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Slope stability at an opencast quartz mine

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SYNOPSIS

An investigation of the slope stability of a pit and failures of spoil dumps, carried out at Silicon Smelters quartz mine, Pietersburg, South Africa, is described. The major geological feature of the mine is the intensely foliated schist in the footwall, with exceptionally low strengths upon extensive discontinuities, and the similarly unusual strength properties of the dumped schist spoil. The selection of slope angles for the pit is described, and a proposal for the stabilization of the failing spoil dump by segregated dumping is explained.

SAMEVATTING

'n Ondersoek van groefhellingstabiliteit en skroothoopmislukkings wat by die Silicon Smelters kwartsietmyn te Pietersburg, Suid-Afrika, uitgevoer is, word beskryf. Die intensiewe bladvormige skis met buitengewone lae weerstandseienskappe en ekstensiewe diskontinuiteite in die vloersone en die soortgelyke buitengewone lae weerstandseienskappe van die skroothoopskis, is die hoof geologiese eienskap van die myn. Die hoekkeuse van groefhellings word beskryf en 'n aanbeveling vir die stabilisering van die skroothoopmislukkings deur afsonderlike storting word verduidelik.

Introduction

Silicon Smelters Mine at Pietersburg is an open-pit mine for the extraction of quartz. The quartz is in contact with a granite mass in the hangingwall and talcose schists in the footwall, within the body of the Witkop Hill, and spoil from the mine is cast upon the slopes of the hill. The site is shown in Fig. 1.

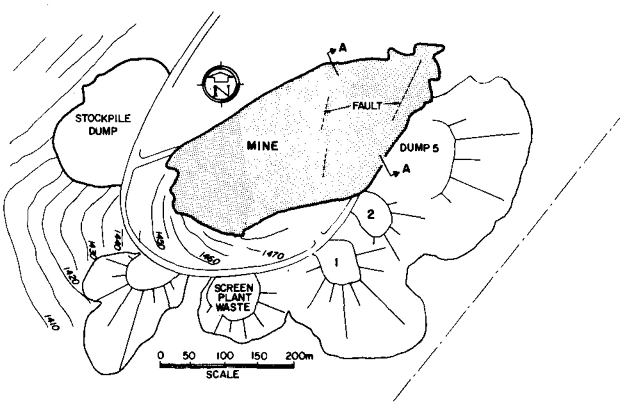

Fig. 1—Plan of the site

At the present stage of mining, a cut of about 25 m in height has been excavated in the schist, with benches of 6 m in height. The interbench slope angle is 70°, and the overall slope angle is approximately 25°. There have been numerous small slips, limited to individual benches, which have not interfered with mining. There is not yet a significant slope in the granite hangingwall. When the mine reaches its final depth, the height of the footwall and hangingwall slopes will be approximately 80 m and 50 m respectively.

The spoil dumps, consisting mainly of schist, rest upon the natural ground surface, which slopes at approximately 15°. The spoil is dumped by trucks on terraces at the dump crest, and the maximum height from toe to crest is 45 m. All the dumps are failing, although the movement is not continuous, but appears to be precipitated by rainfall and by the addition of spoil at the crest.

This paper describes an investigation of the properties of the granite and schist in the open pit, and of the failures in the spoil dumps. The investigation was carried out to provide information for use in the selection of slope angles for the pit and to devise a method for the stabilization of the spoil dumps. It is an unusual case study, in that, despite the low strength characteristics of the schist, which have resulted in failures of the bench and spoil dump, steeper slope angles were recommended than the preliminary design angles, with a substantial potential cost saving to the mine.

The proposed slope angles were based upon information gathered in the existing pit, and they will be reviewed and revised, if necessary, as the pit is deepened.

Geology

When ancient rocks of the Basement Complex were intruded by granite, they were altered to schists, and inliers of these altered rocks were left in the granite. It is probable that the Witkop quartz appeared at the end of the intrusive phase. The quartz body dips steeply to the north, and occurs at the contact between granite to the north and schist to the south. Fig. 2 gives a typical section[1].

The exposed granite is a friable, extremely soft rock, but an examination of the cores indicates that the consistency improves at between 10 and 20 m below the original ground surface.

The schist is a contorted, intensely foliated, talcose soft rock, with small areas of extremely soft rock. Its colour varies from brown and grey-green to shiny black. Many large polished movement surfaces are exposed in the pit. Most of the schist foliation and jointing is north-dipping (i.e. dipping into the pit) at dip angles that are generally steeper than 45°.

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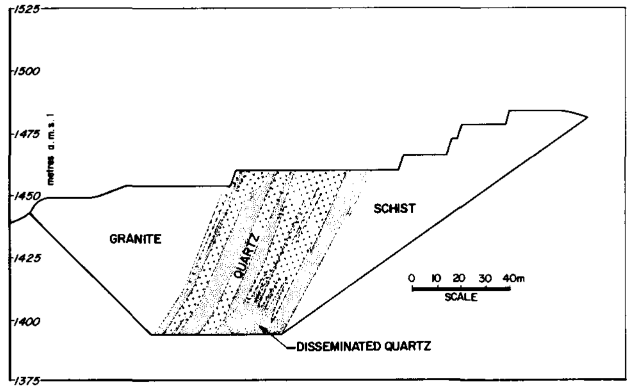

Fig. 2—Section A–A through the proposed open pit

Rock Joints

Joint Strength

Samples of intact granite from depths of 17 to 20 m in exploratory boreholes drilled for the ore-reserve assessment were tested in uniaxial compression. The average strength was 22 MPa. With no joints, such a material could be cut vertically to a considerable height. The average uniaxial compressive strength of the schist was lower than that of the granites (a mean value of 8 MPa), but still sufficient to allow high vertical cuts in unjointed rock.

It is therefore clear that, although the intact schist and granite would be classified as very weak to moderately weak rock[2], slope stability would be controlled by the joint properties of the rocks.

From shear tests upon natural and saw-cut joints in samples under various confining pressures, graphs were produced relating the shear strength of a joint to the depth of that joint beneath the surface of a slope.

Direct shear tests were carried out. The pieces of rock core containing a natural joint or a smooth saw-cut were set into resin in shaped moulds. After the resin had hardened, the two sections were placed in contact in the test apparatus. A chosen normal pressure was applied across the joint, and a gradually increasing shear stress was applied in the plane of the joint surface, by pushing against the upper section of core while restraining the lower section. The shear stress and shear strain were noted as the shear stress increased, and shear failure was considered to have occurred when there was a sudden increase in shear strain. The apparatus (Fig. 3) was developed by the Department of Mining Engineering at the University of the Witwatersrand. A series of tests was carried out on each sample at different normal pressures, and a graph of normal stress against shear strength was produced.

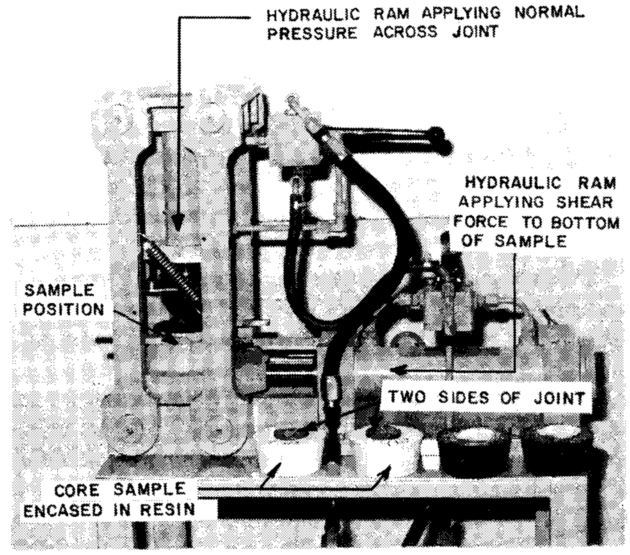

Fig. 3—Joint shear apparatus, University of the Witwatersrand

The results of tests upon natural joints and a saw-cut joint in the granite are shown in Fig. 4. Each set of points represents tests on one sample.

The waviness of the granite joints and the small sample size resulted in great variations in the laboratory results. Machine vibrations prevented the measurement of movements normal to the joint plane, and therefore the results were not corrected for dilation. It is also clear that, at high normal stresses, shear movement began to cause failure of asperities (e.g. the upper set of points in Fig. 4). The shear strength of a granite joint was calculated according to the empirical method of Barton and Choubey[3] from measurements of joint strength and roughness and an assumed base friction angle of 30°. The curved failure envelope thus produced is also included in Fig. 4.

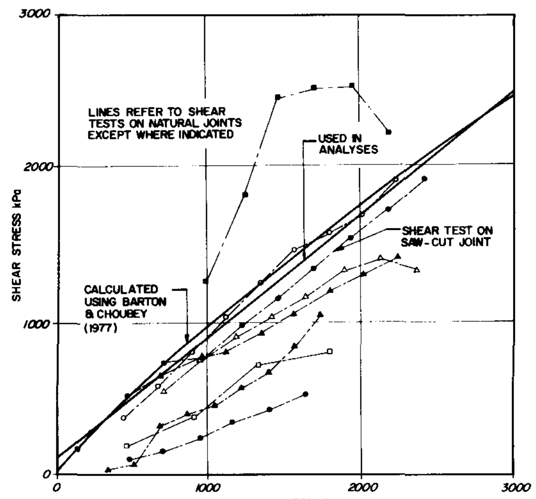

Fig. 4—Shear strength of rock joints in the granite

A linear-strength envelope was used for the stability analyses, with an effective cohesion of 100 kPa and an effective friction angle of 38 °. These values are reasonable when one considers the results of a test upon a saw-cut joint, the results of the calculation using Barton and Choubey[3], and values quoted in the literature.

The tests upon schist samples saw-cut along the foliation produced more uniform results, with a low mean base-friction angle of 18° reducing to 12° at high stresses (Fig. 5).

The change in gradient of the curves must be due to the breakdown of small asperities formed during the preparation of the samples. The failure envelope calculated from the results of Barton and Choubey[3] was for a chosen base-friction angle of 15°. This angle was selected from the

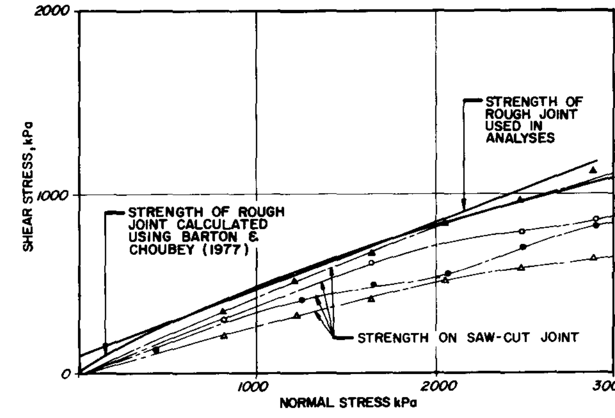

Fig. 5—Shear strength of discontinuities in the schist

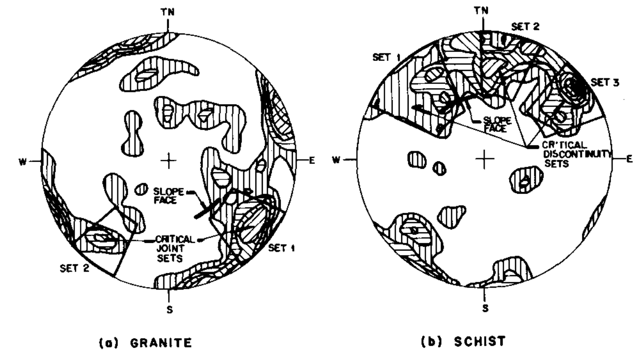

Fig. 6—Upper-hemisphere stereo-projection: Contours of discontinuity poles, showing major sets used in the stability analyses

results of the shear tests as being representative of the schist strength in the normal stress range of 0 to 1500 kPa relevant to the pit slopes.

The linear-strength envelope selected for the analyses, with an effective cohesion of 100 kPa and an effective angle of shearing resistance of 20°, is compatible with the Barton and Choubey curve derived directly from the shear-test results and measurements of joint roughness. The effective cohesion of 100 kPa is considered justified by the stepped failure surfaces observed where slips had occurred between benches. Failure along these surfaces had included tensile failure of intact rock bridges.

Joint Roughness, Alteration, and Gouge

The joints in the granite are generally rough and without gouge. Schmidt-hammer tests indicated that the joint strength was not affected by weathering.

The joints in the schist are also rough. Whereas weathering in the granite at depth was mainly confined to joint surfaces, the schist has decomposed *en masse*, and there are areas where this decomposition has resulted in low intact strength. No preferential weathering in the joints was noted, and the joints are usually free of gouge.

The granite has a dominant joint set dipping south-east at a near vertical inclination, and a cross-joint set dipping east, also at near vertical inclination. Two other steeply dipping sets occur that are particularly important when potential wedge failures are considered. Stereo-projections of the poles of joints in the exposed granite measured with a geological compass are shown in Fig. 6(a).

It was difficult to differentiate between jointing and foliation in the schist, and there was really no reason for them to be separated since they were similar in properties. The stereo-projection of measured poles given in Fig. 6(b) indicates that the major proportion of discontinuities dips at 45 to 90° towards the north into the pit.

Joint Length and Spacing

The measured joints in the granite exposures were generally less than 0,5 m long, but the inspection of old underground workings approximately 4 m below the pit bottom revealed that very large joints are present. The spacing between joints was generally from 0,2 to 1 m.

The schist faces examined were larger than the granite exposures, and many extensive joints could be seen that were continuous over the height of the face. Average joint lengths were about 4 m but, presumably, if the exposure had been higher, much larger continuous joints would have been revealed.

Discontinuities were closely spaced, from 0,02 m, and the impression was gained that an increase in shear stresses would easily create movement on foliation.

For the purpose of the analyses of slope stability, it was assumed that failures would take place along discontinuities, with no portion of the failure surface passing through intact rock, although, as mentioned above, a cohesion intercept of 100 kPa was used for these discontinuities. This assumption of failure on discontinuities was certainly the correct approach to the analysis of the stability of the schist footwall, and was justified for the granite hangingwall by the limited available data on joints.

The mean joint parameters used for the analysis of slope stability are summarized in Table I.

Dip directions, dip angles, and joint lengths are summarized graphically in Fig. 7.

Analyses of Slope Stability

Granite Hangingwall

The joint strengths of the granite are relatively high, and

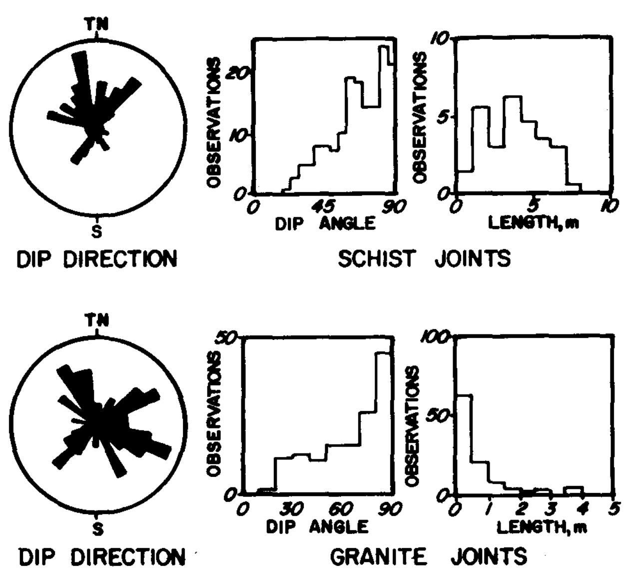

Fig. 7—Joint orientation and lengths

TABLE I

MEAN VALUES OF JOINT PARAMETERS USED IN THE STABILITY ANALYSES

Rock type	Joint set	Apparent cohesion kPa	Tangential friction angle degree	Pore-pressure parameters	Dip direction degree	Dip angle degree
Granite	1	100	38	0,1	128	60
	2	100	38	0,1	220	60
Schist	1	100	20	0,1	316	65
	3	100	20	0,1	053	70

the joint inclinations are generally favourable for a stable slope. However, as shown in Fig. 6(a), two major joint sets were identified that could create sliding wedges.

Combinations of the joint sets were analysed by a computer program that selects joint dip, dip direction, and strength from statistical distributions of these parameters, and evaluates the potential instability of the chosen wedge. The probability of failure of a slope is determined from the analysis of a large number of potential wedge failures[4]. The relationship of slope angle to probability of failure is shown in Fig. 8. The probability of failure of the granite hangingwall increases swiftly as the slope is steepened above 55°.

Schist Footwall

The analysis of the schist footwall was more problematical than that of the granite. Jointing and foliation were generally north-trending and extensive, with the result that it was questionable whether failures would form wedges on two or three planes or whether they would occur as simple two-dimensional slips.

Two-dimensional stability analyses and three-dimensional wedge analyses were carried out by the use of computer programs selecting strength and joint parameters from statistical distributions. In this way, after a large number of potential failures had been analysed, the probability of failure of a slope excavated at a particular angle was derived.

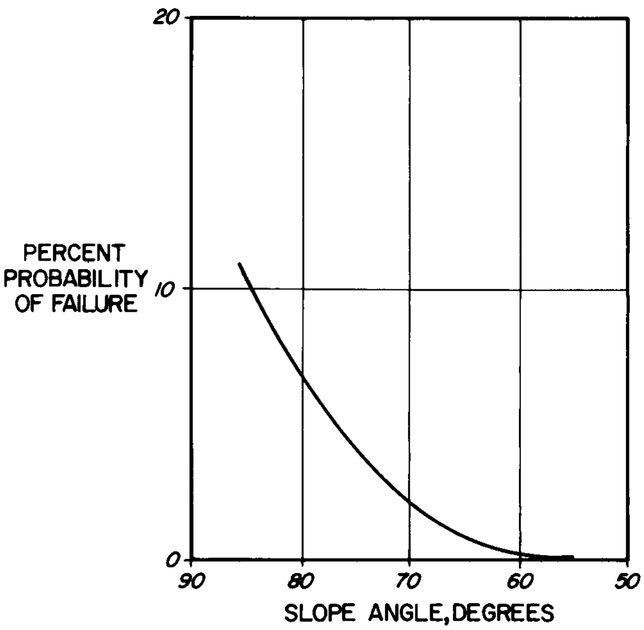

Fig. 8—Effect of slope angle of granite hangingwall upon probability of a two-plane wedge failure

The two-dimensional analysis was carried out by use of the computer program STABL[5], which makes use of the Simplified Method of Janbu[6]. For each selection of rock parameters, the program chooses a number of non-circular failure surfaces, and the factor of safety for each surface is calculated. By restricting the choice of surface to a particular critical region, a distribution of factors of safety for the various selections of rock parameter is obtained, and the probability of failure is derived from this distribution. The rock was given anisotropic strength parameters to account for the effects of jointing and foliation (Table II).

TABLE II

ANISOTROPIC STRENGTH PARAMETERS USED IN TWO-DIMENSIONAL ANALYSES OF THE SCHIST FOOTWALL

Range of failure surface inclination	Effective angle of shearing resistance degree		Effective cohesion kPa	
degree	Mean	Standard deviation	Mean	Standard deviation
45 to 90	20	4	100	50
0 to 45	30	4	375	150

The cohesive strength of 375 kPa used for failure plane inclinations of more than 45° to the horizontal was intended to represent intact strength. It was calculated from a lower bound uniaxial compressive strength. (An intact sample of extremely soft rock schist had been tested in a shear box, producing an effective cohesion of 160 kPa and an effective angle of shearing resistance of 30°.)

Fig. 9 shows the measured dip angles of discontinuities on the schist. It will be noted that only 7 per cent of the discontinuities that might influence two-dimensional stability had a dip angle of less than 45°. Typical non-circular potential failure surfaces are shown in Fig. 10, for a postulated slope angle of 55°.

Potential wedge failures on intersecting joint planes were analysed by use of the program WEDGEPROB. There was only a small probability that a geometrically possible wedge would form when the slope angle was less than 50°. Fig. 11 shows the relationship of probability of failure to slope angle in the schist for both methods of analysis. It is

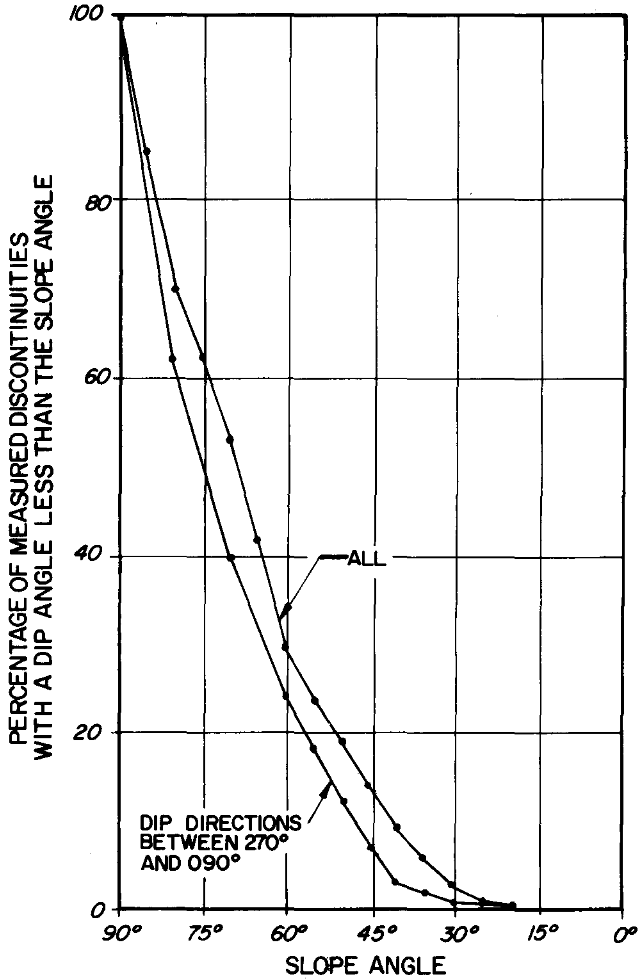

Fig. 9—Dip angle of discontinuities in the schist related to slope angles

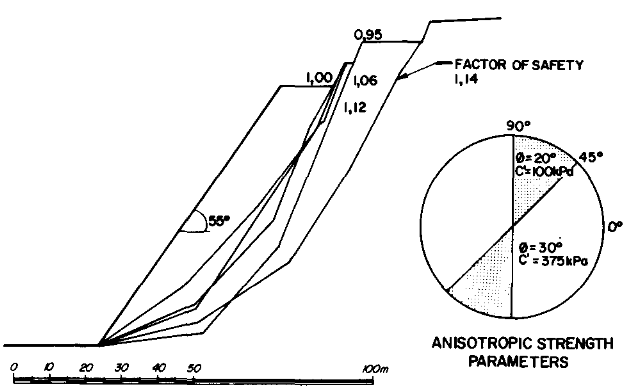

Fig. 10—Potential two-dimensional failures in the schist wall

accepted that these curves have no absolute significance but are design aids for the engineer.

Selection of Slope Angles

Slope angles were selected after careful consideration of the jointing, the results of stability analyses, and the planned pit geometry. It was established that mine bench failures, particularly in the schist, would be unavoidable but of little significance to the mining operation. However, because the pit is small, failures extending over more than one bench are considered unacceptable, and the slope angles were selected accordingly.

It was decided that the upper 10 to 20 m of very weathered, extremely soft-rock granite should be excavated at an overall slope angle of 45°, based on the use of simple stability charts. An inter-ramp angle of 55° was chosen for the deeper weathered, soft-rock granite, giving a probability of failure of 0,2 per cent (Fig. 8). The bench height is 6 m, the inter-bench angle is 70°, and the bench width is 2 m.

The calculated probability of failure of 0,2 per cent is low, but is justified by the preliminary nature of the investigation, the insignificant exposure of granite that was available for measurements, and the immediate disruption of operations that would be caused by a closure of the haulage road.

An inter-ramp angle of 45° was chosen for the schist footwall, giving a probability of failure of 0,1 and 2 per cent for the two methods of stability analysis (Fig. 11). The justification for this selection is similar to that used for the angle of the granite slopes, but was reinforced by observations of the failures that have occurred on individual beaches.

The slope stability of the pit will be re-assessed as mining progresses, and it is anticipated that the analyses of joint measurements and monitoring could result in the steepening of slope angles by up to five degrees.

Cost Implications

The slope angles proposed in the initial mine plan were 45° and 35° in the granite and schist respectively, although it was appreciated that these values might be altered after a geotechnical investigation. Thus, the investigation of pit slope stability has resulted in a 10-degree steepening of the pit slopes. The reduction of spoil volume represents a change in the stripping ratio of spoil to ore from 3,2 to 2,2

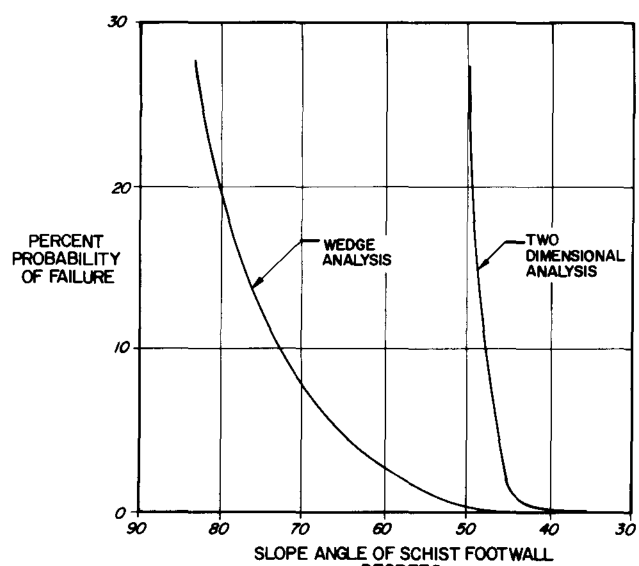

Fig. 11—Variation in the probability of failure of the schist footwall with its slope angle

with a total excavated waste volume of approximately 1 million cubic metres, compared with the initial 1,5 million cubic metres.

Slope Failures at Waste Dumps

Dump failures are nearly always a safety hazard, and they also reduce the total capacity of a dumping area. There were signs of movement at the crest and toe of all the dumps containing schist spoil, and the toe had moved to within 20 m of the boundary of the mine property at one point (Fig. 12). No seepage was observed.

Fig. 13 shows a view of the upper failure surface at section B–B, Dump 5. The underlying material consists of quartz pebbles and boulders in a matrix of silty sand, which is a relatively competent foundation. It was clear from this, and from the appearance of the slips, that failure was occurring through the spoil itself.

When dumped, a granular material without cohesion, such as the schist spoil, forms a slope at its natural angle of repose, and failures can usually be precipitated only by an increase in water pressures within the dump or a decrease of shear strength. Although no seepage was noted, and it is unlikely that there was a water table within the dumps, it is possible that periodic ingress of rainwater destroys an apparent cohesive strength resulting from surface-tension forces between particles. In addition, it is likely that movements within the loose schist spoil will tend to orientate the platelike particles parallel to the direction of movement, reducing the strength to residual values. Finally, a curvature of the shear envelope implies that the resisting force increases at a slower rate than the disturbing force as the dump increases in height.

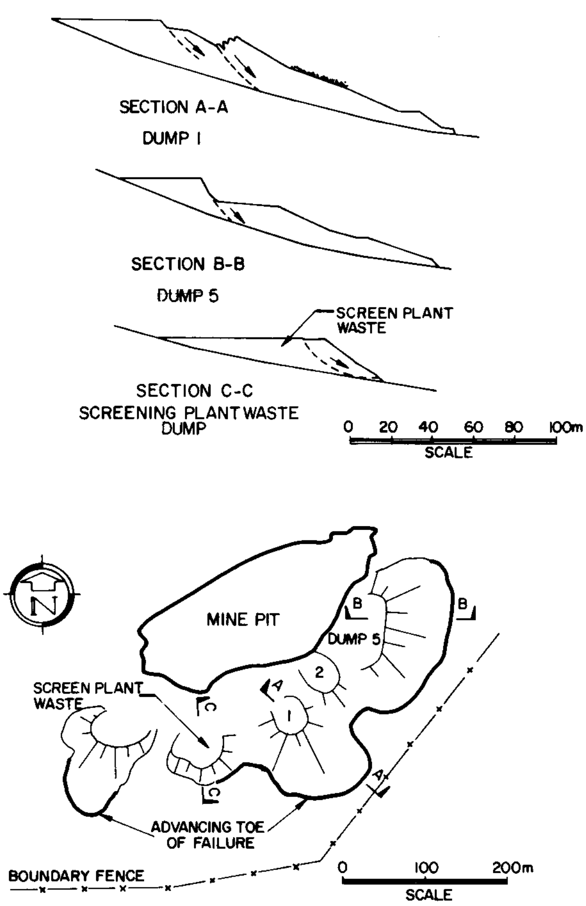

Fig. 12—Sections through failing waste dumps, and plan showing areas affected by failures

Fig. 13—Slickensided failure surface at Dump 5, indicating a 10 m downward slide of the material in the foreground

It is therefore postulated that dump failure results from the following sequence of events.

(a) The dump is able to build up at a slope angle steeper than the mean angle of friction because of apparent cohesion forces due to water surface tension between particles, and because of the curvature of the shear envelope. (An example of this is the slope angle above the failed upper portion of Dump 5, Fig. 13.)
(b) Strains within the dump cause re-orientation of schist platelets, reducing the shear strength on potential failure planes.
(c) The ingress of rainwater removes the restraining surface-tension forces between particles, and may actually cause the temporary development of positive pore-water pressures.
(d) The forces resisting failure are thus reduced to less than the disturbing forces, and failure occurs. The 'run-out angle' (the angle of repose attained after failure) would be less than the initial stable angle of repose, and appears to be 20 to 22°, which is approximately equal to the residual friction angle.

Strength of Schist Spoil

The sample of schist spoil was a gravel in a matrix of silty sand, although, of course, the dumps contain a large proportion of boulders. However, the matrix material is present in sufficient quantity to control the behaviour of the material as a whole. This matrix material was tested in an apparatus consisting of a laboratory ring shear box, which can apply very large strains, to provide information on the reduction of strength with dump movement. It would be expected that, as the spoil is talcose and has platelike particles, the strength will reduce as the particles re-orientate

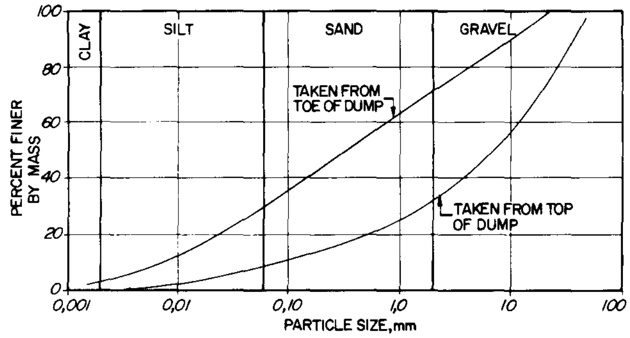

Fig. 14—Gradings of schist samples taken from the spoil dumps

themselves parallel to the direction of movement. The fraction passing a 2 mm sieve size was used.

Fig. 14 shows the particle-size distribution of the sample of schist spoil. The strength envelopes and stress–strain curves are shown in Figs. 15 and 16.

The spoil gave relatively low strengths (a silty sand would be expected to have an effective angle of friction of more than 30°). A linear least-squares analysis of the data gave the following strength parameters:

	Effective angle of friction degree	Effective cohesion kPa
Peak	25	11
Residual	23	8.

However, a better fit was obtained with power curves, which gave correlation coefficients of 0,997 and 0,999 for peak and residual strength respectively. The data are shown in Fig. 15. For the purpose of the analyses, a linear-strength envelope was used, with no cohesion and an effective friction angle of 22°. An effective cohesion was not

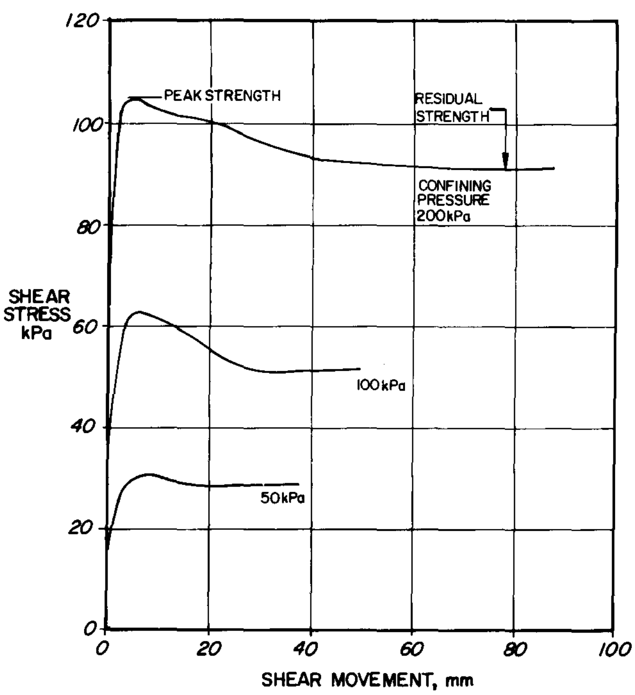

Fig. 15—Stress–strain curves for schist spoil

considered justified for a material that was essentially cohesionless. An effective residual friction angle of 22° was close to the laboratory value, and agreed with the results of a back-analysis of the spoil-dump failure.

The particle sizes indicated that the spoil would not be completely free-draining. This implies that instability could be caused by the ingress of rainwater. No granite spoil was available for testing, but it was considered to have good strength and drainage characteristics.

Stability Analyses and Remedial Measures

The stability of the spoil dumps at Section A–A (Fig. 12) was analysed by the Simplified Method of Janbu in the computer program STABL. With residual-strength

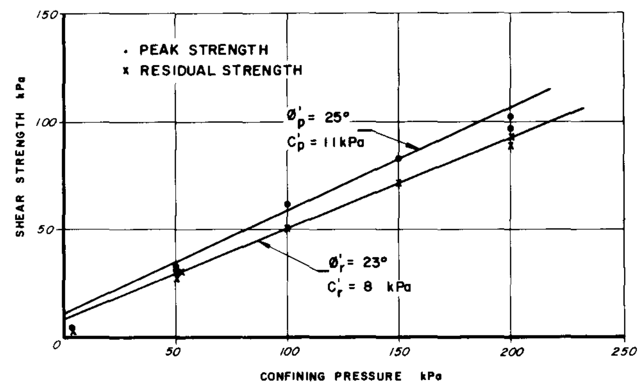

Fig. 16—Shear-strength envelopes for schist spoil

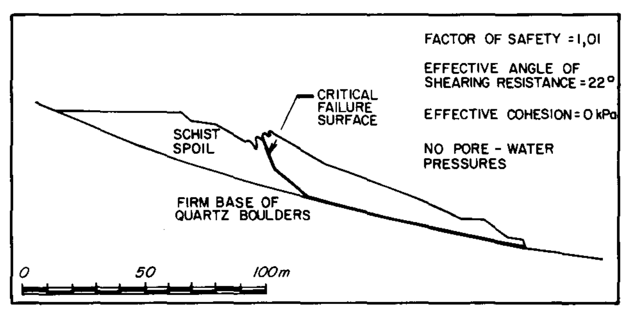

Fig. 17—Results of two-dimensional slope-stability analyses of Dump 1 using the simplified method of Janbu in the computer program STABL

parameters of $\emptyset' = 22°$ and $c' = 0$ kPa, and no water pressures, it was confirmed that the factor of safety against failure was 1, implying that a small increase in water pressures would re-activate the failure (Fig. 17). It seemed probable, therefore, that intermittent movements of the dumps would continue, being precipitated by periods of rain, and a suitable method of halting these movements was sought.

Consideration was given to the use of a segregated dumping system, the more competent granite spoil being placed in a buttress to restrain the schist spoil. Stability analyses showed that there would be an adequate proportion of granite to prevent failure of the dump when the water pressure rose to a value equivalent to a pore-pressure parameter, r_u, of 0,1 (approximately 1 to 2 m of water pressure). The criterion for the dimensions of the buttress was that it should be sufficiently high to prevent failures passing over it. It was assumed that freshly dumped schist would stand at an angle of repose of 26°, but that it would have a stable slope angle after failure of 22° (Fig. 18).

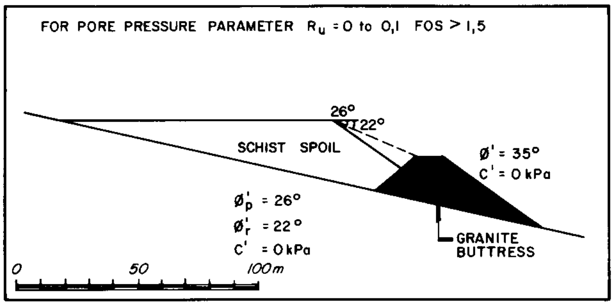

Fig. 18—Proposed segregated dump

Where the dump had already moved to within 20 m of the boundary fence, the width of the buttress would be restricted, but would nevertheless provide sufficient resisting force to prevent movement of the existing dump when r_u increased to 0,1. The factor of safety of the dump was calculated to be 1,15 (Fig. 19) on the assumption that no additional spoil would be dumped in that zone.

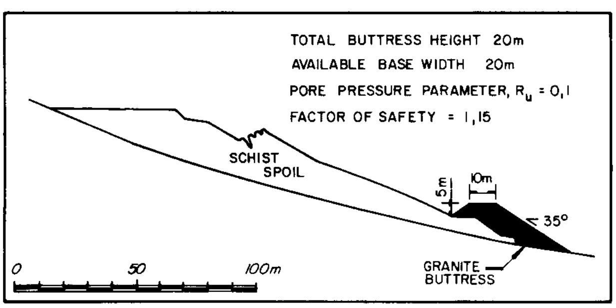

Fig. 19—Section through Dump 1, showing required geometry for granite buttress

Conclusions

The conclusions that were drawn from the geotechnical investigation conducted for the initial design of open-pit slopes at Silicon Smelters quartz mine were as follows.

(1) The pit slopes were steepened by 10° to an inter-ramp angle of 55° in the granite hangingwall and 45° in the schist footwall. This has reduced the stripping ratio from 3,2 to 2,2.

(2) The talcose schist rock in the footwall was found in laboratory tests to have a mean basic friction angle of 18° at low normal stresses and 12° at high normal stresses.

(3) The schist spoil produced a curved failure envelope in laboratory tests. Linear approximations gave peak and residual friction angles of 25 and 23° respectively. The peak and residual apparent cohesion (a result of curvature of the envelopes) was 11 and 8 kPa respectively.

(4) Extensive failures in the spoil dumps were considered to be caused by a combination of the effects of temporary increases in water pressures, the application of additional spoil at the crest (related to curvature of the failure envelope), and the reduction in strength of the platelike particles of schist to residual values.

(5) A system was devised for segregated dumping of spoil, the granite spoil being placed as a buttress to retain the schist spoil.

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