pleistocene deposit layer. When $\Delta s_u / \Delta z$ is less than 2.1 kPa/m, F_s is smaller than 1.27, where F_s increases linearly with $\Delta s_u / \Delta z$. In these cases, the critical slip circles reach the Pleistocene clay deposit. The Pleistocene sand layer exists between the Holocene clay layer and the Pleistocene clay layer. Since the shear strength of the sand layer is larger than that of the clay layer on both sides, the bottom depth of the critical slip circle varies discontinuously at $\Delta s_u / \Delta z$ of 2.1 kPa/m and its depth moves in the Pleistocene clay to the Holocene clay at this point. This implies that the Pleistocene clay deposit governs the stability, if $\Delta s_u / \Delta z$ is less than 2.1 kPa/m.

Mesri (1975) indicated that vane shear strength s_u corrected by Bjerrum's factor (Bjerrum, 1973) and normalized by consolidation yield stress p_y , (s_u / p_y), is a constant value at 0.22 for the variation of plasticity index I_p . Tanaka and Tanaka (1994) applied a vane shear test to Japanese marine clays and indicated that s_u / p_y varies

Table 1. Parameters for stability analysis

Layers	Soil type	Unit weight (kN/m ³)	Strength parameters
1	Reclaimed sand	19.6	$\phi=30^\circ$
2	Concrete structure	21.6	very large
3	Sand	19.6	$\phi=30^\circ$
4	Sand	19.6	$\phi=30^\circ$
5	Sand	19.6	$\phi=30^\circ$
6	Holocene clay not improved	16.9	$s_u = -1.5(z + 19.2)$ kPa
7	Holocene clay improved by sand drain	16.9	$s_u = 49 - 1.5(z + 19.2)$ kPa
8, 9	Holocene clay improved by sand drain	16.9	$s_u = 118$ kPa
10	Pleistocene sand	19.6	$\phi=35^\circ$
11	Pleistocene clay	16.9	$s_u = -1.5(z + 19.2) \sim -2.5(z + 19.2)$ kPa

z (m) is the elevation and the initial seabed is at $z = -19.2$ m

Fig. 3. Calculated relationship between safety factor and strength increase ratio of Pleistocene clay with variation of depth of slip circle

from 0.25 to 0.35, which is higher than Mesri's value. Using the apparent overconsolidation ratio OCR , undrained shear strength increase ratio with depth $\Delta s_u / \Delta z$ is expressed as follows,

$$\frac{\Delta s_u}{\Delta z} = \frac{s_u}{p_y} \cdot \gamma' \cdot OCR \quad (1)$$

where, s_u at $z = -19.2$ m (at seabed) is assumed as 0, and γ' is effective unit weight of the clay. At this site, the average OCR is 1.3 according to Horie et al. (1984) and the average unit weight γ' for the Pleistocene clay in Osaka Bay is 6.9 kN/m^3 ; thus undrained shear strength increase ratio with depth $\Delta s_u / \Delta z$ of 2.2 kPa/m is derived from Eq. (1) when s_u / p_y is assumed as 0.25. The stability analysis is very sensitive to the value $\Delta s_u / \Delta z$ of around 2.2 kPa/m as mentioned above. In order to carry out reliable stability analyses, undrained shear strength of the Pleistocene clay should be determined very carefully. When a slip takes place along a circle passing through the Pleistocene clay during the construction of the seawall, the influenced area of the failure will be quite large and possibly cause serious damage to the construction project. Consequently, accurate estimation of the undrained shear strength is important. In addition, it

should be noted that the undrained shear strength depends on the testing method. Thus, in practical design process, in order to determine the undrained shear strength for design, understanding relations between shear strengths determined by each test is very useful for geotechnical engineers.

In the present study, a series of five kinds of laboratory shear test, UC (unconfined compression) test, UU (unconsolidated undrained), CU (isotropically consolidated undrained), K_0 CU (anisotropically consolidated with a K_0 value and undrained) compression and extension tests, and DS (direct shear) test, was carried out for undisturbed samples of the Pleistocene clays collected for the second phase construction. Mechanical behaviors and shear strength profiles obtained by each test were compared and discussed.

LABORATORY TESTS

In most Japanese construction practices, undrained shear strength of clay layer is usually determined as half of the unconfined compressive strength q_u of an undisturbed clay sample. The reason for using the UC test is not only its simplicity but also its reliability, which has been proved by several case studies of actual failures in coastal areas (Nakase, 1967). For the samples taken from large depth, however, the effect of disturbance on q_u becomes much larger due to fissures in the samples caused by release of high confining stresses. Thus, the q_u is not reliable for evaluating the undrained shear strength of a clay sample collected from a large depth.

Tsuchida (2000) indicated that there are two kinds of disturbance: one is the remolding type, caused by shear deformation, and the other is the crack type, caused by fissures. The UU triaxial test is frequently adopted to recover crack type disturbance. However, this test cannot take account of remolding type disturbance and strength anisotropy. Additionally, effective stress, which governs the soil behavior, is not equivalent to the in-situ condition.

To reduce the effect of disturbance, the recompression method was proposed by Bjerrum (1973), and Berre and Bjerrum (1973), in which the shear test is conducted after a specimen sampled is consolidated under the in-situ effec-

tive stress condition in the triaxial cell. In this method, a specimen trimmed from an undisturbed sample is first fully consolidated anisotropically with σ'_{v0} and $\sigma'_{h0}(=K_0\sigma'_{v0})$, and then is subjected to shear under undrained condition for compression (K_0 CUC) and extension (K_0 CUE).

Generally, as long as soil is normally consolidated in one-dimensional condition, the K_0 value is almost constant, but if a stress history of overconsolidation is applied to the soil, the K_0 value will be changed. In natural soils, the clay is often in a quasi-overconsolidation state because of ageing effects such as secondary consolidation and chemical processes with cementation. The K_0 is possibly influenced by these factors; however, it is not yet clear how much K_0 is influenced by these factors. In this study, the K_0 value under normal consolidation was determined by a K_0 consolidation test in which an undisturbed specimen was consolidated up to more than twice the consolidation yield stress in the triaxial cell. In the present study, $K_0=0.5$ was obtained (Watabe et al., 1998). The test procedure for K_0 -consolidation by triaxial test was described in detail by Tsuchida and Kikuchi (1991).

In Berre and Bjerrum (1973), the tensile strength is defined as half of the deviator stress at an axial strain of 2%, because the K_0 CUE test for Drammen clay dealt with by them shows strain hardening and does not have peak strength. Watabe (1999) showed that this behavior is similar to that of a clay in normal consolidation or light overconsolidation. For this kind of soil, definition of the tensile undrained shear strength is very difficult and this fact is discussed in Koutsoftas and Ladd (1985), Watabe (1999) and Watabe et al. (2000). Since the Pleistocene clay in Osaka Bay is very aged, it starts to show a clear peak both for compression and extension. Undrained shear strength for extension in this study is defined as the peak strength within the 15% axial strain.

Hanzawa et al. (1980) and Hanzawa (1982) defined undrained shear strength for design as the average value of undrained compressive and tensile strengths in the recompression method, and applied the strength to stability analyses of several practical projects. They reported excellent agreement between the analyses and the field behaviors.

Tsuchida (2000) proposed a CU test, in which a specimen sampled is consolidated under the mean in-situ effective stress of $(1+2K_0)\sigma'_{v0}/3$ and is subjected to shear with undrained condition for compression. Because most K_0 -values for Japanese soft marine clays fall in the range of 0.4 to 0.5, the recommended K_0 value for the CU test is 0.5, thus the consolidation stress is $(2/3)\sigma'_{v0}$.

Hanzawa (1995) indicated that shear strength for design can be reliably obtained with easy handling by DS (direct shear) test. This DS test equipment is designed based on Mikasa (1960) and can carry out shear tests under constant volume condition. The specimen sampled is consolidated by the overburden stress, automatically under K_0 condition in the rigid shear box, and is sheared with constant volume.

In this study, these five kinds of shear test were carried

out as follows,

(a) CU test

A specimen sampled was trimmed to 35 mm in diameter and 80 mm in height. This size was the same as that for UU, CU, and K_0 CU triaxial tests. The specimen was sheared with an axial strain rate of 1.0%/min until peak strength was measured. Undrained shear strength s_u was defined as half of the maximum compressive strength q_u .

(b) UU test

A confining stress of 98 kPa, 392 kPa or 686 kPa was applied to a specimen trimmed as above without consolidation. Undrained compression was conducted with an axial strain rate of 1.0%/min until a peak strength was observed. In the same manner as the UU test, undrained shear strength s_u was defined as half of the maximum deviator stress.

(c) CU test

A specimen was consolidated isotropically under $(2/3)\sigma'_{v0}$, and undrained compression was conducted with an axial strain rate of 0.1%/min up to an axial strain of 15%. The undrained shear strength was defined as half of the maximum deviator stress. Tsuchida (2000) also proposed taking 75% of the undrained shear strength measured from the CU test as design strength, to take account of the strength anisotropy and strain rate effects.

(d) K_0 CU test

Anisotropic consolidation was carried out by increasing the stresses linearly from $\sigma'_1=\sigma'_3=(1/6)\sigma'_{v0}$ up to $\sigma'_1=\sigma'_{v0}$ and $\sigma'_3=K_0\sigma'_{v0}=0.5\sigma'_{v0}$ in 720 min, and this condition was maintained until the excess pore pressure was completely dissipated. After that, the specimen was sheared in undrained condition for compression (K_0 CUC) and extension (K_0 CUE) with an axial strain rate of 0.1%/min. The undrained compressive and tensile strengths were defined as half of the peak deviator stresses respectively within 15% axial strain, and the undrained shear strength for design was defined as the average value of these two.

(e) DS test

A sampled specimen was trimmed to 60 mm in diameter and 20 mm in height, and this was inserted to the shear box. The specimen was consolidated completely under σ'_{v0} and sheared in constant volume with a clearance between upper and lower boxes of 0.2 mm. A shear rate of 0.25 mm/min was adopted, which is the same as that in Hanzawa (1995). Details of the test equipment were given in Shibuya (1995). The undrained shear strength was defined as the maximum shear stress. Hanzawa (1989) proposed a correction factor of 0.9 to estimate the shear strength for design considering the strain rate.

CLAY SAMPLE

Undisturbed samples used in this study were collected from points A and B in Osaka Bay for the second phase construction shown in Fig. 1. The borings carried out as deep as -150 m at A in 1981 and -400 m at B in 1995 indicated that the depth of water and thickness of Holo-

Fig. 4. Water content and consistency profile at points A and B

cene clay are 19 m and 23 m, respectively. Below the Holocene clay, lies the Pleistocene clay layer including some sandy soil layers, and its accumulation is in the order of age. The clay at -140 m in elevation was deposited 270,000 years ago and that at -230 m 440,000 years ago.

The upper part of the Holocene clay layer at from 0 to 20 m depths from the seabed will be improved by sand drains; thus its intact undrained shear strength is not important for practical design. Based on the stability analyses mentioned above, undrained shear strength from -40 to -120 m in elevation which affects the stability of the seawall structure, was mainly investigated. The quality of the undisturbed samples was discussed in Watabe (2001).

Figure 4 shows the consistency profiles including liquid limit w_L , plastic limit w_p and natural water content w_n at (a) point A and (b) point B. The distance between these two points is about 800 m. However, the tendencies that the existence of high plastic clay layer with $w_L > 100\%$ at -70 m in elevation and some clayey layers with $w_L = 70 \sim 80\%$ below -80 m in elevation are very similar at these two points. Taking account of the fact that Osaka Bay clay lies very widely and homogeneously, it can be said that soil properties at these two sites are almost identical. In the Pleistocene clay below -60 m, plasticity index I_p ranges from 20 to 70, and liquidity index I_L decreases with depth from 0.65 at -70 m to 0.4 at -120 m.

TEST RESULTS AND DISCUSSION

Figure 5 shows the stress-strain curves observed in (a) UC test, (b) UU test, (c) CU test, (d) K_0 CUC and K_0 CUE tests, and (e) DS test. Effective stress paths are also shown for (c') CU test, (d') K_0 CUC and K_0 CUE tests

and (e') DS test. Here, stresses are normalized by effective overburden pressure σ'_{v0} .

In the UC test for specimens sampled from -103 m (a), after peak strength at a strain of as little as 1.5%, shear strength decreased remarkably down to almost zero showing very brittle failure due to collapse of the structure developed by the ageing effect. Some of the peak strengths normalized by σ'_{v0} are larger than those of the UU test. In the UU test for specimens sampled from -105 m (b), stress-strain curves show very brittle behavior similar to those of the UC test; however, they have some residual stresses after the peak because of confining stress. The peak strengths are not in the order of the confining stresses, although the value of residual strength depends on the confining stresses, with a tendency that the residual strength increases when the confining stress increases. In the CU test (c), stress paths go up vertically to the peak, and stress-strain relationships show strain softening after the peaks.

In the K_0 CUC and K_0 CUE tests (d), the samples from the larger depth seem to show more brittle behaviors. The Pleistocene clay fails with a clearly defined slip surface not only for compression but also for extension. The Holocene clay, however, collapses without clear local failure, as shown in Fig. 6(a) and (b). Therefore, it can be said that the characteristics of shearing behavior of a clay change from ductile to brittle by developing structure due to ageing effects. In the DS test (e), remarkable strain softening is observed after the peak strength.

In the K_0 CUC and K_0 CUE tests (d) and DS test (e), stress-strain curves and stress paths for normally consolidated (NC) clay are also shown for the comparison between the intact clay and the NC clay. Since, NC clay collapses uniformly without local failure as shown in Fig. 6(c), tensile peak strength does not appear but strain

Fig. 5. Stress-strain relationships observed in (a) UC test, (b) UU test and (c) CU test, and stress paths observed in (c') CU test

hardening is observed. For this kind of clay, while compressive strength is defined at only 1% axial strain, extension strength is defined at 15% axial strain. Taking account of strain softening after compression peak, it is very difficult to define the shear strength for design in consideration of the strain compatibility concept proposed by Koutsftas and Ladd (1985). On the other hand, in the DS test, shear strength can be simply defined as a peak stress and this value can be directly used as a design parameter.

Figure 7 shows strength anisotropy obtained in K_0 CU tests defined as the ratio of undrained tensile strength s_{uE} to compressive strength s_{uC} , and the ratio was 0.6~0.65. Tsuchida (2000) indicated that the ratio of the typical Japanese Holocene marine clays ranged from 0.6 to 0.8. It can be said that the ratio of Osaka Bay Pleistocene Clay is in the lower bound of the ratio for Japanese Holo-

cene marine clays.

Figure 8 shows the profiles of apparent strain at failure ϵ_f (or γ_f) in each shear test. Here, apparent shear strain at failure for the DS test in Fig. 8(e) is defined as horizontal displacement divided by the specimen height in the same manner as Hanzawa (1995). In UU tests with different confining stresses (b), and in the UC test without confining stress (a), strains at failure in each test are almost the same, implying that strain at failure ϵ_f is not a function of confining stress.

In the K_0 CUC and K_0 CUE tests (d), strains at failure of compression tests ϵ_{fC} range from 2 to 6% for the Holocene clay, while most of them are as small as 1% for the Pleistocene clay. Strains at failure of extension tests ϵ_{fE} are more than 15% in the Holocene clay, while they decrease with depth and are about 3~4% for the depth of -100 m in the Pleistocene clay.

Fig. 5. Stress-strain relationships observed in (d) K_0 CU tests and (e) DS test, and stress paths observed in (d') K_0 CU tests and (e') DS test

Figure 9 shows undrained shear strength profiles without correction factor determined by each test: (a) UC test, (b) UU test with $\sigma_3=98$ kPa, (b') UU test with $\sigma_3=392$ kPa, (b'') UU test with $\sigma_3=686$ kPa, (c) CU test, (d) K_0 CUC and K_0 CUE tests and (e) DS test. The strength increase ratios with depth $\Delta s_u/\Delta z$ obtained from these tests are 1.85, 1.97, 2.35, 2.42, 2.88, 2.36 and 2.20 kPa/m, respectively as indicated in the figures.

The UC test is equivalent to a UU test with $\sigma_3=0$ kPa; thus if we compare (a), (b), (b') and (b''), it can be said that strength increase ratio $\Delta s_u/\Delta z$ increases when confining stress σ_3 increases in the UU test, from $\Delta s_u/\Delta z=1.85$ kPa/m at $\sigma_3=0$ kPa to $\Delta s_u/\Delta z=2.42$ kPa/m at $\sigma_3=686$ kPa. In a UU test with high confining stress such as (b'') $\sigma_3=686$ kPa, $\Delta s_u/\Delta z$ is larger than that by K_0 CU, because the UU test does not consider strength anisotropy (only considering compression strength) and its strain rate is 1.0%/min which is 10 times larger than that in the K_0 CU test. For typical Japanese marine clays,

undrained shear strength increases 8% when the strain rate becomes 10 times larger from 0.1%/min (Tsuchida, 2000). Thus, in a UU test with high confining stress, an overly large undrained shear strength is possibly obtained.

Generally for K_0 CU tests, an axial strain rate of 0.01%/min is recommended; however, the K_0 CU test in this study was conducted with 0.1%/min, 10 times higher than the recommended value, to carry out the tests efficiently. Thus, the observed shear strength is possibly 8% larger (Tsuchida, 2000), and the $\Delta s_u/\Delta z$ with a strain rate of 0.01%/min in K_0 CU will be 2.19 kPa/m.

Not only the value of undrained shear strength, but also the reliability of these values are very important geotechnical issues in practical design. Here, a standard deviation of $(s_u - s_u^*)/s_u^*$, which is adopted as a representative parameter of data scattering (coefficient of variation) is calculated for each strength profile shown in Fig. 9, where s_u is undrained shear strength obtained by

Fig. 6. Specimens dried in oven after test, (a) Holocene clay, (b) Pleistocene clay and (c) K_0 normally consolidated clay

laboratory test and s_u^* is shear strength calculated by the following equation with strength increase ratio with depth $\Delta s_u / \Delta z$,

$$s_u^* = \frac{\Delta s_u}{\Delta z} (-z - 19.2) \quad (2)$$

Here z is the elevation and $z = -19.2$ m corresponds to the seabed. The values of standard deviation for each test are 0.323, 0.342, 0.439, 0.515, 0.140, 0.075 and 0.234, respectively. The $\Delta s_u / \Delta z$ normalized by $\Delta s_u / \Delta z$ of K_0 CU test, and standard deviation of $(s_u - s_u^*) / s_u^*$ are plotted in Fig. 10. It can be said that the data set with the minimum scattering can be obtained by K_0 CU test. A second-class data set can be obtained by CU and DS tests. These facts imply that the recompression method, in which the specimen is consolidated under the in-situ stress, gives a data set with minimum scattering. On the other hand, UC and UU tests, in which the specimen is not consolidated, give

Fig. 7. Strength anisotropy defined as ratio of tensile strength $s_{u(E)}$ to compressive strength $s_{u(C)}$ in K_0 CU test

very scattered test results. Moreover, geotechnical engineers should be careful about the relations in the order of these undrained shear strengths. Table 2 is a summary of undrained shear strength obtained in this study. In this table, shear strengths corrected with the factors or considering the strain rates as mentioned above are also shown: i.e. with no correction for UC and UU, with a correction factor of 0.75 for CU, with a reduction of 8% for K_0 CU, and with a correction factor of 0.9 for DS.

When $\Delta s_u / \Delta z$ of 2.19 kPa/m for K_0 CU with a strain rate of 0.01%/min is put at 1, $\Delta s_u / \Delta z$ of 2.16 kPa/m for CU test defined with a correction factor of 0.75 becomes 0.99, which coincides well with the $\Delta s_u / \Delta z$ for K_0 CU. However, for direct shear test, $\Delta s_u / \Delta z$ based on a correction factor of 0.9 corresponds to only 0.90. Generally for Japanese Holocene marine clays, the corrected direct shear strength is coincident with the corrected undrained shear strength of K_0 CU, but in this study for Pleistocene clay, the corrected $\Delta s_u / \Delta z$ in direct shear test is 10% smaller than that of the corrected $\Delta s_u / \Delta z$ in the K_0 CU test. According to various case studies on actual failure of marine clays, where a large slip occurring several hours after the threshold of a failure is observed, it is said that the shear strain rate of the actual failure is of the order of 0.01%/min. In these cases, undrained shear strength for design, in which the strain rate is 0.01%/min, becomes as follows:

$$\begin{aligned} \frac{\Delta s_u}{\Delta s} &= \frac{1}{4} \left(\frac{\Delta s_{uC}}{\Delta z} + 2 \cdot \frac{\Delta s_{uD}}{\Delta z} + \frac{\Delta s_{uE}}{\Delta z} \right) \\ &= \frac{1}{2} \left\{ \frac{1}{2} \left(\frac{\Delta s_{uC}}{\Delta z} + \frac{\Delta s_{uE}}{\Delta z} \right) + \frac{\Delta s_{uD}}{\Delta z} \right\} \\ &= \frac{1}{2} \left(\frac{2.19}{2} + 1.98 \right) = 2.09 \text{ kPa/m} \quad (3) \end{aligned}$$

Fig. 8. Strains at failure in (a) UC test, (b) UU test, (c) CU test, (d) K_0 CU test and (e) DS test

Fig. 9. Shear strength profile with depth; (a) UC test, (b) UU test with $\sigma'_3 = 98$ kPa, (b') UU test with $\sigma'_3 = 392$ kPa and (b'') UU test with $\sigma'_3 = 686$ kPa

where, s_{uC} and s_{uE} are the undrained compressive and tensile shear strengths for K_0 CU tests, respectively, and s_{uD} is the direct shear strength with a strain rate of 0.01%/min. Again, according to Fig. 3, at a $\Delta s_u / \Delta z$ of 0.09 kPa/m, which is smaller than 2.1 kPa/m, the safety factor F_s becomes a little smaller, and the stability of the seawall structure is governed by the Pleistocene clay deposit.

The profile of undrained shear strength in the Pleistocene clay layer, represented by a strength increase ratio with a depth $\Delta s_u / \Delta z$, depends on the testing methods. The stability analysis for a typical seawall structure in the

second phase of Kansai International Airport is very sensitive, around $\Delta s_u / \Delta z$ of 2.09 kPa/m. Since the safety factor calculated is smaller than that required in design ($F_s \geq 1.3$) before consolidation in the Pleistocene clay, the margin for safety is not so great. When a slip takes place along a circle passing through the Pleistocene clay, the affected area will be quite large and cause serious damage. Consequently, the design parameter should be determined very carefully with engineering sense, and construction control including field observation will be important.

Fig. 9. Shear strength profile with depth; (c) CU test, (d) K_0 CU test and (e) DS test

CONCLUSIONS

In this study, the UC test, UU test, CU test, K_0 CU compression and extension tests, and DS test were carried out for undisturbed samples, and the following conclusions were obtained by discussing those results.

- 1) In UC and UU tests, shear strength decreased remarkably after reaching a peak with very brittle mechanical behavior; however, the residual strength increased when confining stress increased. Undrained shear strengths obtained by UC and UU tests became larger with an increase of confining stress. Since only a compression test was carried
- 2) In the K_0 CU test, strain softening was observed more remarkably for a sample collected from a larger depth, and the peak for compression and extension became clearer. In extension tests for a specimen in normal consolidation, defining undrained shear strength is difficult because the strength increases

without consideration of strength anisotropy, and the strain rate of 1.0%/min was larger than that of the CU, K_0 CU tests, the shear strength obtained with a large confining stress is possibly higher than that obtained by K_0 CU test. The strain at failure in the UC and UU tests is not a function of confining stress.

Table 2. Summary of laboratory tests for Osaka Bay Pleistocene clay

Test	Confining or consolidation stress	Shearing rate	Definition of s_u	$\Delta s_u / \Delta z$ kPa/m	$(\Delta s_u / \Delta z) /$ $(\Delta s_u / \Delta z)_{K_0CU}$	Standard deviation of $(s_u - s_u^*) / s_u^*$	With correction factor	
							$\Delta s_u / \Delta z$ kPa/m	$(\Delta s_u / \Delta z) /$ $(\Delta s_u / \Delta z)_{K_0CU}$
(a)	UC 0	$\Delta \varepsilon_a / \Delta t = 1.0\% / \text{min}$	Half of peak compression strength	1.85	0.78	0.323	—	—
(b)	UU 96 kPa	$\Delta \varepsilon_a / \Delta t = 1.0\% / \text{min}$	Half of peak deviator stress	1.97	0.83	0.342	—	—
(b')	UU 392 kPa	$\Delta \varepsilon_a / \Delta t = 1.0\% / \text{min}$	Half of peak deviator stress	2.35	1.00	0.439	—	—
(b'')	UU 686 kPa	$\Delta \varepsilon_a / \Delta t = 1.0\% / \text{min}$	Half of peak deviator stress	2.42	1.03	0.515	—	—
(c)	CU $(2/3)\sigma'_{v0}$	$\Delta \varepsilon_a / \Delta t = 0.1\% / \text{min}$	$0.75 \times (\text{Half of peak deviator stress})$	2.88	1.22	0.140	2.16	0.99
(d)	K_0CU $\sigma_1 = \sigma'_{v0}$ $\sigma_3 = \sigma'_{h0} = K_0 \sigma'_{v0}$ ($K_0 = 0.5$)	$\Delta \varepsilon_a / \Delta t = 0.1\% / \text{min}$	Average of compression and extension peak deviator stresses	2.36	1	0.075	2.19	1
(e)	DS σ'_{v0}	$\Delta d / \Delta t = 0.25 \text{ mm/min}$	Peak shear strength	2.20	0.93	0.234	1.98	0.90

Fig. 10. Undrained shear strength increase ratio normalized by ratio of K_0CU test, and standard deviation of $(s_u - s_u^*) / s_u^*$ for each test

with strain hardening. On the contrary, for a specimen of Pleistocene clay with strong ageing effect sampled from larger depth, a clear peak strength was observed.

- 3) The recompression method, in which the specimen is consolidated under in-situ stress, gives test results with much less scattering. On the other hand, UC and UU tests, in which specimens are not consolidated, give very scattered test results.
- 4) For undrained shear strengths obtained by each test with correction factors, the strength of the DS test is smaller than those of K_0CU and CU tests. This fact is derived from the relatively larger strength anisotropy in the Pleistocene clay. Undrained shear strength increase ratio with depth $\Delta s_u / \Delta z$ for design

corresponding to a strain rate of 0.01%/min is obtained as 2.09 kPa/m, considering strength anisotropy. This implies that the Pleistocene clay deposit governs the stability of the seawall structure in the second phase of Kansai International Airport.

- 5) Since the safety factor calculated is smaller than that required in design ($F_s \geq 1.3$) before consolidation in the Pleistocene clay, the margin for the stability on a typical seawall structure in the second phase is not so great. Consequently the design parameters of the shear strength for the Pleistocene clay should be determined very carefully, and construction control with field observations will be necessary to complete the projects safely.

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