EFFECT OF AGING ON THE SHEAR-STRENGTH PROPERTIES OF A NORMALLY CONSOLIDATED CLAY

by

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SYNOPSIS

In the Paper is described a series of triaxial tests on samples which were normally consolidated for different periods of time prior to the subsequent shear test. It is shown that the behaviour of the clay when sheared depends on the "age" of the samples. With time the clay becomes more brittle with smaller failure strains and the undrained shear strength shows a slight increase. A study of the stress-strain curves leads to the conclusion that the effect of time can be explained by the growing of cohesive bonds at the contact points between the particles. The cohesive bonds lead to a greater resistance against a shear deformation, but they are gradually destroyed for increasing strain.

Dans cet article on décrit une série d'essais triaxiaux sur des échantillons qui ont été normalement consolidés pendant différentes durées précédant l'essai de cisaillement. On montre que le comportement de l'argile lors du cisaillement dépend de la durée de consolidation de l'échantillon. Avec le temps, l'argile devient plus friable avec des déformations de rupture plus faible et la résistance au cisaillement "non drainée" augmente legèrement. L'étude des courbes déformation-contrainte, mènent à la conclusion suivante: l'effet du temps peut s'expliquer par l'augmentation des liens de cohésion aux points de contact entre les particules. Les liens de cohésion conduisent à une plus grande résistance à la déformation due au cisaillement, mais ils sont progressivement détruit en raison de l'effort croissant.

INTRODUCTION

As a part of a current research on the stress-strain properties of normally consolidated clays an investigation has been made on the influence of the time on the undrained shear-strength and pore-pressure characteristics of a normally consolidated clay.

There are two different ways in which the time factor may influence the results obtained by a conventional triaxial shear test. In the first place it is a well-known fact that the rate of application of the shear stresses has an influence on the shear strength of certain clay and shale (Casagrande and Wilson, 1951).[1]

Recent research (Bjerrum, Simons, and Torblaa, 1958) has, however, shown that in the second place a clay will behave differently depending on how long a time it has been left in the triaxial cell for consolidation. This means that samples consolidated for different periods of time prior to the shear testing might behave differently during a subsequent shear test depending on the "age" of the sample.

In the above-mentioned Paper it was, for instance, shown that clays which exhibit secondary consolidation will gain in shear strength with time. But also clays which do not show a secondary time effect might change their properties with time and the tests described in the Paper indicated that with time a clay might become more brittle, showing lower strains at failure.

This last-mentioned effect of time is of special interest for an evaluation of the undrained shear strength of normally consolidated clays. As pointed out in various Papers (Bjerrum and Simons, 1960; Bjerrum, 1961), there is a serious disagreement between the undrained shear strengths measured on undisturbed clay and on samples which have been reconsolidated in the laboratory. A possible explanation of this discrepancy may eventually be that an undisturbed clay since its deposition has gained some properties which cannot be reproduced in the laboratory where the time for which the sample is consolidated is of the order of a few days only. A reference should here be given to a previous study of this problem, made by Taylor (1953).

With the purpose of investigating this factor a series of triaxial tests were performed

[1] The references are given on p. 156.

in which samples of a normally consolidated clay, which did not show an appreciable secondary time effect, were sheared after different periods of aging. It is the results of this series of tests which will be described below.

PROPERTIES OF THE CLAY

The clay used for this study is a normally consolidated marine clay located at a place called Skabo, 2 miles west from the centre of Oslo. The undisturbed samples were obtained by the Norwegian Geotechnical Institute's thin-walled stationary piston sampler recovering samples of 80 cm long and 54 mm dia. In order to ensure homogeneity of the material used for the testing, the samples were taken from the same depths, 15–16 m, in eight boreholes made in the same vicinity.

The results of the standard field and laboratory tests carried out at Skabo are summarized in Fig. 1. The data of the clay used in the present programme are summarized in Table 1.

Table 1
Properties of Skabo clay

Water content, %	43·1 (39·4—47·8)
Liquid limit	52
Plastic limit	24
Plastic index	28
Clay fraction $< 2\mu$, %	45
Activity	0·63
Salt content, g/l	25 (24·2—28·8)
Organic content, %	0·32
s_u/p	0·25
Sensitivity	5

From the boring records in Fig. 1 it is seen directly that the clay, is unusually homogeneous with approximately constant Atterberg limits and water content between the depths of 8 and 18 m.

Results of X-ray examination and differential thermal analysis showed that the clay fraction was composed of about 40% illite, 20% chlorite, 25% quartz, and 15% feldspar. Except for the presence of the appreciable amount of chlorite, the mineralogical composition may be considered as typically representative of the clay in the Oslo region.

The results of about forty consolidation tests showed that the soil has a "weathered" zone approximately 11 m thick and below this depth the deposit is normally consolidated. This finding is confirmed by a study of the variation in undrained shear strength with depth. Below a depth of about 11 m the clay exhibits a linear increase of shear strength with depth. The ratio of the undrained shear strength to the consolidation pressure is 0·25 for the layer considered. The undrained shear strength being determined in situ by vane tests and in the laboratory by vane tests and unconfined compression tests.

All samples tested were undisturbed but reconsolidated at a pressure well above the effect stresses which the samples carried in the field, so that they can be considered as normally consolidated.

TESTING PROCEDURE

The tests were carried out with the triaxial equipment developed at the Norwegian Geotechnical Institute. The samples had a diameter of 3·56 cm (10 sq. cm area) and their height was 8 cm. In all tests double membranes separated with a thin layer of silicon grease were used.

In order to ensure that the samples tested were normally consolidated, the consolidation pressure used was 2·5 kg/sq. cm. Six samples were consolidated isotropically at this pressure and six samples were consolidated anisotropically at the major and minor principal stresses of 2·5 kg/sq. cm and 1·5 kg/sq. cm, respectively. For each series two samples were left at the consolidation pressure for a period of 3 days, two samples for 2 weeks and the last two samples for a period of 4 months.

After these periods of aging the samples were subjected to a conventional undrained shear test with measurements of pore pressures. The shear tests were carried out with a back pressure of 2 kg/sq. cm to ensure saturation, the cell pressure and the pore pressure being raised simultaneously. The strain rate used for the shear test was 1% in 48 min.

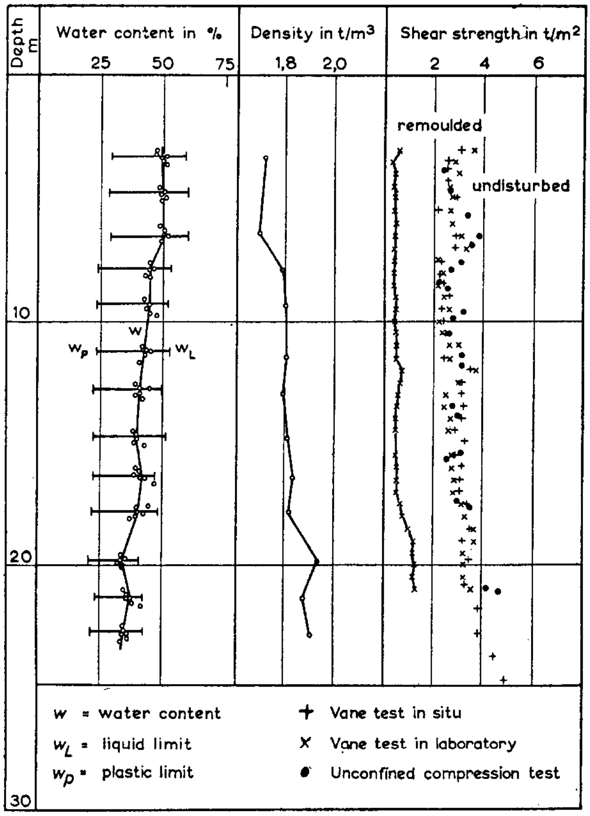

Fig. 1. Geotechnical profile through Skabo clay with results of field and laboratory tests

All tests were performed in duplicate and thus the range of scattering of the test results could be evaluated. The scatter was rather small and reliable comparison of the test results can therefore be made.

DISCUSSION OF TEST RESULTS

Since all tests were performed in duplicate and because samples consolidated isotropically as well as anisotropically were allowed to age for three different periods, the test series includes in total twelve tests. The details of the test results are collected in Table 2 and some of the most important characteristics are summarized in Table 3.

In Table 2 the failure parameters are listed, whereby it has been distinguished between the bulk failure of the sample at the peak value of the principal stress difference $(\sigma_1-\sigma_3)_{max}$ and the ultimate failure of the soil skeleton occurring at maximum effective principal stress ratio $(\sigma_1'/\sigma_3')_{max}$.

Table 2
Results of aging tests on Skabo clay

Depth: m	Type	Time of consolidation	Consolidation pressure: kg/sq. cm		w_i: %	w_f: %	$(\sigma_1 - \sigma_3)_{max}$						$(\sigma_1'/\sigma_3')_{max}$					
			σ_1	σ_3			ϵ_f: %	$(\sigma_1 - \sigma_3)$: kg/sq. cm	Δu: kg/sq. cm	A	σ_1'/σ_3'	Time to failure: min	ϵ: %	$(\sigma_1 - \sigma_3)$: kg/sq. cm	Δu: kg/sq. cm	A	σ_1'/σ_3'	Time to failure: min
15·10	CIU	3 days	2·5	2·5	41·6	33·1	3·8	1·75	1·63	0·93	3·02	200	8·3	1·74	1·81	1·04	3·53	404
15·45	CIU	3 days	2·5	2·5	48·2	38·2	3·3	1·70	1·59	0·94	2·87	223	8·4	1·67	1·82	1·09	3·45	448
15·75	CAU	3 days	2·5	1·5	39·4	34·4	1·9	1·76	0·59	0·78	2·94	335	9·4	1·66	0·77	1·16	3·28	1,128
15·85	CAU	3 days	2·5	1·5	40·0	33·4	2·3	1·78	0·72	0·93	3·28	307	10·6	1·68	0·90	1·33	3·79	1,255
15·40	CIU	2 weeks	2·5	2·5	40·6	32·7	3·6	1·89	1·57	0·83	3·04	195	11·8	1·90	1·77	0·93	3·60	571
15·50	CIU	2 weeks	2·5	2·5	41·2	32·4	4·3	1·90	1·63	0·86	3·18	230	9·3	1·85	1·72	0·93	3·37	456
15·75	CAU	2 weeks	2·5	1·5	42·8	33·4	0·9	1·90	0·47	0·52	2·85	135	5·0	1·76	0·77	1·01	3·41	635
15·85	CAU	2 weeks	2·5	1·5	43·2	33·5	0·8	1·78	0·48	0·62	2·74	124	11·8	1·46	1·00	2·17	3·93	1,382
15·35	CIU	4 months	2·5	2·5	47·8	37·6	2·0	1·85	1·31	0·71	2·55	116	10·1	1·65	1·80	1·09	3·36	452
15·55	CIU	4 months	2·5	2·5	46·5	36·4	2·2	1·97	1·40	0·71	2·78	135	8·9	1·77	1·79	1·02	3·49	435
16·10	CAU	4 months	2·5	1·5	41·9	35·4	0·5	1·74	0·36	0·49	2·53	99	12·0	1·32	0·91	2·84	3·24	1,481
16·20	CAU	4 months	2·5	1·5	43·5	36·2	0·5	2·02	0·34	0·34	2·74	100	11·0	1·54	0·92	1·70	3·66	1,270

Table 3
Summary of test results

Type of test:	Time of consolidation:	Water content:		At $(\sigma_1 - \sigma_3)_{max}$						At $(\sigma_1'/\sigma_3')_{max}$				
		w_i	w_f	ϵ: %	$\frac{\frac{1}{2}(\sigma_1 - \sigma_3)}{p}$	σ_1'/σ_3'	ϕ'	D_M: %	A_f	ϵ: %	σ_1'/σ_3'	ϕ'_u	A	$\frac{\Delta u}{p} + 1 - K$
CIU	3 days	44·9	35·7	3·6	0·346	2·95	29·6°	85	0·94	8·4	3·49	33·7°	1·07	0·72
	2 weeks	40·9	32·5	4·0	0·379	3·12	31·0°	90	0·85	10·6	3·49	33·7°	0·93	0·70
	4 months	37·0	37·0	2·1	0·382	2·67	27·0°	78	0·71	9·5	3·43	33·2°	1·05	0·72
CAU	3 days	39·7	33·9	2·1	0·354	3·11	30·8°	88	0·88	10·0	3·54	34·0°	1·24	0·73
	2 weeks	43·0	33·4	0·9	0·368	2·80	28·3°	85	0·57	8·4	3·67	33·9°	1·59	0·75
	4 months	42·7	35·8	0·5	0·376	2·64	26·8°	77	0·42	11·5	3·45	33·4°	2·27	0·77

Before discussing the effect of the age on the shear strength characteristics, it is of importance to realize that there was no appreciable reduction in water content of the samples during the aging periods. This observation is confirmed by the measurements of the final water contents, see Table 3, which do not show any regular decrease with the period of aging.

From the shear-strength characteristics computed at the peak values of $(\sigma_1 - \sigma_3)$ and σ_1'/σ_3' and listed in Tables 2 and 3 the following conclusions can be drawn:
At $(\sigma_1 - \sigma_3)_{max}$ the effect of an increase of period of aging is:

(*a*) the undrained shear strength $\frac{1}{2}(\sigma_1 - \sigma_3)/p$ increase slightly;

(*b*) the pore-pressure parameter A_f decreases;

(*c*) the strain at failure decreases;

(*d*) the angle of shearing resistance ϕ' mobilized at $(\sigma_1 - \sigma_3)_{max}$ decreases and the degree of mobilization defined as: $D_M = \tan\phi'/\tan\phi'_u$ thus decreases.

At $(\sigma_1'/\sigma_3')_{max}$ the effect is:

(*e*) the maximum principal effective stress ratio σ_1'/σ_3' remains unchanged and so does the ultimate angle of shearing resistance ϕ'_u;

(*f*) the pore-pressure parameter A remains unchanged for the samples consolidated isotropically, but increases for the anisotropically consolidated samples.

The conclusions drawn may well be useful for an evaluation of the effect of time on the conventionally used shear strength parameters, but they contribute only little to a fundamental understanding of the change of properties of a clay occurring when it is left at a sustained consolidation pressure for a period of time. Much can however be learned from a study of the stress-strain properties and for this purpose the test results have been plotted in Figs 2 and 3. For a comparison of the stress-strain properties of a number of tests it has previously been found convenient to make use of the following equation which is generally valid and holds good for any strain:

$$\frac{\sigma_1 - \sigma_3}{p} = (\sigma_1'/\sigma_3' - 1)\left[1 - \left(\frac{\Delta u}{p} + (1 - K)\right)\right]$$

in which p and Kp represent the major and minor principal consolidation pressures. In this equation σ_1'/σ_3' and $\Delta u/p$ are considered to be independent functions of the strain and these functions will be discussed separately.

EFFECT OF AGING ON $\frac{\Delta u}{p} - \epsilon$ RELATIONSHIP

The change of pore-pressure ratio $\Delta u/p$ in relation to strain for all the samples consolidated isotropically and anisotropically are plotted in Figs 4(a) and 4(b), respectively.

Fig. 4(a) shows clearly that in spite of the different period of aging a unique relationship exists between excess pore-pressure and major principal strain. The points resulting from the performed six tests on the isotropically consolidated samples fall very close to a single curve. The conclusion may therefore be reached that aging has no effect on the pore-pressure/strain relationship.

In Fig. 4(b) are plotted the excess pore pressures observed during the shearing of the anisotropically consolidated samples. These test results show a considerable scatter, but a detailed study shows that the variation of the observed pore pressures does not correspond to or is not correlated with any specific variable such as time of aging or the water content. It is believed, therefore, that the scatter is a result of small variations of principal stress ratio during the consolidation (Lo, 1961).

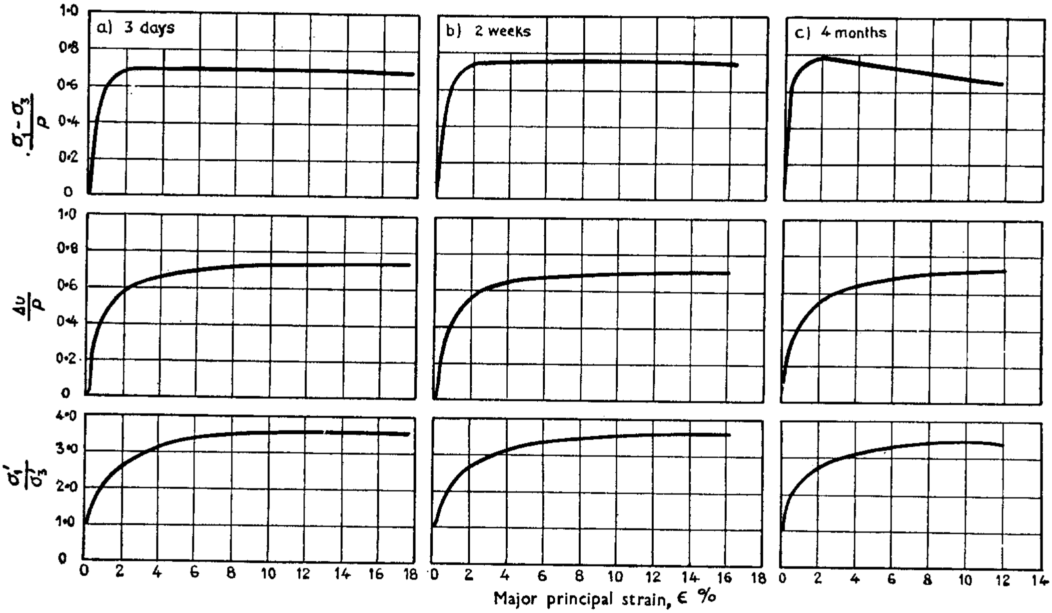

Fig. 2. Results of consolidated undrained triaxial tests on samples consolidated isotropically for various periods of times: (a) 3 days, (b) 2 weeks, (c) 4 months. Consolidation pressure 2·5 kg/sq. cm. Each curve represents the average of two tests

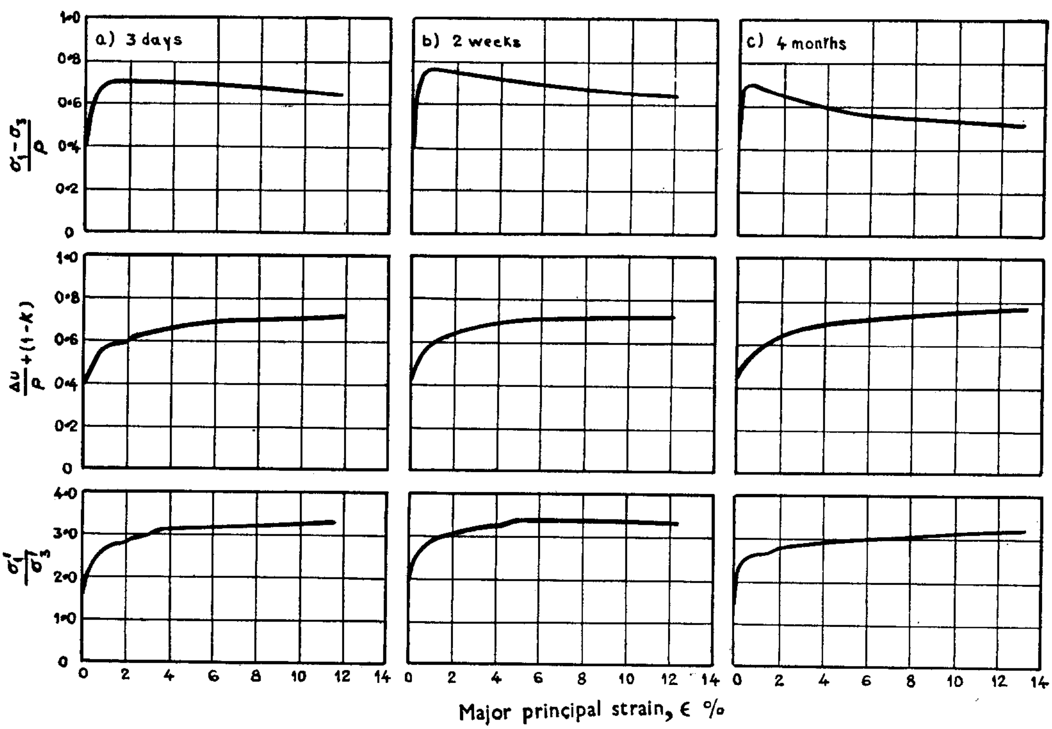

Fig. 3. Results of consolidated undrained triaxial tests on samples consolidated anisotropically for various periods of times: (a) 3 days, (b) 2 weeks, (c) 4 months. Consolidation pressure $p_1 = 2{\cdot}5$ kg/sq. cm, $p_2 = p_3 = 1{\cdot}5$ kg/sq. cm. Each curve represents the average of two tests

The maximum values of $\Delta u/p + (1 - K)$ are listed in Table 3 for comparison. The values determined for the anisotropically consolidated samples are slightly higher than those observed for the isotropical samples, a finding which has been confirmed also for other types of clay (Bjerrum and Lo, 1961). The tests on the isotropical samples show no tendency for this quantity to vary with the time of aging, but a small increase with time of aging was observed for the samples consolidated anisotropically.

EFFECT OF AGING ON $\sigma_1'/\sigma_3' - \epsilon$ RELATIONSHIP

In Figs 5(a) and 5(b) are shown the variation of principal effective stress ratio with strain for samples consolidated isotropically and anisotropically, respectively.

From the plotted curves it is observed at once that at a given strain the effective principal stress ratio is higher for samples with longer period of consolidation. However, the curves converge into a single one as the ratios approach their maximum value. This trend is observed for both the isotropically and the anisotropically consolidated samples, with the exception that the curve for samples consolidated anisotropically over a period of 4 months lies below the two others for large strains.

The maximum values of σ_1'/σ_3' are listed in Table 3 and they show no pronounced variation with the period of aging; if anything, there is a tendency towards a small reduction with aging.

EFFECT OF AGING ON $\frac{\sigma_1 - \sigma_3}{p} - \epsilon$ RELATIONSHIP

From the graphical plots of the test results in Figs 2 and 3 it is directly seen that an aging has a very marked influence on the $(\sigma_1 - \sigma_3)$ — strain relationship. For an increased period of aging the maximum value of $\sigma_1 - \sigma_3$ occurs at a smaller strain and the reduction in $\sigma_1 - \sigma_3$ after failure becomes more pronounced.

Considering the maximum values of $\sigma_1 - \sigma_3$ there is a slight increase in the undrained compressive strength of the samples with aging. This increase is of the order of 6–10% for the range of time variations investigated in the present test series.

Now, the value of $\sigma_1 - \sigma_3/p$ is at any strain a function of the two basic parameters $\Delta u/p$ and σ_1'/σ_3' as expressed by the equation:

$$\frac{\sigma_1 - \sigma_3}{p} = (\sigma_1'/\sigma_3' - 1)\left\{1 - \left[\frac{\Delta u}{p} + (1 - K)\right]\right\}$$

$\sigma_1 - \sigma_3/p$ is thus a product of a "strength term" $(\sigma_1'/\sigma_3' - 1)$ which increases with strain and an "effective stress term" $\left\{1 - \left[\frac{\Delta u}{p} + (1 - K)\right]\right\}$ which in undrained tests on normally consolidated clays decreases for increasing strain. The maximum value of $\sigma_1 - \sigma_3/p$ will, therefore, in general not occur at the strain where the strength term is maximum, but at an intermediate strain where the rate of decrease of the effective stress term just compensates the rate of increase of the strength term, i.e. when:

$$\frac{\dfrac{d\left(\dfrac{\Delta u}{p}\right)}{d\epsilon}}{1 - \left(\dfrac{\Delta u}{p} + (1 - K)\right)} = \frac{\dfrac{d(\sigma_1'/\sigma_3')}{d\epsilon}}{\sigma_1'/\sigma_3' - 1}$$

This fact explains the observation from tests on normally consolidated clays that there is a rise in pore pressure at the peak value of $\sigma_1 - \sigma_3$.

Turning the attention towards the effect of aging, it has been shown above that the

$\Delta u/p - \epsilon$ relationship does not change with aging, whereas the $\sigma_1'/\sigma_3' - \epsilon$ curves show a steeper rise at small strains. Consequently, the $(\sigma_1 - \sigma_3) - \epsilon$ curves will rise more steeply and reach maximum values at smaller strains for longer aging periods. A decrease of failure strain with aging will also mean a corresponding smaller value of excess pore pressure at failure. For example, when the aging period for the CAU tests was increased from 3 days to 4 months the maximum value of $\sigma_1 - \sigma_3$ changed only very little, about 6%, whereas the pore pressure at failure was reduced by about 50%. This explains why the A value is reduced from 0·88 to 0·41 in spite of the fact that the excess pore-pressure/strain

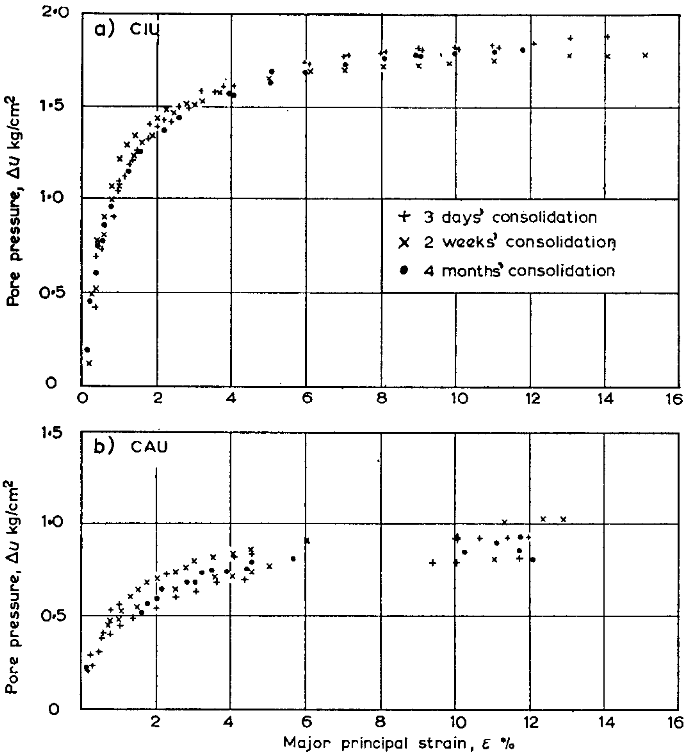

Fig. 4. Excess pore pressure set up during consolidated undrained tests: (a) tests on isotropically consolidated samples, (b) tests on anisotropically consolidated samples. For each type of consolidation are shown the results of six separate tests

relationship is not influenced by aging. Again, as the strain at failure decreases, the σ_1'/σ_3' value at $(\sigma_1 - \sigma_3)_{max}$ is reduced, and for the samples which were consolidated for a period of 4 months the degree of mobilization at failure was as low as 77–78%.

SOME NOTES ON DEVELOPMENT OF A STRUCTURAL STRENGTH WITH TIME

Because the samples showed initially the same structure and because the secondary consolidation was insignificant, the samples for each mode of consolidation had the same structure prior to the shear test. At a given strain it is believed that approximately the same number of contact points are broken, resulting in identical magnitudes of pore pressure set up due to transference to the pore-water of effective stresses originally carried at the contact points. To produce a given strain, a higher value of σ_1'/σ_3' is required for the samples consolidated 4 months than for those consolidated only 3 days. The conceivable effect of aging is therefore a growth of bond strength at the points of contact of the particles.

As the strain increases, movement occurs at most or all of the contact points and the bonds developed during the consolidation period are destroyed. The σ_1'/σ_3' values for different aging periods therefore gradually converge into a single curve and their final maximum value is independent of the initial bond strength. The additional cohesive component of the shear strength gained with time during a consolidation period is therefore destroyed, at small strains and does not contribute to the final strength of the soil skeleton.

It is reasonable to expect that the structure of the soil samples varies with different modes of consolidation. It is therefore also conceivable that the pore-pressure and effective stress-ratio characteristics might be slightly different during the early stages of the shearing process of the isotropically and the anisotropically consolidated samples. At large strains when the

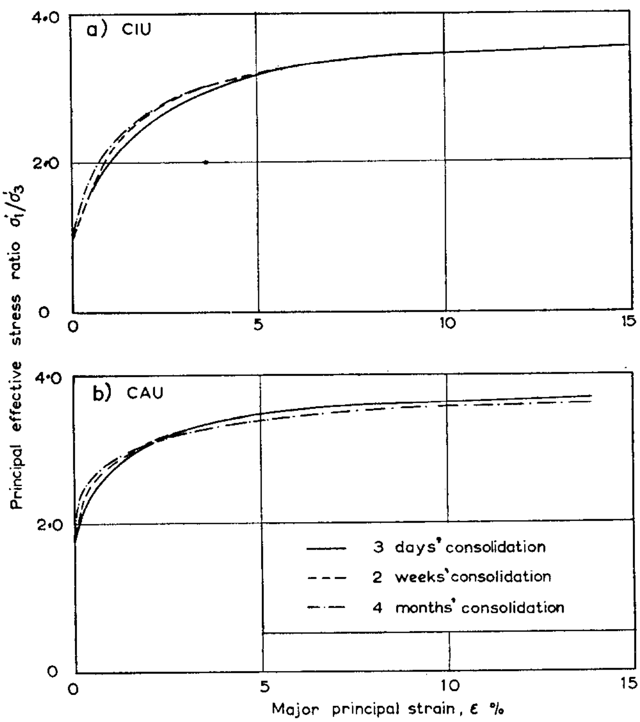

Fig. 5. Principal effective stress ratio as observed in consolidated undrained tests on: (a) isotropically consolidated samples, and (b) anisotropically consolidated samples. Each curve represents the average of two tests.

initial bonds have been destroyed, the mechanical behaviour of the particles is gradually dominated by sliding. It is therefore logical to expect that the ultimate values of the effective principal stress ratio would be the same for both isotropically and anisotropically consolidated samples, and this has been shown to be the case, as seen from Table 3.

There are no reasons to believe that the bonds developed at the contact points during the 4 months of aging included in the present test programme represent the maximum values which can be reached in geological time. On the contrary, there are good reasons to believe that these bonds, which do not show up in conventional shear tests on reconsolidated samples, are of considerably greater importance in nature than observed in the tests.

Even if the test results do not permit a quantitative evaluation of the effect of aging over periods of time on a geological scale, they indicate clearly in a qualitative way the general trends of further aging. A continued growth of bonds at the contact points will lead to a further increase of the steepness of the early part of the stress-strain curve of the clay and the undrained shear strength will therefore be mobilized at a smaller strain. Assuming that the aging has no effect on the pore-pressure/strain relationship as observed in the tests, the excess pore pressure at an undrained failure will be reduced. The degree of mobilization of the effective stress shear strength parameters will therefore also be reduced.

With increasing consolidation time the undrained shear strength will thus to an increasing degree be controlled by the contribution to the strength of the cohesive bonds prevailing at small strains. The frictional component, the mobilization of which requires strain, will be reduced correspondingly.

It should finally be mentioned that the existence of cohesive bonds in clays which are developed with time and are destroyed completely or partly by reconsolidation has previously been reported for a clay from the Göta valley (Bjerrum and Wu, 1960). Terzaghi (1941) suggested that such bonds may be the controlling factor for the equilibrium water contents of natural clay sediments and recent experiments carried out at Purdue University (Leonard and Ramiah, 1960) have demonstrated by consolidation tests on aged samples that cohesive bonds were formed with time leading to an increased resistance against volume reduction.

CONCLUSION

In the described test series, normally consolidation samples were aged for different periods of time before they were subjected to undrained shear tests in the triaxial apparatus. The results can be summarized as follows:

(1) Samples with longer periods of consolidation show at early stages of the tests a greater resistance against a shear distortion. The effective principal stress ratio increases more rapidly at small strains for the older samples. For larger strains this effect of aging diminishes and it disappears completely when the ultimate value of the effective principal stress ratio is reached.

(2) The excess pore-pressure/strain relationship observed during the shear tests were independent of the age of the samples.

(3) The principal stress difference reaches its maximum value at a smaller strain the longer the sample has been left for aging. The maximum value shows a slight increase with the age of the samples.

(4) A study of the stress-strain and pore-pressure/strain curves suggests that the change of behaviour of samples for increasing age is a result of the bonds which develop at contact points between the clay particles. These bonds are gradually destroyed for increasing shear strain. The cohesive contribution to the undrained shear strength is thus believed to increase and the frictional contribution to decrease with the age of the sample.

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