LABORATORY VANE TESTS ON DILATANT SOILS

THE INFLUENCE OF PORE-WATER PRESSURES ON LABORATORY VANE TESTS ON DILATANT SOILS

by

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SCOPE AND CONTENTS:

This thesis describes a study of the influence of pore-water pressures on the measurement of shear strength of remolded silt by the laboratory vane apparatus. A device to prepare saturated soil samples and an experimental technique to measure pore-water pressures on the failure surface produced by rotation of the vane are discussed. Experimental results of vane tests conducted in conjunction with pore-water pressure measurements on remolded silt samples and laminated silt samples are presented and discussed.

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CHAPTER I

INTRODUCTION AND RÉSUMÉ

The field vane shear apparatus (Cadling and Odenstad, 1950) was developed to measure "in situ" the shear strength of sensitive clays. Samples of sensitive clays are difficult to obtain for triaxial or direct shear tests, as these clays undergo a considerable loss in shear strength when disturbed.

A simple field vane apparatus is shown in Figure 1. The vane, consisting of four thin rectangular blades welded to a shaft, is attached to the lower end of the apparatus (vane dimensions: 3 inches high x 2 inches diameter). To perform a vane shear test, the field vane apparatus is inserted into the ground to the required depth and torque is applied to the turning handle manually. The torque is transmitted by two spring balances to a lever and from the lever to the shaft of the vane by an extension rod, protected by a casing pipe. The torque is applied until the soil is ruptured on a cylindrical failure surface conforming to the dimensions of the vane. The maximum torque indicated by the spring balances during a vane shear test is related to the shear strength of the soil. The angle of rotation of the vane can be measured on a protractor by a pointer attached to the lever.

A research program was initiated in January 1960 to investigate the apparently high values for shear strength indicated by the field vane apparatus when laminated (varved) clays with layers of fine sand

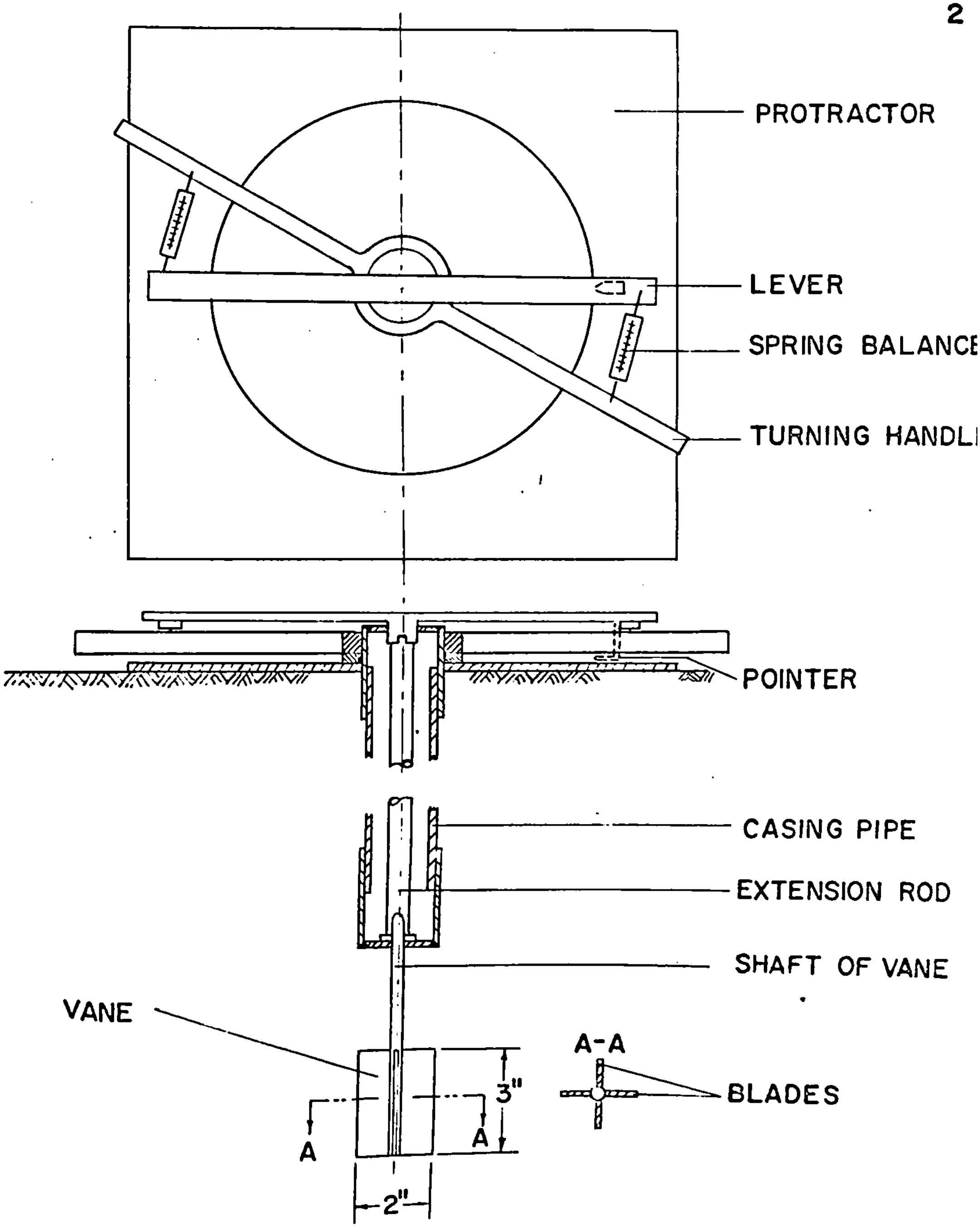


FIG. 1

SIMPLE FIELD VANE APPARATUS.

and silt were tested (Golder, 1960). A theory was proposed by Dr. H. Q. Golder (1960, 1961) that the apparent over-estimation of the shear strength in varved clays may be caused by negative excess pore-water pressures, which are generated during shear deformations in the sand and silt layers. Shear deformations cause expansion (called dilatation) of dense sand and silt and generate negative pressures in the pore-water of the soil. These negative pore-water pressures increase the grain to grain stress and cause an apparent increase in shear strength.

As part of the research program initiated in January 1960, laboratory vane tests (vane dimensions: 1 inch high x 3/4 inch diameter) in remolded clay were carried out by Professor N. E. Wilson to ascertain the applicability of the laboratory vane to clays (Wilson 1961). A laboratory vane apparatus (Figure 14) is normally used to measure shear strength of natural soil samples extracted from the ground and remolded soil samples (which are thoroughly kneaded, or consolidated from a slurry). In addition, laboratory vane tests in dense sand and in sand layers of laminated samples consisting of layers of remolded clay and dense sand were conducted by the present author prior to the testing program herein reported (Wilson, 1961). All laboratory vane tests in sand were conducted in conjunction with measurements of pore-water pressure.

This thesis represents the research work carried out in an attempt to substantiate Dr. Golder's theory for laminated clays with layers of silt. Laboratory vane tests were conducted in remolded silt samples and silt layers of laminated samples consisting of layers of remolded clay and remolded silt. These tests were conducted in conjunction with pore-water pressure measurements to investigate the possible generation of negative pore-water pressure and its effect on torque values. The vane tests were performed in the laboratory on remolded samples to ensure the consistency of the properties of the silt and to obtain reproducible results. The work in the laboratory was also necessary to develop a technique to measure pore-water pressure on the failure surface generated by the vane. Information and techniques gathered from laboratory vane tests on remolded soil samples can be applied to the determination of shear strength values obtained by the use of the laboratory vane on undisturbed soil samples and by the field vane in undisturbed soils.

The preparation of saturated homogeneous and laminated soil samples (5 inches high x 6 inches diameter) is described in Chapter III. Replicate silt samples were prepared for each test series by deaerating and consolidating a slurry of water and silt. The properties of the silt samples indicated a small variation in water content, void ratio, degree of saturation and grain size distribution between the replicate samples; there were also some variations with depth within each sample.

The major portion of the thesis (Chapters IV to VI) describes the technique and experimental results of laboratory vane tests carried out in conjunction with pore-water pressure measurements on remolded silt. Pore-water pressures, generated by the vane during a shear test, were recorded at the instant of maximum torque, which was considered to represent the torque necessary to cause shear failure. The effect of the depth of insertion of the vane and the rate of testing on shear strength measured by the vane was investigated. To confirm the dependence of maximum torque on negative pore-water pressures laboratory vane tests were carried out with negative pore-water pressures applied to the sample; the magnitudes of the applied negative pore-water pressures were considerably greater than those generated by the laboratory vane. The variations of pore-water pressure during laboratory vane tests in remolded silt were investigated by rotating the vane by hand in stages and recording pore-water pressures at every stage. To compare the shear strength measured by the vane with strength values from standardized shear tests, triaxial tests were carried out on remolded silt samples and pore-water pressure measurements were taken.

The first test series, conducted on remolded Fort Dover silt, indicated that the maximum torque value showed a significant dependence on pore-water pressure. No significant dependency of maximum torque on water content of the sample, depth of insertion of the vane or angular speed of the torque dial was observed over the range tested. The average shear strength, measured by the laboratory vane at zero excess pore-water pressure, was significantly higher than the average shear strength approximated from triaxial test results at small negative pore-water pressures. The insertion of the laboratory vane into the silt usually caused positive excess pore-water pressures. In general, soil expansion (dilatation) due to shear deformations generated by the vane, did not decrease positive pore-water pressures below hydrostatic pressure, when the torque was applied immediately after insertion of the vane.

Laboratory vane tests were carried out on silt layers of laminated remolded samples consisting of clay and silt, which had properties similar to the silt samples used in the first test series. No significant differences between results from laboratory vane tests in the silt layers of laminated samples and results from homogeneous silt samples were observed.

Results from laboratory vane tests on silt with different properties from those observed in the first test series indicated an increase in maximum torque with increasing grain size accompanied by decreasing uniformity coefficient.

CHAPTER II

REVIEW OF THE LITERATURE

Numerous articles have appeared concerning the theory of shear strength of soils. In 1773, Coulomb introduced in his essay concerning the earth pressure theory, an equation to determine the resistance of soil to shear (see Golder, 1948). His equation

$$S = c + \sigma_n \tan \phi \qquad (1)$$

stated that the shear resistance (S) may be considered as the sum of cohesion (c) and frictional resistance. The frictional resistance is dependent on the normal pressure (σ_n) acting on a considered plane and on the angle of internal friction of the soil (ϕ).

Hvorslev (1937) discovered that cohesion is a function of the water content. He was able to prove that soils had a characteristic angle of internal friction.

The determination of the shear strength coefficients of soil was complicated by the pore pressure, generated during stress changes, in the fluid occupying the pore space. Terzhagi (1923 and 1938), clarified the pore pressure concept by introducing the principle of effective stress. This principle stated that the effective normal stress is equal to the difference between the applied normal stress and the pressure in the fluid occupying the pore space. This was expressed as

$$\bar{\sigma}_n = \sigma_n - u \qquad (2)$$

where $\bar{\sigma}_n$ denotes the effective normal stress on a considered plane

σ_n denotes the total normal stress, and

u denotes the pore pressure of the fluid occupying the pore space.

The principle of effective stress was related to shear strength by Terzaghi (1938). He subsequently stated (Terzaghi, 1943) that the shear strength depends on the effective (grain to grain) normal stress and not on the total normal stress on a considered plane. Coulomb's equation was, therefore, revised in the following way:

$$S = \bar{c} + (\sigma_n - u) \tan \bar{\phi}$$

or

$$S = \bar{c} + \bar{\sigma}_n \tan \bar{\phi} \qquad (3)$$

where: S denotes the shear strength

$\bar{c}$ denotes the "true cohesion" (which Hvorslev found to depend on water content),

$\bar{\sigma}_n$ denotes the effective normal stress on the failure plane, and

$\bar{\phi}$ is the "true angle of internal friction" determined by effective normal stresses on the failure plane.

Experimental verification of the principle of effective stress required the development of an apparatus to measure pore pressures. Since the pioneer work carried out by Rendulic (1937), the technique of measurement of pore-water pressures in saturated samples has been further developed by Taylor (1944 and 1948), Bishop and Eldin (1950), Hilf (1956), Anderson, Bjerrum, DiBiagio and Kjaernsli (1957). Bishop (1960 b) pointed out that in most modern equipment the operation of the pore pressure measuring systems involved "no-flow" of fluid to or from the pore space. He stated that flow of water caused a considerable time lag in the response of the apparatus for soil samples of low permeability and seriously modified the actual pore pressure in samples with small volumes.

The principle of effective stress has been verified for saturated soils by the experimental work of Rendulic (1937), Taylor (1944 and 1948), Bishop (1950, 1960 a and b) and many others.

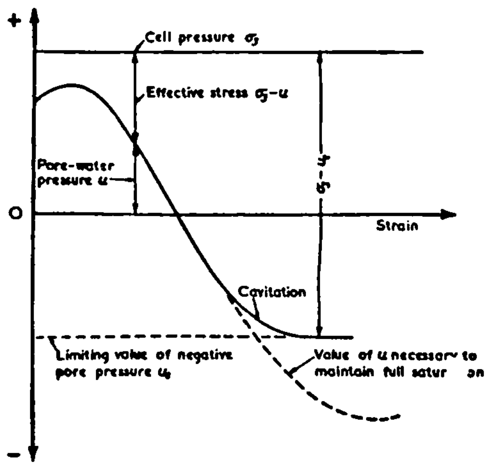
A graphical method for representing effective stresses in Mohr's diagram was devised by Casagrande and Wilson (1953). In the analysis of test data, curves were plotted to represent the locus of points whose coordinates were the shear stress and effective normal stress on the failure plane during a triaxial test. Such curves were considered to be traced by a vector representing the resultant effective stress on the ultimate failure plane and were referred to as "vector curves" (Casagrande and Hirschfeld, 1960).

Bishop and Bjerrum (1960) found that the differences in shear characteristics between sand and clay were caused by the wide differences in permeability rather than by the differences between their frictional properties. They stated that applied stresses would not produce any change in the frictional component of strength until a sufficient time had elapsed for water to leave or enter the system and to produce a change in pore-water pressure.

Golder and Skempton (1948) have indicated that the shear strength values for clay obtained from undrained triaxial tests usually did not increase with increasing confining pressure. (An undrained triaxial test is a shear test conducted at constant water content; no drainage and hence no dissipation of pore pressure is permitted (Bishop and Bjerrum, 1960)). The angle of shearing resistance appeared to be zero when the results of the undrained triaxial tests on clay (Golder and Skempton, 1948) were plotted in terms of total stress in Mohr's diagram. Skempton (1948 b) suggested, therefore, that the shear strength of cohesive soil may be expressed as an "apparent cohesion" (to differentiate it from true cohesion $\bar{c}$) which can be approximated by one-half of the compressive strength obtained from undrained triaxial tests. Golder and Skempton (1948) have shown that in the case of undrained triaxial tests on silt the angle of shearing resistance appeared to be significantly greater than zero; the tentative conclusion of the authors was that this was connected with dilatancy. (Volume increase caused by shear is called dilatancy, (Terzaghi and Peck, 1948)).

Results from shear tests in dense sand indicated an increase in volume of the sand sample during shear (Taylor, 1948; Bishop, 1959). During undrained triaxial tests in dilatant saturated sand, Bishop and Eldin (1950) measured slight increases in pore-water pressure at small strains and a drop in pore-water pressure to great negative values at larger strains (Figure 2). This drop in pore-water pressure was caused by dilatancy.

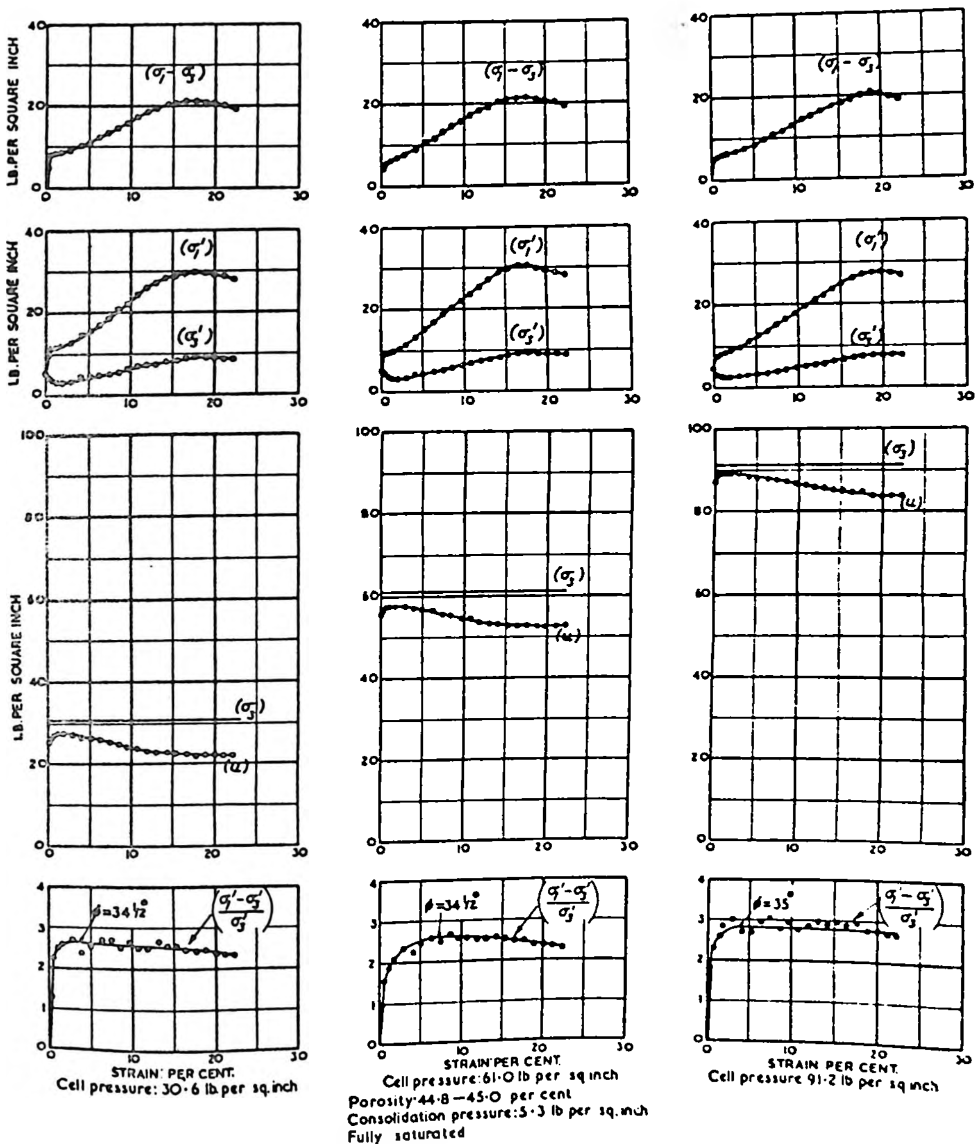
Additional results from undrained triaxial tests (Figure 3) conducted by Bishop and Eldin (1950) on dilatant saturated sand samples indicated that the maximum deviator stress $(\sigma_1-\sigma_3)_{max.}$ and thus the shear strength of the sand, was independent of confining (cell) pressure. The test results (Figure 3) show that an increase in confining pressure (at zero per cent strain) resulted in an increase in pore-water pressure of



Pore-pressure changes in a dilating sample

Figure 2

(After Bishop and Eldin, 1950)



Stress changes during shear

Figure 7

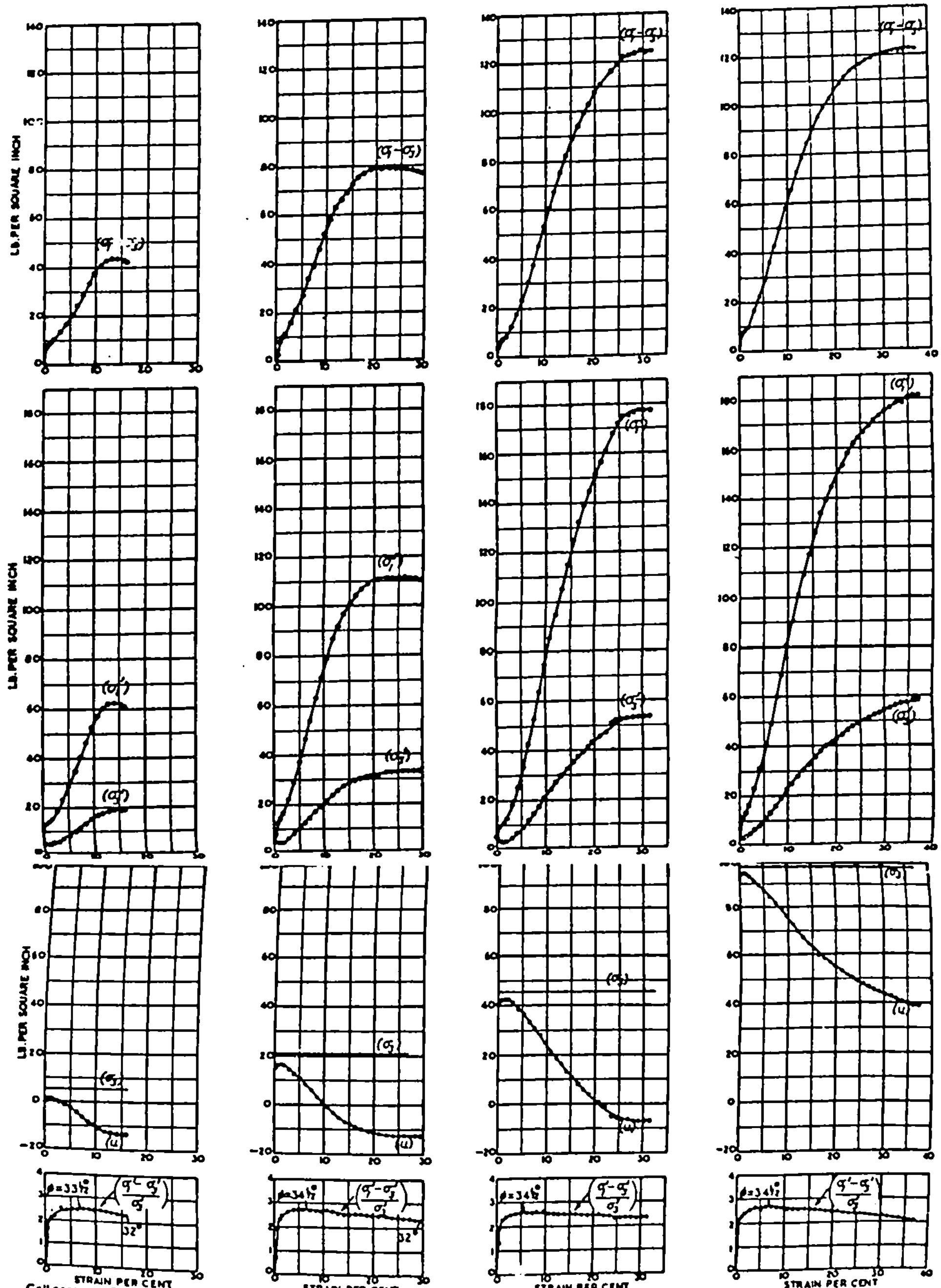
(After Bishop and Eldin, 1950)

about equal magnitude. As the pore-water pressure and confining pressure were approximately of equal magnitude for each test at zero per cent strain, the effective minor principle stress ($\bar{\sigma}_3 = \sigma_3 - u$) decreased to a small value for each triaxial test specimen before a deviator stress was applied. Therefore, an increase in confining pressure had no effect on the shear strength of the sand samples. Figure 3 also indicates that the pore-water pressure decreased due to shear with increasing deviator stress ($\sigma_1 - \sigma_3$), but that the reduction in pore-water pressure was similar for each test. Figure 4 (Bishop and Eldin, 1950) shows that the maximum deviator stress and thus the shear strength of saturated sand was increased by increasing the confining pressure, when an increase in confining pressure resulted in an increased reduction in pore-water pressure due to shear during an undrained triaxial test. The reduction in pore pressure due to shear in Figure 4 was greater than in Figure 3, as a sand with a smaller porosity was used. The reduction in pore-water pressure due to shear increased with confining pressure as a limiting negative pore pressure existed.

The equation for effective stress $\bar{\sigma} = \sigma - u$ indicates an increase in effective stress when the pore pressure decreases and the total stress remains constant. Consequently, the shear resistance of the dilatant sand increased with increasing pore-water pressure as indicated by the revised Coulomb equation

$$s = \bar{c} + \bar{\sigma}_n \tan \bar{\phi} \qquad (3)$$

Bjerrum (1954) indicated that during the last part of an undrained triaxial test on dilatant soils (the part governed by a decrease in pore-water pressure) the effective stress corresponded to stress in a



Cell pressure 5·3 lb per sq inch

Cell pressure 20·5 lb per sq inch

Cell pressure 45·7 lb per sq inch

Cell pressure 98·3 lb per sq inch (Note - This result is obtained by interpolation from two tests at slightly different porosities)

Porosity: 43·3 per cent
Consolidation pressure: 53 lb per sq inch
Initially fully saturated

Stress changes during shear, showing the effect of the limiting negative pore pressure

drained triaxial test. (A drained triaxial test is conducted with open drainage slow enough to prevent any change in pore-water pressure throughout the test).

Bishop (1960 b) stated that in partly saturated soils air and water occupy the pore space and may be in equilibrium at pressures which differ considerably due to surface tension. He pointed out that at low degrees of saturation, the soil particles may not be surrounded completely by the liquid and the pressure of the liquid phase will act over a reduced area.

For partly saturated soils, the following expression has been suggested for the effective stress (Bishop, 1960 b),

$$\bar{\sigma} = \sigma - u_1 + X\,(u_1 - u_2) \qquad (4)$$

where u_1 denotes pore air pressure,

u_2 denotes pore-water pressure and

X denotes a parameter closely related to the degree of saturation which equals unity for saturated soils and decreases with decreasing degrees of saturation.

Bishop indicated that the value of the parameter X was close to unity for degrees of saturation between 80 and 100 per cent; the above equation was then approximated by $\bar{\sigma} = \sigma - u_2$. Hilf (1956) and Bishop (1960 b) found that the technique of measurement of pore-water pressures in partly saturated soils was difficult. For pore-water pressure measurements a probe in the form of a porous element with a very high air entry value was provided and tests were carried out slowly. Air bubbles, entering the pore pressure probe gave erroneous values of pore-water pressure because the volume of the air changed with pressure.

Bishop and Eldin (1950) stated that an increase in pore pressure reduces the volume of the free air in partly saturated samples by direct compression according to Boyles Law and dissolves the free air into the pore-water according to Henry's Law. Bjerrum (1954), therefore, increased the initial pore pressure of partly saturated triaxial soil samples by increasing the confining pressure. This increased the degree of saturation of the pore-water and facilitated pore-water pressure determinations in partly saturated soil samples during undrained triaxial tests. Bishop and Eldin (1950) found, however, that a small volume of air present in a triaxial sample increased radically the compressibility of the fluid filling the voids and caused deformation of the soil skeleton, which was compressed to compensate for the space formerly occupied by air.

Casagrande and Wilson (1953) increased the confining pressure during undrained triaxial tests and applied an initial pore pressure, called back pressure, to the partly saturated samples with the pore pressure apparatus. Lowe and Johnson (1960) have shown, however, that back pressures of the order of 100 psi. to 200 psi. were necessary to achieve 100 per cent saturation; they have not been able to demonstrate conclusively that significant volume changes of the samples were avoided, even by increasing the confining pressure and back pressure simultaneously in small steps.

The field vane shear apparatus was developed so that relatively undisturbed testing in clay was possible even at depth (Carlson, 1948; Skempton and Bishop, 1950; Evans, Sherrat and Calderwood, 1948).

Cadling and Odenstad (1950) showed a cylindrical rupture surface conforming to a cylinder having the dimensions of the vane. This rupture surface was accomplished by turning the vane in a barrel filled with sand or clay and sealed with a rigid cover. The authors introduced the following assumptions for the calculation of the shear strength:

1) the surface of failure is a circular cylinder with diameter and height equal to that of the vane and
2) the stress distribution at maximum torque is uniform across the failure surface, as the vane is replaced by a rigid cylinder to which the soil adheres.

In accordance with these assumptions, the torque required to mobilize the shear strength is

$$T = S(\pi h \frac{d^2}{2} + \pi \frac{d^3}{6}) \qquad (5)$$

where T denotes maximum torque,

S denotes shear strength, (ultimate shear stress),

h denotes height of vane blades, and

d denotes diameter of vane.

Cadling and Odenstad gave no explanation for the large angular deformation (up to 15 degrees) before shear failure. The authors found that the shear strength determined by the vane depended on the speed at which the vane was rotated. They also investigated the effect of the height: diameter ratio of the vane and showed that a ratio varying from 1.25:1 to 3.75:1 had insignificant effects on the shear strength obtained from vane tests on remolded soils.

Skempton (1948 a) conducted a large number of field vane tests and unconfined compression tests with test durations ranging from 1 to 50 minutes. He found that shear strengths measured by the field vane and by the unconfined compression test had the same relationship with time.

Carlson (1948) and Skempton and Bishop (1950) considered field vane tests as undrained tests in clay. Bjerrum (1954) considered shear strength values, as measured by the field vane, to be independent of total stress and compared the shear strength, as measured by the vane, with shear strength obtained from undrained triaxial tests. As the shear strength obtained from undrained triaxial tests was usually in agreement with the shear strength approximated by one half the unconfined compressive strength (Bennett and Mecham, 1953; Golder and Palmer, 1955; Skempton and Bishop, 1950) field vane tests results were generally compared with unconfined compression test results. The results of field vane tests and the results of unconfined compression tests agreed for samples at shallow depth ($<$ 40 feet), but the former exceeded the latter at greater depth (Carlson, 1948; Skempton, 1948 b; Eide and Bjerrum, 1955; Eden and Hamilton, 1957; Eden, 1961; Peaker, 1961). Cadling and Odenstad (1950) showed that shear strength values, as measured by the vane, were in agreement with the shear strength values calculated from eleven slides, whereas unconfined compressive strength values were too small. This has generally been attributed to reduction in shear strength due to increasing sample disturbance with depth for unconfined compression tests (Osterberg, 1956). There exists some experience where the vane gave values for the shear strength which appeared to be too high when checked against actual failures (Golder and Palmer, 1955; Bazett, 1961).

The field vane apparatus, a tool designed to determine the shear strength of clay, has been used frequently for shear strength determination in silty clay, silt and varved clays with layers of silt or sand. Skempton (1948 a) and Anderson and Bjerrum (1950) used the vane in silty clay and found agreement between vane tests and unconfined compression tests up to a depth of about 40 feet. Hill (1956) showed a table (Figure 5) where results from field vane tests and direct shear tests were compared. The table indicates, in general, a higher shear strength measured by the vane than by the direct shear apparatus for silty clay and silt. Metcalf and Townsend (1961) investigated shear strength values recorded for varved clay deposits in Ontario and reported 3 cases of good correlation between results from vane tests and unconfined compression tests and only one case of poor correlation.

Golder (1961) stated that results from field vane tests apparently over-estimated the shear strength in the case of varved clays in which some of the layers were silt or fine sand.

Results from laboratory vane tests (Evans, Sherratt, Calderwood, 1948) indicated a linear relationship between unconfined compressive strength and maximum torque for a remolded clay at various water contents. The laboratory vane apparatus was a hand operated, "stress controlled" model. The authors assumed that failure took place over a cylindrical surface conforming to the dimensions of the vane and calculated the torque to produce failure as

$$T = S\left(\pi h \frac{d^2}{2} + \pi \frac{d^3}{6}\right)$$

This equation was similar to the one later developed by Cadling and Odenstad. Evans, Sherratt and Calderwood were not able to show a cylindrical failure surface conforming to the dimensions of the vane, when a

TABLE I.—COMPARATIVE SOIL SHEAR VALUES; VANE SHEAR *versus* DIRECT SHEAR.

Soil				Laboratory Direct Shear Test			Soil Shear Strength, psi	
Location	Depth, ft.	Type	Moisture, per cent	Unit Cohesion, C, psi	Angle of Friction, ϕ, deg	Weight of Soil Above, N, psi	Calculated	Field Vane
Banfield Expressway Sta. 0 + 18	19.5	Org. clay	242	2.0	5	8.94	2.8	9.4
	27.5	Org. silt loam	268	1.5	34	9.55	6.9	7.9
	32.5		115	1.5	37	10.31	9.2	8.7 and 9.6
Sta. 23 + 85	27.5	Clayey peat	278	1.5	34.5	4.55	4.6	8.3
	39.5	Peaty clay loam	154	0	36.5	6.04	4.5	4.6
Oregon Coast Astoria—W. Lake Sta. 126 + 50	4.5	Peaty silty clay	92	2.5	24	3.28	4.0	3.6 3.2
	11.5		96	0.5	35	6.36	5.0	3.4
	13.5		96	0.5	35	6.68	5.2	3.6
	15.5		96	0.5	35	7.14	5.5	4.9
	16.5		96	0.5	35	7.34	5.6	3.3
Sta. 127 + 50	4.5	Silty clay loam	80	2.0	13.5	2.13	2.5	3.4
	8.5		109	1.0	32	2.80	2.8	3.7
	17.5		60	2.0	30	4.55	4.6	5.5
Sta. 145 + 00	4.5	Silt loam	128	0.5	25	1.71	1.3	2.2
	15.5	Org. clay	54	0	37	4.99	3.9	3.1
Sta. 165 + 00	11.5	Silty clay	99	2.0	4.5	3.60	2.3	2.6
	15.5	Peaty silt	57	1.0	31	5.24	4.1	2.2
	20.5	Org. clay loam	61	1.0	33	5.94	4.9	5.0
Sta. 270 +	6.5	Silty clay loam	91	2.0	7	2.58	2.3	3.0
	7.5	Org. clay loam	71	2.0	30	2.81	3.6	7.4
Sta. 297 + 00	9.5	Silty clay	158	1.5	22	1.28	6.7	3.8
Sta. 310 + 00	9.5	Silty clay loam	87	0	33	1.95	1.3	3.0
Beaver Slough—Bandon County Bridge	8.5	Org. silt	70	1.5	11	3.43	2.2	2.8
Siletz River—Depoe Bay Sta. 72 + 30	16.5	Silty clay loam	81	0.5	27.5	3.50	2.0	3.0
Coos-Bay—Roseburg Coquille—Myrtle Pt Sta. 29 + 00	15.5	Clay	34	7.5	33	11.70	15.0	14.7
Corvallis—Newport Toledo By-Pass Sta. 296 ±	6.0	Silty clay	85	2.0	24	1.04	2.5	2.6
	7.0		85	2.0	24	1.22	2.5	2.8
The Dalles—California Klamth Falls By-Pass Sta. 159	9.5	Peat (Ap '54)	588	6.5	31.5	1.59	7.5	5.3
		(Jl '54)	392	1.5	30	1.71	2.5	2.2

vane, made up of two blades welded at right angles, was rotated in an open container filled with clay. They found, however, that the shear strength calculated from vane tests, using the above equation, was not significantly higher than the shear strength obtained from unconfined compression tests, when the soils tested had an angle of internal friction not greater than 10 degrees. The authors considered the increase of maximum torque with rate of torque application to be caused by viscous effects.

For vane tests conducted in the laboratory at Imperial College the soil sample was covered by a rigid lid with a circular hole in the centre to allow insertion of the vane (Skempton and Bishop, 1950). The laboratory vane test was considered as an undrained test on soft undisturbed or remolded clays for which a compression specimen could not readily be prepared. Shear strength values, calculated from laboratory vane tests using Cadlings equation (5) indicated good correlation with one half of the unconfined compressive strength.

Aldrich (1953) summarized the results of laboratory vane tests conducted at Harvard University (Figure 6).

Shear strength values for remolded clay, calculated from laboratory vane tests using Cadlings equation (5), were in good agreement with one half of the unconfined compressive strength (Wilson, 1961). This agreement existed at various water contents of the clay.

Wilson (1961) stated that, during laboratory vane tests in dense saturated sand, negative pore-water pressures were generated during insertion and rotation of the vane. Results from these tests indicated a dependence of maximum torque on negative pore-water pressures; this dependence varied with container size and depth of insertion. These

TABLE A

Case	Description of Soil	Natural water Content	Atterberg Limits		Shearing Strength Comments
			L_w	P_w	
		%	%	%	
(a)	Very soft uniform dark gray clay	70	57	25	Vane tests gave a strength of about 260 psf. while the shearing strength from unconfined tests averaged 100 psf.
(b)	Soft blue-gray varved clay and silt	40	40	17	Vane tests and unconfined tests agreed closely and averaged about 700 psf.
	Blue-gray silty clay	38	30	21	Vane tests averaged 1,300 while unconfined tests gave about 600 psf. for shearing strength.
(c)	Soft blue-gray silty clay	32	40	22	Vane tests averaged about 950 and unconfined tests 700 psf. Data from a few tests indicated that the vane results were about midway between those from triaxial Q and Q_c tests.

Figure 6

(After Aldrich, 1957)

results were confirmed by further laboratory vane tests in dense sand.

The laboratory vane tests in clay and dense sand (Wilson, 1961) were conducted as part of the research work carried out at McMaster University to investigate the apparently high values for the shear strength indicated by the field vane when laminated clays with layers of fine sand were tested. The laboratory vane tests in sand were carried out by the present author.

CHAPTER III

PREPARATION OF SOIL SAMPLES

1) Description of Material

The material used for the major part of this research program, the "D-test series", was a dark gray silt obtained from Port Dover beach, Ontario. The liquid limit of this silt was 26, the plastic limit 17. These results placed the silt on the plasticity chart just above the A-line. The grain size distribution curves for Port Dover silt obtained from hydrometer analyses (Figure 7) showed a coarse to fine, well graded silt with a uniformity coefficient (D_{60}/D_{10}) of about 6, with a clay fraction of about 10 per cent and with a sand fraction of about 10 per cent. The sand grains appeared to be subangular quartz particles when magnified under the microscope (Figure 8). The range of grain size obtained from the hydrometer analyses was substantiated by measurements of the average diameter of the large and small silt grains magnified 890 diameters (Figure 8). The specific gravity of the silt was determined as 2.72.

For the "A-test series" laminated samples were prepared which consisted of one layer of Port Dover silt and one top layer of clay. The clay, obtained from South Hamilton, Ontario, had a dark brown color, a liquid limit of 35 and a plastic limit of 19.

The grain size distribution curve for Port Dover silt with the fine grains removed (Figure 9) indicated a coarse to medium silt with a uniformity coefficient (D_{60}/D_{10}) of 1.7 and with a sand fraction of about

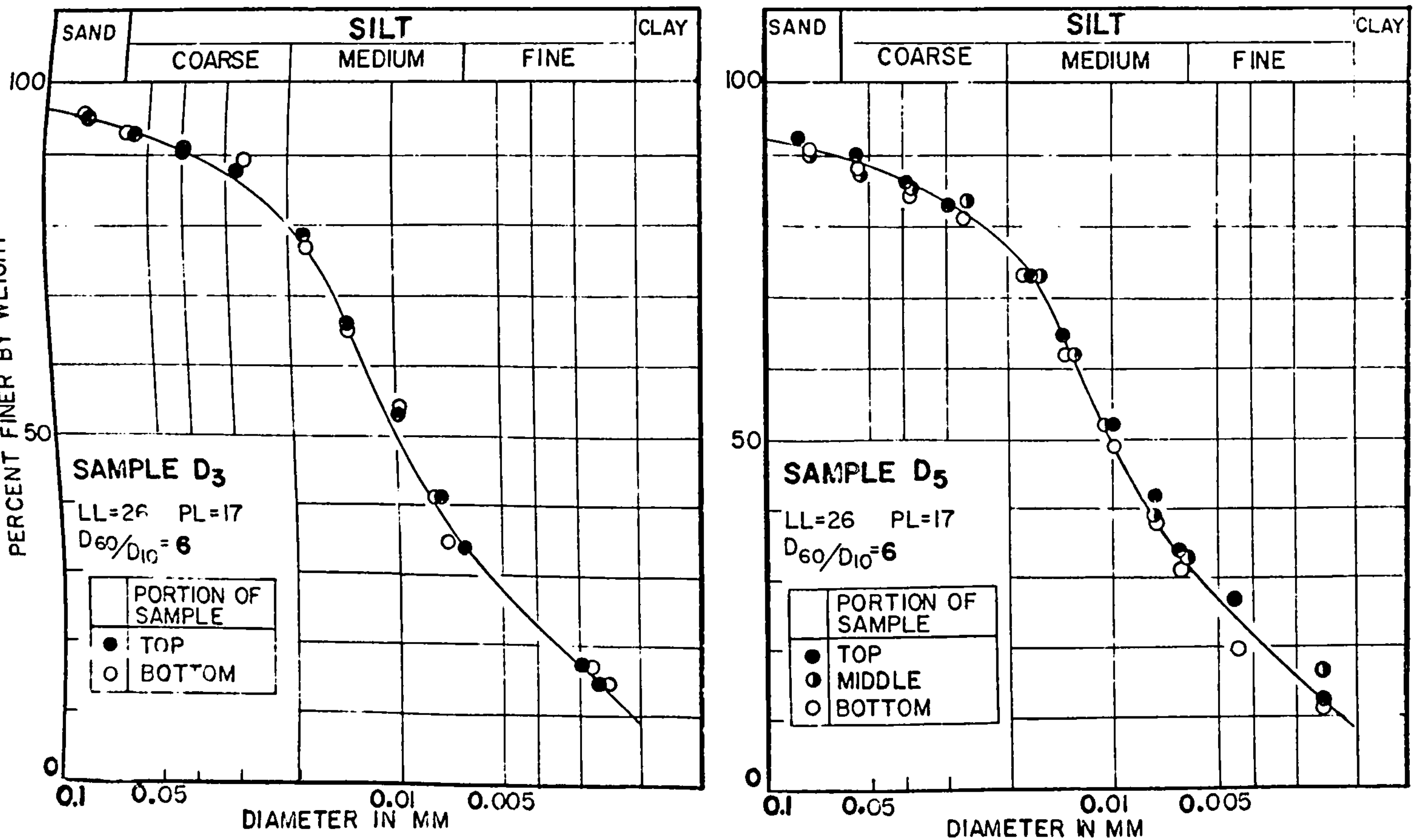
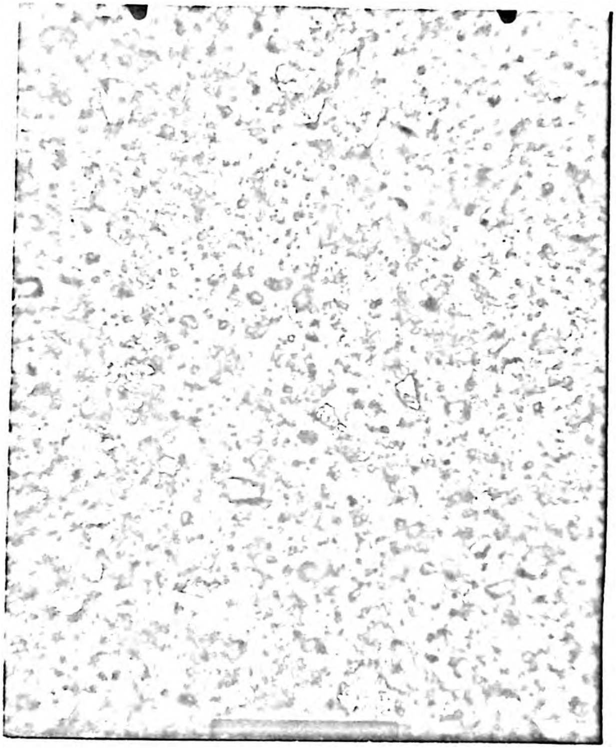


FIG.7

GRAIN SIZE DISTRIBUTION CURVES OF PORT DOVER SILT

*CURVES OF RANDOM SAMPLES D_3 AND D_5 REPRESENT THE GRAIN SIZE DISTRIBUTION FOR SAMPLES OF THE D-AND A-TEST SERIES.



AVERAGE DIAMETER OF SMALL GRAINS 0.001 MM
AVERAGE DIAMETER OF LARGE GRAINS 0.08 MM

MAGNIFICATION 890 DIAMETERS

FIG·8

PORT DOVER SILT GRAINS UNDER THE MICROSCOPE·

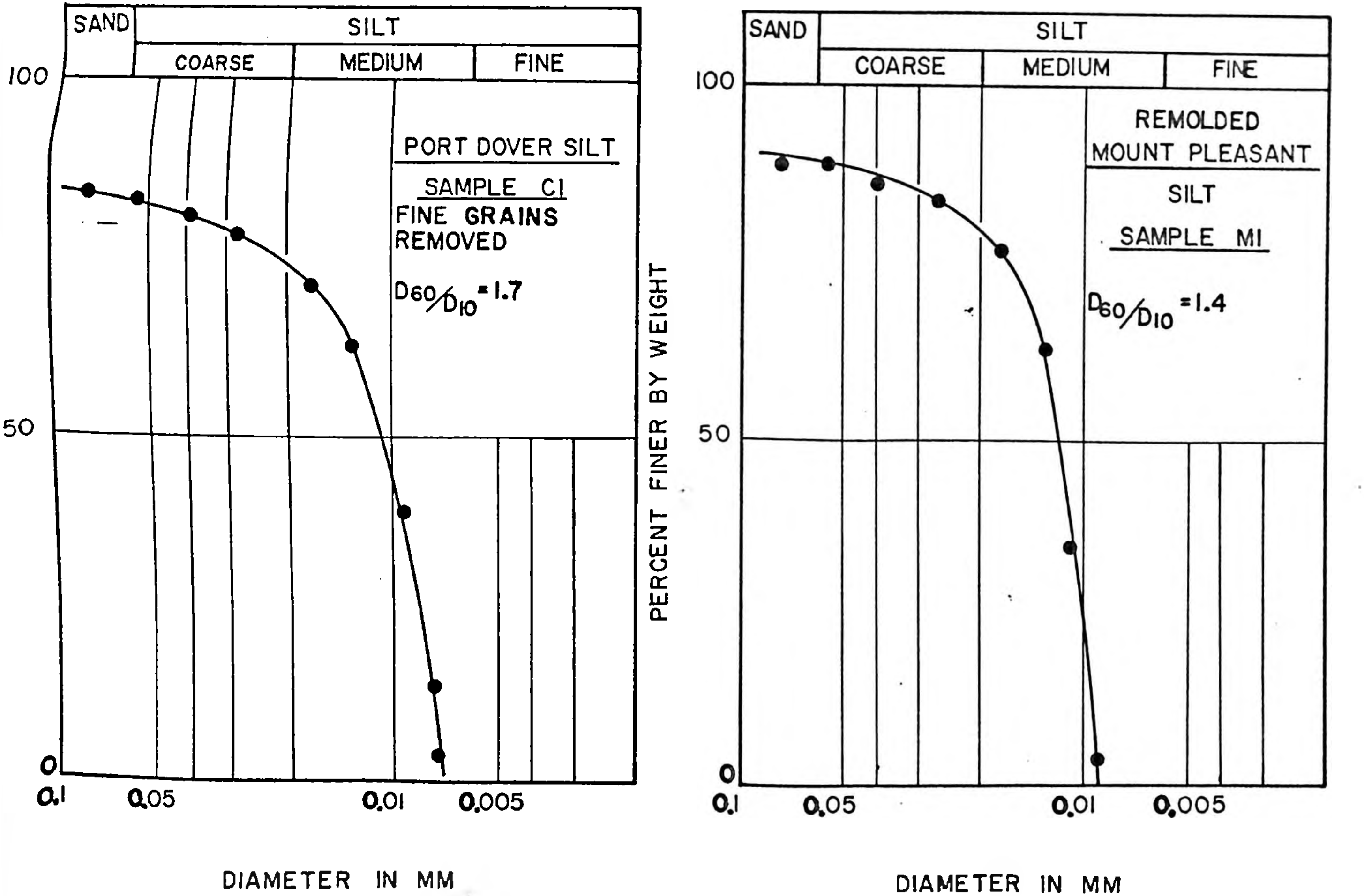


FIG. 9

GRAIN SIZE DISTRIBUTION CURVES
SAMPLES C1 AND M1

20 per cent. This material was used for the preparation of sample C_1. To remove the fine grains a thin slurry of Port Dover silt was allowed to settle in a sedimentation tube until only the fine grains remained in suspension; then the fine grains were siphoned off with the supernatant water.

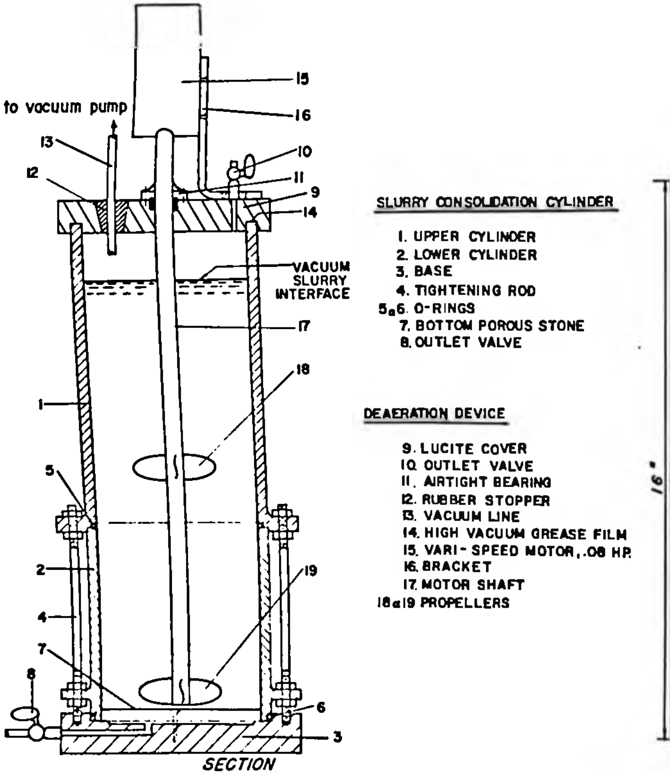
A different type of silt, a yellow silt, was obtained from silt layers, about 1 inch thick, of laminated soils found near Mount Pleasant, Ontario. The grain size distribution curve for this silt (Figure 9) showed a coarse to medium silt, with a uniformity coefficient (D_{60}/D_{10}) of 1.4 and with a sand fraction of about 10 per cent. Sample M_1 was prepared from this material.

To facilitate pore-water pressure measurements, soil samples with a high degree of saturation were prepared by deaerating and consolidating slurries, consisting of water and soil.

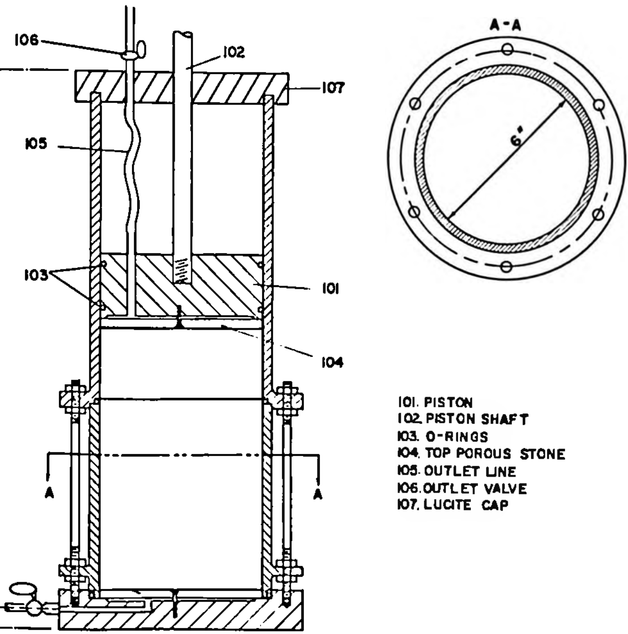
2) Apparatus to Prepare Saturated Homogeneous and Laminated Samples

The apparatus (Figures 10 and 11) was designed to produce, in the shortest possible time, saturated soil samples from slurries. The slurries were deaerated by stirring, under vacuum, at the vapour pressure of water. Stirring action provided a constant renewal of the vacuum-slurry interface and accelerated the deaeration process as air removal was not governed by diffusion of the air through the slurries. Immediately after the deaeration process the slurries were consolidated.

The slurry consolidation cylinder (Figure 10) provided storage for the slurry during the deaeration and consolidation processes and consisted basically of an upper cylinder (1), a lower cylinder (2) and a



FIG·10
SLURRY CONSOLIDATION CYLINDER
WITH DEAERATION DEVICE



SECTION

FIG. 11
SLURRY CONSOLIDATION CYLINDER
WITH PISTON.

base (3). Six tightening rods (4) screwed into the base held the upper and lower cylinders in position. Air-tight seals between the upper and lower cylinders and between the lower cylinder and the base were ensured by neoprene O-rings (5 & 6).

The apparatus was assembled for the deaeration of the slurry as indicated in Figure 10. The slurry consolidation cylinder was sealed on top by a lucite cover (9) which contained an outlet valve (10), an air-tight bearing (11), for the motor shaft (17) and a rubber stopper (12) to seal the vacuum line (13). The vacuum was applied by a Speedivac High Vacuum Pump. An air-tight seal between the lucite cover and the consolidation cylinder was ensured by a high vacuum grease film (14) placed on the top rim of the upper cylinder. Stirring action for the slurry was provided by two propellers (18 & 19) attached to the motor shaft and rotated by a vari-speed motor (15). A bracket (16) connected the vari-speed motor rigidly to the lucite cover which was held in position by the vacuum in the slurry consolidation cylinder.

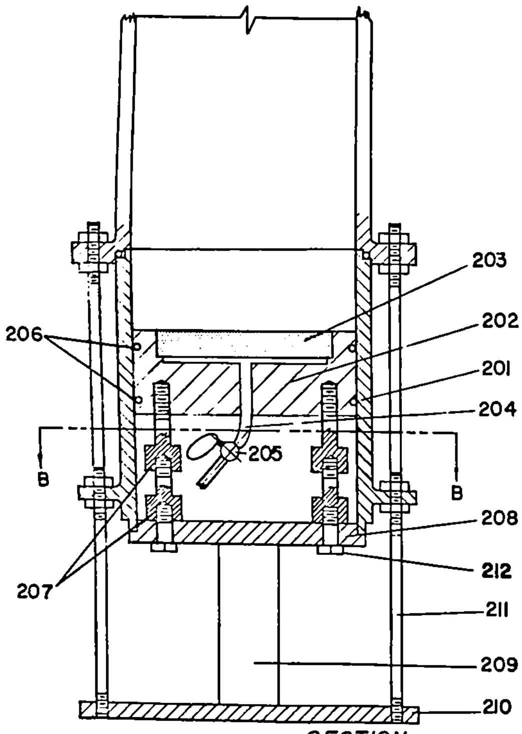
For the consolidation process, the lucite cover with the stirring device was removed. The piston (101) was inserted into the upper cylinder (Figure 11). A lucite cap (107) and two O-rings (103) around the piston provided guidance for the piston; the O-rings also ensured an air-tight seal. The slurry was consolidated by applying an axial compressive force to the end of the piston shaft (102) and by allowing the water to escape through a top porous stone (104) attached to the piston and through a bottom porous stone (7) embedded in the base (Figure 10). After the slurry was consolidated into the lower cylinder, the upper cylinder was removed.

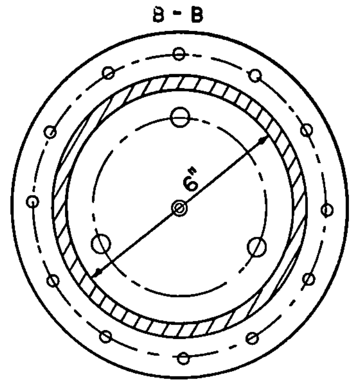
Figure 12 shows the slurry consolidation cylinder altered to prepare laminated samples. The base was removed and the lower cylinder was elevated by a set of six threaded rods (211) screwed into a base plate (210). A lower piston (202) provided an adjustable bottom for the lower cylinder, as the level of the piston could be varied by the adjusting legs (207). Two O-rings (206) ensured guidance and an air-tight seal for the lower piston. The adjusting legs were fastened by three securing bolts (212) to a bottom plate (208) which was supported by a steel rod (209) and tightened against the lower cylinder by the six threaded rods. In this way vertical movement of the lower piston was prevented during the deaeration and compression processes. The clearance below the slurry compression cylinder facilitated removal of the adjusting legs during the preparation of laminated samples. The procedure to prepare laminated samples, with the altered slurry consolidation cylinder, is described on Page 34.

3) Procedure for Preparation of Saturated Soil Samples in the Slurry Consolidation Cylinder

The weight of the slurry necessary to prepare a required sample was calculated from the dry weight of the required sample and the water content of the slurry; the water content was about 1000 per cent.

For the preparation of the soil samples for this research program a consolidation pressure of 20 psi was used; this pressure is equivalent to the over-burden pressure of a submerged soil stratum about 35 feet deep.





201. LOWER CYLINDER FIGURE
202. LOWER PISTON
203. POROUS STONE
204. OUTLET TUBE
205. OUTLET VALVE
206. O-RINGS
207. ADJUSTING LEGS
208. BOTTOM PLATE
209. STEEL ROD
210. BASE PLATE
211 . THREADED RODS
212. SECURING BOLTS

FIG. 12
SLURRY CONSOLIDATION CYLINDER ALTERED FOR LAMINATED SAMPLES.

A) Preparation of Homogeneous Soil Samples

Procedure:

(i) The required amount of slurry was poured into the slurry consolidation cylinder.

(ii) The deaeration device was assembled (Figure 10) and an absolute pressure of 2 millimeters of mercury was applied to the vacuum line. The silt was deaerated by stirring it for 3 hours under vacuum.

(iii) The deaeration device was removed. The slurry was consolidated by inserting the piston (Figure 11) and applying the required axial force to the end of the piston shaft. The top and bottom outlet valves were opened to allow the water to escape through the top and bottom porous stones.

(iv) Random air content tests of the drained water were carried out by "The Alsterberg Modification of the Winkler Method for Determining Dissolved Oxygen". (American Public Health Association Inc., 1960).

(v) The slurry was consolidated until the porous stone of the piston had entered the lower cylinder. In this way a layer of water was kept on top of the sample after removal of the piston to avoid diffusion of air into the sample. The piston was removed with the upper cylinder to consolidate the sample further, using the common technique of consolidation, until primary consolidation had ceased.

This consolidation procedure (i) to (v) was adopted for all silt samples. With this procedure it was possible to turn out one silt sample about 5 inches high and 6 inches in diameter every 48 hours with two loading devices. During the consolidation period, using the common technique of consolidation, some air could diffuse into the sample through the top porous stone. If samples with full saturation throughout should be required, the upper cylinder and the piston could be maintained in place until primary consolidation had ceased. In this case, the pressure on the slurry could be adjusted for frictional resistance of the piston.

The prepared silt samples were checked for a consistent grain size distribution by hydrometer analyses on specimens from the top, middle and bottom portions of a number of samples. The variations in void ratio, water content and degree of saturation with depth were also investigated.

B) Preparation of Laminated Samples of Different Soils

The slurry consolidation cylinder was assembled as in Figure 12, with the distance between the top of the lower piston and the rim of the lower cylinder equal to the thickness of the required soil layer plus the thickness of the porous stone attached to the upper piston.

Procedure:

(i) Steps (i) to (iv) of part III 3 A.

(ii) The slurry was consolidated until the porous stone of the piston had entered the lower cylinder. The upper cylinder was removed. The upper cylinder and upper piston were separated, and the upper cylinder was reassembled.

(iii) The amount of slurry required for the following layer was poured into the apparatus and deaerated as in step (ii) part III 3 A.

(iv) After deaeration the steel rod and the bottom plate were removed and the adjusting legs were shortened by the thickness of the following layer. The bottom plate and steel rod were reassembled.

(v) The slurry was consolidated (as in step (iii) of part III 3 A) forming a layer of soil on top of the previous layer. The axial force applied to the piston shaft during consolidation forced the bottom piston downwards until the shortened adjusting legs reached the bottom plate. The adjusting legs were then fastened to the bottom plate by the securing bolts, to prevent upward movement of the lower piston during vacuum application for the deaeration of the following soil layer.

(vi) Step (iv) of part II 3 A.

(vii) Procedure (ii) to (vi) of part III 3 B was repeated for each layer.

(viii) The slurry of the last layer was consolidated until the porous stone of the upper piston had entered the lower cylinder. The upper cylinder was removed and the sample was further consolidated, using common consolidation techniques, until primary consolidation had ceased.

4) Properties of Silt Samples Prepared in the Slurry Compression Cylinder

The "D-test series," samples D_1, D_2, D_3, D_5, D_6, D_7, and D_9, consisted of replicate homogeneous Port Dover silt samples. Sample D_4 was tested with the vane without measurement of pore-water pressure and sample D_8 was used for consolidated undrained triaxial tests; the results of neither sample are presented in this work. The properties of the samples of the "D-test series" indicated a variation throughout the height of a sample in water content, void ratio and degree of saturation (Table I). The highest void ratio existed in the middle portion of each sample tested (Table I, Figure 13), as this portion was the last to be consolidated. During consolidation of the slurry, soil was first deposited in the top and bottom portion of a sample as the flow of water was directed to the top and bottom stones. The degree of saturation increased with depth for the samples tested (Table I, Figure 13) as the samples were not sealed at the top during the consolidation period using common consolidation technique. The bottom region in which the vane tests took place was almost fully saturated. Data from hydrometer analyses indicated a consistent grain size distribution throughout the height for the samples tested (Figure 7). Differences in the average water content and void ratio between replicate samples (Table I) were caused by variations in consolidation pressure during sample preparation. (As the consolidometer was operated by air pressure, pressure changes in the air line caused variations in consolidation pressure.)

The "A-test series," samples A_1, A_2 and A_4, consisted of a bottom layer of Port Dover silt, about 3 inches thick, and a top layer of South Hamilton clay, 1 inch thick. Sample A_3 was not 100 per cent consolidated and is not presented in this work. The average water content and void ratio of Port Dover silt is lower for the "A-test series" than for the "D-test series" (Table I), as the silt of the laminated samples underwent secondary consolidation during consolidation of the top clay layer.

Water contents and void ratios for sample C_1 and M_1 were not determined.

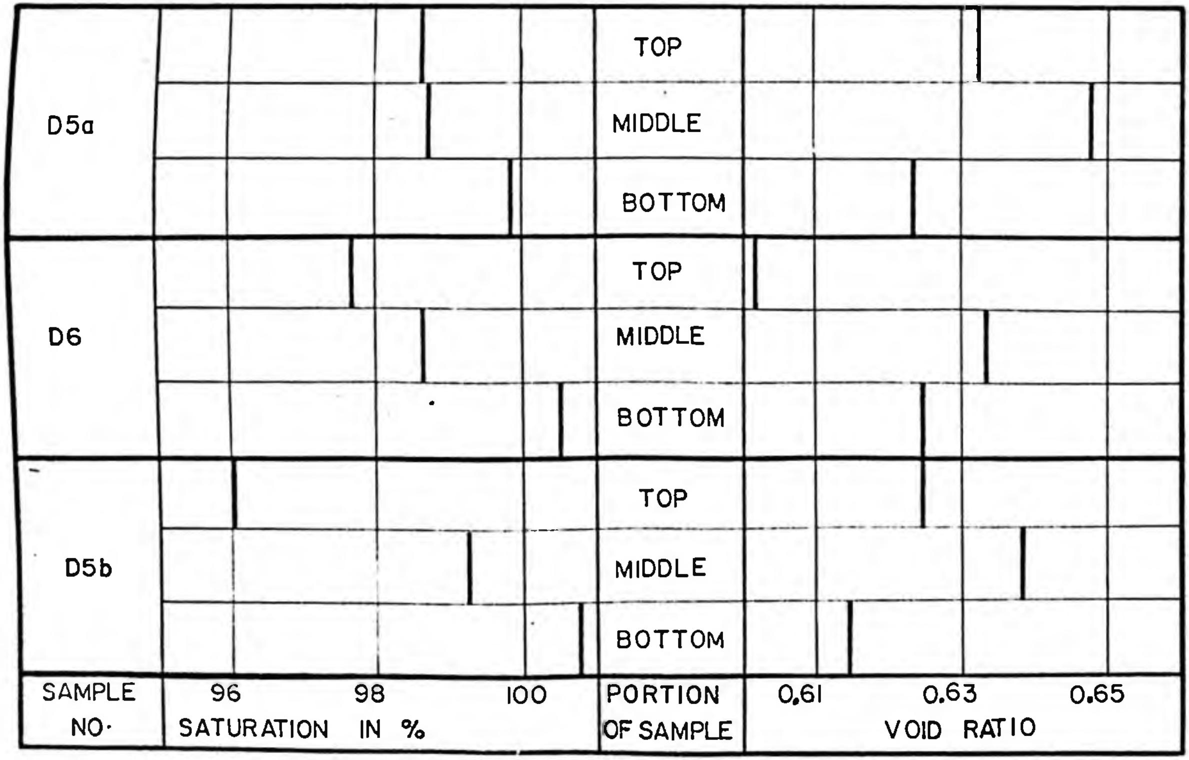


FIG.13

VARIATION OF SATURATION AND VOID RATIO WITH DEPTH. PORT DOVER SILT
SAMPLES PREPARED IN SLURRY CONSOLIDATION CYLINDER.

CHAPTER IV

EXPERIMENTAL APPARATUS AND PROCEDURE

1) Experimental Apparatus

A) Laboratory Vane Apparatus

The laboratory vane, a hand operated basic model, was purchased from Wykeham Farrance Engineering Limited. It was modified by the addition of a variable speed motor (Figure 14). In this apparatus the torque is applied through calibrated torsion springs. Torque angle and strain are indicated on a rotational dial graduated in degrees. The vane (1 inch high x 3/4 inch diameter) was fabricated from a stainless steel bar (Figure 15). During the early laboratory vane tests on silt, the pore pressure probe, a hypodermic needle (0.0614 inches O.D.) was attached to the stationary socket of the apparatus (Figure 15 a) and inserted with the vane into the sample. The hypodermic needle was closed at the pointed end, but had a slot(0.75 inches x 0.0614 inches) facing the vane; a No. 200 stainless steel mesh was inserted into the slot and provided an interface where pore-water pressures were measured. The laminated samples of the "A-test series" and sample C_1 were tested with this arrangement. As pore-water pressure measurements were not consistent, the hypodermic needle was passed through a hollow vane shaft and attached to the vane running along the edge of one vane blade, which was cut back the diameter of the needle (Figure 15 b). The slot with the No. 200 mesh

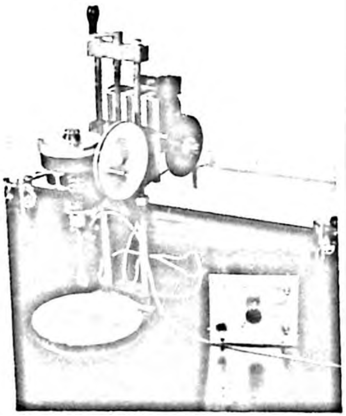
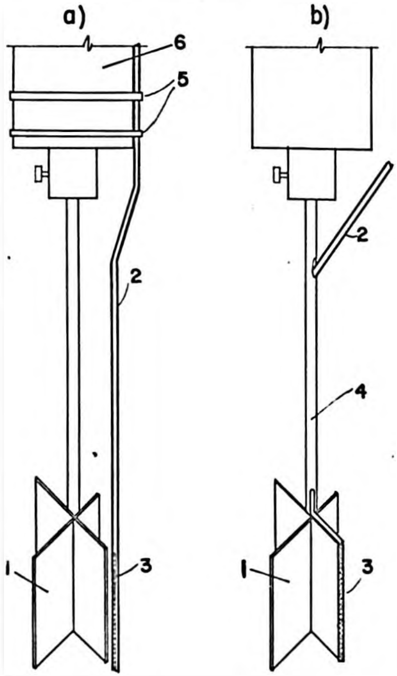


FIG. 14

LABORATORY VANE APPARATUS WITH VARIABLE SPEED MOTOR.



1- VANE 1" x 3/4" DIAMETER
2- HYPODERMIC NEEDLE 0.0614" O.D.
3- SLOT WITH NO. 200 STAIN LESS STEEL MESH
4- HOLLOW SHAFT
5- CLAMPS
6- STATIONARY SOCKET

FIG. 15

METHODS OF PORE WATER PRESSURE MEASUREMENT ON VANE.

represented the outer edge of the vane blade. This arrangement caused some additional disturbance, but improved pore-water pressure readings could only be obtained by measuring it directly on the failure surface in this fashion.

B) Pore Pressure Apparatus

The pore pressure apparatus was a "no-flow" device as developed by the Norwegian Geotechnical Institute (Andersen, Bjerrum, Dibiago, Kjaernsli, 1957). It was modified (Figure 16) by replacing the 1/2 inch O.D. copper tubing and Klinger valves by glass tubing and high vacuum stop cocks. The 1/8 inch O.D. copper tubing connecting the probe with the pore pressure instrument was replaced by 1/8 inch O.D. plastic tubing. These modifications were necessary so that trapped air bubbles in the apparatus could be observed. The 1/8 inch O.D. plastic tubing also provided a flexible connection between the pore pressure instrument and the probe, which rotated with the vane. The deaeration of the water used in the pore pressure apparatus was accomplished by stirring at the vapour pressure of water in a three gallon bottle by a "Magnestir" until air bubbles ceased to appear. Using this procedure, it was possible to reduce the air content of the water to less than one milligram per litre, as determined by random checks. The entire pore pressure device was flushed with this water continuously until it was possible to reduce the internal pressure to the vapour pressure of water by the screw control.

As pore-water pressures generated by the vane were confined to a small volume of pore-water, the "no-flow" condition of the pore pressure apparatus was investigated. It was attempted to control flow of pore-

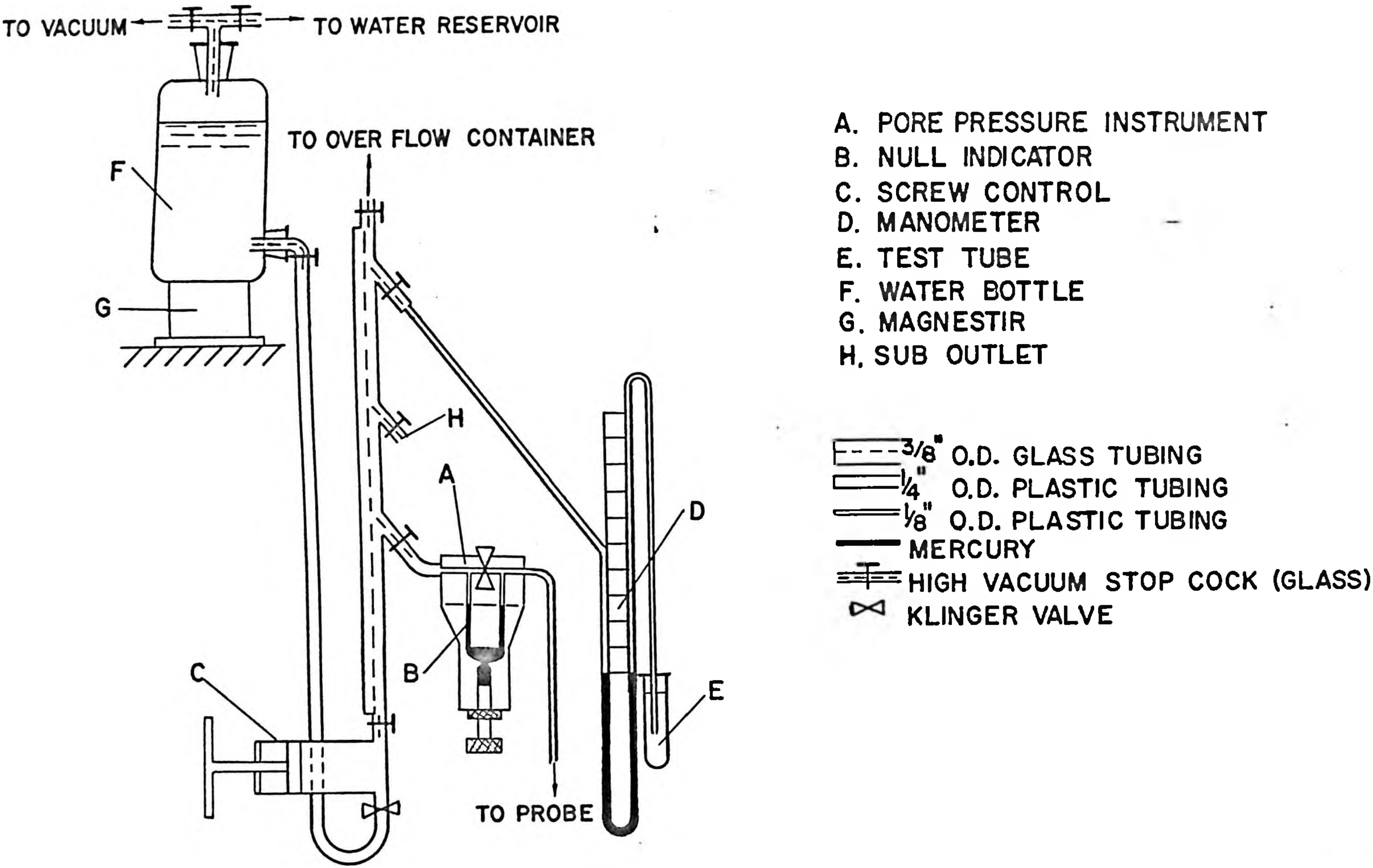


FIG.16

MODIFIED NORWEGIAN PORE PRESSURE APPARATUS.

water during pore pressure measurements by maintaining the null indicator in the pore pressure instrument level (Figure 16). This technique could not prevent the flow of pore-water due to volume changes that took place during pressure variations in the 1/8 inch O.D. plastic tubing connecting the pore pressure instrument and the probe. The pore pressure apparatus was calibrated for these volume changes by closing the plastic tubing connecting the pore pressure instrument and the probe at the end where the probe was usually assembled; the pressure in the pore pressure apparatus was then varied with the screw control and the displacement of the right branch of the null indicator was measured at various pressures. The displacement of the right branch of the null indicator at various pressures is shown in Figure 17 for an 1/8 inch O.D. "Polyethylene" tubing, 2 feet long, used during vane tests, and an 1/8 inch O.D. "Tygon" tubing, 5 feet long, used during triaxial tests. To avoid the flow of pore-water in or out of the sample during a test, the pore pressure scale readings should be taken with the right branch of the null indicator displaced by the amount indicated on the calibration curve (Figure 17). This displacement of the null indicator would compensate for volume changes in the 1/8 inch O.D. plastic tubing due to variations in pressure, but would create a different datum for each scale reading. As the displacement of the right branch of the null indicator was of the order of readability of the pore pressure instrument for the range of pore pressure generated by the vane, scale readings were taken with the null indicator maintained level.

During vane tests with applied negative pore-water pressures the

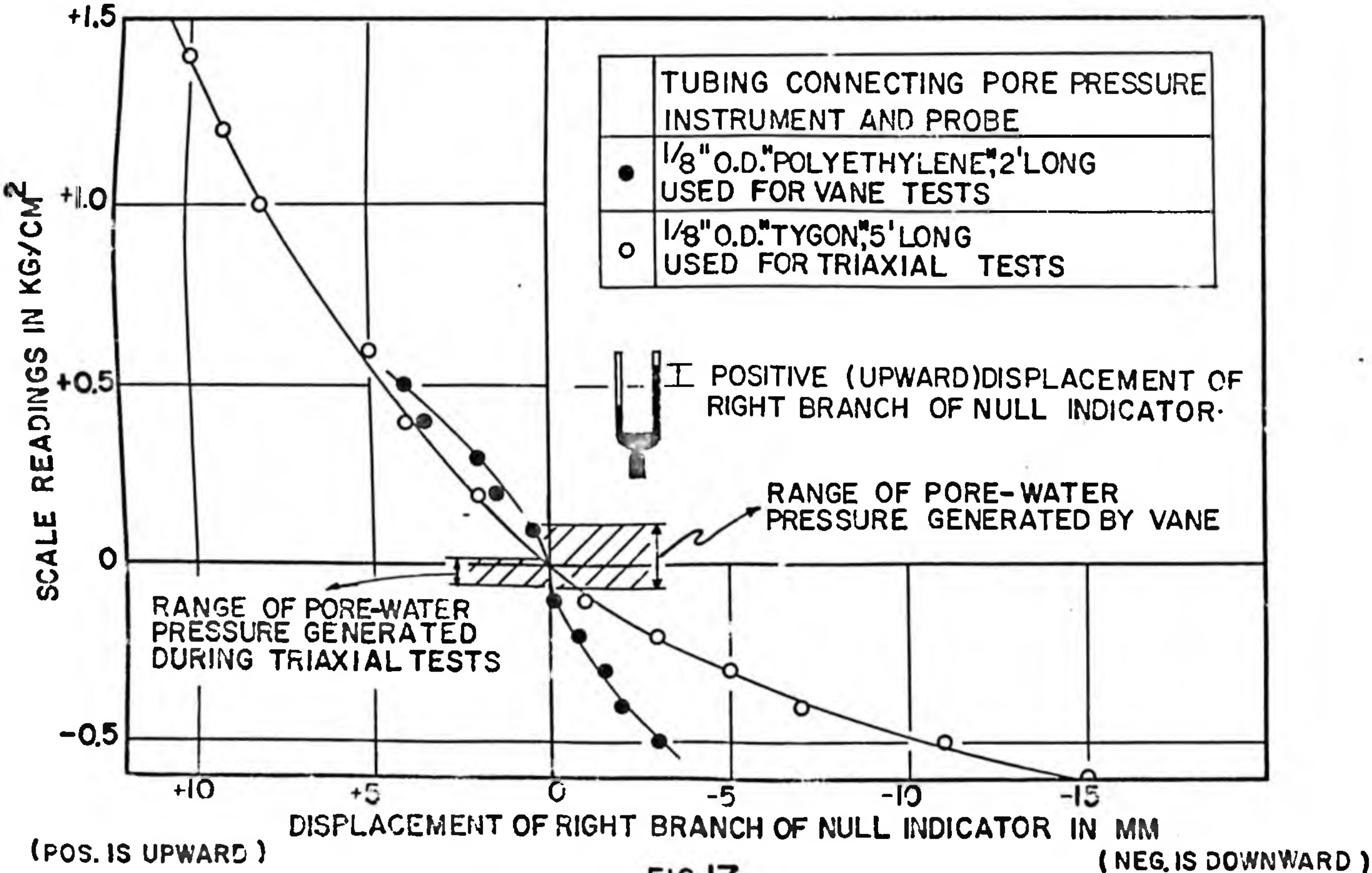


FIG. 17

CALIBRATION CURVES OF "NO-FLOW" PORE PRESSURE DEVICE probe was inserted with the vane before the negative pore-water pressure was applied. Therefore, volume changes were generated by the applied negative pore-water pressure in the 1/8 inch O.D. plastic tubing, connecting the pore pressure instrument and probe, before the vane shear test was conducted. The displacement of the right branch of the null indicator due to pore-water pressure changes generated by the vane during shear deformations was again of the order of readability of the instrument.

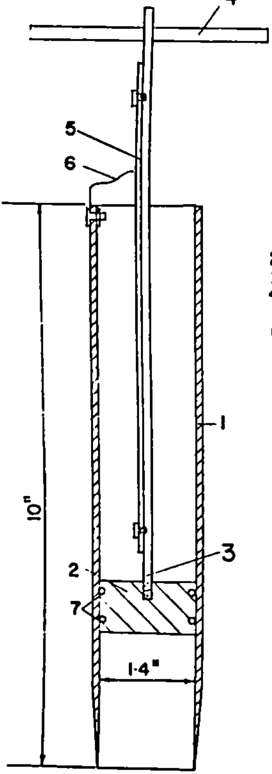
For triaxial tests, the volume of the samples was considered to be large enough to compensate for the small displacement of the null indicator due to pressure changes (Figure 17) over the range tested.

All pore-water pressure readings in this research work were taken with the null indicator maintained level.

C) Triaxial Apparatus

The apparatus used for triaxial compression tests was manufactured by Karol Warner Inc., (Model 500, Serial 33). It was a standard strain controlled apparatus and was designed for soil samples 2.8 inches high and 1.4 inches diameter.

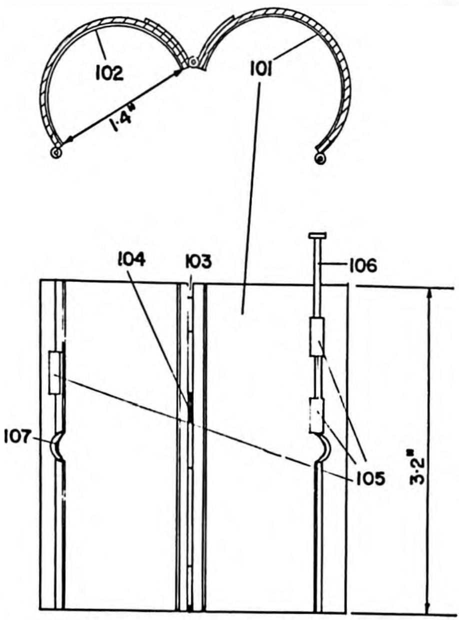
To extract triaxial samples from silt samples (5 inches high x 6 inches diameter) prepared in the slurry compression cylinder a static-piston sampler was used (Figure 18). This device consisted basically of a cylinder (10 inches high x 1.4 inches I.D.) with a lower cutting edge containing a piston with piston rod and handle. Two O-rings provided seal and guidance for the piston. To obtain a triaxial sample, the bottom of the piston was made coincident with the lower cutting edge of the



1· CYLINDER WITH CUTTING EDGE
2· PISTON
3· PISTON ROD
4· HANDLE
5· SCALE
6· POINTER
7· O-RINGS

FIG·18

STATIC-PISTON SAMPLER·



101· HALF CYLINDER
102·RUBBER LINING GLUED TO HALF CYLINDER
103· HINGE
104· COIL SPRING
105·LOCKING RECEPTACLES
106·PIN
107·HOLE FOR RUBBER PAD AND NEEDLE INSERTION

FIG·19

CONFINING CYLINDER FOR TRIAXIAL SAMPLES·
(OPENED)

cylinder. The cylinder was pushed vertically into the silt to the bottom of the sample while the piston was kept statically at the soil surface. The static-piston sampler was then extracted with the soil sample inside. The height of the sample was indicated by a pointer on a scale attached to the piston rod. The static-piston sampler was also used to obtain samples for void ratio and saturation tests.

During the assembly of a silt sample in the triaxial apparatus, a confining cylinder was used to minimize deformation of the sample (Figure 19). This device, consisting of two half cylinders hinged together, was closed around the triaxial sample after enclosure in the membrane until a confining pressure was applied in the triaxial cell. The confining cylinder was then opened by pulling a pin out of the locking receptacles with a wire passing through an air valve in the top of the triaxial cell. (The air valve was closed after removal of the wire). A spring connected to the hinge of the confining cylinder ensured the opening of the device, which fell to the bottom of the triaxial cell.

For pore-water pressure measurements on triaxial samples, the same type of probe was used as developed for the early vane tests in silt (Figure 15 a). This probe, 2 inches long, was inserted through the rubber membrane into the triaxial sample at an angle of about 50 degrees to the horizontal. To reinforce the rubber membrane for insertion of the probe a round pad of synthetic rubber (1/16 inch thick x 3/4 inch diameter) was glued to the outside of the membrane. A pad of natural rubber (1/8 inch thick x 3/8 inch diameter) was then glued to the synthetic rubber pad to provide an airtight seal for the probe. The shaft of the probe was coated above the slot with Pliobond (a trademark of the Goodyear Company) prior to insertion.

2) Procedure

A) Vane Tests with Pore Pressure Measurements

For all vane tests in silt, torsion spring No. IV (0.044 inch-pound per degree) was used except for tests No. 1 and 2 on Sample A_1 for which spring No. III (0.125 inch-pound per degree) was used.

a) Tests with Probe Inserted beside the Vane (Figure 15 a)

(i) The probe was flushed thoroughly by water from the pore pressure apparatus.

(ii) Vane and probe were inserted into the flooded soil until the wire mesh was submerged.

(iii) The probe was flushed again and the hydrostatic pressure was recorded.

(iv) The surface water was drained.

(v) Vane and probe were inserted to the required depth and a pore pressure reading was taken.

(vi) A waiting period of two to three minutes was introduced. Alternative procedure for (vi): No waiting period.

(vii) The torque was applied. Readings of maximum torque, strain and pore pressure readings at maximum torque were taken. (Due to the time required to take pore-water pressure readings it was not always possible to record pore-water pressures at maximum torque.)

b) Tests with Probe Attached to the Vane (Figure 15 b)

Same as (i) to (vii) in section IV 2 A a.

Alternative procedure for step (v) section IV 2 A a:

The vane was lowered to the required depth and the pore pressure was reduced to its lowest value by the screw control; a reduction to vapour pressure of water for a few seconds indicated an air free system.

Alternative procedure for step (vi) section IV 2 A a:

A negative pore pressure was applied to the sample through the bottom porous stone.

This procedure was adopted in order to obtain torque readings at greater negative pore-water pressures. Dilatancy produced a negative pore-water pressure of only 0.85 psi.

During the early tests on silt, the vane was turned by hand corresponding to an angular speed of the torque dial of approximately 10 degrees per second. After the variable speed motor was assembled, tests were carried out at an angular speed of 3 or 4 degrees per second.

A different type of test was run at the end of the research work by applying the torque in stages with short waiting periods between the stages to take readings. It was thus possible to record torque and pore-water pressures throughout the test as well as at failure. (The vane apparatus was slightly modified to make instantaneous torque angle readings possible). The tests were performed in stages of 5 degrees of rotation of the vane, taking for each stage about 5 seconds to rotate the vane and about 10 seconds to take readings. This was equivalent to an average angular speed of the vane of about 0.3 degrees per second which corresponds to an average velocity of the torque dial of about 1 degree per second.

B) Triaxial Tests on Silt

a) Preparation of Samples

(i) Identical to section III 3 A.

(ii) A triaxial soil sample was extracted with the static-piston sampler from the sample prepared in the slurry compression cylinder.

(iii) The piston of the static-piston sampler was moved outwards until a sample 2.8 inches high remained in the sampler. The end sticking out was cut off.

(iv) The sample was pushed into the prepared membrane carefully trying to avoid trapping air. It was then surrounded by the confining cylinder.

(v) The sample was set up in the triaxial apparatus, the pore pressure needle was inserted and a confining pressure applied. The confining cylinder was released and remained at the bottom of the pressure cell during the test.

b) Technique of $\overline{Q}$ Test

A $\overline{Q}$-test is defined as an undrained triaxial test with measurement of pore-water pressures. The compressive strength (i.e., the deviator stress at failure) is found to be independent of the cell pressure, with the exception of fissured clays and compact silts at low cell pressures (Bjerrum and Bishop, 1960). For fully saturated soils, an increase in cell pressure is reflected by an equal increase in pore pressure and the effective stress at failure remains unchanged for tests conducted at different cell pressures.

Procedure:

(i) To check the response of the pore pressure device and the saturation of the sample, the confining pressure was increased in steps of 2 psi up to 16 psi and the pore-water pressure was recorded at each step. Confining pressure and pore-water pressure were then reduced to zero.

(ii) The axial load was applied. Deformation, pore-water pressures and load were recorded at 1/2 minute intervals until the axial load started to decrease.

The strain rate for the triaxial tests on silt was approximately 5 per cent per minute. The tests were conducted without an applied confining pressure.

CHAPTER V

RESULTS AND DISCUSSION OF RESULTS

1) Vane Tests on Remolded Port Dover Silt

Results from vane tests conducted on samples of the "D-test series" indicated an increase in maximum torque with a decrease in pore-water pressure (Figure 20). The regression line (Figure 20) was obtained by the method of least squares (Steel and Torrie, 1960). The line has the property that the sum of the squares of vertical deviations of the data from this line is smaller than the corresponding sum of squares from any other line. The line is valid for the estimation of the maximum torque values from given pore-water pressures. The coefficient of correlation for the test results indicated in Figure 20 was 0.89. Complete absence of correlation is indicated by a value of zero. The standard error of estimate was 0.19 inch-pound for the maximum torque.

The results of statistical analysis indicated that a significant dependence of maximum torque on pore-water pressure existed and that the testing technique developed gave consistent results. The maximum negative pore-water pressure generated by the vane (0.85 psi, Figure 20) did not appear to cause a significant increase in maximum torque from the maximum torque obtained at zero pore-water pressure, as this increase was approximately equal to the standard error of estimate for the maximum torque.

Table I and Table III indicate that a number of additional

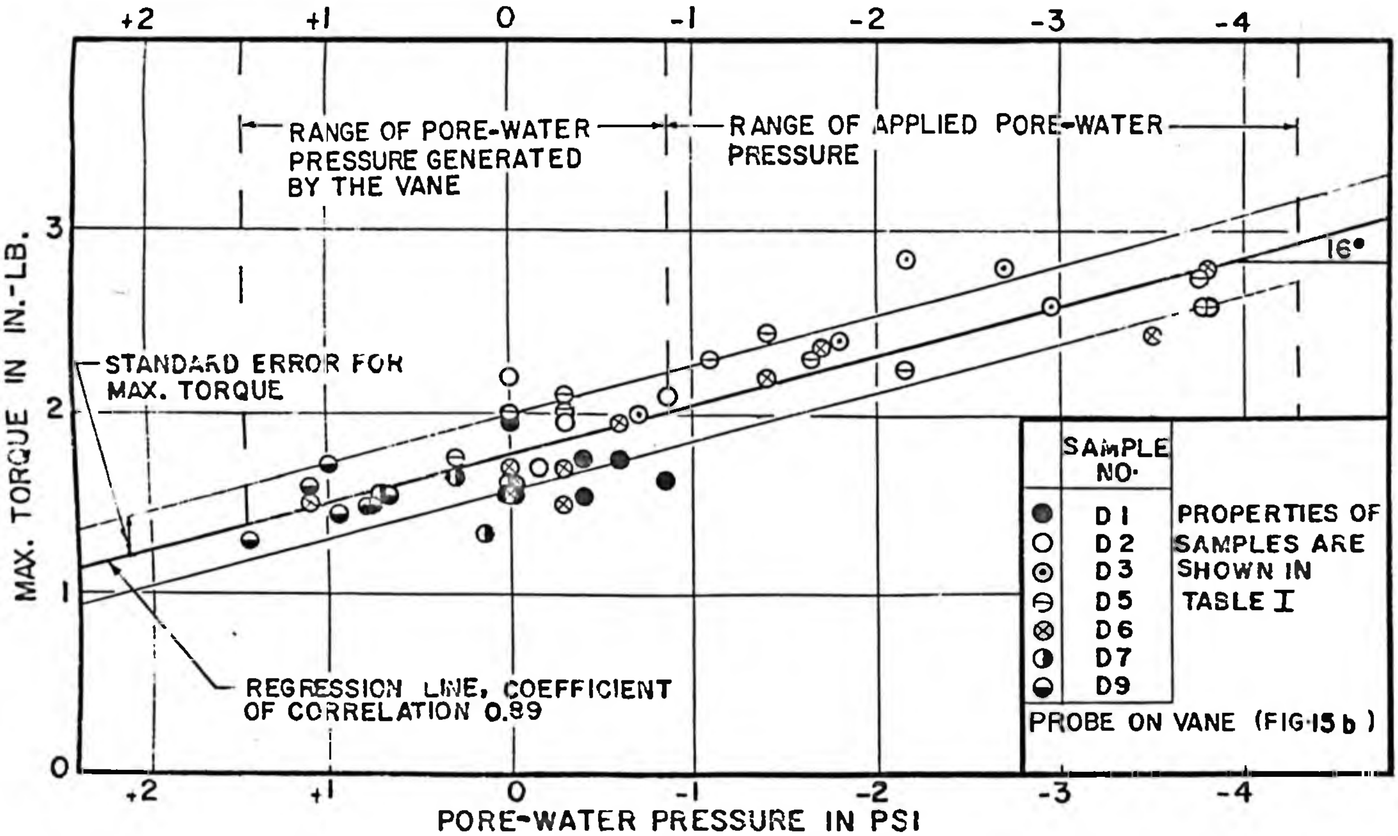


FIG. 20

VANE TESTS ON REMOLDED PORT COVER SILT, MAX. TORQUE vs. PORE-WATER PRESSURE FOR "D-TEST SERIES".

variables could effect the maximum torque/pore-water pressure relationship in Figure 20. The variables encountered were:

1) angular speed of the torque dial (1, 3 or 4 degrees per second),

2) depth of insertion of the vane (2 or 3 inches) and

3) average water content (Range: 22.5 to 24.3 per cent).

To investigate the possible influence of angular speed of the torque dial on test results expressed in Figure 20, the coefficient of correlation was determined for only those tests performed at an angular speed of 4 degrees per second. The value of this coefficient of correlation (0.86) was not appreciably different from the overall coefficient (0.89) calculated for the data in Figure 20. It was concluded that the change in angular speed of the torque dial from 3 to 4 degrees per second did not significantly effect the test results. (The small number of vane tests conducted at an angular speed of 1 degree per second (Table III) was considered to have no appreciable effect on the overall coefficient of correlation calculated for the data in Figure 20, as the 3 points obtained from these tests were almost coincident with the regression line.)

The results in Figure 20 were replotted in Figure 21 to compare maximum torque/pore-water pressure relationships for vane tests conducted at depths of insertion of 2 inches and 3 inches. The coefficient of correlation for vane tests conducted at a depth of insertion of 3 inches only was identical to the overall coefficient of correlation (0.89) calculated for the data in Figure 20 or 21. It was concluded that the variation in depth from 2 to 3 inches did not affect the test results.

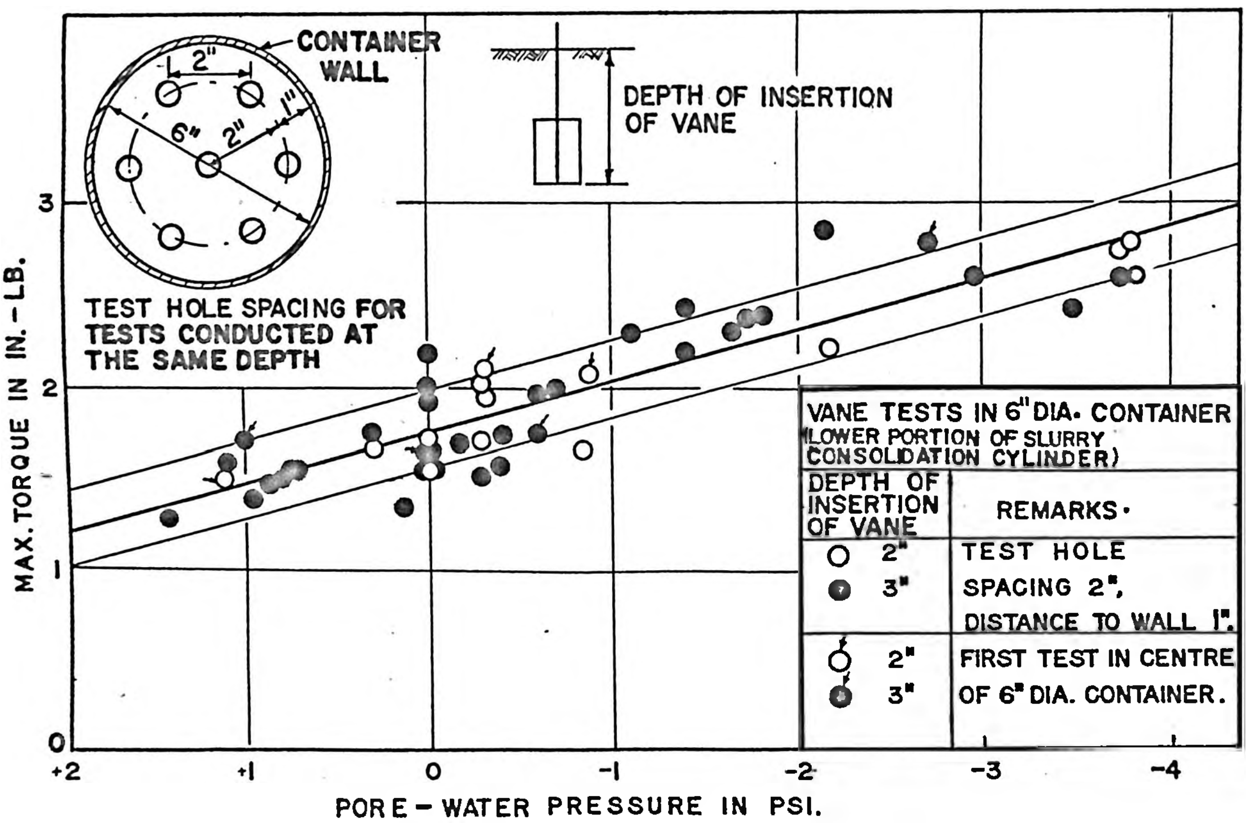


FIG.21

VANE TESTS ON REMOLDED PORT DOVER SILT, EFFECT OF DEPTH, WALL & TEST HOLE SPACING ON RESULTS IN FIG.20

The test hole spacing for vane tests conducted at the same depth in a silt sample (prepared in the slurry consolidation cylinder) and the distance of the test holes from the container wall are also shown in Figure 21. The coefficient of correlation for the tests carried out in the centre of the container was 0.87. With this value for the correlation, no evidence existed that wall effects were present with this type of silt when tests were conducted at a distance not less than 1 inch from the container wall. The vane tests conducted in the centre of the container were the first tests performed on each sample (see remarks Table III). Therefore, the coefficient of correlation for the tests conducted in the centre of the container (0.87) indicated also that the test hole spacing had no appreciable effect on the test results, if the spacing was not less than 2 inches.

From results of vane tests performed in clay (Wilson, 1961), it was noted that an increase in maximum torque with a decrease in water content existed. To investigate the possible effect of water content on the results in Figure 20, the maximum torque values were reduced to equivalent values of torque at zero pore-water pressure by projecting the points in Figure 20 parallel to the regression line. In this way, the standard error for the torque was not altered. The maximum torque values obtained in this fashion were then plotted versus water content, for constant depth and constant angular speed of the torque dial as shown in Figure 22. A coefficient of correlation of 0.5 between maximum torque and water content was obtained for tests conducted at a depth of 3 inches and an angular speed of 4 degrees per second. With this value for the correlation,

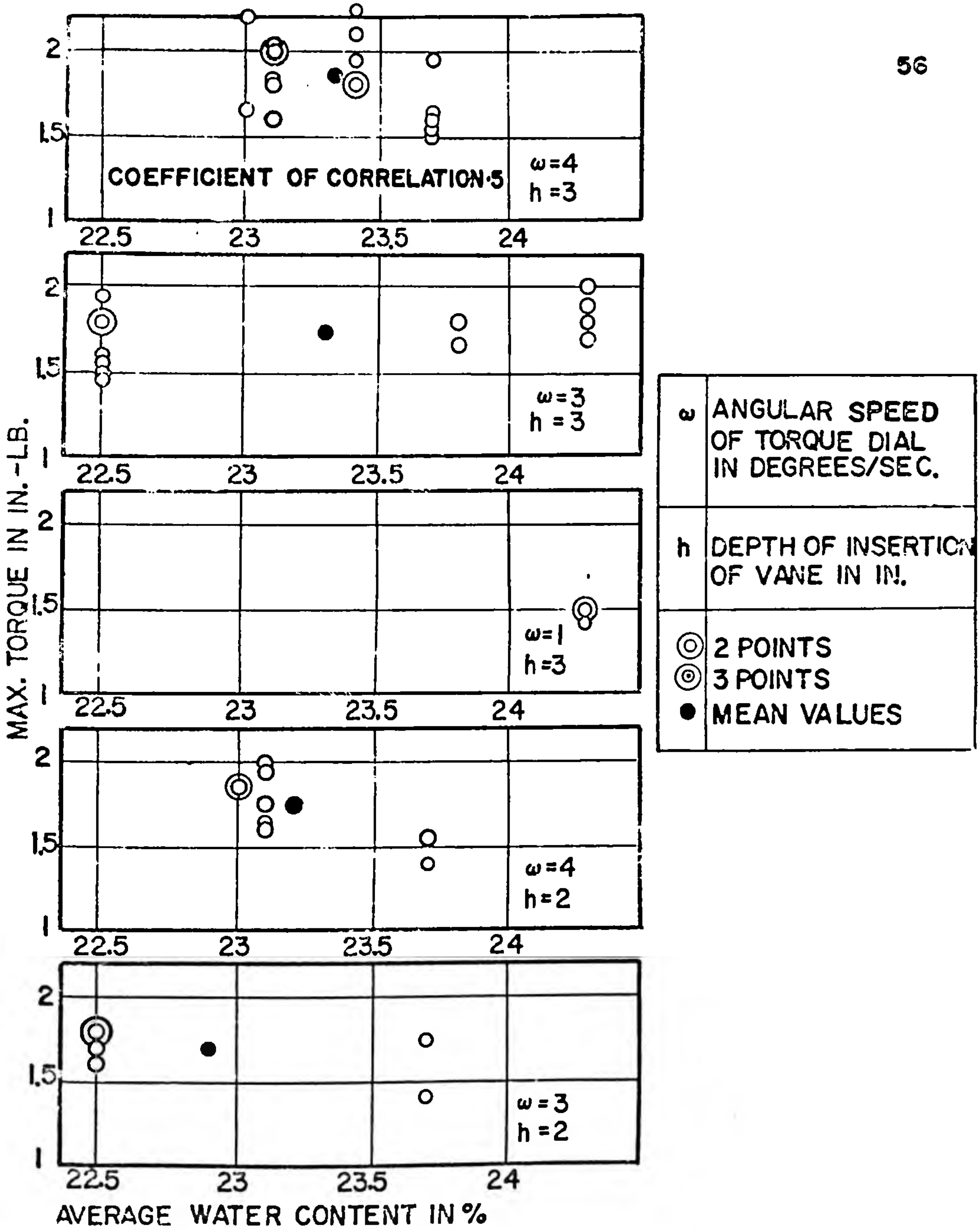


FIG. 22

VANE TESTS ON REMOLDED PORT DOVER SILT, MAX. TORQUE VS. WATER CONTENT; MAX. TORQUE VALUES WERE INTERPOLATED TO AN EQUIVALENT VALUE OF TORQUE AT ZERO PORE-WATER PRESSURE PARALLEL TO REGRESSION LINE IN FIG·20.

no evidence of the dependence of maximum torque on water content existed over the range tested.

A comparison of mean values shown in the individual graphs in Figure 22 indicated a larger average maximum torque required for greater depths and for higher angular speeds of the torque dial. The possibility existed that these differences were mere chance effects (Steel and Torrie, 1960). A test for the significance of differences between two means, an "F-test", was made for tests performed at depths of 2 inches and 3 inches having a constant angular speed of 4 degrees per second. This test consisted of the comparison of the variation of all the tests considered, which represented chance, to the variation between the two means, which represented measurable difference. For this test, the chance variations were notably larger than the observed differences between the means of the two groups. This fact was also observed for tests conducted at an angular speed of the torque dial of 3 and 4 degrees per second for a depth of insertion of 3 inches. It was concluded that, for the range of values of depth or angular speed used, (Figure 22), no significant difference between the means could be established statistically.

Although no statistical evidence was established in Figure 22, a trend to increased torque with increased depth and increased angular speed existed. Further testing is required over a wider range of vane depth and angular speed of torque dial to verify this trend. The variation of torque with water content can be established in the same manner by tests over a larger range.

Additional vane tests conducted over a wider range of angular speed of the torque dial on the silt layer of laminated samples A_1, A_2 and A_4 were available. Maximum torque values were related to angular speed of the torque dial for the D- and A-test series in Figure 23. For the "D-test series" the mean values of maximum torque were chosen from Figure 22; the values of maximum torque for the "A-test series" were obtained from tests within a range of pore-water pressure from +0.3 to -0.3 psi (Table IV). (Tests No. 1 and 2, conducted on sample A_1 using torsion spring No. III, were excluded, as they were the only tests performed with spring No. III.). The data in Figure 23 verified the trend to increased maximum torque with increased angular speed of the torque dial. To determine the effect of angular speed of the torque dial on maximum torque values, the maximum torque value corresponding to an infinitely small angular speed has to be estimated. A curve fitted to the data in Figure 23 can be extrapolated to estimate the maximum torque value corresponding to an infinitely small angular speed. As the data (Figure 23) indicated a non-linear dependence of maximum torque on angular speed at values of angular speed smaller than 3 degrees per second, additional vane tests are required to establish this curve.

An angular speed of the torque dial of 3 degrees per second was the lower limit for turning the torque dial by the motor.

Insertion of the vane usually caused an increase in pore-water pressure (Table III). For some of the vane tests of the "D-test series" a waiting period between insertion of the vane and torque application was employed to allow dissipation of positive pore-water pressures created by insertion of the vane. During these vane tests maximum torque values

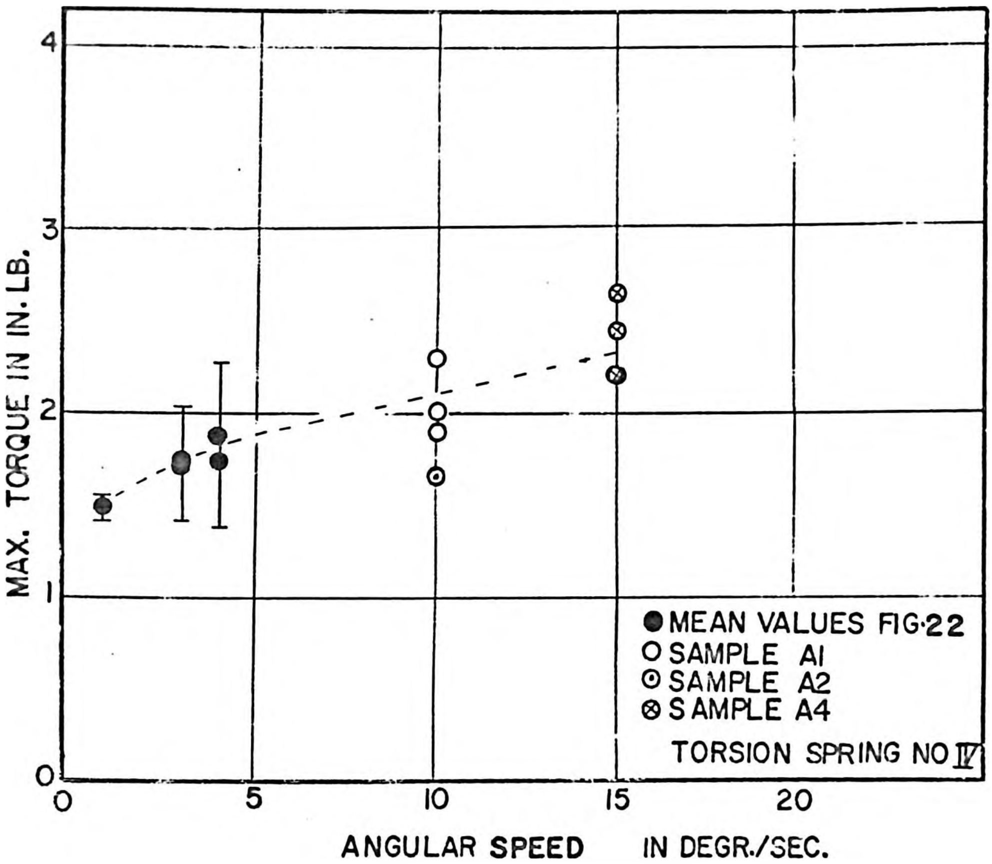


FIG.23

VANE TESTS ON REMOLDED PORT DOVER SILT, MAX. TORQUE VS. ANGULAR SPEED OF TORQUE DIAL.

were usually recorded at negative pore-water pressures (Table III). For vane tests conducted without a waiting period between insertion of the vane and torque application, the maximum torque was usually measured at positive pore-water pressures (Table III). Vane tests conducted with a waiting period before torque application indicated a higher maximum torque than vane tests without a waiting period (Table III and Figure 20). Duration of pore pressure dissipation on the incipient failure surface was about 3 to 4 minutes for 0.8 psi.

To observe the variation in pore-water pressure during a vane shear test, vane tests No. 5, 6 and 7 of sample D_9 were performed in stages of about 5 degrees of rotation of the vane (Figure 24). The time for each stage was approximately 15 seconds. The positive pore-water pressures at zero angle of rotation were caused by the insertion of the vane. The results indicated, in all 3 cases, an initial increase of pore-water pressure during application of the torque until the maximum torque was reached. Then dilatancy caused the pore-water pressure to decrease, but in two out of three cases did not reduce the pore-water pressure in the sample below the magnitude of the pore-water pressure created by insertion of the vane (Figure 24). This explained why the vane tests No. 1 to 4 conducted on sample D_9 at the usual angular speed of the torque dial of 3 degrees per second indicated an overall increase in pore-water pressure during a vane shear test (Table III). It should also be noted that sample D_9 had the highest void ratio of the series and was the least dilatant material. Dissipation of pore-water pressure during tests conducted in stages could account for differences from tests performed at the usual angular speed of the torque dial. Maximum torque was recorded

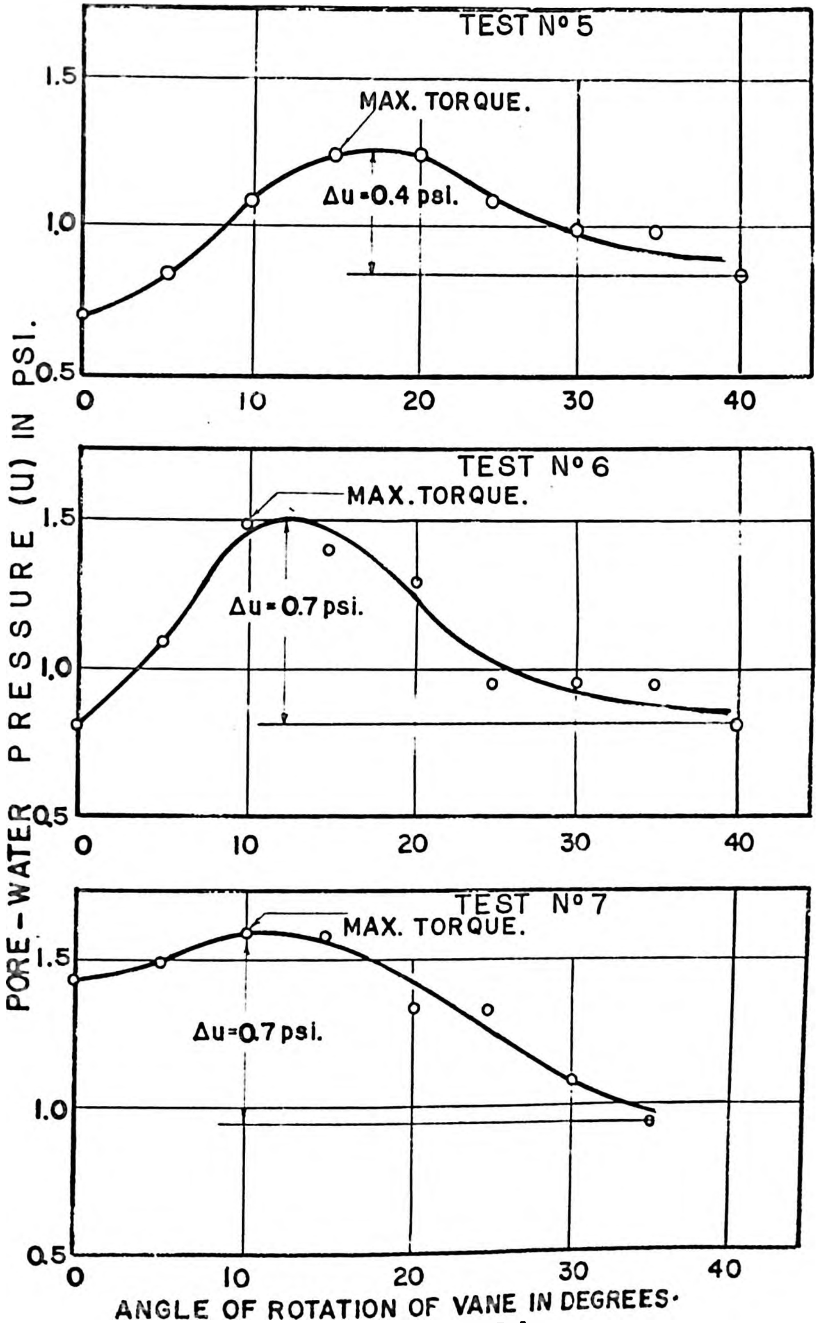


FIG. 24

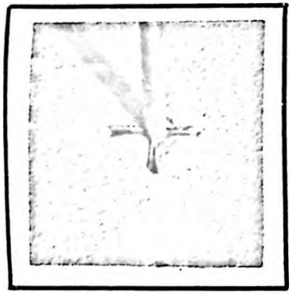
VANE TESTS ON REMOLDED PORT DOVER SILT.
PORE-WATER PRESSURE VS. ANGLE OF ROTATION OF VANE
FOR SAMPLE D9.

after 10 to 15 degrees of rotation of the vane, i.e., about 30 to 45 seconds. As this time was considerably shorter than the time taken for complete dissipation of pore-water pressure, the possibility of drainage for the period of pore pressure increase was reduced. The decrease in pore-water pressure, after the maximum torque was reached, may be influenced by drainage because of the time involved.

Pore pressure variations during undrained triaxial ($\bar{Q}$) tests on Port Dover silt samples D_3 and D_5 are indicated in Figure 32. A comparison of the pore pressure decreases at failure of undrained triaxial ($\bar{Q}$) tests (Figure 32) with the decreases during a vane test (Figure 24), showed that both were of the same order of magnitude. The results from triaxial sample D_5 showed similar characteristic variations in pore pressure as the results from vane tests.

It was attempted to determine the pore-water pressure at maximum torque, as this was considered to represent the torque necessary to cause a shear failure. For vane tests carried out at an angular speed of the torque dial of 3 or 4 degrees per second, inertia of the pore pressure apparatus and inability of the operator could account for errors in pore pressure readings taken at the instant of maximum torque. Due to the time required to adjust the pore pressure apparatus it was difficult to obtain instantaneous readings of a varying pore-water pressure, as the pore pressure in the sample had changed by the time the pore pressure instrument was adjusted. Accurate readings could only be made after the pore pressure remained fairly constant. Maximum torque values in test No. 6 (Figure 24) could have been recorded at a pore-water pressure varying from 1.5 to 0.8 psi due to the limitations of the testing equipment, if the test had been conducted at the ordinary angular speed of the torque dial of 3 degrees per second.

Photographs in Figure 25 indicated the development of a shear failure surface during vane tests. As these tests were conducted at a depth of insertion of 1 inch, it was possible that for tests at a greater depth a different type of shear failure existed. To investigate the failure conditions at a greater depth, a silt sample was prepared in a brass cylinder 5 inches high and 1.4 inches in diameter. The vane was inserted to a depth of 3 inches and the torque was applied until the angle of rotation of the vane was about 20 degrees. The sample then was baked and broken up. The soil fragments (Figure 26) showed the same features as indicated during vane tests at a depth of 1 inch. Considerable deformation (up to an angle of rotation of the vane of 15 degrees) took place during a vane test, as the material detached itself from the back of the vane blades creating a void, before a shear failure surface was visible. (Figure 25). Compressive stresses exerted by the vane blades caused this deformation. These compressive stresses were also indicated by the increase in pore-water pressure up to an angle of rotation of the vane of 12, 16 and 10 degrees (Figure 24). (Cadling and Odenstad (1950) could not satisfactorily explain the fact that rupture in clay did not occur until an angle of rotation of the vane of about 15 degrees was reached). The maximum torque of 1.4 inch-pounds (Figure 25) measured at an angle of rotation of the vane of 10 degrees could be caused by compressive stresses only as it was recorded before a shear failure surface appeared. The torque was smaller (1.3 inch-pounds) when a shear failure surface



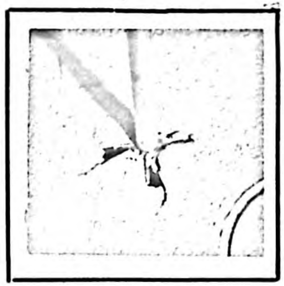
$\beta = 10°$
TORQUE = 1·4 IN.-LB.

$\beta = 15°$
TORQUE = 1·3 IN.-LB.

$\beta = 22°$
TORQUE = 1·3 IN.-LB.

$\beta = 26°$
TORQUE = 1·3 IN.-LB.

$\beta = 10°$
TORQUE = 1·4 IN.-LB.



$\beta = 25°$
TORQUE = 1·3 IN.-LB

β IS THE ANGLE OF ROTATION OF THE VANE.

FIG. 25

VANE TESTS ON REMOLDED PORT DOVER SILT.

TEST IN CYLINDER 5" HIGH X 1·4" DIA·
ANGLE OF ROTATION OF VANE 20°
DEPTH OF INSERTION OF VANE 3"

FIG· 26

VANE TEST ON REMOLDED PORT DOVER SILT·
FRAGMENTS OF A BAKED SILT SAMPLE·

appeared. It can be observed in Figure 25 that the shear failure surface in its early stages did not conform to a cylinder having the dimensions of the vane.

The pore pressure probe attached to one vane blade measured the pore-water pressure at one location of the shear failure surface only, whereas the pore pressure could have a different value at other locations of the failure surface. During the early vane tests the pore pressure probe was attached to a stationary socket of the apparatus (Figure 15 a) and inserted with the vane into the sample beside the subsequent failure surface. This technique of measurement of pore pressure gave inconsistent results. As the location of the pore pressure probe varied with respect to the failure surface for these early vane tests, the inconsistent results could indicate that different pore-water pressures existed at different locations on the failure surface. However, the possibility existed that the pore pressure probe inserted beside the vane was located close to a void created behind a vane blade and caused erroneous pore pressure readings for some vane tests. The formation of voids behind the vane blades was discovered after the tests with the probe inserted beside the vane were conducted.

A shear strength of 1.3 psi was obtained for remolded Port Dover silt from 4 undrained triaxial ($\bar{Q}$) tests conducted on samples of the "D- and A- test series" (Figure 31). The shear strength of each sample was approximated by one-half of the compressive strength $(\sigma_1 - \sigma_3)_{max.}$, measured at small effective minor principal stresses $(0.1 \text{ psi} < \bar{\sigma}_3 < 0.4 \text{ psi})$. This value would slightly exceed one-half of the unconfined compressive strength. By using equation 5 a shear strength of 1.6 psi was calculated for remolded Port Dover silt from 9 maximum torque values measured at zero pore-water pressure (Figure 20). This shear strength, obtained from the vane at zero pore-water pressure, exceeded the shear strength determined from undrained triaxial tests (1.3 psi), by about 23 per cent. (The shear strength calculated from vane tests would exceed one-half of the unconfined compressive strength by an even greater amount). The excessively high value for the shear strength, obtained from vane tests for remolded Port Dover silt at zero pore-water pressure, can be explained partly by the influence of testing speed (Figure 23). Further possible causes for the excessively high shear strength values, obtained from vane tests at zero pore-water pressure, could be the eccentricity of the vane blades, the additional disturbance caused by the probe attached to the vane and a non-uniform stress distribution on the failure surface. The state of stress is an unknown factor in the case of the vane, but can be determined for the triaxial test. For purely cohesive material, the shear stress distribution would be uniform on the failure surface of a vane as it would not be affected by any variation of the normal effective stress. This may be the reason that the results of laboratory vane tests and triaxial tests conducted on cohesive soils with small friction angles ($<$10 degrees) usually agree. As the shear strength of the silt depends on both the cohesive and frictional resistance, a non-uniform distribution of normal effective stress on the failure surface could influence the measured shear strength and cause disagreement between results from vane tests and triaxial tests. The formation of voids behind the vane blades during the initial stages of a vane test (Figure 24) indicates the presence of an effective normal stress on the incipient failure surface created by the vane.

The shear strength of Port Dover silt was calculated from vane test results by using equation 5. This equation was based on the assumption that the shear failure surface created by the vane was the total cylindrical surface conforming to the dimensions of the vane. By considering the reduction of the cylindrical shear failure surface due to the formation of voids behind the vane blades (Figure 25), the shear strength calculated from vane test results would exceed the shear strength calculated by using equation 5. No real conclusions about the validity of the shear strength values calculated from maximum torque values measured at zero pore-water pressure can be drawn, as the maximum torque values may have been caused by compressive stresses exerted by the vane before a shear failure surface was developed. Further testing is required to investigate the validity of the assumptions made for the establishment of equation 5.

2) Vane Tests on Laminated Samples

Vane tests in silt layers of laminated samples A_1, A_2, and A_4 were conducted with the pore-water pressure probe inserted beside the vane. During these tests the vane apparatus was operated by hand. The angular speed of the torque dial was approximately 10 degrees per second for samples A_1 and A_2; this rate of rotation (and a water content of about 22 per cent) caused higher values for maximum torque than those observed in case of homogeneous samples (Figure 23). Tests on sample A_4 were performed at an angular speed of 15 degrees per second.

Data from tests conducted on laminated samples indicated agreement with test results of homogeneous samples of the "D-test series" (Figure 27). The correlation of test results in the case of laminated samples is less than the correlation of results obtained from homogeneous samples, as an improved technique for measurement of pore-water pressures was used for the homogeneous samples (Figure 15). Results from vane tests conducted on sample A_4 were not plotted in Figure 27, as these tests were performed at an angular speed of the torque dial of 15 degrees per second. This angular speed was considered to affect the test results appreciably.

3) Vane Tests on Silt Coarser than Silts Used in "D- and A-test Series"

The grain size distribution curves for samples C_1 and M_1 (Figure 9) indicated a coarser and more uniform material than in the case of the grain size distribution curves representing samples of finer materials, the "D-test series" (Figure 7). Maximum torque values were plotted versus pore-water pressures for vane tests conducted on samples C_1 and M_1 and were compared to the "standard error band" obtained for the "D-test series" (Figure 28). The results indicated that the maximum torque measured at zero pore-water pressure was appreciably greater for the coarser materials (samples C_1 and M_1) than for the finer material (samples in the "D-test series"). The increase in maximum torque with a decrease in pore-water pressure appeared to be greater for the coarser silt (samples C_1 and M_1) than for the finer silt ("D-test series").

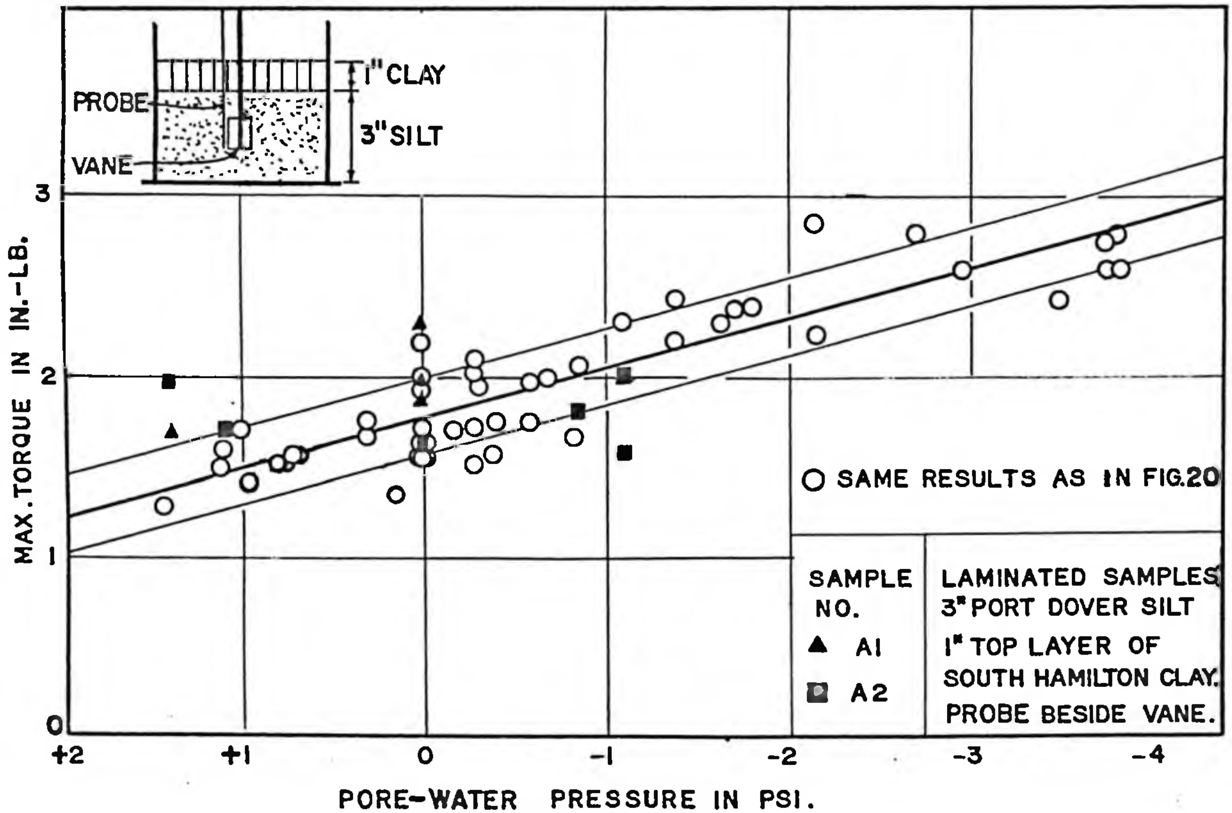


FIG.27

COMPARISON OF VANE TEST RESULTS OF LAMINATED SAMPLES WITH RESULTS IN FIG.20

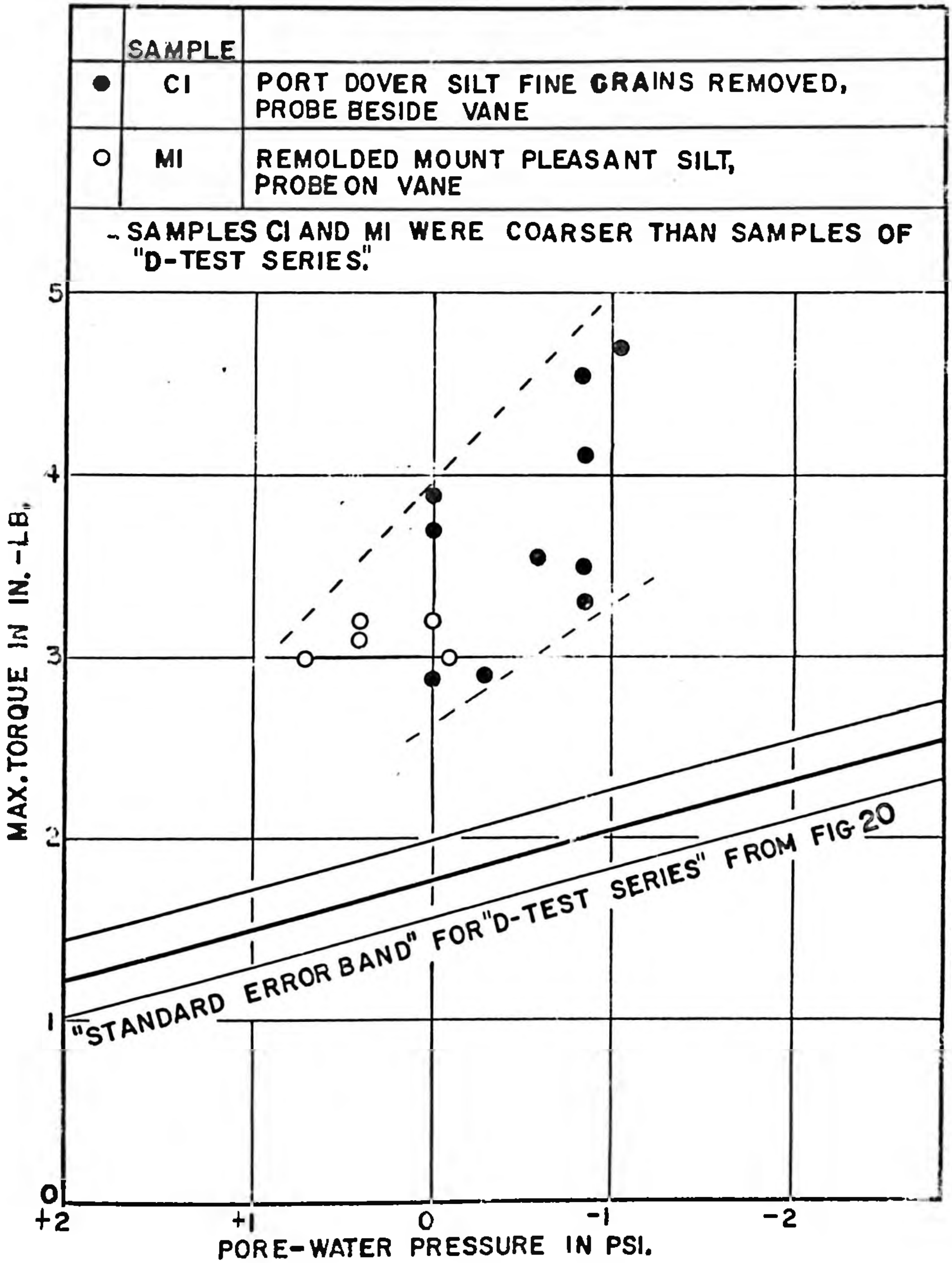


FIG. 28

VANE TESTS ON REMOLDED SILTS, MAX. TORQUE VS. PORE-WATER PRESSURE FOR SAMPLES CI AND MI

The variation of maximum torque with angular speed of the torque dial for sample M_1 (Figure 29) had the characteristics of variations observed in the case of the "D-test series" (Figure 23). The pore-water pressures of the 3 tests conducted on sample M_1 at an angular speed of the torque dial of one degree per second were not measured (Table VI a); the maximum torque values of the remaining tests conducted on sample M_1 were measured at pore-water pressures varying from -0.12 to $+0.7$ psi (Table VI). Therefore, variation in pore-water pressure could significantly affect the maximum torque/angular speed relationship in Figure 29.

4) Triaxial Tests on Remolded Silts

The triaxial samples of the "A- and D-test" series had a degree of saturation varying from 96 to 100.7 per cent. Back pressures were not used for fear of compression of the soil skeleton, which would change the dilatant character (expansion of the soil skeleton due to shear) of the material. Erroneous pore pressure readings, caused by small air bubbles in the probe, were unlikely at the small negative pore pressures encountered. The shear strength obtained from the triaxial test results was approximated by one-half the compressive strength, $(\sigma_1 - \sigma_3)_{max.}$ (Figure 31) measured at low effective minor principal stresses (0.1 psi $< \bar{\sigma}_3 < 0.4$ psi), as the true friction angle of the material was not determined. All triaxial tests presented in this research work were undrained ($\bar{Q}$) tests conducted in conjunction with pore-water pressure measurements with no confining pressure applied. The effective minor principal stress at failure was caused by negative pore-water pressures due to shear and the hydrostatic pressure (0.1 psi measured at mid-height) exerted on the sample by the

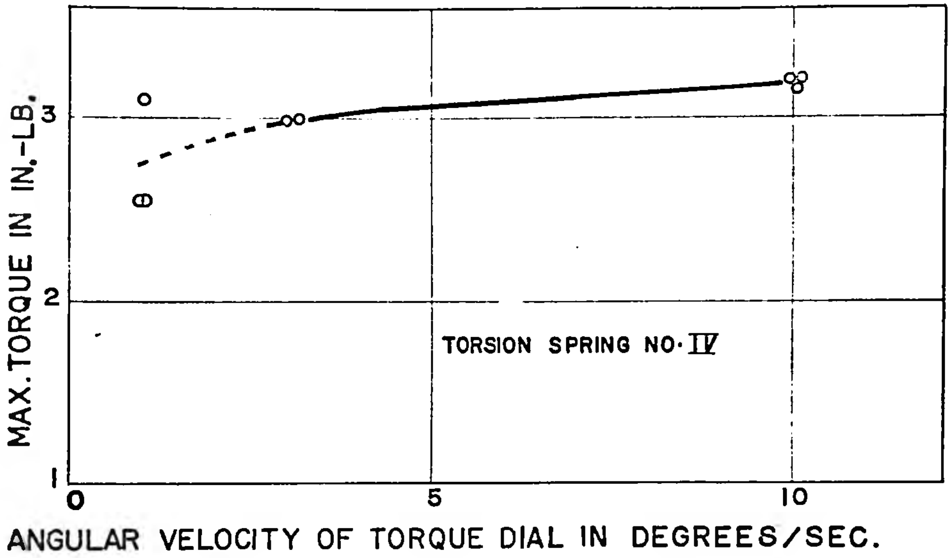


FIG.29

VANE TESTS ON REMOLDED MOUNT PLEASANT SILT.

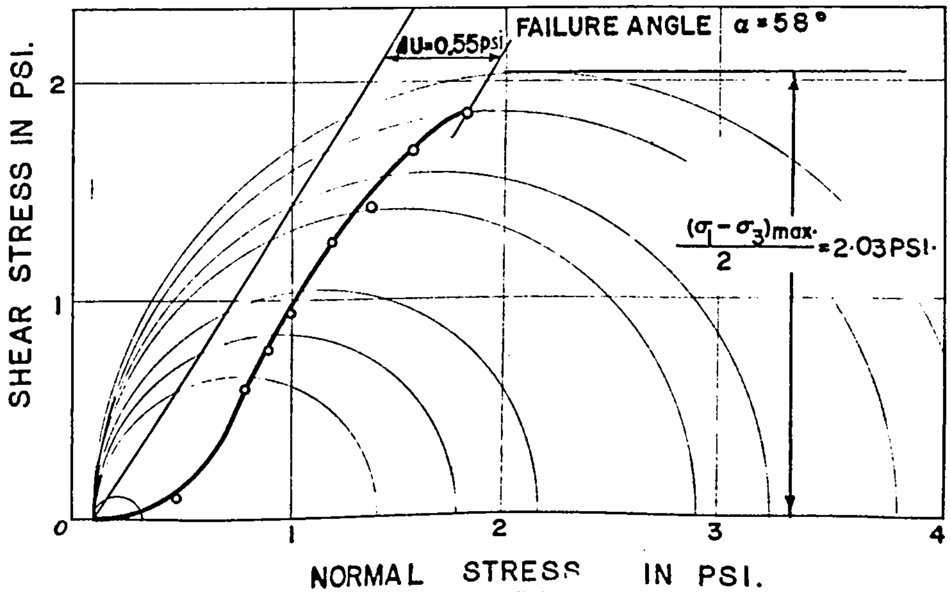


FIG.30

VECTOR CURVE FOR $\bar{Q}$ TEST ON REMOLDED MOUNT PLEASANT SILT.

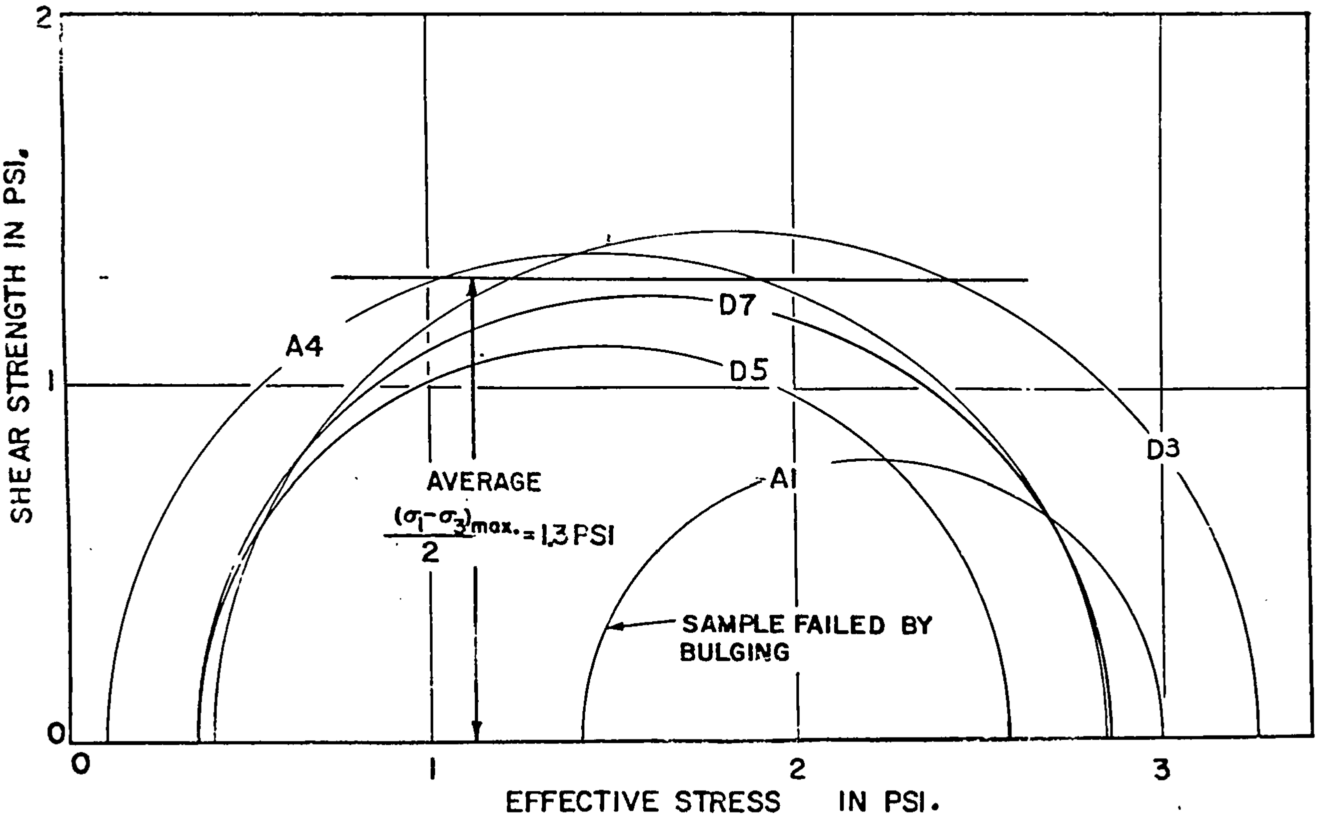


FIG. 31

MOHR'S STRENGTH CIRCLES FOR $\bar{Q}$ TESTS ON REMOLDED PORT DOVER SILT water in the traxial chamber. The shear strength approximated by one-half of the compressive strength was, therefore, only slightly in excess of one-half of the unconfined compressive strength. Pore pressure variations during undrained triaxial tests on Port Dover silt samples D_3 and D_5 are indicated in Figure 32 and on Mount Pleasant silt sample M_1 are indicated in Figure 30.

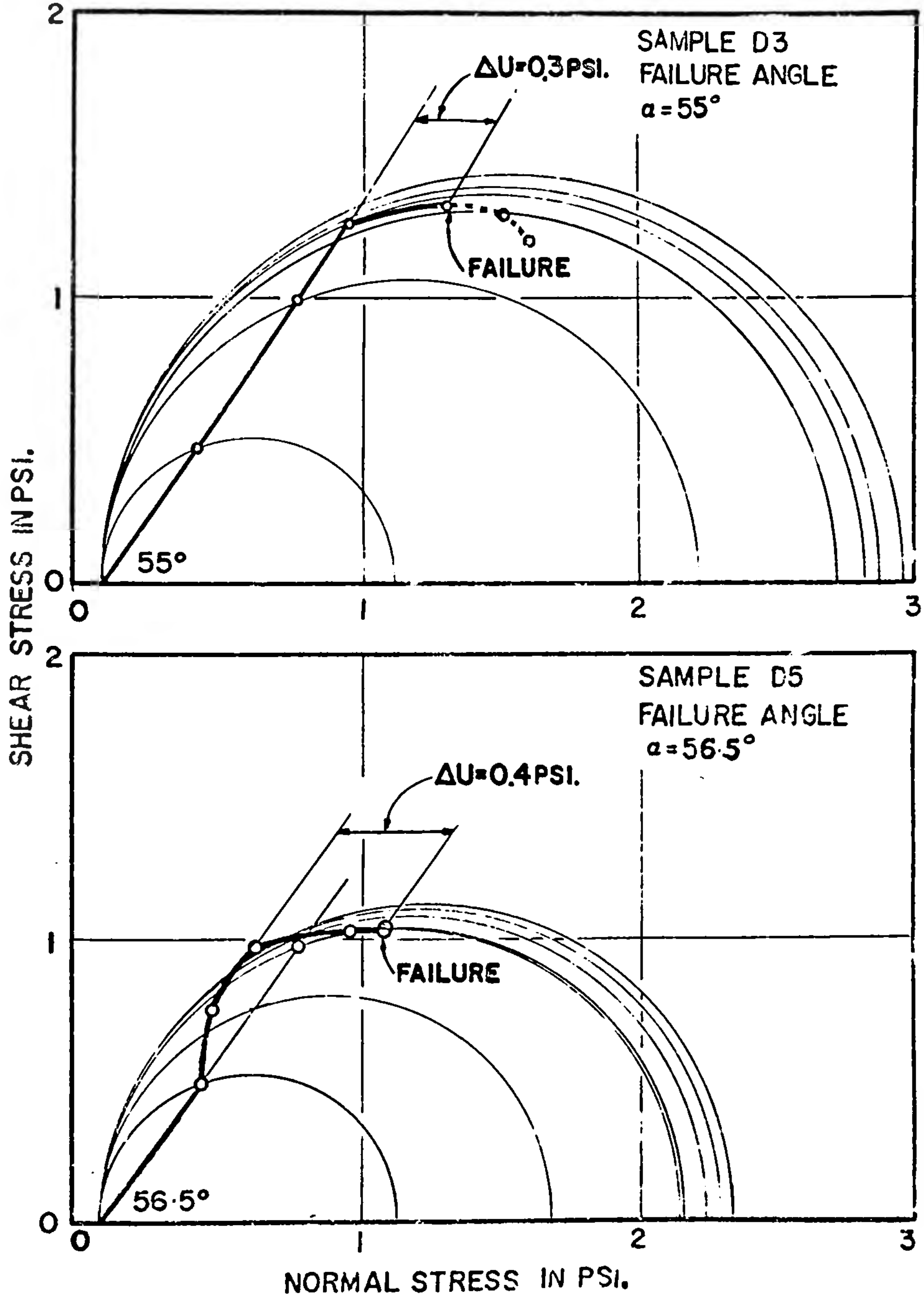


FIG. 32

VECTOR CURVES FOR $\bar{Q}$ TESTS ON REMOLDED PORT DOVER SILT

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Replicates of homogeneous and laminated silt samples prepared in the slurry compression cylinder showed only minor variations in water content, void ratio, degree of saturation and uniformity coefficient.

Consistent results, under the experimental conditions, were obtained with the technique developed to measure pore-water pressures on the outer edge of the vane blade.

The test data indicated that an increase in maximum torque could be correlated to a decrease in pore-water pressure. These results partially substantiate the theory presented by Golder (1960 and 1961) that with varved clays negative pore-water pressures could produce excessive shear strength values. The negative pore-water pressures actually generated by the vane in this study would not appear to cause significant variations in maximum torque values. Further study on generated pore pressures will be necessary to completely substantiate Golder's theory.

No significant dependency of maximum torque on the water content of the sample, the depth of insertion of the vane or the angular speed of the torque dial was observed over the ranges employed with homogeneous samples. Some evidence exists that further increases in the range of angular speed could affect the maximum torque values obtained.

The average of the shear strength, measured by the laboratory vane at zero excess pore-water pressure was significantly higher than the average of the shear strength obtained from triaxial tests.

When shear failure patterns were observed, the experimental results indicated that the basic assumptions used by Cadling and Odenstad (1950) to establish equation 5 were not applicable to the laboratory vane tests conducted in this study. Considerable deformation took place during a vane test, prior to the development of the visible shear failure surface, causing the silt to detach from the back of the vane blades thus creating a void. In addition, the maximum torque was observed during the deformation period. The visible shear failure surface in its initial development did not conform to a cylinder having the dimensions of the vane. Further work should be undertaken to investigate the basic assumptions reached by Cadling and Odenstad (1950).

The employment of the following apparatus is suggested to facilitate further work:

1)a strain controlled laboratory vane apparatus that records torque angle and angle of rotation of the vane throughout a test and has a controlled rate of rotation, and

2)a pore pressure apparatus that would provide instantaneous readings of pore-water pressures while minimizing the flow of pore-water necessary to activate the pore pressure apparatus.

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APPENDIX A

TABLE I

Summary of Properties of Soil Samples Prepared in the Slurry Consolidation Cylinder

1) Properties of Homogeneous Port Dover Silt Samples of the "D-Test Series".

2) Properties of the Port Dover Silt Layer of Laminated Samples of the "A-Test Series".

Sample No.	Water Content: Portion of Sample top o/o	mid. o/o	bot. o/o	average o/o	Void Ratio: Portion of Sample top	mid.	bot.	average	Saturation: Portion of Sample top o/o	mid. o/o	bot. o/o	average o/o
1) D_1				23.7				0.64				100.3
D_2				23.0				0.63				98.7
D_3				23.4				0.64				99.3
D_{5a}	22.9	23.5	22.8	23.1	0.632	0.647	0.623	0.63	98.6	98.7	99.8	99.0
D_{5b}	23.0	23.3	23.0	23.1	0.624	0.638	0.614	0.63	96.0	99.2	100.7	98.6
D_6	21.6	22.7	23.2	22.5	0.602	0.633	0.624	0.62	97.6	98.6	100.5	98.9
D_7			23.8	23.8			0.645	0.65			100.0	100.0
D_9	24.1	24.4	24.4	24.3	0.667	0.668	0.660	0.66	98.1	99.4	100.4	99.3
2) A_1				21.9				0.60				99.4
A_2				22.4				0.61				99.7
A_4				21.5				0.62				100.0

Readability for weight measurements ±0.0005 gm, for volume measurements ± 0.5 cc

TABLE II

Calibration of "No Flow" Pore Pressure Device for Volume Changes in Plastic Tubing Connecting Pore Pressure Instrument and Probe.

1) Plastic Tubing: Polyethylene, 2 ft. long, 1/8 in. O.D. 2) Plastic Tubing: Tygon, 5 ft. long, 1/8 in. O.D.

1) Scale Reading Kg/cm^2	1) Displacement of Right Branch of Null Indicator mm	2) Scale Reading Kg/cm^2	2) Displacement of Right Branch of Null Indicator mm
+0.5	+4.0	+1.4	+1.0
+0.4	+3.5	+1.2	+0.9
+0.3	+2.0	+1.0	+0.8
+0.2	+1.5	+0.6	+0.5
+0.1	+0.5	+0.4	+0.4
+0.0	0.0	+0.2	+0.2
-0.1	-0.1	0.0	0.0
-0.2	-0.8	-0.1	-0.1
-0.3	-1.5	-0.2	-0.3
-0.4	-2.0	-0.3	-0.5
-0.5	-3.0	-0.4	-0.7
		-0.5	-1.1

Displacements caused by negative scale readings are downwards and are indicated by a negative sign.

TABLE III

Summary of Vane Tests Conducted on Homogeneous Fort Dover Silt Samples of the "D-Test Series."

Vane Dimensions 1 inch x 3/4 inch diameter. Torsion spring No. IV (0.044 in-lb/deg). Probe mounted on the vane (Figure 15 b)

Sample No.	Test No.	Depth of Insertion of Vane	Ang. Speed of Torque Dial	Max. Torque	Pore-Water Pressure			
					After Insertion of Vane	After Waiting Period	Applied to Sample	At Failure
		in	deg/sec	in-lb	psi	psi	psi	psi
D_1	1	3	4	1.75	+1.0	not recorded		-0.6
	2	3	4	1.75	-0.15	not recorded		-0.4
	3	3	4	1.55	0.0	not recorded		0.0
	4	3	4	1.55	-0.15	not recorded		-0.4
	5	2	4	1.65	0.0	not recorded		-0.85
	6	2	4	1.55	0.0	not recorded		0.0
	7	3	4	1.95	0.0	not recorded		0.0
D_2	1	2	4	2.1	-0.6	not recorded		-0.85
	2	2	4	1.95	+1.4	not recorded		-0.3
	3	3	4	2.2	+0.15	not recorded		0.0
	4	3	4	1.7	+0.15	not recorded		-0.15

Sample No.	Test No.	Depth of Insertion of Vane	Ang. Speed of Torque Dial	Max. Torque
		in	deg/sec	in-lb
D_3	1	3	4	2.8
	2	3	4	2.4
	3	3	4	2.6
	4	3	4	2.85
	5	3	4	2.0
D_5	1	2	4	2.1
	2	2	4	2.0
	3	2	4	2.75
	4	2	4	2.25
	5	2	4	2.6
	6	3	4	2.6
	7	3	4	2.3
	8	3	4	2.3
	9	3	4	2.45
	10	3	4	2.0
	11	3	4	1.75

TABLE III (continued)

Pore-Water Pressure			
After Insertion of Vane psi	After Waiting Period psi	Applied to Sample psi	At Failure psi
		-3.15	-2.7
		-1.4	-1.8
		-3.7	-2.95
		-2.8	-2.15
		-1.4	-0.7
+0.15	-0.3		-0.3
+0.8	-0.3		-0.3
		-4.2	-3.75
		-2.6	-2.15
		-3.8	-3.8
		-4.2	-3.8
		-1.7	-1.65
		-1.1	-1.1
		-1.7	-1.4
+1.2	0.0		0.0
+1.0	0.0		+0.3

TABLE III (continued)

Sample No.	Test No.	Depth of Insertion of Vane in	Ang. Speed of Torque Dial deg/sec	Max. Torque in-lb	Pore-Water Pressure: After Insertion of Vane psi	After Waiting Period psi	Applied to Sample psi	At Failure psi
D6	1	2	3	1.5	+1.2	0.0		+1.1
	2	2	3	1.7	+1.2	0.0		−0.3
	3	2	3	1.7	+1.2	0.0		0.0
	4	2	3	2.8			−2.9	−3.8
	5	3	3	2.45			−4.3	−3.5
	6	3	3	2.2			−1.7	−1.4
	7	3	3	1.95	+0.6	−0.1		−0.6
	8	3	3	2.35	+0.6	0.0		−1.7
	9	3	3	1.6	0.0	−0.1		0.0
	10	3	3	1.5	+0.3	0.0		−0.3
	11	3	3	1.55	0.0	0.0		0.0
D7	1	3	3	1.65	+0.6	0.0		0.0
	2	3	3	1.55	+0.7	+0.1		+0.7
	3	2	3	1.65	−0.3	No waiting period		+0.3

TABLE III (continued)

Sample No.	Test No.	Depth of Insertion of Vane in	Ang. Speed of Torque Dial deg/sec	Max. Torque in-lb	Pore-Water Pressure: After Insertion of Vane psi	Pore-Water Pressure: After Waiting Period psi	Pore-Water Pressure: Applied to Sample psi	Pore-Water Pressure: At Failure psi
D_7	4	1	3	1.1	0.0	No waiting period		+0.3
	5	2	3	1.35	0.0	No waiting period		+0.15
D_9	1	3	3	1.7	+0.7	No waiting period		+1.0
	2	3	3	1.6	+0.85	No waiting period		+1.1
	3	3	3	1.3	+1.25	No waiting period		+1.45
	4	3	3	1.55	+0.6	No waiting period		+0.7
	5	3	approx. 1	1.5	+0.7	No waiting period		+0.85
	6	3	approx. 1	1.5	+1.1	No waiting period		+0.8
	7	3	approx. 1	1.45	+1.45	No waiting period		+0.95

Remarks to Table III:

The first test on each sample was conducted in the centre of the container, (the lower part of the slurry consolidation cylinder). The pore-water pressure applied to a sample does not represent the pore-water pressure before torque application. Results indicated for test No. 5, 6 and 7 of sample D_9 were taken from Table IIIa; the pore-water pressure at failure for these tests was assumed to correspond to the final pore-water pressure measurement of each test. Cadlings equation 5 , $T = S(\pi h \frac{d^2}{2} + \pi \frac{d^3}{6})$ yields T = 1.1 S for vane dimensions 1 inch x 3/4 inch diameter.

TABLE IIIa

Summary of Vane Tests No. 5, 6 and 7 of Port Dover Silt Sample D9.

Torque and pore-water pressure readings were taken in stages of 5 degrees of rotation of the vane, duration of each stage was about 15 seconds. This test procedure would correspond to an average angular speed of the vane of about 0.3 deg/sec and an average angular speed of the torque dial of about 1 deg/sec.

Test No.	Degrees of Rotation of the Vane	Torque in-lb	Pore-Water Pressure psi
5	0	0.0	+0.7
	5	0.9	+0.85
	10	1.3	+1.1
	15	1.5	+1.25
	20	1.4	+1.25
	25	1.4	+1.1
	30	1.4	+1.0
	35	1.4	+1.0
	40	1.4	+0.85
6	0	0.0	+1.1
	5	1.1	+1.1
	10	1.5	+1.5
	15	1.4	+1.4
	20	1.4	+1.3
	25	1.4	+0.95
	30	1.4	+0.95
	35	1.4	+0.95
	40	1.4	+0.8

TABLE IIIa (continued)

Test No.	Degrees of Rotation of the Vane	Torque in-lb	Pore-Water Pressure psi
7	0	0.0	+1.45
	5	1.2	+1.5
	10	1.45	+1.6
	15	1.4	+1.6
	20	1.4	+1.35
	25	1.4	+1.35
	30	1.4	+1.1
	35	1.4	+0.95

TABLE IV

Summary of Vane Tests Conducted on the Port Dover Silt Layer of Laminated Samples of the "A-Test Series".

Vane Dimensions 1 inch x 3/4 inch diameter.

Torsion spring No. III (0.125 in-lb/deg).

Torsion spring No. IV (0.044 in-lb/deg).

Probe inserted beside the vane (Figure 15a).

The laminated samples consisted of a 3 inch layer of remolded Port Dover silt and a 1 inch top layer of remolded South Hamilton clay.

Sample No.	Test No.	Torsion Spring No.	Depth of Insertion of Vane	Ang. Speed of Torque Dial	Max. Torque	Pore-Water Pressure at Failure	Degrees of Rotation of Vane at Failure
			in	deg/sec	in-lb	psi	deg
A_1	1	III	3	10	4.0	+0.3	
	2	III	3	10	4.0	-0.3	
	3	IV	3	10	2.3	0.0	22
	4	IV	3	10	1.65	+1.4	21
	5	IV	3	10	2.0	0.0	25
	6	IV	3	10	1.9	0.0	18
A_2	1	IV	3	10	1.95	+1.4	14
	2	IV	3	10	1.65	0.0	16
	3	IV	3	10	1.7	+1.1	18
	4	IV	3	10	1.8	-0.85	19
	5	IV	3	10	1.45	-1.1	21
	6	IV	3	10	2.0	-1.1	22
A_4	1	IV	3	15	2.65	-0.85	15
	2	IV	3	15	2.65	-0.15	18
	3	IV	3	15	2.45	-0.0	14
	4	IV	3	15	2.2	-0.3	16

TABLE V

Summary of Vane Tests Conducted on Homogeneous Silt Sample C_1

The silt was Port Dover silt with the fine grains removed (Figure 9).

Vane dimensions 1 inch x 3/4 inch diameter.

Torsion spring No. IV (0.044 in-lb/deg).

Probe inserted beside the vane (Figure 15a).

Angular speed of the torque dial 15 deg/sec.

Water content 24.3 per cent.

Test No.	Depth of Insertion of Vane in	Max. Torque in-lb	Pore-Water Pressure After Insertion psi	Pore-Water Pressure At Failure psi
1	2	3.55	-0.3	-0.6
2	2	3.9	+0.8	0.0
3	2	3.7	-0.8	0.0
4	2	4.55	0.0	-0.85
5	3	4.7	0.0	-1.05
6	3	3.5	0.0	-0.85
7	3	2.9	-0.85	-0.3
8	3	2.9	0.0	0.0
9	3	3.3	-1.1	-0.85
10	3	4.1	-0.3	-0.85

TABLE VI

Summary of Vane Tests No. 1 to 5 Conducted on Homogeneous Mount Pleasant Silt Sample M_1.

Vane dimensions 1 inch x 3/4 inch in diameter.

Torsion spring No. IV (0.044 in-lb/deg).

Probe mounted on the vane (Figure 15b).

Water content 30.8 o/o.

Test No.	Depth of Vane	Max. Torque	Pore-Water Pressure at failure	Angular Speed of Torque Dial
	in	in-lb	psi	deg/sec
1	3.5	3.0	-0.1	4
2	2	3.2	+0.4	10
3	2	3.1	+0.4	10
4	2	3.0	+0.7	4
5	2	3.2	0.0	10

TABLE VIa

Summary of Vane Tests No. 6, 7, and 8 Conducted on Homogeneous Mount Pleasant Silt Sample M_1.

Torque readings were recorded in stages of 5 degrees of rotation of the vane; duration of each step was about 15 seconds. This test procedure would correspond to an average angular speed of the torque dial of about 1 degree/second. Pore-water pressures were not recorded. Depth of insertion of vane was 3.5 inches.

Degrees of Rotation of Vane	Test No. 6 Torque in-lb	Test No. 7 Torque in-lb	Test No. 8 Torque in-lb
0	0.0	0.0	0.0
5	1.5	1.7	1.6
10	2.2	2.3	2.0
15	2.5 (max.)	2.5 (max.)	2.2
20	2.3	2.2	3.1 (max.)
25	2.2	2.1	2.1
30	2.0		2.1
35	1.7		1.9
40	1.6		

TABLE VII

Summary of Undrained Triaxial ($\bar{Q}$) Tests on Fort Dover Silt with Strain Controlled Loading

Triaxial samples (2.8 inches x 1.4 inches diameter) were obtained from silt samples (about 5 inches x 6 inches diameter) of the "D-Test series" and from the silt layer (about 3 inches high) of Laminated samples of the "A-Test series."

Sample No.	Test No.	Confining Pressure	Time of Loading	Compressive Strength $(\sigma_1-\sigma_3)_{max.}$	Pore-Water Pressure at $(\sigma_1-\sigma_3)_{max.}$	Effective Stress at $(\sigma_1-\sigma_3)_{max.}$ $\bar{\sigma}_3$	$\bar{\sigma}_1$	Failure Angle α
		psi	min	psi	psi	psi	psi	degrees
D_3	1	0.1	5.0	2.86	-0.3	0.4	3.26	55°
D_5	1	0.1	7.5	2.23	-0.25	0.35	2.58	55° to 58°
D_7	1	0.1	9.0	2.51	-0.25	0.35	2.86	52°
$(A_1$	1	0.1	10.0	1.41	-1.5	1.6	3.01)	Sample failed by bulging
A_4	1	0.1	8.0	2.75	0.0	0.1	2.85	55°

The average failure angle α was 54.6°. The average true angle of internal friction $\bar{\phi} = 2(\alpha - 45°) = 19.2°$ (Bjerrum, 1954). The average of one half of the compressive strength of samples D_5, D_6, D_7 and A_4 was 1.3 psi. Sample A_1 was excluded from all analyses.

TABLE VIIa

Loading Data of Undrained Triaxial ($\bar{Q}$) Tests Conducted on Sample D_5

Test No. 1 Date: June 25, 1962.

Confining pressure 0.1 psi (hydrostatic pressure of water in triaxial cell)

Time p m	Elapsed Time min	Load lb	Deformation in	Strain o/o	Corrected Area in^2	Total Stress σ_1 psi	Pore-Water Pressure psi	
12:40	0.0	0.0	0.000	0.0	1.54	0.00	0.0	
	0.5	1.6	0.055	1.6	1.57	1.12	0.0	
	1.5	2.6	0.172	6.2	1.64	1.68	+0.15	
	2.5	3.5	0.234	10.3	1.71	2.14	+0.15	
	3.5	3.7	0.400	14.5	1.80	2.15	0.0	
	4.5	4.1	0.520	18.8	1.90	2.25	-0.15	
	5.5	4.4	0.640	23.2	2.00	2.30	-0.25	
	6.5	4.7	0.745	27.0	2.10	2.33	-0.25	max. deviator stress
	7.5	4.9	0.872	31.7	2.20	2.32	-0.25	

TABLE VIIb

Loading Data of Undrained Triaxial ($\bar{Q}$) Tests Conducted on Sample D_3

Test No. 1 Date: June 22, 1962.

Confining pressure 0.1 psi (hydrostatic pressure of water in triaxial cell)

Time p m	Elapsed Time min	Load lb	Deformation in	Strain o/o	Corrected Area in^2	Total Stress σ_1 psi	Pore-Water Pressure psi	
3:20	0.00	0.00	0.000	0.0	1.54	0.00	0.0	
3:20	0.25	1.60	0.040	1.5	1.56	1.10	0.0	
3:20	0.50	3.40	0.115	4.2	1.61	2.21	0.0	
3:21	1.00	4.50	0.191	6.9	1.65	2.82	0.0	
3:21	1.50	4.90	0.270	9.8	1.71	2.96	-0.3	max. deviator stress
3:22	2.00	4.95	0.370	13.4	1.78	2.88	-0.55	
3:23	3.00	5.00	0.540	19.6	1.91	2.71	-0.7	
3:24	4.00	5.20	0.730	26.5	2.09	2.59	-0.7	
3:25	5.00	5.60	0.917	33.3	2.30	2.54	-0.7	

TABLE VIII

Loading Data of Undrained Triaxial ($\bar{Q}$) Test Conducted on Sample M_1

Test No. 1 Date: July 4, 1962.

Confining pressure 0.1 psi (hydrostatic pressure of water in triaxial cell)

Time p m	Elapsed Time min	Load lb	Deformation in	Corrected Area in^2	Total Stress σ_1 psi	Pore-Water Pressure psi
					0.00	0.0
					0.30	-0.3
					1.40	-0.3
					1.78	-0.3
					2.14	-0.3
					2.90	-0.3
					3.25	-0.38
					3.80	-0.4
					4.15	-0.55
					4.00	-0.6