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COMPARISONS OF ONE AND TWO-SPECIMEN CFS TESTS

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SYNOPSIS

The CFS test is designed to permit the determination of the strain variation of the mobilization of c and ϕ components from a test using only a single specimen. This paper includes a review of the theory and performance of this test. The validity of using only a single specimen is investigated by comparing the stress-strain and cohesion-friction-strain results from one-specimen CFS tests with results from identical tests using two specimens. Each such test permits a check of the curve-hopping technique used in the one-specimen test but not required in the two-specimen test.

The paper includes 14 such comparisons, involving nine different normally consolidated soils. Seven are remolded and saturated, with plasticity indexes between 0 and 21% and at 105%. The other soils are undisturbed—one a dry, cemented sand and the other a sensitive Leda clay. Only these two were tested to strains exceeding conventional failure. The remainder investigate the cohesion failure, but the strains do not reach conventional failure. Two of the remolded clays were also tested for the validity of curve-hopping after allowing different times for secondary compression.

All one and two-specimen test comparisons show good qualitative agreement. The quantitative agreement is usually good, particularly for the maximum value of cohesion. However, there is a tendency for cohesion to decrease more rapidly with strain in the two-specimen tests. The comparisons for the undisturbed soils are also encouraging. Increasing secondary time with remolded clay appears to result in improved comparisons. The available evi-

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dence indicates that undisturbed soils can be tested successfully with the one-specimen CFS test. Overconsolidated clays were not investigated.

The paper includes a brief examination of the soil type, speed of compression, drainage, and equipment precision requirements for the successful performance of one-specimen CFS tests. Also considered are constant pore pressure and constant volume versions of the CFS test.

INTRODUCTION

Review of One-Specimen CFS Test.— The laboratory test under investigation is the Cohesion-Friction-Strain (CFS) test. In this test, the investigator attempts to determine the strain-mobilization of the cohesion and friction components of a soil's resistance to shear stress. The test is designed to permit computation of redefined cohesion and friction components[2] of the total shear resistance at selected values of strain during a compression test. From these computed values, curves of the mobilization of cohesion and friction with strain can be determined.

The CFS test procedure (herein referred to as "standard") involves the determination of two stress-strain curves, each at a different constant value of the major principal effective stress, $\overline{\sigma}_1$. The specimen is placed in a triaxial cell, consolidated in any desired manner, and then is subjected to a constant rate of compressive strain. The imposed strain forces the development of a principal stress difference $\sigma_d = \sigma_1 - \sigma_3$, herein called the deviator stress. During this strain, the pore fluid pressure is controlled to maintain a constant preselected value of $\overline{\sigma}_1$. Any increase in pore pressure then reduces effective stresses and weakens the soil with the result (because of the strain control) that a new equilibrium stress-strain curve is established at a lower magnitude of σ_d. Decreasing pore pressure similarly increases σ_d. The one-specimen CFS test procedure consists of alternating between two values of $\overline{\sigma}_1$ in such a way that two stress-strain curves are obtained—one for each $\overline{\sigma}_1$. Any strains where these two curves are both sufficiently well defined, the investigator can mathematically determine the desired shear resistance components.

The CFS test procedure is neither drained nor undrained. Small changes in volume necessarily occur in conjunction with changes in $\overline{\sigma}_1$ at the same strain, but require a void ratio change of less than 1%. A somewhat greater change may occur during strain at constant $\overline{\sigma}_1$. The test is not free-draining because of the imposed pore pressure control.

For the reader who is not familiar with the previous publication describing the CFS test theory and procedures,[3,4] a brief review is presented in Appendix I. However, this paper is part of a sequence, and familiarization with these references should be helpful. They also describe the Norwegian Geotechnical Institute equipment used in this research, as does the paper by Andresen and Simons.[5]

Notation.—The letter symbols adopted for use in this paper are defined where they first appear and are listed alphabetically in Appendix II.

Question of Validity of Use of Single Specimen.—The preceding description indicates that because the performance of the CFS test requires only one specimen, the test may permit the practical determination of the mobilized cohesion and friction components of undisturbed and otherwise nonhomogeneous soils. However, the discussers of the paper by Schmertmann and Österberg[3] indicated that the one-specimen CFS test procedure must be proved valid before the profession can use any advantages it may offer. Quotes from the discussions are reproduced:[6]

> "The determination of the shear characteristics of cohesive soils in an undisturbed state requires a considerable number of samples of practically identical materials which, in most cases, cannot be obtained. If the procedure proposed by Schmertmann and Osterberg should prove sufficiently reliable, this difficulty would disappear, because the procedure would make it possible to obtain the essential data from triaxial tests on a single specimen."

and[7]

> "By such a procedure, it appears possible to determine the cohesion intercept c and the angle of shearing resistance ϕ of a soil from a test upon a single sample. Such a procedure, if valid, would have great value in testing undisturbed soils, for it would avoid the difficulties which stem from the fact that it is almost impossible to obtain two or more genuinely identical samples."
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> "The potential advantages of the "curve hopping" (used in CFS test) or "stage" (used by D. W. Taylor) techniques are so great as to warrant careful investigation into their validity."

Scope of Paper.—Several aspects of the validity of the one-specimen CFS test appear important. Foremost is whether or not a one-specimen test, which defines two stress-strain curves, yields the same computed results as would be obtained from two initially identical specimens each tested at constant $\overline{\sigma}$, and each defining only one of the stress-strain curves of the one-specimen test. This paper deals with this question alone.

Comparative one and two-specimen CFS tests were performed for a variety of soils. In all cases, an effort was made to obtain the comparative specimens in as initially identical a condition as possible. A "Vac-Aire" extrusion

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[2] Definitions are restated in Appendix I.

[3] "An Experimental Study of the Development of Cohesion and Friction with Axial Strain in Saturated Cohesive Soils," by John H. Schmertmann and Jorj O. Osterberg, ASCE Research Conference on Shear Strength of Cohesive Soils, June, 1960, p. 643.

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

[5] "Norwegian Triaxial Equipment and Technique," by A. Andresen and N. E. Simons, ASCE Research Conference on Shear Strength of Cohesive Soils, June, 1960, p. 695.

[6] "Review of Conference," by Karl Terzaghi, ASCE Research Conference on Shear Strength of Cohesive Soils, June, 1960, p. 1118.

[7] "Shear Strength of Saturated, Remolded Clays," by S. J. Johnson and R. V. Whitman, ASCE Research Conference on Shear Strength of Cohesive Soils, June, 1960, p. 1133.

machine[8] was used to prepare duplicate specimens of six cohesive soils, five with plasticity indexes (PI) between 4% and 21%, and the sixth with a PI of 105%. Specimens of Ottawa sand were prepared individually by dry compaction in a triaxial specimen mold. Two natural, undisturbed soils are included. The specimens were trimmed from blocks. One is a lightly cemented angular sand from Tupelo, Mississippi, and the other a sensitive Leda clay from Ottawa, Canada. The comparative specimens of natural soil are possibly not as nearly identical as those from the extruded cohesive soils. Except for the curve-hopping, all the other test conditions such as length of time in primary consolidation and secondary compression, rate of compression, temperature, and stress measurement and pore pressure control accuracy were also duplicated within a comparative test series.

In addition to the variety of soils used, the scope of the comparative tests was restricted to a study of strength component mobilization at low values of compressive strain. Previous one-specimen CFS test research indicated that the cohesion was fully mobilized at much lower strains than was required to develop friction.[3,4] The writer thought it especially interesting to study the cohesion development in the two-specimen test. Restricting the comparative tests to the low strain range permitted a more careful check of the difference in the strain behavior of the two components over the range where they are the most different, and also permitted a more accurate comparison of the computed values of maximum cohesion. However, for the extruded specimens, this meant that the strain did not reach sufficiently high values to attain conventional failure (cohesion plus friction failure).

The scope also includes comparative tests on two of the extruded soils after allowing different times for secondary compression—with all other variables held constant. This was done to obtain a preliminary evaluation of the comparative-test behavior of undisturbed cohesive soils. One of the most outstanding differences between undisturbed and remolded clays is the highly developed structure in undisturbed clay that is associated with long times in secondary compression.[9] After the performances of these tests, an undisturbed cube of Leda clay became available and the comparative CFS test results are included.

PRESENTATION OF DATA

Tables.—Listed in Tables 1 and 2, in order of increasing plasticity index of the soil, are the mineralogy and other classification characteristics of the soils used. The subsequent tables, and the presentation of figures, are also in order of increasing plasticity index.

Table 3 is a summary of the soil parameters that provide the best available information regarding the uniformity of the specimens in each comparative group, or groups, of specimens. Table 3 includes computed values of water content, void ratio, and degree of saturation at the time of placement in the triaxial cell (subscript i for initial) and at the time of removal from the cell (subscript f for final). The void ratio after one-increment, hydrostatic consolidation, but before the CFS test, is also included (subscript o). This value is the average of two computations, one computing forward from the initial condition and the other computing backward from the final condition.

TABLE 1.—SPECIMEN MINERALOGY

Letter Designation	Semi-quantitative mineralogy[a]	Notes
OS	quartz 100%	Standard Ottawa sand
TCS	Kaolinite 5% quartz 85% feldspar 10%	Tupelo lightly cemented, angular sand, undisturbed, air/dry
Q-EPK	same as DWEPK	Mixed with 1/25 M soln. "Quadrofos"
ENID	kaolinite 15% illite 10% vermiculite 10% sepiolite 10% quartz 20% feldspar 15% muscovite 15%	Residual clay from Georgia, extruded with natural water
Leda	Not determined. Contains** illite, chlorite feldspar	Trimmed from undisturbed cube, brittle, lab vane sensitivity approx. 15
JSC	montmorillonite 10% quartz 20% calcite 20% feldspar 20% dolomite 10%	Mixture of splitspoon samples from Florida site, natural water
BBC	illite 45% chlorite 25% quartz 20% feldspar 10%	Boston blue clay from Cambridge, Mass., natural water
DWEPK	kaolinite 95% quartz - less than 5% muscovite 5%	"As received" kaolin powder from Edgar Plastic Kaolin Co., Edgar, Fla., mixed with distilled water
LWC	montmorillonite 85% illite 5% quartz 10%	Clay from Lake Wauburg, Fla., natural water

[a] Mineralogical analyses performed by J. L. Harrison.

Table 4 contains a summary of the pressures, drainage conditions, and consolidation times permitted during the one-increment consolidation. The

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[8] "De-Aired, Extruded Soil Specimens for Research and for Evaluation of Test Procedures," by H. Matlock, C. Fenske, and R. Dawson, Bulletin No. 177, ASTM, October, 1951.

[9] "Undisturbed Clay Samples and Undisturbed Clays," by Karl Terzaghi, Contributions to Soil Mechanics, 1941-1953, Boston Soc. of Civ. Engrs., p. 45.

full cell pressure was applied suddenly without the use of back pressure in the pore water. The drainage aids used include 1/8 in. to 1/4 in. wide filter paper strips the same length as the 8.00 cm. long specimen and placed vertically, with approximately equal spacing, around the circumference of the 3.58 cm diameter specimen. The internal drains are the same as those previously described.[10] The Casagrande semi-log method was used to determine the time to 100% primary consolidation. Secondary times listed are the differences between the total time allowed for consolidation and the primary time indicated. Primary and secondary could not be distinguished during the consolidation time allowed the Leda clay, and the total time allowed is listed under primary.

Table 4 also includes a summary of the effective stresses and compression rates used for the comparative CFS tests. The $\bar{\sigma}_1$ values listed are the intended constant values for each curve (approximately 95% and 75% of the hydrostatic preconsolidation value, $\bar{\sigma}_c$). Reference to Table 4 will show that most of the tests were performed with $\bar{\sigma}_c$ = 3.50 kg per sq cm. This value was chosen for convenience because the piston load cells are then used almost to capacity, and the two stress-strain curves are separated with maximum precision. The final computed values of $\bar{\sigma}_1$ may vary slightly from the intended value, but usually by no more than 0.02 kg per sq cm. The compression rates listed are the averages for the entire strain range of each test. Because of compression and extension of the load cell during curve-hopping, the compression rate between successive data points in the 1-specimen tests may

TABLE 2.–SOIL INDEX VALUES

Soil	G_s Used	Per cent finer than		Atterberg Limits		Activity $\frac{\text{PI}}{\text{\% less thaw } 0.002 \text{ mm.}}$	Liquidity index-start of CFS-test
		200 sieve (washed)	0-002 mm (hydrometer)	LL	PI		
OS	2.660	0	0	nonplastic			
TCS	2.661	5	2	not determined			
Q-EPK	2.609	100	60	29	4	0.07	23%
ENID	2.790	65	20	37	9	0.45	1
Leda	2.800[a]	not determined		36	12	n.d.	220
JSC	2.716	62	13	30	14	1.08	19
BBC	2.810	98	53	38	19	0.36	20
DWEPK	2.609	100	60	52	21	0.35	15
LWC	2.810	100	85	150	105	1.24	14

[a] Information obtained from "The Influence of Rate of Strain on Effective Stresses in Sensitive Clay," by Carl B. Crawford, Special Technical Publication No. 254, ASTM, 1959.

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[10] "An Experimental Study of the Development of Cohesion and Friction with Axial Strain in Saturated Cohesive Soils," by John H. Schmertmann and Jorj O. Osterberg, ASCE Research Conference on Shear Strength of Cohesive Soils, June, 1960, p. 652.

TABLE 3.–SUMMARY OF MEASURES OF TEST GROUP SPECIMEN UNIFORMITY

Fig. No.	Clay	Test No.	Computed Soil Conditions						
			Before Consolidation			Start CFS	After CFS-Test		
			W_i%	e_i	S_i%	e_o	W_f%	e_f	S_f%
1	OS	517	Not Det.	0.521	99.9	0.521	Not Det.	0.540	99.9
		518		0.515	99.9	0.515		0.538	99.9
		519		0.521	99.9	0.521		0.540	99.9
2	TCL	157	1.0	0.73	3.7		Not Determined		
		167	0.8	0.77	2.7				
		168	0.8	0.77	2.6				
3	Q-EPK	155	26.7	0.700	99.5	0.679	26.6	0.690	100.6
		156	26.4	0.693	99.5	0.671	26.9	0.693	101.1
		159	26.4	0.694	99.3	0.674	26.7	0.697	100.1
4	ENID	125	36.2	1.026	98.3	0.792	28.2	0.805	97.8
		133	36.6	1.036	98.5	0.799	28.5	0.807	98.7
		134	36.3	1.022	99.2	0.782	27.4	0.787	96.9
5	ENID	122	36.1	1.031	97.7	0.794	27.4	0.788	97.1
		126	36.1	1.028	98.0	0.790	27.8	0.801	96.9
		129	36.2	1.023	98.7	0.783	27.3	0.780	97.6
6	ENID	115	36.2	1.026	98.5	0.782	27.4	0.788	96.9
		118	36.0	1.019	98.5	0.775	27.5	0.766	100.4
		123	36.1	1.028	98.1	0.788	27.4	0.789	96.7
7	Leda	261	53.2	1.505	99.0	1.391	49.9	1.406	99.3
		263	53.2	1.510	98.7	1.432	51.1	1.441	99.3
		264	53.5	1.528	98.0	1.421	50.9	1.439	99.0
8	JSC	149	23.5	0.640	99.7	0.500	17.9	0.492	98.8
		150	22.9	0.632	98.4	0.497	18.0	0.494	99.1
		151	21.9	0.606	98.2	0.513	17.8	0.489	99.0
9	BBC	100	26.4	0.752	98.8	0.653	22.7	0.653	97.6
		101b		0.744	98.6			0.655	97.9
		101c	26.3	0.742	99.5	0.639	22.4	0.644	97.7
10	BBC	166	26.4	0.759	97.8	0.648	22.7	0.652	97.7
		170	26.8	0.757	99.3	0.648	23.0	0.654	99.0
		171	26.2	0.743	98.8	0.649	22.7	0.648	98.3
		176	26.2	0.743	98.9	0.631	22.8	0.648	99.1
11	DWEPK	98b	38.8	1.013	99.9	0.873	33.7	0.881	99.9
		98d	39.0	1.018	100.0	0.875	33.3	0.877	99.1
		99	39.0	1.017	100.0	0.875	33.4	0.878	99.2
12	DWEPK	135	39.4	1.031	99.7	0.898	34.5	0.906	99.5
		165	39.2	1.031	99.3	0.891	34.0	0.893	99.4
		169	39.4	1.029	99.9	0.885	34.4	0.896	100.2
13	DWEPK	114	40.0	1.408	99.5	0.903	34.2	0.896	99.7
		121	39.7	1.040	99.5	0.904	34.6	0.907	99.7
		127	39.1	1.027	99.3	0.907	34.4	0.900	99.6
14	LWC	160	64.5	1.846	98.2	1.695	61.3	1.743	98.8
		161	64.2	1.838	98.2	1.699	61.9	1.772	98.2
		164	61.7	1.785	97.2	1.668	60.8	1.746	97.9
15	DWEPK	140	39.5	1.025	100.4	0.872	33.7	0.889	98.8
		162	39.4	1.018	100.9	0.871	34.1	0.879	101.2

TABLE 4.—SUMMARY OF COMPARATIVE TESTS

		No. Drainage Aids		1-Increment Consolidat.			CFS-Test		
Fig. No.	Test No.	Internal Wool	Ext. Strips	$\bar{\sigma}_c$ kg per sq/cm	Time (Minutes) Primary	Second.	$\bar{\sigma}_1$ in kg/cm^2 High	Low	Compression Strain Rate (Min. for 1%)
1	517	0	0	2.00	Not Determined		2.00	--	23
	518	0	0	2.00			2.00	1.50	23
	519	0	0	2.00			--	1.50	23
2	157	0	0	1.00	Not Determined		--	u=0.50	168
	167	0	0	1.00			u=0	--	166
	168	0	0	1.00			u=0	u=0.50	167
3	155	5	4	3.50	290	1030	3.30	--	152
	156	5	4	3.50	305	965	--	2.60	152
	159	5	4	3.50	390	905	3.30	2.60	152
4	125	0	4	3.50	60	90	--	2.60	135
	133	0	4	3.50	80	60	3.30	2.60	136
	134	0	4	3.50	75	65	3.30	--	136
5	122	0	4	3.50	90	1300	3.30	--	136
	126	0	4	3.50	80	1330	--	2.60	136
	129	0	4	3.50	65	1380	3.30	2.60	135
6	115	0	4	3.50	80	9695	3.30	--	137
	118	0	4	3.50	65	9705	--	2.60	136
	123	0	4	3.50	60	9670	3.30	2.60	134
7	261	1	12	3.50	11500		3.30	--	2906
	263	1	12	3.50	10790		--	2.60	2827
	264	1	12	3.50	10770		3.30	2.60	2501
8	149	1	4	3.50	150	1085	3.30	2.60	146
	150	1	4	3.50	65	1175	--	2.60	137
	151	1	4	3.50	120	1045	3.30	--	151
9	100	3	3	3.68	190	1070	3.70	3.00	150
	101b	3	3	3.68			--	3.00	150
	101c	3	3	3.68	210	1130	3.70	--	150
10	166	5	4	3.50	150	1230	3.30	2.60	127
	170	5	4	3.50	225	1175	--	2.60	126
	171	5	4	3.50	280	1080	3.30	--	139
	176	5	8	3.50	115	1080	3.30	2.60	140
11	98b	3	3	3.68	80	1220	--	2.95	139
	98d	3	3	3.68	60	1240	3.68	--	139
	99	3	3	3.68	60	1250	3.68	2.98	138
12	135	1	4	3.50	55	1380	3.30	2.60	151
	165	1	4	3.50	70	1345	3.30	--	148
	169	1	4	3.50	65	1345	--	2.60	149
13	114	1	4	3.50	75	9870	3.30	--	146
	121	1	4	3.50	70	9760	--	2.60	150
	127	1	4	3.50	55	9720	3.30	2.60	150
14	160	7	12	3.50	3600	6500	3.30	--	927
	161	7	12	3.50	4300	5100	--	2.60	925
	164	7	12	3.50	4000	5200	3.30	2.60	1665
15	140	5	4	3.50	35	1345	3.30	2.60	148
	162	5	4	3.50	30	1210	3.30	2.60	150

vary as much as 25% from the average of the entire test. When interpreting the experimental results, this compression rate variation was assumed negligible and was disregarded.[11] The section "Consideration of Data" includes additional examination of errors in the computations of the shear resistance components.

Figures.—The upper half of Figs. 1 through 15 present the comparative one and two-specimen stress-strain curves. The solid line at greater deviator stress presents the behavior of a specimen held at a high, constant value of $\bar{\sigma}_1$. The solid line at lower deviator stress presents the behavior of another specimen held at a lower, constant value of $\bar{\sigma}_1$. These two-specimen test curves are well defined by many data points with negligible scatter from the curves presented. These individual points are therefore omitted in the figures to enhance clarity. The dashed lines present the behavior of a single specimen, using the CFS test curve-hopping technique. The open circles are all the data points obtained at the high value of $\bar{\sigma}_1$, and the solid circles all the data obtained at the low value of $\bar{\sigma}_1$. As shown in Table 4, the intended high and low $\bar{\sigma}_1$ values were the same in each comparative one and two-specimen test series. These intended values are also shown in each figure.

The lower half of Figs. 1 through 15 show the results of cohesion and friction computations (circles and crosses, respectively) at various values of axial strain, as calculated from the stress-strain curves directly above. Again, the solid lines represent the results of the two-specimen test and the dashed lines the results of the one-specimen test. The curves shown are the writer's estimate of the best fit through the computed points. It should be noted again that cohesion and friction are components of mobilized shear resistance. At zero strain, the stress is still hydrostatic, and there is a net shear stress of zero on any plane. If the concepts of negative cohesion and negative friction are rejected, then both components must be zero at zero strain (as shown in Fig. 5). This fact was sometimes useful in estimating the best tan ϕ_1 curve. However, the c_ϵ and tan ϕ_ϵ curves were omitted for the strain interval between zero and the first computed point (except in Fig. 5) to restrict the test comparisons to the strain interval studied in detail.

Fig. 1 presents the comparison for tests with Ottawa sand. In each test, the specimen was compacted to approximately 90% relative density and completely saturated. A high relative density was chosen because it was more reproducible.

Data for the Tupelo cemented sand, presented in Fig. 2, resulted from the use of a variation in the "standard" CFS technique described in the references cited and used in all the other tests reported herein. First, the soil was air-dry, and its structure was easily destroyed by water. Air, rather than water, was therefore used as the pressure-controlled pore fluid. Second, two constant $\bar{\sigma}_1$ curves could not be used with this soil because its cohesive strength was primarily caused by a dried kaolinite cement between the sand grains, and the compressive strength of the soil exceeded the cell confining pressure. To maintain a constant $\bar{\sigma}_1$, the pore pressure would have to be able to exceed the confining pressure—which cannot be done in the triaxial test. Testing this soil required another procedure. Two curves were obtained with each representing a constant, though different, value of pore air pressure. This proce-

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[11] Closure to "Cohesion After Non-Hydrostatic Consolidation," by John H. Schmertmann and John R. Hall, Jr., Soil Mechanics and Foundations Division, ASCE, Vol. 88, No. SM 4, August, 1962, p. 163.

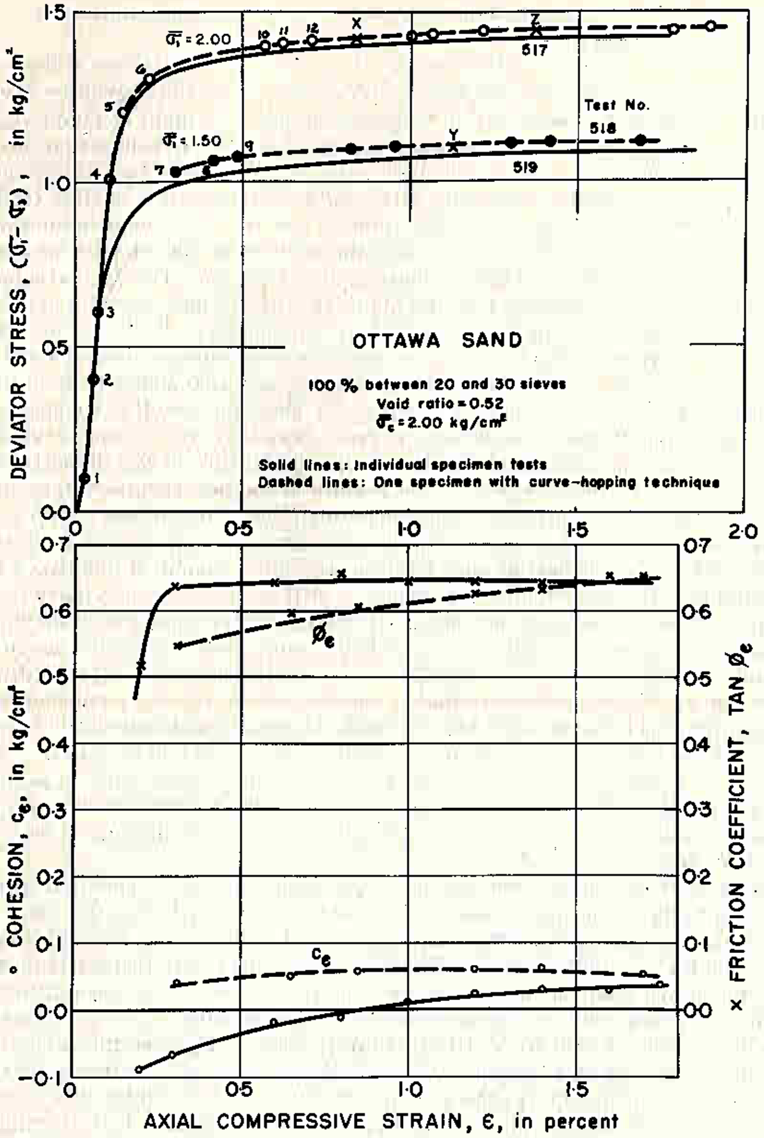

FIG. 1.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, OTTAWA SAND

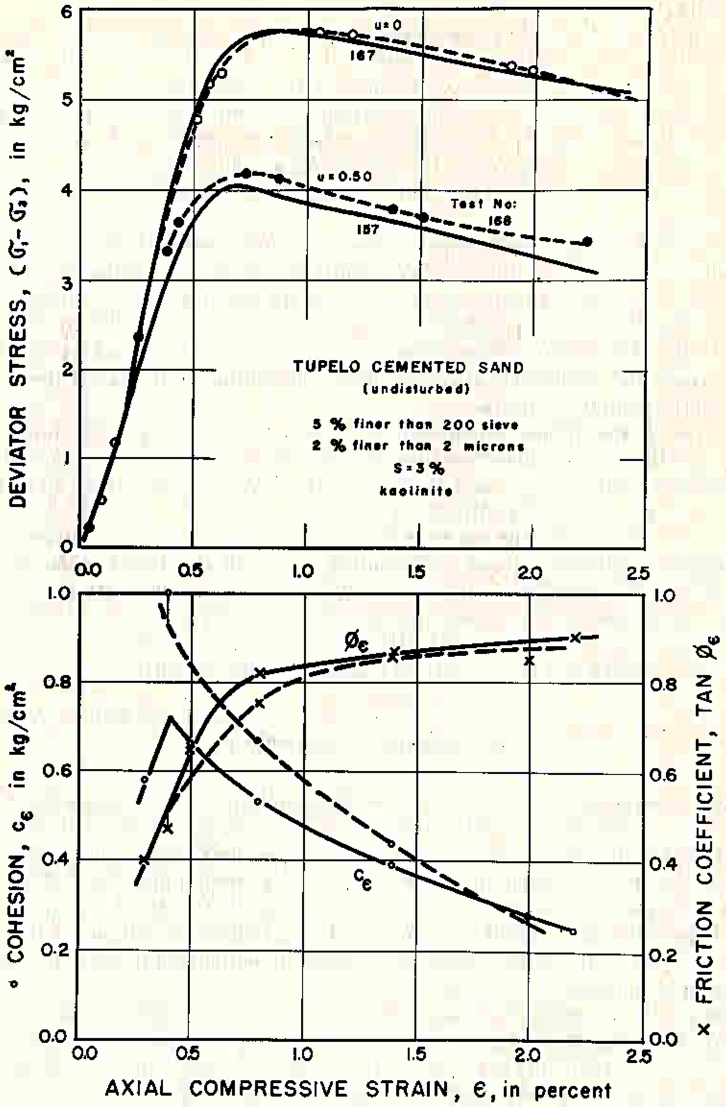

FIG. 2.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, TUPELO CEMENTED SAND

dure also results in two stress-strain points at any strain, each of which is at a different value of $\bar{\sigma}_1$. Calculations for cohesion and friction at this strain can then be made in the same manner as with the "standard," constant $\bar{\sigma}_1$, CFS test procedure.

The original reason for keeping $\bar{\sigma}_1$ constant during the generation of a stress-strain curve was to keep void ratio, and therefore presumably structure, approximately constant in compressible cohesive soil. Soils with significant cement-cohesion are not significantly compressible, and the investigator need not keep $\bar{\sigma}_1$ constant in order to minimize structural changes. Therefore, for the purposes of the comparisons made in this research a constant u version of the CFS test was considered acceptable for the Tupelo cemented sand.

Fig. 3 presents the comparative test results for the Q-EPK. This clay has the same mineralogy as the "Edgar Plastic Kaolin" mixed with distilled water (DWEPK), for which similar comparative tests are presented in Figs. 11, 12, and 13. The difference is that the "as received" powdered kaolin was mixed with a 1/25 M solution of "Quadrofos" $(Na_6P_4O_{13})$. This change resulted in a more dispersed structure after machine extrusion than that of the machine extruded untreated kaolinite.

Groups of specimens of two clays were permitted to consolidate for different lengths of time prior to compression in the one and two-specimen test comparisons. Figs. 4, 5, and 6 present the data for Enid clay, and Figs. 12 and 13 for the untreated kaolinite clay.

Figs. 9 and 11 present the results of a preliminary series of comparative one and two-specimen tests. Subsequent tests on the same soils, shown by Figs. 10 and 12, respectively, permit a check of reproducibility.

Fig. 15 presents the comparative results of two one-specimen tests in which a different sequence of effective stress "curve-hops" was used for each test. The analysis of Fig. 15 will be continued subsequently.

CONSIDERATION OF DATA

When making an evaluation of the agreement, or lack of agreement, between the results of one and two-specimen tests we are faced with the lack of objective standards on which to base such comparisons. Since the point of interest is an evaluation of the curve-hopping technique, the writer herein makes the assumption that the two-specimen test is the standard for comparison. However, the cohesion-friction-strain behavior computed from two-specimen test data is not necessarily a better representation of the strength component behavior of a soil in the field.

Even with a standard, an evaluation of the comparisons depends on whether the cohesion or friction component is of principal interest and on the strain of principal interest. What constitutes a good or poor comparison? Considering that the CFS test may represent the first experimental attempt to separate shear resistance components as a function of strain, and that identical specimens for many natural soils may not be obtainable for use in a two-specimen test, perhaps a deviation as great as 100% is a "good comparison." In view of these uncertainties, this investigation is somewhat general.

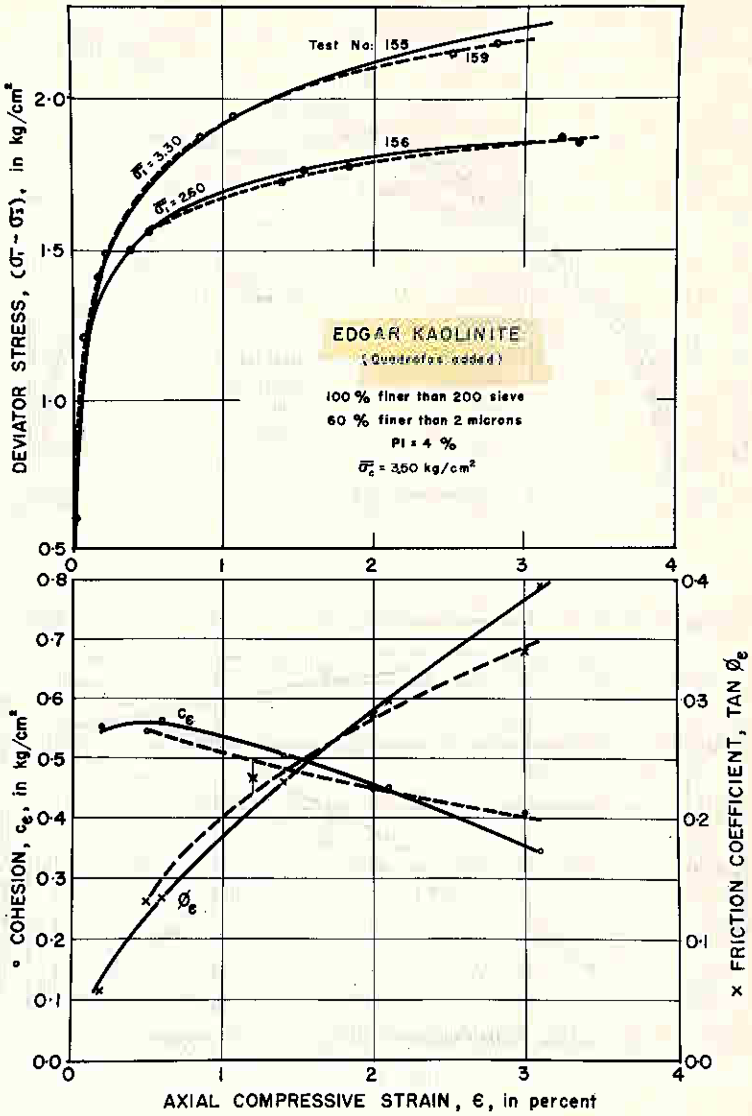

FIG. 3.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED KAOLINITE, DISPERSANT ADDED

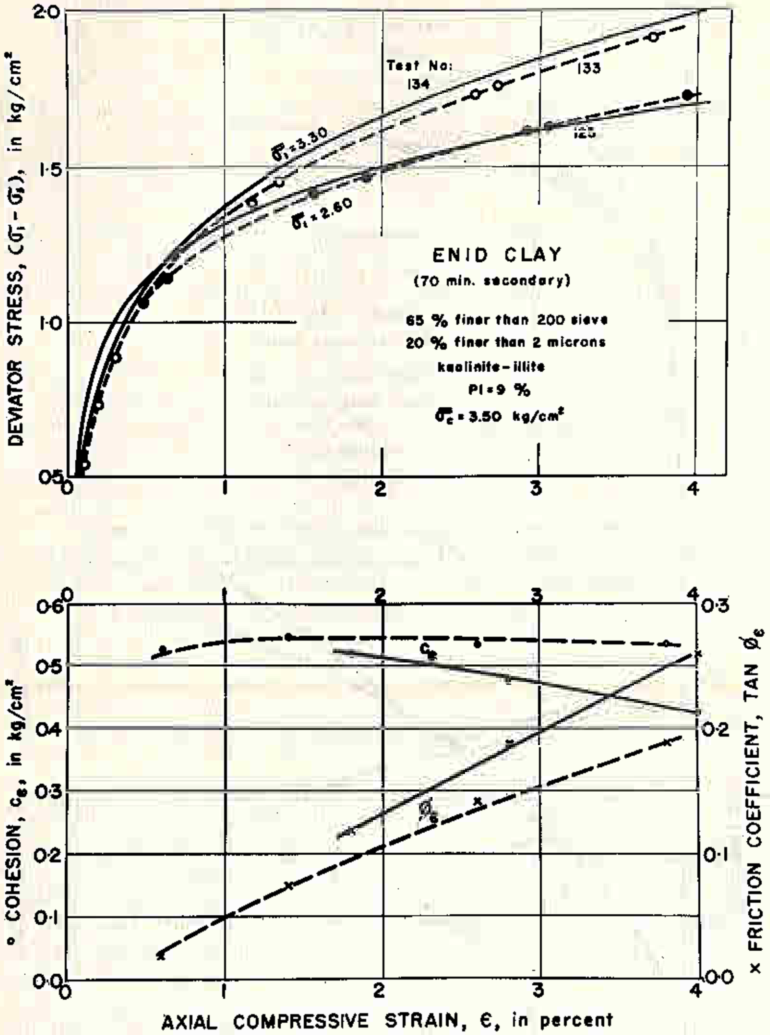

FIG. 4.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED ENID CLAY, 70 MIN SECONDARY

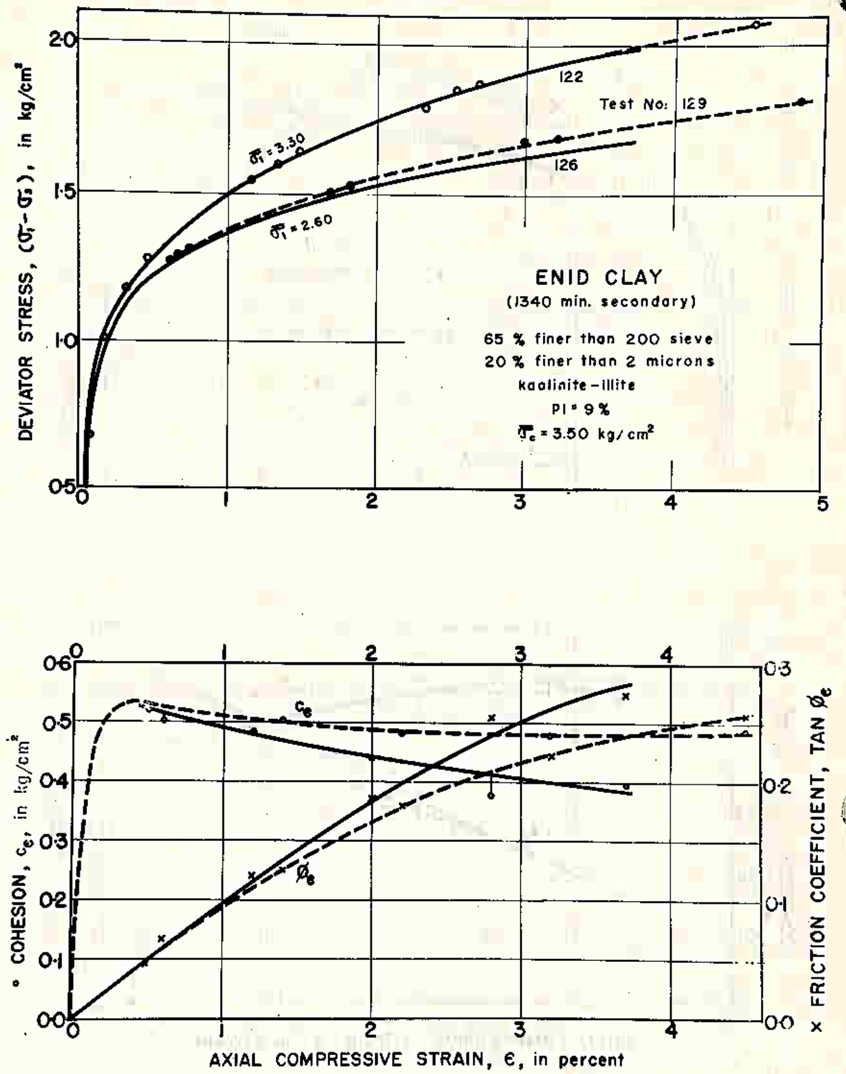

FIG. 5.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED ENID CLAY, 1,340 MIN SECONDARY

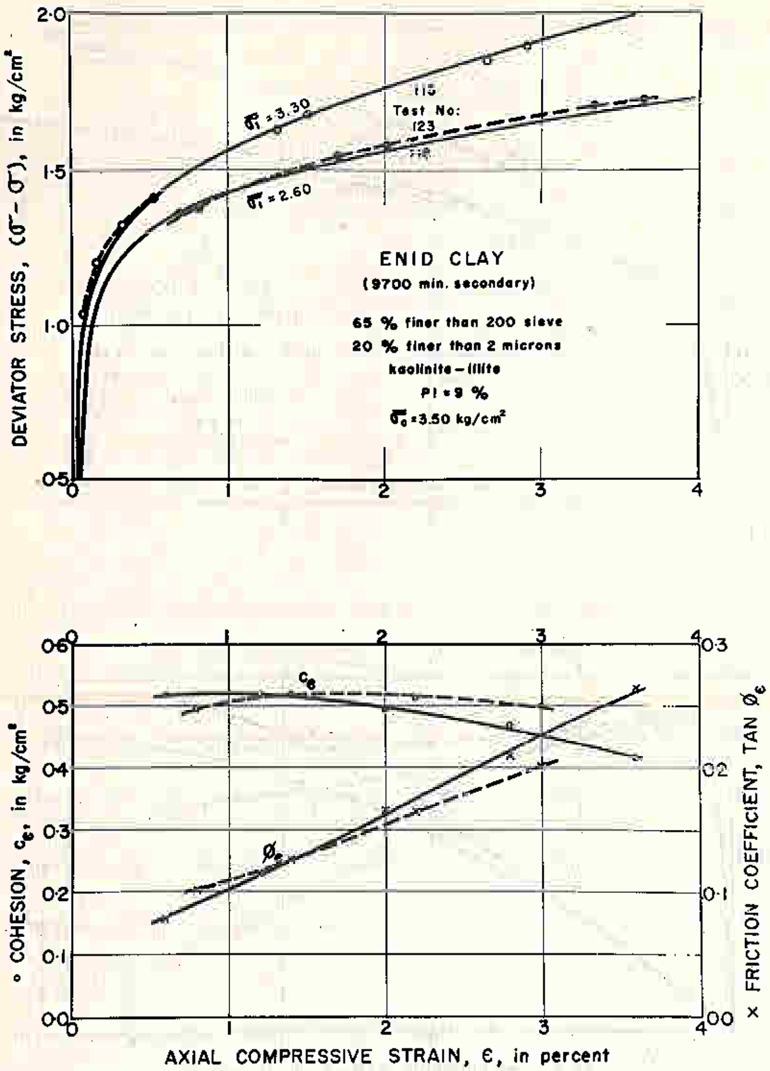

FIG. 6.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED ENID CLAY, 9,700 MIN SECONDARY

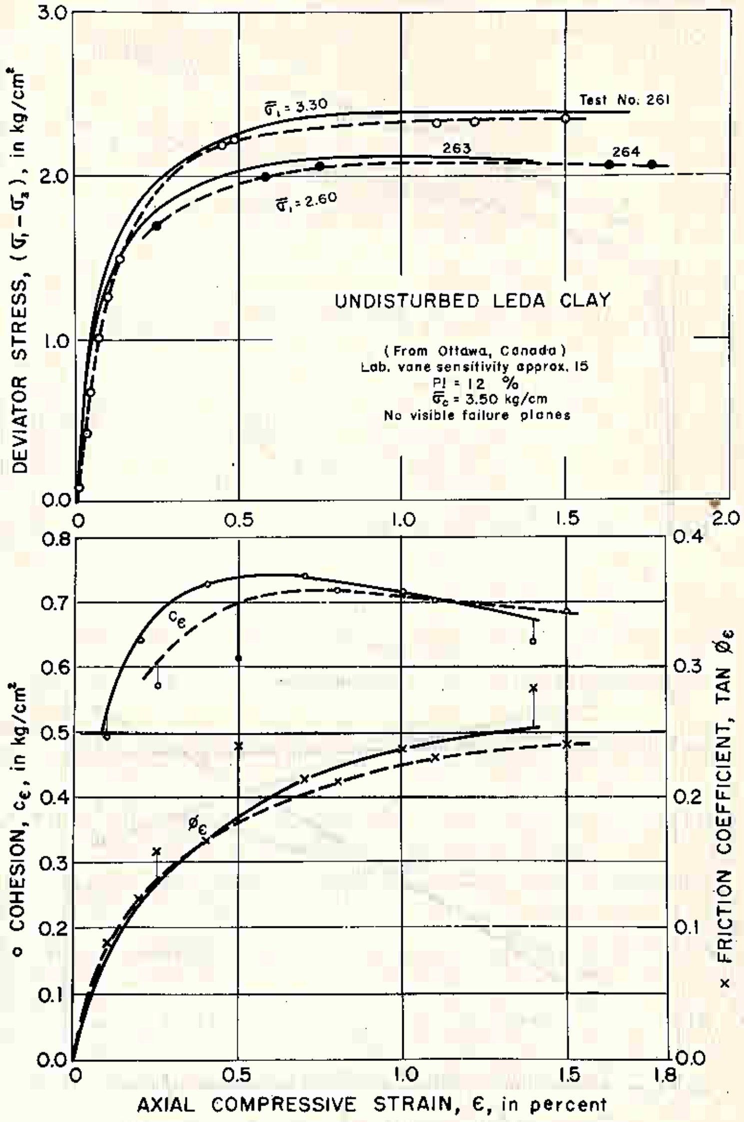

FIG. 7.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, UNDISTURBED LEDA CLAY

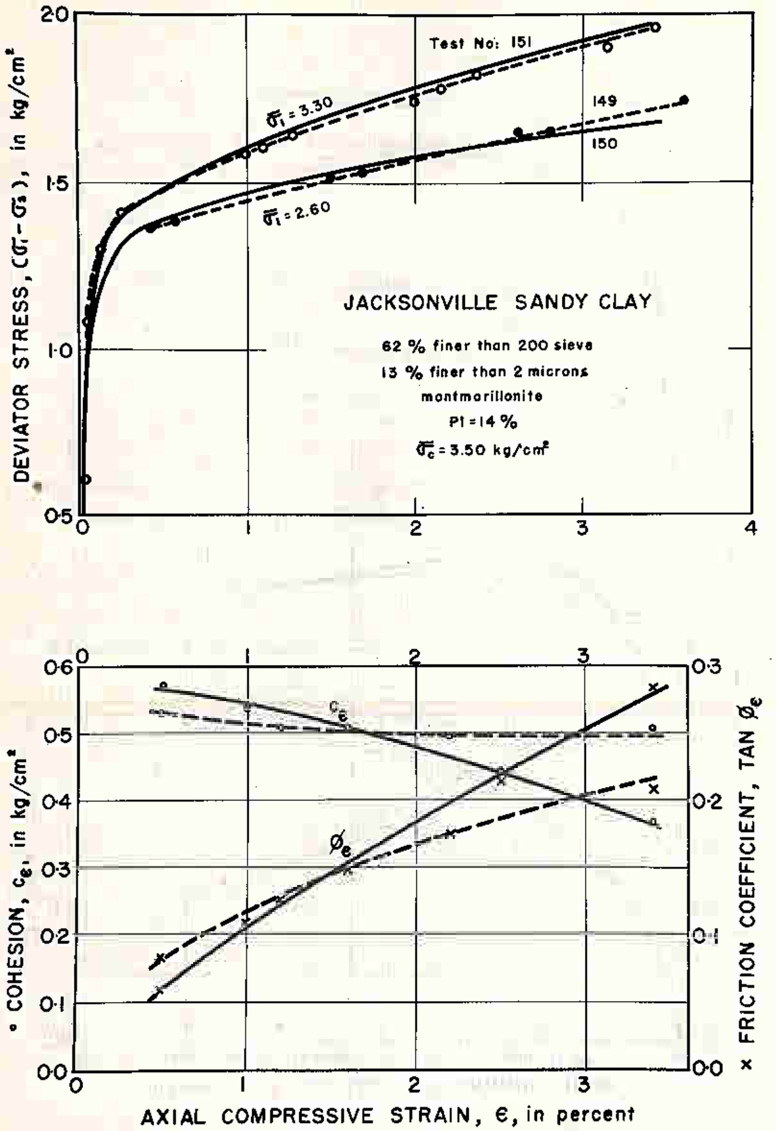

FIG. 8.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, JACKSONVILLE SANDY CLAY

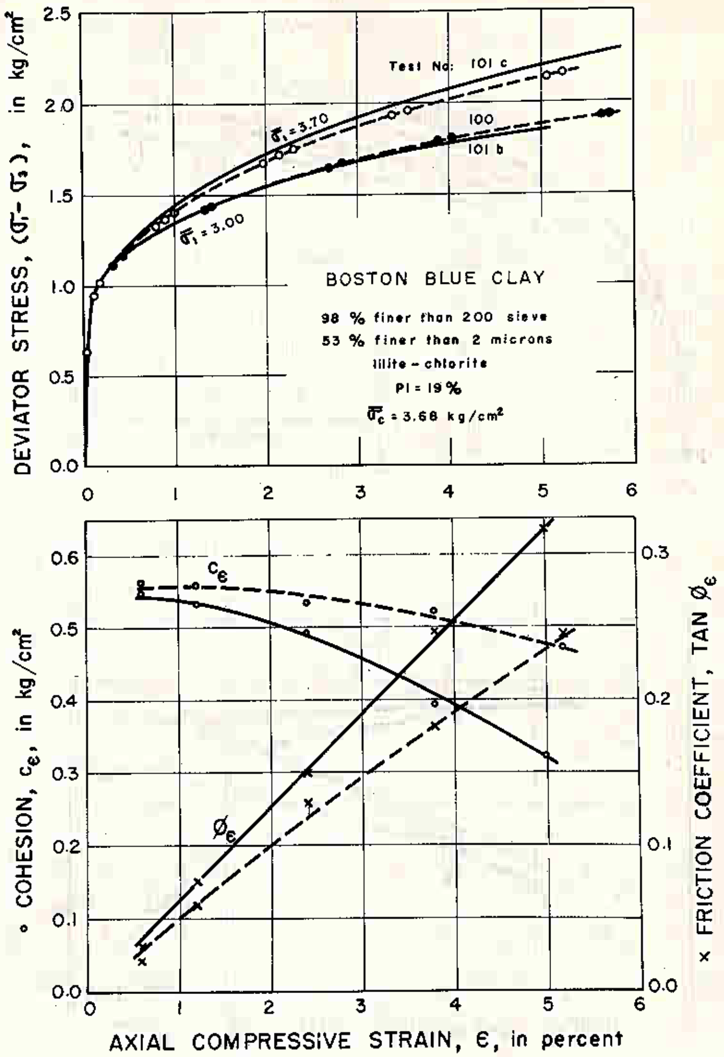

FIG. 9.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED BOSTON BLUE CLAY

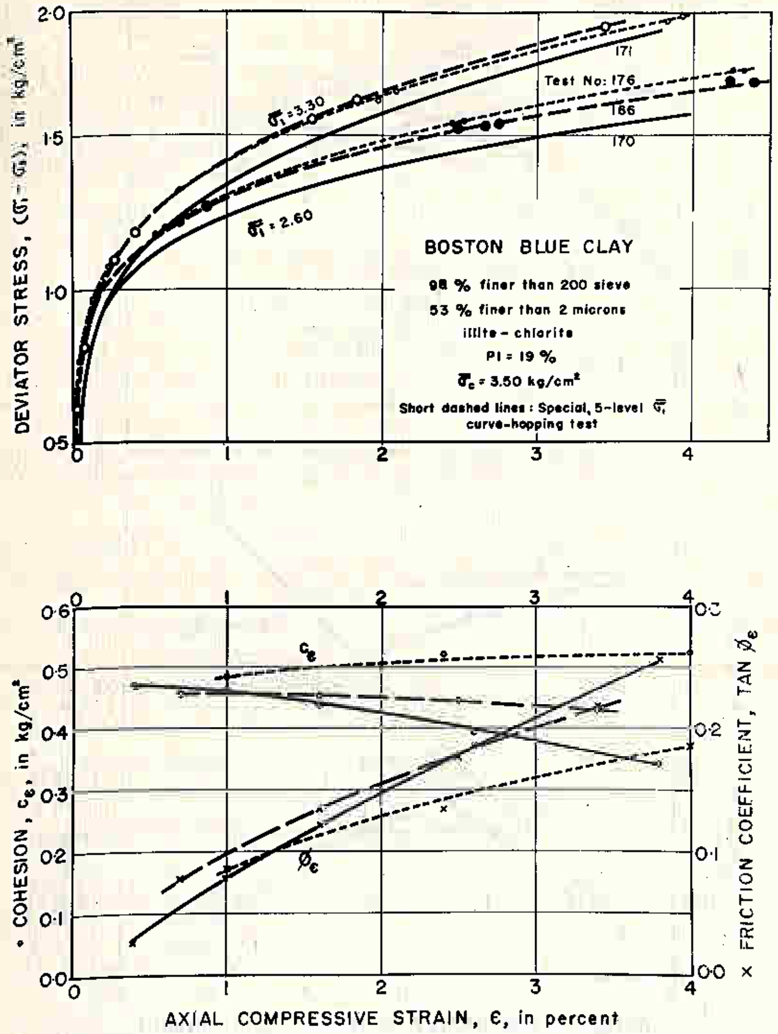

FIG. 10.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED BOSTON BLUE CLAY

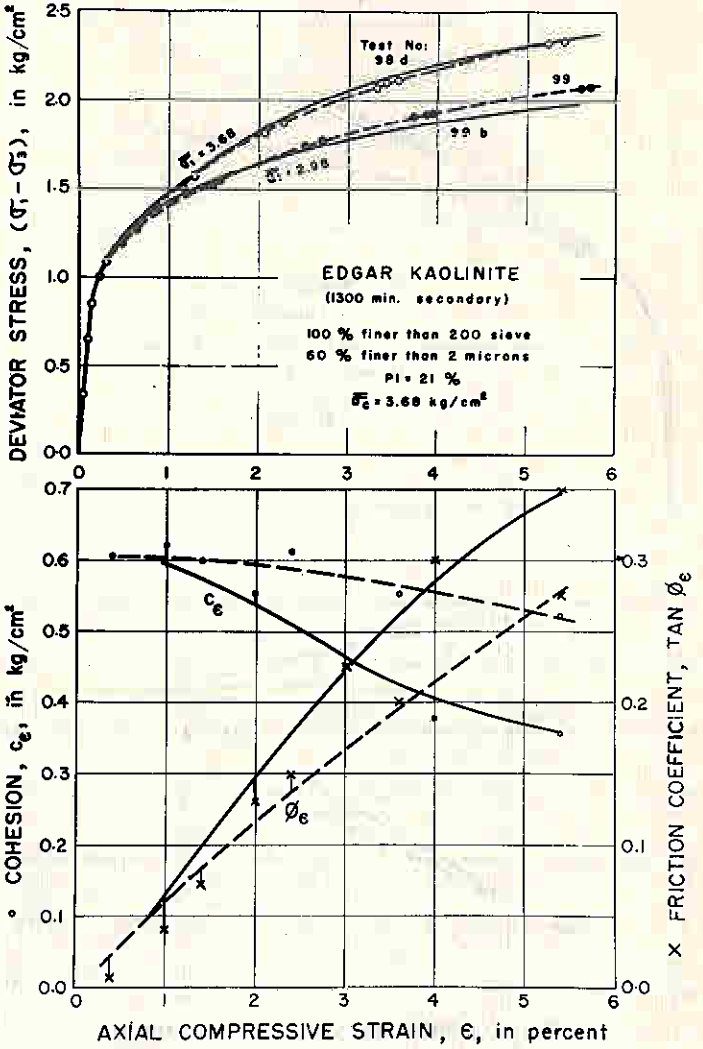

FIG. 11.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED KAOLINITE, 1,300 MIN SECONDARY

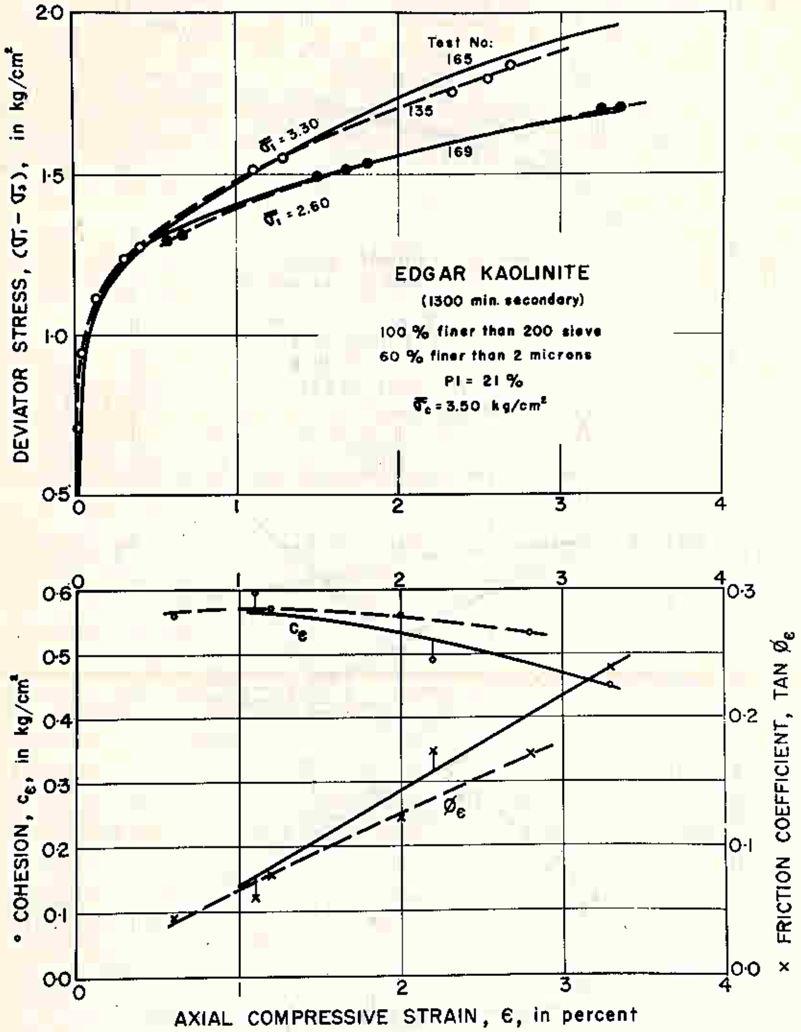

FIG. 12.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED KAOLINITE, 1,300 MIN SECONDARY

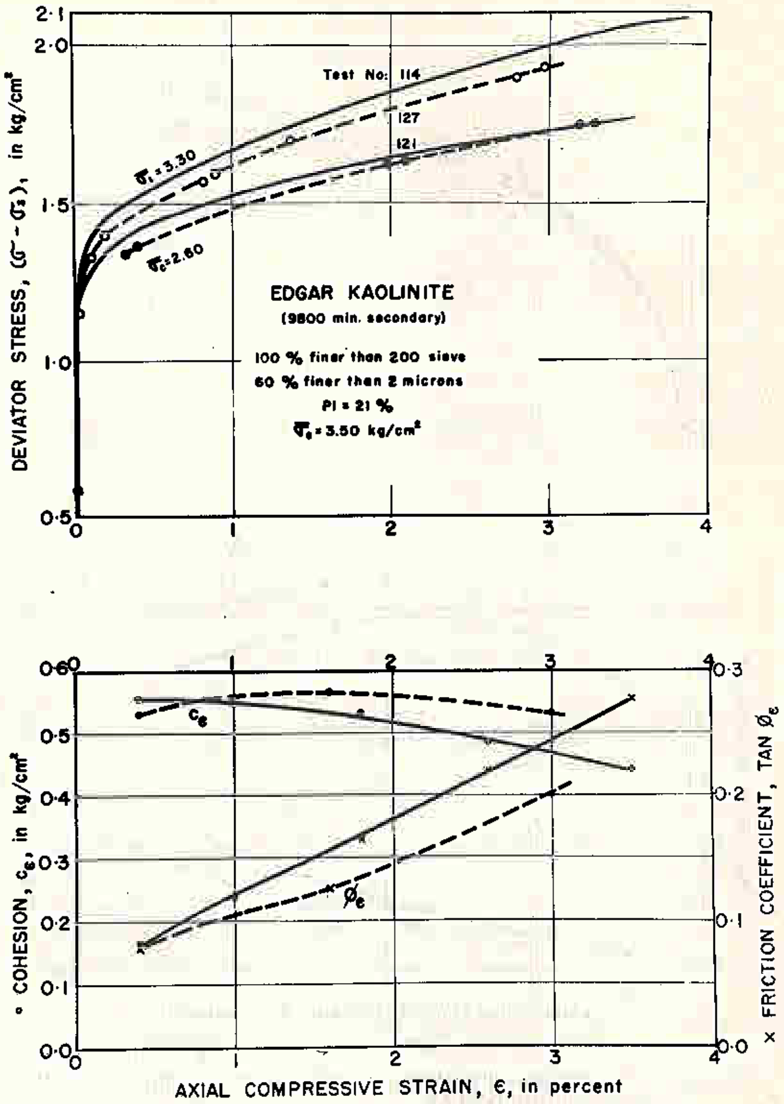

FIG. 13.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED KAOLINITE, 9,800 MIN SECONDARY

$\sigma'_{1h} \approx 3.39$

$\sigma'_{1\ell} \approx 2.69$

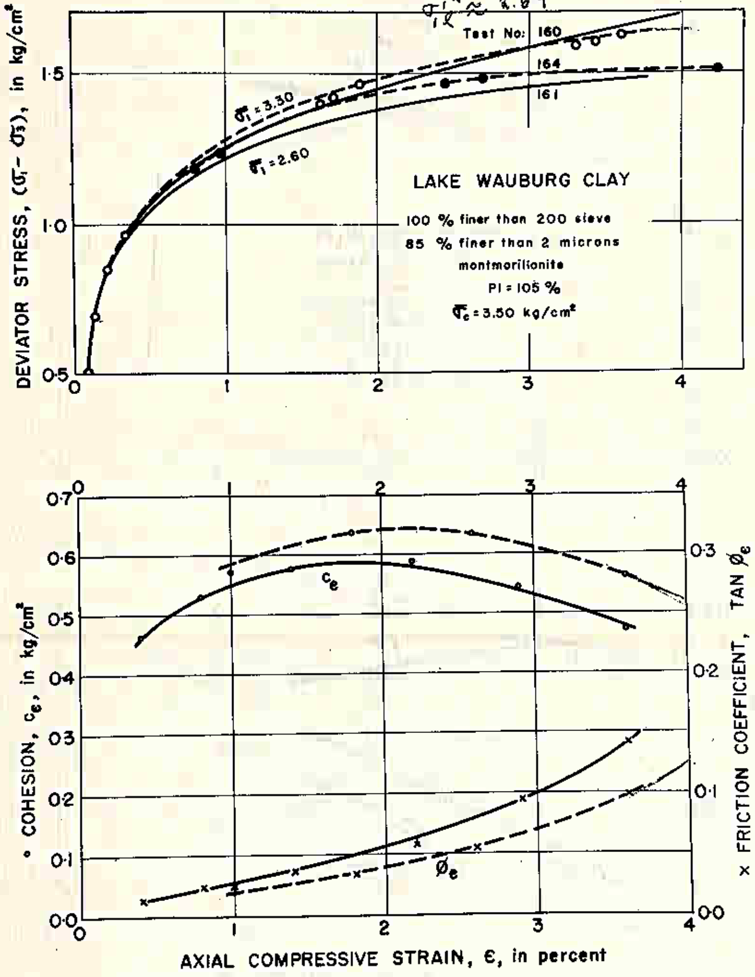

FIG. 14.—ONE AND TWO-SPECIMEN CFS TEST COMPARISON, EXTRUDED LAKE WAUBURG CLAY

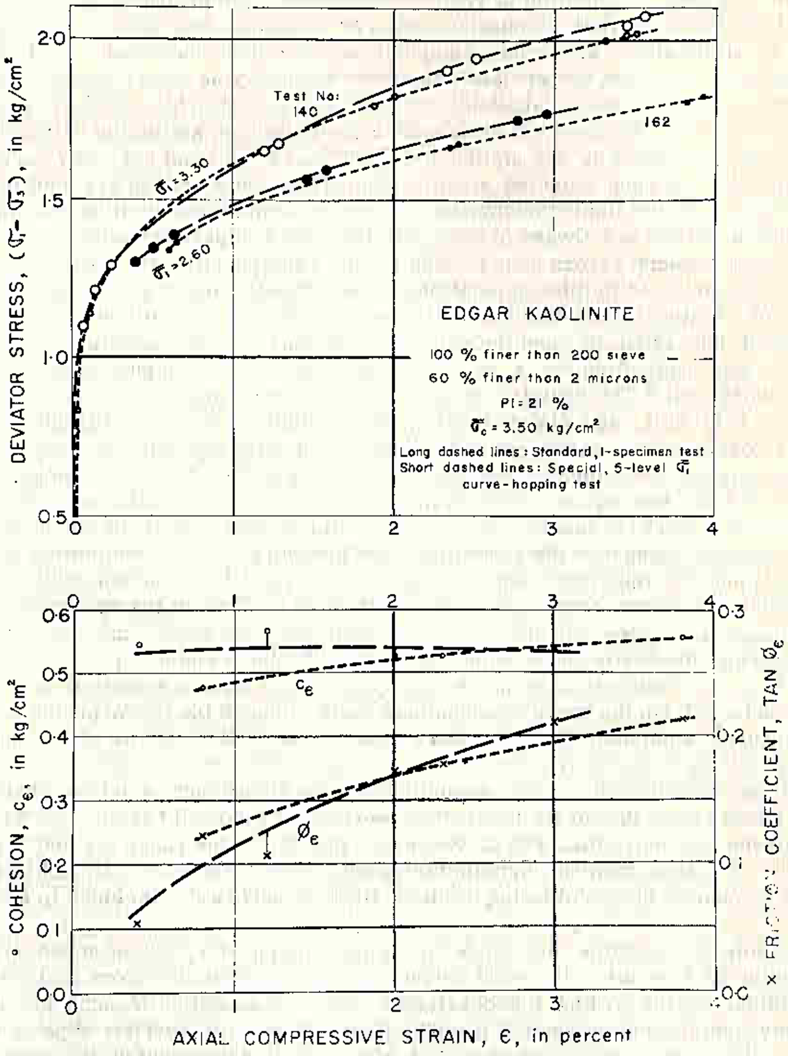

FIG. 15.—COMPARISON OF ONE-SPECIMEN CFS TESTS USING DIFFERENT CURVE-HOPPING SEQUENCES, EXTRUDED KAOLINITE

Measurement and Plotting Errors.—When evaluating the comparisons between the one and two-specimen tests, the reader must be able to separate differences caused by possible errors in experimental measurements, computations, and the plotting of results. Because all computations were done by an IBM 650 or an IBM 709 digital computer, computational errors are essentially eliminated. (Electronic computation is not necessary because of problem difficulty. The writer used computers to save time and manpower and to assure accuracy. Desk calculator methods are also suitable.) Plotting errors can arise when the first set of computer computations, giving the data for the stress-strain curves, are plotted and the values of $\bar{\sigma}_1$, and $(\sigma_1 - \sigma_3)$ are interpolated at each selected strain value. Then these values are sent to the computer for the final cohesion and friction computations. Plotting and interpolation errors are always present, but they are comparatively minor.

Measurement errors can be significant. The experimental measurements directly involved in the computations of the cohesion and friction components at any strain are the triaxial cell pressure, σ_3, the deviator stress applied through the piston, σ_d, and the pore fluid pressure, u. The triaxial cell pressure was controlled by a hydraulic (oil over water) weight-loaded-piston pressure cell.[5] The deviator stress piston load was measured by an external SR-4 load cell, and piston friction was minimized by either oil lubricated ball bearings or rotating piston bushings.[5] The pore pressure was measured with mercury manometers reading to 1.9 kg per sq cm. For pressure greater than 1.9 kg per sq cm mercury manometers were used with the range extended by back pressure from a hydraulic pressure cell. However, these manometers measure the controlled pore pressure value at the bottom of the specimen. An important source of potential error is that the controlled pore pressure may not be the one actually effective throughout the specimen. The drainage aids used and the test procedures (see Appendix I) are designed to eliminate this error and it is not included in Tables 5 and 6.

Data for a statistical study of these plotting and measurement errors were not obtained. On the basis of occasional tests to check the performance of the measuring equipment and continuous observation of the behavior of the equipment during experiments, the writer makes the following estimates of ordinary and extreme error in the magnitude of the significant variables when the stresses are at the levels used in the experiments reported herein. The writer believes that more than 50% of the errors fall within the range termed "ordinary" and less than 10% outside the range termed "extreme." The estimates are presented in the following Table 5. Plotting errors are included in σ_d and u.

Table 6 indicates the range in typical computed values of cohesion and friction as a result of the most unfavorable combination of errors in Table 5, considering the ordinary and extreme cases separately. Because the most unfavorable combination will occur only occasionally, the writer expects that more than 50%, likely more than 90%, of the values will be within the ordinary range and less than 10%, likely less than 2%, will be outside the extreme range.

Cohesion and Friction Comparisons.—It is obvious from the comparative test results (Figs. 1 through 14) that the one-specimen test succeeds well in qualitatively describing the low strain development of both the cohesion and friction components. Thus, one of the principal discoveries of the CFS test research—the rapid strain development of cohesion and the more gradual development of friction—is firmly supported by the results of the two-specimen tests. Of course, the quantitative agreement is good in some comparisons and perhaps only fair, or even poor, in others.

Cohesion.—The agreement in the values of peak (maximum) cohesion is excellent. With the exception of two tests (the cemented sand in Fig 2 and the Lake Wauburg clay (LWC) in Fig. 14) the deviation in peak cohesion is 6% or less. This deviation is within the ordinary range (Table 6). As explained previously, the cemented sand tests involved the use of adjacent undisturbed specimens. It is probable that the specimens were not sufficiently identical and that this accounts for the deviation in the peak cohesion of approximately 25%. The LWC is an exceedingly plastic and impermeable soil, and it is likely that even the slow strain rate (1,665 min per 1% strain) used during the compression of the one-specimen test did not allow sufficient time for pore pressure, or $\bar{\sigma}_1$, equilibrium. This reduces the effect of pore pressure changes and the computed results then indicate a cohesion in error on the high side. Such behavior may account for at least part of the 9% deviation in peak cohesion found in the LWC test (Fig. 14).

TABLE 5.—PRECISION OF MEASUREMENTS AND PLOTTING

Experimental Measurement	Range of Error, in kilograms per square centimeter (Measured - True)			
	Ordinary		Extreme	
σ_3	0.00	+ 0.01	0.00	+ 0.05
u	-0.015	+ 0.025	-0.04	+ 0.08
σ_d	-0.01	+ 0.015	-0.04	+ 0.06

TABLE 6.—RANGE IN COHESION AND FRICTION DUE TO ERRORS SHOWN IN TABLE 5

	Computed Minimum		Assumed True Value	Computed Minimum	
	Extreme	Ordinary		Ordinary	Extreme
(a) Cohesion, c_ϵ, in kilograms per square centimeter					
Low strain	0.35	0.51	0.56	0.60	0.67
High strain	0.21	0.42	0.49	0.55	0.69
(b) Friction Coefficient, tan ϕ_ϵ					
Low strain	0.139	0.066	0.046	0.280	0
High strain	0.424	0.296	0.256	0.220	0.142

The agreement on the strain at which the peak cohesion develops is also good. In only a few cases (Figs. 6, 10, and 13) does there appear to be any important difference in this comparison, and this can probably be accounted for by the flat nature of the cohesion-strain curve. Small errors in cohesion magnitude can result in large errors in the strain position of the peak cohesion.

Comparison of the cohesion values as strain increases shows a general trend for the cohesion to decrease more rapidly with strain in the two-specimen than in the one-specimen CFS test. This deviation varies from a rather small one in Figs. 3, 6, 12, and 14 to a rather pronounced one in Figs. 8, 9, and 11. The cohesion data in Fig. 2 is opposite to this trend, but this may be caused by the special nature of this test, as explained previously. Usually, the greater the strain, or perhaps just the greater the number of curve-hopping cycles, the greater the deviation in cohesion between the two tests, with the one-specimen tests yielding the greater values.

Friction.—With the exception of the Ottawa sand (Fig. 1) and the undisturbed soils (Figs. 2 and 7), the tests were not carried to high enough strains to permit a comparison of the maximum friction values. For the two sands, the agreement was essentially perfect. For the Leda clay it is very good. Such results should be expected for the sands because structure of these soils is not sensitive to effective stress history. The results from Leda clay are surprising.

Comparisons of the strain-rate development of friction indicate, in almost all cases, that the two-specimen test shows a more rapid rate of development than the one-specimen test. It is not known whether this is a cause or consequence of the comparative loss of cohesion with strain.

There is also a tendency (as exhibited in Figs. 3, 6, 8, and 10) for the one-specimen test to have higher friction values at low strain and then to cross-over the two-specimen test with lower values at higher strains. The writer believes that this behavior is caused by experimental errors or by the effect of overconsolidation on the low strain behavior of the low $\bar{\sigma}_1$ specimen in the two-specimen test. No special importance is attached to this cross-over behavior.

Undisturbed Soils.—Because the one-specimen CFS test offers the special advantage of permitting the testing of soils from which duplicate specimens cannot be obtained, as is the case with many undisturbed natural soils, the test comparisons from such natural soils are of special interest. Both the Tupelo cemented sand and the Leda sensitive clay are such soils and the good comparative test results from both are encouraging for the use of the one-specimen testing procedure. Note that the tests for both soils passed through conventional failure.

Because of the difficulty of obtaining duplicate specimens, the writer also approached the problem indirectly via the length of time allowed for secondary compression during the consolidation of two of the extruded cohesive soils. As cited previously, the structural changes resulting from long secondary times are thought to be a major factor accounting for the differences in strength behavior of remolded and undisturbed soils. Therefore, better comparative test agreement with increasing secondary time suggests that undisturbed soils could be successfully tested with the one-specimen test.

Figs. 4, 5 and 6 show the comparative test data for three groups of duplicate Enid clay specimens in which each group was allowed a different time for secondary compression under a constant hydrostatic pressure. These times were approximately 70, 1,340, and 9,700 min, respectively. Although the comparisons of the stress-strain curves and the computed cohesion and friction versus strain are reasonably good for all three groups, the agreement improves with longer secondary time.

Figs. 11, 12, and 13 permit a similar comparison for the kaolinite in which the secondary time was approximately 1,300 min for the groups in Figs. 11 and 12 and about 9,800 min in Fig. 13. A comparison of the results in Figs. 11 and 13 supports the previous conclusion of better one and two-specimen test agreement with longer secondary time. Agreement is approximately the same when comparing Figs. 12 and 13. However, in part, the good results shown in Fig. 12 reflect the fact that the stress-strain curves from the two-specimen test are not properly separated at low strain. Had the specimens been more nearly identical at the beginning of compression, the agreement would not have been as good as that shown in Fig. 12.

On the basis of the limited evidence presented, it appears that the use of the one-specimen CFS test, with its curve-hopping technique, will be successful with undistrubed soils. However, highly overconsolidated cohesive soils (either undisturbed or extruded) were not tested, and such soils may present unexpected problems. But, they could also prove to yield even better test comparisons.

CONDITIONS FOR SUCCESSFUL ONE-SPECIMEN TESTS

Type of Soil.—This paper is partly a survey of those soil types that appear suitable for use with the one-specimen CFS test procedure. From the results presented herein, as well as from experience with many other CFS tests, the writer believes that all soils investigated to date are suitable. Some are more easily tested than others. In general, the higher the plasticity index the more difficult the performance of the test because of the accompanying lower permeability and the fact that more time must be allowed for pore pressure uniformity within the specimen.

For the engineer interested in trying the one-specimen CFS test in his laboratory, the writer suggests that it is easiest to obtain good results on a soil having a low PI; Ottawa sand is an ideal materal for pilot tests.

Choice of Speed of Compression.—It is important that the investigator be aware of the maximum compression rate at which a CFS test can be successfully performed. This is dependent on the speed with which the pore pressure changes imposed during curve-hopping can be distributed throughout the specimen. This, in turn, is dependent on the permeability of the soil and its resistance to the volume changes associated with pore pressure changes. It appears convenient to use the length of time required to complete primary consolidation during the consolidation increment that precedes the performance of the CFS test, as a measure of permeability and resistance behavior. This time is designated t_{100}, and can usually be determined by a method such as the well-known Casagrande graphical method. Based on experience to date (May, 1962) in which t_{100} was obtained during normal consolidation, the writer suggests that the CFS test be performed with a compression rate not greater than 1% axial compressive strain per t_{100} time interval.

The strain interval of interest for a cohesion-friction separation also influences the compression rate chosen. The more detail desired over any strain interval, the slower the required compression rate to permit a sufficient number of curve-hopping cycles to obtain this detail.

Artificial drainage aids can greatly reduce t_{100} and decrease the time necessary for a CFS test. For instance, the writer performed duplicate, one-increment consolidation tests on saturated kaolinite clay specimens 8.00 cm long and 3.59 cm in diameter, drained at the bottom only. Each external drain was one vertical strip of Whatman No. 54 filter paper approximately 1/4 in. wide (1/8 in. when No. 12 used). An internal drain was punched axially with an 0.19 cm diameter needle and filled with a double length of wool yarn. Table 7 presents the results obtained. It is apparent that drainage aids can be effective in reducing the time for a CFS test if this is desired. The problem is usually with the very impermeable clays in which the use of the maximum number of internal drains that will not seriously damage the soil structure still results in high t_{100} values. Then, the CFS test simply must be run slowly, often spanning several days or even weeks.

Precision of Triaxial Equipment.—To perform one-specimen CFS tests with the accuracy shown in Table 6, the investigator must have equipment capable of the precision of measurements indicated in Table 5. Furthermore, this equipment must be used with the care necessary to maintain this precision. Less precise equipment will necessarily result in greater probable error in the individual cohesion and friction determinations.

TABLE 7.—EXAMPLE OF USE OF DRAINAGE AIDS TO REDUCE t_{100}

Number of drains		t_{100}, in minutes
External	Internal	
0	0	550
4	0	210
4	1	70
4	3	55
4	5	33
12	7	22

Although all the published CFS test work has been done with the triaxial machine, it is not essential to use this machine. It is merely convenient because the average stress conditions are known with sufficient accuracy to construct Mohr circles of stress and to interpret small changes in these circles (see APPENDIX I). The CFS test theory is intended to be generally valid. It should be possible to use any of the discussed forms of the test in conjunction with any suitable stress-strain measuring device. It is not certain that curve-hopping will be successful in other strain controlled devices, but the writer knows of no reason to think otherwise.

Variations in the Type of CFS Test.—The test procedure reviewed in the Introduction and in APPENDIX I, examined in previous publications,[3,4] and used for all the one-specimen tests reported herein (except Fig. 2), is called the standard CFS test. This is a constant $\bar{\sigma}_1$ test. This test could not be used with the Tupelo cemented sand and a modification was made, as described previously, resulting in a constant u version of the CFS test. Other versions are possible. One that may be of special interest to the reader, the constant volume CFS test, is considered.

In the constant volume version, the investigator performs the curve-hopping between two predetermined levels of constant volume. The saturated soil is compressed in a strain-controlled test with all drainage valves closed. Pore pressure measurements must be obtained during compression with a suitable no-flow device. The pore pressure line is then opened, and the increment of volume change is imposed by increasing the pore pressure in order to force water in, or by decreasing pore pressure in order to permit the intended volume reduction. After the volume change is accomplished, the line is closed. After allowing time for equilibrium under the new conditions, data are taken at the new volume level. The test consists of hopping between the two volume levels and finally obtaining two stress-strain curves—one for each constant volume. A $\bar{\sigma}_1$ versus strain curve must also be drawn for each constant volume condition. Then, at any strain, the two deviator stress and $\bar{\sigma}_1$ values are interpolated and the cohesion and friction calculations can be performed in the same manner as in the standard CFS test. An example of such a test is available.[15]

This version of the CFS test requires not only a no-flow pore pressure measuring device, but also equipment to measure accurately volume changes of less than 0.5 cu cm forced on the specimen. Thus, this version of the test requires additional equipment. Because pore pressure is measured rather than controlled, and $\bar{\sigma}_1$ interpolation is more difficult, the test is also somewhat less accurate. However, the no-volume-change condition may be of practical interest. The writer has performed a few tests of this type, and the results are similar to those obtained from constant $\bar{\sigma}_1$ tests at the same effective stress levels.

It is also possible to perform a stress controlled CFS test using either of the three versions. However, curve-hopping is not possible, and the investigator must use a two-specimen test with the attendant uncertainty of whether or not specimens have been genuinely duplicated.

Importance of $\bar{\sigma}_1$ Control.—One might ask about the importance of maintaining continuous, accurate control of $\bar{\sigma}_1$ during the CFS test, and particularly during curve-hopping. Can $\bar{\sigma}_1$ control be lost, with considerable deviation from the intended level, and yet return to the correct stress-strain curve as if control was not lost?

In part to answer the above question the writer performed tests in which the sequence of curve-hopping was 3.30, 2.95, 2.60, 2.25, 1.90; 3.30, 2.95 kg per sq cm, and so forth. Note that these five stress levels included the 3.30 and 2.60 commonly used herein for the one-specimen tests. Using only these two of the five stress levels, the stress-strain curves and the cohesion and friction curves computed from them are presented in Figs. 10 and 15 (short dashes). There they can be directly compared with one-specimen tests performed with only two stress levels (long dashes). The comparisons are good and indicate that temporary loss of $\bar{\sigma}_1$ control would have only a minor, and perhaps negligible, effect on the subsequent position of the stress-strain curves at the intended $\bar{\sigma}_1$ levels.

However, if $\bar{\sigma}_1$ control is poor and each point deviates considerably and randomly from the intended value, then $\bar{\sigma}_1$ interpolation at any strain is less accurate and the component separation is less accurate. In other versions of the CFS test it is desirable that $\bar{\sigma}_1$ versus strain plot as a smooth curve so that interpolation can be accurate. The more closely this curve approximates constant $\bar{\sigma}_1$, the more accurate this interpolation.

CONCLUSIONS

From the results of this research involving comparisons of one and two-specimen CFS tests using Ottawa sand, undisturbed cemented sand, and sensitive clay and six machine extruded soils with plasticity indexes between 4 and 21 and at 105%, the writer reaches the following conclusions:

1. The one-specimen test, using a curve-hopping technique, when compared with a two specimen test not using this technique gives excellent qualitative agreement in the curves of cohesion and friction against strain. Quantitative agreement on the maximum cohesion values is also excellent. These comparisons promote confidence in the validity of the use of the one-specimen test with the variety of soil types tested.
2. There is a general tendency for cohesion to decrease, and friction to increase, more rapidly with strain in the two-specimen tests.
3. The tests on the two undisturbed soils showed excellent comparative one and two-specimen test agreement. Increasing the length of time allowed for secondary compression in two of the extruded clays resulted in improved comparative agreement. The writer interprets the improvement as additional evidence that undisturbed soils can be tested successfully with the one-specimen test.

ACKNOWLEDGMENTS

The writer is pleased to thank the Engineering Sciences Division of the National Science Foundation for Grant No. G 14671, which made this work financially possible. The writer also greatly appreciates the encouragement and suggestion by Karl Terzaghi, Hon. M. ASCE, that a study of this type be pursued.

Carl B. Crawford of the Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada, kindly sent the sample of undisturbed Leda clay.

APPENDIX I.—CFS TEST DEFINITIONS, PROCEDURE, AND EXAMPLE ANALYSIS

Component Definitions.—The CFS test research began with an attempt by the writer to find a method of separating the Hvorslev effective cohesion and friction components[12] over the entire strain range of a compression test, rather than only at failure. This proved impossible.[13] New component definitions had to be developed that permitted a variation in void ratio for equal cohesion. However, the variation permitted is slight (less than 1%), and the new definitions are thought to be at least approximations of the Hvorslev components. Hence, the use of the terms cohesion and friction was continued in the writer's previous works[3,4] and, for continuity, in this paper.

The writer now prefers to call CFS test friction and cohesion "D" and "I," respectively. This frees the mind from the many possible prior associations with the terms "friction" and "cohesion" and one can more serenely contemplate the CFS test as a means of distinguishing two arbitrary shear resistance components, D and I, which may or may not have engineering significance.

The following is a rewording of the original definitions of ϕ_ϵ and c_ϵ[3]. The meaning remains essentially the same, but the definitions are now stated in mathematical terms and include component separation on any plane.

"D_ϵ" = Dependent component.—Component of shear resistance mobilized on any plane and at any strain = ϵ, which is dependent on effective stress on that plane according to the equation

$$D_\epsilon = \left[\bar{\sigma}\left(\frac{\Delta\tau}{\Delta\bar{\sigma}}\right)\right]_{\epsilon \atop \Delta\bar{\sigma} \to 0} = \left[\bar{\sigma}\left(\frac{d\tau}{d\bar{\sigma}}\right)\right]_{\epsilon} \quad \ldots\ldots\ldots\ldots (1)$$

when, at ϵ, the shear resistance on that plane changes by $\Delta\tau$ due to $\Delta\bar{\sigma}$. Because there must be no change in soil structure, $\Delta\bar{\sigma}$ must approach zero.

"I" = Independent component.—Remaining component defined by

$$I_\epsilon = \tau_\epsilon - D_\epsilon \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

when τ_ϵ is the total shear resistance mobilized at ϵ on the plane considered.

Fig. 16 graphically illustrates, in the incremental form necessary in experiments, the definitions of these components for the plane of Mohr envelope tangency. This is the only plane considered in this paper.

Of course, it is hoped to eventually prove that these components are directly related to fundamental cohesion and friction. Some data to support this has been published.[14] However, such proof is not essential to appreciate the possible contributions of this paper, which deals with establishing the validity of laboratory technique.

One-Specimen Test Procedure.—The test to be performed is the constant $\bar{\sigma}_1$ CFS test in which two stress-strain curves are obtained, one for each magnitude of $\bar{\sigma}_1$. In the triaxial machine, $\bar{\sigma}_1$ is always on the horizontal plane and is equal to the cell pressure plus the deviator stress minus the pore pressure; or $(\bar{\sigma}_1 = \sigma_3 + \sigma_d - u)$. Then, to maintain $\bar{\sigma}_1$ as a constant when the cell pressure is kept constant it is only necessary to keep

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[12] "Physical Components of the Shear Strength of Saturated Clays," by M. Juul Hvorslev, ASCE Research Conference on the Shear Strength of Cohesive Soils, June, 1960, p. 211.

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

[14] "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. 88, No. SM 3, Proc. Paper 3158, June, 1962, p. 1.

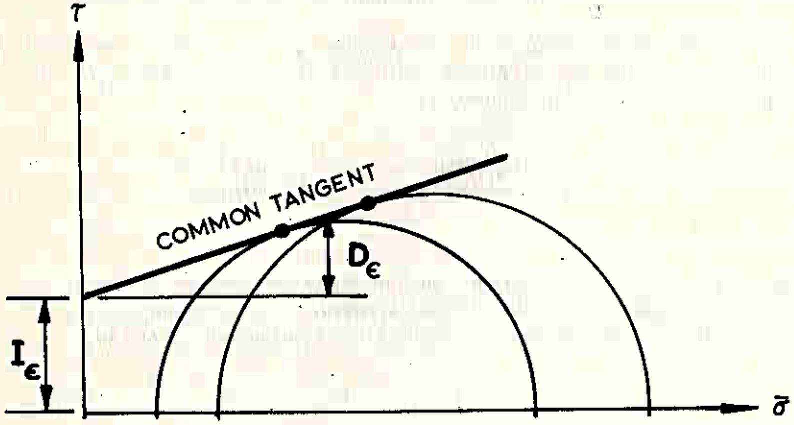
$$u = \sigma_d + \left(\sigma_3 - \bar{\sigma}_1\right) \quad \ldots\ldots\ldots\ldots\ldots \quad (3a)$$

or

$$u = \sigma_d + (\text{a constant}) \quad \ldots\ldots\ldots\ldots\ldots \quad (3b)$$

A different constant is used for each magnitude of $\bar{\sigma}_1$.

The pore pressure is controlled by periodic adjustments to continually (as the soil strains) fulfill the requirements of Eq. 3a. Of course, there must be some assurance that the pore pressure control imposed at the bottom porous stone is the pore pressure within the specimen. Special internal drains can be used to speed pore pressure transmission (see Table 7). In addition, the data is accumulated in such a way as to prove extra points on the stress-strain curves, and these provide additional assurance that $\bar{\sigma}_1$, or pore pressure, equilibrium is practically established.



Both stress circles obtained at strain = ε ,
with less than 1 % change in void ratio.

FIG. 16.—ILLUSTRATION OF "I" AND "D" COMPONENTS MOBILIZED AT STRAIN ϵ

The first ten points in the one-specimen test on Ottawa sand have been numbered in Fig. 1, and they will be referred to in the following explanation.

After consolidation to $\bar{\sigma}_c$ (2.00 in test No. 518) the pore pressure may or may not be increased slightly to establish the magnitude of $\bar{\sigma}_1$ for the upper curve. In test 518, the upper $\bar{\sigma}_1$ curve was at $\bar{\sigma}_1$ = 2.00. Strain controlled axial compression is then imposed and the deviator stress increases and one obtains points 1, 2, 3, and so forth. With the increase in σ_d, the pore pressure must be adjusted, as explained, to maintain a constant $\bar{\sigma}_1$. After point 6, the investigator decided to hop to the lower curve. As explained in the Introduction, the pore pressure increase resulted in the reduction of the magnitude of σ_d that the specimen could sustain at that strain. After some additional strain, during which the investigator periodically adjusts the pore pressure to the changing deviator stress to more closely approach the intended lower magnitude of $\bar{\sigma}_1$, a first data point is obtained at the new magnitude. This is point 7. After continuing the pore pressure control, now much stabilized, point 8 is obtained. The process may be continued and still another point, No. 9, is obtained. The positions of these points on a load-deflection curve that is kept concurrent with the progress of the test indicates that it is likely that they adequately define a portion of the lower $\bar{\sigma}_1$ stress-strain curve. From the complete curves, it may be seen that point 7 alone was adequate to position the lower curve, but point 8 provides additional assurance, and 9 still more, that a practical $\bar{\sigma}_1$ equilibrium was established.

The pore pressure is then decreased and periodically adjusted to maintain the upper $\bar{\sigma}_1$ magnitude. After a similar waiting and adjustment period, during which the controlled strain continues, the investigator ventured taking the data for point 10. The check points 11 and 12 were also obtained. The remaining cycles of curve-hopping are repetitions of the one described. Normally at least one check point is obtained, and often the first is discarded as premature and the check point becomes the new first and is in turn checked.

Example Analysis.—For the reader who wishes a ready reference to the manner in which the lower cohesion friction-strain curves in Figs. 1 through 15 were computed from the upper stress-strain curves, an example analysis follows. The analysis is for the strain ϵ = 0.31% in test 518.

Sufficient data is available to plot two Mohr circles for the strain of 0.31%. The mathematical problem is to fit the common tangent to these two circles, as shown in Fig. 16, and then to extract the slope angle and the tau axis intercept. Analysis of the geometry of the problem gives the following equations for the desired components:

$$\tan\phi_\epsilon = \tan\left\{\sin^{-1}\left[\frac{\Delta\sigma_d}{2\left(\Delta\bar{\sigma}_1\right)-\Delta\sigma_d}\right]\right\} \quad \ldots\ldots\ldots\ldots \quad (4)$$

and

$$c_\epsilon = \frac{\frac{\sigma_d}{2} - \left(\bar{\sigma}_1 - \frac{\sigma_d}{2}\right)\sin\phi_\epsilon}{\cos\phi_\epsilon} \quad \ldots\ldots\ldots\ldots \quad (5)$$

It should be noted that σ_d and $\bar{\sigma}_1$ must be taken from the same curve, but can be either.

For the upper and lower curves, the σ_d values are obtained directly from the stress-strain curves interpolated through the data points and spanning the strain gaps during which the control is for the other $\bar{\sigma}_1$. In this case, the σ_d values are 1.348 and 1.034, for a $\Delta\sigma_d$ = 0.314 . Although the $\bar{\sigma}_1$ values were intended to be 2.00 and 1.50, because of minor control inaccuracies they are usually slightly different from this, and it is necessary to interpolate a value for the strain investigated from the nearest points on each curve. In this case, the interpolated values of $\bar{\sigma}_1$ are 2.005 and 1.521 and $\Delta\bar{\sigma}_1$ = 0.484. Using Eqs. 4 and 5 then gives tan ϕ_ϵ = 0.547 and c_ϵ = 0.040, which are the values plotted in the lower part of Fig. 1.

APPENDIX II.—NOTATION

The following symbols have been adopted for use in this paper:

c = cohesion term in Coulomb's equation;

c_ϵ = cohesion at strain ϵ;

D_ϵ = suggested CFS test component of mobilized shear resistance, replaces $\bar{\sigma}$ tau ϕ_ϵ;

e_f = void ratio of specimen at end of CFS test;

e_i = void ratio of specimen before placing in triaxial cell;

e_o = void ratio of specimen after consolidation, before CFS test;

G_s = specific gravity of soil solids;

I_ϵ = suggested CFS test component of mobilized shear resistance, replaces c_ϵ;

LL = Atterberg liquid limit;

PI = Atterberg plasticity index;

S_f = degree of saturation at end of CFS test, after removal from triaxial cell;

S_i = degree of saturation before placing in triaxial cell;

t_{100} = time required for primary consolidation;

u = pore fluid pressure, air or water saturated;

w_f = water content at end of CFS test, after removal from cell;

w_i = water content before placing in triaxial cell;

ϵ = axial compressive strain (also used in general sense to denote strain);

$\bar{\sigma}$ = effective stress on any plane;

σ_1 = major principal stress;

$\bar{\sigma}_1$ = major principal effective stress;

σ_3 = minor principal stress, equals triaxial cell pressure;

$\bar{\sigma}_c$ = preconsolidation pressure by lab hydrostatic consolidation;

σ_d = greatest principal stress difference, $\sigma_1 - \sigma_3$, deviator stress;

τ = shear stress;

τ_ϵ = shear stress at strain ϵ;

ϕ = angle of internal friction in Coulomb's equation; and

ϕ_ϵ = angle of internal friction at strain ϵ.