Techniques of Field Measurement and Sampling

Méthodes pour Mesures sur Place et Prélèvement d'Enchantillons

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W. H. Ward	*U.K.*
J. Kérisel	*France*
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General Reporter, Division 2 / Rapporteur Général, Division 2

The Chairman

I declare open the third session of the conference, and I ask the General Reporter to introduce the subject of Techniques of Field Measurement and Sampling.

General Reporter

As the Report has already been printed and you all have had an opportunity of seeing it, it would be useless for me to read it, but there are three points which I should like to emphasize.

First, judging from the papers presented to this Division and from the principal publications issued since the last conference, it can be stated that no major development has been made in the various techniques of soil exploration and sampling or in the *in situ* measurements of permeability, water and earth pressure, settlement and deformation. However, the understanding of the convenience of the use of geological, geomorphological, aerial photographs and geophysical reconnaissance methods has been pushed forward and contributed to a better approach to the field investigations for engineering problems.

Secondly, the existence of a large number of different types of devices for measuring penetration resistance of the soil can be considered as harmful for the development of the knowledge of soil properties. In most cases the differences between them are limited to construction details, which are irrelevant as to the practical utilization of penetrometers, but which introduce differences in the obtained values. Such a variety of devices leads to values of penetration resistance which cannot be compared. The same remarks can be applied to the differences in the methods of measuring penetration resistance. It is most desirable that only a few standard procedures and devices should be adopted by the several agencies engaged in soil exploration, so that the results of a field investigation can be properly understood and utilized by any person interested in such determinations.

Thirdly, much attention has been given to the research on the utilization of radioactivity of isotopes and gamma rays in the *in situ* determination of soil densities and moisture content. Results of field investigations are, however, still insufficient for a conclusion on the convenience of the method for current application.

R. Jones (U.K.)

During the discussions held yesterday there were many references to the elastic modulus of soil and often, it seemed to me, the definition of elasticity was ambiguous. A. Dvořák, in Paper 2/3, has drawn a clear distinction between the modulus of deformation obtained from the stress–strain relationship and the dynamic modulus of elasticity derived from seismic tests. The difference between these quantities which A. Dvořák observed on rocks is present to an even greater extent on soil. Indeed, the effect of rate and magnitude of loading on the deformation properties does not appear to have been studied adequately. However, under uni-axial sinusoidal loading experiments we have made at the Road Research Laboratory suggest that for a silty clay soil, ratio between stress and strain is sensibly constant at frequencies greater than 15 c/s and stresses below 10 lb./sq. in.

We are interested in measuring the dynamic shear modulus of soil *in situ* because this quantity influences the stress distribution in a road structure under moving traffic. In soil, the modulus of elasticity often increases with depth, so that the seismic method is not suitable for studying the properties of the top 3 ft. (1 m) of soil in which we are primarily interested.

We have, therefore, developed an experimental technique

Fig. 1

for measuring the velocity of propagation of surface vibrations in soil. The apparatus is shown in Fig. 1. Vibrations are produced by an electrodynamic vibrator resting on the surface of the material under test, and are detected by a vibration pick-up which is moved progressively away from the vibrator. Positions are found which have the same relative phase and the average distance between successive positions is measured to give the wavelength of the vibrations. The product of the wavelength and frequency gives the velocity of propagation: for soil, frequencies between 30 c/s and 300 c/s are normally used. The apparatus is installed in a van and the whole equipment is driven by a small 500 W portable petrol generator.

It is usually found that the velocity is a function of frequency (or preferably wavelength) and the form of the relationship gives an indication of the variation of the dynamic shear modulus with depth. At short wavelengths the velocity becomes sensibly constant and, with a knowledge of its density, enables the shear modulus of the surface soil to be obtained.

Preliminary experiments indicate that where there is definite stratification, the technique can be adapted to give a very good estimate of the thickness of the surface layer.

J. L. Serafim and J. B. Folque (Portugal)

Referring to what is said in the General Report about residual soils and rock and to the field tests presented by A. Dvořák (Paper 2/3), J. B. Folque and I thought it would be of interest to present here the programme of tests under way in Portugal on this subject.

For the construction of Pisões dam, which will be about 2 km long and 100 m maximum height, the most favourable types of structure from the topographical and materials points of view are the concrete multiple arch or massive head buttress types. The foundation exploration indicated altered granites in every degree. The importance of the problem led to an extensive programme of study which included: (a) the identification of the degree of alteration, through either the determination of

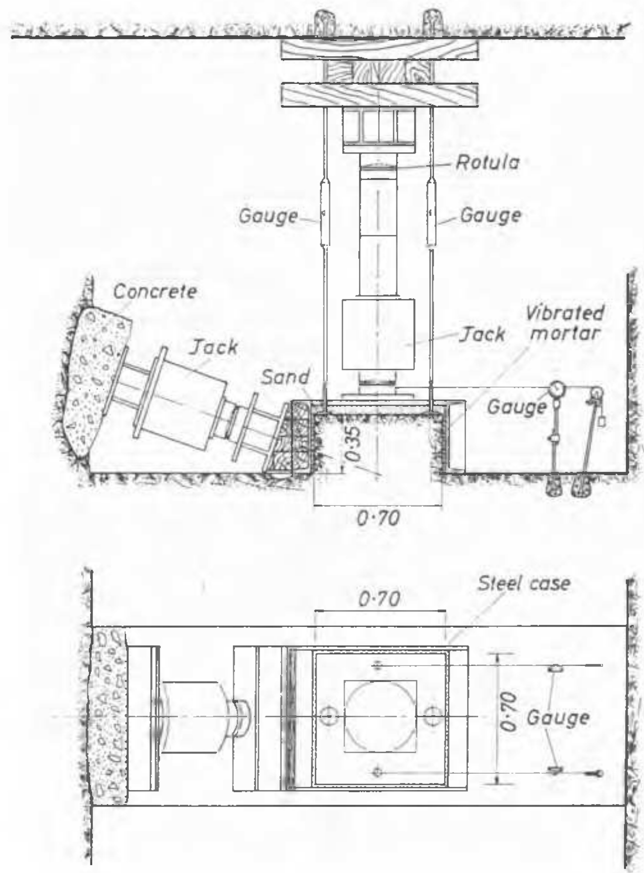


Fig. 2 Loading system for shear test
Système de chargement pour les essais de cisaillement

density or the loss of water of saturated samples, the results obtained indicating that both methods are equivalent; (b) laboratory tests of compressive strength of rock and soil samples, permeability, etc.; and (c) field tests. Such field tests include the determination of the modulus of elasticity in large areas, as described by Rocha, M., Serafim, J. L. and Silveira, A. F. (1955), Deformability of foundation rocks, *Proc. 5th Cong. Large Dams*, Paris, the determination of shear strength in large areas, and the determination of bearing capacity.

Shear tests have been carried out in four trenches, making four or five tests for each trench. The apparatus used is shown in Fig. 3, and is very similar to that which was presented by A. Dvořák. The results of the tests presented in Fig. 3 show not a single straight line but a curve. A point to be noted is the importance of the identification of the rock, because

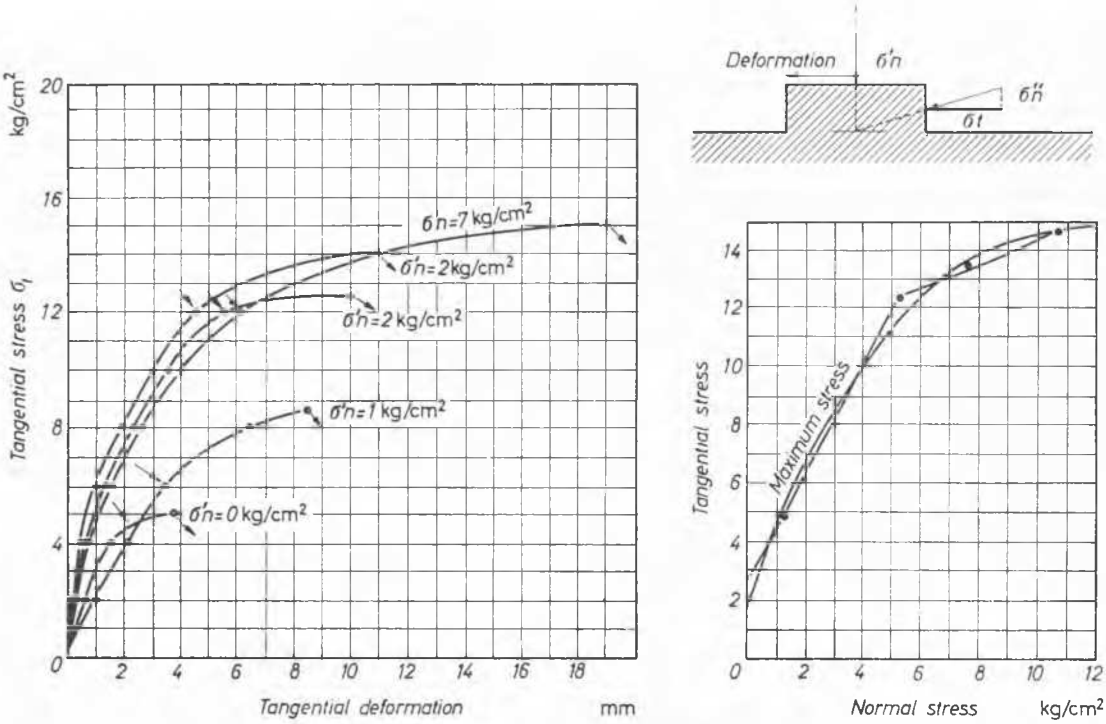


Fig. 3 Results of shear tests in trench No. 3 of Pisões dam
Résultats des essais de cisaillement dans la tranchée No. 3 du barrage de Pisões

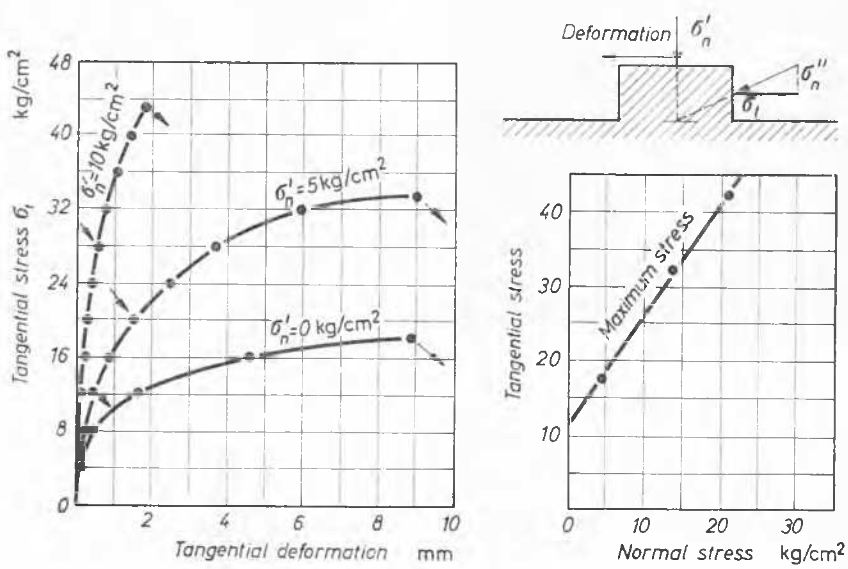


Fig. 4 Results of shear tests perpendicular to schistosity in gallery No. 10 of Miranda dam
Résultats des essais de cisaillement perpendiculairement aux schistes dans la gallerie No. 10 du barrage de Miranda

for each degree of alteration a different shear curve can be obtained.

For the site of another future buttress dam (Miranda dam), shear tests are also being carried out, in this case in galleries. The rock is a schist in various degrees of alteration. The tests are being carried out in three directions: in the direction normal to the schistosity, and in two other directions parallel to the schistosity. Fig. 4 shows the values obtained in this case.

Many other results will be presented when these investigations come to an end.

A conclusion that can be drawn from the studies already carried out in Portugal about *in situ* tests follows. Shear tests, modulus of elasticity tests and bearing capacity tests in altered rocks call for identification tests for determining the degree of alteration.

W. H. WARD (U.K.)

Some years ago we became conscious of the effects of poor sampling on the mechanical properties of soils, now known as sensitive clays. In my view we need to give more attention to the effects of sampling techniques on the properties of stiff clays. I will give you some idea of the effects of normal British sampling techniques on stiff London clay which will, I hope, be

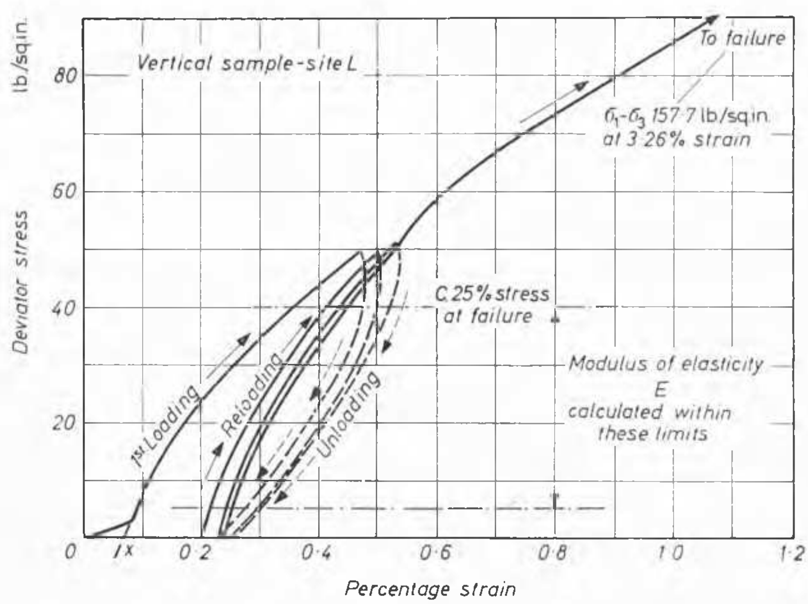


Fig. 5 Typical stress–strain curve showing loading and unloading cycles for calculating *E*
Graphique tension/déformation montrant les cycles de chargement et dechargement pour le calcule de *E*

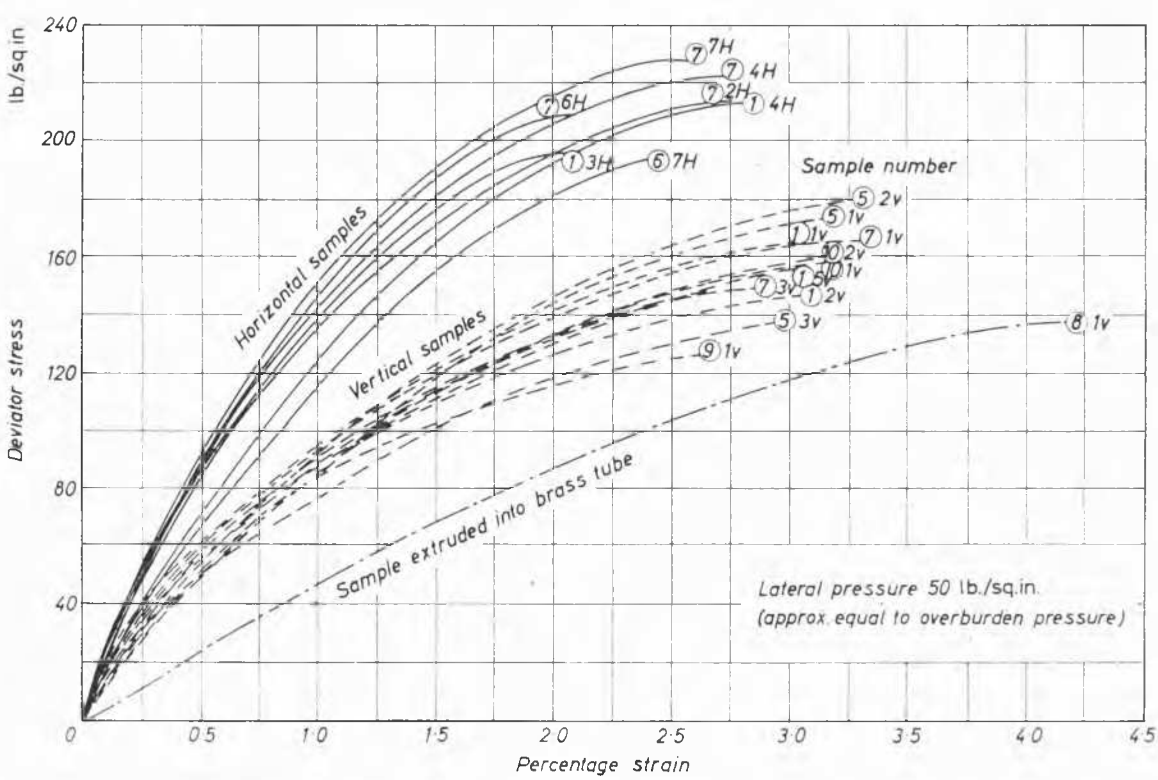
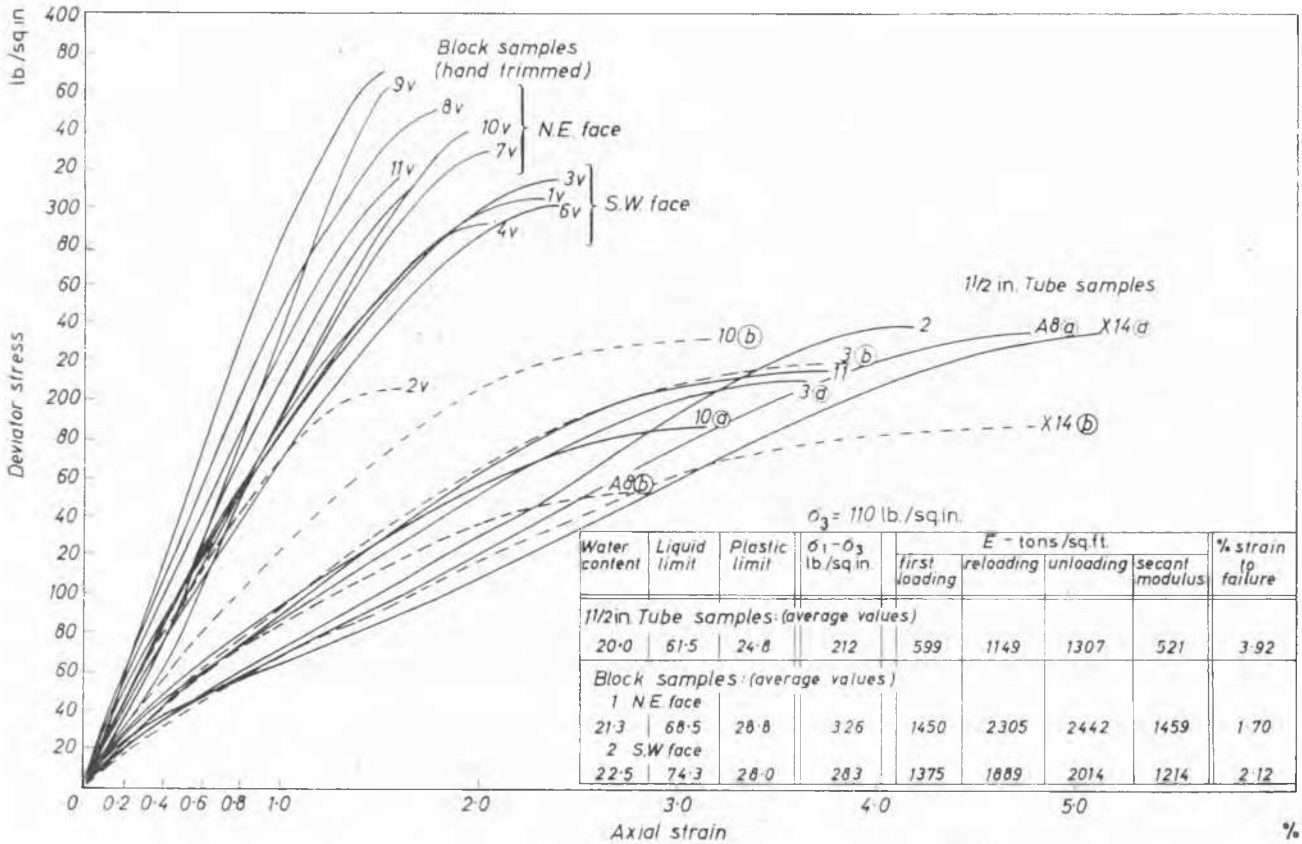


Fig. 6 Stress–strain curves for site L
Graphique tension/déformation pour la position L

of interest to A. W. Skempton and D. J. Henkel in the light of their paper (1a/25) on the properties of London clay and also to A. Dvořák in relation to the remarks in his paper (2/3) about the elasticity of rocks.

In the last few years we at the Building Research Station have been engaged in fairly extensive observations on the linings of tunnels in London, and in conjunction with this work we have been extracting large intact blocks of London clay from the tunnel faces. Cylindrical samples for testing in the triaxial machine have been prepared as carefully as possible from these blocks, using hand saws and shaving knives.

We have been interested mainly in the deformation characteristics of the London clay at very small strains, strains of the same order as those in the tunnel linings, and, for a reason to be explained in a moment, cycles of loading have been applied at small deviator stresses. All samples have been subjected to an all-round pressure approximately equivalent to the total overburden pressure at the site where the samples were



Water content	Liquid limit	Plastic limit	$\sigma_1 - \sigma_3$ lb./sq.in.	E – tons/sq.ft. first loading	reloading	unloading	secant modulus	% strain to failure
1 1/2 in. Tube samples: (average values)								
20·0	61·5	24·8	212	599	1149	1307	521	3·92
Block samples: (average values)								
1 N.E. face								
21·3	68·5	28·8	326	1450	2305	2442	1459	1·70
2 S.W. face								
22·5	74·3	28·0	283	1375	1889	2014	1214	2·12

Fig. 7 Influence of sampling technique and preparation on stress–strain curves of very stiff London clay
Effet de la technique d'echantillonnage et de préparation sur les graphiques tension/déformation d'une argile London clay très rigide

extracted. Fig. 5 gives a plot of deviator stress against percentage strain for a typical sample prepared as described previously, it also indicates the type of loading cycles used. The significant strain differences between the first and subsequent loading cycles will be noticed; Dvořák's comments on this point were interesting. We have regarded this difference as some measure of the effect of extracting the sample from the ground. The earliest portion of the first loading cycle is clearly due to a combination of bedding at the ends of the sampie and a closing up of laminations and fissures, and even the latter portion of the first loading curve does not appear to be entirely representative of *in situ* conditions. The subsequent loading and unloading curves are surprisingly similar and, almost independent of the strength of the clay, we find that the apparent modulus, specified over a small stress range, does not vary much throughout London: the value is about 1000 ton/sq. ft.

The operation, and even the careful extraction of a block from an excavated face, causes the laminations and fissures to open up. Such features may not be particularly visible in the block before it is taken from the face but, within a short while after extraction, the fissures and laminations visibly open; hence the mere operation of extraction causes a significant change in the sample. The insertion of a sampling tube opens up the laminations and fissures even further.

It is also important in sampling to take into account the highly laminated nature of the London clay. Fig. 6 shows the differences between stress–strain curves of groups of samples taken at the same site. In the upper group of curves the deviator stresses were applied in the direction of the bedding, which is generally horizontal in London; in the lower group of curves the deviator stresses were applied normal to the laminations.

We have known for some time that bore hole samples of London clay taken in accordance with the British Standard recommendations give much lower values of compressibility and strength than the block samples we have taken out of the ground, but only quite recently have we had an opportunity of checking this point systematically. In Fig. 6 there is one instance of the effect of taking a standard (1½ in. dia. $\frac{1}{16}$ in. wall, no clearance) small tube sample from one of our blocks; it will be noticed that the tube sample curve breaks away completely from the group of vertical block samples. Fig. 7 demonstrates the sampling effect in greater detail at another site in North London where the depth was 180 ft. below the surface. From two faces of the same tunnel, about half a mile apart, we took a series of block samples and a series of standard (1½ in. dia.) samples on the same day and tested them immediately: the stress–strain curves of *all* samples are given in Fig. 7. The tube and block samples separate out into two distinct groups of curves. The sampling disturbance leads to a very much greater failure strain and the strength is reduced to about 60 per cent.

I believe it is time to take these disturbing effects of sampling in stiff clays into account and also make an effort to improve the sampling techniques.

J. Kérisel (France)

Monsieur le président, messieurs, je voudrais intervenir à propos du module de déformation *E*, d'abord sur une question de terminologie et ensuite sur un appareil de mesure de *E in situ*.

Généralement, en mécanique des sols, on désigne par la lettre *E*, le module statique de déformation plastique qui est une sorte de module de Young généralisé mais qui est, en fait, la pente variable de la tangente à la courbe contraintes-déformations relatives.

E se rapporte à la phase solide dont le resserrement produit la mise en pression de l'eau interstitielle et, pour tous les calculs de consolidation, cette définition est la seule valable.

C'est dire que E ne peut se mesurer sur l'ensemble des deux phases solide et liquide que si l'eau n'est pas en pression; en particulier, si on procède à une expérience de compression simple, on ne peut obtenir que la valeur E non drainée que l'on désigne par E_u.

Cette valeur E_u peut différer assez largement du module E, comme nous le rappellent les expériences de A. C. MEIGH et K. R. EARLY (1a/16) sur la craie.

Mais la valeur de E_u, tout comme les valeurs intermédiaires entre E_u et E, peut présenter un intérêt certain pour diverses applications, comme celle du flambement, par exemple, comme le montre l'article de H. Q. GOLDER et B. O. SKIPP (3b/7).

Qu'il s'agisse de E_u ou de E, il y a intérêt à développer les mesures *in situ* étant donné ce que nous savons sur l'altération possible de E lorsqu'on procède au prélèvement de l'échantillon. W. H. WARD vient de montrér, à cet égard, toute l'altération du module de déformation lorsqu'on procède au carottage d'échantillons de l'argile de Londres.

C'est pourquoi, je crois qu'il n'est pas sans intérêt de vous présenter ici un appareil de mesure de E *in situ* qui est dû à L. MENARD et qui a été mis au point successivement à l'Ecole Nationale des Ponts et Chaussées à Paris et chez le M. PECK aux Etats-Unis.

Il s'agit en fait de la transposition aux sols du procédé d'autofrettage des canons. Nous avons un mandrin central, compris entre deux mandrins de garde soumis tous deux à la même pression. On descend cet engin dans un trou préparé à l'avance, on augmente la pression de la cellule centrale et on trace ainsi le diagramme contrainte radiale-déformations relatives.

Le diagramme d'enregistrement de cet appareil porte en ordonnées les déformations relatives radiales et en abscisses les pressions radiales pour la cellule centrale. Tous ces diagrammes ont l'allure bien connue des diagrammes de butée. Si on applique la théorie de Lamé sur les tubes épais, on en déduit que la pente de ce diagramme par rapport aux déformations est égale à $E/(1+\sigma)$, σ étant le coëfficient de Poisson. De sorte qu'en exploitant la première partie de l'un quelconque de ces diagrammes, et prenant la pente de cette froite à l'origine par rapport aux déformations, on peut calculer E, aux erreurs près sur le module de Poisson.

La deuxième partie de la courbe se rapporte à l'ensemble des équilibres bien connus autour des puits, équilibres concentriques: équilibre plastique distendu et équilibre élastique qui se accorde avec l'équilibre naturel du terrain.

Bien entendu le module ainsi calculé n'est représentatif du module de déformation plastique pour la phase solide que s'il n'y a pas de pression interstitielle appréciable développée dans l'extension du mandrin. Ce sera le cas pour les essais pratiqués sur des sols peu saturés ou présentant un grand Ek, k étant le coëfficient de perméabilité de Darcy. Dans tous les cas, cet appareil présente l'avantage de réduire les possibilités de remainiement du terrain et par conséquent l'altération de E du fait du prélèvement d'échantillons. Avantage qui me concerne pas seulement les argiles sensibles mais aussi les argiles raides.

Nous avons entrepris avec L. Menard une série de comparaisons systématiques des résultats donnés par son appareil et par l'odomèttre.

D. J. PALMER (U.K.)

I should like to add a short postscript to Paper 2/5 by J. G. STUART and myself.

In the paper it is shown that the penetration resistance of the cone penetrometer is of the same order as that of the Raymond penetrometer in sands. The tests presented suggest that the same is true of sandy gravels.

Since the paper was presented a number of tests have been carried out in dense sandy gravel layers at Fawley, sufficiently thick to provide a correlation between the two penetrometers. The results are shown in Fig. 8, together with the results from a bore hole at Tilbury passing through the Thames gravel.

The horizontal axis is the resistance of the Raymond penetrometer and the vertical axis is the cone penetrometer. The points represent the averages of penetration resistance from

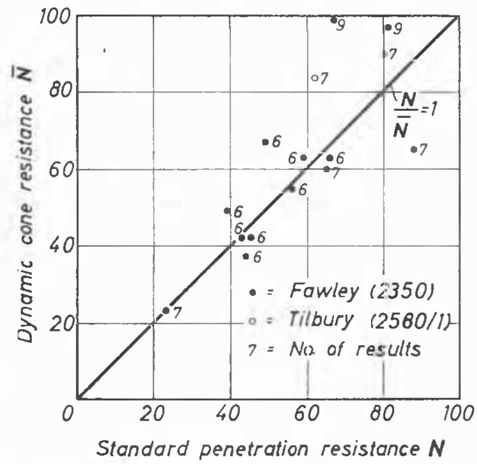


Fig. 8 N versus $\bar{N}$ for sandy gravels. Averages of results for penetrations in sandy gravel of 6 in. and over

N envers $\bar{N}$ pour des graviers sableux. Moyennes des résultats pour des pénétrations de 15·2 cm au plus dans le gravier sableux

each bore hole, and the numbers by them are the number of results from which they were taken.

The overall averages for 94 tests in 14 widely spaced bore holes at Fawley are $N=58{\cdot}5$ (48 tests) and $\bar{N}=63{\cdot}2$ (46 tests). This substantiates our earlier impressions that the penetration resistance for each penetrometer is of the same order in both sands and sandy gravels. The results suggest that the cone penetrometer gives slightly higher penetration resistances than the Raymond penetrometer and generally appear to be in agreement with the work of E. SCHULTZE and H. KNAUSENBERGER described in Paper 2/9.

J. G. Stuart and I would like to express our agreement with the General Reporter's contention that the existence of a great number of penetration test devices is harmful to the development of the knowledge of soil properties. We should like, however, to put forward the claims of the standard penetration test for retention as a soil test well understood by many engineers all over the world; particularly it may be considered a truly standard test in both the U.K. and the U.S.A., besides many other parts of the world. We therefore feel that any recommendations that might be made as a result of this discussion should include the adoption of the standard penetration test as one of the standards.

C. VAN DER VEEN (Netherlands)

I should like to make two comments on the interesting paper (2/9) of E. SCHULTZE and H. KNAUSENBERGER concerning experiments with several types of penetrometers.

The first concerns the remark by the authors at the end of their paper, where it is stated that 'the closest approach to the static structure load is obtained by static penetration tests: however, the performance of these tests according to conventional methods is a rather laborious job . . . etc.' I think that I must disagree with the last conclusion. In the Netherlands for about the last five years types of sounding equipment have been developed both by the Delft Soil Mechanics Laboratory and the Rotterdam and Amsterdam Public Works Offices which are mechanically operated and provided with a very accurate though easily handled measuring apparatus. In this connection, I might also remind you of the sounding equipment of the Swedish Geotechnical Institute, which was demonstrated during the European Stockholm Conference in 1954. These types of equipment are as easily dealt with, in my opinion, as the dynamic penetrometers.

My second remark concerns the deplorable fact that so many types of cones have come into use. The tests of E. Schultze concern nine types of cones. That number could easily have been doubled or even trebled.

It seems to me that the cone penetration test is becoming more and more one of the most important field tests there is. However, the results cannot be evaluated purely theoretically; the cone penetration test is a conventional test in which the measured resistance depends on the dimensions of the cone and the whole procedure of performing the tests, e.g. the speed with which the cone is pushed down into the earth. Therefore, it is of the highest importance—and the General Reporter has pointed this out and I hope I may be allowed to underline it—that standardization of the dimensions and the procedure of the cone test should be effected.

The cone penetration test has been extensively used since about 1930 in the Netherlands, it owed its introduction to the late Professor Buisman, and an effort has been made to arrive at a proposal for a proper standardization, I feel convinced that only an internationally accepted standardization would avoid the confusion and the waste of energy which is caused by the use of so many different and uncomparable types of penetration tests.

I would suggest that the International Society should consider whether it could not see its way to bring forward a recommendation on standardization which might be composed by some appropriate members of the Society.

J. Kolbuszewski (U.K.)

K. H. Head and I would like to make a few remarks concerning the measurements of sand soil and properties in general, and refer here to the following papers: 1a/12, 1a/26, 1b/3, 1b/5, 1b/11, 1b/13, 2/1, 2/2, 2/5, 2/6, 2/8, 2/9, 3a/25, 3a/39 and 3a/42.

Special reference is made to the paper (1a/9) 'Research on Determining the Density of Sands by Spoon Penetration Testing', by H. J. Gibbs and W. G. Holtz, who indicated that the results of so-called standard penetration tests are of no value in many cases unless they are interpreted on the basis of the critical states of sands, that is their loosest and densest packing.

Reading the papers, and our literature in general, one can note that various laboratories use various techniques to determine the degree of relative density of a sand. Voids ratio e, density in lb./cu. ft. γ, or porosity n, are used as the criteria. The choice of the means of description does not matter much as long as its meaning is properly assessed. What matters is the way in which these values were established, especially in the case of the e max and e min, or n max and n min, or γ max and γ min.

We do not get into any trouble when describing the state of a clay in the field. There the techniques involved are standardized. When we deal with sands, the evaluation of the maximum and minimum densities or porosities varies from one investigator to another.

Gibbs and Holtz, for example, on page 36, state: 'The minimum density was found by lightly pouring the dry material into the container of known volume'.

Fig. 9a and b will prove that this statement in itself does not describe the results; what is more, it may be if applied in a laboratory a very misleading technique which will undermine the value of the rest of the research, however well done. When the results obtained are plotted against 'relative density' there is always doubt whether this relative density was measured properly. Light pouring of sand in a dry state will never produce minimum density. The same technique of light pouring with several sands will give for each sand a porosity removed from maximum porosity (minimum density) by varying amounts which will depend not only upon the rate of pouring but also on the more detailed properties of grains, such as their specific gravity, roundness and sphericity.

The maximum porosity of a sand (minimum density) must be measured by depositing this sand in water without entrapped air and the minimum porosity or maximum density by vibration under water (Kolbuszewski, 1950).

No other technique will give reliable results, and it is the authors' opinion that all interpretations of the state of a sand based on techniques different from that above are not accurate or reliable, and cannot be used for generalizations in work on sands.

A series of experiments were carried out in collaboration with P. G. Bossie, B.Sc., A.M.I.C.E., J. A. Bray, B.Sc. and

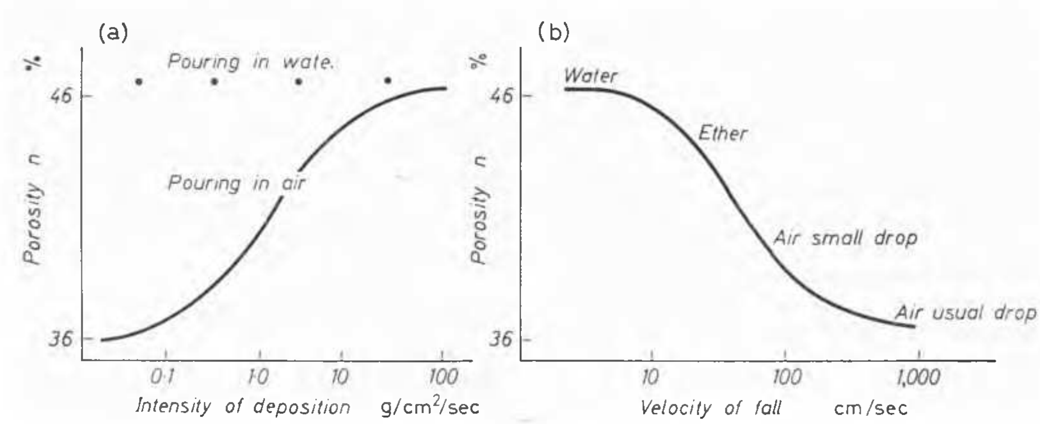


Fig. 9a and b Fundamental factors controlling loose packing of sands, knowledge of which is essential for determining maximum porosity (minimum density) (Kolbuszewski, 1950)

Facteurs fondamentaux gouvernant la condition meuble des sables qu'il faut connaître pour déterminer la porosité maximum (densité minimum) (Kolbuszewski, 1950)

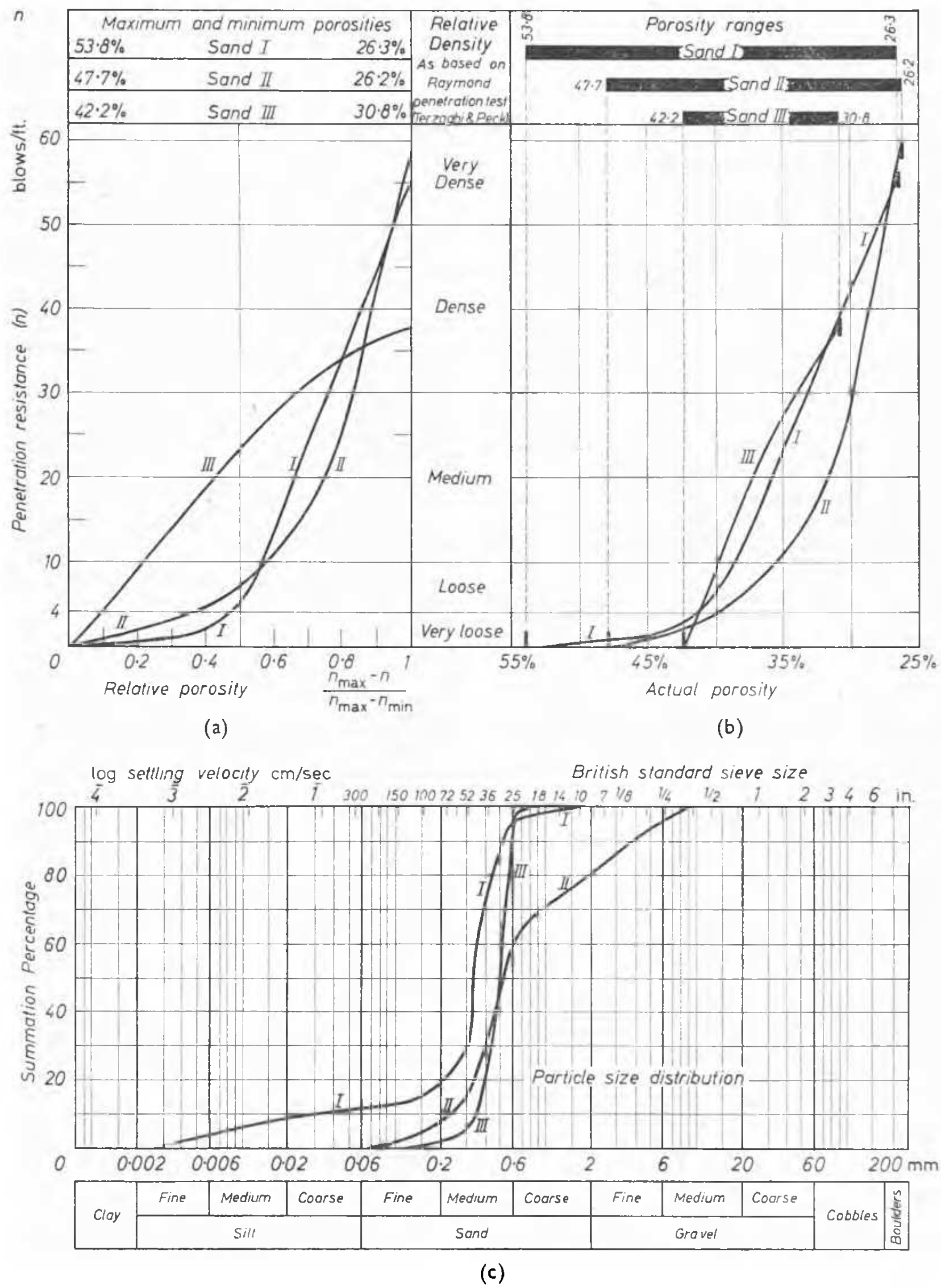


Fig. 10a, b and c Relationship between penetration resistance and porosity for 3 sands
Rapport entre la résistance à la pénétration et la porosité pour 3 sables

G. H. Child, B.Sc., both in the laboratory and in the field. Tests using the standard penetration test (TERZAGHI and PECK, 1948; TSCHEBOTARIOFF, 1951) with sands possessing different values of the relative porosity for the same field porosities (or densities) were carried out.

The values of the limiting porosities were measured in the laboratory, as described above.

Fig. 10a, b and c show the summary of some typical results obtained.

The conclusions derived from the investigation, and from the papers mentioned, are as follows:

(1) The maximum and minimum porosities or densities of sand required for 'relative' values must be measured by techniques described by KOLBUSZEWSKI (1950) until better and more reliable techniques are known.

(2) Results of the so-called standard penetration test and similar techniques must be related to the fundamental properties of materials, and especially to their limiting states of density.

(3) The techniques for evaluation of the loosest and densest states of sands should be internationally standardized to help with interpretation and comparison of data obtained from work on sands in general.

The above general conclusions are in addition to previously gathered data in the laboratory, now supported by field data from which it can be seen that (see Fig. 10a, b and c):

(1) sands I and II show a practically linear relationship between their relative porosities and the number of blows in the standard penetration test for relative porosities greater than 0·5, and the relative density classification as given by Terzaghi and Peck appear to hold good;

(2) below this value the penetration resistance for both these sands is very low;

(3) in the case of the uniform sand III, which has a much narrower range of possible porosities, the penetration resistance is proportional to the relative porosity throughout the porosity range;

(4) it therefore follows that it is not sufficient to estimate the relative density of a sand from the penetration resistance alone without determining other relevant properties;

(5) it can be seen, as an example, that a sand having a penetration resistance of 25 blows per ft. is classified by the standard penetration test as being of medium density.

In our case, sands I and II show relative porosities corresponding to 25 blows of 0·7 to 0·75. For the uniform sand III, however, the relative porosity for the same number of blows is much lower.

On the other hand, at 38 blows per ft. of penetration the opposite is the case—sands I and II are about 0·9 relative porosity, whereas II is at its maximum possible density.

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E. C. W. A. GEUZE (Netherlands)

Referring to the results obtained by some authors who used the Dutch penetration method for different purposes, it seems to me that some confusion still exists about the principles underlying this method.

E. SCHULTZE and H. KNAUSENBERGER (2/9), for instance, refer to a statement by Schubert, that the effect of surcharge in sand layers leads to erroneous conclusions concerning the compaction of the sand. G. PLANTEMA (2/6), on the contrary, has used depth increase as a measure of the porosity and the angle of internal friction, H. J. GIBBS and W. G. HOLTZ (1a/9) found a considerable effect of overburden pressure on the penetration resistance, but their results were obtained by a dynamic method, no direct comparison with a static method being possible. J. Kolbuszewski has already discussed that end of the problem.

Theoretical proof of the fact that the resistance to penetration inside the sand layer depends on the initial state of stress was given by G. de Josselin de Jong at the Paris Conference on the bearing capacity of piles; therefore the result of a penetration test never can produce any positive information concerning the porosity of the sand as long as a certain value of this physical quantity is compatible with wide differing stress ratios.

I applied the same method as that referred to by G. PLANTEMA in his paper (2/6) to this conference a couple of years ago, in order to establish a relationship between the density of sand and the increase of penetration resistance versus depth ratio. This investigation resulted from a discussion with Mr. Golder of Soil Mechanics Ltd., when the point was raised in connection with his intention to investigate the variations of the shearing properties of a hydraulic fill of rather coarse-grained material by an *in situ* method.

The laboratory tests were carried out with the help of a model cone penetration device at one-fifth of the size of the Dutch penetrometer cone used in the fill itself. The coarse-grained material was reduced to the same scale, maintaining the same grading curve and smoothness of the grains. Under these conditions the increase of resistance versus depth ratio should inherently be the same in the model material as in the fill, provided their respective densities and initial states of stress were the same.

In the case of a hydraulic fill of sufficient depth below the water table, the resulting state of stress in deposits of great extent will be governed by gravity, the viscosity of the water and the size and grading of the grain material. If this procedure of sedimentation is reproduced with sufficient accuracy in the laboratory model test, similar increase of resistance versus depth ratios should be found.

If, however, forces of another nature, like pre-loads, vibrations or seepage pressures are subsequently used to obtain higher compaction, the resulting stress ratio may differ considerably from the at-rest state of stress as produced by the aforementioned process of sedimentation.

Proof of the fact that higher stress ratios at a given density produce marked increases of resistance versus depth ratios is shown by nature itself, for instance, in perfectly homogeneous deposits of dune sand along the Dutch coast. Alternating layers of different geological history may be recognized from the pattern of cone penetration diagrams. The effects of high residual, lateral effective stresses in pre-loaded layers as found by extremely high values of the resistance versus depth ratios are in good agreement with the high stress ratios resulting from plate loading tests on pre-loaded surfaces in this area, as found by Keverling Buisman in 1927. In this way I have found penetration diagrams a real help in the study of the different phases building up this geological formation.

These facts in themselves should warn us that the use of the indirect method of assessing the density of a sand deposit is limited to the case of a material obtained by some natural, conservative procedure, so that the resulting state of stress will vary within fairly narrow, well-defined limits. Therefore, it cannot produce a measure of the density if compaction is obtained by artificial means, resulting in an unknown residual state of stress.

The question arises whether we are interested in the density of a sand layer when dealing with practical problems in soil mechanics: certainly not in the physical magnitude as such, but as one of the factors governing the mechanical properties of the layer. We certainly have to go a long way round in order to measure the different factors separately by suitable methods; an interesting subject for my collaborators and myself in the field of research. However, practice has asked and is still asking for *in situ* methods by which the mechanical properties of soil formations can be assessed in a direct and economical way, even if this would be obtained at a certain loss of scientific value. The fact that the Dutch penetration method, though limited to a certain range of practically important soil formations, is being used by an increasing number of workers in the field of soil mechanics is a source of satisfaction to us. It also puts us under an obligation to develop this method into something more than a mere exploration tool, though we are well aware that the simplicity of the underlying principle puts a natural limit to our possibilities, but then the same may be said of any other simple *in situ* method.

H. ZWECK (Germany)

I should like to give you some information about the work we are doing in Germany in connection with the first point to be discussed. The German Society of Soil Mechanics and

Foundation Engineering has set up a working team called 'Sonden' (soundings), on the initiative of E. Lohmeyer. Among other tasks, the team is working on the standardization of equipment for static and dynamic penetration tests; the reason being that several types of sounding apparatus are in use and, as the General Reporter has stated, comparison of the results obtained by them is often rather difficult.

The team is about to define the dimensions of a light and of a heavy dynamic sounding apparatus. The hammer weight of the light one is 10 kg, that of the heavy one 50 kg: in both cases the hammer-drop is 50 cm. For dynamic sounding from the bore hole the standard penetration test with 2 in. outside diameter sampling spoon is to be recommended. A static sounding apparatus is also included in this programme. I expect that the discussion of our committee will soon come to a close and that the results will be published in 1958.

Coming to point 3 posed for discussion, I should like to report on our tests which are being carried out in the laboratory of the Bundesanstalt für Wasserbau. These tests are expected to help in establishing a relationship between penetration resistance of static and dynamic sounding and the relative density of different kinds of sand down to a depth of about 3 m. In these experiments the influence of the water content on the penetration resistance is especially to be investigated, and we are therefore performing these experiments on moist, saturated and dry sands. The tests are carried out in a circular pit which has a diameter of 2 m and depth of 3 m.

As G. Plantema states in his paper (2/6), we also found an increase of penetration resistance with increasing depth, and observed this relationship in both static and dynamic sounding tests. Furthermore, we noticed that the increase of the resistance stopped at a certain depth and remained practically constant further down to a depth of 3 m, this being the limit of our observations. This depth from which the resistance remained constant, and which I shall term now the critical depth, depends on the relative density, on the kind and the grading of the sand, and on the dimensions of the sounding apparatus. It was found that the critical depth varied between a few cm and 1 m. It was always smaller for the lower than for the higher densities.

We also observed that the increase in penetration resistance in the upper part depends, as G. Plantema found, greatly on the moisture content. The same dependence was noticed on the magnitude of the constant penetration resistance below the critical depth. It was always smaller in saturated sand than in moist or dry sand.

When the sounding was performed above the ground water level, the influence of the saturated sand could already be noticed before the point of the penetration apparatus reached the water table.

M. P. P. dos Santos (Mozambique)

In our laboratory in Laurenzo Marques, Portuguese East Africa, we have been using the radioactive method for determining the moisture content and density of soils for a long time. As we have thus accumulated a certain amount of experience, under somewhat difficult tropical conditions, I am quite sure that a brief comment on the papers presented to this conference on the subject may help to clarify a few aspects of the method.

My first remarks will deal with the important matter of calibration, which, to a certain extent, closely controls the accuracy of determinations. To emphasize this point, I would say that quite different results may be obtained according to the diameter of the calibration drums, and the distance between the source and the scaler. Careful planning is needed if one wants to get reliable results; also, the G.M. counters must be frequently and carefully calibrated and for that purpose it is necessary to have at one's disposal an additional source of neutrons, with long half-life.

With regard to field operations, on various types of apparatus, like the one that we are working with, the slow neutrons are detected by the induced radioactivity on sheets of rhodium. The sensitivity of the method will be sharply modified on account of the time factor, particularly if saturation has not been reached; besides that, the residual radioactivity of rhodium plays an important part in disturbing the accuracy of successive measurements. However, the use of modern tubes, filled with BF_3 gas, is expected to improve materially the present unsatisfactory situation.

In summing up, I should like to stress the point that in my opinion this method cannot at present be considered as sufficiently reliable for field routine operations.

B. O. Skipp (U.K.)

I wish to make some comments on Paper 2/2.

As the General Reporter remarked in his Report on Division 1, there is nothing new about the relationship between gamma ray adsorption and density: the technique is, however, novel to engineers and seems most promising. In order to render it practicable it is necessary to improve its accuracy, reduce the cost and bulk of the scaler unit, and ensure safe operation.

In Paper 2/2, Durante, Kogan, Ferronsky and Nosal reproduce one curve, Fig. 5, showing an experimental relationship between $\log_e I_0/I$ and the density in g/cm³ in which there appears to be possible error of about 4 per cent in density.

We are at this moment calibrating a similar device against granular materials in both a saturated and dry condition and it has been possible to reduce the scatter of densities to about $\pm 2{\cdot}5$ per cent. Even so, we are not satisfied that the sands and gravels against which we are doing the calibration are uniformly placed throughout the bulk of 2 cu. ft.

The Compton back scatter method as described by Goldberg, Trescony, Campbell and Whyte (see Refs. Vol. 2, p. 432) has been calibrated both in the laboratory and the field to within $\pm 1{\cdot}5$ per cent. This device can be lowered down a sounding tube and is really a logging instrument.

I would like to ask the authors of Paper 2/2 some questions. First, why was the collimation method impracticable for geotechnical investigations; secondly, how did they measure the densities in their Fig. 5; and thirdly, have they noted any difference in the slope of the curve $\log_e I_0/I$ versus density between dry and saturated materials?

J. Vourinen (Finland)

In connection with the two previous conferences on Soil Mechanics and Foundation Engineering, G. Plantema has reported on the design and operation of an electrical pore water pressure cell, in which wire resistance strain gauges have been employed for measuring the pressure against a deflecting membrane. In view of the existing interest in electrical measuring devices for pore water pressure it is felt justified to give here a preliminary description of a pressure measuring cell, based on electrical principles of another kind.

This cell is still in the experimental stage and therefore its characteristics and possibilities are not yet fully evaluated. As G. Plantema has in his reports very thoroughly dealt with the general requirements for a pressure cell, this contribution is restricted to the actual measuring principle only.

Fig. 11*a* shows a section through the pressure cell. In this cell the water pressure deflects a membrane, which has been chosen from ordinary pressure gauge membranes of commercial manufacture. To measure the deflection of the

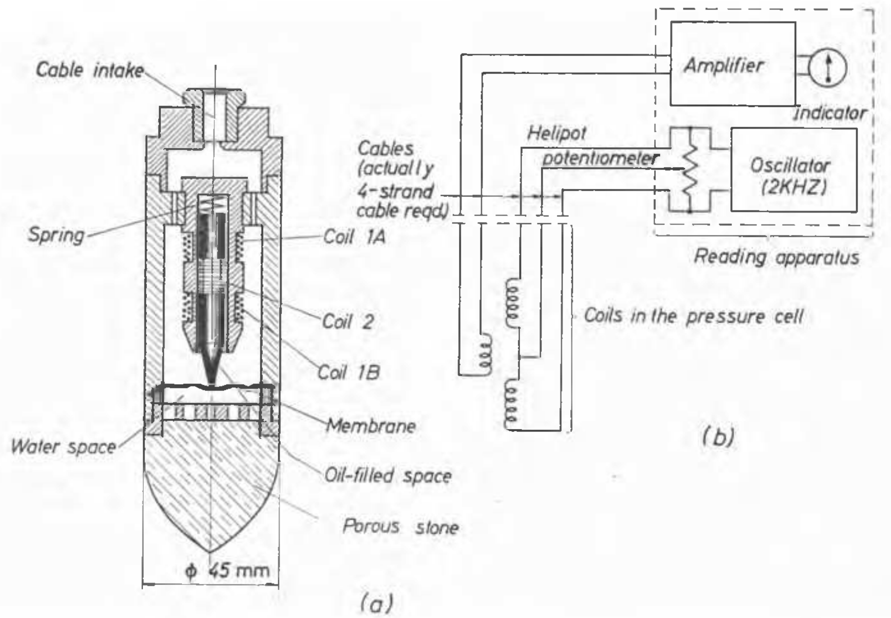

Fig. 11 Electrical pore water pressure cell
Cellule électrique pour mesurer la pression interstitielle

membrane, and thus the pressure, an inductance coil system is used which consists of two fixed coils (coils *1A* and *1B*) and one movable coil (coil *2*), the stem of which is lightly pressed against the membrane by a spring. When the water pressure actuates the membrane, the movable coil strictly follows the deflection of the membrane, and thus the mutual position and also the mutual inductance between the coils is changed. This change of position is measured, using a reading apparatus.

The arrangement of the reading apparatus is shown in principle by Fig. 11*b*. The fixed coils are connected in parallel with a variable resistance (a Helipot potentiometer) and this bridge system is fed by an oscillator. The coils *1A* and *1B* induce a voltage in coil *2*. By adjusting the potentiometer the electrical system can be brought into such a balance that the induced current in the coil *2* is at a minimum. This in turn is shown by the indicator (a micro-ammeter) which is connected to coil *2* by means of an amplifier. In such a way the position of coil *2* and thus each degree of deflection of the membrane is indicated by a corresponding potentiometer scale reading. The present reading apparatus is battery operated and has a weight of 10 kg in total so it is easily transported to the various measuring points.

The pressure cells now fabricated have a measuring range from zero to 30 m of water head. The deflection of the membrane at a pressure of one atm is 0·14 mm, corresponding to a change of 40 mm^3 in the water volume. The reading is possible with an accuracy of $\pm$ 10 cm (0·01 atm) over the whole range. The prototypes have now been in continuous use for about one year without showing any actual defects. As it is evidently possible to improve the reading accuracy a stiffer membrane may be used and so the time of response shortened. It is naturally necessary to calibrate each pressure cell individually for determining the correlation between the pressure and the potentiometer reading.

It is considered that the following points are of interest.

(1) The choice of the membrane is limited only by the dimensions of the cell so that, for instance, ordinary corrugated pressure gauge membranes can be used, to which wire resistance strain gauges could not be cemented. Different membranes may be chosen to correspond to the individual requirements of the different measuring ranges.

(2) The requirements with regard to the insulation resistance of the electrical parts in the cell are lower than when using strain gauges.

(3) In spite of the fact that electronic reading apparatus is used, the operation of the apparatus is extremely simple and the weight and cost of it are very reasonable.

As already mentioned, both the pressure cells and the reading apparatus now described are still in the state of development, and it is felt that both of them can be improved so as to increase their accuracy and reliability in operation. It is intended to prepare a more comprehensive report on this measuring system later, when additional experience in its practical use becomes available. The present cells and apparatus have been designed and fabricated by the Office of Soil Mechanics and Concrete Technology of the Oulujoki Power Company, in Finland. Acknowledgement is due to Mr H. Laine, Research Physicist of the same office, for his work and co-operation in the development of the meter in question.

I. M. Litvinov (U.S.S.R.)

In the Ukrainian building research institute YOUZHNII there has been developed a set of portable equipment 'Type-9' which I introduced for field geotechnical investigation. The equipment is contained in three cases weighing 20 to 25 lb. each is operated by one man.

This equipment replaces completely the complicated and bulky stationary field laboratory and permits all the determinations required by the codes to be made directly on the site, quickly and with a high degree of accuracy.

Here are some examples of use of this equipment:

(a) Selection of the undisturbed soil monoliths.

(b) Determination of various soil characteristics: volume, weight, moisture content, porosity, density, filtration, plasticity, maximum molecular moisture capacity, granulometricity, angles of the slope of repose, swelling, relative modulus of pressure, structural cohesion, shearing cohesion of clayey and silty soils, etc.

(c) Determination of the sedimental properties of the soils, the coefficients of sediment and the size of the possible sediments.

(d) Determination of the compressive properties by means of usual or accelerated method.

(e) Field investigation of the shear properties of the soil.

(f) Determination of the critical soil pressure for the existing codes and technical requirements.

(g) Calculation of the possible settlement of the footings of various buildings.

In addition, this equipment makes it possible to control the quality of different earthworks: construction of soil dams, embankments, dikes, canals, etc.

The high accuracy achieved by this equipment is due to its special design and to the fact that all the determinations are carried out immediately on the site.

The usual error on repeated measurements carried out by generally accepted methods varies between 2 and 10 per cent, whereas when using equipment 'Type-9' this is reduced to between 0·5 and 1 per cent.

The time required for taking an undisturbed sample and making the basic determinations has been reduced to 1/10 or 1/15 of that required for other generally used methods and apparatus.

The drying out of soil samples is carried out in a collapsible cupboard of special design weighing only 3 lb. which allows 50 samples to be dried at the same time with automatic temperature regulation.

The apparatus for shear tests which is included in the set is particularly accurate because of the absence of friction in the apparatus at the metal shear planes. At the same time any possibility of wedge action by particles of sandy soil or of squeezing out of saturated cohesive soils which takes place in other types of equipment is here avoided.

More than 4,000 sets of this equipment have been manufactured in Kharkov in the U.S.S.R. They are being used with success for soil investigation in housing, civil engineering, industry and road construction, etc., both in the U.S.S.R. and in many other countries.

This equipment has been considered to be very important by many scientific and productive organizations for permitting the costs of construction and prospecting to be considerably reduced.

The detailed description of this equipment for the field laboratory 'Type-9' and the method of its operating may be taken from I. M. Litvinov's book *Soil Investigation in the Field* published in 1954.

All information concerning the equipment may be obtained from the Academy of Construction and Architecture of the Ukrainian S.S.R. (Vladimirskaya 24, Kiev, U.S.S.R.).

T. K. E. Kallstenius (Sweden)

The General Reporter has expressed the opinion that nothing of basic interest has been achieved in, among other things, the development of equipment and technique for undisturbed sampling and for the *in situ* measurement of pore water and earth pressures since 1948, when Hvorslev's book appeared. He therefore suggests that the discussion should be concentrated on penetration tests.

The Swedish Geotechnical Institute, of which I am a member, has paid much attention to the above questions and has made progress which we feel to be of a basic nature. The results of this work have not been submitted to this conference as they appear in the Institute's Proceedings, Nos. 10, 12 and 13, which have been published quite recently, and in a paper on undisturbed sampling, which has been published in Swedish and will shortly be available in English. I will thus give here only a brief outline.

As regards soil exploration and sampling, Fig. 12 shows the relationship between the angle of the cutting edge of piston samplers and the unconfined compression strengths. Several types of piston sampler were tested on two sites. We found the angle of the cutting edge to be the most important single influence on sample quality, and we think that this angle may replace Hvorslev's concept 'Area Ratio'. The demands on small angles are, however, often more rigorous than was recommended by Hvorslev.

When sampling clay at depths below 30 to 50 ft. we found it very useful to inject heavy fluids such as trichlorethylene or carbontetrachloride between the sample and the sampler wall. This reduces inside friction and adhesion as the sample is kept floating in the liquid.

The steel foil sampler used in combination with rotary boring has proved to give almost undisturbed samples even in sand and fine gravel. For depths down to 30 to 50 ft. the procedure is rapid and cheap; for greater depths costs increase.

On the subject of *in situ* measurement of water and earth pressure, our Proceedings No. 13 contains basic considerations and a description of a pore-pressure measuring system of a special kind intended for temporary installations. Here great efforts have been made to decrease time lags and make the reading simple and free from manipulations. Since the publication was issued we have carried out two big installations of a more permanent kind, comprising about 100 measuring points. Fig. 13 shows our most recent system of interchangeable parts in which different types of filters can be combined with different measuring pick-ups to suit different conditions. Pick-ups are not damaged during installation and can easily be calibrated *in situ* or exchanged for others better suited to actual conditions, thanks to a detachable connection close to the filter.

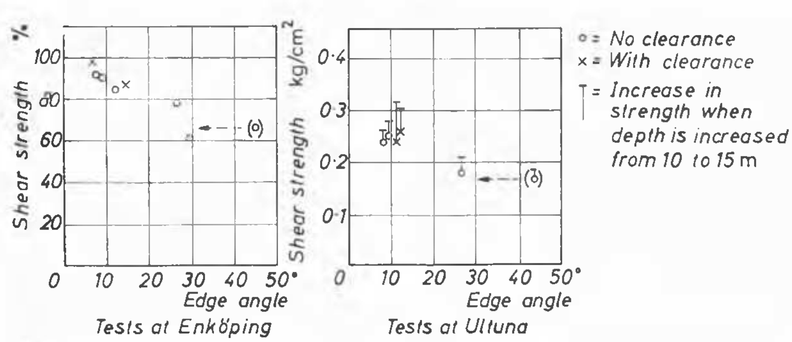

Fig. 12 Different piston samplers—unconfined compression test: relationship between edge angle and sample quality

Différents instruments d'échantillonnage à piston—essai de compression sans contrainte latérale; rapport entre l'angle du bord de l'instrument et la qualité de l'échantillon

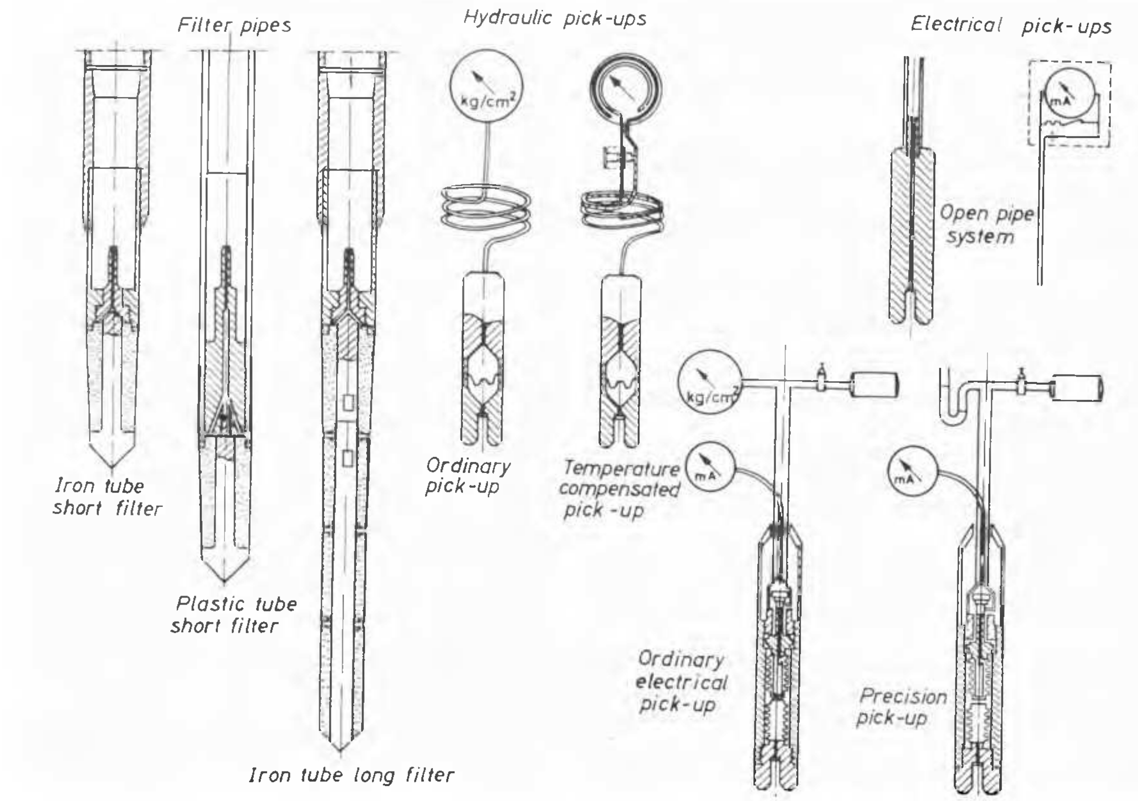

Fig. 13 Pore pressure measuring system S.G.I.
Système S.G.I. pour mesurer la pression interstitielle

Soil pressure measurement has been studied carefully for many years at our Institute and our Proceedings No. 12 shows that it is not possible for good measurements to eliminate the influence of the mechanical properties of the soil close to the cell. The type of cell cover and its travel also have a great influence on the results. Very few workers have studied the behaviour of cell covers against soils, and our research is only one step which ought to be followed by many others.

With regard to the main subject, penetration tests, it must be borne in mind that in cohesionless soils the penetration resistance is mainly dependent on the soil density while in cohesive soils it will be more related to shear strength. In practice, all combinations of these are possible, so that sampling will probably always be necessary if reliable calibration is to be achieved.

However, one does penetration tests mainly to save cost and this ought, therefore, to be regarded primarily as a statistical and economic problem. If it is sufficiently cheap and rapid it can enable extensive borings to be done, giving a good idea of the homogeneity of the ground. As the test provides a large mass of not very precise information, great attention must be paid to simple evaluation. In Sweden we are working along these latter lines, and our rapid sounding machine may be known to many. Good progress is, however, now being made in the development of a somewhat cheaper and lighter equipment and simplified methods of evaluation.

T. K. Chaplin (U.K.)

Referring to Paper 2/7, I hope M. Rocha, U. Nascimento and F. de Castro are successful in using a number of their vibrating wire hygrometers to measure changes in soil moisture. May I bring to their notice the following practical points from my own experience of vibrating wire gauges? My remarks also are equally applicable to the use of vibrating wire gauges to measure neutral pressures in earth structures, which forms part of the General Reporter's fourth proposed topic for discussion (see page 440, Vol. 2).

Paper 3a/10, by R. L'Herminier, M. Bachelier and F. Soeiro, describes an investigation using vibrating wire load gauges, and Paper 5/13 by W. H. Ward and T. K. Chaplin describes the results of investigations using vibrating wire strain gauges in tube railway tunnel linings.

My points are the following. First, a stainless steel wire of high strength is immune to corrosion, but unfortunately has a temperature coefficient of expansion differing slightly from that of any associated ordinary carbon steel. In dry situations a good silver-plated hard drawn high-tensile steel wire, similar to the violin E string, may be adequate.

Secondly, if small changes over long periods of time are of interest, it is essential to refer all readings to an absolute standard of frequency, and not just to the arbitrary scale of a particular control set which may get damaged in use or may be in use on another job at the time you wish to use it. This is not being fussy: I have learnt this lesson the hard way.

Thirdly, the metalwork of the electro-magnet must be insulated from earth.

Fourthly, the electro-magnet must be waxed with a non-cracking tough wax. Hard paraffin wax is a poor substitute.

Fifthly, accidental earthing of the leads between the electro-magnet and the control set, even if only partial, may cause large enough circulating earth currents of electricity to be very troublesome, but this can always be greatly improved if a transformer is inserted between the gauges and the control set, and resonant excitation is used, which, of course, passes easily through the transformer.

Finally, where there are many gauges in an installation, capacitative and inductive interference between leads in a multi-core cable can result in several gauges on a common return lead giving a signal together, when only a single gauge is excited. The ideal is to use no common return leads in the multi-core cable. If the length of lead makes this too expensive, the use of telephone-type selector switches at each end of a single five-core or six-core cable will be satisfactory.

These points that I have made may sound difficult and unexpected, but I would assure you that they have come from practical experience of using large numbers of gauges under difficult site conditions and that such attention to detail is necessary to produce consistently successful results.

B. Rajčević (Yugoslavia)

Monsieur le président, mesdames, messieurs, permettez-moi de vous faire une communication sur les mesures sur place. L'objet de cette communication est de présenter des essais en grande échelle systématiques de compactage réalisés sur chantier en liaison avec l'étude de projet et la construction du barrage de Krupec en Yougoslavie, montraut les résultats préliminairs.

Le matériel utilisé était un rouleau compacteur à pneus de 48 tonnes Albaret, type C 11. Le compactage a été contrôlé par les méthodes suivantes:

(1) Nivellement de précision.

(2) Détermination de la densité (poids par unité de volume) du matériau compacté, par prélèvement d'échantillons dont le volume était déterminé au moyen de sables sélectionnés.

(3) Détermination de la densité et de la résistance à la pénétration, au moyen de l'aiguille de Proctor, d'échantillons préfabriqués en silt sableux et en argile silteuse, préalablement

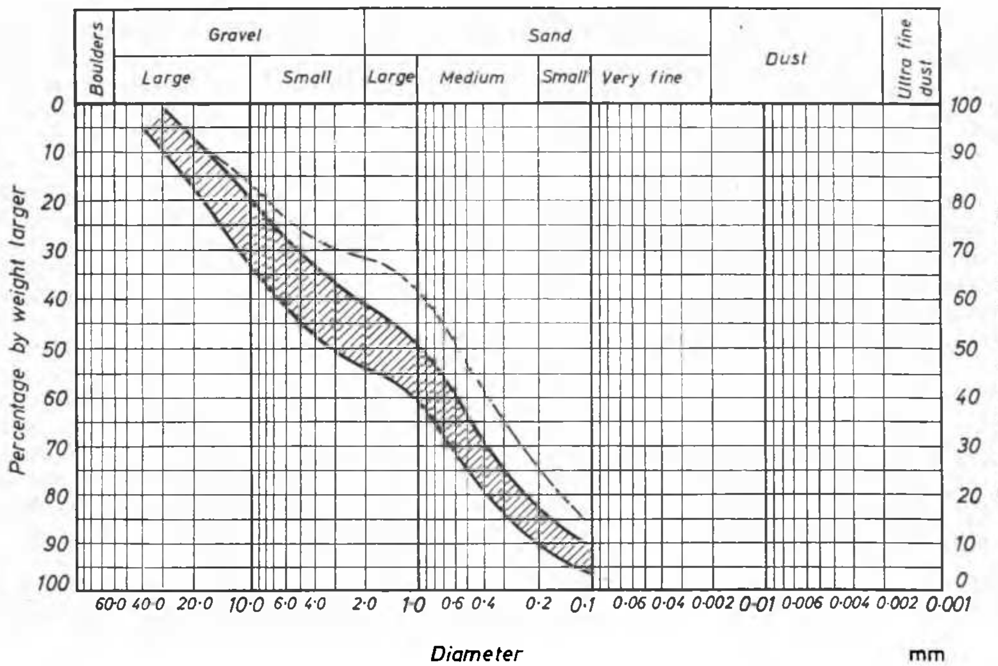

Fig. 14 Grading of dam material
Granulométrie du matériau du barrage

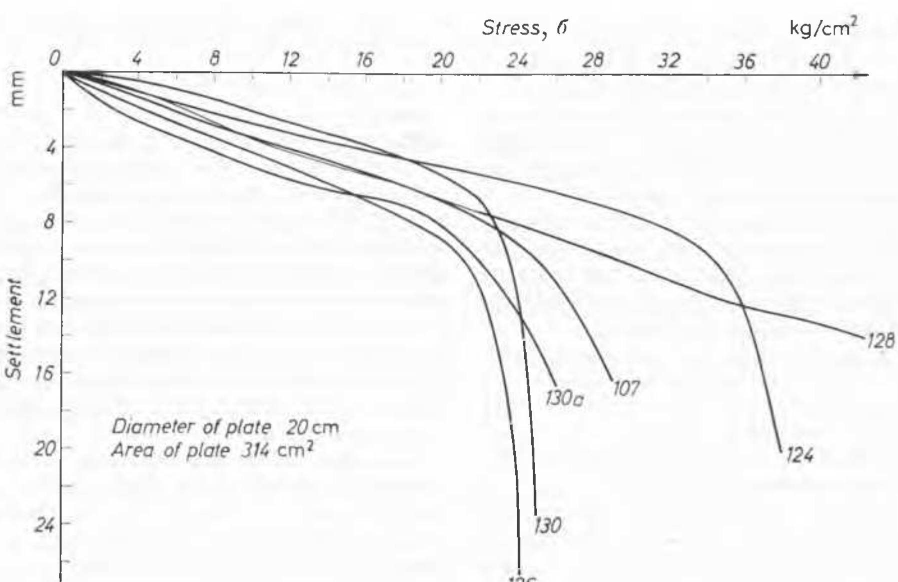

Fig. 15 Results of plate loading tests at Krupec Dam, 1956
Résultats des essais de chargement sur plaque au barrage de Krupec, 1956

incorporés dans le massif et dont les caractéristiques étaient connues à la suite d'essais en laboratoire.

(4) Essais de charge en utilisant une plaque circulaire de 20 cm de diamètre.

Les résultats les plus satisfaisants et intéressants ont été obtenus au moyen de cette quatrième méthode qui présente des avantages importants pour le contrôle de la construction d'un barrage. Jusqu'à présent, en effet, le contrôle du compactage était, le plus souvent, basé sur la notion de 'densité relative' dont la valeur doit être d'au moins 0·7 selon l'expérience du Bureau of Reclamation.

Outre une supériorité sur la méthode des densités qui donne des résultats imprécis et dispersés, la méthode de la plaque chargée présente de plus un avantage considérable, celui de donner des résultats qui peuvent être introduits dans la formule sur la capacité portante critique du sol de K. TERZAGHI and R. B. PECK (1948, *Soil Mechanics in Engineering Practice*, p. 172. New York; Wiley) et utilisant son diagramme pour déterminer l'angle de frottement interne. Connaissant cet angle, on peut alors contrôler directement la stabilité des talus du barrage et ainsi, déterminer le degré necessaire de compactage en fonction du facteur de stabilité imposé per le project.

Nous avons pu, ainsi, observer que la résistance au cisaillement d'un massif compact augmentant avec le nombre de passage du rouleau et, dans notre cas particulier, le nombre économique de passages simples était de 10.

Nous avons également put juger même de l'influence bienfaisante de l'arrosage sur des matériaux graveleux roulés.

W. AICHHORN (Austria)

I should like to mention that since 1932 I have used a similar apparatus to that which J. Kérisel has described. I would emphasize that it is very difficult to interpret the results of this test, and many of the difficulties are referred to in the book *Baugrund und Bauwerk* (Kögler and Scheidig).

L. MENARD (France)

A la suite de l'intervention de W. AICHHORN, je me permets de prendre la parole en vue de préciser certaines données de manière à éviter toute confusion entre le Pressiomètre et l'appareil Kögler.

En 1930, le professeur Kögler, d'Allemagne, a introduit un appareil cylindrique pour réaliser des essais de sol. Comme J. Kérisel vient de l'indiquer cet appareil ne présente qu'une seule cellule. Il présentait ainsi les inconvénients majeurs:

Il ne pouvait éviter que des écoulements plastiques se produisent aux extrémités supérieures et inférieures de la cellule. Le champ des contraintes n'étant pas cylindrique, l'analyse du diagramme pression-déformation devient inextricable.

Du fait des écoulements plastiques aux extrémités les variations de volume de la cellule ne correspondaient aux variations de diamètre du forage au droit de la zone médiane de la cellule. Il est ainsi impossible de mesurer avec une cellule les déformations dans le sol sous l'effet du champ des contraintes.

L'appareil pressiométrique est exposé au deuxième étage.

A. DVOŘÁK (Czechoslovakia)

I have a small contribution to make on Paper 2/5 by D. J. PALMER and J. G. STUART on penetration tests.

We use penetration tests in connection with drilled holes in order to get preliminary information on the soil properties and to reduce the number of laboratory tested samples. I agree with the General Reporter that it is essential to use standard tools and conditions for penetration tests; however, in our country we have soils with comparatively coarse grains, and the standard penetration tool of 2 in. diameter has too small an inner space. For this reason we have introduced for penetration tests the tool which is usually used for undisturbed sampling, with a diameter of 140 mm inside, but until now we have made, whenever possible, a simultaneous standard penetration test. It seems that the number of blows increases approximately linearly with the diameter of the tool when all other conditions are preserved. After taking the sample we make a rough examination of the properties at the place, and when the number of blows as well as other qualities are the same we consider the two undisturbed samples as identical.

There is a fairly good relationship between the number of blows and the porosity or relative density for sands; however, it seems that those relationships change with other factors, for example, with the diameter of the grains, and we get a set of similar lines for coarse, medium and fine sands. We try to use the penetration test also for cohesive soils where the number of blows is probably related to the degree of consistency.

In other circumstances I would not recommend extrapolation of the results, as the penetration per blow decreases considerably with the depth during the test.

Further, it is also necessary to maintain the standard weight of hammer and height of the fall. If we change, for example, the weight and preserve the same kinetic energy, the number of blows decreases with increasing weight of hammer. In addition, it has been noticed that there is an influence of the overburden on the results of penetration tests in bore holes.

J. FLORENTIN (France)

Monsieur le président, mes chers collègues, je voudrais vous dire quelques mots des difficultés que rencontre parfois l'ingénieur à appliquer immédiatement les résultats des reconnaissances *in situ* dont nous souhaitons tous qu'elles se perfectionnent de manière à nous donner des chiffres immédiatement utilisables.

Je vais prendre deux cas.

(1) Le premier site est en bordure de la Seine, à l'aval de Rouen et le sol peut se schématiser ainsi:

de 0 à 10 m de profondeur, sable très fin constituant les alluvions modernes;
de 10 à 20 m de profondeur, sables grossiers, graviers et silex qui constituent les alluvions anciennes;
au-dessous, nous trouvons la craie.

Des pieux avaient été préparés à une longueur de 10 m, comme on avait l'habitude de la faire dans la région. Un grand nombre d'entr'eux n'ont pu être battus au delà de 5 à 6 m de profondeur sans un surbattage très énergique qui introduisait un facteur économique supplémentaire.

Une fois les 5 ou 6 m de profondeur atteints, les pieux reprenaient leur course facilement vers les 10 m habituels. Le maître de l'œuvre décida de faire 6 essais de pénétration qui, pratiquement, confirmèrent les résultats des pieux et qui donnèrent vers 4, 5 ou 6 m de profondeur une couche résistante. En réalité, au cours des sondages, aucune couche spécialement résistante n'avait été rencontrée autour de cette profondeur de 5 m. Ceci a été confirmé par un puits enécuté au voisinage immédiat des pieux en question.

Je suggère que la résistance des pieux et de l'essai de pénétration au cône est due à l'influence de la nappe. Au dessus de la nappe, il s'agit de sable fine peu saturé, situé dans la frange capillaire et dont la résistance augmente avec l'enfoncement du corps étranger que constitue le pieu ou le pénétromètre. Au dessous de la nappe, la pénétration exige une évacuation d'eau qui se fait difficilement étant donné la rapidité de la pénétration eu égard à la très faible perméabilité du sable fin.

(2) Le deuxième cas se rapporte à un équipement hydraulique, mais il s'agit surtout de l'ouvrage de chemin de fer que l'on a exécuté sur le canal de fuite de l'ouvrage, dans une vallée alpestre; la voie séparait l'usine de la rivière dans laquelle devait se faire la restitution. La voie a été mise sur supports provisoires et les piles étaient prévues sur caissons à air comprimé, compte tenu de la très grande perméabilité des alluvions de surface et compte tenu du fait qu'il fallait descendre sous le fond du futur canal.

Le sol comporte trois à quatre mètres d'alluvions grossières: cailloux et graviers et, au-dessous, dans la zone de l'ouvrage, un remplissage fluviatile, d'un creusement glaciaire de très grande profondeur. Ce remplissage est constitué par des sables fins silteux à stratification, je dirai saisonnière, pratiquement visible, que nous avions eu l'occasion d'étudier antérieurement pour le barrage. L'angle intergranulaire était de l'ordre de 38°.

La méthode de travail comportait le relachement de la pression dans le caisson à chaque fin de poste de travail. Cela n'avait aucune importance tant que le caisson était dans les graviers. Mais dans le silt des renards importants se produisaient. A la reprise du travail les ouvriers s'enfonçaient profondément dans le sable silteux remanié. Il était indispensable de descendre dans le silt. Le maître de l'œuvre nous demande de descendre dans le caisson, un essai de pénétration ayant donné 0 comme résistance. Or, 0 n'est jamais un chiffre suffisant pour un ingénieur.

Les décisions prises furent les suivantes: le fonçage du caisson—bétonner, construire la pile et reporter la charge du support provisoire sur la pile. Pratiquement il ne s'est rien passé, même sous la charge d'une locomotive, mis à part le tassement instantané, ceci en raison de la différence qui existe entre un chargement lent et la pénétration dans des silts.

Inutile de vous dire que si cela avait mal marché, on avait prévu d'utiliser les connaissances géologiques de la couche et d'exécuter des puits verticaux de sable avec une mise en charge lente et progressive de l'ouvrage.

Qu'il me soit permis de terminer en souhaitant un agencement des appareils de pénétration, permettant la mesure de la pression des pores du matériau au voisinage de la pointe. En l'absence de cette mesure, trop d'écueils sont possibles.

The Chairman

I now call on the General Reporter to sum up the discussion, and I should like at this stage to thank him for his very interesting introduction.

General Reporter

It seems to me that one of the principal conclusions that we can draw from the discussion we had in this Division is that there are indeed many doubts concerning the standard penetration test which have to be solved before that kind of test can be really useful to the profession. Many authors have pointed out the inconvenience of having so many penetration resistance devices and means to obtain penetration resistance for the purpose of determining mechanical properties of soils.

There is another question which it seems to me is in the mind of every one of us but which was not expressed—namely, is there any real correlation between penetration resistance and bearing capacity of soils?

Brazil is perhaps one of the countries where penetration resistance is most widely used, so that cases where penetration resistance and bearing capacity are correlated can be counted by hundreds. Statistical studies of such correlation have been made, and we know now that deviation for an average bearing capacity for a certain number of blows per ft. can be of 100 or more per cent. For instance, one of the layers of one of our soils is a clayey medium sand with a very low penetration resistance but a high bearing capacity. In one of the foundation programmes in which I was engaged, namely, the foundation of a tall building about 120 m high and an area of construction of about 30×50 m, the average penetration resistance down to the depth of about 30 m was less than 18 blows per ft. According to the textbooks, it was not considered possible to found this building on footings. Nevertheless the building was founded on spread footings resting about 7 m below street level, and the average soil pressure underneath the footings is about 40 t/m^2. After five years the total settlement of the building is only about 3 cm. If such a discrepancy between bearing capacity and penetration resistance is observed in sandy soils, what should we say about the relation between penetration resistance and bearing capacity of clays, where the problem involved is much more complicated than in sand?

The Chairman

I thank the General Reporter for his rather pessimistic final words! I now have pleasure in calling on K. Terzaghi, our President.

K. Terzaghi, President (U.S.A.)

The General Report presented by our colleague and friend, M. Vargas, covered two topics, the geophysical survey and air photo methods, which are not represented by papers. I wish, therefore, to add a few words concerning these two.

So far as geophysical methods of subsoil exploration are concerned, there can be no doubt about their desirability and merits, because they are extremely cheap: they are even cheaper than geologists. They have only one disadvantage, and that is that we never know in advance whether they are going to work or not! During my professional career I have been intimately connected with seven geophysical surveys. In every case the physicist in charge of the exploration anticipated and promised satisfactory results. Yet only the first one was a success; the six others were rather dismal failures.

As far as I know the only rational attempt to ascertain the pre-requisites for a successful geophysical survey was made by the U.S. Waterways Experiment Station in Vicksburg, and the results were published in 1943 in a paper entitled, 'Critical Study of Shallow Seismic Exploration in the Limestone Areas of the Ozark Highland'. Considering the benefits which can be derived from a successful geophysical survey at an almost nominal expense, the value of investigations concerning the prerequisites for success can hardly be over-estimated.

The second topic which is not represented by papers is the aerial survey. For many years I have insisted that the subsoil exploration of dam sites should be preceded by an aerial survey and the study of stereo couples. Examining the photographs under a stereoscopic viewer one can often detect significant geological details which may escape even an experienced observer operating on the ground, because the stereo photos show the topography of the ground surface on an exaggerated vertical scale. On account of the usefulness of aerial surveys the technique of interpreting aerial photographs is making rapid headway.

Considerable attention has been given in the papers to the measurement of pore pressures in sediments. In connection with this topic the question may arise, when is it justified or indicated to measure pore pressures? Everybody agrees that pore pressures ought to be measured in the clay portion of earth dams in order to follow the progress of consolidation during construction, yet few engineers realize that it is equally important to measure the pore pressures in the natural ground prior to construction. Without such measurements, important features of the subsoil conditions may remain undetected. This fact I have noticed on various occasions.

For instance, in 1943 I installed pressure gauges in a thick and apparently homogeneous clay stratum at various depths below the ground surface at the site for an ore yard. At a depth of about 120 ft. the clay deposit rested on hard shale. During the installation of the gauges we discovered to our surprise that the clay stratum was the seat of important excess hydrostatic pressures. At a depth of 70 ft. below the ground surface the hydrostatic head amounted to 30 ft. with reference to the water level in the adjacent river. The existence of these abnormal hydraulic conditions had a decisive influence on the bearing capacity of the clay.

A similar surprising discovery was made at the shore of a lake in the Pacific Northwest. In 1947 a hydroelectric power station was built on the north shore of the lake. According to the results of test borings which were made at the site prior to construction, the site was located above a stratum of very stiff clay with a thickness of about 40 ft. Since the unit load was rather small, no significant settlement was anticipated. Yet, within three years after construction tension cracks in the reinforced concrete tunnels disclosed a horizontal movement of the powerhouse towards the lake over a distance of several inches. At the same time the lake front of the powerhouse settled 4 in. and the opposite one about 7 in., indicating settlement combined with a tilt towards the north. A description of the subsoil conditions can be found in the paper 'Fifty Years of Subsoil Exploration' which I presented at our conference in Zurich in 1953. The pore pressure measurements were made after the movement of the powerhouse was detected; these showed that the base of the clay stratum was acted upon by an excess hydrostatic pressure equivalent to a hydraulic head which varied with the seasons between about 45 and 75. It was this excess hydrostatic pressure which was responsible for the disappointing performance of the powerhouse.

Several years later it was decided to build a new power station on the same lake shore. In order to get away from the seat of the detrimental excess hydrostatic pressures in the subsoil we selected a site adjacent to a rock spur, at a distance of about 2,000 ft. west of the existing powerhouse. At that point the topographic and geological conditions appeared to exclude the possibility of the existence of high excess hydrostatic pressures in the sand strata located beneath the clay, because the base of the clay at the site of the existing powerhouse is located at a depth of about 80 ft. below lake level, whereas at the new site the lake bottom descends at a slope of 1 vertical on 2 horizontal to a depth of 360 ft. below lake level. Nevertheless I considered it necessary to install a few observation wells at the lake shore, to make sure that the pervious strata located beneath the clay freely communicated with the lake.

At the site of the new observation wells the base of the clay was encountered at a depth of about 120 ft. below lake level and 240 ft. above the foot of the steep, submerged slope. Between the depths of 120 and 360 ft. the subsoil consists of silty sand. Nevertheless, as soon as the first boring entered the sand, water started to flow out of the casing, and when a Bourdon gauge was mounted on the upper end of the casing it was found that the piezometric level in the subsoil of the clay stratum was located at an even higher elevation than the corresponding level at the site of the existing powerhouse. Subsequent observations showed that the piezometric head at the new site, with reference to the lake level, varied between about 55 ft. in November and 115 ft. in April, at the time of the melting of the snow. I am still unable to understand how these hydrostatic pressure conditions can exist close to the steep, submerged slope, and why this slope did not fail long ago. On account of these entirely unanticipated conditions we were compelled to establish the new powerhouse on a bench blasted out of the slope of the adjacent rock spur.

The source for the artesian pressure in the silty sand located beneath the north shore of the lake resides in the joint system of the metamorphic rocks out of which the trough occupied by the sediments was carved. In the spring, when the snow melts on the high mountains surrounding the trough, the water table in the jointed rock rises high above the lake level. The silty sand which occupies the bottom of the trough communicates with the joints and its top surface is sealed by the overlying clay stratum. Therefore, the hydrostatic pressure in the sand stratum rises while the water level in the jointed rock goes up.

While I was engaged in the investigation of the site for the powerhouse, I had an opportunity to visit the Big Thompson Valley through which the Canadian Pacific and the Canadian National Railroad descends from the Rocky Mountains to the Frazer River Valley. The Big Thompson Valley is occupied by Pleistocene fine sands and silts which were deposited in an ice-dammed lake. The river flows between terraced slopes on the bottom of an erosion valley which was carved out of the sediments to a depth of more than 100 ft. below the original surface of the sediments. Almost every spring, during the period of melting snows, slides occur which displace the tracks and involve heavy expenditure for repair. Judging from what I have seen on the trip, there is little doubt in my mind that the physical causes of the slides are very similar to those which were responsible for the settlement and displacement of the powerhouse I described. The bottom layer of the Pleistocene sediments communicates with the joint system in the underlying rocks. During the snow-melt the water table in the joint system of the rocks on both sides of the valley goes up and rises high above the level of the present valley floor. As a consequence the pore water pressures in the adjacent silty sediments also go up and the shearing resistance along the potential surfaces of sliding decreases, whereupon local slope failures or slumps occur.

On account of the frequency of the occurrence of artesian conditions at sites where they are not anticipated, the subsoil exploration should always be combined with the measurement of the elevation to which the water rises in the casings from different depths below the ground surface. If water flows out of the casing a Bourdon gauge should be mounted at the top of the casing. Wherever artesian conditions are encountered, a permanent observation well or pore pressure gauge should be installed, to determine the seasonal variations of the pressure.

The discussion of the artesian pressures leads me to the flow slides which occur from time to time in the silty sediments forming the bottom of Norwegian fiords. I owe to L. Bjerrum a description of some of these slides. The movement starts with a local slope failure at some point of the shore line and it proceeds from the point of origin to a distance of many miles from this point at a rate which decreases from an initial value up to 15 miles per hour to several miles per hour in the final stage. Cables which cross the area involved in the movement are snapped, though the area may be almost horizontal. Last fall, in a paper entitled 'Mechanism of Submarine Slope Failures', I expressed the opinion that the movements could be accounted for only by a metastable structure of the sediments. However, considering the fact that the rock surface topography of the Norwegian fiord country is strikingly similar to that of the valleys in the Pacific Northwest, it is conceivable that part of the unusual performance of the fine-grained sediments in the fiords may be due to an upward flow of water out of the joints of the rock into the overlying sediments. Such an upward flow would also account for the development of a metastable structure of the sediments. I do hope that our Norwegian colleagues will investigate these possibilities.

The Chairman

I thank our President for his very interesting final words.

G. Meardi (Italy)

In Paper 6/16 I presented three diagrams of penetrations in very soft silty clays which may be interesting to this session. The abscissae of those diagrams represent the sinking of the penetrometer in cm per blow instead of the usual blows per ft. of sinking. The penetrometer employed had a conical point of 51 mm base, a rod of 33 mm diameter and a casing of 50 mm diameter, with a 73 kg hammer and a 75 cm fall. The rod and casing were in sections of 1½ m.

The diagrams show that the first hammer blow after the driving in of the casing thrust the point into the soil to a depth twice as great as that obtained by the following blows. This fact emphasizes that in very soft clays not only is the casing necessary but it is essential also that this casing should follow the point after each blow.

On the same site the sampling of the soft clay was made by means of a sampler longer than that usually employed. The sampling tube was 2 m long and 1 mm thick and once the sample was obtained it was used as a container. In this way I obtained samples with thin strata of shells practically undisturbed.

With the same method and using tubes 2·5 m long I have recently been able to sample successfully the marine mud of the Spezia Bay, because the long tube reaches the underlying, more consistent, silty clay, which serves to keep the entire sample in the tube.

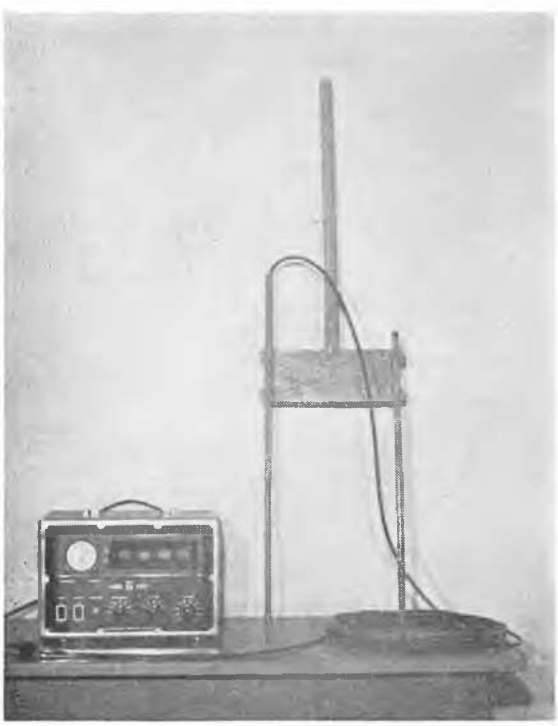

Fig. 16

H. U. Smoltczyk (Germany)

I would like to comment on the field investigation of soil densities.

In Berlin, we have used gamma ray absorption for determining the density of sand to about 10 m depth, and below the water table. A drill hole was sunk to about 1 m above the depth we wanted to test. From there we drove the double-tube test apparatus down to the proposed depth. The two mantel tubes

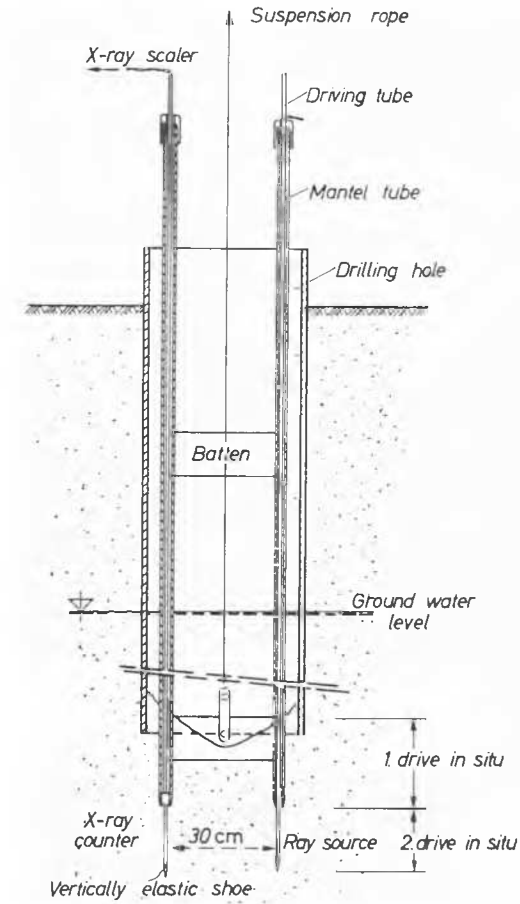

Fig. 17 Testing the soil density of deep layers by means of x-ray absorption
Essais radiographiques pour mesurer la densité du sol dans les couches profondes)

contained smaller tubes, having a cobalt source at the tip of one, and a ray meter fixed at the tip of the other; in principle the same apparatus as described in Paper 2/2. The mantel tubes were open at the top, and the inside apparatus could be driven out into the undisturbed soil after having reached the test point

We have also used the system of a radio-active fork with considerable practical success in order to control fill materials.