INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

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Slope Stability — General Report

Stabilité des Pentes

P. LA ROCHELLE   Professor, Université Laval, Dept. of Engineering, Québec, G1K 7P4, Canada
R.J. MARSAL   Professor, Instituto de Ingenieria, Ciudad Universitaria,
                         Apdo. 70—472, Mexico 20, D.F., Mexico

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INTRODUCTION

At the last International Conference on Soil Mechanics
and Foundation Engineering held in Tokyo in 1977, a
State-of-the-Art report on "Slopes and Excavations" was
presented by Morgenstern, Blight, Janbu and Resendiz
(1977); this subject matter had been considered so
vast and so varied that it was divided into five main
parts in terms of the nature of the soils encountered:
the normally and lightly overconsolidated clays,the
heavily overconsolidated clays, the residual soils,
the compacted soils and the mining and industrial waste.
In spite of the combined efforts of these four Authors,

the State-of-the-Art reporter conceded that it was
impossible to embrace all aspects of that theme within
the framework of the conference proceedings. For the
present Session 11 of the 10th Conference, the
Organizers have specially requested that the Reporters
do not prepare a State-of-the-Art Report, but rather
limit the content of their General Report to a review
and analysis of the papers submitted to this Confer-
ence, or presented elsewhere since the Tokyo Confer-
ence and which are relevant to the themes of this
Session.

Many meetings dealing with slope stability problems
have been held during the last four years.

- The third International Symposium on Landslides
(ISL) was organized under the initiative of the
Indian engineers and held in New Delhi just over one
year ago in 1980. In addition to the 83 papers included
in Volume 1 of the Proceedings of the Symposium, the
following State-of-the-Art reports and special lectures
were presented and are printed in Volume 2:
. Landslides in sensitive clays (B.B. Broms and
  T. Stål);

. Slope stability studies in centrifuge (R.S. Scott);

. Factors affecting the selection of shear strength para-
  meters in slope stability analysis (N.R. Morgenstern);

. Creep in natural slopes and cuttings (G. Ter-Stepa-
  nian);

. Critical evaluation of the approaches to stability
  analysis of landslides and other mass movements
  (N. Janbu);

. Use of computers for slope stability analysis (D.G.
  Fredlund);

. Instrumentation: its role in landslide prediction and
  control (M.Fukuoka);

. Electro-osmotical stabilisation of slopes (C.Veber
  and F. Hilbert);

. Geological aspects of landslides with particular
  reference to the Himalayan region (V.S. Krishnas-
  wamy).

- For the meeting of the Executive Committee of the
ISSMFE in Oaxaca in 1979, an International Symposium on
Soil Mechanics was organized by the Sociedad Méxicana
de Mecanica de Suelos. Two general reports which had
a direct bearing on slope stability problems were pre-
sented: one by N. Janbu who discussed the mechanisms of
failure in natural and artificial soil structures, and
one by N.R. Morgenstern who reviewed the geotechnical
behaviour of clay shales.

- The 6th Asian Regional Conference on Soil Mechanics
and Foundation Engineering held in Singapore in July

1979 included a session on "Slopes and excavations" with a general report prepared by P. Lumb.

- A two-day Specialty Conference on Slope Stability Problems in Urban areas was held in Toronto, Canada, in August 1980.

Taking into account the numerous papers which have appeared in the different journals, and in the proceedings and reports of international and national meetings held within the last four years, the Reporters could have easily been led to write a State-of-the-Art report if they had not made an effort to confine their review and comments to the topics directly covered by the papers presented to this session. However, in trying to do so, the Reporters are aware of the fact that some valuable papers may have been neglected in their review; nevertheless, they hope that their report will remain helpful to the attendants of Session 11 and eventually to the readers of the Conference Proceedings.

In all, 49 papers are presented to this session on slope stability; this number constitutes an increase of 50% over the 32 papers presented on the same topic in Tokyo. The four themes retained for this session are:

Theme 1: Detection and classification of potential landslide areas

Theme 2: Analysis of slope stability, including dams.

Theme 3: Factors influencing the short- and long-term stability of slopes.

Theme 4: Slide warning systems and methods of prevention of landslides.

It should be noted that for the convenience of presentation, the order proposed by the Organizing Committee for themes 2 and 3 has been reversed. The papers were divided up amongst the four themes as follows:

Theme	Number of papers
1	2
2	37
3	3
4	7

The table of papers given below includes the list of authors, titles of the papers and some keywords which may be found useful. Many of the papers allotted to theme 2, which represents the bulk of this Session, could also have been included in other themes as many of the authors giving case histories and analysis of slope failures, also comment on different aspects of the stability problem.

Each of the themes will be discussed separately and reference (underlined) will be made to papers presented to this Session, and dealing with the theme subject.

THEME 1: Detection and classification of potential landslide areas

The identification and classification of landslide areas have been the object of many recent studies (Varnes 1978, Rib and Liang 1978). This exercise for which many factors need to be taken into account, constitutes the first stage in the elaboration of geotechnical maps prepared with the purpose of defining landslide hazard zoning. Different sectors of our profession are concerned by this problem. The Commission on Landslides and other Mass Movements of the International Association of Engineering Geology (IAEG) is presently working under Dr D. Varnes on the preparation of a document giving the guidelines for landslide hazard zoning, it is understood that this document will soon be available through UNESCO (Paris).

In September 1979, the IAEG has also held in Newcastle-upon-Tyne a Symposium on "Engineering geological mapping for planning and construction in civil engineering". The numerous papers presented at the symposium originated from many countries and are published in the bulletins No. 19 to 21 of the IAEG; they give many case histories of site and regional mapping pratice which are quite valuable for engineers involved in this type of problem. During the recent years, there has been a growing interest for hazard and risk mapping, which is probably due to an increase of human activity into more uneven and critical areas; a Symposium devoted to this topic has been presented at the 1980 International Geological Congress in Paris. It is thus surprising, and unfortunate, that only two papers are presented under this theme in our session, as this theme aims at preventing disasters and losses of life which should be the ultimate social concern of the geotechnical engineers.

After the occurrence of a large landslide, described by Berntson and Lindh, which destroyed an area of the city of Tuve, the Swedish Government decided to carry out a national landslide hazard mapping program. The preliminary results of this program presently confined to 10 cities in the south west part of Sweden are given by Viberg. The existing landslides and their environment have been investigated as a basis for a mapping and classification method; their sizes, shapes and frequencies usually provide a good idea of the landslide susceptibility of a given area; this is normally the first characteristics that an engineer will examine when he is called to judge on the probable stability of a natural slope in a post-glacial clay area. The Author gives interesting graphs showing the influence of human activity and of climate on the frequency of landslides; as will be seen under theme 3, several papers in this Session emphasize the effect of rainfall on the stability of slopes. An interesting land form feature which has been stressed by the Author is the fact that the slides will often retrogress up to rock outcrops, to frictional material, or to clay ridges around old landslide scars; the same feature is often observed in Canadian sensitive clays. As expected, erosion is identified as a common cause of landslide; by increasing the stress level at the toe of a slope, it may act as a trigger to the first slide which often precedes a larger flow-slide in these clays. But, in order for the slide to retrogress, other conditions must be met, such as unfavourable soil properties, general topography, hydrogeological conditions, etc.. The slope angle has been used by the Author as a first step in the establishment of stability criteria, the other factors being the surrounding topography, infiltration and groundwater level conditions, shear strength values, active erosion and previous landslides.

The Reporters wonder, however, why Viberg has put so much emphasis on defining the frequencies and sizes of landslides and seems to have placed the item of "previous landslides" at the bottom of the list when establishing the stability criteria. There are known cases in Canadian sensitive clays where all the conditions for landslides are met according to the Author's main criteria: steep high slopes with marginal factors of safety, high sensitivity, and uncontroled human activity such as important fills on top of slopes, etc. Nevertheless, no recent or old slides are observed in these regions, while adjacent regions are very much affected by major landslides. This observation tends to show that there may be some secondary characteristics which are sometimes unidentified and play a major role in the stability. Regional history should then be taken into consideration; the Reporters would certainly evaluate the item "previous landslides" to be as important if not more, than the angle of the slope.

The establishment of zoning for landslide hazard mapping with the purpose of land management eventually requires that specific geotechnical studies be made, including soil investigations and stability analyses. This necessitates the investment of budgets which have to be adapted to the real needs. Sällfors and Tägnfors propose a way to use the available resources so as to optimize the results; they have defined five different levels of slope stability analysis starting from a general geological and soil mapping down to a research-oriented analysis including elaborate testing. The reliability of the factor of safety will be essentially a function of our knowledge of the soil and of the increasing cost of the investigation. Three avenues of improvements of the cost/benefit ratio are suggested by the Authors. A better use should be made of the experience obtained in one given area, including the empirical knowledge of soil parameters and the observations gathered from local people and contractors; this would help to address the efforts to the important points. Secondly, long series of pore pressure observations should be collected and classified over many years and for different types of terrains and topography; the Reporters agree that this type of information is badly missing in soil mechanics and would be of great help to the engineer who has to choose the worst probable conditions of water pressure to evaluate the stability of a slope. Thirdly, the Authors suggest to use more flexible programs allowing changes of data without too much cost; such a program has been developed at Chalmers University. In this respect, the Authors refer to the use of the slope stability charts proposed by Janbu (1954). The Reporters would like to emphasize the value of these charts for the engineers involved in stability problems; it is indeed unfortunate that these charts which were partly published in a discussion by Janbu (1967) were not given more publicity, as they constitute a most useful tool for preliminary slope stability analysis and could be a valuable time and money saver.

THEME 2: Analysis of slope stability, including dams

Although there have been some pessimistic comments expressed in the literature lately on the lack of improvement of our design methods and of our capacity to investigate failures, the Reporters believe that the numerous studies and case histories which are published regularly help us to progress, at a slow pace may be, but perceptibly towards a better understanding of the soil behaviour and of the potential and limitations of our methods of analysis. Obviously we have not, and we will probably never reach a point where good engineering judgment is not essential anymore in decision making; nevertheless, it is believed that the engineering analysis of stability problems has become a valuable tool in the hands of a design engineer.

For the analysis of the stability of natural or man-made slopes, the three main questions which have been discussed for some years and are receiving more and more precise answers are:

- How accurate are the methods of analysis?

- Should the slope be studied in terms of total or effective stresses?

- What are the strength and pore pressure values acting in the failure zone at failure?

In the light of the results of many studies published during the last four years, the Reporters would like to suggest some answers to these questions in the hope that it might stimulate discussion during the session.

2.1 Accuracy of the methods of analysis

The limit equilibrium methods of analysis are widely used by design engineers when dealing with problems of stability of slopes, either natural or cut slopes, or slopes of embankment on rigid or on soft foundations. Many such methods are available in practice and the most common ones call on the principle of slices to allow taking into account the complex geometry and the variable soil and pore pressure conditions of a given problem. The accuracy of these methods are often questioned although the errors resulting from uncertainties on the geometry, pore pressures and values of mobilized shear strength are in most cases more important.

The methods most commonly used in practice are:

- The ordinary method of slices of Fellenius
- The modified or simplified Bishop's method
- Janbu's generalized method of slices
- Morgenstern - Price's method
- Spencer's method

The first two methods do not satisfy all the moment and force equilibrium equations and can only accomodate circular slip surfaces, while the three last methods satisfy all equilibrium equations and may be used to calculate the factor of safety along any shape of slip surfaces. These different methods, plus other more elaborate ones, were compared by Fredlund and Krahn (1977) and by Duncan and Wright (1980) in an attempt to evaluate their respective accuracy and reliability. The results which came out of these two separate studies and of other papers presented to this Session, are very similar and the conclusions may be summarized as follows:

- The equilibrium methods which satisfy all conditions of equilibrium give accurate results which do not differ by more than ± 5% from the "correct" answer (Duncan & Wright 1980); these Authors have considered that the "correct" answer is the result obtained by the log spiral method because it satisfies all conditions of equilibrium and is not a method of slices. Bishop's simplified method is also equally accurate (Wright et al. 1973). This conclusion is further substantiated in a paper by Fredlund et al. where the Authors present a general limit equilibrium method of slices (GLE method) and show how the above-mentioned methods are special cases of the general formulation; the different methods are compared to the GLE method and it can be inferred that the three non-circular methods give equivalent results. It should be noted, however, that Resendiz (1974) has challenged the conclusions of Wright et al. (1973) by comparing the results of the analysis performed by means of limit equilibrium methods, to those obtained by a F.E. analysis using the hyperbolic stress-strain representation proposed by Kondner (1963); the safety factors were 10% to 30% higher when computed by the F.E. method, the larger difference being obtained on flatter slopes. The differences obtained by this Author are so important that they may need to be qualified by further discussion. The Reporters believe that, in cases where failure occurs, the FEM is not the best method to evaluate stability conditions.

- The design engineer can thus choose, amongst the three non-circular and the Bishop's simplified methods the method of stability analysis which will fit his needs, and be confident of obtaining a correct answer in terms of the mechanics of the problem (Duncan & Wright 1980). Kisiel et al. have used the theory of characteristics to compute the upper bound value of the factor of safety and, for the case studied, they have reached the conclusion that the methods of Bishop and Janbu give values

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2	Mikasa, M. et al.	Stability test of a rockfill dam by centrifuge.	Model preparation, test method, displacements, water filling.
2	Mulder, G. et al.	Some properties of sandasphalt as it is used in hydraulic structures.	Stress-strain relationships, dutch cell, deformations, slope erosion, lining.
2,3	Nagarkar, P.K.	Failures of a monozone earth dam of expansive clay.	Old dam, 30 m high, slope failures, stability analysis, desiccation and cracking.
2	Papadopoulos, B.P. Anagnostopoulos, A.G.	Factors affecting the stability of slopes.	Limit equilibrium, parabolic vs circular cylindrical slips, anisotropy, non-homogeneity.
2	Pastor, J.	Analyse limite et stabilité des fouilles	Uniform surcharge, analytical and numerical analysis, plane deformation.
2,3	Runesson, K. et al.	On the behaviour of natural slopes in clay.	Silt layers, pore pressure, combined analysis of deformation and instability, critical state.
2	Salençon, J. Tristán-López, A.	Stabilité d'ouvrages en sols cohérents anisotropes.	Bishop's formula, anisotropic strength criteria, failure coefficient, static and cinematic approaches; tables.
2	Sotiropoulos, E. Cavounidis, S.	Irregular stress-strain and progressive failure.	Overconsolidated marl, fissure material, peak strength, progressive failure.
2,3	Tsytovich, N.A. Ter-Martirosyan, Z.G.	Short-term and long-term stability of slopes.	Visco-plastic flow, theory, case history.
2	Tinoco, F.H.	Analysis of a cutting in overconsolidated clay.	Mechanism of failure, pore pressure, shear strength, stability analysis.
2	Vallejo, L.E.	Stability analysis of mud flows on natural slopes.	Mudflow structure, mechanics of movement, stability analysis.
2,3	Vallejo, L.E. Edil, T.B.	Stability of thawing slopes: field and theoretical investigations.	Depth of thaw, field measurements, type of material, limit equilibrium.
2	Wackernagel, A.	Long term behaviour of dam in a period of 108 years.	Material, settlement observations, stability analysis.
2,3	Widdis, T.F. Clapham, H.G.	End of contruction failure of a slope in a stiff boulder clay at Glenhove.	Excavation, wedge failure, inclined clayey seam, laboratory tests, stability analysis.
2,3	Wiseman, G. et al.	Stability of a heterogeneous sandy coastal cliff.	Triaxial tests, soil strength parameters, design curves, comparison with field information.
3	Brand, E.W.	Some thoughts on rain-induced slope failures.	Infiltration of water, suction changes, modes of failures, laboratory tests, field measurements.
3	Jappelli, R. Musso, A.	Slope response to reservoir water level fluctuations.	Weathered rock, residual strength, mechanism of failure, pore pressure.
3	Massarsch, K.R. Broms, B.B.	Pile driving in clay slopes.	Induced pore pressure, disturbance, displacements, induced slope failure.
4	Berntson, J. Lindh, B.G.	The Tuve landslide, a photo documentation.	Rehabilitation of the area, excavation and rockfills, sheet piling, limestone columns.
4,3	Cartier, G. et al.	Glissement de talus ferroviaires: surveillance et prévention.	Slope movements, field observation, corrective measures.
4	Huder, J. Duerst, R.	Safety considerations with respect to slope stability for the cut at the North Portal of the Born tunnel (Switzerland).	Pile wall, anchor rods, monitoring system, pore pressure, stability analysis.
4,3,2	Mitchell, R.J. Williams, D.R.	Induced failure of an instrumented clay slope.	Induced pore pressures, piezometers, extensometers, deflectometers and distortion gauges, warning devices, sensitive clay, displacements.
4	Nonveiller, E.	Effect of bored horizontal drains on slope stability.	Seepage to drains, consolidation; length, spacing and location of drains; time vs safety gain.
4	Shields, D.H. Harrington, E.J.	Measurements of slope movements with a simple camera.	Simplified photogrammetry equipment, computations, monitoring.
4	Viggiani, C.	Ultimate lateral load on piles used to stabilize landslides.	Failure modes, rigid and hinged piles, comparison with full scale tests, slope stabilization.

* The first theme number relates to the main subject of the paper, and the other numbers, in italic, indicate other theme subjects referred to.

which are only slightly lower than that obtained by the more elaborate plastic analysis.

- When it comes to the choice of the method, the experience of the Reporters is that Bishop's and Janbu's methods are easy to program and far cheaper to use than the Morgenstern-Price method. According to Fredlund and Krahn (1977), the Spencer and the Morgenstern-Price methods are at least six times as costly to run as Bishop's or Janbu's methods.

The engineer will then have the choice between a circular or a non-circular slip surface method in the light of the shape of the anticipated slip surface, or of the observed failure surface. The shape of these surfaces will greatly depend on the geometry of the problem and on the stratigraphy of the soil profile. A shear failure in a steep slope of homogeneous soil will usually develop along circular surfaces; however, if weak layers or seams are present in the soil profile, the failure surface will have a tendency to follow as much as it is kinematically possible these planes of weakness and to adopt a non-circular shape. In these cases, if the stability analysis is made along circular surfaces, thus ignoring the detrimental influence of the weak strata, the factor of safety obtained may be overestimated as it may not correspond to the most critical surface.

One of the shortcomings of the non-circular surface method of analysis rests in the fact that the computer programs do not include an automatic search for the most critical surface. Although this subroutine is easy to include in a program using the equation of a circular surface, this is not so for a non-circular surface for which the computer program leaves it to the user to choose by trials the most critical surface; this requires a good deal of judgment and is obviously a time-consuming operation. Fortunately, a paper submitted to this Conference by Celestino & Duncan explains a simple numerical method whereby a search for a critical non-circular surface can be automatically performed by iteration; a subroutine which can be introduced into a computer program of a non-circular method has been developed and is graciously made available by the Authors without cost. Two examples are given in the paper to illustrate the benefit of this method.

In spite of their apparently good performance, these methods of slope stability analysis remain the object of many criticisms resulting mainly from the assumptions on which they are based. It may be useful to review them in order to better understand the limitations of these analyses:

- The normal stress on the shear surface depends on the static equilibrium of the slice and is quite independent of the stress-strain characteristics of the soil. The distribution of the normal stresses along the circular surface as determined by means of Bishop's simplified method has been compared in the case of a steep slope to those obtained for the linear elastic stress distribution and the difference was found to be quite large (La Rochelle 1960), but it diminishes as the slope becomes flatter (Wright et al. 1973). If these elastic stress distributions were representative of the real conditions on the failure surface at failure, which is not the case, the obvious implication would be that for the effective stress analysis, where the strength depends on the effective normal stress acting on the failure surface, the calculated strength at any point on the surface could differ appreciably from the strength mobilized in the field. It should be realized, however, that the methods for comparison used by different authors to substantiate this point (in the

present case the linear elastic distribution which certainly does not model the soil behaviour) cannot really be applied to calculate the normal stresses on the failure plane at failure. For failure conditions, no techniques are presently available which would allow us to model exactly the behaviour of the soil in order to establish an exact quantitative basis of comparison.

- In all equilibrium methods, an assumption is implicitly made that the soil is rigid perfectly plastic, which means that the strength mobilized may remain constant for large strains. As mentioned by Duncan and Wright (1980) these methods are not applicable to brittle or "strain softening" materials such as stiff-fissured clays and shales; in these cases a value of strength lower than the peak must be chosen. This is the procedure which is followed when either the residual, the fully-softened or the large strain strengths are used in a stability analysis.

- The equilibrium methods of the slope stability analysis most widely used involve the assumption that the factor of safety is the same for all the slices. Wright et al. (1973) have calculated the factor of safety along a shear surface by means of a finite element analysis for cases where Bishop's method was giving a factor of safety of 1.0. The results have shown that the local factors of safety obtained by FEM with linear elasticity were different from 1.0 and even appreciably lower than 1.0 in some parts of the failure surface. One should indeed question the validity of methods of analysis which incorporate in the average such low factors of safety. Nevertheless, according to these Authors, the magnitude of the differences in the average values of the factors of safety obtained by both methods for slopes of 1.5:1, or flatter, varies from zero to only 4.5%. Considering the large spectrum of soil properties and of slope angles embraced in the study, and also the differences in the soil behaviours postulated in both the FEM and the equilibrium method, an error of the factor of safety of less than 4.5% resulting from this assumption is surprisingly small and quite tolerable.

- Three other arguments questioning the validity of these methods are presented in a critical paper by Tavenas et al. (1980). The first one follows from the definition of the factor of safety normally introduced in the methods of stability analysis. The mobilized shear strength is usually defined by:

$$\tau = \frac{c'}{F} + \frac{\sigma' \tan \phi'}{F} \qquad (1)$$

where σ' and τ represent the applied normal and shear stresses on the potential failure surface, c' and tan ϕ' are the cohesion and the friction coefficient, and F is the factor of safety. Janbu (1973) has suggested a simplified form of Coulomb's equation which can be written in terms of the "attraction" $a = c' \cotan \phi'$, so that equation (1) giving the mobilized strength becomes:

$$\tau = (a + \sigma') \frac{\tan \phi'}{F} = (a + \sigma') \tan \rho \qquad (2)$$

or

$$F = \frac{\tan \phi'}{\tan \rho} \qquad (3)$$

This formulation illustrates more clearly the problem resulting from the definition of the safety factor. For a deposit limited by a horizontal surface, the safety factor should be $F \to \infty$. According to (3), this implies that tan ρ must be equal to zero, which is a condition only achieved in an isotropic state of stress. In natural deposits where an anisotropic state of stress given by

$$(\sigma_h'/\sigma_v') = (\sigma_3'/\sigma_1') = K_o$$

has generally developed, a factor of safety much less than infinity, and of the order of 1.6 to 1.7, is calculated on the basis of (3). As concluded by Tavenas et al. (1980), this raises the question of the relevance of equations (1) or (3) for describing the stability conditions in natural anisotropic soils. This is probably the most severe criticism formulated against our methods of stability analysis.

- The second argument concerns the indiscriminate use of the Mohr-Coulomb criterion to represent the failure conditions in a natural clay. According to Tavenas and Leroueil (1977), the strength of the natural clays is entirely represented (Fig. 1) by its limit state surface
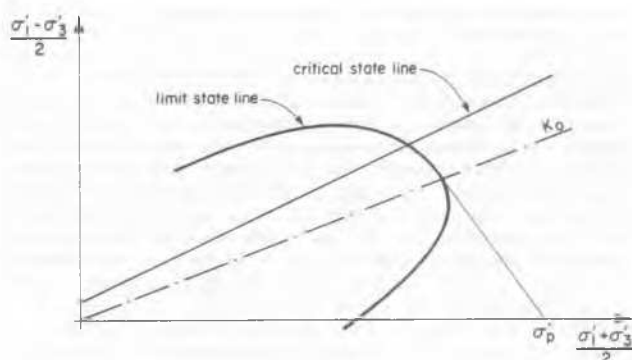

Fig. l——TYPICAL LIMIT STATE CURVE FOR NATURAL CLAYS.
(from Tavenas et Leroueil,1979)

and its critical state line which is identical to the Mohr-Coulomb criterion. When the effective stress path followed by a clay element reaches the limit state curve, which is time and strain-rate dependent, the clay yields and the ensuing stress path aims towards the critical state corresponding to the void ratio of the clay. It can then be inferred that the direct use of the Coulomb equation relating the strength to the overburden effective stress as it is done in the methods of stability analysis is questionable. However, as mentioned by the Authors, in the case of natural slopes where,failure can develop slowly, it is possible that the clay has a partially drained response and that its void ratio has time to adjust to the ambiant effective stresses, in which case the use of the Mohr-Coulomb criterion would possibly be less erroneous than in other cases of stability. Tavenas et al. (1980) further mention that the only case where the predominant effect of void ratio could be neglected is in the reactivation of an old slide along a well-defined existing failure surface where the Mohr-Coulomb criterion would become applicable

- The third argument proposed by Tavenas et al. (1980) follows a discussion by Bishop (1974) to the effect that the effective stress paths found in most practical cases are appreciably different from those implied in all the methods of stability analysis; as a result, the degree of mobilization of the shear strength will differ from that given by equation (1). Tavenas et al. (1980) have well illustrated this point for cases of loading and unloading of a clay mass and have arrived finally at a very negative conclusion on the validity of our methods of stability analysis.

A similar argument is presented in this session by Juárez-Badillo who has based his reasoning on direct shear stress paths and has also concluded that the factors of safety as we introduce them in the method of analysis lead to some error.

In the light of all these formulated criticisms, it is puzzling to realize that, in spite of their apparent defects, our methods of stability analysis yield results which seem to reflect reality. It appears that we are getting the right answer but for the wrong reason. Is it because our errors are cancelling each other? Or would it be that the soil does not behave at all in nature as we think it should? In the opinion of the Reporters, the problem is not really solved; but in the meantime, the fact remains that we have a proven engineering tool which allows us to carry out with a certain degree of confidence some stability analyses which are quite valuable for design purposes although the results obtained must not preclude experience and good engineering judgment from prevailing in decision making.

2.2. Effective versus total stress analysis

There has been more and more evidence accumulated in the last few years to the effect that stability analyses are more reliable when performed in terms of effective stresses than total stresses. The difficulties encountered with the total stress or $\phi=0$ analysis, result not so much from the method of analysis in itself which, as mentioned by Tavenas et al. (1980), presents probably fewer sources of errors than the effective stress analysis, but are due to the fact that there are probably as many values of undrained shear strength as there are methods to measure it. For many years, the $\phi=0$ analysis has been associated with the short-term stability cases, and the effective stress, to the long-term. However, this association of concepts has been challenged more and more by case studies which show that the $\phi=0$ analysis was generally unreliable in short-term cases and that, in order to obtain satisfactory results, some empirical correction factors have to be applied to the undrained shear strength used in the analyses (Bjerrum 1972).

It is now recognized that, even in the case of excavated slopes which represent the ideal short-term case, the effective stress analysis may be a better tool for design than the $\phi=0$ analysis provided that good measurements of pore pressures are available. The failure at the Kimola floating canal has shown clearly that the $\phi=0$ analysis was most unreliable in that case (Kankare 1969), and that the effective stress analysis gave good results with the measured pore pressures; this was confirmed by a study performed by Kenney & Uddin (1974) who have also shown that the stability of the excavated slope started to decrease immediately following completion of excavation as the measured pore pressure which had dropped during excavation started to increase. It should be mentioned, however, that this conclusion is presently challenged by Leonard (1980) who imputes the failure to the presence of a thin weak seam of clay in the foundation of the slope. Juárez-Badillo also expresses concern that discontinuities are often neglected to the benefit of simpler geometrical surfaces. The Reporters agree that stratigraphical features and anomalies should be identified and taken into account in the analysis; however, they remain intrigued by the fact that in many cases the failure has been back-analysed with success although the presence of a significant discontinuity was not taken into account in the analysis. The case of the Kimola canal (Kenney & Uddin 1974) discussed above and the failure of the riverbanks preceding the dramatic flow-slide of Saint-Jean-Vianney (La Rochelle 1975) are two such examples, which lead us to believe that in some instances the presence of stratigraphical anomalies may only be the small difference between stability or failure conditions. In these two cases, the analyses have indicated that the soil masses were at the limit of equilibrium; it is then conceivable that the stratigraphical anomalies had only made the failure possible at that moment.

As shown by Kenney & Uddin (1974) the drop of pore pressure due to an excavation will dissipate with time. Skempton (1977) has observed that the equilibration of the pore pressure in cuts in London clay could take many years to occur due to the low permeability of that clay. However, Leonard (1979) has suggested that in the cases mentioned by Skempton (1977), the delay in the failure of the cuts could be explained by the fact that a structural breakdown occurs and continually generates negative excess pore pressures. This suggestion is substantiated by the fact that there is a very consistent correlation between the slope angle and the time to failure; the structural breakdown being more rapid in steeper slopes, they fail sooner. As a result, the stability will decrease with time but the rate of decrease does not depend exclusively on the rate of equilibration of pore pressure but also on the rate of creep which will affect the limit state curve as discussed by Tavenas and Leroueil (1981).

In the light of the above discussion on the different factors which may influence the stability of a clay mass, it becomes obvious that the evolution of all these factors can be followed only by means of the effective stress approach.

As will be discussed later, many authors of papers presented at this conference arrive at the same conclusion, even for cases of short-term stability. However, it will be seen that in some of the papers, the undrained shear strength approach is still preferred even for the stability analysis of natural slopes.

2.3 Strength and pore pressure at failure

The other question which needs to be considered concerns the choice of the strength parameters and of the pore pressure values acting in the zone of failure at failure. This question has been discussed at length by different authors and will also probably be dealt with at some other session of this conference.

2.3.1 Effective strength parameters

The choice of the effective strength parameters for overconsolidated fissured clays has been studied especially by Skempton (see Skempton (1977) and his previous papers). This choice is particularly significant in the case where the soil exhibits a strain-softening behaviour for which three levels of strength must be considered.

First, the "peak" strength which is often very well defined in the laboratory tests, is seldom mobilized in nature; this is confirmed by the conclusions of some of the papers presented at this Session. There are many reasons why the measured peak strength should not be relied on in practice: the presence of fissures, the effects of softening, the anisotropy, the rate effect, and possibly the progressive failure, are all factors which will tend to erase the peak in nature. In fact, one wonders if the peak strength is not more a property of the test rather than a property of the soil. It is interesting to note at this point that the same arguments apply against the use of peak strength in soft sensitive clays, except that the softening in these clays does not usually result from an increase in water content but rather from a destructuration due to shear.

In studying the first-time slides in the Brown clay, Skempton (1977) has come to the conclusion that in these overconsolidated fissured clays, the displacement at failure was sufficiently large to reduce the strength to its "fully-softened" value, which he also called "critical state" strength. This value can be determined by measuring the strength of remoulded,

normally consolidated clay. This is the strength level which was found to be mobilized in the first-time slides in London clay. For the sensitive clays, this second level of strength has been referred to under different names: post-peak strength (Lo 1972), strength at large strain (SALS) (La Rochelle et al. 1974), or residual strength (Lefebvre & La Rochelle 1974). However, the concept represented by these three different names is the same and corresponds to a strength value determined on a stress-strain curve obtained by triaxial test, past the peak, at about 5% to 15% axial deformation where the strength shows a tendency to stabilize with increasing strain. This is the level of strength that was found to be mobilized in the "first slide" in sensitive clays, which usually precedes a larger flow-slide (Lefebvre and La Rochelle 1974).

The third level of strength mobilization in terms of effective stresses is the residual strength which implies a very large movement along thin zones, or failure planes, with a resulting parallel re-orientation of the clay particles. Skempton (1977) states that for London clay the residual strength is not relevant to the first-time slides and that it applies only to post-slip movements or re-activated slides as observed in many clay shales in the world (Marsal 1979; Cancelli and others). In sensitive clays, part of the confusion in the names mentioned above comes from the fact that the residual strength, and the strength at large strain cannot be distinguished very clearly. Lefebvre & La Rochelle (1974) have reported reversal shear box test results made on sensitive clays which give the same strength parameters as the strength at large strain; this is the reason why these Authors have used the term residual. However, if the concept of residual strength implies a re-orientation of the clay particles, as it seems to be the case in overconsolidated clay or clay shales, this concept is not relevant to the behaviour of the sensitive clays and should not be used in this context; the term strength at large strain (SALS) should be preferred.

2.3.2 Undrained strength values

The choice of the "correct" value of undrained strength is still more complex as there are many different values of undrained strength which depend mostly on the methods of measurement. The abundant data available in the literature show that the undrained strength values are different if measured by unconfined, or unconsolidated-undrained, or isotropically or anisotropically consolidated undrained triaxial tests, by direct simple shear test, or by any field method as the vane, pressiometer, etc.. It is only at random that two tests will give the same results. Of all the field tests, the vane is the most commonly used because of its simplicity and low cost; but the difficulties associated with the field vane strength are well publicized and this strength is not used anymore without referring to Bjerrum's correction factors or to some other factors based on personal experience. Even with the use of such correction factors the outcome has been sometimes erratic, the reason being that the vane test is no more than an index test and that its application remains essentially empirical. In an attempt to propose more realistic values of undrained strength, La Rochelle et al. (1974) have suggested that the "undrained strength at large strain" (USALS) defined in unconsolidated-undrained (UU) triaxial tests would be more relevant; Trak et al. (1980) have later found that, for Canadian sensitive clays, this strength is equivalent to the value of $0.22\ \sigma_p'$ suggested by Mesri (1975) where σ_p' is the preconsolidation overburden pressure. These new approaches have been used successfully in the analysis of the stability of embankments on soft clay foundations. It would seem, however, that the "USALS" or the

0.22 σ_p' value could possibly be the strength mobilized
behind unstable slopes (Tavenas et al. 1980). In the case
of many Norwegian landslides studied by Aas, the un-
drained strength value mobilized at failure seems to be
close to 0.22 σ_p' as discussed below. Larsson (1980) has
shown that the undrained strength normalized at about
0.22 σ_p' is also applicable to soft inorganic Scandinavi-
an clays. Trak et al. (1980) and Larsson (1980) both
mention that this empirical relation is not verified in
organic soils.

2.3.3 Pore pressure

In the following section dealing with the factors
influencing the stability of slopes, references will
be made to the importance of the intensity of rainfall
as a factor of instability of slopes. It is evident
that the value of the pore pressure acting in the zone
of potential failure is of the uppermost importance to
evaluate correctly the stability. Unfortunately, this
factor is not always treated with the attention that
it deserves; much effort is sometimes devoted to define
and to analyse in detail the soil characteristics in
the soil profile, but when it comes to determining the
pore pressures, the measurements are often scarce or
the chosen values are based on some assumptions.
Neither the intensity nor the network of pore pressure
should be neglected.

In a paper presented to a different session of this
Conference by Leroueil & Tavenas, it is shown that an
error of ±0.5 m in the estimated position of the water
table could result in an important error in the calcu-
lated factor of safety; for a typical clay slope with
a height of 8 m and an inclination of 23º, the factor
of safety would vary from 0.99 to 1.33. But it is not
only the depth of the water table which is important
but also the general pattern of the groundwater regime
in the clay slope; this was studied by Lafleur &
Lefebvre (1980) who have shown how upward gradients
can develop at the toe of the slopes, phenomenon which
has been observed to occur in areas where the sensi-
tive clay slopes are notoriously unstable (La Rochelle
et al. 1970).

The use of r_u coefficients may be justified for
preliminary studies in areas where sufficient experience
has been accumulated; however, when a judgment has to be
made on the stability of one given slope of some impor-
tance, measurements should be made to evaluate not only
the depth of the water table but also the direction of
the gradient in the cross section of the slope.

2.4 Comments on papers

In the following section, brief comments will be made
on the papers which have not been referred to in the
above remarks and which deal with the subjects relevant
to theme 2:

- Methods of analysis   - Earthflows
- Cuttings   - Creep movements
- Natural slopes

2.4.1 Methods of analysis

Following a series of short term failures which occurred
in the Fiumicino area during the construction of a net-
work of trenches, Cavalera & Scarpelli have undertaken
a theoretical study to evaluate the effect of the
strength anisotropy of the clay on the stability ana-
lyses. The experimental study of the failures has shown
that the most critical failure surfaces were in good
agreement with the observed movements; the values of the
factor of safety based on the uncorrected vane strength
were somewhat higher than 1.0 which seems to agree with
the curves proposed by Bjerrum (1973). However, noting

that the evaluation of the anisotropy effects may be
influenced by the experimental procedure, the Authors
have worked out a theoretical model to show how and to
what extent the anisotropy effect may influence the
stability analysis. The pore pressure increment was
expressed in terms of the first and second invariants
of the stress increments according to Henkel's expres-
sion; in the plane strain case the soil was assumed to
be elastic in order to evaluate the intermediate stress.
The equilibrium equations and the failure criterion
used by the Authors proved that the equations descri-
bing the state of stress in a cohesive anisotropic
dishomogeneous medium is of the hyperbolic type. The
solutions of the differential equations obtained show
that as the slope angle decreases, the safety factors
given by the slip circle analysis are increasingly
higher when compared to a lower bound solution which
is assumed to be exact. The Authors state in their con-
clusion that "an effective stress analysis, taking into
account the real distribution of the pore pressures
that arise at failure seems to be more appropriate for
dealing with the anisotropy of clays and its effect
on stability problems". If the Authors had any avail-
able data to substantiate that statement, they would
be quite welcome.

Using the principle of limit equilibrium, Papadopoulos
& Anagnostopoulos have computed the stability numbers
($c/\gamma H$) for slopes on cohesive soils, assuming para-
bolic slip surfaces. Three cases are analyzed, namely
1) isotropic soil (c = const); 2) anisotropic soil
(c_{max}/c_{min} = K) and 3) isotropic soil having a
cohesion that increases with depth (c = $s_u z/H$, where
s_u = undrained cohesion at depth H, z = depth,
H = height of slope). The Authors compare the results
thus obtained with the stability numbers for circular
slip surfaces and conclude that: 1) for isotropic
soils and smooth slopes, the circular arc yields
lower values of the safety factor; 2) the stability
numbers are greater in the case of parabolic slip
surfaces for steep slopes and high values of K, and
3) the parabolic arc surfaces are more critical than
the circular ones when the cohesion increases with
depth. Also the influence of the frictional compo-
nent is investigated in the above case studies. The
Reporters suggest that before going into a more
elaborate method of analysis, it would be wise to
apply it to well documented case histories available
in the literature and to find out whether or not the
parabolic assumption is better than the circular arc
proposition.

The influence of anisotropy on slope stability has also
been considered by Salençon and Tristán-López who are
proposing a rigorous method to define some coefficients
of stability in cohesive anisotropic soils. The Authors
give charts which allow the user to obtain, for a simple
idealized geometry, the "failure coefficients" required
to determine the stability. In order to understand the
basis of that method, the readers must refer to previous
papers of the senior Author. Unfortunately, the charts
are based on the K_2 coefficient proposed by Bishop
(1966), which is the ratio of the undrained strength
C(45) measured on a triaxial specimen cut at 45º from
the field vertical, divided by the undrained vertical
strength C_v. It is now well recognized that the C(45)
strength measured by increasing the total principal
stress inclined at 45º from the effective principal
stress applied in the field during consolidation, has
no practical significance; this stress system usually
causes the sample to buckle during compression (Pagano
& Halpin 1968, Saada 1970).

Federovsky et al. propose three methods of stability
analysis to solve problems of homogeneous and hetero-

geneous slopes, in two and three dimensions, one of them based on the Coulomb's law and Kötter's equations, and the other two, on the limit equilibrium of the potentially sliding mass. The treatment of this matter is theoretical; little information if any, is given to prove the merits of the above mentioned methods over the traditional ones applied in geotechnical practice.

The pore pressure changes are known to be a major factor in the stability of natural slopes. Their influence may be still more important in the case of stratified soils where more permeable silty seams can transmit the pressures more rapidly. Starting from this premise, Runesson et al. have made a study of a clay mass by means of a finite element method analysis where special attention was paid to the influence of a silt seam embedded in the clay. Taking into account the observed behaviour of the Swedish clays, including creep, the Authors have chosen an elastic-viscoplastic-plastic model. This model is based on the limit and critical states and takes into account the creep and the anisotropy of clay; the description of the model is very succinct so that it is indeed difficult to judge its implications. Neither is it quite clear what the Authors mean by the stabilising effect resulting from an increase of pore pressure until the "dry side" of the locus is obtained; it is not at all evident that this is the case in terms of failure or deformation. Two case studies are presented including the Tuve landslide; in that latter case, the factor of safety β is defined in a way which is difficult to visualize and would also need some clarifications.

To increase the allowable (code) surcharges on top and near the edge of cuttings and trenches, Bilz & Brödel propose to account for the three-dimensional effect of a limited slide in the conventional stability analysis. Based on the three-dimensional model developed by Baligh and Azzouz (1975) and typical design parameters, the Authors discuss some of the results obtained through computations for different heights, lengths and slope inclinations of the cuts, as well as for selected values and dimensions of the surcharge. The increase of stability due to the three-dimensional geometry depends mainly on the soil cohesion and is independent of the friction and slope inclination; the resulting increase of the factor of safety is rated at about 20 percent. These results are valuable for the design of excavations in urban and industrial areas where working space is limited.

Azzouz et al. are proposing one more refinement in the method of analysis of an embankment on soft soil with the purpose of taking into account the end effect of a failure surface which in the circular arc analysis is assumed to have an infinite length, whereas in practical cases, it develops along a finite length. The Authors are using both the uncorrected field vane strength and the SHANSEP (Stress History and Normalized Soil Engineering Properties) strength profile; they are also adding strength anisotropy and strain compatability effects in their analysis. In doing so, the Authors imply that the structure of the clay is well preserved at high stresses and that the "peak strength" value is obtained in the direct simple shear apparatus; two implications on which the Reporters entertain some serious doubts. Trak et al. (1980) have shown that the normalized strengths determined by Ladd and Foott (1974) on five clays are equivalent to 0.22 σ_p' for normally or lightly overconsolidated clays which in turn is equal to the values of undrained strength at large strain (USALS) as suggested by La Rochelle et al. (1974) ; this seems to eliminate the possibility that the peak strength value be taken into account in the SHANSEP method and it excludes the necessity to introduce a strain compatibility effect. The Reporters are also puzzled by the fact that, of the four cases studied by the Authors, the embankment where the

failure surface was found by the Authors to be surprisingly long shows the largest end effect.

Sotiropoulos and Cavounidis give one more example where the peak strength cannot be relied upon in stability analysis. They have made laboratory tests on samples of an over-consolidated fissured blue marl which was identified as the stratum in which most of the observed slides originated. The stress-strain curves indicated a typical strain-softening behaviour; however, a certain number (6 to 20%) of the tests did not reach the peak but failed with a "trunkated peak" which pointed out the presence of some discontinuities. The Authors quite rightly suggest that the presence of fissures is the most probable explanation. The Reporters would like to suggest that, before getting involved in trying to explain the field behaviour by a progressive failure approach, they should rather investigate the scale effect of their testing which were made on small diameter (37 mm) samples. Rowe (1972) has stressed the importance of using large samples in fissured materials; this was also emphasized by Lo (1970) who has shown that, the operational strength of such fissured materials in the field could be considerably smaller than the strength values obtained on small samples in the laboratory.

Foerster and Georgi suggest the use of the finite element method and of a general constitutive equation for solving time-dependent stress-strain and stability problems. They had to idealize the strain-softening behaviour which no one has yet succeeded to model exactly with the FEM. A certain number of assumptions were used for the behaviour of the material. In the example illustrated in figure 7 of the Authors' paper, for which linear elasticity is used, the first elements to fail are located at the surface of the slope at about mid-height. This seems to disagree with the results obtained by different authors using similar conditions of linear elasticity who observed that a plactic zone is first formed at the toe of steep slopes, or at a certain depth below the surface of the slope for flatter slopes (La Rochelle 1960; Yamanouchi et al. 1978; Phukan et al. 1970). Field observations have also confirmed that bulging starts at the toe of a slope in cohesive material before failure actually occurs (Skempton & La Rochelle 1965).

Based on the limit analysis, Pastor investigates the stability of uniformly loaded, homogeneous slopes of infinite extent assuming that the soil behaves according to the Tresca or Coulomb materials. The Author applies to the theoretical results to a cylindrical excavation and compares the ratios γH/c obtained with the Tresca and Von Mises criteria. The results are very valuable from a fundamental point of view, provided that we accept the premise that the Tresca and Von Mises criteria are applicable to soils; Bishop (1966) has shown that these criteria lead to a very substantial overestimate of the strength in soils.

Acevedo et al. present a statistical approach to perform what the Authors have called a retrogressive analysis, and that the Reporters understand to be a back-analysis of a slope. They claim that their method will allow a more rational back-analysis of a slope taking into account the natural scatter of the c, φ parameters, which are understood to be the effective stress parameters. The Reporters must admit that after having read this paper, they would not know how to apply the suggested method and therefore cannot comment on the merits of it.

2.4.2 Cuttings

Two papers in this Session report on effective stress analyses of cuttings in overconsolidated clay.

Tinoco presents a method attempting to include the effect of pore pressure equalization and softening in the analysis of stability of a cut. The Author uses a limit equilibrium method where the swelling process is represented by an horizontal force acting at the base of the slice and which is proportional to the effective overburden weight of the soil before excavation; the shear strength of the stiff fissured clay is based on some modified Hvorslev parameters which the Author has proposed in an other paper presented to this Conference. The limiting value of r_u just after excavation is calculated by means of the equilibrium equations where the coefficient of swelling is taken into account. The case of the Northolt cutting is used as an example which shows how the increase in pore pressure, from negative values after excavation to positive values after pore pressure equalization, had reduced the effective cohesive component and the swelling force. The Author concludes that the distribution and magnitude of pore pressures and their variation with time are essential for the evaluation of the stability of cuttings in overconsolidated stiff clays if effective stress analysis is to be used.

An end-of-construction failure of a slope in stiff Boulder clay is of a rare occurrence. In the case reported on by Widdis & Clapham, the cause of failure could be easily traced to the existence of a smooth pre-existing shear plane in an inclined layer of silty clay some 1 m from the base of the excavation; this seemed to be an ideal case for using residual strength parameters to analyse the stability. The excavation was made in a natural slope and the section of the excavation included a step in its upper part to act as an interceptor drain. Tension cracks formed at the top of the slope and a regressive movement took place in subsequent weeks. The form of the slide mass was well defined as a triangular wedge sliding on an inclined plane. The analysis was then reduced to study the equilibrium of the wedge sliding along the clay seam with some water pressure in the crack behind the wedge. In this case, the calculated factor of safety depends essentially on the height of water in the crack and on the assumed shear strength. The Authors have tried many such assumptions and they have concluded that the analysis using the residual strength parameters in the clay seam conformed well to the observed mode of failure, whereas those in terms of total or peak effective strengths did not.

When discussing the problem of cut slopes in residual soils, Barata and Danziger are more concerned by the necessity to insure that the movement will be limited to a value which is technically, aesthetically and psychologically tolerable; this problem seems to be critical in large area of Brazil where 50 to 80 m high cuts are built in residual terrains of weathered rock. These slopes may be the seat of large displacements without really failing, up to a point where some discontinuities, such as fissures and steps, can be observed to form at the surface; the "separations" or steps resulting from shear displacements are especially objectionable. The Authors describe how these "separations" form and increase as the excavation of the cut proceeds, and they define the failure as that state in which the excavated slope undergoes progressive displacements even though the shear stress stops increasing. For design purposes, the Authors suggest to adopt a criterion of limited displacements and find that the direct shear test is probably the most representative of the soil behaviour in the slope. As a tentative suggestion needing more research, in the case of a plastic stress-strain behaviour of the soil, the maximum shear stress admissible should correspond to a deformation of about half the deformation at failure; this would possibly be sufficient to avoid "separation" at the top of the slope. Lower values

would be required for brittle soils. It should be noted that the cut slopes studied by the Authors usually have a critical surface which is shallow and steep, with a deep or non-existent water table, so that the study is made in terms of effective stresses.

The paper by Bell, Theissen & Tsai is a good illustration of the fact that there may be as many undrained shear strength values as there are methods of measuring it; the selection of a value which is representative of the soil behaviour thus becomes quite problematic. The Authors are analyzing slopes for an 8.5 m excavation in the well known San Francisco Bay mud. To define the strength profile, they have made unconsolidated-undrained triaxial tests, consolidated-undrained direct shear tests, laboratory vane tests and field vane tests, and have found appreciable differences between the different types of tests. The larger strength values were given by the small laboratory vane; this was also observed on tube samples of Canadian sensitive clays by La Rochelle and Lefebvre (1971) who have attributed this higher strength to the fact that the small laboratory vane test involves only the core of the tube sample which is the least disturbed part of the sample. For the tests on the San Francisco Bay mud, a reduction factor of 45% was applied on the small laboratory vane test, and 20% on the direct shear data. The Authors then proceeded to interpolate the mean values of all these corrected tests and defined a mean shear strength curve which was checked by using it in an analysis of two failed slopes and was found to underestimate the mobilized shear strength by about 10%; the lower boundary of the strength envelopes gave a better estimation of the mobilized shear strength. This case proves the complexity and the uncertainties associated with the use of the undrained shear strength to which empirical reduction factors must be applied. It also shows the necessity to find a better solution; the undrained strength at large strain (USALS) may offer a valuable alternative. The deep cut given in figure 5 of the Authors's paper is somewhat similar to the section of the Kimola canal, and it would have been interesting that an effective stress analysis be made to compare the effectiveness of the methods.

2.4.3 Natural slopes

Of all the papers presented at this Session and dealing with natural slopes, at least nine originate from Scandinavia; this shows the importance of the landslide problem in that area. Two major landslides are described: the Tuve landslide which occurred in Sweden in 1977 and has been the object of major corrective works which will be discussed later, and the landslide of Rissa in Norway which is probably up to now the best documented slide in sensitive clays.

The frequency of large landslides, of the order of millions of m^3, is fairly high in Norway where they occur at intervals of about 4 years. It is then not surprising that much effort is made to try and explain these catastrophic landslides. The "flake-type" failures which involve a large monolithic volume of clay sliding suddenly and rapidly along a near horizontal surface are studied by Aas. Using an idealized flake-type slide and analysing the stress conditions along the failure or sliding plane by means of the simple shear box test, the Author illustrates how the shear strength mobilized along the failure plane is lower than the theoretical value given by the angle of friction ϕ'; a critical shear stress level is obtained at which any sudden increase in stress will result in a marked increase in pore pressure and an overall decrease in the effective stress leading to failure. The reasoning based on the simple shear test device is difficult to follow and the interpretation of the test results is associated with some

uncertainty as mentioned by the Author, because the horizontal boundary stress on the specimen is not exactly known. Nevertheless, it is quite interesting that for four of the five case histories presented by the Author, the ratio of the undrained strength s_u to the effective vertical pressure σ_0' vary between 0.16 and 0.22 both in the laboratory test and on the field. Considering that this clay is nearly normally consolidated, this ratio seems to be close to the value of $s_u/\sigma_p' = 0.22$ found by Trak et al. (1980) and Larsson (1980) to be applicable to the problem of stability of embankments on sensitive inorganic clays where σ_p' is the preconsolidation pressure. However, the "long slope" cases in Scandinavian clays given by Larsson (1980) seem to indicate generally that an undrained strength value lower that $0.20 \cdot \sigma_p'$ is mobilized.

The spectacular Rissa landslide which was triggered by a small excavation work and extended over an area of 330 000 m², involving a volume of about 5 to 6 million m³ is described in detail by Gregersen. The Author gives a good acount of the main events and of the two different stages of sliding. As mentioned above, the slide was triggered by an excavation at the toe and the first stage occurred as a retrogression due to the low sensitivity of the clay and to topographical restraints; moreover, it was thought that in the area affected by the first stage of the slide, the shear stress level was appreciably below the "critical shear stress". Then followed the spectacular movements of flake-type slides which are described in detail with eye-witness accounts, The elaborate vestigations and mapping of the clay have shown that a band of quick clay 200 to 300 m wide existed all along the mountain side where fresh water under artesian pressure was infiltrating from the bedrock and leached the clay. From the laboratory test results, figure 12 of the paper, it is interesting to note that the maximum shear strength given by the direct simple shear corresponds to the large strain strength of the triaxial compression test and gives a ratio s_u/σ_0' of 0.20 which, according to Aas is the strength mobilized at failure in Rissa; this seems to show that the "large strain strength" determined by a compression triaxial test would be just as accurate to analyse the failure, and certainly much simpler than using a mixture of data from compression tests and extension tests in the triaxial apparatus plus direct simple shear tests.

The stratigraphy of a deposit and the presence of softer or more silty layers at some depth below surface is often of major importance in the evaluation of the stability of a slope. Many cases presented to this Session or discussed elsewhere (Leonard 1980) illustrate these points. Bernander and Olofsson suggest that the presence of a brittle layer could explain the translational displacement of a large land area; they are probably referring to the flake-type slide discussed above. The Authors assume that the failure zone is developing in a brittle stratum located at some depth below surface; the stress-strain properties of that stratum are "taken to be known" with the time-dependency corresponding to the rate of application of the driving force. The shear deformation is presumed to occur primarily in the brittle stratum and the compression of the upper layer to be of an elastic nature. With all these assumptions, the Authors propose an equilibrium equation which can be solved by a step-by-step method, either by hand or by computer, and which simulates a progressive shearing in the brittle zone. This analysis seems interesting but it raises many questions. How do we determine the time-dependent stress-strain properties of the brittle layer so that it be compatible with the elastic deformation of the overlying layer and with the time-dependent driving force? How do we identify the brittle layer in a clay deposit which is essentially brittle throughout its depth? In their conclusions,

the Authors advise against using peak shear strength as a basis for design; the Reporters agree and would further suggest that this advice should also apply to the stability analysis of natural slopes. This would eliminate the necessity to resort to a progressive failure type of analysis to explain failures in brittle materials.

Cancelli presents a detailed description of the properties of the Lugagnano clay in Northern Italy, together with the analysis of a periodical slide and of a first-time slide which occurred in this formation. The soil is a silty, heavily overconsolidated and fissured clay which can be defined as a "weakly marly clay". The peak, the fully-softened and the residual strengths were determined in the laboratory and were used to perform stability analyses. For the two slopes studied, piezometric data were obtained. The study gave very consistent results: for the periodical movement, the residual angle of friction ϕ_r' is mobilized, and for the first-time slide, the mobilized angle of friction is close to the fully-softened value. Moreover, the Author has observed that only the slopes with an angle $\beta < 5°$ to $6°$ are free from landslides, which corresponds to the situation where the residual angle would be close to mobilisation, with the water table coinciding with the surface.

The control of the regression of coastlines which are liable to be eroded by waves and sometimes wind action is a subject of concern in many countries. A very interesting study of the mechanism and rates of retreat of the cliff fronts along the north shore of Lake Erie has been presented by Quigley et al. (1977). A paper presented in this Session by Wiseman et al. describes a study made along the Mediterranean coastline of Israel in view of developing some stability criteria for a troublesome 13 km length of cliff. The Authors give a good description of the different geological units found in the typical profile and explain how undercutting erosion either by waves or by wind action can cause collapse of the more resistant layers of hard calcareous sandstone. Even after the undercutting was controled by different protective structures, shallow slips have continued to occur; it is to this problem of stability that the Authors have addressed their efforts. In such formations of cemented sand or silt, or of laminations of calcareous sandstone and sand as found in unit 2, located at the lower part of the slope, the variability in the degree of cementation is such that laboratory tests are hardly significant. The Authors have then chosen the right approach of looking at the way nature behaves and then attempt to establish criteria on that basis. Many slope sections, both stable and unstable, were measured and compiled in terms of the behaviour of the different geological units, and the results were plotted in graphs of height versus slope of unit 2 together with curves obtained by using Taylor's charts; the Authors have concentrated their attention to unit 2 because their observations led them to believe that this is the seat of the shallow instability. Analysis of actual failures using simplified Bishop method completed the data and helped in establishing useful design charts. As mentioned by the Authors, this study is indeed an interesting example of the successful "cooperation of engineering, geologist and geodetic personnel making use of field observation and measurements, laboratory testing and analysis".

Mention should be made of the very elaborate study which was performed by Yamanouchi et al. (1977) on the problem of cuttings in the Pumice-flow soil deposit of the southern Kynshu area of Japan. This soil, called "Shirasu" is a volcanic ash and sand mixed with pumice which is susceptible to flow with rainfall. The study included strength and stress analysis which led to the choice of an optimum slope angle.

2.4.4 Earthflows

Earthflows constitute in some countries problems which are as challenging for engineers and can be as disastrous for the population than any other type of mass movement. In this Session four papers dealing with earthflows have been presented.

Manfredini et al. describe extensive landslides which are affecting the Crete Nere or black shale formation in Italy. The Authors give a detailed description of the geological origins and characteristics of this formation which is essentially composed of argillaceous layers interstratified by rare and thin lithoid layers. The mineral content in the argillaceous layers is high and is constituted mainly of illite and kaolinite. The material is highly slaking and is affected by shear deformation. Weathering plays an important role in the evolution of the slope; where earthflows are underway, the original structure of the material has been completely obliterated by mechanical and chemical degradation. The surface displacement is of the order of 2 to 4 cm/day and is shown to be directly influenced by the rainfall intensity. The inclinometer readings seem to indicate that the sliding mass moves as if it were a rigid body. A thorough geotechnical analysis of the earthflow and of the intact material has been carried out and shows the influence of the remoulding and weathering on the flowing material. The shear strength was determined by triaxial drained and undrained tests and by shear box tests on the intact, the earthflow and the remoulded material. The results of the back-analysis indicated that the behaviour of the slide is consistent with the residual strength values obtained in the laboratory. This is a very well documented case history containing many valuable results. The Authors conclude, however, that progressive failure probably played an important role due to the considerable difference between the peak and the residual strengths; the Reporters want to emphasize that such a brittle material can indeed be subjected to progressive failure but only if an over-stressed condition prevails in the field, which is not the case in this problem as conceded by the Authors. This case seems to be only one other example where the peak strength measured in the laboratory is not mobilized in the field due to softening effect, fissures, rate effect, or creep, etc...   •

The mud flows which affect mainly the surface layer of natural slopes are studied by Vallejo and are defined as a mixture of hard clay fragments or pieces of rock floating in a matrix of liquid-like clayey slurry. These flows usually occur in two stages: the first one is due to one of many possible causes such as dynamic loading, increase in pore pressure, rainfall impact, thawing, etc.. After the failure stage, if enough water is available, a transition to a condition of streaming occurs, which implies flowing at low velocity. Based on observations made in an experimental channel, the Author has studied the evolution of the flow, of the bulbous ridge or snout and has suggested a factor of safety based on simple static conditions of equilibrium. The limit equilibrium conditions of the advancing front of a mudflow on the London clay slope as well as for the experimental flow was successfully assessed by the analysis proposed by the Author who has also shown the predominance of the free surface slope on the behaviour of the mudflow. The Author obtains a simple equation for the safety factor based on the concentration of clay or rock pieces per unit volume of the mudflow mixture; this parameter might not be easy to predict in nature. The Author concludes that, in the cases where the inclination of the free surface and of the bed of mudflow is small, the gravity shear stress causing the movement is a function of the slope of the free surface and is independent of the slope of the bed.

In the paper by Vallejo & Edil, the stability analyses of thawing slopes in cohesive soils are examined in the light of the particulate structure approach suggested by the senior Author in previous papers. By investigating the behaviour of a thawing slope in cohesive soils, they have observed that when a critical depth of thaw is reached in the face of the slope a combination of soil lumps and muddy water will be formed and will slide. They analyse the stability assuming that the effective strength parameters as determined in shear box tests are mobilized and thus seem to obtain good results for that slope. They also question the validity of other approaches proposed by different authors but they do it on scant evidence. For example, measuring the undrained strength with a small vane at 10 cm depth in a thawing soil and assuming that this is the strength which is acting at the critical depth of 25 cm is somewhat speculative. The argument that an excess pore pressure as small as 0.5 kPa, which is only 0.1 the overburden pressure cannot develop at a depth of 25 cm in a thawing soil is not evident to the point of rejecting a priori the relevant theory. The Reporters would question the fact that the Authors are generalizing their observations to different climates. The evolution of the frost line in the soil is quite different in the southerly temperate climate of Michigan from what is observed in more northerly climates; consequently, the behaviour of the slopes can also differ appreciably.

2.4.5 Creep movements

Creep movements are very wide spread in nature and have retained the attention of many researchers. The State-of-the-Art report presented by Ter-Stepanian (1980) at the ISL in New Delhi presents a good review of this phenomenon affecting the slopes. As discussed by Varnes (1978), the limit between the terms flow and creep is rather vague; often people dealing with slow movements in slopes which are not failing will use alternatively the terms creep or flow.

The three cases presented by Disli and Recordon constitute such an example of slides which are qualified by the Authors of "viscous flows" and are studied by means of creeping tests. These movements occur in the prealpine and glacial clayey formations in Switzerland. Two of these slides can be qualified of "earth flows" as they are continuously supplied at their upstream end by disintegrating cliffs. In these cases the rates of displacement were quite important up to 3 m per year measured in some points of the flow. In the other case, the movement was initiated by the construction of a road embankment and the rate which was quite high at the beginning was reduced following the construction of surficial drainage works. The Authors have observed in situ a stationary creep with random small accelerations or decelerations which are somewhat related to the rainfall intensity, or to the loading or unloading of the surface; the variation of the dynamic viscosity is small. It would then seem that this flow behaves according to Bingham's law of constant viscosity. The laboratory tests made by means of a direct shear apparatus confirmed the existence of a linear relationship between the logarithms of the shear rate and of time. However, quite different results were obtained by means of a ring shear apparatus developed by the Authors where three different phases of rate shear variation with time were observed to occur; these rates were probably influenced by consolidation at the early stage of the test. On the basis of their observations and tests, the Authors consider that this material should be treated as a solid or viscous liquid rather than as a rigid solid-perfectly plastic.

Tsytovitch and Ter-Martirosyan consider that the determination of rheological parameters by loading in the

laboratory gives no idea of the rheological behaviour of the soil mass which has been under stress for a long time; these Authors prefer to base their evaluation on field observations. They are concerned by the cyclic movements of the landslides resulting from the changes in the initiating factors and especially the rainfall intensity. In the case where the slope has a porous stratum, they are proposing to apply a theory of the propagation of pressure "waves" in a deformable porous medium. According to this theory, accumulated plastic deformations may result from increases in stress which lead to a stress in excess of the long-term strength of the soil. They consider the soil to behave as a visco-plastic medium in accordance with Bingham's law. They obtain a satisfactory agreement between the field and laboratory investigations and the calculated values of displacements.

It may be relevant to mention that, in his discussion on the rheological basis of the mechanism of slope failures, Ter-Stepanian (1979) has come to the conclusion that the Bingham's law of constant viscosity of soils is generally incorrect except in the slopes for which the stress state has been established long ago and where all the processes remain unchanged.

The phenomenon of creep has been mostly associated to the residual soils, clay shales, overconsolidated clays, and other similar soils. It was generally considered that the sensitive quick clays, and especially the reputedly "cemented" sensitive clays of Eastern Canada, had such a brittle behaviour that the slope failures in these soils would occur very suddenly without any perceptible previous creep movements. However, during the last decade or so, many field observations (Mitchell & Eden 1972) and laboratory studies (Vaid & Campanella 1977; Tavenas et al. 1978) have shown that considerable creep can take place in sensitive soils before failure occurs. In the light of the limit state model, Tavenas & Leroueil (1981) have discussed the mechanism of occurrence of creeping movements and their consequence on natural slope behaviour. They present an interesting observation where a downward creep movement or settlement of about 1 m has affected the crest of a natural slope over a period of 40 years before failure occurred. An extremely valuable field experiment has been made recently by Mitchell & Williams who have artificially produced the failure of an instrumented slope in the sensitive clays of the Ottawa area by pumping water into recharge wells behind the slope. They have measured surface movements up to 50. mm, more than 15 days before failure. Eden (1977) and Ter-Stepanian (1980) have also discussed the problem of creep movements in sensitive clay slopes.

2.5 Dams and comments on papers

The increasing demand for energy in most of the countries has promoted the development of hydroelectric projects, which include the construction of high and large embankment dams. Before 1960, the height of these structures was less than 100 m, with few exceptions. The experience gained at that time was very valuable, but not adequate to design earth and rockfill dams in the 100 to 300 m high range. Very little was known about the mechanical properties of free draining masses composed of rock fragments, and for economical reasons, deep and narrow damsites having irregular geometries were selected, the crest length-height ratio being less than three in some cases. These facts impelled two main lines of action: 1) the research on rock fill materials and 2) the monitoring of embankment behaviour.

The investigation of the mechanical properties of coarse granular materials demanded the design and construction of testing equipment and the development of techniques to handle and prepare large specimens (1 to 2 m^3).

Gradually, results of the drained shear strength, stress-strain characteristics and compressibility became available, and today there is a reasonable amount of experimental information on rockfills (Fumagalli 1969; Marachi et al. 1969; Becker et al. 1972; Marsal 1973 & 1977).

To observe the behaviour of zoned embankments it was required to adapt and improve monitoring devices, to develop other instruments and the techniques to place them in the dam during construction, and to take measurements periodically. To facilitate the interpretation of these measurements and as a frame of reference, the stresses and deformations were computed for different construction stages, using the finite element method (non linear, elastic). Notwithstanding the limitations of both field measurements and the theoretical evaluation of stresses and deformations, the comparison of observed and computed values allowed to evaluate qualitatively the behaviour of the structure, and if necessary, to improve the design in the course of construction. This approach to the design of rockfill dams like El Infiernillo, La Angostura and Chicoasen in Mexico, for example, has disclosed information related to the development of plastified and tensile zones, arching between banks, interaction between the impervious core and shells, cracking, etc. (SRH-CFE-UNAM, 1976; Marsal and Moreno 1979) as well as to observe the performance of the structure upon the first filling and subsequent operation of the reservoir (CFE 1980; Moreno & Alberro 1981; Alberro & Moreno 1981). A review of current trends in design and construction of embankment dams was undertaken by Wilson and Marsal (1979).

From the above introductory remarks, one is tempted to ask: what is the role of the stability analysis in the case of a high earth and rockfill dam? The Reporters believe that the stability analysis is a good exercise for the purpose of distributing the available materials within the embankment, and also to have some measure of the safety against failure and of its variation for the different geometries of the dam cross-section. On this matter, several papers presented to this Session which will be briefly commented below might be of some help in selecting the most convenient type of section subject to review after the preliminary stress-strain analyses. But, as outlined at the beginning of these remarks, the designed cross-section may undergo during construction successive adjustments to take into account topographical and geological details, variation of materials used, and results of field measurements.

The following comments on the papers will be classified under five subheadings: 1) limit equilibrium methods; 2) testing of physical models; 3) construction pore pressures; 4) embankment performance, and 5) sand-asphalt facings.

2.5.1 Limit equilibrium methods

In practice, when dealing with homogeneous embankments, the methods discussed in the previous section of Theme 2 are usually applied. Design parameters are based on results obtained from conventional laboratory tests on specimens prepared according to compaction specifications and fully saturated; however, in some cases, the strength-deformation characteristics of the as-placed material (in non saturated condition) are required in order to evaluate the embankment behaviour during construction. Effective stress analyses are performed with pore pressures estimated from shear tests and/or seepage flow nets. The assumed mechanism of failure is the slip circle (two-dimensional case) which gives the minimum factor of safety (F); the accepted values of F vary from 1.0 to 1.5 for different working conditions of the structure (full reservoir, drawdown, seismic action or combination of such conditions).

Fredlund et al's critical review of several methods of stability analysis (Fellenius, Bishop, Janbu, etc.) as compared to the GLE formulation, gives a clear insight of the variation in the factors of safety that could be expected from the application of the above methods.

Given the locations of critical slip surfaces connected to intrinsic variations of soil properties and other uncertainties derived from geotechnical practice, Grivas establishes the relationship between the factor of safety (FS) and the probability of failure, assuming three different models for the probability distribution of FS; the procedure is applied to a case study and the factors of safety are compared with the value obtained by means of the Bishop's simplified method, the latter being smaller than the most probable FS estimation. Yong et al. (1977) working on the instability risk of the slopes at Davidson Corners, Ottawa, find a similar result. The probabilistic approach may be a valuable tool in dam engineering, because the variation of the material properties and the uncertainties connected to construction practices can be reliably accounted for.

The design of a zoned embankment raises several questions related to the applicability of the conventional static methods of stability analysis, namely: shape, of the slip surface, strain compatability and time effects. Non linear elastic stress-strain analysis (FEM) have revealed, for instance, some interaction processes between zones that are not taken care of by the commonly used procedures; the stress state developed during construction may be far from the stress distribution assumed for design purposes. Furthermore, no account is made of 1) the stress changes caused by the first filling of the reservoir and 2) the time-dependent phenomena (delayed deformation by arching or interaction) which also involve redistribution of stresses. However, hundreds of embankments of this type have been built around the world based upon the above mentioned methods of stability analysis (Janbu 1967; Morgenstern & Price 1965), and with few exceptions, they have shown reasonably good behaviour. Approved engineering practices as well as construction experience are the necessary ingredients of such an engineering task and may be largely responsible for the successes on records.

To reconcile the above-mentioned facts one has to take into account that 1) the materials commonly used in earth and rockfill dams are really plastic for large deformations (greater than 5 percent) and 2) the peak strength is not observed in well compacted granular materials when subjected to stress levels higher than 5 kg/cm^2 (Marsal 1977).

The methods of slope stability analysis applied to zoned embankments are based on non-circular types of slip surfaces (wedges or combination of cylinders). As mentioned in a previous section of this report, the contribution by Celestino & Duncan on this subject will greatly help the design engineer in searching for the critical slip surface by means of numerical computation. On the other hand, Martins et al. investigate the limit equilibrium of a non-circular sliding mass composed of several blocks, in which the shear strength is mobilized at both the slip surface and internal boundaries when failure occurs. This procedure like the method proposed by Fredlund et al. may find application to analyse the stability of earth and rockfill dams.

2.5.2 Testing_of_physical_models
To investigate the stability of Gongen Dam, Japan, several models were tested in a centrifuge. This structure is a zoned embankment 32.6 m high resting on a deep alluvial deposit which includes a 2 m thick clay stratum.

The height of each model was chosen to reproduce in a 150 g acceleration field, the stresses on the prototype. The tests were divided in three stages, namely: 1) gradual acceleration up to 150 g; 2) upstream water level changed from the base to 90 percent of the dam height (5.5 cycles); and 3) inclination of the model up to failure. Mikasa et al. describe the displacements and settlements of the impervious core in stages 1 and 2 as well as the corresponding volume and shear strains, and for stage 3 the inclination angles that induced failure of the models. The measured deformations at stage 2, as commented by the Authors, are similar to those reported by Marsal & Ramirez de Arellano (1967) for El Infiernillo Dam, which were based upon field measurements. The deformation of both the clay stratum at the foundation and the downstream rockfill showed less influence on a central impervious core than in an inclined core, as originally proposed for the Gongen Dam. The technique applied in this particular case, however involved, might be of great help to investigate more closely some phenomena that are being disclosed by field instrumentation (SRH-CFE-UNAM 1976; Marsal & Moreno 1979).

2.5.3 Construction_pore_pressures

Measurements performed during the construction of Ramganga Dam, India, are described by Goel & Das; these observational data are compared with values computed with analytical methods (Hilf, Bishop and Li, Gibson). The embankment is 128 m high and has a central vertical core made of compacted clay shale with slopes 1 V to 0.25 H, supported by pervious zones of crushed sandstone. To monitor pore pressures, USBR twin-tube hydraulic piezometers were installed at about the maximum cross-section of the dam; also, settlements were measured by means of six USBR deformeters.

Comparison of the observed and predicted values indicates that pore pressure development in the core does not coincide with the theoretical curves, and that the monitored pore pressure ratio at the end of construction ranges from 0.1 to 0.2, whereas those given by the Hilf, Bishop & Li, and Gibson methods are appreciably higher.

2.5.4 Embankment_performance
Only three papers presented to Session 11 contribute to this subject that has evolved significantly in the last decades as a result of field measurements and numerical analyses of stresses and deformations in embankment dams. Due to the influence of placement procedures and defects of the devices themselves, behavioural monitoring has to be considered currently as a qualitative, valuable tool. On the other hand, the 2D and 3D non linear elastic models that can be handled with powerful computers, are limited by lack of constitutive laws for the different materials used in earth and rockfill dams. For the above reasons, case histories on this topic continue to be important sources of information to guide the research in both fields of embankment engineering.

Nagarkar et al. describe several partial failures registered during 100 years of operation of the Waghad Dam, India. This is a homogeneous embankment, 32 m high, resting on weathered rock; it was built with highly plastic, expansive clay. Due to overtopping, the earth structure was breached upon the first filling and repaired; as this construction operation was barely finished, a slip developed at the downstream slope. Twenty years later, another partial failure in a different section of the same slope was triggered by heavy rainfall; water precipitation was scarce (half the mean annual value) in the previous two years. A third slip

was observed in 1919 at the upstream slope, the scarp 75 m in length running along the pool edge of the crest. Finally, in 1976 a failure located at the same section as the second slip developed under similar conditions of rainfall.

After the last failure, investigations were undertaken with the opening of trenches at several locations to observe the soil mass, obtain block samples and perform laboratory tests. With this geotechnical information, the four slips mentioned above were analysed using the Bishop's simplified method and the Swedish slip circle; these studies comprised both upstream and downstream slopes subjected to drawdown and steady seepage conditions respectively, assuming full saturation by rainfall above the phreatic line. Based on this careful investigation the Authors conclude that 1) the Bishop's simplified method back-predicted more accurately the observed failures, and 2) the dams built with highly plastic clays require a cover 2 to 5 m thick, to minimize cracking due to drying.

Wackernagel discusses the performance of a 9 m high earth dam, built in 1866 near the city of Basle, Switzerland. It was overtopped and breached immediately after the first filling; during its long life, it was overtopped several times undergoing minor damage. Recent studies performed by means of two borings, undisturbed sampling and laboratory tests, have disclosed that the dam is of the homogeneous type and is resting on a 1.5 m deposit of gravelly materials with low clay content. According to topographical records the crest has settled about 1 m (12% of its height) in 108 years, as compared to 47 cm estimated upon consolidation test results. From this, the Author has concluded that the settlement was partly caused by creep of the embankment; he then applied the residual strength determined in direct shear tests to evaluate the current factor of safety, which he finds close to one (FS = 1.14).

A third paper by Justo & Saura describes the performance of the asphaltic concrete facing of the Venemo rockfill dam by comparing the displacements measured one year after the first filling and the corresponding values computed with the three dimensional finite element method, assuming linear elastic behaviour of the rockfill mass. The modulus of deformation of this material was adjusted so that the maximum computed and the measured displacements be equal, with a Poisson's ratio of 0.25. A wide range of values of E and ν for the asphaltic concrete of the facing were tried in the computations because of the variability shown by the laboratory test results. With the above assumptions the Authors discuss the effect of 1) the above design parameters on the distribution of principal stresses at the facing for the full reservoir condition; 2) the inclination of abutment slopes on the tensile stresses in the asphaltic lining, and 3) the variation of the mechanical characteristics of the facing on the global deformation of the dam; the influence of the latter factor is very modest.

2.5.5 Sandasphalt facings

On this topic that is rather marginal to Theme 2, Mulder et al. present some interesting information on the stress-deformation characteristics of hot mixtures made of 3 to 5 percent by weight of bitumen and sand. This material is used in the Netherlands as a protection of the outer slopes of dry and submerged sand deposits. The Authors discuss the viscous and visco-plastic behaviour of the sandasphalt and propose a model for large strains which can be introduced in the stability analysis. Early experience and the experimental results mentioned above seem to indicate that this type of facing

is promising, particularly in coastal and river engineering.

THEME 3: Factors influencing the stability of slopes

Whenever a slide occurs in a given area, the tendency for the technical man, and sometimes also for the legal man, is to find "the" culprit, or "the" cause of the slide. However, it is now generally agreed upon that in most cases, more than one factor contributes to the instability of a given soil mass, and that the "identified" cause is often that minor event which has actually triggered the movement. When the Organizing Committee decided to use the term "factor" rather than "cause", they made a judicious choice since that term implies a less exclusive relationship with "effect".

It is not the purpose of this report to review all the factors which may contribute to slope sliding movements; this subject has been covered very expertly by Varnes (1978). To remain within the scope of a general report, it has been judged preferable to deal with the factors which are mentioned in the papers presented to this Session.

3.1   Pore pressure increase

Of all the contributing factors referred to in the many papers presented to this Session, the pore pressure increase is by far the one which recurs most often. Pore pressure may increase in a soil mass either as a result of natural climatic conditions such as rainfall or snowmelt, or else it can be induced by different interventions of man.

3.1.1 Rainfall

Many of the papers which are discussed under the other themes present graphs which show how the rainfall intensity influences the rate of displacement or the number of slides. In describing the slides and flows occurring in the black shales of Italy, Manfredini et al. give data showing a very good coincidence between the rainfall intensity and the rate of displacement. Different results were obtained by Disli & Recordon in the course of their study of three cases of viscous flows in the prealpine clayey formations of Switzerland; in one of the three cases, the Authors have observed a random rate of displacement which is only slightly related to rainfall; unfortunately, no information is given on the two other cases studied. Tsytovitch and Martirosyan have measured the pore pressures at different depths in a soil mass and have shown that the spells of rainfall are translated as pressure "waves" into porous strata; however, there is no evidence given in the paper to the effect that the rate of displacement follows the pressure waves. Nishida et al. (1979) give also a theoretical analysis of the magnitude of pore pressure resulting from rain water permeation.

A high intensity of rainfall, can often result in a marked increase in the frequency of landslides. This has been observed, for example, by Fukuoka (1979) who has presented some elaborate observations leading to a relation between rainfall intensity and the number of landslides per unit area. The relationship between the season and the frequency of landslides is also well established in the northern countries where snowmelt combines with rain to produce critical conditions; Viberg shows that for the whole of Sweden there are two peaks in the histogram of slide frequency, one in the spring and one in the fall; but it is also interesting to note that in northern Sweden the slide frequency is higher during the spring while in southern Sweden, it is higher during the fall, which is an

indication of the influence of snowmelt in spring-
time. According to the Author the same trend was found
by Jörstad (1970) in Norway. In the sensitive clay
deposits of Québec, Canada, Lebuis & Rissman (1979)
have established an histogram which gives a very
predominant peak during the month of April and May when
the snowmelt combined with rain can produce the worst
possible conditions of infiltration; this is
illustrated in Fig. 2 where Tavenas & Leroueil (1981)
compare the variation of the slide frequency with
seasons in Scandinavia and Québec. The groundwater
recharge due to snowmelt is a predominant factor in
Québec where the snow cover is important; in fact,
nearly all the major landslides in Eastern Canada have
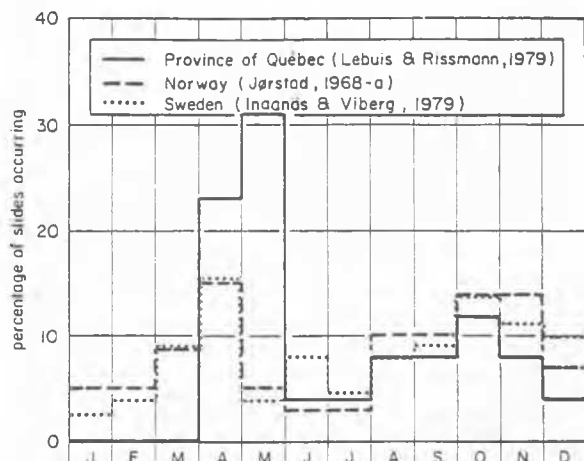occurred during springtime.

Fig. 2 — SLIDE FREQUENCY VERSUS MONTH OF THE YEAR
IN QUÉBEC, NORWAY AND SWEDEN.

(from Tavenas et Leroueil, 1981)

When dealing with the residual soils of Hong Kong,
Brand has to face a somewhat different problem where
the balance between infiltration and pore pressure
suction plays a major role. In these cases of rain-
induced failures in slopes of unsaturated soils, there
seems to be no rational procedure for design or ana-
lysis. The infiltration causes a reduction in the pore
suction, hence reducing the strength until failure
which takes place under almost constant stress condi-
tions, as in most natural slope failures. Obviously,
this problem can only be rationally dealt with in terms
of effective stresses and the Author proposes to
simulate the failure conditions by means of triaxial
tests in which the total stresses would be maintained
constant and the pore pressure increased. The Author
also emphasizes the importance of the behaviour of the
soil at failure: if it tends to dilate at failure, suc-
tion will increase and the action of infiltration will
be counteracted; if it compresses, the suction will
rapidly decrease with a resulting flow occurring. This
phenomenon will of course depend on the stress level
relative to the preconsolidation pressure below which
dilation is observed to occur during shear in residual
soils. The solution of the problem is complex as there
are many experimental difficulties related to these
kinds of soils and tests. The Author proposes to
monitor soil suction on a long term basis in a number
of selected slopes.

From all these observations, it is evident that rain-
fall intensity is a major factor in the occurrence of
landslides, earth flows or creep movements. However,
surprisingly little effort has been devoted to relate

the rainfall intensity to the pore pressure or pore
suction in the ground as suggested by Sällfors and Tägn-
fors. Such a compilation made over a few years could
allow the use of a probabilistic approach to define the
most probable maximum pore pressure which may develop
in some given topographical and geological conditions.
The numerous factors which must be taken into account
have been discussed by Tavenas & Leroueil (1981) who
show the complexity of this problem and conclude that
the safe approach is to determine the upper limit of
groundwater conditions. However, a few uncertainties
remain when it comes to evaluating the effects of this
maximum groundwater on the pore pressure in the clay
mass; this emphasizes the importance of pore pressure
measurements.

Rainfall can sometimes be troublesome to man-made
structures. Nagarkar et al. discuss the possibility
of failure of the downstream slope of a dam in India
due to the infiltration of water in cracks which might
have formed down to the phreatic line during drought;
they show that by taking this factor into account in
their stability analysis, the computed factor of
safety would be appreciably lower than unity.
Cartier et al. describe the behaviour of railroad em-
bankments which were built during the last century with
clayey and silty soils excavated from each side of the
fill to minimize handling. Due to the lack of compaction
and to the poor quality of the fill material, the em-
bankments are deforming and their surfaces are forming a
pool under the ballast where rainwater accumulates and
infiltrates through cracks into the fill material; water
collects also at the toe with the results that the em-
bankment material has generally a high water content and
a low shear strength.

3.1.2 Induced pore pressure

Pore pressure can be induced in different ways in a
soil mass and eventually lead to a mass movement or a
slide. The most evident example is given by Mitchell
& Williams who have pumped water into a recharge well
and measured the resulting deformation up until a slide
occurred; a similar experiment had been attempted in
Japan some ten years ago with very dramatic consequen-
ces.

Japelli & Musso report the case of a "slide under con-
trol" which is affecting the bank of an artificial res-
ervoir in Sicily. This slide has many characteristics
which are similar to the Vajont rockslide. The mass in-
volved in the movement is of such an extent that only
the lower part is influenced by the changes in the res-
ervoir water level. The formation includes a bed of
Cretaceous limestone overlain by a chaotic structure of
limestone, marl, green and brown clay. Of particular in-
terest is the orientation of the rock layers which fol-
lows the slope of the surface and becomes horizontal in
its lower part and at the reservoir bottom. According to
the observations, the failure plane may be either near
the contact of the debris with the bedrock, or in the
bedrock itself depending on the section studied. A marly
clay layer has been identified in one area at the con-
tact between the debris and the bedrock, with a residual
angle $\phi' = 13°$ measured on a remoulded sample of that
clay. The slide was first observed during the initial
filling of the reservoir in 1963 and is under observa-
tion ever since. The Authors present some very interest-
ing measurements showing how the rate of displacement
reacts to the rate of water level rise and mention that
the influence of the rainfall intensity on the slope
movement is not evident. If the marly clay layer with a
residual angle of 13° is assumed to be located all along
the failure surface, the calculated factor of safety is
1.0 for the condition when the lake is empty. However,
if the reservoir level is increased to normal operation

level, the factor of safety drops to 0.92. This may be an indication, as mentioned by the Authors, that the clay layer or clay fillings which are probably more or less continuous govern the stability.

Massarsch and Broms discuss the problem of pore pressure induced by pile driving in sensitive soils and the adverse consequences that it may have on the stability of slopes. This is a most welcome paper in these times of lost assumptions. The Authors first analyse the generation of pore pressure due to pile driving by means of the theory of expansion of spherical and cylindrical cavities and propose a graph combining both types of expansion and enabling the prediction of pore pressure in the plastic zone around a single driven pile. They then proceed to discuss the effect that pile driving may have on the disturbance of the clay and the ensuing decrease in its strength; the Authors mention clearly that the sudden increase in pore pressure does not affect directly the stability, as the undrained shear strength of a cohesive soil is independent of a change in total stress; however, when the excess pore pressure dissipates by radiating to adjacent zones, the swelling which occurs can result in a gradual decrease of the strength. One of the most problematic situations arises when permeable seams are located in the clay slope section; the driving of piles will then produce an increase in pore pressure which will immediately reduce the shear strength of cohesionless soils or cause some liquefaction if the soil is a loose saturated sand or silt. Runesson et al. discuss the influence of pore pressure increase in more permeable seams and show how this can lead to the failure of a slope. This situation is still more critical if the seams happen to be dipping towards the toe of the slope. Massarsch & Broms refer to many cases of slides due to pile driving in Scandinavian clays which are reported in the literature. One case has also occurred recently in Canada (Carson 1979) where a large landslide in a sensitive clay could be attributed to liquefaction during pile driving of a silty layer at some depth below the toe of a slope. The Authors are presenting a very well documented case history where they have measured at the toe of a slope the deformations and pore pressures generated by pile driving; they have come to the conclusion that preboring was essential in order not to endanger the stability of the slope. This paper is a valuable contribution and the Reporters would like to take this opportunity to emphasize that, in spite of the "lost assumption" (Golder 1979), pile driving can produce slope failures, especially in stratified and/or sensitive soils.

3.2 Erosion

Erosion can probably be considered as the second most important factor of slope instability; it is often identified as the triggering agent of large flow slides in sensitive soils, and as the main factor of bank regression and slides in less sensitive clays and clay shales. A study by Bjerrum et al. (1969) has shown the role of erosion in quick clay slides, and more recently, Williams et al. (1979) have studied the rate of recession of a riverbank due to erosion in the Ottawa area; erosion also plays an active role in areas where clay shales are predominant and may lead to large retrogressive landslides as observed in the Saskatoon area by Hang et al. (1977).

It is then somewhat surprising that only two papers in this Session refer to erosion as an important factor of instability for the slopes under study. Wiseman et al. studying the problem of regression of the coast line along the Mediterranean Sea in Israel, have described two types of erosion: the classical action of the waves which are undercutting the toe of the slope in the soft rock, and the wind action which is eroding the lightly cemented sand in the upper half of the slope and undermining the overlying hard sandstone. In his paper on the evolution of natural slopes in overconsolidated marine clays, Cancelli discusses the case of a first-time slide which resulted from stream erosion increasing the mean angle of slope of a hillside that had never been affected by preceding slides.

The fact that, in this Session, so little mention is made of erosion as a factor contributing to instability should not lead us to a wrong appraisal of its importance. In the opinion of the Reporters, erosion remains one of the main factors and should probably be placed by order of importance immediately after pore pressure increase. It is a known fact that some important landslides could have been avoided, or delayed appreciably, if erosion of the toe of the slopes had been controled.

3.3 Stratigraphical discontinuity

The importance of the stratigraphy, or of the presence of a discontinuity in the soil profile is a factor which is often overlooked and can make a considerable difference between stable conditions and a disastrous slide; the examples to substantiate this point are abundant. In the present Session, many papers discuss the adverse influence of seams of clay, silt, or of brittle material as a factor of instability (Bernander & Olofsson, Runesson et al., Massarsch & Broms, Tsytovitch & Ter-Martirosayan, Japelli & Musso). In two cases, failure was observed to have taken place along some discontinuities such as a marly clay seam at the contact with or in the bedrock (Jappelli & Musso), or in a pre-sheared layer of silty clay below a wedge of Boulder clay (Widdis & Clapham). Vajont remains probably the most dramatic example of the importance of stratigraphic features, but there are also other examples discussed by Leonard (1980). The case of Saint-Jean-Vianney (Tavenas et al. 1971) offers an example which underlines the importance of not only the presence of strata, but also of their dip. As reported by La Rochelle (1975), on the south side of the river where the flow-slide took place, the stratifications were dipping $1°$ to $2°$ towards the river while on the north side, they were dipping inland; on the north side where impressive clay cliffs were exposed, only minor slides have occurred although the upper 20 m thick layer of clay was observed to move out more than 8 cm along stratification planes. In this case the characteristics of the strata helped to explain the extent of the flow-slide but, as mentioned in section 2.2 above, the first slip of the river bank could be back-analysed successfully without taking into account the presence of the strata.

3.4 Human activity

One other factor, human activity, has been mentioned by three Authors and is of some importance especially in soft clay areas. The spectacular slide of Rissa has been triggered by man. Gregersen reports that the slide was initiated by an earth fill of 700 m^3 placed on the shore of lake Botnen. Dysli & Recordon also report that one of the earth-flows that they have been studying resulted from the construction of a highway across the area. The influence of human activity is also well illustrated by Viberg who gives a graph showing marked increase in landslide frequency since 1890, period which corresponds to the beginning of the construction of railroad lines and of the industrial era.

3.5 Other factors

In some countries, frost is a factor of degradation and weathering of soils which decreases the strength of the soil and may lead to earthflows which are usually of a surficial nature; such a case is discussed by Vallejo

& Edil. The volume of soil displaced is usually small in the temperate climates, but in periglacial areas, thawing can be a major factor of alteration of landform which occurs mainly as a solifluction phenomenon; McRoberts & Morgenstern (1974) have given a good description of the mechanism and extent of this process of degradation of slopes in northern Canada.

THEME 4: Slide warning systems and methods of prevention of landslides

Most of the papers related to this theme in the Session refer to measures which are applied in practice to prevent sliding or to improve stability; a single contribution deals with warning systems. However, between the Tokyo International Conference (1977) and the present one, many papers on this matter have been included in the following publications.

- The Proceedings of the International Symposium on Landslides, New Delhi, India, 1980: Session V was devoted to prediction of landslide behaviour (11 papers), and Session VI delt with landslide control measures and their efficacy (15 papers); these contributions are included in Volume I of the proceedings; Volume II contains the General Reports by Bhandari, and by Datye on the above mentioned subjects, respectively, and also the invited lecture on "Instrumentation, its role in landslide prediction and control" presented by Fukuoka.

- The Special Report 176 on "Landslides, analysis and control", published by the Transportation Research Board, National Academy of Sciences, Washington D.C., USA, 1978; Chapters 8 and 9 contain information on design procedures and remedial measures for soil and rock slopes, and chapter 5 on field instrumentation.

- The proceedings of the Sixth Asian Regional Conference on Soil Mechanics and Foundation Engineering, Singapore, 1979; several papers on landslide stabilization using retaining walls, caissons, perfo-bolts and prestressed anchors, drains, etc.

Also, a number of papers have been submitted·to Session 12 of the present International Conference, which describe some methods, for soil improvement that may also be applicable to prevent landslides.

4.1 Warning systems

To study the evolution of a slide by means of observational techniques, Mitchell & Williams carried out measurements on a slope of Champlain clay, Canada, which was brought to failure by increasing the hydraulic pressure in recharge wells. Previous stability analysis of this slope showed that the safety factor was close to unity. The Authors comment that the visual observation of cracks and deformation at the toe of the slope, piezometric monitoring and measurements by means of microseismic or acoustic methods are not clear indicators of the instability of natural slopes on clays of this type, which are overconsolidated, fissured and very plastic (w_L = 66%, w_p= 8 to 10).

The instrumental data collected at the test slope revealed the response of the different instruments installed (piezometers, deflectometers, shear distortion measuring devices and extensometers). Following these results, the Authors recommend, for monitoring the long-term behaviour, the use of extensometers near the surface connected to an alarm device which will be triggered when the slope deformation reaches a specified limit value. Maintenance of such a device is simple and can be adjusted periodically so as to have a complete record of the slope movement. This paper also contains information on the behaviour of sensitive clays, as indicated in a previous section of this Report.

The use of a photogrammetric method to observe slope movements is advocated by Shields and Harrington. Although their paper refers specifically to the detection of displacements in open mine excavations, the method might be applied to earthflows and may provide detailed information on the major features of this type of slide. The author comments on how to get acceptable accuracy with conventional equipment.

4.2 Prevention of landslides

Drainage is one of the most commonly used means of improving the stability of natural slopes and cuttings. Nonveiller investigated the effect of horizontal drains on slope stability taking into account the time required to establish the flow of water in the domain of each drain, which in turn depends on its length and spacing, as well as on the coefficient of consolidation of the soil. Based on the theory of consolidation and by means of finite differences, the Author estimates water pressure dissipation, increment of effective stresses in the zone of influence of the drain, corresponding settlements and the time required by the process. With a parametric approach, two slopes (2:1 and 3:1) are analyzed, varying the length, spacing and location of the drains· This theoretical study discloses that short drains installed in the face of the slope have less stabilizing effect than long drains bored at the base, the latter being also more effective than a drainage ditch excavated at the toe. Another conclusion of practical interest derived from the above mentioned study is the evaluation of the time necessary to increase the safety factor in terms of the coefficient of consolidation; it is shown that satisfactory stabilisation can be achieved within one month in sandy or silty material, while it may take six months in clays. The design of horizontal drains for soil slopes was studied by Kenney, Pazin & Choi (1977) who provide design charts as suggested guidelines. A case history of the stabilization of a landslide in stratified sensitive clay by means of subhorizontal drains pushed into the toe of the slope is also reported by La Rochelle et al. (1977) with an evaluation of their beneficial effects; drainage has increased the factor of safety of the slope by 30%. Among the papers on stability analysis (Theme 2) presented to Session 11, two refer to drainage as a method of improving stability (Dysli & Recordon; Manfredini et al.). It is also worth noting that a paper by Deniau et al. presented to Session 3 describes a new technique to develop a deep drainage trench into the soil by digging a slurry trench with a bio-degradable slurry mixed with clean sand; this is found to be a very promising technique.

Another means of improving the stability of slopes consists of driving piles across the unstable mass down to the underlying stable soil; the location of the potential slip surface and the shear force needed to increase the safety factor by a given amount, is usually accomplished by performing conventional stability analyses. Viggiani discusses the mechanisms of failure of the piles based on the Broms' concept evaluating the yield value of the soil-pile interaction in the case of cohesive soils. The Author considers three modes of failure for both the rigid pile and the plastically hinged piles, for which the formuli and corresponding design charts were developed. Agreement between the theoretical expressions and full scale measurements taken from the literature appears to be qualitatively satisfactory. A similar type of protective measure is described by Huder & Duerst, consisting of a tieback intermittent pile wall that was built at the entrance of the Born's railroad tunnel in Switzerland. Since raising of the piezometric level can induce a critical

stability condition, geodetic surveys as well as tilt and pore pressure measurements are periodically performed to prevent an accident; drainage and dewatering systems were considered not effective in this case.

The photo documentation presented by Berntson & Lindh on the Tuve slide near Gothenburg, Sweden, illustrates several of the stability measures that were taken in risk areas around the landslide, to protect private property from damage. Among these measures, should be mentioned: 1) the excavation of trenches through the slide mass down to bedrock or frictional soil, subsequently filled with rockfill; 2) sheet pile walls anchored with tierods in the bedrock, and 3) circular sheet pile walls combined with limestone columns to help reconsolidation of the clay. The Authors comment that the most suitable method was the buttressing of the landslide edge with rockfill placed in a previous excavation carried down to bedrock or to frictional soils.

Cartier et al. report on the studies recently undertaken in France to analyze the performance of railroad embankments in view of the increased frequency of maintenance and repair works. Most of these earthfills were built during the last century with equipment and specifications that are far from acceptable by current standards. Based on field observations and records of the rail levelling in a number of sections of the railway system, a classification of the problems associated with the stability of these embankments was attempted. From this study, it was concluded that: 1) most of the detected instabilities are related to the nature of the materials used in construction, to the action of rainwater on the top of the embankment, and to the erosion and infiltration of water at the bottom of the fill due either to its proximity to a river or to a lateral borrow poorly drained, and 2) the best indicator of the embankment stability or instability is the horizontal displacement of the upper part of the earthfill due to plastic deformation induced by repeated loading. The accumulated levelling corrections of the rails plotted in terms of time, also give a good indication of the earthfill behaviour.

After reviewing the recent technical literature, the Reporters feel that there has been a growing emphasis placed on the use of heavy works to prevent or control landslides. During the last four years, many examples have been described of buttressed retaining walls, anchored sheet piles walls, caissons, piled walls, piles, rock dykes, elaborate drainage works, etc... It might be useful to look more closely, in the near future, at the medium-term performance of these structures and at their cost/benefit ratio.

FINAL COMMENTS

Following their extensive review of all the papers presented to this Session, the Reporters have acquired certain perspective views which they would like to share with the readers for the benefit of discussion. They consider that these papers give an important amount of valuable data which will contribute to improve our knowledge of slope behaviour. It has become obvious that much work remains to be done on a regional basis and that the accumulated knowledge cannot be generalized to all geographical areas. The Reporters certainly do not share the pessimistic views that our knowledge on slope stability problems has not improved in spite of all the efforts and expenses. Some of the papers presented to this Session deal with the fundamental aspects of slope stability, and others present case histories supported by field observations; both of these categories of papers are leading to progress.

One of the most important responsibilities of the geotechnical engineer involved in landslide problems is the detection and classification of risk areas:

- Judgment on the stability of natural slopes in a given region can be based on many data, the first one to be considered being the geomorphology of the area.

- In order to help a better evaluation of the stability of a slope in a given area, the collection and processing of long-term data on rainfall and its influence on the pore pressure in different terrains should be considered essential. Some special attention should also be devoted to artesian pressure.

- Stratigraphical discontinuities and their implications on the stability of the slope should be investigated most carefully. However, it should be realized that such a feature may be neither a sufficient nor a necessary condition for a landslide to occur.

The limit equilibrium methods of stability analysis satisfying the equations of static equilibrium and also Bishop's simplified method are sufficiently accurate to be used as a valuable engineering tool both for homogeneous or heterogeneous soils. The FE method is also a valuable tool but only for defining qualitatively the stress-strain behaviour of the soil mass before failure; however, field measurements can help to quantify approximately the computed results.

There is more and more evidence tending to show that the effective stress analyses are more reliable than the total stress, even for cases of short-term stability.

According to accumulated evidence, of the three levels of shear strength which are considered in this report in the case of strain-softening materials (peak, fully-softened or large strain, and residual), the peak strength does not seem to be mobilized in nature. For the first-time slide, the mobilized strength on the failure surface at failure corresponds to the "fully-softened" value in overconsolidated clays, and to the "strength at large strain" in normally consolidated clays; this implies that the strain compatibility does not need to be taken into account for all practical purposes. The residual strength only applies to reactivation of old failure surfaces in overconsolidated, fissured clays and clay shales.

Of all the factors identified as contributing to the instability of slopes, the most recurrent and important one is the increase in pore pressure; this may result from natural climatic conditions such as rainfall and snowmelt, or from other intervention such as human activity. It should be emphasized that pile driving can cause failure of a slope in certain soils.

No generalization of approach to the problem of slope stability will ever be possible, as too many variables are involved. Each problem has to be studied within its regional context. However, the basic principles and the proven experiences accumulated in the field of geotechnical engineering may facilitate the transfer of valuable approaches between geotechnical regions. In that perspective, the Reporters believe that the contributions to this Session have served a useful purpose.

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REFERENCES

Abbreviations

CSMFE: Conference of Soil Mechanics and Foundation Engineering.
ASCE : American Society of Civil Engineers
JSMFD: Journal of the Soil Mechanics and Foundation Division
JGED: Journal of the Geotechnical Engineering Division
ICOLD: International Commission on Large Dams

Alberro, A.J., Moreno, G.E. 1981. Interaction phenomena in Chicoasen Dam: construction and first filling. Submitted to the XIV ICOLD, Brazil.

Baligh, M.M., Azzouz, A.S. 1975. End effects on the stability of cohesive slopes. ASCE, JGED, Vol. 101(GT-11): 1105-1117.

Becker, E., Chan. C., Seed, H.B. 1972. Strength and deformation characteristics of rockfill materials in plane strain and triaxial compression tests. Report No. TE-72-3 Dept. of Civil Eng., University of California, Berkeley.

Bishop, A.W. 1966. The strength of soils as engineering materials, Géotechnique, Vol.16(2): 89-130.

Bishop, A.W. 1974. Discussion, Session 4, Interpretation Proc. Symp. on Field Instrumentation in Geotechnical Eng. Halsted Press Book. London, England:666-672.

Bjerrum, L., Løken, T., Heidberg, S., Foster, R. 1969. A Field study of factors responsible for quick clay slides. Proc. 7th ICSMFE, Vol.2:531-540.

Bjerrum, L. 1972. Embankment on soft ground. Proc. Specialty Conf., Performance of Earth and Earth Supported Structures, ASCE, Vol. 2:1-54.

Bjerrum, L. 1973. Problems of soil mechanics and construction on soft clays. Proc. 8th ICSMFE, Moscow, Vol. 3:11-159.

Carson, M.A. 1979. Le glissement de Rigaud du 3 mai 1978. Geogr. Phys. Quat., Vol.33(1):63-92.

Comision Federal de Electricidad (CFE) 1980. Performance of El Infiernillo and La Villita Dams, including the earthquake of March 14, 1979. CFE Pub., Mexico.

Duncan, J.M., Wright, S.G. 1980. The accuracy of equilibrium methods of slope stability analysis. Proc. of the Int. Symp. on Landslides, New Delhi, Vol.1:247-254.

Eden, W.J. 1977, Evidence of creep in steep natural slopes of Champlain sea clay. Can. Geot. Journ., Vol. 14(4):620-677.

Fredlund, D.G., Krahn, J. 1977. Comparison of slope stability methods of analysis. Can. Geot. Journ., Vol. 14(3):429-440.

Fukuoka, M. 1979. Causes of landslides - earthquakes, rain and pore water pressures. Proc. 6th Asian Regional CSMFE Singapore, Vol.1:221-224.

Fumagalli, E. 1969. Tests on cohesionless materials for rockfill dams. Proc. of the ASCE JSMFD, Vol.95(SM1): 313-330.

Golder, H.Q. 1979. The lost assumption, Géotechnique, Vol. 24(4):480-482.

Hang, M.D., Sauer, K., Fredlund, D.G. 1977. Retrogressive slope failure at Beaver Creek, South of Saskatoon, Saskatchewan, Canada. Can. Geot.Journ., Vol.14(3): 288-301.

Inganäs, J., Viberg, L. 1979. Inventering av Lerskred i Sverige. Proc. Nordic Geotechnical Meeting, Helsinki, :549-556.

Janbu, N. 1954. Stability analysis of slopes with dimensionless parameters. Harvard Soil Mech. Series No. 46:81 p.

Janbu, N. 1967. Discussion on "Dimensionless parameters for homogeneous earth slopes". By J.M. Bell, ASCE, JSMFD, Vol. 93(SM&):367-374.

Janbu, N., 1973. Geotekniske Stabilitetsanalyser. Institute for Geoteknikk of Fundamenteringslaere, Trondheim, Norwegian Institute of Technology, Bulletin No.7.

Jørstad, F.A. 1968. Leirskred i Norge. Norsk Geografisk Tidsskrift, Bind 22, Hefte 3:214-219.

Jørstad, F.A. 1970. Leirskred i Norge, Norwegian Geotechnical Institute, Publikasjon No.83, 1-6. Oslo.

Kankare, E. 1969. Failures at Kimola floating canal in southern Finland. 7th ICSMFE, Mexico, Vol.2:609-616.

Kenney, T.C.., Pazin, M., Choi, W.C. 1977. Design of horizontal drains for soil slopes. ASCE JGED, Vol.103 (GT11):1311-1323.

Kenney, T.C., Uddin, S.1974. Critical period for stability of an excavated slope in clay soil. Can.Geot. Journ. Vol.11(4):620-623.

Kondner, R.L. 1963, Hyperbolic stress-strain response: cohesive soils. ASCE JSMFD, Vol.89(SM1):115-143.

Ladd, C.C., Foott, R. 1974. New design procedure for stability of soft clays. ASCE JGED, Vol.100(GT7):763-786.

Lafleur, J., Lefebvre, G. 1980. Groundwater regime associated with slope stability in Champlain clay deposits. Can.Geot.Jour., Vol.17(1):44-53.

La Rochelle, P. 1960. The short-term stability of slopes in London clay. Ph.D. thesis, Imperial College, University of London, England.

La Rochelle, P., Chagnon, J.Y., Lefebvre, G. 1970. Regional geology and landslides in the marine clay deposits of eastern Canada. Can.Geot.Journ. Vol. 7(2):145-156.

La Rochelle, P., Lefebvre, G. 1971. Sampling disturbance in Champlain clays. Sampling of Soil and Rock, ASTM STP 483:143-163.

La Rochelle, P., Trak, B., Tavenas, F., Roy, M.1974. Failure of a test embankment on a sensitive Champlain clay deposit. Can.Geot.Journ., Vol.11(1):142-164.

La Rochelle, P. 1975. Causes and mechanisms of landslides in sensitive clays with special reference to the Québec area. Proc. Mass Wasting, 4th Guelp Symp. on Geomorphology:173-182.

La Rochelle, P., Lefebvre, G., Bilodeau, P.M. 1977. The Stabilization of a slide in Saint-Jérôme, Lac Saint-Jean. Can.Geot. Journ., Vol. 14(3):340-356.

Larsson, R. 1980. Undrained shear strength in stability calculation of embankments and foundations on soft clays. Can.Geot.Journ., Vol.17(4):591-602.

Lebuis, J., Rissman, P. 1979. Earthflows in the Québec and Shawinigan areas. Geological Association of Canada Congress, Québec, Brochure for field trip B-11.

Lefebvre, G., La Rochelle, P. 1974. The analysis of two slope failures in cemented Champlain clays. Can.Geot. Journ., Vol. 11(1):89-108.

Leonard, G.A. 1979. Stability of slopes in soft clays. Proc. 6th Pan. Am. CSMFE, Vol.1:223-275.

Leonard, G.A. 1980. Investigations of failures. 16th Terzaghi lecture, Hollywood, Florida. To be published in ASCE JSMFD.

Lo, K.Y. 1970. The operational strength of fissured clays. Géotechnique, Vol.20(1):57-74.

Lo, K.Y. 1972. An approach to the problem of progressive failure. Can. Geot. Journ., Vol.9(4):407-429.

Marachi, N.D., Chan, C., Seed, H.B., Duncan. J. 1969. Strength and deformation characteristics of rockfill materials. Report No. TE-69-5, Dept of Civil Eng., University of California, Berkeley.

Marsal, R.J. 1973. Mechanical properties of rockfill. Casagrande Volume, Embankment-Dam Engineering, John Wiley and Sons, N.Y.:109-201.

Marsal, R.J. 1977. Research on granular materials. Contribution of the Instituto de Ingenieria, UNAM to the 9th ICSMFE, Tokyo.

Marsal, R.J. 1979. Stability investigations related to clay shales. Int. Symp. on Soil Mech., Oaxaca, Vol. I:51-74.

Marsal, R.J., Moreno, G.E. 1979. Investigations on the design and performance during construction of Chicoasen dam, Mexico. CFE contribution to the XIII ICOLD, India.

Marsal, R.J., Ramirez de Arellano, L. 1967. Performance of El Infiernillo Dam. 1963-68, ASCE,JSMFD, Vol.93:265-298.

Mesri, G. 1975. Discussion on New design procedure for stability of soft clays. ASCE, JGED, Vol.101(GT4): 409-412.

McRoberts, E.C., Morgenstern, N.R. 1974. The stability of thawing slopes. Can. Geot.Journ., Vol.11(4):447-469.

Mitchell, R.J., Eden, W.J. 1972. Measured movements of clay slopes in the Ottawa area. Canadian Journ. of Earth Sciences, Vol.9:1001-1013.

Moreno, G.E., Alberro, A.I. 1981. Behaviour of Chicoasen dam: construction and first filling. Submitted to the XIV ICOLD, Brazil (in press).

Morgenstern, N.R., Blight, G.E., Janbu, N., Resendiz,D. 1977. Slopes and excavations. State-of-the-Art report to Session 3. 9th ICSMFE, Tokyo, Vol.2:547-604.

Morgenstern, N.R., Price, V.E. 1965. The analysis of the stability of general slip surfaces, Géotechnique, Vol. 15(1):1-31.

Nishida, Y., Yagi, N., Futaki, M. 1979. The theoretical analysis on the pore pressures due to rain water permeation in the ground, Proc. 6th Asian Conf. on SMFE, Singapore, Vol. 1:241-244.

Pagano, N.J., Halpin, J.C. 1968. Influence of end constraint in the testing of anisotropic bodies. Journ. of Composite Materials, Vol.2(1)18-31.

Phukan, A.L.T., Lo, K.Y., La Rochelle, P. 1970. Stresses and deformations of vertical slopes. Proc.11th Symp. on Rock Mech., Berkeley.:193-212.

Quigley, R.M., Gélinas, P.J., Bou. W.T., Packer, R.W. 1977. Cyclic erosion-instability relationships: Lake Erie north shore bluffs. Can. Geot. Journ., Vol.14 (3):310-323.

Rib, H.T., Liang, T. 1978. Recognition and identification. Landslides analysis and control, Special Report 176, Edited by R.L. Shuster & R.J. Krizek, Transportation Research Board, Washington D.C., Chapter 3:34-80.

Resendiz, D. 1974. Discussion on Accuracy of equilibrium slope stability analysis. ASCE JGED, Vol.100 (GT8):967-970.

Rowe, P.W. 1972. The relevance of soil fabric to site investigation practice. Géotechnique, Vol.22(2):193-301.

Saada, A.S. 1970. Testing of anisotropic clay soils. ASCE JMSFD, Vol.96(SM5):1847-1852.

Skempton, A.W. 1977. Slope stability of cuttings in Brown London clay. Proc. 9th ICSMFE, Tokyo, Vol. 3: 261-270.

SRH-CFE-UNAM. 1976. Behaviour of dams built in México. Contribution of the Secretaria de Recursos Hidraulicos (SRH) Comision Federal de Electricidad (CFE) y Universidad Nacional Autonoma de México (UNAM), to the XII ICOLD, México.

Tavenas, F., Chagnon, J.Y., La Rochelle, P. 1971. The St-Jean-Vianney landslide: observations and eyewitnesses accounts. Can. Geot. Journ., Vol.8(3):463-478.

Tavenas, F., Leroueil, S. 1977. Effects of stresses and time on yielding of clays. Proc. 9th ICSMFE,Vol.1, Tokyo." 319-326.

Tavenas, F., Leroueil, S. 1979. Clay behaviour and the selection of design parameters. Proc. 7th European CSMFE, Brighton, Vol.1:281-291.

Tavenas, F., Leroueil, S. 1981. Creep and failure of slopes in clays. Can. Geot. Journ., Vol.18(in press).

Tavenas, F., Leroueil, S., La Rochelle, P., Roy, M. 1978. Creep behaviour of an undisturbed lightly overconsolidated clay. Can. Geot. Journ., Vol.15(3): 402-423.

Tavenas, F., Trak, B., Leroueil, S. 1980. Remarks on the validity of stability analyses. Can. Geot. Journ., Vol.17(1):61-73.

Ter-Stepanian, G. 1979. Rheological basis of mechanism of slope failures. Proc. Int. Symp. on Soil Mech., Oaxaca, Vol.1:155-160.

Ter-Stepanian, G. 1980. Creep on natural slopes and cuttings. Proc. Int. Symp. on Landslides, New Delhi, Vol.2:95-108.

Trak, B., La Rochelle, P., Tavenas, F., Leroueil, S., Roy, M. 1980. A new approach to the stability analysis of embankments on sensitive clays. Can. Geot. Journ., Vol.17(4):526-544.

Vaid, Y.P., Campanella, R.G. 1977. Time-dependent behaviour of undisturbed clay. ASCE JGED, Vol.103 (GT7):693-709.

Varnes, D.J. 1978. Slope movement: types and processes. Landslides analysis and control, Special Report 176, Edited by R.L. Schuster and R.J. Krizek, Transportation Research Board, Washington D.C., Chapter 2:11-33.

Williams, D.R., Romeril, P.M., Mitchell, R.J., 1979. Riverbank erosion and recession in the Ottawa area, Can. Geot. Journ., Vol. 16(4):641-650.

Wilson, S.D., Marsal, R.J. 1979. Current trends in design and construction of embankment dams. ASCE publications, N.Y.

Wright, S.G., Kulhawy, F.H., Duncan, J.M. 1973. Accuracy of equilibrium slope stability analysis. ASCE, JSMFD, Vol. 99(SM10):783-791.

Yamanouchi, T., Mochinaga, R., Gotoh, K., Murata, H. 1978. Studies of cut-off slopes in a pumice-flow soil deposit and their applications to the design standards for an expressway. Proc.9th Int. CSMFE, Tokyo, Case History Volume:61.

Yong, R.N., Alonso, E., Tabba, M.M., Fransham, P.B. 1977. Application of risk analysis to the prediction of slope instability. Can. Geot. Journ., Vol. 14(4): 540-553.