Session 11

Slope Stability

Stabilité des Pentes

Chairman	N. Janbu (Norway)
Co-Chairman	J.N. Hutchinson (UK)
General Reporter	R.J. Marsal (Mexico)
Co-Reporter	P. LaRochelle (Canada)
Technical Secretary	L. Viberg (Sweden)
Panelists	P. Anagnosti (Yugoslavia), G. Pilot (France), E. Slunga (Finland), W. Wolski (Poland)

N. Janbu, Chairman

INTRODUCTION

The Chairmanship, the General Reporters and the Panelists have decided to conduct the discussion periods around five questions:

1. How would you analyse the long-term stability of a natural slope in clayey soil? (Give: Type of analysis, strength parameters, testing methods)
2. How would you analyse the short-term stability of a cut slope in clayey soil? (Give: Type of analysis, strength parameters, testing methods)
3. In your opinion and experience, what are the merits and defects of the limit equilibrium and the finite element methods to investigate the stability of man-made earth structures (embankments) for:
   - non-homogenous type of sections?
   - embankments built on soft, compressible foundations?
4. From your regional experience, what are the main factors which should be taken into account in establishing a landslide hazard zoning?
5. From your regional experience what would be your first choice of means to:
   - give warning of a landslide?
   - prevent a landslide?
   - correct a landslide?

The ground rules for discussion are as follows: We will deal with one question at a time, from 1 to 5. A prepared panel contribution is given first, then an interchange of views between panelists and officers, before inviting floor discussers. In addition, to prepared floor discussions we sincerely hope for spontaneous reactions, to try to make the session as much alive as possible.

G. Pilot, Panelist, and P. Pouget

CONSTRUCTION DE REMBLAIS SUR PENTES ARGILEUSES
Embankment Construction on Clay Slopes

La construction de remblais sur versants naturels argileux plus ou moins stables est une nécessité dans la construction des infrastructures de transport. Cette opération se révèle difficile et génératrice d'importants glissements de terrains.

L'étude de la stabilité de tels ouvrages pose un certain nombre de problèmes, parmi lesquels :

- quels paramètres de résistance au cisaillement employer pour étudier la stabilité du versant naturel seul ?
- comment identifier l'état de stabilité d'un versant ?
- comment calculer la stabilité du remblai construit sur un versant à la limite de la stabilité ?

La réponse à la première question, dans le cas des argiles raides surconsolidées, fréquemment rencontrées en France, est bien connue : la nécessité de tenir compte des pressions interstitielles comme facteur essentiel de la stabilité, oblige à faire l'analyse de la stabilité en contraintes effectives donc d'utiliser les paramètres effectifs de résistance au cisaillement. Dans le cas de versants argileux susceptibles de subir un premier glissement ce sont les caractéristiques de pic d'un essai consolidé drainé qui sont représentatives, avec cependant une restriction sur le terme c' dont la valeur apparaît parfois ainsi surestimée. Dans le cas des versants naturels ayant déjà glissés, les caractéristiques résiduelles d'un essai consolidé drainé à la boite de cisaillement (alterné ou par rotation) représentent la résistance au cisaillement (avec $c'_r = 0$) mobilisée le long de la surface de glissement (SKEMPTON 1964). On a eu la possibilité d'instrumenter en France avec des piézomètres et inclinomètres plusieurs sites instables d'argile raide, notamment dans les argiles du Lias et de confirmer que ce sont bien les valeurs résiduelles de la résistance au cisaillement qui permettent d'expliquer l'instabilité (BLONDEAU et al 1977)

En ce qui concerne l'identification de l'état de stabilité d'un versant il est bien évident que les seuls indices géomorphologiques annoncent souvent l'activité de certains glissements de pentes naturelles. Les cas les plus difficiles concernent les formations argileuses dont l'histoire géologique suggère l'instabilité mais dont les indices géomorphologiques sont insuffisants pour attester l'existence d'un glissement actif. Dans de tels cas les mesures inclinométriques réalisées avec les appareils récents, très précis permettent de détecter assez rapidement de faibles mouvements profonds dont l'incidence en surface

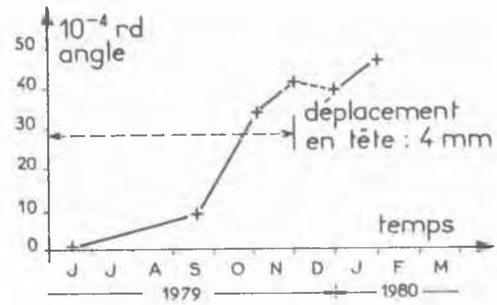

Figure 1 - Mesure de la rotation d'un tube inclinométrique à 6,50 m de profondeur dans le sol du site de Sallèdes.

est peu perceptible. Suivi, sur le versant de Sallèdes (Puy de Dôme) qui accueille les remblais expérimentaux des Laboratoires des Ponts et Chaussées, on a mis en évidence une zône de rupture à 6,5 m de profondeur ; la figure 1 montre les rotations mesurées en fonction du temps à ce niveau avec un inclinomètre LPC dont la précision est de l'ordre de quelques 10^{-4} radian : une rotation de 40 x 10^{-4} radian en 6 mois révèle le mouvement sans ambiguité alors que le déplacement en tête du tube, 4 mm pendant cette même période passe inaperçu.

La question du calcul de la stabilité d'un remblai construit sur versant argileux à la limite de stabilité demeure difficile ; elle pose en effet les problèmes suivants : quelles sont les surpressions interstitielles générées par le chargement du remblai ? Le calcul doit-il être mené en contraintes totales ou en contraintes effectives et quels paramètres de résistance au cisaillement implique-t-il ?

Pour apporter des éléments de réponse à ces questions on a construit sur le site de Sallèdes mentionné ci-dessus un remblai expérimental édifié jusqu'à la rupture (BLONDEAU-MORIN 1980).

La hauteur du remblai mesurée à la rupture sous la crête de talus, 5,35 m est à rapprocher des valeurs déterminées a priori selon les diverses méthodes mentionnées au tableau I : on note que ce sont les calculs en contraintes effectives effectuées à partir des caractéristiques résiduelles qui encadrent le mieux la valeur mesurée.

Un second remblai est en cours d'édification, par phases, afin de déterminer les paramètres d'alarme à retenir et de tenter de mieux maîtriser ces difficiles constructions (CARTIER-POUGET 1981).

Tableau I - Calculs prévisionnels de stabilité ; hauteur à la rupture du remblai de Sallèdes selon diverses méthodes.

calcul	mesure	non circulaire	circulaire
Contraintes totales	scissomètre (s_u)	12 m	16 m
	triaxial (C_u)	6 m	8 m
Contraintes effectives	triaxial (c', ϕ')	-	7-10 m
	boite (c'_r, ϕ'_r)	6 m	4- 7 m

Références :

CARTIER G., POUGET P., (1981). Paramètres d'alarme pour les ouvrages construits sur versants instables. C.R. 10è Congrès Inter. Méc. des Sols et Travaux de Fond. STOCKHOLM. Vol. 4.

BLONDEAU F., PILOT G., PERROT A. (1977). Etude en vraie grandeur des versants naturels instables C.R. 9è Congrès Inter. Méc. des Sols et Travaux de Fond. TOKYO. Vol.2, pp.21-24.

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G. Cartier and P. Pouget (Written discussion)

PARAMETRES D'ALARME POUR LES OUVRAGES CONSTRUITS SUR VERSANTES INSTABLES
Warning parameters for Constructions on Instable Slopes

La définition d'un matériel d'alarme et de ses conditions d'utilisation nécessite un choix préalable et une bonne connaissance du ou des paramètres à surveiller. Le problème se pose notamment lors de la construction ou de l'exploitation de remblais sur versants argileux instables pour lesquels on peut suivre, soit les paramètres hydrauliques qui se trouvent directement à l'origine des causes du glissement, soit la vitesse de déplacement du sol qui gouverne directement la sécurité de l'ouvrage.

Si la figure 1 montre qu'il existe effectivement une nette concordance entre les fluctuations des pressions interstitielles au niveau de la surface de glissement et l'évolution du phénomène, la corrélation n'est par contre pas simple à établir. Les mesures précédentes tendent par exemple à prouver que la vitesse des déplacements (V) entre deux instants est liée non seulement au niveau moyen de la nappe (u) mais également à l'amplitude et au sens (alimentation ou drainage) des variations de pression interstitielle (Δu) entre ces deux instants. Cela impose donc que l'on réalise un enregistrement continu des mesures (cf figure 2). Ceci étant établi, on constate alors une nette corrélation entre la vitesse des déplacements et le coefficient de sécurité au glissement de l'ouvrage calculé suivant une méthode classique.

Finalement, force est de constater que si l'existence d'une chaîne "précipitations-pressions interstitielles-glissement-désordres sur l'ouvrage" plaide pour un suivi des paramètres hydrauliques permettant de connaître l'évolution de la sécurité le plus en amont possible, les corrélations entre les différents maillons restent à établir de façon plus systématique à partir de constatations et mesures in-situ et,

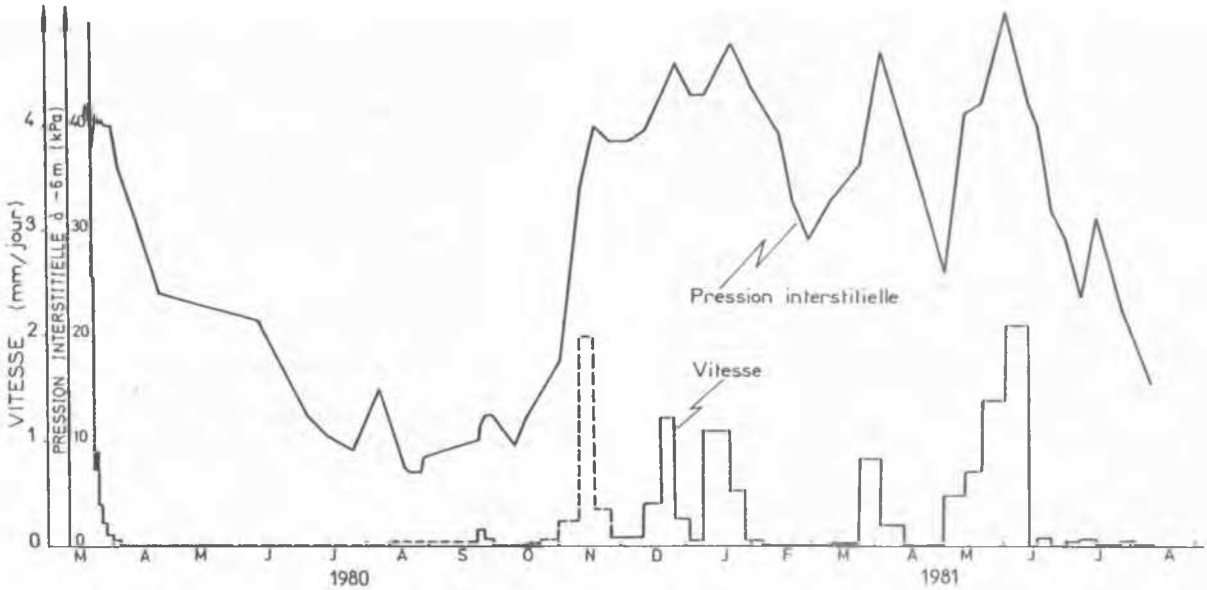

Fig.1 : Evolution dans le temps des déplacements et des pressions interstitielles

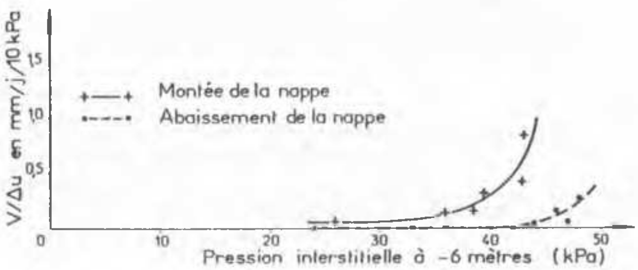

Fig. 2 : Relation entre la vitesse des déplacements et les conditions hydrauliques.

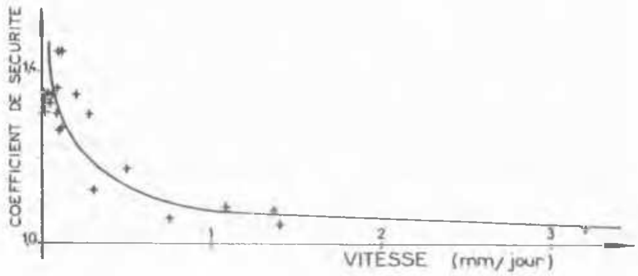

Fig.3 : Relation entre les déplacements et le coefficient de sécurité

dans l'avenir, grâce au développement de modèles rhéologiques plus performants.

Références :

BLONDEAU F., MORIN P. (1980) Rupture d'un remblai expérimental sur versant à Sallèdes, Bull. de Liaison des L.P.C., (106), 133-137.

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G. Aas (Oral discussion)

I would like to comment upon the proposal made by general reporters as to apply to so-called "large strain strength" in stability calculations of natural slopes in quick clays.

Let us look at a clay element along a typical potential sliding surface below a gently inclined slope in quick clay (Fig. 1). The directions of the principal stresses acting on this element are such that the most critical plane does not coincide with the potential sliding surface. However, if one could imagine a situation where all of the elements along the sliding surface were subjected to a stress condition corresponding to yielding shear stress along those critical planes, and at the same time the shear strength was fully mobilized in the active and passive zones of the potential sliding body, the clay layer would be free to deform like a huge shear test specimen and at the same time developing pore pressures.

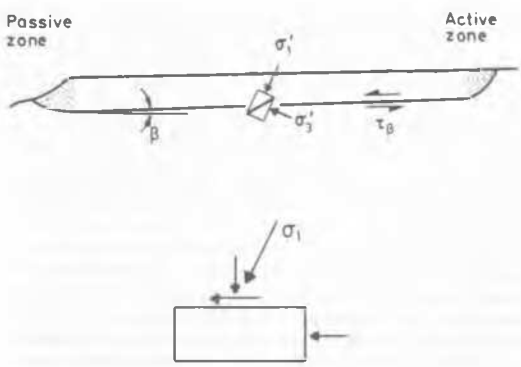


Fig. 1. Stress conditions along potential sliding surface in quick clay

Of course, a complete failure in sense of sliding would not be kinematically possible along those planes where yielding first took place. However, increasing pore pressures mean decreasing effective stresses, and if the rise in pore pressure was really dramatic, the clay would have to mobilize its full friction angle also along other planes, for instance those coinciding with the potential sliding surface, in order to compensate for the loss in effective stress in an effort to withstand the existing shear stresses.

As you will understand the critical value of shearing stress along the potential sliding surface is not a simple function of the friction angle of the quick clay and the effective normal stress on this plane just prior to failure. In contrary, it is entirely defined by the start of yielding along any plane and by the rate or extent of the subsequent rise in pore pressure.

As shown in my paper to this conference, yielding which initiates the structure collapse in a quick clay takes place for a mobilized friction angle substantially lower than the ultimate value of φ'. This is illustrated by an effective stress path from an undrained compression triaxial test in Fig. 2.

A relevant pore pressure parameter for this quite peculiar condition of increasing pore pressure at constant shear stresses and constant total stresses was found to be the ratio $\frac{\Delta u}{\sigma_0' \cdot \Delta \sin \varphi'_{Mob}}$ which seems to be a rather consistent soil parameter. This ratio is 4 - 8 times higher for a quick clay than for a clay of low sensitivity. The values observed for Norwegian quick clays are more than sufficient to provide for a completion of the structure collapse, although the ratio between shear stress and effective normal stress along the final sliding surface just prior to failure only amounting to 0.15 - 0.20 corresponding to a mobilized friction angle along this plane of the order of 10 degrees.

In my opinion, the critical shear stress along a potential sliding surface in quick clay could best be determined, either on the basis of the critical value of friction angle corresponding to peak shear strength in undrained triaxial tests - or from the undrained shear strength (τ_H) measured in direct simple shear tests. The Mohr-Coulomb-diagram in Fig.3 illustrates how the critical

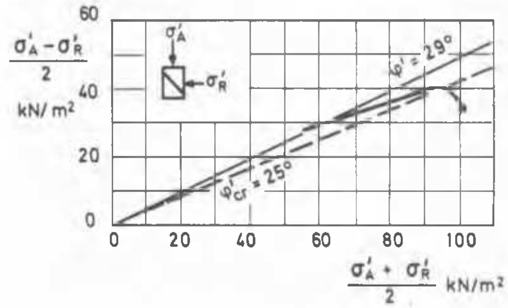
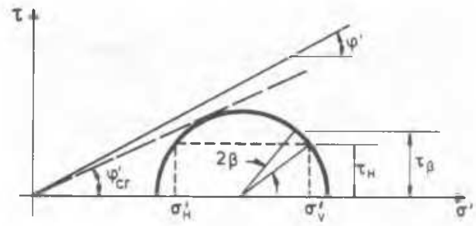


Fig. 2. Typical effective stress path for a quick clay

shear stress along a plane inclined β degrees depends upon φ'-critical or τ_H.



$$\tau_\beta = f_1(\varphi'_{cr}, \beta, \sigma_V', \sigma_H')$$

$$\tau_\beta = f_2(\tau_H, \beta, \sigma_V', \sigma_H')$$

Fig. 3. Mohr-Coulomb-diagram representing stress conditions on sliding surface

A critical shear stress determined in this way has been found to agree fairly well with the calculated average value of shear stress along the sliding surface in a number of Norwegian landslides.

B. Trak (Written discussion)

ON STABILITY ANALYSIS OF FLAKE-TYPE SLIDES IN QUIC-CLAY SLOPES

In a paper to Session XI, Åas presented interesting results of stability analyses of flake-type slides in quick, normally consolidated clay slopes in Scandinavia. As determined from total stress analyses, the average strength mobilized on the actual failure surface is related to the effective overburden pressure by a ratio varying between 0,15 and 0,20. Åas also indicates that the results obtained from a back-analysis of actual failures are very close to the one determined by direct simple shear (DSS) tests in the laboratory.

Although the failure conditions and mechanisms are probably very different in slopes and embankment foundations, it is surprising to note that similar results are obtained by back calculating the average mobilized strength at failure under embankments. As indicated by the General Reporter, Trak et al. (1980) have found that for inorganic sensitive clays of medium to high plasticity, the mobilized strength at failure was equal to 0,22 σ_p'. This relationship was originally proposed by Mesri (1975) who interpreted data given by Bjerrum (Fig.1). He found that the mobilized strength $\mu\, c_u$, where μ is the vane correction factor proposed by Bjerrum (1972), is related to σ_p' by a ratio of about 0,22 for clays with $I_p > 20\%$ and somewhat less for low plasticity clays. It is interesting to note that Åas gives similar values of mobilized strength for Scandinavian clays of low plasticity.

Fig.2 shows the theoretical safety factor at failure for cuts and unsupported excavations plotted against the plasticity index of the clay. On the same figure is shown the line derived by Bjerrum (1972) from a similar plot for embankment failures. It can be seen that this line satisfactorily represents the relationship between the safety factor and Ip for cuts and unsupported excavations. As the $c_u = 0{,}22\ \sigma_p'$ method was derived from this same line, it should also be applicable to the design of short-term cuts in soft clays.

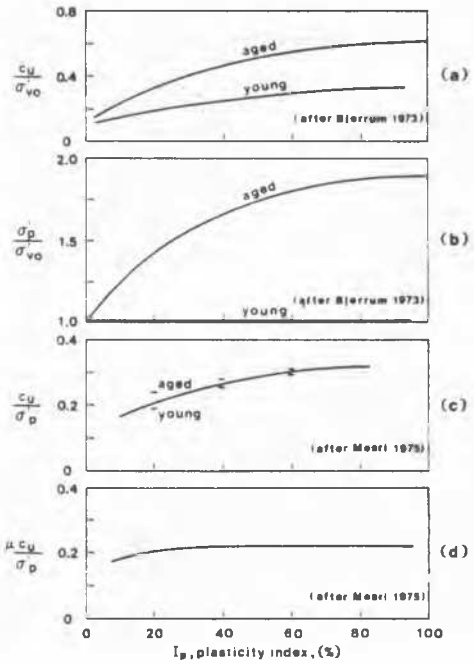


Fig. 1. Typical ratios for normally consolidated late glacial and postglacial clays.

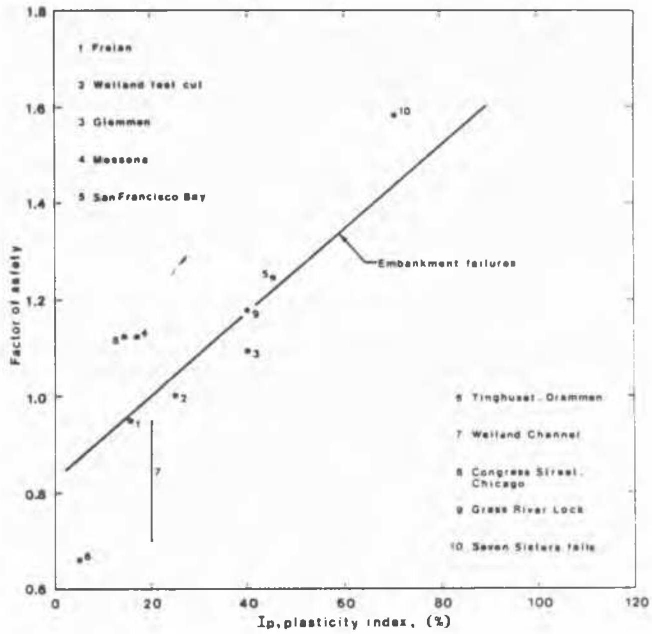


Fig. 2. Theoretical factor of safety at failure for cuts and unsupported excavations plotted against the plasticity index of the clay (After Bjerrum 1973)

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

G.A. Leonards (Oral discussion)

The General Reporters struck an optimistic note concerning the state-of-the-art for analyzing stability of slopes, in spite of some recently published "pessimistic papers". They did not refute the reasons for the "pessimism" reflected in the papers they cited, and offer as their raison-d'etre only the fact that back-analyses of failures often result in safety factors approximately equal to one. I cannot accept the rationale for this argument because the literature is replete with back-calculated safety factors equal to one although it is now known that some of the premises in the analysis were faulty. Moreover, a number of back-analyses giving safety factors both considerably greater than and less than one are also extant. However, the fact most damaging to their argument is that there is no evidence the number of unexpected failures is in any way abating. Consider the case of the landslide at Tuve (Berntson and Lindh, 1981; Jansson and Stål, 1981). Who among us, given the opportunity to investigate that site before the failure, would have had the conviction to insist on evacuating the public and to define the size of the area to be evacuated?

Our Norwegian colleagues have provided us with, perhaps, the first - and certainly the best - documented evidence of the anatomy of a landslide (in quick clay) by providing the fascinating movie shown during the Conference and by publishing two associated papers (Gregersen 1980 and 1981). A comparison between the results of stability analyses and what actually happened is presented below. To conserve space, reference will be made to Figures presented in Gregersen's papers.

The Rissa slide was initiated by dumping spoil from a small excavation to form a low embankment adjacent to Lake Botnen. According to the movie, this initial slide occurred very rapidly and slipped as a unit into the lake. However, as shown by Fig. 3 (Gregersen, 1980) the width of the filling constituted only about 30 percent of the width of the slide. Thus, although the slide was approximately 2-D, the loading was definitely 3-D; nevertheless, analysis of the slide assumed the loading was 2-D. As the SF was calculated to be 1.50 before the filling, and 0.95 after (Fig. 13, Gregersen, 1980), it could hardly be less than 1.25 if the loading were considered to be 3-D. Thus, invoking weighted shear strengths according to the relative lengths along the slip surface corresponding to active triaxial, simple shear, and passive triaxial, and approximating the slip surface by an equivalent circle, cannot be a correct representation of the mechanics of the slide -- although the back-calculated SF was ≃ 1.

The initial slide retrogressed slowly (for about 35 minutes) in a direction more or less parallel to the lake (Fig. 2, Gregersen, 1980 and Fig. 3, Gregersen, 1981). Then, suddenly, a large mass of soil - the largest single unit of the entire slide - first subsided and then slid in a direction perpendicular to that of the previous (and subsequent) retrogression. An analysis of this slide was not presented. Could it be shown that the direction of movement corresponds to the most critical stability condition? Furthermore, the entire slope did not slide; the energy of the moving mass was sufficient to push the soil in front of it into the lake. Could an analysis show that the actual slip surface was the most critical one? I have emphasized elsewhere (Leonards, 1979 and 1981), how dangerous it is to give explanations of only a portion of a failure. If the concepts used cannot explain all observed events, they have repeatedly been shown to be faulty.

I am convinced that important factors affecting stability of slopes (and embankments) in soft clays are not even considered by the approach taken in the present state-of-the-art. Some of these factors are summarized below:

1) Sedimented clay deposits are inherently "varved" (Leonards, 1977 and 1981). Thin seams with lower shear strengths are not uncommon yet current methods for obtaining the strength profile are not well adapted to locating them. Even when weak zones are detected by chance they are generally ignored; "representative" strengths are chosen by some "averaging" process and the analysis carried out using slip circles. For example, at Rissa, both the boring profile (Fig. 11) and the results of laboratory strength tests (Fig. 14, Gregersen, 1981) clearly show evidence of a weak seam at a depth of about 8 m. In the future the location and effects of thin weak seams will have to be accounted for in much better ways than they are at present; fortunately, the tools for doing this are at hand.

2) Even within seams that are comparatively "homogeneous", there are random variations in shear strength. These variations are such that for calculated safety factors of 1.2 to 1.5, based on mean strengths, the probability of failure could be as high as 1 in 10 (Leonards, 1977). This means that a failed slope could have a correct safety factor (based on mean strengths) of 1.25 to 1.50! Force fitting a safety factor of one to such situations can hardly be expected to yield consistent predictions of stability in other cases.

3) Much has been written about the existence of a critical shear stress at which the clay structure breaks down (see Aas, 1981 for recent review). Due to the accompanying increase in excess pore pressure, failure occurs in undrained shear at a value of τ_f/σ' which is less than tan ϕ', where ϕ' is the angle of shearing resistance mobilized in drained shear. The effect is more pronounced in highly sensitive soils, but it also occurs in less sensitive soft clays. While the concept is valid it does not go far enough, because clay structures can break down due to shear creep and/or secondary compression even if the stress level is not increased. For this reason it is necessary to think in terms of a critical combination of volumetric and shear strain at which the clay structure will break down (Leonards, et al., 1981). Admittedly, this creates a problem with the frame of reference for strains but the concept is necessary to understand why sudden undrained failures can occur, without changes in loading, in slopes that have been stable for long periods of time.

4) It is well recognized that stored strain energy in rocks and stiff clays can have profound effects on the behavior of the mass if the opportunity for release of the stored energy is provided (Bjerrum, 1967). I believe that release of strain energy is of no less importance in the behavior of soft clay masses. For example, lateral squeezing of a weak clay seam into the brook at Tuve probably triggered the initial slide in a direction perpendicular to that of squeezing, which established conditions necessary for the large retrogressive slide that ensued. A similar phenomenon may have caused subsidence and sliding of block "B" (Fig. 4, Gregersen, 1981) in a direction perpendicular to that along which the slide was previously retrogressing. In the movie an underwater "jet" moving rapidly out into Lake Botnen can be observed. It was as though a giant tube of toothpaste with its outlet below the water line was suddenly squeezed. Although I viewed the film five times, I could not establish the relation between the "jet" moving out into the lake and the state of sliding on the land. Perhaps study of the unedited film will reveal such a relation, and thereby throw light on the effects of sudden releases in strain energy on the mechanics of landslides. In any case, current methods of stability analysis do not consider the existing state of stress and strain prior to a change in loading, and hence, are inherently incapable of accounting for the development of tension cracks or for the development of progressive failure.

I object to labeling the identification of uncertainties in the present state-of-the-art "pessimistic". As every practicing engineer knows full-well, there is no escape from making prompt decisions when dealing with practical problems, irrespective of the uncertainties which are extant. However, I firmly believe that such decisions must be made in the light of experience within a given geologic region - experience that has been carefully digested in terms of local practice in site investigation, sampling, testing, and analytical procedures - and that better judgments can be made if the engineer is more aware of the nature of the uncertainties involved than if he is less aware of them. It is a sobering thought for all of us to realize that virtually all of this experience in the realm of slope stability has been garnered from back analyses on the basis of interpretations made from the slide debris, and not on the movements that initiated the instability. There is nothing to gain, and much to lose, from promulgating the view that present analyses are, in themselves, adequate for design purposes!

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P. La Rochelle, Co-Reporter (Written discussion)

In his discussion on the General Report of session 11 on Slope Stability, Professor G.A. Leonards has raised some objections to a comment made by the Reporters concerning the pessimism of some recently published papers. Unfortunately, the brisk reaction of the discusser was spurred by a misinterpretation of the Reporters' comments. It is always a challenge for a writer to make comments in such a way that there remains no possibility of misunderstanding; it is as great a challenge for an involved reader not to interpret these comments in his own perspective.

The Reporters would like to refer the Discusser to the exact text of the General Report where it is quite clear that the comments on "pessimism" are directed only to those statements "on the lack of improvement of our design methods and óf our capacity of investigate failure". On the basis of their personal practical experience, the Reporters cannot concede that the numerous studies and the tremendous amount of observations being made on slope stability problems are useless; such a contention would be not only defeatist but contrary to reality. It is our opinion that we are progressing "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".

A careful reading of the General Report will permit a reader to judge whether it is justified to contend that the "Reporters struck an optimistic note concerning the state-of-the-art for analysing stability of slopes". On the contrary, the General Report presents the numerous criticisms addressed to the methods of slope stability analysis and states that it is indeed puzzling to realize that, with all their apparent defects, these methods of stability analysis yield results which seem to reflect reality. In other words, there are no fundamental reasons why these methods of analysis should give "the" good result; nevertheless, one has to admit that, when properly calibrated on regional conditions and with local procedures, they can become a valuable engineering tool if used with good engineering judgment. In that sense, we have to concede that these methods have some potential as well as limitations.

There are many embankment dams and many natural and excavated slopes which have been analysed with success in different parts of the world using these methods. The fact that frequent failures occur does not imply that the approach as a whole is useless; but it means that our degree of confidence cannot and will certainly never be 100%. One has to bear in mind that for each spectacular failure reported in the literature, there may be several successfully designed or analysed slopes performing so satisfactorily that their history is not worth publishing.

The Discusser quite rightly stresses the fact that there are many important factors affecting the stability of slopes in clay masses which often are not even considered by the present methods of analysis. The Reporters agree that the force fitting of a safety factor of one to cases of failure of such masses should not be used as a basis to explain the behaviour of the soil unless all the factors have been taken into account; and for most cases, this is beyond the possibilities of the present state-of-the-art. There is no doubt that such cases as the Tuve and Rissa landslides defy by their extent and their complexity the state-of-the-art of stability analysis.

Ten years ago, the senior Reporter was in charge of the geotechnical study of a large, disastrous and deadly flowslide in a sensitive clay in Saint-Jean-Vianney, Québec; like the Rissa and Tuve landslide, the Saint-Jean-Vianney flowslide is a case where nobody could have predicted the evolution and extent of the slide. From that extensive study, however, it could be suggested that, if the opportunity had been given to make a detailed study of the slope of the river bank which failed and triggered the flowslide it might have been possible to establish that the stability of the slope was close to limit equilibrium; but again, nobody could have inferred that the safety of the villagers located at more than 600 m away was threatened.

In this particular case, a stratigraphical characteristic was partly responsible for the spreading of the flowslide over a very large area, and this was indeed a very significant factor to consider in the evolution of that slide. Ever since Terzaghi has stressed the importance of the "minor geological detail", the possible occurrence of anomalies has been a major concern for experienced engineers dealing with the stability of clay masses. On many occasions during the last few years, Professor Leonards has greatly contributed to our understanding of stability by reminding this fact with due insistence. We now have powerful tools, as the piezocone, to locate seams and other discontinuities in clay masses, but it is not always possible to clearly appraise their influence on the stability.

In spite of all our efforts these discontinuities may remain elusive; however, we must be careful not to succumb to the "anomaly syndrome" and decide that it is useless to perform stability analyses because there may always be a probability that somewhere in the clay mass a stratigraphical discontinuity may be present and invalidate the analysis. Are we gaining much in promulgating that present analyses are, in themselves, useless for design purposes? The Reporters think that, on the contrary, the geotechnical engineers would be losing much by accepting that hypothesis.

R. Lundström (Oral discussion)

DEVELOPMENT OF THE TUVE LANDSLIDE WITHIN THE PASSIVE ZONE

Comparison between the Surte and Tuve landslides

The landslide at Surte, which occurred on the 29 September 1950 and that at Tuve on the 30 November 1977 show many similarities. The most obvious similarities are the initial slides, from which slides developed both backwards, up the slope, and forwards, down the slope. The factor of safety in the area in which the initial slide took place was close to 1.0. The initial landslide was triggered by the actions of Man, in combination with unfavourable natural conditions. The porewater pressure has had an adverse effect in both cases. The landslide at Surte was probably triggered by piling. At Tuve, residential development in combination with heavy traffic and resulting shock waves through the clay have been initiating factors in the presence of high pore-water pressure.

In a comparison, it should also be noted that the extent of the slides in both cases was large, which may be partially due to the incidence of quick clay in certain parts. What may require special explanation and has been discussed from different theoretical points of view is how the large extent of the forward-moving or passive zone could develop. The Surte landslide moved down to the River Göta Älv from the initial slide area, a distance of about 350 m. The distance between the lower limit of the initial slide at Tuve and the most distant limit of the passive zone is between 350 and 400 m.

In the report on the Surte slide in the Geological Survey of Sweden (SGU) (Paper Ser Ca No. 27) "The Landslide at Surte on the River Göta Älv" (C Caldenius and R Lundström), the undersigned explains how the development of the landslide within the passive zone could take place. The large kinetic energy which the sliding masses received from the initial slide and the secondary slides were tranformed into work; the passive earth pressure of the soil mass ahead of the sliding masses was exceeded and the sliding masses were displaced and lifted.

The kinetic energy of the active soil masses

The kinetic energy of a mass in movement is $\frac{m \cdot v^2}{2}$ in the SIS-system and $\frac{m \cdot v^2}{2\ g}$ in the MKSA-system

where

m is the weight of the mass in Newton (N) and kilogram (kg) respectively

v is the velocity of the mass in m/s and

g is the acceleration due to gravity, expressed in m/s^2 (= 9.80665)

In the following the figures in the MKSA-system will be put in brackets.

The kinetic energy is consequently a work according to the dimensions. In addition to a certain elastic compression or fragmentation of the masses, the kinetic energy was converted into work in the form of friction forces acting along the length of the retardation movement.

In the cases of Surte and Tuve, the masses from the initial slides and the secondary slides received large kinetic energy, whereas the masses ahead of them in the passive zone were at rest when the initial slides started.

According to an estimate, the velocity of the soil masses in the Surte slide amounted to 1.5 m/s (page 55 in the SGU paper Ca No. 27). The velocity of the soil masses in the Tuve slide is given as walking speed, or 1.5-2.0 m/s.

If we assume that the bottom of the slide lay at a depth of 12 m below ground level, the soil masses in the initial Tuve slide can be estimated at approx. $2.4 \cdot 10^5$ m^3 or approx. $3.8 \cdot 10^6$ kN ($3.8 \cdot 10^5$ t). This assumes that the initial slide involved an area of about $135 \cdot 150$ m^2, which is justified from descriptions of how the slide developed. The soil masses of the secondary slides in the active zone can be estimated at approx. $4.5 \cdot 10^5$ m^3 or approx. $7.2 \cdot 10^6$ kN ($7.2 \cdot 10^5$ t) or totally with the initial slide $11.0 \cdot 10^6$ kN ($11.0 \cdot 10^5$ t). See Fig. 4.

If we assume that the bottom of the slide lay at a depth of 30 m within the area of the initial slide and that the secondary slides in the upper part lay at a depth of 12 m, the masses within the initial slide area may be estimated at approx. $6.1 \cdot 10^5$ m^3 or approx. $9.7 \cdot 10^6$ kN ($9.7 \cdot 10^5$ t) and for the secondary slides at approx. $8.5 \cdot 10^5$ m^3 or approx. $13.5 \cdot 10^6$ kN ($13.5 \cdot 10^5$ t). This can be compared to almost five 200 000 t tankers moving at a speed of 3 - 4 knots in the case of the initial slide and almost seven such tankers in the case of the secondary slides or together eleven and a half 200 000 t tankers moving downwards the slope.

Certain observations can be made from the slides in Surte and Tuve as regards the manner which the passive soil masses have affected the active masses. In Surte, projecting terraces, as shown in Figure 1, could be seen in the north-western part of the slide limit.

A similar pattern, in the form of clay elements that had been sheared off could be observed in the Tuve slide (Figure 2).

The surfaces of these elements in Tuve slope downwards in the direction of slide travel, which is interesting and is explained in the text below.

Figure 1. Projecting terraces at the north-western slide limit at Surte

Figure 2. Clay elements that have been sheared off, in the upper passive zone of the Tuve slide. Note the inclination of the surface of the elements. The direction of the slide is towards the right in the photograph

Destructive shearing into elements

The destructive shearing of the soil masses in the passive zone may be explained as follows. The masses that have started to move, the active masses, exercise a continual pressure on the masses ahead of them until the active masses have been retarded and come to rest. The active forces, as shown in Figure 3, may be obtained by considering an element within the passive masses.

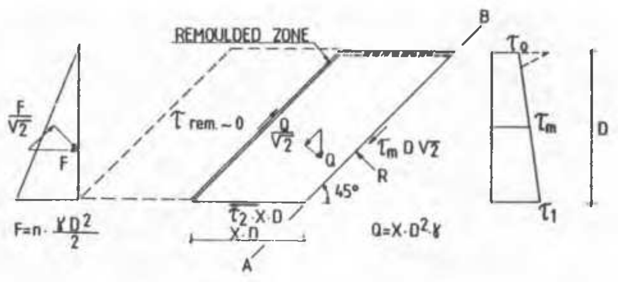

Figure 3. The forces that act on an element within the passive soil masses, as long as the active masses retain their kinetic energy and have not been retarded

Projection along the line A - B gives:

$$n \cdot \frac{\gamma \cdot D^2}{2\sqrt{2}} - \frac{x \cdot D^2 \cdot \gamma}{\sqrt{2}} - \tau_m \cdot D \cdot \sqrt{2} - \frac{\tau_{2} \cdot x \cdot D}{\sqrt{2}} = 0$$

$$x = \frac{n \cdot \gamma \cdot D - 4\tau_m}{2\,(D \cdot \gamma + \tau_2)}$$

The horizontal force H has been taken as $n \cdot \frac{\gamma D^2}{2}$, where n is a dimensionless coefficient.

The results of the investigation of Tuve show that τ_m can be taken as approx. 17 kPa (1.7 t/m^2), assuming a bottom sliding surface at a depth of 12 m below the original ground level. The shear strength in the bottom sliding surface, taking different pore pressure situations into account, can be taken as between 0 and 25 kPa (2.5 t/m^2). In the first calculation of x, τ_2 is taken as 10 kPa (1.0 t/m^2). The average bulk density is 1.6 t/m^3. If n, taking certain dynamic forces into account, is taken as 1.5, x will be 0.54, i.e. the length of the sheared soil element at ground level amounts 0.54 x 12 = 6.5 m. If n is taken to be 1.0, the value of x drops to 0.31 and the length of the element at ground level falls to 3.7 m. If $\tau_2 = 0$, i.e. the pore water pressure has reduced the shear strength to 0, this affects the length of the sheared-off elements in the passive zone very little. The value of x will only increase from 0.54 to 0.57 and from 0.31 to 0.32.

If we assume that the sliding bottom lies at a depth of 30 m, τ_m can be taken to be 27 kPa (2.7 t/m^2) and τ_2 may vary between 0 and 40 kPa (4.0 t/m^2). For calculating x, τ_2 is assumed to be reduced to about half, or 20 kPa (2.0 t/m^2), because of high pore-water pressure. The value of x then amounts to 0.37 for n = 1.0 and 0.61 for n = 1.5, i.e. the length of the sheared-off element in the ground will be 11.1 and 18.3 m respectively. If τ_2 = 0, these dimensions increase negligibly to 11.7 and 19.2 m respectively.

What may be noted as especially interesting is that the value of x increases negligibly when $\tau_2 = 0$, which is dependent on the resisting force of the weight of the element. Even if there is a very weak layer at a certain level, the passive masses are successively sheared into elements. Consequently the shearing-off a single large element along the weak layer does not take place. It can be said in summary, that the length of the sheared-off element at ground level or along the sliding surface - the length x · D - amounts to between about a third of the depth of the slide from the ground level, when n is 1.0 to just over half of the depth of the slide when n is 1.5.

Comparison between the kinetic energy and the work required to shear the active zone into elements

It may be regarded as an almost impossible task to calculate exactly how the kinetic energy is converted into work, with a breaking down of the natural structure of the clay and the movement of soil along the sliding surfaces. An opinion as to whether the kinetic energy in the active soil masses is sufficiently large to shear the passive soil masses into pieces may be obtained by rough calculations. In the Tuve slide, the soil masses involved in the initial slide amounted to about 3.8 · 10^6 kN (3.8 · 10^5 t), as described above for a sliding bottom at a depth of 12 m. With a velocity of 1.7 m/s, these masses obtain a kinetic energy of approx. 5.6 · 10^5 kNm (0.56 · 10^5 mt). The soil masses of the secondary slides in the active zone can be estimated at approx. 7.2 · 10^6 kN (7.2 · 10^5 t) as described above. The kinetic energy of the secondary slides amounts to 10.6 · 10^5 kNm (1.1 · 10^5 mt), giving a total of 16 · 10^5 kNm (1.6 · 10^5 mt).

In calculating the work required to move a soil mass it is assumed that the undisturbed shear strength has been transformed into a disturbed shear strength as a result of a displacement of 0.05 m. This figure is probably smaller in the case of quick clay. In the following estimate, an average shear strength of 17 kPa (1.7 t/m^2) has been assumed for all sliding surfaces in the Tuve landslide, which implies that reductions in the value of τ_2 have not been taken into account in this case as they were above. The calculation is only intended to give an indication of the order of size of the work required to move the soil. It is also assumed that the shearing occurs in elements whith a length at ground level of 5 m as described above, and that the bottom depth of the slide is 12 m. The total surface where movements take place is obtained as follows. The area of the bottom sliding surface is 320 x 500 m^2 = 1.6 · 10^5 m^2 and in the surfaces at 45° $= \frac{320}{5} \cdot 12 \cdot \sqrt{2} \cdot 500 = 5.4 \cdot 10^5$ m^2 or a total of 7.0 · 10^5 m^2. The work in these areas amounts to

$$7.0 \cdot 10^5 \cdot 0.05 \cdot 17 = 5.9 \cdot 10^5 \text{ kNm } (0.59 \cdot 10^5 \text{ mt}).$$

With the bottom of the slide at a depth of 30 m, the kinetic energy of the initial slide at a velocity of 1.7 m/s amounts to 14 · 10^5 kNm (1.4 · 10^5 mt) and the total energy of the secondary slides amounts to 20 · 10^5 kNm (2.0 · 10^5 mt), giving a total of approx. 34 · 10^5 kNm (3.4 · 10^5 mt). If it is also assumed for the purposes of the estimate that the average shear strength amounts to 27 kPa (2.7 t/m^2) in all sliding surfaces and that the length of the elements sheared off amounts to 12 m, the work required to achieve a displacement of 0.05 m along the sliding surface formed as above may be found to be:

$$\left[1.6 \cdot 10^5 + \frac{320}{12} \cdot 30 \sqrt{2} \cdot 500\right] \cdot 0.05 \cdot 27 = 9.8 \cdot 10^5 \text{ kNm}$$

$$(0.98 \cdot 10^5 \text{ mt}).$$

In a comparison between the estimated total kinetic energy and the work calculated to achieve a displacement of

0.05 m in the sliding surfaces that develop, the work amounts to about one third of the total kinetic energy or almost the kinetic energy, or less, of the initial landslide. There is thus a surplus of kinetic energy for further displacement and breaking down of the masses in the passive zone. To this can be added that it appears probable that the masses in the secondary slides have moved faster than walking speed, since these masses have mostly moved in areas of quick clay. Unfortunately no velocity has been noted for the secondary slides. However it should be noted that the kinetic energy is directly proportional to the square of the velocity, so that a small increase in velocity provides a considerable addition to the kinetic energy.

The breaking down of the quick clay into more or less completely disturbed layers would probably have contributed to the large area encompassed by the slide. It should be possible to liken the quick clay to a hydraulic fluid wedge when the masses in the active zone meet the passive zone. The lifting has taken place within the passive area in the slip surface zone so that the sheared-off elements have got he slopes shown in Figure 2.

Conclusion

Since very large masses have been put in motion in the landslides at Surte and Tuve and also in many other slides, it is necessary to introduce the dynamic forces into the explanation of the behaviour of the passive zone. The above facts and calculations show that it is possible to explain the large extension of the passive zone in the Tuve landslide by taking the dynamic forces into account.

Figure 4.

The Tuve landslide with the course of events during the slide according to eyewitness accounts. The big figures show the order in which the different parts started to move (after U. Fält, 1978). Event 1 and 2 started almost at the same time.

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E. Slunga, Panelist

ANALYSING THE SHORT-TERM STABILITY OF A CUT SLOPE IN CLAYEY SOIL

According to the practice I am used to work with the short term stability of a clay slope is always assessed by a total stress analysis, at least at first, because of the simpler way of determining the shear strength of clay and of the easier calculation methods compared with an effective stress analysis.

In those loading conditions, where remarkable changes in pore pressure are caused or the soil formations are complex, the effective stress analyses are also used. When using the effective stress analysis, enough resources should be reserved for the purpose to make the investigator able to measure the pore pressure. Poor knowledge of pore pressure will not increase the reliability of a stability analysis.

The stability analyses are mostly performed by using circular slip surfaces. If the geometry or stratigraphy of a construction site shows that soil masses will probably slide along a non-circular surface, then also analyses with this type of slip surface are made, usually with the method of Janbu.

A total stress analysis is often made on basis of field vane tests, because this method makes one able to get quickly and cheaply many investigation points and whole profiles through the weak soil layers. It has been found out in practice that the peak shear strength determined by standard tests often has to be reduced before using it in a stability analysis. The total reduction factor (μ) can be written (Helenelund 1977)

$$\mu = \mu_T \cdot \mu_A \cdot \mu_P$$

where the need for reduction is assumed to consist of the influence of time to failure (μ_T), the influence of anisotropy (μ_A) and the influence of progressive failure (μ_P).

In normally or slightly overconsolidated homogeneous, inorganic clays the reduction factors proposed by Bjerrum or by the Swedish Geotechnical Institute are used and they have been found to work satisfactorily in the frame of the used safety margin (mostly $F \geq 1,3...1,5$), The factor of safety at failure (F_f) without shear strength reduction is often found to be 1,0... 1,3 or less, depending on the situation. There are also some exceptional cases e.g. that of Kimola canal, where the overconsolidation ratio was 1,5...7 and F_f = 1,55...1,74 (Kankare 1969).

In varved and in organic clays or in clays of high plasticity the situation can be different. In the case of Saimaa canal (Slunga 1973) the clay was varved and slightly overconsolidated ($p_c/p'_o = 1,3...1,6$, $\tau_{fu\ vane}/p'_o = 0,40$). During the excavation works three greater failures occurred. The length of the greatest failure was 240 m and the height of the slope about 6,3... 6,8 m. The reduction factors μ according to the usually available methods (Helenelund 1977) are shown in the table 1.

Table 1. Reduction factor μ of the undrained vane shear strength according to various methods at Saimaa canal.

Reducing method (Helenelund 1977)	μ
Swedish Geotechnical Institute	1,00
Pilot	1,00
Bjerrum, $\mu_T \cdot \mu_A$	0,95
-"- , μ_T	0,80
Aas ($\mu_T \cdot \mu_A \cdot \mu_P$)	0,69
Friis, $\mu_T \cdot \mu_A \cdot \mu_P$	0,71-0,78
Dascal & Tournier (1975), $\mu_T \cdot \mu_A \cdot \mu_P$	0,73
Back-analysis (Slunga 1973)	0,76-0,79

The great difference between the Bjerrum's reduction factor and that from back-analysis is explained by the fact that Bjerrum's reduction factor includes the influence of the normal stress anisotropy, $\mu_A = 1,0...1,2$. In the varved clay at Saimaa Canal the μ_A-factor is estimated to be 0,86. If we take into account separately the influence of time to failure $\mu_T = 0,80$ according to Bjerrum's case records and the factor $\mu_A = 0,86$ we get $\mu = 0,80 \times 0,86 = 0,69$ which is the same as the reduction according to Aas and about 10 % less than the reduction factor 0,76 from the back-analysis.

A progressive failure has usually not been taken into account separately. The importance of the separate treatment of progressive failure will obviously depend on the local soil and loading conditions as well as the method used when reducing the shear strength of the clay. There is often also a certain overlapping of the influence of the various factors.

In the excavation case presented in Fig. 1, the liquid limit of the clay is $w_L = 90$ %, humus content 2,5...3 % and $\tau_{fu,vane}/p'_o \approx 0,55$ (Slunga 1979). A stagewise excavation method was used. One day after reaching the final bottom level a small failure occurred (length/height of the failured slope $\approx$ 3). The reduction factors μ

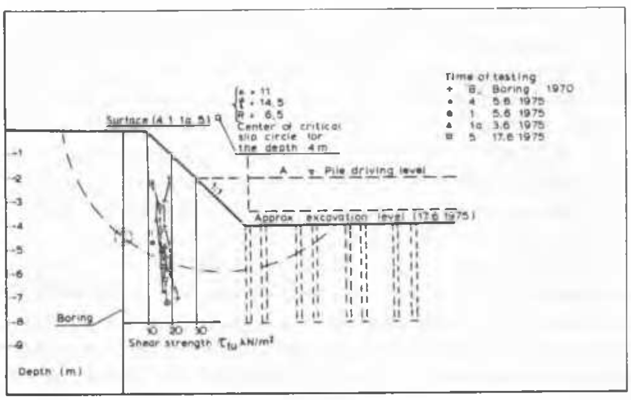

Fig. 1. Comparison of shear strength during various stages of working.

according to various methods are in this case shown in the table 2.

Table 2. Reduction factor μ according to various methods in the case of Salo.

Reducing method	μ
Swedish Geotechnical Institute	0,90
Pilot	0,80
Bjerrum, $\mu_T \cdot \mu_A$	0,75
Bjerrum, μ_T	0,66
Aas ($\mu_T \cdot \mu_A \cdot \mu_P$)	0,55
Dacal & Tournier (1975), $\mu_T \cdot \mu_A \cdot \mu_P$	0,62
Back-analysis (Slunga 1979)	0,67-0,75

The back-analysis is made for a long slope. No influence of the traffic on the neighbouring streets is taken into account. The shear strength of the dry crust is assumed to be the same as that of the clay under the crust.

The use of various reduction methods requires obviously also the use of various factors of safety. It is therefore in important cases preferable to check the level of safety also by comparing the factor of safety calculated without shear strength reduction to the empiric factor of safety at failure (F_f) according to the local experience taking into account the type of failure and of soil and the influence of the surrounding area.

Vane test has to be made with a normal vane (H = 2D) the speed of rotation being 0,1°/second. The vane test results can be influenced by various factors. Some laboratory tests are always made for controlling the vane tests. The laboratory tests in question are often: unconfined compression test, triaxial compression test (UU or CU), direct shear test, cone test. The use of vanes of various shapes can also be accounted when the anisotropy or other conditions are exceptional.

Unconfined compression test has been found suitable for testing of samples taken from dry crust or from other overconsolidated layers, where the shear strength can be influenced by fissures. The samples are, if possible, consolidated to natural stress conditions.

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P. Anagnosti, Panelist

FACTOR OF SAFETY AND ACTUAL DISPLACEMENTS

The conventionally applied methods of slope stability, based on limit equilibrium along potential slip surfaces, cannot provide the values of displacements and cannot define actual stress-strain conditions either above or below assumed slip surface. The stress distribution along assumed slip surface is based on the assumption of the equal mobilization of the available shear strength in all points laying on preselected potential slip surface (concept of average factor of safety). This distribution does not correspond to the actual stress field. (Fig.1) This is why the slopes with materials of very different rigidity are to be considered for the case of progressive failure.

The application of conventional stability analyses is therefore justified in cases where actual stress-strain field has no importance for practical consequencies, and when the displacements on the slope surface may take the values which are required to mobilize the shear strength along potential slip surfaces. These displacements should not change the geometry of the slope and continuity of the soil mass.

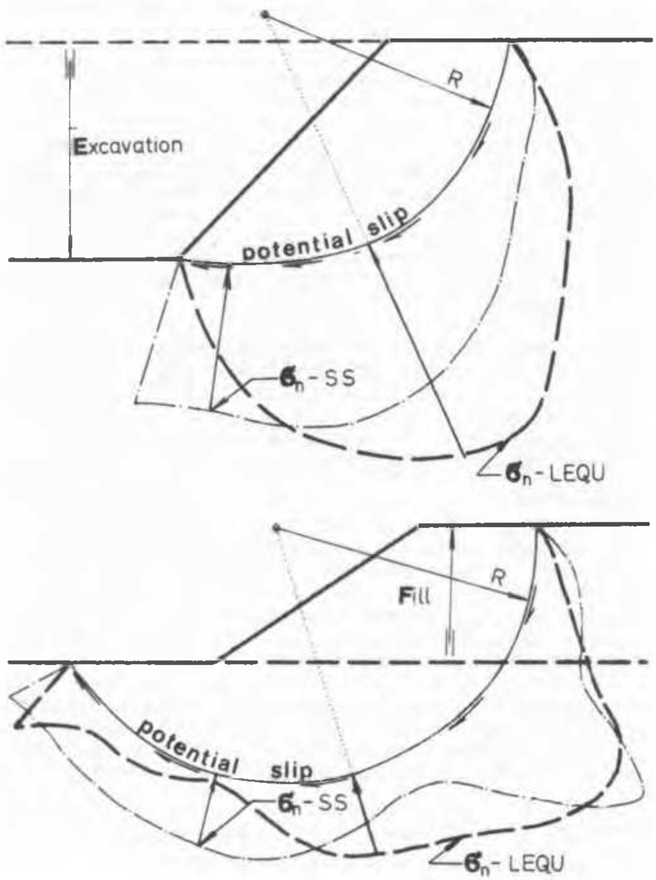

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Fig. 1

The selection of the appropriate safety factor appears to be more matter of experience than of judgment, and precedent similar cases are of paramount importance in providing the confidence in the adequacy of the applied computations and selected safety factors.

The application of residual shear strength values are often associated with acceptance of significantly lower safety factors. This may be considered as equivalent safety margine in relation to the failure state of the slope, but only in those problems where the actual displacements along the slope and in the ground may be accepted. (Fig.2)

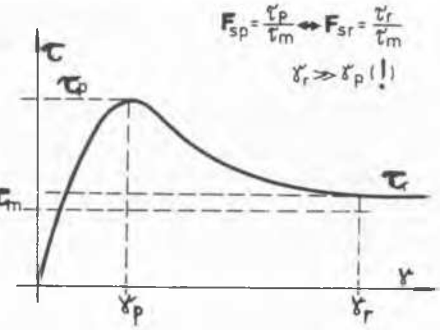

Fig. 2

The practical case of the slope designed with low value of F_{sr} (factor of safety based on residual shear strength), is presented here-below. (Fig.3) The slope of inclination of 4^0 was composed of clay having $LL \leqslant 113\%$ and consistency index $I_c \geqslant 0,60$. The design of the cut in the lower part of the slope was based on short term stability conditions, and for the long term conditions the stability factor of $F_{sr} = 1,10$ was considered as safe assumption, with residual shear strength of $\phi_r=6^0$. The peak strength was twice the residual one, and the equivalent factor of safety $F_{sp}=2,2$. The weakest zone was the contact between slope deposit and weathered marl underlaying it. The excavation was limited to appr. 2,5 - 3 m and remained stable, but the upper part of the ground appr.30 to 60 m from the cut experienced displacements of the order of 10 to 30 cm, and heavy damages occured in the houses situated 60 m away from the excavation limit. The remedial works consisted in placing coarse grained fill on the lower part of the slope ensuring $F_{sr} = 1,5$ and after that, the displacements seased and further damages of the houses have been prevented.

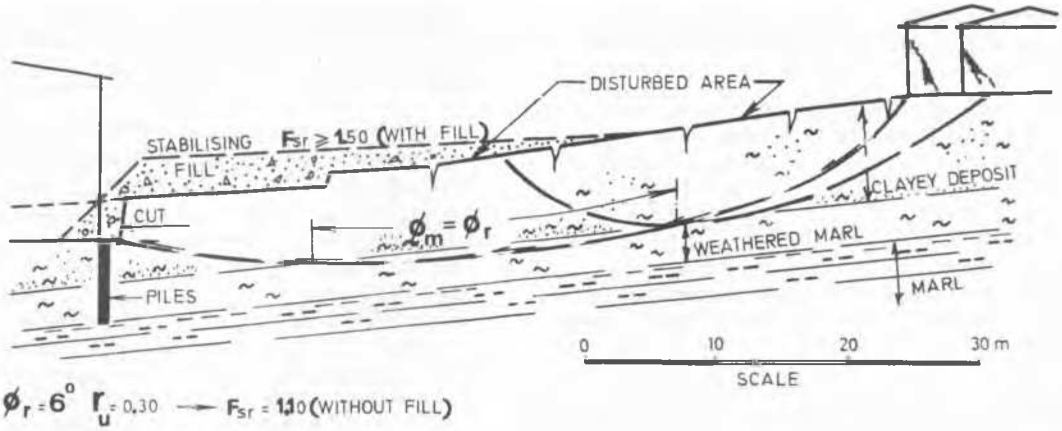

Fig. 3

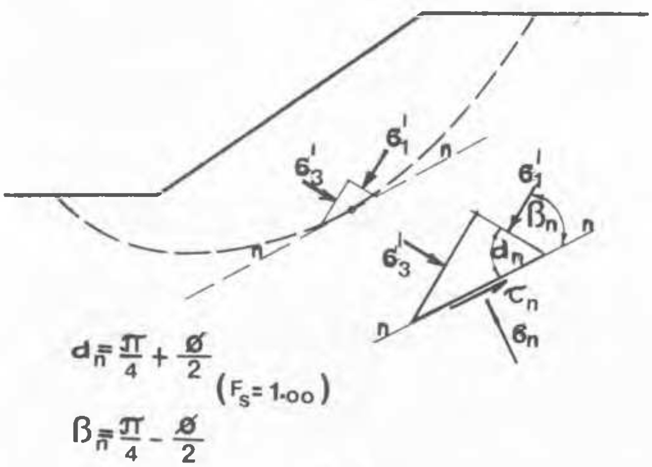

Fig. 4

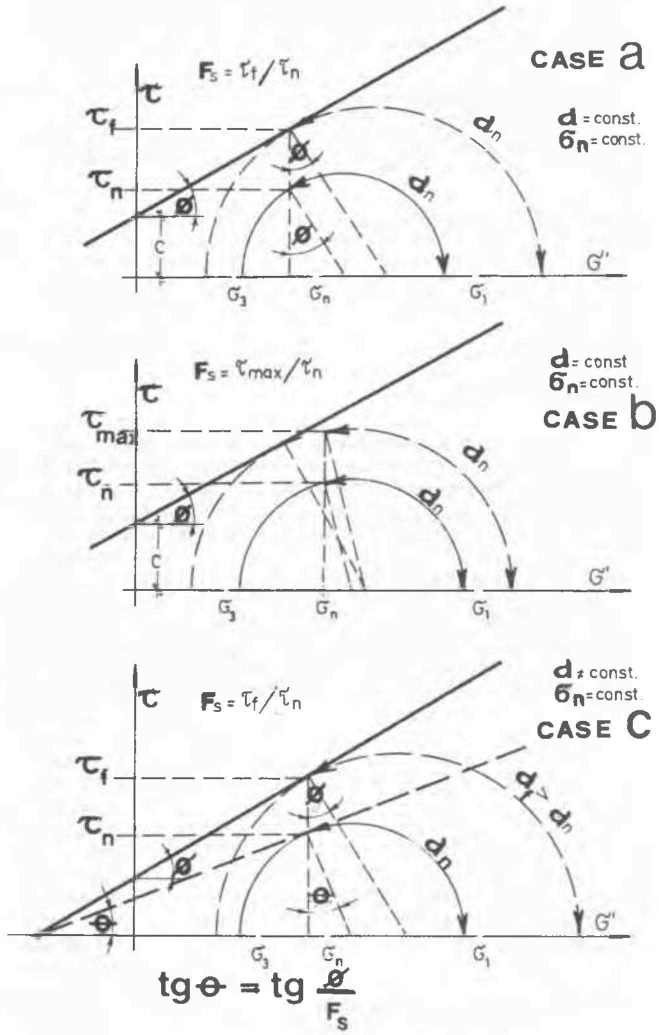

Fig. 5

The main experience that one can derive from the cited case is the necessity to consider allowable displacements in connection of the applied factor of safety, and consider that the equivalency of factors of safety in relation to the peak and residual shear strength is dependent on the value of displacements that might take place.

If no precedent cases are available for selecting the adequate factor of safety, the stability problems which are dependent on allowable displacements have to be analysed by some stress-strain methods. The available numerical methods are providing the possibility to determine probable displacements in cases when factors of safety are sufficiently large, i.e. when the zones where limit state of stress is taking place are small and cannot significantly influence stress-strain distribution. The particular difficulty in computing the actual state of displacements is associated with the initial state of stress-strain state which is governed by geological processes in natural slopes, and foundations of man-made structures. The establishing rational relationship between safety factor in limit equilibrium methods, and actual safety margine based on the computed stress-strain state of the slope is associated with principal difficulties.

The shear and normal stresses which are acting on the point of the assumed slip line (Fig.4) cannot be uniquely related to some other state of stress which is bringing that point into the state of limit equilibrium. Therefore different safety factors may be derived for the same point, always defined as the ratio of the available shear strength and actual shear stress in the predetermined plane. (Fig.5)

The case (a) is showing the factor of safety derived under condition of the constant direction of principal stresses, case (b) is showing same conditions, but with critical values of shear and normal stress appearing in some other plane, case (c) is showing critical shear and normal stress acting in the predetermined plane, but under changed direction of principal stresses where the condition of $tg\,\theta = tg\,\varphi / F_s$ is satisfied.

The presented cases demonstrate that even one would start from the same definition of the safety factor (as the ratio of available strength and actual stress in the selected plane) still there will be possibilities to derive the different F_s values. It appears, that the right procedure is to relate the computed displacements (by numerical or other suitable methods defining approximate state of stresses and strains) to the safety factors determined in classical way i.e. by limit equilibrium methods. This is, however, the only way to overcome the lack of field experience for the particular case under consideration.

O. Orrje (Oral discussion)

INDUCED PORE PRESSURES DURING PILE DRIVING IN A CLAY SLOPE

In the article presented to this conference by Massarsch and Broms concerning pile driving in clay slopes it has been stated that sufficient data on soil conditions and test results reported in the literature concerning this problem, have only been available from six test sites.
In this discussion one more case is presented wich has been carried out last autumn at the city block Jungfru Lona in Stockholm.

The building site in which two large buildings had been planned, is situated close to an excavation for the Stockholm subway, wich was made in 1964. The soil consists of a top layer of gravel fill which rests on rather soft clay. The planned buildings were to be founded on endbearing concrete piles. To reduce the pore pressure in the clay during pile driving, clay cores were taken in zones up to a distance of 15 m from the slope. No significant movements of the soil mass were observed during the pile driving. Vibrating wire piezometers were installed in 6 points within the zone with the lowest safety factor, with a distance to the nearest piles of 4 m.

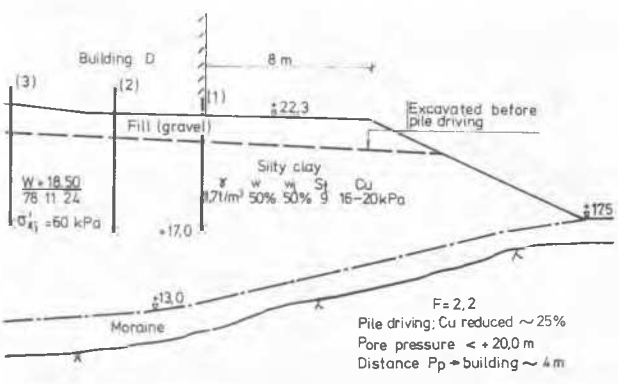

Fig. 1

Fig 1 shows a section through the slope with the layer of gravel fill on top. The fill was excavated before pile driving.

Stability calculations showed that the stability safety factor for the slope was 2.2 in the finished state. With the necessary safety against failure of the slope, 1,5, the pore pressures in the failure zone, during pile driving were not allowed to rise above the level of + 20 m. Since clay cores were taken before the driving of the piles, no large exess pore pressures were expected.

As can be seen in fig 2 large pore pressures were obtained during the installation. These pore pressures decreased rather rapidly. When the pile driving started close to 1 and 3, the pore pressures increased. The high peak values decreased rather rapidly. The pore pressure during pile driving never did increase above the critical value + 20, but were as large as + 19.7 m, even though clay cores had been taken before the driving of the piles.

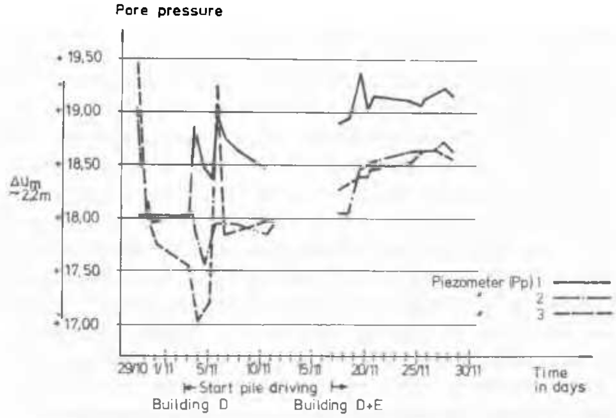

Fig. 2

To estimate the upper limit of the maximum excess pore pressures the formula by Lo had been used.

$$\Delta u_m = \left[(1-k_0) + \left(\frac{\Delta u}{p}\right)_{max}\right]\sigma'_{li}$$

The formula gives the maximum excess pore pressure Δu_m as a function of the initial effective vertical pressure in the ground before pile driving σ'_{li} and of the maximum pore pressure ratio $(\frac{\Delta u}{p})_{max}$ determined from consolidated undrained triaxial tests. The maximum pore pressure ratio has been given by Lo (1968) as a function of the sensitivity of the clay as shown in fig. 3.

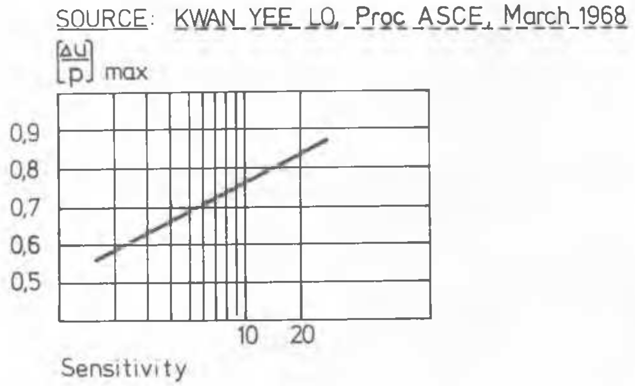

Fig. 3

In fact,the formula by Lo is only valid in the zone of maximum pore pressure surrounding a pile. The measured pore pressures were expected to be considerably lower than the calculated values according to the Lo formula. The calculated values of excess pore pressures were 68to 74 kPa, while in the present case the measured maximum value was 22 kPa.
As can be seen from the present case, rather large pore pressures can occur when piles are driven in a clay slope even when clay cores are taken before pile driving. The measured pore pressures wich were obtained in the present case were as large as 1/3 of the maximum pore pressures developed in the critical zone surrounding a pile when no clay cores have been taken. The formula of Lo was also very useful in order to estimate the upper limit of the maximum excess pore pressures close to the pile groups.

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W. Wolski, Panelist

MERITS AND DEFECTS OF THE LIMIT EQUILIBRIUM AND FINITE ELEMENT METHODS TO INVESTIGATE THE STABILITY OF EARTH STRUCTURES

In practical cases the limit equilibrium methods /LEM/ which inolude Swedish,Bishop`s, Janbu´s and Morgenstern´s methods /tab.1/are mostly used by engineers when stability of earth structures is analysed.The finite element method/FEM/, at present stay of its development, is rather used as an auxiliery tool.

TABLE I

Methods and Parameters Used in Stability Analysis

Methods		Parameters
Fellenius **Simplified Bishop´s** **Janbu´s** **Morgenstern-Priee´s**	L E M	φ **Angle of Friotion** c **Cohesion**
Finite Element **F E M**		φ **Angle of Friotion** c **Cohesion** E **Youngs Modulus** μ **Poisson Ratio or** φ, **c,E,**μ **and additional: K , G,eto.**

In spite of an undoubtedly more universal oharaoter of FEM, the above practice results from the four main defeots of FEM, when applied to stability analysis. These defeots are as follows:

- **inadequacy of soil model used,**
- **uncertainties of soil parameters and diffi- oulties with their assesment,**
- **uncertainties of geometry of the slide,**
- **difficulties to programm,associated with oost of computation /far more expensive than LEM/**

Most esential among the above mentioned defe- ots,in opinion of the author, are difficul - ties with the assesment of the parameters. As an example of the influence of soil parame - ters on the results of stability analysis, diagram given by Martins and others in their paper presented to this conference in Volume 2/page 466/ can be adopted; authors using FEM according to Zienkiewicz and Cormeau /1974/ with their own modification, has shown the influence of Poisson ratio on safety factor. Taking also into consideration that Poisson ratio diffioult to asses, ohanges with water oontent/fig.1/, as was shown by Wolski and others/1978/ it oan be concluded, that FEM which eploi more parameters than LEM,Poisson ratio inoluded/see tab.1/gives the results more uncertain.

However, there are several cases, when FEM applied as an auxiliery way of assesment of the strain and stress destribution, make the results of stability analysis by LEM more reliable. Three examples of structures whose stability analysis is more adequate when FEM is applied are given in/fig.2/.Common feature of these three oases are existanoe of two different types of soils: of high and low

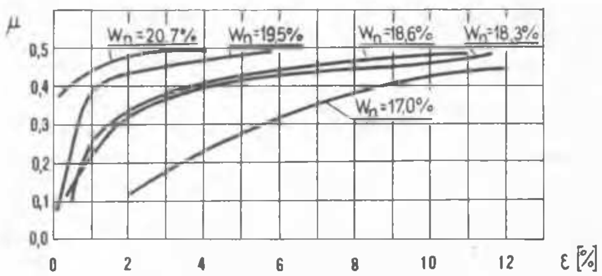

Fig.1 Influence of water oontent on μ

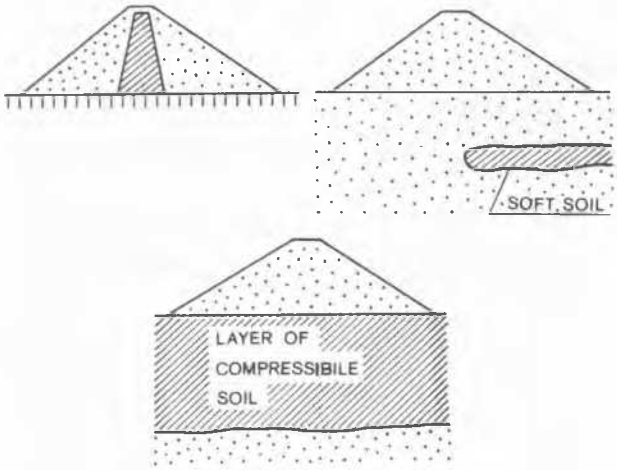

Fig.2 Embankments with oompound sliding surface

compressibility. Strains and displacements whioh occure in oontact areas of different soils oomplicate the geometry of sliding surface.

One of the possibilityes of application of FEM to analyse the stability of embankment on soft soils is presented in figs.3,4. Finite element analysis anable us to estimate plastified zones under given load. Assuming that within this zones only the residual strength are involved, a solution is more realistic, than in olasical LEM approach. Increase of the load oause an increase of the plastified area what reseltsin diminishing of the safety factor.

Summing up it is worth to point out that FEM is still in preliminary stage of application to stability analysis. This prospeotive method needs farther research both oonoerning soil model as well as methods of estimation of parameters.

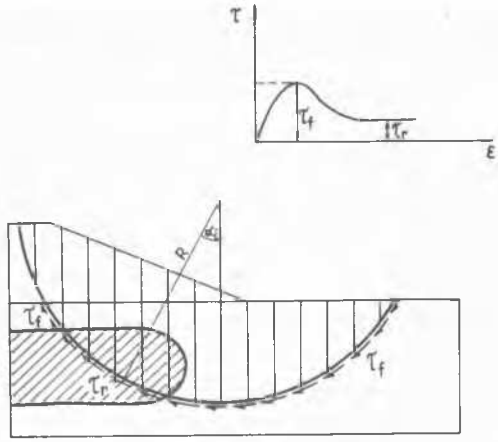

Fig. 3 Destribution of plastified zones with increase of the hight of the embankment

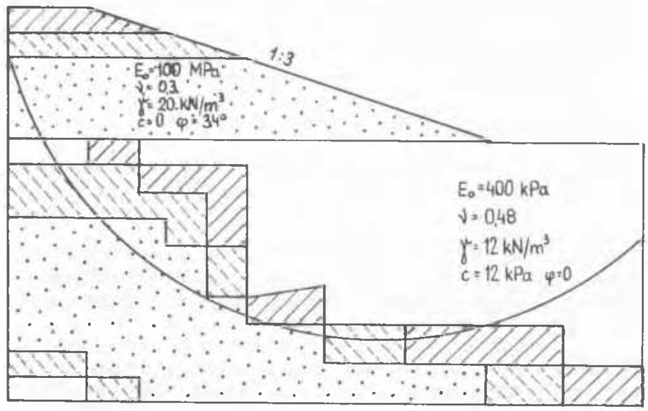

Fig.4 LEM stability analysis with plastified zone /Garbulewski, 1981 /

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M. Dysli (Written discussion)

M. le Professeur W. WOLSKI a soutenu dans son intervention que la méthode des éléments finis conduisait à des résultats fort différents de ceux obtenus avec les méthodes classiques d'évaluation de la stabilité d'une pente. Cette opinion mérite une brêve et légère contradiction.

Les méthodes classiques utilisent toutes une loi des matériaux rigide - parfaitement - plastique (Mohr - Coulomb) sur une surface de rupture choisie à priori; elles excluent en outre toute redistribution des contraintes dans le massif de sol.

La méthode des éléments finis permet elle l'usage de n'importe quelle loi des matériaux non linéaire. La loi facilement utilisable avec cette méthode et la plus proche de la loi de Mohr - Coulomb est une loi élasto - plastique avec le critère de rupture de DRUCKER et PRAGER (1952); le critère de Mohr - Coulomb présente en effet certaines difficultés d'utilisation dans un modèle d'éléments finis, car la contrainte intermédiaire est indéfinie. La surface de rupture n'y est, en principe, pas définie à priori; elle est l'enveloppe de la zone des éléments plastifiés par dépassement des contraintes de rupture données par la loi choisie.

En utilisant cette méthode avec un algorithme itératif approprié - ce qui n'est pas le cas

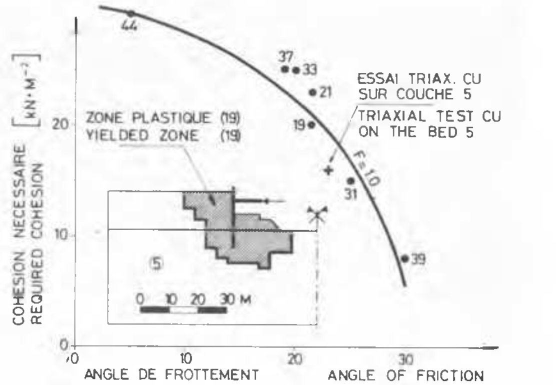

Fig. 1 - Exemple d'une évaluation de stabilité de talus par la méthode des éléments finis (programme ADINA)

dans beaucoup de programmes basés sur cette méthode - les résultats sont en général très proches de ceux obtenus par une méthode classique; la figure 1 est un exemple de résultats d'une telle analyse par la méthode des éléments finis. Il faut cependant noter, qu'avec la méthode des éléments finis, le coefficient de sécurité ne peut être appliqué qu'aux caractéristiques des sols et que, pour un équilibre limite, la convergence du processus itératif peut être très longue.

La comparaison entre ces deux types de méthode est académique car l'usage de la méthode des éléments finis, pour une simple vérification de stabilité d'un talus, est très coûteuse relativement aux méthodes classiques mêmes sophistiquées. La méthode des éléments finis apporte cependant un renseignement supplémentaire qui peut être déterminant dans beaucoup de cas: c'est l'évaluation des déformations. DYSLI, FONTANA et RYBISAR (1979), comme du reste d'autres auteurs, ont, par exemple, montré l'importance de la détermination des déformations dans l'étude pratique des enceintes de fouilles.

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J.N. Hutchinson, Co-Chairman

LANDSLIDE HAZARD ZONING. OPENING REMARKS

Landslide hazard zoning involves the division of the whole land surface within a chosen region into discrete areas, which are then ranked according to their degree of actual or potential hazard from landslides or other mass movements on slopes. There has been a great development in this field over the past decade, carried out chiefly by engineering geologists and geomorphologists. The inclusion of the theme of landslide hazard zoning, for the first time, in the programme of our international conference is a reflection both of the growing importance of this activity and of the increasing participation of geotechnical engineers in it.

An excellent introduction to the subject has been prepared for UNESCO by Mr D J Varnes of the USGS and the IAEG Commission on Landslides and other Mass Movements. This monograph, which was referred to by our General Reporter, is expected to be published by UNESCO late this year. Such zonation is based upon three basic principles: - that the past and present are frequently keys to the future - that the main conditions causing landsliding can be identified and - that degrees of landslide hazard can be estimated. The factors to be considered will vary somewhat from place to place, but will typically include presence of former landslides, slope inclination, lithology, geological structure, geohydrology, etc. Because of the general regional nature of such studies they are normally carried out at Level 1 as defined by Sällfors and Tägnfors, in Vol. 3 of the Proceedings, with little geotechnical or sub-surface data. For each of the relevant factors, individual factor maps, or corresponding data banks, are then usually produced. Finally, these are superposed, or integrated, to form a composite map, on the basis of which the zonation into degrees of landslide hazard can be made.

G. Pilot, Panelist, and G. Champetier de Ribes

CARTOGRAPHIE DES GLISSEMENTS DE TERRAIN. LANDSLIDE MAPPING

A la suite de graves glissements de terrain survenus dans les Alpes françaises en 1970, la Direction de la Protection Civile a lancé une action de cartographie des Zones exposées aux Risques de Mouvements du Sol (action ZERMOS). La méthode de travail élaborée à cette occasion a été appliquée à des régions morphologiquement et géologiquement très variées (CHAMPETIER DE RIBES, HUMBERT, 1974) ; il existe actuellement une trentaine de cartes ZERMOS établies à l'échelle du 1/25.000.

La carte d'appréciation du risque dans une région donnée résulte de la synthèse d'informations sur les facteurs suivants :

- facteurs topographiques : une carte des pentes du terrain naturel est établie,
- facteurs géologiques et structuraux : sur la base de la documentation géologique locale, des photographies aériennes, d'éventuels sondages existants et de la visite du site, on reporte les éléments suivants sur une carte :
  . la nature du substratum rocheux,
  . les formations meubles de couverture,
  . les caractéristiques structurales.
- facteurs hydrogéologiques et géomorphologiques la documentation hydrogéologique, la collecte sur place des informations, les photographies aériennes permettent de dresser une carte sur laquelle figurent notamment les indices des divers types d'instabilité constatés (éboulements rocheux, glissements, coulées de boue etc...) ainsi que leur acuité (mouvements actifs, instabilité "dormante" etc...).

Au niveau de ces cartes de facteurs on dispose donc essentiellement d'informations objectives. Ces informations sont rassemblées et conduisent à une appréciation de la stabilité par zônes de risques (ce jugement présente alors une certaine part de subjectivité).

- une représentation symbolique de la nature des instabilités : par traits pour les instabilités, par tiretés pour les instabilités potentielles.
- des zones de couleur présentant la gradation du risque :
  . la dominante rouge marque le danger existant (mouvements naturels actifs),
  . la dominante orange attire l'attention sur l'existence de conditions défavorables (mouvements anciens, extension possible de mouvements actifs ...) ,
  . la couleur verte exprime la stabilité naturelle.

On notera que cette représentation se veut neutre vis à vis des temps d'apparition de nouveaux phénomènes naturels tandis qu'une certaine appréciation de l'incidence des travaux humains peut être mentionnée dans les zônes oranges.

On soulignera que ces cartes sont accompagnées d'une notice importante dans laquelle sont justifiés les jugements retenus.

D'une façon générale les cartes présentées de cette manière s'appuient principalement sur la récurrence dans une même région de phénomènes d'instabilité naturelle, mais ne font pratiquement pas appel aux concepts de mécanique des sols si ce n'est dans l'appréciation qualitative des facteurs défavorables à la stabilité (hydraulique, état résiduel de résistance au cisaillement ...) ; les formations montagneuses généralement traitées se prêtent évidemment mal à une appréciation plus fine.

On a cependant tenté d'appliquer les résultats quantitatifs provenant d'études de stabilité dans une région peu accidentée, dans l'Est de la France, entre Nancy et Metz où dominent les versants argileux des argiles du Lias (BLONDEAU et al. 1973, Perrot et al. 1978).

La figure 1 montre une vue d'un glissement typique de versant naturel qui affecte ces formations. On a voulu évaluer le coefficient de sécurité en divers points des sites argileux de cette zône ; cette évaluation, simplement basée sur une hypothèse de surface plane de glissement réel ou potentiel nécessite tout de même la connaissance de la pente du talus naturel, de l'épaisseur de la couche d'argile, des caractéristiques mécaniques de l'argile et l'orientation d'un écoulement supposé uniforme.

Fig.1 - Vue aérienne du glissement naturel de Cosny dans les argiles du Lias.

Ce dernier facteur, essentiel pour l'appréciation de la stabilité est généralement lié, dans ces formations, à la présence et à la position d'un horizon perméable de grès fissuré fortement aquifié, situé sous la couche d'argile ; le coût de l'investigation permettant de fournir cette information est hors de proportion avec la dotation financière d'une telle carte ; l'on a donc du constater que, compte tenu de ce facteur et des quelques autres, il était impossible de porter une appréciation fine de la stabilité à ces échelles de l'ordre du 1/25.000.

Au plan de l'utilisation des cartes ZERMOS, on précisera qu'elles ne sont considérées que comme des documents techniques et non pas réglementaires.

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E.W. Brand (Oral discussion)

It has been emphasized in the Author's paper to this Conference that the classical methods of slope stability assessment are difficult to apply to slopes in residual soils and, more particularly, to such slopes subjected to heavy rainfall. This is the situation in Hong Kong, where only 1,050 km^2 of land are occupied by more than five million people. Because land is such a scarce resource in Hong Kong, the Geotechnical Control Office has embarked upon a programme of Geotechnical Area Studies aimed at mapping the Territory for land resource planning and management purposes.

The Geotechnical Area Studies are being carried out in two parallel series. The 1:20,000 scale Studies produce sufficient information for outline planning purposes on a regional basis, whereas the 1:2,500 scale Studies are sufficient for detailed assessments of specific areas identified from the regional Studies.

Each Geotechnical Area Study consists of a terrain evaluation, based on aerial photograph interpretation, and an engineering geological assessment, based on field reconnaissance and mapping and on a review of existing site investigation data.

A report is published at the completion of each Study. This is accompanied by a series of maps and transparent overlays, which include a terrain classification map, engineering geology map and Geotechnical Land Use Map (GLUM). Where appropriate, an engineering data sheet, surface hydrology map, landform map, erosion map, physical constraints map and vegetation map are included in addition.

The overall geotechnical assessment of an area is presented in the Geotechnical Land Use Map (GLUM), which classifies land into four classes according to its apparent geotechnical limitations. The GLUM classification system presently employed is synthesised in Table 1.

Table 1. The Geotechnical Land Use Map Classification System

CHARACTERISTICS OF GLUM CLASSES	CLASS I	CLASS II	CLASS III	CLASS IV
GEOTECHNICAL LIMITATIONS	Low	Moderate	High	Extreme
SUITABILITY FOR DEVELOPMENT	High	Moderate	Low	Probably Unsuitable
ENGINEERING COSTS FOR DEVELOPMENT	Low	Normal	High	Very High
INTENSITY OF SITE INVESTIGATION REQUIRED	Normal	Normal	Intensive	Very Intensive
TYPICAL TERRAIN CHARACTERISTICS (Some, but not necessarily all, of the stated characteristics will occur in the respective Class)	Insitu terrain with gentle slopes (0 - 15°), without severe erosion or instability. Cut platforms in insitu terrain.	Insitu terrain with slopes between 15 & 30°, without severe erosion or instability. Insitu terrain of gentle slopes associated with drainage, but with no instability. Colluvial terrain with gentle slopes (0 - 15°), without severe erosion or instability.	Insitu terrain with slopes between 30 & 60°, without severe erosion or instability. Insitu terrain less than 15°, with history of landslips. Colluvial terrain less than 15°, with evidence of instability. High to moderate fill-slopes.	Very steep insitu slopes (> 60°) and cliffs. Steep to very steep insitu and colluvial slopes, with history of instability. Colluvial terrain with gentle slopes, but associated with instability and drainage.
NOTE : THIS CLASSIFICATION SYSTEM IS INTENDED AS A GUIDE TO PLANNERS AND IS NOT TO BE USED FOR A DETAILED GEOTECHNICAL APPRAISAL OF INDIVIDUAL SITES				

T. Stål (Oral discussion)

SURVEYING POTENTIAL LANDSLIDE RISK ZONES IN SWEDEN

The surveying of potential landslide risk zones in developed areas has been in progress in Sweden for the last two years. The long-term object is to take preventive measures in these areas to give the people who live and work there a greater sense of security. The Swedish Geotechnical Institute was commissioned by the Government to carry out this work following the big landslide at Tuve.

The frequency of slides in clay in Sweden has increased during the twentieth century. During the last month we have had 3 slides. Most of the slides have been caused by human activities.

The cost of damage that has occurred can now be estimated at 50 to 100 million Swedish kronor per year.

Fig. 1.

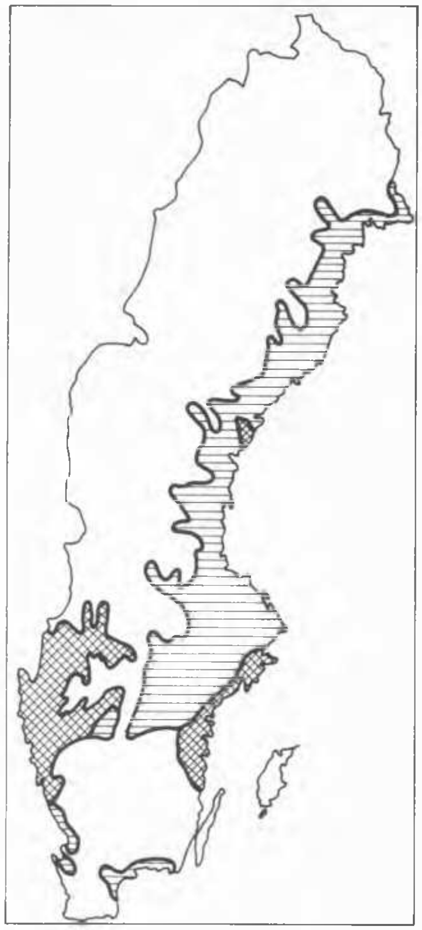

Fig. 1 shows the areas in Sweden where most of the landslides have occurred. These zones are regarded in this survey as zones of primary interest. The total area of clay in these zones is about 5000 square kilometres.

Because of greatly restricted financial resources, the survey will only cover a minor part of the areas of primary interest, mainly those with the highest frequency of previous landslides in the west coast region of Sweden.

The survey work has been divided into five stages. The first stage consists of locating areas of clay within the boundaries of the developed areas where the ground surface slopes more steeply than 1 in 50. This is a first, rough sorting out of the areas in which there are physical conditions for instability. These maps have been issued officially and the local authorities must ensure that no new buildings are constructed and that no changes are made in the ground levels in potential risk zones, before detailed stability calculations have been carried out.

The remaining four stages included in this overall survey are aimed at successively reducing the area of the potential risk zones. The four stages consist of: file investigations, terrain classifications, field investigations and stability calculations.

The terrain classification involves dividing the areas into sub-areas with different conditions for instability. The division is carried out on the basis of high pore pressure potential or high shear stress potential. The sub-areas that do not contain either of these risk elements can be excluded from further investigations. The risk areas are further divided into three degrees of priority, which take into account the slope height and the depth of clay.

Finally general stability calculations are carried out in order to obtain a rough assessment of the risk factor for the initial slides. It is the intention to rank the priority of the individual zones on the basis of assessed risk level. The final report will be issued in two years on maps showing the remaining pontential risk zones.

To avoid creating unrest among those who live and work in these risk zones and to prevent a drop in property values, the survey work in progress must be completed with detailed investigations, and preventive measures against slides must be taken where this if found necessary. Our survey work may otherwise cause more harm than good.

T.B. Edil (Oral discussion)

COASTAL LANDSLIDE HAZARD ZONING
Détermination des Zones à Haut Risque de Glissement de Terrain dans les Régions Côtières

This discussion is about the main factors which should be taken into account in establishing a landslide hazard zoning based on regional experience. The factors considered for such a zoning effort for the coastal areas of the state of Wisconsin in the Great Lakes region of North America are presented herein. Much of the shoreline of Lakes Michigan and Superior in Wisconsin is an area of high, steep bluffs developed in unconsolidated glacial deposits. Most of these deposits are fine-grained sediments subjected periodically to accelerated wave erosion due to fluctuating lake water levels. Extensive damage to property takes place due to resulting landslides and complex slope processes (Edil and Vallejo, 1977; Vallejo and Edil, 1979). These bluffs are natural slopes which may be subject to rapid evolution under the action of waves and other environmental forces such as surface runoff, rain impact, freezing and thawing, etc. Factors considered for hazard zoning in this coastal area included the following:

1. Coastal conditions: Since wave action is the main triggering mechanism for initiating and perpetuating the mass-wasting processes in coastal slopes its significance has to be assessed. In this connection presence of shore protection structures, harbors, and geomorphological features such as wetlands and terraces are important. In unprotected bluff areas the severity of wave action in terms of both the erosion of native bluff materials and the removal of mass-wasting materials collected at the toe are to be considered. Influencing factors include wave energy (wind velocity, duration and fetch) and erodibility of coastal materials. Quantitative analysis of aerial photographs taken at different times combined with on-ground measurements has been useful in determining erosion rates.

2. Geological and geotechnical investigations: These are usually limited in extent and generalizations are required. Relationship of geotechnical properties to glacial stratigraphic units along Wisconsin's Lake Michigan shoreline has been developed and used successfully (Mickelson et al, 1979).

3. Probabilistic techniques: By its very nature, hazard zoning requires the use of such techniques for: (a) assigning hazard potential and (b) choosing slope stability parameters. A probabilistic approach, utilizing the known limited information on soil properties, bluff geometry and stratigraphy, and groundwater conditions along with the Monte Carlo method of generating distribution of these variables has been used effectively for landslide hazard potential assessment along the southwestern Lake Superior shoreline (Schultz, 1980).

4. Set-back zoning in undeveloped coastal areas: In order to minimize future economic losses, development in coastal areas may be limited to zones set back a certain distance from the shoreline.

5. Slope stabilization if economical: When there is extensive development in the tableland, structural solutions are considered. Complete design must include shore protection, slope stabilization, vegetation, etc. (Edil, 1980) (see the table below).

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PROCESSES	SOLUTIONS	
	STRUCTURAL (STABILIZATION): DESIGN	NONSTRUCTURAL (MANAGEMENT): PREDICTION
TOE EROSION	SHORE PROTECTION (groins, seawalls, etc.)	SHORE RECESSION RATE (long-term,cyclic)
DEEP ROTATIONAL SLIPS	SLOPE STABILIZATION (regrading, dewater.)	STABLE SLOPE ANGLE AGAINST DEEP SLIPS
FACE DEGRADATION AND SHALLOW SLIPS	SURFACE PROTECTION (vegetation, drainage)	ULTIMATE ANGLE OF STABILITY

R.K. Bhandari (Oral discussion; invited)

"SLIDE WARNING SYSTEMS AND METHODS OF PREVENTION OF LANDSLIDES"

DETECTION OF AVALANCHES

In India, every year, dozens of people get buried under avalanches that play havoc with property and communication system inflicting untold misery and staggering economic loss. In central Europe alone nearly 100 people get killed every year by avalanches; the number of victims all over the world is far greater.

OBJECTIVE

Detection of an avalanche release is essential so that:

(a) Measuring devices, taperecorders, cameras etc. could be switched on.

(b) People could be warned, roads could be closed in time, and people carried to shelters.

(c) Assessment could be made, based on avalanche speed, if vehicles could travel past the hazard zone in time or they are got to be stopped.

(d) Road clearing or stabilization operations could be planned.

(e) Researches could be advanced to improve concerned technology.

DEVICES FOR DETECTION OF AVALANCHE RELEASE

(a) Wire or special switches operated by snow pressure that electrically or hydraulically release signals. The system can be used only for one avalanche release and thereafter requires new wires to be fixed. The threat bv secondary avalanches or those occurina in auick succession would therefore remain.

(b) Elastic switch poles which turn to an upriqht position, are also used. These however, cannot, control an extended area.

(c) For speed measurements of explosion-induced avalanches, photo-electrical barriers are sometimes used. However, to keep the optics free of snow is as much a problem as the closing of road under the threat of an avalanche.

(d) Pulsed Radar, discussed herein.

TYPES OF RADAR

1. Small economical and simple devices used for controlling speeds of vehicles or those used for raising alarm against thieves. These have a range of about 40m which is low. Besides, everything within the response area is recorded. For example, an insect in front of radar antenna can provoke more signal than a distinct snow slide.

This type of Radar is therefore not very useful.

2. Pulse Radar provides a range of about 5km against targets with a Radar cross-section of $1m^2$, if magnetrons of 5 kw peak power are used with frequency of 10 GHz.

What does the Radar do?

1. It determines the release time to the precision of a second.

2. Measures speed of avalanche, and variations in the speed.

3. Friction on sliding surface of the avalanche also be estimated from the knowledge of speed of movement.

Radar can be mounted on an stable opposite slope(Fig 1).

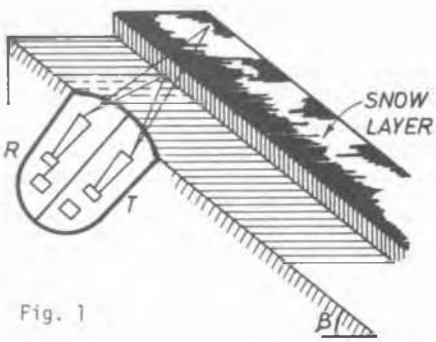

Fig. 1

As soon as the initiation of movement is detected, warning could be flashed out for protection. For closing and opening of highways at the time of avalanches, Fritzsche (1979) has observed certain speed relations. For highways where a minimum speed of 36 km/hour may be expected, a protection distance of 250m will be possible for an avalanche track of 1km. For ordinary roads, if vehicle speed of only about 18 km/hour is expected, a protected distance of 125m will be possible for an avalanche track of 1km. The plan view is shown in Fig 2.

If timely warning does not reach, Radar can once again come to rescue by aiding search of avalanche victims (Fig 3). The first avalanche victim detector came from Bachler : transmitter 200 KHz, transistor as receiver. The first one used in practice was developed by Lawton in 1969, but its weight was too high. Now the frequency of 2275 Hz makes this system considerably cheaper. In Austria, the Technical University of Graz has developed a system known as

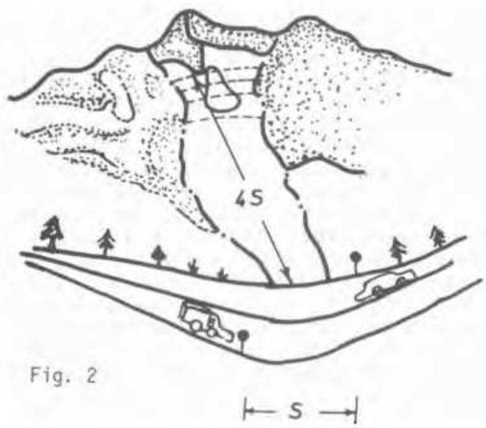

Fig. 2

Pieps 1 working on the same frequency but having lesser weight. In 1973, the Swiss Army did introduce a transmitter-receiver systems 'VS 68' operating on a frequency of 457 KHz (weight 300 gms). In 1978 however, an improved model (Pieps 2) was introduced. It works on the R6 batteries and reaches a technical range of approximately 45m transmitting time 500h. (Fritzsche, 1979).

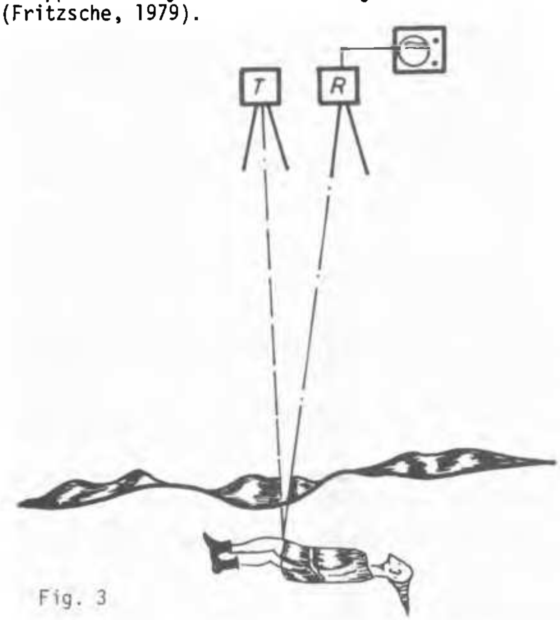

Fig. 3

In homogenous snow (dry or humid) the search of avalanche victims is quite easy. Problems arise, however, when snow contains lumps that cause reflections similar to those by the human body. Well trained operators are therefore needed for recognising such pseudo-targets (Schogel and Fritzsche, 1974).

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R.L. Schuster and D.R. Nichols (Oral disscussion)

WARNING PROCEDURES FOR POTENTIAL LANDSLIDES - EXPERIENCES IN THE UNITED STATES

Des Procédés d'Avertissement d'un Eboulement de Terre Potentiel - ce q'on Fait aux Etats-Unis

Landslide researchers commonly think primarily of "hardware" in considering warning systems. Keep in mind, however, that warning procedures, i.e., the means of issuing warnings to responsible government officials and the public, are a vital aspect of any landslide warning system.

In the "Disaster Relief Act of 1974" the Congress of the United States stated that "the President shall direct appropriate Federal agencies to provide technical assistance to State and local governments to insure that timely and effective disaster warning is provided." Responsibility for doing so with respect to landslides and other geologic hazards was delegated to the U.S. Geological Survey. As the first step in discharging this responsibility, the USGS issued a statement of "Warning and Preparedness for Geologic-Related Hazards--Proposed Procedures" (U.S. Department of the Interior, 1977). In this statement, a geologic hazard was defined as "a geological condition, process, or potential event that poses a threat to the health, safety, or welfare of a group of citizens or to the functions or economy of a community or larger governmental entity" (p. 19292).

Three categories of "warning" were set up under this statement; depending on the magnitude of risk and the time of potential occurrence of the geologic process, a hazard notification is issued as a notice, a watch, or a warning. These categories are defined as follows:

> "Notice of potential hazard--The communication of information on the location and possible magnitude or geologic effects of a potentially hazardous geologic event, process, or condition.
>
> "Hazard watch--The communication of information, as it develops from a monitoring program or from observed precursor phenomena, that a potentially catastrophic event of generally predictable magnitude may be imminent in a general area or region and within an indefinite time period (possibly months or years).
>
> "Hazard warning--The communication of information (prediction) as to the time (possibly within days or hours), location, and magnitude of a potentially disastrous geologic event or process" (p. 19292).

To date under this system, "warnings" have been issued for four landslide hazards or hazards involving potential landslides as side effects of another type of geologic hazard. An interesting example of the issuing of a "warning" under this system was the 1978 notice of potential hazard for a landslide on Pillar Mountain adjacent to the harbor of the City of Kodiak, Alaska. The Pillar Mountain slide, which at present is approximately 520 m wide at its base and extends to an altitude of 343 m, is believed to be an ancient slide that is in danger of being reactivated because of excavation at its toe (Kachadoorian and Slater, 1978). Should the slide mass fail abruptly, perhaps triggered by an earthquake, it would have the potential of displacing a large volume of water in the harbor and creating a wave comparable to, or greater than, the destructive tsunami that hit the City of Kodiak during the 1964 Alaskan earthquake. The issuance of the notice of potential hazard had two positive effects: (1) alerting local officials and the populace to the danger, and (2) inducing the State of Alaska to provide funding for study, instrumentation, and monitoring of the slide to help in selecting methods of mitigation. On the other hand, local officials and the people of Kodiak generally were unhappy with the warning because of the alleged negative economic impact on the city and its port operations caused by publicity given the potential hazard. Not to provide such information, however, could have moral and possibly legal implications.

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T. Løken (Written discussion)

MAPPING OF POTENTIAL QUICK LANDSLIDE AREAS IN NORWAY

The geomorphology of Norway clearly reflects the glacial activity during several periods of extensive glaciations. In the valley and fjord districts of western and northern Norway characterized by steep mountain slopes and bedrock cliffs, snow avalanches and rockfalls occur every year. Whereas in the lowland regions around the Oslofjord and Trondheimsfjord, where large areas with soft postglacial marine clays are encountered, quick clay landslides are relatively common.

In connection with all kinds of natural hazards in Norway, a governmental organization, the National Fund for Natural Disaster Assistance, is responsible for damage compensation, site investigations and building of necessary safety structures. The Norwegian Geotechnical Institute has been engaged by this organization as technical advisory consultant in connection with snow avalanches, rock falls and slides, earth and debris flows and clay landslides. During the last years, three pilot projects on natural hazard mapping has been carried out (Hestnes and Lied, (1980).

Due to the time and scope limitations of this session, I will consentrate on one of these projects, the mapping of potential quick clay landslide areas. Marine clay deposits in Norway cover approximately 5000 km^2, and about 1000 km^2 of these areas may present a potential quick clay sliding danger.

The mapping started in 1980 in a pilot area of the Verdalen community in the Trondheim region, and yielded very promising results. In 1981, NGI has just started an eight-year national project of mapping 80% of the marine clay deposits. The remaining 20% is considered to lie in the small and isolated areas in the several narrow fjords along the coast of Norway, and is therefore much more expensive to map than major deposits in the Oslofjord and Trondheimsfjord areas.

The mapping project has the following basic principles:

- Only potential quick clay landslide areas are considered. The frequency of occurrence and the nature of quick clay landslides, i.e. very rapid failure without any previous warning warrant this. Quick clay landslides cause the largest life and property losses.

o The mapping aims at delimiting quick clay areas with unfavourable topography with respect to potential slide activity. In this stage of mapping, neither detailed geotechnical investigations nor calculations of slope stability are performed.

The mapping is organized as cooperative effort between the Norwegian Geological Survey (NGU) and the Norwegian Geotechnical Institute (NGI). An existing national programme by NGU to produce Quaternary maps has been accelerated to cover all actual areas with marine clays.

NGI's investigation is based on airphoto interpretation combined with topographical maps in order to register areas of potential landslides. NGI then checks if quick clays can be found in these areas mainly by simple rotational soundings.

If quick clay is found in such an area, the area is claimed as a potential risk of quick clay landsliding and zoned as such on the hazard maps. On the other hand, if no quick clay is found, the registered areas will have no zoning on the maps.

In Norway the national building code states: "Ground can only be built on if there is sufficient safety against subsidence, inundation, landslides, etc.". This implies that within the zoned areas, geotechnical consultants should be engaged in the future before any kind of construction activity is started.

The map with zoned areas "which have a potential risk of quick clay landslides", will be produced in the scale 1 : 20 000 with 5 m elevation curves. (In some areas a scale of 1 : 50 000 may be more economical). This level of mapping is financed by the Government, to a total cost of Nkr. 18 mill. It can be mentioned that the total refund by the insurance companies and compensation by the National Fund for Natural Disaster Assistance in connection to the Rissa landslide (Gregersen, 1981) was of the same order. This implies that if the mapping programme can prevent the damage from one new slide of the magnitude of the Rissa slide, the investment will have already been cost effective.

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J.N. Hutchinson, Co-Chairman

LANDSLIDE HAZARD ZONING. CLOSING REMARKS

For many geotechnical engineers, familiar with the considerable difficulty of deciding the factor of safety of a single slope, even with the benefit of a comprehensive site investigation and laboratory programme, there is a natural tendency to be sceptical of the value of the methods by which regional hazard zonation is carried out. I hope that this afternoon's discussion has shown something of the value of such zonations.

Before we leave this subject, it is well to remember that in many of the mountainous regions of the world, and particularly those such as the Andes and the Himalayas which are more tectonically active, the hazards presented by landslides can be particularly severe and lead to a great loss of life. The humanitarian and economic contributions which can be made by landslide hazard zoning in these regions is evident, and I believe that it is part of our general responsibility to do all we can to encourage and further such work.

A.J. da Costa Nunes (Oral discussion)

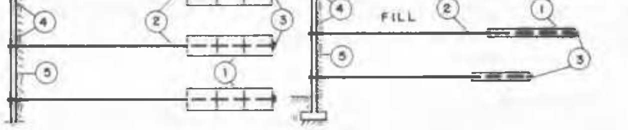
MICRO-ANCHORS

1. INTRODUCTION

The use of prestressed, grouted tie-backs in slope retaining walls during the last twenty years has inspired the development of a new, more economical method for supporting vertical wall-retaining fills using prestressed anchors installed during fill placement. This new process, called micro-anchoring, has been in use since 1977.

2. ANCHOR COMPONENTS (See Figure)

Micro-anchors consist of prestressed, ungrouted anchors with small working loads, composed of prefabricated horizontal plates set within the fill during its placement and compaction, and whose other components (rods and fixing heads) are similar to those used for grouted anchors. The retaining walls are made of reinforced concrete, poured-in-place or prefabricated slabs or small vertical columns joined to slabs, installed manually or by



using light hoisting equipment. The micro-anchoring plates which transmit the loads from the rods to the soil are made up of prefabricated reinforced concrete slabs (1) with cross-sections varying from 6 x 30cm to 8 x 30cm - and lengths from 2 to 5m. The anchor rods (2) pass through a hole along the longitudinal axis of the slabs and are fixed at their far end (3). The rods are steel bars or - wires although oftentimes bars made of resins reinforced with fiberglass are used in highly corrosive soils. The steel bars or wires should be protected by an anti-corrosive coating. The fixing heads(4) are simply a means of connecting the anchor rods to the wall(5) after the prestressing has been applied.

3. STATIC CALCULATIONS

A retaining wall is considered to be subject to active thrust loads from the proper soil mass and from any actual or future applied loads. The flexibility of the anchor rods permits the wall to suffer the necessary deformations for development of the active thrust.

Under the action of the horizontal components of these loads, the wall is supported by the anchor rods by placing them in tension. In the case of poured-in-place walls, the system can be considered to be similar, for static calculations, to a flat slab whose columns are the anchor rods. The calculations are made using the simplified methods which have shown to provide satisfactory results. After the working loads of the anchors have been determined, the bending moments for the various wall sections are calculated. The wall thickness is, in general, determined from the punching shear resistance to the load transmited from the anchor rods which should in turn be designed to withstand these loads and also those applied during tests.

4. ANCHOR STRENGTH

The capacity of each anchor slab depends on its shape, dimensions, depth and on the local soil characteristics. Under the loads of the anchor rod the slab resists, similar to a pile, through its tip resistance as well as through its perimeter resistance (friction and adhesion). From this principle comes the theoretical method for calculating the load capacity. In practice, these values are always tested during construction through the prestressing and testing procedures. Previously obtained test values can be used to evaluate strengths when soil and slab materials remain constant in much the same way as strengths for grouted anchors are based more on tests than on theoretical considerations for soil-anchor interaction.

Besides the soil-slab contact resistance, the compacted soil must have sufficient strength to resist the stresses from a single slab anchor as well as those from the group action of the slab anchors. There are various methods for evaluating this strength, called "internal strength", one of which is presented in the paper by Ranke and Ostermayer (1968).

In the study of soil strength it is also necessary to consider the creep effect which results in the shortening of the free length of the anchor rod with a decrease in its prestressing load.

Another failure form which should be examined is the so-called "external failure" considered to occur along a circular, cylindrical surface passing through the base of the wall (or below it if we are dealing with fill placed over soft soils) and behind the anchors.

5. TESTS

The test methods used for micro-anchors to verify their adequacy for the actual site conditions are those contained in Brazilian standard NB-565/77 for grouted anchors; that is, acceptance, basic and qualification tests.

The acceptance tests are performed on all the anchors before their being connected to the wall to verify their strength and that of the adjacent soil. The load is applied till 80% of the test load limit (which is at most equal to 90% of the yielding load for the anchor rods) at which time displacements are measured until stabilization is reached, although the load must be maintained for at least 15 minutes in cohesive soils or 5 minutes in non-cohesive soils. In ten percent of the anchors the loads are applied till the test load limit.

The basic and qualification tests are more sophisticated tests designed to determine more completely the characteristics of the anchors to be tested. The difference between them is that in the basic test a posterior excavation is made to examine "in situ" the anchor slab and its contact with the soil. The two tests are similar in all other respects: the anchors are loaded in various cycles from an initial value F_0, which is not greater than 10% of the anchor rod yielding load, and each cycle is composed of increasing load stages, varying at the most 15% of the rod yielding load, with each stage being maintained until stabilization is obtained. After the maximum load for each cycle is reached and stabilization is obtained, the load is reduced to the initial value F_0. A new load cycle is then begun.

Plots of load vs. displacement and time vs. displacement are made for all types of tests performed. Load vs. elastic displacement and load vs. plastic displacement plots are obtained from the load vs. displacement curves and are used to calculate and verify the free lengths of the anchor rods and soil-rod friction. The creep characteristics of the soil are determined for each load stage from the time vs. displacement curve.

6. REFERENCES

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6.2 Ranke, Armin and Ostermayer, Helmut (1968) - Beitrag Zur Stabilitatsuntersuchung. Mehrfach Verankerter Baugrubenumschliessungen. Die Bautechnik (10) - Berlin.

A.W. Malone (Written discussion)

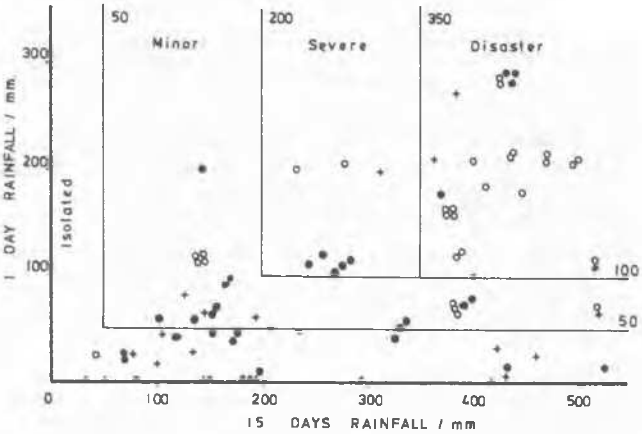
TIMING OF LANDSLIDES IN HONG KONG IN RELATION TO RAINFALL

This contribution deals with one aspect of observations of some 87 landslides recorded in Hong Kong in the years 1978, 1979 and 1980; namely the timing of the landslides in relation to rainfall.

Hong Kong receives on average 2.2m of rain annually. In 1978 2.1m of rain fell in the 6 summer months of May to October. 51 landslides were recorded by the government department concerned with building safety, although undoubtedly many more occurred. In 1979, 28 landslides were recorded and in 1980, a very dry year, only 8 were recorded. (The 87 cases reported here include failures in soil and decomposed rock cuttings and fill slopes, boulder falls and collapses of retaining walls. 10 more cases are known, but full details are not available). Most of the landslides were small, being less than 100 cu.m. in volume and quite shallow, having depth to length ratio less than 0.2. However, several were more than 15m high.

Now, it is intuitively obvious that a rainstorm is more likely to give rise to landslides if it occurs at a time when the ground is already saturated. To examine the importance of antecedent rainfall the 87 recorded landslides of the years 1978, 1979 and 1980 were plotted in the form suggested by Lumb (1975). Rainfall in the 15 days prior to the landslide is plotted against

rainfall on the day of the landslide. On this basis Lumb distinguished between "disaster" days, "severe" days and "minor" days, his classification being based on number of incidents per day.



Data for 1978 – 1980

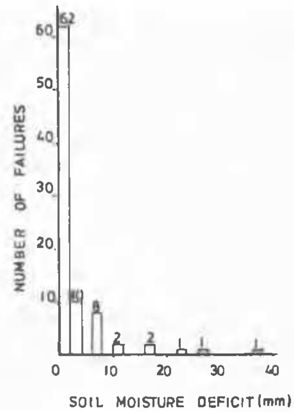
- o Landslides occurring on a day when 8 or more landslides were recorded (3 days thus)
- + Landslides occurring on a day when 1 landslide occurred (22 days thus)
- • Remaining landslides (15 days)

ANTECEDANT RAINFALL AND PREDICTIVE ZONES

after Lumb 1975

It will be seen from examination of the above figure that the data for the 87 recorded landslides in the years 1978 to 1980 are disposed generally as might be expected, in that the data for landslides occurring on a day when only one landslide occurred tend to fall in the less severe zones than the data for landslides occurring on a day when many landslides occurred. However the data are not so well disposed as those presented by Lumb and the difficulties of prediction will be apparent; though it must be said that this data is sparse compared to the data cited by him which spans many years, including much more severe years than 1978, 1979 or 1980, and does include a predominance of small landslides.

Prompted by the work of Crozier and Eyles (1980) in New Zealand, the timing of failures in relation to soil moisture deficit was examined. The soil moisture deficit data was taken from work by Leach and Herbert (1982) in Hong Kong.



It is apparent from the above figure that over 70% of the failures recorded took place on a day when the soil moisture deficit was close to zero - 2mm or less.

In Hong Kong, where landslides pose a serious social and economic problem a reliable landslide warning system would be of great value. These results suggest that a warning system based on soil moisture deficit and daily rainfall forecasts may have promise.

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J.N. Hutchinson, Co-Chairman

SOME ADDITIONAL REMARKS

Concerning methods of giving warning of landslides, it seems generally more promising, and direct, to monitor pre-slide movements although, as shown by G. Cartier, monitoring of the corresponding pore-water pressures may also be helpful.

As a result of the great variety of good instruments that have been developed, case records of pre-slide movement are becoming numerous. In addition, it is now quite feasible to install systems which are capable of giving automatic warning of impending failure. The problem remains, however, of deciding what limits of movement can be tolerated in any particular case. To make progress in this area, we need, on the one hand, to continue to make high-quality observations of pre-failure movements in all types of soil and rock, and on the other hand, to develop sounder geotechnical/rheological theoretical and experimental bases against which to compare and evaluate our field measurements.

N. Janbu, Chairman

CLOSURE OF SESSION AFTER ORAL DISCUSSION

In a short while the Closing Session for the whole conference will take place in this room. We will therefore have to wind up Session 11 now.

I hope you will agree with us that as a whole this Session proved to be successful, particularly in the sense that the floor has been very active during discussion periods. Despite strict timing, several contributions from the floor had to be referred to written contributions. We apologize for that.

A very important part of the Session dealt with landslide warning systems, methods of prevention, and landslide hazard zoning. These topics were given particular attention, after introductory surveys from several parts of the world. New ideas about the possibility of advance detection of landslide have also been presented to us. On the other hand, the important legal aspects of property reevaluation due to hazard zoning was not covered adequately. For several reasons, therefore, it is advisable that these new topics form a part of future sessions on slope stability.

The classical topics of slope stability, such as numerical analyses, short- and longterm considerations, and case record evaluations received, as usual, considerable attention.

From a fundamental point of view, no major new ideas were brought in, but evidently statistics, probability and variational techniques are being used to increasing degree. So also are finite element analyses, finite differences, methods of characteristics and velocity fields. Nevertheless, it is still the modelling of soil behaviour, and the corresponding soil parameters that decide the reliability of the final numerical result.

I will summarize my own opinion about today's state of art in slope stability analysis as follows:

- we are overqualified to carry out numerical analyses.
- we are, by comparison, gravely underqualified to model actual soil behaviour.

Let this be a challenge to us all for the future!

Along the same line: The task of designing a stable slope requires knowledge of soil behaviour during a transition from one state of equilibrium to another, different state of equilibrium - but still below failure, for sufficient safety. Since safe design is a major part also in geotechnical engineering, I draw the following conclusion:

- we are doing much too much testing on "strength" alone
- by comparison, the prefailure behaviour of soil still remains almost unknown, both from a fundamental and a practical point of view.

Please, let us do something about it, to remedy an almost intolerable situation.

In closing, let me, on behalf of the chairmanship, be allowed to express our gratitude to all contributors and participants to this session. Thank you - all of you!

P.M. Acevedo, A.L. Kurzulovič and J.P. Molina (Written disc.)

ABOUT THE REGRESSIVE ANALYSIS APPLICATIONS

The Authors feel that the regressive analysis methodology of slope stability described in their paper[1], should be accomplished with an example. For this purpose, it was analysed the slope of figure 1, considering

$\gamma = 1.85\ (t/m^3)$ and $\gamma_{sat} = 2{,}10\ (t/m^3)$

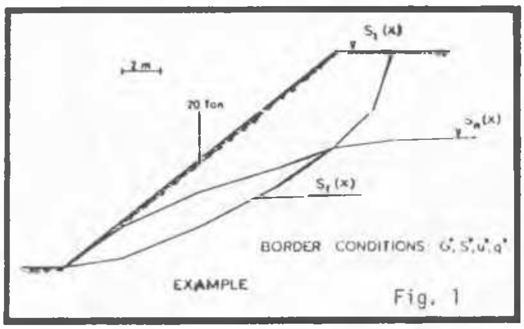


Fig. 1

For the static case, defined by $k_h = 0.0$ the results of the Janbu's method of analysis[2] are:

$F(\overline{c} = 1\ (t/m^2),\ \overline{\Phi} = 15^{\circ}) = 0.814$

$F(\overline{c} = 5\ (t/m^2),\ \overline{\Phi} = 35^{\circ}) = 2.985$

then applying equation (1):

$0.814 = \xi^* \cdot 1{,}0 + \Omega^* \cdot \mathrm{tg}\ 15^{\circ}$

$2{,}985 = \xi^* \cdot 5.0 + \Omega^* \cdot \mathrm{tg}\ 35^{\circ}$

where:

$\xi^* = \xi\ (G^*, S^*, u^*, q^*)$

$\Omega^* = \Omega\ (G^*, S^*, u^*, q^*)$

Solving the system of equations, is possible to obtain:

$\xi^* = 1.695$

$\Omega^* = 0.360$

Then for the slope of figure 1 and $k_h = 0.0$, the factor of safety is defined by:

$$F = 1.695 \cdot \mathrm{tg}\overline{\Phi} + 0.360 \cdot \overline{c} \qquad \text{(a)}$$

In the same way, for the seismic case, is possible to obtain:

$$F\ (k_h = 0.1) = 1.460 \cdot \mathrm{tg}\overline{\Phi} + 0.321 \cdot \overline{c} \qquad \text{(b)}$$

and

$$F\ (k_h = 0.2) = 1.265 \cdot \mathrm{tg}\overline{\Phi} + 0.290 \cdot \overline{c} \qquad \text{(c)}$$

In figure 2 are plotted the relationships (a), (b) and (c), and the results obtained directly by means of the Janbu's method of analysis (dots). This figure indicates a good agreement of the regressive relationships for F and the results obtained performing directly a slope stability analysis.

Finally, is possible to analyse the effect of the scattering of $\overline{c}$ and $\overline{\Phi}$. It is assumed that $\overline{c}$ has a mean value of 1 (t/m^2), a maximum probable of 1.3 (t/m^2) and a minimum probable of 0,7 (t/m^2), and $\overline{\Phi}$ a mean value of 30º, a maximum probable of 32º and a minimum probable of 28º.

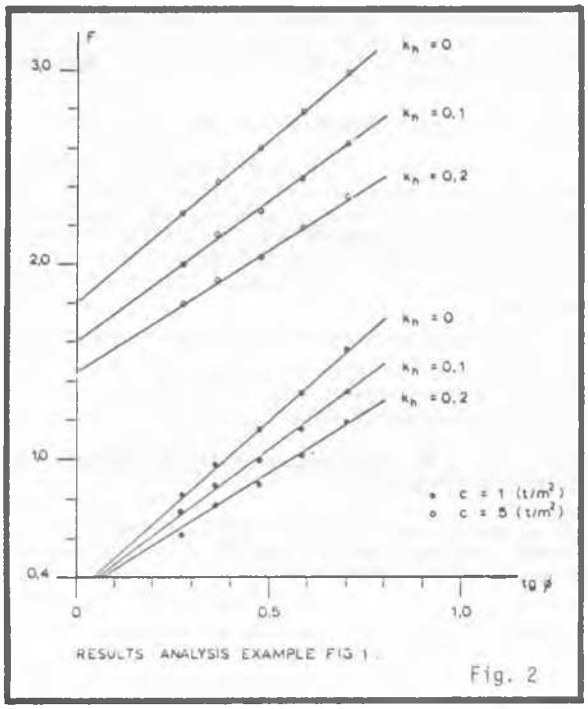
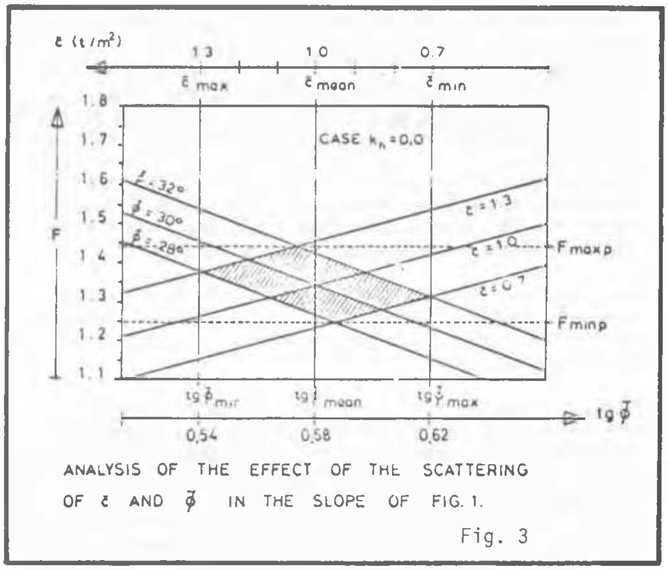


Fig. 2



ANALYSIS OF THE EFFECT OF THE SCATTERING OF c̄ AND φ̄ IN THE SLOPE OF FIG. 1.

Fig. 3

Then is possible to draw the system of axis of figure 3 and using the relationship (a), draw the curves with cohesion constant $\bar{c}$ = 0.7 (t/m²), $\bar{c}$ = 1.0 (t/m²) and $\bar{c}$ = 1.3 (t/m²), and the curves with angle of friction constant $\bar{\Phi}$ = 28º, $\bar{\Phi}$ = 30º and $\bar{\Phi}$ = 32º. The intersection of these curves defines the most probables maximum and minimum values of the factor of safety: F_{maxp} = 1.44 and F_{minp} = 1.25, because the values maximum maximorum and minimum minimorum, F ($\bar{c}$ = 1.3 (t/m²), $\bar{\Phi}$ = 32º) and F ($\bar{c}$ = 0.7 (t/m²), $\bar{\Phi}$ = 28º) are extreme cases.

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T.K. Chaplin (Written discussion)

ON CLOSING REMARKS BY THE CHAIRMAN

The Session Chairman (Prof. Janbu) has said that we are 'overqualified' in analysis; this seems open to question (for instance, de Josselin de Jong has pointed out that variational methods are invalid for slope stability). In current methods there are snags, and this contribution will also look at the work principle.

For the usual limit equilibrium, we use forces and moments. Unless the system is exactly in balance, *uniform* displacements (rotations) are implied when equating forces (moments). To get final equilibrium for a 'rigorous' solution, we typically have to use unbalanced forces. Thus we relate gravity & surcharge force movements to implied movements along the slip surface. Other (e.g. graben) deformation modes are *not* checked.

For a routine job it would hardly be practical to fully define the moving soil mass. But we might assume a simple *displacement pattern* in it by Eq. (1), see Fig. 1. At point P, velocity is

$$\vec{V}_P = \left(\sum_{n=1}^{k} \vec{V}_n \cos^6\theta_n L_n / r_n^{\,2}\right) \Big/ \sum_{n=1}^{k} \cos^6\theta_n L_n / r_n^{\,2} \qquad (1)$$

where $\vec{V}_n$ applies at the midpoint of section n of length L_n. Work by gravity might use several parts of a slice. Internal shearing would often be ignored. Weights and surcharges are not now transmitted vertically downwards, so a more accurate vertical pressure might be given near the centre of a dam.

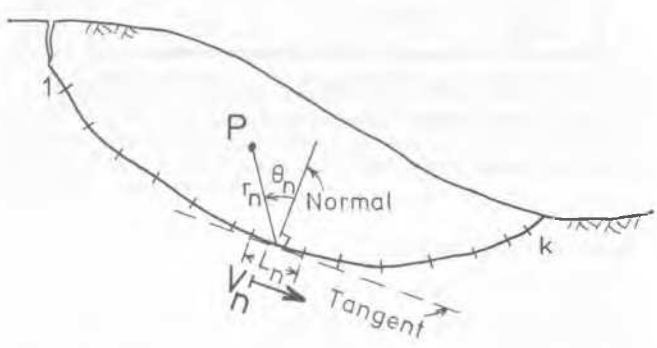

Fig.1 Velocity by weighted mean at point P

In Britain, one assumes the main computer slice programs would be based on Bishop for circular surfaces, Morgenstern & Price for non-circular ones. Neither seems to specify any action if tension is implied, whether between slices or on slice bases (where u is large). An acceptable 'rigorous' analysis would have the thrust line well within the moving mass, near the slip surface if close to a tension crack. Using the Morgenstern & Price method (with Morgenstern's own program, kindly given to the writer), one expects some of the thrust line outside the moving mass (even going off to infinity) if any tension occurs near the knee of the slope. The method may fail to converge for obscure reasons.

A slip surface which is a *logarithmic spiral* for reduced ϕ', or a surface with the same physical property, has the same F for *any* normal pressure distribution on the failure surface (compatible with applied forces). If numerical methods omit steps to handle this, they could break down. As the worst surface in uniform soil might be close to a spiral, this may not be a remote problem, but F is then insensitive to assumed stresses.

The 'conventional' method, as used by May and Brahtz in 1936 or earlier, can be made better in two ways: scaling all normal forces (by the same multiple, slightly above 1.0 in general) to get *vertical force balance*, and *only* where a slice is moving upwards, basing its base total force on W.secα instead of W.cosα (before scaling, the normal stress would equal the overburden stress). Especially if u is high, with these two changes it might give a good initial F value for a more complicated method.

To sum up, a 'black box' is only an aid to our engineering judgement and geological sense, and we must always stay wary. It would be wise to use more than one method on most near-critical failure surfaces, especially if the lowest acceptable F value is near or below 1.5.

S.S. Coric (Written discussion)

A CONTRIBUTION TO RESEARCH ON THE STABILITY OF NATURAL SLOPES
Contribution à la Recherche de Stabilité des Pentes Naturelles

This contribution concerns the natural multi-layered slope stability (Mokroluški stream, south of Belgrade) on which a nine floor apartment building with a slab type shallow foundation has been built. The stability of the slope has been analyzed by the finite element method.

On the basis of data from standard laboratory triaxial tests (CU, dia 3,81 cm) the stress - strain relationship for the various soil and rock zones have been obtained (Fig. 1.).

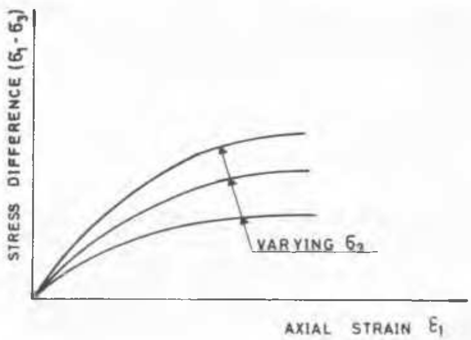

Fig. 1. Stress-strain curves (schematic)

An analytical stress-strain model is then developed

$$\sigma_1 - \sigma_3 = a \cdot \frac{\varepsilon_1}{b + \varepsilon_1} \qquad (1)$$

where

$$a = \alpha_1 + \beta_1 \cdot \sigma_3 + \gamma_1 \cdot \sigma_3^2 \qquad (2)$$

$$b = \alpha_2 + \frac{\beta_2}{\sigma_3} + \frac{\gamma_2}{\sigma_3^2} \qquad (3)$$

In these equations $\alpha_1 \dots \gamma_2$ are parameters whose values may be easily determined from the results of triaxial tests.

The stress-strain relationship expressed by eq. (1) may be most conveniently employed in non-linear stress-strain analyses since it enables the tangent Young's modulus corresponding to any point on the stress-strain curve to be determined. If the value of the minor principal stress is constant, the tangent Young's modulus E may be expressed as

$$E = \frac{d(\sigma_1 - \sigma_3)}{d\varepsilon_1} = \frac{a \cdot b}{(b + \varepsilon_1)^2} \qquad (4)$$

For the purpose of the study described herein Poisson's ratio was assumed to be constant for each zone of the ground.

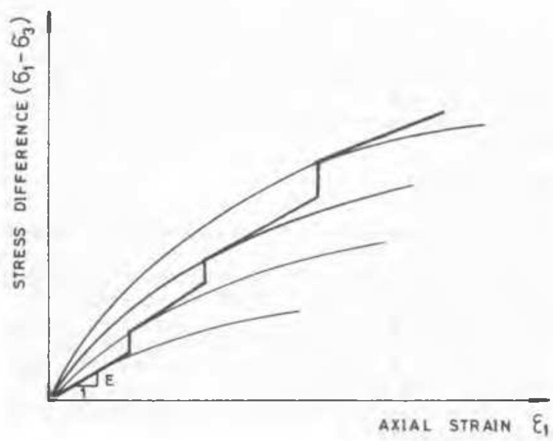

Fig. 2. Scheme of the incremental procedure

The weight of the building was applied in a series of increments. The proposed incremental procedure takes into account the influence of stress level and confining pressure on tangent modulus values (Fig. 2.).

The finite element analysis indicated the zone of greatest deformations. Hence, position of likely slip surface is determined.

The safety factor for the slope is defined by the limit deformation method

$$F_S = \frac{\tau_L}{\tau_S} \qquad (5)$$

where

τ_L - shear strength for the limit deformation ($\varepsilon_1 = \varepsilon_{1L}$)

τ_S - shear stress for the actual deformation ($\varepsilon_1 = \varepsilon_{1S}$)

The proposed safety factor is a function of both stress-deformation and strength characteristics of soils.

J.N. Hutchinson and R.K. Bhandari (Written discussion)

"STABILITY OF MUDFLOWS ON NATURAL SLOPES"
(Paper 11/43, Vol. 3, by L.E. Vallejo)

There is much that we disagree with in this paper. Although initially we accepted the, then current, term "mudflow" to describe the mass movement at Beltinge (Hutchinson 1970), we rapidly concluded (Hutchinson and Bhandari 1971) that such features, moving predominantly on discrete boundary shears, would more properly be termed "mudslides". The term mudflow is now generally confined to the very rapid, wetter, less clayey and more stream-like mass movements which characterise many mountainous regions. In his paper, the Author seems to use this term to mean sometimes mudflow and sometimes mudslide, also without distinction to whether these are occurring in a sub-aerial or a sub-marine environment.

Further confusion results from the introduction of Campbell's "sliding and streaming stages". While we do not question the validity of these in the cases studied by Campbell (1975), it is incorrect to apply them to the Beltinge mudslide, as is done in the paper.

The Author states that his stability analysis applies to the "streaming stage" of a "mudflow", but the treatment, in fact, addresses only the basal sliding case. A detailed critique of the method of analysis adopted, the values of parameters assumed and the conclusions reached will be given elsewhere. The main present comment is that the relevant c_u value in equation (7) is that obtaining at the basal slip surface of the mudslide. In his Table II, however, the Author uses a value which applies just beneath the dried crust, typically 3 to 4 metres above the basal slip surface (Hutchinson 1970). The factor of safety of 0.9 that is thereby calculated is therefore meaningless and there is no basis for the Author's conclusion 3. Similarly little weight can be attached to the factor of safety quoted in Table II for the experimental mudflow, on account of the tiny scale of this, the absence of the "particulate structure" assumed in the Author's theory and the failure to establish the reliability of the c_u value of 0.03 kN/m² that is used.

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E. Ledeuil,(Written discussion)

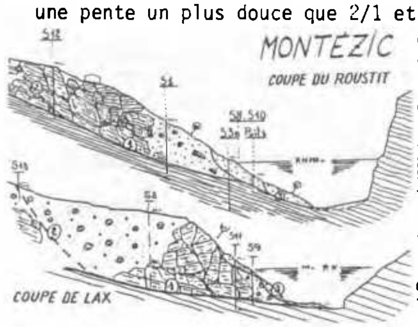
ANALYSE D'UNE STABILITE DE PENTE DE TERRAINS EN PLACE.
GLISSEMENT DE LAX ET DU ROUSTIT

RESUME: Il s'agit après diverses reconnaissances d'utiliser le programme "STABAR" mis au point pour l'étude de stabilité pseudo statique de pentes sur des matériaux non homogènes à lignes de rupture composites. (1 cercle suivi d'une droite inclinée et se terminant par un cercle... choisi par le programme en fonction de la nature du matériaux.)

1. ANALYSE DU PROBLEME

1.1. Un grand glissement d'origine quaternaire s'étend sur 1 km de long et 500 mètres de hauteur. Un talus d'équilibre existe mais quelques loupes proches de la retenue de Couesque (qui en baigne le pied) se remettent en mouvement à la suite de fluctuations rapides du niveau souvent en corrélation avec une forte pluviométrie. Or, l'exploitation de la retenue de Couesque dans le cadre de l'aménagement de transfert d'énergie de Montézic va amplifier et rendre plus fréquents les mouvements d'eau au pied du glissement.

1.2. Dans la zone du Roustit le bed-rock a été trouvé avec une pente un plus douce que 2/1 et le terrain de surface avec une pente de 3/2 dégage un dièdre d'épaisseur croissante avec la hauteur. Sur le bed-rock une couche poreuse de cailloux broyés ($\varphi = 39°$) de faible épaisseur est le siège des circulations d'eaux. Sur cette couche se trouve une couche à matrice argileuse.



1.3. Toutes les couches diminuent de puissance en pénétrant dans le lac, la couche de cailloux drainés ne fait plus que 20 cm d'épaisseur à 25 mètres sous le niveau normal de la retenue.(visible dans le puits).

2. MECANISME DE RUPTURE

2.1. Si de grosses pluies alimentent cet horizon poreux surmonté d'une couche épaisse étanche, ou si la retenue baisse rapidement, la quantité d'eau ne peut s'évacuer assez vite et l'horizon reste en pression telle une canalisation. Les pressions crées et les faibles caractéristiques des matériaux entourant cet horizon poreux créent les conditions de rupture soit au contact du rocher créant ce même matériau poreux soit juste au dessus dans les matériaux argileux.

2.2. Par contre, une rupture plus importante du versant semble peu probable car la nature caillouteuse des matériaux après rupture avec élimination plus rapide de l'eau en exès et la pente globale soit du substratum soit du talus extérieur doivent permettre de retrouver très vite l'équilibre précaire et qui restera toujours précaire, l'avancement du talus est comparable à celui d'un glacier.

Une seule solution permettrait une stabilisation......... Il s'agirait de buter le pied par un apport de matériau en fait cet apport se fera tout seul petit à petit, sans risque de voir la vallée se boucher (il y a 15 mètres de hauteur de disponible au fond pour laisser une exploitation normale, et l'apport prévisible à chaque rupture est limité à quelques centimètres, au vu des relevés effectués depuis 4 années).

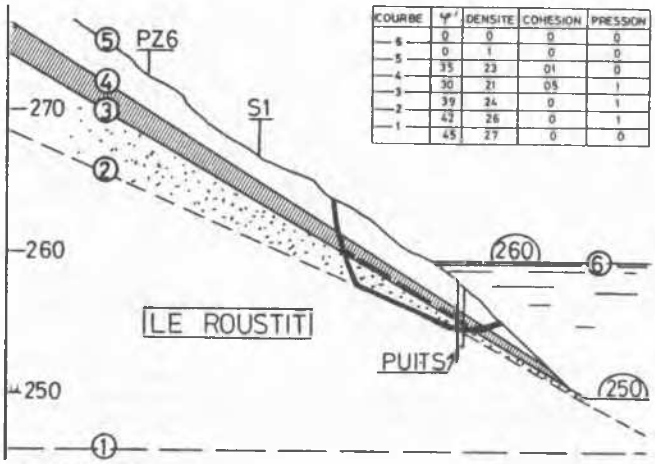
3. UTILISATION DU PROGRAMME "STABAR"

La ligne de rupture composite utilisée laisse apparaître:

3.1. Un cercle à l'entrée qui se rapproche d'une droite inclinée sur la verticale

3.2. Une droite inclinée (le programme peut ou non choisir cette ligne ou un cercle complet comme pour la zone de LAX ou le bed-rock est plus profond).

3.3. Un cercle pour sortir à un point défini que l'on peut faire varier bien entendu. Le programme en choisit le rayon et la position pour qu'il soit tangent à la droite citée en § 3.2., et qu'il sorte en faisant un angle $(\frac{\pi}{4}-\frac{\varphi}{2})$ avec l'horizontale.



COURBE	φ'	DENSITE	COHESION	PRESSION
	0	0	0	0
6	0	1	0	0
5	35	23	01	0
4	30	21	05	1
3	39	24	0	1
2	42	26	0	1
1	45	27	0	0

4. RESULTAT

Sans entrer dans les détails on obtient rapidement une stabilité limite, les conditions de pression d'eau étant prépondérantes.

R.W. Lumsdaine and K.Y. Tang (Written discussion)

ON STABILITY ANALYSIS IN HONG KONG

Several of the conference papers discussed various methods of slope stability analysis and their relative abilities to produce meaningful and useful results. Of course, new improved methods of analysis are useful only if they are used carefully and correctly in the first place. This aspect was the subject of an exercise that was recently carried out by the Geotechnical Control Office (GCO) of the Hong Kong Public Works Department.

In Hong Kong, the minimum acceptable factors of safety against failure of a soil slope (using limit equilibrium analysis) under various conditions have been specified in the Geotechnical Manual for Slopes (Geotechnical Control Office, 1979). The hilly natural terrain makes it necessary to work to factors of safety as low as 1.2, and so the implications of errors in the formulation and application of the various methods of slope stability analysis are very serious. This situation prompted the GCO to initiate an exercise to determine the quality and computational correctness of slope stability analyses that are carried out by both private firms and Government offices using various types of calculators and computer programs.

The basis of the assessment was a set of 29 test problems that consisted of several well defined cross-sections and slip surfaces with various combinations of soil strength parameters, water conditions and surcharge loadings. These problems were sent to all private firms and Government offices that are involved in the assessment of stability of soil slopes. Fifty-nine sets of solutions were submitted to the exercise. Eleven of the solution sets used 'rigorous' methods of analysis (Morgenstern and Price, Janbu Rigorous, Sarma), 43 sets used a 'simplified' method of analysis (Janbu's Routine Procedure), and 5 partial sets (which were ignored) used 'other' methods (Bishop Simplified, Modified Fellenius).

When the 'rigorous' and 'simplified' results were plotted separately, a wide spread of answers was observed for nearly all the problems (Fig.1). Each submission was carefully scrutinised and all apparent sources of error were identified. Participants whose submissions contained errors were asked to correct, recalculate and resubmit their problems.

The following types of errors were detected in the 43 sets of 'simplified' solutions :

(a) Incorrect use of Janbu's correction factor - 74% of sets

(b) Incomplete convergence - 22% of sets

(c) Incorrect input data
- significant number - 50% of sets
- minor number - 30% of sets

It was difficult to determine the reason for the spread of answers from the 11 sets of 'rigorous' solutions due to the more complicated analytical aspects of the methods and the incompleteness of output that was produced by most of the programs. Four of the programs were found either to have serious deficiencies or to have been used incorrectly. Very few errors in input data were detected.

Once errors had been corrected and seriously deficient programs had been eliminated, the agreement between answers from a given type of analysis for any given problem was usually quite reasonable (Fig.2). On the basis of the set of 29 test problems, the 'simplified' and 'rigorous' methods of analysis produced similar results for problems with cohesive soils. In problems with cohesionless soils the 'simplified' methods of analysis produced lower factors of safety (average - 8%) than 'rigorous' methods.

The main conclusions concerning the quality of slope stability analysis in Hong Kong that were drawn from this exercise were :

(a) The majority of firms in Hong Kong that carry out soil slope stability analysis use Janbu's Routine method of analysis.

(b) The majority of these firms have been using this method in an incorrect manner.

(c) Many firms use this method despite having access to sophisticated computer hardware.

(d) Most users of slope stability analysis programs in Hong Kong are very careless with the accuracy of the input data, even under test conditions.

(e) The overall standard of documentation of slope stability analysis programs in Hong Kong is poor.

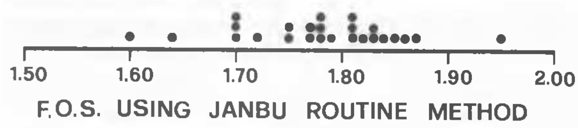


Fig.1 Initial submissions for problem no. 28

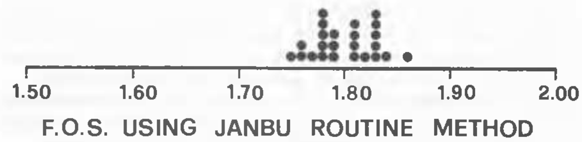


Fig.2 Recalculated submissions for problem no. 28

REFERENCE

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M. Popescu (Written discussion)

COEFFICIENT OF EARTH PRESSURE AT REST IN SLOPE STABILITY

The relevance of in-situ stresses to stability of altered slopes, including cuts and embankments, was not widely recognized until the finite element method became available as a valuable tool for stress analysis (Popescu, 1978). The inherent stability of natural slopes may be also considered in relation to the existing stress field in the slope (Chowdhury, 1978).

To study the influence of in-situ stresses on the inherent stability of uniform natural slopes against failure along planar surfaces, a series of analyses have been performed using various values of the coefficient of earth pressure at rest, K_o, defined as the ratio of the effective normal stresses on horizontal and vertical planes respectively. The existing stress field in an uniform slope is related to the conjugate stress ratio, K, defined as the ratio of the total stress on a vertical plane, acting parallel to the slope and the total stress on planes parallel to the slope, acting in a vertical direction. The relationship between K and K_o is dependent upon the slope inclination, β, and the pore water pressure ratio, r_u, as shown in fig.1. When $\beta=0$ and $r_u=0$, $K=K_o$.

The directional factor of safety, F_s, is defined as the ratio of shear strength to shear stress on the relevant plane. Considering long-term stability of a cohesive slope at the residual state as regards its shearing resistance ($c=c_r=0$, $\emptyset=\emptyset_r$), plots of $F_s/tg\emptyset$ for different values of failure plane inclination, α, and slope inclination, β, are shown in fig.1a and 1b, respectively. For any failure plane wich is not parallel to the surface of a slope, the factor of safety is highly dependent on the value of K_o. It is intersting to find that for horizontal failure surfaces the factor of safety becomes infinite only when K=0.

While the K_o coefficient is one of the key parameters in analysis of both natural and altered slopes, there is no satisfactory laboratory method of measuring K_o on "undisturbed" samples, due to the stress relief effects on laboratory samples. The in-situ measurements of lateral stresses is possible by means of total pressure cells, hydraulic fracture of pressuremeter tests. The hydraulic fracture method cannot be used in soils with $K_o>1$ because horizontal fractures would form and the test would measure the overburden pressure. For these soils the use of self boring pressuremeter is advocated (Wroth, 1975).

Taking into account the above mentioned considerations, the careful laboratory tests performed by Brooker and Ireland (1965) to measure magnitude of horizontal stresses exerted by soil against unyielding horizontal constraint under increasing and decreasing vertical stresses, appear valuable in determing the K_o as a function of the stress history and other properties of the soil studied. On the basis of Brooker and Ireland data, completed with self boring pressuremeter data available in literature, the following relationship was derived for determining at rest coefficient K_o as a function of the overconsolidation ratio OCR and plasticity index, I_p:

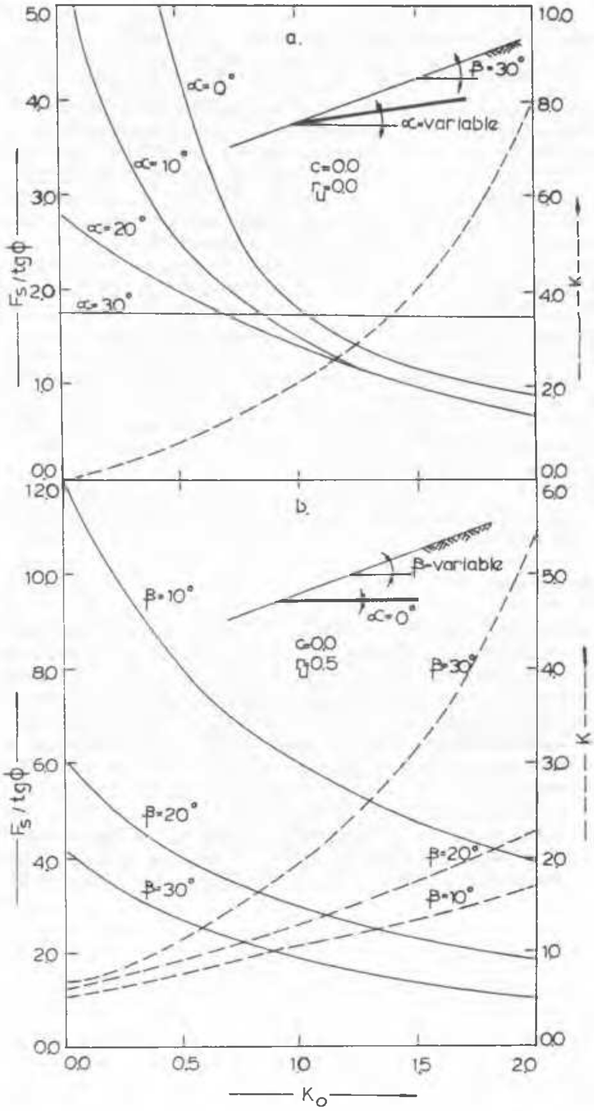

Fig.1

$$K_o = OCR \cdot K_{nc} - (OCR-1)\frac{\ln OCR}{\sqrt{OCR}}(0.45+0.005\ I_p) \qquad (1)$$

where K_{nc} denotes the K_o value for normally consolidated clay, wich may be estimated from the simplified Jaky relationship ($K_{nc}=1-\sin \phi'$) or in the absence of an appropiate value of the effective angle of shearing resistance, from the empirical relationship :

$$K_{nc} = 0.27 \log I_p + 0.20 \qquad (2)$$

It is to be noted that equation (1) reproduces the experimental evidence wich shows that an optium condition exists at wich the cohesion and the frictional effects retain the greatest radial stress and consequently display the highest K_o value. Due to this fact at an OCR>10, clays of medium plasticity develop higher value of K_o as compared to either low or high plasticity clay. The proposed correlation may be subjected to revision as additional information becomes avaible, especially from the measurements made with the promissing new self-boring instruments. However it must be emphasise that such correlations are not a substitute for fuller site investigation, but only an adjunct that should increase the engineer's confidence in his judgement.

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G. Ter-Stepanian (Written discussion)

CREEP IN ALPINE CLAYEY FORMATIONS
Fluage des Formations Argileuses Alpines
(Paper 11/14, Vol. 3, by M. Dysli and E. Recordon)

M.Dysli and Prof. E.Recordon presented an excellent paper on relationship between the rheological properties of soils and the long-term behavior of landslides; this problem is amongst the most important ones in modern science on landslides and in the soil rheology both.
Attempts made so far showed that this problem is difficult and depends on many factors. The Authors studied in detail three cases and found that the soil should not be considered as a rigid - perfectly plastic material.
The landslide at the Bois Carrien-Coupe was obviously in the phase of depth creep before the placing of the fill; the creep rate decreased at first under the weight of the fill and increased afterward as a result of the continuing creep. Analogous case was observed in Sochi on the Black Sea shore;use of creep hodographs was advantageous in this case (Ter-Stepanian,1971).
The landslide at the Villarbeney is of great interest because of the unusual soil properties and type of slope deformation. The zone of the depth creep bulged downhill while the creep rate decreased because of the landslide widening.
The real behavior of slopes is a result of the superposition of influences of seasonal changes (moisture, temperature, etc.) on the general trend. From this standpoint the young and old slopes are quite different. In young slopes as that of cuttings or fills the age of the induced stresses is short, the dynamic viscosity increases rapidly with the time and it is possi-

ble to detect the general trend in spite of the inevitable influence of the seasonal effect. In old slopes the stress age is too great compared with the duration of the study and even with the human lifetime; therefore the change of the dynamic viscosity is negligible and the influence of the seasonal effect remains only.
The explanation of changes of the creep rate in both cases - Bois Carrien and Villarbeney given by the Authors seems to be correct and adequate. A special attention deserve the results of the laboratory tests. In experiments with a rotational device where the normal and shear stresses were applied simultaneously the Authors found that the dynamic viscosity at high shear stresses depends to a less degree on the void ratio than at low shear stresses. This result is understood since the higher the shear stress the greater the stress dependent part of the strength, i.e. of the friction. Our early tests with the mica powder (a material with no cohesion) showed that if the normal and shear stresses both are applied simultaneously the stress-strain relation is smooth (Ter-Stepanian,1936). It should be emphasized that results of the laboratory rheological tests are of limited value in analysis of the depth creep of old slopes. Laboratory samples are small, the sampling is connected with the disturbance of the soil structure and considerable change of the stress state, the duration of tests is too short, the geological features cannot be modelled satisfactorily. Therefore the use of the field measurements is preferable being free of these limitations.
The writer proposed a method enabling to determine the soil viscosity based on the inclinometer measurements of the planar creep (Ter-Stepanian and Simonian, 1978). The dynamic viscosity η is

$$\eta = \frac{B\,T}{\gamma_z}\,\frac{a' + \sigma'}{A + \rho' z}\,(z - d_s)$$

where $a' = c' \cot\varphi'$; $A = a'/g\cos^2\beta + \rho_w d_p$; $B = A\rho\tan\beta/(A + \rho' d_s)$; c', φ', σ', ρ_w, ρ, ρ', g and β are the International Symbols in Soil Mechanics; z is the depth of the plane under consideration; d_p is the depth of the piezometric level; d_s is the depth of lower boundary of the rigid zone; T is the duration of creep observations and γ_z is the angular creep strain corresponding to this time interval.
Based on data contained in the paper by M.Dysli and E.Recordon the dynamic viscosity of the soil in Villarbeney (borehole F1, depth z = 15 m) was determined approximately, assuming that c' = 10 kPa and the groundwater level is at the surface (d_p = 0); the rest data were obtained from the Fig. 6 of the paper: d_s = 11.7 m; T = 515 days = $= 4.45 \cdot 10^7$ s; γ_z = 0.049. Using these data we get the field value of the dynamic viscosity of the soil $\eta = 2.76 \cdot 10^9$ kPa·s.
The use of the true initial data will enable to get more relevant value of the field viscosity of the soil.

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G. Ter-Stepanian (Written discussion)

STABILITY ANALYSIS OF MUDFLOWS ON NATURAL SLOPES
Analyse de la Stabilité des Ecoulements de Boue sur des Pentes Naturelles
(Paper 11/43, Vol. 3, by L.E. Vallejo)

Prof. L. E. Vallejo presented a valuable paper on mudflows containing important statements.The following discussion is aimed at bringing together our standpoints on the mechanism of mudflows. The paper deals with a phenomenon which has not been studied sufficiently until now. There is not even a common opinion about the terminology and definition of mudflows. A number of synonyms is used in technical publications: mudflow, debris flow, debris avalanche, etc. A comprehensive review of this problem was made by Varnes (1978). The phenomena referred to by the Author having such low velocities as 2 mm/day or even 4.5 m/day may hardly be called mudflows; they cannot cause any severe loss of life. They should be rather denoted as earthflows.
The real mudflows causing severe loss of life and property, as mentioned by the Author are widespread in young mountain countries; these disasters are rapid events of short duration and have maximum velocities measured by meters per second. Their mechanism remains still poorly investigated.

There are at least three enigmatic contradictions in the mudflow behaviour which may serve as touchstones for every theory of mudflows:
1. Mudflow masses have a very high mobility allowing them to travel with a great velocity and at the same time they have a very low water content usually not exceeding 14-15%; on the other hand the mud masses are able to be easily solidified, when they are stopped, transforming into a dense mass without any sorting, sometimes with some small discharge of water.
2. The traveling masses have an insignificant corrasion effect on the mudflow beds; on the other hand they are able to be enriched maximally by fragments extracted from their beds.
3. The traveling masses are able to transport on great distance large boulders and rock fragments in suspended state, the volume of fragments exceeding 1 m^3 and even reaching up to 4-5 m^3, although the density of the mud matrix (about 1600 -1700 kg/m^3) is much smaller than the density of the floating fragments (about 2200-2400 kg/m^3) and therefore the buoyancy is unsufficient to

keep these fragments in the suspended state. The writer has proposed several years ago the theory of the avalanche-type mechanism of cohesive (hydrodynamic) mudflows (Ter-Stepanian, 1968) which explains their behaviour as follows. Loosely detrital and fine disperse material is formed in mudflow seats as a result of physical and chemical weathering; on ample watering this material is able to transform into a mudlike mass rich in colloidal and clay particles. Heavy rains change this material into a viscous clayey matrix with coarse fragments flowing downhill. The density of the clayey matrix is much higher than that of the water (e.g. if $\rho_s = 2700$ kg/m^3 and w = 120% the density of the matrix will be $\rho_m = 1400$ kg/m^3). However the submerged density of the rock fragments in such matrix remains still high enough and the buoyancy alone is insufficient to keep these fragments in the suspended state. The clayey matrix is viscous, little permeable and is exposed to high interstitial pressure, dependent on its density and piezometric head. Therefore another force is originated in the clay matrix, namely the seepage force j, which is directed along the flow lines, i.e. toward the free surface of the mudflow and is equal to $j = i \rho_m g$, where i is the hydraulic gradient and g is the acceleration due to gravity. Thus the seepage force is added to the buoyancy enabling the fragments to float. Calculations confirm that this mechanism explains the suspended state of rock fragments in the mudflows.
The abovementioned two other contradictions may be explained by this mechanism too.
The corrasion effect of mudflows on their beds is low because of the high neutral pressure in the matrix and hence the small shear strength of mudflow masses. On the other hand if the mudflow bed is permeable and cohesionless, as the boulder beds, the clayey matrix may penetrate into pores and cause the buoyancy enabling the boulders to be involved into the motion.
Easy solidifying of hydrodynamic mudflows with little discharge of water when they are stopped follows from the fact that coarse fragments are composing a rigid framework and they have generally a conservative structure.
The described cohesive mudflows were denoted by the writer as "hydrodynamic" mudflows because the hydrodynamic or seepage pressure ensures the buoyancy of fragments. This type of mudflows differs from the cohesionless mudflows which are abundant in water and where the turbulence is the main source of the rock fragment buoyancy; such mudflows are called the "turbulent" ones.

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