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THE ANGLE OF SHEARING RESISTANCE IN COHESIVE SOILS FOR TESTS AT CONSTANT WATER CONTENT

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One very simple method of analysing stability problems in cohesive soils, which is much used in England, is that known as the $\phi = 0$ method. 1) In this method it is assumed that under conditions of no water content change, the soil shows an angle of shearing resistance ϕ equal to zero. However, it is known that tests do not always show $\phi = 0$ and the choice of the values of the shear characteristics to be used in the analysis is usually the most difficult part of the problem. The authors have examined several hundred sets of test results on very diverse soils and have found that they can be conveniently classified into groups which are given below.

The tests were all immediate triaxial compression tests. The technique employed in England in this test is different from that in use in some other countries, although it corresponds to the "quick" triaxial test as used in the U.S.A. 2) The sample of soil is a cylinder 1½" in diameter and 3¼" in length. It is surrounded by a rubber membrane 0.01" thick which is sealed to solid metal end plates in contact with the ends of the sample. The amount of water inside the membrane cannot change during test. Thus the mean water content of the sample must remain constant, although migration of water within the sample is not prevented by the conditions of the test. The sample, sealed inside its rubber membrane and end plates, is then placed in a cell in which it is surrounded by water under pressure. This fluid pressure acts all round and on the ends of the sample. The vertical pressure is then increased by means of a piston, until failure occurs. During the test the sample is completely isolated and no measurements of volume change or internal pore-water pressures are made. The rate of strain during test is about 1 per cent per minute and failure is reached, with most soils after about 10 minutes.

The test results fall into the following groups:-

I. Soft fully-saturated clays
II. Stiff fully-saturated clays
III. Fully-saturated remoulded clays
IV. Partially saturated undisturbed soils
V. Partially saturated remoulded soils
VI. Clay shales & siltstones
VII. Saturated undisturbed silts.

I. SOFT FULLY-SATURATED CLAYS $\phi = 0$.

Many examples of this type of material were met with. Most of them were soft recent alluvial clays in some cases containing peat. They were nearly all completely saturated and all had liquid limits greater than about 38%. In all cases ϕ was equal to zero, although cases occurred where variation in strength of the material from one test to another resulted in an unusually large or small Mohr circle under one lateral pressure. Seven examples are given in Table 1. Example b may be only 98% saturated and may exhibit a small ϕ of 1°. Example f is London clay (Eocene) softened by weathering near the surface, all the others are recent marine or river alluvium. The Mohr circles for example c are shown in Fig. I.

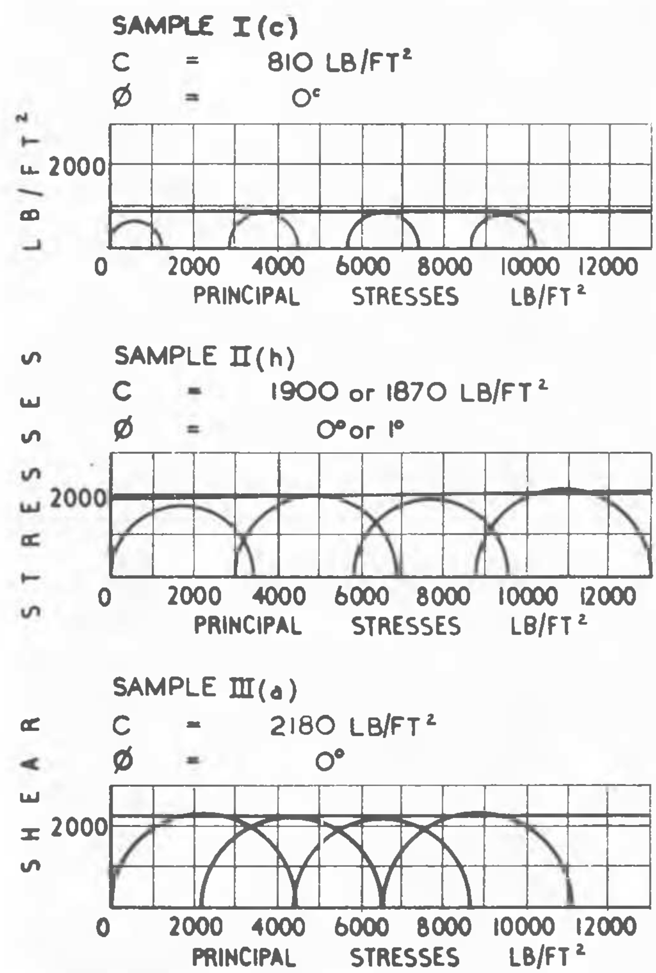

FIG.1.2.3

II. STIFF FULLY-SATURATED CLAYS $\phi = 0$.

Many stiff clays occur in England, the most important being the stiff fissured London clay. A large number of test results were available on this material, and these establish without doubt that although a considerable variation may occur between single results, the mean of a large number of tests is the same whatever the lateral pressure. The strength under zero lateral pressure is, however, consistently lower by about 20% than that under any other pressure. This is attributed to the effect of the fissures and does not occur with the same material when remoulded. Other stiff clays also occur, some of which are not fissured, and eight examples of different types are given in Table II. All the samples are fully or almost fully saturated and in each case $\phi = 0$ or very slightly more, but never exceeds 2°. The liquid limits range from 34 to 111 but all the samples with liquid limits below 40 are boulder clays of glacial origin.

III. FULLY SATURATED REMOULDED CLAYS $\phi = 0$.

When testing clays containing stones, the authors have sometimes found it necessary to remove the stones and remould the clay before testing. This procedure is permissible in soils such as boulder clays which have little structure, and also in the case of material which is to be used, or has been used as fill. By very heavy compaction it is possible to reduce the air voids practically to zero in which case ϕ is found to be zero. Four examples are given in Table III Example d has a liquid limit of 26% and is only 95% saturated and there are indications of a small ϕ of ½°.

IV. PARTIALLY SATURATED UNDISTURBED SOILS $\phi > 0$.

Clay soils which contain some air, normally exhibit a ϕ in immediate triaxial tests. Four examples of tests on undisturbed samples of soil are given in Table IV and example c is plotted in Fig. 4.

V. PARTIALLY SATURATED REMOULDED SOILS $\phi > 0$.

Remoulded soils which contain air also exhibit a ϕ. Five examples of different soils are given in Table V. Examples d and e are taken from a series of tests in which the soil was compacted in a standard manner at different moisture contents and then tested. Fig. 5 shows the values obtained for c and ϕ at different moisture contents. The density curve is also given.

TABLE I

SOFT FULLY SATURATED CLAYS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture Content %	σ_{III} lb/in^2	$\sigma_I-\sigma_{III}$ lb/in^2	C lb/in^2	ϕ°
a) Gosport.[8] Very soft recent marine clay. Normally consolidated.	100	82	29	58	0 20 40	2.90 2.94 2.88	210	0
b) Fens.[9] Soft lagoon silty clay. Normally consolidated	98 to 100	46	25	41	0 15 30 45	6.8 6.8 7.2 8.1	500	1°
c) Neath.[10] Soft recent marine clay. Normally consolidated.	97 to 100	168 (peat in sample)	60	105	0 20 40 60	9.5 12.0 12.2 11.0	810	0
d) Clyde Estuary.[10] Soft recent silty clay. Normally consolidated	100	56	22	46	0 20 40 60	2.5 4.5 3.4 3.0	250	0
e) Avonmouth.[10] Soft recent silty clay. Normally consolidated.	96 to 100	60	23	45	0 30 60	7.8 7.8 7.8	560	0
f) Wembley.[10] Soft weathered London clay. Eocene. Over-consolidated.	100	59	19	29	0 20 40 60	11.5 12.5 12.0 11.0	850	0
g) Newport, Mon.[10] Soft intact grey clay. Recent river alluvium.	100	61	24	44	0 30 60	3.2 3.2 4.2	250 or 230	0 ½°

TABLE II

STIFF FULLY SATURATED CLAYS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture Content %	σ_{III} lb/in^2	$\sigma_I-\sigma_{III}$ lb/in^2	c lb/ft^2	ϕ
a) Durban.[11] Firm estuarine clay. Highly over-consolidated	100	111	28	58	0 15 30 45	15.7 16.8 15.9 17.2	1,180 or 1,150	0 1°
b) Peterborough[12] Very stiff laminated Oxford clay. Intensely over-consolidated.	100	87	30	21	0 30 60	93 86 86	6,300	0
c) Cambridge,[10] Stiff fissured clay. Gault. Over-consolidated	98	74	29	30	0 30 60	25.3 27.5 28.1	1,950	0 or 1°
d) Conisborough.[10] Stiff weathered clay.	100	42	14	13	0 30 60	34.7 34.7 34.8	2,500	0
e) Manchester.[10] Stiff sandy boulder clay.	100	34	14	16	0 30 60	33.6 32.9 36.0	2,450	1°
f) Wrexham.[12] Stiff laminated clay. Pleistocene. Normally consolidated.	100	36	18	28	0 20 40 60	28.0 33.0 27.0 29.0	2,100	0
g) Walton.[13] Stiff fissured London clay. Eocene. Over-consolidated. Mean of 50 tests at each pressure.	100	70	25	27	0 20 40 60	26.4 31.7 32.9 34.1	2,250 or 2,160	0 2
h) Leyton.[10] Stiff fissured. London clay. Eocene. Over-consolidated. Single tests.	97 to 98	66	20	27	0 20 40 60	23.9 27.1 25.6 29.1	1,900 or 1,870	0 1°

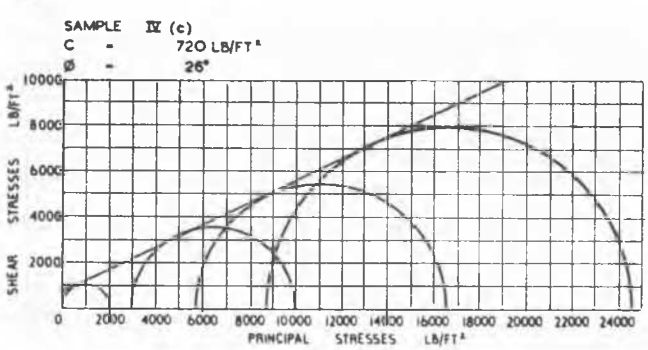

FIG.4

VI. CLAY-SHALES & SILTSTONES $\phi > 0$.

It has been shown by Terzaghi 3) that the area of contact between the grains in most soils is very small (less than 1%), but it is probable that in materials with very low porosities, such as slates the area of contact is appreciable. Between these types are shales and clay-shales in which it is possible that some small area of contact occurs. If so, these soils would be expected to show a ϕ in immediate triaxial tests. The seven examples given in Table VI all show a ϕ. Some of these soils do not appear to be saturated although

TABLE III

FULLY SATURATED REMOULDED CLAYS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture Content %	σ_{III} lb/in^2	$\sigma_I - \sigma_{III}$ lb/in^2	c lb/ft^2	$\phi°$
a) Muirhead.[12] Boulder clay. Optimum moisture content 16.5%	100	40	17	17	0 15 30 45	30,3 29.9 29.7 31.3	2,180	0
				21	0 15 30 45	10.7 11.8 10.6 10.3	780	0
b) Hyde Park.[13] Brown London clay.Mean of 10 tests at each pressure.	100	82	23	33	0 30 60	26.9 28.3 27.6	2,000	0
c) Ireland.[10] Gravel, sand and clay. Optimum moisture content 16%	100	42	20	22	0 20 40 60	5.8 - 5.0 5.0	380	0
d) Ireland.[10] Gravel sand & clay. Optimum moisture content 10%	95	26	11	15	0 20 40 60	10.3 14.5 15.5 15.5	1,050	0 or ½

TABLE IV

PARTIALLY SATURATED UNDISTURBED SOILS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture Content %	σ_{III} lb/in^2	$\sigma_I - \sigma_{III}$ lb/in^2	c lb/ft^2	$\phi°$
a) Blue Mountain[14] Derm, U.S.A. Reddish brown mottled clay.	87	30	19	19	0 14 42 125	20 58 91 234	820	28
b) Soulthorpe.[10] Brown fine sand with some silt and clay.	72	18	11	13	0 20 40 60	14.1 44,1 65.2 87.0	1,570	19
c) Prestwick.[10] Soft brown sandy clay.	90	65	17	25	0 20 40 60	14,5 48.7 74.9 110.0	720	26
d) Conisborough.[10] Firm clay and weathered siltstone.	80	36	20	12	0 30 60	33.5 58.5 91.6	1,700	20

TABLE V

PARTIALLY SATURATED REMOULDED SOILS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture content %	σ_{III} lb/in^2	$\sigma_I - \sigma_{III}$ lb/in^2	c lb/in^2	$\phi°$
[10] a) Brynmawr. Boulder clay.	61	c35	c23	15	0 30 60	71.5 88.0 97.0	4,400	10
[10] b) Newport, Mon. Devonian Marl.	92	33	20	18	0 20 40 60	75.0 85.0 110.0 120.0	4,200	17
[10] c) Newport, Mon. Weathered clayey Devonian Marl.	92	56	20	22	0 20 40 60	34.3 75.0 90.0 115.0	1,700	24
[10] d) Ireland. Gravel, sand and clay.	46	42	20	10	0 20 40 60	85 123 164 203	3,500	30
[10] e) Ireland, Gravel, sand and clay.	37	27	12	6	0 20 40 60	78 124 175 225	3,000	33

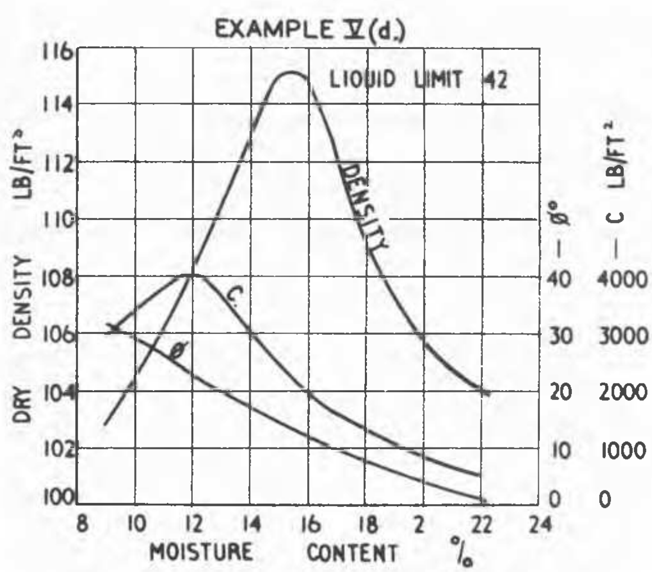

FIG.5a

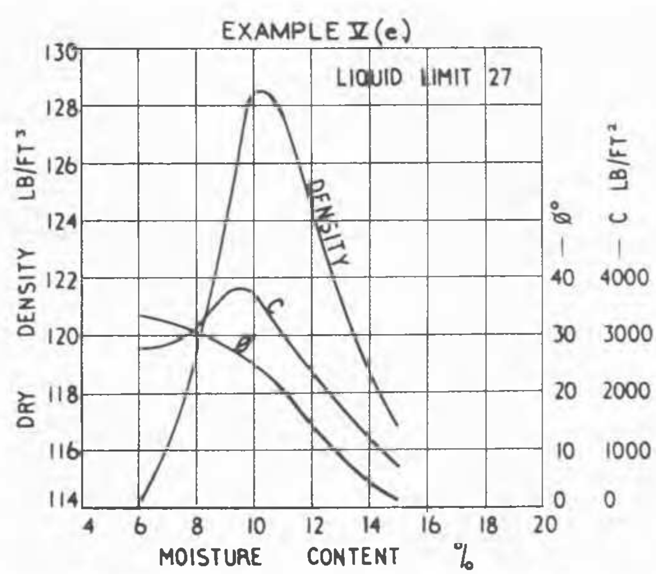

FIG.5b

they occur well below ground water level and in these cases the ϕ may of course be due to an air content. However, three of them are definitely saturated. The liquid limits of all these soils are about 30 - 35.

VII. SATURATED UNDISTURBED SILTS $\phi > 0$.

The last group is most interesting. A few silts have been tested which give a ϕ even though saturated. Not many of these materials have been found and some results have had to be rejected since the water contents were lower in the stronger samples, although the authors are of the opinion that this did not account completely for the increase in strength. Five examples are given in Table VII. Sample a is a recent marine alluvium. Samples, b, c and d are Eocene silts taken from a depth of about 40 - 50 feet on the same site but not at the same time. They are permiable and definitely frictional. Sample e was taken from a Pleistocene deposit formed in a glacial lake. The soil becomes more silty with increasing depth and samples above sample e showed no ϕ but had a higher liquid limit. A typical test result is given in Fig. 7.

TABLE VI

CLAY SHALES AND SILTSTONES

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture content %	σ_{III} lb/in^2	$\sigma_I - \sigma_{III}$ lb/in^2	c lb/ft^2	$\phi°$
10 a) Portishead. Stiff red clay shale Kemper Marl.	81	41	27	20	0 30 60	30 71 110	1,440	23
10 b) Portishead.Stiff red & green clay shale, Kemper Marl.	100	29	23	19	0 20 40 60	38 69 109 133	1,600	27
10 c) Newport, Mon. Hard fissured red siltstone. Devonian.	91	30	17	9	0 20 40 60	23.0 49.7 90.0 125.0	900	28
10 d) Newport, Mon. Firm red weathered siltstone. Devonian.	100	32	16	10	0 20 40 60	24.5 47.2 55.0 57.5	1,440	16
10 e) Severn Estuary. Firm red clay shale. Kemper Marl.	100	30	17	15	0 20 40 60	41.5 62.0 78.0 85.5	2,300	16
10 f) Waun-y-Gilfach. Laminated grey clay shale. Coal measured.	86	Not	known	9	0 20 40 60	17.6 63.1 90.7 121.5	1,600	25
10 g) Dormanstown. Hard grey shale	90	c36	c22	18	0 40 60	13.5 60.5 86.5	700	22

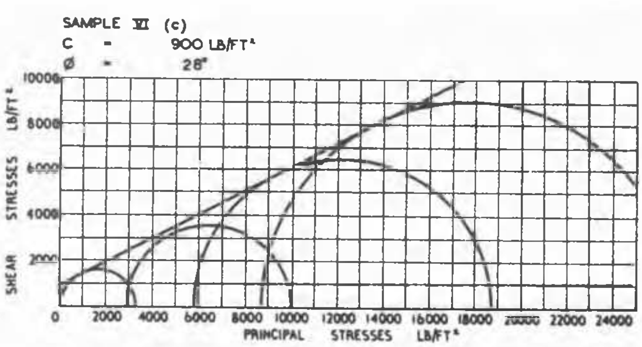

FIG.6

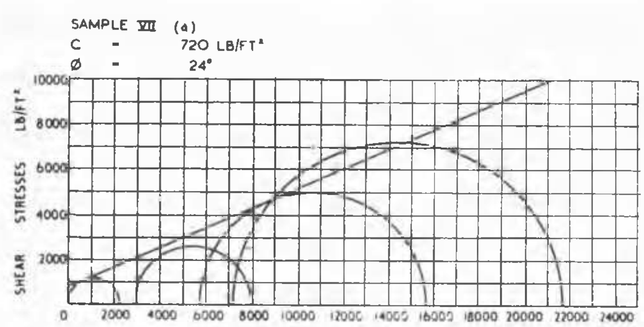

FIG.7

DISCUSSION.

The results quoted in this paper were obtained in the course of routine tests on a large number of samples tested for the analysis of very diverse problems. The classification suggested in this paper was not envisaged when the tests were carried out and in some cases the lack of some piece of information or doubt about a test result has forced the rejection of several examples which could have been included had check tests been carried out at the time of test.

With saturated clays, whether soft or stiff, undisturbed or remoulded, sufficient tests have been done to prove that there is no increase in shear strength for the conditions of this test. This result is confirm-

TABLE VII

SATURATED UNDISTURBED SILTS

Description	Saturation %	Liquid Limit %	Plastic Limit %	Moisture content %	σ_{III} lb/in^2	$\sigma_I - \sigma_{III}$ lb/in^2	c lb/ft^2	$\phi°$
a) Clyde Estuary.[10] Compact slightly clayey silt. Recent.	100	31	21	24	0	16.0	720	24
					20	35.5		
					40	68.8		
					60	100.0		
b) Poole.[10] Compact grey-white silt. Eocene.	100	23	19	21	0	13.0	450	38
					20	115.0		
					40	110.0		
					60	200.0		
c) Poole.[10] Firm silty-clay.	100	32	15	19	0	19.1	750	34
					30	81.4		
					60	185.0		
d) Poole.[10] Firm very silty clay.	100	30 to 35	12 to 15	15	0	77.5	4,040	18
					20	97.4		
					40	110.5		
					60	129.0		
e) Wrexham.[10] Firm silty clay. Pleistocene.	100	23	15	16	0	18.8	1,000	19
					20	38.3		
					40	34.8		
					60	75.0		

ed by the results of a large number of tests carried out in the U.S.A. under similar conditions. 2) This result presumably means that the effective pressure does not increase with the lateral pressure, as shown by Terzaghi, 3) 4) Rendulic 5) and Jürgenson. 6)

It is significant that no soil in Table I has a liquid limit lower than 46%. A careful search through several hundred test results showed that no soil which would fall into Group I had a liquid limit less than about 38%. In Group II the only soils with liquid limits below 40% were boulder clays or were of glacial origin i.e. soils in which the clay fraction was not very active being composed of mechanically comminated rock flour. Cooling 7) has shown that the particle size distribution of these soils is likely to be the same as that of a "normal" clay soil with a higher liquid limit.

In Group VII the number of tests is small but ϕ is definitely greater than zero. No sample in this Group had a liquid limit greater than 35%. The authors' tentative explanation is that these silts have a dilatant structure. This phenomenon is familiar in dense sands and is the name given to an expansion of the material consequent upon shear deformation. Such an expansion, which need only occur on the plane of shear failure, can result in a drop in pore-water pressure which may even become negative. Since the total pressure remains unchanged the effective pressure must increase by a corresponding amount. The shear strength therefore increases with lateral pressure. This effect would only be expected in soils with a pronounced structure and may not occur with the same soil when remoulded. Further it is likely to be restricted to soils of a certain particle size. In this respect it should be noted that the liquid limit is not an index of particle size and the particle size of the various soils in Group VII may be considerably more alike than would be expected from the liquid limits.

In the partially saturated soils of Groups IV and V the increase in strength with lateral pressure is easily explained by immediate compaction and what might be called "micro-consolidation" into the air voids.

In the case of the clay-shales and siltstones of Group VI it seems possible that there may be an appreciable area of grain to grain contact, in which case the effective pressure can increase immediately with increase in lateral pressure. On the other hand the possibility of dilatancy in these materials cannot be ruled out.

The authors are aware that in the type of triaxial test in use in Holland and Belgium angles of shearing resistance greater than zero are habitually obtained for fully saturated clays. The authors are of the opinion that the evidence indicates that this is due to a certain amount of consolidation occurring during the test. This point is important and the authors hope that full discussion will result in agreement.

TENTATIVE CONCLUSIONS.

1) All clays which are fully saturated, whether soft clays of recent origin or stiff fissured clays which have been over-consolidated, have an angle of shearing resistance of zero for the conditions of these tests, where no water content change is allowed to occur under the applied stress.

2) All clays which are partially saturated, either because they exist above present

ground water level or because they have been excavated and recompacted, exhibit an angle of shearing resistance always greater than zero.

3) Clay-shales and siltstones give a value of ϕ greater than zero. In some cases this may be due to partial saturation, but the evidence suggests that there may also be some other cause such as appreciable area of contact between the grains.

4) Some saturated silts give values of ϕ greater than zero even when tested under conditions of no water content change. These silts have liquid limits less than about 35%. The reason for this result is not known but it is suggested that it may be connected with the phenomenon of dilatancy which could cause an increase in effective pressure due to a decrease in the pore-water pressure.

REFERENCES TO BIBLIOGRAPHY.

1) Skempton A.W. and H.Q. Golder "Practical Examples of the ϕ = 0 Analysis of Stability in Clays". Paper to this Conference.
2) Rutledge P.C. "Triaxial Shear Research." Waterways Experiments Station, Vicksburg U.S.A. (1947)
3) Terzaghi K. "Simple Tests determine Hydrostatic Uplift." Engineering News Record 116 p. 872. (1936)
4) Terzaghi K. "Tragfähigkeit der Flachgrundungen." Int. Assoc. Br. Str. Eng. p. 659. Paris (1932).
5) Rendulic L. "Ein Grundgesetz der Tonmechanik und sein experimenteller Beweis." Der Bauingenieur 18 p. 459 (1937).
6) Jürgenson L. "The Shearing Resistance of Soils." Jour. Bost. Soc. C.E. July 1934.
7) Colling L.F. "Development and Scope of Soil Mechanics." Inst. C.E. London 1946.

REFERENCES TO TEST RESULTS IN TABLES 1 TO 7.

8) A.W. Skempton "The Geotechnical Properties of a Deep Stratum of Post-glacial Clay at Gosport." Proc. 2nd Int. Conf. on Soil Mechanics 1948.
9) A.W. Skempton "A Slip in the West Bank of the Eau - Brink - Cut." J. Inst. C.E. Vol. 24 No. 7 1945.
10) Soil Mechanics, Ltd., Unpublished Records.
11) A.W. Skempton Unpublished Tests at Imperial College.
12) A.W. Skempton Unpublished Tests at B.R.S.
13) H.Q. Golder "The Shear Strength of Clay and its Measurement." Inst. C.E. Glasgow Association, 9th March, 1945.
14) A. Casagrande "Progress Report No. 4 on Cooperative Triaxial Shear Research." Harvard University, 1941.

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II d 3

A STUDY OF THE IMMEDIATE TRIAXIAL TEST ON COHESIVE SOILS

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1. INTRODUCTION.

Two basic types of compression test are used for investigating the shear characteristics of cohesive soils. Firstly the immediate triaxial test, in which the specimen is stressed under conditions of no water content change. The unconfined compression test is a special case of the immediate triaxial test, where the applied lateral pressure is zero.
Secondly, the equilibrium triaxial test in which the specimens is allowed to attain equilibrium under an applied hydrostatic pressure before being tested in compression x).

The immediate triaxial test is, from the practical point of view, the more useful and provides the basis for the Φ = 0 analysis of stability in saturated clays. The equilibrium test is less easily interpreted and may, indeed, be misleading in the evaluation of stability problems (Terzaghi 1947).

In the present paper an attempt is made to study the immediate triaxial test from three points of view:

i) to assess the significance of the inclination of the shear planes in compression specimens of cohesive soils

ii) to obtain a theoretical expression for the pore water pressure set up in a saturated clay when stressed under conditions of no water content change

iii) to obtain a theoretical expression for the compression strenght of a saturated clay, as measured in the immediate triaxial test, in terms of the true cohesion and true angle of internal friction as defined by Hvorslev (1937).

The treatment is approximate, but it is presented in the hope that it may prove useful in further research work.

2. THE IMMEDIATE TRIAXIAL TEST.

In the immediate triaxial test a cylindrical specimen initially in equilibrium at some particular water content under a capillary pressure xa) p, is placed between non-porous end pieces and covered with a thin rubber envelope. The specimen is subjected to a hydrostatic pressure σ_3 and the axial pressure is then progressively increased until failure occurs under a total applied axial pressure σ_1 No water content change is allowed to take place during the test.

With saturated clays the compression strength ($\sigma_1 - \sigma_3$) is found to be a constant,

x) In the U.S.A. these two types are referred to as the "quick" and "consolidated" triaxial tests (v. Triaxial Shear Research 1947).

xa) In Laboratory work on remoulded clays p is the pressure under which the specimen has been consolidatio prior to the test.