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SLOPE STABILITY STUDIES AT KNOB LAKE

D.F. COATES, M. GYENGE AND J.B. STUBBINS

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SLOPE STABILITY STUDIES AT KNOB LAKE *

D. F. Coates,[1] M. Gyenge[2] and J. B. Stubbins[3]

Abstract

The object of the slope-stability studies at Knob Lake has been to obtain measurements in the open pits which, together with the results of laboratory tests, would permit the prediction of instability in the incompetent wall rocks. Some work is also being done in the brittle, hard wall rocks; however, only data concerning the incompetent rocks are presented here.

An initial study of the slides that had occurred in the past indicated that it might be possible to determine the strength of wall rocks at any particular section by obtaining core samples that could be tested in the laboratory. Then based on the strength obtained from the laboratory tests together with some knowledge of the structural geology and using the appropriate theory, it was hoped that ultimate pit slopes that would become unstable could be identified before the slopes were created.

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* FMP 64/121-MRL

[1]Head, [2]Project Leader, Mining Research Laboratories, Fuels and Mining Practice Div., Mines Br., Dept. Mines & Tech. Surveys., Ottawa.

[3]Chief, Engineer, Knob Lake Operation, Iron Ore Company of Canada, Schefferville. Que.

Laboratory studies were initially carried out on bag samples that were recompacted in the laboratory to the same densities that were measured in the field. The results of these tests indicated that the recompacted samples had strength values commensurate with the strength deduced from previous slides. A series of operations were then conducted to obtain undisturbed samples for laboratory testing. This work is still proceeding but difficulties have been encountered in sampling and preparing specimens from a material which on a large scale is soft but on a small scale is made up of blocks of hard rock in a softer matrix.

Laboratory Tests

The orebodies occur generally in structural troughs, formed either by distorted and broken synclines or by the intersection of homoclinally folded strata and a major strike fault (1). Ruth slate and Wishart quartzite lie adjacent to the ore zone and hence form the walls in many of the pits.

The Ruth slate is finely laminated, fissile and ferruginous. It is commonly interbedded with thin layers of chert. Near the ore the slate is soft and composed chiefly of an intimate mixture of very fine-grained quartz, sericite, kaolinite and iron oxides. On a sample passing the 4-mesh about 55% of the particles are smaller than the 200-mesh.

The Wishart quartzite stratigraphically underlies the Ruth slate. It is generally composed of equigranular quartz grain cemented by an extremely fine-grained quartz. The rock also contains minor amounts of iron and alumina. Near the orebody the rock is altered by the partial removal of the quartz cement leaving a rock that is more or less friable.

Initial laboratory testing of these rocks was done on broken material recompacted back to in-situ densities. Triaxial compression tests were used to determine the variation of the strength of the samples with strain rate.

The tests showed that on both rocks whereas the strength would be decreased by long duration loads, or by slow strain rates, compared to relatively quick tests, the stiffness (modulus of deformation) of the samples did not decrease with decreasing strain rate. From these tests, it was concluded that the rocks would not exhibit creep or viscous properties aside from any deterioration, of course, that would occur with weathering. In addition, the appropriate strength values for long-duration loadings in material equivalent to these uncemented samples would be represented by an angle of internal friction of 34° with cohesions between 450 psf and 750 psf. These numbers should represent minimum strengths as some formation might not have been completely altered and thus would have higher cohesion through some retained cementation.

Slides

In Table 1 the various slides that have occurred in these incompetent wall rocks are listed. These slides have been analysed to determine the effective cohesion that was acting at that section. As the general, undisturbed water table is usually less than 100 ft below the surface in this area, it has been assumed for the purpose of calculating the effective cohesion in these slides that the ground-water level behind all slopes was half the slope height. In addition, it was assumed that the effective angle of internal friction was 34° and that all slides were preceded by tension cracks in the crests which extended to a depth of one quarter of the height of the slopes.

TABLE 1

Compilation of Slides

Slide No.	Height, ft	Slope Angle degrees	Cohesion, psf
1	100	36	410
2	136	41	765
3	114	42	665
4	132	40	695
5	140	43	860
6	106	44	690
7	71	52	770
8	110	36	310
9	67	43	530
10	80	50	870
11	110	50	1200
12	54	47	530
13	94	38	530
14	130	35	340
15	145	48	1500

The average cohesion acting in these slides is calculated to be 710 psf. The standard deviation is 317 psf and the coefficient of variation 45%. If the above coefficient variation is compared with those obtained for manufactured materials such as concrete and steel, it is found that these supposedly uniform materials have a dispersion in their strength values of the order of 25%. Whereas these wall rocks are more variable than these construction materials, the difference is not so great that a similar approach cannot be used for the structural problem of slopes.

It can be seen from the above compilation of slides that the average value of cohesion in these walls fell within the range obtained from the recompacted samples in the laboratory. The range of values, however, was greater than that obtained on the laboratory samples. Values greater than about 750 psf probably indicate that some cementation existed in the ground. For computed values less than about 450 psf, as shown by some of the cases in the above table, it is possible that the actual ground-water level behind these slopes was greater than the height assumed for the calculations. In other words, had there been factual information on these cases, the computed cohesion would have been greater than the numbers obtained and the dispersion of the total reduced.

Case History A

In Figure 1 the cross-section of a pit wall is shown in which several slides occurred during the mining of the ore. The ground consisted of the altered Ruth slate and Wishart quartzite, which in this area is relatively massive with dip angles between 40° and 75°.

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 deduced to be about 184 ft. The toe of the slide was subsequently cleaned up when excavating for the ramp that was required for mining to proceed down to the lower levels.

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 cracks moved at the rate of about 5 in. per week outwards and 1 in. per week downwards. No actual slide occurred, nor was there any evidence of movement in the toe zone.

In the spring of 1962 another rotational shear slide, connecting with one of the tension cracks in the crest, started. At this time the crest was at El. 370, 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 the movement was about 316 ft as shown in Figure 1. 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 was 12 ft horizontally and 3 ft down.

Figure 2 shows the appearance of some of the cracks in the crest during July 1961. Figure 3 shows the appearance of a small slide in the same wall just beside the main slide zone that occurred during July 1961. The total movement of this small slide during its active period, which was typical of slides in this material, was at the rate of about 5 ft per month.

Figure 4 shows the deterioration of the ground at the crest of the slope during July 1962. Figure 5 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 a point that has been mentioned before (2). Dispersion in strength values must be considered when dealing with the fracture of any material. In other words, if a slope were designed on the basis of the average strength with a safety factor of 1 (assuming it were possible to determine such an average which included geological factors), then it could be expected that failure would occur for 50% of the wall rocks. This would arise from 50% of the material having a higher strength than the average and 50% having a lower strength than the average.

With sufficient information on the dispersion of strength values it would theoretically be possible to determine the slope angle either to eliminate all slides or to permit some acceptable but low percentage of the wall ground to fail. 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 probably follow that, modified by the consequences of failure, optimum economy would require that some of the walls should fail.

The above case history illustrates that with care it is sometimes possible to live with a slide, and thus the explicit decision to permit some minor percentage of the ground to fail would be quite reasonable.

Case History B

Figure 6 shows the plan and Figure 7 the cross-section of the pit wall in which several slides occurred over a period of years.

The first slide in this area occurred during the summer of 1961 and was located at Section 38. This slide took place across very flat bedding in altered Ruth slate. The movement was quite gradual.

During the winter of 1961-62 stripping operations took the toe down to El. 2060 with the crest at this section being at El. 2230. Along the wall at El. 2130, as shown in Figure 7, a haul road was located. The part of the wall between El. 2130 and El. 2060 had an average angle of 45°. Figure 8 shows the appearance of part of the face on May 14, 1962.

During May, 1962, mining operations in the next lower lift approached the toe of the wall in this area. On May 31 tension cracks opened up at the edge of the haul road at El. 2130. During the night of May 31 the wall failed between section 36 and 37 and the edge of the road dropped 5 ft. The geological structure included a large synclinal fold which plunged toward the pit together with some smaller drag folds such as are shown in Figure 8.

As mining operations required the use of the haul road, fill was dumped at the crest of the slide to maintain the road width. Slow, continuous movement was observed during the balance of 1962.

During the winter of 1962-63 the faces of the slope were cleaned and mining continued to El. 1980. The slope angle between the toe at this elevation and the haul road was about 35°. The slide continued to move in spite of sub-zero temperatures.

In the spring of 1963 the sliding motion increased, and it was apparent that the major portion of the bank was failing along a width of over 400 ft. The motion was slow and continuous.

The movement of the wall was observed on several hubs placed on the surface of the ground and also on the new tension cracks which had formed across the haul road. By July this crack had opened up to a width of 2 ft with the edge of the road dropping about a foot as shown in Figure 9.

The toe of the slide continued to move out, and by the end of September it had moved as much as 50 ft. It was judged that the failure plane was a combination of plane failure in the central part, probably on the interface between the slate and the underlying quartzite, together with rotational failure at the ends of the segment.

It is possible that failure was assisted by the effects of seepage water at this interface. During the summer of 1963 as part of the experimental remedial program, three horizontal drains were drilled through the slate into the more porous quartzite with the hope of decreasing the pore-water pressures and thus increasing the sliding resistance. A small flow was obtained, as shown in Figure 10, which increased during rainy periods; however, by the end of July the continued movement of the slide broke the pipes and the drainage action ceased.

A main haul road was then required below the slide area at El. 1920. To safeguard this road the average angle of the bank in this zone was decreased by moving back the wall above the old haul road. This work was done during the winter of 1963-64 with the removal of approximately 100,000 cu yards. When the ground at the crest of the slope was removed, the sloughed material at the toe of the slide was excavated with the material above the toe moving down under its own weight without any hangup. However, the removal of the slide material from the toe reactivated the old slide as shown by the cracks in Figure 11.

By the spring of 1964 the toe of the bank had been stripped down to El. 1920. A new, larger slide outlined in Figure 6 then started to move down very quickly. The sliding surface in this case was in still slate.

By analysing the conditions of the slide and assuming that the ground-water conditions would remain substantially the same, it was deduced that the stable ultimate slope angle here would be about 30°. The reason for this angle being less than the angle of internal friction of 34° is that the seepage pressures of the ground water must be of a significant magnitude. Any alteration of the ground-water conditions so that the ground-water level was a different ratio to the height of the slope than had prevailed in the past would modify the calculated ultimate stable slope. However, experience in these studies has shown that in these bedded and altered deposits it is very difficult to obtain information on the general ground-water level in the immediate area of slide (no holes can be collared in the moving and sloping ground and, for analyses purposes, this is where they are needed) and, with the low permeability of much of the material that is failing, wells are not very successful in draining the critical area.

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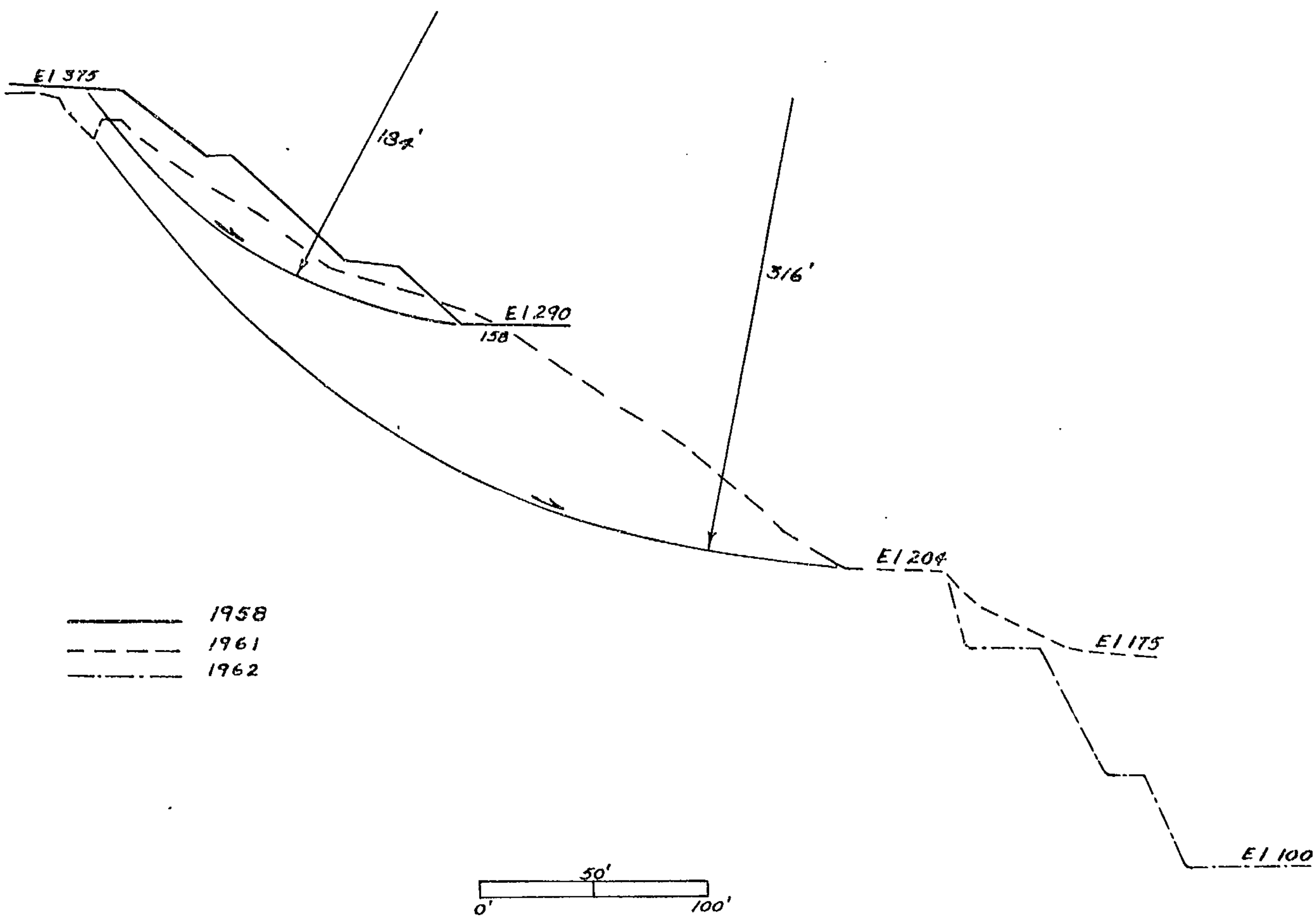


Figure 1. Cross-section of the pit wall for Case History A

Figure 2. Tension cracks appearing in the crest of the wall in Figure 1, July, 1961.

Figure 3. A small slide in the same wall, adjacent to the main slide zone, July, 1961.

Figure 4. The deterioration of the crest of the slope of Figure 1, July, 1962.

Figure 5. The continuing cracking in the same crest, September, 1962.

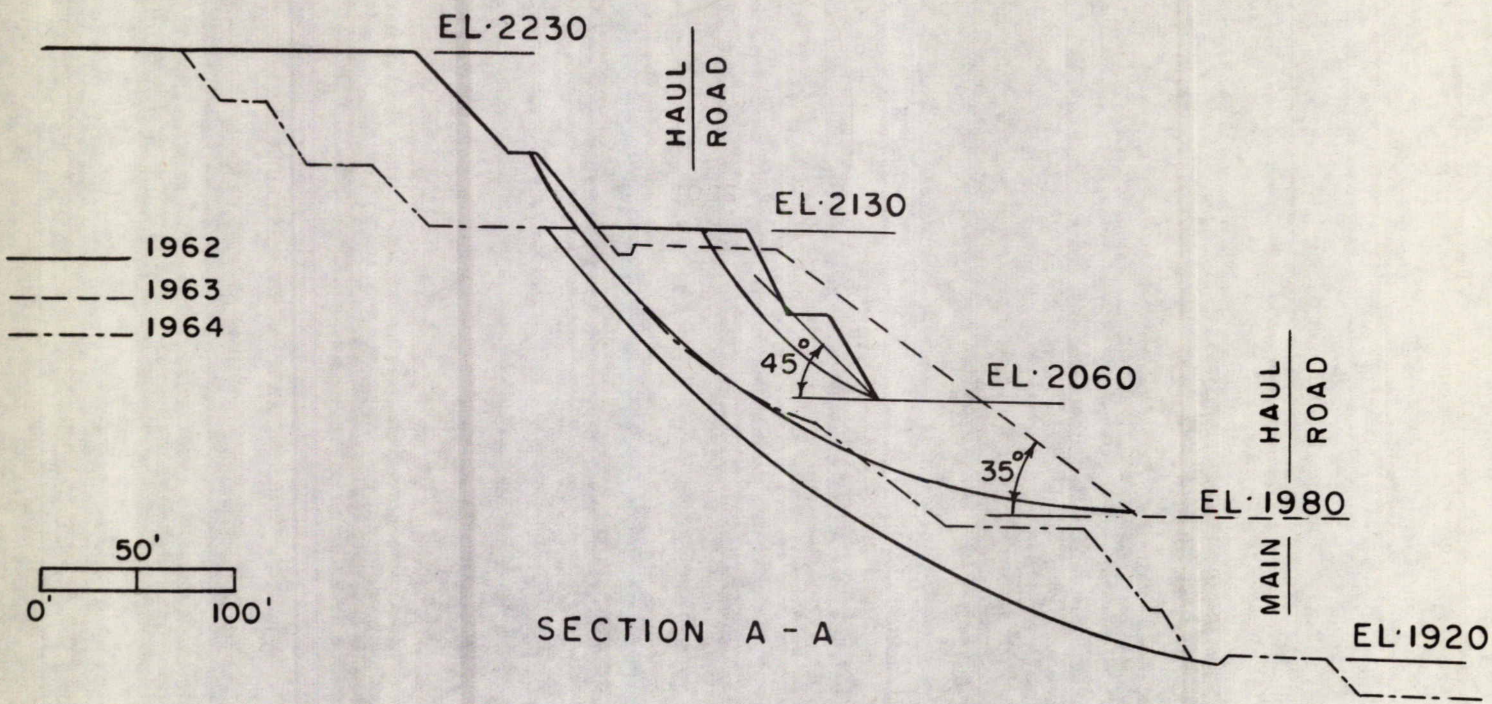
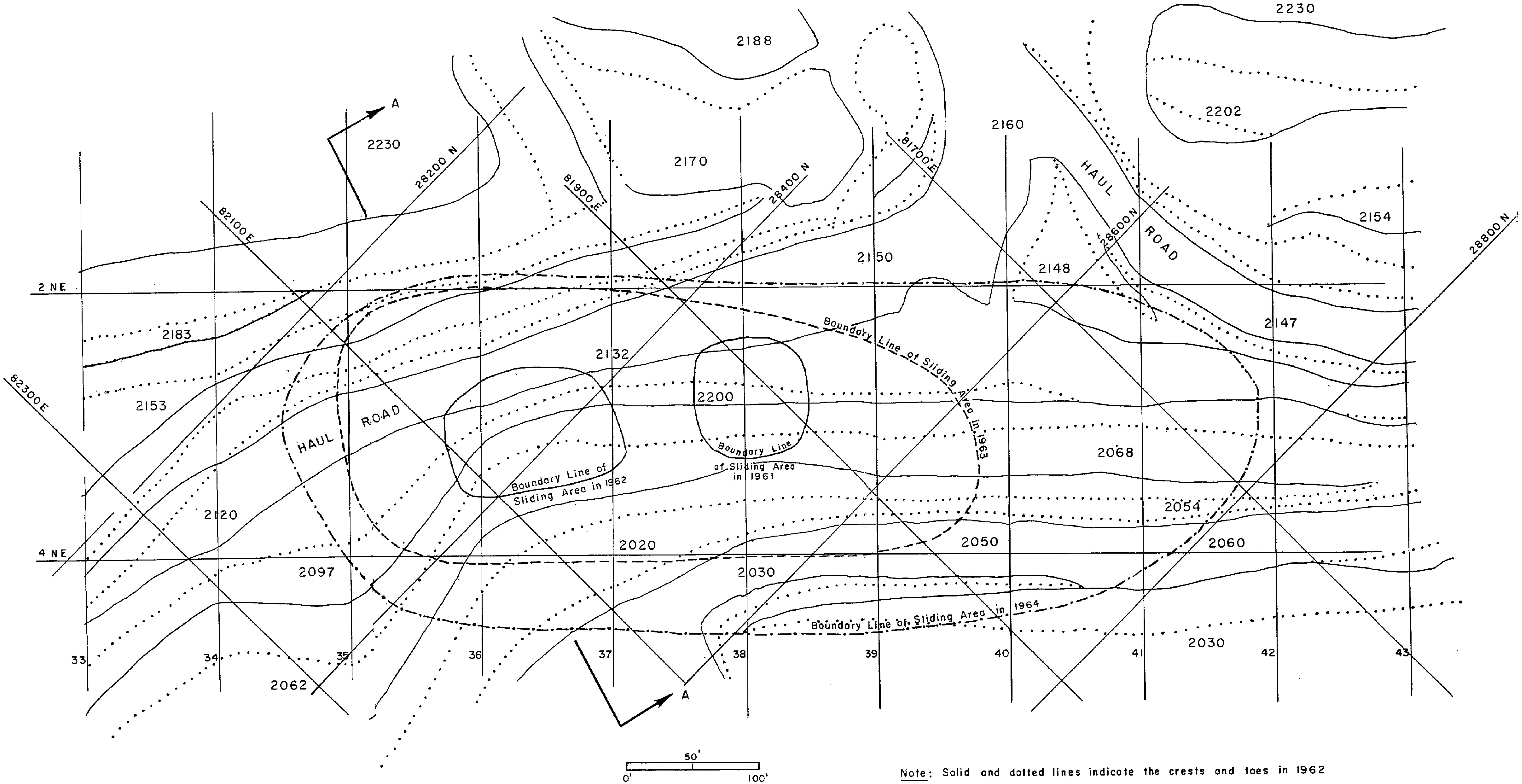


Figure 7. Cross-section of the pit wall for Case History B.

Figure 8. Appearance of part of the face before the first slide occurred, May, 1962.



Note: Solid and dotted lines indicate the crests and toes in 1962

PLAN OF THE PIT WALL FOR CASE HISTORY B

FIGURE 6

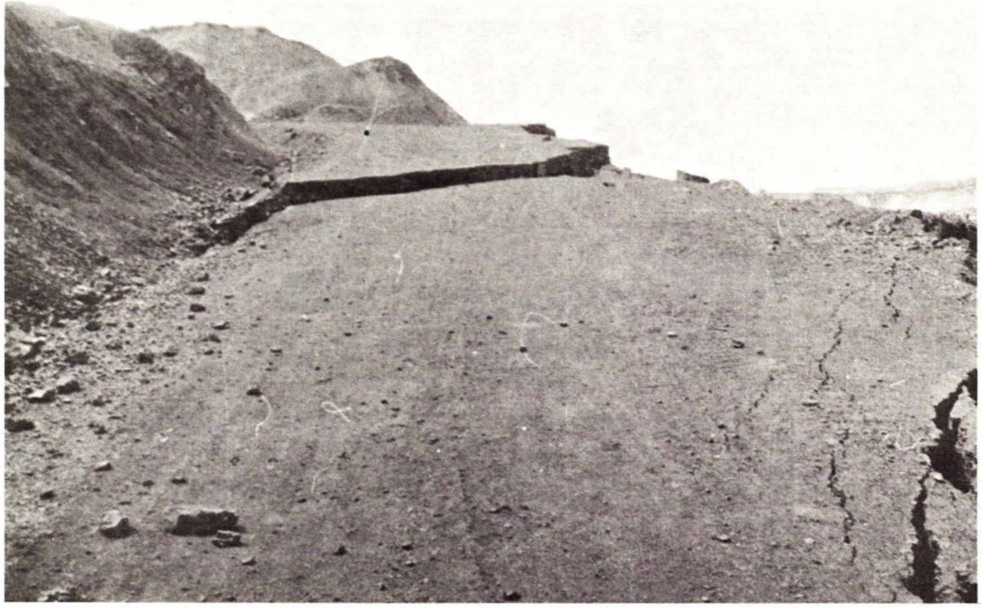


Figure 9. Two-foot-wide crack at the crest of the second slide, July, 1963.

Figure 10. Horizontal drainpipe, July, 1963.

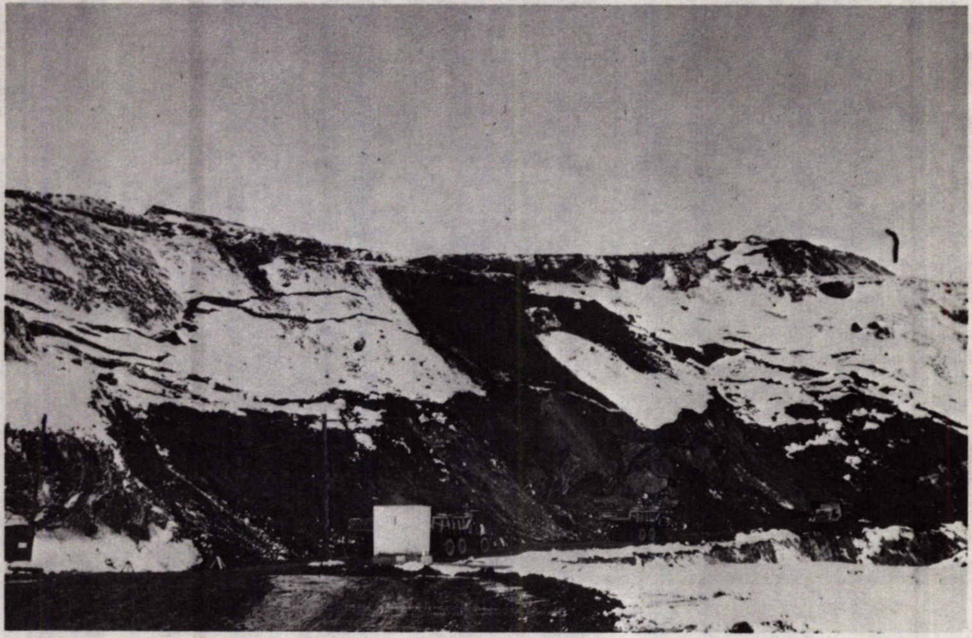


Figure 11. Shovel digging at the toe reactivates the slide, winter of 1963-64.

DISCUSSION

Professor B. Ladanyi, Dept. Civil Engineering, Laval University, Quebec.

Would it not be possible, in the case of such an incompetent rock, to use some kind of deep penetration test for extrapolating the results obtained on the core samples in the laboratory?

AUTHOR'S REPLY

Consideration has been given to the use of a borehole penetrometer, which permits the measurement of the resistance of any stratum to the penetration of a standard size piston for a given indentation at any location along a borehole. To conduct a general appraisal of this technique such a penetrometer is being constructed in Ottawa after the design made by the Pittsburgh office of the U.S. Bureau of Mines. Our intention is to study the correlation between penetration readings and laboratory tests on samples from the same borehole.