Earth Dams, Slopes and Open Excavations

Barrages en terre, talus et tranchées ouvertes

General Report

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Introduction

The behaviour of slopes both man-made and natural, in soil and rock, must surely have aroused the interest of intelligent observers from the earliest times.

In a paper to this Conference Rao (6/31) describes some ancient earth dams in South India, the earliest being the Motitalav Tank built during the period 1 000-1 100 A.D. to a height of 80 ft. with slide slopes of 1·75 : 1 (upstream) and 1·4 to 5·6 : 1 (downstream).

The most spectacular progress in embankment building has been made, however, during the past thirty years. In this period the maximum height of earth and rockfill dams has increased approximately five-fold to over 500 ft.

Walker and Harber (6/39) give a brief account of the Trinity Dam in California, U.S.A. which when completed as scheduled in 1962 will rise to a maximum height of 537 ft. on a difficult site.

We may therefore be tempted to think that all is well in this particular field and indeed one can detect these days a feeling of complacency in much of the related literature.

The real position in our understanding of slope stability is far from secure, however, and we cannot be content with design methods that lack scientific justification.

It is pertinent, therefore, that the main topic suggested for discussion in this Division is that of Slope Stability with particular reference to long term behaviour and the effects of engineering construction.

The 39 papers submitted to Division 6, fall conveniently into the following categories :

(a) The design and construction of embankments and earth dams;
(b) Excavations;
(c) Embankment and slope failures;
(d) The theory of the stability of slopes;
(e) Miscellaneous topics.

Design and construction of embankments and earth dams

As has been mentioned in the Introduction, paper (6/39, Walker and Harber) describes the highest earth and rockfill dam yet attempted. Apart from its height and volume the construction of this dam presents problems of the stabilisation of landslide areas and considerable foundation excavation.

The paper takes the form of a progress report up to January 1960 and it is to be hoped that further information on this project will become available. Of particular interest would be the detailed analysis of the landslide problems.

A description of another proposed dam is given by Gilg and Gerber (6/16). The dam, situated in the Mattmark Valley, Switzerland, is a rockfill type with upstream sloping core and rests on extensive quarternary deposits.

Rao (6/31) and Grandi et al. (6/17) describe progress and experience with earth dam construction in India and Argentina respectively. In both these countries there is considerable activity in this field.

MacDonald, De Ruiter and Kennedy (6/25) report increased interest and activity in earth and rockfill dam construction in Canada. They give extensive information on the properties of the materials (mainly glacial tills) that have been used to provide the impervious zones of these dams.

A problem met frequently in dam construction is that of the stability of the slopes forming the reservoir that impounds the stored water. Failure of these slopes, apart from endangering adjacent facilities can create embarrassing silting problems. Finzi and Niccolai (6/13) report the successful stabilization of slopes in " sandy-gravelly soil with some content of silt and clay " forming the banks of the Monguelfo Reservoir in Italy. The technique adopted was to place a twin-zoned shell, the outer zone being of dumped and sluiced rockfill and the inner of transition material. The construction was successfully tested by filling and drawdown before the reservoir was put into service.

In commenting on engineering works that are either completed or in the design stage the General Reporter faces an almost impossible task in seeking to offer constructive suggestions. Successful completion of work obviously means that the primary requirements of safety is satisfied, but only the designer is in possession of the information necessary to answer the question " was it completed in the most economical manner? ".

In this context the paper by Sowers and Gore (6/35) serves to emphasise the importance of field tests as an adjunct to design as a major contribution to overall economy. These authors describe full scale construction tests on weathered sandstone-sandy clay and on broken sandstone, supplemented by in-place permeability and shear tests.

In the latter tests they used a large shear-box (6 ft. × 6 ft. × 1 ft. 6 ins. deep) to determine the properties of broken rock. In commenting on the values of the angle of friction obtained in the field (45°) and in the laboratory triaxial compression tests on the minus 5 mm material (42°) the authors suggest that the difference may be attributed to the presence of the larger rock pieces.

The writer cannot see that size alone should account for the difference, unless there is a distinct change in mineralogical character and/or shape of the larger pieces. In view of the differing test conditions it is considered that the agreement between laboratory and field tests is reasonably good. Although it has been suggested from time to time that size alone may influence these test results, in the writer's opinion the field tests will have to be conducted with the same degree of precision as has been obtained in the laboratory before a satisfactory conclusion can be made; and this will entail careful consideration of volume changes in relation to the applied stresses.

The paper by MAYER and HABIB (6/27) is concerned with ensuing stability of an embankment when there is little or no control over the materials of construction. The authors give an excellent account of the combination of field observation with engineering analysis leading to successful control of two steel-plant slag dumps.

The analysis, at least as far as the in-situ soil is concerned, was carried out using applied stress parameters derived from undrained and consolidated-undrained shear tests. Subsequently, the authors report field observations of the pore pressures developed in these foundation soils. It would appear therefore that this paper would be even more valuable if stability analyses based on effective stress parameters had been made so that the influence of the pore pressures could be assessed more accurately.

An interesting development is reported by ROCHA, FOLQUE and ESTEVES (6/33). These authors have considered the possibility of using cement stabilized soil (cement percentages 2-5 per cent) in the construction of earth dams. Whilst the work is still only in the exploratory stage it is of sufficient interest to warrant continued attention. The authors rightly emphasise the potential difficulties that may arise with the development of stiff-brittle characteristics in these materials and the consequent need for a thorough evaluation of current design processes in relation to these characteristics.

Although not reported to this Conference, one of the most significant developments in dam construction in recent years is that of the self-spillway rockfill type described by Wilkins [1956]. In these dams, stability is provided by rockfill shells with an upstream sloping core to retain the water. Spillway capacity is provided by stopping the sloping core below the crest and permitting overflow through the downstream rockfill.

Excavations

Forming open excavations is frequently far more hazardous than embankment construction. In embankments the designer usually has a good measure of control over the materials to be used and, particularly in earth dams, can exercise considerable care in their selection.

In cuttings, however, the material has to be accepted in its natural state and every soil engineer is aware of the likely variation of ground conditions.

Cuttings also differ from embankments in that they normally involve considerable stress relief of the in-situ soils with the consequent problems of expansion and effective stress change in highly overconsolidated clays; and our knowledge of the properties of these materials is very limited.

Eight of the papers submitted to this division of the Conference are concerned with some aspect of excavations.

Further evidence of the use of accepted techniques for seepage control, such as grout curtains, electro-osmosis and sand drains, are reported but a novel development is the use of drilling mud (bentonite suspensions) to provide support for trenches during excavation. As far as the author is aware this method has not been extensively reported in engineering literature prior to this Conference.

Two papers, both from France and very similar in content, deal with this aspect of construction. CHADEISSON (6/9) describes the use of percussion drilling equipment (mounted on rails and capable of transverse movement) in forming continuous trenches. The trench is supported during excavation by circulating drilling mud and concrete is then placed by tremie methods to form a continuous diaphragm. The practice that has developed is that of forming alternate panels of the diaphragm at the same time. Examples are given of the use of the method in cofferdam construction, cut-off walls, a bridge pier and a retaining wall forming the basement of a building. Unfortunately no indication of the maximum trench width is given but it is claimed that depths of 50 m can be attained.

The paper by BARBEDETTE and BERRA (6/4) describes the patented " Titania " process which differs from that dealt with by Chadeisson primarily in the digging tool used. The " Titania " method employs a rotary instead of a percussion drill. There is also provision for a temporary metal sheath to act as shuttering for the concrete when alternate panels of a continuous diaphragm are being formed. The authors also point out that the method may be used for under-reamed bored piles with shaft diameters up to 1·3 m and enlarged bases of 2·3 m diameter. The maximum width of trench quoted is 1 m and the maximum depth 30 m.

It is clear that these developments are an important contribution to engineering construction and the methods described will obviously find wide application where other method are either uneconomical or impracticable.

Another important development is that dealt with by TSYTOVICH and KHAKIMOV (6/38) who report continued progress in the application of artificial freezing of the ground as an aid to construction. Although ground freezing has been used in this way for many years, these authors have done much to place its use on a more rational and scientific basis. The gap between engineering construction and scientific appraisal is frequently so wide that any attempts to bridge it are most welcome. The paper describes the successful application of this technique to the sinking of deep shafts for underground railways, to the protection of the foundations of hydraulic structures and to the sinking of deep shafts in unstable soils for mining operations.

The real value of the paper lies however in the development of expressions for estimating the freezing time and time of closing of ice-soil cylinders (Khakimov) (it is claimed that field observations of freezing time are within 15 per cent of the predicted value) and the attention paid to the problem of creep (Tsytovich) in ice-soil systems. Of particular interest is the conclusion reached by the last named author that the frictional component of the long-term strength of frozen soil is independent of the negative temperature and may be taken equal to the angle of internal friction of the un-frozen soil.

It would be of considerable value to know however whether the angle of internal friction of un-frozen soils referred to by the author is determined in terms of applied or effective stresses, as presumably the value for frozen soils is an applied stress parameter.

Yet another method of ground water control which has received increasing attention in recent years is that utilizing sand drains in one form or another. Two of the papers in this Division are concerned with this aspect of construction. OTT, BERG and CHAPPUIS (6/29) describe investigations concerned with the construction of the Reichenau dam in Switzerland. The dam is founded on alluvial deposits and the groundwater regime is separated from the impounded water by an impermeable layer. Model studies were made of the effect of the ground water on the associated hydraulic structures using the reo-electric method. The authors concluded that in these conditions vertical drains are more effective than horizontal drains. HOLM (6/19) deals with the question of the stabilisation of a slope in the notorious Norwegian quick clays. It was decided that owing to proposed industrial construction and local evidence of previous slips, stabilizing measured incorporating vertical sand drains were warranted. The paper describes the details of the preliminary investigations and the installation of these drains together with an appraisal of their performance in service.

During the past twenty years or so the use of electro-osmosis as a construction aid has been pioneered by the first named author of paper 6/8, L. CASAGRANDE. This paper gives yet another example of the success of this method in appropriate circumstances. During the construction of the foundations for a bridge across the Little Pic River, Ontario, Canada a slip occurred in the non-plastic silt (rock flour) that forms the eastern bank of the river. The slope at the time was estimated to be at 1 : 2·5. After electro-osmotic treatment the work was successfully completed with the slope in the silt standing at 1 : 1 on a height of 50 ft. In the General Reporter's opinion the evidence presented in this paper is a notable example of the contribution of tension in the pore water to the strength of these soils. Although the authors recognise the development of this pore water tension, particularly in the laboratory tests, it is perhaps regrettable that they should refer to it as a 'by-product' of the drainage process. These tensions are surely essential to the success of this process.

It would also be of interest to the Conference if the authors could offer further comment on the question of stability after completion of construction. Do they envisage any permanent increase in strength of this material independent of the ground water level? It appears that on cessation of the electro-osmotic treatment the strength obtained during construction could deteriorate progressively with the rise of ground water level and the consequent relief of water tension.

The last paper to be dealt with in this section concerns a topic on which there is very limited information in engineering literature, namely, ground displacement following major excavations.

In paper 6/40 WARD discussed the effect of a 40 ft. deep excavation in London clay on the alignment of and stress conditions in the underlying underground railway tunnels. Most comprehensive and careful instrumentation was used and the results were correspondingly precise. It is interesting to note that the author concludes that the vertical movements observed in the tunnels followed elastic behaviour and in general these movements lie within the values predicted on the basis of simple elastic theory and reported at the last Conference [Williams, 1957]. Following these observations an attempt has been made to calculate the effective Young's Modulus for London clay and the mean value obtained, 1 600 tons/ft^2, compares well with a previous estimate of 1 200 tons/ft^2.

Embankment and slope failures

The examination of full scale failures and the analysis of the conditions leading to failure of slopes are most necessary and fruitful studies. Indeed it has frequently been observed that engineering progress depends on the stimulus of failure. In this group there are 6 papers that deal with various aspects of this question.

In paper 6/21 ISHII, KURATA and HASEGAWA describe the failure of relatively low (18 ft.) embankment built across Kinkai Bay, Japan, for reclamation purposes. The foundation appears to be a very soft, normally consolidated marine clay. On the basis of unconfined compression tests carried out prior to construction it was estimated that at all stages of construction a minimum factor of safety of 1·2 would be obtained. Shortly after the water on the inside of the embankment had been drawn down, failure occurred. The analysis of failure given by the authors is not very convincing. In the first place the major cause of failure is attributed to the fact that the average in-situ density of the fill was 2·1 ton/m^3 as compared with the assumed value of 1·8 ton/m^3. It would appear that this discrepancy would have been sufficient to reduce the construction factor of safety (1·2) to just below unity before drawdown. Furthermore, it is not clear in the paper how the authors arrived at the shear strength values used in the analysis. One wonders whether applied stress strength values have been used in an effective stress analysis, and the General Reporter would be glad to be reassured on this point.

Two of the papers (6/26 and 6/11) are concerned with the failure of river embankments under flood conditions. MARSLAND (6/26) describes the failure of such an embankment in the Thames Valley, England, in 1953.

Following a most thorough and painstaking examination of the field conditions supplemented by exhaustive laboratory tests and in-situ measurements of pore pressures he shows conclusively that failure was due to uplift pressures in a gravel layer under the embankment. This gravel layer had direct access to the tidal waters so that the extreme head of water produced by the high tides was transmitted directly to the layer under the embankment.

The author also demonstrates that instability under these conditions could have been predicted from existing methods of effective stress analysis.

This paper is an object lesson in the value of careful evaluation of failure mechanisms supplemented by appropriate field and laboratory tests.

DOMJAN (6/11) describes conditions in an embankment on the River Danube in Hungary similar to those discussed by Marsland. Donjan's work however does not reach the standard of the previous paper. His experimental technique designed to illustrate the effect of pore air pressures on the maintenance of positive pore fluid pressures is ingenious and has undoubted application in soil mechanics, but it is difficult to understand its particular relevance in the present context.

It would appear that the hydraulic conditions ($i = 1$) necessary to initiate the piping described by the author are present in the gravel layer if, as he mentions, the uplift pressure in this layer is virtually equal to the head of water in the river.

The remaining papers in this group deal with failures in natural slopes of rock and soils.

FUKUOKA (6/15) describes investigations on a slide in Japan of such magnitude that a number of dams are involved. The chief interest in this paper lies in the techniques used to study the movements concerned. Sliding surfaces are located by measuring the distortion of deep boreholes with the aid of clinometers.

A similar technique has been reported by Wilson [1959] in relation to rock movements in deep open cut mines and the General Reporter has been concerned recently with the use of this method to study movements in 200 ft. thick seams of brown coal in the Latrobe Valley, Victoria, Australia.

Another interesting development is the measurement of radio activity at the ground surface.

According to Fukuoka, fracture of rocks at depth will permit the rise of radon gas through the fissures with consequent increase in surface radio activity.

The method proposed for arresting the movement is that of horizontal drainage galleries driven to the shale layer that is considered to be the main sliding layer. The location of these horizontal drains has been materially assisted by the chemical analysis of ground waters.

SUKLJE and WIDMAR (6/37) give a most detailed account of the investigation of a major landslide at Gradot in Yugoslavia. In some respects the Gradot landslide is similar to that described by Fukuoka in that the authors consider the failure surface to occur in a highly over-consolidated clay layer that is overlain by sandstone, conglomerate and tuff.

According to their hypothesis the 'cohesion' of the overburden has been overcome decades or centuries before the sliding owing to the unequal deformation of the clay layer, and this situation has been aggravated by earthquakes. They visualise that just before sliding the strength of the clay justified the assumption that $C' = 0$, implying that the effects of over-consolidation had been overcome during the long-term creep. With this assumption and using extensive test results they derive a mean factor of safety of 1·14.

The authors then observe that this factor of safety " covers the effect of long term creep on the decrease of the friction resistance". It is difficult to see the justification for this conclusion from the paper. However the complexity of the problem is such that at the present state of knowledge the estimation of the factor of safety within 15 per cent could be regarded as encouraging. In general terms the authors' hypothesis certainly appears reasonable.

HENKEL (6/18) also described slide movements in a clay layer that is interbedded with rock strata. In this case, however, the movements took place along a plane surface obliquely to the dip of the strata. A method of analysis has been developed for this case by the author. The resulting calculations offer some evidence that the empirical assumption of $C' = 0$ is applicable in these circumstances. As he correctly observes, however, it is not possible to obtain a precise answer owing to the simplifying assumptions involved in the analysis and the complexity of the problem. Nevertheless there is supporting evidence in the stability of nearby slopes that the results obtained are reasonable.

These last three papers serve to emphasise the increasing inter-relation between the fields of soil mechanics and the more recent developments in rock mechanics. In view of the occurrence of bedding planes and joints in most rock masses they cannot be considered as continua but must rather be analysed as discrete solids, particularly where large rock masses are concerned.

Some preliminary analytical work along these lines [Trollope, 1960] has shown close correspondence between the behaviour of idealised rock systems when compared with that of similar masses composed of sand or other granular material.

It appears likely therefore that many of the concepts of soil mechanics will be directly applicable to stability studies involving in-situ rock and this approach offers much promise.

The theory of the stability of slopes

Until quite recently it has been generally accepted that the first successful attempt at evaluating the problem of deep-seated slope movements was due to Petterson [1916]. Development of Petterson's method by Fellenius led in turn to the now well known Swedish Circular Arc Method of Stability Analysis.

In 1956 however the University of Toronto Press published a most remarkable book entitled " Landslides in Clays " by Alexandre Collin [1846). This book is a translation of Collin's pioneer work during the early part of last century, in which he recognised the rotational character of slips in clay soils and was also the first to use measured values of the shear strength of these soils in stability analysis. In his foreword Mr. Robert F. Leggett describes the leading role he played in what may well be called the romance of the re-discovery of Collin's work. The translation of Collin's book is accompanied by a most extensive and illuminating biography by Professor A. W. Skempton.

Without detracting from the value of the work of the Swedish engineers in the early part of this century it is fitting that we should at this International Conference, the first to be held in France, pay tribute to this eminent French engineer and accord him his rightful place in the history of our subject.

Space does not permit here a consideration of the various attempts dating from Coulomb [1773] and subsequently followed by Français [1820] and Culmann [1866] (see Skempton — in Collin [1956]) in which plane slip surfaces were assumed.

It is clear however that by 1930, the analytical method of assuming plane-strain with a circular slip surface and assessing the shear strength in terms of applied stress parameters (c, φ) had become widely known.

The next major development, the introduction of the effective stress concept into stability analysis, had taken place by 1936.

In the First International Conference on Soil Mechanics and Foundation Engineering, Terzaghi [1936] described the modification of the Swedish Method of slices to take into account the influence of pore pressures. At the same time May [1936] published the method in use at the U.S. Bureau of Reclamation.

Two other equally important developments taking place at this time were the recognition of the need to express the shear strength of soils in terms of effective stress parameters (c', φ') and the consequent interest in the measurement of pore pressures both in the laboratory and the field.

As far as the writer is aware the U.S. Bureau of Reclamation were the first to combine effective stress stability analyses with laboratory and field measurement of pore pressures as a routine procedure for the design of earth dams, and with minor variations the method is in widespread use at the present time.

Parallel with the development of the method of slices many writers turned their attention to other graphical methods. Of these the friction circle method has proved the most popular, principally owing to Taylor's [1937] classical analyses. Because the friction-circle assumption demands, a priori, the vectorial separation of the c (cohesive) term and φ (frictional) term the custom developed of expressing factors of safety with respect to cohesion and friction separately. This led to considerable confusion in the meaning of these factors when compared with that customarily used in the slices method — where the definition is that of the ratio of activating moment; resisting moment. It is interesting to note therefore that two of the current papers, FRÖHLICH (6/14), DE BEER and LOUSBERG (6/6) revert to this latter method of defining the factor of safety, although De Beer and Lousberg develop a preference for expressing the factor in terms of the activating force vector. This introduces no difference when simple circular arc sliding surfaces are involved but, as the authors point out, this definition can be used in cases where the failure surface is non-circular.

In the General Reporters opinion the expression of the factor of safety in terms of moment equilibrium remains the most satisfactory at the present time. Of the two methods reported here that by De Beer and Lousberg (referred to as S_1) is preferable because it involves less ambiguity in the definition of the additional moment required to cause rupture than does Frohlich's 'impulse'.

Too much importance should not, however be attached to the 'exact' calculation of factors of safety at this stage. The intrinsic errors in these methods of analysis preclude calculation of a 'correct' value and the selection of a factor for design purposes still demands careful engineering judgement.

In a paper by ESCARIO (6/12) the author seeks to emphasise the limitations of the conventional slices method. Adopting an extreme set of assumptions he calculates a negative value for the lowest factor of safety. Accepting these assumptions the General Reporter has been able to verify that negative values are obtained in this way. This is not surprising as it follows from the assumed values of pore pressure. What is surprising, however, is the big difference between these values and those obtained, according to the author, using e.g. Bishop's method. Apart from magnitude, the difference is in reverse order to that previously claimed [Bishop, 1955]. In the absence of detailed calculations it is difficult to comment on the observations.

It is suggested that the author should make available typical calculations for each of the methods used, so that readers may have an opportunity of assessing these points further.

An alternative approach to the problem of slope stability which has received increasing attention in recent years is that of examining equilibrium conditions along assumed failure surfaces utilising Kotter's equation of modifications thereof (RODRIGUEZ 6/34). The principal weakness of the analyses so far published, however, is that they do not consider pore pressure (u) as an independent variable and Carillo [1942] has pointed out that neglect of this factor introduces significant errors.

A similar objection may be raised in connection with the paper by KOPACZY (6/23) who examines the problem of slope stability using the classical concepts of plastic rupture in a c/φ material. There is no guarantee, however, that solutions obtained in this way are correct, even for idealised c/φ materials. The 'correct' solution must satisfy both static and kinematic requirements, and so far the only technique evolved has been that of establishing upper and lower bound solutions [e.g. see Prager and Hodge, 1951]. The solution given by Kopaczy is likely to be one of the lower bound solutions.

It must not be thought that the General Reporter is deprecating the work in these last two papers. Such theoretical investigations are indispensable to the progress of the subject and are to be encouraged. Nevertheless it cannot be expected that they will replace existing design methods for the stability of slopes until full account is taken of effective stress conditions.

Another interesting theoretical development is that given by STROGANOV (6/36) on the application of visco-plastic concepts to the behaviour of soils. Little is known of this aspect of rheological behaviour in soils and this contribution is important in that it opens up ideas new, at least to the General Reporter.

In view of recent attempts to define the shear strength of clays more accurately, the expression used by Stroganov for the failure stress at zero shear strain velocity is of particular interest. The expression given is

$$\tau = (H + \sigma) \tan \psi$$

where τ represents the intensity of tangential stress
H — cohesion
σ — the mean normal stress
and $\tan \psi$ — the friction coefficient for an octahedral area or "Botkin constant".

The implication that the 'cohesive' strength component may be represented by an effective stress term ($H \tan \psi$, where ψ is a fundamental friction coefficient) is in accord with a hypothesis advanced by the General Reporter [Trollope, 1960].

One also finds considerable sympathy with the theme of BALUSCHEFF's paper (6/3) in which the author emphasises that failure of a slope may frequently occur after overstressing of the foundation. Although it has been shown [Trollope and Morgan, 1959] that the shear stresses that can be developed at the base of an embankment can greatly exceed those used by the author this aspect of failure is worthy of continued attention.

In their paper on the stresses and deformations in cores of rockfill dams, NONVEILLER and ANAGNOSTI (6/28) assume that owing to differential settlement of the shells and the core a state of limiting plastic equilibrium may develop in the latter. Evidence of field pressure measurements that are in reasonable agreement with the theory are also presented. Although the analysis suffers from all the inherent defects of the assumption of an idealised c/φ material it is nevertheless a most useful contribution in that it illustrates how internal stresses are likely to be distributed in these circumstances.

Finally, as far as this group of papers is concerned, the paper by LANE (6/24) is a most solutary contribution. The author presents charts on stability derived from observation of natural slopes in a given material. Althought it is too much to hope that this alone will yield sufficient design information it is well to remember that such observations are a most valuable aid to theoretical and laboratory investigations.

Miscellaneous topics

There remain in this Division seven papers that, although concerned with related aspects, do not fit readily into the previous groups.

Two of these are concerned with transient soil water flow conditions. That by BROWZIN (6/7) deals with the flow in homogeneous earth dams following sudden drawdown. This is an important topic related to stability and the author has made a valuable contribution both theoretically and experimentally.

KASHEF (6/22) has studied the transient flow conditions in artesian wells. This is a most difficult problem to solve exactly and the author appears to have developed a finite-difference solution which is sufficiently accurate and much simpler than previous solutions.

PATEL and MAHESHIVARI (6/30) have made a most interesting and valuable study of the influence of the location of an upstream sloping filter on the flow net and hence the stability of earth dams. Their work is based on model experiments in which steady-state conditions were developed. Consequently care must be taken in extrapolating their findings, in the case of complete drawdown, to materials that are not free draining. The whole question of transient flow conditions following draw-down is more complicated than the authors have assumed, as Browzin's (loc. cit) work shows, and this topic is worthy of more detailed attention.

Both BAZANT (6/5) and HUANG WEN-XI (6/20) are concerned with aspects of the stability of saturated sand under dynamic loading. The former author criticizes the interpretation of previous tests results and seeks to show how they should be represented in terms of dimensional analysis.

The second author reports a series of experiments on the measurement of excess pore pressures induced by dynamic loading. The results confirm the development of these excess pore pressures and this is related to the failure of sand by liquefaction. This is a most valuable contribution, although it is limited at present by the fact that the applied stresses in the tests, are not completely known. It is to be hoped that the author will continue work along the lines he suggests, utilizing triaxial compression control.

COEN et al. (6/10) give a most valuable account of extensive field and laboratory investigations into the properties of grouting mixtures. Their results will doubtless be of considerable interest to engineers concerned with this aspect of construction control.

The last paper to be noted is that by AISENSTEIN, DRAMANT and SARDOFF (6/2) describing the use of a fat (montmorillonitic) clay as an impervious blanket to seal a leaking reservoir. The authors report failure of the seal when not supported continuously and ascribe the failure to a mechanism similar to piping in which the clay fails owing to the development of high hydraulic gradients at the bottom of the blanket. From this they conclude that the 'effective blanket' is confined to the lower 25-30 cm depth of soil and that this should be protected by a covering layer to prevent drying out and to resist excessive swelling. The authors do not mention the influence of the dispersing characteristics of the clay on accession of water. The General Reporter would like to suggest that this could be a significant influence on the suitability of clay soils for this purpose, and in this context Skempton's 'activity' could be a most useful aid in preliminary classification.

Summary and Conclusions

The principal conclusion to be made at the present time is that the slip circle method, using effective stress parameters, remains the most satisfactory method for the analysis of slope stability. Despite important developments in other theoretical studies, chiefly those incorporating theoretical concepts of limiting plastic equilibrium, it is not to be expected that these will seriously challenge the conventional approach for general use until the more advanced theories take effective stress conditions more fully into account.

The obvious interest of a number of workers in various parts of the world in the more sophisticated theories augurs well however for future development along these lines.

After a period of about 25 years in which there has been considerable confusion in the use of factors of safety with respect to cohesion and friction — primarily in the use of the friction-circle method — there is some indication of a welcome return by protagonists of this method to the definition of the factor of safety in terms of the overall ratio of activating to resisting force or moment vectors, (De Beer and Lousberg (6/6) Fröhlich (6/14)). This should lead to a closer (but not exact) correspondence between the factors computed by both friction-circle and Swedish (slices) method of analysis.

It must not be concluded, however, that all is well and that we need not look further than circular arc methods in stability analysis. The fact remains that there is no theoretical justification for their acceptance and in the General Reporter's view they have been successful despite their obvious inaccuracies and not because of their close approach to reality.

The most likely explanation appears to be that the errors inherent in the assumed stress distributions (activating) are compensated for by the neglect of the contribution of negative pore pressures (water tension) to soil strength (resisting).

It is a salutary thought that, if the development of water tension is accepted; and there is sufficient evidence available to permit its acceptance; then few if any of the analyses of slope failures in the field carried out up to the present time are valid because of neglect of this factor.

In his General Report to this Division of the last Conference Walker [1957] observed that the role of water tension should assume increasing importance in the assessment of slope stability. It is regrettable therefore that so little attention is paid to this topic in the papers to this Conference.

There is yet another aspect of the influence of water tension that deserves mention and this concerns the field measurement of pore pressures. Where piezometers of the type in which the water in the measuring system has access to the surrounding soil through a porous disc, are used in unsaturated materials, a significant unknown zero error is introduced into the measurements. From the aspect of construction control the error is in a favourable or safe direction but it detracts from the accurate assessment of the in-situ stress conditions.

The most significant development in construction is that incorporating the use of drilling mud to permit continuous trench excavation without additional support (Chadeisson 6/9, Barbedette and Berra 6/4). This technique will undoubtedly have increasing application in both foundation and cut-off wall construction.

The long term stability of slopes and the effects on engineering construction—The Organizing Committee has suggested that discussion at the Conference should centre around these topics. The General Reporter's task in this regard is made difficult in that of the 39 papers in this division only two contribute directly to this subject. (Henkel 6/18 and Suklje and Vidmar 6/37). Both these papers offer further evidence in support of the $c' = 0$ hypothesis. An attempt will therefore be made to assess this hypothesis in the light of recent developments.

The statement that $c' = 0$ in long-term stability analyses is an empirical one derived from observation supported by field evidence [Skempton, 1957]. It has, however, important implications in the behaviour of soils in shear. The principal implication is that the effects of overconsolidation are dissipated in the course of time. It has been well established that, on stress release, clay soils tend to swell and unless adequate free water is available pore water tensions result. The rate at which the shear strength decays is therefore a function of the availability of free water. A further significant factor, however, is the evidence offered by Roscoe, Schofield and Wroth [1959] and supported by Parry [1959] concerning the concept of a critical void ratio for clays in shear. According to this concept overconsolidated clay soils will tend to dilate in shear and so also contribute to the development of water tension.

It may be concluded, therefore, that the dissipation of pore water tension with time is an important factor in explaining the decrease of shear strength leading to long term instability.

An equally important consideration in the analysis of stability is the possibility of an increase in shear stresses in a slope with time. It has been shown [Trollope and Morgan, 1959] that such increases can occur in granular materials owing to differential foundation movements and it has been suggested that the heave following slope excavation provides the necessary deformation conditions for these increased stresses to develop. The type of deformation analysed by Nonveiller and Anagnosti (6/28) in their paper to this Conference can similarly contribute to an increase in shear stress.

It would appear therefore that both strength loss and stress increase are possible contributors to loss of stability with time.

Yet another factor is the possibility of significant tension forces, particularly in steep slopes. The existence of these forces has long been acknowledged and in their description of the Gradot landslide Sulkje and Vidmar (6/37) describe very deep tension cracks. Where such tension forces do exist then, in clay soils, they can only be resisted to any significant degree by pore water tension. The release of pore water tension by intermittent water accession could therefore also contribute to progressive slope deterioration.

In an attempt to provoke discussion a hypothetical summary of the possible stress conditions in soil slopes is given in Fig. 1. It should be emphasised however that this summary is tentative and is only intended to indicate a possible regime as a basis for discussion.

The extreme outer configuration AB is the 'arch' formation described by Sokolovsky [1957]. Maintenance of this profile is dependent on the idealised properties of a c/φ material in which presumably it is implicit that the c term is a measure of the invariant tensile strength of the material.

The line AB is analogous to the 'dry' angle of repose for a clay soil. Its significance is that, if clay soils may be considered as granular materials formed of domains [Trollope, 1960], then at slopes less than AC these soils can adopt an equilibrium arrangement in which there are no tensile forces between particles (domains). At slopes greater than AC, tensile forces between particles are necessary for equilibrium.

It is doubtful if the line AC has any physical significance as it is unlikely that a ‘dry’ clay soil exists naturally. The ultimate ‘angle of repose’ for a clay soil is that represented by AD where seepage occurs parallel to, and the phreatic line is coincident with, the surface AD [see Skempton, 1957]. Significant tensile stresses can be resisted by clay soils only if a continuous water phase in tension exists. The permanent retention of this tensile strength is therefore unlikely, in view of dessication and/or water accession.

It is visualized therefore that the failure mechanism in slopes steeper than AD is initiated by tension failure followed by progressive deterioration of strength as ground water flows down the resulting tension crack, until failure occurs along a surface such as EFA. Provided free water is continuously or intermittently available, this process will be repeated until the ultimate slope AD is reached.

Slopes flatter than AD are unlikely to fail unless foundation deformations are excessive.

Following the above argument it is suggested that discussion be centred on the following topics:

1) The factors contributing to the decrease of soil strength with time in slopes;

2) The possible variation of internal stresses in slopes and embankments with time;

3) Do significant tension fields exist in clay slopes and if so can clay soils resist these tensile forces?

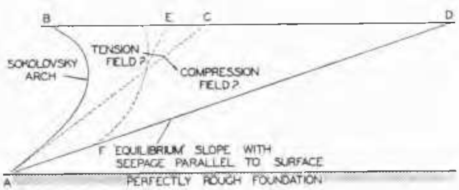

Fig. 1

Résumé et Conclusions

La principale conclusion à tirer à l'heure actuelle est que la méthode du cercle de glissement avec pour paramètres les contraintes effectives, reste la plus satisfaisante pour l'analyse de la stabilité des talus.

Malgré l'importance d'autres études théoriques, en particulier de celles comportant l'application de la notion de l'équilibre limite en plasticité, il ne semble pas que l'on puisse envisager pour le moment une généralisation dans les applications usuelles.

Par contre, l'intérêt qu'un grand nombre de chercheurs portent à ces études permet de prévoir qu'un jour ces théories plus compliquées pourront aboutir. Après une période de vingt-cinq ans pendant laquelle il y a eu une certaine confusion dans l'application des coefficients de sécurité, en ce qui concerne tout au moins les valeurs de la cohésion et du frottement à faire intervenir dans le calcul du cercle de glissement, il semble que l'on revienne à la définition rationnelle du coefficient de sécurité comme le rapport des forces actives aux forces passives (De Beer et Lousberg (6/6) Fröhlich (6/14)). Ceci devrait conduire à une correspondance meilleure sinon exacte entre les éléments calculés par la méthode du cercle de glissement et par la méthode suédoise des tranches.

Il ne faut pas en conclure cependant que tout est pour le mieux et que nous ne devons pas chercher plus loin que la méthode du cercle de glissement en matière d'analyse de stabilité. Il reste le fait que cette méthode n'a pas de justification théorique et votre Rapporteur Général estime que les résultats satisfaisants qu'elle donne sont obtenus malgré des défauts évidents et non parce qu'elle serre de près la réalité.

L'explication la plus vraisemblable paraît être que les erreurs de la répartition des contraintes sont compensées par le fait que l'on néglige la participation de la tension superficielle à la résistance du sol. Lorsque l'on tiendra compte de l'existence de cette tension superficielle, et la preuve est faite de la nécessité d'en tenir compte, un petit nombre seulement des analyses de rupture d'équilibre des talus, telles qu'elles ont été faites jusqu'ici, restera valable.

Dans le Rapport Général présenté à cette même Section à la dernière conférence [1957] M. Walker a fait observer que l'on devrait progressivement attacher une importance croissante à la tension superficielle dans le calcul de l'équilibre des talus. Il est regrettable que ce point ait aussi peu retenu l'attention dans les rapports présentés à cette Conférence.

Il y a encore un autre aspect de l'influence de la tension superficielle qui mérite d'être indiqué ; c'est celui qui concerne la mesure sur le terrain.

Lorsque les piézomètres, où l'eau est en contact avec le sol par l'intermédiaire d'une paroi poreuse, sont utilisés dans des terrains non saturés, les mesures comportent une erreur à l'origine indéterminée. D'après le mode de construction des appareils cette erreur est dans le sens de la sécurité. Néanmoins elle ne permet pas de connaître les conditions de contrainte exactes.

Le perfectionnement le plus remarquable en matière de travaux est l'utilisation des boues de forage pour permettre l'exécution de tranchées continues sans soutènement (Chadeisson 6/9, Barbedette et Berra 6/4). Cette technique aura certainement des applications de plus en plus importantes aussi bien en matière de fondations que pour l'exécution d'écrans d'étanchéité.

Stabilité des talus. Evolution en fonction de temps. — Le Comité d'Organisation a proposé que la discussion à la conférence soit centrée sur ce point. La tâche du Rapporteur Général à cet égard est rendue difficile par le fait que sur les 39 rapports de la Section 6 deux seulement s'appliquent à ce sujet : Henkel (6/18) et Suklje et Vidmar (6/37). Ces deux rapports apportent des éléments complémentaires à l'appui de l'hypothèse $c' = 0$.

Je ferai donc un effort pour présenter cette hypothèse à la lumière des études récentes. Le fait qu'en matière d'analyse de stabilité à long terme il convient de prendre $c' = 0$ a un caractère empirique [Skempton, 1957]. Il a cependant des conséquences importantes quant au comportement des sols au cisaillement. La première est que les effets de la surconsolidation se dissipent en fonction du temps. Il est bien établi que lorsque les contraintes se relâchent les sols argileux ont une tendance à gonfler et qu'il en résulte des tensions capillaires sauf dans le cas où il y a de l'eau libre disponible.

Le degré de détérioration de la résistance au cisaillement est donc une fonction des disponibilités en eau libre. En outre Roscoe, Schofield et Droth [1959] ainsi que Parry [1959] ont démontré l'existence d'un volume critique des vides dans les argiles au moment du cisaillement. D'après eux les argiles surconsolidées tendront à gonfler et provoqueront également des tensions capillaires. La dissipation de la tension superficielle dans le temps est donc un facteur important susceptible d'expliquer la réduction de la résistance au cisaillement conduisant à une instabilité à long terme.

Une considération d'importance égale dans l'analyse de la stabilité des talus est la possibilité de l'augmentation des contraintes de cisaillement avec le temps. Trollope et Morgan [1959] ont montré que des modifications de cette nature peuvent se produire dans des matériaux granuleux du fait de mouvements différentiels de la fondation et il semble que le gonflement qui suit le découpage d'un talus puisse provoquer les déformations nécessaires pour produire cette augmentation des contraintes.

Le type de déformations analysées par NONVEILLER et ANAGNOSTI (6/28) dans leur rapport peut également provoquer une augmentation des contraintes de cisaillement. Il semble donc que la diminution de la stabilité en fonction du temps peut s'expliquer à la fois par une réduction de la résistance et une augmentation des contraintes. Il faut en outre indiquer la possibilité d'efforts importants de traction, en particulier dans les talus à pic.

L'existence de ces forces est connue depuis longtemps et, dans leur description du glissement de terrain de Gradot, SUKLJE et WIDMAR (6/37) donnent la description de fissures de traction très profondes.

Lorsqu'il existe des contraintes de traction dans des terrains argileux il n'y a pratiquement que les tensions capillaires qui puissent les contrebalancer. C'est pourquoi la réduction des tensions capillaires par l'intervention d'eau supplémentaire peut aussi participer à la désorganisation progressive des talus.

Pour faciliter la discussion, j'ai résumé dans la Fig. 1, les états de contrainte possibles dans les talus. Je tiens à bien préciser que ce résumé n'est pas définitif et a simplement pour objet de constituer une base de discussion.

La forme extérieure AB est l' " arc " décrit par Sokolovsky [1957]. L'existence de ce profil dépend des caractéristiques c et φ du matériau, où la cohésion C mesure la résistance à la traction du matériau considéré.

La ligne AB correspond à l'angle du talus naturel " sec " pour un terrain argileux. Si des sols argileux peuvent être assimilés à des matériaux granuleux constitués par des ensembles [Trollope, 1960], lorsque la pente est moindre que AC ces sols peuvent adopter une disposition d'équilibre dans laquelle il n'y a pas de force de traction entre les ensembles. Lorsque la pente est supérieure à AC il faut qu'il y ait des tensions entre les éléments pour qu'il puisse y avoir équilibre.

Je ne sais si la ligne AC a une signification physique quelconque, de même qu'il est improbable qu'il existe une argile naturelle " sèche ".

L'angle de talus naturel pour un sol argileux est celui représenté par AD lorsque la nappe phréatique coïncide avec la surface et que les écoulements ont lieu parallèlement à celle-ci [Skempton, 1957].

Les sols argileux ne peuvent résister à des efforts de traction de quelque importance que s'il s'agit de matériaux saturés où l'eau interstitielle est en tension. Il n'est donc pas probable que le sol conservera cette propriété de résister à la traction du fait, soit d'une dessication ou d'une imbibition possible.

Il paraît donc que le mécanisme de la rupture d'équilibre dans les talus d'inclinaison supérieure à AD commence par des ruptures de traction suivies par une diminution de la résistance progressive à mesure que l'eau de la nappe pénètre dans les fissures de traction jusqu'à ce qu'une rupture se produise le long d'une surface comme EFA. S'il existe une alimentation en eau continue ou intermittente le processus se répètera jusqu'à ce que le talus ait atteint sa pente définitive AD.

Les talus ayant une pente inférieure à AD paraissent peu susceptibles de se rompre sauf dans le cas de déformations importantes de la fondation.

Comme suite à ce qui a été dit ci-dessus, je propose les sujets suivants pour la discussion :

1) Recherche des éléments qui contribuent à la diminution de la résistance du sol en fonction du temps dans les talus ;

2) Modification des contraintes internes dans les talus et les remblais en fonction du temps ;

3) Existe-t-il des zones soumises à des efforts de traction dans les talus constitués par de l'argile et, dans ce cas, les sols argileux peuvent-ils résister à ces efforts ?

References

[1] BISHOP, A. W. (1955). The Use of the Slip Circle in the Stability Analysis of Slopes, *Geotechnique 5.*

[2] CARILLO, N. (1952). Differential Equation of a Sliding Surface in an Ideal Saturated Plastic Soil, *J. Math. Phys.*, 21.

[3] COLLIN, A. (1956). Landslides in Clay (Eng. trans by William R. Schriever). University of Toronto Press.

[4] MAY, D. R. (1936). Proposed Methods of Calculating the Stability of Earth Dams, *Proc. 2nd Cong. Large Dams 4.*

[5] PARRY, R. H. G. (1958). Correspondence " On the yielding of soils ", by Roscoe et al., *Geotechnique 8*, 4.

[6] PRAGER, W. and HODGE, P. G. (1951). Theory of Perfectly Plastic Solids. Wiley.

[7] ROSCOE, K. H., SCHOFIELD, A. N. and WROTH, C. P. (1958). On the yielding of soils, *Geotechnique 8*, 1.

[8] SOKOLOVSKY, V. V. (1957). Some Problems of Soil Pressure, *Proc. 4th Int. Conf. S.M. & F.E*

[9] SKEMPTON, A. W. (1957). The Stability of Natural Slopes in Clay, *Proc. 4th Int. Conf. S.M. & F.E.*

[10] TAYLOR, D. W. (1937). Stability of Earth Slopes, J. Boston Soc. C.E.

[11] TERZAGHI, K. (1936). Critical Height and Factor of Safety of Slopes against Sliding, *Proc. Ist. Int. Conf. S.M.& F.E.*

[12] TROLLOPE, D. H. (1960 a). The Mechanics of Rock Slopes, *Paper to Annual Convention A.I.M.E.*, New York.

[13] — (1960 b). The Fabric of Clays in relation to Shear Strength, *Proc. 3rd Aust.-N.Z. Conf. S.M.& F.E.*

[14] — and MORGAN, J. R. (1959). Stress Systems in Simple Slopes of Granular Materials, *C.E. Trans. Inst. of Engrs, Aust. C.E.I.*, 1.

[15] WALKER, F. C. (1957). General Report Div. 6, *Proc. Proc. 4th Int. Conf. S.M. & F.E.*

[16] WILKINS, J. K. (1956). Flow of Water through Rockfill and its Application to the Design of Dams, *Proc. 2nd Aust.-N.Z. conf. S.M. & F.E.*

[17] WILSON, S. D. (1959). The Application of Soil Mechanics to the Stability of Open Pit Mines, *Symposium on Rock Mechanics. Colorado School of Mines* (April).