Stability and Deformations of Slopes and Earth Dams, Research on Pore-Pressure Measurements, Groundwater Problems

Stabilité des talus et des digues en terre, pression de l'eau interstitielle, problèmes se rattachant aux nappes phréatiques

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Oral Discussion / Discussion orale:

MM. *A. Mayer*, *R. L'Herminier* et *P. Habib*, présenté par M. *A. Mayer*, France
Mr. *R. Peterson*, Canada (presented by Mr. *W. Schriever*)
Prof. *L. Šuklje*, Jugoslavia
Mr. *W. G. Holtz*, U.S.A.
Mr. *T. Middlebrooks*, U.S.A.
Mr. *B. Löfquist*, Sweden
Mr. *A. L. Little*, Great Britain
Mr. *A. Penman*, Great Britain
Mr. *F. C. Walker*, U.S.A.
Mr. *Ch. I. Mansur*, U.S.A. (presented by Mr. *W. J. Turnbull*)
Prof. *K. Terzaghi*, U.S.A.

Written Discussion / Discussion par écrit:

M. *J. Abollado*, Spain
Prof. *Z. Bazant*, Jr., Czechoslovakia
Mr. *T. Edelman*, Netherlands
Mr. *A. Lazard*, France
Messrs. *C. I. Mansur* and *I. R. Compton*, U.S.A.
M. *U. Nascimento*, Portugal
M. *B. Rajčević*, Yougoslavie
Mr. *A. Wackernagel*, Switzerland

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General Reporter
Session 8
Rapporteur général
Session 8

The General Reporter

Professor *Meyer-Peter*'s clear presentation of the difficulties encountered in the design of the Marmorera dam forms an excellent introduction to this afternoon's session, demonstrating the importance of the different subjects which we have to discuss.

The first part of our discussion deals with the stability of slopes. I have suggested that the discussion should be concentrated on the limitations of the $\varphi = 0$ analysis and the methods which should be applied in cases where the $\varphi = 0$ analysis does not form a reliable basis for the design of slopes.

The next question concerns the stability of earth dams. I will briefly mention a few of the most important points which are related to this problem.

If we are in the situation where we need to calculate the stability of an earth dam, we have to decide if the analysis is to be carried out in terms of total stresses—which means that we directly use the shear strength found by consolidated un-

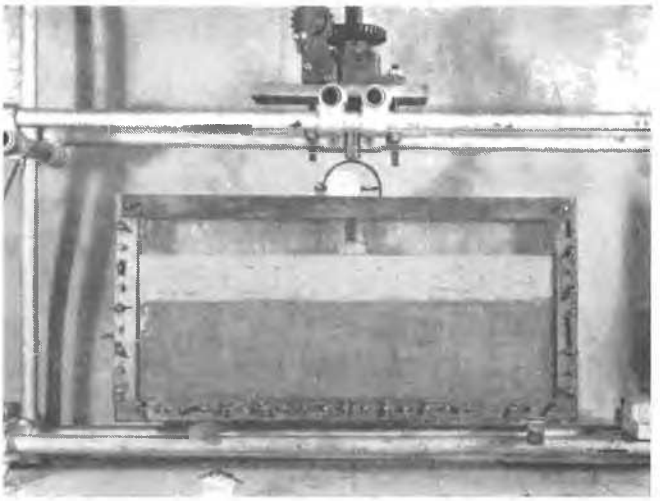

Fig. 1 Appareillage pour les essais de fondation sur double couche
Apparatus for Two-Layer Foundation Tests

drained tests—or in terms of effective stresses—using drained tests or consolidated undrained tests with pore pressure measurements.

It is my personal opinion that only an analysis in terms of effective stresses will lead to the right result. This involves, however, the difficulty of having to estimate the pore water pressures for the different cases which should be covered by the stability analysis.

For this reason I propose for discussion the pore water pressures during construction, during rapid drawdown, and I would like to add a specially dangerous condition, namely a rapid draw-down in soil with dissolved air in the pore water.

A question which is related to the stability analysis is which placement water content should be aimed at during compaction of impervious fills in earth dams.

This questions seems to have divided earth dam-engineers into two groups, a dry group and a wet group. From both groups contributions to the discussion will be given and I hope that we thereby will obtain more information about the experiences on which the two different points of view are based.

The fourth part of our discussion is related to seepage and ground-water problems. I do not believe that much fundamental progress has been made since 1948 on this question, but very valuable field information has been published.

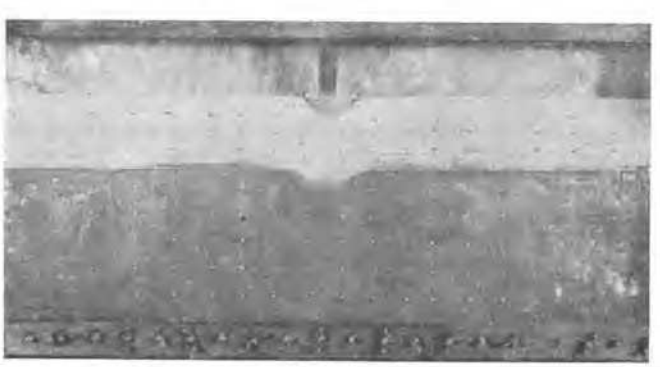

Fig. 2 Photographie avec exposition continue pendant le poinçonnement. Les deux plans de rupture verticaux sont visibles dans la partie centrale au droit des bords de la fondation
Continuous Exposure Photograph During Punching. The Two Vertical Planes of Rupture are Visible in the Central Part Beneath the Foundation Boundaries

I hope that the discussion will be lively despite the fact that this is the last session of the Conference.

Le rapporteur général propose de concentrer la discussion sur les thèmes suivants: la validité de l'analyse $\varphi = 0$, les pressions interstitielles, la teneur en eau lors de la mise en place et les problèmes de filtration.

MM. A. Mayer, R. L'Herminier et P. Habib (présenté par M. Mayer)

Dans sa communication lors de la session 4, le 19 août, M. *Button* (de Grande-Bretagne) a posé la question des fondations sur double couche de matériaux de caractéristiques mécaniques différentes.

M. *Buisson*, Rapporteur général, a bien voulu indiquer que la question pouvait se traiter sur modèle réduit et nous avons précisé que des essais avaient déjà été faits au Laboratoire du Bâtiment et des Travaux Publics à Paris. – Faute de temps, il n'a pas été possible de donner, lors de la discussion de la session 4, les résultats des essais. Le fait cependant qu'ils ont été entrepris à l'occasion d'accidents de digues fondées sur des couches superposées de sable et de vase nous a suggéré d'en exposer les résultats à l'occasion de l'étude de la stabilité des talus et des digues.

Plusieurs accidents avaient eu lieu, où le mur de quai, fondé sur une couche de sable remplissant une souille de vase, s'était affaissé de plusieurs mètres, verticalement, dans des conditions qui rendaient impossible l'application des méthodes d'études usuelles par les surfaces de glissement.

On décida de faire des essais sur modèle réduit en utilisant deux couches, la couche supérieure constituée par du sable fin, la couche inférieure par une graisse consistante de coefficient de frottement nul.[1]) Ces essais ont été confiés à M. *Tcheng* qui les a réalisés à la suite de ceux exposés dans la communication (Comptes Rendus 1953, vol. II, p. 283).

Le principe de l'essai est indiqué sur les Fig. 1 et 2.

Aussi longtemps que l'épaisseur de la couche de sable n'atteint pas plusieurs fois le diamètre de la charge appliquée en surface, le cisaillement du sable se produit suivant deux plans verticaux et la couche plastique inférieure est poinçonnée comme par un mandrin ayant pour section droite celle de la plaque chargée (Fig. 2). Ce type de rupture se produit pour des épaisseurs de sable largement supérieures à celle nécessaire à la formation des courbes classiques du type spirale logarithmique. Ce mode de rupture, à notre connaissance, n'a jamais été décrit. Lorsque l'épaisseur du sable atteint 4 à 5 fois le diamètre de la fondation, on retrouve les surfaces classiques (*Peynircioglu*, Comptes Rendus 1948). Le taux de travail est alors égal à celui que l'on obtient sur une couche de sable semi-infinie.

Des essais sont en cours pour obtenir des indications quantitatives sur les conditions de répartition d'une charge par une couche de sable ou de matériaux sans cohésion surmontant une couche plastique. Mais, dès à présent, on peut considérer comme certain que la répartition à 45°, habituellement adoptée, est trop optimiste.

Cette observation peut comporter de très nombreuses applications: en premier lieu les murs de quai fondés sur massifs en enrochements assis sur du sable remplissant une souille creusée dans une vase molle, qui ont été à l'origine de cette étude, et aussi les essais de module de réaction effectués sur

[1]) Le principe des maquettes en graisse consistante pour l'étude de la rupture lorsque la résistance au cisaillement est constante est du à M. *Frontard*.

une fondation de matériaux graveleux assise sur une plateforme d'argile ou de limon. L'amélioration due à la présence de la couche de fondation pourra, de façon systématique, ne pas être aussi importante qu'on aurait pu l'espérer.

The authors treat the problem of the foundation of dikes on a layer consisting of two different materials. Model tests have been carried out using sand for the upper layer, and grease for the lower layer. The results obtained show that the usually accepted 45° spreading of load is too optimistic.

Mr. R. Peterson[1]) (presented by Mr. W. Schriever)

The $\varphi = 0$ method of analysis is favoured for stability studies in clay by the P.F.R.A. in connection with its programme of water development projects in Western Canada. Although there have been only a limited number of opportunities to check this method against actual behaviour, it has been noted that in several cases failure has occurred where there was a calculated factor of safety in excess of 1.5. These cases have been associated with saturated highly plastic clays and it appears that such behaviour is in line with the findings of *Casagrande* and *Wilson* as noted by the reporter. They found that under sustained loading the stress required to cause failure in clays and clay shales is considerably less than the strength indicated by normal laboratory compression tests. The following is an experience with earth dykes on a highly plastic clay foundation near Winnipeg, Canada.

The project involved about 8 miles of dyke up to 20 ft. in height, consisting of a clay core with granular shoulders. The foundation was a highly plastic clay with a liquid limit of 75 to 100 and a plasticity index of 55 to 70 with a natural water content varying from 35 to 60%. While the dykes were designed mainly on the basis of general considerations, the stability was checked assuming an arc through the core and the foundation, using the $\varphi = 0$ method with the strength based on unconfined compression tests. The analysis indicated that the minimum factor of safety was about 1.5. No movements of the completed dykes occurred immediately following construction and it was therefore assumed that the dykes would be stable. However, several failures occurred from 6 months to 4 years following construction. These were on arcs through the core and the foundation with each involving a length of embankment 100 ft. to 400 ft. In the areas where failures occurred, it is believed that the computed factor of safety was of the general order of 1.5 to 2.5. Following this experience and the report of studies of *Casagrande* and *Wilson*, a few creep tests involving sustained loads have been carried out on highly plastic clay. The results indicate strength losses of at least 50% of the normal value, a trend very similar to that reported by *Casagrande* and *Wilson*. This would seem to be the most logical explanation for the delayed failures in the field.

In view of the above and until such time as further studies have been carried out it would seem that high computed safety factors are justified for this type of material.

M. *W. Schriever* donne lecture d'une communication de M. *R. Peterson* concernant la rupture de digues reposant sur des argiles plastiques. Ces digues ont été calculées sur la base de la méthode d'analyse $\varphi = 0$, le coefficient de sécurité calculé dépasse 1,5. Des désordres se sont produits de 6 mois à 4 ans après la construction. Se basant sur quelques essais l'auteur attribue ces ruptures au fluage qui, parfois, diminue jusqu'à 50% la résistance au cisaillement.

[1]) Prairie Farm Rehabilitation Administration, Saskatoon, Canada

Prof. L. Šuklje

Le Rapporteur général M. *Bjerrum* a proposé que la discussion porte sur les limites de la validité de la méthode $\varphi = 0$. Dans les nombreuses analyses de glissements exécutées en Yougoslavie, au Laboratoire des Sols à l'ETS à Ljubljana, nous n'avons pas eu souvent l'occasion de recourir aux avantages de la méthode $\varphi = 0$. Il s'agissait, cependant, dans la plupart des cas de glissements ayant lieu dans des couches désagrégées et décomposées de marnes, de grès, de schistes, etc., donc dans des sols pour lesquels l'application de la méthode $\varphi = 0$ n'a jamais été recommandée. Pour les glissements très fréquents dans le flysch éocène nous avons fait l'expérience suivante:

Si nous mettons des échantillons de cette argile limoneuse et sableuse, produite par la désagrégation et par la décomposition dans les appareils de cisaillement, à l'état remanié près de la limite de liquidité, nous constatons par un essai demi-lent – c'est-à-dire avec une graduation de la contrainte tangentielle de $\sigma/40$ toutes les cinq minutes – un angle de frottement qui varie, selon la composition minéralogique des sols, de 16 à 26°. Les valeurs moyennes des échantillons pris tout près des surfaces de glissement varient entre 16 et 20°; les valeurs correspondantes de la limite de liquidité s'élèvent de 52 à 54%, et exceptionnellement jusqu'à 108% – pour les matériaux contenant de la terra rossa; les valeurs de l'indice de plasticité sont de 29 à 31, resp. 74%. La résistance au cisaillement des mottes isolées correspond, par rapport à cet angle de cisaillement, aux pressions qui dépassent les pressions géologiques actuelles. Cependant les sols glissent, souvent sur des pentes ayant une inclinaison de 13 ou, encore sur des pentes moins inclinées, si la surcharge produite par les remblais cause les conditions de glissement équivalentes. On ne peut pas arriver dans le domaine des conditions de glissement déterminées par les essais géotechniques, même ceux exécutés avec des échantillons remaniés, autrement qu'en tenant compte de la souspression et de la pression du courant de l'eau. Comme il y a dans cette masse argileuse, d'ailleurs peu perméable, de nombreux passages où l'eau peut circuler, et, comme nous l'avons constaté quelquefois, une saturation évidente des sols, un tel effet d'eau souterraine est probable.

Les surfaces de glissement sont très clairement développées, ordinairement près de la limite inférieure de la zone de désagrégation, c'est-à-dire près des couches solides qui, dans les domaines des glissements, se trouvent généralement à une profondeur de 6 à 12 m. Quelquefois un secteur important des surfaces de glissement est à peu près parallèle à la surface de la pente; elles ont cependant, lorsqu'elles se forment sous l'influence de la surcharge des remblais, une forme circulaire assez régulière. Pour éviter la résistance plus élevée des mottes isolées, les surfaces de glissement cherchent des voies de moindre résistance entre les mottes et prennent alors une forme ondulée. La mise en action de la résistance au cisaillement totale qui, dans les différentes parties de la surface de glissement et dans un sol hétérogène n'est pas simultanée, peut, de même, jouer un rôle non négligeable.

Une correspondance très nette entre les périodes de pluies et les vitesses de déplacement fut constatée, bien que la surface de glissement ait été très profonde, dans le cas du grand glissement enregistré à Zalesina (Croatie), développé dans des couches de Raibl supérieur. La supposition que la résistance au cisaillement est constante n'a pu être appliquée, même pas aux analyses des glissements produits dans les argiles maritimes tertiaires. Bien que ces argiles soient assez homogènes, leur cohésion diminue considérablement dans la direction de la sur-

face. En outre, elle peut être influencée, par exemple, par une mince couche sableuse.

En ce qui concerne l'effet exercé par la pression du courant ou la sous-pression d'eau, nous avons observé leur efficacité très nette sur les bords graveleux des retenues de la Drave, où les pentes naturelles, ayant une inclinaison entre 38 et 42°, ont été érodées par les oscillations quotidiennes du niveau de l'eau. La perméabilité du gravier n'était pas suffisante pour empêche l'effet de la pression retardée du courant de l'eau.

En général, les expériences acquises montrent que l'eau souterraine peut en réalité réduire à rien la stabilité des talus, non seulement par la dissolution des forces de cohésion (adhésion), mais aussi par l'effet mécanique de la sous-pression ou de la pression du courant. Si l'on constate que les calculs géomécaniques donnent, pour l'appréciation des tassements, des valeurs plutôt pessimistes, il faut constater que dans beaucoup de cas, il serait difficile de prévoir, sur la base d'examens superficiels des sols, les glissements qui se sont produits. Le calcul montrerait une sécurité qui n'existe pas.

Il faut donc choisir avec prudence les bases des calculs qui servent à l'appréciation de la stabilité des constructions aussi importantes que le sont les digues en terre. Dans les cas où l'effet mécanique de l'eau interstitielle n'est pas suffisamment clair, il convient d'appliquer des hypothèses et des méthodes de calcul plus pessimistes. Il faudra, sous le même point de vue, reconsidérer également l'application de la méthode $\varphi = 0$.

The author explains that most of the landslips in Jugoslavia occur in heterogeneous clay soils, to which the $\varphi = 0$ analyses cannot be applied with success. The shearing strength of the slopes can be reduced to the values which correspond only to actual effective pressures. It is of great importance to include the influence of pore-water pressure and the pressure of the flowing water within the massif.

Mr. W. G. Holtz

There were a few points on the question of pore pressures as measured by the United States Bureau of Reclamation which the Reporter brought out. During this discussion session, Mr. *Walker* will handle the field measurements of pore pressures; I will discuss for a moment the question of the laboratory pore pressure tests. Let me read the Reporter's comment: "In the Bureau of Reclamation *Earth Manual* (1951), a pore pressure apparatus for the direct laboratory measurements of construction pore pressures on sealed samples, is described. However, the samples are loaded with hydrostatic pressures; as it is known that the principal stress ratio in the dam is far below unity, this procedure may lead to much too high values of pore one, this procedure may lead to much too high values of pore pressures, particularly for soils compacted at relatively low moisture contents."

May I explain that the pore pressure test should not be confused in any way with the triaxial shear test or with pore pressure measurements for the triaxial shear test. The original purpose of the pore pressure test described in the Manual was to study the volume change versus pore pressure characteristics, as a check on the validity of our method of estimating pore pressures by the use of *Boyle*'s and *Henry*'s laws. This method was described by *Hilf* in the last conference proceedings and has been described by other Bureau authors in several publications[1]) since 1939. Very good correlations were obtained between our Laboratory pore pressure test and our method of computing the pore pressure from volume change.

We really do not use this test to a great extent any more, because it has fulfilled its research value. We now use the one-dimensional consolidation test to compute the pore pressure values required for design and construction control purposes. The pore pressure test is used only to study the changes of volume versus pore pressure on sealed soil specimens, and the various loads are applied by hydrostatic pressure all around the sealed specimen. The method of applying the stresses and the stress distributions are really of no consequence in the test, as only the volume changes, and their relation to the resulting pore pressures were desired.

While I am at the speakers' stand, I would like to discuss another matter that I have been thinking about since Dr. *Terzaghi* talked about "soil creep" last Tuesday. The same thing has just been mentioned by Mr. *Schriever*, who has just presented Mr. *Peterson*'s discussion. As you may recall, Dr. *Terzaghi* mentioned that in some cases creep started when the stresses were only about 50% of the maximum deviator stresses measured in the laboratory. While our triaxial shear research has not been directed towards the study of creep, some of the results that we have obtained may be applicable to creep studies. We have established in the laboratory that the maximum pore pressure occurs at the minimum volume point, as it should. At this point, of course, the effective minor principal stress is also a minimum. It has been observed that for several types of plastic soils at various moisture conditions, the minimum volume may occur at some fraction of the maximum deviator stress. In some instances the minimum volume condition has occurred at less than one-half of the maximum deviator stress. Beyond this point, the volume of the specimen increases. As shearing movement takes place as additional stresses are applied, we might say that the shear failure has already begun. Perhaps it is near the stress condition corresponding to the minimum volume condition that creep may start. We have considerable laboratory data, some of which has been published by other societies,[2]) that can be studied if any of you are interested.

L'auteur répondant à une question du rapporteur général expose que le Bureau of Reclamation effectue la mesure de la pression interstitielle en fonction des variations de volume. Ces mesures sont faites à l'aide d'un œdomètre. En outre les résultats obtenus dans les laboratoires du Bureau of Reclamation indiquent que le fluage commence probablement sous l'effet des tensions qui correspondent aux conditions du volume minimum.

Mr. T. Middlebrooks

Your General Reporter has asked me to say a few words on the policy of the U.S. Corps of Engineers, concerning the placement moisture and density in earth dams.

Safety and economy are the prime controlling factors. After safety has been assured, we strive to build the most economical structure as possible. This means that placement moisture and density will vary with field conditions. Dr. *Löfquist* of Sweden has an excellent example which he is presenting in his discussion. In his case the core material was too wet for normal compaction. It was placed at a high moisture content and rolled

[1]) See also: *L. W. Hamilton* (1939): The Effects of Internal Hydrostatic Pressure on the Shearing Strength of Soils. Proc. A.S.T.M., vol. 39, p. 1100; and *W. G. Holtz* (1948): The Determination of Limits for the Control of Placement Moisture in High Rolled-Earth Dams. Proc. A.S.T.M., vol. 48, p. 1240.

[2]) *W. G. Holtz* (1947): The Use of the Maximum Principal Stress Ratio as the Failure Criterion in Evaluating Triaxial Shear Tests on Earth Materials. Proc. A.S.T.M., vol. 47, p. 1067, 1947); and *A. A. Wagner* (1950): Shear Characteristics of Remolded Earth Materials. A.S.T.M., Symposium on Triaxial Testing of Soils and Bituminous Mixtures, S.T.P. No. 106, p. 192, 1950.

only with a tractor, and careful pore pressure observations were made during and after the construction.

In addition to the regular criteria relating to stability and permeability we give special attention to the core material in order that it may satisfy the following criteria:

(1) It must be placed at a density and at a moisture, which will not allow further consolidation on saturation.

(2) It must be sufficiently plastic so that differential settlement will not cause cracks to develop through it.

Both of these criteria can be easily satisfied when the core material is placed well on the wet side of an optimum. In fact this is the only way criterion 2 can be fully satisfied.

You will certainly ask: what about pore pressure? My answer is that it is just another factor which must be considered in any design. It is not an overriding or controlling factor. My suggestion is that in cases where pore pressure will decrease the stability, that the previously mentioned criteria be applied to only the central portion of the core or impervious section, say at width of $\frac{1}{3}$ to $\frac{1}{4}$ the height of the dam.

It is my opinion that the danger of developing excessive pore pressures in modern rolled fill dams has been greatly overemphazised. I assure you that the danger of placing the fill on the dry side of optimum can be much greater. It is a simple matter to install piezometers and observe the pore pressure as it develops. The dangers inherent in a dry fill are not apparent in most cases until the reservoir is full.

In answer to the General Reporter's specific question, on what modifications in cross sections are contemplated by the Corps of Engineers, due to placements on the wet side, my answer is none. We will check the stability during construction more closely and install more piezometers. Since any pore pressure which develops will dissipate quickly after construction is completed, a lower factor of safety is tolerated during this period. Effective stresses are used in the stability analysis for all cases.

Before closing I would like to make a few comments concerning Mr. *Walker*'s paper, on the design of earth dams for pervious foundations.

I regret that I must strongly disagree with my American colleague, concerning *Bligh's* and *Lane*'s theory in the design of earth dams on pervious foundation.

It might be admitted that *Bligh*'s Creep theory was of value in 1916 even though at that time *Darcy*'s law was available and would have given much more useful information. It is also considered possible that *Lane*'s weighted creep theory may have had some application to concrete structures in 1935. However, in the year 1953, the science of Soil Mechanics has developed to such an extent that these rule-of-thumb theories have no

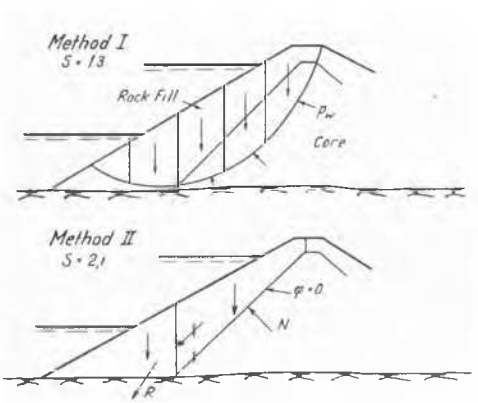

Fig. 3 Two Analysis Methods for a Rock Fill Dam
Deux méthodes d'analyse d'un barrage en enrochement

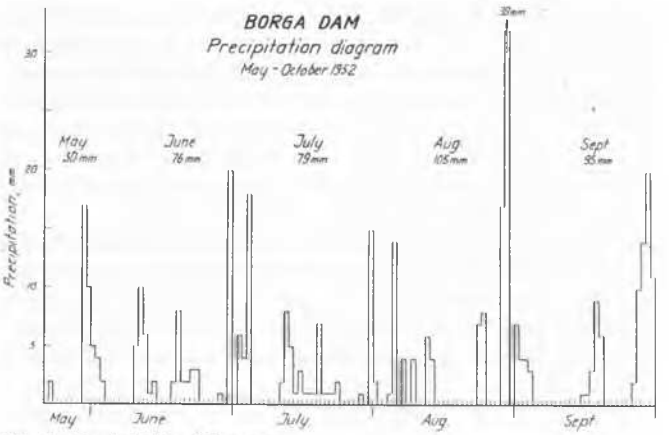

Fig. 4 Precipitation Diagram
Diagramme indiquant les précipitations

place in modern earth dam design. Even in 1935 *Creager*, *Harza*, and *Justin* considered that *Lane*'s creep theory had only limited application. *Casagrande*, in his discussion of the paper, pointed out that the use of creep theory could give misleading results and he outlined sound scientific principles which could be used.

L'auteur est d'avis que le noyau d'un barrage en terre doit être mis en place à une densité et une teneur en eau telles qu'il n'en résulte aucune consolidation pendant la saturation. En même temps il est nécessaire que le noyau soit suffisamment plastique afin qu'aucune fissuration n'apparaisse à la suite de tassements différentiels.

En matière de pression interstitielle l'auteur est d'avis que cette question n'est pas de première importance. Pour les stades de construction on peut calculer un coefficient de sécurité inférieur à celui de l'ouvrage définitif.

En réponse à une question posée par M. *Walker*, sur la fondation des barrages en terre sur des couches perméables, l'auteur est d'avis que la méthode de Bligh et Lane est dépassée aujourd'hui.

Mr. B. Löfquist

I have two subjects for discussion. Firstly I wish to draw attention to the paper by Mr. *Nonveiller* "The stability of dam slopes composed of heterogeneous material" (Proceedings 1953, vol. II, p. 268).

When using the Swedish circle method for analysing the stability of the upstream slope during rapid draw-down, the distribution of the normal stresses along the slip surface ought to be considered in some cases. I will not go into details, but will give an example.

Fig. 3 shows a rock fill dam with an impervious earth core. An analysis according to usual methods gives a safety factor of 1.3 (method I in Fig. 3). Due to pore water pressure, the shearing resistance is very low in the core. Stabilizing forces are mainly mobilized in the lower part of the slip circle in the rock fill.

In the analysis in the figure below the simple assumption is made that no shearing resistance exists in the boundary surface of the core. The weight of the upper right part of the rock fill increases the normal pressure on the lower horizontal slip plane. The safety factor is 2.1.

A detailed investigation may result in a somewhat lower factor than 2.1, but the example shows that the stability of the upstream slope in such dams is more favorable than is evident from the analysis methods usually applied.

The second subject is "Placement moisture content". In most parts of Scandinavia the construction season for earth

dams is about 6 months. The summer climate, however, is often wet and the natural moisture content of the impervious earth is high. Fig. 4 shows a precipitation diagram for an actual dam site. During the few days without rain the air was humid and the wet fill could not dry out. Under these conditions the normal rolled fill method is not applicable. An earth core with alternating dry layers and wet layers is dangerous in view of possible uneven settlements.

Therefore during the last two years the Swedish State Power Board has applied a wet fill construction method which renders possible continuation of the work even during rain. The principles are given in Fig. 5. The upper part of the figure shows compaction curves by various kinds of compaction for a typical

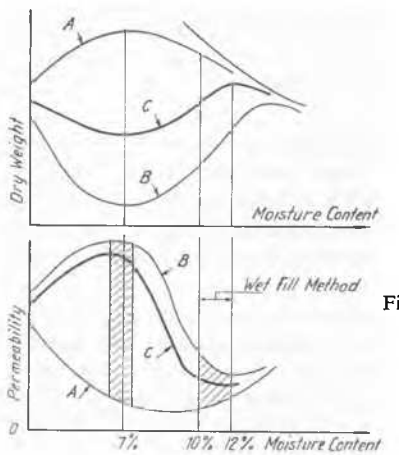

Fig. 5 Dry Density and Permeability vs. Water Content by Different Compaction
Poids volumétrique sec et perméabilité en fonction de la teneur en eau

impervious moraine with practically no clay content. It is similar to the material in the core of the Marmorera Dam about which we have heard in the lecture of Prof. *Meyer-Peter*.

Compaction with sheepsfootrollers (upper curve A) gives an optimum water content of 7%. At this water content the dry density has a minimum if the soil is not at all compacted (lower curve B); a maximum exists, however, at about 14% for this curve. A heavy bulldozer gives the compaction curve C with an optimum moisture content of 12%. The wet fill method presupposes that the placement moisture content is held at about this value and that the soil is compacted only with a heavy bulldozer.

In fact, this method is advantageous also in other respects. The lower part of the diagram shows the influence on permeability. The permeability is about the same at the higher water content as at the Proctor optimum. Most important is, however, that the possible range of variations in permeability is much smaller with the wet fill method than with the rolled fill method. If a layer in the core is not compacted, the permeability still remains good. The earth core will be much more homogeneous than by any dryer method.

The pore water pressure is initially about 100%, but owing to the comparatively high permeability, the consolidation time is short. In two dams 75 and 100 ft. in height the consolidation time was only 1–2 months.

The total settlement in laboratory tests is 4–5%, but as a result of the rapid consolidation the completed dam settles only about 0.2%.

The wet fill construction method is suitable in wet climates for dams of small and medium height. The core width ought to be restricted in consideration of consolidation and of stability, and also because of the fact that trucks cannot be driven on the core fill.

Many engineers object to a moisture content so far above the Proctor optimum value, but my opinion is that the wet fill method is suitable for conditions such as are prevalent in great parts of Scandinavia and in many other places.

Se référant à l'article de M. *Nonveiller* l'auteur expose les raisons pour lesquelles il considère la méthode suédoise préférable, pour le calcul du talus amont d'un barrage en terre hétérogène, aux méthodes analytiques couramment appliquées (Fig. 3).

En raison des conditions climatiques de la Suède (Fig. 4), les ingénieurs se sont vu récemment contraints de compacter le matériau du noyau des barrages en terre de petite et de moyenne grandeur avec une teneur en eau supérieure à l'optimum de Proctor et avec des bulldozers lourds. Les résultats sont illustrés par la Fig. 5.

Mr. A. L. Little

I should like to draw attention to two cases of pore water pressure measurements being carried out in Britain.

In case A the measurements were made in the slopes of a dam at present under construction in Wales. The dam which has a puddle core is situated in a glaciated valley and is being built of the local boulder clay; the matrix of this material having a liquid limit of 25% with a plastic range of 8%. The material is being compacted so that the matrix has an average dry density of 128 lbs./ft³ which is 95% of the Proctor maximum dry density for the matrix only, and at a moisture content of 12% compared with the Proctor optimum of 10%; i.e. the fill is wetter than optimum as placed.

Pore pressure cells were installed by the Building Research Station in the down-stream slope last summer. Construction work ceased in October 1952 for the winter.

During building, as material was placed, the pore water pressure immediately increased to nearly 100% of the total pressure. This pressure then rapidly dropped to about 70% of the total pressure. Thereafter observations of the pore water pressures were continued and it was found that the dissipation of pressure was much slower than had been anticipated. To obtain more information, simple stand-pipes were installed. The readings from the stand-pipes were found to agree very well with the readings of the pressure cells. Investigations were made on the stability of the bank, under the guidance of Prof. *Skempton*, using the measured values of pore water pressure. Calculated values of pore water pressure from laboratory measurements of soil properties were produced, which agreed with the observed values.

It is interesting to compare these results some for another earth dam being built in Scotland. This second dam which has an articulated concrete core is also being built of boulder clay, but of a more permeable type. The liquid limit of the whole material could not be determined because of the large number of stones which it contained, but the liquid limit of the matrix material is appreciably lower than in the previous case.

In this second dam a similar installation of pressure cells and stand-pipes has been made, but no appreciable pore water pressure has been recorded. Indeed, water poured into the stand-pipes drained away quite rapidly.

In this case the compaction gave a matrix dry density of about 130 lbs./ft³, which was 95% of Proctor, and at a moisture content of 7% compared with the optimum of 8%; in other words the fill is dryer than optimum.

It is thus interesting to see that slightly unfavourable conditions of placing may have a serious effect, at any rate, when the fill is of a clay type with a relatively low permeability. These results illustrate very well the "tight-rope walk" referred to by Mr. *Bjerrum* in his general report.

L'auteur donne quelques indications sur des observations de pression interstitielle faites sur deux barrages en terre en Grande-Bretagne, dont les noyaux sont constitués de moraines de fonds. Dans l'un (teneur en eau 12%, optimum de Proctor 10%) la pression est montée jusqu'à 100% de la pression totale pour tomber bientôt à 70%. Dans l'autre (teneur en eau 7%, optimum de Proctor 8%) on n'a pas constaté de pression interstitielle.

Mr. A. Penman

The Piezometer Apparatus just mentioned by Mr. *Little* was developed at the Building Research Station in England. It closely follows that used by the U.S. Bureau of Reclamation, and consists of a piezometer point which is buried in the fill of the earth dam, and is connected by polythene tube (3 mm bore × 1 mm wall) to a Gauge House outside the dam. Pore water can pass from the fill through a porous stone to the water contained in the polythene tube, and its pressure is transmitted to a Bourdon Gauge which forms part of the apparatus inside the Gauge House (Fig. 6).

Two polythene tubes are connected to each point, so that water can be circulated to fill the tubes initially, and to remove any gas which may subsequently occur. De-aired water is circulated from a perspex pressure tank by pumping air into a rubber bladder (beach ball) contained in the tank. The water returns, carrying air bubbles, to a second perspex tank, and circulation is continued until no further bubbles appear. Either of the two tubes from the point can be connected to the Bourdon Gauge, and a difference in reading shows the presence of air in the tubes, which is removed by further circulation.

This apparatus has been installed in two earth dams in Great Britain which will be about 120 ft. high when complete. In one of them, in Scotland, no pore water pressures developed during construction, while in the other, in Wales, the initial pore-pressures have been 100% of the overburden. Fill placement was stopped last winter, and the measured dissipation has agreed fairly well with *Terzaghi*'s consolidation theory, using the results of oedometer tests on the fill material.

Two methods have been used to calculate the initial pore-pressures:

(*a*) The Bureau method, employing *Boyle*'s and *Henry*'s Laws for the compressibility and solubility of the air in the fill, using the results of oedometer tests on samples 4 in. diameter × 4 in. long (10 cm diameter × 10 cm long).

(*b*) Direct measurement of pore-pressure developed in a sample of fill compacted into a triaxial cell. Hydrostatic pressure was applied to the sample through the rubber envelope, and the developed pore-pressure measured by a servo-apparatus which required no volume change.

Both methods were in fair agreement, and agreed with the pore pressures measured in the dams.

I think all I need to say here is that the average placement water content of the dam in Wales was about 2% above its Proctor optimum, whereas the dam in Scotland was placed at about 2% below the Proctor optimum. We think these examples substantiate the Bureau method of estimating initial pore pressures and illustrate the high pressures which can develop when fill is placed above its Proctor optimum water content. Before I stood up I thought we were very concerned about the Welsh dam, because of the high pore-pressures that have developed, but after what Mr. *Middlebrooks* has just said, I think we probably should begin to worry about the dam in Scotland!

L'auteur décrit l'appareil piézométrique mis au point par la Building Research Station et donne les premiers résultats des mesures effectuées avec ce dernier ainsi que les méthodes de calcul de la pression initiale.

Mr. F. C. Walker

Before getting on to the subject of ground water, I would like to discuss Mr. *Middlebrooks*' comments, for a moment. I am fully in agreement with most of Mr. *Middlebrooks*' comments. But there is a difference in our experiences that is largely connected with our design practices. If *Middlebrooks*

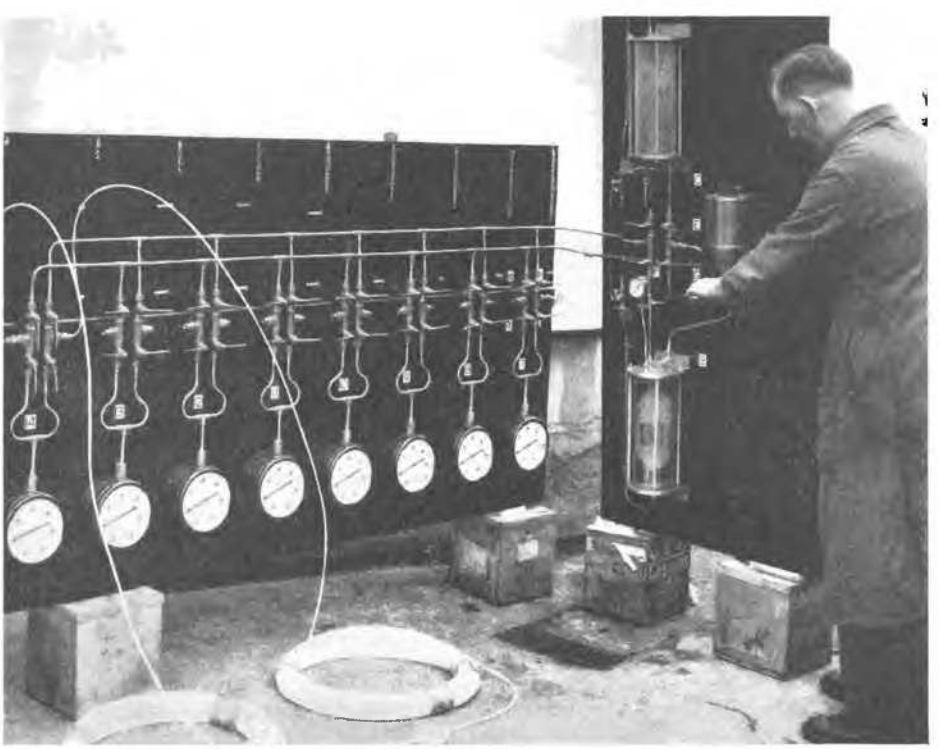

Fig. 6 Piezometer Used by the Building Research Station
Appareil piezométrique mis au point par la Building Research Station

had been in my position and I in his, our policy positions might well be reversed.

The Bureau of Reclamation has done the bulk of its work in the high dry plains and in the mountains of the Western part of the U.S. The Corps of Engineers, on the other hand, has concentrated most of its work on the wet lower alluvial valleys where the soft foundations are the rule. In consequence it is practical for us, in the Bureau of Reclamation, to take advantage of any possible saving in structure by consideration of the pore pressures to reduce the cost of the dam. In one instance we studied recently, we found that on a dam costing approximately 10 million dollars, we could reduce the costs of that dam by approximately 1 million dollars, by requiring control for the pore pressures. On the other hand, I must further emphasize Mr. *Middlebrooks*' point, that there is a great danger in constructing embankments too dry.

When I had to present papers for the 1948 High Dams Conference and the Rotterdam Conference, I was not yet in a position to say whether it was practicable to control pore pressures and prepare designs assuming such control. We had such plans under way but did not have the evidence of performance. I did not think that we had sufficient new information to present to this group at this time, to make the prepared paper worth while. However, on the numerous discussions I have heard here, I see that I have been mistaken. We have found that the pore pressures can be controlled under construction and kept to very low values, under the climatic conditions with which we have had to work in the Bureau of Reclamation. I will readily admit that in England and in Sweden and in many tropical countries such procedures are impossible. But if there is money to be saved and the climate permits then I think that it is well worth while to go after pore pressure control. Our experience lead me to believe that a structure of less than 100 ft. in height will not show a great saving in cost, whether pore pressures are considered at high values or at low, because other factors in the design predominate. But for a dam of about 200 ft. height, costing approximately 3 million dollars, we can, if we can keep the pore pressures down to about 30% uplift, reduce the costs of that structure by about 500,000 dollars. If it is cheaper to take a foundation and remove the material that can readily consolidate, about which Mr. *Middlebrooks* is so concerned, that should be an economic factor to consider. If on the foundations such as Mr. *Middlebrooks* had to contend with, it is impossible to remove the foundation to a solid base, of course, it is more practicable to design for settlement and to accept high pore pressures. In the Corps of Engineers, there is by no means a uniform policy in the treatment of pore pressures, at least as far as I have been able to find, so that there is one group that simply makes a direct shear test and this is not adjusted for pore pressure and no correction for pore pressure is made in a dam. That procedure has the advantage of simplicity, it has the disadvantage of uncertainty as to what pore pressures truly may be.

Mr. *Middlebrooks* also commented on my paper on seepage, that the methods of *Bligh* and *Lane* had long since fallen into disuse. My comment on the paper was intended to show that these surveys provided a means for recognizing a variation in permeability between the horizontal permeability and the vertical permeability in materials, which has since to a large extent been forgotten by most Soils Engineers. In consequence dams are occasionally being designed which, I think, may be in a dangerous condition. On the dam to which I referred we fell into that error. We designed the dam, which we felt was adequately safe against piping, with the ratio of approximately 10:1 between the height of the reservoir and the distance the water would have to flow. Nevertheless, under this circumstance piping did develop and I can only attribute that to the fact that the foundations have a high horizontal permeability as compared to the vertical permeability. Both the Corps of Engineers and the Bureau of Reclamation are in the process of designing dams on pervious foundations for flood control purposes. Such structures may stand for years without ever being filled and then, when suddenly filled by flood flows there is no chance to take precautionary measures. I feel therefore that this factor is one that needs considerable attention when such structures are built. We cannot yet effectively test such foundations, and should therefore take advantage of all possible empirical knowledge in formulating our designs.

L'auteur est d'avis que pour les barrages en terre construits dans des régions à climat plutôt sec, le contrôle de la pression interstitielle est nécessaire. Il permet de construire économiquement, surtout pour les barrages dont la hauteur dépasse 70 m.

En ce qui concerne la méthode de *Bligh* et *Lane*, l'auteur est d'avis que cette méthode empirique indique clairement la différence entre la perméabilité horizontale et la perméabilité verticale. Ce point est important notamment pour les fondations de barrages et digues reposant sur des sols de fondation perméables. A plusieurs occasions cette considération a été négligée et des dommages s'en sont suivis.

Mr. Ch. I. Mansur[1]) (presented by Mr. W. J. Turnbull)

Control of seepage beneath dams or levees founded on pervious materials may be achieved with varying degrees of effectiveness by toe drains, pervious landside blankets, cutoffs, impervious riverside blankets, and drainage wells along the downstream toe of the embankment.

As shown by *R. A. Barron* (Proceedings 1953, vol. II, p. 195), shallow toe drains are not very effective in reducing substratum pressures where the foundation is stratified or becomes more pervious with depth. *Barron* also suggests that for such conditions pressure reduction can be obtained more effectively by means of relief (or drainage) wells that penetrate down into the more pervious strata.

In studying the control of seepage beneath levees along the Mississippi River by means of drainage wells, several sand models were constructed and tested with both homogeneous and stratified sand foundations.[2]) The results of these studies illustrate the fact that if effective pressure reduction is to be obtained with drainage wells, it is necessary that the screens for the wells penetrate adequately the deeper, more pervious strata.

One of the models tested had an impervious top stratum, an open seepage entrance face, and a pervious foundation 100 ft. deep consisting of three strata of sand–25, 25, and 50 ft. thick, with relative permeabilities of 1:2:6.4 downward, respectively. The line of wells was 1000 ft. from the open seepage entrance or river. Wells on 100-ft. centers fully penetrating the coarse sand stratum reduced the pressure landward of the levee to 5 per cent of that on the levee; whereas wells on the same spacing, but only penetrating the top 25 ft. of fine sand, only reduced the landward pressures to 42 per cent of that on the levee. In a homogeneous sand stratum of the same dimensions, the same "25 per cent" penetration wells reduced the pressure landward of the wells to 15 per cent of that on the levee.

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[2]) *Turnbull, W. J.*, and *Mansur, C. I.*: Relief Well Systems for Dams and Levees—Separate No. 192, American Society of Civil Engineers.

L'auteur décrit des essais de puits filtrants sur modèles réduits dans des couches stratifiées de perméabilités différentes. Des essais ont montré une diminution de la pression efficace lorsque les puits sont pratiqués dans les couches inférieures dont la perméabilité est plus grande.

The General Reporter

The chairman, Professor *D. W. Taylor*, has asked me to make a few remarks before we leave the important questions, that were now discussed.

From what we have heard today and from information obtained from the papers and from previous discussions I think we may conclude that the $\varphi = 0$ analysis forms a reliable basis for an evaluation of the stability in saturated homogeneous clay. For design purposes it is, however, necessary to consider a possible change in safety factor with time. If the change in effective stresses with time results in an increase in safety factor, no further analysis is required. If, however, a decrease in effective stresses leads to reduced stability, the $\varphi = 0$ analysis should be followed by an additional analysis of the final stability.

One of my English colleagues described, the other day, the $\varphi = 0$ analysis as an "end of construction" method. I like very much this designation, and I think it is worth while remembering if the method is used for design purposes.

The stability of slopes in stiff fissured clay cannot be analysed by an ordinary $\varphi = 0$ calculation. As pointed out by *Terzaghi* in 1936 and confirmed by English experiences in 1948 by *Skempton* the shear strength of stiff fissured clays may decrease with time in cases where the fissures can open.

Very few examples are known which allow a judgement of the reliability of the $\varphi = 0$ analysis as a basis for design of slopes and cuttings in over-consolidated clays. Such clays are known to be dilatant, which means that a part of the undrained shear strengths is due to negative pore water pressures. It may, therefore, be possible in some cases that a softening takes place with time as, for example, is known to be the case in compacted clays. Probably the slide described by Mr. *Peterson* belonged to these cases, the natural water content of the clay being of the same order as the plastic limit.

It was interesting to learn about the policy and experiences of the Waterways Experiment Station and the Bureau of Reclamation. As I understood the contribution to the discussion I think we might agree that the stability of a rolled earth dam has to be analysed in terms of effective stresses. Pore water pressures have to be considered for the various stability cases and during construction the stability should be controlled by direct measurements of pore water pressures.

As pointed out by Mr. *Middlebrooks* a high placement water content of the core or a central part of the core in a rolled earth dam is a remedy which can be used, if necessary, to counteract the danger of cracks due to uneven settlements. On the other hand Dr. *Löfquist* prefers a high placement water content for technical reasons independently of the danger of cracking.

That a high placement water content results in high pore water pressures has been clearly demonstrated by Mr. *Little*'s and Mr. *Penman*'s observations from two English earth dams. In this connection I would recommend earth dam engineers to study the experiences collected from slips in English earth dams with puddle clay cores.

Le rapporteur général fait un résumé de la discussion: la méthode $\varphi = 0$ a ses limites, elle n'est pas applicable dans les cas suivants: variations des contraintes effectives, argiles rigides et fissurées et préconsolidées avec des pressions interstitielles négatives. Il constate qu'un groupe d'ingénieurs préfère utiliser sur le flanc humide de la courbe Proctor. Finalement, le rapporteur recommande l'étude des expériences recueillies en Angleterre sur les barrages à noyaux foulés (puddle cores).

Prof. K. Terzaghi

In his general report Mr. *Bjerrum* has called our attention to the differences of opinion which exist concerning the water content at which the construction materials for the impervious section of earth dams should be placed and the subsequent discussions have shown that this is a controversial issue indeed.

Some engineers recommend placing the material at a water content slightly above the optimum because they wish to avoid the formation of cracks across the dam and subsequent failure by piping due to the widening of the cracks by subsurface erosion. The recent failure of the Stockton Dam in California was probably caused by such a process. On the other hand the advocates of placing the material at a water content slightly below the optimum are guided by the intention of reducing the porewater pressures to the inevitable minimum, because an increase of the porewater pressures is associated with a decrease of the factor of safety of the slopes of the dam with respect to sliding. Hence, if one is called upon to design a homogeneous earth dam one has the choice between the devil and the deep sea.

I personally prefer the deep blue sea to the devil because if I fall into the sea I may swim but if the devil gets a hold of me I am out of luck. The deep blue sea corresponds to placing the material at a water content slightly below the optimum, because one can always design a dam in such a manner that the formation of cracks remains relatively inconsequential whereas the pore water pressures inevitably reduce the shearing resistance of the material. An increase of the pore water pressure requires flattening the slopes, which increases the cost of the dam.

The danger of erosion in cracks across homogeneous earth-dams can be eliminated by providing the dam with a narrow vertical core made out of filter material. In 1947 I designed a dam equipped with such a core. The dam is located west of Rio de Janeiro. It is about 110 ft. high and it is made out of residual, lateritic clay. The drain curtain is supplemented by a row of filter wells which were drilled through badly fissured decomposed gneiss into sound rock. The degree of saturation of the construction material was kept between the limits of 80% and 90%. The downstream portion of the dam rests on a horizontal filter layer and as a consequence no water can enter this part of the dam. The reservoir was filled two years ago. The drainage system operates strictly in accordance with the forecast and the loss of water due to seepage is very small because the integrity of the creep layer covering the submerged portions of the slopes of the valley on both sides of the dam was re-established before the reservoir was filled.

On another dam of a similar kind I inserted a thick layer of silty sand between the upstream slope of the body of the dam and the gravel base of the riprap. The dam is also equipped with a filter core. If a crack is formed across the dam, the silty sand is washed into the crack and since the silty sand cannot pass across the filter core, the crack is rapidly sealed. This conclusion has been confirmed by laboratory tests.

Since the programme of this session also includes seepage problems I wish to add a few remarks concerning the protection of dams against piping in those rather frequent instances in which the water percolates through sediments with an erratic pattern of stratification, while at the same time the construction of a cutoff is impracticable or uneconomical. On such condi-

tions the estimates of the loss of water due to seepage are utterly unreliable and the location of the points where subsurface erosion may start are unknown before construction begins. At the present time I am engaged in the design of no less than three dam sites where such conditions prevail.

At one of these sites water will escape out of a reservoir through a buried valley towards the foot of a very steep and rather unstable slope. The site is described and illustrated by Fig. 9 in my paper "Fifty Years of Subsoil Exploration" in the third volume of the Proceedings of this Conference. The danger of piping due to subsurface erosion is being eliminated by the construction of a drainage curtain across the narrowest part of the buried valley. The filter material has been deposited in superimposed tunnels. During the excavation of these tunnels a large pocket of open-work gravel has been encountered in spite of the fact that none of the test trenches located at the two ends of the buried valley disclosed the existence of such a gravel in the valley fill.

The dimensions and the degree of continuity of the bodies of open-work gravel are unknown and cannot be ascertained at a reasonable cost. Therefore an estimate of the losses due to seepage is impracticable and the drainage conduits have been dimensioned on the basis of an estimate involving the most unfavorable assumptions compatible with the geological character of the site.

At another site, located in the Sierra Nevada, an earth dam is to be built, the centre-line of which intersects one small ice-contact delta and three terminal moraines separated from each other by a thin and discontinuous till sheet. The till rests on a thick stratum of fluvio-glacial material, locally covered by lacustrine clay. The fluvio-glacial stratum rests on an older till sheet. A bulldozer cut through one of the moraines disclosed an erratic pattern of stratification distorted by the effects of an ice-thrust produced by a temporary advance of the preceding Pleistocene glacier. Deposits of this kind have been piled up by Nature in a short time and with appalling carelessness. A few years ago I have seen in the Canadian Rockies two large terminal moraines which did not yet exist in 1915.

The Sierra Nevada dam is under construction. Yet we do not know and cannot find out whether the loss of water due to seepage will be one, five or 15 ft^3/sec. We also do not know what the hydrostatic pressure conditions will be in the sands underlying the upper till after the reservoir is filled. Therefore, the following measures have been taken. Bleeder wells have been installed along the toe of the dam, with a spacing between 50 and 200 ft., and a large pile of filter material has been accumulated close to the midpoint of the row of wells. During the first filling of the reservoir the hydrostatic pressure conditions in the pore water of the subsoil beneath the toe of the dam will be determined by means of observation wells located between the bleeder wells. If the observations show that at some points the existing provisions against piping are inadequate, supplementary bleeder wells will be drilled at these points. Springs which come out of the ground downstream from the dam will be covered with inverted filters. The drainage conduits have been dimensioned on the basis of the most pessimistic assumptions concerning the quantity of discharge.

I do not doubt that most of the practicing engineers in the audience prefer problems of this kind to the more conventional ones. Soil mechanics combined with engineering geology makes us keenly aware of the various hazards involved in the project and we have also learned how to act as soon as the location of the danger points is disclosed by the results of our observations. At the same time we have the comforting feeling that we are more than mere calculating machines. We have an opportunity to act as engineers who match their wits against the treacheries and inconsistencies of nature. This has been the essence of engineering ever since this profession came into existence.

L'auteur parle en faveur de la mise en place à une teneur en eau légèrement inférieure à l'optimum, la mise en place à une teneur en eau plus élevée entraînant une pression interstitielle qui réduit la résistance au cisaillement. Cette méthode exige des talus à pentes plus faibles et, de ce fait, cause une augmentation du coût de la construction. Mais, d'autre part, lorsque l'on construit avec une faible teneur en eau, il convient de prendre des mesures pour empêcher l'eau de s'écouler par les fissures. On peut éliminer le danger d'érosion interne en exécutant des filtres verticaux dans la partie aval.

L'auteur cite un cas où l'on a inséré, du côté amont, une couche de sable limoneux entre l'assise de l'enrochement et le massif d'appui. Si des fissures venaient à se produire, le sable limoneux serait entraîné dans les fissures et, étant donné qu'il ne peut passer au travers de la zone filtrante, il les colmatera rapidement. L'auteur décrit également des mesures destinées à prévenir la filtration et l'érosion à travers des couches perméables de stratification irrégulière dans les cas où la construction d'un diaphragme est impossible. Dans le premier des cas cités, l'eau de filtration est détournée au moyen d'un écran filtrant construit conformément à une technique employée dans les mines. Dans le deuxième cas, une série de puits filtrants ont été installés au pied du barrage; leur nombre peut être augmenté si la nécessité s'en fait sentir après l'achèvement des travaux. Les sources qui pourraient apparaître sont aveuglées à l'aide de filtres inversés.

L'auteur conclut en disant que la plupart des ingénieurs praticiens qui se trouvent dans l'auditoire préfèrent ce genre de problèmes aux problèmes traditionnels. La combinaison de la mécanique des sols et de la géologie technique nous amène à détecter tous les imprévus qu'un grand projet ne manque pas de rencontrer. L'ingénieur a également appris quelles sont les mesures à prendre dès que le résultat des observations signale un point dangereux. Ces travaux lui apportent la conviction stimulante d'être plus qu'une simple machine à calculer. Ils nous fournissent l'occasion d'appliquer nos connaissances et de triompher des tricheries et des inconsistances de la nature. Telle a été l'essence de notre rôle depuis l'origine de notre profession.

M. J. Abollado

Un des problèmes les plus étudiés dans les communications présentées au 3e Congrès de la Mécanique du Sol est celui des rapports entre les nappes d'eau et les rivières; une étude approfondie s'avère nécessaire dans les problèmes de fondation et d'étanchéité des barrages de retenue.

Il semble qu'il y a des cas de caractéristiques différentes de ceux envisagés, avec grand soin, il est vrai, dans diverses communications présentées sur ce thème. En effet, il s'agit de rivières avec des épaisseurs d'alluvions considérables et de perméabilité appréciable dans des climats plutôt humides, c'est-à-dire où le débit de la rivière et celui du cours d'eau phréatique sont comparables et d'une certaine importance.

Je crois intéressant d'étudier ces phénomènes dans d'autres pays, à climats arides ou semi-arides, dans lesquels l'alimentation locale des nappes est très irrégulière, particulièrement en fonction du temps, et où la rivière, provenant d'une région plus élevée, avec précipitations plus abondantes est la source d'eau de la zone et où, en plus, les alluvions apportées sont en quantité moins grande et de nature limoneuse ou argileuse.

Dans ces conditions, les nappes sont alimentées et contrôlées par la rivière et circulent dans le rocher pour autant que sa nature, plus ou moins perméable, le permet.

Tel est le cas, par exemple, des aménagements de la Rivière Muluya, situés dans le nord du Maroc, dans la zone frontière

des protectorats espagnol et français où les deux administrations sont en train de mener à bout en commun la mise en valeur de terrains de bonne qualité auxquels l'eau fera donner de magnifiques rendements (Fig. 7).

Qu'il nous soit permis de faire une petite digression géologique indispensable, croyons-nous, pour la plus exacte compréhension du phénomène. Les terrains affectés par les travaux du barrage du Mechra Klila supérieur, mis à part les alluvions et terrasses du quaternaire récent (Q), sont situés dans les deux étages Kiméridgien (Km 1) et Portlandien (Pt) et dans les couches inférieure (M1) et moyennne (M 2) du Miocène. Le Kiméridgien inférieur est formé par un massif calcaire, légèrement dolomitique, avec quelques géodes et conduits de dissolution; le Kiméridgien moyen est formé par des calcaires littés comprenant quelques joints marneux, et le Kiméridgien supérieur par des marnes calcaires avec intercalations minces de calcaires.

Le Portlandien comprend des calcaires marneux en bancs d'épaisseur moyenne et le Miocène inférieur est représenté par un facies détritique de conglomérats et de grès. Le Miocène moyen est franchement marneux.

Tectoniquement il s'agit d'une structure de grands blocs faillés, dans le Jurassique, qui se sont déplacés les uns par rapport aux autres et, en plus, par deux discordances marquées entre ce terrain et le Miocène inférieur (phase pré-rifienne) et entre celui-ci et le Miocène supérieur.

Au point de vue qui nous intéresse, je ne retiendrai que la grande faille de la rive droite, parallèle à la rivière et avec un rejet de plus de cent mètres, ainsi que les deux transversales de deux ravins du même côté. La première est remplie par des produits argileux du type imperméable, tandis que les deux autres ne le sont pas. Celle que nous désignons par les lettres $f_1 f_1$ a été le siège d'importantes circulations carstiques c'est pourquoi le barrage a été implanté en amont de cette faille.

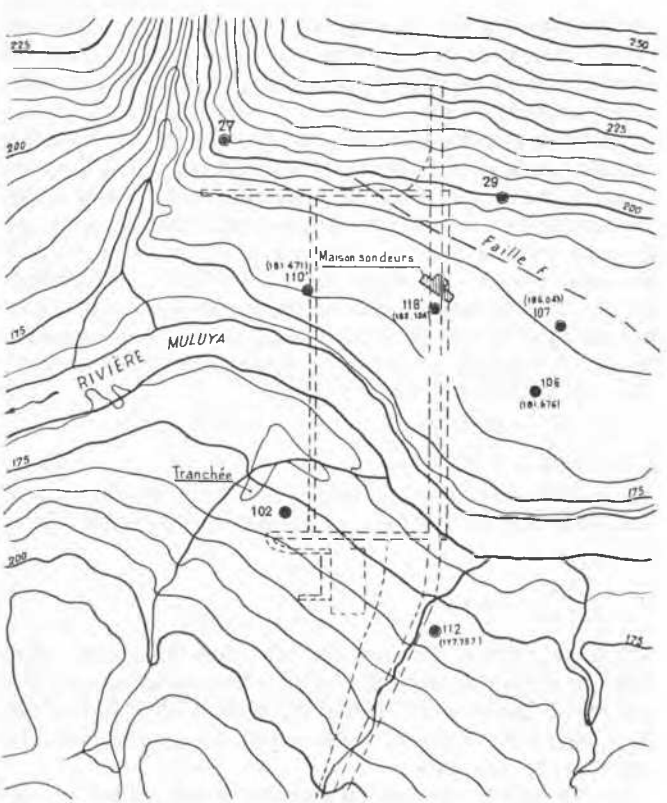

Fig. 7 Situation de l'aménagement de la rivière Muluya
Dam Site on the Muluya River

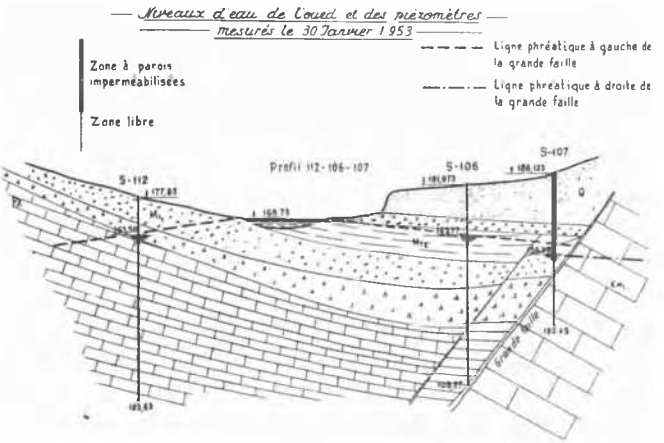

Fig. 8 Coupe passant par les sondages 102, 106 et 107
Section Through Boreholes 102, 106 and 107

Le rocher le plus suspect, au point de vue imperméabilité, est le calcaire dolomitique qui est baigné par la retenue sur quelques centaines de mètres en amont du barrage.

Des investigations minutieuses à l'aide de sondages et de galeries ont été faites et, quoique quelques détails n'aient pas été éclaircis, les résultats obtenus méritent de retenir l'attention.

Toutes les mesures systématiques enregistrées par des tubes piézométriques montrent que le niveau de la nappe phréatique est partout inférieur à celui de la rivière et démontrent clairement l'influence des variations de cette dernière. Nous allons étudier en détail les coupes caractéristiques.

Dans la première coupe transversale (Fig. 8) à la rivière l'on voit une dénivellation marquée entre les nappes de chaque côté

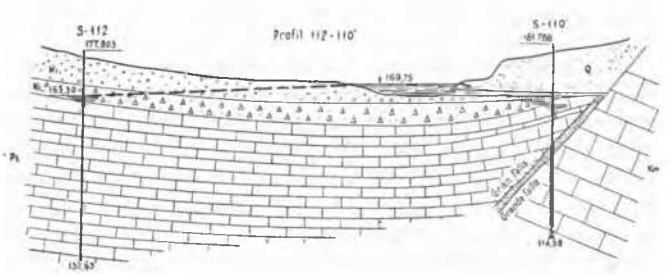

Fig. 9 Coupe passant par les sondages 102 and 110
Section Through Boreholes 102 and 110

de la faille, ce qui indique que cette dernière est imperméable. L'entrée de l'eau dans la rive droite se vérifie surtout dans la couche de graviers de la base de la terrasse. La pente de la ligne piézométrique est plus accentuée du côté calcaire portlandien que du côté terrasse, indiquant par là une perméabilité moins élevée que la terrasse. La seconde coupe (Fig. 9), en aval de la précédente, traverse une épaisseur plus petite du tertiaire et, pour une raison analogue, présente une pente moins prononcée dans les marnes miocènes. Ceci fait que le bord sur la faille du niveau piézométrique est moins bas que dans la coupe antérieure.

Les deux autres coupes (Fig. 10 et 11), parallèles à la rivière et qui, en quelque sorte, mettent en rapport les deux coupes précédentes donnent des résultats que nous pensons être plus intéressants encore.

Dans la coupe à droite de la faille (Fig. 10) nous trouvons une nappe phréatique de faible pente, sauf dans la proximité du ravin aval dans lequel il nous a été impossible de déterminer

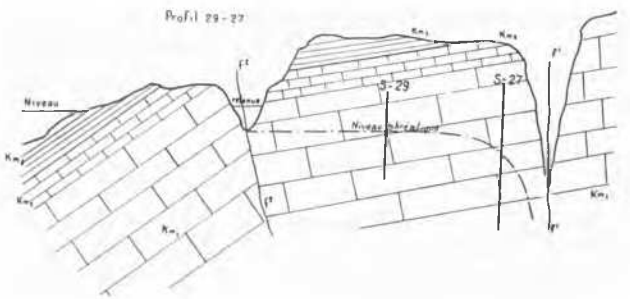

Fig. 10 Coupe à droite de la faille – sondages 29 et 27
Section on the Right Hand Side of the Fault—Boreholes 29 and 27

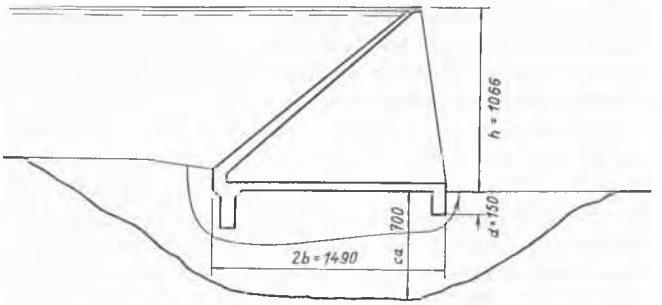

Fig. 12 Piping Under the Ashley Weir—Dimensions in mm
Phénomène de renard sous le barrage d'Ashley – dimensions en mm

aucune nappe phréatique; les niveaux déterminés sur la rive gauche descendent rapidement, dénotant une imperméabilité d'ensemble sur le côté du ravin f_1f_1.

Il est intéressant de noter que des sondages en profondeur exécutés à une centaine de mètres en aval du ravin ont localisé dans la rivière un niveau phréatique plus bas encore peut-être dans un Lusitanien plus marneux ou imperméable.

Nous n'avons pas déterminé le niveau en amont de la faille; f_2f_2; c'est dans cette zone que doivent être exécutés les travaux d'ancrage du voile au barrage dans le Kiméridgien supérieur marneux qui se trouve au niveau de la rivière, à l'extrémité gauche de cette coupe, à quelque huit cent mètres en amont du barrage.

The author describes groundwater conditions in an arid zone at Mechra Klila (Northern Morocco). The groundwater comes from the river Muluya and the groundwater levels are dependent on the varying rock permeability and on the existence of cracks in the soluble limestone.

Prof. Z. Bazant, Jr.

It is generally accepted that the validity of any scientific theory is proved if the theory is in agreement with tests and experience. The author compared his theory of failure by shear in *vibrationless cases* with the tests in Fig. 8 of his article (Proceedings 1953, vol. II, p. 203) and the results are satisfactory. In order to provide further relevant evidence by experience one case of piping under the non-overflow section of a weir is given below. It is the Ashley Weir near Pittsfield, Mass., U.S.A., where the piping took place in 1909 (Fig. 12). The dimensions of the weir were given by *Ziegler* (1927). The dimensions ratio was $d/2b = 1.5/14.9 = 0.1$, and the head at the moment of piping was $h = 10.66$ m. Simplifying the foundation to the form of Fig. 5b we obtain for fine sand without vibration ($\varphi \geqslant 12°$)

$$K_1 \frac{h}{d} = 5.8.$$

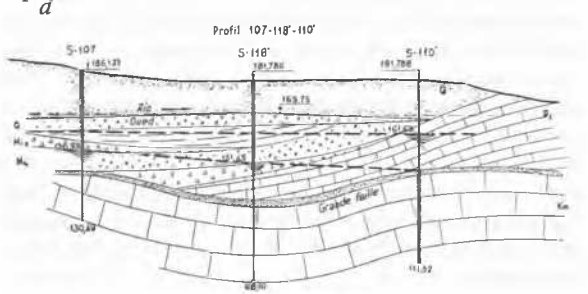

Fig. 11 Coupe à gauche de la faille – sondages 107, 118 et 110
Section on the Left Hand Side of the Fault—Boreholes 107, 118 and 110

As on the verge of stability $K_1 = 1$, the first approximation of the appropriate depth of foundation should be

$$d = \frac{10.66}{5.8} = 1.8 \text{ m}.$$

The computed d differs from d assumed in the ratio $d/2b$. Therefore we repeat the computation choosing another d until we find by trial the final solution $d = 2.1$ m, for which the computed d equals the assumed one. The actual depth being only $d = 1.5$ m, the piping according to the author's theory developed of necessity.

The theory of *vibration cases* has not yet been proved by tests. The values of the angle with φ_v vibration recommended by the author as dependent only on the grain size are a great simplification of a very complex case. It seems that the angle φ_v is not only a function of the grain size but also of the relative density of the subsoil, of the head and the discharge of water flowing over the weir and finally of the area, the mass and the direction of vibration of the apron. Therefore only experience on existing projects can decide whether the theory is applicable, and we can expect to find only the local value of φ_v depending on the type of construction of the weir. In order to test the applicability of the theoretical solution we give the following two examples:

As the first example relating to vibrating structures of the spillway sections of weirs let us consider the Lloyd Dam on the river Indus near Khairpur (Fig. 13). After *Khosla* (1936) the head is there $h = 5.83$ m. We assume for this sand ($D_{50} = 0.3$ mm) with the angle φ_v vibration $= 3°$, for which we take the coefficient $K_2 = 0.61$ for $G_t = 1.8$ (Fig. 3; Proceedings 1953, vol. II, p. 201). Further we insert into equation (22) the known $K_6 = 0.16$ which is the average of the values measured by electric analogy upstream and downstream from the sheet pile wall at the toe. Then we obtain

$$K_1 \frac{h}{d} = \frac{K_2}{K_6} = \frac{0.61}{0.16} = 3.8$$

assuming the validity of our theory for $d/2b = 3.66/58 = 0.06$ too. If we take $K_1 = 2$ for the sheet pile wall we finally obtain the depth

$$d = \frac{2.5.83}{3.8} = 3.1 \text{ m}.$$

The actual depth of the sheet pile wall below the base is 3.66 m which is in good agreement with the above computation. It is assumed in the computation that the depth of scour, from which we measure the depth of the foundation, coincides with the depth of the weir base.

As the second example of vibrating structures let us consider the Rosetta Weir of the Mohammad Aly Dam on the river Nile at El Manashi (*Wehe*, 1949). The weir has four rows

of sheeting (Fig. 14). The head is $h = 3.80$ m and the ratio $d/2b = 3.05/60 = 0.05 < 0.1$. We assume that the downstream heavy riprap is carried away and the scour reaches the base of the weir. For fine sand and silt, with the angle at vibration $\varphi_v = 1°$, we take it that

$$K_1 \frac{h}{d} = 2.1$$

according to Fig. 5b, roughly neglecting the inner sheet pile walls.

Inserting $K_1 = 2$ for the sheet pile wall we obtain the first approximation

$$d = \frac{2.3.80}{2.1} = 3.60 \text{ m}.$$

The computed d differs from the assumed $d = 3.05$ m. The final solution is obtained by trial and shows $d = 4.0$ m. The computed d is greater than the actual d, because we did not take into account the influence of the inner sheet pile walls. This omission increases the residual head h_r and consequently d too.

The agreement of theory with experience of course will have to be exemplified in cases dealing with different $d/2b$. Further the angles with φ_v vibration suggested by the author have still to be verified by tests. Until this has been done, it may be objected that the suggested φ_v may represent the angles of shearing resistance multiplied by some unknown factors of safety.

Finally one improvement can be added to the chart in Fig. 3 (Proceedings 1953, vol. II, p. 201). The limit relative pressure head h_r/d is 1.11 for $G_t = 1.8$, $\varphi = 14°55'$, and 1.66 for $G_t = 2.2$, $\varphi = 15°50'$. This holds for the supposition that we can omit the second max. φ_v which is located near $+Y$ and which appears for $G_t = 1.8$ beginning from $h_r/d = 1.05$, for $G_t = 2.0$ from 1.31 and for $G_t = 2.2$ from 1.57. No centers of trial circles fall in the quadrant below the axis $+Y$, because above the center this would presuppose a horizontal component of movement of sand grains being against the horizontal component of the direction of seepage in the upper part of the segment. Therefore some transient zone may also exist in the upper left quadrant along $+Y$. It seems justifiable to exclude this transient zone from our considerations and with it the second max. φ_v.

References

Khosla, R. B. A. N., Bose, N. K., McKenzie Taylor, E. (1936): Design of Weirs on Permeable Foundations. Simla, India, pp. 43, 127, and Table VI, 3.

Wehe, H. C. (1949): Neue Stauanlagen im Niltal. Die Bautechnik, vol. 26, p. 41.

Ziegler, P. (1927): Der Talsperrenbau. Berlin, Ernst, p. 232.

L'auteur a appliqué sa théorie du soulèvement hydraulique (voir Comptes Rendus 1953, vol. II, p. 203) à quelques exemples tirés de la littérature technique et a trouvé une bonne concordance.

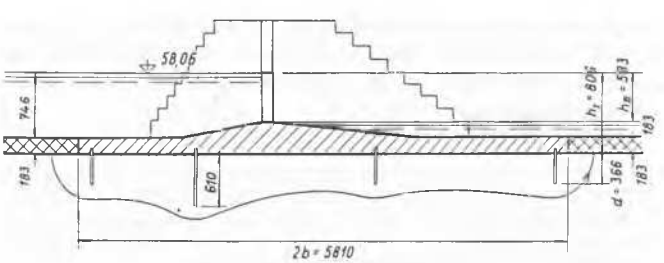

Fig. 13 Lloyd Dam on the River Indus
Le barrage de Lloyd sur l'Indus

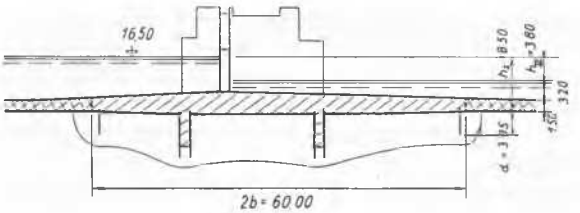

Fig. 14 Rosetta Weir on the River Nile
Le barrage de Rosetta sur le Nil

Mr. T. Edelman

I should like to say a few words about a remark that the General Reporter made about my paper (Proceedings 1953, vol. II, p. 219). In this paper I showed that the compressibility of the soil does not influence the results of an inflow test. Mr. *Bjerrum* believes this to be wrong, and he suggests that the error results from the assumption that the same modulus of volume change can be used for consolidation and for swelling.

I should like to state here that I used this assumption with the intention not to make the derivation of my formulae too intricate. The result of this simplified derivation was to neglect the influence of compressibility over the whole range of values which the modulus of elasticity may have in any existing soil.

Therefore it seems to me that it does not matter whether the modulus of compressibility for swelling is five or ten times greater than the modulus for consolidation, these differences being always of secondary importance compared to the fact that it allows us to neglect compressibility for each value the modulus of compressibility may possess. Therefore I believe it to be right that the compressibility of the soil does not disprove the results of a piezometer test. This conclusion seems to be confirmed by the results of the many piezometer tests obtained in the field by Dutch agricultural engineers.

L'auteur répond au rapporteur général qui a critiqué sa formule pour les essais de perméabilité. La dérivation de formule montre qu'il est permis de négliger la compressibilité du sol pour toutes les valeurs possibles existant dans la nature. Une différence entre les valeurs des modules pour compression ou dilatation n'influence donc pas la validité de la formule.

Mr. A. Lazard

I should like to say some words on a proposal of our General Reporter with which I cannot agree; he said that the best way of computing the safety factor is to divide C and tg φ by the same number n.

Let us consider the calculations of dry slopes. There are three stages:

(1) We make a double hypothesis: we know the exact values of C and tg φ in each point, and that these values are constant in the whole mass.

This double hypothesis is very questionable but we cannot do better for the moment.

(2) We make calculations for a given inclination of the free surface.

For a flat surface the problem has been solved by French scientists: *Resal, Frontard, Caquot, Mandel.* In a paper (Travaux, 1947 and 1948) I have shown that for slopes that are not too great the results obtained by the Swedish method are nearly correct. In more complex cases we can only use the Swedish method, but we must bear in mind that it is only an approximation and that it may lead to errors.

(3) We have tried to allow a safety margin. The proposal of the general reporter can be widely applied but I do not think

that it is always correct. In my opinion it is not advisable to deal in the same manner with C that is a stress and with tg φ that is dimensionless. In doing so we leave $H = C/\text{tg}\,\varphi$ constant, which quantity Mr. *Caquot* called hydrostatic pressure and which is of such great importance in his theorem of corresponding states.

There are several ways of allowing safety margins but if we want to go on using this measure I should like to make a proposal to the Conference: My proposal is based upon statistics and probabilities to an extent which is now frequently adopted in Structural Engineering: let us take into account the scattering.

I propose to take the mean values minus a certain multiple (say 3 or 4 times) the arithmetic mean deviation each for C and tg φ in cohesive soils. In order to answer Messrs. *Casagrande* and *Geuze*'s questions I propose that the scattering should be determined in future research.

Finally I think that we are under a misapprehension when we believe a safety of 1.5 approximatively has been ensured. There are many examples of cases in which slides have occurred with such a safety factor. Mr. *Bjerrum* himself referred to such a case two days ago.

L'auteur n'est pas d'accord avec le rapporteur général quand celui-ci propose de définir un facteur de sécurité en divisant C et tg φ par le même nombre n.

Cette manière de faire – déjà fort répandue – paraît incorrecte car elle traite sur le même pied une quantité telle que C qui est une contrainte et une quantité telle que tg φ qui est sans dimension. Ce faisant d'ailleurs, on laisse constante la quantité $H = C/\text{tg}\,\varphi$ qui joue un si grand rôle dans le théorème des états correspondants de M. *Caquot*.

L'auteur suggère de prendre en compte la dispersion des résultats et de ne retenir pour C et tg φ que leur valeur moyenne diminuée d'un certain nombre de fois (par exemple 1,5 ou 2) l'écart quadratique moyen. Si l'on connaît à peu près l'intervalle de variation possible de tg φ pour les sols pulvérulents, l'étude de la dispersion de C et tg φ pour les sols cohérents n'a pas encore été entreprise sérieusement.

Il existe de nombreux exemples de glissements de talus où le facteur de sécurité, appliqué à la méthode suédoise, était soi-disant de 1,5.

Messrs. C. I. Mansur and J. R. Compton[1])

Mr. *Walker* (Proceedings 1953, vol. II, p. 294) has properly emphasized the importance of *controlling seepage* in the design of earth dams founded on pervious foundations. Before proper seepage control measures can be devised, it is necessary first to achieve an understanding of the factors involved in the seepage problem. Some of these include the head on the dam, characteristics of the dam and foundation including dimensions, permeability, degree of stratification, location of seepage entry (thickness and permeability of upstream blanketing), seepage exit, and handling of seepage water after it emerges. The cost, permanence, and maintenance of the control measures are also of great importance.

Before the above factors can be evaluated properly and control measures designed, it is necessary to obtain a reasonably clear picture of the geology and properties of the formation involved by means of field exploration and laboratory tests. The field exploration may necessarily include the taking of undisturbed-type samples, and pumping tests to determine the permeability of the pervious aquifer and possibly of individual strata by means of piezometers and well-flow meters. Reasonably undisturbed samples of sand, where gravel is not present, can be obtained by use of Shelby tube samplers, drilling mud, and proper technique.[2])

Mr. *Walker* has pointed out that even with the best exploration and testing the true nature of a foundation cannot be predicted with a high degree of reliability, and because of this and the apparent lack of agreement between theoretical analyses and field observations he seems to favor an empirical approach, or one based largely on field observations, to the problem. In dealing with a number of seepage problems in connection with dams and levees built by the Corps of Engineers, mathematical formulas, flow nets, and both sand and electrical analogy models have been most helpful to the writers in understanding and solving specific seepage problems. Of course, precise agreement between the results of such analyses and field performance should not be expected but neither is it required if a suitable factor of safety is incorporated in the design. The principal problem in seepage analyses by means of mathematical formulas, flow nets, and models is not in the methods but in the correct determination of the characteristics of the foundation, and proper application of the formulas, flow nets, or models to the problem. However, with reasonably good knowledge of the characteristics of the foundation, proper use of the above tools of analysis, and by exercising good engineering judgment based, where possible, on field experience and actual observations, it is believed that a better solution can be obtained in this way than with too much dependence on empirical methods.

Mr. *Walker* lists two principal means of reducing the seepage pressures at the downstream toe of a dam on a pervious foundation: namely, an impervious cutoff trench beneath the central or upstream section of the dam, or a system of relief wells beneath the downstream section or at the downstream toe of the dam.

The results of a study[3]) of the efficacy of partial cutoffs by the Waterways Experiment Station showed that cutoffs with less than about 95% penetration of the pervious foundation effected little significant reduction of seepage pressures and flow. Where the depth of the pervious foundation is such that a complete cutoff can be constructed economically, the cutoff trench is considered to be superior to a relief well system, since once constructed it requires no maintenance and is a positive control.

Where the depth of pervious foundation is great, say in excess of 30 ft., careful consideration should be given to the installation of a system of relief wells in the foundation beneath the downstream toe[4]). Such a system can be designed, which will reduce the seepage pressures beneath the downstream toe to tolerable limits. While maintenance may be required occasionally, proper design and careful installation will reduce the maintenance costs to relatively insignificant figures.

Mr. *Walker* has presented some very interesting and worthwhile field observations on hydrostatic pressure conditions in and beneath dams located on pervious foundations. The value of the observational data would have been enhanced, had general information on the characteristics of the foundation soils been shown on Figs. 1–6.

[1]) Engineers, Embankment and Foundation Branch, Soils Division, Waterways Experiment Station, Corps of Engineers, Vicksburg, Mississippi, U.S.A.

[2]) *Goode, T. B.* (1950): A New Method for Obtaining Undisturbed Sand Samples. Engineering News-Record, October 12, vol. 145, pp. 40-42.

[3]) *Mansur, C. I.*, and *Perret, W. F.*: Efficacy of Partial Cutoffs for Controlling Underseepage Beneath Dams and Levees Constructed on Pervious Foundations. (Vol. 5.)

[4]) *Turnbull, W. J.*, and *Mansur, C. I.* (1953): Relief Well Systems for Dams and Levees. Separate No. 192, American Society of Civil Engineers.

In all major hydraulic structures where seepage may be a problem, an adequate number of hydrostatic and seepage measuring facilities should be incorporated in the basic design. Such should be properly installed, maintained, observed at appropriate intervals and reservoir stages, and then the data analyzed carefully. Such information, although seldom directly applicable to another specific problem, is invaluable to the engineer designing seepage control measures.

Les auteurs énumèrent d'abord les facteurs qui influencent les problèmes de filtration. A l'opposé de M. *Walker*, qui est partisan des méthodes empiriques et des observations sur place, les auteurs sont d'avis qu'il est possible de calculer la filtration avec exactitude. Lorsqu'il est possible de pénétrer le schiste perméable jusqu'à la zone imperméable, il est préférable de construire un mur parafouille. Lorsque l'épaisseur de la zone perméable dépasse 10 m, les auteurs préconisent les puits filtrants pour réduire la pression de la filtration. Finalement ils proposent d'incorporer dans tous les ouvrages hydrauliques des appareils pour la mesure de la filtration et de la pression hydrostatique.

M. U. Nascimento

Je voudrais me rapporter à deux points du sujet de cette session.

Le premier concerne les mouvements lents des talus dus aux variations volumétriques des sols du fait des variations saisonnières d'humidité, tels qu'ils ont été observés dans les argiles basaltiques de la région de Lisbonne.

Schématiquement, on peut expliquer ces mouvements de la façon suivante:

Lorsqu'on volume d'argile de poids P, reposant sur un plan horizontal, subit un retrait σ, les efforts de cisaillement f développés entre l'argile et l'appui sont égaux. Le retrait aux extrémités est $\delta/2$, et l'qxe du volume ne se déplace pas (Fig. 15a). Il en est de même dans le cas d'une dilatation (Fig. 15b).

Mais si le volume se trouve sur une pente, sa stabilité étant assurée, l'allure du phénomène est différente.

En effet (Fig. 16a), aux deux forces f, représentant la résultante nulle, vient s'ajouter la composante tangentielle T du poids propre qui, étant la résultante du système des forces agissant sur le volume, l'oblige à descendre, retrait (Fig. 16a) et dilatation (Fig. 16b) s'étant produits.

La conséquence pratique de ce phénomène est un mouvement vers le bas, entre les couches superficielles des talus argileux, dont la valeur peut parfois être très élevée.

Ce phénomène a été observé sur un modèle soumis à des variations d'humidité.

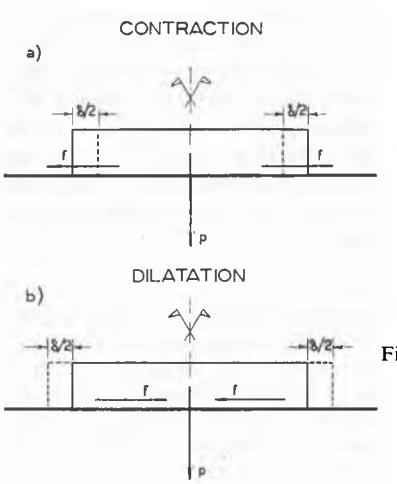

Fig. 15 Cas de contraction et de dilatation sur plan horizontal
Case of Contraction and Dilatation on a Horizontal Plane

Le deuxième point que je voudrais mentionner se rapporte à la pression interstitielle comme facteur de la stabilité des talus des barrages en terre.

On admet d'habitude dans les calculs de stabilité que la phase solide au-dessous de la ligne de saturation subit la poussée d'Archimède. Dans mon article «Capillarity and Soil Cohesion», auquel j'ai déjà fait référence dans la première session, je démontre que cette poussée ne peut exister que si la succion

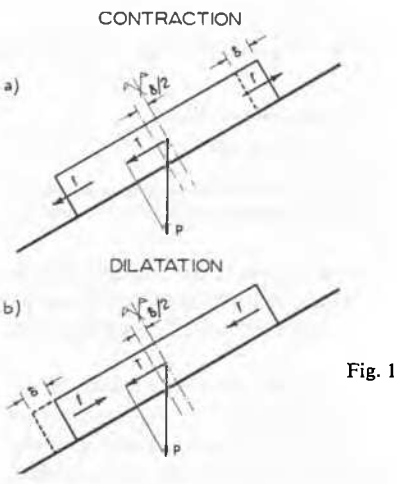

Fig. 16 Cas de contraction et de dilatation sur plan incliné
Case of Contraction and Dilatation on a Sloping Plane

capillaire est constante en profondeur. Je crois donc qu'il y aurait avantage à prendre en considération l'influence de la succion capillaire dans de tels cas.

Références

Arantes, E. de, et *Oliveira*, etc. (1951): L'étude du sous-sol des routes. Rapport portugais présenté au IXe Congrès International de la Route, 2e Question, 30, Lisbonne.

In the first part of his contribution the author gives a schematic description of the influence exerted on clays by seasonal volume variations due to changes in the water content. Whenever such variations take place on a slope, creep is the result.

In the second part he points out the importance of capillary suction in the stability of earth dam slopes.

M. B. M. Rajčević

Considérant les questions proposées pour la discussion, il nous a paru intéressant de donner quelques précisions sur la technique que nous employons pour la *construction des barrages en terre*.

Au cours de ces dernières années, nous avons projeté et construit une douzaine de barrages en terre, d'où nous avons tiré notre propre expérience.

Notre technique ne comprend pas la méthode Proctor de contrôle sur le chantier, ni même la méthode de compactage en laboratoire.

Nous déterminons la densité sèche à obtenir au moyen d'essais à l'œdomètre; cette densité doit correspondre à une consolidation de 100% sous une charge égale à la surcharge que supporte chaque point du barrage. Nous ne faisons donc que le compactage nécessaire pour atteindre ce résultat, d'où l'économie de notre technique.

En ce qui concerne la teneur en eau lors de la mise en place, nous ne travaillons pas avec la teneur en eau optimum déterminée par l'essai Proctor. Nous travaillons avec une teneur en

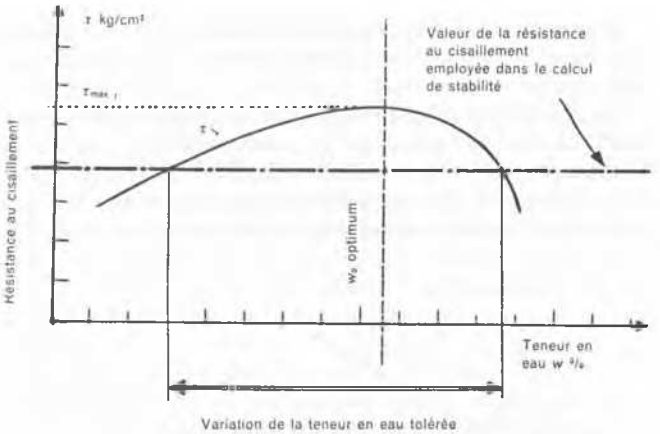

Fig. 17 Résistance au cisaillement en fonction de la teneur en eau
Relationship Between Shear Resistance and Water Content

eau égale ou même supérieure à la teneur en eau naturelle du matériel, suivant les circonstances, la seule condition étant que cette teneur en eau soit inférieure à 80% de la teneur en eau de saturation.

Dans les cahiers de charges on prévoit que la tolérance admise pour la teneur en eau, est fonction de la résistance au cisaillement, cette résistance au cisaillement devant toujours rester supérieure à la valeur choisie pour les calculs de stabilité.

Ce qui précède est concrétisé par le schéma de la Fig. 17.

Toutes ces valeurs sont déterminées, non seulement par des essais de laboratoire, mais aussi par des essais de compactage sur le chantier à l'échelle de la grandeur naturelle.

Cette méthode nous permet notamment de poursuivre les travaux pendant une plus grande période de l'année, ce qui est une source d'économies.

Cette technique n'est peut-être pas applicable dans tous les pays, mais elle s'est montrée particulièrement avantageuse en Yougoslavie, étant donné la structure économique, le climat et la nature des sols du pays (Fig. 18).

Références

Middlebrooks, T. A. (1952): Progress in Earth-Dam Design and Construction in the United States. Civil Engineering, N.Y., vol. 22, no. 9, pp. 702–710.

Fig. 18 Barrage de Vlasina – Premier barrage en terre construit en Yougoslavie (1937). Projet: *B. M. Rajčević*
Vlasina Dam—the First Earth Dam Built in Yugoslavia (1947). Designer: *B. M. Rajčević*

Rajčević, Bogdan (1948): Zemljane vodojaže. Rad, Beograd.
Rajčević, Bogdan (1951): Vlasinska zemljana vodojaža. Beograd.
Rajčević, Bogdan (1951): Geomehaničke analize pri gradnji zemljanih vodojaža. Elektroprivreda, Beograd.
Rajčević, M. B., Jevdjević, V. M. (1951): Some Special Features of Designs and Constructions for Earth Dams and Rockfill Dams in Yugoslavia. IV Congress on Large Dams, vol. 4, New Delhi.

The author gives some details of 12 earth dams constructed in Jugoslavia. The dry density to be obtained in the dam is calculated on the basis of oedometer tests. This density should correspond to a consolidation of 100% under the load at every point in the dam. The moisture content of the material used should be less than 80% of saturation. A variation is allowed in so far as the resulting shear strength is higher than that assumed in the stability analysis (Fig. 17). This method makes it possible to extend the working period and is more economical (Fig. 18).

Mr. A. Wackernagel

The *placement moisture* content is to be adapted to the requirements of the different parts of the structure.

A typical case for varying the placement moisture content was encountered during construction of Konar I Dam in India.

The dam is built of earth with a central concrete spillway. The joint between the earth and the concrete parts is built at a pressure head of 130 ft.

The joint is subject to large differential settlements, i.e. no settlement of the concrete dam and full settlement of the adjoining earth dam. Also during earthquakes considerable differential movements may occur.

Furthermore the danger of piping had to be prevented along the concrete-earth joint.

The problem has been solved by inserting a clay blanket between the concrete surface and the earth fill.

The clay, with a fraction of 40% passing No. 200 mesh, an optimum moisture content (OMC) of 15% and a plastic limit of 16% was placed at a moisture content of 18%, being both on the wet side of OMC and of the plastic limit, thus allowing for considerable differential movements without rupture. The material was compacted by hand and later on by pneumatic tampers reaching a degree of compaction of 86% maximum standard Proctor density, this being equivalent to 115 lbs./cu.ft.

The faces of the concrete dam are everywhere inclined, so that all gaps due to movements should be closed immediately by the overburden pressure. The maximum pressure gradient along the joint is approximately 1 : 2.

At the same time the main earth fill, consisting of Muram and decomposed rock (see Proceedings 1953, vol. II, paper 8/11 by *E.* and *G. Gruner*), is placed 1–2% on the dry side of OMC. This is possible firstly because the foundation consists of rigid material, secondly because the Proctor curve is fairly flat and thirdly because the additional settlement on saturation in the laboratory amounted to only 0.1% of the height of the sample for an overburden of 60 ft. of soil, the settlement for the same overburden before saturation being 2%, if the material is placed at 3% on the dry side of OMC.

L'auteur présente un exemple pour la discussion de la teneur en eau lors de la mise en place. Pour le barrage en terre de Konar I la teneur en eau appliquée à la jonction entre la portion en terre et la portion en béton est supérieure à l'optimum Proctor en raison des tassements différentiels prévus. Pour la partie en terre on a choisi une teneur en eau sur le côté sec du OMC, des essais préalables ayant révélé un faible tassement lors de la saturation.