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BY

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THE INFLUENCE OF STRAIN ON SHEARING RESISTANCE OF SENSITIVE CLAY*

BY CARL B. CRAWFORD[1]

SYNOPSIS

The effective stress shear strength parameters of a sensitive clay are subject to a wide degree of interpretation because the pore water pressure does not reach equilibrium at maximum stress. A simple method of obtaining shear strength in terms of fundamental parameters, believed to be equal to Hvorslev's true parameters, is described. A revised interpretation of true cohesion for undrained tests is developed. Water content variation during the test is discussed, and the mechanism of friction and cohesion is related to degree of deformation.

In accordance with the fundamental principle of effective stress, the shearing resistance on any plane in a soil depends not on the total stress but on the effective stress on the plane (1).[2] The effective stress is the so-called grain-to-grain stress found by subtracting the pore water pressure from the total stress. It is clear, therefore, that values for shear strength measured or applied without regard to pore water pressure may be subject to considerable error. Knowledge of this fact has created a great deal of interest in the measurement of shearing resistance in terms of effective stresses and the use of effective stresses in field application.

Although the use of effective stress analyses is a great advance over the total stress method, it is still an empirical approach dependent upon the accuracy of arbitrary tests, usually on very small specimens, in representing field conditions. Much of the study and application of effective stresses has been on remolded or compacted soils in which stress history and sensitivity may have little influence. Because extrapolation of this experience into the realm of natural soils may not be justified, a special study of the shear strength properties of undisturbed specimens of a sensitive clay is being carried out by the National Research Council of Canada, Division of Building Research. This paper describes a series of tests in which the effective stress shear strength parameters are subject to a wide range of interpretation, and an effort is made therefore to interpret the results in terms of fundamental parameters.

SHEAR STRENGTH PARAMETERS

Thirty years ago Terzaghi (2) showed that if two identical specimens of clay are brought to the same water content—one by simple compression, the other by additional compression followed by rebound—the former will be considerably stronger than the latter.

This important discovery that the stress history of a clay would have a

* Presented at the Sixty-fourth Annual Meeting of the Society, June 25–30, 1961.

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

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considerable influence on its strength was followed by the classical work of Hvorslev (3) who systematically traced the variation in the shearing resistance of remolded clay as it was consolidated, rebounded, and recompressed. He was able, by relating shearing resistance to the "equivalent consolidation pressure" (that is, the pressure on the virgin consolidation curve corresponding to the void ratio at failure), to obtain an expression for shear strength in the form of two parameters which were unaffected by stress history. These two parameters, called the "effective angle of internal friction" and the "effective cohesion," represent a frictional component of shearing resistance which depends only on the effective stress on the plane of failure, and a cohesive component which depends only on the void ratio in the plane of failure. Although Hvorslev still prefers the original terms (4), they have become widely known as the "true angle of internal friction" and the "true cohesion" to distinguish them from empirical parameters in terms of effective stresses (5–7).

The measurement of the Hvorslev parameters is usually such an involved procedure, requiring a good supply of uniform material, that it has been carried out for only a very few natural soils. This is why engineers have continued to rely on empirical parameters determined in terms of total applied stress or in terms of applied effective stress in which a relationship is established between shearing resistance and effective stress over a wide range of water contents. The values obtained in terms of total stress are applicable only if no drainage occurs in the test or in the field problem. The values in terms of effective stress can account for long-term pore pressure changes, but if a substantial proportion of the shearing resistance of a soil actually consists of true cohesion the direct application of empirical parameters in terms of effective stresses may be of limited accuracy. In other words, as the difference between the Hvorslev true angle of internal friction and the angle of shearing resistance in terms of effective stress increases, the departure from reality becomes of greater significance.

SENSITIVE CLAYS

Sensitive clays present a special problem in the determination of effective stress parameters because the pore water pressure induced by shearing continues to increase after the maximum stress has been applied and the selection of the point of failure is controversial. This has led to a consideration of progressive shear strength parameter mobilization (8,9) and of failure criteria other than a maximum deviator stress. In a discussion submitted to the recent ASCE Research Conference on Shear Strength of Cohesive Soils (10), the author presented test results showing that for a sensitive marine clay the computed angle of shearing resistance, ϕ', increased with strain and was subject to variation of interpretation from 22 to 34 deg. Osterman (11) reported similar test results on some Swedish clays in which "angles of up to 41 deg were obtained for very soft clays, which cannot reasonably be correct." Although not a great deal of information is available, the variety of soils in which pore pressures continue to increase after the maximum deviator stress has been applied may be quite extensive. Parry (12) reports this phenomenon for the insensitive London clay.

PROPERTIES OF THE SOIL

The tests reported in this paper were made on samples of a late glacial marine clay called "Leda" clay. The properties of this clay, which occurs extensively in eastern Canada, have recently been summarized (13). The primary set of tests were performed on specimens cut from undisturbed block samples obtained from a small tunnel at a depth of 33 ft. The second set of tests are from a previously reported study of strain rate influences (14) and are quoted in support of the conclusions drawn from the primary tests.

TABLE I.—PROPERTIES OF THE SOIL.

Soil Properties	Specimens 96-4	Specimens 83-27
Depth, ft	33	16
Elevation, ft	222	234
Natural water content, per cent	58	67
Liquid limit, per cent	53	65
Plastic limit, per cent	25	26
Specific gravity	2.80	2.80
Clay size, per cent	62	60
Salt in pore water, gm per l	2.0	0.5
Field vane strength, kg per sq cm	0.55	0.64
Sensitivity (approx)	50	20
Preconsolidation pressure, kg per sq cm	2.4	2.3

To reduce the scatter in test results, special efforts were made to obtain samples of nearly uniform water content, although this is not a general feature of the clay. Further, the samples were obtained from a depth believed to be below surface weathering effects. The characteristics of the two sets of samples, which were located about three miles apart in the City of Ottawa, are summarized in Table I.

TEST METHODS

Consolidated undrained triaxial tests were performed on specimens 10 sq cm in area and 8 cm high, wrapped in prepared

TABLE II.—TEST RESULTS.

Series[a]	Specimen	$\bar{\sigma}_c$[b]	$(\bar{\sigma}_1 - \bar{\sigma}_3)_f$[c]	T_f,[d] hr	Average Water Content, per cent: Initial	Average Water Content, per cent: Failure	Water Content of Slices, per cent: 1	2	3	4	5
A	96-4-1	2	1.26	1.0	59.4	51.7	...	...	...	...	...
	2	3	1.81	1.3	60.7	47.1	46.2	46.7	47.0	47.3	47.5
	3	4	2.55	1.9	59.0	41.8	...	...	...	...	...
	4	6	3.84	1.9	59.0	37.3	36.0	36.4	36.7	37.4	37.7
B	5	2	1.29	1.2	55.0	49.5	48.0	48.8	49.9	51.0	...
	6	3	1.84	1.3	57.3	45.6	44.6	45.2	45.5	45.0	44.9
	7	4	2.33	1.6	55.0	40.9	39.7	40.3	40.5	40.9	41.1
	8	6	3.82	2.2	60.6	38.9	37.6	38.1	38.7	38.9	...
C	9	2	1.23	6.2	58.0	49.7	47.9	48.4	49.2	49.9	49.5
	10	3	1.88	6.4	55.7	44.2	43.5	43.9	44.4	44.6	44.4
	11	4	2.41	6.5	58.3	41.8	40.5	41.1	40.9	42.2	42.8
	12	6	3.75	7.1	58.2	38.4	37.7	37.8	38.0	38.0	...
D	17	6	3.96		58.4	37.2	...	...	...	...	...
	18	6	4.18		56.4	35.2	...	...	...	...	...
Averages for sliced specimens						43.3	42.1	42.7	43.1	43.5	44.0

[a] Series A—consolidated-undrained tests with back pressure,
B—consolidated-undrained tests without back pressure,
C—consolidated-undrained tests with back pressure, deviator stress held constant for 5 hr, and
D—consolidated-undrained stage tests with back pressure.

[b] $\bar{\sigma}_c$ = cell pressure—back pressure, kg per sq cm.

[c] $(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ = deviator stress at failure kg per sq cm

[d] T_f = time to failure, hr.

filter paper drains as described by Bishop and Henkel (15) and enclosed in a single thin rubber membrane (0.01 cm thick). In series A, C, and D, cell pressure was then increased until small increments in cell pressure created equal increases in pore water pressure. This always occurred before the cell pressure reached 0.5 kg per sq cm. Cell pressure was then increased to 2.5 kg per sq cm while a back pressure of 0.5 kg per sq cm was maintained on the pore water, using equipment previously described (16). Drainage was allowed for one day. For tests at higher cell pressures, daily increments of 1 kg per sq cm were applied, allowing at least two days for drainage under the last increment. In series B, the specimens were consolidated without any back pressure, but just before shearing, the cell pressure and pore water pressure were increased simultaneously by 1 kg per sq cm. After consolidation was complete, the drainage was discontinued, and the specimens were strained by axial loading at a rate of about 2 per cent per hr while pore water pressure was measured at the bottom plate.

In series C, the deviator stress was held constant at about one third of its maximum value for a period of 5 or 6 hr. During this period, the specimens continued to deform, and pore pressures increased slowly. Controlled axial strain was then continued by further axial loading.

The two specimens of series D were subjected to multiple-stage tests. This technique has been described recently by Kenney and Watson (17). In these tests, a back pressure was applied to the pore water during consolidation and shear. Consolidation pressures of 2, 3, 4, and 6 kg per sq cm were used in successive stages. After each consolidation stage, the dimensions of the specimen were computed, and an axial load was applied by controlled straining without drainage until a maximum deviator stress was reached.

TEST RESULTS

Test results are summarized in Table II. The relationship between strain and deviator stress and pore water pressure

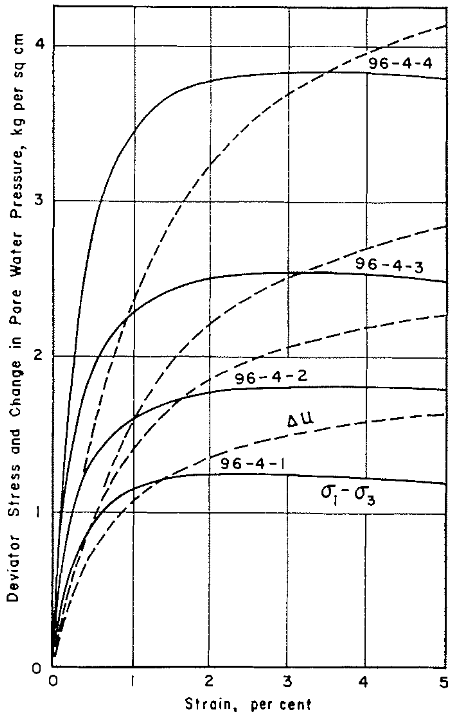

FIG. 1.—Typical Test Results.

for the four specimens of series A is shown in Fig. 1. Similar results were obtained for the other tests, but the pore pressure measurements for series B (consolidated without back pressure) appeared to be less reliable. For specimen 96-4-7, in particular, the measured pore pressure was low and not consistent with the other tests. In series C, about 0.2 per cent strain occurred during the period of constant deviator stress, and pore pressure increased in proportion to strain as shown in Fig. 2. Specimen 96-4-10 was inadvertently stressed to 46 per cent of failure and has therefore greater pore pressures than normal.

and the average variation is shown in Fig. 3. Slice 1 is the slice enclosing the failure plane. The position of the other slices is shown in the figure. The average water

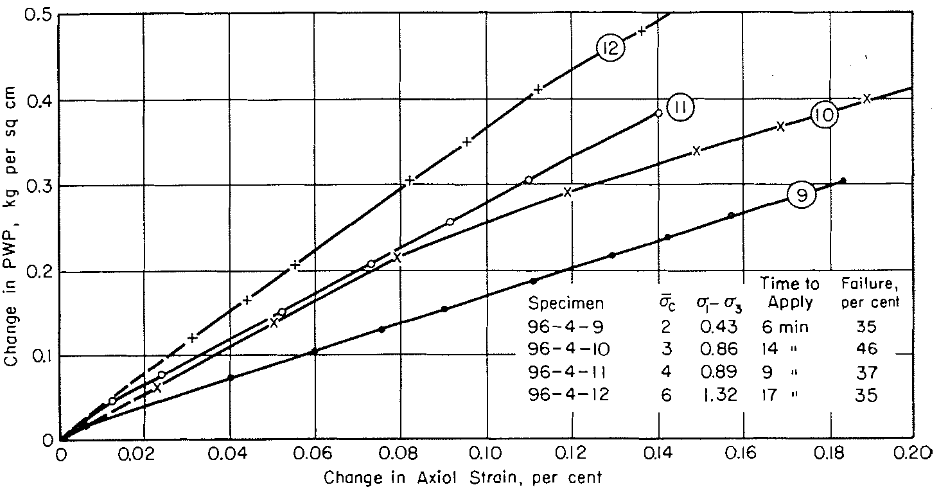

FIG. 2.—Relation Between Pore Pressure and Strain at Constant Stress.

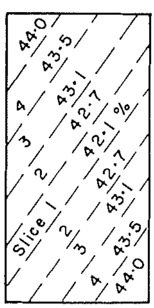

FIG. 3.—Average Water Content of 10 Specimens After Shearing.

Ten of the specimens were removed as quickly as possible from the triaxial chamber and divided into slices parallel to the failure plane in order to measure water content variation between slices. Specimens in which the failure plane was not clearly visible were compressed rapidly in unconfined compression to establish the failure plane. The water content of each slice is given in Table II, content of the ten specimens at failure is 43.3 per cent. The average water content of the slice enclosing the failure plane is 42.1 per cent. The water content of all of the specimens decreased consistently toward the failure plane, averaging 44 per cent at the extremities to 42.1 per cent at the failure plane.

Two specimens from an adjacent block sample were consolidated under one loading increment of 6 kg per sq cm and then removed from the cell quickly without shearing. End slices were removed, and the specimens were divided into upper, middle, and lower sections. Each section was trimmed concentrically into an outer, intermediate, and central portion. Water contents of each portion were determined. These tests revealed a pattern of water content that suggests different degrees of consolidation throughout the specimen. The tests show the outer shell to have been about 1 per cent dryer than the central portion (Fig.

4). Most of slice 1, obtained from sheared specimens, came from the inner portion of the sample, while most of slice 4 or 5, for instance, came from the outer shell, and it is therefore reasoned that slice 1 was wetter than average before shearing. Since it was dryer than average after shearing, this suggests an even greater movement of water during shear than is indicated by measurements on slices parallel to the failure plane.

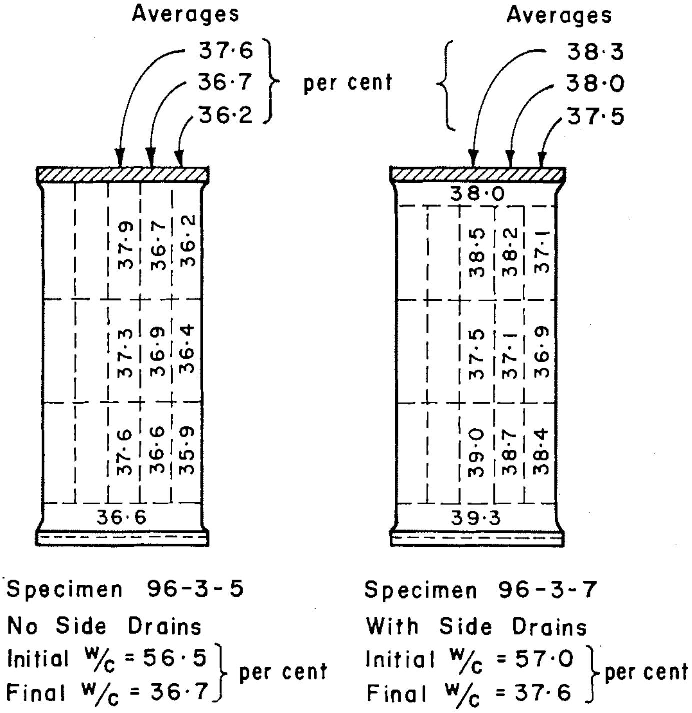

FIG. 4.—Water Content Variation After Triaxial Consolidation.

DISCUSSION OF TEST RESULTS

The fact that the computed angle of shearing resistance, ϕ', of a sensitive clay is subject to a wide interpretation is disturbing. Consider specimen 96-4-4, consolidated under an all-round pressure of 6 kg per sq cm. Since this pressure is well above the natural preconsolidation pressure of the soil, it may be considered to be normally consolidated with $c' = 0$. At 1 per cent strain, when 90 per cent of the maximum deviator stress had been applied, the computed ϕ' was 19 deg, at 95 per cent it was 21 deg, and at 100 per cent it had risen to 26 deg, at a strain of nearly 3 per cent. At approximately 10 per cent strain, the maximum principal effective stress ratio, $\bar{\sigma}_1/\bar{\sigma}_3$, occurred and the computed ϕ' was 34 deg. The application of the last 10 per cent of maximum deviator stress has therefore increased the possible computed ϕ' by as much as 15 deg, but only after considerable strain. This poses the important question of whether or not the triaxial test satisfactorily represents *in situ* conditions of stress and strain for this sensitive, brittle soil.

It is thought that after about 80 to 90 per cent of the maximum stress has been applied, the soil begins to creep and build up pore pressures which are not compatible with field phenomena. In the following analysis therefore the maximum deviator stress is considered to represent failure, and an effort is made to interpret test results at stresses well below failure. At half maximum deviator stress, for instance, the specimen has a factor of safety, F, of 2. Since $c' = 0$, the shearing resistance at stresses less than failure may be represented by:

$$S = \bar{\sigma}\,\frac{\tan \phi'}{F} \qquad (1)$$

where ϕ', the maximum angle of shearing resistance, is equivalent to the "maximum angle of obliquity" at failure as defined by Taylor (18). At half maximum deviator stress, the maximum angle of obliquity, α_m, is

$$\sin^{-1}\left(\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_1 + \bar{\sigma}_3}\right) \qquad (2)$$

Then:

$$\tan \phi' = F \tan \alpha_m$$

$$= 2 \tan \alpha_m$$

For specimen 96-4-4, when $F = 2$, $\alpha_m = 9$ deg and $\phi' = 17$ deg.

When ϕ' was computed in this way at various degrees of maximum stress, it was found to be fairly constant up to about half maximum deviator stress. Further

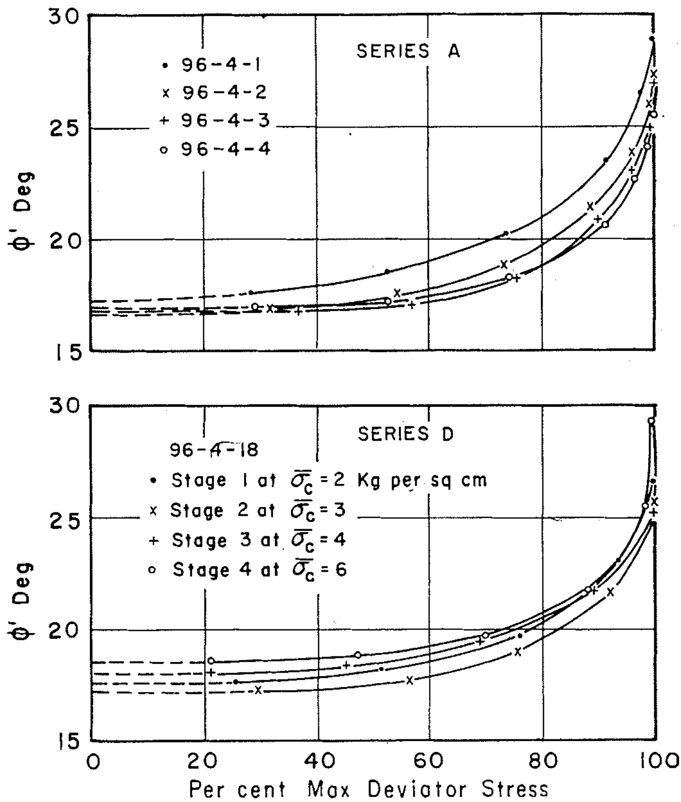

FIG. 5.—Relation Between ϕ' and Deviator Stress.

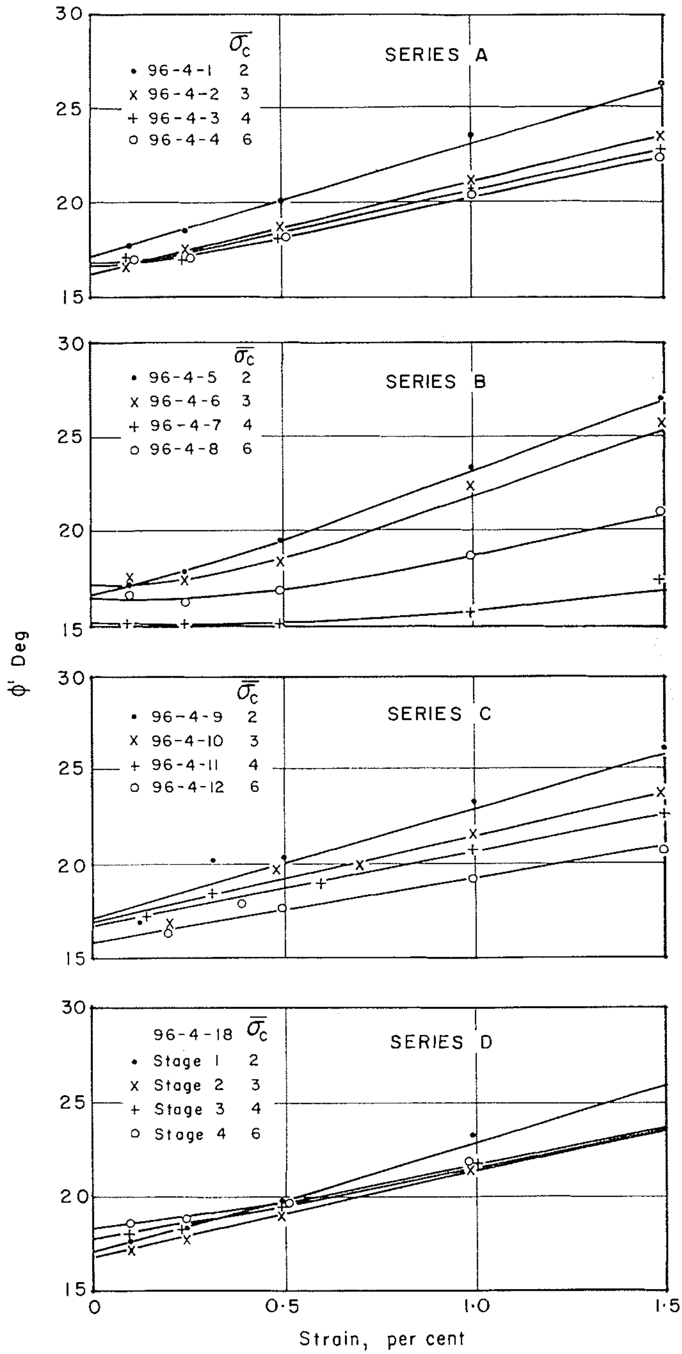

FIG. 6.—Relation Between ϕ' and Percentage Strain (Specimens 96).

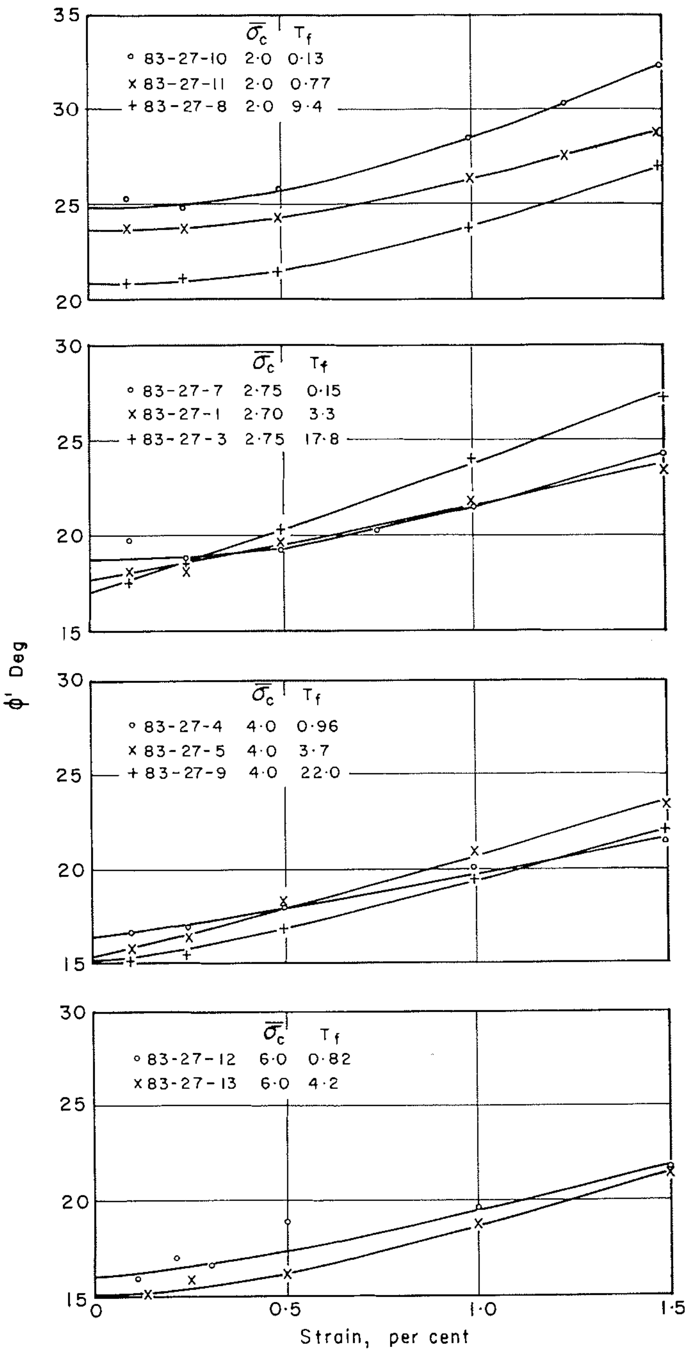

FIG. 7.—Relation Between ϕ' and Percentage Strain (Specimens 83).

stressing caused a marked increase in computed ϕ'. In Fig. 5, values are plotted for specimens of series A and D. A simpler relationship was found between ϕ' and per cent strain. Additional test results plotted in this way are shown in Fig. 6. Curves were drawn through points up to maximum deviator stress, but since the curves were practically straight beyond 1 per cent strain the values are shown only to 1.5 per cent strain.

The significant feature of these relationships is that the computed ϕ' at low stresses is practically constant at an average of about 17 deg and is independent of consolidation pressure. Note that specimen 96-4-1 lies slightly above the average because the laboratory consolidation pressure is slightly less than the *in situ* preconsolidation pressure. Series B test results are scattered, probably because specimens were not consolidated under back pressure and the measured pore pressure is less reliable. Series C shows considerable scatter in the low strain range because ϕ' was computed just before and just after a period of several hours at constant stress. The stage test of series D is especially significant because it agrees with the other tests and can be carried out on only one specimen over a wide stress range.

The series of previously published tests on Leda clay (14) have been analyzed in the same way and are shown in Fig. 7. These results, which were obtained from samples from another location and in which the rate of strain was varied from about 0.1 per cent to 10 per cent per hour, reveal a similar value for ϕ' of about 15 to 17 deg at low strains which appears to be independent of rate of strain. Tests at consolidation pressures less than 3 kg per sq cm give higher values of ϕ' and appear to be affected by the natural overconsolidation.

From these test results, it is thought that the computed ϕ' of 17 deg is a fundamental parameter equivalent to Hvorslev's true friction angle ϕ_e. For discussion, reference is made to Fig. 8 which shows Mohr stress circles for specimen 96-4-4 at various degrees of strain up to 10 per cent. The maximum angles of obliquity at strains of 0.25 per cent and 0.5 per cent are 5 and 9 deg, respectively, and the computed values of ϕ' are 16.9 and 17.1 deg. In this stress range, rather little change occurs in the effective stress on the plane of maximum

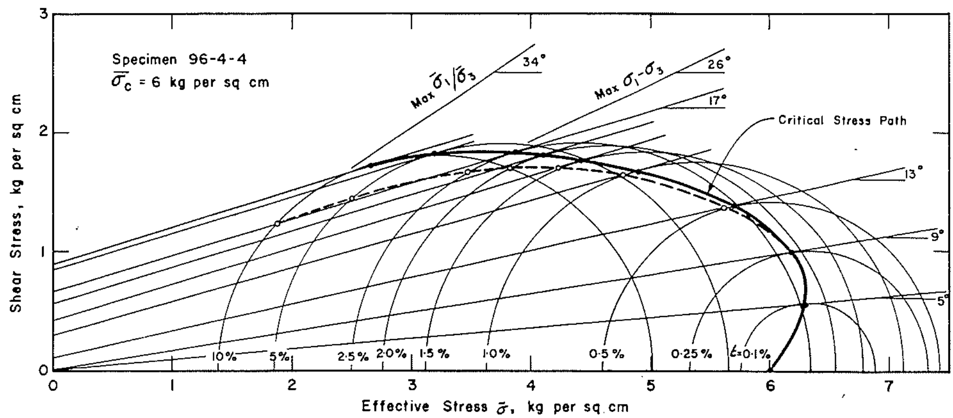

FIG. 8.—Effective Stress Circles at Various Degrees of Strain.

obliquity. It is reasonable to suppose that the true frictional resistance is not fully developed. At 2.5 per cent strain, the maximum deviator stress is reached. Since effective stress on the failure plane is reduced to about 60 per cent of its maximum, the frictional resistance is fully developed and the shearing resistance envelope is drawn at an angle of $\phi_e = 17$ deg intersecting the ordinate axis at a value of 0.65 kg per sq cm. This is regarded as the true cohesion, c_e, defined by Hvorslev and is thought to be due to prestressing (19) on the failure plane. Other values at various strains are shown. At 0.5 per cent strain, it is thought that some true cohesion is developed due to prestressing even though friction is not fully developed.

A solid curve is drawn through points of tangency to the Mohr circles to indicate the effective stress on the most critical shear plane which gradually steepens as friction is mobilized. This "critical stress path" is similar to but not quite the same as the "vector curve" employed by Casagrande and Wilson (19). A set of "critical stress paths" obtained in test series A is illustrated in Fig. 9. These indicate that the magnitude of true cohesion is dependent on the consolidation pressure as postulated by Hvorslev for remolded specimens. In the lower part of the figure, the relationship between true cohesion and consolidation pressures is illustrated. The Hvorslev "coefficient of effective cohesion" is 0.11. Although systematic observations have not been made, it has been casually observed that the shear planes at failure in this material are inclined at about 55 deg to the horizontal, giving further confirmation to the reported value of ϕ_e (1).

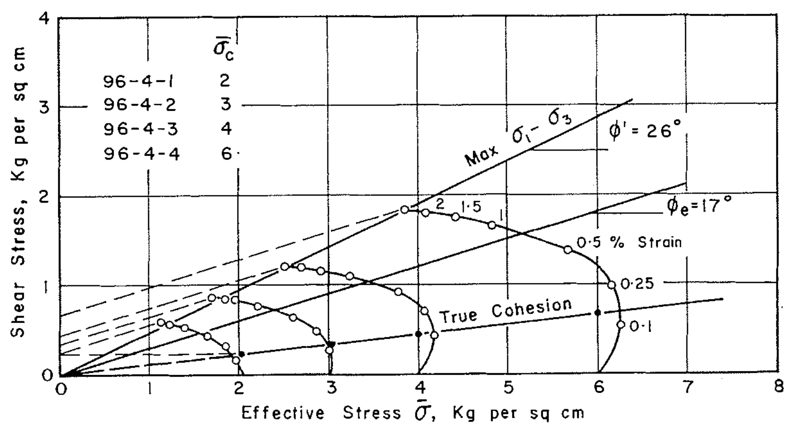

FIG. 9.—Effective Stresses on Most Critical Shear Planes.

DRAINED TESTS

Some consolidated drained tests have been made on this sensitive clay, but none is reported in the paper. It has been found that when axial stress is increased with full drainage, the specimen deforms up to 30 per cent strain or more before maximum strength is reached, and it has been argued that such a test has no practical significance (10,14). If the specimen is caused to fail by decreasing the lateral stress, the relationship between computed ϕ' and strain is similar to that measured by an undrained test, but considerable strain is again necessary for failure (10). The reasoning applied to consolidated undrained tests can not be extended to the drained tests because there is no period of more or less constant effective stress on the failure plane.

For relatively insensitive remolded clay, Henkel (20) found strength in terms of effective stresses to be independent of type of test, drained or undrained, but he was unable "to relate shear strains measured in the various types of tests." He drew attention to the necessity of relating shear stress and shear strain for practical problems and cautioned against extending his laboratory results to field problems.

For both undisturbed and compacted clays, Hirschfeld (21) found the relationship between strength and effective stress to be dependent on test method (drained or undrained). From a study of many test results, he concluded that time of loading and prestress are the most important factors influencing the relation between shear strength and effective stress in a saturated clay.

OTHER SOILS

Test results for a Drammen clay have been reported in sufficient detail by Bjerrum and Simons (9) to permit determination of its "true angle of internal friction" by the described method. This clay, with a liquid limit of 37 per cent, plastic limit of 18 per cent, and natural water content of 34 per cent, is computed to have a true angle of about 18 deg.

Tests described by Simons ((22), Fig. 1) on a highly plastic clay from Canada, for which sufficient information was available, gave a computed true angle of internal friction of about 16 to 17 deg. Tests by the Division of Building Research on samples from the same area gave approximately the same value.

VARIATIONS IN SPECIMEN DURING TEST

The results of several investigations on the stress variation in a specimen during a triaxial test were summarized by Hvorslev (23). Shockley and Ahlvin (24) reported significant stress variation and opposite volume change tendencies in various portions of sand specimens being sheared in a triaxial device and concluded that "interpretations of test data based on total volume changes may be completely at variance with those determined on the basis of changes in the failure zone alone." The late D. W. Taylor reported that "non-uniform conditions of stress and strain occur in all types of tests, and failures are always, to a degree, progressive" (18). Taylor measured pore pressures within clay specimens, and after shear he measured the variation in water content. His results, just recently published by Whitman (25), show variations of $1\frac{1}{2}$ per cent. Similar results are quoted in this paper.

It is difficult, if not impossible, to make allowance for stress variation and water migration within a specimen during the triaxial test. It is useful, however, to attempt to judge the significance of these complications in special tests on reasonably uniform specimens. This is done with reference to Fig. 8. Initially, it is reasoned that as the shear stress approaches its maximum value the structure of the sensitive clay breaks down and transfers normal stresses to the pore water, causing water to flow from the most critical plane. As friction develops progressively, the critical plane steepens to an angle of $45 + \frac{\phi_e}{2}$ to the horizontal. If the material remained homogeneous, the critical plane would maintain its position, but if consolidation occurs on the plane, it will tend to move up or down or to steepen. If a progressively steepening plane due to mobilization of friction is visualized, it is thought that this trend will continue because a shift up or down in the specimen would require the critical plane to move into a region previously affected by consolidation. If the critical plane steepens due to progressive drying, the failure envelope will intersect the Mohr circles which are computed from total stresses minus measured pore water pressure, and the "critical stress path" will trend toward that shown by a broken curve in Fig. 8. Casagrande and Wilson (19) used similar reasoning to explain variation in the shape of "vector curves" depending on type and rate of stressing.

FRICTION AND COHESION

There has not been satisfactory agreement on the true mechanism of friction and cohesion in clay soils. On the basis of experiments with remolded clays, Rowe (26) suggested that true cohesion may be unstressed at equilibrium and that creep would result if true cohesion were mobilized. Lambe (27) saw no fundamental difference between the mechanism of friction and cohesion. He argued (28) that cohesion was mobilized at very small strains, and at some further degree of strain it ceased to contribute to shearing resistance. From a comprehensive set of tests on a variety of soils, Schmertmann and Osterberg (29) conclude that the "cohesion strength component generally develops to its maximum value at an axial compressive strain of less than 1 per cent, while the friction angle requires a much greater strain to reach its maximum value." For normally consolidated Lilla Edet clay, Bjerrum and Wu (7) found true cohesion to be proportional to the consolidation pressure. Specimens reconsolidated in the laboratory to pressures less than the preconsolidation pressure were found to have additional true cohesion. This they attributed to rigid bonds between particles that suddenly break down at pressures above the natural preconsolidation pressure.

In a well-documented paper, Rosenqvist (30) traced the development of physicochemical concepts of soil water systems and emphasized the importance of considering a portion of the water in a soil as part of the soil particles. This concept led him to conclude that when the pressure between mineral grains is released they will continue to stick together because of adhesional forces. This is thought to be a most important concept and is employed in the following explanation for the development of friction and cohesion in the Leda clay.

It is believed, based on test results reported in this paper, that all shearing resistance is of a frictional nature (that is, dependent on normal effective stress on the plane of shear) and that cohesion is caused by the inability of the interparticle forces to decrease quickly (Rosenqvist's adhesion) when external loads are released. It follows that the frictional component (ϕ_e = 17 deg) is independent of time or stress path. The cohesion component, on the other hand, is dependent on stress release and may be expected to begin to dissipate as soon as it is mobilized. It will probably not decrease to zero. Soils which have been rebounded geologically might be expected to retain a minimum cohesion. This could account for the "threshold effect" observed by Bjerrum and Wu (7).

Friction in clays will probably not be as simple as friction between ordinary plane surfaces since it is not thought to involve direct mineral contact. Rather, it will be increased by decreasing distance between particles and by enlarged overlapping of water films. It is probable that the decreasing distance between mineral grains brought about by increasing stress is not compatible with the minerals' demand for complete surface layers of water and that long term adjustment will take place as particles move relative to one another seeking equilibrium. This adjustment is considered to reduce the overlapping of water films with a consequent reduction in the so-called "true cohesion." Time-dependent decrease in the empirical cohesion intercept c' has been shown in the laboratory (14) and in the field (31), and this may logically be interpreted to indicate similar variation in the true cohesion c_e.

APPLICATION

The Hvorslev parameters are considered to represent the correct mechanism of shearing resistance in a soil. Empirical parameters in terms of effective stresses are thought to provide a satisfactory basis for stability analyses provided the stress range during test corresponds to that in the field. A valuable comprehensive study of evidence supporting this view has been presented recently by Bishop and Bjerrum (32). The correlations between analysis and performance always appear to be less satisfactory for natural soils than for compacted soils. This is usually attributed to greater variations in the properties of natural soils and to unknown influences of weathering. A further, and perhaps more important, reason may be errors in the laboratory values of temporary and permanent cohesion, especially in the low stress range. It is not easy to evaluate the true strength parameters in the over-consolidated range, but one of the most difficult soil engineering problems involves unloading of natural soils. It is clear therefore that much more work must be done on natural soils in this important stress range.

CONCLUSIONS

The following conclusions are based on relatively few consolidated undrained test results on a sensitive marine clay. Some evidence of a certain general applicability is quoted, but the extrapolation of test results on sensitive clays is thought to be hazardous.

1. The measurement of shear strength parameters of a sensitive clay in terms of effective stresses applied in the triaxial compression apparatus is subject to an unsatisfactory range of interpretation.
2. The computed ϕ' is directly related to strain in the specimen. At stresses up to about one half of failure stress in the material tested, it is reasonably constant at about 17 deg. On the basis of the Hvorslev hypothesis, this angle is thought to be equivalent to the "true angle of internal friction."
3. This "true angle of internal friction" is independent of cell pressure and rate of strain.
4. "True cohesion" is considered to be equivalent to a frictional phenomenon caused by intrinsic pressures which are not immediately reversible when stresses are released but which will decrease with time.
5. True friction is considered to be mobilized at very low strain (less than 1 per cent in the specimen considered) with true cohesion mobilized as required during stress release.
6. Measurements show that nonuniform consolidation occurs due to application of all-round pressure and that water moves away from the plane of shear. This is thought to influence effective stress parameters, especially at high strains.
7. Most reliable pore pressure measurements are obtained when specimens are consolidated under a back pressure.
8. Under constant axial stress, pore pressure increases in direct proportion to strain.

Acknowledgments:

The author is grateful for the assistance and constructive criticism of associates in the Division of Building Research. Special appreciation is due to D. C. MacMillan and B. J. Bordeleau for able assistance with testing and computations. The author has been encouraged by R. F. Legget, Director of the Division, and this paper is published with his approval.

LIST OF SYMBOLS

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,
$\bar{\sigma}_c$ = effective consolidation stress, kg per sq cm,
$(\bar{\sigma}_1 - \bar{\sigma}_3)_f$ = deviator stress at failure, kg per sq cm,
T_f = time to failure, hr,
F = factor of safety,
α_m = maximum angle of obliquity,
c' = cohesion intercept in terms of effective stresses, kg per sq cm,
ϕ' = angle of shearing resistance in terms of effective stresses,
c_e = true cohesion, kg per sq cm, and
ϕ_e = true angle of internal friction.

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DISCUSSION

Mr. Werner E. Schmid[1] (*presented in written form*).—Mr. Crawford presents a series of most interesting results and he should be commended for the diligence with which the reported experiments were carried out. By far the most significant results of his investigations are the wide variations that are possible in the interpretation and evaluation of shear strength data with pore pressure measurements. The effective angle of shearing resistance ϕ' is found to vary, depending on the strain, from $\phi' = 19$ deg to $\phi' = 34$ deg. Similar results have been observed also by this writer. Such wide variations must clearly be disturbing to anyone who has to use laboratory data of such nature for the design of field installations. These variations, of course, result from the build up of pore pressure with strain and time. For this and a number of other reasons, we have concluded it is unwise to assess the shear strength of a clay soil in the field from laboratory triaxial tests with pore pressure measurements. They are subject to too wide a fluctuation depending entirely upon the strain at which the tests are evaluated, the compliance of the measuring system, the point of pressure pickup, etc.

As a consequence, we are proposing an entirely different shear strength theory[2, 3] which redefines the whole problem and introduces a new concept of shearing strength. A detailed exposition of these concepts is beyond the scope of this discussion; however, the basic principles are outlined briefly:

Clay Soil as an Engineering Material.—It is generally known and accepted that the composition of an engineering material is the most important factor that determines its properties. We utilize this knowledge to manipulate the properties of alloys and of plastics, as well as other materials. The carbon content of a steel determines its yield strength and ductility, and the water-cement ratio of a concrete is the most critical parameter for concrete strength. The composition parameters are very sensitive; that is, their slightest change has a great effect upon the material properties. When we conduct triaxial tests in the soil mechanics laboratory, the samples are usually consolidated at different lateral pressures. This results in different water contents. Hence, samples thus consolidated *are really different engineering materials!* In view of our experience of the sensitivity of composition parameters, the notion that the change in shear strength of a whole series of engineering materials can be expressed by one constant parameter ϕ' in the equation $s = c + \sigma \tan \phi'$ appears to be a vain and naive hope.

In order to get significant and meaningful results we should, therefore, carry out our tests on samples that have the same composition, that is, the same

[1] Associate Professor, Civil Engineering Dept., Princeton University, Princeton, N. J.

[2] W. E. Schmid, Y. Klausner, and C. F. Whitmore, "Rheological Shear and Consolidation Behavior of Clay Soils," Progress Report to the Office of Naval Research, Princeton, University, (1960).

[3] W. E. Schmid, "New Concepts of Shear Strength for Clay Soils," *Journal*, Soil Mechanics Div., Am. Soc. Civil Engrs. (in press).

water content. If such triaxial tests are carried out (undrained quick tests), we find that on the basis of total stresses the failure envelope in a Mohr diagram is a horizontal line, that is, $\phi = 0$. This means that the Mohr-Coulomb failure theory is not applicable, the shear strength *does not* directly depend on the normal stresses, and the general acceptance of the Mohr-Coulomb theory stems from a confusion of cause and effect.

Shear Strength of Saturated Clay Soils.—On the basis of the concepts above, we formulated a shear strength concept which does not separate s into friction and cohesion components but considers the total value of the shear strength which is found to depend on (*a*) the water content of the clay soil and (*b*) the duration of the load. It is also proposed that the best measure for shear strength is the octahedral shear stress. Since the maximum shear stress, $\frac{1}{2}(\sigma_1 - \sigma_3)$, varies only by—at most—15 per cent from the octahedral shear,[4] the former may be substituted without significant loss of accuracy. Thus we may write:

$$s = \frac{1}{2}(\sigma_1 - \sigma_3)_{\max} = Ae^{B(w-w_0)}$$

where:

A and B are material constants that vary with the load duration, and

e is the base of the Naperian logarithm.

Water Content Variation.—In view of the concepts above, the variations of the water content within a triaxial specimen become most interesting, and Crawford's data are highly significant. They confirm observations in our laboratory where within a research program on shear strengths of clay soils we observed the variation of water content within triaxial test specimens before and after shearing. Our specimens were made of remolded, homogeneous, saturated clay of illite and—in the second series—kaolinite type clay minerals. We found that:

1. After consolidation (with side drains) the water contents were highest at the bottom of the specimens, somewhat lower at the top, and lowest at mid-height. Radially, the water content decreased from a maximum at the center to a minimum at the edge. The range of the differences was between 1 and 2 per cent.

2. After shearing, the failure zone always had the smallest water content with deviations from the average water content of the sample by again 1 to 2 per cent.

Deviator Stress versus Stress Difference.—The only displeasure this writer has with Mr. Crawford's otherwise excellent paper is the unfortunate choice of the term stress deviator for the principal stress difference $\sigma_1 - \sigma_3$. While we recognize that this mistake is made frequently in soil mechanics circles, we feel it is time to correct this situation and avoid this constant source of confusion. The term deviator stress has been defined in classical mechanics as the difference between the principal stress and the mean hydrostatic stress, and we should respect this definition:

$$\text{Deviator stress } S_1 = \sigma_1 - \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3).$$

Inclination of Failure Surface.—Saada[5] has shown recently that the inclination of the failure surface on a triaxial specimen should be equal to the octahedral angle $\alpha = 54°45'$. This is in excellent agreement with the inclination of 55 deg observed by Crawford as well as Saada's own observations.

[4] N. M. Newmark, "Failure Hypotheses for Soils," *Proceedings*, Am. Soc. Civil Engrs., Research Conference, Shear Strength Cohesive Soils, Boulder, Colo., June, 1960, pp. 17–32.

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MR. DONALD M. BURMISTER[6] (*presented in written form*).—The influences of strain on stress-strain and shearing strength responses of clays appears to be more characteristic of certain preconditioned states than of inherent sensitivity of clay as such. Soil is a most unusual construction material because its responses in laboratory tests and its field performances with regard to foundation support of structures and stability of earthworks and natural slopes are strongly influenced and conditioned by the kind, relative dominance, and sequences of conditioned imposed. There are no simple model-prototype relations between laboratory soil responses and field performances which can be used invariably and reliably. Soil is always a prestressed and prestrained material with a prestress at any depth at least equal to the weight of overburden.

A first fundamental concept of soil performances states that soils having the same identifications but different prestress, confinement, and preconditioning are in reality different materials, because they possess essentially different responses and performances. A sample of clay which has been taken from its prestress and confinement environment is a radically different material from that in the natural "in place" state. Furthermore, soil samples may be adversely altered in some degree by sampling disturbances. A second fundamental concept, therefore, follows in accordance with the principles of prototype triaxial testing of soils that the original prestress and confinement conditions must be restored on a triaxial specimen, and thereafter the new conditions imposed by construction of structures and earthworks must be applied, all with proper regard to kind, relative dominance, and sequences of conditions. Thus prototype stress-strain and shearing strength responses of clay soils can be consistently and reliably obtained that have direct and valid applications in evaluation and solutions of foundation and stability problems and in predicting full scale field performances.

The prestressing of triaxial specimens of clay by complete consolidation to increasing stress levels corresponding to different depths represents only one aspect of prototype testing. The prior determination of the natural prestress, state of consolidation, and undisturbed quality and "goodness" of samples are essential in triaxial testing in order to set up proper triaxial test conditions and procedures. Prototype consolidation tests are required with an unloading-reloading stress cycle and characteristic triangle construction to establish the prestress and state of consolidation.[7] If a clay is normally consolidated with the prestress equal to the present overburden, the starting point of straining "in place" under imposed stresses is at this prestress. The significant and usable stress region for triaxial testing is therefore just above this natural prestress level on the primary consolidation curve and associated triaxial curve. Stress-strain responses below the prestress are merely a laboratory reloading phenomenon to restore this prestress.[8]

More frequently than is supposed, clays have been over-consolidated under prestresses greater than the present overburden. For over-consolidated clays, the starting point of straining "in place"

[6] Civil Engineering Dept., Columbia University, New York, N. Y.

[7] D. M. Burmister, "Strain-Rate Behavior of Clay and Organic Solids," Papers on Soils—1959 Meetings, *ASTM STP No. 254*, Am. Soc. Testing Mats., pp. 90–92, Fig. 1, pp. 99–104 (1959).

[8] D. M. Burmister, "The Importance of Natural Controlling Conditions upon Triaxial Compression Test Conditions," Triaxial Testing of Soils and Bituminous Mixtures, *ASTM STP No. 106*, Am. Soc. Testing Mats., pp. 256–261, Figs. 1 and 2 (1950).

under imposed stresses is at the overburden stress in a natural unloading stress cycle. The significant and usable stress region for triaxial testing is on a relatively flat reloading stress cycle just above the overburden stress to the natural prestress and continuing thereafter on a steeper primary consolidation curve and associated triaxial curve.[8]

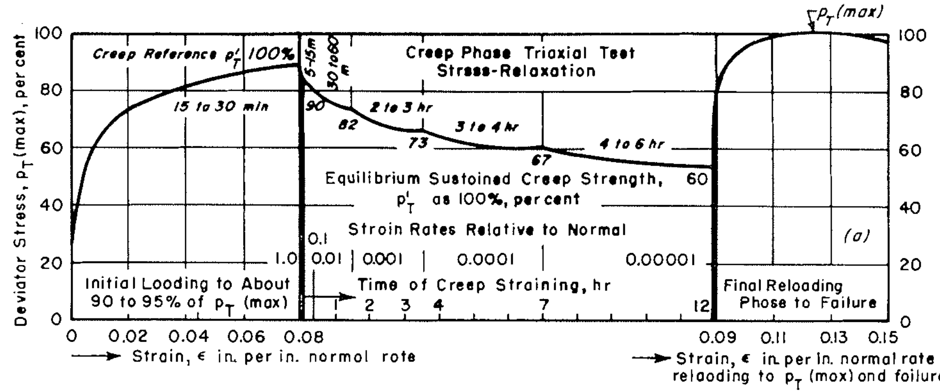
Furthermore, the loading and confinement conditions in sequence have important influences on stress-strain responses and mobilizable shearing strength for foundation support, but more critically for stability of structures and earthworks under unbalanced stress conditions. The strains in the latter case are much smaller—first, for unloading conditions, such as exposure of a slope by excavation and resulting instability conditions and, second, for conditions of lateral yielding of retaining structures with accompanying relief of stress from the "earth pressure at rest" to the active pressure state.[9] Prototype triaxial testing requires that these strain and lateral confinement conditions be simulated by proper testing procedures in sequence which will have to be learned by experience. In addition, the initial prestressing and preconsolidation of triaxial specimens to required stress levels should be in accordance with vertical and lateral stresses increased in a definite ratio corresponding to the state of consolidation, instead of by hydrostatic preconsolidation.[9]

The controlling influences of slow plastic creep, however, are considered to be more important than the above influences for clay soils. Creep phenomena were treated rather completely in the paper entitled—"Strain-Rate Behavior of Clay and Organic Soils."[7] In the period from 1933 to date, there have been many opportunities to observe creep phenomena in action, to study the nature and controlling influences of creep, to evaluate the conditions that govern creep, and to design bulkheads, retaining structures, earth dams, and embankments over clay and organic soil deposits against failure eventually by slow creep. Methods and apparatus for triaxial creep testing were developed and used since 1945. Cases 1 and 2 describe the nature of the problems involved in stability of embankments.[7]

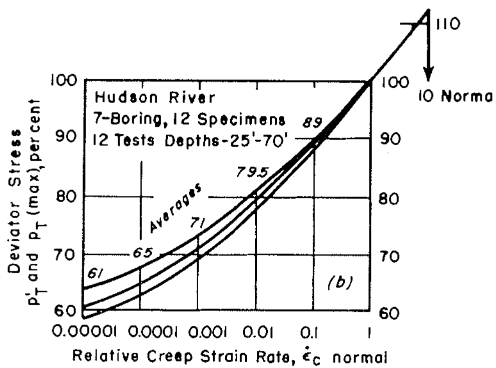
Observations, experiences, and investigations of mass slide failures have shown that a mass slide has practically always been initiated by a phase of slow plastic creep.[7] The initial elastic slumping downward of a slope induces shearing stresses beneath the toe of a slope. If these shearing stresses fall within the range of appreciable creep strain-rates, then creep is initiated beneath the toe of the slope, which is the stress region most vulnerable to creep action. When lateral subsurface creep displacements accumulate to sufficient magnitudes beneath the toe of a slope, settlement of this toe portion must inevitably result. This, in a real sense, "pulls the rug" from under the slope and tends to tilt the slope or embankment outward. As a consequence, well-known tension cracks are formed back of the crest of the slope, which are evidence of creep conditions. Some shearing strength necessary for stability is thus destroyed progressively as these cracks deepen, causing a greater unbalance of stress conditions. As creep is now propagated back beneath the slope, the creep displacements accumulate finally to critical magnitudes. Eventually, these sequences of events lead to a rapid mass slide failure, especially if heavy rains fill the tension cracks and induce additional sliding forces, as a "trigger" action.

Creep is essentially a stress-deformation stress-relaxation phenomenon,

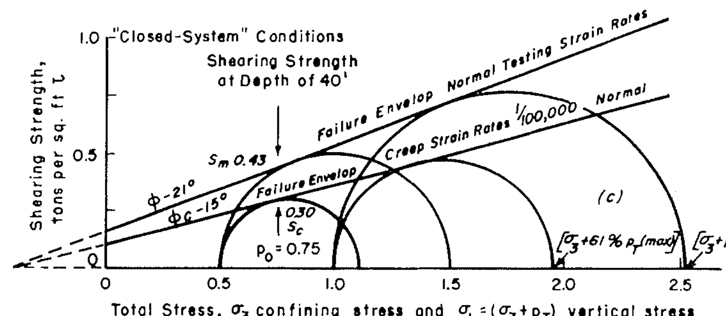
[9] D. M. Burmister, "The Place of the Direct Shear Test in Soil Machines," Symposium on Direct Shear Testing of Soils, *ASTM STP No. 131*, Am. Soc. Testing Mats., pp. 6–11, Figs. 3 and 4 (1952).



(*a*) Triaxial test stress-strain curves with creep phase.



(*b*) Equilibrium sustained stress mobilization under decreasing creep strain rates. A stress relaxation phenomenon.



(*c*) Mohr stress circle analysis of failure conditions at normal triaxial testing rates and creep strain rates 1/100,000th normal.

FIG. 10.—Prototype Triaxial Stress-Strain and Creep Responses to Determine Equilibrium Sustained Creep Shearing Strength. (Dark gray ORGANIC SILT, little fine sand, low plasticity range and average moisture content.)

which is illustrated in Fig. 10(*a*). A preconsolidated triaxial specimen is strained under "closed system" conditions by application of a triaxial or deviator stress, P_T, to about 90 per cent of the maximum mobilizable stress. Thereafter, a creep phase of testing is performed in five stages with the creep strain rate being reduced successively to $\frac{1}{10}$ of the previous strain-rate and finally to a value of $\frac{1}{100,000}$ of normal triaxial testing strain-rates. The creep strain-rate was held constant at each stage until the deviator stress under strain control loading backed off by stress-relaxation to a stable equilibrium sustained stress value for that strain-rate. The length of time to reach equilibrium conditions, as noted in Fig. 10(*a*), increased considerably at the lower strain-rates. The creep strain-rate of $\frac{1}{100,000}$ normal is of the order of the very slow creep rates which may be expected to occur in natural clay deposits.

There are a number of important facts concerning creep phenomena which should be recognized and understood. First, the equilibrium sustained creep strength is strain-rate dependent. Second, an equilibrium sustained creep stress is possible at each strain-rate level with creep continuing indefinitely at that rate to final failure. Third, the sustained creep stress in Fig. 10(*b*) tends by the curved relationship toward a rather definite minimum yield value. Fourth, in a Mohr stress circle analysis in Fig. 10(*c*) for total stresses under "closed system" conditions, the creep failure envelope for $\frac{1}{100,000}$ normal strain rates falls considerably below the maximum failure envelope for normal testing strain rates. Fifth, this creep phenomenon cannot be attributed to a serious breakdown of soil structure by shearing disturbance effects at this shearing displacement but represents a stress-relaxation characteristic of creep in clay and organic soils. Figure 10(*a*) shows in a rerun test phase at normal testing strain rates immediately after the creep phase that the maximum shearing strength is still fully mobilizable. Sixth, it is evident that cohesion and angle of friction are not stable inherent properties of clay and organic soils, as supposed, but are subject to stress-relaxation influences. Furthermore, cohesion cannot be separated strictly from friction because cohesion is really only a "memory" of a previous prestressed friction state. Seventh, it becomes evident that shearing failure occurs under creep conditions when the accumulated creep displacements exceed a value where a "slip plane pattern" is finally fully developed with an accompanying permanent loss of some shearing strength. By ignoring creep conditions, a factor of safety of 1.4 using an indicated shearing strength of 0.43 tons per sq ft on the maximum failure envelope of Fig. 10(*c*) would be completely wiped out under creep conditions with a mobilizable sustained creep shearing strength of 0.30 tons per sq ft, and failure would be inevitable when the creep strains have accumulated to such magnitude as to develop the failure slip plane pattern with the sustained creep shearing stress never rising above the creep level.

In view of these considerations, a fundamental concept may be stated that if the shearing stresses at the toe of a slope or embankment can be kept with a reasonable factor of safety of 1.25 to 1.50 below a value where creep cannot proceed at strain rates greater than $\frac{1}{100,000}$ normal, then creep shearing displacements would not accumulate to sufficient magnitudes within the lifetime of a structure to initiate the final stages of failure by a mass slide, provided that no new or unaccounted for conditions developed not covered by the above margin of safety.

MR. JOHN H. SCHMERTMANN[10] (*by*

[10] Assistant Professor, Civil Engineering Dept., University of Florida, Gainesville, Fla.

letter).—The many contributions of this very interesting paper by Mr. Crawford include a possible new method for the computation of Hvorslev's effective, or "true," friction component of soil strength, ϕ_e. This method offers the tremendous advantage of obtaining ϕ_e from a single undrained test with pore-pressure measurements. *If valid*, the present difficulty of obtaining the necessary group of identical test specimens would be overcome, and the testing of natural soils for ϕ_e and c_e would be practical. The Hvorslev components would then be freed for use in the practice of soil engineering. In view of its potential importance, this discusser herein further examines the validity of the proposed method of determining ϕ_e.

Conforming to the author's definitions and notation, the following is a derivation of another expression for tan ϕ' to be used in subsequent discussion:

$$\tan \phi' = F \tan \alpha_m = F \tan \left[\sin^{-1} \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_1 + \bar{\sigma}_3} \right]$$

$$\text{or:} \quad \tan \phi' = \frac{F}{2} \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\sqrt{\bar{\sigma}_1 \bar{\sigma}_3}} \quad \ldots\ldots\ldots (1)$$

In an axial compression, triaxial test

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \frac{(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{F} \quad \ldots\ldots\ldots (2)$$

Substitution of Eq 2 into Eq 1 gives

$$\tan \phi' = \frac{1}{2} \frac{(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{\sqrt{\bar{\sigma}_3^2 + \frac{\bar{\sigma}_3(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{F}}} \quad \ldots (3)$$

Since $F > 2$ and $\bar{\sigma}_3 > (\bar{\sigma}_1 - \bar{\sigma}_3)_f$ during the deviator stress range considered, this permits $\sqrt{\bar{\sigma}_3^2 + \frac{\bar{\sigma}_3(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{F}}$ to be approximated by $\bar{\sigma}_3$ and Eq 3 is simplified to

$$\boxed{\tan \phi' = \frac{1}{2} \frac{(\bar{\sigma}_1 - \bar{\sigma}_3)_f}{\bar{\sigma}_3}} \quad \ldots\ldots\ldots (4)$$

Applying Eq 4 to two of the typical tests detailed in Fig. 1, and using the data in Table II with the assumption that $\bar{\sigma}_3 = \bar{\sigma}_c$, which is equivalent to assuming negligible Δu developed during compression, the following comparative results are obtained:

Test No.	$\bar{\sigma}_c$, kg per sq cm	Values of ϕ', deg		
		From Eq 4	From Fig. 5	
			0 per cent Deviator Stress	50 per cent Maximum Deviator Stress
96-4-2 . .	3	16.8	16.6	17.3
96-4-4 . .	6	17.7	17.0	17.1

This agreement indicates that Eq 4 is a good approximation, and also that negligible pore pressure developed during the 0 to 50 per cent range of maximum deviator stress.

A major portion of the author's argument to support his conclusion that the low-strain ϕ' and the Hvorslev ϕ_e are equal is that the computed values of ϕ' are approximately constant from 0 to 50 per cent maximum deviator stress and are approximately independent of consolidation pressure in the normal consolidation range. From Eq 4 it is obvious that ϕ' is approximately constant if $\bar{\sigma}_3$ is approximately constant. This occurred in the author's tests because only small pore pressure developed during strain to 50 per cent maximum deviator stress. It may well occur with most cohesive soils. However, attaching a special true-friction significance to this pore-pressure behavior may be unjustified. First, we must learn the reason for this behavior and then, if possible, relate it to a carefully defined friction or cohesion component. This has not been done.

Replacing $\bar{\sigma}_3$ with $\bar{\sigma}_c$ in Eq 4, which is a reasonable approximation when Δu is small, makes tan ϕ' equal to the well-known c/p ratio. This ratio is often shown graphically, on a semi-log plot with void ratio, as a parallel relationship between normal consolidating pressure and strength. It thus conveniently illustrates that the strength of normally consolidated clay is some constant proportion of the consolidating pressure. Since the clay also decreases in void ratio with increasing pressure, it presumably increases its true cohesion capability. Why then, if $\tan \phi' = \tan \bar{\sigma}_e$, should true friction be the only variable determining this proportion?

An additional argument against considering $\phi' = \phi_e$ is their inconsistency at zero strain. At this strain the shear strength mobilized to resist the external hydrostatic consolidation pressure is zero. The Mohr stress circle is a point on the $\bar{\sigma}$ axis. The sum of the true cohesion, c_e, and the true friction, $\bar{\sigma} \tan \phi_e$, components of soil strength mobilized at zero strain is then also zero. Since $\bar{\sigma}$ is equal to $\bar{\sigma}_c$, either $c_e = -\bar{\sigma}_c \tan \phi_e$ or both c_e and ϕ_e are zero. Rejecting negative values for cohesion or friction angle the conclusion is that ϕ_e must equal zero at zero strain. Again referring to Eq 4, which is exact at zero strain because F is infinite, we see that the defined ϕ' cannot be zero at zero strain. In Figs. 6 and 7, the author shows ϕ' to have values from 15 to 18 deg at zero strain, but ϕ_e must be zero.

This discusser considers that the author's conclusions 2 and 5 are not substantiated. Consequently, the validity of the suggested method for determining the true strength components from a single undrained test is in doubt.

As stated by the author in his quotation from Schmertmann and Osterberg (29), this discusser believes that the low-strain strength of a cohesive soil is primarily due to cohesion. In this reference, cohesion is defined as that component of mobilized shear strength, at any strain and with constant structure, which appears independent of changes in effective stress. There is a great deal of evidence, both published[11] and unpublished, from the results of CFS-tests[12] to support this opinion.

MR. C. B. CRAWFORD (*author's closure*).—Mr. Schmid emphasizes the fact that in a series of consolidated undrained triaxial compression tests, each specimen is at a different water content and is therefore a different engineering material. In the author's view this statement is valid only because the volume deformation of a laboratory specimen under changing effective stress is probably much different than that of an element of soil *in situ*. Since effective stress parameters c' and ϕ' are not fundamental cohesion and friction components, it is clear that the more they depart from the equivalent fundamental components, the greater is the danger of misunderstanding.

The argument that the Mohr-Coulomb failure theory does not apply for undrained tests on saturated clay at constant water content is not valid. According to the "principle of effective stresses," shearing resistance depends on the effective stress on the failure plane, and since in an undrained test at constant water content there is no change in effective stress (no matter what total stress is applied), there is correspondingly no change in shear strength. This concept has been explained by Rutledge,[13] and

[11] J. H. Schmertmann and J. R. Hall, Jr., "Cohesion After Non-Hydrostatic Consolidation," *Journal*, Soil Mechanics and Foundations Division, Am. Soc. Civil Engrs., August, 1961, pp. 39–60.

[12] The CFS-test is a triaxial test especially devised to permit the determination of cohesion and friction as functions of strain. Details are presented in the references cited.

[13] P. C. Rutledge, "Review of the Cooperative Triaxial Shear Research Program of the Corps of Engineers," Waterways Experiment Station, Vicksburg, Miss. (1947).

the limitations of the $\phi = 0$ analysis have been described by Skempton.[14]

Mr. Schmid's observations of water content variation in a specimen due to shear confirm those reported in the paper. This water movement will also be a test difficulty even though shear strength is not divided into cohesion and friction components. The objection to the term "deviator stress" is valid even though usage is well established in soil mechanics literature.[15]

Mr. Burmister draws attention to the important considerations of environment between test and prototype. This cannot be overemphasized. It may be the source of most of our difficulties. In particular, he points out the need in testing natural soils to reestablish the original prestress and confinement conditions that existed before sampling and then to observe the stress-strain response in the appropriate working stress region. It may be pointed out that since deformation depends only on changes in effective stress, the working stress region should be in terms of effective stress.

Mr. Burmister goes on to explain features of creep phenomena based on observations in terms of total stresses on a "closed system" or undrained test condition. His fifth point that "creep phenomena cannot be attributed to a serious breakdown of soil structure" is at variance with ideas expressed in the paper. The reason for this appears to be due to the difference in interpretation depending on whether total stress of effective stress information is available.

Triaxial tests on Leda clay (unpublished) indicate that specimens can be subjected to unloading cycles or to creep under constant load without significantly changing the ultimate shearing resistance in terms of total stress provided that times to failure are not significantly different. This confirms Mr. Burmister's

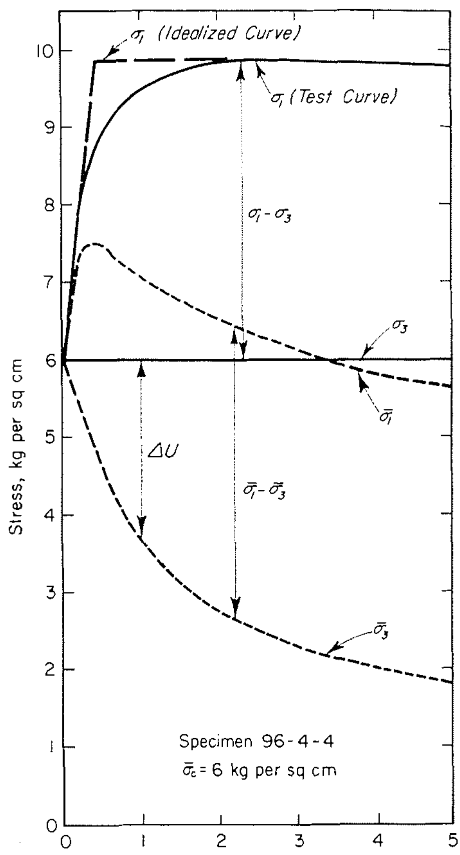


FIG. 11.—Total and Effective Principal Stress Variation with Strain.

observations in Fig. 10(*a*) and leads to his conclusion regarding structural breakdown. If pore pressures are measured, however, it can be demonstrated that the development of pore pressure is directly related to strain (Fig. 2). The only explanation for this seems to be that the sensitive structure breaks down during shear strain causing local con-

[14] A. W. Skempton, "The $\phi = 0$ Analysis of Stability and Its Theoretical Basis," *Proceedings*, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 1, p. 72–78 (1948).

[15] Definitions of Terms and Symbols Relating to Soil Mechanics, (D 653 – 58), 1958 Book of ASTM Standards, Part 4.

solidation and creating pore pressures in the undrained specimen. If the variation in principal stresses, both total and effective, for specimen 96-4-4 are examined in Fig. 11, it is seen that while the principal stress difference in each case is the same at any particular degree of strain, the effective stresses are steadily decreasing. If effective stresses govern shearing resistance, how then can this resistance be maintained approximately constant while the effective stresses decrease? To explain this, the author has used the same concept as stated by Mr. Burmister, that "cohesion is really only a memory of a previous prestressed friction state."

Mr. Schmertmann shows arithmetically what the author attempted to demonstrate graphically in Fig. 8, namely, that the development of pore-water pressure has little influence on effective stresses during application of the first half of the maximum deviator stress. The author attaches no significance to the pore-pressure behavior during the test in attempting to separate friction from cohesion. All reasoning is based on computed effective stresses, assuming that the total stresses and pore pressures can be measured with reasonable accuracy.

It is fortunate though that effective stresses remain relatively constant during initial loading, as this leads to the deduction that a frictional resistance is steadily developed due to strain on the potential failure plane. The author does not agree with Mr. Schmertmann that ϕ_e must be zero at zero strain. ϕ_e is a property of the material that requires shear strain to develop it. The factor of safety was introduced to try to take account of the degree to which friction is developed with stress application. The author has published curves showing the development of computed ϕ' from 0 to 35 deg as a function of strain (10). Of course ϕ' should not be a variable, but the curves were used to illustrate the need to agree on a reasonable failure criterion that is compatible with strains *in situ*. To be precise, the curves of Figs. 6 and 7 should have been broken at very low strains as are the curves of Fig. 5.

The author suggests that all shearing resistance is of a frictional nature and reserves the cohesion term for that portion of the frictional resistance which results from the "memory" of previous stresses and appears to be independent of applied effective stresses. These stresses are considered to be "locked-in" but are thought to decrease with time. Cohesion is attributed therefore to prestress with a superimposed time factor. Only the maximum possible cohesion is related to void ratio which is in turn dependent on stress history. To this extent the author's concept of cohesion differs from that of Hvorslev.

The unusual behavior of this soil is that it can maintain a particular shearing resistance while the effective stresses on the failure plane are reduced by more than one-half. The author suggests that an effort was made to "learn the reason for this behavior and then, if possible, relate it to a carefully defined friction or cohesion component." The success of this reasoning is definitely a matter of opinion.

It is true that the development of pore pressure up to one half maximum deviator stress has relatively little effect on effective stresses; $\bar{\sigma}_3$ is reduced by only a little more than 10 per cent. Up to this point, then the test is almost a drained test, and since the effective stress on the critical plane is so high, the computed ϕ' angle is little affected by the small pore pressure change. It is the pore pressure change during application of the second half of the deviator stress that is important. During this period, $\bar{\sigma}_3$ is reduced by about 60 per cent. The question is: Does this drastic change in effective stress under applied deviator stress occur in the field in the same manner? The author's contention is that it does not, that it is a test phenomenon, and that the interpretation and extrapolation of the laboratory test result should be given special consideration.

By introducing the concept of factor of safety, the author has, in effect, inferred a straight-line stress-strain relationship which is shown in the accompanying Fig. 11 together with the test curve for specimen 96-4-4. Which curve more closely represents the relation between stress and strain *in situ*? Which curve should be used to compute c' and ϕ'?

The author suggests that the actual relationship is closer to the "idealized curve" than to the "test curve." This is not to say that the "idealized curve" represents exactly what happens in nature but to suggest that it is the best approximation until very near to failure. This is a judgment factor based on the knowledge that the stress-strain curve is displaced more and more from the "idealized curve" as sample disturbance increases and on the belief that slope creep is not a dominant property of these brittle, sensitive clays. Failure is usually sudden and catastrophic, whereas creep occurs slowly and under drained conditions. This would result in consolidation and increased strength.

Referring to specimen 96-4-4, it is noted that at a strain of 0.2 per cent a deviator stress of 1.75 kg per sq cm has been applied and pore water pressure is 0.55 kg per sq cm. Over this stress range, both stress and pore water pressure are almost exactly proportional to strain. Pore pressure parameter A is equal to one third, exactly what it should be according to elastic theory. On further loading the proportionality of stress and strain does not apply but the increase in pore pressure continues to be proportional to strain until about 0.4 per cent strain. At this point, about 70 per cent of maximum deviator stress has been applied, and there is a drastic change in the ability of the specimen to resist the increase of major principal effective stress. It could be argued that the specimen had failed at less than 0.5 per cent strain. High A values are obtained in the laboratory only after large strains, and it is questionable if they have any practical significance in this material.

The critical question concerning shearing resistance is whether fundamental cohesion and friction parameters exist. Can strength be divided into two components, one component depending on applied effective stress and a second component independent of applied effective stress? Mr. Schmid thinks this cannot be done and is in favor of discarding the Mohr-Coulomb failure theory. This paper emphasizes the difficulty of interpreting tests on sensitive clay in terms of the Mohr-Coulomb theory, but in the author's view, this theory has too many advantages to warrant its discard at the moment. There is reason to believe that careful study of deformation in terms of effective stresses will result in satisfactory understanding of shearing resistance in clay in terms of the Mohr-Coulomb theory.