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To topple or not to topple: managing failures at a case study site

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Abstract

This paper presents a case study of managing toppling failures at a short-life open pit lithium mine in Australia, where adverse slope performance was encountered early in operations. The geological setting comprises a steeply dipping pegmatite orebody hosted within regionally foliated phyllite. The dominant failure mechanism was flexural toppling along sub-vertical foliation planes, with additional complex toppling and minor wedge failures observed. Slope instability was exacerbated by high pore pressures, poor understanding of rock mass conditions, and limited initial slope monitoring.

A revised geotechnical model was developed based on updated structural mapping, back-analysis of failures, and field observations. Stability analyses indicated that even at shallow slope angles, deep-seated toppling could occur in weak, foliated materials under elevated pore pressures. Slope design modifications included flattening inter-ramp slope angles and implementing a practical depressurisation strategy using horizontal drain holes. A significant uplift in slope monitoring – including prisms, radar, drone surveys and extensometers – improved risk management.

The case study demonstrates that in structurally complex geological settings, especially those with pervasive foliation, a robust and evolving geotechnical model is essential. The findings underscore the sensitivity of flexural toppling mechanisms to groundwater conditions and highlight the importance of adaptive design and operational controls in short-life mining environments.

Keywords: *flexural toppling, slope stability, geotechnical model, depressurisation, pegmatite*

1 Introduction

This paper presents the difficulties in managing toppling failures at a small, short-life open pit in Australia. Open cut mining commenced in 2022 with a conventional truck and shovel operation. It was expected that open pit excavation would be complete within 2–3 years. The pit was to be mined in 2 stages:

1. Stage 1: initial pit, about 350 m long, 180 m wide and 90 m deep.
2. Stage 2: final pit, about 685 m long, 500 m wide and 170 m deep.

The authors became involved at this site in 2023 after mining commenced and geotechnical issues were encountered in terms of slope performance and operations. At that time, both pit stages were being mined, requiring inter-stage mining management.

The case study site is a pegmatite lithium deposit, where the pegmatite has intruded into a regionally metamorphosed phyllite country rock. Pegmatite bodies enriched in spodumene are typically gently dipping, so the sub-vertical igneous deposit at this site is unusual (Phelps-Barber et al. 2022; Weir et al. 2023).

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Given the narrow, lenticular shape of the pegmatite, the open pit slopes are predominantly within the surrounding phyllite country rock. A key characteristic for slope design in these rocks is the foliated texture due to directional stresses related to regional metamorphism.

Toppling failures involve the rotation of columns or blocks of rock about a fixed base. This type of failure mechanism is typically encountered in bedded or foliated rock masses characterised by continuous, closely spaced discontinuities that steeply dip back into the slope. Toppling is typically classified into three types: flexural, block, and block-flexural toppling. However, a number of more complex toppling failure mechanisms are also possible (Wyllie & Mah 2004). The study of Sjoberg (2000) developed a conceptual model for the development of a large-scale flexural toppling in hard rock in six stages.

The following sections review the initial slope performance for the case study site, the geotechnical model and how this evolved from the design study to operations. The paper also looks at the slope performance, operational challenges and adopted slope design modifications.

2 Initial slope performance

The mine slope failure database recorded 24 instability events in the first 7 months of operations and performance of the pit slope was considered poor. This number of failures was particularly high given the limited slope development and that it was dry season during that time. Figure 1 shows the locations and examples of the recorded events. Of the 24 events, 17 were specific instability events and the remaining 7 were recorded as ongoing development of existing failures.

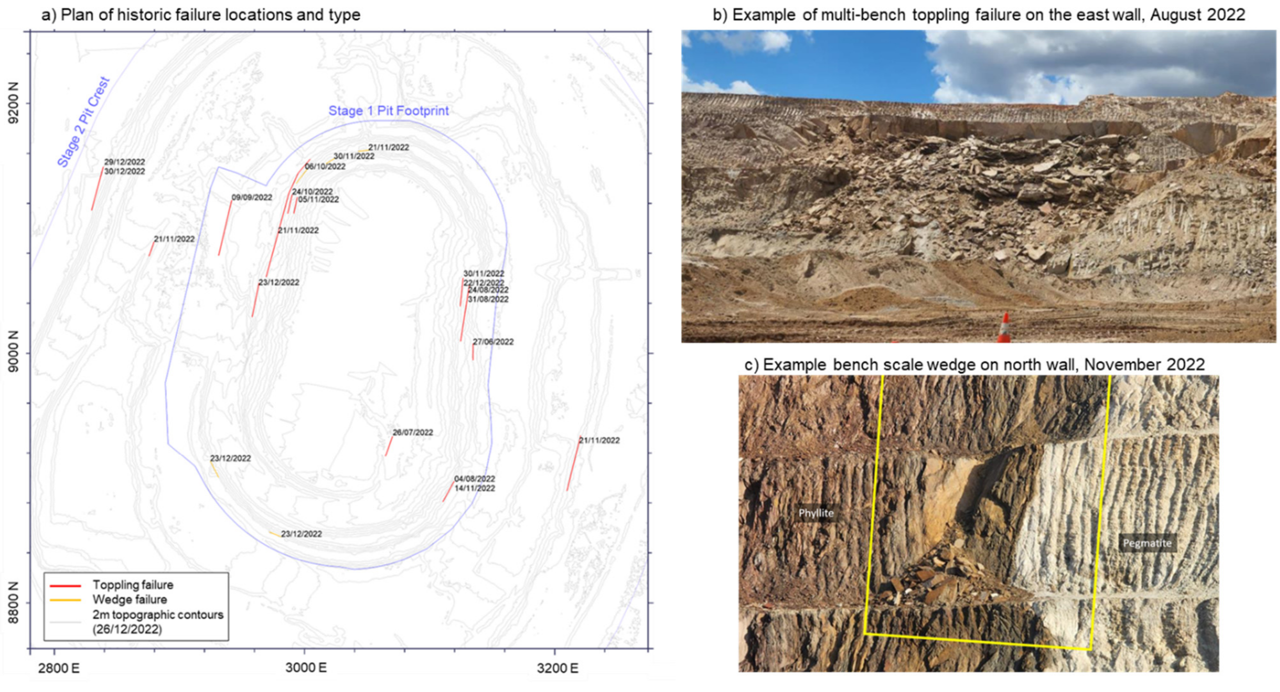

Figure 1 Historic failures (June 2022 to January 2023) with (a) plan location and examples of (b) toppling failures and (c) wedge formation on end walls

The recorded instability events are summarised as follows:

- Flexural toppling failures, typically located on the east and west walls, which have long strike lengths and oriented such that they are susceptible to flexural toppling from the weak, continuous sub-vertical foliation.
- Some complex toppling failures were recorded on the east wall, which included some planar slide component on shears subparallel to foliation.

- The failures were typically bench-scale, however, there are examples of precursor single bench failures progressing to larger multi-bench toppling failures (Figure 1b). This was a concern given the limited wall development at this time.
- Minor, bench-scale wedge failures were observed on the north and south end walls (Figure 1c). These wedges were formed from the intersection of foliation and a joint set. Given the relatively small-scale, these were operationally manageable.
- About 50% of the failures had a trigger such as blasting or rainfall recorded. Of these triggers, rainfall had the highest number of observations.

In addition, durability of the slopes in the upper weathered zone was low with slopes showing cracking, deformation and erosion with mining and exposure. The critical failure mechanisms for consideration in slope design for each pit wall are summarised in Table 1. These were identified based on an observational engineering geology model and slope performance to date.

Table 1 Observed failure mechanisms

Pit wall	Critical failure mechanism	Engineering geology controls
East and west	Flexural toppling along foliation, as the pit is progressively excavated	Pervasive, closely spaced foliation in phyllite Minor faults and shears parallel to foliation Low intact strength in upper weathered zone Pore pressures
North and south (end walls)	Wedges	Geometry and spacing of joints and foliation

Slope monitoring up until January 2023 was limited to about 5 prisms, monitored weekly. This meant that slope management and operation was heavily dependent on visual observations by the onsite geologist. This resulted in significant operational stand-down periods after an instability.

There were other operational difficulties associated with excavation of the weathered phyllite. The weathered phyllite is rich in micaceous minerals which form micaceous silts and clays. These soils commonly exhibit a low in situ density, are highly erodible and hydrophobic making dust control difficult.

Trafficability was also a significant operational issue at the site in both dry and wet season. This has resulted in significant cost associated with importing of material to the site for lining of the haul roads. These operational difficulties and associated costs could have been anticipated during early planning stages, given these are common issues associated with phyllite and other schistose rocks.

Given the adverse slope performance and operational difficulties an overall design update and slope management review was undertaken. This was also prompted by the development of tension cracks parallel in strike to foliation, some 50 m behind the pit crest on the east and west walls during the 2022/2023 wet season. This model and design update is documented in the following sections.

3 Design update

3.1 Available data

Six boreholes were diamond drilled and geotechnically logged as part of the 2018 open pit site investigation for the feasibility study. Borehole locations are shown relative to the ultimate pit crest and the orebody in Figure 2. Of the six geotechnical holes drilled, all have oriented core data collected. The boreholes were predominantly drilled for resource estimation purposes, with 5 out of 6 geotechnical boreholes drilled into the ore deposit, instead of into the open pit walls.

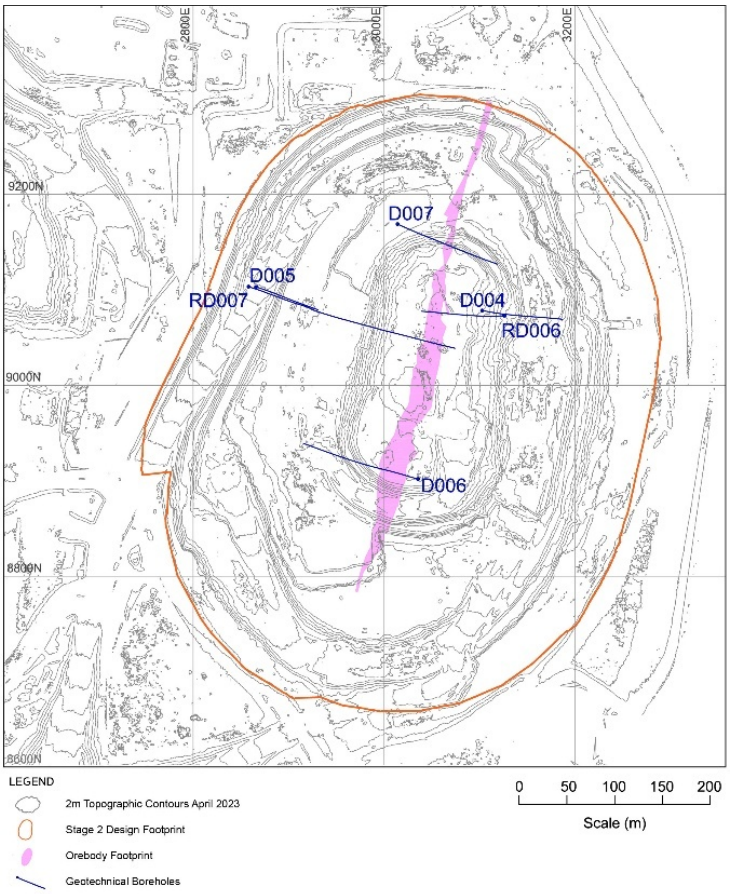

Figure 2 Geotechnical borehole locations relative to the ultimate (Stage 2) pit crest and orebody footprint

Field and laboratory testing included a small database as follows (number of available tests in brackets):

- Point load strength index testing (48)
- Soil characterisation testing (e.g. particle size distribution, Atterberg limits) (6) and CUPP triaxials (2)
- Rock testing (unconfined compressive strength [UCS] [7], indirect tensile [3], slake durability [4])
- Direct shear testing of natural defects (8).

The overall quantum of testing was appropriate for the scale and short life of the pit. However, all UCS testing conducted on phyllite material was deemed invalid in the feasibility study due to failure along existing fabric. Unfortunately, the angle to foliation was not recorded, nor were the original laboratory test certificates available, which meant that the test results could not be further investigated.

Hydrogeological testing and groundwater data relevant for geotechnical purposes was limited to slug testing and standing water level measurements in environmental water bores distributed across the site. The closest water monitoring bore was about 100 m from the open pit crest.

Structural mapping was an important data source for comparison against available borehole data. The mapping database at the time comprised 120 structural measurements collected on the north and eastern walls of the Stage 1 pit.

3.2 Geotechnical model

3.2.1 Introduction

A geotechnical model is a conceptual representation of the subsurface conditions at a specific site, used to support design, construction and inform risk management for open pit slope design and operations. The primary components of the model include geology (lithology and alteration), structure, soil and rock mass, geomorphology and hydrogeology. The components of a geotechnical model are shown conceptually in Figure 3. The influence of each component on open pit slope design will vary with geological environment and site-specific ground conditions. The following sections summarise the available data, key components of the geotechnical model for the study site and how understanding of the model has evolved.

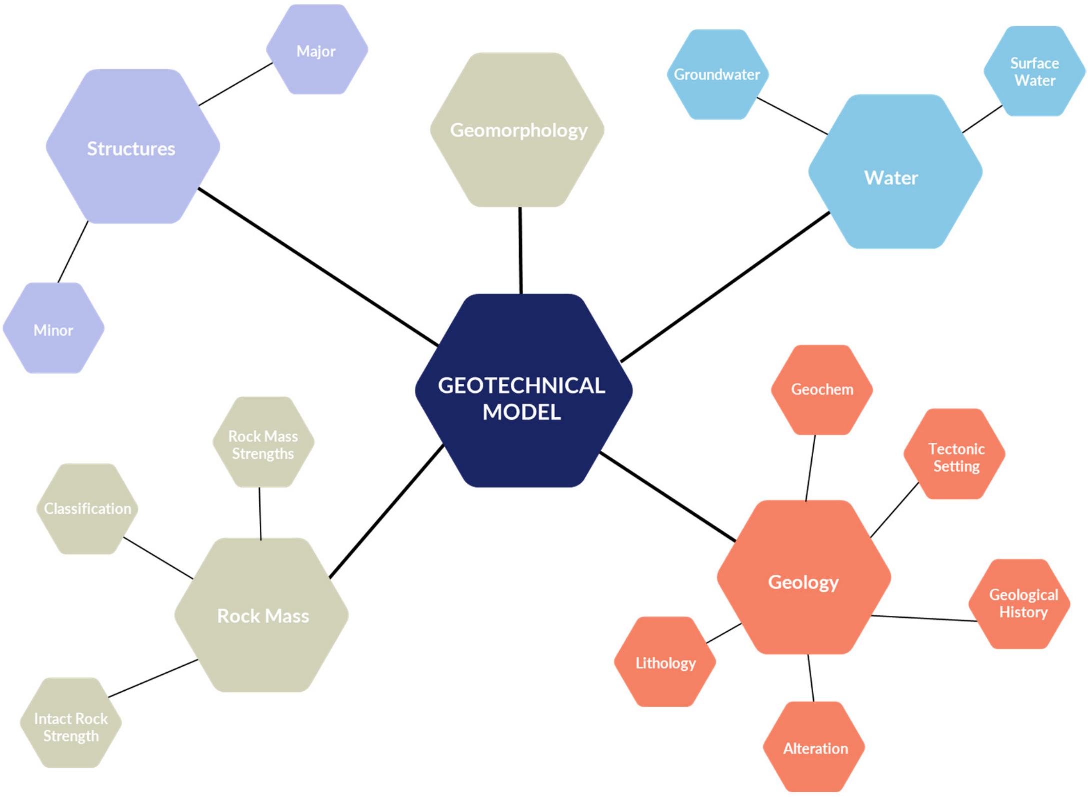

Figure 3 Conceptual figure illustrating the key components of the geotechnical model (after Weir & Fowler 2025)

3.2.2 Geotechnical model: geology and geomorphology

The geology at the case study site is characterised by thin surficial sediments (typically less than 3 m thick) overlying two primary rock types:

- Phyllite: a fine grained metasedimentary rock type (similar in character to schist).
- Pegmatites: an igneous rock which intruded into the phyllite.

The pegmatite forms a primary sub-vertical orebody up to 35 m wide, Figure 4. The orebody is a lozenge shape, about 300 m in length tapering to the north and south (Figure 1).

The depth of weathering was interpreted to be 30–50 m deep during pre-mining studies.

Geomorphology does not have a significant impact on design at this site, with the site located within a broad historic flood plain with very little vertical relief.

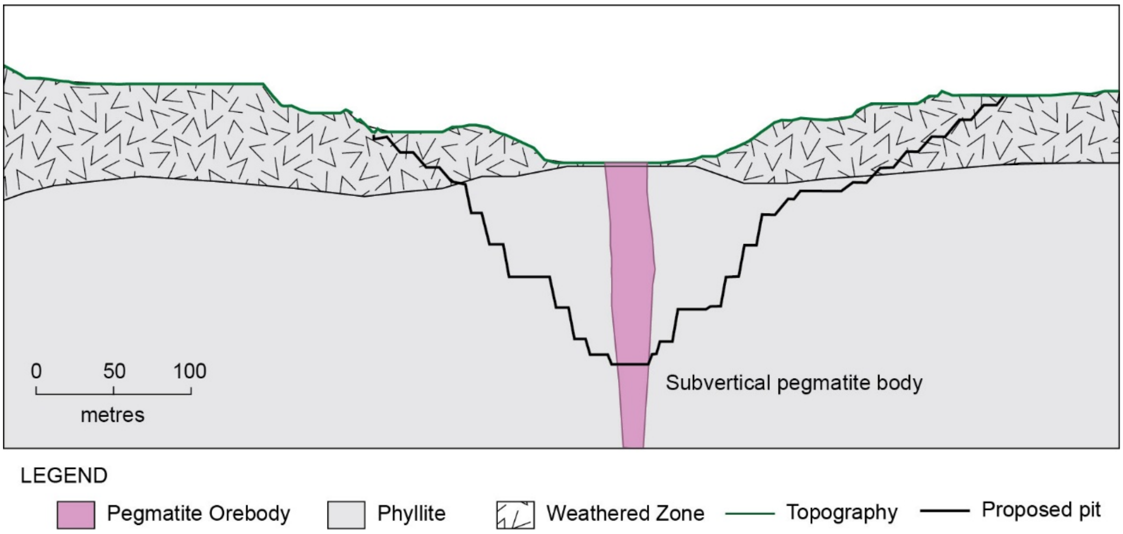

Figure 4 Cross-sections through case study site highlighting the steep pegmatite body

3.2.3 Geotechnical model: rock mass

Rock mass conditions are controlled by both the geology and surface processes at the site. The rock mass units comprise:

1. Weathered zone: extremely weathered in situ rock and residual soils. This zone is typically soil to low intact rock strength and is about 50–70 m thick. This zone is about 20 m thicker than was anticipated in pre-mining studies.
2. Transitional zone: increased intact rock mass strength, weathering along discontinuities
3. Fresh high strength rock which is subdivided into phyllite and pegmatite

The rock mass strengths for the various units are summarised in Table 2. Properties for the transitional and were based on geotechnical logging data, laboratory testing and published values of m_i.

Table 2 Rock mass properties

Rock mass unit	Mohr–Coulomb parameters		Generalised Hoek–Brown			Modulus (MPa)(1)
	Cohesion (kPa)	Friction angle (°)	Uniaxial compressive strength (MPa)	Geological strength index	m_i	
Upper weathered	30	24	–	–	–	300
Transitional	80	28	–	–	–	750
Fresh pegmatite	–	–	120	70	20	–
Fresh phyllite	–	–	80	67	10	29,650

(1) Rock mass elastic modulus calculated using the Generalised Hoek–Diederichs method and using a typical modulus ratio of 550 for phyllite

The properties of the upper weathered zone were based on back-analysis of two multi-bench failures, one on the west wall and one on the east. The back analysed strengths indicated a substantially lower cohesion than the pre-mining estimate, which was 120 kPa and based on 2 soil triaxial tests. This may be a consequence of limited borehole data behind the pit walls for the pre-mining studies. In addition, the core photography and logging for the feasibility study was undertaken at the core shed instead of at the drill rig. Observations of recent drilling at the site has shown that as the core dries out, in the weak materials there is an increase in intact strength, such that material strengths are overestimated within a few days at the core shed. This could have resulted in overestimation of strengths at the depths relevant for the upper weathered zone and increased the interpreted thickness of the transition zone in the previous interpretation.

The fresh rock for both the pegmatite and the phyllite typically has a high intact strength (>80 MPa). The main differentiator between units (besides lithology) is foliation in the phyllite, which may impact the overall rock mass strength. The presence of foliation can result in a strong strength anisotropy for the country rock, particularly those with a sedimentary protolith. The anisotropy can make assessment of intact strength difficult and sample selection for testing requires careful consideration. Given the high rock mass strengths in fresh rock, an understanding of the geological structures is an important component of the geotechnical model.

3.2.4 Geotechnical model: structure

The structural geology is critical to the engineering geology model and slope design for pegmatite deposits (Weir et al. 2023). Here the structural geology model is referring to both major structures (i.e. faults and shears) as well as the structural fabric (i.e. foliation and jointing patterns). The structure model is again strongly controlled by the pegmatite body, both in terms of major structures and fabric.

Two stereographic projections (stereographs) from borehole data are presented in Figure 5. The stereographs show structures measured from oriented core of the 5 pre-mining boreholes through both pegmatite and the phyllite. The pegmatite stereograph shows joints oriented perpendicular and parallel to the pegmatite body, which are interpreted as cooling and contact joints, respectively. Cooling related joints are oriented perpendicular and parallel to the dip of the pegmatite due to heat flow and a decrease in volume during cooling (Figure 5b). Some of the jointing is of similar orientation to the phyllite (Figure 5c) and so may also be tectonic joints.

Within the phyllite foliation is sub-vertical and striking north-northeast, parallel to the pegmatite body (Figure 5c). It is likely that the magmatic fluid forming the pegmatite exploited the foliation as a plane of weakness during the intrusion event. The foliation forms closely spaced planes of weakness, particularly in the upper weathered zone. Given the steep dip and character of the foliation, where the pit slopes are parallel to the foliation they are prone to flexural toppling style failures.

The structural domains (a volume of rock with similar structural patterns and characteristics) at the case study site reflects the lithological control on structural patterns. The structural patterns and character are sufficiently different between the pegmatite and the phyllite, so that two structural domains were defined.

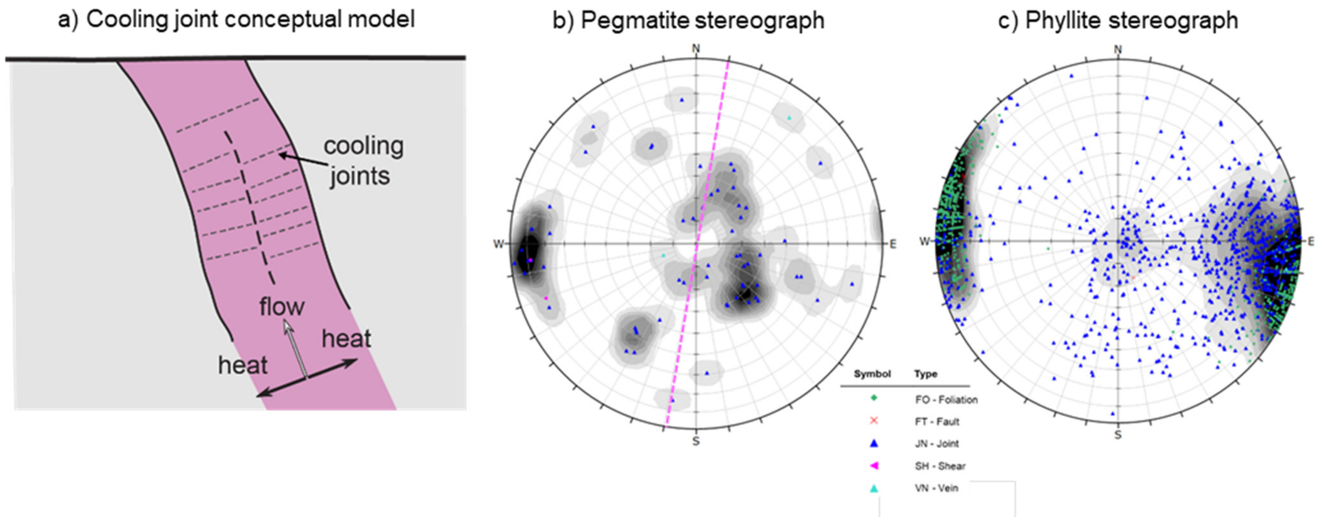

Figure 5 Pegmatite jointing patterns. (a) Conceptual model of thermal (cooling) joints and stereographs from oriented core for (b) pegmatite and (c) phyllite. Pink dashed line indicates strike of the pegmatite body. Stereographs are lower hemisphere, equal area

As part of a slope design update, foliation orientation was reviewed from the available borehole. This included the development of downhole tadpole plots of oriented core data, as well as cross-sections showing boreholes and measured foliation orientations. Overall, the foliation orientation did appear to have a higher degree of variability around pegmatite intrusions. Away from the pegmatite intrusion, the foliation appears to be more consistent, with a dip greater than 80°. However, there may still be variation encountered in the ultimate pit walls not evident in the available data.

3.2.5 Groundwater and surface water

The case study site has a tropical climate with clearly defined dry and wet seasons. The average annual rainfall is approximately 1,700 mm, with most rainfall occurring during the wet season from November to April.

Groundwater and pore pressure assumptions for the design update relied upon historical data, studies and field observations which identified:

1. the phyllite is a fractured rock aquifer
2. groundwater flow direction is towards the northeast, which is parallel to the foliation.
3. standing water level observations from bores in 2021 indicate a depth to groundwater is about 0.5 to 1.6 metres below ground level (mBGL) during the wet season and between 4 to 12.5 mBGL during the dry season.
4. there were visual observations of seepage from fractures on bench faces.

The groundwater data and model are limited, particularly with regards to understanding of actual pore pressures currently acting in the slope. In addition, there are unlined mine water dams and sediment ponds directly to the south of the pit. It is considered highly likely that these structures are leaking and recharging the groundwater system.

Given the above data and observations, it was assumed that there were comparatively high pore pressures and steep hydraulic gradients behind the pit walls. Recharge from rainfall and slow drainage contributes to soft ground conditions for traffic across benches.

3.3 Stability analysis and design updates

3.3.1 Upper weathered zone

Slope stability assessments were undertaken considering the updated geotechnical model, observed failure mechanisms and existing slope performance. The initial focus of the assessment was on the upper weathered zone so that the Stage 2 pit design and ultimate pit crest could be rapidly updated with mine planning.

The upper weathered zone assessments included kinematic and statistical assessment of defects assessing the range of relevant slope aspects as well as 2D limit equilibrium (LE) analysis of multi-bench and inter-ramp scale stability using an anisotropic strength model to define the discrete angle over which the foliation strength applied.

The analyses indicated that to meet a design acceptance criterion of Factor of Safety (FoS) of 1.2 the slope design in the upper weathered zone required modification as follows:

1. Inter-ramp angle within the upper weathered material had to be reduced from 32 to 27°. This was addressed through a combination of reduced bench face angles and increased berm widths.
2. Two wide (14 m) geotechnical berms were necessary to manage slope height and angle. The geotechnical berms improve multi-bench slope stability in the upper slope and allow for management of any potential instability.
3. The analyses also highlighted the sensitivity of slope stability to groundwater, with FoS reduced by 0.2 for the wet season groundwater model. In ground conditions with sub-vertical foliation and a

deformed rock mass, depressurisation through horizontal drain holes is particularly effective at improving slope stability.

An alternative design option for mine planning to consider was the incorporation of an additional haul road in the upper slope instead of the geotechnical berms. This would provide the necessary slope breaks and alternative access if required. The existing pit design had a single access haul road that traverses all walls, any failures that impact this haul road would have significant consequences for ore recovery.

Horizontal drain holes (HDH) were recommended, with a length of at least 50 m and at a spacing of 20–25 m. The holes would need to be lined with slotted PVC. Careful planning and management of the water produced by the holes would be required, including location of lined sumps on the geotechnical berms. Due to a lack of pore pressure, data assumptions were made regarding groundwater levels, pore pressures and the effectiveness of the HDH. These assumptions have significant implications for slope stability analyses. It was recommended that vibrating wire piezometers be installed to confirm pore pressure assumptions within the stability analyses.

3.3.2 Ultimate pit: weathered and fresh rock

To assess slope stability multi-bench and global scale slope stability of the Stage 2 west wall several failure mechanisms were considered relevant and appropriate analysis methods were utilised. The LE analyses and kinematic results are within industry standard design acceptance criteria. The kinematic analyses indicated flexural toppling is the critical control on slope stability. This is consistent with observed multi-bench slope failure mechanisms in the weathered zone to date.

With regards to flexural toppling failure, due to foliation dipping steeply back into the slope, this was assessed using 2D finite element (2DFE) stability analysis. The 2DFE stability analysis was undertaken for a single representative cross-section through the west wall. The analyses were completed using Rocscience's RS2 software. The analyses considered variable slope geometries, groundwater scenarios and foliation characteristics. Other salient features of the model are summarised as follows:

1. Foliation defects were incorporated into the model as a parallel deterministic 2D 'joint' network with defect spacing matched to observations. Foliation was modelled at 80°, dipping back into the slope.
2. Modelled defects extend continuously across the slope to reflect the extensive continuity of natural foliation defects.

The analyses indicated that for the original design, prior to excavation of the −80 mRL bench (second bench in fresh rock), slope deformation is limited to the upper slope with localised toppling at the crest resulting in total displacements in the order of 250 mm, Figure 6a. This was in keeping with observations of slope behaviour and deformation in the slopes at that time. Following excavation of the −80 mRL bench, the 2DFE analysis predicted the formation of a deep-seated toppling failure mechanism involving the coupling between slope deformation in the upper slope and lower slope (Figure 6b).

It was noted previously that toppling failures are highly sensitive to pore pressures and that there was notable uncertainty around this input. It is also of note that 2DFE modelling of flexural toppling is highly sensitive to other inputs, such as defect stiffness, for which there is also significant uncertainty at the case study site (and most sites). Sensitivity analyses were run to provide a range of results to be considered and benchmarked against observed slope behaviour to date.

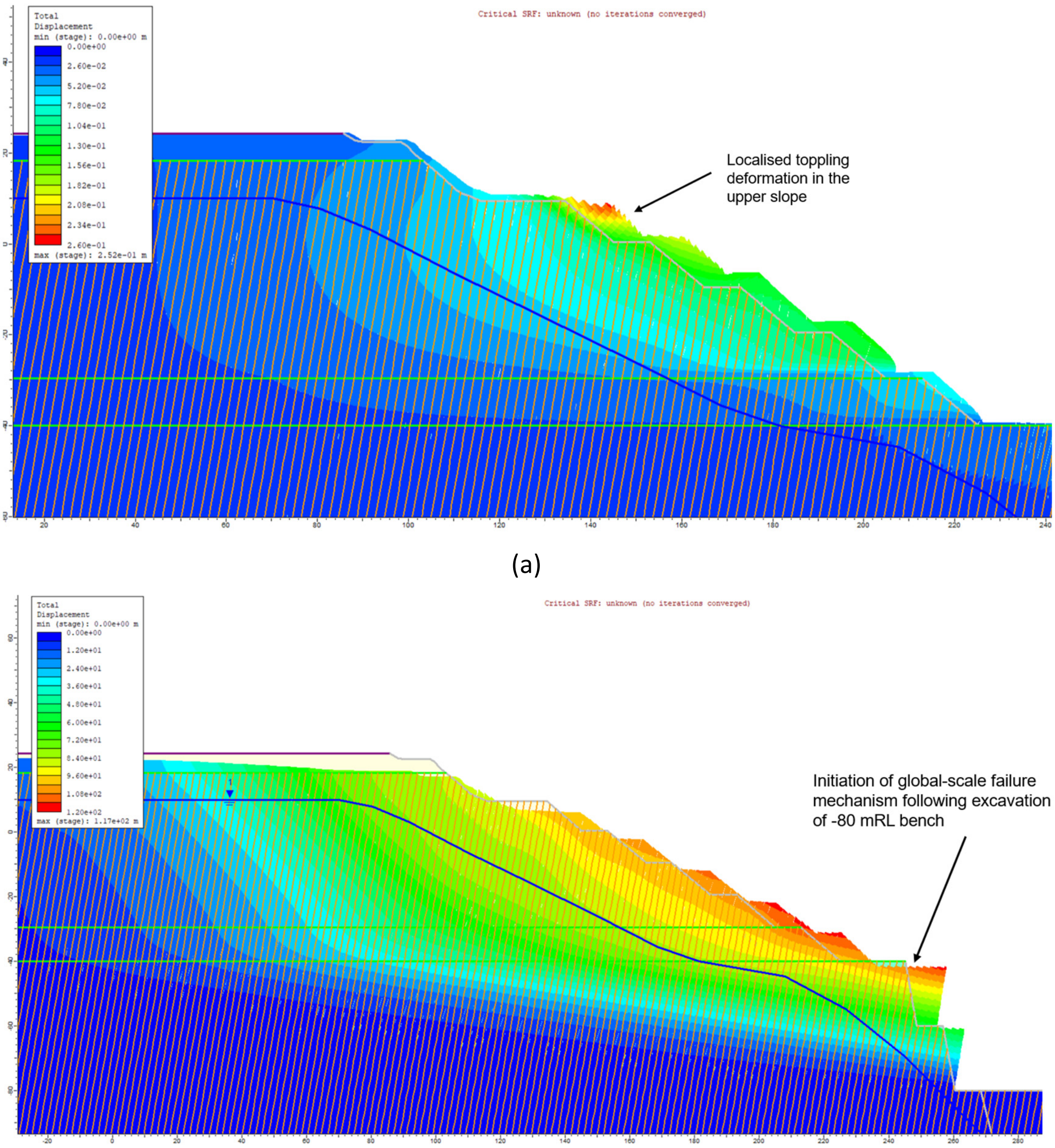

Figure 6 Example 2DF3 model showing predicted slope deformation in upper slope following excavation to (a) −60 mRL and (b) the −80 mRL bench

To eliminate the risk of deep-seated toppling, unrealistically shallow overall slope angles are required. The formulation of a slope design needs to consider the geotechnical model and slope failure mechanisms, as well as the expected operational life and slope management practices.

With consideration to a short-term (1–2 years) pit slope design life a deformable slope geometry was adopted. This design still resulted in a minor flattening of the inter-ramp angle due to a reduction in bench height from 20 to 10 m. The slope deformation was expected to be manageable using strict operational controls including:

- Surface water is effectively managed to minimise surface water infiltration, particularly into the east and west walls.
- Slope depressurisation drilling is completed as mining progresses through the lower slope. Horizontal drain depressurisation will require drain holes at least 50 m deep to be effective.

- Good wall control blasting practices are employed to minimise damage to the surrounding rock mass with the primary concern being dilation of foliation within a blast-damaged zone impacting bench and bench-scale instability.
- Extensive monitoring to be undertaken including drone inspections, prisms, radars, and crack meters.

These analyses confirmed the existing understanding that toppling failures are sensitive to slope height and groundwater. In weak materials deep-seated toppling failures can develop at low inter-ramp angles.

4 Design implementation

4.1 Slope performance

While slope deformations and some failures were still observed, the overall slope performance was improved, attributed to both the design and depressurisation program (Figure 7). In addition, there was a significant uplift in slope monitoring that enabled improved risk management and reduce stand-down times after a failure or heavy rainfall. The depressurisation and slope monitoring program are discussed further in the following sections.

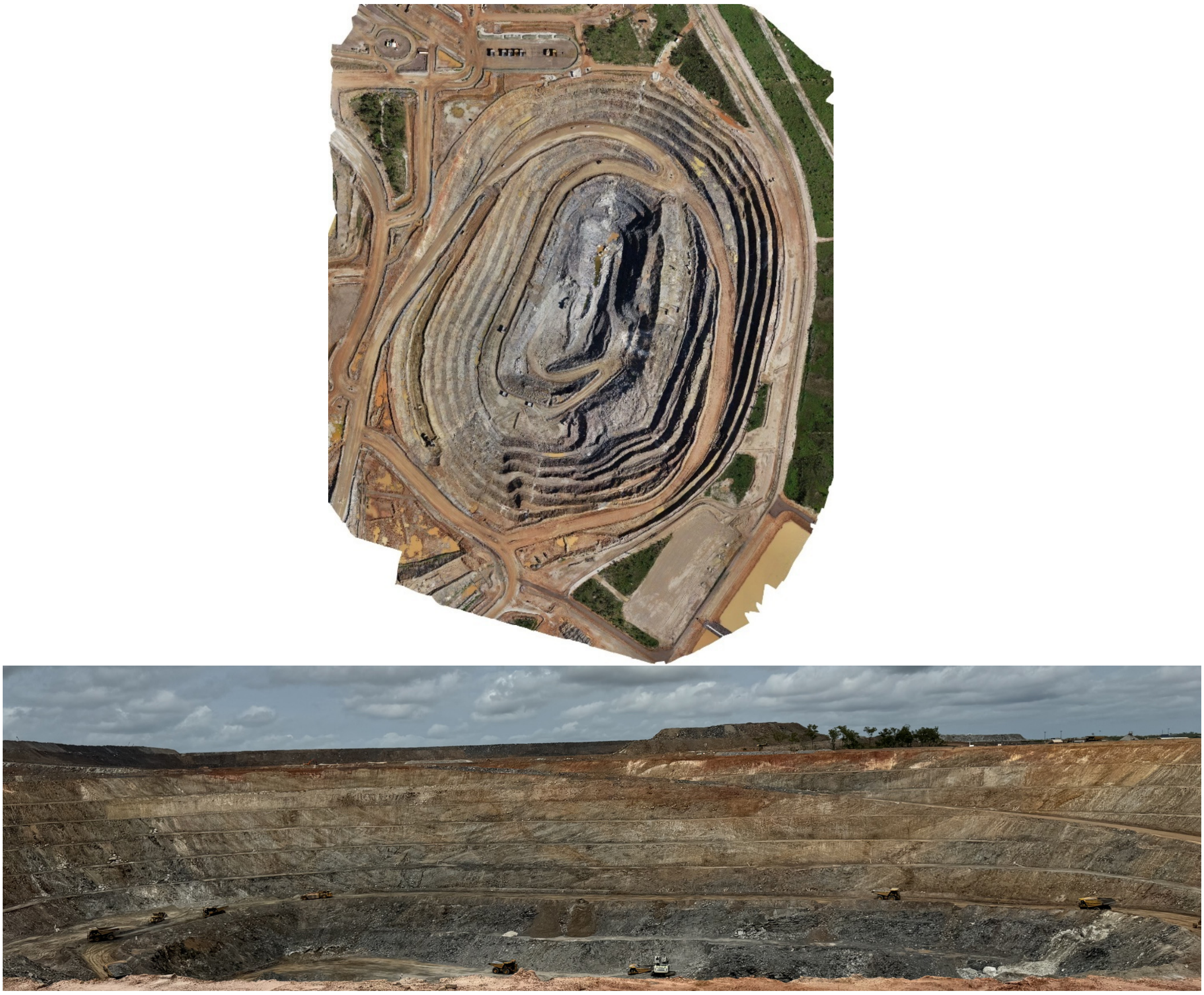

Figure 7 Aerial view of the site and an overall photo of the upper west wall in December 2023

4.2 Depressurisation

From industry experience and the stability analyses of this study it was known that toppling failures are highly sensitive to pore pressures. Slope management practices at the site have been updated to include the installation of 25 m long HDH installed at a spacing of 25–30 m spacing in the upper weathered zone. This was the maximum length possible for a rig that was already available on site, with the cost of a rig that could achieve the 50 m considered prohibitive at the time. The HDH are lined with slotted PVC to assist in maintaining the holes. The HDH are shorter than was recommended but could be rapidly achieved with a rig available on site. Sourcing of a specific rig for the HDH was cost prohibitive for the site.

The drain holes were effective at producing water (Figure 8a). Initially the water was allowed to drain and accumulate on berms, which allowed the water to re-enter the system as well as exacerbating trafficability issues. Consequently, a reticulation system was developed to capture the water and transport it to a lined sump on the endwall (Figure 8). The reticulation system was developed by site personnel and was cost-effective with material sourced from the local hardware store.

Given the limited length of the HDHs the volume of water discharged is notable and indicates that substantial groundwater remains within the slopes (Figure 8). The water is likely contained along foliation and the HDH are ideal for targeting this. In addition, there is potential for compartmentalisation and perched water tables around veins, faults and other significant structures.

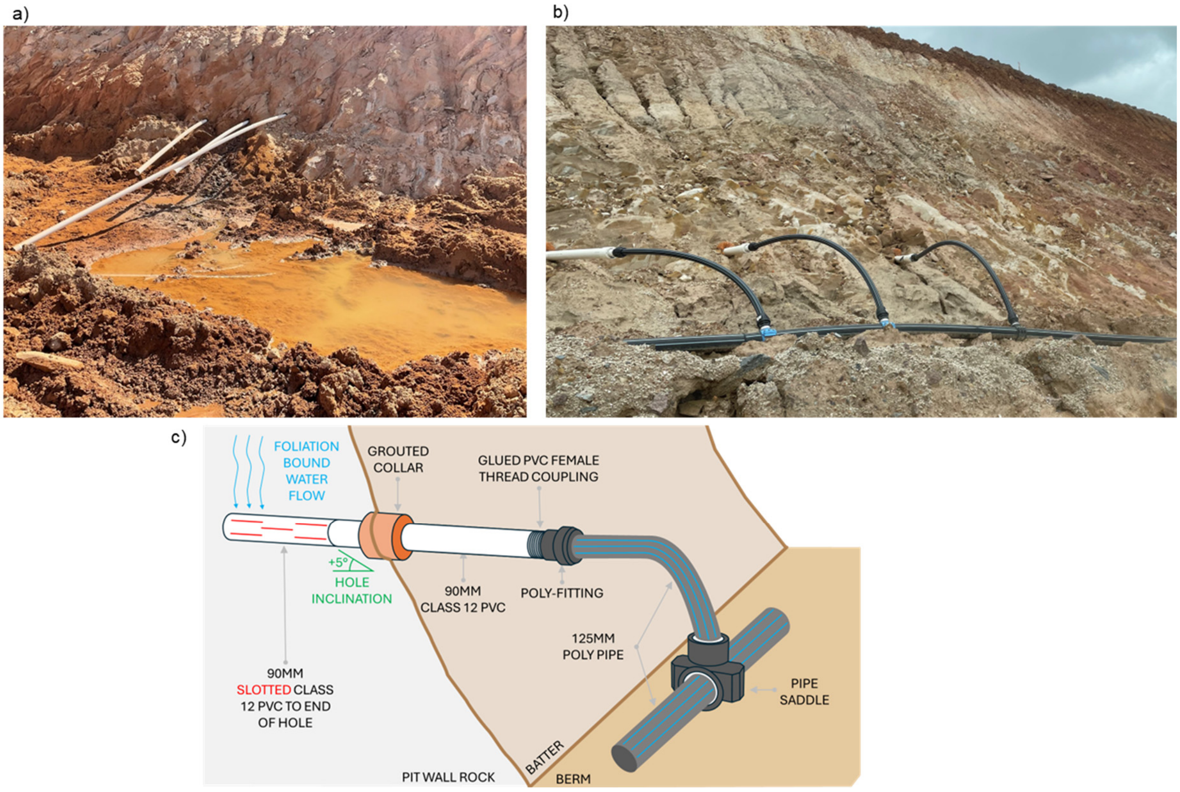

Figure 8 Horizontal drain holes. (a) Initial installations with water accumulating on berms; (b) Reticulation system installed; (c) Conceptual sketch of the installation

4.3 Monitoring

With ongoing operations, daily inspections continued to be an important part of the slope management program. However, there was a significant uplift in the slope monitoring program to support observations made in the field. The additional monitoring and instrumentation are summarised in Table 3. The weekly geotechnical hazard maps were able to be supported by data. This improved general slope and risk management, allowed improved communication, confidence in the production team and reduced stand-down times.

Table 3 Uplifted monitoring program

Slope monitoring	Comment
Daily drone flights	Excellent communication tool, consistent flight path flown each day by field technicians Weekly pit digital terrain model draped with image
Survey prisms	Coverage improved with an uplift from 5 to 55 prisms monitoring Stage 2 slopes Measurements still picked up manually once a week by survey department
Crack extensometers	Simple crack extensometers established on tension cracks around the pit crest Monitored daily by field technicians
Radar	Real time monitoring and alarming for east and west wall Critical monitoring for maintaining safe access to the pit

5 Conclusions

The case study presented highlights the critical importance of a well-developed geotechnical model in understanding and managing slope instability mechanisms, particularly flexural toppling, in foliated rock masses. Poor initial slope performance at the site was largely attributed to an overestimation of the rock mass strength in the weathered zone.

Toppling failures – primarily flexural and complex – were the dominant failure mechanisms, exacerbated by steeply dipping foliation, high pore pressures. These failures were difficult to manage with limited slope monitoring resulting in significant impacts to ore recovery. The refinement of the geotechnical model during operations, coupled with targeted design updates and operational controls, significantly improved slope performance. These updates included slight reduction in inter-ramp angles, introducing wide geotechnical berms, and implementing a cost-effective depressurisation program.

The study also demonstrated that even in short-life operations, appropriate slope design is critical. For toppling failures the design is highly sensitive to pore pressures, slope heights and angles. In weak materials like the upper weathered zone, deep-seated flexural toppling mechanisms can develop even at very low inter-ramp slope angles. The success of the implemented design changes relied heavily on ongoing observational input, practical adjustments in HDH designs by the site team, and a major uplift in slope monitoring systems.

Overall, the experience at this site reinforces the value of an adaptive, model-informed approach to slope design and management, particularly in structurally complex geological settings.

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