SPECIALTY SESSION 6

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STABILITY OF SLOPES OF DEEP EXCAVATIONS AND NATURAL SLOPES

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Chairman Prof.N.N.Maslov (USSR)

Experience in applying remedial measures against landslides brought to the fore the discrepancies between results obtained by design methods and actual conditions.

At the same time landslides which are sometimes associated with the movement of hundreds of thousands and even scores of millions of cubic meters of soil,(e.g.the Vajont,Italy, the Zeravshan,USSR.etc.)are known to endanger the very existence of human settlements,motorways cultivated lands.Particularly so where dams are concerned which often entail the greatest hazards. Of major importance are also earthslides and cave-ins occurring at deep construction excavations or open-pit mining.

Hence many investigators in Europe,the USA, Japan and in this country focus their efforts on evaluating the effectiveness of design procedures relating to actual earth movements. To name but a few they are Skempton A.and F.Delory,Great Britain,F.Hutchinson,Sweden, Norway and Great Britain,H.Peynircioglu,Turkey,M.Vargas,Brazil,R.Sevaldson,Norway et al.

The infeasibility of applying data from laboratory tests in forecasting slope stability seems to have been conclusively proved by Masami Fucuoka,Japan (1973).A surficial landslide is described (the sliding layer being 1m thick) causing loss of human life every year. When conducting tests on another slope, 15 research workers and correspondents perished.

Doubting the reliability of research findings,the author suggests that in estimating the efficacy of remedial measures against landslides the sliding angle for clays be assessed at 10^{o}. The cohesion value is evaluated from recalculation for landslide sites, with the safety factor F equal to unity. A case is cited when cohesion appeared to range between 0.2-0.4 ton/m^2. This and similar studies carried out with reference to specific stituations reveal a relatively good agreement between calculated and actual field observation data (the stability factor,$F \approx 1.0$) when natural slip surfaces are well defined.On the other hand,when establishing the position of the slip surface by calculations using,for instance,different methods assuming a circular cylindrical slip surface,the design values of the internal friction angle φ and of cohesion,C,being determined from laboratory findings,there was a marked discrepancy between the calculated values of the F factor and the ones for actual earth movements (cf H.Peynirciogly who obtained F=3.60).

In such situations the problem arises of gaining an insight into the mechanism of the phenomenon which often results from development of slide-surfaces in the soil mass of natural slopes mainly formed of clayey soils of a solid or semi-solid consistency,their shearing resistance coefficients being much smaller as against the experimental ones in the first place in respect of cohesion.USSR investigators (as will be shown below) attribute the latter phenomenon to the disturbance of the rigid irreversible component of cohesion (structural cohesion,C_s)commencing already in the pre-sliding period at relatively small magnitudes of the strains occurring in the slope thickness when clays begin to creep.

Scientists outside this country and first of all A.Skempton (1964-1970) treat the problem differently.A.Skempton considers that the initial disturbance (first-time slide) after A.Skempton) of the stability of slopes composed of solid and semi-solid mainly nonfissured clays will be associated with the weakening of the practically maximum initial or "peak" shearing strength of soils.

However,he admits that in some cases the mechanism of the phenomenon may be different. In particular where there are old sliding surfaces in the heavily fissured clay thickness (ancient rock-slides and tectonic planes). Here the cracks are the cause of discontinuity and stress-concentrations throughout the clay thickness. Opening of fissures enhances the effect of such factors as weathering,increase in moisture content etc. reducing the strength of the soil until it attains a fully softened state,with a limit residual strength. Such a progressive failure mechanism in the earth mass is supposed to result in the formation of shear surfaces characterized by minor displacements before the first-time slip occurs.

The process progressing, a continuous slide surface may form over which the soil strength drops to its lowest limiting residual value. Corroborating the experimental findings of Japanese (Saito and Uezava, 1961) and a Jugoslav (Shuklje, 1964) researchers A. Skempton asserts that such a conditions which leads to a vigorous development of the sliding mechanism occurs, as a rule, after the displacement of the earth mass by some inches.

The effect of fissures in undermining slope stability and inducing cave-ins is emphasized in the paper by T. Iamanouchi and H. Murata (1973).

The choice of most efficient techniques for estimating slope stability under conditions of progressive stability deterioration is reviewed by A.W. Bishop (1973). An endevour to bring the design methods closer to actual conditions gave birth to A. Skempton's widely known suggestion to accept in slope stability design the general assumption that cohesion of clay soils equals zero ($\bar{c}=0$). Such a proposal bears a somewhat arbitrary and empirical character and stands in need of theoretical substantiation. The more so, that the exclusion of cohesion ($\bar{c}=0$) from calculations in many cases results in the value of F dropping below unity. E.g. F=0.73 and F=0.69, respectively, for landslides at Lodalen, Oslo and Selset, Great Britain.

Of great value is the comprehensive analysis of experimental data carried out by S. Thomson (1971) on a large first-time slide (Alberta, Canada). The author stressed the discrepancy between laboratory test results and actual landslide situations. There were reasons to assume that the internal friction angle corresponded to the peak (maximum) shearing force. The slope failed at cohesion well below the peak value. Significant is the low value of the stability factor at cohesion, c=0 which testifies to some cohesion in the soil even at long-term residual strength $F_{c\ peak}=1.37$, $F_{c=0}=0,74$. This supports to some extent the Reporter's opinion about the effect on the residual soil strength of coherence, Σ_w, which is a term of cohesion C_w of clayey soils.

In assessing the agreement between calculated and field data it is not clear which calculation procedure was used to assign the design values of the angles of internal friction and cohesion (φ_{cal} and C_{cal}) from the available set of experimental evaluations. Owing to the frequently considerable scatter of experimental points one is liable to choose them in a rather arbitrary manner.

It would seem that the method of calculating pore pressure must rigorously match the actual conditions of hydrostatic suspension and hydrodynamic pressure throughout the soil mass, though it is not always so.

2. With the aim in view of evolving a conclusive solution to the above problems several research engineers, viz. Nguyen Chap. V. Braslavsky and Z. Ragozina working under the Reporter undertook a large reasearch program that in our opinion presents some interest.

Among the considerable number of landslide site descriptions available, we selected 49 cases on which most comprehensive observation and measurement data were reported. These data were treated with the aid of different conventional methods, the stability factor values being determined from formulae strictly adhering to the routine recommended by their authors.

In addition at two landslide sites (at the town of Ul'yanovsk, USSR) extensive field an experimental investigations were carried out.

The data were treated by Z. Ragosina (1971-1973), the total number of calculations performed exceeded 400.

The analysis given herein is based on the assumption that one of the design procedures is optimal, most adequately incorporating all the natural conditions and yielding a safety factor approaching unity /$F \approx 1.0$/. Recognizing that the calculation results are bound to be affected by the technique of determining the mechanical design characteristics from the set of experimental points (scatter), the probable effect of the techniques on the values of the design factors φ_{cal} and C_{cal}, as well as on the magnitude of possible deviations from their arithmetical mean value was analysed. To attain this objective 38 kinds of soils of different composition, state and genesis were analysed with the aid of different procedures recommended by their authors or by Standards.

Analysis of the calculation results revealed that:

a) Calculation of the position of slide surfaces, in particular of circular cylindrical ones (no account being taken of the dynamic action of ground waters furnished for landslide sites a safety factor, F=2-3 and even higher ones, which substantiates a situation frequently encountered.

b) The values of F obtained for landslide sites by different methods proved to approximate each other.

c) From the analysis carried out it appears impossible to explain this discrepancy or decrease it by the application of the most efficient method of evaluation which results in reduced values of the design factors φ_{cal} and C_{cal}. One should bear in mind that for a number of reasons φ_{cal} and C_{cal} are usually chosen with a certain margin of safety and their grouth would only bring about an increase of F. The rich fund of experience accumulated while using the conventional procedures of estimating φ_{cal} and C_{cal} in the design of retaining. Structures, including the major gravity dams built in the USSR at comparatively low prescribed values of the safety factor, $F \approx 1.30$.

In the event φ_{cal} and C_{cal} were several times overestimated as against their actual values which might explain the value of F=2-3 cited earlier, then major failures would have taken place, which was not the case.

d) At the same time calculated values of F for landslide sites with a fixed sliding surface at values of φ_{cal} and C_{cal} assigned from samples obtained in slide surface zones turned out to be sufficiently near unity ($F \approx 1.0$) quod erat demonstrandum. Thus conventional methods for establishing these factors seemed to be recognized as valid.

e) Quite satisfactory in every case ($F \approx 1.0$) proved the data obtained by the approximate method of the equistrong slope F_σ suggested by the Reporter which is extensively used in partical engineering-geology in this country (N.N.Maslov, 1949).

The essence of the method, which procedes from principles that basically differ from those of other calculation techniques, consists in assuming that in the state of equilibrium the angle of the slope $\alpha_{\bar{z}}$ at the depth $\bar{z}$ from the edge of the slope equals the angle of shear $\psi_{\sigma z}$ at this level that is to be found from

$$\psi_{\sigma z} = \mathrm{arctg}\, F_{\sigma z} \qquad (a)$$
$$F_{\sigma z} = \mathrm{tg}\,\varphi_{cal} + \frac{C_{cal}}{\gamma \cdot z} \qquad (b)$$
$$\alpha_{cr} = \min \psi_{\sigma z} \qquad (c)$$

Here $F_{\sigma z}$ is shear resistance coefficient. The close agreement of calculation results by the F_σ method with the data obtained from landslide experience almost in every case is not yet fully explained.

The F_σ method may define the position of the overstressed zone (focus) of initial falure of soil strength in the slope under conditions of progressing deformation and general disturbance of slope stability. A specific feature of the method lies in considering the diminishing effect of cohesion on the soil strength with the increasing height of the slope. This is also altested by the results of 10 summer field observations on the behaviour and failure of cutting and embankment slopes undertaken by Masasuke Watari, 1973.

Based on his studies Masasuke Watari proposes to take the cohesion for 5m and 25m high slopes as 0.05 kg/cm^2 and 0.25 kg/cm^2, respectively. For slopes of different height the value of cohesion is derived by linear interpolation.

4. Examination of calculations performed led to the following conclusions:

1. Various methods for determining the safety factor, yielding nearly the same values for a landslide site testify to the hopelessness of refining these methods without introducing into them fundamental modifications reflecting the nature of the phenomenon under study.

This consideration is valid in reference to the main calculation schemes: a) when assessing the position of the slide surface from calculations, b) in the presence of a natural slide surface. This point was earlier emphasized by Prof. Frölich (1961).

2. A good fit of calculated to actual data on the F values received by the F_σ method based on a different principle emphasizes the significance of approximate calculation methods and justifies their further theoretical refinement.

3. Application of conventional calculation methods, with the slide surface computed requires the introduction of modifications: either improvement of the calculation scheme making it more adequate in describing natural conditions, or an appropriate reduction in time under suitable conditions of the design value of cohesion ($\bar{c}_{cal}$) defined from a set of laboratory data.

4. Most promising in relation to modernizing the calculation procedures (see point 3a) are:

a) Incorporation of the frequently occuring constant "density-moisture content" across the depth of the soil mass, as well as the constancy of true angles of friction and total cohesion corresponding to natural density-moisture content at the level in hand.

b) Making allowance for the hydrostatic and, when suitable, for both the hydrostatic and hydrodynamic (seepage) head due to groundwater, including in the latter case the actual soil mass volume of the slope where the aquifer is located.

c) Account must be taken of the gradual development in time of the slip surface with progressive fissuring of the slope and increase in time of stress concentrations in some zones of the soil mass:

5. Reduction of the design value of cohesion ($\bar{c}_{cal}$) according to item 3b is feasible only to such an extent and in such situations when rigid residual cohesion is disturbed (e.g. cemented clays, solid and semi-solid clayey soils) under conditions of:

a) Accelerated weathering progressing with the depth

b) Prestraining of the slope thickness in particular with the onset of creep in clayey soils[x].

The great significance of rheological phenomena in the development of landsliding mechanisms is almost universally accepted nowadays by scientists both in this country and abroad.

Thus Glazer and Kaczynski (1973) proceeding from field observations on the behaviour of slopes of excavations from 100 to 200m deep come to the conclusion that their stability is a function of time and the rheological properties of soils. Disregard of these characteristics may result in overestimating clay strength by 30 to 40%.

B.Simpser stresses the need of taking into account the time factor in forecasting of landslides, soil strength diminishing with time, or being increased due to thixotropic phenomena. His opinion is based on landslide observations in slopes excavated in montmorillonite clays with a moisture content of about 500% (Mexico-city).

K.F.Lo and C.F.Lee (1973) also attach much importance to the time factor and rheological

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[x] Note. It does not seem to be warranted to completely neglect in computations the total cohesion of clay soils without considering its character (of theory of residual shear resistance), and the present analysis does not bear it out in the absence of deformations in the slope thickness.

soil properties when evaluating slope stability. They outline analytical procedures and forcasting techniques confirmed by field data. Saito and G.Iamada (1973) in much the same way connect the disturbance of slope stability with time and the rheological soil properties. They quote an example of a very accurate prediction of the time of a major landslide (130.000 m^3). Strong emphasis is placed by Sh.Shadunts (1973) on the effect of rheological phenomena in the development of surficial landslides.

6. Thus contemporary calculation procedures for estimating the stability of slopes where sliding processes have not yet occured turn out to be hardly adequate. Hence the necessity is felt of their further refinement.

7. The future development of theory connected with the above problems (see item 6) will take a considerable time.

It seems advisable to make some suggestions, naturally, subject to modifications and checking. The recommendations listed below conform in the main to the trend in the remedial measures against landslides which are extensively used in the USSR.

8. An indispensable condition for a substantiated calculation of slope stability is the suitability of the calculation procedure chosen for the observed or predicted type of landslide in accordance with a classification related to the chosen method.

9. The recommendations proceeding from the USSR experience gained in implementing remedial measures against landslides are based on the following assumptions and correlations (N.N.Maslov, 1941, 1949, 1968a and 1968b):

a) A general formula to describe shear resistance, $\tau_{\sigma w}$ of clayey soils under the load, σ, a soil density-moisture content, w

$$\tau_{\sigma w} = \sigma \, tg\, \varphi_w + \Sigma_w + C_s \qquad (1)$$

or

$$\tau_{\sigma w} = \sigma \, tg\, \varphi_w + \bar{\bar{C}}_w \qquad (2)$$

where

$$\bar{\bar{C}}_w = \Sigma_w + \bar{\bar{C}}_s \qquad (3)$$

Herein φ_w is the true angle of internal friction at density-moisture content W; Σ_w - is the coherence of cohesion under the same conditions of aqueous-colloidal nature of reversible character; $\bar{\bar{c}}_s$ = is rigid irreversible structural cohesion (cohesion due to cementation, paroptesis as well as the molecular character of dense soil consistency); $\bar{\bar{c}}_w$ - is total cohesion of clay.

b. The rheological properties of clays are defined in a general case by the coherence, Σ_w

1. From the above considerations the criterion for the occurrence of creep (N.N.Maslov, 1949) is

$$\tau > \sigma \, tg\, \varphi_w + C_s \qquad (4)$$

simultaneously.

2. In situations when creep is observed and the rigid structural cohesion is disturbed ($\bar{c}$=0) long-term soil strength

$$\tau_\infty = \sigma \, tg\, \varphi_w + \Sigma_w \qquad (5)$$

3. Creep cannot occur when

$$\tau_\infty < \sigma \, tg\, \varphi_w + \bar{\bar{C}}_s \qquad (6)$$

Here τ is tangential (shearing) stress acting at a point in the soil mass over variously oriented areas.

10. The solution of problems pertaining to landslide sites with natural slide surfaces is based on Maslov's trinomial /R.1/. The true friction angle φ_w and coherence Σ_w corresponding to the natural density-moisture content of the soil are incorporated in the calculations. Structural cohesion is assumed to be zero (c_s=0); soil coherence is substituted for cohesion, i.e. $\bar{\bar{c}} = \Sigma_w$

11. When slope stability is assessed by methods requiring the determination of potential sliding surfaces by calculations viz. by any technique using circular cylindrical slide surfaces the design value of cohesion is computed twice:

1. To establish the possibility of creep i.e. of the breaking up of rigid structural bonds in the soil ($\bar{\bar{c}}_s$) at $F < 1.0$, assuming the friction angle $\varphi = \varphi_w$ and cohesion $c = c_s$, i.e. $\Sigma_w = 0$.

2) To ascertain actual slope stability:

a) when no deformation is expected to occur as of item 1, φ_w is taken into account and the total cohesion $c_w = \Sigma + \bar{\bar{c}}_s$

b) when such a deformation does occur, the computations are based on the conditions x/ $\varphi = \varphi_w$; $\bar{\bar{c}} = \Sigma_w$ and $\bar{\bar{c}}_s = 0$.

12. With an aquifer present in the soil mass its dynamic action must be allowed for as well as that of hydraulic uplift or a seepage head in case they are observed.

13. Doubtlessly the suggestions adduced do not fully elucidate all the aspects of the problem. We only strive to contribute to its solution in the light of the experience gained in the application of remedial measures against landslides.

Note x/. Total cohesion is subdivided into the coherence term and structural cohesion term ($C_w = \Sigma_w + C_s$) using one or several conventional methods with the object of verifying the results, the method of repeated shearing among them (Svir'stroi, 1927-1930). Shearing of plates (an artificially cut specimen the correlation $c_w = f(w)$ the crushing strength of disturbed and undisturbed specimen at identical density-moisture content etc. being also applied for the purpose.

Note xx/. Considering the fact that all conventional methods in question yield approximately similar or even identical values of the safety factor, the simplest routines are preferred, particularly those for which tables and calculation curves are available.

Prof.A.W.Bishop (England)- Vice-Chairman

The subject of our session is the stability of deep excavations and natural slopes. As was said by Prof.Lambe, General Reporter to Session I, the job of the engineer is to predict. As Prof.Maslov has indicated in his opening remarks, prediction in the case of slopes, cut or existing is natural strata is a difficult task, associated in same geological strata with major uncertainties.

How should the engineer predict? He certainly cannot do this on the basis of experience alone, or even experience supported by field measurements. He must also understand the physical basis of the phenomena which he is trying to control. Professor Goldstein's remarks in Main Session I are relevant to this point.

This understanding involves three aspects. The first is the geological context in which the problem is set.

The second involves the physical and chemical properties of the soil strata (as far as these control) the engineering behaviour of the soil.

The third aspect is the mechanism of failure or deformation, which it is essential to understand to provide a relevant mathematical analysis.

The most basic physical principal of Mechanics Soil and Rock is the principle of effective stress, which indicates the role of water pressure, both positive and negative, in controlling the strength and deformation of soils and rocks. This perhaps needs emphasis in clay soils in which the absence of free water on the unstressed surface of a trial pit or of a sample from a boring may give the impression, to the geologist in particular, that there is no free water and that hydrodynamic forces can be neglected.

In this connection I would like to illustrate the role of negative pore pressure in controlling the delayed failure of excavated or natural slopes. Fig.1 is taken from

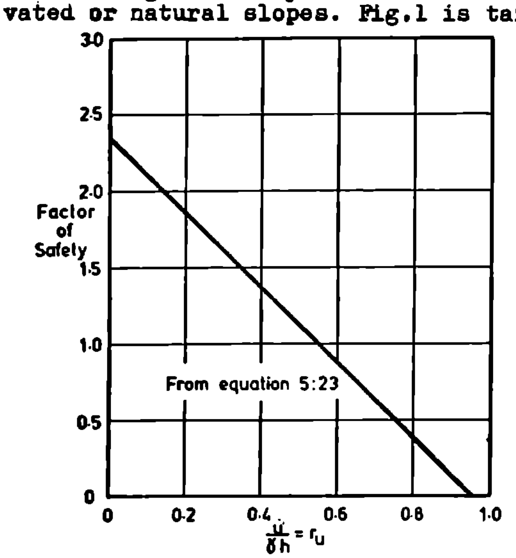

Fig.1. Relationship between factor of safety, F, and pore pressure ratio, r_u: Daer Dam, slip circle no. 4 – after Bishop, (1952).

a paper which I presented jointly with Dr. Bjerum to the Conference at Boulder, USA, in 1960. It illustrates how the pore pressure in the ground decreases and becames negative due to the decrease in total stress on making an excavation, this decrease being particular large in the case of overconsolidated strata. The decrease is a consequence of the fact that parameter the pore pressure B=

for saturated soils to a close approximation. It is one of the few laws in Soil Mechanics on which one can rely. The pore pressure is further modiffied by the change in shear stress according to the semi-emperical expression $\Delta u = B(\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3))$, where A can be expressed either for axially symmetrical or plane conditions strain. Direct confirmation of this prediction has recently been obtain by my colleagues Dr. Vaughan and Miss Walbanke, using special piezometer for measuring negative pore-pressures in situ. This work will appear in a future issue of "Geotechnique" and their main conclusions are Illustrated by Fig.2. You will see the initial pore pressure profile before excavation, the predicted pore pressure immediately after excavation, and its predicted long term profile. You may see that the immediate profile shows negative pressure to depths of 10-15 metres, while the long term profile is largely positive. You will also see that the observed pore pressures after a time of 9 years are still substantially negative, having values of up to about -7 meter of water. These lie close to the curve produced by using the theory of consolidation /or swelling/ with parameters based on tests on large laboratory samples. The clay is mainly the blue London clay.

It is clear from this example that it will be many more years before the negative pore pressure have dissappeared and the equilibrium values established. It follows that many cases of delayed failure which have been attributed to rheological effects or the effects creep discused in Dr. Bjerrum general report can be more simply, and correctly, attributed to changes in pore pressure after excavation. This effect may also be present where successive slides occur in nature consequent a toe erosion by a river or by the sea.

I wish not to turn to the meaning of the factor of safety usually denoted by the symbol F. I have discussed the safety of progressive failure elsewhere and will simply refer to the role of pressure pore water.

A typical relationship between calculated factor of safety and pore water pressure, expressed as a ratio of the weight of soil above is given in Fig.3. This assumes that a state of limiting equilibrium would exist when the mobilized shear stress is give by the expression.

$$\tau_m = \frac{c'}{F} + \frac{tg\,\phi'}{F}(\sigma - u)$$

where c' and ϕ' denote cohesion and angle of shearing resistance in terms of effective stress, and σ denotes total normal stress and u denotes pore water pressure. Now c' and ϕ' can generally be estimated without great difficulties, with due allowance for

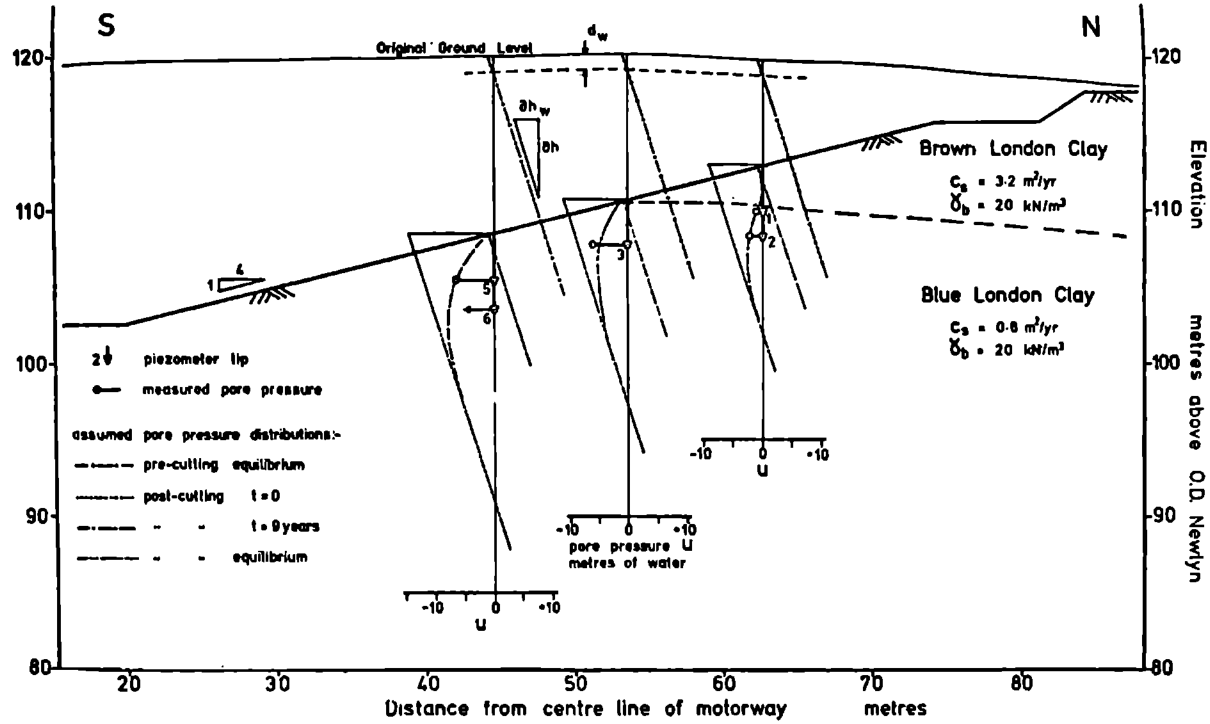

Fig. 2 Edgwarebury Cutting Section showing Pore Pressures

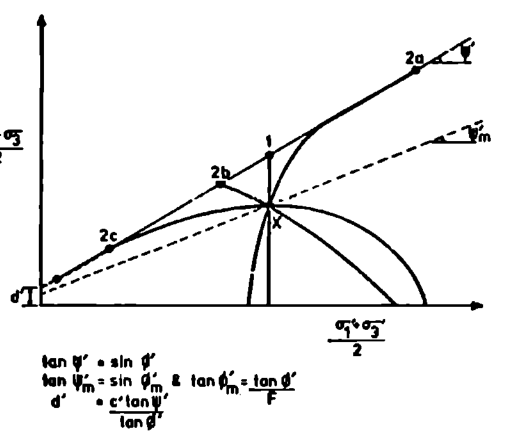

Fig.3. Range of idealized stress paths for soils of low plasticity tested under undrained conditions.

stress system, anisotropy and time effects. However the value of u raises an interesting philosophical problem, which is often overlooked. Is the pore pressure an independent variable or is it influenced by its magnitude of τ the shear stress?

Three cases may be distinguished:

I. The case in which pore pressure is an Independent variable, for example steady seepage or rapid drawdown in relatively permeable soils, where the poro pressure is derived from a flow pattern determined by the geometry of the boundaries, the relative permeabilities of strata and when relevant, the drawdown rate.

2. The case in which pore pressure is directly influenced by the magnitude of the shear stress τ because undrained conditions effectively exist, and the pore pressure changes in a way which can be observed in an appropriately planned laboratory test, subject to the limitation of sample disturbance. This results from low coefficient of consolidation or long drainago path.

3. The case in which the pore pressure is controlled both by τ and by the elapsed time since excavation as in my earlier example. Here an undrained analysis would give only one bound to the factor of safety, and this would be an overestimate.

Clearly in case I and case 3 and effective stress analysis is the only logical way of solving the problem. In case 2 either a total stress analysis or an effective stress analysis may be used, but case 2 represents only the short term factor of safety, and this value will always decrease, both due to pore pressure changes a shown by Fig.1 and due to rheological effects.

However, even when the effecive stress analys's is appropriate, the value of F must be qualified for the reasons illustrated in Fig.4. Here are shown potential stress paths along which one might proceed in various soils if a small increment of shear stress were applied to a slope having an apparent factor of safety F of about 1.5. For a soil in which pore pressure is an independent variable we proceed up stress path XI without any change in pore pressure to meet the failure envelope. The reserve shear stress is unambiguously defined.

However for boulder clay or till, which reduction produces a pore pressure on shear,

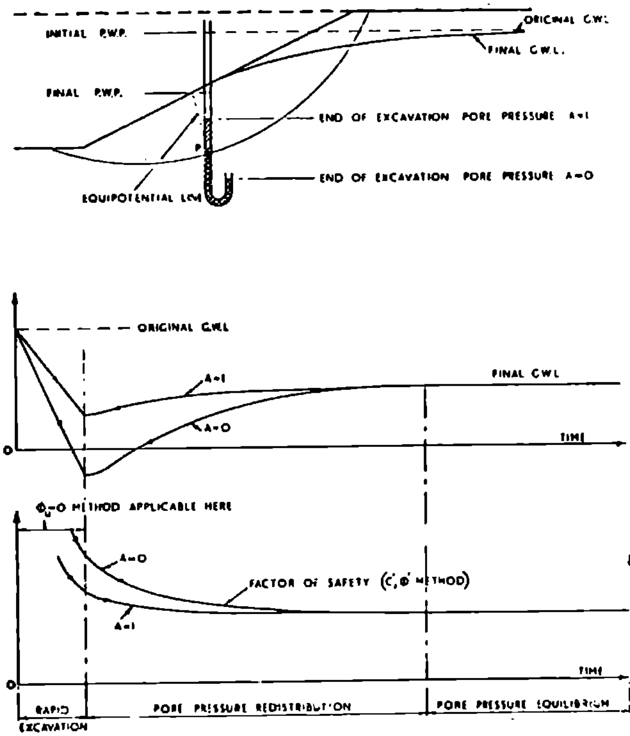

Fig.4 The changes in pore pressure and factor of safety during and after the excavation of a cut in clay.

an undrained increment would follow path X 2-a, and lead to strength greater by a factor of 1.5 or 2 than the apparent strength based on the existing observed pore pressure. This greater strength would only exist as a reliable component of our safety factor if the pore pressure did not rise again to the initial value, which in the case of long term stability would be controlled by the boundary geometry and drainage.

In contrast, for a very sensitive clay or quick clay undrained stress path would be as X2c, and the reserve strength would be less than the apparent reserve strength, and indeed might be negligible. A small disturbance might lead to complete rapture and the fact that the fully drained strength would have been greater would be no longer of relevance.

A less sensitive clay would be as illustrated by X2b, with some reserve strength, but smaller than the apparent value indicated by the existing pore pressure and the effective stress analysis.

This is but one illustration of the necessity of understanding the factors controlling shear strength if one is to avoid drawing incorrect inferences from field observations even when pore pressure have been measured.

Chairman Prof. N.N.Maslov (USSR)
Thank you Prof. Bishop for your report.

Our discussion will be continued by the report of Prof. Ter-Stepanyan.
Prof.Ter-Stepanyan, will you please.

Prof.G.I.Ter-Stepanyan (USSR)

Our experience proves that the determination of the landslide mechanism is difficult and sometimes impossible if geological and hydrogeological data are used only. The difficulties increase considerably in cases of big, old and complex landslides. In such cases important features of landslide dynamics should be taken into account. Analysis of creep rate hodographs was used successfully in such intricated cases. The method was described in a paper presented to the Mexico conference. Some examples will be shown participants of the Sochi tour of the present conference.

A very instructive case was met in Balchik, Boulgaria. Big and old landslides are developing in this area during last milleniums. Joint investigations made by the Boulgarian and Armenian Academies of Sciences have shown the complex mechanism of this sliding. It is a threestoried landslide. The first story are giant blocks of marles and carboniferous clays with horizontal layers which break away from plateau and creep slowly over clay surface toward the sea. The rate of displacement increases as the creep proceeds. The creep rate of the most advanced block reaches 0.6 meters per year. The second story are massives of rocks with inclined layers formed in spaces between blocks by their filling with rocks, creeping from the block slopes. The third story are subsurface landslides being developed in the loose products of weathering on massives located between blocks. It was evident that control of first and second story sliding is impossible. Prognosis of sliding and recommendations for construction were given.

It is safe to say that there are numerous multistoried landslides which represent insuperable obstacles when studied by usual geological methods; they may be successfully analysed using this procedure.

Creep hodographs may be used for determination of the deformed state of the landslide body. A semigraphic method of strain calculation was worked out based on analysis of creep rate hodographs. Direct measurements may be performed for this purpose too. This approach to the slope dynamics study may be used for better understanding of events which take place during the depth creep phase on slopes and for layout and efficiency analysis of the landslide control.

Analysis of slope dynamics open new and important possibilities for investigation of landslide mechanism. Necessary conditions for it are systematic geodetical observations of slope dynamics, made during the phase of depth creep of slopes, when the landslide control is the most effective.

Chairman Prof. N.N.Maslov. (USSR)
Thank you Prof.Ter-Stepanyan for your discussion. The next will be Mr. Feda.

Mr.Feda will you please.

Mr.Feda J. (Czechoslovakia)

On rock slopes of high mountains of Slovakia similar slope movements were observed like those described by Zischinsky (1966). Fig.1 (Nemčok,1972) presents two typical

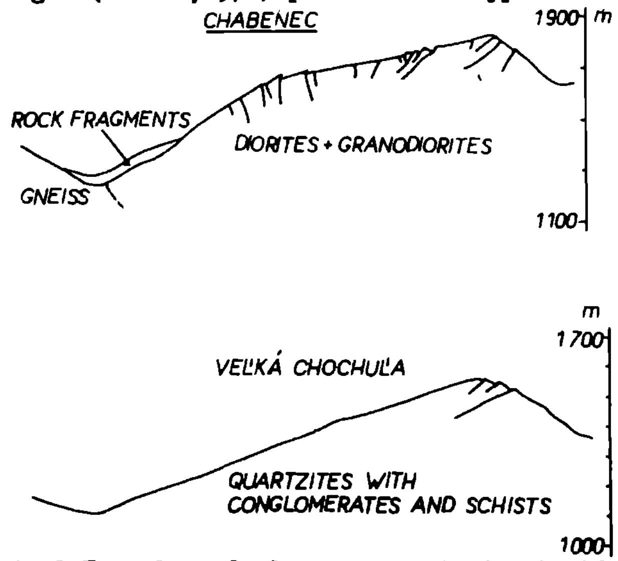

Fig.1.Examples of slope movements in the high mountains of Slovakia (Nemčok,1972)

examples showing marked regions of discontinuous deformations (tension cracks and shear displacement surfaces).

To explain these deformations and thus to enable their prognosis call for an appropriate theoretical idea about the mechanism of slope movements. A sufficiently general theory should be valid both for rock slopes of high mountains and for slopes of soil cuts which are genetically related (erosion valleys and cuts both cause a substantial unloading of the slope toe and corresponding stress redistributions). This assumption being valid one may use the experiences with soil slopes of maximum height difference of some tens of meters for natural rock slopes ten times higher.

According to Fig.1 slopes locally failed although the total stability was not surpassed. The problem faced is therefore one of progressive slope failure. Fig.2 presents a

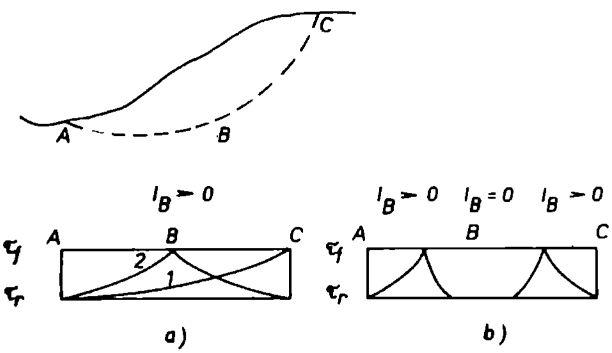

Fig.2.Shear stress distribution at progressive failure:a-for brittle behaviour (Bishop,1971),b-for brittle-ductile behaviour

natural slope with a potential rupture surface. Decisive for the failure progression will be the mechanical behaviour of the rock (soil) mass. This can be either brittle (dilatant) or ductile (contractant). Using Bishop's (1967) index of brittleness.

$$I_B = \frac{\tau_f - \tau_r}{\tau_f} \qquad (1)$$

(τ_f, τ_r - peak and residual strengths) for brittle range $I_B > 0$, for ductile $I_B=0$. The value of I_B depends on the rock (soil) nature, stress level (plus temperature,water content etc. effects) and on the loading path.

For triaxial tests the transition from brittle to ductile behaviour corresponds to a certain transition cell pressure σ_r^T (ductile behaviour for $\sigma_r > \sigma_r^T$). The value of σ_r^T used to be e.g. 0.7 (St.Vallier clay-Lo, 1972); 2 to 3 (Neogene clays of Western Bohemia); 5 to 15 (Ordovician shales of the Prague bedrock according to their degree of weathering); 125 (residual granitic sand, Southern Bohemia-Feda,1973); 800 to 4000 (different rock types-Beyerlee, 1968)- all values in kg/cm^2.

Ideally ductile behaves also a rock (soil) mass displacing along a preexisting sliding surface but in this case there is no accompanying contractancy. Finally it is important to realize that for plane strain condition I_B is greater than for axially symmetrical stress state.

Based on theoretical considerations and field measurements of clay cuts (Bishop,1971) one may assume first to originate tension cracks at the slope crest C (Fig.2),then after sliding surfaces at the slope toe A,later on (or simultaneously) sliding surfaces near the slope crest C and the failure spreads toward the slope center B.

For brittle behaviour progressive failure has a dilatant form and the shear stress distribution on the progressing failure surface may be presented by Fig.2a (Bishop,1972):1-if failure spreads from A to C, 2-if failure spreads from A and C to B.

At the slope toe A and crest C the value of the confining pressure being small one may assume brittle behaviour. The mass will dilate,its volume increases and sliding surfaces (i.e. relatively thin failure zones) will originate. In the middle B of the slope the stress level could be so high as to induce a ductile behaviour. Then $I_B = 0 (\tau_f = \tau_r)$, the volume of the mass decreases (contractancy) and a broader shear zone develops. Its nature may resemble the marble samples of von Karman (1911) tests in the ductile range: the rock mass will be crushed into relatively small fragments so to be like a granular material (von Karman's samples at high pressures consisted after the test of firmly compacted powder. Assuming $I_B > 0$ at A and C and $I_B=0$ around B the stress distribution follows Fig.2b. This is quite similar to the proposal of Gol'dštein and Babickaja (1964).

If the nature of the stress distribution diagram can be assumed in advance the slope safety factor may be computed according to classical methods. A serious problem which I will not deal with is to determine the strength of rock (soil) in situ.

Taking into account a progressive failure after Fig.2b the mechanism of the slope failure describes Fig.3. After contractant fai-

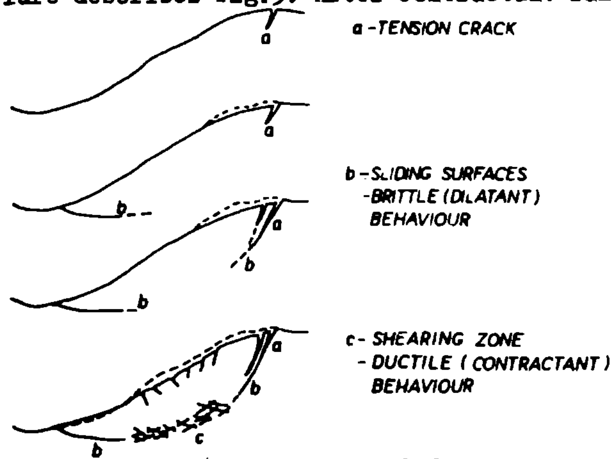

Fig.3. The progress of slope deformations assuming brittle (in a and b regions) behaviour of rock (soil) mass

lure zone c being formed the volume decrease of the middle part of the slope takes place which initiates secondary sliding surfaces in the slope body (near its surface the behaviour is again brittle). Final slope deformation corresponds quite well to the actual one of Chabenec on Fig.1.

If there are some preexisting sliding surfaces in the slope body then no contractancy takes place and the deforming slope will probably reveal no secondary sliding surfaces. Consequently the last stage of slope deformation on Fig.3 will not occur.

Natural slope behaviour may therefore be explained even under a simplified assumption of homogeneity and isotropy of rock mass. In reality this is scarcely fulfilled on a small scale since in a rock mass there exists as a rule a system of joints. In fact a volume decrease of a rock mass is possible only owing to its porosity, jointing, fissuration etc. The above assumption of isotropy and homogeneity is therefore acceptable only on a big scale, i.e. it has a statistical meaning only.

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Chairman Prof. N.N.Maslov (USSR)

Than you very much Mr.Feda for your interesting report. I invite now Dr. Madej from Poland for reporting.

Mr.Madej, will you please.

Dr.Jerzy Madej /Poland/

One of the ways leading to the optimum method of slope stability analysis is the accurate solution of the method of slices. In order to obtain such solution, it is necessary to make an additional ssumption, which allows the determination of the distribution of the internal slice forces. Accurate solutions were proposed by Kenney (1956), Janbu (1957) and Morgenstern and Price (1965). All these solutions however contain some simplifications and the result is that the condition of moment equilibrium in the individual slices is not fulfiled. Because of that, an accurate solution of the slice method has been developed and it is presented below.

In the proposed solution all the assumptions of the method of slices are used and it is interesting for us to determine the internal forces X and E. The additional assumption in this solution is the shape of the function yx of distribution of the X forces, as shown in Fig.1. This function enables the determination of ordinates of the function ydx:

$$ydx_i = yx_{i+1} - yx_i, \qquad (1)$$

which describes the ΔX force distribution. The sum of the ordinates ydx_i must be equal to zero and thus the equilibrium of the X forces is satisfied. The functions yx and ydx are dimensionless. In order to obtain the values of the force ΔX_i in every slice, it is necessary to multiply the ordinates of the function ydx by the multiplier Mp, as below:

$$\Delta X_i = ydx_i \, Mp. \qquad (2)$$

The Mp value is constant for all slices and it has the dimension of force. This coefficient is one of the unknowns which must be determined. The second unknown is the stability factor of the slope, F. This factor is calculated from the condition of force equilibrium

$$\sum\left[(W-\Delta X)\tan\alpha - \frac{(W-\Delta X-u\,b)\tan\Phi'+c'b}{F\,m_\alpha\,\cos\alpha}\right]=0, \qquad (3)$$

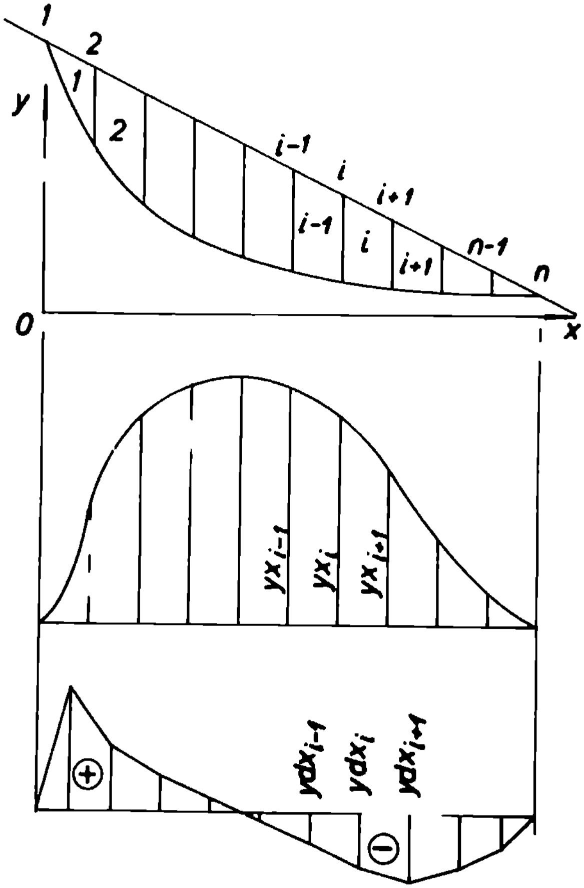

Fig . 1

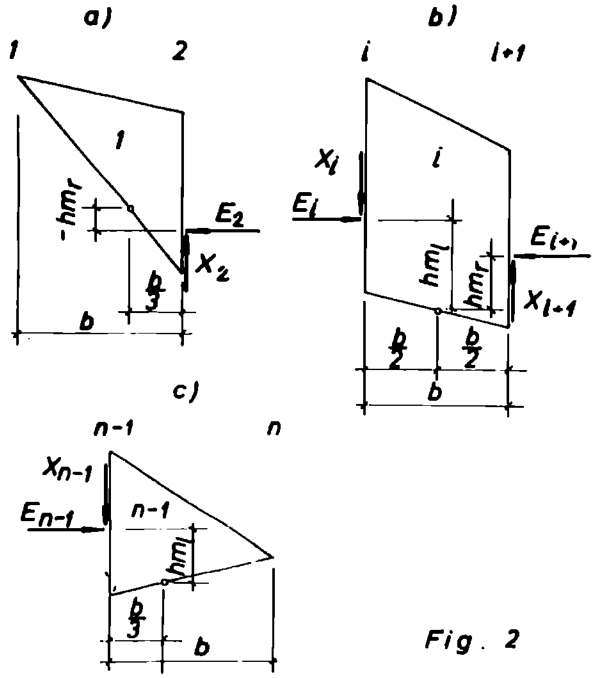

Fig . 2

which was developed by Bishop (1955).

The solution is obtained by the method of successive approximations, beginning from an arbitrary value of the coefficient Mp. The ΔX_i forces, corresponding to this coefficient, are substituted into the equation (3) and a value of the F is chosen such that this equation is satisfied. While determining the values of the X and E forces on the boundaries between the slices, the moment equilibrium in the last slice of a potential slide is proved, as follows.

The sum of moments in the first slice (see Fig.2a) is described by means of the equation

$$M_1 = - X_2 \frac{b}{3} + E_2(-hm_r), \tag{4}$$

from which a location of the thrust line on the right side of this slice is obtained, namely:

$$hm_r = - \frac{1}{3} \frac{X_2 b}{E_2} \tag{5}$$

In the remaining slices (see Fig.2b) the moments are calculated according to the equation

$$M_i = E_i hm_l - E_{i+1} hm_r - \frac{b}{2} X_i + X_{i+1} \tag{6}$$

were four internal forces must be taken into account. Using the formula

$$hm_r = \frac{E_i hm_l - \frac{b}{2}(X_i + X_{i+1})}{E_{i+1}}, \tag{7}$$

the location of the thrust line in the whole sliding mass is determined. The sum of moments in the last slice (see Fig.2c), calculated by the formula

$$M_{n-1} = E_{n-1} hm_l - X_{n-1} \frac{b}{3} \tag{8}$$

should be equal to zero. This condition is reached by means of the changes of the Mp coefficient, and every change requires a complete iteration.

Experience shows that the solution of this problem for one function yx, using the computer, requires making no more than 20 iterations. The proposed solution does not yield a unique result. It is necessary to prove the correctness of the location of the thrust line and the stress state on the boundaries between slices for several functions of the distribution of the X forces.

The comparison of the mentioned propositions of the accurate solution of the slice method shows good qualitative and quantitative agreement of results, as it is seen in Fig.3. The distribution of the internal forces X and the location of the thrust line has the same character, and the relative height of the thrust line, $\frac{t}{h}$, reaches the reasonable values within the range of 0.25–0.50. The values of the F_v coefficient on the boundaries between slices are similar as well. It is rather surprising that the relative differences between the values of

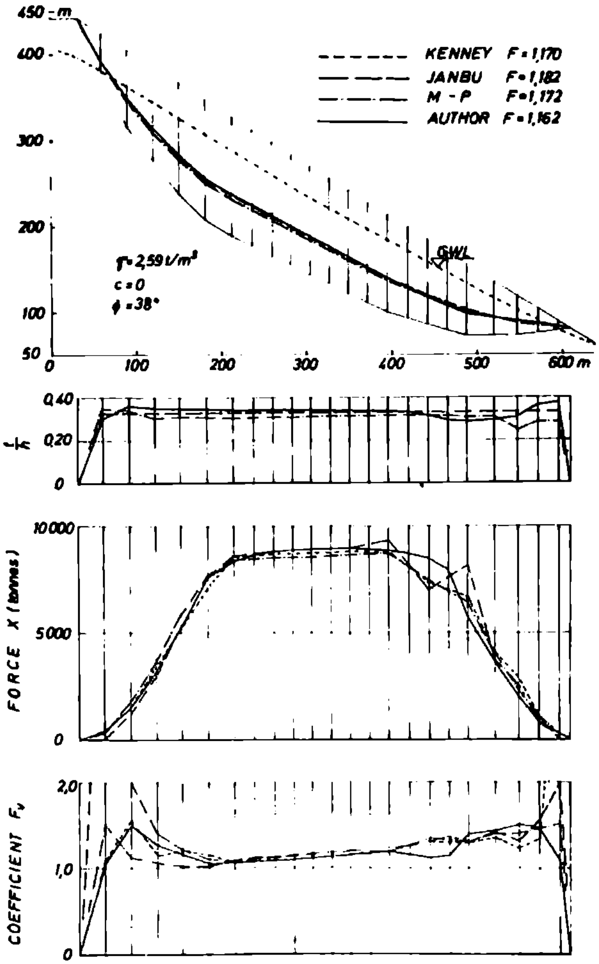

Fig. 3

the stability factor F are 1.0 to 2.0 per cent, not only in the case presented in Fig.3.

These results are to say the compared methods are essentially identical. It should be noted however, that Janbu s proposition can be used in the case of a regular shape of the slip surface and division into slices of more or less the same width, and Kenney's proposition causes difficulties when computers are used. The proposition presented here of the accurate solution of the method of slices does not warrant obtaining a unique solution, similar to Morgenstern-Price's method, but in contrast to the mentioned methods it gives the solution which fulfils rigorously the conditions of the static equilibrium of every slice in the sliding mass.

Chairman Prof.N.N.Maslov (USSR)

Thank you very much Mr.Madej. The next will be Mr. Perlea (Romania).

Mr. Perlea, will you please.

Mr. Vlad D.Perlea (Romania)

It is often necessary to estimate the stability of a slope although the strength characteristics (in particular Ø and c) are not known with the desired accuracy. A typical example of such a case are fly ashes from power plants in hydraulic fills, that are characterized by a high degree of heterogeneity and unisotropy due to their deposit conditions. The stability of such formations will be dealt with in the following.

A corresponding method for analysis in such cases is the resistance envelope procedure (Casagrande, 1950), whose defining is consistent with the old expression (after Fellenius) of the safety factor, derived from the moment equilibrium for circular sliding surfaces:

$$F = \frac{c.L + tg\,\emptyset \sum_{i=1}^{n} \sigma'_{ni}\ \Delta l_i}{\sum_{i=1}^{n} W_i \sin \alpha_i}$$

which, using notations $\bar{\sigma}'_n = \frac{\Sigma \sigma'_{ni}\ \Delta l_i}{L}$

(the average effective normal stress on slip surface) and

$\bar{\tau} = \frac{\Sigma W_i \sin \alpha_i}{L}$ (the average shear stress on slip surface), becomes:

$$F = \frac{c + tg\,\emptyset\,\bar{\sigma}'_n}{\bar{\tau}} \quad \text{or} \quad \bar{\tau} = \sigma'_n \frac{tg\,\emptyset}{F} + \frac{c}{F}$$

The latter expression for the safety factor is accepted also by methods which assume other distributions of the normal stress, than the Swedish slice method, or other shapes of failure surfaces than circular.

The foregoing relationship provides for a mean to find again the safety factor in a graphical way, using a $\bar{\tau} - \sigma'_n$ diagram where the ordinates of the Mohr envelope can be compared to those of some points, that by their corresponding $\bar{\tau}$ and σ'_n values characterize different potential failure surface (fig. 1b). For the fill given in fig.1a the envelope of these points has been drawn in fig.1b.

Owing to the fill heterogeneity and to the impossibility of obtaining undisturbed representative samples, the strength parameters mentioned in fig. 1b (viz. the computed safety factor too) could not be considered as reliable. Consequently, in view of the further elevation of the hydraulic fill by another 25m it was deemed necessary to separate the part to be completed by future sediments from the existent fill, with the help of a horizontal impervious blanket covered with a drainage system, and to drain the toe of the existent fill. By this structural modification, the flow net is radically changed (fig.2a) (Zaharescu and Perlea, 1972).

To justify the suggested solution under the conditions of unknown shear strength parameters, an approach has been done to express

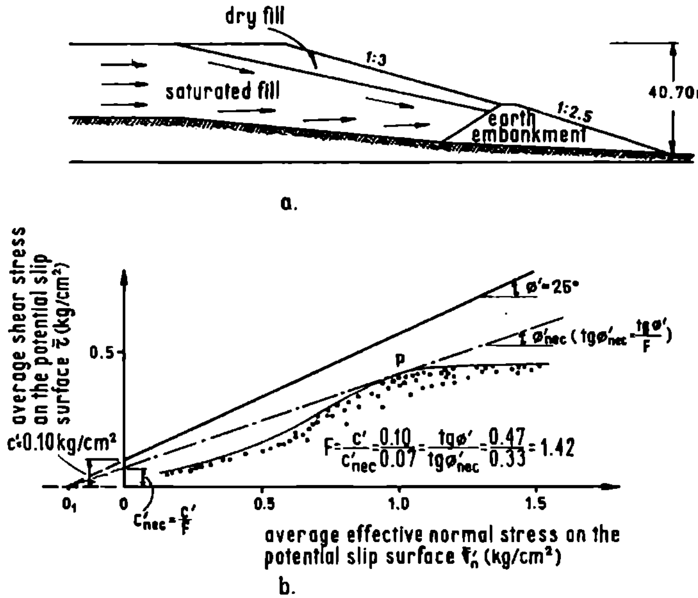

Fig.1. The existent fill. a. Cross section. b.Safety factor determination using the envelope of the points representing average shear stress ($\bar{\tau}$) as a function of the average effective normal stress ($\bar{\sigma}'_n$) on the potential slip surface.

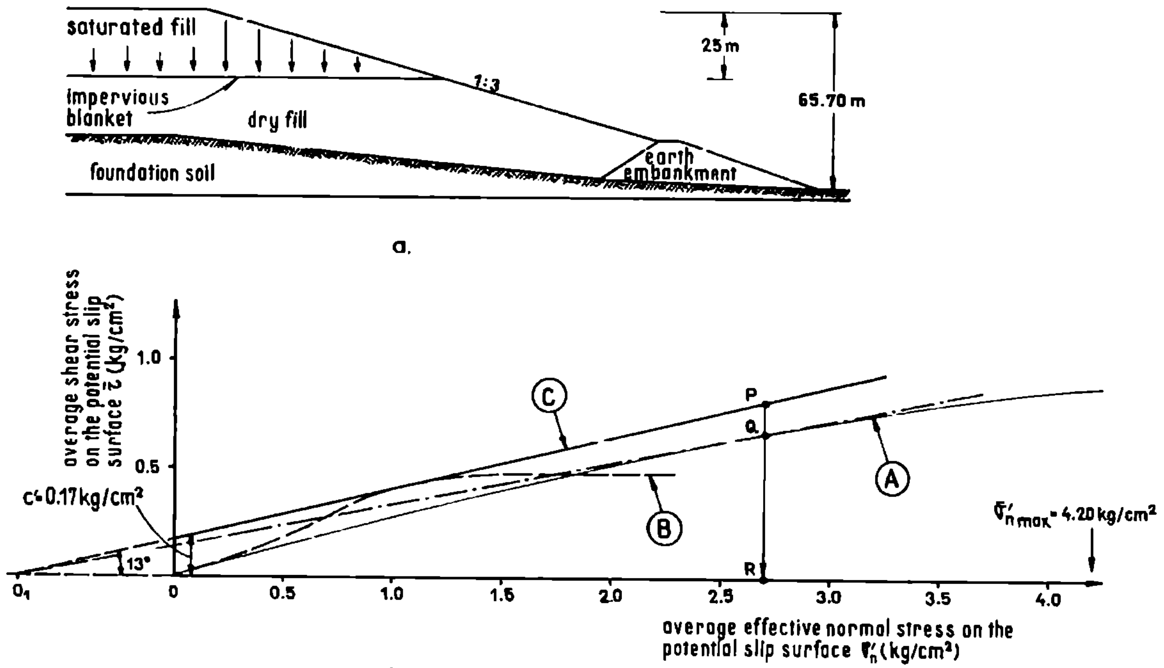

Fig.2. The superelevated fill. a.Cross section corresponding to the suggested solution. b.Safety factor determination using the resistance envelope for the superelevated fill (A) and considering that the Mohr envelope (C) corresponds to a limit equilibrium for the existent fill,namely it is tangent to the envelope (B) drawn in fig.1b

quantitatively the superlevated fill stability, considering the existent fill at limit equilibrium (the safety factor equal to 1.0). Therefore, in fig.2b the resistance envelopes for both the existent fill and the superelevated one were drawn. One can see that even for a small value of the internal friction angel (Ø=13°)the fill stability is improved.

In fig.3a comprehensive comparative analysis is done; it is given that for Ø 20°,very likely a value,the stability is improved by more than 40%, and critical slip surfaces are shallow (corresponding to small σ'_n values) remaining above the impervious blanket.

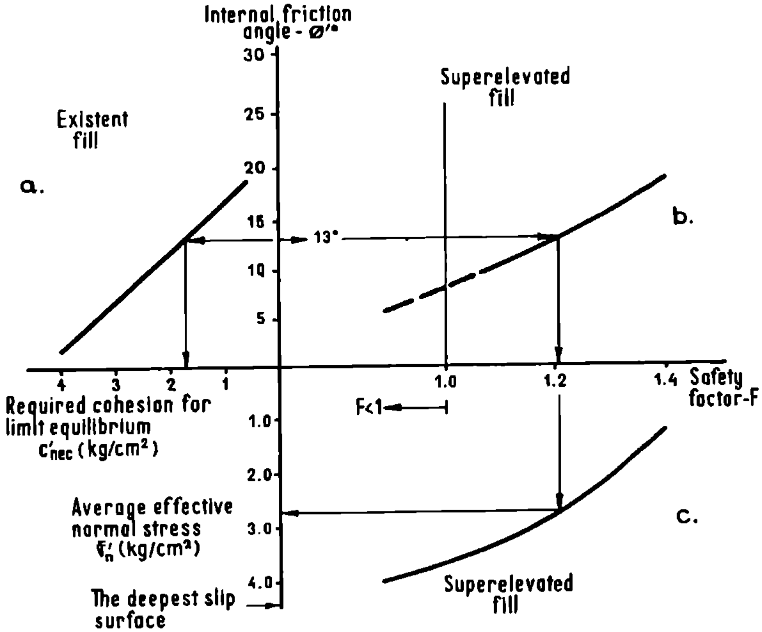

Fig.3. Stability conditions for the superelevated fill. a.The required cohesion (c_{nec}) for the existent fill limit equilibrium,depending on the internal friction angle (Ø) . b.Safety factor (F) for the superlevated fill depending on Ø and c corresponding to the limit equilibrium in the existent fill. c.Average effective normal stress on critical slip surfaces in the superelevated fill (σ'_n) corresponding to several possible situations

The resistance envelope is useful also for stability charts drawing,employing dimensionless parameters (Janbu,1968).It permits the comparison of different analysis methods too, the accuracy of which relies on a significant criterion i.e.the approximation on normal stress distribution along the slip circle (as well as σ'_n value).Finite element method may be very useful for accurate diagrams drawing, that would permit a quick stability estimate in different conditions,using dimensionless parameters.

Kenney (1967) used Bishop's method of slices (Bishop,1954) for plotting the resistance envelope.However,any method in which normal stress depends on shear strength parameters cannot lead to an unique envelope which might be used to the above specified purpose. Therefore, it is recommended to draw a diagram similar to that presented in fig.4; by superposition of curves obtained using two different methods (Fellenius' and Bishop's), the results provided by these methods can be compared subsequently.

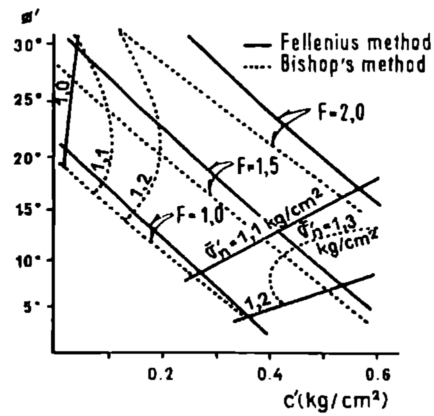

Fig.4.Curves of equal safety factor and equal average effective normal stress for the existent fill obtained by two methods: Fellenius and Bishop

Chairman N.N.Maslov (USSR)

Thank you Mr. Perlea for your report. Now I want to pass the word to Mr.Slunga Mr. Slunga will you please.

Mr.Slunga E. (Finland)

Saimaa canal connecting the Finnish Lake of Saimaa to the Gulf of Finland at the town of Viipuri in the Russian territory was built during the years 1963...1968. The length of the canal is about 45 km, the depth from the water level 5,3 m and the bottom width in general 28m (Fig.1).

using the circular cylindrical slide surface. The consideration is made by concentrating the values of the shear strength and the unit weight measured from various soil layers at a homogeneous part of the slope into one cross section and calculating the standard deviations for active and passive moments based on the deviations of these measured quantities. The anisotropy of the shear strength is taken into account in the calculations. The measured values of the shear strength and the unit weight have been observed to follow the normal distribution.

The statistical factor of safety $\bar{F}_p$/2/ has been calculated from the equation

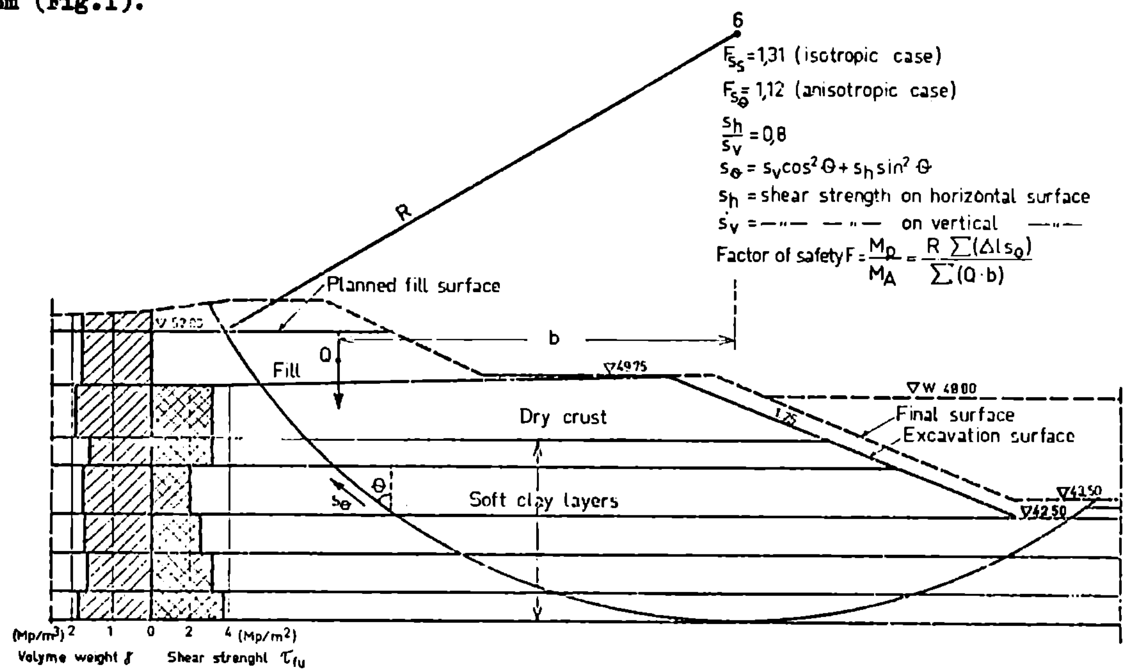

Fig 1. A cross section of the clay slope at Saimaa canal

Characteristic for the canal area are wide layers of sediments mainly of clay and silt. They are slightly overconsolidated and anisotropic. The ratio between the shear strength on the horizontal surface and that on the vertical surface is about 0,8.

When planning the canal the stability calculations were performed mainly by help of the Ø=0 method and most of the calculations were based on field vane (H/D=2) tests. The factor of safety for the short-term failures and the theoretical cross-section was 1,30. If the anisotropy is taken into account according to Lo /1/, the factor of safety decreases by 14% or to a value of 1,12. Soon after the excavation work was completed and before the canal was filled with water, three greater failures occurred. The total length of them was 550 m.

After the occurrence of the slides the stability of the slopes has been examined statistically on the ground of the Ø=0 method and

$$\bar{F}_p = 1 + t_p \frac{\sqrt{\sigma^2 M_p + \sigma^2 M_A}}{\bar{M}_A}$$

where t_p= coefficient corresponding to the risk of failure p% (from tables)

σM_p= standard deviation of the passive moment

σM_A= standard deviation of the active moment

$\bar{M}_A$= average value of the active moment

The risks of failure corresponding to the factors of safety 1,3, 1,4 and 1,5 based on ordinary field vane tests (H/D=2) are 15%, 4,5% and 1% resp., according to the calculations (Fig.2). During the construction work 550m or 14% of the slopes failed along a part of the canal, where the total length of weak slopes was 3850m. Some of the weak slopes were supported before the excavation. If one half of the slopes supported

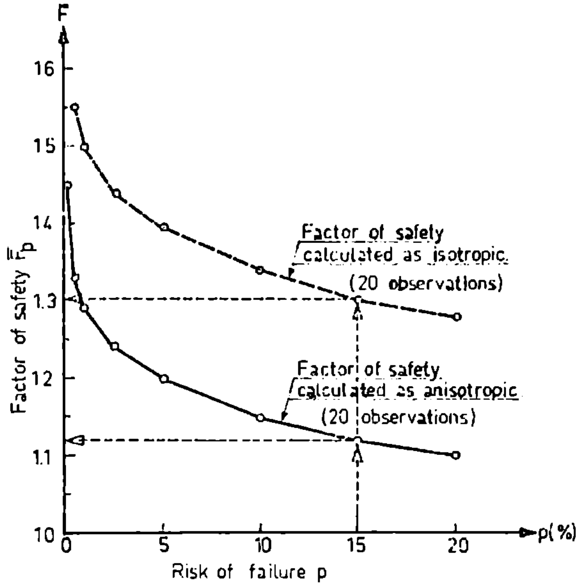

Fig. 2. An example of the relationship between the risk of failure and the statistical factor of safety in anisotropic soil based on vane tests (H/D=2.) Anisotrophy is taken into account in calculating the risk of failure.

is assumed to fail, the total length of failing slopes would increase to the value of 19%.

Figure 3 shows results of calculations concerning the economic optimum value of the

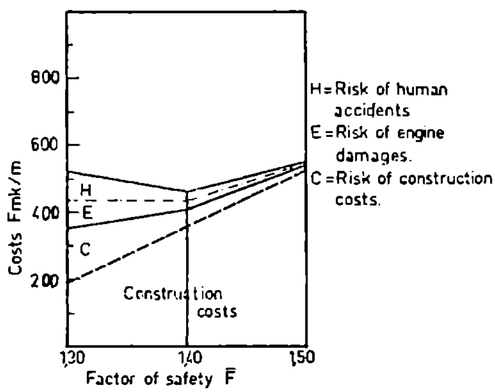

Fig 3 Economic optimum value of the factor of safety.

factor of safety in relation to the foregoing values of the risk of failure. The cost values used in the calculations, excluding the costs of human accidents, are actual costs of the failures at the Saimaa canal. In spite of the great risk of failure (15%), a factor of safety 1,3 is economically most favorable, if only the construction costs and a portion of repair costs of failures determined by the risk of failure are taken into account. If the extra costs of possible machine damages and human accidents also are noticed, the optimum factor of safety increasases. There were no human accidents in the failures at the Saimaa canal. Therefore the extra costs of human accidents have been estimated in the way employed when calculating the economic optimum value for the investments in the planning of higways. It is assumed that one person will die in each failure.

In the foregoing statistical consideration of the slope stability the variables include the undrained shear strength and the unit weight of the soil. There are, however, many other variables influencing the stability of slopes, as e.g. the variations of loads, the time-dependence of the shear strength and the accuracy of the methods of calculation. When developing the consideration the influence of these other factors should be investigated.

Chairman Prof. N.N.Maslov (USSR)

Thank you very much Mr.Slunga for your contribution.
Mr. Yamanouchi, will you please.

Mr. Yamanouchi T. (Japan)

This speaker, Yamanouchi, and Murata[1] have submitted a paper on the brittle failure of the undisturbed samples of a volcanic ash soil "Shirasu" to the Main Session I. I would like to add a new consideration to the paper, in reference to the stability of steep cut-off slope of the soil, referring the result of an analysis of the two-dimensional isotropic slopes by means of finite element method by Kawamoto[2]. Its computation was conducted with the element division and the boundary conditions as shown in Fig.1. The practical cut-off slope of the soil is approximately 80 degrees for preventing the slope surface from an erosion by stream.

In the slopes steeper than 60 degrees of three kinds of slopes, tensile stress zone appears as seen in Fig.2. When the maximum tensile stress exceeds the tensile strength of "Shirasu" material, tensile failure arises in the slope nearly in parallel with the slope surface. Besides this tensile failure, shearing failure is brought about at the part near the toe resulting from the maximum shearing stress. Such a combinative failure may be a cause of block-like failure especially at the part neat the toe, and this consideration seems to acoount for the actual failure as observed at the field.

A different hypothetical cause was suggested by us in that paper for explaining the brittle failure of the slope. I hereafter wish to investigate quantitatively the problem by comparing the stresses obtained from the elastic analysis as above referred with the strengths of the undisturbed samples as reported in that paper, aiming at finding the major cause of the failure for the purpose of establishing the countermeasure against the failure.

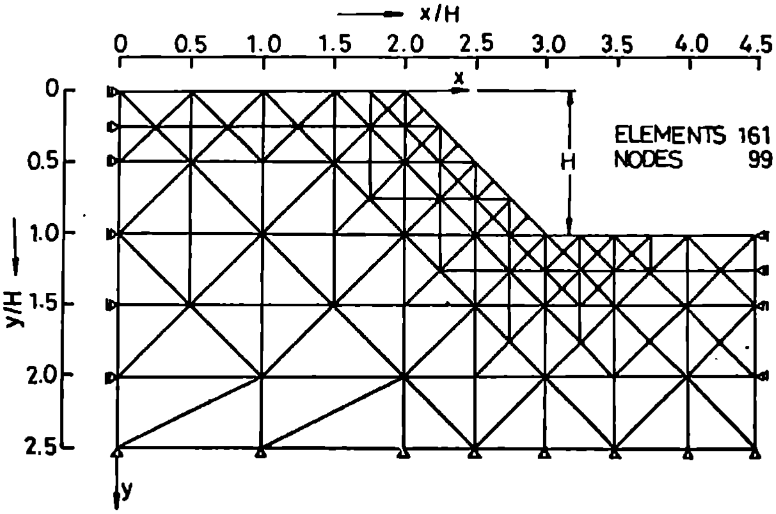

Fig.1

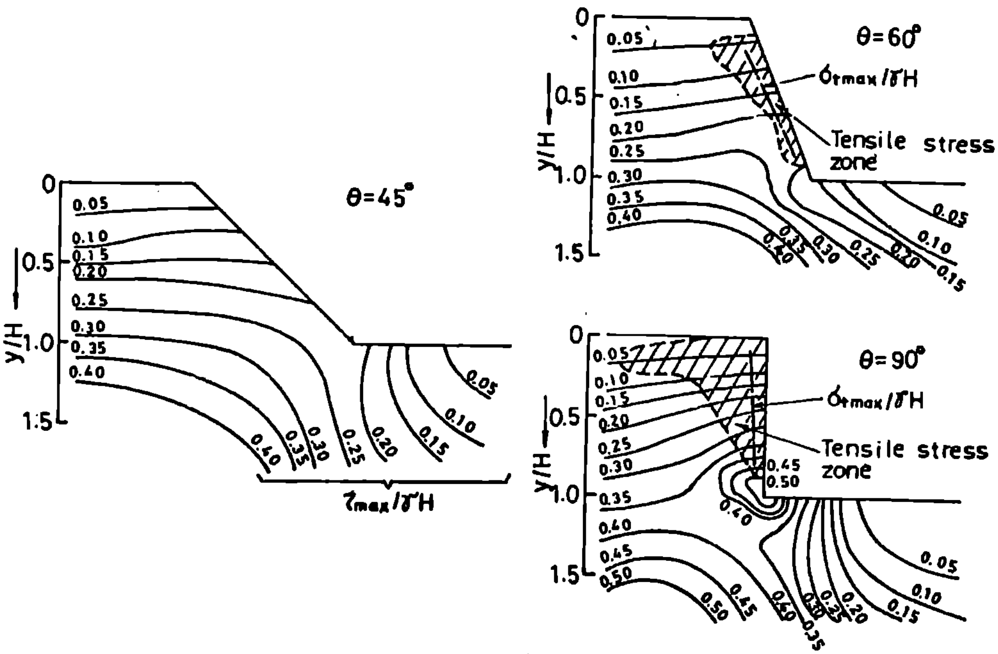

Fig.2

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Chairman Prof. N.N.Maslov (USSR)

Thank you very much Mr. Yamanouchi for your report.
The next will be Miss Turovskaya from the USSR. Miss. Turovskaya will you please.

Miss. A.Turovskaya (USSR)

1. The deformation with the invariable velocity is typically for the creep establish stage. This stage takes place in the pre-limiting state during which the gradual weakness grounds are forming.

These preliminary deformations in the pre-limiting state are ordinary almost for any soil failure. But there are only various time durations to reach the critical state.

The achievement of the critical deformation ε_{cr} testifies about the soil transition into the new state (so-called translimiting state). The progressive soil failure in this condition is accompanied by a sharp decrease its long-term strength till residual value. It takes place mainly by disturbing of the unrecovering rigid bonds. The soil structure rearrangement in the shear zone also contributes to this process decreasing the strength of the recovering viscous bonds.

2. There are four conceptions of the clay strength properties: the standard strength (S), the long-term strength (S_t), the limit of the long-term strength (S_∞) and the residual strength (S_{res}). Let us make the analysis of the correlation between the limit of the long-term strength and the residual strength.

If the soil has mainly such bonds, which line viscous in this stress condition, then critical deformation value (ε_{cr}) is comparatively great. It can be reached by long acting shear stresses which are only somewhat more than the residual strength. It's typically for such plastic clays that the limit of the long-term strength is practically equal to the residual strength ($S_\infty \approx S_{res}$).

But if a soil has high invariable regidity, then his critical deformation is small. Accordingly, the possibility of the long-term deformation growing and the weakness of the viscous bonds are limited by small value ε_{cr}. Therefore the creep deformation are put an end under comparatively high stresses nearing the standard strength. That is why the long-term strength limit of the redid clays is as a rule high. It is characteristic for such clays that the long-term strength limit is exceeded greatly the residual strength ($S_\infty \gg S_{res}$).

3. The preparation of the slide process in the clay soils may be started in the stable slope long before the average shear stress on the potential displacement surface (τ_{av}) would be more than long-term strength corresponding to some time t. It happens because of the development of the creep deformations.

When τ_{av} is only a little more than S_∞ the long decrease of the slope steepness occures still in the pre-limiting state for the plastic clays without the sharp display of the slide displacements. It takes place because of the creep deformations. Usually there is no translimiting state under these conditions as a great redistribution of the shear stress does not take place and because the strength decrease along the shear plates after the achievement of the critical deformation practically does not occur.

If for the redid clays the average shear strength on the displacement surface is equal to long-term strength limit then the forming of this surface results in further reduction of its strength to residual value. The deformation takes place in the translimiting state for such clays, even if the shear stress only somewhat more than S_∞ unlike from plastic clays. It occurs as a distinct slice displacement which is the active the more difference is between the long-term strength limit and residual strength.

If there is a shear stress considerable exceeding S , then decrease of the slope steepness occurs in the plastic clays too as a slide displacement. It's accompanied by frail soil disturbing with further reduction its strength in the translimiting state.

However the surplus shear stress ($\Delta\tau = \tau_{av} - S_{res}$), defining the activity of the slide displacement in the translimiting state is not great usually for plastic clays. That is why the frail soil failure does not happen sharp in the beginning of the slide cycle.

The preliminary deformations in the pre-limiting state can be developed by the influence of the considerable shear stress in the slopes of redid clays with great height and steepness. Then deformations in the translimiting state will develope with large velocity and will happen as catastrophe when the shear stress would be considerable more than the residual strength.

The limit decrease of such slide steepness is usually arrived in consequence of some successive slide cycles. It's accompanied by the reduction of the strength to residual value S_{res}.

Chairman Prof. N.N.Maslov (USSR)

Thank you very much Miss.Turovskaya.
Now I invite to make a report Miss. Emelyanova.
Miss. Emelyanova will you please.

Dr. E.P.Emelyanova, (USSR)

The procedures of the estimation of slope stability have been improved by taking into consideration the influence of underground water, pore pressure, progressive failure etc. Great attention is given to rheological processes in the preparational stage of the formation of a landslide. However there are two essential circumstances which are almost wholly disregarded: 1) the kinematics and dynamics of the main stage of landslide displacement, and 2) the development of landslides in three-dimensional space.

The acceleration, the velocity and the total length of the displacement of a landslide depend on the magnitude of unbalanced forces after failure of slope and full separation of this landslide body. These unbalanced forces are equal to the difference between moving forces and forces of resistance to motion.

With the exception of landslides, originating at earthquakes, explosions and other dynamic influences, the magnitude of moving forces immediately after failure as a rule is the same, as in the moment of failure, and diminishes gradually during motion if the sliding surface were concave. The forces of resistance diminish after failure from pique value to residual one. For pique strength I understand here no pique strength determined on a little sample in a laboratory, but the maximum total strength of slope as a whole in the moment of failure, taking into account progressive failure and the influence of scale.

The bigger is the difference between the pique strength in massif and the residual strength, the bigger will be the magnitude of unbalanced forces, the larger will be the velocity and total displacement of the landslide. As had been shown by author earlier (1968) for two-dimentional case and a circular sliding surface the maximum velocity of a landslide V_{max} can be determined by following expression:

$$V_{max} = \frac{(C_p - C_r)\,L}{W}\sqrt{gR}$$

where C_p - pique strength along the sliding surface

C_r - residual strength

L - the length of sliding surface

W - the weight of landslide body (for a unit of its length along the base of the slope)

g - the acceleration of gravity

R - the radius of sliding surface

The magnitude of the total displacement of a landslide is also in direct proportion to difference between the pique strength and the residual one.

If analising landslides we would take into consideration their velocities and values of their displacement, we could estimate the proximity of strength at failure to residual strength. The strength at the moment of failure by no means can be equal to residual strength if the landslide have gained a great velocity.

As for two-dimentional case all suggested methods of calculation of factor of safety in three-dimentional space at any form of sliding surface mean the simultaneous failure at all its points. Meanwhile not only the failure developes the most unevenly along the base of the slope, but sometimes the separation of landslide body from the slope is not complete. Part of it, most frequently one of its flancs remains attached to the slope. This part is deformed by torsion, and the separated part bends at motion.

For example, the landslide of 1968 on the river Belaya (fig.IA) at its Northern flanc

Fig.1. Examples of landslides not fully separated from the slope and undergoing bending: A. Landslide on r.Belaya; B.Landslide at Vajont Dam. I- Fissures and boundaries of landslides; 2- lines of equal horizontal displacement; 3- marks.

was not separated by fissure. The horizontal displacement of this landslide increased gradually from the North to the South to 339cm at mark N3. The angle of the deflection of the landslide body was about 1°30'.

The displacement of the well-known landslide at Vajont dam from 1960 to catastrophic failure in 1963 also was a rotation around its almost immovable Eastern flanc. As can be seen on fig.IB, constructed on the base of data given in the paper of L. Müller (1964), the magnitudes of the displacement of this slide gradually and rather uniformly increased in the direction of minor landslide of 1960. The angle of deflection in this case was equal only 13'-16'. Apparently on the west side of the Vajont slide its full separation along the bedding surface occured long before the catastrophe. At the moment of catastrophe failure took place only on the east part of the slide, where the sliding surface intersected the bedding surfaces.

So not only the static conditions, but the characteristics of landslide movement should be considered at analysis of landslide accidents. More complete consideration will contribute to more realistic estimation of factor of safety of slopes.

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Chairman Prof. N.N.Maslov (USSR)
Thank you Dr.Emelyanova for your report. The next will be Dr.Brandl from Austria. Dr. Brandl will you please.

Dr.H.Brandl (Austria)

Laboratory experiments, field tests and observations in nature show, that calcium is considerably improving the shear strength only in connection with a flocculant structure of the soils. Calcite in extremely fine or even cryptocrystalline state of division (clay marls etc.) or calcium-carbonate-concretions don't influence the shear resistance effectively.

In landslide prevention practice flocculation can be attained by a stabilization with lime. The angle of internal friction and the residual shear strength increase by a gradual diffusing of lime. The intensity of chemical and physical reactions depends on content in active clay-minerals and movable silica. The maximum hitherto obtained comes to about $\Delta\phi' = 15°$. Cohesion increases approximately linearly with the logarithm of time; nevertheless it is preferable not to take this fully into account for stability analysis.

Stabilization of slippage-prone slopes by lime-piles may be performed practically by pouring in lime-water or ramming down dry calcium hydroxide, caustic lime respectively into bore holes or pipes (sometimes perforated), which must reach beneath the possible planes of sliding. The introduction of lime plugs only influences the topmost layers. The vertical holes are made by auger boring or by ramming (vibrating) pipes into the soil. Their mutual distance (0,5 to 3m- screen) and diameter (3 to 50 cm) depend on the slippage-proneness and on the permeability of the soil; a possible injection pressure is influenced by the same factors. In principle one should prefer more thin "lime-piles" instead of few holes of a large diameter. The optimal diameter is supposed to be about 5 to 10 cm. The bore holes being filled should be shut on their surfaces by ramming in some fine grained soil or by a stopper. There are no investigations about the influence of an injection pressure or steel piles remaining in the soil. Pipes are generally pulled again, as additionally stabilizing effects are supposed to be only exceptional cases. Possible injection pressures depend on the overburden by the surface soil, which is usually low; the longtime process of the stabilization can hardly be quickened.

The range of application of the lime-piles is somewhat narrow. Longtime effects and the sphere of influence within the slope are limited by the permeability of the soils: a high colloidal content certainly favours intensively chemical and physical reactions near the bore hole but it reduces a farther penetrating of the stabilizator into the surrounding soil; the cations are transported only very slowly by the pore fluid.

Montmorillonitic clays, which are usually prone to progressive failures by formation of slickensided slip surfaces, are-within 1 to 3 month-sufficiently stabilized only in the nearer vicinity of the lime piles. Complete lime diffusing throughout the soil will probably last some years; the spreading of the calcium (and magnesium) ions is favoured by the gradual flocculation of the colloidal grains, which is responsible for an increase in permeability.

Slide-prone phylites, chlorite schists and their weathered products being characterized by a great content of flat shaped, frequently orientated particles are usually highly permeable, but they haven't enough chemical activity: there is only a little flocculation or agglomeration when lime is added; the dominating platy character remains.

The method is most suitable for loosely packed loams, which are permeable and react intensively with lime according to their great ionic exchangeability and content of semi-movable silica.

By adding lime not only the shear strength parameters are improved: water content and swelling capacity are reduced too (mainly by caustic lime). These effects can be utilized fully only if seepage waters are drained away and possible water pressures are relieved by effectively functioning drainage facilities.

The investigations about the longtime durability of lime-stabilizing effects are not yet finished; but due to the irreversible reactions between soil and lime (flocculation, changes in specific areas and mineralogical character, cementation etc) a gradual decreasing of the angle of internal friction and the minimum shear strength toward the original values of the non-treated soils is not to be expected.

Calculating the safety factor of stabilized slopes it must be taken into account, that only the minimum shear strength parameters determined by multiple shearing are guaranteed under all circumstances. The results of stability analysis are more influenced by the scattering of the strength characteristics and by the assumptions on force actions of water than by the existing methods of calculation

Chairman Prof. N.N.Maslov (USSR)

Thank you Mr.Brandl for your report. Prof. Vyalov will you please be the next.

Prof.S.S.Vyalov (USSR)

The slow displacement of a slope is usually regarded as viscous flow, described by Bingham's law. Generally speaking, this assumption is true. However, certain corrections must be made. Firstly, let us clear up whether the displacement of slopes is similar to ideally viscous flow. If it is, then the zero velocity condition on the slip surface would be observed, for instance, at the contact with underlying rock in the case of an inclined slope underlayed by rock. In-situ investigation data does not substantiate this condition. Usually, not only intrinsic viscous flow of the landslide body occurs, but also its slip along the slope, similar to the slip of an ice block.

In the Permafrost Laboratory of the Gersévanov Research Institute of Bases and Underground Structures, researchers Sadovsky and Bondarenko conducted tests (on a plane-parallel shear apparatus) on the shear of soil along a rock base. They found that in addition to shear proper of the specimen, there is also shear within the limits of the thin contact layer, and the total displacement u consists of two parts: the viscous flow u_f and the contact shear in the contact zone u_r, i.e. $u=u_f+u_r$. While the viscous flow progresses with an approximately constant velocity, the contact slip develops into a progressive stage with a sharp increase in velocity, leading to failure of the slope. Viscous flow occurs when the shear stress exceeds the limit of long-term strength $\tau_\infty = C+\sigma_n \tan\varphi$. The contact shear can be represented as a displacement of a soil mass along a slip surface. This slip begins when bonds are ruptured between the mass being displaced and the underlying soil. Evidently, in this case, the strength criterion will be the residual strength $\tau_r = \sigma f$.

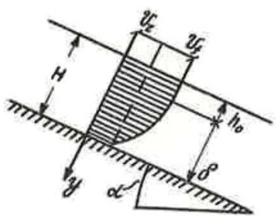
It follows that the velocity of the slope displacement is made up of the velocity of the Bingham viscous flow

$$\frac{dV_f}{dy} = \frac{1}{\eta_f}\left(\tau - \tau_\infty\right)$$

and the velocity of the contact slip

$$\frac{dV_r}{dy} = \frac{1}{\eta_r}\left(\tau - \tau_r\right)$$

Accordingly, the displacement velocity diagram of the landslide body along a slope is the sum of the curvilinear diagram of viscous flow and the rectangular diagram of contact slip (Fig.1). The displacement velocity of



the surface will equal $V=V_f+V_r$

where

$$V_f = \frac{\gamma \sin\alpha}{2\,\eta_f}\left[2y(H-h_o)-y^2\right]; \quad h_o = \frac{C+\gamma \cos\alpha\, tg\varphi}{\gamma \sin\alpha};$$

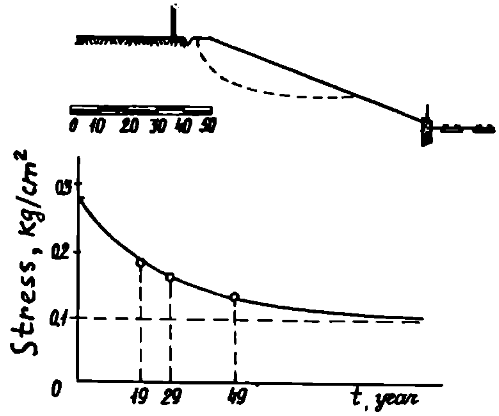
$$V_r = \frac{\gamma \sin\alpha + \gamma f \cos\alpha}{2\,\eta_r}$$

Knowing the strength and creep characteristics of the soil, we can determine the conditions under which slow viscous flow of the slope begins, develops into progressive slip and ends in failure.

In evaluating the magnitude of the long-term strength τ_∞ its variation with time in the process of creep should be taken into account

$$\tau = \frac{\beta}{\ln(t/B)}$$

A comparison is given in Fig.2 of data



calculated by the above formula with data of field tests. These data were obtained by Prof.Suklje by treating investigation results of a number of failures of slopes of cuts in London clays. The long-term strength proved to be much less than the strength determined by short-term testing. This is precisely what lead to failure. The greater the difference in the magnitudes of the above-cited characteristics, the longer the time before failure (from 15 to 47 years).

The curve shown by a solid line was plotted according to the above formula.

It is evident, that the agreement is satisfactory.

Chairman Prof.N.N.Maslov (USSR)

Thank you Prof.Vyalov for your contribution. Now I invite Mr. Janbu (Norway).

Mr. Janbu will you please .

Mr.Nilmar Janbu (Norway)

I would like to report to you, very briefly, two important experiences gained during the last year. In both cases effective stress analyses had to be applied in order to understand fully, and in detail, what actually happened in the field.

The first case deals with a large braced excavation in Oslo. The deepest part was 11m in a medium stiff clay of low plasticity and low sensitivity. According to Aas (1973) the stability of the bottom of the excavation was initially estimated at about 1.3 on total stress basis, using the undrained shear strength of the clay. This was considered sufficient for the short term condition anticipated prior to excavation.

However, after a few weeks of open excavation a noticeable bottom heave developed over a limited zone, with a corresponding increase in the subsidence outside the braced wall.

Hence, it was evident that failure was developing at least over a limited zone in the surrounding clay (Aas 73).

An effective stress analysis for this case, using hydrostatic pore pressures from the bottom of the excavation, yields a factor of safety very close to unity. More important, the effective stress analysis leads to a more realistic estimate of the size of the critical zone of yielding more in accordance with the field observation. The conc lusion to be drawn from this experience is:
The stability of deep cuts in medium stiff clay (or stiffer) should preferably be analysed on the basis of effective stress, even if the cuts is to be open for just a few weeks. One important issue involved in this type of analysis is the proper assessment of the pore pressure for the duration of the c construction period. This fact once more underscores the necessity for improving our skill in pore pressure predictions. This is to emphasize the great importance of professor Bishop's contribution a while ago.

The second case deals with a substantial slope failure in overconsolidated clay. The slide took place on Aug. 15th 1972 in Trondheim. A 3-8 m thick fill had been placed on an initial slope of about 1:2 few weeks before. The slope height was about 25 m, the length of the slide was roughly 160 m, while the depth of sliding was about 20 m at its maximum, see fig. 1a.

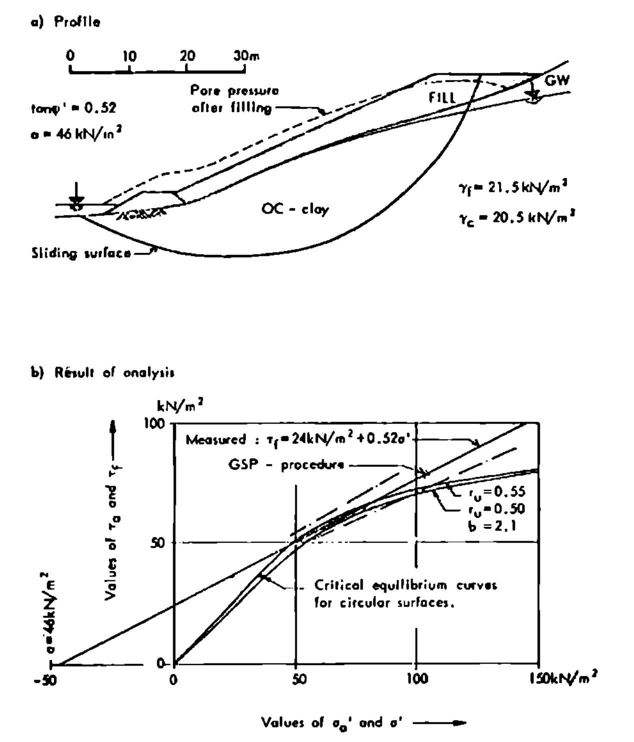


Fig. 1.
The Kroppan Slide in Trondheim, Aug. 15. 1972. Profile and result of analysis.

This slide has been analysed on the basis of effective stress yielding a factor of safety close to unity, and the calculated shape and location of the critical shear surface was very near the actual sliding surface (Janbu 73). A total stress analysis, for short term stability, may also yield an average factor of safety not very far from unity, but the scatter of undrained strengths is very large, which means that average values are questionable. Moreover, the location of the critical shear surface for the total stress analysis is very far from the actual sliding surface. The conclusion to be drawn from this experience is:
For fills on overconsolidated clay slopes an effective stress analysis of the stability should be considered a must. Also in these cases the estimate of the pore pressure due to loading, and the change of pore pressure with time after loading, is the most challenging part of the job. But it can be done, since it has to be done.

Altogether, I am convinced that practicing engineers should use effective stress analysis to a much greater scale than is costomary today, even if it may require a little more effort to obtain the proper parameters. Personally, I have become increasingly convinced that the field of application of total stress analysis, using undrained shear strength, is much more limited in validity than is recognized until now.

For instance, I am unable to understand the logic in using short term, total stress analysis for the long term, drained static bearing capacity of friction piles in clay. I am also unable to understand why we should still cling to a total stress analysis for the stability of permanent cuts and excavation slopes, since the international literature is full of case records of slope failures occuring a long time after the cut, that is under effective stress conditions.

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Chairman Prof. N.N.Maslov (USSR)

Thank you Mr.Janbu for your discussion.
Now I want to pass the word to Mr. Wagner from Austria.
Mr. Wagner will you please.

Mr.Wagner (Austria)

One type of landslide, which frequently occurs is explained by the "Contact Surface Theory". The "Contact Surface Theory" by Prof.Veder states, that water accumulation between two soil layers whose surfaces are close together is caused by thermodynamic forces. This accumulation of water reduces the frictional force in the contact surface and leads to a sliding movement of the upper layer on the layer beneath.The investigated thermodynamic forces,which lead to a movement of the usually adsorbed water in the contact surface are caused by a temperature gradient, by an ion concentration gradient and by an electric potential gradient.By decreasing the gradients, a reduction of the thermodynamic forces on the water particles is achieved.A decrease of the gradients should be attained by driving in metal bars under possible face of slide.When a metal bar is introduced,the following processes take place in the surrounding soil: 1. metal ions are released from the metal bar in the pore water of the soil and 2. differences in voltage and temperature between the two layers of soil are reduced. Metal ions hinder the movement of water mole cules and thus stop the accumulation of water in the contact surface.The factor most influential in stabilizing the soil in the area of the metal bar is,however,the transport of ionsl In order to understand these processes quantitatively,investigations are carried out in the field as well as in the laboratory. In the diffusion experiment, the movement of these ions in the soil is studied with the help of radio-chemical investigative means.Two differently prepared layers of soil are placed in cylindrical forms. The upper layer of soil contains a definite concentration of radioactive trivalent iron. The radioactive trivalent iron diffuses into the lower layer. A transport of mass is connected with the diffusion. In a second series of experiments,the cylinder specimens are prepared in such a way that the upper soil layer in the cylinder is mixed with a certain amount of trivalent iron not radioactivated. The lower layer of soil is prepared with a bivalent iron not radioactivated. In the course of several weeks,a balance will be achieved the soil layers. All investigations are carried out in triplicate experimental sets. In the first set, the cylinders as described above, are examined. In the second set, the metal bars are placed in the center of the cylinders. Construction steel such as is usually used to reinforce parts for building construction is employed for the bars. The third set includes similarly centrally placed aluminium bars. Parallel to these experiments,continuos investigations are made in accordance with classical soil physics. With the increase of stability in the area of the metal bar, an increase in the shear strength and a decrease in the water content occurs. These investigations can be of some help in solving the problems of regions which suffer from landslides.

Chairman Prof.N.N.Maslov (USSR)

Thank you Mr.Wagner for your report. Now Mr.Benarroch will be the next. Mr.Benarroch will you please.

Mr. Ruben Benarroch (Venezuela)

SYNOPSIS: General information on the factors that influence the analysis of foundations on steep slopes are discussed. The lack of appropriate methods to interpret and analyse shear strength and weathering of the schists formations limits the types,depth and bearing capacity of the foundations. The extent and direction of foliation are critical factors to be evaluated before any further analysis and design of structures can be carried out.

INTRODUCTION: Caracas, is Venezuela's capital city with a population of about two million. It is located at 900 mts. above sea level on a tightly confined and elongated valley 25 Km. long and 5 Km. wide. Its population is accommodated at an average density of about 180 persons per developed hectare.

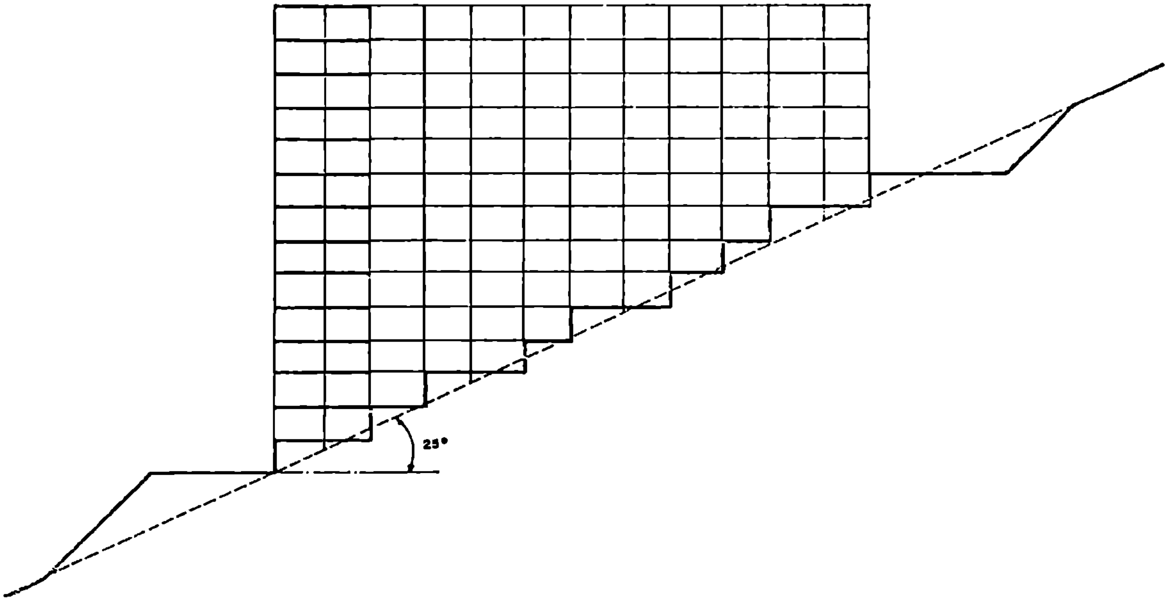
Present population growth,conpled with land scarcity, requires that housing and building construction takes advantage of the city's peripheral steeply-sloping mountains where many of the half- a million people that live in squatter dwellings (locally known as "ranchos") reside. Founded in 1928 to provide housing finance and accommodation for low-income levels of the community,the governments public housing agency, Banco Obrero, is alleviating the housing shortage by constructing housing units as rapidly as possible. It too is building on the valley's sloping hillsides because of the land shortage.

Banco Obrero has developed two solutions for taking advantage of sloping sites: (a) Tall buildings up to 18 stories high; and (b) Individual housing units with the objective of replacing the "ranchos" and improving living conditions in these areas.

The design and construction of tall buildings on steep slopes have presented complex problems with regards to foundation analysis and design,the behavior of the building structure as a system, and the need for developing appropriate construction methods to meet stringent costs requirements. The problems encountered in foundation analysis and design are herein described. 5164

BUILDING ON SLOPES: The type of loads considered in the design of tall buildings for the Caracas area comprise dead,live and seismic loads. As of now, Banco Obrero has located most of its large hillside buildings on terraces of a slope not exceeding five percent. However it is presently preparing some prototype buildings proposed for slopes up to 40 percent. Column spacing for these prototypes is about 4.70 mts. Column loadings for Banco Obrero buildings range from light loads for single-story houses to as much as 500 tons for tall apartment blocks. The prototype envisions loadings of between 70 to 370 tons depending on the number of floors. (See Fig.1).

Figure 1



GEOLOGY: Micaceous schist is the main methamorphic rock and outcrops in most of the hills around Caracas. The schist has a very variable composition and the constituents commonly found are mica, quartz and graphite. The schist is covered by a layer of residual soil that varies in thickness and composition. The thickness is about 2 mts. and the residual soil is classified as clayed sand and clayed silt.

GROUND WATER: The ground-water table in these rock formations is too deep to have any influence on foundation design. A good drainage system is required, however, to prevent erosion and deep weathering produced by intense rainfall.

FIELD AND LABORATORY TESTINGS: Schists are materials that present practical difficulties for sampling and testing because of the nature of the material. Interpretations of field and laboratory tests are complicated by their anisotropy, irregular weathering and the presence of diaclases. Field tests normally consist of direct shear tests and vertical and horizontal loadings of steel H piles. Triaxial shear, direct shear and confined compression tests are also performed in the laboratory. The following parameters are adopted $c=0.3$ Kg/cm^2 and $\emptyset=30^\circ$. It is assumed that weathered soft rock behaves like soil.

TYPES OF FOUNDATIONS: Shallow and deep foundations are considered. Individual and combined footings are analysed by Meyerhof's method. Frequently the column loads are found to impose a bearing pressure greater than subsoil bearing capacities obtained from the Meyerhof formula (1957).

Experience using this formula indicates that local Caracas mountain schist has a bearing capacity only of between 1 Kg/cm^2 and 2 Kg/cm^2 when $\beta=25^\circ$ slope ($\approx 40\%$ gradient) and when the above mentioned parameters are applied. This low bearing capacity is a result of the poor quality rock usually found in the upper ground layers. For this reason a safety factor of six is recommended. Square footings in such conditions will permit loadings up to 100 tons (assuming a maximum practical size of footing of 3m^2 with a minimum separation between footings of 4.5m), and are thus appropriate for conventional buildings of up to about five stories.

The bearing capacity of Caracas schist increases with depth; its shear strength improves and the incidence of weathering decreases. Whenever column loads exceed 100 tons drilled piers are mostly used instead because piles driving is difficult because of the schist, and also because it requires heavy equipment that is hard to handle on steep slopes.

CONCLUSIONS:

1.- Banco Obrero needs to build tall structures on steep slopes to supply low-income housing in a context of scarce land availability.

2.- Shallow foundations are recommended for column loads of up to 100 tons on Caracas hillsides. Belled piers are now being considered for column loads greater than 100 tons.

3.- Load characteristics, ground slope conditions, potential seismic effects and the low shear resistance of the subsoil (as well as its weathering and erosion due to water runoff) are the factors that mostly determine the depth and type of foundations.

4.- The lack of an appropriate theory of rupture to evaluate anisotropy and the complex effects of foliation in general makes it difficult to interpret the shear strengths of schists.

5.- The determination of the extent of foliation and weathering in Caracas schists is necessary. Geophysical methods combined with infrared aerial photography, although costly, may be capable of providing the required information.

6.- A knowledge of the direction of the foliation in schists is critical in the determination of the stability of hillside areas.

7.- Research on the behavior of foundations placed on a normal direction to the foliation is important and urgent.

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Chairman Prof. N.N. Maslov (USSR).

Thank you very much Mr. Benarroch.

We have finished the program of our Session. We have heard some very important and interesting reports.

All the material of this Session will be analysed with great care. The reports will be published in the IV volume of the Conference Proceedings.

Allow me to thank you on the behalf of the Conference Organizing Committee and personally for your active participation in this Session.

With this I announce that Specialty Session 6 of the VIII International Conference on Soil Mechanics and Foundation Engineering is closed.

RHEOLOGICAL STUDIES ON THE STRENGTH OF KRAKOWIEC CLAYS

Z. Glazer, R. Kaczyński /Poland/

Development of open - cast mining often necessitates the construction of 100 - 200m high escarpments in cohesive soils. Such escarpments have to last for shorter or longer periods of time. Observations carried out on natural slopes show that inclinations of slopes of the same height depend on time factor. Thus, rheological studies appear necessary for establishing coefficient of slope stability calculated on the basis of strength parameters of blanket - forming soils.

The paper presents results of studies on the creeping of Krakowiec clay samples subjected to unconfined compression. The Krakowiec clays represent marine, silty, illite and marly, consolidated, semi-compact clays and heavy loams of the Tertiary age. Natural planes of weakness are quite common in these clays. Soil massif built of these clays represents an anisotropic, heterogoneous and discontinuous medium. /Glazer and Kaczyński, 1970, 1971/.

The first stage of the studies on cree - ping comprised evaluations of the strength of these clays to unconfined compression. The technique involved compressing cylindrical clay samples /38 mm in diameter, 76mm in height; natural moisture and orginal texture/ rate of loading equal

$$\frac{d\sigma}{dt} = 0.08\ \mathrm{kG/cm^{-2} \cdot min/}.$$

It was found that the strength to compression oriented normally to clay lamination was $R_c\perp$ = 11,2 kG/cm^2, and that oriented paralelly to the lamination $R_c\parallel$ = 14.4 kG/cm^2; strength ratio, $R_c\parallel : R_c\perp = 1.2$. Interdependance $\sigma = f\ /\varepsilon/$ is given in Fig.1.

The second stage of the studies comprised evaluation of interdependance $\varepsilon = f\ /t/$ on the assumption that σ_o is constant. On the basis of the obtained value Rc, further studies were made through introduction of successive loadings equalling 0.9 Rc; 0.8 Rc; 07 Rc; and 0.6 Rc /Kłeczek, 1967/. The results obtained show that the studies on creep may be performed at compressions oriented normally to lamination equalling σ_o = 0.6 $R_c\perp$, and compressions oriented parallely to the lamination, equalling σ_o = 0.7 $R_c\parallel$. Thus, it appears that for long - lasting escarpments it is not justified to accept strength parameters calculated without taking into account creep value, as in such parameters overestimated by about 30 - 40 percent are obtained.

The studies carried out on creep showed a deformation change taking place in time, and accompanied by a decrease in the rate of deformation/Fig.2/. Provided that the relaxation time is not taken into account, Kelvin's model may be adapted as the first

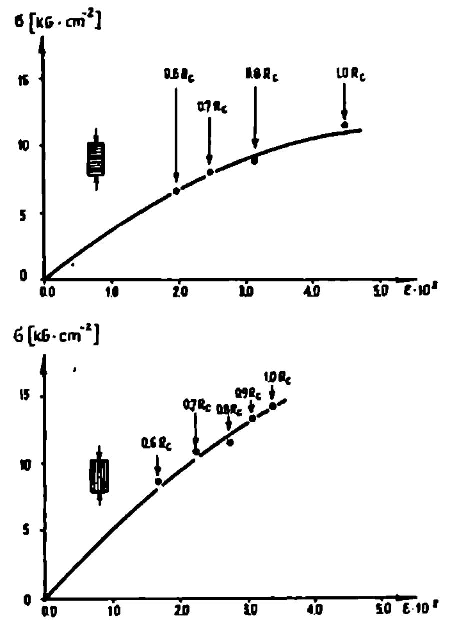

FIG.1 RELATION $\sigma = f(\varepsilon)$ FOR KRAKOWIEC CLAYS IN DIRECTIONS NORMAL AND PARALLEL TO LAMINATION

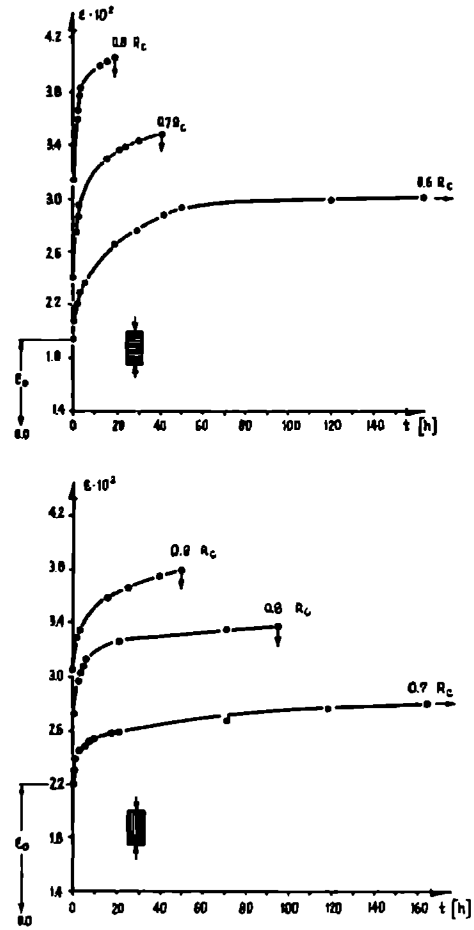

FIG.2 CREEP CURVES $\varepsilon = f(t)$ FOR KRAKOWIEC CLAYS IN DIRECTIONS NORMAL AND PARALLEL TO LAMINATION

approximation in mathematical description of the course of creep curve obtained. In that model the state is described by the equation:

$$\sigma = E \cdot \varepsilon + \eta \frac{d\varepsilon}{dt} \qquad (1)$$

In that model, the creep curve is represented by the following equation:

$$\varepsilon = \frac{\sigma_0}{E} - \left(\frac{\sigma_0}{E} - \varepsilon_0\right) e^{-\frac{1}{\eta} E \cdot t} \qquad (2)$$

Explanations of particular symbols used are given in Tab.1, Fig . 1;2.
Provided that:

$$\frac{\sigma_0}{E} = m, \quad \frac{\sigma_0}{E} - \varepsilon_0 = n, \quad \frac{1}{\eta} E = p$$

and it is found that $\varepsilon = m - n e^{-pt}$ (3)
; where m, n, p are constant.
When $t \to \infty$, $\varepsilon_{(t \to \infty)} = m = \frac{\sigma_0}{E}$

When m constant is obtained ,it is ossible to calculate the value n. In order) evaluate p constant, equation /3/ should be logarithmized, which gives:

$$\lg(m - \varepsilon) = \lg n - pt \lg e \qquad (4)$$

On the basis of the laboratory results obtained, there were found the values m, n, and p, and thereafter, the values of Young's modulus E, coefficient of viscosity, η , and time of retardation of elasticity T.The data obtained for samples of Krakowiec clays for compressions normal and parallel to their lamination are given in Tab.1.

RHEOLOGICAL PARAMETERS OF KRAKOWIEC CLAYS

TAB. 1

DIRECTION OF LOAD TO LAMINATION	YOUNG'S MODULUS E [kG/cm²]	COEFFICIENT OF VISCOSITY η [kG·cm⁻²min]	RETARDATION TIME T [min]
	220	244 000	1160
	380	840 000	1980

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STEADINESS OF TALUS SLOPES. V.V.Chkheidze /USSR/

When designing and erecting large hydrotechnical constructions and highways in the regions of mountains,there inevitably arise very many problems which are connected with the steadiness of talus slopes, that should be solved. The influence of different factors on talus slopes steadiness has not been yet sufficiently studied. The talus has been studied in the reservoir of Inguri power station and also in 22 specific regions of Georgia. But it's very difficult to perform shear tests in the field conditions,determining the angle of internal friction and clutch. On this basis the tentative table with data on physical,mechanical and design characteristics is of practical importance for the calculation of steadiness of talus slopes. According to the data of the shear tests,the author tabulated design constructions of talus of different petrographic content (see Table I).

Table I

Names of talus according to the petrografic content	Slaty	Marly	Sand-stone	Lime-stone	Talus of eruptive rocks
Angle of natural gradient	28^o-32^o	27^o-32^o	32^o-35^o	33^o-38^o	35^o-40^o
Mechanical content	finly crushed	-"-	massive fragment	-"-	-"-
Porosity %	40-45	40-45	30-35	35-40	30-40
Specific gravity of chips t/m3	2,5-2,6	2,5-2,6	2,5-2,7	2,6-2,8	2,7-2,9
Volume weight with the natural moisture T/m3	1,7-1,8	1,7-1,9	1,8-2,0	1,8-2,0	1,9-2,0
Angle of internal friction	20^o-22^o	25^o-27^o	25^o-30	32^o-35^o	38^o-40
Clutch kg/cm2	0,10-0,20	0,20-0,30	0,20-0,30	0,20-0,30	0,30 0,40

Obtained data of physicomechanical characteristics of talus ground are recommended by the authors to use for a design of steadiness of talus and constructions upon them in the case of difficulty when determining them by the field method. It's established by the tests, that more than 80mm of friction in talus doesn't influence the value of the internal friction angle. In passing from a loose packing to a very close one at an optimum humidity, the increases of the angle reaches $14^o30'$. The angle internal friction and clutch vary considerably with the content of clay fractions more than 30%. For talus with clay content more than 30%, water saturation reduces sensibly the shearing strength by 30-40%, and the clutch by 10-15%. According to the above mentioned data, the steadiness of slopes was determined. There are many different methods of talus steadiness determination to date. Calculations were made by the following methods: Kulman's method, area and moment methods, Sokolovsky-Senkov's method, method of horizontal forces, Maslov-Berer's method, method of projection,method of steadiness of slopes "F p" and others. For calculations without a fixed natural shift surface in both conditions, it's advisable to use the method of uniform strong slope "F p" and Sokolovsky-Senkov's method. Prof.Maslov's methods of uniform strong strong slope "F p" are advantageous. For determination of steadiness of constructions on talus it's necessary to use the method of areas, considering the action of extra loads on talus slopes and supporting walls. When estimating the steadiness of talus slopes of mountain water storage basins by statistical-mathematical methods, it's necessary to make calculations for several conditions:- natural (before the filling), already filled and after the decrease of water level in reservoir. The conclusions and suggestions made in the present paper can be recommended for use in mountain regions of the USSR and also abroad.

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STABILITY OF SLOPE INTERCEPTED BY A RETAINING STRUCTURE. H.Y.Fang[1], T.Atsuta[2], W.F.Chen[3] (USA)

The problem of the stability of a slope intercepted by a retaining structure as shown in Fig.1 frequently has appeared in engineering practice but design data to assess the optimum location of installation, penetration depth, and strength of the retaining structure are very scant. This lack of detailed information is due largely to the difficult procedures of analysis encountered when the conventional limit equilibrium method is used. In particular, the friction condition between the soils and the wall poses special problems and the strength consideration of the wall can further complicate the computational procedures. However, as in previous works /1/ and /2/ on the stability of slopes, the upper bound theorem of the generalized theory of perfect plasticity /5/ can be used to obtain the solutions for the critical height of the problem.

Herein, the upper bound theorem of limit analysis is employed to obtain the closed-form solutions to the problem. It is found that the limit analysis method provides a convenient and effective method of analysis for the stability of slope intercepted by a retaining structure. The tabulated results and design charts will be useful in the design of such retaining structures on a structurally unstable and weak soil slope.

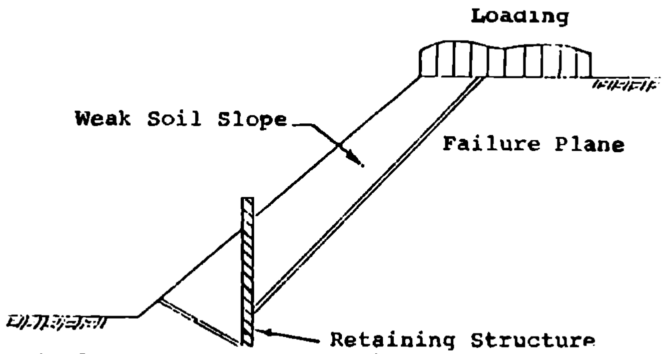

Fig.1. Weak Soil Slope Intercepted by a Retaining Structure

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LANDSLIDE AND DEEP EXCAVATION. M. FUKUOKA
(Japan)

1. Determination of soil oonotanto.
Landslides are defined as a movement of a soil mass whose daily speed is typically in the range of 0.1 to 10mm in Japan. Landslides are caused by cutting and filling operations as well as natural movement. Usually it is difficult to obtain a good testing result to account satisfactorily for the movement of sliding, even if shearing tests are often made on the samples taken from the actual sliding surface. It is necessary, in general, to assume the shear constants, namely, cohesion c and angle of internal friction ϕ , when some preventive measures are to be taken for the landslide.

Combinations of angle of internal friction ϕ , and cohesion c, yielding a factor of safely equal to unity are determined by the stability analysis retrogressively. Then, some possibility is sought to raise the factor of safety up to 1.1 to 1.3 by taking into account some counter measures such as drainage and etc.

The author assumed the relationship between σ , v and τ as illustrated in Fig.1, where σ is effective normal stress; τ , shearing resistance; v, displacement velocity of shear; c and ϕ , cohesion and angle of internal friction when displacement velocity is very slow; f, a constant depending upon soil property and thickness of sliding surface. Fig.2 shows a slice of a sliding mass and its state of movement. The equation, quite similar to the one in Fig.1, implies that the displacement velocity changes with the height of ground water level, thickness of soil mass, and slope angle. The displacement velocity is assumed linealy proportional to each of these factors, when the changes are small. There is a close relationship between displacement velocity and amount of rainfall, therefore, soil constants such as c , ϕ and f can better be obtained from the observation data. Preventive measures such as underground drainage, removal of a part of soil mass, and retaining structures may be designed using these constants. Because landslides are very complicated phenomena, the effectiveness of the counter measures should be checked during construction. The design should be changed if necessary.

2. Method of accurate measurements of displacement velocity.
Measurements of displacement velocity of landslides can be made by means of clinometers, inclinometers, and strain gages. It is very important to measure displacement velocity accurately both in magnitude and direction in order to check as soon as possible the effectiveness of counter measures. This is also important to recognize the area of movement in a very short time. Fig.3 shows a very simple strain gage used by the author, which has span length of about 60m and is capable of measuring the displacement with an accuracy 1 : 1000 mm. The peg P_1 was set on the sliding mass and the peg P_2 was set on the firm ground (Fig.3). The second peg P_2' was set on the firm ground, too, but P_1, P_2 and P_2' were placed at the three corners of a triangle. Displacement velocity was as small as 0.1 mm per day, but the velocity and the direction of movement at point P_1 was obtained in a matter of a week.

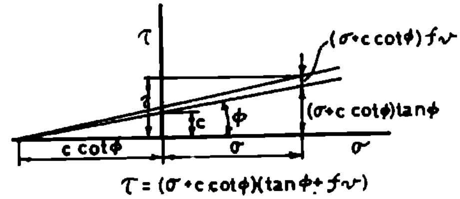


Fig.1 Relationship between σ , v and τ .

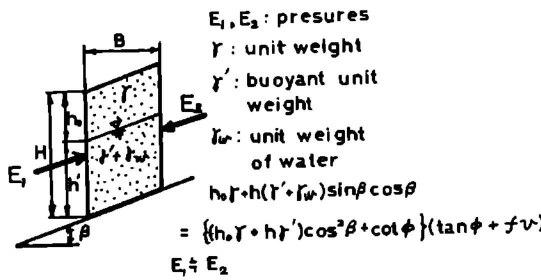


Fig.2 Equation of Movement

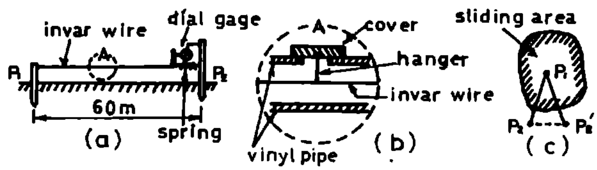


Fig.3 Strain Gauge

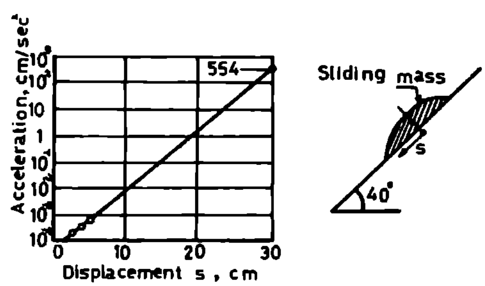


Fig.4 Relationship between Displacement and Acceleration

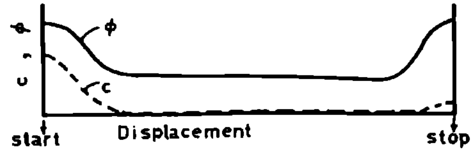


Fig.5 Relationship between Displacement and c, ϕ

3. Landslides of thin layer on hillside by heavy rain.

Many lives have been lost by landslides of thin layer on hillsides by heavy rain every year. The layer is covered with trees and grasses. Roots of plants penetrate deep into the base, and help the thin layer attached to the base. It is impossible to find out whether the layer will slide down by heavy rainfall by analysing with the soil constants from laboratory tests. A thin layer of loam on a slope with inclination 40 - 45° recently slid down near Tokyo. Fig.4 shows an example of relationship between acceleration and displacement that occurred in this occasion. The acceleration increased with displacement. The ground surface at which the failure took place was perfectly saturated by heavy rain storm, and the frictional coefficient between the sliding soil mass and the ground underneath was about 0.2. The speed of the soil mass was extremely high, and a vigorous disturbance took place. Then the soil mass lost its speed. Fig.5 shows relationship between displacement and c, ¢ schematically. Velocity of a sliding mass changed with displacement also, and therefore, we may say that the acceleration changed with velocity.

4. Method of stabilizing deep cut with piles.

Fig.6 shows an example of stabilizing a deep cut (15 - 20 m) with rows of concrete piles, a landslide took place during the construction work of a highway. Fig.7 shows another example of stabilizing a deep cut (about 20m) with frame works of large diameter piles. The finite element method was used to analyse stresses for design, but satisfactory results were not obtained. Measurements of stress and deformation during construction has been conducted to check safety. Fig.8 shows moment and displacement of a pile penetrating vertically through a sliding plane. These formulas are useful for design.

5. Part of removal of sliding mass for control.

A series of strain gages (spanning about 10 m) are set along a center line of a sliding mass to measure the expansion and contraction of the surface. Generally, the upper part is extended and the lower part contracted. An active pressure may work on the vertical wall of the extension zone, and on the other hand a passive pressure may act on a vertical wall of the extremely contracted zone. When removal of a part of the sliding mass is contemplated, it is not reasonable to remove only the top of the sliding mass at the extension zone. It is also not recommendable to remove the top of the sliding mass at the extreme contraction zone, which is acting as a retaining structure for the slide.

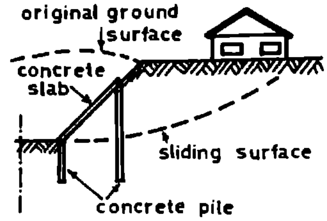


Fig.6 Deep Cut for Road Construction

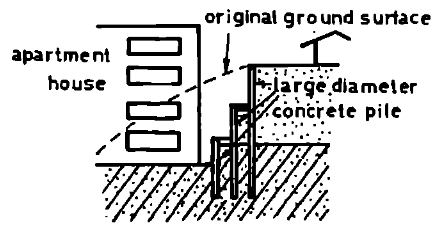
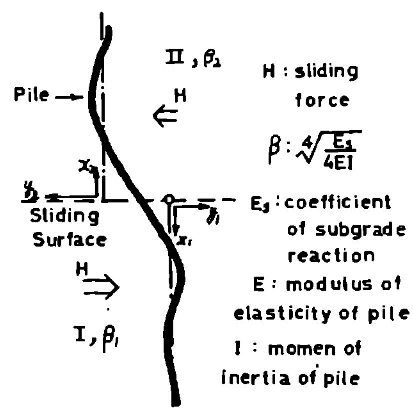


Fig.7 Deep Cut for Apartment House



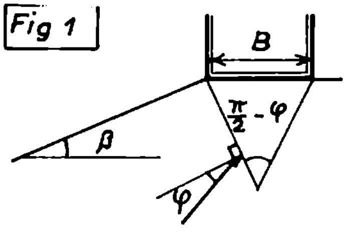
$$\begin{cases} y_1 = \dfrac{-H}{4EI\beta_1^2} e^{-\beta_1 x_1}\left\{\left(\dfrac{1}{\beta_2}+\dfrac{1}{\beta_1}\right)\cos\beta_1 x_1 - \left(\dfrac{1}{\beta_2}-\dfrac{1}{\beta_1}\right)\sin\beta_1 x_1\right\} \\ y_2 = \dfrac{-H}{4EI\beta_2^2} e^{-\beta_2 x_2}\left\{\left(\dfrac{1}{\beta_1}+\dfrac{1}{\beta_2}\right)\cos\beta_2 x_2 - \left(\dfrac{1}{\beta_1}-\dfrac{1}{\beta_2}\right)\sin\beta_2 x_2\right\} \end{cases}$$

$$\begin{cases} M_1 = \dfrac{H}{2} e^{-\beta_1 x_1}\left\{\left(\dfrac{1}{\beta_2}+\dfrac{1}{\beta_1}\right)\sin\beta_1 x_1 + \left(\dfrac{1}{\beta_2}-\dfrac{1}{\beta_1}\right)\cos\beta_1 x_1\right\} \\ M_2 = \dfrac{H}{2} e^{-\beta_2 x_2}\left\{\left(\dfrac{1}{\beta_1}+\dfrac{1}{\beta_2}\right)\sin\beta_2 x_2 + \left(\dfrac{1}{\beta_1}-\dfrac{1}{\beta_2}\right)\cos\beta_2 x_2\right\} \end{cases}$$

Fig.8 Pile penetrates through Sliding Surface

ESSAIS DE FONDATIONS SUPERFICIELLES SUR TALUS

Y. LEBEGUE (France)



Des essais de fondations reposant au sommet d'un talus ont été réalisés en vue d'examiner la validité de formules de force portante obtenues par extension de celles de CAQUOT et KERISEL (1966) ; on admet que sous la semelle se forme un coin solidaire de la fondation et présentant une section triangulaire isocèle d'angle au sommet ($\pi/2 - \varphi$)(LEBEGUE 1974)

ETUDE EXPERIMENTALE

1 - Deux dispositifs d'essais ont été utilisés :
. à petite échelle : semelle de largeur B1 = 6 cm et de longueur 20 cm, placée entre les 2 faces verticales opposées d'une cuve de 20 cm d'épaisseur ;
. en semi-grandeur nature : fondation de largeur B2 = 20 cm et de longueur 100 cm, foncée dans une fosse de section carrée de 3 m de côté avec un vérin de 100 tonnes.
Dans leur longueur les semelles comportent 3 parties, 2 latérales de garde et une centrale où sont effectuées les mesures de précision.

2 - Le milieu expérimenté est un gros sable de rivière sec, essentiellement siliceux.

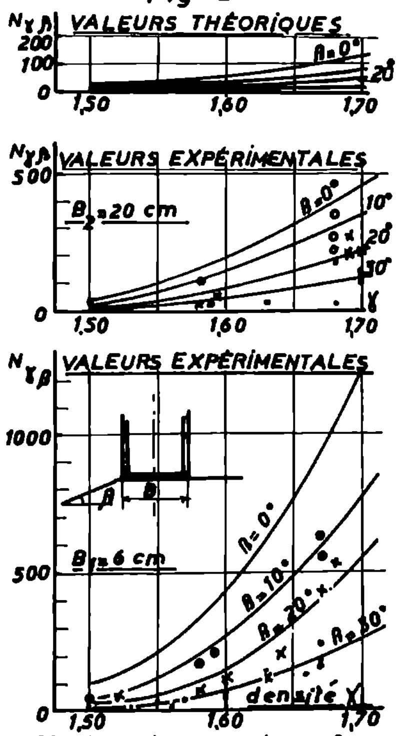
3 - Le programme expérimental a comporté pour les 2 fondations et diverses densités de sable, des essais en surface et à 2 profondeurs différentes pour des talus d'inclinaison 0°, 10°, 20° et 30°.

RESULTATS

1 - Observations
La rupture se produit toujours du côté du talus. La forme du coin solidaire de la semelle et celle de la surface de rupture sont voisines du schéma théorique admis. L'enfoncement nécessaire à la rupture est d'autant plus faible que la densité est plus élevée.

2 - Résistance à la rupture : sur la fig. 2, seulement pour les fondations superficielles, on a porté en fonction de la densité γ du sable, les valeurs théoriques et expérimentales pour les 2 modèles, sous la forme du coefficient du terme de surface pour une inclinaison β du talus $N_{\gamma\beta} = q/0,5\ \gamma B$

- Fig 2 -



(q contrainte limite moyenne). - Conformément aux calculs, la résistance croît en fonction de la densité γ et donc du frottement φ du matériau, ainsi que de la profondeur de fondation ; elle diminue lorsque l'inclinaison du talus augmente. Mais les coefficients expérimentaux de force portante sont supérieurs à leurs valeurs théoriques, et ce d'autant plus que le milieu est plus serré et la semelle moins large (effet d'échelle).

3 - Influence de l'inclinaison du talus
Pour l'étudier on a calculé les rapports $N_{\gamma\beta}/N_\gamma$ et $N_{q\beta}/N_q$ pour les termes de surface et de profondeur. Sur les fig. 3 et 4 on a donné les valeurs correspondant au sable mis en place aux densités de 1,69 (φ = 40° à l'appareil triaxial) et de 1,57 (35°). L'inclinaison du sol diminue sa force portante et les coefficients théoriques et expérimentaux coïncident bien.

Fig. 3
γ = 1,69
φ = 40°

(Points expérimentaux pour
B1 •
B2 ⊙)

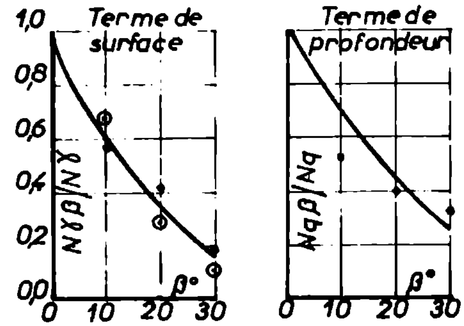
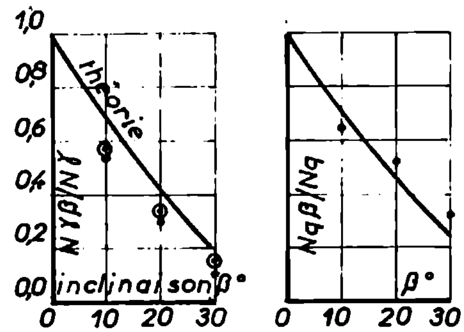


Fig. 4
γ = 1,57
φ = 35°



CONCLUSIONS

1 - L'allure du phénomène observé correspond bien à la théorie.
2 - Les coefficients expérimentaux de force portante sont supérieurs à leurs valeurs théoriques, et font apparaître un net "effet d'échelle" dans les milieux sableux. Cependant la différence entre les résultats théoriques et expérimentaux s'atténue lorsqu'on atteint les fondations réelles.
3 - L'inclinaison du sol entraîne une diminution de sa force portante dont la théorie rend bien compte.

On retrouve les particularités observées sur les fondations habituelles, reposant sur sol horizontal. Ces conclusions justifient l'utilisation des formules proposées lorsque l'assise est inclinée.

Références :
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LEBEGUE (1974) - Etude Expérimentale des Fondations Reposant sur un Talus. A paraître dans les Annales de l'Institut Technique du Bâtiment et des Travaux Publics - PARIS.

ANALYSIS OF PROGRESSIVE FAILURE IN CLAY SLOPES.

K.Y. Lo and C.F. Lee (Canada)

This paper describes an approximate solution to the problem of progressive failure. The solution itself is composed of two parts:

(a) A finite element analysis of the stress distribution in the slope (taking into account the strain-softening behaviour of the material), followed by an assessment of slope stability in terms of a "residual factor" (Skempton, 1964).

(b) Wherever applicable, a subsequent incorporation of the time element into the analysis for an estimation of the time of slope failure.

Stress Analysis in Strain-Softening Material

The method used in the finite element solution consists essentially of the following steps of operation:

(a) An elastic analysis is performed with modulus E_1.

(b) The state of stresses determined are compared with a chosen failure criterion (fully drained or undrained). "Excess" stresses are removed from "overstressed" elements by applying systems of stresses equal in magnitude but opposite in sign. Simultaneously, the strains for these "overstressed" elements are brought back to the peak condition.

(c) New stiffness matrices are generated in each subsequent step of stress release and transfer, and the post-peak relationship is obeyed in elements where the peak strength is exceeded.

(d) The process is repeated until convergence is finally obtained (i.e., no significant amount of "excess" stress can be detected in any element).

An Integrated Approach to Stability Analysis

The residual factor R, as originally defined by Skempton (1964), is "that proportion of the total slip surface in the clay along which its strength has fallen to the residual value." This physical definition will be presently retained, and its numerical value is now given by $R = D/L$, where L = length of the slip surface, and D = that portion of L along which residual strength operates.

To take into account the effect of the "overstressed" zone on slope stability, it is convenient to define a "corrected" factor of safety F_o such that the "residual strength" governs the "overstressed" portion of the potential slip surface, and the peak strength governs the pre-peak portion of it.

The above approach may be extended to treat the time-dependent phase of progressive failure. For clays that exhibit decrease in drained strength with time, the overstressed zone will propagate; hence, the residual factor increases and the factor of safety decreases. It is clear, therefore, that both the residual factor and the factor of safety are functions of time after slope formation. For a complete quantitative analysis of the problem, it is convenient to establish the following relationships:

(a) A relationship between the residual factor and factor of safety,

(b) A relationship between the residual factor and time, and finally

(c) A relationship between factor of safety and time.

To establish relationships (b) and (c), the rate of decrease of drained strength with time must be known. The effect of time on the drained strength has been discussed by Lo (1972), who suggested a linear relationship between the drained strength and logarithm of time to failure, with the drained residual strength as the lower limit. Given this (or other) rate of drained strength decrease, then for each time interval "t" after slope formation, the finite element analysis is made using the drained strength which is operative at t. The extent of the "overstressed" zone and the residual factor R may therefore be determined. The factor of safety at t (F_t) may be computed by an appropriate limit equilibrium analysis, taking the overstressed zone into account. The procedure is repeated for several time intervals, from which R and F_t are determined for different times. The relationships of R versus F_t, R versus t, and F_t versus t are therefore established.

Case Histories

To illustrate the above approach to stability analysis, three case histories will be examined presently. All of these are first-time slides in slopes of London clay, which have been chosen because:

(a) Documented first-time slides in London clay cover a wide spectrum of time to failure, from shortly after slope formation to over 80 years.

(b) The soil parameters required as input data in the finite element analysis are well-defined and well-documented for London clay. An average value of K_o = 2.5 can be taken from Skempton (1961) and Bishop et al (1965). An average value of the pre-peak modulus E_1 of 2000 psi. can be measured from the stress-strain curves given by Bishop et al (1965), and can be used in the stress analyses.

The case histories analysed are the Northolt slide of 1955, the Sudbury Hill slide of 1949, and the Upper Holloway slide of 1951. These cuttings failed 19, 49 and 81 years, respectively, after slope formation.

A detailed description of the Sudbury slide may be found in Skempton (1964), and Skempton and Hutchinson (1969). Fig. 6(a) shows the section which failed in 1949. Average values of index properties for London clay in this area are w = 31, LL = 82, and PL = 28. Drained tests on brown London clay at seven different sites from depths between 6 and 22 ft. yield the peak parameters c' = 230 psf. and ϕ' = 20°. These tests were made on 6 cm. square shear box and 1½ in. x 3 in. triaxial specimens, typically with a time to peak failure of about one day.

The residual strength parameters are $c_r' = 20$ psf. and $\phi_r' = 13°$.

Stability analyses with non-circular sliding surfaces yield a factor of safety of 2.27 with respect to peak strength, and 0.74 with respect to residual strength. It follows that the average strength mobilized at the time of failure lies between the peak and the residual values. In other words, prior to the occurrence of the slide, some portion of the slip surface had fallen to the residual state. Obviously, the effect of time cannot be disregarded in this case.

For the purpose of the present analysis, the rate of reduction in drained strength with time has to be determined. Let this be denoted here by "k", and let its value be expressed in percent per log cycle of time. For the present purpose, it is not unreasonable to assume a value of k = 6% as the average rate of strength decrease per log cycle of time under the drained condition, but values of k of 4% and 10% are also used to complete the analysis. An alternative view is to backfigure k using this record, and the value determined will be applied to other case records in the same clay, so that the consistency of results may be examined.

The propagation of the failure zone during 4 log cycles of time was thus determined, and Fig. 6(a) shows the results for the k = 6% case. The change of residual factor with time may then be deduced, Fig. 6(b). The graphical relationship between the factor of safety and the residual factor is shown in Fig. 6(c). With the residual factor at the end of each log cycle of time determined in the finite element analysis, the corresponding factor of safety can be read off from Fig. 6(c). This makes possible the final plot of factor of safety versus time shown in Fig. 6(d). On extrapolating the curve to factor of safety = 1.0, the time of slope failure can be estimated. It is interesting to note that the k = 6% case gives a predicted time rather close to 40 years, whereas the k = 10% case predicts failure in no more than 3 years. The very high sensitivity of the predicted time to the value of k chosen is a consequence of the logarithmic relationship between strength and time. To evaluate the admissibility of the results, additional case histories have to be analysed on the basis of the k = 6% assumption, and the overall consistency may then be verified.

Using k = 6% thus deduced, the Northolt and Upper Holloway slides were similarly analysed, and the results are plotted in Fig. 9. It will be seen that the predicted times to failure are not unreasonable.

Detailed discussions of the method and analyses performed are contained in a paper to the Main Session of this Conference (Lo and Lee, 1973).

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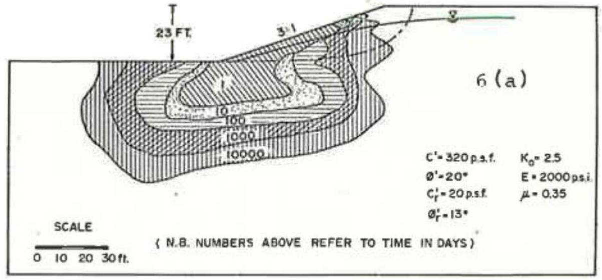
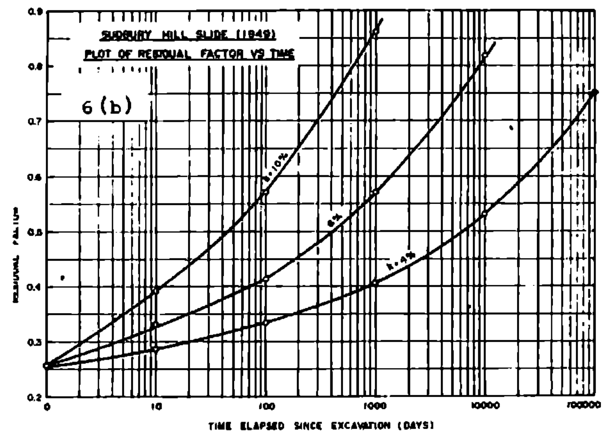
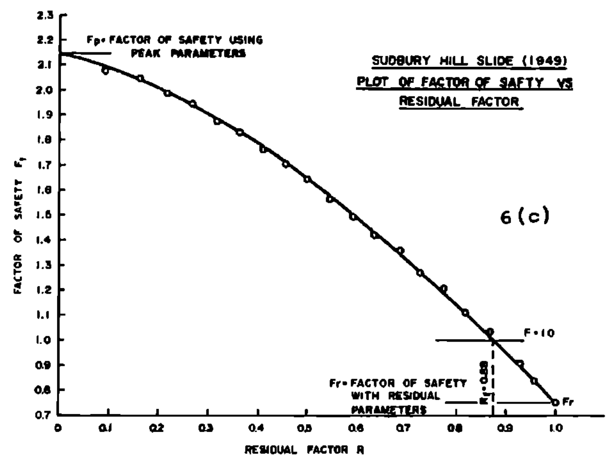
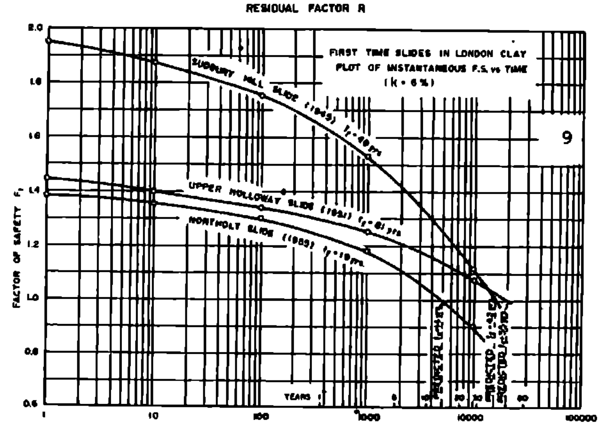


SUDBURY HILL SLIDE 1949 · PROPAGATION OF FAILURE ZONE WITH TIME







CENTRIFUGAL SIMULATION OF NONCONSOLIDATED BANKS AND THEIR BASES. Y.N.Malushitsky(USSR)

Problems of defining the limiting equilibrium of the nonconsolidated dumps-banks or their bases have no satisfactory analytical decisions since during piling contrary processes simultaneously take place:consolidations and risings of the shearing forces which are in the complex dependence from such factors as variable composition of the rock mass,their initial moisture and density, velocity and way of piling and so on.The simplest way of finding the needed decisions is in the application of the method of the centrifugal simulation,which is based on the criterion of the approximate similarity of G.I.Pokrovsky.The models of dumps banks and their bases built from the materials of their natural prototypes(keeping their physical and mechanical properties)are loaded instead of the volume forces of the weight (acting on the prototype)by the volume centrifugal forces,being created in the rotary models in the scales which are reverse to the linear dimensions model scales.

The indices of stability of the dump-bank are:I)critical altitude of the slope H_{cr} above which the slope,stockpiled under constant angle of rolling down the rock α_{rol} begins spontaneously to decrease the angle, 2)dependence $\alpha_{lim}=f(H)$,where α_{lim} -limiting resultant angle of the slope and H - height of the slope,which is higher than H_{cr} We distinguish two meanings of α_{lim} : α'_{lim} up to the arising of through surface of the arising in the dump and α''_{lim} - after the arising of through surface of the sliding; $\alpha_{rol} > \alpha'_{lim} > \alpha''_{lim}$.

The appearance of the models is determined by the geometric similarity of the contours and deformation of the nature and model and by the identity of the physical state and properties /w -moisture, δ -density, φ and c -strength/ of materials of the dump-bank in the equivalent as to positions of time and space points of the prototype and model.

Values of the summary errors resulted in the simulation are investigated by means of the comparison of the results of the simulation of the models alike of different scales,and the comparison of these results with the data of the studied natural prototypes-dumps. They are in the range of 4-7%. The index of the stability of the nonconsolidated dependence from rock composition of the dump-bank or from its base,from moisture of the rock /w %/ and their initial density / δ T/m^3 / from the capacity / Q m^3/h/ and the way of stockpiling the dump-bank. The finding of the degree of influence of each of these factors in the form of pair dependences is realized by the method of centrifugal simulation by the way of changing the numerical value of the corresponding index in the series of testing models under condition of the preservation of the constancy of the values of the other indices.

Using in simulation the factor experiment method we find the influence on the stability of the bank during simultaneous acting of several variables,for example, $H_{cr}=f(w,\delta,Q)$. For this the plan of the experiment is being made in such a way that it comprises all the possible combinations of the equations of the variables.On the base of the treatment of the results of the experiment we make the mathematical model of the process, the analysis of which gives the possibility to determine the nature and degree of the influence on initial value H_{cr} ,not only separate variables but their joint influence. As the initial we take linear model of the process,which is enough acceptable for our case. The equation of the full mathematical model is:

$$H_{cr}=b_0+b_1w'+b_2\delta'+b_3Q'+b_{12}w'\delta'+b_{13}w'Q'+b_{23}\delta'Q'+b_{123}w'\delta'Q',$$

where $b_0, b_1, b_2, \ldots b_{123}$ -regression coefficients being determined by the calculation.w', δ', Q'--dimensionless(normalized)values of the variables.

After carrying out the calculations and the substitution of the dimensionless values of the variables by their natural values / w, δ, Q /quantitative expression of the critical altitude H_{cr} is crystallizing in a simple formula,defining not only dependences of the critical altitude on principal factors but on their interaction.

For example,for the dumps of the tertiary clays the calculating formula is:

$$H_{cr}=176{,}74-4{,}83w-9{,}08\delta-0{,}031Q+0{,}001wQ$$

/in m/.

Curves $\alpha_{lim}=f(H)$ and determinations H_{cr} when $\alpha_{rol}=35^{\circ}$,founded on the results of the dumps models of these clays at different conditions of moisture,original density and capacity of the stockpiling are shown on Fig.I.Dispersion of the curves shows the significance of each variables.

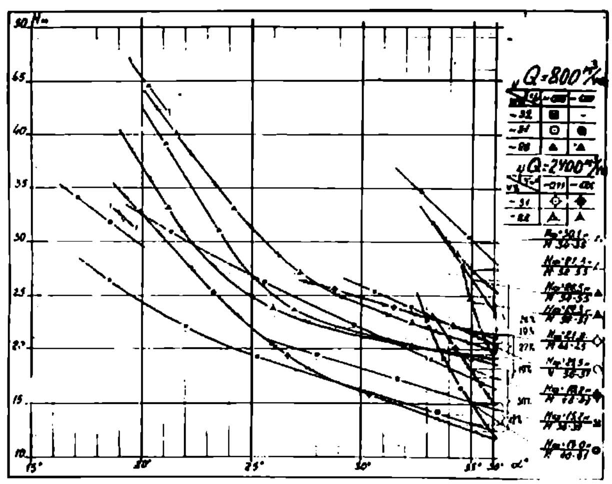


Fig.I.Dehendences $\alpha_{lim}=f(H)$ for dumps of the tertiary clays with variables w, δ, Q.

THE EVALUATION OF THE EXISTING METHODS OF ANALYSIS BASED ON THE STUDIES OF ACTUAL LANDSLIDES. Maslov N.N., Braslavsky V.D., Nguen Tchap (USSR)

46 slopes existing in the USSR, Britain, Norway and the Democratic Republic of Viet-Nam have been analysed. Composed of various clayey soils, the slopes were from 6m to 100 m high, with the slopes angles from 10° to 50°, and were divided into two groups: group I - dry slopes, viz. those having no water bearing strata of a definite thickness; group II - wet slopes, viz. those subjected to the effect of ground waters. Stability has been evaluated by methods of round-cylinder-shaped surfaces of sliding (Taylor, Terzaghi, Tchugaev, Fyodorov, Krey) as well as of the pre-set (determined by geology) surfaces of sliding (Maslov-Berer, Tchugaev, Shakhounyants); by a method of ultimate equilibrium developed by Sokolovsky V.V., and by an approximation method "F_p" (of a slope of equal strength) which was developed by Maslov N.N. and according to which the slope angle for the limit condition of the slope is equal to the angle of shear strength corresponding to the given formation. The design values of ϕ ; C are based on the experimental, arithmetic-mean values.

It has been determined that for all the methods of round-cylinder-shaped surfaces of sliding (RCSSS) stability factors (F) are almost similar and considerably exceed I. Thus, for example, for the slopes of the first group, according to the method of Terzaghi, F = 2.3-4.2, while for the slopes of the second group, F = I.2-2.0 (Fig.I).

According to the methods of the pre-set surfaces of sliding the values of F are close to I. According to the method of Sokolovsky for the first group F = I.2-3.9, while for the second one F $\approx$ I. It is worth mentioning that F $\approx$ I (Fig.I) for the slopes of both groups when the approximation method "F_p" is applied.

It can be supposed that the excessive values of F obtained by the methods of "RCSSS" resulted from an incorrect position of the surface of sliding because the values of F calculated by the methods of the pre-set surfaces of sliding were close to I. Yet, in the methods of "RCSSS", despite their different prerequisites, the values of F were almost identical. Thus, the position of the surface of sliding can be disregarded when F is determined. It has been revealed that for the slopes of the first group the water effect was insignificant. Under wet conditions obtained in particular by the methods of "RCSSS" F_{wet} was equal to 2.74 against F=3.30 and F_{wet} =3.34 against F = 3.6. For the slopes of the second group for which the methods of "RCSSS" were applied, F was lower but as compared to the slopes of the first group the experimental value of $C_{des.}$ for the slopes of the second group was also low.

It can be supposed that the exceeded values of F resulted from the use of the excessive shear strength parameters (ϕ ,C). An appropriate analysis has been made to determine the said parameters by various (adopted in practice) methods of averaging the values of ϕ and C. Such a procedure,

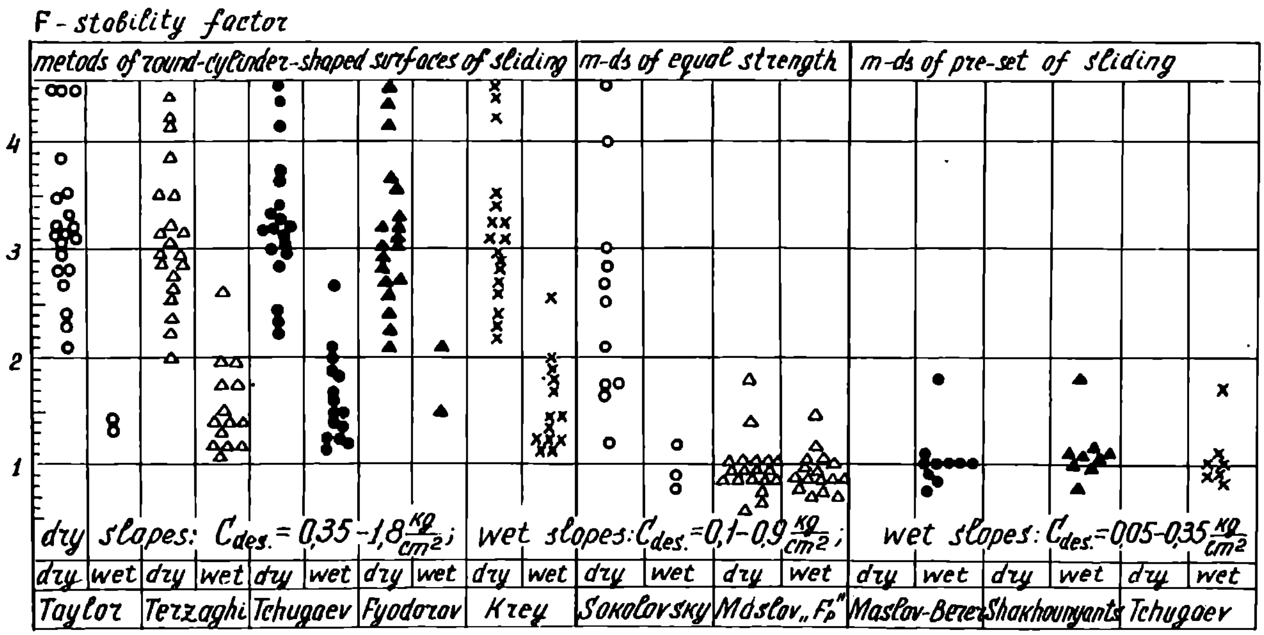


Fig. I. Stability Factors of Actual Landslides Based on Various Methods of Calculation.

the validity of which is still to be proved, makes it possible, by applying the methods of "RCSSS", to obtain the reduction of the values of F to F = 1.5 - 2.0 for the slopes of the first group and F = 1.10 - 1.5 for the slopes of the second group. Consequently, it has become impossible to prove the exceeded values of F by the excessive design values of Φ and C.

An attempt has been made to find the explanation of this phenomenon in the excessive values of cohesion. Various calculations have been made to evaluate friction (F_ϕ at C=0) and cohesion (F_c at Φ =0) stability.

It has turned out that for the slopes of the first group composed of hard and semi-hard soils, with the pre-set values of $C_{des.}$ and those of Φ = 0, the values of F_c is far greated than I, viz. the value of $C_{des.}$ is overevaluated. Besides, at C=0 and Φ =constant, F_ϕ =0.5 - 0.90 (Fig.2) which makes it possible to conclude that cohesion cannot be completely excluded from calculations as proposed by A.Skempton (1957).

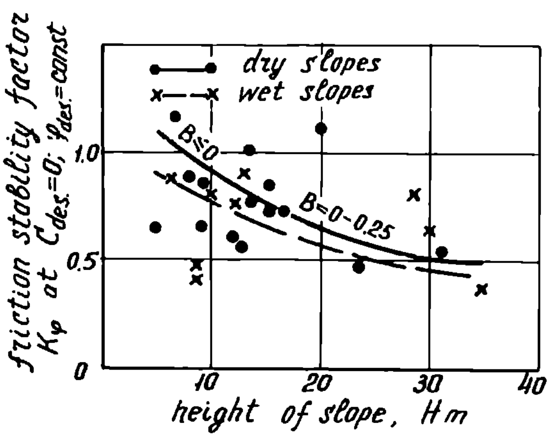


Fig.2. Relation of Friction Stability Factor at C=0 From Height of Slopes.

It has been ascertained that when the "RCSSS" methods are applied the values of F come closer to 1 for the very high slopes. This also proves that in the given case the value of $C_{des.}$ has been overevaluated since it is common knowledge that with the increase of the height of the slopes cohesion becomes less important in providing slopes stability. The analysis has shown that when applying the "RCSSS" methods it is necessary either to somewhat reduce the value of cohesion against its design value (without equalling it to zero) or not to take $C_{des.}$ for the whole surface of sliding, viz. again to reduce its specific value.

The first approach seems more acceptable since in this case, apparently, only cohesion Σ_w is to be taken into account as the cohesion of the reversible nature (according to Maslov N.N. $\tau_{pw} = \sigma \cdot tg\, \Phi_w + C_w$ where $C_w = C_c + \Sigma_w$) while irreversible structural cohesion C_c must be excluded from the calculations. Thus, it has been revealed that for the old slopes the value of $C_{des.}$ should be reduced against the experimental values especially for the slopes composed of hard and semi-hard soils.

The data available make it possible to assume that the reduction of $C_{des.}$ in the soils of the strata of the slope accounts for irreversible structural cohesion being disturbed in the course of slope deformation which might occur under conditions of creep.

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THE STUDY OF THE LANDSLIDE SLOPE STRESS STATE FOR ESTIMATING ITS STABILITY. Maximov S. USSR.

Slope stability is usually determined by means of static analysis for which it is always necessary to use certain schemes of rock massive. However this schematic approach can lead to the essential divergence of the analysis from the nature and mechanism of sliding that can cause wrong estimation of stability. Besides, when using analysis, the attention payed to the problem of stress state of rocks in zones of sliding is not sufficient, the distribution of stress in the landslide body and its influence on the formation of stability not being taken into account as well. That is why the calculation of stability, which we get, is not sufficiently accurate.

So, several scientists /Tertzagy K. and Peck R. 1948, Ter-Stepanjan 1957, Emelianova 1960, 1972, Popov and Maximov 1960/ suggested that the stability of the rock mass should be determined by means of factor of safety, which is the ratio of soil strength to the value of maximum tangental stress. For this purpose we should know the distribution of strength properties of soils composing the slope and distribution of tangental stress in this massive.

Determination of the strength of soils, i.e. their shear resistance, is usually carried out by means of laboratory or special field testings.

One can measure the stress state of rocks in field environment. The other method is laboratory modeling when values of stress and their distribution can be observed on the models, imitating the rock massive of rather complicated geologic structure. To determine the stress for the state preceding the failure the method of photoelasticity or tensometric net can be used and for the state of equilibrium or failure- the method of equivalent materials. Modeling allows us to determine the values of stress in different points of section, the complete state of stress distribution, their direction, to reveal the zones of stress concentration, the parts of tensil stress and so on.

Observing the requirements of similarity theory such researches allows to evaluate the stress state of the massive rather completely.

Comparing the strength and maximum tangential stress one can pick out the zones of the strength or even higher. In this case it is possible to have the loss of stability and failure. At the same time the zones are revealed where the stress is lower than the strength, i.e. where stability is secured. This analysis allows to direct the research of rock properties primarily to the region where the rooks can be unstable.

The research of the stress state of several landslide slopes was carried out at the Department of Soil Engineering and Engineering Geology of Moscow State University. Among them was the high bank slope of the river Oka, composed of permiane deposits /alternation of marls, weak sandstones and clays/ covered by the mantle of Quarternary deposits. The layer of clay with higher compressibility and low strength properties /angle of internal friction 16°, cohesion 0.6 kg/cm^2/ is found in the permiane deposits. Other deposits of this age possess higher property indexes. Landslide accumulations are located in the low part of the slope and are represented by crushed, remolded and saturated soils with very low strength index /angle of internal friction 6°, cohesion 0.2 kg/cm^2/ and high compressibility.

Stress state study of the slope was carried out by photoelasticity method on flat models imitating non-uniform geologic structure of the slope.

Distribution of the maximum tangental stresses shows that in the permiane rocks, the layer of weak clay including, they do not exceed 4.0-4.5 kg/sm^2. The calculated strength value of this clay with the action of normal stress taken into account, is equal to 5.0 and more kg/sm^2. Thus, the strength of the clays appears to be higher than the stresses, and there should not be any squeezing of clay. The existence of additional load of landslide body to the foot of the slope still increases their stability.

The stability state of the landslide masses is basically different. The modeling showed that tangental stresses in this part of the slope reach 1.5-2.0 kg/sm^2 while the calculated strength is 1.2-1.5 kg/sm^2. That points to their equilibrium or unstable state. In nature this is the moving landslide. The studies which were carried out helped to choose the correct antylandslide measures.

The mentioned above study of stress state of landsliding slope for determination its stability shows the possibility of such approach, though it cann't replace completely the static analysis of stability.

Mr. P.A. Rochette (England)

SUMMARY. The study of the properties of the stress ratio diagram ($\frac{q}{p}$-p in log-log scale) emphasizes simple stress relationships and the need to measure the friction coefficient mobilised in the same conditions of roughness, moisture and stress than in the field. A principle of behaviour for unloading conditions is proposed together with the subsequent design of rock slopes and excavations. The additional shear resistance due to the occurence of cohesion or of geometrical irregularities at the surface of the discontinuity, has not been investigated in the scope of this communication.

1. Principles for stress relationships for a flat discontinuity in rock. Fig.1-1 summarizes the phenomenom of friction occuring at the real contacts of a unit area of surface. The proposed formulation of the stress ratio as being proportional to p^{-a} is compatible with the many possible processes of friction (see fig.1-1a).

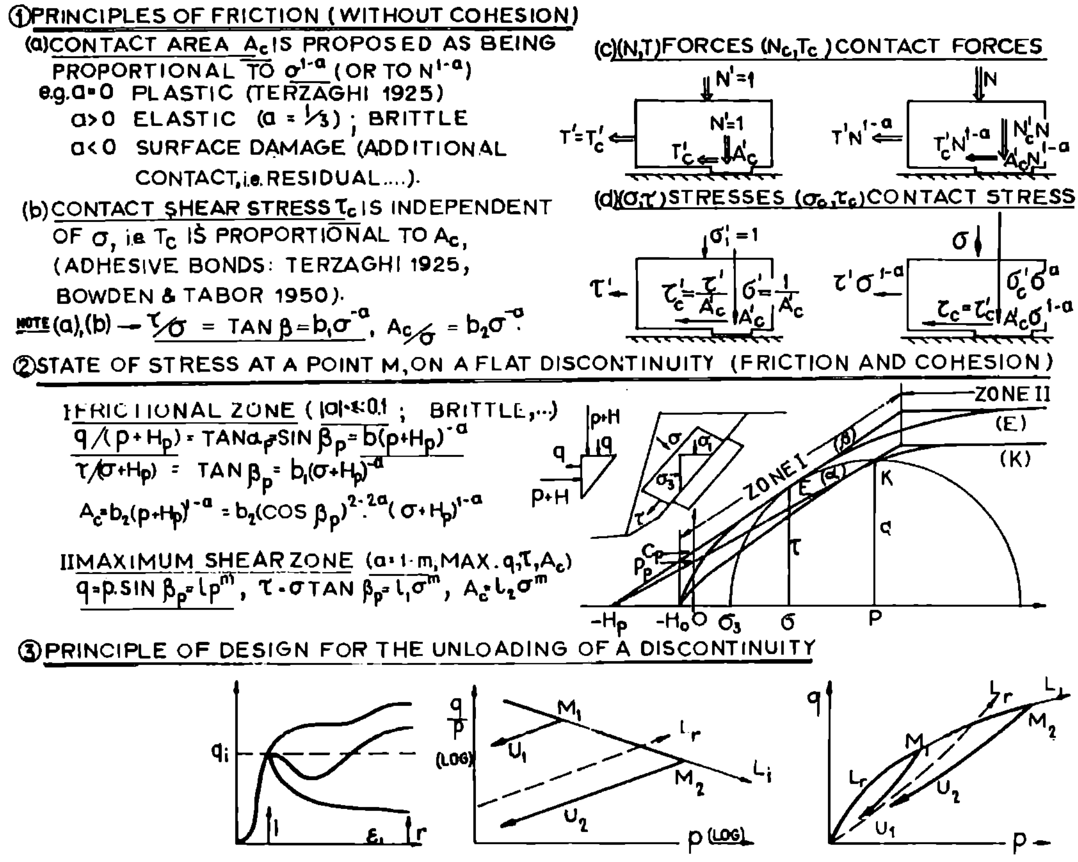

Fig 1 PRINCIPLE OF STRESS RELATIONSHIPS FOR A FLAT DISCONTINUITY DURING LOADING AND UNLOADING.

The state of the stresses at a point M, on a flat discontinuity is given by a Mohr circle on fig.1-2. When the pressure p increases, the above expression of the stress ratio (described also in fig 1-2) corresponds to an envelope (E) or to a curve (K) in the p-q diagram.

The curves (K) or (E) show two zones described in fig 1-2 as: 1) a frictional zone with 'a' much smaller than 0.1(brittle, elastic behaviour); 11)a zone of shear resistance slightly increasing or maximum (a of the order of 0.85).

2. Principle of determination of the initial and residual curves in the stress ratio diagram $\frac{q}{p}$ -p).

The proposed relationship for the stress ratio in terms of p gives straight lines in the double logarithmic diagram as illustrated by the central sketch of fig 1-3. This applies to the initial and residual values of the stress ratio and for any curve representing a given constant shear strain. The mechanism itself is confirmed by the experimental data. Fig.2,3 and 4 show the straight lines joining the test data found by G.H.Coulson (1971; the values have been tabulated on page 101 to 103).

3. Results of the experiments. The data mentioned illustrate the following results consistently obtained in fig 2,3, and 4.

a) the friction coefficients, initial or resi-

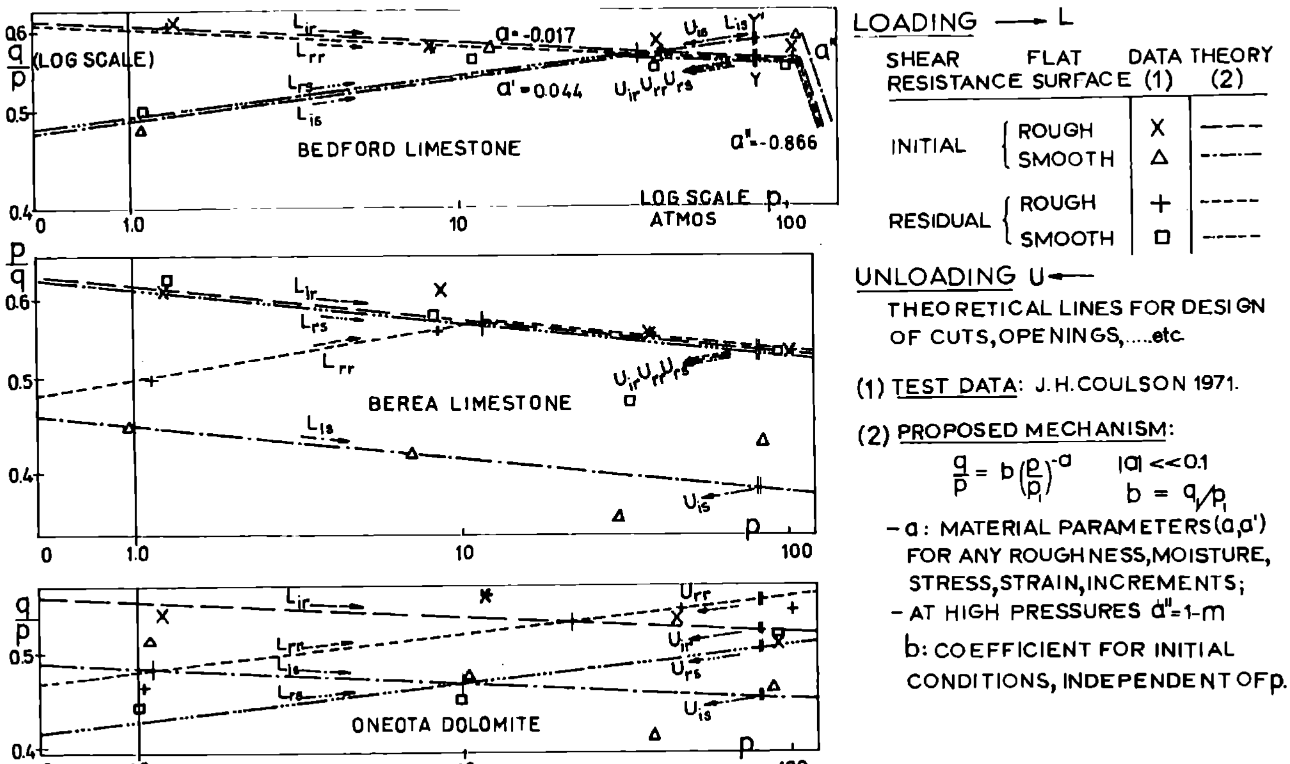

Fig2 STRESS RATIO DIAGRAM FOR FLAT DISCONTINUITIES IN SOFT, POROUS ROCK; PRINCIPLE OF DETERMINATION OF INITIAL AND RESIDUAL (OR CONSTANT SHEAR STRAIN) CURVES (L) FOR INCREASING p, (U-UNLOADING).

dual, may increase or decrease depending on the initial conditions of the surface and on the level of the pressure p. However it appears that only two slopes a and a' can be used, all the increases being parallel to each other (a' positive), and the decreases having the same slope a. The initial friction ratio is generally higher for a rough surface, but the residual coefficient may decrease in proportion of the surface roughness. In brief the figures 2 to 4 reveal general trends of behaviour but the stress ratios should be measured for the field conditions of the discontinuities.

b) at high pressures the shear resistance q increases only slightly and the ratio decreases with a slope of the order of 0.85 (see for instance fig 5).

c) the same friction coefficient, initial as well as residual, may vary with increasing pressure according to a slope a (or a'), but suddenly the slope may change to the other value a' (or a), for reasons not yet clearly understood. The general mechanism remains a decrease of the coefficient for a brittle or elastic behaviour, and an increase when the contact area becomes excessive by surface damage.

4. Principle of design for rock excavations or loading cycles.

a) the stress ratio at the initial conditions before unloading should be measured directly, since it may vary considerably with the past stress and surface history (see for instance fig 5a).

b) the unloading line should be measured, or assumed to take the slope a' (most conservative case).

c) the straight line of the $\frac{q}{p}$ -p diagram corresponds to a decreasing value of the residual angle of friction. For instance from 28° to 21.5° for line (Z) in figure 5a. The p-q diagram in fig 5a should be inaccurate to distinguish the decrease of the mobilised angle of resistance at low pressures. This mechanism is also illustrated in fig 1-3: The mobilised angle may eventually remain higher or lower than the residual angle classically obtained at large deformation but at a different level of stress p.

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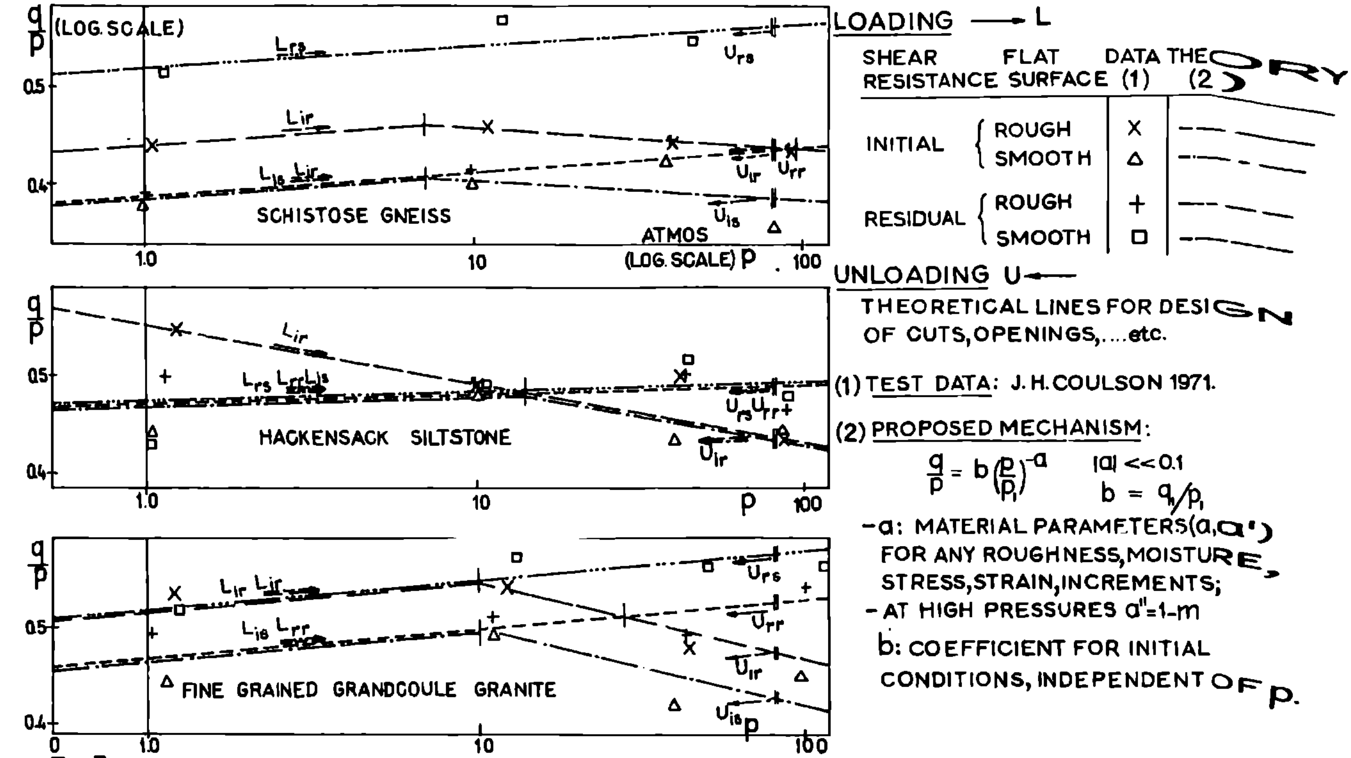

Fig.3 STRESS RATIO DIAGRAM FOR FLAT DISCONTINUITIES IN SLICKENSIDED, OR MICACEOUS, OR FINE GRAINED ROCK; PRINCIPLE OF DETERMINATION OF INITIAL AND RESIDUAL CURVES (L) FOR INCREASING p; (U = UNLOADING).

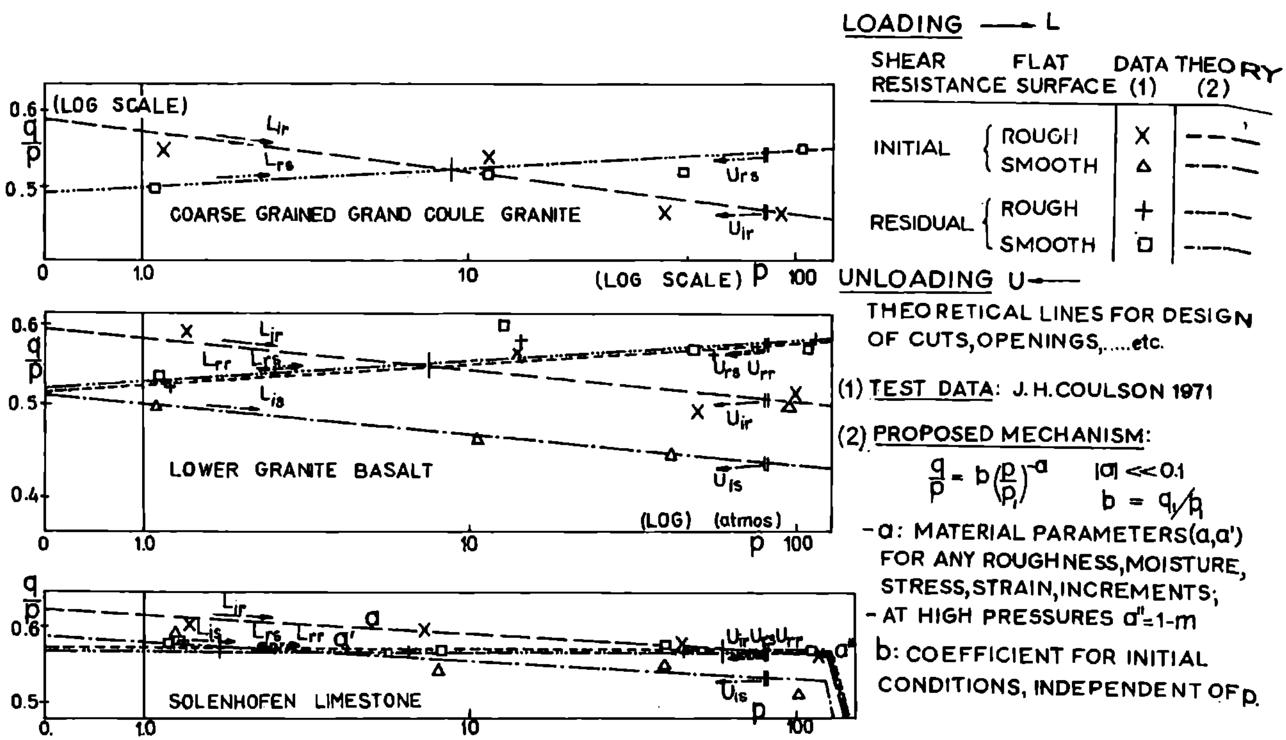

Fig.4 STRESS RATIO DIAGRAM FOR FLAT DISCONTINUITIES IN HARD ROCK; PRINCIPLE OF DETERMINATION OF INITIAL AND RESIDUAL (OR CONSTANT SHEAR STRAIN) CURVES (L) FOR INCREASING p; (U = ULOADING).

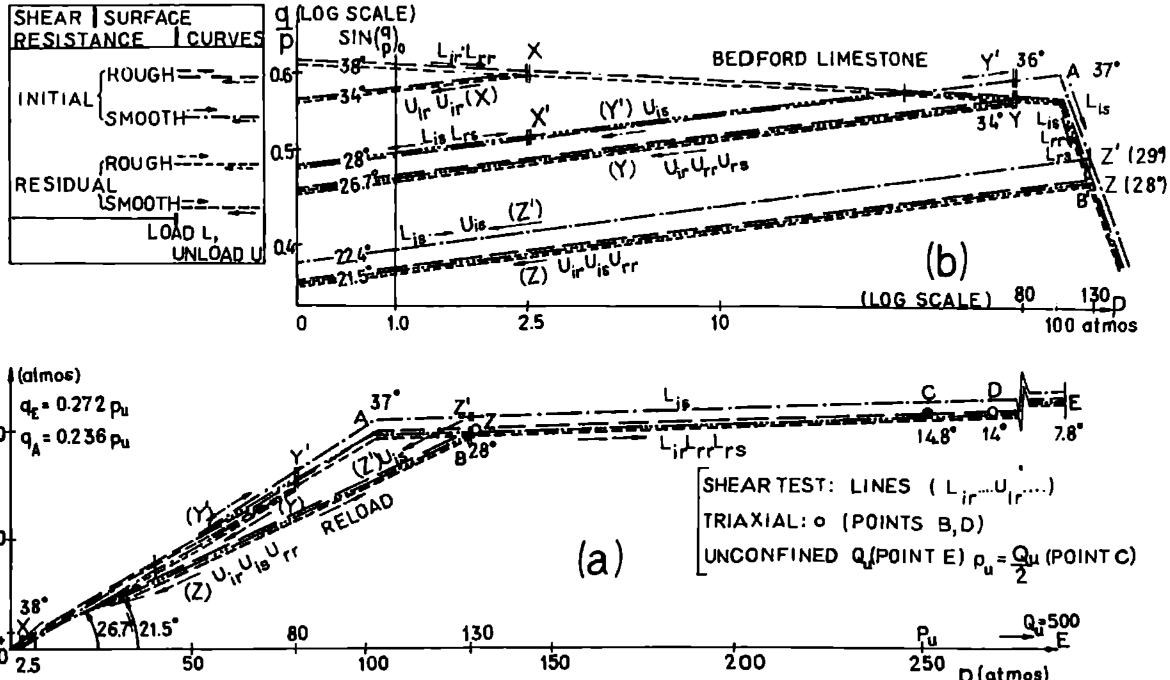

Fig.5 PRINCIPLE OF DESIGN FOR ROCK EXCAVATIONS. OR LOADING CYCLES. (a) INACCURACY OF q/p IN THE p-q DIAGRAM (b) DETERMINATION OF THE STRESS RATIO q/p FOR FLAT DISCONTINUITIES, FOR GIVEN INITIAL CONDITIONS OF ROUGHNESS, STRESS, STRAIN (eg INITIAL, RESIDUAL).

ON THE PROBLEM CONCERNING THE CALCULATIONS OF LANDSLIDE STABILITY Rogozina Z.I.(USSR)

At present there is a great number of methods to calculate the stability of landslides and artificial slopes. Yet, the results of calculations are very often far from being valid for practical purposes so that some experts deny both the practical significance of shear strength factors calculated under laboratory conditions and the expediency of such calculations.

The discrepancy of the results of landslide stability calculations based on the existing methods of calculation is accounted mainly for the fact that the conditions and particularly a force (hydrostatic and hydrodynamic pressures) of ground waters affecting the stability of landslides have not been properly taken into account in the course of calculations.

During the recent years the author of the present article has systematized and analysed the data on the matter in question, has made field and laboratory studies of the two areas of the Volga landslides in the town of Ulyanovsk and has made some landslide calculations to reveal if various effects of undersurface (ground) waters (additional wetting, hydrostatic and hydrodynamic pressures) on the sliding strata could be of any significance for the stability of landslides. The results of the studies have made it possible to come to the following conclusions as regards the problem of the stability of landslides and the steps to be taken to eliminate the effect of landslides:

1. It has often been overestimated (exaggerated) that the overwetting (increase of moisture content) of the rocks in the zone of shear affects the stability of a landslide. In many cases the landslides developed or ceased to develop without the change of moisture content of the soils in the zone of shear. In the given case a possibility of additional moistening of the sliding strata plays an insignificant role (the depth of moistening is 1.0 - 1.5 m) whereas the hydrostatic and hydrodynamic pressures are often of greater and even decisive importance as regards the landslides of the superficial nature.

2. In case the sliding rock has a free strata of ground waters the effect of the stability of the superficial strata should be taken into account together with the suspension effect as well as the seepage (hydrodynamic) pressure. If in the zone of contact there is a pressure horizon the back pressure occurring there should be taken into account when the values of normal stresses affecting the contact surface are determined.

3. The ground waters run in the strata of landslides along the cracks and voids. In fact, the original gradient makes it impossible for the ground waters to run through the pores of a clayey soil. Nevertheless, the analysis of the data of the calculations has shown that it is necessary take into account the bulk density of the soil affected by the original gradient when evaluating the value of hydrodynamic pressure.

4. The characteristics of the shear strength of the soils obtained under laboratory conditions correspond sufficiently to the actual conditions. The results of the calculations cannot be valid for practical purposes because the effect of the ground water force on the stability of landslides which is associated with the geological and hydrogeological conditions of landslides has not been properly taken into account.

5. It can be noted that the lowering of a ground water level (or limiting its rise) in the strata of the upper deposits is highly effective and is one of the main measures to prevent landslides and to secure the stability of the landslides under consideration (the superficial landslides).

6. There is a similiarity of the final results of the calculations made in accordance with the various methods of calculation (the method of horizontal forces worked out by Maslov - Berer, and the method of inclined forces worked out by Tchugaev R.R.) to evaluate the stability of the superficial sliding strata by taking a certain surface of sliding under natural conditions.

7. It is to be noted that a possibility has been revealed to evaluate the stability of slopes by calculation methods using the soil characteristics obtained as a result of the conventional laboratory studies but with due regard for the hydrostatic and hydrodynamic effect of the ground and underground waters on landslides.

The measures taken to prevent landslides in both of the said areas in order to lower the ground water level in the sliding strata, and consisting in the regulation of the run-off of the atmospheric as well as the snow water and the drainage of the underground waters have proved the validity of the forecast that the lowering of the level of the said waters affects the stability of landslides. In fact, no landslides have been observed.

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FORECASTING AND RESULT IN CASE OF LANDSLIDE AT TAKABAYAMA. Michitaka Saito and Gōji Yamada (Japan)

General description.
The forecasting method of slope failure, based on creep-rupture characteristics of soil was published in series in the previous Proceedings of the Conferences. Recently such a noticeable record of a landslide was obtained, that the failure time estimated with this method was announced officially in advance on the previous day of failure, and it resulted in good coincidence with the actual failure time. In this paper authors describe an outline of the case as a positive proof for the reliability of this method.

In January, 1970, a landslide occurred on the Iiyama Line of the Japanese National Railways, and collapsed a half of Takabayama Tunnel, 187 m in total length. Slumped debris of the slide, amounted to 130,000 m^3 covered a good length of local highway and a part of Shinano River, the longest in Japan, and it took 10 months before railway traffic was resumed through a new tunnel, constructed parallel to the old one. The range of the landslide is shown in Fig. 1.

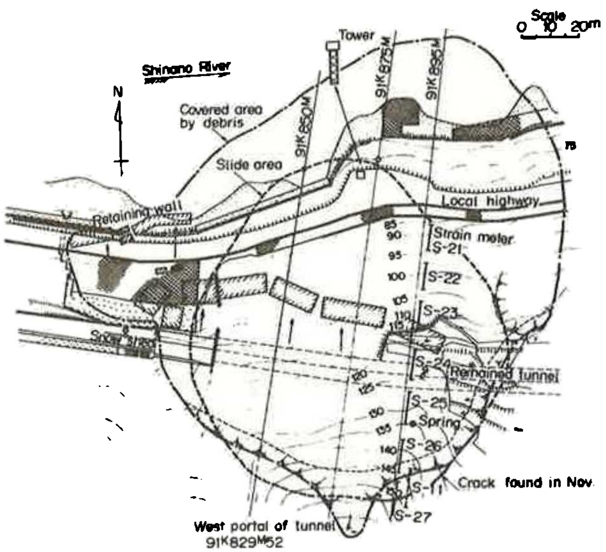

Fig. 1. The range of landslide at Takabayama

Geological formation of the site is sedimentary rock of Miocene to Pliocene Period, consisting of alternation of mudstone and sandstone. The stratum is slightly folded and shows syncline structure along Shinano River and anticline along ridge parallel to the river.

Progress of slide.
Some deformation of the tunnel for fear of landslide had been noticed from its construction on. In April, 1969, unusual distortion was found; crack opening on the slope above the west portal of the tunnel increased continuously, and the west portal itself was observed to displace toward the river, but those remarkable deformations were limited to the neibourhood of the portal. Then, the National Railways switched over to extremely careful observation, by dispatching six staffs every day since April, and with seven watching staffs to stay near by the site throughout the day since July.

In August, distortion became to increase remarkably, due to heavy rainfall, 618 mm in total in 20 days; e. g. opening of crack in concrete of the portal increased 17 mm in a day. In order to avoid unexpected accidents, railway traffic operation was suspended on 11th of August, and such countermeasure works were carried out, as drainage by horizontal drilling, construction of retaining walls along the river side to stabilize steep slope above the tunnel, removing heavily cracked portion of the tunnel, 20 m long from the west portal, and construction of snow shed, 40 m long, adjacent to the remained tunnel.

In September, railway traffic operation was resumed. Since about this time, many cracks appeared at the central part of the tunnel. Drainage by radial drilling in horizontal or obliquely upward direction from the inside of the tunnel was tried, but remarkable effect could not be obtained. Displacement of the tunnel was still in progress with nearly constant rate of 1.0-1.5 mm per day, according to transit surveying.

On 7th of November, a long tension crack was found on the slope above the central part of the tunnel, 40 m upward from the center line. Then, extensive investigation for landslide movement was performed over the area. Relative displacement measured by strain meter across this crack showed 3-6 mm per day on and after 25th of November.

On 14th and 15th of December, heavy snowfall was experienced, and almost all measuring devices on the ground surface failed in use. So, strain meters were replaced with those of remote-recording type and buried in the ground. Observations were resumed on 31st of December, 1969. At this time displacement rate attained to 20 mm per day. The measuring devices continued to work normally and furnished useful data until just before occurrence of landslide.

With the approach of failure, cameras and movies were arranged at the opposite site of the slide, accompanied by lightening the site with illuminations. A lot of picture were taken at the moment of failure, and the results were fairly satisfactory though in snowstorm.

Site investigation and observation.
Subsurface investigation by drilling was first carried out at three spots near the west portal in 1966. When distortion became remarkable in August, 1969, drillings at 10 spots, selected so as to form three sections, were carried out, and some of them were inserted with plastic pipes attached with wire strain gauges. Measurements of bending strain of pipes by these strain gauges were fairly effective to decide the location of slip surface. One of typical sections of slope and estimated slip surface is shown in Fig. 2.

Displacements of the tunnel toward the river were surveyed with transits, by measuring discrepancy of stake head from the line of vision. Subsidences were also measured with levels. A typical progress of the movements is shown in

Fig. 3.

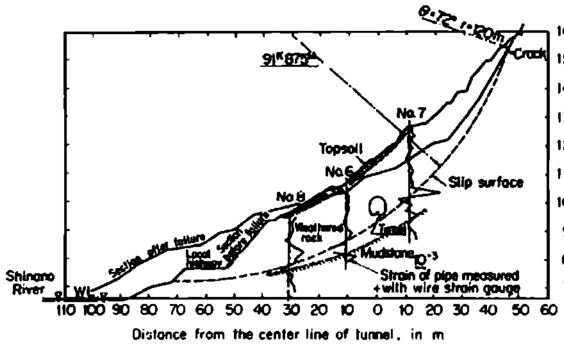

Fig. 2. Section of landslide at Takabayama.

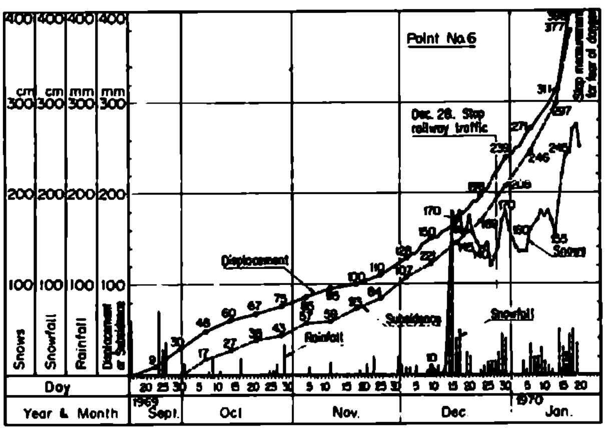

Fig. 3. Displacements and subsidence inside of the tunnel.

Relative displacements of slope were measured by strain meters using invar wire stretched across tension crack or along slope line. The devices were already used around the west portal since 1966, but it was after Spring of 1969, that the devices were used concentratedly. After remarkable distortion was found above the central part of the tunnel devices were reset so as to fit to new cracks. Those devices, exchanged with remote-recording type after heavy snowfall in December, were also reset near the same locations as before. The locations of the devices are shown in Fig. 1, with symbols from S-21 to S-27. The records of relative displacements with those devices are shown in Fig. 4, in which symbol S-11 is the former one. It can be seen clearly that no remarkable increase in strain is obtained except for S-27, which was set across tension crack.

Forecasting of failure time and the result.

Forecasting of the time of slope failure was carried out using the creep curve of S-27 shown in Fig. 4. As it was rather difficult to fix the boundary of creep range, the very time of failure was estimated by both methods in the secondary and the tertiary creep range.

As for the former, transient strain rate was adopted, and failure time was estimated by the following formula:

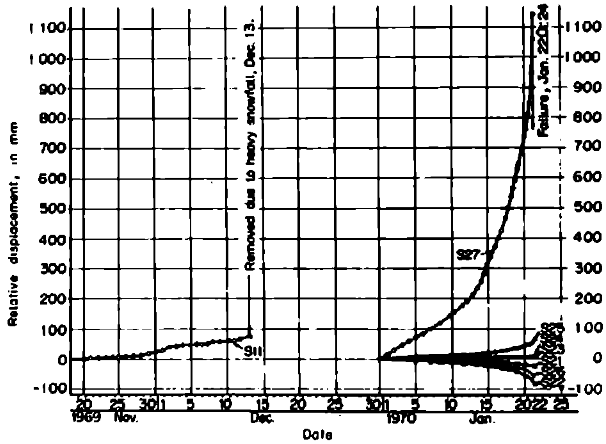

Fig. 4. Relative displacements measured by strain meters from Nov. 1969 to the time of failure on Jan. 22nd, 1970.

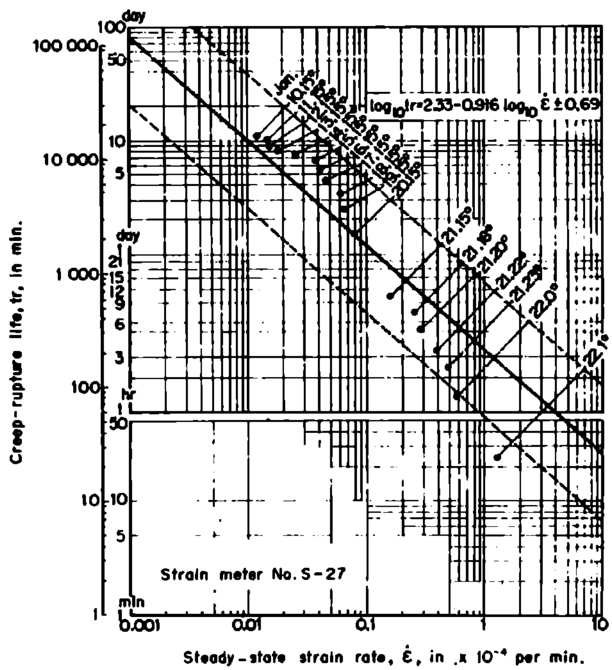

Fig. 5. Forecasting based on steady-state strain rate in the secondary creep range.

$$\log_{10} t_r = 2.33 - 0.916 \times \log_{10} \dot{\varepsilon} \pm 0.59,$$

where t_r : creep rupture life in min.

$\dot{\varepsilon}$: transient strain rate in 10^{-4} per min.

Daily estimation of failure time by this method is shown in Fig. 5. In the previous day of failure, hourly estimation was tried. From these results it may be said that the estimation was fairly in good coincidence with the actual time, and it was rather in safe side until two days before.

As for the latter, estimation of failure time in the tertiary creep range is done by substituting the creep curve to a logarithmic one. This method has the advantage that, the nearer failure comes, the more reliable estimation is obtained, and moreover, has additional merit to enable to adopt graphical analysis. The result of graphical analysis is shown in Fig. 6. It can be seen that the estimation shows exceedingly good convergence to the fatal line.

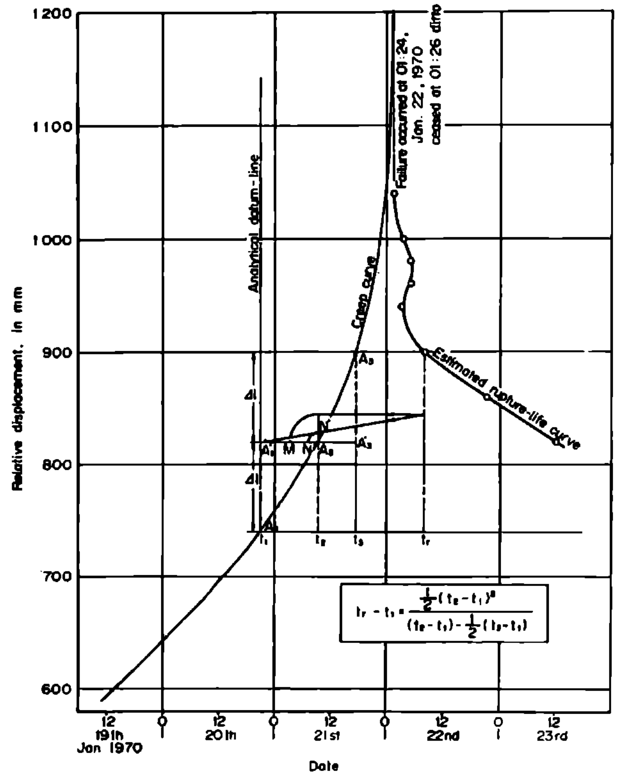

Fig. 6. Forecasting by means of substituted logarithmic curve in the tertiary creep range.

Time after time, estimation was tried repeatedly, and on 20th of January, it seemed that serious consequence would be impending and inevitable. The unprecedent announcement was pronounced by the head office and the district bureau of the National Railways at 5 p. m. of 21st that the slope would fail at the coming midnight or before dawn. The estimation was revised every hour, and seemed to converge to the dead of night. Estimation of failure time on this stage is shown in Table I. The final announcement was made at the midnight that the failure would occur at 1:30 a. m. on 22nd, according to the analysis in the tertiary creep range.

After all, the slope above Takabayama Tunnel began to fail at 1:24 a. m. on 22nd, accompanied with terrific roaring, and ceased to move after 2 minutes. The difference between estimated and actual failure time is 6 minutes; therefore, the estimation of failure time can be said to come true almost exactly.

Table I. Estimated failure time.

Estimated date	Strain rate $\times 10^{-4}$ per min	Estimated failure time	
		Secondary creep range	Tertiary creep range
Jan. 21st. 20:00	0.29	21st. 23:00 - 22nd. 07:00	22nd. 03:00
21:00	0.31	21st. 23:00 - 22nd. 07:00	22nd. 05:00
22:00	0.39	22nd. 00:00 - 22nd. 06:30	22nd. 05:00
23:00	0.49	22nd. 00:45 - 22nd. 05:40	22nd. 03:30
22nd. 00:00	0.69	22nd. 01:17 - 22nd. 04:50	$\boxed{\text{22nd. 01:30}}$*

* $\boxed{\text{22nd 01:30}}$ is the final announcement for forecasting.

After the incident, a personnel in charge of observation and estimation of slope failure told us that he could not believe until the moment that the slope would really fail soon, because the shape of the slope did not show any change as before, though the strain meter was recording rapid progress in relative displacement.

Conclusion.

In case of landslide at Takabayama, estimated failure time came very closely to the actual time, and the difference was no more than 6 minutes. Although this surprising outcome resulted principally from extremely cautious attention and unyielding spirit of the personnel in charge of lookout, it is not impossible to deny some good fortune. At the present situation, it can be said that the reliability of forecasting is not beyound one day, or may be said to be within several hours.

Acknowledgement.

All observation data used in this report were offered by the Japanese National Railways. Authors wish to express their appreciation to the National Railways, especially to Dr. S. Kobashi, in charge of observation and estimation, research member of Railway Technical Research Institute, for generous permission to use those valuable data.

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THE CALCULATION OF FLOW SLIDES IN TOP-SOIL FORMATIONS. K.Sh.Shadunz (USSR)

As it was mentioned by a number of investigators the flow slides of the top-soil formations (Howe,1907.Meyerhof,1957) are widely spread in many countries all over the world. The Black Sea coast of the Caucasus, Moldavia, the Ukraine and the Volga region are the most frequent areas of occurrence in the USSR(Zolotarev,1956). According to the nature of the movement and strength soil indexes the flow slides occupy an intermediate position between the clay flows and the deep slope creeps. The soils in them may be regarded as the rheological bodies described by Bingham-Shvedov' equation with internal friction proportionate to the normal pressure being considered(Vyalov, 1959).

$$\tau = \tau_{lim} + \eta \frac{dV}{dy} + \sigma \tan\varphi$$

where τ_{lim} - ultimate shear stress, η - plastic viscosity coefficient, $\frac{dV}{dy}$ - velocity gradient, σ - normal pressure, φ - angle of internal friction with "continuous"shear. The same equation for the determination of maximum soil displacement rates were arrived at by Maslov(1968)for the deep slope creeps, Shischenko and Yesman(1966)for the flow of the sticky-plastic liquids, Tevsadze(1966)for the earth flows on the basis of the above dependence. The equations were deduced for the wide flows (plane problem).

$$V_{max} = \frac{\gamma}{\eta}\left[H(H-H_s) - \frac{(H-H_s)^2}{2}\right](\sin\alpha - \cos\alpha\tan\varphi) - \frac{\tau_{lim}}{\eta}(H-H_s)$$

where H - the depth of displacement layer, the retaining structure zone being part of it, H_s is a stiff layer, γ is a bulk specific gravity, α is an incidence to the surface of displacement. For the cases of yield and plastic yield consistence Maslov (1968) gives the following formula:

$$V_{max} = \mu \frac{\gamma}{\eta} J_x H^2,$$

where $\mu = \frac{3}{8}$, J_x - hydraulic gradient of section x of the clay flow length. In the both formulas the velocity is proportionate to the square of the layer thickness. For shallow slides-flows of surface formations a different expression must be introduced. Taking into account the fact that flow slides have mostly elongated shape with slightly changing width and depth all over the length,it is possible to determine the value of the displacing tangent stress for the conditions of the volumetric problem as follows:

$$\tau = \gamma R J$$

where γ is the unit weight, R is a hydraulic radius of the flow cross section, $J = \sin\alpha$ is the dip of free surface. The ultimate condition for the beginning of the movement will be:

$$\tau - (\tau_{lim} + \sigma\tan\varphi) \geq 0$$

from which we can determine the minimum size of the flow section which can move or the size of the "structural" zone of the flow:

$$R_s = \frac{1}{\gamma J}(\tau_{lim} + \sigma\tan\varphi)$$

The maximum rate at the surface of the gradient layer or the rate of movement of the structural zone will be expressed by the formula:

$$V_{max} = \frac{1}{\eta}\left[0.5\gamma(J - \cos\alpha\tan\varphi)(R^2 - R_s^2) - (\tau_{lim} - qJ)(R - R_s)\right]$$

Verification of the formula obtained was done by the calculation and simulation of the flow slide situated near the town of Adler. During the period of exploration its length was approximately 180 m, width - from 15 to 30 m, the surface area - 4500m^2. The average dip J was 0,28. The depth of the sliding layer varied from 0,7 to 2 m. Under natural moisture content the average soil strength characteristics of the slide body proved to be: W=29%, c=0 06 kg/cm^2, φ= 4°; the bulk specific gravity 1,83 t/m^3, consistence limits W_L=47%, W_p=24%. Viscosity coefficient of the crushed soil from the tongue slide was determined by the viscosimeter PB-8 and model tests. The flow slide simulation was done in the glass tray. The likeness of the experimental and natural conditions was gained by loading the surface and increasing the soil moisture content as compared with natural conditions. There were determined the scales of viscosity, density and displacement rate. η determined by the simulation was put into the formula proposed by the author. According to the calculation the rate of rock slide displacement with the average dip 0,28 was approximately 10 cm within 24 hours. The rate of the movement observed during rainy periods was of the order of 20 cm. Thus, formula 7 gives sufficiently good approximation for natural measurements.

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A SOIL MECHANICS AND CONSTRUCTION PROBLEM IN MEXICO CITY CLAY. B. Simpser (Mexico)

Introduction. During the construction of a foundation for a building in Mexico City Clay, when the excavation was well advanced and the stability of the excavation slopes was very critical, a 5.5 m high berm was urgently placed to prevent slides of the slope and the failure of an adjacent structure. At the same time, an analysis was requested, to determine a constructive method to excavate safely and economically under the berm that was specifically placed there to prevent slides, till −5.75 m in order to pour the foundation slab, and then, to further excavate 3 m more in trenches, to a depth of −8.75 m, to pour the foundation ribs and pile caps.

The soil is a sensitive clay (CH), its average water content is 365% with a maximum of 520%. The average unconfined compressive strength is 0.40 Kg/cm2. The water table is erratic in a reasonably homogeneous soil.

Three dimentional effect of excavation boundaries on slope stability. This analysis, in top of the assumptions made by the Swedish Method (the Mohr-Coulomb law is valid, a constant shear stress acts at the same time in all the sliding surface, between the lateral boundaries there is a plane strain state and the failure surface is a circular cylinder), is based on the additional assumption, extrapolated from the circular failure surface, that the circular cylindric surface of the Swedish method has a spherical segment at each end of the slope, and that the centroidal axis of the circular cylindric failure surface o' (Fig. 1) is unchanged by the addition of the spherical segments.

The area of the spherical segments, varies generally between 0.125 and 0.25 of the area of a sphere, and is a function of the excavation geometry.

As the soil tends to rotate about an axis normal to the paper through point o (Fig. 1), all the shear forces opposing this tendency, on the surface of the spherical segments, are tangent to the spherical segments and perpendicular to the axis of rotation, therefore, the additional resisting moment due to one spherical segment when the soil tries to slide, can be estimated with the following approximate relation:

$$M = Acr$$

where M is the additional resisting moment in Ton x m; A is the area of a spherical segment, computed from: $f4/3\pi R^3$, in m2; f is a nondimentional factor depending on the geometry; R is the radius of the critical failure circle, in m; c is the cohesion in Ton/m2; r is the segment o−o' (Fig. 1) in m, the approximate average value for the moment arm for the shear forces acting on the surface of the spherical segments.

The restraining effect of the boundary decreases gradually along the length of the slope, as the distance from the boundary increases. The effect of the boundary is felt on the slope till a distance of approximately 1/2 cb (Fig. 1), measured along the length of the slope, which around 9 m for the case studied.

Based on the previous assumptions and using the stability number defined by Taylor(1937), the section B of the slope in study which is 3 m away from the toe of the southern slope, had an increase in its slope stability safety factor, already corrected to take into account the effect of tension cracks (Terzaghi, 1943), of approximately 52% (from 0.85 to 1.35) when the live loads acting on the slope were considered. In section D, which is 6 m away from the toe of the northern slope, the increment in its slopestability safety factor was of approximately 35% (from 0.86 to 1.16), which is a smaller increment than for section B because it is separated from the lateral boundary of the slope by a larger distance than section B.

Influence of a pile on the stability of a slope. Piles were already driven, and one row intersected the failure surface that would exist if the piles would not exist. It was obtained theoretically that the direct influence exerted by the pile on the slope stability, is felt in a slope length of approximately two pile widths, measured from the pile centroid. Further away, the pile influence is transmited to the adyacent soil by a three dimentional stress field.

Assuming tentatively that the failure surface has not been altered by the presence of the pile, and assuming that the part of the pile intersected by the failure surface is deep enough in the soil so that it can act as if it were fixed, an increment of approximately 18% in the safety factor of the stability of the slope was obtained.

By allowing the failure surface to change due to the presence of the piles, a new critical cylindric failure surface was obtained with a higher safety factor than the one obtained from the critical failure surface when the pile did not exist, but 50% smaller than the safety factor obtained assuming that the failure surface without piles was not modified by the presence of a pile.

From the previous results, it can be said that the mechanism by which the pile increases the safety factor in the particular case studied, is by modifying the circular cylindric failure surface in the vicinity of a pile, so that the failure surface is tangent but does not intersect it.

For the case studied, the pile increased the safety factor from 8% to 10% in the vertical plane that contains the pile and cuts the slope transversely. This increase, decreases along the length of the slope as the distance from the pile increases.

Time effect. As the safety factor of the slope stability is directly proportional to the cohesion, and the cohesion changes with the time, the safety factor is a function of time. By the use of the coefficient of rheological decrease, it can be estimated the approximate reduction of cohesion in percentage, at discrete times, using logarithmic interpolation to estimate intermediate values.

Safety factor surface. For the case in study, the safety factor for the stability of a section in a slope is a function of its distance from the lateral boundaries, its distance from piles, and the time elapsed. (Fig. 2).

FIG. 1

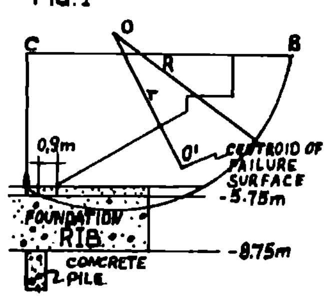
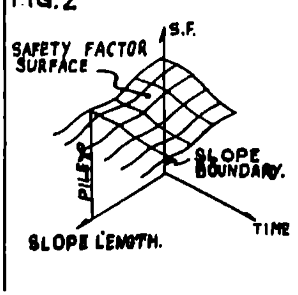


FIG. 2



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THE LESUEUR LANDSLIDE, A FAILURE IN UPPER CRETACEOUS CLAY SHALE. S. Thomson (Canada)

Abstract

The development lands associated with river valleys brings the question of valley wall stability into sharp focus.

In late April of 1963, a distinct but small crack was observed to arc across the lawn of the Lesueur property. As the summer advanced the crack widened and a small scarp formed. On September 3, 1963 a major slide occurred leaving a scarp 22 feet high and a part of the house cantilevered over the edge.

Site surveys were carried out to establish the soil profile and piezometric levels. Soil parameters were established in a laboratory program using triaxial and direct shear equipment. The stratigraphic profile consists of the fine glacial lake sands, till, terrace sands and gravel overlying clay shales and bentonitic clay shales of Upper Cretaceous age. The preslide slope angle was 26° and the valley wall height was about 100 feet.

The analyses were in terms of effective stresses and used a non-circular failure surface comprised of a steeply dipping section and an essentially horizontal section. The results of the analyses indicated a "first time" failure but that the overall strength that was mobilized was less than full peak strength. It appears that the peak angles of shearing resistance are mobilized but the cohesion developed has a value between peak and residual.

The slope failure is postulated as resulting from removal of toe load due to lateral erosion of the river.

Introduction

The Lesueur Landslide is located on the outside of a bend of the North Saskatchewan River about 4 miles northeast of the eastern outskirts of the City of Edmonton, Alberta, Canada, Figure 1.

The river banks are marginally stable for many miles including those within the city (Hardy 1957; Thomson 1970). The rapid population growth is exerting pressure for the development of lands bordering the valley, the valley walls, and the terraces close to water level. This pressure, coupled with increasing land values, brings the question of valley wall stability into sharp economic focus. Assessment of the stability and recommendations for development or solutions of problems posed by development schemes is clearly the task of the geotechnical engineer and the engineering geologist.

One of the tools useful in landslide analysis is case histories. It is the purpose of this paper to present an account of one landslide and its analysis.

Geologic Setting

The bedrock of the area is Upper Cretaceous in age and has been designated as the Edmonton Formation (Ower, 1962). It is a brackish to freshwater deposit and consists of rather poorly indurated claystone, mudstone, siltsone, and sandstone strata interbedded. The dip is southwesterly at 20 feet per mile. The general depositional environment is also apparent from the presence of coal seams that range from a few inches to several feet thick. Bentonite is a common admixture in some of the strata and forms an occasional pure seam several inches to one foot thick. The presence of bentonite, which is derived from altered volcanic ash, seriously affects the engineering characteristics of the strata.

At the close of the Cretaceous period, sediment deposition slowly changed from a deltaic environment to terrestrial. The later deposits are referred to as the Paskapoo Formation. During the Oligocene, regional uplift occurred and subaerial erosion began. It has been estimated that some 2000 feet of sediment has been eroded from a broad area of the province. All of the Paskapoo and the upper part of the Edmonton Formation have been removed.

The preglacial topography indicated by bedrock contours (Carlson, 1967) was gently undulating and the drainage pattern was dendritic. The interesting feature of the bedrock topography is a preglacial bedrock ridge trending northeast immediately under the Lesueur landslide site. One might expect this subsurface configuration to affect groundwater levels in the immediate vicinity of the landslide.

The only recorded advance of ice into the Edmonton area during the Pleistocene was of Wisconsin age (Bayrock and Hughes, 1962). During the early part of this period the rivers of the province developed a series of terraces and deposited a quartzitic sand and gravel on them. These deposits are called the Saskatchewan Sands and Gravels and rest unconformably on the bedrock. Their lithology and the complete absence of highly metamorphic or igneous rocks denotes their original origin to be the Rocky Mountains to the west.

The Wisconsin Ice advance is recorded by two till sheets which may be differentiated in outcrop but are difficult to separate by borehole methods (Westgate, 1969). From a geotechnical point of view they are similar enough to be considered as one unit. Both tills are dense in the undisturbed state but contain vertical joints. There is the usual wide range of grain sizes with the clay sizes containing a high proportion of montmorillonite. The till contains highly metamorphic and igneous rocks and pebbles that are of Shield origin. Outwash gravels that are occasionally found between the till sheets or associated with them are easily differentiated from the older Saskatchewan sands and gravels on the basis of pebble lithology.

The regional slope of the land is northeasterly and hence glacial advance from the same direction was upslope. During the waning phases of glacier retreat, many temporary ice marginal lakes were formed. One such lake, named Glacial Lake Edmonton (Bayrock and Hughes, 1962), inundated the general Edmonton area. It has been demonstrated that this short lived lake was ice marginal in many areas. The northern shore of this lake was about a mile or so north of the Lesueur landslide site. The lake sediments that cap the valley walls in the slide area are, therefore, silty sands with some clay inclusions that may have been ice rafted.

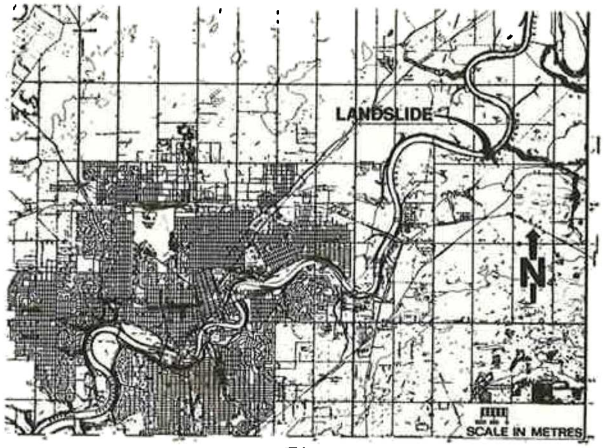

Fig.I

The terrace immediately east of the landslide site, Figure 2, is about 25 feet above river level and has been dated at 6500 years by radio carbon techniques and the Mazama Ash (Westgate, 1969). The slopes of the valley wall on the eastern side of this terrace and about 2000 feet south of the landslide site show evidence of very old landslides. These slopes were surveyed and slope angles in the range of 14 to 16 degrees with the horizontal were observed. It is inferred that these slopes are in the order of 6500 years old and that the old landslides were associated with bank erosion when the river was on the terrace. At this time, a short reach of the river was flowing north off the terrace. It is pertinent to note that downward erosion of the river has only been about 25 feet in the past 6500 years.

Upstream from the general area of this terrace, the North Saskatchewan River channel has been eroded entirely in post-glacial time. The banks are steep and active erosion is taking place at present. Downstream, the valley widens abruptly and the river is flowing in a broad preglacial valley. The present course of the river in the vicinity of the Lesueur slide has swung from its old northerly direction to its present roughly east and north direction in the past 6500 years. The movement of the river in a southeasterly direction is evident from the meander scrolls, visible on air photos, opposite the slide area. One is not surprised to find landslides clearly discernable on the right bank for some distance downstream of the slide.

Landslide Observations Prior To and During Failure

About 1960, the older original Lesueur house was extended eastward by the addition of a large room. The general construction was a concrete block basement wall resting on simple spread footings. Above ground the house was a single storey finished with wood siding.

In late April of 1963, a distinct but small crack and a slight downwarping of the ground surface was noticed through the remnants of the snow cover. The crack started on the eastern edge of the lawn arced across it and appeared to stop at the basement wall about 10 feet from the north corner of the house. No cracks were observed in the concrete blocks or on the inside of the basement wall. The inside walls had been finished, however, with quarter inch plywood sheets nailed to studs which were, in turn, fastened to the wall. About mid May, the crack was clearly discernable having widened to two or three tenths of an inch. There was still no sign of cracking in the house wall. No sharp vertical displacement was apparent at the crack and the downwarping did not appear to have increased. By mid July, a distinct scarp had developed which by mid August had increased to 12 to 14 inches high. The situation continued to deteriorate slowly and on September 3, 1963 at 10 a.m. the scarp had increased to 2 feet. By 8 p.m. the same day it was some 6 feet high. At 8 a.m. the following day, September 4, the scarp was 22 feet high and a corner of the house was cantilevered over the edge, Figures 3 and 4. There were no signs of distress in the basement wall or floor other than a minor spalling off of a thin dressing mortar that had, presumably, been originally applied below the ground line as a waterproofing. From the original valley wall to the scarp the maximum distance was some 50 feet. The slide was 170 feet wide at the top but fanned out to nearly 1000 feet at river level. At the toe, the river bank averaged 10 feet above water level. During the day of failure, material at the toe was observed to move out along a practically horizontal plane and, after a small amount of cantilevering, chunks dropped off into the river where they accumulated. There was no apparent upward movement at the toe. As toe debris accumulated, cracks opened up parallel to the river bank and a few feet back from it. The next day, the toe area was a jumble of blocks and soft ground with some fallen trees to add to the disorder. The planar movement observed during failure was not in evidence.

Southward from the river, a band of small spruce and birch trees were tipped uphill. The angle of tip was not measured but appeared to be remarkably uniform, that is the trees were largely parallel in their tipped state. One was given the impression that a rectangle in a vertical plane had been pushed into

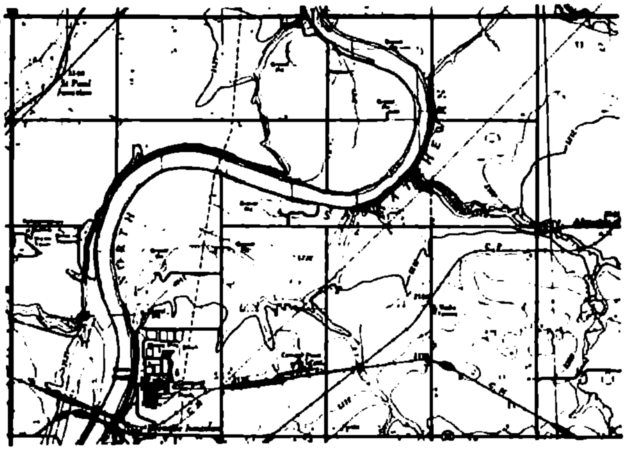

Fig.2

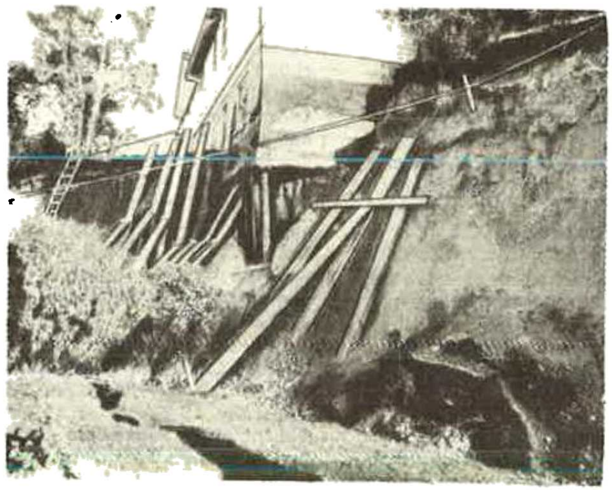

Fig.3

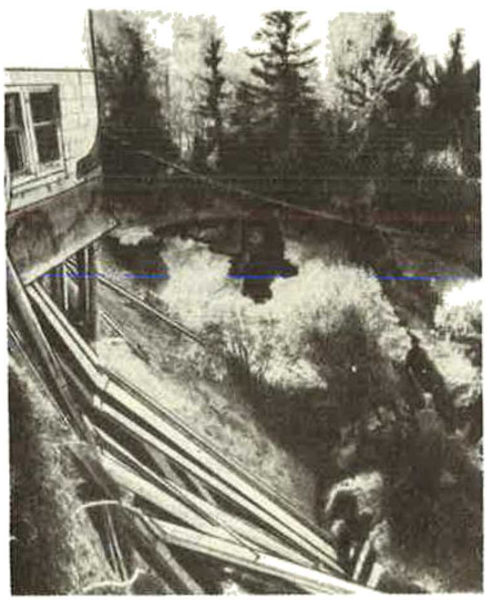

Fig.4

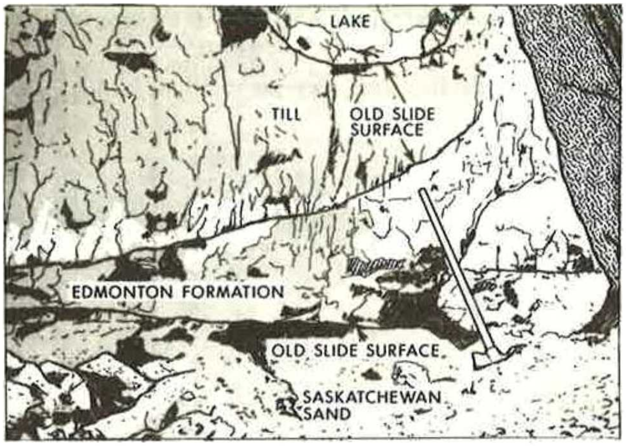

Fig.5

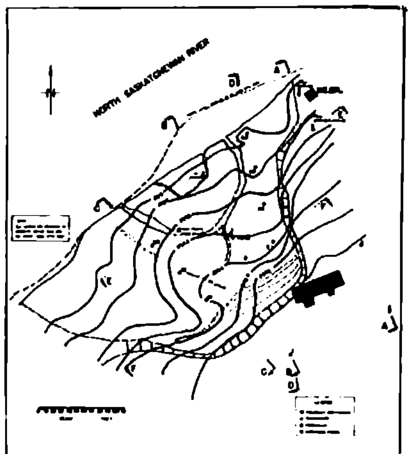

Fig.6

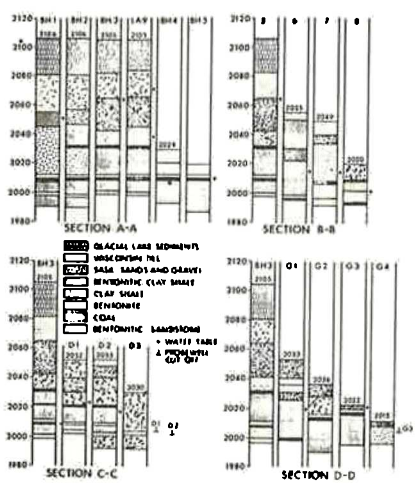

Fig.7

a rhombohedral shape by movement along the lower side. The uphill side of this band of trees terminated abruptly at a tension crack and a small scarp. Up the slope to the top of the slide the trees appeared to have undergone translation only and were not visibly tipped.

At the scarp, a series of cracks parallel to the scarp opened up across the part of the lawn involved in the slide. As the failure progressed the cracks widened and the blocks bounded by the fissures dropped downward toward the scarp and formed a series of graben-like features. These are shown on Figure 4.

On the east flank of the slide, a face was exposed by the movement that presented unmistakable evidence of earlier slide activity. A wedge of the bedrock was in a till mass and a gently curved, apparent slip surface could be traced for a distance of some 30 feet along the exposure. A sketch of the exposure is on Figure 5. The disturbed bedrock wedge was on top of the Saskatchewan sands and could not be explained as an erratic. These sands suggested a high level terrace and it is probable that the slide occurred not long after deglaciation. In any event, this evidence suggests that at least the upper half of the slope had undergone movement prior to the present slide.

On the west flank, the edge of the slide mass did not provide any exposures more than one or two feet high. Evidence of prior movement was suggested by a tree that had a butt diameter of almost 2 feet and which came out of the bank nearly horizontally for 10 feet and then curved abruptly upward.

The high face on the eastern flank was likely influenced by the deep V-notch stream, Figure 2, that provided sufficient drainage to stabilize the soil mass. On the western flank the original slope was somewhat flatter hence the lateral edge of the sliding mass was defined only by the low ridge. Throughout the entire movement, the only noise came from the occasional snapping of tree roots.

The presence of other terraces suggested that the flat area at the toe in the vicinity of the tipped trees about 20 feet above river level represents the vestiges of a terrace that erosion has recently destroyed.

Field Investigation

In November 1963, a preliminary subsurface exploration was carried out by drilling boreholes 1 and 2, shown on Figure 6, to depths of 119 feet and 116 feet respectively using a Failing 1500 truck-mounted drilling rig. The holes were drilled dry to a depth of 50 feet and water-flush drilling was then used to the bottom of the hole. Three inch diameter Shelby tubes were driven but due to the hard nature of the soil no suitable "undisturbed" samples were obtained. Cold weather and lack of sufficient funds postponed further work.

The slide study continued the following spring. A topographical survey shown on Figure 6 was carried out, all major tension cracks were mapped and a portion of the unfailed slope to the west was included. The pre-slide slope angle was in the order of 23 degrees and the post-slide profile was almost 17 degrees.

During the summer of 1964 thirteen boreholes were drilled as shown on Figure 6. The holes were numbered 3 to 8, G1 to G4, and D1 to D3. In general, depths of the drill holes were carried to approximate river level.

Several drilling techniques were used. The G-series of holes were advanced using a truck-mounted Failing CSD 2 rotary rig. These holes were planned for probe well installation hence undisturbed sampling was not attempted. Disturbed samples for classification tests were obtained at intervals of 5 feet in addition to hole logging. Boreholes 3 and 8 were drilled using a Boyles BBS 2 truck-mounted rotary drill. A core barrel having tungsten carbide bits successfully obtained samples of the harder mudstones but was unsuccessful in the sandstones. In borehole 3, continuous samples were obtained from a depth of 63 feet to 103 feet with the core barrel having a face discharge bit using drilled mud. Borehole 8 was drilled dry using air as the drilling fluid. The D-series and holes 4 to 8 were drilled using a truck-mounted Mobile B40 drill using 4.5 inch diameter flight augers. Shelby tube sampling was tried with indifferent success.

The hole logs derived from the drilling program are given in Figure 7 (a) to (d).

Groundwater levels were noted and water samples were obtained using a simple bailing tool. Five Bishop type, porous ceramic piezometers were installed. Two were located in borehole 4 and one each in boreholes 5, 7, and 8. The former two holes were located near the unfailed soil immediately west of the toe area. The latter 2 piezometers were located in the failed mass.

Mercury manometers, de-airing apparatus and ancillary controls were housed in a small, skid-mounted building located just west of the toe of the slide. Polythene coated 4.75 mm O.D. nylon tubing connected the tips and the guage house. These lines were placed 3 feet below ground level in a narrow trench. Twelve inches of dry sawdust were placed over the tubing before backfilling. Groundwater was used in all the lines. A small propane heater kept the chill out of the guage house and the system appeared a function satisfactorily all winter.

The probewells, boreholes G1 to G4 and D. to De were cased with 3 inch diameter aluminum pipe. Thirty foot lengths of pipe were butt welded as required. The angular space around the pipe was backfilled with sand to within 1 foot of the surface. This last foot was filled with cement grout and a cement cap was formed around the pipe at the surface.

Pore pressure readings were taken over the period November 1, 1964 to April 15, 1965 at intervals of about 2 weeks. The piezometric levels are given on Figure 8. From May 1965 to January 1966 only occasional observations were made. The readings in December 1965 and January 1966 indicated a pressure 15 feet below tip elevation and therefore were considered unreliable. It is likely that the sawdust insulation became saturated and the entire system froze off. In the spring of 1966, some further landslide movement involving a central lower portion of the original landslide, completely disrupted the piezometer installations.

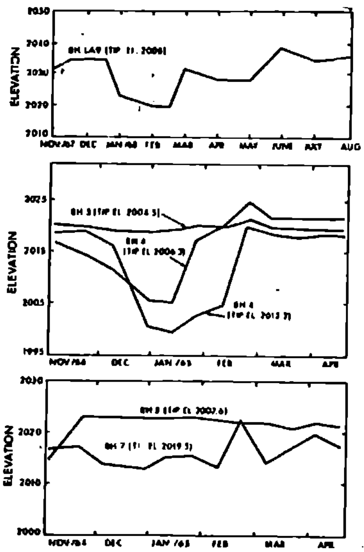

Fig.8

During the winter of 1964-65, triaxial tests were carried out on the undisturbed core samples of the Edmonton Formation bedrock. Peak angles of shearing resistance and cohesion were obtained in terms of effective stress. Studies were concentrated on the bentonitic members of the formation which comprised the major soil type above the plane of failure.

Three of the probewells, D1, D2, and G3 squeezed shut or sheared off due to continued creep that occurred between October 1964 and April 1965. The upper limit of the slide zone was thus located at these three points at an elevation of approximately 2004 feet.

In late July 1967, another hole, designated LA 9, was drilled at the Lesueur site adjacent to the earlier borehole 3. The Pitcher sampler (Terzaghi and Peck, 1967, p. 311) was successful in retrieving excellent cores of the clayey mudstones of the bedrock. The water level in this borehole remained very close to elevation 2070 for the next three months. In early November, 1967 a 1.5 inch pipe with a small well point was placed in borehole LA 9. The tip was surrounded by sand and a seal of bentonite some 6 feet long was tamped above this. A transducer piezometer (Brooker and Lindberg, 1965; Brooker et al, 1968) was installed and readings of water pressure were obtained covering the period November 1967 to August 1968. The piezometric levels are shown on Figure 8. During the summer and fall of 1967 previous piezometric data within the slide mass were checked and found to be essentially the same.

The piezometric data obtained from the installation in hole LA 9 gave a maximum water pressure corresponding to a water table at elevation 2038. The former water elevations in the scarp area resulted from a perched water table. A more realistic phreatic surface could now be employed in stability analyses.

The samples retrieved from this latest borehole were subjected to a series of reversing direct shear tests and the residual angle of shearing resistance was determined. These data augmented earlier strength testing.

A single, simple profile of the slope was taken in the fall of 1967 and the average slope angle had decreased to 14 degrees.

Laboratory and Field Data

Boreholes 1, 2, 3, and LA 9 which were drilled on the top of the valley wall, and boreholes 4 and 5 which were drilled near the toe, provide a consistent stratigraphic profile of pre-slide conditions. At Elevation 2000 there is a layer of bentonitic sandstone two feet thick which is overlain by a bentonitic clay shale. A bentonite seam six inches thick resting on a coal seam one to two feet thick occurs at Elevation 2010. There are overlain by a clay shale whose upper surface is Elevation 2044. A thin coal seam appears at Elevation 2031. The Upper Cretaceous clay shales are overlain by Pleistocene Saskatchewan Sands and Gravels which, in boreholes 3 and LA 9 extend from Elevation 2044 to 2064. From Elevation 2064 to 2082 is Wisconsin till above which there is 20 to 22 feet of glacial lake silty sands. Ground surface is Elevation 2105.

The stratigraphic succession has been constructed from drilling through the slide mass. As might be anticipated, there are some anomalies. The recent slide itself and possible previous slides caused some disturbance to the strata. The drilling technique and logging the hole from material retained in the auger bit, is imprecise enough that thin strata can be missed or not clearly recognized.

In Profile B-B Figure 7(b) there appears to be some downward displacement of the beds. In Profile C-C the thin bentonitic sandstone stratum appears reasonably continuous at Elevation 2004. This stratum is missing in Profile D-D but the bentonite-coal combination suggest some uplift of the beds. These somewhat anamalous movements may reflect local undulations of the bedrock or may be caused by sudsidiary post-slide movements.

In boreholes 6 and G1, the sand and gravel is overlain by the bedrock. This has been interpreted as evidence of a very old landslide. The sands and gravel in boreholes D1, 6, and G1 is stratigraphically lower than in holes D2, 7, G2. These sets of holes define two profiles trending NE-SW paralleling the scarp. The latter set of holes are riverward from the former. It is possible that these gravels define an old, high level course of the river and it was into this old channel that the ancient landslide occurred.

Based on Profile A-A and to a lesser extent Profile C-C, the stratigraphic section given on Figure 9 was chosen for the analyses.

Table 1 summarized the results of about forty Atterberg Limit tests and twenty-five grain size analyses. With the exception of the lake sands and the till fraction coarser than 0:074 mm, grain size distribution was determined by hydrometer analyses. The lake sediments vary from nearly clean fine sand to a silty sand having a few per cent clay sizes. The till is remarkably uniform with regard to classification data. The bedrock shows a very wide range of results but the highly plastic nature is dominant and depends largely on the clay mineral content. The bentonite has very high plasticity characteristics as might be expected. X-ray diffraction patterns for the clay size fraction of typical samples are given in Table 2.

The determination of the strength parameters of the glacial lake silty sand and the sands and gravels were determined by direct shear tests on disturbed samples remoulded to approximate field densities. The till parameters were determined from Pitcher samples using consolidated undrained triaxial tests with pore pressure measurements for peak parameters and drained direct shear tests for residual parameters. The results of these tests are given in Table 3.

Sample preparation of the Upper Cretaceous materials was more difficult and the natural variations revealed by the classification data manifested themselves as a scatter of the data. Figure 10 is a Mohr Diagram showing peak angles of shearing resistance (ϕ_p') and cohesion (c_p') in terms of effective stress. Two envelopes have been drawn giving values of $\phi_p' = 24^o$ and $c_p' = 1300$ psf for the clay shale and values of $\phi_p' = 14^o$ and $c_p' = 865$ psf for the bentonitic clay shale. (See Table 4).

The residual angles of shearing resistance are plotted on Figure 11 and, while showing some scatter also indicate that, at large deformations, the clay mineral montmorillonite exerts some influence. An effective residual angle of 14^o was accepted for the clay shale. The bentonitic clay shale had a residual angle of 10^o.

From a study of the piezometric data, a piezometric surface was drawn on Figure 9. The question immediately arises as to whether or not this represents pre-slide water conditions. A definite answer is not known but such evidence as is available suggests the piezometric surface postulated is reasonable. The perched water table noted in borehole 3 did not change appreciably during a three year period (borehole 3 drilled July 1964, borehole LA 9 drilled July 1967). The water levels in the boreholes in the lower part of the slide area maintained a relatively constant elevation during the same period. The water elevations in Figure 8 for piezometers in the slide mass and those immediately to the east suggest that pre-slide water levels were approximated to a reasonable degree.

Landslide Analyses

The failure surface was defined by the visible portion of the scarp face, which was some 30 feet high, and by the closed-off probewells. The observed behaviour of the material at the toe during failure suggests that the lower portion of the slip surface is planar. This data combined with the flat lying stratigraphy, lead to the consideration of two failure surfaces. The first consisted of the downward projection of the scarp to meet a horizontal line drawn through the closed-off probewell points. The second slip surface cut a corner off the first. The scarp face was projected downward to the top of the clay shale and this point was joined to a point on the horizontal portion of the slip surface just to the left of a closed-off probewell. These failure surfaces are shown on Figure 9. Subsequent analyses showed the latter surface to be the more critical hence it was used in the majority of the trials.

The slip surfaces as defined lend themselves to analysis using the method of Morgenstern and Price (1965). All the analyses were in terms of effective stresses.

The first analysis assumed all soils at peak values of effective strength and yielded a factor of safety, F, of 1.37. The second analysis assumed all soil parameters at their residual value and F was 0.54. Using all peak angles of friction but zero cohensin for all soils, F was 0.74. These preliminary analyses suggest that the soil parameters being mobilized at incipient failure were greater than residual, less than peak at least on some portion of the sliding surface and that some cohesion must be acting.

The problem becomes one of assigning soil parameters to the various strata throughout the profile consistent with observation and natural processes. The following concepts were used as a guide in subsequent slip surface analyses.

The air photos and ground reconnaissance indicate that the pre-slide surface of the valley wall comprised old slump topography. The upper few tens of feet were the exposed scarp face of an old slide modified by erosion and vegetation. This interpretation is consistent with the old slides suggested by the drill hole results and the section exposed on the east flank of the recent slide. There was no field evidence or any old ground movements south of the original valley wall. Hence it would seem reasonable that the pre-slide valley wall represented the southerly limit of former landslide activity. In addition, observations discussed previously suggest that these old slides occurred at the time the Saskatchewan sands and gravels were being laid down as a terrace deposit. From these concepts it follows that the steeply dipping part of the recent failure surface, as represented by the scarp face and its downward extension, is a "first time" failure and the soils

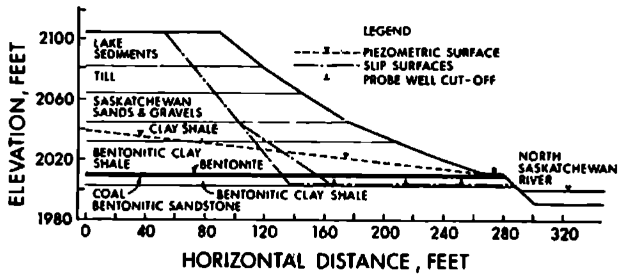

Fig.9

Table 1

Results of Classification Tests

	WL	I_p	% S	% Si	% C
Lake Sands	Non Plastic		66	25	9
			95	5	0
			60	33	7
Till	40	20	35	35	30
	38	20	33	37	30
Bentonite	434	385	0	5	95
	214	150	15	5	80
Clay Shale	100	75	20	35	45
Bentonitic Clay Shale	200	170	17	28	55

Table 2

Results of X-Ray Diffraction Tests on Clay Fraction of Typical Samples

Soil	Per Cent Montmorillonite	Per Cent Illite	Per Cent Kaolinite/Chlorite
Till	50	30	10
Clay Shale	80-100	10-0	10-0
Bentonite	100	-	-

Table 3

Strength Parameters for
Pleistocene Soils in Terms of Effective Stresses

Soil	Peak Parameters c'_p	ϕ'_p	Residual Parameters c'_r	ϕ'_r	Total Unit Weight
Silty Sand	0	41	0	38	112
Till	660	24	0	22.6	132
Sand and Gravel	0	38	0	35	120

Angle in degrees

Cohesion in lb. per sq. ft.

Unit weight in lb. per cu. ft.

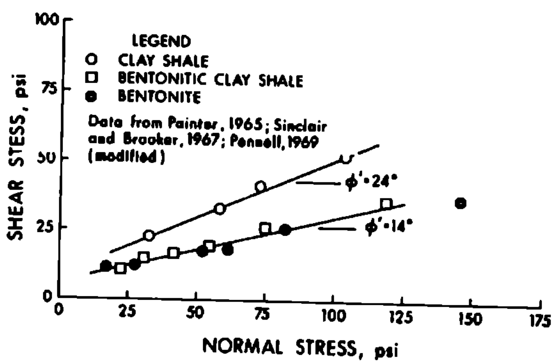

Fig.10

Table 4

Strength Parameters for

Upper Cretaceous Soils in Terms of Effective Stresses

Soil	Peak Parameters c'_p	ϕ'_p	Residual Parameters c'_r	ϕ'_r	Total Unit Weight
Clay Shale	1300	24	0	17	114
Bentonitic Clay Shale	865	14	0	10	112
Bentonite	800	14	0	8	108
Coal	0	32	0	32	87

Angle in degrees

Cohesion in lb. per sq. ft.

Unit weight in lb. per cu. ft.

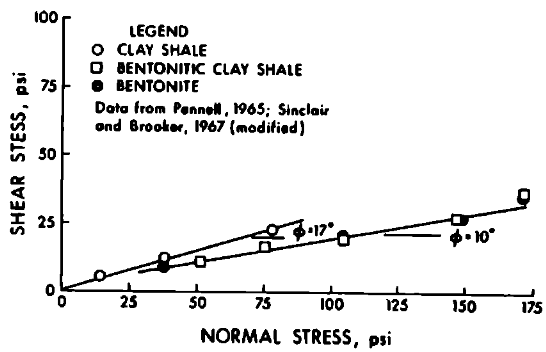

Fig.II

associated with it are mobilizing peak parameters.

The preceding reasoning considered in conjunction with the results of the preliminary analyses require that the shear resistance mobilized along the horizontal part of the slip surface was less than peak. Two factors that might lead to slope instability are removal of "toe load" and a softening of the soil by rebound due to unloading. The first of these is represented by erosion of the terrace by lateral migration of the river. This phenomenon has been observed at other similar locations (Thomson, 1970, Hayley, 1968). The terrace erosion is not a steady, gradual process but rather one that involves a small slide followed by a slow removal of the slide debris by the river. The immediate bank is steepened, another slide occurs, and the process is repeated. During the interval between slides there would be a decrease in soil strength in the toe area due to swelling and horizontal deformation.

Erosion of the terrace would also lead to a decrease in confining pressure in the general toe area which would be accompanied by an increase in moisture content due to swelling. Associated with the increase in moisture content is a decrease in strength.

The development of large shear strains due to unloading would also produce a decrease in clayeysoils.

To investigate the influence of a decreasing strength along the horizontal part of the slip surface, a series of analyses were made which simulated lower strengths marching inward from the river. The soils through which the steeply dipping part of the slide surface passed were considered to be mobilizing peak strength parameters. The horizontal portion of the slip surface was divided into three approximately equal sections. In the first analysis, the outer one third was considered to be at residual strength. In two subsequent analyses the outer two thirds and finally the entire hroizontal portion was at residual. The factors of safety for the latter two analyses were 1.03 and 0.94 respectively. These results indicate that the strength decrease migrated inward from the river along almost the entire horizontal portion of the slip surface. These analyses involving the residual value of strength are an extreme combination and would appear to be an unlikely situation in the field.

It is possible that the cohesion intercept for the clay shale and the bentonitic clay shale could have been overestimated in interpreting the laboratory strength test results. This could arise if there were some curvature of the Mohr envelope for low values of the normal stress. To investigate the effects of lower strength values the clay shale was assigned varying values of cohesion. The upper stratum of the bentonitic clay shale was assigned values of 500 psf along the inner third, 350 psf along the central third, and 200 psf along the outer third of the horizontal part of the failure surface. All angles of shearing resistance were maintained at peak values. For the given values assigned to the bentonitic clay shale, reducing the cohesion of the clay shale from 1300 psf to 400 psf resulted in F = 1.10. Similarly, reducing the angles of shearing resistance of the lake sands and the Saskatchewan sands to 32° from their peak values, resulted in F = 1.05. Therefore, low values of cohesion must act along the horizontal part of the failure surface.

Another series of analyses varied the value of cohesion of the clay shale stratum as well as the various values of cohesion for the bentonitic clay shale. All shear angles were retained at peak values. Assuming the clay shale to have a cohesion of 700 psf, the upper bentonitic clay shale 500 psf and the lower portion of the slip surface to be at 350 psf on the inner half and 200 psf on the outer half, F was 1.06. Decreasing the clay shale to 400 psf then to 100 psf, maintaining the clay shale at the previous values, the factors of safety were 1.04 and 1.02 respectively. It is important to note that the strength still must be remarkably low along the horizontal part of the slip surface despite considerable reductions in cohesion in other strata.

Another possible factor is fluctuations in the water table within the slide mass arising from freezing of the soil surface or rising water levels in the toe area due to increasing river discharge. The previous marching in technique was employed and again the results were similar in that low strengths had to be acting along the lower portion before the factor of safety dropped to values close to unity. A low water table in the terrace improved the factors of safety but they still remained low.

A series of analyses were conducted to assess the stability of the upper portion of the valley wall. The slide surface was chosen similar in shape to preceding surfaces with a lower horizontal portion at the contact of the clay shale and the bentonitic clay shale. The upper soils were assigned their peak strength parameters but the soil strength along the horizontal portion of the assumed slip surrace was taken as $c = 500$ psf and $\phi = 14^{\circ}$. The factor of safety for these conditions was 1.6. Allowing the cohesion to drop to zero decreased the factor of safety to 1.4. The upper part of the valley wall is therefore stable and some confirmation is obtained that the landslide acted as a large, single block.

The stability of the terrace was also investigated. The results of the analysis indicate that unstable conditions exist adjacent to the river bank. This merely confirms the observations noted previously.

The results of the circular arc failure surface analyses present some interesting comments despite the fact that the landslide is clearly a block glide movement. Several analyses were carried out using ICES-LEASE Program (Bailey and Christian, 1969) which is based on the method of Bishop (1955). The critical failure surface was chosen such that it was tangent to the sandstone layer at Elevation 2003 and passed through the lawn level in the vicinity of the know scarp. Using peak strength parameters throughout, the factor of safety was found to be 1.34. Using residual parameters for soils the factor of safety dropped to 0.55. Assigning peak parameters to all soils except the lower bentonitic clay shale which was considered at residual, the factor of safety was 0.98. The results of these analyses are remarkably in agreement with those of the non-circular method. The circular slip surface breaks the lower slope in the vicinity of the junction of the valley wall and the terrace. The role played by erosion of the terrace is not evident using the circular arc procedure.

Failure Mechanism

All the analyses indicate that the shear strength of the soil along the lower part of the slip surface had decreased to a value considerably lower than that expressed by peak parameters. This decrease in strength was brought about by erosion of the terrace by lateral migration of the river. The destruction of the terrace involves small landslides in which one or two tens of feet of terrace are affected. These small slides generate deformation which leads to strength decrease of the soils in the vicinity as well as reducing confining pressure of strata inside the sliding mass. As a consequence swelling occurs and moisture contents increase which lead to a strength reduction.

As the terrace became narrower, the overall slope stability decreased until some triggering mechanism precipitated the major failure. The slight ground displacements noted in April indicated failure to have occurred sometime during the winter and suggest that the triggering mechanism was a build up of pore pressures in the toe area due to ground freezing.

Once overall failure occurred, the deformation involved brought about a gneral decrease in shear resistance until the catastrophic movements occurred.

It would appear that a key issue in achieving some degree of stability is the prevention of the small landslides along the terrace, that is, arresting the erosion. The problem then assumes a different character but probably is of lesser magnitude than that of stabilizing a slope some 100 feet high. For any particular but similar field situation the time interval between successive small slides at the terrace level is not known and in any event would be variable. The minimum width of the terrace before the major failure is also an unknown. Both of these unknowns are functions of several natural phenomena among which are the unpredictable river discharges and the variation of pore pressures within ghe slide mass. It, therefore, becomes a difficult task to entice owners to spend money on stability measures that often appear to have a rather speculative nature. Once cracks or other definite failure signs appear in the scarp area, it would seem that a major slope stability problem is involved in addition to arresting erosion.

Physical Chemical Effects

A subsidiary program concerned with the physical chemical characteristics of the clay shales was carried out. Samples of the groundwater were analyzed for salt content and for alkalinity (in terms of the bicarbonate radical), the sum of the calcium plus magnesium cations and the sodium cation. The results are presented in Table 5. Except for boreholes 3 and G1, the very high proportion of the sodium cation is notable.

The cation exchange capacity and the exchangeable cations were determined on many of the disturbed samples that were obtained. The cation exchange capacity of a sample is determined by the absorption of ammonia and represents the amount of cations absorbed on the surface area of the soil. It is usually expressed in milliequivalents per 100 grams of air dried soil. The exchangeable cations were 100 grams of air dried soil. The exchangeable cations and the cation exchange capacity may be taken as a crude measure of the salts in the pore water.

Previous work has indicated that an increase in the proportion of the absorbed sodium cation and a decrease in the salt content reduce the angle of shearing resistance (Thomson, 1963, Locker, 1963).

There is some evidence that, in soils having essentially no salts in the pore water and if sodium occupies 25 per cent or more of the exchange positions, the soil behaves as if it were homionic in sodium. Kenney (1967) reported on a series of direct shear tests on natural soils and pure minerals. He found that the clay mineral montmorillonite exerts a profound influence and results in low residual angles of shearing resistance. In addition, decreases in pore water salt cortent and increased in absorbed sodium cause a decrease in the residual angle for montmorillonitic soils.

The cation exchange data versus depth is plotted on Figure 12 for borehole 7. The notable feature is the abrupt decrease in the pore water salt content near the failure plane (approximately Elevation 2004) and the sharp increase in the per cent of absorbed sodium on the exchange complex. These phenomena may influence (i) the low peak and residual angles for the soils rich in montmorillonite, (ii) the peak and residual angles of the bentonite and bentonitic soils having nearly the same values, (iii) the continued creep of the slide mass in the years since the original failure, and (iv) some of the decrease in strength parameters of the clay shales. The latter two could result from changes in the absorbed complex due to swelling or to freer movement of the groundwater due to an increase in permeability resulting from fracturing of the soil mass as a consequence of the failure. Figure 12 indicates that soils outside this slide zone have a different absorbed cation complex than similar soils within the slide mass. If it is assumed that preslide conditions were reasonably uniform then Figure 13 suggests that some cation exchange did occur.

Summary

The Lesueur landslide appears to be a "first time" slide that mobilizes overall strength values less than full peak. In particular, it appears that peak angles of shearing resistance are being mobilized but that the cohesion is between the peak and residual values. This is consistent with the analyses of Skempton (1970).

It will be noted that in one set of analyses, the strength along the horizontal portion of the failure surface was at residual strength values while in another comparable series the angle of shearing resistance was assumed at its peak value but the cohesion was reduced to low values. However, these two series of analyses yielded similar values of the factor of safety within practical field limits and indicate, therefore, that the influence of peak and residual angles of shearing resistance for the bentonitic clay shale cannot be clearly differentiated in this field situation.

The results of the analyses in which the values of cohesion were varied for the clayey members, although indicating the necessity for decreased strength along the horizontal part of the failure plane, also indicate the difficulty in drawing conclusions concerning the mobilization of soil parameters in the non-homogeneous strata of the Great Plains Region.

The soil parameters may be affected to some degree by changes in the absorbed cation complex. Changes in soil behaviour also may be influenced by physical chemical phenomena although these changes may be associated with post-slide changes in the pore water regime.

The major factor leading to the landslide was the erosion of the terrace and a triggering mechanism is suggested as a build-up of water pressure in the toe area. Arresting lateral erosion of the terrace is suggested as being central to preventing the major landslide. This problem has a different character to that of stabilizing a major slide but must be carried out before cracking of the ground or other signs of movement appear in the scarp area.

Acknowledgments

The original investigation of the Lesueur landslide was carried out by W.T. Painter as part of an M.Sc. program. Subsequent field work was performed by D.G. Pennel who, in conjunction with P. Ali, carried out the laboratory testing program. The physical chemical data were obtained by L.J. Bednar. The financial assistance of the National Research Council of Canada under Grant No. A3559 is gratefully acknowleged.

Table 5

Results of Groundwater Analyses

Borehole No.	pH	Alkalinity Equivalent HCO_3 ppm	Ca and Mg ppm	Na ppm
3	7.6	410	140	0
4	7.7	1020	36	304
5	7.4	725	252	179
6	7.6	480	224	203
7	8.1	780	22	297
D1	7.3	660	109	152
D2	8.7	900	144	136
G1	7.7	715	426	0

from Painter 1965 (modified)

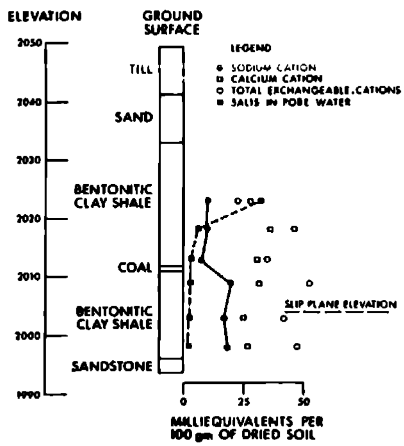

Fig.I2

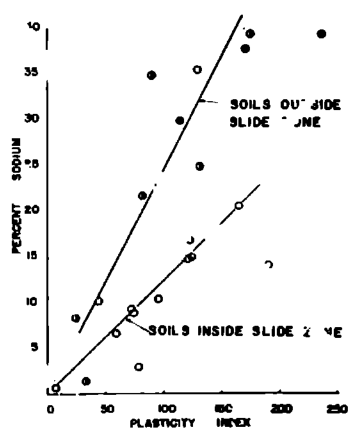

Fig.I3

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STABILITY OF SLOPES IN THE DOGGER SERIES
OF JURA. Andreas K.Wackernagel (Switzerland)

SYNOPSIS. The shearing strength of the highly overconsolidated clay shales and marl shales in the Dogger formation of Jura were investigated during construction of the Swiss National Highways. It was found that in deep cuts always the residual shear strength determined by direct shear tests should be taken into account.

SHEARING RESISTANCE OF THE DOGGER SHALES

During construction of the Swiss National Highway N2 in the Jura ranges the clay shales of the Dogger series of Jura caused various slides. The formations of the Dogger comprising clay shales or marly shales are the following:

- Callovien clay
- Blagdeni marl
- Sowerbyi clay
- Murchisonae marl
- Opalinus clay

The shearing strength was investigated by triaxial and direct shear test. During these investigations the direct shear test led to useful results.

The residual angle of shearing resistance obtained is plotted as a function of the clay content of the samples in Fig. 1.

Although these shales are highly overconsolidated clays having a consistency index well above 1.0 and a high cohesion above 10 t/m2 the points fit well into Skempton's Diagram (1964). The actual slides which had occurred during construction were recalculated. They indicated that the angle of shearing resistance in the sliding surface corresponded to the residual angle. No cohesion at all or a cohesion of 2.0 t/m2 at most was left in addition to the frictional resistance.

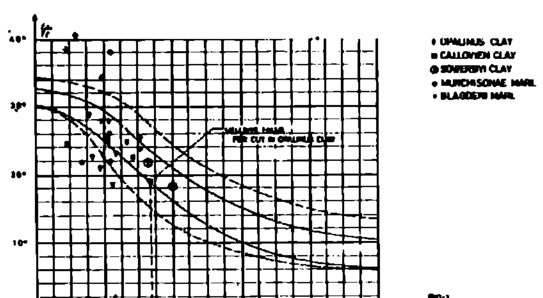

FIG-1 RESIDUAL ANGLE φ_r OF SHEARING RESISTANCE IN THE DOGGER SERIES OF JURA

CASE HISTORY OF CUT IN OPALINUS SHALE

A cut near Augst had to be executed entirely in Opalinus shale whose characteristics were the following:

Water content w	5.7 - 17.5 %
Liquid limit w_L	37.9 - 39.6 %
Plastic limit w_P	16.6 - 21.0 %
Consistency Index I_C	1.20 - 1.56
Clay content < 0.002 mm	32.0 - 41.0 %
Cohesion c determined from unconfined compression test	12 t/m2

Typical grain size distribution curves are given in Fig. 2.

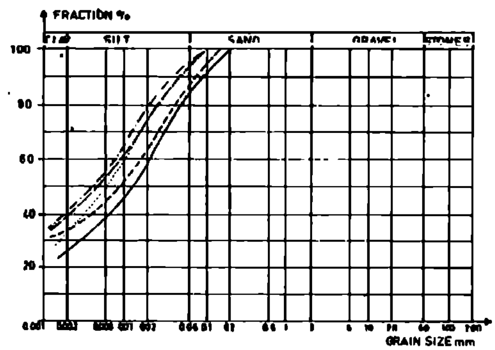

FIG 2. TYPICAL GRAIN SIZE DISTRIBUTION OF OPALINUS CLAY

The shear parameters assumed for the design of the cut as obtained by preliminary tests were $\varphi_r' = 20^{\circ}$, $c_r' = 1.0$ t/m2.

As there was a ground water table parallel to the natural ground surface, the pore water pressures were eliminated by bored horizontal drainage pipes having a length of 40 m. The design slope of the cut was 1 : 2.5 as shown in Fig. 3.

The cut was excavated successfully in summer 1969 However in winter due to freezing some of the drainage pipes stopped functioning. Hence a first slide occurred in February 1970 followed by a second slide in September 1970. By recalculation the following shear parameters were obtained

$$\varphi_r' = 18.5^{\circ} - 22^{\circ}, \quad c_r' = 0,0 \text{ t/m2}$$

depending upon the possible variations of the groundwater table. Those values nearly confirmed the design assumptions. It is however interesting to note the disappearance of a cohesion.

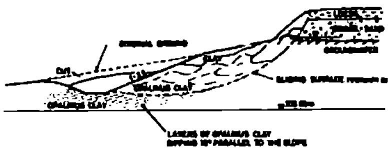

FIG-3 CROSS SECTION OF CUT NEAR AUDST WITH SLIDE OCCURRED, Km 128.0 (DR L HAUBER)

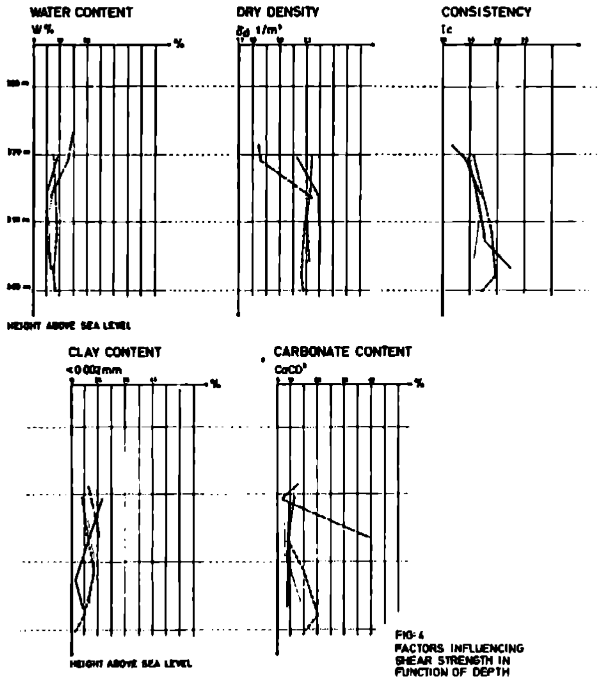

FIG-4 FACTORS INFLUENCING SHEAR STRENGTH IN FUNCTION OF DEPTH

It was decided to arrive at a solution which does not depend entirely upon drainage. For the further design a friction angle of φ'_r = 18.5° and no cohesion at all was assumed. This corresponded to the minimum residual shear strength determined by direct shear test. This assumption was necessary up to a depth of approximately 10 m below the surface up to which the slides had occurred. It was however not known whether very deep-seated sliding surfaces are possible.

Hence it was necessary to investigate by a number of bore holes how the shearing strength varies in function of depth. The factors influencing the shear strength are

- Water content
- Dry density
- Consistency Index
- Clay content $<$ 0.002 mm
- Carbonate content $CaCO_3$

These factors are plotted in Fig. 4 in function of depth.

It is seen that density, consistency and carbonate content gradually, however slightly, increase with increasing depth, while water content decreases.

The shear parameters φ'_{res} and c'_{res} are plotted in function of depth below the surface of the shale in Fig. 5.

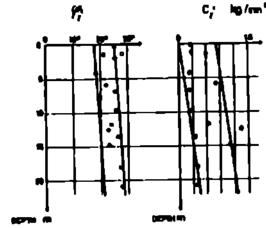

FIG-5 SHEAR PARAMETERS c'_r and φ'_r IN FUNCTION OF DEPTH IN OPALINUS CLAY

Also here it can be observed that there is a slight, but noticeable increase of the parameters. Hence for the design it was assumed that in a depth from 0.0 to 10.0 m there is c' = 0.0 t/m2, from 10.0 m to 20.0 m there is c' = 1.0 t/m2, below 20.0 m there is c' = 2.0 t/m2. By these assumptions deep-seated sliding surfaces are no longer critical. This fact is also observed practically.

The residual shear strength together with the maximum shear strength is shown in Fig. 6.

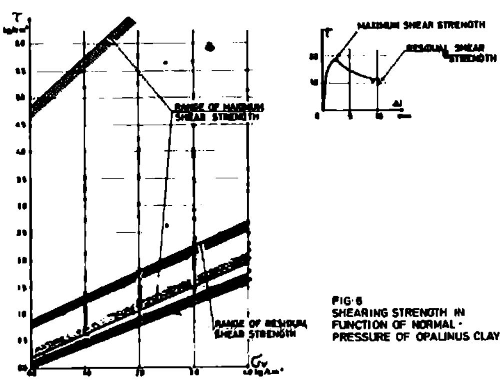

FIG-6 SHEARING STRENGTH IN FUNCTION OF NORMAL-PRESSURE OF OPALINUS CLAY

It can be seen that the maximum strength can be several times higher than the residual strength. In the observed sliding surfaces, however, only the residual strength can be taken into account.

CONCLUSION

Although the shales in the Dogger series of Jura are highly overconsolidated clays, they have a rather low residual shear strength. This was observed when a number of slides occurred during construction of the Swiss National Highway accross the Jura ranges. In a cut near Augst the residual shear strength was found to increase slightly with depth. This increase is however sufficient in the case investigated to prevent the occurrence of deep seated sliding surfaces.

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FOR THE RELATIONSHIP BETWEEN LARGE SCALE EARTHWORK AND THE OCCURANCE OF LANDSLIDE IN NATURAL SLOPE. Masasuke Watari* /Japan/

In order to research the relationship between the occurance of landslide which has more-than 5 metres thickness and the influence to their safety factors by deep excavation or large fill, the author has tried to analyse the stability from many examples for the past 10 years.
To calculate the safety factor, the Swedish sliced method was used. It was applied to the actual slided surfaces which were occurred by artificial causes like to deep excavation or large fill.
From results of many soil tests with undisturbed samples at slided surfaces, it was supposed that the cohesion of clay is almost proportionate to the depth of slided surface. (cf. Table I)

Table I

Depth of Sliding Surface (m)	Cohesion of Sliding Surface Kg/cm²
5	0.05
10	0.10
15	0.15
20	0.20
25	0.25

Therefore, the internal friction angle was calculated by reverse operation in which the factor of safety was assumed to be 1.00 without artificial causes.

As the results of the calculation, it was recognized that the scale of artificial causes (the diminution of safety factor) would be controled by material of slided mass and that the behavior of sliding movement and the methods of countermeasure for landslide would be much influenced by the material, too.
Owing to these recognition, the author considered a classification of landslide by sliding material.
The critical diminution value of safety factor in natural slope by earthwork and the methods of countermeasure to landslide occurance became to be able to be established by this classification as Table II.

Table II Classification of Landslide

Type of Slide	Infance	Youth	Full Age	Old Age
Sliding material on the Head	Weathered rock	Heavy weathered rock	Soil with large gravels	Clayey soil with gravels
Sliding material on the Tang	Soil with large gravels	Soil with gravels	Soil with gravel and partly clay	Clay
Mean velocity of sliding	More than 2cm/day	0.5-2.0 cm/day	0.2-0.5 cm/day	Less than 0.2 cm/day
Continuity of the movement	Unexpected and closed short time	Sometimes intermittant but closed short time	Intermittant, sometimes continuous	Continuous, sometimes intermittant
Type of movement	Glide with plain sliding surface	Cylindrical slide	Slump with many slides	Creep
Critical diminuation of F.S. by earthwork	more than 10%	10-5%	5-3%	Less than 3%
Forecast of slide occurance	Very difficult	Difficult	Easy	Very easy
Countermeasure	Removal of Head, counterweight fill, or deep drainage	Removal of Head, Counterweight fill, or deep drainage & surface	Horizontal drainage, Pylings	Give up or change the earthwork, or removal of all slide mass

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BRITTLE FAILURE OF A VOLCANIC SOIL "SHIRASU" IN REFERENCE TO THE STABILITY OF NATURAL AND CUT SLOPES. T. Yamanouchi, H. Murata /Japan/

The so-called "Shirasu" which is widely distributed in South Kyushu, Japan can be classified into unwelded pumice flow, pumice flow, and their secondary deposits. Shirasu is known in Japan as a typical "structurally unstable soil", and lately it has been the cause of frequent damage involving slope failures and other accidents, resulting from heavy rains and earthquakes.
The periphery of Shirasu plateau often maintains enduring stability with the slopes of several tens of meters in height and stands almost perpendicularly. Such steep slopes as these have been positively applied as a traditional method to cut slopes for about 300 years, since they are effective for preventing slope surface from stream erosion by letting the top surface of loam be an impervious cover.
This paper deals with the brittle failure of undisturbed Shirasu with an eye on the fact that failures are often caused by cracks given to steep natural and cut slopes by the action of tension instead of circular sliding, except for the cases of erosive failures. The authors carried out uniaxial and triaxial compression tests as well as Brazilian and uniaxial tensile tests on vaious kinds of undisturbed Shirasu specimens, and investigated such failure criterion as is suitable for undisturbed Shirasu by comparing those failure conditions with such failure criteria as Mohr-Coulomb, Coulomb-Navier, Griffith and Modified Griffith. They made clear that Coulomb-Navier and Modified Griffith criteria are applicable to Shirasu.
Furthermore, they showed the relations among moisture content ratio, degree of saturation and tensile strength or degree of brittleness on Shirasu.
Based on the above laboratory study on undisturbed specimens of Shirasu, such failures of steep natural and cut slopes of Shirasu as actually arising in the field are explained.