shear strength as measured in the laboratory ($c' = 720$ lbs/sq.ft, $\phi' = 35^{o}$) has been mobilised then for a factor of safety of 1.0, implying a condition of incipient failure in the slip, the porewater pressure ratio, r_u, will fall between the values of 0.33 and 0.41 depending upon the absence or presence of tension cracks. These values are consistent with rainwater entering at the top of the slope under conditions of sustained rainfall. Under conditions of sustained rainfall it is assumed that any tension crack present would be completely filled with water exerting a hydrostatic pressure against the walls of the crack, hence increasing the actuating force on the potentially unstable mass.

However, there was a further complicating factor in that the condition of fissuring in the vicinity of the slip was abnormal. Fissuring has the effect of reducing the available shear strength and the laboratory values may be too high for this case. Fissuring may be taken into account here by an arbitrary increase of the factor of safety which represents the condition for incipient failure. In this case it was estimated that the factor of safety should be increased from 1.0 to say 1.20 or 1.30 if laboratory shear test data were to be employed. If either of these values be considered for the data in Fig. 15 it will be seen that the pore pressure ratio, r_u, still falls within reasonable values.

The analysis of the bench failure indicates that the porewater pressure in soil behind the slope was definitely positive. This may be due to either rainwater or unrelieved water pressures as consolidation is taking place in the soil. The latter alternative seems unlikely as the failure surface of the slip was near to the original ground surface of the bench before the failure took place, so the resistance to drainage of porewater during consolidation was low.

Bench failures of this type provide important field evidence for correlation with laboratory tests. However, their statistical importance with respect to all the benches of equal height and in the same material which have not failed, should be considered carefully, especially when irregular conditions such as abnormal fissuring are present.

If at all possible during engineering operations involved in cutting slopes in soil material, full scale tests may be carried out by inducing bench failures similar to the one discussed above. This may be accomplished in a number of ways, e.g.

(i) increasing the height and angle of slope until failure occurs

(ii) applying an excess load at the top of the slope

(iii) removing material from the toe of the slope

In this way invaluable information will be obtained which would greatly facilitate future design work.

The importance of observing natural shear manifestations and the conducting of full scale tests cannot be over emphasised.

(c) Porewater pressure predictions and measurements.

1. Variation of porewater pressures due to de-watering.

The prediction of porewater pressures in the soil behind a slope probably constitutes the most difficult phase of stability analysis using effective stresses. The problem usually arises because porewater pressure conditions are not static.

When mining operations are carried out in areas where the ground water table is near the surface, a considerable lowering of this water table results, either as a result of pumping from underground workings or from the sumps of open pits into which the water from the surrounding area flows. This water table lowering results in an increase of effective stress in the soil due to the removal of buoyancy due to the porewater. The principles involved in the increase of effective stress of a soil as a result of lowering the ground water are illustrated in Figs. 1 (a) & (b) (Appendix VI.)

If a water table is lowered by 100 ft. it results in an increase of about 3 tons/sq.ft. in effective pressure (σ - u) on all of the strata below a level approximating to the original water table position. This is a considerable increase in load which would have a profound effect on the stability of open pit slopes thus affected. Further, the drainage process which accompanies the water table lowering will result in seepage of the retained water down to the new water table. This must have the effect of washing fines out of the soil structure and thus lowering the cohesive strength of the soil.

The increase in effective pressure referred to above results in a compression of the soil grain skeleton with an associated decrease in the volume of voids. This requires an expulsion of water from the voids of the saturated soil or an expulsion of water and air from the soil where air entry has already occurred. This in turn involves a flow problem and, for flow of the porewater to take place, time is required. Hence the decrease in water pressures in compressible soil above the sink stratum involves a time lag which is a function of the permeability of the soil. A sink stratum refers to a porous stratum in otherwise relatively impermeable strata. Pumping the drained water from a sink stratum causes the water pressure in this stratum to become atmospheric. The sink stratum in this case is assumed to have considerable lateral extent so that we can consider the water pressure behaviour in a typical column of soil.

In consolidation theory the curves of the porewater pressure at various stages in time from the initial curve at $t = t_0$ to the final curve at $t = t_\infty$ are known as isochrones.

An evaluation of the time effects of consolidation on the soil strata and the associated progressive reduction in porewater pressures is carried out in Appendix VI.

The evaluation is based on the conditions prevailing at the pit which was under investigation. In this case the water table has been lowered 400', say from 0' to 400'. The strata affected consists of a soft to firm, fairly cohesive sandy silt.

Initially, before pumping, the porewater pressure will follow the line marked t = 0 in Fig. 16 (b). After pumping has progressed for 23 years the porewater pressure will be represented by the line marked t = 23 yrs. on the figure. In this way the reduction of porewater pressure proceeds as shown by the remaining isochrones shown on Fig. 16 (b).

Appendix VI shows details of calculations and plotting of isochrones which illustrate the progress of consolidation with time, and the corresponding state of porewater pressures. See Figs. 16 (a) and 16 (b).

Fig. 16 (a) shows isochrones drawn with the assumption that no air entry takes place and that the negative pressures are fully developed.

Fig. 16 (b) shows the case where air entry is assumed to take place at a negative pressure of minus 1 atmosphere. As seen from the isochrones a slight development of negative pressures is estimated with depth. This may be erroneous.

The isochrones discussed above deal with the case where the sink strata were assumed to be horizontal.

Fig. 17 illustrates the effect of a dip in the sink stratum. The water table in the sink drops with time so that the starting point of the isochrone will fall. Depending upon the rate of draw-down of water the isochrones may not be as regular as has been indicated. Nevertheless, the general pattern is very similar to that shown on Fig. 16 (b).

While the drainage processes of the types shown in Figs. 16 (b) and 17 will apply to deep pumping from strata which may be considered as semi-infinite sinks, these conditions will apply only if the overlying compressible stratum remains undisturbed. As soon as a major excavation is made into the strata and its depth exceeds the water table depth, the flow pattern will be modified by drainage into the excavated pit. Drainage will always prefer paths of smaller total resistance to flow, in general paths of smaller length. Close to the pit slope the pattern of Fig. 18 is preferred but as any vertical section recedes from the pit slope the patterns of Figs. 16 (b) and 17 start to predominate. The resultant flow situation at any section will be determined by vector summation.

It is seen from the isochrones that there is a possibility of the development of negative porewater pressures. This is a matter that deserves a great deal of consideration as these negative pressures may contribute substantially to the stability of the slopes since it is clear that in the shear strength equation

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

Considerable advantage can be gained if the term u is negative and can be relied upon to stay negative. Because of the difficulties involved, negative pore pressures are usually neglected in normal slope design and it is possible that in many cases great economic advantages are forfeited.

The negative porewater pressures, as discussed, are caused by capillary forces. [2] All water within the capillary zones in soils is under tension; in other words its pressure is less than atmospheric pressure.

The force across any unit of area within a soil mass, as explained in previous discussions, is made up of intergranular pressure and water pressure. The combined pressure may be expressed

$$\sigma = \bar{\sigma} + u$$

where:- σ is the combined pressure

$\bar{\sigma}$ is the normal component of the intergranular pressure

u is the water pressure

Therefore at all points where the stress in the water is negative, the intergranular pressure must be of greater magnitude than the combined pressure. This leads to greater strength in the soil, since the strength characteristics of the soil depend directly on the intergranular pressure.

Capillary pressures, and the increased shearing strengths they produce in fine soils, may be of appreciable magnitude. Calculations reveal that they may be as high as 15 Kg per sq.cm. or ± 215 lbs. per sq. ft. in clays with a high colloidal content.

2. Settlement of the de-watered strata.

Ground water lowering as discussed above results in consolidation of the compressible and semi-compressible soils. This may cause appreciable surface subsidence if the depth of water lowering is great. This subsidence is also a function of time and records of surface movements will provide important indirect information regarding the thickness of strata and the progress of de-watering. Such observations of surface subsidence would have to be referred to remote fixed bench marks.

If estimates of settlement are made for various thicknesses of strata and porewater pressure distributions, these results can be compared with the actual settlements measured and a fair idea can be obtained of the state of porewater pressures in the strata. This may obviate to a certain extent the need for taking large numbers of porewater pressure measurements from boreholes, which are extremely difficult where the strata are not permeable.

A discussion on the method of settlement estimation based on results obtained from laboratory tests follows:

Consolidation tests were carried out, in the Civil Engineering Laboratories at the University on behalf of the author, on soil samples obtained from the strata under investigation. The results of the consolidation tests are tabulated in Appendix VII, Tables I and II. The data obtained included observations of the rates of compression and the coefficient of consolidation c_v has been calculated. See Figs. 1 (a) and (b), Appendix VII. This coefficient is a function of the permeability (k) and the void ratio (e) and is used for convenience in calculating time rates of dissipation of porewater pressures during consolidation. The mean figure $c_v = 1.0 \times 10^{-2} cm^2/sec.$ has been taken for the strata tested. This value was used in determining the isochrones discussed above.

Void ratio (e) was also plotted against the logarithm of the applied pressure (log p). The slope of the curve of the semi-logarithmic plot represents C_c which is called the compression index for the soil. See Figs. 2 and 3, Appendix VII.

By definition $C_c = \dfrac{\Delta e}{\Delta(\log_{10} p)} = \dfrac{\Delta e}{\frac{dp}{p} . 0.435}$

$$\therefore \quad \Delta e = C_c . \frac{dp}{p} (0.435)$$

But the ultimate settlement

$$\rho_u = \frac{2H}{1 + e_o} (e_o - e)$$

[e_o is the void ratio at the maximum past pressure exerted on the soil and is also determined from the Δe vs. log p curve. See Figs. 2 & 3, Appendix VII.]

$$\rho_u = \frac{C_c}{1 + e_o} \; 0.435 \; \frac{\Delta p . H}{p}$$

$$= \frac{C_c}{1 + e_o} \; 0.435 \; \frac{p_2 - p_1}{\frac{1}{2}(p_2 + p_1)}$$

ρ_u represents the ultimate settlement.

To find the settlement after 34 years for example, we must determine the Time Factor which in this case is T = 0.075. The percentage consolidation is then read off Fig. 19.

$$\%U = 30\% \text{ approx.}$$

Therefore total estimated settlement of the surface of the 400' stratum after a period of 34 years

$$= \frac{30}{100} \times \rho_u$$

$$= \frac{30}{100} \times 15.45 \text{ ft.}$$

$\therefore$ <u>Settlement</u> = <u>4.6 ft.</u>

See Appendix VII, Fig. 4, for results and calculations.

CHAPTER 5.

THE STABILITY OF ROCK SLOPES.

The analysis of stability of rock slopes is confusing to the engineer. This seems to stem from the lack of any grounds for a generalisation of approach to the problem. Comparison of different slope problems is difficult as there are usually few common denominators in any two slopes.

A number of differential classifications have been carried out on rock and soils. The object in each case has been the establishment of some convenient, though arbitrary, dividing line to differentiate between rock masses and soil masses. For the purpose of this problem a structural differentiation seemed to be the most suitable. A simple but effective system was worked out by Coates & Brown (5) and is recorded here.

"It is suggested that as a structural material rock differs from soil in two respects:

(1) Rock strengths are of a higher order of magnitude than soil strengths.

(2) Rock masses are seldom found that form comprehensive masses similar to cohesive soils, i.e. rocks almost always contain a family of joints, not to mention the possibility of other structural features, which divide the mass into a system of blocks. The

effects of bedding, schistosity, etc. can be considered analytically as being additional complexities superimposed on this block system."

The author has found this structural differentiation most useful in that, while it points out the obvious ways in which rock and soil masses differ it also gives a clearer picture of the similarity between the two, and it is upon this similarity, rather than difference, that the author bases his approach to the problem.

Consider a large mass of rock which has been fractured into blocks of some reasonably small size, say for argument's sake, in the region of 10-foot cubes. If the mass of rock is large enough we can compare it to a mass of soil, where the blocks of rock are likened to grains in the soil. Where the blocks are cemented together by material in the joints and this joint-material is strong, the rock mass may be considered to behave as a cohesive soil. On the other hand, where the joints are open and the mass has some finite void ratio, it may be considered to behave like a granular soil.

This comparative assumption is gaining popularity for the solving of many types of rock mechanics problems, not to mention its suitability in analysing slopes excavated in fractured and jointed rock. It is generally recognised that rotational shear slope failures do occur in jointed rock material in exactly the same manner as if the material had been a cohesive soil. This would indicate that we could represent the strength of a jointed rock by means of the Coulomb equation, i.e. we consider it to have some cohesion and some measurable constant internal angle of friction.

The frictional properties of joints in rocks has been the subject of an interesting paper by Professor J.C.Jaeger (13). The conditions for sliding over artificial joint surfaces were studied using triaxial testing apparatus and the results obtained agreed reasonably well with the simple theory for a constant coefficient of friction, μ, where $\mu = \tan \emptyset$; $\emptyset$ being the internal angle of friction for the material.

The measured coefficients of friction differed by surprisingly little between surfaces at various inclinations and when the inclination of the plane was greater than 65° to the axis of the specimen, failure took place through the rock ignoring the joint. This critical angle, in this case 65°, will naturally vary depending on the strength properties of the rock under test.

The following approach to stability problems in rock slopes is suggested:

(1) After carrying out adequate geological exploration of the affected area the engineer should be in a position to decide whether a rotational shear

failure is possible in the slope causing concern. In other words whether the ratio of the scale parameters of the slope and the material forming the slope is such that the slope material may be considered to behave as a soil as mentioned above. If this assumption does appear to be valid then the procedure would follow that which was discussed at length in Chapter 4.

(2) If, however, the geological structure of the slope suggests that a rotational shear failure is unlikely to occur, the approach would be somewhat different. For failure to take place in the slope, therefore, one assumes that this would take place along a plane, or several planes, of inherent weakness in the slope material. Thus, the conclusion is a simple though distressing one, namely, each slope of this type has to be considered as a unique individual, there being no panacea for the solution of this type of problem. This conclusion was brought home clearly to the author during a recent visit to an open-casting operation in Southern Rhodesia. There were a number of open pits on this particular property, all of which were excavated in rock material. The geology of the area was complicated but the structural details did not vary too drastically from pit to pit. During a preliminary investigation it soon became evident that the slopes in no two pits were alike, indeed, no two slopes in the same pit were alike. Each slope in

each pit would have to be considered separately were they to be analysed for stability.

There were a number of interesting slope problems on the property, and one of particular interest, which accents the value of this type of work.

The situation at the time is illustrated by Fig. 30, which is a sketch of a section through the slope in question.

The "superimposed weight" in Fig. 30 refers to the crushing section of the ore beneficiation plant. At a glance it is seen that the zone of talc schist could present a very dangerous condition especially if water were allowed to enter the talc zone at the surface. The rock below the talc zone was a competent hard rock while that above the talc was hard but traversed by joint planes. In this particular case the mining company was contemplating cutting back the slope as indicated by the dotted lines for the recovery of additional ore. Luckily the proximity of the crusher station at the top of the slope was taken heed of and further investigation revealed the presence of the extremely dangerous talc zone. Steps will now be taken to stabilise the slope more satisfactorily and this will most likely be done by creating an overburden dump in this section of the pit to provide some lateral restraint to the slope.

Finally then, it seems that the experience of the engineer on the job is the all-important factor in the solving of slope problems, and the way to obtain experience that can be relied on in this field is by attempting to understand the underlying principles of the problem.

CHAPTER 6.

THE PRESENCE OF UNDERGROUND WORKINGS BELOW OR ADJACENT TO THE OPEN PIT.

The mention of this subject always results in a variety of reactions from different people. Some are alarmed at the prospect whilst others consider it to be a rather innocuous pastime. At the outset the author had no hesitation in joining the ranks of the alarmed but this opinion was later modified.

Wilson, S. D. (4) makes reference to an open pit excavation in the U. S. A. which is being made in caved ground and which seems to be quite stable from a slope stability point of view.

The Western end of the pit, with which the author was concerned, had been undermined by underground caving but besides the fact that considerable surface subsidence was taking place, the slopes in that area did not seem to be adversely affected.

It seems then that excavating in caved ground is feasible enough, from a slope stability point of view, as long as the problem is approached in a level-headed manner. It would obviously not be advisable to excavate in ground where caving is currently taking place. The ground would have to be left for some time until the large scale settlements had taken place and the settlement - time plot had fallen to a reasonably small value. Other precautions would have to be taken in slope stability calculations, as the ground in caved areas would be badly cracked. This may be a problem of the same type as fissuring requiring only a flatter slope than that calculated for the same ground in an intact state.

The best approach to this problem for a reliable solution would be the establishment of the co-operation of experts on surface settlement caused by underground excavations.

The author does not profess to be an expert on surface subsidence, caused by underground excavation, but a small discussion here is relevant to some previous comments made in this dissertation.

The following is a summary of the present accepted subsidence theory (11) and it will be shown how this agrees with observations made by the author. The comparison is interesting and illustrates the possibilities entailed in greater co-operation with ground subsidence experts.

The limit of influence of an area of underground extraction at the surface, or at any intermediate plane, has been defined by a line drawn from the edge of the excavation to a point at the surface, at which ground movement is a minimum. The angle this line makes to the vertical, has been called "draw" in British practice, and corresponds to the angle to the horizontal in Continental terminology.

The accepted limit of influence (or zone of influence) is dependent upon the accuracy of measurement, and at any plane considered above the mining area, defines precisely what is implied, the limit of movement arising from mining. The fracture of the surface, when it occurs, is always nearer to the edge of the mining area and would if it occurred, be within the extensional, lateral differential zone, and probably where this movement is a maximum. See Fig.20.

It is interesting to compare the above with what was actually observed while carrying out a slope stability investigation on an open pit which was partially undermined.

The situation at the particular mine is represented well by the "dynamic case" as put forward in the subsidence theory as discussed above. The dynamic case is illustrated by a sketch in Fig. 21.

Cracks indicating fracturing were observed in the bench faces of the open pit in question at a position corresponding to the point of maximum extension on the extension-compression curve of Fig.21. These cracks are shown in Plates I and II. The cracks are an almost perfect example of failure taking place under Rankine active stress conditions. The angles of intersection of the cracks to the vertical were measured and by equating these to $45 - \phi/2$ a value for ϕ, the internal angle of friction for the material, was obtained which compared favourably with

ϕ values obtained in triaxial tests. The fact that the ground appeared to be failing in a Rankine active state fits in well with the above subsidence theory.

A better understanding of subsidence and caving mechanisms will be of invaluable assistance in the work of interpreting these full-scale shear manifestations, already referred to in this dissertation.

CHAPTER 7.

SCIENTIFIC OPEN PIT PLANNING AND CONTROL.

The problem of slope angles in open pit mines ultimately becomes one of costs and it is therefore inevitable that stability and economic considerations be linked in open pit planning to produce the optimum results for any projected open pit.

"Computations of reserve tonnages, stripping ratios, and grade of ore has long been a revolting aspect of the young mining engineer's job. Weeks at the desk calculator may turn into months before a single job is completed." (7)

In his statement Mr. R. L. Wilson probably expresses the strong opinions held by most mining engineers intimately associated with the planning of an open pit mining project. The job of detailed planning is a monstrous, but very necessary, operation. The time-consuming nature of the calculations is illustrated by the current trend of large American open pit operators towards the use of electronic computers in their planning computations.

When planning operations to take place in an open pit project one, two or even 5 years hence, many intelligent guesses have to be made. Upon these guesses depends the accuracy of the planning calculations. While it is necessary to maintain as high a standard of accuracy as possible in planning calculations, it does pay to keep them in perspective with the guesses made. Planning engineers are often frustrated to find that what they planned and what was actually done are two different things, after they had spent months of anguish with a planimeter and a desk calculator planning a project in detail.

The author now proposes a system for future planning of open pit operations which will produce swift and relatively accurate results.

The system may be described essentially as follows:-

(i) The setting up of a mathematical function which represents the growth in volume of an open pit which has a particular geometric shape. At the same time, splitting the function into that portion which represents the volume of ore excavated and that portion which represents the volume of over-burden stripped. The ratio of these would, at any stage of the operation, represent an overall stripping ratio for the pit.

(ii) By substituting numerical values for the various parameters in the functions it is possible in a very short time to obtain overall stripping ratios for the pit at any stage of the planned operation. The results of these calculations may be represented graphically in a number of very useful ways.

(iii) Further, by differentiating the above functions with respect to any of the main parameters, e.g. depth of pit, strike length, dip length, etc, it is possible to obtain a function representing the "Instantaneous stripping ratio" at any given value of the chosen parameter, i.e. for a pit with a depth = H ft. it is possible to determine the stripping ratio involved in increasing the depth from H ft. to $(H + \Delta H)$ ft.

(iv) In the same way as above, numerical values may be substituted for the parameters in the function rendering quick results which may also be conveniently represented graphically.

This particular method of analysis has some useful applications. They are:-

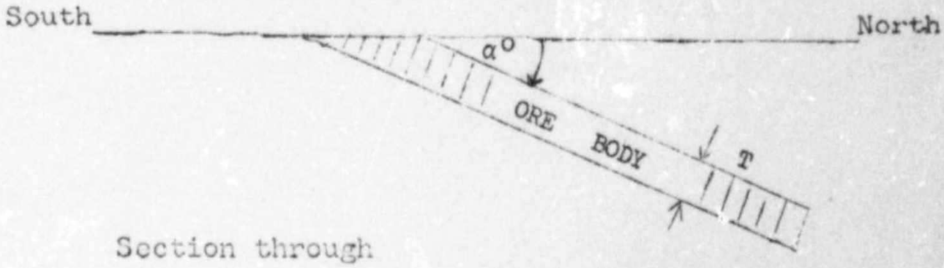
(i) Determining the correct geometric shape of an open pit, given a certain geometric shape of the ore-body.

(ii) Current planning and control of open pit operations.

Examples of these analyses have been appended, together with tabular and graphical representation of the results.

(i) Determining the correct geometric shape for an open pit given a certain shape of ore-body.

Description of the ore-body to be considered.



Section through
Ore Body

The ore-body is tabular with a constant thickness T ft. and constant dip to the horizontal of α^o. The lateral extent of the ore-body is defined by length L ft.

Two geometric shapes of open pits suitable for mining this ore-body are analysed in the appendix and the comparison of results is both interesting and revealing.

The two pits analysed will be referred to henceforth as Pit A and Pit B.

See Figs. 1, 2 and 3 appended for shapes and dimensions of Pits A and B.

Pit A is a rectangular excavation with a constant top length on strike.

The pit constants have been expressed in the following general terms. See Fig. 1.

Ore-body:	Dip	$= \alpha^o$
	Thickness	$= T$
	Sub-outcrop depth	$= D$
	Average width at bottom of box-cut	$= B'$

Pit Excavation:

Top length on strike $= L$

North, or mining face, slope $= \beta^{o}$

East face slope $= \theta^{o}$

West face slope $= \Psi^{o}$

Ground slope (towards north) $= i^{o}$

Ground is level, East-West

Depth of pit from North rim $= H$

It is seen from the geometry of this pit that the strike length of the ore-body exposed decreases with depth at a rate depending on the slopes of the East and West faces, θ^{o} and Ψ^{o} respectively.

This decrease in the strike length of the ore-body with depth was thought to be a disadvantage and for comparison purposes Pit B was designed. The design of Pit B is centred around keeping the strike length of the ore-body constant with depth. To do this the rectangular shape of Pit A was abandoned and in Pit B the East and West side slopes were inclined outwards to the North as shown in Fig. 2. The angle of inclination is calculated in appendix II.

Besides the outward inclination of the side slopes, the pit constants used in the appended analyses for Pit B are the same as those used in Pit A. See above.

Discussion of the results obtained.

In Figs.23 (a) and 23 (b) overall stripping ratio's are plotted as the ordinate, against the total volume of ore recovered, plotted as the co-ordinate.

It is seen from the curves obtained for Pit A and Pit B that in the initial stages Pit A works at more economical stripping ratios than Pit B, for the same volumes of ore recovered. At a certain stage of the operation (i.e. when volume of ore recovered = 3.6×10^8 cu.ft., for $\beta = 35^o$; and vol. of ore recovered = 4.6×10^8 cu.ft., for $\beta = 45^o$) the two curves cross over and Pit B becomes increasingly more economic than Pit A.

It is also seen from Figs.24(a) and 24 (b) that the pit can be kept far shallower following the method of Pit B, whilst recovering the same volume of ore. This is illustrated by the following example which is read off the curve for $\beta = 35^o$:-

To recover 5×10^8 cu.ft. of ore, Pit A would have to be 1,200 ft. deep while Pit B would only have to be 640 ft. deep. This is an important factor from a slope stability point of view.

The conclusion is, therefore, that as a short term proposition Pit A would be preferable, but if the grade of ore permitted a far larger operation with the removal of large volumes of ore, Pit B would then be better in the long run.

In countries where the fullest exploitation of natural resources is essential the method employed for

Pit B would be highly desirable.

Similar useful information is obtained by plotting curves with instantaneous stripping ratios as the ordinate instead of overall stripping ratios. See Fig. 25.

The object of the exercise is to illustrate what sort of factors are revealed by the analyses and the examples worked out are not claimed to be ideal.

(ii) Current Planning and Control of Open Pit Operations.

In the current planning and control of an open pit project the engineer must know how he stands at any time with respect to:-

(a) The economics of the operation.

(b) The safety of the operation.

(c) What the current overall and instantaneous stripping ratios are.

A control chart, therefore, which could readily illustrate these things would be extremely useful.

Two control charts are proposed. One illustrates the position in relation to instantaneous stripping ratios and the other illustrates the position with respect to overall stripping ratios. See Figs. 27 & 28.

In Fig. 27, instantaneous stripping ratios have been plotted against depth of pit in feet. A family of curves is obtained for different values of β, each curve representing the growth of the pit in depth for a particular value of β. (β being the slope of the overburden on the winning side of the pit.)

The family of curves obtained may be used as a basis for a control chart. For the purposes of an explanation a hypothetical example will be considered.

Assume that the Company mining open pit B decided on the following general policy.

(a) Mining from a depth of 0 ft. - 200 ft. with $\beta = 15^{\circ}$.

(b) Mining from 200 ft. the slope angle β is slowly increased until at a depth of 400 ft., $\beta = 35^{\circ}$.

(c) The slope angle $\beta = 35^{\circ}$ is carried to a depth of 680 ft. after which it is gradually increased to obtain a slope of 45° at 800 ft.

Fig. 26 illustrates this mining policy.

The progress as outlined above can now be plotted as a superimposition on the abovementioned family of curves, and it is represented by the curve plotted with small circles on Fig. 27. From this curve it is possible to read off directly the instantaneous stripping ratio at any stage of the operation. It will be convenient to refer to this curve as the "instantaneous stripping ratio characteristic" hereafter.

In a similar manner it is possible to obtain an "overall stripping ratio characteristic" as illustrated in Fig. 28.

It remains now to introduce safety and economic limitations into the control chart.

From stability computations for the pit slopes one is able to plot a curve of safe angle of slope vs. depth of pit. If this curve is superimposed on the control charts (see dotted curve on Fig. 28) it forms a "safety barrier" through which the stripping ratio characteristic should not be allowed to pass. As long as the stripping ratio characteristic remains above and to the left of the dotted curve, therefore, conditions in the pit will be safe from a slope stability point of view.

Before applying an economic limit to the control chart a divertionary discussion is thought necessary.

We require to know at what level of stripping ratio we cease to make a profit. A factor called the "economic stripping ratio" is employed to do this.

It is defined below and illustrated by a hypothetical example.

<u>Let</u>:

(i) the revenue obtained from mining 1 ton of ore - - - - - - - - - - - <u>= a</u>

(ii) the cost of the entire mining and metallurgical operation, excluding cost of stripping overburden, to process and sell the copper from 1 ton of ore - - - - - - - - <u>= b</u>

(iii) the cost of mining 1 ton of ore - - - $\underline{= c}$

<u>Then</u>:

(iv) a - b = money available for stripping overburden.

(v) $\frac{a - b}{c}$ = economic stripping ratio.

Revenue from 1 ton of ore at 4% cu.

(Price of copper = £220/ton)

Revenue $= \frac{4}{100} \times 220 =$ £8.16.0d.

If a = 176/-

b = 128/-

a - b = 48/-

c = 4/-

$\therefore \frac{a - b}{c} = \frac{12}{1}$

$\therefore$ Economic stripping ratio = 12:1

<u>Note</u>: This example is purely hypothetical.

The value of the "economic stripping ratio" obtained in the example is 12:1. This value obviously will be subject to fluctuations depending on market conditions, working costs and a host of contributory factors. Its value can, however, be determined with a fair degree of accuracy at any instant of time. This value can be plotted on the control chart by relating its value at any time with the progress of the pit in depth. See Fig. 28 where the curve has been plotted arbitrarily as a horizontal line representing a value of 12:1.

This curve forms the "economic barrier" on the control chart and to mine economically the stripping ratio characteristics should remain below this curve. This is a very rigid condition as far as the overall stripping ratio characteristic is concerned, but it can easily be seen that brief excursions of the instantaneous stripping ratios across this economic barrier are acceptable as long as the overall profitability of the operation is not adversely affected.

In the development of the above theory an actual mining policy was plotted on a control chart. One might ask, "Is the cart before the horse?" Indeed it is when considering the problem from a planning point of view, but it is not when considering the problem from a control and checking point of view.

The planning aspect reveals itself quite clearly. All we have to do is to plot an "ideal" path for the stripping ratio characteristics on the charts, depending on the current market conditions and other influencing factors, making sure that the characteristic remains between the safety and the economic limits, and then relate the information back to the formulation of a corresponding mining policy. The progress of this policy could then be checked by plotting the actual current positions on the control charts.

The curve plotted with little crosses on Fig. 27 represents such a planned characteristic and Fig. 29 represents the mining policy formulated from it.

ECONOMIC IMPLICATIONS.

Having established the various physical merits and de-merits of the two methods of mining the ore-body, i.e. either by means of Pit A or Pit B, it would be necessary to carry out economic valuations of the two methods to determine which, in fact, was the more economic proposition. An example of such a comparative valuation calculation is shown below.

A number of simplifying assumptions were made for the purposes of illustration of the valuation calculation. The object of the example is to illustrate the method of approach to this calculation, and not to carry out an actual valuation calculation.

For the set of circumstances chosen for the example it appears that Pit B is a better proposition than Pit A, the former having a present value of £25 million whilst the latter has a present value of £21 million. One should tread warily at this stage, and bear in mind the assumptions made.

They are:-

(a) The life of both pits is given as 15 years. This necessitates a far higher rate of production in

Pit B than in Pit A, which in turn entails higher initial expenditure on capital equipment.

(b) The cut-off grade has been fixed at a constant value of 1.5% copper for both pits, and similarly, a constant mill head grade of 2.5% copper has been assumed.

(c) The final depth of both pits has been taken as 800 ft. This may not be the optimum depth for both pits.

It is suggested that a number of similar calculations be carried out, varying each of the above factors in turn, while keeping the others constant. Having covered a comprehensive combination of possibilities a pattern should emerge which could be used as a basis for an optimal analysis.

The number of calculations need not be excessive. The following combination is suggested.

(1) <u>Cut-off grades of</u>:

(a) 1.5%) with their corresponding
(b) 1.0%) mill-head grades.
(c) 0.7%)

(2) <u>Total life of pit of</u>:

(a) 15 years) with corresponding
(b) 25 years) rates of production.
(c) 35 years)