A Method for Determining the In Situ K_o Coefficient

Méthode pour Déterminer le Coéfficient K_o

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SYNOPSIS Most of the available testing methods for determining the coefficient of earth pressure at rest give only "apparent" value. A new testing method on the basis of a different concept has been developed for the determination of the "in-situ" coefficient of earth pressure at rest for clays in laboratory. Results of experimental investigation indicate that the proposed method is reliable and can be applied to both normally consolidated clays and lightly overconsolidated soils.

INTRODUCTION

The coefficient of earth pressure at rest, K_o, of a soil is defined as the ratio of the horizontal and vertical effective stresses acting on the soil under no lateral strain condition. It is well recognized that the stress-strain behavior or strength and deformation characteristics of a particular soil element underground can only be reasonably determined by testing a soil specimen which has its initial stresses and strains, if possible, been completely returned to its in situ condition. The most which can be done in the laboratory is to reconsolidate an "undisturbed" specimen under its in situ stresses. The in situ vertical stress of a soil mass can be accurately determined from the overburden pressure and pore water pressure. However, direct measurement or determination of the horizontal stress presents serious difficulty, which at the present can only be estimated from the results of K_o determination. POULOS and DAVIS (1972), SIMONS and SOM (1969) have found that the use of an incorrect value of K_o for the estimation of horizontal consolidation pressure has serious effects on the deformation parameters of a soil, such as Young's modulus, Poisson's ratio and coefficient of volume compressibility.

Since direct measurement of in situ stresses in the field presents serious difficulties owing to disturbance of stress pattern resulting from the necessity of installation of measuring devices, the principal sources of information about the K_o value are laboratory tests in which field conditions are closely simulated. The first reliable laboratory test for K_o determination was developed by BISHOP and HENKEL (1957) and BISHOP (1958) by using triaxial cells. The test is based on the principle that no lateral strain of the soil specimen is allowable during the entire testing process. This testing concept has then been adopted by many other researchers such as DAVIS and POULOS (1963), BROOKER and IRELAND (1965), CAMPANELIA and VAID (1972). However, due to intrinsic deficiency of the testing concept, this type of no lateral strain tests would yield only an apparent value of the coefficient of earth pressure at rest, K_{oa} (KENNY, 1967; POULOS and DAVIS, 1972; BJERRUM and ANDERSEN, 1972). The value may differ significantly from the K_o value existed in situ, designated as K_{of}.

A method for the determination of K_{of} in the laboratory has recently been developed by POULOS and DAVIS (1972). The method is applicable only to normally consolidated clays. The results indecated that the K_{of} values determined by this method were generally higher than the K_{oa} values. BJERRUM and ANDERSEN (1972) using the principle of hydraulic fracturing have developed a successful field method for measuring the in situ lateral pressures in normally consolidated clays. They also found that the K_{of} values were higher than the K_{oa} determined in the laboratory.

This paper presents a different method for laboratory determination of the K_{of} which appears to be rather simple in operation and also applicable to lightly overconsolidated clays. The method is termed as the Allowable Deviator Stress Testing Method (ADSTM).

BASIC CONSIDERATIONS

"Undisturbed" samples are generally used in the laboratory strength and compressibility tests. An isotropic residual effective stress $\bar{\sigma}_r$, which is equal to the negative residual pore pressure, usually exists in a soil specimen. In the traditional K_o-triaxial test suggested by BISHOP (1958), the state of stresses and strains at each stage of the test is shown in Fig. 1(a). By considering the testing procedure and condition of the soil specimen, it can be concluded that the traditional K_o testing method has at least two shortcomings which are quite significant. Firstly, the disturbed zones commonly exist on the periphery of a prepared test specimen are not consolidated before the K_o consolidation process. This always causes the soil skeleton to strain radially and weakens the soil structure. The soil skeleton tends to become more compressible during the subsequent K_o consolidation. As a result, the amount of horizontal stress required to maintain zero lateral strain during the test would be less than the actual in situ horizontal stress. The stress condition after consolidation thus may depart considerably from the in situ condition. Secondly, although it could be reasonably assumed that the cross-sectional area of good quality "undisturbed" specimen may approximately represent its original size under the in situ stresses, the process of specimen saturation or application of back pressure frequently causes the initial effective stresses in the specimen to change under the condition of zero lateral strain. This change is probably due to the non-linearity and inelastic behavior of the soil which cause the specimen to contract if the same amount of stress is reimposed on the specimen after application of the back pressure, i.e. a partial unloading process.

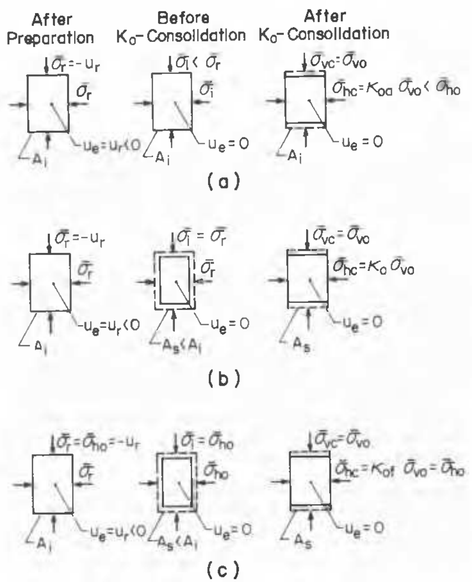

Fig. 1 Stress and Strain Conditions during K_o Tests.

The result of these shortcomings is to cause the specimen to behave differently from that in situ. The K_o value as determined by this method is therefore not truly representative of the in situ value and should be considered as an apparent value, K_{oa}.

The traditional testing method can be improved by removing these major shortcomings. This might be accomplished by consolidating the specimen with consolidation pressure equal to $\bar{\sigma}_r$ before K_o consolidation as shown in Fig. 1(b). Not only the disturbed zones are consolidated but also the effective stresses can be maintained before the K_o consolidation. More reasonable result of the K_o value can therefore be expected. Unfortunately, methods of measurements of u_r or $\bar{\sigma}_r$ in an "undisturbed" specimen are still unreliable up to date. This approach thus does not appear to be very practical.

It is quite possible that the residual effective stress $\bar{\sigma}_r$ of a good quality "undisturbed" specimen may approximately be equal to its corresponding in situ horizontal effective stress $\bar{\sigma}_{ho}$. If this were the case, the stress and strain conditions at each stage of a K_o test can be expressed as shown in Fig. 1(c). The specimen is first consolidated isotropically under $\bar{\sigma}_{ho}$ before K_o consolidation. The cross-sectional area of the specimen after initial consolidation, or the area to be maintained during K_o consolidation, would be more representative of that when the specimen is in equilibrium with the corresponding in situ stresses. The change of horizontal stress during K_o consolidation should therefore be zero in order to maintain the no lateral strain condition of the specimen when consolidated under in situ stresses $\bar{\sigma}_{vo}$ and $\bar{\sigma}_{ho}$. Hence, it can be expected that a reasonable K_{of} value can be determined by first consolidating the test specimen isotropically under $\bar{\sigma}_{ho}$ and then testing the specimen to find out how much deviator stress the soil skeleton can take without causing any lateral strain. The rationality of the proposed procedure in determining the K_{of} value of a soil can also be explained from the point of view of theory of elasticity (CHANG, 1973).

Unfortunately, the $\bar{\sigma}_{ho}$ is an unknown value which in fact is the one to be determined. Therefore only an assumed value of horizontal stress $\bar{\sigma}_{hc}$ can be used in the isotropic consolidation stage prior to the application of deviator stress. The K value determined in case $\bar{\sigma}_{hc} \neq \bar{\sigma}_{ho}$ may differ from that when $\bar{\sigma}_{hc} = \bar{\sigma}_{ho}$. Hence the proposed method would have only very limited value unless the K value determined by the proposed method is proved to be not affected by or insensitive to the value of $\bar{\sigma}_{hc}$ used in the test. A series of tests have been carried during the research on two artificially sedimented normally consolidated clays for this purpose. The results in Fig. 2 show that the

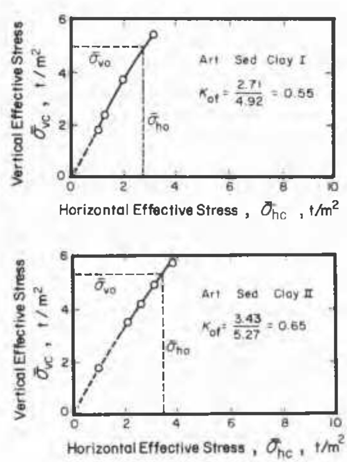

Fig. 2 Variation of Vertical Effective Stress with Horizontal Effective Stress For Two Normally Consolidated Artificially Sedimented Clays.

K values determined by the proposed method are resonably constant when the isotropic consolidation stresses applied are in the reconsolidation range of the soil, which covers the in situ stress. It can therefore be concluded that the proposed method of K_{of} determination is feasible as long as the initial isotropic consolidation stress chosen for the test near and not larger than the corresponding in situ horizontal effective stress. This procedure also appears to be feasible for slightly overconsolidated clays as indicated by the data shown in Fig. 3.

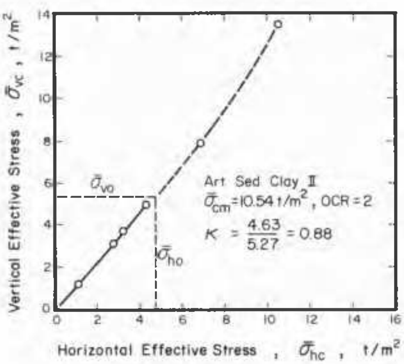

Fig. 3 Variation of Vertical Effective Stress with Horizontal Effective Stress for A Lightly Overconsolidated Clay.

TESTING PROCEDURE

An "undisturbed" soil specimen is set up in a conventional type triaxial cell and saturated with a seleced back pressure. The set up and saturation process may follow those standard procedures suggested by BISHOP and HENKEL (1957). The saturated specimen is first consolidated under an isotropic stress. A value equal to 0.4-0.5 $\bar{\sigma}_{vo}$ for lean clay and 0.5-0.6 $\bar{\sigma}_{vo}$ for fat clay, which are generally slightly lower than the possible $\bar{\sigma}_{ho}$ value, is suggested to be used. The lateral stress is then kept constant while the vertical stress is adjusted step by step so that the stress ratio, K, decreases from one to smaller values. During the test, the vertical deformation of the test specimen is recorded from a dial gauge at convenient time intervals. The value is multiplied by the cross-sectional area of the specimen after initial consolidation to obtain the volume change of the specimen. This calculated volume change, ΔV_c represents the vertical strain of the specimen. The amount of water drained out of the specimen, measured from a volume change measuring device, is also recorded at the same time. This measured volume change, ΔV_m, represents the volumetric strain of the specimen. The deviator load increments are applied gradually. The time interval used for each step of deviator stress application is dependent upon the strain condition of the specimen as observed from the volume change, the vertical compression and the rate of consolidation. In the initial stage, the ΔV_m is relatively larger, due to specimen contraction, than the ΔV_c, sooner adjustment should be made. Upon further increase of deviator stress, the value of ΔV_c becomes closer to ΔV_m and the time interval used for load application should be larger. When the ΔV_c is found to be approaching the ΔV_m, the amount of load-increment should be reduced. Also, the time interval load application is extended to a longer period in order to determine whether further increase of deviator stress is required. No further increment of deviator stress is applied when agreement between ΔV_c and the volumetric strain ΔV_m is achieved, that is when the lateral strain of the test specimen is zero. The ratio of horizontal and vertical effective stresses acting on the soil specimen after completion of consolidation under this zero lateral strain condition is taken as the K_{of}.

TEST RESULTS AND DISCUSSION

Several series of laboratory tests were carried out on a number of undisturbed samples of normally consolidated clays. They included two natural deposits and two artificially sedimented clays with known stress histories. Table 1 shows the general properties of these soils. The clays were tested in the laboratory to determine their K_{of} values by both ADSTM and the method proposed by POULOS and DAVIS (1972). The procedure of Poulos and Davis' method is analogous to that for estimating the preconsolidation pressure of a soil in one dimensional test but the test is carried out in a triaxial cell. In addition, the Bishop's method (BISHOP, 1958) was also used to determine the K_{oa} for comparison. A summary of the test results is presented in Table 2.

Although the testing concepts of the two methods in determining the K_{of} value are quite different, the results presented in Table 2 indicate that the two methods gave almost identical results. Since POULOS and DAVIS (1972) has proved that their proposed method is reliable and can be applied to all normally consolidated clays, agreement of the test results reflects that the ADSTM is also reliable and can be used for determining the K_{of} value of normally consolidated clays.

The K_{oa} values determined by Bishop's method are always lower than the K_{of} values. The reasons contributed to the difference have been discussed earlier. Much evidences, such as comments made by KENNEY (1967) and results obtained by both POULOS and DAVIS (1972) and BJERRUM and ANDERSEN (1972), have indicated that the K_o value as determined by the traditional testing procedure is too low.

Of course, the best way of determining the K_{of} value is to measure the in situ lateral stress directly in the field such as the method proposed by BJERRUM and ANDERSEN (1972). But in most of the cases, it would be much more convenient and economical to determine this parameter in the laboratory. Both Poulos and Davis' method and the ADSTM can be used for this purpose. By comparing these two methods, the ADSTM offers the advantages of being simpler in operation, shorter time required, and more flexible.

No method, up to date, has been developed to determine the K_{of} values for overconsolidated clays, i.e. clays with their in situ stresses different from the maximum stresses they have encountered. According to the testing concept on which the ADSTM is based, the suggested procedure should be applicable to all soils with K_{of} value smaller than one. A series of test was carried out on an artificially sedimented clay consolidated under an overconsolidation ratio of 2. The result shown in Fig. 3 appears to indicate that the ADSTM can also be extended to determine the K_{of} value of lightly overconsolidated clays.

Table 1 General Properties of Soil Sample

Soil Property	Natural Bangkok Clay	Artificially Sedimented Clay I*	Natural Nong Ngoo Hao Clay	Artificially Sedimented Clay II**
Specific Gravity	2.70	2.77	2.72	2.78
Ave Natural Water Content, %	87.3	60.1	51.2	94.0
Liquid Limit, %	97	72	51	108
Plastic Limit, %	33	35	21	48
Plasticity Index, %	64	37	30	60
Soluble Salt Content, NaCl gm/l	5.8	35.0	13.3	35.0

*Maximum preconsolidation pressure = 4.92 t/m^2
**Maximum preconsolidation pressure = 5.27 t/m^2

Table 2 Summary of Test Results on K_o Determination for Normally Consolidated Clays

Test Series	Soil Type	Effective Overburden Pressure, t/m^2	Testing Method	K_{of}	K_{oa}
I	Natural Bangkok Clay	5.98	ADSTM	0.75	-
			P & D	0.73	-
			Bishop	-	0.63
II	Artificially Sedimented Clay I	4.92	ADSTM	0.55	-
			P & D	0.55	-
			Bishop	-	0.45
III	Natural Nong Ngoo Hao Clay	6.85	ADSTM	0.64	-
			P & D	0.64	-
			Bishop	-	0.45
IV	Artificially Sedimented Clay II	5.27	ADSTM	0.65	-
			P & D	0.64	-
			Bishop	-	0.46
V	Artificially Sedimented Clay II	21.08	ADSTM	0.76	-
			P & D	0.75	-

However, when the stresses acting on a soil are rather small, e.g. when soils taken from rather shallow depths are tested, the ADSTM becomes insufficient. This is because that both the volumetric and vertical strains encountered during the deviator stress testing process are so small that the error may be rather large. Use of the ADSTM to determine the K_{of} values for heavily overconsolidated clays with their K_{of} value larger than one still needs further investigation.

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