Chapter 10.4
SLOPE STABILITY

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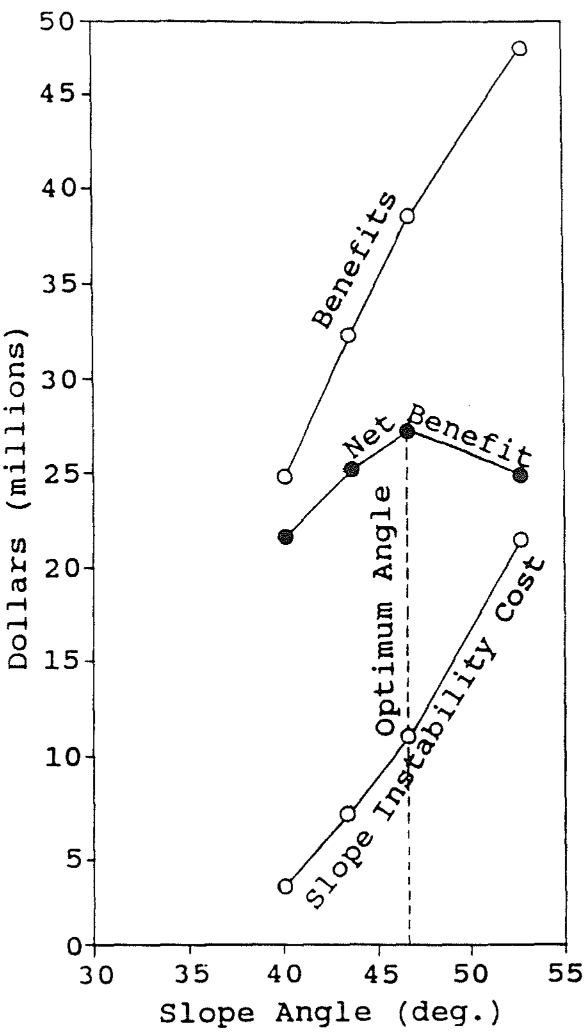

Fig. 10.4.1. Cost benefit curves.

10.4.1 INTRODUCTION

10.4.1.1 Design Approach

In the design of a typical open pit, increasing the slope angle decreases the stripping and/or increases the recoverable ore, which produces a higher benefit or return on investment (Fig. 10.4.1). However, increasing the slope angle decreases the stability of the slope. Because of the variability of geologic structure and rock properties, there is not a unique angle below which there is no slope instability and above which massive failure occurs. More typically, as the slope angle is increased, the number, size, and movement rate of slope failures increases. These slope failures result in operating costs such as the expense of cleaning up failed material, lost production, and unrecovered ore.

At steeper slope angles, the cost of slope instability increases more rapidly than the benefits. Thus the net benefit curve obtained by subtracting the cost of instability from the gross benefit has a maximum. The slope angle at which this maximum occurs is the *optimum angle,* since mining at a flatter angle results in higher stripping costs and reduced ore recovery. Conversely, mining steeper than the optimum results in slope instability costs greater than the increased ore recovery.

Slope design is the process of determining this optimum angle for input into pit design. The slope stability portion of slope design is the prediction of the slope instability as a function of slope angle.

10.4.1.2 Stability Criteria

From the standpoint of simple mechanics, the *stability of a slope* is the ratio of the strength of the material to the stresses in the slope. If the stress exceeds the strength, the slope is unstable; conversely, if the strength exceeds the stress, the slope is stable. This ratio is termed the *safety factor* and has been the basis for stability analysis in civil engineering for many years. Because of the variability of rock properties, uncertainty in the measurement of these properties, and the influence of quasi-random events, such as earthquakes and rainfall, the stresses and strengths used in stability are estimates of populations with significant distributions rather than single values. For this reason, safety factors greater than one have been used for slope design. An alternate approach to defining stability is to use the reliability method, whereby the probability of whether or not a slope will be stable is calculated from the distribution of input values.

Slope instability does not necessarily mean slope failure from the operational standpoint. It is not uncommon for a slope to become unstable, with the resulting displacement being less than 3 ft (1 m). Whether an unstable slope results in significant cost to the operation depends on the rate of movement, the type of mining operation, and the relationship of the unstable material to the mining operation. Unstable areas with displacement rates of over 4 in. (100 mm)/day have been successfully mined by truck and shovel operations. On the other hand, a few inches (millimeters) of displacement of the rock under a crusher, conveyor, or building may require extensive repair. When the rate of displacement is such that it disrupts the operation or the movement produces damage to mining facilities, it is considered an operational slope failure. A similar economic concept was used by Varnes (1958) to distinguish between creep and landslides. He restricts the lower limit of the rate of movement of landslide material "..to that actual or potential rate of movement which provokes correction or maintenance."

10.4.1.3 Safety

Another aspect of slope stability is *slope management.* In an optimized slope, some slope failure can be expected but the specific location and time of instability cannot be predicted with any certainty. Also the stability analyses utilized in design, with very few exceptions, are static solutions that do not provide

Table 10.4.1. Comparative Approximate Fatality Rates (per 10^6 hours of exposure)

Highway travel		1.9
Air travel		2.4
Motorcycle travel		4.4
Cigarette smoking		2.6
Open pit mining		0.42
Falls of rock	0.01	
Runs of muck, stockpiling, etc.	0.03	
Fall of person	0.05	
Vehicle accident	0.10	
Miscellaneous	0.23	
Logging		0.94
Construction		0.26

Source: Coates, 1977

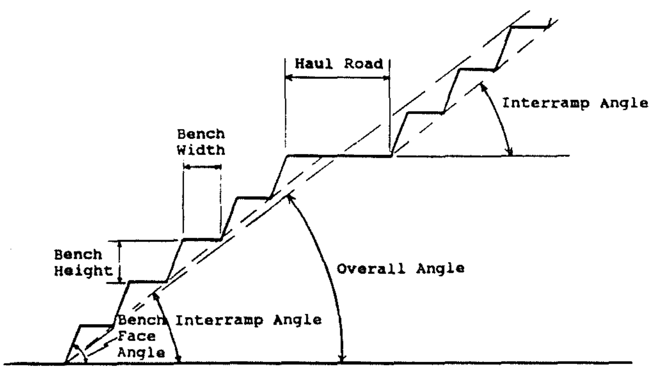

Fig. 10.4.2. Typical design cross section.

estimates of the rate or magnitude of displacement. Therefore, to provide safe working conditions and minimize the economic impact of slope instability, there should be a program of displacement monitoring to provide advance warning of major slope displacement, accompanied by design of remedial measures. In spite of the uncertainty in slope stability, the safety record has been excellent compared with mining in general and other activities (Table 10.4.1). With an appropriate slope management program, it should be possible to mine steep slopes with an equal or greater safety record.

10.4.1.4 Slope Geometry

There are three major components of a pit slope: bench configuration, interramp slope, and overall slope (Fig. 10.4.2). The *bench configuration* is defined by the bench face angle, the bench height, and bench width. The *interramp angle* is the slope angle produced by a number of benches. Where there are haul roads, working levels, or other wide benches, the *overall slope angle* is the angle of the line from the toe to the crest of the pit; the slope angle will be flatter than the interramp angle. It is important in slope design to consider these components. For example, in the case of bedding dipping into the pit at 40°, the daylighting plane shear criteria would result in a design angle of 40°. If this angle were used for the overall slope angle, haul roads cut into the slope would undercut the bedding and result in interramp instability. In addition, there would be almost no catch benches left, as the bench face angle would be steeper than the bedding.

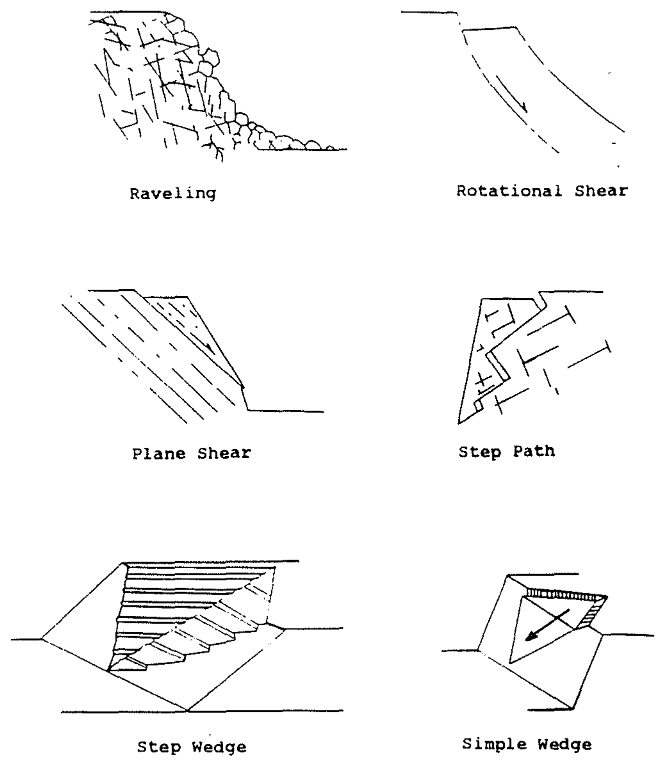

Fig. 10.4.3. Typical failure models.

10.4.2 INSTABILITY MODELS

In order to make a quantitative estimate of the stability of a slope, analytical models amenable to mathematical solutions must be used. The requirements of these models are the failure geometry and assumptions regarding material properties and stress distributions. (In the following discussion the term "failure" is used for simplicity and to be consistent with prior usage.)

10.4.2.1 Geologic Model

The rock of a slope can be considered to consist of the following components.

Intact Rock: The primary unbroken rock as determined from a piece of core cut for compression testing. The term rock substance has also been used for the unbroken rock.

Fractures: The geologic structures such as joints, bedding, foliation, and minor faults that break the intact rock into more or less discrete blocks. *Discontinuity* is a term that is also used for fractures.

Rock Mass: The combination of intact rock and fractures considered as a unit. Soil could be considered a special case of rock mass.

Major Structures: Geologic features such as faults that are large enough to be mapped and located as individual structures. There is actually a continuum between fractures and major structures, but the differentiation is useful for design purposes.

10.4.2.2 Sliding Block Geometries

The sliding block failure mode refers to a situation in which displacement occurs along one or more geologic structures, and the failure mass is considered to be a rigid block or a number of blocks. These geometries are shown in Fig. 10.4.3.

Plane Shear: The plane shear is the simplest geometry consisting of a single plane striking nearly parallel to the slope. The structure must have a dip flatter than the slope angle (daylighted) and must be long enough to reach the surface or a tension crack. Since the stability analysis is two dimensional, the width of the failure must be great enough that the end results are negligible or there are boundary structures that define the lateral extent of the failure.

Step Path: The step path geometry occurs where there is a fracture set dipping into the pit in the plane shear orientation, but no individual fracture is long enough to form a plane shear geometry. Sliding is assumed to occur along fractures in the plane shear orientation (the master joint set) and separation along fractures approximately perpendicular to the master joint set or tensile failure of the rock between the master joints.

Wedge: The wedge failure geometry is the result of two planar geologic structures intersecting to form a detached prism of material. Sliding can occur down the intersection or on one plane with separation on the other plane. In some cases the sliding on one plane will be a rotation rather than simple translation.

Step Wedge: The step wedge is similar to the simple wedge except that one or both of the failure surfaces are step paths.

Two-block: The two-block is a two dimensional plane shear geometry where there are two plane shear structures dipping into the pit, with a third structure dipping back into the wall that divides the failure into an active and a passive block.

Slab: Where there is bedding or foliation parallel to the pit, slope instability can occur even though the structures are not daylighted. The possible failure mechanisms are crushing at the toe, a two-block geometry formed by joints at the toe, and buckling.

10.4.2.3 Nonplanar Failure Surfaces

Rotational Shear: In a soil or weak rock mass slope where there are no geologic structures that control the failure, the most unstable failure surface is approximately a circular arc. The radius and location of the most unstable circle (the critical circle) depends on the material properties and must be found by iterative solutions of trial circles. The stability of the circular arc is usually analyzed by the method of slices. The failure is divided into a series of vertical slices so that the failure surface can be approximated by planar segments. The driving forces and resisting forces on the failure surface at the base of the slice, as well as the interslice, forces are summed up over the slices. The method of slices is normally a two-dimensional analysis.

General Surface: The general surface is a mixed mode failure in which part of the failure surface is structurally controlled and part is failure through the rock mass. An example would be a nondaylighted plane shear. The method of slices can be used to analyze the stability of the general surface.

10.4.2.4 Other Models

Block Flow: Compared with underground rock mechanics, the stresses in a pit slope are low and do not exceed the rock mass strength. Thus most slope instability is controlled by geologic structure. However, in deep pits, there is the possibility that the stresses in the toe of the slope would be sufficient to result in the crushing failure of the rock mass, particularly if there was a high horizontal stress. This mode of instability was referred to as *block flow* by Coates (1981).

A conceptually possible variation of the block flow would be a situation where the rock mass under confinement in the slope wall yields plasticly. The resulting deformation would be plastic flow such as occurs in a glacier. At the surface of the pit slope, where there is no confinement, secondary sliding block failure would occur similar to the calving of a glacier. This is a possible explanation for situations where instability occurs in a relatively flat slope, and the back analysis indicates an anomalously low shear strength.

Toppling: Where there are steeply dipping structures that result in blocks with a large height-to-thickness ratio, the *toppling* failure mode has been postulated. For toppling to occur, the center of gravity of the block must be outside the toe of the block. Therefore, sliding or crushing of the toe must occur before toppling is initiated unless the slope is mined steeper than 90°. Because of this, toppling is most commonly observed as a secondary failure mechanism resulting from displacement caused by another mode of instability.

An exception to this generalization is where ice wedging or pressure from water-filled cracks causes toppling, as in the case of the Hells Gate Bluffs failure in Fraser Canyon, British Columbia (Piteau et al., 1976).

Rockfalls and Raveling: Bench faces are normally cut as steeply as the loading equipment can dig them. As a result, individual blocks in the face are at or close to limiting equilibrium, and disturbing forces can dislodge them. The primary disturbing forces are freeze/thaw and water from rainfall. The action of these disturbing forces can dislodge individual blocks, producing a rockfall. The dislodging of large numbers of blocks is termed *raveling*. Weathering can also produce raveling by the deterioration of the material supporting the blocks. Although in principle, the stability of individual blocks could be analyzed, there is no practical method of conducting stability analyses for raveling on a pit scale. The design approach is to provide for adequate catch benches.

10.4.3 STRESSES IN A SLOPE

Although most stability analyses assume simple gravitational body loading to calculate the stress on a failure surface, it is recognized that the actual stress magnitude and orientation is affected by the in situ stress field, the geometry of the pit, and the variation in material properties.

10.4.3.1 In Situ Stress

Simple gravitational loading would produce a vertical stress equal to the weight of the overlying material, and according to elastic theory, the horizontal stress would be a function of the vertical stress and Poisson's ratio. For the common value of 0.25 for Poisson's ratio, the horizontal stress would be 1/3 the vertical stress. Measurements of in situ stress in underground mines have demonstrated that the horizontal stress can be greater than the vertical stress, as a result of active or residual tectonic stress. The horizontal stress is not equal in all directions, either. In the absence of in situ stress measurements or other indications of a high horizontal stress, the most reasonable assumption is that the horizontal stress is equal to the vertical stress.

10.4.3.2 Slope Geometry

There is a stress concentration at the toe of a slope that is a result of the deflection of stresses around the toe. A high horizontal stress produces a greater toe stress than simple gravity loading. The effect of in situ stress and slope geometry for a plane strain analysis is shown in Fig. 10.4.4. It should be noted that the toe stress is much more dependent on the pit depth and the ratio of horizontal to vertical stress than on the slope angle.

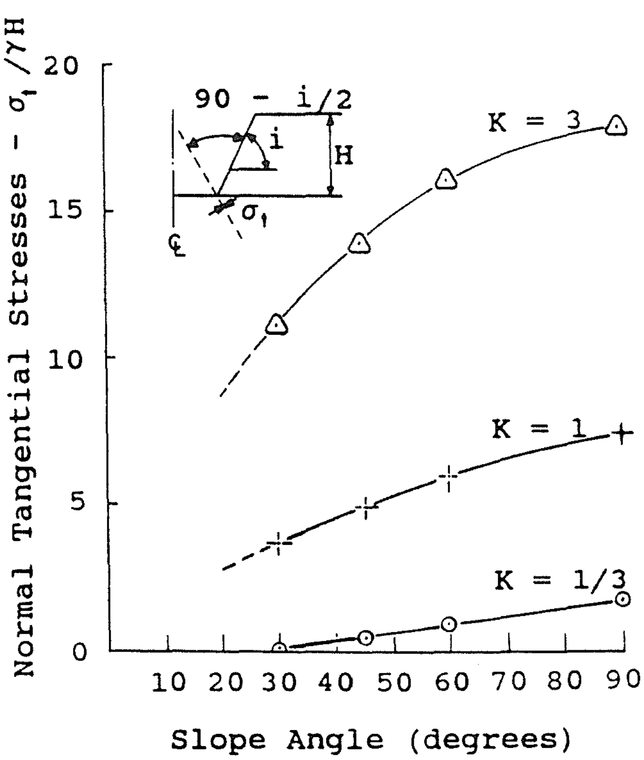

Fig. 10.4.4. Variation in plane strain of the toe stress.

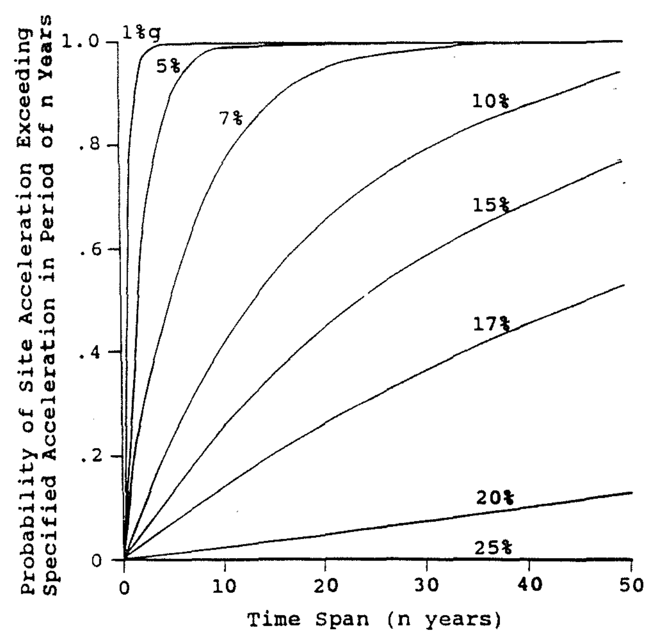

Fig. 10.4.5. Site acceleration probabilities as a function of time. Attenuation calculated according to Patwardhan (1978).

10.4.3.3 Material Properties

Finite element analyses have shown that major stress concentrations can be produced where there are rocks of differing stiffness in the slope. Stiffer rock units carry more load and thus have a stress concentration. Of particular concern for slope stability is the development of high shear stresses in the vicinity of the contact between rocks of differing stiffness.

10.4.3.4 Seismic Acceleration

The shock wave from an earthquake exerts a temporary additional stress on a slope that can cause instability. This has been demonstrated by the number of landslides triggered by earthquakes (Glass, 1982), although this record is misleading with regard to rock slopes; as saturated soil slopes are subjected to liquefaction, which would result in much greater displacement at lower seismic loading. Thus it is appropriate to include the affect of dynamic stresses in the stability analysis of slopes.

The classic method of including the effect of earthquakes in stability analysis is the pseudo-static approach whereby the maximum site acceleration that could be produced by an earthquake is input into the stability analysis as a horizontal force. This approach is excessively conservative when applied to pit slopes, for the reasons listed in the following.

Probability of Occurrence: The maximum earthquake may have a very low probability of occurrence during the critical exposure time of a pit slope. Although the life of a pit may be 20 years or more, the maximum height and angle only exists at the end of the mine life. Therefore, when analyzing the stability of the final slope, the exposure time for that slope geometry is only a few years.

For the cost-benefit approach to slope design, a probabilistic risk analysis of seismic dynamic loading can be used. From the historical earthquake record, a time history of acceleration at the site can be obtained by using empirical attenuation relationships to convert the magnitude and distance for each earthquake to a site acceleration, and calculating the probability of occurrence of site accelerations for time intervals with Gumbel's extreme value theory (Fig. 10.4.5).

Slope Response: The response of a specific failure geometry is a function of the frequency and duration of the ground motion as well as the maximum acceleration. Even if a failure geometry is unstable at an acceleration below the maximum, resulting from ground motion, the amount of total movement may be only a few inches (tens of millimeters). Although a pseudo-static stability analysis would give a safety factor less than one, the displacement would be below the range of what would be considered an operational slope failure.

To estimate the displacement resulting from seismic acceleration, the linear acceleration dynamic response (LADRS) technique developed by Glass (1982) can be used. In this technique the displacement is calculated for small time steps using a digitized model accelerogram and the displacement summed over the duration of the accelerogram. The failure criteria can be expressed as a maximum permissible displacement specific to the slope situation being analyzed. For slopes without facilities such as crushers or conveyors, a maximum displacement of 8 to 12 in. (200 to 300 mm) would be appropriate. Where facilities are present, the displacement tolerance of the structure would be the criteria.

10.4.4 DATA COLLECTION

Collecting adequate and appropriate data for stability analysis is a key aspect of slope design. Obtaining incorrect results from slope stability analysis is predominantly the result of failing to analyze the critical failure mode or not having the suitable estimates of the input parameters such as rock strength or the geometry of geologic structures. With the use of computers, our ability to construct mathematical models and perform the calculations exceeds our ability to collect adequate input data for the models. There are two aspects to the problem of data collection: sampling and measurement.

To illustrate the problem, let us take the specific task of determining compressive strength. This is usually measured by conducting a compressive test on a cylinder of rock 2 to 3 in. (50 to 75 mm) in diameter and 5 to 8 in. (125 to 200 mm) in length. The population of interest (referred to as the target population by statisticians) includes all the cylinders in the volume of rock that could be involved in slope instability. Based on slope behavior and stress considerations, this volume would extend one pit depth back from the design pit and one half the pit depth below the bottom of the design pit. It is obvious that all of the target population could not possibly be tested, so the strength distribution must be estimated by inference, using the test results from some small, hopefully representative, fraction of the target population.

The availability of samples for testing is determined by access, which would be the ground surface, the pit wall, underground workings, and drillholes. Where there is no preexisting pit or underground workings, and the ground surface is covered by alluvium, access to samples is restricted to drillholes. These accessible samples are referred to as the sampled population. The samples that are actually collected and tested are referred to as the sample population. To make valid statistical inferences about a population, every member of the population in question must have an equal likelihood of being sampled, and the tested samples must be an unbiased representation of the population. This can be true regarding the sampled population, but making the step from the sampled population to the target population is more difficult. Because of the restricted access, not all members of the target population are available for sampling even if a specific rock mechanics drilling program is conducted, as there are surface topographic restrictions on locating drill sites, and holes may not be completed because of bad ground. As pointed out by Cochran et al. (1954), "No statistical processes can make the step from sampled population to target population. It can only be done by judgment, intuition, and subject matter knowledge."

Sample disturbance and the difficulty of reproducing the in situ field conditions introduce measurement uncertainties. In the case of rock mass strength, the sample size required makes direct testing prohibitive. Indirect methods, such as modeling the rock mass by compositing the intact rock strength and joint strength, or a rock mass classification with subsequent correlation to empirical behavior are generally employed.

In the case of geologic structure data collection, parameters such as orientation, length, and spacing are geometric rather than scalar, and cannot be measured at a point. This results in a window problem, particularly in the case of fracture length. If the fracture is larger than the observation window, such as a bench face, the length cannot be directly measured. This is why surface mapping is preferable to drillhole data where the core diameter is the window. There is also an orientation bias, as a linear sampling window such as a drillhole does not intersect fractures parallel to the window.

Data collection should be well organized, with specific objectives regarding the use of the data and the quantity required (Table 10.4.2). Collecting data for data's sake should be avoided, as it will result not only in information that is not used, but the possible omission of information needed. Ongoing data reduction is important in order to determine whether a sufficient quantity of appropriate data is being collected.

10.4.4.1 Geology and Major Structure

Conventional geology provides the distribution of rock types and alteration, and the location of major structures. Geologic data should be in the form of a surface map, cross sections, and level maps. It is preferable to have two sets of documents—the factual sheets that show only the actual observations and a set of interpreted maps and cross sections.

For the design of final pits, a geologic map of a trial pit design and cross sections normal to the pit wall should be constructed.

10.4.4.2 Rock Fabric

Rock fabric is the orientation, length, and spacing of fractures. These are the geometric attributes used in stability analysis and in characterizing the rock mass. On a pit scale, the number of fractures such as joints are too numerous to map. Fracture mapping, therefore, consists of measuring the attributes of a subset of the total fractures and characterizing the population with distributions of the attributes.

It has been found from detailed mapping that the orientation of fracture sets has a normal or bivariate normal distribution. Since the orientation is vector quantity, it is properly a spherical normal distribution. However, for the limited range of attitude for a specific fracture set (100), the simple normal distribution is adequate. In the case of folded rock, the poles of the bedding planes fall along a great circle.

The measurable aspect of joint size is the trace length, which is the intersection of the joint and the mapping surface. The negative exponential appears to be the best distribution for trace lengths on the basis of fit to mapping data and theoretical considerations. Models such as the circular disc and the Poisson flat have been postulated to describe joints in three dimensions. These models can be used to correct for the observation window limitation.

Several common mapping methods are available.

Fracture Set Mapping: This is a modification of conventional joint mapping where fracture sets are identified by eye, and the orientation, length, and spacing are recorded. If joints or other structure orientations have been recorded during regular geologic mapping, they can be compiled and used in slope design.

Detail Line: The detail line method is a systematic spot-sampling method in which a measuring tape is stretched along the bench face or outcrop to be measured. For all the fractures along the tape, the point of intersection with the tape, orientation, length, roughness, filling type, and thickness are recorded. To get an adequate representation of the fabric, 100 to 150 fractures need to be mapped. This is the least subjective method, as individual fractures are recorded, and it provides the most detailed length and spacing data. It is relatively inefficient, however, as more observations are made on closely spaced fracture sets than are required for adequate statistical representation.

Cell Mapping: In this method, mapping surfaces such as a bench face are divided into cells. Normally, the width of the cells is made equal to the height of the cells. Within each cell, the fracture sets are identified by eye, and the orientation, length, and spacing, characteristics are recorded. Cell mapping is a combination of fracture set mapping and detail line, with the efficiency of visual identification of fracture sets and some of the more rigorous measurements of detail line.

Oriented Core: To obtain subsurface fabric, oriented core can be used. In inclined holes excentrically weighted, imprinting devices can be used to determine the orientation. In vertical holes, a scribing technique coupled with a downhole compass must be used. Oriented core provides information on fracture

Table 10.4.2. Checklist for Preliminary Slope Stability Data

MAPS AND SECTIONS	
Regional (1:50,000 or USGS quadrangles)	
Surface topography	( )
Surface geology	
Outcrop	( )
Interpretation	( )
Typical geologic cross-sections	( )
Hydrology	
Drainage areas and surface flow	( )
Groundwater	
Water levels in drillholes	( )
Contour map of piezometric surface	
Mine Area (1:5000 or 1:10,000)	
Surface topography	( )
Surface geology	
Outcrop and drillhole locations	( )
Interpretations	( )
Geologic cross-sections	
Surface and drillhole data	( )
Interpretations	( )
Groundwater	
Water levels in drillholes	( )
Contour map of piezometric surface	( )
Pit (1:1000 or 1:2000)	
Surface topography	( )
Trial pit plans	( )
Geology	
Surface outcrop	( )
Surface interpretations	( )
Grid cross-sections	
Drill data	( )
Interpretations	( )
Radial cross-sections	( )
Drill data	( )
Interpretations	( )
Level maps	( )
Structure contour maps	( )
GRAPHS	
Precipitation	( )
Water levels in drillholes	( )
Stream flow	( )
RQD of drill core	( )
TABLES	
Surface fracture data	( )
Drill core fracture attitude	( )
RQD of drill core	( )
SAMPLES	
Type samples for each rock type	
Uniaxial compression (6 per rock type)	( )
Triaxial compression (6 per rock type)	( )
Disc tension (3 per rock type)	( )
Direct shear (3 per rock type)	( )
PHOTOGRAPHS	
Drill core before splitting	( )
General views of pit	( )
Typical outcrops	( )
Major structure exposures	( )
Aerial photographs	( )
OPERATING PITS SHOULD ADD THE FOLLOWING:	
MAPS AND SECTIONS	
Unstable areas (pit scale)	
Topography	
Before movement	( )
After movement	( )
Plan showing unstable areas	( )
Tension cracks and displacement vect	( )
Geologic cross-sections through slide	( )
Groundwater	
Surface seepage and water levels in drillholes and blastholes	( )
GRAPHS	
Displacement measurements	( )
Bench face angle vs. mine coordinates	( )
TABLES	
Displacement measurements	( )
Bench face angle data	( )
Blasthole data	( )
Production data	( )
Mining Costs	( )
Stockpile capacities	( )
Rock strength data	( )
PHOTOGRAPHS	
Unstable slopes	( )

orientation and spacing, but the length of fractures cannot be directly measured.

10.4.4.3 Rock Properties

Since the spatial variability of rock properties is large, the potential for sampling error is greater than the measurement error. For this reason, it is preferable to use simple test methods for a number of samples than to use an expensive precise method on one sample.

For the shear strength of fractures and fault gouge, the direct shear test is recommended as it is a simulator of field conditions. Since the shear/normal failure curve may be nonlinear, it is important to use normals that represent the expected range of normals for potential failure geometries in the slope. The tests at each normal should be run with sufficient displacement to obtain a residual shear strength, as the residual shear strength usually is a better estimate of in situ strength than the peak strength. To obtain the shear/normal relationship, a curve can be fitted to the shear normal values for a range of normals. Some stability analyses, such as the modified Bishop method of slices, require a linear failure curve of the classic relation,

$$S = c + n\tan\phi \qquad (10.4.1)$$

where ϕ is friction angle, c is cohesion, n is normal stress, and S is shear stress. This is a linear failure curve and is often a good fit, particularly for fault gouge. The more general curve is the power with an intercept, that is

$$S = c + kn^m \qquad (10.4.2)$$

where c, k, and m are constants.

Commonly, fractures have the simple power curve,

$$S = kn^m \qquad (10.4.3)$$

The linear is a special case of the power with intercept where tan (?)

$m = 1$, in which case k becomes tan and c is cohesion. The linear fit to an actual power curve can be an adequate predictor of shear strength except at low and high normals where the curves diverge. When using these strength estimates, it is useful to think of cohesion as a mathematical intercept rather than an intrinsic property of the material.

For intact rock, unconfined compression and Brazilian disc tension tests are recommended. In addition to obtaining the compression and tension strengths, the intact rock shear strength can be approximated by a fit to the tension and compression Mohr circles using the relationships,

$$\phi = \arcsin[(U-T)/(U+T)](0.85) \qquad (10.4.4)$$

$$C = 0.5T\tan\phi(1/\sin\phi + 1)(0.98) \qquad (10.4.5)$$

where U is uniaxial compression, and T is tensile strength.

The constants 0.85 and 0.98 are factors developed from comparison between triaxial testing and the simple linear fit to the uniaxial and disc tension strengths. For most stability analyses, the failure surface is not under high confinement, so triaxial testing is not necessary.

The uniaxial compression tests can be gaged to obtain the Young's modulus and Poisson's ratio for the intact rock.

Index tests such as the point load can also be used to evaluate the spacial variability of intact rock strength.

For the rock mass where direct testing is not possible, indirect methods such as the rock mass rating (RMR) classification and back analysis must be used.

10.4.4.4 Hydrology

Standard hydrologic procedures such as piezometers and pump tests can be used to obtain the current pore pressure distribution and the permeability for predicting changes in pore pressure with time and changes in pit geometry. Simple techniques, such as measuring the water level in drillholes, are effective procedures. Two factors need to be considered, however:

1. Water behavior in rock slopes is a fracture flow phenomenon, and porous media analysis, while useful at a regional scale, may be a poor predictor of pore pressure at pit slope scale.

2. The critical factor in slope design is the pore pressure rather than the quantity of water. A low permeability rock mass may yield very little water and appear "dry," yet have significant pore pressure.

10.4.4.5 Stress Measurements

The most cost-effective stress measurement techniques are overcoring methods such as the "door stopper" or the triaxial gage. Because of the practical limitation of most current overcoring techniques to hole depths of 100 ft (30 m), underground openings are needed to penetrate far enough into the slope to get away from the surface effects. Also overcoring is usually not successful where the rock quality designation (RQD) is less than 50%, which is often the case with deposits such as porphyry copper. Alternates to current overcoring techniques, such as hydrofracturing, have potential where overcoring is not feasible.

It is useful to conduct a finite element analysis using assumed stress to evaluate the mud for in situ stress. determination

Detailed measurement techniques are discussed in Chapter 10.3.

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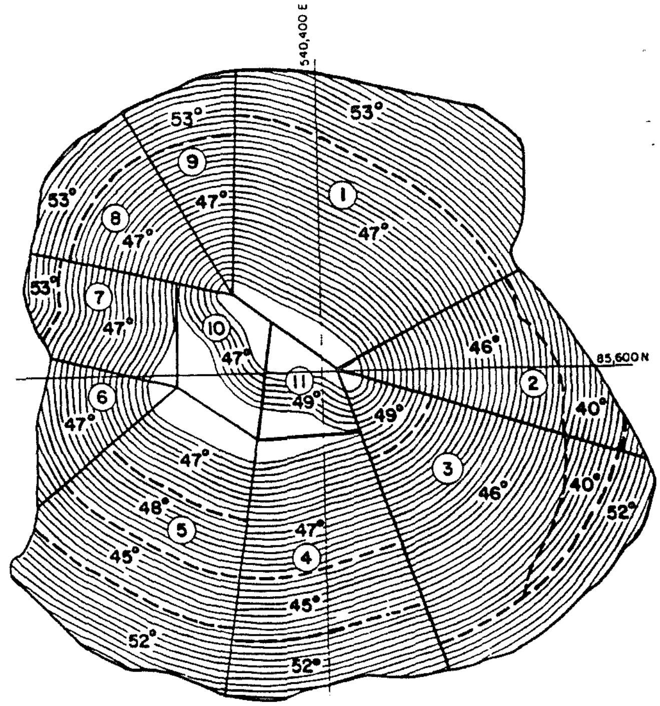


Fig. 10.4.6. Cuajone design sectors and recommended interramp angles.

10.4.5 DESIGN

Steps in slope design are the following:

1. Define design sectors.

2. Conduct a bench design analysis to determine the maximum interramp slope.

3. Conduct interramp design analysis using economic criteria for the selection of interramp angles.

4. Evaluate the resulting overall slope for potential instability, and modify the design if required.

Slope design is an interactive process as a trial pit is required to select design sectors, but the development of a trial pit requires slope angles.

10.4.5.1 Design Sectors

To conduct stability analyses and develop optimum slope angles for input into pit design, the proposed pit must be divided into design sectors that are sections of the pit with similar geologic and operational characteristics (Fig. 10.4.6).

The first criterion for the selection of design sectors is the structural domain, which is an area within which the rock properties and fabric are consistent. Typical structural domain boundaries are lithologic contacts and major structures which separate areas of dissimilar fabric.

The second criterion is wall orientation. Since rock is usually anisotropic, different wall orientations within the same structural domain can have significantly different modes of instability and different optimum angles. An extreme example of this is a dipping coal deposit where slab slides occur in the footwall at a slope angle of 18°, whereas 150-ft (45-m) high benches in the same lithologic sequence in the highwall orientation are stable at 70° with only minor step path raveling.

A third criterion for defining design sectors is operational considerations. Because of the higher cost of slope failure, sections of the pit wall that will contain in-pit crushers, conveyors, or haul roads require different stability criteria than the same wall orientation in the same structural domain.

Since a pit geometry is required to define design sectors, slope design is iterative with mine planning. A preliminary set of slope angles must be provided so that a trial pit can be developed. After the optimum angles are selected and a pit designed, the pit plan must be reevaluated to determine whether the design sectors need to be changed because of changes in the pit geometry.

For each of the design sectors, the rock fabric and major structure orientation data can be plotted on a stereographic projection. This diagram is used to determine failure modes and select structure sets for stability analyses.

10.4.5.2 Bench Design

Bench faces are normally mined as steeply as possible so that some bench-scale rockfalls and raveling can be expected. Thus it is customary, and in many cases mandated by mining regulations, that catch benches be left in the pit wall to retain rockfalls and raveling. Bench design is the process of conducting stability analyses to estimate the minable bench face angles, selecting the bench width, and, to a limited extent, the bench height. The bench height is controlled by the height of the mining levels, but it is possible to increase the height by leaving catch benches on every other level (double benching) or every third level (triple benching).

Based on an analysis of rockfall mechanics, Ritchie (1963) developed width and depth criteria for a ditch at the toe of a slope to protect highways from rockfalls. Falling rocks impact close to the toe of the slope, but, because of horizontal momentum and spin, can roll considerable distances from the toe. The concept of Ritchie's design 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 a berm can be substituted for the ditch. A modification of Ritchie's design that can be used to determine the minimum bench width for bench heights from 30 to 100 ft (9 to 30 m) is

4.5 m + 0.2 bench height (m)

5/B

$$\text{minimum bench width} = 4.5 \text{ ft} + 0.2 \text{ bench height} \quad (10.4.6)$$

or 15ft + 0.2 b.h. (ft)

with a minimum 4-ft (1.2-m) high berm on the edge of the bench.

Recent work with mathematical simulation of rockfalls has indicated that this criterion may be conservative, and the simulation method has the potential for more site-specific bench width criteria (Evans, 1989).

For a given bench height and corresponding bench width, the upper limit of the interramp angle becomes a function of the bench face angle. The bench face angle, however, is not a unique value, as variability of the rock fabric results in varying amounts of backbreak. Backbreak is defined as the distance from the design crest to the as-mined crest. Because of this variability, it is preferable to use a reliability approach rather than using the mean bench face angle. (Calculating an interramp slope using the minimum bench width and the mean bench face angle results in 50% of the benches being too narrow.) The procedure is to select a percentage reliability and use the cumulative frequency distribution of the bench face angle to find the angle where the percentage greater is equal to the reliability (Fig. 10.4.7). This gives the design bench face angle to use, with the minimum bench width and the bench height, to calculate the interramp slope angle.

The percent reliability represents the percentage of the bench along a given level that would be wider than the minimum required bench width to catch rockfalls. The reliability should be selected on the basis of the potential for rock fall and the exposure of personnel and equipment. For example, the catch bench in raveling ground above a haul road requires a greater reliability than catch benches in a stripping area with more competent ground. In practice, reliabilities from 60 to 90% have been satisfactory.

In an operating property, the actual bench faces can be measured, and the measured bench face angle distribution can be used in design. Where existing bench faces are not available, a bench-face angle distribution can be obtained by running a stability analysis of a vertical face. For this analysis, the plane shear, wedge, and step path analyses are run using the fracture data. The height analysis should be incremented in steps up to the bench height, and the resulting backbreak composited, as short fractures that would not result in full bench failure can still cause crest backbreak. This bench face angle distribution is referred to as the theoretical bench face distribution, as the effect of blasting and digging is not included. If there is a strong geologic control such as bedding or foliation, the measured and theoretical bench face angles are the same. Where no strong structure exists, the theoretical bench face angles should be reduced to include the effect of blasting. Based on comparisons that have been made between measured and theoretical angles, the reduction should be between 10 and 20°, depending on the controlled blasting to be used.

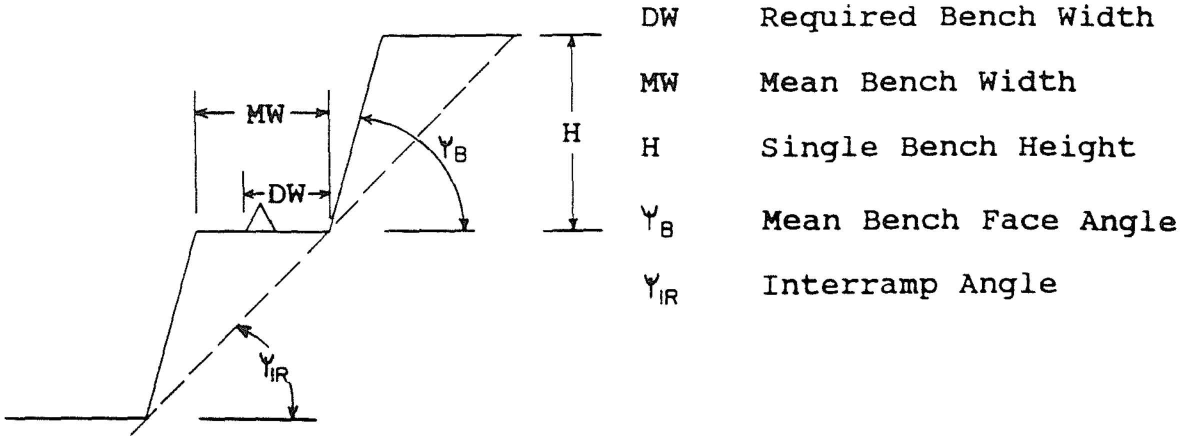
10.4.5.3 Interramp Design

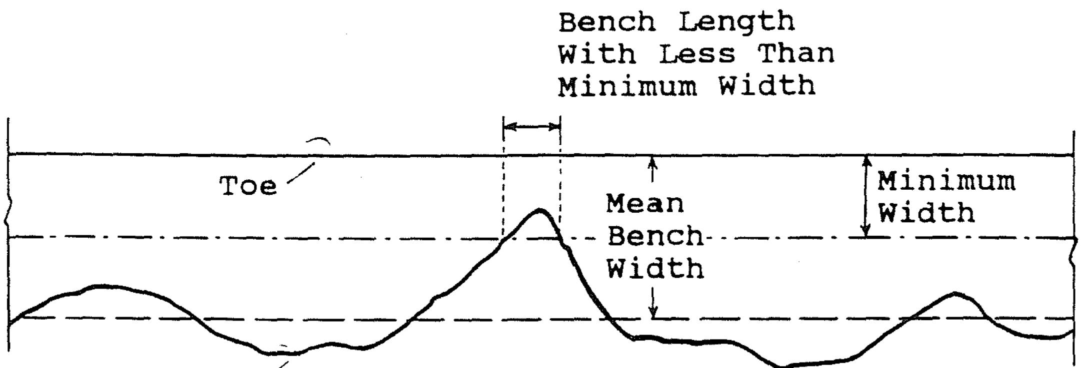
The stability of interramp slopes is primarily controlled by intermediate and major structure failure geometry. Where major structures can be specifically located in space, the geometry relative to the slope can be defined and a discrete stability analysis can be conducted. Commonly, however, the number of mapped structures is large and the distance between the mapping sites and the design wall is greater than the length of the structures. In this case, the structural data must be considered a statistical representation of the structures that will occur in the design slope, and a probabilistic analysis is required.

To obtain the input for stability analysis, the wall orientation can be plotted on a lower hemisphere stereographic plot of the poles of the fractures and the major structures. The fractures and major structures are sorted into design sets based on their orientation relative to the orientations for failure modes, as shown on Fig. 10.4.8; and the distribution of orientation, length, and spacing can be computed for the design set. These design sets may not correspond to geologic sets, although the boundaries of the sets may be adjusted to avoid splitting a geologic set. An advantage of this approach is that it is based on kinematic tests for viable failure geometry and makes it unnecessary to test all the structures for each failure mode.

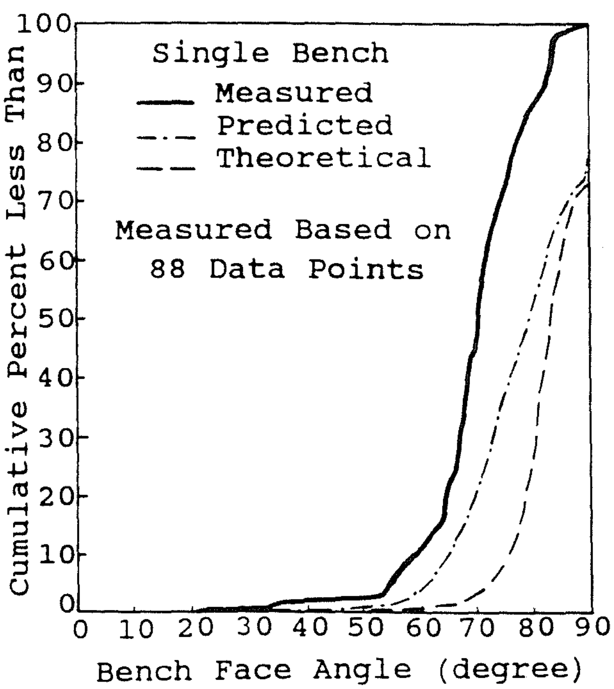
Major Structures: In the case of through-going major structures, where the geometry is known, a safety factor can be calculated for specific slope angles and slope heights using analytical models described in the references for the appropriate failure model. For a deterministic design, the slope angle with the desired safety factor would be selected.

In the reliability method, the probability of sliding can be calculated by Monte Carlo sampling of the shear strength distribution to obtain a distribution of safety factors; and computing the area of the safety factor distribution that is less than one (Fig. 10.4.9). Other techniques can be used, such as the point estimate method (Harr, 1984) or calculating the probability that





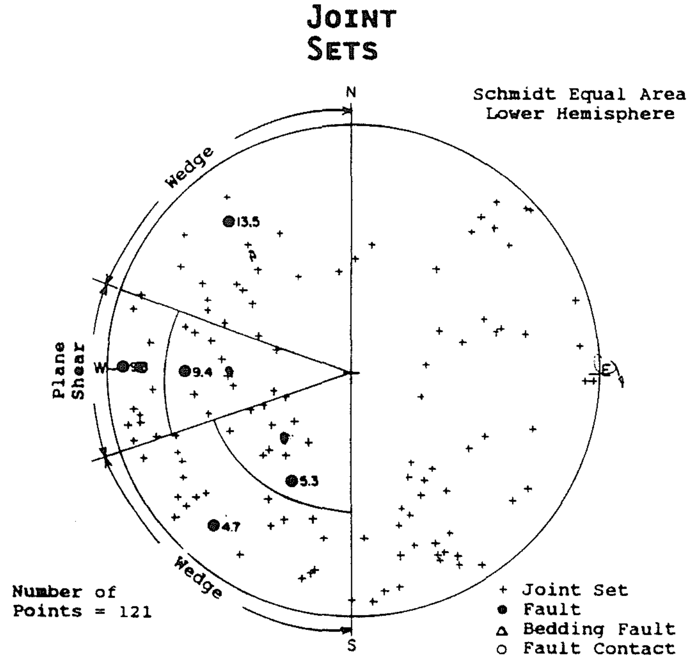
$$\text{Reliability} = \frac{\text{Bench Length Greater Than Minimum Width}}{\text{Total Bench Length}}$$



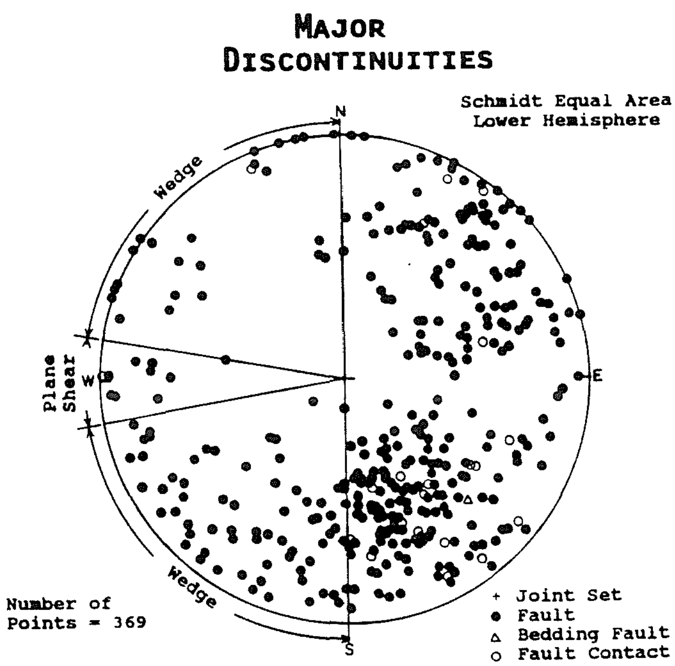
Single Bench Measured Bench Face Angles
SB Mean Face Angle: 69.9
DB Mean Face Angle: 69.9

Single Bench (Ht=50ft)			Double Bench (Ht=100ft)		
Slope Angle deg	% Reliability: Min. Bch Width of 25 ft percent	Mean Width ft	Slope Angle deg	% Reliability: Min. Bch Width of 35 ft percent	Mean Width ft
30	98	68.3	30	98	136.6
31	98	64.9	31	98	129.8
32	98	61.7	32	98	123.4
33	98	58.7	33	98	117.3
34	98	55.8	34	98	111.6
35	98	53.1	35	98	106.2
36	98	50.5	36	98	101.0
37	98	48.0	37	98	96.1
38	98	45.7	38	98	91.4
39	97	43.4	39	98	86.9
40	94	41.3	40	98	82.5
41	91	39.2	41	98	78.4
42	90	37.2	42	98	74.4
43	89	35.3	43	96	70.6
44	87	33.5	44	93	66.9
45	85	31.7	45	91	63.4
46	78	30.0	46	90	59.9
47	71	28.3	47	89	56.6
48	60	26.7	48	88	53.4
49	55	25.1	49	86	50.3
50	40	23.6	50	83	47.3
51	33	22.2	51	78	44.3
52	29	20.7	52	71	41.5
53	24	19.4	53	61	38.7
54	19	18.0	54	55	36.0
55	15	16.7	55	43	33.4
56	13	15.4	56	36	30.8
57	10	14.2	57	32	28.3
58	3	12.9	58	28	25.8
59	1	11.7	59	23	23.4
60	1	10.5	60	19	21.1

Fig. 10.4.7. Catch bench design. Conversion factor: 1 ft = 0.3048 m.



Design Set	Dip Direction Mean (degrees)	Dip Direction S.D. (degrees)	Dip Mean (degrees)	Dip S.D. (degrees)	Length Mean	Length S.D.	Spacing Mean	Spacing S.D.
4.7	38.1	22.3	71.7	10.2	6.7′	4.1′	.54′	.41′
5.3	52.7	13.5	34.6	7.2	4.8′	5.2′	.65′	.55′
9.8	86.1	13.9	79.6	5.6	3.9′	3.3′	.51′	.34′
9.4	91.1	12.9	45.5	11.5	5.2′	3.9′	.49′	.16′
13.5	133.7	20.3	52.6	13.6	6.8′	5.3′	.61′	.49′



Length Mean	Spacing (Mean) LW	RW	PS
233′	417′	1139′	2963′

Conversion factor: 1 ft = 0.3048 m.

Fig. 10.4.8. Design set determination.

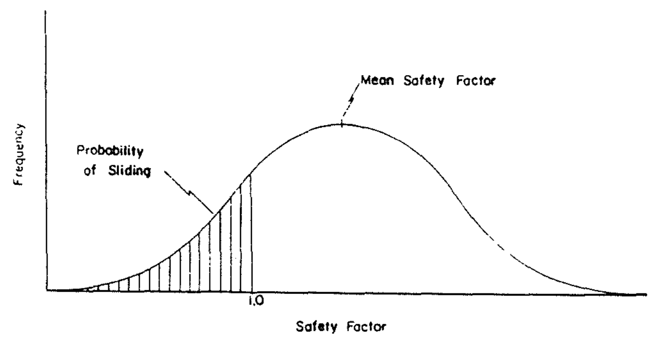


Fig. 10.4.9. Example distribution of safety factors used to calculate probability of sliding.

the shear strength is less that the strength required for a safety factor of one.

Because of the variability of the shear strength, a safety factor greater than one is used to reduce the risk of instability to an acceptable level. One problem with this is that a given safety factor will have a different level of risk depending on the dispersion of the input parameters. The advantage of the reliability approach is that it deals directly with the risk.

Failure Volume Estimation: Where the geologic structures compose a statistical population, the probability of failure for the single occurrence of a specified failure mode is a function of the probability that the structures exist and form a viable failure geometry, as well as the probability of sliding. The probability of existence is calculated from the orientation, length, and spacing of the structures.

To calculate the expected number of failures and the expected failure volume for input to a cost-benefit analysis, the probability of failure for the possible failure modes must be calculated for a range of heights and angles and then composited.

Fig. 10.4.10 is an example of the number of failures and failure volume as a function of slope angle for the design sector of a large pit.

Cost of Failure: Given the expected number of failures and the expected failure tonnage, the cost of slope failure can be estimated. Failure costs consist of cleaning up failure material, repairing haul roads, repair of facilities, lost production due to disruption of operations, the value of lost ore buried by a failure, and engineering costs. The method used to estimate failure cost is a "what if" mine planning procedure. A failure is postulated for a design sector, a plan for responding to the failure is made, and the cost of the plan is estimated. These exercises are useful whether or not a full cost-benefit optimization is done, as they can lead to modifications of the mine plan that will reduce the impact of slope instability.

10.4.5.4 Overall Slope

The overall slope is usually flatter than the interramp slope because of ramps or other step-outs. Thus the overall slope normally will be more stable than the interramp except for stress-induced failure or failure modes not analyzed for the interramp.

Block flow potential can be analyzed by the finite element technique. Finite element has been a time-consuming and expensive analysis in the past. However, with the faster computers and better software currently available, it has become more feasible. A quick check can be made for block flow potential using the charts developed by Coates (1981) (see Fig. 10.4.4). If the charts do not indicate block flow potential with any regional stress assumption, finite element is not needed unless there is a high contrast in stiffness between adjacent materials in the slope. The charts assume a homogenous material and would therefore not indicate stress concentrations produced by stiffness contrasts.

Changes in the overall slope angle have relatively little effect on the stress concentration at the toe of the slope, where a greater concentration could produce block flow. Therefore, block flow potential would not be a suitable method for selecting overall slope angles. A more effective design approach would be to design the slope based on other criteria, and to make provision in the mine plan for step-outs, if needed in the toe area of the pit to reduce the stress concentration produced by the notch effect of the bottom of the slope. The loss of ore from step-outs at the toe would have less economic impact than the amount of stripping required to have the same effect on block flow potential.

Rotational shear analysis should be run for the overall slope, even on rock slopes, to verify that it would not be a critical failure mode. Rotational shear would be a primary method of analysis for both interramp and overall slopes in alluvium and low rock-mass strength slopes such as soft coal measures.

The general surface analysis should be used for the overall slope to evaluate mixed mode failure types where part of the failure is structurally controlled and part is failure of low rock-mass strength. Nondaylighted wedge and plane shear failures in which the weak rock at the toe fails are becoming recognized as a more significant failure mode. This is in part because pits are becoming steeper and deeper, and partly because more pits have been designed for the simpler sliding block failure modes.

10.4.5.5 Slope Support

Ground support techniques such as cable bolting have not had wide application in open pit mining, although the effectiveness is well established in underground mining and in civil construction. One reason is the uncertainty of the ultimate pit geometry. Where the ultimate pit is defined by an economic cutoff, changes in the price of the commodity or in operating costs change the location of the pit slope. It is difficult to justify the expense of cable bolting when there is a reasonable chance that the there will be a new pushback and the bolts will have to be mined out. Corrosion is another problem with bolting, particularly in copper mines where acid mine water is very corrosive. In large pits where the bolt length would have to be in excess of 500 ft (150 m) to include potential failure surfaces, bolting would be difficult and expensive.

A current application of support, employed in Australia in particular, is the bolting of bench faces to reduce ravel and steepen the faces. By reducing raveling, catch benches can be made narrower, which can increase the interramp angle if multibench failure geometries are stable. In some cases, catch benches could be eliminated by using a combination of bolting and meshing to prevent raveling.

There are special situations where bolting is warranted. Even small-scale failures could damage in-pit facilities such as crushers and conveyors, resulting in expensive repair costs and a long period of lost production while the equipment is being repaired or replaced. In this case, bolting to improve the reliability of the slopes that affect the facility would be appropriate. An example of this is the bolting of the haul road in the Ertsberg Pit (Mealey and Nicholas, 1986). The haul road was the only access to ore, and a bench-scale wedge failure of the haul road would stop production. Because of the steep interramp slope, repair of the haul road would have been difficult, so the face below was cable-bolted to increase the reliability of the haul road.

The contribution of a bolt to the shear resistance is composed of three parts:

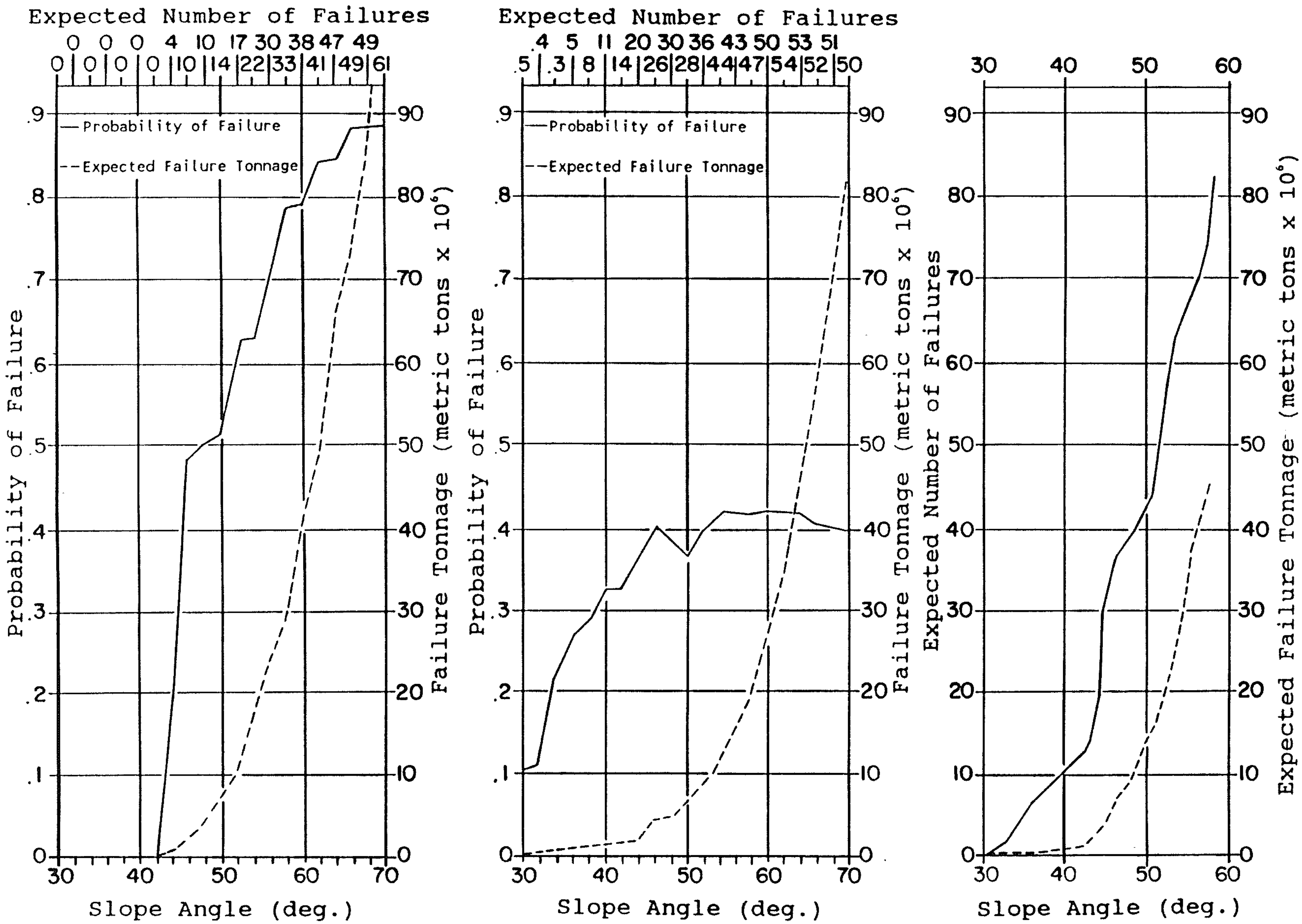


Fig. 10.4.10. Expected failure tonnage curves. Conversion factor: 1 ton = 0.9072 t.

tonnes ?

$$\text{tension} = B\cos\alpha \quad (10.4.7)$$

$$\text{dowel} = 0.5B \quad (10.4.8)$$

$$\text{friction} = B\sin\alpha \tan\phi \quad (10.4.9)$$

where B is the bolt tensile strength, and α is the angle between the bolt and the shear surface. The dowel strength, which is the strength of the bolt in shear, is taken as one-half the tensile strength. Assuming the bolt acts as a Tresca material (Dight, 1982), the tensile and dowel strengths are not mutually exclusive, and the net shearing resistance of the bolt is

$$\text{net resistance} = (\text{tensile}^2 + \text{dowel}^2)^{1/2} + \text{friction} \quad (10.4.10)$$

Untensioned, fully grouted bolts are easier to install and less expensive than tensioned bolts, and there is considerable evidence that a fully grouted bolt reaches full tension with a very small displacement. The argument for tensioned bolts is that displacement required to tension an untensioned bolt may result in loss of the peak shear strength of the rock and may cause cracking of the grout, which would expose the bolt to corrosion. It is doubtful, however, that the benefit of tensioned bolts justifies the additional cost.

Shotcrete has also been used successfully on a small scale to stabilize progressive raveling failure affecting haul roads.

10.4.5.6 Controlled Blasting

Production blasting is designed to fragment rock for loading. At the slope wall, this fragmentation results in backbreak, which reduces the bench face angle and results in flatter slope angles or narrower catch benches. To reduce this backbreak the fragmentation of the final wall must be reduced by controlled blasting.

It has been found that a blast shock wave with a peak particle velocity greater than 25 in./sec (625 mm/s) initiates cracks in rock and produces significant damage above 100 in./sec (2.5 m/s). The peak particle velocity is a function of the charge weight and the distance from the charge. The relationship is

$$V = k(D/W)^b \quad (10.4.11)$$

where V is peak partial velocity, D is distance, and W is instantaneous charge weight (see Chapter 9.2.2). The constants k and b are a function of the rock and the type of blasting, so are site specific. Typical values for open pit blasting are $b = -1.6$ and $k = 26$ to 260 (Oriard, 1982). This relationship can be used to determine the maximum charge weight per delay required to keep the peak particle velocity below 25 in./sec. (625 mm/s).

Reducing the number of holes per delay will reduce the peak particle velocity, but for the perimeter row of holes and the buffer row, a production hole charge is usually too large, and must be reduced. To maintain the same powder factor, the hole spacing must be reduced concurrently with the reduction in hole charge. In practice, this method of controlled blasting has been shown to increase the measured bench face angle by 5° (Savely, 1986).

Presplitting, where a closely spaced line of holes with a light powder charge is shot before the main blast, can produce a smooth face with minimum damage. Presplitting is usually not necessary and is not effective in closely jointed rock.

10.4.6 SLOPE MANAGEMENT

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 the establishment of operational contingencies.

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

1. *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.

2. *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.

3. *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 without warning, the converse is not always the case. Deformation and tension cracks can occur without major displacement.

10.4.6.1 Detection of Instability/Monitoring

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.

The objectives of a pit slope monitoring program should be

1. To maintain safe operational procedures for the protection of personnel and equipment.

2. To provide advance notice of instability so that mine plans can be modified to minimize the impact of slope displacement.

3. To provide geotechnical information for analyzing the slope failure mechanism, for designing appropriate remedial measures, and for conducting future redesign of the slope.

An effective slope monitoring program consists of the systematic detection, measurement, interpretation, and reporting of evidences of slope instability. Measurements are normally made of both surface and subsurface displacement in order to provide an accurate assessment of slope instability (see Chapter 10.3).

Surface Displacement: Surface displacement measurement by means of tension crack mapping, extensometers, and survey points is still the most cost-effective monitoring method. All three procedures should be used as no one method would give the entire picture. Fig. 10.4.11 shows a typical surface monitoring layout.

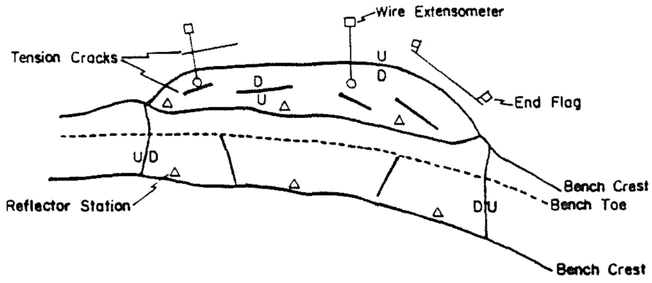


Fig. 10.4.11. Tension crack map.

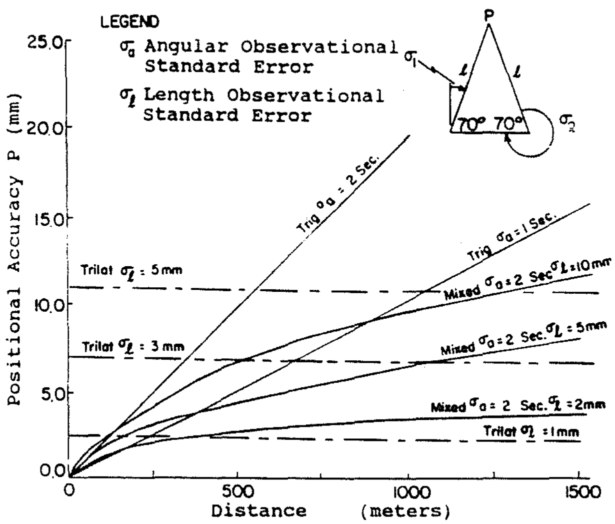


Fig. 10.4.12. Positional accuracy of P by triangulation, trilateration, and triangulateration (Ashkenazi, 1973). Conversion factors: 1 in. = 25.4 mm, 1 ft = 0.3048 m.

1. Tension crack mapping: Tension cracks are an early, obvious indication of instability. By systematically mapping the cracks, the geometry of a failure can be better defined. All cracks should be mapped regardless of apparent cause. Often cracks which appear to be the result of local bench failure or blasting form a pattern showing an impending larger failure when plotted on a pit map.

The ends of the cracks should be flagged or marked so that on subsequent visits new cracks or extensions of existing cracks can be identified.

2. Extensometers: Portable wire extensometers should be used to provide monitoring in areas of active instability across tension cracks. These monitors can be quickly positioned and easily moved. The extensometer should be positioned on stable ground behind the last visible tension crack, and the wire should extend out to the unstable area. For warning devices, or for information on deformation within a sliding mass, wire extensometers can be placed at any strategic location. Anyone working in the area can make an immediate check on slope movement by inspecting the instruments.

3. Survey monitoring: Monitoring prism targets with the geodimeter total station continues to provide the most detailed movement history in terms of displacement directions and rates in the unstable areas. To keep within an accuracy of 0.05 ft (15 mm), the two-second geodimeter should have a maximum range of 5000 ft (1500 m) (Fig. 10.4.12).

Backsights can be taken on other instrument stations or on reference points outside the pit for calibration. In addition to a backsight, each instrument station should have a reference point on stable ground. This reference point is used to check the stability of the instrument station and to calibrate the EDM. Since the displacement measurements are relative, reproducability is often more important than absolute distances.

Subsurface Displacement: Surface displacement measurements do not determine the subsurface extent of instability, although it is possible to make inferences from displacement vectors. There are many situations where measurement of subsurface displacement is needed. These measurements are commonly made utilizing shear strips, borehole inclinometers, and borehole extensometers.

1. Shear strips: Shear strips in a borehole will help to locate the position where the hole is cut off. Either commercial segmented strips or a coaxial cable with a fault finder can be used. These systems have the limitation of being go/no-go devices.

2. Borehole inclinometers: A borehole inclinometer that measures the angular deflection of the hole will give the deformation normal to the hole.

3. Borehole extensometers: Borehole extensometers will give the deformation parallel to the borehole.

Precision, Reliability, and Cost: The number of different devices that can be used for monitoring, as well as the precision and sophistication of the devices, are a function of the ingenuity, time, and budget of the engineer in charge of monitoring. Since none of these factors is infinite, hard choices must be made. are Some general guidelines for decision making follow.

1. Measure the 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 0.4 in. (10 mm) is inexpensive compared to measuring to 0.0004 in. (0.001 mm). A micrometer is unnecessary for monitoring slope movement that has a velocity of 2 in. (50 mm)/day.

4. Redundancy is required: No single device or technique tells the complete story. A single extensometer or survey point cannot indicate the area involved in the instability, and, if it is destroyed, the continuity of the record is lost.

5. Timely reporting is essential: Data collection and analysis must be rapid enough to provide information in time to make decisions.

Monitoring Schedule: A definite monitoring schedule should be established. The frequency of monitoring is a function of the precision of the system, the rate of movement, and how critical the area is. Table 10.4.3 is a suggested schedule. If there is heavy rain or a large blast in the area, additional measurements should be made.

Cooperation between operations and engineering is important. Equipment operators often have an intuitive feel for ground conditions. Any changes in the condition of an area observed by operators should be reported to engineering for followup.

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.

Table 10.4.3. Suggested Monitoring Schedule

Mining	Velocity Ft/Day	Velocity Mm/day	Visual Inspection	Extensometers Extension	Crack Map	Survey[3]	Piezometers
Active	0	0	Daily[1]	—	Monthly	Monthly	Monthly
	< 0.05	< 15	Daily[1]	Daily[2]	Weekly	Monthly	Weekly
	0.05–0.17	15–50	Each Shift[1]	Each Shift[2]	Daily	Weekly	Daily
	0.17–0.30	50–100	2 × Shift	2 × Shift	Daily	Daily	Daily
Inactive	0	0	Monthly	—	Monthly	Quarterly	Monthly
	< 0.05	< 15	Monthly	Monthly	Monthly	Monthly	Monthly
	0.05–0.17	15–50	Daily	Daily[2]	Weekly	Weekly	Weekly
	0.17–0.30	50–100	Daily	Daily[2]	Daily	2 × Week	Daily
	< 0.30	< 100	2 × Day	2 × Day[2]	Daily	2 × Day	Daily

Note:
1. Some mining codes require inspection of working face at beginning of each shift.
2. Extensometers should have warning lights.
3. If extensometers are not installed, survey observations should be on extensometer schedule.

5. Magnitude, bearing, and plunge of resultant (total) 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 monitoring data are graphically displayed. For engineering purposes, the most useful plots are

1. Horizontal position.
2. Vertical position (elevation vs. change in horizontal position, plotted on a section oriented in the mean direction of horizontal displacement).
3. Displacement vectors.
4. Cumulative total displacement vs. time.
5. Incremental total displacement rate (velocity, usually in ft or m/day) vs. time.

All graphics should be kept up-to-date and should be easily reproducible, for ease of distribution. By studying several graphs simultaneously, the movement history of a particular slope can be determined.

Precipitation data should also be recorded in order to evaluate possible correlations with slope displacement. A gage located at the mine site can be used to measure occurrences and amounts of precipitation. In addition, measurement of the average daily temperatures will provide some indication of freeze and thaw periods.

The location of mining areas and the number of tons mined should also be recorded on a regular basis, because slope displacements are often associated with specific mining activity. One method of cataloging this information is to plot the mining area and then note the number of tons mined and the date on a plan map of the pit. A histogram can be made of tons mined vs. time, and this plot can then be compared to the total displacement graphs.

A formal monthly slope stability report should be prepared, containing the data listed in Table 10.4.4 and recommendations on the appropriate response to current instability. This ensures that mine management receives the appropriate information and provides the discipline to document slope behavior. Direct informal communication should also be maintained with pit operations on a daily basis.

Table 10.4.4. Monitoring Data Presentation

Graphs
Cumulative Displacement vs. Time
Velocity vs. Time (ft or m/day, semi-log plot)
Precipitation vs. Time
Water Levels vs. Time
Mining vs. Time
Maps and Sections
Pit Map with Location of Unstable Areas
Location of Monitoring Points with Displacement Vectors
Tension Crack Map
Horizontal Plot of Location with Time
Vertical Plot of Location with Time
Map of Piezometric Surface
Cross Section of Unstable Area

10.4.6.2 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 comparisons conducted. The following is a list of 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 trade-off between the value of lost ore and the cost of cleanup.

5. Partial clean-up 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.

10.4.6.3 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.

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