The Warm Blanket of Geotechnical Databases and the Reality of Rock Mechanics

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Abstract. The limited success of simplified slope stability assessment tools and concerns related to slope performance resulted in a review of an open pit mine located in South Western United States. 3D numerical was used to assess the LOM design which led to improved slope stability, while maintaining key safety and production targets.

An advanced inelastic strain-softening constitutive model (IUCM) was used with FLAC3D (finite difference) and SlopeX software to assess slope stability of the LOM design. The weak nature of the rock mass necessitated that time-dependent behaviour was accounted for, which typically cannot be considered by simplified slope stability assessment approaches.

The limited availability of a well-developed geotechnical database required that model inputs (material properties) were primarily derived through rigorous back analysis of slope behaviour, as recorded by slope monitoring equipment. Significantly, subsequent slope performance of the LOM compares favourably with original forecast results derived from the advanced numerical software and IUCM.

This paper demonstrates that the derivation of material properties along with the simulation of complex rock mass behaviour can be successfully achieved despite a limited geotechnical database. This underlines the importance of applying the correct assessment tools and rock mass constitutive models in order to produce safe, reliable open pit mine designs.

1 Introduction

1.1 Statistical Versus Engineering Analysis

Rock masses are naturally occurring substances that are typically variable and complex in their makeup. It is this feature that geostatistics and resource modeling of mineral deposits attempts to understand through the development of a geological drillhole database with sufficient quality and quantity. It would seem reasonable, therefore, that the same method could be used when characterising variability of rock mass quality for geotechnical assessment purposes. Considerable resources are regularly allocated to develop geotechnical databases through approaches such as field work (core logging and mapping) and laboratory/index testing. The resulting database is often used to classify rock mass quality (Q, RMR, GSI etc.) and/or to develop block models and synthetic

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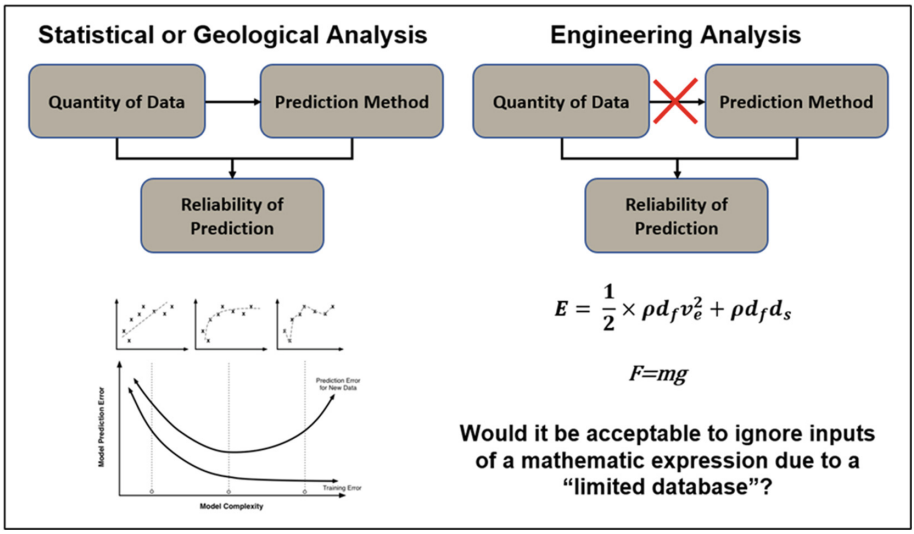

Fig. 1. Statistical Analysis Compared to Engineering Analysis

rock masses on which geotechnical assessments are based. However, it takes considerable time and expense to develop a geotechnical database, meaning data quantity and overall quality can be adversely affected if such resources are not available. Faced with such dilemmas, many practitioners advocate a 'simple' geotechnical assessment tool be used. However, doing this inadvertently assumes a false equivalence between rock mass variability (statistical analysis) and rock mass response to loading (engineering analysis) (see Fig. 1). In reality, these analyses approaches are vastly different and should not be confused nor conflated. A well-developed geotechnical database cannot make up for the shortcomings of an overly simplified engineering assessment tool. But is the reverse true? Can an appropriate engineering assessment tool make up for the shortcomings of a limited geotechnical database? This paper will show, through a slope stability case study, that this can be achieved.

1.2 Slope Assessment Methods

Slope stability assessments are commonplace throughout the mining industry and are largely used to evaluate safety and economic risk. The different assessment approaches largely fall into three categories; empirical methods; limit equilibrium methods; and numerical methods.

Empirical methods (formal and informal) have been applied in mine design for many decades. The reliability of empirical methods is reliant on the database of cases (on which they are based) being comparable with the specific condition being assessed. Furthermore, such methods cannot take into consideration site-specific aspects, such as unusual 3D mine geometries or ground conditions. It is for this reason that many slope assessments are now completed using limit equilibrium or numerical methods, which can take many site-specific aspects into account.

The application of limit equilibrium and numerical methods for slope stability assessments is widespread. However, there are considerable and well-documented differences between these methods. Aspects such as equilibrium, internal slope stresses, rock mass deformation, slope failure, and kinematics are assessed in considerably different ways. Significantly, these differences can affect assessment outputs beyond factor of safety (FoS) results (Chiwaye, 2010). According to Lorig and Varona (2004), the main contributions of numerical models in rock slope stability analyses can be summarized as:

- Numerical models can automatically extrapolate potential failure mechanism(s) and the most probable failure surface (or shear zone).
- Numerical models can implicitly or/and explicitly incorporate significant geological features (such as faults, weathered zones, etc.) and groundwater conditions providing more realistic analysis than classical analytical models in which the conditions are frequently oversimplified leading to conservative solutions.
- Numerical analyses can help to explain in more detailed manner the physical behaviour of slope instability.
- Numerical methods can test, in a relatively short time, multiple topographical geological and failure mechanism situations and propose different design options.

1.3 Numerical Methods and Constitutive Models

While numerical methods are regularly used to assess rock slope stability, many geotechnical practitioners overlook several key aspects; the numerical code and constitutive model used. Consequently, many assume that all numerical methods generate the same or similar results. This is incorrect. As is discussed below, selecting the correct numerical code and constitutive model is critical if a reliable slope stability assessment is to be produced.

Numerical Codes

Vakili and Sandy (2014) provide a detailed discussion on common numerical modelling methods used in the mining industry. The most widely applied numerical modelling methods in the mining industry are described below. It is intended that these be general, non-technical descriptions, intended to enable key concepts to be understood by a wider audience in the mining industry. Consequently, it should be noted that some of the terms used might not be accurate from a purely technical perspective. It is important to also note that all analysis software and tools have their own benefits and limitations. Furthermore, there is no single tool or software that is applicable to every single scenario. Each method is described in terms of its predictive power. Predictive power here is measured as the ability of a model to forecast displacement, stress, extent of fracturing, and extent of failure induced as a result of a single excavation; in a broad sense, its ability to predict or replicate the observed response to mining:

- **Elastic Methods:** This method assumes a linear elastic mechanical response for the rock material. Consequently, its predictive power significantly reduces with more

complex conditions (such as anisotropic, highly stressed, and squeezing rock). Interpretation of results is achieved through indirect and empirically-derived failure criteria. Consequently, reliability of this method outside its predictive limits depends greatly on the adopted failure or interpretation criteria and its calibration efforts. The most widely used 'elastic' software codes are MAP3D and EXAMINE3D.

- **Inelastic Time-Independent (Implicit) Methods:** The inelastic time-independent (implicit) method can account for both elastic and non-elastic mechanical responses of rock material. However, this method uses an implicit (time-independent) numerical technique which finds a solution by solving an equation involving both the current state of the system and its future state. This means that these codes are usually unable to follow the development of rock mass failure, including progressive damage processes in a system, nor extreme material non-linearity such as softening. The predictive power of this method is usually better in small deformation/strain conditions, and should always be accompanied by appropriate calibration and manual material changes when used outside its predictive limit. The chosen failure criterion can also greatly influence the predictive power of this method. Abaqus/Implicit and RS2/RS3 are the most commonly used software codes that employ the inelastic time-independent method.
- **Inelastic Time-Dependent (Strain Dependent) (Explicit) Methods:** This method estimates the future state of a model from its current state in an automatic time stepping manner, which enables the development of a failure, including non-linear material changes due to excessive damage (i.e., new phases of stress and strain). This method can accommodate a wide range of rock mass conditions, including high-stress and high-deformation situations. When used with an appropriate constitutive model (failure criterion) and carefully selected input parameters and pre-mining stress state, this method offers a reasonable level of accuracy. Consequently, often only minor calibration efforts are required when monitoring results become available, meaning that it is more a case of validation than calibration. The outputs from this approach are often directly comparable to outputs from real-life monitoring data such as displacement, strain, and stress. Abaqus/Explicit and FLAC3D are the most commonly used continuum software codes that feature this method, while ELFEN, 3DEC (if used in fully-discontinuum mode), or PFC are the more commonly used discontinuum examples.

Constitutive Models

A constitutive model is a mathematical representation of a material that describes the full mechanical behaviour from none to complete damage. Depending on its complexity, a constitutive model depends on a number of parameters that usually represent some mechanical characteristics of the material being modelled. Unlike discontinuum codes, continuum numerical methods depend heavily on constitutive models to represent how a rock mass responds to loading. Much like numerical methods, the subject of constitutive models is wide but usually falls into one of three categories (Vakili, 2017):

- **Linear Elastic Model:** The linear elastic model assumes that rock behaves elastically under applied loading, meaning that it will return to its initial shape and size when any disturbing forces are removed. In reality, the rock may remain elastic only up to

about half to two-thirds of its peak strength. Consequently, this model increasingly loses its predictive power as the level of loading and failure increases in the rock mass. Elastic models require only a few input parameters, and there's often a standard and easy guideline for selection of these inputs.

- **Strain-Softening-Mohr-Coulomb Model:** The Mohr–Coulomb peak strength criterion and associated constitutive models have gained wide acceptance and application in the field of geotechnical engineering. This criterion is obtained from two key parameters; cohesion (c) and friction angle (ϕ). Although, more advanced numerical assessments often couples the Mohr–Coulomb failure criterion with a strain-softening post-peak model to simulate the full stress–strain behaviour of the rock mass. (Note that there are other, less well known variations also by some practitioners.)
- **Proprietary Model:** These models include proprietary algorithms (i.e., owned / commercialised intellectual property) and processes that are used to solve the full stress–strain behaviour within the numerical model. While the capability and accuracy of these models is often promoted by their developer, several fundamental issues arise, including lack of transparency (such as technical details, theories, and limitations) and difficulty to independently review (due to limited information provided or access to the material model). Examples of advanced rock mechanics constitutive models include, but are not limited to LR2 (Levkovitch, Reusch, & Beck, 2010) and IMASS (Itasca, 2022).

Improved Unified Constitutive Model (IUCM)

The Improved Unified Constitutive Model (IUCM) was developed with the intention to address the limitations of the current modelling practices described above in this paper. This model gathers the most notable and widely accepted previous research work in the area of rock mechanics and integrates it into a unified constitutive model that can better and more accurately predict the stress–strain relationships in a continuum model. The IUCM was developed after extensive calibration against well-established case histories and independent field applications, including weak and strong rock masses (Vakili, 2016; Vakili, 2017). Such case histories include (but are not limited to):

- Triaxial test results:
  - Various rock mass failure mechanisms which are dependent on different confinement levels (Tiwari & Rao, 2006).
  - Analytical relationships proposed by Hoek and Brown (1980), and laboratory test results outlined by Donath (1972), and McLamore and Gray (1967).
- Failure mechanisms in massive, brittle rock associated with the Mine-by-Experiment project (Vakili, 2016).
- Complex buckling failure mechanism and effects of rock strength anisotropy associated with vertical shafts (Vakili, Sandy, & Albrecht, 2012; Vakili, Albrecht, & Sandy, 2014; Watson, Vakili, & Jakubowski, 2015).
- Back analysis of observed hanging wall damage in a deep underground open stope (Vakili, Albrecht, & Sandy, 2014).

- Pillar stability assessment of a massive brittle rock mass (Vakili, 2016).
- Open pit mine slope stability assessments (Sainsbury, Vakili, Lucas, & Hutchison, 2016; Hutchison, Chambers, Gannon, & Oko-oboh, 2017; Roach & Johnston, 2020; Vakili, Watson, Abedian, & Styles, 2020; Ford, Lucas, & Vakili, 2020; Lucas, Vakili, & Hutchison, 2020; Hutchison, Morrison, & Lucas, 2020).

It should be noted that the IUCM model calibration results for open pit applications compare favourably with more traditional elastic and coupled strain-softening-Mohr-Coulomb (SSMC) models. The linear nature of the residual envelope in the IUCM with

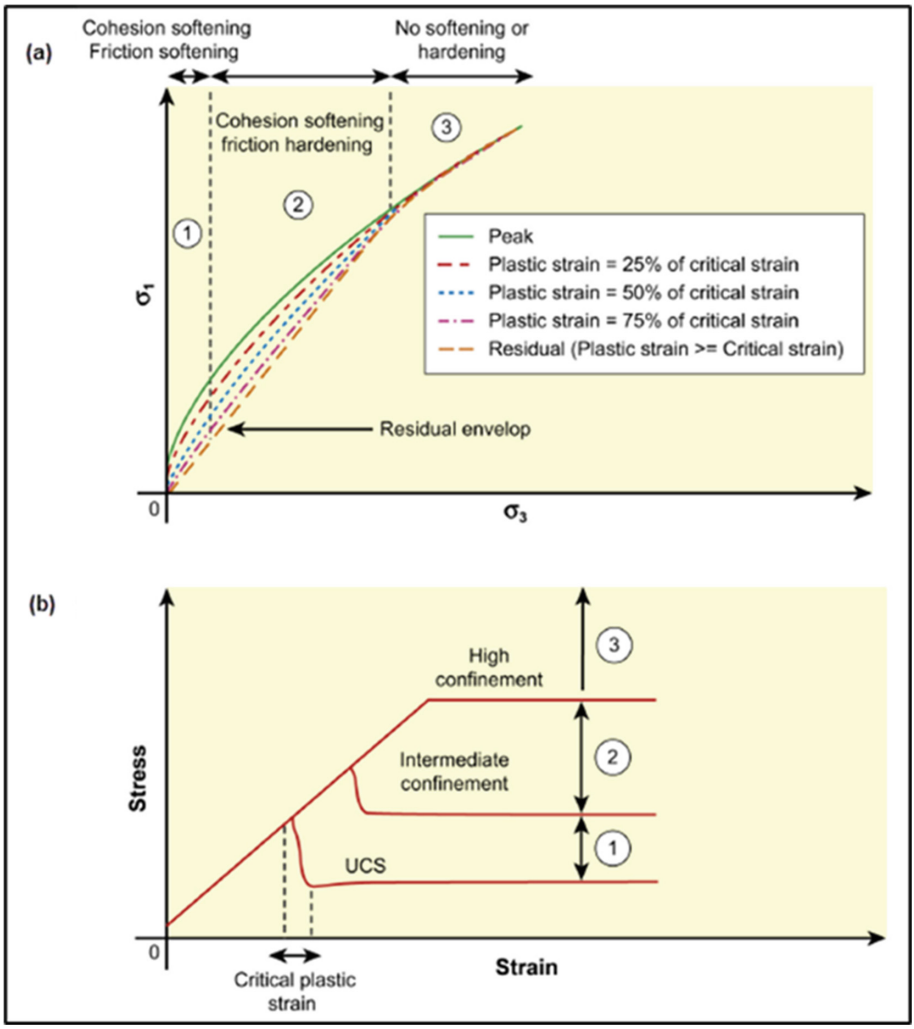

Fig. 2. Conceptual Representation of the Post Peak Rock Mass Behaviour as Represented in the IUCM; (a) Principal Stress Space; and (b) Stress-Strain Space (Vakili, 2016)

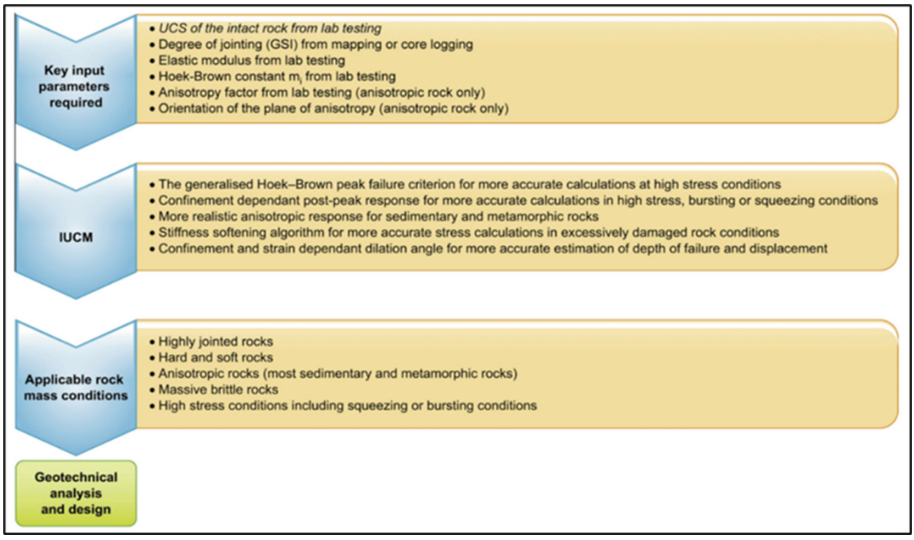

Fig. 3. Conceptual Description of the IUCM

regards to the peak Hoek-Brown envelope, replicates cohesion and friction softening at lower confinement levels and cohesion-softening and friction-hardening at high confinement levels (Fig. 2). This allows progressive failure to occur near the boundary of excavations, while limiting the propagation of yield or plasticity zones away from excavation boundaries (Vakili, 2016). A conceptual description of the IUCM is presented in Fig. 3. Where possible, the development of new theories or techniques was avoided when devising the IUCM, with a focus on well-accepted and widely applied rock mechanics techniques and theories. Furthermore, the inputs, limitations, and form of the IUCM are transparent and without ambiguities often associated with proprietary and commercially sensitive aspects of some models.

2 Open Pit Case Study

A technical review of an open pit mine located in South Western United States was undertaken after slope performance concerns were identified by site personnel. This included large tension cracks developing behind the open pit crest, along with large displacement occurring over multiple benches (particularly where deposit-scale faults coalesce with the open pit design surface and each other). Consequently, concerns were raised that the LOM design may lead to large-scale slope instability that results in unacceptably high risk associated with safety, the environment, and mine production. It is noted that the rock mass was composed of a number of pit-scale faults, and was characterised by several weak rock units. Furthermore, it was recognised that complex ground conditions and time-dependent slope behaviour demanded a more advanced assessment approach to reliably forecast slope stability and effectively manage associated hazards. Significantly, the geotechnical database was limited to some mapping data and material properties used

in a previous 2D limit equilibrium assessment. A high-quality slope monitoring database of prism and radar data was available.

2.1 Slope Stability Assessment Approach

Given the limited geotechnical database, it was decided that a two-phase approach would be used; back analysis of slope failures using slope monitoring data to derive material properties, followed by forward analysis of the LOM design. Both phases of the study applied the SlopeX plugin (Vakili, Watson, Abedian, & Styles, 2020) and FLAC3D software (Itasca, 2012), along with the IUCM.

It was assumed that the adopted vertical in situ stress is lithostatic (i.e., based on the density of the overlying rock) and that the major horizontal in situ stress magnitude equals the vertical stress magnitude, oriented east-west.

Site personnel initially indicated that ground water was located beneath the LOM open pit design. Consequently, the models did not include a phreatic surface (i.e., were run 'dry'). (It should be noted that subsequent information revealed several faults contained ground water, suggesting results may be 'best case' in affected areas.)

2.2 Phase 1: Model Calibration

A calibration model was constructed using available data from field mapping, prism, and scanning monitoring records, previous 2D limit equilibrium assessment assumptions, photographs, and conversations with site personnel. **The absence of core logging and laboratory testing data should be noted.** Therefore, an iterative approach was applied:

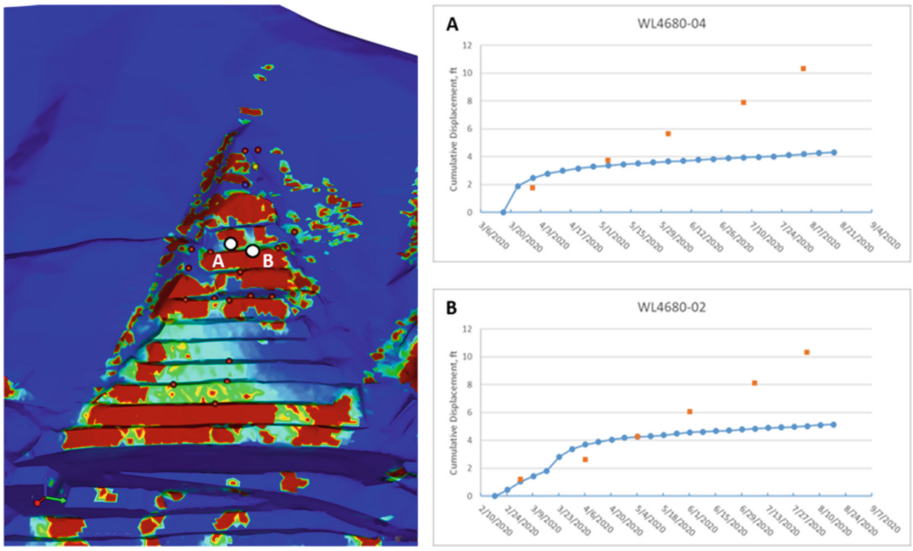

Fig. 4. Example of Phase 1 Results. Displacement of Upper Slope Compared with Prism Data (Blue = Prism Data, Red = Calibration Model)

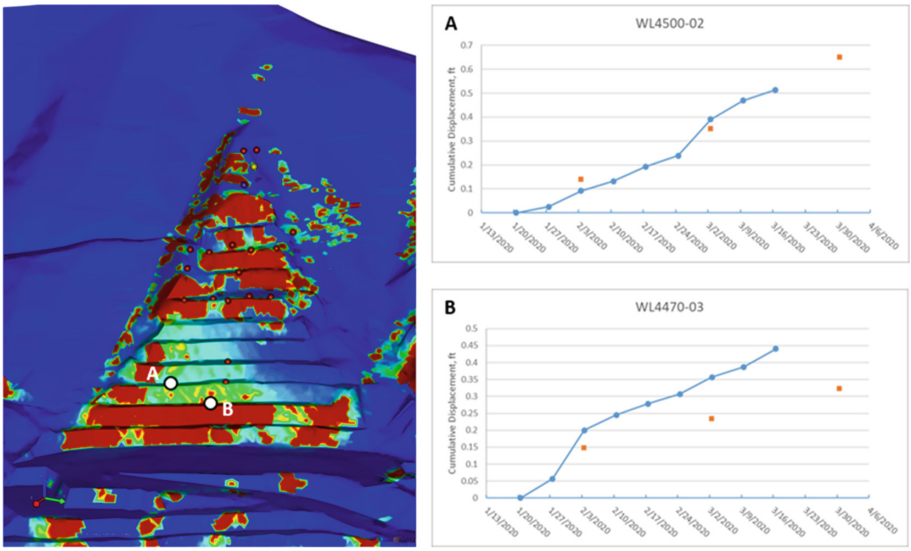

Fig. 5. Example of Phase 1 Results from July 2020. Displacement of Lower Slope Compared with Prism Data (Blue = Prism Data, Red = Calibration Model)

- Step 1: Apply rock mass and fault material properties used in previous 2D limit equilibrium assessments and exposure mapping. All key geological features (such as lithology and fault wireframes) were included in the numerical models. Results were compared to historical rock mass response to mining (i.e., slope monitoring data and photographs).
- Step 2: Access monitoring data records from the slope, selecting data points from representative areas of the slope that have undergone various ranges of displacement (i.e., small to large).
- Step 3: Based on key monitoring prism data and model runs, modify initial material properties to match monitoring data.
- Step 4: Re-run numerical models iterations until a close fit with monitoring data results and overall slope behaviour is identified.

2.3 Phase 1: Model Results

Examples of results from the Phase 1 calibration model are presented in Fig. 4 (upper slope) and Fig. 5 (lower slope). It can be seen that displacement results compare favourably with slope prism monitoring data for the same mining period. Model sections through the West Lobe slope suggest that shallow bench-scale displacement and deeper shear strength results compare favourably with slope behaviour observed in the field.

Results indicated the rock mass to be significantly weaker than was thought to be the case prior to commencement of mining in 2017 and previous 2D limit equilibrium assessments (Table 1). Therefore, initial material properties were downgraded by approximately 30% during the calibration phase. This was commensurate with historical time-dependent 'creeping' behaviour recorded on the slope.

Table 1. Material Properties Developed from Calibration Model

Geotechnical Domain		GSI	Sigci (MPa)	mi	V	coh (kPa)	φ (°)	Ei (MPa)	D	Residual Friction Angle (°)
Initial	Pinal Schist	40	35	8	0.3	230	51	12,500	0	45
	Schultz Granite	55	100	29	0.2	1200	68	2,465,600	0	45
	Apache Leap Dacite	60	70	25	0.2	850	65	2,842,700	0	45
	Apache Leap Vitrophyre	35	25	4	0.2	300	45	725,200	0	45
	Faults(1)	25	25	2	0.2	20	25	725,200	0	45
Final	Pinal Schist	30	20	8	0.3	230	51	543,900	0	44
	Schultz Granite	40	30	29	0.2	1200	68	739,700	0	44
	Apache Leap Dacite	45	21	25	0.2	850	65	852,800	0	44
	Apache Leap Vitrophyre	25	8	4	0.2	300	45	217,500	0	44
	Faults(1)	20	8	2	0.2	20	25	217,500	0	30
	Joints	5	2	25	0.2	20	25	435,100	0	29

(1)Fault Zone assumed to be ~2–3 m wide

2.4 Phase 2: Forecast of LOM Design Results

Using the material properties derived from the Phase 1 calibration model, a forward analysis model was run to assess slope behaviour of the LOM design (finishing in February 2022). The focus being to confirm that the LOM can be achieved. Results of model velocity and FoS are presented in Fig. 6 and Fig. 7 respectively.

2.5 Comparison of Forecast Model with Final Slope Behaviour

Results from the Phase 2 forecast model suggested that slope stability of the LOM open pit design met industry standard acceptability criteria:

- Overall slope angle-scale slopes were predicted to be stable throughout the LOM design, with a FoS > 1.3. This is in line with the acceptability criteria for this scale of slope with moderate consequence for failure (Read & Stacey, 2009). Model velocity results also met or exceeded recommended thresholds of 1e-7 m per timestep (Lorig & Varona, 2000).
- Inter-ramp angle-scale slopes were also predicted to be stable throughout the LOM design, with a FoS > 1.2. While this also met or exceeded the acceptability criteria

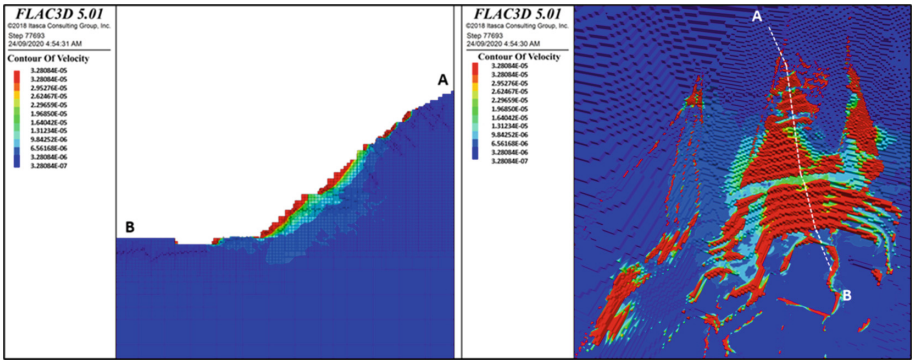

Fig. 6. Forecast Model Velocity Results of LOM (February 2022) Open Pit Design, Suggesting Shallow, Bench-Scale Failure (i.e., No Deep-Seated Slope Failure)

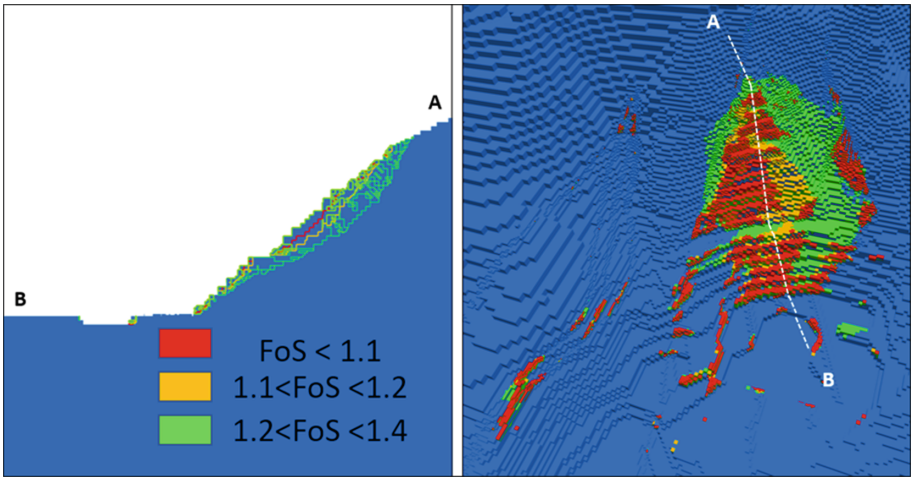

Fig. 7. Forecast Model FoS Results of LOM (February 2022) Open Pit Design, Suggesting Shallow, Bench-Scale Failure (i.e., No Deep-Seated Slope Failure)

of > 1.2 FoS for slopes with moderate consequence for failure (Read & Stacey, 2009), results suggested retrograde surficial bench-scale degradation would likely occur (particularly where deposit-scale faults coalesce with open pit design and each other).

The Phase 2 model forecast results were compared with actual LOM prism monitoring array records at completion of mining (i.e., LoM) in February 2022. As was predicted, overall slope stability was maintained, despite considerable retrograde surficial bench-scale degradation. Furthermore, the forecasted extent and location of degradation (displacement) compared closely with the actual data as can be seen in Fig. 8 and Fig. 9.

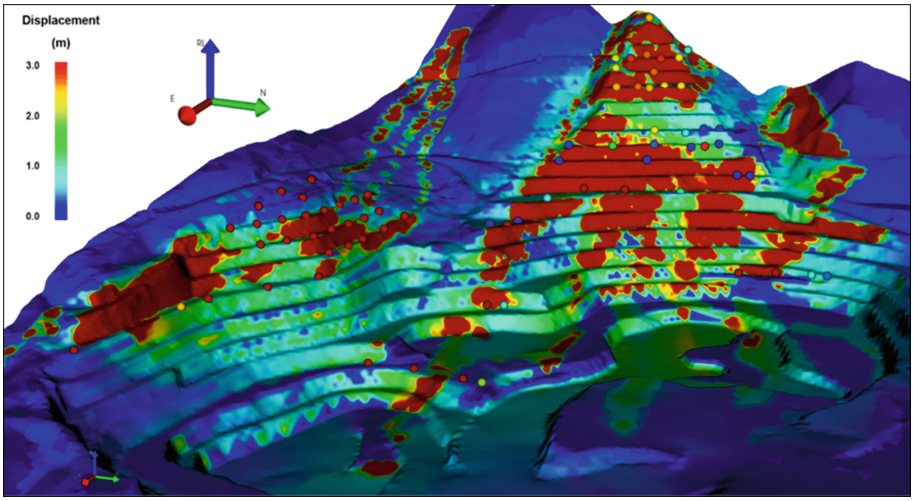

Fig. 8. Forecast Model Displacement Results Projected onto The LOM (February 2022) Open Pit Design, Along with Actual Prism Data Collected Over the LOM (Spheres)

Fig. 9. As-Built LOM Slope at Completion, February 2022 (Looking North)

2.6 Conclusion

This paper shows that an appropriate engineering assessment tool can make up for the shortcomings of a limited geotechnical database. While a well-developed geotechnical database is always preferable, it is demonstrably not necessary (i.e., statistical and engineering assessments are independent).

Other benefits of using an appropriate engineering assessment tool, as shown through this case study, include (but are not limited to):

- Confidence that the overall slope stability would be maintained throughout the LOM design.
- Confirmed the need for rigorous slope monitoring practices (including prism and radar monitoring arrays, along with regular visual inspections).
- Provide a "road map" for when and where retrograde surficial bench-scale degradation would likely occur throughout the LOM (as evidenced by the position of prism locations and forecast slope behaviour in Fig. 8).

2.7 Limitations and Uncertainties

There are a number of limitations and uncertainties that should be considered when assessing numerical modelling results of open pit slope stability. A summary of these limitations include:

- The numerical modelling often does not account for any dynamic effects associated with blasting, nor the effects of rock mass degradation caused by weathering and erosion.
- The confidence in the condition and location of modelled fault structures (which can greatly influence results).

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