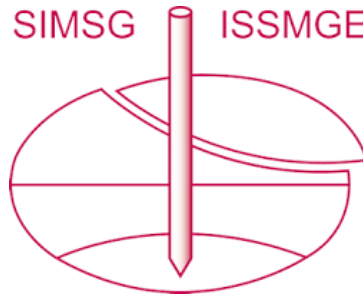


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There is, however, a much bigger difference between the two methods of testing in the same size box. This is also borne out by a few tests carried out at a very rapid constant rate of strain, which also gives lower values of ϕ . This indicates that the value of ϕ obtained does not depend to any great extent on the size of the machine (over a limiting size), but that discrepancies between measured values of ϕ and the results of field or model loading tests etc., may lie in the different types of failure induced.

3. The stress distribution within the sample and along the failure plane is not discussed in this paper, but preliminary examinations of the type of failure plane by Kotter's equation indicate that the deviation from the average of the normal stress on the failure plane is not serious.

VI. CONCLUSIONS.

A full range of cohesionless soils, ranging from sands to gravels and sandy gravels, has been tested in a shear box taking samples 1 foot square and 6 inches thick. For sands it has been found that the results obtained in the standard 6 cm. shear boxes are in good agreement with those obtained in this large box. It should be noted however that in the standard boxes the results appear to depend to some extent on the manner of applying the shear force.

It was found that materials with a uni-

form grain size, whether a medium sand or pebbles about 1 inch in diameter, gave similar results when plotted in the form of ϕ against porosity which were quite distinct from the relationships found for the graded materials.

The graded sands and sandy gravels lay more or less in the same general zone, but the lower porosities of the well graded sandy gravels resulted in higher angle of friction than the graded sand. The sandy gravels were found to have angles of friction as high as 50° at a dense packing.

VII. ACKNOWLEDGEMENTS.

For permission to publish details of the machine and of the Walton tests the Author is indebted to H.F. Cronin, M.I.C.E., Chief Engineer of the Metropolitan Water Board.

The Author is grateful to J.B. Miners, A.C.G.I. B.Sc. (Eng.) A.J. Smallman, A.C.G.I. B.Sc. (Eng.) and M.A.A. Hafez B.Sc., for carrying out much of the testing programme referred to, and to Assistant Professor A.W. Skempton for his constant interest.

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Proc. 2nd Int. Conf. Soil Mechanics.

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THE SHEARING RESISTANCE OF SOILS AS DETERMINED BY DIRECT SHEAR TESTS AT A CONSTANT RATE OF STRAIN

J. MAC-NEIL TURNBULL, Assistant Research Officer
State Rivers and Water Supply Commission, Victoria, Australia.

- a) Three samples of soil, compacted at or near the optimum density, have been tested in direct shear in accordance with the latest developments of the procedures described elsewhere 1). Tests were made at three rates of strain: namely, 0.22, 0.055, and 0.007 inch per hour
- b) Specimens were tested under normal stresses between 570 and 27,000 lb. per sq. ft., and after the determination of the maximum shearing resistance the cohesion, if any, was obtained by a new method 1).
- c) The results were checked by tests wherein specimens were preconsolidated at a particular normal stress before testing at reduced normal stresses, also for conformity with certain geometrical requirements.
- d) The same soils were also tested by means of the triaxial compression test in the Research Laboratory of the Melbourne and Metropolitan Board of Works in accordance with procedures described by D. F. Glynn 2).

1. The particle size distribution curves of the three soils are shown in Figure 1, and the classifications 3), Atterberg Limits, and compaction data are given in Table I.
2. The results of direct shear tests at various rates of strain are shown in Figures 2, 3, and 4; and the test curves, including the determination of the cohesion, are shown in Figures 5, 6 and 7. Data for tests on sample No. 2 are given in Table II. In order to check the values obtained by this new method for obtaining the cohesion of each specimen

under a particular normal stress, several tests were made after preconsolidation and unloading to a lower normal stress. It will be seen that the results by the two methods of testing are in close agreement in Figures 2(a) and 4(a). In Figure 3(a) agreement is also obtained when the particle displacement correction is made, and indicates one advantage of the additional information obtained by the new method. The curves of Figures 3(a) and 6(a) show the effect of a partial breakdown of the soil particles during

TABLE I

PHYSICAL CHARACTERISTICS OF THE SOILS TESTED

Sample No.	Classification Soil	Sp. Gr.	L.L.	P.I.	S.L.	Optimum Moisture Content ¹ Per Cent.	Optimum Density ¹ lb./cu.ft.	Shear Test Specimens Compacted At	
								Moisture Per Cent.	Density lb./cu.ft.
1	0.184mm./2.781	2.75	44	23	17	18.9	105.1	20.2	104.2
2	0.736mm./3.787	2.71	18	1	13	11.2	123.7	12.0	122.4
3	0.368mm./3.339	2.72	28	6	19	-	-	16.6	107.3
1 Compacted under 40 blows per layer.									

TABLE II

DATA FROM TESTS ON SAMPLE NO. 2 IN FIGURE 6.

Fig. No.	Test No.	Applied Normal Stress		Load and Strain after Figure 9.						Max. Shearing Resistance Corrected lb./sq.ft.	Cohesion lb./sq.ft.
		Consolidation lb./sq.ft.	Test lb./sq.ft.	S _m lb.	l _m in.	a ₁ lb.	l ₁ in.	a ₂ lb.	l ₂ in.		
6(a)	1	1,500	1,500	175	0.106	70	0.349	83	0.814	1,270	0
	2	2,500	2,500	239	0.092	72	0.198	81	0.800	1,950	25
	3	4,500	4,500	376	0.098	83	0.250	99	0.702	3,410	10
	4	10,000	10,000	773	0.157	156	0.308	244	1.110	7,160	215
	5	12,000	12,000	875	0.128	139	0.255	228	1.103	8,520	250
	6	18,500	18,500	1,165	0.236	37	0.315	86	0.891	12,920	290
	7	27,000	27,000	1,754	0.197	116	0.320	-	-	18,840	-
6(b)	8	18,500	3,000	257	0.104	66	0.316	-	-	2,250	-
	9	18,500	4,500	413	0.101	107	0.254	-	-	3,570	-
	10	18,500	6,000	486	0.115	114	0.325	-	-	4,360	-
	11	18,500	9,000	703	0.123	161	0.442	-	-	6,430	-
6(b)	12	4,500	4,500	334	0.134	52	0.327	-	-	3,270	N.D.
	13	18,500	18,500	1,105	0.205	0	-	-	-	12,670	N.D.

Sedimentation Analysis — Sieve Analysis

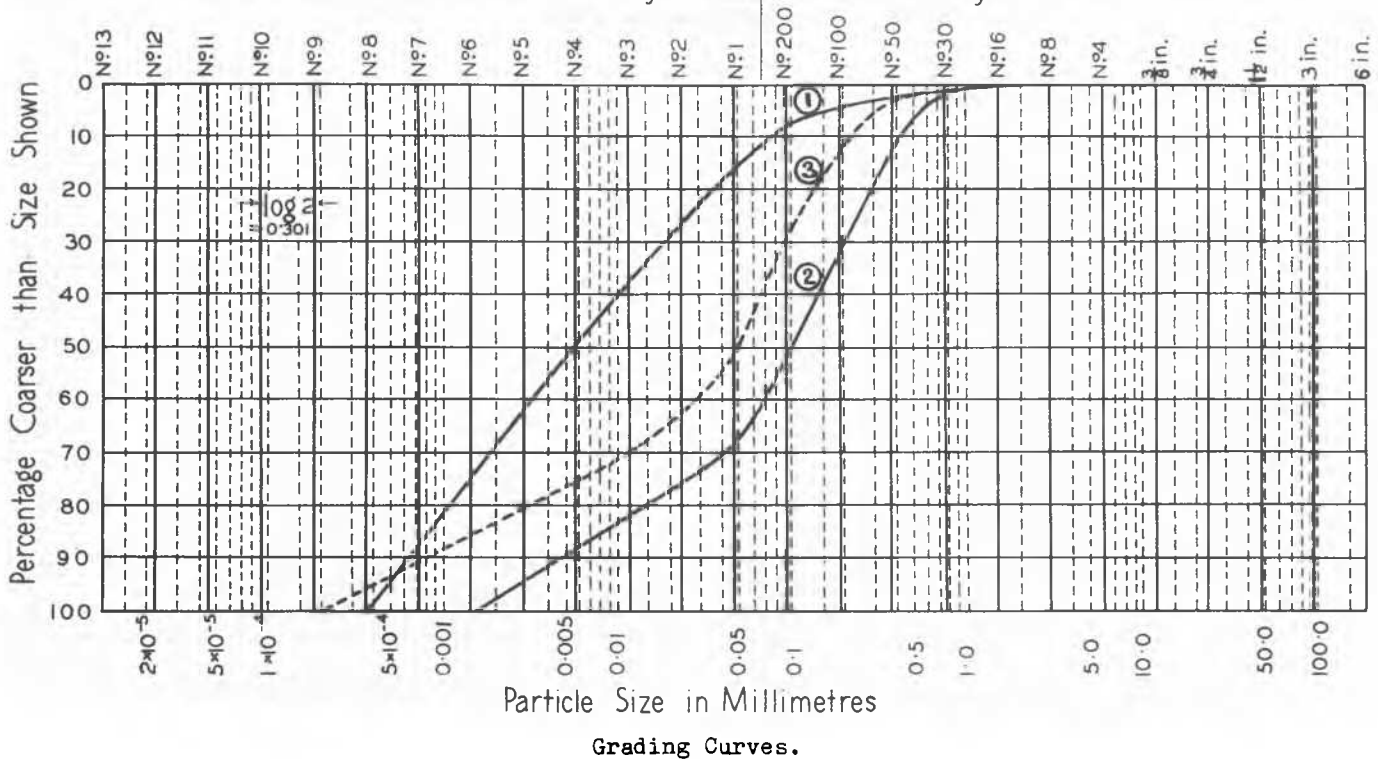


FIG. 1

the test.

3. The values of the cohesion and angle of internal friction for three applied normal stresses are given in Table III.

4. The results of the triaxial compression tests as reported by D.F. Glynn are given in Table IV.

The effects of particle displacement appear to have affected the results for sample No. 2 (see Figure 1 and Table I).

5. The lower limit for the normal stresses on the unloading curve, Figure 8, should be restricted to not less than one-fourth or one-fifth of the maximum normal stress for which the cohesion and angle of internal friction are required. The time interval between unloading and testing should be short, 30 minutes being used in the present tests. For the tests at 0.007 in. per hr., the rate of strain was increased to 0.045 in. per hr. after the first portion of the cohesion curve was obtained. In the slow consolidation tests the period between increments of applied normal stress was 48 hours.

6. Five geometrical requirements which enable a check to be made on the curves are shown in Figure 8 for a soil whose shearing resistance is required under an applied normal stress OS_1 : namely,

a) The maximum shearing resistance, after correction for any particle displacement, as obtained from tests of specimens consolidated and tested at each of several normal stresses at a rate of strain of 0.22 in. per hr. should lie on one straight line $A_1'' A_2'' A_3''$, the extrapolation of which will intersect the horizontal axis at a point O'' .

b) The values of the cohesion for each of the above tests should also lie on a straight line $B_1'' B_2'' B_3''$, the extrapolation of which

should intersect the horizontal axis at the same point O'' .

c) The results for specimens preconsolidated to normal stress OS and tested at reduced normal stresses should fall on a straight line $A_1'' E_1 E_1''$, where cohesion = $O'' E_1'' = S_1 B_1''$. (These tests are only used for confirmation purposes.)

d) For other rates of strain, such as 0.007 in. per hr., other curves such as $B_1' B_2' O'$, $A_1' A_2' O'$, and, if it could be determined,

$A_1' E_1 E_1'$, where cohesion = $O' E_1' = S_1 B_1'$.

e) The point E_1 on the ordinate through the origin of applied normal stress is a fixed point for any one sample at the particular normal stress considered.

For sample No. 1 at $OS_1 = 18,500$ lb. per sq.ft.,
 $OE_1 = 2,800$ lb. per sq.ft.

f) Plotting the capillary stress against the rate of strain, both to arithmetic scales, gives a straight line and provides a means of estimating the effect on the shearing resistance of tests at infinite time. For many cohesive soils the effect of a reduction below 0.007 in. per hr. is very slight. It will be noted that the capillary stress is a real stress, as indicated by $S_3 A_3''$ being greater than $S_3 A_3'$.

7. When the effects of particle displacement are present the cohesion is determined from the second drop in the cohesion curve as shown in Figure 9, and

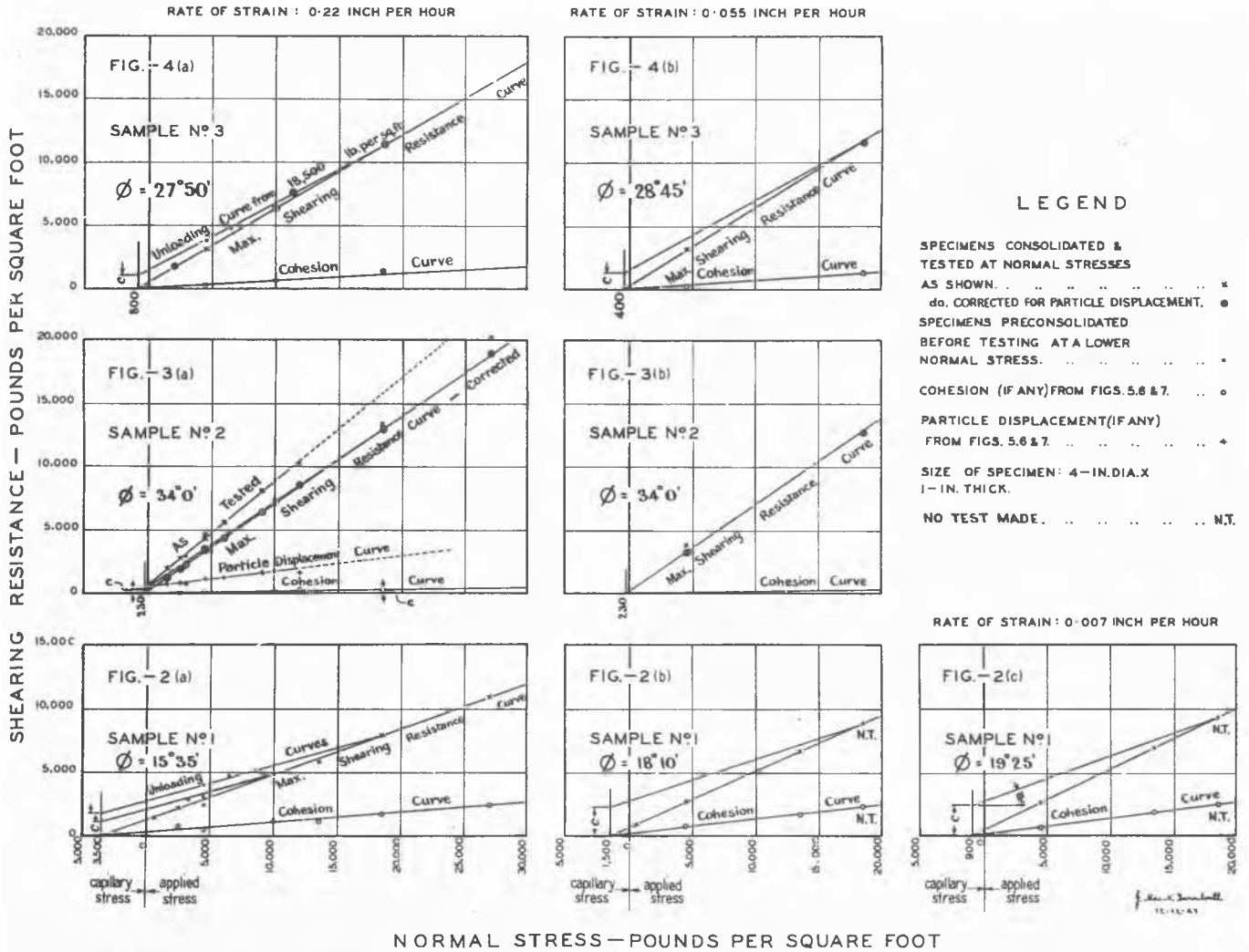
$$\text{Cohesion} = a_2 \times \frac{\text{Area at } |_2}{\text{Area at } |_m} - a_1 \times \frac{\text{Area at } |_1}{\text{Area at } |_m}$$

TABLE III
RESULTS OF DIRECT SHEAR TESTS.

Sample No.	Rate of Strain in. per hr.	Applied Normal Stress lb. per sq. ft.	Capillary Stress lb. per sq. ft.	Total Normal Stress lb. per sq. ft.	Cohesion lb. per sq. ft.	ϕ	$\tan \phi$
1	0.22	18,500	3,500	22,000	1,800	15° 35'	0.279
		10,000	3,500	13,500	1,100	15° 35'	0.279
		4,500	3,500	8,000	650	15° 35'	0.279
1	0.055	18,500	1,500	20,000	2,300	18° 10'	0.329
		10,000	1,500	11,500	1,300	18° 10'	0.329
		4,500	1,500	6,000	700	18° 10'	0.329
1	0.007	18,500	900	19,400	2,500	19° 25'	0.352
		10,000	900	10,900	1,400	19° 25'	0.352
		4,500	900	5,400	700	19° 25'	0.352
2	0.22	18,500	230	18,730	290	34° 00'	0.674
		10,000	230	10,230	160	34° 00'	0.674
		4,500	230	4,730	75	34° 00'	0.674
2	0.055	Same as for 0.22 in. per hr.					
3	0.22	18,500	800	19,300	1,100	27° 50'	0.528
		10,000	800	10,800	620	27° 50'	0.528
		4,500	800	5,300	300	27° 50'	0.528
3	0.055	18,500	400	18,900	1,330	28° 45'	0.549
		10,000	400	10,400	730	28° 45'	0.549
		4,500	400	4,900	340	28° 45'	0.549

TABLE IV
RESULTS OF TRIAXIAL COMPRESSION TESTS.

Sample No.	Cohesion lb. per sq. ft.	ϕ	$\tan \phi$
1	2,700	20° 50'	0.38
2	2,600	39° 20'	0.82
3	1,600	29° 40'	0.57



LEGEND

SPECIMENS CONSOLIDATED & TESTED AT NORMAL STRESSES AS SHOWN. *

do. CORRECTED FOR PARTICLE DISPLACEMENT. ●

SPECIMENS PRECONSOLIDATED BEFORE TESTING AT A LOWER NORMAL STRESS. +

COHESION (IF ANY) FROM FIGS. 5.6 & 7. ○

PARTICLE DISPLACEMENT (IF ANY) FROM FIGS. 5.6 & 7. +

SIZE OF SPECIMEN: 4—IN. DIA. X 1—IN. THICK.

NO TEST MADE. N.T.

Shear Tests.

FIG.2 3 4

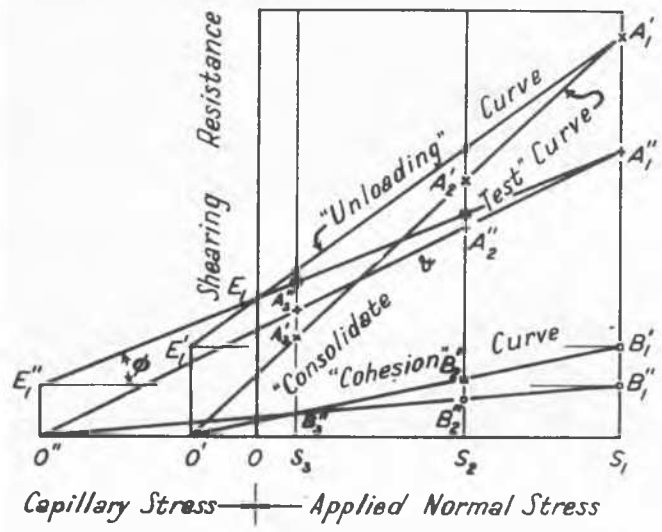


FIG.8

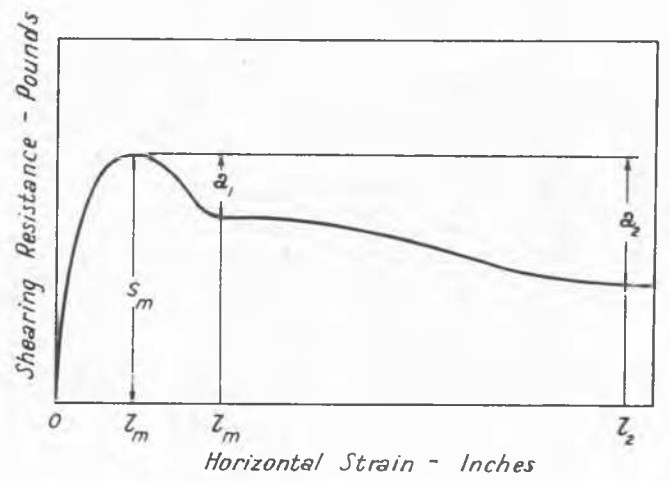
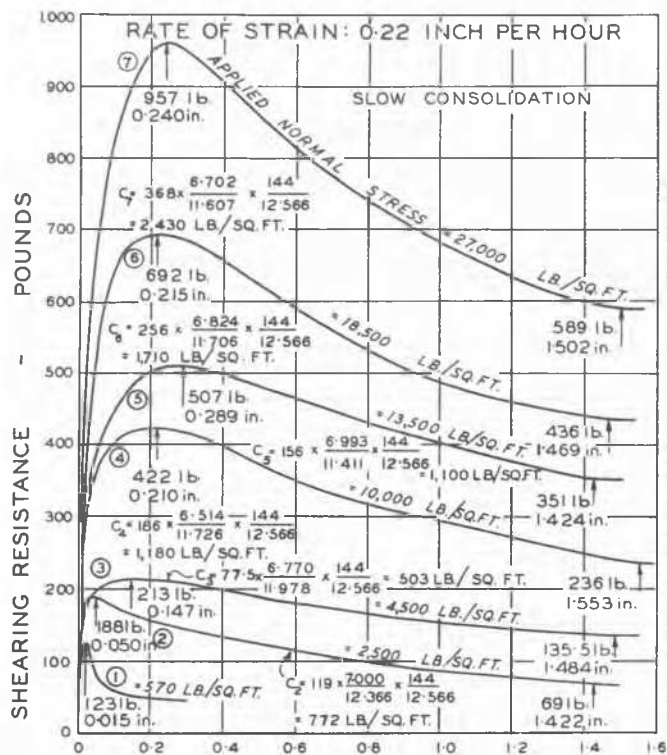
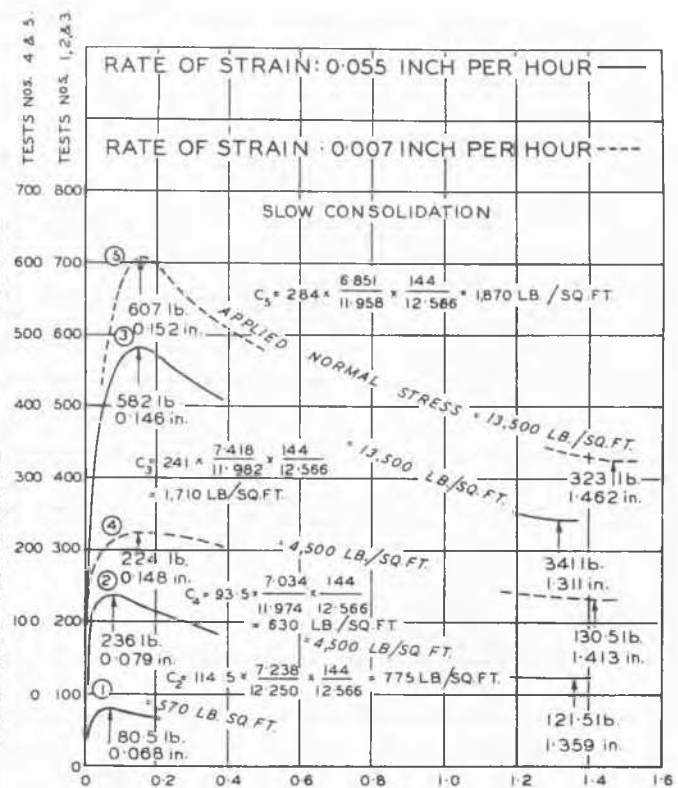


FIG.9



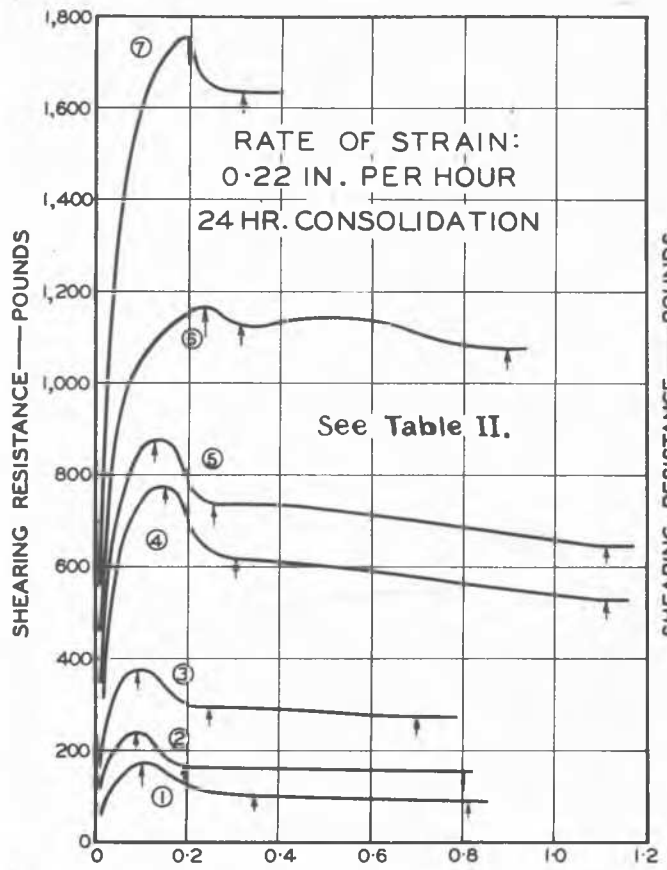
a. Horizontal strain-inches.

Cohesion Tests on Sample no. 1.



b. Horizontal strain-inches.

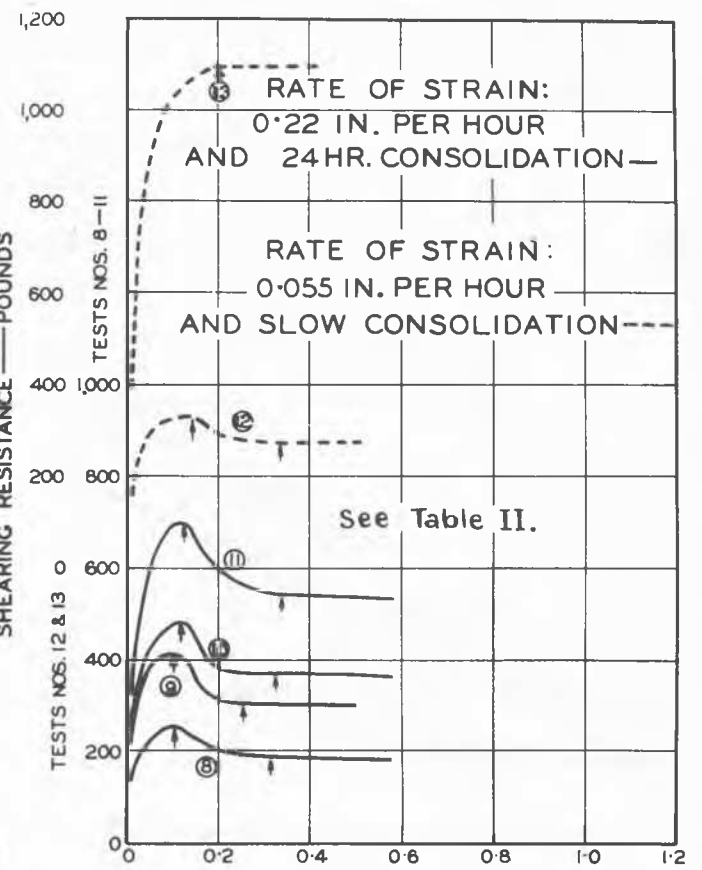
FIG.5



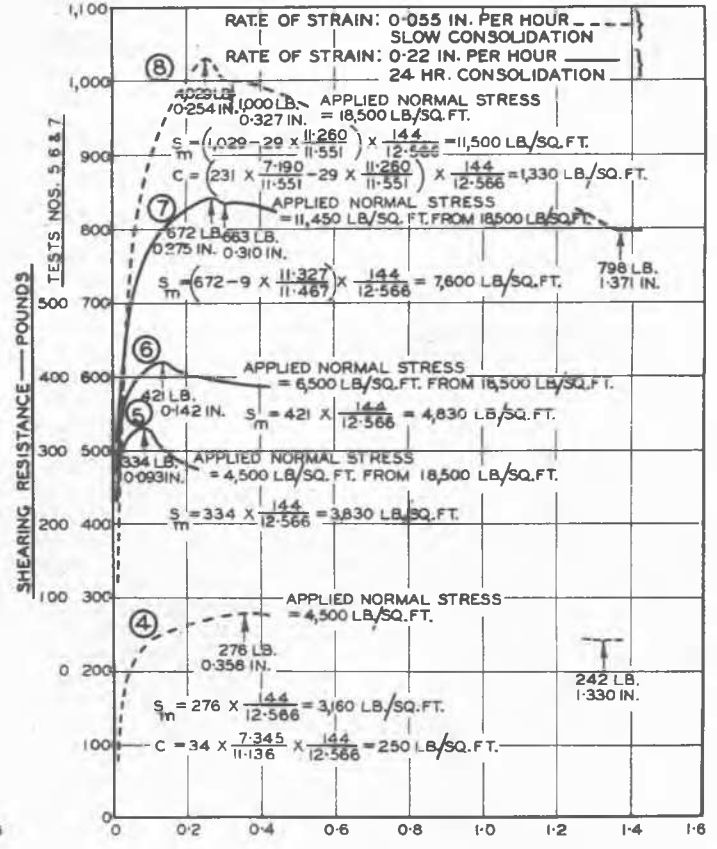
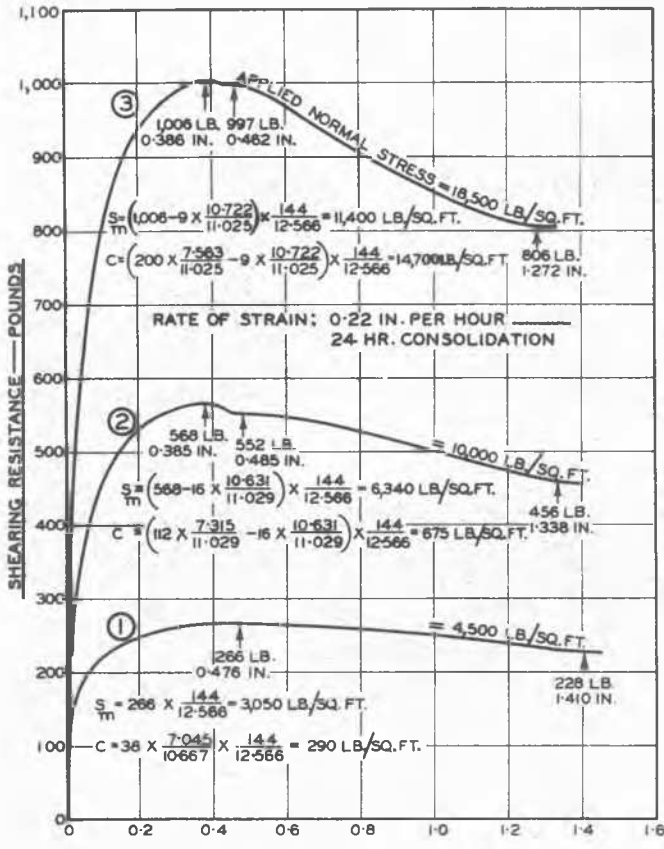
a. Horizontal strain-inches.

Cohesion Tests on Sample no. 2.

FIG.6



b. Horizontal strain-inches.



a. Horizontal strain-inches. Cohesion Tests on Sample no. 3, b. Horizontal strain-inches.

FIG. 7

The maximum shearing resistance is reduced by the particle displacement correction expressed by the last term in the above equation, which vanishes when there is no particle displacement as in the curves of Figure 5. The maximum shearing resistance and the cohesion are reported in lb. per sq. ft. on the assumption that the load is carried by the initial cross-sectional area of the specimen. The applied normal stress remains constant throughout the test.

8. Appreciation is expressed to Lewis R. East, Chairman, State Rivers and Water Supply Commission, for permission to publish this paper; and W.J. Corrigan who made all the tests; also D.F. Glynn for permission to quote the results of his tests.

SUMMARY.

- a. Direct shear tests were made on samples of soil under several normal stresses at various rates of strain.
- b. The cohesion was determined for each particular normal stress by a new method.
- c. The accuracy of these determinations of the cohesion was checked by a series of tests in which specimens were preconsolidated

at the normal stress for which the information was desired, and the maximum shearing resistance was determined at reduced normal stresses.

- d. The results of tests on a homogeneous sample of soil must comply with certain geometrical requirements.
- e. Comparison of these results with those obtained on the same soils by means of triaxial compression tests shows that the latter make no allowance for the effects of particle displacement or capillary stress.

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