Approaches for Estimating Slope Breakback and Stability Longevity for Closure of Large Open Pits

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Abstract. Currently, few guidelines exist for how a practitioner should tackle the quantitative assessment of long-term slope performance for open pit mine closure design. Material degradation, resulting from cumulative incremental time-dependent deterioration and propagation of episodic damaging events (e.g., precipitation and seismicity) is of paramount importance in the closure context, as rockmass competence degradation largely controls long-term slope performance. Deterioration rates are governed not just by intrinsic (material specific) and extrinsic (environmental) factors, but also can be significantly affected by previous mining influences (principally blast damage, slope oversteepening, stress relief, etc.). Suggestions for various empirical, analytical and numerical modelling approaches for assessing impact of degradation on closure design risk are outlined and three key geotechnical closure design challenges for supporting design recommendations commensurate with planned post mining land use (PMLU) risk are discussed. These include:i) defining appropriate safe pit crest standoff distances, ii) assessing likely extent of long-term pit crest creep-strain tolerance zones, and iii) quantifying long-term overall slope stability. A risk-based approach is proposed, comprising initial susceptibility screening for time-dependent degradation, followed by increasing analysis rigour (commensurate with risk), starting with empirical assessments, moving then to first principle deterministic limit equilibrium calculations and finally escalating to numerical assessments involving sequentially calibrated time-stepped degradation models.

Keywords: Open Pits · Slope Stability · Crest Breakback · Closure · Longevity

1 Introduction

Assessing the long-term stability of engineered cut slopes is becoming an increasingly important focus for large operating open pits approaching mine closure. Final cut slopes may occasionally, but not always, be blasted to a higher standard than often employed during routine production, leaving competent final wall conditions that may help prevent rapid regression. Contrast this with open pits where 'goodbye' cuts have aggressive

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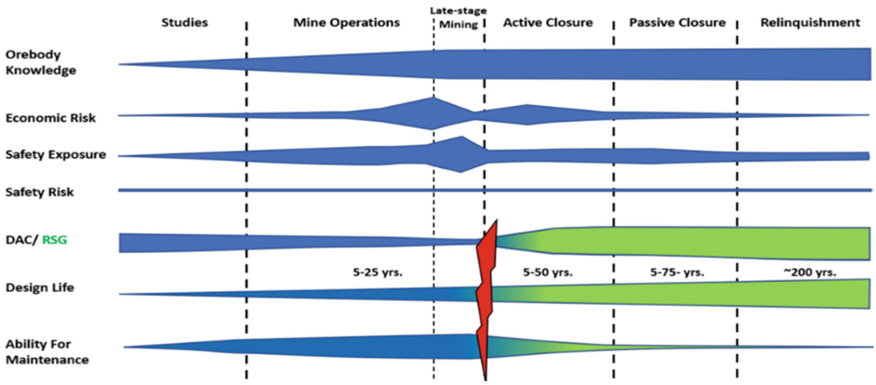

Fig. 1. Evolution of key characteristics throughout the mine life cycle. Lightning bolt marks the point where Operations-Focus DAC must change to a Closure-Focus RSG. Width of time bars defines degree of focus intensity at that study stage

designs (with lower than usual design acceptance criteria), or where 'final' wall conditions have subsequently been disturbed by cave mining and/or by induced subsidence. For these mines, preparation for closure must also examine potential long-term retrogression associated with uncontrolled crest breakback due to oversteepening and/or induced subsidence. These types of final walls can no longer be considered as well engineered pit slopes; but for closure planning, their condition must, at least, be well understood. Design for pit closure must however consider post-mining land use (PMLU) whilst also dealing with such extremes of pre-closure slope geometry.

Figure 1 conceptually illustrates the evolution of key characteristics through the typical mine life cycle of planning, development, and closure for an open pit mine. Although several of these key factors evolve during this cycle (such as orebody knowledge and mine design life), for effective slope management it is important that the safety requirement for zero harm with as low as reasonably possible (ALARP) risk principles governs both operational and closure decisions. Consequently, a major change in design outlook focus post-mining is required, and design acceptance criteria (DAC), or closure Relative Stability Guidelines (RSG), as proposed by Carter et al. [8], must be adjusted in alignment with confidence, as well as with consequence and stability reliability objectives.

2 Material Characterization and Degradation

For closure implementation to be effective, planning vision needs to adopt a much longer-term perspective than the typical 100-year design life normally considered for (civil) engineered structures located on or within a rock mass [18]. Typically, open pit mine cutback designs range between 5- to 25-year operational life. For mine closure, however, design life must look in the range of several hundred years. Consequently, risk-based approaches must be adopted. In addition, comprehensive understanding is needed of long-term, rocktype-specific degradation processes.

Material deterioration is rarely given much attention in conceptual or operations design stages. Typically, only the most obviously susceptible materials receive any design consideration. Even during operations, time-dependent material degradation influence on poor slope performance is oftentimes overlooked (i.e., the root cause of instability may be misdiagnosed). In an open pit setting, two principal mechanisms for material degradation are recognised, namely: i) environmental (weathering and pore pressure fluctuations), and ii) mining-induced influences (i.e. unloading, blast damage etc.). Donati et al. [13] highlight the influence of seasonal and continuously active events causing permanent deformation within a slope due to accumulation of slope damage through rockmass dilation and intact rock fracturing. Thus, for closure, damage appraisal must extend to include long-term creep and seismic loading.

Approximately one third of large open pits around the world are excavated within rock types of low durability, susceptible to severe weathering. Slope deterioration in these rock masses is a problem that must be addressed for pit closure as, with time, significant reduction in slope stability can occur due to degradation effects. Mining-induced damage effects can also aggravate degradation processes, leading to increased excavation-related fracturing as a result of blast damage and / or excavation geometry interaction with rockmass, structure and stress. Fracturing related with blast damage, stress relief (unloading) and/or long-term progressive creep processes and transient pore pressures, can lead to strength degradation and deterioration, even in hard rock.

Fookes et al. [17] define three essential factors that must be considered for safe excavation, construction and serviceability over the design life of a slope, namely: i) current geotechnical characteristics, ii) dominant time-related processes that control the engineering behaviours of the involved soil and rock masses, and iii) anticipated rate of deterioration of geotechnical properties. Pit design procedures to account for current geotechnical conditions are well established, including recognition that susceptible materials need to be identified to receive special attention.

Many sedimentary and volcaniclastic rocks show marked weathering susceptibility irrespective of geographical location, whereas almost all rock types under tropical and/or high humidity/rainfall or freeze/thaw conditions show increased propensity for degradation and extreme weathering. Deep saprolitic profiles are common throughout the tropics and dominate the behaviour of the upper part of many deep open pits in these climatic zones. Extensive alteration halos also exist in many mines worldwide, irrespective of latitude location, and the degree of degradation of the parent competent rock in these halos around and adjacent to high-grade ore bodies can be substantial.

For closure, pit slope design must consider all these time-dependent degradation influences on long term slope stability. All potentially degradable rock types in a slope that might affect final pit wall stability need to be defined, categorized, and then tested so that their degradation-prone characteristics in a pit closure context can be properly understood. To achieve this understanding, requires classical geomorphology knowledge applied with an engineering geological, soil mechanics and geotechnical engineering viewpoint to ultimately 'put numbers to science' that can be used for slope design purposes. As aspects of this conundrum have been studied for decades, with widely different viewpoints on how to categorize / analyse the processes, the authors have consolidated these various differing perspectives within Table 1 into a 3-stage recommended approach to allow material degradation consideration in closure design.

Table 1. Suggested process for characterizing material degradation for closure slope design

	1: Initial Screening	**2: Material Strength Assessment**	**3: Parameter Assignment for Stability Analysis**
Precedent Based & Lab Data	*Qualitative Observations* • Ascribe deleterious mineralogy • Define weak, poorly cemented materials • Check for poor 3rd cycle slake durability • Note high core tray degradation • Check observed rapid excavation deterioration and degradation*	*Lab or Field Benchmarks* • Determine weathering index strengths (Reidmuller, 1997), [21, 27] • Check laboratory testing of material properties relevant to weathering • Undertake back-analysis calibration as a proxy for predicted degraded strength [16]	*Weathering Profile Prediction* • Make geomorphological best estimate, with and without denudation (erosion) • Define time dependent degradation function (e.g., use Fish function relationships In FLAC for instance, if time-stepping to replicate degradation)
Classification Based	*Predictive Weathering Classification* • Follow guidelines for predicting qualitative behaviour with respect to weathering (per [23] • Conduct Slope Stability Probability Classification, SSPC [19, 20, 24] • Undertake Rockslope Deterioration Assessment, RDA [35], also [36]	*Material Degradation Parameters* • Estimate degradation RMRs (per [2]) • Assess governing MRMR change [28] • Assess time-sequenced degradation in GSI (see [32]) • Assess D factor change [9]	*Strength Weakening Assessment* • Estimate Hoek-Brown D factor gradation with depth as function of pit excavation geometry (e.g., [42] and assign through depth zoning in *RS2*, *SLIDE*,*FLAC*, etc. • Evaluate appropriate stress corrosion and degradation parameters • Define strain weakening constitutive behaviour model combined with GSI reduction (per [15])

*Degradation in rock competence occurs as a consequence of all forms of weathering including chemical and mechanical (e.g., freeze-thaw, thermal slabbing and various slaking) processes.

2.1 Initial Screening

Hack [21] recommends benchmarking ground conditions in similar exposures over a range of excavation dates to establish weathering susceptibility. Predicting change due to weathering without adequate calibration can be challenging due to variability of both intrinsic (inherent material attributes) and extrinsic (environmental) changes.

It should be appreciated that rates and depths of weathering can be highly variable, depending on local differences in rockmass characteristics. Various index tests have been trialled but with limited success. Notwithstanding the challenges created by over-simplification, a combination of red flagging indexes is suggested for initial Stage 1 screening, namely: mineralogy (XRD), slake durability, strength (UCS) and observed performance. Of the three empirical weathering prediction systems, cited in Table 1 the Relative Determination Assessment (RDA) approach [35] is more intuitive and thus somewhat easier to effectively implement than the Slope Stability Probability Classification (SSPC) system, [20]. However, while useful for helping with categorizing susceptible rock masses that need specific mitigation measures to ensure long term stability, classification systems alone cannot realistically be used for detailed design, and are thus suggested to be used solely to initially define and assess the possible extent of potential problems.

2.2 Material Strength Assessment

In general, for the second step in applying the logic of Table 1, two approaches have merit, one adopting a scale of weathering grades and the second adopting modified rock mass classification indices. Characterisation by weathering class (per IAEG or ISRM scales) is standard practice in most investigation studies and may be quantified in terms of rock mass ratings, laboratory test results or by instability back analysis. Anticipated degraded rock performance can usefully then be inferred for equivalent weathering classes, i.e., current Grade II material degrading to Grade III in future will have the same strength range as current Grade III material. Alternatively, rock mass classification ratings can be adjusted for future weathering and degradation, following the principle that increasing weathering grade is associated with decreasing rockmass competence and reduced joint strengths [7].

2.3 Strength Assignment for Stability Analysis

Stage 3 in Table 1 is arguably the most challenging and also most influential. This is because weathering profiles are always complex and irregular, [1, 37], making them difficult to predict. Lessons from work on saprolites and residual soils point to instability being directly related to relict rock structures within the soils [29]. This makes forecasting of such pit slope behaviour difficult without good understanding of local conditions. These difficulties are further compounded when the combined effects of future environmental (climatic, seismic, etc.) and anthropogenic (i.e., human influence) changes need consideration. Although probable slope degradation effects can often be assessed qualitatively using geological judgement and/or conditional applied modelling following gradational strength weakening rules,again, this type of evaluation needs thorough weathering process understanding for predictions to have any credibility. Also, modes and scales of instability, and susceptibility to strength degradation must be considered (Table 2).

Advanced numerical analyses offer a systematic process for problem screening and assessing sensitivity for longevity planning and closure design. However, even the most sophisticated modelling codes remain wanting in that time-dependency cannot readily be accounted for explicitly. Care must further be taken in domaining, [5], as the location of estimated boundaries can substantially affect pit stability assessment outcomes. Thus, a significant degree of engineering judgement is warranted in progressing to designs, and even more importantly, to understanding the sensitivity of instability modes to ongoing weathering. Early recognition will help focus further characterization studies and assist in estimating suitability of different mitigation options for ensuring reliability of final overall pit slope design profiles.

3 Closure Slope Design Analysis

Analysis methods (empirical, analytical, limit equilibrium and numerical) available for pit closure slope design are identical to those that are regularly utilized for operations design, but they need to be applied with additional long-term vision considerations:

Three aspects need special attention:

1. Long-term (PMLU weathered profile) properties need to be assumed, rather than short-term (freshly blasted/excavated) operation-era properties.
2. Water conditions will be different at closure, sometimes varying markedly from operations; according to the long-term stabilized PMLU vision, with the proviso that no actively managed engineering solutions can be considered viable, and many other passive systems will also degrade in effectiveness. Therefore, there will be a need to study time evolution of pit lakes, as well as long-term changes in water pressures within the slopes, to identify critical conditions.
3. Long return period extreme events (e.g., cyclonic rainfall, earthquakes) also need consideration, even though these are seldom addressed for most pit operations.

Although the same approaches as used in operations pit design may be appropriate for use in closure design, additional longevity-related factors need assessment, and most will remain uncertain due to a lack of data. Thus, in addition to all common modelling issues, it must be expected that results from long-term predictive PMLU modelling will be even more dependent on the assumptions inherent in the analyses.

3.1 Analysis Approaches

One of the most important reasons for undertaking detailed review of operations era pit slope design analyses, but with a closure perspective [8], is to establish required set-back distances behind each sector around the pit crest so that such limits can be delineated in planning documentation. Often also there is a need for verifying and establishing strain damage delineation zone boundaries, behind the pit crest (as discussed later, in more detail). Precision in delineation of such strain boundaries and set-back lines and hence

Table 2. Modes of Instability and Long-term Weathering Susceptibility

	Modes of Instability	**Time-Dependent Susceptibility**	**Comments**
Inter-ramp / Overall Slope	Toppling	Low to moderate	Typically, self-stabilizing (e.g., toppled blocks buttress the slope and limit progressive dilation), provided the mechanism is not triggered by undercutting of weak material part way up the slope (i.e., buttressing ineffective due to slope geometry).
	Wedge/planar (daylighting structure)	Moderate	Long-term stability determined by time-dependent shear strength fatigue on controlling structures. Tight, healed/clean, undulating structures often less susceptible to time-dependent deterioration than filled planar structures.
	Wedge/planar (with non-daylighting structure)	Moderate to high	Long-term stability determined by time-dependent shear strength fatigue on controlling structures and toe rock mass properties. Toe buttress rock mass deterioration generally controls stability.
	Circular / rotational	High	Materials most susceptible to rotational instability, are typically weak and/or highly jointed, and thus vulnerable to both further degradation and resulting shear strength reduction.
Crest / Toe Impact	Ravelling	Moderate to high	Deterioration and loosening of all but the most massive and competent slopes are unavoidable, resulting in localized progressive crest loss and subsequent rockfall hazards.
	Wedge/planar structure	Moderate	Long-term stability determined by time-dependent shear strength fatigue on controlling structures. Tight, healed/clean, undulating structures often less susceptible to time-dependent deterioration than filled planar structures.

selection of appropriate analysis approaches will thus depend on project stage and data availability.

For conceptual planning, empirical and analytical approaches are most common, recognizing that pit slope designs and final limits will almost certainly change during mining. Initial concept designs may be based on published recommendations, such as the Western Australia Department of Mines Guidelines recommended slope design breakback angles of 25° for weathered (oxidised) pit slopes and 45° for unweathered (unoxidised) bedrock horizons. The authors urge extreme caution in using these types of guidelines, as they are generally rocktype specific and when used for other rocktypes have been demonstrated to be unreliable [10, 39]. Rather, it is suggested to use local empirical pit slope breakback performance data (if available), or adopt simple limit equilibrium analytical calculation methodologies (as shown in Fig. 2) as a starting point for establishing initial set-back lines per domain around a pit crest.

Application of the latter approach is straightforward, relying only on making some reasonable assumptions of discontinuity fabric and typical rockmass competence and long-term strength and groundwater characteristics of the pit walls per domain around the pit shell. Some reasonable assessment of controlling parameters and proper domaining of pit wall conditions is, however, critical for reliable forecasting of post mining land use performance [5, 8].

For use of the charts included in Fig. 2, consideration should first be given as to whether the domained slope will behave as a competent rockmass; typically, with high m_b values as shown towards the right side of the two charts in Fig. 3, with failure assumed on a given discontinuity fabric or through the rockmass, on, en-echelon joints. Alternatively, it could behave as a blocky or weak rockmass, generally with a low m_b value (more towards the left side of the diagrams in Fig. 3), for which circular styles of failure geometry might be more representative.

Estimating appropriate parameters can be challenging, but Fig. 3 provides some guidance on probable strength relationships for different slope behaviour for a wide distribution of slope height vs slope angle data points for natural and open pit slopes, with corresponding rock competences, defined in terms of Hoek-Brown m_b values. Representative parameters can be picked off the charts or algebraically evaluated using the equations provided in Carter & Carranza-Torres [6].

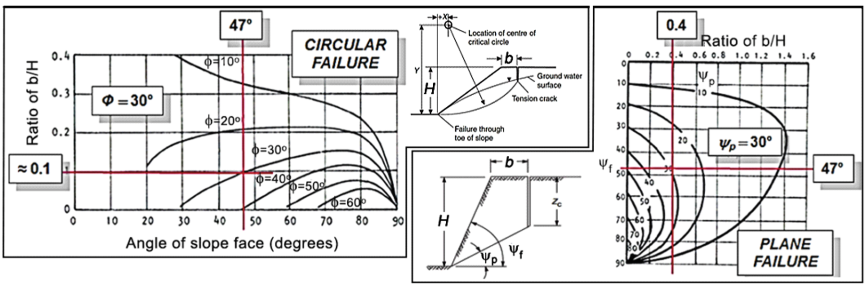

Fig. 2. Analytical assessments for slope set-backs utilizing comparative tension crack location charts for circular saturated and planar dry failure geometries

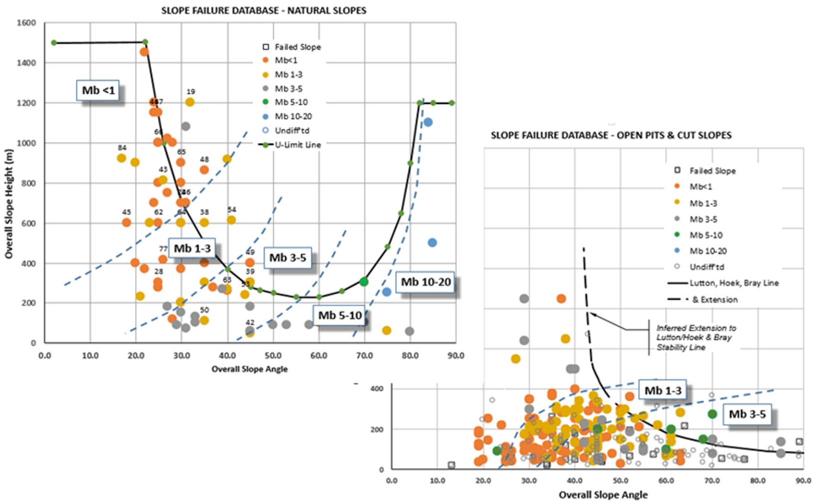

Fig. 3. Postulated slope height-vs slope angle relationships for natural and open pit slopes plotted as a function of rock competence, expressed in terms of Hoek-Brown m_b values (from [4], based on data compiled by [31, 44–45] and [46, 47]). (NB: See original papers for listings of data-points per reference numbers in graphs)

3.2 Detailed Analysis for Final Closure Plan Submissions

Building on the initial assessments discussed above, as one moves into detailed design, of primary importance is better definition of the extent and degree of probable adverse surface displacements that could occur behind the pit crest. Refining the precedent-based empirical assessments undertaken in early closure planning, requires, as credible as possible, an evaluation of current and future stability states, and potential induced strain impact extending behind the pit crest. These assessments can be done using a variety of different software (including Limit Equilibrium codes such as SLIDE) or can be undertaken in a more integrated manner using distinct element modelling that can consider both structure and material influences on stability and induced strain.

Although there are no universally accepted methodologies for evaluating long-term slope behaviour, a starting point is to estimate "time to slope failure". Observations and theory tell us that the lower the safety margin, the less time remains before the slope fails. This postulated time-failure relationship can be represented graphically as shown in Fig. 4, based on the equations detailed in Appendix I.

If the estimated time to failure picked off the chart in Fig. 4 exceeds the probable planned, say 200 year closure warranty period, perhaps no further analysis is required. However, in most cases, particularly where poor rockmass conditions exist or where aggressive operational slope design has prevailed, this will prove unlikely. In addition, typically it can be expected that Regulators examining a final closure plan will require that careful analysis and a well-calibrated slope model has been used to assess current and future stability states and likely pit crest strain profiles.

Two approaches can be considered for this detailed evaluation stage:

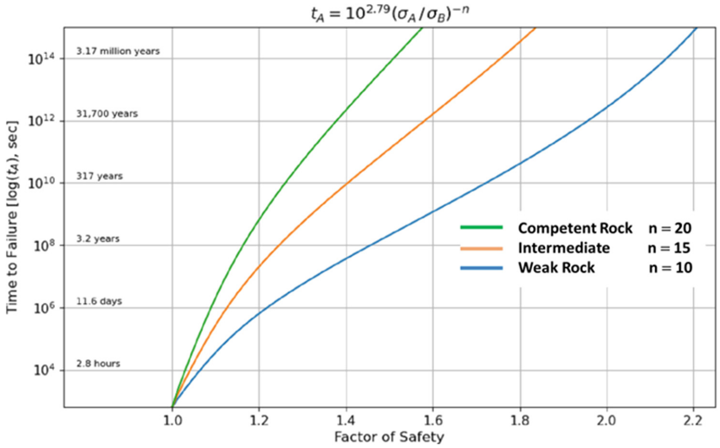

Fig. 4. Relation between Factor of Safety and Time to Failure for different rock competency based on static fatigue theory [12] [n = 20 for competent massive brittle rock, UCS > 250 MPa,n = 15 for strong, dry sedimentary rocks, 25 MPa < UCS < 250 MPa; and n = 10 for weak rocks, UCS < 25 MPa, where n = the exponent of the rate of stress corrosion, which, in this situation can be considered analogous to Hoek-Brown m_i as descriptive of rock competence, where σ_A/σ_B and t_A are as defined in Appendix I]

1) *Perform long-term stability analysis using assumed final conditions.* In this approach, time-dependency is implicit in the selection of modelling parameters and conditions. This is the most commonly applied approach for detailed evaluation; with the advantage being that it is relatively easy to perform with limit equilibrium or numerical methods. The disadvantage for limit equilibrium approaches is that important intermediate transient conditions (e.g., water influence) may be missed, and estimates of induced strain behind the pit crest are difficult to undertake without extra refinement.
2) *Perform long-term time-dependent analysis by following evolution of specific modelling parameters.* Explicit modelling of this type requires carefully applying time-dependent well-calibrated viscoelastic or viscoplastic behaviour models. This approach is less common and requires significant numerical modelling know-how to correctly implement and choose appropriate input values. Implicit modelling of time-dependency stability change however also requires careful calibration, which can be achieved by attempting to link strength weakening to plastic strains associated with simulated benching of the open pit and/or seasonal climate events. The modelling of annual cyclic events also introduces an implicit means to model time. However, this again necessitates that good monitoring data be available for proper validation. These types of implicit treatments, if well calibrated, though, will also allow use of more familiar behaviour criteria such as Mohr-Coulomb and Hoek-Brown (with which most users will have more experience), allowing intermediate stability conditions to be considered. Use of these more familiar approaches may have the advantage also of being more readily understandable, thus helping speed any approval process by being more easily checked by Regulatory Authorities.

Pit Crest Strain and Damage Criteria Guidelines. Open pit induced subsidence can typically be addressed in terms of degree of disturbance, based schematically on three physically distinct zones proceeding outward from the pit crest: i) zone of large-scale surface rupture/cracking (fractured zone), ii) zone of small-scale surface displacement (continuous surface subsidence zone), and iii) stable zone. The limit that bounds the "fractured zone" from the "continuous subsidence zone" is defined as the "fracture limit". The zone of surface subsidence of most concern in the context of this paper, occurs mainly outside the fracture limit, characterized by a combination of horizontal strain and angular distortion (or maximum shear strain). Different combinations of those two parameters define damage categories for surface infrastructure, as shown in Fig. 5 (per [22]).

Typically, the fracture zone limit is considered to coincide with the onset of severe damage, marked by the region (4–5) boundary in Fig. 5. It should be noted, however, that these descriptive surface damage categories were developed based on observations of typical residential and industrial building damage over coal mine subsidence troughs and thus should be applied with caution when they are used to define possible damage limits in other rock engineering situations and with different infrastructure.

Overall Stability Assessment. 2-D and 3-D numerical modelling utilizing explicit methods can effectively be applied predictively for analysis of PMLU long-term slope performance, as codes like *FLAC3D* and *3DEC* can readily be time-stepped through a progressive degradation sequence, starting from the initial pit slope conditions to those existing at the end of operations and beyond. Similarly, inclusion becomes possible of specific time-sequenced decommissioning of actively managed engineering measures, such as deep pumping wells installed during mining to improve stability. Dewatering systems and wells can be installed in operations, then eliminated in the time-stepping sequence, replicating the transition to full mine closure. Diminished effectiveness can also be time-stepped for other permanent measures that might be prone to maintenance failure (e.g., replicating depressurization efficiency decay with horizontal drains).

Effective calibration can be achieved where slope performance and early warning systems have been monitored throughout the transition phase to full closure. Often the most useful instrumentation records for such purposes are those obtained when specific

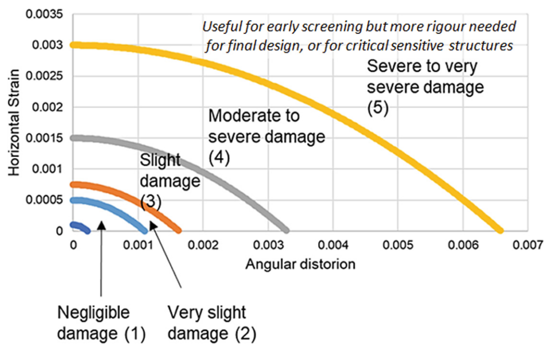

Fig. 5. Different categories of building damage as determined from a combination of horizontal strain and angular distortion (modified after [22])

(often periodic) accelerations are measured in response to increased precipitation and/or mining, especially where corresponding decelerations have also been recorded on the same instruments associated with implementation of engineered control measures or when mining was temporarily stopped.

Figure 6 provides an example of output from such a well calibrated model. The top left diagram (Fig. 6a) shows the basic *UDEC* model created to help forecast future PMLU pit slope response based on calibration to actual slope performance through the transition phase from operations to pit closure.

The geology in the *UDEC* model (as shown in Figs. 6a and 6b), includes a series of folded and faulted sedimentary units, with bedding and cross-jointing that make the slope susceptible to toppling and localized bench instabilities. Rock mass properties utilized in the model were derived from evaluation of geotechnical borings and field-based assessments. The treatment of time-dependency in the model was implemented implicitly using a Hoek-Brown constitutive behaviour model with strain weakening. First, Geological Strength Index (GSI) values were derived from the geotechnical data using the charts by Marinos & Hoek [34] and Marinos et al. [33] for tectonically disturbed sedimentary rock. From these, residual Hoek-Brown strength parameters were established based on Cai et al. [3] to represent a degraded, strain-weakened state resulting from relaxation, slope displacements, and progressive failure. Based on the peak and residual properties derived, intermediate points were then extrapolated for plastic strain increments based on those determined from laboratory cyclic loading tests and acoustic emission results by Eberhardt et al. [14], and then calibrated through a back analysis of earlier slope acceleration events.

The model was built to first simulate the mine sequencing and excavation of the benches to the pit bottom (Fig. 6a), using monitoring data to calibrate the base case. Options were then explored for assessing pit closure conditions, simulating different rock fill buttress options (Fig. 6b). Pore pressures were accounted for in the form of an effective stress analysis, which entailed applying an updated porewater pressure field corresponding with each mined bench. Specific data points were implemented using phreatic surfaces derived from hydrogeological modelling.

To help ensure reliability, the full simulated sequence was calibrated against prism monitoring data. Figure 6c, and d show the results for the model state corresponding with the end of mining. Toppling is observed in the upper slope, similar both in extent and magnitudes to that observed in the monitoring data. The results indicate that the slope remains in a relatively stable state after the pit bottom is reached, although some ongoing movements periodically occur in response to simulated seasonal precipitation.

A key question asked of the modelling was then, how the long-term stability of the slope would evolve, not just in response to future major precipitation events, but also in response to mine closure mitigation plans to infill the pit lake and then construct a stabilization buttress. Figure 6e provides key insights for this question, showing the continuation of the model for an extended period incorporating the pit lake and post-mining seasonal precipitation events. Significant, continued movement of the pit wall is indicated, suggesting that the slope likely might be approaching its limit equilibrium state. However, a decrease in stability due to the influence of progressive failure was also seen in the results showing the slope's response to recurring seasonal precipitation events,

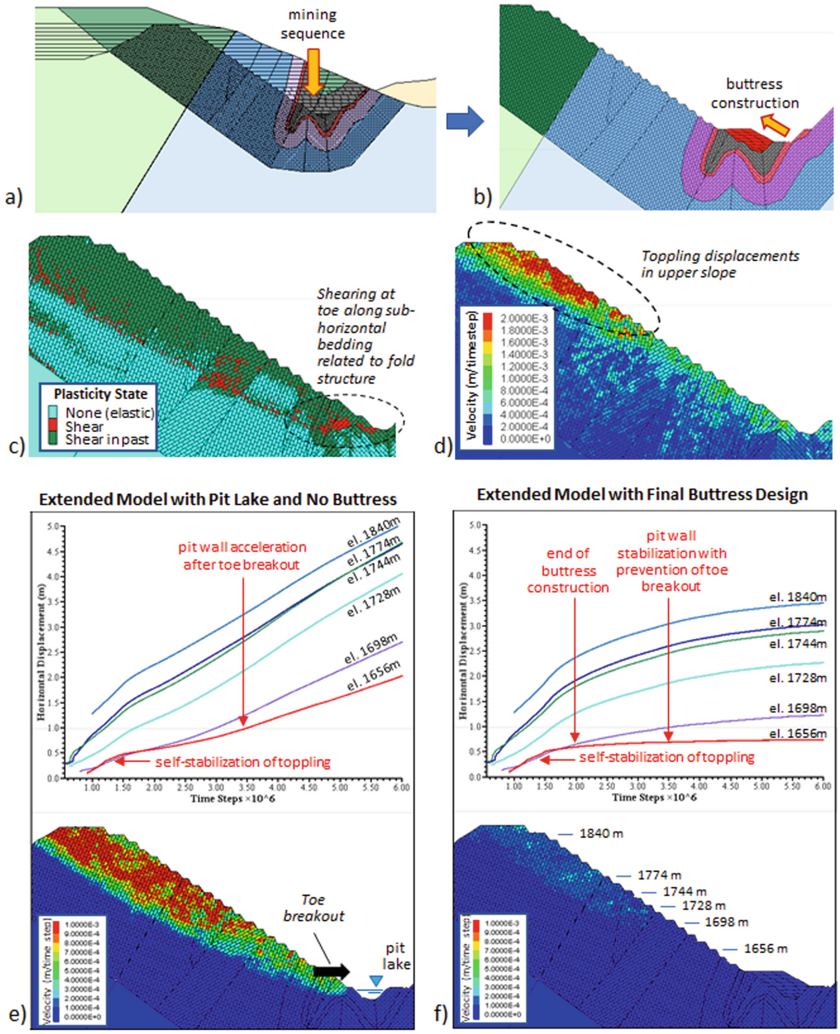

Fig. 6. *UDEC* model for case study. b) Model means to simulate construction of a buttress for mine closure. c) Modelled slope performance with respect to yield state and d) relative velocities, up to the end of mining. Results show movement of the upper slope developing through toppling of steeply dipping beds. e) Results assuming the long-term performance of the slope, including the development of a pit lake. This shows that without confinement at the toe, shearing along bedding joints contributes to progressive failure and toe breakout. f) Comparison of the same model but with simulated buttress design to achieve long-term stability after mine closure

(Figs. 6d and 6e) resulting initially in localized yield and weakening, but eventually promoting the development of full slope toe breakout.

Figure 6f when compared to 6e, illustrate the benefits of constructing a substantial toe buttress, as a planning approach for closure of the mine. Close inspection of the results show that while the toppling failure mechanism is self-stabilizing to an extent (i.e., slowly decreasing slope displacement rates), the construction of the slope buttress for mine closure effectively stops the deformations at the toe of the slope. This in turn leads to significant decreases in the toppling deformations in the upper wall.

It must, of course, be emphasized that while good well calibrated models can yield significant insight. All such numerical results are highly dependent on the model inputs, especially the interpreted geology, but also the estimated rock mass properties and pore pressure values, and the degree of achieved calibration to known conditions. As with all models, accurate formulation of the slope geometry and geology in the *UDEC* model shown in Fig. 6 was critical for achieving believability. This is not atypical, as it should be appreciated that as a general rule, the more sophisticated the model the greater often is the sensitivity to the interpreted structural geology, which in turn then directly controls the modelled slope kinematics and displacements. Acknowledging that many simplifications must necessarily be built into such models, the diagrams in Fig. 6 are nonetheless illustrative of the insight that can be gained in understanding degrading slope situations. These models must however be built extremely carefully, adhering to all details of the available structural geology and assumptions regarding observed failure mechanisms. If this is achieved such that the modelled displacements are consistent with monitoring data then such models can provide valuable predictive tools allowing insights into slope failure mechanisms, long-term stability state, and evaluation of the benefits or otherwise of various mine closure mitigation options.

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APPENDIX I - Derivation of Static Fatigue Law Relationship

In order to assess time dependency as a function of long-term rock strength loss, let us consider application of the static fatigue law formulated by Damjanac & Fairhurst [12]. Equation 1 (Equation 4 in their paper) states that $n\log(\sigma_A/\sigma_B) = \log(t_B/t_A)$, where σ_B and σ_A respectively represent short term (intact) lab. Strength and long-term degraded strength and t_B and t_A are the corresponding short and long-term times to failure.

For slope longevity evaluation in the context of this paper, the goal is to relate Factor of Safety to the time to failure for rock slopes of varying rock type and competence (this is described by n). The first step in applying this approach would be fitting n in this equation to available lab data. Assessing three different competence rock types, allows graphing of FoS results as per Fig. 5 in the paper [typically $n = 20$ for very competent massive brittle rock, where UCS $>$ 250 MPa; $n = 15$ for strong, sedimentary rocks where 25 MPa $<$ UCS $<$ 250 MPa; and $n = 10$ for weak rocks, where UCS $<$ 25 MPa]. As an example, take $n = 20$ as representative of test data for competent rock and fit this with Eq. 1: $\left[\frac{\sigma_A}{\sigma_B}, \log(t_A)\right] = [0.80, 4.73]$ as per lab. Data in the original paper.

This results in the following expression: $20\log(0.8) = \log(t_B) - 4.73$. Solving for $\log(t_B)$ gives $\log(t_B) = 2.79$ and substituting this back into Eq. 4 and solving for t_A gives an expression allowing estimation of time to long-term failure as a function of strength drop typical for failure of such a rocktype, viz: $t_A = 10^{2.79}\left(\frac{\sigma_A}{\sigma_B}\right)^{-n}$.

However, because Factor of Safety to time to failure is desired instead of driving stress ratio at failure to time to failure, the following expression was developed by gradually reducing UCS to get failure in a 500 m high modelled slope as a means to convert this driving stress ratio to Factor of Safety:

$$FoS = 1.95\left(\frac{\sigma_A}{\sigma_B}\right)^4 - 6.67\left(\frac{\sigma_A}{\sigma_B}\right)^3 + 8.83\left(\frac{\sigma_A}{\sigma_B}\right)^2 - 5.62\left(\frac{\sigma_A}{\sigma_B}\right) + 2.52, \left[0 \le \frac{\sigma_A}{\sigma_B} \le 1\right]$$

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