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Advantages of geotechnical data collection during pit wall construction for open pit slope design optimisation

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Abstract

Pit slope design is often considered final at the detailed design for life of mine pit walls. During construction of these walls, critical information from newly exposed benches can be overlooked, particularly if a slope reconciliation plan has not been produced. The absence of slope reconciliation may lead to unexpected slope instability, which could result in not achieving design acceptance criteria values, increased effort being required to maintain bench width for management of rockfall risk, mitigation measures to manage loss of benches, step-outs to manage multi-bench and inter-ramp instabilities, or complete loss of a pit wall.

The purpose of geotechnical rock mass and structural models generated for slope stability analysis is to predict the conditions at the time of construction. As these models are only as good as the data provided, they often include gaps in knowledge where little or no data is available. This is particularly challenging in areas of variable rock mass and structural conditions.

This paper documents the approach used to consider the high degree of spatial variability in geotechnical properties during construction of the Lihir Mine, located in Papua New Guinea. Lihir Mine is geotechnically characterised through geostatistical block modelling approaches to better identify the spatial variability of geotechnical properties within geotechnical domains. The use of block modelling tools has allowed for greater resolution of predicted input parameters for slope stability analyses. During construction, variation of predicted versus as-built geotechnical conditions are observed and documented as part of the slope reconciliation study. Onsite geologists and engineers collect key information from exposed faces through geological mapping, rock core logging, observations during construction and monitoring data review, where these data are compared to predicted conditions. The magnitude of variation is then assessed to determine its potential impact on slope stability and areas where additional slope stability analyses may be required.

Keywords: *stability analysis, geotechnical characterisation, slope optimisation, slope stability, geotechnical model*

1 Introduction

Newmont Corporation's Lihir Mine (Lihir) 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 amongst 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 present challenges for geotechnical characterisation and slope stability assessment. These complex conditions are represented through a 3D geotechnical model

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involving geostatistical analysis and block modelling to provide greater resolution of predicted conditions for use in 2D and 3D stability analyses (Darakjian et al. 2023). During the construction phase of this study, validation of predicted versus as-built conditions was conducted through several onsite data collection methods. This paper provides an overall summary of the methodology adopted and general outcomes of the reconciliation work completed.

2 Predicted conditions

In typical geological settings in open pit mines, the rock mass strength and quality improve with depth where near-surface conditions comprise soil and weak rock. While this is evident at Lihir there is also a high degree of variability in the rock mass conditions within individual domains, including 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 (Darakjian et al. 2023). Thus, a 3D geotechnical model was developed to represent both project-wide and localised conditions to best predict the geological, rock mass and structural environments.

This paper presents a high-level summary of the development of the geotechnical and structural models, and should be read in conjunction with the referenced Darakjian et al. (2023) and Stoch et al. (2023) papers for further detail.

2.1 Geotechnical model

This 3D geotechnical model was developed by delineating large geotechnical domains and identifying areas within each domain exhibiting a high degree of spatial variability, creating smaller subdomains. Geotechnical domains were generated based on geology and alteration, and the smaller subdomains were generated based on rock mass strength and the geological strength index (GSI).

2.1.1 Rock strength and geological strength index block model

The rock mass strength and GSI block models were developed based on available downhole data, including unconfined compressive strength (UCS), field estimated strength (FES), point load test, rock quality designation (RQD) and joint condition (JCon) data, to estimate intact strength and the GSI.

In the rock mass strength model, all data sources were converted to an equivalent UCS strength and assigned to blocks. For the GSI model, downhole Rock Mass Rating (RMR76, RMR89) (Bieniawski 1976; Bieniawski 1989) data was not sufficient to generate GSI estimates by block due to the limited number of detailed geotechnical logs. Therefore, an alternative GSI estimation method based on empirical relationships, using a larger dataset of reliable RQD information and average JCon ratings (Bieniawski 1989) by geotechnical unit (Equation 1) (Hoek et al. 2013), was selected to generally capture changes in rock quality within the slope and assign broadly representative values of GSI to those zones.

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

Using the various data sources, statistical and geostatistical analysis was conducted to investigate data distribution and model spatial continuity. Rock mass strength and GSI block models were generated using Vulcan (Maptek 2022) software to constrain the rock strength and quality within the geotechnical model domains. A parent cell size of 12 (X) by 12 (Y) by 12 m (Z) (a 12-m^3 block) was used to achieve an acceptable resolution of geotechnical domains and to match mining bench heights (Darakjian et al. 2023).

2.1.2 Rock mass model

As the rock strength block model was constrained within geotechnical model domains it was then superimposed on the geotechnical model to create 3D visualisation of spatial variability within each geotechnical domain, as shown in Figure 1.

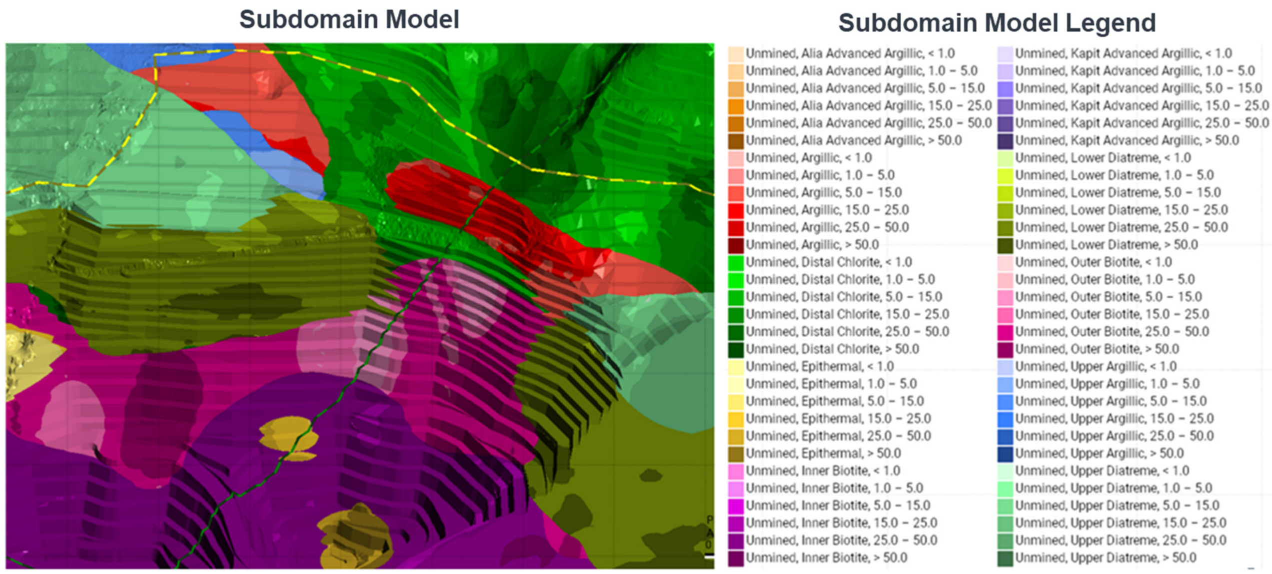

Figure 1 Exposed subdomain model on design pit wall and associated legend (Darakjian et al. 2023)

These subdomain models are visually represented by rock mass strength and GSI bins summarised in Table 1.

Table 1 Unconfined compressive strength (UCS) and geological strength index (GSI) subdomain model bins

UCS bins	GSI bins
<1 MPa	<1
1–5 MPa	1–20
5–15 MPa	21–40
15–25 MPa	41–60
25–50 MPa	61–80
≥50 MPa	81–100

2.2 Structural model

Given the complex environment at Lihir, a comprehensive and fully constrained structural model was required to best predict structural conditions. The model was based on historical understanding of the tectonic evolution, late diatreme intrusion, significant metasomatic overprint, ongoing hydrothermal activity and weathering of the country rock.

To address these challenges, an evidence-based approach was adopted for structural model development and characterisation. The approach integrated disparate datasets, including high-resolution photogrammetry, field mapping data, core logs and downhole photographs (Stoch et al. 2023). Structural delineations were informed by both surface-derived data – like geological traverses and field mapping – and subsurface remote data, including geophysical logs and rock core imagery. By combining these complementary data sources, the model provides a robust interpretation of structural features, capturing the spatial variability, continuity, geometry and key characteristics.

Each structural feature was plotted in a 3D point cloud and categorised based on its identified structure type. Features exhibiting similar characteristics, conforming with the understanding of the local geological and structural evolution, were grouped and assigned specific fault designations. The grouped data were subsequently utilised to create individual faults in 3D, ensuring they align with the historical understanding of the structural conditions and fault trends. Crosscutting relationships were then identified and interpreted to understand the hierarchy of faulting and to appropriately terminate fault sets where they intersect, completing the fully constrained structural interpretation shown in Figure 2.

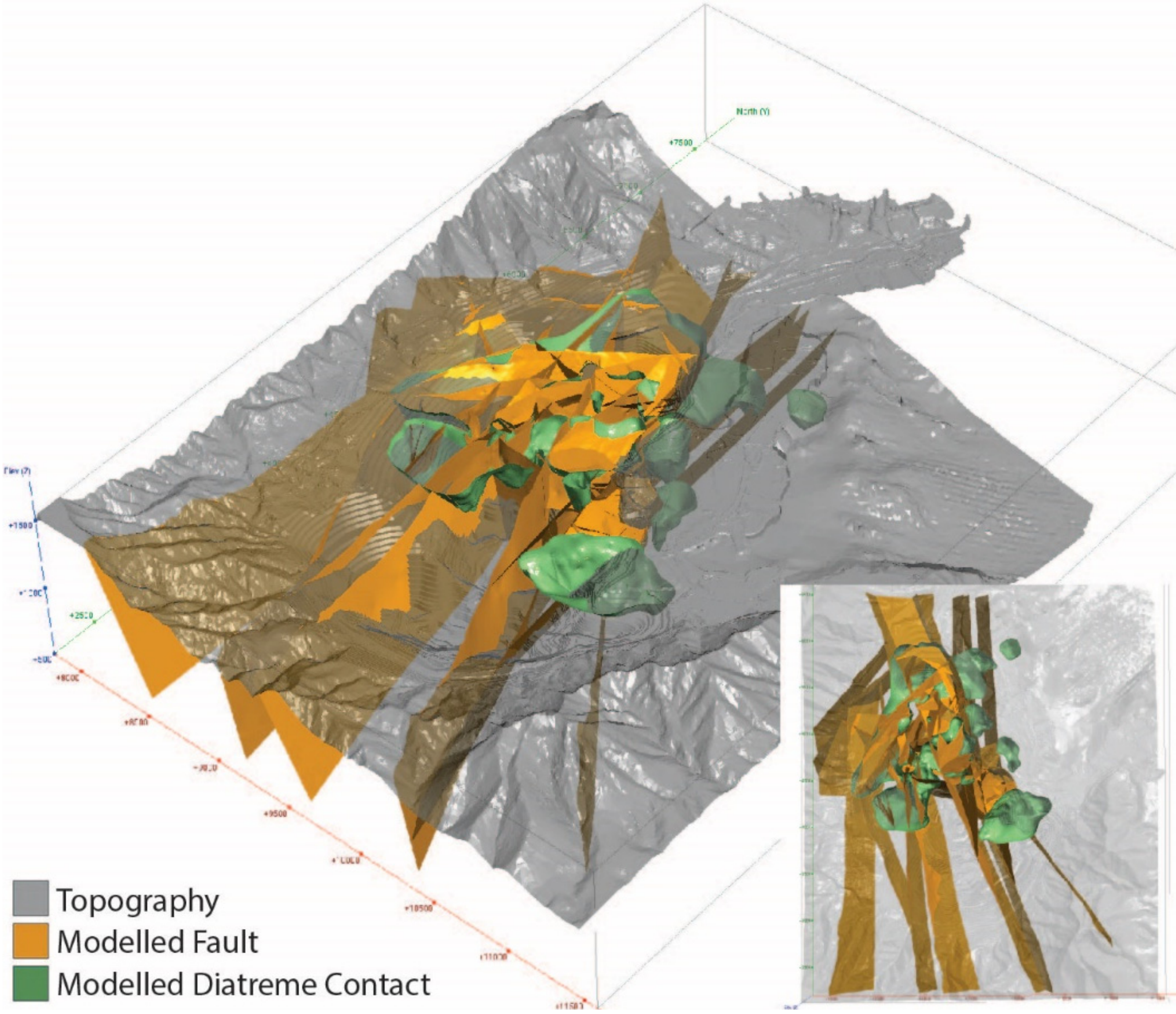

Figure 2 Modelled fault network across the Lihir Mine project extents, incorporating all available geological, geophysical and geotechnical data (Stoch et al. 2023)

2.3 Limitations and uncertainty

The 3D geotechnical and structural models developed to predict site conditions are based on various data sources collected from site, where each data source used has limitations and an associated level of confidence. Limitations in downhole data include sampling bias in laboratory strength testing (where typically more competent core is collected), subjectivity in FES and the performance of downhole equipment (i.e. downhole orientation tools). Surface data also presents challenges such as the resolution of photogrammetry imagery, impacts of blasting and scaling on exposed walls, and subjectivity in the delineation of features. Data limitations and uncertainty, and their impacts on geotechnical and structural models, can often be improved by utilising larger datasets from various data sources.

Further limitations in this approach include the resolution of the geotechnical model and the spatial variability of the structural model. The geotechnical block model was developed with a resolution of 12 m^3 blocks, matching the height of one single bench. This allowed for fast 3D numerical modelling run times while maintaining an appropriate project-wide model resolution. While this represents the general spatial variability within geotechnical domains, it does not capture smaller-scale zones of varying geotechnical conditions as described in further detail in Section 3. The structural model was developed by data points collected and categorised in a point cloud. The space between points generating an individual fault may be a large enough gap to misrepresent smaller localised variability regarding the location and orientation of the fault, which is also described in further detail in Section 3.

2.4 Data confidence

Prior to construction a detailed design study was conducted to identify potential data gaps and appropriately reconcile these areas through further onsite data collection. Priority of data collection was based on areas where previous kinematic, 2D limit equilibrium (LE) and 3D Fast Lagrangian Analysis of Continua (FLAC) (ITASCA 2024) modelling indicated potential slope instability. Additional data was collected through mapping, photogrammetry, drilling and downhole instrumentation to minimise data gaps, within reason. During construction, onsite data collection was conducted by engineers and geologists to reconcile as-built conditions to predicted conditions, identify new observations that may impact pit designs, and adjust 3D geotechnical and structural models locally, where required.

Collecting as-built data during construction significantly improves confidence of the geotechnical model, which in turn improves confidence of pit designs. This was particularly important for this case study as the pit design targets high-grade ore, requiring steepening of pit walls in areas including zones of weaker rock mass. Where variations are identified and analysed in slope stability models, operations benefit in two key ways: by identifying opportunities for optimisation of designs and anticipating potential failure mechanisms. Onsite data collection allowed for up-to-date high-confidence data for redesigning stability analyses in areas where the variation of as-built versus predicted conditions exceeds the range of existing sensitivity analysis. This proactive approach allows for timely design adjustments, helping to minimise risk and impacts associated with slope instability. Additionally, proactive reconciliation of predicted designs at the beginning of construction enables the optimisation of slopes at the start of mining, where slight adjustments to the steepness of benches can eventually add up to significantly reduced waste stripping and improved ore extraction.

3 As-built conditions

The as-built conditions were documented through mapping, photogrammetry, drilling and monitoring data collected during the construction phase, which was then compared against predicted conditions (see Section 2) to determine if updates to slope stability models were required. These comparisons required consideration of the resolution of the geotechnical models used for analysis, which were based on block models with block sizes of 12 m^3, compared to mapping and downhole drilling data which was collected on the metre-by-metre scale from drillholes and flitch exposures that were typically 4 to 6 m high and approximately 100 m long. As a result, comparisons were reviewed both broadly on an inter-ramp and overall wall scale as well as on a smaller bench scale to determine the extent of model update required.

The criteria for defining a magnitude of variation between predicted and as-built conditions as being significant enough to warrant potential updates to slope stability analyses were developed mostly through expert site-specific knowledge and understanding of slope performance from previous pit phases in similar rock. These empirically developed, evidence-based general criteria were defined as follows:

- Geological contacts were located more than 12 m from their location as predicted by the design model.
- Rock strength or GSI differed from the design model prediction beyond the range of existing sensitivity analysis impacting the design acceptance criteria (DAC).
- Faults were located more than 10 m from their location predicted by the design model.
- Faults were identified and projected to be oriented adversely for slope stability.

Model and slope design updates are particularly important where the variations impact the DAC beyond the range of existing sensitivity analysis, especially for weak rock domains and exposures. Any one of the above criteria justify an update to the geotechnical model. The following sections describe the onsite data collection methods and observed differences between predicted and as-built conditions.

3.1 Mapping

Bulk mining was undertaken in flitches involving mining stages of 4 to 6 m heights to achieve an overall bench height of 24 m. After completion of the excavation, geological mapping of the exposed face was undertaken to allow encountered conditions including geology, strength, GSI, defects and major structures to be recorded. Photogrammetry was also used to capture the post-mining condition of the face, using both aerial drone and manual (on foot) methods to allow further detailed interpretation of encountered conditions for both the current mining phase and future geological model development.

Although more complex 3D methods (i.e. LeapFrog [Seequent 2024]) were used for the model generation, simpler methods were also adopted onsite where an initial comparison of the mapped conditions was undertaken using spreadsheets for more efficient data collection (Figure 3). These spreadsheets were then compared to the 3D models to identify zones where predicted conditions are not generally representative of as-built conditions. The reconciliation of the encountered ground conditions were required to confirm assumptions which fed into the kinematic, 2D LE and 3D FLAC (ITASCA 2024) modelling work, where they were used to assess bench scale, inter-ramp and overall pit slope stability (refer to Section 4).

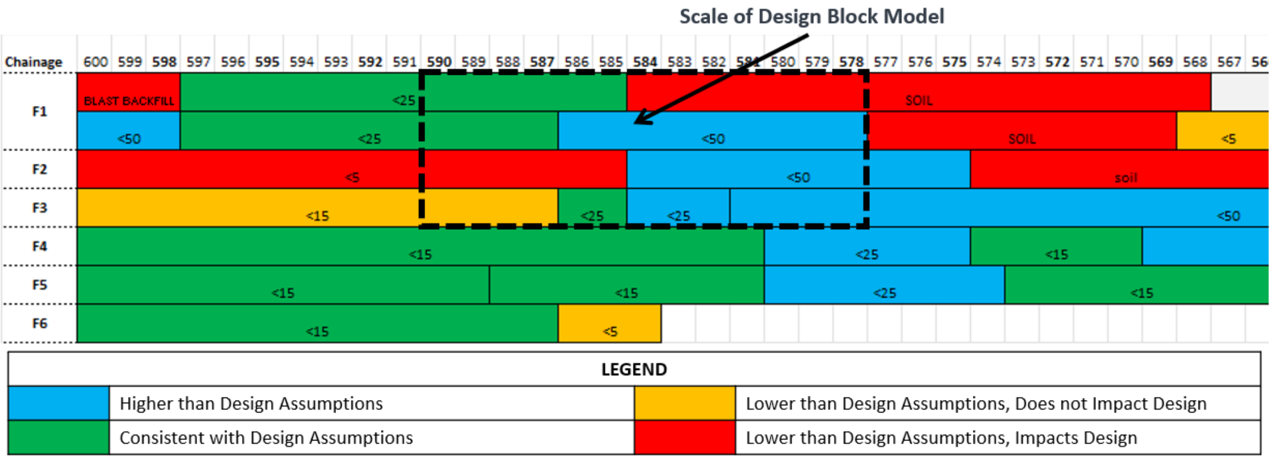

Figure 3 Example of site mapping records being used to reconcile strength against a design block model

Further detail on the GSI, rock strength and structural data collection is provided in the following sections.

3.1.1 Geological strength index

The GSI was visually assessed using the Marinos & Hoek (2000) GSI chart as part of the mapping processes and was reconciled with the block model predictions as shown in Figure 4. The GSI is presented as a mapped minimum and maximum as it is typically mapped in the field within GSI bin ranges, defined in Table 1.

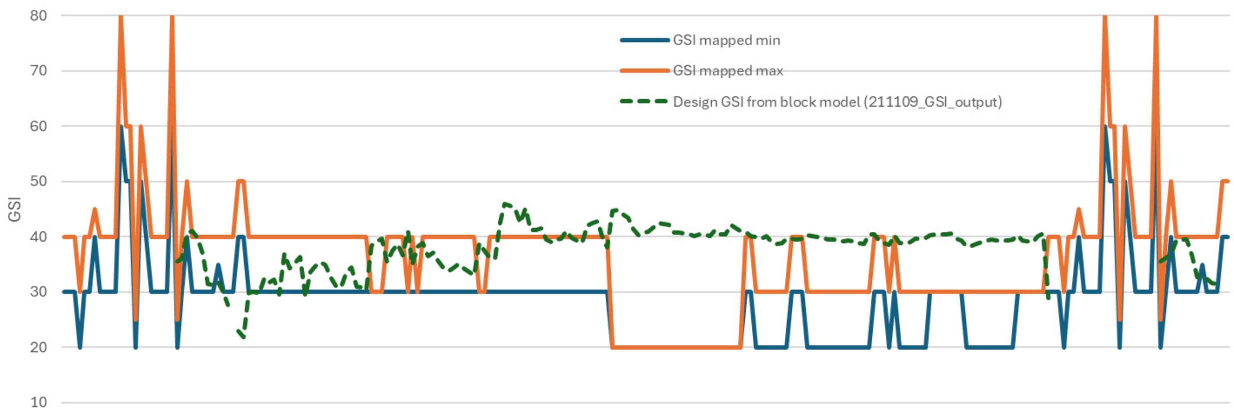

Figure 4 Example of mapped geological strength index (GSI) as a range versus GSI from the block model

Results indicate mapped GSI generally captures broad trends in rock quality, however, it appears to be variable across a mapped length. This is likely due to the small-scale, localised variability in rock quality, subjectivity in GSI mapping undertaken by various individuals (particularly when mapping at a safe distance from the wall), and potential impacts from blast damage and construction which reduces the true GSI of the intact rock. This difference between GSI from design to as-built was observed to vary by up to ±20 points, which was considered as part of revised 2D LE and FLAC3D slope stability analysis as described in Sections 4.2 and 4.3.

The observations of this review also supported discussions around improvements to blasting techniques, including implementing trim shots and controlled blasting in the direction of open faces to minimise rock mass damage.

3.1.2 Strength

Pocket penetrometer and Schmidt hammer testing was conducted onsite to aid in estimating the undrained shear strength and UCS of soil and rock units exposed in the pit slope. Site-specific calibration was undertaken for the Schmidt hammer using concrete cubes of various ages and strength to confirm a site law for the equipment being used. A power function formula (shown in Equation 2) was subsequently adopted, which has a similar form to other published relationships (Brencich et al. 2013) to assess rock strength.

$$UCS\ (MPa) = 0.02\ Schmidt\ hammer\ rebound\ value^2 \tag{2}$$

During geological mapping, soil strength was estimated using tactile observation and rock strength estimated through the FES ISRM method (Brown 1981). These observations were compared against the results of pocket penetrometer and Schmidt hammer testing to arrive at the mapped intact strength value.

Whilst the pocket penetrometer and Schmidt hammer rebound values varied over discrete parts of the slope, the collection of several readings resulted in a range of values measuring typically well within the mapped range of soil and rock mass strength. This highlighted the benefit of systematic testing during a mapping campaign to provide a repeatable approach that can be used by several different people to achieve consistency in strength data capture in soil and rock. The ranges applicable to the tools used for the current study are shown in Figure 5 below. Where there is some difficulty in assessing rock strength within the R1 to R2 strength range, a possible solution is the use of pocket penetrometers that are designed for assessing the early strength of concrete which cover strengths up to approximately 5 MPa, albeit with some loss of resolution and accuracy over the soil strength range.

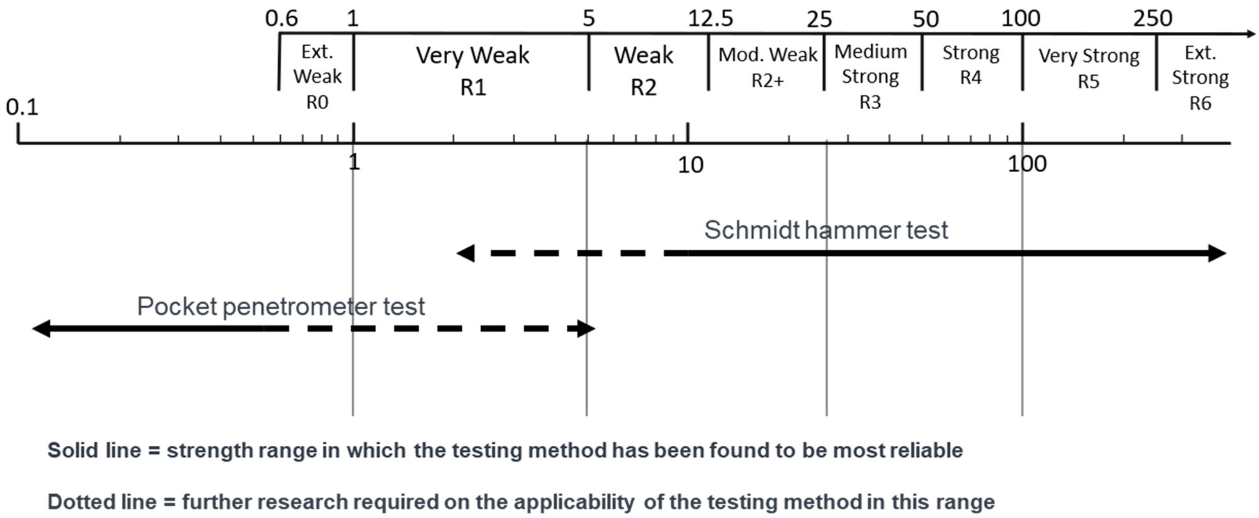

Figure 5 Applicability of pocket penetrometer and Schmidt hammer strength testing methods based on site calibration

Similarly to the mapped GSI, strength mapping results indicated that general capture predicted broad trends in rock with some localised variability in rock mass strength. It was also observed during onsite strength mapping that some clay-rich zones can experience strength degradation over time due to exposure to air and the environment. This strength degradation in certain geological units was further assessed in 2D LE to understand the magnitude of impact on the slope Factor of Safety (FoS), as described in Section 4.2.

3.1.3 Structures

Structural data was collected during mapping of newly exposed faces to capture feature type, estimates of orientation, persistence and other visible characteristics that may impact slope stability. These mapped features were then compared to the structural model, described in Section 2.2, to identify potential variation in the modelled structures.

Where the magnitude of variation criteria were met, as defined in Section 3, initial assessment of the revised parameters was conducted through conventional kinematic methods. The assessment involved identifying the potential structural failure mechanism, whether it was planar, wedge or complex (i.e. failure along structures and weak rock mass), and selecting the most appropriate method of analysis. This review is further described in Section 4.1.

3.2 Drilling

Review of drilling data allowed for more subsurface remote reconciliation of 3D models, targeting areas deeper than the directly exposed bench faces as well as areas potentially impacting future bench construction. This allowed for improved future planning of design and construction, and identification of areas potentially requiring shallowing or optimisation.

Geotechnical rock core logging data and associated core photos were reviewed to compare against the predicted conditions of rock mass strength and GSI. Average rock mass conditions within geotechnical domains were observed to generally represent the conditions in logging data and core photographs. However, due to the 12-m geotechnical block model size, smaller zones of variable geotechnical conditions were not appropriately represented, particularly around fault zones. Figure 6 presents an example of smaller-scale variation in downhole GSI data, where a zone of poor rock mass was identified at approximately 45 m downhole which was not represented in the block model. However, this zone aligned well with a previously modelled fault, providing reconciliation to the structural model. Section 4.3 provides greater detail on how faults are represented in FLAC3D models.

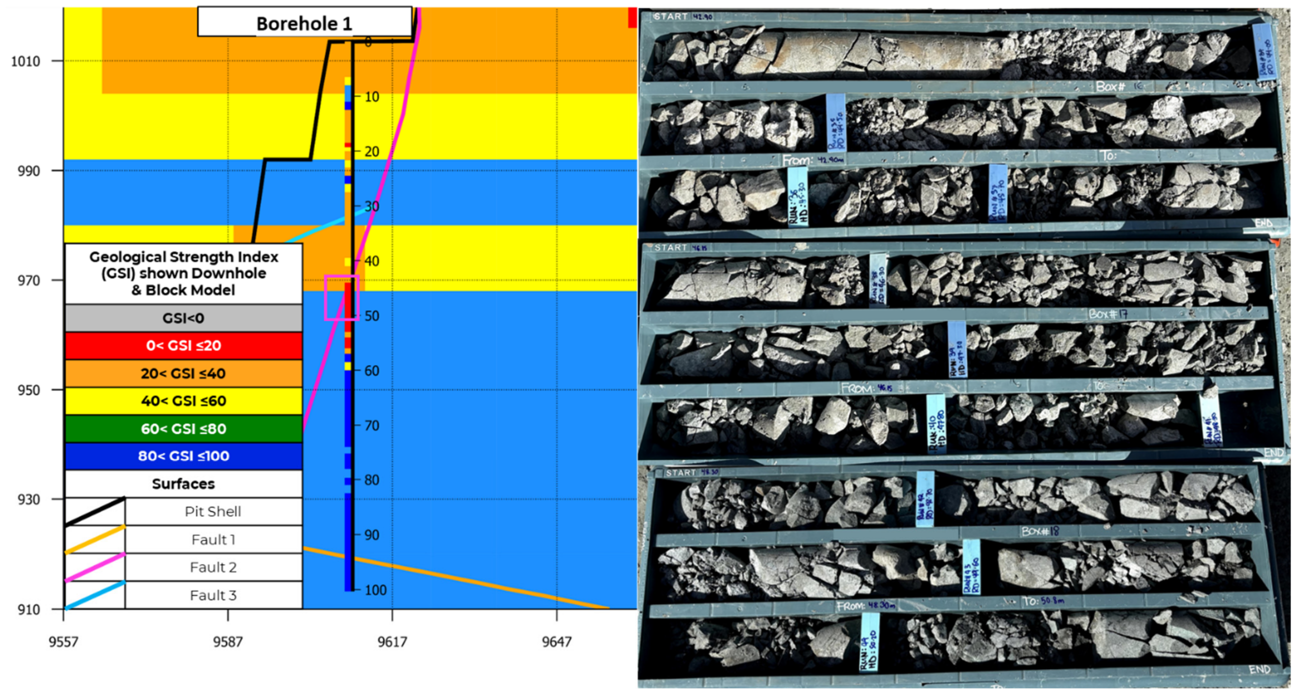

Figure 6 Example of rock mass and structural model reconciliation review through downhole data

Overall, review of the downhole data indicated that while the block models did not capture variability in geotechnical conditions on a smaller scale (i.e. <12 m), the block models were generally representative of the broader conditions. Where variations were identified, sensitivities in slope stability analyses were conducted (and described in greater detail in Sections 4.2 and 4.3).

3.3 Monitoring

Several types of slope movement monitoring have been utilised onsite including radar, prisms, extensometers, time-domain reflectometer (TDR) cables, inclinometers and vibrating wire piezometers. This monitoring allowed for comparison of in situ deformation to the predicted deformations from the 3D numerical modelling, described in Section 4.3. Where monitored deformations exceeded predictions, targeted follow-up investigations and increased monitoring were carried out. Monthly reviews of monitoring data were undertaken to assess whether slope behaviour, including groundwater pressures, are consistent with the design predictions. The boreholes drilled during construction were utilised to both validate predicted ground conditions, as described in Section 3.2, and to install downhole instrumentation as part of the ongoing monitoring program.

The main outcomes from the monitoring include:

- Most of the movement observed onsite occurs following a blast or after sustained periods of rainfall.
- Some movement has been observed in sections of the pit wall where it was anticipated in the detailed design study. Given that these areas were predicted to have a lower FoS, enhanced monitoring incorporating survey prisms and focused radar monitoring was initiated at the beginning of construction to more accurately detect movement within these regions.
- Typically, minor movement of the slope has occurred as mining continues and is managed on a local scale.

TDRs, extensometers and inclinometers have contributed to understanding the pit wall response during excavation. Collecting data through these subsurface remote measures, along with radars and surface prism readings, enhances our understanding of the ground movement. This improved understanding aids in forecasting the performance of specific rock types in future construction.

In considering how to use and review monitoring data from a pit slope it is important to recognise that the FoS of a slope cannot be directly assessed in the field. In view of this, monitoring must be focused on quantifiable performance (i.e. assessment of the deformation of pit slopes and groundwater levels). Therefore, this approach requires use of advanced numerical modelling techniques, such as finite difference or finite element software. Further background on this subject may be gained from the Alberta Dam and Canal Safety Directive (Government of Alberta 2018), which identifies the importance of focusing design and monitoring of dams on quantifiable performance.

During the Lihir project, expected pit slope deformation was estimated during design and was used to set trigger levels using a trigger action response plan (TARP) for both cumulative overall slope movement and for rapid changes in slope movement. Groundwater pressures were also measured and compared against the pressure assumptions used in FLAC models to allow adjustments to be made to the planned slope dewatering design (i.e. through horizontal drain holes).

4 Slope stability analyses

Based on findings of the as-built data collection described in Section 3, slope stability analyses were required in areas where the in situ conditions deviated from predicted modelled conditions by amounts exceeding the criteria required for a revised model update (defined in Section 3). Where these in situ conditions appear less favourable than predicted conditions (i.e. weaker rock mass strength/GSI, newly identified structures or more adversely dipping structures), a revised analysis may indicate more conservative slope designs are required. Alternatively, where in situ conditions appear more favourable, a revised analysis may indicate optimisation of the pit designs.

As previously indicated, the difference in scale between the geotechnical block models (12 m^3 blocks) and the onsite data collected (≤1 m) introduces complexity when directly comparing in situ to predicted conditions. This complexity was addressed through various considerations:

- Reconciliation of bench scale mechanisms separately from inter-ramp and overall wall scale mechanisms
- Sensitivities in slope stability analyses to understand the impacts of variation in rock mass strength, rock mass quality and hydrogeological conditions
- Capturing impactful but smaller-scale variations (i.e. <12 m thickness) in in situ conditions by adding zones in models matching the location, thickness and characteristics of the identified variation.

4.1 Kinematic analysis

In cases where new structures were identified, previously modelled structures required localised revisions or movement was observed from monitoring data, further investigation to identify potential structural conditions to confirm the possible failure mechanisms was conducted. This included reviewing known structures – both from face mapping and the structural model – and evaluating how these faults interact with the geometry and orientation of the pit slopes. Initial review involved creating 3D planes of newly identified structures in LeapFrog (Seequent 2024) and visually assessing spatial relationships and interaction with existing structures and the design pit wall in 3D. This review allowed for identification of potential failure mechanisms and selection of the most appropriate method for analysis. Planar and wedge mechanisms were assessed through RocScience's RocPlane and SWedge software (RocScience 2023a and 2023b), respectively. Figure 7 presents an example of two identified structures forming a wedge and potentially impacting slope stability. For more complex failure mechanisms, such as a wedge forming along two or more structures and weak rock mass, slope stability analysis was conducted in FLAC3D (described in Section 4.3).

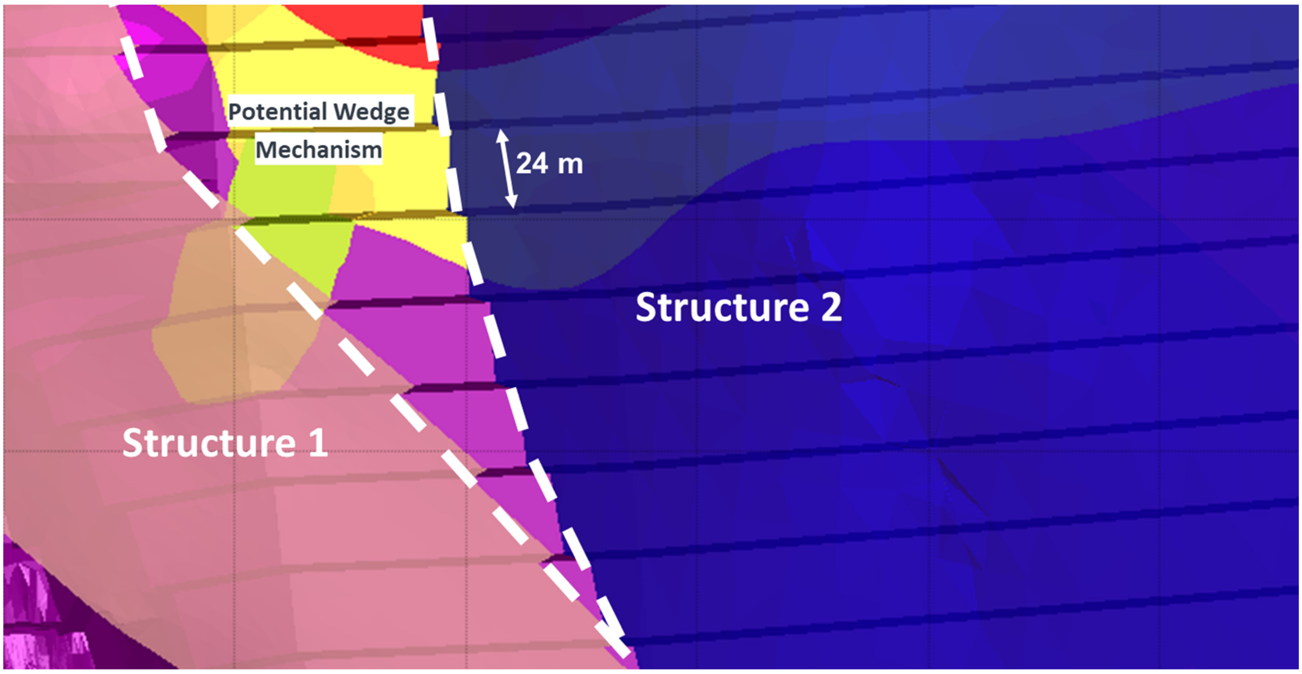

Figure 7 Example of 2 structures forming a wedge and potentially impacting slope stability

Where these wedge mechanisms were identified during construction, a conventional kinematic analysis was conducted using SWedge (RocScience 2023b). Orientations for the two identified structures creating the wedge and the pit design wall were used to calculate FoS values and compare results to the DAC (i.e. FoS ≥ 1.1 for bench scale mechanisms and FoS ≥ 1.3 for inter-ramp mechanisms).

Based on the results, potential adjustments to the pit design were reviewed to continue meeting DAC. Where analyses indicated FoS values below acceptable thresholds, mitigation methods such as shallowing of the pit walls, reducing bench height and increasing bench widths were required to maintain the integrity of the pit wall and reduce the risk of rockfall hazards.

4.2 2D limit equilibrium analysis

The rock mass strength and GSI block models were compared to the data collected during construction to identify the magnitude of variation between the predicted and as-built conditions. Where these conditions varied by more than the criteria defined in Section 3, slope stability analyses were revisited to identify the potential for shallowing or steepening of the pit walls.

Variations in intact rock strength and GSI can quickly be analysed using Rocscience's Slide software (RocScience 2025) on bench, inter-ramp and overall wall scales. Where revisions to the models were required, re-assignment of rock strength blocks were conducted and models re-run to determine impacts on slope stability. An example of a simplified bench scale model sensitivity to capture observed strength degradation in clay-rich rock is presented in Figure 8. This analysis assumed dry slopes at the bench scale and simulates strength degradation by applying a 50% UCS strength reduction on the material. While this estimate may be considered a 'worst-case scenario', it is important to understand the impact if this magnitude of strength degradation does occur. The outcome of this sensitivity indicated that the designed 60° bench face angles continue to meet the DAC. Note that, in this region, the pit slope designs are governed by inter-ramp stability and, while steeper benches may be achieved on a bench scale, inter-ramp stability designs require shallower bench face angles and wider bench widths.

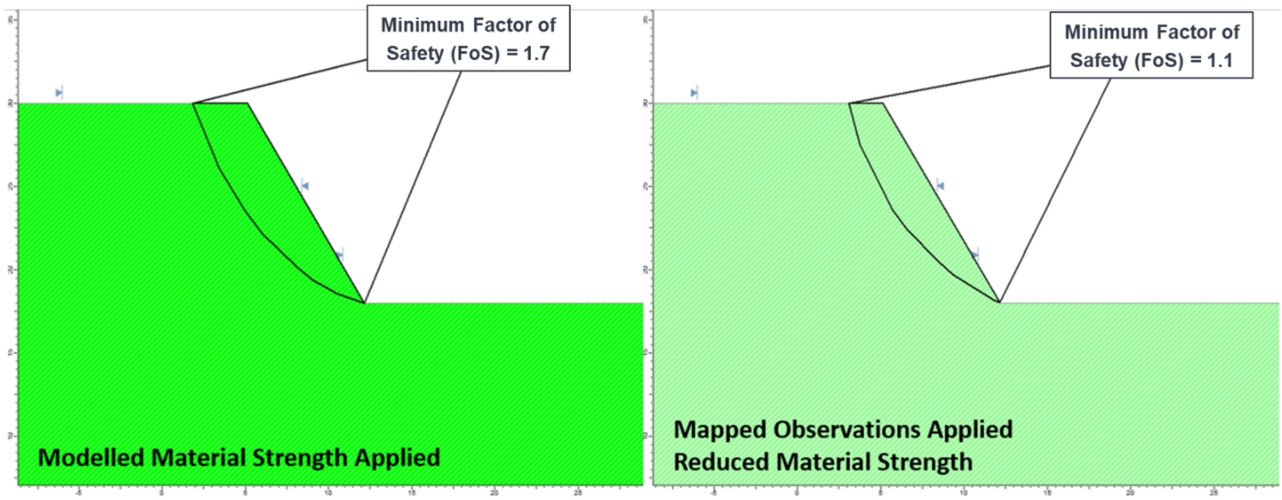

Figure 8 2D limit equilibrium model sensitivity applying observed conditions

In cases where variations of strength and GSI occurred over a smaller area (i.e. smaller than the block model resolution), a zone was manually created in the model to simulate these smaller variations in strength. This was particularly important when identifying zones of weaker material that may form sliding planes.

A 30-m blast damage zone simulated in the 2D LE models using a disturbance (D) factor was also reviewed during mapping and subsequent analysis review. If it appeared that the face mapping of GSI adequately captured blast damage, then analyses were conducted with and without a D-factor as a sensitivity.

The results of the revised models indicated areas where localised shallowing was required to manage poorer than predicted rock mass strength and quality to meet the DAC. Alternatively, localised steepening was addressed in zones with improved as-built conditions. This was particularly beneficial in areas where more competent geotechnical domains were exposed earlier than expected in the slopes.

4.3 3D numerical modelling

3D numerical modelling of the pit slope was undertaken using FLAC3D (ITASCA 2024). An overall pit wall scale model was created for the project which incorporates the geological domains, GSI block model, strength block model, structural model (major faults) and an assumed pore pressure grid. The model mesh comprises hexahedral elements set up to be consistent with the block model resolution of 12 m^3 but refined in the vicinity of major faults and the slope face to 3 m resolution. For reference, the overall model covers an area approximately 2.7 km wide by 2.5 km deep by 1.2 km high.

Rock mass strength within the model was assessed using the UCS and GSI values from the block models to select the parameters to be used on an element-by-element basis. Where major faults pass through the pit slope, these were modelled by applying a 50% GSI reduction for elements within approximately 10 m of the fault wireline. This 50% GSI reduction approach was validated through back analyses of instabilities involving faults within the project area. Similarly to the 2D LE analysis summarised in Section 4.2, block modelled strength and GSI values were adjusted in areas where onsite data collection indicated variation in predicted conditions. These models were then re-run to analyse the impacts of identified variations, which would then inform required larger-scale pit design changes.

The objective of the 3D numerical modelling work was to assess performance of the slope at inter-ramp and overall wall scales. It was also used to assess the behaviour of the slope where larger geometric changes were being made (i.e. changes to ramp locations and widths, testing different inter-ramp angles and assessing sensitivity of the design to key design input parameters). As discussed above, simple kinematic or 2D LE methods were used to assess variations from design models on a small to medium scale.

The FLAC3D modelling provided insights into areas of the slope with potential for larger movements, indicated as areas with lower strength reduction factor values. This enabled slope monitoring to focus on these identified areas with increased risk. An example of this involves an area within a bullnose of a pit slope which was identified as having increased risk of instability due to the predicted and mapped geotechnical conditions, as shown in Figure 9. This increased surveillance allowed for more proactive implementation of controls to ensure the safety of mine personnel and equipment, including establishment of an exclusion zone. A revised pit shell was developed that allowed mining to continue safely below the slope with a step-out. The bullnose subsequently experienced progressive failure over several months, which involved increased deformation and cracking at the slope crest that eventually lead to collapse into the exclusion zone that had been established.

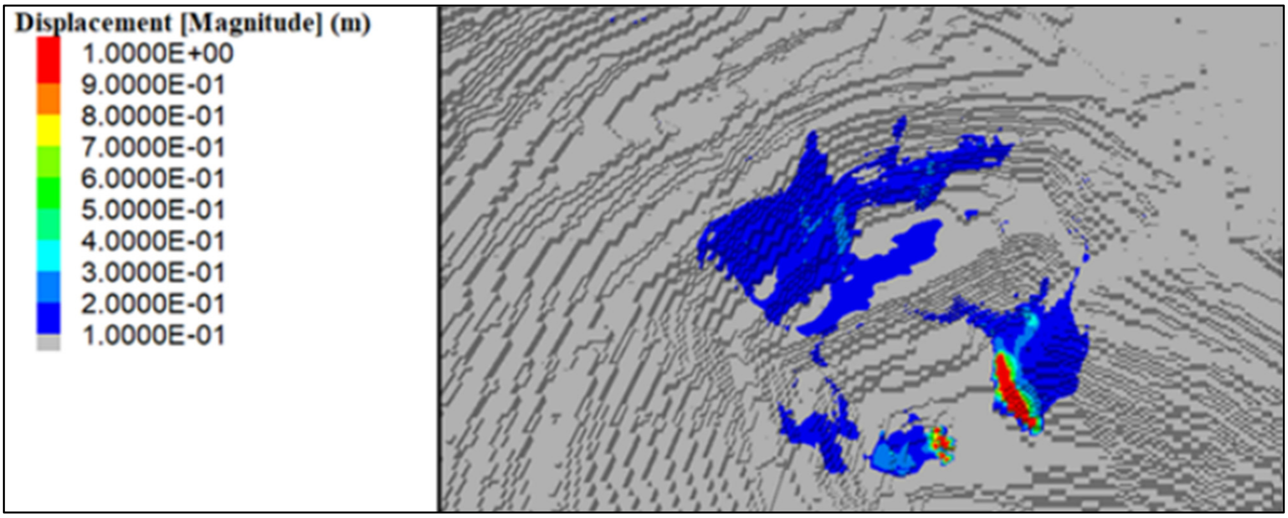

Figure 9 FLAC3D output showing deformation predicted at the bullnose of the pit

5 Conclusion

This paper presents a case study on the reconciliation of predicted geotechnical conditions through in situ data collection during pit construction. The objective of this case study was for early identification of potential failure mechanisms, monitoring for compliance to the DAC and exploring opportunities for slope optimisation in a complex geological setting. The slope reconciliation approach for open pit construction described in this paper offers several advantages as it:

- improves confidence of the modelled geotechnical conditions through in situ data collection, leading to more reliable pit designs
- identifies opportunities for optimisation of designs based on verified site conditions, leading to a reduced risk of steeper slopes
- anticipates potential failure mechanisms and proactively targets slope monitoring to manage the safety of personnel and equipment during construction
- anticipates early planning of pit design adjustments to mitigate slope failures that may impact construction schedule and required step-outs, reducing potential for ore recovery
- enables optimisation of slopes at the start of mining, where slight adjustments to the steepness of benches can eventually add up to significantly reduced waste stripping and improved ore extraction.

Proactive planning of slope reconciliation offers advantages such as improved accuracy in identification of potential failure surfaces, improved confidence in stability analysis, enhanced pit slope designs and effective risk management during construction. These benefits contribute to the development of safe and cost-effective slope designs, reducing the potential for slope failures and improving the long-term stability of slopes.

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