THREE CASE STUDIES FOR SHORT TERM STABILITY OF SOFT CLAY DEPOSITS

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ABSTRACT

Three case studies for short term stability problem for the structures constructed on normally consolidated clays at and near failure are described in detail including discussions of the shear strength of soft clays determined by various methods currently practiced.

Undrained strength to be used for stability analysis is briefly summarized and the shear strength determined by various methods are discussed as well as considerations on reliability of stability analyses. It is shown that 1) shear strength of soft clay deposits determined by the methods currently practiced is uncertain and 2) results of stability analyses evaluated using such uncertain strength values are unreliable.

Results of stability analyses for the three structures constructed on normally consolidated clays at and near failure are described in detail as well as the undrained strength characteristics of the clays at the three sites. Stability analyses were carried out using the strength values from 1) the modified Bjerrum's method, 2) the SHANSEP method, 3) in-situ vane test, 4) unconfined compression test and 5) Bjerrum's stability chart. Factors of safety obtained from each of the methods are considerably different from each other and strongly demonstrated the importance to recognize the regional variations in evaluating the proper soil behaviour.

Key words: anisotropy, clay, consolidated undrained shear, shear strength, stability analysis, test procedure (IGC : E 6/D 6)

INTRODUCTION

Stability of soft clay deposits is one of the major subjects in geotechnical engineering and many efforts have been devoted on this theme. As a consequence, various methods for determining the shear strength of clay in-situ for practical use have been proposed and practiced. However, it should be noted that each of the methods is generally based on different concepts and there usually

exist considerable differences in the shear strength values obtained from each of the methods. In spite of this fact, there have been reported many papers to say that quite good agreement was obtained between results of stability analysis and field behaviour of clay foundations. When these reports are examined, however, it is sometimes found that strength determination is questionable or even wrong. It is desirable to clarify uncertainties involved in shear strength de-

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terminated by various methods currently practiced and in stability analyses for soft clay deposits.

This paper first describes the undrained strength to be used for practical stability problems and then discusses uncertainties and limits involved in shear strength determined by various methods and in stability analyses having been reported. Three case studies for the structures constructed on normally consolidated clays at and near failure are described in detail as well as the undrained strength characteristics of the clays found at the three sites. Results of stability analyses obtained using strength values determined by various methods are presented and compared with the field behaviour and actual failure plane of the structures.

UNDRAINED STRENGTH TO BE USED FOR STABILITY ANALYSIS

The shear strength of a clay to be used for stability analysis will be given by Eq. (1) when its undrained strength is noted.

$$S_{up} = S_{uf} \times \mu_A \times \mu_R \times \alpha \times \beta \quad (1)$$

where S_{up} = undrained strength for stability analysis

S_{uf} = in-situ undrained strength

μ_A = correction coefficient for strength anisotropy

μ_R = correction coefficient for strain rate effect

α = correction coefficient for S_{uf} due to consolidation or rebound

β = correction coefficient for progressive failure

In-situ undrained strength is the strength such as compression, extension or direct simple shear strength in the field. It is possible to use in-situ vane strength as S_{uf} in Eq. (1) from practical point of view as will be demonstrated later. The value of μ_A depends on what strength is used as S_{uf} . The value of μ_R will depend on various factors such as property of clay and loading conditions, and would not be unconditionally determined. Ladd and Foott (1974) pointed out that this value is affected by plasticity of

the clay and creep susceptibility. The value of α is less than 1.0 when long term stability of excavated slope is examined, while will be equal to or more than 1.0 for short term stability problems including under multi-stages loading conditions. The value of β should depend on many factors such as property of clay, kinds of stability problems (short or long term), loading conditions, rain fall and many other factors. Quantitative evaluation of this value will be very difficult as demonstrated by Skempton (1977) in his extensive studies on long term stability for London clay.

The relationship shown in Eq. (1) should be considered in determination of the shear strength in terms of effective stress because 1) c' and A (pore pressure coefficient) values, particularly A values, depend on the strain rate, 2) the effect of stress induced anisotropy on ϕ' and A values have been reported to be significant (Ladd, 1965; Ladd, et al., 1977; Mitachi and Kitago, 1980), and 3) the effect of progressive failure is common for both effective and total stress analyses.

UNCERTAINTIES INVOLVED IN SHEAR STRENGTH DETERMINED BY VARIOUS METHODS AND IN STABILITY ANALYSIS

Unconfined compression strength, q_u and in-situ vane strength, $S_{uf(v)}$ have long time been used for short term stability analysis based on an assumption that a relationship shown by Eq. (2) holds for these two strengths.

$$\left. \begin{aligned} q_u/2 &= S_{uf} \times \mu_A \times \mu_R \\ S_{uf(v)} &= S_{uf} \times \mu_A \times \mu_R \text{ or } S_{uf} \times \mu_A \end{aligned} \right\} \quad (2)$$

The values of $q_u/2$ are significantly smaller than the in-situ compression strength, $S_{uf(c)}$ because of the stress release and mechanical disturbance through sampling process. However, this strength is of unconservative from the view point of strength anisotropy and the strain rate effect because it is compression strength at rapid strain rate (generally 1.0%/min in Japan). In the use of $q_u/2$ values for stability analysis, it is assumed that these effects will be cancelled.

The reason why the strain rate effect is sometimes considered for $S_{uf(v)}$ is that strength decrease due to stress release and mechanical disturbance is less in in-situ vane test (hereafter called F.V. test) than in unconfined compression test (hereafter called q_u test). The relationship shown by Eq. (2) may hold for some clays from practical point of view, however, this is fundamentally empirical. As a consequence, stability of clay foundations evaluated from these two strengths is generally unreliable as demonstrated later. It should be noted here that the use of $q_u/2$ values for stability analysis may give unconservative factor of safety when recent development in undisturbed sampling technique is considered.

Bjerrum (1972) proposed a stability chart in which factor of safety calculated using $S_{uf(v)}$ values measured in the field is corrected according to the plasticity index, I_p , of the clay. This chart is based on an assumption that factors affecting the shear strength of in-situ clay can be directly related to the I_p . Schmertmann and Morgenstern (1977) pointed out the importance at the outset regional variations in soil behaviour and insisted that engineering geology is reflected in more than the I_p . The author agrees with their opinion.

Strength ratio between compression and extension strength obtained from K_0 -consolidated triaxial tests for four marine clays found in the Arabian Gulf and in Tokyo Bay are plotted versus I_p in Fig. 1. The strength ratio indicates a reverse tendency

Fig. 1. Strength ratio between compression and extension strength, $S_{u(e)}/S_{u(c)}$ obtained from K_0 -consolidated triaxial tests plotted versus I_p for four marine clays

against Bjerrum's indication. It should also be pointed out that $S_{uf(v)}$ values are considerably higher than the actual strength values for such clays which contain thin sand seams, shell fragment and organic material. Such clays, which were probably formed under mild and humid environment, are very commonly found in the world. It is the author's opinion that the Bjerrum's stability chart is highly empirical and its use should be limited as one of the checking methods at the design stage as will be demonstrated later.

Ladd and Foott (1974) proposed the SHANSEP (Stress History and Normalized Engineering Properties) method which is based on a quite simple assumption that the overconsolidated state of a given in-situ clay was caused by stress release alone. In this method, in-situ undrained strength, S_{uf} of a clay is given by Eq. (3) as presented in Fig. 2.

$$S_{uf} = S_{un}/\bar{p} \times n \times \bar{p}_c \quad (3)$$

where $S_{un}/\bar{p}$ = undrained strength ratio in the normally consolidated state

$\bar{p}_c$ = preconsolidation pressure

n = correction coefficient for strength decrease due to rebound

The values of n are obtained from K_0 -consolidated (at overconsolidation ratio, OCR = 1.0) and K_0 -rebound (at corresponding OCR in the field) undrained strength tests. They conducted a lot of stability analyses using the strength values obtained from this method and reported that results of stability

Fig. 2. Idealized plot showing the S_u - $\bar{p}$ path and the undrained strength for N.C. aged clay and O.C. young clay

analyses showed good agreement with the field behaviour of clay foundations (Ladd and Foott, 1974; Azzouz, et al., 1981).

Hanzawa (1979), Hanzawa, et al. (1980), and Hanzawa and Kishida (1982) proposed the modified Bjerrum's method which strongly objects to the concept adopted in the SHANSEP method and also reported that results of stability analyses obtained using the strength values determined by this method showed quite good agreement with the field behaviour of structures constructed at and near failure. In this method a specimen prepared from an undisturbed sample is K_0 -consolidated at the same stress in the field and then subjected to shear. Reconsolidation at the field stress is terminated when excess pore water pressure has mostly dissipated (for triaxial tests) or when primary compression has been achieved (for direct simple shear tests). The value of S_{uf} of normally consolidated aged clay (hereafter called N.C. aged clay), which is in an over-consolidated state even though it has not been subjected to release of the overburden because of the additional strength developed by aging effect, is given by Eq. (4) as presented in Fig. 2 (Hanzawa and Kishida, 1981; 1982).

$$S_{uf} = S_{un} / \bar{p} \times \bar{p}_c \quad (4)$$

The shear strength obtained from the modified Bjerrum's method and the SHANSEP method is S_{uf} in Eq. (1) and then correction for strength anisotropy and the strain rate effect is required when used for stability analysis. Noticing that simple shear strength approximately equals to the average strength between compression and extension strength, Ladd and Foott (1974) used the shear strength obtained from simple shear test for such a case where flat level ground is subjected to loading. Hanzawa et al. (1979) used triaxial compression and extension strength for stability analysis as indicated in Fig. 3 where S_u is defined by $(\sigma_1 - \sigma_3)_{\max} / 2$. The strain rate to be adopted in determining the undrained strength for stability analysis would not be unconditionally determined as previously described. The

Fig. 3. Schematic presentation how to use $S_{u(c)}$ and $S_{u(e)}$ for stability analysis

Table 1. Strain rate recommended

Researcher	Strain rate (%/min)
Berre & Bjerrum	$2-3 \times 10^{-5}$ (triaxial)
Ladd & Foott	$0.8-1.5 \times 10^{-2}$ (triaxial) 8×10^{-2} (simple shear)
Kishida et al.	$10^{-3}-10^{-2}$ (triaxial)

strain rate recommended by Berre and Bjerrum (1973), Ladd and Foott (1974), and Kishida et al. (1983) are summarized in Table 1. Application of the two methods were discussed in detail by Hanzawa and Kishida (1982). They clearly indicated that the shear strength of aged clay can be determined by the modified Bjerrum's method, while the use of the SHANSEP method should be limited to young clay.

LaRoche et al. (1974) and Trak et al. (1980) conducted stability analyses using the undrained strength at large strain ($\div 15\%$) from UU or CU triaxial compression tests and reported that good results were obtained. This method is so called the USALS (Undrained Strength at Large Strain) method. The undrained strength obtained from this method is given by Eq. (5) according to their explanation.

$$S_u \text{ from USALS} = S_{uf} \times \beta \times \mu_A \times \mu_R \quad (5)$$

They said that the undrained strength at large strain is probably the strength after correction for progressive failure and incorporates the effects of anisotropy and strain rate for some sensitive cemented clays in Canada. But there is few possibility that this statement can be applied to the other clays. In addition, testing procedure in this method is uncertain whether CU or UU test is carried

out. Because of these reasons, it can be stated that the USALS method is of highly empirical and be available only to some Canadian clays.

Janbu (1977) summarized a lot of case studies for both short and long term stability problems and pointed out that stability analyses with the effective stress method showed quite good agreement with the field behaviour of clay foundations. In his effective stress analysis, however, the effects of strength anisotropy and strain rate as well as the effect of progressive failure were not considered at all though it was not known how was the strain rate in triaxial compression test that he carried out. For long term stability problems, particularly for heavily overconsolidated clay, it is suggested that the effect of progressive failure should be considered. There is a question on the case studies reported by Janbu at least regarding this point, " $\beta=1.0$ ". On the other hand, Foott (1977) reported that factors of safety with the effective stress method for embankments constructed at and near failure were 25% to 50% larger than the more correct values given by the $\phi=0$ method. Hanzawa et al. (1982) also reported a case study which supports Foott's indication and demonstrated the importance to consider the strain rate effect in evaluating excess pore water pressure.

From the discussions given in this section, it can be concluded that 1) the shear strength values obtained from various methods currently practiced are uncertain and 2) results of stability analyses obtained using such uncertain strength values are unreliable even though it is reported that good results were obtained.

STABILITY ANALYSES FOR THREE STRUCTURES CONSTRUCTED AT AND NEAR FAILURE

The author has been concerned with the stability analysis for harbor facilities which were constructed on soft marine clays under very low factors of safety. In this section

of the paper, stability analyses for three structures constructed at and near failure are described in detail as well as the undrained strength characteristics of the clays found at the three sites. The three case studies were experienced in 1976, 1978 and 1981, respectively. Since the author's opinion on the shear strength of soft clay deposits has been progressed through these period, it should be pointed out here that the procedures for determining the shear strength for the stability analysis used for each case study is somewhat different from each other.

Outline of Three Case Studies

The three case studies described in this section are as follows: -

1) Fao Steel Jetty in the Arabian Gulf in 1976

This is a jetty constructed on the right bank of the river mouth of the Tigris and Euphrates rivers in the Arabian Gulf. A stability analysis was carried out using $q_u/2$ values after two months of commencement of the construction. A minimum factor of safety, $F.S._{min}$ of 0.6 was obtained from the analysis. In order to evaluate the possibility of sliding failure, a series of field and laboratory investigations were carried out parallel with the construction. Through investigation a special geotechnical property of Fao clay was brought to a light. The modified Bjerrum's method was indeed developed by the investigation conducted for the Fao Steel Jetty. A stability analysis was carried out using the strength values obtained from the modified Bjerrum's method and a $F.S._{min}$ value more than 1.2 was obtained. The construction of the Jetty was progressed and completed on time. The Fao Steel Jetty is in a stable condition up to the present time. This case study was already reported in detail by Hanzawa (1977) and Hanzawa et al. (1979).

2) Khor Al-Zubair Embankment in the Arabian Gulf in 1978

Five embankments for preloading were rapidly constructed on Khor Al-Zubair clay found on the flood plain developed by the

Tigris and the Euphrates rivers. This site is about 70 km apart from the Fao Steel Jetty. In order to shorten the preloading time, the embankment was designed under a minimum possible safety. Because of a slight difference in slope and height between the designed embankment and the constructed embankments, one embankment failed immediately after completion and two others were probably on the verge of failure judging from their settlement behaviour during construction. This case study was already reported in detail by Hanzawa et al. (1980).

3) Daikoku-cho Dike in Tokyo Bay in 1981

A dike for reclaimed land was constructed on a highly plastic marine clay found in Tokyo Bay. Immediately after completion, it failed and sunk into the sea. The author was given a chance to investigate the cause of failure. This case study was already reported in detail by Kishida et al. (1983).

Undrained Strength of the Clays Found at the Three Sites

1) Fao clay

The largest characteristic for Fao clay ($I_p=15\%$) found at the river mouth of the Tigris and the Euphrates rivers was that $\bar{p}_c$ values obtained from a oedometer test, in which consolidation load was applied by doubling the previous load and loading time was 2 hours, were smaller than $\bar{p}_0$ values. However, ground settlement was not observed in Fao city which was developed on Fao clay about 200 years ago and in-situ vane test results did not suggest any possibility of underconsolidated state. It was suggested, therefore, that the underconsolidated state of this clay was caused by a special property of Fao clay. A special oedometer test, in which consolidation load approximating $\bar{p}_0$ was applied in small increments by 10 to 20 kN/m² reaching 2 hours in consolidation time, was carried out. The values of $\bar{p}_c$ obtained from the two kinds of oedometer tests are indicated and compared with $\bar{p}_0$ in Fig. 4. As indicated in the figure, $\bar{p}_c$ values obtained from the special oedometer

Fig. 4. Comparison of preconsolidation pressure, $\bar{p}_c$ values obtained from the conventional and the special oedometer tests for Fao clay and their relations to the effective overburden pressure, $\bar{p}_0$

test showed good agreement with or slightly higher than $\bar{p}_0$. From the tests results shown in Fig. 4, an important geotechnical property of Fao clay was suggested as follows:-

[Fao clay is seriously disturbed in structure by stress release through the sampling process. This change in structure is so large that it can not be explained by the decrease in effective stress alone. It would be rather correct to say that sampling gives a clay quite different from the clay in the field. However, the undisturbed structure in the field can be restored when the sample is reconsolidated at the same stress it carried in the field. In order to ensure this complete restoration, consolidation load must be applied in small increments. In the conventional oedometer test, the field structure can not be completely restored because of large increments in consolidation load. This gives smaller $\bar{p}_c$ values than the actual values. In-situ undrained strength of Fao clay, therefore, can be obtained from consolidated undrained strength tests in which the specimen prepared from an undisturbed sample is reconsolidated at the same stress it carried in the field and then subjected to shear]. Bjerrum (1973) suggested the possibility that some kinds of clay drastically change their

Fig. 5. Comparison of $S_{uf(v)}$ (from F. V. test), $S_{uf(c)}$ and $S_{uf(e)}$ (from the modified Bjerrum's method) for Fao clay

Fig. 6. In-situ undrained strength ratio, $S_{uf(v)}/\bar{p}_0$ and $q_u/2\bar{p}_0$ values of Fao clay plotted versus elevation

structure when sampled. It can be concluded that Fao clay is one of such clays and can be classified as a collapsible clay. As previously described, the modified Bjerrum's method was indeed developed by the test results obtained from the special oedometer test.

The relationship between $S_{uf(v)}$ (from F. V. test), $S_{uf(c)}$ and $S_{uf(e)}$ (from K_0 -consolidated triaxial compression and extension tests at the strain rate, $r=0.09\%/min$ following the modified Bjerrum's method) is presented in Fig. 5. A unique relationship was obtained among the three strengths as follows:-

$$\left. \begin{aligned} S_{uf(c)} &= 1.25 S_{uf(v)} \\ S_{uf(e)} &= 0.75 S_{uf(c)} = 0.94 S_{uf(v)} \end{aligned} \right\} \quad (6)$$

On the other hand, a total of 78 F. V. tests were performed. The values of $S_{uf(v)}$ as the ratio to $\bar{p}_0$, $S_{uf(v)}/\bar{p}_0$ are plotted versus elevation in Fig. 6 together with $q_u/2\bar{p}_0$ values. The values of $q_u/2\bar{p}_0$ show considerably smaller values than $S_{uf(v)}/\bar{p}_0$ values, which must have been caused by the special property of this clay previously pointed out. When the $S_{uf(v)}/\bar{p}_0$ values are represented by the solid line shown in Fig. 6, the values of $S_{uf(c)}$ and $S_{uf(e)}$ can be easily obtained using the strength ratio shown in Eq. (6)

In carrying out the stability analysis, $S_{uf(c)}$ and $S_{uf(e)}$ values were used for the right side and for the left side of the center of the arc, respectively as shown in Fig. 3.

When the value of $S_{uf(v)}$ is used as S_{uf} in Eq. (6), the correction coefficient for anisotropy, μ_A can be given using the length of the arc, l_1 and l_2 shown in Fig. 3 as follows:-

$$\mu_A = [1.25l_1 + 0.94l_2] / [l_1 + l_2] \quad (7)$$

It was impossible to conduct the test to investigate the strain rate effect because of the capacity of triaxial apparatus prepared at the site. In the stability analysis it was assumed that the correction coefficient for the strain rate effect, μ_R is 1.0.

$$\mu_R = 1.0 \quad (8)$$

The undrained strength for the stability analysis, S_{up} from 1) the modified Bjerrum's method, 2) F. V. test and 3) q_u test can be obtained using the $S_{uf(v)}/\bar{p}_0$, $q_u/2\bar{p}_0$ -elevation relationship (shown in Fig. 6), strength ratio (shown in Eq. (6)) and μ_A (shown in Eq. (7)) as follows:-

$$\begin{aligned} S_{up} &= S_{uf(v)}/\bar{p}_0 \times \bar{p}_0 \text{ (F. V. test)} \\ &= q_u/2\bar{p}_0 \times \bar{p}_0 \text{ (} q_u \text{ test)} \\ &= S_{uf(v)}/\bar{p}_0 \times \bar{p}_0 \times \frac{[1.25l_1 + 0.94l_2]}{[l_1 + l_2]} \\ &\quad \text{(modified Bjerrum's method)} \end{aligned} \quad (9)$$

The values of S_{up} obtained from Eq. (9) are presented in Fig. 7 where the values from the modified Bjerrum's method is for the case, $l_1 = l_2$.

Fig. 7. Undrained strength of Fao clay for stability analysis determined by 1) the modified Bjerrum's method, 2) F.V. test and 3) q_u test plotted versus depth

Fig. 8. Comparison of $S_{uf(v)}$ (from F.V. test), $S_{uf(c)}$ and $S_{uf(e)}$ (from the modified Bjerrum's method) for Khor Al-Zubair clay

2) Khor Al-Zubair clay

Fig.8 indicates the relationship among $S_{uf(c)}$, $S_{uf(e)}$ (from the modified Bjerrum's method at $r=0.09\%/min$) and $S_{uf(v)}$ (from F.V. test) of this clay. A unique relationship was obtained as shown by Eq.(10).

$$\left. \begin{aligned} S_{uf(c)} &= 1.6 S_{uf(v)} \\ S_{uf(e)} &= 0.7 S_{uf(c)} \end{aligned} \right\} \quad (10)$$

In order to obtain the undrained strength ratio in the N.C. young state, $S_{un}/\bar{p}$, K_0 -

Fig. 9. Compression strength in the normally consolidated state, $S_{un(c)}$ of Khor Al-Zubair clay obtained from K_0 -consolidated triaxial test plotted versus vertical consolidation stress, $\bar{\sigma}_{vc}$

consolidated triaxial compression test, in which specimens prepared from undisturbed samples were K_0 -consolidated at $OCR=1.0$ and then subjected to shear by compression, was carried out at $r=0.09\%/min$. The undrained strength ratio corresponding to compression strength in the N.C. young state, $S_{un(c)}/\bar{p}$ was obtained as shown in Fig. 9.

$$S_{un(c)}/\bar{p} = 0.355 \quad (11)$$

Since Khor Al-Zubair clay ($I_p=30\%$) is a N.C. aged clay (Hanzawa, et al., 1980; Hanzawa and Kishida, 1982), the strength ratio shown in Eq.(10) is constant irrespective of OCR values. When the strength ratio in Eq.(10) is multiplied by $S_{un(c)}/\bar{p}$ value, the values of $S_{un(e)}/\bar{p}$ and $S_{un(v)}/\bar{p}$ can be given by:

$$\left. \begin{aligned} S_{un(e)}/\bar{p} &= 0.355 \times 0.7 = 0.25 \\ S_{un(v)}/\bar{p} &= 0.355 \times 1/1.6 = 0.22 \end{aligned} \right\} \quad (12)$$

The values of OCR of this clay are then given using Eq.(4) as follows:-

$$\left. \begin{aligned} OCR &= \bar{p}_c/\bar{p}_0 = S_{uf}/\bar{p}_0 \div S_{un}/\bar{p} \\ &= S_{uf(c)}/\bar{p}_0 \div 0.355 \\ &= S_{uf(e)}/\bar{p}_0 \div 0.25 \\ &= S_{uf(v)}/\bar{p}_0 \div 0.22 \end{aligned} \right\} \quad (13)$$

A lot of oedometer test, in which consolidation load was applied by doubling the previous load and consolidation time was 2 hours, were conducted. The values of OCR from the oedometer tests and from the Eq.(13) are compared in Fig.10. As indicated

Fig. 10. Comparison of overconsolidation ratio, OCR values determined by 1) oedometer test, 2) F.V. test, 3) K_0 -consolidated triaxial compression and extension tests for Khor Al-Zubair clay

Fig. 11. Correction coefficient for strength decrease due to rebound, n for Khor Al-Zubair clay obtained from K_0 -consolidated and K_0 -rebounded triaxial compression tests plotted versus OCR

in the figure, OCR values from the two tests showed good agreement with each other.

Determination of the undrained strength for the stability analysis was also carried out by the SHANSEP method. Fig. 11 indicates the n values (shown in Eq. (3)) versus OCR obtained from CK_0RK_0UC test in

Fig. 12. Change in undrained strength with change in strain rate for Khor Al-Zubair clay obtained from CK_0UC_r test

which specimens from undisturbed samples were first K_0 -consolidated at $OCR=1.0$, then K_0 -rebounded at a given OCR and finally subjected to shear by compression at $r=0.09\%/min$. Determination of the undrained strength by the SHANSEP method is then available using OCR values and n values shown in Figs. 10 and 11.

In order to investigate the strain rate effect on undrained strength, CK_0UC_r test, in which specimens were K_0 -consolidated at $OCR=1.0$ and then subjected to shear by compression under different strain rates, was conducted on a remoulded and reconsolidated sample. The change in undrained strength with the strain rate is presented in Fig. 12. Since all the triaxial tests except this test were subjected to shear at $r=0.09\%/min$, the strength ratio at a given r and at $r=0.09\%/min$, $\frac{[S_u \text{ at a given } r]}{[S_u \text{ at } r=0.09\%/min]}$ is plotted versus strain rate in the figure. The strength at $r=10^{-3}$ to $10^{-2}\%/min$ is about 90% of that at $r=0.09\%/min$. Then the value of μ_R is given by:

$$\mu_R=0.9 \quad (14)$$

On the other hand, embankment is constructed on flat level ground. The value of μ_A is given by Eq. 15 when $S_{uf(c)}$ value is taken as S_{uf} in Eq. (1).

$$\mu_A=(1+0.7)/2=0.85 \quad (15)$$

The values of S_{up} for the stability analysis can be obtained using OCR values (shown

Fig. 13. Undrained strength of Khor Al-Zubair clay for stability analysis determined by 1) the modified Bjerrum's method, 2) F.V. test and 3) the SHANSEP method plotted versus depth

in Fig. 10), $S_{un(c)}/\bar{p}$ value ($=0.355$ shown in Fig. 9), $\mu_A=0.85$, $\mu_R=0.9$ and the n values (shown in Fig. 11) as follows:-

$$\begin{aligned} S_{up} &= 0.355 \times \text{OCR} \times \bar{p}_0 \times 0.85 \times 0.9 \\ &= 0.27 \bar{p}_c (\text{modified Bjerrum's method}) \\ &= 0.27 n \bar{p}_c (\text{SHANSEP method}) \\ &= 0.355 / 1.6 \times \text{OCR} \times \bar{p}_0 \times 0.9 \\ &= 0.20 \bar{p}_c (\text{F.V. test}) \end{aligned} \quad (16)$$

The values of S_{up} from Eq. (16) are presented in Fig. 13. It can be seen that there is considerable differences in S_{up} values obtained from each of the methods.

3) Daikoku-cho clay

The values of $q_u/2$, $S_{uf(v)}$ and $S_{uf(c)}$ obtained from q_u test, F.V. test and the modified Bjerrum's method ($r=0.12\%/min$) are compared in Fig. 14. A unique relationship was obtained among the three strengths as follows:-

$$\left. \begin{aligned} S_{uf(v)} &= 0.9 S_{uf(c)} \\ q_u/2 &= 0.8 S_{uf(c)} \end{aligned} \right\} \quad (17)$$

Compression and extension strengths in the N.C. young state, $S_{un(c)}$ and $S_{un(e)}$ obtained from K_0 -consolidated triaxial tests are plotted versus vertical consolidation stress, $\bar{\sigma}_{vc}$ in

Fig. 14. Comparison of $q_u/2$ (from q_u test), $S_{uf(v)}$ (from F.V. test) and $S_{uf(c)}$ (from the modified Bjerrum's method) for Daikoku-cho clay

Fig. 15. Compression and extension strength in the normally consolidated state, $S_{un(c)}$ and $S_{un(e)}$ of Daikoku-cho clay obtained from K_0 consolidated triaxial test

Fig. 15. The values of $S_{un}/\bar{p}$ and the strength ratio, $S_{un(e)}/S_{un(c)}$ were obtained as follows:-

$$\left. \begin{aligned} S_{un(c)}/\bar{p} &= 0.36 \\ S_{un(e)}/\bar{p} &= 0.20 \end{aligned} \right\} \quad (18)$$

$$S_{un(e)}/S_{un(c)} = 0.20/0.36 = 0.56 \quad (19)$$

Since Daikoku-cho clay is a N.C. aged clay (Kishida, et al., 1982), the strength ratio shown in Eq. (19) is constant irrespective of OCR values.

The dike was constructed on flat level ground, the value of μ_A is given by $(1+0.56)/2=0.78$ when $S_{uf(c)}$ value is taken as S_{uf} in Eq. (1).

$$\mu_A = (1+0.56)/2 = 0.78 \quad (20)$$

Fig. 16 summarizes the change in undrained strength with the strain rate obtained from CK_0UC_r test the same as that carried out for Khor Al-Zubair clay. Since all the triaxial tests except this test were conducted

Fig. 16. Change in undrained strength with change in strain rate for Daikoku-cho clay obtained from CK_0UC_r test

Fig. 17. Correction coefficient for strength decrease due to rebound, n for Daikoku-cho clay obtained from K_0 -consolidated and K_0 -rebounded triaxial compression tests plotted versus OCR

at $r=0.12\%/min$, $\frac{[S_u \text{ at a given } r]}{[S_u \text{ at } r=0.12\%/min]}$ is plotted versus strain rate in the figure. The strength at $r=10^{-3}$ to $10^{-2}\%/min$ is about 85% of that at $r=0.12\%/min$. The value of μ_R is then given by:

$$\mu_R = 0.85 \quad (21)$$

Determination of the in-situ undrained strength by the SHANSEP method was also carried out though this method can not be used for aged clay. Fig.17 indicates the n values versus OCR values obtained from CK_0RK_0UC test at $r=0.12\%/min$.

The final problem in determining the S_{up} values is how to determine the values of $\bar{p}_c$ or OCR. Uncertainties of $\bar{p}_c$ values from the oedometer tests have been reported by many engineers (Leonards and Raminah, 1960; Crowford, 1964; Mesri, 1975) and it was already demonstrated for Fao clay that $\bar{p}_c$

Fig. 18. Comparison of OCR values determined by the standard and the special oedometer tests, and the K_0 -consolidated triaxial compression test for Daikoku-cho clay

values drastically changed with the load incremental ratio. In order to investigate this problem, two kinds of oedometer tests were conducted for Daikoku-cho clay as follows:-

1) Standard oedometer test in which consolidation load was applied by doubling the previous load and consolidation time was 24 hours.

2) Special oedometer test in which consolidation load was applied in small increments by 10 to 20 kN/m^2 and consolidation was terminated when primary compression was achieved.

The values of OCR obtained from the two kinds of oedometer tests and from the strength test in which OCR value is given by $S_{uf(c)}/\bar{p}_0 \div 0.36$ for this clay, are compared in Fig.18.

1) The values of OCR obtained from the standard oedometer test and from the strength test showed good agreement with each other.

2) The values of OCR from the special oedometer test showed some 20% to 30% greater values than from the standard oedometer and the strength tests.

As previously described, it was demonstrated for Fao clay that $\bar{p}_c$ values of this

clay should be determined by the test approximately coinciding to the special oedometer test conducted for Daikoku-cho clay. The oedometer test conducted for Khor Al-Zubair clay was performed following the two conditions, 1) loading time was 2 hours and 2) load incremental ratio was doubling the previous load. It is suggested that $\bar{p}_c$ values obtained under the above two conditions would be an intermediate value from the standard and the special oedometer tests. These facts demonstrates that it will be very difficult to determine the proper $\bar{p}_c$ value from the oedometer test alone. On the other hand, it is suggested that OCR values of N.C. aged clay from the strength tests are less affected by load incremental ratio and loading time because these factors in the triaxial tests to determine both strengths in the field and in the N.C. young state were equivalent each other (Hanzawa, 1982). Because of this reason, OCR values represented by the solid line in Fig.18 were adopted in strength determination (Kishida, et al., 1983).

The values of S_{up} for Daikoku-cho clay can be given using OCR values (shown in

Fig.18), $S_{un(c)}/\bar{p}$ value ($=0.36$), μ_A ($=0.78$), μ_R ($=0.85$), n values (shown in Fig.17) and the strength ratio (shown in Eq.(17)) as follows:-

$$\begin{aligned}
 S_{up} &= 0.36 \times \bar{p}_c \times 0.78 \times 0.85 \\
 &= 0.24 \bar{p}_c (\text{modified Bjerrum's method}) \\
 &= 0.24 n \bar{p}_c (\text{SHANSEP method}) \\
 &= S_{uf(v)} \times \mu_R = 0.9 \times 0.36 \times \bar{p}_c \times 0.85 \\
 &= 0.28 \bar{p}_c (\text{F.V. test}) \\
 &= q_u/2 = 0.8 \times 0.36 \times \bar{p}_c = 0.29 \bar{p}_c (q_u \text{ test})
 \end{aligned}
 \tag{22}$$

Fig.19 indicates the S_{up} values versus depth determined by each of the methods.

Results of Stability analyses for three structures

Stability analysis for sliding failure was carried out by the modified Fellenius method which is commonly used in Japan. In carrying out stability analysis, it is an important check point whether or not strength increase due to consolidation takes place during construction. This was evaluated by the comparison of $\bar{p}_c$ and $[\bar{p}_0 + \Delta p, U]$ values where Δp and U are the additional stress induced by fill and the degree of consolidation at the completion of construction. In calculating Δp and ΔU values, it is required to know loading conditions, unit weight of fill materials and clay properties such as drainage conditions. Table 2 summarizes loading conditions, construction rate, unit weight of fill materials, drainage conditions and comparison of $\bar{p}_c$ and $[\bar{p}_0 + \Delta p, U]$ for the three cases. Internal friction angle of fill materials used for stability analysis are also indicated in the table. As indicated in the table, strength change due to rebound or consolidation was considered in stability analysis for the Fao Steel Jetty and the Daikoku-cho Dike, respectively.

The values of $F.S._{min}$ obtained are summarized in Table 3 with the field behaviour of each structure. The followings can be pointed out:-

1) $F.S._{min}$ value for the Fao Steel Jetty obtained using $q_u/2$ values is indeed less than half $F.S._{min}$ value from the modified Bjerrum's method. The reason for this can

Fig. 19. Undrained strength of Daikoku-cho clay for stability analysis determined by 1) the modified Bjerrum's method, 2) the SHANSEP method, 3) q_u test and 4) F.V. test plotted versus depth

Table 2. Summary for loading conditions, unit weight of fill material, drainage conditions and comparison of $\bar{p}_c$ and $\bar{p}_0 + \Delta p \cdot U$

Site	Loading conditions	Construction rate	Unit Weight of fill	Drainage conditions	$\bar{p}_c$ and $\bar{p}_0 + \Delta p \cdot U$
Fao Steel Jetty	Back fill 2 m to 3 m thick Excavation to -10m	3 months for back filling	$\gamma_t = 20 \text{ kN/m}^3$ (measured) $\phi = 35^\circ$ (assumed)	$c_v = 300 \text{ cm}^2/\text{d}$ $H = 30 \text{ m}$	$\bar{p}_0 + \Delta p \cdot U \div \bar{p}_c$ Strength decrease due to excavation was considered using the n -OCR relationship
Khor Al-Zubair Embankments	No. 1	11.0 m high	$\gamma_t = 19-20 \text{ kN/m}^3$ (measured) $\phi = 35^\circ$ (assumed)	$c_v = 600 \text{ cm}^2/\text{d}$ $H = 7.5 \text{ m}$	$\bar{p}_0 + \Delta p \cdot U < \bar{p}_c$ Therefore strength change during construction was not considered
	No. 2	11.5 m			
	No. 3	11.5 m			
	No. 4	11.5 m			
	No. 5	11.5 m			
Daikoku-cho Dike	15.0 m high	60 days	$\gamma_t = 14-16 \text{ kN/m}^3$ (measured) $\phi = 35^\circ$ (assumed)	$c_v = 100 \text{ cm}^2/\text{d}$ $H = 5.0 \text{ m}$	$\bar{p}_0 + \Delta p \cdot U > \bar{p}_c$ for the upper 2 m and strength change was considered for this zone in stability analysis

Table 3. Factors of safety for each structure obtained from stability analyses

Structure	Field behaviour	Minimum factor of safety, F.S. _{min}				
		Mdf. Bjerrum's method	SHANSEP method	F.V. test	q_u test	Bjerrum's chart
Fao Steel Jetty	stable	1.43		1.24	0.68	1.26
Khor Al-Zubair Embankments	No. 1	stable	1.10—1.16	0.79—0.83	0.88—0.93	0.91—0.96
	No. 2	verge of failure	1.01—1.06	0.72—0.76	0.81—0.86	0.83—0.88
	No. 3	critical for failure	1.06—1.12	0.76—0.80	0.85—0.89	0.88—0.92
	No. 4	failed	0.97—1.02	0.69—0.72	0.77—0.82	0.79—0.83
	No. 5	stable	1.15—1.21	0.83—0.88	0.91—0.95	0.94—0.98
Daikoku-cho Dike	failed	0.91—1.03	0.85—0.97	1.00—1.09	1.02—1.12	0.81—0.88

be attributed to the special property of Fao clay that it drastically changes the structure when sampled.

2) The field behaviour of 5 embankments constructed on Khor Al-Zubair clay showed excellent agreement with the results of stability analysis calculated using the strength values from the modified Bjerrum's method. F.S._{min} values obtained using the strength values from F.V. test and the SHANSEP method were too low and far from the field behaviour of embankments. This clearly indicates that 1) $S_{uf(v)}$ values should not be directly used for stability analysis before the strength ratio to an inherent in-situ strength is obtained and 2) the SHANSEP method can not be used for aged clay.

3) F.S._{min} values obtained using the strength values from the modified Bjerrum's method showed good agreement with the field behaviour of Daikoku-cho Dike. It

should be noted, however, that F.S._{min} values obtained using $q_u/2$ values were not so different from the F.S._{min} values by the modified Bjerrum's method. It is possible, therefore, to conduct rational design and quality control works using $q_u/2$ values provided a proper F.S._{min} value such as F.S._{min} = 1.3 is maintained.

4) F.S._{min} values obtained using the Bjerrum's stability chart were too low and far from the field behaviour for Khor Al-Zubair Embankments and Daikoku-cho Dike.

Fig. 20 presents 1) the circular arc giving F.S._{min} value obtained from the stability analysis which used the strength values from the modified Bjerrum method and 2) the actual slip plane observed in the field for No. 4 embankment of Khor Al-Zubair Embankment and Daikoku-cho Dike. The critical circular arc obtained from the calculation showed relatively good agreement with the

Fig. 20. Comparison of the critical circular arc obtained from the stability analysis and the actual failure plane for a) No. 4 Embankment of Khor Al-Zubair Embankments and b) Daikoku-cho Dike

actual slip plane.

CONCLUDING REMARKS

Three case studies for short term stability problems for the structures constructed on normally consolidated clays at and near failure were described in detail. Also presented discussions of the shear strength determined by various methods currently practiced. It was revealed that 1) the shear strength determined by the methods currently practiced is uncertain and 2) stability of clay foundations evaluated using such uncertain strength is less reliable. Actually, results of stability analyses for the three structures reported in this paper strongly supported this conclusion.

The author has been concerned with the stability problems for harbor facilities constructed on soft clay deposits under very low factors of safety. Through his experience, the author keenly felt the importance to understand 1) the difference in geological history (including environmental conditions) of a given clay in-situ since deposition and 2) the effect of this difference on the shear strength characteristics. Since the geological history of soils in-situ is seldom same, their properties are fundamentally different from

each other. For example, Fao clay and Khor Al-Zubair clay are found at the same area only 70 km apart from each other. However, their geotechnical properties are considerably different from each other as presented in this paper. This difference was caused by the difference in aging effects that the two clays have been subjected to (Hanzawa, 1982). It was also revealed that proper determination of the preconsolidation pressure of the three clays was difficult by oedometer test alone irrespective of the quality of undisturbed sample. It should be realized in the author's opinion that the appropriate evaluation of the regional variations in soil behaviour is the most important factor to obtain the proper solution for geotechnical problems.

NOTATION

- F. S. $_{min}$ = minimum factor of safety
 I_p = plasticity index
 K_0 = coefficient of earth pressure at rest
 n = correction coefficient for strength due to rebound
 $\bar{p}_c$ = preconsolidation pressure
 $\bar{p}_0$ = effective overburden pressure
 q_u = unconfined compression strength
 S_u = undrained strength
 S_{uf} = in-situ undrained strength
 $S_{uf(c)}$ = in-situ compression strength
 $S_{uf(e)}$ = in-situ extension strength
 $S_{uf(v)}$ = in-situ vane strength
 S_{un} = undrained strength in the normally consolidated state
 S_{up} = undrained strength for stability analysis
 $S_{un}/\bar{p}$ = undrained strength ratio in the normally consolidated state
 α = correction coefficient for strength due to rebound or consolidation
 β = correction coefficient for progressive failure
 μ_A = correction coefficient for strength anisotropy
 μ_R = correction coefficient for strain rate effect

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