CHAPTER 4

# Managing and Analyzing Overall Pit Slopes

**Richard D. Call,* Paul F. Cicchini,* Thomas M. Ryan,* and Ross C. Barkley***

## 4.1 INTRODUCTION

Many large porphyry copper mines, such as the Chuquicamata Mine and the Bingham Canyon Mine, have slopes in excess of 600 m. Several of these mines have final mine plans that include sectors with overall slope heights of more than 1,200 m. Because of their large size and the corresponding large stripping volumes, there is often an economic incentive to design working pit slopes near the optimum overall slope angle. However, experience has shown that a significant amount of slope displacement can be expected at economically optimum slope angles. Because of their size, most large open pits have enough operational flexibility to tolerate considerable slope displacement, provided the slope "failures" can be adequately modeled and predictions regarding their size and behavior can be made.

The analytical tools used to predict the expected number and volume of multibench, structurally controlled failures (wedge, step-path, plane shear, and step-wedge) are well developed. Probabilistic models have been developed to estimate the number, size, and displacement resulting from structurally controlled rock-slope failures (CANMET 1977). However, overall slope displacements related to yield in weak rock masses require more complicated numerical models to estimate the degree of instability. These models should consist of accurate characterizations of the rock units, the major-structure locations, the rock-mass strengths and elastic properties, the hydrologic conditions, and the in situ stress. If reliable and predictive rock-mechanics models of strength, stress, and displacement can be constructed for the overall pit slopes, mine plans and operational procedures capable of tolerating overall slope instability can be developed, thus (1) improving operational safety, (2) increasing production efficiency, (3) improving mine economics, and (4) extending the mine life.

The geotechnical analysis of large-scale overall slope instability involves these primary areas of study:

- Slope monitoring and kinematic displacement modeling
- Geological, geotechnical, and hydrological analysis
- Stress, displacement, and stability modeling

## 4.2 FAILURE CLASSIFICATION

Post-failure models are primarily based on empirical relationships derived from slope-monitoring data (Cruden and Mazoumzadeh 1987; Voight, Orkan, and Young 1989). Examination of displacement data led to the development of two general displacement models: a progressive failure model, for which slope displacement will continue to accelerate to a point of collapse (or greatly accelerated movement), and a regressive failure model, for which the slope will decelerate and stabilize. Zavodni and Broadbent (1982) defined regressive and progressive failure stages for several large-scale open-pit slope failures and related these stages to failure geometry. Savely and Call (1991) expanded this work into a useful description of failure characteristics and expected regressive or progressive slope behavior (Figure 4.1).

A progressive slope becomes less stable with time and can exhibit sudden, large movements. Ryan and Call (1992) reported a wide range of precollapse velocities for progressive slope failures. Savely and Kastner (1982) and Kennedy and Niermeyer (1970) discuss operational and monitoring procedures for minimizing the impact of progressive slope failure on mining.

Regressive slope failures often occur as a result of rock mass yielding near the toe of the slope. In the United States, case histories of regressive slope failures in open-pit mining have been documented in the literature since the early 1950s (Bisbee, Arizona, and Butte, Montana). Many rock-mass movements in high open-pit slopes decelerate with time, provided the ratio of driving to resisting forces decreases with displacement. Surface and subsurface monitoring data from several slope failures display displacements that are extremely variable in direction and in magnitude.

Slope displacements, whether progressive or regressive, must be monitored carefully to ensure operational safety and to determine the impact on slope stability of changes to the mine design and operations.

The remainder of this chapter discusses the characteristics, behavior, and analysis of regressive, overall pit-slope failures that tend to stabilize with displacement and time. Geotechnical analysis and experience with several regressive slope failures demonstrate that mining can continue in areas of large-scale displacement, provided the failure is regressive in character, the failure mechanism is well defined, monitoring procedures are established and enforced, and an effective slope-management program is in place.

### 4.2.1 A Displacement Model of Regressive Slope Failures

Slope-monitoring data consistently indicates that most regressive slope failures occur in response to mining activity. A typical response is shown in Figure 4.2.

As mining proceeds along a level within a zone that is in active yield, confining stresses are reduced, excess strain energy is induced, and displacement of the rock face is produced. These slope displacements can be either elastic or plastic, depending on the state of stress and the strength of the nearby field rock mass. Monitoring data often demonstrate that slope displacement initially develops in lower-strength, highly fractured rocks near the toe of an active mine slope. Toe displacement directions are typically at very low angles (5° to 20° from horizontal). Displacements higher in the slope do not develop instantaneously with displacement at the toe and can have highly variable directions. Since the slope is not acting as a rigid block, displacements will tend to develop in a time-dependent manner, resulting in differing rates of

* Call & Nicholas, Inc., Tucson, Arizona.

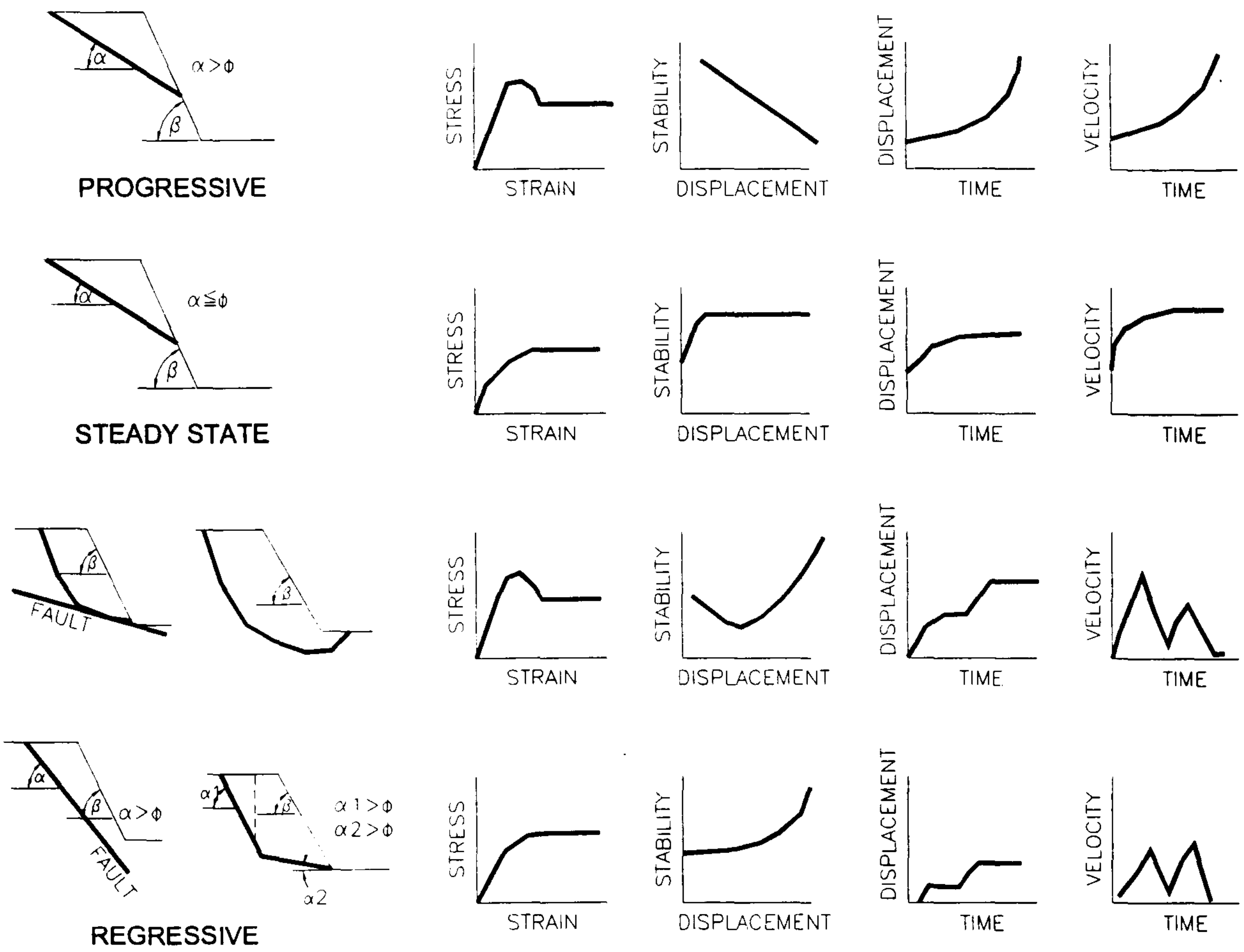


**FIGURE 4.1 Classification of progressive and regressive failure**

movement from point to point within the slope (Figure 4.3). This time dependency can also be demonstrated in regard to surface and subsurface displacement.

## 4.3 GEOLOGICAL CHARACTERISTICS OF LARGE-SCALE REGRESSIVE SLOPE MOVEMENTS

The following geologic characteristics are common to documented, large-scale regressive slope failures (Figure 4.4):

- Low rock-mass strength in the toe
- A ubiquitous joint set that dips into the pit
- High-angle faults or continuous joints that form back and side releases for the slope movement
- Saturated toe, excess hydraulic gradients, and compartmentalized groundwater conditions
- High in situ horizontal stresses

### 4.3.1 Low-Strength Rock Mass

Unfortunately, thick zones of low-quality rock are associated with the processes of ore emplacement. Regional-scale tectonics, contact metamorphism, and the release of volatiles from the magmatic melt all contribute to the degradation of the rock within and surrounding porphyry deposits.

In slopes that have experienced large-scale regressive displacements, the zones of low rock-mass strength generally exceed 50 m in thickness; these zones have a rock-mass rating (RMR) value of less than 40 and a rock-quality designation (RQD) of less than 30. Rock-mass characteristics rar[illegible]m low RQD zones with clay-filled fractures and moderate wall-rock alteration to pervasively clay-altered zones in which the original rock has been obliterated by mechanical and hydrothermal actions.

Varying degrees of clay alteration are present in zones of low rock-mass strength, as well as in the bordering transition zones. In the low rock-mass strength zone, clay alteration varies from a pervasive fracture filling with weak to moderate wall-rock alteration to complete alteration of the rock to clay by mechanical and hydrothermal processes.

Clay alteration extends beyond the low rock-mass-strength zone but is usually confined to larger structures. In some cases, a transition occurs between weak clay-altered rock and fresh rock. Transition zones are characterized by clay-filled fractures and faults; however, the wall-rock alteration is diminished and the RQD improves.

Clay adversely affects the stability of slopes for two reasons. First, clay alteration of intact rock and clay infilling of fractures reduces rock-mass strength. Second, clay impedes the natural or induced drainage of the slope, resulting in higher water pressures within the rock mass.

### 4.3.2 Ubiquitous Joint Set

Ubiquitous joints that dip toward the pit at 35° to 45° are a common characteristic of these regressive failures. In general, such structures do not daylight in the overall slope but provide a plane of weakness within the slope.

The principal stress aligns with the dip direction of the ubiquitous joint as mining progresses (Amadei, Savage, and Swolfs

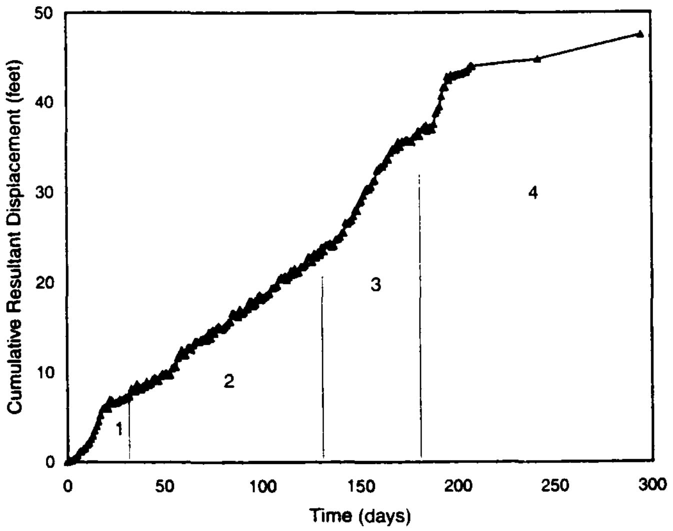


**FIGURE 4.2 Four regressive slope displacement cycles displayed on a graph of cumulative resultant displacement**

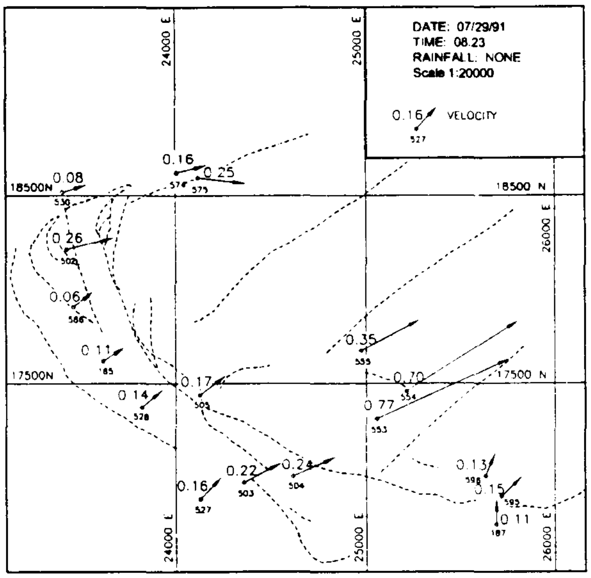


**FIGURE 4.3 Measured slope velocities of a large-scale slope failure (Note the high velocity measured directly above the active mine bench at the 2,680 level.)**

1987). This stress alignment enhances slope movement along the ubiquitous joints and may extend the joints.

### 4.3.3 High-Angle Back and Side Releases Formed by Faults

High-angle faults that strike parallel to subparallel to the trend of the pit wall form a back release for downward slope movement. These structures diminish the rigidity of the rock mass, form discrete kinematic blocks, and compartmentalize groundwater. In addition, because of the large contrast in rigidity between the fault structures and the surrounding rock, the maximum principal stress aligns parallel to these structures on the upslope blocks.

Faults or continuous joints, which strike from perpendicular to oblique to the trend of the pit wall, provide side releases for the slope movement.

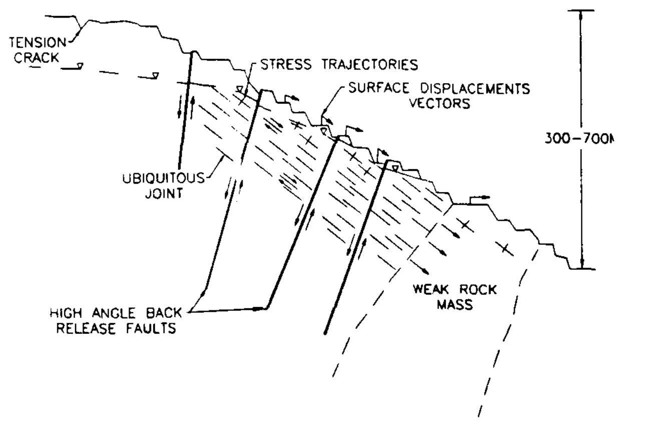


**FIGURE 4.4 Geologic conditions observed in large-scale regressive slope failures**

### 4.3.4 Groundwater Conditions

Groundwater causes a destabilizing effect in these large-scale slope failures. In a free-draining slope, the hydraulic gradients are generally low, as evidenced by a gentle drawdown cone and a dry slope. However, clay-filled faults and fractures greatly reduce the permeability of the rock mass comprising the slope. Excess pressure gradients can occur, because the clay inhibits free drainage when the stress state in the rock is changed by mining. In these cases, the drawdown cone is very steep, resulting in groundwater close to the pit face. This pressure creates a significant buoyant force on potential failure surfaces, which reduces the ability of the slope to resist shear. Driving forces increase further if the water pressure acts along a vertical back release.

Slope movements are quite sensitive to changes in water pressure. The onset of slope failure dilates the rock mass, creating additional storage. This added storage results in a lowering of the water level and a temporary increase in stability. In very dry regions, the lack of recharge may stabilize the slope or may cause diminished slope movements for several pushback cycles. However, in areas of high recharge, the increase in stability is short-lived, and mine personnel should continue to monitor the slopes for increased pore pressure.

### 4.3.5 Horizontal Stress

When designing overall pit slopes, high horizontal stresses related to regional tectonics should be evaluated. As mining progresses, high horizontal stresses accentuate the deviatoric stress in the rock mass near the toe of the slope. If the rock mass is not strong enough to withstand the deviatoric stress, it will yield. Higher horizontal stresses will cause more rock-mass yielding, leading to larger slope displacements.

## 4.4 FAILURE MODELING AND STABILITY ANALYSIS

The geologic characteristics listed in this chapter often combine to cause the large-scale regressive slope failures observed. The following is a suggested sequence of events that explains the slope displacements:

1. Initiation of movement by active mining in the weak toe rock.
2. Propagation of movement upslope due to successive release of kinematic blocks.
3. Deceleration to a more stable state.

### 4.4.1 Initiation of Movement

Mining through low-strength rock in the toe reduces the confining stress and creates an overstressed condition. Excess stress is relieved by movement in the toe of the slope and by stress transfer into rock that can carry additional load. The maximum principal stress is commonly horizontal at the toe, resulting in strong horizontal displacement.

Due to the pervasive clay fillings and wall-rock alteration in the weak toe rock, natural or induced drainage is limited. Pore-water pressures in this low-permeability saturated rock can be high due to stress redistribution. The high pore-water pressure reduces the ability of this weak rock mass to resist shear. In some cases, this material becomes amenable to drainage as the displacing rock mass dilates.

### 4.4.2 Propagation of Movement Upslope

Displacements propagate upslope as confinement is reduced in successive upslope kinematic blocks. Upslope movements occur parallel to the non-daylighted joint sets and along high-angle release faults.

A common characteristic of these failures is a differential shear along the high-angle faults that form the back releases of identifiable kinematic blocks. Along a particular fault structure, the upslope block drops relative to the downslope block (Figure 4.4). This differential movement results from a significant vertical stress gradient across the faults. Due to the low strength and modulus of the fault structure, the maximum principal stresses on the upslope block align with the fault. This, coupled with the relief of lateral confinement due to downslope displacements, can result in a stress condition analogous to active earth pressure. The additive effects of the deep shear often result in the observed differential displacement along the release faults. Field observation of this post-deformation expression along high-angle faults has misled many investigators into referring to this failure mechanism as deep-seated toppling. As such, it implies that deep-seated toppling of these upslope blocks initiates the slope failure when, in fact, the toppling of the blocks is a secondary effect due to shear deformation initiating in the toe of the slope.

The high-angle release faults that form the kinematic blocks also tend to compartmentalize groundwater. Groundwater lowers the resistance of the rock mass to shear, resulting in a deeper zone of shearing along ubiquitous joint sets. In addition, water pressure acting on the backplanes increases the driving forces.

Lowering pore-water pressure in the upslope kinematic blocks greatly reduces overall slope displacements by reducing the driving forces and increasing the shear resistance. Additionally, stabilizing the upslope blocks imparts less stress on the weak toe material. Installation of dewatering systems usually pays dividends because slightly steeper slopes can be achieved and delays in mining operations caused by a moving slope can be reduced.

### 4.4.3 Reduction of Slope Movement

Slope displacements decrease as a result of the following:

- Halting or lowering the production rate in the weak toe
- Lowering of the pore water pressure due to dilation of the displacing rock mass
- Displacing to a more stable geometry
- Unloading the active block of the failure

### 4.4.4 Stability Modeling

Simple rigid-block models are not appropriate for analyzing rock-slope stability when the primary mechanism of failure is plastic (toe) to pseudoplastic (shear along ubiquitous joints) yielding of the rock mass. Limit-equilibrium methods have limited application because they cannot predict either strain or the extent of slope deformation. For these reasons, the combination of discrete element and continuum numerical methods is finding widespread application for the overall stability analysis of these high, open-pit slopes. The sophistication and efficiency of these models has improved rapidly.

When using these numerical methods, it is critical to understand the mechanisms producing the slope failure, the strength characteristics of the rock mass, and the hydrology of the pit. If modeling is performed for a failing slope, it is refined to correspond to the historical slope displacements and the measured in situ stresses.

Parameters that influence the model response include in situ stress, rock-mass strength, elastic properties, structure, and hydrology. Because displacements are often not elastic, superposition is not valid, and a careful simulation of historical mining is required for stability analysis. A historical match between the model and the slope often can be obtained through several stress paths. Therefore, careful reasonability checks on (1) the loading and unloading of the model; (2) the geomechanical, geological, and hydrological parameters used in the model; and (3) the displacement, stress, strain, and constitutive state at each model step.

It is also important to understand the limitations and assumptions inherent to the particular numerical method selected. A continuum model may be suitable for the back-analysis of an existing failure but its usefulness in predicting the stability of future mining geometries may be severely limited. In a jointed rock mass, both the deformation characteristics of the rock mass and the ability of the rock to transfer and relieve stress are influenced by joint slip. Appropriate deformation characteristics of the rock mass can be simulated in a continuum model by reductions to the intact rock modulus of deformation. However, stress relief due to mining cannot be properly simulated since the effects of stress transfer due to joint slip cannot be accounted for in a continuum. This limitation becomes particularly acute when a large volume of material must be excavated, such as in the case of mining from a premine topography (known stress state) to an ultimate pit geometry (simulated stress state).

Another important issue to consider when setting up a numerical model is the manner in which the in situ horizontal state of stress will be initialized. The state of stress can be initialized through application and adjustments to a distributed load applied to the boundary of the model or through direct initialization of a stress state within each element. In the latter case, the model will have a zero displacement boundary, and in the former, it will not. Although identical initial stress states can be created with either approach, the induced stresses from the simulation of future mining cuts will be very different. The boundary-loaded model simulates an active, driving horizontal force, creating much greater stress concentrations around the vicinity of the excavation and more adverse stability conditions than those resulting from the zero displacement boundary conditions. Therefore, the user must have a clear understanding of the stress environment to select the appropriate boundary conditions. In the absence of strong evidence to support an active stress regime, zero displacement boundary conditions will generally be assumed.

A large geomechanical database must be developed to define the parameters used in the model. This database should include the following:

1. Rock types observed from surface and subsurface geologic mapping
2. Block sizes
   a. Cell mapping
   b. RQD logging of core
   c. Surface classification mapping of weak rock masses
3. Intact shear strength
   a. Uniaxial compression tests
   b. Brazilian disk tension tests
   c. Triaxial compression tests
   d. Point load tests

4. Fracture shear strength
   a. Small-scale direct-shear tests of core
   b. Large-scale direct-shear tests of rock blocks
5. Orientation of geologic structure
   a. Rock fabric mapping
   b. Oriented coring
   c. Geologic mapping of major structures
6. Hydrological characteristics
   a. Geologic mapping and drilling to define character of water-bearing rock
   b. Measuring water levels in exploration and geomechanical drill holes
   c. Reporting wet blastholes
   d. Reporting seeps observed on the slope
   e. Slug testing of exploration and geomechanical holes to provide a rough estimate of transmissivity
   f. Performing pump tests with observation holes for definition of transmissivity and storage in more homogeneous aquifers
7. In situ stress measurements

Rock-mass strength and elastic properties and hydrologic parameters are defined from the data mentioned in items 1 through 7. These data are used to zone the rock into discrete domains that possess similar engineering characteristics. Quantitative methods have been developed to classify rock masses according to rock type, block-size distribution, intact rock shear strength, fracture shear strength, orientation of the geologic structures, pore-water pressure, and in situ stresses. The RMR (Bieniawski 1993) and Q-system (Barton, Lien, and Lunde 1974) rock-mass classifications have been used extensively to choose appropriate underground mining methods and to estimate support requirements for underground openings, settlements in rock-mass foundations, and the rock-mass shear strength.

Of particular importance in constructing numerical models of pit slopes is the definition of the rock-mass shear strength and the deformation modulus. Several empirical classification schemes have been developed to estimate the rock-mass strength and the elastic properties. The majority of these methods rely on assessments of the RMR. Experience with numerical slope modeling has shown that the rock-mass strength and elastic properties derived from these classification schemes do not always provide model responses that match the slope performance or the measured slope displacements. An alternative method has been developed that relates the strength of the rock mass directly to the degree of fracturing present, through a combination of the intact rock strength and the natural fracture strength as a function of RQD.

To determine the modulus of deformation for the rock mass, Bieniawski (1978) proposed the following relationship for the correlation between RQD and the ratio of the rock-mass modulus to the intact rock modulus:

$$r = E_m/E_i \qquad \textbf{EQ. 4.1}$$

Where:

$E_m$ = rock-mass deformation modulus
$E_i$ = intact rock deformation modulus

$$r = \alpha e^{\beta(\%\mathrm{RQD})} \qquad \textbf{EQ. 4.2}$$

For:

$\alpha = 0.225$
$\beta = 0.013$

Deere and Miller (1966) demonstrated that the elastic modulus for intact rock can be related to the intact compressive strength and defined a narrow range of observed ratios between elastic modulus and compressive strength for [illegible]ttle and soft materials. Consequently, it seems reasonable to expect that a similar relationship may exist between the rock-mass modulus and the rock-mass strength. Back-analysis of slope failures indicated that the estimation of rock-mass strength does follow Bieniawski's relationship for predicting deformation modulus; however, the strength properties were found to vary according to the square of the modulus ratio ($r^2$). For example, if the square of the modulus ratio ($r^2$) is 0.3, the estimated rock-mass strength is derived by compositing 30% of the intact rock strength with 70% of the natural fracture strength. The resulting equations for predicting the rock-mass friction angle and cohesion are

For RQD > 50% to 60%:

$$C_m = \gamma[r^2 c_i + (1 - r^2)c_j] \qquad \textbf{EQ. 4.3}$$

$$\phi_m = \tan^{-1}[r^2 \tan(\phi_i) + (1 - r^2)\tan\phi_j] \qquad \textbf{EQ. 4.4}$$

Where:

$\phi_m$ = rock-mass friction angle
$C_m$ = rock-mass cohesion
$\phi_i$ = intact rock friction angle
$c_i$ = intact rock cohesion
$\phi_j$ = joint friction angle
$c_j$ = joint cohesion

And:

$\gamma = 0.5$ to 1.0
$\gamma = 0.5$, jointed medium to strong rock (>60 Mpa)
$\gamma = 1.0$, massive weak to very weak rock (<15 Mpa)

For simplicity, the rock-mass strength equations have been presented for a linear Mohr–Coulomb failure envelope. The rock-mass shear strength can be mapped to a power envelope by regression techniques using the calculated percentage of intact rock ($r^2$ * 100) and the power strength envelopes of both the intact rock and the fracture shear data.

The intact compressive strength exerts the primary control on the constant gamma ($\gamma$) in Eq. 4.3. However, the appropriate gamma ($\gamma$) value is also influenced by the degree of fracturing. In general, the gamma ($\gamma$) value increases as the intact compressive strength decreases, and as the fracture intensity becomes greater, the gamma ($\gamma$) value lessens.

Subsequent application of these equations indicated that for RQD values less than approximately 50%, Eq. 4.4 tended to overpredict the rock-mass friction angle. Consequently, the constants alpha ($\alpha$) and beta ($\beta$) in Eq. 4.2 were revised to provide a better fit to back-calculated rock-mass friction angles for lower RQD rock masses.

For RQD < 40 to 50% (estimation of rock-mass friction angles only),

$\alpha = 0.475$
$\beta = 0.007$

The two relationships presented for predicting rock-mass friction angle do not follow a smooth transition between 40 and 60% RQD. Modifications to the equations in this RQD range are currently being investigated, and the authors hope to publish the results of this work in the near future.

Because the above relationships for predicting rock-mass strength are based on RQD, it is important to recognize that RQD can be an imprecise indicator of the degree of fracturing at RQD values below approximately 20% and above approximately 80%. To overcome this deficiency, a relationship based on fracture frequency is preferable. However, existing mine databases typically either lack these data or the information is very limited. Fortunately, this situation is rapidly changing as greater focus is being placed on the compilation of a comprehensive geomechanical database during exploration drilling. Once more extensive databases of fracture frequency are available, these relationships can be readily converted and extended to find wider applicability in strongly fractured, as well as in massive, rock units.

Empirical-based methods for predicting rock-mass properties have been widely used and often successfully applied in the field of rock mechanics. However, more rigorous methods for determining rock-mass strength characteristics are still needed. The use of three-dimensional numerical methods to model the mechanical response of the rock mass to various loading conditions shows some promise. To be effective, these models must accurately portray the intact rock properties, the orientation of stress fields, the orientation and spacing of discontinuities, and the strength and elastic properties of the discontinuities and/or discontinuity fillings.

Although numerical methods have proven valuable in defining the failure mechanism of these large-scale slope failures and in predicting the response of the slope to future mine plans, they are not adequate for modeling the combined responses of mechanical loading, deformation, and water flow. The large size of these slope models places a constraint on the number of sensitivity analyses for various static water conditions that can be performed in a reasonable period. Future versions of the numerical methods must efficiently represent the mechanical/hydrological interaction of large pit models. The following topics should be addressed by updated numerical methods:

- Consolidation
- Efficient establishment of hydraulic gradients in complex hydrological conditions
- Change in hydraulic gradients resulting from rock-mass dilation during failure

## 4.5 DISPLACEMENT RATES AND OVERALL SLOPE ANGLE

A range of overall slope angles can be excavated in regressive slope failures. Steeper slopes experience greater velocities and larger overall displacements. However, there is a maximum limit to the angle that can be excavated before a progressive accelerating slope failure will occur. Steepening the upslope geometry beyond this critical angle results in driving forces that are greater than the resisting forces in the upslope kinematic blocks. These upslope blocks can drive the toe to an accelerated failure condition. A fundamental role of stability modeling is to determine the critical slope angle and identify whether this accelerated condition is beginning to develop. Figure 4.5, a plot of model velocity histories for three slope geometries, illustrates the use of modeling in defining the critical slope angle.

## 4.6 OPERATIONAL PROCEDURES TO MINIMIZE THE IMPACT OF SLOPE DISPLACEMENT ON MINING

Because of the operational flexibility in most large open-pit mines, slope displacement does not necessarily constitute "failure" from the standpoint of mine management. This relationship between theoretical and operational slope failure in mining has been discussed (Munn 1985). In particular, the real hazard to mine operations is often the potential for greatly accelerated movement occurring near equipment and personnel. If this can be mitigated, a significant amount of slope displacement can often be tolerated with routine mine operations such as dozing and additional shovel shifts for cleanup. Mining in areas of large-scale slope instability generally results in unsteady production. Provided a mine slope failure is regressive in character, slope displacement can be controlled using specific operational procedures: dewatering, additional stripping, control of the excavation geometry, and control of the excavation rate. The effect of these controlling measures can be assessed and predicted with geotechnical analysis.

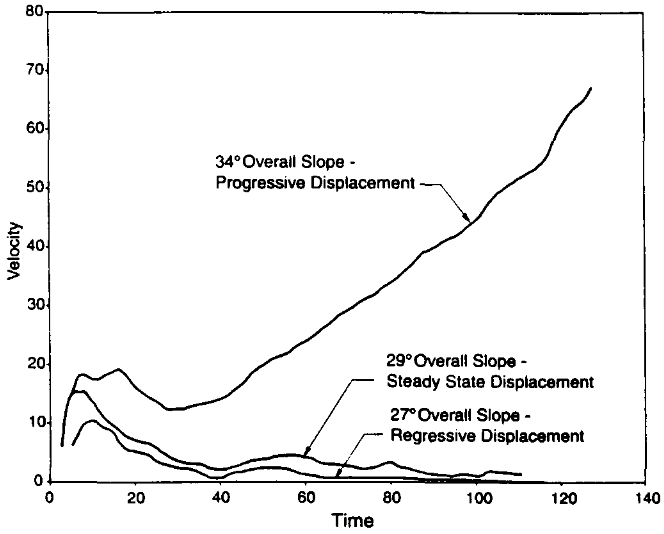


**FIGURE 4.5 Velocity histories for three modeled overall slope geometries**

### 4.6.1 Stepout Versus Unload

At Twin Buttes during the past five years, it has been demonstrated that periodic, small (less than 20 ft) stepouts into the pit could effectively decrease slope displacement in an area of historical slope failure (Ness 1992). There is often a trade-off between the cost of cleanup or additional stripping and the value of ore lost due to a stepout. There also may be an optimum location for a stepout in a pushback, and this can be defined by the geotechnical engineer through a stability analysis.

### 4.6.2 Controlling Excavation Rates

Controlling the mining rate is another way to maintain displacement velocity to minimize its impact on mine operations. This technique is difficult to implement and requires significant flexibility from mine operations. Once displacement velocity of a slope region reaches a limit defined by the rock mechanics staff (through analysis and experience), mining is discontinued until an acceptable relaxation limit is achieved. Savely (1993) discusses how this approach is optimized by maximizing operational efficiency in relation to the moving slope area. At several properties, short mining periods and shorter delays have been found to be more cost-effective than long mining periods and longer delays. This can also be demonstrated mathematically with displacement modeling (Figure 4.6).

Since the vast majority of accelerating and decelerating displacement curves for rock slopes approximate either an exponential or a power function, the relationship between the time spent in excavation and the time required for relaxation is not linear. The higher displacement rates associated with longer mining periods require a substantially greater proportion of time for deceleration. Stability analysis and slope-monitoring data can be used to assess optimum extraction rates.

### 4.6.3 Pushback Width

Optimum pushback widths can be defined from a geotechnical perspective as well as a mining perspective. Practical mining experience suggests that narrow pushbacks within failed slopes are difficult to maintain. This can also be demonstrated numerically with detailed stress and energy analyses of rock slopes. The role of the geotechnical engineer should be to determine whether there is an advantage in changing the pushback geometry because of either an existing slope instability or the location of a major geologic structure.

Stability analysis of overall slope failures using continuum models demonstrates that stress concentration occurs at the toe

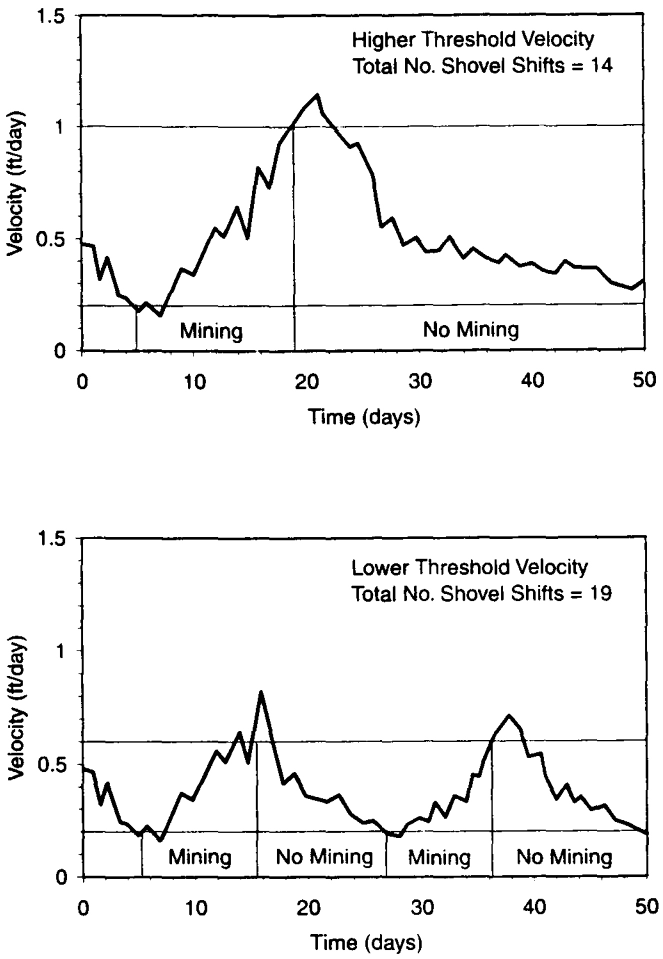


**FIGURE 4.6 Optimizing production in a displacing slope (Note the lower "no mining" threshold velocity of 0.6 ft/d allows 19 shovel shifts, while the higher "no mining" threshold velocity of 1.0 ft/d allows 14 shovel shifts.)**

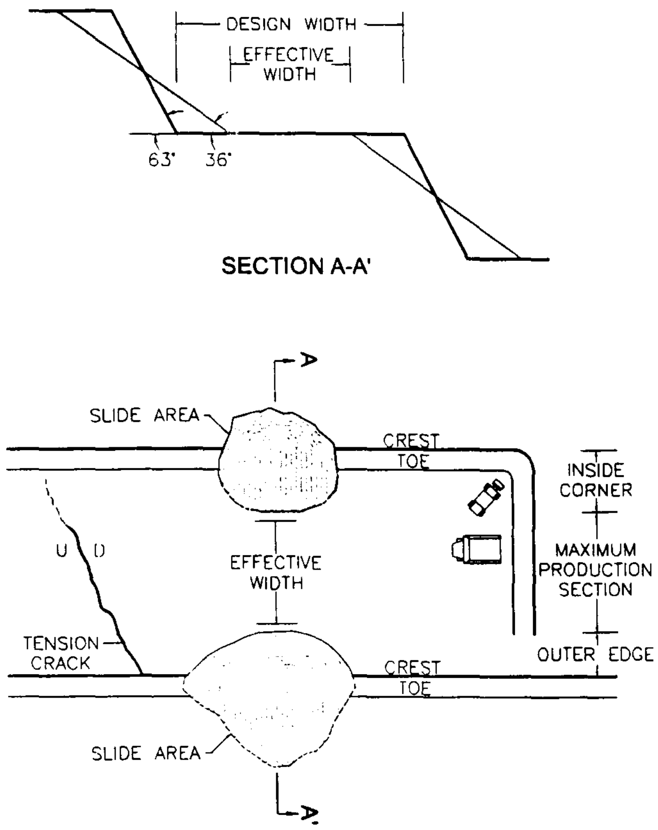


**FIGURE 4.7 Operational considerations for pushback width**

of slopes as they are excavated; this has been validated by in situ stress measurements at several mines. In the case of the excavation occurring within a plastic zone, a change in the stress and energy state occurs, which results in slope displacement. The stress path for excavation is a combination of lateral extension and axial compression. Strain energy for underground excavations has been analyzed in detail by others (Salamon 1984; Farmer 1986), and similar methods can be applied to pit slopes that are at yield. In general, smaller excavations result in less energy changes and less displacement. However, because of the stress concentrations that develop in the toe of a slope, strain-energy potential within the slope is not uniform. This leads to a nonlinear relationship between pushback width and strain energy (or maximum shear stress) for varying sizes of incremental mining cuts. When narrow pushbacks are mined, excavation takes place within the zone of stress concentration, and the resulting displacements are typically a high percentage of the overall pushback width. This renders narrow pushbacks difficult to maintain and results in either excessive time spent in additional slide cleanup or frequent, unplanned stepouts into the pit. If stepouts must be taken too often, it may not be possible to complete the pushback, and ore production may be lost. Additionally, if strain softening of the rock mass occurs with displacement, there is considerable geotechnical incentive to *mine out* the existing failed rock as much as possible to take advantage of the higher strength associated with a less disturbed rock mass.

For maximum production efficiency, a pushback must be wide enough for double spotting trucks and three lanes of traffic. For a normal rectangular cut with the digging face at right angles to the pit wall, the inside corner is relatively inefficient because either a wider swing or single spotting is required. The outer edge is similarly inefficient, in addition to having a less-than-complete digging face. Thus, a narrow pushback, with the majority of the digging time in the inside corner and the outer edge, will produce fewer tons per shovel shift (Figure 4.7).

Where there is overall slope displacement, the effective pushback width will be reduced by bench sloughing. For example, if a 15-m-high bench dug at 63° sloughs to a 36° angle of repose, 6.5 m of the pushback width will be lost. If sloughing of the working bench and the bench below occurs in the same section, the effective width of the pushback would be reduced by 13 m. Thus, in this case, pushbacks for a displacing slope should be designed at least 13 m wider than pushbacks for a stable slope.

In particular, the following sequence is to be avoided:

1. A production schedule is planned with a minimum width pushback and full shovel and truck efficiency.
2. Slope displacement reduces the effective width of the pushback, and the tons per shovel shift decreases.
3. At the lower production rate, ore will not be uncovered in time, so the pushback width is decreased in an attempt to uncover ore.
4. The reduction in pushback width results in one-way traffic, more single-side loading, and delays from tension crack offsets. This further reduces the tons per shovel shift.
5. Steps 3 and 4 are repeated until the pushback is not minable.

Another factor that must be considered in production scheduling is the horizontal displacement. If the slope experiences horizontal displacement on some level between one pushback and the next, additional material will have to be mined to achieve the design toe.

### 4.6.4 Slope Dewatering

Dewatering has been demonstrated in several cases as an effective control for displacing slopes (Argall and Brawner 1979). Cost–benefit analyses indicate that dewatering programs are one of the most cost-effective mechanisms available for improving slope stability for both stable and unstable ground. In many low-permeability rock masses, reduction of pore pressure prior to failure is difficult with conventional methods but is readily achievable after failure. Dewatering below the pit bottom is often required to achieve an acceptable pore pressure condition for the overall slope, and an analysis of aquitards and compartmentalization of groundwater must be completed by the geotechnical engineer to focus the proper level of effort on dewatering.

## 4.7 CONCLUSIONS

Improvements in continuum and discrete element models have enhanced the ability of engineers to design high overall pit slopes that often experience significant slope instability. These models can be used to predict the development of rock-mass movements, which previously could only be evaluated empirically. As the sophistication of the stability models improves, more data are required to extend the model; particularly high-quality data are necessary for calibrating the model. Better estimates of the state of stress and pore pressure are needed in addition to improvements in the techniques used for rock-mass strength prediction. In particular, stability models that are capable of modeling dynamic processes should continually be developed.

Provided enough data can be gathered to acquire a firm understanding of the failure mechanism, stability models that will enable engineers to provide operations personnel with reliable predictions of the impact of mine excavation on the stability of the overall pit slopes can be developed. If pit slope analyses can be developed in a spirit of cooperation between the geology, engineering, and planning staffs, geotechnical analyses can be run in concert with mine planning to create an optimum slope design.

Experience with several, extensive open-pit slope failures indicates that mining can be successful in areas of large-scale slope stability if variance in production can be accepted in the unstable pit sectors. If such fluctuating production rates can be tolerated, specific operational procedures can be used to minimize the impact of slope displacement on mine operations. Geotechnical analysis can be used to identify the procedures required to allow mine operations to work either within or near areas of large-scale slope instability. Such analyses can greatly improve the possibility of successful mining in these areas.

## 4.8 REFERENCES


Amadei, B., W.Z. Savage, and H.S Swolfs. 1987. Gravitational stresses in anisotropic rock masses. *Intl. Jour. Rock Mech. Min. Sci. & Geomech. Abstr.*, 24:5–14.

Argall, G.O., and C.O Brawner. 1979. Mine drainage. *Proceedings 1st International Mine Drainage.* Denver, Colorado.

Barton, N., R. Lien, and J. Lunde. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mech.*, 6:183–236.

Bieniawski, Z.T. 1973. Engineering classification of jointed rock masses. *Trans. South African Inst. Civ. Eng.*, 15:335–344.

Bieniawski, Z.T. 1978. Determining rock mass deformability–Experience from case histories. *Intl. Jour. Rock Mech. Min. Sci.*, 15:237–247.

Call, R.D., and J.P. Savely. 1991. Open pit rock mechanics. In *SME Mine Engineering Handbook*. New York:AIME, pp. 860–882.

Cruden, D.M., and S. Mazoumzadeh. 1987. Accelerating creep of the slopes of a coal mine. *Rock Mech. & Rock Engrg.*, 20:123–135.

Deere, D.U., and R.P. Miller. 1996. *Engineering classification and index properties of intact rock.* Air Force Laboratory Technical Report No. AFNL-TR-65-116. Albuquerque, New Mexico.

Farmer, I.W. 1986. Energy based rock characterization. In *Application of Rock Characterization Techniques in Mine Design.* ed. M. Karmis. New York:AIME, pp. 17–23.

Kennedy, B.A., and K.E. Niermeyer. 1970. Slope monitoring systems used in the prediction of a major slope failure at the Chuquicamata Mine, Chile. *Proceedings on Planning Open Pit Mines with Special Reference to Slope Stability.* Amsterdam:AA. Balkema, pp. 215–225.

Munn, F.J. 1985. Coping with wall failures. *Canadian Mining and Metallurgical Bulletin*, No. 78-884, 9–62.

Ness, M.E. 1993. Personal communication.

Ryan, T.M., and R.D. Call. 1992. Applications of rock mass monitoring for stability assessment of pit slope failure. *33rd U.S. Symposium on Rock Mechanics*, Santa Fe, New Mexico.

Salamon, M.D.G. 1984. Energy considerations in rock mechanics. Fundamental results. *Jour. South African Inst. Min. Metal.*, 84:8:233–246.

Savely, J.P. 1993. Slope management strategies for successful mining. *Proceedings International Congress on Mine Design*. Ontario, Canada.

Savely, J.P., and V.L. Kastner. 1982. Slope instability at Inspiration's Mines. In *Proceedings 3rd International Conference on Stability in Surface Mining.* ed. C.O. Brawner. New York:AIME, pp. 609–634.