OPEN PIT ROCK MECHANICS

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INTRODUCTION

Historically, the application of rock mechanics to surface mining has primarily been the analysis of the stability of slopes and was considered more theoretical than applied. In the first edition of *Surface Mining* (Pfleider, 1968), slope stability was included in the section on research and development. In the intervening years, slope design has become an integral part of mine planning rather than a research curiosity. This has come about with the development of improved data collection techniques (Call, et al., 1976), computer-aided stability analysis (Ross-Brown, 1979), and a reliability cost-benefit interfacing with mine planning (Kim, 1977). Since 1968, two comprehensive reference works on slope design have been published (Hoek and Bray, 1981; CANMET, 1976).

In addition to slope design, there are a number of other applications of rock mechanics to surface mining that utilize a common database. Table 1 lists the applications and the database. For purposes of this section, soil mechanics and rock mechanics are not differentiated. The underlying mechanics of stress strain and strength are the same, regardless of the classification of the material, as are the basic principles of determining the physical properties of a material and analyzing its response to imposed stress. Therefore, when reference is made to *rock*, it may be geologically a soil.

It has been the experience of the authors, both as mine staff geotechnical engineers and as consultants, that ground control difficulties more often have been the result of inadequacy in organization and implementation of a rock mechanics program than the lack of, or failure of, the technology to solve the problem. Therefore, in this section, the authors have placed the emphasis on what to do, when to do it, and how to integrate rock mechanics with other aspects of mining rather than on the theoretical aspects of mathematical analysis.

An effective rock mechanics program requires careful organization and planning. Data collection must precede analysis so that problems are anticipated; otherwise, there may be insufficient time to collect data and important information may be lost. For example, displacement and water level time trends cannot be measured retroactively, bench faces may become covered or inaccessible, and drill core ground up for assay. On the other hand, there is never the time, manpower, or budget to measure everything; so the analytical approach must be kept in mind during data collection to ensure the appropriate information is collected with the resources available.

Interpretation and analysis should be kept current with data collection; otherwise, data collection can become an end in itself. File cabinets full of raw data may give the appearance of productivity but do not, in and of themselves, result in an optimum mine plan.

In the subsequent section, the emphasis is on data collection first to provide the background for analysis and designs. Geologic data collection and presentation are discussed in more detail than other aspects of rock mechanics because of their importance and the number of times we have found the geologic database inadequate.

Design Approach

Mine design and operational decisions are primarily costbenefit optimizations. The objective is to extract the mineral

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Application	Struc- tural Do- mains	Major Struc- ture	Fabric	Sub- stance	Frac- ture	Hydrol- ogy	Stress Field	Seis- micity	Mining Plans	Equip- ment	Produc- tion Rates and Costs	Current Slope Geome- try and Dis- place- ment
Slope design	1	1	1	1	1	1	2	1	1	3	2	2
nope manage- ment Diggability Blasting Trafficability	1 1 1	1 2 2	1 1	1 1 1	1 2	1 2 2 1	3 2		1 2	2 1 1 1	2 2 1	1
ity	1			1	1	1				1		
Crushing and grinding Leaching	1 1	2	1 1	1 2	2	1			2	1		

Table 1. Rock Mechanics Applications and Data Requirements for Surface Mining

reserve at the lowest cost, or to accept or reject a mining option on the basis of benefits being greater or less than the cost. In this context, the role of rock mechanics is to predict the behavior of the rock in response to mining in a manner that costs and benefits can be assigned.

The prediction of rock behavior is by no means straightforward. To make a rational analysis, a conceptual model must be developed that is mathematically tractable and cost effective. The complexity of natural materials and processes precludes an exact modeling; thus, an analysis is only an approximation of the real world.

Even with the simplification of modeling, the present analytical capability exceeds the ability to obtain the prerequisite input data on material properties, geology, and site conditions for the following reasons:

1) Material properties vary from point to point and access is limited, so obtaining representative samples is difficult.

2) There are uncertainties in field measurements and testing.

3) The magnitude and time of occurrence of the phenomena that affect rock behavior, such as rainstorms or earthquakes, are governed by such a complex interrelationship of factors that it approaches a chance event.

This uncertainty in the input data and analysis precludes an exact prediction of rock behavior. For this reason, the probabilistic approach has gained widespread acceptance in open pit rock mechanics. By appropriate sampling and testing strategies, the distribution of rock properties can be quantitatively estimated. These distributions can be used in explicit mathematical models or in Monte Carlo simulations so that the results of an analysis can be computed as a probability distribution rather than a deterministic single value based on an average or assumed single input value. For example, the stability of a slope can be expressed as the probability of failure, which is more useful for an economic risk analysis than a safety factor.

GEOLOGY

Applied geology emphasizes an overall view of general geology of the mine, which includes the distribution of rock units and the spatial relationships of major structures. For specific rock mechanics studies, additional information on major structure and rock fabric is needed. Rock fabric is the orientations and characteristics of minor structure, such as joint sets and foliation.

Major structures are treated as individuals in slope design because their location and extent are known. Bench face mapping and geologic interpretation are used to define major structure. Rock fabric, however, is treated statistically in design because a single orientation measurement and property determination on a joint may or may not represent those of the joint set. The statistics of populations where orientations and characteristics are described, not by a single value but by a distribution of values, are needed because of the recognized variability in joint properties within the joint set. Cell mapping, set mapping, and detail line mapping are used to obtain data for statistical analyses.

Bench Face Mapping

The bench face mapping technique is described by Peters (1978). This is the basic method for pit mapping whereby major structures, such as faults and contacts (traceable for at least two benches), are described by the points where they intersect the toe and crest of the bench. True strike directions are seen only where structures cross bench levels. A map showing several bench levels should be carried in the mapping

to tie geology through to successive benches (Fig. 1). In addition to plotting the major structure, comments should also be made on rock type, alteration, mineralization, rock hardness, and fracture frequency.

An appropriate mapping scale is the key to successful pit mapping. Mapping should provide sufficient information for interpretation, and it must be provided in time to be useful. Thus, a mapping scale should be selected that will allow mapping to keep pace with the mining. Extensive detail in one small area is less important than providing general geologic knowledge for the entire mine. Ideally, mapping is kept current within 100 m (400 ft) of the mining advance. The following guidelines can be used for selecting an appropriate mapping scale:

Pit Diameter	Mapping Scale
<500 m (<1500 ft)	1:400 (1:600)
500-1500m (1500-5000 ft)	1:1000 (1:1200)
>1500 m (>5000 ft)	1:2000 (1:2400)

Data from field sheets should be posted onto mylars the same day the mapping is done. These mylars should contain only factual information, not interpretation.

Geology Interpretation and Presentation

Interpretations of geology should be made on section mylar or sepia copies from the factual mylars. Keeping factual information separate from the interpretations makes reinterpretation at a later date possible with a minimum of effort.

The overall geologic picture is provided by a complete set of interpretive geology level maps and cross sections at a scale that shows the complete view of the mine plan. Generally, these maps are at the same scale as that used for mine planning. If geologic information is merely collected and not interpreted, the geology will not be used effectively in ore reserve estimation, ore control, mine planning, or slope design. Mine geologists must extend interpretations, based on intuition if necessary, for a distance of one and a half pit depths beyond the final pit limit. Otherwise, mine planners and engineers must make the necessary geologic assumptions usually based on less knowledge than the geologists have. One assumption that can be made in error is that no useful geologic information is available.

From the basic level maps and cross sections, geology is easily transferred to plans and the current pit composite. Geology maps are made for every planned mining level. Cross sections are made to show views of geology in the vertical plane on intervals determined by the ability to project geologic information. Overlays are generally made on selected maps to show alteration, mineralization, grade, or other features of interest.

Level maps and cross sections contain the following basic information: (1) pre-mine topography, (2) current topography, (3) original surface geology, (4) drill holes that show geology and grade, (5) old underground workings, and (6) mining push backs and final pit limit. The process of developing geology plan maps is illustrated in Fig. 2. Cross-section maps are developed in a similar manner.

Rock Fabric Data Collection

To develop an understanding of minor geologic features, such as joint sets or foliation, mapping must provide a systematic and consistent measurement. This implies the use of coding forms suitable for data entry and compatible with computer processing (Fig. 3). Coding provides the desirable aspect of a checklist to ensure that all needed information



Fig. 1. Bench face mapping field sheet.

is recorded. Minor geologic features are analyzed statistically, which requires that an unbiased sample of the true distribution of values be obtained. All numbers should be recorded exactly, and not rounded, to provide continuous rather than discrete information.

There are three methods for obtaining rock fabric data: (1) cell mapping (Call, et al., 1976); (2) set mapping (Call, et al., 1976); and (3) detail line mapping (Piteau, 1970; Call, et al., 1976). Each method has a specific purpose, and each has advantages and disadvantages.

Cell Mapping: This method is used when there are large, extensive exposures of rock, such as along benches in an open pit or on large natural outcrops. Consecutive mapping cells are established along the strike of the exposure. Orientations are characteristics of the most significant structures are recorded. This mapping provides a continuous measurement that gives an estimate of the recurrence of structure orientations and, thus, a probability of occurrence. Experience has shown 30 to 40 cells are needed in each structural domain to describe the statistical distributions needed in slope design.

Cell mapping is subjective because it relies on the observer's judgment as to which fracture sets are significant; therefore, an observer bias is built into the data. The advantages to this method are that fracture characteristics and their frequency of occurrence can be determined over large areas in relatively short time.

Set Mapping: This method is used in place of cell mapping when rock exposures are not suitable for establishing consecutive cells, or for reconnaissance-type mapping. It provides information on fracture orientations and characteristics, but not quantitative information on recurrence over a large area. It is a fast mapping method, but like cell mapping, the observer makes a judgment as to the significance of each fracture set, thus introducing an observer bias to the data.

Detail Line Mapping: This method has the least observer bias of the three methods for obtaining statistical information on the fracturing, but it also provides the least area coverage. Generally, this method is used to determine rock fabric when there is no previous knowledge of the fracture patterns or distribution forms of the fracture characteristics. This method is a spot sampling technique where sampling sites are chosen along the bench face or other available rock exposure.

At each mapping site, a measuring tape, which serves as a reference line, is stretched along the rock exposure. The orientation and characteristics of every fracture or its projection, which intersects the line, are recorded. Usually, an arbitrary length cutoff of about 0.3 m (1 ft) is used and fractures with lengths less than the cutoff are not recorded.

Major disadvantages to this method are that it is tedious and time-consuming, and only a small area is covered. Unless supplemental cell mapping or set mapping is done, significant fracture sets can be completely missed.

Rock Fabric Data Processing

Data from the coding forms are entered into the computer from which Schmidt plots (lower hemisphere, equal area projections) are produced (Billings, 1954; Hoek and Bray, 1970). These plots are used in determining structural domains and potential failure modes (Fig. 4).

Conventional methods for developing histograms and determining distribution forms are used to describe the fracture orientations and characteristics in preparation for stability analysis and slope design.

Structural Domains

A structural domain is an area usually bounded by a major structure, such as faults or contacts, where orientation patterns of the fractures and their characteristics can be considered similar. An example of structural domain definition is given in Fig. 4, and more discussions can be found in Chap. 2 of the CANMET Pit Slope Manual (1976).

Oriented Core

Oriented core is often used to supplement surface mapping data and to collect samples of natural fractures in areas behind proposed pit walls and at depth. Orientation of the entire core from the drill hole is not necessary for fracture studies. All that is needed is a statistical sampling of the fracture attitudes. For this reason, some of the simpler and less expensive core orientation methods, such as the clay imprinter are recommended (Call, et al., 1982). More sophisticated methods of core orientation are available from Christensen in Salt Lake City.

ROCK MECHANICS PROPERTIES AND ROCK MASS STRENGTH

It is important to differentiate between the rock substance which is intact rock, and the rock mass, which includes both intact rock and rock fabric.

Strength refers to the "maximum stress that a body can withstand without failing by rupture or continuous deformation." Application in analysis and design determines the loading conditions which define the strength value of interest.

In open pit slope design, compressive strength of the substance is important as a classification criteria. Also, block flow stability analysis, where crushing of the rock is consid-



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DATA SHEET FOR CELL MAPPING

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Fig. 3. Rock fabric data sheets.

ered, uses the mean and standard deviation of compressive strength as input parameters (Coates, 1981).

In crushing, drill performance, and excavation studies, compressive strength becomes important because of the relationships that might be developed between strength and energy requirements, abrasiveness, drilling rate, and digging or ripping characteristics. When potential failure planes are not continuous, intact rock bridges must be broken for failure to occur and the rock substance tensile and shear strength are required.

In rock mechanics the strength of the rock mass will determine its behavior under stress. However, rock mass strength cannot be obtained by laboratory testing or direct measurement; it must be inferred from the measurable components of the rock mass: rock substance strength, fracture strength, rock fabric, major structure, and block size. Determining rock fabric and major structure characteristics were discussed previously. The other parameters can be determined from laboratory tests, field methods, and estimation methods (Table 2).

Laboratory Tests

Uniaxial and triaxial compression tests are conducted in the laboratory on cylindrical samples, usually of drill core, to provide rock substance strength. Deformational properties, Poisson's ratio, and modulus of deformation are also determined from the uniaxial test.

Poisson's Ratio (μ) is the ratio of lateral strain (ϵ_{iat}) to longitudinal strain (ϵ_{iong}) under normal uniaxial stress (σ_n) and is a measure of the directional variability in the deformability of the rock substance.

Modulus of Deformation (E) is a measure of the stiffness of the rock substance under normal uniaxial stress. It is the ratio of uniaxial stress to longitudinal strain.

Tensile Strength

Of the two most common tests for determining tensile strength, indirect tension (Brazilian) and direct tension, the Brazilian test is the least expensive, easiest, and most commonly used.

A Brazilian test consists of diametrically loading a disk of rock core until it fails. Theoretically, the diametrical loading induces a tensile stress in the center of the disk, and failure occurs parallel to the direction of loading.

The direct tension test pulls a cylindrical rock sample at both ends until the specimen fails.

Direct Shear

The direct shear test consists of taking two blocks of rock that are separated by a natural fracture, applying a load perpendicular to the fracture, then measuring the shear load required to displace the blocks relative to each other. Samples of fault gouge or low strength rock, where shear failure of the rock substance is expected, are tested in a similar manner by shearing through a single block or core of the intact

Fig. 4. Structural domains and rock fabric plots.



Rock Mass Component	Laboratory Test	Field Method	Estimation Method
Substance compressive strength	Uniaxial, Triaxial	point load test, Schmidt rebound hammer	manual index tests
Substance tensile strength	Indirect (Brazilian), Direct	none	tensile strength = uniaxial com- pressive strength \div 10
Substance shear strength	Direct	in-situ direct, vane	shear strength = uniaxial com- pressive strength ÷ 2
Fracture shear strength	Direct	in-situ direct, tilt	Barton's classfication, back analy- sis of failure, geometries
Rock fabric	N/A	mapping	none
Major structure	N/A	mapping	none
Block size	N/A	RQD, core logging, screen tests, inspection of muck piles and dumps, volumetric joint count, block size index, photoanalysis	simulation of fracture geometry

Table 2. Methods of Determining Characteristics of the Components of the Rock Mass

material. Although the shear strength of intact rock can be obtained from triaxial tests, the direct shear test is preferred by the authors since it more closely simulates the stress conditions used in slope stability analysis.

The resulting relationship between normal stress and shear stress can be analyzed statistically, and a mathematical linear or power *best* fit can be calculated:

- $\tau = C + \sigma_n \tan \phi \text{ (linear)}$
- $\tau = k \sigma_n^m \text{(power)}$

where τ is the shear strength, C is cohesion, σ_n is the normal stress, ϕ is the friction angle, and k,m are power curve parameters relating shear strength and normal stress.

Commonly, the linear approximation is used because the values of friction angle and cohesion are easy to relate to field problems and it has been an accepted method for many years. However, the power fit is considered more representative of the shear strength along fracture surfaces. At low normal loads, common to slope stability problems, the upper fracture surface tends to ride up and over irregularities on the lower surface. As the normal load is increased, shearing of the irregularities occurs. The resulting power relationship shows a steep curve at low normals that tends to approach zero at zero normal load, instead of a mathematically determined cohesion intercept as in the linear model. The linear intercept leads to overestimation of the available shear strength at low normals. For a limited range of normals, the linear fit is a reasonable estimator of the shear strength. Because of the potential nonlinearity of the shear strength curve, it is important to conduct shear tests in the anticipated range of normals.

Table 3 lists some typical strength and deformation properties of rock substance for some common mine rocks. Table 4 lists some typical fracture strength properties. Values from these tables might be used for preliminary analysis until results are available from site-specific testing.

Field Methods

The Schmidt hammer is a tool originally developed for testing concrete. The hammer is essentially a spring-loaded piston. The cocked piston is placed against the rock to be tested and triggered. The height of the rebound of the piston is measured, which is a measure of the rock hardness. A correction is made to standardize results based on the orientation of the hammer during testing. Schmidt hardness is related to the compressive strength of the rock substance (Brown, 1981). The test is unreliable and considerable variablity in strength estimates occurs. Several measurements should be taken at each site.

Point load tests can be done on either rock core or irregularly shaped specimens. The test equipment is a portable machine similar to a core splitter, consisting of a loading apparatus and an additional system to measure load and distance between the loading plates. A point-load strength index is calculated from the failure load and the sample dimensions. Corrected results from this testing often correlate with uniaxial compressive strength of rock substance. It is a simple, reliable, and inexpensive means to measure substance strength and the results are useful for rock classification purposes. Procedures for conducting these tests have been developed by the International Society for Rock Mechanics (Brown, 1981).

In situ direct shear tests can be used to test shear strength of low strength soil-like rocks and fracture surfaces in hard rock. The advantage to this test is reportedly that in situ site conditions, which can influence shear strength, are included in the testing. However, the sample preparation at the site, in fact, changes the actual conditions. The main disadvantage is that the test is time-consuming and can be expensive. In the majority of cases, it is better to collect a number of samples and test them in the laboratory than to spend the same amount of money on one in situ test. The objective should be to describe the variability in rock mass strength and, although less precision may introduce some variability, it is better to have a statistical sampling instead of one precise point.

Vane shear tests can be conducted in soil-like materials, and there are ASTM standards for conducting these tests (Sowers and Sowers, 1970). The vanes are essentially two crossed blades on a rod. The vanes are forced into the soillike substance and then rotated. The torque required to shear the soil-like substance is measured. Shear strength is correlated to the size of the vane and the torque.

Tilt tests to determine shear strengths of fractures or rock fill in the field have been suggested by Barton (1982). He proposes corrections to extrapolate the information for use in design. This test is simple and inexpensive and, in the case of fractures, simply involves a measure of the size of the blocks tested and measuring the tilt angle at which one block

MINE OPERATIONS

Table	3.	Rock	Substance	Properties
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	Uniaxi St	al Compr rength, ps	essive Si*	Brazilia Str	n Disk 1 ength, p	^r ensile si				Cot	nesion, p	osi	Density, pfc
Material	Typical	High	Low	Typical	High	Low	Typical	High	Low	Typical	High	Low	Typical
Igneous Intrusive Fresh granite Altered granite	30,000	50,000	20,000	2000	5000	1000	55	65	45	3000	4500	1500	167
per) Quartz monzonite Quartz diorite Diabase	12,000 14,000 27,500 40,000	20,000 20,000 40,000 57,000	6,000 8,000 16,500 20,000	1200 3150 3000	2000 4750 5200	700 1550 800	48 60	60	36	3200	4800	1600	165 164 175 185
Igneous Extrusive													
Rhyolite Dacite Andesite Basalt Welded tuff	21,500 19,000 24,000 20,000 10,000	31,500 39,000 50,000 18,000	6,500 9,000 14,000 3,000	1050 800 1050 1900 300	3000	90 0	46 49 36			2800 5400 900			147 162 162 167
Metamorphic													
Gneiss (foliated) Schist (to folia-	19,500	35,000	10,500	1500	2100	90 0	50	70	45	3200	4700	1700	170
tion) Sehiet (1 to folio	6,800	8, 300	5,300										172
tion) Quartzite Dolomite Slate	13,500 35,000 25,000 26,000	18,000 55,000 45,500 33,000	9,300 15,000 4,500 19,000	1000 2300 1900 2450	1200 5000 3400	800 1000 400	52 50 45	65 60 60	45 40 40	4300 1000 4000	8000	2000	172 170 175 170
Sedimentary													
Siltstone (mid-US) Sandstone (ce-	900	1,800	500	50	9 0	30							131
mented) Limestone Clay shale (mid-	12,000 16,000	32,000 26,000	8,000 8,000	900 1100	1800 1800	0 50	44 46	55 57	33 35	1600 2500	3200 3700	0 1200	125 156
US) Conglomerate	25,000	8,000	500	300	50 0	50	44	50	35	1150	1700	50	144
(southwest)	1,200	2,000	400	300	50 0	0	35	40	20	250	40 0	10 0	138
Miscellaneous Coal (subbitumi-													
nous, lignite) Fault gouge (clay with rock) Fault gouge (clay)	2,500	4,500	500	300	500	100	47	65	30	40	60	20	81
Broken rock													120
graded) Sand and silt Clay													120 125 100

*Metric equivalents: psi \times 6.894 757 = kPa; pfc \times 16.018 46 = kg/m³.

of rock slides on another. For rock-fill materials, Barton proposes the construction of a tilt box to contain the rock fill, but the test procedure is the same as for fractures.

Rock Mass Characteristics

Block size is an important characteristic of the rock mass for crushing and grinding, leaching, excavation, drilling, and blasting, as well as the effect it has on rock mass strength.

Block size index is one measure of block size (Brown, 1981). To determine the index, a bench face is selected that appears consistent in fracturing and rock type. Typical maximum, minimum, and mode block sizes are measured, and the number of different fracture sets bounding the measured block are noted. Information on the number of fracture sets forming the block is used to adjust block volume calculations. Size distribution curves might be estimated from this data for each structural domain.

The volumetric joint count is the sum of the number of joints per meter for each joint set. A bench face is selected as in the block size index determination. For each joint set, average true spacings of the joints in each set are calculated from the number of joints in the set occurring over a specified distance measured normal to the joint set. The volumetric joint count is the sum of the number of joints per unit length for all sets. The following is an example.

Set 1: 6 joints in 20 m (65.6 ft) Set 2: 2 joints in 10 m (32.8 ft)

SURFACE MINING

	Peak Fi	riction A	Angle	Peak C	ohesion	, psi	Resid	ual Frict Angle	ion	Residu	al Cohe psi	sion,
Material	Typical	High	Low	Typical	High	Low	Typical	High	Low	Typical	High	Low
Igneous intrusive												
Fresh granite Altered granite (porphyry copper)	34	40	° 30 25	24	40	7	30 30 27	35	28	1 5	2	0
Quartz diorite Diabase	36 44	35	25	20	40	U	23	27	20	1.5	3	0
Igneous extrusive Rhyolite Decite												
Andesite Basalt Wolded tuff	40 26	45 65	35	120	250	50 25	37 35	45 38	25 31	20		
Metamorphic	01	05	55	90	150	20	50	00	40	20		
Gneiss (foliated) Schist (to foliation)	30	35	25				32	35	30			
Quartzite Dolomite Slate	34						28 31 28	32 36 30	26 26 25			
Sedimentary												
Siltstone (mid-US) Sandstone (cemented) Limestone	40 33 37	50 43 47	30 23 27	50 20 50	65 30 100	40 0 0	27 33 37	34 43 47	18 23 27	5 2	15 26	0 0
Clay shale (mid-US) Conglomerate (southwest)	20 39	32 45	14 30	2.5 50	7 90	1 25	15 32	18 35	8 29	0.3 15	0.5 25	0 0
Miscellaneous												
Coal (Subbituminous, lig- nite) Fault gouge (clay with rock)	26 21	27	15	12 21	35	10	12 19	25	8			
Fault gouge (clay) Fault breccia												
Broken rock Gravels (well graded) Sand and silt Clay	37 40 35 21	41 44 41 30	34 36 30 10	10	20	5	37 40 35 14	41 44 41 16	34 36 30 11			

Table 4. Rock Fracture Properties

*Metric equivalent: $psi \times 6.894757 = kPa$.

Set 3: 20 joints in 10 m (32.8 ft) Set 4: 20 joints in 5 m (16.4 ft) Vol. ____ Count = 6/20 + 2/10 + 20/10 + 20/5= 0.30 + 0.20 + 2.00 + 4.00 = 6.5 joints/m³

Block shape, number of joint sets, and joint lengths should be recorded to give a better description of the block size.

Screen tests could also be devised for size gradation determinations by constructing several size grizzlies and coarse screens. These are time-consuming tests and equipment construction can be expensive. Screening can be done only on blasted or loose rock and representative sampling is difficult.

Inspection of muck piles and dumps also gives an indication of block size and shape. The procedure is similar to the block size index where maximum, minimum, and mode block sizes are measured.

Usually, blasting tends to open up existing fractures and actual rock breakage is small. Therefore, the block sizes measured from blasted rock and dumps are often a reasonable estimate of in situ block size. Rock quality designation (RQD) and core logging also provide information on the degree of fracturing, thus, the block size. In some instances, screening of the entire drill core will give a size distribution curve that is reasonably accurate for the lower range of block sizes. RQD, in the strictest sense, is a measure of all pieces of core greater than 10 cm (4.0 in.) in length expressed as a percentage of the total length in the drill run, and could be considered as a measure of ± 10 cm (± 4.0 in.) rock blocks.

Estimation Methods

The Unified Soil Classification, which includes both quantitative and descriptive information, is a classic manual index test to determine characteristics of soils (Lambe and Whitman, 1969). Other soil indices can be found in Sowers and Sowers (1970). In geology and rock mechanics work, the rock hardness index proposed by Jennings and Robertson (1969) and Piteau (1970) are used (Table 5). Kirsten (1982) gives a field identification procedure for estimating compressive strength and vane shear strength.

Rule of thumb criteria for estimating strength has de-

veloped over the years. One is for tensile strength of rock. Tensile strength is one-tenth to one-twentieth the compressive strength. One-tenth is generally the closer estimate. A similar rule might be used for shear strength of intact rock substance. Maximum shear strength will not exceed half the uniaxial compressive strength.

Barton (1982) has proposed a classification scheme based on the roughness of joints and joint wall hardness. This classification is used to estimate peak shear strength. Caution in using this method is warranted because analysis of many slopes should not be done using peak shear strength values. Residual strengths may be a better estimate of actual conditions.

If slopes are available where failed geometries are present, back analysis can be done to determine strength values required for stability by assuming the failures are at limiting equilibrium. In many cases, these results are the "best estimates" for shear strength because they have the loading, geometry, and shear strength relationships of actual field conditions. However, one significant parameter that often cannot be reconstructed for the back analysis is the water condition at the time of failure. The water condition can be very critical to the stability calculation, and the assumption used will affect the results of the back analysis.

It should be noted that stable slopes are also useful indicators of strength because they give a lower bound, just as failed slopes give the upper bound of shear strength. If enough slopes in both failed and unfailed conditions can be observed, the actual strength values can be bracketed, and the ability to accurately estimate shear strength improves. McMahon (1976) reports on his study and use of back analysis in slope design.

Simulation to predict block size is becoming more common. The usual procedure is to randomly sample joint orientations and spacings and to make some assumption regarding joint lengths to model the fracturing. Numerical methods are generally used to calculate the size and number of blocks. Conceivably, this modeling could be used to develop size distribution curves for the rock blocks. There are, however, still major difficulties to overcome regarding appropriate length values before this method can be applied by people other than specialists. Additional work is being done using key block theory to describe rock block shapes and volumes (Goodman and Shi, 1985).

Table 5. Rock Hardness Index

Grade	Description	Field identification	Approximate range of uniaxial compressive strength, MPa
S1	Very soft clay	Easily penetrated several inches	< 0.025
S2	Soft clay	Easily penetrated several inches	0.025-0.05
S3	Firm clay	Can be penetrated several inches by thumb with moder- ate effort	0.05-0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great ef- fort	0.10-0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.50
S6	Hard clay	Indented with difficulty by thumbnail	> 0.50
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0–25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological ham- mer	25–50
R4	Strong rock	Specimen requires more than one blow of geological ham- mer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to frac- ture it	100-250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	> 250

Classification Methods	Rock Substance Strength	Drill Core Quality, RQD	Joint Spacing	Joint Orientation	Joint Strength	Rock Genesis	Ground- Water Conditions
CSIR rock mass rating (Bieniawski,	-						
1974)	X	Х	X .	Х			Х
NGI tunneling index (Barton, et al.,							
1974)	Х	Х	х	Х	Х		Х
Coates, 1981	х		х			х	
Deere, 1968	Х		х			X	
Müller and Hofmann, 1970	X		X				
Zavodni and McCarter, 1977	Х	х	х				х
GSL rock quality (Franklin, et al.,							
1971)	Х		х			х	
Kirsten, 1982	х		х	х	Х		

Table 6. Attributes Used in Some Developed Classification Schemes

Plate tests and radial jacking tests can be used to estimate the deformation characteristics of the rock mass.

ROCK MASS CLASSIFICATION

Rock mass classification schemes are generally not sufficient design criteria for slopes. Their applicability has more merit for excavating, crushing, and leaching. Franklin, et al. (1971) discussed classification schemes based on fracture spacing and compressive strength of the rock substance to describe the rock mass. They then related it to the excavation characteristics of digging, ripping, and blasting.

Kirsten (1982) adapts classification to excavation and calculates an excavability index which relates to the equipment needed. He has developed his classification for both soil-like and hard rock applications.

A useful classification scheme provides comparisons of rock mass properties within a mine as well as a comparison between different mines. The classification should indicate some behavioral characteristic in a quantifiable manner. Then the classification is used to predict rock mass behavior in an area where the classification system is identifiable, which allows better planning and design before mining begins.

Available Classification Schemes

Classification schemes have been developed for a variety of purposes. Some mines may be able to adopt one of these classifications or modify it slightly to fit their needs. Others would have to develop their own classification based on the attributes of interest. Table 6 presents a list of some of the better known classifications and the attributes upon which they are based. Table 7 presents current classification schemes and their applications.

Developing a Specific Classification Scheme

Classification schemes are based on time, space, physical properties, and relationship between properties. An example of a time-related attribute would be the seasonal fluctuation in ground-water level or the time-dependent movement of a pit slope. Space-related attributes are most common and would include the variations in such attributes as rock hardness or fracture frequency over mine areas, which in themselves are examples of physical properties of the rock mass. Relationships between properties would be exemplified by RQD, a property of the rock mass that is dependent upon both rock hardness and fracture frequency (Deere, 1968). Coates (1981) gives further discussion on developing meaningful classifications.

The most effective method for developing a useful classification is to decide on the attributes of interest and to develop a set of overlay maps displaying the areal distribution. Varnes (1974) is an excellent reference on the method of attribute selection and map development.

SLOPE DESIGN

Slope design involves analysis of the three major components of a mine slope: bench configuration, interramp angle, and overall slope angle (Fig. 5). Bench configuration is

Classification Method	Ground Stability	Block Size Distribution	Bearing Capacity	Diggability	Blasting Requirements	Crushing/ Grinding Requirements
CSIR rock mass rating						
(Bieniawski, 1974)	Х					
NGI tunneling index						
(Barton, et al., 1974)	Х					
Coates, 1981	Х					
Deere, 1968	Х					
Müller and Hofmann, 1970	Х	Х	Х			
Zavodni and McCarter,						
1977	Х	Х				
GSL rock quality (Franklin,						
et al., 1971)	Х		Х	Х	X	
Kirsten, 1982		х		Х	Х	

Table 7. Information Provided by Some Developed Classification Schemes



Fig. 5. Definition of bench face, interramp, and overall angles.

defined by bench height, width, and face angle; the interramp angle is defined by the bench configuration; and the overall slope angle is defined by interramp sections separated by haul roads or mining levels. If through-going structures do not produce the possibility of large-scale failure and joints lengths are short, all slope angles will depend on the bench configuration.

Fig. 6 is a flow chart of the slope design process from data collection to mine design. After data collection, the steps in the design process are:

1) Determine design sectors.

2) Conduct stability analysis to estimate probability of failure and expected failure tonnages for bench interramp and overall slopes.

3) Develop maximum interramp slopes based on catch bench criteria.

4) Determine optimum slopes with a cost benefit analysis.

Design Sectors

Design slope angles within an open pit are influenced by rock strength, geologic structure, hydrologic conditions, pit wall orientation, pit wall height, ore distribution, and operational conditions.

Since any or all of these parameters vary from place to place in an open pit, the pit must be divided into design sectors within which these parameters are similar or will have a similar impact on slope design. Structural domain



Fig. 6. Slope design flow chart.

boundaries are a primary criteria for sector limits. Changes in wall orientation are logical sector boundaries. Ore distribution and operational considerations affect economic considerations. For example, a concentrator on the edge of the pit would require a higher reliability for the slope for the same economic optimization than a similar pit wall without the concentrator.

Determining design sectors, and slope design in general, is necessarily an iterative process. The slope engineer needs the position, orientation, and height of the pit walls to design the slopes, but the mine planner needs the slope angles to design the pit geometry. Therefore, a pit plan has to be developed based on assumed slope angles. The design sectors are then selected and optimum slope angles are determined. Given these angles, the pit has to be redesigned and the slope angles reevaluated based on the new geometry. This is a formidable task using manual mine planning techniques. However, a new reserve pit can be developed in a few hours at a reasonable cost using floating cone or other pit design computer programs.

Stability Analysis

Stability analysis begins by selecting appropriate numerical models of potential failure modes for each design sector. These models are simplified geometric representations of the actual expected failure mechanisms. Typical failure models are shown in Fig. 7.

Plane Shear Failure: Plane shear failure occurs when the geologic structure has a strike parallel to or nearly parallel to the strike of the slope face and a dip flatter than the slope angle. Plane shear analysis determines the risk of sliding along structures of this type. Controlling factors in the analysis are (1) orientations of geologic structures, which determine whether the fractures have strikes (within about 20°) to the strike of the face and are daylighted (dip angles less than slope angle); (2) structure lengths, which determine



Fig. 7. Typical failure modes.

the probability of having a continuous through-going fracture; (3) structure spacing, which indicates the number of potential failure surfaces in the slope; and (4) structure shear strength, which determines the probability that, if a fracture satisfies all other criteria, the slope will displace along that fracture.

Step Path Failure: In step path failure, as in plane shear failure, it is assumed that sliding occurs along geologic structures subparallel to the slope. However, whereas plane shear displacement is assumed to occur along a single surface, the step path model assumes that failure is due to the combined mechanisms of sliding along surfaces dipping out of the slope (the master joint set) and either separation along geologic structures that are approximately perpendicular to the master set (the cross joint set) or tensile failure of the intact rock connecting members of the master set. Since the step path model does not depend on continuity of the master set, it often has wider applicability than does the plane shear model.

Simple Wedge Failure: Simple wedge geometry is the result of two planar, or nearly planar, geologic structures intersecting to form a completely detached prism of material. The weight of the material and acting hydrostatic forces drive the prism down the line of intersection. To be kinematically viable, the line of intersection must be daylighted. This implies that not only must the plunge of the intersection be less than the dip of the slope, it must also be directed toward a free face; i.e., the bearing of the intersection must be oriented within 90° of the dip direction of the slope.

Step Wedge Failure: Step wedge failure is similar to simple wedge, but in this case the structures that intersect to form the wedge do not need to be single, continuous features. Rather, as with step path, the combination of different structural sets forms the failure surfaces. A variation on this mode is a wedge formed on one side by a single planar surface and on the other by a step path geometry. There is a lack of sufficiently developed analysis for this failure mode, and simplifications are necessary if analysis is required.

Topping Failure: Some authors have proposed this failure mode as a primary failure mechanism (Goodman, 1980; Hoek and Bray, 1981; Brown, 1981). The topping failure mechanism relies on the development of thin slabs of rock that dip away from the slope face. In order to be a viable failure mode, the weight of the slabs must be directed outside their bases. Unless the slabs are very thin, sliding or crushing at the toe must occur before topping is initiated, and the analysis should concentrate on this toe area as the primary failure mechanism. Topping then would be a secondary failure mechanism that would have implications in progressive failure of a slope but would not be used as a primary analysis for slope design.

Rotational Shear Failure: Rotational shear failure occurs in slopes composed of material with low intact rock strength and sparse or nonexistent geologic structure, or material in which the geologic fabric is essentially randomly oriented. The failure surface, generally assumed to be circular or log spiral arc, represents the trajectory of the minimum ratio of shear strength to shear stress. The analysis determines the position of this critical failure surface, which is a function of slope geometry, material strengths, unit weights, and pore water pressure. Using a Monte Carlo simulation technique, material properties are varied for different conditions of slope geometry and pore water pressure distributions. The resulting distribution of the ratio of shear strength to shear stress provides an estimate of the probability of failure.



Fig. 8. Design set determination.

Block Flow

For deep pits or where the rock substance strength is low, the stresses in the wall, particularly in the toe area, can exceed the compressive strength of the rock substance. This can lead to crushing and progressive deterioration of the slope. Coates (1981) presents a simplified analysis to check for the potential of block flow. If the simplified analysis indicates that a potential, more detailed investigation is warranted, use a finite element analysis to estimate the stress distribution and a comprehensive triaxial strength testing program conducted to estimate the distribution of rock strength.

Determination of Potential Failure Geometry

By plotting the pit wall orientation of a design sector on Schmidt plots of the rock fabric and major structures, the impact for stability analysis can be developed (Fig. 8). The fractures and major structures are sorted by the failure-type orientations and the attitude, distribution length, and spacing distributions computed. These design sets may not correspond to geologic sets although the orientation boundaries can be adjusted somewhat to avoid splitting a geologic set. We have found that defining sets by visual or mathematical analysis, while appropriate for geologic fabric analysis, is less satisfactory for slope design, and it is best to use the wall orientation for determining design sets.

Probability of Instability

Determination of the probability of slope instability depends on the ability to quantify the variable character of the geologic parameters. Since physical geologic conditions, such as discontinuity lengths, orientations, spacings, and shear strength vary within the rock mass, statistical distributions are used to represent these geologic parameters. Estimation of these statistical distributions generally requires a representative statistical sample, which often consists of many observations.

In the structurally controlled failure models, the probability of failure (P_f) for a single occurrence of a specified failure mode has three parts:

1) the probability that the dip exists (P_d) ;

2) the probability that the structure is long enough (P_1) ; and

3) the probability of sliding (P_s) .

The probability of dip (P_d) and the probability of length (P_i) are calculated from the statistical distributions of the geologic structures.

The probability of sliding (P_s) is determined by calculating the probability that the shear stress exceeds the shear strength along the failure surface. This probability is calculated from the distribution of safety factors generated either by a technique called Monte Carlo simulation, which involves an iterative process of randomly sampling strength values from the shear strength distribution and subsequently calculating the safety factor, or by the application of closed form mathematical modeling.

Using the calculated mean and standard deviation of the distribution of safety factors and assuming a standard normal distribution, the probability of sliding, or the percentage of the total area of the distribution less than 1.0, can be calculated.

The probability of failure (P_f) for a single occurrence of

the particular failure mode is the probability that the mechanism is viable and that it will displace.

$$P_f = P_1 * P_d * P_s.$$

Since more than one potential occurrence of a specified failure mode can occur in a design section, the expected number of failures is the probability of failure times the probability of occurrence of the structures that constitute the failure geometry. Although the actual number of failures that will occur may be more or less than the expected number, it is the best estimate for design.

Utilizing the expected number of failures and the values calculated in the stability analysis, a probability of failures and expected failure volume curves can be developed (Fig. 9). The curves for all the potential failure modes can be composited to produce an expected failure volume curve for the design sector (see Fig. 10).

In the case of bench analysis, the distance the failure breaks back from the crest of the bench is composited rather than the failure volume.

Slope displacement will occur if the dynamic forces generated by earthquake-induced ground motion are large enough. The response of a slope to the external forces generated by an earthquake will depend mostly on the ground acceleration, the duration of the event, the rock mass strength, and the slope geometry. Slope movement, if it occurs during the seismic event, is assumed to cease when the event ceases. By calculating the total displacement that occurs during the event, a failure can be defined as that situation where displacement is great enough to disrupt normal mining operations (Glass, 1982). Probabilities of failure are generally increased, but often not significantly, when earthquake forces are included.



Fig. 9. Composited slope heights for multiple bench wedge analysis, wedge HL2R1, sector H.



Fig. 10. Sector E, composite failure mode multibench, number of failures and failure tonnage.

Bench Design

Bench faces are normally mined as steeply as possible; as a result, rock falls and raveling are inevitable. Thus, it is customary, and in many cases mandated by mining regulations, that catch benches are left in the pit wall to retain rock falls and raveling.

Analyses of rock fall mechanics by Ritchie (1963) demonstrated that falling rocks impact relatively close to the toe of the slope, but because of horizontal momentum and spin, can roll considerable distances from the toe. Based on his analysis, Ritchie developed width and depth criteria for a ditch at the toe of a slope to protect highways from rock fall. The concept was that the rock would impact in the ditch and the side of the ditch would stop the horizontal roll.

It is not practical to excavate a ditch in an open pit catch bench, but the same effect can be achieved by casting up a berm (Fig. 11). Assuming the berm can be emplaced with slopes of 1.25 to 1, the modification of Ritchie's criteria presented in Fig. 11 is recommended for open pit catch benches. For a given bench height and corresponding design bench width, the upper limit of the interramp slope angle becomes a function of the bench face angle.

The bench face angle, however, is not a unique value because the variability of the rock fabric produces varying amounts of backbreak. Backbreak is defined as the distance from the design bench crest to the actual bench crest. Fig. 12 is an example of the cumulative frequency distribution of measured bench face angles and theoretical bench face angles. The theoretical bench face angle is obtained from stability analyses assuming a vertical bench face and is the upper limit



Fig. 11. Design catch bench geometry. Minimum bench width = 4.5m + 0.2H; berm height = 1m + 0.04H.

Bench Height,	Impact Zone,	Berm Height,	Berm Width,	Bench Width,
m	m	m	m	m
7.5	1.5	0.8	2	3.5
15	5	1	3	8
30	7	1	3	10

of possible bench face angles because it does not include the effect of blasting and digging. Comparison of measured and theoretical face angles at several properties gave a difference of 19° to 25°, except where the bench face was controlled by a strong geologic structure, such as bedding or foliation. In those cases, the measured and theoretical bench face angles were the same.

For an operating property, the measured bench face angles can be used for design. For a new property, the theoretical bench face angle adjusted for the effects of blasting must be used. Rather than choosing the mean bench face angle which would result in 50% of the catch benches less than the design width, or the minimum bench face angle which would result in unnecessarily flat slope angles, it is recommended that a desired catch bench reliability be chosen based on the potential for rock fall and the exposure of personnel. Catch benches in raveling ground being mined by front-end loaders should have a higher reliability than catch benches in massive ground mined with a large rope shovel. A bench face angle should then be chosen to give the desired reliability. For example, if a 90% reliability is desired, the bench face angle would be the angle where 90% of the bench faces will be steeper than the design angle shown in Fig. 12. Using the reliability criteria, the average bench width will be greater than the design width. In the sample shown in Table 8, a 52° interramp slope with a 90% reliability for a 10-m (32.8-ft) bench width has an average bench width of 15.4 m (50.5 ft).

Production bench heights are selected on the basis of equipment size and grade control requirements. There is considerable benefit in increasing the bench height on final



Fig. 12. Bench face angle distribution.

pit walls by leaving a catch bench every other mining level (double benching). A 5° to 8° increase in interramp slope may be achieved by double benching, assuming the same bench face angle. There is also the possibility of increasing the bench face angle with double benching because the majority of backbreak occurs as small failures of the crest. These small failures have less effect on the face angle of a high bench.

Controlled Blasting

The objective of production blasting is to produce the fragmentation required for excavation; however, the blast damage resulting from uncontrolled production blasting at the final wall reduces the bench face angles and, hence, the interramp slope angle. Therefore, controlled blasting in the vicinity of the final wall is desirable.

Measurement of blast damage indicates that a peak particle velocity of about 63.5 cm/s (25 ips) produces displacement on existing fractures and creation of new fractures. By monitoring blasts, the relationship between scaled distance and peak particle velocity can be established. Fig. 13 shows the results of such monitoring. Since the relationship is sitespecific, it is preferable to monitor blasts at the property. The curves in Fig. 13 should be used only as a first estimate.

This scaled distance relationship can be transformed to a curve showing the maximum charge weight vs. distance from the face for 63.5 cm/s (25 ips) peak particle velocity at the face (Fig. 14). At distances where the maximum charge is greater than the charge per hole, blasting can be controlled by limiting the number of holes per delay. At closer ranges, the charge per hole must be reduced and the spacing adjusted on the holes *picked*.

The position of the blastholes relative to the catch benches should also be considered. By strategically placing the holes, damage to the catch benches can be reduced (Fig. 15). If blastholes are placed directly over the underlying bench crest, the subgrade damages the crest and reduces the bench width.

Slope Angle	Median, Face 75°		Bench Width, 80% Reliability Face 68.1*		90% Reliability Face 65.8°		
	SB	DB	SB	DB	SB	DB	Offset
34	18.2	36.4	16.2	32.4	15.5	31.0	4.0
35	17.4	34.8	15.4	30.8	14.7	2 9.4	4.0
36	16.6	3 3.3	14.6	29.2	1 3.9	27.8	4.0
37	15. 9	31.8	13.9	27.8	13.2	26.3	4.0
38	15.2	30.4	13.2	26. 3	12.5	2 4.9	4.0
39	14.5	29.0	12.5	25.0	11.8	2 3.6	4.0
40	13.9	27.7	11.8	23.7	11.1	22.3	4.0
41	13.2	26.5	11.2	22.5	10.5	21.0	4.0
42	12.6	25.3	10.6	21 .3	9.9	19.8	4.0
43	12.1	24.1	10.1	20.1	9.3	18.7	4.0
44	11.5	23.0	9.5	19.0	8.8	17.6	4.0
45	11.0	22.0	9.0	17.9	8.3	16.5	4.0
46	10.5	20.9	8.5	1 6.9	7.7	15.5	4.0
47	10.0	19.9	8.0	1 5.9	7.2	14.5	4.0
48	9.5	19.0	7.5	15.0	6.8	13.5	4.0
49	9.0	18.0	7.0	14.0	6.3	12.6	4.0
50	8.6	17.1	6.6	13.1	5.8	11.7	4.0
51	8.1	16.3	6.1	12.2	5.4	10.8	4.0
52	7.7	15.4	5.7	11.4	5.0	10.0	4.0
53	7.3	14.6	5.3	10.5	4.6	9.1	4.0
54	6.9	13.8	4.9	9.7	4.2	8.3	4.0
55	6.5	13.0	4.5	8. 9	3.8	7.5	4.0
56	6.1	12.2	4.1	8.2	3.4	6.8	4.0
57	5.7	11.4	3.7	7.4	3.0	6.0	4.0
58	5.4	10.7	3.3	6.7	2.6	5.3	4.0
59	5.0	10.0	3.0	6.0	2.3	4.5	4.0
60	4.6	9.3	2.6	5.3	1.9	3.8	4.0
61	4.3	8.6	2.3	4.6	1.6	3.1	4.0
62	4.0	7.9	1.9	3.9	1.2	2.5	4.0
63	3.6	7.2	1.6	3.2	0.9	1.8	4.0
64	3.3	6.6	1.3	2.6	0.6	1.1	4.0

 Table 8.
 Catch Bench Widths for Single and Double Benching in the

 Diorite East Structural Domain (Using Measured Bench Face Angles)



Note: Single bench height = 15.0 m.

Fig. 13. Ground response to blasting (from Oriard, 1971).

Cost-Benefit Analysis

The simplest method for determining the optimum slope is to compute the incremental cost of slope failure and the incremental benefit per degree of slope increase (Fig. 16). If the incremental benefit is greater than the incremental cost, it is profitable to increase the slope angle. As the incremental cost exceeds the incremental benefit, it becomes unprofitable to increase the slope angle. In general, incremental benefit decreases while incremental cost increases with slope angle. The angle where the two curves cross is the economic optimum, as shown in the example in Fig. 16.

The cost of slope failure is determined by assigning cost models to expected failure volumes and possible mining responses. The benefit is the market value of the recoverable commodity minus the mining and processing costs. Kim (1977) discussed cost models and cost benefit analysis in more detail.

A more sophisticated analysis is to run a Monte Carlo simulation of the sequenced mine plans, applying the probability of failure schedule to include slope failure costs in the cash flow analysis. This way, the effect of interim slope failure and the time cost of money can be included. This type of analysis was developed for the CANMET *Pit Slope Manual* (Kim, 1977).

The cost-benefit approach, using probabilities of failure, provides a methodology by which the risks and costs of failure can be compared with the corresponding benefits for MINE OPERATIONS



Fig. 15. Blasthole layout.

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Fig. 16. Idealized incremental cost-benefit curves showing the economic optimum interramp angle.

any design. The benefit-cost approach does provide an optimized design unlike safety factor methods where the design is considered to be so conservative that little risk or cost of failure is expected. This apparent conservatism can be misleading, as evidenced by the failures of a large number of supposedly *safe* geological designs.

The fact that a slope is designed with some risk of failure must not be viewed as a disregard for safety concerns. Almost any economically viable option will have some probability of failure, and it is better to be aware of the level of that risk. Sufficient, suitable monitoring must be provided to detect instability at an early, noncritical stage to allow for remedial engineering and initiation of safety measures.

MONITORING

In any open pit mine, some slope instability can be expected. The instability can vary from bench sloughing to large-scale slope movement. Because of the inherent variability of rock strength and geologic structure, the uncertainties associated with sampling and measuring rock characteristics, and the mathematical and geometric approximations of the stability analysis, even a *safe* slope, designed to some customary safety factor, has a finite probability of instability.

Acknowledging that slope instability can occur leads to commitment to a monitoring program that ensures safe working conditions. The objectives of pit slope monitoring are (1) to maintain safe operational practices for the protection of personnel, equipment, and plant facilities; (2) to provide advance notice of instability, thus allowing for the modification of mine plans to minimize the impact of slope displacement; and (3) to provide geotechnical information to use for analyzing the slope failure mechanism, for designing appropriate remedial measures, and for conducting redesign of the slope.

Surface displacement measurement using conventional survey equipment and extensometers has been the most

widely used method for monitoring. It is still the most cost effective.

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A survey network consists of targets on the pit slope and instrument stations from which angles and distances to the targets are measured. If a total station EDM instrument is used, approximately 3 min will be required for each reading; thus, about 30 to 40 prism targets can usually be surveyed in a half day. Using a distance meter EDM and a theodolite in combination takes about 5 min per reading.

The survey network has several primary functions.

1) It establishes a surveillance system to detect initial stages of slope instability.

2) It provides a detailed movement history in terms of displacement directions and rates in unstable areas.

3) It defines the extent of the failure area.

Tension Crack Mapping

One early, obvious indication of slope instability is the development of tension cracks. By systematic mapping of these cracks, the extent of the unstable area can be established. The ends of the cracks should be flagged so that on subsequent inspections new cracks or extensions of existing cracks can be identified.

Wire Extensometers

Portable wire extensioneters can be used to monitor areas of active instability and to provide backup for the survey system. They should be positioned on stable ground behind the last visible tension crack and the wire should extend out to the unstable area. The length of the extensioneter wire should be limited to approximately 60 m (197 ft) because sag can produce inaccurate readings. Usually 15 to 20 kg (33 to 44 lb) of counterweight is needed for such a length, depending on the weight of the wire.

Wire extensioneters can be set up as a warning device by affixing a switch several centimeters (inches) above the counterweight. Significant displacement will trip the switch and activate a warning light or siren.

Other Surface Displacement Devices

Tiltmeters and manometers can be used to measure displacement across tension cracks when the displacement is predominantly vertical.

Subsurface Displacement Devices

Subsurface information on instability is needed when surface displacement cannot be used to infer the extent of the instability. Shear strips or a coaxial cable with a fault finder can be used to locate failure surfaces, but these systems are go/no-go devices. Borehole inclinometers measure angular deflection of the borehole and will give the deformation normal to the hole, thus locating failure surfaces. Borehole extensometers measure deformation parallel to the borehole. These extensometers are costly and difficult to use and are only suitable for special applications.

Piezometers measure ground-water levels and pore pressure. Measuring ground-water levels is an important part of monitoring and simple standpipe open piezometers are usually sufficient. However, if there are areas of low permeability or confined aquifers, or when rapid response to reduction in pore pressure needs to be monitored, pneumatic or electric devices may be required.

Microseismic monitoring has historically been expensive because of the electronic equipment needed. However, the lessening cost of equipment in recent years and its increased reliability are making this technique more attractive. Experiments with microseismic recordings have established that there is a correlation between rock noises and slope movement.

Guidelines for Monitoring

1) Measure obvious things first. Surface displacement is the most direct and most critical aspect of slope instability.

2) Simpler is better. The reliability of a series system is the product of the reliability of the individual components. A complex electronic or mechanical device with a telemetered output to a computer has significantly less chance of being in operation when needed than do two stakes and a tape measure.

3) Precision costs money. The cost of a measuring device is often a power function of the level of precision. Measuring to $\pm 1 \text{ cm} (0.39 \text{ in.})$ is inexpensive compared to measuring to $\pm 0.0001 \text{ cm} (0.000039 \text{ in.})$. A micrometer is unnecessary for monitoring slope movement that has a velocity of 5 cm/ d (2 ips).

4) Redundancy is required. No single device or single technique tells the complete story. Backup devices are needed.

5) Timely reporting is essential. Data collection and analysis must be rapid enough to provide information in time to make decisions. Reducing last week's data and telling the mine superintendent that the slope was moving Thursday when a shovel was buried Sunday does not lead to pay raises.

Data Reduction and Reporting

The following measurements or calculations should be made for each survey reading:

1) Date of reading, incremental days between readings, and total number of days the survey point has been established.

2) Coordinates and elevation.

3) Magnitude and direction of horizontal displacement.

4) Magnitude and plunge of vertical displacement.

5) Magnitude, bearing, and plunge of resultant displacement vector.

6) Rates of horizontal, vertical, and resultant displacements.

Both incremental and cumulative displacement values should be determined. Calculating the cumulative displacement from initial values rather than from summing incremental displacements minimizes the effects of occasional survey aberrations.

Slope displacements are best understood and analyzed when the data are graphically displayed. For engineering purposes, the most useful plots are:

1) Horizontal position (northing vs. easting).

2) Vertical position (elevation vs. change in horizontal position, plotted on a section in the mean direction of horizontal displacement).

3) Displacement vectors (plotted on a plan map).

4) Cumulative total displacement vs. time.

5) Incremental total displacement rate (velocity) vs. time.

6) Schmidt plots of total displacement vectors.

Daily precipitation and the number of tons mined beneath the slope should be added to the graphs as histograms to compare these records with slope movement.

A monthly slope stability report should be prepared for mine management. This report serves the dual purpose of providing information to decision makers and providing the discipline to document slope behavior. Direct, informal communication also should be maintained with pit operations on a daily basis in the case of mining in an active slide area.

REMEDIAL MEASURES

With an economically optimized slope design, some degree of slope instability can be expected. Minimization of the adverse effects of slope instability must be accomplished through judicious mine planning and establishment of operational contingencies.

There are several principles of slope mechanics that should be kept in mind in dealing with slope instability.

Slope failures do not occur spontaneously. A rock mass does not move unless there is a change in the forces acting on it. The common changes that lead to instability in an open pit are removal of support by mining, increased pore pressure, and earthquakes.

Most slope failures tend toward equilibrium. It is an observed phenomenon that as a slide displaces, the toe pushes out and the crest recedes. Such displacement reduces the driving force and increases the resistance force so that the displacement rate is reduced until movement stops. When high pore pressures are involved, a similar balance is attained. Displacement causes dilation of the rock mass. As a result, pore pressures drop and the effective shear strength increases. This mechanism explains the stick slip movement of some slides, in which recharge increases the pore pressure in tension cracks, resulting in renewed displacement. There are exceptions to this generalization, but they are usually the result of reduction of shear strength due to shearing of asperities or changes in the forces acting on the rock mass.

A slope failure does not occur without warning. Prior to major movement, measurable deformation and other observable phenomena, such as development of tension cracks, occur. These phenomena occur from hours to years before major displacement. However, single bench sloughing directly associated with mining does occur rapidly. While a slope failure does not occur rapidly without warning, deformation and tension cracks can occur without major displacement.

Detection of Instability

The first step in slope management is the identification of potential failure areas such as faults, breccia dikes, and/ or jointing with attitudes that would form a failure geometry. Data for this identification would come from geologic pit mapping. Areas of higher water levels are also potentially unstable and should be identified.

The second step is monitoring areas that are potentially unstable and/or show evidence of instability by displacement and tension cracks.

On the basis of monitoring and mapping, the geometry of a failure can be determined and predictions made of future behavior.

Slide Management

When instability occurs there are a number of response options:

1) Leave the unstable area alone.

2) Continue mining without changing the mine plan.

3) Unload the slide through additional stripping.

- 4) Leave a step out.
- 5) Partial cleanup.
- 6) Mine out the failure.
- 7) Support the unstable ground with cable bolts.

8) Dewater the unstable area.

The choice of options or combination of options depends on the nature of the instability and the operational impact. Each case should be evaluated individually and cost-benefit SURFACE MINING

comparisons conducted. The following are guidelines on the choice of options.

1) When instability is in an abandoned or inactive area, it can be left alone.

2) If the displacement rate is low and predictable and the area must be mined, living with the displacement while continuing to mine may be the best action.

3) Even though unloading has been a common response, in general it has been unsuccessful. In fact, there are situations involving high water pressure where unloading actually decreases stability.

4) Step-outs have been used successfully in several mines. The choice between step-out and cleanup is determined by the tradeoff between the value of lost ore and the cost of cleanup.

5) Partial cleanup may be the best choice where a slide blocks a haul road or fails onto a working area. Only that material necessary to get back into operation need be cleaned up.

6) Where the failure is on a specific structure and there is competent rock behind the structure, mining out the failure may be the optimum choice.

7) Mechanical support may be the most cost-effective option when a crusher, conveyor, or haul road must be protected.

8) Where high water pressure exists, dewatering is an effective method of stabilization that may be used in conjunction with other options.

Contingency Planning

Mine planning should have the flexibility to respond to slope instability. Rather than an "after the fact" crisis response to forced deviation from a rigid mine plan, contingency plans should be prepared in advance so that the response to slope instability is well thought out.

Operational flexibility should be built into the mining plan; for example:

1) Adequate ore should be exposed and accessible so that production is not dependent on a single location.

2) There should be more than one access road into the pit for service vehicles.

3) Whenever possible, double access to working benches should be maintained.

4) Production scheduling should have a provision for slide cleanup.

Excavation

Equipment selection and estimation of production rates and costs require a knowledge of the rock mass properties. A simple classification system based on rock substance compressive strength and fracture frequency can be used (Franklin, 1971) (Fig. 17). Kirsten (1982) has developed a classification that includes fracture and equipment characteristics.

Regardless of the classification system used, it is important to zone the excavation area rather than use average values. Although the average rock properties may indicate that blasting is not required, ignoring the 20% that does require blasting can have a devastating impact on mining costs.

CRUSHING AND GRINDING

Miners often attempt to improve fragmentation by altering blasting practices, but generally after experience and disappointment, will go to crushing because it is the only way to guarantee a uniform product size. In hard rock,





fragmentation can only be improved to a block size consistent with the fracture intensity. This is because the fractures are inherently weaker than the surrounding rock substance and the principles of fracture mechanics apply. These principles do not allow significant rock breakage, particularly with low velocity explosives such as ANFO, because less energy is required to open or extend existing fractures than to initiate new fracture and break rock substance. Some pulverization of the rock occurs immediately near the blasthole and, if costs allowed, high velocity explosives and a blasting pattern could be designed to pulverize the entire rock mass. Practically, however, the volume of pulverized rock in a blasthole is very small and probably insignificant when compared to the total volume of rock that is thrown into the muck pile.

Another consideration in attempting to improve fragmentation is that unless care is taken, mining costs could be significantly increased if the stability of final pit walls is decreased because of blast damage. To properly determine crushing requirements, the rock hardness, the size of the blocks to be crushed, and the tonnages needing crushing need to be known. These are parameters that will, in part, determine energy requirements, crusher size, grizzly size, abrasiveness, maintenance schedule, and equipment life. Siting of the crusher might be based on proximity to the material that requires the most crushing; thus, spatial distribution of large blocks must be known. Ultimately, conveyor belt size and other processing equipment will be selected based, in part, on the ability to crush materials to a specified size.

Rock Hardness

Compressive strength is the best indicator of rock hardness for crushing. Compressive strength can be estimated from laboratory tests on core from drill hole or rock blocks, from point load tests on irregular rock samples or drill core, or from a rock hardness classification. If rock hardness is plotted on level maps next to the drill hole locations or sample sites, the spatial distribution of hardness can be defined.

Block Size Prediction

Block size distribution can be determined by simulation and direct measurement. Both estimation methods require a knowledge of *fracture frequency*, i.e., a count of the number of fractures per meter (foot) without regard to orientation or the manner in which the fractures are separated. Rock Quality Designation, RQD, can also be used. A relationship between RQD and fracture frequency was reported by Priest and Hudson (1975). First it is necessary to define areas of similar fracture frequency on level maps. A good first approach is to contour fracture frequency on each level. Areas where data cannot be obtained from core holes can often be supplemented by mapping underground drifts and pit faces. Commonly, extrapolation is necessary, and this can lead to more sophisticated methods of estimation techniques, such as the geostatistics used in ore reserve estimation.

Simulation

After areas of similar fracture frequency have been defined, simulation of the fracturing in the rock mass can be done using Monte Carlo sampling of orientations and spacings. Block sizes can be calculated if all fracture lengths are assumed continuous, which is not a bad assumption because fractures are extended during the blasting. This type of simulation has been done to predict cavability for block caving (White, 1979). A simulation requires good input data, which includes nearly complete knowledge of the fracture orientations and the distribution of spacings. This data can be time-consuming to obtain and often requires special expertise in data reduction.

There are mathematical models that assume equidimensional fragment shape and that consider the probability of a drill hole or mapping line intersecting the maximum dimension of the fragment. This method results in an approximation of a size distribution curve based on fracture frequency. A curve would be estimated for each area defined on the level maps. Again, the mathematics are somewhat complex, and the results should be checked with direct measurement before extrapolation is done.

Direct Measurement

Most mining companies will not have the means or the desire to attempt simulation or mathematical modeling. Even when these methods are used, some direct measurement is still required to confirm predictions. Again, in direct measurement, areas of similar fracture frequency should be defined on level maps. Direct measurement is essentially a largescale screening of as much tonnage from the muck pile as is feasible. A set of grizzlies or screens or some combination of screens and grizzlies would be constructed and several tests conducted to develop the size gradation curves for each area defined on the level maps.

Another less accurate method might be simply to estimate percentages from the muck pile. This would be a very crude approximation and many inaccuracies could result, but these approximations would still be better than no attempt to quantify the fragmentation and the crushing requirements. Consideration should be given to photogrammetry to assist in the estimation. Franklin, et al. (1988), report on encouraging results using photoanalysis to determine size distribution of blasted rock.

Both direct measurement methods require fracture frequency to be estimated prior to the blast to correlate fracture frequency with the resulting size distribution curve.

The Geotechnical Model

Once rock hardness and fracture frequency with the corresponding size distribution curves have been defined for each level, combining the data into a final rock classification for crushing requirements is done. A final level map is produced with areas defined according to rock hardness and block sizes. The result is a geotechnical model that can be used to estimate hardness, size, and tonnage in the same manner that tonnage and grade are estimated in ore reserve estimation. If a product of, say, less than 5 cm (2 in.) is desired, a tonnage by level which requires further crushing can be measured. It would be possible with this information to determine crushing requirements for each period in the mine life.

LEACHING

The geotechnical model is also useful for estimating the product size that ultimately reaches the leach dump. Recovery in a leaching operation is determined in large part by the amount of surface area on an individual rock fragment or block that is exposed to the leach solutions. If the tonnages of each size material are known, approximations can be made regarding the available surface area. Leach tests can be designed to give a more accurate prediction of expected recoveries. Thus, a more accurate estimation of recovery from the leach system can be made.

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