FRICTION AND COHESION OF SATURATED CLAYS[a]

Discussion by Vojtěch Mencl, John H. Schmertmann, and R. C. Bea

VOJTĚCH MENCL.[25]—The concept of the authors to find the proportions of cohesion and internal friction in the strength of a saturated clay is based on the assumptions that (1) all specimens of a saturated cohesive soil having

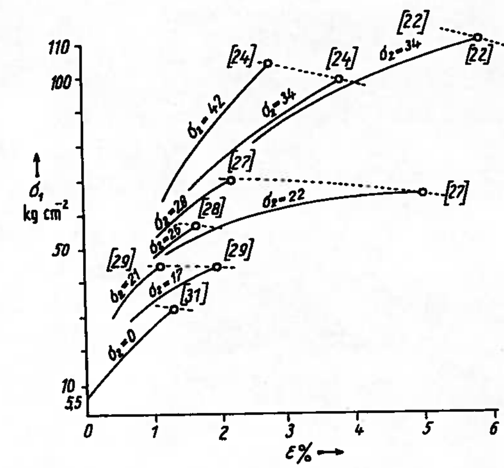

FIG. 24.—NEOGENE CLAY STONE: STRESS-STRAIN CURVES AND LINES OF EQUAL STRENGTH

the same density have also the same true cohesion, and (2) that condition 1 applies for equal percentage of strain. The writer wishes to demonstrate that assumption 2 may not be fulfilled in general. It can be proven that it is not fulfilled for cohesive soils in which the bonds of cementation prevail over those caused by molecular forces.

In order to find the proportions of cohesion and internal friction of clay stone (strength in simple compression of 31 kg per sq cm), the writer tested the specimens in triaxial apparatus, but not to failure. After the tests, the confining pressure, σ_2, was removed and the specimens were subjected to a

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[a] June 1962, by T. H. Wu, A. G. Douglas, and R. D. Goughnour (Proc. Paper 3158).
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simple compression test. The typical values in kg per cm^{-2} of strength reached in these tests are given in brackets at the ends of the stress-strain curves in Fig. 24 and the curves of constant strength in these tests are drawn (dotted lines). These additional tests show to which degree the integrity, that is, cohesion, of the material has been destroyed during the triaxial test, a phenomenon termed the Bauschinger effect in the theory of strength of materials.

Thus, if cohesion is defined as that part of strength that follows from the integrity of the material, its value can be found by using the Bauschinger effect. The remainder of the strength reached during the triaxial test is then due to internal friction. If so, Fig. 24 shows that the cohesion of specimens subjected to the same percentage of strain decreases with increasing confining pressure, a phenomenon that can probably be caused by the injury of the bonds of the material either in the sense of Griffith's theory or by the "vagabundierenden Spannungen" introduced by M. Rŏs.[26]

JOHN H. SCHMERTMANN,[27] A. M. ASCE.—The subject is of fundamental importance for a better understanding of the engineering behavior of cohesive soils. For the first time, data have been published relating the Hvorslev effective components, c_e (authors' C_e) and ϕ_e, and those determined by means of variations of the CFS test.[13,28] This is particularly interesting to the writer and the following discussion is primarily speculation on the relationship between these two sets of components. Also mentioned is the potential superiority of using methods of determining cohesion-friction-strain behavior that require only a single specimen and the possible special importance of $\bar{\sigma}_1$ in cohesion. Supplementary data are also offered.

Effective and CFS Components.—Hvorslev[5,11] succeeded a long time ago in determining soil strength components which he called "effective cohesion, c_e" and "effective friction, φ_e." Hvorslev formulated the results of experiments and the resulting equation had the simple Coulomb form, thus defining c_e and ϕ_e. Because these components were based on strength behavior at different effective stresses but equal void ratio on the failure plane at failure, and c_e at a given void ratio appeared independent of stress history, they have also become known as the "fundamental" or "true" components of soil strength. The experimental method of determining c_e and ϕ_e from normally consolidated and overconsolidated specimens of a soil was subsequently developed by Hvorslev and Karl Terzaghi, Hon. M. ASCE. A refinement of this is the authors' Method 1. The original method also serves to demonstrate the physical meaning of these components. Other methods are described by L. Bjerrum.[29]

It is only comparatively recently that there has been some question of the uniqueness of the c_e and ϕ_e components. They are usually determined using the $(\sigma_1 - \sigma_3)_{max}$ values from each test involved in the determination and,

26 "Versuche zur Klärung der Frage der Bruchgefahr," by M. Rŏs and A. Eichenger, E. M. P. A., Zurich, 1926.

27 Assoc. Prof. of Civ. Engrg., Univ. of Florida, Gainesville, Fla.; on leave from the Norwegian Geotech. Inst., Oslo, Blindern, Norway.

28 "Cohesion After Non-Hydrostatic Consolidation," by J. H. Schmertmann and J. R. Hall, Jr., Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 87, No. SM 4, Proc. Paper 2881, August, 1961, pp. 39-60.

29 "Theoretical and Experimental Investigations on the Shear Strength of Soils," by L. Bjerrum, Publication No. 5, Norwegian Geotech. Inst., 1954, Oslo, Norway.

heretofore, it has often been tacitly assumed that both components also represent the maximum values of the components. The recent research into the pore pressure-strain and stress-strain properties of cohesive soils has led some investigators to re-examine the uniqueness of the effective components.

The Skempton "A" pore-pressure parameter has consistently been found to be an important, and sometimes dramatic, function of strain and overconsolidation ratio. This suggests that the important details of particle-to-particle geometries and the physical conditions at their contact zones, or simply "soil structure," is also an important function of strain and overconsolidation ratio. With soil structure a function of strain, it is but a small step to realize that fundamental cohesion and friction must also be functions of strain. The uniqueness of c_e and ϕ_e then depends on the uniqueness of soil structure at $(\sigma_1 - \sigma_3)_{max}$. There seems to be no conclusive reason to believe that this is the case; for instance, there is considerable evidence that soil structure at $(\sigma_1/\sigma_3)_{max}$ is a physically more unique situation in saturated soil sheared without drainage. If cohesion and friction are functions of strain, then designers are faced with another difficulty. The normally consolidated and overconsolidated specimens can reach maximum $(\sigma_1 - \sigma_3)$ at different strains. Then, to what strain in other laboratory specimens, or the soil in the field, do the c_e and ϕ_e values belong? This question is discussed subsequently.

Hvorslev's original work was with two remolded clays which he tested under drained conditions in stress-controlled direct and torsion shear machines. Maximum $(\sigma_1 - \sigma_3)$ and $(\bar{\sigma}_1/\bar{\sigma}_3)$ occurred simultaneously in his drained tests. Again referring to Hvorslev,[11] Fig. 35 shows what he refers to as typical stress-twist curves for the two clays, obtained from a torsion machine. The twist angle can be converted to shear radians by his formula (Eq. 41), which assumes uniform strain distribution. If the same assumption is made for the triaxial test specimen, then the axial compressive strain that produces a given shear strain on the failure plane can be computed.[30] The approximate translations from Fig. 35 into equivalent undrained triaxial strains at failure for the Vienna and Little Belt clays, respectively, are: Normally consolidated, 40% and 26%; overconsolidation ratio of 5, 4%, and 9%. It seems that these clays failed only after considerable strain.

Perhaps, when comparing normally consolidated and overconsolidated specimens at equal void ratio at failure, Hvorslev was also comparing them at $(\sigma_1/\sigma_3)_{max}$ and at large strain—strains perhaps large enough essentially to destroy the different structural effects of different stress history. The comparison was then at essentially the same structure and, under these strain conditions, the void-ratio value was a good parameter of soil structure. Equal void ratio meant equal structure. Strength comparisons at equal structure but different effective stresses then permitted the successful separation of the fundamental components.

However, what about strains other than large values? As lesser strains are considered, the difference in soil structure between normally consolidated and overconsolidated specimens becomes greater—even at the same void ratio. This seems clearly evidenced by their progressively (toward lesser strain) greater difference in pore-pressure behavior in constant volume tests. Method

30 "A Study of Shear Failure in Certain Tertiary Marine Sediments," by James P. Gould, Research Conf. on Shear Strength of Cohesive Soils, ASCE, 1960, p. 631.

1 is consequently unsuitable for the determination of the fundamental components at a particular strain other than a large one. A more detailed treatment of the argument leading to this conclusion can be found in a paper by the writer.[31]

On the basis of Hvorslev's work, together with the subsequent work of many who, in different ways, showed the importance of soil structure, the writer proposed definitions of cohesion and friction that were more general, in that strain is considered as an independent variable. To emphasize this, a c_ϵ and ϕ_ϵ notation is used. The CFS test was then developed as a practical means to measure the proposed components. As in Hvorslev's studies, the effective stress is varied at essentially constant soil structure and the resulting different magnitudes of shear resistance are studied and components separated.

However, the equal void-ratio requirement for equal structure is modified and structure is considered to be still essentially constant after a slight (less than 1%) variation in void ratio. Accepting this permits the investigator to impose small differences in effective stress between two or more initially identical specimens compressed to the same strain. Instead of specimens at equal void ratio, they are at slightly different void ratio but at nearly equal overconsolidation ratios. This is believed to be much better assurance of essentially constant structure at a given strain. With the CFS definitions, cohesion is not directly related to void ratio.

As the authors explain, they used variations of the CFS test. Thus, their conclusions are tied to the component definitions connected with the CFS test. Their use of the c_e and ϕ_e notation over the entire strain range tends to cloud the preceding point.

The introductory study by the authors showing the check between the results of Methods 1 and 2 is interesting and valuable. It is obviously of considerable importance to demonstrate the meaning of c_ϵ and ϕ_ϵ in already familiar terms. If it can be shown that $c_\epsilon = c_e$ and $\phi_\epsilon = \phi_e$ at large strain, which from the preceding discussion seems a reasonable possibility, then confidence in the use of c_ϵ and ϕ_ϵ at other strains should be greatly increased. The authors have demonstrated this equality for the first time. Their confidence reached the state that they used the c_e and ϕ_e notation at all strains rather than just the ϵ = 10% that they investigated. This may be premature. However, the writer can present some data in support of the possibility that they are equal at large strains after reaching $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$.

Additional Data Showing That $c_e = c_\epsilon$ at Large Strain After $\bar{\sigma}_1/\bar{\sigma}_3$ Failure.—Fig. 25 presents the results of a comparative determination of the Hvorslev effective and the CFS test components. All tests are on machine extruded duplicate specimens of commercial kaolinite mined at Edgar, Fla. The CFS tests are Nos. 254 and 242. The tests used to determine the effective components are Nos. 238, 240, 241, 250, and the higher $\bar{\sigma}_1$ curve from 254. For all tests, the calculated void ratios varied from 0.880 to 0.900 at the strain of the component calculation. The corresponding range in $\bar{\sigma}_e$, determined from a well-defined, one-increment, hydrostatic, virgin consolidation curve, is 3.8 kg per sq cm to 2.8 kg per sq cm. Additional numerical information relating to spec-

31 "An Experimental Study of the Development of Cohesion and Friction with Axial Strain in Saturated Cohesive Soils," by J. H. Schmertmann, thesis presented to Northwestern University at Evanston, Ill., in 1962, in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

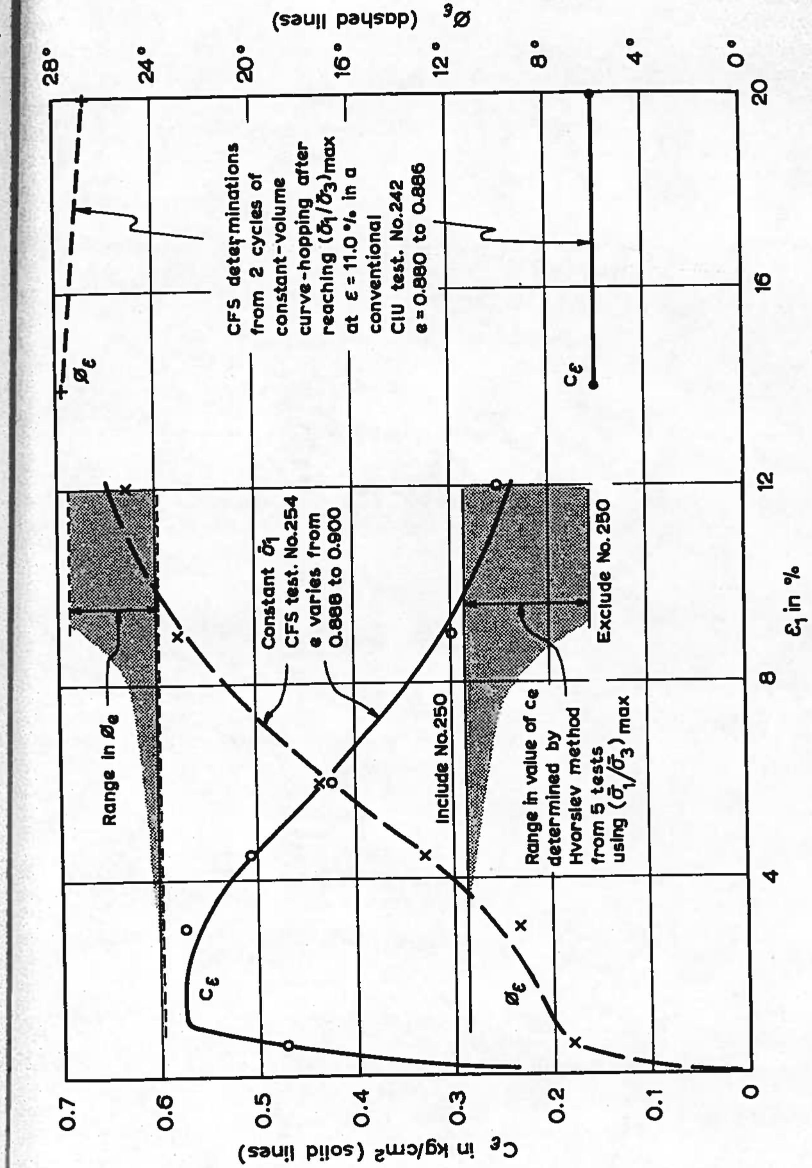

FIG. 25.—COMPARISON OF HVORSLEV EFFECTIVE COMPONENTS WITH RESULTS OF CFS TESTS

imen uniformity, consolidation pressures and times, and stresses and strains at $\bar{\sigma}_1/\bar{\sigma}_3$ failure is presented in Table 9.

Test No. 254 is a "standard" constant $\bar{\sigma}_1$ CFS test with the specimen normally consolidated to $\bar{\sigma}_c$ = 3.50 and curve-hopped between the values of 3.30 kg per sq cm and 2.60 kg per sq cm. Individual cohesion and friction values computed from this test are shown, as well as the curves based on these points. This test appeared to be essentially at $\bar{\sigma}_1/\bar{\sigma}_3$ failure at the final 12% strain.

Test No. 242 ended as a one-specimen version of the authors' Method 2. It began as a conventional Consolidated-Isotropically-Undrained(CIU) test with pore pressure measured. However, after reaching $\bar{\sigma}_1/\bar{\sigma}_3$ failure at a strain of 11.0%, the test was continued, at the same controlled strain rate, as a constant-volume CFS test. In this version of the CFS test, the curve-hopping is done between the undrained volume of the specimen and a controlled increase in volume (in this case, Δe = 0.006) forced on the specimen by a temporary increase in pore pressure. After the required volume change occurs, the pore-pressure control line is closed, time is allowed for pore-pressure equilibrium to be established again, and undrained pore pressures are again measured (this time with the specimen volume slightly increased). Effective stresses are then slightly, but significantly, different. The decrease back to the initial

TABLE 9

Test No.	238[a]	240[a]	241[a]	242	250	254
Initial condition of specimens:						
void ratio	1.026	1.022	1.021	1.016	1.020	1.015
% saturation	99.8	99.7	99.3	100.1	99.4	99.3
Consolidation pressure (kg per sq cm):						
normal	3.0	4.0	5.0	3.5	7.0	3.5
rebound		2.0	1.0		1.0	
Times (in minutes)						
total normal	1,330	1,140	1,220	1,090	1,340	51,650
primary	85	80	100	85	65	55
rebound		1,130	1,120		11,560	
At $(\bar{\sigma}_1/\bar{\sigma}_3)$ maximum						
$\bar{\sigma}_1$	2.91	3.22	2.98	3.14	1.00	3.30
$\bar{\sigma}_3$	0.89	0.99	0.90	0.98	0.05	1.01
e	0.896	0.890	0.893	0.880	0.893	0.895
ϵ (%)	9.5	9.5	9.0	11.0	0.8	12.0
strain rate (% per hour)	2.3	2.3	2.3	2.3	2.3	0.5

[a] These tests were conventional consolidated isotropic undrained, with pore pressure measured (GEONOR equipment).

Note: Because three of the five tests used in the Hvorslev component determinations were undrained, no volume-change energy correction was applied in these determinations.

volume is accomplished similarly, but utilizes a forced pore-pressure reduction. Two such cycles in test No. 242 permitted CFS computations at strains of 14% and 20%. The results compare favorably with an imagined extension of the CFS curves from test No. 254. The details necessary for the CFS computations from test No. 242 are presented in Fig. 26 by the stress-strain curves and $\bar{\sigma}_1$ for each data point.

For the purpose of determining the effective components, the writer prepared a number of normally consolidated and overconsolidated specimens and then subjected them to triaxial compression at either constant volume (Nos. 238, 240, and 241) or at constant values of $\bar{\sigma}_1$ (Nos. 250 and 254). These tests, which computed to have essentially equal void ratios but different effective stresses at $\bar{\sigma}_1/\bar{\sigma}_3$ failure, were then used to compute the components. Only the five tests noted, out of nine attempted, met the void-ratio requirement. These five tests had an average specimen e_f ranging from 0.890 to 0.896, which represents a $\bar{\sigma}_e$ range of only 3.3 kg per sq cm to 3.0 kg per sq cm.

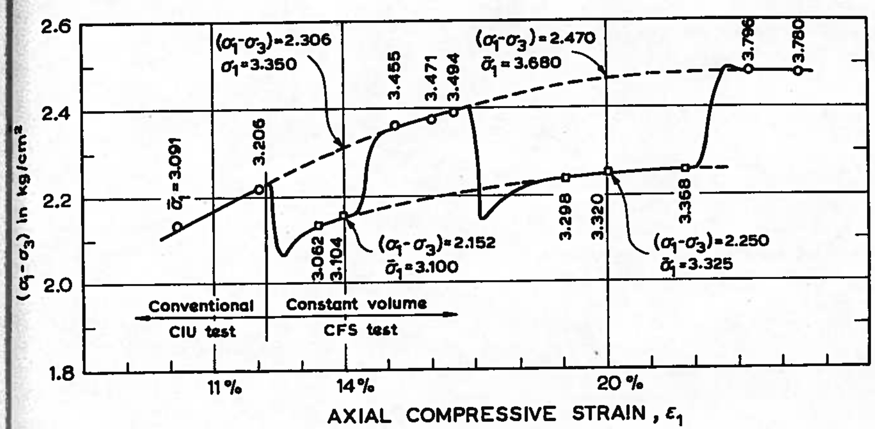

FIG. 26.—DETAILS OF CFS PART OF TEST NO. 242

Four of these five tests reached failure between 9% and 12% strain, but the fifth (No. 250) reached failure at only 0.8% strain. If only the four tests are used for the c_e extrapolation, then the extrapolation is long and uncertain but the applicable strain is better defined. If test No. 250 is included, then the c_e extrapolation is a short one and not uncertain but the applicable strain is uncertain. Sufficient data are presented in Table 9 for the reader to attempt these extrapolations. The results of the writer's attempts, both excluding and including test No. 250, are also presented in Fig. 25.

From Fig. 25 it appears that the two sets of components are in reasonable agreement. Perhaps these data, supplementing the authors' data, permit the tentative conclusion that the e and ϵ components are at least a good approximation of each other at large strain after $\bar{\sigma}_1/\bar{\sigma}_3$ failure. If so, then an investigator may use a procedure similar to test No. 242 as a practical method of determining the fundamental strength components after failure in a conventional test.

These data also serve to illustrate another point. Effective cohesion, c_e, can be considerably smaller than the value of $(c_\epsilon)_{max}$. In this case, the comparison is approximately 0.2 kg per sq cm to 0.57 kg per sq cm. This is due to the pronounced, early-strain cohesion "peak" that is often computed from CFS tests.

Use of More Than One Specimen for CFS Test.—The technique of Method 2 and the reported creep and relaxation tests involves a long extrapolation of the effects of small differences in effective stress between two or three, hopefully initially identical, clay specimens. The 5% and 10% strains in Fig. 7 are extreme examples. Small initial differences in the supposedly identical specimens can greatly alter the results of such extrapolations. However, if the CFS test is strain controlled, then it can be performed using a single specimen and thereby greatly increase the assurance of essentially constant structure in all soils and particularly in nonhomogeneous soils. However, the extent of nonhomogeneity is always uncertain. This can be illustrated by data presented by the authors (see Table 8).

Specimens Nos. 321 and 328 appear to be good duplicates. On the other hand, specimens Nos. 9 and 30 should be nearly identical from their water contents, but the $(\sigma_1 - \sigma_3)_f$ values are different. Thus, there is uncertainty even with carefully prepared remolded duplicates. According to Table 4, Fig. 7 should include points from three specimens, as does Fig. 6, but data from specimen 304 are missing. Perhaps if the authors presented the computed CFS curves obtained from the 304-305, 304-306, and 304-305-306 combinations as well as 305-306 (Fig. 7), the comparisons would serve as a useful illustration of the possible importance of small initial differences between specimens in multi-specimen CFS tests.

The one-specimen test is recommended wherever possible. The basic technique of such a test is explained in the literature.[13,28] Another paper that further explains technique and presents extensive data substantiating the validity of the one-specimen test has been accepted for publication. When stress control is required, as with the creep and relaxation tests, an increment of strain using strain control can serve to give a one-specimen determination of cohesion and friction at the beginning of the increment. This involves a short extrapolation of the CFS curves to zero CFS strain for that increment. This method is amply illustrated.[28,32] Even when the test is to be at constant volume, a one-specimen procedure can be utilized, as explained previously (Fig. 26).

Cohesion and $\bar{\sigma}_1$.—The writer's belief that the magnitude of $\bar{\sigma}_1$ is of great importance in determining the value of peak cohesion, irrespective of clay structure, is supported.[13,28] Perhaps $\bar{\sigma}_1$ is also important in determining non-peak values. The authors could provide valuable additional data if, to the extent possible, the cohesion-strain behavior for the different structures are compared with respect to $\bar{\sigma}_1$ values during strain.

32 "An Experimental Study of Cohesion and Friction During Creep in Saturated Clay," by R. G. Bea, thesis presented to the University of Florida, at Gainesville, Fla., in 1960, in partial fulfilment of the requirements for the degree of Master of Science (unpublished).

It is possible that $\bar{\sigma}$ on the failure plane may be more fundamental than $\bar{\sigma}_1$. In that case, $\bar{\sigma}_1$ can be considered as an approximate measure of $\bar{\sigma}$.

Supplementary Data.—Supplementary data are offered herein with the hope that the authors may find them useful in their research. Figs. 28 and 29[13] show data similar to Fig. 9. Data on the time transfer of cohesion to friction have been presented.[17,32] An interesting study of creep has been made.[32] All of these data are from tests on remolded kaolinite and remolded and undisturbed Boston blue clay.

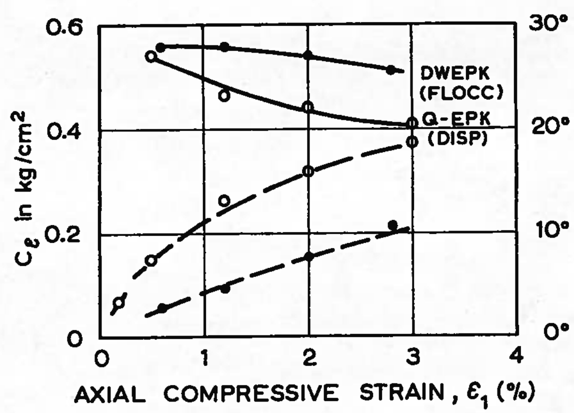

FIG. 27.—CONSTANT -$\bar{\sigma}_1$ CFS COMPARISON OF RELATIVELY FLOCCULATED AND DISPERSED KAOLINITE

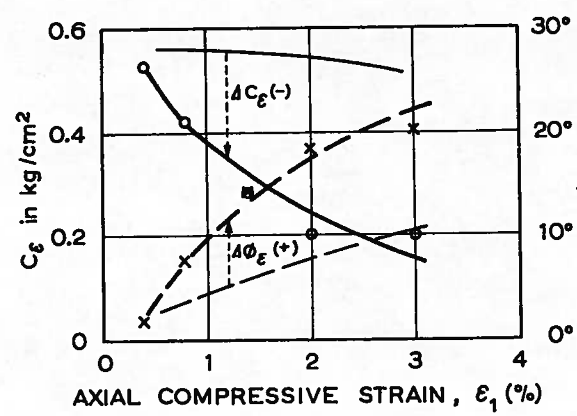

FIG. 28.—COHESION TO FRICTION TRANSFER IN STRESS CONTROL TEST ON KAOLINITE

Fig. 27 presents the results of one-specimen, constant $\bar{\sigma}_1$, strain-controlled CFS tests on remolded specimens of kaolinite. Two-specimen tests, in which a different specimen is used to obtain each of the constant $\bar{\sigma}_1$ stress-strain curves, gave similar results. The Q-EPK has a trace of "Quadrofos" deflocculant added and has a greatly dispersed structure compared to the DWEPK. One evidence of this is a PI reduction of from 21% to 4%. It is seen that, at the same $\bar{\sigma}_1$, and with prior consolidation procedures identical, the more dispersed structure has the more pronounced cohesion peak, indicating that results are not always the same as suggested by Fig. 8.

Fig. 28 shows the results of a two-specimen CFS test on remolded kaolinite (without deflocculant added). Stress control was used, involving approximately ten increments with each constant stress level held for 3 days. Initial consolidation procedures and subsequent compressive stress levels were identical for all specimens. The only difference was in the magnitude of constant $\bar{\sigma}_1$: One was 3.30 kg per sq cm and the other 2.60 kg per sq cm (both rebounded from $\bar{\sigma}_c$ = 3.50 kg per sq cm). When the results are compared with the same DWEPK clay in Fig. 27 (light lines in Fig. 28), it appears that the greater the strain, the greater the cohesion to friction transfer. At strains below the peak cohesion, there was no evidence of transfer. This supports the authors' findings from their flocculated clay. It should be noted that the problem of not-quite duplicate specimens occurred here and the higher $\bar{\sigma}_1$ stress-strain curve used to compute c_ϵ and ϕ_ϵ was the average of two tests. The results shown in Fig. 28 should only be used qualitatively.

Acknowledgments.—L. Bjerrum, Director of the Norwegian Geotechnical Institute, kindly reviewed this discussion. His suggestions added to its possible value.

R. G. BEA,[33] A. M. ASCE.—In 1960, an experimental study of the development of cohesion and friction during creep in saturated clay was made by the writer.[32] The results of this experimental study are presented herein as a supplement to the creep study phase of the investigation made by the authors.

The experimental program, conducted under the direction of J. H. Schmertmann, A. M. ASCE, at the Soil Mechanics Research Laboratory of the University of Florida, Gainesville, Fla., was divided into two groups of tests. The first group consisted of a series of tests to determine the characteristic variation of cohesion and friction in the clays not having been subjected to creep action. The second group consisted of a series of creep-strength tests in which the hydrostatically consolidated samples were subjected to a constant load, permitting no drainage, for a period of time and then tested to determine the influence of creep action on the cohesion-friction-strain (CFS) characteristics of the clay. The determination of the CFS characteristics were accomplished using the triaxial compression test technique developed by Schmertmann.[34]

Two different clay soils and two types of samples were used in the test series. These included both remolded and undisturbed samples of near-saturated Boston Blue clay, designated BBC and U-BBC, respectively, as well as remolded samples of Kaolinite clay, designated DWEPK. All samples were 8.00 cm in height and 3.59 cm in diameter. The clays, sample preparation,

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34 "An Experimental Study of the Development of Cohesion and Friction with Axial Strain in Saturated Cohesive Soils," by J. H. Schmertmann and J. O. Sterberg, June 1960 Research Conf. on Shear Strength of Cohesive Soils, ASCE, July, 1961.

TABLE 10.—TEST SUMMARY

Test No.	Creep-strength test series	Sample type	Consolidation period, in minutes	CFS Test comp. rate, in 10-3 mm per min	Creep load, in kg	Creep period, in days	Initial adjustment, percentage	Second-stage creep rate percentage	Strain during creep, percentage	Peak pore-water pressure, in kg per sq cm	At End of Creep Test, kg per sq cm: Pore-water pressure	At End of Creep Test, kg per sq cm: Deviator stress	Void-ratio change due to testing
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
B-1		DWEPK	1,000	5									+0.003
B-2	A	DWEPK	1,300	5	11.0	9	0.60	0.008	0.71	0.77	0.50	1.12	+0.006
B-3		DWEPK	1,200	5	16.6	10	1.73	0.005	1.82	1.85	1.50	1.69	+0.008
B-4		DWEPK	1,000	5	20.0	11	4.75	0.012	5.00	2.56	1.51	1.96	+0.012
B-5		BBC	1,100	5									+0.003
B-6	B	BBC	1,200	5	11.0	19	0.52	0.009	0.82	0.76	0.14	1.12	+0.007
B-7		BBC	1,300	5	16.6	19	2.68	0.005	2.82	2.15	1.96	1.64	+0.009
B-8		BBC	1,200	5	19.0	19	6.14	0.011	6.30	2.60	1.57	1.82	+0.017
B-9	C	BBC	6,800	1.5	19.2	18	2.30	0.006	2.77	1.80	0.12	1.89	+0.005
B-10		BBC	6,800	1.5	19.2	2	2.22	---	2.37	2.08	2.03	1.90	+0.009
B-11		BBC	6,800	1.5	19.2	9	1.45	0.015	1.58	1.90	1.62	1.91	+0.002
B-12		BBC	6,800	1.5									+0.001
B-13	D	U-BBC	1,500	1.5	19.2	11	2.21	0.022	2.51	1.90	1.90	2.02	+0.007
B-14		U-BBC	1,300	1.5	19.2	3	1.80	---	1.97	1.78	1.78	2.00	+0.000
B-18		BBC	6,800	1.5									+0.011
S-91		U-BBC	1,300	5									+0.005

533 min/5

Genor triaxial testing equipment, and general experimental technique have been described in detail in previous papers.[34,28] A summary of the experimental conditions and results of the tests included in this study is given in Table 10.

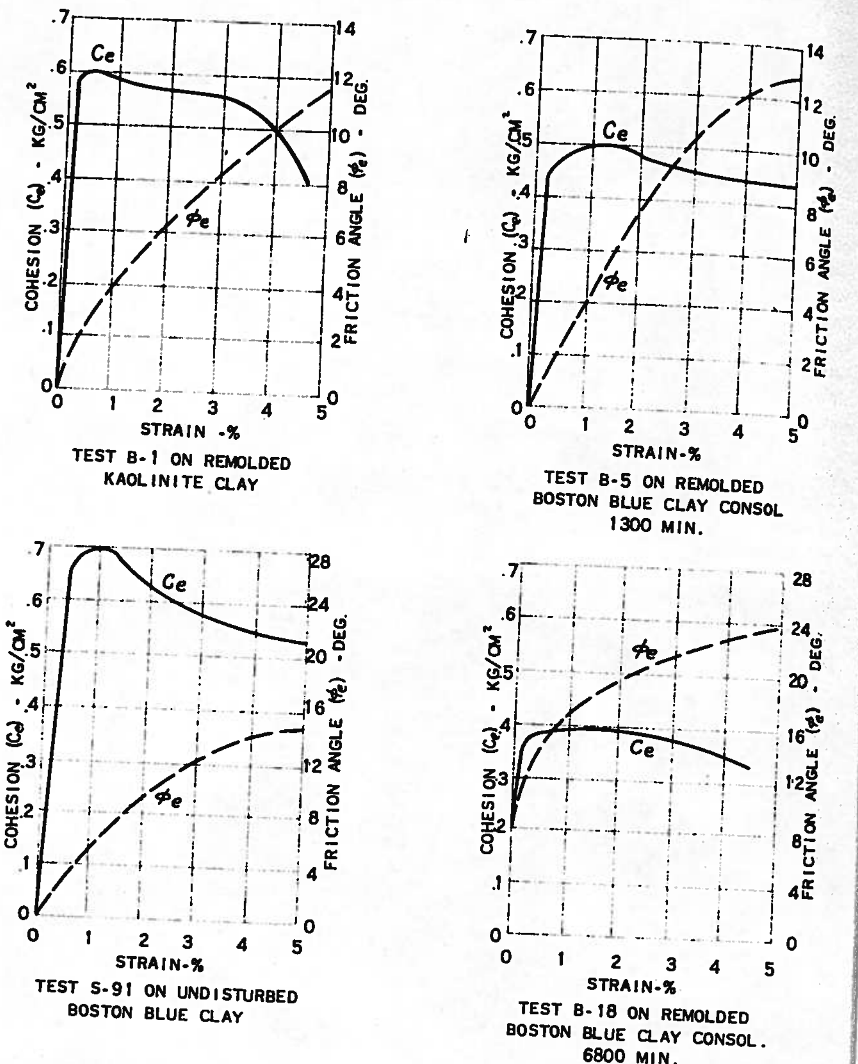

FIG. 29.—TYPICAL COHESION-FRICTION-STRAIN VARIATION FOR SATURATED CLAYS TESTED

The results of the first group of tests, which were performed to determine the characteristic CFS behavior of the soils included in this study, are presented in Fig. 29. These tests indicate that the cohesion develops to a maximum value at low compressive strains, whereas the friction angle requires much greater strains to reach its maximum value. Several tests in this group were performed on samples that had been consolidated 6,800 min rather than the usual 1,200 min to 1,300 min. The samples that had a longer period for consolidation showed a much higher friction angle at equal strains (see Fig. 29, Tests B-5 and B-18). The variation of cohesion with strain was essentially the same as noted in the other tests.

The second group of tests, termed creep-strength tests, generally were performed on a series of three specimens; these series are given test designations A, B, C, and D, respectively. The details of the test series are given in Table 10. Measurements were taken of axial compression, pore-water pressure, and time. All of the creep tests were performed at constant water content and at constant load.

The CFS results of test series A performed on remolded Kaolinite clay samples are given in Fig. 30. These results clearly indicate the influence that creep action has had on the CFS characteristics. Initially, the friction angle is much greater and the cohesion much less than is found in the standard test at the same strain. With the increased strain rate of the CFS test, the friction angle quickly decreases, reaches a minimum value, and then again begins to build at the higher strains along the same lines as the standard test. The cohesion, on the other hand, increases rapidly at low strains, reaches a maximum, and with higher strains generally follows the same lines as the standard tests. The stepping action noted in some tests is thought to be caused by the formation of failure planes in the samples at these high strains.

Test series B, shown in Fig. 31, on remolded Boston Blue clay was performed under the same conditions as those in test series A. The same striking CFS behavior is noted. The absence of the tendency in the BBC samples for the friction angle to increase at high strains is probably due to this sample having reached a maximum frictional resistance at this strain rate.

In the C series of tests, shown in Fig. 32, the remolded Boston Blue clay samples were allowed to consolidate approximately 5,600 min longer than the series B samples before beginning the creep-strength tests. In addition, the strain rate used in the CFS test was reduced for these and the remaining tests in order to enable a better definition of the cohesion-friction behavior at low test strains. The high initial build-up of friction in the CFS test after creep is not so evident as in the test series B, but the high initial friction and low initial cohesion are evident. Again, there is the same general variation of cohesion and friction with CFS test strain, except that in this BBC series there is a definite tendency for the friction to increase again after reaching a minimum value.

The CFS behavior of the undisturbed Boston Blue clay, test series D shown in Fig. 33, is somewhat different from that of the remolded soil. There is still the high initial build-up of friction, but there is now a definite cohesive strength present initially. The cohesion quickly increases with strain whereas the friction remains essentially constant. As the CFS test strain is further increased, the cohesion and friction remain essentially constant. This absence of the tendency for the friction angle to increase with strain may be because the sample has already mobilized the maximum friction; thus, the cohesive component of the strength is forced to carry the remainder of the stress.

In the creep-strength test series, there were three samples that had high initial values of cohesion (tests B-10, B-13, and B-14). Test B-10 was tested

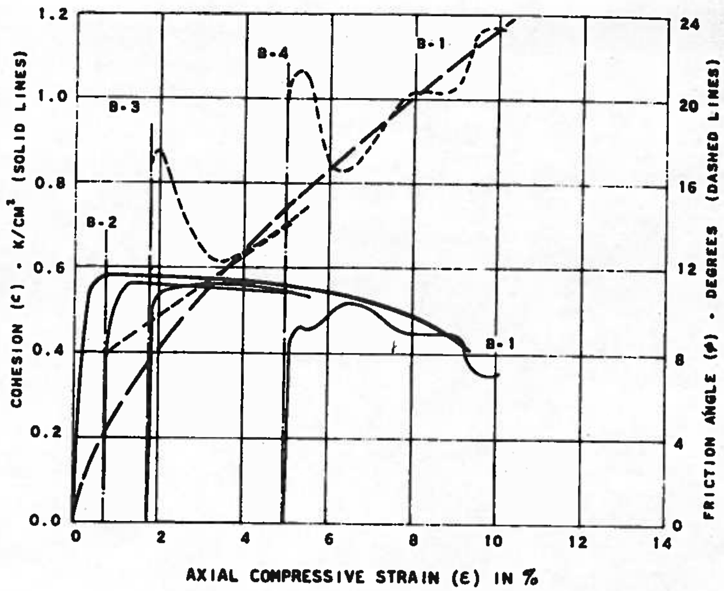

FIG. 30.—COMPUTED RESULTS OF CFS TESTS ON DWEPK SAMPLES

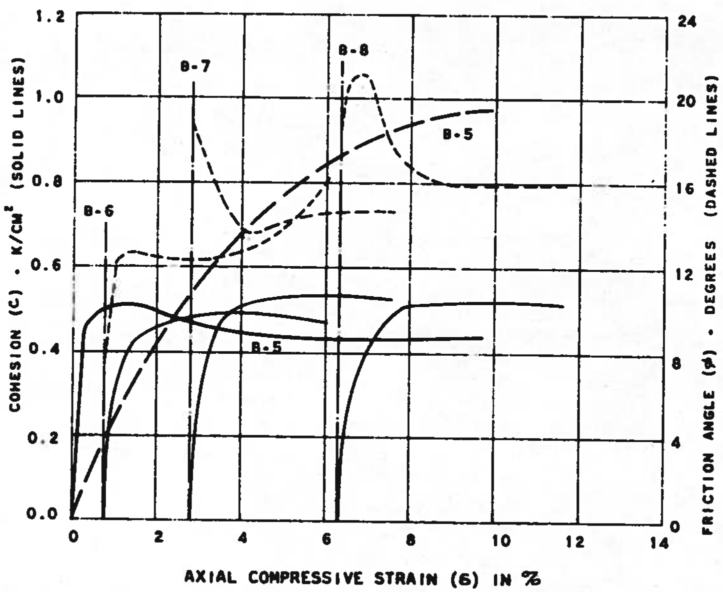

FIG. 31.—COMPUTED RESULTS OF CFS TESTS ON BBC SAMPLES

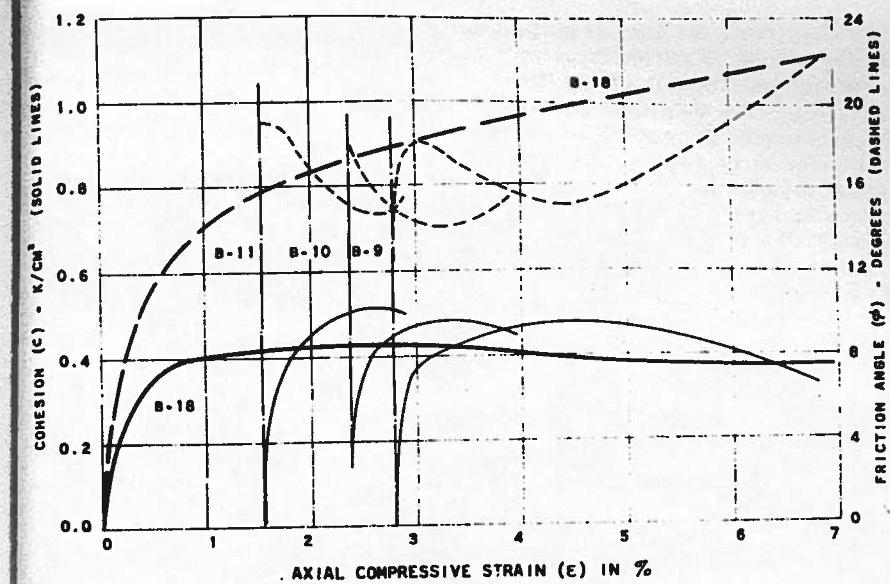

FIG. 32.—COMPUTED RESULTS OF CFS TESTS ON BBC SAMPLES (tc - 6,800 min)

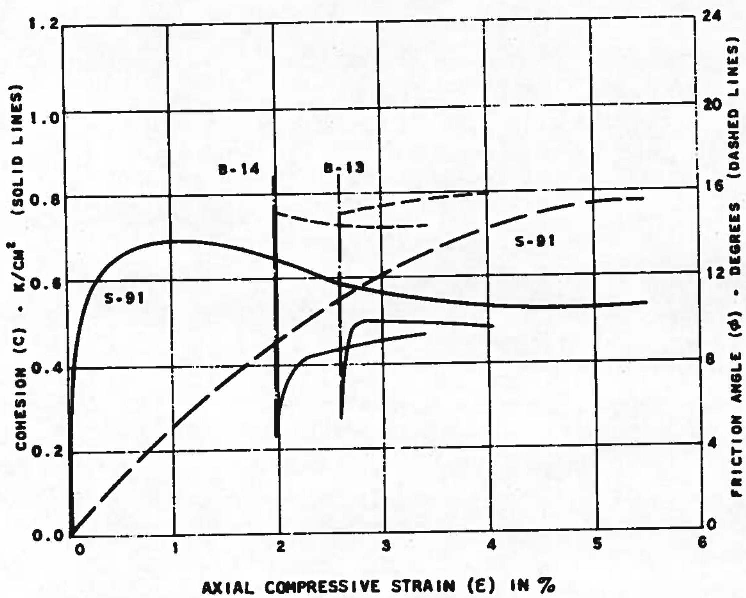

FIG. 33.—COMPUTED RESULTS OF CFS TESTS ON U-BBC SAMPLES

two days from the time of initial loading, at which time the sample was still in the transition period from initial adjustment to the visco-plastic stage of creep action. Tests B-13 and B-14 were performed on the undisturbed Boston Blue clay. The CFS test subsequently indicated that it was not possible for these samples to mobilize any greater frictional resistance. The creep rates for these three samples (see Table 10) were significantly higher than those found in the other tests. This would suggest the possibility that the rate of creep in the visco-plastic stage of creep action may depend on the degree of mobilization of the cohesive component of strength.

The creep deformation characteristics of the soils used in this study are presented in Figs. 34 and 35. The creep curves generally were divided into two characteristic stages: (1) An initial stage characterized by a high strain rate and large deformations, and (2) a second stage beginning within 1,000 min to 4,000 min in which the strain rate remained essentially constant. These creep characteristics were obtained for the zero to full loading period of 1 hr, for the remolded and undisturbed samples, and for creep periods of 2 days to 19 days.

A comparison of test series A and B is shown in Fig. 34. These tests indicate that the Kaolinite samples generally had less initial adjustment than the Boston Blue clay samples. The second-stage creep rates at the same loads were approximately the same in the two soils.

A comparison of the tests on remolded and undisturbed samples of Boston Blue clay is shown in Fig. 35. Fig. 35 shows the influence of load, consolidation period, and structure of the soil on the creep characteristics of the same soil. A comparison of tests B-8 and B-13, both subjected to approximately the same creep load, indicates the marked influence of structure on the creep characteristics of the sample. The undisturbed samples had an initial adjustment that was approximately 35% of that for the remolded sample under the same conditions, whereas the creep rates for the undisturbed samples were approximately twice that for the remolded soil. A similar comparison of tests B-8 and B-10 shows the effect of consolidation period. The remolded samples that were consolidated for 6,800 min had an initial adjustment that was approximately 35% of that for the samples consolidated 1,200 min.

Examination of the standard CFS test for these samples will provide an explanation for this widely differing behavior. The samples that mobilized high cohesive and frictional strength at low strains showed less initial adjustment under the creep loads. The samples that had a greater ultimate frictional strength had lower second-stage creep rates. Thus, it appears that the creep characteristics of a clay will depend on its CFS characteristics determined before creep.

The variarion of pore-water pressure with time is shown for the test series in Figs. 36 and 37. For the majority of the samples there was a peak in the pore-water pressure in the early part of the creep test. This usually occurred a short time after the total load had been imposed and while the sample was in the transition from the first to the second stage of creep. In the remolded samples, the peak pore-water pressure was generally followed by a decrease in pressure. This indicates the occurrence of a stress transfer during the creep process; a transfer from the pore water to the structure of the soil. Unlike the other tests, the pore-water pressure in the undisturbed Boston Blue clay test series (Fig. 37) quickly built up, then was maintained essentially constant for the remainder of the creep test. This would indicate that it

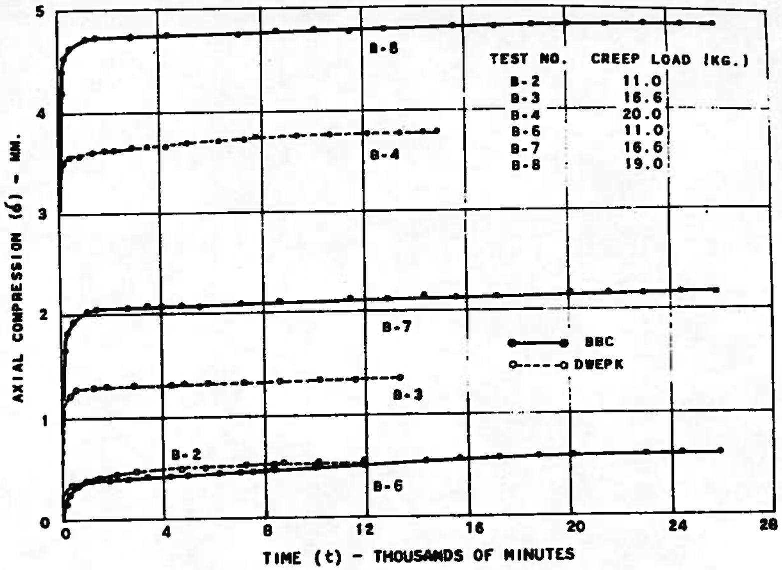

FIG. 34.—CREEP CURVES FOR BBC AND DWEPK SAMPLES, TEST SERIES A AND B

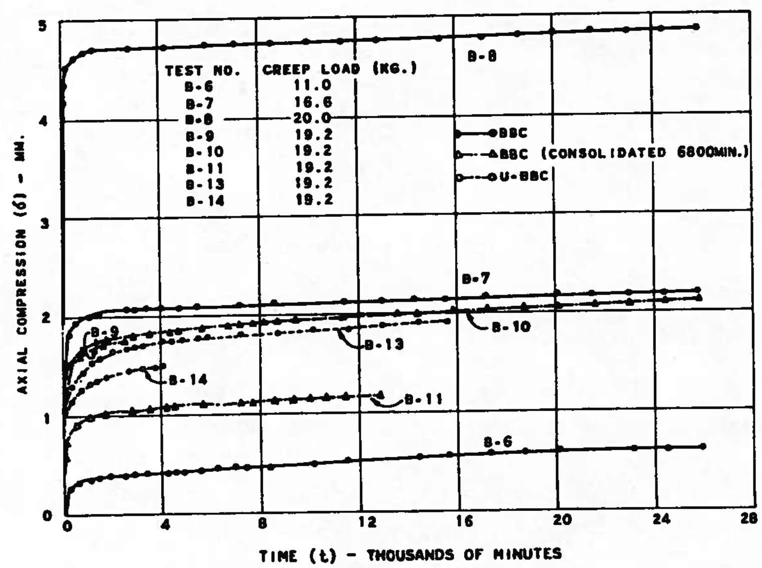

FIG. 35.—CREEP CURVES FOR BBC AND U-BBC SAMPLES, TEST SERIES B, C, AND D

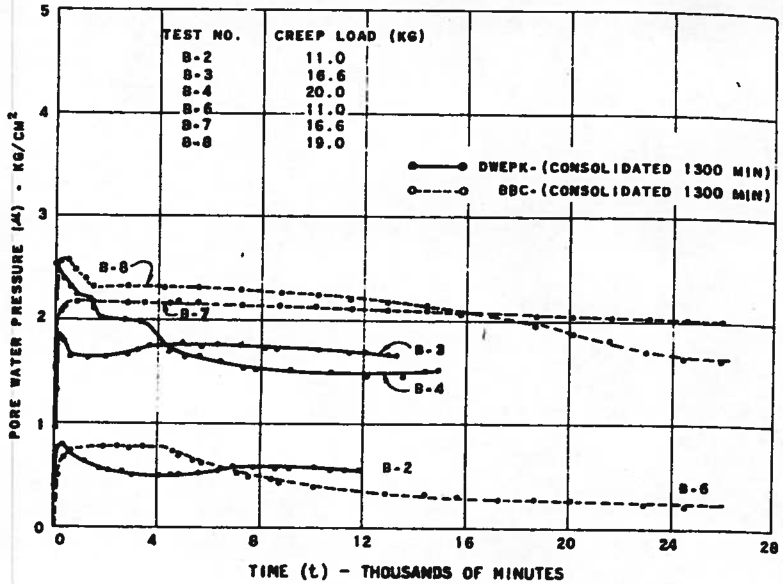

FIG. 36.—PORE-WATER PRESSURE VERSUS TIME, TEST SERIES A AND B

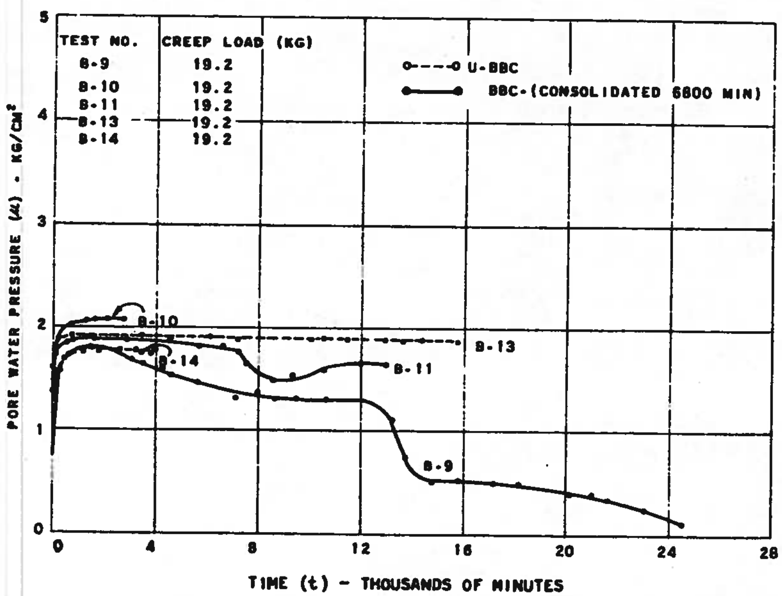

FIG. 37.—PORE-WATER PRESSURE VERSUS TIME, TEST SERIES C AND D

was not possible for the undisturbed samples to effect such a transfer of stress with time as was noted in the remolded samples. This is further evidenced by the high second-stage creep rate and the existence of a significant initial cohesive strength.

Thus, this experimental evidence indicates that there is a definite transfer of stress to the frictional component of strength during creep in a near-saturated clay. The cohesive component of strength was found to be nearly zero after creep periods of 2 days to 19 days except in the undisturbed samples, which exhibited relatively high initial values of cohesion. The undisturbed samples had large second-stage creep rates and there was no evidence of a decrease in the pore-water pressures as was found in the remolded samples. The mineralogy, structure, and consolidation period were found to have a marked influence on the CFS, creep, and pore-water pressure characteristics of the near-saturated clays tested during this investigation.

The rheological characteristics of clay soils are finding wider application as the field of soil mechanics is explored and developed. Some theoretical and experimental investigations into creep, plastic flow, and relaxation phenomena in clays and their influence on strength have been made, but the information is limited and experimental data are meager. The authors have made a valuable contribution to the promising field of clay rheology. It is hoped that the reported results of the writer's experimental program will further clarify and extend the findings of the authors.