Keynote Paper:

Investigating the Effects of Mining-Induced Strain in Open Pit Slopes

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Abstract

Modern open pit slope design is increasingly requiring meaningful forecasts of slope deformation behaviour during mining, in addition to traditional design approaches used to satisfy slope stability. As open pit mines become deeper and stability mechanisms become more complex, optimizing ore extraction while maintaining stability may require detailed design investigations to maximize slope angle while maintaining acceptable levels of risk. In situations where ultimate pit mining encroaches on surface or underground infrastructure, design acceptability criteria require incorporation of deformation tolerances in addition to traditional factor of safety or probability of failure criteria. This requires numerical modeling techniques, such as distinct element modeling, that incorporate a detailed representation of the engineering geology and groundwater conditions and the mining excavation sequence. Validation of model input parameters is established through careful calibration of model displacements with actual slope monitoring data during back analysis. Slope deformation forecasts can then be used to define acceptable movement magnitude, rate and strain threshold criteria.

This paper describes the use of detailed distinct element modeling to investigate the influence of strain-induced degradation (disturbance) of the rock mass that occurs in slopes as a result of stress relief and relaxation during mining. Recognizing the potential adverse effects of mining-induced strain is important in defining appropriate design strength parameters, and hence, successful geotechnical slope designs. As current analytical methods may be limited in addressing the time-dependent strength behaviour of geological materials, deformation forecasts must be continually evaluated with slope displacement monitoring to confirm the validity of analytical forecasts. To safeguard against destabilizing slope conditions, if they were to occur, empirical failure time prediction and runout assessment methods can be used to define slope monitoring thresholds and hazard zones.

1 Introduction

In jointed rock masses, structural geology often plays an important, if not controlling, role in slope stability. Representing the influence of structural discontinuities in the design of stable bench, interramp and overall slope geometries can be critical in defining and optimizing slope angles. Where geological structure is continuous relative to the scale of an excavation, simple planar, wedge and toppling failures can be assessed using basic limit equilibrium methods (Hoek and Bray, 1981). However, where geologic structure is discontinuous and involves complex interaction with the rock mass, more complex failure modes may occur, such as step path or complex wedge failures that combine shear or tensile failure through intact rock or rock bridges along the failure path. Limit equilibrium analysis methods can be used to evaluate step path failures or instability that involves rock bridging, but these methods do not provide an evaluation of deformation and strain related changes in shear strength and rock mass conditions that precede progressive failure development.

This paper provides an overview of the capabilities of distinct element modeling in providing retroactive assessment and predictive forecasts of slope deformation and stability mechanisms in open pit mine slopes. General observations from modeling can provide valuable insight into the influences of strain-induced yielding that can occur as a result of mining. Depending on the orientation and continuity of geological structure, mining-induced strain or creep displacements may lead to destabilizing conditions that result in failure over time, unless they are recognized in advance and mitigated. Empirical methods provide a useful means of estimating the timing and potential impacts of failure, and improve safety. Examples of these conditions are presented.
2 Distinct element modeling of complex slope stability and deformation

2.1 Background
Distinct element numerical modeling codes such as UDEC (Itasca 2011) and its three-dimensional equivalent 3DEC (Itasca 2003), provide the ability to model complex geological structure, rock mass and groundwater conditions, the interaction of mining activity and related displacement behaviour, and the potential for progressive failure development. This capability is advantageous for the investigation of slope movements that develop prior to, and coincident with, a failure condition, or in environments where design tolerances to slope deformations are low, such as in areas of permanent mine facilities, critical haul roads or crown pillars. Due to the number of input parameters required in distinct element models, a high level of input data quality is required. A more detailed description of the UDEC numerical modelling methodology summarized in this paper is presented in Rose and Scholz (2009).

2.2 Assessment of slope conditions to determine numerical modeling requirements
In consideration of whether detailed numerical modeling may be appropriate for a given slope, an evaluation should first be carried out to determine whether stability conditions can be adequately explained using simpler, more traditional stability analysis methods. Factors that require consideration include the stability mechanism, structural complexity and whether adequate information exists to calibrate a detailed numerical model and provide meaningful modeling results. As shown in Figure 1, a series of checks can be applied to determine whether detailed numerical modeling may be required:

![Figure 1. Chart illustrating general process for determining whether detailed numerical modeling is warranted.](chart.png)
Information that is required to carry out detailed UDEC modeling is summarized as follows:

- a detailed geological model defining the distribution of major lithologies, faults and/or structural zones;
- structural discontinuity fabric defining the orientations of predominant structural sets, persistence and spacing, and peak/residual shear strength characteristics;
- geotechnical properties defining the rock mass shear strength and deformation properties of the main geotechnical units, as derived from laboratory testing and rock mass classification;
- a comprehensive database of displacement monitoring data;
- detailed historical and forecasted mining sequences;
- piezometric monitoring and groundwater modeling results defining the distribution of pore pressures during various stages of mine development and closure; and
- information defining the insitu stress regime, if available.

2.3 Model representation of physical slope conditions

2.3.1 Structural discontinuity fabric and shear strength

Development of structural fabric in distinct element models requires scaled representation of semi-continuous to continuous structural features that are important to kinematically possible failure modes or may influence slope deformation. Non-important structural features such as discontinuous joints are generally accounted for in the characterization of rock mass strength, as represented by finite difference zones (Section 2.3.2). Peak and residual strength conditions should be incorporated to account for the effects of shear strain on discontinuity shear strength. Figure 2 illustrates the modeling of discontinuities that exhibit peak-residual behaviour such as joints, as compared to major fault zones that may only exhibit residual shear strength behaviour.

Model discontinuity (joint) properties that are subject to peak and residual slip criterion include:

- joint friction angle ($\phi'$);
- joint cohesion ($c'$);
- joint tensile strength, which is typically zero;
- joint normal stiffness ($jkn$); and
- joint shear stiffness ($jks$).
2.3.2 Rock mass strength conditions and yield softening criteria

As illustrated in Figure 3, rock mass conditions are represented in model finite difference zones using a Mohr-Coulomb elastic-plastic constitutive model. Mohr-Coulomb strengths are derived from the 2002 Hoek-Brown criterion at maximum confining stresses ($\sigma_{3\text{max}}'$), or alternatively normal stresses ($\sigma_n'$), that are representative of interramp to overall slope conditions. Strain softening criteria are simulated with FISH routines to update the constitutive parameters of yielded finite difference zones after each excavation from undisturbed to disturbed rock mass strength conditions defined by Hoek-Brown Disturbance (D) factors of 0 and 1, respectively. Softening of rock mass modulus ($E_m$) to disturbed values is carried out following each excavation step for each finite difference zone that is indicated to be in a current state of yield or has yielded in the past. Rock mass dilation angle is generally approximated as one-quarter to one-eighth of the rock mass friction angle, except for fault zones or low quality rock masses where dilation angle is zero. Poisson’s ratio is estimated using laboratory testing results or empirical estimates.

Figure 3. Illustration of rock mass strength conditions represented in finite difference zones with Mohr-Coulomb yield criterion (strength envelopes represented by dashed lines). The Hoek-Brown (2002) strength criterion is used to define undisturbed and disturbed conditions (solid lines).

As illustrated on the stress plot in Figure 3, the transition between disturbed and undisturbed conditions (thick grey dashed line) in an excavated slope occurs as a function of the tensile or shear damage that develops from stress relief and slope relaxation during mining. This transition can occur over significant depths and is sensitive to the structural, rock mass and groundwater conditions in the slope. The general shape of the disturbance transition curve may exhibit sub-linearity relative to the disturbed and undisturbed non-linear rock mass strength envelopes. This sub-linearity may justify the use of Mohr Coulomb or bilinear strength criterion, or alternatively, s-shaped strength criterion or rock damage transition behaviour described by others (e.g., Kaiser and Kim 2008, Diederichs et al. 2007, and Carter et al. 2008). As shown on Figure 4, the range of stress or alternatively depth of the disturbance transition zone can be estimated from the percentage of yielded elements and contacts in distinct element models. This information can be used to estimate appropriate ranges of Hoek-Brown D factor for limit equilibrium modeling. Selection of appropriate depths of disturbance requires consideration of whether slope stability conditions are dominated by geologic structure, rock mass conditions, or a combination of both.
2.4 Calibration of existing slope conditions based on back analysis

2.4.1 Simulation of the actual stress-strain path of a slope with bench-by-bench mining

Due to the amount of time required to develop detailed distinct element models, a common approach is to model slope conditions with a single excavation stage under elastic conditions, followed by analysis of elastic-plastic conditions. The main disadvantage in simplifying the mining excavation sequence is that slope deformation and stability is assessed relative to displacement endpoints rather than the actual stress-strain path that a slope undergoes during mining. This is illustrated in the following example:
Consideration of a single staged excavation may not identify the acceleration trend in slope displacements during mining of the final benches. Acceleration trends in displacement rates can be important in identifying potential for strain-induced degradation of structural and rock mass strength conditions, or progressive failure development. This, in turn, could be important in the consideration of long-term slope stability conditions based on the observation of increasing strain rates and magnitudes, even if the final slope configuration is determined to be stable.

2.4.2 Representing the excavation sequence

Simulation of multi-bench mining conditions should include the staged representation of at least one prior mining phase. This is important to represent the actual mass removed during mining and to limit inertial shock effects in the model that may adversely (unrealistically) affect the modeling results. As shown on Figures 6 and 7, calibration is established through back analysis of slope displacements on the existing mining phase by first removing the previous slope on a bench-by-bench basis. Following each excavation, time stepping is carried out to equilibrium to verify that conditions are stable before proceeding to the next excavation stage.

![Figure 6. Illustration of benched mining sequence including representation of the previous and current mining.](image)

Once model calibration is achieved, model forecasts are carried out for subsequent mining.

One advantage of simulating bench-by-bench mining is the ability to forecast incremental bench velocities as a function of incremental model displacements relative to the bench mining schedule, as illustrated in Figure 7.

2.5 Forecasts of slope deformation and stability

Once modeled slope conditions are back analyzed and calibrated based on slope monitoring data (grey shaded areas on inset graphs in Fig. 7), displacement and velocity forecasts can be developed for comparison with actual monitoring installations during mining. These can be developed in the form of surface monitoring points for comparison with prism monitoring data or as subsurface displacements that can be compared with inclinometer monitoring data. Provided that the modeling results meet acceptability criteria, this provides a useful means of defining threshold movement criteria that can be evaluated on an ongoing basis. Trigger action response procedures (TARPs) can be developed to include re-evaluation of slope conditions, if actual displacements or velocities exceed model forecasts.
Identification of potential instability

Identification of potential indicators of instability from modeling results can be important with respect to both short-term and long-term stability of a slope. A number of stability indicators can be used to identify the potential for instability, including:

- model plasticity and discontinuity slip defining patterns that are consistent with potential mechanisms of instability;
- strength reduction factor (SRF) indicating a low factor of safety;
- elevated levels or contrasts in shear strain and/or a high percentage of yielding in rock bridges; and
- elevated model velocity, indicating potential for continued creep-displacements or strain.

Empirical assessment of strain-induced instability

The ongoing effects of strain from mining-induced ground relaxation and stress relief, as well as seasonal variations in pore pressure, can lead to time-dependent deformation and degradation of slope conditions leading to progressive slope failure, as described by Martin (1993) and Zavodni (2000). To accommodate uncertainties in time-dependent behaviour of slopes that may not be effectively accounted for in analytical models, empirical methods can be used to safeguard against progressive slope failure by predicting the time and associated runout distances of a potential slope instability.
3.1 Forecasting potential slope failure time using linear estimates of inverse-velocity trends

The inverse-velocity method, developed by Fukuzono (1985), provides a useful tool for interpreting slope monitoring data, with the objective of anticipating or predicting slope failure. The concept of inverse velocity for predicting the time of slope failure was developed based on previous Japanese work and on large-scale well-instrumented laboratory tests simulating rain-induced landslides in soil. The conditions simulated in the laboratory were considered to be characteristic of accelerating creep (i.e., slow continuous deformation) under gravity loading. When the inverse of the observed rate of displacement (“inverse velocity”) was plotted against time, its values approached zero as velocity increased asymptotically towards failure. A trend-line through values of inverse-velocity versus time could be projected to the zero value on the abscissa (x-axis), predicting the approximate time of failure, as shown on Figure 8.

\[
\frac{1}{V} = [A(\alpha - 1)]^{\frac{1}{\alpha - 1}} \cdot (t_f - t)^{\frac{1}{\alpha - 1}}
\]  

[1]

\( t \) is time, \( A \) and \( \alpha \) are constants and \( t_f \) is the time of failure. In the laboratory measurements preceding failure, \( \alpha \) was found to range between 1.5 and 2.2. As shown on Fig. 8, the curve of inverse velocity is linear when \( \alpha = 2 \), concave when \( \alpha < 2 \) and convex when \( \alpha > 2 \). Based on the results of laboratory testing, Fukuzono concluded that a linear trend fit through inverse-velocity data usually provided a reasonable estimate of the time of failure, shortly before failure occurred.

As documented by successful predictions of slope failures in Rose and Hungr (2007), the assumption of linear trend fits of inverse-velocity data can be applied to estimate the time of slope failure, provided that forecasts are adjusted to accommodate potential trend changes. Figure 9a is a plot of inverse velocity versus time showing the trends of nine survey prisms located at various elevations on the southeast wall of the Barrick Goldstrike Betze-Post open pit, Nevada, USA, over the last six weeks preceding the S-01-A slope failure in August 2001. Filtering (data smoothing) of two-hour robotic total station prism monitoring measurement data was achieved by calculating six-day average (incremental) slope distance velocities to reduce the effects of instrument error in low-level velocity values. As displacement rates increased, a clear inverse-velocity trend developed and began to converge on a failure time of August 29, 2001. Linear regression was applied to the inverse velocity values, which defined a coefficient of determination (R²) of 99% for all nine prisms. The failure occurred on the predicted date encompassing an overall slope height of 550m and an estimated 47 million tonnes. The instability occurred over several hours as a series of nested wedge failures and rock avalanches.
In evaluation of the inverse-velocity trend lines in Figure 9a, it is noteworthy that the shallower sloping trend lines, defining the minimum values of $-A$, corresponded to prisms located higher on the slope where displacement rates were greater and tension cracks were first observed. The steeper trend lines, representing larger values of $-A$, corresponded to prisms located lower on the slope that exhibited lower displacement rates, but higher rates of acceleration up to the time of failure. These observations have proven useful in the identification of conditions that could potentially lead to progressive slope failure in other slope instabilities, as discussed in Section 3.3.

### 3.2 Assessment of debris runout potential and hazard zoning

Evaluation of the potential impacts of slope failure requires consideration of the potential failure mechanism, the type and nature of the materials involved (e.g., rock or soil, peak or residual strength), the degree of saturation, and the surface roughness or effective friction angle of the runout surface. Analytical codes, such as the Dynamic Analysis code, DAN-W, developed by Hungr (1995), are available to evaluate the potential impacts of debris runout from slope failure or rock avalanches. However, as is the case for other analytical methods, these models require proper calibration, which may not be possible if deteriorating slope conditions are observed and available time for modeling is limited. As a starting point, empirical methods can be used to estimate potential failure runout distance based on empirical databases of historical failures or landslides.

One empirical method was developed by Tianchi (1983), who carried out statistical analysis of data from 76 major rock avalanches that occurred in the European Alps between 1901 and 1974. He found that a strong correlation exists between avalanche volume ($V$) and the maximum vertical drop ($H$) to the maximum horizontal distance travelled ($L$) by the avalanche, a relationship previously considered by Heim (1932) and Scheidegger (1973). The tangent of the $H/L$ ratio is referred to as the Fahrböeschung ($\alpha$), overall travel angle or runout angle, and is defined by the angle measured from the top of the failure scarp to the limit of the runout debris. Figure 10a shows the graphical relationship defined by Tianchi (1983), with data points for four failures added from the south wall of the Barrick Goldstrike Betze-Post open pit. Figure 10b shows the same relationship, but in terms of the runout angle on the $y$-axis.
Figure 10. Correlation between the H/L ratio and rock avalanche volume (a) compiled from a database of 76 European rock avalanches by Tianchi (1983). Data points for four failures are added from the south wall of the Betze-Post open pit, Nevada, USA (inset legend). The right graph (b) shows runout angle derived from the tangent of (H/L) on the y-axis.

Equation [2] was derived from the best fit least squares regression line on Figure 10a, using a logarithmic relationship (with base 10) between the H/L ratio and volume in cubic metres. A standard deviation of 0.118 delineates the lower bound and upper bound dashed lines relative to the solid mean fit line.

\[ \log(\frac{H}{L}) = 0.664 - 0.1529 \log V \]  

[2]

An important observation from Figure 10b is that small failure volumes of about 100,000 m³ have a mean runout angle of 38°, which is a typical angle of repose for fragmented rock. By rewriting Eqn. [2] in terms of L, mean horizontal runout distance in metres can be calculated as a function of slope height from Eqn. [3]:

\[ L(\text{mean}) = \frac{H}{10^{(0.664 - 0.1529 \log V)}} \]  

[3]

An upper bound runout distance can be estimated from the minus one standard deviation H/L ratio from Eqn [4]:

\[ L(\text{upper bound}) = \frac{H}{10^{(0.546 - 0.1529 \log V)}} \]  

[4]

As seen on Figure 10, the majority of the historical landslide data plot above the minus one standard deviation line for volumes less than about 25 million cubic metres. This indicates that provided that failure conditions do not define fluid runout potential, this relationship may provide an upper bound estimate of runout distance from the known location of tension cracks on an unstable slope.
3.3 Documented instability on the South Wall of the Barrick Goldstrike Betze-Post Pit

Over the last ten years, four large slope instabilities have occurred on the south wall of the Barrick Goldstrike Betze-Post open pit. The timing of each of these instabilities was predicted using the inverse-velocity method described in Rose and Hungr (2007). The runout limits of the two most recent failures, S-07-B and S-09-B, were predicted using the mean Fahrböeschung relationships defined by Tianchi (1983) shown on Figure 10 and in Eqns. [2] and [3]. Two of the instabilities, S-01-A and S-09-B, occurred on ultimate pit walls following the completion of mining. The S-05-B and S-07-B failures occurred during mining, but were forecasted in advance and had no adverse impacts on the mining operation.

Table 1 summarizes the volume, mass and plan source area of the instabilities. Also included are the minimum values of \(-A\) from the inverse-velocity trend lines that were used to forecast the failures. These trends were derived from prism slope distance inverse velocity with data averaging increments ranging from two to six days.

### Table 1. Summary of Slope Instability Details on the Betze-Post South Wall between 2001 and 2009.

<table>
<thead>
<tr>
<th>Failure</th>
<th>Volume (Mm³)</th>
<th>Mass (Mtonnes)</th>
<th>Area (10⁴m²)</th>
<th>(-A_{(\text{min})}) (cm⁻¹)</th>
<th>Runout Angle (°)</th>
<th>Rock type</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-01-A</td>
<td>19</td>
<td>47</td>
<td>367</td>
<td>0.010</td>
<td>22.5</td>
<td>Intrusives</td>
</tr>
<tr>
<td>S-05-B</td>
<td>2</td>
<td>5</td>
<td>51</td>
<td>0.042</td>
<td>27</td>
<td>Metasediments</td>
</tr>
<tr>
<td>S-07-B</td>
<td>0.2</td>
<td>0.5</td>
<td>29</td>
<td>0.165</td>
<td>38</td>
<td>Metasediments</td>
</tr>
<tr>
<td>S-09-B</td>
<td>1.1</td>
<td>2.7</td>
<td>42</td>
<td>0.079</td>
<td>29</td>
<td>Metasediments</td>
</tr>
</tbody>
</table>

Figure 11. Plan map showing limits of Zones S-01-A, S-07-B and S-09-B. Zone S-05-B was mined out.
3.3.1 Relationship between failure size and inverse-velocity trends

Figure 12a shows the correlation between the minimum value of $-A$ and failure volume for the four historical failures on the south wall (Table 1). The minimum values of $-A$ correspond to prisms in the active zones of the instabilities, in the area of tension cracks in the upper portions of the failures. The relationship between $-A$ and failure volume is defined by Eqn. [5], with an $R^2$ of 98%.

$$-A \ (cm^{-1}) = 384 \ V^{-0.628}$$  \ [5]

Figure 12b is a plot of the minimum value of $-A$ versus the plan area of three instabilities in metasedimentary rocks on the south wall, two of which are seen on Figure 13. As all of these failures involved a component of shearing along bedding with consistent dips of about ten degrees towards the pit, the assumption was made that failure size could be related to failure area based on an assumed relationship between depth and areal extent. The relationship between $-A$ and plan area (square metres) of the failures is defined by Eqn [6], with an $R^2$ of 99%.

$$-A \ (cm^{-1}) = 1.01e^{-6.18E-05 \times \text{Plan Area}}$$  \ [6]

Figure 12. Correlation between the slope of linear inverse-velocity trend lines ($-A$) and failure volume (a) for four historical instabilities on the south wall of the Betze-Post pit. The right graph (b) shows $-A$ versus the plan area of three failures in metasediments.

The data from the failures supports an inverse relationship between failure size and rate of acceleration, as indicated by values of $-A$. Larger failures tend to move at lower rates of acceleration than smaller failures, and are therefore easier to detect. Similarly, creep displacements in previously failed (residual) materials commonly move at low acceleration rates. This is illustrated by the blue symbol on Figure 12b that represents residual materials within Zone S-09-B (Fig. 13). This suggests that inverse-velocity trends, once detected, can be compared to potential failure size to determine the nature of the slope movement materials and associated hazards. Intact materials may define brittle behaviour and therefore require lower monitoring thresholds to accommodate higher acceleration rates and greater runout potential. Conversely, different monitoring thresholds may be justified for residual materials on the basis of creep behaviour and low rates of acceleration.
3.3.2 Assessment of slope monitoring thresholds for intact ground

To determine whether established monitoring thresholds will provide adequate warning time of impending slope failure, monitoring thresholds can be calculated from Eqn. [7], according to required warning time (T) prior to failure.

\[
\text{Monitoring Threshold (cm/day)} = \frac{1}{T \text{ (days)}} \times \frac{1}{-A \text{ (cm}^{-1})} \]

[7]

By substituting Eqn [5] for \(-A\) in Eqn [7], slope monitoring velocity thresholds for intact ground on the south wall are plotted in Figure 14a according to different advanced warning times and potential failure volumes. Figure 14b shows calculated monitoring velocity thresholds according to plan surface area from Eqn. [6]. These plots indicate that lower thresholds are required to provide adequate warning time for smaller failures.

As an example of slope monitoring thresholds, “Warning” and “Critical” movement rates of 38 and 76 mm/day (1.5 and 3 inch/day) were defined for the south wall, represented by the orange and red dashed lines on Figure 14, respectively. For a potential failure volume of about 100,000 m³, these thresholds correspond to calculated advanced warning times of one day and 12 hours, respectively. As discussed in Section 3.2 and shown on Figure 10b, this volume defines the approximate limit where materials run out at an angle of repose of 38°. Previous instabilities of this size, or smaller, were contained by available catchment on the slope, but still require consideration of rockfall potential. Zone S-08-B, seen on Figures 11 and 13, provides an example of these conditions.
3.3.3  Hazard zoning and protection measures

Hazard zones are defined across the south wall, below areas of observed tension cracks, bounded by major faults. Details of hazard zoning assessments carried out for mining on the 11th West Layback are discussed in Armstrong and Rose (2009). Since mining was completed on the 11th West Layback in late 2009, a 70 to 100m wide by 125m high waste rock buttress has been constructed to stabilize the lower south wall and to provide access to the in-pit backfill dumps on the east side of the pit (Fig. 13). An approximately 8m deep catchment ditch and rockfall impact berm is maintained on the inside of the buttress against the toe of the south wall. Rockfall and debris runout modeling is carried out to evaluate potential hazards and design mitigation measures.

The south wall is continually monitored with a Reutech MSR 300 slope monitoring radar and a robotic total station prism monitoring system. Both systems send warning messages to Mine Dispatch and geotechnical and mine engineering senior personnel in the event of a threshold level being exceeded. Standard operating procedures and TARPs are clearly defined to evacuate and barricade access to the slope within a distance of 180m of the highwall in the event of a Critical threshold being confirmed. The haul road to the in-pit backfill dumps is periodically closed during elevated slope movement periods. Continual evaluation of slope conditions is carried out to assess potential rockfall and runout hazards, and mine operating procedures are adjusted, as required.

4  Conclusions

Rock slope stability mechanisms in large open pit slopes involve the complex interaction of geologic structure, rock mass, groundwater and in-situ and induced stress conditions. Stress relief and slope relaxation related to excavation induces strain that can result in dynamic changes in strength conditions over the life of a mined slope. Even following mining, seasonal changes in pore pressures can result in strain-induced degradation of slope conditions. To accommodate uncertainties in time-dependent behaviour of geologic materials that may not be effectively accounted for in analytical models, empirical methods can be used to safeguard against progressive slope failure, if it were to occur. The inverse-velocity relationship by Fukuizono (1985) and Fahrböeschung...
relationships developed by Tianchi (1983) help improve safety by allowing prediction of the timing of slope failure and associated runout distances. However, in the application of these methods, it must be recognized that these tools can aid in the engineering judgement process, but should not be applied on the basis of misperceived accuracy. This is particularly important if fluid runout potential may be present. The empirical relationships developed by Tianchi (1983), and shown on Figure 10, are based on a database of fragmental rock avalanches and should not be applied to saturated weak rock or soil-like materials.

When dealing with the potential for slope failure, a strict philosophy should be maintained where mining is not allowed to continue on or below a slope that it is expected to fail. Adequate time (e.g., days) should be provided for evacuation of established hazard areas, thereby safeguarding against the possibility that empirical or analytical predictions are wrong. Not all acceleration trends lead to slope failure (e.g., regressive displacement, strain hardening), and as such, false alarms should be anticipated as part of the ground control management process. In that regard, the occurrence or recurrence of false alarms in slope failure time predictions should not lead to complacency with respect to mine operating procedures or the arbitrary adjustment of threshold criteria in unstable slope areas.

Data from four significant instabilities on the south wall of the Barrick Betze-Post open pit indicate an inverse relationship between the minimum negative slope of the inverse-velocity trend line, an indicator of acceleration rate, and failure size. This relationship can be used to define appropriate threshold monitoring levels to provide adequate warning time of impending slope failure. Although experience with this approach is currently specific to the south wall of the Barrick Goldstrike mine, it is presented with the anticipation that the general approach may be successfully applied in other mining operations that have documented monitoring data from prior slope instability. Further development of this method will hopefully expand on this experience.

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6 References


