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THE STABILITY OF SLOPES IN OPEN PITS

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THE STABILITY OF SLOPES IN OPEN PITS

By

D. F. COATES[1]

INTRODUCTION

Many open pits in Canada are currently being planned with depths of the order of 1000 ft. The competition between safe slopes and maximum profits is thus providing greater incentives than before for improving the design of these types of mines.

If it is accepted that the objective of any work in rock mechanics is to provide guidance for the judgment that must be exercised in making decisions on planning and operations, then it can be claimed that within reason studies on pit slopes stability are valuable. If, however, from rock mechanics investigations definitive slope angles are desired for each pit wall, then the warning must be given that not only can this not be done for hard rocks at the present time but that it will be a difficult goal to obtain within the foreseeable future.

This being the situation, what explicit benefits come from slope stability studies or how can the design of pit walls be improved? Before answering this question a few terms will be explained so that the terminology related to this field can be used.

TYPES OF SLOPE FAILURES

The four general types of failure, as shown in Fig. 1, are distinguished by the mechanics of the breakdown of the ground.

Rock falls

Rock falls occur when loose rock develops with the slope angle being greater than the angle of repose of this loose material. The blocks of rock fall or roll to the bottom of the slope. The cause of this type of slope failure can be considered to be the result of a breakdown in the tensile strength of the rock mass.

Rotational shear

Rotational shear failure occurs when a segment of the slope fails by rotation on a more or less circular arc. This type of failure only occurs in rocks that will yield, similarly to structural steel, before rupturing. This yielding is necessary so that high local stresses are prevented from developing. Rupture then occurs when the average shear stress over a critical surface, which happens to be very nearly circular, exceeds the average shear strength.

Plane shear

Plane shear describes the slope failure when sliding occurs along an extensive geological plane of weakness. Failure results from the critical location of such planes with respect to the geometry of the wall. Figs. 2 and 3 show two situations where the geology would favour this type of failure.

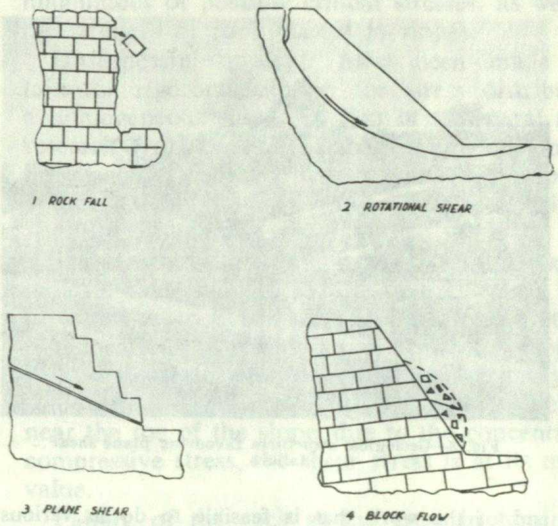

FIG. 1—Classification of types of stope failure.

Block flow

This type of failure is imagined to occur in a uniform, hard rock mass that does not contain extensive planes of weakness oriented in a critical direction. The method of failure is visualized as being a general breakdown of the rock mass as a consequence of the crushing, at the points of highest stress, in the brittle blocks comprising the mass. As individual blocks are crushed, increased loads are thrown on to the adjacent blocks whose strength in turn is exceeded. It is thought that this type of progressive action, or working of the ground, continues until a general breakdown of the slope occurs.

SLOPE STABILITY INVESTIGATIONS IN HARD ROCKS

To return to the question raised in the introduction of what can be done to be of help in this field, a review can be made of the work that is being done

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FIG. 2—Geological structures favouring plane shear failure.

FIG. 3—Geological structures favouring plane shear failure.

and of the work that is feasible to do in various situations. This review can be made with respect to the type of failure that is occurring or is anticipated.

In hard rock slopes, the most troublesome type of failure is that arising from the development of loose rock on the surface of the slope. Very often this action alone prevents the excavation of slopes steeper than the angle of repose of the loose material. Consequently, any methods of preventing or controlling such failures would permit, in many cases, steeper slopes to be used.

Some investigations are being conducted on the effectiveness of modifying blasting patterns adjacent to ultimate slope faces. In the construction industry the use of closely spaced holes that are not loaded and the use of lines of holes of reduced diameter, spacing and weight of explosive, have proven to be effective for many situations. Whether such techniques are feasible in mining depends on the nature of the rock, the cost of the increased drilling and the saving in waste excavation or clean-up. More experiments along these lines would seem to be of value in many cases.

Besides attempting to prevent rock falls by modifying blasting procedures, some control can be exercised. A rigorously scheduled program of scaling is, of course, important on most properties. In addition, periodic benches of extra width can be included in the wall design to catch rock falls and prevent their causing damage at lower levels. Rock anchors have been used in limited areas to prevent such falls, and mesh has been used to control the actual fall as shown in Fig. 4. Where mesh is used

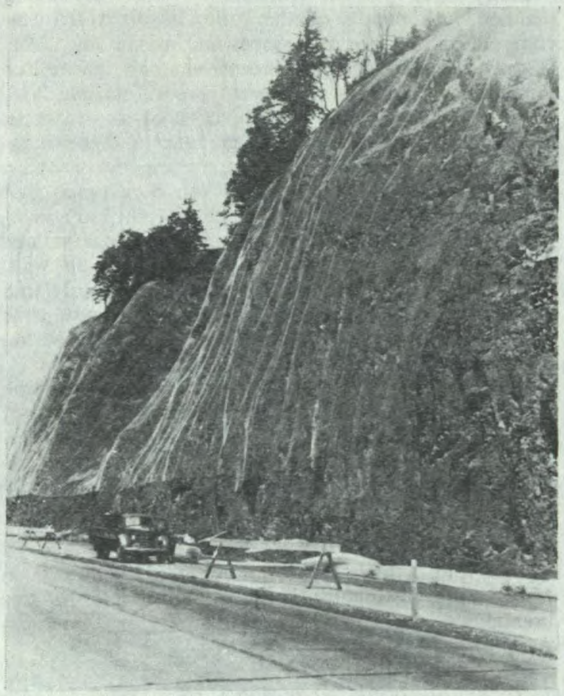

FIG. 4—Controlling rock falls with wire mesh.

it has been found to be preferable to pin the mesh only at the top, otherwise loose rock can develop and be temporarily retained until the load becomes excessive; then the mesh will fail and spill the accumulated rock down the slope.

A more extensive type of failure that can occur in hard rocks is the movement of a large block of ground by plane shear failure on a geological surface of weakness such as a fault, a weak bed or a sill or dyke of altered rock. In normal circumstances it is difficult to know if this type of failure is impending or not. However, as this mode of failure generally affects the entire height of the slope, it can also be expected to affect a length of the wall of a similar order of magnitude. Consequently, it requires an extensive surface of weakness to produce this type of failure. The corollary to this requirement is that the detection of any such surface of weakness within a critical zone in the slope would not be too difficult.

Although seismic methods might be tried, the most positive technique in current use is to conduct a careful drilling program to obtain cores throughout the zones where a surface of weakness would have to be located to cause trouble. Any loss of core, even as little as 1 in. in a 10 ft run, might be significant. Consequently, observations on the drill behaviour, e.g., loss of pressure on the bit, rough running, etc., become very important to assist in deciding on whether any lost core arises from a soft layer or from other accidental causes such as grinding or possibly an inefficient core catcher. A device that might also be used for examining a zone where core has been lost is the borehole penetrometer. This instrument can be inserted in a borehole and the force measured for the penetration of a small piston for a specified distance into the wall of the hole. For particularly important investigations borehole cameras or TV cameras can also be used.

When such an extensive plane of weakness has been detected in a critical zone, the strength or resistance to sliding along this plane can be investigated if cores can be obtained that contain the weak material. These cores can then be tested in the laboratory to obtain some information on the strength of the weak layer. It is then possible to make a two dimensional analysis to determine the slope angle that would avoid such a plane shear failure. Alternatively, in special circumstances a steeper slope might be designed and deep rock anchors used to increase the strength along the potential surface of failure.

However, the imponderable factor that arises in this problem is the effects of the ends of the mass in resisting this failure. In other words, the three dimensional aspects are very important. It has been found, for example, in investigating such a situation that, whereas failure might have occurred, the section was restrained from moving by the more stable rock, owing to modifications in the geology of the adjacent sections.

A strong element of engineering judgment must be exercised in making decisions in these cases. However, if an element of risk is accepted, some monitoring technique should be employed to obtain some warning of any developing instability. Microseismic monitoring, or listening for the rock to start working, currently is being tried, but no conclusion can yet be reported on its degree of success. Alternatively, crest deformation measurements between pins set on a line normal to the wall can provide a warning of developing instability.

Other techniques might be tried although they are more complicated than the previous two mentioned. For example, borehole extensometers, whereby the expansion of the ground over the length of the borehole is measured, might be a useful monitoring technique. Also, the use of stressmeters to measure changes in stress with time, and possibly even for measuring absolute stress within the slope, might have some usefulness although these are expensive techniques. The measurement of the sonic velocity between stations along a wall is another possible technique that might detect changes that occur with time. However, although this technique looks interesting for underground work no attempt has been made yet to use it in open pits (Larocque, 1964).

If a slope in hard rock is not governed either by the possibility of rock falls or plane shear failure, then the only remaining mode of failure is by block flow. Unfortunately, at the present time it is impossible to make a theoretical analysis of the stability of such a slope. More research is required on the stress distribution, in particular the locations and magnitudes of possible critical stresses, as well as on the strength of rock masses in slopes.

Unsuccessful attempts have been made to obtain the rigorous solution for stress distribution in a homogeneous slope. In lieu of a general solution, research work is being conducted to determine the patterns of stress distribution in typical slopes using photoelasticity (Gyenge and Coates, 1964; Long, 1964).

These stress distribution studies and experience indicate that if a block flow failure occurs the rupture of the rock is likely to be initiated at one of two locations. Behind the crest of the slope there is a zone of tensile stress that might easily exceed the normally low tensile strength of any rock. Then, near the toe of the slope, due to the concentration of compressive stress, the shear stress is at its maximum value.

Failure of the rock at the crest in tension, however, does not necessarily mean that the entire slope will fail. It has been observed many times that tensile cracking has occurred without the slope failing. However, it does produce a more severe stress distribution that might be all that is required to cause a shear failure of the slope.

With shear failure being initiated near the toe of the slope in a restricted zone, the slope itself need not fail. However, a progressive action is visualized whereby the zone of high stress concentration is shifted back from the immediate toe area, which can then cause additional shear failure, and so on until the entire slope breaks down. Field measurements, however, are required to substantiate this concept of progressive failure.

The determination of the strength of the rock mass in a slope is even more difficult than the determination of the stresses. Other than to classify the rock substance, there is little use in testing the normal small-size core samples in the laboratory as this information can not be applied in any practical manner to the problem.

The strength of a rock mass in a slope is probably affected by the orientation and frequency of joints (Long, 1964). The determination of the orientation of the various systems of joints must usually be done on a statistical basis, thus requiring a large number of observations. These observations can be obtained by measurements on the joint faces exposed by the excavation of the wall and from observations in drill holes using either a borehole camera or a closed circuit TV camera.

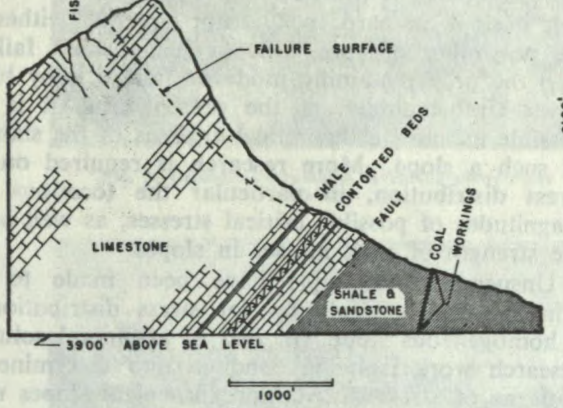

FIG. 5—The Frank Slide, Alberta, Canada.

These techniques are expensive and not perfect from a scientific point of view. For example, observations on the face of a wall are biased towards those joints with strikes and dips parallel to the wall. Borehole observations are somewhat better, but are biased against orientations normal to the axis of the hole; i.e., their trace is less conspicuous than joints that approach being parallel to the axis of the hole.

At the present time, the proper use of such joint data in an analysis of the mechanics of the slope is still a matter of theoretical consideration, and, as the expense of such studies is relatively great, this type of work must be considered to be still in the realm of research rather than practical engineering.

The techniques that can be used to exercise some control or to obtain some warning of incipient failure by block flow are the same as those described above as being useful for incipient plane shear failures; i.e., crest deformation measurements, possibly microseismic monitoring, the use of borehole extensometers, or other untried techniques.

An example of a probable block flow failure is provided by the Frank Slide in Alberta, Canada, where 80,000,000 tons of rock flowed as a fluidized mass two and a half miles in less than 100 sec. (Anon., 1912). Eye witnesses described the movement of the mass as being like a viscous fluid. The blocks varied in size from powder up to 40 ft with the common size being between 3 ft and 20 ft.

The geometry of the slope at the Frank Slide is shown in Fig. 5. The rock was a strong limestone with some layers of shale, sandstone and coal at the toe. The maximum slope angle was 53° with a height of 2,100 ft. The limestone dipped 49° into the slope and contained a well developed family of joints. The outline of the failed mass is also shown in Fig. 5.

Several factors combined either to increase the shear stresses or to decrease the shear strength of the mass so that failure occurred. First, the temperatures during the days preceding the failure varied from 70°F. during the day to 0°F. during the night causing melting of the surface snow in the day and freezing of the resulting water in the cracks at night. Second, ground water seepage added to the normal shear stresses. Third, compressible layers of shales and sandstones below the limestone at the toe of the slope probably aggravated the stress distribution. Fourth, underground workings in the toe area added to this affect. Fifth, earthquakes that occurred during the preceding months could have marginally affected the strength of the rock mass.

Failure in the toe zone must have started at least seven months before the actual slide, as the coal in the underground workings had been breaking itself during this period of time. The excavation of the coal can be thought of as artificially decreasing the average strength and increasing the compressibility of the toe material so that failure would be initiated and ultimately would progress throughout the slope. The slide zone coincided exactly with the length of the workings along the toe of the mountain.

To obtain some practical guidance on the design of slopes in hard rock where the possibility of a plane shear failure is excluded, the only practical approach at the moment is through case histories of either previous slides in the same formation as the planned wall or in previous stable slopes in the same formation. For example, Fig. 6(a) shows a slope that was surveyed in a steeply dipping dolomite that contained a few solution caverns and in some zones was severely brecciated with secondary calcite cementation. The strength of the rock substance, including some brecciated zones, was of the order of 26,000 lb./sq. in.

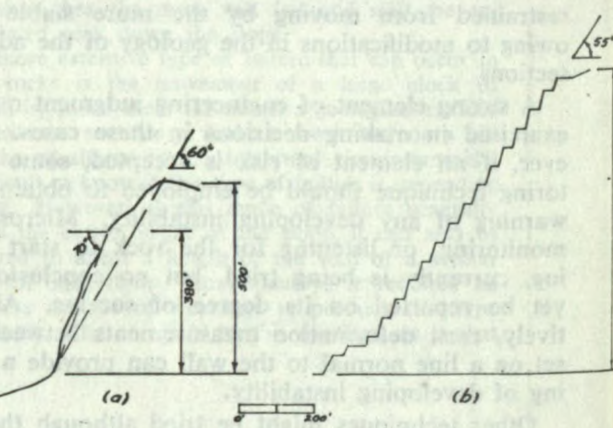

FIG. 6 (a)—Observed high slope in dolomite.
FIG. 6 (b)—A planned slope in the same dolomite.

An open pit mine was being planned that would include a nose of the same formation. As this nose projected into the ore zone, the large amount of waste excavation that it represented would be influenced very significantly by the appropriate slope angle. Some guidance in determining this slope angle was obtained from the slope that had been surveyed and was known to have been stable. It is possible that the rather awesome slope shown in Fig. 6(b) would be stable; however, to eliminate the possibility of a plane shear failure, investigations were required to determine if any major structural planes of weakness existed within a critical zone in this part of the formation.

SLOPE STABILITY INVESTIGATIONS IN SOFT ROCKS

In rock that will yield without rupture at points of stress concentrations, it has been established that the methods used in soil mechanics can predict stable slopes (Coates, *et al*, 1963). These methods consist in obtaining samples from the ground in which the slopes are to be cut, in testing these samples in the laboratory for their strength, and then, with an analytical procedure, in determining the critical average slope angle for any given height of slope.

Where the wall rocks are of this type, the procedure is not only feasible but has the beneficial effect of showing that the appropriate slope angle varies with the height of the slope. In other words, if different parts of the pit are to have different depths, then logically the design slope angles should be different. Of course, the practical aspects of the layout of ramps and other considerations can override the purely technical factor of slope stability.

Another aspect of slope failure is the dispersion in strength values that must be considered when dealing with the fracture of any material (Coates, *et al*, 1963). In other words, if a slope were designed on the basis of the average strength with a safety factor of 1, then it could be expected that failure would occur for 50 per cent. of the wall rocks. This would arise from 50 per cent. of the material having a higher strength than the average and 50 per cent. having a lower strength than the average.

Table 1 shows a typical array of actual slides in a soft rock. Fig. 7 is a photo of one of these slides. The angle of internal friction of this material was 37°, and it was assumed that the ground water level was equal to 50 per cent. of the height of the slope. If the more probable situation, that the ground water level had been close to the surface of the slope, is assumed for Slide No. 7, the average cohesion of this soft rock would be analysed to be 1,000 lb./sq. ft or 7·6 lb./sq. in. This was very close to the results obtained from laboratory testing.

TABLE 1
Rock slides

Slide No	Height (ft)	Width (ft)	Slope angle, degrees	Cohesion (lb/sq ft)
1	215	290	51	1490
2	163	240	51	1150
3	51	300	60	685
4	173	120	50	935
5	138	125	56	1250
6	119	100	54	950
7	86	150	49	660
8	115	50	57	825
9	95	100	66	1195

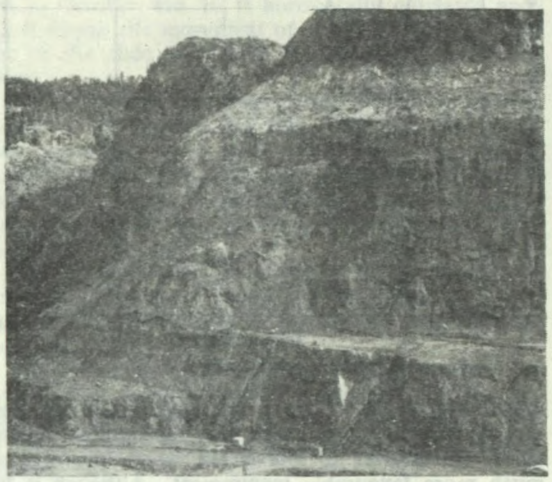

FIG. 7—A slide in a 215 foot high wall in soft rock.

It should be noted that for these nine slides the standard deviation of the cohesion is 240 lb./sq. ft, and the coefficient of variation is 22 per cent. From this it follows that 50 per cent. of the ground would have cohesions less than 7·6 lb./sq. in., and from statistical calculations it can be shown that one-third of the ground would have cohesions less than 78 per cent. of 7·6 lb./sq. in., or 5·9 lb./sq. in. Thus, to eliminate slides in this rock, slope angles considerably less than those determined from the average strength would be required.

In Fig. 8 typical design curves are shown that illustrate this point. Curve I indicates the required slope angle for the various heights of the slope to eliminate almost all danger of slope failure. Curve II would give the appropriate slope angle if 10 per cent. of the wall ground were permitted to develop slides. To determine the optimum design, it would be necessary to compare the cost of slides with the cost of waste excavation required to eliminate the possibility of these slides. The conclusion would, of course, be subject to modification in the light of other factors such as the consequences, aside from the cost of clean-up, of such slope failures.

Fig. 8 also includes the effect of changing the ground water regime behind a pit slope. In the case of Curve II the ground water level is equal to half the height of the wall. If it is possible to draw this

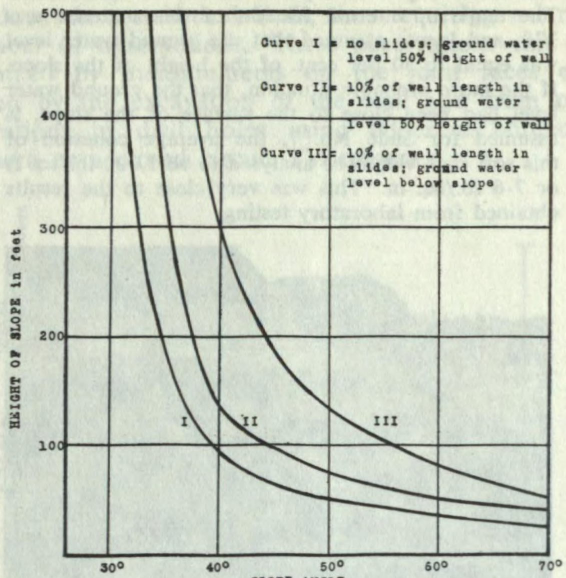

FIG. 8—Typical stability curves for soft rock.

water level down so that it no longer influences the stability of the slope, then the appropriate design curve moves over to Curve III, which provides a much more favourable requirement. Although it is easy to point out the implications of this type of action, it is not so easy to design a system that will draw down the water table in a rock mass and to ensure that the system is being effective. Actually, more field trials and measurements are required on this factor of controlling the ground water levels before it can be assumed to be one of the options available to the mine planner.

CASE HISTORY

In Fig. 9 the cross-section of a pit wall is shown in which several slides occurred during the mining of the ore. The ground consisted of a weak, altered slate and quartzite, which was massive and layered with beds dipping into the pit at angles between 40° and 75°.

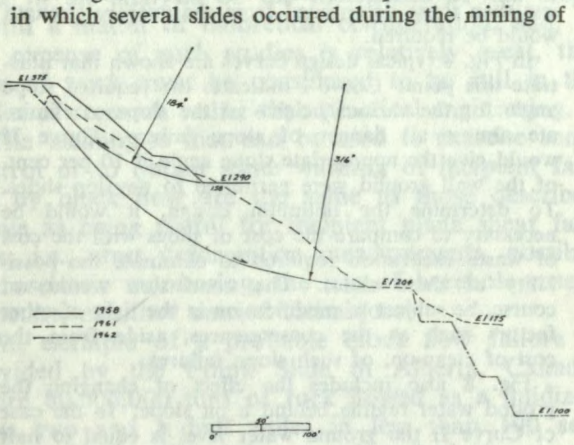

FIG. 9—Cross-section of the pit wall in altered slate and quartzite.

In 1958 when the crest of the slope was at El. 375 and the bottom of the pit was at El. 290, a rotational shear slide occurred that caused the crest of the wall to move down some 9 ft and out 4 ft. The radius of the circle of failure was about 184 ft. The toe of the slide was subsequently cleaned up when excavating for the ramp that was required for the mining to proceed down to lower levels.

FIG. 10—Tension cracks appearing in the crest of the wall in Fig. 8, July 1961.

In July 1961 with the toe at El. 204, new cracks opened up in the crest of the wall. For a period of about two months the crest moved at the rate of about 5 in. per week outwards and 1 in. downwards. No slide occurred, nor was there any evidence of movement in the toe zone.

In the spring of 1962 another rotational shea slide connecting with one of the tension cracks in the crest started. At this time the crest was at El. 370,

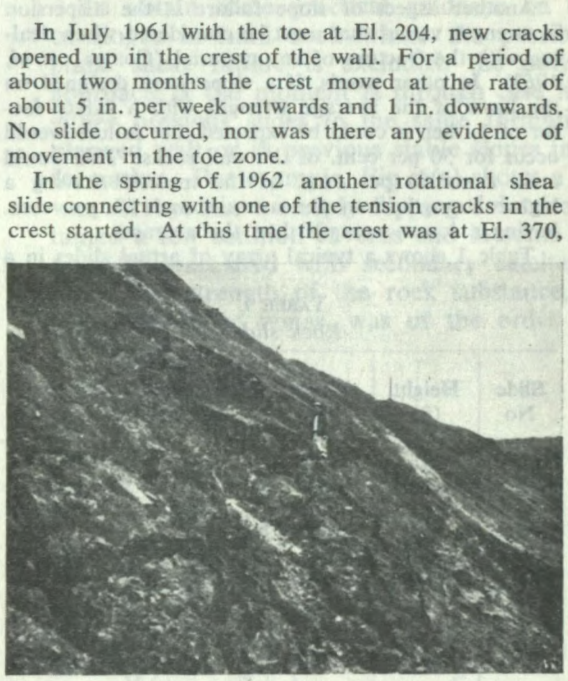

FIG. 11—A small slide in the same wall adjacent to the main slide zone, July 1961.

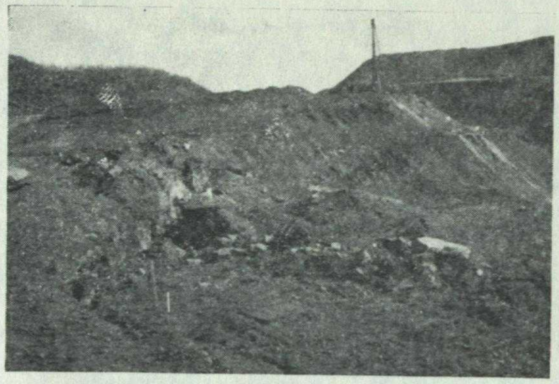

FIG. 12—The deterioration of the crest of the slope of Fig. 8, July 1962.

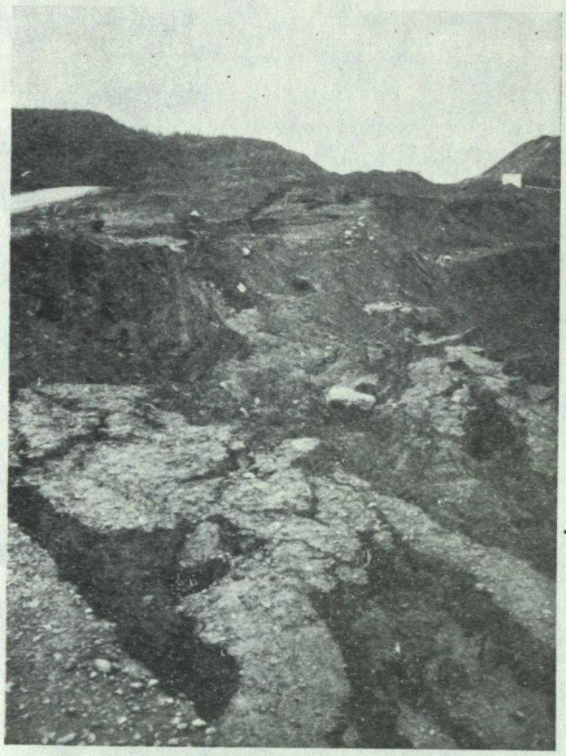

FIG. 13—The continuing cracking in the same crest, September 1962.

the toe at El. 175 with the ramp at El. 204. The movement of the wall was measured on several hubs placed on the surface of the ground. From this information it was deduced that the radius of movement was about 316 ft as shown in Fig. 9. The slide encroached on the ramp at El. 204, but the waste material only had to be dug out twice to permit mining to proceed.

The maximum hub movement, which was measured close to the crest, during the period from August to October was 16 ft horizontally outwards and 15 ft down; the movement of the hub closest to the toe of the slide moved 12 ft horizontally and 3 ft down.

Fig. 10 shows the appearance of some of the cracks in the crest during July 1961. Fig. 11 shows the appearance of a small slide in the same wall just beside the main slide zone that occurred during July 1961. The movement of this small slide during its active period was at the rate of about 5 ft per month.

Fig. 12 shows the deterioration of the ground at the crest of the slope during July 1962. Fig. 13 shows the continuing cracking at the crest during September 1962.

Until October 1962 the slide material at the toe of the wall continued to be excavated until the pit was completed down to El. 100. The filling of the pit with waste from the adjacent pits then started immediately.

The history of this slide illustrates the point mentioned above: to eliminate all slides would often require unacceptably conservative slope angles and with care it is sometimes possible to live with a slide.

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