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PORE PRESSURES WITHIN SOIL SPECIMENS IN TRIAXIAL
COMPRESSION

BY

CARL B. CRAWFORD

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LES PRESSIONS INTERSTITIELLES A L'INTERIEUR
DE SPECIMENS DE SOL SOUMIS A UNE COMPRESSION
TRIAXIALE

SOMMAIRE

Des mesures de la pression interstitielle à la base et à l'intérieur de spécimens non remués d'argile sensible, pendant la consolidation et le cisaillement, indiquent que le défaut d'étanchéité de la membrane n'est pas grave dans le cas d'essais ordinaires. On décrit une technique permettant de mesurer les pressions à l'intérieur des spécimens. Dans le sol mis à l'essai, il se développait des pressions interstitielles plus élevées à la partie inférieure du spécimen qu'à son centre, et ceci était imputé à l'encastrement. Dû au fait que le manquement se produit près du bas des spécimens, la pression mesurée à l'embase est jugée satisfaisante pour le calcul des tensions effectives au point de manquement.

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PORE PRESSURES WITHIN SOIL SPECIMENS IN TRIAXIAL COMPRESSION

BY CARL B. CRAWFORD,¹ Psnl. Memb. ASTM

SYNOPSIS

Pore pressure measurements at the base and within undisturbed specimens of a sensitive clay during consolidation and shear indicate that membrane leakage is not serious for ordinary tests. A technique for measuring pressures within the specimen is described. In the soil tested, greater pore pressures were developed at the lower end of the specimen than in the center and this was attributed to end restraint. Because failure occurs near the bottom of the specimens, the pressure measured at the base plate is considered to be satisfactory for the computation of effective stresses at failure.

The effective stresses in a saturated soil specimen during shear tests in the laboratory are computed by subtracting the measured pore water pressure from the total stresses. With the refined equipment now available it is possible to measure total stresses with reasonable accuracy. Pore pressure observations and interpretation are less reliable for at least two reasons: First, the pore pressure may vary throughout the specimen, and the point at which it is measured may not be representative of the conditions in the shearing zone. Second, pore pressures may be incorrectly attributed to the shearing action when they are in fact due to leakage from the cell or to a secondary consolidation effect. This paper describes some tests made at the Division of Building Research of the National Research Council to assess the reliability of normal pore pressure observations during triaxial loading of undisturbed specimens of clay.

THE SOIL

The tests were made on specimens trimmed from an undisturbed block sample of Leda clay which had been obtained at a depth of 33 ft from a small tunnel in the city of Ottawa. The soil has a natural water content of about 58 per cent, a liquid limit of 53 per cent, and a plastic limit of 25 per cent. The pore water contains about 2 g per liter of salt, and the sensitivity is about 50. A series of triaxial compression tests made on an adjacent block of soil from this tunnel have been described (1).²

MEMBRANE LEAKAGE

Peculiarities in triaxial test results have often been attributed to leakage of water through the rubber membrane. In some cases where the membrane is damaged during installation or where proper seals are not made at the ends, a substantial leakage may be revealed by routine observations, but for ordi-

¹ Head, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada.

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

nary short-term tests, leakage through the membrane itself is not thought to be serious. This has been the writer's experience and it agrees with the views of Casagrande and Hirschfeld (2).

Some simple checks have been made which tend to confirm that membrane leakage is normally small. Several thin rubber membranes (Ramses type, 0.005 cm thick) were subjected to osmotic pressure by filling them with salt solution and immersing them in distilled water. Even at a salt concentration of 150 g per liter the leakage through a membrane was only about 0.1 g per day. A wet cup vapor permeability test, such

applied to the end caps under the membrane. Doubling the membrane had no apparent effect.

A further assessment of membrane leakage was made by consolidating a specimen for four days with a cell pressure of 4.5 kg/cm² and a back pressure on the pore water of 0.5 kg/cm² and then closing the drainage and measuring the pressure at the base plate for an additional day. The pore pressure at the base of this specimen (96-4-19, Table 1) increased from 0.5 to 0.8 kg/cm² in 8 hr and then remained approximately constant overnight. Considering that part of the increase is due to a secondary

TABLE 1—TEST RESULTS.

Specimen	Natural Water Content, per cent	Effective Consolidation Pressure, kg/cm ²	Total Length of Test, hr	Water Expelled, cm ³	Change in wt, g	Difference, cm ³	Difference per hr, cm ³
96-3-9.....	57.0	5	101	18.7	15.4	3.3	0.033
-10.....	59.7	5	147	17.8	17.7	0.1	0.001
-12.....	57.6	5	71	17.6	16.2	1.4	0.020
-14.....	58.4	5	149	16.5	16.5	0	0
-15.....	56.8	5	68	15.4	15.6	-0.2	...
-16.....	57.1	5	46	15.1	15.5	-0.4	...
96-4-19.....	57.5	4	78	14.1	12.9	1.2	0.015
-20.....	59.1	4.1	147	16.3	15.5	0.8	0.005

as is applied to building materials, caused a flow through a similar area of membrane of only about 0.2 g per day. Leakage under triaxial test conditions would probably be much smaller than in these special tests.

A few tests have been made in which the triaxial equipment has been assembled in the routine manner but with a lucite cylinder in place of the soil specimen. These have included single and double membranes with a layer of vaseline between, and the end caps have been used perfectly clean or smeared with grease. In all these tests the rate of leakage was less than 0.2 g per day at cell pressures of 4 kg/cm². Minimum leakage was obtained when grease was

consolidation effect, the membrane leakage does not appear to be significant in this case.

This specimen was then axially loaded to 0.9 kg/cm² in 8 min and the strain was held constant at 0.14 per cent for 5 hr. During this time the axial stress had to be reduced to 0.48 kg/cm² to prevent further axial strain, while the pore pressure increased from 1.12 to 1.29 kg/cm². An identical specimen (96-4-20) performed similarly. Previous work (1) showed a simple relationship between pore pressure and axial strain at constant axial stress, but these present observations indicate that the pore pressure will increase even though axial strain for the entire specimen is held

FIG. 1—Test Specimen with Needles Installed Showing No-Flow Indicators.

TEST APPARATUS AND TECHNIQUES

All tests were carried out in a precision triaxial cell with rotating bushing as described by Andresen and Simons (3). The undisturbed clay specimens, measuring 8 cm in height and 10 sq cm in area, were enclosed in a single rubber membrane. Observations of rate of consolidation showed no appreciable difference whether or not filter-paper side drains were used, so they were not used in the tests reported here.

Pore Pressure Needles:

A number of variations were tried before a satisfactory technique was developed for measuring pore pressures within the specimen. This was achieved

FIG. 2—Pore Pressure Null Indicator.

constant. This is attributed to structural collapse under the all-round effective stresses: a secondary consolidation effect. The axial strain under constant stress, which was previously observed, may result from this breakdown of structure rather than cause it.

by inserting into the soil medical hypodermic needles (0.12 cm OD). The pore pressure system was prepared by drawing de-aired water through it. The water in the needle was then quick-frozen with dry ice to prevent the entry of soil during insertion. Two needles were installed

in each specimen, one extending along the axis to the middle of the specimen from the top cap and one extending horizontally to the center of the specimen, 2 cm above the bottom as shown in Fig. 1.

Efforts were made to seal the hole through the membrane using liquid latex, but the easiest and most reliable seal was obtained using an ordinary clear rubber cement called "Kleen Stik."

in which a slug of mercury could be inserted into the pore pressure line leading from the cell to the pump in order to indicate flow (Fig. 2). It could be mounted with a minimum of pressure line and connections between the mercury slug and the cell. During flushing of the pore pressure system the slug of mercury rests in a recess in front of the screw piston. When the assembly is completed the device is tilted and the

FIG. 3—Relationship of Volume Change and Pore Pressure to Time After Beginning of Drainage.

No-Flow Indicator:

Owing to their relatively small sphere of influence, the pore pressure needles require a much more rigid system than is necessary to measure pressures at the base plate. The ordinary U-tube no-flow indicator and associated connections and copper lines did not seem to perform adequately when used with the needles. Response to pressure changes was slower than desired. In spite of great care, unexplained movements occurred in the mercury columns even when valves were closed and pressures were constant.

A simple device was therefore built,

piston is given a slight turn to force the mercury to the center of the horizontal capillary. When pressures are generated at the needle they are compensated by a manual pressure system leading to the opposite end of the mercury. A test set-up showing the needles with two of the no-flow indicators is illustrated in Fig. 1.

TEST RESULTS

Of the several tests made with internal pore pressure measurements, six were done in an identical manner with a single membrane, no side drains, and with pore pressure needles installed as

shown in Fig. 3. These six tests were free of serious leaks; the pore pressure equipment functioned satisfactorily, and the results were similar in each case. The water expelled into the buret during consolidation is compared in Table 1 with the change in weight during the test.

Several factors operate to cause disagreement between the observed volume change and weight change during a test. As mentioned previously, leakage

special precautions were taken, every joint and valve is a potential leak which, due to evaporation, may not be apparent. In more recent tests the "Klinger" valves have been replaced by "circle-seal" valves, and this seems to have improved the results. These observations indicate the order of possible inaccuracies in the measurements.

Two of the tests in which the observations are most complete have been

FIG. 4—Variation of Pore Pressure and Stress with Strain.

through the membrane itself is not thought to be especially serious. In two cases (specimens 9 and 12), some leakage around the end caps may have occurred. The absorption of water during the time taken to dismantle the apparatus increases the weight measured at the end of test and decreases the computed change in weight. Therefore, the difference between the volume and weight measurements may be a little less than indicated in Table 1. Negative differences must be due to leakage from the volume measuring system. Although

selected to illustrate the test results. In each case the cell pressure and the pressure in the pore pressure system were advanced simultaneously to 6 kg/cm². Then the drain was opened but a back pressure of 1 kg/cm² was maintained during consolidation to minimize the effect of air in the system. The response of the pore pressure through the needles, after the drainage valve was opened to the base plate, is shown in Fig. 3, together with the per cent volume change as indicated by the expelled pore water. These observations are in line with

expectations and indicate that the pore pressure devices are operating satisfactorily.

Following consolidation, the specimens were axially strained without drainage at about $\frac{1}{2}$ per cent per hr, while pore pressures were measured at the needles and at the base plate. Specimen 12 was strained for 24 hr, but when specimen 14 had reached nearly 1 per cent axial strain, the rate was increased to about $2\frac{1}{2}$ per cent per hr. The measured vertical stress and pore pressures for the two tests are shown in Fig. 4.

The performance of specimen 96-3-12 is generally representative of all of the test results, except that the pore pressure response to axial loading was slightly more than for other tests. At a strain of 0.2 per cent, approximately one half of the ultimate deviator stress was acting on the specimen, and pore pressure parameter A , which is the ratio of pore pressure to deviator stress, was about 0.9 according to the base plate measurement (4). In most tests strained at 0.5 per cent per hr, the value of A at half failure load was about 0.5 to 0.6, and in tests strained at faster rates it was even less (5). The most important observation is that at a particular strain the pore pressures in the lower quarter of the specimen are fairly uniform, but higher than pore pressures near the center. At low strains the difference is of greater significance than at failure.

Specimen 96-3-14 was representative of other tests until the rate of strain was increased from 0.5 to 2.5 per cent per hr. The increased rate of strain appeared to reduce the rate of pore pressure increase and caused a noticeable increase in shearing resistance. The pore pressure gradient from center to base plate was also increased by the more rapid loading.

Discussion:

The necessity of assessing shearing resistance in terms of effective stresses

has emphasized the need for reliable measurements and a better understanding of pore pressure development during testing. A special "Pore Pressure and Suction in Soil Conference" was held in London in 1960. Later in the same year the "Research Conference on Shear Strength of Cohesive Soils" sponsored by the American Society for Civil Engineers featured many references to pore pressure variations and difficulties of measurement.

Bishop et al (6) noted that "pore pressure gradients may arise owing to nonuniformity of stress during the shear stage due to end restraint and a tendency to narrow zone failure which appears to be characteristic of some samples." Henkel (7) confirmed the occurrence of "localized thin failure zones" in undisturbed specimens, and he recommended slower rates of testing than for remolded material. Marsal and Resines (8) shared the concern for errors due to friction at the end plates. Shockley and Ahlvin (9) provided evidence of significant nonuniformity through a triaxial specimen which could have a great influence on pore pressures.

Whitman (10) reported extensive studies on undisturbed Boston clay and noted a tendency for end pore pressures to lag behind center pore pressures at the faster rates of strain. This correlated with higher water contents near the ends of the specimen after the test. Bishop et al (6) in their closing discussion reported the pore pressure at the base of compacted specimens to be greater than at mid-height. This was attributed to the influence of end restraint. Gibbs et al (11) described test results on compacted clay in which pore pressures measured in the center and at the ends were identical. These variations in observed pore pressures illustrate the uncertainties associated with any rigorous interpretation of their significance.

The results on the Leda clay are

equally confusing. They show that even at the relatively slow rate of strain (0.5 per cent per hr) the pore pressure in the center of the specimen lags behind the pore pressure at the base by as much as 0.5 kg/cm² but once the maximum deviator stress is reached the pressures are almost equalized. Previously published tests (1) on soil from an adjacent block sample showed, however, that the soil from the failure plane is always substantially drier than that at the ends. This does not correlate with the pore pressure measurements, although it should be noted that the water content observations were made on specimens that had been strained well past failure when pore pressure gradients appear to be insignificant. In addition, the top pore pressure needle was located at the geometric center of the specimen, but the failure plane invariably passed through the lower half of the specimen. A more complete discussion of failure planes in a similar soil has been presented (12). It seems reasonable, therefore, to assume that the pore pressure on the failure plane and that at the base plate are almost equal. It follows from Bishop's explanation that the pore pressures are induced primarily by localized stresses due to end restraint, and the reduction in effective stress causes the failure plane to develop near the base. This is consistent with the fact that the porous base plate offers more restraint than does the acrylic plastic top cap.

CONCLUSIONS

On the basis of these tests the following conclusions are drawn:

1. In carefully prepared short-term triaxial tests, membrane leakage does not

seriously affect pore pressure measurements.

2. In sensitive undisturbed Leda clay, pore pressures can develop under constant stress or even when average stress is decreasing. This is attributed to a secondary consolidation effect in which the collapsing soil structure transfers some of the applied stress to the pore water.

3. Hypodermic needles inserted in the test specimens appear to provide satisfactory pore pressure pick-ups provided a small volume pore pressure system with few opportunities for leaks is used.

4. Under the initial slow axial loading the test specimen develops higher pore pressures at its lower end than at the geometric center. After failure the pore pressures tend to equalize.

5. Increasing the rate of strain during a test reduces the pore pressure slightly and causes increased shearing resistance.

6. The higher pore pressure at the base is attributed to disturbance caused by end restraint, and the resulting decrease in effective stresses are thought to be the reason for the fact that failures occur near the base.

7. Pore pressures measured at the base plate are thought to represent satisfactorily the pressures on the failure plane, but owing to the variety of experiences reported in the literature, further research on this problem is necessary.

Acknowledgments:

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DISCUSSION

G. E. BLIGHT¹—Three of the papers in this session^{2,3,4} have concerned themselves wholly or in part with problems arising from the presence of nonuniform pore pressures within a triaxial specimen during testing. This is therefore an opportune time to present some additional data on this subject.

Profiles of Pore Pressure:

Mr. Crawford has drawn attention to the number of different and apparently conflicting observations on pore pressure profiles published in the proceedings of the 1960 conference at Boulder. It was reported by various workers that the

pore pressure at the midheight of a specimen could be greater or less than that at the ends, and explanations of how this came about were far from complete.

A partial explanation of the shape of the pore pressure profile set up in a triaxial test can be obtained from elastic theory. The effects of frictional end restraint on the stresses in a cylinder of elastic material loaded axially between rough rigid platens were first investigated by Filon.⁵ His results, in a simplified form, have been summarized in Fig. 5. The stresses shown in Fig. 5 represent the average stresses over the diameter of a cylinder at various distances from midheight. It will be noted that at the midheight of the cylinder the average stresses approximate to the applied stresses, namely,

¹ University of the Witwatersrand, Johannesburg, South Africa.

² C. B. Crawford, "Pore Pressures Within Soil Specimens in Triaxial Compression" (see p. 192).

³ N. E. Simons, "The Influence of Laboratory Stress Path on Triaxial Test Results" (see p. 270).

⁴ G. E. Blight, "The Effect of Nonuniform Pore Pressures on Laboratory Measurements of the Shear Strength of Soils" (see p. 173).

⁵ L. N. G. Filon, "On the Elastic Equilibrium of Circular Cylinders Under Certain Practical Systems of Load," *Phil. Transactions*, Royal Society, Vol. 198 (A), 1902, pp. 147-223.

FIG. 5—Stresses in a Cylinder of Elastic Material Axially Loaded Between Rough Rigid Platens (After Filon).

FIG. 6—Theoretical Pore Pressure Profiles Calculated From the Stress Profiles Given in Fig. 4.

$\sigma_1 = Q$
 $\sigma_2 = \sigma_3 = 0$

however, these stresses are considerably higher than the applied stresses.

At the restrained ends of the cylinder, body of saturated soil subjected to shear-

ing stresses can be described by the equation

$$\Delta u = \frac{\Delta \sigma_3 + \Delta \sigma_2}{2} + \frac{A[2\Delta \sigma_1 - (\Delta \sigma_2 + \Delta \sigma_3)]}{2}$$

where A is a pore pressure coefficient for the soil.

In a triaxial specimen subjected to an applied stress change, Q , $\Delta \sigma_1$, $\Delta \sigma_2$, and $\Delta \sigma_3$ will vary down the height of the specimen. One has only to study the results of any undrained triaxial com-

A values. Some profiles calculated in this way are shown in Fig. 6.

The measured pore pressure parameter $\Delta u/Q$ has been written as

$$\frac{\Delta u}{Q} = \frac{\Delta \sigma_2 + \Delta \sigma_3}{2Q} + \frac{A[2\Delta \sigma_1 - (\Delta \sigma_2 + \Delta \sigma_3)]}{2Q}$$

A , the true pore pressure parameter for the soil at any stress ratio

$$\frac{2\Delta \sigma_1}{(\Delta \sigma_2 + \Delta \sigma_3)}$$

has been assumed to vary down the

- A. Loading, axial strain 5 per cent.
- B. Loading, axial strain 14 per cent.
- C. Unloading, axial strain 18.4 per cent.
- D. Reloading, axial strain 17.8 per cent.
- E. Reloading, axial strain 18.6 per cent.

The degree of equalization of pore pressure at 14 per cent axial strain was 40 per cent.

FIG. 7—Experimental Pore Pressure Profiles Measured During Undrained Loading of a Specimen of Saturated Clay.

pression test,⁴ (such as shown in Fig. 9), to realize that the parameter A depends a great deal on the stress ratio

$$\frac{\Delta \sigma_1}{\Delta \sigma_3}, \text{ or } \frac{2\Delta \sigma_1}{(\Delta \sigma_2 + \Delta \sigma_3)}$$

Hence A , too, will vary down the height of the triaxial specimen. If the stresses shown in Fig. 5 are assumed to apply, and if a suitable variation of the parameter A is assumed, pore pressure profiles can be calculated for different

height of the specimen in accordance with the equations

$$A_E = \alpha A_C$$

$$A = A_C \left[1 + (\alpha - 1) \left(\frac{z}{H} \right)^2 \right]$$

The subscripts E and C refer respectively to the ends and midheight of the specimen; H is half the specimen height; and z is measured from midheight toward the ends of the specimen.

Three conclusions can be drawn from Fig. 6:

1. Various combinations of true pore pressure values A_E and A_C can lead to profiles of pore pressure in which the midheight value is either greater than, less than, or equal to the values at the ends.⁶

2. The pore pressure parameter measured at the ends of a triaxial specimen does not correspond to the true pore pressure parameter A for the soil. The pore pressure parameter measured at

various stages during the undrained loading and unloading of a saturated remolded clay specimen. It is apparent that most of the types of profiles predicted in Fig. 6 can in fact occur. During loading the pore pressure at midheight was initially greater than that at the base (Profile A). At a later stage the profile was reversed (Profile B). Unloading again caused a reversal of profile (Profile C). On reloading the profiles reverted to a type similar to Profile B .

The degree of equalization of pore pressure in both tests at 10 per cent axial strain was 33 per cent.

FIG. 8—A Comparison Between the Maximum Pore Pressure Difference Measured in Triaxial Tests Using Conventional End Platens and Lubricated End Platens.

midheight in a rapid test is much closer to the true value.

3. Although the pore pressures measured in a very slow test will result in accurate measurement of shear strength parameters in terms of effective stress, the corresponding values of the pore pressure parameter A may not be accurate.

Fig. 7 shows a series of experimental pore pressure profiles measured at

⁶ It is not suggested that the values of A_E , A_C , and α used in Fig. 5 are all values likely to occur in practice. The values used are possible but were selected mainly to illustrate extreme types of possible pore pressure profiles.

Special Measures to Eliminate the Effects of End Restraint:

It appears that if end restraint due to friction could be eliminated from the triaxial test, the lot of the soil testing engineer would be much improved. Unfortunately, measures which reduce the effects of end restraint usually result in very difficult experimental techniques. Two techniques which have been used with varying success will now be described:

The Use of Lubricated End Platens—

The triaxial specimen is loaded between platens having smooth polished parallel

faces. Each end of the specimen bears against a disk of thin rubber membrane which in turn is separated from the platen by a layer of silicone grease.⁷ The specimen is located by a peg in the

used to measure pore pressures or to permit drainage in a drained test.

Fig. 8 shows the results of pore pressure measurements on two specimens of a compacted clay loaded at identical

Volumetric strains occurred in the central failure zone in a special test in which the central zone was isolated from the remainder of the specimen.

FIG. 9—A Comparison of Over-All Volumetric Strains in a Conventional Drained Triaxial Test.

center of each platen. If these pegs are made of porous material, they can be

rates between lubricated and conventional rough platens. Until an axial strain of 10 per cent was reached, the lubricated platens caused a large reduction in the maximum pore pressure difference. At larger strains end restraint

⁷The coefficient of friction between latex and polished brass lubricated with silicone grease is 0.018. Between latex and lubricated polished perspex it is 0.019.

appeared to increase. Tests on remolded soils have not been as successful as tests on compacted soils, although a significant reduction in pore pressure difference has been achieved. Reductions in maximum pore pressure difference brought about in this way allow the use of reduced testing times for undrained tests. Testing times for drained tests would not be affected.

Isolation of the Central Failure Zone—Reference to Fig. 5 shows that if the central two thirds of a triaxial specimen could be isolated from the end sections, and if measurements could be made only on this central portion, far more uniform stress conditions would apply. Isolation of the failure zone can be achieved by sealing the end one-sixth portions of a specimen in self-vulcanizing latex and then enclosing the central two thirds together with the sealed end portions in a membrane in the usual way. In undrained tests pore pressures can be measured at a probe at midheight or at the base through a hole drilled in one of the end disks and then filled with a fine saturated sand. Drainage can take place through holes in the end disks or via filter paper strips passing between the outer membrane and the sealed end disks.

Fig. 9 compares measurements of over-all volumetric strain in a conventional drained test with volumetric strains taking place in the central two thirds of a specimen with sealed disks at its ends. The soil was a heavily over-consolidated saturated clay. The rate of dilatation of the central portion was almost twice that of the specimen as a whole. This is of importance when applying a dilatation correction to failure stresses in drained tests.⁸ Although the central failure zone of the specimen tested in the conventional way was dilating, the end zones must have been compressing

or at least were dilating at a very much slower rate. These observations are similar to those made by Shockley and Ahlvin⁹ in large-scale triaxial tests on dry sand.

Conclusions:

1. The shape of the pore pressure profile set up in a rapid triaxial test depends on the pore pressure parameter A for the soil and its variation with principal stress ratio. True values of parameter A cannot be obtained from base measurements in a conventional triaxial test.

2. Elimination of the effects of end restraint allows shorter testing times to be used in undrained triaxial tests and should give truer measurements of the pore pressure parameter A .

3. The rate of dilatation measured in a conventional drained triaxial test may be considerably less than the rate of dilatation of the failure zone.

R. E. OLSON¹⁰.—This discussion is divided into two parts covering pore water pressure measurements with needles and membrane leakage.

Pore Water Pressure Measurements with Needles:

The author used needles successfully in the measurement of pore water pressures in specimens of sensitive Leda clay. It appears that the high permeability and sensitive brittle structure of such clays make them ideally suited for such measurements. Needles would appear to be ideal for use with impervious clays too, but the following difficulties are encountered:

1. The insertion of the probe and the

⁹ W. G. Shockley and R. G. Ahlvin, "Non-Uniform Conditions in Triaxial Test Specimens," *Proceedings, Am. Soc. Civil Engrs., Research Conference on Shear Strength of Cohesive Soils*, 1960.

¹⁰ Associate professor of civil engineering, Dept. of Civil Engineering, University of Illinois, Urbana, Ill.

⁸ A. W. Bishop, Correspondence, *Géotechnique*, Vol. II, 1950, pp. 113-116.

FIG. 10—Influence of Temperature on the Apparent Pore Water Pressure in a Specimen of Illite.

rigidity of the probe during the shearing stage may disturb the specimens and alter the pore water pressures. The problem appears to be particularly severe when tests are performed on highly sensitive but not brittle clays, because these clays have a very small resistance to disturbance.

2. If the specimen fails at strains above a few per cent, it appears that relative displacements between the tip of the needle and the soil may induce significant changes in the measured pore water pressure.

3. The small contact area between the needle and the soil may pose serious problems if the ambient temperature changes. If the temperature increases, the fluid in the measuring system expands. Since expansion is inhibited at the measuring end of the pore pressure line by the null indicator, the water must flow into the specimen. For relatively impervious clays, the resistance of the clay to the flow causes increased apparent pore water pressures. The resulting error is a function of the rate of change of temperature and of the contact area between the sensing element and the soil. As an example of this problem, a specimen of dispersed sodium illite was consolidated to 80 psi, and the cell pressure and pore water pressure were simultaneously increased by 40 psi and held constant for three days. Then the back-pressure valve was turned off and measurements of pore water pressure and ambient temperature were obtained for ten days (Fig. 10). These pore water pressures were measured through a 1.5-in.-diameter porous stone. The use of a pore pressure needle would have resulted in much greater changes in apparent pore water pressure because of the smaller contact area between the probe and the specimen.

Membrane Leakage:

The author attempted to measure

leakage of cell fluid through thin rubber membranes and concluded that membrane leakage was not a serious problem for short-term triaxial tests. He noted that doubling the membrane had no apparent effect on the rate of leakage.

The writer has obtained measurements of leakage through 0.002-in.-thick commercial membranes by using a 1.5-in.-diameter by 3.00-in.-high Norton P2120 porous stone in place of the soil specimen in a triaxial device and measuring the apparent volume changes under constant cell pressure. The membranes were bound to the base pedestal using rubber bands and to the top cap using two rubber O-rings. Measurements were taken for periods up to six months to ensure representative leakage rates. The following are typical observations:

1. Using a single membrane with distilled water on both sides and with the same amount of dissolved gas in the water on both sides, the leakage rate varied from 0.002 ml/day at a cell pressure of 10 psi to 0.008 ml/day at 75 psi.

2. With distilled water in the cell, 1-N CaCl_2 in the stone, and a cell pressure of 20 psi, the leakage rate was 0.090 ml/day for a single membrane and 0.026 ml/day for two membranes separated by a thin layer of silicone high-vacuum grease.

3. Using a single membrane and 1-N NaCl in the stone, the leakage rate varied from 0.032 ml/day at a cell pressure of 5 psi to 0.050 ml/day at 80 psi.

4. For comparison, direct nitrogen leakage through two membranes at a cell pressure of 100 psi was 720 ml/day with no grease and 500 ml/day with the membranes separated by a thin layer of silicone high-vacuum grease. For nitrogen in solution in water, the leakage rate was of the order of 50 ml/day.

It appears that membrane leakage can be reduced to about 0.002 ml/day if two membranes separated by silicone

high-vacuum grease are used, provided that the concentration of dissolved gas and dissolved salt is the same in the cell fluid as in the pore fluid of the specimen and provided that cell pressures less than 120 psi are used. If lower leakage rates are desired, it appears that mercury should be used as the cell fluid. These conclusions are in substantial agreement with those expressed by the author.

WERNER E. SCHMID¹¹—The careful experiments and research on the pore pressure variation within triaxial specimens by Crawford are rather interesting. His paper, as well as the one by G. E. Blight, appears to be one of the more significant developments of this symposium for the proper assessment of the shearing strength of clay soils.

From 1957 to 1959 we measured pore pressures in a series of triaxial cell tests at various locations inside a test specimen. The measurements were carried out by using commercially available pneumatic relays that were modified for pore pressure measurement. They established automatically a no-flow counter pressure equilibrium and indicated the pressure required to prevent flow on an ordinary bourdon gage. Since these relays were small and cheap, a number of them could be used simultaneously. Thus a set of twelve such relays were arranged to measure pore pressures at three different radial distances and at four different heights of a triaxial test specimen 8 in. in diameter and 20 in. high. The results obtained¹² were rather interesting:

1. The pore pressures measured varied from point to point in a similar manner as reported by Crawford and Blight.

2. Pore pressures varied not only from

point to point but also with time and with the time rate of loading.

3. Pore pressures were also a function of the strain.

4. The pore pressures were always measured with a certain time lag observable by the fact, for example, that an instantaneous increase in the cell pressure $\Delta\sigma_3$ required a certain time before the pore pressure apparatus indicated a corresponding increase in Δu . This time lag was only partially a function of the over-all compliance of the measuring system.

As a consequence of all this we considered it quite tenuous to assume, as is usually done, that the value of the pore pressure u , wherever, whenever, and by whatever means observed (usually at the porous stone end of a specimen) is—*eo ipso*—the correct value for the proper determination of the shear strength parameters of a clay soil.

The problems and questions raised by our observations were so serious that, in our opinion, they drew the reliability of the pore pressure approach into serious doubt. It would have been easier perhaps to suppress our misgivings and calm our conscience by assigning all the inexplicable variations and discrepancies to the variation of Skempton's pore pressure coefficient A . Skempton in 1954¹³ derived a relationship between the pore pressure change Δu and the accompanying changes in total stress as:

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \dots (1)$$

According to Skempton $B = 1.0$ for saturated soils. If this formula was valid in our case the coefficient A would have had to vary from point to point within the test specimen and also from instant to instant during the test, because with $\Delta\sigma_3 = 0$ for a saturated clay, Eq (1) reduces to $\Delta u = A\Delta\sigma_1$ or $A = \Delta u/\Delta\sigma_1$.

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¹² W. E. Schmid, Y. Klausner, and Ch. F. Whitmore, "Rheological Shear and Consolidation Behavior of Clay Soils," *Progress Report to the Office of Naval Research*, Princeton, December, 1960.

¹³ A. W. Skempton, "The Pore Pressure Coefficients A and B ," *Geotechnique*, Vol. IV, No. 4, pp. 143-147.

Thus the pore pressure coefficient A is nothing but the pore pressure measured as a fraction of the applied stress $\Delta\sigma_1$, and since Δu usually varies not only with stress but also with strain and time and, as we have observed, also with the location in the test specimen, it really cannot be considered more than a convenient function that may vary arbitrarily in such a fashion as to match the experimentally observed pore pressures u with the applied stress. A function that varies in such a fashion is deceptive because it permits to establish an apparent relation between any two arbitrary variables—however related or unrelated these may be. More than anything else, therefore, this pore pressure coefficient to us appeared as one of those “fudge factors” that are always necessary to reconcile inadequate theories with experimental data.

Several attempts to get around our dilemma, such as averaging in various combinations, were unsatisfactory, and we finally decided to investigate whether there were alternate methods of assessing shear strength—if possible without having to measure pore pressures. We soon found that the requirement of knowing the pore pressure resulted from uncritically accepting the Mohr-Coulomb failure hypothesis for clays, and that this was not at all the one and only possible hypothesis available. Among several others there were at least two, namely the Tresca and the von Mises hypotheses, that were promising. In the course of our investigations we became more and more convinced that the undue emphasis on pore pressure measurements was not at all advancing shear strength research but actually was stultifying it.

To avoid any misunderstanding, we find it necessary to clarify one point: We do not claim that pore pressures do not exist (as we are sometimes accused), nor do we claim that they are worthless

under any and all circumstances. Field values of pore pressures may be invaluable for assessing the degree of consolidation of a clay layer and from that estimating the resulting shear strength.

What we do claim is that the efforts necessary to assess the shear strength of clay soils on the basis of the effective stress principle with pore pressure measurements in the laboratory are not commensurate with the accuracy, the reliability, and the usefulness that results, considering all the assumptions and uncertainties that are inherent to this method. These conclusions were reached as a consequence of the relatively great variability of the pore pressures that were observed within a test specimen throughout the loading process. It is therefore a great satisfaction that other researchers like Crawford and Blight report similar experiences in pore pressure measurements as we have observed. Perhaps our arguments will now find a more open-minded audience.

In our search for a fresh approach to the shear strength problem we found that soil engineers had inherited many habits of thought and action or had accepted them without critical analysis. It is refreshing to observe how more and more people now are suggesting to rethink the problem from the very beginning.

We are gratified to see Messrs. Roscoe, Schofield, and Thurairajah state, as we have done earlier,¹² that the extended von Mises or Tresca failure criteria are more relevant than Mohr-Coulomb. We only wish to add here that they are also less cumbersome in their application to practical design problems.

Mr. Crawford states that pore pressure measurements taken at various points within a soil specimen are confusing. We agree. With that much evidence to the contrary it is difficult, however, to accept his hypothesis that it seems reasonable

to assume that the pore pressures on the failure plane and those at the base plate are almost equal. This appears to be a typical argument to brush the inconsistencies under the rug: Why get confused? Let us measure pore pressure at the base and assume that all is well. Nobody can get confused that way.

Unfortunately, in the research field of shear strength of clay soils we find a seemingly unalterable, generally accepted dogma about the validity of the Mohr-Coulomb hypothesis. Actually this hypothesis, as it is applied, is a strangely construed mixture of experimental observations, assumptions, and speculation. We believe one of the most urgent requirements to clear up the problem would be to separate fact from fiction.

A detailed discussion of a more promising shear strength theory would be beyond the scope of this discussion.^{14,15} We may, however, just summarize some points that are essential:

1. Clays in general are visco-elastic-plastic materials and their load-deformation behavior is time-dependent. The theory that satisfactorily permits to describe this behavior, therefore, must be a rheological one.

2. We must recognize that the shear strength of a clay soil is also a rheological quantity, that is, that it is not a unique value but it runs through a spectrum of values depending on the rate of loading.

3. The von Mises or Tresca failure conditions are independent of any normal stress and therefore do not require any pore pressure measurement, nor do they require the artificial separation of shear strength into a cohesion and a friction component. These components in any event always have to be recom-

bined in the solution of practical problems.

4. We must recognize the fact that, when a given soil changes from one effective stress to another effective stress, it really changes into a different engineering material.

5. The shear strength can be expressed in analogy to the old Mohr-Coulomb hypothesis by using the concept of the intrinsic stress such that $s = p_i \tan \phi^*$ where $\phi^* = (\sigma_i + \sigma - u) \tan \phi^*$ where p_i is the intrinsic stress and exists between the soil grains in the absence of any pore pressure or exterior stress.¹⁶

Finally we wish to commend the careful experimental work by Mr. Crawford. Only those who have ever measured pore pressures in the laboratory can appreciate the diligence required to produce results as Mr. Crawford has done.

CARL B. CRAWFORD (*author's closure*)—Until recently some of the unusual features of triaxial test results were assumed to be caused by membrane leakage. Professor Olson's extensive test results are therefore a welcome corroboration of the writer's observation that leakage through the rubber membrane is not a serious problem for ordinary tests. Leakage due to improper seals can, of course, be very serious.

Professor Olson's study of gas diffusion is also of special interest. Gas diffusion can be just as serious through plastic hose as through the membrane, and it should not be used in the pore pressure system without checking this property. A simple test of plastic hose is to fill a short length with de-aired water, seal both ends, and observe it for a few days. Usually air voids will begin to appear within a few hours.

¹⁴ W. E. Schmid, "New Concepts of Shearing Strength for Clay Soils," Part 1, *Sols—Soils*, Vol. 1, 1, Paris, 1962, pp. 31–42.

¹⁵ W. E. Schmid, "New Concepts of Shearing Strength for Clay Soils," Part II, *Sols—Soils*, Vol. 1, 2, Paris, 1962, pp. 19–26.

¹⁶ W. E. Schmid and S. Kitago, "Shear Strength of Clays and Safety Factors as a Function of Time," *Proceedings*, Sixth International Conference ISSMFE.

Now that pore pressure variation within triaxial specimens is a well established phenomenon, it is probable that much research will be devoted to it, and the problems with the use of probes, as cited by Professor Olson, will require continuing attention. The problem of inserting a needle into the undisturbed sensitive Leda clay is probably not nearly so serious as it is in most remolded soils, because the tiny hole required is made very easily with a drill (slightly smaller than the needle), and this appears to have little effect on the relatively strong undisturbed specimen. The limited number of tests with needles showed no tendency to lower strengths.

The second and third problems noted are difficult to assess and will require further study. In the tests described in the paper the cell pressure and the pore pressure systems were increased simultaneously and after standing at equilibrium for a short time the drainage was opened, giving results such as those shown in Fig. 3. Attempts to follow the pore pressure change resulting from increments of ambient pressure were generally unsatisfactory due to plugging of the needle. This suggests an outward flow which could not be overcome. The method can only be evaluated, therefore, by judging the performance during consolidation and shear. Obviously improvements in the technique are required, but the measurements made at present do not appear to be unreasonable.

Temperature influences on measured pore water pressure must be of considerable importance as indicated by Professor Olson and other contributors to the conference. This is another factor in testing that must have more attention in the future, especially for long-term tests during which substantial temperature changes occur.

Professor Olson recognizes the need

for internal pore pressure measurements in order to reduce testing times. The writer has tried to indicate in discussion of Dr. Blight's paper¹⁷ that the pore pressure variations in a sensitive clay are due largely to time-dependent structural collapse, and if the test is to represent undrained shear, it may be impossible to achieve pore pressure equalization with the base plate at practical rates of shear. Nevertheless the base measurements quoted in the paper are considered to be satisfactory because the shear plane is so near to the base.

Dr. Blight has produced valuable information regarding the pore pressure and dilatancy variation within triaxial specimens and has demonstrated the influence of end restraint on pore pressures. His Fig. 5, however, appears to be an oversimplification, since the average stress, σ_1 , over the diameter at the base of a cylinder must equal the applied stress, Q . The vertical stress at the edge of the base may be greater than the applied stress.

Some confusion may result from Dr. Blight's reference to the variation in parameter A at various levels in the specimen. Parameter A is a reflection of the dilatancy characteristics of the soil and should be the same at equal levels of stress and strain for a uniform soil. That it varies is due to the inequalities of stress and strain at any particular time during the test, and it is suggested that it is more correct to emphasize this fact.

Professor Schmid doubts that it is reasonable to assume that pore pressures on the failure plane are almost equal to those measured at the base plate. This was deliberately stated with caution in the conclusions because of the many uncertain influences, even though Fig. 3 shows it to be true for typical tests. The purpose of the study was to

¹⁷ See p. 173.

evaluate this very question, and the results may be interpreted in a variety of ways depending on the reader's point of view.

Professor Schmid finds most of the common shear strength concepts applied to soils to be unacceptable. The writer's only response to this is that the alternatives are, at the moment, less acceptable. The problems of stress, strain, and pore pressure variations do not disappear with the rejection of Mohr-Coulomb or by abandoning the

triaxial shear device, although both of these possibilities merit study. It is probably true that most research workers relied too heavily on base measurements of pore pressure during the time of the London Conference in 1957, when the effective stress concepts were first developing on a practical basis. But this situation was corrected quickly, and by 1960 many references could be found to the possible errors in accepting base measurements, and this is now a subject of intensive study.