DIRECT-SHEAR TEST FOR EFFECTIVE-STRENGTH PARAMETERS[a]

Discussion by William W. Moore, and John H. Schmertmann

WILLIAM W. MOORE,[23] F. ASCE.—The data presented by O'Neil focus attention on an important aspect of shear strength determination which, in the writer's opinion, has been too frequently overlooked in soil mechanics practice. Specifically, there appears to have been far too much reliance placed on peak shear strength, without recognizing the possibly critical significance of time-rate of loading and of pore-pressure conditions created during the test. The effects of time-rate of loading have been described by several other investigators.[24,25,26,27,28,29]

Certainly, one of the practical factors that has led to the common use of peak strengths in quick tests has been the long time required and the relatively high costs of making slow or completely drained shear tests or tests with reliable pore-pressure measurements. The writer has found it helpful to make direct shear tests, and sometimes other types of shear tests, with various rates of loading or with stress increments of loading in order to evaluate the stresses that may produce continuing progressive rupture. Experience of the writer's firm has indicated that the stress capable of producing slow failure under long-time stress for different soils in quick tests.

A practical example of potentially dangerous results based on peak strength tests may be demonstrated by stability analyses for cohesive soils in which lower stresses are capable of producing progressive deformation and, in some cases, even ultimate failure under prolonged loading. For such soils, it is often difficult to obtain a correct selection of shear strengths on the basis of yield-point values, or estimated effective stresses, without going to the rela-

a August 1962, by Hugh M. O'Neil (Proc. Paper 3232).

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24 "Final Report to U. S. Waterways Experiment Station on Investigation of Effect of Long-Time Loading on the Strength of Clays and Shales at Constant Water Content," by A. Casagrande and S. D. Wilson, Harvard Univ., Cambridge, Mass., 1949.

25 General report for discussion of "Laboratory Investigation, Including Compaction Tests, Improvement of Soil Properties," by E. C. W. A. Genze, Proceedings, 3rd Internatl. Conf. on Soil Mechanics and Foundations Engrg., Zurich, 1953, Vol. 3.

26 "Soils," Chapter 10 of "Building Materials, Their Elasticity and Inelasticity," by A. W. Skempton and A. W. Bishop, North-Holland Publ. Co., Amsterdam, 1954.

27 "Some Deformation Characteristics of Cambridge Gault Clay," by W. J. Thompson, thesis presented to the University of Cambridge, England, in 1962, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

28 "Dynamic and Static Resistance of Cohesive Soil," by W. S. Housel, Special Tech. Publ. No. 254, ASTM, 1959.

29 "The Strength of Compacted Cohesive Soils," by H. B. Seed, J. K. Mitchell, and C. K. Chan, ASCE Research Conf. on Shear Strength of Cohesive Soils, Boulder, Colo., 1960.

tively costly methods of consolidated triaxial or direct shear tests, or tests with quite complex and difficult laboratory pore-pressure measurements.

In some soil types, it is difficult to interpret graphs of the test results so as to determine accurately the stress that would represent an incipient yield-point condition. This is the factor that is intended to be evaluated by the method proposed by W. S. Housel[28] of plotting the rates of shear movement for stress increments above a yield point. This can be done by graphical interpretation of the test data leading to selection of a stress value that may be expected to resist slow progressive deformation.

The test technique described by the author appears to provide a tool whereby the practicing engineer might obtain a reliable determination of the effective stresses and strengths within test time intervals and laboratory cost factors that should compare favorably with more conventional quick test methods.

It will be most interesting to study the results of additional experience with the use of the R_0 test suggested by the author. On the basis of available information, it is the writer's expectation that the suggested technique will prove to be one of the most important advances in shear testing tenchiques that has been offered in many years. The method proposed apparently offers the possibility of rapidly determining the effective shear stresses in cohesive soils, which are those soils wherein the effects on the apparent shearing strength produced by varying speeds of shear movement and laboratory test conditions are most important. These effects of transient pore pressures and time-rate of deformation have no doubt introduced large and sometimes dangerous errors in using peak values from conventional strain-control test methods for certain project conditions.

JOHN H. SCHMERTMANN,[30] A. M. ASCE.—The R_0 test will undoubtedly increase in popularity, both because of its economy and because interest in the direct shear test itself will be renewed as the author's and other improvements in the design of direct shear equipment evolve into general practice.

It is well to remember the underlying assumption of the R_0-test. Reworded, it is that the internal changes in effective stress in an undrained, constant-volume test can be completely replaced by external changes in a drained, constant-volume test. This is only approximately true in the direct shear apparatus because internal changes are isotropic whereas the external are anisotropic—but perhaps the approximation is ordinarily a good one. In this respect, the triaxial test apparatus is superior for a R_0-test. It also follows that the R_0-test is not suitable for partially saturated soils because the internal changes in effective stress will not be accurately reflected by the external pressure changes necessary to maintain constant volume. Nor will the test be suitable for saturated soils wherein the magnitude of the pore pressure can influence structural changes which in turn influence effective stress, but such behavior is probably rare.

Although the direct shear test does approximate plane strain, it is well known that the shear-strain distribution is nonuniform because the deformations are concentrated in an undefined zone. The volume change characteristics of this shear zone may be somewhat different than the gross behavior

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of the entire specimen. Thus, pore pressures in this critical zone may or may not be somewhat different than u_e obtained from the R_0-test; this is not known at present and the point requires future investigation.

EPK Kaolinite for Standard Clay?—O'Neil used kaolin obtained from the Edgar Minerals Corp., Gainesville, Fla., as one of his test clays. The writer has used this clay extensively and takes this opportunity to suggest that perhaps it would make an ideal standard clay—much as Ottawa sand is often used as a standard sand. The following outline details this suggestion. Because the author now also has experience with this clay, his comments would be valuable.

Advantages of having a standard clay (this is also recommended elsewhere[31]);

1. Better communication of experimental results, because the nature of the clay would be a matter of common understanding.

2. Routine work can be avoided because such data would become available in the literature.

3. It would facilitate common, or associated, efforts between laboratories. This is now often difficult because of limited clay supply, disagreement on clay to be investigated, or reluctance to trouble another investigator with requests and explanations. Perhaps agreement on at least one clay (basic kaolinite, which can be obtained easily, cheaply, and independently) would be desirable and practical.

Suitability of EPK kaolinite:

4. General

a. It is readily available by direct order from EPK.Co., in convenient powdered form. It is inexpensive (approximately $7.50 per 50-lb bag plus shipping). This clay has proved exceptionally uniform for the 70 yr it has been mined at this location.[32] (It should be noted that the writer has no financial or other interest in this company. Such small orders would be of negligible business value and constitute a special service by the company.)

b. Remolded specimens have many of the behavior characteristics of undisturbed clays (such as preconsolidation breaks in the e-log-p curve, sensitivity, and development of failure planes), offering the uniformity of the former and the reality of the latter.

c. Remolded specimens are easily prepared in a near-saturated condition (99.5+%) by adding water to the kaolin powder and vacuum extrusion, such as by "Vac-Aire" machine.[33]

d. In the as-received condition, this kaolin is approximately 95% kaolinite—the simplest and best understood clay mineral. It is a particularly pure and well crystallized kaolinte.[34] Its specific gravity is near the theoretical value for kaolinite (2.609).

31 Discussion by W. Schaad and L. Bjerrum, Proceedings, 2nd Internatl. Conf. on Soil Mechanics and Foundations Engrg., Rotterdam, 1948, Vol. VI, p. 79.

32 "Kaolinitic Sediments in Peninsular Florida and Origin of the Kaolin," by E. C. Pirkle, Economic Geology, Vol. 55, No. 7, November, 1960, p. 1382.

33 "De-Aired Extruded Soil Specimens for Research and for Evaluation of Test Procedures," by H. Matlock, Jr., C. W. Fenske, and Raymond F. Dawson, Bulletin No. 177, ASTM, October, 1951.

34 "Fundamental Factors Affecting the Deformation Behavior of Clay-Water Systems, I. Raw Material Studies," by C. W. Ormsby, Report No. 5658, Natl. Bur. of Standards, U. S. Dept. of Commerce, Washington, D. C., November, 1957.

e. When prepared with as-received powder, its LL and PI of approximately 52 and 21, respectively, place it near the mid-point between clays of low and high plasticity; so perhaps it can be considered an average cohesive soil with regard to plasticity. Its activity is 0.35.

5. Use of chemical additive: This kaolin is responsive to additives such as the sodium polyphosphate dispersants. Its engineering properties can be varied over a wide range by the before (during remolding) or the after (leaching) addition of these chemicals in trace amounts. For example, mixing the as-received kaolin powder with a 0.04 M solution of $Na_6P_4O_{13}$ (commercially known as "Quadrofos") instead of distilled water causes the following changes:

a. The structure changes from flocculated to dispersed.

b. The PI decreases from 21 to 4.

c. The thixotropy after extrusion increases from none to significant.

d. The permeability decreases from approximately 2×10^{-8} cm per sec to 6×10^{-9} cm per sec at the same void ratio (0.75).

e. The compression index decreases from approximately 0.50 to 0.05.

f. The sensitivity increases from 2 to high values (if additive mixed by leaching).

g. The stress-strain characteristics change to make the clay more brittle and to have an increased tendency to form failure planes.

h. The proportion of secondary to primary consolidation increases, as does swelling tendency. Both secondary and swelling effects are not pronounced and are slight in the untreated clay.

6. The action is the same, but the effects can be reduced, if a more dilute solution of "Quadrofos" is used or if a weaker additive (such as $Na_2HPO_47H_2O$) is used.

7. Because the kaolin is received in powdered form, the properties of prepared specimens can also be varied easily by mixing the powder with other granular components. For example, such components could be bentonite powder to produce a swelling tendency, graded sand to simulate a more typical compacted soil, or the commercially available quartz silts to produce clayey silt.

8. The comparative high permeability of this clay offers the following important practical advantages:

a. Ease of saturation.

b. Rapid consolidation.

c. Rapid pore-pressure response.

d. Ease of leaching operations.

9. Pleasant advantages for those who must work with the clay in the laboratory are its aggreeable white color, lack of odor, and the ease with which it can be trimmed and cleaned.

Most of the preceding would also apply to other commercial kaolin powders. However, because kaolins may vary significantly from one source to another, if a standard is to be developed, it is necessary that it be obtained from only one source, as is Ottawa sand. Edgar kaolin seems to be particularly suitable.

The small differences reported by O'Neil between his values for the kaolin's Atterberg Limits and effective stress envelope and those by the writer and Osterberg are most probably due to experimental differences rather than a

difference in the two batches of kaolin. It is interesting to note one agreement. O'Neil's value of 27° for the ϕ'_e value of the kaolin agrees well with a range of 24° to 27.5° determined in triaxial tests, after $(\sigma'_1/\sigma_3)_{max}$, by two other methods.[35]

[35] Discussion by J. H. Schmertmann of "Friction and Cohesion of Saturated Clays," by T. H. Wu, A. G. Douglas, and R. D. Goughnour, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM1, February, 1963.