INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

SECTION IV

STABILITY AND DEFORMATIONS OF EARTH CONSTRUCTIONS

GENERAL REPORT

A.E. BRETTING (Denmark)

IV a. EMBANKMENTS OF HIGHWAY AND RAILROADS.
(2 papers).

In this subsection two papers are received, both Dutch. CUPERUS and de NIE describe in their paper: "Strengthening the Road-Bed of a Railway, supported by Soft Soil and situated amidst the Buildings of the Central Part of Town" interesting observations of railway banks on soft subsoil, mainly peat. Measurements of pore water pressure and displacements are made.

In the second paper by de NIE: "Undulation of Railway Embankments on Soft Sub-Soil during passing of Trains" observations of the undulations of an embankment on similar ground are observed. It is surprising to note that only an insignificant part of the waves are rising above the line of equilibrium.

IV b. DAMS AND LEVEES. (14 papers).

In this subsection the studies for several earth dams are described. It seems that English engineers mostly prefer the $\varphi = 0$ analysis when studying the stability against sliding, especially when the dam is founded on the tertiary London clay, as f.inst. in the case of the dam for Walton Reservoir No. 1 described by W.A. BISHOP in his paper: "Some Factors involved in the Design of a Large Earth Dam in the Thames Valley". The uppermost part of the clay had such a small cohesion that stability was only secured when consolidation of the clay could keep pace with the construction work, which had to be regulated accordingly.

By the construction of the dam for Muirhead Reservoir, Scotland, (JAMES A. BANKS: "Construction of Muirhead Reservoir, Scotland") slides occurred and the height of the dam had to be diminished. Soil studies were made, but the question of pore water pressure does not seem to have been examined.

Most authors employ circular arc slides for their studies of dam stability and much attention is paid to the determination and avoiding of pore water pressure in the dam during construction.

Two Australian papers treat this subject. A. RUFENACHT: "Pore Pressure Assumptions for Stability Studies of Earth Dams" states that the pore water pressure produced during construction of the dam, before the filling of the reservoir, is usually fixed according to empirical assumptions. He tries to give a more rational method for determining this pressure, using the well-known consolidation theory of Terzaghi-Fröhlich.

It does not seem obvious that correct results can be obtained, as full saturation of material is assumed during consolidation. The author states that the compaction should if possible be on the dry side of the optimum after Proctor's method.

In his paper "Stability Analysis in the Design of Earth and Rockfill Dams" R.C. CLARKE gives a complete analysis for a combined earth and rockfill dam. The method is the usual, em-ploying circular arc slides, but for the pore water pressure at the end of the construction the assumption seems to be empirical.

American engineers have paid special attention to the study of construction pore water pressures.

In his paper: "Estimating Construction Pore Pressure in Rolled Earth Dams" J.W. HILF gives a relatively simple theory which takes the air in the voids into consideration. He states on basis of field measurements that fluid pressures of considerable magnitude can develop in an unsaturated rolled earth dam during construction.

By combining Boyle's law for compressibility of air with Henry's law for solubility of air in water he finds the increase in air pressure after consolidation.

It seems that the symbol P in equation (1) means the increase in air pressure after consolidation, not the pressure itself, as indicated. No drainage is assumed.

In this way the effective stresses are found; when full saturation is reached they cannot increase further. A method for finding the relation between total stresses and pore water pressure is given.

For grains not passing the No. 4 sieve, which are usually not included in the laboratory tests, a simple correction can be made, provided that the amount of these grains is less than about 20%.

The proposed method is compared with the results of field observations giving actual piezometric pressures and vertical consolidations.

In the paper by F.C. WALKER and W.W. DAEHN: "Ten Years of Pore Pressure Measurements" an interesting description of the development of measuring apparatus for pore water pressure is given.

A description is given of a piezometer installation, where pore water pressure can be transmitted through connecting water filled tubes to Bourdon-tube gauges placed at the downstream slope of the dam. No flow from the embankment is caused by the measurements. Actual measurements from several dams are given.

The authors recommend strongly that such measurements should be done so that a continual check can be maintained on the stability of the dam and valuable data can be collected for the design of new dams.

In their paper: "Deformability of Earth Materials and its Effect on the Stability of Earth Dams Following a Rapid Drawdown" R.E. GLOVER, H.J. GIBBS and W.W. DAEHN try to study the deformability of earth materials and its effect on the stability of earth dams following a rapid drawdown on a purely theoretical basis.

They find that no greater pore water pressures than those of the gravity flow system need ordinarily to be used

When the correlation with actually observed data does not seem to give very near quantitative accordance it is not surprising.When

it is considered that the arrangement of the grains shown in fig. 1 can hardly be considered as probable.

In the paper by R.R. PROCTOR: "Earth Dam Design, Construction and Performance" the author also stresses the importance of considering the pore water pressure.

The method of calculation shown in Fig.1 should be approximately correct, even if the explanation given on page 2 seems hard to understand.

The following comments are given by the reporter:

If the streamlines of the flowing groundwater are supposed to be parallel to the surface of the stream, the distribution of head $(z + \frac{p}{\delta})$ is "static" $(z + \frac{p}{\delta} = const.)$ in lines perpendicular to the surface and it is easily shown that the resultant effect of the water pressures along the three sides of the element considered will be a bouyancy B which is not vertical but perpendicular to the surface of the water and equal to the volume of the element multiplied by the specific weight of the water. This gives a component horizontal force equal to $B \sin \beta$ and a component vertical force $B \cos \beta$. (β = inclination of groundwater surface).

Considering the whole sliding volume it is evident that the water pressures in the sides of the elements can be considered as internal forces which do not influence the sliding so that only the pore water pressure in the sliding circle itself needs to be considered. The result will of course be the same as the above mentioned.

Proctor stresses the importance of supervision of the work, so that accordance with provised figures is secured. The soil moisture must, to prevent saturation and occurrence of pore water surpressure, be well below the moisture for optimum of density.

In another paper: "The Elimination of Hydrostatic Uplift Pressures in New Earthfill Dams" Proctor gives more detailed recommendations for this purpose.

It seems that Proctor does not consider the effect of the air in the voids and its absorption by the water.

In the paper: "Stability Analysis by Application of the Elastic Theory" JAMES G. PATRICK employs the theory of elasticity on earth dams.

The dam is divided in horizontal strips and the total stresses in the foundation ground are found by superposition (on page 3 is indicated that they are added algebraically). These stresses are compared with the strength of the soil, and a curve for the variable factor of safety is drawn.

Patrick requires that no point in the foundation should be overstressed.

It deserves further discussion if this criterion should be more generally adopted. In many cases, even for homogeneous cohesive soils an even distribution of shear stresses in the sliding surface can not be presumed. Stiff clays will often give way for mean stresses, which are considerably lower than it should be supposed after laboratory tests.

It seems as if the clay fails bit by bit, especially if the resistance is considerably diminished by remoulding.

In the paper: "The Significance of Prestress in Embankment Dams" PAUL BAUMANN discusses the importance of prestresses with special reference to earthquakes and other vibrations.

The surface of the compacted material must be surcharged to prevent deconsolidation due to vibrations or drying out; especially

this is valid for non-cohesive granular materials.

If sufficiently prestressed such dams will according to Baumann act as monolithics and can be calculated accordingly.

Synopsis of subsections IV a and IV b.

The shearing strength in the sliding surface is now often determined on the basis of triaxial tests. Due regard is then to be taken to the pore water pressure reducing the effective stresses.

The correct determination of the pore water pressure especially in dams under construction is difficult, but experience shows that considerable surpressure can exist in the interior of an unsaturated rolled dam.

Methods are proposed for the control and possible prevention of pore water pressure.

It seems important to take the air in the voids into consideration.

Measurement of actual pore water pressure during construction and in service is recommended and will probably in the future be generally employed for more important dams.

It ought to be discussed whether a more general use of the elastic theory for the determination of actual shear stresses is to be recommended, and whether it should be required that the shearing strenth should not be surpassed in any single point of the foundation of dams.

IV c. EXCAVATIONS AND SLOPES. (16 papers).

In this subsection several cases of flow slides in loose sand are reported.

R.B. PECK gives in his paper: "Description of a Flow Slide in Loose Sand" a report of such a slide in Chicago. No seepage pressure was present, and the primary cause seems to have been the collapse of the extremely loose structure of the fine sand so that it was completely liquified. Relatively small disturbances caused by a gap in a steel sheet piling wall caused the flow to continue even when an extremely flat slope was attained. The small density of the fine sand was indicated by the number of blows needed to drive a split spoon 1 foot.

W.H. WARD in his paper: "A Coastal Landslip" reports a slide from Castle Hill, Newhaven, Sussex, where fine liquified sand seems to have played an important role besides the occurrence of soft clay.

In a paper: "A Study of Foundation Failures at a River Bank Revetment" CH. SENOUR and W.J. TURNBULL report slides in the Mississippi River which also seen to be caused at least partially by the loose structure of fine sand. Studies are continued, especially with regard to natural density of the foundation sands.

A detailed description of "Coastal Flow Slides in the Dutch Province of Zeeland" is given by A.W. KOPPEJAN, B.M. van WAMELEN and L.J.H. WEINBERG.

Enormous flow slides occur at different places in Holland, and the studies of these particular slides are very illuminant.

The critical density of the fine sand is determined by laboratory tests, where the volume changes are found as a function of the difference between the two principal stresses.

It is found that for increasing values of $(\sigma_1 - \sigma_3)$ i.e. of the shearing stress τ; the sand first get a decrease in volume followed later by an increase. The turning point gives the means of determining the critical density where a shear stress causes considerable decrease in volume and consequently col-

lapse of the structure of the sand and corresponding surpressure in the pore water and reduction of the effective friction.

It is concluded that sands with an angle of internal friction less than 37° have a density below the critical value.

The angle of friction of the sand in the natural state is determined indirectly by penetration tests.

Several slides in fissured clay are reported: F.L. CASSEL has described several English slides in his paper: "Slips in Fissured Clay".

The fissured clays are generally of older geological age, and are characterized by their macroscopic structure; they consist of small polyhedric fragments of hard clay, parted by numerous shiny sliding surfaces. When water is present a softening of the clay in the fissures proceeds and a strong reduction of shearing resistance follows as explained by Terzaghi already in 1936. Many years, up to 50, 70 or even 90 may pass before slide occurs.

Cassel warns against basing the design of such slopes on the usual tests on undisturbed samples.

A.E. SKEMPTON in his paper: "The Rate of Softening in Stiff Fissured Clays with Special Reference to London Clay" makes on attempt to find a time scale for the softening of the clay in the fissures. The method is purely empirical The shear strength is plotted against the time.

In several reported slides the pore water pressure undoubtedly played an important part in the failure of the slope.

L. MARIVOET in his paper: "Control of the Stability of a Sliding Slope in a Railway Cut near Wetteren" reports the studies made after the main slide in 1943. Piezometric readings of the pore water pressure were made and circular slides were studied. The factor of safety was found approximately equal to unity.

S.J. JOHNSON in his paper: "Failure of an Excavation Slope" describes a large failure by the construction of a lock near the Mississippi River. A thick bed of plastic clay was overlying a deep sand stratum which was under hydrostatic head from the Mississippi River.

R.F. TILLMANN in a paper: "Failure of a big retaining wall at "Wienfluss" creek-shore in Vienna, VI" reports a big slide where several houses were involved. The ground water has played an important part in the slide. Tillmann states that the usual analysis gives a factor of safety of abt. 1,5, and warns against confidence in such results. The calculations are not given in detail, but it seems doubtful if the pore water pressure has been correctly introduced.

T.K. HUIZINGA in his paper: "Two failures with cut-off walls" describes failures where the elastic deformation at several sheet pile walls acting in series seems to be the main cause of the failure.

Similar experiences are known from elsewhere, when the cohesion of the clay material is sufficiently high, so that the clay of the upstream side does not follow the deformation of the sheet piling.

C.S. PROCTOR in his paper: "Underpinning and Under-Drainage to check Earth Sliding and Settlements at a large Cement Plant" has described considerable movement of earth caused by ground water in combination with underlying soft mud. The measures taken were exclusion of the ground water from the whole endangered foundation by means of a tight steel sheet piling and an outside drainage ditch combined with a tight paving of the surface.

Among the papers in this subsection is an interesting report on the enormous slides of the Panama Canal: B.V. BINGER: "Analytical Studies of Panama Canal Slides". They result in a graphic representation giving the slope of the cut as a function of its depth, provided a factor of safety of 1,3.

K.S. LANE: "Treatment of Frost Sloughing Slopes" explains the phenomenon of "solifluction" and states that covering of the surface with a thin blanket of pervious soil or cinders is an adequate preventive.

Synopsis of subsection IV c.

Flow slides are caused by extremely loose state of fine sand. A critical density exists, above which the danger of flow slide is imminent. Density can be studied on the site by means of penetration tests.

Fissured clay requires special caution in the application of usual methods of soil mechanics. So far no systematic method of calculation is possible, but empirical data of value for practical construction work are available.

Pore water pressure is an important factor by the stability of slopes. It seems that it is not always taken into consideration in a correct way.

Elastic deformations of sheet piling can often be the cause of crevices on the upstream side, reducing the effectiveness of the sheet piling wall considerably.

IV d. MISCELLANEOUS. (4 Papers).

In this subsection is a paper by W.J. TURNBULL: "Utility of Loess as a Construction Material", in which detailed information concerning the stability of slopes in loess is given as well as concerning the use of loess as a dam material.

It is stated that important engineering structures can safely be built of and on loess soil, but that special methods of design and construction have to be employed.

Great settlements are to be expected and a considerable water content of the soil during placing is to be used so as to prevent cracks during settlement.

A factor of safety of abt. 1,1 - 1,2 is used by the determination of stability of slopes.

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SUB-SECTION IV c

EXCAVATIONS AND SLOPES

IV c 14

DISCUSSION

J.E. LEWIN (U.S.A.)

I should like to discuss the paper IVc 12 of Mr. K.S. Lane.

However because Mr. Malcor mentioned the question of drainage and requested information on this subject, I would like to call the attention to Russian publications.

One book of four hundred pages deals with all the different conditions Mr. Malcor described and covers the problem of various drainage conditions in a very complete manner.

I was reading the paper of Mr. Lane and was very much impressed by it. To the author's explanation of mechanics of frost sloughing I would like to add the phenomenon of build up of ground water pressure under an impervious frozen blanket. Such an impervious blanket is formed if the surface layer contains moisture. This moisture freezes and forms an impervious surface layer. Therefore, I consider the methods suggested by the author to combat frost sloughing as correct, since they eliminate any accumulation of water and subsequently of any ice in the surface layer.

Under the frozen impervious layer the ground water moves freely, although it might be in undercooled state. Such flow of undercooled water under frozen ground had been observed in Russia by Tolstychin. Figure 1 shows plainly a subterranean flow under an arch of ice.

If the frozen surface layer is to thin to resist the uplift pressure of the ground water, a sloughing occurs. However, if at the lower
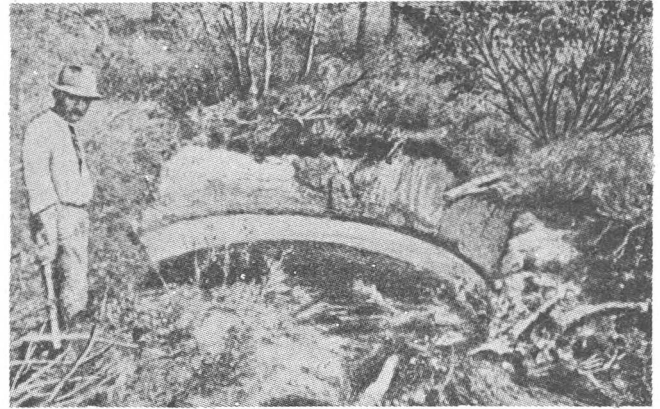

FIG. 1

end of the slope the ground is cut and allowed to freeze, the flow of groundwater is interfered with. Subsequently the ground water pressure is built up and the sloughing occurs. Therefore I consider the provision of drainage duct at the foot of the slope, as shown in author's fig. 3 as objectionable and increasing the danger of sloughing.

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IV c 15

DISCUSSION

F.L. CASSEL (England)

I am discussing the paper IVc 6. Several of the published papers deal with stiff fissured clays, a soil material which has given considerable trouble. Several other papers refer to this soil as well. There is general agreement that these troubles are caused by the progressive deterioration of these clays with time. I should like to make a few remarks on Prof. Skempton's paper. By following a very interesting line of thought, the author has tried to find an empiric expression for the rate at which this deterioration proceeds.

If such law existed it would be of great interest, not only for the Scientist, but also for the designing Engineer, as well for the design of the slopes as for the remedies for slips and slides. The results presented in the paper, however, do not agree with other observations. My own experiences are probably numer-

ically more limited than those of Prof. Skempton. But they extend from plastic LONDON clay to very dry and hard Liad clays, while Prof. Skempton's examples, at least those described in the paper, are exclusively drawn from London clay, probably mainly plastic, as described in my own paper. These examples with the exception of the Watford Bypass slip, describe slips behind retaining walls of relatively low height, and not slips in unsupported cuttings. All the slips, with the exception of the Wembley case occured after a relatively short period. This case, however, lies back so long that no proper soil-mechanical examination either in laboratory or at the site exists.

I have tried to use the notions of Prof. Skempton's paper for the slips in Lias clay which are described in my paper IVc 5 and I found that they do not fit into the scheme

	Age years	Average depth of slip below slope ft.	Average depth of slip below original surface ft.	Slope of cut	Calculated mean shear strength at failure lbs/sq.ft.	Original shear strength
Toddington	40	14-15	36	2.5:1	365-385	3600-3700
Hook Norton	70	15-16	45	2:1	410	1400-2200
Hullavington	44	10	30	2:1	250-310	2800-3200

It will be seen that it would not have been possible to predict the slips or the time of their occurrence by the use of Prof. Skempton's formules or curves.

In a large area, containing numerous cuttings in plastic London clay, very few deep seated slips occured. Most were shallow, indicating creep and continuous surface movement. Considering the great lengths of cuttings and the innumeral retaining walls in that area, of the same depth and age, which have not failed, it becomes obvious that other factors play a very considerable role.

One of the author's examples, the "Mill Lane" failure, is also mentioned in another paper (VIIIf 2) where it is pointed out that the calculated shear strength was 5.5. lbs/sq. in., much less than the real shear strength found in numerous tests, a proof that no general decrease in strength with depth, but reduction of strength in a fissured zone had occured.

The main point I wish to make therefore, is that the assumption made by Prof. Skempton that the deterioration of the stiff fissured clays proceeds uniformly at a definite rate from the surface to the depth is not proved at all, and is probably not correct. The deterioration is not only, and not even prevalently, a function of time and depth - (which depth, that below surface or that below original surface?) - but of many factors, of which I mention only the topography of the site, the climatic and seasonal influence of precipitation, the local ground water movement, the amount of drainage provided and others.

But in addition I wish to point out that even if it could be ascertained, by calculation, by tests, or by observation, to what ultimate low value of shear strength a particular clay can decay or at which low value a failure would occur no economic design could be based on such low values. 40 or 70 ft. deep cuttings just cannot be designed for slopes 7:1, at least not in European countries, nor would retaining walls of that height, able to resist the pressure of such softened clays, be economically possible. Remedies must be looked for to prevent or slow down the natural deterioration, by altering the variable factors, which are not depth or time.

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IV c 16 NOTES ON THE POSSIBILITY OF APPLICATION OF THE METHODS OF SLOPE STABILITY CALCULATIONS ON A GROUND NON STRICTLY HOMOGENEOUS
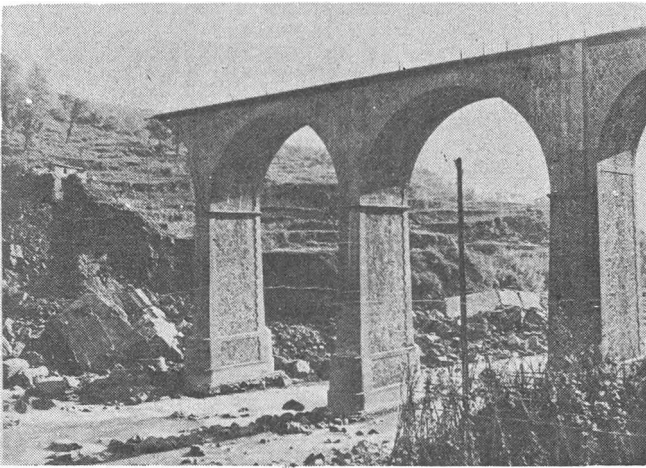
J. ABOLLADE Y ARIBAU (Spain)

It is on July 1942 that the writer received the order of proceeding to study the possibility of consolidation or rebuilding of a beautiful viaduct on the Fluvia River (province of Gerona) by the Pirinean Range, whose left abutment and first pile of this side were turned down on the extraordinary rains of the spring 1940, which so many ravages caused on the Catalan Region.

The problem was a definite one and it was necessary to calculate forces and stresses to make possible the project of slide resisting structures to confront them, on the economic standpoint mainly, with the building of a new bridge. The ruined structure has 30 m high and three arches 18 m width on the centre, with one of 14 m on the left bank and three on the right one of the same width.

There were many incidents on the foundation period on the left bank and the characteristics of the ground very discused but the viaduct was finished and the traffic on it commenced in 1903. Moreover in the following years many small motions on the left abutment were observed and in the year 1929 some soundings on ground and foundation were made.

The stormy rains of 1940 performed two different effects, the water saturation of the clayey marls of the grounds and the foot attack of the slope by the river. The viaduct went away. The region where the bridge stands is a very interesting one from the geological standpoint because it is one of the scarce volcanic regions of Spain and the basaltic formations have a great development. The right abutment is at the foot of a basaltic colum-

The Viaduct and the left bank after the slide

Bed circle. Cohesion only } Swedish method
Bed circle. Cohesion and friction }
Frontard's cycloid.
Soundings curve.

nade of almost 100 m high but the left one
is on the eocene "flisch", the geologic ground
which has produced more griefs to the engi-
neers. The words "flisch" means in some Swiss
dialects sliding ground.

On the case we are also on a techtonic
zone so many fractured, that it was found
many faults and discontinuity on the strata.
The succession of sandstones and limestones of
a maximum thickness of two meters each is in-
terlayed with thin ones of clayeys marls of
some decimeter of thickness.

On the doubt of the method of investiga-
tion to be followed it was decided to realise
a complete series of coreborings and try to
deduct from the form of the slide-curve ob-
tained the possibility of coming at conclu-
sion worth of confiance.

It is interesting to verify how, in our
case, are applicable the admitted formulae for
the study of slides on hillsides not only for
its own value as pure mechanics but as a con-
firmation and complement for the geologic
standpoint which is furnished by the core-
borings. This is as much important as the con-
sequences infered from the comparison of the
different methods of employed calculation al-
low us to judge on the validity of every one
and to resolve on the applicability of the
more adopted at the peculiar conditions of our
case.

These ones are so special, we think, that
it may be dubious if the classic calculation
and his basic hypothesis are utilizable.

There is question of an alternate succes-
sion of different rocks of so distinct hardness
that it seems it is not allowed to employ the
methods based on the mass homogenity and where
the constants of friction, coefficients of
cohesion and density are uniform. It is clear
that if the ground dimensions affected by the
slide would be smaller ones and comparable
with the strata thickness, this would be exact
and there is not anything to do, but as this
is not the case, we think it is possible to
realize the study.

The first produced effect is a series of
massif settlements which produce the fracture
of the differents strata, which are converted
on anything as a true rockfill with more or
less voluminous blocks but which are very much
smaller if we consider the phenomen on its
whole.

The second is the clayey character of
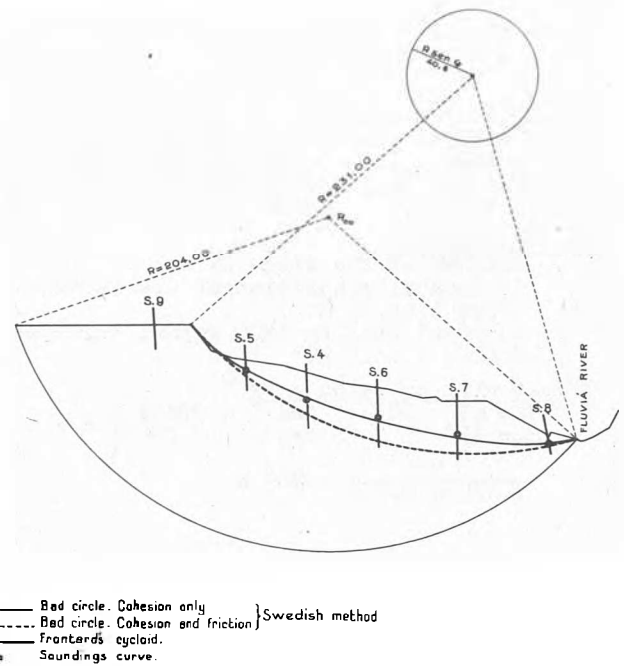
the marls and these ones fulfill after the
settlements all the orevasses and fractures,
making, finally, a sort of moving concrete.

It is clear, also, that the obtained
sliding surfaces are not so uniform in the
ground as given by the calculation and it is
found some undulations and breaking points on
the passage through blocks, but if the general
dimensions are a great ones the average sur-
face will not much differ from the real one
if the assumed hypothesis are well adopted.

Finally the existence of a stratification
whose dipp is roughly parallel to the hillside
gives a tendency at a sort of polarisation of
the stresses in the direction of the marls,
which is the feeblest and more unstable part
of the whole, and, over all, the more sensible
at the water imbibition; its characteristics
being very variable with the amount of con-
tained water.

In conclusion, it results that if we were
obliged to determine directly on the core cy-
linders from the borings the physical con-
stants of the ground, we will obtain very dif-
ferent results and will be very difficult to
precise an intermediate value utilizable. The
study, on the whole, with the borings aide
which we go to do, will enable us to say not
the exact value of the constants neither its
median value, but if the grounds behavior is
of a homogeneous equivalent mass or, if hetero-
geneous, its degree of heterogenity is less
than the dimensions of the sliding hillside
would permit and tnis with so much reason than
the obtained results, as we will see at the
final of this paper are satisfactory concor-
dant.

Two are the processes of calculation we
have used, based on the paper of Entrecanales
(On stability of ground slopes in coherent
soils; Revista de Obras Públicas, Madrid 1941).
These two are the swedish method of the very
bad circle which has justified its name be-
cause the obtained dimensions and depth of the
slide are greater enough that the real ones
and the Frontard's cycloid whose exactitude
is very much satisfactory, as we must hope
because the later curve is deducted on the

hypothesis of Rankine, where the pressure on a vertical plane is parallel to the hillside, it is to say, in our case, to the stratification. The discrepancy with the result given by the core-borings is on the order of two meter at most, on testborings of 25 m, it is to say less than 8% which is acceptable. On the whole the surface determined by the testboring are of less depth than the Frontard's cycloid, but this is favourable to the stability.

Swedish Method

Heighth of the slide 77 m

Horizontal projection of the maximum slope line 35 m

Supposed density 1800 kg/m^3; tgi = $\frac{77}{235}$ = 0,327; i = 18°10'

Circle with cohesion only

α = 59° 30'   sen α = 0,862

ψ = 13°   sen ψ = 0,225   } R =

$$\frac{77}{0,225 \times 0,862} = 204 \text{ m}$$

height with the horizontal projection we have found as the most probable value of 14° 30' for the friction angle and 0,28 kg/cm^2 for the cohesion (See quoted paper).

We could write at the following lines the complete development of the analytic computation of the determined points of the Frontard Cycloid; with the supposed constants, employing the well known parametric equations. The quoted number of words of these papers make more useful to consign only the results.

Maximum thickness of the wedge

$$\varphi_1 = \frac{C}{v} \frac{Cos}{sen(1-\varphi) \cos i} = 24,99 \text{ m}$$

sensibly lesser than the swedish method and more approaching to the reality as we will see.

As a final summary we present on the following table the depths which on every boring gives the swedish method, the Frontard's and the analysis of the core samples with the correspondent differences

Borings	Samples	Swedish	Differences	Frontard	Difference
5	17,00	22,00	+ 5,00	17,00	0,00
4	25,00	35,00	+ 10,00	24,00	- 1,00
6	23,00	38,00	+ 15,00	25,00	+ 2,00
7	19,00	30,00	+ 11,00	21,00	+ 2,00
8	8,00	10,00	+ 2,00	9,00	+ 1,00

Cohesion factor

$\frac{Co}{V_h}$ = 0,145; $\frac{Co}{V}$ = 0,145 x 77 = 11,165

Co = 11,165 x 1800 = 20097 kg/m^2 ↔ 2,01 kg/cm^2.

Circle with cohesion and friction

After some trials on the abacus of the quoted paper the very bad circle obtained which passes by the foot and crest of the slide with cohesion and friction joined is as follows.

R = 2h = 231; $\frac{C}{Co}$ = 0,15; c = 0,15 x 2,01 = 0,301 kg/cm^2

φ = 12   tg φ = 0,212

We have traced on the profil this very bad circle, which gives a maximum depth below the unslided ground slope before the slide on 38 m.

Frontard Cycloid

After some trials to adjust the total

As a resume of this tentative paper we think that Frontard curve is well adapted to the cases where there is a stratification parallel to the slope and that on the routine practice the swedish method gives a security coefficient very much greater than it is commonly supposed.

It may be also interesting to state that the conclusion of the study of the viaduct problem led us to the building of a new one as the slide-resisting structures on the less conservative hypothesis of the Frontard cycloid led us to a true bridge on reinforced concrete sunk on the ground with a probability from resisting a new slide least enough to that a new bridge of smaller highness and of isostatic type.

The determination of pressures and stresses on the slide were made with a graphical method. The area affected was divided on vertical strips on which were placed its soliciting forecits weight, the cohesion and friction. A funicular poligons give us the desired forces.

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IV c 17

W.H. WARD (England)

In his report, Dr. Bretting, refers to my paper "A coastal landslip". He seems to have gained an incorrect impression of the facts. The fine sand in its original state was quite dense and it was only subsequent to internal erosion and after movements of the upper cliff had taken place, that the sand became loose and liquified. All the clays were originally stiff and it was only after considerable landslip movements had occurred that the clays progressively became softer.

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SUB-SECTION IV d

MISCELLANEOUS

IV d 5

DISCUSSION

D.P. KRYNINE (U.S.A.)

I am going to make just a few little remarks about the papers IVd 2, Vb 2, Vb 3, VIa 10, and VId 5.

In analyzing stability of vertical slopes, both supported by walls and unsupported, extreme importance should be paid to the condition of statics $\Sigma M = 0$. To satisfy this condition a part of the earth mass adjacent to the vertical slope may carry a load considerably different from that of gravity.

In the case of an unsupported vertical slope, to balance the moment caused by the lateral pressure H and its reaction H, a couple formed by the vertical forces V – V (fig. 1) is needed. In this way the vertical slope AB carries a load higher than $h\gamma$, where h is the height of the wall and γ the unit weight of the earth material. If the mass is enclosed in a box as in the experiments by Mr. Wynne – Edwards (page 65, vol II of the Proceedings of this Conference) the lateral pressure H is the passive resistance of the wall opposite to that removed. The free face AB is carrying in this case a load exceeding 112 x 10 = 1120 lb. Apparently this is the cause of the steepness of the failure line (51° instead of 45°)

An interesting settlement pattern was observed by Mr. L.F. Cooling in the Oxford silo (fig. 4, page 136, vol II) when the settlement was at a maximum at the ends of the wings, but the settlement diagrams for empty and full bins were not parallel. Apparently, due to the rigidity of the interior wall of the wing (next to the tower) there was a difference in passive resistance of the walls acting from the tower. This difference corresponds to force H in Fig. 1. Hence the overloading of the ends of the wings when the bins were full.

The theory of transfer of pressures by shearing forces should be applied to fig. 3, page 169, vol II. In this case the shearing stresses acting upward pick up the weight of the hanging middle part and transfer it to the supports, the interior edge of the support being considerably more loaded than the exterior. This small remark is not intended to
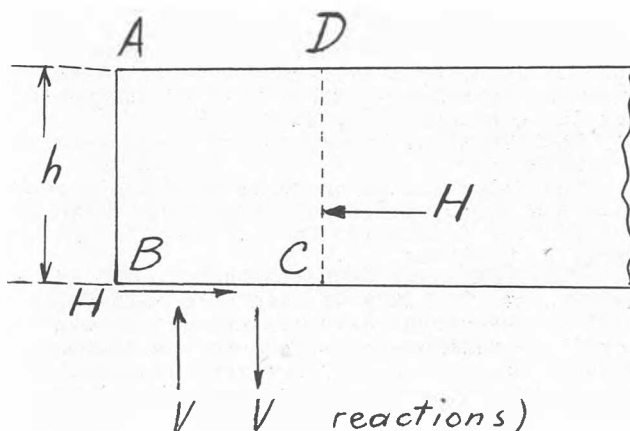
Forces acting on part ABCD of an earth mass, besides its weight

FIG. 1

decrease the value of the otherwise interesting and important paper by Mr. K.W. Mautner.

As to the pressure on flexible walls, the speaker believes that a flexible wall is that producing deflections large enough as to mobilize considerable shearing stresses and thus redistribute the conventional triangular lateral pressure. In this way such definitions of a flexible wall as a wall deflecting $\frac{1}{2}$ or some other percent of its span, should not be used. In the opinion of the speaker, what counts is the absolute value of the deflection not a percentage of its span. A wall may be flexible, but its model deflecting the same percentage of the span, may be rigid.

Without entering into details of the large and interesting research work done in the Princeton University (paper Vb 2), the speaker shares the opinion of the author of that paper as to the stress pattern shown in fig. 3 a, page 82, vol II.

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