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ANALYSES OF PIT SLIDES IN SOME INCOMPETENT ROCKS

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ANALYSES OF PIT SLIDES IN SOME INCOMPETENT ROCKS

by D. F. Coates, K. L. McRorie and J. B. Stubbins

Twenty-two pit slides that occurred in two Canadian open pit mining properties are analyzed. Information on the results of laboratory tests of the rocks and a brief description of the geological environment are also presented. One deduction from the study is that slides seem to be predictable by using established theory. This corroboration indicates that, contrary to normal mining practice, the appropriate slope angle should vary with the height of the wall. In addition, if the general ground water level around an open pit can be reduced, the appropriate slope angles of the walls could then be increased by a determinable amount. The ultimate objective of these investigations is to determine the relationship between the percentage probability of failure of any slope designed with specific strength parameters. With this relationship it would then be possible to determine optimum slope angles.

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Because of changes in various economic factors, open pits are now being mined to much greater depths than previously. The need to keep stripping ratios low without endangering lives and equipment provides a strong incentive to determine the maximum practicable wall slopes. Applied research work is currently being conducted to establish the methods for analyzing optimum slopes.

This paper gives the results of the initial studies that were made on two properties. The work has indicated the possibility of predicting slides in these rocks. More detailed studies are now being made to obtain information on important factors that have had to be assumed for this preliminary work, e.g., ground water levels, failure surfaces, deformation rates and variation of wall strengths.

To assist in studying and discussing the subject of slope failures, all slides may be classified into one of four groups. Fig. 1 presents a description of a suggested classification system.

The first type of failure described in Fig. 1 is that of the rock fall. This is simply the fall of loose blocks when the slope angle is greater than the angle of repose of the blocks.

Rotational shear failure, the second type, produces a movement of an almost undisturbed segment along a circular or spoon-shaped surface and occurs mainly in nonbrittle materials. Such materials typically would be either soils (cohesive or granular) or a lightly consolidated unjointed rock similar in physical properties to cohesive soils.

Plane shear failure, the third type, results when a weak geological surface exists within the slope in such a direction as to provide a preferential path for failure.

Block flow, the fourth type, is the term given to a slope failure when there is a general breakdown of the rock mass. To understand the nature of this type of failure it is necessary to recognize that brittle rocks differ from soils. For example, brittle 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 blocks may or may not be cemented together. However, the strength of the cement between the blocks is normally less than that of the blocks themselves.

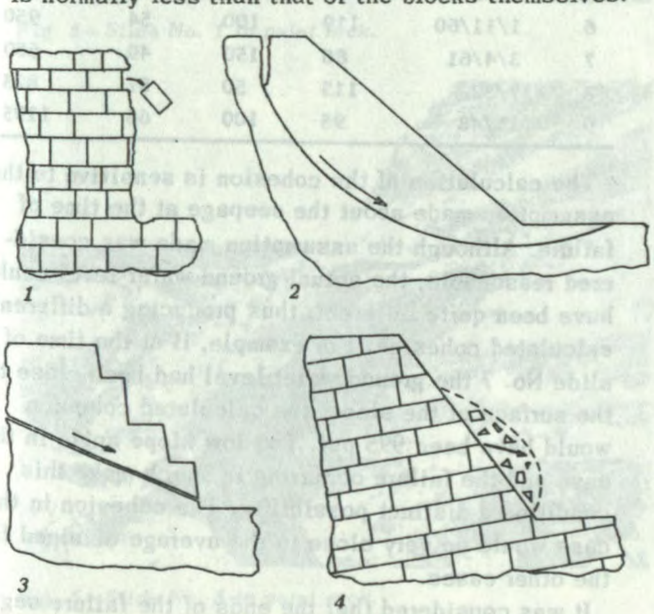

Fig. 1 — Classification of types of slope failure: 1) rock fall, 2) rotational shear, 3) plane shear, 4) block flow.

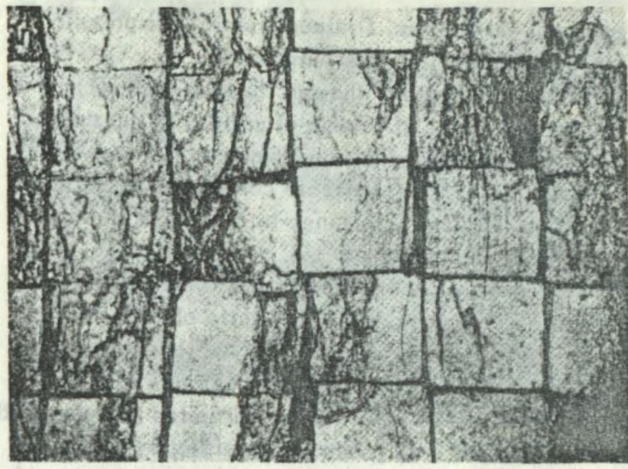

Fig. 2 — Individual blocks failing within a block mass subjected to pressure.

In addition, the basic nature of brittle materials is that they rupture completely on failure and thus are not likely to permit any plastic yielding or redistribution of stresses before the general failure of the mass. Consequently, when a typical brittle rock mass, without complicating structural features, is subjected to stresses high enough to cause crushing of the blocks, failure of the mass will usually occur (see Fig. 2).[1]

PAINT ROCK SLIDES

Nine slides have been recorded in a material that is known on the property as paint rock. This paint rock is part of the footwall in the Steep Rock ore zone, which is a steeply dipping hematite-goethite deposit some seven miles long. The dip of the ore varies between 62° and 77°.

The paint rock is a soft, incompetent, fine-grained mass of quartz, pyrolusite and kaolin with subangular fragments of chert, hematite and goethite. When a sample is broken down into its individual particles with a rubber tipped pestle, a sieve analysis on the −4-mesh fraction shows that about 20% of the particles are smaller than the 200-mesh sieve. The formation is generally made up of laminations less than 1 cm thick.

Two open pits have been mined out of this ore zone, the Errington and the Hogarth, from which the following slide data has been obtained. The Errington mine was about 3000 ft long and the Hogarth mine was close to 5000 ft long. A vertical depth of 300 to 400 ft of ore was mined out of each pit; however, since the bedrock originally underlay some 250 ft of overburden at the bottom of Steep Rock Lake, the pit walls were, in some cases, considerably higher than the mining depth.

At the beginning of the investigation it was not known whether the paint rock was a purely cohesive material or whether it would have distinct frictional properties. Consequently, recompacted laboratory samples were used to obtain some of the basic strength parameters. Drained triaxial compression tests were used for this purpose.

Field density tests showed that the void ratio (volume of voids to volume of solids) of the undisturbed paint rock might vary between 0.4 and 0.7. The laboratory samples were compacted to different void ratios within this range to determine the effect of density on strength.

The results of one series of triaxial tests run on saturated samples with a void ratio of 0.56 showed the material to have an effective angle of internal friction of 36° and an effective cohesion of 6 psi. Another series of tests run on saturated samples with a void ratio of 0.49 produced an effective angle of internal friction of 38° and an effective cohesion of 13 psi. In both series the rate of strain was 0.288% per min. The Mohr failure envelopes for these two series of tests are shown in Figs. 3 and 4.

For the case of rotational shear failure,

$$i = f\left(\frac{c}{YH}, \frac{d}{H}, \phi\right)$$

where i is slope angle measured from the horizontal at the point of incipient failure, f is the symbol for a functional relationship, c is cohesion of wall rock, Y is bulk density of the wall rock, H is height of slope, d is depth from the crest of the slope to the ground water level and ϕ equals the angle of internal friction of the wall rock.

This expression indicates that the slope angle, i, varies with the parameter c/YH. It is known that the slope angle increases with c/YH. For example, if the height of slope, H, is increased, c/YH decreases and the maximum stable slope angle, i, also decreases.

Similarly, the slope angle varies or increases with d/H and ϕ. Thus, if the ground water level rises, d/H decreases and consequently the slope angle, i, decreases. The quantitative functional relationships for most cases cannot yet be expressed analytically; however, stability charts can be used for the approximate solution for an unknown parameter.[2,3]

In Table I the cases of slope failure in the paint rock are listed. The height recorded is that of the actual slide zone before failure. Similarly, the slope angle is the average angle to the horizontal of the actual slide. The width recorded is the average width which might have influenced the amount of shear resistance the ends of the slide would have had on the critical shear stress. The majority, if not all, of the failures probably occurred by rotational shear.

The cohesion listed in the table has been determined by using an effective angle of internal friction of 37° and by assuming the ground water level behind the slope to be at an elevation equal to half the height of the slope. Circular arcs of failure with tension cracks equal to 50% of the height of the slope are assumed in these analyses.

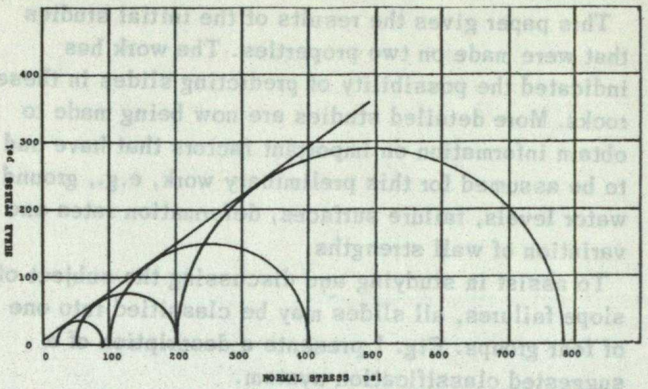

Fig. 3—Mohr diagram for paint rock with a void ratio of 0.56.

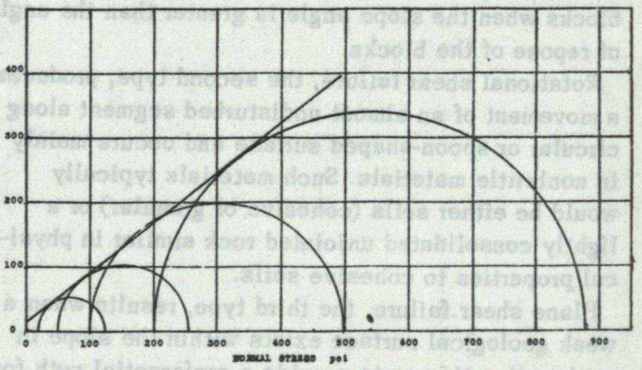

Fig. 4—Mohr diagram for paint rock with a void ratio of 0.49.

Table I. Paint Rock Slides

Slide No.	Date	Height, Ft	Width, Ft	Slope Angle, Degrees	Cohesion, Psf
1	3/11/60	215	290	51	1490
2	1/60	163	240	51	1150
3	—	51	300	60	685
4	11/24/60	173	120	50	935
5	12/6/60	138	125	56	1250
6	1/11/60	119	100	54	950
7	3/4/61	86	150	49	660
8	7/52	115	50	57	825
9	11/48	95	100	66	1195

The calculation of the cohesion is sensitive to the assumption made about the seepage at the time of failure. Although the assumption made was considered reasonable, the actual ground water level could have been quite different, thus producing a different calculated cohesion. For example, if at the time of slide No. 7 the ground water level had been close to the surface of the slope, the calculated cohesion would have been 995 psf. The low slope angle in this case and the failure occurring in March make this condition a distinct possibility. The cohesion in this case would be very close to the average obtained for the other cases.

It was considered that the ends of the failure segments might have a differential effect on the computed cohesions from the different slides. Therefore, an average slide segment was postulated, and a correction factor that varied with the width and height of the slide segment was computed. This factor was then included in the analyses to eliminate the resistance of the ends.

The average cohesion deduced from these slides, using the full seepage case for slide No. 7, was found to be 1100 psf or 7.6 psi. The standard deviation for these nine slides is 240 psf and the coefficient of variation is 22%. In other words, about two thirds of the observed failures produced computed cohesions within ±22% of the average. The laboratory results for cohesion (6 to 13 psi) bracket the average value obtained from analyzing the slides.

Fig. 5 shows slide No. 1 in paint rock. The height of this slide was 215 ft with an average slope angle of 51°. Fig. 6 shows slide No. 5 in paint rock. The height of this slide was 138 ft with an average slope angle of 56°. Fig. 7 is a face-on shot of the same slide, demonstrating that the end effects would be very important and would, if not eliminated, seriously distort the computed cohesion. Fig. 8 shows slide No. 7 in the paint rock. The height of this slide was 86 ft with an average slope angle of 49°. Since the slide is on a nose of wall the resistance on the ends of the sliding segment would be very small.

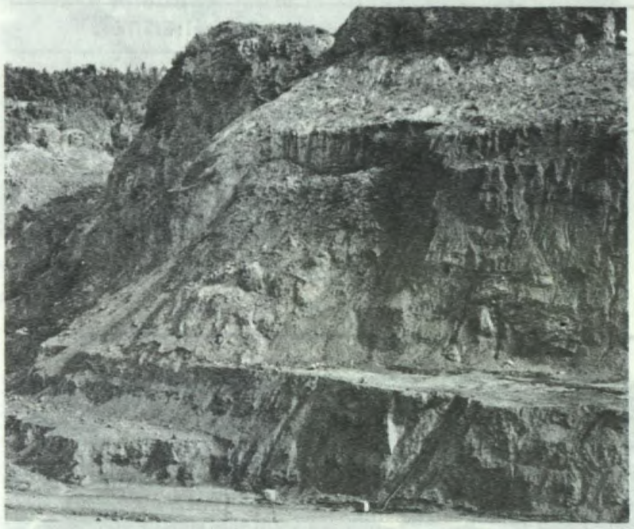

Fig. 5 — Slide No. 1 in paint rock.

Fig. 6 — Slide No. 5 in paint rock.

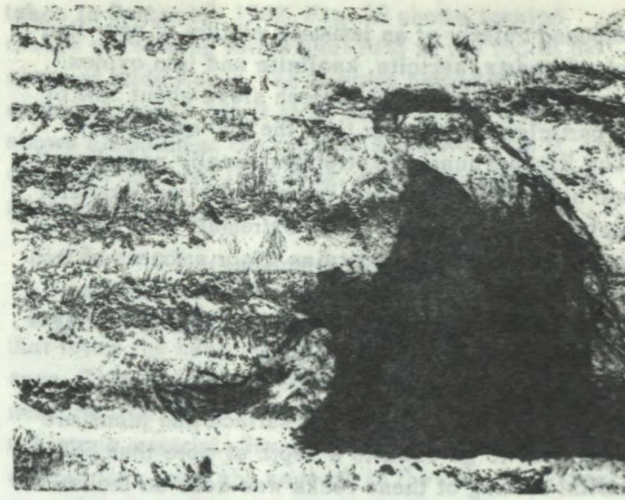

Fig. 7 — Detail of slide No. 5 in paint rock.

Fig. 8 — Slide No. 7 in paint rock.

ALTERED SLATE AND QUARTZITE SLIDES

Six slides have been recorded in the associated Ruth and Wishart formations in the Ruth Lake mine of the Iron Ore Co. of Canada in northeastern Quebec and Labrador. The orebodies in this area occur generally in structural troughs, formed either by distorted and broken canoe-synclines or by the intersection of homoclinally folded strata and a major strike fault.[4] In the Ruth Lake mine these two formations lie adjacent to the ore zones and hence form in most places the pit walls. Most of the ore lies below the water table, which is generally less than 100 ft below the surface in the mining area.

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 sieve about 55% of the particles are smaller than the 200-mesh sieve.

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

The Ruth Lake mine is about 5000 ft long; mining is carried on 250 ft below the surface and plans are being made to go down to a depth of more than 550 ft.

Initial testing of these rocks was done on broken material recompacted to in situ void ratios. Undrained triaxial compression tests with pore pressure measurements were used to determine the strength of the samples. The initial series of tests on the Ruth slate on saturated samples with a void ratio of 1.06 produced an effective angle of internal friction of 33° and an effective cohesion of 7 psi. A similar series of tests on the quartzite produced an effective angle of internal friction of 34° and an effective cohesion of 15 psi. These tests were run at a strain rate of 0.60% per minute.

Additional tests were run on the quartzite to determine if slower strain rates would affect the strength properties of this rock. At a strain rate of 0.08% per min the stiffness (modulus of deformation) of the samples was not decreased. Hence the rock should not exhibit creep properties at stresses below failure. The strength parameters, however, were affected by the decreased strain rates. An angle of internal friction of 34° and a cohesion of 5 psi seem to represent minimum values that would exist for long duration loadings. Also, tests on unsaturated samples indicated that an addition to the safety factor in any slope would result if unsaturated conditions occurred.

In Table II the cases of slope failure in these altered slates and quartzites are listed. The cohesion has been calculated assuming the effective angle of internal friction was 34°, the ground water level behind all slopes was 50% of the slope height and tension cracks had occurred to a depth of 25% of the height of the slope. Circular arcs of failure are used in these analyses.

The average cohesion in these slides was found to be 680 psf or 4.7 psi. The standard deviation is 151 psf and the coefficient of variation 22%. These coefficients of variation are surprisingly low when it is considered that current construction projects only produce concrete with coefficients of variation between 18% and 25%.[5]

Because of the lack of information on past slides,

Table II. Altered Slate and Quartzite Slides

Slide No.	Date	Height, Ft	Slope Angle, Degrees	Cohesion, Psf
1	6/29/58	100	36	410
2	7/19/61	136	41	765
3	11/17/60	114	42	665
4	9/13/61	132	40	695
5	4/11/61	140	43	860
6	5/31/62	106	44	690

Fig. 9 — Aerial photograph of Ruth Lake mine, September 1962 (photo by Hunting Survey Corp. Ltd., Toronto).

Fig. 10—Tension cracks developing at the top of the slope of slide No. 2 in altered slate and quartzite.

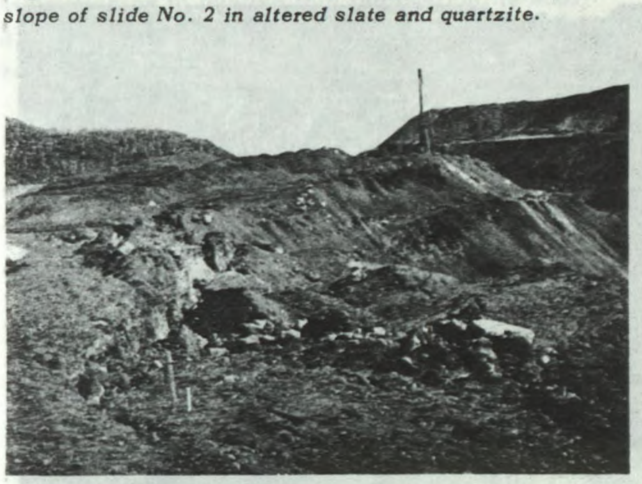

Fig. 11—The crest of slide No. 2 in altered slate and quartzite.

Fig. 12—Some remains of slide No. 6 in altered slate and quartzite.

no correction was made in Table II for the end effects on the sliding segments. The average cohesion obtained from these slide analyses was very close to the results of the laboratory tests at slow rates of strain. With an end correction the average computed cohesion would be decreased by about 15%, which would still make it very close to the laboratory value.

Fig. 9 is an aerial photograph of Ruth Lake mine taken in September 1962. Fig. 10 shows tension cracks developing in the haul road at the top of the slope of slide No. 2. The average height of the wall at that time was 136 ft at an angle of 41°. Fig. 11 shows slumping at the crest of slide No. 2 as it appeared one year later. Fig. 12 shows some of the remains of slide No. 6.

ASH ROCK SLIDES

Seven slides have been recorded in the ash rock that forms the hanging wall of the Steep Rock ore zone. This rock varies from a soft, altered material near the iron ore to a competent, brittle somewhat schistose material. Six of the slides occurred in the altered rock, with the seventh in the more competent part of the formation.

The ash rock has some unusual lithologic features. A typical specimen contains dark green to black, lenticular, aphanitic, serpentinized fragments generally less than ½ in. in size in a greenish schistose matrix. The rock is probably a pyroclastic of an unusually basic type.[6] Alteration can consist of a decrease in silica content with an increase in iron content in the form of either hematite, limonite or pyrite.

Triaxial compression tests were conducted on core samples of the more competent rock at confining pressures of 100, 500 and 1000 psi. From the fracture angles shown by these tests it might be deduced, by being somewhat selective, that the angle of internal friction of these samples should be between 34° and 46°. The samples were not sufficiently homogeneous to determine the angle of internal friction from a Mohr envelope.

In addition, cyclical stress applications in a uniaxial compression test to increasing levels of stress showed that there was no plastic component of strain in the competent ash rock at compressive stresses up to 5000 psi.

The data on the seven slides in the ash rock are recorded in Table III. The cohesion is computed assuming that the effective angle of internal friction was 40°, that the ground water level was at a height of 50% of the slope, that the tension cracks extended

Table III. Ash Rock Slides

Slide No.	Date	Height, Ft	Width, Ft	Slope Angle, Degrees	Cohesion, Psf
1	7/52	160	350	48	840
2	8/59	100	200	56	820
3	6/53	85	50	61	780
4	10/27/60	85	200	63	1160
5	12/12/60	145	125	40	610
6	3/2/61	84	125	46.5	410
7	—	445	450	51	2620

to 50% of the depth of the slope and that failure occurred along circular arcs.

Excluding slide No. 7, which was in the competent part of the formation, the average calculated cohesion was found to be 770 psf or 5.3 psi. The standard deviation is 250 psf and the coefficient of variation is 33%.

It is probable that slide No. 7 is an example of block flow failure. In this case the underlying assumption of plastic redistribution of stress concentrations enabling average shear stresses to be compared to average shear strengths would not be applicable.

Furthermore, it is possible that the frictional resistance would not be mobilized before the cohesion in the material was broken down. In an attempt to appraise this possibility the angle of internal friction of the rock in slide No. 7 was assumed to equal zero, and the critical average shear stress was computed. This stress amounted to 17,600 psf or 122 psi. This is still considerably lower than the shear strength of the core samples of the competent rock where the average uniaxial compression strength was about 4500 psi. Consequently, the mechanism of stress concentrations on blocks within the formation and progressive failure probably was effective in this case.

Fig. 13 shows slide No. 6 in ash rock. The height of this slide was 84 ft at an average angle of 46.5°. Fig. 14 shows slide No. 7 in ash rock. This wall was 445 ft high at an angle of 51° before the slide. The appearance of the wall after the slide leads one to conclude that the failure was by block flow.

Fig. 13 – Slide No. 6 in ash rock.

Fig. 14 – Slide No. 7 in ash rock.

CONCLUSIONS

The initial appraisal of the predictability of slides in these various materials has been encouraging. Consequently, work is continuing to determine actual ground water levels, failure surfaces, deformation rates and the strength and variation of strength of the in situ material by undisturbed sampling and laboratory testing.

By corroborating accepted theory, the analyses of these slides implies quite clearly that the slope angle of pit walls should vary with the height of the wall, and that if ground water levels can be reduced, the design slope angle can be increased (see Fig. 15).

It may be possible to establish an approximate probability curve, showing the percentage probability of failure for any slope designed with certain strength parameters. When this curve is defined it will be possible to determine the volume of slides that should be tolerated for maximum economy of mining. This outcome is anticipated by recognizing that, even with a moderate variation in strength properties, the slope angle required to eliminate all failures would generally be unacceptably low (see Fig. 15).

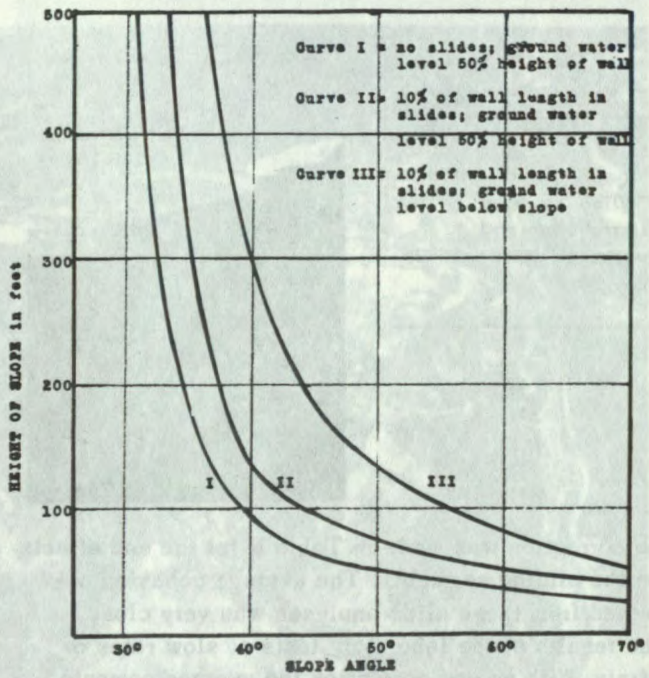

Fig. 15 – Typical stability curves for incompetent rock. Curve I: no slides, ground water level 50% height of wall; Curve II: 10% of wall length in slides, ground water level 50% height of wall; Curve III: 10% of wall length in slides, ground water level below slope.

Recognizing that some slides should be expected for maximum economy, inspection and operating procedures must be adopted that will avoid endangering lives; these may consist of having scheduled observations of critical wall crests, of excavating all the initial loose rock, of scaling all the newly developed loose rock, and of giving some consideration to minimizing the loose rock that develops through modified blasting procedures.

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REFERENCES

[1] J. Venter and P. Stasse: Pressure and Movement in Soft Rocks, *International Strata Congress*, Leipzig, 1958.

[2] D. Taylor: *Fundamentals of Soil Mechanics*, John Wiley & Sons, New York, 1948.

[3] N. Janbu: Stability Analysis of Slopes with Dimensionless Parameters, *Harvard Soil Mechanics Series*, 1954, No. 46.

[4] J. Stubbins, R. Blais and I. Zajacki: Origin of the Soft Iron Ores of the Knob Lake Range, *Canadian Inst. of Mining and Metallurgy Bulletin*, 1961, vol. 61, p. 43.

[5] E. Abdun-Nur: How Good is Good Enough, *Journal of American Concrete Inst.*, 1962, vol. 59, p. 31.

[6] A. Jolliffe: Geology and Iron Ores of Steep Rock Lake, *Economic Geology*, 1955, vol. 50, No. 4.