

THE STABILITY OF SLOPES

IN

OPEN CAST MINES

AND

THE ECONOMIC IMPLICATIONS THEREOF.

A DISSERTATION

PRESENTED TO THE DEPARTMENT OF MINING ENGINEERING,
UNIVERSITY OF THE WITWATERSRAND.

IN PARTIAL FULFILMENT OF THE REQUIREMENTS
FOR THE DEGREE OF MASTER OF SCIENCE
IN ENGINEERING.

SUBMITTED BY:

DAVID CHARLES BRINK
B.Sc.(Eng.)(Rand).

JANUARY, 1962.



DECLARATION BY CANDIDATE.

I, DAVID CHARLES BRINK, hereby declare
that this dissertation:

- (a) is my own work, and
- (b) has not been submitted for a
degree to another University.

Signed . *D. Brink* .

S U M M A R Y.

The author presents in this dissertation a review of the knowledge to date on the subject of slope stability in open pit mines.

After an introductory discussion the problem is presented as it would appear to engineers in general as opposed to specialists. This is followed by a chapter on the importance of geological and ground water investigations as a tool for the solving of the slope problem. The more sophisticated details of the problem are dealt with in the chapter on soil mechanics, which is followed by more general discussions on the link with rock mechanics and the influence of underground excavations below or adjacent to pit slopes.

Finally, the economic aspect involved in planning and controlling slope angles in open pit mines is considered. This part of the problem is often neglected by investigators although it is one of the motivating factors behind the investigation of the problem itself.

ACKNOWLEDGMENTS.

The author wishes to acknowledge the assistance given by the following persons:

Professor J. E. Jennings and Professor R. A. L. Black, joint supervisors in the preparation of this work, for invaluable guidance and assistance throughout the work.

Dr. G. E. Blight for the soil tests and for work on the stability of the slopes.

Mr. L. P. Nutt for assisting with the mathematics relating to the economic aspects of the problem.

The Management and Staff of the NChanga Consolidated Copper Mine for kind assistance and co-operation throughout the initial investigation of this problem.

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CHAPTER 1.

INTRODUCTION.

(i) STATEMENT OF THE PROBLEM

It is required to determine at what angle any slope in natural ground should be cut so that it should be stable and safe in all respects, yet not over-safe to the detriment of the economics of the excavation.

In some cases, the most economically advantageous slope, for the mining of a mineral deposit in stable ground, is of such a flat angle that the problem of safety does not arise. It is where we require to cut steep slopes, for economic reasons, in unstable ground that the engineer is faced with a most tricky optimisation problem; a problem which is often made complex due to the vague definition and difficult evaluation of the parameters involved.

(ii) DISCUSSION

This work is based predominantly on the information obtained by the author while taking part in a slope stability investigation carried out on a large open pit situated on the Northern Rhodesian Copperbelt. The investigation is still in progress and will be for a number of years. Although a final solution, which can be used for design purposes, has not yet been found, a great deal has already been learned. This is an attempt to record the state of

knowledge of the subject that was attained during the year of 1961. Notwithstanding the above remarks, this is not a case study. The subsequent remarks have been expanded wherever possible with a view to general application.

The reader will soon note a heavy bias towards the use of soil mechanics theory as opposed to that of rock mechanics. This is due partly to the greater suitability of the former theory to all problems of this kind which by their nature deal with unstable material; and partly due to the fact that the investigation discussed above was intimately concerned with slopes cut in a predominantly soil material.

CHAPTER 2.

SLOPE STABILITY IN OPEN PIT MINES.

In all ground slopes whether natural or man-made forces exist which tend to cause movement of material from high points to low points. Further, nature teaches us that where the ground is strong it can support the slopes of mountains and gorges. Whereas, in areas where the ground is weak, the terrain is flat because the ground in these areas cannot support slopes. It is where man creates slopes which are steeper than the natural slopes in a particular type of ground that slope stability problems arise.

The problem of stability of slopes is one that has received attention for many years. This is understandable considering the dangers of potentially unstable slopes and the economic disadvantages of "over-safe" slopes. Much of the work done in the past has been directed towards attempted classifications of slope failures. It seems that as the details of cases have been examined the resultant classification systems have become increasingly complex. The use of such classification systems is extremely limited particularly if they include a high degree of complexity.

A better, although more difficult, approach to the problem may be an attempt to really understand the fundamental theoretical causes of slope instability, to evaluate their importance and relative magnitudes

and finally to analyse their effects. Once engineers have this knowledge at their disposal the problem of various types of failure will resolve itself to the simple process of selecting that part of the known stability theory found to be most applicable to the problem. Classification of the different types of failure then would serve only to aid the engineer in determining clearly in his own mind what has taken place, i.e. he would be able to "state the problem", and then the formulation of a solution to the problem would be a matter of course. For this purpose classification systems can be simple and of a general nature.

It has long been recognised that if a soil has some measure of cohesion then the safe slope angle may vary from 90° in the case where the vertical height is small, or moderate, to a flat angle where the height is large. Further, it is also known that time plays an important part and the safe slope angles may be considerably reduced when long term stability is important. Broadly, the whole of the previous statement applies to rock slopes as well especially if these are fractured to any extent. This comparison of behaviour between soil and rock masses will be dealt with more fully in the chapter dealing with the rock mechanics aspect of the problem.

To get some idea of the economic savings that might be brought about by pursuing this problem of

slope stability a little further the following examples provide some food for thought.

"One million extra tons of waste would have to be mined as a result of an average slope being reduced by 1° in a pit 1000 ft. x 1000 ft. x 400 ft. deep. This, of course, is not particularly large when a typical pit in the south-western United States of America is considered. Here 4000 ft. x 5000 ft. x 1000 ft. deep would not be unusual dimensions, in which case a 1° change in an average slope would involve 20 million tons of material and a 10° change would require an extra 220 million tons to be mined". (5)

After indicating the importance of determining, accurately, the critical angle of slope for an open pit mine the logical sequence is to think immediately of what factor of safety should be applied to this slope. Taylor (2) describes the difficulty involved in choosing a factor of safety and also the different types of safety factors that can be applied. For simplicity, and suitability of practical application, it has been decided in this work to employ the factor of safety with respect to shearing strength, denoted F_s . The theoretical aspect of this will become clear in the chapter dealing with the soil mechanics aspect of the problem of slope stability.

The factor of safety, F_s , is normally taken as 1.5 in the design of slopes such as earth dams. In construction work of a temporary character where a

reasonable failure risk may be accepted it is usual to take a lower value of F_s . The selection of the value of F_s to be used is important since the greater the safety (higher F_s) the flatter must be the slope. A value recently adopted by Professor J.E.Jennings (3) in this type of work is thought to be a reasonable one. The following is quoted:

"An open pit mine may be considered as a temporary engineering construction and hence a value of F_s less than 1.5 may be adopted. In normal construction the factor should not fall much below 1.3."

By "temporary engineering construction", in the above quote, is meant work which is attended by engineers in the course of construction. A lower factor of safety can be accepted in this case because the work is under observation and if anything tends to go wrong the men and machines are on the site to take immediate action to remedy it.

Slope failures seldom if ever take place without giving some kind of warning. Whether or not the warnings are recognised and taken heed of is another matter. Engineers concerned with slope construction or the cutting of slopes in natural ground should therefore familiarise themselves with the types of warnings which are likely to expose an imminent failure. The most common type of warning is the formation of tension cracks at the top of the slope.

In some cases diagonal tension cracks may become evident on the side of the mass in which failure is about to take place. Movements of the potentially unstable mass if detected also provide adequate warning. The best way of detecting such movements is by carrying out routine survey traverses across the potentially dangerous area, starting from remote bench marks. Imminent failure is usually indicated when the deflection-time plot shows a reversal from decelerating to accelerating movement. Other more sophisticated methods of obtaining warning are by means of installing stress-meters in boreholes and measuring the stress variation with time. Any sharp increase in stress would indicate that all was not well. Slope indicators, too, are useful in this respect. Although they are primarily used to determine the position of the surface of movement, they also indicate any movement of the mass which might be taking place.

After taking the initial piezometer readings in boreholes near a slope for determining values of porewater pressures for design purposes it may be wise to continue these piezometer readings on a periodical basis as it is a valuable means of detecting any considerable change in seepage and porewater pressure conditions, the consequence of which is discussed fully in the chapter dealing with the soil mechanics aspect of the problem.

Classification of slope failure types has been mentioned above and it was decided that for the

purpose of this work a simple system would be settled for. In fact it has even been decided to avoid a distinction between rock and soil, which would be arbitrary anyway. Four types of incipient failure are considered.

(i) FAILURE BY ROTATIONAL SHEAR.

This is probably the most important type of failure. It forms the theme throughout this dissertation and for that reason there is no point in expanding on it here.

(ii) PLANE SHEAR.

Masses of rock and soil very often fail in shear along a plane. Failure occurs when the shear stress exceeds the shear strength along such a plane within a slope. Failure usually takes place along some plane of weakness in the mass, usually of geological origin. [See Fig. 4 (a)]

(iii) FLOW FAILURES.

This type of failure is not readily susceptible to analysis and fortunately is seldom encountered in mines.

(iv) ROCK FALLS.

This is a problem which is generally easy to overcome. It is usually caused by the scaling of an otherwise competent slope or else loose material falling down a slope which is made steeper than the natural angle of repose for the loose material.

This type of failure is usually of no considerable magnitude and would seldom if ever be catastrophic. [See Fig. 4 (b)]

The listing of methods for the prevention and control of slope instability fall under the same criticism as that leveled at classification of slope failure types. While it is necessary to know the tried and tested methods for control of unstable slopes, no really effective control measures can be applied to any case without understanding the basic underlying principles of the problem. The reason for this is that the obvious methods of control are not always the most effective, or effective at all for that matter; and obscure methods exist which might provide relatively simple solutions. The validity of this will be realised while reading the section in the soil mechanics chapter dealing with water problems.

C H A P T E R 3.

THE IMPORTANCE OF STRUCTURAL GEOLOGY AND
GROUND WATER INVESTIGATIONS.

The problem of slope stability is more difficult when one is dealing with slopes cut into natural soils or when failures occur as landslides in natural mountain slopes. Most of the research work to date concerns the design of artificial slopes such as are constructed in earth dams, but nevertheless information is also available on the mechanics of failure of slopes cut in natural soils. The principal difference between natural and artificially formed slopes lies in geological features. In the former case one has to deal with natural variations in the soil and with fissures, joints and laminations which are beyond the engineers' control. In applying theory which has been devised for a slope composed of homogeneous soil to these conditions it is therefore necessary to be cautious and to try to make allowances for the geological variations.

Stability computations of natural slopes are as accurate as the knowledge of the properties of the materials which make up the slope. This summarises the problem which engineers face when tackling a slope stability problem. Sophisticated methods exist for analysing the mechanics of slope failures but the information used in these analyses is usually severely lacking in even modest accuracy.

Geological information is essential whether slopes are to be designed by traditional rule-of-thumb or by theoretical analysis. Dip, strike, and location of all bedding, schistosity, laminations, dykes, sills and faults, should be known. As much information as possible should be obtained on direction and filling of joint systems. The location of breccias, particularly weak and unstable materials, and of permeable layers should be given attention. (5)

To obtain the information described above extensive investigation programmes sometimes have to be undertaken where complex geological structures exist. On the other hand investigations may be a relatively simple matter in areas where the geological features are of a regular nature. A comprehensive investigation may consist of the following parts:-

- (a) Topographical and geological survey
- (b) Geophysical subsurface investigation
- (c) A drilling and sampling programme

The three sub-sections will be expanded below:-

(a) Topographical and geological survey

The observation of cracks, ground movement, soil creep causing tilting of trees and telephone poles, etc. should form part of the survey together with a detailed study of surface water conditions. These observations should be additional to the normal surface geological mapping of soil and rock types,

outcrops, etc. Aerial photography is frequently useful for general topographical surveys as features such as changes in vegetation, which sometimes indicate important geological trends, become obvious.

(b) Geophysical subsurface investigation

This method is often warranted for the determination of structural features such as the position of the bedrock surface or the location of large faults. It is, however, a relatively inaccurate method and should only be considered for determining general trends.

(c) Drilling and sampling programme

This is an aspect of the geological and ground water investigation which can yield invaluable information. Not only does it yield information with regard to stratigraphic and structural details, but also with regard to ground water conditions and strength properties of the materials encountered which are determined from samples taken.

Porewater pressures measured by means of piezometers lowered down boreholes yield information concerning ground water tables, the presence of perched water tables, and if the measurements can be taken accurately enough, flow nets can be worked out. Considerable difficulty is experienced with piezometer measurements in relatively impermeable material because although fairly high porewater pressures may exist there is such

a slow rate of flow that measurements taken with instruments which depend on flow taking place are impractical. The development of an effective "null-flow" piezometer for this type of work would greatly facilitate investigations.

During the slope stability investigation of an open pit which had slopes consisting predominantly of highly decomposed material it was realised that "undisturbed" samples, of the materials which were taken for the purpose of strength determinations, from deep boreholes, were useless. This was because the techniques available for cutting and retrieving the samples from the bottom of the boreholes were unsuitable and did not adhere sufficiently to requirements for undisturbed samples. A surface exploration of the materials in the pit was subsequently resorted to. Figs. 5 - 7 show three soil profiles drawn on the North slope of the open pit. These were recorded in accordance with recommended practice in South Africa. (9) The information obtained from the soil profiles provided a valuable link with the general stratigraphy of the area. Further, while taking the profiles, information was obtained relating to jointing and the concentration thereof.

Undisturbed block samples of the materials in the pit were also taken along the same section lines as the

soil profiles were taken. See Figs. 8 - 10. The samples were taken from 1 - 2 feet below current bench levels. The dimensions of the samples were: diameter 14" and height 8". In taking the samples rigid precautions were adhered to, to obtain undisturbed specimens and the samples were immediately sealed, with a $\frac{3}{4}$ " layer of a 60 : 40 mixture of paraffin wax and petroleum jelly in heavy wooden crates and then dispatched for testing at the Civil Engineering laboratories of the Witwatersrand University. The results of the tests are given in appendix IV.

The familiarity that the engineer gains with the soil properties and geological features of a particular site, whilst carrying out the profiling and sampling aspects of the investigation, is invaluable. It holds him in good stead if he is called upon to apply safety factors to laboratory or other strength determinations. After all the careful testing of materials and the sophisticated stability analysis, the engineer's judgment and experience will probably still carry the most weight and it is the cultivation of this judgment during the investigation which is important.

G.P.Tschebotarioff (10) makes some valuable comments relating to geological investigations for engineering purposes. "Few engineers can attain the skill which is often necessary to properly interpret complicated local geological features, since such skill can be attained only through constant practice." The closer

co-operation of geologists is therefore encouraged in all major problems where geological features and ground water conditions are important. Geologists can detect much more quickly than engineers the presence of local geological features of engineering importance, such as dangerous faults in the bedrock or pervious, water-bearing, or soluble veins.

C H A P T E R 4.

THE SOIL MECHANICS ASPECT OF THE PROBLEM.

(a) General discussion of theory.

The forces existing in a ground slope are predominantly the component of gravity and the forces caused by the presence of water behind the ground slope. These forces are resisted by the shear strength of the slope material. The available shear strength (S) of the material is given by the Coulomb equation where:

$$S = c + \sigma \tan \phi$$

c = the cohesion of the soil

ϕ = the angle of internal friction of the soil

σ = the intergranular pressure in the soil
normal to the plane of failure.

A slope in a homogeneous cohesive soil tends to fail by a mass sliding down a surface which is fairly close to a cylinder. If the slope is long then a slice of unit length may be taken giving a circular slide of the type shown in Fig. 11. The stresses in the soil at the top of the arc (ZONE I, Fig.11) tend to approach "active" pressure conditions with the development of tension cracks in the upper surface. At the bottom of the arc (ZONE III, Fig.11) the soil stresses tend to approach the "passive" conditions.

Active and passive pressure conditions refer to the limiting relationships which can exist between horizontal and vertical pressures in a soil mass.

In the active state the soil is yielding horizontally: the horizontal pressures will be minimum and this will usually result in some tension near the surface. In the passive state the soil is compressed horizontally and horizontal pressures will be maximum.

Let us consider a slide along the arc of a circle, length l , with centre O . See Fig. 12.

Before continuing, the method of stability analysis to be used will be discussed.

The method of stability analysis adopted by the author is that known as the Swedish method of slices. It was developed originally by W. Fellenius and is based on the statical analysis of the mass above any trial failure arc, with this mass considered to be made up of a number of vertical slices. The reason for the adoption of this method of analysis is that, in the author's opinion, it offers more "flexibility" than most other popularly used methods. The "flexibility" refers to the problem of analysis of natural slopes where special geological features have to be taken into account, and indeed they can be, using the method of slices. The inaccuracies that are introduced by the use of this method as compared to the more sophisticated methods are on the conservative side. Finally the method of slices lends itself to simple graphical solution which sometimes eliminates tedious computations.

At this stage it is assumed that there is no water in the soil forming the slope. The effect of ground water will be dealt with later in the development of the theory.

In Fig. 12:-

l = length of failure arc

c = the unit cohesion per unit of slice width acting along the arc of sliding

N = the force, normal to the sliding arc, which induces friction

W = the gravitational force

T = the slide inducing force for each slice.
It is tangential to the arc of sliding at its point of application.

The slide resisting force = $c + N \tan \phi$

$$\begin{aligned} \therefore \text{Factor of Safety } (F_s) &= \frac{\text{shearing resistance}}{\text{shearing force}} \\ &= \frac{c.l. + N \tan \phi}{T} \end{aligned}$$

In this way the factor of safety for any slope may be determined given the remaining parameters.

[NOTE: $N = \sigma \times \Delta l$ IN THE ABOVE ANALYSIS]

Instability of open pit mine slopes often results from excessive hydrostatic uplift pressures and seepage forces behind the face (4). The presence and the magnitude of these hydrostatic uplift pressures and seepage forces is very often deceptive to mining engineers

in general especially in pits with slopes cut in cohesive soils of low permeability. Often in these cases the flow of water by seepage is so slow that it is lost by evaporation as soon as it emerges on the face of the slope.

When there is water in the soil behind a slope, part of the total weight of the soil is carried by pressure in the porewater of the soil. This gives rise to a porewater pressure, usually denoted by u , in the soil and this pressure contributes nothing to the shear strength of the soil along the sliding arc. This can be viewed as a buoyancy effect and the Coulomb equation is rewritten:

$$S = c' + \sigma' \tan \phi'$$

where σ' is the effective or intergranular pressure in the soil which is equal to the total pressure σ , minus the porewater pressure u .

[This is stated in Terzaghi's effective stress law $\sigma' = (\sigma - u)$. c' and ϕ' are the cohesion and internal friction angle parameters with respect to effective stresses.]

Alternatively $S = c' + (\sigma - u) \tan \phi'$

If in addition, the water in the pores flows through the soil under some pressure gradient, then the resistance to flow offered by the soil particles results in a

reaction force on the soil which is called the seepage force.

Fig. 13 shows the consequence of a seepage flow from an elevated water table into a cutting. The resultant seepage force is denoted P and the porewater pressure at any point on the underside of the sliding arc is denoted u .

D.F.Coates and A.Brown (5) state that this mechanism is particularly noticeable after heavy rain-storms and spring runoffs and claim that in extreme circumstances the shear stress can be increased by as much as 24% of the pre-existing stress. Put in another way, the effect could be equivalent to an increase in slope angle by as much as 13° .

Water behind rock slopes seems to be just as critical as for soil slopes, especially in zones which are highly fractured and fissured. S.D.Wilson (4) quotes one case where faulting and jointing and water pressures combined to cause a failure in a rock pit at the low slope angle of 21° .

D.F.Coates and A.Brown (5) quote the case of the famous Frank slide where a block-flow failure occurred in ground which was jointed and where the joints were filled with water. The result was that 80 million tons of rock moved $2\frac{1}{2}$ miles across the valley in less than 100 seconds.

In the above discussion porewater pressure and seepage forces were mentioned as separate entities. In fact they are dependent upon one another. D.W.Taylor (2) proves that in a stability analysis the actuating force may be taken as either the combination of total weights and boundary neutral forces or the combination of submerged weights and seepage forces. In the following analysis the former will be used. It has been argued in some circles that in the Swedish slice method of analysis no account is made of the intergranular forces acting on the sides of the slices because these are statically indeterminate. This is true. Further it appears from the geometry of a slope that these forces are unbalanced in favour of the actuating force, hence reducing the factor of safety. If, therefore, in the analysis we take account of seepage forces in obtaining the resulting actuating force, we will take account of the unbalanced side forces and will be on the "safe" side. This is an argument that designers of slopes might well bear in mind.

The actual stability analysis theory will now be developed further.

In Fig. 14 a section is shown which is assumed to be of homogeneous soil through which a steady state of seepage is occurring as represented by the equipotential lines, which are shown by dashed curves. This section has been arbitrarily divided into five vertical slices of equal width. Let it be assumed that the circular

arc on the section is an arbitrarily chosen trial arc.

From the equipotential lines the neutral pressure may be determined at any point on the section. Having determined the neutral pressure at a number of points along the trial arc a pressure head diagram may be drawn. This is shown as the curved line below the trial arc. On the sides of the slices pressure head diagrams are also shown.

For the steady seepage case with the flow not known, the vectors representing neutral forces on the sides and bases of the slices are completely defined, and from the pressure head diagrams shown in Fig. 14 these vectors may be determined.

The assumption made in this analysis is that the lateral combined forces are in balance on the sides of each slice. The use of this interpretation of the analysis does not require the determination of the neutral pressures on the sides of the slices, and thus it saves a considerable amount of labour. Since it gives conservative results, (2) by reducing the total resisting force, it is probably the more satisfactory procedure for general use.

In this case the factor of safety

$$F_s = \frac{c.l. + (N - u) \tan \phi}{T}$$

where u is the total force of the soil water exerted

on the bottom of the soil slice.

It is thought unnecessary to enter into a detailed description of an actual stability computation here. Descriptions of these types of analysis are readily available in most good textbooks on soil mechanics.

(b) Shear Strength Predictions.

(1) Laboratory Tests.

During the course of a stability investigation on a certain pit the author took some undisturbed soil samples from the various strata which were present in the pit slopes. The samples were sent to the Civil Engineering Department of the Witwatersrand University for testing. The effective stress parameters c' and ϕ' were determined in saturated-drained triaxial tests on specimens cut from the undisturbed samples.

No rules can be set up for the methods to be used for testing different types of soils. The advice of soil mechanics specialists should be sought in each case to determine the type of triaxial tests to be carried out on undisturbed soil specimens. The tests should be as closely representative as possible to the conditions pertaining in the natural soil, at the site under investigation. In this case the method of testing used was that recommended by Bishop and Henkel (12) with recent modifications by Blight (8).

The test results are given fully in Appendix IV and may be summarised as follows:-

STRATUM	No. of Test Specimens	From Saturated-drained triaxial tests	
		Parameters	
		c' lbs/ft. ²	ϕ' °
No.3 Reddish-grey firm jointed clayey silt	4	720	37
No.4 Pinkish-grey soft & firm jointed micaceous clayey and sandy silt	4	720	34
Mean Design Values		720	35°

No further discussion will be entered into on the subject of laboratory testing for determining the values of c and ϕ . Instead the author wishes to discuss the importance of observing natural phenomena with regard to obtaining further information of the soil strength characteristics. These natural phenomena are an invaluable supplement to results obtained in the laboratory.

To obtain shear strength data suitable for final design of open pit slopes a large number of laboratory tests have to be carried out on undisturbed samples and even then they form a poor statistical representation of the slope as a whole. Very often it is difficult or impossible to obtain good undisturbed samples from any suitable depth behind the slope, with the result that samples have to be cut one or two feet from the slope surface where the soil may have been altered by exposure.

If, therefore, natural phenomena could be successfully and accurately observed and interpreted they would be a great help, especially if they were on a large scale. These phenomena would represent large scale tests of the material which would be far more representative and statistically acceptable than the laboratory tests.

It is interesting to compare the laboratory test data, discussed above, with two full scale shear manifestations which were observed in the same pit during the above-mentioned stability investigation.

(ii) Natural Manifestations of shear strength.

1. The Western End of the investigated pit was underlain by underground workings. Surface subsidence resulting from the underground caving was taking place and cracks had appeared on some of the upper benches as illustrated by photographs, Plates I and II. These cracks were surveyed and have been analysed in Appendix III.

Where the cracking is taking place the ground is failing in horizontal tension and the stresses in the soil are in the condition of active pressure. If the cracking takes place as if in a two dimensional system, i.e. if the cracks appeared in very long lines on the surface of the ground, there should be two sets of complimentary cracks intersecting at an acute angle of $(90-\phi)^{\circ}$.

Appropriate rotation to give sections normal to the strike of the cracks is done in Appendix III and this gives a mean value of $\phi = 42^\circ$. This is greater than the angle ϕ measured in the triaxial tests. This may be due to the fact that the caving cracks are three dimensional and not two dimensional as required by the theory above or, alternatively, it may be due to a more sophisticated reason in that in the triaxial tests the volume changes during shear are not identical to those which have taken place in the caving process.

The interpretation of the cracks is not yet entirely clear and it is a matter which deserves considerable thought. However, the results that have been obtained are thought to supply an important correlation with laboratory results.

2. During January 1960, a bench failure took place in the pit under investigation. An attempt was made by the Civil Engineering Department of the Witwatersrand University to analyse the failure using the mean reduced values of c' and ϕ' found from laboratory tests discussed above. The analysis is shown in Appendix V.

Certain assumptions had to be made, in particular whether tension cracks were present and fully developed at the top of the slope before failure occurred and, further, the magnitude of the porewater pressures in

the soil behind the face of the slope had to be estimated. Porewater pressures can easily exist in such a case and they will be due principally to entrance of rain water into the tension cracks at the top of the bench. This is probably the case, as the slide took place during a period of fairly heavy rainfall.

In the analysis of the slip, porewater pressures have been considered in terms of the porewater pressure ratio r_u .

The porewater pressure ratio r_u is defined as follows:-

$$r_u = \frac{u}{\gamma h}$$

When the water table is at the surface the maximum value of r_u is given by

$$\text{Max } r_u = \frac{\gamma_w h}{\gamma h} = \frac{\gamma_w}{\gamma} \approx \frac{1}{2}, \text{ i.e. } \frac{62.5}{120} \text{ in this case.}$$

where : γ is the total, or bulk density of the natural soil.

γ_w is the density of the water = 62.5 lbs/cu.ft.

h is the depth of any point in the profile.

The results of calculations for various conditions of the two unknowns, namely, the presence of tension cracks and the values of porewater pressures, have been summarised in Fig. 15. The data show that if the full

Author Brink David Charles

Name of thesis The Stability Of Slopes In Open Cast Mines And The Economic Implications Thereof. 1962

PUBLISHER:

University of the Witwatersrand, Johannesburg

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