Earth and Rock Dams, Slopes, and Open Excavations

Barrages en terre et en roche, talus et tranchées ouvertes

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Chairman: R. J. MARSAL (Mexico)

I open Session Eight, Division 6, of this Conference. In the first part of this Session we have a special lecture on modern Canadian dams, which will be delivered by Mr. J. K. Sexton. Mr. Sexton is Vice-President and Director of Civil Engineering of Montreal Engineering Company Limited. He has been working in the civil and hydraulic aspects of hydro- and thermo-electric power since 1928, when he got his B.SC. degree from the University of Saskatchewan. Outside Canada, Mr. Sexton has done consulting work in the United States, China, India, the Caribbean area, and Central and South America. He holds the position of Study Director of the Power Study of South Central Brazil with headquarters in Rio de Janeiro.

Mr. Sexton has become very well known among engineers since he was Vice-President of the International Commission on Large Dams, Chairman of the Canadian National Committee on Large Dams, and also Chairman of the Hydraulic Power Section of the Canadian Electrical Association. It is my privilege to give the floor to Mr. Sexton.

(*Mr. Sexton's lecture appears on pp. 168–84.*)

CHAIRMAN MARSAL

Thank you very much, Mr. Sexton, for your most graphic presentation of the work being done by Canadian colleagues in this field. As an engineer devoted to the design and construction of dams in Mexico, I admire the effort and the ingenuity displayed in the works you just described. I am sure that all present agree that we owe a vote of thanks to Mr. Sexton. There will be no recess at this time, so that we may proceed directly with the panel discussion.

The subject of this Session is one of great interest to engineers engaged in the building of water storages, roads, and industrial plants. Recent progress in the technology of large rockfill masses, both in equipment to haul and place the material as well as in the determination of its mechanical properties, has vitalized this field of soil mechanics. On the other hand, the demand for better roads and the ever increasing size of industrial facilities, has resulted in a variety of problems related to the stability of high slopes or deep excavations. The number and quality of contributions presented on the first matter is indicative of the activity developed in earth and rock dam construction around the world. Contrary to expectations, papers on slopes and open excavations were few, although these topics are by no means less important.

In the last decade we have observed an appreciable increase in the height of earth and rock dams, particularly those being designed for power generation. The advent of high dams built with a rather thin clay core and shoulders of pervious materials has raised questions which are not really new, but are nevertheless important because of their implications for the behaviour of the structures.

One-dimensional compression tests are run with rockfill materials to determine their compressibility. Time-settlement curves disclose a delayed action in the solid skeleton. Although of different nature from the consolidation process in a clay, this phenomenon can be explained by means of a probability analysis of the random path of particles during compression. The effect of saturating the specimen in the laboratory confirms what is to be observed at the upstream shoulder of a dam upon first filling of the reservoir.

Triaxial tests performed with granular materials reveal that the principal stress ratio at failure diminishes for increasing confining pressures, corresponding strains being surprisingly high. In some well compacted but poorly graded rockfill samples, deviator stresses kept increasing even for axial strains of 20 per cent. Volumetric changes measured in these tests show no dilatant effect for high stress levels. Test results seem to indicate that the solid skeleton of a granular mass during shear is separated into two parts: one is the particle structure that transmits stresses and the other the group of grains that remain idle within the voids. This may lead to new theoretical developments.

Breakage of grains becomes an important feature in both the compressibility and the shear strength of granular materials. To understand this phenomenon, a theory has to be developed based on intergranular forces, particle concentration, and number of contacts and crushing strength of grains. Papers recently published in several countries show this trend.

Parallel to the laboratory and speculative work, measurements in dams and slopes are being made at an increasing rate. Instruments of various types are installed both in the core and rockfill shoulders. Not long ago, the usual devices were piezometers and cross-arms. Settlements of monuments were also reported. To-day, several rockfill dams have strain meters, inclinometers, lying tubes, pressure cells, accelerometers, and seismoscopes in them as well as piezometers and cross-arms. There is no question that this field information properly processed and correlated with laboratory data will bring about a significant improvement in the methods for designing rockfill dams.

Finally, I would like to encourage discussions by members of the panel and the audience. To save time, I recommend the use of the telegram type of language suggested by Professor Casagrande.

The General Reporter of this Session is Professor Dinesh Mohan, Director of the Central Research Institute, Roorkee, India.

General Reporter: D. MOHAN (India)

Since writing my General Report I have been informed that the height of the Oroville Dam in the U.S.A. is now going to be 770 ft due to an extra 22.5-ft excavation in the foundation. Another report says that the height of the Nurek Dam in the U.S.S.R. is going to be 984 ft now and that the Soviet engineers are testing a 28-ft high scale model of the dam. The U.S.S.R. claims to be building an even higher dam, an arch dam on the Inguri River. Since we have, in this Conference, distinguished engineers from U.S.A. and the U.S.S.R. I do hope we will have some further interesting information coming forward on these two dams during the course of our discussions today.

Discussion on the subject of earth and rockfill dams, slopes, and excavations may be broadly divided into two categories: (1) analytical aspects dealing with the stability of slopes; and (2) practical and other construction aspects. In this morning session I would suggest that we discuss the first aspect, viz., various new hypotheses and concepts put forward on the stability analysis of earth slopes.

An interesting development on this subject that has taken place only during the last decade is the increasing use of digital computers. With this powerful and speedy device at our disposal there is no doubt that we can have a quick evaluation of the various hypotheses put forward by various workers on the analytical aspect of the stability of slopes.

Let us next look at the sliding surface. The shape of the sliding surface has again been discussed in the papers. A number of views have been advanced, one of them being the concept of two flat surfaces, one being the internal surface of the side fill and the other a surface inclined at a critical angle to the bed plane. I am sure there will be some useful comments on this and other aspects of the problem during the current discussions.

Another important subject is the safety factor. In selecting the proper factor of safety the concept of the theory of probability put forward by one of the authors is of interest and there is little doubt that if a more rational approach could be introduced in determining the factor of safety it would lead to greater economy and safety in many cases.

The rheological properties of clays in relation to their strength and compressibility characteristics are also important and as was suggested in the first technical session on general soil properties it is time that more serious attention was given to the basic properties of the soil. This would have a marked influence on the design of earth structures.

On the subject of negative pore-water pressure I would like to correct an impression that might have been conveyed by my report on paper *6/16* by *Little* on the "Value of Field Studies in Earth Dam Designs." On further study of the paper, I find that the author does appear to have made use of the negative pore-pressure concept in the design of the Mangla Dam. It would therefore be useful if the experiences and views of other workers on this important aspect of negative pore pressures were brought forward in the course of discussions this morning.

I have mentioned only a few important topics which might provide subjects for a fruitful discussion. Although the Organizing Committee of this Conference has allowed me ample time for the introduction and summarizing of my report I feel that, having already published my report, which most of you have probably read, I should take the least possible time now and provide more time for discussions amongst the panel members and others from the floor. However, since within the short time at our disposal it may not be possible to discuss all the analytical aspects of design of earth dams I would suggest the following topics for discussion at this morning session: (1) the introduction of visco-elastic, visco-plastic, and elasto-plastic principles in stability analysis; (2) the mechanism of development and shape of slips in embankments and cuttings; (3) the variation of stress with time and the effect on stability of normal stress distribution along the slip surface; (4) the importance of the role of water tension and negative pore pressure in slope stability analysis; (5) the creep strength of soils and their influence on stability of slopes with particular reference to their long-term behaviour and its effect on engineering construction.

(*Professor Mohan's General Report appears on pp. 270–3.*)

CHAIRMAN MARSAL

Thank you very much Professor Mohan for your interesting report on papers presented to this part of Division 6. No doubt, the discussion will prove a stimulating one. The panel members are Mr. Queiroz from Brazil, Professor Nonveiller from Yugoslavia, Mr. Folque from Portugal, and Dr. Golder from Canada. The Panel Members will now start to discuss the topics proposed by the General Reporter. We will try to hold a lively discussion through brief and precise comments. Please indicate to me when any of you wish to comment. I will ask Mr. Queiroz to start the discussion on cut slopes.

Panelist: L. DE A. QUEIROZ (Brazil)

This topic, the mechanism of development of slips in embankment and slopes, presented by the General Reporter for discussion leads to many aspects of the problems connected with slope stability and with stability analyses of embankments, and natural and cut slopes. In soils engineering we are striving to achieve a design which can lead to stable and economically satisfactory structures. However, the development of the slips in embankment slopes and natural or cut slopes is not well enough known to permit the soils engineer to feel confident about the computed values for the apparent safety of the designed slopes.

This confidence can only be achieved with field experience, with complete observations obtained during construction and after completion of the structure. Even so, the designer may have to face unexpected field conditions which can lead to instability and the related problems which must be dealt with immediately.

The search for the proper knowledge of the controlling factors of the over-all stability of a structure is most important. Trying to uncover the mechanism of development of slips may seem practically irrelevant since for the typical slides, the process is generally rapid and no corrective measures can be taken to prevent the movement of the mass. However, if the mechanism can be visualized thus providing additional information about the behaviour of the materials involved, it will eventually complete the mosaic of the full understanding of the problem.

It has to be also realized that mass movement due to shear failure of soil elements within the mass cannot be fully understood since our knowledge of the shear strength of the soil and other materials involved under different load and strain conditions is also a controversial matter. Progressive failure and the redistribution of stresses along potential sliding surface are problematic aspects and, at present, impossible to ascertain. Consequently, research and works of stability analysis must leave the theoretical approach and, if possible, measure in the field those factors and aspects which govern stability.

Two cases will be presented to this Conference which will show the need of complete correlation between observations in the field and existing theory to achieve proper and safe design. It can also be added that the complexity of the problems of slope stability, in most of the cases, can be such that no immediate progress can be foreseen as far as any theoretical treatment is concerned: the trial and error method still is the only road for us.

The first case deals with stability of cut slopes and to illustrate I have chosen one very recent case from Brazil. It can be considered valid for those areas where weathering has produced a deep mantle of residual soil and weathered rock. In a particular location (Fig. 1) where an earth dam and power plant are being built the natural soil slopes are quite steep, of the order of 1 on 1.5, with sound rock at great depth. In some spots near to the site old scars and talus deposition at the toe of the slopes can be detected, indicating local instability.

FIG. 1. General view of the site showing steep natural soil slopes.

For the excavation of spillway and power house, a cut with total height of 250 ft was made. The design was based on the assumption of keeping about the same the natural slope, that is, 1 on 1 slope with 3 to 5 m berms every 10 m in height. The cut intercepted clayey soil in the upper 10 m and residual soil from there to the bottom. This residual soil presented such denseness that it was not possible to drive the sampling spoon of the Standard Penetration test and did not show the texture of the parent rock of the critical areas of the slide. Moreover, the usual technique of rock drilling did not recover any rock core samples. There is no doubt that for such a material and designed slopes, the factor of safety is above one, according to routine stability analyses and shear parameters adopted for the materials.

With the cut almost completed some slides developed in part of the area (Figs. 2 and 3). These slides started with the development of movement and cracks along very neatly defined shear planes (Fig. 4). These planes were found to be generally coincident with the jointing system of the parent rock. No sounding could have disclosed the occurrence of such planes in the soil mass. Furthermore the occurrence of a very thin layer of more clayey and blackish soil along those planes was found. The mechanism of these slides can be established as starting along those planes possibly triggered by seismic waves from nearby blasting of the rock

FIG. 2. View of site showing two slides.

FIG. 3. Typical slide. Note bodily movement of the soil mass with berms still maintained at 10-m intervals.

FIG. 4. Shear surface along pre-existing shear plane.

excavation. Failure along those planes progressed at a slow rate and the strain measured at the surface reached several centimeters without mass sliding. During the process, the weight of the potential sliding mass was gradually transferred to the soil along the potential sliding surface which was able to carry the additional load until the shear stress reached the shear resistance. The sliding surface, although laterally following those planes, had the common aspects of the circular slide surfaces.

Thus, minor geological details dictated the behaviour and the mechanism of the slide. They showed that in spite of existing conditions for stability for the majority of the cut slope, the slides developed where they were present. Moreover the sliding process mechanism cannot be cited as starting at the top or bottom; in this particular case it started at neither place.

It can be inferred from this and from other reported cases, that slope design can only be achieved when all the conditions revealed during construction are taken into consideration, and so the risk of eventual slides cannot be entirely eliminated.

Another case to be mentioned deals with the design of embankment slopes of compacted materials. In Brazil, extensive and predominant use is still made of residual soils for compacted embankment construction and earth dams. These embankments are usually compacted at about optimum moisture content, with an average 98 per cent of Standard Proctor maximum density, as has been reported also to this Conference by *Vargas, Silva, and Tubio (6/33)*. Safety factors computed on the light of existing theories were found to be much higher in the field because of the low value of the pore pressure developed in the compacted material. In some cases negative pore pressures were still measured even when relatively large fill loads had been applied.

One reason for this is the large difference between the optimum water content in the field and the plastic limit. In one recent case this difference was found to be 10 to 12 per cent of the dry weight. Therefore, all the stability analyses based upon larger pore pressure as a result of laboratory initial measurements became of limited value. For such and similar cases whatever minor differences in stability computations may exist or may be claimed, the controlling factor is and may be beyond assumed hypotheses and may completely overshadow those refinements.

It should be added that some interesting papers reporting observational data and tentative correlation with theories, have been presented to this Conference with particular attention to the measurement of slope deformation before actual slides occurred. Among these papers are *Aisenstein (6/1)* from Israel, *Mencl, et al. (6/18)*; from Czechoslovakia, *Saito (6/24)* from Japan, and *Ter-Stepanian (6/32)* from the U.S.S.R. However at this same Conference more papers on the theoretical aspects of stability analysis without any correlation with field data will be encountered. Special care has to be taken with this approach in order to protect the designer, contractor, and clients against dangerous pitfalls.

CHAIRMAN MARSAL

Thank you very much Mr. Queiroz. Professor Nonveiller, have you any comments on this topic?

Panelist: E. NONVEILLER (Yugoslavia)

Prior to failure of a slope, plastic deformation will occur: the upper zone on the steeper part of the slip surface will tend to dilate, and the active state of plastic equilibrium will be approached at failure; the zone above the lower part of the slip surface will compress and the passive state will be approached at failure. The consequence of this change of the state of stress is a slight difference of the actual shear strength in the two zones, depending on the type of material. In a slope of cohesionless pervious material tan ϕ changes with the porosity (e); tan ϕ will decrease slightly in the active zone and increase in the passive zone. Failure will occur progressively starting in the upper section of the slope, where the strength has decreased as a consequence of deformation. The opposite course will develop in a slope of cohesive material of low permeability where strains will cause pore pressure, which will be negative in the upper dilating zone and positive in the compressing zone. Thus the actual shear strength will increase slightly in the upper part and decrease in the lower part of the slope. Again the failure will be progressive starting in the lower part of the slope where pore pressure decreases the actual strength.

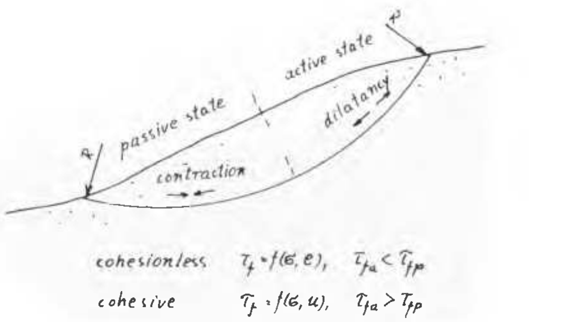

FIG. 5. Sketch showing the deformations of the sliding segment, shear strength in active and passive zones, respectively.

The first type of failure can be illustrated by the catastrophic slide in the Vajont Reservoir. The shear strength on the upper part of the slope was much lower because the bedding planes which dip parallel to the slope are filled with clay. The shear strength in the lower part was much higher because interlocked rock fragments, rock within folded limestone beds, resisted movement there. Consequently failure started in the upper part of the slope and the scar all along the upper limit of the unstable zone was clearly visible two years before the final collapse occurred. The second type of failure often occurs on slopes of normally saturated consolidated clays with subsequent retrogressive slides in the upper zone after the initial slide.

Thus a general rule about the mechanism of development of slips does not exist. The mechanism depends on the influence of the deformation of the slope prior to failure on the strength of the soil along the slip surface.

CHAIRMAN MARSAL

Thank you very much Professor Nonveiller.

Panelist: H. Q. GOLDER (Canada)

I would like to ask a question of Mr. Queiroz. In view of the variable nature of residual soil, to what extent were borings and normal laboratory testing carried out in the example described of a failure in residual soil?

PANELIST QUEIROZ

As I mentioned in my contribution, it is very difficult to establish a testing programme and to ascertain the shear strength of such materials. The criterion has been to use the existing experience on similar cuts along the area, and, when possible, to design the slope in such a way that the safety factor is above unity. Since the material varies so much with depth, one cannot use theoretical or laboratory testing to determine the factor of safety of these materials.

I would say, therefore, that this tends more to the empirical side of design, using field observations for checking. On the face of the slope under discussion two major slides, comprising about 30 per cent of the area, occurred.

PANELIST GOLDER

Do you allow then for some failures?

PANELIST QUEIROZ

These failures were not expected; in fact, the materials in the slide area and the intact area are generally the same. So actually, you cannot pinpoint a laboratory testing programme which would have provided a foolproof design.

Panelist: J. B. FOLQUE (Portugal)

I would like to have Mr. Queiroz's opinion on a question that is closely related to the one put forward by Dr. Golder. Do you not think it is possible to perform a stability analysis of these cut slopes using a somewhat conventional method, assuming that the potential surfaces of sliding are planes in determining the shear strength of the anisotropic material that constitutes the slope? Of course, following the slide it would be necessary to perform field tests rather than laboratory tests.

PANELIST QUEIROZ

That would be a very difficult matter, since as you may have seen, one of these slides did not intercept all the cut slope. (It was located from the top to approximately mid-height of the slope.) The other one did intercept deeper areas. Therefore, even if you can assume a slip surface, you probably could not determine the shear strength which should be used for a theoretical approach.

PANELIST GOLDER

I agree with Professor Nonveiller that it is important to realize that there are many different kinds of slips. Some may start at the bottom and some may start at the top. However, we know they will finish at the bottom and it is the engineer's job to stop them starting. We normally do this by assuming a mechanism of failure which we know is not entirely true, and using a factor of safety based on experience, from which we know that with a given value the chances of failure are small. If further study is made of the real mechanism of the slip, the end result should be that we will be able to use a lower factor of safety in design. This is the criterion by which the success of, and value of, any further study of the mechanism should be judged.

PANELIST NONVEILLER

Fig. 6 illustrates the behaviour of the 60-m-high Peruča Dam on the River Cetina. It is a rockfill dam with a clay core. The computed settlement of the core with the arching effect is shown by the dashed line. The solid lines show the settlement during the seven years since completion of the dam. Three facts are evident from this diagram. (1) The settlement of the rockfill shell is much lower than that of the core, causing positive arching and a favourable stress distribution. (2) the rockfill shell is still settling at a slow rate. (3) the settlement of the core is less than was computed and it tends to lag behind the computed time–settlement curve. This can be attributed to the higher mobilization of the shear strength of the interface between core and shell than assumed in the computation. The horizontal displacement of the dam crest has been negligible, which may indicate that the actual safety factor of the downstream slope is higher than computed ($F_s = 1.5$). Further theoretical and experimental study will be necessary in this area. Comprehensive measurement of stress and deformation on existing dams will contribute a great deal to an increase in the accuracy of our present design practices.

CHAIRMAN MARSAL

Thank you, Professor Nonveiller. Dr. Golder, would you like to proceed with the topics for discussion?

PANELIST GOLDER

I have been concerned with two separate problems recently in which the time factor has been important and unusual. Both jobs were highway embankments.

In the first, 12 ft of muskeg and very soft clay were excavated and rock fill was then tipped on the remaining clay, causing shear failure and displacement of some of the clay. This is an accepted method of building an embankment over soft clay soils. In general, once the original failure has taken place, the embankment is expected to be stable since re-consolidation of the disturbed clay takes place with a corresponding increase in strength. Three months after completion this embankment failed—a typical cylindrical slide. This failure is difficult to explain.

It was found from the records of the job that after the initial construction, the height of the bank was raised some 4 ft in order to use up excess rock and to improve the grade. This is a somewhat unusual procedure, but if this extra weight was going to cause failure one would have expected failure to occur at the time the rock was placed. If failure did not occur then, it would have been concluded that the bank was safe, since consolidation of the clay under the weight of the rock would gradually increase the factor of safety. The only explanation I could produce for the failure was that

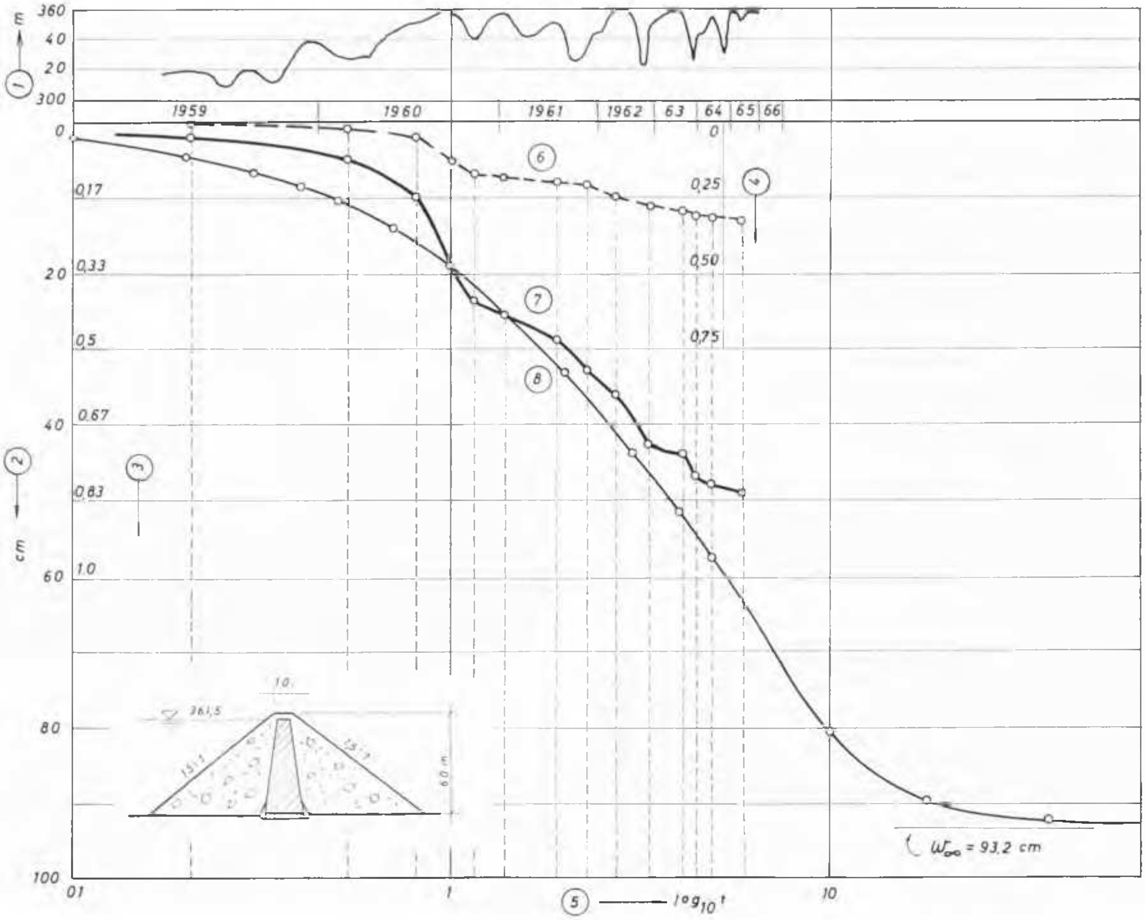

FIG. 6. Settlement of core and shell of Peruča Dam: 1, reservoir water levels; 2, settlement in core; 3, settlement of core, per cent of height; 4, settlement of rock fill, per cent of height; 5, time, one unit = 16 months; 6, settlement of rock fill; 7, settlement of core; 8, computed settlement of core.

after placing the extra 4 ft of rock, the factor of safety was just over unity in terms of the shear strength measured with normal rates of loading, that is, quickly, but that the shear strength under long-term loading was lower. Thus we have two long-term effects working against each other, consolidation slowly increasing the strength and long-term loading at a stress near the failure point, in effect lowering the strength. The reduction in strength under long-term loading won.

In the second case an embankment was being built over a soft varved clay and a failure occurred during construction. A normal total stress analysis gave a factor of safety of unity. The interesting thing about this failure was that a section of the bank close to the failure had been instrumented to measure settlement and pore water pressure. When the bank was placed the pore water pressure rose to 85 per cent of the weight of the fill. Over a period of two years the bank settled two ft, but the water pressures dropped only 15 per cent. Further, the rate of settlement was not dropping as one would expect—there seemed to be some secondary effect causing settlement not due to consolidation. This was unusual because the clay was varved and contained definite silt layers.

Mr. K. Y. Lo, who carried out a careful investigation of the failure, put forward the following theory to explain the observed facts. At the instrumented section the factor of safety was just over unity (based on vane and laboratory tests at this chainage). The shear stresses in the clay were close to the shear strength. Large shear strains occurred and a slow plastic deformation took place. This caused an increase in pore pressures and settlement. Thus, although consolidation was occurring, the pore pressures remained high and the rate of settlement did not decrease as expected. It is hoped to be able to check this theory by simple instrumentation of an adjacent section where the strength of the clay is higher and the same phenomenon should therefore not have occurred. It is also planned to unload partly the instrumented section and to observe the change in pore pressure and settlement.

Thus, where we are working with factors of safety just over unity there are time effects which must be taken into account. With more normal factors of safety these effects are not of importance.

CHAIRMAN MARSAL

Have you any comment, Mr. Folque?

PANELIST FOLQUE

I think it is difficult to agree with the statement that the shear strength decreases when the stress is high, approaching failure. I think that it is necessary to deal with the long-term strength, not with the peak value.

PANELIST GOLDER

My point is that if the shear strength is determined in the normal manner, that is using vane tests and compression tests, what is known as the normal shear strength of the material is obtained. Now the question is why did the bank not fail when the extra four ft of rock were added, rather than later, when one normally would have expected an increase in shear strength due to consolidation?

PANELIST QUEIROZ

One point, which could be mentioned in connection with this embankment failure, is the foundation material. If certain construction methods were used, remoulding of the clay in the foundation could lead to a secondary type of consolidation with a very slow rate of strain, resulting in very small creep type movements from the beginning leading to failure.

PANELIST GOLDER

Do you agree then with my suggestion that if you have a shear stress which is very close to what I will call the shear strength measured at normal rates and if you then apply that shear stress for a long period, you may get failure, because if you do your shear-strength measurements much more slowly, not allowing consolidation, you may get a lower strength?

PANELIST QUEIROZ

Yes, I agree.

PANELIST FOLQUE

It is also possible to postulate that the shear strength of clay is nil in the very long term view.

PANELIST GOLDER

I cannot agree with Mr. Folque in that regard as it implies a behaviour similar to that of a viscous material. I believe that for engineering purposes, there are shear stresses that one can apply to a clay soil and design safe structures with those stresses.

PANELIST FOLQUE

I think that all of us can agree that ultimate strength cannot be accurately determined with the usual methods.

CHAIRMAN MARSAL

I would like to hear Mr. Folque discuss visco-elastic, visco-plastic, and elasto-plastic principles.

PANELIST FOLQUE

I should like to comment on questions (*a*), (*c*), (*d*), and (*f*) of the General Report. These questions are closely interrelated in one aspect of the problems they raise: the rheological behaviour of compacted soils. As a matter of fact, the introduction of rheological principles into stability analysis as stated in point (*a*), the variation of stress with time and creep strength referred to in points (*c*) and (*d*), and the influence of age on the strength mentioned in point (*f*) all depend upon a deeper understanding of the rheological properties of soils. The introduction of visco-elastic, visco-plastic, or elasto-plastic principles in stability analysis is a very desirable aim but it seems very difficult to perform the accompanying computations by analytical methods.

Owing to the work of Taylor, Goldstein, Tan and other researchers we are beginning to have well-grounded ideas about the rheological equation of the state of compacted soils in the moisture content range between the "as compacted" and the "quasi-saturated" situations. But we also know that the rheological behaviour of soil is very complex, its structure having elastic, plastic, and viscous elements, and that the rheological equations of state are non-linear. Therefore it seems important to stress the possibility of performing stability studies by means of scale models. The first task is to establish the rheological equation of state of the soil and the second to search for a convenient material with which to construct the model in order to obtain the required similitude relations. Even studies of the variation of stress with time and creep strength of soils can be forwarded by the use of scale models. The effect of age on strength and compressibility, as well as the fundamental rheological study by means of laboratory tests, must be studied on samples taken from both the downstream and upstream zones of dams already constructed.

CHAIRMAN MARSAL

Professor Nonveiller, do you have any comment on this topic?

PANELIST NONVEILLER

When considering problems of immediate interest in the design of new high dams not only the rheological properties of clay but also those of rockfill are very important. We need both a theoretical rheological model for rockfill and a practical means to establish the relevant parameters. In this field I think much has yet to be done.

PANELIST GOLDER

Everyone assumes that measuring the shear strength of clay is difficult, and that measuring the shear strength of rockfill is simple. This is not so. It is only recently that investigations such as Dr. Marsal's have shown that the angle of internal friction of coarse granular materials under very high pressures is much less than the normally accepted 40° to 45°.

I would like to ask Dr. Marsal if he considers that he can take part in the discussion, what values of ϕ he uses in dam design when he gets these low values in the triaxial test.

CHAIRMAN MARSAL

Although not a member of the panel, I will be glad to take up the point raised by Dr. Golder. When we designed El Infiernillo Dam, a value of 45° was assumed as an average of the angle of internal friction for the rockfill shoulders. In fact, this material was a sound, quarry blasted, silicified conglomerate. However, it was decided to build a triaxial apparatus big enough to run tests on specimens 113 cm in diam, the maximum particle size being 20 cm.

Tests showed that the principal stress ratio varies from 6 to 4 for confining pressures of 1 to 25 kg/sq.cm. Therefore, the average angle of internal friction was 41°. We had to enlarge a berm in the downstream slope in order to meet the usual requirements for stability.

I might mention that we have already tested rockfill samples that, for the same range of confining pressures, have principal stress ratios at failure varying from 5 to 3, that is, angles of friction between 42° and 30°. These particular results were also obtained with sound rockfill samples, but they were not as well graded as the El Infiernillo material.

PANELIST GOLDER

The reduction in angle of friction with increase in pressure is not new. It has been demonstrated earlier by Terzaghi (1930) and Golder (1941). In discussion of the latter paper, I suggested that a probable limit for the value to which the angle of friction of a dense granular material dropped with increasing pressure was the angle of internal friction for the same material in a loose condition, since this value seemed to be independent of the pressure applied.

Another important effect is the size of the grains in granular material. A value for the angle of internal friction of 40° or 45° is often assumed for rock. This is not by any means true. For the same mineral, for the same grain shape, for the same shape of grading curve, and the same density, I can see no logical reason why the angle of friction should increase as the over-all size of the particles increases. In dense granular material the angle of friction drops as pressure increases, but this is not so with a loose soil. Therefore, in high dams where the pressures are high, if compaction does not result in a higher value of angle of internal friction, why do we compact the material? This statement has created some amusement but I did not intend to be funny. I feel that although compaction is sometimes necessary, it is not always essential and we need to revise our thinking on this matter.

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PANELIST NONVEILLER

I agree with Dr. Golder that in many cases we do not need to compact rockfill but this view cannot be adopted as a general rule. One might object to his view that the shear strength obviously increases with compaction. The peak strength of compacted cohesionless material is mobilized at low strain. Only a progressive failure could cause increased deformation and reduced final strength, but such a failure is not likely in a composite dam section consisting predominantly of rock. On the other hand the deformability of rock fill is reduced by compaction, an important development in dams with a central core.

CHAIRMAN MARSAL

We will continue after a short recess.

(There followed a brief intermission.)

CHAIRMAN MARSAL

The Session is open to discussions from the floor.

C. VEDER (Austria)

I would like to comment on preceding reports (among others, that by *Aitchison and Wood (6/2)* on the necessity of studying the electro-physical and electro-chemical behaviour of certain soils.

THE STABILIZATION OF LANDSLIDES

I think that the phenomena of certain landslides as described, for example, by Mr. Queiroz can be explained by my theory. As already mentioned in preceding papers of mine, the I.C.O.S. firm, of which I am the consulting engineer, succeeded in stabilizing landslides of a certain type simply by introducing metallic conductors into the ground. These put the electrical potential existing between the two layers of clay of different nature in short circuit. No outside electric current is necessary. The principle of this procedure is shown in Fig. 7.

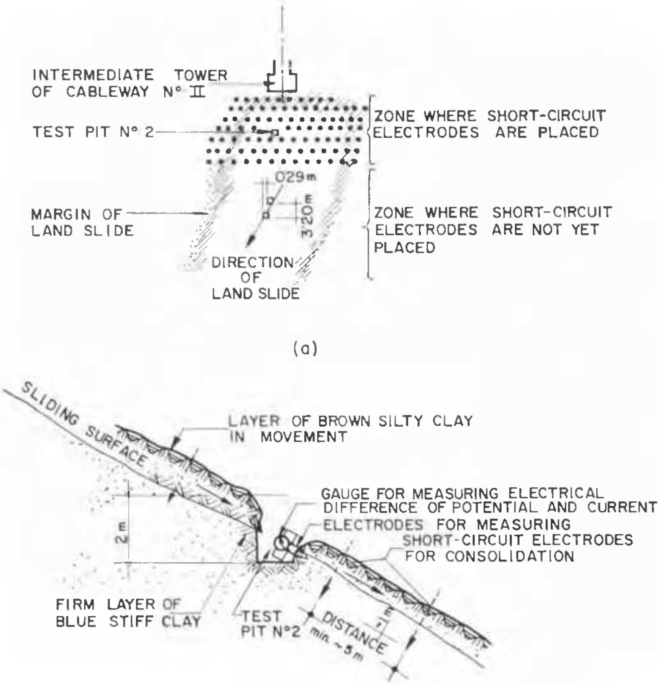

FIG. 7. Landslide stabilization. (*a*) plan; (*b*) section through test pit.

FIG. 8. View of the Bosco landslide before treatment.

I have observed that in certain regions (Apennines, Sicily, the eastern Alps) a layer of brown silty clay lies upon a lower layer of blue stiff clay. Between these two layers a more or less thick, discontinuous film of water can be observed (independently of whether the ground is horizontal or inclined). Even on slopes of only 5–10° the upper layer starts slipping upon the lower layer, probably due to the presence of the water film. Between the two layers there exists a difference of electrical potential and the electric current I measured to be about 20mV and 3μA. The presence of water, building up a pore pressure between the two layers, I attribute to an electro-osmotic phenomenon. I concluded that, if the electric current were eliminated, the water ought to disappear, and the landslide dry out and become stable.

In order to eliminate the electric current, metallic conductors (steel or aluminium bars) were pushed through the upper layer which was moving down into the lower fixed layer.

FIG. 9. Same view of the Bosco landslide after treatment with short-circuit electrodes.

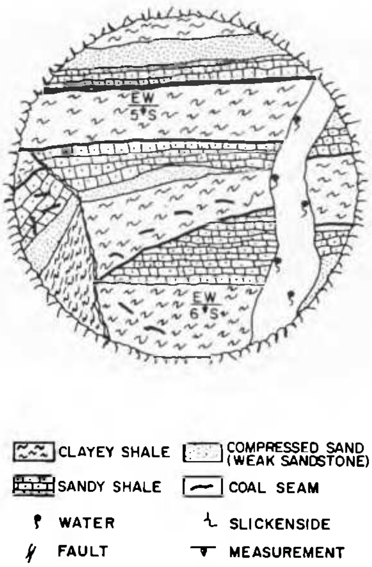

FIG. 10. Geological section of tunnel to be excavated through soils having a great swelling tendency.

After approximately one month the landslide area dried out and the movement stopped. I have succeeded in stabilizing five landslides of this type; one of them (near Bosco Campofranco in Sicily) has been dry and firm for more than four years and undoubtedly will remain so. The maximum distance between the electrodes at Bosco was 5m × 5m and I have the impression that even wider spacing might be possible. Fig. 8 and Fig. 9 show the Bosco landslide before and after the introduction of the short-circuit electrodes. As was pointed out before, there was no electric current introduced from outside.

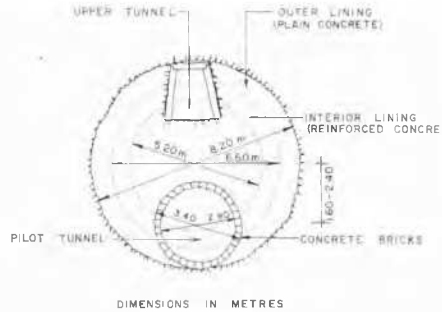

FIG. 11. Proposed method of excavation utilizing the old Austrian method of pilot tunnels lined with concrete bricks.

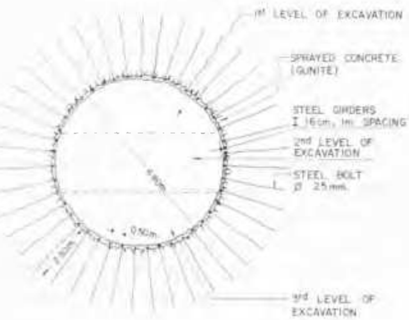

FIG. 12. Section through 6.8-m-diam. experimental tunnel.

STABILIZATION OF TUNNEL EXCAVATIONS IN SWELLING SOILS

In 1958 we applied for and obtained a German patent for the consolidation of swelling soils in tunnels by introducing a short circuit between the inner zone and the outer zone by means of metallic conductors. According to my theory, there is in certain swelling clays a difference of electric potential between layers which differ in composition. In 1963–64 such a work was successfully carried out. A tunnel was to be excavated through a soil with great swelling properties. It consisted of clayey grey schist, fine grey sandstone, and seams of coal or greyish fine sand. Water appeared as humidity, drops, or even small springs. The moisture in the tunnel atmosphere was far below the medium limit (Fig. 10). The pilot tunnels (Fig. 11) were driven with great difficulty, and the enlargement of the tunnel to its final shape provoked repeated swelling and rupture of the lining. The rate of advance of the tunnel was as low as 10 m/month. On our advice the current between the zones next to and further from the excavation was measured in a borehole and it turned out to be in the order of magnitude of about $100\mu A/2.5m$, using non-polarized electrodes. There may be some error of quantity of little practical importance, but not of quality. I proposed to introduce from the tunnel excavation into the surrounding soil, a number of steel bars 25 mm in diam and 2.5 m in length, at 0.5-m intervals (Fig. 12).

After the steel bars had been introduced, the swelling phenomenon disappeared and it was observed that the electric current in the borehole in the neighbourhood of the bar also disappeared. It can be concluded that, in a manner similar to that described for stabilizing landslides, steel bars, by short-circuiting the two diverse layers, eliminated the electric current between them and consequently the water transport from the remote mountain mass toward the tunnel

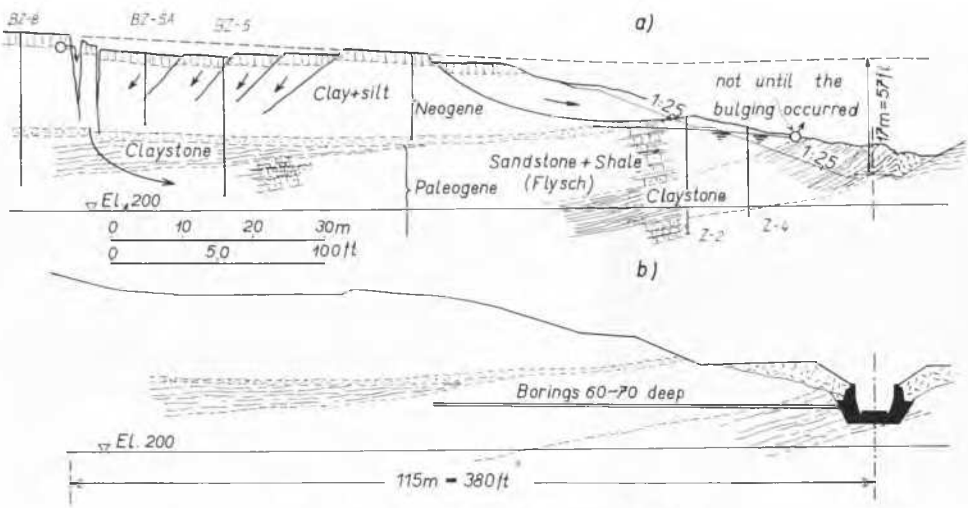

FIG. 13. Section through movement zone.

excavation. The tunnel was afterwards lined with a layer of concrete gunite into which H-beams, 16 cm long, were embedded at 1-m intervals. The lining was then completed by a ring of concrete 0.80 m thick (instead of the 1.50-m ring originally planned). In this way work was speeded up by three times and the cost was of course considerably lowered.

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V. MENCL (Czechoslovakia)

I am afraid that the question submitted by the General Reporter concerning the mechanism of slips cannot be answered. Several types of landslides are to be distinguished according to engineering-geological conditions. To illustrate how strange the mechanism of a slide may be let me describe one briefly.

A large landslide occurred in a long railway cut located in a Tertiary basin of the Carpathians in Czechoslovaka after the excavation was finished (Fig. 13). The landslide would not have occurred if a large horizontal residual stress had not existed in the mass. The stress amounted to about 5 kpcb^{-2}. The claystone—the weakest element in the mass —was rather hard and, except for the subsurface water and the water at the end of the cut, the excavation was nearly dry.

Nevertheless a two-stage landslide occurred. The first stage was a "first-storey" slide immediately following the excavation work. The samples (about 120) of the neogene material revealed that the density of the material in the neighbourhood of the cut was only about 92 per cent of that elsewhere. The second stage which occurred several months after the first one consisted of an upheaval of about 5 m of the bottom of the cut. Also, a large crack opened at a distance of about 100 m from the bottom of the cut. This was followed by several backward partial slips as well as by the unexpected appearance of water in the cut.

FIG. 14. View showing backward partial slips.

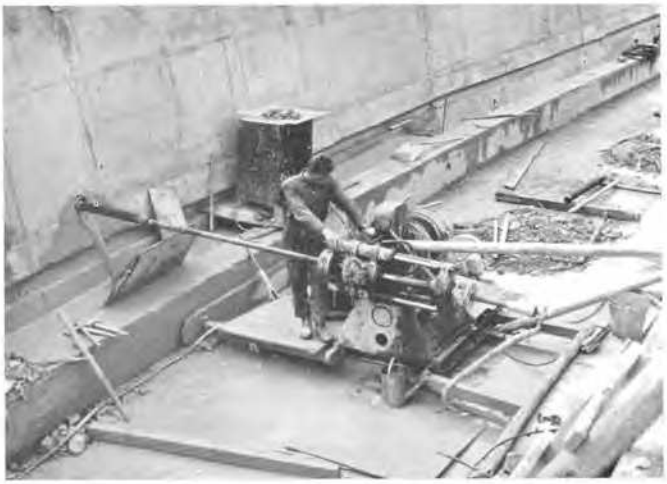

FIG. 15. View showing drilling of horizontal drain holes.

As space does not permit me to present the statical analysis, let me present only its results. In order to stabilize the movement, I proposed two principal measures: to construct an invert frame at the bottom of the cut loaded with fill and to bore horizontal drain holes into the slope. A stability factor of 1.15 could be reached using these measures. Fig. 14 shows the backward partial slips near the end of the cut. Fig. 15 shows the drilling of the drain holes. Two types of bits were used. One type was a homemade chisel bit (Fig. 16).

FIG. 16. Homemade chisel bit and perforated drilling tube.

FIG. 17. View showing nearly completed construction.

Perforated drilling tubes were left in the hole as casing. The other type of bit used was the Scholler-Blockmann rotary bit. The construction is seen in Fig. 17 nearing completion.

Before concluding allow me to submit a question to my Norwegian and Canadian colleagues: can we be certain that landslides in sensitive clays have nothing in common with the uplift of water coming from the bedrock?

A. W. SKEMPTON (Great Britain)

Following the stimulating lecture by Professor Haefeli, I would like to make a few remarks on creep in clay slopes. Clay slopes are subjected to soil creep and weathering and, under certain circumstances, to shear creep as well. Soil creep is restricted to shallow depths, and weathering is a very slow process. From an engineering point of view we are therefore chiefly concerned with shear creep, since this can lead to large displacements or large forces on structures.

At least three types of shear creep can be distinguished. (1) In cuttings where progressive failure is taking place, leading eventually to a "first-time" slip, the rate of creep increases with time, and the shear stresses are greater than the residual strength of the clay. An example is provided by the Kensal Green slip in London Clay (Skempton, 1964). (2) After a landslide has taken place the clay at and near the toe may be broken up to considerable depths and to such an extent that rapid softening can occur during rainstorms. Mud flows then develop, and these exhibit creep, probably of an intermittent nature. So far as I am aware, this type of movement has not yet been analyzed quantitatively. (3) In the main mass of a landslide, the clay has been reduced to its residual strength along the slip surfaces (Skempton, 1964). When the shear stresses increase above the residual strength, as a result of a small excavation near the toe, for instance, or a rise in the piezometric line, quite rapid creep movements may occur. Conversely, if the piezometric line is lowered, either naturally or by drainage works, creep appears virtually to be eliminated (Terzaghi, 1950, Fig. 15 relating to a landslide in the mountains west of Rio de Janeiro). Hence the occurrence or absence of this type of creep should be susceptible to analysis.

As supporting evidence, I will mention some tests recently completed by Mr. D. Petley at Imperial College. It now seems reasonably certain that the strength of a clay along

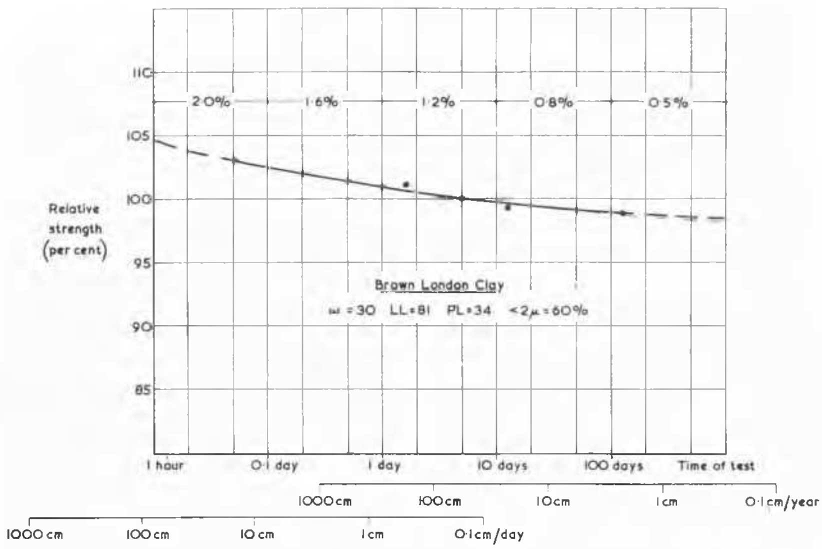

FIG. 18. Effect of rate of displacement on residual strength.

a plane cut in a specimen, before testing, is a close approximation to the lowest residual strength the clay can mobilize. But there remains the question of the extent to which this strength is influenced by rate of displacement. The tests to which I have just referred show that the residual strength is, in fact, affected very little by rate of displacement (Fig. 18); and it is easy to suggest that there is a limiting stress, only a few percentage points lower than the residual strength as measured in laboratory tests lasting only a few days, below which no continuous shear creep can take place. In other words, I am suggesting that the residual strength of clays is closely analogous to the "fundamental strength" of rocks, as defined by Griggs (1936). Also, from the data plotted in Fig. 18, it might be expected that a relatively small increase in stress above the residual strength would lead to rather rapid displacements. Hence, creep movements in this category should be very sensitive to piezometric fluctuations —a conclusion which is in accordance, for example, with field observations on landslides in Japan (Fukuoka, 1953).

The idea that continuous deep-seated shear creep cannot occur at stresses appreciably below the residual strength may, however, be a contradiction of the phenomenon of cambering and valley bulging, as described from several areas in England from Yorkshire to the Weald. Valley bulging, in particular, shows that large distortions can take place in clays and clay shales, probably over long periods of time, and apparently at low shear stresses. Some form of creep or plastic flow has undoubtedly occurred, but possibly under conditions of deep freezing and thawing in the Pleistocene epoch (Kellaway and Taylor, 1953). Thus, at present, it is difficult to argue one way or the other from this field evidence.

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L. BJERRUM (Norway)

Those interested in the stability of natural slopes in sensitive clays will remember Mr. Hutchinson's investigation of the landslide at Furre in Norway where a large flake of silt and gravel failed monolithically on a 10-cm-thick layer of quick clay. A computation of the stability showed that the shear strength at failure corresponded to an effective angle of internal friction of 6°, which is only a fraction of the value measured in the laboratory.

In a contribution to the discussion in Division 2 (pp. 327–8), I described a new simple direct-shear apparatus in which a sample could be failed along horizontal planes in simple shear. Some drained tests have been carried out with a quick clay in this apparatus, and the results are very interesting as they may help to throw light on the discrepancy between the field and the laboratory shear strength found at the Furre landslide and at several other slides in natural slopes in soft clays. The clay used for the tests is the same Manglerud quick clay I described in my discussion in Division 2 (pp. 327–8).

Fig. 19 shows the results of three of the most successful drained simple-shear tests on specimens consolidated at pressures equal to the effective overburden in the field. As observed from the top diagram, the stress-strain curves have a steep slope at small strains. At a certain critical shear stress the curves show a sharp bend. The bend is followed by a stage where the increase in shear stress with strain is slight. With further straining the stress-strain curves rise again and this continues until the end of the test. The shear stress at which the sharp bend occurred was nearly the same for all tests, the ratio of shear stress to vertical pressure being of the order of 0.20, about 40 per cent of the ultimate value. The lower diagram shows the volume change. At low strains the rate of volume change was small. At about 2–3 per cent strain it increased rapidly and reached a maximum at the same strain at which bending starts on the stress-strain curve. With further straining the rate of volume change decreases regularly.

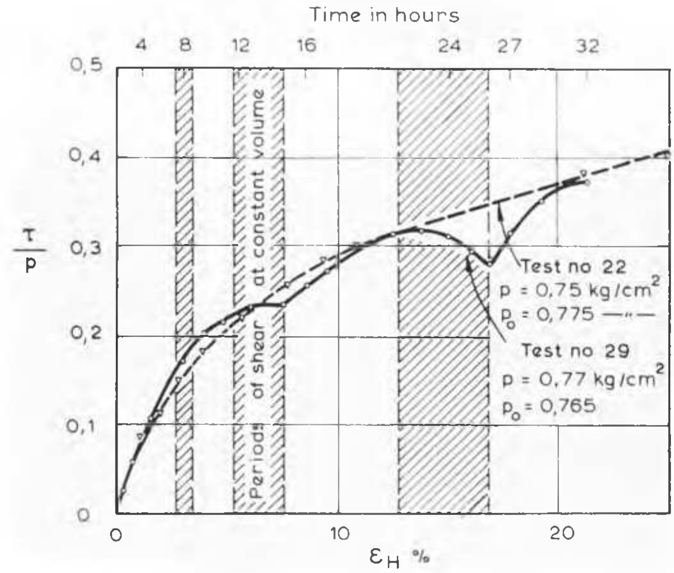

FIG. 19. Results of three drained simple-shear tests on specimens of a quick clay consolidated at a pressure equal to the previous overburden in the field, representative for tests showing a "critical shear stress."

From the results of the drained direct-shear tests it is concluded that in an undisturbed clay the structure will start to break down at a certain critical shear stress which is much smaller than the ultimate drained shear strength. Whereas the clay shows a brittle behaviour below the critical value, the mobilization of additional strength requires large strains and large volume changes.

A drained clay element which is subjected to a shear stress near the critical value is, therefore, in a very delicate state of equilibrium. A small sudden increase in a shear stress will start the collapse of the structure resulting in large strains and increases of compressibility. The collapse of the structure will be accompanied by a simultaneous rise in pore pressure, and if the pore pressure cannot disperse rapidly enough, the corresponding reduction in effective stresses may offset the increase in strength with strain and a failure will occur.

The effect of applying small "undrained" increments in shear stresses to a clay specimen during a drained test was studied in some special tests. During various stages of a drained test further consolidation of the specimen was prevented and the test was continued for a while as a constant-volume test. The result of this test is shown in Fig. 20. The first interruption in the drained test was made when the shear stress was relatively low, corresponding to $\tau/p = 0.16$. At this stage a substantial increase in shear stress could be applied, under conditions of no further volume change, without leading to a failure. Consolidation was therefore again permitted and the test continued as a drained test. The second interruption was made when the shear stress just exceeded what was believed to be the critical stress, the τ/p ratio being 0.23. At this stage an increase in τ/p of only 0.010 (i.e., 4.5 per cent) applied under undrained conditions was sufficient to bring the specimen to failure. The third change in drainage conditions was made when $\tau/p = 0.32$. At this stage a failure occurred for an increase in shear stress of only 0.3 per cent. The ultimate value of τ/p which represents the drained shear strength was not measured but is probably of the order of 0.45. The stress-strain curve observed in a parallel drained test with no undrained interruptions is shown for comparison.

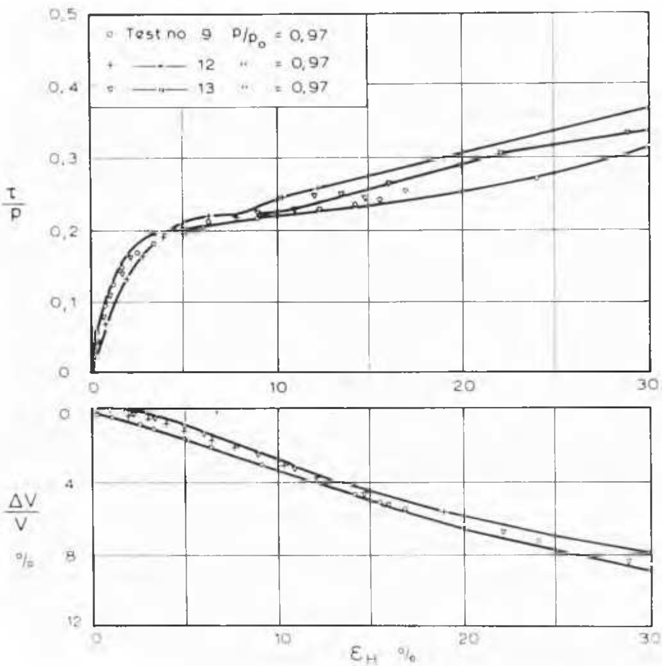

FIG. 20. Results of special simple shear test on a specimen of a quick clay consolidated at a pressure equal to the previous overburden in the field. At three different stages the drained test was interrupted by intervals during which the volume of the sample was maintained constant, indicated by the shaded areas in the figure.

The phenomena described are believed to be of importance in the mechanism of some types of landslides in slopes in sensitive clays. It is, for instance, believed that the above findings might contribute to a better understanding of the landslide at Furre. A large flake of soil resting on a thin layer of quick clay slid out as a consequence of a very small, sudden increase in shear stress resulting from a relatively small failure of the river bank (Hutchinson, 1961). This happened even though the shear stresses were only a fraction of the ultimate drained shear strength.

These results lead to the conclusion that it is dangerous to evaluate the stability of natural slopes in compressible clay with a sensitive structure on the basis of a conventional $c' - \phi'$-analysis. A realistic study of the stability should include a consideration of the effect of small sudden increments in shear stress; as would occur if there was a minor local slip at the toe of the slope.

N. Janbu (Norway)

First, I would like to make a few comments on stability investigations in general, and then some remarks on the paper by *Nonveiller (6/20)*. In general, it is widely recognized that stability investigations, at least for important jobs, contain several components. Some major components may be listed as follows: (1) geological conditions requiring a survey; (2) mechanical properties requiring material testing; (3) minimum safety factor requiring calculations; (4) state of stress requiring calculations; (5) synthesis—practical judgment with all factors considered. These components appear obvious, with perhaps one exception—the state of stress. Personally, I believe that in the future, more efforts should be made to obtain at least an approximate knowledge of the state of stress in addition to the minimum safety factor. This aim appears to be of particular importance for high earth—and rockfill dams and for large natural slopes.

To illustrate briefly the necessity of obtaining knowledge of the state of stress, reference is made to Fig. 21. If the shearing resistance envelope is curved (as it very well may be, as is underscored in contributions during this Conference), then it appears quite obvious that from both testing and calculation points of view it is essential to know where on the stress axis our problem is located. For instance, Fig. 21 indicates that it is perhaps of limited practical value to carry out laboratory model tests at small stresses to simulate a much higher stress condition in the field.

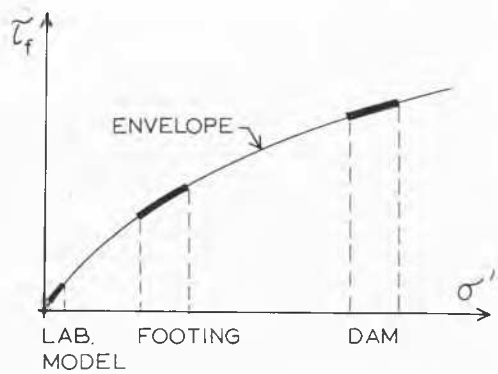

FIG. 21. Illustration of the importance of knowing the state of stress relevant to the stability problem at hand.

Our next step in getting additional information relevant to the final question—"Is this profile stable?"—would, in my opinion, be to include more rationally the results of stress-strain analysis of soils, because only then can we make further progress towards answering the much disputed and important question about the possibility of progressive failure. This would, of course, involve a departure from the purely plastic approach, and it may therefore at present be somewhat far-fetched.

To date most numerical calculation procedures of stability analysis are concerned only with the factor of safety and omit consideration of the state of stress. *Nonveiller*'s paper *(6/20)* is another interesting attempt to obtain the factor of safety for slip surfaces of arbitrary shape. In short, Nonveiller considers moment, horizontal, and vertical equilibrium, but only the over-all moment equation is solved (implicitly) with respect to the safety factor; the other conditions are formulated but not rationally solved. Therefore, additional simplifying assumptions are made about internal forces between layers; and the reader may be completely unaware of the one statically indeterminate quantity involved.

Nonveiller's statement regarding my procedure is very misleading. This can be proved mathematically (as shown at the end of this discussion), by solving Nonveiller's own Eq (14) with respect to F_s, because then one obtains, term for term, a formula identical to my own. Since the formulae are correct, we are only left with the question of convergency, a question which is now much clarified thanks to the circumstance that my procedure has been programmed and used for some time in the U.S.A.

The misleading statement may be due to the use of the somewhat incomplete presentation made in 1954; in 1957 the more complete procedure of slices, including calculation of the state of stress, was submitted to the London Conference (Janbu, 1957).

Speaking only about calculation of safety factors, the two procedures are in fact similar, the main difference in approach being that Nonveiller uses an arbitrary pole for the over-all moment equation, whereas I prefer to be in the ground, so to speak, when considering moment equilibrium. This is not merely a formality because it contains one of the essential ideas behind my procedure, since I still believe that only then can one get sufficiently simple formulae for obtaining both the factor of safety and the state of stress simultaneously by successive approximation, and because at the same time it makes it clear to the reader that he is actually dealing with a procedure containing one statically indeterminate quantity. Moreover, it should be noted that both procedures lend themselves to successive approximation, and therefore no reliable statement can be made about relative accuracy unless the theoretically rigorous values are known, and the iteration is carried far enough for both procedures.

For the mutual benefit of those interested in safety factor calculation, I would therefore like to see Professor Nonveiller clarify the following questions: (1) Is the calculated factor of safety really independent of the location of the moment point? (2) How reliable are Nonveiller's additional assumptions about the internal forces between slices? (3) Can this procedure be extended to include approximate calculations of the state of stress, so that more correct values of internal forces can be introduced in the safety factor calculations? If these questions could be clarified favourably, then I believe there are many who may find Nonveiller's method of approach more attractive and more complete.

Regarding the relative accuracy of different methods of approach I believe that one should more frequently exchange typical profiles for checking other procedures, because in the end it is not the result of just one chosen surface that matters as much as the answer to the all-important question of the stability of the entire profile itself, and in that respect we may have different methods of approach. Such exchanges of profiles and calculation procedures would help considerably in avoiding gross misunderstandings, and above all those investigators concerned then have the opportunity of learning from one another.

MATHEMATICAL PROOF

Reference is made to *Nonveiller*'s paper (*6/20*). The author's definitions are transferred to Janbu's notations (1957) as follows for finite differences:

$$\begin{aligned} B\sin\Psi &= \Delta Q, \quad B\cos\Psi = \Delta P, \quad \Delta T/b = t \\ (W_1 + W_2 + B\cos\Psi)/b &= p, \quad b = \Delta x \end{aligned} \tag{a}$$

Then Nonveiller's notation $[m]$ defined under his Eq (13) reads,

$$[m] = \tau_f \Delta x \sec\alpha \tag{13a}$$

in which the shearing resistance τ_f equals

$$\tau_f = \frac{c' + (p + t - u)\tan\phi'}{1 + \tan\alpha\tan\phi'/F_s}. \tag{13b}$$

Introducing Eqs (a) and (13a) into Nonveiller's Eq (14) (p. 525) one gets

$$\sum\left[\tau_f\Delta x\cos^{-2}\alpha/F_s - \Delta Q - (p+t)\tan\alpha\Delta x\right] = 0$$

which when solved with respect to F_s = constant yields, boundary conditions observed,

$$F_s = \frac{\sum \tau_f \Delta x \cos^{-2}\alpha}{\sum (p+t)\tan\alpha\Delta x + (Q - E_b)} \tag{14b}$$

which together with Eq (13b) is identical with my Eqs. (8b) (Janbu, 1957). The one statically indeterminate quantity, t, is found approximately by introducing an assumed line of thrust.

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A. J. DA COSTA NUNES (Brazil)

The discussion of the methods for stabilization of slopes seems to us of great practical interest—especially for those emergency cases where failure has already started.

REDUCTION OF THE SLOPE ANGLE

Except for very particular cases in which the slope stability is chiefly menaced by superficial erosion, the reduction of the slope angle is the most radical and obvious measure to be taken. The methods available for stability computation allow us to estimate the angle of permanent stability fairly accurately. In general, it is convenient to shape the slope with little benches or berms in order to facilitate drainage and inspection, and to reduce the erosion produced by superficial water. The slope stability depends approximately on the average inclination of the line obtained by linking the slope toe to its top, independently of the berms. Nevertheless, it is evident that these berms themselves have stable slope angles. This point is much more important in the case of slopes on heterogeneous soils.

DRAINAGE

Drainage of the infiltration and superficial water is one of the most important factors in slope stability. The correlation between rains and slides is well-known. Water causes instability due to: (1) development of static pore water and percolation pressures, even small ones; (2) reduction of the parameters of shearing resistance chiefly the cohesion of silty clayey soils; (3) increase of weight—reason for sliding; (4) subsurface erosion (piping); (5) superficial erosion. The need for drains at the points where water leaves the slope must always be emphasized.

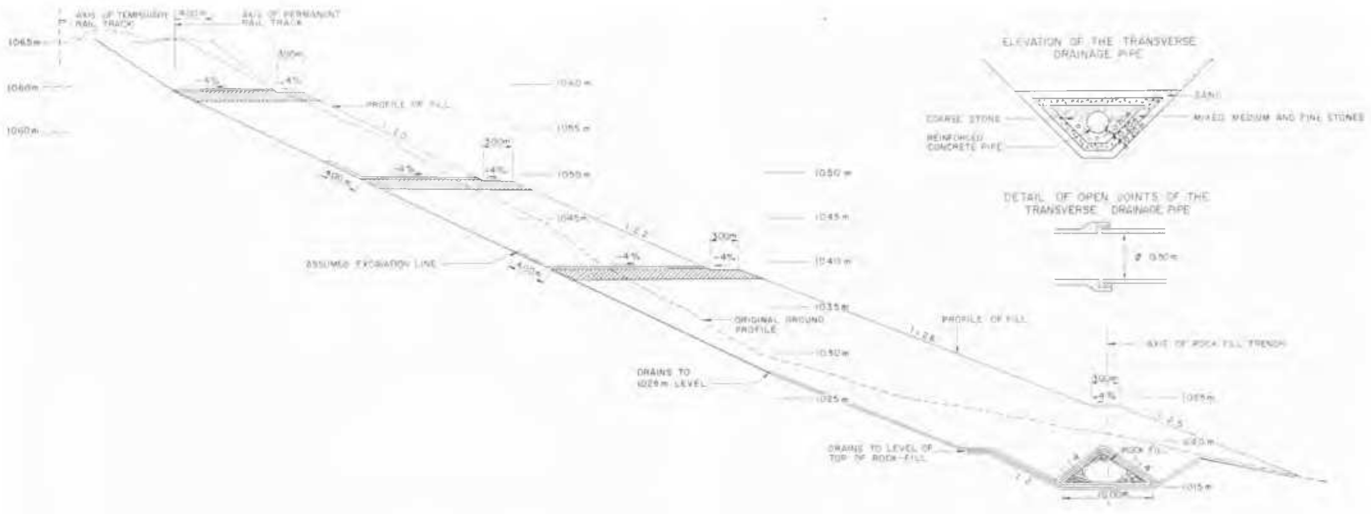

FIG. 22. Bichancos's fill (Estrada de Ferro Central do Brasil) showing details of subsurface drainage.

SUPERIMPOSED BERMS

A berm at a slope toe reduces the average slope angle as mentioned earlier and, because of the favourable position of the additional weight with reference to the centre of sliding, its influence as a counterweight is very often conspicuous.

STRENGTHENING OF THE SHEARING RESISTANCE AT THE SLOPE TOE

Theory and practice show that the shearing stresses in a slope are very high on the foundation, near the slope toe. Under these conditions the increase of stability desired is very often obtained by increasing the shearing resistance of the foundation at the slope toe. This can be accomplished by the use of steel or wooden piles or other structural material that also transfers the load to deeper layers. Alternatively a submerged dam of material having a high shearing resistance might be used. Using this approach, we have stabilized Bichanco's fill for the Estrada de Ferro Central do Brasil (Brazilian Central Railway) (Fig. 22).

RETAINING WALLS

When a slope is very steep a retaining wall at the toe may be the solution. The wall must be provided with drainage and the excavation to its foundation must not menace the slope whose stability is to be improved. Sheet piles constitute a very efficient type of retaining wall, but good foundation conditions and a place for anchoring them outside the sliding mass are required for their use. Very often blocks of rock and stiff layers located in the soil mass do not permit the driving of piles.

BOLTING

To drive anchor bars through a mass of soil, anchoring them outside the sliding mass, is a new method found to be most efficient in stabilizing slopes as well as tunnels. This procedure was the subject of another discussion in this Conference (pp. 526–7).

FILLING OF EROSION CREVICES AND SLOPE REVETMENT

In many cases the principal factor in the deterioration of slopes is erosion due to superficial water. In these cases it is important to fill the erosion crevices with resistant materials such as cement mortar, and to cover the slopes with some bituminous or cement coat. Gunite and soil cement have been employed successfully as cement mortar. We have had very good results filling the erosion holes with soil cement. A very interesting job was the Cadeb one in Rio de Janeiro (Fig. 23). To face slopes against erosion bituminous paint is also very useful.

FIG. 23. Slope stabilization with soil cement (Cadeb, Rio de Janeiro).

PLANTATION OF THE SLOPE

Plantation is efficient against superficial erosion and also contributes to preservation of the soil's moisture. Plants must be carefully selected, keeping in view local climate and flora.

Note however that plantation has no effect on deep earth sliding.

SEALING OF CRACKS

Due to shrinkage by desiccation clayey slopes crack. Water enters the interior of the slope mass through the cracks and reduces the cohesion. It is recommended that such cracks be closed with cement mortar or bituminous paints. We have had very good results with cement grouting or soil-cement mixtures in fissured slopes. This process is, as a matter of fact, recommended in literature pertaining to railway slopes. In some special cases of fissured rock, grouting increases the stability considerably.

I. C. DOS M. PAIS-CUDDOU (India)

The problems of earth dams and slopes have been discussed at length in the papers submitted in this Division, with regard to investigation, design, and construction and post-construction observations. I would like to relate my experiences with the design and performance of some earth dams constructed in India.

EARTH FILL FOR THE HIRAKUD DAM

It is very important to keep in mind, when designing earth dams, that locally available materials are to be used in the interests of economy. It may not always be possible under these conditions to get the correct materials for the core and outer zones of the dam, and therefore, for optimum utilization of the locally available materials, a homogenous dam or one with a selected material for the core and relatively pervious materials with higher density for the outer zones may have to be constructed. An example of this type of construction is the high Hirakud earth dam, the first of its kind to be built in India in the state of Orissa. The materials used for the outer zones were not free-draining, as they should have been. What was wanted was to provide for the outer zones a material with high dry density and relatively pervious to that of the core of the dam.

SLIP ON THE UPSTREAM SLOPE OF AN EARTH DAM

An instance of a slip that occurred in the upstream slope of one of the dams brought out some interesting results after the investigation. The earth fill in the dam was rolled and compacted under optimum conditions of moisture and density. After nearly 6 years, a slip occurred on the upstream side and after a lapse of another 3 years another slip occurred last year, more or less on the same pattern. The soil samples from the body of the dam indicated that, at a certain level, a 6-to-12 in. layer of the soil was soft compared with the rest of the dam. It is possible that this slip, which occurred when the water level in the reservoir was very low, may have been due to low shearing resistance at this level. Apparently at the time of construction, which was spread over a number of years, work was stopped at this level during the monsoon period and later when work was resumed sufficient care was not taken to remove the soft slushy layer at the top. This probably became saturated and swelled during the heavy rains and should have been removed before construction was begun again and fresh layers were laid and compacted. The question of failure due to drawdown was ruled out as the reservoir level was lowered slowly over a period of 6 months or so, the reservoir water being mainly used for irrigation.

SEEPAGE ALONG THE DOWNSTREAM SLOPE

Another of our dams, which was built using clay for the core and random fill for the outer zones, showed signs of sweating on the downstream slope when the reservoir was filled to full capacity for flood storage. In this particular case the full reservoir level was about 2 ft above the top of the clay core and water must have flowed over the core and through the random fill on top of it. Although a horizontal filter had been provided downstream at the base of the random fill, the water did not find its way to this filter to be safely drained away. The sweating could therefore be ascribed to some horizontal lenses of impervious material formed due to percolating water carrying the finer materials in the fill down lower at various levels on the downstream side thereby preventing downward flow and causing sweating on the downstream slope. This could have been prevented if an inclined filter had been provided on the downstream side of the clay core to trap the seepage coming from the upstream side.

BREACH IN AN EARTH DAM

Recently there was a breach in one of our earth dams when it was being filled for the first time. Due to heavy rains, rapid filling of the reservoir occurred and it was observed that water was oozing from the downstream toe of the dam. Piping and scouring of the downstream slope and subsequent breach in the dam occurred. The cutoff provided for the dam was a puddle trench carried down to rock; grouting in the bedrock was also provided. The failure might have been due either to settlement of the earth fill and subsequent cracks or to some other causes. It would be interesting to know of experiences in other countries with the behaviour of dams filled by rapid inflow from a heavy rainfall.

COMPACTION CONTROL

Lastly, I would like to mention an experience of mine in compaction of earth fill. The usual procedure is to determine the necessary number of passes and thickness of the layers by compacting the fill in a test embankment to a predetermined optimum moisture and density. In this case it was observed that the optimum density could not be achieved after a number of roller passes for the residual soil and weathered rack used for the fill. On checking the compaction characteristics of the soil before and after it was rolled in the embankment, it was observed that the kneading action of the rollers had changed the particle size distribution of the soil and that the density after rolling was lower. Hence the desired compaction on the basis of the earlier tests could not be relied upon.

These are a few experiences of the behaviour of dams in India and of the materials used for construction. It would be interesting if similar or other observations have been made in other countries where earth dams are being constructed.

G. SEADEN (Canada)

It is with great interest that we have read paper *6/29*, Compressibility of broken rock and the settlement of rockfills, by *Sowers, Williams, and Wallace*. Our recent experience with a large (1,000,000 cu.yd.) reclamation project appears to confirm certain findings of their research programme.

During the site preparation for the universal international exhibition to be held in Montreal in 1967, an area in the vicinity of MacKay Pier has been reclaimed by means of rock fill originating from a quarry located in the adjacent riverbed (Fig. 24). The rock used was dark grey Utica shale which displays a definite horizontal bedding and a fine network of random calcareous veins. Unconfined compressive strength of this rock displays a wide scatter ranging from 1000 to 10,000 lb/sq.in. with an average value of approximately 4000 lb/sq.in. and when quarried out, it rapidly disintegrates into small flat particles due to a complex physico-chemical process.

Previous experience (Peckover and Tustin, 1958) with this type of rock had indicated that indiscriminate dumping in high lifts would result in significant settlements of the fill over a prolonged period of time, a development which was incompatible with the almost immediate projected use of the reclaimed area for the construction of a network of roads, services, and buildings. It was decided, therefore, to

FIG. 24. Over-all view of the site under reclamation.

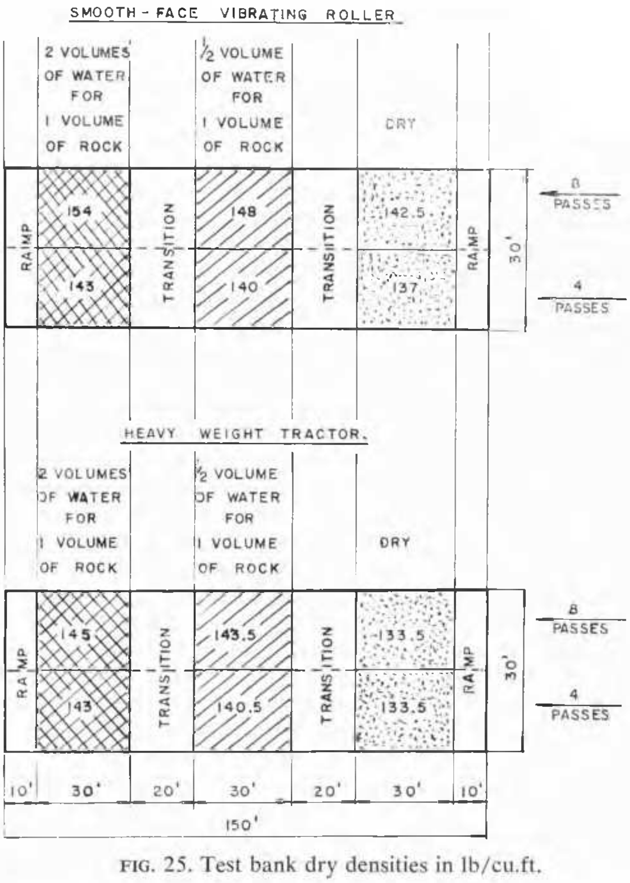

FIG. 25. Test bank dry densities in lb/cu.ft.

place the rock in shallow layers and subject it to heavy compaction so that the resulting material would have minimum settlement characteristics.

Since the usual standards of density (Proctor, etc.) cannot be satisfactorily used in the case of a soft coarse material, it was decided to refer the compacted field density to the solid dry density of the rock measured at approximately 170 lb/cu.ft. All crushed rock was to be compacted to a minimum of 85 per cent of the above-mentioned solid dry density. A test bank was established in order to determine the type of compacting equipment, the amount of compaction required, and the effect of sluicing.

Two types of compacting equipment were tested, a 6-ton variable frequency vibrating smooth face roller and a heavy (20 tons) crawler tractor and the test bank was divided into areas of different compaction effort (4 passes and 8 passes) and different amounts of sluicing water (dry, ½ volume of water for 1 volume of rock, 2 volumes of water for 1 volume of rock). The water was added by means of spray nozzles, in such a manner that no localized washing out would occur. Half of the water volume was applied prior to the compaction of each layer, while the other half was added after 50 per cent of the compaction was carried out. The results of the test bank are shown in Fig. 25.

From this information and from the visual observation of the test bank as well as of the subsequent fill operations, we have concluded that the effect of the sluicing water in case of soft rock such as shale is probably threefold: it accelerates the disintegration of shale into smaller particles (a phenomenon relatively independent of pressure); it facilitates relative movements between rock particles within the compacted mass; and it breaks up the crust of fine particles, which forms on the surface, particularly when smooth face compacting equipment is being used. The last phenomenon results in a migration of fines into the rock fill and yields an open surface which transmits the compaction effort more readily. Sieve analyses of the compacted material (Fig. 26) indicated the presence of such fines as well as the crushing

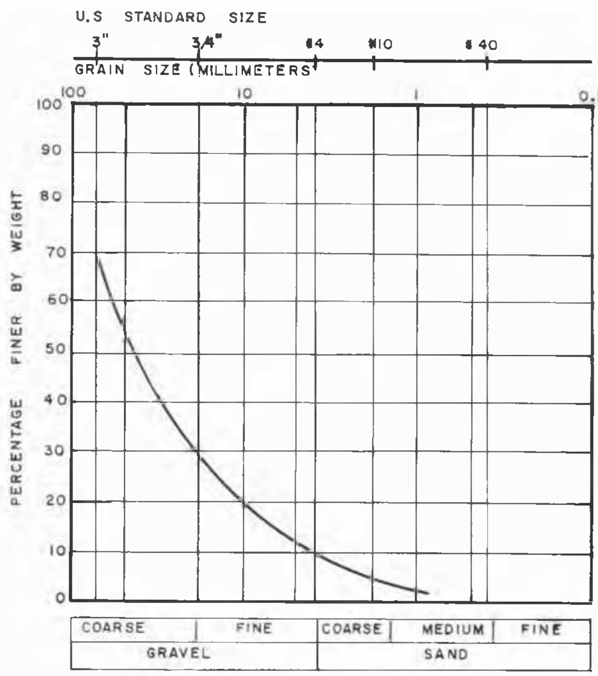

FIG. 26. Mean sieve analysis of crushed shale after compaction.

effect of the compaction from the original quarried 4 to 12 in. size stones.

In the subsequent filling operations, the amount of water applied continuously had been reduced to ⅛ volume of water to 1 volume of rock with success and the compaction effort was established at 8 passes of the smooth face roller. The resulting density of the rock fill, frequently controlled by means of large size (200–300 lb) field density tests, gave consistent results of 145 ± 3 lb/cu.ft. The fill has been in place for over one year, and although it has not been possible to obtain accurate settlement values, various observations would indicate that it does not exceed 0.5 per cent of the fill height.

ACKNOWLEDGMENT

The author wishes to acknowledge the permission of the Canadian Corporation for the 1967 World Exhibition to publish the results of tests which were carried out on their behalf.

REFERENCE

PECKOVER, F. L., and TUSTIN, T. G. (1958). The St. Lawrence Seaway–soil and foundation problems, *Engineering J.*, Vol. 41 (Sept.).

CHAIRMAN MARSAL

Now, our General Reporter will comment on the outcome of this Session.

GENERAL REPORTER MOHAN

All you will agree that we have had a very interesting and fruitful discussion amongst the panel members and from the floor. I feel that the Chairman deserves to be complimented on the skilful manner in which he organized discussions among the panel members.

There is still no general agreement on the mechanism of the development of slides. Although the problem is more of academic interest, collection of some more data on this subject will be useful. A significant point regarding the importance of the rheological properties of rock fills has been made and the need for proper testing of heterogeneous material in a rock fill has been rightly stressed. Proper evaluation of the angle of internal friction in a rock fill under high pressure does appear to be necessary.

The influence of residual stresses on slides in certain clays is now recognized. A new concept of shear creep has been introduced by Professor Skempton and it deserves serious consideration. Professor Bjerrum has also given an interesting account of critical shear stress and its importance.

It is indeed difficult for me to summarize all the discussions in such a short time and I shall look forward to reading the printed report of them all.

(The remarks of the General Reporter for Session 8 presented to the Closing Session appear on p. 596.)

CHAIRMAN MARSAL

I want to express many thanks to our General Reporter for his interesting review of the Session. The members of the Panel deserve warm applause for their enlightening remarks. Also, I offer my deep appreciation to contributors from the floor. I close Session Eight.

WRITTEN CONTRIBUTIONS CONTRIBUTIONS ÉCRITES

P. ANAGNOSTI (Yugoslavia)

Several papers presented to this Division discuss the true meaning, numerical estimation, and definition of the factor of safety. Various statements and viewpoints have been expressed, and attention should be paid to the conclusions in paper *6/17* (*Londe and Sterenberg*).

It is understandable that the statically indeterminate stress distribution along a slip surface requires the established stress-strain or stress-strain rate relations for the rigorous computation of the factor of safety. As these relations have not been satisfactorily established thus far, computations of the factor of safety in all common methods are based on the failure conditions in terms of stresses only. This is also valid for estimation of the bearing capacity of footings, piles, and all similar cases where the limit equilibrium (failure) conditions of a soil are of importance.

The definition of the factor of safety is not a unique one in engineering science, and, in general, it can be attributed to any of the unknown values in the analysis that is, to shear strength, angle of slope, active forces, and so forth. Its value is determined in general by tests or by experience and depends on the unknown value to which it was attributed. In stability analysis the most common meaning of the factor of safety is the degree of the acceptable mobilization of the shear stresses along the most unfavourable surface selected. This surface, usually called the slip surface, replaces the unknown envelope of the failure planes in the state of the limit equilibrium of the soil continuum before noticeable, large displacements are developed.

The value of the stability factor in the analysis of slopes depends on the distribution of the resistance forces along the slip surface. This distribution is not arbitrary in any respect, because it must satisfy the equilibrium with external forces and at the same time remain physically admissible.

For homogeneous slopes and circular slip surface Frölich and Biarez proved that the difference between factors of safety obtained on the basis of the most favourable and most unfavourable distributions of the resistance forces is not larger than 10 per cent. For non-homogeneous slopes and non-circular slip surfaces this difference can be significantly larger. In order to obtain more limited distribution of the resistance forces along the slip surface, the interslice forces were introduced into the analysis. This provided additional limitations for distribution and the physical admissibility of interslice forces. Satisfying the equilibrium equations for each slice, with limitations regarding physical admissibility and degree of shear strength mobilization along the slip surface and on interslice surfaces, it can be required that the maximum shear strength mobilization on the slip surface and interslice surfaces does not differ, but also larger or smaller mobilization on the interslice surfaces can be allowed. The larger the shear-strength mobilization along the interslice surfaces, the more significant the deformations along the slopes can be expected to be. These deformations should not affect the stability of the slope as a whole.

The total number of the available equations for equilibrium of each slice is still not sufficient for estimation of the factor of safety and additional assumptions for each slice have to be implemented. The value of the factor of safety, the distribution of the interslice and resistance forces, and the mobilization of the shear strength along the interslice surfaces depend on these assumptions. Fig. 27, illustrating the simplest example, shows that the factor of safety varies between values of 2.00 and 1.50 for assumptions of full shear-strength mobilization and no mobilization on the interslice surfaces. The limit position of interslice forces which are physically admissible has been examined and the corresponding region of the positions of the resistance forces found. Therefore, the physical admissibility criterion has been satisfied and the difference between the minimum and maximum values of the factor of safety is large.

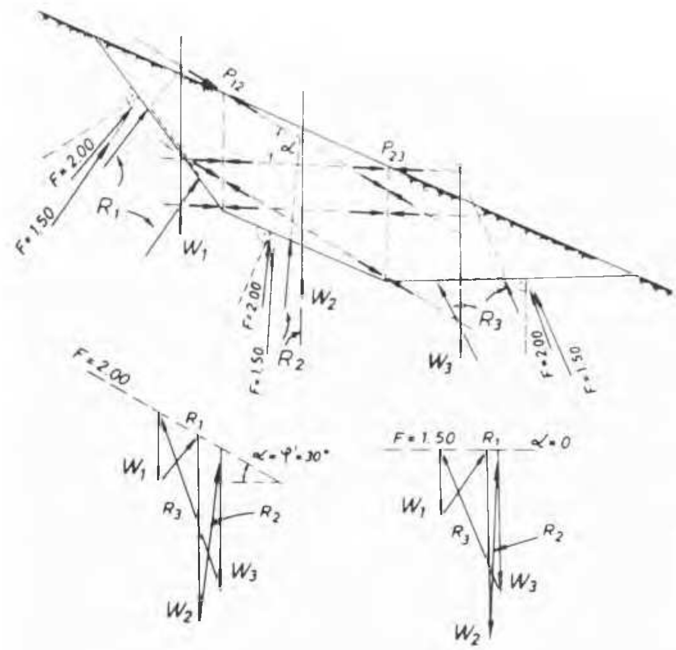

FIG. 27. Example showing the variation of factor of safety with different assumptions regarding shear-strength mobilization on the interslice surfaces.

In the case of more than 4 slices the positions of the interslice forces depend on their inclinations and *vice versa*. Thus for each particular assumption the relevant distribution of interslice and resistive forces and a corresponding factor of safety can be found. It is, however, desirable to select a possible assumption parameter which will not significantly change the factor of safety even for limit possible values of this parameter. The attempt has been made in the paper by Morgenstern and Price recently published in *Géotechnique*, where the numerical solution for computation of the factor of safety for the slip surface of any shape is presented. The changeable parameter is the distribution of the inclinations of the interslice forces. According to the cases examined this parameter does not have much influence on the value of the factor of safety. The use of the electronic computer can quickly provide the results of all required values for verification of the physical admissibility of distributions and positions of resistance and interslice forces.

In conclusion it can be pointed out that although there is now no possibility of relating state of stress to the state of strains, the application of the limit equilibrium analysis can provide the values of the factors of safety, which has been proved by the present experience. These values can be used as a sound basis for structures of homogeneous or non-homogeneous sections and for structures whose dimensions are comparable to the preceding ones.

R. A. BARRON (U.S.A.)

The expression relating the shear stress to the effective normal stress and the shear strength parameters c' and $\tan \phi'$ given in Eq (1) of paper *6/20* by *Nonveiller* contains an inferred assumption that the vector curve (Fig. 28) from point A to point B on the strength curve is a vertical line. This condition exists in the drained direct-shear test, although the strength curve for such a condition may not be the same as that given by c' and $\tan \phi'$ because of prestressing of the soil in the latter case. When the soil tries to expand during undrained shear, vector path AC, the expression of Eq (1) is safe but it is not safe for a soil which tends to consolidate under shear, vector path AD. For a soil which tends to a structural collapse on shear, vector path AE, the expression is most unsafe.

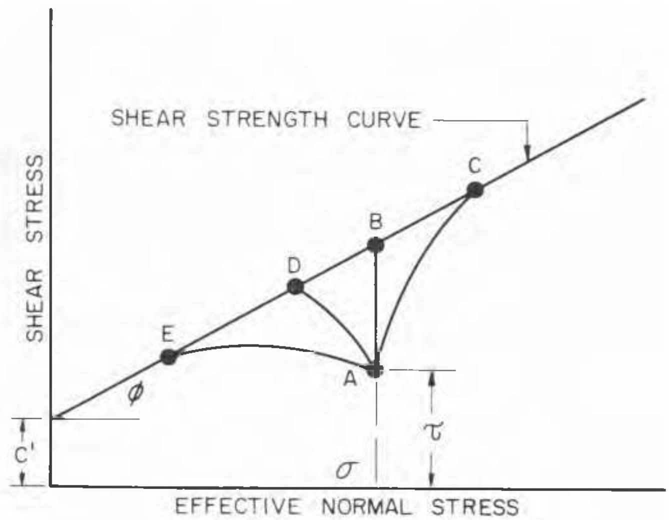

FIG. 28. Possible vector curves for different soil properties.

A convenient way might be to plot a total stress shear strength curve with the associated pore pressures induced by stress changes for use in a stability study. By such a procedure one would obtain an estimate of pore pressures induced by stress changes in an undrained state. Pore pressures caused by seepage would be added to the stress-induced pore pressures to obtain the total pore pressures. Such an estimate would be of value when field pore pressures are obtained during construction. Difficulties now exist in using such stress-induced pore pressures obtained from laboratory specimens because of the stress changes in undisturbed samples, the non-uniform stress conditions in laboratory specimens, and the fact that such specimens are tested as cylinders and not in plane strain. Further research is needed to determine the extent of influence of each of these factors.

When piezometers are installed in the field to observe pore pressures induced by stress changes, undisturbed samples of soil should be taken at the piezometer tip locations and tested in the laboratory to develop the relationship between stress-induced pore pressures for various degrees of consolidation under the stress changes for future comparison with field measurements.

R. W. BRANDLEY (U.S.A.)

The special lecture entitled "Modern Canadian Dams," presented by Mr. J. K. Sexton of Montreal Engineering Company, showed a clay cutoff which is being used in the

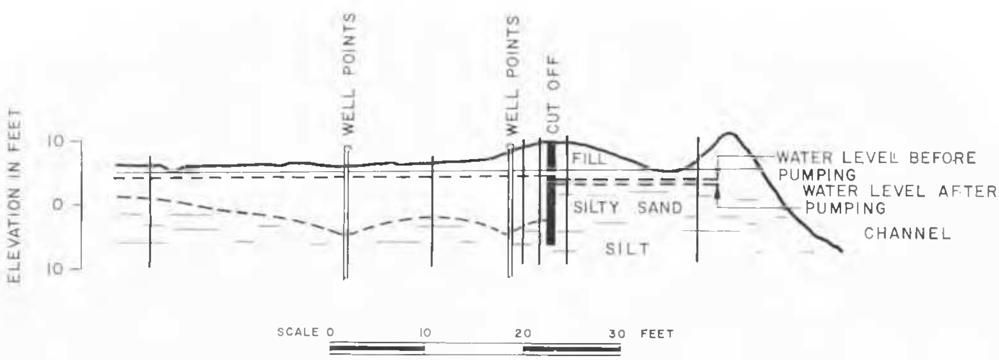

FIG. 29. Section through test area showing drawdown of the water table due to the well points.

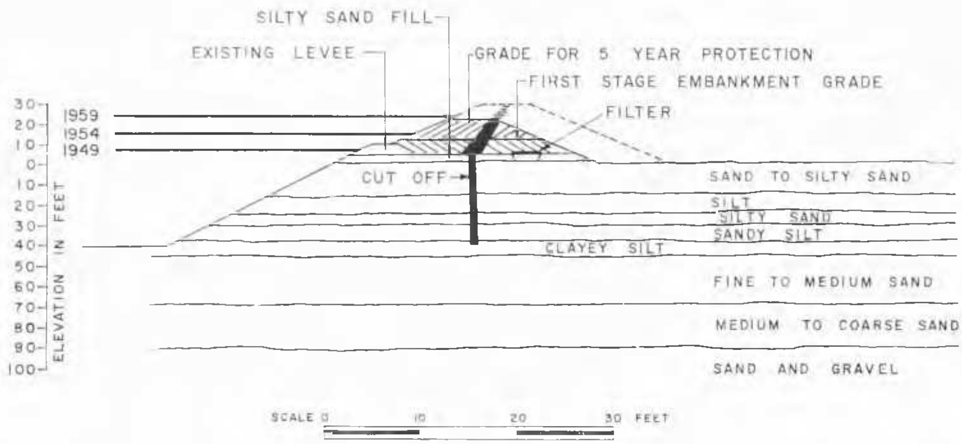

FIG. 30. Typical section showing the cutoff wall and the stage construction of the levees.

construction of one of the dams in British Columbia, Canada. The core trench excavation was kept open using a bentonite clay slurry. The trench was backfilled with a specially graded sand, gravel, and clay soil.

This process was used in 1950 during the construction of a levee system around Terminal Island in Southern California, where the Union Pacific Railroad Company had an oil field. The island had subsided some 15 ft by 1950 and was subsiding at the rate of approximately 1½ ft/yr at the time the levees were constructed. The levees were constructed for the purpose of protecting the land on Terminal Island from the ocean in order that oil production could continue. It was proposed to raise the levees some 15 to 20 ft in stages as required.

The soil stratification throughout the Terminal Island site consisted of varying strata of sands, silts, and sands and gravels. Before any construction was undertaken, the maximum head between the high tide and the lowest land behind the levee was approximately 5 ft. Even with these low heads, there was evidence of piping behind the levees and a very wet condition existed throughout the site.

A cutoff was designed for the entire area on which levees were to be placed. This cutoff was carried into the silt or clayey silt material which had a low coefficient of permeability and to sufficient depth to control the flow of water under the levee section and the hydrostatic uplift pressure behind the levee. The cutoff in the existing ground was constructed by the use of heavy-duty trenching equipment which cut a trench 3 ft wide and to depths ranging from 15 to 42 ft. The trench was kept open by filling it with a drilling mud, similar to that used in the drilling of oil wells. The drilling mud was composed basically of bentonitic clay with a sufficient amount of boron added to increase the unit weight of the material. The drilling mud held the trench open adequately but it was found that the trenching operations through the drilling mud caused sands to settle to the bottom of the trench creating a pervious zone 10 to 20 ft deep in the bottom of the trench. This sand was pumped from the bottom of the trench with a certain amount of the drilling mud and the slurry was centrifuged again using the oil well drilling equipment; the sand was thereby removed and the drilling mud was reused. After the trench had been completed and cleaned, it was backfilled with an air-dried bentonitic type of clay obtained from the Mojave Desert. This clay was broken down to approximately 1-in., maximum size, and was compacted in place by means of a drop weight compactor. No effort was made to grade the backfill materials and samples cut from the core trench six months later showed that the clay had become a homogeneous mass due to expansion with saturation and there was little evidence of the clay materials penetrating adjacent sand soils. The clay cutoff in the levee section was constructed as a portion of the levee.

In order to determine the effectiveness of the clay cutoff, a test section was installed. This test section consisted of constructing a cutoff well through the silty sand layer into a silt stratum. Two rows of well points were placed on the land side of the cutoff and the well points were pumped until a stabilized flow condition was obtained. The drawdown of water table was observed by means of piezometers. Fig. 29

shows the installation of the test section and the drawdown of the water table. It can be seen that the cutoff wall effectively cut off the flow of water.

After the test section had been completed, the remaining cutoff walls and levees were constructed around the property, as shown in Fig. 30. The installation has been so successful that even with additional subsidence, the land behind the levees has dried up and the seepage through the cutoff has been extremely small. Uplift pressures behind the levees, as measured by piezometers, are low.

J. T. LABA (Canada)

Pietkowski and Czarnota-Bojarski (6/22) have presented a very interesting paper in which the time-space relationship for a quick earth slide is discussed. However, I would like to take issue with some aspects of their analysis and their main conclusion. The authors state that the theoretically calculated time of failure introduced in their paper agrees quite well with phenomena observed in nature. Unfortunately, they do not supply us with the "actual observations" with which the theoretical calculations are said to correspond. The direct application of Newton's law of motion to the calculation of the time-space relationship of a rapid earth slide, where the deformable body of sliding earth mass is considered to be rigid, may lead to contradictory conclusions.

It is true, as mentioned by the authors, that the well-known procedure elaborated by Fellenius for examination of the stability of slopes is based on the principle that the investigated mass of earth is a "stiff block." However, the method is used mainly to derive an expression for the factor of safety, when only prior to, and at the instant of sliding, the whole mass is considered to be intact. Thus, when the "method of slices" is used, it is common to assume that forces developed on vertical planes between adjacent slices maybe neglected, as they do not affect the equilibrium of the mass as a whole.

On the other hand, the authors' discussion of the time-space relationship deals with the soil from the time of the instant of sliding to the time when the whole earth slide comes to rest. During this time, the sliding earth mass, in overcoming the resisting action of its internal forces, will assume a completely new shape. In these circumstances, to consider the sliding soil mass a rigid body is rather far from reality.

From trial studies, the authors have observed that the sliding earth mass will stop at the distance shown on their Fig. 1, after the centre of gravity moves a distance of 175 cm; this, being observed, can be accepted without discussion. By applying "dynamic computation" to the deformable body, the authors simplified a rather complicated problem. The time t_6 was calculated by assuming that the motion of the centre of gravity of the discussed slide was rectilinear with constant acceleration. The numerical value of the acceleration was taken as being equal to one-half of the acceleration produced by "resulting force" $S = 6.4$ tons, tangent to the failure line of sector 6. The difference in direction between the rectilinear motion of the centre of gravity and its assigned acceleration appears to have been disregarded by the authors.

The final position of the earth mass after sliding indicates that the soil particles located in different sectors of the investigated earth slide were compelled to travel unequal distances before reaching their final position. Assumption of a constant value of linear acceleration would lead to the conclusion that the sliding earth mass will not come to rest at a certain moment, but gradually, starting with particles travelling the shortest distance. On investigation of sector 6, we could say that some of its soil particles travelled as far as 400 cm, if they moved with average acceleration $p = 126$ cm/sec^2, and then the required time would be 2.52 sec. Since this does not agree with the time, t_6, of 1.67 sec derived by authors, we may conclude that the time t_6 does not include the whole earth mass, but only the soil particles which moved a distance of 175 cm. On the other hand, the assumption that the whole of the sliding earth mass will come to a stop at the same time would lead to the conclusion that the various particles had to travel at different accelerations.

It is evident that "dynamic computation," when used to evaluate the time required for the earth slide to complete its cycle, breaks down in detail, as it neglects the actual behaviour of the sliding earth mass and furnishes only approximate results. Furthermore, the total time, T, as calculated by the authors is not defined by any visual boundaries which could be successfully checked during experimental work.

In their theoretical procedure, the authors divided the whole earth slide into six sectors. The partial times t_1 to t_5 required to destroy the links of cohesion along the sliding surface of sectors 2 and 6, by a movement of 4 mm, were calculated. Following the method presented by the authors, it is evident that for each sector discussed, the principle that the acceleration should be proportional to the resulting force S and in the same direction was followed. Limited movements of 4 mm producing local soil consolidation only were assumed to be rectilinear and partial times from t_1 to t_5 were obtained.

However, the procedure used to evaluate the largest of the partial times t_6 is questionable. The time t_6 required for the centre of gravity of the whole earth mass to move 175 cm, was calculated (disregarding the direction of acceleration) by assuming that the earth mass will travel with an initial acceleration of $p = 253$ cm/sec^2, equal to the acceleration computed for the lowest sector 6.

Another factor, which also appears to be disregarded, is that the acceleration computed for the lowest sector is always proportional and in the same direction as the sector's resulting force S. The arbitrary selection of the required number of slices will affect not only the direction of this force (always tangent to the rupture line of the sector but also, slightly, its value. In the authors' way of thinking, there is no provision for the above.

It is evident that the method introduced by the authors is applicable only to cohesive soils, as the total sum of partial times t_1 to t_5 (0.23 sec) was derived from the arbitrary assumed shearing movement of 4 mm required to destroy the links of cohesion. Also, assumption 3, that the first limited movements produce local consolidation only, could require a more complicated analysis, including the effects of the motion of the shearing strain on the volume change and the pore water pressure change. Furthermore, the validity of the calculated time required to overcome cohesion resistance depends completely on the correctness of the authors' assumption that sliding always begins at the top and proceeds downwards. Contrary to this, *Mencl (6/18)* refers to a case history where a slope failed due to residual stresses causing heaving near the toe of the slope.

In conclusion, it is obvious that in the "dynamic computation" applied by the authors to earth slides, the required time for this phenomenon to occur was directly proportional to the square root of the distance by which the earth mass moved and inversely proportional to the square root of the acceleration. The time and the movement if obtained from the experiment could determine the value of the actual average acceleration in the direction of movement. The comparison between theoretical and experimental values of the acceleration could be a valuable contribution, and support theoretical computations or establish some degree of accuracy. Unfortunately, in their paper, the author did not supply the experimental results observed by them in support of the validity of their theoretical investigation.

L. P. MISHU AND A. G. ALTSCHAEFFL (U.S.A.)

The study presented by *Scherrer (6/25)* deals with the volume change of a partly saturated *in-situ* foundation soil when subjected to wetting. A similar phenomenon occurs in clays when compacted dry of optimum moisture content.

The U.S. Bureau of Reclamation, (U.S.B.R., 1962) has reported on the volume changes occurring in natural foundation soils upon wetting; the *in-situ* density was considered an important criterion in determining if the resultant volume change would be so rapid as to be termed a "collapse." Settlements approaching 3 ft for foundations on a 20-ft thick sandy layer of soil subjected to wetting have been reported (Jennings and Knight, 1957). The importance of the degree of saturation from tests on partly dried slurry samples was suggested (Jennings and Burland, 1962).

The rapid volume changes due to wetting of two compacted clays: the "B" horizon of a glacial silty clay and a limestone residual soil ($w_L = 43$ per cent, $w_P = 22$ per cent, $I_P = 21$ per cent) have been studied (Mishu, 1963). Samples were compacted with a kneading compactor to densities approximating those produced by the standard Proctor compaction effort, as well as higher and lower densities; compaction curves for the Crosby "B" soil are shown on Fig. 31. Three degrees of saturation (dry of optimum, wet of optimum, and at optimum water content) were used as initial conditions. The samples were subjected to various applied pressures in one-dimensional compression in a fixed ring consolidometer (diam. 2½ in., height 1 in.). Volume changes were measured during loading in the partly saturated state, as well as during and after wetting. An example of the relation between these volume changes is shown on Fig. 32.

The results of wetting tests on Crosby "B" soil are shown in Fig. 33. The volume changes indicated are due only to wetting after "equilibrium" had been reached under the applied pressure in the partly saturated state. The major part of the volume reduction occurred immediately upon the

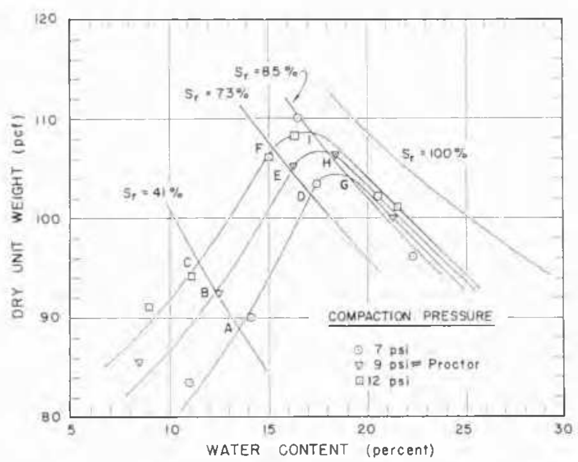

FIG. 31. Kneading compaction curve—Crosby "B" soil.

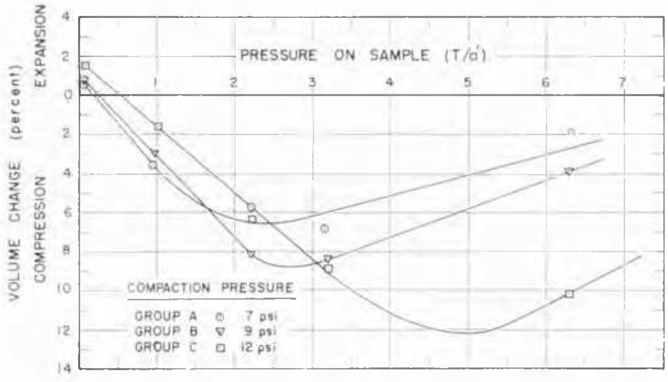

FIG. 32. Volume change due to wetting—Crosby "B" soil—$S_r = 41$ per cent.

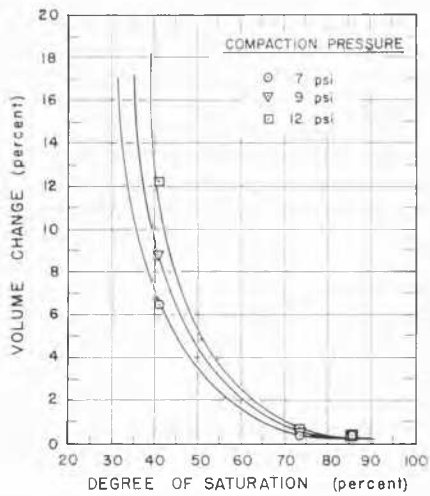

FIG. 33. Maximum volume reduction due to wetting—Crosby "B" soil.

addition of water to the sample and can be termed a "collapse" of the soil. A different applied pressure is associated with the maximum amount of "collapse" for different compaction efforts used in sample preparation; furthermore, the higher the compaction effort, the larger the maximum "collapse" volume change upon wetting.

The higher the initial degree of saturation, the less the amount of "collapse" upon wetting, as shown in Fig. 33 for Crosby "B" soil. Similar phenomena were observed in the case of the limestone residual soil. It was noted that volume changes exceeding one per cent are observed at compaction water contents approximating 2 per cent dry of optimum.

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E. NONVEILLER (Yugoslavia)

For about 60 years the stability analysis of slopes has been based mainly on the assumption of plastic failure on circular cylindrical slip surfaces, the sliding earth body being regarded as rigid. The problem is statically indeterminate and some assumptions must be made in order to replace the deformation conditions available for the solution of elastic systems. This is usually achieved by assumptions on the distribution of the interslice forces; in some methods even these are neglected. Bishop has developed a general numerical solution which gives an accurate value of the safety factor for any condition of loading, pore pressure, and hydrostatic pressure, and of shear-strength characteristics along the slip surface. Similar solutions exist for slip surfaces of general shape. An analysis of the results obtained by all these methods will show that the assumed distribution of stresses along the slip surface or on the interslice planes does not have a great influence on the resulting safety factor. These methods are satisfactory for the solution of most cases where the actual slip surface is cylindrical-circular or elongated.

The results can however, be seriously in error, if the shape of the slip surface implies the occurrence of important deformations in some zones of the sliding body. In such cases, the basic assumption of all these methods—that the earth body is rigid—does not hold true and the deformations of the sliding body influence the computation result to a high degree.

In 1954, Samsioe presented a solution for plastic failure of a triangular wedge with hydrostatic load on one boundary. In this case the critical slip surface was convex (Fig. 34a). This theoretical result was also found in experiments carried out independently by Dr. Reinius in Sweden and by myself. Fig. 34b shows the slip surface of a composite dam section resulting from these studies. A graphical solution for such cases is available in which the sliding wedge is divided along secondary slip planes and the safety factor is found by trial and error.

In such solutions the assumed degree of mobilization of the shear strength along the interfaces between materials of different properties influences the result of the computation to a high degree. From the polygon of forces shown on Fig. 34c, it is seen that the required magnitude of force E_A on the interface (a-b) needed for equilibrium decreases and the available resistance E_P increases with increasing obliquity (δ); thus the safety factor increases also. The obliquity (δ) depends mainly on the differential deformations within the sliding wedge due either to different settlement of the materials or to deformations induced by the external load or to both. The assumption of the stress distribution within the sliding body has a great influence on the resulting safety factor in such cases.

A. PIASKOWSKI and Z. KOWALEWSKI (Poland)

It was shown in our paper *(6/21)* that, when determining the active earth pressure of the soil skeleton, p_a, it is advisable to take into account the arching which decreases the pressure p_a in short trenches at greater depths. As an illustration a nomogram was given in the paper which was calculated for the definite ratio $\gamma' : \gamma = 0.57$. It follows from successive analysis that instead of Eqs (4a) and (4b) only one formula

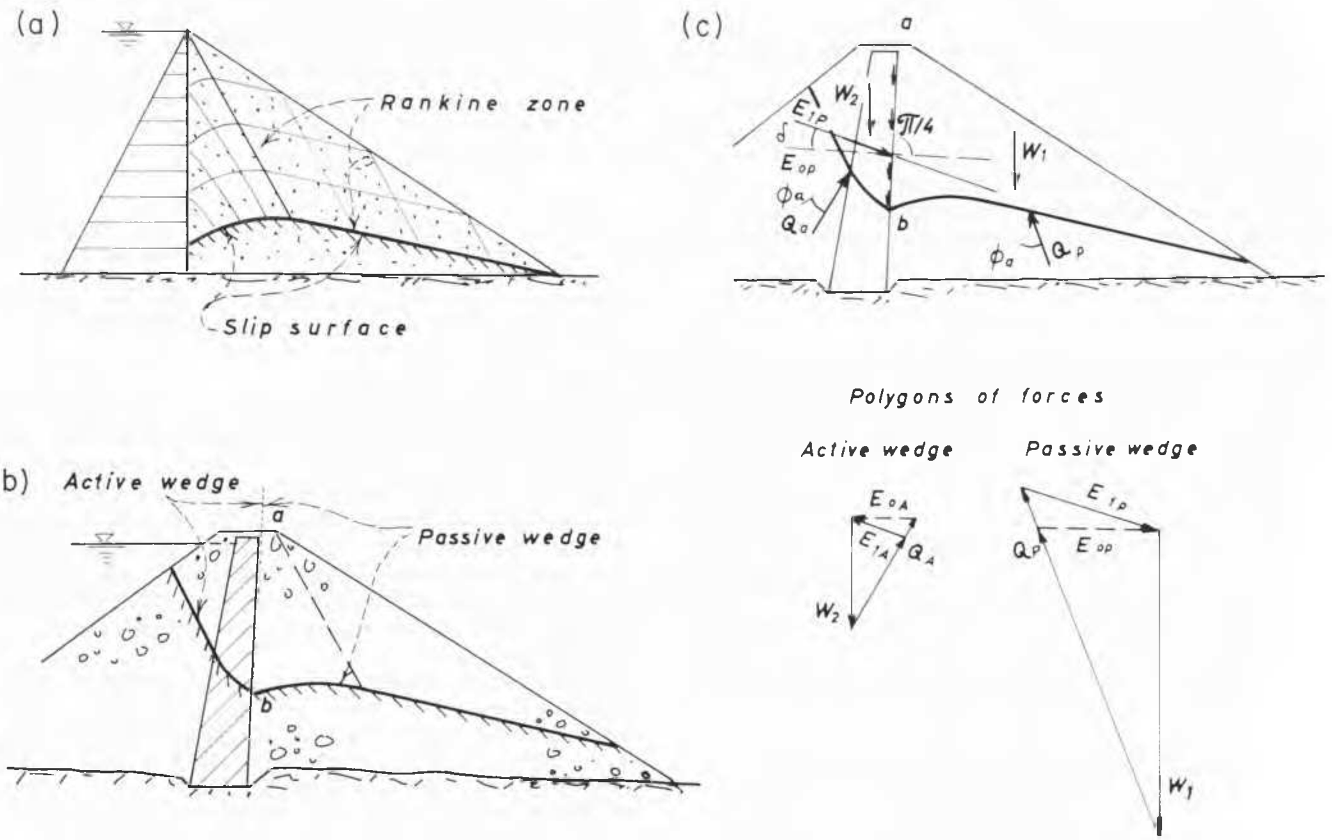

FIG. 34. Convex slip surface failure plane found both by theory and experiment.

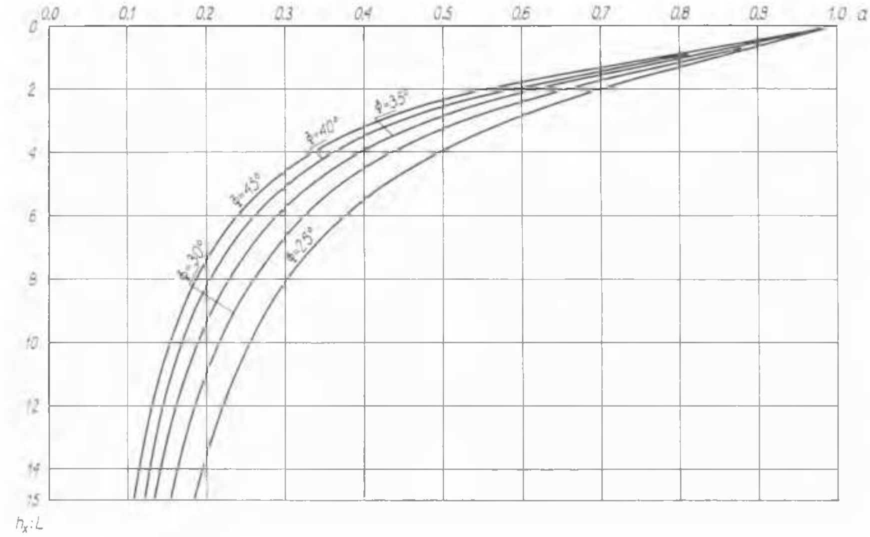

FIG. 35. Nomogram for use in the application of clay suspensions for stability of vertical sides of deep trenches without the use of strutting.

may be used which also takes into account different ratios of $\gamma' : \gamma$:

$$p_a = a\left[1 - \left(1 - \frac{\gamma'}{\gamma}\right)\beta\right]\gamma h_x \cdot \tan^2\left(45^0 - \frac{\phi^0}{2}\right) \quad (1)$$

in which $\beta = 0$, when $h_x < h_w$, and $\beta = (h_x - h_w) : h_x$, when $h_x > h_w$, where a is the value according to the nomogram shown in Fig. 35 dependent up $h_x : L$ and ϕ.

Quantitatively similar results are obtained when the method, proposed by G. Schneebeli (1964) is applied. This method, however, gives slightly higher values of pressure p_a than does Eq (1). The method, based on the theory of pressure of loose materials in silos, takes into account only the local plastic equilibrium at the trench side in case of arching in the vertical plane of the side.

The problem of the correct calculation of the unit weight of suspension required to safeguard the stability of a trench side seems to have an essential practical significance. It has been proven in field research work that the average digging speed decreases as depth increases but at a much faster rate than should result from the extension of excavator working cycle alone. This seems to confirm the view expressed in our paper that the hydrostatic pressure of suspension against the trench bottom might result in the increase of shearing resistance by the excavator grab bucket. With a given trench depth, the pressure will obviously be higher, the greater the unit weight of suspension filling the trench.

Fig. 36 shows the results of measurements of the digging speed of an excavator equipped with a special grab bucket. The trench was designed for making a watertight diaphragm made of cohesive soils (Piaskowski, 1965) and had in this case an average width of 1.35 m.

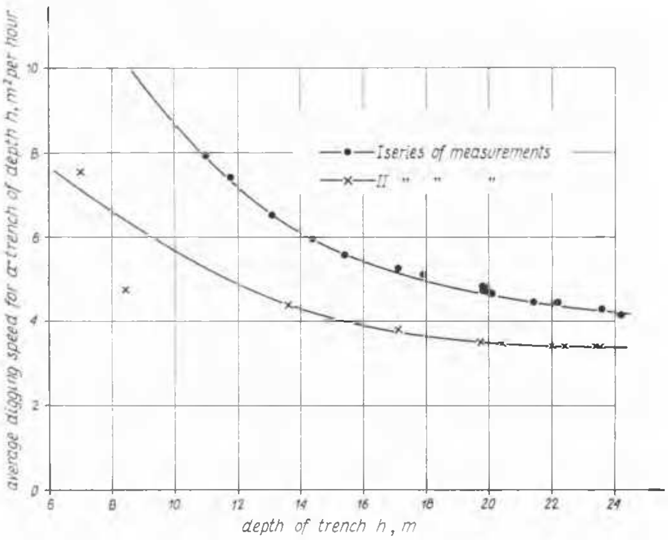

FIG. 36. Results of actual measurements of the digging speed of a trench excavator.

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Precise geodetic observations on large natural slopes with a sufficiently dense network of surface benchmarks show, as a rule, noticeable divergences from the distribution of directions and rates of landslide displacement which follow from the classical theories. These cannot be explained solely by errors of measurement. In some cases the explanation of these variations is found in the underground relief of landslide beds (Ter-Stepanian, 1960). In other cases they reflect the peculiarities of the mechanism of landslides, a matter of much deeper interest.

The author has worked out a graphical method of decomposition of vectors of the rate of depth creep which permits the recognition of different types of sliding. Joint application of this method with an analysis of landslide fissures (Ter-Stepanian, 1962) and of deformations of structures makes it possible to reveal a much more complicated mechanism of landslides as compared with the commonly known ones.

An interesting case of a multi-level landslide was observed in Sochi, on the shore of the Black Sea; it occurred along a funicular railway. The foundation of the slope was composed of layers of Eocene marls and Oligocene argillites and sandstones dipping downward parallel to the slope.

These indigenous rocks were overlain by a thick rock mass of blocks of these same argillites and sandstones. These blocks have been displaced due to huge landslides which took place in the Upper Pleistocene period. These rock masses, which reach depths up to 60 m, still continue to creep very slowly. They represent the first, lowest level of the landslide. The length of the creeping mass exceeds 900 m and the breadth reaches 500 m. The rate of secular depth of the first level was measured to be several millimeters per year.

A layer of crushed argillites is located above the blocks of argillites and sandstones. It was formed as a result of subsequent weathering and sliding of the blocks. The depth of the layer of crushed argillites reaches 20 m. Planar depth creep is developed in these masses. They represent the second level of landslide. The rate of depth creep of this level exceeds 1 cm/yr. The tunnel of the funicular passes through this section of landslide.

A colluvial sheet, with a thickness of up to 6 m, is located above the crushed argillites. Here a slow earthflow occurs. It is the third level of landslide. The slow earthflow twists on the slope and forms an insel near the upper end of the tunnel. Immediately above this exit the upper station of the funicular is located; it is founded on the creeping soil masses of the third level. The movements of this station and of the exit average 25 mm/yr, a movement equal to the difference of the rates of depth creep of the second and third levels of the landslide.

Apparently multi-level landslides, which were revealed for the first time in Sochi are not uncommon, especially on the Black Sea shore. Their mechanism is determined by the geological structure of the slope, its geometrical features, and the history of sliding.

A more detailed analysis of this case will be published later.

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