Field Investigation, Including Compaction Control, Soil Stabilisation, Technique of Field Observations

Recherches et essais du sol sur place, y compris contrôle de la compaction, stabilisation des sols, technique des observations sur le terrain

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Mr. *W. J. Turnbull*
General Reporter Session 3 – Rapporteur général Session 3

The General Reporter

The preparation of the report for Session 3 included a closure or summary section. This summary section was prepared with considerable effort and was designed also to serve for brief oral presentation at this conference. Further condensation is not believed justifiable. Consequently, at the risk of repetition to some of you, believed to be greatly in the minority, who may have read my report in Volume 2 of the Proceedings of the conference, I herewith present this summary to you.

The following discussion, under four main groupings, furnishes a summary of the detailed reviews. The first grouping discussed is field exploration and sampling. Greatly increased understanding and use of geomorphology and aerial photographs in the preliminary location of structures, estimates of subsurface conditions, and planning of subsequent detailed explorations have contributed more to the general improvement and economy of field investigations than other recent developments.

Simpler and more reliable field equipment for electrical resistivity and seismic refraction methods of exploration have

been developed, and there has been a great reduction in both size and weight of such equipment.

Advances in the design and operation of sounding rods and cone penetrometers have increased possible rates of progress, permit more accurate determination of the skin friction, or practically eliminate skin friction with a consequent increase in attainable depths of exploration. Empirical and theoretical relationships between the point resistance and the strength or bearing capacity of soils have yielded satisfactory results for certain soil conditions, but recent investigations indicate that these relationships are not unique and that the point resistance is a function of many other factors, including minor details in design and operation of the equipment. Exploration by cone sounding has great potential advantages, but a considerable amount of basic research remains to be performed.

The penetration resistance of drive samplers is also a function of many factors, and the limitations of correlations between this resistance and the strength or relative density of soils are being recognized more than formerly. The collection of data for improvement and delimitation of such correlations is continuing, particularly in the U.S.A. and Canada.

Stabilization of bore holes with drilling fluid instead of casing is increasing because the method often is faster and cheaper, decreases danger of disturbing the soil, and facilitates removal of gravel and retention of samples of soft and cohensionless soils.

Distinct advances have been made in the formulation of directions and in the development of equipment and techniques for obtaining undisturbed samples of soils, including saturated sands, but satisfactory methods for undisturbed sampling of gravelly soils are not yet available. There has been an increase in use of improved sampling equipment and techniques, but there is also a need of increased publicity and recognition of the fact that the results of tests on samples obtained with improper equipment or methods may be very misleading, when such samples are believed or considered to be undisturbed.

Recently developed or improved bore hole cameras provide complete photographic coverage of the walls of bore holes and permit definite determination of the location and form of faults, fissures, channels, and cavities.

The second grouping of the summary covers the field studies of properties of soil in situ. As indicated, available correlations of the penetration resistance of penetrometers or samplers with the density and strength of soil in situ are subject to definite limitations. More reliable data on the strength of soil in situ can often be obtained by vane tests. Equipment and techniques for vane tests have been improved, and the tests are being used on an increasing scale, but additional basic research is needed on possible limitations of the test or on the reliability of the results obtained in various types of soils.

Lighter and more mobile equipment for field loading or plate bearing tests has been developed, but these tests are still relatively time-consuming and are often replaced with faster and less expensive methods of investigation, especially since the limitations of results of plate bearing tests are becoming better known.

Sources of error in determination of ground-water levels and pore-water pressures are better known than formerly, and definite improvements have been made in measuring equipment and techniques, especially in the design and installation of piezometers and hydrostatic pressure cells for use in the less permeable soils. These advances are important since determination of pore-water pressures and their changes during and after construction is becoming a practical necessity for proper design and control of construction of many large foundations and earthen structures.

Attempts are being made to replace field pumping tests for determination of the permeability of soil in situ with simpler and less expensive methods, such as observation of flow from or to shallow trenches, borings, or piezometers. These latter methods are theoretically feasible, but the measurements may be subject to significant errors, and additional research and improvements in techniques are needed to determine the influence of or eliminate sources of error.

Investigations of the magnitude and distribution of residual and induced stresses in soil masses are being continued by several organizations. New or improved soil pressure cells have been developed, but it is not yet certain that any of these cells, when buried in moist or saturated soil, will remain completely reliable or retain their calibration characteristics over protracted periods of time.

The third grouping discusses strengthening of soils by compaction and stabilization. Field and laboratory investigations of the compaction of soils are commanding the keen interest and efforts of many investigators. One of the principal difficulties in laboratory testing is practical duplication of the soil structure, density, and strength obtained by field compaction. Fully satisfactory testing equipment and techniques have not yet been developed, but laboratory compaction by kneading shows greater promise of success than static, impact, or vibratory compaction.

Methods of field compaction vary with the type of structure and soil, climatic conditions, and local preferences. Sheepsfoot rollers are used extensively for compaction of more or less cohesive soils, but agreement has not been reached on the optimum weight of the roller, of the foot area and pressure, and corresponding variations in the required number of passes. Such agreement may never be possible due to the great number of variables involved. Heavy rubber-tired rollers, with tire pressures up to or even above 100 pounds per square inch are being used on an increasing scale, especially in the U.S.A. for compaction in depth and of soils with little or no cohesion. Smooth steel rollers are often preferred in Great Britain because climatic conditions usually require compaction of the prevalent cohesive soils at their natural water content.

Vibrations unquestionably increase or facilitate the compaction of cohesionless soils, and utilization of vibrations in compaction is under active investigation, but really successful vibrating compaction rollers have not yet been developed. Heavy plate vibrators give excellent compaction in depth and can be used to advantage in confined spaces and on special and smaller jobs.

Field compaction is usually accomplished with water contents at slightly above, or somewhat below, the optimum for the soil, depending on the type and general design of the structure. In very humid climates it may be necessary or preferable to use the soils at their natural moisture content and tolerate a corresponding lower strength of the compacted soil. Significant advances have been made in methods of moisture control, execution of control tests, and statistical evaluation of the results.

Intense research and practical development of stabilization of soils by admixture with other soils or with various inorganic and organic products are in progress. Cements and bituminous products are the most commonly used additives. Recent investigations indicate that the addition of small amounts of lime or soluble calcium salts extends successful stabilization with cement to relatively plastic clays and organic soils. An increase

in strength or a decrease in the required amount of cement can be obtained by proper selection and blending of local soils, proper moisture control and compaction, and by thorough mixing of additive and soil.

Many chemicals and natural or synthetic organic materials are being investigated and also used in experimental and practical soil stabilization. Some of these materials are expensive but also quite effective, and may ultimately become economical in use because the amounts required are small and current prices of the additive may be reduced through mass production on increased demand.

Observations on completed structures comprise the fourth and last grouping. Continued basic research on soil properties, in both field and laboratory, is required for real progress in the design of foundations and earthen structures, but equally and possibly more important are observations of the behavior of the structure and the foundation soil during and after construction. Such observations may permit corrections in design to meet unforeseen difficulties, and they furnish much needed data for verification or improvement of general theories and design procedures.

Recent advances consist primarily in the assembling and amplification of widely scattered data on equipment and techniques of field observations. The design, methods of installation, and observation of settlements points in the interior of earth masses have been improved; equipment for surface measurements of very small and slow displacements has been developed, and inclinometers have been built or adapted for checking the position of pile points or determination of horizontal movements or creep. Advances in methods for determination of changes in water content, porewater and soil pressures are discussed in the foregoing section of field studies.

Far too few sufficiently detailed observations of the behavior of foundation and earth structures are being made, and it is to be hoped that the importance and value of such observations will be given increased recognition by foundation and construction engineers and by owners of structures.

It would be desirable if all discussions of papers or questions concerning the subjects assigned to this session could be presented for consideration during the session. This is not possible, and it is suggested that preference be given to certain topics which appear to have commanded the greatest interest in the papers submitted to the conference or published since 1948. These topics are (1) determination of density, strength, and other properties including stress and strain characteristics of soil in situ by means of cone penetrometers or vane tests, and (2) factors to be considered in selecting placement water contents and exercising proper density and moisture control during field compaction of soils. The detailed report for Session 3 listed as a third topic for discussion the heaving and settlement of structures as a result of foundation and moisture changes; however, this item will be more fully covered under Session 4 and consequently it is not discussed here.

I also feel that the problems of obtaining, preserving, and preparing undisturbed soil samples for testing are worthy of additional discussion and, in general, should be given greater consideration and thought than they have received from many engineering and construction organizations.

Le rapporteur général dresse un aperçu général des travaux exécutés à ce jour dans le cadre de la Session 3 (pour plus amples détails voir Comptes Rendus 1953, vol. II, p. 319).

Mr. C. Van der Veen

In accordance with the proposal of the General Reporter to limit the discussions to some topics, I should like to say a few words on the method of determining properties of soil in situ by means of the cone penetrometer.

In two papers, one by *Begemann* and the other by *Haefeli* and *Fehlmann*, special attention has been given to the way in which skin friction can be measured with the aid of the cone penetrometer. *Begemann* critizises the ordinary way in which this is done and proposes the use of an adhesion-jacket-cone, as he calls it, which consists of a cone with a loose sleeve placed behind it. It is obvious the predetermination of the skin friction along the shaft of a pile by way of an ordinary cone penetrometer cannot be considered as a solved problem. Perhaps *Begemann*'s proposal will bring some improvement. Nevertheless I should like to make some critical notes concerning this friction sleeve.

First of all I doubt whether this method can be very accurate. The proposed length of the sleeve is 10 cm, which gives, a diameter of 3.6 cm, and a surface of the sleeve of 113 cm². The area of the cone is 10 cm². Now the friction is determined by measuring friction and cone resistance together and then measuring the cone resistance separately. The difference between these two values gives the friction along the loose sleeve. That means that for a variation of 1 kg/cm² in the point resistance, we have a variation of 0.1 kg/cm² or 1 t/m² in the measured friction. Now everyone acquainted with the use of the cone penetrometer knows that, as a rule, much larger variations occur, even when advancing the cone only a few cm further into the ground. Therefore it seems to me that in most cases the friction thus determined cannot be very accurate. The author who has realised this difficulty suggests two solutions: one consists in placing the loose sleeve immediately behind the cone, and the other in placing it as far away as possible. To my mind he has not clearly shown how this should improve the method.

My second remark concerns the state of stress which occurs around the loose sleeve when this is pushed down. This differs from the stress which is encountered when a tube of a much greater length is pushed into the ground. In the case of a loose sleeve, there is, at the upper and lower end of the sleeve, a discontinuity in the state of stress, which may be of importance

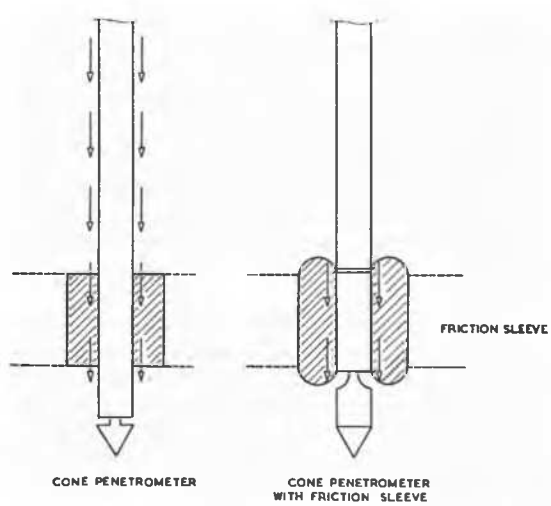

Fig. 1 Cone Penetrometer with Friction Sleeve
Pénétromètre à cône avec manchon de frottement

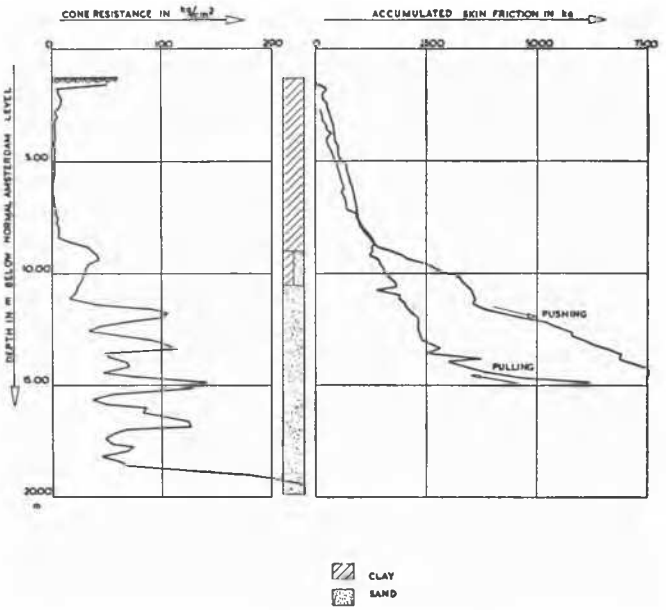

Fig. 2 Skin Friction During Driving and Extraction
Frottement latéral au cours du battage et de l'arrachage

in view of the limited length, 10 cm only, of the sleeve. These boundary influences tend to give a too high friction value, as illustrated in Fig. 1. In this connection, one is tempted to ask about the influence of the open space which exists between sleeve and cone, which, in the case on non-cohesive soils, can lead to erroneous conclusions, as the soil will penetrate into this open space.

Another solution is given by *Haefeli* and *Fehlmann*. The authors have tried to determine the friction by pulling the rod and measuring the resistance with and without the sleeve. In doing so, they introduce a new element which, it seems to me, is no improvement. It is not certain whether the skin friction is the same, when pulling a rod and pushing it down. There are indications that this is not so. This may be illustrated by Fig. 2 which gives the friction measured by means of a penetrometer according to the ordinary method. One measurement was taken while pushing the tubes into the ground, the other curve, when pulling it out. This appears to give only a small difference in clay, but the friction in the sandlayer is far less in the case where the tubes are being pulled. Therefore I doubt whether *Haefeli* and *Fehlmann*'s method will give reliable results.

Finally I should like to call your attention to the fact that the curve of the accumulated friction is composed of the friction measured by the friction sleeve, at a series of points. This means that a constant error in the method of measuring, for instance caused by friction between rod and tube, or a constant error in the measuring device, is multiplied by the number of measurements.

L'auteur commente la méthode proposée par M. *Begemann* (Comptes Rendus 1953, vol. I, p. 213) pour l'estimation de la résistance à l'adhésion. M. *Begemann* propose de placer le manchon soit immédiatement derrière le cône ou le plus loin possible de ce dernier. L'inégalité du manche de l'appareil de sondage, au-dessus et au-dessous du manchon, entraîne, à son avis, l'enregistrement de valeurs de frottement trop élevées. L'espace compris entre le cône et le manchon devrait également être pris en considération.

Pour ce qui est de la méthode proposée par MM. *Haefeli* et *Fehlmann*, l'auteur met en doute l'assertion émise par les auteurs, c'est-à-dire que le frottement latéral est le même au cours du fonçage et de l'arrachage. La différence est minime dans les argiles, mais importante dans les sables.

Finalement l'auteur souligne que, en déterminant le frottement latéral de point en point, l'on multiplie une erreur initiale.

M. M. Buisson

Les forages et essais de laboratoire sont coûteux, longs et ne donnent que des renseignements incomplets, parce que nous savons maintenant que l'homogénité du sol est souvent un mythe.

L'auscultation complète des sols ne peut se faire, pour le moment, que par l'appareil de pénétration pour déterminer le plafond et la base de chaque couche. Mais des difficultés importantes existent pour en tirer tous les enseignements nécessaires. La résistance de pointe d'un pieu peut certainement être déduite sans difficulté de l'essai si l'on est sûr du sol sous-jacent, et dans le cas des sols fins. Il n'en est plus de même avec les graviers ou galets, les renseignements donnés pouvant être fortement erronés. De plus, en ce qui concerne l'évaluation du frottement on se heurte aux difficultés suivantes:

1° Les caractéristiques de frottement acier sur sol sont inférieures à celles de sol sur sol. La cohésion mesurée est inférieure à la cohésion réelle. L'eau ressuée par le sol ne peut être absorbée, ce qui se produit dans le cas des pieux battus, en béton. La résistance tombe alors à zéro dans la zone traversant les limons ou les sables fins argileux. Il est alors illusoire d'essayer de tirer un renseignement de l'essai qui est ainsi faussé.

2° La résistance réelle au cisaillement sol sur pieu pourra être fort différente suivant qu'il s'agit de pieux foncés ou forés du fait de la thixotropie ou, au contraire, de la détente qui se produit en milieu sableux.

3° Le tube se désolidarise du sol – ce qui est mis en évidence par l'annulation prématurée de la résistance à l'arrachage. De plus, la plupart des résistances données par les courbes d'arrachage se trouvent être inférieures aux courbes de résistance latérale au fonçage; quoique la résistance initiale au début de l'arrachage soit souvent égale, dans le sable, et qu'elle soit nettement supérieure, dans les argiles thixotropiques selon la durée d'arrêt. Quoi qu'il en soit, la constatation précédente montre aussi une perte partielle de résistance latérale qui confirme que la résistance latérale totale mesurée au fonçage est, sans doute, inférieure à la résistance réelle. Cela s'explique par le fait de la constitution d'un corps porteur toutes les fois que l'on force le sol à s'écarter et parce que le glissement favorise la formation de ce corps porteur par colmatage progressif. Les calculs effectués pour déduire les caractéristiques de frottement des essais de pénétration doivent tenir compte du fait précédent.

Je ne parlerai pas des inconvénients qui naissent de la forme même des appareils et qui ont été mis en évidence. Toutefois, je crois personnellement que les systèmes proposés par MM. *Begemann*, *Haefeli* et *Hvorslev* qui utilisent seulement le tube au voisinage de la pointe doivent pouvoir résoudre finalement la difficulté n° 3.

La vitesse de fonçage ne permet donc pas de déceler les propriétés thixotropiques des argiles. Il faut prendre la précaution d'arrêter le sondage en cours de route pour pouvoir déterminer, par deux points au moins, la résistance finale en employant la loi logarithmique que j'ai mise en évidence (Congrès de Paris, 1952). Un moyen plus sûr consiste à prélever des échantillons intacts et à mesurer les résistances à l'écrasement sur échantillons remaniés 1 jour et 10 jours après, la teneur en eau restant constante.

De même, MM. *Mayer* et *L'Herminier* ont apporté une contribution importante, permettant d'évaluer la résistance de frottement et de cohésion, en utilisant les données de l'appareil vers la pointe. Toutefois, la résistance latérale mesurée ne pouvant être comparable à la résistance réelle, le long des pieux, pour la raison N° 1, il me paraît impossible d'aboutir à un résultat correct, sauf par excès de sécurité.

Je suggère à MM. *Begemann*, *Haefeli* et *Hvorslev* de procéder à des essais comparatifs avec un tube rugueux au voisinage de la pointe. Je dois procéder moi-même à de tels essais prochainement.

Quant aux essais de battage qui sont employés depuis si longtemps en France pour la reconnaissance du sol en profondeur, ils ne peuvent être interprétés correctement que comme auxiliaires, lorsque des forages et surtout des essais de pénétration statique permettent d'avoir une connaissance complète des sols traversés, et en se basant sur ceux-ci pour étalonner en quelque sorte les résultats obtenus par battage et réduire au minimum le nombre des essais de pénétration.

En conclusion, mon avis personnel est qu'à l'heure actuelle, la reconnaissance du sol doit se faire conjointement par les deux procédés pénétration statique et pénétration dynamique. De plus les sondages doivent être complétés par des sondages réels avec prélèvements intacts dans l'argile et prélèvements dans les couches de sable et gravier.

Etant donné d'autre part que M. *Golder* montre dans sa communication que, dans certains cas, les prévisions déduites des essais de pénétration sont quelquefois excessives, il serait nécessaire que celui-ci fasse connaître la méthode employée pour ces prévisions ainsi que la nature du sol. – Jusqu'à plus ample informé, il semble nécessaire de prévoir encore des essais statiques de pieux, sauf dans les cas où aucune erreur d'appréciation ne peut être commise.

Enfin, en ce qui concerne l'utilisation des sondages par pénétration pour la fixation des contraintes admissibles des fondations superficielles, celle-ci ne deviendra réellement possible, sans essais complémentaires, que lorsque quelques progrès pourront encore être réalisés – à la fois en ce qui concerne la comparaison des conditions de rupture des fondations profondes (réalisées dans le pénétromètre) et de celles des fondations superficielles, et en ce qui concerne l'évaluation de la compressibilité au moyen d'une formule analogue à celle donnée par M. *Buisman*, et à laquelle M. *Marivaet* faisait allusion dans son rapport (Comptes Rendus 1953, vol. I, p. 418).

The author comments on the difficulties which arise when determining the skin friction in static penetration tests and also on the application of penetration tests to foundation problems. The measured skin friction is generally less than the true friction; the true skin friction in a pile depends on the method of installation. Therefore soil investigations should be carried out using two different methods.

Driving tests can be interpreted only by comparison with the borings and the static penetration tests. Before we can apply the penetration method to obtain the elements which enable us to determine the dimensions of shallow foundations, further research will have to be carried out.

Mr. O. Godskesen

The Danish Cone Penetrometer which was earlier called the Pocket-size spring-scale cone has been used for many years in Denmark, and I want briefly to tell you more about it. The Cone penetrometer has a 60 degree cone, similar to the Swedish drop-cones, and a spring-balance which measures the force necessary to push the cone 10 mm into the clay[1]). The newest type of the penetrometer is shown in Fig. 3. It weighs only 60 g and can therefore be carried easily in the pocket. This pocket-size penetrometer has proved a success; it indicates in numerical values the consistency of a cohesive soil, especially when used in excavations or on freshly taken samples (samples still in the tube). The index of the spring-scale cone (the cone penetrometer index) is given as the force in kg which causes a 10 mm penetration of the cone into the soil. In ordinary soil with a varying constency many rough measurements are of greater value than a few accurate ones. Experience seems to show that the index number given by the Danish Cone Penetrometer is approximately twice the safe bearing capacity which is ordinarily used for soil (between 1 and 5 kg/cm²). When previous borings carried out with a weighted drill point[2]) have shown that there are no soft deposits below, we may often assume that the permissible bearing capacity is approximately half of the index of the cone penetrometer. The maximum bearing capacity of clay depends on many conditions, one of which is the sensitiveness to settlements of the designed construction, but actually it seems that the index number is close to, or slightly less than, the expected maximum bearing capacity.

These approximations tally well with some investigations reported by *A. W. Skempton* and *A. W. Bishop*, and *Hugh Q. Golder* and *W. H. Ward*, in "Géotechnique" (Dec. 1950, pp. 99 and 126). From these investigations we also find that the index number can be assumed to be $7\tau_{max}$ for London clay and Shellhaven clay. I have also obtained the same value for the fat and firm Skive clay in Denmark. For the very soft Norwegian clay from Horten however $3\tau_{max}$ must be used.

[1]) Conf. Soil Mech. Harvard 1936, vol. II, p. 38, and *L. Bendel* (1948): Ingenieurgeologie, vol. II, p. 108 and 233.

[2]) Conf. Soil. Mech. Harvard 1936, vol. I, p. 311, and vol. III, p. 269.

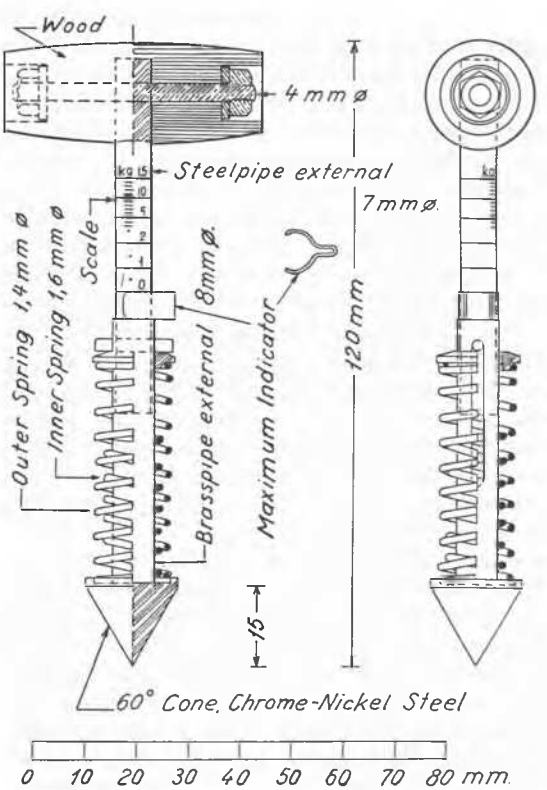

Fig. 3 Small-Size Cone Penetrometer
Pénétromètre à cône, petit modèle

I do not know the exact relationship for other clays, but there seems to be a certain conformity with the sandy Danish clays.

L'auteur présente l'appareil de pénétration à cône danois – modèle de poche – pour l'estimation rapide et sommaire de la résistance dans les sols cohésifs.

Prof. G. F. Sowers

The desirable qualities in a compacted fill are primarily high strength, low compressibility and freedom from swelling and shrinking. Research during the past two decades indicates that for any given soil these properties are largely controlled by the soil's moisture and density—the higher the density and the lower the moisture (within limits), the better the compacted soil.

It is the problem of the constructor, then, to produce a fill of high density and low moisture, and do it with the least expenditure of effort. Minimum effort or cost is particularly important, for if the cost of obtaining density exceeds the benefits obtained, the work will be futile.

In the past, soil engineers have emphasized the need of controlling and evaluating compaction and have left the problem of obtaining it economically to the constructors and equipment manufacturers. Messrs. *Turnbull*, *Shockley* and *Lewis* have pointed out some factors involved in getting better compaction. It is the writer's intention to review the basic principles involved and to point out what can be done in the future.

The concept of optimum moisture, introduced by *R. R. Proctor*, was an important step toward efficient compaction. *Proctor* found that for each given process of compaction, there is a certain moisture content which results in maximum soil density. By proper moisture control, the constructor can get the most out of any process he might use.

The concept of optimum moisture, however, is not the complete answer to compaction efficiency. First, it may not be possible to obtain the optimum due to weather conditions, for example. Second, the optimum moisture may not be the best from the standpoint of strength or other properties. Therefore, in recent years, more and more attention has been paid to the other factors envolved in compaction efficiency: the characteristics of the equipment and its use.

Compaction of a cohesive soil is essentially consolidation or compression with limited lateral support. Consolidation is achieved by the application of pressure—the higher the pressure the greater the density. Some rebound or decompression takes place when the pressure is released. This means that part of the effort or work expended in compacting a soil is wasted. A second application of the same pressure will produce additional consolidation, but not so much as the first. The rebound will be proportionally greater. Each successive application of the same pressure will produce less and less consolidation and relatively more rebound, until the two are equal, and no additional consolidation occurs.

This knowledge of the mechanics of compaction by consolidation leads to two conclusions. First, an increase in the number of successive applications of pressure, such as by increasing the number of passes of a roller, will not lead to a proportional increase in density. This is shown by Fig. 4 in the paper by Messrs. *Turnbull* and *Shockley*, a statement of Mr. *Lewis*, and has been demonstrated many times elsewhere. For example when a sheepsfoot roller "walks out" of the ground, the soil is approaching the point where compression and rebound are equal, and little additional compaction is obtained with many additional passes.

Second, the most efficient compaction is obtained by a compaction procedure which utilizes only the first cycle of compression and rebound of the soil. This has been clearly demonstrated by recent research at Georgia Institute of Technology. Different methods of compaction, such as tamping, hammering and squeezing, were employed, each exerting the same total amount of work in the process. The fewer the tamps or applications of pressure necessary to exert the same work, the greater was the density obtained.

Obviously, if the number of applications of pressure is to be kept to a minimum, the pressure exerted must be high enough to produce the necessary consolidation without repeated applications. This has led to the development of heavier and heavier rollers in recent years. The limit of pressure is the bearing capacity of the soil. If the pressure exceeds this, the compacting device becomes a chopper, or a mixer, or a plow. The General Report mentions the development of a miniature load plate or penetrometer by the Waterways Experiment Station to pre-determine the maximum soil pressure. The LBA Laboratories of Atlanta have successfully computed the maximum pressure from the results of triaxial shear tests, using *Terzaghi*'s formulas for bearing capacity.

Theory indicates that the bearing capacity of soils having internal friction (most compacted cohesive soils) is partially dependent on the size of the loaded area. The greater the width or diameter of the loaded area, the higher the pressure which can be sustained without failure.

Theory also indicates that for a given contact pressure, the stress at any point below the soil surface increases with an increase in the width or diameter of the loaded area.

Both considerations lead to the conclusion that a wide loaded area is desirable. This is confirmed by the results of tests described by Mr. *Turnbull* and Mr. *Shockley*. Also, recent laboratory research at the Georgia Institute of Technology indicate that as the ratio of tamper diameter to soil layer thickness increases, so does the efficiency of the compaction produced.

There appears to be a limit to the increase in effectiveness obtained by increasing the width of the loaded area. Mr. *Turnbull* and Mr. *Shockley* concluded that increasing the area of a sheepsfoot lug from 14 sq.in. to 21 sq.in. did not result in materially better compaction than when using a 6 in. layer and 250 psi pressure. The research at Georgia Institute of Technology indicates that the beneficial effect decreases as the ratio of diameter to layer thickness approaches 1. If the *Turnbull-Shockley* data are reduced to the same ratio, their optimum ratio of diameter to thickness would be about $\frac{2}{3}$. The cause of this limitation appears to be the fact that a rigid load will bridge small soft areas and overstress harder areas with a corresponding loss of efficiency. This may explain why the rubber tired rollers with their flexible loading have been so successful even though their print widths are large.

Time is certainly a factor in the consolidation process if the soils are saturated or nearly so. The effect of time on the consolidation of partially saturated soils, such as those involved in compaction, is not clearly understood. It would appear to be a factor in that the longer the load is applied, the greater the time allowed for secondary consolidation. Tests at the Georgia Institute of Technology indicate that methods of compaction involving dynamic application of pressure produce less compaction than do static methods for the same amount of work exerted. The writer believes that further study of this point is necessary.

Les densités de plus en plus élevées exigées par les travaux modernes de fondations mettent en premier plan les méthodes nou-

velles développées pour obtenir une haute densité avec un minimum d'effort. Nous fondant sur nos connaissance actuelles, nous pouvons dire que la méthode que nous recherchons devrait présenter les avantages suivants:

1° Une surface de chargement aussi vaste que possible;
2° Un dispositif flexible d'application des charges ou un dispositif s'adaptant aux irrégularités produites au moment du chargement dans les parties molles;
3° Une pression suffisamment élevée pour que la compaction désirée soit obtenue après un seul passage sur toute la surface;
4° Une épaisseur de la couche n'excédant pas 1,5 fois le diamètre de la charge.

Mr. N. D. Lea

I would like to speak briefly about the relationship between the shear strength determined in situ by vane tests and the shear strength determined in the laboratory, with particular reference to the interesting paper by Prof. *Skempton* and Mr. *D. J. Henkel* (Proceedings 1953, vol. I, p. 302).

We have had several opportunities in Canada of studying the shear strength of the Leda marine clays of the St. Lawrence valley. We have used a number of laboratory methods, as well as field vane tests. In studying our results we have considered the slope of the strength-depth line. It has generally been found, by investigators in England, Sweden, Canada and the United States that, with the vane tests, we really do get an increase of strength with depth. Professor *Skempton* has considered the ratio c/p for a normally consolidated clay which is, in fact, the slope of this line if it goes through the origin. Since some of our studies dealt with pre-compressed clays, we have used the ratio $c_2 - c_1/p_2 - p_1$ which is the slope of the line even though it does not go through the origin. It is assumed that the strength decrease due to unloading is not important (Fig. 4).

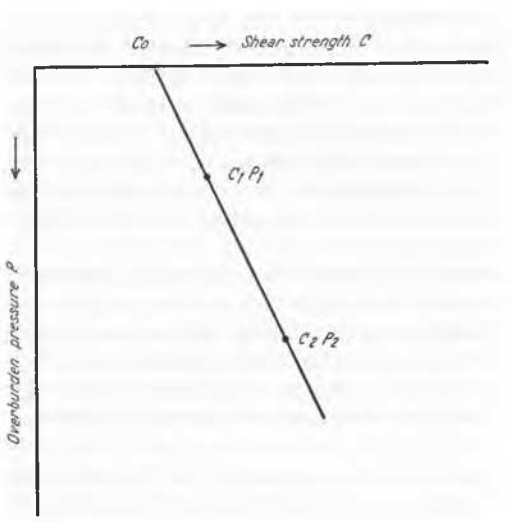

Fig. 4 Slope of the Strength/Depth Line in Preconsolidated Soils
Pente de la ligne force/profondeur dans des sols préconsolidés

Taking the results which we have obtained, and plotting them on the Figs. 8 and 9 of paper 3/17 (Proceedings 1953, vol. I, p. 302), we find that our points fall in the same region but they increase the scatter somewhat.

For inorganic clays the slope of the strength-depth line is 0.2 to 0.4. This slope is useful in studying test results and in estimating the precompression load which is c_0 if the origin cohesion is zero.

In comparing this slope with laboratory tests, we have found that the best method was to use consolidated quick tests, with specimens consolidated at a pressure greater than the precompression load. Taking a series of such tests we find that the slope of the *Mohr* envelope for total stresses is the same as the slope of the c/p line determined in the field by vane tests. This has been our experience on several projects. The magnitude of the shear strength determined in the field with vane tests also checks against the magnitude determined from the *Mohr* envelope if we use, as the normal pressure, the sum of the overburden pressure and the precompression load. This comparison assumes the horizontal and vertical stresses in the ground are equal.

It is interesting to consider the influence of the origin cohesion, which we were considering this morning, upon the interpretation of vane tests. When I came to our hotel at noon, I looked at one of our projects where we have had a very low precompression load and I found that it could be interpreted in just the same way, as Mr. *Jakobson*'s results (Proceedings 1953, vol. I, p. 35). But the interesting thing was that we had analyzed it as though it were slightly precompressed and built the structure accordingly. It has performed quite well, without the great settlements which would be expected if it were not precompressed. Thus it occurred to me that this origin cohesion might for practical purposes have exactly the same influence as shear strength derived from precompression. I think, it would be quite interesting if, on projects similar to Mr. *Jakobson*'s, we had the results of consolidation tests so that we could see if this origin cohesion shows up in the consolidation test, as a part of the precompression load.

We have also compared the results of unconfined compression tests on shelby tube samples with similar tests on samples from the *Kjellman* foil sampler and we have found that the samples from the *Kjellman* sampler in some instances gave definitively higher strength. We believe that this may be due to the structure of the clay which was chunky. The planes of weakness may develop into open fissures in some methods of sampling.

L'auteur compare les essais de résistance au cisaillement à l'aide de l'appareil à palettes et les essais rapides en laboratoire sur des échantillons consolidés. Dans les sols préconsolidés la ligne reliant les points représentant la pression de surcharge aux points représentant la résistance au cisaillement n'atteint pas zéro. L'inclinaison peut être exprimée comme suit: $c_2 - c_1/p_2 - p_1$.

Des résultats d'expériences pratiques nous autorisent à assimiler la cohésion d'origine de M. *Jakobson* à la précompression. Finalement l'auteur souligne que les essais sur des sols prélevés à l'aide du tube de carottage de *Kjellman* donnent une résistance plus élevée que les essais avec l'appareil de prise d'échantillons de *Shelby* ce qui semble dénoter une déformation moins accentuée.

Mr. D. J. Maclean

I should like to make some comments on soil compaction, on the basis of experience gained by the Road Research Laboratory in Great Britain. In the compaction of soils for road embankments, the primary consideration is to prevent non-uniform vertical movements of the pavement after construction. Stability problems seldom arise because of the shallowness of the earth works, compared for example with earth dams.

With regard to the state of compaction to be obtained in soil we feel that this can more logically be specified for British conditions in terms of the percentage air content of the soil than as a percentage of the maximum dry density obtained in

a laboratory compaction test. In Great Britain, the natural moisture contents of cohesive soils invariably exceed, by several per cent, the optimum moisture content obtained in the standard laboratory compaction test. This has led to many instances where a state of compaction has been specified which is only obtainable in practice by drying the soil, an unpractical process in Great Britain. This difficulty would not arise if the required compaction had been specified in terms of a maximum permissible air content.

In a paper by *Croney* (Proceedings 1953, vol. I, p. 13), which might better have been included in the papers for this session, it has been shown that soils in Great Britain tend to reach an equilibrium moisture content beneath impervious areas. This condition which can be calculated from laboratory suction and loading tests on the soils, is in Great Britain generally very much closer to the natural moisture content than the optimum moisture content obtained from the standard laboratory compaction test. This means that, if the soil fill for a road embankment is compacted at the natural moisture content, there will be little tendency for the soil to become dryer or wetter after the construction, and consequently little change, either in the strength of the subgrade or in the volume of the fill. On the other hand, were the fill placed at a lower moisture content, there would be a tendency, possibly over a period of many years, for water to enter the fill, from below as a result of differences in suction, or from the sides and top. This would lead in road embankments constructed with cohesive soils to the development of undesirable irregularities in the road surface.

In dry climates the moisture content of the natural soil may be much lower than the theoretical equilibrium moisture content under an impermeable surface. In such climates wetting of the soil in embankments from beneath is likely to be counteracted considerably by drying from the sides and top. There appears, however, to be little doubt that there is a tendency for the soil to become wetter under large covered areas, such as building foundations, an almost unpredictable condition of dynamic equilibrium being established when the water drawn up, as a result of differences in suction, is equal to the water lost by evaporation and transpiration at the boundaries of the covered area. Here the tendency is for a high moisture content to be established near the centre of the foundation area. We agree with Mr. *Jennings* of the National Building Research Institute of South Africa that this is probably the primary cause of the heaving of buildings constructed on expansive soils.

In our view, measurements of moisture content generally provide an unsatisfactory means of detecting moisture movements of this type, because of the influence of soil type and density on the moisture content. The migration of moisture can best be detected by means of porewater pressure measurements. In this connection, there is a need for satisfactory gauges to be developed to measure negative pore water pressures, outside the range of the ordinary tensiometer.

L'auteur fait quelques observations sur le compactage des sols, qui sont basées sur les expériences faites au Road Research Laboratory en Grande Bretagne. Il estime que dans les conditions prévalant en Angleterre, il vaut mieux caractériser la compaction par le pourcentage de la teneur en air que par la densité sèche. L'humidité naturelle des sols cohésifs dépasse invariablement de plusieurs pour cent la teneur en eau optimale obtenue en laboratoire d'après la méthode standard de compaction, et, par conséquent les chaussées ne peuvent être compactées à la densité sèche optimale. D'autre part M. *Croney* (Compte Rendus 1953, vol. I, p. 13) a constaté que le régime d'humidité qui s'établit avec le temps au-dessous des revêtements imperméables est plus rapproché de la teneur en eau naturelle que de la teneur optimale. En conséquence les sols sont compactés à teneur en eau naturelle en prévision des mouvements différentiels. Même dans les pays à climat sec la teneur en eau qui s'établit au-dessous du centre des grandes surfaces couvertes, telles que les fondations de bâtiments, accuse la tendance indiquée plus haut, ce qui, par la suite, entraîne des soulèvements. L'auteur estime préférable de mesurer les variations de l'humidité sur la base de la mesure de la pression de l'eau interstitielle que sur la détermination de la teneur en eau naturelle.

Mr. M. Peleg

I realise that the small-scale works carried out in Israel are by no means comparable with the large schemes carried out in bigger and more developed countries. But I believe that the results of this work may be of interest, especially for the countries that are in the same region or in a similar climate.

In the Jordan Valley, a small housing project of adobe houses, comprising about 30 dwelling-units was carried out in the summer of 1952 by the Housing Department of the Ministry of Labour and under the technical guidance of the Division of Earth Construction of the Israel Institute of Technology. Preliminary tests were carried out in order to determine the method of stabilizing the earth material and construction of the houses.

Owing to the presence of sulphates (approximately 0.35%) the addition of cement to the heavy soil proved to be unsatisfactory. (Test blocks showed signs of deterioration.) The blocks, $10 \times 25 \times 40$ cm, were moulded with addition of straw only (1–2% by weight) in wooden forms which were removed immediately after preparation. The earth-water-straw mixture was kneaded in a primitive way with the feet.

A special feature of this project is 8 houses with domed roofs, made of earth blocks to which 1–2% of heavy fuel oil was added for stabilisation. The mortar for the construction of the domed roofs was of the same composition. Walls and roofs were coated from the outside with "Flintcote" which showed good adhesion, both to the earth material and the sand-lime-cement. From the inside, the walls and domed roofs are coated with the same asphalt emulsion, to which water was added in order to reduce the cost, as no protection against moisture was necessary, only improvement of adhesion, was desired.

In some houses cracks appeared in the walls, apparently caused by too shallow foundations on the heavy soil having swollen because of heavy rains during the previous winter. Cracks appeared in the outside plaster, due to too early plastering before the joints of the walls had settled sufficiently. These cracks were filled with a mastic made of "Flintcote" and sand.

The small project and the tests carried out in connection with it are being followed up this summer by a larger-scale building scheme, carried out by the Department for Earth Construction of the Ministry of Labour with the support of U.N.O., and for which much more modern machinery and tools are being used.

L'auteur donne quelques indications sur des essais entrepris en Israël en vue de la construction de maisons d'habitation en blocs de torchis. La présence de sulfates dans les sols excluait l'emploi du ciment. Les blocs employés pour la construction des murs contenaient 1–2% de paille environ; pour la construction des toits voûtés l'on a eu recours à une adjonction d'huile lourde. La face intérieure des murs et des toits a été revêtue d'une couche de «Flintcote», c'est-à-dire d'une émulsion bitumineuse.

Des fissures ont apparu dans les murs; elles sont dues d'une part au tassement du sol lourd de fondation, de l'autre au fait que la couche de «Flintcote» a été appliquée trop tôt.

Messrs. A. A. Maxwell[1]) and R. A. Ford[2])

The calibrating equipment devised by Ir. *G. Plantema* (Proceedings 1953, vol. I, p. 283) is believed to be a definite contribution to the art of producing controlled calibration conditions for earth pressure cells. The size of the calibration tank with respect to the size of the soil pressure cell is probably of equal importance to the fact that calibration can be carried out under different ratios of vertical to lateral pressure. The elimination of side friction is also an extremely important and valuable factor in a calibration tank.

The presence in a soil mass of a type of pressure cell, such as used by the author, which has a rigid rim and a large flexible membrane, is likely to cause large changes in the stress pattern. A large flexible membrane will probably deflect more in the center than toward the edges. As a result the cell reaction is not a measure solely of the vertical pressure but is a composite of several stress components, and a variety of calibrations may be needed.

The successful use of such a cell rests largely in the selection of calibration conditions and how well these calibrations correspond to the actual field conditions under which the measurements are made. It appears that the author has made a wise selection in his calibration device.

The WES earth pressure cell, developed during the period from 1938 to the present, utilizes the same principles as the *Plantema* cell, but there are important differences in detail design. Some initial difficulties were diminished by use of a more rigid face plate instead of a very flexible face plate or membrane. The advantage of this is that the rigid face plate acts somewhat like a platform scale and sums up the forces applied to its area. The type of deflection characteristic which is incorporated in the WES cell seems to simulate more practically the actual strain conditions which would exist in a mass of fairly homogeneous material. The over-all deflection characteristics of the WES pressure cell are within the modulus of the deformation range 10,000 psi to 60,000 psi.

There has been considerable improvement of the original WES pressure cell over that described in the report, "Soil Pressure Cell Investigation", TM 210–1, July 1944. The cells described in that report performed satisfactorily in laboratory tests but their behavior in the field, especially when placed below the water table, was far from satisfactory, as leaks usually developed and made the electrical circuit inoperative.

Several major changes have been made in recent years which have greatly improved the performance of the WES cells in the field. The cell is now made of heat-treated stainless steel (type 416 or 17–4*p*H) which minimizes creep, after effects and mechanical hysteresis and in addition provides some degree of protection from trouble caused by corrosion; mercury is used as a pressure-transmitting medium instead of transformer oil; the space below the diaphragm is filled with dry nitrogen to minimize corrosion caused by presence of moisture and oxygen; the electrical cable now enters the cell through a hermetic seal which prevents leakage of water into the cell; and four SR–4 strain gages are used within the cell instead of two. Two of the gages are placed near the edge of the cell diaphragm and are subject to compressive strains, the other two are placed near the center of the diaphragm and are subject to tensile strains. The four gages comprise the four arms of a Wheatstone bridge circuit; thus a full bridge circuit is now employed within the cell rather than the half-bridge circuit used in the earlier models. This results in greater sensitivity, about 3000 micro inches per inch for full capacity load, and eliminates the undesirable effects of resistance changes in the cables.

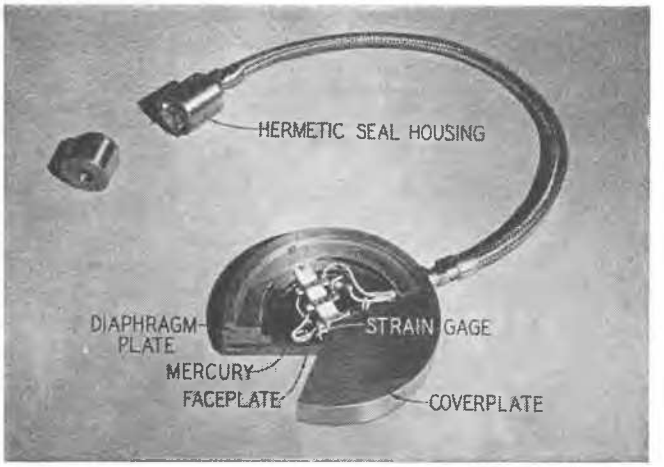

Fig. 5 WES Earth Pressure Cell
Cellule WES pour la mesure de la pression

The results of these improvements are becoming evident from an installation of 18 cells on the stilling basin wall and outlet conduit of an earth dam. After a period of almost four years 16 of these cells are still operating satisfactorily, the performance of one is doubtful, and one is inoperative because of reasons not yet determined.

The following three figures show the principal features of the improved WES cell. Fig. 5 is a photograph of the cell with a portion cut away so that the interior may be seen. This cell is 6 inches in diameter and 1 inch thick. Fig. 6 shows two vertical sections, one through the pressure cell and one through the hermetic seal cable housing. Fig. 7 shows the electrical circuit schematically and also the location of the gages on the diaphragm.

Les auteurs commentent tout d'abord la cellule de pression et l'équipement d'étalonnage faisant l'objet de la communication de

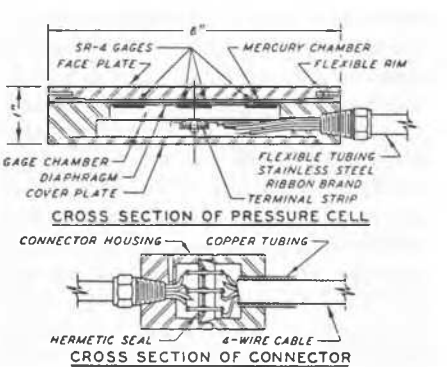

Fig. 6 Cross-Section WES Earth Pressure Cell
Coupe transversale d'une cellule WES pour la mesure de la poussée des terres

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[2]) Engineer, Instrumentation Branch, Technical Services Division, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, U.S.A.

M. *Plantema* (Comptes Rendus 1953, vol. I, p. 283). La difficulté rencontrée dans l'emploi d'un appareil de ce genre réside dans le fait que la cellule est emboîtée dans un cadre rigide, ce qui peut engendrer des effets d'arc à la suite desquels des forces de cisaillement sont mobilisées et mesurées.

Les auteurs décrivent ensuite un modèle perfectionné de cellule de pression mis au point par la W.E.S., qui comprend une face plus rigide que le précédent, ou membrane, et différentes améliorations (métal non-corrosif, mercure, sceau hermétique, quatre jauges SR de déformation). Voir Figs. 5, 6 et 7.

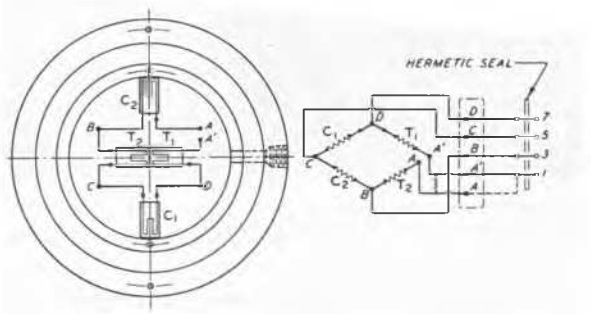

Fig. 7 Circuit Diagram—WES Earth Pressure Cell
Diagramme du circuit – Cellule WES pour la mesure de la pression

The General Reporter

During the few minutes left for this session, it would possibly be desirable to give a brief summary of my opinion concerning the status of the work covered by Session 3. However, in my capacity as Reporter, I find myself in the difficult situation of having to choose between doing this or allowing the remainder of the time to be taken up by people who have requested the floor. In fairness to these people, I am yielding the floor to them and request that they condense their remarks to two or three minutes each. I now invite Mr. *Croney* to take the floor.

Le rapporteur général renonce à faire un résumé des travaux de sa session et préfère donner immédiatement la parole aux orateurs.

Mr. D. Croney

I just wanted to refer to one point on a paper which was actually considered at the first session yesterday but which, as Mr. *Maclean* said this afternoon, might more suitably have been considered in this session. Mr. *Turnbull* has also referred to others of my papers on the subject of soil moisture. Mr. *Wolpert* said yesterday that alpha in the equation $\alpha P - S = U$, which I quote relating normal pressure, suction and porewater pressure, has a mathematical significance only. He stated that if it were regarded as that fraction of the overburden or normal pressure carried by the porewater, the equation was in disagreement with *Terzaghi*'s consolidation equation. This is not so.

When a sample of a saturated, compressible soil is loaded, the equation $\alpha P - S = U$ holds while the consolidation process is taking place and when it is completed, but of course the suction S and the porewater pressure U must be measured at the same instant, i.e. no consolidation or change of moisture content must occur between the measurement of these quantities. If, in a normal consolidation test, the suction of the soil was measured before the application of the normal load and the equation $\alpha P - S = U$ applied, α would certainly appear to be less than 1, but only because the suction at the end of the test should have been used.

Defining the terms, as they are defined in my paper, I believe that I am correct in saying that the expression $\alpha = 1$ means the normal pressure is wholly carried by the porewater. Equally, in a material of rigid structure or in a clean compacted sand, the expression $\alpha = 0$ means that no part of the normal pressure is carried by the porewater. I thought I would just like to clear up this point.

L'auteur répond à M. *Wolpert* qui, au cours de la séance 1, a mis en doute la valeur de la formule avancée par lui, c'est-à-dire: $\alpha P - S = U$ (Proceedings 1953, vol. III, p. 114). Selon M. *Wolpert* α aurait une valeur mathématique seulement. L'auteur expose que si S et U sont déterminés en même temps au cours de l'essai, sa formule est valable; $\alpha = 1$ indique que la pression normale est entièrement supportée par l'eau interstitielle.

M. J. M. Pappaert

J'ai quelques mots à vous dire à propos de compactage. L'un des orateurs a déjà évoqué la question, mais je crois utile d'y apporter un complément. Les remarques que je vais tenter de vous exposer rapidement sont fournies par une expérience toute récente qui doit encore être poursuivie sur une large échelle. Il s'agit de la construction d'un aérodrome pour avions lourds, actuellement en cours au Congo Belge. Le site choisi présente, quant au matériau naturel, une grande uniformité; il se compose en ordre principal d'une épaisse couche de sable légèrement argileux qui doit être compactée avant la pose du revêtement. Comme les roues des avions exerceront sur la piste des efforts isolés de près de 50 tonnes, le maître de l'œuvre – qui est aussi auteur du projet – a estimé que le moyen le plus efficace de réaliser la compaction consisterait à utiliser un engin qui engendre, à peu près et même par excès, les pressions effectives qu'exerceront plus tard les avions lourds sur le revêtement et son support. Vous avez déjà pressenti qu'il a donc été fait usage d'un «supercompacteur» *Porter* pouvant être chargé jusqu'à 200 tonnes, à partir d'un poids propre à vide de 45 tonnes environ.

Conformément aux règles américaines du C.A.A., il a été prescrit de réaliser le revêtement (béton asphaltique ou de ciment), avec ses couches sous-jacentes d'empierrements, à même le sable naturel et de compacter celui-ci de manière que dans les 50 cm supérieurs on atteigne un Proctor de 100% (poids volumétrique sec 2,020 g/cm³) et, dans les 50 cm suivants, un Proctor de 95%.

Des essais préalables ont été faits au Laboratoire de l'Institut Géotechnique de l'Etat (Belgique). Parmi ces essais nous citerons la recherche des Proctors et C.B.R. correspondant respectivement à 25, 50 et 100 coups par couche. Les courbes de variation du C.B.R. en fonction de la teneur en eau ont accusé une forme en cloche asymétrique très typique, se caractérisant par un flanc «humide» très abrupt. Cela signifie qu'il est prudent de se tenir un peu en deça des optima, c'est-à-dire sur le flanc «sec» (dry side) des courbes. Comme les pourcentages d'eau optima, relatifs aux trois énergies croissantes citées plus haut, avaient été trouvés respectivement égaux à 8,2% à 7,6% et 7% tant pour le C.B.R. que pour la densité sèche, on estima d'abord qu'il était indiqué de s'en tenir aux environs de la plus faible de ces valeurs lors du compactage de la piste d'essai; les valeurs satisfaisantes du C.B.R. et de la densité sèche, obtenues après un nombre suffisant de passes, semblaient d'ailleurs confirmer ce point de vue. Cependant, lors d'un passage imprévu du Porter chargé à 160 tonnes, et ce à l'époque où l'humidité moyenne du sol était devenue supérieure à celle admise pendant le compactage, on fut surpris de constater des affaissements tout à fait anormaux. Ce fait a suscité des observations extrêmement précieuses pour la

détermination définitive des teneurs en eau les plus convenables à associer à des énergies extérieures données.

Tout d'abord, la nécessité est apparue de pouvoir procéder, si les délais d'exécution l'autorisent, à un étalonnage de l'engin de compactage, c'est-à-dire au relevé expérimental des courbes du genre Proctor propres à divers chargements du rouleau. En effet, l'équivalence du travail mécanique – développé d'une part en laboratoire et d'autre part sur chantier – ne semble pas pouvoir être établie a priori sur la base de considérations purement théoriques, d'autant plus que sous un rouleau à pneus par exemple, le sol ne subit pas les mêmes transformations internes que sous la dame Proctor.

Un autre point est celui du choix de la teneur en eau optimum à différents niveaux, lorsque l'on compacte à profondeur relativement grande à l'aide d'une forte charge superficielle; pour que celle-ci ait un rendement maximum, il est souhaitable que la teneur en eau puisse augmenter, dans une certaine mesure, avec la profondeur. Cette variation pourrait se traduire par une courbe caractéristique établie à partir des valeurs expérimentales des énergies capables du rouleau et à partir des tensions en divers points du massif chargé. Sans doute n'est-il pas aisé de choisir, au cours de l'exécution des travaux, le moment le plus opportun où ces conditions idéales d'humidité se réaliseront plus ou moins, soit par les pluies soit par des arrosages artificiels. De plus les phénomènes de perméabilité et de pression interstitielle viennent encore compliquer le problème!

Voilà les quelques réflexions que j'ai désiré soumettre à votre attention.

The author reports on the experience gained in laboratory and in situ tests (Proctor, C.B.R.) when compacting the subsoil of a runway in the Belgian Congo. The subsoil of clayey sand was compacted by means of a Porter supercompactor (200 tons).

Further, the author draws attention to the necessity of determining the Proctor curves in the field with compaction by heavy equipment, and also to the difficulties which arise when compacting the lower layers at a minimum moisture content that increases with depth.

The General Reporter

No time was available to the Reporter to analyze the formal discussions presented by the ten discussers. It is obvious from listening to their reports that about seven out of the ten discussers made direct or indirect reference to the topics listed by the Reporter as most desirable for discussion.

In general I believe it can be said that this conference has demonstrated that a great amount of progress has been made in the field of work covered by Session 3. It is believed that the conference further indicates that we have not yet developed all investigational work to the fullest. Much work and investigation still need to be done. The field of investigation in connection with the development of the strength of soil in place by means of the cone penetrometer, the vane shear device, and similar instruments can be very fruitful. This Reporter desires to thank the various chairmen and committees of the conference for a job which is being very well done.

Le rapporteur général constate avec plaisir les grands progrès réalisés depuis le dernier Congrès. Néanmoins les investigations n'ont pas encore atteint le but final. Le rapporteur remercie enfin les présidents et comités qui ont aidé ce congrès.

Dr. L. Bendel

During the last ten years extensive research on the determination of the physical properties of soil by means of cone penetrometers has been carried out in my laboratory. The measurements have included:

(1) the penetration per blow;
(2) the blows per foot of penetration;
(3) the magnitude of penetration of the cone in each layer.

The weight of the hammer was 100 lbs. and the height of fall varied from 20 to 30 inches.

Good agreement has been found in a large number of measurements:

(*a*) Relation between the penetration (C) of the cone and the compression index K in the formulae

$$S = \int_0^h K \log\left(\frac{p_0 + \Delta p}{p_0}\right) dh$$

S = settlement of a circular plate 30 cm in diameter
p_0 = initial pressure
Δp = additional pressure
dh = height of the layer.

$$K = K_t \log\left(\frac{C_a + \Delta C}{C_a}\right) + K_a \text{ in } \% \qquad (1)$$

C = observed penetration of the cone in mm, $C = C_a + \Delta C$
K_a = compression index, if the penetration (C) of the cone is $C = C_a = 10$ mm and $\Delta C = 0$ mm
K_t = compression index if the penetration (C) of the cone is $C = (C_a + \Delta C) = 10\ C_a$.

Examples:

Sand: $K_{\text{sand}} = 3{,}4 \log \frac{C}{10} + 4$ in %

Peat: $K_{\text{peat}} = 9{,}7 \log \frac{C}{10} + 8$ in %

(*b*) *Relation between the penetration* (C) *of the cone and the angle* φ *of internal friction.*

$$\varphi = \varphi_a - \varphi_t \log\left(\frac{C_a + \Delta C}{C_a}\right) \text{ in degrees.}$$

φ_a = angle of internal friction, if the penetration (C) of the cone is $C = C_a = 10$ mm
φ_t = angle of internal friction, if the penetration (C) of the cone is $C = (C_a + \Delta C) = 10\ C_a$.

Examples:

Sand: $\varphi_{\text{sand}} = 37° - 8 \log\left(\frac{C}{10}\right)$ in degrees

Peat: $\varphi_{\text{peat}} = 27° - 16 \log \frac{S}{10}$ in degrees

(*c*) *Relation between the penetration* (C) *of the cone and the stress* σ *in the rod of the penetrometer.*

$$\sigma = \sigma_a - \sigma_t \log\left(\frac{C_a + \Delta C}{C_a}\right) \text{ in kg/cm}^2;$$

σ_a = stress in the rod, if the penetration (C) of the cone is $C = C_a = 10$ mm
σ_t = stress in the rod, if the penetration (C) of the cone is $C = C_a + \Delta C = 10\ C_a$.

(*d*) No relation has been found between the penetration (C) and the permeability or the water content.

Conclusion:

There exists a relationship between the penetration of a cone and the physical properties of the soil.

L'auteur soumet des formules exprimant la relation entre la pénétration d'un cône et les propriétés des sols.

Prof. R. Cebertowicz

L'Institut de Travaux Hydrauliques de l'Ecole Polytechnique de Gdansk annexée à l'Académie Polonaise des Sciences, a développé un procédé simple pour l'amélioration de mauvais sols de différents types, tels que sables peu denses, glaises, argiles et sols contenant des matières organiques (tourbe, marne lacustre, etc.).

Les recherches de MM. *R. Haefeli* et *W. Schaad* de l'Institut de Mécanique des sols de l'Ecole Polytechnique Fédérale de Zurich – où le soussigné a travaillé de 1942 à 1945 en qualité d'assistant scientifique – ont été le point de départ de nos études pour la mise en pratique des phénomènes électro-cinétiques.

On peut comparer l'effet des phénomènes électro-cinétiques à l'effet d'une pompe à double action dans laquelle des solutions chimiques coulent de l'anode à la cathode, sous l'influence d'un courant électrique faible et régulier.

En introduisant des solutions chimiques dans un mauvais sol, nous pouvons améliorer sensiblement l'une des plus importantes qualités de ces sols, c'est-à-dire leur cohésion. Les sols deviennent plus cohérents et plus imperméables.

Sur la base d'expériences, tant au laboratoire que sur place, nous avons constaté, que le poids volumétrique unitaire augmente, que les vides deviennent plus petits et que l'angle de frottement interne s'accroît de 30% environ; les sols peu denses deviennent plus compacts et leur sensibilité se rapproche de celle des roches excavables à la pioche; l'eau interstitielle en excédent peut être éliminée facilement, etc.

Nous avons fait plusieurs expériences pratiques qui ont été couronnées de succès:

1° Reprise en sous-œuvre de divers bâtiments qui s'étaient fortement tassés; la capacité portante des sols a été à peu près quadruplée par le traitement.

2° Stabilisation de talus, par exemple à l'église Ste-Anne à Varsovie.

3° Solidification du sol de fondation en une paroi verticale qui est à même de remplacer un rideau de palplanches métalliques.

4° Exécution dans les sols de gorges d'eaux et de fouilles ouvertes perméables jusqu'à une profondeur de 8 m au-dessous du niveau phréatique.

5° Travaux pour assurer des puits de mines; dans un cas, une couche de sable aquifère située à une profondeur de 96 m a été étanchée sous une charge d'eau de 50 m, rendant possible par là le fonçage de puits jusqu'à 225 m.

6° Etanchement de matériaux de construction: bois, béton, maçonnerie, etc.

7° Vivification et développement, respectivement anéantissement de cultures arborifères.

Le coût de l'application pratique de la méthode électro-cinétique est très peu élevé; dans plusieurs cas le coût n'a pas dépassé le dixième des frais entraînés par n'importe quel autre type de fondation classique.

Le soussigné a présenté une note à l'Académie Hongroise des Sciences sur l'application pratique des phénomènes électro-cinétiques, note qui a été publiée dans la presse technique hongroise.

L'application pratique des phénomènes électro-cinétiques pour l'amélioration des sols de fondation nous semble être d'une importance telle qu'elle gagnerait à être appliquée tant théoriquement que pratiquement par tous les laboratoires qui s'occupent des problèmes relatifs au mouvement des fluides dans le sol.

The author comments on the electro-chemical method as applied in Poland for the treatment of soils. This method is based on research carried out by Messrs. *Haefeli* and *Schaad.* It has been successfully applied in Poland for underpinning, the stabilisation of talus, the construction of vertical earth walls, cuts, shafts through water-bearing sands, and improvement of tree-bearing soils.

Dr. C. L. Dhawan[1])

Soil stabilization is a process by which the soil is made hard, resistant to water and free from volume changes resulting from swelling and shrinkage. The conception of a stabilized soil is similar to that of cement concrete. Therefore a mechanically stabilized soil should have a well-balanced proportion of sand, silt and clay.

The grading proposed for the Punjab (India) alluvial soils, belonging to the kaolinite group is as follows:

Clay = 8 to 15%
Silt = 12 to 25%
Sand = 60 to 80%

The following experiments were conducted to determine the validity of the above specifications:

(i) Effect of soaking in water;
(ii) Stabilization with cement;
(iii) Shrinkage factor.

The results proved conclusively that the above grading was an ideal one for kaolinite type of soils.

In the above paragraphs the effect of grain size distribution on the stability of a soil has been discussed. But we cannot ignore the role of the chemical properties of the soil colloids. It is a matter of common observation that the soils with the same composition of minerals and the same grading behave differently, if one has calcium as its exchangeable base and the other has sodium. The larger the capacity of the clay mass to carry exchangeable bases, the greater is the importance of the effect of the exchangeable bases on the physical properties of soils. Therefore, it becomes necessary that while evaluating the specifications of soil-stabilization, due attention should be paid to the chemical aspect of the question viz. type of clay mineral nature of exchangeable base, nature of salts, humus etc. etc.

L'auteur indique la granulométrie requise pour stabiliser mécaniquement un sol naturellement kaolinitique. Par ailleurs la composition chimique du sol joue un rôle important dans ce phénomène.

Dr W. Fisch

La méthode de la résistivité électrique est appliquée en Suisse depuis 20 ans surtout pour les investigations concernant les problèmes de fondation (profondeur du fond rocheux, nature du recouvrement etc.), la reconnaissance de vallées cachées, les gisements de matériaux de construction et l'étude des nappes aquifères.

Les courbes enregistrées à l'aide de la méthode géoélectrique

[1]) Physical Chemist, Irrigation and Power Research Institute, Amritsar, Punjab, India.

dans les terrains meubles indiquent une extrême sensibilité à la granulométrie et par conséquent à la perméabilité des sols. On peut distinguer deux cas principaux:

Au-dessous de la nappe, par exemple dans les alluvions aquifères, les facteurs dominants sont la minéralisation de l'eau (dureté, chlorures) et la teneur en eau (fonction de la composition granulométrique). La Fig. 8 représente un sommaire statistique de la résistivité des nappes de 7 régions prospectées. Il en résulte qu'en tenant compte de la chimie de l'eau on arrive à distinguer les dépôts de granulation différente. La résistivité varie dans la proportion de 1 à 7 environ du sable fin (section inférieure d'une ordonnée) au gravier sableux (section supérieure de la même ordonnée).

Au-dessus de la nappe, les relations entre la granulométrie et la résistivité sont dominées par la teneur en eau pelliculaire. Or, cette dernière dépend de la surface spécifique qui devient énorme pour les grains minces. En outre, plus les grains sont fins, plus l'eau y reste immobilisée par les forces d'adhésion et, par conséquent, cette eau est très minéralisée et très conductrice. Il s'ensuit que la diminution de la résistivité correspond à une diminution de la taille des grains, comme le montre l'étude d'un éboulement de Flysch (Fig. 9). Dans l'éboulement même la résistivité varie de 2500 à 90 ohm-mètres, ce qui correspond à un matériel allant de blocs de quelques pieds cube à des débris de quelques millimètres enrobés dans de la vase fine, c'est-à-dire de la roche broyée. Il est intéressant de noter que les paquets irréguliers à grain grossier se trouvent uniquement dans la partie supérieure de l'éboulement, où ils ont pu échapper partiellement au concassage, tandis que dans la partie

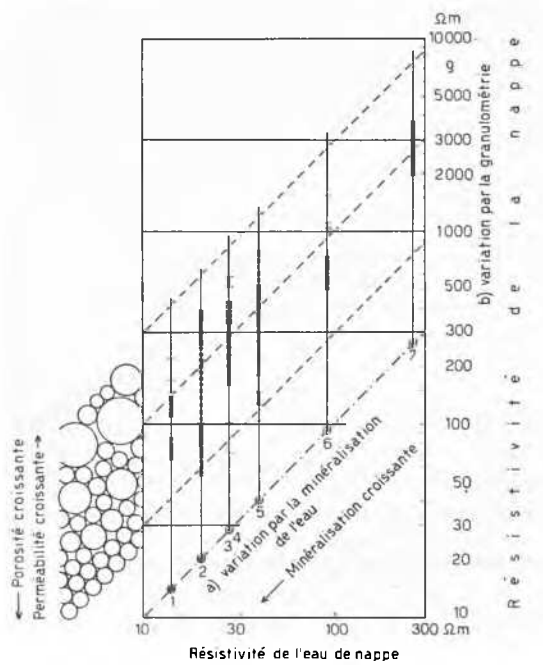

Fig. 8 Alluvions aquifères: variation de la résistivité électrique en fonction *a)* de la minéralisation de l'eau, *b)* de la granulométrie. Régions prospectées: 1 Eiken (Argovie, Suisse), 2 Studen (Berne, Suisse), 3 Delle (Belfort, France), 4 Villnachern (Argovie, Suisse), 5 Rupperswil (Argovie, Suisse), 6 Zwischbergen (Valais, Suisse), 7 Remiremont (Vosges, France)
Water-bearing alluvia: Variation of Electric Resistivity in Relation to (*a*) the Mineral content of Water, (*b*) the Grain Size Distribution. Prospected Areas: 1 Eiken (Argovie, Switzerland), 2 Studen (Berne, Switzerland), 3 Delle (Belfort, France), 4 Villnachern (Argovie, Switzerland), 5 Rupperswil (Argovie, Switzerland), 6 Zwischbergen (Valais, Switzerland), 7 Remiremont (Vosges, France)

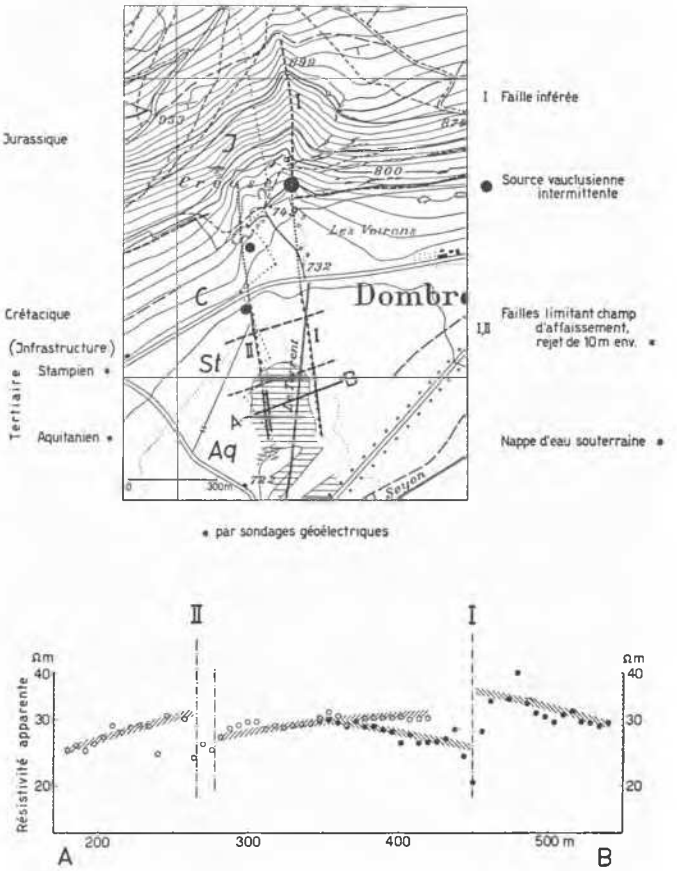

Fig. 10 Reconnaissance géoélectrique de deux failles, limitant un paquet effondré. Profondeur de pénétration 80 m. Val-de-Ruz (Neuchâtel, Suisse)
Electrical Resistivity Survey of Two Hidden Faults Adjoining a Sunken Part. Penetration Depth 80 m. Val-de-Ruz (Neuchâtel, Switzerland)

inférieure, les roches glissant sous la charge des parties supérieures ont été presque complètement broyées. Les résultats ont été vérifiés par des forages et des essais de laboratoire.

Récemment un nouveau système de sondages géoélectriques a été développé et éprouvé. Il permet de constater, même à travers des recouvrements considérables, la présence et la position exacte des failles, des systèmes de fissures et des zones fracturées ou mylonitisées, dont la connaissance est importante en vue des travaux d'injection. La Fig. 10 illustre deux failles jalonnées par des sources et limitant un paquet effondré dont la présence a été enregistrée par le décalage brusque dans l'allure des deux abaques de résistivité. La faille I accuse en outre une résistivité minimal. La profondeur de pénétration de ces sondages est de 80 m environ. Ce procédé de prospection s'applique sur des échelles et avec des pénétrations variées, selon l'objet et le but des études.

L'auteur exprime ses remerciements à la direction de la Société Romande d'Electricité, Montreux-Clarens, et au Service des Ponts et Chaussées du Canton de Neuchâtel qui l'ont aimablement autorisé à publier les résultats ci-dessus.

The author gives some examples of soil resistivity surveys. The electric resistivity of the soil depends on the grain size distribution and, accordingly, on the permeability. Beneath the groundwater table, for instance in water-bearing deposits, the most important factors are the salts concentration and moisture content. Above the groundwater table the relationship between grain size distribution and resistivity is influenced by the water film.

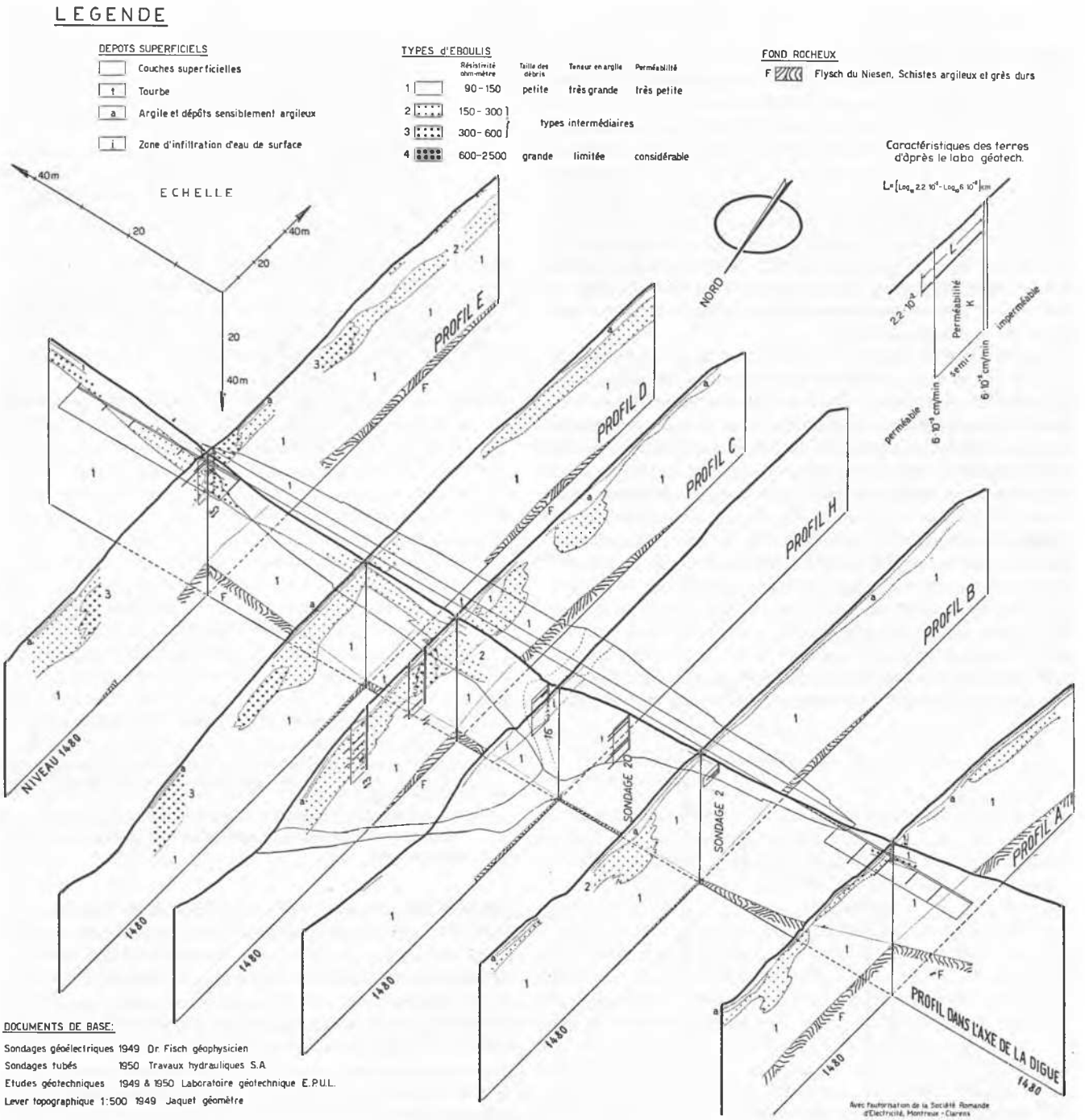

Fig. 9 Surélévation du Lac d'Arnon (Berne, Suisse): composition granulométrique d'un éboulement et profondeur du fond rocheux, d'après sondages géoélectriques
Damming of the Arnon Lake: Grain Sizing of Earth Slide and Depth to Bedrock Deduced by the Electrical Resistivity Method

Dr. K. F. G. Keil

Since my report (Proceedings 1953, vol. I, p. 147) the Hydraton method has been successfully applied in the winter of 1952/1953 to lining the bottom and the slopes of a drainage ditch, depth 10 m, in the brown coal district of Central Germany. The ditch is dug in loam; the slope is 1:1.7. Bottom and slopes are lined with a Hydraton stabilizing carpet (Figs. 11 and 12) made with loess loam (liquid limit: 25%). The carpet is 25 cm thick in order to avoid cracks and tearing, which the lignite mines on both sides of the ditch might cause. This carpet was placed at a water content of 38%. Rainfall (max. 66 mm day) and temperature (min. 0° C) have not affected the work; the carpet is proof against sliding. Tests have shown that Hydraton can just as well be placed under water at the same water content.

Thus it has been demonstrated that Hydraton may be placed in large quantities and in any climate below the optimum moisture content; that it is stable and resists shearing on 1:1.7 slopes in spite of excess pore water pressures.

L'auteur donne un nouvel exemple de l'application d'un sol stabilisé à la construction des digues en terre par la méthode Hydraton.

Figs. 11/12 Hydraton Stabilizing Carpet on the Loam Banks—Slope 1:1.7—of a Drainage Ditch in the Brown Coal District of Central Germany. Loess Loam: Liquid Limit 25%, Water Content 38%
Tapis stabilisateur Hydraton sur digues de limon – pente 1:1.7 – d'un fossé de drainage dans la région des mines de lignite de l'Allemagne Centrale. Limon lœss: limite de liquidité 25%, teneur en eau 38%

Dr. R. Nieder

Strip pits in the Rhineland lignite mining district will go down to 975 feet within the next years. Before the lignite exploitation can be carried out, the ground water level has to be lowered within a large area; this is accomplished by drilling filter wells of corresponding depths. Since boring these wells using the existing drilling methods would be money- and time-consuming, a new drilling system by suction with reversed circulation has been applied. Drilling in an uncased hole which must be filled with water up to ground level characterizes this new drilling method, which requires special drilling rigs. These rigs were originally built by the *Winter Weiss Co.* in Denver (Portadrill Reverse Circulation); now they are also constructed by the Salzgitter Maschinen A.G. at Salzgitter and by the Westdeutsche Bohrgesellschaft in Cologne.

A Portadrill rig has been in operation for one and a half years; also one rig of each of the other two makes has been in operation for half a year. Up to the present time 60 wells of 40 to 46 inches in diameter, ranging from 162 to 380 feet deep, have been drilled by the Portadrill rig in sands and clays of the lignite-bearing formation. In these beds the average drilling progress per working shift of eight hours was nearly 50 feet, which is about 15 times faster than the progress of the earlier wells drilled by "dry" drilling (using drill pipes without circulation).

Five additional wells, ranging from 162 to 442 feet in depth, were drilled in the gravel deposits of the Rhine. These wells showed an average drilling progress of 21 feet per working shift in comparison to about one foot 7½ inches with other drilling systems.

Up to the present each of the Salzgitter and Westbohr rigs has completed 10 wells 40 inches in diameter in the Rhine gravels. The average drilling rate of both rigs is steadily increasing from one well to the next, and at the moment amounts to 50 feet per working shift, which surpasses the Portadrill rig. Advances of 91 feet per working shift and 244 feet per day have been recorded (Fig. 13). The depths reached are particularly worth mentioning: while the Portadrill rig could only reach a depth of 442 feet, the Westbohr and Salzgitter rigs went down considerably deeper, probably much deeper than 1000 feet. The maximum drilling depths recorded up to the present time are 803 (Fig. 13) and 978 feet respectively (Fig. 14).

L'auteur attire l'attention sur des forages sans tubages avec circulation réverse exécutés dans les mines de lignite de la Rhénanie. Des profondeurs allant jusqu'à 326 m ont été atteintes. Les forages sont utilisés comme puits filtrants aux fins d'abaisser le niveau de la nappe phréatique.

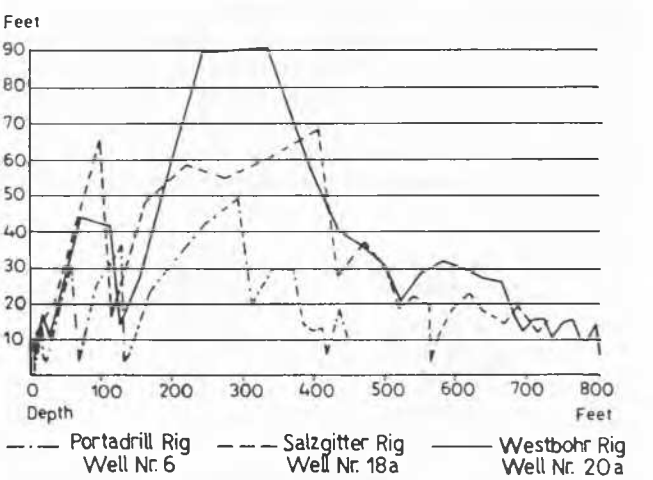

Fig. 13 Drilling Progress per Working Shift, Dirmerzheim
Forage par équipe de travail, Dirmerzheim

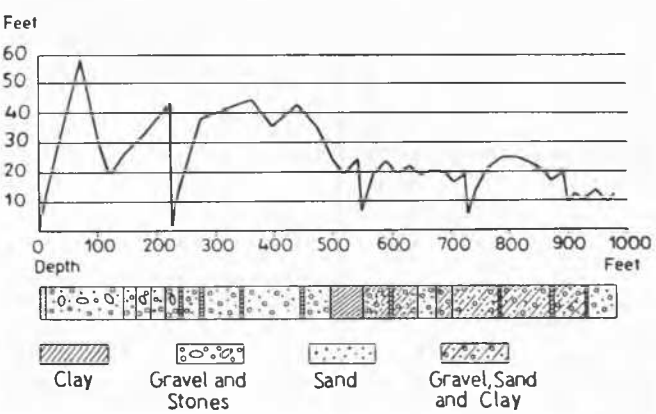

Fig. 14 Drilling Progress per Working Shift with Reference to Type of Formation and Depth in Well No. 24 near Dirmerzheim
Forage par équipe de travail par rapport au type de formation et à la profondeur – Puits No 24 près de Dirmerzheim

Mr. S. Stump

The penetration apparatus developed by the author in 1946 (see Proceedings 1948, vol. III, p. 212) eliminates the skin friction along the rod by means of a casing, and has a broadened point. Besides this, there are in use other systems designed to eliminate the skin friction.

In order to compare the results obtained with two methods of eliminating skin friction, the author has carried out a number of tests in different soils of Switzerland using the dynamic penetration apparatus. These tests were carried out without casing (Fig. 15a) and with casing (Fig. 15b), all these tests being located within a small area. As a control, a boring test was made. The dynamic penetration resistance was calculated according to *Stern*; it could almost as well have been represented by the number of blows for a given penetration. From the results obtained—which were put on show at the exhibition organized at this Third International Conference at Zurich —two typical examples are shown in Figs. 15c and 15d.

Fig. 15c shows the results in a profile in cohesive material. In the tests without casing the total dynamic penetration resistance increases with depth much more than in the test with a casing. This is attributable to a skin adhesion in addition to the point resistance. This means that behind the point in the test without casing, the clay flows back into the space left by the passage of the point, and acts on the rod.

Fig. 15d shows the results of two tests in noncohesive material. There is only little difference between the two tests due to inhomogeneity in the soil. We may conclude that the walls of the hole left along the rod after the passage of the broadened point do not collapse in the tests without casing. Therefore practically no skin friction is exerted on the rod.

If we wish to measure only the point resistance in different soil layers, an apparatus with casing gives results which are easier to interpret.

L'auteur se réfère à des études comparatives sur les résultats obtenus dans des essais de pénétration à l'aide d'une sonde de battage à pointe élargie, avec ou sans tubage. Aucune différence n'a été relevée dans les matériaux pulvérulents; par contre dans les matériaux cohésifs la résistance est plus élevée dans les essais sans tubage, le matériaux exerçant un frottement latéral sur la tige.

Messrs. W. J. Turnbull and R. G. Ahlvin[1])

Considering the small amount of data available on measured stresses in the base and subgrade of flexible pavements, the paper submitted by Mr. *Plantema* (Proceedings 1953, vol. I, p. 289) should be of great interest. Tests of this nature require an elaborate setup of test equipment and tedious work on the analysis of data. Mr. *G. Plantema* is to be commended for this work. For the benefit of engineers interested in stress distribution studies in soil masses, the writers wish to limit their discussion of this paper to a few general comments and to present a brief résumé of the extensive work being conducted at the Waterways Experiment Station, Corps of Engineers.

The distribution of stresses beneath flexible pavements is very complex. The knowledge of vertical stresses beneath the loading plate owing to applied loads on the plate which Mr. *Plantema* has presented here is of considerable value, but some knowledge of the horizontal and shear stresses is necessary to an understanding of the distribution of stresses in a mass. Also, it is necessary to gain an understanding of the magnitudes and effects of stresses caused by the weight of overburden and those "built-in" by compaction and repeated loading.

[1]) Engineer, Chief of Reports and Special Projects Section, Flexible Pavement Branch, Soils Division, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, U.S.A.

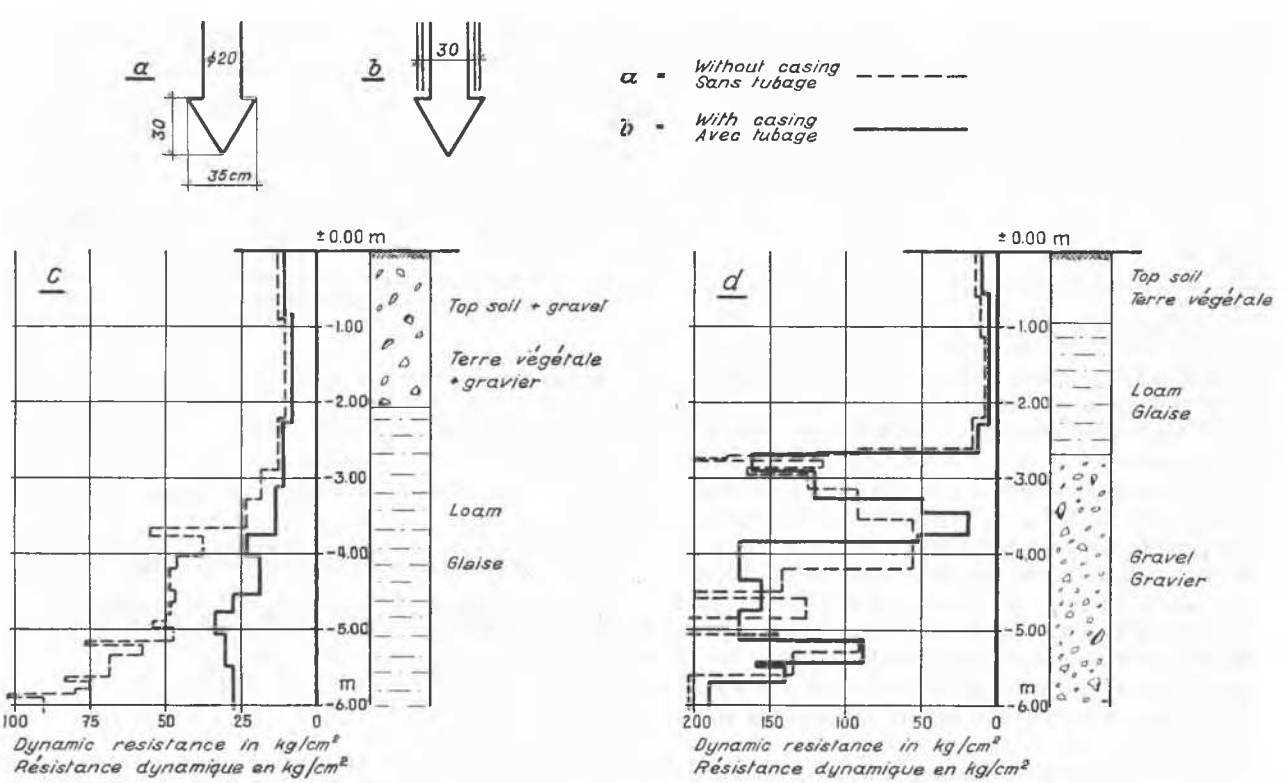

Fig. 15 Dynamic Penetration Tests With and Without Casing
Essais de pénétration dynamique avec et sans tubage

Horizontal stresses as well as vertical have been directly measured at the Waterways Experiment Station using pressure cells. The cells used will hold a fairly constant zero position over a relatively long period of time and can be used to measure "built-in" and overburden stresses as well as those induced by surface loads. Shear stresses have also been determined.

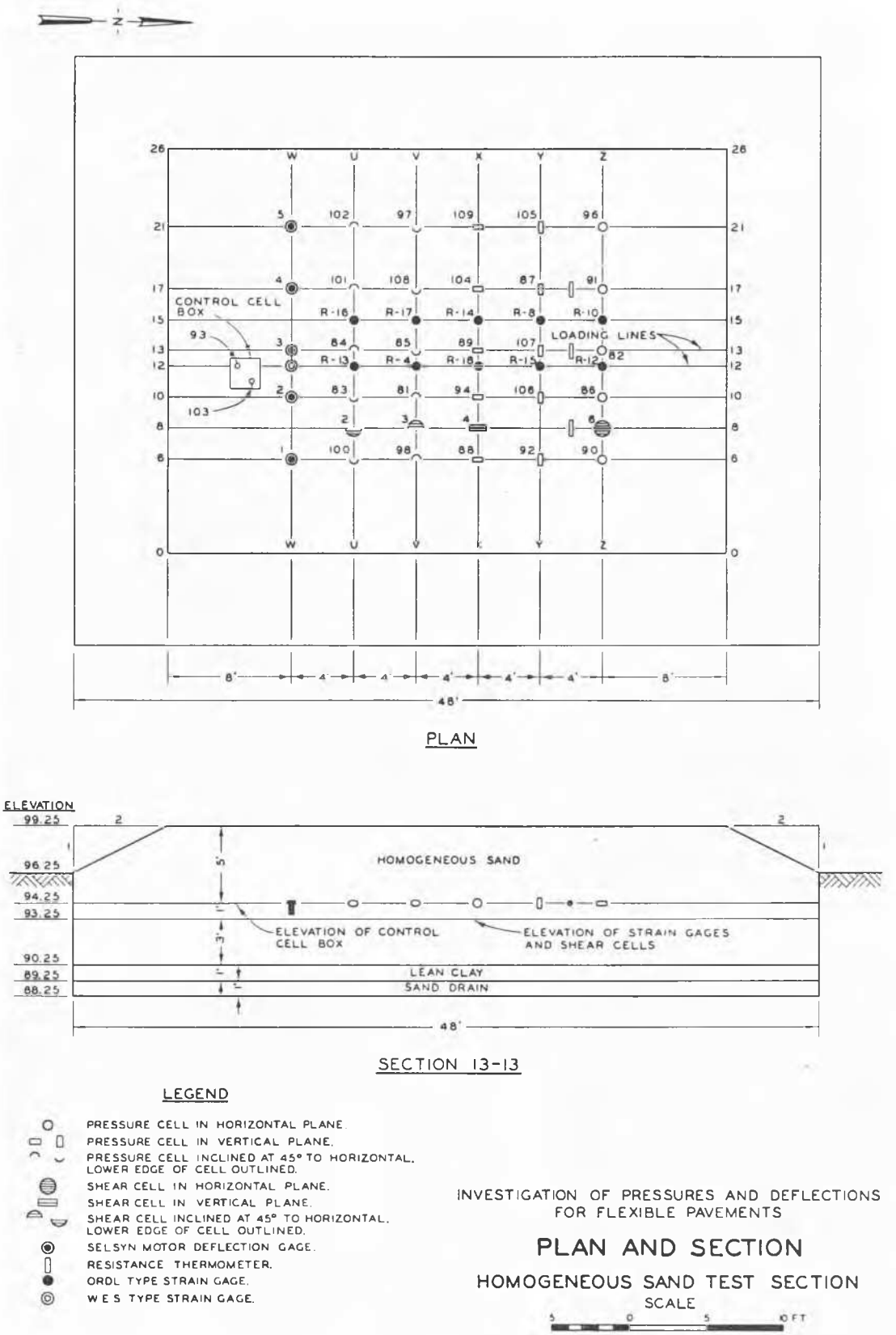

Fig. 16 Homogeneous Sand Test Section
Tronçon d'essai en sable homogène

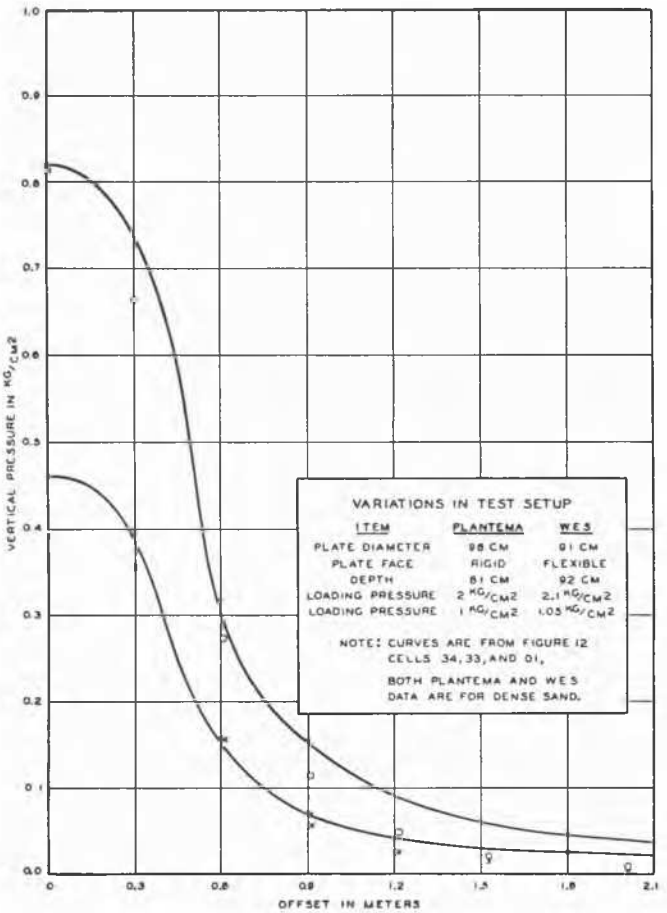

Fig. 17 Comparison of Test Data: Plantema vs. WES
Comparaison des résultats d'essais d'après Plantema et WES

This has been accomplished by placing pairs of pressure cells perpendicular to one another and in similar positions with respect to the load. Half the difference between the measurements from these cells is the shear stress at 45° to them. A few cells have been tried which measure shear stress directly. These cells were suggested by Mr. *R. R. Philippe*, designs suggested by Dr. *D. W. Taylor*, and design details worked out by Dr. *A. C. Ruge*. Results from installations in a homogeneous dry sand have been good.

The Waterways Experiment Station is currently conducting studies similar to Mr. *Plantema*'s on distribution of stresses, strains, and deflections in soils under simulated airplane wheel loads. This is part of a long-range program for improving the present CBR method of design of flexible pavements for airfields and for developing a more rational method of design.

Two test sections have already been constructed and tested, a homogeneous clayey silt section, described in a previous paper,[1]) and a similar section of homogeneous sand, shown in Fig. 16. Some of the results from the clayey silt section have been published[2], [3], [4]); results from the sand section will follow.

Currently, theoretical studies and instrumentation developments are being made prior to the construction of a two-layered (crushed stone on clayey silt) test section. The test data compare reasonably well with theoretical results using *Boussinesq*'s theory.

Figs. 17 and 18 compare some of the test data from the homogeneous sand section with curves from Mr. *Plantema*'s data. There are some variations in the test equipment to be considered when comparing these data. Despite this, there appears to be reasonably good agreement.

Les auteurs, se fondant sur les recherches auxquelles ils travaillent actuellement, ajoutent quelques remarques complémentaires à la communications de M. *G. Plantema* sur la pression dans l'infrastructure des pistes flexibles au cours d'essais de charge. La répartition des charges est compliquée. Les charges horizontales et la force de cisaillement doivent être mesurées; à celles-ci viennent s'ajouter la contrainte due au poids mort, celle induite par la compaction, plus celle causée par les essais de charge. Les résultats de différents essais, illustrés par les Figs. 16–18, sont comparés aux résultats obtenus par M. *Plantema*.

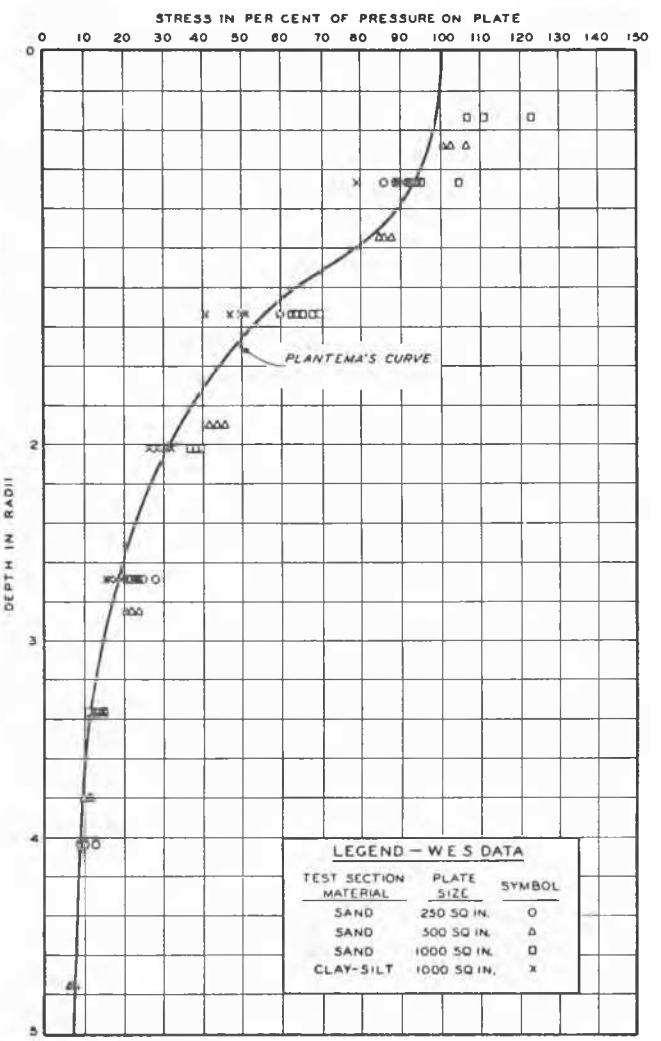

Fig. 18 Vertical Stress under the Centre of a Circular Load
Contrainte verticale au-dessous du centre d'un charge circulaire

[1]) *Turnbull, W. J., Boyd, W. K.*, and *Fergus, S. M.* (1948): Stresses and Displacements in a Homogeneous Soil. Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, vol. V, p. 162.

[2]) Waterways Experiment Station (1951): Investigations of Pressures and Deflections for Flexible Pavements, Report No. 1, Homogeneous Clayey-silt Test Section. TM 3–323, March.

[3]) *Foster, C. R.*, and *Fergus, S. M.* (1951): Stress Distribution in a Homogeneous Soil. Highway Research Board, Research Report No. 12-F, January.

[4]) *Foster, C. R.*, and *Fergus, S. M.* (1950): Supplement to Stress Distribution in a Homogeneous Soil. Highway Research Board Proceedings, p. 175.