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THE INFLUENCE OF RATE OF STRAIN
ON EFFECTIVE STRESSES IN SENSITIVE CLAY

BY

CARL B. CRAWFORD

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THE INFLUENCE OF RATE OF STRAIN ON EFFECTIVE STRESSES IN SENSITIVE CLAY

BY CARL B. CRAWFORD¹

SYNOPSIS

Triaxial compression tests with pore water pressure measurements on undisturbed samples of a sensitive marine clay common to Eastern Canada and United States are reported. The stress and pore water pressure at failure depend on rate of strain. It is suggested that a breakdown of soil structure determines the level of pore water pressure. On a Mohr stress diagram it is shown that the failure envelope in terms of effective stresses is dependent on rate of strain application, and the implications associated with this observation are discussed.

LIST OF SYMBOLS

f	= failure condition,
ϵ	= strain, per cent,
T_f	= time to failure, hr,
σ	= normal stress, kg per sq cm,
u	= pore water pressure, kg per sq cm
Δu	= pore pressure change during shear, kg per sq cm,
$\bar{\sigma}$	= effective stress, kg per sq cm,
σ_c	= consolidation cell pressure, kg per sq cm,
$\sigma_1 - \sigma_3$	= deviator stress, kg per sq cm,
A	= pore pressure parameter—ratio of pore pressure change to deviator stress during shear,
c'	= cohesion intercept in terms of effective stresses, kg per sq cm,
ϕ'	= angle of shearing resistance in terms of effective stresses.

as compression, distortion and a change of the shearing resistance are exclusively due to changes in the effective stresses” It was Terzaghi’s appreciation of effective stresses in soils that distinguished his earlier work (2), but this was a clear statement of the “principle of effective stresses” as it is known today. (The effective stress on any plane is the so-called grain-to-grain stress and is found by subtracting the pore water pressure from the total stress on the plane, that is $\bar{\sigma} = \sigma - u$.) Jurgenson (3) and Casagrande (4) had already published some laboratory proof of the importance of effective stresses in clay, but it was on the more extensive work of Rendulic (5) and Hvorslev (6) that Terzaghi based his classic statement.

At the First International Conference on Soil Mechanics and Foundation Engineering in 1936, Terzaghi (1)² stated that for soil materials “all the measurable effects of a change of the stress, such

The significance of effective stresses in soils was emphasized in the review of the U.S. Corps of Engineers triaxial shear research program by Rutledge in 1947 (7). Because of the difficulty of determining pore water pressures in the laboratory and in the field, every effort was made in this review to interpret shear

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² The boldface numbers in parentheses refer to the list of references appended to this paper.

strength in terms of total stresses. At the Second International Conference in 1948 however, Skempton (8,9) compared triaxial test results in terms of total and effective stresses and pointed out certain important limitations to analyses in terms of total stresses.

Following the Second International Conference, a great effort was made to investigate the shearing strength of clay soils in terms of effective stresses. New equipment was developed and much testing was carried out, particularly on uniform remolded clay specimens. This work confirmed Hvorslev's earlier research, and for a number of soils an appreciation of the strength and deformation properties under effective stresses was obtained. Typical of the developments is the work of Gibson (10), Casagrande and Wilson (11), Bjerrum (12), Skempton and Bishop (13), and Hilf and Gibbs (14).

In more recent years field evidence has been obtained to support the validity of effective stress analyses. For heavily overconsolidated clays, it has been clearly shown by Henkel and Skempton (15) and by Henkel (16) that stability analyses using shear strength parameters in terms of total stresses can lead to a serious over-estimate of safety factor. Sevaldson (17) and Bjerrum and Kjaernsli (18) present field evidence to show similar, although smaller, errors in the total stress analysis of normally consolidated clays.

SOME CANADIAN STUDIES

In an effort to apply the well-developed principles outlined above, the Division of Building Research in Canada began standard triaxial shear tests on a saturated, sensitive, marine clay found commonly in the Ottawa region. In a series of consolidated-undrained (consolidated-quick) triaxial tests on borehole samples of this clay it was noticed that the pore water pressure continued to rise after the sample had failed in

shear, that is, maximum deviator stress, $\sigma_1 - \sigma_3$. At first this was attributed to a delay in pore pressure pick-up, and the rate of strain was reduced from 2 per cent per hr to 0.25 per cent per hr in order to allow more time for pore pressure in the sample to reach equilibrium with the measuring apparatus. It was found, however, that although the rate of strain was reduced to one eighth of the original rate the pore pressure continued to increase after failure in much the same manner. Because the properties of the borehole samples were considerably affected by the variable nature of the clay it was impossible to study satisfactorily the effect of strain rate on shear strength and pore water pressure. Therefore block samples of the soil were obtained from an excavation in the city of Ottawa and tested at various degrees of consolidation and at different rates of strain. This paper describes the tests and discusses the results.

PROPERTIES OF THE SOIL

The marine clay, called in Canada, Leda clay, is widely distributed through the valleys of the Ottawa and St. Lawrence Rivers. Its geology and geotechnical properties have been described by Eden and Crawford (19). It is in general a highly plastic, highly compressible and sensitive postglacial clay which was deposited in the Champlain Sea as the glaciers retreated from North America. It is usually slightly preconsolidated, brittle and sensitive, often remolding to a liquid consistency. It is usually highly stratified with plasticity, grain size and water content varying appreciably over a few inches of vertical profile. The mineralogy is not established beyond question, but it is thought to contain appreciable amounts of feldspar, illite, and chlorite.

Because of the stratification, homogeneous samples are not easy to obtain.

In a detailed examination of a 22-ft deep excavation only one layer about 4 in. thick was found which had an almost uniform water content. The tests reported in this paper were carried out on undisturbed specimens cut from block samples from this layer. The layer, at a depth of 16 ft, is well below the brown

tent in the pore water is less than 0.5 g per liter and the liquidity index is 1.05.

CONSOLIDATED-UNDRAINED TEST RESULTS

Thirteen consolidated-undrained axial compression tests were carried out on 1.4-in. diameter specimens in accordance

TABLE I.—CONSOLIDATED-UNDRAINED TEST RESULTS.

Series	Sample	Cell Pressure, σ_c , kg per sq cm	Water Content		e_f , per cent	T_f , hr	$(\sigma_1 - \sigma_3)_f$, kg per sq cm	u_f , kg per sq cm	$\Delta u'_f$, kg per sq cm	A'_f	$\Delta u''_f$, kg per sq cm	A''_f
			Initial, per cent	Failure, per cent								
A.....	83-27- 8	2.0	66.8	61.0	1.1	9.4	1.50	1.96	0.96	0.64	0.90	0.60
	-11	2.0	66.8	62.4	1.3	0.77	1.80	2.14	1.14	0.63	0.98	0.55
	-10	2.0	67.4	64.4	1.3	0.13	1.98	1.99	0.99	0.50	1.02	0.51
B.....	- 3	2.75	66.8	52.8	1.5	17.8	1.72	2.64	1.64	0.95	1.75	1.02
	- 2	2.75	66.8	53.3	1.5	6.0	1.74	2.55	1.55	0.89	1.55	0.89
	- 1	2.70	66.7	55.1	1.5	3.3	1.72	2.57	1.57	0.91	1.34	0.78
	- 6	3.00	67.0	54.2	1.6	0.4	1.90	2.76	1.76	0.93	1.30	0.68
	- 7	2.75	67.4	53.7	1.5	0.15	2.00	2.38	1.38	0.69	1.30	0.65
C.....	- 9	4.0	67.1	46.2	1.8	22.0	2.04	3.42	2.42	1.19	2.26	1.11
	- 5	4.0	66.4	47.2	1.6	3.7	2.22	3.26	2.26	1.02	2.30	1.04
	- 4	4.0	66.8	46.6	1.7	0.96	2.38	3.03	2.03	0.85	1.96	0.82
D.....	-13	6.0	66.5	41.5	1.8	4.2	3.34	4.10	3.10	0.93	3.23	0.97
	-12	6.0	66.6	42.0	1.5	0.82	3.46	3.76	2.76	0.80	2.73	0.79

$\Delta u'_f$ = u_f -induced back pressure,

$\Delta u''_f$ = u_f -pore water pressure at start of shear,

Pore pressure parameter $A'_f = \frac{\Delta u'_f}{(\sigma_1 - \sigma_3)_f}$,

Pore pressure parameter $A''_f = \frac{\Delta u''_f}{(\sigma_1 - \sigma_3)_f}$.

oxidized zone but within the fissured crust which often extends to depths greater than 20 ft.

The soil in this thin layer had an average natural water content of 66.8 per cent. The field vane strength was 0.64 kg per sq cm undisturbed and 0.03 kg per sq cm remolded giving a sensitivity just over 20. The liquid limit averages 65 per cent and the plastic limit 26 per cent. The material is about 60 per cent clay size and 40 per cent silt size. The salt con-

with the procedures described by Bishop and Henkel (20). Filter paper side drains and 0.01-cm thick rubber membranes were used but no correction for drain and membrane strength has been applied to the results. Each specimen was consolidated in stages until practically no water was flowing out of the specimen and in all cases more than one day was allowed between the last increment of cell pressure and the start of shear. The total time allowed for consolidation

varied from 7 to 14 days. After consolidation, the pore pressure measuring equipment was connected to the bottom drainage stone, and cell pressure and pore pressure were raised simultaneously

FIG. 1.—Typical Test Results.

by 1 kg per sq cm to provide a back pressure in the pore water. Then pore water flow was stopped and the pore pressure allowed to come to equilibrium. From 2 to 6 hr was generally allowed for this, but in two cases (83-27-11 and 83-27-1) the sample stood overnight.

Complete test results are given in Table I and typical stress-strain and pore water pressure curves are shown in Fig. 1. In all cases failure was assumed at maximum deviator stress and these values are shown plotted against time to failure, T_f , in Fig. 2. Shown on the same plot are the results of recent tests by Bjerrum et al (21) on a normally consolidated marine clay in Norway; these results also agree with the extensive research of Casagrande and Wilson (22).

The measured changes in pore water pressure during shear, Δu_f , are indicated in Table I. Slightly different values are obtained depending on whether the back pressure is subtracted from the total measured pore pressure at failure or whether the initial pore pressure (at start of shear) is subtracted. The greatest variation occurred in samples 83-27-6 and 83-27-1 where the initial pore pressure (at start of shear) was 1.46 and 1.23 kg per sq cm respectively. In all other tests the initial pore pressure was within 15 per cent of the back pressure (that is, 1.0 kg per sq cm). Because the magnitude of pore water pressure at the start of shear is a factor which affects the magnitude of pore pressure change during shear, it would have been better to have consolidated the specimens under a fixed back pressure (20, p. 113). This would have avoided the ambiguity between $\Delta u'_f$ and $\Delta u''_f$ as shown in Table I.

Because of the possible ambiguity in pore pressure change at failure, both values are used to determine pore pressure parameter at failure A_f as defined by Skempton (23):

$$A_f = \frac{\Delta u_f}{(\sigma_1 - \sigma_3)_f}$$

and these are entered in Table II and plotted on Fig. 3. It has been shown by

FIG. 2.—Deviator Stress at Failure.

FIG. 3.—Pore-Pressure Parameter A_f at Failure.

Henkel (24) that the pore-pressure parameter A_f depends somewhat on the degree of overconsolidation. This has been considered in drawing one curve to represent

tests at cell pressures of 4 and 6 kg per sq cm (normally consolidated specimens). The preconsolidation pressure for the soil is approximately 2.0 to 2.5 kg

per sq cm,³ but there was no justification for grouping A_f values from tests at cell pressures of 2.8 kg per sq cm with those at the higher cell pressures. Samples

From the averaged curves for deviator stress (Fig. 2) and for A_f (Fig. 3), the average pore pressure at failure was calculated ($\Delta u_f = A_f(\sigma_1 - \sigma_3)_f$) for times

TABLE II.—AVERAGED TEST VALUES.

T_f , hr.....	0.10				1.0				10			
Test Series.....	A	B	C	D	A	B	C	D	A	B	C	D
σ_c (kg per sq cm).....	2.0	2.8	4.0	6.0	2.0	2.8	4.0	6.0	2.0	2.8	4.0	6.0
$(\sigma_1 - \sigma_3)_f$	2.00	2.10	2.62	3.73	1.75	1.87	2.37	3.48	1.50	1.61	2.11	3.22
A_f	0.5	0.63	0.66	0.66	0.57	0.78	0.85	0.85	0.64	0.93	1.04	1.04
Δu_f	1.0	1.32	1.73	2.46	1.0	1.46	2.02	2.96	0.96	1.50	2.20	3.35
$(\bar{\sigma}_3)_f$	1.0	1.48	2.27	3.54	1.0	1.34	1.98	3.04	1.04	1.30	1.80	2.65
$(\bar{\sigma}_1)_f$	3.0	3.58	4.89	7.27	2.75	3.21	4.35	6.52	2.54	2.91	3.91	5.87
$(\bar{\sigma}_1/\bar{\sigma}_3)_f$	3.0	2.42	2.16	2.05	2.75	2.40	2.20	2.15	2.44	2.24	2.17	2.22

σ_c = maximum consolidation pressure,

$(\sigma_1 - \sigma_3)_f$ = deviator stress at failure,

A_f = pore pressure parameter at failure,

Δu_f = average pore pressure change during shear,

$(\bar{\sigma}_3)_f$ = minor principal effective stress at failure,

$(\bar{\sigma}_1)_f$ = major principal effective stress at failure,

$(\bar{\sigma}_1/\bar{\sigma}_3)_f$ = principal effective stress ratio at failure.

FIG. 4.—Averaged Pore Pressure Changes at Failure.

tested at a cell pressure of 2 kg per sq cm are slightly overconsolidated and the A_f values are considerably lower.

³ J. J. Hamilton and C. B. Crawford, "Improved Determination of Preconsolidation Pressure of a Sensitive Clay," discusses the difficulties in determining the preconsolidation pressure of this soil. See p. 254 of this publication.

to failure of 0.1, 1.0, and 10 hr as shown in Table II and Fig. 4. Also listed in Table II are the effective principal stresses and the effective principal stress ratios at failure. From the averaged test results shown in Table II, Mohr stress circles are plotted in Fig. 5 and the failure envelopes are compared in Fig. 6 for failure times of 0.1, 1.0, and 10 hr.

In all but two tests, the loading of specimens was carried on continuously past failure and in some cases to more than 15 per cent strain. Preliminary work had included a number of tests in which the deviator stress $(\sigma_1 - \sigma_3)$ was held constant for several hours and several in which strain was held constant. It was noted that under constant deviator stress the pore pressure continued to rise and under conditions of constant strain the pore pressure decreased. To investigate this feature, samples 83-27-12 and 83-27-13 were loaded to about one-third of failure stress and then held at constant deviator stress for several hours before

continuing the loading. Figure 7 shows the increase in pore water pressure with time, and Fig. 8 shows the increase in

ship between Δu and strain is noted. For sample 83-27-13 (strained at 0.5 per cent per hr) the relationship is simple pro-

FIG. 5.—Mohr Stress Circles and Failure Envelopes.

FIG. 6.—Failure Envelopes by Various Procedures.

pore water pressure with change in percentage strain. No simple relationship between pore pressure increase, Δu , and time is observed, but a definite relation-

portion. For sample 83-27-12 (strained at 2.0 per cent per hr) there appeared to be a lag in pore pressure response of a little less than 0.1 kg per sq cm when

loading was stopped, and this lag was made up during a strain increment of 0.03 per cent (time interval about 40 min).

To investigate whether pore pressure gradients exist in test specimens, these

DRAINED TEST RESULTS

To study drained (slow) test results on this soil, three specimens were consolidated under cell pressures of 2.0, 2.8, and 4.0 kg per sq cm respectively and then sheared at a rate of strain of 0.17

FIG. 7.—Change in Pore Pressure with Time under Constant Stress.

FIG. 8.—Relation Between Pore Pressure and Strain under Constant Stress.

two samples were divided after testing into slices parallel to the apparent failure plane (as shown in Fig. 9) and the water content of each slice was determined very accurately. The soil was found to be about 1 per cent on the dry side of average around the failure region and about 1 per cent on the wet side of average in regions farthest from failure planes.

per cent per hr under full drainage conditions. This rate is equivalent to the undrained test where time to failure is 10 hr. Test results, in Fig. 10, show a sharp change in the stress-strain relationship below 1 per cent strain and then a steady deviator stress increase to 20 per cent strain. No distinct failure condition is indicated.

The Mohr failure envelope falls considerably below the envelope for undrained tests until rather high strains have taken place. The envelope then indicates a negative value for the cohesion intercept c' and a high value for ϕ' (27 deg at 15 per cent strain shown on Fig. 6). Because of the unsatisfactory nature of drained test results, other rates of strain were not tried.

DISCUSSION OF RESULTS

These tests demonstrate the significant influence of the rate of strain on deviator stress and pore water pressure at failure in an undisturbed sensitive clay.

FIG. 9.—Water Content Variation at End of Test in Two Samples.

angle of shearing resistance in terms of effective stresses increases from 17.5 to 23 deg as the time to failure increases from 0.1 to 10 hr. A corresponding decrease in the cohesion intercept is observed. Of even more importance is the substantial decrease in shearing resistance of specimens below the preconsolidation pressure as the testing time increases.

The first question to be answered in evaluating these results is whether the measured pore pressures satisfactorily represent the actual pore pressures in the specimen. The water content distribution in two specimens after failure (Fig. 9) suggests that a pore pressure gradient was developed in each specimen during shear. Since pore pressures

build up as a result of shearing strains, it is not surprising that the maximum pore pressure is at the shear plane and that this results in a flow of water away from the shear plane. Both of these tests were strained well past failure so that only a portion of the water was moved before failure. Nevertheless it is evident that there is a small pore pressure gra-

FIG. 10.—Drained Test Results.

dient between the failure plane and the base plate of the triaxial cell.

Results shown in Fig. 8 give a further clue to the relationship between actual and measured pore pressures. Specimen 83-27-13 was loaded to a deviator stress of 1 kg per sq cm in about 60 min and then the deviator stress was held constant and the strain and pore pressure measured. It is seen that there is a simple relationship between strain and pore pressure. If a substantial pore pressure gradient had existed, a period of adjustment would have been expected.

Specimen 83-27-12 was loaded in exactly the same manner but at five times the rate used in 83-27-13. Here the adjustment period is evident and by projecting back the pore pressure - strain curve a pore pressure lag of about 0.09 kg per sq cm is suggested.

In addition to these specific tests all of the regular pore pressure - strain curves were compared, and although the final pore pressure at failure increased with increasing time to failure the shapes of the curves were quite similar for specimens loaded at rates of strain varying by a factor of 100, that is, 0.12 per cent per hr to 12.0 per cent per hr.

It is concluded therefore that undrained tests strained at a rate of 2 per cent per hr (failure time 40 to 60 min) will allow satisfactory measurement of pore water pressure using side drains as recommended by Bishop and Henkel (20).

If there is only a small lag in pore pressure response, the reason for the continuing rise after failure (Fig. 1) and for greater pore pressure development in slow tests (Fig. 4) requires further consideration. It is suggested that the pore pressure level in the sample is a function of structural breakdown in the sample and this in turn is related to strain. Lambe (25) explains that remolding increases the order of particle orientation and decreases the volume of an undisturbed marine clay. If, as in undrained triaxial tests, the volume cannot change, the remolding caused by straining along the main shear plane will transfer applied stresses to the pore water. In addition to raising the pore water pressure throughout the sample, this will cause a redistribution of water along the pressure gradient as shown in Fig. 9.

The curves of Fig. 8 show a very regular relationship between pore pressure change and strain under constant deviator stress. This suggests a con-

tinuing breakdown of structure, particularly along the plane of maximum shear stress, resulting in an increase in pore pressure. This phenomenon, perhaps a secondary consolidation effect, has been suggested before by Casagrande and Wilson (11) and by Bjerrum and Rosenqvist (26). If this interpretation is accepted, then it is apparent that the pore pressure at failure will be higher in a very slow test than in a rapid test simply because more time has been allowed for structural breakdown. Also it is known that remolding causes a strength decrease and so it is not surprising that, as strain rate decreases, the Mohr circles at failure become smaller (lower deviator stress) and shift to the left in the Mohr diagram (higher pore pressure).

In these tests, the two tendencies combine to give a slightly steeper failure envelope above the preconsolidation pressure at slow rates of strain. Below the preconsolidation pressure, the time effect on pore pressure is negligible but the strength decrease is very substantial. The remolding apparently destroys the effect on strength of preconsolidation. From tests on a normally-consolidated Norwegian clay, Bjerrum et al (21) give results which suggest a decreasing angle of shearing resistance, ϕ' , at slow rates of strain. This is an opposite tendency to that shown in this paper. It is noted from Fig. 2 that the failure stresses of the Norwegian soil are affected by rate of strain in the same way as the Canadian soil, but in Fig. 3 it is shown that the time effect on the pore pressure parameter A_f is greater in the normally-consolidated range ($\sigma_c = 4$ or 6) for the Canadian soil. This may be due simply to the greater sensitivity of the Canadian clay causing a greater shift to the left in the Mohr diagram.

At strains beyond failure, the pore water pressure in the consolidated-undrained tests continued to rise until it

approached the cell pressure. (Minor principal effective stresses reduced to 0.5 kg per sq cm in some cases.) At 6 per cent strain the pore-pressure parameter A ranged from about 1.1 to 1.5 depending on the cell pressure. Beyond failure, however, there appeared to be no relation between pore pressure and deviator stress. It is suggested therefore that for this type of soil the high values for pore-pressure parameter A which are measured after failure are meaningless.

In drained tests the minor principal effective stress remains constant while the major principal effective stress increases. Therefore during shear the area of the sample remains reasonably constant while the height decreases in proportion to the strain. This apparently causes general remolding and structural breakdown all through the sample, and although the water content decreases appreciably the loss of strength due to remolding is sufficient to result in a net reduction in shear strength. Drained test failure envelopes shown on Fig. 6 are quite unrealistic in representing the strength of this material since at strains of 10 or 20 per cent the material is quite different from its natural state.

Having eliminated the drained test as a useful method of evaluating this type of soil and shown the futility of interpreting test values at strain beyond failure, the problem of selecting an appropriate consolidated-undrained test envelope remains. This problem will not be solved by studying a few test values on an isolated sample of soil. It is probable however that some trends can be discerned which will aid in the judgment necessary in applying test results.

It is probable that for special "end of construction" problems (see for instance Bishop and Henkel (20)) experiences with the use of the $\phi = 0$ analysis also apply to effective stress analyses in this clay. The 5-min unconfined compression test

has found wide and satisfactory use in this case and therefore the equivalent envelope ($T_f = 0.1$ hr) is not unreasonable. More often, however, it is desirable to compute the "long-term stability" using effective stresses. In these cases it would appear, from these results, that little reliance should be put on the additional strength due to preconsolidation because this strength decreases substantially under slow strains such as might exist in long-term stability problems.

It is also questionable whether the higher strength at slow strain rate ($T_f = 10$ hr) in the normally consolidated range would exist in nature. It is so critically dependent on structural breakdown and probably indirectly on sample disturbance and method of consolidation in the test, that is, isotropic or anisotropic, that it would be better to ignore it. Perhaps the most reasonable "long-term" analysis should be based on a 1-hr test failure envelope through the origin.

One of the notable features of this sensitive clay is its association with flow slides. The laboratory tests reported in this paper may be of some assistance in understanding flow slide mechanism. It is apparent from the tests that pore pressures increase due to strain. In the triaxial test the absolute value of pore pressure seems to be limited only by the cell pressure and by a limiting value of principal stress ratio $\bar{\sigma}_1/\bar{\sigma}_3$ as $\bar{\sigma}_3$ approaches zero. In nature the pore pressure in a soil layer remolded at depth is limited only by the overburden pressure. Therefore once a failure plane is established the pore pressure on the plane will immediately develop to a value equal to the overburden pressure allowing the material to "flow" away.

CONCLUSIONS

Conclusions from tests reported in this paper must be qualified by the knowl-

edge that the tests describe only one block of a very heterogeneous soil deposit. The fundamental value of the results is increased by the uniformity of this specific sample, but the general conclusions need verification at shallower and deeper depths. From the tests the following conclusions are drawn:

1. The failure deviator stress decreases with increasing time to failure in accordance with a pattern established by others.
2. The ratio of pore pressure change during shear to deviator stress at failure

$$A_f = \frac{\Delta u_f}{(\sigma_1 - \sigma_3)_f}$$

increases significantly with time to failure and in a pattern depending on preconsolidation.

3. The pore pressure change during shear increases with increasing consolidation pressure. In addition there is a time effect which results in higher pore pressures as time to failure increases for specimens tested at pressures above the preconsolidation pressure.

4. Slow straining appears to destroy

the additional strength developed by preconsolidation.

5. The Mohr failure envelope is significantly influenced by testing time, and it is reasoned that the 1-hr test best represents critical field conditions and allows sufficient time for a satisfactory pore pressure equilibrium in the specimen.

6. Pore pressures are dependent on strain and may be considerably influenced by sample disturbance and isotropic consolidation.

7. Drained tests do not satisfactorily evaluate this sensitive clay.

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REFERENCES

- (1) Karl Terzaghi, "The Shearing Resistance of Saturated Soils and the Angle Between the Planes of Shearing," *Proceedings, First International Conference on Soil Mechanics and Foundation Engineering*, Cambridge, Mass., Vol. 1, pp. 54-56 (1936).
- (2) Karl Terzaghi, "Friction in Sand and Clay," *Engineering News-Record*, Vol. 95, No. 26, p. 1026 (1925).
- (3) L. Jurgenson, "The Shearing Resistance of Soils," *Journal*, Boston Soc. Civil Engrs., Vol. 21, No. 3, pp. 184-217 (1934).
- (4) L. Jurgenson, "The Shearing Resistance of Soils," *Journal*, Boston Soc. Civil Engrs., Vol. 21, No. 3, pp. 223-225 (1934). Discussion by A. Casagrande:
- (5) L. Rendulic, "Ein Grundgesetz der Tonmechanik und sein experimenteller Beweis," (The Fundamental Law of Clay Mechanics and the Experimental Proof), *Baugingenieur*, Vol. 18, pp. 459-467 (1937).
- (6) M. J. Hvorslev, "Ueber die Festigkeitseigenschaften gestoerter bindiger Boeden," (On the Physical Properties of Disturbed Cohesive Soils), *Ingeniorvidenskabelige, Skrifter A*, No. 45 (1937).
- (7) P. C. Rutledge, "Review of the Cooperative Triaxial Shear Research Program of the Corps of Engineers," Waterways Experiment Station, Vicksburg, Miss., April 1947.
- (8) A. W. Skempton, "Study of the Immediate Triaxial Test on Cohesive Soils," *Proceedings*, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 1, pp. 192-196 (1948).
- (9) A. W. Skempton, "The $\phi = 0$ Analysis of Stability and Its Theoretical Basis," *Pro-*

- ceedings*, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 1, pp. 72-78 (1948).
- (10) R. E. Gibson, "Experimental Determination of The True Cohesion and Angle of Internal Friction in Clays," *Proceedings*, Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. 1, pp. 126-130 (1953).
 - (11) A. Casagrande and S. D. Wilson, "Prestress Induced in Consolidated-quick Triaxial Tests," *Proceedings*, Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, Vol. 1, pp. 106-110 (1953).
 - (12) L. Bjerrum, "Theoretical and Experimental Investigations on Shear Strength of Soils," *Publication No. 5*, Norwegian Geotechnical Inst., (1954).
 - (13) A. W. Skempton and A. W. Bishop, "Soils," Chapter 10 of "Building Materials, Their Elasticity and Inelasticity," Amsterdam, North Holland Publishing Co. (1954).
 - (14) J. W. Hilf and H. J. Gibbs, "Triaxial Shear Tests Holding Effective Lateral Stress Constant," *Proceedings*, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, pp. 156-159 (1957).
 - (15) D. J. Henkel and A. W. Skempton, "A Landslide at Jackfield, Shropshire, in a Heavily Overconsolidated Clay," *Géotechnique*, Vol. 5, No. 2, pp. 131-137 (1955).
 - (16) D. J. Henkel, "Investigations of Two Long-term Failures in London Clay Slopes at Wood Green and Northold," *Proceedings*, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 2, pp. 315-320 (1957).
 - (17) R. A. Sevaldson, "The Slide in Lodalén, October 6th 1954," *Géotechnique*, Vol. 6, No. 4, pp. 167-182 (1956).
 - (18) L. Bjerrum and B. Kjaernsli, "Analysis of the Stability of Some Norwegian Natural Clay Slopes," *Géotechnique*, Vol. 7, No. 1, pp. 1-16 (1957).
 - (19) W. J. Eden and C. B. Crawford, "Geotechnical Properties of Leda Clay in the Ottawa Area," *Proceedings*, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, pp. 22-27 (1957).
 - (20) A. W. Bishop and D. J. Henkel, "The Triaxial Test," Edward Arnold, Ltd., London (1957).
 - (21) L. Bjerrum, N. Simons, and I. Torblaa, "The Effect of Time on the Shear Strength of a Soft Marine Clay," *Proceedings*, Brussels Conference on Earth Pressure Problems, Vol. 1, pp. 148-158 (1958).
 - (22) A. Casagrande and S. E. Wilson, "Effect of Rate of Loading on Strength of Clays and Shales at Constant Water Content," *Géotechnique*, Vol. 2, No. 3, pp. 251-263 (1951).
 - (23) A. W. Skempton, "The Pore-pressure Coefficients A and B," *Géotechnique*, Vol. 4, No. 4, pp. 143-147 (1954).
 - (24) D. J. Henkel, "The Effect of Overconsolidation on the Behavior of Clays during Shear," *Géotechnique*, Vol. 6, No. 4, pp. 139-150 (1956).
 - (25) T. William Lambe, "The Structure of Inorganic Soil," *Am. Soc. Civil Engrs.*, Vol. 79, Separate No. 315, Oct. 1953.
 - (26) L. Bjerrum and I. Th. Rosenqvist, "Some Experiments with Artificially Sedimented Clays," *Géotechnique*, Vol. 6, No. 3, pp. 1-13 (1956).

DISCUSSION

MR. RONALD C. HIRSCHFELD¹ (*by letter*).—Our knowledge of the pore pressure characteristics of sensitive clays is meager, despite the fact that pore pressures may play an especially critical role in the shearing of such soils. This paper is a worthwhile contribution on this subject, in addition to its principal contributions on the influence of rate of strain.

The discussor is in complete agreement with the author that decreasing rates of strain will probably produce larger pore pressure buildups. However, from an experimental point of view, one critical factor was omitted in the discussion of the test results. Water may leak from the triaxial chamber into the specimen, either through the membrane directly or through the bindings and drainage connections at the cap and base. A very small quantity of water leaking into the sample could produce a large buildup of pore pressure. Unless conclusive experimental evidence is included to prove that such leakage did not take place during the long-time tests, it is not possible to determine whether the buildup of pore pressure was caused by a time-dependent structural breakdown or by leakage.

The author states that the drained envelope cannot be used as the relation between shear strength and effective stress because of the large strains associated with the drained test. This seems a rather strong statement in view of our poor understanding of the strength characteristics of soil *in situ*. For example, it

has been found in laboratory tests that the stress-deformation characteristics of many clays are dependent on the rate of strain, the strain at failure increasing with decreasing rate of strain for most of the clays tested. Although it does not seem plausible that a sensitive clay could be strained very far at constant water content without failing, it is quite possible that the failure strains *in situ* might be much larger than those which are found in the various types of laboratory test in which the soil is sheared at constant water content. Furthermore, the drained envelope which the author shows in Fig. 6 is conservative compared to the undrained envelopes. We must use the conservative envelope for design purposes unless convincing proof can be shown why a less conservative envelope would be satisfactory.

Two minor comments should be made concerning the terminology. It is not always clear in the text whether the author is referring to a strength envelope in terms of "effective" stress or "total" stress. Also, the term "undrained" can be ambiguous when used alone. It would be clearer to specify "consolidated-undrained" or "unconsolidated-undrained" to avoid any confusion.

MR. T. CAMERON KENNEY² (*by letter*).—Recently two papers were published concerning the influence of rate of strain on the effective stresses induced in sensitive clays; Bjerrum, Simons and Torblaa (1)³ tested a normally consolidated ma-

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³ The boldface numbers in parentheses refer to the list of references appended to this discussion.

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rine clay having a sensitivity varying from 1.5 to 7.0 and the author tested a lightly overconsolidated marine clay having a sensitivity greater than 20 (Table III). In both cases, the testing programs consisted of a series of consolidated drained and consolidated undrained triaxial compression tests performed at different rates of strain. In most respects the major conclusions of both papers were the same; that is, in undrained tests the magnitude of the induced pore pressures increased as the rate of strain decreased and the magni-

soil at the following three stages during a compression test:

(a) During isotropic consolidation when the stresses all round the sample are equal.

(b) At the point during the test when the shear strength parameters of the soil attain their maximum values.

(c) At any stage of the test between (a) and (b).

In the first case, no shear stresses are applied to the sample, and, therefore, the shear strength parameters of the soil are not mobilized; in other words, the mo-

TABLE III.

Clay	Reference	Natural Water Content (w_n)	Liquid Limit (w_l)	Plasticity Index (I_p)	Activity	Natural Sensitivity (S_t)
Cornwall.....	...	31	28	14	0.58	6
Wallaceburg.....	...	40	41	27	0.57	4
Allanburg.....	...	22	29	15	...	2.5
Bersimis.....	...	28	33	13	0.47	...
Ottawa.....	Crawford (2)	67	65	39	0.65	20
Saco River.....	Taylor (4)	...	46	30	...	10
Boston.....	Lambe (5)	...	...	...	...	8
Drammen.....	Kjaernsli (6)	33	36	16	...	5
Fornebu.....	Bjerrum et al (1)	35 to 55	36 to 59	15 to 34	0.34 to 0.61	1.5 to 7.0

tude of the maximum deviator stress decreased as the rate of strain decreased. However, with respect to the apparent shear strength parameters c' and ϕ' at failure, the results of the two papers were not fully in agreement. Bjerrum et al found in the case of drained tests that c' and ϕ' remained effectively constant for different rates of strain, and in the case of undrained test c' remained effectively constant and ϕ' decreased with decreasing rates of strain. Crawford, on the other hand, found for the case of undrained tests that c' decreased and ϕ' increased with decreasing rates of strain.

It is proposed to suggest an explanation for these contradictory test results by introducing the concept of mobilization of the shear strength parameters.

Let us consider a sample of saturated

bilized values of true cohesion and true angle of internal friction are equal to zero.

In the second case, the shear strength parameters are mobilized to their fullest extent.

In the third case, the shear strength parameters are only partially mobilized to balance the externally applied shear stresses.

This concept of shear strength parameter mobilization can be expressed mathematically by considering the geometry of the Mohr stress circle. The deviator stress acting on an element of soil may be expressed in the following manner:

$$(\sigma_1 - \sigma_3)$$
$$= 2 \left\{ \frac{c'_{mo} \cdot \cos \phi'_{mo} + (\sigma_3 - u) \cdot \sin \phi'_{mo}}{1 - \sin \phi'_{mo}} \right\} \dots (1)$$

FIG. 11.—Results of Drained and Undrained Triaxial Compression Tests on a Normally Consolidated Clay.

where:

σ_1, σ_3 denote the total major and minor principal stresses respectively,
 $\sigma_1 - \sigma_3$ denotes the compressive stress,
 c'_{mo}, ϕ'_{mo} denote the mobilized values of the shear strength parameters, apparent cohesion and angle of shearing resistance, and u denotes the pore water pressure.

From the above equation, it is seen that in a standard drained compression

test where $\sigma_3 - u$ remains constant, the degree of mobilization of the shear strength parameters depends upon the magnitude of the applied shear stresses;⁴ the larger the applied shear stresses, the more fully mobilized the shear strength parameters become. It is also seen that the sample exhibits its maximum com-

⁴ Rowe (3) has postulated that the degree of mobilization of true cohesion also is dependent upon the rate of strain.

Ottawa (<i>N</i>)	<i>U</i>	39	53	30—	8—	31+	5+	3.2—	24+	39	14	25	2	20	0.81—	5.0	16.3
	<i>U</i>	85	42	71—	25—	46+	5+			96	46	39	2	17	0.70—		
	<i>D</i>	28	...	63+	28	...	20+			...	...	...	...	...	...		
	<i>D</i>	40	...	94+	40	...	20+			...	...	...	...	...	...		
	<i>D</i>	57	...	148+	57	...	20+			...	...	...	...	...	...		
Saco River (<i>N</i>)	<i>U</i>	74	...	60—	18—	56+	6+	0	33+	85	37	37	2	23	0.67—	0	23
Boston (<i>N</i>)	<i>U</i>	55	32	32	9	46	14	0	35	60	23	32	2	26	0.70	0	26
Drammen (<i>N</i>)	<i>U</i>	21	...	24—	9—	13+	3+	0	29+	25	9	12	2	28	0.96—	1.4	23
	<i>U</i>	43	...	43—	14—	29+	4+			48	19	24	3	26	0.87—		
	<i>U</i>	85	...	94—	33—	52+	5+			105	44	41	3	24	0.81—		
Fornebu (<i>N</i>)	<i>U</i>									Time to failure: 0.5 to 12 hr						0.6	30
	<i>U</i>									Time to failure: 29 to 460 hr						0.6	26
	<i>D</i>	Time to failure: 20 to 700 hr							0.6	30							

N = Natural sample*R* = Remolded sample*U* = Undrained test*D* = Drained test σ_c = Consolidation pressure w_f = Water content at failure ϵ = Sample strain

$$M = \frac{\tan \phi'_{mo}}{\tan \phi'_{max}}$$

NOTE.—All stresses are measured in psi

FIG. 12.—Relationship Between the Angles of Shearing Resistance of Normally Consolidated Clays Obtained from Drained and Undrained Compression Tests.

test, it is seen from Eq 1 that the degree of mobilization of the shear strength parameters is dependent not only upon the magnitude of the applied shear stresses but also upon the magnitude of the induced pore pressures. It follows that maximum compressive strength which a sample exhibits in an undrained state is dependent upon the magnitude of the induced pore pressures and the degree of mobilization of the shear strength parameters. Therefore, the shear strength parameters do not have to be fully mobilized when the compressive strength of a sample reaches its maximum value, and, conversely, the compressive strength of the sample does not have to reach its maximum value when the shear strength parameters are fully mobilized.

FIG. 13.—Relationship Between Natural Sensitivity and Degree of Mobilization of ϕ' at $(\sigma_1 - \sigma_3)_{\max}$.

pressive strength when the shear strength parameters are fully mobilized. However, in the standard undrained test where $(\sigma_3 - u)$ generally varied throughout the

The concept of mobilization of the shear strength parameters is shown graphically in the accompanying Fig. 11 by the results of consolidated drained

and consolidated undrained compression tests on natural samples and on remolded natural samples of a normally consolidated silty clay. The test data in Fig. 11 is presented in the form of stress paths where measured values of $\bar{\sigma}_1$ and $\bar{\sigma}_3$ are plotted for various values of strain throughout any test. Any point on a stress path, whose position is defined by particular values of $\bar{\sigma}_1$ and $\bar{\sigma}_3$, can be represented by a Mohr stress circle and a line joining two points on different stress paths can be related to the tangent of the corresponding stress circles in the following manner:

$$\sin \phi'_{mo} = \frac{\left(\frac{\Delta \bar{\sigma}_1}{\Delta \bar{\sigma}_3} - 1 \right)}{\left(\frac{\Delta \bar{\sigma}_1}{\Delta \bar{\sigma}_3} + 1 \right)} \dots \dots (2)$$

$$c'_{mo} = \frac{\alpha}{2} \{ \sec \phi'_{mo} - \tan \phi'_{mo} \} \dots (3)$$

where α denotes the $\bar{\sigma}_1$ intercept of the line joining the two stress points produced back to the condition of $\bar{\sigma}_3 = 0$.

The envelope of the stress paths defines the condition of maximum mobilization of the shear strength parameters; if the direction of a stress path is towards the envelope, the shear strength parameters of the soil are becoming more fully mobilized, while if the direction of the stress path is away from the envelope, the shear strength parameters are becoming demobilized.

From the limited available data (Table IV) it is noted that the envelope of the stress paths is essentially common to both drained and undrained tests. In Fig. 12, the angle of shearing resistance determined from drained tests, ϕ_D , is plotted against the angle of shearing resistance determined from undrained tests for the conditions of maximum deviator stress, ϕ'_{mo} , and maximum mobilization of shear strength parameters, ϕ'_{max} . It is shown that ϕ_D and ϕ'_{max} are very nearly equal, and this suggests that these param-

eters are dependent solely upon the fundamental shear strength parameters of the soil, true cohesion, and true angle of internal friction.

In Table IV, a summary of the results of compression tests on several different soils is given. For the condition of maximum deviator stress, the values of mobilized angle of shearing resistance, ϕ'_{mo} , and degree of mobilization, $\frac{\tan \phi'_{mo}}{\tan \phi'_{max}}$, have been calculated; in those cases where $c'_{mo} > 0$, it was assumed for simplicity that $c'_{mo} = c'_{max}$. From the results of these calculations it is seen that the degree of mobilization for the condition of maximum deviator stress is high when the test consolidation pressure is low, and is lower when the consolidation pressure is higher. This is especially evident for clays of high natural sensitivity, as shown in Fig. 13, where the degree of mobilization has been plotted against natural sensitivity. This suggests that the manner in which the shear strength parameters are mobilized is dependent upon the stability of the soil structure; within the preconsolidation stress range of sensitive soils, the soil structure is stiff and brittle while in the normally consolidated stress range the soil exhibits a plastic and compressible structure.

It is desired to make the following points:

1. The maximum values of the shear strength parameters, c'_{max} and ϕ'_{max} , found from drained tests and undrained tests on the same soil, are very nearly equal (Fig. 12) and this suggests that these values are dependent only upon the fundamental properties of the soil, true cohesion, and true internal friction.

2. There is some evidence from the tests reported by Bjerrum et al (1) that, in the case of drained tests, the rate of strain has little effect upon the maximum values of the shear strength parameters.

The explanation suggested for this phenomenon was that the fundamental shear strength parameters, true cohesion, and true angle of internal friction possibly decrease with decreasing rates of strain and that this effect is compensated by an increase in the fundamental shear strength parameters due to secondary consolidation.

It would be extremely interesting if the author could report on the effect of rate of strain on the maximum values of the shear strength parameters determined from the undrained tests on Ottawa clay. As in the drained test, it might be expected that a decrease in true cohesion and angle of internal friction might take place, but this might be compensated to some extent in the undrained test by increased values of induced pore pressures.

3. If the maximum deviator stress failure criterion is used to interpret the results of undrained tests, it is shown in Table IV and Fig. 13 that the value of the mobilized angle of shearing resistance is dependent upon the stability of the soil structure with respect to the applied stresses. For this reason, the results of any single test might be expected to be dependent upon the type of consolidation (isotropic, anisotropic), the magnitude of the consolidation stresses with respect to the preconsolidation stresses, the natural sensitivity of the soil, the degree of sample disturbance, the type of shear test (cylindrical compression, plane strain), and the rate of strain.

The usual method of determining the apparent shear strength parameters is to consolidate several samples at different pressures and to determine the effective stresses for the condition when the deviator stress is a maximum. If the degree of mobilization of the shear strength parameters is high and is approximately equal in each individual test, realistic values of c' and ϕ' will be obtained. This occurs in the case of insensitive soils.

However, if the degree of mobilization of the shear strength parameters is widely different in each individual test, it is possible that unrealistic values of c' and ϕ' will be obtained. This occurs in the case of very sensitive soils such as Ottawa clay where, for one series of tests reported, Crawford obtained a negative value of c' .

From the above discussion, the following question is raised of the type of results which can be obtained from undrained tests when use is made of the maximum deviator stress criterion, and this question is particularly important when dealing with sensitive clays: Of what value is this failure criterion for determining the effective stress shear strength properties of a soil from the results of undrained tests, and under what conditions can they be applied?

At maximum deviator stress, a sample exhibits its maximum strength with respect to the applied total stresses and, therefore, we may say that the maximum deviator stress failure criterion is a satisfactory "total stress" failure criterion. With respect to effective stresses, however, it has been shown that the mobilized values of the shear strength parameters at maximum deviator stress are dependent upon such factors as the relative magnitude of the induced pore pressures, and this means that these parameters are only applicable where the external stresses and the pore pressures are equal to those measured in the test. Surely, this is only a round-about manner of applying a total stresses analysis. The application of an effective stress analysis to a problem where the pore pressures are unable to dissipate by drainage is only possible if an accurate estimation of the induced pore pressures can be made and if the mobilized shear strength parameter values can be predicted for the stress-strain conditions involved.

failure were too large. In nature, however, large strains take place in sensitive soils under the conditions of no excess pore pressures (drained conditions), and the fact that it may be difficult to duplicate in the laboratory does not mean that this condition cannot be studied. If the only difficulty encountered in this problem is that of having to produce large shear strains, the suggestion is made to use some type of ring shear apparatus.

MR. C. B. CRAWFORD (*author's closure*).—In his discussion, Mr. Hirschfeld has drawn attention to the possibility of water leakage from the triaxial chamber into the specimen. There is little doubt

TABLE V.—COMPARISON OF SHEAR STRENGTH PARAMETERS AT MAXIMUM DEVIATOR STRESS AND AT 6 PER CENT STRAIN.

Approximate Rate of Strain, per cent per hr	at $(\sigma_1 - \sigma_3)_f$		at 6 per cent strain	
	c' , kg per sq cm	ϕ'	C' , kg per sq cm	ϕ'
15.....	+0.25	17.5	+0.22	23.5
1.5.....	+0.1	20	+0.12	28
0.15.....	-0.1	23	-0.06	37

that water can pass through the membrane. This has been demonstrated in standard vapor permeability apparatus and by application of osmotic pressures. It has not been possible, however, to measure flow in a mock triaxial test. In earlier tests, an attempt was made to trace leaks using an ink solution in the triaxial cell. This revealed no leaks through the membranes but did show stains just beneath the top cap which was made of lucite. Subsequent tests showed this could be eliminated by smearing "Apiezon grease" on the cap before positioning the membrane.

In view of the difficulties of assessing leakage by mock tests, the author has relied on observations during each test to indicate whether the result is being af-

fected by leakage. Serious leakage is always indicated by a continuing flow of water out of the specimen at the end of consolidation. This is confirmed by a difference between measured and computed volume after the test. Less serious but still significant leakage will be indicated by slow changes in the pore water pressure after drainage has been cut off but before shearing is started. Neither of these indications were evident in any of the tests reported. Furthermore the shapes of the pore pressure - strain curves were similar for tests run at rates of strain

FIG. 15.—Undisturbed and Remolded Leda Clay at Natural Water Content (65 per cent).

differing by a factor of 100. This could not happen if significant leakage was occurring. Finally the pore pressure changes with time shown in Figs. 7 and 8, are, in the author's opinion, convincing proof that the pore pressure changes are not time dependent as they would be under the influence of leakage. Nevertheless, the possibility of leakage of water to the specimen must continue to be a possible source of error in the triaxial test and further research into the matter is more than justified.

Mr. Kenney has raised a most important question—What is the criterion of failure in a compression test on sensitive clay? In his discussion he develops

a concept of shear strength parameter mobilization and shows that with increasing sensitivity there is an increasing discrepancy between the maximum angle of shearing resistance⁵ and the angle of shearing resistance which is mobilized at maximum deviator stress. Further he shows that the maximum angle of shearing resistance is relatively independent of method of test (drained or undrained) and of the nature of the specimens (natural or remolded). There is substantial disagreement therefore between the conclusions stated in the paper and those proposed in the discussion. The reason for the disagreement is a difference in interpreting failure in the specimen.

It should be pointed out that the two undrained tests on Ottawa (Leda) clay quoted by Mr. Kenney in Table IV were on the only two specimens for which (owing to space limitations) detailed stress-strain information was given. Unfortunately, one specimen (83-27-3) was strained very slowly and the other (83-27-12) was strained quite quickly. Furthermore, the former test appeared to be influenced by preconsolidation while the latter was not. These two reasons contributed to the rather low $\phi'_{mo} = 16.3$ deg, which is quoted. To assist in the further consideration of the test results Fig. 14 is now added showing the effective stresses developed in one group of consolidated-undrained tests which were strained at a rate of approximately 2 per cent per hr. Values are plotted at strains of 0, 0.25, 0.5, 1, 2, 4, and 8 per cent. Maximum deviator stress is indicated by an asterisk. On the same plot the three consolidated-drained test results are shown up to the maximum strain of 20 per cent.

As requested in the discussion further

⁵ In this discussion parameters are always quoted in terms of effective stresses.

information on the effect of rate of strain on the maximum values of the shear strength parameters determined from the undrained tests on "Ottawa clay" is given in Table V. Although a value of ϕ' as high as 37 deg is quoted in the table this is not quite the maximum possible value but too few of the tests were carried past 6 per cent strain to give further reliable results. It will be noted from these results that ϕ' continues to increase with increasing strain past maximum deviator stress and at high strains the effect of rate of strain is even greater than at low strains. In the author's view, the negative values of c' are not significant. Theoretically the extended envelope should pass through the origin but the slightest error in testing can displace the cohesion intercept significantly.

The Mohr envelope at 6 per cent strain for consolidated-undrained tests carried out at a rate of strain of approximately 15 per cent per hr ($T_f = 0.1$ hr) is shown in Fig. 6. At slower rates of strain higher envelopes were obtained. It is apparent, then, that if large strains were accepted in the undrained tests and maximum values plotted the shear strength parameters quoted in the paper would be in reasonable agreement with those quoted by Mr. Kenney. Agreement of the drained test results is less certain since even at 20 per cent strain the deviator stress was still increasing (Fig. 10) and so failure in the usual sense was not reached.

Returning to the question of failure criterion, it is the author's belief that the amount of strain cannot be ignored in assessing test failure conditions in a brittle, sensitive clay. For insensitive clays, the failure envelopes for drained and undrained tests should, theoretically, agree and much evidence has been published in support of this view. For a fairly sensitive clay, Mr. Kenney has shown (Fig. 11) reasonable agreement between

drained and undrained test failure envelopes if samples are strained sufficiently to allow maximum mobilization of shear strength parameters.

But, in permitting large strains the specimen is seriously remolded. In the drained test, where volume change is allowed to occur, the remolding probably occurs throughout the specimen. In the undrained test the strain and remolding are probably concentrated around the plane of maximum shear stress. This results in a redistribution of pore water as shown in Fig. 9 and at the failure plane a condition of partial drainage exists. This must result in a strength increase and a shift of the failure plane to a plane of lower shear stresses.

Basically the author disagrees with the contention that large shearing strains occur in sensitive clays in nature. While creep is a common phenomenon in insensitive soils, the failure of sensitive soils is sudden with little or no warning. Recently a flow-type landslide occurred in Leda clay at the site of the little village of Nicolet, about 75 miles north-east of Montreal (7) and although nearly a quarter-million yards of sensitive clay flowed out of a crater in the heart of the village in about 5 min, no previous sign of creep had been observed.

There is no comparison between the shear strength properties of undisturbed and remolded sensitive clay. Remolding results from deformation. Figure 15 illustrates the effect of complete remolding. Here a slice has been removed from an undisturbed specimen, remolded, and deposited beside the specimen. It is suggested that during the triaxial test the

ability of the specimen to resist the cell pressure is decreased by the remolding that occurs because of strain. In the undrained test this results in a transfer of stress from the soil structure to the pore water. The increase in pore water decreases the effective stress on the potential failure plane and the shearing resistance should decrease. There is evidence, however (Fig. 9), that while the pore pressure is changing, water is moving away from the potential failure plane causing an increase in strength on the critical plane. Under increasing strain the net result of the two effects is an increasing angle of shearing resistance. It is the author's contention that any field condition in which this type of soil deforms under fully drained conditions results in increasing stability and that triaxial tests carried to large strains have no practical significance.

It is quite evident that for a sensitive clay the choice of "failure criterion" has much more influence on the shear strength parameters than any other single factor. In the original analysis much effort was made to establish a more satisfactory criterion than maximum deviator stress. Other criteria such as the variation in principal effective stress ratio were studied but none gave a noncontroversial failure. Finally it was concluded that large strains could not be tolerated in evaluation of failure and maximum deviator stress was accepted as the most realistic failure criterion.

The author is grateful for the sound, constructive criticism offered by Messrs. Hirschfeld and Kenney.

REFERENCES

- (1) L. Bjerrum, N. Simons, and I. Torblaa, "The Effect of Time on the Shear Strength of a Soft Marine Clay," *Proceedings*, Brussels Conference on Earth Pressure Problems, Vol. 1, pp. 148-158, (1958).
- (2) P. W. Rowe, " $C_e = 0$ Hypothesis for Normally Loaded Clays at Equilibrium," *Proceedings*, Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, pp. 189-192, (1957).
- (3) D. W. Taylor, "Shearing Strength Determinations by Undrained Cylindrical Compression Tests with Pore Pressure Measurements," *Proceedings*, Second International Conference on Soil Mechanics and Foundation Engineering, Vol. 5, pp. 45-49, (1948).
- (4) T. W. Lambe, "Soil Testing for Engineers," John Wiley and Sons, Inc., New York, N. Y., pp. 135-137, (1951).
- (5) B. Kjaernsli, "Stabilitetsundersøkelse av Elvebredden på Bragernes i Drammen," *Publication No. 18*, Norwegian Geotechnical Inst., (1956).
- (6) C. B. Crawford and W. J. Eden, "The Nicolet Landslide of November 1955" (*to be published in Engineering Geology, Case Histories*, The Geological Society of America No. 4).

A list of all publications of the Division of Building Research is available and may be obtained from the Publications Section, Division of Building Research, National Research Council, Ottawa, Canada.