SSIM 2023 – PM Dight (ed.)

doi:10.36487/ACG_repo/2335_21

Challenges of characterising a highly altered and variable rock mass for open pit slope design optimisation

T Darakjian *WSP, Canada*

S Luck *WSP, Australia*

D Luck *WSP, Australia*

C Xu *WSP, Australia*

K Moffitt *Equilibrium Mining, Australia*

S Nicoll *Newcrest Mining Limited, Australia*

D Tennant *Newcrest Mining Limited, Australia*

F Pothitos *Newcrest Mining Limited, Australia*

Abstract

Geotechnical characterisation is generally carried out by subdividing a rock mass into a number of unique geotechnical domains, each exhibiting similar geotechnical properties. Geotechnical data for each domain are then analysed to develop representative parameters for each domain. This approach is not optimal for rock masses that have significant local-scale variability in geotechnical character.

This paper documents the approach used to consider the high degree of spatial variability in geotechnical properties for the Lihir Mine, located in Papua New Guinea. The Lihir Mine is situated within the Luise volcanic crater, part of a volcanic island arc chain within the New Ireland arc-trench complex, southwest of an inactive subduction zone. The Luise volcano has previously been interpreted as a mafic to intermediate volcano, with an underlying porphyry system. Following volcanic sector collapse, the lithostatic load was rapidly decreased. This led to boiling of mineralised fluids and resulted in the formation of an epithermal gold deposit (Blackwell et al. 2010). Maar-diatreme activity then continued within the caldera, leading to the formation of diatreme eruptive centres and crater lake sediments. The geological history has resulted in a complex geological and structural environment with a high degree of geotechnical variability. Geotechnical characterisation has involved the use of geostatistical block modelling approaches to better identify the spatial variability of geotechnical properties within each geotechnical domain. The use of block modelling tools has allowed for greater resolution of input parameters for both 2D and 3D stability analyses.

Keywords: *caldera, diatreme, ordinary kriging, 3D modelling, continuity, data spatial distribution, block model, stability analysis, geotechnical characterisation, slope optimisation, slope stability, weak rocks*

1 Introduction

Newcrest Mining Limited's Lihir Mine is situated within the Luise volcanic crater, part of a volcanic island arc chain within the Papua New Guinean New Ireland arc-trench complex, southwest of an inactive subduction zone, as part of the Tabar-Lihir-Tanga-Feni island chain. The mine is host to the Ladolam deposit, which is among the youngest (<1 Ma) gold deposits in the world (Davies & Ballantyne 1987; Moyle et al. 1990; Carman 1994).

Complex tectonic development, volcanism, destructive alteration, late diatreme activity, ongoing hydrothermal activity and quaternary deposition, discussed in the following sections, present challenges for geotechnical characterisation and slope stability assessment. The development of a 3D geotechnical model, geostatistical analysis and block modelling provide greater resolution of input parameters for both 2D and 3D stability analyses.

1.1 Geological conditions

The Lihir deposit is interpreted to have been a mafic to intermediate volcano with an underlying porphyry system. Volcanic sector collapse resulted in lithostatic unloading and boiling of mineralised fluids, thereby resulting in the formation of an epithermal gold deposit and significant hydrothermal alteration (Blackwell et al. 2010). Maar-diatreme activity then continued within the caldera, leading to the formation of diatreme eruptive centres and crater lake sediments. Surficial quaternary sediments were then deposited along the coastline and within fluvial channels, with hydrothermal activity continuing today.

1.1.1 Alteration

Alteration within the Luise caldera is defined by an underlying porphyry system overlying a low-sulfidation hydrothermal system and active hydrothermal alteration. Three characteristic alteration events are evident. From oldest to youngest they are porphyry, epithermal and argillic. Porphyry alteration is defined by biotite, anhydrite and pyrite proximal to the intrusive sources, grading more distally to chlorite and carbonate assemblages within the original volcanic mound. Epithermal alteration was associated with volcanic sector collapse, lithostatic unloading and boiling, whereby hot acidic fluids resulted in dissolution of calcium-bearing minerals (anhydrite and carbonates), creating porosity within the host rocks with lateral flooding of potassium feldspar (K-feldspar), pyrite and quartz. Argillic hydrothermal alteration causes feldspars (K-feldspar, albite) to degrade to clays (illite, montmorillonite), which continues to the present day. Alteration has a significant influence on rock mass strength and other mechanical properties (Goulet et al. 2022).

1.1.2 Diatremes

Diatremes are present within the Luise caldera as late-stage phreatomagmatic eruptive centres, typically observed as barren, milled matrix to clast supported breccias containing angular felsite clasts, accretionary lapilli and floating volcanic blocks with altered zonation (Lawlis 2020). Diatremes exhibit geotechnical properties that are independent of the surrounding alteration units. In the areas of interest for pit wall design, the diatremes have been interpreted, modelled and subdivided into upper and lower diatreme to account for the distinct lithological, textural, strength and mechanical properties.

1.1.3 Structures

The Lihir island has undergone protracted deformation, with the earliest deformation initiation determined by northwest and/or southeast-directed compression and west-northwest-directed extension, linked with fault and vein formation during early porphyry-style alteration. Protracted/multistage, northwest-directed extension was dominant during porphyry- to epithermal-stage development (Sykora et al. 2018). Faults and vein development underwent reactivation under extensional conditions. Reactivation produced northeast-striking tensile to hybrid and breccia veins with high-angle dips and rhombic dilational jogs, localising high-grade mineralisation (Sykora et al. 2018).

The northeast- to east-northeast-striking faults present at both the island scale and the deposit scale were inherited from a tectonically generated structural grain. These faults have been reactivated throughout the evolution of the deposit. Similarly orientated deep-seated faults are considered to have contributed to the northeast elongation of the volcanic amphitheatre.

Further deformation comprises late-stage diatreme intrusion and subsequent subsidence, resulting in faulting and shearing along diatreme contacts, as well as post-emplacement annular gravity-driven slumping (Cole et al. 2005). This annular faulting is typically exacerbated where pre-diatreme structures, such as regional faults (in this case northeast/east-northeast faults) or those created during initial caldera formation, occur.

1.2 Geotechnical parameters required for stability assessments

In typical geological settings in open pit mines, the rock mass strength and quality improve with depth where near-surface conditions are comprised of soil and weak rock. While this is evident at the Lihir Mine, there is also a high degree of variability in the rock mass conditions within individual domains as well as zones of poor rock mass conditions at depth. This complex geotechnical environment results in difficulties designing the open pit slopes as well as subsequent optimisations if these conditions cannot be understood or well-represented in the slope design analysis. As a result, dividing the large geotechnical domains into smaller subdomains to capture the potential instability is crucial for the slope design process. Thus, a rock strength block model was required to present the spatial variability of strength and to facilitate the subdomaining of the primary geotechnical domains based on strength.

Rock mass strength is represented in Lihir design analyses using the Hoek–Brown failure criterion. This requires an estimate of the intact Hoek–Brown strength parameters and the geological strength index (GSI): a measure of rock quality (blockiness and fracture conditions). GSI is often approximated from the logged rock mass rating (RMR76, RMR89) (Bieniawski 1976, 1989) where core logging data are available to estimate GSI. However, in this case, limited drillhole data were available to calculate RMR. Specifically, historic holes did not capture all the data required and/or data were inconsistent. In the absence of sufficient, reliable RMR data, an alternative GSI estimation method based on empirical relationships using rock quality designation (RQD) and joint condition rating (Bieniawski 1989) was selected to generally capture changes in rock quality within the slope and assign broadly representative values of GSI to those zones. RQD is collected at Lihir from most drillcore, not solely geotechnical drillholes, and is therefore available for a large dataset of holes in the project area. Given the data density of RQD, a block model of RQD was required to allow a more accurate estimation of GSI rather than relying on averages for each domain.

The approach for geotechnical assessments documented in this paper relies on both rock strength and RQD block models to present the spatial continuity and variability of rock strength and rock quality. This approach allows the rock mass conditions to be defined on a block-by-block basis for stability modelling.

2 Block model methodology

The rock strength, RQD and GSI parameters were defined as part of a detailed characterisation study and were incorporated into block models to provide local rock mass properties for use in geotechnical slope stability analysis.

2.1 Geotechnical parameters

Geotechnical parameters were validated and used to develop block models to capture the spatial variability of strength and rock quality, and to facilitate subdomaining of the primary geotechnical domains based on strength.

The rock strength block model utilised validated downhole unconfined compressive strength (UCS), field estimated strength (FES) and point load test (PLT) data to estimate intact strength, where all data sources were defined by UCS strength. The rock quality block model utilised the average joint condition ratings for each geotechnical domain and block-estimated RQD values to calculate GSI for every block/element in the block model.

There are a number of sources of intact rock strength data available:

- Laboratory strength testing – UCS and triaxial compressive strength (TCS) testing on drillcore samples.
- Field strength estimates:
    - Point load testing – PLTs are undertaken on samples in the field or core shed at the time of geotechnical logging. This is a simple index test where samples are loaded to failure (either axially or diametrically).

   ○ ISRM strength rating (ISRM 2007) – A simple FES rating system was used to log the strength over geotechnical intervals.

Each of these data sources have limitations and an associated level of confidence. Laboratory compressive strength testing provides the highest level of data confidence but can be subject to sampling bias, where typically more competent (i.e. stronger) samples are selected for testing. There is also a practical limit to how many tests can be undertaken and therefore there is rarely a good spatial coverage of laboratory samples for a pit wall design, particularly for Lihir where there are large local-scale variations in strength within each geotechnical domain.

Point load testing is an index test that relies on site-specific (or geotechnical domain-specific) correlation factors to estimate the UCS of the intact rock. PLTs can be subject to the same bias as UCS testing (skewed to higher strength samples) but because of the simplicity of the test and that it can be carried out easily at the core shed, many more tests can be carried out; resulting in greater spatial coverage of strength data over the area of interest compared to laboratory testing alone.

Field-logged ISRM strength ratings (ISRM 2007) are based on physically indenting, scraping or hitting the core with a hammer. This method of strength estimation is subjective, and the accuracy of the logged values relies heavily on the training and experience of the logger. The benefit of the ISRM strength rating is that it provides a continuous log of strength downhole and is not subject to the same sampling bias as the other two data sources. In relatively weak fractured rock, ISRM strength rating logs are critical for identifying the areas of weak to very weak core that are not competent enough to sample for either laboratory or point load testing. ISRM-field estimated FES was collected according to the ISRM (2007), where the average UCS was estimated for each strength range for use in the block models.

2.2 Statistics and geostatistical approach

Prior to block model estimation, statistical and geostatistical analysis was completed to investigate data distributions and model spatial continuity. Histogram lengths for FES indicated that a composite length of 2 m would be appropriate, however, the average distance between PLT data points was 8 m. To reduce the sample imbalance between the FES and PLT/UCS data while maintaining variability in the dataset, a composite of 4 m for FES was selected. Straight compositing was applied to the PLT and UCS data points. Exploratory data analysis was conducted on the composite data for rock strength (FES/PLT/UCS) and RQD via descriptive univariate statistics and distribution comparisons. Some high values were observed within the rock strength dataset. No data were removed from the rock strength dataset prior to estimation as a decision was made to use a high value filter to restrict the influence of the high values during the estimation process. Cumulative probability plots were generated to determine stationarity. The plots display some flat, horizontal patterns of strength in some areas due to the categorical nature of the FES data.

Variographic analysis was undertaken to model spatial continuity of rock strength and RQD in each of the geotechnical domains. The objectives of the variography were to:

- Establish the directions of major rock strength, or RQD continuity, for each geotechnical domain.
- Quantify spatial continuity (variability, anisotropy and overall continuity).
- Provide variogram model parameters for use in geostatistical grade interpolation (ordinary kriging).

Directional variography requires search tolerances to be used for calculation of variograms because drillhole samples are not typically perfectly aligned in a given direction in 3D space and are not equally spaced along that direction. This requires the use of angular and distance tolerances. Figure 1 illustrates the relationship between the angular and distance tolerances (shown as generic values) with respect to the direction in which the variogram is required to be calculated.

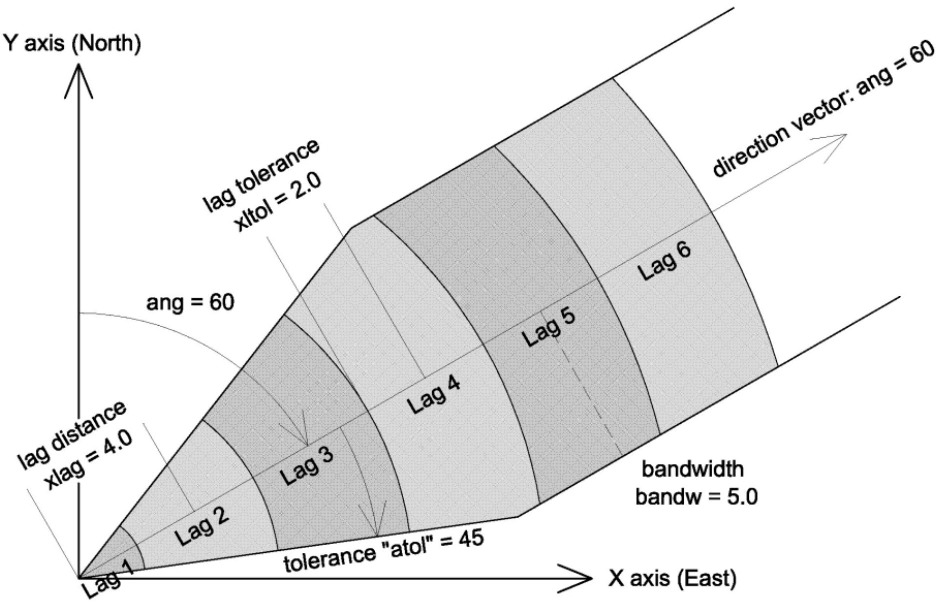

Figure 1 Definition of variogram tolerances

An overview of the variography procedure utilised is as follows:

- Directional variograms were calculated using a 40 m lag interval. This is a compromise between the X and Y direction drill spacing available in each geotechnical domain.
- The nugget variances were modelled from average downhole variograms based on a 4 m lag for rock strength and a 2 m lag for RQD, reflecting the downhole composite spacing. The downhole variogram model provided the nugget for the directional variograms.
- 3D model fitting was based on experimental variograms produced with a major axis aligned with the orientation of greatest continuity.
- Correlograms were used for variogram modelling as they provided clearer interpretable structures.
- Spherical models were used to derive all variogram parameters from the experimental variograms.

The models were fitted using WSP proprietary software, OBO (WSP 2023), which allows different sills and ranges to be modelled for each continuity axis. Fitted variogram models were then used for estimation search orientations, distances and kriging weights.

2.3 Block model and estimation

Vulcan™ (Maptek 2022) software was used to create a geotechnical block model to constrain the rock strength and RQD estimation within the geotechnical model domains. The block model was aligned to the north–south and east–west orientation. A parent cell size of 12 (X) × 12 (Y) × 12 m (Z) was used to achieve acceptable resolution of geotechnical domains and to match mining bench heights.

Ordinary Kriging (OK) was used for estimation of rock strength and RQD within the geotechnical domains using Vulcan™ (Maptek 2022) software. OK is a commonly used linear interpolation approach which considers spatial variability and incorporates variography. A conventional directional search approach was applied, with orientations defined from the variography. A two-pass approach was used. Pass 1 used the search orientations and distances from the variography. Pass 2 used the search orientations and distances from the variography multiplied by two to populate additional blocks in the model (at a lower level of confidence). A high yield filter was applied to reduce artificial inflation of the block estimate in the vicinity of high value outliers. A spatial influence distance of 24 m (X), 24 m (Y) and 12 m (Z) was used where samples were above 200 MPa.

Block estimation for rock strength and RQD was validated via visual analysis in Vulcan™ (Maptek 2022), swath plots and block mean versus sample mean statistics for each geotechnical domain. Local variations between the estimated block values and sample values were observed, reflecting the variable nature of the deposit. However, general agreement was observed between composited samples and block values for all estimation validation methods. An example of the swath plot validation for inner biotite (MPa) is provided in Figure 2. Based on this validation process, the models were deemed acceptable for use.

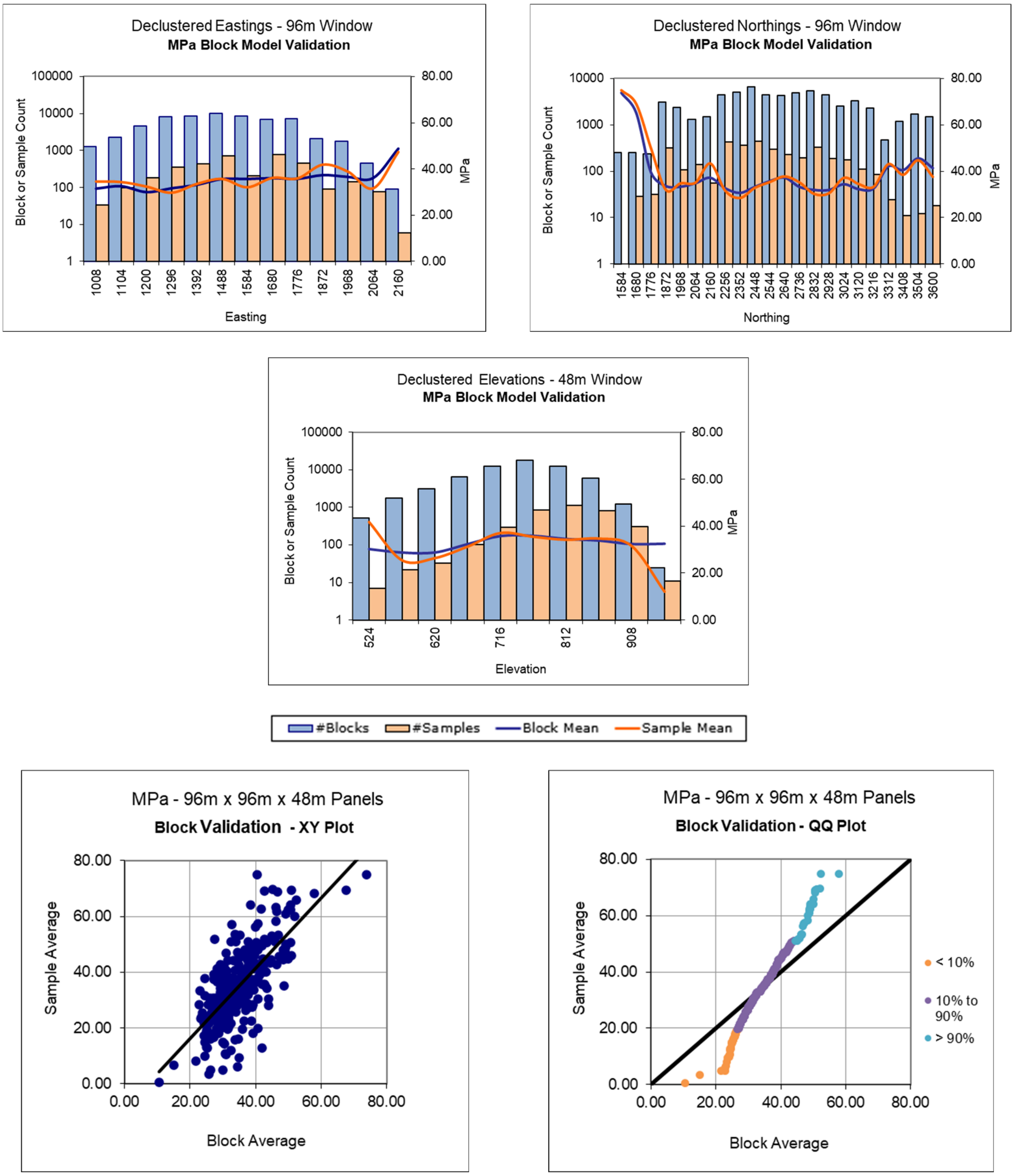

Figure 2 Examples of swath plots, Q-Q plots and XY plots for the inner biotite domain (MPa)

As previously discussed in Section 1.2, the GSI estimation method based on empirical relationships using RQD and joint condition rating (Bieniawski 1989) was selected in the absence of sufficient and reliable rock mass rating downhole data. The GSI block model was calculated using the RQD block values with the average joint condition ratings for each geotechnical domain and the following equation (Hoek et al. 2013).

$$GSI = 1.5\, JCond_{89} + RQD/2 \quad (1)$$

It is recognised that the GSI estimates developed for this study are indirect and empirical in nature and, as such, are not expected to be accurate for each and every drilling interval. Instead, these values are expected to generally capture changes in rock quality within the slope and assign broadly representative values of GSI to those zones.

3 Application to slope stability

Geotechnical slope design criteria were evaluated through a series of kinematic analyses, 2D limit equilibrium method (LEM) analyses and 3D numerical stress modelling (FLAC3D [Itasca Consulting Group Inc. 2019]). Model inputs were sourced from the characterisation of the project area, including the rock strength and rock quality block models generated.

The approach to estimating the intact rock strength and rock quality presented in the preceding sections was used to develop rock mass strength envelopes for each of the geotechnical domains. For units classified as rock (generally with intact unconfined strengths greater than 5 MPa for the purpose of this study), rock mass strength has been evaluated using a Hoek–Brown strength criterion. This non-linear failure envelope is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}(m_b \frac{\sigma_3'}{\sigma_{ci}} + s)^a \quad (2)$$

where:

σ_1'	=	maximum principal stress.
σ_3'	=	minimum principal stress.
σ_{ci}	=	UCS of the intact rock material.
m_b, s, and a	=	material constants.

m_b is a reduced value of the material constant m_i and is expressed as:

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \quad (3)$$

where:

GSI	=	geological strength index.
D	=	disturbance factor .
m_i	=	intact material constant.

Both σ_{ci} and m_i are estimated from curve-fitting the intact failure envelope using UCS, TCS and tensile strength data. The resulting parameters define the rock mass failure envelopes.

Statistics on the block model did not indicate a relationship of decreasing GSI with decreasing strength and therefore a uniform GSI is assumed for each geotechnical domain for defining the intact failure envelopes, independent of material strength. A volumetric average of GSI for each domain was calculated from the block modelled values within the project area and was selected for design analyses in all 2D LEM models. The 3D numerical model adopts the GSI values from the block model on an element-to-element basis to more accurately capture variability in the rock quality.

The soil materials (UCS <1 MPa) and the weak rock transitional materials (UCS = 1–5 MPa) have been characterised using a Mohr–Coulomb failure criterion, a more appropriate failure criterion for soils. Strength parameters for soil materials have been based on laboratory testing. For the transitional (weak rock) materials, empirical judgement has been used to downgrade intact strengths to account for the weakening impact of fabric/structure on the overall strength of the materials.

The ranges of strength for each subdomain have been broadly adopted from the ISRM strength rating ranges to maintain consistency with logging procedures. Since a large proportion of the argillic, diatreme and Ailaya advanced argillic units fall within the R2 strength category (5–25 MPa), a further subdivision was deemed necessary in those units to differentiate between material with an unconfined strength of 5–15 MPa from material with an unconfined strength of 15–25 MPa. Strength categories used for subdomaining the geotechnical model are listed in Table 1.

Table 1 Strength categories used for subdomaining the geotechnical model

Strength subdomain	ISRM strength rating	UCS range (MPa)	Rock mass failure criterion used in analysis
Very weak	R0 – R1	0 – 5	Mohr–Coulomb
Weak (low)	R2a	5 – 15	Hoek–Brown
Weak (high)	R2b	15 – 25	Hoek–Brown
Medium strong	R3	25 – 50	Hoek–Brown
Strong	R4	>50	Hoek–Brown

Geotechnical parameters were provided for each alteration unit and strength subdomain considered in stability assessments. The subdomain properties were developed primarily for the 2D LEM analyses where consistent properties have been applied to each geotechnical subdomain in those analyses. For the 3D numerical model, the GSI and UCS are input on a block-by-block basis from the block model. The m_i value is determined based on the UCS (the m_i is adopted from the appropriate strength envelope for that material).

The block models and associated geotechnical properties are able to better capture spatial variability of strength rather than using uniform 'mean' strengths for each geotechnical domain. This has allowed for more detailed optimisation of slope designs based on anticipated conditions to be exposed in the pit slopes. Areas where ground conditions were anticipated to be less adverse for slope stability allowed for localised optimisation of the design slopes.

3.1 Slope design criteria development

A combination of kinematic and generic 2D LEM methods were used to develop inter-ramp (IR) and bench designs that meet the defined design acceptance criteria (DAC) for the Lihir Mine. A 3D review of the subdomain models and the interaction with proposed pit shell designs allowed for identification of anticipated rock mass conditions exposed in the slope and directly behind the slope to guide the selection of rock mass strength parameters to be considered in the design. Figure 3 presents a plan view of the preliminary pit design, an oblique view of the west wall and the corresponding subdomain model legend as an example of exposed rock mass properties on the pit wall for use in the slope design criteria development.

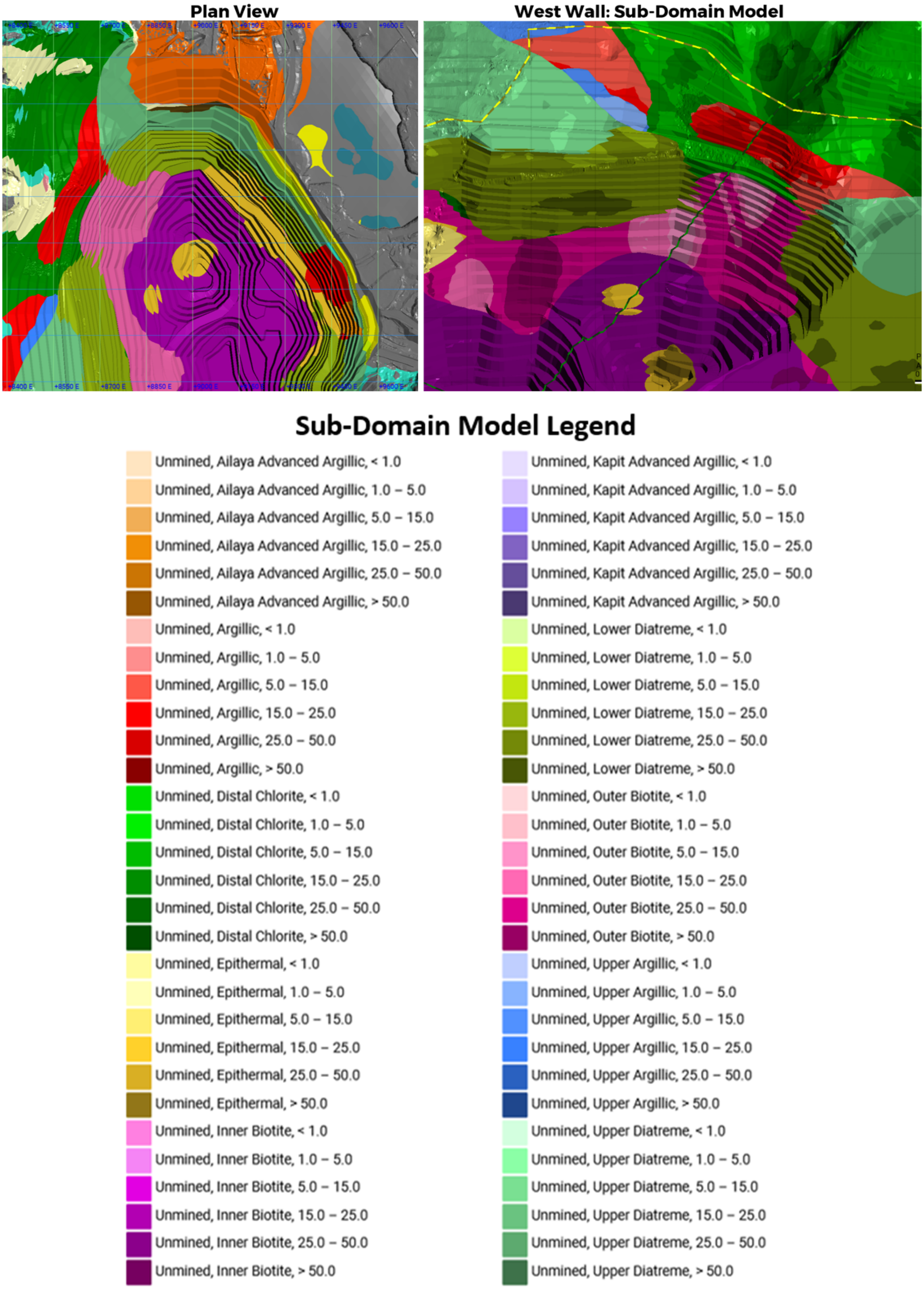

Figure 3 Plan view of the preliminary slope design, oblique view on the west wall and legend of the exposed subdomain model

For IR design, the achievable IR slope angles for both kinematic and rock mass mechanisms (meeting the respective DAC) were evaluated for each design domain. This rock mass and kinematic critical failure mechanism review was conducted to inform the selection of achievable inter-ramp angle (IRA) for each domain. The controlling mechanism was identified as the one that resulted in the shallowest slope angle required to meet the DAC.

3.1.1 Generic 2D limit equilibrium method

Simple generic 2D LEM models of representative bench and IR stack heights were developed for each rock mass domain and used to conduct a rock mass stability review of each alteration unit as part of the critical mechanism review. Representative rock mass strength parameters were adopted for each rock mass strength subdomain exposed on the preliminary design pit shell.

The IRA results were summarised for the IR DAC (i.e. Factor of Safety [FOS] = 1.2) and provided for each alteration unit. These results were then compared to outcomes of the conventional kinematic analysis (described in Section 3.1.2) to define the critical failure mechanism for each domain.

3.1.2 Conventional kinematic analysis

Kinematic analyses on a bench and IR scale were undertaken to assess potential structural controls to the proposed preliminary slope design. Design domains were selected based on geotechnical domain, structural domain and wall orientation.

Dips (Rocscience Inc 2021a) was used to initially identify which of the identified structure sets form potential planar, wedge or toppling structural mechanisms. Identified potential failure mechanisms were further analysed using RocPlane (Rocscience Inc 2021b) for planar, and SWedge (Rocscience Inc 2021c) for wedge mechanisms, to calculate Probability of Failure for various slope dip angles. Potential for toppling mechanisms was not identified in this review. A probabilistic analysis was employed which uses statistical distributions of structural orientations derived from core logging data to randomly generate plane or wedge failure geometries with a given slope orientation. The FOS is calculated for each statistically realised block, and the resulting FOS distribution is used to calculate Probability of Failure for slope dip angles assuming that failure occurs at FOS = 1.0. These results were then compared to outcomes of the generic 2D LEM analyses (described in Section 3.1.1).

3.1.3 Slope design recommendations

The slope design recommendations represent the achievable IRAs from the critical governing failure mechanisms identified from comparison of the kinematic and generic 2D LEM analyses. The controlling mechanism was identified as the one that resulted in the shallowest slope angle required to meet the DAC.

The results of the analyses allowed for slope design parameters specific to the anticipated conditions. These designs were then used to regenerate the ultimate pit shell for review in detailed 2D LEM and FLAC3D (Itasca Consulting Group Inc. 2019) models.

3.2 2D limit equilibrium method

Once slope designs were developed using the slope design criteria, 2D LEM models were constructed using the Rocscience program Slide2 (Rocscience Inc 2021d). Sections for 2D analyses were selected along the most adverse areas of the designs considering the geotechnical characterisation, subdomain model and structural conditions. Figure 4 provides an example of a section selected along the west wall targeting the weaker upper argillic, argillic, and upper diatreme material anticipated to be exposed in the upper slopes. The subdomain legend in Figure 3 corresponds to the subdomain model shown in Figure 4.

For each slope stability analysis section, the material properties were assigned based on anticipated exposed rock mass conditions from the subdomain model. This allowed for specific and targeted slope stability review along areas of the pit wall deemed most adverse to informing potential ‘worst-case’ conditions in the design.

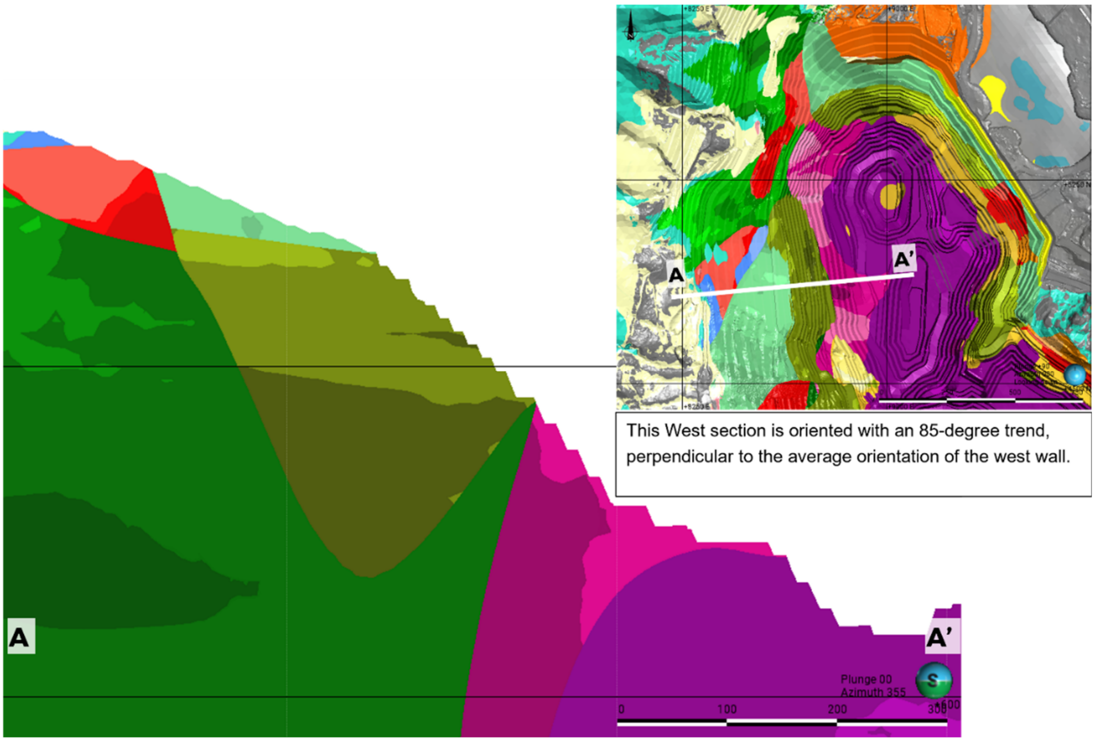

Figure 4 A 2D LEM section along section A–A′, selected along the west wall of the pit, with the subdomain model

3.3 3D numerical modelling

The 3D numerical stress model using FLAC3D (Itasca Consulting Group Inc. 2019) was developed based on the geological, structural and strength data as discussed in previous sections. In this model, the GSI and UCS properties were prescribed discretely on a block-by-block basis from the block models. Rock mass strengths were assigned on an element-by-element basis using the UCS and GSI from the block models to delineate the appropriate strength category, with the m_i assigned based on the appropriate intact strength curve fit. This model focused on capturing the deformation patterns and critical failure mechanisms which develop through predictive forward analysis of planned mining. Figure 5 provides an example of the FLAC3D (Itasca Consulting Group Inc. 2019) model inputs showing anticipated exposed geotechnical domains on the ultimate pit shell.

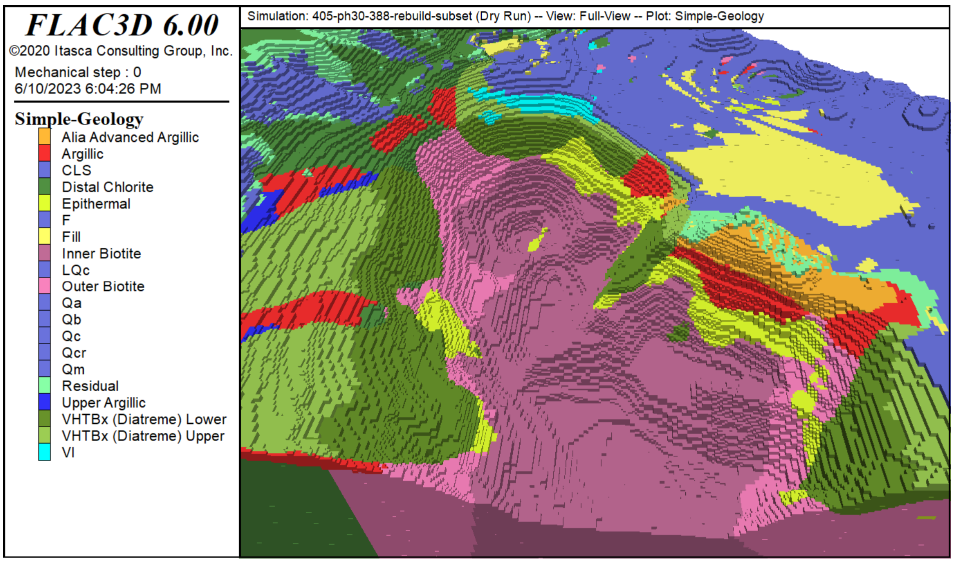

Figure 5 FLAC3D (Itasca Consulting Group Inc. 2019) model of the ultimate pit and geotechnical domain

4 Conclusion

This paper presents a case study of incorporating block modelling for geotechnical characterisation and slope optimisation in a complex geological setting. The geotechnical characterisation and block modelling for slope stability assessments consist of the following steps:

- Review and understand geological conditions of the project site and identify the potential source of variability.
- Identify geotechnical parameters required for subdomaining.
- Collect and validate the geotechnical data for subdomain model inputs.
- Conduct statistical analysis. This step includes understanding the distribution and limitations of data and the determination of composite length.
- Conduct geostatistical analysis. This step consists of the determination of the direction of continuity and quantifying parameters for spatial continuity for each domain and choosing an interpolation method.
- Generate the block model. This step involves block size selection and validation of the model output.
- Conduct quality checks and validate the block model to confirm appropriate representation of subdomain conditions.
- Utilise the block model and associated strength parameters in slope stability assessments.

The block modelling approach for slope design described in this paper offers several advantages:

- Adequate representation of geotechnical conditions – The block modelling approach has facilitated a better presentation of the spatial continuity and variability of geotechnical properties within each geotechnical domain when compared to using mean or median values as input parameters on a domain-basis.
- Identification of potential failure surfaces with higher statistical confidence – Data is based upon spatial distribution of geotechnical parameters influencing failure mechanisms by domain.
- Improved stability analysis – The use of block modelling tools has allowed for greater resolution of input parameters for both 2D and 3D stability analyses.
- Enhanced geotechnical design – Incorporating block models allows for a more comprehensive geotechnical design of slopes, facilitating assessments of the strength and behaviour of various geotechnical domains.
- Effective risk management – The block model approach provides a clearer understanding of potential failure mechanisms and their probability of occurrence. By considering the geotechnical complexities of the slope, engineers can identify high-risk areas and develop targeted monitoring and mitigation strategies. This allows for proactive measures to be implemented, reducing the likelihood of slope failures and minimising potential consequences.
- Optimal resource allocation – Resources may be allocated more efficiently during slope design and construction. By adequately characterising the geotechnical conditions, unnecessary excavation or reinforcement measures can be avoided in areas where the slope is inherently stable. This optimisation of resources helps reduce costs, increase project efficiency and improve overall design sustainability.

Updates and validations of the model over the mine life are also required in the reconciliation process upon acquisitions of new data for future design studies. Furthermore, multiple indicator kriging (MIK) may be investigated for future work to further improve the block modelling methodology.

The MIK method is capable of modelling highly variable data using non-linear transformation and estimation of the local distribution at each unsampled location, providing a measure of local uncertainly via the variance of the distribution. Conditional simulation methods may also be used to model local uncertainty through generation of equiprobable realisations and could also be considered as a more rigorous approach to provide risk-quantified local estimates.

Acknowledgement

The authors acknowledge and thank Newcrest Mining Ltd for its logistical and technical support with the compilation of this paper.

References

Bieniawski, ZT 1976, 'Rock mass classification in rock engineering', in ZT Bieniawski (ed.), *Symposium Proceedings of Exploration for Rock Engineering*, vol. 1, pp. 97–106, A.A. Balkema, Cape Town.

Bieniawski, ZT 1989, *Engineering Rock Mass Classifications,* Wiley, New York.

Blackwell, JL, Cooke, DR, McPhie, J & Simpson, KA 2010, 'Lithofacies associations and evolution of the volcanic host succession to the Minifie ore zone, Ladolam gold deposit, Lihir Island, Papua New Guinea', *Economic Geology*, vol. 109, no.4, pp. 1137–1160, https://doi.org/10.2113/econgeo.109.4.1137

Carman, GD 1994, *Genesis of the Ladolam Gold Deposit, Lihir Island, Papua New Guinea,* Doctoral dissertation, Monash University, Melbourne.

Cole, JW, Milner, DM & Spinks, KD 2005, 'Calderas and caldera structures: a review'*, Earth-Science Reviews*, vol. 69, no. 1–2, pp. 1–26, https://doi.org/10.1016/j.earscirev.2004.06.004

Davies, RH & Ballantyne, GH 1987, 'Geology of the Ladolam gold deposit Lihir Island, Papua New Guinea', *Proceedings of the Pacific Rim Congress 87*, Australasian Institute of Mining and Metallurgy, Melbourne, pp. 943–949.

Goulet, A, Grenon, M & Hadjigeorgiou, J 2022, 'Understanding the impact of alteration on rock mass strength'*, Geotechnical and Geological Engineering*, vol. 40, no. 5, pp. 2533–2552.

Hoek, E, Carter, TG & Diederichs, MS 2013, 'Quantification of the geological strength index'*, Proceedings of the 47th US Rock Mechanics/Geomechanics Symposium*, American Rock Mechanics Association, Alexandria.

ISRM 2007, *The ISRM Suggested Methods for Rock Characterisation, Testing and Monitoring: 1974-2007*, Springer Cham, Cham.

Itasca Consulting Group Inc. 2019, FLAC3D — *Fast Lagrangian Analysis of Continua in Three-Dimensions*, version 6.0, computer software, Itasca, Minneapolis, https://www.itascacg.com/learning/tutorials/flac3d-6-0-built-in-model-generation-tools-and-workflow

Lawlis, EEC 2020, *Geology of the Kapit NE and Coastal Ore Zones, Lihir Gold Deposit, Papua New Guinea*, Doctoral dissertation, University of Tasmania, Hobart.

Maptek 2022, Vulcan, version 2022.4.1, computer software, Maptek, Denver.

Moyle, AJ, Doyle, BJ, Hoogvliet, H & Ware, AR 1990, 'Ladolam gold deposit, Lihir Island', *Australasian Institute of Mining and Metallurgy*, vol. 14, pp. 1793–1805.

Rocscience Inc 2021a, *Dips*, version 8.0, computer software, Rocscience Inc, Toronto, https://www.rocscience.com/software/dips

Rocscience Inc 2021b, *RocPlane*, version 4.0, computer software, Rocscience Inc, Toronto. https://www.rocscience.com/software/rocplane.

Rocscience Inc 2021c, *SWedge*, version 7.0, computer software, Rocscience Inc, Toronto. https://www.rocscience.com/software/swedge.

Rocscience Inc 2021d, *Slide2*, version 9.0, computer software, Rocscience Inc, Toronto. https://www.rocscience.com/software/slide

Sykora, S, Selley, D, Cooke, DR & Harris, AC 2018, 'The structure and significance of anhydrite-bearing vein arrays, Lienetz orebody, Lihir gold deposit, Papua New Guinea', *Economic Geology*, vol. 113, no. 1, pp. 237–270, https://doi.org/10.5382/econgeo.2018.4550.

WSP 2023, *OBO*, computer software, version 16.01, Golder Associates, Toronto.