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# Les propriétés des sols et leur mesure

## Soil Properties and their Measurement

**Sujets de discussion :** Dispersion des essais de mécanique des sols. Dissipation de la pression interstitielle. Propriétés visco-élastiques des sols : Existence d'un seuil de Bingham. Fluage.

**Subjects for Discussion:** Scatter of soil mechanics tests. Pore-pressure dissipation. Visco-elastic properties of soils : the existence of a Bingham limit. Creep.

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*Vice-Président / Vice-Chairman :*

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*Rapporteur Général / General Reporter :*

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*Contributions écrites / Written Contributions*

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C. MEYERHOF

Rapporteur General, Division I / General Reporter, Division I

**Le Vice-Président :**

Messieurs, au nom du Comité d'organisation du Congrès, j'ai l'honneur d'ouvrir la première séance de discussion du Cinquième Congrès de la Société internationale de Mécanique des sols, séance consacrée à la section 1. « Propriétés des sols et leur mesure ».

Je prie M. Glossop, de la délégation britannique de bien vouloir assurer la présidence de cette section, assisté de

M. Meyerhof, de la délégation du Canada, comme rapporteur, de M. Bishop, de la délégation britannique, de M. Breth, de la délégation allemande, de M. Denissov, de la délégation soviétique, et de M. Marchand de la délégation française, comme membres du groupe de discussion. M. le président va vous indiquer la façon dont la séance va se dérouler.

#### Le Président :

This morning we are to discuss soil properties and their measurement. This is, of course, a very large and complex subject, and so as to give continuity to our proceedings the Organising Committee has suggested that the spoken discussion should be limited to three aspects of the subject. These, as you know from page 37 of the Programme are — (a) Scatter of soil mechanic tests; (b) Pore-pressure dissipation; (c) Visco-elastic properties of soil; the existence of a Bingham limit. Creep.

Accordingly I propose that we should give priority to these three subjects, though if time permits other subjects may be introduced later. Of course, written contributions on any subject connected with Section 1 may be submitted and will be published in Volume III. These should not exceed 600 words in length and they should be submitted within one month of the closing of this Conference.

As regards the order of our discussion, I will first call upon Mr Meyerhof to make his report. The discussion will then be opened by the panel, or discussion group, whose names have been given to you by Mr Wahl. They will discuss these three subjects in turn (that is to say, they will first discuss (a), then (b) and then (c)). They will be allowed 10 to 15 minutes each. After the panel has introduced the subject in this way we will have a short break of 10 minutes or so, and during that 10 minutes if any other members — and I hope there will be many — of the audience wish to contribute will they please write their names and the subject they wish to discuss (that is, (a), (b) or (c)) on a piece of paper and send it up to the platform. Mr Meyerhof and I will, during the interval, go through these and call upon the members to speak. In making their discussion they should come up to the tribune and speak through the microphone here. I would also like to remind them to speak slowly and clearly, since every word they say must, of course, be interpreted and tape recorded. I am afraid that, again, time must be limited because no doubt many people will wish to take part. Therefore I ask members to speak for not more than 5 minutes.

I now ask Mr Meyerhof to give his report.

#### Le Rapporteur Général :

Monsieur le Président, Mesdames, Messieurs, j'ai présenté en anglais le rapport général de cette section, et je voudrais vous en faire un résumé en français, pour marquer le caractère international de cette conférence, et pour remercier nos hôtes de leur aimable hospitalité.

La première section du Congrès de Mécanique des Sols est consacrée aux propriétés des sols et à leur détermination. Il s'agit d'un des problèmes fondamentaux de notre science, et dont l'importance est soulignée par l'envoi de soixante-douze communications, le plus grand nombre jamais obtenu dans aucune autre section. Mon rapport a donc été basé principalement sur l'analyse des communications au présent Congrès; les publications parues depuis le dernier Congrès de Londres sont brièvement mentionnées.

On peut distinguer trois grands problèmes concernant les propriétés des sols : classification et description; propriétés physico-chimiques et mécaniques, y compris la perméabilité; méthodes et appareils de mesure des propriétés des sols. Ces questions ont été étudiées dans les sept sections principales de notre rapport de manière à faire ressortir le compor-

tement fondamental des sols, qui a donné lieu, récemment, à une recherche intense sur l'aspect physico-chimique, du cisaillement et de la déformation. Néanmoins, je suggère que pour nos conférences prochaines cette section soit divisée en une section 1A concernant la classification et description des sols avec les méthodes et appareils correspondants, et une section 1B concernant les propriétés physico-chimiques et mécaniques, y compris la perméabilité, avec les méthodes et appareils correspondants.

Voici un bref résumé de mon rapport général :

Dans plusieurs pays les études régionales des sols peuvent être groupées, et les renseignements obtenus utilisables sous forme de cartes géotechniques et de mémoires pour les avant-projets d'ouvrages en terre et de travaux de fondations dans la région même. On n'observe pas de contribution majeure dans l'identification et la classification des sols, mais des rapprochements nouveaux ont été proposés entre les propriétés descriptives et mécaniques. La nature et le comportement du complexe eau-sol ont été précisés par des méthodes physico-chimiques dans les domaines des échanges de bases, de la stabilisation, et du gonflement. On note également un progrès dans l'étude de l'électro-osmose, du gel, et de la perméabilité d'un sol partiellement saturé; néanmoins les principes de base des effets constatés et les modalités de l'application pratique ne sont pas encore établis.

Les propriétés visco-élastiques des sols sont mieux connues, et les caractéristiques de déformation déterminées à contrainte constante et dans le cas de répétition des efforts, mais fréquemment sans drainage et sans mesure des pressions interstitielles; et les propriétés des sols drainés, primordiales pour les problèmes de stabilité à long terme, ne sont pas connues suffisamment. La plupart des essais présentés renvoient à des contraintes triaxiales, et les relations visco-élastiques dans un champ de contraintes biaxiales sont méconnues. La consolidation axiale et radiale a été étudiée pour des sols partiellement ou complètement saturés et également dans le cas d'argiles anisotropes; la compressibilité des sols pulvérulents a été envisagée, le tassement étant jusqu'à maintenant basé sur des méthodes semi-empiriques.

C'est dans le domaine de la résistance au cisaillement qu'on trouve le plus grand nombre de communications, relatives à des considérations théoriques et expérimentales diverses. Les paramètres les plus utiles concernent la résistance effective, et nombreux sont les mémoires basés sur des essais triaxiaux. La résistance au cisaillement avec drainage et à long terme, de même que les paramètres résultant de compressions biaxiales ou d'autres modes de chargement, devraient être étudiés davantage. L'appareillage de compression triaxiale et de consolidation a été amélioré, et de nouvelles méthodes de mesure de la pression interstitielle ont été mises au point.

Beaucoup d'intéressantes communications invitent à la discussion, mais les sujets doivent être limités à trois problèmes importants :

1. La dispersion des essais de Mécanique des Sols, le degré de dispersion devant être étudié d'abord sur une propriété donnée d'un certain sol. Il importe de distinguer d'une part la dispersion due à la diversité des appareils, des opérateurs, et des techniques d'essais et d'interprétation, et d'autre part celle due aux variations du sol lui-même.
2. La dissipation de la pression interstitielle, qui peut être estimée dans le cas de matériaux isotropes complètement saturés et consolidés sous une charge constante ou uniformément variée. Il serait désirable d'envisager les cas d'autres types de chargement, de sols anisotropes, et de sols partiellement saturés où interviennent à la fois les pressions de l'air et de l'eau.
3. Les propriétés visco-élastiques des sols : existence d'un seuil de Bingham; problème du fluage. Vu l'importance

des propriétés de déformation visco-élastique et de résistance à long terme, il serait utile de considérer la répartition du seuil de Bingham entre la cohésion effective et l'angle de résistance, et de discuter le fluage d'un sol drainé pour différentes conditions de chargement.

#### Le Président :

Thank you, Mr Meyerhof. I will now ask the panel to open the discussion of subject (a). Dr Bishop, would you be so kind as to open the discussion?

#### M. BISHOP (Grande-Bretagne)

In my contribution to this discussion I want to consider the problem facing the engineer; whether to accept the scatter of his test results and analyse them statistically, drawing sometimes rather pessimistic conclusions, or whether to track down the source of the scatter and see if it does not in fact often lie in the testing technique rather than in the natural properties of the soil.

There are two particular sources of error to which I wish to refer. The first arises from the fact that in undrained triaxial compression tests run at conventional rates of testing there may be large differences in pore water pressure between the middle and ends of the specimen. This is partly a consequence of the restraint caused by the rigid platens at the ends of the specimen and partly a function of the properties of the soil itself. We were aware that this difference might occur, but in recent years we have found that its magnitude is much greater than anticipated and can lead to a very serious error in soil properties based on a single point pore pressure measurement unless the test is run sufficiently slowly for equilibrium to be set up within the pore water throughout the whole specimen.

This may be illustrated by test results presented at the recent Conference on the Shear Strength of Cohesive Soils held by the American Society of Civil Engineers. Fig. 1 shows the difference between the pore water pressure measured at the base of the specimen and that measured at its mid-height in a test of 8 hours duration on a sample of compacted clay shale 8 inches in height. The difference is of the order of 10 lb./sq. in. at failure at a rate considered to be on the conservative side in ordinary testing practice.

In Fig. 2 the corresponding results of a test on the same soil of about 120 hours duration are illustrated. The difference between the base and mid-height values of pore water pressure has decreased to about 2 lb./sq. in. which can be ignored without serious error.

The effect of unequalized pore pressures on the position of the failure envelope is illustrated in Fig. 3. The base measurement of pore water pressure in the 8 hours test leads to a cohesion intercept of 11 lb./sq. in., while the mid-height measurement indicates an intercept of only about 1 lb./sq. in. This difference has very serious implications from a design point of view, and in many of the test results still being published this factor is overlooked.

In Fig. 4 the difference in pore water pressure between the middle and ends of the specimen is plotted against the time to reach 10 per cent axial strain. Given sufficient time the pore pressure will equalize and a valid result can be obtained from a single point measurement. The time required, however, for this particular soil (22 per cent clay fraction) and sample height (8 inches) is about 10,000 minutes or 7 days, unless special means are taken to accelerate the equalization of pore pressure. Strips of filter paper around the

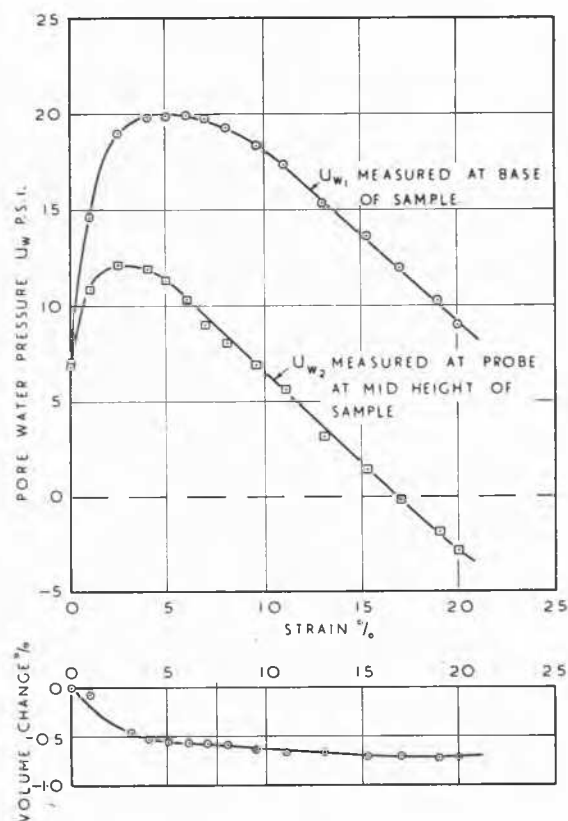
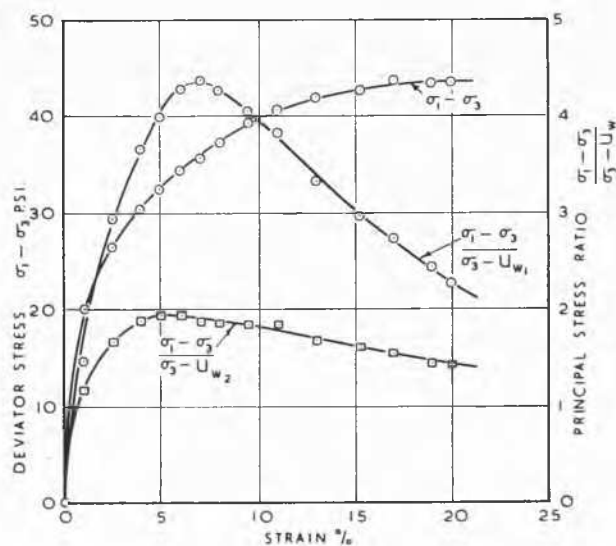


Fig. 1 Effect of rate of strain on pore pressures measured at end and centre of a 4" dia  $\times$  8" high sample of compacted shale. Rate of strain: 20 % in 8 hours.

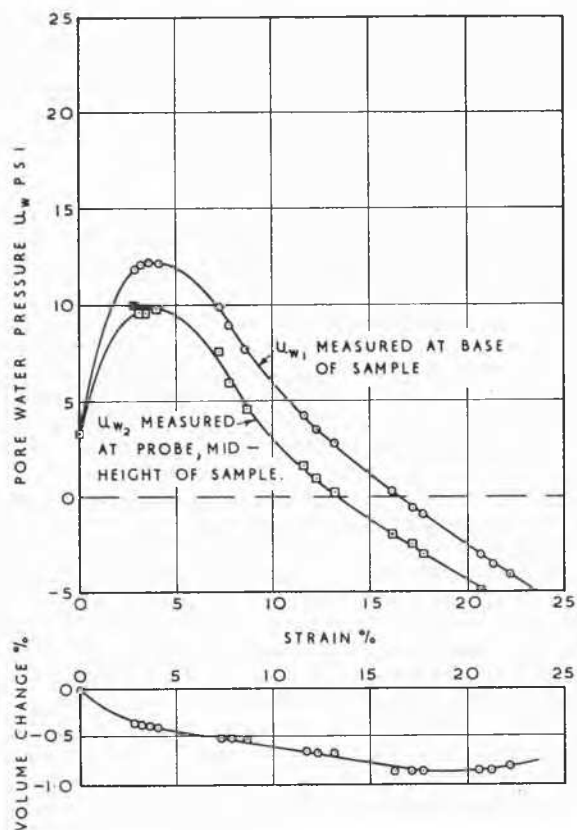
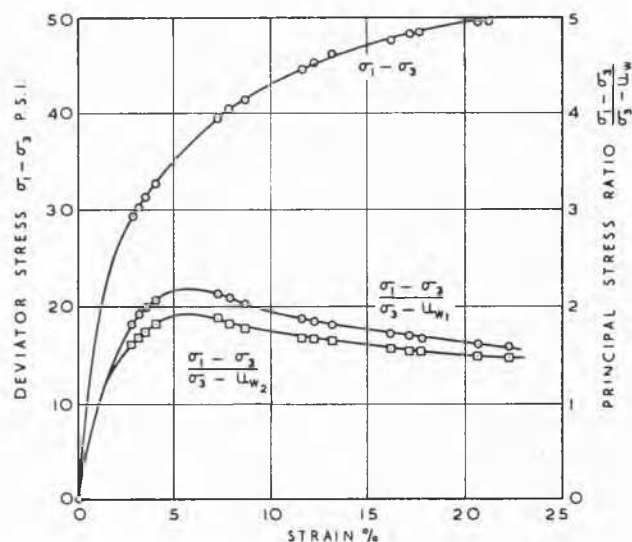


Fig. 2 Effect of rate of strain on pore pressures measured at end and centre of a 4" dia  $\times$  8" high sample of compacted shale. Rate of strain: 20 % in 120 hours.

perimeter of the sample may be used for this purpose in saturated soils.

Fig. 5 shows the water content profiles of two typical samples of the compacted clay shale, taken after time had been allowed for the pore water pressure gradients to approach zero. The profiles clearly reflect the observed differences in pore water pressure.

For different types of soil this difference may be a difference not only in magnitude but also in sign. For normally consolidated sensitive clays the pore pressure is higher in the middle of the sample than at the ends. This is illustrated by the results of tests by Taylor and Clough (1951) on un-

disturbed Boston clay, one of which is plotted in Fig. 6. Also plotted are the results of a test on a saturated remoulded clay, which is lightly over-consolidated. Here the pore pressure difference is small. In contrast the compacted clay shale shows a large difference, the pore water pressure being lower in the middle than at the ends.

A theoretical relationship has been worked out by Dr R.E. Gibson between the duration of the test, expressed as a dimensionless time factor, and the degree of equalization of the initially non-uniform pore pressure. For a given height of sample and coefficient of consolidation the appropriate testing time can be selected. This relationship is plotted in

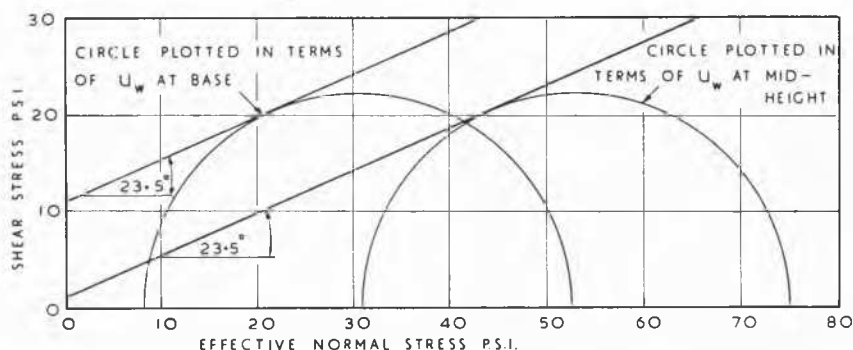


Fig. 3 Effect of unequalized pore pressures on apparent position of failure envelope for compacted shale. Rate of strain: 20 % in 8 hours.

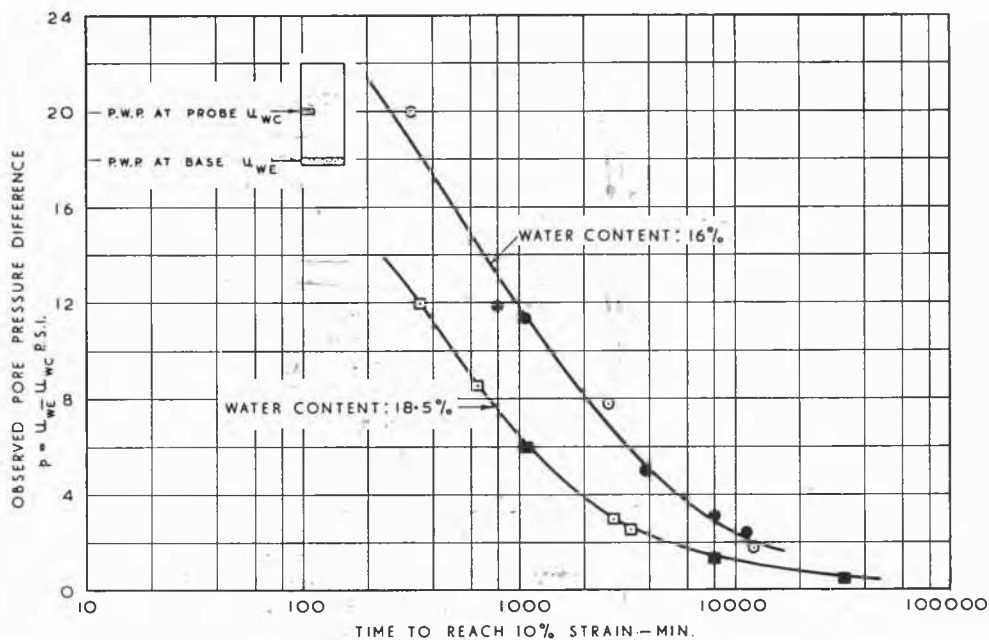


Fig. 4 Effect of time of testing on equalization of pore pressure during shear of compacted clay shale.

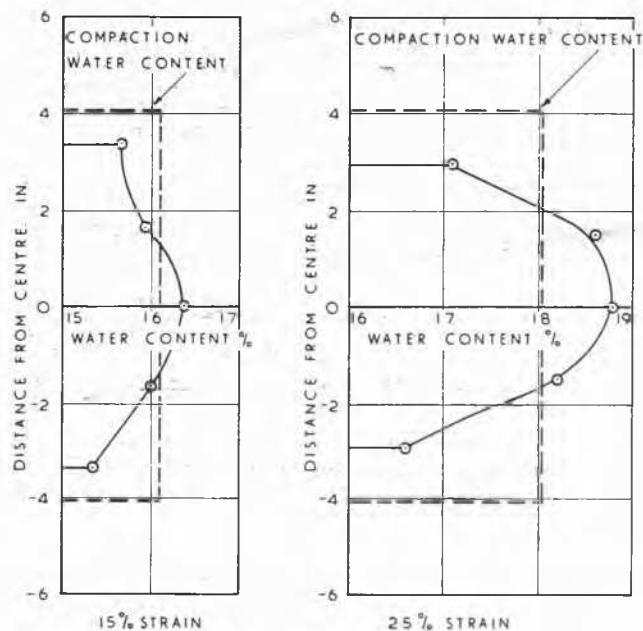


Fig. 5 Equilibrium water content profiles.

Fig. 7, together with the results of tests carried out at Imperial College and at M.I.T., which are in reasonable conformity with it.

The second error to which I wish to refer arises in partly saturated soil from the difficulty in distinguishing between the pore water pressure and the pore air pressure. In Fig. 8 the results are plotted of some triaxial tests in which fine ceramic discs of high air entry value were used to measure the pore water pressure, the pore air pressure being measured separately with discs of coarse woven fibre glass cloth at the opposite end of the sample. The difference between the pore water pressure and the pore air pressure due to surface tension

varies from zero to nearly 20 lb./sq. inch in the series of tests illustrated.

If the strength is plotted against effective stress on the basis of the difference between total stress and pore water pressure the relationship shown by square symbols is obtained. This line, if extended, would give a negative cohesion intercept. If the results are plotted in terms of the difference between total stress and pore air pressure the relationship shown by round symbols is obtained. This corresponds to a large positive cohesion intercept.

Tests on fully saturated samples in which there is no ambiguity about the determination of effective stress, lead to

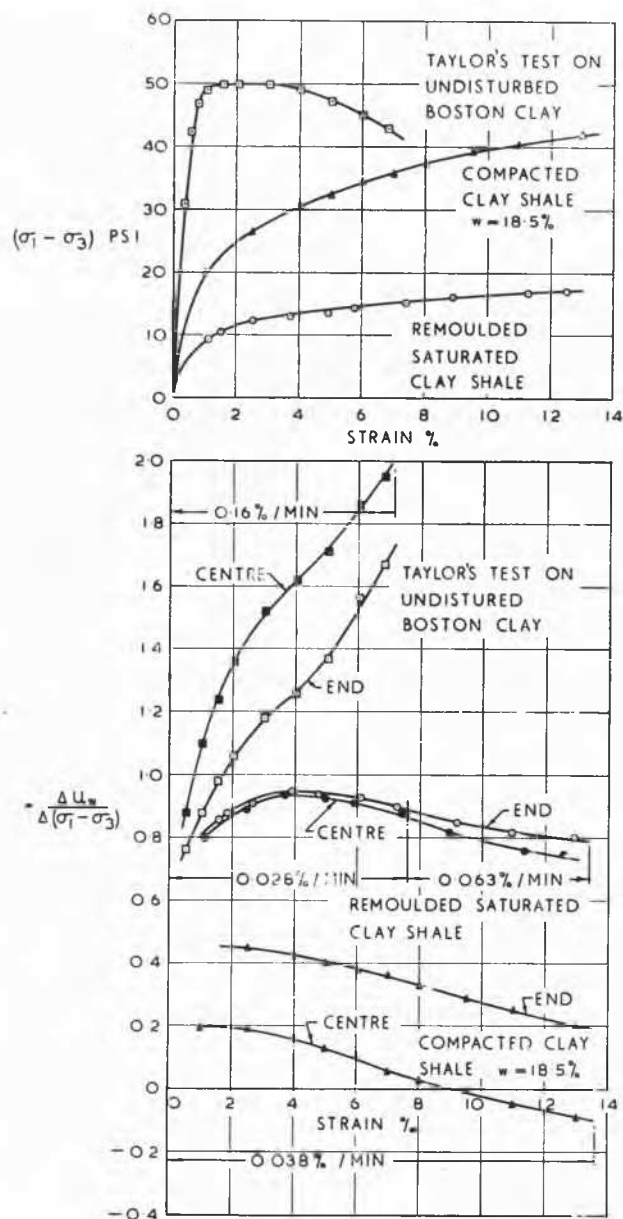


Fig. 6 Difference between centre and end pore pressure measurement for three soils representing a wide range of « A » values.

an intermediate position for the strength plot, with a small positive cohesion intercept.

Since in the partly saturated soil neither the air pressure nor the water pressure can in general act over the whole of the surface of the soil particles, the use of one or the other alone will lead to an incorrect estimate of effective stress. This has led to the proposal of a modified effective stress equation :

$$\sigma' = \sigma - u_a + \chi (u_a - u_w)$$

where  $\sigma'$  denotes effective stress,  
 $\sigma$  — total stress,  
 $u_a$  — pore air pressure,  
 $u_w$  — pore water pressure,  
 and  $\chi$  — a parameter varying between 1 for fully saturated soils and 0 for dry soils, depending mainly on the degree of saturation, but also on soil type and cycle of wetting or drying, etc.

This equation is discussed in a paper to this Conference (Bishop and Donald, 1961) and a more general examination of the problem is to be found in the Proceedings of the Conference on Pore Pressure and Suction in Soil, London, 1960.

If the additional factor controlling effective stress in partly saturated soil is ignored, apparent inconsistencies may arise between the results of different types of test on the same sample. In particular, if coarse grained porous discs of low air entry value are used, pore air pressure will usually be measured, and this, as shown by Fig. 8 will lead to a very high apparent cohesion intercept, which is not a true property of the soil. If the porous disc, being initially fully saturated, sometimes measures pore water pressure, sometimes pore air pressure, considerable scatter of the results may result.

I have drawn attention to these two sources of error, because very inconsistent results may be obtained if they are not given full consideration, and the application of statistical methods will not tell you which is the right result to apply in any particular practical problem.

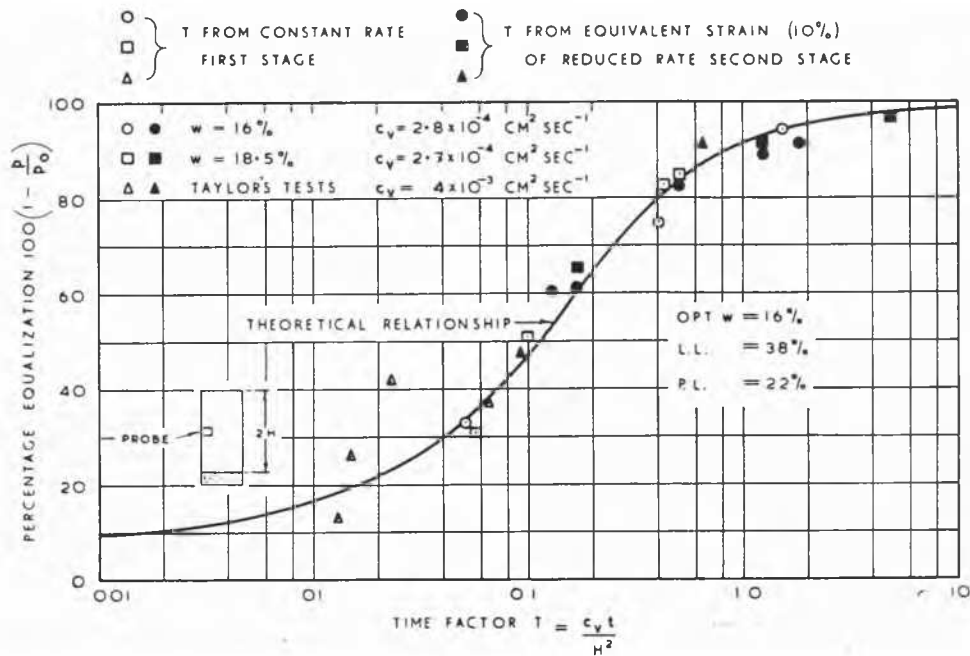


Fig. 7 Comparison of test results with theoretical relationship between percentage equalization of pore pressure in undrained tests and time factor. Compacted clay shale at two water contents and Taylor's tests on undisturbed Boston clay.

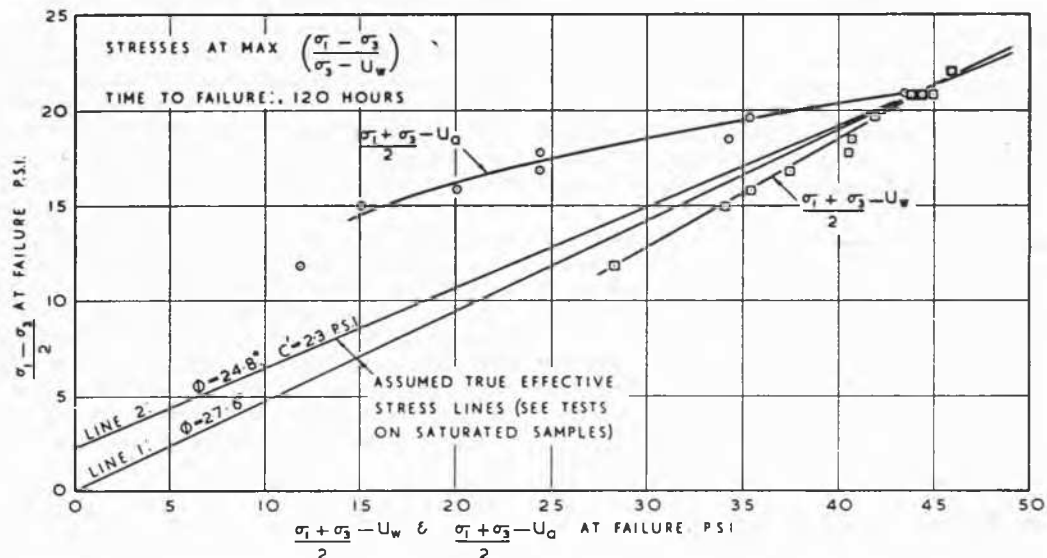


Fig. 8 Triaxial tests on a compacted shale (clay fraction 22 %) compacted at a water content of 18.6 % and sheared at constant water content.

## Références

- [1] BISHOP, A.W. ALPAN, I., BLIGHT, G.E., and DONALD, I.B. (1960), "Factors controlling the strength of partly saturated cohesive soils", *Proc. Conf. on Shear Strength of Cohesive Soils*, Amer. Soc. Civ. Engrs.; pp. 503-532
- [2] —, BLIGHT, G.E. and DONALD, I.B. (1960). *Discussion. Proc. Conf. on Shear Strength of Cohesive Soils*, Amer. Soc. Civ. Engrs.; pp. 1 027-1 042.
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- [4] *Proc. Conf. on Pore Pressure and Suction in Soils*. Butterworths, London, 1960.
- [5] TAYLOR, D.W. and CLOUGH, R.H. (1951). *Research on shearing characteristics of clay*. M.I.T. Report.

## M. MARCHAND (France).

Je serai tenté d'abord de répondre à M. Bishop, car son exposé m'a vivement intéressé, ayant eu récemment quelques difficultés à interpréter certains essais de mesure de pression interstitielle effectués sur des matériaux grossiers, non saturés, au cours de cisaillements à l'appareil triaxial. Je crois que l'on pourrait tirer deux conclusions de son exposé, peut-être un peu extrêmes :

— D'une part pour faire des mesures de pression interstitielle valables, il est nécessaire de faire des essais assez lents. Dans ce cas au lieu d'essais non drainés et lents, on pourrait aussi bien faire des essais drainés qui se passent de mesures de pression interstitielles, au moins dans de nombreux cas

où l'intérêt des essais non drainés avec mesure des pressions interstitielles réside dans la rapidité d'exécution de l'essai.

— D'autre part, il semble que lorsqu'on augmente la pression dans la chambre triaxiale, les différences entre les pressions de l'air et les pressions de l'eau diminuent, du moins les différences relatives, d'où l'intérêt des essais à très forte charge pour déterminer avec un minimum d'erreur l'angle de frottement interne des matériaux non saturés.

Je voudrais passer maintenant plus spécialement à une question de dispersion des essais de mécanique des sols. Je crois qu'il y a beaucoup à dire là-dessus. La dispersion commence à l'échantillonnage, suit dans l'essai proprement dit et se termine dans son interprétation.

Tous les essais n'ont pas la même importance. J'ai l'impression que pour les essais de classement, on peut se contenter d'une large dispersion étant donné que l'on en attend surtout des renseignements qualitatifs. On pourrait en dire autant des essais de compactage et de tassement parce que l'on observe fréquemment des différences suffisamment sensibles entre le laboratoire et le chantier pour que l'on n'exige pas de ces essais une grande précision.

Par contre, l'essai de cisaillement, lui, requiert la plus grande précision, car on l'utilise directement dans des études de stabilité, souvent avec des coefficients de sécurité très réduits qui ne tolèrent pas une grande marge d'erreur.

Pour l'utilisateur que je représente, n'ayant pas personnellement de laboratoire, le premier problème consiste à faire la part entre la dispersion inévitable et l'erreur. Actuellement je crois qu'il n'y a pas d'autre méthode pour y arriver que de multiplier les prélèvements, les essais, les méthodes d'essais, et même, lorsque c'est possible, les laboratoires, pour essayer de diminuer l'influence du matériel et du personnel.

Il sera certainement possible de diminuer ce nombre d'essais nécessaires lorsqu'auront été établies les cartes géotechniques dont M. Meyerhof parle dans son rapport général, cartes qui permettront de préciser certains rapports entre les essais de classement et les essais mécaniques. Naturellement, cela ne dispensera pas de faire des essais mécaniques, mais cela guidera l'opérateur et lui montrera tout de suite si le résultat obtenu est normal ou non, le résultat anormal pouvant d'ailleurs être valable. Dans le premier cas on se contentera de quelques essais de contrôle, dans le second il faudra faire une étude complète et préciser ou corriger la carte.

Toutefois, il y a une difficulté particulière pour l'essai de cisaillement dans ces cartes géotechniques, parce que les appareillages varient beaucoup d'un laboratoire à l'autre; ils sont souvent de conception originale; les méthodes d'essai aussi. Donc, je me demande s'il ne serait pas intéressant de normaliser certains types d'essais de cisaillement, ce qui n'empêcherait pas d'ailleurs de faire des essais non normalisés et au besoin de changer les normes si l'on s'aperçoit qu'elles sont dépassées par la technique.

De toutes façons le projeteur doit rester extrêmement méfiant devant les progrès de la technique; par exemple si un essai nouveau, indiscutablement correct, montre que les résistances au cisaillement étaient jusque là sous estimées, cela ne veut pas dire qu'il faille aussitôt raidir les pentes des barrages en terre en projet. Il faut peut-être corrélativement changer le coefficient de sécurité du calcul de stabilité.

En ce qui concerne le cisaillement, j'ai l'impression qu'une des causes de dispersion les plus nettes réside dans les diverses natures des éprouvettes que l'on cisaille. On utilise, en général, trois éprouvettes, et l'on sait qu'elles ne sont jamais absolument identiques. Cela peut conduire à des erreurs sensibles si, considérant isolément un essai sur trois éprouvettes, on l'interprète individuellement par une cohésion et un frottement auxquels on est tenté de conférer une réalité physique indépendante du domaine restreint de l'essai. Par exemple, sur soixante essais, on obtient par cette méthode d'interprétation individuelle des angles de frottement qui varient entre

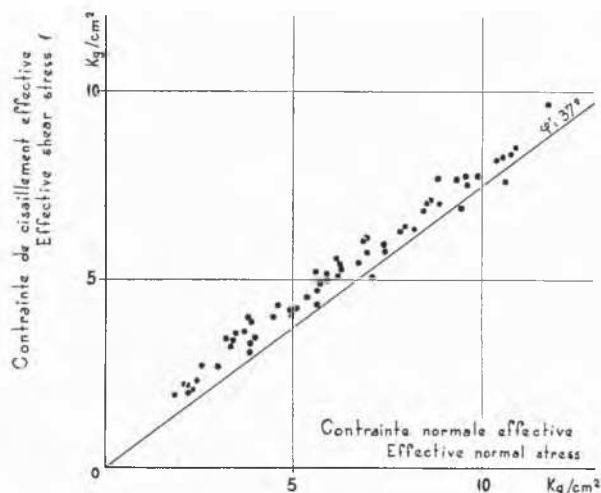


Fig. 9 Cisaillements triaxiaux non consolidés et non drainés normaux et cell-test. Matériaux 0,20 mm compensés. Contrainte sur le plan de rupture en fin d'essai.

31° et 39°; quand on porte tous les résultats sur le même graphique, on s'aperçoit que l'on peut retenir en fait un angle de 37° avec une bonne précision, ainsi qu'en témoigne la figure 9; une petite erreur sur la pression interstitielle, par exemple attribuable au défaut de saturation, a vite fait de faire basculer la droite de Coulomb, surtout si l'on travaille dans un domaine de pression restreint. A défaut d'un grand nombre d'essais on peut toujours diminuer les risques de l'interprétation individuelle en précisant bien le domaine d'emploi de la formule en cohésion et frottement.

Dans le but de réduire cette dispersion due aux éprouvettes, tout en cherchant à réduire les manipulations dans le cas de matériaux compactés et de cisaillement directs, nous avons utilisé un moule Proctor spécial que l'on pouvait diviser en un certain nombre de boîtes. Je crois que l'on avait ainsi le maximum de chances d'obtenir des éprouvettes analogues, à condition d'éliminer les extrémités du moulage.

Il y a aussi une méthode intéressante, la méthode hollandaise du Cell-test, qui se contente d'une seule éprouvette. Elle permet de gagner du temps, mais en contrepartie elle exige une grande dextérité de l'opérateur pour ne pas trop déformer l'éprouvette au premier chargement.

Pour ces essais systématiques ayant partiellement pour but d'estimer la dispersion, on peut souvent se contenter d'essais un peu simplifiés. Ainsi nous utilisons assez couramment en France, dans les cas des sols grossiers, la méthode de compensation pour préparer le matériau des éprouvettes compactées; elle consiste à remplacer les gros éléments par des éléments plus petits en poids égal. Par exemple, pour représenter un matériau naturel 0-200 mm, nous avons fabriqué des matériaux 0-60 mm où la partie 60-200 mm du matériau naturel était remplacée par du 20-60 mm, des matériaux 0-20 mm où la partie 20-200 mm était remplacée par du 5-20 mm. Et même, nous avons été, pour tester la méthode, jusqu'à fabriquer du 0-5 mm, où la partie 5-200 mm était remplacée par du 2-5 mm. Il faut vous dire que la partie 5-200 mm représentait 60 pour cent du matériau, c'était donc une compensation extrêmement poussée. Néanmoins, on a obtenu des résultats valables, même avec ce 0-5 mm, comme on peut le voir sur la figure 10. Tout ce que l'on a constaté, c'est qu'au compactage la granulométrie subissait un changement notable, mais c'est un autre phénomène indépendant de la méthode de compensation.

Cette méthode de compensation n'est d'ailleurs qu'exceptionnellement utilisée pour des essais de cisaillement car il est bien rare que la granulométrie des terres le justifie. Par

Matériaux compensés Soil with compensation	Essais non consolidés non drainés Unconsolidated and undrained tests
0 - 5 mm	• Normal
0 - 5 mm	▲ Cell-test
0 - 20 mm	• Normal

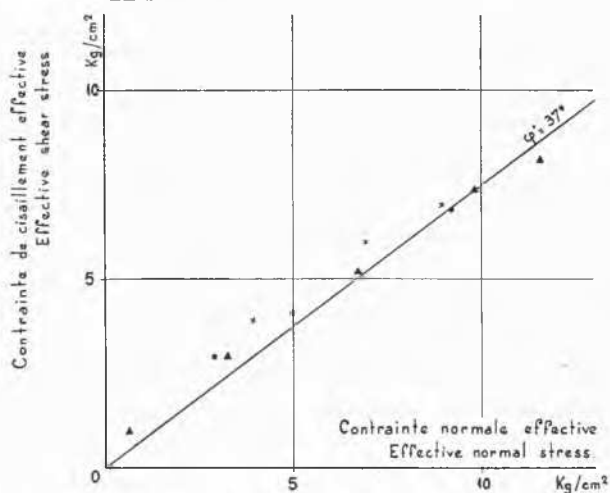


Fig. 10 Cisaillements triaxiaux.

• Cisaillement direct, drainé (S) Eprouvilles : $\phi 100$ mm hauteur 50 mm	• Direct shear test, drained (S) Specimens : $\phi 100$ mm height 50 mm
▲ Cisaillement triaxial, consolidé non drainé avec mesure des pressions interstitielles (R) Eprouvilles : $\phi 100$ mm hauteur 250 mm	▲ Triaxial shear test, consolidated undrained with pore pressure measurements (R) Specimens : $\phi 100$ mm height 250 mm
Matériau 0-20 mm compensé	Soil 0-20 mm with compensation

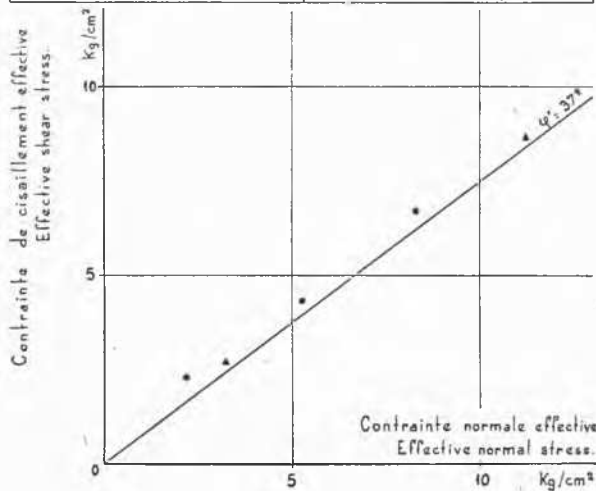


Fig. 11 Effet de paroi.

contre son emploi pour le compactage est plus courant. Des essais comparatifs, en compactage et perméabilité, portant sur des matériaux compensés 0-60 mm placés dans des moules de 400 mm de diamètre et des matériaux compensés 0-20 mm placés dans des moules CBR de 150 mm de diamètre, ont montré que les résultats différaient suffisamment peu pour que soit justifiée la validité de la méthode.

Dans le même esprit de simplification, je crois que l'on peut essayer de s'affranchir de certaines règles habituelles surtout lorsque ces dernières conduiraient à renoncer à l'essai, un essai imparfait étant préférable à pas d'essai du tout. Ainsi, dans un cisaillement direct avec des boîtes telles que l'épaisseur totale de l'échantillon soit de 50 mm nous avons utilisé des matériaux compensés avec des éléments allant jusqu'à 20 mm, ce qui, à priori, paraissait extrêmement abusif. Néanmoins, la figure 11 montre que le résultat est très correct et en très bonne concordance avec l'essai triaxial correspondant, sur éprouvette de 100 mm de diamètre et 250 mm de haut.

Finalement, en travaillant dans les meilleures conditions possibles, je crois que l'on devrait arriver, sur l'essai de cisaillement drainé, à une précision de l'ordre de 1 ou 2°, ce qui est amplement suffisant si l'on considère que le matériau naturel ou fabriqué varie toujours assez fortement d'un point à un autre; on est donc obligé de tenir compte d'un matériau sinon moyen du moins limite du côté de la sécurité et la dispersion propre de l'essai devient assez négligeable.

Néanmoins, il convient que les essais soient parfaitement exécutés, et que les normes soient toujours bien respectées. En particulier, il faut se méfier de la routine qui tend à faire dévier tout doucement des normes, ce qui peut conduire parfois à des résultats tout à fait scabreux.

En conclusion je vous prierais de bien observer que mon propos était essentiellement pratique et n'avait nullement la prétention d'apporter des précisions aux remarquables études récentes sur le cisaillement des sols, en particulier à celles présentées il y a un an à la Conférence de l'Université du Colorado.

#### Le Président :

Thank you very much, Mr Marchand. Well, that concludes the panel's discussion on subject (a). We now go on to subject (b), the dissipation of pore pressures — and I will ask Mr Denissov if he will be so kind as to open the discussion.

#### N. DENISOV (U.R.S.S.)

I should like to say a little about overestimation of the influence of pore pressure on clay strength in the problem of the stability of clay slopes.

The problem of pore pressure dissipation is only problem (b) in the list of subjects for discussion but I think this is a very important question — in fact, it is problem No. 1 in modern soil mechanics.

At the present time much attention is being paid to the effect of pore pressure upon the strength of soils. Of special interest are the well-known papers by Bishop, Skempton and a number of other scientists and engineers.

The problem of the pore pressure increase's influence upon soil strength is, generally speaking, that of their dependence on pressure. And the problem of the pore pressure increase's effect on clay slope stabilities is reduced to that of the possibility and degree of clay strength drop as the result of their unloading.

For sand, the strength of which is solely due to the effect of internal friction, the influence of pore pressure is surely and easily determined by the simplest experiments. When, however, sand is transformed into sandstone, which is accompanied by the development of "stiff" cementation bonds, the possibility of the effect of pore pressure upon the strength is eliminated. It is obvious that pore pressure can neither influence the strength of cementation bonds nor the strength of the particles.

The state of sandstone in natural conditions may be considered as near that of unjacketed samples in the laboratory. The experiments made with sandstone, marbles and concretes carried out by different researches (A.W. Skempton,

1961) show that pore pressure increase under such conditions does not cause any drop in the strength of materials.

In recent years many papers have been published (Ivan Popov, U.S.S.R., 1941, V. Florin 1959, N. Denisov 1941 and 1956, N. Denisov and P. Rebinder, 1946, T.W. Lamb 1961, L. Bjerrum and T.H. Wu, 1960, I. Rosenquist, 1959, H. Seed, J. Mitchell and C. Chan, 1960), in which more and more attention has been paid to the concept that the strength of clay is to a great extent due to the existence of stiff bond between the particles, developing because of the cementation of soil. The effect of the increase of pore pressure on strength can be manifest here as well as for sandstone only after the collapse of this bond. Just by the effect of cementation bond and its elimination can be explained the well-known difference in the strength of natural clay and remoulded samples of equal density.

Now let us consider the clays having no cementing bonds; their cohesion results from the influence of Van-der-Vals's forces, etc. It is well known that both the drained strength of these soils and their density increase under the growing pressure.

The pore pressure developing under the place of contact of the soil and the loaded surface somewhat diminishes the effective influence of the consolidating pressure. This, of course, tells on the rate of soil strength increment. Pore pressure dissipation in time is accompanied by a greater influence of an external load and by a higher degree of soil consolidation and its strength.

When a decreasing pressure acts upon such soils, their density and strength remain practically unchanged.

To prove this one can refer to the results of laboratory investigations and field observations which show the considerable strength of *over-consolidated* clays. It can be explained that in the process of previous consolidation their strength grew mainly because of bond increment accompanying the rise of particle concentration in a unit volume. The above-said may be illustrated by the test results for shear resistance of the alluvial clay (Fig. 12) in the normally consolidated state (line 1) when the density changed with pressure increase, and in the over-consolidated state (line 2) when the density change resulted from pressure decrease.

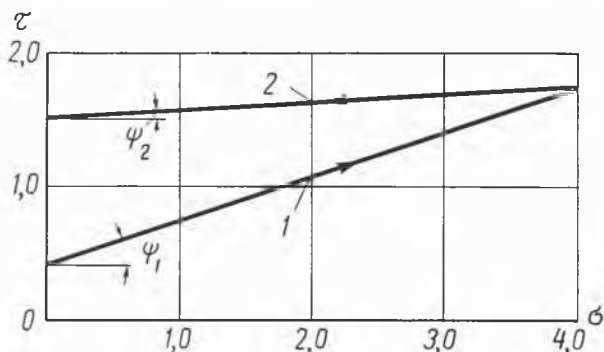


Fig. 12

The experiments were carried out under drainage and the stresses at the abscissa axes are effective. The figure clearly shows that the angle  $\psi_1$ , typical of the soil strength increase under loading, is substantially larger than the angle  $\psi_2$ , typical of strength drop under unloading. Hence, for different schemes of stress changes we must use different expressions showing relations between shear resistance  $\tau$  and consolidating pressure  $\sigma$ :

For loading

$$\tau_{\sigma'} = C_{\min} + \sigma \operatorname{tg} \psi_1$$

where  $C_{\min}$  — initial cohesion

$\sigma$  — pressure.

For unloading

$$\tau_{\sigma''} = C_{\min} + \sigma \operatorname{tg} \psi_1 - \Delta\sigma \operatorname{tg} \psi_2$$

$\Delta\sigma$  — pressure decrease.

From the above-stated, it follows that the pore pressure increment in the non-cemented clays forming the slope and diminishing effective stresses cannot substantially influence the strength of the clays and the stability of the slope.

At the same time it is clear that the use of the ordinary value of the *angle of friction*, the pore pressure action on the stability of slopes being taken into consideration, leads to a considerable overestimating of this influence. Thus the shear strength of the alluvial clay (Fig. 12) under the pressure of 4 kg./cm<sup>2</sup> is 1.73 kg./cm<sup>2</sup>. The pore pressure increase by 2 kg./cm<sup>2</sup> leads to a corresponding decrease of effective stress. If the shear resistance, corresponding to this new stress, is determined from the straight line 2 we receive 1.62 kg./cm<sup>2</sup>, and from the straight line 1 only 1.06 kg./cm<sup>2</sup>.

Thus with correct estimate of pore pressure influence the strength of the clay decreased by 6 per cent, and when using the relation illustrated by the straight line 1, the strength of the clays decreased by 40 per cent.

This can explain the fact that the coefficients of stability for a number of calculated slopes are much less than one. It is necessary to note that, as opposed to sands, the shear resistance of clays can change with the stress alternating only in cases when the alterations of this state is accompanied by the change of density and structure. Pore pressure fluctuations in the marine clays which are, in the slopes, in an over-consolidated state, cannot be accompanied by the volume change of these clays. That is why their strength can decrease in time as a result of the physical and chemical influence of pore water but not of its mechanical action.

According to the above-stated it can be said that the effect of pore pressure undoubtedly influences the strength of sand. The effect of this pressure upon the strength of clay soils with natural structure is usually estimated on the basis of laboratory tests on jacketed samples.

No doubt, there are natural conditions corresponding to these laboratory tests when a layer of a water-saturated soil is between the strata of practically impervious soils. However, the state of clay strata similar in their permeability somewhat differs from these conditions.

One thinks that the difference in permeability is the only reason for the distinction of properties of clay and sand. This explanation is not enough. The main reason for it is the different degree of colloidal-chemical influence in the strength of clay and sand.

From this point of view, because of the usual ignoring of both the residual nature of the strength of clay and the gain in strength of clay soils and the influence of cementation bonds, the effect of pore water pressure is substantially overestimated.

Whilst expressing a doubt about the possibility of the effective influence of pore water pressure on the strength of clay soils, it is at the same time impossible to neglect the great importance of observation of the rate of pore pressure dissipation. These observations give reliable data which enables one to judge the extent of the completeness of the process of the compression or expansion of soils and the rate of these processes. This in turn enables one to adjust the rate for construction work.

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#### Le Président :

Thank you very much, Dr Denissov. Would Dr Bishop care to continue the discussion?

#### M. BISHOP

I will try and deal first with one of the points which Prof. Denissov has just raised.

I have the impression that the cemented bonds to which Prof. Denissov refers have not been wholly over-looked in previous work relating to the principle of effective stress. Work which we carried out some years ago (Bishop and Eldin, 1950) on the influence of a finite contact area between the particles showed that the difference between the total stress  $\sigma$  and the pore pressure  $u$  controlled the deformation properties when the contact area was large just as it does in an uncemented sand at ordinary stresses where the contact area is extremely small. Tests on concrete by the U.S. Bureau of Reclamation (McHenry, 1948) and by other workers indicate that the strength of such materials, at least at com-

plete rupture, is controlled, to a close approximation, by the stress difference  $\sigma - u$ .

More recent tests in Portugal by Serafim (1954) show that the principle of effective stress applies also to the deformation of cemented materials, although account has to be taken of the behaviour of the material forming the bond.

Since time is short I could not do better than refer to the introductory address given by Prof. Skempton to the Conference on Pore Pressure and Suction in Soils in London (1960), in which much of this data is summarised. Theories were also presented in this address which apply to both sands and similar materials with an extremely small contact area and to concrete and natural sandstone and other rocks where the cemented area is very large. It appeared from Prof. Skempton's investigation that there is no discontinuity in the application of the principle of effective stress.

As far as I can understand from the previous discussion, the suggestion that pore pressure has no influence on strength, in the presence of stiff cementation bonds is a theoretical concept not yet supported by experimental evidence. I would agree that more attention needs to be given to the behaviour of natural samples at very small strains, but field evidence as well as laboratory tests will be needed before we reject a principle which hitherto has been extremely useful to us in understanding the properties of soil.

My second point relates to the dissipation of pore pressure in the field. During the construction of the Selsset dam in the north of England the opportunity was taken of confirming the design assumptions by a detailed series of field measurements of pore pressure.

There were two aspects to the problem. The dam itself was built on a soft boulder clay foundation which it was uneconomical to strip, and 18 inch diameter sand drains at 10 ft. centres were used to accelerate the consolidation of the foundation. Piezometers were placed at 14 points in the middle of groups of these sand drains to measure the dissipation of pore pressure and thus enable the field value of coefficient of consolidation to be compared with the laboratory value.

In addition the fill was to consist of the same impervious clay, having a permeability of about  $1 \times 10^{-8}$  cm. per sec. The average rainfall was expected to be about 20 inches in each construction season. The control of water content necessary to give low pore water pressures would have been uneconomical under these circumstances. The embankment

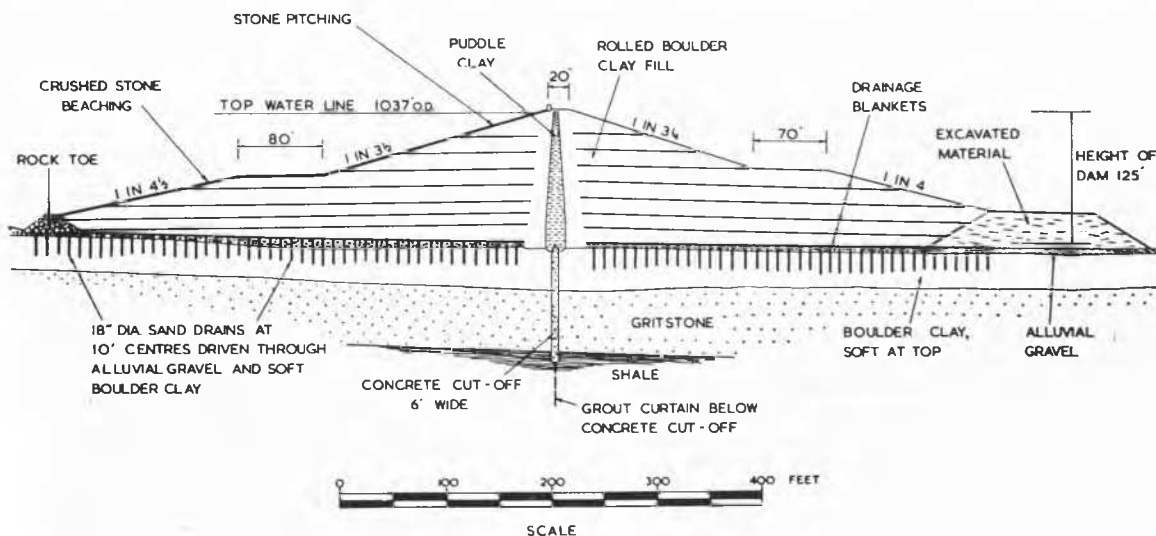


Fig. 13 Selsset Reservoir : Typical cross-section of embankment.

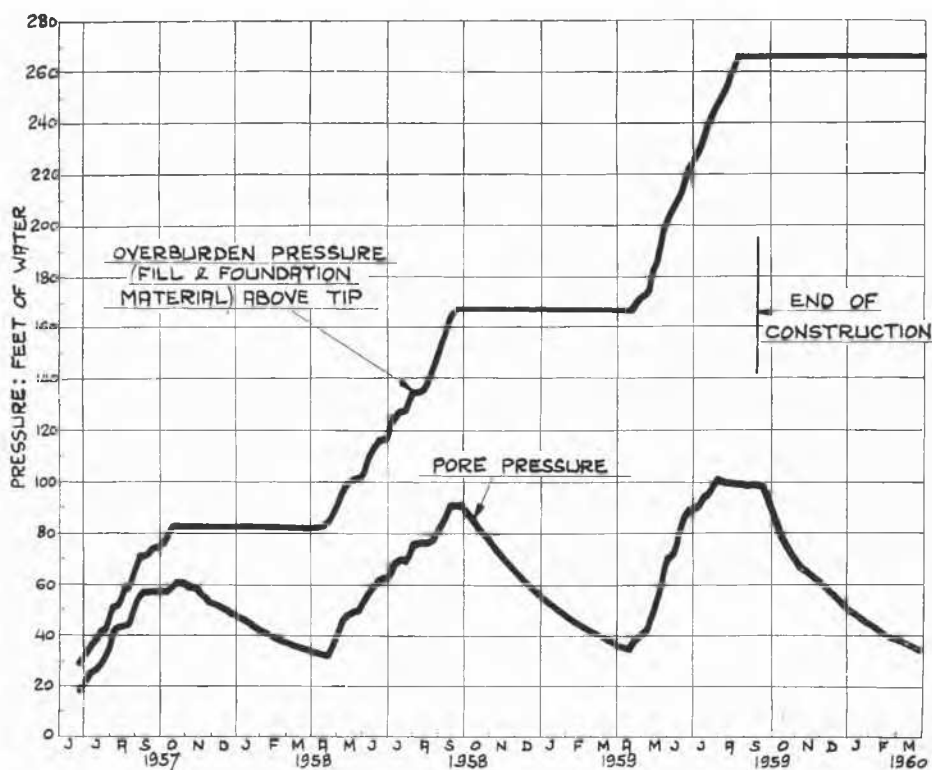


Fig. 14 Observations of pore water pressure in the clay foundation.

was therefore divided into layers sandwiched between horizontal drainage blankets at 15 ft. centres to permit rapid consolidation of the fill. In the lower part of the dam vertical as well as horizontal drains were used to take advantage of the horizontal permeability if it proved to be greater than that in a vertical direction. The cross section is shown in Fig. 13. Fuller details of the construction, instrumentation and analysis of the results will be found in other papers (Bishop, Kennard and Penman, 1960; Kennard and Kennard, 1962; Bishop and Vaughan, 1962).

The observations are at present being analysed in detail, but the preliminary results indicate that the foundation pore pressures have dissipated a little faster than predicted. Fig. 14 shows the build-up of load on the foundation and the corresponding changes in pore water pressure. The rapid increase in pore pressure during the construction season and the decrease during the shut-down period are most marked.

The preliminary estimate of coefficient of consolidation  $c_v$  from the field results gives a value approximately three times the laboratory value based on 4 inch diameter samples. In selecting the samples in the trial pits we obviously had to avoid the most stoney zones of the boulder clay, and this would have weighted the average of the laboratory tests to give a lower value of  $c_v$  than would be obtained from fully representative samples. However, considering the possible errors in measurement and analysis, I think the agreement is not unsatisfactory, particularly as the field value lies on the right side from the construction point of view.

In the compacted fill, which was mainly placed wet of the optimum water content, the average field value of the coefficient of consolidation as determined from the piezometer results is about 1.5 times the laboratory value based on samples from which material greater than 3/8 inch diameter was excluded. Within the fill itself material up to about 1 ft. diameter was included. The agreement is satisfactory — the error is again on the safe side.

These results show that we can rely on dissipation of pore

pressure in field problems and that we can make valid predictions on the basis of laboratory tests provided we use a certain amount of caution in selecting representative values.

My third point is that the final value of pore pressure depends not only on the rate of dissipation but also on the initial value of the pore pressure, and this depends on the magnitudes of the three principal stresses in the field. The General Reporter has drawn attention to the lack of data about the behaviour of soil under plane strain conditions, which apply in many engineering problems. I will give a few illustrations, therefore, of the kind of results we have been obtaining with the plane strain apparatus at Imperial College. Dr Clive Wood, Dr Cornforth and I hope to publish the detailed results in the near future.

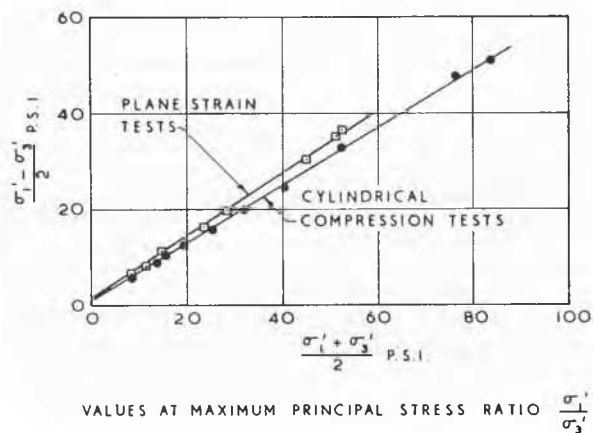


Fig. 15 The relation between strength and effective normal stress; a comparison of the results of plane strain and cylindrical compression tests on compacted soil.

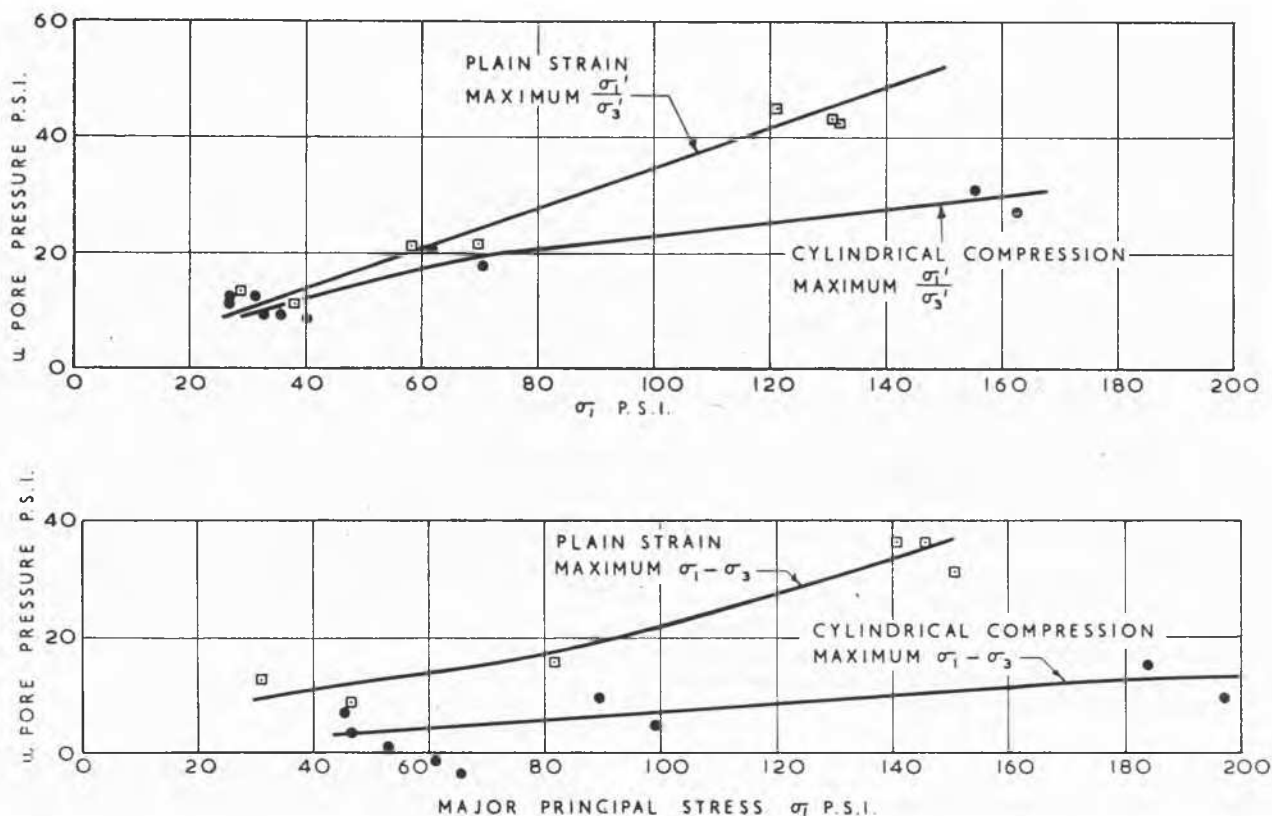


Fig. 16 The relation between pore pressure and major principal stress in undrained tests: a comparison between plane strain and cylindrical compression tests.

The apparatus will be familiar to those of you who visited our laboratory at the time of the London Conference. Samples 4 inches in height, 2 inches in thickness and 16 inches in length are tested in a cell in which the intermediate principal stress is measured under the condition of zero strain in that direction. Tests have been completed on computed fill and cohesionless sand and are at present in progress on a saturated clay.

Fig. 15 shows the relation between strength and effective normal stress for the compacted soil. A higher strength is clearly obtained in plane strain, the difference in the tangent of the angle of shear resistance  $\Phi'$  being about 10 per cent. This is significant from the engineering point of view although not dramatic. The difference is on the safe side if our estimates of factor of safety are based on cylindrical compression tests.

However Fig. 16 shows that the pore pressures set up under plane strain conditions are higher than those obtained in the conventional cylindrical compression tests, both at the maximum principal stress ratio and at the maximum deviator stress. Field values of pore pressure may therefore considerably exceed estimates based on the results of cylindrical compression tests, though if provision has been made for their observation and control this situation should be revealed and met without difficulty.

The tests on sand, Fig. 17, show the same trend towards a higher strength in plane strain in the denser samples. In the loose samples the difference is small. The trend towards a higher strength is accompanied by a smaller strain at failure and a smaller increase in volume, in the drained tests, before failure occurs. This corresponds to the higher pore pressures observed in the undrained plane strain tests.

The difference in the angle of internal friction of about  $4^\circ$  in the case of dense sand would imply an increase of 10 to 20 per cent in the factor of safety, in a slope stability problem,

over that based on cylindrical compression tests. In terms of bearing capacity the implied difference is very much greater. In Fig. 18 the increase in bearing capacity of a strip footing if the estimate is based on the plane strain test rather than the cylindrical compression test is plotted against porosity. The difference ranges between 30 per cent and 13 per cent. It is an interesting reflection that many people have obtained

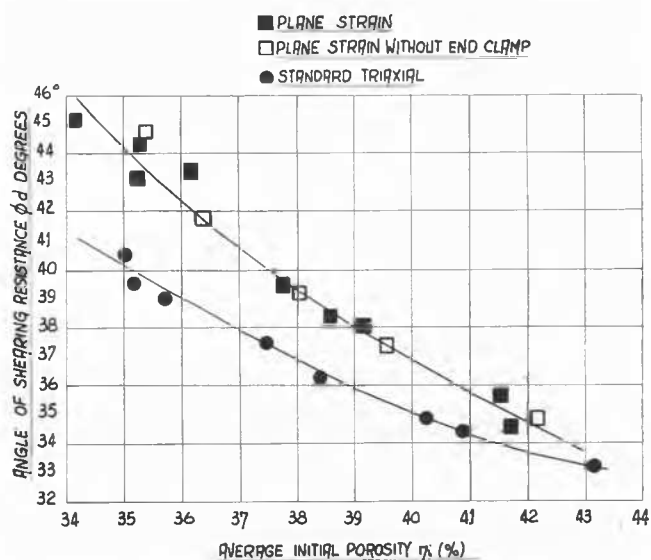


Fig. 17 The relation between drained angle of internal friction and initial porosity: a comparison between plane strain and cylindrical compression tests on sand.

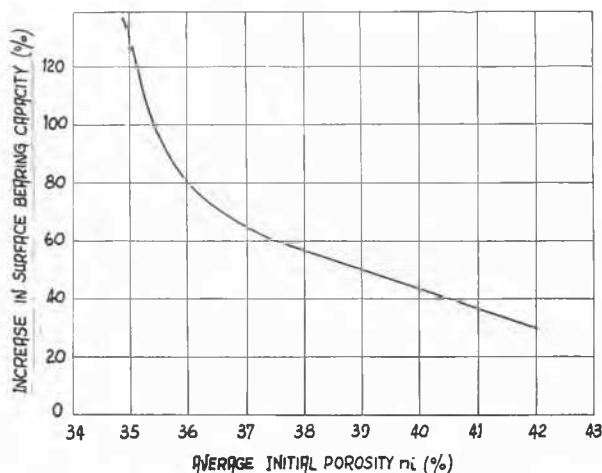


Fig. 18 The influence of type of test on calculated bearing capacity.

agreement between a proposed theory and the results of bearing capacity tests in terms of the cylindrical compression test. I hope they can now provide an equally rational explanation in terms of the plane strain test!

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#### Le Président :

Thank you, Dr Bishop. Would Dr Breth care to make a comment?

M. H. BRETH (Allemagne)

Mr Bishop's most interesting investigation shows us that the air pressure and the water pressure in the pore space of partly saturated soils are different. One can expect that those pressures will be in a state of equilibrium if the soil is consolidated.

In this state air pressure and water pressure in the pore space should be equal. If the air cannot get out of the pore space, or only after a long time, a residual pore pressure should occur.

During drained tests we several times measured a residual pore pressure. After what Mr Bishop said several problems

result for the praxis. With the usual measuring system we receive only the water pressure or the water pressure influenced by the air pressure. The result is that we do not obtain any regularity. We have measured pore water pressure in partly saturated soils in the center of large samples by triaxial tests and found out that pore water pressure depended upon the dimension and the rate of the load increment.

The influences changed with the preparation and the compaction of the sample and with the state of consolidation.

For instance, in a blanket of clay of 6 m. thickness under a dam, pore pressure due to quick drawdowns showed greater change during the first year than during the following years. Again we can see the influence of time.

Thus it will be very difficult to draw conclusions from the results of investigation into actual problems of engineering construction.

#### Le Président :

Thank you, Mr Breth.

I would like to introduce Prof. Geuze to this discussion. I think you know Mr Tan Tjong Kie was originally a member of the panel. He is unfortunately unable to attend and Prof. Geuze at very short notice has been so kind as to join us. Would you like to make some comments on section (b), Prof. Geuze?

M. E. GEUZE (Etats-Unis)

Thank you Mr Chairman, I would like to do that very much. May I use the privilege of continuing my remarks on question (c) after those I will make in connection with question (b). My reason for this is that the phenomenon of pore pressure and its dissipation obviously depends on the difference between the elastic and the viscous properties of the soil. These properties are in fact so closely interrelated, that we could hardly discuss either one of them without reference to the other.

Magnitudes of pore pressure in the voids of a supposedly saturated soil system depend to a great extent on the nature of the forces acting between the particles of the system. In the Terzaghi theory of consolidation these forces are assumed to be in the nature of a one-to-one relationship between the stresses and the strains. Considering the comparatively ideal mechanical properties of the pore fluid the relationship between the effective and neutral part of the stress is therefore a simple one.

If however the nature of the stress-strain relationship is complicated by a deviation from the assumed linear, Hookean behaviour of the soil skeleton, we can only assess its effect on the bulk behaviour indirectly through the measurement of pore pressure.

I understand from Dr Denissov that he has considered this effect in dealing with soil systems subjected to cementation bonds between the particles.

The properties of the soil structure then become of primary importance and I actually consider this to be our central problem in the study of the mechanical properties of saturated clays. Since we are obviously not in a position to remove the fluid from the voids without affecting the magnitude (and sometimes even the nature) of the particle bonds, we can only assess the effects of the nature of these bonds on the behaviour of the soil structure by indirect means, i.e. by the measurement of the pore pressures in the voids of the soil system (which we normally do when measuring on the gravity scale) and the stress-strain (both hydrostatic and deviatoric components) relations of the bulk material.

This problem appears when we consider the rate of pore pressure dissipation in elasto-viscous soil systems. The classic concept of Terzaghi's consolidation theory is then complica-

ated by the fact that the soil particle structure will show time-dependent deformations in shear and compression, which can not be explained by a simple elastic stress-strain model of the soil structure. In other words the Terzaghi theory of consolidation cannot be applied to soil systems with elasto-viscous properties. Successful attempts have been made over the last seven years to use simplified rheological models of elasto-viscous soil systems in order to establish their mechanical properties both in consolidation and in stress-strain behaviour of saturated clay systems (Tan Tjong Kie 1954, Schiffman 1959, Gibson 1960). These simplifications mostly bear on the elastic nature of the hydrostatic stress-strain relationship and the unique stress-rate of strain relationship of the deviatoric part of the stress system. Both the bulk modulus and the viscosity are then constants.

More recent investigations (De Josselin de Jong and Geuze, 1957) have shown that the viscosity of these saturated clay systems in a state of flow is of an even more unique nature than assumed previously. Arbitrarily chosen programs of loading and unloading by varying the deviator of stress accordingly resulted in proportional rates of flow. This evidence indicates that flow is a unique property of the system and that the viscosity is a constant parameter over a certain range of the time-scale.

Later investigations (Geuze, 1960) have showed proof of the fact, that these systems also possess elastic properties in the lowest range of stress. Complete reversibility was obtained at programs of loading and unloading with increasing magnitudes of the deviator of stress. This behaviour proves the absence of permanent parts of the strain, which are an indication of the slip of particles with respect to each other (Tan Tjong Kie, 1954). It therefore strongly supports the hypothesis of bonds existing between the particles, which can be overcome only at a certain level of the shear stress.

Two other observations are significant in this respect. Identical loading programs repeatedly performed on the same clay sample did not show essential differences in the stress-strain behaviour.

The maximum level of the elastic stress deviator did not vary substantially at repeated performances provided that the permanent strain at that level was kept under control. The transition from the completely reversible state of strain to those of the partly reversible state clearly showed.

The above mentioned facts have proved beyond doubt that the clay system has elastic properties within a certain limit of shear stress. The reasons why these properties did as yet not clearly show up in the conventional types of shear test has to be ascribed to the low level of the yield stress and the lack of information on the reversibility of the strains below that stress level.

The sharp transition in the strain behaviour is undoubtedly due to the application of the stress deviator as a loading system, which mobilizes the bonds oriented in the principal direction of shear simultaneously. As pointed out at an earlier opportunity (Geuze, 1948) the conventional type of loading in triaxial testing involves a rotation of the principal direction of shear, which in turn mobilizes the frictional forces between varying portions of the particles of a granular system according to the directions of their contact planes.

It is an interesting point, that the yield limit of the elastic range will give us an opportunity to give the often misused term *cohesion* its proper physical significance, instead of its current geometrical meaning as the (assumed) point of interception of the Mohr envelope with the shear stress coordinate axis.

An additional point about the strain-time behaviour at instantaneous loading is the retardation effect by the viscosity of the pore fluid as shown at the higher stress levels within the elastic range. This phenomenon repeats itself at any state of deformation involving the change of position of the

particles. We can as yet not make a clear distinction between retardation effects due to the fluid viscosity and the structural viscosity of the particle system. It is obvious — as stated in the early part of this discussion — that their mutual interference in the state of flow, has to be investigated in order to establish the effect of the fluid phase on the viscous properties of the soil structure. We can not however hope to obtain this information by simply removing the water from the voids or through its displacement by another type of fluid with a larger or smaller viscosity, as the magnitude of the bond forces and their mechanical nature largely depend on the nature of the fluid.

The limited amount of information obtained from a certain number of clays puts the yield limit at about 50-90 gr./cm<sup>2</sup>. Beyond this limit permanent strains will appear. They are an indication of the permanent changes of the soil particle structure. The increase of the permanent deformation with time at a constant magnitude of the stress represents the state of flow, which depends on the viscosity of the soil structure. No information is as yet available on the effect of the pore pressure on the rate of the deformation.

As shown by Casagrande and Wilson (1954) flow may be followed by failure at a constant magnitude of the stress. In summarizing their results these authors concluded, that the lower the stress the longer the period of time to failure. In one particular case the failure stress could be lowered by as much as 80 per cent in comparison with the failure strength at the normal rate of deformation, when a sufficiently long time to failure was allowed for.

This statement is significant because it indicates that below a certain rate of loading the failure condition is much stronger affected by the permanent part of the strain than by the stress level.

The ambiguity of this behaviour may well be ascribed to the interference of what is commonly known as thixotropy (Mitchell 1960), which lowers the resistance of the soil structure at high rates of strain and thereby shortens the time to failure period.

As has already been mentioned by the General Reporter, the stress-strain-time relationships for saturated clays under conditions of no drainage have been studied in some detail. The amount of information under various conditions of drainage and for anisotropic materials is very limited indeed. By the design of the tests these results do not warrant conclusions in view of the time dependent volume changes of the soil structure and its flow properties. These factors should have a substantial effect on the dissipation of the pore pressure.

The fact that the simplest case of deformation, the one-dimensional consolidation process, is obviously subjected to flow is a sufficient proof of this conclusion.

The so-called secondary time-effects, which occur under conditions of residual pore-pressure gradients at ever decreasing rates of strain, deserve our special attention. As I recently observed, these effects may even take place in saturated clays when subjected to hydrostatic states of stress, which should not produce shear in the bulk of the material.

Evidently the shearing type of deformation then occurs in the microstructure of the clay. This type of test may serve as a means to study the viscosity of the soil structure by the increase of the particle bonds with time at extremely small rates of strain.

These time dependent effects in the strain history of certain clays may well invalidate the relationships between the shear strength and the water content of these clays, which are mostly based on test results of the routine type where these bonds are lost at the state of failure. They would however show up in the type of test, as previously described, which allows for states of equilibrium at small strains and small rates of strain.

It is this point, which I understand to be raised by

Dr Denissov in connection with pore pressure, occurring in systems where these bonds had developed either through the process of consolidation in nature or by a rest period after dissipation of pore pressure in the consolidation test. My own measurements of pore pressure at the base of the consolidation samples showed, that the magnitude of the pore pressure at repeated loading decreased when the rest periods after previous loading increased.

The preceding statements and conclusions indicate that we have to study closely the properties of the bonds existing between the particles of a clay system, because they largely define the properties of the bulk of the material in either of the well known mechanisms used in soil mechanics to predict soil behaviour at varying states of stress.

From what I gathered Dr Bishop said with reference to statements made by Dr Skempton, the effective area of contact between the particles would be significant in the explanation of mechanisms of saturated clays. I do not think this to be a factor of such importance, because the bonds define the strength of clay structure even when the contact areas between the particles are negligible. I think that the magnitude of the bond forces and the nature of these bonds are of a decisive importance. His reference to properties of other building materials such as concrete f.i., does not apply as the bonds then are of a completely different nature.

Also, the analysis of consolidation tests in clays should be based on the actual states of stress and on the stress-strain-time characteristics of the material.

When looking at this subject from the point of view of its practical importance in foundation engineering, it is obvious to me that our primary interest is to distinguish the various types of cohesive materials as to their tendency to flow over long periods of time. Some clays are particularly susceptible to the effects of flow on their strength and we should therefore be careful in handling these in the designs of foundation structures.

#### Le Président :

Before we continue with our discussion of the visco-elastic properties of soil Dr Denissov would like to say a few words.

#### M. DENISOV

Dr Bishop's speech was a very interesting one, and I have listened to it with great pleasure, but I do not agree with some of his opinions. He said that one finds pore pressure in cemented materials such as concrete, rock, etc. It is true and I know the papers of the London Conference about Pore Pressure. But it is not proved that strength of cemented materials and rocks is decreasing due to the pore pressure increases. In my speech I have spoken about the overconsolidated clays in natural slopes. In this clay we found pore pressure, but I do not think that the fluctuations of pore-pressure in natural conditions can influence the stability of slopes. Therefore the pore-pressure increase cannot be the reason for essential decreasing of the safety number of clay slopes in natural conditions.

I have very little time so I shall say only that it is quite possible to understand many soil phenomena without taking into consideration the influence of pore pressure.

The increase of clay strength with accompanies the pressure increase is not due to the pore pressure dissipation. The reason for this is increase of cohesion due to the particles' concentration in the unity of volume. It is very important that this considerable gain in strength remain after the unloading of the clay.

#### Le Président :

Dr Bishop wishes to speak on the subject of the visco-elastic properties of soil, but I think only to make one comment.

#### M. BISHOP

Perhaps I could add one comment from a practical point of view on the dissipation of pore pressure in full-scale problems. The dam to which I referred earlier in this discussion has been built and is behaving satisfactorily. One of the reasons for our use of dissipation of pressure as a construction technique was that an earlier dam of very similar clay in an area of similar rainfall had slipped during construction when a little over half the height proposed for the Selset Dam (Banks, 1948). No special provision for the dissipation of pore pressure had been made in this case. Now, whether or not the theory is right, it is evident that the dissipation of pore pressure has a very marked effect on the shear strength of clay in full scale examples such as these. Until Prof. Denissov produces contrary evidence I think we must accept that there is some validity at least in the idea that changing the pore pressure alters the shear strength and deformation properties of actual soils.

I would like also to comment on what Prof. Geuze has said. He referred, as did the General Reporter, to the small amount of data we have so far on the rheological properties of soil measured in terms of effective stress parameters. One of the reasons for emphasising this point is indicated by my earlier remarks about the non-uniformity of pore pressure within soil specimens tested under undrained conditions. Tests carried out at a constant average water content — these have figured very largely in the literature of rheology — may be merely reflecting the effects of redistribution of water within the sample, and not the basic properties of the material.

We should therefore welcome the limited amount of data presented to the Conference — there is one paper from Japan, I believe — in which the long-term properties have been presented in terms of effective stress. Such data will enable us to make some estimate of the basic rheological properties of the soil.

Results published by Bjerrum, Simons and Torblaa in 1958 showed that in long-term undrained triaxial tests the pore water pressure increased very considerably with the duration of the test. This may be partly a function of the true properties of the soil or it may be partly a consequence of the test procedure, but it certainly had a very marked influence on the reduction in strength of the samples.

In this connection it is worth recalling that the figure Prof. Geuze gave for the reduction in strength was from tests by Casagrande and Wilson, in which about 80 per cent of the strength was lost, on a long-term basis, in one or two cases. These were, as I recall, undrained tests, and I do not think we shall find any such marked decrease in drained tests. There will be a decrease, but it will be much smaller in magnitude, otherwise most of the world would be as flat as Holland.

While on this subject I would like to draw attention to a very important paper given by Dr Hvorslev to the Research Conference on the Shear Strength of Cohesive Soils organised by the American Society of Civil Engineers last year. In this paper he considered in detail the physical components of the shear strength of saturated clays — cohesion, true angle of internal friction and that part of the strength which depends on the rate of volume change at failure — and the extent to which they are time-dependent. Anyone who wishes to take a broad view of rheological properties should include this in his reading, as well as the more widespread — and sometimes misleading — literature based on the results of undrained tests.

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- [2] BJERRUM, L., SIMONS, N. and TORBLAA, I. (1958). "The effect of time on the shear strength of a soft marine clay". *Proc. Brussels Conference on Earth Pressure Problems.*, vol., pp.148-158.
- [3] HVORSLEV, M. J. (1960). "Physical Components of the Shear Strength of Saturated Clays". *Proc. Research Conference on the Shear Strength of Cohesive Soils, ASCE*, pp. 169-273.

## Le Président :

Would you like to comment, Dr. Geuze?

## M. GEUZE

In the first place, Mr Chairman, the flatness of Holland has nothing to do with the objections of Dr Bishop. But that is not the essence of what I have in mind.

Dr Bishop particularly mentioned a type of test in which drainage was allowed to take place during a long-term creep process. It is obvious that my statements were limited to the case of no drainage because it is only for that particular state that the strain-time relationship are not or hardly affected by the spherical component of the state of stress. As previously shown (Geuze, 1948) both the consolidation test and the (direct) shear test are essentially of the mixed type as in both tests the strains result from the combined action of compression and shear.

If drainage is allowed to take place, we have no means to distinguish between the creep in consolidation by the spherical component of the state of stress and the creep or flow caused by its deviatoric component. This distinction was made in a hypothetical manner on the basis of results obtained with standard type equipment in my paper to the 1948 I.C.O.S.E.F. It seems however that similar results of a more dependable nature will appear in tests as previously described.

Dr Bishop's reference to the observed creep behaviour of the material under drainage conditions therefore is not valid under conditions of constant water content; I am afraid that I can not agree with his statement with regard to the tests as carried out by Casagrande and Wilson.

## Le Président :

To conclude this opening panel discussion I think Prof. Meyerhof would like to say a few words.

## Le Rapporteur Général :

We have listened to an interesting panel discussion in which the three main subjects suggested at the end of the General Report have been considered. Regarding the scatter of soil mechanics tests M. Marchand has mentioned that the variation in the field can be greater than in the laboratory and he drew particular attention to the difference between problems of local stability and those of general stability, and the use of soils in their natural state as distinct from soils in the compacted state. I think that some of these factors are already taken care of in engineering practice by the different margin of safety which we use in connection with these soil properties. Thus, the factor of safety in the design of earth dams is commonly of the order of 1.5, this being a problem where we use soils in the compacted state. In foundation engineering, on the other hand, we use a factor of safety of 2 to 3, since we are here concerned with soils in their natural

state in which the variability is greater and where we have to provide a similar margin of safety as in the structure carried by these foundations.

The second problem dealt with the pore pressure dissipation. Here we had the interesting results of plane strain compression tests compared with those of triaxial tests. After reviewing the papers to the Conference I made the comment that we need more data on the important case of plane strain, which holds in earth dams, the stability of slopes and retaining walls and in strip foundations. It was therefore very gratifying to hear Dr Bishop giving the results of his important tests. He showed that the strength under plane strain compression is generally greater and the failure strain is generally smaller than in the axial symmetrical case of compression and the pore pressure is greater in plane strain than in triaxial compression. These factors have important consequences for our understanding of the fundamental properties of soils.

He also drew attention to the effect of these results on foundation problems. In my paper on piled foundations to this Conference (3B/16) I mentioned the lack of such data and indicated that a difference of only about 10 per cent in the angle of internal friction is necessary for us to be satisfied with the present theories of strip foundations on sands. In other words, it is now not necessary to revise the theories, but we did not have the right soil strength data to go with these theories. As I mentioned in Germany last week, if we use the right theory with the right soil tests we get the right answer!

Under the third subject heading for discussion, Dr Geuze referred to the importance of the visco-elastic properties of soils, the effect of stress reversals on the Bingham limit and the viscosity of soil-water systems. It was interesting to hear that the Bingham limit of clays of 30 to 90 gm./sq. cm. which he mentioned, is of the same order of magnitude as the shear strength at the liquid limit. Whether this has any connection I do not know, but it was interesting to observe this simple approximate relationship.

## Le Président :

Well, ladies and gentlemen, that concludes the panel's contribution to this discussion. I now propose that we adjourn until a quarter past 11. Will those members of the Conference who wish to take part in the discussion later be so kind, during this interval, as to write their names and nationalities, and the subject on which they want to speak, on a piece of paper and send it up here, so that Mr Meyerhof and myself can deal with them before we resume our discussion?

(La séance fut suspendue de 11 h. à 11 h. 15)

## Le Président :

We now open the second part of our discussion. We begin with subject (a) and I will ask Mr Ranganatham to make the first contribution.

## M. B. V. RANGANATHAM (Inde)

The problem of scatter of soil mechanics tests can best be studied with remoulded specimens. In case of undisturbed soil samples it will be difficult, if not impossible, to distinguish the scatter of test results from the variability of the particular soil under test.

A special technique has been adopted to prepare remoulded clay samples at the Soil Mechanics Laboratory of the Indian Institute of Science, Bangalore. The technique, very briefly, is as follows:

The clay water suspension is stirred well with a half horse-power stirrer for sufficient time to ensure thorough and uni-

SERIES NO. →		1			2			3			4			VARIATION %	
PROPERTY MEASURED ↓		A	B	VAR. %	A	B	VAR. %	A	B	VAR. %	A	B	VAR. %	AVER.	MAX.
FINAL THICKNESS (INS)		·475	·470	0·33	·498	·492	0·61	·383	·373	1·32	·475	·464	1·17	0·86	1·32
FINAL M.C. (%)		29·64	28·78	1·47	29·57	28·76	1·39	51·44	51·27	0·17	43·21	42·01	1·41	1·11	1·47
$C_v$ IN $\text{IN}^2/\text{MIN} \times 10^{-4}$ when final pressure is	1 ton/ft. <sup>2</sup>	3·48	3·54	0·85	9·30	8·65	3·62	106·5	102·8	1·77	1·45	1·35	3·57	2·45	3·62
	2 ton/ft. <sup>2</sup>	4·17	3·99	2·16	8·45	7·74	4·39	112·0	103·4	3·99	1·62	1·55	2·21	3·19	4·39
	4 ton/ft. <sup>2</sup>	4·75	4·70	0·54	7·92	7·34	3·80	85·86	86·50	0·37	1·65	1·53	3·77	2·12	3·80
$C_c$		·465	·495	3·12	·335	·345	1·47	1·092	1·162	3·11	·610	·595	1·24	2·24	3·12
$M_v$		·0475	·0525	4·90	·0372	·0404	4·12	·0668	·0733	4·64	·0548	·0577	2·58	4·06	4·90
RECOVERY RATIO (X)		10·23	10·20	0·15	9·43	9·72	1·51	2·37	2·46	1·86	24·00	23·21	1·67	1·30	1·86

INITIAL THICKNESS OF SAMPLE = 0·76"

Fig. 19

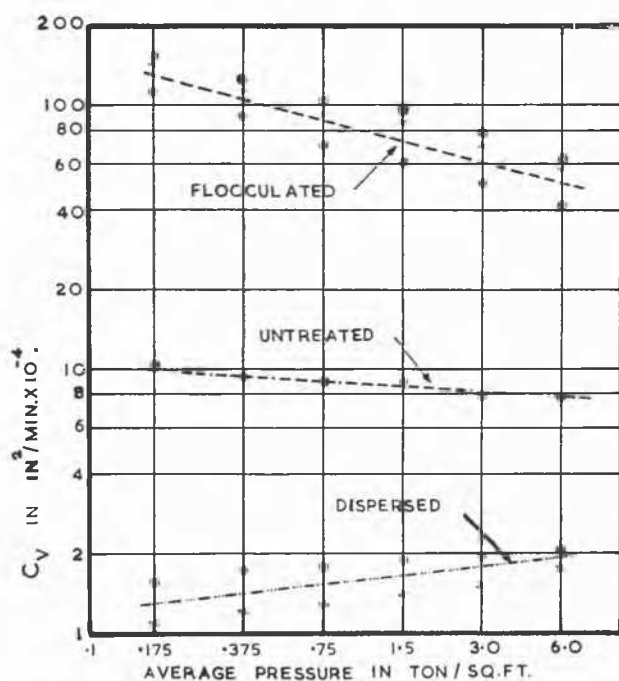


Fig. 20

form mixing. It is then poured into a specially designed porous cylindrical mould. A porous circular disc is placed on top of this suspension, the load on which is gradually increased to reach a final intergranular pressure of 0·05 tons/ft.<sup>2</sup> in about ten days. The samples so obtained are of adequate consistency to be handled and trimmed in the standard consolidation rings.

A wide series of standard oedometer tests, using such remoulded samples with two tests in each series, has been conducted. The slight variations in all the significant properties measured are clearly brought out in the typical test results shown in the Table (Fig. 19).

I have observed some very interesting changes take place when an expansive tropical clay is treated with a flocculant or a dispersant. When treated with lime, a flocculant, there is a marked increase in the coefficient of consolidation and a decrease in the recovery ratio (vide Fig. 20). When the same clay is dispersed the phenomenon is reversed, i.e., there is a decrease in the coefficient of consolidation and an increase in the recovery ratio. This shows that the flocculation of expansive clays results in a better drained and more stable structure.

Another striking feature which in spite of carefully conducted repetitions was observed in a total of over a hundred laboratory compression curves, is shown in Fig. 21. The early deviation from the straight line is probably due to the

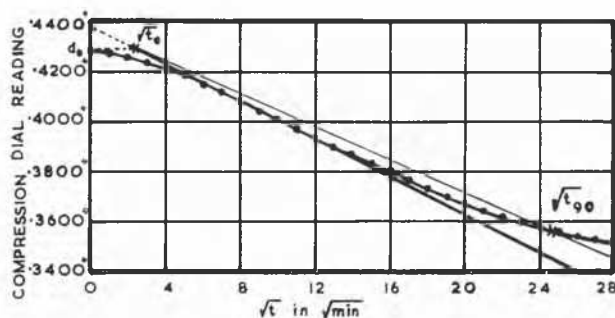


Fig. 21

time lag experienced in the actual pore pressure build-up within the sample, as against the instantaneous build-up assumed in the theory.

With regard to the statistical correlation of index properties and the compressibility characteristics, I wish to bring to your attention an important point. Index properties are necessarily determined from remoulded samples while consolidation tests are carried out on undisturbed soil samples, being designed to predict the settlement of buildings. Hence such correlations should at best be limited to the regions where the sensitivity of the clay has been established to be about unity.

**Le Président :**

Mr Ashbee, would you care to make a contribution?

**M. R. A. ASHBEE (Grande-Bretagne)**

I would like to limit my remarks to what I said in a written contribution \*, in which I dealt with one or two difficulties we had in getting consistent test results in undrained triaxial tests from silty soils being sampled under water. Thank you.

**M. J. OSTERMAN (Suède)**

In the panel discussion, Dr Bishop showed a diagram on the differences of soil strength in the cases of plain shear and of triaxial tests. I noted that the differences seemingly should vanish when the angle of shearing resistance approximates a certain value.

It seems to me that this angle is about the same as in the case of the critical void ratio, in Sweden often supposed to be about  $32^\circ$  for sands. At this state of density, I suppose there is very little reason for differences existing.

In other states of density it is necessary to study the boundary energy corrections when comparing angles of shearing resistance. If this is not done, the values found at structural break-down will be extremely low.

In division 1 of the Conf. Proceedings 1961, a diagram is given where the measured and the calculated strength results do not agree. I have a strong feeling that these calculations do not correspond to nature; for instance values of earth pressure at rest are applied to the failure process.

Sometimes it seems difficult to separate drained shear from undrained. This question should, however, be connected to dilatancy and contraction of the soil skeleton at a virtual shear. When dealing with sensitive clays, the points mentioned above are of special interest.

In this connection the changes in the pore pressure are important, see for instance the paper on yielding of soils by Roscoe et al., *Geotechnique* 1958.

**M. KEZDI (Hongrie)**

Monsieur le Président, Mesdames, Messieurs, je voudrais présenter quelques observations à propos de la dispersion des résultats des essais de mécanique des sols.

Ces remarques touchent l'essai œdométrique, qui est en usage encore, mais peut-être pas pour très longtemps, et sert pour le calcul des tassements des fondations.

Nous avons fait de nombreux essais avec une argile remaniée et homogénéisée. Les dimensions et les rapports de la hauteur et du diamètre variaient entre 20 cm de diamètre et 10 cm de hauteur, jusqu'à 5 cm de diamètre et 1 cm de hauteur, avec les rapports :  $H/D = 2:1$  et  $1:10$ .

Le sol était une argile saturée, avec un indice de plasticité de 30 pour cent, une teneur en eau de 26 pour cent à 28 pour cent. En traçant les courbes œdométriques, les courbes de compression, nous avons eu une dispersion considérable qui atteignait 100 pour cent.

Nous avons essayé de réduire cette dispersion en considérant la friction sur la surface latérale des échantillons cylindriques. Mon adjoint à l'Université, M. Balla, de Budapest, a mis sur pied dans ce but une théorie fournissant l'état complet des tensions dans les échantillons, et nous avons fait usage aussi des formules approximatives de Muhs et de Kany. Nous avons tracé les courbes. Résultat : la dispersion restait presque du même ordre.

Ensuite, nous avons pris les valeurs correspondant à une pression verticale de 1 kg par centimètre carré, et nous avons calculé de nouveau les tassements spécifiques en considérant ces points comme points initiaux communs. Dans ce cas, les courbes nouvelles se trouvaient dans une bande assez étroite, donnant une dispersion de 7 pour cent.

En conclusion, nous avons constaté que la dispersion est causée par les petites irrégularités et inégalités sur les surfaces des échantillons, des troubles initiaux dans le chargement, et non pas par la différence des dimensions et des rapports des échantillons.

**M. K. H. ROSCOE (Grande-Bretagne)**

Firstly may I say how much I welcome Dr Bishop's doubts concerning the application of results of triaxial tests to the many plane strain problems that occur in soil mechanics.

When in 1958 Drs Schofield, Wroth and I published our concept of representing the yielding of soils by a unique surface in a space of three variables, two relating to stresses and a third to voids ratio, such as shown in Fig. 22, a great deal of judgement had to be used when assessing the values of triaxial tests. We accepted only the results of triaxial tests in which the sample was "work hardening", but treated with great caution all results in which samples "work softened" because they tended to fail in a thin zone. Furthermore we published in 1959 some criticisms of the triaxial method of testing to which I could add extensively in the light of recent tests carried out at Cambridge. I doubt the validity of the results from Dr Bishop's plane strain apparatus to the same extent as triaxial test results since the former apparatus is prone to many of the same potential sources of error as the latter.

In 1946 I began to devise a plane strain simple shear apparatus which would impose uniform strain throughout a soil mass since under these conditions measurement of boundary movements would give a true measure of changes of the voids ratio. Subsequent improved models are described in these by Wroth (1958), Poorooshasb (1961) and Thurairajah (1961).

Poorooshasb's model, shown in Fig. 23, was used to carry out all the tests with varying degrees of controlled dilatation which are described in paper 1/51. Steel balls (1 m.m. dia.) were selected as the medium for these tests since they did

\* See page 117.

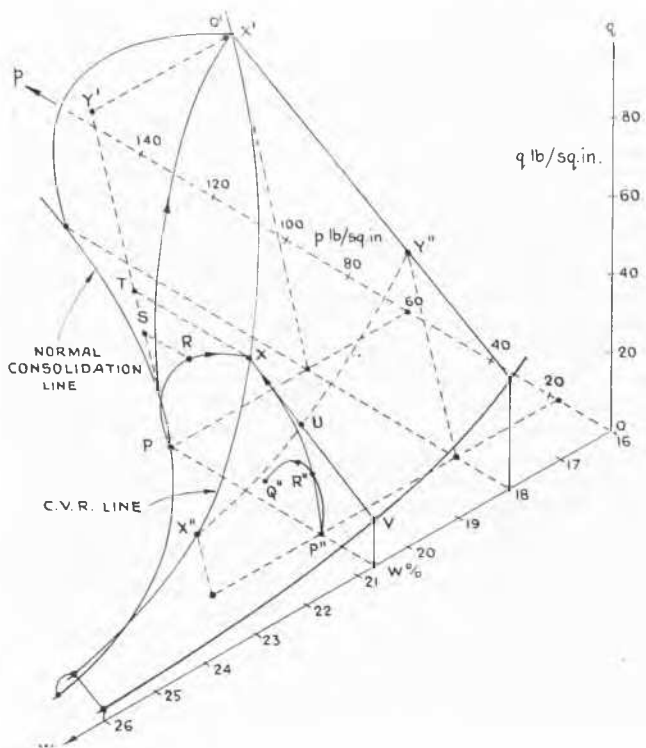


Fig. 22 Isometric view of yield surface for Weald Clay.

$$q = \sigma_1 - \sigma_3 ; p = \frac{\sigma'_1 + 2\sigma'_3}{3} ; w = m/c$$

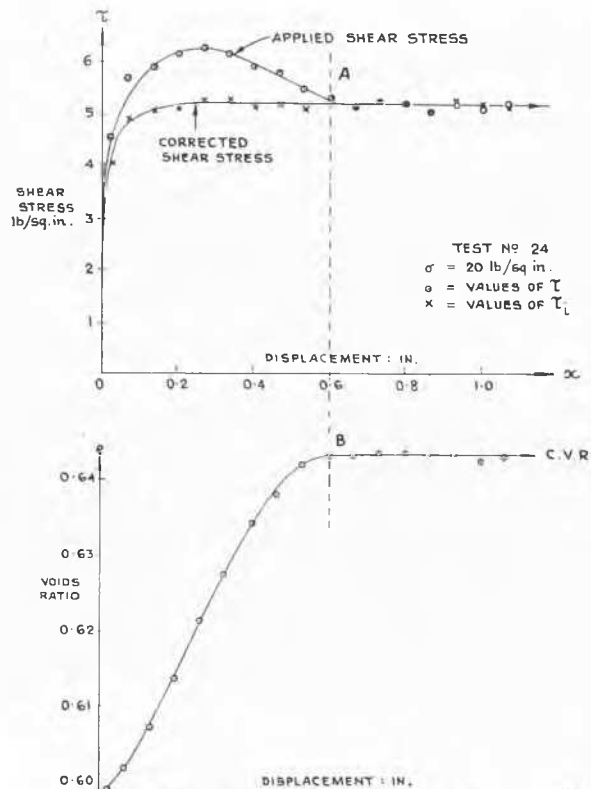


Fig. 24 Simple shear test on steel balls.

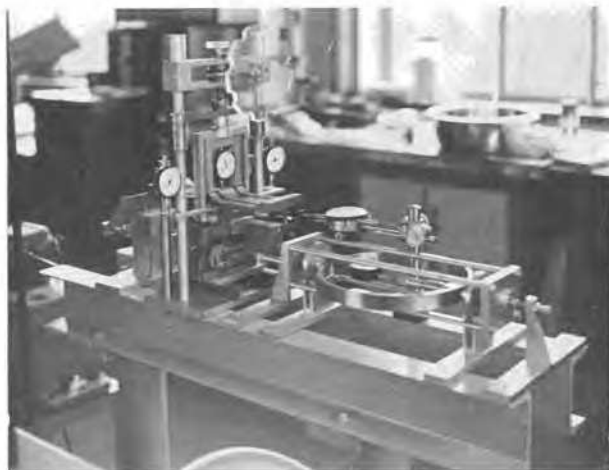


Fig. 23 Simple shear Apparatus.

not break or distort and thereby cause errors in the measurement of voids ratio.

Fig. 24 shows "stress-strain" and "voids ratio-strain" curves for drained simple shear tests on such a medium and the corrected stress  $\tau_c$  is obtained from the applied stress  $\tau$  as described in paper 1/51. Note the constancy of the voids ratio and the shear stress for strains (exceeding a shear angle of  $37^\circ$ ) to the right of the dotted line AB, when the originally randomly packed sample has reached the critical state at which it will continue to yield at constant stress and constant volume. In triaxial tests on this medium this constancy cannot be achieved; the sample appears to continue to dilate despite axial strains exceeding 30 per cent.

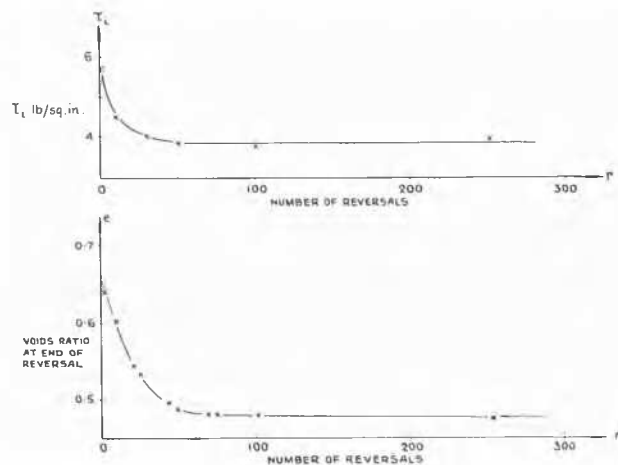


Fig. 25 Reversal tests on steel balls.

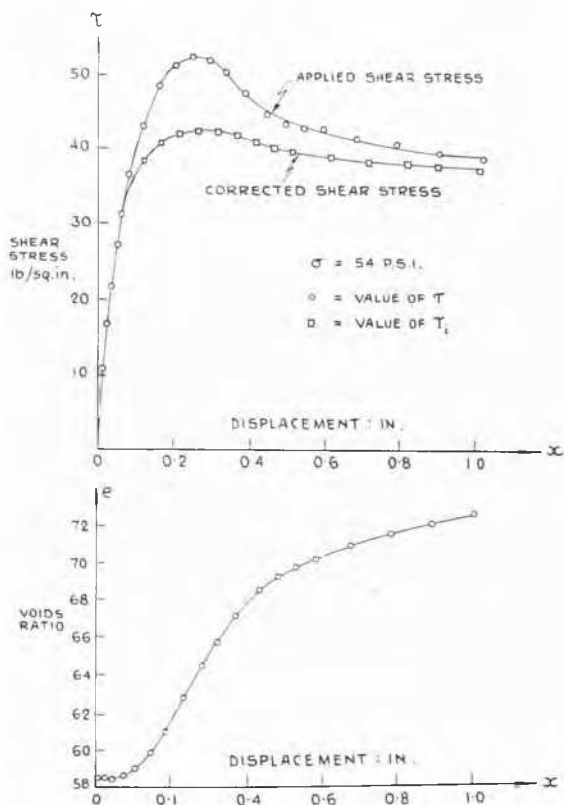


Fig. 26 Simple shear test on Leighton Buzzard sand.

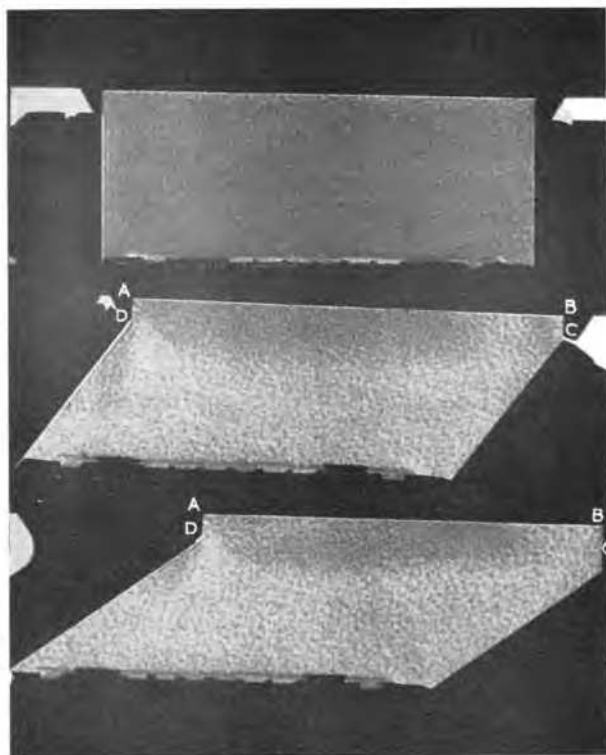


Fig. 27 Leighton Buzzard sand in simple shear strains.  
Top (0°), Middle (38°), Bottom (63°).  
 $e_0 = 0.59$      $\sigma = 11.6$  psi

Now consider the results of tests on sand. I do not believe that an apparatus exists that will properly measure the stresses and voids ratio changes of a sand when subjected to shear stress. Typical results of a drained simple shear test are shown in Fig. 26 and it will be noted that despite achieving a constant shear stress the sample appears to be continuing to dilate despite a shear strain exceeding 50°. The same trouble occurs in triaxial tests. To investigate this Thurairajah (1961) developed a model of the simple shear apparatus in which X-rays may be passed through a sample throughout a test. Fig. 27 shows three such photographs.

The upper photograph shows the initial uniformity of the packing of the sample and the middle and lower photographs clearly show how uniform the dilatation is throughout the sample except for the "dead zone" ABCD which continues to dilate and reduce in size up to the limit of travel of the apparatus. The apparatus is now being modified to eliminate this dead zone.

In conclusion may I make a plea for a closer study of the reliability of test methods. Only the results of reliable tests should be used to develop hypotheses otherwise the latter merely explain the peculiarities of the test equipment rather than the medium under test.

**Le Président :**

We pass to subject (b). Dr Šuklje ?

**M. ŠUKLJE (Yougoslavie)**

In the paper 1/23 Prof. J. Brinch Hansen has adopted the Terzaghi consolidation theory for compressibility coefficients increasing with time due to secondary time effects. Thus as the pore pressure dissipation corresponding to the so improved classical theory is accepted regardless of the layer thickness, the new method cannot be applied when the plastic resistance gives rise to a faster pore pressure dissipation of the sample (e.g. in the case presented in the Fig. 1, 2, 3, 4 and 7c of my above mentioned paper). If however the primary consolidation curve of the sample is close to the improved classical theory, the Brinch Hansen method and the isotaches method give comparable results. In Fig. 28 a comparison is shown for the lacustrine clay sample presented on Figs. 5 and 6 of my paper (Šuklje 1957).

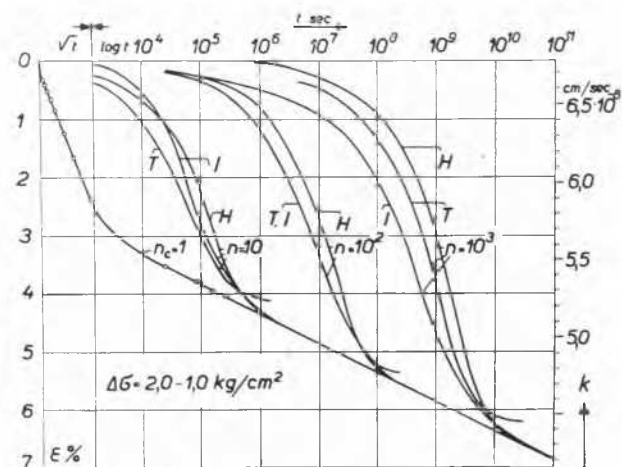


Fig. 28 Discussion on pore pressure dissipation.

In the first moment after the load increment the volume change of a saturated oedometer sample is zero provided the compressibility of the pore water is neglected. Assuming the validity of basic stress-strain relationship of the math-

ematical theory of elasticity, and considering zero lateral strain in oedometer tests, the additional intergranular stresses at the beginning of the consolidation process of an anisotropic sample are zero if the condition

$$1 - (\mu_{12}\mu_{21} + \mu_{23}\mu_{32} + \mu_{31}\mu_{13}) - (\mu_{12}\mu_{23}\mu_{31} + \mu_{21}\mu_{32}\mu_{13}) = 0 \dots (1)$$

( $\mu_{ik}$  is the Poisson's ratio governing the influence of the stress  $\sigma_k$  on the strain  $\varepsilon_i$ ) is not fulfilled. If it is fulfilled, the initial pore pressure is determined by the equation

$$\frac{\Delta u}{\Delta \sigma_1} = \frac{1 - \mathcal{C} \frac{\Delta \sigma_3}{\Delta \sigma_1}}{1 - \mathcal{C}} \quad (2)$$

The coefficient  $\mathcal{C}$  is given by the expressions

(a) in the case (2)  $\equiv$  (3) :

$$\mathcal{C} = 2\mu_{13} \frac{E_1}{E_3} \quad (3a)$$

( $E_i$  is the deformation modulus in the direction ( $i$ )) under the condition

$$2\mu_{13}\mu_{31} = 1 - \mu_{23} \quad (1a)$$

(b) in the case (2)  $\equiv$  (1) :

$$\mathcal{C} = \frac{1}{2\mu_{31}} \cdot \frac{E_1}{E_3} \quad (3b)$$

under the condition

$$2\mu_{13}\mu_{31} = 1 - \mu_{12} \dots (1b)$$

(c) in the case (1)  $\equiv$  (2)  $\equiv$  (3), that is for isotropic soils

$$\mu = 0,5 \quad (1c)$$

$$\mathcal{C} = 1 \quad (3c)$$

and consequently

$$\Delta u = \Delta \sigma_1 = \Delta \sigma_3 \quad (2c)$$

In the non confined conditions the assumption of zero volume change in the first moment of consolidation leads to the equation

$$\frac{\Delta u}{\Delta \sigma_1} = \mathcal{A} + (1 - \mathcal{A}) \frac{\Delta \sigma_3}{\Delta \sigma_1} \quad (4)$$

with the following definition of the coefficient  $\mathcal{A}$

$$\mathcal{A} = \frac{\mathcal{C}_{c1}}{\mathcal{C}_{c1} + \mathcal{C}_{c2} + \mathcal{C}_{c3}} \quad (5)$$

where

$$l_{c1} = \frac{3}{E_1} (1 - \mu_{21} - \mu_{31}) \quad (6)$$

and similarly for  $\mathcal{C}_{c2}, \mathcal{C}_{c3}$ . In the figure 29 the values of  $\mathcal{A}$  are presented as functions of the ratio  $\mathcal{C}_{c3}/\mathcal{C}_{c1}$  for the cases.

(a)  $\mathcal{C}_{c2} = \mathcal{C}_{c3}$ ,

(b)  $\mathcal{C}_{c2} = \mathcal{C}_{c1}$ .

The comparison between these theoretical values of  $\mathcal{A}$  and the empirical values presented by Skempton and Bjerrum (1957) for different soils might be of interest. The relation seems to be reasonable.

Prof. Skempton and Dr Bjerrum (1957) have inserted the

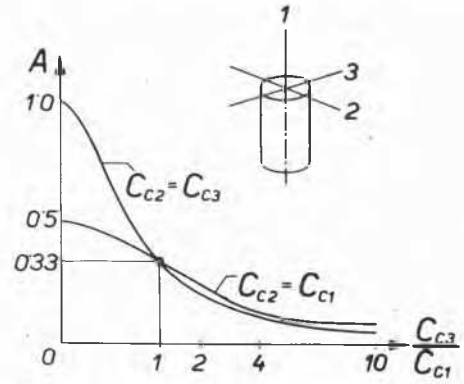


Fig. 29 Discussion on pore pressure dissipation.

initial pore pressure  $\Delta u$  given by the equation (4) into the consolidation settlement calculus expressed in the form

$$\rho_c = \int_{z_1}^{z_2} m_v \Delta u dz \quad (7)$$

From the previous deductions it can be seen that this procedure is justified only if the Young law can be assumed and if the cases of anisotropic soils satisfying the condition (1) are disregarded. Respecting however the plastic behaviour of soil and considering that different intergranular pressures can be attributed to a certain void ratio at different creep rates, the initial pore pressure of a saturated oedometer sample is not necessarily equal to the load increment. The accuracy of such a concept has been proved by the isotaches sets presented in my above mentioned paper (Šuklje 1957) and recently it was verified by the strain and pore pressure versus time relationship obtained in undrained triaxial tests and presented by Mr K.Y.Lo in the very interesting paper 1/37. In cases when the pore pressure in the oedometer sample at the very beginning of the load step is smaller than the load increment, the reduction coefficients proposed by Skempton and Bjerrum lose their theoretical justification. On the other hand the increase of the compressibility coefficient with the layer thickness due to the secondary time effect has to be taken into account.

Prof. E. Schultze and Mr A. Moussa have presented in the paper 1/58 valuable experimental information about the factors affecting the compressibility of sand. It would be desirable to complete their study by investigating the time effect. Some long term observations of the compressibility of a dry fine sand (1 per cent  $< 0,06$ , 50 percent  $< 0,06/0,2$ , 49 per cent  $< 0,2/0,6$ ) have been made in the Laboratory of Soil Mechanics at the University of Ljubljana. Figure 30 shows the dial reading — logarithm of time curves for a sample the initial void ratio of which ( $e_{in} = 0,8896$ ) was obtained by free filling of the sand into the oedometer. In this case the time effect is very important. The moduli of compressibility corresponding to various reading times and to the extended logarithmic lines of last readings are presented in Fig. 31 for the same sample (A a) as well as for a parallel test of another free filled sample (A b) ( $e_{in} = 0,8935$ ) and for a moderately dynamically compacted dry sample of the same sand ( $e_{in} = 0,8028$ ).

Concluding it can be said that in predicting settlement, its development and pore pressure dissipation the plastic behaviour of soils has to be considered and that its influence on the pore pressure dissipation as well as on the development and the end value of settlement depends on the soil character, on its stress history, on the test conditions

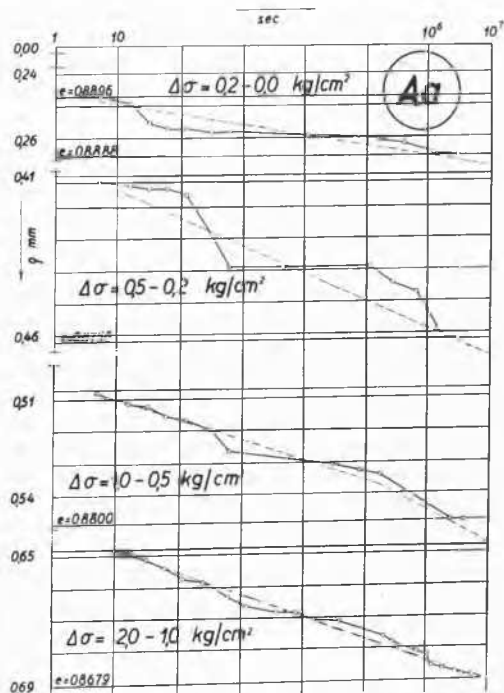


Fig. 30 Discussion on pore pressure dissipation.

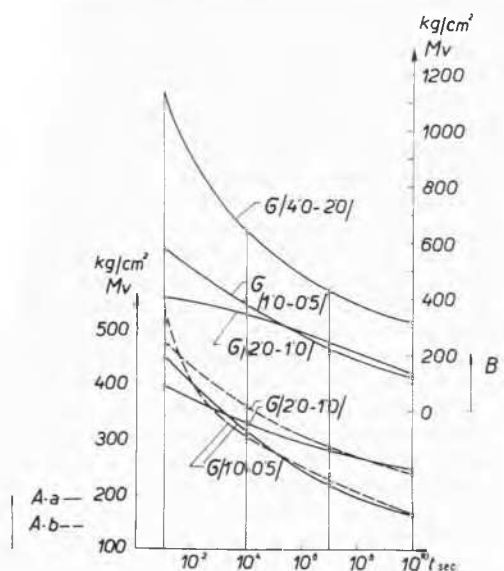


Fig. 31 Discussion on pore pressure dissipation.

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M.T.W. LAMBE (Etats-Unis)

The reporter has requested that the delegates discuss the question of field pore pressures.

At Lagunillas on the east coast of Lake Maracaibo, Venezuela the Creole Petroleum Corporation was faced with the necessity of constructing three very heavily loaded tanks on weak subsoils. Because of the serious stability and settlement problems, the weak underlying clay was instrumented with a number of piezometers and then preloaded. Pore pressure readings were taken during the preload construction and for a considerable period of time after the load was fully on. The case is considered not only interesting but also most significant since there was convincing evidence that the foundation clay at the time the pore pressures were measured was very close to complete shear rupture.

Based on laboratory triaxial tests, the pore pressure parameters of the foundation clay were determined. Using the pore pressure parameter  $A$  and the expression developed by Skempton in 1954, i.e.,

$$\Delta u = \Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)$$

estimates were made of the maximum pore pressure to be encountered at a number of locations in the soft clay. Various techniques of determining  $\Delta \sigma_1$  and  $\Delta \sigma_3$  were employed. A comparison was made between the computed and measured excess pore pressures. The laboratory tests showed that the pore pressure parameter  $A$  was far from a constant but, in fact, depended very significantly on strain and stress history of the soil. An estimate was made of the shear displacement which would occur under the full preload and for this shear strain, the pore pressure parameter  $A$  was estimated to be 0.85.

A comparison of the actual field measured pressures with those computed by various schemes showed that the use of the Skempton equation with  $\Delta \sigma_1$  and  $\Delta \sigma_3$  as determined by elastic theory gave values very close to the field measured ones. The agreement between the theoretical and actual pore pressures was so close that one suspects that a fortuitous cancellation of errors contributed to the agreement.

A paper entitled, "Pore Pressure in a Foundation Clay", has been prepared and submitted to the A.S.C.E. for publication. This paper describes in detail the determination of the pore pressure parameter  $A$ , the estimation of the maximum pore pressures, and compares the calculated with the measured pore pressure values.

## Le Président :

Mr Anagnosti, Yugoslavia.

M.P. ANAGNOSTI (Yougoslavie)

The stress strain relation in clay is the most important problem for theoretical, testing and field investigations. In connection with stress-strain relationship, attention should be paid to the report 1/37 presented to the conference by Mr Low, where the first results of the testing program in the Norwegian Geotechnical Institute is described. As it is stated in the report the unique relationship exists between pore pressure and axial strain in the undrained (constant

and on the layer thickness. In the paper 1/63 Prof. Tan Tjong Kie has presented a brilliant general solution of predicting the consolidation process. The application of this solution requires a considerable experimental and mathematical effort. Nevertheless it can be hoped that the rigorous solution of some fundamental simple problems will show the practical applicability of the new method as well as the limits of the applicability of classical solution and of new methods considering the secondary time effect in a simpler way.

volume of voids) triaxial test. It means that a unique relationship seems to exist between the deviatoric part of the strain tensor and pore pressure in the triaxial test. It would be also of special interest to investigate the relationship between spheric strain tensor and pore pressure in the triaxial test.

The prediction of pore pressures based on  $A$ ,  $B$  or  $\bar{B}$  coefficients of the well known Skempton equation is not always convenient in design work because the stress state in geotechnical problems is only very seldom well defined. The recent measurements of the pore pressure during the change of loading have shown the considerable differences between the pore pressure values estimated by usual simplified methods and those measured in field.

When there is a question of evaluating soil properties as for example the pore pressure dissipation, a very slow rate of load increasing may involve the occurring of viscous and thixotropic properties, the role of which may be such that it could not be neglected. These phenomena as well as dilatancy should be investigated and excluded for the adequate comparison of the results from triaxial, shear box, ring shear, plane strain and other tests. Soil classification methods are still based on rather empirical methods (Atterberg limits), not being connected with evaluation of the general physical properties like viscosity, plasticity, etc.

The total number of 14 unknowns of soil media, as stated by Mr Caquot, is sufficiently large to operate with but it seems that it would be possible in practical work as well as in research work to select two or three main properties of the soil in treating the problem. Evaluation of these characteristics should be sufficient to provide a clear picture of phenomena and to solve the problem, as well as to give more detailed information on the soil media from a scientific point of view.

#### Le Président :

Time is running short, and I must ask the remaining speakers to be as brief as possible. Would Mr Rowe care to say a few words?

#### M. P. W. ROWE (Grande-Bretagne)

Dr Bishop referred to the fact that at Selset the pore pressures dissipated at a rate which was about three times that which might have been expected from tests on small samples and from that I take it that the coefficient of consolidation necessary to fit curves for radial dissipation to sand drains would have to be about three times that measured in the laboratory. This I think is a very interesting result, because we do not always get this sort of result. I do not think it should be minimised, for the cost of sand drains is inversely proportional to the coefficient of consolidation. If the factor is 3, as it was at Selset, it means one-third the cost of the sand drains might have been spent. There were 15 miles of sand drains at Selset and one might infer that 5 miles of drains would have been necessary. There is quite a large engineering cost in the difference and I think that this is a subject for very much further research. We know certain sources of error that can arise. One is in the use of filter paper drains, where the papers are assumed, if they are used, to be fully effective; but in point of fact filter paper drains in the triaxial cell are compressed by the membrane and they are not fully effective, and one can get factors of error up to 5 to 1 in using filter papers.

There is a small error in the assumption that the stress system in the triaxial or in the oedometer is identical to that in the field. There may be laminations in the ground. I do not think that occurred at Selset, but I am surprised at the number of soils that come into my laboratory, which show natural laminations — and by laminations I do not mean

varves of alternate layers of sand, silt and clay — I mean that when the sample is cut and split it will show a fine feathering, and that these laminations can be reproduced even after remoulding the soil to the liquid limit, reconsolidating, cutting and splitting. You can still see those laminations. In fact, quite recently we have had a considerable amount of trouble to find a natural soil in England which did not do this.

There is a fourth source of error which has not even begun to be investigated where sand drains are concerned and that is the consideration of the stiffness of the sand drains themselves. At Selset the drains were 18 in. diameter and as close as 10 ft. apart and they therefore represent a very considerable piling system beneath the foundation. For the soil between the drains to settle more than that column of sand, as they must presumably do if the clay is soft and requires the sand drains, then the stress is transferred in a redundant structure to the unyielding component and some increase in effective stress will occur on the columns of the sand. Stress will come off the clay, and this will be recorded naturally, as a decrease in pore pressure, because the actual applied stresses are decreasing.

#### Le Président :

We now pass to section (c). I ask Mr Wiseman if he will open the discussion.

#### M. G. WISEMAN (Israël)

I read with great interest Mr J. Folque's paper (1/19) on the "Rheological Properties of Compacted Unsaturated Soils". A little over a year ago, Mr Amir, a graduate student working under my direction at the Israel Institute of Technology, investigated the influence of test duration on the strength of compacted clays. During the course of this investigation he attended a series of lectures on Rheology by Prof. Marcus Reiner and was stimulated to attempt describing the results of his investigations in terms of an approximate rheological model given to mathematical analysis. It is interesting to note that the rheological model arrived at is identical to that proposed by Mr Folque. It should be pointed out however that a shortcoming of this model is that it does not show the permanent deformation of partially saturated soils upon removal of the applied stress.

The difficulty Mr Folque had in fitting his constant stress test results to the mathematical expression for his rheological model may very well be due to thixotropic effects. For two clay soils tested by Mr Amir, one a highly plastic clay (CH) from the north of Israel and the other a clay of low plasticity (CL) from the south, it was found that the total deformation at 90 per cent of the strength i.e. close to failure, was a function of the curing time of the specimens decreasing with increasing curing time. By curing time I refer to the time elapsed between compaction and testing. In the tests referred to, the range of times was from 10 minutes to 30 days, the water content being maintained constant. The tests were of the incremental loading unconfined compression type with a total time to failure from the first load application of about 10 minutes. When the total deformation was separated into instantaneous deformation (as defined by Casagrande and Wilson (1951)) and creep it was found that the instantaneous deformation was independent of curing time while the creep decreased significantly with increased curing time. To put it crudely, instead of a Newtonian fluid in our dashpot we have apparently a thixotropic gel. It is hence suggested that a rheological study of laboratory compacted clays should include, as a first stage, a study of the thixotropic characteristics of the soils.

In closing I would like to state Mr Chairman that Mr Folque is to be congratulated on his most interesting contribution to these proceedings.

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## Le Président :

Would Prof. Leonards care to take part?

M. G. A. LEONARDS (Etats-Unis)

I would like to deal briefly with four points on the general problem of pore pressure dissipation and creep.

First of all, in the very important aspect of assessing the significance of creep on pore pressure dissipation in terms of field measurements, it seems essential to me to distinguish between those structures and foundation conditions in which the significant aspects result in a large load increment ratio being applied to the soil and those structures in which the load increment ratio may vary from something very large near the surface to something very small within the significant zone. By load increment ratio I mean the ratio of the stress increment to the previous effective stress.

There is ample evidence to show that if the load increment ratio is sufficiently large creep effects can be largely neglected, at least for a nominal time period.

Secondly I would like to call attention to the fact that there is a large category of problems in which creep phenomena are very significant in terms of the deformations and deformation rates produced rather than the limiting shear strength and relationship of this shear strength to effective stress. I am referring now to such problems as the build-up of pressure behind a relatively rigid structure (such as tunnels, retaining walls, etc.) or the lateral forces that may be produced on a piling system due to creep effects, and I would like to suggest that this problem is not being tackled to the degree that it warrants.

Thirdly, numerical values were quoted with respect to the stresses at which a Bingham limit exists and they were indeed very low; but I would like to suggest that these values might be very considerably raised in problems where the rate at which the stress is applied is very large, particularly to very rapid rates of loading and to shock loadings, where the Bingham limit may be well within the significant stresses that are being investigated.

Lastly, I do not share Prof. Geuze's pessimism with regard to the necessity of studying the creep pore pressure dissipation phenomena exclusively by indirect methods. The possibility exists, it appears to me, by modifications or improvements of freeze drying technique or other methods, to study directly the effective stress deformation time relationship.

M. J. D. COLEMAN (Grande-Bretagne)

Strain increment and stress data for partly saturated soils subjected to a combined stress system may be plotted in a principal effective stress space with respect to soil volume (Croney, Coleman and Black, 1958). The line  $\sigma'_1 = \sigma'_2 = \sigma'_3$  is the axis of four types of surface.

A yield surface, enclosing stress points corresponding to a linear visco-elastic reversible condition of the soil. The mean radius of a normal section of this surface (a yield locus) increases with the amount of plastic work  $W_p$  expended by the stress system upon the soil, a condition which for undrained tests can be expressed as:

$$W_p = \int (\sigma_1 \cdot d\epsilon_1 + \sigma_2 \cdot d\epsilon_2 + \sigma_3 \cdot d\epsilon_3) = f_I(J'_1, \Delta'_2, \Delta'_3)$$

per unit volume  $V_s$  of soil. The deformation modulus also increases by work hardening.

A failure surface, which encloses all effective stress points for the soil, and upon which  $W_p$  all appears as heat in the soil, if shear is homogeneous.

Surfaces of constant water content, containing undrained effective stress paths starting from a common point upon the axis. For a saturated clay the surfaces cut the axis orthogonally, and, if the clay is normally consolidated, they also cut the failure surface in a plane which itself cuts the axis orthogonally. For unsaturated soils, the surfaces cut the axis orthogonally only if the mean total stress is constant.

Surfaces giving the strain increment ratios may be produced by multiplying each strain increment by a deformation modulus to preserve dimensions, and plotting the products as a vector in stress space. The direction cosines of the vector are the strain increment ratios, and if these quantities depend solely upon the current stress, the vectors can be drawn as the normals to surfaces in stress space. The surfaces cut the axis orthogonally, and all the normals are in a plane orthogonal to the axis at failure, when  $d\epsilon_1 + d\epsilon_2 + d\epsilon_3 = 0$ .

Each of these surfaces must be expressible in terms of scalar quantities including the invariants of tensors. In Cartesian notation, the tensors available comprise  $\sigma_{ij}$ ,  $d\sigma_{ij}$ ,  $\sigma'_{ij}$ ,  $d\sigma'_{ij}$  and  $d\epsilon_{ij}$ , together with the deviator of each, and any three independent invariants may be selected for the purpose. The second and third invariants ( $\Delta'_2$  and  $\Delta'_3$ ) of the stress deviator, and the first invariant  $J'_1$  of the effective stress tensor are used for the surfaces in Fig. 32. It may be noted that the effective stress and total stress have the same stress deviator only if the effective stress is a linear combination of total stress and pore water pressure, pore air pressure, osmotic pressure of the soil water, temperature, etc.

$$f(J'_1, \Delta'_2, \Delta'_3) = 0$$

$$J'_1 = \sigma'_1 + \sigma'_2 + \sigma'_3 = 3p'$$

$$\begin{aligned} \Delta'_2 &= -(\sigma'_1 - p')(\sigma'_2 - p') - (\sigma'_2 - p')(\sigma'_3 - p') - (\sigma'_3 - p')(\sigma'_1 - p') \\ &= \frac{1}{2}[(\sigma'_1 - p')^2 + (\sigma'_2 - p')^2 + (\sigma'_3 - p')^2] \\ &= \frac{1}{6}[(\sigma'_1 - \sigma'_2)^2 + (\sigma'_2 - \sigma'_3)^2 + (\sigma'_3 - \sigma'_1)^2] \end{aligned}$$

$$\begin{aligned} \Delta'_3 &= (\sigma'_1 - p')(\sigma'_2 - p')(\sigma'_3 - p') \\ &= \frac{1}{3}[(\sigma'_1 - p')^3 + (\sigma'_2 - p')^3 + (\sigma'_3 - p')^3] \end{aligned}$$

Fig. 32 General equation to yield surface, failure surface, surfaces of constant water content, and surfaces giving strain increment ratios, in principal effective stress space.

These equations seek an analytical extension of existing treatments of phenomena involving combined stress in soils. The extension to include the third invariant  $\Delta'_3$  is regarded as essential, since experiment shows that the surfaces are not of revolution (of equation  $f(J'_1, \Delta'_2) = 0$ ) about the axis, but have only three planes of symmetry at 120°. An invariant failure criterion for soil

$$3 \cdot \Delta'_2 = ((2 \cdot J'_1)/3 + (\Delta'_3/12)^{1/3})^2 \cdot \sin^2 \Phi$$

using these principles has recently been proposed (Coleman, 1960). It coincides with the Mohr-Coulomb criterion for

axially symmetrical failure in both compression and extension,  $\Phi$  being the angle of internal friction (upon an effective stress basis) in each case.

To draw the two alternative failure loci, circles with centre on the axis, and radii proportional to  $1/(3 - \sin \Phi)$  for triaxial compression failure, and  $1/(3 + \sin \Phi)$  for triaxial extension failure, are drawn. The Mohr-Coulomb joins the six points thereby produced with straight lines, implying constancy of  $\Phi$  for all stress states. For the invariant locus, a further circle of radius proportional to  $1/3$  on the same scale is drawn, giving six more points where  $\Delta'_3 = 0$ , the twelve points then available being joined by a smooth curve. Stress states lacking in axial symmetry now give a maximum value of  $\Phi$  (Habib, 1953).

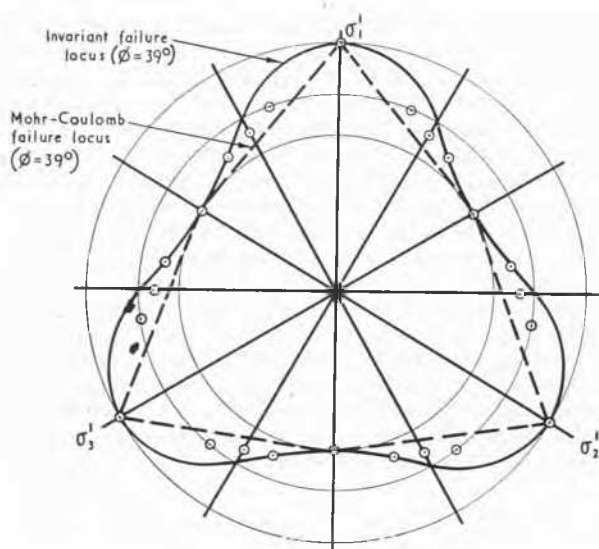


Fig. 33 a Experimental failure locus for closely graded sand (Kirkpatrick 1957) and two interpretations.

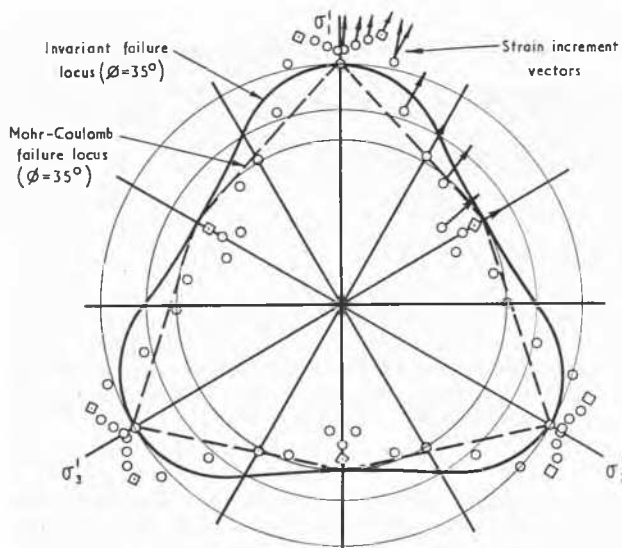


Fig. 33 b Experimental failure locus for a remoulded silt (Haythornthwaite 1960) and two interpretations.

In Fig. 33 (a) the two theoretical failure loci with  $\Phi = 39^\circ$  are compared with the experimental points of Kirkpatrick (1957), for a closely graded sand. In Fig. 33 (b) the two theoretical failure loci with  $\Phi = 35^\circ$  are compared with the ex-

perimental points and strain increment vectors of Haythornthwaite (1960), for a remoulded silt. In each Figure,  $\Phi$  is that observed in undrained triaxial compression.

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## Le Président :

I think you will all agree this is a very interesting discussion, and I propose to go on a few minutes more. I will ask Mr Ménard if he would care to make a contribution.

M. MÉNARD (France)

Mon intervention est relative à l'étude expérimentale du comportement rhéologique des sols, c'est-à-dire des variations du champ de déformation en fonction du temps d'application du champ de contraintes.

Je me bornerai à présenter les résultats d'ensemble suivants :

Les phénomènes rhéologiques sont d'essence entièrement différente pour un champ de contraintes du type déviatorique et pour un champ du type sphérique. Ainsi, dans le premier cas les déformations rhéologiques s'accroissent avec la pression, tandis qu'ils décroissent dans le second cas.

Les phénomènes rhéologiques ont un caractère particulier pour chacun des stades de déformation du matériau (stade des micro-déformations, stade des déformations pseudo-élastiques, stade plastique, etc...).

Si l'on prend comme exemple le cas du champ déviatorique et que l'on représente sur un diagramme en ordonnées les déformations rhéologiques et en abscisses la variable pression, l'on obtient la courbe type de la Fig. 34.

Il apparaît que pour des faibles déformations, les phénomènes rhéologiques sont pratiquement négligeables (phase OA); pour la phase AB, ils sont proportionnels à la pression, la courbe représentative étant une droite de pente relativement faible.

Par contre à partir d'une certaine pression que nous appelons pression de fluage ( $P_f$ ), les phénomènes rhéologiques s'intensifient très rapidement (phase BC).

Il peut être intéressant de représenter simultanément sur le même diagramme le champ de déformations total avec la pression, et d'étudier les correspondances entre les deux courbes.

On s'aperçoit ainsi que la phase AB (phénomènes rhéologiques proportionnels à la pression), correspond à la zone pseudo-élastique de la courbe des déformations totales, et que la phase BC correspond à la zone plastique.

Dans les analyses précises des phénomènes rhéologiques apparaissant avant la pression de fluage, on considère le rapport  $f$  des tassements différés au tassement partiel correspondant à l'incrément de pression correspondant.

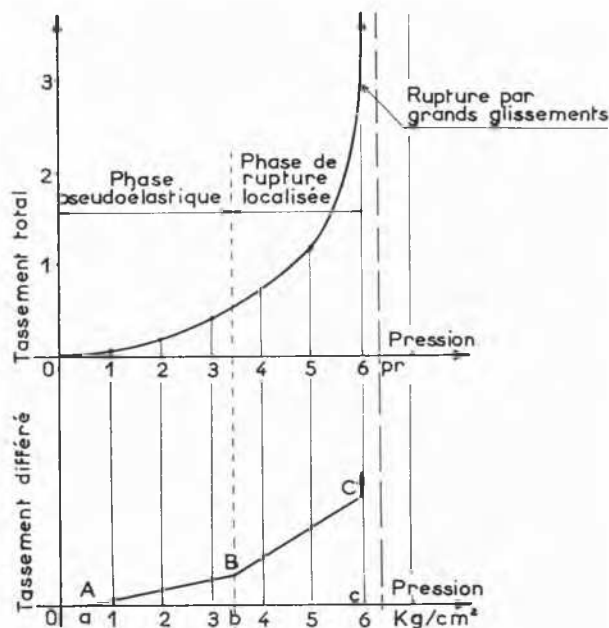


Fig. 34

La comparaison des valeurs ainsi obtenues pour différents terrains avec le même processus expérimental fait apparaître la propension relative de ces terrains au fluage.

Si l'essai des chargements est effectué dans les conditions standards de l'essai pressiométrique, ce rapport est appelé coefficient de fluage. Des résultats statistiques ont montré que ce coefficient dépend très étroitement de la structure du matériau et en moyenne on peut donner le tableau suivant des coefficients :

sable .....	2 à 10 pour cent
limons .....	10 à 15 —
argiles .....	15 à 20 —
tourbe et argile tourbeuse...	20 à 50 —

Les résultats expérimentaux ont montré qu'il était illusoire de faire intervenir dans la représentation mathématique des phénomènes les schémas classiques utilisés en rhéologie (schéma de Kelvin, etc...), car à l'inverse de la plupart des autres matériaux, les sols ne présentent aucun des caractères des corps dits « boltzmaniens ».

#### Le Président :

I think we have one more speaker, M. Silvan Andrei.

M.S. ANDREI (Roumanie)

En ce qui concerne les questions présentée par M. J.A. de Wet dans son rapport « Utilisation de la notion d'Energie en Mécanique du sol » (1/68) je voudrais présenter, quelques aspects qui préoccupent le service Fondations de l'Institut de Recherches scientifiques pour la Construction et l'Economie des Constructions, Bucarest (Roumanie) pour traiter les mécanismes qui intéressent la Mécanique du sol.

En pratique, il y a des situations où apparaissent avec une importance variable différents phénomènes (compression, expansion, circulation de l'eau), traités jusqu'à présent isolément.

Afin de les examiner d'une manière unitaire, il est nécessaire d'établir les éléments communs de liaison (ou si l'on veut) le « dénominateur commun » qui à notre avis, sont représentés par le transfert de masse et d'énergie. Chaque action

extérieure de nature mécanique, thermique, électrique conduit à des changements d'énergie et de masse qui se manifestent (à l'extérieur) sous la forme des phénomènes mentionnés plus haut. En ce sens, l'évaluation des changements énergétiques occupe la première place.

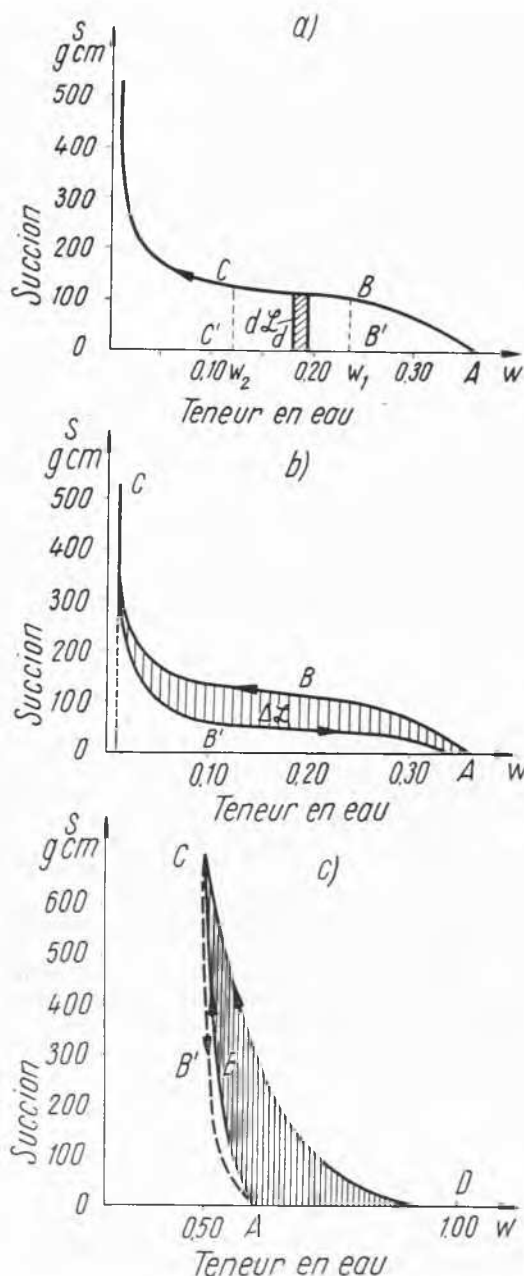


Fig. 35 Variation de la succion avec la teneur en eau (a et b, sable fin de Malinc; c, argile grasse).

Si on analyse, par exemple, le mécanisme du drainage (Fig. 35 a) et si on tient compte du fait que du point de vue énergétique la succion  $s$  représente exactement le travail effectué par le milieu extérieur pour transformer en eau libre un gramme d'eau retenue avec la succion  $s$ , il en résulte que pour obtenir une réduction d'humidité  $dW$  il est nécessaire de dépenser par unité de poids du matériau sec, une énergie :

$$d\mathcal{L}_d = -s \cdot dW \quad \dots (1)$$

représentée par la surface hachurée. Le résultat est négatif car, par rapport au milieu extérieur, il s'agit d'une perte

d'énergie utilisée pour augmenter l'énergie cinétique des molécules dipolaires d'eau. Dans le cas de l'humidification, il y a réduction de la liberté de mouvement des molécules d'eau absorbée; leur énergie cinétique est cédée au milieu extérieur, le plus souvent sous forme de chaleur de mouillage ou d'expansion. Lorsqu'il s'agit d'un mécanisme de drainage permettant de passer de l'humidité  $W_1$  à l'humidité  $W_2$ , le travail spécifique de drainage sera :

$$\mathcal{L}_d = \int_{w_1}^{w_2} s \, dw$$

c'est-à-dire exactement l'aire de la zone BCC' B'.

Pour pouvoir suivre le développement du phénomène en fonction du temps il est nécessaire d'introduire la notion de puissance spécifique de drainage

$$\mathcal{P}_d = \frac{d\mathcal{L}_d}{dt} \quad \dots (2)$$

Si l'on considère comme vitesse de drainage la quantité d'eau drainée dans un intervalle donné d'un volume de terre unitaire

$$V_d = - \frac{\gamma_s}{1 + \varepsilon} \frac{dw}{dt}$$

les relations (1) et (2) donnent une proportionnalité directe entre la vitesse et la puissance spécifique de drainage :

$$V_d = \frac{\gamma_s}{1 + \varepsilon} \frac{\mathcal{P}_d}{S}$$

En utilisant ces notions, il en résulte que dans un cycle de drainage-humidification une certaine énergie égale à la surface de la boucle d'hystérésis est employée irréversiblement (Fig. 35 b), conclusion qui est d'ailleurs en accord avec le second principe de la thermodynamique. De même l'aire ABCD (comprise entre deux branches de drainage d'un sol argileux et l'axe des abscisses peut être interprétée comme étant équivalente à l'énergie employée pour reposer les particules de terre (Fig. 35 c).

Pour la compression du sol sous l'action d'une pression  $p$  on peut définir d'une manière similaire, par unité de volume du squelette solide un travail de compression  $d\mathcal{L}_c = -p d\varepsilon$  (Fig. 36 a) ou par unité de poids du matériau solide, le travail spécifique de compression

$$d\mathcal{L}_c = -p \frac{d\varepsilon}{\gamma_s}$$

Le signe  $-$  apparaît car il s'agit d'une perte d'énergie du milieu extérieur utilisée pour vaincre les forces qui s'opposent à l'action de placer les particules dans un état plus compact et de réduire la teneur en eau. On peut montrer, comme nous l'avons fait plus haut, qu'entre la vitesse de compression et la force de compression il existe une proportionnalité directe. L'aire de la boucle d'hystérésis ABCD' (Fig. 36 b) représente l'énergie perdue au cours d'un cycle de chargement-déchargement, et l'aire ABCD (Fig. 36 c), l'énergie employée pour mettre les particules dans un état plus compact.

Il en résulte qu'entre les mécanismes de drainage-humidification et de compression-expansion il existe une grande ressemblance; pour les terrains argileux dans le domaine de la saturation il s'agit d'une identité.

On peut analyser de la même manière, du point de vue des changements énergétiques, les autres mécanismes qui nous intéressent comme la migration de l'eau, les changements de

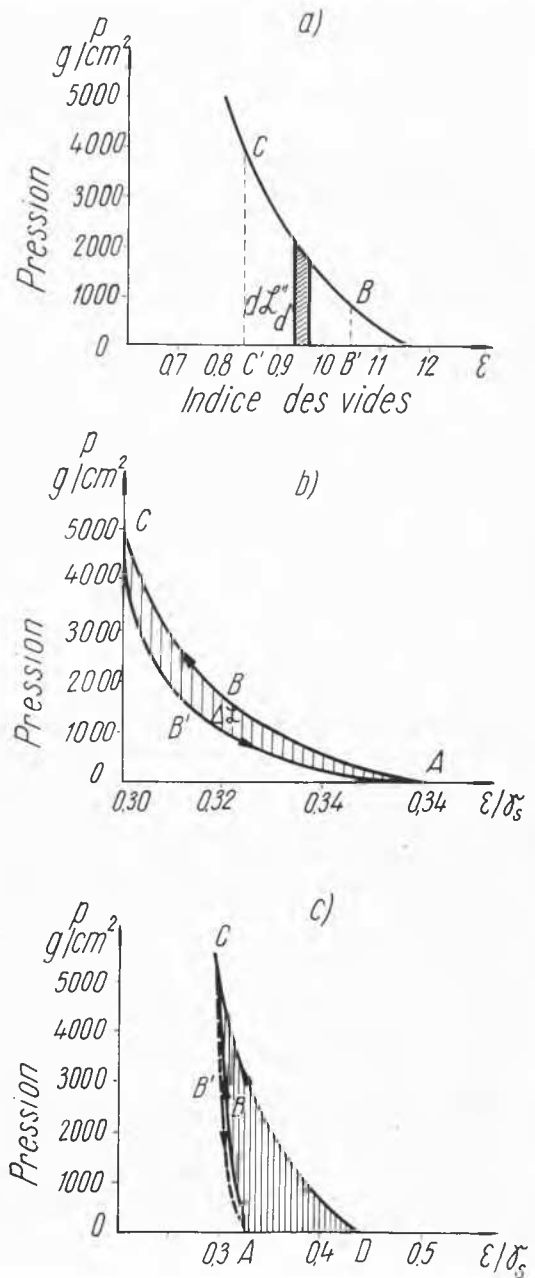


Fig. 36 Variation de la porosité avec la teneur en eau pour un silt de calica sabanga.

phases etc. Par exemple pour la migration de l'eau le travail spécifique de migration aura pour expression :

$$d\mathcal{L}_m = \frac{1 + \varepsilon}{\gamma_s} K_w i^2 \, dt$$

où  $K_w$  représente le coefficient de perméabilité qui correspond à l'humidité  $w$ , et  $i$  est le gradient hydraulique.

La considération des changements énergétiques présente l'avantage de permettre une meilleure compréhension des différents mécanismes et aussi de les traiter d'une manière unitaire. La Fig. 37 présente comme exemple les valeurs du travail d'humidification d'une argile grasse, d'un loess et d'un sable fin déduites d'une part de la courbe succion-humidification ( $s - w$ ) correspondante au domaine de dessiccation dans le vide et d'autre part de la courbe chaleur de mouillage-humidité ( $q_u - w$ ); on peut observer sur les dia-

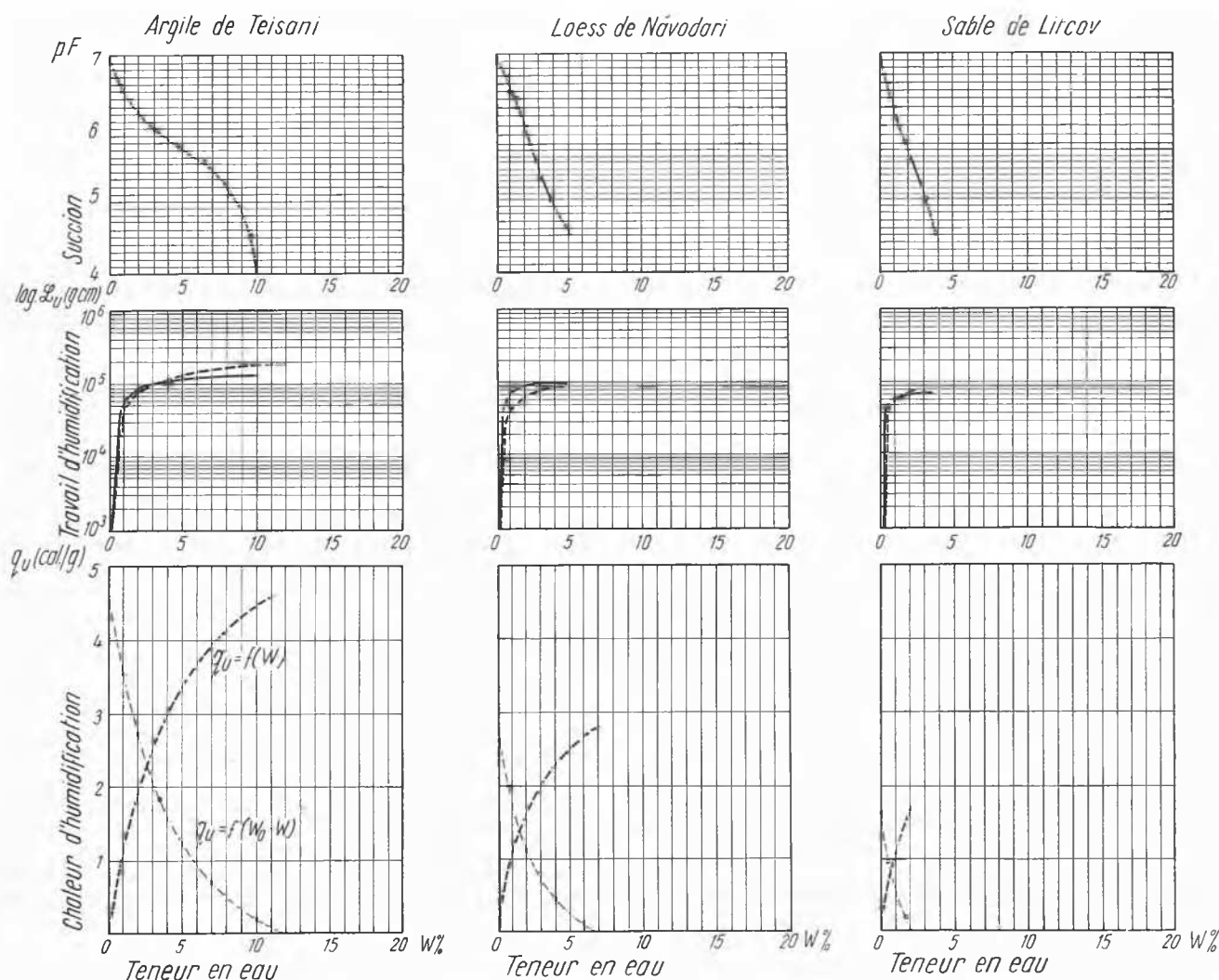


Fig. 37 Comparaison entre le travail spécifique de mouillage  $\mathcal{L}_u$  déduit des courbes succion-teneur en eau et chaleur de mouillage-teneur en eau.

grammes centraux qu'il existe une relation assez bonne. Les notions de travail et de puissance spécifique de drainage (humidification) sont utilisées dans les problèmes de drainage (y compris l'abaissement de la nappe aquifère) des sols à grains fins, dans lesquels, à cause des forces d'interaction entre l'eau et le squelette solide, il est nécessaire d'employer au commencement une énergie supplémentaire pour le drainage du massif :

$$\mathcal{L} = \int \mathcal{L}' dv$$

où  $\mathcal{L}$  est le travail de drainage qui correspond au volume unitaire et  $dv$  le volume élémentaire.

En général, à cause de la complexité des phénomènes qui sont étudiés en mécanique des sols, et compte tenu des moyens de calcul offerts par la technique moderne, nous ne considérons plus nécessaire la tendance à réduire les phénomènes à des schémas, dont il résulte inévitablement un éloignement de la réalité; le chemin à suivre c'est une analyse approfondie. En partant de l'analyse locale de l'élément différentiel, en appliquant les lois de la conservation de la masse et de l'énergie et en utilisant une des méthodes approximatives de calcul (relaxation, différences finies) on doit déterminer les changements d'état (efforts, déformations, changements de phases) du sol et les influences qu'ils auront sur les fondations.

De même, par exemple, dans le cas d'une fondation de poids  $G$  ayant un tassement  $\delta$ , l'énergie de la sollicitation extérieure  $G \cdot \delta$  devra être dissipée dans le massif de terrain au cours des phénomènes de circulation d'eau, de déformations élastiques et plastiques, etc. Pour pouvoir suivre le développement du mécanisme, il est nécessaire de partager le terrain en volumes élémentaires et d'établir pour chacun, à des intervalles de temps fixés (dont la grandeur dépend de la précision voulue) quels sont les changements d'état compatibles avec les lois de la conservation de la masse et de l'énergie. Cette manière d'aborder le problème des tassements présente l'avantage de permettre l'étude d'une stratification quelconque, d'une modification des sollicitations extérieures et permet d'obtenir le développement des tassements en fonction du temps.

Nous considérons que cette manière d'approcher le problème doit être utilisée aussi pour d'autres phénomènes qui présentent de l'intérêt dans la mécanique du sol, car elle offre l'avantage d'une unité de traitement, sur la base commune des changements de masse et d'énergie.

L'élaboration concrète d'une telle méthode de calcul est un problème qui trouvera une solution dans l'avenir et qui nécessite des recherches laborieuses concernant les changements de masse et d'énergie qui accompagnent l'application d'une série de sollicitations sur le sol.

## Références

- [1] CRONEY, D. (1959). "The movement and distribution of water in soils", *Geotechnique*, 3, 1; pp. 1-16.
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- [3] SCHOFIELD, R.K. (1935). "The  $pF$  of the water in soil". *Trans. 3rd Int. Congr. Soil. Sci.*, 2; pp.37-48.

## Le Président :

That ends our discussion today, or perhaps I should say we will carry on with our discussion over the luncheon table. Thank you very much.

I must apologise to those members who wished to speak but for whom we could not find time.

(La séance fut levée à 12 h. 50.)

## Contributions écrites/Written contributions

M.S. ANDREI

En ce qui concerne les problèmes présentés dans la communication de M. le Professeur Renato di Martino « La théorie de l'eau pelliculaire et ses applications à la cohésion du sol et à d'autres phénomènes » (1/40), je voudrais faire état de quelques résultats obtenus dans les laboratoires où je travaille (Incerc, Bucarest) concernant l'influence des conditions d'humidité sur la résistance mécanique des sols.

Notre attention s'est portée d'abord sur les sables et notamment sur la variation de leur résistance à la traction, comme suite aux effets capillaires de l'eau interstitielle. Mais puisque ces résistances ont des valeurs très réduites, il a été nécessaire de réaliser un dispositif pour lequel l'effet du poids propre de l'échantillon de sol et les frottements soient négligeables.

Le dispositif simple (Fig. 38) dans lequel le moule rempli

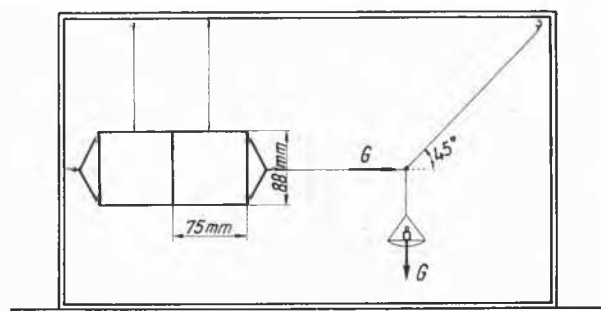


Fig. 38 Schéma du dispositif pour déterminer la résistance à la traction des sables.

de sable compacté à divers états de tassement et d'humidité est suspendu à l'aide de fils de soie, s'est montré sensible à des poids d'un gramme, ce qui permet de mesurer les efforts de traction de l'ordre de  $0,01 \text{ g/cm}^2$ . Les résultats obtenus (Fig. 39 a) montrent qu'il existe une humidité optimum pour laquelle la résistance à la traction est maximum et qui croît en fonction du tassement du sable; pour des degrés d'humidités inférieures, la résistance à la traction devient plus faible (son influence sur le compactage est réduite), et tend vers zéro pour le sable tout à fait sec; pour des degrés d'humidité supérieurs à l'humidité optimum, la résistance est également réduite, mais la possibilité d'essai est limitée par l'apparition d'une accumulation d'eau dans la partie inférieure de l'échantillon. Il en résulte également que la résistance à la traction s'accroît avec la réduction de la dimension des particules à cause de l'augmentation du nombre de points de contact entre les particules (Fig. 39 b).

Pour caractériser l'état de tension de l'humidité on a recours à de petites enveloppes ( $20 \times 20 \text{ mm}$ ) en papier filtre dans lesquelles se trouve un morceau de papier filtre plié. Ces enveloppes sont placées dans l'échantillon jusqu'à ce que nous soyons sûrs d'avoir atteint l'équilibre thermodynamique

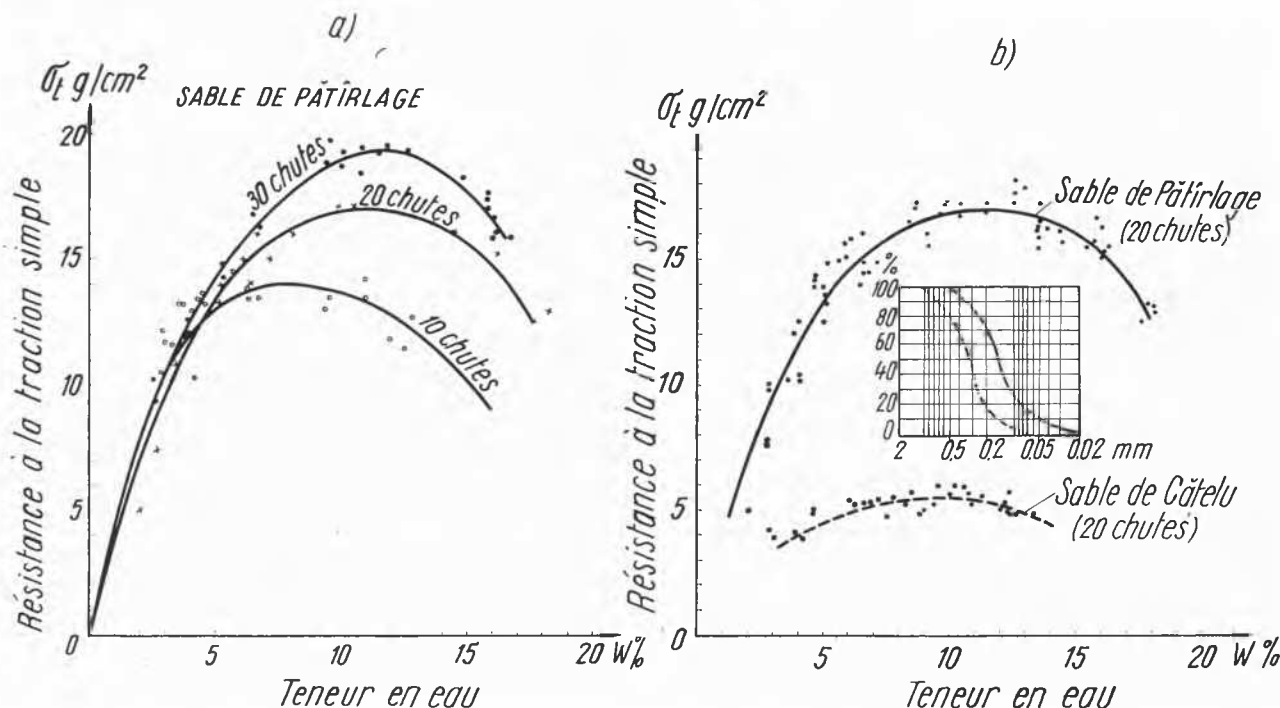


Fig. 39 Variation de la résistance à la traction du sable avec la teneur en eau :  
a) sable fin de Pătrîlăgele à différents états de compactage;  
b) sable fin de Pătrîlăgele et sable moyen de Cătelu.

quand le potentiel d'humidité du corps étalon (le papier filtre) et du sol sont égaux.

Comme potentiel d'humidité on a utilisé celui proposé par Luikov, lequel est défini comme le rapport entre l'humidité du papier filtre correspondante à un certain état et celle correspondante à l'hygroscopicité maximum :

$$\theta = \frac{w}{w_{\max}}$$

A l'heure actuelle nous sommes en train d'effectuer des essais pour établir le rapport entre le potentiel de Luikov et la succion.

## Références

- [1] LUIKOV, A.V. (1956). « L'échange de chaleur et de masse dans les procès de dessiccation », *Gosenergoidat*, Moscou.

M. R. A. ASHBEE (Grande-Bretagne)

## Use of the One - Dimensional Consolidation Test to Assess Ground Pressures and Soil Water Conditions

### Introduction

Accurate predictions of settlement depend to some extent upon accurate prediction of initial in-situ ground pressures. These initial ground pressures are produced by the soil overburden, but they are considerably affected by flotation effects due to the presence of ground water. Unfortunately ground water conditions are not always accurately revealed in the normal exploratory borehole; a perched water table is not easy to locate and even more difficult is to know the true water table in a clay where free water is absent and flotation effects depend largely on the clay being 100 per cent saturated.

Possibly the most common error is to assume flotation where in fact it does not occur. In consequence the initial ground pressure is under-estimated and predicted settlements are over-estimated.

To help overcome these drawbacks it has been found useful to examine the pressure in a consolidation test necessary to return a sample to its in-situ moisture content and to compare this pressure with those calculated for overburden and for overburden with full flotation.

This comparison leads to three applications:

- (1) In-situ ground pressures are indicated. Perched or discontinuous water tables can be detected.
- (2) If ground pressures are known accurately then sample disturbance due to water absorption can be detected. This can be useful in silty or sandy soils where sampling under water is often unavoidable.
- (3) Bulk densities can be corrected to their in-situ values.

### Theory and assumptions

When a soil sample is removed from the ground the release of pressure causes an elastic expansion of the soil skeleton. This gives rise to negative pore water pressures and the sample will try to absorb air (or water if it is available); also gases in the pore water itself may come out of solution. The net result is that any sample placed in the consolidation press is *not* 100 per cent saturated.

The main assumption made in this contribution is that if the sample is recompressed to its original intergranular pressure, then it will return to its original volumetric conditions. In particular it is assumed that if the sample was satur-

ated in the ground, then it will return to saturation [in] the test at the same moisture content.

The obvious objections to these assumptions are:

- (1) Lateral restraint conditions are different.
- (2) Ground water pressures are not reproduced in the test.
- (3) Hysteresis effects in the test make for uncertainty.

Fortunately in spite of these apparent objections this method has proved useful.

### Method

A normal consolidation test is carried out with the sample immersed in water. It is assumed that at the end of such a test the sample is saturated and use is made of this final saturated condition, together with the initial moisture content, in subsequent calculations.

Heights of samples are used in preference to void ratios, since these can be plotted directly from the test results. (Proportional height changes can also be used directly in settlement computations, which saves the unnecessary conversion to void ratios and then back again).

The sample height of interest is the "Initial Saturation Height", that is, the height or thickness of the sample corresponding to saturation of the sample at its initial moisture content.

A typical consolidation against pressure curve is shown in Fig. 40.

The deductions to be drawn from any position of the "Initial Saturation Height" are shown for the first application mentioned in the introduction.

If ground pressures are accurately known (second application) then any deviation of the pressure, corresponding to saturation height, from the calculated pressure can indicate sample disturbance.

For the third application; the bulk density used in pressure calculations should be the in situ value. In saturated ground this will generally be the density corresponding to the saturation height. Bulk densities should, however, only be corrected to pressure values which have been checked for compatibility with all the available site and test information.

### Calculations

#### (a) Initial Saturation Height

$$h_{\text{sat}} = h_f \frac{(1 + w_o G_s)}{(1 + w_f G_s)} \quad (1)$$

where

- $h_f$  is the final measured height, or thickness, of the consolidation sample at the end of the test.
- $w_f$  is the final moisture content (at  $h_f$ ).
- $w_o$  is the initial moisture content.
- $G_s$  is the specific gravity of the soil particles.

It will be noted that  $h_f$ ,  $w_f$  and  $w_o$  are values which can be easily measured to a good degree of accuracy. Even if there are slight errors in  $G_s$ , due to average or assumed values, they will tend to be self cancelling in this type of formula.

#### (b) Bulk density Corrections

The bulk density theoretically required is that at the initial saturation height. Other values, above or below the saturation height, may be indicated by pressure corrections made in the light of other information.

At the Saturation Height ( $h_{\text{sat}}$ )

$$\gamma_{\text{sat}} = \frac{W_f + A \gamma_w (h_{\text{sat}} - h_f)}{A h_{\text{sat}}} \quad (2)$$

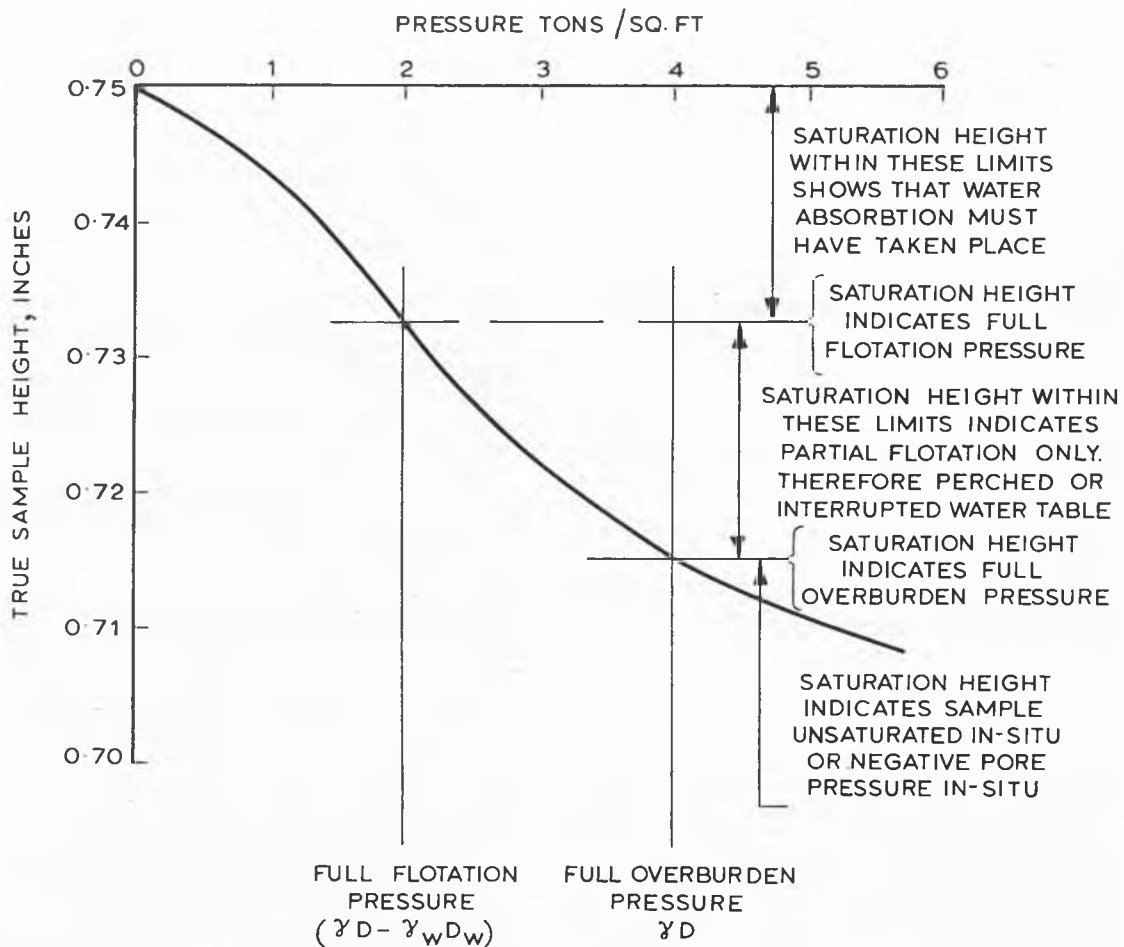


Fig. 40

below  $h_{\text{sat}}$

$$\gamma = \frac{W_f + A\gamma_w(h - h_f)}{Ah} \quad (3)$$

above  $h_{\text{sat}}$

$$\gamma = \frac{W_f + A\gamma_w(h_{\text{sat}} - h_f)}{Ah} \quad (4)$$

where

- $\gamma$  = Required Bulk Density at Height  $h$
- $\gamma_{\text{sat}} = \text{---} \text{---} \text{---} h_{\text{sat}}$
- $\gamma_w$  = Bulk Density of Water
- $W_f$  = Final Weight of Wet Sample at height  $h_f$
- $h_f$  = Final Height of Sample
- $h_{\text{sat}}$  = Initial Saturation Height of Sample
- $h$  = Height of Sample at which the Bulk Density  $\gamma$  is required.
- $A$  = Area of Sample.

For a 3 inch diameter sample this reduces to:

$$\gamma_{\text{sat}} = \frac{W_f + 115 \cdot 8(h_{\text{sat}} - h_f)}{1 \cdot 855 h} \text{ lb./cu. ft.} \quad (5)$$

where

$W_f$  is the final wet weight in grams and all heights are in inches.

#### Machine Accuracy

Attention is drawn to the need for accurate measurement

of the true sample height. Many machines set their dial gauges to an arbitrary starting point at the beginning of each test. This is not good practice. Dial gauges should always read the actual sample thickness. It is advisable to calibrate any machine for errors due to distortion during loading. If a steel sample is placed in the machine, in place of the normal clay sample, any dial gauge readings during loading are "calibration errors". Such errors can be significant and should be corrected for in a normal test.

The initial height of the sample, when it is first placed in the consolidation press, is not an accurate or reliable basis for calculation. Bedding-in and other errors occur; its use is therefore to be avoided where possible.

#### Acknowledgment

This work was carried out on British Railways under Mr A. H. Toms. It was initiated when difficult soil conditions were encountered at Redbridge Viaduct, near Southampton. Undrained triaxial tests, on the laminated sand and silty clay, gave variable results. Some samples yielded plastically and showed no ultimate stress even up to 40 per cent strain. This was shown by the above method to be due to supersaturated conditions. Examination of the saturation height has proved illuminating on other jobs as an auxiliary method. It is most important, however, to appreciate its limitations. The various theoretical objections are dealt with earlier. It is always advisable to examine the results from a complete site rather than just one sample. Also, results should be compatible with each other and with the other technical data

available. The most obvious incompatibility, for example, is a drop of ground pressure with depth which, of course, cannot generally occur.

M. R. A. ASHBEE

### The Effects of Step-Strain Phenomena on Settlement Calculations

The paper by Trollope and Chan to the American Society of Civil Engineers on Step-Strain Phenomena is not only extremely interesting but gives a valuable lead to increasing the accuracy of settlement predictions.

Trollope and Chan show very convincingly that clay particles can exhibit properties analogous to static and sliding friction.

Possibly an over simplification of the model for clay is given in Fig. 41.

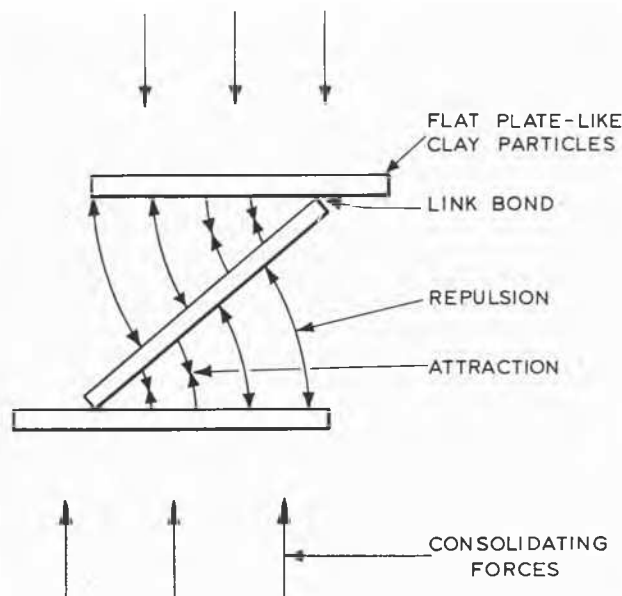


Fig. 41

The step-strain phenomena described, explained on the basis of sliding and static friction between particles, leads to the conclusion that there is a minimum stress increment necessary to cause consolidation or settlement movement from an equilibrium state. This means that we can cut down the zone of influence of a foundation, in settlement calculations, to a definite stress boundary instead of the rather arbitrary boundaries often used at present.

Predicted settlements can be reduced in this way but further experimental data is needed on the magnitude of stresses in step-strain phenomena particularly during a consolidation test.

### Référence

D.H. TROLLOPE and C.K. CHAN, 1960, *Proc. Am. S.C.E. SM2*, April, pp. 1-39.

M. J. BERNEDE (France)

### Cohésion des sables en place

Pour l'interprétation de certains essais de portance, nous avons été amenés à rechercher l'existence d'une certaine cohésion dans les sables en place, et à la mesurer.

La grande hauteur de fronts de carrières quasi verticaux était en outre un indice à la fois de son existence et aussi de sa pérennité.

Quelques sables ont été étudiés, mais en particulier le sable de Fontainebleau dont les grains, compris entre 0,1 et 0,3 mm sont constitués de quartz pur.

La cohésion en place a été, en général, évaluée rapidement avec une bonne approximation mais par un très grand nombre de mesures, à l'aide d'un poinçon circulaire de très petite section : 0,5 cm<sup>2</sup>. Le terme de portance relatif au frottement interne se trouve ainsi éliminé.

On a donc :

$$\phi \neq 1,3 C N_c.$$

Une valeur arbitraire majeure étant admise bien entendu pour l'angle de frottement, afin de définir une valeur approchée de  $N_c$ .

Des mesures complémentaires sur éprouvettes découpées en place dans le massif de sable ont permis, dans quelques cas, de recouper les mesures précédentes : rupture en compression de prismes encore attenants par une base, flexion de poutres encore soudées à leurs extrémités.

Des valeurs de 100 gr/cm<sup>2</sup>, quelques fois supérieures à 200 gr/cm<sup>2</sup> ont été relevées. Il s'agit bien entendu de sable propre, friable, hors de zone de grès en état de cimentation évident.

Pour faible qu'elle soit en général, cette cohésion est un facteur important que le fait de négliger peut conduire à un défaut de sécurité : nous voulons parler des essais de table dans laquelle il y a lieu de tenir compte de la tenue de la structure en place, notamment, et ne pas attribuer la résistance totale au frottement interne seul.

Pour savoir dans quelle mesure on peut compter ensuite sur cette cohésion, il faut tout d'abord connaître son origine. De par la nature quartzreuse des sables, l'action eau-silice fut mise en cause a priori. Toutefois il semble, à la suite des premières investigations que la cohésion soit plutôt le fait de présences salines que de gel.

Des analyses d'eaux souterraines paraissent le montrer, qui accusent des concentrations silice/sels, dans le rapport de 1 à 10. En outre, tout ensemble de grains agglomérés se dégrade immédiatement au contact de l'eau mais reprend aussitôt par séchage sinon tout, du moins, une grande part de sa cohésion. Cette rapide réversibilité nous a paru confirmer notre interprétation.

Avant les expériences de MM. Denisov et Reltov qui rejoignent nos préoccupations personnelles, nous aurions pensé que le processus de formation d'une adhérence intergranulaire sensible attribuée au gel de silice était un phénomène très lent : la cimentation des grès quartzreux se produit en effet à l'échelle géologique.

M.H. BOROWICKA (Autriche)

A long-term program has been started at the Technical University of Vienna in 1957 in order to study the effect of scattering and the nature of effective cohesion in silty remoulded clays. The program was based on the hypothesis that, in addition to the well-known porewater pressure or suction, a secondary liquid stress be efficient attributed probably to the absorbed water and that in a certain degree an interchange of absorbed water and free pore water might be possible during a shearing test. For this reason direct shear tests at constant volume with Hvorslev's ring shear apparatus were chosen (see paper 1/6).

A good deal of evidence has been obtained from the test results on the existence of a secondary liquid stress and also of a slight interchange of absorbed water and free pore water during shearing. Our concept of clay structure deduced from the test results is the following : Even in a clay which is

considered to be consolidated, a secondary liquid stress can still be present which may be a pressure or suction according to the history of the clay. Therefore, the stresses that have been considered until now to be effective are not exactly the real grain-to-grain stresses, but the secondary liquid stresses must be added resp. subtracted in order to obtain the really effective stresses in the aggregate.

When a shear load is applied, the secondary liquid stress may behave in a different way from that in the free pore water according to the kind of loading and other potential influences.

If the secondary liquid stress is a pressure (which is the case with first consolidated soils), it may dissipate steadily or in steps. In this case, the pore water pressure will increase by the same amount by which the secondary liquid pressure diminishes, i.e., steadily or in an unsteady manner. However, it may also occur that the initial secondary liquid pressure remains unchanged up to rupture. In this case lower shear strength apparently results though the normal stress in the shear plane is an effective one in the sense hitherto used. It is, therefore, the case of scattering.

In a preloaded clay, the secondary liquid stress is a suction. As the deformation in such cases is very small during shearing, the secondary suction remains almost unchanged up to rupture and causes the effect of cohesion. According to our concept of clay structure, the real or effective cohesion is, therefore, also an apparent one. But if we produce a high rate of shearing deformation in overconsolidated soils as can be done by reversing the direction of shear (see paper 1/6) the effect of cohesion disappears. The aggregate of clay is therefore, according to our concept, an ideal friction material. The objection might be made that cohesion is caused by a shear strength due to the viscosity of the absorbed water. But this assumption does not explain the existence of a secondary liquid pressure. As secondary pressure and suction behave in a very similar manner we can only assume that they have the same origin i.e. a secondary liquid stress.

It is quite obvious that a stress in the liquid of a clay need not remain constant with time and may drop to zero under certain conditions, especially where deformation is expected to be high.

The remarks made above refer to stable silty clays only. There are other rather special varieties of clay in which a structural change takes place at rupture reducing the angle of internal friction to one half or even less.

More information on these problems may be found in the third paper of "Mitteilungen des Institutes für Grundbau und Bodenmechanik", Technical University of Vienna.

M. J.B. BURLAND (Afrique du Sud)

In paper 1/3 Bishop and Donald deal with effective stresses in partly saturated soils. The possibility of evaluating the behaviour of partly saturated soils in terms of effective stresses is an important consideration but necessarily implies that the principle of effective stress is valid for such soils. A brief review of the definition of the principles of effective stress together with an examination of the requirements for its applicability to partly saturated soils might assist in clarifying the problem.

Terzaghi in his work on fully saturated soils defined the effective stress  $\sigma$  in a soil as the excess of the total applied stress  $\sigma$  over the pore pressure  $u$ .

$$\text{i.e. } \sigma' = \sigma - u \quad \dots (1)$$

He then stated that all measurable effects of a change in stress, such as compression, distortion and a change of shearing resistance are exclusively due to changes in effective stress as defined above.

The effective stress is seen to be made up of two components; one due to applied loads and the other due to pressure in

the pores of the soil. When the pores contain more than one fluid, as is the case with a partly saturated soil, the pore pressure is made up of two components and is defined by the symbol  $u^*$ . The expression  $[u_a - \chi(u_a - u_w)]$  in Bishop's and Donald's equation (1) is therefore equal to  $u^*$ .

$$\text{i.e. } \sigma' = \sigma - [u_a - \chi(u_a - u_w)] \quad \dots (2) \\ = \sigma - u^*$$

The question now arises as to whether the principle of effective stress is valid in partly saturated soils i.e. are the changes in volume and shear strength of partly saturated soils uniquely related to changes in effective stress?

In the test described by Bishop and Donald (paper 1/3)  $\sigma_3$ ,  $u_a$  and  $u_w$  were all varied during shear so as to keep  $(\sigma_3 - u_a)$  and  $(u_a - u_w)$  constant. During these changes the stress strain curve remained smooth and it must therefore be inferred that the effective stress did not change sharply during the test and that the soil behaviour was independent of absolute values of  $\sigma_3$ ,  $u_a$  and  $u_w$ .

The above test cannot be taken as a general proof of the validity of the principle of effective stress because although  $\sigma'_3$  hardly changed during the test, neither did its two components  $(\sigma_3 - u_a)$  and  $\chi(u_a - u_w)$ . In fact any small change in  $\sigma_3$  that might have occurred could only have resulted from changes in the degree of saturation affecting the magnitude of  $\chi$ . If the principle is to be proved rigorously and generally it will be necessary to show that variations in  $(\sigma_3 - u_a)$  and  $\chi(u_a - u_w)$ , such that their sum remains constant, have no effects on the behaviour of the soil. Such a test would be very difficult to perform since it would require a knowledge of  $\chi$  at all times during the test. This introduces the problem of the measurement of  $\chi$ .

The methods of measuring  $\chi$  outlined by Bishop and Donald tacitly assume that the principle of effective stress is valid for partly saturated soils i.e. they assume that the void ratio and shear strength of a given soil is a unique function of the effective stress in the soil. Such an assumption, while being valid for most saturated soils, has not yet been shown to be valid for partly saturated soils and there is some evidence available to show that in certain cases it is definitely not valid. The most important example of a soil which does not obey the principle of effective stress is one that exhibits a tendency to collapse on wetting under load. The phenomenon of collapse will be discussed in some detail in Section 3 A.

The above remarks are not intended in any way to detract from the value of Bishop's and Donald's work. They are presented rather with a view to stimulating further and deeper thinking into the very complex problem of partly saturated soil behaviour particularly when it is considered in relation to effective stresses.

M. T.K. CHAPLIN (Grande Bretagne)

For the first time at an International Conference there are several papers describing or using measurements of the compressibility of sand and other soils with a granular structure. Prof. Schultze and his co-authors have presented a most remarkable series of papers (1/56, 1/57, 1/58 and 2/17) on this topic, and the first part of my paper in Division 3 B (paper 3 B/6) also deals with it. It is very heartening to find such an interest being taken at last in this seriously neglected part of Soil Mechanics.

The most important methods at present available for measuring the compressibility of sands and other soils with a granular structure are as follows:

- (a) Conventional oedometer, as used by Prof. Schultze and others.
- (b) Rowe's confined compression apparatus.

- (c) Triaxial compression tests in which the lateral strain is kept at zero throughout the whole test, using a lateral strain indicator such as that developed by Dr Bishop.

Using the first two methods, in all the tests which I have been able to find, arching and side friction seriously affect the results in all cases, except at lower pressures where the sand is dense or very dense (there is not generally enough information to be more precise). The results of Messrs. Schultze and Moussa (paper 1/58, p. 336) plotted in their Fig. 2 show slopes on a log-log plot of strain and pressure which vary from about 0.5 for the denser samples to lower values for those at higher void ratios. I feel reasonably sure that this variation of slope is almost entirely due to the method of test rather than to the effect of altering the initial void ratio. If I am right, this means that the forms of Figs. 3, 5, 6 and 9 of Messrs. Schultze and Moussa's paper are incorrect. In particular, one would expect that in their Fig. 6 (p. 338), the strain at 10 kg/cm<sup>2</sup> would be almost exactly 10<sup>0.5</sup> or about 3.2 times the strain at 1 kg/cm<sup>2</sup>, assuming that the true compression strain is proportional to the square root of the applied pressure (see my paper 3 B/6, Part 1, for reasons). This seems to be true to a remarkable degree of accuracy in Fig. 6 for relative densities of 0.5 and over. I would suggest that even at  $p = 1$  kg/cm<sup>2</sup> the measured strains are too low at relative densities of up to about 0.3, so it seems very unlikely that Messrs. Schultze and Moussa are correct in their suggestion that the strain at a given pressure varies exponentially with the relative density.

To measure the compressibility of sand with locked-up stresses (due to compaction and pre-loading) in it, one can only use something like Rowe's confined compression apparatus (method (b)) or an oedometer with pressure measurement over a small central portion. In either case the diameter/thickness ratio of the sample must be several times greater than those normally used, in order to reduce arching and side friction to an acceptably low value.

The great importance of compressibility tests lies as much in their application to the fundamental theories of discrete solids as in their straightforward use. Messrs. Kallstenius and Bergau's paper to this Conference (1/28, p. 165), Dr Trollope's paper to the Fourth Conference (vol. II, p. 382, 1957), Messrs Wilson and Sutton's paper to the Second Conference, as well as many recent papers by Mindlin, Deresiewicz and others in the United States, all show the growing interest in the mechanics of discrete solids. As far as I know, the two main ways of deducing the behaviour of single inter-particle contacts in a cohesionless soil from the behaviour of a normal-sized sample is to measure either its compressibility (change of volumetric strain) or its change of wave velocity with change of pressure. I have not yet found any natural sand or even a lean clayed gravel which appears to compress in a way corresponding to the Hertzian concept of spheres. So far the available evidence shows that the contacts behave like those marked with an asterisk in Table 2 of my paper (3 B/6, vol. II, p. 34).

Finally I would like to point out that the constancy of  $K_0$  in a sand at any one porosity over a wide range of pressures, which was first noted by Dr Bishop in 1958 (Brussels Conference on Earth Pressure Problems), seems to be due to two things :

- (a) Constancy of the overall particle geometry during the whole test, which implies low strains.
- (b) A constant ratio of the average tangential and normal compliances at individual particle contacts as the pressure increases.

A constant value of  $K_0$  over a wide range of pressure cannot therefore tell us anything about the nature of the effective

shape of the inter-particle contacts. Incidentally, condition (b) is theoretically true both for spherical contacts before the yield stress is reached, and for those at which the geometry remains essentially unchanged (marked with an asterisk in Table 2 of my paper 3 B/6) as the loads increase, even with the accompanying plastic deformation.

M. D.H. CORNFORTH (Grande Bretagne)

Side filter drains are used to speed up drained triaxial tests in soils of low permeability by shortening the drainage path in the soil. At full efficiency, theoretical considerations suggest that homogeneous isotropic soil specimens draining from both ends and the radial boundary consolidate approximately 100 times faster than specimens draining from one end only (Bishop and Henkel, 1957). The efficiency of radial drainage depends on the relative permeabilities of the soil and filter paper drains. There are the following three conditions :

1.  $k$  filter drain  $\gg k$  soil, full efficiency
2.  $k$  filter drain  $> k$  soil, partial efficiency
3.  $k$  filter drain  $\leq k$  soil, zero efficiency

Fig. 42 shows the consolidation data obtained in triaxial tests on saturated samples of sandy silty clay (decomposed volcanic rock) from the Far East. The two samples were taken close to each other in the same borehole and classification tests showed that they were almost identical in moisture content, Atterberg limits and grading. Side filter drains were placed on the three test specimens from Sample No. 492/20, but they were not used on Sample No. 492/21. All the specimens were drained from one end only.

Comparing the two sets of results, the filter drains on Sample No. 492/20 appear to have been only partially effective. If the specimens of Sample No. 492/21 (without filter drains) had been drained from both ends during consolidation, the parameters  $t_{100}$  would have been less than those in Sample No. 492/20 employing filter drains. Now, although a still faster consolidation could be achieved by using filter drains and double drainage, the filter drains clearly have poor efficiency on this soil i.e. the tests are approaching condition 3 in which the soil and filter paper permeabilities are similar. For the particular filter paper used, Whatmans No. 5, the limiting soil permeability at which filter drains usefully assist drainage is approximately 10<sup>-6</sup> cm/sec. This is a lower permeability than most engineers imagine and the tendency in the past has been to specify filter drains on almost all soils finer than clean sands. However, it is advantageous to omit the side drains whenever possible for the following reasons :

- (i) They affect the strength of the soil, especially at low consolidation pressures.
- (ii) Using filter drains, it is virtually impossible to prevent trapping air between the membrane and specimen; this can affect the volume change readings during consolidation, or can increase the back pressure required to saturate the specimen because the entrapped air has to be dissolved into solution in addition to the pore air voids.
- (iii) When the efficiency of radial drainage is unknown, the consolidation data cannot be used to calculate  $c_v$  and  $k$  of the soil.

Checking through the records of recent triaxial tests carried out in the laboratory of Soil Mechanics Ltd, there is at least one other set of data on 6 specimens of the same soil which confirms that the filter drains are ceasing to be useful in assisting drainage when the soil permeability approaches 10<sup>-6</sup> cm/sec. Almost all soils with this permeability have been tested with filter drains. However, from Fig. 42, it may be

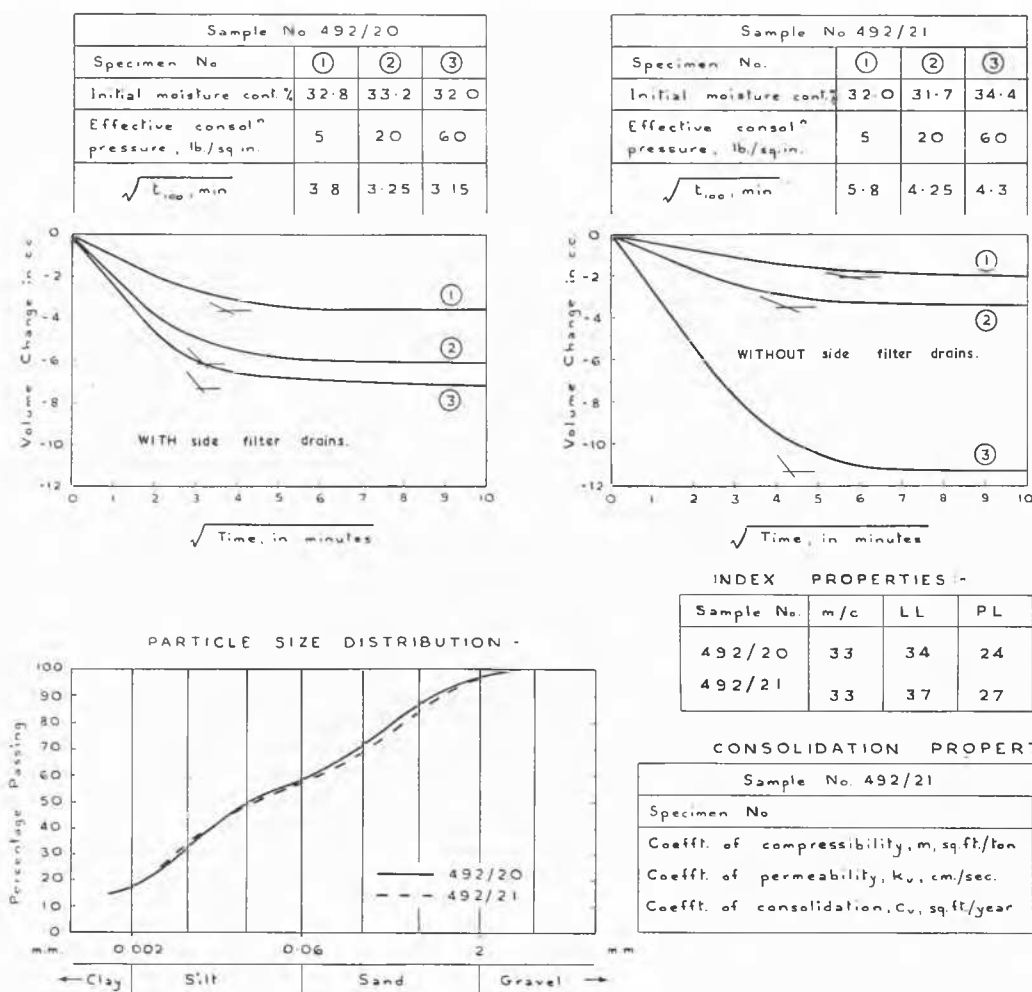


Fig. 42 Use of side filter drains in the triaxial test.

deduced that the borderline is reached when  $3'' \times 1\frac{1}{2}''$  size specimens with filter drains have consolidation parameters  $t_{100}$  in the range of 9 - 15 minutes. Soil samples in this group — 9 in all — have been examined and compared with other soils on which filter drains would, or would not, serve a useful purpose in assisting drainage. From this examination, soils in the category of borderline cases appear to have the following characteristics in common :

Visually they can be described as clayey silts or perhaps very silty clays with a clay fraction in the range 10-20 per cent. Well-graded soils in this category have a slightly lower clay fraction with just sufficient clay present to act as a binder to the coarser material.

Now, the relationship between the parameter  $t_{100}$  and the time to failure in a drained test,  $t_f$ , can be calculated from the equations in Bishop and Henkel (1957) as follows :

$$\begin{aligned} \text{Side drains and double drainage : } t_f &= 20 t_{100} \\ \text{Single or double drainage : } t_f &= 8\frac{1}{2} t_{100} \end{aligned}$$

Therefore, soil with a permeability of  $10^{-6}$  cm/sec., such as Sample No. 492/21, will have a theoretical time to failure in double drainage of approximately 40 minutes. A Laboratory Assistant requires a testing time of at least 2-3 hours in order to calculate the results as the test proceeds. Hence, there is no advantage in using filter drains on this soil, even if a more permeable filter paper is available.

In summary, there are two main points. The first point is that the side filter drains cease to be efficient in assisting

drainage when the soil permeability is approximately  $10^{-6}$  cm/s. The second point is that the theoretical time to failure in drained tests, published in Bishop and Henkel (1957), suggests that soil at this permeability does not, for practical purposes, need filter drains, especially on the smaller sizes of test specimen. Therefore, filter drains can be safely omitted on soils with a permeability of up to  $10^{-6}$  cm/sec, and this group includes soils with an appreciable clay fraction.

M. C.B. CRAWFORD (Canada)

One of the most difficult problems in soil engineering is to relate, with sufficient accuracy, the shear strength of a soil measured in the laboratory to its actual value in the field. A great step forward was made at the time of the Third International Conference when the practical usefulness of effective stress concepts of shearing resistance was widely displayed. At the Fourth International Conference these gains were consolidated by further documentation although there was some concern about the reliability of laboratory tests for certain soils (Hvorslev 1957). One of the major contributions of this, the Fifth International Conference, is a series of papers drawing attention to the uncertainties associated with deformation and volume change during shear testing.

It had been common practice to neglect the effects of strain and volumes changes in computing  $c'$  and  $\Phi'$ . This did not appear to lead to serious error with remoulded soils on which

much of the fundamental shear research had been done. But with sensitive soils it soon became apparent that the development of pore pressure during shear was associated with collapse of the clay structure and was therefore dependent on deformation of the test specimen. It followed that if deformation in the test was not compatible with deformation *in situ* a completely misleading interpretation might be made. Tests on the sensitive Leda clay of Eastern Canada revealed that the computed  $\Phi'$  could vary from  $22^\circ$  to  $35^\circ$  depending on the interpretation of failure. It was further reasoned that the more disturbed the test specimen, the higher would be the computed value for  $\Phi'$  (Crawford 1960).

In a recent series of tests on the Leda clay (Crawford 1961) it was found useful to follow graphically the variation of effective stresses on the potential failure plane. By computing  $\Phi'$  at very low strains, and correcting the computed value according to the proportion of maximum shear stress applied, it was found to be relatively independent of consolidation pressure and of rate of strain. This computed value of  $\Phi'$ , averaging about  $17^\circ$ , was considered to be equivalent to the Hvorslev "true angle of friction." The ability of the specimen to resist shear at greater strains, even though the effective stresses decreased to less than half their original magnitude, was attributed to "true cohesion".

Much more work will have to be done to substantiate the concepts mentioned above but enough has been done to show that, for sensitive clays, the shear strength parameters in terms of effective stresses cannot be obtained in the usual manner. Since  $c'$  and  $\Phi'$  are not fundamental parameters, a routine effective stress analysis may depend for its accuracy on a complete compatibility of stress path, deformation, and rate of strain between laboratory test and field condition.

This points up the need for a more fundamental understanding of cohesion and friction in natural soils. Research in mineralogy and soil physics will be invaluable but it is probable that the engineering answer will be obtained by physical testing. Chances appear good that the shearing resistance may be satisfactorily divided into two parts, one part dependent on the effective stress on the shear plane and one part independent of the effective stress. For sensitive clays, however, these components will have to be more fundamental than our present concepts of  $c'$  and  $\Phi'$ .

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M. R. DAVIDENKOFF (Allemagne)

La méthode la plus répandue pour mesurer la résistance des sols au cisaillement consiste actuellement dans l'exécution de deux séries d'essais : essais non-drainés et essais drainés. Les résultats des premiers essais s'appliquent au calcul de la stabilité de l'ouvrage immédiatement après son exécution, ceux des deuxièmes essais au calcul de la stabilité à long terme.

On a parlé ici de la difficulté de mesurer correctement les caractéristiques de cisaillement du sol dans différents appareils. Le problème de l'emploi de ces caractéristiques dans les calculs est aussi un problème d'actualité qui n'est nullement résolu par la méthode courante ci-dessus mentionnée. En fait, comme le montrent les observations sur le tassement des ouvrages (Nichiporovich, 1955), un tassement considé-

nable se produit pendant la construction, c'est-à-dire que l'essai non-drainé sur un échantillon du sol naturel donne dans ce cas des résultats trop désavantageux. D'autre part, l'utilisation dans le calcul de la stabilité à long terme des valeurs de la « cohésion apparente »  $c'$  et de « l'angle de frottement apparent »  $\phi'$  qu'on obtient à partir des essais drainés ne peut pas être justifiée,  $c'$  et  $\phi'$  n'étant ni la cohésion ni l'angle de frottement du sol. Ce ne sont que les caractéristiques fondamentales du sol, comme les appelle M. Bjerrum, c'est-à-dire la cohésion vraie  $c_e$  et l'angle de frottement vrai  $\phi_e$ , se rapportant à un état du sol caractérisé par sa densité et sa teneur en eau, qui peuvent être employés dans les calculs de la mécanique des sols.

Dans les travaux de Hvorslev, Bjerrum, Jakobson et autres, différentes méthodes de conduire les essais de cisaillement pour obtenir les valeurs  $c_e$  et  $\phi_e$  ont été décrites. Une nouvelle méthode a fait l'objet du très intéressant rapport de M. Maslov présenté à ce Congrès. En faisant les essais de cisaillement sur des sols non consolidés, Maslov obtient un angle de frottement « réel » et la cohésion vraie pour différents degrés de consolidation du sol. Les essais montrent que non seulement la cohésion mais aussi cet angle de frottement, dans lequel est impliquée éventuellement l'influence de la pression interstitielle, si elle existe dans le cas considéré, diminuent considérablement avec l'augmentation de la teneur en eau du sol.

On peut aussi se poser la question en vue de simplifier les essais, s'il est nécessaire, pour un sol saturé, d'effectuer les essais de cisaillement avec des échantillons placés sous l'eau; ceci a conduit l'auteur à étudier de plus près le problème de la pression capillaire.

L'étude des travaux sur la capillarité des sols (Vageler, 1932, et autres) a permis de constater que, contrairement à l'opinion courante, la hauteur de montée capillaire croît avec la diminution de la grosseur des particules du sol seulement jusqu'à une certaine limite (2,5-3 m pour les limons) et puis décroît, pratiquement jusqu'à zéro pour les argiles lourdes. Cela signifie que la cohésion due à la pression capillaire ne joue aucun rôle dans les argiles, ce qui a été prouvé expérimentalement par Habib (1953).

On peut étudier l'action de la pression capillaire sur des échantillons saturés, mais ne se trouvant pas sous l'eau, cisailés sous une charge moindre que la pression de consolidation. La pression capillaire devrait alors remplacer la différence entre la pression de consolidation et la pression agissant pendant l'essai. Des essais préliminaires sur une terre glaise (essais actuellement poursuivis) ont montré, au contraire, que la résistance au cisaillement pour différentes pressions n'est pas la même et que la courbe intrinsèque se rapproche d'une droite inclinée de l'angle de frottement vrai sur l'axe des pressions.

M. R. DI MARTINO (Italie)

Among the phenomena covered by the pellicular water theory as exposed in my report *The Pellicular Water Theory in Relation to Soil Coherence and Other Phenomena*, there is the distribution of pellicular water in the soil on the grain size basis.

Because of the general shortage of space, I have given only a few words to this subject, and therefore I feel the need of adding something to make it more understandable.

I briefly sum up here that the pellicular water of unsaturated soils forms intergranular rings as well as a continuous monomolecular film surrounding the rest of the particle surface and that the free water of the rings can have pressure deficiency or compressive force or no pressure at all. By the movement of the film the pressure tends to be the same all over the soil to reach a condition of equilibrium.

Now in such a condition of equilibrium the amount of

pellicular water belonging to one grain of soil depends upon the number of points of contact. As a matter of fact, if in one grain we have only two particles, there will be only one ring of water; for three particles there will be two rings and so on: the smaller the particles the greater will be the amount of pellicular water. The limit of this increase is given by the relationship:

$$w = (\text{water content}) = \frac{3n^2(n-1) \times \text{volume of ring} \times G \text{ water}}{n^3 \times \text{volume of sphere} \times G \text{ soil particle}}$$

(this relationship working only for an open-packed arrangement of spherical particles).

By understanding clearly this concept of the equilibrium of the intergranular water we can easily accept that two soils having quite different humidity can be in contact and yet keep their own humidity, as long as they have two correspondingly different particle sizes, and we can also visualise the movement from a drier to a wetter soil.

M. VENTURA ESCARIO (Espagne)

### The Magnetic Triaxial Apparatus

It is well known that in the triaxial test now being used the minor and intermediate principal stresses are equal. However it is very interesting to know the effect of the intermediate principal stress; it seems, according to some investigators, that the intermediate principal stress may have an appreciable effect on the coefficients of pore pressure; the problem of plain-strain, so frequent in practice, also asks for a knowledge of the effect of the intermediate principal stress.

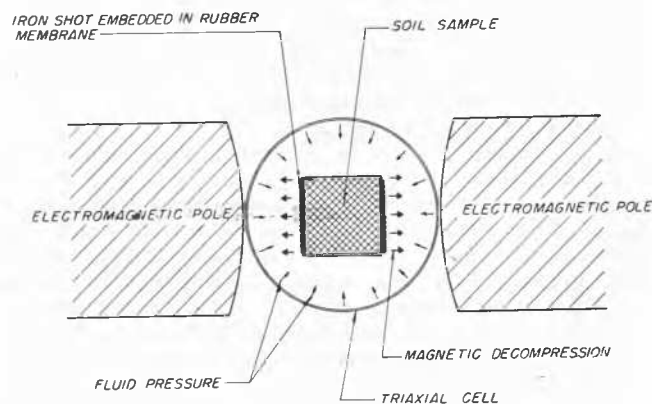


Fig. 43 Plan view of the magnetic triaxial apparatus.

With this in mind, I have in progress, at the Laboratorio del Transporte y Mecánica del Suelo of Madrid, the study of an apparatus of the type shown in Fig. 43. A square sample — instead of a circular one — is introduced into a triaxial cell made out of non-magnetic material, for 1.5" normal type samples; a layer of iron shot embedded in the rubber membrane is placed on two opposite faces of the sample. Both magnetic surfaces are then submitted to the effect of the two poles of an electromagnet. The fluid of the triaxial chamber is subjected to a certain pressure; the electromagnetic forces working on the iron shot result in a decompression of the corresponding stresses, in the desired magnitude. In that way, we can get any combination of values for the intermediate and minor principal stresses.

I have already carried out a preliminary study of the apparatus with the cooperation of technicians in problems of magnetism, and we have arrived at the conclusion that, in

order to reach a decompression of 5 kg/cm<sup>2</sup>, an electromagnet, with a 10 cm gap, of about 300 kg in weight and 18 000 loops, would be needed.

In the final design we have the intention of reducing these dimensions to a substantial extent, in order to reach decompressions of only about 2.0 kg/cm<sup>2</sup>.

Among the difficulties arising with an apparatus of this type we can mention the fact that, on the faces with iron shot, not only normal stresses, but also tangential ones, are originated. These however may be reduced to negligible values by making the poles of the electromagnet big enough.

The possible lack of uniformity in the stresses at different points on a face of the sample, or the edge effect, will be compensated, if necessary, by changing in a convenient manner the dimensions of the iron shot throughout the various areas.

We are also studying a way of measuring in a prototype sample the stresses originated, in order to be able to make out a direct calibration of the apparatus. One of the procedures we intend to apply is the installation of strain gauges at suitable points of an artificial sample. We have designed other methods for making a direct calibration of the apparatus, whose details I will not describe here.

We cannot yet say whether this apparatus will fulfil or not its aim, since it is still in its design phase; but the studies performed on it up to now look rather encouraging, and we hope to have it finished within the next year.

The use of magnetic forces in the triaxial test also has other possible applications, such as the use of pulsating or vibrating forces, very fast loading processes, etc.

We would appreciate any criticism or suggestion about this apparatus, and this is the reason why we present it for discussion in its preliminary stage of design.

MM. V. A. FLORIN and V. P. SIPIDIN (U.R.S.S.)

### Some Problems Pertaining to Creep and Consolidation of Saturated Soils

Deformations of saturated, and in particular clayey soils are characterized by a prolonged intensification in function of time, which, as is known, is explained by simultaneous processes of consolidation and creeping of the soil skeleton. The consolidation process consists in changing of both porosity and water content, followed by unsteady percolation, while some quantity of water is being squeezed from the soil pores. Creeping is determined by viscous resistance to reciprocal displacements of solid particles arising during soil skeleton consolidation.

The process of the diminishing of percolation phenomena during consolidation of saturated clayey soils in laboratory conditions may be described by means of the diagram of pore pressure variations in function of time obtained experimentally. Fig. 44 shows the diagram of variations of pore pressure absolute values  $P_t$  caused by loading. As seen from this diagram the values of pressure in water  $P_{\max}$  are in reasonable agreement with those obtained from theoretical analysis.

Fig. 45 shows the experimental curves representing relative

pressure variations  $\frac{P_t}{P_{\max}}$  in function of time and relative value of soil layer compression  $\frac{S_t}{S_{\text{stab}}}$ . As it may be seen from

this diagram the curve representing the relative value of compression continues after pore pressure dissipation. This confirms the fact that in the initial stage of consolidation of soils under investigation (sand exclusive) there prevail percolation phenomena, after the diminishing of which deform-

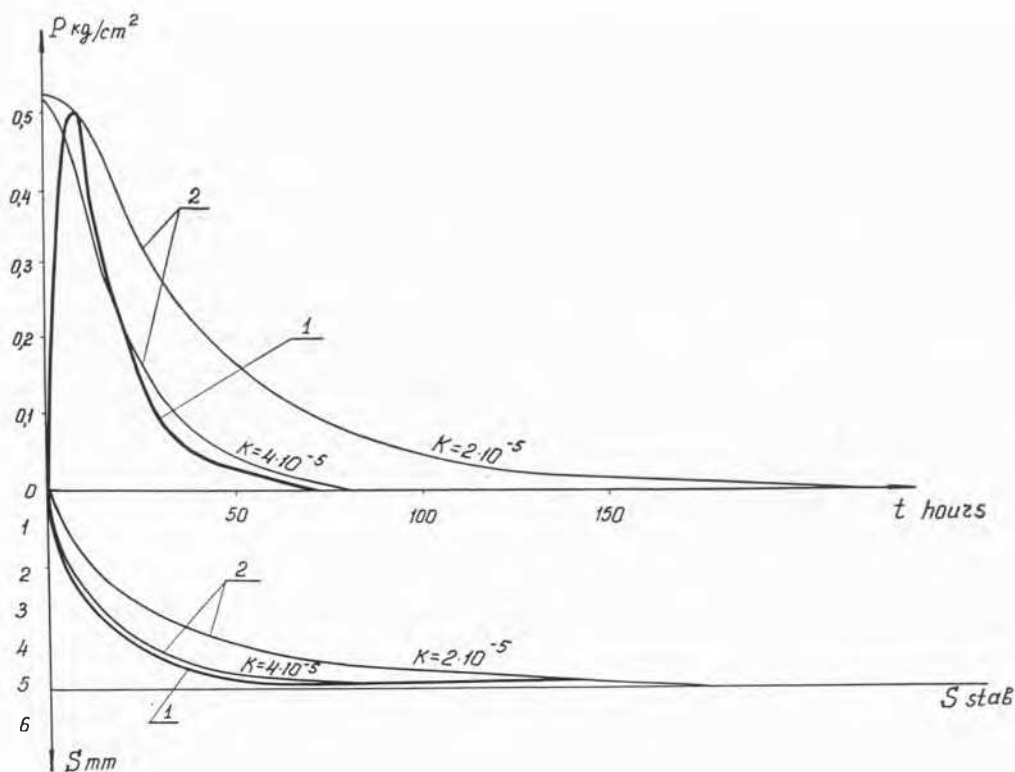


Fig. 44 Curves representing pore pressure variations and settlement of cambrian clay samples. 1. Experimental curve; 2. Theoretical curves.

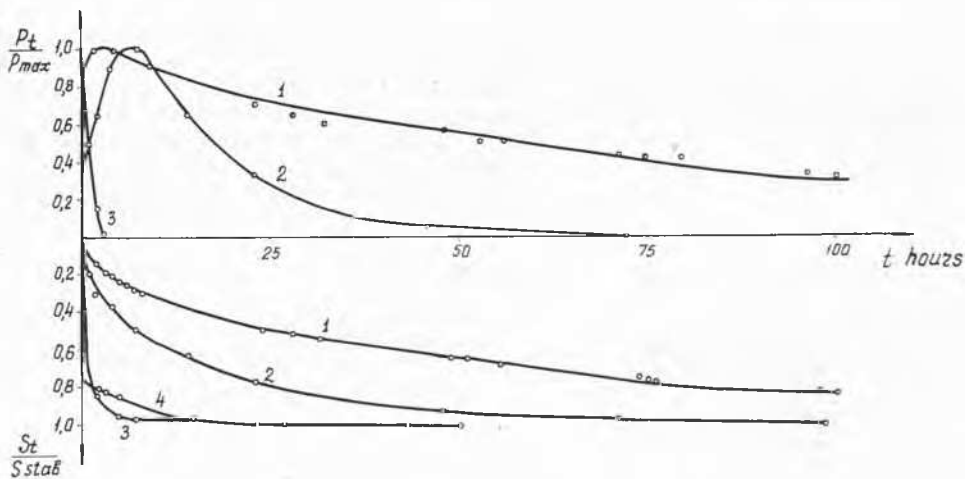


Fig. 45 Curves representing pore pressure variations and settlement of different soil samples. 1. Bentonite; 2. Cambrian clay; 3. Sandy loam; 4. Sand.

ations are determined only by the phenomena of creeping and viscous resistance to reciprocal movements of soil particles. Thus, time duration of the percolation period of consolidation and the beginning of the period of creeping may be determined by the diagram of pore pressure variations. It is found from Fig. 45 that duration of consolidation of a dusty sand sample is explained solely by deformations of creeping because sand consolidation occurs without an increase in pore pressure.

In order to analyze the characteristic features of the process of consolidation one may also use the curves of deformation rate variations in function of time plotted according to the data obtained in common practice of compression tests.

As seen from Fig. 46 the period of sharp change in deformation rate approximately corresponds to the period of decrease in pore pressure. Thus, the turning point of the curve representing in the diagram the rate of deformation is an evidence of occurrence of the period of creeping, the phenomena of "secondary compression" and "secular time effect" being included in the term "creep".

The description of displacements, corresponding to the period of percolation consolidation is made by methods used in the theory of consolidation of saturated medium [1]. As to the periods of creeping, it is recommended to use the methods of the theory of linear creep, considering that squeezing of water from the soil pores came to an end.

Let us denote the moment of load application by the sym-

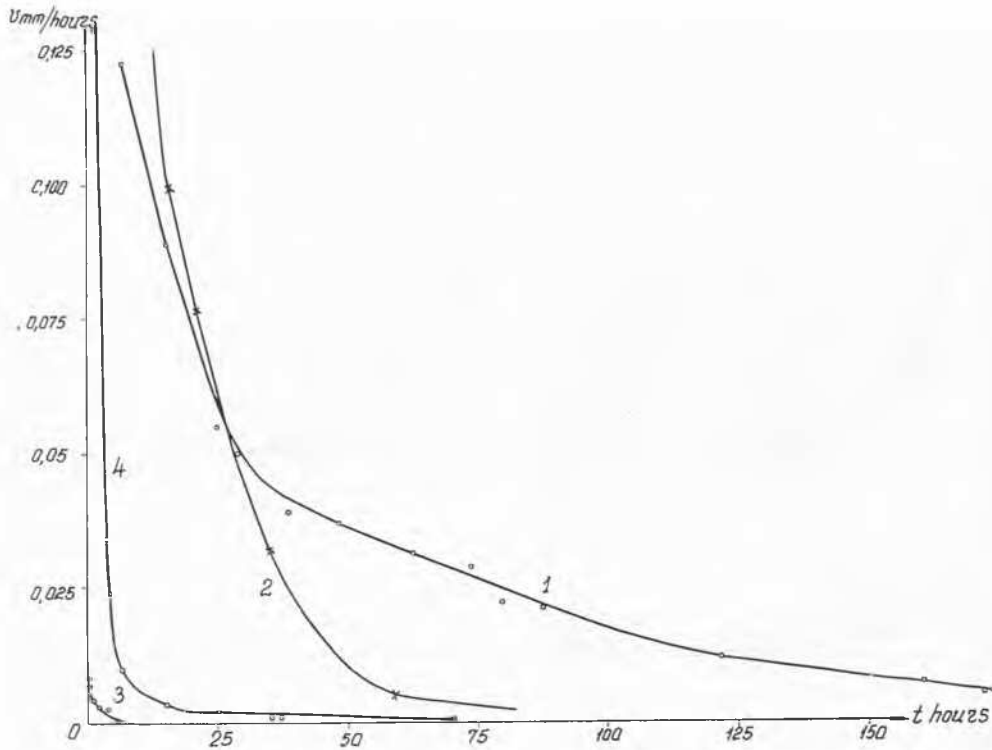


Fig. 46 Curves representing variations of different soil sample deformation rates.  
1. Bentonite; 2. Cambrian clay; 3. Sand; 4. Sandy loam.

bol  $\tau_1$ , and the moment of time, for which the deformation is determined, by  $t$ .

Then, for example, the full axial deformation under invariable by time unit loading will be expressed as

$$\delta(t, \tau_1) = \frac{1}{E(\tau_1)} + C(t, \tau_1),$$

where

$E(\tau_1)$  is the modulus of instantaneous deformation,

$c(t, \tau_1)$  is the measure of creep.

The vertical movement of the point under investigation on the soil surface may be written in the following form

$$s_t = \int_0^h e_z(t) dz = \sum_0^h e_z(t, z) \Delta z.$$

In case the applied load is variable with time the expression for deformation  $e_z$  may assume the form of integral equation. Under constant loading the expressions for two-dimensional conditions of deformation as well as for the three-dimensional problem are of the following form respectively

$$e_z = (1 + \mu) [(1 - \mu) \sigma_z(\tau_1) - \mu \sigma_x(\tau_1)] \delta(t, \tau_1),$$

$$e_z = \{ \sigma_z(\tau_1) - \mu [\sigma_x(\tau_1) + \sigma_y(\tau_1)] \} \delta(t, \tau_1),$$

where

$\mu$  Poisson's ratio, assumed as being equal for both elastic and non-elastic parts of deformation and independent of time.

The stress components in the above expressions may be determined by the methods used in the theory of elasticity. In such a case it is necessary that the measure of creep at one-axial compression should be proportional to the meas-

ure of creep at shear [2]. In view that it is the soil skeleton that is accepted for the model of creeping medium, it seems that this requirement can be easily met with because both types of deformations are determined by the phenomena of viscous displacements of solid particles and pore water, of which the soil skeleton is composed.

The horizontal movements of structures may be determined in the same way.

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M. B.P. GORBUNOV (U.R.S.S.)

## Investigation of Soil Structure by Filtration Method

Considering water as a Shvedov-Bingham, the following filtration rate (2) is obtained :

$$V = K \left[ i - \frac{i_0}{3} \left( 4 - \frac{i_0^3}{i^3} \right) \right] \approx K (i - 4/3 i_0) \dots (1)$$

$$K = A \frac{(e - e_0)^3}{1 + e} \dots (2)$$

$$A = \frac{\gamma_w g}{2 \eta v^2} (1 + e_0)^{2 \frac{1-m}{m}} \dots (3)$$

$$i_0 = \frac{\tau_w v}{\gamma_w g (e - e_0)^{1-m}} (4)$$

where

$e_0$  = quantity of adhering water;  
 $i_0$  = initial filtration gradient;  
 $v$  = specific soil surface, i.e. surface of particles per unit of volume of hard phase / $sm^{-1}$ ;  
 $m$  = coefficient of shape of particles: for spherical = 3, scale = 1, uncertain shape = 2.

In the case of  $\tau_w = 0$   $i_0 = 0$  equation (1) changes into the Darcy formula.

Employing the above mentioned equations, it is possible to calculate  $e_0$  and  $v$  which may be used as indications of the soil structure. For this aim compression tests are to be performed with soils with disturbed and undisturbed structure, determining  $K$  for each load stage (1, 2, 3).

Preparation of soil with disturbed structure is similar to the preparation for soil classification analysis. The initial moisture content is about  $L_L$ . As a result we obtain :

$K_1; K_2 \dots K_a; K_a + 1; K_a + 2 \dots K_b$   
 $e_1; e_2 \dots e_a; e_a + 1; e_a + 2 \dots e_b$

i.e. the observation series are divided into two groups.

The experiments determined that  $e_0$  and  $v$  are constant if appropriate preparation and removal of loads during filtration tests are performed. Therefore, they may be calculated in the following way :

$$e_0 = \frac{\sum_{a+1}^b e_i \sum_1^a \sqrt[3]{K_i(1+e_i)} - \sum_1^a e_i \sum_{a+1}^b \sqrt[3]{K_i(1+e_i)}}{(b-a) \sum_1^a \sqrt[3]{K_i(1+e_i)} - a \sum_{a+1}^b \sqrt[3]{K_i(1+e_i)}} \quad (5)$$

$$A = \frac{\left[ \frac{\sum_1^a \sqrt[3]{K_i(1+e_i)}}{\sum_1^a e_i - ae_0} \right]^3}{\left[ \frac{\sum_{a+1}^b \sqrt[3]{K_i(1+e_i)}}{\sum_{a+1}^b e_i - (b-a)e_0} \right]^3} \quad (6)$$

$v$  is calculated according to  $A$  and  $e_0$  by formula (3), tangential resistance of water  $\tau_w$  by formula (4).

$v$  as an integral characterizes the soil classification. It may be used for classifying the soil [3].

Soil	$v$
Sand .....	< 1 200
Sandy clay .....	1 200 to 12 000
Loam .....	12 000 to 60 000
Clay .....	> 60 000

Experiments have shown that  $e_0$  showing the total hydrophylic nature and degree of dispersion, cannot be used for classification.  $v$  gives stable and characteristic values.

In cohesive soils with undisturbed structure the value of  $v$  will change according to the pressure;  $e_0$  conditionally may be considered constant, being determined by tests of the soil with disturbed structure and using which it is possible, by the porosity and coefficient of permeability, to calculate the value of the specific surface of the soil with undisturbed structure  $v_1$ , at each load stage, by the following formula :

$$v_1 = \sqrt{\frac{\gamma_w g (e - e_0)^3}{2 K \eta (1 + e_0) (1 + \varepsilon)}} \quad \dots (7)$$

The ratio of the specific surfaces of the soil with disturbed

and undisturbed structure is known as the aggregate index and is designated by the letter  $f$

$$f = \frac{v}{v_1} \quad (8)$$

The average number of particles adhering in aggregates is determined as the cube of this value.

Considering  $f$  as the function of pressure we introduce the value indicating the mechanical workability of the aggregates

$$\bar{\varepsilon} = \frac{\Delta f}{\Delta p} : f \quad \dots (9)$$

The sign shows the character of coherence; the value the strength of the structure.

When brittle coherence  $\bar{\varepsilon} < 0$ , elastic  $\bar{\varepsilon} > 0$  and  $\bar{\varepsilon} = 0$  the aggregates or separate particles are so strong, that they do not change in the limits of the pressures used in the tests.

By size the soils may be classified in groups :

Group	$f^3$
Non-aggregate .....	1
Fine-aggregate .....	1 to 10
Average-aggregate ...	10 to 100
Coarse-aggregate ....	100 to 1 000
High-aggregate .....	> 1 000

The experiments have determined that  $\tau_w$  measured by thousands of dynes/cm<sup>2</sup> is based in slays  $i_0$ , significantly exceeding unity.

The described method may be known as the filtration analysis of soils. As a result of its application, certain physical properties are obtained, characterizing the size of particles, number of particles adhering in aggregates, the value and character of cohesion.

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MM. I. M. GORKOVA, N. A. OKNINA, V. F. CHEPIK (U.R.S.S.)

## Deformation Features and Diffusion Permeability of some Sedimentary Rocks

In the Soviet Union the Laboratory of Hydrogeological Problems of the Academy of Sciences of the U.S.S.R. has instituted a new trend in soil mechanics studies of sedimentary rocks as natural structured dispersion systems. This trend is based on modern concepts of the newest branches of colloidal chemistry and physico-chemical mechanics.

Experimental studies have been carried out of the deformation and rheological conduct as well as of the diffusion permeability of certain sedimentary rocks with a natural and disturbed structure. The age, properties and composition of these rocks and the research methods have been described previously. The following groups of rocks characterized by specific structural and deformation features are distinguished:

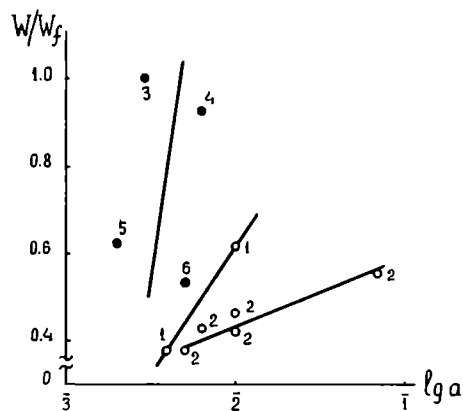


Fig. 47 Dependence of the compressibility coefficient ( $a$ ) in sedimentary rocks with a natural structure upon their relative humidity  $W/W_f$  :  
1. Khvalynian clays; 2. Jurassic clays; 3. Pure chalk; 4. Clay chalk; 5. Marl; 6. Clay-marl.

1. Rocks with unstable loose coagulation structures ( $E/E_f > 1$ ). They include recent marine sediments, Quaternary clays, chalk, loesses, quicksoils, etc. All these rocks with a different degree of water saturation possessing a varying strength in the range of undisturbed structures (depending upon their hydrophilicity and the degree of synergetic strengthening) are characterized by a capacity of sharply losing their stability in the narrow interval of strains due to a destruction of the loose structure and the release of immobilized pore water. They are inclined to become suddenly liquefied under strains exceeding the dynamic flow limit,  $P_{k-2}$ , causing slumps of slopes, collapses, subsidences and sags as well as drastic landslides. The plastic strength of these rocks ( $P_m$ ) determined by cone immersion varies from 0.05 to 42 kg/cm<sup>2</sup>. The ultimate uniaxial compression strength amounts to 12-20 kg/cm<sup>2</sup>. The sensitivity of these rocks varies from 1.7 for marine sediments to 1000 for chalks, depending upon the strength of the natural structure. When natural structural bonds are disturbed, the rocks acquire a large high-elasticity

( $\lambda = \frac{E_1}{E_1 + E_2} = 0.5 - 0.8$ ) and a high compressibility

( $a > 0.05$ ). They are characterized by irreversible structural deformations and an absence of creeping capacity. When the structure of water-saturated rocks is subjected to a maximum strain their viscosity drops by several tens of the orders. The viscosity ratio of the least and of the most destroyed structures  $\eta_0/\eta_m$  amounts to  $10^2 - 10^6$  and more. The thixotropic structure recovery proceeds in time. The diffusion coefficient for these water-saturated rocks is  $> 0.6$ . Rocks of this group change their properties most drastically when their structure is disturbed. Vibrations are especially dangerous for them.

2. Rocks with compact, synergetically strengthened structures ( $E/E_f = 0.5$ ;  $W/W_f = 0.45$ ). They include hard clays with a low water content characterized by distinctly elastic properties in a definite range of strains (up to 2 kg/cm<sup>2</sup>), a very low high-elasticity ( $\lambda = 0.08 - 0.10$ ) and a tendency for a plastic flow (creep), virtually without any destruction of the structure owing to its instantaneous thixotropic recovery. They are characterized by a low compressibility ( $a \approx 0.005$ ) and a reversible character of deformations. The plastic strength of these rocks amounts to 20-35 kg/cm<sup>2</sup>, the ultimate compression strength comes to 15-20 kg/cm<sup>2</sup>. Their sensitivity is low (2-1).

Because of their capacity to creep these rocks form stream-

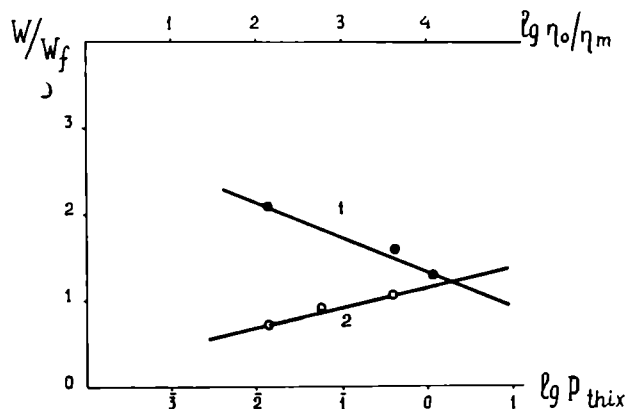


Fig. 48 Changes in the viscosity ratio  $\eta_0/\eta_m$  (1) and thixotropic recovery of the rocks  $P_{thix}$  in time (g/cm<sup>2</sup>, min.), (2) depending upon their relative moisture content  $W/W_f$ .

ing or heaving types of landslides at the foot of the slopes. The diffusion permeability of the rocks is low —  $K < 0.3$ . Increase of moisture content causes a very drastic change in the properties of these rocks, determining a swelling pressure up to 12 kg/cm<sup>2</sup> and considerable deformations during swelling and slaking.

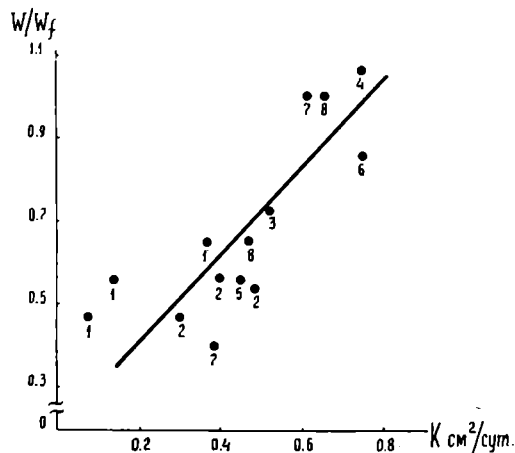


Fig. 49 Dependence of the diffusion coefficient of the chlorine ion upon the relative moisture content of sedimentary rocks at a temperature of 20 °C.  
1. Khvalynian clays; 2. Clay-marl; 3. Clay chalk; 4. Pure chalk; 5. Marl; 6. Recent marine sediments; 7. Montmorillonite; 8. Kaolinite; 1-6 samples with a natural structure; 7-8 samples with a disturbed structure.

3. The intermediate group of rocks ( $E/E_f = 1 - 0.50$ ;  $W/W_f = 1 - 0.45$ ) is represented by plastic clays and marls. These rocks have a capacity to creep in a certain much narrower strain interval, but are characterized by a sufficiently sharp drop of viscosity under strains exceeding the dynamic flow limit, which in practice coincides with the creeping limit. The viscosity ratio of the least and of the most disturbed structures  $\eta_0/\eta_m$  for them varies from 400 to 20. Their sensitivity is from 9 to 2. Their plastic strength comes to 4-10 kg/cm<sup>2</sup> for clays and 17-20 kg/cm<sup>2</sup> for marls and marly clays. The ultimate uniaxial compression strength amounts to 12 kg/cm<sup>2</sup> for clays and 27 kg/cm<sup>2</sup> for marls. These rocks possess an average compressibility and are characterized by an irreversible character of deformation.

The rheological features of the rocks, their elastic and plastic properties, capacity for flow, creep, etc., as well as their diffusion permeability are mainly determined by the concentration of the dispersion phase (structure). That is why they are similar for natural and disturbed structures of dispersed rocks. The strength features of the rocks and their high-elastic properties (determining the rigidity or mobility of structural bonds) are determined also by the strength of interparticle cohesion. This factor is of a varying nature depending upon the character of the structural bonds and the degree of synergetic strengthening of the rocks.

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### The Dynamics of Soils in the Inelastic Range (One-Dimensional State of Loading)

In volume I, division 1/11 (pp. 71-74) of the Proceedings, M. Davidov (U.S.S.R.) presents a mathematical method for the calculation of special cases of stress wave propagation in soils [3].

The present paper deals with the general problem, and in particular with the phenomenon of the propagation of stress and deformation through non-linear, inelastic materials. If for instance an atomic bomb explodes above an underground shelter, a wave of great intensity travels through the ground. Because of the inelasticity of the soil, the wave is gradually attenuated. This effect is important for a proper design of the shelter, as it is necessary to know the pressure and the ground motion at the depth of the constructions.

There are a great many other dynamic problems in civil engineering, where the inelasticity of the materials plays a decisive part, e.g. the study of pile-driving [2], of the action of traffic impacts on roads and runways and of the dynamic compaction of soils [6].

In order to tackle the problem of one dimensional stress wave propagation in soils quantitatively, we first have to find a set of adequate material properties. The investigations show that it is possible to determine a dynamic stress—strain relation for a given type of soil. This relation character-

izes the materials exhaustively and will allow calculating the phenomenon theoretically.

### Experimental Setup

The present work required first a set of reliable dynamic measuring devices. The following apparatus had to be developed:

#### Dynamic soil pressure cell.

A pressure cell was necessary for the laboratory and field tests. It had to be capable of being used in soils like gravel and stoney materials with edged components. The maximum pressure was 30 kg/cm<sup>2</sup> (about 420 psi.), the rise time about one millisecond. There was no such gauge available, so a new one had to be developed. The open cell consists of two rigid plates of aluminum, held together by three supporting rings. Strain-gauges are mounted on these rings and connected in such a way that the electrical outputs of all the rings are summed up. Thus, the total output becomes independent of the local arrangement of the loads on the cell. This is particularly important for pressure measurement in coarse-grained soils. The natural frequency of the cell as a whole is 2 000 cps. It can be shown by integration of the vibration equation that this is sufficient for rise times down to one millisecond.

#### Accelerometer.

The field tests required the measurement of fast displacements of points buried below the ground surface. Barium titanate accelerometers were unsatisfactory for measuring impacts. A device based on strain-gauges was constructed, in which a mass is carried by a steel cylinder. The effects of transverse accelerations were eliminated by a symmetrical layout of the whole device. Fast transient movements were measured with it and checked by high speed film recordings. Fig. 50 shows a comparison between the twofold integrated acceleration signal and the actual displacement vs time curve. The agreement is very satisfactory up to a time of about 25 ms.

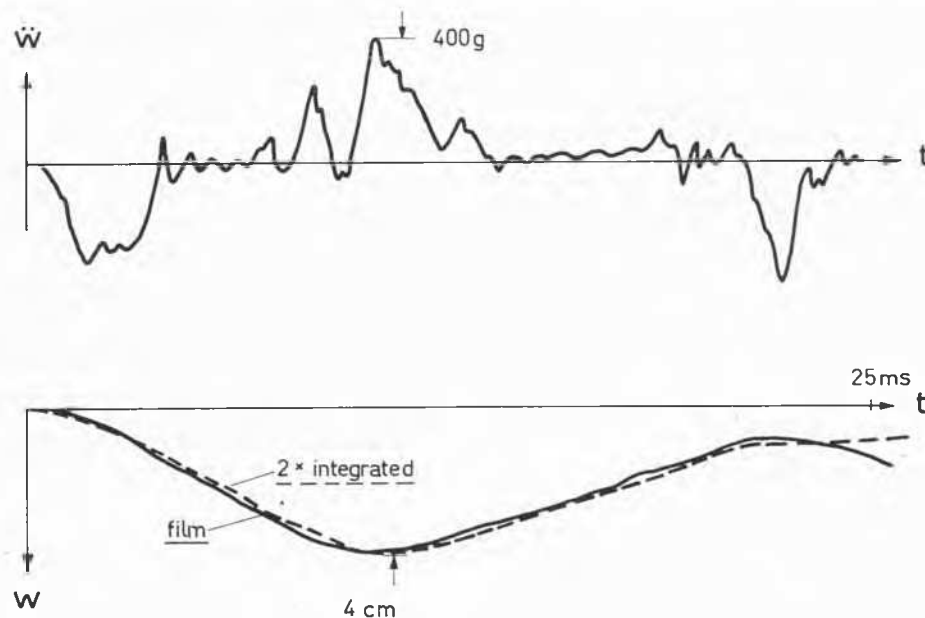


Fig. 50 Diagrams as functions of time  $t$  showing acceleration signal (top), twofold integrated signal (dotted line, bottom) and displacement curve from high speed film.

### Dynamic oedometer.

The central part of the experimental program was the determination of the dynamic stress-strain relation. The corresponding apparatus is called "dynamic oedometer" in analogy to the static device. As with the latter, it is necessary to put a sample of the soil — disturbed or undisturbed — into a rigid steel casing. No friction is allowed on this casing. In the apparatus developed (shown schematically in Fig. 51) this dynamic friction was measured by supporting the casing by four pressure sensitive elements. The friction was negligible in the arrangement shown (plastic coating between casing and soil). The pressure was measured on the upper and lower surface of the specimen, and both recordings coincided sufficiently well, as a relatively short length was chosen. The deformation was recorded by two potentiometers.

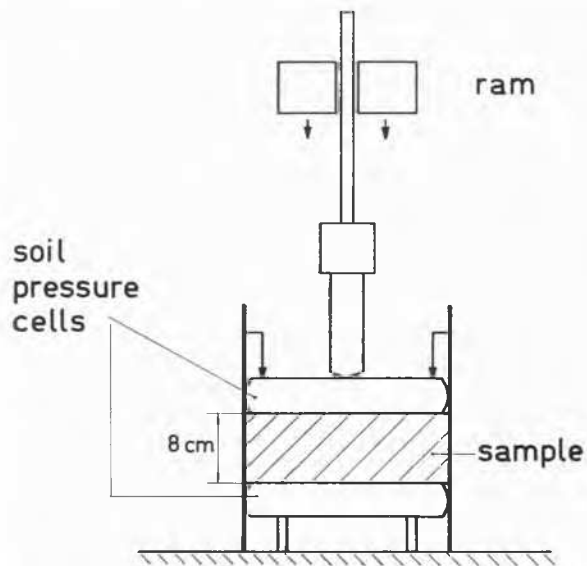


Fig. 51 Dynamic oedometer (vertical section), showing soil sample in rigid casing, pressure cells and deformation gauges as well as loading device (ram).

### Dynamic Stress-Strain Relations

A set of dynamic stress-strain relations is shown in Fig. 52. Most of the deformations are inelastic. They decrease as the number of blows increases. The loading curve is non-linear, as could be anticipated from static tests. If a lower peak pressure is used for the test, the loading curve is the same as in a test with a higher peak value (the curve is shifted to the right because of the higher initial density).

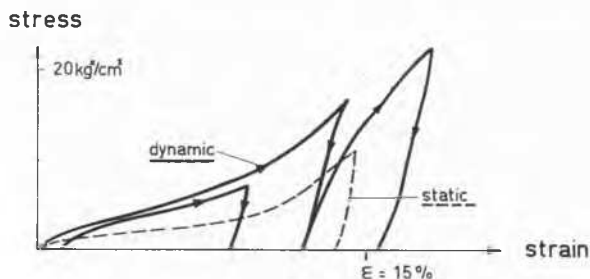


Fig. 52 Dynamic stress-strain relations for gravel. Curves for loading, unloading and reloading. Comparison between dynamic and static diagram.

Comparisons were made between the specific compaction energy and the standard Proctor test results. The energies did not differ very much for a small number of blows, but the discrepancy became large with increasing number of blows. This is not astonishing, as considerable energy is wasted in the Proctor test by pushing away the material from below the ram without compaction effect.

A theoretical approach was made to investigate the effect of the air in the pores on the soil being subjected to transient loading.

### Theoretical Approach

The problem of onedimensional wave propagation normally leads to a system of two quasilinear differential equations of the hyperbolic type. Such a problem is usually solved by the method of characteristics. It can be shown, however, that this method does not apply if the material exhibits general inelastic behaviour, unless very special assumptions are made [3]. As practical problems require solutions for real materials and real boundary conditions, a method was sought for which no assumption had to be made regarding the stress-strain relation, the heterogeneity of the soil and the loading conditions.

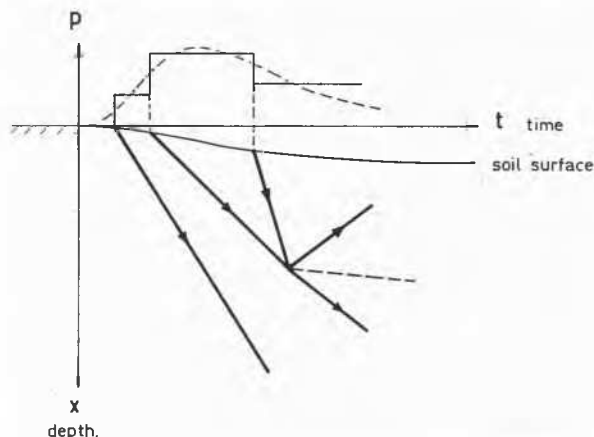


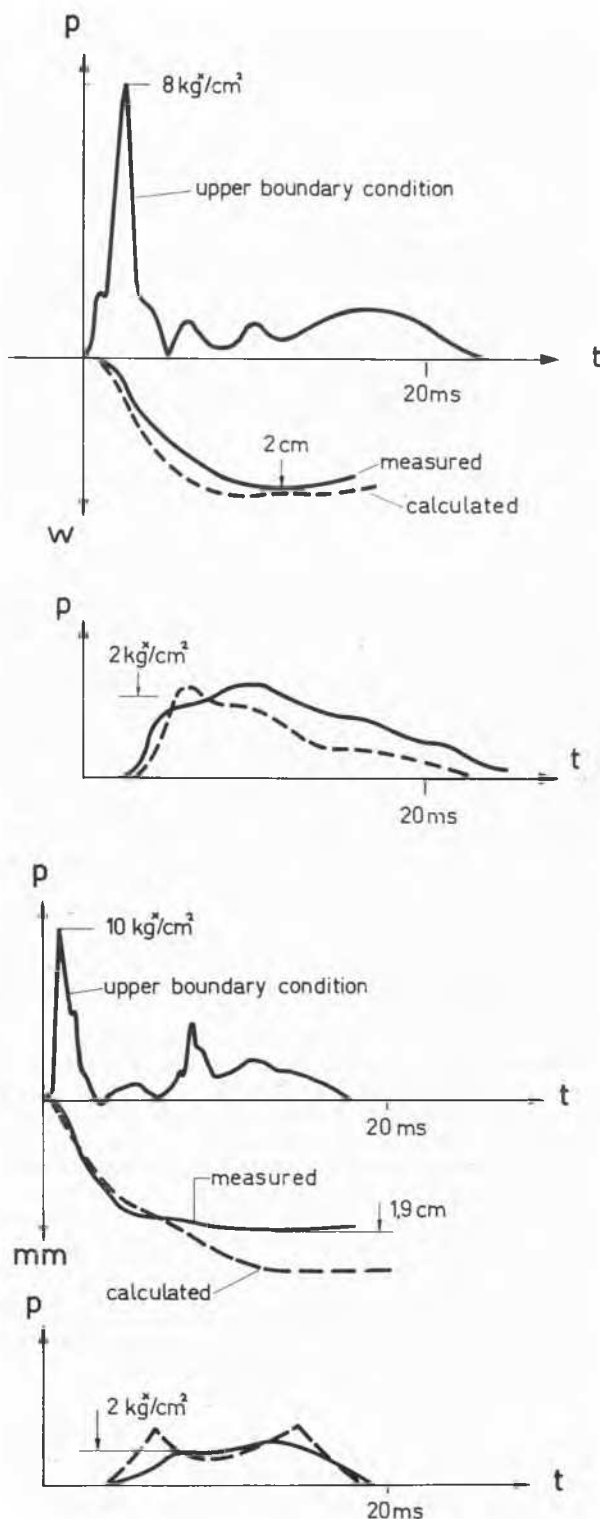
Fig. 53  $x$ - $t$ -plane showing propagation of loading waves and reflection of unloading wave.

This method is illustrated by Fig. 53. The given pressure time curve is divided into several abrupt steps [4]. To calculate the propagation of such a pressure pulse, a continuity equation, Newton's law and the stress-strain relation for every element are established. The result is shown in the  $x$ - $t$ -plane (Fig. 53). When unloading takes place, a wave of greater velocity travels through the soil and tries to overtake the loading wave. At the meeting point reflection occurs, because the unloaded material has a greater particle velocity than the loading wave. This phenomenon is calculated similarly to the "simple wave", only the aforesaid set of equations have to be solved for the upper and the lower part with regard to the meeting point.

Reflection at rigid or yielding surfaces and refraction at heterogeneities of any kind do not present difficulties other than the unloading waves.

### Experimental Verification

After having established a method of calculation and a basic experimental method, it is interesting to compare the results of theory to practical measurements. Tests were made in the dynamic oedometer and under field conditions. In order



Figs. 54-55 Comparison between calculated (solid) and measured (dotted) pressure and displacement curves vs. time.

to get a marked attenuation effect, relatively long samples were chosen for these tests.

Figs. 54 and 55 show the comparison between theory (dotted lines) and test results (solid lines). The agreement is quite good if we consider the inherent uncertainties in real

materials and the difficulties in the measurement of fast transient tests.

### Conclusions

The whole method suggested for the treatment of one-dimensional stress wave problems comprises the following steps.

1. If necessary, the problem to be solved is approximated by a one dimensional state of loading. In an atomic explosion for instance a relatively large area is simultaneously loaded by the same pressure, so that the wave can be taken as one dimensional in many cases, at least to a certain depth.
2. The dynamic stress-strain relation has to be determined or estimated. In an estimation of such a relation for fissured rocks the amount of fissures per unit of depth will help to determine the plastic part of the deformations.
3. The calculations have to be made, taking into account the given boundary conditions and heterogeneities and, of course, the stress-strain relation.
4. From the calculated  $x-t$ -plane, we can derive any desired information, e.g. peak pressure vs. depth, acceleration, displacement, etc.

It is felt that this investigation might be helpful in designing underground shelters, where an approximate estimation of pressures and accelerations is very desirable for the calculation of the whole construction.

### Acknowledgements

The present paper is an abstract from the doctor's thesis (1961) of the author. He is greatly indebted to Prof. G. Schnitter, and to dipl. Ing. Ch. Schaerer as well as to a number of their collaborators for their assistance and instruction. The author also acknowledges gratefully the help of the Swiss Military Department, especially of dipl. Ing. W. Gagg, colonel.

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There are a number of papers in the Proceedings dealing with strength-deformation properties of granular materials. The following notes on data obtained from drained triaxial tests on sands may be of interest to the participants. The comments serve to enlarge a statement made in the introduction to the paper by Chaplin [3], p. 33, vol. II of the Proceedings, and are concerned with the components of strength of granular materials.

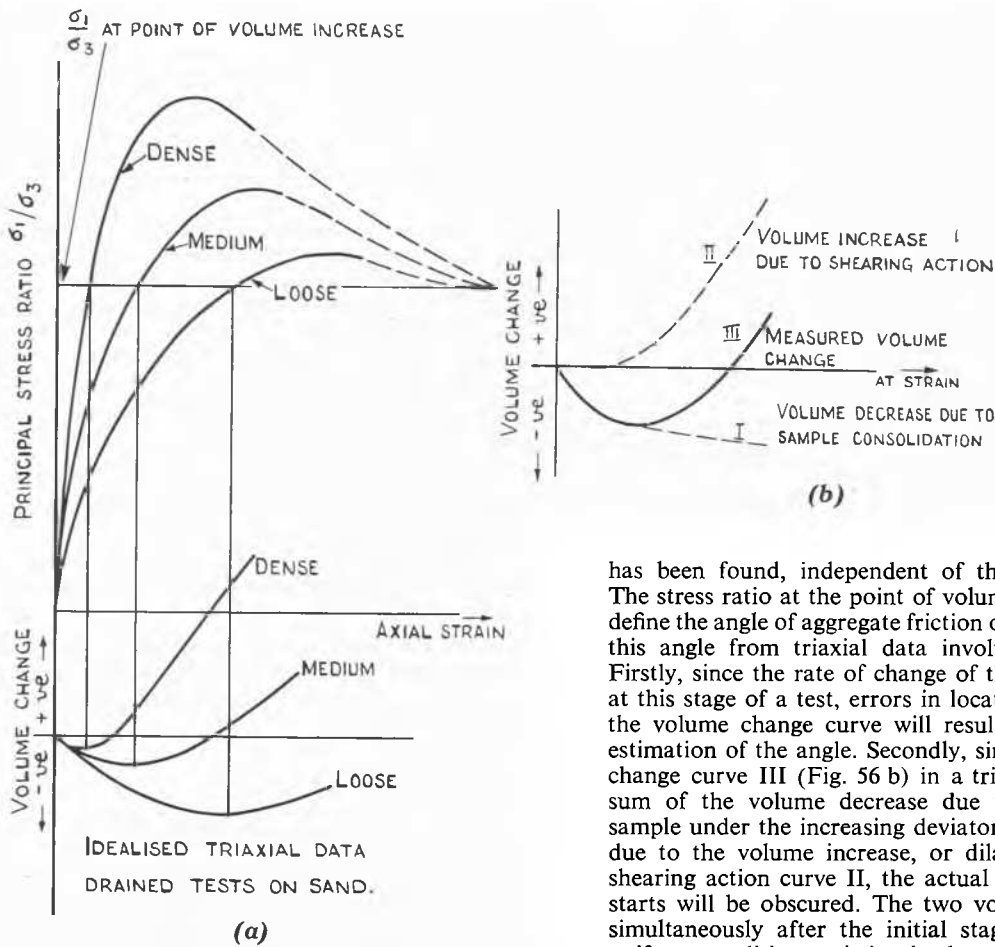


Fig. 56

Fig. 56 a shows stress-strain-volume change curves for a sand in dense, medium and loose states of compaction, as typically obtained in the triaxial compression test with  $\sigma_1$  increasing. The form of the volume change curves are similar, irrespective of porosity, and each shows an initial volume decrease followed by a volume increase. The looser samples may not, however, show an overall increase in volume, over the original volume, at the end of a test. This form has been noted in all tests performed by the writer although such high relative porosities as those obtained by Bjerrum, Kringstad and Kummeneje [2] in vol. I., p. 24 were not achieved.

The results of tests on Loch Aline sand published in 1954 [4] showed that the lowest point on the volume change-axial strain curves, — that is the point of minimum volume of the sample, — corresponded to a constant stress ratio for the sand, irrespective of the porosity. This stress ratio was approximately equal to the stress ratio at the ultimate stage of the test where the volume tends to become constant. Further tests have shown a similar tendency for a different sand and also for small glass spheres. The effect is illustrated somewhat ideally in Fig. 56 a.

A possible interpretation of these findings is to assume that the stresses at the point of volume increase result in the mobilisation of the quasi frictional components of strength, as distinct from other components such as dilatancy which are developed after this stage. To avoid confusion in the nomenclature the friction component is here termed aggregate friction and as well as pure friction includes effects of rolling and rotation which are similar in character to friction. The aggregate friction component of strength should be, as

has been found, independent of the porosity of the sand. The stress ratio at the point of volume increase will therefore define the angle of aggregate friction of the material. Observing this angle from triaxial data involves certain inaccuracies. Firstly, since the rate of change of the deviator stress is high at this stage of a test, errors in locating the turning point on the volume change curve will result in larger errors in the estimation of the angle. Secondly, since the measured volume change curve III (Fig. 56 b) in a triaxial test is the algebraic sum of the volume decrease due to consolidation of the sample under the increasing deviator stress, curve I, and that due to the volume increase, or dilatancy, produced by the shearing action curve II, the actual point at which dilatancy starts will be obscured. The two volume changes will occur simultaneously after the initial stages because of the non-uniform conditions existing in the triaxial sample. The effect will be to produce an overestimation of the angle of aggregate friction.

Experimental data showing the independency of the stress ratio at the point of volume increase with porosity, for Loch Aline sand, is illustrated in Fig. 57 a. The tests at the lower porosities were originally performed to provide sets of Mohr envelopes, and the ranges of porosities over which the results are applicable are indicated. The stress ratio defines an angle slightly in excess of  $28^\circ$  and is represented by curve III in Fig. 57 b. Curve I, (Fig. 57 b) represents the peak angle of friction, and curve II represents the peak angle of friction adjusted to eliminate the dilatancy contribution. This adjustment has been made using the Bishop [1] energy correction. Fig. 57 b infers that there is either an additional contribution to strength other than dilatancy and aggregate friction, which varies with porosity, or that the energy correction does

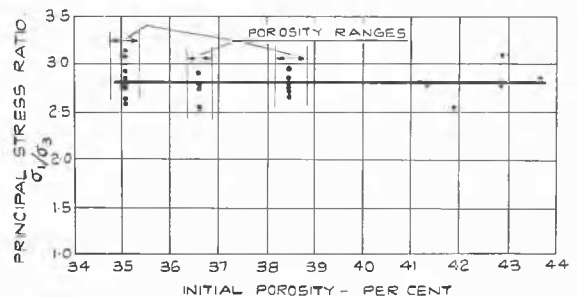


Fig. 57 a Triaxial compression tests  $\sigma_1/\sigma_3$  at point of volume increase. Loch Aline sand.

### Pore Pressure in the Soil Consolidation Process

Since clay particles are not in direct contact with each other, stresses in liquid film plastifying the contacts of particles should be considered effective, while stresses in liquid filling the rest of the pore volume are neutral.

At the moment of applying external pressure (at  $S < 1$ ) the latter is taken by the films in particle contacts, i.e. by effective stresses. Neutral stresses remain unchangeable at this moment. During the consolidation process as water from particle contacts is being squeezed out and bubbles of gas entrained in pores are being compressed the neutral stresses increase. The value by which the neutral stress is increased in the consolidation process will be pore pressure.

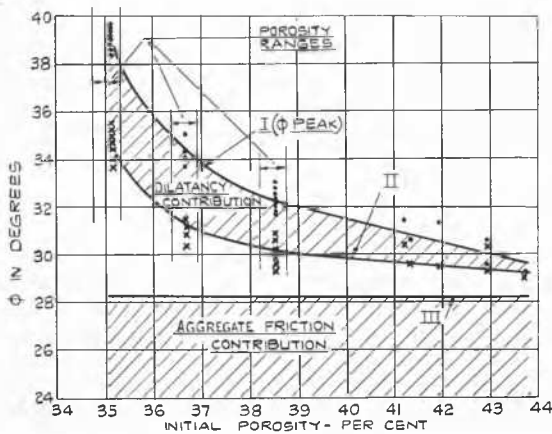


Fig. 57 b Triaxial compression results. Factors contributing to shear strength of Loch Aline sand.

not completely account for the contribution of dilatation in the compression test.

Fig. 58 shows the results of a limited number of tests on artificially graded Leighton Buzzard sand. Insufficient data has been obtained to establish the influence of grading on the  $\phi$ -porosity variation. The trend however shows a different  $\phi$ -porosity relationship for grading 2 than for gradings 1 and 3. The reason for illustrating Fig. 58 is to indicate that the aggregate friction is a property of the grain material and is apparently independent of grading as well as porosity.

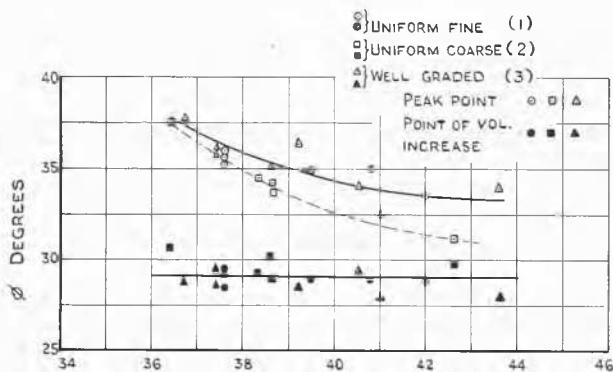


Fig. 58 Initial Porosity — %. Triaxial Compression tests. Artificially graded Leighton Buzzard sand.

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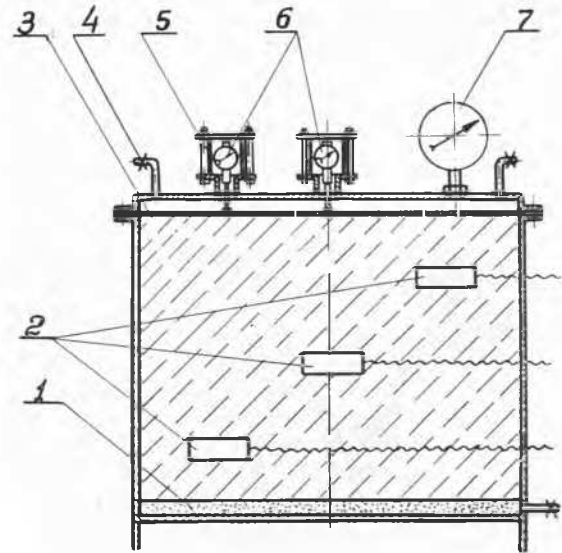


Fig. 59 Scheme of installation for investigation of one-dimensional consolidation process.

1. Drainage; 2. Piezodynamometers; 3. Rubber membrane; 4. Cocks; 5. Plexiglass cylinders; 6. Indicators; 7. Manometer.

The investigation of one-dimensional consolidation process was carried out on the installation scheme which is given in Fig. 59. The soil of given density and water content was placed in the chamber 40 cm dia. and 32 cm height which was closed by the watertight cover; rigidity of the bottom as well as chamber cover was provided by means of radial steel ribs. The soil surface settlement was registered by two indicators enclosed in the watertight cylinders made of plexiglass. Sand drainage 3 cm thick was placed into the soil base. In the undrained tests water seepage from the drainage was not allowed.

Pore pressure was measured at three levels by means of piezodynamometers (Fig. 60) used in the field conditions (earth dams, foundation "soil"). The piezodynamometer consists of a cylindrical cell filled with water, inside of which the wire dynamometer is placed for measuring only neutral pressure. Small value deflection of the wire dynamometer membrane forming the cavity with volume about 15 mm<sup>3</sup> per 1 kg/cm<sup>2</sup> of pressure has no effect on measured pore pressure value, i.e. inertness of the gauge has no practical importance.

Simultaneously tests were carried out on some types of clay in the oedometer by undrained and drained systems followed by measuring pore pressure by an automatically operated compensating apparatus (Fig. 61). Water volume

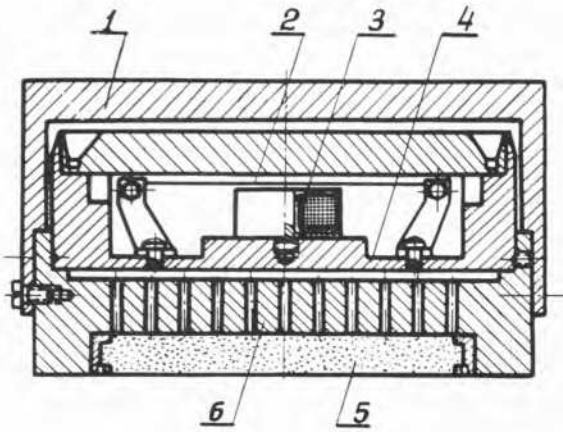


Fig. 60 Piezodynamometer.  
1. Cell; 2. Wire; 3. Electromagnet; 4. Membrane;  
5. Filter; 6. Perforated plate.

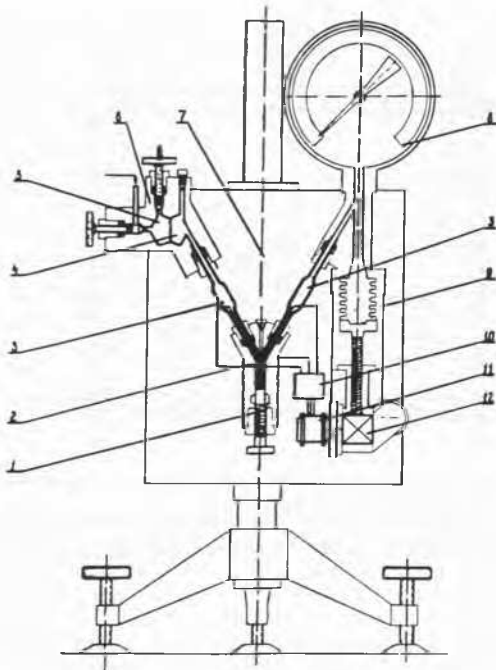


Fig. 61 Apparatus for pore pressure measuring.  
1. Mercury level regulator; 2. Mercury; 3. Oil; 4. Dividing diaphragm; 5. Water; 6. Cock; 7. Contacts;  
8. Manometer; 9. Uplifting mechanism; 10. Electron relay; 11. Electrometer; 12. Worm reducer.

necessary for measuring pore pressure did not exceed  $0.2 - 0.3 \text{ mm}^3$ .

Test results allow the following conclusions :

At partial soil saturation pore pressure from applied load gradually increased as water from particle contacts is being squeezed out and gas bubbles entrained in pores are being compressed.

Under conditions of undrained tests pore pressure increases simultaneously across the whole soil volume (Fig. 62) in accordance with total pressure.

Steady pore pressure in undrained tests is expressed by the following equation :

$$u = \frac{-\mathcal{M} + \sqrt{\mathcal{M}^2 + 4u_0\sigma}}{2}$$

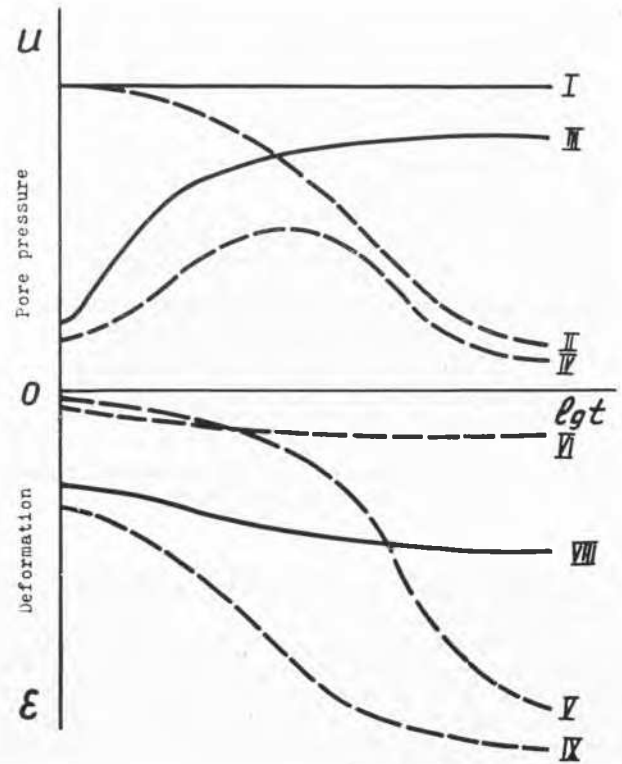


Fig. 62 Pore pressure during consolidation process.  
— undrained system; I, II, III, —  $S=1$   
- - - - - drained system; III, IV, VII —  $S=1$   
V—relative volume of squeezed water.

where  $\mathcal{M} = nE(1 - S + \beta S) + u_0 - \sigma$

$u$  = initial pore pressure;  $\beta$  = Henry ratio.

At full soil saturation ( $S = 1.0$ ) according to the following equation :

$$nE(1 - S + \beta S) = 0$$

$$u = \sigma \text{ and } \mathcal{D} = \frac{u}{\sigma} = 1$$

In the case of tests by the drained system pore pressure increases with various rates in different parts of the sample depending on the length of the filtration path. The shorter the filtration path and the lower soil saturation is, the slower the pore pressure will increase and the lower the value it will attain.

During the first period of consolidation, when pore pressure increases, the volume of water squeezed out of the soil will be less than the volume of its compaction by the value of the compression volume of entrained gas bubbles which is dissolved.

The second period is characterized by the lowering of pore pressure. In this period the volume of soil compaction is equal to the difference between the volume of squeezed water and volume of expansion of entrained gas bubbles, considering also its transition from dissolved into free state as the pore pressure decreases.

For the third period the slow compaction attenuation is typical; usually at low practically constant pore pressure (secondary compression).

Under full saturation the pore pressure attains the value of the external load at the moment of its application.

At high saturation and plastic consistency of the soil the

first period of the consolidation process appears to be of short duration and for such soils is not clearly shown.

Obviously, for this reason it was generally stated that pore pressure reaches its maximum value at the moment of load applying as well as for partially saturated soil.

M. B. LADANYI (Yougoslavie)

La présente discussion se rapporte à la communication 1/51, présentée par H. B. Poorooshasb et K. H. Roscoe.

Dans cette communication, les auteurs traitent, entre autres, le problème de l'influence de la dilatation du sol lors du cisaillement sur la résistance au cisaillement mesurée, et ils proposent une nouvelle formule (15) destinée à corriger, pour cette influence, la valeur du déviateur mesurée lors d'un essai triaxial.

Je suis heureux de constater que les auteurs de la communication 1/51 sont arrivés aux conclusions analogues à celles publiées dans les Réf. [1] et [2]. En effet, pour le cas de la symétrie cylindrique des contraintes, une formule analogue à (15) a été proposée en 1959 dans ces références.

En utilisant les notations de la communication 1/51, cette dernière formule se présente sous la forme suivante :

$$q_L = (\sigma'_1 - \sigma'_3) + \frac{3}{2} p \frac{\delta v}{\delta \gamma} \quad (1)$$

([1], éq. 23-21; [2], éq. 33)

où 
$$p = \frac{1}{3} (\sigma'_1 + 2\sigma'_3)$$

est la contrainte sphérique, et

$$\delta \gamma = \delta \varepsilon_1 - \delta \varepsilon_3$$

est l'accroissement de la distorsion des facettes primitivement inclinées à 45° sur la direction des facettes principales.

Il est facile de démontrer que la formule (1) ci-dessus est identique à la formule (15) proposée dans 1/51.

Signalons également qu'à partir des mêmes considérations, dans le cas de la déformation plane, mais sous réserve que la position des axes principaux reste constante au cours du cisaillement (essai « biaxial »), une formule analogue à (1) et (15) peut être déduite, notamment :

$$q'_L = (\sigma'_1 - \sigma'_3) + \frac{3}{1 + \nu} p \frac{\delta v}{\delta \gamma} \quad (2)$$

([1], éq. 23-40; [2], éq. 49),

où 
$$p = 1/3 (1 + \nu) (\sigma'_1 + \sigma'_3),$$

$$\delta \gamma = \delta \varepsilon_1 - \delta \varepsilon_3$$

et  $\nu =$  coefficient de Poisson.

Pour tenir compte du fait qu'une partie de l'énergie lors de la distorsion est absorbée dans la consolidation, et par tant ne doit pas être considérée lors de la correction du déviateur, les auteurs de la communication 1/51 proposent d'introduire un paramètre isotrope  $r$ , qui devient égal à  $p$  lorsque  $\delta v'$  tend vers  $\delta v$ , et s'annule, pour les sols sans cohésion, lorsque  $\delta v'$  tend vers zéro.

Cependant, même lorsque  $r = 0$ , et que la formule (15) reste valable, il faut, à notre avis, faire une remarque complémentaire concernant la valeur de  $\delta v$  à introduire dans cette formule. En effet, dans un essai triaxial, comportant une variation simultanée des contraintes déviatoriques et de la contrainte sphérique (comme p. ex. l'essai avec  $\sigma_3 = \text{const.}$ , ou un essai non drainé), la variation totale du volume unitaire peut être représentée par la somme

$$\nu = \nu_c + \nu_d \quad \dots (3)$$

où  $\nu_c =$  « compression », définie comme la part de la variation du volume unitaire totale  $\nu$  due à la variation de la contrainte sphérique  $p$ , sans modification des composantes déviatoriques des contraintes, et  $\nu_d =$  « dilatance », définie comme la part de la variation du volume unitaire totale  $\nu$  due à la modification des contraintes déviatoriques à contrainte sphérique constante.

Pour les accroissements correspondants des variations du volume unitaire, il faut également écrire d'après (3) :

$$\delta \nu = \delta \nu_c + \delta \nu_d \quad \dots (4)$$

Une application correcte de la formule (15) exige que l'accroissement de la variation du volume unitaire  $\delta \nu$ , introduit dans la formule, ne contienne pas la part  $\delta \nu_c$  due à la consolidation isotrope lors du cisaillement. On peut se rendre compte de ce que cette dernière limitation est nécessaire, lorsqu'on applique la formule (15) au cas de la consolidation isotrope. Dans ce cas, si on introduit dans (15) la valeur totale  $\delta \nu = d\nu_c = 3\delta \varepsilon_1$ , on obtient  $q_R = \infty$ .

Par contre lorsque, dans la formule (15), on n'introduit que la variation de volume  $\delta \nu_d$  due au phénomène de dilatance, on obtient, dans le cas de la consolidation isotrope :

$$\delta \nu = d\nu_d = 0,$$

et la formule (15) fournit la valeur correcte  $q_R = 0$ .

En d'autres termes cela signifie qu'une formule de correction comme (15), (1) ou (2) ne peut être appliquée directement et sans élimination préalable de  $\delta \nu_c$ , que dans les deux cas suivants :

1. A chaque instant d'un essai de cisaillement, où la contrainte sphérique est maintenue constante lors du cisaillement (essai du type  $p = \text{const.}$ ), étant donné que, pour ce type d'essai, on a à chaque instant  $\delta \nu_c = 0$  et  $\delta \nu = \delta \nu_d$ .

2. A l'instant de la rupture d'un essai de cisaillement du type quelconque, étant donné qu'à l'instant de la rupture, la condition  $p = \text{const.}$  est satisfaite par définition, et par conséquent,  $\delta \nu = \delta \nu_d$ , quelles que soient les variations des contraintes qui se sont produites avant la rupture.

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M. T. E. PHALEN (Etats-Unis)

## Discussion of some "Grading-Density Relationships for Sands", by Bruce Hutchinson and David Townsend (1/27)

During the process of investigating the age old problem of relative densities from field and laboratory, a series of tests were conducted at Northeastern University in Boston, Massachusetts. These tests are still under way. Basically, a series of tests were conducted on a set gradation of gravel and sand utilizing five different hammers and lengths of free fall to determine maximum and minimum densities. The tests verify:

1. As the compactive effort (i.e. the energy imparted to the soil) increases the maximum and minimum densities increase.
2. As the grain size of the  $D_{10}$  and  $D_{60}$  sizes approach one another, extreme difficulties were noted in obtaining a

clear picture of what the maximum and minimum densities were.

3. The shape of the grain structures also had a marked affect on the densities that were developed.

4. Greater difficulties in obtaining maximum and minimum densities were noted as the  $c_u$  approached unity.

In essence, tests that are currently being conducted to provide more technical information concern the following questions and topics concerning the consolidation problem.

1. The effect of compactive energies on various  $c_u$  values.
2. The effect of these same compactive energies on the same  $c_u$  values but utilizing different grain shapes.
3. To investigate the typical curve of water content vs. dry density, which apparently is a cubic equation, to see what practical criteria can be applied to a theoretical curve.

The laboratory tests along this line will consist of standard compaction tests on natural and laboratory prepared soil gradations together with photomicro graph techniques for establishing grain size and shape.

These tests in part will also enhance and add to our basic knowledge of the compaction problem and will add to the developments presented by Messrs Hutchinson and Townsend.

M. T. E. PHALEN

#### Discussion on "Optical Methods of Measuring the Cross Section of Samples in the Triaxial Test" by V. Escario and S. Uriel (1/15)

It was with avid interest that the technique of measuring cross sectional areas of samples subject to triaxial testing was read. This work coincides with similar experiments carried on by Woodland G. Shockley and Richard G. Ahlvin of the Waterways Experiment Station at Vicksburg, Mississippi, some time prior to 1960. The technique basically consisted of photographing the specimen during various stages of testing and noting the changes in the rectangular grid pattern placed on the specimen skin prior to loading. This technique is further described in the Shockley and Ahlvin paper entitled "Non-Uniform Conditions in Triaxial Test Specimens" published in the proceedings of the A.S.C.E. Research Conference on Shear Strength of Cohesive Soils in Boulder, Colorado, June, 1960.

The importance of accurately describing the phenomena of changes in area and volumes are presented in a discussion of this paper by the author. This technique would assist tremendously in the authors' hypothesis on the mechanism of failure of a non-cohesive soil as presented in this discussion which briefly states:

1. At the instant the load is applied there is a slight decrease in volume throughout the specimen as the smaller particles drop into the larger voids.
2. As this slight decrease in volume occurs, the particle movement at the top and bottom is toward the center of the specimen setting up frictional shearing stresses between plate and sample.
3. As this frictional shearing stress develops this increases the major principal stress in a conical area dependent on the angle  $\Phi$ .
4. In this conical area the minor principal stress becomes tensile in character. This small tensile stress may act as a mechanism to separate particles instantaneously, thus increasing voids slightly, locally, in this area but sufficient to cause other grains to fill the void, thus adding to the increase in density.

5. The stresses in this conical area are changing continuously in order to maintain an equilibrium status.

6. This increase in density and subsequent increase in the effective principal stress allows this area to withstand higher shearing stresses.

7. The area above and below the conical areas are unaffected by the frictional shearing stresses, provided the relationship between the angle  $\Phi$ , the diameter, and the height are such that the two conical areas do not intersect.

8. The area in the middle unaffected by frictional shearing stresses decreases in void ratio and increases in density to a critical point where the grains then begin to roll over one another and the density decreases and the void ratio increases.

9. The weight of the sample should be considered when making shear strength calculation particularly if the sample is large.

10. The friction shearing stress existing in a triaxial test on non-cohesive soil is dependent on the unit weight of the material and an angle  $\phi$  a property of the material.

11. The height of the specimen should be determined as a function of the original density and the angle  $\phi$ .

The author suggests that the technique could be improved to obtain better results by:

1. Instituting a photography technique that will take pictures at predetermined time intervals during the entire test set-up.
2. Cameras recording diameter changes shall be so located that the diameters are not measured in the same plane. This would assist in showing the difference in diameters as may be affected by non-homogeneous and non-isotropic conditions.

M. A. PIASKOWSKI (Pologne)

I should like to refer to the paper of Mr M. Said Youssef (vol. I, pp. 419-421). It must be stressed that the problem of the influence of temperature on the obtained values of different characteristics of soils seems to be very important and it is worth more attention. Similar work was carried out also in the Institute of Building Technics in Warsaw; generally speaking the character of the obtained relations was more or less similar to the ones given in the above mentioned paper. Only the change of measured parameters with temperature was not only much smaller, but often it was too small for accurate measuring. The Fig. 63 shows for instance the influence of temperature on the value of the liquid limit of a soil.

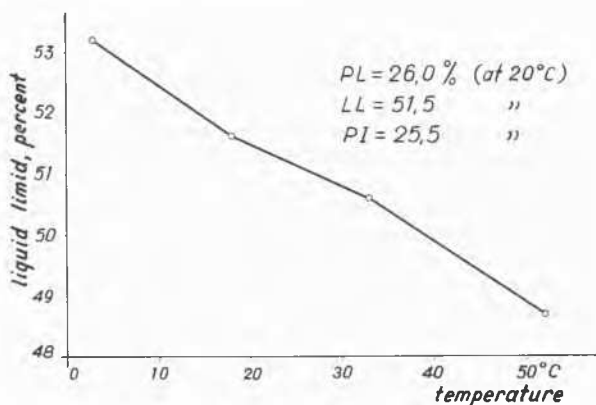


Fig. 63

It should be supposed that the soils on which the research was carried out by Mr M. Said Youssef contained perhaps especially active clay minerals or had a comparatively high content of organic matter and this could explain such a big influence of temperature; it seems however that for the common European soils this influence is in any case somewhat smaller.

M. P.W. ROWE (Grande Bretagne)

In paper 1/51 the Authors have used the form of energy correction for use with a shear box type test after D.W. Taylor, namely :

$$\tan \varphi_r = \tan \varphi_{\max} = \tan \theta$$

where

$$\tan \theta = \frac{dy}{dx} \text{ or } \frac{\Delta V}{\delta \Delta},$$

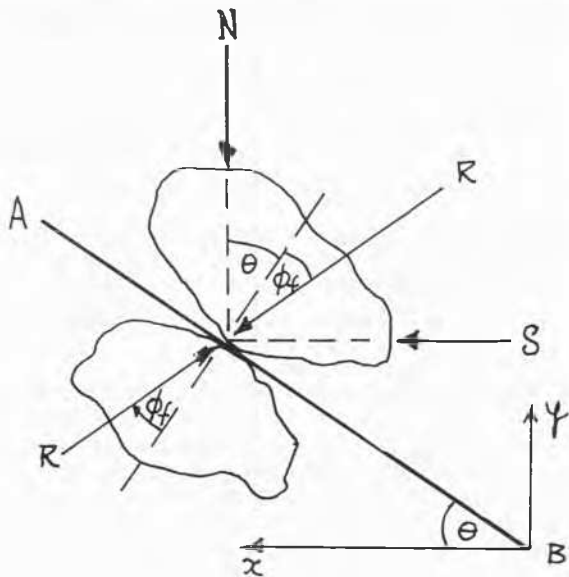


Fig. 64

According to Fig. 64, during dilatancy particles shear on interfaces  $AB$  inclined at  $\theta$  to the direction of the shear force  $S$ . Let  $\varphi_f$  equal the angle of shearing resistance on the plane  $AB$ .

Then

$$\frac{S}{N} = \tan (\varphi_f + \theta)$$

Now  $S$  is composed of three components

$$S = S_1 + S_2 + S_3$$

$S_1$  is the component due to the external work done in dilating against  $N$

$$\text{Thus } S_1 \cdot dx = N \cdot dy$$

$$\text{or } S_1 = N \cdot \tan \theta$$

Then

$$\frac{S_2 + S_3}{N} = \frac{S - S_1}{N} = \tan \varphi_{\max} - \tan \theta$$

and this is the form of correction used by A.W. Bishop and A.W. Skempton 1950; R.E. Gibson, 1953; K.H. Roscoe, A.N. Schofield, C.P. Wroth, 1958; and H. Poorooshab and K.H. Roscoe 1961;

where

$$\tan \varphi_r = \frac{S_2 + S_3}{N}, \text{ and } \varphi_r \text{ is}$$

assumed to be an angle of shearing resistance corrected for dilatancy.

But additional internal work in friction has to be done simply because of dilatancy, and this is the components  $S_2$ .

The force  $S$  induces a normal reaction on plane  $AB$  equal to  $S \sin \theta$ . Its frictional strength is  $S \sin \theta \tan \varphi_f$

$$S_2 \cos \theta = S \sin \theta \tan \varphi_f$$

$$S_2 = \tan \theta \tan \varphi_f$$

Then

$$S_3 = S - S_1 - S_2$$

$$S_3 = S [1 - \tan \theta \tan \varphi_f] - N \tan \theta$$

$$S_3 = N \tan (\varphi_f + \theta) \times [1 - \tan \theta \tan \varphi_f] - N \tan \theta$$

or

$$\frac{S_3}{N} = \tan \varphi_f$$

Therefore  $\varphi_r > \varphi_f$  for dilating soils and the difference increases with the degree of dilatancy.  $\varphi_f$  is a fundamental parameter which depends on the true angle of friction between grains,  $\varphi_u$ , and the degree of remoulding expressed by a factor  $K$ , at the instant of observation, where  $\tan \varphi_f = K \cdot \tan \varphi_u$ . But  $\varphi_r$  is not so fundamental because it varies with the degree of dilatancy. The above equations were first stated by Newland and Allely, 1957, but have received little attention. With decreasing porosity  $\varphi_r$  increases, but  $\varphi_f$  decreases towards a value equal to  $\varphi_u$ , whereas Newland and Allely appear to have expected  $\varphi_f$  to be a constant. The significance of this cannot be explained here but it would be useful if values of  $\varphi_f$  could be included in future publications rather than  $\varphi_r$ .

M. H.U. SMOLTCHYK (Allemagne)

I should like to comment on the rheological aspects of the mechanical models used for symbolizing soil bodies. There are two dangers to be kept in mind. Firstly, the three classical model elements of rheology are all developed for continuous media and neglect volume plasticity. Therefore, these patterns do not cover all essential features of soil deformation. Secondly, in stating the rheological properties of soils, most papers deal with cohesive soils. It should be noted, however, that similar phenomena can be observed in dry sands. This is very conclusively shown by the paper of Messrs Seed and Chan (1/59). Of course, these effects are not as spectacular and of the practical importance they obviously are in clays. Anyway, a sand shows all three components of deformation : elastic strain, limited plastic flow (which accords with plastic strengthening in terms of the theory of plasticity) and viscous flow (which covers the time delay of both elastic strain and plastic flow). I would therefore like to present the mechanical soil pattern of Fig. 65.

A sequence of some  $k$  particles stands on a rough surface, each of them with a spring ahead that may be damped. At rest, there is a small free distance between the spring and the next particle which symbolizes a void ratio. The damping of the spring is necessary if we assume the friction on the surface to be a friction at rest only, the particle moving freely as soon as the movement starts. When the first element is subjected to a load  $P$ , it overcomes its friction ("the Bingham limit") and starts moving until its spring comes into action. Balance is achieved, if the next element counterbalances  $P$  minus the friction at rest of the first element (being

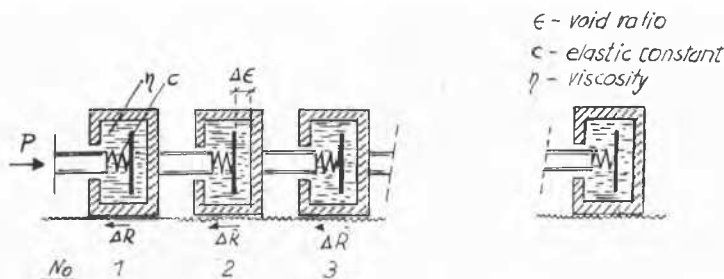


Fig. 65 Linear soil model.

efficient again as soon as the movement stops), and so on. Each amount of external load is so accompanied by a certain amount of limited plastic deformation. Failure occurs when even the last element starts moving. So, we get a non-continuous pattern which truly reflects the extending area of limited plastic deformation which follow an increasing load to mobilize the necessary internal friction. It is possible to extend this pattern to a compound model with more than one sequence of elements (Fig. 66) and, at last, to a three-dimensional type (Fig. 67).

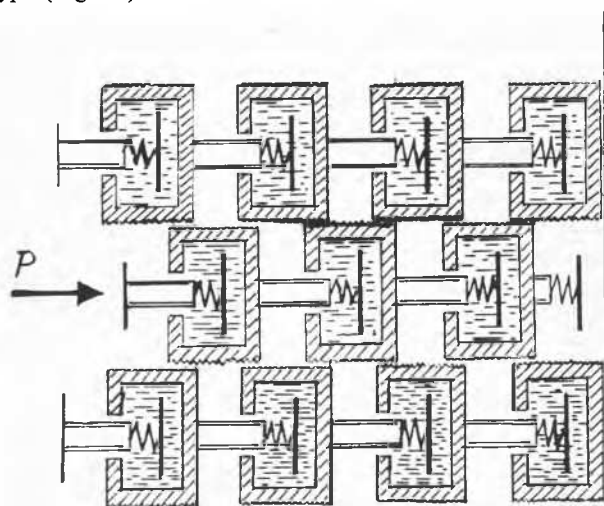


Fig. 66 Linear compound soil model.

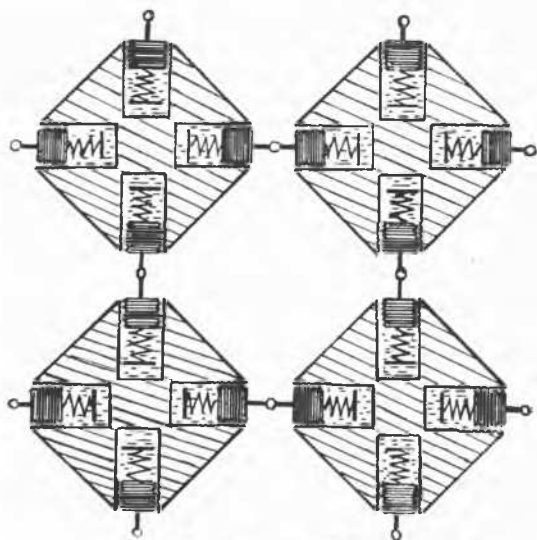


Fig. 67 Three-dimensional soil model.

Another point, which refers to the paper 1/66 of Messrs Vialov and Skibitsky on the rheology of soils. In their figure 2 they use a Mohr envelop which is continuously curved in a parabolic manner with a vertical tangent at zero shear stress. This, in turn, means that there are indefinitely many states of stress possible which lead to failure because the tangent circle  $\sigma_{2,k}$  of this point (Fig. 68) has a finite radius.

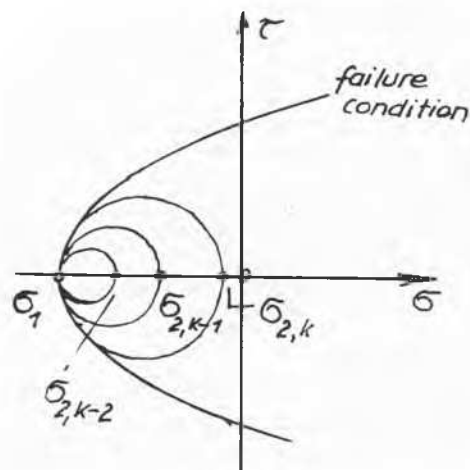


Fig. 68

All circles with the same amount of first principal stress but a smaller radius as well fulfill the failure condition. This seems to be contradictory to physical observance. A failure condition may be of a curved character in the Mohr plane but must not have a vertical tangent at zero shear stress, in order to get definite conditions.

M. E. SPENCER (Grande Bretagne)

#### The Relationship between Porosity and Angle of Internal Friction

In paper 1/5, L. Bjerrum, S. Kringstad and O. Kummeneje give results from triaxial tests on a fine sand in which the angle of internal friction ( $\varphi$ ) is plotted against the porosity ( $n$ ) at failure. The value of  $\varphi$  is shown to decrease sharply as the maximum porosity is approached.

I have obtained very similar results in a theoretical analysis relating  $\varphi$  and  $n$  to the angle of friction between the particles ( $\mu$ ) for spherical particles. In Fig. 69 I have shown the curve (dotted) given by my theoretical relationship (taking  $\mu = 16.6^\circ$ ) superimposed on that given in Fig. 7 of paper 1/5. It can be seen that the two curves are very similar in shape and that the increasing steepness of the curve for the sand as the maximum porosity (46.2 per cent) is approached is repeated in the theoretical curve for the spherical particles as the maximum porosity of 47.6 per cent is approached.

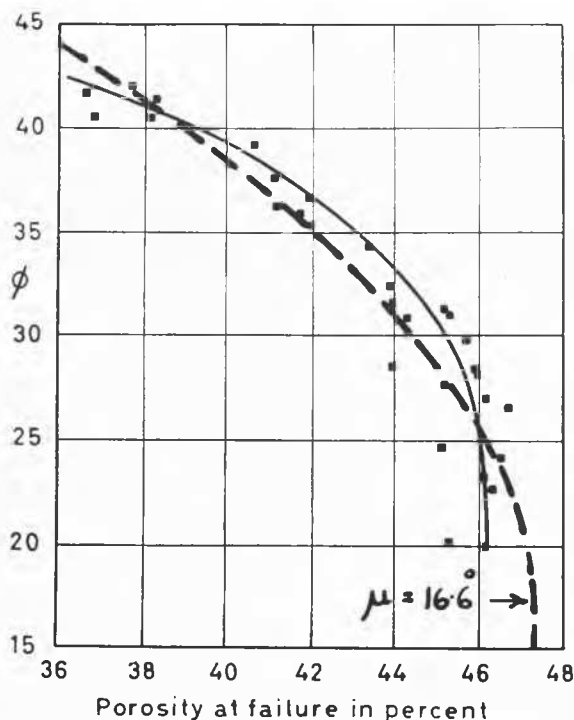


Fig. 69 Porosity at failure versus angle of internal friction. This is a copy of figure 7 from paper 1/5 onto which an additional curve (shown dotted) has been superimposed.

*The relationship between the angle of internal friction ( $\phi$ ) and the angle of friction ( $\mu$ ).*

With regard to the effect of  $\mu$  on  $\phi$ , I wish to refer to paper 1/10 by P. Dantu in which the relationship between  $\mu$  and  $\phi$  for spherical particles is given in Fig. 8 (M. Dantu's paper). In Fig. 70, M. Dantu's curve is shown on that which I obtained (shown dotted). It can be seen that the two curves are somewhat similar in shape, though mine is much the steeper of the two.

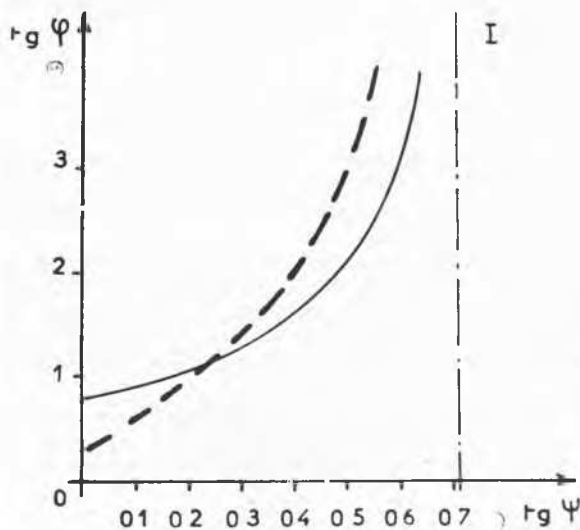


Fig. 70 This is a copy of figure 8 from paper 1/10 onto which an additional curve (shown dotted) has been superimposed.

#### Derivation of relationship between $\phi$ , $n$ and $\mu$

Though space does not permit me to give the full derivation of relationship here, I would like to discuss its basis briefly. I shall be glad to go into further detail privately with anyone interested.

In deriving the relationship between  $\phi$ ,  $n$  and  $\mu$ , I have considered an imaginary triaxial specimen consisting of spherical particles packed in a perfectly regular pattern. The spheres were arranged in layers in a triangular pattern with the spheres in one layer fitting in between those in the adjacent layers. I have calculated for this packing, the effect of increasing the spacing of the spheres in each layer on the porosity and upon the deviator stress required to cause slipping to occur on an oblique plane. Hence I obtained the relationship between  $n$  and  $\phi$  in terms of the angle of friction ( $\mu$ ) between the particles. This relationship is illustrated in Fig. 71.

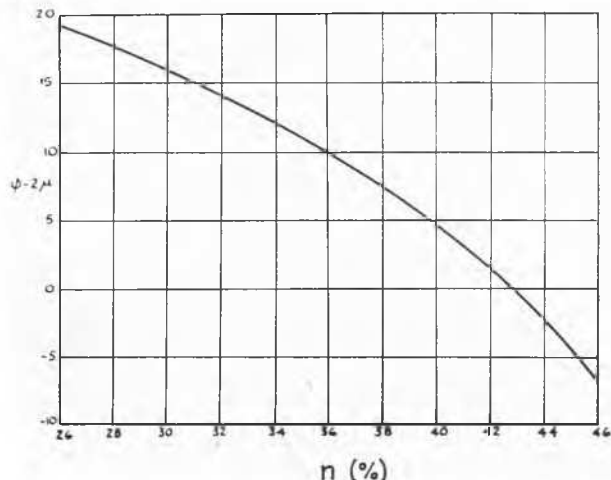


Fig. 71 The relationship between the porosity ( $n$ ), the angle of internal friction ( $\phi$ ) and the angle of friction particle on particle ( $\mu$ ).

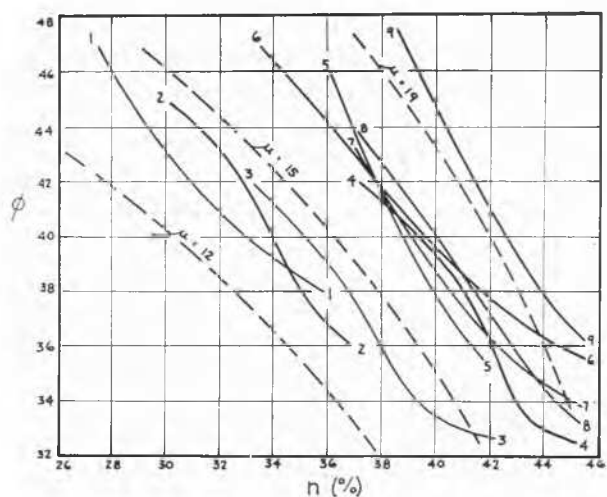


Fig. 72 Angle of internal friction versus porosity for a number of granular materials (sources given in table 1). The dotted curves show the theoretical relationship for different values of  $\mu$ .

Using this relationship I have plotted a family of curves showing the variation of  $\phi$  with  $n$  for different values of  $\mu$  and have superimposed on this graph curves obtained by different workers (given in Table 73) from shear tests on different types of granular materials. These curves are shown

in Fig. 72. The similarity between the theoretical curves and the experimental ones is quite striking.

Table 73

Curve No.	Soil Type	Reference
1	Heathrow gravel - well graded.	A.W. Bishop, 1948. A large shear box for testing sands and gravels. Proc. 2nd Int. Conf. on Soil Mech. and Found. Eng. vol. I, p. 207.
2	Walton Sandy gravel.	
3	Brasted sand.	
4	Ham river sand - uniform.	
5	Chesil Bank pebbles uniform.	
6	Silt	A.D.M. Penman (1953). Shear characteristics of a saturated silt measured in triaxial compression. Géotechnique, Dec. 1953.
7, 8, 9	Sand.	A.W. Bishop and A.W. Gamal Eldin (1953). The effect of stress history on the relation between $\phi$ and porosity in sand. Proc. 3rd Int. Conf. on Soil Mech. and Found. Eng. vol. I; p. 100.

I have also carried out triaxial tests on specimens formed of spherical particles of a number of different materials. For glass ballotini in particular, the agreement between the experimental and theoretical results was quite good as can be seen in Fig. 74 in which the points represent the relationship between  $\phi$  and  $n$  for a value of  $\mu$  of  $6.8^\circ$  (the average value of  $\mu$  obtained for glass in sliding tests was  $6.7^\circ$ ).

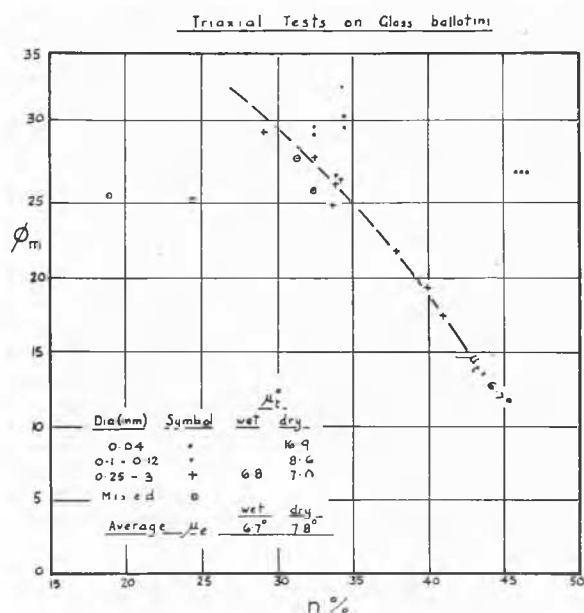


Fig. 74 Angle of internal friction versus porosity for glass ballotini. The points were obtained from triaxial tests; the dotted line shows the theoretical relationship.

I have also checked the validity of the theoretical relationship by plotting the values of  $\mu$  implied by the triaxial tests against those determined in sliding tests. The comparison

is shown in Fig. 75, in which  $\mu_t$  represents the value of the angle of friction obtained by applying the results of the triaxial tests to the theoretical relationship (between  $\phi$ ,  $n$  and  $\mu$ ) and  $\mu_e$  represents the value obtained for the angle of friction in sliding tests. In general there is good agreement between  $\mu_t$  and  $\mu_e$ ; the discrepancy in respect of the very smallest sizes of glass particles tested may well have been due to an increase in the friction between the particles which occurs when the forces between them are very small, as is the case in specimens consisting of a very large number of very small particles.

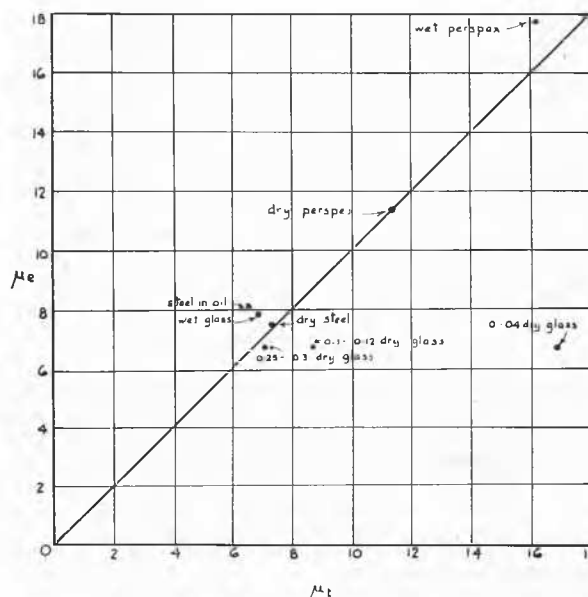


Fig. 75 Comparison of values of angle of friction obtained from sliding tests ( $\mu_e$ ) with value obtained from triaxial tests using the theoretical relationship ( $\mu_t$ ).

The full details of the derivation of the theoretical relationship between  $\phi$ ,  $n$  and  $\mu$  and of the experimental work carried out in this project have been given in a paper submitted to *Geotechnique*.

M. G. STEFANOFF (Bulgarie)

Dans leurs deux rapports, Scherrer (1/55) et Karlsson (1/29) ont décrit la détermination de la limite de liquidité à l'aide d'un cône semblable à celui qui a été présenté au Congrès de Londres (1).

Je voudrais remarquer que le désavantage fondamental de la mesure de la consistance, indépendant de la manière de détermination, reste qu'on la détermine sur des échantillons remaniés. Dans le rapport de Karlsson sont montrées des possibilités pour déterminer la consistance sur des échantillons intacts.

Je veux ajouter que la détermination de la consistance sur des échantillons intacts s'avère possible à l'aide d'un cône. Je regrette de ne pouvoir présenter ici ces matériaux.

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- [1] STEFANOFF, G. (1957). *Session I. Proc. 4th International Conference on Soil Mechanics and Foundation Engineering*, London, vol. III; p. 97.

The rheology of soils has become a very promising and important aspect in the fundamental research on the physico-chemical and mechanical properties of soils and their application to practical problems. For a satisfactory understanding of the rheology of soils the following 3 points need careful research: 1) structure mechanics under various physico-chemical conditions; 2) a suitable testing procedure for more objective determinations of the rheological quantities in combinations with long term field observations; 3) new theories and methods of computation for practical applications. The importance of point 1 has also been stress-

ed by Goldstein (1960) who points out that a mere mathematical description of laboratory data is insufficient for an understanding of the rheological properties.

In order to study the rheology of clays and loess we have carried out a large series of rheological, physico-chemical, mineralogical, swelling and permeability tests. For these soils we found a lower yield-value  $f_1$  or  $f_2$  and an upper yield-value  $f_3$ , completely in agreement with the important results by Ménard ((1/42), Murayama-Shibata (1/46) and Folque (1/19). The lower yield value will be denoted  $f_1$  or  $f_2$  dependent on the internal structure as it will be discussed further on. In the rate of permanent shear/stress curves (Fig. 76) both lower yield values are shown as a Bingham limit, whereas  $f_3$  marks the transition from continuous to accelerating flow. In many cases we have found a fairly good linearity of the stress-strain isochrones (Fig. 77 b).

On the basis of a study mentioned above in points 1 and 2, I have devised the schematic structural networks and corresponding rheological models for fat clays A, over-consolidated clays B and Lanchow loess shown in the Figs. 78, 79 and 80 respectively. The hypothesis of a card-house structure dominated by corners-flat surface contacts in Fig. 78 a (Tan 1957) has been ingeniously proved by Rosenquist (1959) who further correctly stresses the need for a careful re-examination of the fundamental concepts of soil mechanics. The model in Fig. 78 b showing its basic model (series coupling of spring  $H_1$ , Kelvin element K and dashpot N-F) parallelly coupled to a spring  $H_2$ , holds for stresses less than the yield-value  $f_2$ . For stresses exceeding  $f_2$  (order of 0.01-0.02 kg/cm<sup>2</sup>) many weak contacts in the clay network are disrupted; in the model then the contact D will be broken,  $H_2$  disconnected and the rheological behaviour can sufficiently be described by the basic model. The dashpot N-F describes permanent flow, consisting of Newtonian flow represented by a Newtonian element N and retarded irrecoverable flow represented by a Kelvin element F, non-linear with respect to stress (non-linearity expressed by arrows). The latter component is negligible for fat clays and can be discarded. On the long term the linear Kelvin element K reaches its maximum extensibility (few weeks for fat clays) and the further continuous flow is governed by the dashpot N'. Hence for practical purposes the simple Maxwell model has been obtained (Fig. 78 d). This model can also describe flow non linear with the time,

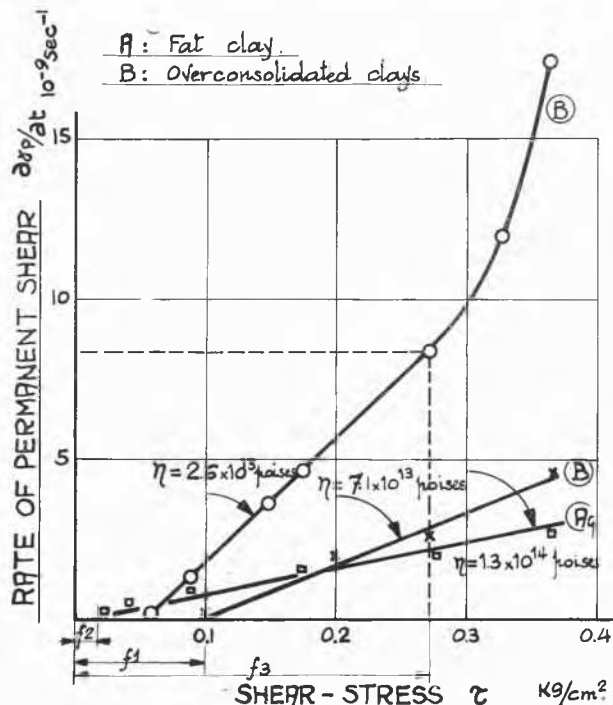


Fig. 76 Bingham flow - curves for clays.

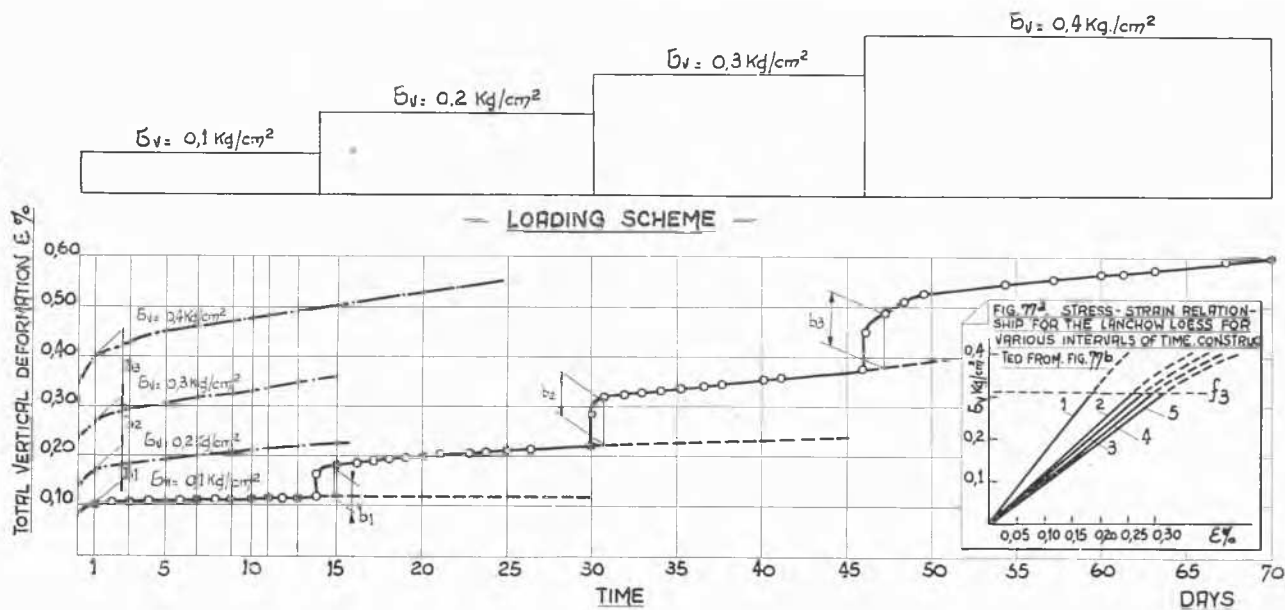


Fig. 77 a Creep tests on the Lanchow loess.

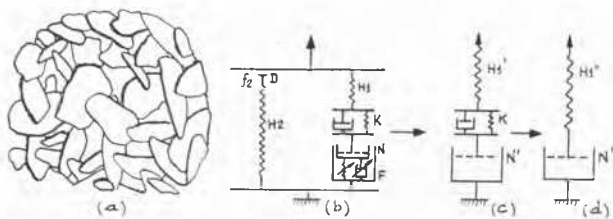


Fig. 78 Schematic structure and rheological models for fat-clays A.

if a suitable operator form or integral equation will be taken; for flow with  $\log t$  the Maxwell operator becomes :

$$\psi = G \cdot \eta \cdot s / (\eta \cdot s - G \log cs), \quad (\text{Tan, 1/63}).$$

For over-consolidated clays (Fig. 79) the sand-silt grains are mutually cemented by amorphous silica, sesqui-oxides, clay-particles in edge-flat surface and flat-flat-surface contacts. Here deformation can only readily be detected, as soon as the local stresses exceeding the lower yield-values  $f_1$  disrupt the cementing agents and surpass the mutual friction between the sand-silt particles. This concept of the yield-value  $f_1$  is described by the friction element  $f_1$  parallelly coupled with the models in Fig. 78.

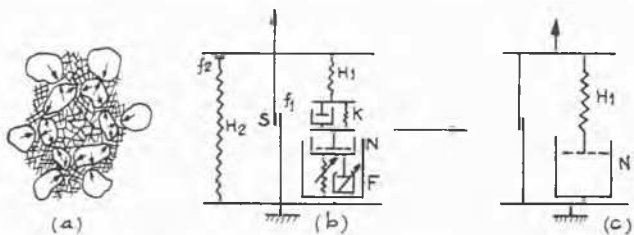


Fig. 79 Schematic structure and rheological models for over-consolidated clays B.

For Lanchow loess the rheological properties are mainly governed by the cementing bridges holding the sand silt particles in a highly porous network (porosity 1.05, water content 8-13 per cent); the main constituents of the cementing agents: soluble salts, illites, montmorillonites, amorphous silica govern the peculiar behaviour of loess towards water. In a certain range of water contents a linear stress-strain relationship has been measured (Fig. 77 b) described by the model in Fig. 80 c.

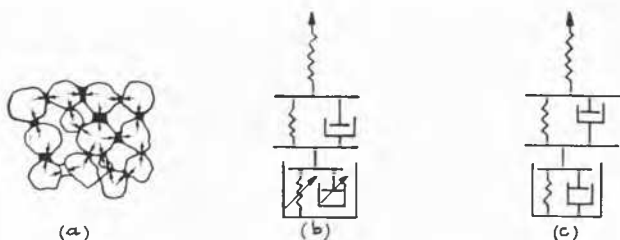


Fig. 80 Schematic structure and models for loess.

ations continue on the long duration; long term settlement observations and creep records for slopes, as far as two types of secondary time-effects and flow show: 1. linear with  $t$ ; and 2. linearly with  $\log t$ . These long term properties can be accounted for by means of a dashpot allowing irrecoverable flow, linear or non-linear, with the time (Figs. 78 and 79). All models allowing continuous flow of course lost their physical significance for  $t = \infty$ , but do hold their validity for  $t$  of the order of the life-time of engineering constructions.

With reference to the very interesting paper by Seed and Chan (1/59) on the vibration characteristics of soils I wish to remark that non-elastic effects in soils can be measured as a damping varying with the frequency and can be interpreted as a complex dynamic modulus  $\lambda = \lambda_1 + i\lambda_2$ . Both  $\lambda_1$  and  $\lambda_2$  are generally functions of the frequency; but for stiff clays (frequency 0-50 Hz) they may be constants. In dynamic problems account can be taken of wave attenuation due to visco-elasticity, by taking the complex modulus  $\lambda$  instead of the usual Youngs modulus  $E$ .

The fact that the shear strength of clays decreases with the time (even 50 per cent and more for some fat clays) casts some doubt on the relative value of routine shear tests and their interpretation. The shearing rate/stress curves for the failure period show a Bingham limit and beyond a non-linear portion become approximately linear (Geuze-Tan 1953) as it also has been found by Vialov-Skibitsky (1/66). For visco-elastic clays and loess I have remarked that for stresses exceeding the upper yield value  $f_3$  accelerating flow occurs as a result of structural disintegrations. As the course of this process is dependent on the stress path it follows that the ultimate state of failure is influenced by the method of testing, as, for instance, the rate of deformation etc. Therefore I am just studying the possibility of whether the upper yield value can give a more objective criterion for limit equilibrium than the usual shear strength. For this purpose the 3 dimensional yield surface should be studied in comparison with the shear strength surface in the  $\sigma_1, \sigma_2, \sigma_3$  space or in the  $\tau, \sigma$  diagram as it has been attempted by Vialov and Skibitsky.

On the basis of test results and theoretical considerations I have devised the following testing procedure to be carried out on a series of identical samples :

a. Isotropic consolidation tests in triaxial cells (all-drained) with measurement of the volume change with the time, every sample being tested under a different constant hydrostatic stress.

b. After completion of the consolidation process undrained creep tests should be performed on the same samples keeping the same hydrostatic stress but in combination with a step-wise deviatoric stress system (Fig. 77 a) with recording of the vertical deformation and the waterpressure with the time. Every loading interval is to be taken so long as to allow a sufficiently accurate measurement of the continuous flow, and the number of steps should be increased until failure occurs. Alternatively, this second part of the tests can be carried out under all-drained conditions, their interpretation, however, being more complicated.

c. Simultaneously long term consolidation tests in the gauge oedometer should be carried out with measurement of the lateral stress with the time.

This testing method enables us to determine the following relationship and rheological quantities : 1. stress strain-relationship with the time (Fig. 77 b constructed from test results in Fig. 77 a by means of the superposition theorem); 2. creep and flow function under shear; 3. coefficients of isotropic and one-dimensional consolidation and from the lateral pressure-time relationship, also the ultimate settlement after the secondary period (Tan 1957); 4. Lower and upper yield values and ultimate shear strength as a function of the hydrostatic stress.

Murayama-Shibata and Folque propose modified three parameter models. In my opinion the cardinal point of interest for practice is to know how the shearing creep deform-

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M. F. TERRACINA (Italie)

## Contribution à une étude systématique des constantes et des variables des sols

Les problèmes géotechniques sont toujours des problèmes de prévision du comportement du sol sous l'influence des charges et des circonstances. Il est par conséquent très utile d'avoir une connaissance claire des données de départ et de distinguer celles qui sont permanentes de celles qui sont susceptibles de variation.

Si l'on néglige les cas exceptionnels de broyage des grains, d'échanges d'ions et de véritables actions chimiques, on peut admettre comme constantes dans un sol les caractéristiques des grains considérés en eux-mêmes, c'est-à-dire la composition minéralogique, le poids spécifique moyen, la forme et les caractères des surfaces, les dimensions (composition granulométrique).

Parmi les variables, il convient de distinguer les caractères variables principaux, suffisants pour caractériser ce qu'on pourrait appeler l'état du sol, soit, essentiellement :

- a) Les rapports entre ses composants (solide, air, eau) qui, en négligeant les variations de poids spécifique, peuvent être exprimés par deux données convenablement choisies parmi les suivantes : indice des vides, compacité, porosité, densité totale, densité sèche, teneur en eau, degré de saturation;
- b) Pour certains sols l'orientation réciproque des grains (type d'agrégat).
- c) Les pressions intergranulaires, la pression hydraulique (neutre), éventuellement les mouvements de l'eau.

La température est, en général, négligeable, sauf pour certains problèmes où elle joue un rôle par son influence sur la viscosité de l'eau et sur la formation de glace.

La prévision de l'état futur, qui dépend des charges et des autres circonstances extérieures, météorologiques, etc., constitue la solution même des problèmes de déformation.

Le comportement mécanique dépendant aussi de l'état et des charges, une connaissance approfondie de ses lois devrait permettre de le déduire a priori; nous sommes bien loin de cela; une partie de ses caractéristiques doivent être essayées ou vérifiées, au moyen d'expériences convenablement choisies et conditionnées. Le coût du prélèvement des échantillons et la difficulté qu'on rencontre pour les avoir réellement intacts amènent cependant à valoriser au maximum les connaissances indirectes.

Le progrès de la géotechnique doit donc aider à déduire le comportement mécanique futur en partant d'un nombre minimum de données qui seront toujours de trois catégories :

1. Caractéristiques du matériau solide;
2. Etat actuel (a, b, c);
3. Prévision des facteurs qui influencent l'état.

Il est important de remarquer que l'on doit considérer, au

même titre que les constantes, les relations (éventuellement les inégalités) caractéristiques d'un matériau solide et dont la détermination est en effet souvent un moyen d'individualisation. Telles les limites d'Atterberg qui, en définissant l'intervalle d'humidité dans lequel le sol conserve une plasticité pratiquement définie, suffisent quelquefois pour caractériser un matériau, mieux que ne le ferait, par exemple, l'analyse granulométrique.

Sur le même plan, on doit considérer les limites inférieures et supérieures de compacité et d'humidité dans lesquelles un sol peut, pratiquement, exister; on peut représenter, ainsi qu'on l'a fait dans la Fig. 81, ces limites dans le plan  $w, e$  (humidité, indice des vides) sur lequel on peut aussi tracer les courbes fermées qui limitent les champs de possibilité d'un matériau; dans la Fig. 81, deux champs sont indiqués, l'un pour un sable, l'autre pour une argile. Nous pensons que l'examen de la forme de ces surfaces est déjà très instructif.

Entre les constantes, dans le sens indiqué, on peut encore comprendre, par exemple, le module de recouvrement élastique (décompression), la densité maximum et l'humidité optimum, résultantes d'un essai de compacité du type Proctor, et aussi le coefficient de portance CBR lorsque la préparation de l'échantillon est faite suivant des règles générales établies à l'avance. Pour chaque type de sol (dans le sens de matériau solide), individualisé par un processus quelconque, on pourrait, à la rigueur, prévoir ces données.

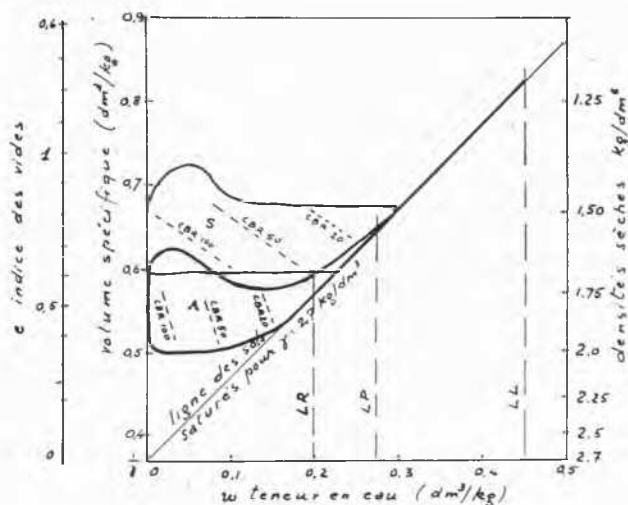


Fig. 81. Champs d'une argile A et d'un sable S. Lignes CBR = 20-50-100.

La cohésion, l'angle de frottement interne, la perméabilité, le CBR sur l'échantillon donné, la compressibilité, etc. sont, en général, nettement influencés par l'état initial de l'échantillon. A propos de la compressibilité, rappelons que lorsqu'elle suit la marche logarithmique habituelle, l'indice de compression est à peu près constant pour un matériau solide (remanié) et, en effet, on a pu établir des relations (grossières il est vrai) entre cet indice et la limite de liquidité.

Dans les limites où chacune de ces variables peut être considérée comme fonction de la seule composition volumétrique du sol, on peut alors établir, pour un sol (matériau solide), une famille de lignes selon la méthode ordinaire de représentation graphique des fonctions de deux variables. Ces lignes, qui ont été tracées dans la Fig. 81 pour le CBR sont, à leur tour, à considérer comme des caractéristiques du matériau solide et doivent ainsi être incluses parmi les propriétés que l'on peut déduire de l'examen d'un échantillon, même altéré, du sol.

Toutes ces caractéristiques, variables dans le temps, changent

souvent aussi dans l'espace qui intéresse une certaine œuvre. Or, pour les problèmes pratiques, c'est cet espace, ce *fond*, qu'on doit considérer, avec ses hétérogénéités horizontales et verticales; naturellement, son étude doit être faite sur la base des variables précédemment énumérées, mais il peut donner lieu à des caractéristiques particulières; quand le fond est homogène, elles sont reliées d'une façon directe et simple aux constantes et aux variables déjà indiquées; mais, en général, elles doivent être conçues comme dépendant d'une œuvre et de charges déterminées : tels le coefficient de réaction (global) d'une fondation, sa capacité portante.

A côté donc des

1. caractéristiques du matériau solide,
2. et des caractéristiques du sol en un état déterminé, on doit considérer
3. les caractéristiques d'un *fond* sous des charges ou, tout au moins, sous un type de charges données.

Les premières sont variables dans l'espace; les secondes dans le temps et dans l'espace; les troisièmes concernent l'espace intéressé dans son ensemble et elles peuvent varier dans le temps. C'est donc seulement d'une façon tout à fait conventionnelle qu'on pourrait les appeler, par exemple, respectivement, constantes, variables et survariables.

Ces remarques peuvent servir à coordonner bien des connaissances; peut-être signifient-elles peu pour les spécialistes; cependant, il est certain qu'il est très utile aux autres de les comprendre et de les assimiler; or, c'est justement des non spécialistes qu'il faut bien se préoccuper, car ils représentent la plupart de ceux qui doivent poser les problèmes, comprendre l'utilité de certaines recherches, apprécier à leur juste valeur les résultats et, enfin, les utiliser. Expliquer les caractéristiques qui ont été énoncées équivalait à éclairer la portée des lois fondamentales de la géotechnique et faciliter la compréhension des grandeurs employées. Ce ne sont pas là des choses de peu d'importance. Dans la phase de formation des sciences principalement, une bonne partie du progrès consiste à établir des indépendance et des types de dépendances : c'est ce qu'on fait aussi en géotechnique.

M. M.K. TERZAGHI (Etats Unis)

#### Discussion of Professor A.W. Skempton's Paper "Horizontal Stresses in an Over-Consolidated Eocene Clay"

In his paper on "Horizontal Stresses in an Over-Consolidated Eocene Clay" our President, Prof. Skempton, provided us with a most valuable contribution to our knowledge of the stress conditions in undisturbed, overconsolidated clays. The essence of his findings is represented by the curves  $K_0$  and  $K_p$  in Fig. 82.

The ordinates indicate depth below the ground surface, the abscissas of  $K_p$  are the values of the coefficient of passive earth pressure, and those of  $K_0$  represent the corresponding values of the ratio between the horizontal and the vertical effective stress in the clay *in situ*. Both curves appear in Prof. Skempton's Fig. 3. The numerical values required for computing the values of  $K_p$  were obtained by means of triaxial compression tests, and the procedure for determining the values of  $K_0$  is described in detail in Prof. Skempton's paper.

If the values of  $K_0$  and  $K_p$  shown in the diagram are reasonably correct the conclusions at which Prof. Skempton arrived can be supplemented by a few others with a more general validity. In order to arrive at these supplementary conclusions I introduced into the diagram the curve labeled  $\epsilon$ , representing the relationship between depth and the vertical strain  $\epsilon$  which the clay has experienced since the time when the removal of the overburden started. This strain was com-

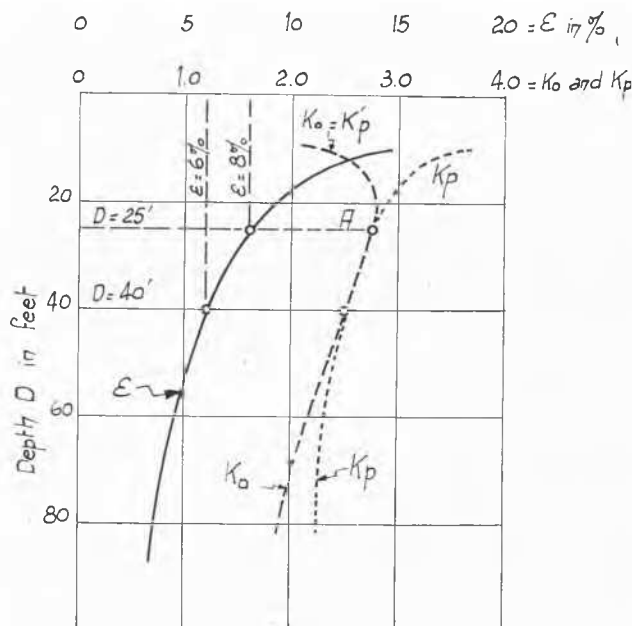


Fig. 82

puted on the assumption that the horizontal dimensions of the clay remained unaltered, while the water content and as a consequence the vertical dimensions of the clay increased. The present values of the water content of the clay at different depths are shown in Fig. 3 of Prof. Skempton's paper. The strain  $\epsilon$  was estimated on the assumption that the water content of the clay prior to the removal of the overburden was equal to the present water content at a depth of 120 ft., which is 27 per cent. In reality it was probably somewhat lower and as a consequence the real strain values were somewhat greater than those shown in the slide.

The data shown in the figure indicates the following relationships :

At depths in excess of 40 ft., the value of  $K_0$  is smaller than  $K_p$ . Therefore below that depth the greatest shearing stress in the clay is smaller than the shearing resistance of the clay. However with decreasing depth, the difference between the two values decreases and at a depth of 40 ft. it becomes equal to zero.

Between depths of 40 and 25 ft.,  $K_0$  and  $K_p$  are equal. This indicates that the maximum shearing stress in the clay is equal to the full shearing resistance of the clay. Within this depth zone the strain increases from 6 per cent to 8 per cent. Yet the shearing resistance of the clay remains unimpaired whereas under laboratory conditions the shearing resistance of a precompressed clay starts to decrease at a very much smaller strain. The difference appears to be due to the slow rate at which the deviator stress increased in the field.

Above a depth of 25 ft. the strain increased from 8 per cent to more than 15 per cent. On account of the brittleness of the clay the increase of the strain to values of more than 8 per cent was in my opinion associated with the formation of fissures. As the strain further increased, the joints opened up, water invaded the joints and the clay disintegrated. As a consequence of this process the clay turned into an aggregate of hard fragments imbedded in a softer matrix, its shearing strength became progressively smaller than that of the clay in an intact state and the corresponding values,  $K'_p$ , of the coefficient of passive earth pressure are smaller than  $K_p$ . Unlike the strain in the clay below a depth of 40 ft., that above a depth of 25 ft. cannot be considered too small to mobilize the full shearing resistance of the clay. Hence, if

my interpretation is correct, the abscissas of curve  $K_0$  above point  $A$  in Fig. 82 would be equal to the coefficient of passive earth pressure  $K'_p$  of the fissured clay. In other words, above point  $A$  the difference between the abscissa of  $K_0$  and  $K_p$  would be exclusively due to the effect of excessive strain on the shearing resistance of the clay and, as a consequence, on the value of the coefficient of passive earth pressure.

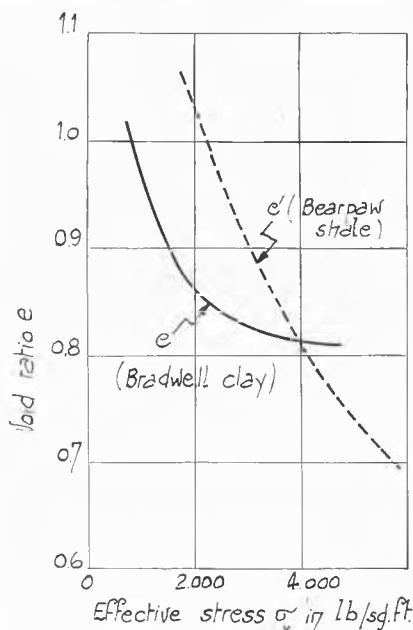


Fig. 83

In Fig. 83 the curve labelled  $e$  represents the relationship between the effective overburden pressure (values of  $\sigma'_v$  in Fig. 3 of Prof. Skempton's paper) and the corresponding void ratios  $e$  for rebound. For the Bearpaw shale in Saskatchewan, which is rather similar to the London clay at the Bradwell site, the same relationship is represented by the dash curve labelled  $e'$ . The striking difference between the curves  $e$  and  $e'$  is due to the fact that the Bradwell clay was precompressed under a load of the order of magnitude of 10 t./sq. ft. and the Bearpaw shale under a load of roughly 100 t./sq. ft. At any depth the strain is roughly proportional to the slope of the curves shown in Fig. 83 at the effective overburden pressure prevailing at that depth.

According to my hypothesis, fissures developed in both the Bradwell clay and the Bearpaw shale, when the strain has reached a value of about 8 per cent. However on account of the difference in the shape of the rebound curves for the two materials, this condition is satisfied in the Bearpaw shale at a very much greater depth below the present ground surface than in the Bradwell clay. Within this depth the structure of the Bearpaw shale is disrupted by a network of slickensided joints, the spacing of which increases with increasing depth. The presence of these joints and the softening effect of the water circulating in the joints reduce the shearing resistance of the shale to a small fraction of that of the intact shale under the same effective load. As a consequence they also reduce the coefficient of passive earth pressure and the maximum value which the horizontal pressure in the jointed zone can assume. Thus the conditions which were encountered at the Bradwell site appear to be a small scale replica of those in the Bearpaw shale.

The fact that the walls of the joints in the Bearpaw shale are slickensided indicates that the deformation of the clay in the jointed zone was associated with a small displacement

along the walls of the joints. That is a condition which cannot be reproduced in the laboratory but I have encountered it in heavily precompressed clays at many other localities. It appears that it is a consequence of the very low rate of deformation. If a heavy structure is erected on a layer of stiff, jointed clay, the joints in the clay surrounding the portion located directly beneath the loaded area open up, water invades the open joints and the clay gets softer. This process may account for the importance of the "secondary time effects" in the settlement of some of the structures which have been built on such clays. It may also have some bearing on the formation of the "valley bulges" on the bottom of valleys carved out of such clays.

M. W.J. THOMPSON (Grande Bretagne)

In their paper "A study of the one-dimensional consolidation test" Leonards and Girault describe how they have managed substantially to reduce side friction in the one-dimensional consolidation test by means of teflon coatings on the consolidometer rings, lubricated with a grease containing molybdenum disulphide.

In my research work into the deformation characteristics of remoulded, saturated Cambridge Gault clay, I am reducing side-friction in the one-dimensional test by enclosing the sample in a tensioned, very thin rubber sleeve on the outside of which a low friction coefficient silicone grease is applied. This arrangement was the suggestion of my research supervisor Mr K.H. Roscoe. By means of a specially-built apparatus it has been possible to measure directly the friction obtained with this arrangement and the results indicate that the friction actually present is of a small order.

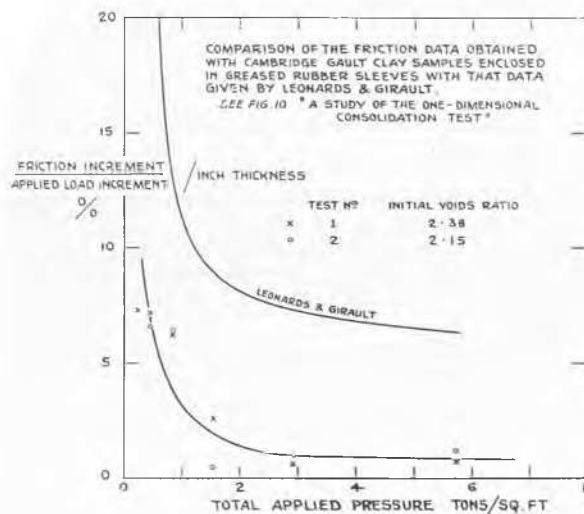


Fig. 84

An alternative and more convenient presentation of the results is offered by the following plot.

Tests in which pore pressures have been measured at two different radii at the base of a sample have shown insignificant variation between the two positions both with and without the presence of the rubber, although where the rubber is present the resultant friction reduction gives rise to correspondingly larger pore pressures.

The dependence of the shape of the settlement-log time curve on load increment ratio and the corresponding dependence of the time rate of secondary compression on total load and load increment ratio as observed by Leonards and Girault for undisturbed Mexico City clay, is also charac-

FRICTION MEASUREMENTS IN ONE-DIMENSIONAL CONSOLIDATION  
TESTS ON SAMPLES ENCLOSED IN GREASED RUBBER SLEEVES  
CLAY TESTED - REMOULDED, SATURATED CAMBRIDGE GAULT.

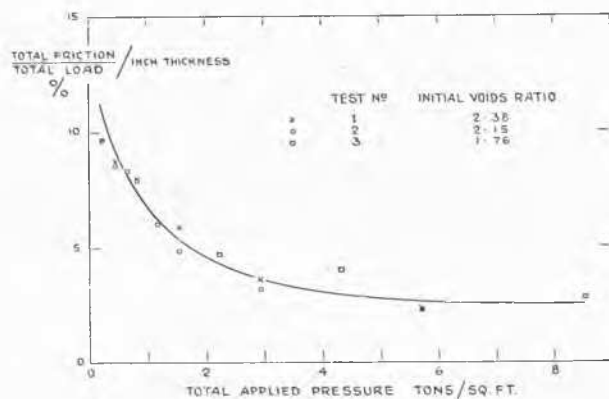


Fig. 85

teristic of the remoulded Cambridge Gault. These dependencies are considered by Leonards and Girault to represent a deterrent to the possibility of representing the consolidation process by rheological models. However, the rheological model is simply a pictorial representation of a stress-strain relationship whose parameters will vary not only with load increment ratio but also with total load, voids ratio, etc. Should the specified stress-strain relationship satisfy the observed results for varying load increment ratio and the other possible imposed conditions then surely it may be considered satisfactorily to represent the one-dimensional consolidation process for the clay under consideration.

M. A.H. TOMS (Grande Bretagne)

The writer is concerned at the disproportionate amount of effort concentrated on academic theories based on hypothetical soil properties or soil properties deduced from short term tests, in comparison with the amount of work which appears to be in progress on endeavours to unravel the true nature of soil bonds and their relation to the long-term strength of soils.

Workers in problems concerning the long-term stability of slopes excavated in natural over-consolidated clay soils have frequently confirmed the writer's opinions, namely that calculations based on shear strengths determined from short-term laboratory test values are often incapable of explaining how failures of such slopes could have occurred. In general, the shear strengths determined from such tests on the soil at any point other than the actual slip zone are far higher than the calculated values for borderline equilibrium.

At the Conference Prof. Skempton stated that, in a slip recently investigated, the moisture content of the soil in the remoulded shear zone was much higher than in the body of the unsheared soil and, also, that in such stiff fissured clays it appeared impossible to predict accurately, from tests on the virgin soil, the behaviour of new cuttings through such clays.

The writer draws attention to the fact that as long ago as 1939 in the, now classic, Sevenoaks Slip and repeatedly since, on the Southern Region of British Railways, it has been shown that, in slips in stiff fissured clays, the only soil encountered which has a shear strength low enough to account for the slipping is the very narrow remoulded slip zone, often only an inch or two thick. The moisture content of this clay

is always much higher than that of the unsheared clay above and below it. The mystery is therefore how it is that clay, which according to normal laboratory tests is far too strong to fail does, in fact, gradually yield and, in doing so, absorb water to soften it down to this ultimate condition. Some twelve years ago the writer concluded that the current theories as to the constancy of soil strength parameters must be in error and tests were commenced, in the British Railways, Southern Region Laboratory, on the behaviour of  $1\frac{1}{2}$  diameter samples of over-consolidated clay at constant overall moisture content when subject to constant sustained axial stresses which were various fractions of the stress required to cause failure at normal rates of strain.

Subsequently, the testing technique was modified to permit drainage from the ends of the specimens which were then subjected to appropriate all-round overburden stress. It was found repeatedly that with London Clay, for instance, failure could occur, after several months' loading at stresses less than half of the "rapid" failure stress. This experience indicates that creep tests appear to be a far more logical approach to the problem of the stability of slopes in stiff fissured clay than do conventional tests based on rapid rates of shear strain.

For this reason and as a result of lengthy experience of the behaviour of old railway cuttings and embankments formed from stiff fissured clays, the writer considers that cutting and bank slopes in such materials can as yet only be designed safely on a judicious combination of case history and comparative soil tests. He is also of the opinion that research into the slow shear deformations of clays under sustained stress is one of the most urgent of all soil mechanics studies needing to be pursued.

At the same time, it seems essential to concentrate all available resources on fundamental studies, using the electron microscope and any other suitable means, to discover the true nature of the so-called "cohesion" and "frictional" bonds between soil particles.

To the writer it seems illogical to expect that if all soil grains are entirely surrounded by water films of one sort or another, there can be either true friction or constancy of cohesion under sustained shear stress. The creep test results seem to support this view.

M. C. VEDER (Italie)

#### An Investigation on the Electrical Phenomena at the Area of Contact between Bentonite Mud and Cohesionless Material

A series of tests was conducted at the Soil Mechanics Laboratory of the Construction Science Department of the Milan Polytechnic for the purpose of investigating various aspects of two particular phenomena: the supporting action of bentonite muds on the face of cohesionless material, and the formation of a bentonite "lining" on the side-walls of cohesionless material when placed in contact with these muds<sup>1</sup>.

At the same time, as these tests were being carried out, investigations were made by the author on the electrical phenomena occurring simultaneously at the area of contact of the various "phases" under observation, namely, the bentonite mud, and the cohesionless material itself (a mixture of sand and gravel was used, according to the curve (b) of Prof. Meardi's report).

#### Description of the test tank

The tests were carried out in a large tank with wooden sides and glass ends through which the phenomena could be

1. See: Prof. G. Meardi, *Support of vertical sides of earth cuts by means of bentonite muds*.

observed. The tank was 1,50 m long, 1 m wide and 0,70 m high.

A glass plate partitioning the tank longitudinally and sliding between two wooden guides could be raised gradually by means of a hand winch bringing into a gradual contact the cohesionless material under examination and the bentonite mud.

Electrode in natural sand Polarity +  
Electrode in bentonite mud Polarity -

Measurements taken between electrodes inserted into the saturated sand and the bentonite mud

At a distance of 1 cm from the bentonite lining<sup>1</sup>.

Time	mV	$\mu A$
1st reading at start .....	32,50	20,00
3 minutes reading .....	10,50	19,00
10 minutes reading .....	9,60	19,80
15 minutes reading .....	9,50	20,00

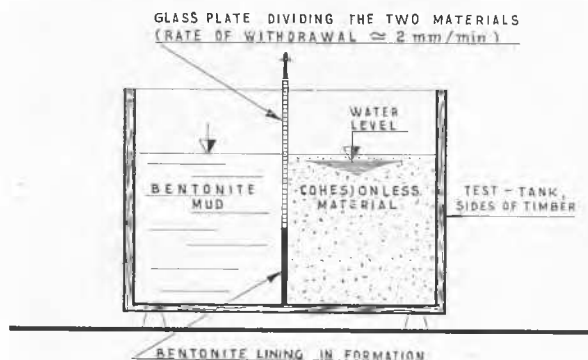


Fig. 86 Schematic section through the test tank.

### Measuring electrodes

For measuring the electric potential (millivolts) and the current (microamperes) different types of electrodes, placed into the bentonite mud and the cohesionless material at different distances from the bentonite lining, were used :

- (a) Small round brass rods 20-30 cm long and 3-5 mm in diameter.
- (b) Brass plates  $170 \times 100 \times 1,5$  mm, or  $80 \times 40 \times 1,5$  mm.
- (c) Fink type (0,4 mm) Filters consisting of a copper rod inserted into a solution saturated with  $Cu SO^4$ . (The Fink Filter is similar to a test tube for chemical purposes, being made of porous material).

### Measuring instruments

For the measurement of the potentials existing between the two phases a moving coil millivoltmeter and a moving coil microamperemeter of S.E.M. Recagni manufacture were used. These instruments measure tensions to an accuracy of 0,5 mV and currents to an accuracy of 0,5 microamperes.

### Measurements of the spontaneous electrical forces

The measurements obtained are shown in the following table together with the values of the electrical forces registered at the time when the currents and the tensions achieved stabilization.



Fig. 88 Measurement of the spontaneous electric potential between the two phases.

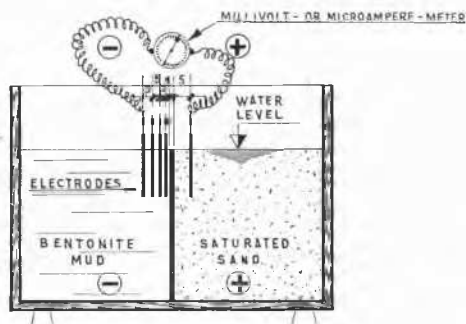


Fig. 87 Arrangement of electrodes for measuring the spontaneous electric potential and electric current.

### Conclusions

(a) There exists a distinct polarity between the two "phases", a negative polarity in the bentonite mud and a positive polarity in the cohesionless material, damp or saturated with water.

(b) By connecting electrically the two "phases" the existence of electric tensions ranging from a few mV up to tens of mV may be observed.

(c) By connecting electrically the two "phases" the existence of an electric current ranging from a few microamperes up to tens of microamperes may be observed.

1. Measurements at distances of 3, 5 and 8 cm gave practically the same results.

(d) When the two "phases" are placed in short-circuit, tensions and currents reach peak values. After about ten minutes, they stabilize themselves at lower values.

(e) Tensions and currents are maintained for periods of at least ten days. Measurements were not made over longer periods because other tests concerning the statics of both "phases" had to be carried out with the same equipment.

(f) When varying the distance from the bentonite lining of the electrodes inserted into the bentonite mud, no significant difference of electrical forces was noted, whereas the difference was greater when the distance of the electrodes in the cohesionless material was varied. Using different types of electrodes the results of the tests were practically the same.

The tests reported above are, therefore, only first steps in a research which may yield many important results by operating, for instance, with different electronic measuring instruments and by varying the parameters of the measurements, such as mud concentration, various chemical additives, origin of the mineral (bentonite), etc.

Parallel to these laboratory tests, in the course of the works for the Milan Underground Railway, field tests were also carried out in order to observe the function of the bentonite lining in upholding the vertical face of the excavation. The bentonite level in the trenches could be lowered to 3,0 m below the top of the excavation without reducing the static equilibrium condition of the vertical face of the dry cohesionless gravel and sand.

M. C. VEDER

**Tests on the introduction of a current by means of electrodes inserted and distributed in the two "phases" (bentonite muds and cohesionless material saturated with water) of similar and opposite polarity to those existing naturally (saturated sand—positive; bentonite mud—negative)**

The factual result obtained, namely the evidence of the existence of electric potentials and currents between the two "phases", as described in the paper Ch. Veder : "An investigation on the electrical phenomena at the area of contact between bentonite mud and cohesionless material", led logically and immediately to the conception of introducing currents into the system formed by the two "phases" of both similar and opposite polarity to those existing spontaneously.

The tests were carried out in the following manner : The test tank was filled to a depth of about 43 cm by successive layers of non-compacted sand completely saturated. Then, by holding the upper 3 cm of the stratum of the cohesionless material in place, a trench 20 cm wide was excavated under bentonite mud at constant level. When the trench (which, as in all the tests, was perfectly vertical) had been completed, electrodes were introduced. A brass plate electrode 1,34 m  $\times$  0,66 m and 0,7 mm thick was inserted into the bentonite mud in the centre of the tank and several brass rod electrodes 50 cm long and 8 mm in diameter were inserted into the sand 5-6 cm away from the bentonite lining. Their disposition is shown clearly in Fig. 89.

The electrodes were connected by 0,8 mm copper wire, and at first natural existing current was measured. Then a direct current was introduced into the circuit through a 4,5 volt battery which was regularly changed every 4-5 hours. In this manner the current flowing was practically constant throughout the tests.

Throughout the three tests the water level in the cohesionless material was maintained at 3 cm below the surface of the material. The bentonite mud was of a uniform density of 1 060 g/dm<sup>3</sup>.



Fig. 89 Introduction of an electric current of natural polarity.

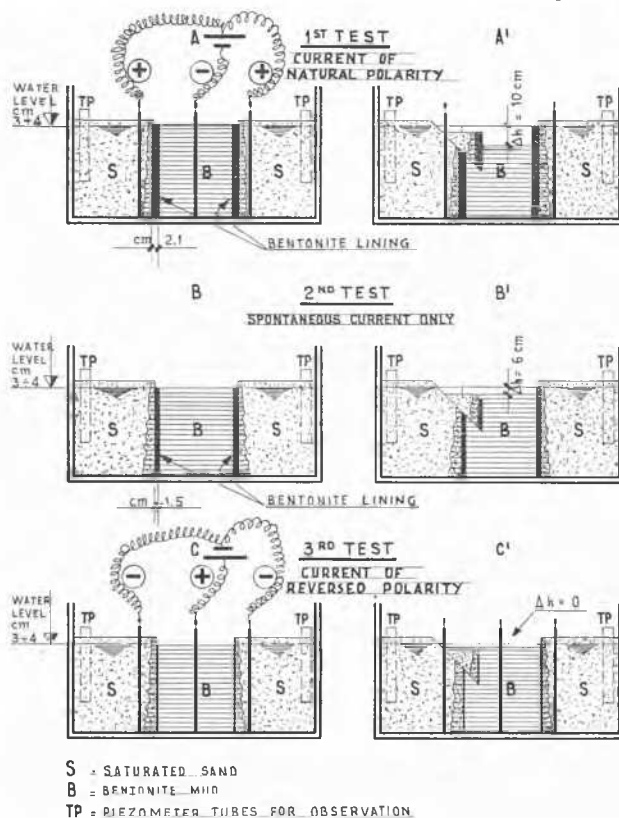


Fig. 90 Tests of the introduction of an electric current between the two phases.



Fig. 91 Thick bentonite-lining formed after introduction of an electric current of natural polarity.



Fig. 92 Collapse of the vertical face after introduction of an electric current of reversed polarity.

To summarise, it was noted that :

In the first case, the current helped in the formation of the lining whose thickness reached 2,1 cm; the wall collapsed when the level of the mud was lowered by  $\Delta h = 10$  cm.

In the second case, when no current was introduced but the natural current, the thickness of the lining was 1,5 cm as normal, and the vertical face collapsed when the level of the mud was lowered by  $\Delta h = 6$  cm.

In the third case, the current gradually weakened the lining, especially in the area close to the electrodes, and in fact caused the face of the cohesionless material to collapse although the level of the bentonite mud had not been lowered, but it led to the formation of a solid lining of bentonite on the central electrode.

### Conclusions

1. The formation of the bentonite lining on the vertical face of cohesionless material can be altered by introducing between the two "phases" a direct electric current. If the polarity is the same as that already existing naturally, this will help the formation of the lining and towards a more favourable condition of the static equilibrium of the vertical face of the soil; if the polarity is opposite, the lining weakens until it is completely disintegrated and causes the collapse of the vertical face of the soil.

2. On this point, it should be noted that the introduction of an electric current in small diameter boreholes has already been adopted in the field of soil exploration in order to obtain variations of the thickness of the lining of the dense drilling muds used for the drilling operations.



Fig. 93 Bentonite lining formed on the plate electrode having positive polarity.

The points recorded are clear evidence of the importance of the formation of the bentonite lining for the static equilibrium when excavating subsoils under bentonite muds. In addition to allowing the mud to exert efficiently its hydrostatic pressure, such a lining constitutes, in the opinion of the author, in itself a membrane with its own resistance, as revealed by triaxial tests reported in another paper. It seems that it holds together the individual grains of the adjacent subsoil also and prevents them from freeing themselves and subsequently sliding.

### On the Dynamics of the Antarctica Ice-cap.

The report deals with the problem of ice-cap dynamic equilibrium. The given solution of the problem proceeds from rheology theory principles and is based upon the data of investigations carried out by the writer at the Soviet Second Antarctic Expedition.

The formation and movement of an ice-cap depend, on one hand, on the existing conditions of supply and consumption of solid precipitation, and, on the other hand, on the physical properties of the ice (the latter depending on its temperature). The basis of the solution in question is assumed to be the following :

(a) The physical relationship (according to Glen and Nye's theory) between the shearing stresses and the rate of ice deformation, which, at low stresses, characterizes a viscous flow, and, at high stresses, a block sliding.

(b) The condition of dynamic equilibrium, setting a close relation between the ice-cap surface configuration and the stresses acting along the ice-cap bed.

(c) The condition of substance balance (the supply of solid precipitation is equal to its consumption), setting a close relation between the accumulation and the configuration of the ice-cap surface and the speed of the ice movement.

As a result, equations have been obtained, determining the ice-cap surface configuration, the stress distribution along its bed and the ice-sheet movement speed versus the accumulation and thermal regime of the ice-cap and the contour of its bed. It has been demonstrated that the surface of ice-caps may be of a most different shape depending on the character of their accumulation and thermal regime.

It is proved that the ice-cap surface configuration of East Antarctica presents a true half-ellipse, the theoretical curve corresponding well with the data of natural observations.

The Greenland ice-cap configuration is also elliptic but its parameters are different, the dome is more gently sloping, which is conditioned by the difference in the character of the solid precipitation distribution and in the thermal regimes of Antarctica and Greenland.

The movement speed of the East Antarctica ice-cap surface has been computed. It changes from the centre towards the periphery according to a law, which is close to hyperbolic. The theoretical calculations show a fair coincidence with the data of natural observations.

M. P.H.D.S. WIKRAMARATNA (Ceylan)

The theoretical correlation of the angle of shearing resistance  $\Phi$  with the intergranular friction angle  $\psi$  given by Dr P. Dantu in Paper 1/10 appears to differ even more from the range of experimental results on practical granular materials than the other well-known correlation given by Caquot (1934), Penman (1953), Bishop (1954), Kjellman (1955), Newland and Alley (1957), as shown in Fig. 94.

Visual observation of the continually varying geometry of the points of contact between the grains along the slip surface within a random two dimensional granular mass has led to a theory (Wikramaratna, 1960) which gives  $\Phi$  in terms of  $\psi$  and a parameter which is dependent upon the shape of the grains.

A paper on the subject is under preparation for publication elsewhere; given below is a bare outline of the theory.

Considering the idealised random two dimensional model of the slip surface built up of equal cylindrical surfaces shown in Fig. 95 it is found that if the packing is dense, the effective transfer of the forces  $N$  will occur as shown in Fig. 96 a (with volume increasing). If the packing is loose two cases

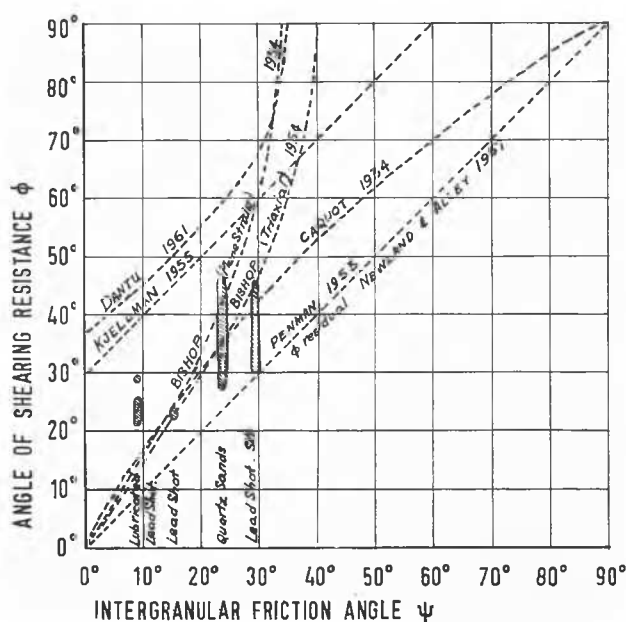


Fig. 94 Correlation between the angle of shearing resistance  $\Phi$  and the intergranular friction angle  $\psi$ , given by Caquot, Penman, Bishop, Kjellman, Newland and Alley and Dantu. Experimental values are taken from publications by various authors including Newland and Alley, Bishop and Penman.

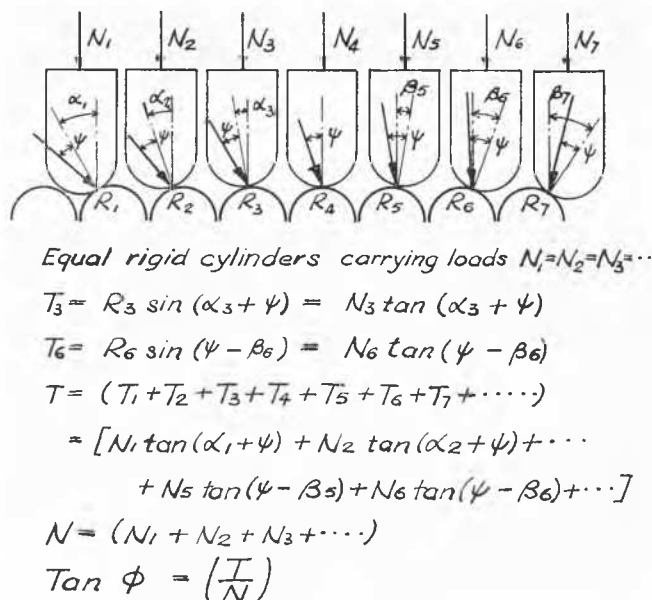


Fig. 95 Idealised random model of the slip surface, built up of equal cylindrical surfaces.

have to be considered as shown in Fig. 96 b. With plausible assumptions for the distribution of reactions  $N$  and limiting sliding friction at all effective contacts two curves shown in Fig. 97 are obtained, thus giving a range of values for  $\Phi$  corresponding to a dense and a loose mass. The amount of interlocking between the grains is accounted for by the variation of the magnitude of the angular limit of the summation from  $30^\circ$  for round grains to  $45^\circ$  for grains with the greatest amount of interlocking (see Fig. 96). Despite the use of a two dimensional model to explain a three dimensional problem, available published experimental data and

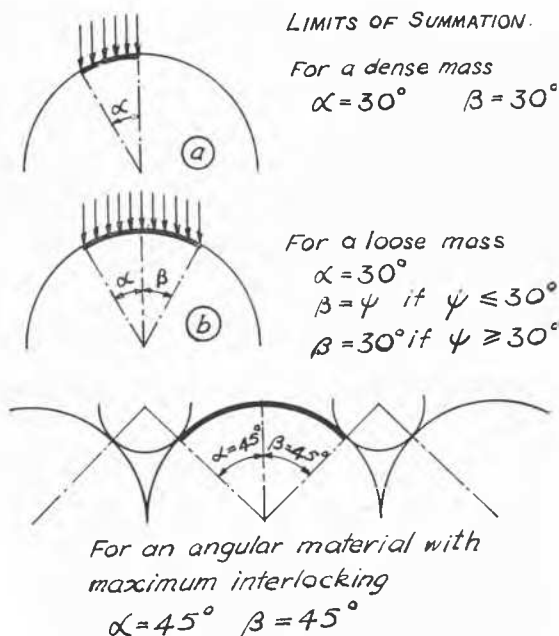


Fig. 96 Limits of summation for dense and loose masses of round material and the modifications necessary to account for angular material.

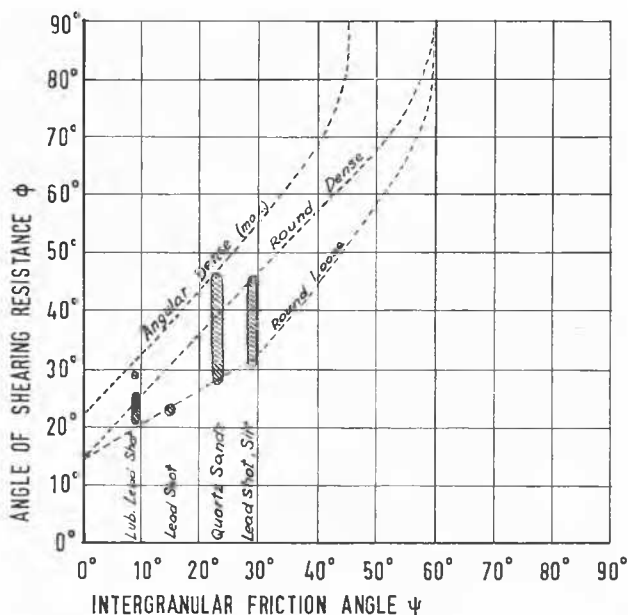


Fig. 97 The resulting correlation between the angle of shearing resistance  $\phi$  and the intergranular friction angle  $\psi$ . Experimental values are the same as in Fig. 94.

these theoretical curves appear to be in reasonable agreement.

The values of  $\phi$  have been derived assuming limiting sliding friction at all effective contacts. However, if the mass is very loose the geometry or architecture of the grains will be highly unstable and the tendency will be for more grains to roll rather than to slide. Hence, the value of  $\phi$  for a very loose mass will tend to be lower than indicated in Fig. 97 for a loose mass. This appears to be a reasonable explanation of the results reported by Dr Bjerrum et al in Papers 1/4 and 1/5 for very loose sands.

With increasing magnitudes of normal stresses effects other than sliding between "rigid" grains, assumed above,

may cause other noticeable effects. Hence any complete theory for the strength of a granular mass has to take into account not only the plastic failure at the points of intergranular contact, but also the effect of the mechanical failure of whole grains by crushing, whether it be by random brittle fracture of an isotropic grain or by failure along a surface of weakness such as a cleavage plane. Resistance to failure may even tend to increase due to modification of the shape of the grains due to plastic flow at points of intergranular contact with materials like lead shot.

In literature on the mechanics of grinding and crushing of materials it has been demonstrated that for a given material the energy expended in crushing is directly proportional to the increase in surface energy which is itself proportional to the increase in total surface area. Therefore it would be profitable to study the variation of the particle size distribution of a granular mass at different stages of failure during a shear test. Such a study may provide clues to a better understanding of conflicting deductions (from tests on what appears to be identical granular material) regarding dilatancy and energy corrections as well as the effect of the intermediate principal stress.

## Références

- [1] WIKRAMARATNA, P.H.D.S. (1960). "The strength of a cohesionless granular mass". *Proc. Ceylon Association for the Advancement of Science*, Part I Sectional Programmes and Abstracts, pp. 26-27.

M. G.S. ZOLOTAREV (U.R.S.S.)

## Engineering Geological Studies of Weathering Crust of Volcanic rocks (illustrated by the Example of Ceylon and other Regions) (1)

Studying the crust of weathering of Archean metamorphous and crystalline rocks in the north-western part of Ceylon as correlated to published data on weathering other rocks makes it possible to draw some conclusions.

In engineering geological studies it is reasonable to part weathering crust into two zones and horizons of weathering, the composition, structure and properties of the rocks being different within the limits of the above zones. Zoning is used when describing the sequence of weathering crust in prospecting holes and exposures by external features of weathering and also when using different methods in studying changes of mineral and petrographic composition based on the data on density and characteristics of weathered rocks.

Weathering of metamorphous and crystalline rocks in tropical zones consists in a multiple complex of various processes lasting longer than those in zones of continental climate, chemical changes of rocks being however more considerable, kaolinites and free aluminium and ferric oxides being formed. Various depths and natures of weathering crust materials at different geomorphological elements are typical of them making it possible to distinguish the horizons and zones of weathering of different ages.

Zoning of the weathering crust in the granite and gneiss in the region of the headworks on the Malwatu Oya River as well as engineering geological studies of the crust result in another evaluation of the rocks in different formations as compared to that recommended in published handbooks. Due to high density ( $\Delta = 2$  g per cu.cm or higher), low seepage rates and high strength of the red-coloured rubble-

1. The materials of the USSR Institute of Giprovdokhoz and Irrigation Department, Ceylon, are used in the report.

clayed formations of "A" horizon they may be a reliable foundation for earth dams up to 20-30 m high. Thus, weathered rocks in the foundations of structures are not to be removed and that part of weathered layers which is not destroyed with shrew tracks or tree roots is to be conserved as much as possible. The availability of a natural waterproof screen makes it unnecessary to grout earth dam foundations for the purpose of preventing seepage through the lower strata of the weathering crust or the cracks below.

Mean and high concrete dams can rest on the rocks of "D" weathering horizon and partially of "C" horizon

providing for grouting those horizons and underlying granite-gneiss to make them stronger (monolithic) and prevent undesirable seepage conditions.

For many centuries the rocks of "A" horizon and partially of the "B" one have been used in Ceylon as natural building materials for bodies of earth dams, dykes, embankments, etc. When disintegrated quartzite is available they are also used as aggregates for concrete. The density of soils in earth dams or dykes is to reach the state corresponding to the density of rubble-clayey formations in the upper strata of the crust of weathering.