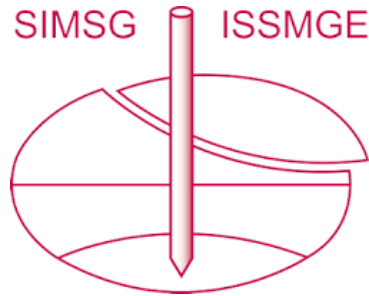


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Stress-Strain Behaviour of Anisotropically Consolidated Clays during Undrained Shear

Le Comportement contrainte-déformation durant le cisaillement non-drainé d'argiles consolidées de façon anisotrope

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SUMMARY

The behaviour of anisotropically and isotropically consolidated samples of six normally consolidated clays is compared in consolidated-undrained triaxial compression tests. Anisotropic consolidation had little effect on the ratio of undrained shear strength s_u to major principal stress at consolidation, while both $\bar{\phi}$ and A_f at failure were generally decreased. It is shown that the strength parameters of anisotropically consolidated samples cannot be predicted from test results on isotropically consolidated samples.

The *in-situ* undrained strength of clay is not a unique function of water content or consolidation pressure, and hence is not independent of total stress path, as commonly assumed. Triaxial data on two normally consolidated clays starting with K_0 stresses show a twofold variation in s_u depending on whether the axial pressure is increased or decreased. Possible implications of such effects on current practice in selecting values of s_u for $\phi = 0$ stability analyses are discussed.

SOMMAIRE

On compare le comportement d'échantillons d'argile anisotrope et isotrope, normalement consolidés, au moyen de l'essai de compression triaxiale sans drainage. L'effet de la consolidation anisotrope sur le rapport entre la résistance au cisaillement à teneur en eau constante, s_u , et la contrainte principale à la fin de la consolidation, n'a pas été considérable, tandis que $\bar{\phi}$ et A_f à la rupture ont généralement diminué. On trouve que l'on ne peut pas prédire les paramètres de résistance au cisaillement des échantillons consolidés anisotropiquement à partir des essais effectués sur des échantillons consolidés isotropiquement.

La résistance *in situ* sans drainage de l'argile n'est pas une relation unique de la teneur en eau ni de la pression de consolidation. C'est pour cette raison que la résistance n'est pas, contrairement aux croyances populaires, indépendante de la trace des contraintes totales. Les essais triaxiaux commençant avec une pression K_0 de deux échantillons d'argile normalement consolidés, prouvent que la valeur de s_u peut doubler, qu'il s'agisse d'une augmentation ou d'une diminution de la pression axiale. On discute les conséquences de ces effets sur la pratique courante dans le choix des valeurs de s_u pour l'analyse des stabilités employant $\phi = 0$.

CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS with pore-pressure measurements are run on samples of saturated clays to obtain values of undrained shear strength; to measure effective stress envelopes at failure, and to obtain values of stress-strain modulus and the pore-pressure parameter A . Isotropic consolidation stresses are frequently used in these undrained tests (called $\bar{C}\bar{U}$ tests). However, the stress system acting on *in-situ* clays is generally anisotropic, i.e., the horizontal and vertical stresses are not equal. This paper compares the stress-strain behaviour of anisotropically consolidated and isotropically consolidated samples of six saturated normally consolidated clays as measured in undrained triaxial tests. The effects of rotation of principal planes during undrained shear are discussed.

The ratio of horizontal to vertical effective stress in one-dimensionally consolidated horizontal soil deposits is called "coefficient of earth pressure at rest" and is denoted by $K_0 = \bar{\sigma}_{hc}/\bar{\sigma}_{vc}$. The variation in K_0 with overconsolidation ratio (OCR) is plotted in Fig. 1 for several clays. K_0 for normally consolidated clays (OCR = 1) generally equals 0.6 ± 0.2 and has been found to be related to the slope of the effective stress envelope ($\bar{\phi}$ = friction angle) at maximum obliquity by $K_0 = 1 - \sin \bar{\phi}$ (Bishop, 1958; Simons, 1958). As a clay deposit is rebounded, K_0 increases and

becomes greater than unity for OCR values exceeding about 3.5 ± 1 ; in other words the horizontal pressure becomes larger than the vertical pressure. Thus the deviation of *in-situ* stresses from an isotropic condition is most pronounced in normally consolidated and in highly overconsolidated clays, whereas in slightly overconsolidated clays the stresses are approximately isotropic.

TRIAXIAL COMPRESSION TESTS ON NORMALLY CONSOLIDATED CLAYS

The ratio of undrained shear strength s_u to major principal consolidation stress $\bar{\sigma}_{1c}$ from consolidated-undrained triaxial compression tests on normally consolidated clays with zero cohesion intercept is given by (Skempton and Bishop, 1954):

$$\frac{s_u}{\bar{\sigma}_{1c}} = \frac{[K + A_f(1 - K)] \sin \bar{\phi}}{1 + (2A_f - 1) \sin \bar{\phi}}$$

where failure is defined at maximum stress difference and $s_u = (\sigma_1 - \sigma_3)_{\max}/2$; $K = \bar{\sigma}_{3c}/\bar{\sigma}_{1c}$; $\bar{\phi}$ = friction angle at failure; $A_f = (\Delta u - \Delta \sigma_3)/(\Delta \sigma_1 - \Delta \sigma_3)$ at failure. If A_f and $\bar{\phi}$ are independent of K , it follows from Eq (1) that anisotropic consolidation decreases $s_u/\bar{\sigma}_{1c}$ if A_f is less than one, increases the ratio if A_f is greater than one, and has no effect

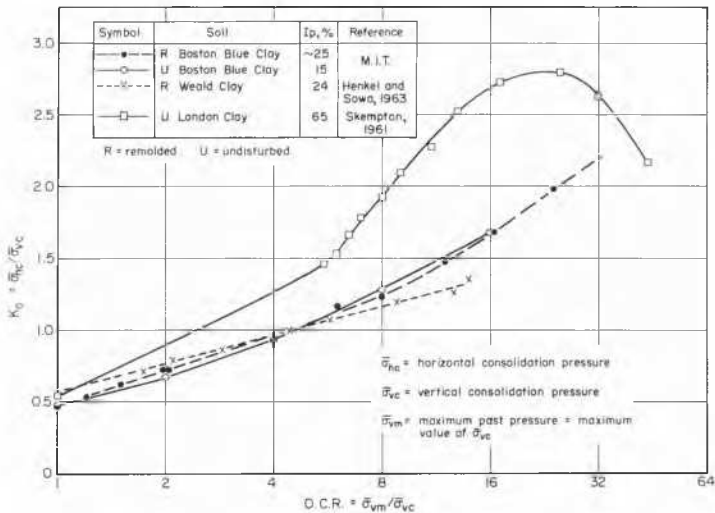


FIG. 1. Coefficient of earth pressure at rest versus overconsolidation ratio (log scale)

if A_f equals one. However A_f and $\bar{\phi}$ are not generally independent of the value of K as shown by the data in Table I from consolidated-undrained triaxial compression tests with pore-pressure measurements on six remoulded and undisturbed clays of varying plasticity. The $\bar{C}\bar{U}$ tests ($K = 1$) on the six clays gave $s_u/\bar{\sigma}_{1c} = 0.28 - 0.45$, $A_f = 0.80 - 1.10$, and $\bar{\phi} = 24 - 37^\circ$, all at $(\sigma_1 - \sigma_3)_{max}$. The $\bar{C}\bar{A}\bar{U}$ tests, for which K was approximately equal to K_0 , showed the following effects of anisotropic consolidation (see cols. 6-11 in Table I): (1) the change in $s_u/\bar{\sigma}_{1c}$ was generally small with a maximum increase of 10 per cent and a maxi-

imum decrease of 15 per cent; (2) the value of A_f decreased by a significant amount except for the remoulded Weald and Vicksburg Buckshot clays; (3) the friction angle $\bar{\phi}$ decreased by 0 to 4° ; and (4) the strain at failure was considerably smaller, being generally less than 1 per cent versus 2.5 to 15 per cent for the $\bar{C}\bar{U}$ tests. For the condition of maximum obliquity (cols. 12 and 13), the $\bar{C}\bar{A}\bar{U}$ tests showed lower values of shear stress but essentially the same effective stress envelope.

Effective stress paths from $\bar{C}\bar{U}$ and $\bar{C}\bar{A}\bar{U}$ tests on tube samples of Kawasaki clay are compared in Fig. 2 in which

TABLE I. CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS ON NORMALLY CONSOLIDATED CLAYS (ALL STRESSES IN KG/SQ.CM.)

1 Soil	2 Classification data			5 Activity	6 Consolidation		8 At $(\sigma_1 - \sigma_3)_{max}$			12 At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$		
	$w_L, \%$	$I_p, \%$	$I_L, \%$		$\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{1c}$	A_f	$\bar{\phi}^\circ$	$\epsilon_f, \%$	$q/\bar{\sigma}_{1c}$	$\bar{\phi}^\circ$
Remoulded Boston Blue clay (fresh water) ($s_1 = 5-10$)	33	15	375	0.5	4-6 3-6	1.00 0.54	0.30 0.33	1.10 0.60	27.5 26.5	2.5 0.4	0.29 0.27	32.5 33
Remoulded Weald clay*	46	24	48	0.63	2-7.5 2-7	1.00 0.61	0.32 0.27	0.92 26	26 7	15 —	— —	— —
Remoulded Vicksburg Buckshot clay	63	39	230	0.75	3-6 3-6	1.00 0.54	0.28 0.28	1.05 1.05	24 23.5	6 0.7	0.27 0.25	25 25
Undisturbed Kawasaki clays I and II ($s_1 = 10$)	80 (50-100)	38 (20-50)	80 (60-100)	~1 (0.7-1.7)	1.5-6 2-5.5	1.00 0.52	0.45 0.42	0.80 0.50	37 33	4.5 0.9	0.44 0.40	38.5 39
Undisturbed Brobekkveien, Oslo clay† ($s_1 = 5$)	39	18	72	0.5	1.5-4 2-12	1.00 0.47	0.35 0.32	0.95 0.75	30.5 27	3.5 0.6	0.35 0.28	31 31.5
Undisturbed Skabo clay‡ ($s_1 = 2-6$)	52	29	65	0.6	2-6 2-5	1.00 0.47	0.32 0.32	1.05 0.75	30 26.5	5.3 0.6	— —	— —

*From Skempton and Sowa (1963); †From Simons (1960); ‡From Landva (1962).

Notes: Col. 1: $s_1 =$ sensitivity; Col. 4: I_L values at remoulding and *in situ* for remoulded and undisturbed samples respectively; Col. 5: activity = $I_p/(\% < 2 \text{ microns})$; Col. 6: range of values of $\bar{\sigma}_{1c}$ prior to shear; Cols. 7-13: average values from tests within the pressure range in Col. 6; Cols. 10, 13: values of $\bar{\phi}$ assuming zero cohesion intercept; Col. 11: $\epsilon_f =$ axial strain at failure; Col. 12: $q = (\sigma_1 - \sigma_3)/2$.

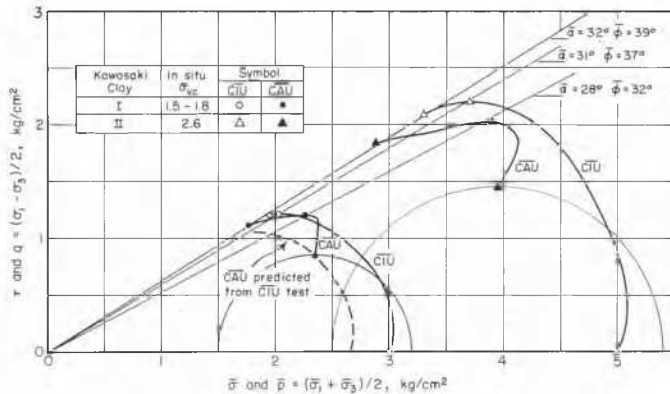


FIG. 2. Effective stress paths from consolidated-undrained triaxial compression tests on normally consolidated Kawasaki clay.

half the stress difference $q = (\sigma_1 - \sigma_3)/2$ is plotted against the average effective stress $\bar{p} = (\bar{\sigma}_1 + \bar{\sigma}_3)/2$. These data again illustrate that anisotropic consolidation, relative to isotropic consolidation, lowers the friction angle at $(\sigma_1 - \sigma_3)_{max}$, although the effective stress envelope at maximum obliquity is practically unchanged.

The strength behaviour of anisotropically consolidated samples is sometimes derived from **CTU** test data (Taylor, 1948, p. 387; Henkel, 1960) by assuming that effective stress paths from **CTU** tests represent a unique relationship between shear stress and effective stress. If such were true, a **CAU** test consolidated under stresses falling on the $q - \bar{p}$ path of a **CTU** test would have an effective stress path during subsequent undrained shear that coincided with the path of the **CTU** test. The test data in Fig. 2 show that the strength behaviour of anisotropically consolidated samples can not be predicted from **CTU** test data. (Compare the measured **CAU** test having $\bar{\sigma}_{1c} = 3.20$ with the extension of the **CTU** test shown with a dashed line.) Similar data on the other clays* in Table 1 also show that effective stress paths from **CTU** tests are not unique. Values of $s_u/\bar{\sigma}_{1c}$ for **CAU** tests with $K = K_0$ predicted from **CTU** stress paths will generally be less than the measured values (the Weald clay in Table 1 is an exception).

Fig. 3 compares stress-strain curves in a dimensionless form from **CTU** and **CAU** tests on normally consolidated samples of the Kawasaki and Boston Blue clays. The anisotropically consolidated samples show a much lower strain at failure, a larger drop off in stress difference at strains beyond failure, and a lower A parameter at small strains but a higher one at the larger strains. The excess pore pressure Δu is reduced since a large portion of the stress difference [equal to $(\bar{\sigma}_{1c} - \bar{\sigma}_{1e}) = \bar{\sigma}_{1c} (1 - K)$] was applied prior to undrained shear. Bjerrum and Lo (1963) suggest use of the relationship $\Delta u/\bar{\sigma}_{1c} + (1 - K)$ to correlate pore pressures from **CTU** and **CAU** tests at the condition of maximum obliquity. The test data in Fig. 3 at maximum obliquity and similar data by Bjerrum and Lo

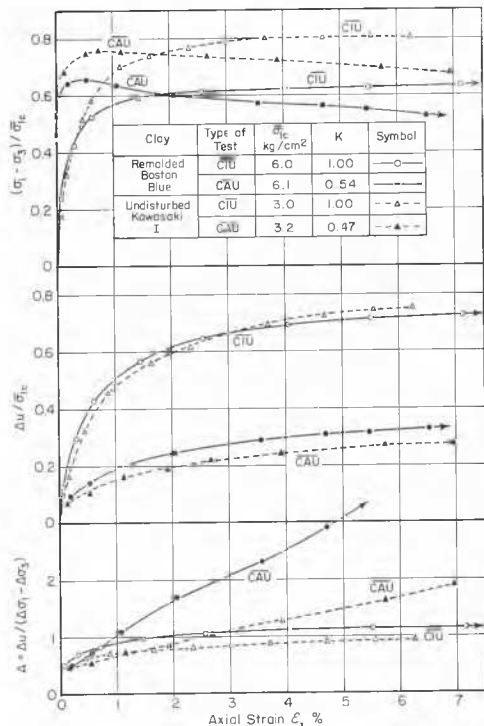


FIG. 3. Effect of anisotropic consolidation on the stress-strain behaviour of two normally consolidated clays.

(1963) show that **CAU** tests yield values of adjusted pore pressure equal to or slightly higher than those from **CTU** tests.

*Whitman, et al. (1960) presented stress paths from **CTU** and **CAU** tests on Vicksburg Buckshot clay.

STRENGTH BEHAVIOUR AS INFLUENCED BY TOTAL STRESS PATH DURING UNDRAINED SHEAR

In selecting a value of undrained shear strength s_u for use in a " $\phi = 0$ " stability analysis, it is commonly assumed that the value of s_u is not affected by different total stress paths during undrained shear. Data are presented in Fig. 4 to show that the undrained strength of clays is not necessarily "unique" but can indeed vary several fold depending on the stress system applied during shear.

A large chunk of Boston Blue clay, prepared by consolidating a slurry to 1.5 kg/sq.cm., was cut up for subsequent strain-controlled triaxial tests. Three specimens (the CAU, CA-UU, and CAU-RE tests) were consolidated in small increments under approximate K_0 stresses to $\bar{\sigma}_{1c} = \bar{\sigma}_{ac} = 4$ kg/sq.cm. and $\bar{\sigma}_{3c} = \bar{\sigma}_{rc} = 2.16$ kg/sq.cm. The effective stress paths during undrained shear in terms of the axial stress $\bar{\sigma}_a$ and radial stress $\bar{\sigma}_r$ are plotted in Fig. 4. The

test, which is representative of the stress conditions under the centre line of a circular excavation (i.e., unloaded area), was sheared undrained by decreasing the axial stress.* The direction of the major and minor principal planes rotated 90° when σ_a became less than σ_r , and the sample failed in extension since the intermediate principal stress σ_2 equaled the major principal stress σ_1 (both equalling σ_r).

At failure, i.e. $(\sigma_1 - \sigma_3)_{\max}$, the specimens showed:

CAU test	$s_u/\bar{\sigma}_{1c} = 0.33$;	$s_u = 1.35$ kg/sq.cm.
CA-UU test	$s_u/\bar{\sigma}_{1c} = 0.275$;	$s_u = 1.12$ kg/sq.cm.
CAU-RE test	$s_u/\bar{\sigma}_{1c} = 0.165$;	$s_u = 0.66$ kg/sq.cm.

Thus the value of s_u for three specimens consolidated under essentially identical stresses, and hence with identical water contents at failure, varied by a factor of two because of the different stress systems applied during shear. A similar test series on normally consolidated samples of Kawasaki clay II yielded $s_u/\bar{\sigma}_{1c} = 0.44, 0.37, \text{ and } 0.22$ for the CAU, CA-UU, and CAU-RE tests respectively. The range in strengths for identical samples was also two-fold. Data on variations in stress-strain curves with different stress systems were presented elsewhere (Ladd, 1964).

CONCLUSIONS

One-dimensional, as contrasted to isotropic three-dimensional, consolidation of normally consolidated clays of low to moderate sensitivity causes the following effects with undrained triaxial compression tests:

1. $s_u/\bar{\sigma}_{1c}$ is practically unchanged (± 15 per cent).
2. Both ϕ and A_f at $(\sigma_1 - \sigma_3)_{\max}$ are generally decreased; the axial strain is much smaller.
3. At maximum obliquity, ϕ is essentially unchanged, the A parameter is often greatly increased, and $(\sigma_1 - \sigma_3)/\bar{\sigma}_{1c}$ is usually decreased.

These effects cannot be predicted from the results of CU tests.

Although $s_u/\bar{\sigma}_{1c}$ is approximately independent of K for failure in triaxial compression, its value can be highly dependent on the type of total stress system applied during shear. For example, triaxial specimens of a clay normally consolidated with K_0 stresses and failed by increasing the radial pressure may have only half the undrained strength of identical specimens failed by increasing the axial stress. Simple shear tests have also produced much lower values of $s_u/\bar{\sigma}_{1c}$ than triaxial compression tests (Bjerrum, 1961). The most important variables are probably the values of K at consolidation, σ_3 at failure, and the direction of σ_1 at failure relative to its direction after consolidation (i.e., rotation of principal stresses). The magnitude of the change in $s_u/\bar{\sigma}_{1c}$ with different stress systems would generally increase with the sensitivity of the clay.

The common assumption that the *in-situ* undrained strength is a constant is open to question since most field cases involve stress systems which vary from point to point (an example being different amounts of rotation of principal stresses along a curved failure surface). With footings on normally consolidated clays, the average value of s_u along a potential failure surface may be far less than that corresponding to triaxial compression. If such is true, the reported successful use of unconfined compression tests

*The difference in stress paths from the CA-UU and CAU-RE tests between $K = K_0$ and $K = 1$ is due to experimental scatter.

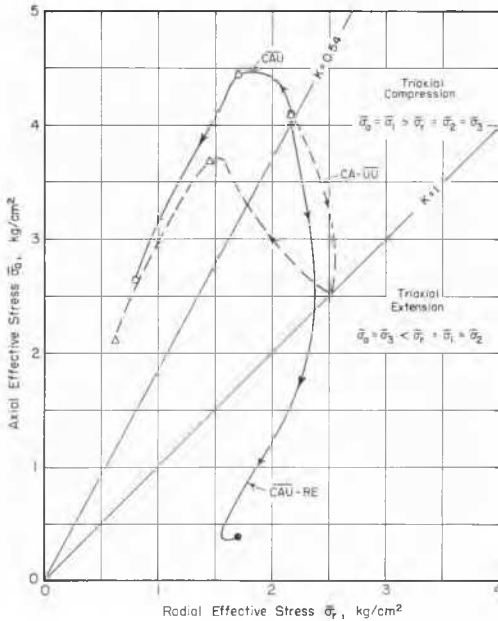


FIG. 4. Effective stress paths from consolidated-undrained triaxial tests with different stress systems on normally consolidated Boston Blue clay.

CAU test was sheared by increasing the axial total stress σ_a with $\Delta\sigma_r = 0$. This total stress path is representative of the stress conditions under the centre line of a circular loaded area (note that the value of $\Delta\sigma_r$ is inconsequential as long as $\Delta\sigma_a > \Delta\sigma_r$). In the CA-UU test the sample was sheared undrained by first decreasing σ_a until $\sigma_a = \sigma_r$ and then increasing σ_a until failure occurred. This test represents an unconsolidated-undrained compression test on a "perfect" sample, wherein "perfect" sampling means that no disturbance is given to the sample other than that involved with the release of the consolidation shear stresses (see Ladd and Lambe, 1963; Skempton and Sowa, 1963). The CAU-RE

for $\phi = 0$ analyses (Bishop and Bjerrum, 1960) must be partly due to compensating errors. A decrease in the value of the *in-situ* s_u (relative to s_u for shear in compression) caused by different total stress paths and a decrease in the s_u of unconfined compression tests caused by disturbance during sampling (Ladd and Lambe, 1963) are possible compensating errors.

Additional research on the effects of sample disturbance and effects of total stress path is required before the engineer can select with confidence the proper values of undrained strength for $\phi = 0$ stability analyses in situations where experience has not yet developed an empirical set of guidelines. In particular, programmes of field and laboratory strength tests should give thought to the total stress path followed by different clay elements in the field.

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