ratios are possible corresponding to the points A and B in Fig.No.1, and in accordance with the displacement of the individual grains, one or the other process takes place within separate zones in the interior of the sand, so that tension becomes inhomogeneous, which is the cause of the progressive nature of the deformations. This change-over of the grains from one tension ratio to another takes place without retardation in the case of dry sand, as no resistance is offered by air. If, however, the pores contain water, the process is retarded, which leads to a break-down of the granular structure within a considerable sphere thus causing the jerky process of deformation. Figure No. 2 allows a comparison of the results produced by prism-tests carried out by the author with water-saturated and damp sands of various humidity (containing different percentages of water) and a granulation of from 0.15 - 0.25 mm grain-diameter, and an initial void ratio of ε_o = 0,61. The interrupted lines represent sand saturated by water, fully drawn-out lines represent humid sand containing a maximum quantity of water, whilst dotted lines show sand with low percentage of water. Break tensions σ_b are given for each curve, Figure No. 3 gives a diagrammatic description of the process.

As follows from what has been said above, deformation by jerks in the case of tests carried out with sand cylinders is caused by pore water and it has proved impossible in this case in spite of the most carefully selected test-apparatus to avoid considerable dispersion within the sphere in which deformation increases in a progressive manner, - which is in direct opposition to results obtained by tests carried out with dry sand.

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II d 13 SHEARING STRENGTH DETERMINATIONS BY UNDRAINED CYLINDRICAL COMPRESSION TESTS WITH PORE PRESSURE MEASUREMENTS

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GENERAL

Data on magnitudes of shearing strengths are needed in the solutions of many types of practical problems relative to the stability of earth embankments and foundations. When the soil in question is a thick deposit of highly impervious, completely saturated clay, there often is opportunity for practically no drainage during shear, and laboratory tests for the determination of shearing strengths for such cases must be tests in which no drainage is allowed. If a limited amount of drainage occurs, the best approach to the problem usually is by the determination of shearing strengths for both the completely undrained and the completely drained cases, followed by an interpolation that must be based largely on judgment. Tests with complete drainage often require much time, but they offer no particular difficulties of interpretation. Tests in which no drainage occurs are more difficult of interpretation and are the subject of this paper.

Fundamentally, the shearing strength in all instances depends on the intergranular pressures that prevail when failure is taking place. However, in tests in which no drainage occurs, there is a gradual build up of pore-water stress that can only be measured by special testing apparatus; thus the pore-water stress and the intergranular pressures at failure are of unknown magnitude when ordinary testing methods are used. However, the intergranular pressures that are attained at failure in tests that do not drain bear definite relationships to the intergranular pressures prevailing at the start of the tests, and in some cases these initial intergranular pressures are known. Therefore, the logical relationship for use in the case without drainage is one in which the shearing strength is expressed as a function of the pressure to which the sample was consolidated previous to shear.

If the initial consolidation pressures on a given soil in nature are the same on all orientations of planes within the sample, relationships between shearing strength and consolidation pressure may be predicted from the data of a standard type of shear test. x) In general, however, the original intergranular pressures in nature have a principal stress ratio that is of unknown magnitude, but that is known to be greater than unity, and the general form of the relationships between shearing strength and consolidation pressures, applying for any given principal stress ratio, can be obtained only by the use of special tests in which pore pressures are measured.

This paper presents examples of tests on saturated, or nearly saturated clays, in which pore pressures are measured. The main object of the paper is to present methods for the interpretation of these test data. The methods presented apply in fullest degree to clays that have not been precompressed; that is, to clays which have never been consolidated to pressures greater than those existing at the start of the shear test. Therefore, the method as it applies to clays that have not been precompressed will be presented first, and the precompressed case will be considered later.

Before proceeding with the main subject, special and simplified conditions of testing, that impose limitations on the applicability of the results, will be mentioned. The tests that are reported in this paper are cylindrical compression tests (often called triaxial tests) of more-or-less standard form, with equal minor and intermediate principal stresses acting throughout. They should, without exception, be run on specimens that are in good

x) These tests were proposed by A. Casagrande and are known as "quick-consolidated" or "consolidated-undrained tests" 1), 2).

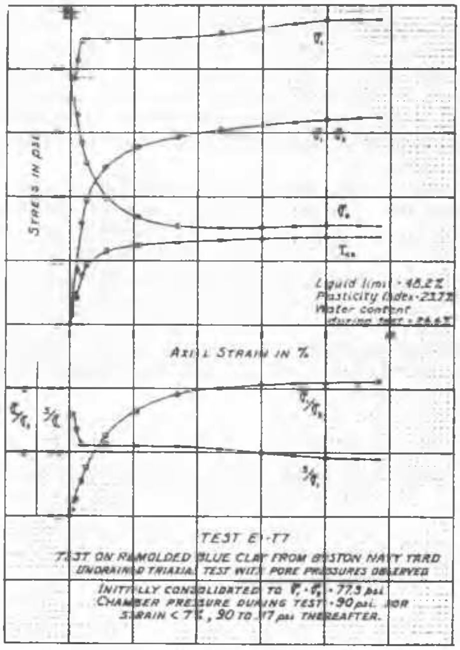

FIG.1

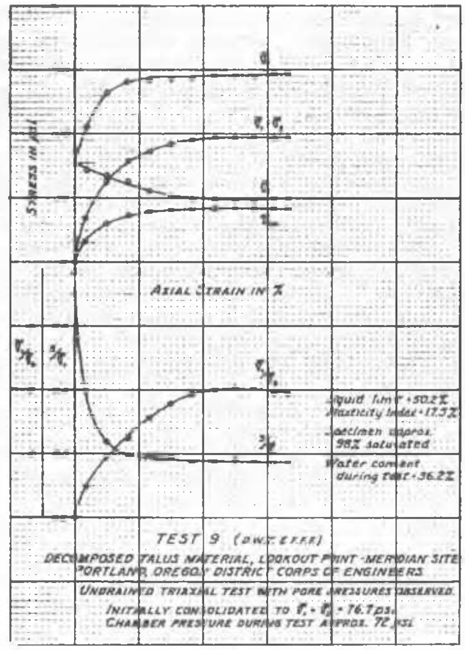

FIG. 2

condition, but that undoubtedly always are somewhat changed from their natural undisturbed state. The tests are run directly to failure with no reversals of loading. All details according to which actual conditions in the soil mass in nature differ from these testing conditions represent possible differences between the strengths obtained in tests and the strengths in nature. The possibility of such differences should not be disregarded, but it is believed that they are no greater than must be recognized as existing in the great majority of analyses in which soil properties are used.

TEST DATA

Figures 1 to 3, inclusive, represent shear tests of the undrained type, in which the clay was initially consolidated to a uniform consolidation pressure (principal stress ratio of unity), this consolidation pressure being larger than any intergranular pressure previously acting on the material. These tests were conducted in cylindrical compression apparatus on specimens of 2.8-inch initial diameter and approximately 6.5- inch length. The important respect according to which these tests differ from standard tests is that pore water pressures were observed in the sample during shear. The pore-pressure measuring unit can be described briefly as a porous device inserted diagonally into the centre of the sample and connected by a small plastic tube containing air-free water to a finebore tube; the pressure required to hold the top of the water column at a fixed point in the fine-bore tube is equal to the pressure in the pore water of the sample. x)

The observed quantities in these tests are the chamber pressure σ_{ch}, the axial strain e_a, the axial load Q, and the pore water pressure u. The deviator stress $\sigma_1 - \sigma_3$ is taken as $Q(1-e_a)/A_o$, where A_o is the original area of the sample and $A_o/(1-e_a)$ is the approximate area at any stage of the test. At any time during the test the intergranular major and minor principal stresses are designated by $\bar{\sigma}_1$

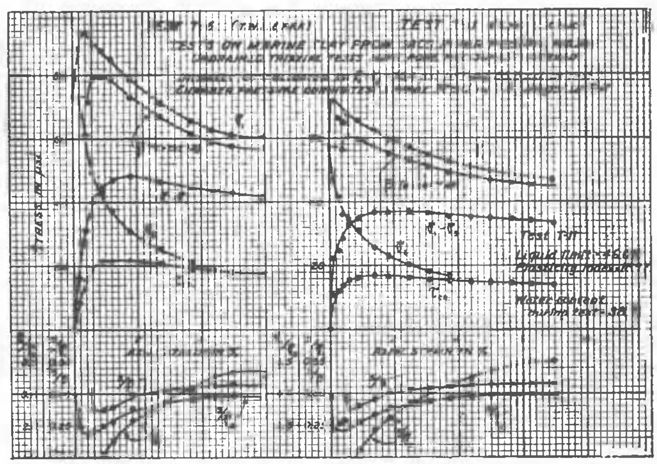

FIG. 3

and $\bar{\sigma}_3$ and they may be expressed

$$\bar{\sigma}_3 = \sigma_{ch} - u$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 + (\sigma_1 - \sigma_3)$$

The stress τ_{cr}, which is the shearing stress on the plane of maximum obliquity, may be obtained graphically from Mohr Diagrams of the type illustrated in Fig. 4 or may be computed by the use of the expression

$$\tau_{cr} = \frac{(\bar{\sigma}_1 - \bar{\sigma}_3)\sqrt{\bar{\sigma}_1 \bar{\sigma}_3}}{\bar{\sigma}_1 + \bar{\sigma}_3}$$

The shearing strength, s, may be defined as the maximum value τ_{cr} will attain under increasing shearing strain; after τ_{cr} has reach-

x) For a more complete description of the apparatus and the pore-pressure measuring device, see 3) and 4).

ed its maximum value and is decreasing, the shearing strength at any given strain is equal to value of τ_{cr} at that strain.

The three soils represented in Figs. 1 to 3 inclusive are of entirely different type. Fig. 1 represents a remolded clay that has been used extensively in research on clays; its main interest is this fact, since data on this remolded material have little value in practical problems. The decomposed talus material and the marine clay of Figs. 2 and 3, respectively, are from far separated origins. The strength of the talus soil changes a minor amount when remolded, whereas the marine clay loses 90 per cent or more of its strength when remolded at given water content; also, in other respects, the two soils have extreme differences in properties. In the talus soil, approximately two per cent of the pore volume is occupied by air; the other samples represented by the figures are in essentially saturated condition.

No detailed discussion need be devoted to the curves of the upper plots of Figs. 1 to 3, with the exception of the curve labeled $\bar{p}$ in Fig. 3, which will be discussed later. In the lower plot, the curve of $\bar{\sigma}_1/\bar{\sigma}_3$ appears and is a curve that has long been in common use. The ratio $s/\bar{\sigma}_1$ also is represented in the lower plot, s being the shearing strength as previously defined and $\bar{\sigma}_1$ the intergranular major principal stress at the given axial strain; the plot of this ratio presents the concept underlying the method of interpretation which is the main subject of this paper.

A relationship that is of particular interest is shown by the two tests of Fig. 3. These two tests are on specimens initially consolidated to different pressures and all stresses shown by the curves of the upper plot are larger for Test T-6 than for Test T-11. However, it is seen that almost identical results appear for the two tests in the lower plots. Furthermore, many other comparisons could be presented to prove that this similarity holds essentially true at all pressures in most soils, subject only to the condition that there be no precompression. From this evidence it may be concluded that the lower plots of Figs. 1 to 3 are valid for the soils represented at any magnitude of consolidation pressures and at any void ratio or water content.

SOIL PRESSURES THAT ARE KNOWN IN NATURE

The pressures acting within a soil mass on planes parallel to ground surface are sometimes called overburden pressures. The intergranular components of these pressures are designated intergranular, overburden pressures. Overburden pressures can usually be estimated with reasonable accuracy. In a homogeneous soil mass with no horizontal variation and with a surface that extends a considerable distance at a constant slope, the overburden pressure acts vertically as shown in Fig. 5 and may be expressed $p = \gamma z \cos i$, in which i is the slope angle, z is the depth below ground surface, γ is the true unit weight and p is a combined, or intergranular-plus-neutral pressure.

If the ground surface is level, the overburden pressure is a principal stress. For the case without precompression, which is the case now under consideration, the overburden pressure is the major principal stress and it may be expressed

$$\sigma_1 = \gamma z$$

If the elevation of the free water surface is known, or if water pressure values are available from observations, the neutral pressure may be subtracted vectorially from the combined pressure to determine the overburden, intergranular pressure. For example, the intergranular pressure on the plane parallel to ground surface at a depth of 20 ft below the surface of a 6 to 1 slope composed of soil with a true unit weight of 120 lb per cu ft, when the water pressure at this depth is 780 lb per sq ft, may be obtained as follows: in lb per sq ft, the combined pressure is (120)(20)(6/$\sqrt{37}$) or 2370; its shearing component is (2370)(1/$\sqrt{37}$) or 390; its direct component is (2370)(6/$\sqrt{37}$) or 2340; and shearing and direct components of intergranular pressure are, respectively, 390 and 2340-780, or 1560.

The pressures on all other planes are greatly dependent on the strain history of the soil. The pressures on vertical planes may have values anywhere between the minimum or active value and the maximum or passive value. However, when the soil is not precompressed, as in the case under consideration, the pressures on vertical planes are usually somewhat smaller than the overburden pressures.

Thus the complete stress systems in the underground are indeterminate and any predictions of shearing strengths for the undrained case are complicated by the fact that to a degree these strengths depend on stresses that are indeterminate.

STRENGTH PREDICTIONS FOR CLAY THAT IS NOT PRECOMPRESSED

In spite of the fact that certain stresses that effect the shearing strength are of indeterminate nature, predictions of minimum and maximum values of possible strengths can be made and, fortunately, there is not a great difference between these extremes in many instances.

For example, reference will first be made to Fig. 2. Since it is known that the $s/\bar{\sigma}_1$ curve is valid for this clay under shear without drainage for all consolidation pressures, the value of $s/\bar{\sigma}_1$ is, for all consolidation conditions, between 0.55 and 0.285. The figure indicates that at an axial strain of about 1.8 per cent, the principal stresses are respectively equal to 142 and 60 psi, their ratio being 2.4. It may next be noted that for clays initially consolidated to these pressures, or to any other principal stresses in the ratio 2.4, the behavior during shear is shown by the portion of the curves to the right of an axial strain of 1.8. Thus it may be concluded that if the $\bar{\sigma}_1/\bar{\sigma}_3$ ratio to which the sample is consolidated initially is greater than 2.4, the $s/\bar{\sigma}_1$ ratio must be between 0.300 and 0.285, or

$$0.300\ \bar{\sigma}_1 > s > 0.285\ \bar{\sigma}_1 .$$

Possibly the majority of cases encountered would have pre-shear $\bar{\sigma}_1/\bar{\sigma}_3$ ratios greater than 2.4, but this cannot be assured. At least it can be said that it is entirely conservative under all values of $\bar{\sigma}_1$ and for all consolidation pressures to assume that for this soil

$$s \geqq 0.285\ \bar{\sigma}_1$$

Referring now to Fig. 1 it may be noted that there is a some what smaller difference than in Fig. 2 between the maximum and minimum values, and for the complete range of possible pre-shear $\bar{\sigma}_1 / \bar{\sigma}_3$ values

$$0.36\ \bar{\sigma}_1 > s > 0.29\ \bar{\sigma}_1$$

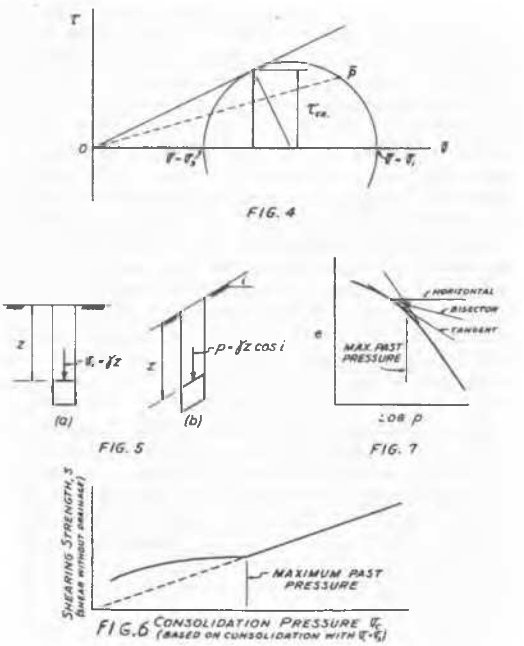

FIG. 4, 5, 6, 7

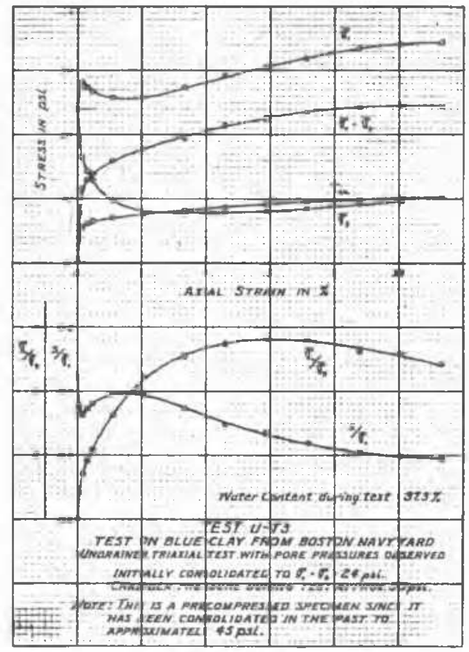

FIG. 8

Because of the considerably different shape of curves of major principal stress in Fig. 3 as compared to Fig. 2, the $s/\bar{\sigma}_1$ curves are also of different shapes for the two soils represented. In Fig. 3.

$$0.295\ \bar{\sigma}_1 > s > 0.235\ \bar{\sigma}_1 .$$

For any case in which the ground surface is level, values of the overburden intergranular pressure $\bar{\sigma}_1$ can usually be estimated, as has been explained. Present understanding of $\bar{\sigma}_1/\bar{\sigma}_3$ values existing in nature, or of the axial strains that have occurred during the history of clays, will seldom be sufficient to permit the determination of anything but the ranges of the strength values. However, the above considerations indicate that the ranges are often quite small and, therefore, the minimum value will often serve as a satisfactory estimate of the shearing strength.

In connection with a clay such as that represented in Fig. 2, it should be noted that conventional tests with no pore pressure observations can furnish only the initial point (from quick-consolidated, or consolidated-undrained tests) and the final point (from slow, or drained tests) of the $s/\bar{\sigma}_1$ curve. Thus the concepts that have been outlined can seldom be used when only conventional test data are available.

The above considerations apply to cases in which the ground surface is initially level. For example, if a deep ditch is to be excavated rapidly into a clay deposit that is not precompressed and that has a level surface, the shearing strengths that would apply in stability analyses of the banks can be predicted by this method. However, it is likely that opportunities for applications with sloping ground will be at least as frequent as those with level ground. Even if the slope of the terrain is somewhat irregular in such cases, an average slope may often be used without appreciable additional inaccuracy.

An example based on sloping ground is illustrated by curves of $\bar{p}$ and $s/\bar{p}$ in Fig. 3. In this example, the direct and shearing components of the intergranular stress $\bar{p}$ are in the ratio 4 to 1, as was the case in the numerical illustration previously given; stress $\bar{p}$ thus has an obliquity angle α that is equal to $\cot^{-1}4$. Values of $\bar{p}$ are easily obtained by graphical procedure; Mohr circles can be drawn, using known principal stress values, and on each plot a line at a 4 to 1 slope determines $\bar{p}$, which is represented by the distance from 0 to the point labeled $\bar{p}$ in Fig.4. The lower plots of Fig.3. show that for the given obliquite angle

$$0.315\ \bar{p} > s > 0.275\ \bar{p}$$

with a considerably smaller range prevailing when the initial $\bar{\sigma}_1/\bar{\sigma}_3$ exceeds approximately two. Below sloping ground, however, it should be noted that the obliquity angle will usually have values that vary with depth.

CONSIDERATIONS FOR PRECOMPRESSED CLAYS

When a clay has been compressed in the past under pressures greater than now acting, it is well known that the strength is greater because of this past pressure. The strength envelope for such a case is of the shape shown in Fig. 6.

The method that has been outlined cannot be used directly in such a case, but it can be used to establish bounding values. Expressions such as those given above, based on tests on specimens that have not been precompressed, and based on the use of present values of overburden pressure $\bar{\sigma}_1$ or $\bar{p}$, give strength predictions that are always smaller than those prevailing at the time of the investigation. The same expressions, using the maximum past overburden pressures instead of the present values, give maximum strengths of the soil in the past, and these values are somewhat larger than the present values. It is well known from common information on compression and expansion characteristics of clay that expansions resulting from removal of load are much smaller than virgin compressions occurring under the same load change. Therefore, present strengths tend to be much nearer to the larger of the two above-mentioned bounding values, and especially so if the present overburden pressure is a fairly large fraction of the maximum past overburden pressure. This condition is shown by Fig. 6. The maximum past value of intergranular major principal stress may be estimated with reasonable accuracy from the compression curves that are obtained in consolidation tests, by a method proposed by A. Casagrande 5). This method, which is shown in Fig. 7, may be used for cases of both level and sloping ground surface.

There are more direct methods for determining the shearing strength of precompressed clays, the commonest being by cylindrical compression tests, either confined or unconfined, on undisturbed samples at their natural water content and by tests on samples reconsolidated to their natural overburden pressure. x) Such tests, however, give strengths that usually do not bear a constant ratio to the overburden pressure. Possible further research will lead to a more general method of type similar to that presented in this paper for determining the strengths of precompressed clays. Such a method quite possibly can be based on the use of curves of the type appearing in Fig. 8, which represents a compressed clay, but which has the same plots as used in Figs. 1 to 3.

FINAL COMMENTS

Study of pore pressure data shows that shearing strengths, for cases in which no drainage occurs, depend on the ratio of the principal stresses under which the soil is consolidated before the shear occurs; therefore, for rational predictions of shearing strengths for this case, data on pore pressures are essential.

The method presented in this paper is as general as can possibly be obtained for soils that have not been precompressed. For precompressed soils, the method gives upper and lower limiting values that, at least, are of value as checks on strengths obtained by other methods.

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II d 14

A STUDY ON PLASTIC CLAY OF THE PARIS REGION

J. FLORENTIN
G. L'HERITEAU
E. BRANDA

SUMMARY OF THE FRENCH REPORT

1) Origin of the samples

Carrières des Etablissements Etienne Henry, PROVINS.

2) Characteristics of the Clay

Natural Water Content	27.2 %
Specific Weight	2.69
Liquid Limit	117.5 %
Plastic Limit	32.7 %
Plasticity Index	84.8 %

3) Nature of the Study

Determination of strength by means of the triaxial compression device. In our device the vertical pressures are exerted through a piston with weights or tanks filled with water. The lateral grasp is exerted by means of glycerine under pressure. This device allows the measurement of vertical and horizontal deformations by means of comparators.

4) Tests on undisturbed samples

These tests yielded unconclusive results even for samples of close-by origin in the pit, i.e. $\varphi = 22^\circ$; C = 3,90 kg/cm2 (55.5 psi) and $\varphi = 20^\circ 30'$; C = 1,85 kg/cm2 (26.4 psi). These characteristics are obtained from the intrinsic curves.

The angles of internal friction which may be deduced from the gradient of the shearing surfaces, are very irregular.

Seen through a magnifying glass, the undisturbed clay is traversed by a network of fine cracks. These cracks cause the shear resistance to vary. The angle of internal friction φ and the cohesion C, which may be derived from the intrinsic curves do not correspond to the facts.

5) Tests on remolded and similar samples

Consolidation through compression of satur-