DIVISION 2

Soil Properties – Shear Strength and Consolidation

Propriétés des sols—Résistance au cisaillement et consolidation

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DESIGN FOR STABILITY IN CIVIL ENGINEERING, when reduced to its simplest form, can be visualized as a comparison between stressing and the ability of materials to withstand stress. The ability of material to withstand stress is mainly measured by its mechanical properties, determined under stress conditions similar to those prevailing in the structure. It is only natural then that one of the first tasks of any branch of civil engineering dealing with a given material should be the thorough study of its mechanical properties.

For this reason, the sections of soil mechanics conferences, meetings, and technical publications referring to mechanical properties always have the most papers. The variety and complexity of the mechanical properties of soils make these studies imperative at a time when knowledge of them is still limited. The emphasis on this facet of soil mechanics was maintained in the interim between the Fifth and the Sixth International Conferences. Numerous contributions to the subject were made at regional and local conferences as well as at such special international conferences as the Second Panamerican Congress (Brazil, 1963), the Second Asian Regional Conference (Tokyo, 1963), the Symposium on Laboratory Shear Testing of Soils (Canada, 1963), the Second European Conference (Germany, 1963), and the Conference on Design of Foundations for Control of Settlement (U.S.A., 1964). As well, specialized technical magazines, such as *Géotechnique*, the *Journal of the Soil Mechanics and Foundation Division of the A.S.C.E.*, and others publishing soil mechanics papers, devoted a large portion of their pages to contributions which increased the knowledge of the mechanical properties of soils. The Sixth International Conference is no exception to this trend; sixty-one papers have been assigned to Division 2, even though it has been limited to the shear strength and deformation of soils.

When a design for stability is confined to problems of strength and deformation in engineering practice, the contrast between stressing and material properties can frequently be separated into two independent or nearly independent operations. First, the stressing load is compared with the failure strength of the material to ensure that a reasonable margin or factor of safety exists between them; secondly, deformations are computed and checked against some previously established allowable values, usually empirical in nature. This simple scheme may often be complicated by the interdependence between failure strength and deformation which sometimes occurs in statically indeterminate problems. Yet, in soil mechanics, the discrimination is frequently clear cut and solutions are said to be controlled either by failure or by settlement.

Based on this assumption, the papers assigned to Division 2 have been divided into (1) those concerned mainly with the shear strength of soils under static loading and (2) those dealing mainly with soil deformation under static loading and soil behaviour in general under dynamic loading.

I. SHEAR STRENGTH OF SOILS UNDER STATIC LOADING

As in other branches of civil engineering dealing with non-elastic, creeping materials, strength problems are much easier to solve than deformation problems. By the same token, the determination of the strength parameters also proves easier.

Soil mechanics has gone a long way in devising methods to determine these parameters in the field and, through adequate samples, in the laboratory. Study of the various typical soil materials and correlation of the results to form rational laws which permit extrapolation and understanding of a material's behaviour are essential if reliable solutions to practical problems are to be found. Limitations of time and expense make for solutions based on a few tests, comprising only simple stress-strain conditions. For them to provide efficient answers, however, it is necessary for the engineer to have a circumspect general knowledge of soil behaviour, with sufficient insight to extrapolate beyond the bounds of limited testing.

Much has been done to forward this aim, but the variety of soil types and conditions and the diversity of states of stress make the task ahead still formidable. Twenty-nine of the papers presented in Division 2 deal with some aspect of the shear-strength behaviour of soils. They are, of course, only a small fraction of all the papers on the subject published since the last international conference.

Laboratory Testing Devices

The versatility of the triaxial cell permits the control of a variety of stress conditions, a control not obtainable with other existing standard testing apparatuses. In particular, control of consolidation and of pore pressures, coupled with the possibility of reaching a state of complete saturation through the use of back pressure, has led recently to the encasing of other laboratory testing devices in triaxial cells in order to study the implications of the results obtained. For example, in the laboratory of the General Reporter's consulting firm, direct shear tests on soft rock with the use of back pressure and pore-pressure measurements have been made by placing a shear ring inside a triaxial cell.

Kenney and Landva (2/28) report details of an apparatus, based on this idea, which was used to study the implications of the results of the shear strength obtained with the vane tests. By attaching a retractable vane that is pushed into a triaxial cell, samples of saturated clay can be sheared under selected control conditions, such as degree of consolidation, pore-pressure development, and so forth. At present, it is reported, the vane-triaxial apparatus is being used to study the dimensions of the surface of failure in relation to the vane dimensions, the shear stress distribution at the end faces of the cylindrical surface, and the effect of the rate of strain on shear strength. No results of tests are given in the paper; only a general statement on the effect of the rate of strain is advanced.

Pore-pressure measurement has become a routine operation in both commercial and research triaxial testing. Its use is also being extended to other types of test, such as direct shear and consolidation tests, and will spread further with time. It may be that in the future practically no testing of saturated soils will be made without pore-pressure measurement. Consequently, the development of devices which are easy to use and simple to control is very important in order that common errors resulting from inadequate air purging of tubes and pipes, leakage of pore water, uncontrolled volume change, and so forth may be avoided. *Leussink and Prange (2/33)* discuss the development of a new electronically operated pore-pressure transducer which can be actuated by short waves, thus avoiding any connection between transducer and receiver. At maximum pressure, actuation requires a volume change of only about one cu. mm. Also, since the transducer is of small dimensions, several of them may be distributed at various selected points when testing large-diameter samples. As with all the existing devices to measure pressures, this transducer, when placed inside a soil mass, has the disadvantage of introducing into the mass a medium of different rigidity which in turn changes the distribution of stress. However, with the small size of the present device, this may prove to be a minor disadvantage. This problem does not apply to pore-pressure devices placed at the top and bottom of samples, however, for in such a case the mode of stress distribution is already determined by the rigidity and roughness of the end caps.

Torsional Field Shear Tests

Field testing to obtain mechanical properties is used particularly for soils as distinct from other structural materials. The difficulty in obtaining adequate samples, coupled with relatively low strength of soil, has prompted the use of field tests to obtain the mechanical properties desired. In fact, the use of field tests on an empirical basis was the only tool available to engineers before the advent of soil mechanics. Many of these empirical tests, improved by basic knowledge of soil mechanics principles, are still in general use, as are some sounding and penetration tests. Realization of their shortcomings has led to the development of more rational field tests, among them field torsion tests and pressuremeter tests. Field torsion tests provide an opportunity of obtaining a clear-cut determination of peak and residual values of the shear strength "as in nature."

Among field torsion tests, the vane tests have become an integral part of the routine for the determination of the undrained shear strength of clays. Especially suitable for saturated soft to medium clays, they provide the best available field procedure for determining such a strength; they have been particularly useful where soft quick clays cover extensive areas. *Aas (2/1)* gives further insight into the meaning and reliability of the strength values obtained with the vane. Varying the ratio of diameter to height of vane, a study was made of the relation between shear strength on vertical and horizontal planes, and it was found that the ratio is approximately proportional to the K_0 state of the clay. Allowing a period of rest after pushing the vane and before turning it resulted in a considerable gain of strength. On the other hand, lowering the rate of strain produced, in general, a decrease of strength, showing what appears to be a very important effect of creep.

Helenelund (2/22) reports on a hand-operated torsional field shear test apparatus developed to measure the shear strength of glacial till. It may be used to perform tests in open pits applying normal pressures which may reach values of the order of 2 to 4 kg/sq.cm. depending on size. Ideas for further developments are also advanced.

Pore-Pressure Build-up in Laboratory Tests

In undrained triaxial tests, the increase of the deviator stress causes a pore-pressure build-up which at failure may be positive, nil, or negative. The ratio A_f between pore pressure and deviator stress at failure is an expression of the ultimate soil volume change tendencies. Experience has shown that, for non-sensitive, normally consolidated clays, A_f reaches values, in isotropic triaxial compression tests, which are in the vicinity of $+1$, while for sensitive clays and other soils with collapsible structures, such as loess, whose soil structure collapses while the soil is being sheared, A_f becomes greater than 1. For overconsolidated clays the value of A_f is smaller than 1, and may be nil or become negative, depending on the overconsolidation ratio of the material.

Denissov (2/15) proposes a mechanistic explanation of that behaviour, starting from two premises. (1) Normally consolidated, non-sensitive clays derive their strength mainly from attraction forces; their behaviour is proposed as standard. (2) Sensitive clays and other plastic soils with collapsible structures derive their strength from both attraction forces and cementing bonds. While bond persists, these latter types of soils are underconsolidated because their void ratios, for a given all-round pressure, are higher than they would be if the same materials behaved like normal clays, as happens when their structure is destroyed by remoulding. For this mechanistic reason, when underconsolidated soils fail in shear, the collapse of their structures creates a tendency for further soil consolidation, building up a pore pressure which is higher than the deviator stress. On a similar basis an explanation is given for the behaviour of overconsolidated clays.

Measurement of pore pressure in soil tests has shown that the propagation of this pressure through the soil mass is not instantaneous even if soils are fully saturated by the application of high back pressures. Because of this time lag, among other things, triaxial tests with pore-pressure measurements cannot be run as fast tests but require an increase in the deviator stress over approximately 8 to 12 hours for samples of clay two inches in diameter. Observations made on earth dams and other earthworks have shown the same behaviour. Systematic studies of this peculiar hydraulic behaviour of the soil water may prove to be very helpful. They might have, for example, an important bearing on the design of the drawdown state of earth dams. *Barberis (2/4)* describes an attempt to make such a systematic study. It has been performed by measuring the time lag for pore pressure in a triaxial cell using partially saturated soil samples; the degree of saturation unfortunately is not reported. None the less, the experiments show that the time lag is a function of soil permeability, being inversely proportional to it; more permeable soils need less time for pore-pressure propagation than do more impervious materials. The time required for pressure propagation also seems to be a function of the magnitude of the pressure applied, a conclusion that should be accepted with caution, as it may be only the result of dissolving more air in water for higher pressures. This same reason may also explain the faster propagation obtained on loading as compared with unloading. It is felt that to make the investigation more useful it should be continued, testing fully saturated and partially saturated soils with known degrees of saturation.

Frequently, problems must be analysed in which the effect on bearing capacity and settlement of either continuous rainfall or a temporary rise in the groundwater table have to be taken into account. The usual method in such cases is to determine soil properties after full saturation because the mechanism of such processes is not well understood. In this way the problem is placed on the safe side, with a possible sacrifice in economy. For the above reason, a knowledge of the mechanism of mutual air and water displacement from soil voids may be useful. The experimental analysis is not an easy task as the speed of mutual displacement should be taken into account, for it may be of the utmost importance.

The first part of the paper by *Havlíček and Myslivec (2/21)* bears on such a problem. Three saturated samples of sand, 5 cm high, placed in cylindrical receptacles with porous stone bases, were connected by flexible tubes to burettes. Lowering the burette, the head of water necessary to force air into the sample was measured, until a value comparable to the air-entry value was obtained and the sample became air permeable. Raising the head of water did not restore saturation and air remained trapped in the samples. Full saturation was again obtained by immersion of the samples over an extended period of time.

These experiments should be considered only preliminary and do not allow any degree of generalization. Furthermore, many of the proposals made in the paper in relation to the air solution in water seem to be at variance with Henry's law.

Shear-Strength Components

Soil materials with tensile strength, when subjected to any type of compressive condition conducive to failure, appear to develop their strength as the summation of two factors, cohesion and internal friction. Other factors, such as the effects produced by delivery or expenditure of energy during shear volume change, are also present. Disregarding variations in the latter effects, shear strength can be taken solely as a function of cohesion and internal friction. When expressed in terms of effective stress, these two factors are known as Hvorslev's parameters.

In terms of effective stress, the Mohr-Coulomb envelope of a clay is an inclined line which may or may not pass through the origin, depending on whether the clay is normally consolidated or preconsolidated. Every ordinate on this line is said to be the summation of cohesion and internal friction. Following the hypothesis that internal friction depends only on normal stress on the plane of failure, which means supposing that the angle of internal friction is a constant, independent of water content and stress history, Hvorslev discovered that cohesion is also independent of stress history and only a function of water content. His expression of true cohesion and true angle of internal friction, as opposed to apparent cohesion and apparent angle of internal friction, is one of the classic expressions of soil mechanics giving the physical components of the shear strength of clays.

To obtain Hvorslev's parameters, it is necessary to perform tests that compare the strength of clay samples which are of the same material and have identical water contents, so that they may have the same true cohesion, but which have different stress histories, so that they may have different normal stresses on the planes of failure. For this purpose, various methods of testing a clay have been proposed. They are summarized by *Noorany and Seed (2/39)* who indicate their shortcomings. To avoid these shortcomings, a new method is proposed in which two identical samples are first consolidated anisotropically under the same stress conditions. After consolidation, with both samples at the same water content, drainage is stopped and the samples are sheared following two different stress paths, furnishing two different normal effective stresses on the failure plane. As with many of the methods proposed to separate Hvorslev's parameters, the two Mohr circles obtained may be very close to each other for some clays, and thus obscure interpretation.

Hvorslev's expression (Hvorslev, 1960) implies that cohesion originates on bonding forces, which are a function of water content, whereas internal friction depends only on normal pressure on the plane of failure. Another idea has been advanced to discriminate between the physical components of the shear strength of soils. It was first sketched by Taylor in his *Fundamentals of Soil Mechanics* (1948), where he contended that cohesion may not be physically different from internal friction, but may only be an expression of what he called the intrinsic pressures. To him an internal stress would be a function of the preconsolidation pressure.

The view that there is no fundamental difference between cohesion and friction has been put forward lately by other workers and is developed in detail by *Parry (2/42)*. This paper refers only to fully saturated remoulded clays and starts with a short discussion of the nature of interparticle force network. The hypothesis is advanced that the net stress arising from the force network must be one of repulsion, whose magnitude, at any void ratio, is a function of stress history, depending on whether the clay is normally consolidated or preconsolidated. At any given void ratio, the repulsive forces are supposed to have their maximum value for normally consolidated clays and their minimum value for the clay condition represented by points on the rebound envelope curve of all possible swelling lines. It is assumed that ultimate failure will develop along what has been called the critical void ratio (C.V.R.) line. The virgin consolidation line and the rebound envelope curve enclose the locus of all points representing the clay behaviour, the C.V.R. line being located symmetrically between them.

With these basic ideas, a simplified mechanistic model is put forward and a mathematical expression derived which yields the shear strength as a function of only one intrinsic constant. The expression is compared with test results and with Hvorslev's equation. From this comparison, it is shown that the new expression appears to fit the test results better.

The new expression may prove to be an important contribution to the interpretation of soil behaviour. As with most theories, however, it may have validity only in a limited range. Among other things, it is based on the assumption that the ultimate condition always develops along the C.V.R. line, a hypothesis that has been contested elsewhere in papers presented to this same Conference. Also, the existence of a net repulsive stress seems to be open to question in non-remoulded clays and in remoulded clays with thixotropic hardening, as it would appear to imply that soil disintegration would occur under zero effective normal pressure, something that has yet to be proved. Nevertheless, the equation put forward by *Parry (2/42)* may possibly be disconnected from the real nature and sign of those forces and still maintain its value within a given range of applicability yet to be established.

Fagnoul (2/16) reports the results of triaxial and shear tests on normally consolidated silts, silty clays, and clays, performed first on undisturbed samples and then on the same materials after remoulding the soils at the liquid limit. The tests are intended to show that in normally consolidated cohesive soils remoulding destroys the cohesion intercept, c', but introduces no change in the effective angle of internal friction, ϕ'. It is hypothesized that the identity in the ϕ' values derives from the fact that deformation to failure produces remoulding in the plane of failure and suggested that, in problems where cohesion is disregarded, testing of samples remoulded at the liquid limit will solve the problem of finding the effective angle of internal friction. This suggestion is not clearly understood since it is well known that, in terms of effective stress, normally consolidated soils have a zero cohesion intercept. Moreover, some of the results reported give what appears to be an exceedingly low effective angle of internal friction and an unusually high cohesion c', values that require a further explanation.

Analysis of the Standard Triaxial Test

Now that techniques for performing triaxial tests have matured, and the standard compression test with the use of back pressure to ensure saturation and the measurement of pore pressures and other details of apparatus and testing procedure to provide reliable results have been developed and are in general use, one of the next steps in research should be to devote time to determining the real implications of standard triaxial compression tests. In practice, a problem very seldom appears in which radial or cylindrical symmetry exists as it does in the standard triaxial tests. On the contrary, other states of stress are very often present. Loading tests and other measurements made on earth structures, such as earth dams, indicate some divergence between theoretical predictions, based on the standard triaxial tests, and field observations. Is this divergence a consequence of faulty theories or a defect in the interpretation of soil properties due to the inadequacy of the method of testing?

Broms and Jamal (2/10) give a critical analysis of the triaxial test in cohesionless soil, trying to show that because of non-uniform stress distribution of radial and tangential stresses (σ_r and σ_θ) in planes perpendicular to the major principal stress, interpretation of test results considering $\sigma_2 = \sigma_3$ or $\sigma_\theta = \sigma_r$ may lead to erroneous evaluations (underestimations) of the cohesion and the angle of internal friction of soils.

To prove the point, consolidated-undrained tests with pore-pressure measurements were performed on sand, comparing solid cylindrical specimens with hollow ones in which the central core was replaced by water. During the undrained part of the tests, the water of the central core was kept either at constant pressure or at constant volume. When kept at constant volume, it developed a pressure which was different from the chamber pressure, showing that the average σ_r was smaller than the average σ_θ and that the state of stress was not cylindrical. A real cylindrical state of stress was forced on tests in which the pore water pressure was kept constant at the same level as the chamber pressure.

Results show that for dense sands the angle of internal friction obtained in these special types of tests is 3° to 4° higher than that in standard tests, while in sand of medium to low relative density the difference is negligible. Only a few tests are reported, all of them for small chamber pressures and always with the sand behaving as dilatant. More tests are needed to substantiate the analysis.

Anisotropic States of Stress

For reasons of simplicity, the practical routine determination of the shear strength of soils in the triaxial apparatus is made under a state of stress with radial or cylindrical symmetry. Moreover, samples are first subjected to a hydrostatic state of stress in a so-called isotropic consolidation process and then sheared in compression increasing the vertical load. This type of test has come to be called an isotropic triaxial compression test. In nature, only singular points or singular lines are subjected to a cylindrical state of stress. Also, isotropic consolidation is a rather rare event. Usually, the soil is stressed under plane strain, as, for example, in continuous footings, earth dams, natural and artificial slopes, or under other deformation conditions giving a stress distribution with unequal stresses, such as for individual spread footings. Moreover, a rotation of principal stresses is frequently produced when failure ensues.

It has been tacitly accepted in the past that, because of the limitations imposed by nature on the problems to be solved in soil mechanics, the values obtained from the standard triaxial test, in spite of their possible inaccuracies, constitute a sufficiently good approximation. This statement may be true for certain situations, such as simple failure conditions under triaxial compression, but it is being realized more and more that anisotropy in consolidation and non-symmetrical states of stress may, in other circumstances, influence the behaviour of soils to such a high degree as to invalidate the use of the standard triaxial test completely. For example, it seems now to be beyond doubt that stress-strain relations under plane strain are very different from those obtained under a cylindrical state of stress. Both laboratory tests and deformation measurements made on earth- and rockfill dams seem to support this view.

Information on the stress-strain and strength behaviour under such states of stress is scarce, and comparisons between standard triaxial test results and those obtained on tests with varying states of stress, especially plane strain, are much needed. At this stage of development, it would seem that the standard triaxial test will continue to be the basic routine testing technique; therefore investigations should be directed towards the possibility of establishing corrections to be applied to the results of such tests to obtain the behaviour under other states of stress.

To predict soil behaviour on the basis of standard triaxial tests then, it is absolutely necessary to have a thorough knowledge and understanding of the difference in mechanical properties resulting from distinct stress patterns. Only then those problems where the standard tests may still be used, with due corrections if necessary, could be separated from those where, on the contrary, testing simplification cannot be justified. Since the principle of superposition does not hold true in soils, it may be stated that simplification will seldom be possible where deformation problems are involved.

For these reasons, investigations of the effect of anisotropic consolidation and non-cylindrical states of stress on the stress-strain relation and shear strength of soils constitute one of the most promising and most urgent topics of research. Four of the papers presented to Division 2 refer to tests with anisotropic states of stress; one of them reports results of tests on sand and the other three deal with tests on remoulded clays. Their findings are conflicting; two of the papers indicate that anisotropy of stress has little effect on the shear-strength parameters of remoulded clay, whereas the third one gives results showing a very important difference.

The research reported by *Broms and Casbarian (2/9)* deals with the behaviour of a remoulded clay tested as consolidated undrained with pore-pressure measurements. Hollow cylindrical specimens were placed in a triaxial cell which could rotate through the application of a torque. Results show that both the rotation of the principal stresses and the change of the intermediate principal stress have a very important effect on the shear-strength parameters at failure; substantial variations result in the deviator stress, in the coefficient A_f, and in the angle of internal friction ϕ'.

Rotation of the principal axes produces an increase in A_f and a decrease of both $(\sigma_1 - \sigma_3)$ and ϕ'. That is, it leads to a decrease in the strength of the material. Change of the intermediate stress has not such a clear-cut effect. It increases the value of A_f; it first increases and then decreases the value of $(\sigma_1 - \sigma_3)_f$, as the test deviates from the standard and from axial compression passes to axial extension; it considerably increases the effective angle of internal friction.

It would be useful to have stress-strain comparisons between the tests reported and standard triaxial tests. The tests in which rotation of the stress axes was superimposed with an anisotropic state of stress indicate that this effect can be linearly added.

Ladd (2/31) reports on the analysis of consolidated-undrained triaxial tests with pore-pressure measurements performed on saturated, normally consolidated clays under both isotropic and anisotropic consolidation to show what difference in behaviour results from the effect of anisotropy. Some experiments were also performed to find out the effect that the total stress path may have on the undrained strength of such materials.

Findings show that neither the relation between undrained shear strength and major principal consolidation stress nor the angle of internal friction ϕ', obtained from the Mohr-Coulomb envelope, are changed significantly. Major differences appear in the strain at failure, which is considerably smaller, and in the value of the A parameter which, in some of the clays, is shown to be decreased at peak failure by anisotropic consolidation, but considerably increased beyond. It is also shown that the effective stress paths of anisotropically consolidated clays cannot be predicted from standard tests, as would be inferred from Henkel's early hypothesis that the effective stress path represents a unique relation between shear and effective stress. It is concluded that the stress-strain behaviour of anisotropically consolidated samples cannot be predicted from standard test data. The influence of the stress path is specially brought forward by the values obtained for the undrained strength of the same clay following three different stress paths after anisotropic consolidation. The values obtained differ twofold when a shear compression failure is compared with a shear extension failure. A short discussion in the paper considers the possible implication of these results on the $\phi = 0$ stability analysis.

Lorenz, et al. (2/34) refer to the development of a triaxial apparatus in which the three principal stresses can, at least theoretically, be varied at will. It is based on the idea of separately loading the faces of a cube-shaped specimen.

The apparatus has been used to test, under plane strain conditions of stress, a dry sand in loose, medium, and dense states of compaction. Seventy-three tests are reported in which the soil was first subjected to a state of stress $\sigma_1 = \sigma_3$, which varied between 0.5 and 3.5 kg/sq.cm. Then σ_3 was kept constant while σ_1 was steadily increased to failure. Measurements made during the tests included deformations ϵ_1 and ϵ_3 and the induced stress σ_2 for $\epsilon_2 = 0$. Calculations provided the volume change, ϵ_v, the distortion, $\epsilon_d = \epsilon_1 - \epsilon_3$, Poisson's ratio, $\mu_P = \Delta\epsilon_3/\Delta\epsilon_1$, and the tangent modulus of deformation, $E_P = \Delta\sigma_1/\Delta\epsilon_1$. Typical curves are given showing results and an analysis is made of the values obtained, fitting equations to express deformations and moduli as a function of the stresses σ_1 and σ_3. Unfortunately no comparisons are made with results of standard triaxial isotropic tests on the same sand. It appears evident, however, that plane strain behaviour differs from standard test behaviour as can be seen from the stress-strain curves reported. It is shown that Poisson's ratio is independent of relative density and that E_P decreases linearly with deviator stress.

The apparatus is an interesting new development for its versatility in varying at will the three principal stresses. For plane strain it might be easier to control test conditions by using a prismatic specimen encased in a triaxial cell, as has been done elsewhere. The triaxial cell facilitates sample saturation by back pressure and the measurement of pore pressures. It would be valuable to know how such provisions can be made in this apparatus.

Further details of tests would also be welcome as some of the statements made in the author's analysis are not totally clear. The following points of clarification are suggested: (1) A comparison of results with those obtained from standard triaxial tests should be made. (2) An explanation of the unique value obtained for the angle of internal friction should be offered, since it is reported that test results show that the angle of internal friction at maximum volume is uniform and independent of relative compaction. In the opinion of the General Reporter, these are the friction components of the angle of internal friction which do not include the effect of dilatancy as it appears to be very clear for both medium and dense sands, with volume expansion in the latter case. Yet in the paper it is stated that ". . . contrary to triaxial tests, no volume expansion, compared with the initial density, is reached." This expansion, however, should be measured only under the deviator stress, in which case it seems there would not be any fundamental difference. (3) The effect of relative density on the modulus of deformation E_P also requires clarification. The statement that E_P is only slightly dependent upon the porosity seems to contradict Fig. 5.

Shibata and Karube (2/48) report an investigation to compare the behaviour of normally consolidated, remoulded clay under standard triaxial and other stress conditions. For this purpose, a triaxial cell was adapted to hold prismatic specimens to whose faces stresses could be applied in such a way as to vary at will the relation between σ_1, σ_2, and σ_3. Undrained tests with pore-pressure measurements were performed after the clay had been consolidated under an equal all-round pressure varying from 0.5 to 3 kg/sq.cm.

Findings show that anisotropic stressing does not essentially influence shear strength. There is only a small increase which does not seem to be significant for practical purposes. It does significantly influence the stress-strain relation and the pore-pressure development during shear. As has already been shown by others, strains decrease with anisotropy, this decrease being proportional to the difference between the two minor principal stresses.

Pore-pressure development is shown to be only a function of the mean pressure and of the octahedral shear, multiplied by the ratio of the coefficient of compressibility and of the coefficient of dilatancy, irrespective of the relative value of the intermediate stress and of the water content at the start of shearing. Below a certain ratio of octahedral shear stress to mean normal stress, the pore pressure becomes simply a function of the mean normal stress and is independent of the octahedral shear stress.

Description of the method used for applying the intermediate stress is not clear, and details of the apparatus used would be welcome.

Time Effects

It is generally known that, to a greater or lesser degree, the ultimate strength of all engineering materials is a function, among many other factors, of the rate of loading. Furthermore, there are materials, such as some clay soils, in which this effect may be very important and require special attention. The long-term stability of some natural slopes and artificial cuts can only be explained on the basis of a highly decreased long-term shear strength, which has also been called the residual strength. The peak strength and the residual strength obtained in current laboratory tests seem to provide reasonable values for two particular points in the relation between time and shear strength. These points correspond to static short-duration loading and, with reasonable accuracy, to very long-term loading.

Skempton, in the Fourth Rankine Lecture (1964), has explained brilliantly how this long-term time effect develops. He described how field evidence has shown that, to explain slides on natural slopes and artificial cuts in certain soils, angles of internal friction as low as $\phi' = 10°$ and cohesion $c' = 0$ have to be acting, while for other soils no such decrease in strength has been observed. He assigned the difference to the presence or absence of fissures or joints in the clay soils.

The development of test procedures that would permit discrimination between clay soils liable to decrease their strength from those only slightly affected and that, as well, would allow an estimate of possible minimum long-term strength is of the utmost importance. The determination of the residual strength for very large shear deformations is one of those procedures. To complete this study, results would be required for loadings of very short duration, such as transient loadings, and for rates intermediate between short- and long-term loadings. A great advance has been made in this direction and published in international technical magazines and at previous conferences, but much still needs to be done to clarify the effect of time on the behaviour of soils.

Borowicka (2/8) reports on a new test technique designed to distinguish between soils that are highly affected by loads of long duration from those that are not or are only slightly influenced. For this purpose, a sample is first loaded to beyond peak failure in a shear box at constant volume and the test repeated by inverting the shearing load. This double shear test is later repeated, maintaining the normal stress constant for reasons of simplicity. Repeated shearing is continued until an ultimate state is reached.

It is proved that, depending on the colloidal content of the clay and on other still unknown properties, angle of internal friction for ultimate shear strength may vary between its value on first loading and a value less than 10°. This finding agrees with that reported by Skempton.

Schmid and Kitago (2/45) report an attempt to establish a relation between shearing time and shear strength for three fully saturated, normally consolidated clays within a very limited range of time. The clays were tested in a consolidated-undrained condition in the triaxial cell with loading times varying only between 10 seconds and 90 minutes. The results are correlated introducing a new concept called the intrinsic stress. This is a re-statement of a concept used by Taylor many years ago, which is here made equal to the consolidation pressure for normally and preconsolidated clays. Practical implications are discussed.

Shear Strength of Sand

That the stress envelope, or intrinsic line, for sand is not a straight line is well known. The curvature is small, however, and within a limited range of normal stress, it can be considered as straight. Yet, extrapolation beyond such a range is not always warranted. Shearing under high normal pressures introduces further complications due to degradation. Within the range where degradation is not an important factor, the stress envelope of a sand is a function of relative density, the grain surface roughness, the grain size distribution, and the normal pressure under which shear is produced.

De Beer (2/6) focuses attention mainly on the effect of the normal pressure and refers to testing where it was shown to have a very important influence on the measured angle of internal friction of the material. The mean normal stress σ_m is used as the independent variable. For the sand tested, a significant variation of the angle of internal friction was obtained when the mean stress changed from a very small value to 10 kg/sq.cm. For a given relative density, a variation of the tangent angle to the intrinsic curve of the order of 4° to 5° was obtained when σ_m was varied between 1 kg/sq.cm. and 10 kg/sq.cm., and of the order of 10° when σ_m was varied between 0.02 kg/sq.cm. and 10 kg/sq.cm. This last value is of the same order of magnitude as the variation in angle of internal friction between minimum and maximum relative density.

Numerous investigations have proved that the shear strength mobilized by soil increases with shear deformation, reaches a maximum, and then either remains constant or drops down to a lower value called the residual strength. Depending on the nature and mechanism of the field problem, experience has indicated that in some cases the maximum value controls ultimate strength, while in others the residual or some intermediate value is the determinant. There are situations, however, in which the deformation necessary to reach the maximum value may turn out to be incompatible with the problem. In such cases, shear strength mobilization at lower deformations must be taken into account.

Havlíček and Myslivec (2/21), in the second part of their paper, analyse the shear-strength mobilization of two sands and two gravels tested in a shear box, showing how the angle, ϕ', changes with horizontal shear-box displacement, starting from the peak value and changing to the residual value. In identifying the peak and the residual values a nomenclature is used which may be misleading. The peak value is called the static strength and the residual value the dynamic strength, a nomenclature that should be reserved to differentiate between one loading and repeated or transient loading strengths. Shear tests were also performed on gravels composed of flat particles placed horizontally in one series of tests, and vertically in another. Rotation of particles and dilatation in this last series produced an ordinate at the origin, apparently equivalent to cohesion, that is called τ_0 in the paper. To the General Reporter, τ_0 is liable to be a function of the relation of the size of the shear box to the size of gravel particle and more a product of the laboratory than of nature, where the possibility of such an arrangement of particles is very unusual.

To date no systematic study has been made in which the individual effects exerted by the various factors influencing the shear strength of sand have been separated. Yet, specific knowledge of this type should prove very useful in clarifying the behaviour of such materials and, through it, in allowing better judgment in solving practical problems. It may even happen that some currently held ideas of long-standing may prove incorrect.

Kirkpatrick (2/29) reports on a step in the right direction to obtain this specific knowledge. Using laboratory-prepared sands and glass beads, drained triaxial tests were performed to obtain the angle of internal friction ϕ' and the friction component ϕ_f (equal to ϕ' minus the dilatancy friction component) of materials with a certain grain size, and containing particles of uniform shape and roughness. In this way the effect of grain size, as the only variable, and, for each grain size, the effect of porosity, were studied. Contrary to more or less accepted opinion, it was found that ϕ' increases as grain size decreases. The value of ϕ_f instead apparently remains constant, as if it were a particle property, though the test results show a very significant scatter, which in the paper is assigned to causes beyond the scope of the investigation. For a given grain size, the value of ϕ' decreases linearly with increasing porosity and, follows a curved line with relative porosity.

Graded sands, prepared in the laboratory by mixing, in three different proportions with various fractions of each grain size, were also tested. The results obtained, however, are not clear cut. Within the same scatter as before, it would appear that the value of ϕ_f could also be taken as independent of grading, grain size, and relative porosity. On such a hypothesis, all variations of ϕ', for any sand made with particles of similar shape and surface roughness, could come only from the dilatancy component.

The team working on soil mechanics under Professor Roscoe at the Engineering Laboratories of Cambridge University, England, published a paper entitled "On the Yielding of Soils" in 1958. In it an attempt was made to extend Hvorslev's theory on the physical components of the shear strength of soils beyond the peak value and into the ultimate state. The new theory states that, when sheared, soils reach an ultimate state under continuing shear strain in which either the rate of volume change or the rate of pore-pressure change becomes zero. For a given soil, all the stress paths, represented in effective stress–void ratio (p, q, and e) space, whatever the stress conditions leading to ultimate failure, should end on a line called the critical void ratio line. The Cambridge team found convincing experimental evidence to support their theory in undrained tests on normally consolidated and slightly overconsolidated clays, but left much to be proved when the theory was used in drained tests or in undrained tests on highly overconsolidated dilatant clays. Similar doubts arose from the analysis of the behaviour of cohesionless materials.

To fit experimental values into their theory, the Cambridge team has published several papers (1963, 1964), as indicated in their reference list, in which critical analyses have been made of the triaxial tests, emphasizing errors involved in the evaluation of results and discriminating between the energies absorbed by the soil during stressing: dissipated energy, boundary energy, and elastic energy. The paper by *Thurairajah and Roscoe (2/52)* belongs to this series and shows that, in order to fit undrained and drained tests of granular media into the yield theory, the deviator stress obtained from triaxial tests has to be corrected for boundary and elastic energy absorption. The corrected stress paths obtained show that soil yielding lies on an inclined plane containing the e axis in the p, q, and e space.

High Confining Pressures

The ever increasing height of earth- and rockfill dams, and the need for a better insight into the deformations accompanying the very high confining pressures being induced near pile points in sand, together with the full realization that in stiff clays point resistances developed by piles may be very much higher than were previously predicted, all have launched in the last few years a world-wide study of the behaviour of both compacted and natural soils under elevated cell pressures.

Most of the important laboratories, both in the research and the practical fields, have built high-pressure cells and are currently testing soils to discover their behaviour under such conditions. Published results are still few in number, but it is expected that many papers will be written on the subject in the near future.

The paper by *Bishop, et al. (2/7)* is one of the investigations on this subject presented to this Conference. It describes the apparatus, developed at the Imperial College, for the basic research necessary to clarify the analysis of the behaviour of soils under high pressure. One of the questions raised is whether the fundamental equations of soil mechanics hold true without adjustment. The results reported, obtained while performing a multiple-stage undrained test with constant effective pressure ($\sigma - u$) and two widely varied total pressures, σ, one in the low range and the other in the high range, show that the principle of effective stress holds true for the higher confining stress used (about 70 kg/sq.cm.). More tests of a similar nature are said to be in progress. Should their results, as is to be expected, confirm those given in the paper, further research on the subject could be limited to the strength and deformation behaviour of soils under high pressures.

Results of study on this latter aspect show one feature that is common to all investigations reported to date: the deformations required in triaxial compression tests to develop maximum strength are much greater under high pressures than under low pressures. Similar results were obtained in testing other non-elastic materials, such as concrete. Yet, the few existing field observations on earth dams, working under plane strain, appear to show deformations which are much smaller and do not bear a relation to those observed in the triaxial tests. Laboratory tests have shown that under plane deformation, strains to failure are much smaller, but the matter has only started to be clarified and still requires detailed study.

In line with what is already known from concrete, marble, and other porous materials, results of strength to failure tests published to date show systematically a flattening of the stress envelope with the increase of the confining pressure. In granular soils this phenomenon is accompanied by a grain-size degradation, a circumstance that may or may not add to the above trend. In fine soils, for example, for pressures up to 100 kg/sq.cm. no breakage is expected, yet the results reported to date show a flattening for some of them which is more pronounced than in sand and gravel.

There are, however, tests, such as those by *Insley and Hillis (2/23)* referring to a glacial till and by *Wissa, et al. (2/60)* who were concerned with soils stabilized either with hydrated lime or Portland cement (*infra*, p. 21), where no flattening of the stress envelope was observed. *Insley and Hillis (2/23)* describe tests performed on samples of compacted glacial till, 6 inches in diameter and 12 inches high, under a maximum confining pressure of 31.5 kg/sq.cm. The results show that, up to that confining pressure, there is very little effect on the effective shear stress parameters. The most important effect of the confining pressure was observed in the pore pressure developed by the deviator stress. The value of $\bar{A}_f$ at failure increased significantly with confining pressure. No data are given on degradation. The very high effect of moulding water content on the cohesion intercept appears to be rather unusual.

The testing of soils under high pressures poses a number of practical questions which must be answered in order to solve many design problems properly. Among these, probably the most important are the following:

1. To what extent are the deformations obtained in triaxial tests to develop the full shear strength of a soil representative of the actual behaviour of that soil in the field? Should soils in the field require similar deformations to develop full strength? The use of this strength may be open to question as a failure condition may appear at a smaller deformation due to settlement or other soil displacement. Spiral columns in reinforced concrete pose precisely such a problem; piles in soil mechanics may do the same. It may happen that, under certain conditions, full friction resistance is not compatible with full point resistance because the pile settlements required to develop them are very different.
2. Should such cases arise, as some field work seems to indicate, the development of failure criteria based either on deformation or on some other idea may be necessary.

Typical Soils

The systematic study of typical soils covering large areas of the earth's surface is of very great interest because it leads to a better understanding of the behaviour of each material and it eventually allows the solution of practical problems with lower test expense than would otherwise be needed.

Furthermore, soils consist of such a variety of materials that the study of their various kinds and the understanding of their properties represents one of the important tasks of research. That task is further enhanced when regional or world-wide problems are involved.

Debaille and Ghiste (2/14) refer to the properties of a cretaceous chalk that constitutes the subsoil in the region of Mons, Belgium. They have studied the shear strength of the material, in the field with the Dutch static penetrometer and in the laboratory by the triaxial testing of samples that are reported to be rather disturbed because they were obtained by driving a sampler. Tests are also reported on the same material compacted in the Proctor mould and in another smaller mould. Throughout the paper a comparison is made of the penetrometer results, interpreted on the basis of De Beer's paper (1948) entitled "Données concernant la résistance au cisaillement deduites des essais de pénétration en profondeur." The authors tend to force a concordance which is rather difficult to see, especially when one takes into account that the paper itself states that, in such types of soils, the point and the friction resistance measured with the penetrometer leave much to be desired and are of very doubtful value. Furthermore, to fit results in one of these comparisons, a cohesion c' with an average value of 1.27 kg/sq.cm. was discarded as too small and variable.

In this paper the values of c' and ϕ' were determined for both saturated and unsaturated soils in quick tests with pore-pressure measurements. It would be very useful, to clarify the results, to know which procedures and methods were used to perform the tests and measure pore pressures as certain of the results given which relate the effective angle of internal friction and porosity are at variance with those found elsewhere in other types of soils.

Peat is a soil with properties which do not fit into the framework of knowledge built up for other materials in the field of soil mechanics. It is a special type of material with properties which are very peculiar. The understanding of these properties and the devising of means to determine them is very important in places where a considerable thickness of peat covers the earth's surface causing particular problems in the construction of roads and lightweight structures. Its very low strength and exceedingly high deformation requires close study to determine its properties, since necessarily it must be used near the limit of failure.

Hanrahan and Walsh (2/19) report on the methods used and the values obtained in a laboratory investigation to determine the shear strength and settlement characteristics of peat under varying conditions of stress and strain, including measurements of the modulus of elasticity and Poisson's ratio. Vane tests were used to obtain the shear strength. Settlement was studied with samples moulded in large tanks and strains were measured to study settlements and surface displacements. The modulus of elasticity was measured in unconfined compression tests and Poisson's ratio derived from the relation of E and G, this last value being obtained from the vane tests.

Schultze and Horn (2/46) report on a systematic study made at the Technische Hochschule of Aachen, Germany, covering various aspects of the engineering behaviour of silts. Silts are very common soils. Most of them have physical properties located within a narrow range and, besides, those of Aeolian origin frequently form deposits which are very uniform.

This paper deals only with the shear strength of samples remoulded at a water content higher than the liquid limit. Both drained and undrained triaxial tests were carried to an unusually high deformation, exceeding 30 per cent, to reach the maximum deviator stress. In undrained triaxial tests with pore-pressure measurements, failure was determined at both maximum obliquity and maximum deviator stress, this last criterion being adopted for analysis. It was found that, for both types of tests, the effective angle of internal friction is constant and the cohesion is zero, irrespective of preconsolidation.

The zero cohesion intercept is said to be due to dilatation of the silts. This statement needs further explanation. The General Reporter has had the opportunity to test many silts with results, based on maximum obliquity (Mohr-Coulomb envelope), which appear to be at variance with those found at Aachen. In a paper published in the *Proceedings* of the Second Panamerican Conference (Moretto, *et al.*, 1963), for example, it is indicated that, because of silt dilatation, the effective angle of internal friction varies with the degree of compaction or preconsolidation pressure, a finding that has been repeatedly observed and that seems to contradict that of *Schultze and Horn (2/46)*. Furthermore, a small cohesion intercept is always present in our tests for preconsolidated silts.

Schultze and Horn (2/46) also report on a study of the effective stress path during undrained triaxial testing. Assuming the maximum deviator stress as a failure criterion and counting on a zero cohesion intercept, a single undrained triaxial test with pore-pressure measurement is proposed to obtain all the shear parameters of silts. The effective angle of internal friction is obtained by joining the point of maximum deviator stress with the origin. The true cohesion and true angle of internal friction could then be obtained by the following reasoning: since the maximum obliquity condition appears significantly early in the test, the stress path going from this point to the maximum deviator stress point lies on Hvorslev's envelope, because that portion of the path represents impending failure at constant water content with varying effective pressure on the plane of failure. It apparently develops clearly because of the effect of silt dilatation in decreasing the pore pressure and it provides a new method of obtaining the physical components of that material.

Varved clays are important and common materials in many places in the northern hemisphere. They are usually very soft, presenting serious practical problems both in failure and settlement. For these reasons, a thorough knowledge of their behaviour is especially important. *Townsend, et al. (2/54)* report an inquiry into the behaviour of such clays when tested, consolidated and undrained, in the triaxial cell, with special emphasis on the pore pressure built up in the two materials forming the varves, and on the effect of the water migration, required to equalize pore pressure between them, upon the shear strength of the soil.

Immediately after the application of a shear stress, pore pressures are higher in the clay layers, because of their higher compressibility, than in the silt layers. With time and under constant volume, equalization is produced by consolidation of the clay layers and softening of the silt layers, the final strength being a function of such contrary effects, of which the softening of the silt layers is shown to be the most important. Theoretical interpretations of these effects on shear strength, both in terms of total and effective stresses, are presented and compared with experimental results. Some of the reported experimental results are rather unusual and, because of this, open to question. For example, the effective angle of internal friction given for a soil made exclusively of the silty material composing the varved clay tested appears to be exceedingly high, while that for the clay material seems to be rather low.

Stabilized Soils

Soil studies in highway engineering have developed along a line which, in many details, diverges from fundamental ideas currently held in other branches of soil mechanics. Fostered by the necessity of solving urgent practical problems of a very complicated nature on the basis of rather simple tests performed with easy routine procedures, highway engineers have developed an empirical "know-how" which has connected soil behaviour, as an integral part of pavement, with simple soil tests. Empirical "know-how" has been and will always be an important part of engineering knowledge; its main shortcoming is that extrapolation beyond its limited range of validity proven by experience is very seldom possible without assuming unknown risks. On the contrary, knowledge based on fundamental ideas and on general laws of physics allows a deeper insight into the problems and a much safer extrapolation beyond the boundaries of existing experience.

Development of such rational knowledge in soil mechanics requires the use of theories expressing structural behaviour on the basis of fundamental soil parameters, obtained from the strength to failure under the stressing condition developed under pavements, and of the stress-strain relation arising under such conditions. These parameters are not necessarily the usual ones obtained when testing soils under static loadings. They may require consideration of dynamic loading as well.

Visual observation shows that soils stabilized with cementing agents acquire the texture of soft rocks when compared with the soils from which they originated. Used mainly in highway work, their mechanical properties, as has been said, have been measured mostly by empirical procedures used in such branches of engineering practice to emphasize specifically the quality of the materials as integral parts of pavements. The determination of the effective strength parameters by use of modern triaxial testing techniques has been less common.

The paper by *Fossberg (2/18)* is a report on some fundamental properties of a lime-stabilized clay in which a comparison is made between the effective strength parameters obtained under static loading conditions first for a highly plastic clay soil and then for the same soil stabilized with varying percentages of hydrated lime for different curing periods. All samples were tested in a saturated or nearly saturated state. Consolidation and suction properties, as well as permeability and box shear drained failure strength, are compared.

Lime stabilization produces cementation and has the same effect on the soil as a pre-consolidation load, which increases with lime content and curing period. By the same token, it increases suction, decreases permeability, and increases shear strength. Based on Hvorslev's parameters, adding lime has an immediate effect of increasing the true angle of internal friction and the true cohesion of the soil. Yet, time of curing increases only the cohesion, the true angle of internal friction remaining practically constant.

Wissa, et al. (2/60) report on the effective strength parameters of two soils, a silt and a clay, stabilized with either varying quantities of hydrated lime or varying quantities of Portland cement and allowed to cure moist for periods ranging from 4 days to 4 months. Undrained triaxial tests with pore-pressure measurements were performed for consolidation pressures varying from a small value to 60 kg/sq.cm. The following findings are reported: (1) The Mohr-Coulomb envelope is a straight line throughout the whole range of consolidation pressures. (2) Stabilization increases both the effective angle of internal friction and the cohesion intercept, its effect depending on soil type and the amount and type of the stabilizing agent used. (3) Increased curing time does not affect the angle of internal friction, it only increases cohesion, a finding which agrees with that reported by *Fossberg (2/18)*. (4) Soil rigidity, measured by the initial modulus of deformation, was shown to be proportional to the deviator stress, increasing with consolidation pressure and soil improvement. (5) Stressing beyond peak deviator stress leads eventually to a state at which the pore pressure and shear stress remain essentially constant with further straining. At this ultimate condition, cementation has been destroyed and only internal friction remains. The ultimate angle of internal friction obtained is larger than the peak angle of internal friction, a finding which is difficult to understand. (6) Pore-pressure development was normal; however, pore-pressure response to variation in consolidation pressure is rather unusual, and is attributed to a so-called effect of mineral skeleton on such response. This abnormal behaviour is at variance with observations made in the laboratory of the General Reporter's firm on saturated soft rock where a full response has always been obtained.

Angle of Internal Friction versus *Void Ratio*

In their book *Traité de mécanique des sols* (1956), Caquot and Kérisel showed that a limited relation existed between void ratio and angle of internal friction of sands and they expressed that relation with the formula $e \tan \phi' = 0.5$ to 0.6. *Caquot and Kérisel (2/11)* now propose to extend this relation to clays and other saturated soils and, at the same time, correct the formula for sands to read: $e_f \tan \phi' = 0.45$ to 0.55, in which e_f is the final void ratio on the failure plane.

That a rough relation may exist between effective angle of internal friction and void ratio in sands is easy to understand. Because of the mechanism of failure, the value of ϕ' in sands depends to a great extent on relative density and, since maximum and minimum void ratios vary within a relatively narrow range when all possible sands are considered, within that range of variation and to the extent to which ϕ' depends only on relative density, $\tan \phi'$ is a direct function of e. The possibility of extending such a type of relation to clays is not so evident. Interlocking is mainly responsible for the relation between relative density and $\tan \phi'$ in sands. But interlocking plays quite a different and a much smaller role in clays, and dispersion is so high that in the opinion of the General Reporter any such relation loses much of its possible practical value.

Progress between Conferences

Soil mechanics, as a branch of civil engineering, has matured sufficiently to have passed through the early stage of spectacular advancement. Most of the fundamental knowledge of its framework is now generally well developed and progress is now measured in the gradual transformation of theories and of the furthering of the experimental determination of material properties, with the aim of a better interpretation of the variety of stressing conditions that are produced in nature and of the manner in which they are withstood by soils. All this work has as its main purpose the closing of the enormous gap still existing between theory and practice.

The many papers written for conferences, meetings, symposia, technical magazines, and bulletins since 1961, including those that constitute the subject matter of this Report, have unquestionably signified advances in the improvement of methods of testing and in the understanding of the behaviour of soils when stressed to their failure load under static loading. Yet, the task ahead is still so formidable and the contributions continuously being made to this slow progress so great in number and frequently so fragmentary in nature that a comprehensive assessment would require a length of time beyond the reach of this General Reporter. For these reasons, this discussion of progress between conferences is limited to a short enumeration of the principal subjects covered by those contributions without itemizing the papers in which they were made so as to avoid involuntary and inevitable omissions.

Laboratory techniques have continued to improve and new developments have been reported in the design of both triaxial and direct shear apparatuses. Analysis of particulars of testing, such as the effects of non-uniformity of distribution of both effective stresses and pore pressures, and details of the influences derived from stress history, stress paths, and rate of strain were made by various techniques of testing. Design of apparatuses for testing soils under high pressures, new devices for pore-pressure measurements, and methods for testing under anisotropic states of stress are probably the subjects that received more attention. Particularly important progress was made in the design and construction in Mexico and in Germany of triaxial equipment to handle very large samples under pressures comparable to those existing in high earth- and rockfill dams and the development in England of a method to eliminate friction at the end caps restraining the samples in triaxial tests.

Fundamental studies of the shear strength behaviour of soils were also continued. Some of them pursue a general theory which would give the mechanism of the ultimate shear strength behaviour, both for remoulded clays and sands. Undisturbed soils were very seldom included in these investigations. Other studies refer to methods of obtaining the components of the shear strength of clays, either based on Hvorslev's criteria or supported by new ideas. In addition, investigations were made of the frictional characteristics of soil minerals and of the shear strength of chemically purified clay, minerals, etc.

The representativeness of laboratory undrained testing of samples of saturated clays was the subject of two important papers published in 1963, one by Skempton and Sowa in *Géotechnique* and the other by Ladd and Lambe in the *Proceedings* of the Ottawa Symposium on Laboratory Shear Testing of Soils. In them, following the same line of thought, analyses were made of the reliability of unconsolidated-undrained test results, obtained from undisturbed samples, in relation to the shear strength of the soil in the ground, arriving at somewhat conflicting conclusions as to the importance, on the obtained laboratory strength, of the soil disturbance coming from the stress change produced by sampling.

As has been common in the past, the study of strength properties of typical soils was the subject of several of the papers published between conferences; decomposed rocks, silts, and compacted soils were among the materials which received preferential attention.

The effect of varying the stress path, and especially that produced by reaching failure under a so-called anisotropic state of stress, with particular reference to plane strain occupied the attention of many investigators. The study of this problem seems to have made a good start.

Shear-strength behaviour of soils and rock fragments under high confining pressures has been one of the favourite subjects of research. The papers published on the subject are not numerous but interest in the problem is great and has led to the construction of many high-pressure cells so that most good soil mechanics laboratories are now provided with at least one such device for testing fine-grained soils.

Great progress was made on the study of the long-term strength of overconsolidated fissured clays, particularly through the Fourth Rankine Lecture which elucidated the mechanism of the problem and supported results with a large number of case records. Very little was published, by contrast, on the strength behaviour of non-saturated soils, an outstanding problem in the routine of everyday soil mechanics practice, about which present knowledge is slight. The few papers published on the subject refer mainly to the application of the principle of effective stress to such soils, but practically no progress was made on representative testing in relation to field soil behaviour.

Subjects for Panel Discussion

The following subjects are proposed for panel discussion:

1. Shear-strength behaviour of soils under an anisotropic state of stress with special reference to plane strain, making comparisons with the behaviour of the same soils when subjected to standard triaxial tests.

2. Shear-strength behaviour of soils under high confining pressures, differentiating cases where particle degradation may be a factor (sand and gravel) from those in which negligible or no degradation takes place (silts and clays).

3. Effect of very large sample size on the shear strength of fine soils, coarse soils, and rockfill materials, with reference to the effect that large particle-size degradation may have on that strength when tests are run under sufficiently high confining pressures.

4. Shear-strength behaviour of non-saturated soils with special reference to methods of testing to obtain the strength parameters to be used in foundation design, taking into account possible change in strength due to consolidation, expansion, and/or wetting during or after construction, but excluding consideration of problems related to small buildings on expansive clays.

5. The subjects just described constitute some of the many topics where information appears to be in one of the three following states: (*a*) sufficient knowledge exists to allow a collection of information which would prepare the way for further advances; (*b*) a certain amount of work has been done on the subject and a *résumé* of conclusions is necessary to orient whatever new investigations are convenient; (*c*) so little is known about the problem that a start should be made to induce research to cover the gap. This last statement applies especially to the fourth subject of discussion. A large number of the problems faced in soil mechanics practice refer to the design of simple foundations placed on unsaturated soils. Yet, not much is known of the shear-strength behaviour of such soils, a problem that constitutes one of the fields of practical research that needs more attention.

Closing Remarks

It was said at the beginning of this Report that soil mechanics has gone a long way towards devising methods to determine the shear-strength parameters in the field and, through adequate samples, in the laboratory.

The rational framework built up by the science of soil mechanics indicates solutions for problems controlled by failure under static loading—which constitute by far the great bulk of practical problems, solutions that require the determination of the shear-strength parameters of the soils involved. Yet, the use made of progress in sampling and testing is rather limited and a really alarming amount of practical work is performed ignoring it totally. Non-scientific solutions, such as many of the widely popular penetration tests, take their place.

This situation arises partly from sampling difficulties caused by the nature of soil and the impossibility of making rational field tests, partly from an exceedingly false prestige acquired by the penetration tests, even for soils where sampling is cheap, efficient, and easy, and partly from a need for extended studies of the behaviour of typical soils in nature and of the representativeness of both field and laboratory strength tests. Furthermore, a great number of the laboratory studies on the shear strength of soils ignore the possible relation of their results to field behaviour.

It is hoped that those contributing discussions to this session will keep in mind the shortcomings of this situation. Soil mechanics has now become an integral part of everyday civil engineering from which the profession expects the efficient solution of their problems, simple and complicated, routine and spectacular, dull and dramatic, with scientific effectiveness, so as to provide the cheapest safe solution which would make good use of existing natural possibilities.

Sujets de discussion pour le comité d'experts

Les sujets suivants sont proposés:

1. Le comportement des sols à la résistance au cisaillement, sous un état anisotropique de contrainte avec référence spéciale à la déformation plane, établissant des comparaisons avec le comportement des mêmes sols lorsque soumis aux conditions d'essais triaxiaux standards.

2. Le comportement des sols à la résistance au cisaillement sous des pressions œdométriques différenciant les cas où la dégradation des grains peut être un facteur — tel que pour le sable et le gravier — de ceux n'accusant qu'une faible ou aucune dégradation tels que les limons et les argiles.

3. L'effet produit par une forte dimension des échantillons sur la résistance au cisaillement des sols à grains fins, sols à grains grossiers et matériaux de remblai rocheux relativement à l'effet exercé par la dégradation importante de la dimension des grains sur cette résistance lorsque des essais sont effectués sous des pressions œdométriques suffisamment élevées.

4. Le comportement des sols non remaniés, à la résistance au cisaillement, avec référence spéciale aux méthodes d'essais en vue d'obtenir les paramètres de résistance devant être utilisés pour le calcul des fondations — en tenant compte d'un changement possible de la résistance dû à la consolidation et l'expansion ou au mouillage durant et après la construction, mais abstraction faite des problèmes relatifs aux petits bâtiments sur argiles d'expansion.

5. Les sujets décrits ci-avant constituent quelques uns des multiples thèmes où l'information semble se classer dans l'un des trois états suivants: (*a*) Connaissances suffisantes disponibles permettant de rassembler les renseignements qui favoriseraient l'avancement. (*b*) Un certain travail a été accompli sur le sujet, et un *résumé* des conclusions est nécessaire afin d'orienter toute nouvelle recherche. (*c*) Les données sur ce problème étant assez rares, il serait sage d'encourager les recherches afin de combler cette lacune. Cette dernière déclaration s'applique tout particulièrement au quatrième sujet de discussion. Un grand nombre des problèmes rencontrés dans la technique de la mécanique des sols se rapporte à la conception de fondations simples reposant sur des sols non remaniés. Cependant, on ignore à peu près tout du comportement de tels sols — à la résistance au cisaillement — un problème qui fait partie d'un des domaines de la recherche pratique et qui nécessite la plus grande attention.

Remarques de la fin

Il a été dit au début de ce rapport que la mécanique des sols a largement contribué à la conception de méthodes pour déterminer les paramètres sur le chantier et par la suite en laboratoire à l'aide d'échantillons adéquats.

Le cadre rationnel établi par la science de la mécanique des sols présente quatre problèmes qui sont contrôlés par la rupture sous charge statique — qui constitue de beaucoup la majeure partie des problèmes pratiques — solutions qui exigent la détermination de la résistance au cisaillement des paramètres des sols en question. Jusqu'à présent le prélèvement d'échantillons et l'essai n'ont que très peu profité du progrès, et une proportion alarmante du travail pratique est effectuée sans en tenir compte. Les solutions non scientifiques, tels que plusieurs essais de pénétration fort répandus, leur sont préférés.

Cet état de chose est en partie imputable aux difficultés de prélèvement d'échantillons suscitées par la matière du sol, et de l'impossibilité de procéder à des essais de chantier rationnels; une autre raison pour cet état de chose serait la fausse considération acquise par les essais de pénétration, même dans le cas des sols où le prélèvement d'échantillons est peu coûteux, efficace et facile; enfin, une troisième raison viendrait de la nécessité d'études approfondies sur le comportement de sols typiques de nature, et la représentativité des essais de résistance effectués sur le chantier et en laboratoire. De plus, un grand nombre d'études en laboratoire sur la résistance des sols au cisaillement ignore la relation possible de leurs résultats avec le comportement sur le chantier.

Il est à espérer que ceux qui ont participé à ces discussions se souviendront des désavantages de cette situation. La mécanique des sols est maintenant une partie intégrale du génie civil courant, sur lequel la profession compte afin de résoudre efficacement leurs problèmes, simples et compliqués, courants et spectaculaires, ternes et dramatiques, avec une efficacité scientifique, afin de fournir la solution la plus sûre et la moins coûteuse — faisant usage des possibilités naturelles existantes.

II. SOIL DEFORMATION UNDER STATIC LOADING AND SOIL BEHAVIOUR UNDER DYNAMIC LOADING

In the preceding section of this General Report it was stated that practical problems in soil mechanics can usually be separated into two kinds, depending on whether they are controlled by failure or by settlements. This section of the report is devoted to those papers which refer to theories and soil properties relating to soil deformation under static loading and general soil behaviour, shear strength, and deformation under dynamic loading.

Soil Deformation under Static Loading

Settlement problems in soil mechanics have proved to be some of the most difficult confronting the civil engineer. When the deformation parameters are to be obtained by laboratory testing, difficulties start with the sampling operations. Most soils are very sensitive to these operations and while, with good sampling, the shear strength is only slightly affected, even the best sampling methods introduce a significant change in soil deformation characteristics. In addition, the oedometer test commonly used in routine laboratory work to obtain these parameters is not necessarily suitable for that purpose because it is only truly applicable to a very limited range of problems where a confined, uniformly loaded, relatively thin layer of soft, normally consolidated clay is involved. Furthermore, some of the field tests frequently used in practice (dynamic penetration tests, etc.) provide results which are, at best, only rough images of the soil's behaviour.

For these reasons, the accuracy of the solutions for problems of deflection or settlement is generally much less than for those giving the failure load and, for problems controlled by settlements, more uncertain. Moreover, the precision expected and sometimes required, in order to meet the design standard of the superstructure, is frequently greater.

Other factors are also responsible for this disagreement. The most important of these stems from the fact that stress conditions developing in the field are very complicated and difficult to reproduce in the laboratory in routine practical studies of their relation with strains. Since the principle of superposition is not applicable, extrapolation from simple tests, not reproducing field stress conditions, becomes uncertain.

Of the sixty-one papers assigned to Division 2, twenty-five deal with some aspect of soil deformation under static loading.

Extensions of Terzaghi's Theory of Consolidation

Terzaghi's theory of one-dimensional consolidation is based on well-known assumptions about soil parameters, such as constant coefficient of permeability and constant coefficient of volume change, and a given set of well-known boundary conditions. By changing these assumptions and/or conditions, mathematical solutions for various other situations for one-, two-, and three-dimensional consolidations have been obtained to express how the pore pressure and the degree of consolidation change with time. Three papers in this division refer to these extensions of Terzaghi's theory.

De Leeuw (2/32), starting from the general differential equations for three-dimensional consolidation, found a mathematical solution applicable to cylindrical bodies stressed symmetrically under plane strain and valid for any boundary condition. An example is given applying the solution obtained to a case with vertical sand drains. *Martins (2/37)* presents a solution for the unidimensional consolidation of a layer of clay with a linearly varying coefficient of permeability, obtained by both analytical and numerical methods. His results are compared with Terzaghi's solution, using a weighted average coefficient of permeability, and show that, provided the average coefficient of permeability is determined while considering the clay layer as a stratified mass, the difference between the exact and the approximate solutions is very small. Isocrones and degree of consolidation–time factor curves for various rates of permeability variations are given. Finally, *Verigin (2/57)*, using the fundamental principles of Terzaghi's consolidation theory, develops analytical solutions for the process of consolidation and the determination of settlement in a half-space of saturated soil acted upon by lineal and surface loads of rectangular and circular shape when the surface itself is a plane of drainage.

New Theory of Consolidation

As a theory trying to predict the dissipation with time of the stresses produced on the liquid face of saturated soil, Terzaghi's one-dimensional theory of consolidation has played the well-known, outstanding, and uncontested role of initiating the science of soil mechanics and of introducing the concept of effective stress. Universally used to compute the magnitude of settlement and its progress with time, the outcome has been variable and controversial. Even if field conditions coincide fairly well with the fundamental theoretical hypothesis, the comparisons between computations and measurements are not always satisfactory because, among other reasons, the theory ignores secondary time effects. When used beyond its limited range of validity, with preconsolidated clays for example, it usually leaves much to be desired, often predicting magnitudes of settlement several times larger and rates of settlements much slower than those later observed. To obtain better results a search is being made for new theories or for adjustments to the old one.

The paper by *Marsal (2/36)* is an attempt to develop a new theory of consolidation based on the analysis of the properties and behaviour of the soil grain skeleton, without giving consideration to the actions of either the gas or the liquid phases of the material.

Starting from the hypothesis that the soil skeleton constitutes a discrete isotropic body made of grains having a variety of shapes and sizes, statistical methods are used to analyse soil deformation for one-dimensional compression using a mechanical model in which the actions of external forces on particles are divided into two parts: constant intergranular forces; erratic constant forces.

With this mechanism, making use of a stochastic method of analysis, the one-dimensional displacement of a particle is determined through a probability function of time and, by simple extension, the same function is used to measure the change in grain concentration of the soil mass, for the purpose of obtaining the one-dimensional compression or settlement. This function, being the fundamental solution of the Fokker-Plank equation, leads to the statement of the differential equation relating grain concentration, one-dimensional co-ordinate, and time. By simple transformation, grain concentration is converted into strain and an expression for percentage of consolidation is obtained as a serial function of dimensionless factors called diffusion and drift time factors. Finally, because of difficulties in convergence of the series, a simplified expression is proposed instead.

No applications of the new theory are given. It is only remarked that the solution obtained indicates that the process is greatly influenced by the thickness of the layer, and it is further pointed out that this result is of importance for application, since the compressibility parameters of soils are obtained by testing samples whose thickness is very small compared to that of the stratum in the prototype.

Effect of Stress Condition on Settlement

The oedometer test represents a very particular stress condition, giving results which relate only the main principal stress to the soil deformation, while ignoring the possible effects of the other principal stresses defining the field state of stress. To obtain the deformation parameters required to take these effects into account, tests may be needed in which the stress condition is completely controlled during the whole period of testing, so that the parameters may be referred to it and later be extrapolated to other states of stress. For this purpose, the most versatile apparatus at present appears to be the triaxial cell.

Akai and Adachi (2/2) refer to consolidation tests performed in a triaxial cell, with some modified details, under isotropic and K_0 states of stress (unidimensional consolidation) using remoulded samples in which only radial drainage was permitted in order to avoid a stress gradient along the sample, not accounted for in the stress analysis.

Analyses are made for variations with time of volume change, axial strain, pore-pressure dissipation, and the ratio $K = \sigma_3'/\sigma_1'$ necessary to obtain zero lateral strain, and comparisons are made between isotropic and unidimensional consolidation. For the same major vertical stress, vertical strain in unidimensional consolidation is found to be about three times that developed under isotropic conditions. Relating volume change to mean effective stress σ_m', it is shown that the existence of a deviator stress results in a larger volume change in unidimensional consolidation than in isotropic consolidation. This difference is assigned to an effect which, in the paper, is called the negative dilatancy effect. Moreover, the vertical strain, expressed as a function of σ_m', is three to four times larger in unidimensional than in isotropic consolidation. Finally, the total settlement ρ, or axial strain in unidimensional consolidation, is considered equal to the sum of the isotropic settlement ρ_c produced by σ_m', plus the pure distortion settlement ρ_γ, originated by the deviator stress, plus the negative dilatancy settlement ρ_d. For the clay studied it is stated that the percentage contribution is as follows: $\rho_c = 30$ per cent, $\rho_\gamma = 60$ per cent, and $\rho_d = 10$ per cent. It is not very clear how these effects are obtained from the tests and a further explanation would be needed.

After consolidation, samples were taken to failure. Two graphs are included that show the ratios of deviator stress and pore pressure to mean pressure as a function of axial strain; a third graph gives the stress paths followed in isotropic and K_0 triaxial tests. It is shown that the strain and the pore-pressure parameter, A_f, are considerably smaller in unidimensionally than in isotropically consolidated samples. On the other hand the angle of internal friction is not affected by the stress path.

It seems apparent that, to determine the deformation that a soil will undergo under a given change of stress may require making the soil being tested follow, as closely as possible, the same stress path to which the material will be subjected in nature. Compressibility is a consequence of the sum of distortion and volume change effects and both are a function of the principal stress ratios. In elastic materials, the principle of superposition permits the derivation of the deformation or total strain in a given direction as a function of the state of stress and of two constant and independent parameters: the modulus E and Poisson's ratio ν. In non-elastic materials such deduction is not possible because both E and ν vary with the state of stress. It would appear, therefore, that the only rigorous way to obtain the true deformation parameters consists in following the path of the stress change. For that purpose the soil, after sampling, must be first brought back to the stress prevailing in the ground and from then on it must follow the changes it will undergo on loading.

The procedure is easy to describe, but very difficult to perform. For this reason, simplified methods which would lead to approximate results, sufficiently accurate for practical purposes, will always provide an important contribution to settlement computation. The one-dimensional consolidation test, performed with an oedometer, is one of these simplified procedures.

The development of any simplified solution implies the use of limited stress ranges, such as the range of variations in principal stress ratio, and the need to state the scope of its applicability.

The paper by *Janbu and Hjeldnes (2/24)* is a contribution to these problems. It refers to soils, such as clean sands and saturated clays (the latter stressed in undrained conditions), whose shear strengths under the states of stress that are studied in the paper are defined by one parameter, namely, the angle of internal friction or the undrained apparent cohesion. Three problems are analysed.

First, a study is made of the range of the principal stress ratio $K' = \sigma_3'/\sigma_1'$ as a function of the percentage of shear strength mobilized, expressed through a safety factor F. It is then suggested that, when no rotation of stresses is produced, triaxial anisotropic tests may be made with a constant stress ratio, equal to the mean value between the initial K_0 condition and the final effective stress ratio K', to determine the compressibility. Furthermore, it is shown that, theoretically, a direct linear relation exists between K_0 and the ratio (s/p) of undrained shear strength to overburden pressure for normally consolidated clays.

Secondly, a study is made of the elastic deformations under an anisotropic state of stress, with a principal stress ratio K, showing that for elastic materials the shear stresses (distortion) are responsible for a major part of the total strain developing along the direction of the major principal stress. Furthermore, it is shown that the elastic theory appears to be applicable for determining the compression of saturated clays under undrained conditions (initial settlement) and that it explains why the modulus of deformation in the field is from three to four times higher than that obtained from unconfined compression tests.

Finally, results are reported of drained tests on two sands, performed under constant stress ratios, $K' = \sigma_3'/\sigma_1'$, to determine stress-strain curves and to derive the tangent modulus as a function of the stress ratio. It is shown that, for the materials tested, the elastic theory, introducing in the equations the pertinent tangent or secant modulus of deformation, may be used to determine settlement as long as the stress ratio $K' \gtrless 0.55$. For lower values of K', falling within the usual practical range, the divergence appears to be too high.

General Stress-Strain Relations

A fundamental study of the stress-strain pattern of soils is of engineering interest when related to the solution or the understanding of deformation engineering problems or the behaviour in shear failure of the materials involved. Since both are very important matters, the analysis of tests is generally accompanied by a qualitative description or a record of the stress-strain behaviour.

Unconfined compression and triaxial tests, either drained or undrained, show curved stress-strain relations which, on first approximation, can be assimilated to parabolas, a condition that is common to most non-elastic materials. Quantitative analysis, however, has centred in the past mainly on confined compression tests because the most important deformation problems of soil mechanics (settlements) have been analysed mostly on the basis of such tests.

The paper by *Chaplin (2/12)* deals with the analysis of such a relation in undrained triaxial tests. After showing that stiffness in sand increases exponentially with relative density, stress-strain relations for various other granular materials are given in which it is shown that a parabolic law governs the shape of the greater part of the respective curve.

Brinch Hansen (2/20), using as a basis some considerations about the movement in nature of soil particles, developed an expression for the ratio between shear stress and strain which, through substitution of numerical constants, is eventually changed by three empirical equations applicable, respectively, to soils behaving as do soft clays, loose sands, and dense sands. These equations are used to state a definition of failure for materials which do not exhibit a peak strength. In spite of being arbitrary, it appears worth quoting: "Failure corresponds to the load at which the deformation is twice the deformation at 90 per cent load." Furthermore, for the same types of soil, the deformation at half the failure stress is only 10 per cent of the deformation at failure as defined above. Consequently, where the laboratory tests are directly representative of field behaviour, a factor of safety of 2 will limit deformations to approximately 10 per cent of the failure value. The equations mentioned are later substituted by other empirical expressions of a simpler and more explicit nature. And other empirical expressions are proposed for the volume and linear deformations of isotropic and anisotropic compression, for the oedometer test and for the triaxial test.

To a great extent deformation problems are solved by resorting to the equations of the theory of elasticity. This theory is based on a non-frictional, purely cohesive material whose properties are not time dependent, so that the effect of both internal friction and time on the stress-strain fields originated by loading is not taken into account.

The existence of internal friction means, among other things, a difference in behaviour in tension and compression which is not considered in the theory of elasticity. When this effect is taken into account, it is demonstrated that the relation between shear stress and strain is not an invariant, G, but a modulus that is dependent on the average normal stress, as soil shear testing clearly indicates, and that shear introduces a change in volume. By the same token, volume change depends not only on σ_m but also on the shear stress. The rheological effects, derived from what could be called the time action, further complicate the problem.

The paper by *Vyalov (2/58)* refers to plasticity and creep in cohesive media and thus deals with these subjects. It is shown that, neglecting the time action, the stress-strain field of frictional materials is given by a curve in the space $\sigma - \epsilon - \tau$ whose projections in the $\sigma - \epsilon$ and $\tau - \epsilon$ planes correspond to unconfined compression and pure shear. Starting from the general Volterra-Boltzmann equations, simplifying assumptions are made by considering, first, that projections in the $\sigma - \tau$ plane are straight lines, as soil experience shows, second, that projections in the $\sigma - \epsilon$ plane for different τ values follow an exponential law and, third, that the tensile strength H is a constant independent of soil state. From these assumptions a new general equation relating stress to strain is found and a simplified solution obtained. The new equation is then further developed introducing the rheological effect resulting from the time factor and various elasto-plastic behaviours.

Wroth (2/61) states that in another paper, still to be published at the time this General Report was written, the use of certain exponential functions is suggested, as a mathematical model of the shearing behaviour of granular media, to take into account the actual stress-strain characteristics observed for soils, instead of the conventional assumptions of perfectly elastic or perfectly plastic behaviour. In this paper the use of these exponential functions, which had been applied to shear tests, is extended to triaxial tests using as parameters the mean normal stress p, the deviator stress q, the void ratio e, and the deviator strain $\epsilon = \frac{2}{3}(\epsilon_1 - \epsilon_3)$. It is shown that, for both undrained and drained tests on normally consolidated clay, a unique relation exists between q/p and ϵ and that, for the same value of ϵ, the values of q/p, for different degrees of drainage, are directly proportional for tests starting at the same void ratio e_0, with a particular reference made to that relation when fully drained and undrained tests are compared. For tests not complying with this condition, it is suggested that the value of the artificial strain parameter $(1 + e_0)\epsilon$ be used instead to obtain such proportionality.

Pore Pressures

Pore-pressure measurement is, to a certain extent, hampered by the volume change required to activate the measuring device. Though volume change is compensated for by using one of the various methods developed to maintain the volume constant, a temporary local consolidation of the soil results and true instantaneous measurements are not possible. When these types of measurements are required, the best technique is to couple high sensitivity with pre-setting of the device as closely as possible to the expected pore pressure.

Pore-pressure measurements in oedometer tests are especially delicate and require that volume change and time lag for volume compensation be minimized as much as possible. Otherwise, the disturbance produced becomes large enough to influence seriously the pore-pressure-dissipation process developing during consolidation. Furthermore, in oedometer tests drained from the top and connected to a pore-pressure measuring device at the bottom, the process of consolidation is affected by the flexibility of the measuring system because some water must flow into the system before a pressure change is recorded. Therefore, the measured pressures are at variance with those expected from theory for an ideal no-flow measuring device. Experience shows that there is a delay in maximum pressure build-up and that its value never reaches the theoretical maximum.

Perloff, et al. (2/43) develop a mathematical solution by which the effect of the volume change in the measuring system is taken into account in the evaluation of pore-pressure build-up throughout the consolidation test specimen as a function of time. It is deduced that the pore water pressure at the base goes to zero instantaneously upon application of the load and then increases to a peak value before dissipating. In this process, approximately the lower 15 per cent of the specimen is subjected to swelling. The measured pore pressure at the base is smaller than the Terzaghi theoretical value during the initial stages of the test and greater during the later stages. It is predicted that measured pore water pressures should coincide with Terzaghi's theoretical values for a time factor $T = 0.39$, giving, in this way, a means of fitting theoretical with experimental results.

The paper by *Northey and Thomas (2/40)* is a contribution to the study of the response of pore-pressure measuring devices when applied to the determination of the pressure build-up in oedometer tests. The pressure build-up is also studied in relation to the speed of loading and to the effect of oedometer wall friction.

The sensitivity of the pore-pressure device required to obtain adequate response is shown to depend on soil compressibility. For large samples of highly compressible clays, the usual null indicator device involving a rather large volume change may be adequate, but a small sample of a soil of low compressibility requires a measuring system with extremely low volume change.

It is also shown that rapid loading (at 20-minute intervals) leads to a sharper pore-pressure response when compared with standard 24-hour-cycle loading. The difference is assigned to the effect on pore pressure of the greater rigidity of the soil structure, built up as a result of the secondary consolidation period of the 24-hour cycle.

A study is also made of the effect of side friction. The use of an oedometer with chromium-plated walls produced a friction of about 20 per cent at early stages, which decreased to only 5 per cent in later load increments. No improvements are reported by coating the walls with various lining and lubricating materials.

In settlement problems, the importance of correctly predicting the preconsolidation stress of clay deposits cannot be overemphasized. Yet, it is general knowledge that this prediction is very difficult and that all existing procedures based on laboratory tests leave much to be desired. Any attempt to find a more reliable precedure, if successful, may be an important practical contribution.

Togrol (2/53) reports on such an attempt. Only slightly overconsolidated soils, which are located between the virgin consolidation line and what has been called the critical overconsolidation line and which do not dilate in triaxial shear, were studied. In such soils, pore-pressure development in isotropically consolidated undrained compression testing derives only from the tendency of the materials to compress during shear. Furthermore, pore-pressure build-up is consistent and more uniform than in highly overconsolidated soil.

Drawing on these experimental facts, and following the line of thought developed by Roscoe, Schofield, and Wroth (1958) in their paper, "On the Yielding of Soils," it is indicated that the ultimate pore pressure at failure for a given soil is a unique function of water content and overconsolidation ratio. Therefore, to determine the preconsolidation stress of a clay sample, it would suffice to obtain the slopes of the virgin and of the overconsolidation lines and then to perform an isotropically consolidated undrained triaxial test to measure the pore pressure at ultimate yielding failure. As an example, a nomogram is shown relating pore water pressure at failure, overconsolidation ratio, and water content at failure for a given clay. The success of the method is still open to question and dependent on many factors, among them the degree of agreement existing between the yielding theory and the real behaviour of soils and on the uniformity of the pore pressure build-up, for which reported experimental data are scarce.

Primary and Secondary Consolidation

Within its limits, Terzaghi's theory of consolidation is a simplified mathematical interpretation of a highly complicated phenomenon. The degree of agreement between theory and reality, provided the boundary conditions correspond to those of the theory, as in oedometer tests, and due allowance is made for sample disturbance, depends on the development of phenomena of a creep nature which are responsible for what is known as secondary consolidation. These phenomena are always present, but their relative importance is widely different for different materials. Approximation of results to Terzaghi's theory varies with that relative importance. To take secondary consolidation into consideration various modifications of Terzaghi's theory have been proposed, one of the latest by Gibson and Lo (1961). The main purpose of the paper by *Christie (2/13)* is to check these theories by making long-duration oedometer tests in which the effect of secondary consolidation is emphasized. Each load increment was left one month and during this time both settlement and mid-plane pore-pressure observations were made.

Deriving the necessary parameters from the tests, a comparison is made between the theories of Terzaghi and of Gibson and Lo, and the experimental results. It is shown that for the materials tested the second theory fits the measured pore-pressure dissipation more closely. An important part of the paper is devoted to the peculiarities of pore-pressure measurement in consolidation tests and to the influence that the volume change has on the process of consolidation, as a result of the flexibility of the pore-pressure measuring system. Using Terzaghi's theory, an expression is derived of the variation of the pore pressure u at the base of the sample which provides curves relating pore pressure with time. They have the same form as those determined experimentally and reported by *Northey and Thomas (2/40)*. It is also shown that entrapped air may increase artificially the real flexibility of the measuring device.

It is generally agreed that settlement of structures due to stress deformation of plastic soils can be assigned to three simultaneous, though more or less independent causes: (1) settlement due to soil distortion without volume change, also called instantaneous or immediate settlement because it is usually taken as non-time-dependent; (2) primary consolidation settlement resulting from pore-pressure build-up; and (3) secondary consolidation, a settlement of a creep nature, whose physical mechanism remains unknown. All of these are, in reality, time-dependent phenomena.

When a buried layer of laterally confined, soft, saturated clay, of a small thickness in relation to the loaded area, is responsible for most of the settlement, the contribution of primary consolidation is, for certain clays, overwhelming in relation to the other causes and, within the accuracy of the problem, the latter frequently can be ignored. However, the case of the deep, comparatively thin, confined, soft clay layer has lost much of its original importance in building foundation practice. On the one hand, recognition of the influence of such buried layers in foundation behaviour and, on the other hand, the advancement of modern construction methods, has made it frequently cheaper to bypass such layers, when present, by making deeper foundations, thus avoiding the problems arising in semi-rigid framed reinforced concrete and steel structures from large settlements which increase the cost of the structure due to the greater stiffness required and bring unwanted troubles in building operations.

Present problems in settlement computation are much less unexpected yet much more difficult. There is a demand for methods which will provide, with reasonable accuracy and a minimum of guessing, the settlements originating in both saturated and unsaturated soils under triaxial states of stress inducing volume change and distortion. However, no such methods are available. For saturated soils a very valuable attempt has been made to meet the situation through a short cut which, using the existing knowledge and current methods of testing, applies correcting factors to the results obtained from the unidimensional method of settlement computation. It is described by Skempton and Bjerrum (1957) in their paper, "A Contribution to the Settlement Analysis of Foundations on Clay." In this method, total settlement is taken as the summation of immediate settlement and corrected consolidation settlement. It may well be that solutions of this type will prove, in the end, to be the most practical approach to settlement problems, especially because they retain simplicity through the use of oedometer and isotropic triaxial tests. However, in its present state of development, the method appears to have too many limitations. Almost inevitably, it leads in practice to a good deal of guessing about the values of the correcting factors, which involves a corresponding amount of uncertainty in the final results. Furthermore, calculated immediate settlements are always much higher than the measured values and, to obtain adequate results, the moduli of deformation derived from undrained triaxial tests must also be corrected. Secondary consolidation is ignored.

For these reasons, this General Reporter believes that, to take a real step forward, a novel approach may be needed, which, taking into account all the complexities would reach a new stage in settlement analysis. From it practical simplified solutions could be evolved later.

One facet of the research required to reach this starting point must be the study of the deformation properties of soils under various stress conditions and the discrimination of the physical components contributing to them; the other facet is the mathematical analysis of soil stressing and the development of calculation methods with computational helps to supplement the existing ones.

De Josselin de Jong and Verruijt (2/25) deal with one of the many sides of this study. They report consolidation tests on spherical samples, stressed radially on the surface and draining on the centre. In such a test $\sigma_1 = \sigma_2 = \sigma_3$ at every point and no distortion is applied to the sample, if the small effects resulting from the pore-pressure gradient are disregarded. Deformation due to primary consolidation, without shear distortion, as a function of pore pressure can be obtained by mathematical derivation taking into account the boundary conditions. When performing tests in which both pore pressure and volume change are measured, the part of this volume change pertaining to primary consolidation should end, by definition, at zero pore pressure. All volume change beyond this limit would pertain only to secondary consolidation. Three tests are reported, two on a preconsolidated clay stressed below the preconsolidation load, the other on the same material normally consolidated. The preconsolidated clay tests show good agreement between theoretical primary consolidation, total pore pressure, and volume change. No such agreement was obtained for the normally consolidated material. However, to arrive at any conclusion, more test results would be needed.

Mechanical models constitute more or less simplified abstractions for interpreting complicated phenomena. Clay consolidation, both one- and three-dimensional, has proved to be a difficult process to interpret because in it pure dissipation of pore pressures is intermingled with secondary time effects and other creep and rheological phenomena of a visco-elastic nature with non-uniform structural deformations.

Thompson (2/51) refers to studies made on a remoulded, saturated, silty clay tested in the oedometer and in the triaxial cell. A simple spring model is proposed to interpret the consolidation of such a clay at the initial stages, when good agreement exists with Terzaghi's theory. After longer times, however, the volume reduction is found to be at variance with this equation and, to explain it, more complicated models are required which would take into account the increasingly important effects of secondary consolidation and creep. It is further stated that mechanical models starting from Terzaghi's theory of consolidation provide only qualitative results because they assume isotropy. To obtain quantitative values non-uniform conditions should be taken into account and, for that purpose, a new visco-elastic model is proposed.

Lateral Pressures in Swelling Soils

In semi-arid zones, highly plastic clays undergo important volume changes which produce movements in the earth's surface to depths which vary with climatic conditions and with soil stratification. The surface goes up and down with moisture change and simultaneously vertical cracks are opened and closed. Frequently, the most dangerous movements are a product of soil swelling, resulting from closing the evaporation surface on the area covered by the structure. When these movements are opposed by the weight and the strength of the structure, high pressures develop. To solve engineering problems related to this particular soil behaviour it may be necessary to determine the amount of potential swelling deformation and the pressure required to resist such deformations, totally or partially. The methods developed to obtain values of these quantities are based on the use of a modified oedometer test which allows the measurement of either the swelling pressure under constant volume or the amount of swelling under constant stress. In the General Reporter's experience, at least for undisturbed soils, these tests would appear to be merely indicative, not selective; they may sort out high swelling soils from low swelling soils, a fact that often can be obtained just as well by site observation, but seldom yield quantitative data on the magnitude of the swelling pressures and/or deformation to be used in design, thus giving no clear picture of the soil behaviour.

Komornik and Zeitlen (2/30) report on the development of a device to measure lateral swelling pressure which consists of a consolidation ring, modified by having the centre part trimmed to a thin section. Strain gauges attached to this part of the ring are used to determine the lateral pressure for a negligible lateral deformation. Only preliminary test results are given. It would be interesting to have more results of both undisturbed and compacted soil and if possible an itemized discussion of their relation to field behaviour.

Clay Deformation under Tensile Stresses

Man-made structures subject soils essentially to a compressive state of stress. Yet tensile stresses may develop occasionally, for example behind open cuts or when a cohesive soil mass is subjected to seismic pulsating action. The knowledge of the deformation behaviour of cohesive soils under tensile stresses may therefore throw some light on soil reaction when subjected to such states of stress.

The paper by *Šuklje and Drnovšek (2/50)* is a preliminary incursion into the study of the deformation of cohesive soils under a state of stress involving a tensile principal stress. It deals first with a theoretical analysis undertaken to derive expressions for the tensile deformations on the assumption that the material is elastic, yet has orthogonal anisotropy, for both the plane strain and plane stress states of stress. Then tests are reported on two clays at natural consistency showing that, to fit experimental results to theory, the effect of anisotropy must be taken into account.

Behaviour of Typical Soils

Granular media may constitute the only soil materials in which the possibility of establishing rather simple relations between fundamental soil properties and simple physical properties, such as void ratio or relative density, appears to be more or less straightforward, especially when experiments are confined to limited groups of such materials—to those having grains with similar surface and shape properties, for example. It is thus conceivable that general laws giving approximate values of modulus of deformation and Poisson's ratio may be found for river sands with low mica contents. However, these laws would be different for sands with high mica content. To derive such laws, studies must be made to determine how the modulus of deformation and Poisson's ratio vary with the state of stress, the value of the confining or the mean pressure, and the stress history.

Karst, et al. (2/26) report such a study made on three granular materials in a rather dense state, two of them natural river sands and the third one an aggregate of glass beads. Drained triaxial tests were performed following stress paths comprising isotropic compression, K_0 condition, isotropic tension, and a combination of them. It is found that, on loading a given granular material for the first time, the relation between $(\sigma_1 - \sigma_3)/\sigma_3$ and axial strain ϵ_1, in an isotropic compression test, can be approximately represented by a unique curve, independent of lateral pressure. The same statement can be made for the relation between σ_1/σ_3 and ϵ_1. These relations imply that peak failure is reached for a unique value of σ_1/σ_3, independently of the intensity of the confining pressure, a statement that holds true only within the range where the effective angle of internal friction is practically a constant. By the same token, deformation at peak failure is supposed to be independent of confining pressure, while the modulus of deformation is linearly proportional to confining pressure. It is proposed that the same conclusions be taken as valid, on first approximation, for anisotropic triaxial tests. Experimental results put forward to support the statement appear, in the view of this General Reporter, to disprove it, however.

When subjected to a repeated loading, granular materials become work-hardened and the modulus increases greatly, the soil behaving almost as an elastic material. An exponential law relates the modulus to confining pressure which, however, also is shown to be a function of many other factors such as stress path, amplitude of stress variation, etc. Curves are given relating modulus of deformation to confining pressure for first loading and repeated loading for a triaxial hydrostatic stress variation and for a triaxial isotropic compression stress path, with constant confining pressure, over a given range of stress cycling.

From K_0 and repeated loading tests, it is stated that K_0 varied between 0.30 and 0.35. Poisson's ratio, after repeated loading, was also very uniform, varying between 0.30 and 0.33, regardless of the nature of the sand.

Rowe and Shields (2/44) report on the use of consolidation tests, performed with the classical oedometer device and with the same device modified by a new technique, to introduce radial horizontal drainage directed towards a small central sand drain (1/20 of the oedometer diameter) located in the centre of the sample. With this device, the horizontal and vertical coefficients of consolidation have been determined for both a uniform remoulded clay and an artificially laminated material made of alternating layers of clay and more permeable ceramic discs. The project's purpose was to study the behaviour of laminated and varved clays during consolidation. The average coefficient of horizontal consolidation of the layered system is compared with the same coefficient for the clay material. As has been shown in a previous paper, this ratio varies with the relative length of the drainage path. Furthermore, the experimental ratios obtained from the above-mentioned tests are compared with those predicted by a theory of Rowe (1964), obtaining what appears to be a reasonable agreement.

The soil mechanics laboratory of the Technische Hochschule of Aachen has for many years been engaged in a study of penetration tests, both static and dynamic, and their relations to relative density, overburden pressure, and the compressibility of soils. Even though penetration tests, and especially dynamic penetration tests, such as the so-called standard penetration test, are at best very crude driving tests, with the shortcomings inherent in them, they frequently provide the only data relevant to soil strength and deformation obtainable during subsoil investigations in sand and fine gravel strata. Consequently, until more rational methods are devised, a correlation of penetration resistance and fundamental soil properties, obtained by systematic testing and statistical analysis, may prove to be very valuable in extending and correcting existing correlations, because many of them are presently based on very limited evidence.

The investigation reported by *Schultze and Melzer (2/47)* was made to compare sand compressibility, as determined in confined compression tests, with the static cone penetration resistance and with the dynamic standard penetration test for various overburden pressures. For that purpose a tank 3 m in diameter and 5.5 m high was filled with a dry sand at various uniform relative densities. The groundwater level could be raised from the bottom of the tank to a level 2 m below the sand surface. Equations are given for obtaining the modulus of deformation as a function of the number of blows and of the penetration resistance for various overburden pressures. Similar relations are also given for relative density. In the General Reporter's opinion, it is unfortunate that for the dynamic penetration tests the thick-walled, two-inch, split-spoon sampler was retained. On the basis of today's knowledge this is one of the poorest types of sampling tools devised. Such a crude sampler can be supplanted without extra cost by much better tools designed to recover whole samples of sands under all conditions, something that is very seldom achieved with the split-spoon sampler in clean sands under water (Moretto, 1963a). Furthermore, the qualified experience behind the standardized use of the split-spoon sampler is too limited to justify its continuing use.

It is generally known that non-saturated cohesive soils with a given natural or compacted density, when stressed under undrained conditions, increase their deformation with water content. Highway engineers, in particular, have realized this for a long time and design methods, such as the California Bearing Ratio method, take this into account empirically by soaking the compacted soils before testing to obtain conditions which will generally resemble the future soil state in the highway structure. Other people have been tempted by the temporary increase in rigidity that is produced when a soil partially dries out after compaction and have used such a technique in highway construction in arid and semi-arid climates.

Souto Silveira (2/49) reports on a systematic statistical research project to study how the modulus of deformation of a typical compacted soil, tested undrained in the triaxial cell under a single confining pressure and a single rate of strain, varies with water content and dry density. It is found that, within the degrees of compaction having practical significance, the modulus of deformation is mainly dependent on water content, the degree of compaction not being significant. It is also found that up to about 70 per cent of the maximum deviator stress, the relation of stress to strain is practically linear.

Permafrost soils are a special problem and their peculiar characteristics require special study. Their behaviour is so highly influenced by the distribution of ice content and by the properties of solidified water that their treatment constitutes a specialized branch of soil mechanics. Moreover, thawing of pore water brings out very distinctive consolidation phenomena that do not fit into the general framework of normal consolidation theories.

The paper by *Tsytovich, et al. (2/55)* is an important contribution to the understanding of the mechanism of consolidation of permafrost soils which, at one boundary, come in contact with thawing temperatures, as when buildings or other structures are erected on such soils and a temperature change introduced on a limited area at the surface. It is shown that thawing is responsible for the greater part of the settlement. The thawing effect develops as a jump-like change in void ratio which, for a given soil and within a limited range of pressures, can be assumed to be linearly proportional to external pressure. After thawing, phenomena similar to normal primary and secondary consolidation develop. Experiments, performed on special non-heat-conducting oedometers with frozen samples exposed on one face to a constant temperature, are described, the pore pressure developed during the tests analysed, and the mechanism of resulting consolidation explained. Two theoretical approaches for two possible conditions at the thawing boundary are proposed.

Loess soils in a loose state are among the more difficult materials to sample undisturbed. They have a very sensitive structure which may easily be damaged by sampling methods. Only samples cut from open pits maintain their natural properties and even they may be damaged in transit or while cutting out specimens in the laboratory. In the General Reporter's experience, however, the importance of this damage varies depending on whether the shear strength of the soil or its deformation is to be measured. The loss of strength may be kept relatively small by using adequate simple sampling tools and methods. At the same time, to avoid serious damage to the soil rigidity, the utmost care is necessary. For the above reasons, settlement computations for structure founded on loess soils are very unreliable, though in soils with a highly collapsible structure the soil parameters obtained from oedometer tests may be more representative than for other soils, even for an unconfined layer of considerable thickness.

The paper by *Varga (2/56)* is an outstanding example of the failure of Terzaghi's solution for settlement computations when used to solve problems other than those for which it was developed. Varga compares the moduli of deformation obtained from oedometer tests and from loading tests. He also compares these computations with measured settlements. From both comparisons, it is shown that the modulus of deformation obtained from the oedometer is always very much smaller and has very little resemblance to the value derived from field measurements. Most of the difference is assigned to sample disturbances; yet, in the General Reporter's view, for loess that is not highly collapsible a large part of the discrepancy may be a result of the inadequacy of the oedometer test and of the simplified method of computation in general use for calculating settlements.

Due to its spongy nature, peat is possibly one of the soil materials in which settlement computations can be made with sufficient accuracy using the deformation parameters obtained from an oedometer test, even when the contributing layer is neither thin nor confined. The use of the oedometer test does not necessarily involve the mechanism of Terzaghi's theory of consolidation, and in fact much of the settlement originating in soils with collapsible structures does not recognize transfer of pore-pressure build-up as the main motivating cause. The collapse of the structures into a more stable framing of particles also plays a very important part. This collapse may be initiated by the pore-pressure build-up, but it is essentially an independent phenomenon. Consequently, the mechanism of consolidation is at variance with Terzaghi's theory.

Wilson, et al. (2/59) refer to the first stage of an investigation directed toward the determination of the mechanism of peat consolidation. For this purpose a sample of remoulded peat has been studied, analysing the soil behaviour when specimens of various heights are consolidated in the oedometer. By plotting the logarithm of the rate of consolidation against the logarithm of time, it is shown that the consolidation process can be divided into two distinct parts with very different rates. As is the case with common soils, the first part is called primary consolidation and the second, secondary consolidation. Curve-fitting formulae are given to express these two consolidations, from which the change in void ratio at any given time can be calculated.

Measured pore pressures are lower than the applied pressures and their peak values show a time lag that appears to develop as demonstrated by *Northey and Thomas (2/40).*

Progress between Conferences

In reviewing the specialized technical magazines one is impressed by the small number of contributions on the subject of soil deformation under static loading. The contrast with the number of papers published on shear strength is remarkable. Most of the contributions on the subject were reported to the European Conference on Soil Mechanics and Foundation Engineering (Germany, 1963) and to the Conference on the Design of Foundations for the Control of Settlement (U.S.A., 1964), conferences that were called to discuss settlement problems alone. Inasmuch as the papers presented were reviewed in detail and commented upon by the general reporters of those conferences and discussed during their meetings, it appears superfluous to refer to them further. Only their general conclusions will be mentioned and, for that purpose, attention will be focussed on the discussions held during conference meetings. (At the time this General Report was written only the discussions of the European Conference were available.)

When trying to express in a few words the atmosphere of the European Conference, in agreement with what has been said in this General Report, the impression obtained is that solutions for settlement are making little progress and that renewed examination of the problem may be necessary. This was clearly stated in the opening address of the European Conference where Dr. Bjerrum said: "It is, however, my feeling that what is still more needed today is a fundamental understanding of the factors governing the compressibility of clay. It is my hope that the discussions at this conference will concentrate on this subject. Should this be the case, this conference may lead to a rapid development of our knowledge of settlements. . . ."

Statements following a similar line of thought were made by this General Reporter in the discussion of Division 1b of the Second Panamerican Conference (1963) referring to deformation problems in soil mechanics (Moretto, 1963b).

Subjects for Panel Discussions

This General Reporter suggests that panel discussions be focussed on soil deformation properties related to settlement problems with the main emphasis placed on a critical analysis of the tests currently used to obtain the compressibility of soils and on the possible paths to be followed for a complete re-examination of the problem, having in view whatever development of new and more appropriate methods of testing may be deemed necessary. On that basis, the following subjects are proposed for panel discussion.

1. New trends and envisaged future development in laboratory testing methods to obtain the stress-strain behaviour and soil deformation parameters of both saturated and unsaturated soils representative of their behaviour in the field, with whatever adjustments may be possible to overcome or to take into account the large effects of inevitable sample disturbance.
2. State of the knowledge and envisaged future development of rational field methods to obtain the stress-strain behaviour and the parameters defining soil deformation.

Sujets de discussion pour le comité d'experts

Ce Rapporteur général suggère que les questions devant être discutées par le comité soient concentrées sur les propriétés de déformation du sol par rapport aux problèmes des tassements — en soulignant l'importance d'une analyse critique des essais couramment en usage afin d'obtenir la compressibilité des sols et des mesures à prendre en vue d'un nouvel examen approfondi du problème — en envisageant le développement de nouvelles méthodes d'essais plus appropriées qui seront jugées nécessaires. Sur ce principe, les sujets suivants sont proposés afin d'être discutés par le panel.

1. Les nouvelles tendances et le futur développement envisagé concernant les méthodes d'essais en laboratoire en vue d'obtenir le comportement tension-déformation et les paramètres de déformation du sol, pour les sols saturés et non saturés, représentatifs de leur comportement sur le chantier, et tout ajustement possible permettant de surmonter ou de considérer les effets importants du remaniement inévitable des échantillons.
2. L'étendue des connaissances et le futur développement envisagé des méthodes rationnelles sur le chantier afin d'obtenir le comportement tension-déformation et les paramètres définissant la déformation du sol.

General Behaviour, Shear Strength, and Deformation of Soils under Dynamic Loading

During the past 10 to 15 years there has been a marked increase in the study of soil behaviour under dynamic loading. On the basis of a recent inventory made by the Soil Mechanics and Foundation Division of the American Society of Civil Engineers it has become one of the main topics of current research in the United States. There have been various motivating reasons, the following appearing to be the more important.

1. Realization by highway engineers that the development of rational methods of pavement design requires the consideration of the behaviour of the subgrade, the base courses, and the wearing surface under the action of transient repeated stresses.
2. The appearance of new dynamic problems in connection with the structural requirements of facilities for launching or operating spacecraft, missiles, and heavy weapons.
3. The need to study the effect of nuclear and other very powerful blastings.
4. The ever present interest in the behaviour of soils under the action of repeated stresses produced by seismic action and machine foundations.

Many of these studies have been made by subjecting soil samples to transient or repeated stresses in a triaxial device. Moreover, triaxial or other similar devices have also been used to measure dynamic properties through wave propagation, either at or away from the natural frequency. Earlier methods, using loading plates or concrete blocks acted upon by vibratory forces, appear to be losing favour. Instead, field tests using vibratory techniques to produce and measure the velocity of waves propagated in the ground are being used more extensively. Eight of the papers assigned to Division 2 refer to the dynamic behaviour of soils.

Effect of Loads of Very Short Duration

Fifteen years ago, pioneer work by Casagrande and his co-workers showed that, for stressing of very short duration in the range called transient loading, which is that requiring less than one minute to produce failure, the undrained shear strength of clays increases with loading rate. Subsequent investigations by others have confirmed these findings.

The deformation is also time dependent. For a given stress level, it increases with the time allowed to reach that stress. It is considered to be composed of three parts: truly elastic strain, taken as equal to the recoverable deformation that the soil would undergo when the time of loading tends to zero; plastic strain, resulting from particle displacement of a viscous nature, taken as equal to the non-recoverable deformation produced during the period of increasing load; and creep, also a viscous type of phenomenon, not necessarily distinguishable from plastic strain, that is assumed equal to the increase in deformation produced under constant stress. For finite loading times, the three types of strain develop simultaneously and it is conceivable that transfer from elastic to plastic strain may occur under sustained stress.

Discrimination between strain components in actual tests is very difficult, though it may be of great interest to understand response under exceedingly fast loading.

Mitchell and McConnell (2/38) refer to an investigation in which tests were made to separate elastic response from plastic effects in compacted specimens of a kaolinite clay which were subjected to a single deviator stress pulse of different duration (range: 0.006 to 1.50 seconds) for five different stress levels varying between about 15 per cent and 70 per cent of the unconfined compression strength of the material. By plotting the stress recovery as a function of load duration, the test results were extrapolated to zero time load duration and an estimate of the magnitude of the pure elastic strain obtained.

Two methods of compaction were used to prepare the specimens: kneading, developing what elsewhere has been termed a dispersed structure; and static, providing what has been called a flocculent structure.

For samples with a dispersed structure, it is shown that, for stress levels in excess of 15 per cent of the unconfined compression strength, the recoverable strain decreases as load duration increases, thus illustrating the transfer of elastic to plastic deformations as a result of stress relaxation within the specimen. On the contrary, for samples prepared by static compaction, the recoverable strain is found to be essentially independent of load duration. In this comparison, the recoverable deformation of specimens with a flocculent structure was found to range between 60 and 90 per cent of the total deformation, whereas in specimens with dispersed structures values of the order of only 15 to 30 per cent of that deformation were reached. Plotting the extrapolated recoverable elastic strain for zero time of load duration as a function of stress intensity, it is further found that the elastic response is non-linear with stress.

Olson and Kane (2/41) report on an investigation undertaken to determine the undrained failure and stress-strain behaviour of compacted clay specimens placed in a triaxial cell under high confining pressures, equal to about 7 and 70 kg/sq.cm. respectively, and subjected to a single deviator pulse with a duration ranging from 100 seconds to 3 milliseconds.

Results confirm those obtained in previous investigations on the effect of strain rate on shear strength and deformation of soils. It was found that the compressive strength increased about 10 per cent for each tenfold increase in the rate of deformation, so that the dynamic Mohr-Coulomb failure envelope could be derived from the conventional static envelope by simply increasing the static strength in the appropriate ratio, obtained from the difference in rate of deformation. The same general law was found valid for the secant modulus at one per cent axial strain for the specimens tested with a confining pressure of 70 kg/sq.cm.; while, for those tested at 7 kg/sq.cm., pre-stress due to compaction (of the order of 14 kg/sq.cm.) made the secant modulus independent of rate of strain. Furthermore, for all tests the strain to failure was found to be independent of either confining pressure or strain rate.

The investigation also included the study of creep at constant load after rapid stress rise. It was found that creep did not take place until the stress difference reached about 30 per cent of the failure deviator stress at a given rate of deformation.

Repeated Stress Application

To a great extent, the study of the behaviour of soil under the effect of repeated loading has been limited in the past to the determination of the natural frequency of large soil masses subjected to vibrating impulses transmitted by bearing plates or concrete blocks of different sizes placed on the ground surface. Theoretical derivations made on the hypothesis of an elastic behaviour of the material provided the deformation parameters, such as the dynamic modulus of elasticity. Tests of this type are very useful, but as do all indirect methods of measurements, they have the shortcoming of needing an idealized theoretical deduction to arrive at a physical constant.

The systematic study of deformation parameters, under repeated loading of specimens placed in triaxial cells to control the stress conditions, is of more recent date. *Folque (2/17)* refers to such types of tests carried out on a silty clay material compacted at normal Proctor density with a moisture content 2 per cent below optimum, giving a degree of saturation of about 85 per cent.

Samples were tested undrained under either a pulsating hydrostatic state of stress ($\sigma_1 = \sigma_2 = \sigma_3$) or a pulsating deviator stress and constant confining pressure. Volumetric strains, axial strains, and pore pressures were measured during the tests. Tests under a hydrostatic state of stress provide results indicating that the relation between volumetric strain and effective pressure ($\sigma - u$) is not very much affected by the frequency of pulses and no difference is found with static loading tests. Triaxial compression tests show, on the contrary, that in terms of effective stresses, for a deviator strain of one per cent, the behaviour of the samples is strongly affected by the frequency of pulses and that, at equal axial strain, for loads applied during identical time duration for the same mean effective stress $(\sigma_1' + \sigma_3')/2$, the deviator stress $(\sigma_1' - \sigma_3')/2$ decreases when frequency increases.

In the paper by *Kawakami and Ogawa*)2/27) a rheological model is first proposed, whose behaviour is taken as similar to that of the soil under the application of repeated stresses. Then the results of triaxial tests on five compacted soils, comprising a silty loam, a clay, and three different clay-sand mixtures, are reported. They were subjected to six different confining pressures, varying from 0 to 5 kg/sq.cm., and four cycling deviator stress ranges with a constant minimum stress equal to 0.26 kg/sq.cm., and a maximum stress varying from 0.77 to 2.05 kg/sq.cm. applied 10,000 times. Axial strains were measured, rates of strain determined, and moduli of resilient deformation computed.

It is stated that the axial strain increased with the intensity and the number of stress applications. However, after fifty to one hundred stress applications its rate of increase became very small and could no longer be measured with the dial indicator used for the tests (1/100 mm graduation). Other statements made about the variation of the moduli of resilient deformation, with the number of stress applications are not clear to this General Reporter who would welcome a further comment by the authors of the paper. Several figures are shown in which curves giving the relations between confining pressure and axial strain, modulus of resilient deformation, ultimate stress, and yield stress are represented for various cycling stress ranges and water contents at 10,000 cycles.

Makhlouf and Stewart (2/35) report on research undertaken to investigate the deformation properties of air-dried Ottawa sand, in both loose and dense states, when subjected to a cycling deviator axial stress in a triaxial cell under varying confining pressures, ranging from 0.5 to 5 kg/sq.cm. Previous investigations have shown that the modulus of deformation after a great number of cycles can be taken as a measure of the dynamic modulus of the material. The present tests show that, with repeated loading, the sand tends to become elastic in the range of cycling and that most of the inelastic behaviour of the material is overcome on first loading. Beyond the range of cycling the behaviour is not changed. It is shown that the measured dynamic modulus of deformation depends on confining pressure, range of cycling, magnitude of either the maximum or the minimum stress enclosing the range, and, to a lesser degree, relative density.

The last part of the paper by *Karst, et al. (2/26)* refers to an investigation following the same lines as the one just reviewed. Reported findings are reviewed *supra*, p. 236.

Wave Propagation Methods

Estimates of dynamic moduli, through the use of the well-known expression that, for elastic materials, relates Young's modulus, E, and the shear modulus, G, to mass density and to either longitudinal or shear wave velocity, have been in use for a long time. They provide one of the most widely applied methods for obtaining these moduli.

Bamert, et al. (2/3) report an investigation undertaken to study wave propagation through two soils, a gravel and a loamy loess, under varying confining pressures in order to compare the estimated moduli of elasticity with those obtained from repeated slow loading tests. For this purpose, a large triaxial cell, 25 cm in diameter, was provided with hollow filter discs in which a pressure cell was fitted. In the centre of these cells a dynamic gauge was inserted, having the same height and modulus of elasticity as the surrounding filter.

To perform the tests the samples were first subjected to states of stress with varying ratios $\sigma_1 : \sigma_3$ ranging from $\sigma_1 > \sigma_3$ to $\sigma_1 < \sigma_3$. Then, maintaining the state of stress constant, a weight was dropped on the sample head inducing an impulse which travelled along the sample length and was reflected at its base. Measurements were taken at head and base to obtain frontal and peak wave velocities from which dynamic moduli E_{d_1} and E_{d_2} were respectively determined. The experiments proved that the frontal velocity is significantly higher than the velocity at which the pressure peak travels through the sample.

Comparison with repeated slow loading tests shows agreement with the values E_{d_2} obtained from peak wave velocity, but much lower values than those derived from frontal velocity, as $E_{d_1} = 2E_{d_2}$ for the gravel and $E_{d_1} = 3E_{d_2}$ for the loamy loess. It is shown also that for both materials the dynamic modulus increases very significantly with all-round confining pressure.

The first part of the paper by *Bažant and Dvořák (2/5)* is concerned with a description of the effects that vibration produces in saturated sand as an acceleration applied to the bottom of a sand layer increases from zero to a value several times that of gravity. They are classed as dynamic stability, compaction, liquefaction, and motion of sand. Coulomb's equation for static loads, in terms of effective stress, is adjusted adding the effective and pore pressures produced as a result of vibration impulses. On the hypothesis that the angle of internal friction does not alter during vibration, the conditions leading to dynamic stability, compaction, and liquefaction are discussed and compared with the results of some simple tests in which a glass container filled with saturated sand was placed on a vibrating table moving vertically and the acceleration needed to reach those conditions measured.

The second part of the paper by *Bažant and Dvořák (2/5)* refers to a simplified method of producing vibrations in concrete blocks of different sizes placed on the ground surface or foundation level. Vertical and horizontal vibrating impulses are separately induced by striking a block with single blows of hand-operated hammers. The displacements of the blocks and of the surrounding ground surface due to oscillations are recorded by vibrographs placed on the top of the blocks and on the ground surface. Considering the block as a body with one degree of freedom supported by elastic subsoil, the spring constant c and the modulus of elasticity E can be determined from the measured frequency n_0 of free oscillation and the known weight W of the block. The use of the equations for the impact of two free bodies, hammer and block, is suggested to check the measured displacement amplitudes.

Progress between Conferences

The number of papers referring to the dynamic properties of soils is increasing rapidly and this trend will probably continue. The motivating reasons for this development were given earlier. Allowing for the language limitations of this General Reporter, by far the greatest number of contributions seem to have been made in the U.S.A., particularly in relation to highway engineering. The Symposium on Soil Dynamics held by the A.S.T.M. and published in 1962 had eight papers devoted to testing methods, instrumentation, and soil behaviour. Of special interest are the contributions made to the International Conference on the Structural Design of Asphalt Pavements (University of Michigan, U.S.A., 1962), where ten papers were published and discussed, among them one by the team working in the California Division of Highways in which a preliminary method of designing flexible pavements on the basis of the dynamic behaviour of its component parts is described. More papers related to the dynamic behaviour of compacted soils have been published, in technical magazines such as the *Proceedings* of the Highway Research Board, *Géotechnique,* the *Journal of the Soil Mechanics and Foundation Division,* and other magazines carying soil mechanics papers.

The behaviour of soil under single transient loads, such as that produced by blasting, has also been discussed in the specialized literature. Particular reference should be made to the paper published by Wilson and Sibley (1962).

Studies devoted to undisturbed soils, to obtain a better understanding of their behaviour under dynamic loads imposed by machine foundations and seismic action, have been less favoured. Notwithstanding, mention should be made of the work performed by F. E. Richart and his co-workers on wave velocity and damping through granular materials published at the Second Panamerican Conference and in the *Journal of the Soil Mechanics and Foundation Division* of the A.S.C.E.

Subjects for Panel Discussion

Taking into consideration that this is the first time that the subject of soil behaviour under dynamic loading has been put forward for panel discussion in our international conferences, and realizing that the main interest of participants is centred on foundation and earthwork engineering, the following general topic for discussion is suggested. The state of knowledge of the behaviour and testing methods for the determination of the mechanical properties of soils when subjected to: *(a)* seismic loads, *(b)* vibrations induced by machine foundations. Even though the dynamic effects of seismic loads and those induced by machine foundations are essentially of the same nature, sufficient differences appear to spring from the amplitude, frequency, acceleration, and duration of the vibrations produced, to justify a separate treatment. Also, earthquake engineering embraces a large number of problems related to the stability of earth dams, natural slopes, and other large masses of soil which involve stress conditions requiring separate attention.

Sujets de discussion pour le comité d'experts

Considérant que le comportement du sol sous un chargement dynamique est discuté pour la première fois à nos congrès internationaux, et réalisant que l'intérêt principal des participants est concentré sur les travaux de fondations et de terrassements, le sujet suivant, d'ordre général, est suggéré. L'état de la science relativement au comportement et aux méthodes d'essais en vue de la détermination des propriétés mécaniques des sols lorsque soumis aux: (1) charges sismiques, (2) vibrations induites par les fondations de machines. Alors même que les effets dynamiques des charges sismiques et ceux induits par les fondations de machine sont essentiellement de la même nature, des différences suffisantes se manifestent par l'ampleur, la fréquence, l'accélération et la durée des vibrations produites, pour justifier une attention toute spéciale. De plus, l'étude des secousses sismiques embrasse un grand nombre de problèmes se rapportant à la stabilité des barrages en terre, talus naturels et autres masses importantes de sols avec conditions de contraintes nécessitant une attention particulière.

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