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# Stability and Deformation of Slopes, Earth Dams and Groundwater Problems

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

## GENERAL REPORT

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### Introduction

The rapid development of the subjects covered by Session 8 finds expression in an increasing volume of publications. A great number of papers, books and reports has appeared since the Conference in Rotterdam 1948 containing a wealth of experience, theories, field observations and test results. To this conference 22 papers are presented in Session 8. There is altogether such a comprehensive selection of raw material available that a general report necessarily must be confined to a discussion of some few important topics.

Owing to this limitation the papers presented to this Conference are only considered in the report if the paper can form a basis for a discussion of the selected topics.

At the end of the Report a list of publications is printed which, however, only includes papers directly referred to in the Report.

### (1) Stability of Slopes

When, after the Conference in 1948, the reporter tried to summarise the main results, he found that the English presentation of the  $\varphi = 0$  analysis for investigating stability problems in clay was one of the most important contributions. Not only was the fundamental basis for this method and its range of validity presented, but also a number of actual slides were investigated and the simplicity and validity demonstrated.

Since the Rotterdam Conference this method has been further supported, partly by theoretical considerations (*Skempton*, 1948<sup>1</sup>; *Hansen and Gibson*, 1949) and partly by examples from actual slides. *Cadling and Odenstad* (1950) have presented 11 further slides for which the  $\varphi = 0$  analysis showed safety factors close to unity, the shear strength being determined by the vane. *Wenner* (1951) describes 4 Swedish landslides for which he found a "brilliant confirmation" of the correctness of the  $\varphi = 0$  analysis. An analysis of 8 American slides

(*Berger*, 1950) resulted in a very good agreement for 6 cases, and similar results are reported from New Zealand (*Murphy*, 1951).

Taking into consideration that this method has been used with success for more than 20 years in several European countries, and that the validity now has been demonstrated in more than 30 cases, it would be expected that this method would now be generally accepted and applied. However, a review of papers published since 1948 proves that this is not the case. Several authors express their scepticism, probably as they have misunderstood the method. Very often it is not realised that the shear strength found by unconfined compression tests, undrained triaxial tests, or vane tests includes the frictional resistance which can be mobilized if the clay is brought to failure without water content change. They, consequently, fear that the  $\varphi = 0$  analysis "would lead to a conservative design, since the resistance offered by the normal forces would not be included". Others do not realize the limitations of the  $\varphi = 0$  analysis; for instance they try to use the  $\varphi = 0$  analysis on the actual sliding surface, an attempt which will result in erroneous safety factors.

A number of American authors reject the  $\varphi = 0$  analysis referring to analysis of actual slides which resulted in safety factors greater than unity. *Larew* (1952) has quoted 3 slides and of the 8 slides analysed by *Berger* (1950), 2 showed too high calculated safety factors. In an Annual Report (1950-51) Purdue University also records too high safety factors for actual slides, and Northwestern University found that "some slopes indicated as being safe by conventional methods of analysis, have actually failed".

No detailed information is available about the slides investigated, and it is, therefore, not possible to draw any conclusion from these cases. The very high safety factors, found for some of the reported slides indicate that the clay was

fissured (this is confirmed for one of the slides reported by *Berger*). As pointed out by *Terzaghi* (1936) and *Skempton* (1948), slopes in stiff fissured clay cannot be analysed in the ordinary way due to a decrease in strength with time, which it is difficult to observe with ordinary sampling and testing technique.

The question of which test should be preferred for an analysis of a clay slope, will probably be discussed in Sessions 2 and 3. The low safety factors obtained from unconfined compression tests in sensitive clays (*Cadling* and *Odenstad*, 1950) can be explained by the sample disturbance.

As the  $\varphi = 0$  analysis forms a simple basis for stability analysis in clay and its reliability has been proved in a relatively large number of cases, it must be expected that the method will be generally accepted. Presumably the appearance of the vane will contribute greatly to this. An account of the limitations of the method is, for this reason, very important, and it would be of great interest to obtain detailed information about those cases for which the  $\varphi = 0$  analysis apparently failed.

In this connection the attention is drawn to the effect of the rate of strain. Since 1948 this factor has been further investigated, and in a paper *Casagrande* and *Wilson* (1951) state that under sustained loading the stress required to cause failure in clays and clay shales is considerably reduced. Even if the conclusions can only be taken as tentative, the authors consider it as probable that this effect can explain slides in slopes which have been standing for many years without apparent movement. This effect may mean that the strength found by standard 5-minute vane or unconfined compression tests has to be corrected for the rate of loading. Further research and a comparison between laboratory results and field observations is, however, necessary; the above mentioned agreement between calculations and field observations points to the fact that the effect of the rate of loading either is small in normal clays, or possibly is compensated for by other factors which act in the opposite direction (*Skempton* and *Bishop*, 1950).

Even if the  $\varphi = 0$  analysis shows satisfactory results if used for analysing failures, provided that the samples tested represent the failure zone at the moment of failure, its use for design is limited to cases with no change in effective stresses and water content, or to cases where the effect of such a change can be relied on to increase the factor of safety. The soil engineer is thus often faced with stability problems for which the assumptions in which the  $\varphi = 0$  analysis is based do not hold good. Besides the above cases in which a change in effective stresses can take place, can be mentioned stability problems in silty soil, partly saturated or dilating clays.

In the opinion of the reporter the most logical approach is in such cases to carry out the stability analysis with respect to effective stresses. The problems concerned with such an analysis will be further discussed below in connection with the stability of earth dams.

An example on an actual slide which probably could only be predicted by an analysis with respect to effective stresses, is given in a paper presented to this conference by *Harty* (8/13). An 8.5 m high embankment was constructed 1928 in Ireland for storage purposes on deposits of clay and silt overlying a fine sand. In 1948 a slide took place on the downstream side of the dam, and subsequently an investigation of the stability was made. Probably the slide was caused by a reduction in the strength of the clay deposits due to high pore water pressures in the sand below the downstream toe of the dam. An attempt was made to analyse the slide based on vane tests, but the scattering in the observed values are too great to allow any conclusion to be drawn.

The big landslides on the Kent Coast in Southern England are described in a paper by *Toms* (8/20). Thorough investigations have cleared up the geological conditions and pore water pressures have been measured. The paper contains an analysis of the stability based on the  $\varphi = 0$  principle, but no agreement is found between measured strength and shear stresses calculated from the slide. This result seems to be quite reasonable as the clay involved is heavily overconsolidated and, therefore, probably dilatant and fissured. The paper also includes a discussion of the change in shear strength which takes place due to progressive reduction of the overburden by previous slides. In this connection it must be emphasised that even if only very small water content changes are observed by unloading of an overconsolidated clay, a reduction in effective stress will always be accompanied by a corresponding decrease in shear strength. It would be very interesting if an analysis of the stability in terms of effective stresses (using the measured pore water pressures) could be published. The reporter would also propose the investigation of the possibilities of utilizing the low ground water level observed in the underlying Green sand for a stabilization of the area. If at the toe of the cliff a number of vertical filter wells through the water bearing zones were drained into the Green sand with the aid of small diameter borings, a favourable reduction in pore water pressures in the sliding masses would seem possible.

It is rather impressive in Annual Reports and different papers to read the descriptions of the big landslides, earth flows, creeping slopes and similar phenomena. In Japan, Yugoslavia and also Switzerland enormous masses are flowing or creeping downwards.

A number of earth flows in Japan are described by *Fukuoka* (8/9). Rate of movement is quoted, and it is stated that a distinct relationship exists between precipitation and velocity of the sliding masses. The paper and its figures illustrate the difficulties encountered in analysing or attempting to stop the slides. It is easily understood when *Fukuoka* in an Annual Report concludes that "a landslide devil seems to laugh at human incompetency".

## (2) Earth Dams

### (a) General review

Engineers who have tried to follow all the publications dealing with earth dam problems, have had a hard, but rewarding job. Particularly since 1945, tremendous progress has been made in the design, construction and control of earth dams.

Outstanding contributions to our knowledge of the behaviour of earth dams have been made by the U.S. Bureau of Reclamation. Pore water pressures and settlement observations from a number of dams are presented in different papers, and the collected experience has proved to be of great value for practice as well as for fundamental research. Some important field observations are presented by the Bureau of Reclamation engineers to this conference also. Seepage and filter problems have been thoroughly studied by the U.S. Corps of Engineers, and the results are presented in a number of detailed reports and papers.

During recent years an increasing number of earth dams have been constructed in Europe, and simultaneously the soil mechanics laboratories have started the scientific investigations of the different problems met with in earth dam design. The number of English, French and Swiss papers dealing with earth dam problems has suddenly increased since the war. This work has already resulted in outstanding publications, mainly as ad-

vantage could be taken of previous fundamental research. Particular attention should be drawn to a recent doctor's thesis from the University of London by *A. W. Bishop*. In this thesis *Bishop* gives an eminent treatment of the stability of earth dams. Based on the results of fundamental research, he succeeds in deriving approximate design values for the pore water pressures during construction and rapid draw-down. The reporter considers this result as the most important finding published since 1948.

Since 1948 a Congress on Large Dams has been held in New Delhi (1951). One of the questions discussed at this conference, was the design and construction of earth dams and rock fill dams with their core walls and diaphragms. 23 papers were delivered in this section, giving a broad outline of the present stage and presenting a wealth of experience and field observations. It is difficult to draw boundaries which could prevent overlapping between the two conferences. But as the discussion at the Large Dam Congress mainly covered practical questions, it seems reasonable that the discussion at Session 8 of this conference should concentrate on the basic questions: Stability analysis of earth dams and placement moisture content. Seepage through dams will be considered in chapter 3 "Seepage and groundwater problems".

#### (b) *Stability analysis of earth dams*

The calculation of the stability of an earth dam is probably one of the most difficult questions in soil mechanics. And at the same time it is one of the most important questions, as it contains practically all the problems met with in other cases of stability analysis. Numerous papers dealing with the stability of earth dams are published, but the opinions are so strongly divergent that it seems impossible to draw any generally agreed conclusions. The reporter has realised that a discussion of the theoretical refinements is only of limited value at the present stage of development. He has, therefore, tried to select some principal points in the divergent conceptions, putting them forward as possible subjects for discussion.

Reviewing recent papers, one has the impression that only very few authors prefer the "*elastic method*" of analysing the stability of earth dams, consisting of a calculation of the stress distribution in critical zones of the dam and comparing this with the allowable strength of the soil. In a paper delivered to the Congress on Large Dams in 1951 *Bennett* attached much importance to the elastic method and recommended its use, together with the slip circle method by design of important embankments. If, however, the design of earth dams should be based on the requirement that no point should be overstressed, this would correspond to considerably flatter slopes than are normally used in existing dams designed on the slip circle method. For instance *Bishop* (1952) found by relaxation analysis of the stress distribution in earth dams that for safety factors lower than 1.8 on slip circle methods some local over-stress is bound to occur.

The majority of authors prefer the second way of approaching the stability analysis, which is called the *limit design method* or the *rupture surface method*. The characteristic feature of this method is that the analysis is concerned with a state of equilibrium assuming that failure occurs along a continuous surface of rupture. In cases where the safety factor is 1.0, this method will lead to an approximate result without further assumptions. But in normal design the limiting equilibrium is not realised and it is, therefore, necessary to postulate a change in the actual conditions which would bring the dam up to the point of failure. This can be made in different ways resulting

in different definitions of the safety factor, and the divergencies met with in the various types of analysis are mainly due to different methods of bringing about limiting equilibrium. Some authors prefer to calculate the critical pore water pressures which would bring the dam to failure, other to determine the maximum height the dam could be raised to or the steepest slope which could be used without causing failure. But the various types of analysis will result in inconsistent safety factors, and they are all of them of limited value to the designing engineer.

The most logical approach to establish incipient failure in the analysis of the stability of an earth dam seems to be obtained by changing the shear strength properties of the soil. In the  $\varphi = 0$  analysis this is easily made by dividing the shear strength with a factor, defined as the safety factor, if thereby limiting equilibrium is obtained. Correspondingly in cohesionless materials  $\tan \varphi$  is divided with the safety factor. For dam materials which in terms of effective stresses show cohesion as well as friction, i.e.

$$s = c + (\sigma - u_w) \tan \varphi$$

the obvious method of realizing incipient failure is to divide  $c$  as well as  $\tan \varphi$  by a factor, defined as the safety factor (*Terzaghi*, 1943). *Golder* and *Ward* (1950) state that this method of defining the safety factor has now been accepted in British practice.

This method of defining the safety factor possesses such powerful advantages that it is easily understood that it is generally preferred. It leads to a very simple procedure of stability analysis, and the safety factor gives an intelligible picture of the stability. But an important question still remains unspecified, namely, if in impervious materials the pore water pressure ( $u_w$ ) should be inserted with the actual values, this means the values which could be measured with piezometer cells installed in the dam. Or should the pore pressures correspond to the failure condition given by reduced  $c$ - and  $\tan \varphi$ -values. To be said in favour of the first method is that the use of the actual pore water pressures allows that the estimated values of pore water pressures can be checked by field observations, a fact whose importance cannot be overestimated as it provides against unforeseen deviations from the design assumptions.

The general trend in the published papers is that the analysis of the stability of earth dams is carried out in terms of effective stresses. This principle has been further supported by the research carried out by *Bishop* (1952), who demonstrates e.g. that the values  $c$  and  $\tan \varphi$  in the above equation are almost independent of whether the material is consolidated anisotropically or—as is normal in laboratory testing—under an all-round fluid pressure; the pore water pressure set up by a change in stress, is, on the other hand, much influenced by the initial principal stress ratio.

An interesting attempt to analyse the rapid draw down case in terms of total stresses is published by *Golder* and *Ward* (1950). They express the shear strength by the equation

$$s = c + \sigma \tan \varphi$$

in which  $\sigma$  is the total normal stress.  $c$  and  $\varphi$  cannot be considered constant, but vary with the pressure under which the material is previously consolidated (before draw down). In this way the difficult estimate of the pore water pressures is evaded. This procedure seems correct if the consolidated undrained shear tests on which the analysis is based, are carried out with samples consolidated anisotropically with the same principal

stress ratio as could be expected in the dam (*Hansen, Gibson, 1949*). Such a testing procedure is possible, but elaborate as a great number of tests are required.

Also *Taylor (1951)* proposes a similar method for stability analysis in terms of total stresses in partially saturated soil. The effect of the principal stress ratio is mentioned by *Taylor*, but not considered in the stability analysis.

Taking into consideration that the stability analysis of the case with full reservoir and steady seepage necessarily has to be analysed in terms of effective stresses and that the use of total stresses in the analysis prevents a check with field observations as failure values of the pore water pressure are supposed, it seems logical to carry out the stability analysis for all cases by using effective stresses.

As mentioned above, the estimate of the pore water pressures is the most difficult step in a stability analysis in terms of effective stresses. The determination of the pore water pressures developed in impervious materials during construction and rapid draw-down, present special difficulties. However, since 1948 progress has been made in our understanding of the factors which control the pore water pressures.

In the Proceedings of the Rotterdam Conference *Hilf* described a method of estimating the *construction pore water pressures* developed in the air and water in the pores of a rolled impervious fill provided that no drainage takes place during the construction. By this method the pore water pressures are calculated using *Boyle's Law* for compressibility of air and *Henry's Law* for solubility of air in water. The compressibility of the soil is taken from oedometer tests. This procedure has been generally accepted as a reasonable approach to the problem, and the laterally confined compression is believed to be a satisfactory approximation to the state of stresses in the middle of the dam.

In the Bureau of Reclamation *Earth Manual (1951)*, a pore pressure apparatus for 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 one this procedure may lead to much too high values of pore pressures, particularly for soils compacted at relatively low moisture contents. Field observations of pore water pressures have now been published in several papers (*Walker and Daehn, 1948; Daehn and Hilf, 1951*). It would, therefore, be of great value if the Bureau of Reclamation could present during the Conference a comparison between observed and calculated construction pore pressures and the test data on which it is based.

The development in laboratory technique has made it possible to measure the pore water pressures in undrained samples subjected to arbitrary state of stresses in a triaxial cell. This direct method of predicting the construction pore water pressures appears to be very promising. *Bishop (1952)* proposed that tests should be run either at a constant effective principal stress ratio, corresponding to a certain factor of safety, or with no lateral yield, which seems to correspond to the actual state of stresses in the central part of the dam. In a recent publication the results of such tests are presented (*Bjerrum, 1953*).

Summarizing these results it is seen that construction pore pressures can either be calculated as described by *Hilf (1948)*, or measured at unconsolidated samples loaded in the triaxial cell. In either case, the construction pore pressures should be determined for different water contents in order to investigate how the stability depends on the placement conditions of the fill. As the pore water pressures are extremely sensitive to a

change in placement water content, it seems hazardous to use rule of thumb methods of expressing the pore water pressures as a percentage of the overburden.

The second important question is the evaluation of the *pore water pressures set up during a rapid draw-down*. In relatively pervious materials in which excess pore water pressures due to the change in stresses can be neglected, the pore water pressures can be determined from flow net constructions. But in impervious material matters are more complicated.

Firstly it is necessary to determine the pore water pressures which exist with full reservoir immediately before the draw-down. As in design work direct measurements seldom are available an estimate must be based on a flow net.

During the draw-down the change in total stresses will cause a change in pore water pressures. Now the state of stresses after a draw-down is unknown and can only be derived by a complicated analysis. And even if fundamental research has succeeded in establishing how the pore water pressures change for a change in total stresses under undrained conditions, the experimental determination of the necessary soil coefficients would require comprehensive testing. It is thus outside the scope of normal design work to aim at a rigorous solution. Maximum values of pore pressure in saturated fills may, however, be obtained by assuming that the soil is relatively compressible. On this basis *Bishop (1952)* arrives at a simple solution as he found that the *change* in pore water pressure during a rapid draw-down equals the change in total overburden pressure even allowing for the change in shear stress. Similar results are stated by *Terzaghi (1943)* and used in designing Swiss dams (*Haefeli, 1951*), but in these cases the effect of the shear stresses was not considered.

If, however, the material contains air in the pores before the draw-down (compressed or dissolved), still higher values of the pore water pressures during the draw-down may exist. The expansibility of the air will attempt to maintain the pore pressures by a decrease in pressures (*Bureau of Reclamation, 1951; Reinus, 1948; Bishop, 1952*). The opinions about this effect are somewhat divergent. *Glover and Cornwell (Bureau of Reclamation, 1951)* consider the grain structure as more expandable than the pore fluid and find consequently a temporary reduction in pore water pressures. As this effect may lead to severe assumptions for the design, it would be interesting to discuss the question at the Conference.

Several authors spare no pains in order that their sliding surface fulfil the criterion that the angle between the sliding surface and principal stresses is  $45^\circ \pm \varphi/2$ . French authors have contributed a number of theoretical investigations on this question. Also at this Conference the use of the logarithmic spiral is considered in a paper by *Fröhlich (8/8)*. However, there is no fundamental difference between the use of a circle and the use of a spiral (see i.g. *Coenen's* paper to the Rotterdam Conference, 1948) as sliding surfaces, and this question will, therefore, not be treated further.

In zoned dams the most dangerous sliding surface will often be composed of a circle and a straight line. This result is again obtained from model tests reported by *Nonweiler (8/16)* in a paper to the Conference, "The stability of slopes of nonhomogeneous dams". As the distribution of the normal stresses along the sliding surface is unknown, serious errors may be introduced in the normal analysis of the stability of zoned dams. This question is still unsolved and it would, therefore, be interesting to determine the actual failure distribution from *Nonweiler's* tests. However, the small scale of the model tests, the questionable boundary conditions in the model box and the im-

portant friction from the side walls invalidate the test results, so that it seems difficult to draw any conclusions.

More important are *Haefeli's* investigations (1951) of the stability of zoned dams during a rapid draw-down. *Haefeli* found that the complete draw-down is not the most dangerous case. The minimum value of the safety factor is often reached at an intermediate stage of the water level. In the case described, the minimum value of the safety factor is about 10% smaller than for a complete draw-down. This effect seems to be related to the use of circular sliding surfaces, and the conclusions lead to a recommendation of the use of a straight line as a sliding surface through the pervious zone.

### (c) Impervious fill

A statistical treatment of the data of the impervious material used in 37 American earth dams is presented in a paper by *Esmiol* (8/7). It is interesting to see the wide range of the physical properties of the tabulated soil types. But it is still more impressive to find the applicability of impervious fills based on physical properties only, neglecting whether the natural moisture content is above or below the optimum. European engineers will envy the Bureau of Reclamation engineers the favourable climatic conditions under which their dams are constructed.

*Gruner* (8/11) describe the use of a particular material as fill for an Indian earth dam. At the dam site the rock is weathered "in situ", the upper layers being a plastic mica-rich impervious material while in greater depths pervious gravel-like decomposed rock is found. This favourable formation leads to a cross-section of the dam with a core constructed of the most weathered material while the less decomposed rock is used as pervious material in the outer sections. It is interesting to see that even though the dam is placed close to a violent earth quake centre, the design is based on extremely low safety factors.

A number of difficulties met with in earth dam design in India are described by *Rao* (8/18). The paper gives a broad outline of several big earth dam projects and some data of the fill.

### (d) Placement moisture content

At the Rotterdam Conference the important conclusion was reached that increased stability during the construction of earth dams can be obtained if the impervious material is placed with a water content well below optimum. In Bureau of Reclamation manuals a moisture content 1–3% below optimum is recommended, and even in countries with wetter climate conditions low placement water content are aimed at.

In 1951 this principle was attacked by *Casagrande*. An analysis of partial failures in two dams showed that even relatively small differential settlements due to a compressible foundation can result in the development of dangerous cracks in impervious fills. As differential settlements are often encountered in earth dam design *Casagrande* proposes the use of a higher water content in the core so that it will be able to accommodate itself to differential settlements. This proposal is now accepted by the U.S. Corps of Engineers, which "favours placement of all earth dam embankments on the wet side of optimum" (*Middlebrooks*, 1952).

Realizing that this means a very large increase in construction pore pressures, this criterion will result in a design with flatter slopes. In an actual case the reporter found from an analysis of the stability during construction of a 30 m high earth dam that the safety factor would drop from 1.90 to 1.05

if the placement water content increased from 1½% below to 1% above optimum. These figures show clearly the economical consequences of this question.

In order to prevent settlements on saturation it is generally accepted that the fill never should be placed at a moisture content lower than a critical value. The necessity of this criterion is clearly demonstrated in a very interesting paper by *Peterson* and *Iverson* (8/17). Two homogeneous dams constructed of plastic clay, poorly compacted at a water content far below optimum, failed during the first filling of the reservoir. Investigations showed that the failures were due to piping through fissures and tunnels resulting from a collapse in the structure on saturation. In other similar cases severe settlements were observed during the first filling of the reservoir.

Taking the three above mentioned factors into consideration the choice of placement moisture content will be a sort of "tight-rope walk" for the designing engineer. And in countries with unsettled climate daily showers may invalidate even the most deliberate choice.

In opinion of the reporter these problems are very important, and he therefore proposes them for discussion at the Conference. It would be interesting to be informed by the U.S. Corps of Engineers what modifications in the cross sections of earth dams would result from a use of placement moisture contents well above optimum. Also it would be valuable to be informed about experience with rolled earth dams which show differential settlements.

One of the most interesting papers presented in this section deals with the compressibility of rolled fill materials. *Gould* (8/10) describes the settlements observed on a number of Bureau of Reclamation dams. The measurements demonstrate the effect of the placement water content on the compressibility. Also this paper confirms that an impervious fill compacted dry is sensitive to stress changes after completion of the dam. By correlating pore water pressure and settlement observations the compression curves for the fills are derived. If the corresponding laboratory curves are corrected for the content of stones, an agreement with the field curves is found. This finding is important as it indicates that the laterally confined consolidation test reproduces with a good approximation the state of stress at least in the central part of the dam.

## (3) Seepage and Groundwater Problems

### (a) General review

Earlier, it was a characteristic feature of publications dealing with groundwater problems that they either were the result of theoretical, and often complicated, mathematical studies, or they were papers treating the problems from a purely practical point of view. To day it is a pleasure to establish that it is attempted to an increasing degree to bridge these two distinct methods of solving the same problems. The theorist shows a better understanding of the necessity of presenting the results of his investigations in such a way that they are comprehensible and useful also for the practising engineer. And in papers representing the practical point of view the limitation of empirical methods and rules of thumb are realized.

The most important difficulty in the application of theoretical investigations arises from the anisotropy and the lack of homogeneity of natural soils. Particularly is the effect of an anisotropy on the seepage pressure of decisive importance.

The discrepancies between conventional theoretical studies and observations in practice are demonstrated in several papers published since 1948. Special attention is drawn to the reports

published by the U.S. Corps of Engineers and to an interesting paper by *Walker* (8/21) presented for this conference.

After having studied recent publications dealing with ground-water and seepage problems, the reporter feels that there is an urgent need for the development of a method for reliable determination of the permeability in horizontal and vertical direction of a natural soil stratum. Owing to difficulties by sampling of pervious materials the most promising methods will probably be obtained by field tests.

#### (b) Seepage beneath dams and sheet piles

The seepage problems met with in design of dams on pervious foundations have been thoroughly investigated in the past few years. Instructive examples on results obtained by cooperation between theory and practice can be found in publications from the *U.S. Corps of Engineers*. At the Conference in 1948 investigations of the effect of partial cut-offs for controlling seepage beneath dams and levees were reported. In 1949 a number of model tests on relief wells was described in a Technical Memorandum, and it is stated that well designed relief wells can reduce effectively excess hydrostatic pressures. Various foundation conditions are investigated and design rules derived. In due time it will be interesting to be informed about experience from long time performance in practice of relief wells (danger of plugging etc.).

At this Conference a theoretical study of the effect of toe drains is presented by *Barron* (8/1). *Barron* has obtained a mathematical solution for a few simple cases, and he demonstrates that the effect of toe drains on the seepage pressure on the base of a top impervious blanket is small if the foundation shows a growing permeability with increasing depth.

Experience from early earth dams and especially from Fort-Peck Dam shows that a steel sheet-pile cut-off cannot be depended upon to reduce seepage appreciably (*Middlebrooks*, 1952). An experimental investigation of the flow of water through the interlocks of various types of sheet-piles is presented for this conference by *Jaspar* and *Ringheim* (8/15). It is interesting to study the measured flow through the interlocks; the application of the results in practice seems, however, to be limited as field conditions generally deviate too much from the laboratory conditions.

In an interesting paper, *Walker* (8/21) presents observation of seepage pressures from three Bureau of Reclamation dams. The measurements show a concentration of head loss near the toe of the dam, which only could be predicted if the anisotropic permeability conditions are taken into consideration. *Walker* (8/21) gives a short summary of geological conditions encountered in the design of dams on pervious foundations, leading to a recommendation of a semi-empirical approach based on thorough geological analysis and field observations.

The problem of hydraulic soil heaving beneath cofferdams and in sheeted excavations is still a subject for investigation and discussion. In his "Theoretical soil mechanics" *Terzaghi* succeeded in expressing the danger of heaving by a consideration of the "lifting" of a prism adjacent to the sheeting. The average head on the base of the prism remains to be estimated or computed.

Charts for the residual head at the lower end of sheeting for varying geometrical conditions have been given by *McNamee* (1949) and by *Mandel* (1951). *McNamee* distinguishes between heaving and boiling, the latter being related to the stability of a small element of the surface adjoining the sheet pile. The safety factor against boiling is calculated for different widths of a long sheeted excavation and presented in a graph. *Mandel*

calculated the danger of heaving by a static consideration of the danger that the weight of a soil column outside the sheet pile should cause a heave of the soil prism adjacent to the inside of the sheeting.

A different approach to the solution of the problem of soil heaving at dam toes and sheeted excavations, is presented by *Bazant* (8/2). Considering the stability of a segment body rather than a prism and accounting for friction along the arc, the author arrives at limiting criteria in terms of the residual head at the toe or lower end of sheeting. The theory was found to be in fair agreement with the results of a number of model tests. For the case of a single row of sheet piling the theory of *Bazant* may be compared with the *Terzaghi* theory, resulting in an agreement if the above mentioned average head at the base of *Terzaghi* prism is set approximately equal to 75 % of the residual head at the lower end of the sheet piling.

The above mentioned theories show different concepts of the mechanics of hydraulic soil heave. It would be interesting at the Conference to discuss this question, and be informed about results of experience from actual cases of piping in practice.

#### (c) Groundwater problems

Theoretical analyses are presented in papers by *Irmay* (8/14) and *Edelman* (8/6). *Irmay* presents a mathematical theory for the flow of liquids through porous media with a space-varying coefficient of permeability (by the author termed conductivity). The method of flow-nets not being applicable to cases of continuous variation of the permeability, the general equations are worked out for computation of pore-pressure in one-dimensional confined flow, as well as for the discharge in some cases of unconfined flow.

The theories developed by *Edelman* (8/6) are applied to a determination of time lag in observations of ground water levels. Tidal movements, pumping tests and inflow tests are considered in detail. For the last case the dubious result is obtained that the compressibility of the soil does not influence the results of inflow tests. This finding probably results from the erroneous assumption that the same modulus of volume change can be used for the consolidation, which takes place during the first part of the test, and the swelling, which dominates the end of an inflow test.

As measurements of pore water pressures and field determination of permeabilities receive increasing importance, fundamental research on sources of errors and irregularities in groundwater observations is needed. *Hvorslev* (1951) has contributed very much to this research with an investigation of time lag in groundwater observations. To this *Hvorslev's* report is exemplary in exact and clear representation of the results of the research.

The analysis of groundwater flow by means of analogy method measuring electric potentials in a conducting liquid is considered in three French papers: by *Cambefort* (8/3), *Chadeisson* (8/4), and *Habib et Sabarly* (8/12). All papers are apparently related to the same project, a 10 m deep excavation in pervious alluvium near a river. *Chadeisson* makes an attempt to interpret a pumping test. He demonstrates that some of the conditions governing the groundwater flow can be represented in the model tank, such as regular anisotropy ( $K_{hor}$ ) ( $K_{vert}$ ), pervious or impervious lenses, and the shape of the excavation. *Cambefort* describes the effect of the river by a drainage or by a feeding of the ground-water and proves that electric analogy in model tanks can only be obtained by eliminating the river. *Habib* and *Sabarly*, in their paper, deal with the seepage into the excavation, with and without an impervious wall extending

below the excavation bottom. Also, measurements of the electric potential representing the uplift pressure at the bottom of the proposed caisson are presented, and the factors affecting this pressure are discussed.

The practical approach to groundwater problems is represented by *Collingridge* and *Offer's* paper (8/5). Three examples on groundwater lowering in connection with deep excavations are described and discharges and piezometer observations are presented in diagrams. It is interesting to state that the design and layout of installations of even very large projects are based on permeability determinations from grain size analysis. The yield of the wells is calculated from *Dupuit's* formula. As stratified soils are also encountered in the reported cases, it would be interesting to be informed about the effect of the anisotropy in the permeability.

The surface subsidence in Mexico City is well known from the geotechnical literature. *Zeevaert* (8/22) describes the geological conditions in the Mexico valley and describes the particular soils which form the underground in Mexico City. The results of pore pressure measurements are shown and compared with the observed settlements. Most interesting is Fig. 4 in *Zeevaert* papers, showing the geotechnical data of a soil profile down to 70 m depth and including the results of shear strength, pore pressure and preconsolidation load determinations.

## Summary with Proposals for Discussion

### (1) STABILITY OF SLOPES

Two approaches are possible for an analysis of the stability of slopes. The first method is the  $\varphi = 0$  analysis, characterized by the assumptions of constant shear strength for an undrained change in total stresses. This method is extremely simple as the results of unconfined (or undrained triaxial) compression tests or vane tests can directly be inserted in the stability analysis. Its reliability has now been demonstrated in more than 30 cases.

The second approach is based on an analysis in terms of effective stresses. The procedure as used for design purposes is described in detail in the chapter dealing with stability analysis of earth dams. Analysing actual slides, both methods will show the safety factor 1.0.

*Proposals for discussion. The discussion proposed is concentrated on the limitations of the  $\varphi = 0$  analysis and the reliability of an analysis in terms of effective stresses as found from actual slides.*

### (2) EARTH DAMS

As the Large Dam Conference in Delhi in 1951 covered a number of practical questions concerning the design and construction of earth dams, the discussion proposed is concentrated on the questions: Stability analysis of earth dams, and Placement moisture content.

#### (a) Stability analysis of earth dams

Two approaches are available by analysing the stability of earth dams. The "elastic method" is based on a comparison of the actual shear stresses in the dam and the allowable shear strength of the soil.

In the limit design or surface rupture method a state of equilibrium is analysed, assuming that failure occurs along a continuous surface of rupture. In normal design limiting equilibrium is not realized and it is, therefore, necessary to postulate a change in actual conditions bringing about a state of

failure. This can be made in different ways resulting in different methods of analysis. The most logical approach to a solution of this problem is to divide  $c$  and  $\tan \varphi$  in *Coulomb's* equation by a factor, defined as the safety factor if thereby incipient failure is obtained.

It is theoretically possible to analyse the stability of an earth dam during construction and rapid draw-down in terms of total stresses. Such a procedure requires, however, that the anisotropical state of stresses in the dam is considered in the shear tests from which the shear strength values are evaluated. As this would lead to a very elaborate laboratory testing, and as the stability of the cases with full reservoir and steady seepage cannot be analysed in terms of total stresses, it is in general recommended to carry out the stability analysis by using effective stresses.

The most difficult step in an analysis of the stability in terms of effective stress is the estimate of the pore water pressures developed in impervious materials during construction and rapid draw-down.

The construction pore pressure in non-saturated soils can be calculated using *Boyle's* Law for compressibility of air and *Henry's* Law for solubility of air in water and based on oedometer tests for determination of the relation between volume change and effective stresses. The pore water pressures can also be measured in the laboratory on undrained samples subjected to increasing total stresses in a triaxial cell. The tests can be run with a constant ratio between the effective principal stresses, corresponding to a certain safety factor, or with no lateral yield of the sample, which seems to correspond to the actual state of stresses in the central part of the dam.

For estimating pore water pressures during a rapid draw-down no exact solution is available. Recent investigations, in which the pore water pressure set up by a change in shear stresses are also considered, showed that maximum values are obtained if assuming that the change in pore pressures during a rapid draw-down is equal to the change in total overburden pressures. However, in not saturated soil still higher values may exist.

In zoned dams the most dangerous sliding surface will often be composed of a circle and a straight line. Serious errors may be introduced by conventional stability analysis as the distribution of the normal forces along the sliding surface is not taken into consideration. This question is, however, still unsolved.

*Proposals for discussion. The discussion proposed is confined to the main principles of stability analysis ("elastic" method versus "surface of rupture" method, total stresses versus effective stresses) and the definition of the safety factor. Also the question of the pore water pressures (during construction and rapid draw-down) which have to be inserted in the analysis, is proposed for discussion.*

#### (b) Placement moisture content

In order to reduce the pore water pressures during construction of earth dams it is generally aimed at placing impervious fills with a moisture content below the *Proctor* optimum.

However, recent investigations of dams on compressible foundations have established that differential settlements can result in development of dangerous cracks in impervious fills. This finding has led to a recommendation of a placement moisture content of the impervious core on the wet side of the optimum so that it better will be able to accommodate itself to differential settlements.

A third factor which should be taken into consideration by choice of placement moisture content, is the danger of settle-

ments on saturation. Prevention of this phenomenon leads to the requirement that the fill should never be placed at a moisture content lower than a critical value.

As the behaviour of a fill is very sensitive to changes in moisture content a consideration of the above mentioned three factors will in certain cases make a choice of placement moisture content very difficult.

*Proposals for discussion. As topics for the discussion the reporter proposes the factors which should be considered by choice of placement moisture content including also the climatic conditions.*

### (3) SEEPAGE AND GROUNDWATER PROBLEMS

#### (a) General review

It is stated that it is to an increasing degree attempted to bridge theoretical and practical points of view in publications dealing with groundwater problems. A review of recent papers shows an urgent need for information of the anisotropy in permeability in natural soils.

#### (b) Seepage beneath dams and sheet piles

Experience and theoretical studies show that steel sheet pile cut-off cannot be depended upon to reduce seepage in pervious foundations appreciably. Recent papers describe the use of impervious blankets and relief wells, and it is stated that properly designed relief wells are effective reducing excess hydrostatic pressures beneath the downstream toe of dams. Observations of seepage pressures beneath dams on pervious foundations show a dangerous concentration of head loss near the toe of the dam, which leads to a recommendation of a careful investigation of the anisotropy in permeability.

Hydraulic soil heaving in sheeted excavations is discussed theoretically by several authors. It is distinguished between boiling, a local failure at the surface near the sheet pile, and heaving, a general upheaval of a soil prism adjacent to the sheeting. The mechanics of heaving are still an unsolved problem even if *Terzaghi's* approach to a solution is believed to give satisfactory results in practice.

*Proposals for discussion. The subjects selected for the discussion are, the problems related to pervious dam foundations with different values of permeability in horizontal and vertical directions and the question of hydraulic soil heaving, including comparison between theory and practice.*

#### (c) Groundwater problems

Since 1948 measurements of pore water pressures and field determination of permeabilities have received increased importance. Research on the sources of errors in groundwater observations has resulted in a better understanding of the time lag phenomenon. Analogy methods of measuring electric potentials in a conducting liquid are used in analysing groundwater flow.

## Sommaire et propositions pour la discussion

### 1° STABILITÉ DES TALUS

Il est possible d'analyser la stabilité des talus de deux manières. La première méthode selon laquelle on admet  $\varphi = 0$  est caractérisée par l'hypothèse que la résistance au cisaillement est constante lors d'une variation sans drainage des contraintes totales. Cette méthode est extrêmement commode en ce sens que les résultats des essais de compression simples (ou des essais triaxiaux sans drainage) ou encore des essais à l'appareil à palettes peuvent être directement introduits dans le calcul

de la stabilité. La sûreté de la méthode a été confirmée dans plus de 30 cas.

La seconde méthode fait intervenir les contraintes efficaces. Le procédé, tel qu'il est utilisé pour l'établissement des projets est décrit en détail dans le chapitre se rapportant aux barrages en terre. Lors de l'analyse des glissements qui se sont effectivement produits les deux méthodes conduisent à une valeur du coefficient de sécurité égale à 1,0.

*Propositions pour les discussions. Nous proposons de concentrer la discussion sur les limites de la validité de la méthode  $\varphi = 0$  et sur l'écart qu'on peut avoir entre le calcul à l'aide des tensions efficaces et les valeurs observées sur des glissements qui se sont effectivement produits.*

### 2° BARRAGES EN TERRE

Du fait que la Conférence des Grands Barrages (New Delhi, 1951) a traité un certain nombre de questions pratiques se rapportant à la façon de projeter et de construire les barrages en terre nous proposons de centrer la discussion sur les questions suivantes:

Calcul de la stabilité des barrages en terre; teneur en eau lors de la mise en place.

#### a) Analyse de la stabilité des barrages en terre

On connaît deux méthodes de calcul de la stabilité des barrages en terre. La «méthode élastique» est basée sur une comparaison entre les tensions de cisaillement efficaces dans le barrage et la résistance au cisaillement admissible du sol.

Dans la méthode de l'équilibre limite ou de la surface de glissement on analyse un état d'équilibre en admettant une rupture le long d'une surface continue de glissement. Dans les projets de constructions la condition de l'équilibre limite n'est, dans la règle, pas réalisée et il est nécessaire de postuler un changement des conditions effectives qui permette d'arriver à l'état de rupture. Ceci peut être fait de diverses façons et il en résulte diverses méthodes de calcul. La voie la plus logique consiste à diviser  $c$  et  $\text{tg } \varphi$  dans l'équation de *Coulomb* par un nombre, défini comme coefficient de sécurité, de façon à arriver aux conditions du début de rupture.

Il est théoriquement possible d'analyser la stabilité d'un barrage en terre durant la construction ou au cours d'un rapide abaissement du niveau dans le réservoir en introduisant explicitement les tensions totales. Un tel procédé requiert toutefois que l'on tienne compte de l'état des contraintes anisotropiques dans le barrage lors de l'exécution des essais de cisaillement au laboratoire lesquels permettront d'estimer la résistance au cisaillement. Comme cela nécessiterait une technique de laboratoire très poussée et que, d'autre part, les cas se rapportant au réservoir plein avec écoulement stationnaire ne peuvent pas être calculés en introduisant les contraintes totales, il est en général recommandé d'exécuter le calcul en se servant des tensions efficaces.

Le point le plus difficile dans le calcul de la stabilité au moyen des tensions efficaces est l'estimation de la pression de l'eau interstitielle qui apparaît dans les matériaux peu perméables lors de la construction ou d'une vidange rapide du réservoir.

La pression interstitielle dans les sols non saturés peut se calculer pour la période de construction en se servant de la loi de *Boyle-Mariotte* pour la compressibilité de l'air et de la loi de *Henry* pour la solubilité de l'air dans l'eau, et en se basant sur des essais œdométriques pour la relation entre la variation de volume et les tensions efficaces. Les pressions de l'eau interstitielle peuvent aussi se mesurer au laboratoire sur des échantillons non drainés soumis à des contraintes totales croissantes

dans une cellule triaxiale. Les essais peuvent être effectués avec un rapport constant entre les contraintes principales efficaces, ce qui correspond à un certain coefficient de sécurité, ou en empêchant le mouvement latéral de l'échantillon, ce qui semble correspondre à l'état réel des tensions dans la partie centrale du barrage.

On ne possède pas de solution exacte pour estimer la pression interstitielle lors d'une vidange rapide du réservoir. Des recherches récentes, tenant également compte de la pression interstitielle produite par un changement des contraintes de cisaillement, montrent qu'on obtient des valeurs maximales en admettant que la variation de la pression interstitielle lors d'une vidange rapide est égale à la variation de la surcharge totale des couches sus-jacentes. Toutefois dans les sols non saturés, il peut encore exister des valeurs plus élevées.

Dans les barrages hétérogènes la surface de glissement la plus dangereuse se compose souvent d'un cercle et d'une ligne droite. Il est possible que par une analyse conventionnelle de la stabilité on introduise de sérieuses erreurs, du fait que la distribution des efforts normaux le long de la surface de glissement n'est pas prise en considération. Cette question n'est toutefois pas encore résolue.

*Propositions pour les discussions:* Nous proposons de limiter la discussion aux principes généraux du calcul de la stabilité (méthode «élastique» – méthode de la «surface de rupture», contraintes totales – contraintes efficaces) et à la définition du coefficient de sécurité. Nous proposons d'inclure également dans la discussion la question des pressions interstitielles à introduire dans le calcul pour le cas de la période de construction et celui d'une vidange rapide.

#### c) Teneur en eau lors de la mise en place

Dans le but de réduire la pression interstitielle pendant la construction des digues en terre on tâche en général de placer les masques étanches avec une teneur en eau inférieure à l'optimum de Proctor.

Toutefois des recherches récentes sur des barrages reposant sur un sous-sol déformable ont montré que les tassements différentiels peuvent produire des fissurations dangereuses dans les masques étanches. Cette constatation conduit à recommander une teneur en eau à la mise en place supérieure à l'optimum de façon à permettre une meilleure adaptation aux tassements différentiels.

Un troisième facteur à considérer lors du choix de la teneur en eau à la mise en place est le danger de tassement à la saturation. Pour prévenir ce phénomène, il faut exiger que le massif ne soit jamais mis en place avec une teneur en eau inférieure à une valeur critique.

Du fait que le comportement d'un massif est très sensible à des variations de la teneur en eau il sera dans certains cas très difficile de concilier les trois facteurs mentionnés.

*Propositions pour les discussions:* Comme sujets de discussion le rapporteur propose les facteurs dont il faut tenir compte pour le choix de la teneur en eau lors de la mise en place en incluant également les conditions climatiques.

### 3° PROBLÈMES D'INFILTRATION ET DES EAUX SOUTERRAINES

#### a) Revue générale

Nous constatons que l'on cherche de plus en plus dans les publications relatives aux problèmes de la filtration à relier les points de vue théorique et pratique. Une revue d'articles récents montre la nécessité urgente d'informations sur l'anisotropie de la perméabilité des sols naturels.

#### b) Infiltration au-dessous des barrages et des palplanches

L'expérience et des études théoriques montrent que l'on ne peut trop compter sur les parafoilles en palplanches pour réduire de façon appréciable l'infiltration dans les fondations perméables. Des études récentes décrivent l'usage de tapis imperméables et de puits filtrants et l'on constate que des projets bien établis de puits filtrants réduisent de façon effective la pression hydrostatique au-dessous du pied aval des barrages. Les observations de la pression d'infiltration au-dessous des barrages fondés sur un sol perméable révèlent de dangereuses concentrations des pertes de charge au voisinage du pied du barrage, ce qui amène à recommander une étude poussée de l'anisotropie de la perméabilité.

Le soulèvement du sol dans les excavations à l'abri de palplanches fait l'objet de discussions théoriques de divers auteurs. On distingue entre le renard, rupture locale au voisinage de la palplanche et le soulèvement général d'un prisme adjacent à la palplanche. Le mécanisme de ce soulèvement est encore un problème irrésolu, même si l'on admet que la façon de procéder de Terzaghi donne en pratique de bons résultats.

*Propositions pour les discussions:* Les sujets choisis pour la discussion sont les problèmes relatifs aux fondations perméables des barrages avec des valeurs différentes pour les perméabilités horizontale et verticale et la question du soulèvement des sols avec, en particulier, comparaison entre théorie et pratique.

#### c) Problèmes des eaux souterraines

Depuis 1948 on a attribué de plus en plus d'importance aux mesures de la pression de l'eau interstitielle et à la détermination sur place de la perméabilité. Les recherches sur les sources d'erreur dans les observations des eaux souterraines ont conduit à une meilleure compréhension du phénomène des écarts de temps. Pour l'analyse du mouvement des eaux souterraines on utilise l'analogie avec la répartition du potentiel électrique d'un liquide conducteur.

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