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MATAKANUI
GOLD LIMITED

GEOTECHNICAL ASSESSMENT
OPEN PIT & UNDERGROUND MINING
RISE & SHINE DEPOSIT

BENDIGO-OPHIR GOLD PROJECT
OTAGO, NZ

REPORT 24053

Prepared for:

Matakanui Gold Limited
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In association with:

GEORGE, ORR and Associates (Australia) Pty Ltd Consulting Engineering Geologists

EXECUTIVE SUMMARY

Geotechnical assessment of proposed mining at the Rise and Shine (RAS) deposit has been based on discussions with Matakanui Gold Limited geological and mining personnel, inspection of the general topographic setting, surface exposures and cores from geotechnical and selected exploration/ resource definition boreholes, and information/ data gathered for and results from prior geotechnical investigation.

Mining at RAS will be carried out in schists of the Haast Group from Textural Zone Three (TZ3 chlorite zone) and Textural Zone Four (TZ4 biotite zone).

This report concludes that, with the recommended program of ongoing analysis and application of appropriate, practicable methods in mining and ground performance monitoring; open pit and underground stability of planned excavations at RAS can be managed safely and adequately.

Open Pit Mining

Open pit mining at RAS will be influenced significantly by the typically *poor* rock mass quality of country rocks comprising dominantly TZ3 schists and lesser proportions of the upper zones of the TZ4 schists; major structural features, the Thomson Gorge Fault (TGF), and RAS Shear Zone (RSSZ)/ upper TZ4; other faults (known but yet to be clearly defined) and groundwater pressures. It is possible that mine stability may be affected by transient seismic disturbances over the life of mine.

Foliation/ shear fabric, dipping shallowly to the north north-east is pervasive and has potential to be a significant/ dominant influence on pit wall stability locally. The significance of this influence varies locally and is dependent on the relative orientations of the wall and the fabric. Other faults exist; however, further structural interpretation is essential to identify potential for these features to adversely influence wall stability.

It is unlikely that hydrostatic pressures due to groundwater in wall rocks at RAS can be totally dissipated. However, active wall depressurisation using sub-horizontal boreholes is inferred to be applicable at RAS and is recommended to be included as an integral component of open pit mining.

Further hydrogeological investigation and assessment is required/ recommended to aim to identify the likely response and effectiveness of RAS lithology to implementation of active dewatering and wall depressurisation measures. Installation of piezometers is recommended on a number of sections to monitor groundwater pressures/ levels behind and beneath pit walls (and assess effectiveness of depressurisation measures).

Natural seismicity will be an intermittent and transient, but unpredictable hazard to mine stability at RAS; hence allowance for possible earthquake disturbance must be included in mine design. Earthquake disturbance applied in analyses was that from a 4.5M event occurring at the RAS site inducing a lateral acceleration of 0.13g. Events of greater magnitude are possible, albeit of lesser potential. For example, there is a 75% probability of an M8+ event on the Alpine Fault within the next 50 years. Greater disturbance from such events could result in pit wall instability.

Trim blasting methods are recommended for final wall development where blasting is required to enable productive excavation. However, it is recommended that final face profiles in TZ3 are always mechanically excavated, even when blasting. Presplit may be viable in more competent TZ4 schists (away from RSSZ).

Pit wall stability monitoring must be established as soon as is practicably possible following commencement of mining. Techniques which would be suitable include:

- Electro-optical distance measurement (EDM) of arrays of fixed prisms (preferably automated)
- LiDAR scanning to check compliance with design and incremental surface displacements
- Daily photographic/ video scan via georeferenced UAV/ drone-captured imagery, which can also be used to photogrammetrically map structures exposed in pit walls

Localised use of artificial reinforcement and support techniques may be required; however, these techniques cannot be applied generally at RAS. Limitations on the effectiveness of conventional reinforcement/ support techniques result fundamentally from generally *poor* rock mass quality which variously inhibits or precludes load transfer between the rock mass and reinforcing elements.

Wall exposures of poor/ very poor quality rock will locally require surface treatment (sealing and/ or support) to prevent/ retard slope degradation.

Given the complexities in loading and rock stress changes which will occur as mining progresses, conduct of three-dimensional analysis of proposed mining (open pit and underground) is recommended strongly. Complexities in stress distribution and re-distribution are related variously to topographic variation RAS lode geometry, throughgoing major structures, assessed *poo*r to *fair* rock mass quality, geometrically complex intersections between excavations and geological units, anisotropy in naturally weak materials, geological structures and groundwater influences.

Recommended base case wall design parameters for the proposed RAS pit are listed by design domain in Table ES1, with domains illustrated in Figure ES1.

Variations to previously recommended open pit wall designs are derived from:

- Assumed successful active depressurisation
- Application of anisotropic shear strength in TZ3, restricting applicability of the weakest shear strengths to directions sub-parallel to the TGF
- An alternative approach used in modelling hydrostatic pressures.

Note that stability analyses (in both previous and current assessments) assumed no wall damage from blasting.

Experience may well dictate that upper batters be limited to 10m height to (say) 30m depth to accommodate the inferred lesser competence in near-surface materials.

Potential upside in wall design parameters (noted in Figure ES1) is inferred based on consideration of:

- Higher Factors of Safety (FOS) obtained in the modelled pit sectors
- The unrealistic assumption the analyses make concerning geometry of geological units and structures into the third dimension
- In reality wall stability will be greater than suggested by two-dimensional modelling. confinement.

Table ES1 RAS open pit recommended base case wall design parameters

SECTOR	FACE HEIGHT	FACE ANGLE	BERM WIDTH	INTER-RAMP ANGLE	Potential upside
WEST	15m	50°	9m	34.8°	Unknown
NORTH-EAST	15m	60°	6m	45.7°	? 49.1° IRA
EAST	15m	60°	7.5m	42.9°	? 45.7° IRA
SOUTH-WEST	15m	50°	7.5m	36.8°	Unknown

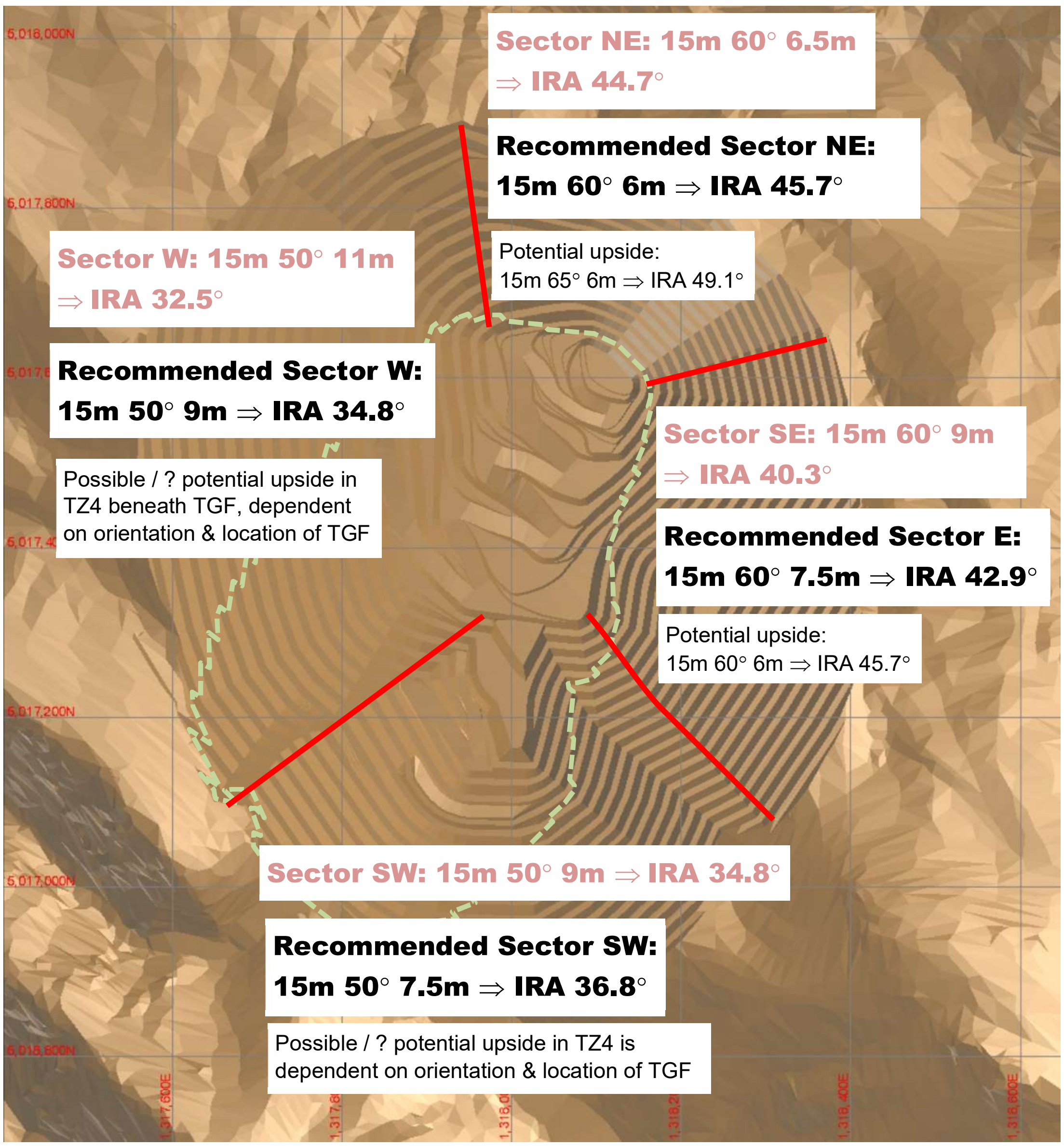

Figure ES1 Proposed open pit wall design parameters
Pit wall intersection with TGF shown as dashed green outline

Underground Mining

Development of a decline access in establishment of underground operations at RAS must deal with the *poor* rock mass quality TZ3 schists and upper zones of the TZ4 schists for several hundred metres from the planned decline portal position in the southern flank of Shepherds Valley.

Ground reinforcement and support schemes for the upper decline and upper levels stope access are to be based on floor to floor application of fibre-reinforced shotcrete (FRS) and bolts. Locally *fair* to *good* rock mass conditions cannot be assumed to compensate for the dominant influences of *poor* and *very poor* zones.

Provision for groundwater drainage through the applied FRS will be essential. While an array of holes bored through the FRS provides the simplest drainage system, other more sophisticated methods are also available (for example, mats or mesh incorporated into the surface support to capture and control inflow).

Inspected cores (and core photographs) from the lower zones of the TZ4 schist indicate significant improvement in rock mass quality at depth, which is anticipated to permit gradual reduction in ground support capacity in lower levels mined beneath the hangingwall of TZ4 and the RSSZ. It is anticipated that the ground support scheme in lower levels may be based on friction bolts and mesh.

The TGF and RSSZ are dominant regional structures which have potential to adversely impact future stope back stability. Undercuts resulting from stope back overbreak into/ through the TGF and exposing the TZ3 would be prone to cave, increased ore dilution and possibly compromising stope stability in adjacent areas.

Given the *poor* quality of the rock mass at the footwall of TZ3 (immediately overlying the TGF and RSSZ) non-conventional stoping methods will be required to recover ore at the TGF-TZ4 hangingwall contact to avoid excessive dilution and preclude uncontrolled caving into TZ3.

Options outlined for this mining are variously based on high capacity ground support schemes (full surface support and closely spaced bolting); and formation of temporary sill pillars followed by controlled panel caving and formation of artificial pillars.

Modestly sized longhole open stoping (LHOS) can be adopted for recovery of TZ4/ RSSZ ore beneath the supported hangingwall zone. Stoping panel sizes must be limited to prevent development of excessive cumulative hangingwall spans.

LHOS must be mined in a primary-secondary stope sequence. Hydraulically placed engineered/ cementitious backfill is required to facilitate extraction of secondary stopes and maximise tight filling.

Assessment of empirically derived ratings for typical rock mass quality, indicated sustainable LHOS spans are governed by stope back capacity, where the hydraulic radius (HR) for unsupported ground is ~ 1.4m (say 10m × 4m) and radii for supported ground range from 3.3m (20m × 10m) to ~ 5.0m (20m × 20m). Unsupported stope sidewalls have HR between ~ 8.0m and ~ 12.0m. It is not practicably/ economically possible to provide support to LHOS sidewalls.

Assessment of potential for development of subsidence of the area(s) directly overlying stoping panels is necessary and in the first instance can be examined as a component of recommended three-dimensional numerical modelling.

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1. Introduction

This report summarises preliminary findings of a review of proposed open pit and underground mining of the Rise and Shine (RAS) deposit at Matakanui Gold Limited's (Matakanui Gold, Matakanui, MGL) Bendigo-Ophir Project in the Central Otago goldfields of the South Island, New Zealand.

The review has considered geotechnical investigations and assessments performed (variously by others) on ground and groundwater conditions influencing surface and underground mining at the deposit and the design parameters derived, in turn, for potential open pit development and underground stoping.

The review has been performed at the request of Rod Redden, Study Manager of Matakanui.

1.1 Scope of Work

The requested work entailed:

1. Initial review and feedback regarding the geotechnical guidelines provided for the main RAS pit, identifying whether there are clear opportunities for improvement.
2. Full first principles investigation and analysis through to mine design guidelines to support Reserves for RAS.

This report presents a summary of review findings and recommendations compiled to date.

1.2 Data Sources

Findings and recommendations provided are variously based on:

- Discussions held severally and variously with Matakanui personnel, including Damian Spring, CEO; Sam Smith, Executive Director; Alex Nichol, Geology Manager; Mark Mitchell, Technical Services Manager; and Rod Redden, Project Manager.
- No sub-surface ground investigations were performed for the current assessment; rather assessments presented herein rely heavily on information presented in PSM Consultants Pty Ltd Report (PSM) PSM5131-003R [1] (draft) dated July 2024 to Santana Minerals Limited (Santana, SMI).

  PSM investigations were performed according to current industry practice and reported results are accepted as reliable.
- Information and summary details of investigations, assessment and recommendations contained in:
  - *Structure and geochemistry of the Rise & Shine Shear Zone mesothermal gold system, Otago Schist, New Zealand* – Cox, MacKenzie, Frew, Craw & Norris
  - *Probabilistic seismic hazard assessment of New Zealand* – Stirling, McVerry, McGinty, Villamor, van Dissen, Cousins & Sutherland
  - Santana Minerals
    Bendigo-Ophir Gold Project Pre-Feasibility Study report, 2024
  - Matakanui Gold general geology, topography

Inferences drawn regarding ground conditions and possible mining configurations from investigations at RAS can be applied for high level assessment pf open pit mining at the Srex (SRX)/ SrexEast (SRE) and Come in Time (CIT) satellite deposits; SRX/ SRE to the south south-east and CIT to north north-west of RAS.

2. Background Information

2.1 Location & Past Mining

The Bendigo Goldfields region is located ~ 17 km north-east of Cromwell, Central Otago, New Zealand (Figure 1). The RAS deposit lies in the Dunstan Mountains/ Range.

Gold mining in the Bendigo Goldfield was initiated in the early 1860s, following discovery of alluvial gold in the quartz rich region. Mining of alluvial and in situ gold (open pit and underground) continued until the early 1940s when operations ceased production following withdrawal of a government funded mining subsidy.

Intermittent exploration in the region has been performed since the 1980s; however, no operations of significant scale have been established to date.

2.2 Geology

Information is variously derived/ paraphrased from the following documents, and discussions with SMI Geology personnel:

Cox, L., 2003
Rise and Shine shear zone, Central Otago, New Zealand
BSc Honours thesis, University of Otago

Cox, L., D. McKenzie, D. Craw, R. Norris & R. Frew, 2006
Structure & geochemistry of the Rise & Shine Shear Zone mesothermal gold system, Otago Schist
NZ J Geol & GeoPhys, 2006, Vol 49: 429-442

PSM Consult Pty Limited, 2024
Bendigo-Ophir Gold Project, Pre-Feasibility Geotechnical Assessment
PSM Report PSM5131-003R DRAFT report to Santana Minerals Limited (unpublished)

Institute of Geological & Nuclear Sciences (GNS), various
General geological background information

The basement rock of the Dunstan Range is formed from the Haast Schist Group, a grey quartz-feldspathic metagreywacke interlayered with micaceous meta-argillite and greenschist. The schists have a pervasive schistose fabric, with shallow dips reflecting broad regional-scale warps which developed during regional deformation.

The Dunstan Mountains are a doubly-plunging domal culmination with limbs dipping to the north-west, north-east, south-east and south-west. The core of the mountains in the south are strongly foliated and segregated garnet-biotite-albite zone schists (Torlesse Terrane Textural Zone IV (TZ4)). These schists are separated to the north by the gently north north-easterly dipping normal-slip Thomsons Gorge Fault (TGF) from a chlorite schist zone (Torlesse Terrane Textural Zone III (TZ3)). This low-angle fault zone has resulted in shear zone-hosted gold mineralisation in the form of the Rise and Shine Shear Zone (RSSZ).

The TGF and RSSZ dip gently to the north north-east at ~ 20 to 25°. The regional scale TGF is (unusually) consistently and uniformly planar.

The TGF is a regional metamorphic discontinuity, ‘thinly’ separating lower-grade TZ3 schists against higher-grade TZ4 schists (Figures 2 and 3). The TZ4 core of the range is effectively a range-scale footwall block with the TZ3 schists forming northern and southern hangingwall blocks.

RSSZ-hosted quartz-gold mineralisation extends at least 16 km across the Dunstan Range. The shear zone is on average ~ 21m in width (intrashear zone) and up to ~ 90m locally and is bounded by main hangingwall and footwall shears.

Cox (2003) implies that the RSSZ is unlikely to be the western continuation of the Hyde-Macraes Shear Zone.

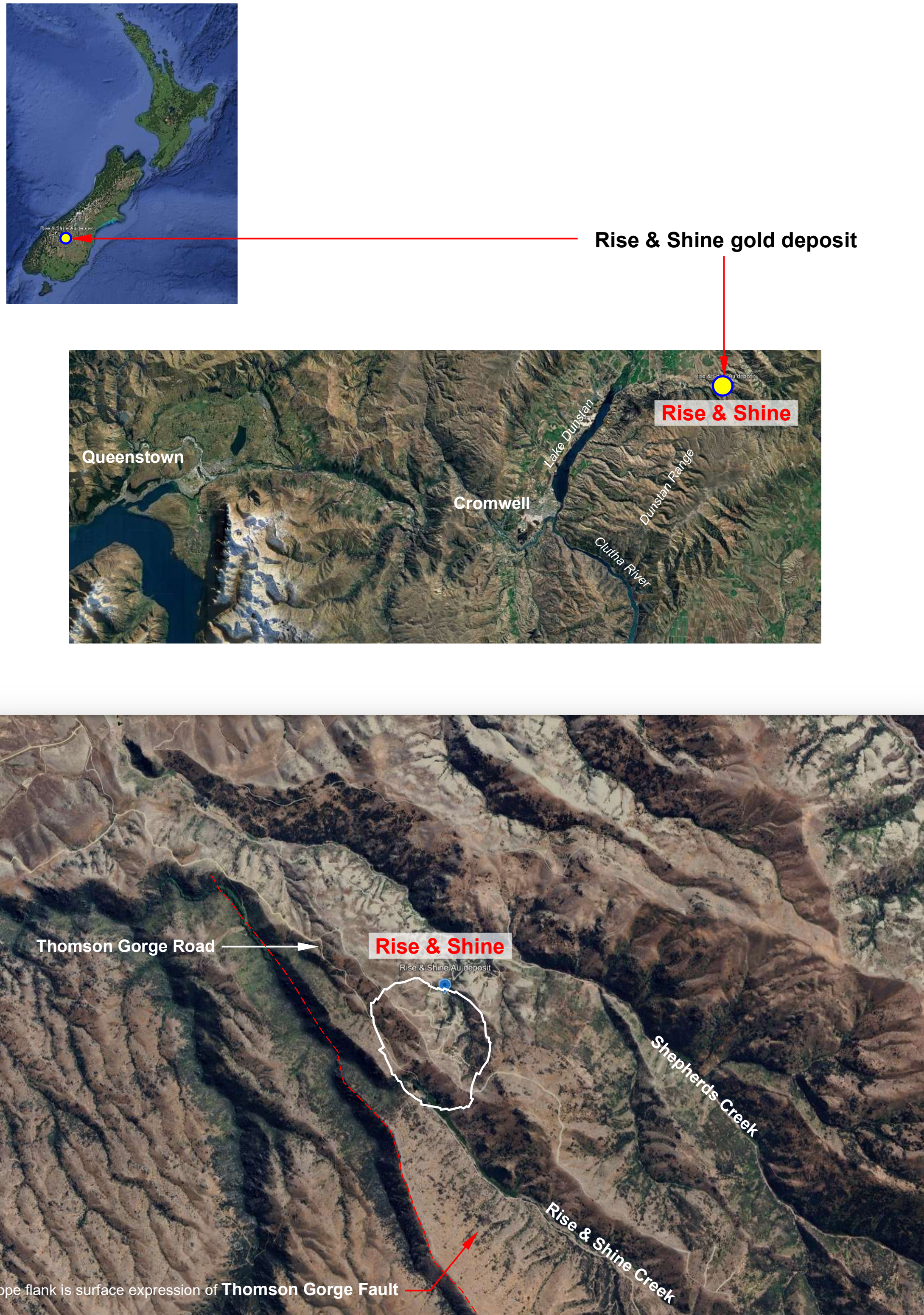


Figure 1 **Rise & Shine deposit location with crest footprint of conceptual open pit design**
Dashed red line is fault intersection with topographic surface

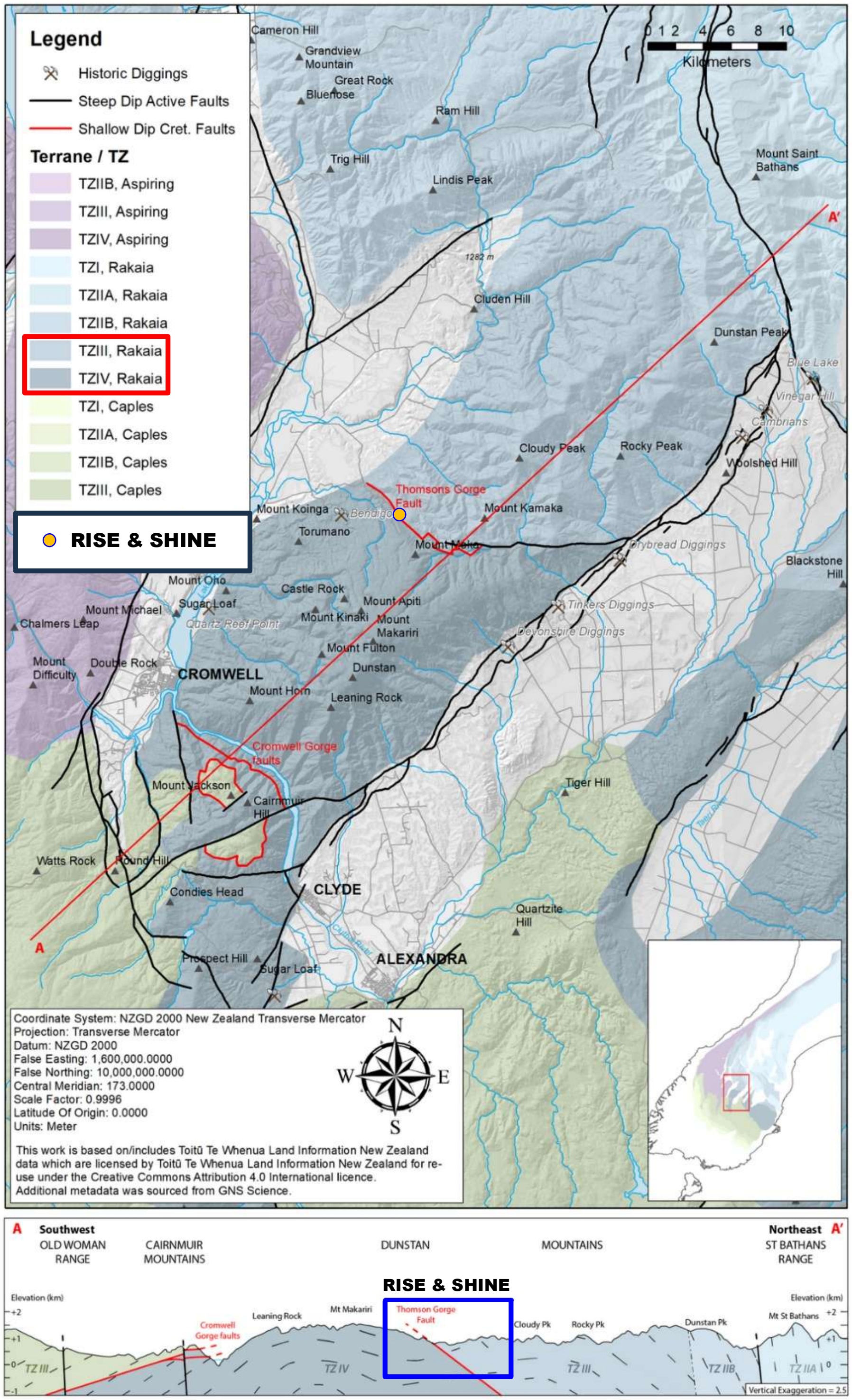


Figure 2 Map & geological cross-section illustrating textural zones of Haast Schist in the Dunstan Mountains region, Otago, New Zealand
Cross-section inspired by Mortimer *et al* (2023) NZJGG

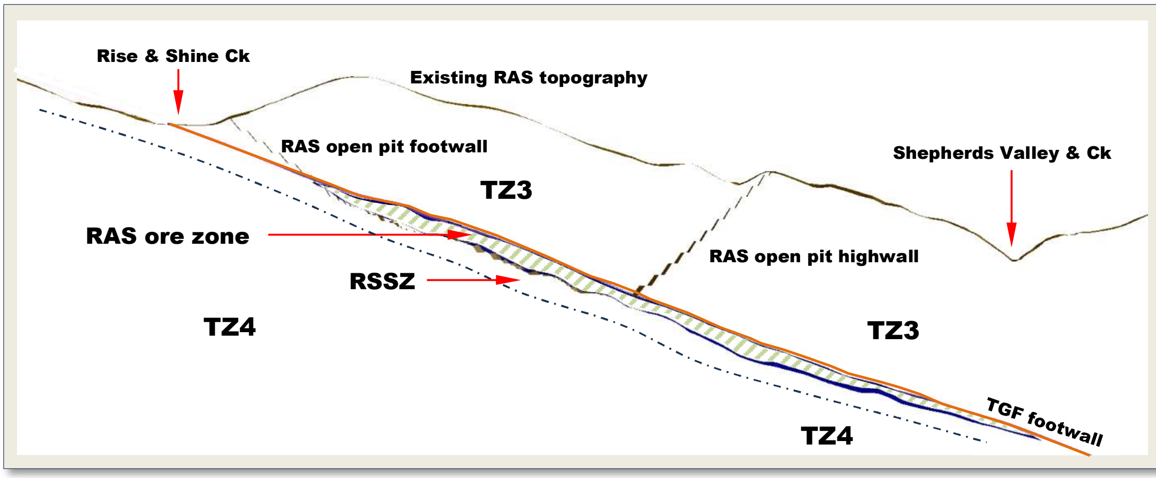


Figure 3 Schematic section on notional centreline of RAS ore zone (Section bearing ~ 010°)

2.3 Proposed Mining

The review has examined a number of open pit designs/ concepts and one (1) underground configuration provided by Matakanui:

- The current conceptual ultimate pit design (File: *ras_phase7_v3_clipped.dtm*) is shown as Figure 4. In this configuration the minimum floor elevation is 385mRL; maximum ultimate wall height is ~ 315m (west wall); while the elevation difference between the highest crest position and the pit floor is ~ 405m. The strike length of the pit (sub-parallel to the dip direction of the orebody) is ~ 945m, with a maximum width of ~ 950m.

  Wall profiles for this conceptual design are based on 15m high batters mined at 50° or 60° with berms of various widths (depending on assessed rock mass quality/ competence):

  - 15m high face, 50° face angle, 9m wide berm, for an inter-ramp angle (IRA) of 34.8°, W & NW walls
  - 15m, 60°, 6m, IRA of 45.7°, NE wall
  - 15m, 60°, 7.6m, IRA 42.7° E wall
  - 15m 50°, 7.5m berm, IRA 36.8°, S & SW walls

  The ultimate pit is to be mined in seven (7) stages.

  Figure 4 shows the completed seventh (7th) stage.

  Figure 5 shows the initial five (5) development stages of the pit.

  Figure 6 shows Stage 5 against the proposed ultimate pit.

  Figure 7 is a long-section showing Stage profiles on the approximate mid-line of the ore zone

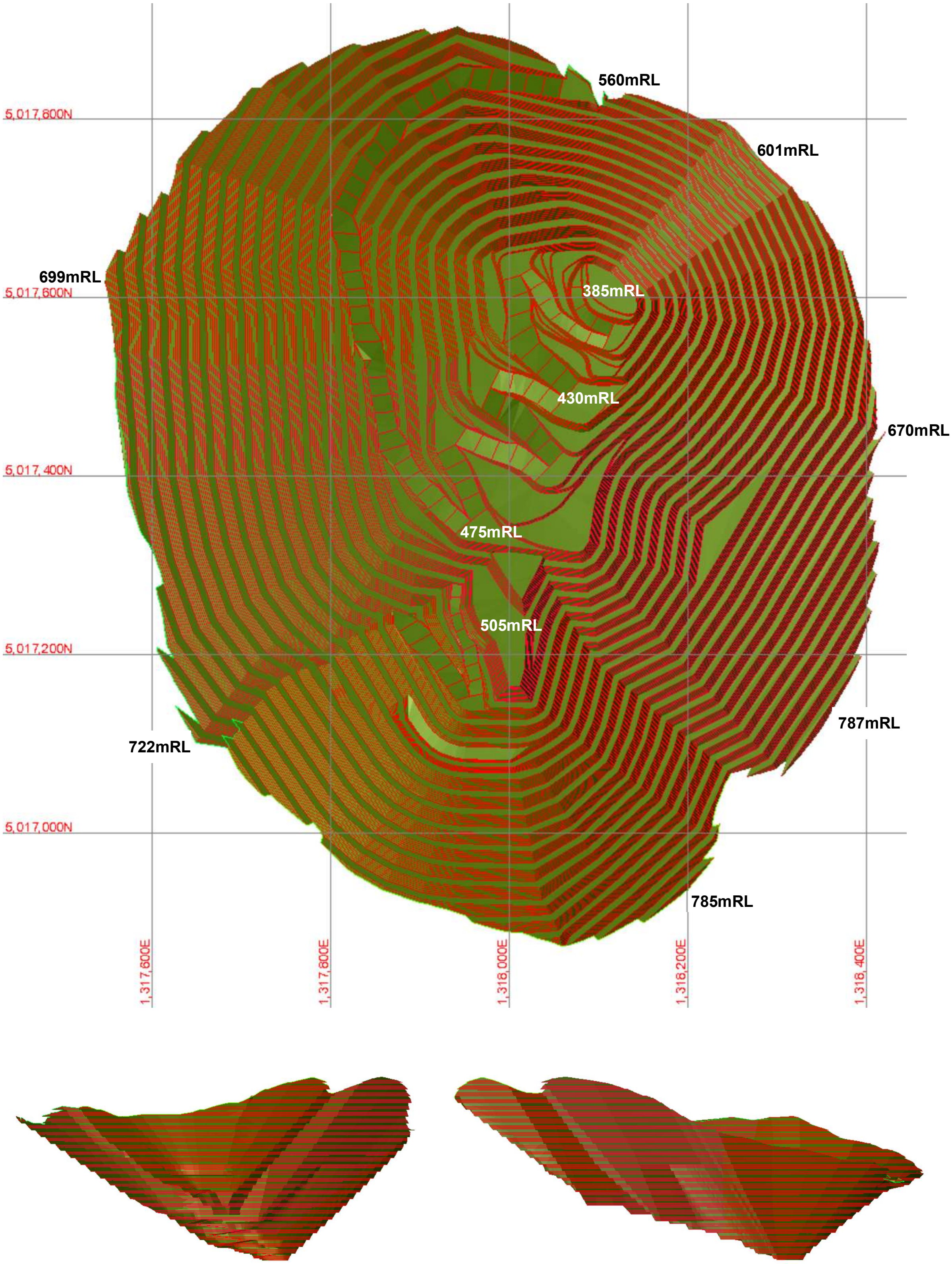
- Conceptual underground configuration (Figures 8 & 9)

  File: *v10 solid* Access development
  File: *v10 tall stopes 10* Conceptual stoping panels

At present plans for underground mining are based on developing decline accesses from portals on the southern flank of Shepherds Valley (Figures 8, 9 and 10).

Following this plan, initial development would be in TZ3 for ~ 500m in the Main decline and ~ 570m in the Ventilation decline. After developing through the core of the TGF most remaining access development would be in TZ4. This conceptual access configuration requires some decline segments and local cross-cut access and production drifts to be developed in TZ3 (Figure 11).

The stoping blocks shown in Figure 10 are purely illustrative. The poor quality of the lower TZ3, TGF core and upper TZ4/ RSSZ preclude direct application of conventional/ contemporary methods. Discussion on this aspect is presented in Section 6, Underground Mining Assessment.



Profile looking north **Profile looking west**

Figure 4 Rise & Shine ultimate open pit design
Source: Matakanui, file: *ras_phase7_v3_clipped.dtm*

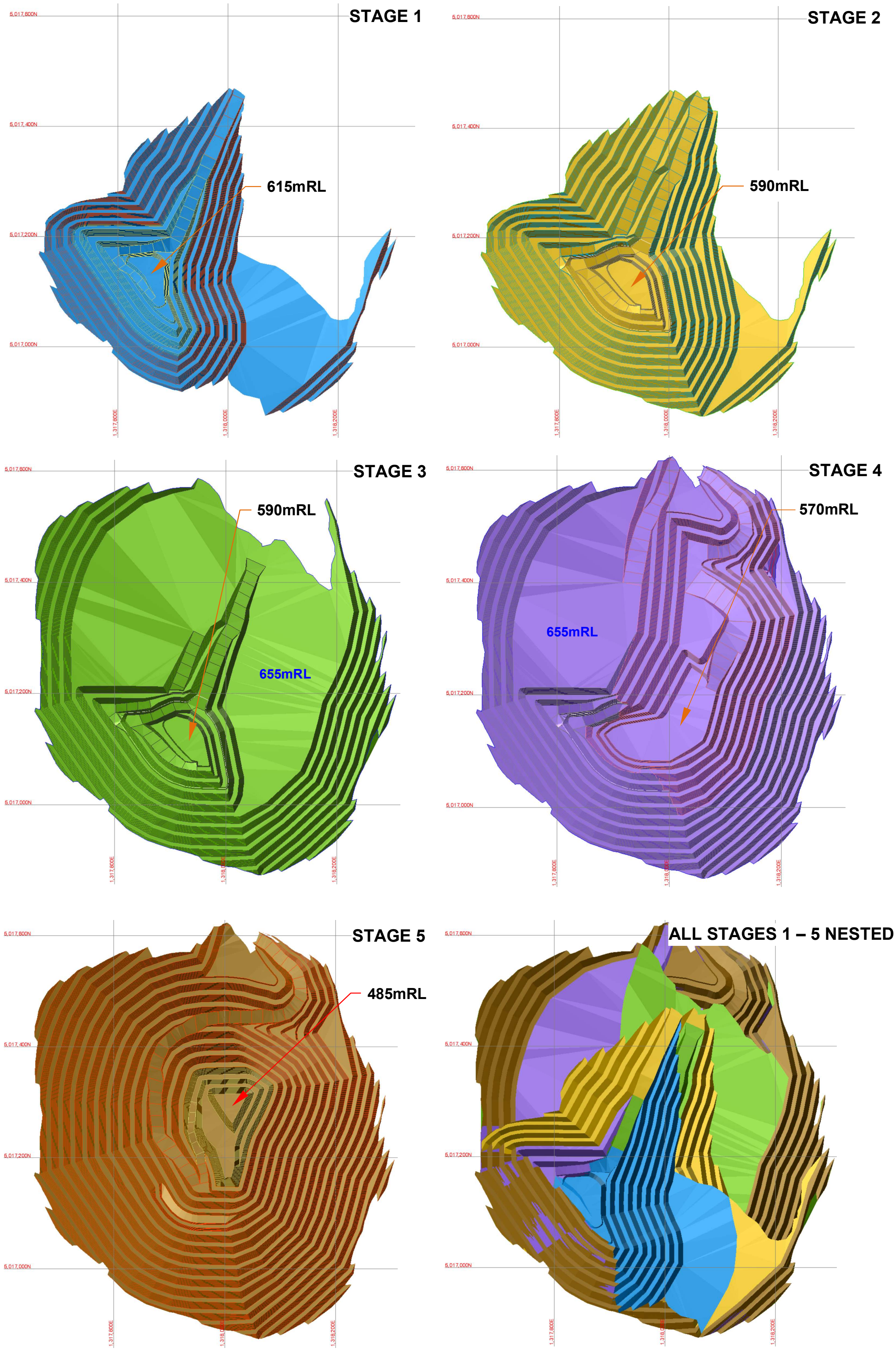


Figure 5 RAS open pit sequential development to Stage 5

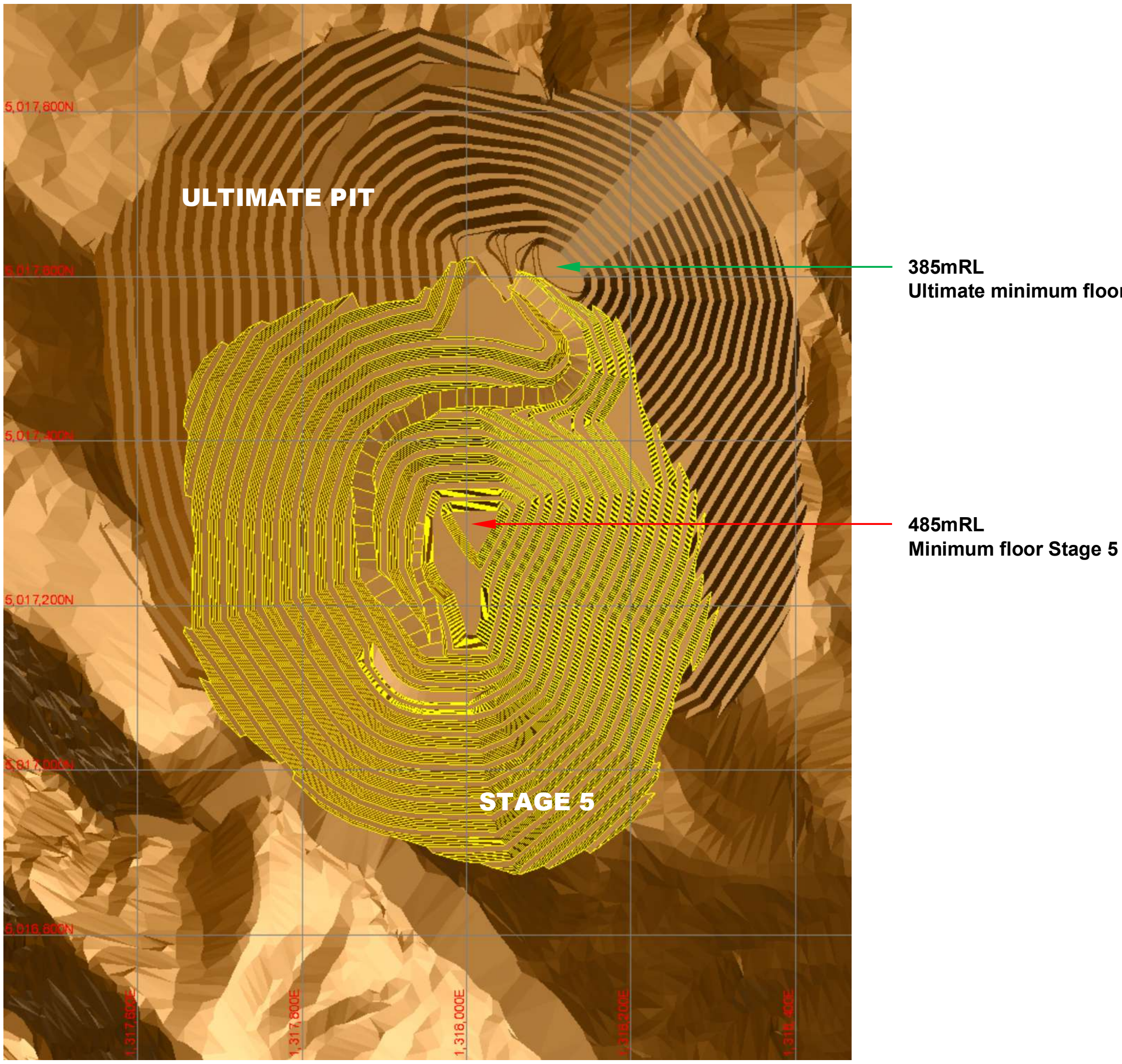


Figure 6 Rise & Shine: Stage 5 & ultimate open pit configurations
Source: Matakanui

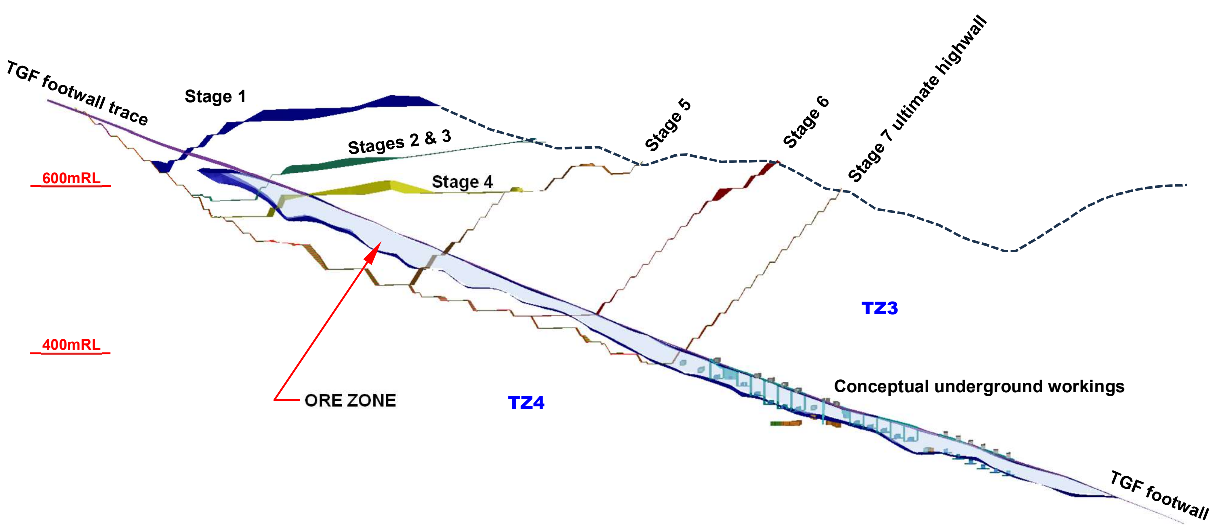


Figure 7 Rise & Shine: Stages 1 to 5 & ultimate long-section on approximate pit mid-line

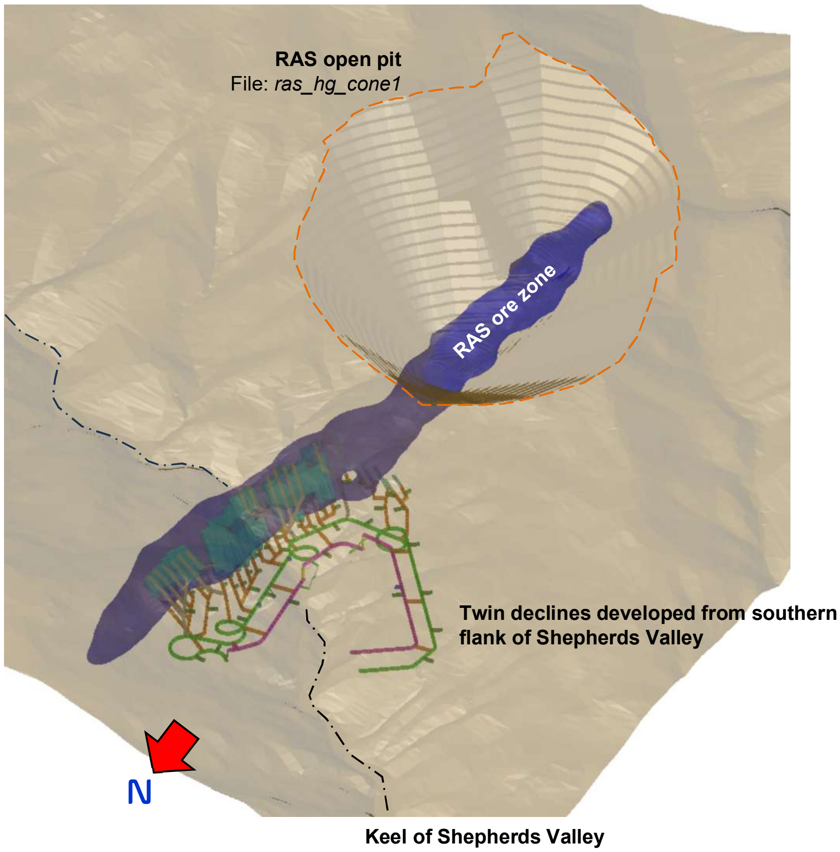


Figure 8 Conceptual RAS underground operations configuration & topography
Oblique viewing direction ~ 145°

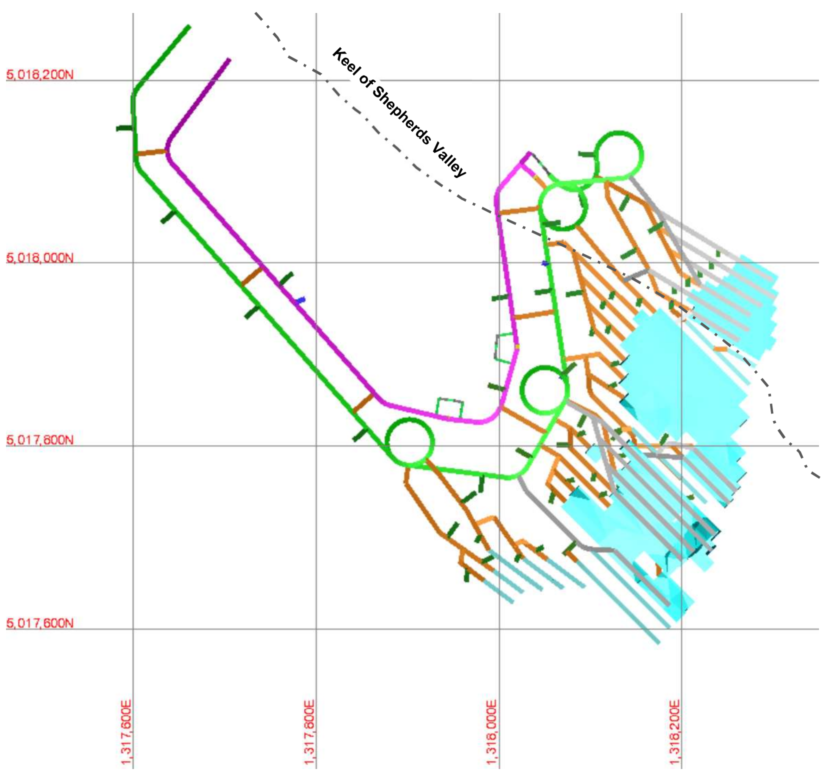


Figure 9 Plan view: conceptual RAS underground operations configuration

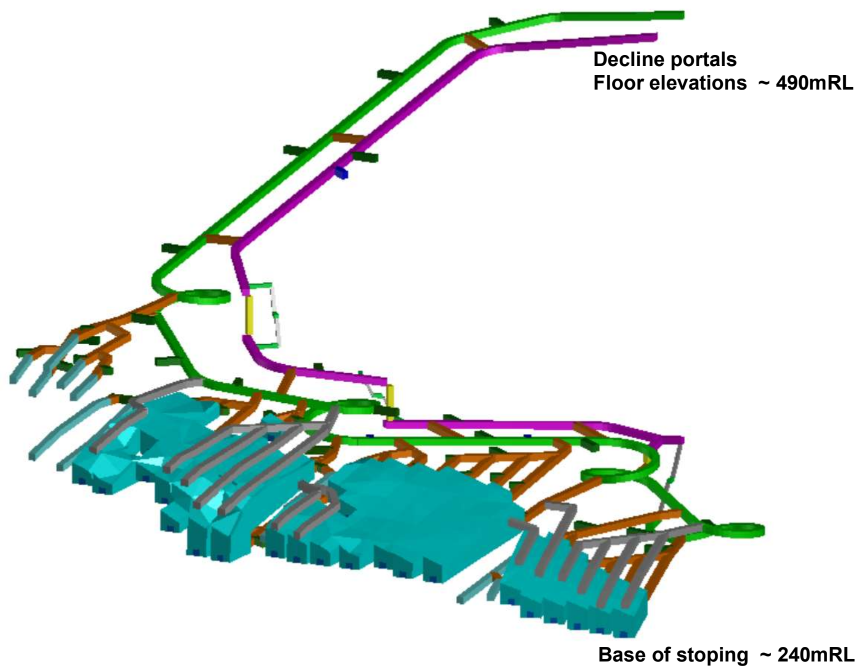


Figure 10 Conceptual RAS underground configuration
Oblique viewing direction ~ 290°

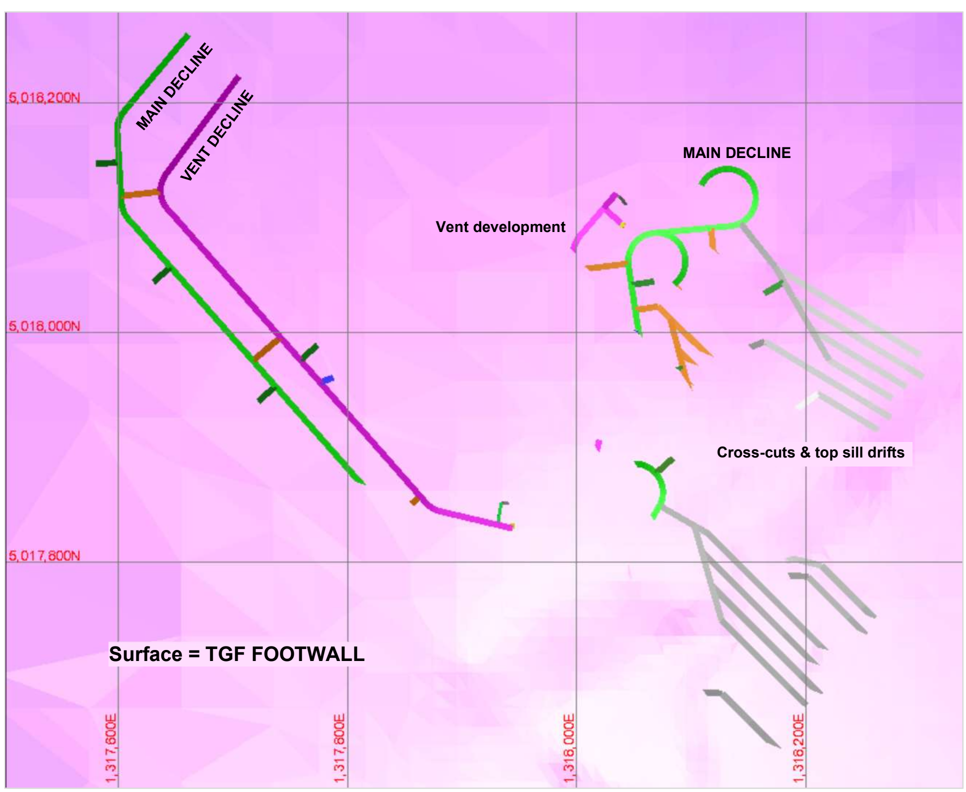


Figure 11 Conceptual RAS underground access segments above TGF (in TZ3)

2.4 Previous Geotechnical Investigation & Assessment

Previous PFS assessment was performed by PSM Consult Pty Ltd (PSM) and reported in mid-2024 [1].

The PSM study used industry-standard methods of investigation and assessment and was able to reference direct experience in geotechnical investigation, assessment and review of operations at the OceanaGold Macraes Gold Mine, located ~ 90 km east south-east of the RAS deposit. Macraes is set in similar geological and geotechnical settings to those at RAS; although the topographical settings of the two areas differ significantly.

The Scope of Work assigned to PSM was fulfilled, albeit that due to absence of specifically applicable data, some recommendations were generic, for example, relating to decline portal establishment, ground support schemes for underground access and stoping parameters.

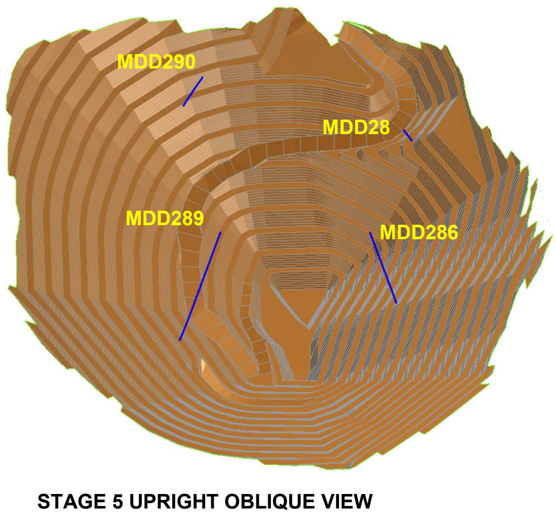
Data Adequacy & Assessment

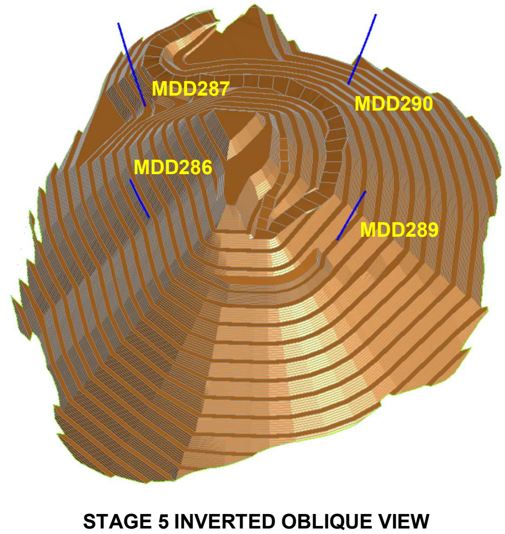
The RAS PFS had limited sub-surface data coverage. This is inferred to reflect time constraints and SMI has both awareness and acceptance of needs for further investigation and assessment. It is not unusual for PFS investigations to have limited investigative coverage, necessitating broad extrapolation regarding ground conditions and requiring further data for appropriate feasibility assessment.

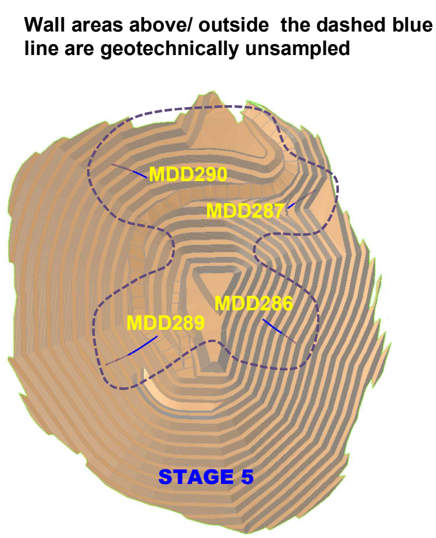
The four (4) centrally located geotechnical boreholes used in the PFS do not provide sufficient or adequate coverage for an open pit of the dimensions proposed for RAS. Much of the core from boreholes MDD286, 287, 289 & 290 is from well within the excavated void of any given pit, with only the lowest third of each borehole being close to or within a planned wall position.

Figure 12 shows the traces of these geotechnical boreholes with respect to intermediate Phase 5 and ultimate pit Phase 7 design (*ras_phase5_v3.dtm* and *ras_phase7_v3.dtm*) examined in this study. The upright and inverted images show the wall zones cores from these boreholes and reveal extensive areas which are unrepresented.

This temporary lack of relevant data has been redressed by SMI. A geotechnical drilling program has been designed and executed. Information able to be obtained from this program will be adequate in identifying ground conditions up to (at least) Phase 5 of the proposed open pit mining sequence.







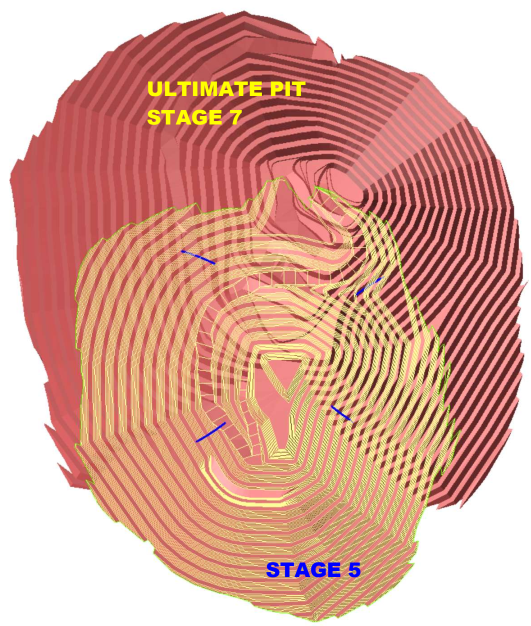


Figure 12 Coverage provided by geotechnical boreholes MDD286, 287, 289 & 290

While it is possible, there is no surety the geotechnical information obtained from the PFS boreholes (via conventional geotechnical logging and televiewer scanning) is representative of conditions in and behind the upper walls of a proposed pit; additional geotechnical drilling is necessary to sample these presently overlooked sectors directly.

The PFS geotechnical boreholes are also remote from the proposed underground mining area. In this case referral to existing resource borehole cores is available to assist in more detailed assessment of stoping methods and parameters; however, further drilling is likely required to obtain samples from stoping zones for additional physical rock properties measurement and will be required for assessment of portal areas and underground access development alignments.

Rock mass quality and physical properties have been empirically assessed (using the Geological Strength Index GSI [2] and rock mass quality rating Q [3] methods) using data obtained from geotechnical logging (conventional and televiewer scanning).

The rock mass strength properties derived using this process were used to define rock mass quality and in slope stability analyses and assessment of underground access support requirements and preliminary stoping parameters.

The empirical approaches applied are currently industry-accepted methods; however, there are limitations which need to be understood and variations that must be considered. Rock mass classification methods do not involve analysis of specific failure mechanisms or the requirements to counteract potential destabilising stresses/ forces. These methods are 'applicable' only when data required for rational analysis are unavailable or inadequate. It is pertinent to note that experience indicates that empirical methods can variously make overestimates or underestimates of rock mass capacity; hence measures indicated by the method can be inappropriate.

The restricted data coverage (discussed previously) further limits confidence in inferred rock mass capacity and anticipated ground response to mining activity.

PSM Recommendations

PSM recommendations for open pit wall design have been derived considering assessed rock mass quality, the results of kinematic stability analysis (undercutting potential), limit equilibrium analyses and reference to experience in similar geological and geotechnical settings. Design acceptance was based on achieving a <u>minimum</u> Factor of Safety (FOS) ≥ 1.20 against slope failure, based on limits presented in Read & Stacey, *Guidelines for Open Pit Slope Design*, 2009 (Table 9.9). Note also, Table 9.4 from Read & Stacey, which notes the <u>mean</u> FOS for slopes should be ≥ 1.30 (Figure 13).

The recommended wall design parameters, which assume passive dewatering only (that is, pumping from the open pit only) are listed in PSM Table 23, reproduced as Figure 14 below.

Potential slope disturbance from natural seismicity was <u>not</u> considered by PSM.

Table 9.9: Typical FoS and PoF acceptance criteria values

		Acceptance criteria[a]		
Slope scale	Consequences of failure[b]	FoS (min) (static)	FoS (min) (dynamic)	PoF (max) P[FoS ≤ 1]
Bench	Low–high	1.1	NA	25–50%
Inter-ramp	Low	1.15–1.2	1.0	25%
	Medium	1.2	1.0	20%
	High	1.2–1.3	1.1	10%
Overall	Low	1.2–1.3	1.0	15–20%
	Medium	1.3	1.05	5–10%
	High	1.3–1.5	1.1	≤5%

a: Needs to meet all acceptance criteria
b: Semi-quantitatively evaluated

Table 9.3: FoS and PoF guidelines

		Acceptable values		
Consequence of failure	Examples	Mean FoS	Minimum P[FoS < 1.0]	Maximum P[FoS < 1.5]
Not serious	Individual benches; small (< 50 m), temporary slopes, not adjacent to haulage roads	1.3	10%	20%
Moderately serious	Any slope of a permanent or semi-permanent nature	1.6	1%	10%
Very serious	Medium-sized (50–100 m) and high slopes (<150 m) carrying major haulage roads or underlying permanent mine installations	2.0	0.30%	5%

Source: Priest & Brown (1983)

Figure 13 Slope design Factor of Safety criteria
after Read & Stacey

Table 23 – Summary of the Recommended Pit Slope Design for RAS Open Pit

Wall	Aspect (°)	Unit	IRA (°)	BFA (°)	Berm Width (m)	Bench Height (m)	Controlling Mechanism
Southwest	350 – 065	All	34.8 #	50	9	15	Foliation/Foliation shears dipping towards the northeast.
West	065 – 160	TZ3	32.5	50	11	15	Planar sliding along the obliquely dipping TGF.
		TZ4	45	60	6.5	15	Planar sliding along faults and shears identified behind the pit wall.
Northeast	160 - 235	All	45	60	6.5	15	
East	235 – 350	TZ3	40	60	9	15	Planar sliding along the obliquely dipping TGF.
		TZ4	49.2	70	7.5	15	Planar sliding along faults and shears identified behind the pit wall.

\# IRA values in boxes replace incorrect values missed by PSM in editing & review

Figure 14 PFS recommended wall design parameters
after PSM

PSM conclusions relevant to mine design were:

- Assessed potential for basal sliding along the TGF controls the design of slopes in the TZ3 schist.
- Pit slope designs were provided for passively depressurised walls, noting the expectation that steeper walls could be developed using horizontal drainage from pit wall and pumping from ex-pit bores.
- Understanding structural geological conditions is critical to mined open pit and underground mine design and further work is required to refine structural models.
- Additional geotechnical drilling is required.

PSM listed risks and uncertainties related to open pit and underground mining as follows:

- Limited knowledge of locations and characteristics of major structures (faults and shears) and associated potential to adversely influence future pit wall stability.
- Possibility that structural conditions and/ or rock mass conditions vary across the deposit area.
- Potential that rock mass conditions are worse than anticipated.
- Possible inadequacy of or inability to reduce groundwater pressures.
- Possibility for elevated pore pressures in TZ4.
- Complex failure mechanisms:
  - Possible potential for large scale planar sliding on sheared foliation fabric and/ or unidentified faults and shears.
  - Adverse combinations of faults and shears disturbing slope stability at inter-ramp scale.
- Potential adverse influence of natural seismicity (regionally) or mining-induced seismicity (locally).
- Stope back instability resulting in increased overbreak and dilution.
- Inadequate ground support.
- In situ stresses are greater than assumed, requiring increased levels of ground support.
- Inadequate separation between open pit and underground openings leading to instability.
- Potential for portal instability.

Future work requirements listed by PSM:

- Detailed outcrop mapping.
- Preliminary structural model to confirm the presence and condition of the regional lineament model. Further investigation is required to identify defect characteristics and distributions and assess potential impacts on slope and underground opening stability.
- Verification of rock mass conditions and zoning; further sub-division within TZ3 and TZ4 units.
- Additional rock strength testing.
- Pit scale groundwater model to assess compartmentalisation of pore pressures and capacity for depressurisation.
- In situ stress testing.
- Three-dimensional stability analyses to assess complex fault interactions with pit geometries.
- Stability analyses subject to regional seismicity.
- Assessment of underground opening stability and access development support requirements.
- Box-cut and portal design.

3. Review of Geotechnical Conditions

3.1 Rock Strength

Assessment of intact rock strength for this review/ re-assessment has referred to (and concurs with) the PSM evaluation of rock physical properties. PSM considered field estimated intact strengths against results from point load testing on cores; results from laboratory-based measurement of rock strengths; and observations made in site mapping; and inspected and check-logged geotechnical cores.

Intact rock strengths vary widely. Fabric has a significant influence on rock competence (dependent on load magnitude and direction of loading). Faulting and shearing have caused significant disturbance to the RAS rock mass, to the extent that there are (inferred structurally-defined) zones of disintegrated rock, reduced to soil level competence.

The weaker rocks can be assumed to be the dominant control on behaviour of as-excavated mine openings. The strength of intervening rock intervals between weak zones can be almost irrelevant to the rock mass response to mining activity.

Tabulated results from rock physical properties testing are listed in Tables 9, 10, 11 & 12 of the PSM report [1] and are reproduced in Figure 15 below.

Rock strength testing remains relevant to comminution assessment and characterisation of waste rocks.

3.2 In Situ Rock Stress

No *in situ* stress measurements have been performed at RAS.

In the first instance this is not a critical issue (at PFS), given the moderate depths of proposed mining; however, given the assessed variable but typically *poor* rock mass quality and potential for structural influence it is important that the stress tensor is identified prior to commencement of underground mining. It is inferred that the near surface ambient rock stress will be low to moderate; high stresses would be variously dissipated by surface relaxation or shed from the weak, *poor* quality rocks.

The PSM inference that in situ stresses can be based on experience in similar conditions elsewhere in the Otago Goldfields is reasonable. The ambient in situ principal stresses are thus:

Major Principal Stress (σ_1) moderately steeply westerly plunging at ~ 40° / 278°.

Pre-mining σ_1 magnitude is assumed to be ~ 2.2 times overburden ($\sigma_V = 0.027 \times$ depth), which in the upper underground levels (at ~ 270m below surface) would be ~ 16 MPa.

Intermediate Principal Stress (σ_2) moderately steep easterly plunging at ~ 51° / 090° and ~ 1.75 times overburden (σ_V); ~ 13 MPa at 270mbs.

Minor Principal Stress (σ_3) sub-horizontal, north trending at ~ 01° / 005° at ~ 13 MPa magnitude.

Changes in stresses due to concentrations and/ or reductions resulting from mining-induced stress re-distribution are expected to be moderate; however, total/ cumulative spans need to be managed to control local stability and maintain the integrity of the mine structure. As noted, it is possible, however, that low stresses could be problematic locally.

Figure 15 reproduces the PSM report [1] Tables 9, 10, 11 & 12, summarising rock mass quality assessment and rock strength test results.

Table 9 – Summary of Rock Mass Units at RAS

Rock Mass	Description	Estimated Field Strength [(1)]	RQD
Weathered TZ3	Highly weathered to moderately weathered schist	Weak to Strong	Poor to fair 0% to 50%
TZ3	Unmineralised Chloritic Schist	Moderately Strong to Strong	Poor to good 0% to 90%
TGF	Highly Altered TZ3 unmineralised Chlorite Schist with clayey gouge zones - sheared	Weak to Moderately Strong	Poor to good 0% to 100%
TZ4	Unmineralised and mineralised (in parts) biotite schist	Strong to Very Strong	Fair to very good 50% to 95%

(1) Logged under Guideline for the field classification and Description of Soil and Rock for engineering purposes. NZ Geotechnical Society Inc, December 2005 (Reference 10).
(2) Geological Strength Index.

Table 10 – Summary of 2024 SI Valid Point Load Tests

Rock Mass Unit	Valid Axial Tests	Valid Diametral Tests	$I_{s(50)}$ Results	
			Range	Mean
Weathered	0	11	0.014 – 0.816	0.38
TZ3	0	93	0.003 – 1.61	0.28
TZ4	0	50	0.041 – 5.16	1.77
Total	**0**	**151**		

Table 11 – Intact Rock Strengths – Comparison of Data Sources

Rock Mass Unit	Logged Field Estimated Strength (MPa)	UCS Testing – (MPa)		Diametral Point Load Equivalent UCS (MPa)	
		Range	Mean	Range	Mean
Weathered	Extremely Weak 1 – 5	n/a	n/a	1 - 5	2
TZ3	Moderately Strong 20 – 50	7.7 - 35	20.4	1 - 50	3
TZ4	Very Strong 50 – 100	35 - 110	70	1 - 100	40

Table 12 – Summary of Triaxial Test Results and Adopted Shear Strength for the TGF

Structure	Mohr-Coulomb Shear Strength				Description
	Lab Testing		Adopted		
	c' (kPa)	ϕ' (°)	c' (kPa)	ϕ' (°)	
TGF	33	24	50	20	Taken from one sample (MDD286 131.18 to 131.35 m) logged FES of Strong (50 – 100 MPa)

Figure 15 RAS rock strength testing results
after PSM

3.3 Rock Structure

Stereographic assessment (accepting as valid the data collected by SMI and PSM and processed by PSM as presented in PSM reporting [1]) indicates the dominance of shallow, north north-easterly dipping shearing (RSSZ), faulting (TGF and associated sub-parallel faults and shears), and pervasive schistosity/ foliation fabric. Partings on the schist fabric are common and numerous fissile zones are evident in inspected geotechnical cores.

The range of defects assessed and listed in listed in PSM Table 7, reproduced in Figure 16.

Table 7 – Summary of Defect Sets Identified in the 2024 Televiewer Data

Lithology	Defect Type	Set	Dip (°)	Direction (°)
TZ3	Foliation	1	20 ± 25	025 ± 30
	Joint	1	50 ± 30	025 ± 35
		2	45 ± 25	200 ± 35
	Faults & Shears	1	40 ± 25	025 ± 40
		2	45 ± 25	205 ± 30
TZ4	Foliation	1	30 ± 15	025 ± 35
	Joints	1	45 ± 30	035 ± 35
		2	60 ± 30	110 ± 35
		3	30 ± 30	290 ± 35
	Faults & Shears	1	35 ± 30	035 ± 35
		2	50 ± 25	120 ± 30

Data derived from televiewer logging of geotechnical & resource boreholes

Figure 16 Defect set orientations
after PSM

Oriented defect data collected by PSM during the PFS geotechnical study were analysed using the Rocscience program DIPS [4].

Figure 17 is a lower hemisphere equal angle projection of fault and joint orientations in the RAS area. These orientations have been derived/ inferred from review of geotechnical logs and observations by PSM [1].

Stereographic projections of faults, foliation defects and joints derived by PSM from televiewer logging data of RAS resource boreholes are shown for TZ3 and TZ4 in Figures 18 and 19, respectively.

Mean orientation of the TGF and schist fabric is notionally ~ 20 to 25° to a dip direction of ~ 025°. There are wide local variations to both dip and dip direction of defects.

Structures at other orientations at RAS include structures antithetical to the TGF and schistose fabric with a mean orientation of ~ 45° / 205°; and cross-cutting faults/ joints at ~ 55° / 115° and ~ 55° / 305° (which are possibly conjugates to schistosity and antithetical structures).

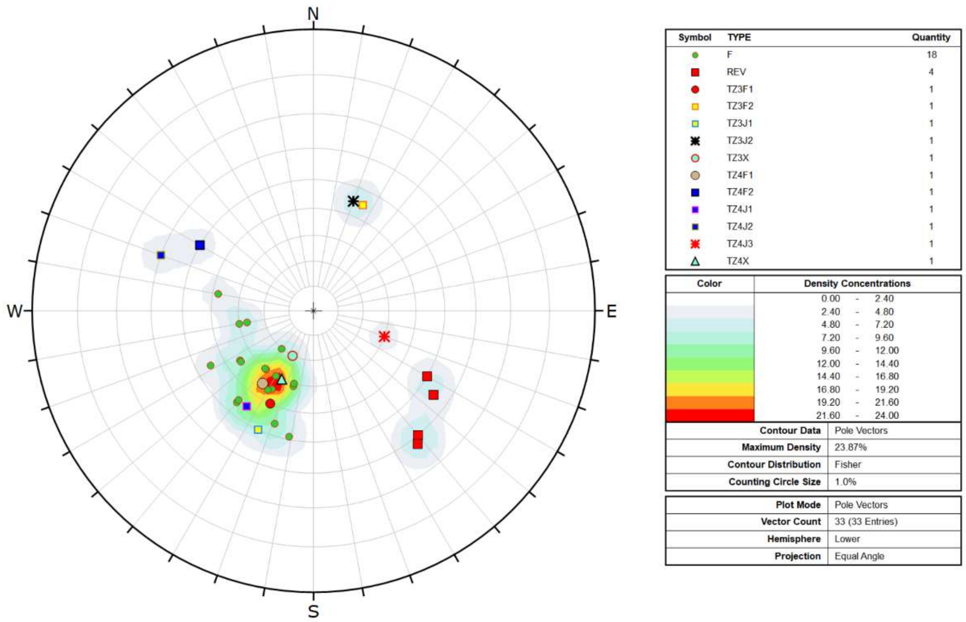


Figure 17 RAS faults/ fault zones from logged & mapped data &
TZ3 & TZ4 mean structural orientations data derived from televiewer logging
Source PSM [1] & SMI

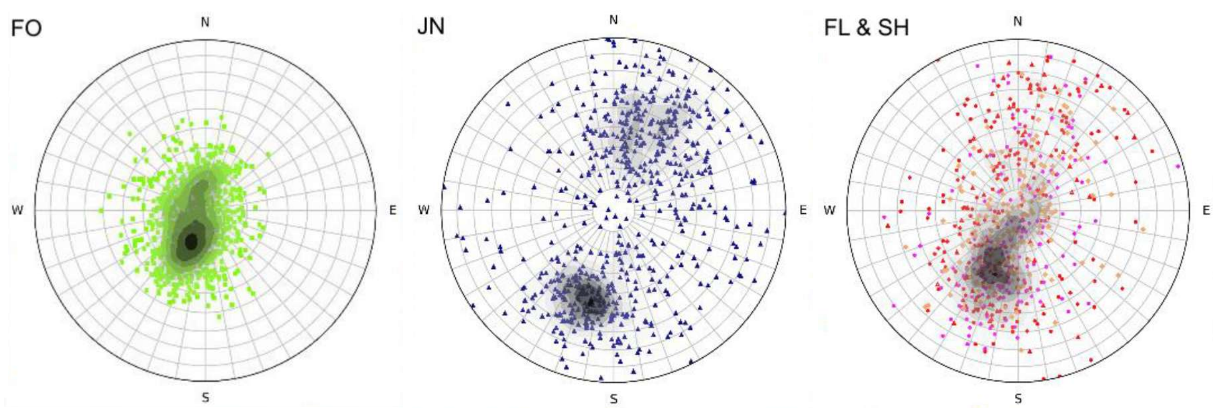


Figure 18 TZ3 structural orientation data derived from televiewer logging
Source PSM [1]

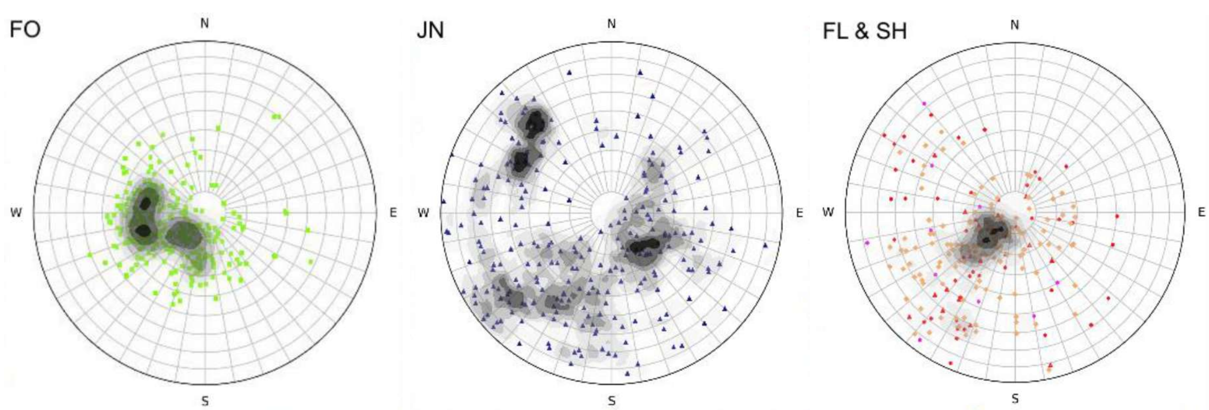


Figure 19 TZ4 structural orientation data derived from televiewer logging
Source PSM [1]

3.4 Hydrogeology

Information regarding groundwater, relevant to geotechnical assessment of proposed mining at RAS, has been drawn from a Komamawa Solutions Ltd (KSL) report to SMI [5].

In summary:

- ⇨ Groundwater is present although likely compartmentalised (major structures ± infill act as barriers/ aquitards/ aquicludes), water transmission rates in intact basement schists are inferred to be low
- ⇨ The water table mimics the land surface, and is located at ~ 40m depth near ridges and close to surface in valleys
- ⇨ Hydraulic conductivity of both TZ3 and TZ4 units is low (at 10^{-8} ms^{-1})
- ⇨ Permeability is secondary (flow is in fractures/ joints)
- ⇨ Inflow to mined voids will be low to moderate
- ⇨ Groundwater recharge is limited by low permeability.

Groundwater pressures are inevitably detrimental to pit slope stability. Rock mass relaxation resulting from excavation unloading will dissipate groundwater pressures (at least temporarily). Further active depressurisation measures will be required to maximise sustainable slope angles.

The following information has been drawn from the KSL report to SMI (locally précised):

There is appreciable passage of water through fractured basement rocks due to their wide and pervasive distribution across Central Otago; however, much of the potential groundwater recharge of excess precipitation is rejected at the soil / regolith interface due to the generally low permeability of the fractured rock. This water instead feeds surface stream flows. The steep slopes of the Dunstan Mountains are prone to shed excess precipitation into surface stream flows than rain falling on gentler slopes or flats. There are few signs of surface water over schist rock being lost to groundwater, neither are there observations of schist bed creeks making discrete gains from groundwater.

The basement of the Bendigo District and much of Central Otago is Haast (Otago) Schist of the Torlesses Supergroup. The Haast Schist was formed from marine sediments that underwent deep burial and regional metamorphism so that the sandstone, siltstone and mudstone have been somewhat homogenised in metamorphic consolidation. The original porosity of the sediments was lost due to compaction and metamorphism, although subsequently successive phases of uplift and crustal flexure led to the penetration of several generations of fractures and joints. These fractures and joints, plus fault brecciation and shear zones, provide secondary permeability and porosity to the schist rock in the Bendigo District. Some rock fractures have been infilled with quartzose or calcareous vein-work, and some shears contain significant clay infill.

Measurements of the hydraulic conductivity of TZ4 schist within the Cromwell Gorge in connection with lake shore stabilisation geotechnical investigations found in situ TZ4 rocks averaged 5.5×10^{-7} ms^{-1} with a minimum of 3.1×10^{-10} ms^{-1}.

There is no significant difference in permeability between TZ3 and TZ4 units. Measurement of RAS hydraulic conductivity in TZ4 (nine (9) packer lugeon tests at depth) indicated a mean of 4.6×10^{-8} ms^{-1}, while TZ3 schist over the RAS pit area averaged 2.7×10^{-8} ms^{-1} (six (6) packer tests).

Overall, the water table tends to mimic the land surface across areas of sharply undulating terrain. Depth to water was measured in eighty (80) separate locations RAS, Come in Time, and SREX deposit areas to extend to as much as 42m directly beneath steep ridges, while saturation approaches ground surface at slope bottoms. Groundwater transmission rates in the schist basement are inferred to be low, especially within the intact parts of the schist basement.

Measured deep groundwater pressures within the low permeability TZ4 (footwall) schist is elevated and may be higher than the overlying land surface. Flowing artesian pressures were encountered four (4) open resource holes, suggesting groundwater pressure compartmentalisation.

The proposed RAS mine would comprises a surface mining pit and underground workings, each following RSSZ mineralisation. Both open pit and underground operations workings would experience inflows from the surrounding fractured schists, ~ 14 to 28 Ls^{-1} for the pit and ~ 24 Ls^{-1} into underground workings.

Surface water management planning would conduct flow from remnants of the intersected RAS Creek to be diverted around the pit (via a channel). Peak flood flows could escape the channel and enter the RAS pit.

3.5 Rock Mass Quality

The physical properties and capacity of the RAS rock mass have been estimated using empirical methods. Data obtained from geotechnical logging have been reviewed and cores and exposures inspected to assist derivation of appropriate parameters to describe material properties in assessing open pit slope design, underground access support requirements and preliminary stoping parameters. As noted, however, there are limitations to the use/ applicability of these methods.

There is high variability of rock mass conditions related to structure, deformation and alteration within both the TZ3 and (upper) TZ4 which makes identification of appropriate rock mass strengths, to be used in analysis, somewhat difficult. 'Typical', mean, median, or various percentile conditions/ parameters can be identified from logging ± mapping data; however, rock mass performance is dictated by the capacity of the weakest element exposed in the mined structure (for example, the slope or stope), that is, by the behaviour of the element subjected to the greatest stress (relative to capacity).

General application of the strength of the weakest element is usually overly conservative, while use of any value greater than mean or median is unrealistic.

Application of the median value or an assessed 'typical' value in initial assessment is reasonable; however, provided the median/ typical value is accurate, in approximately 50% of cases analysed stability conditions will be under-estimated while the other half will be over-estimated.

In stoping assessments first quartile values/ rating are often used, with the expectation that 25% of cases may have poorer performance while 75% will be better.

At RAS excavations in TZ3 and (to an inferred lesser extent) in the upper portion of TZ4 (within the RSSZ) behaviour will be influenced significantly by the frequent presence of foliation-parallel faults and shears which are interlayered with rocks of moderate strength/ quality, albeit containing pervasive schistose fabric.

A median or mean shear strength value applied to an entire unit is misleading, in that while that strength may or may not be accurate for some parts of the rock mass; it will be highly inappropriate in others (variously too strong or too weak). It is possible that general assignment of a median rock mass strength could be inappropriate everywhere within the rock mass of concern.

Ideally the locations and extents of all rock mass material and strength variations would be known, represented numerically and modelled to assess and predict responses to excavation. This is not possible at present and may remain so indefinitely.

Use of the GSI System [2] to identify rock mass shear strength in the foliated and faulted TZ3 rock mass at RAS, and designation of the TZ3 rock mass as *blocky/ disturbed* to *very blocky* may over-estimate the capacity of the unit. There are zones within TZ3 which are fissile (closely laminated) and other zones which are effectively soils. Figure 20 shows the GSI graphical assessment.

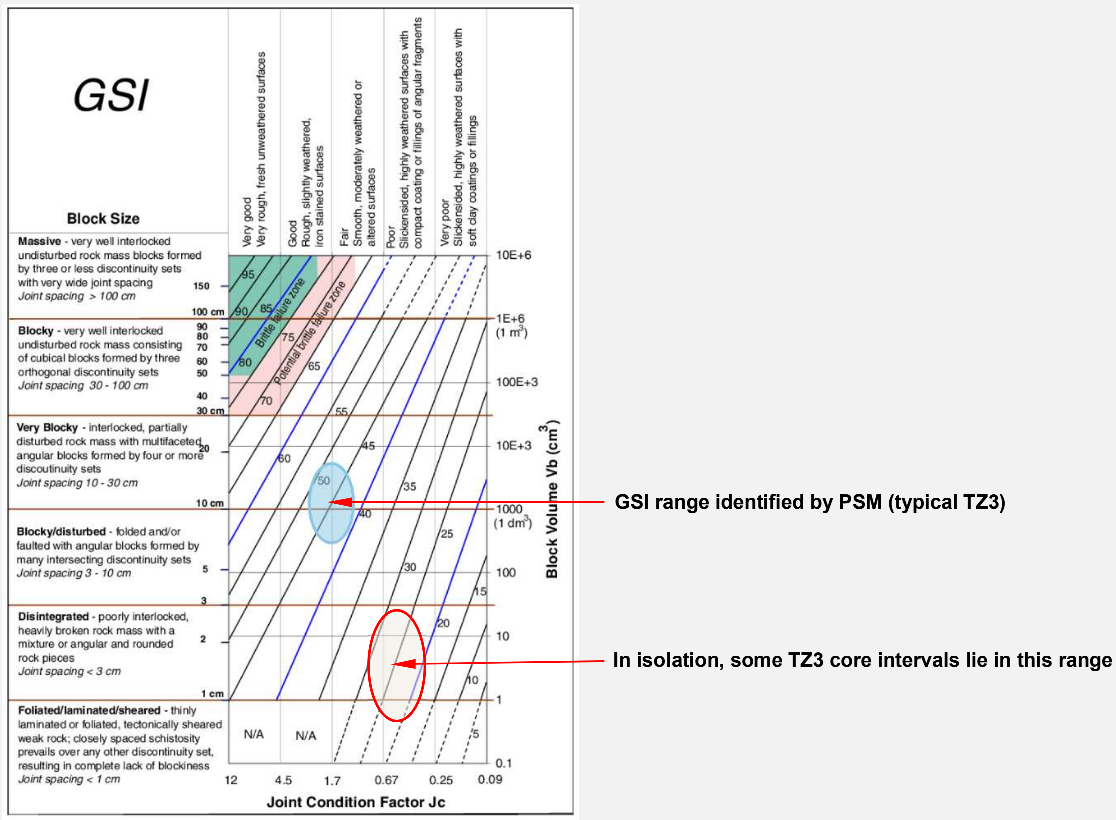
In laminated/ sheared ground, the rock mass feature controlling strength and deformability is not that derived from rock-to-rock contacts of blocks or of broken rock pieces (as in a breccia), but rather the shear strength of polished/ slickensided surfaces or the inter-laminae sheets of fissile zones or of the fines along the numerous foliation or shear surfaces/ zones.

While the RAS rock mass is not as pervasively disturbed or as fissile as that in the case referenced by Hoek *et al* [2] it is possible that *poorer* quality RAS zones could adversely affect slope or stope stability.

Poor quality rock mass conditions are frequently evident within the hangingwall and footwall to the core of the TGF. In observed geotechnical cores, these zones lie within and around targeted ore zones.

Ground support schemes recommended by PSM based on empirically derived rock mass quality assessed from the geotechnical logging data are reasonable for access development but are lacking with respect to stope stability assessment.

The proposed approach to exposing longhole open stope (LHOS) backs is inferred to be non-viable. Cable bolt reinforcement into the lower TZ3 zone (from a single top sill drive) has a low likelihood of success, and (the alternative of) allowing the backs to cave would introduce excessive dilution which could become uncontrollable.



The following comments regarding performance of disintegrated/ disturbed rock mass zones in schistose rocks are from Hoek *et al*, 1998[2]:

> The Geological Strength Index (GSI) classification scheme, through which the rock mass strength and deformability parameters are estimated based on the rock mass structure and discontinuity surface condition does not adequately describe some of the rock mass types commonly encountered in the RAS bedrock.
>
> The materials not included are the thinly foliated or laminated, folded and predominantly sheared weak rocks of non-blocky structure. In these rock masses the strength and deformability characteristics are not governed by rock-to-rock contacts of angular or rounded rock pieces but rather by the displacements along the numerous very thinly spaced presheared and slickensided foliation planes of the rock mass.
>
> A new foliated/laminated/ sheared rock mass structure category is proposed to accommodate these rock types in the lowest range of applicability of the GSI system. Given the presheared nature of the rock's discontinuities their surface condition could not be classified either as very good or as good and therefore the classification is non-applicable. For the remaining fair to very poor surface qualities the equivalent GSI contours now range from the new value of 5 up to 30.

Figure 20 TZ3 GSI: PSM[1] typical rating & inferred rating range for interval of poorest quality

3.6 Seismicity

Natural seismicity will be an intermittent, transient hazard to mine stability at RAS. Potential for earthquake disturbance must be allowed for in mine design.

Statutory regulations and guidelines provide the background against which mine design must be based.

From ***Ground or Strata Instability in Underground Mines and Tunnels*** Worksafe NZ, 2016

Section 2.1.3 Seismic Activity

> Natural seismicity is an earthquake that is caused through natural earth processes and needs to be considered in mine or tunnel design. Understanding the location and seismic hazard profile of major fault zones capable of producing strong ground motions at the mine or tunnel site is important. The competent person considers this when undertaking the geotechnical review. The potential for earthquakes may need to be factored into the design, both during excavation and construction, as well as the final tunnel or roadway formation.
>
> Mining-induced seismicity occurs as a result of stress redistribution around underground openings. In some cases, such stress changes may trigger a sudden slip on a fault. This is almost always accompanied by ground vibration which may cause considerable damage to underground openings. In other cases, abutments or pillars may become overloaded and yield suddenly.

Section 3.1.1 Components of a Geotechnical Assessment

> The geotechnical assessment should cover proposed activities over the whole life of the mine or tunnel, from the feasibility study stage, operation of the mine or tunnel, to the final closure and abandonment of the mine or the full life of the tunnel including, *inter alia*:
>
> - Earthquake potential, depending on the location of the operation in relationship to fault lines.

The lateral acceleration associated with the seismic disturbance at RAS with a 10% probability of exceedance in 50 years (a 475-year return period) is 0.36g (Figure 21) [6]. To withstand acceleration of such a magnitude overall slope angles would need to be reduced, likely by ≥ 5° to provide an adequate FOS against rotational shear sliding.

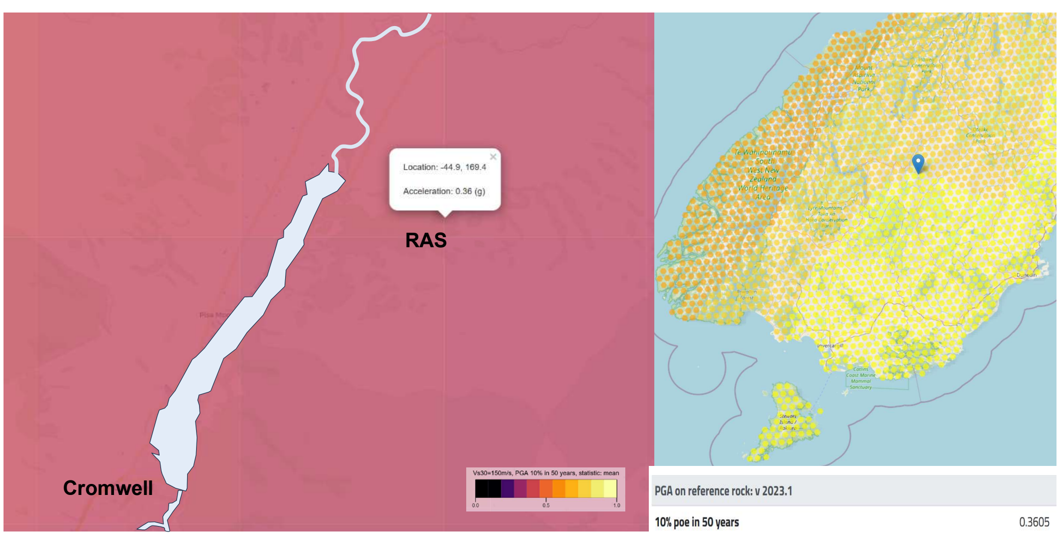


Figure 21 **RAS regional PGA with 10% probability of exceedance in 50 years**
0.36g (475 year return) (subtle, progressive reduction in potential disturbance to east)
Sources https://nshm.gns.cri.nz/HazardMaps & https://af8.org.nz/what-is-af8

For assessment of RAS mine stability, records were examined for earthquakes of magnitude ≥ M3 occurring within ~ 25 km of RAS over the past 51 years. Twenty-two (22) earthquakes occurred proximal to RAS, between 01 January 1974 and 31 December 2024 (Figure 22).

Events in this period included three (3) events in the immediate vicinity of the RAS site with magnitudes:

- M3.1 (16 March 2011)
- M3.3 (04 December 2011)
- M3.8 (30 November 2011).

Two (2) events in the area and timeframe were ≥ M4; M4.0 (11 June 1990) and M4.4 (02 July 2017), both of which were ~ 20 km west of the site, in the Mt Pisa vicinity.

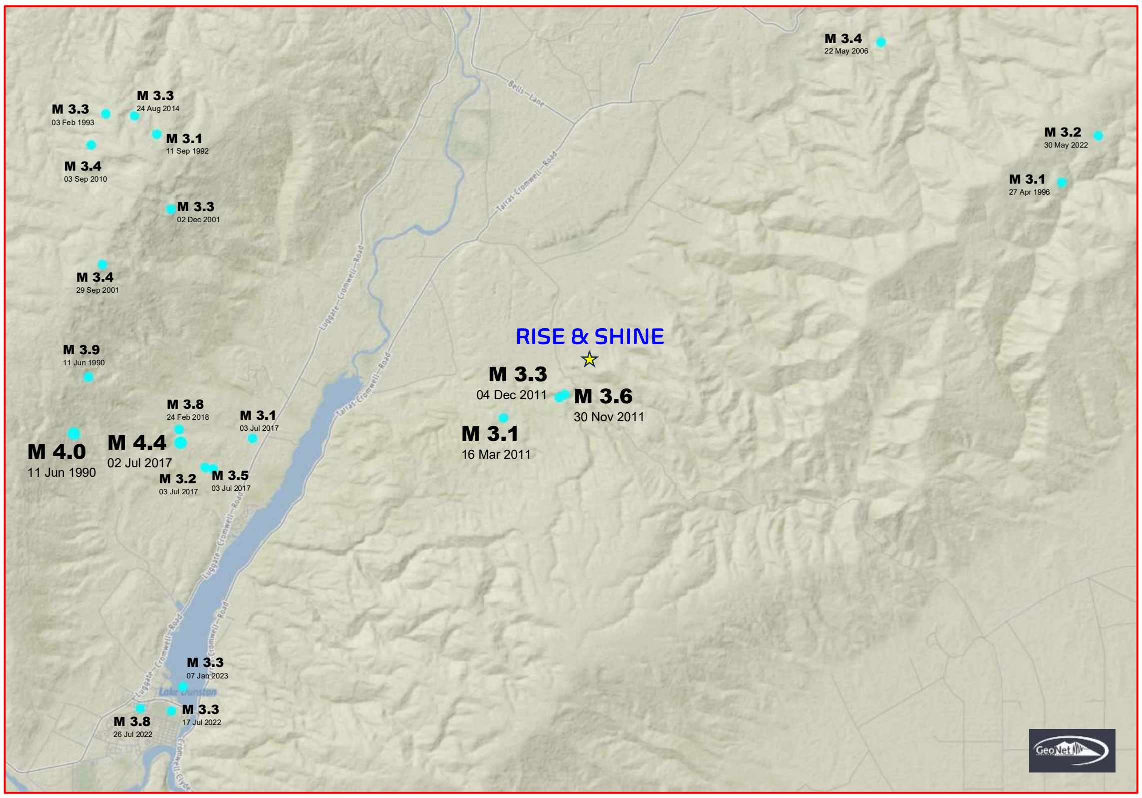


Figure 22 Earthquakes > M3 proximal to RAS, January 1974 - December 2024

The earthquake disturbance adopted for mining assessment at RAS is a lateral acceleration of 0.13g.

This is inferred to be the disturbance generated by a 4.5M event occurring at the RAS site.

Wider ranging searches were also performed; findings are illustrated in Figure 23. Records indicated 259 earthquakes (M3.3 to 4.4) for Otago & Southland, and 2,806 and 3,078 such events for Fiordland and Nelson & West Coast, respectively.

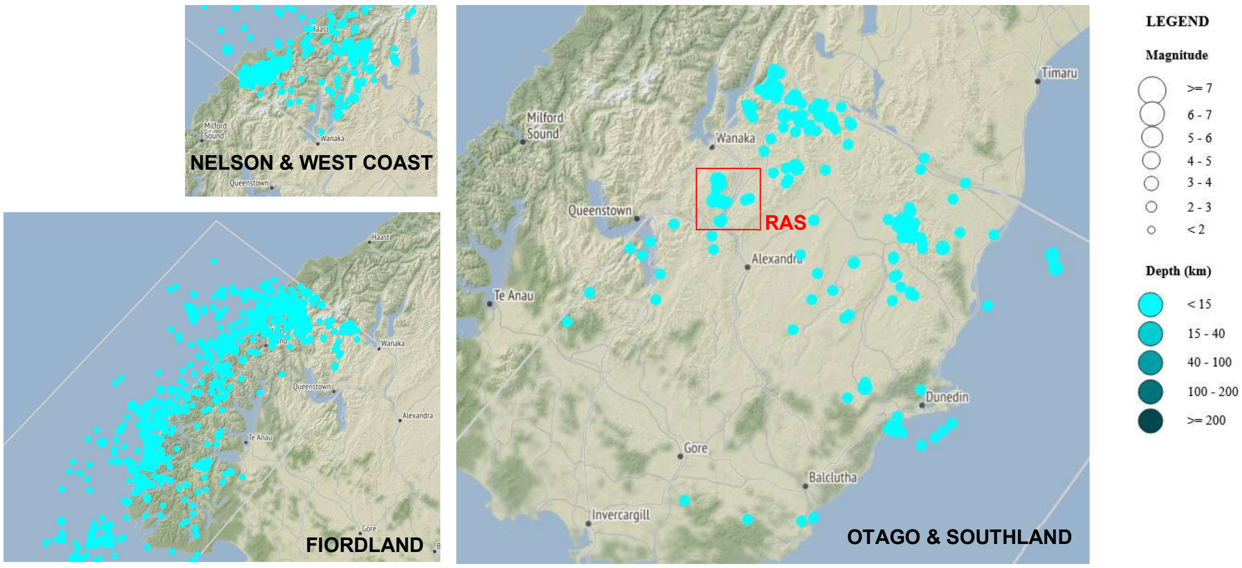


Figure 23 Earthquakes > M3 southern & western South Island January 1974 - December 2024

It is noted that the next large Alpine Fault earthquake has the potential to cause severe damage and disruption across the entire South Island, with major consequences for the rest of the country [7]. There is a 75% probability of an Alpine Fault earthquake within the next 50 years, and an 82% probability that earthquake will be an M8+ event.

The shaking generated by a M8+ Alpine Fault earthquake will be felt across the South Island and lower North Island. At its nearest point the Alpine Fault is ~ 110 km north-west of RAS.

At the RAS the expected disturbance from an M8+ event on the Fault would be *strong* (MMI 05) [#], possibly up to *slightly damaging* (MMI 06) (Figures 24 & 25).

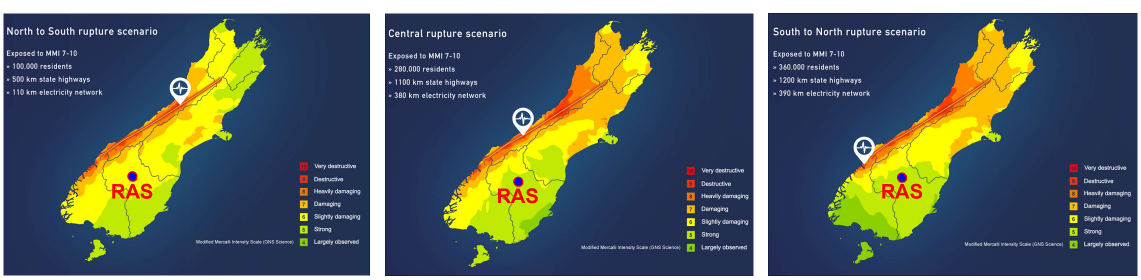


Figure 24 Inferred earthquake intensity distribution for northern, central & southern slip initiation points on the Alpine Fault (Source GNS NZ)

[#] Modified Mercalli Intensity Scale (MMI) indicates the intensity of an earthquake at a location as noted by the severity of ground shaking at that location and the effects of that shaking on people, manmade structures, and the landscape. It is an arbitrary ranking based on observed effects; there is no mathematical basis to the scale.

MMI 05
Generally felt outside & by almost everyone indoors. Most sleepers are awakened & a few people alarmed. Small objected shifted or overturned & pictures knock against walls. Some glassware & crockery break & loosely secured doors may swing open & shut.

MMI 06
Felt by all. People & animals are alarmed & many run outside. Walking steadily is difficult. Furniture & appliances may move on smooth surfaces & objects fall from walls & shelves. Glassware & crockery break. Slight non-structural damage to buildings may occur.

Figure 25 Modified Mercalli Scale for earthquake intensity

Comment: Seismicity & Pit Slope Stability

While earthquakes are noted to have frequently triggered landslides in natural slopes, there are no known, positively identified, occurrences of earthquakes initiating large open pit slope failures. Read & Stacey *Guidelines for Open Pit Design* (2009) [8] note the "*considerable debate about the need to perform seismic analyses for open pit slopes*" and that "*is the main reason why seismic loading is often ignored in open pit slope design*".

They concede, however, that a property located in a seismically active region and a large earthquake were to occur near a mined slope the effects could experience significant disturbance if weak soil-like materials are involved and "*Additionally, other mine infrastructure, especially tailings dams, may be and have been affected by earthquakes*". Accordingly, documenting the regional seismicity and integrating it with the geological mode is important.

Read & Stacey raise the question of the need for seismic analyses for open pit slopes and cite several instances where high magnitude events in highly active seismic zones close to operating large open pit mines in hard rock conditions have not produced significant slope instability. The mines cited include the Bougainville and Ok Tedi mines in Papua New Guinea, and a number of mines in Chile and Peru. It is conceded, however, that "earthquakes have produced small shallow slides and rockfalls in open pits but none on a scale sufficient to disrupt mining operations".

Excerpts from Read & Stacey [8] Section 3.5 ***Regional seismicity*** and Section 10.3.3.5 ***Seismic analysis*** are attached.

In the RAS case, concern regarding seismic disturbance relates to the assessed *poor* quality of the TZ3 rock mass. This assessment is based on conditions observed in cores and in available (albeit limited) exposures. Despite the *poor* quality, which results from a mix of highly fractured and locally disintegrated rock intercalated with layers of competent schist, it is inferred that *in situ* TZ3 rocks will not respond as a soil-like mass under transient seismic loading. Rather, it is inferred that the response of the unit to strong seismic disturbance would involve local fretting/ sloughing where poorer quality zones are exposed.

4. 2025 Geotechnical Assessment

The current reassessment of geotechnical conditions and implications for mining is based on existing data. Collection of further geotechnical data is required, and SMI has commenced a program of investigative drilling targeting likely ultimate open pit wall positions. The gathered data, once added to the existing database will be used in feasibility level assessment of proposed mining.

The existing geological and geotechnical data, gathered variously by SMI and PSM for previous geotechnical assessment by PSM have been reviewed and adopted for reassessment of ground conditions influencing behaviour and stability of open pit slopes, underground access development and stope openings formed during proposed mining on the RAS deposit.

The data have been collected using industry standard best practice and are found to be reasonable, albeit with acceptance of the inherent variabilities of the country rocks at RAS. Such variations/ scatter in rock and rock structure characteristics are typical of those associated with economically exploitable mineral deposits. While findings have been accepted (largely) at face values, interpretations of likely rock mass behaviour may differ.

Findings of high level hydrogeological assessment [5] of the deposit by Komamawa Solutions Ltd (KSL) available for this reassessment had not been made by the time PSM reported to SMI. The PSM assumptions generally align with the KSL findings.

While further measurements are required of RAS country rock and orebody physical properties, it is unlikely that the spread of results, for example, of rock strengths, will lessen. And it is pertinent to note, the weakest materials, which will likely dictate behaviour in response to mining, often cannot be tested (meaningfully).

Geotechnical reassessment of open pit and underground mining at RAS has therefore been based on:

- Discussions held severally and variously with Damien Spring, Executive Director and CEO; Sam Smith, Executive Director; Rod Redden, (Redden Mining Consulting Mining Engineers) Project Study Manager; Alex Nichol, Geology Manager; and Exploration/ Mine Geologists of SMI, prior to, during and following a site visit held from 09 to 13 January 2025.

  Discussions concerned requirements of the reassessment, mining history of the district, conceptual/ proposed mining configurations, general and site geology and structural geology, groundwater, prior geotechnical assessment of proposed mining, ground reinforcement and support schemes and stability monitoring of as-mined openings/ structures.
- Data provided by SMI, existing topography, drillhole plans/ traces, core logs and photographs, open pit designs/ conceptual shells, underground access and mineable stope optimiser (MSO) shapes, supplied electronically, variously in pdf, dxf, Surpac str/ dtm, jpg & csv, formats.
- Inspections of the RAS and surrounding area, including satellite deposit areas, made in the company of SMI Geological personnel during the January 2025 site visit.
- Inspection of cores and review of logs from geotechnical boreholes MDD286, 287, 289 & 290; core photographs from these geotechnical boreholes; inspection of core photographs and geology logs from twenty-eight (28) core resource definition boreholes (relevant to assessing ground conditions above and within proposed stoping blocks).
- Information and data referenced in the PSM geotechnical assessment [1] and appendices and selected figures (supplied by SMI) – no opportunity available for collection of new data, rather based on critical review of data and general information provided by SMI and provided in the PSM report data and discussion.
- New Zealand Geological and Nuclear Sciences (https://www.gns.cri.nz/) for general background geological information and seismicity data.
- A high level assessment of hydrogeological conditions at RAS compiled by Komamawa Solutions Ltd (KSL) (supplied by SMI).

5. Open Pit Assessment

It is inferred that there will be opportunity for mechanical excavation in the upper benches of the planned RAS pit. The effectiveness of mechanical excavation is expected to reduce with depth; however, conditions in core suggest that resort to localised or general use of drill and blast methods could revert to mechanical excavation at greater depth, potentially in a repetitive sequence. In some locations, and possibly generally, despite the excavatability of materials, resort to light blasting may be necessary to ensure consistent fragmentation and maintain required productivity levels.

It is essential that appropriate perimeter methods are used in all locations wherever blasting is required to form final batters. The proposed wall designs assume that excavation and perimeter blasting (where required) will be performed using industry best practices, and that results will be consistently effective.

Care must be taken to ensure that production blasts do not pre-condition/ disturb/ damage wall rocks. Wherever possible, it is recommended that final batter profiles in TZ3 (and potentially within the upper levels of TZ4) be formed by mechanical excavation. The back row of the limit blast (adjacent to the wall) should stand off from the wall to a point where the (anticipated) edge of the blast damage/ disturbance zone is at least ~ 1 to 2m off the designed batter position.

It is expected that excavation of the few metres to the final batter position will vary from very easy to being somewhat difficult. The approach aims to minimise blast disturbance to the batter and ensure that the berms retain adequate rockfall arresting capacity.

Where mechanical excavation to final wall positions is impractical, it is expected that best results would be obtained using trim blasts. Trim blasts must be fired to a free face, and preferably two free faces. A free face is one where all broken stocks and rill material are removed from the face and toe of the shot. This is critical in allowing good burden relief of the face, thus providing opportunity for burden relief throughout the pattern. Without face relief movement of the body of the pattern is blocked, energy dissipates in all directions, including into the wall. At RAS such over-confinement of energy would be highly conducive to wall damage, for example via intensive fragmentation (without displacement) or gas penetration causing block heave and/ or release of load fracturing.

Implementation of all perimeter blasting methods requires an elevated level of supervision in the field and stringent application of simple field controls. The return to the operation can be expected via reduced time in wall scaling, retention of berm crests and cleaner walls (less loose material), and thus safer pit operating conditions.

5.1 Potential Slope Failure Mechanisms

The pervasive, shallowly north north-easterly dipping foliation and sub-parallel faulting and shearing within TZ3 upper pit wall rocks and the RSSZ provide the dominant influence on RAS ground behaviour and thus on pit wall stability.

Other structures are evident/ anticipated and comprise features sub-perpendicular to the dominant fabric; antithetical structures dipping steeply to the south south-west; and sub-vertical faults and joints striking sub-parallel to the dip direction of the major structural features. Some 'randomly' oriented defects are also noted in core (inferred to be dominantly minor faults and joints).

The conditions and variations observed in TZ3 and (to an inferred lesser extent) in TZ4 schists could enable several failure mechanisms:

- Block/ slab sliding on undercut foliation fabric where the local fabric dip exceeds the effective limiting shear resistance acting on the basal/ lateral surfaces of the block (base friction + surface irregularity + cohesive effects – groundwater pressure).
- Rock mass failure via rotational shear through weak rock ± stepwise failure surface development on defects (foliation, faults/ shears, joints) and rupture through intact rock bridges.
- Block sliding displacement along the TGF (a feature frequently evident in limit equilibrium slope stability analyses).
- Fretting of FZ/ SZ materials, undercutting more competent blockier fresh rocks, potentially triggering further upslope instability.
- Local structural/ block failure promoting fretting/ more fretting of weak materials and/ or possibly acting as the initiator of rotational shear instability. Rotational shear could occur through the weaker zones of the RAS rock mass and/ or via development of stepwise failure paths variously sliding along existing defects (foliation in particular) and rupture through weak intact rock bridges.
- Active-passive mechanisms where the mass of the upper slope is sufficient to cause outward and possibly upward displacement of the lower wall of the pit.

5.2 Slope Stability Assessment

6.2.1 Kinematic Slope Stability Analyses

While block/ wedge instability in slopes in the RAS area is theoretically possible, analyses show that most wedges (defined by defects at the mean orientations of identified structural sets) which could be exposed in major walls of a future RAS pit would be stable.

Kinematic stability analysis using stereographic methods considering identified mean defect orientations (from 2024 geotechnical logging) and inferred fault orientations (derived from PSM interpretations and logs) show potential for block/ wedge sliding from southern, western and eastern sectors. Figure 26 shows potential for sliding along defects with shear strengths defined by a 30° friction angle from a 60° batter (facing 020°) in the south south-western sector of a RAS open pit, generated using the Rocscience program DIPS [4].

Wedges daylighted in the sector are denoted by the square symbols in the red and yellow crescents. Wedges with intersections in the red crescent area are more likely to slide as they exit the slope at moderate to high angles to the strike of the batter.

The actual potential for wedge sliding is likely significantly lower than that suggested by the stereoplot which assumes ubiquitous presence and throughgoing persistence for all defects. Coincidence of some pairs of defects (particularly faults) may occur only at a singular location or may be impossible. However, while each fault is present as a singular surface/ zone, presence of sub-parallel sub-ordinate structures (joints), which could contribute to local instability, is assumed.

Analyses using the Rocscience program SWEDGE [9] to analyse the stability of tetrahedral wedges defined by all possible paired combinations of defects with assessed mean orientations, show that unstable wedges could be exposed on southern, south-western and western walls. These theoretically unstable wedges are defined by steep easterly dipping and moderately steep north-easterly dipping joints and faults. These two defect sets provide the greatest potential for walls in these south-westerly sectors (facing the north-eastern sector of the pit). Figure 27 shows the wedge with minimum FOS against sliding in the south-western sector. While these defects are neither (obviously) prolific nor pervasive, some localised batter scale wedge instability is possible.

Structurally controlled instability via block sliding on undercut foliation fabric or associated sub-parallel faults/ shears is inferred to provide greater potential for significant wall instability than does exposure of wedges in future pit walls. A near-pervasive shear-parallel foliation fabric exists in the TZ3 schist. Faults/ shears sub-parallel to the TGF are abundant in cores from the TZ3 and contribute to the block sliding potential.

Lateral and rear release structures coincident with an undercut are needed to enable detachment and subsequent displacement of undercut blocks. For example, a detachable block could be defined by the TGF, a defect sub-perpendicular to the fault zone and sub-vertical northerly-trending joints.

As well as via appropriately oriented singular shear-parallel structures, sliding may be provided by an en echelon grouping of minor structures connected via rupture through weak intact rock or through inherently weak zones.

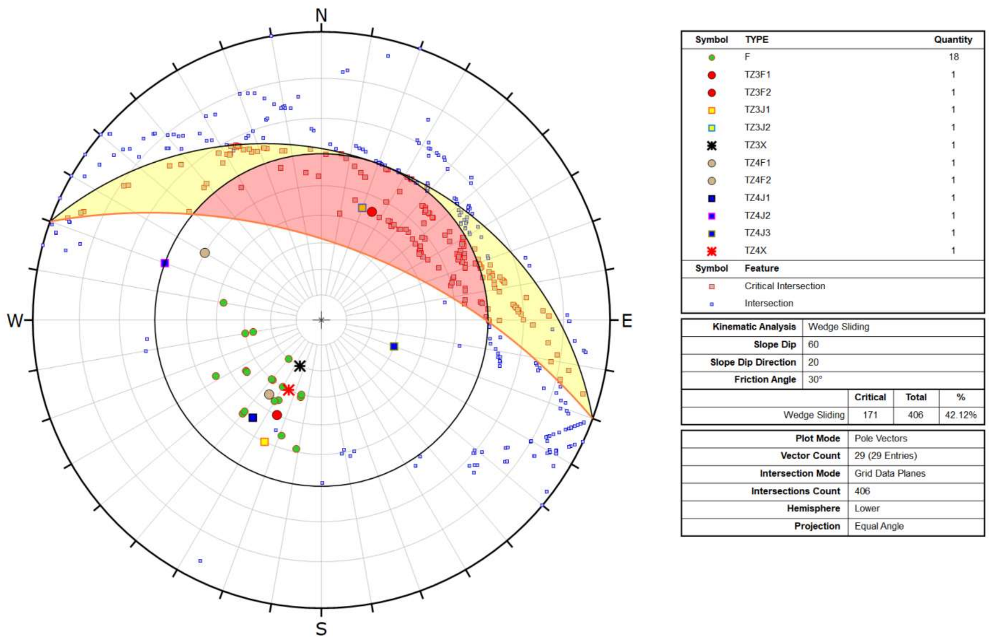


Figure 26 Stereographic tetrahedral wedge stability analysis: 60° batter facing 020°
Using assessed mean defect orientations & inferred fault orientations, assuming ubiquitous presence of continuous defects

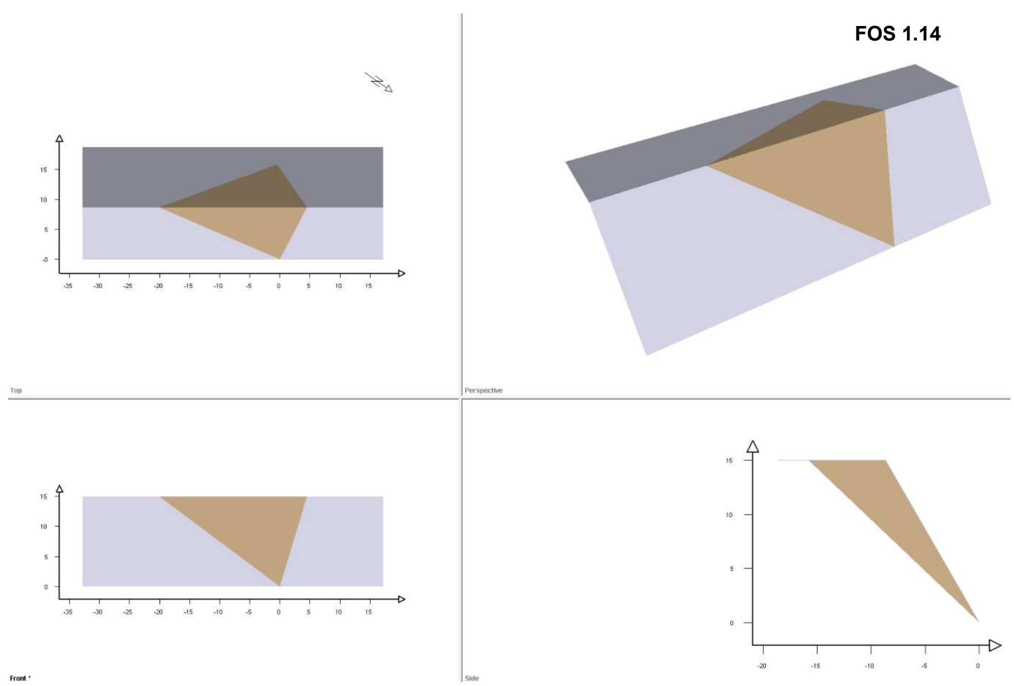


Figure 27 SWEDGE assessment of sliding potential: 60°, 15m high batter, facing 060°
Wedge with lowest FOS from assessment of all combinations of all defects (mean defect orientations)

5.2.2 Slope Limit Equilibrium Analysis

Assessment of rotational shear in fresh rocks is often simply a demonstration of very high FOS and low probability of failure (POF) or indeed improbability of failure. However, at RAS the typical poor quality and pervasive schistosity and high frequency of parting on fabric (in core) and the faulted/ sheared nature of TZ3 rocks are such that rotational shear could develop, perhaps most readily triggered by localised instability (for example, minor structurally controlled block sliding).

Once initiated, quasi-circular/ parabolic failure surfaces could develop variously via shear displacement along foliation and across-fabric rupture through 'intact' TZ3 rock and along the significantly weaker core of the TGF.

Completely dry slopes are not anticipated.

Undissipated groundwater pressure will exacerbate potential for failure. Full slope saturation is, however, unlikely in that unloading (by excavation) will relieve pressure and some drainage from the disturbed zone immediately behind pit walls is expected.

Semi-saturated/ partially depressurised slope conditions are assumed to be achievable at RAS via active dewatering using in-pit and ex-pit bores and wall depressurisation (installation of sub-horizontal depressurisation boreholes).

It is pertinent to note that LEA does not include the influence of structure. Selective/ localised reduction of material shear strengths or inclusion of weak layers can be included to represent known structures; however, the approach does not always adequately reflect structural influence.

Some of the slope conditions modelled are hypothetical and unlikely to exist.

The earthquake disturbance applied for this assessment was that inferred from a 4.5M event occurring at the RAS site.

Records from the past 50 years include twenty-two (22) events ≥ M3 and ≤ M4.5 within an ~ 25 km radius of the site, including three (3) events proximal to the RAS site with magnitudes of M3.1 (16 March 2011), M3.3 (04 December 2011) and M3.8 (30 November 2011). Two (2) events in the area and timeframe were ≥ M4, specifically M4.0 (11 June 1990) and M4.4 (02 July 2017), both of which were ~ 20 km west of the site, in the Mt Pisa vicinity (Section 3.6).

For the RAS site (fundamentally a rock site) a seismically-induced lateral acceleration of 0.13g was applied to a partially saturated/ partially depressurised 260m highwall slope (east wall) to represent likely worst case conditions during the life of the project.

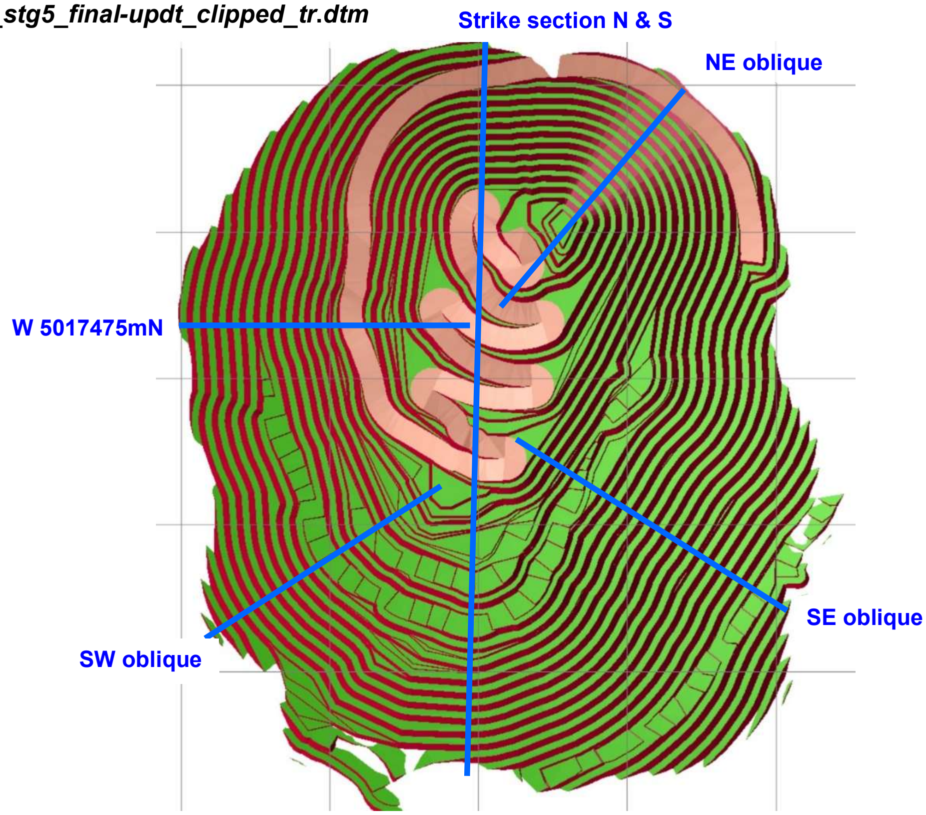
The disturbance anticipated from an event at RAS with a 10% probability of exceedance in 50 years is 0.36g. Two-dimensional LE analysis indicates that a disturbance of that magnitude would cause instability in a 200m high slope mined at any overall angle > 30° in TZ3 schists.

Two-dimensional limit equilibrium analyses (LEA, LE analyses) have been performed using the Rocscience program SLIDE2 [10] on several sections cut through the several designs provided by SMI (Files: *240928_stg5_final-updt_clipped_tr.dtm* (December 2024)*; ras_hg_cone1.dtm* (January 2025) and *ras_phase7.dtm* (May 2025)).

The pit configurations developed in various design phases differ only slightly; the critical sectors remain common and obtained FOS are similar. The critical wall sections are those in north-eastern, eastern, western/ south-western and southern sectors which variously include the highest, steepest and structurally most complex settings. Figures 28 and 29, respectively show pit designs and analysis section locations for December 2024 & January 2026 designs and the May 2025 ultimate pit design.

Needs remain for detailed design review/ revision to avoid a number of issues which, as designed, variously challenge practicability and establishment/ maintenance of wall stability. For example, review and adjustment of convex wall sectors (bullnoses, however subtle) and mergers between design sectors.

240928_stg5_final-updt_clipped_tr.dtm



ras_hg_cone1.dtm

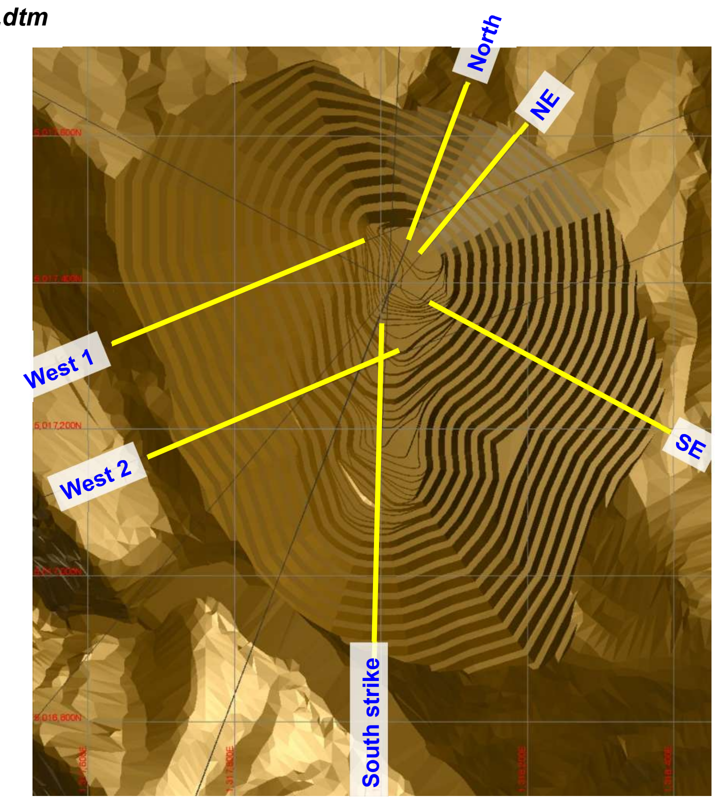


Figure 28 Limit equilibrium analysis sections for alternative pit designs
Design files: *240928_stg5_fiinal-updt_clipped_tr.dtm* (upper) & *ras_hg_cone1.dtm*

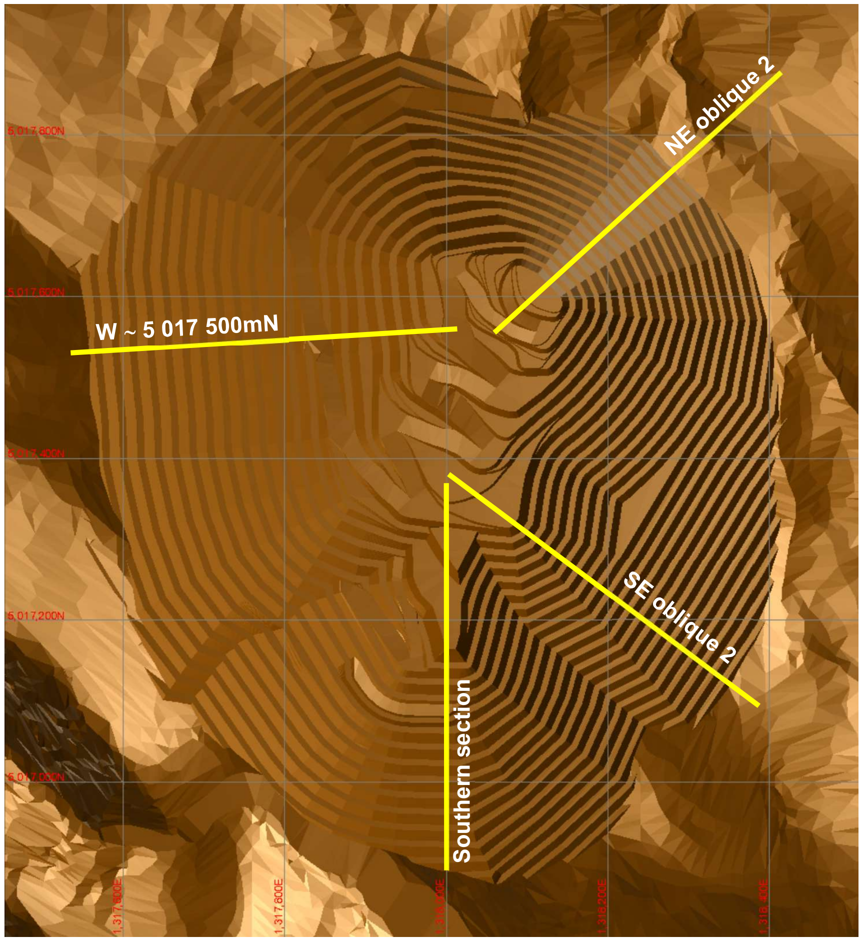


Figure 29 Limit equilibrium analysis sections for updated pit designs, May 2025
Design files: *ras_phase1* through *ras_phase7.dtm*

Material shear strength parameters adopted for the analyses were those derived by PSM. These rock mass shear strength properties are listed in Table 16 of the PSM report [1], reproduced in Figure 30.

Figure 31 illustrates the LE analysis sections cut through the May 2025 Phase 7 (ultimate) pit.

Table 16 – Adopted Rock Mass Shear Strength Parameters

Rock Mass Unit	Unit Weight (kN/m^2)	Mohr-Coulomb		Generalised Hoek Brown			Q	
		c' (kPa)	ϕ' (°)	m_i	GSI	UCS (MPa)	Range	Typical
Weathered Schist	20	100	38	N/A				
TZ3 (50 mbgl)	27	N/A		12	45	25	0.06 – 3.52	0.27
TZ3 (250 mbgl)							0.03 – 3.52	0.13
TZ3 (400 mbgl)							0.03 – 3.52	0.13
TGF [(1)]	20	50	20	N/A			0.003 – 0.08	0.02
TZ4 (250 mbgl)	27	N/A		12	60	75	0.22 – 44.55	3.09
TZ4 (400 mbgl)							0.11 – 22.28	1.24

(1) All depth ranges.

Figure 30 LEA adopted material shear strength parameters
after PSM

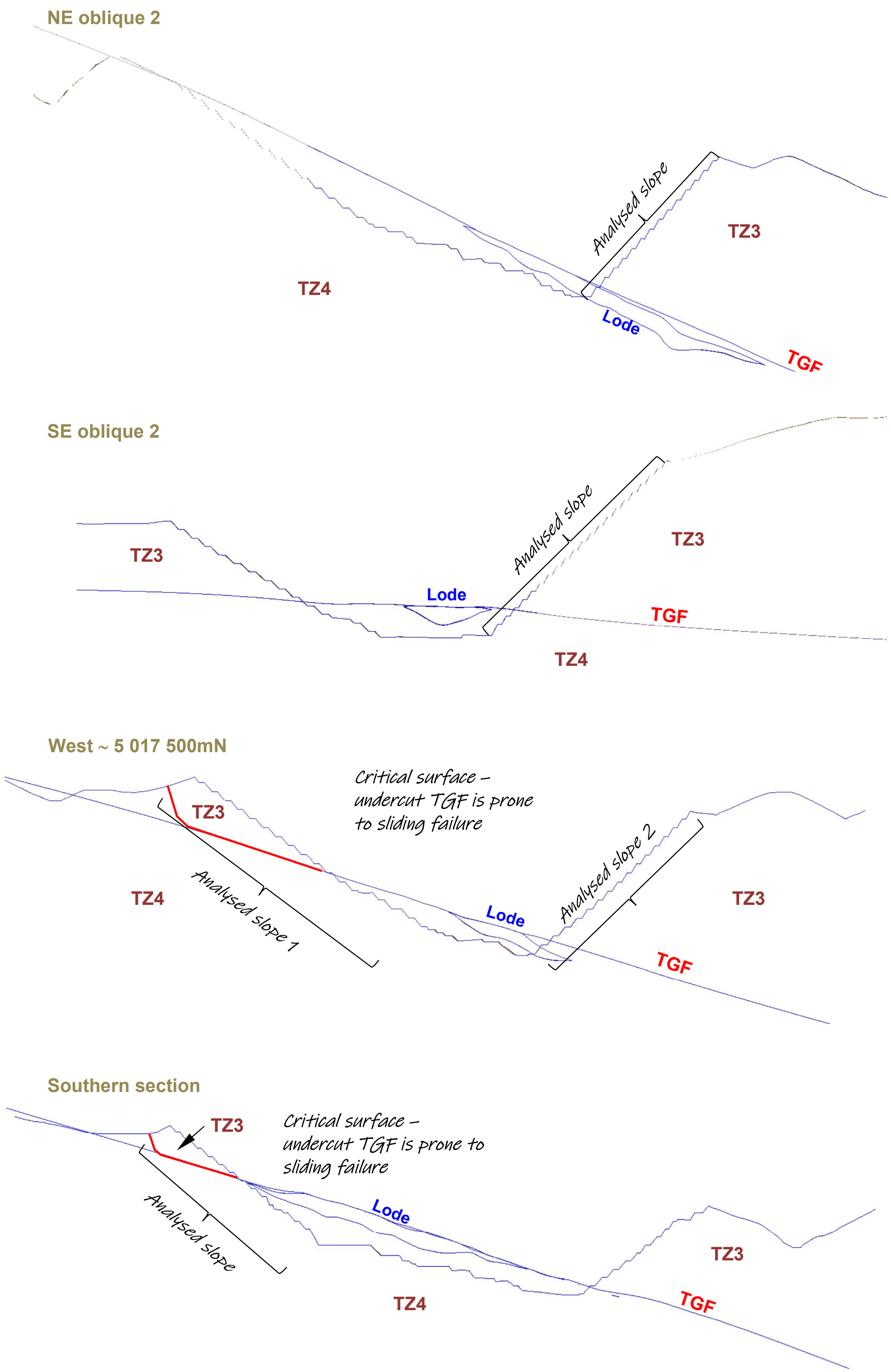


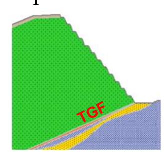
Figure 31 LE Analysis sections (May 2025 pit design, Phase 7 (ultimate pit)

LE analysis results are summarised in Table 1 (January 2025 design) and Table 2 (May 2025 design). Analyses were performed for various slope groundwater conditions under static and pseudo-static (simulated seismic) loading conditions:

➲ *Dry slope, static loading*	No hydrostatic pressure = fully drained/ depressurised
➲ *Saturated slope, static loading*	Below pre-mining SWL phreatic surface at slope surface
➲ *Semi-saturated slope, static loading*	Phreatic surface geometry to reflect inferred effect of active slope depressurisation
➲ *Dry slope, seismic loading*	No hydrostatic pressure = fully drained/ depressurised Lateral acceleration 0.13g
➲ *Saturated slope, seismic loading*	Below pre-mining SWL phreatic surface at slope surface Lateral acceleration 0.13g
➲ *Semi-saturated slope, seismic loading*	Phreatic surface geometry to reflect inferred effect of active slope depressurisation Lateral acceleration 0.13g

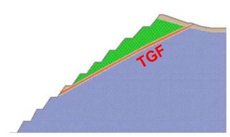
The surfaces with minimum FOS indicated by LE analysis of RAS pit walls are typically deep into/ behind the slope whereas it is inferred that the depth of initial instability, likely structurally controlled, would be significantly shallower and occur at batter to multi-batter scale. Nevertheless, the loss of confinement caused by such an event or events can lead progressively to widespread instability. Extension of collapse can vary from progressive creep to apparently 'instantaneous' collapse.

On all sections where the TGF dips into the wall, the surface with minimum FOS breaks out along the core of the TGF (that is, daylights at the point the slope intersects the TGF). The length of the minimum FOS surface on the TGF is significant and can be ≥100m.



The 2D situation modelled does not match reality. The 2D analysis assumes that the distribution of materials persists infinitely whereas the geometry of the pit wall-TGF intersection changes gradually but significantly around the pit perimeter. Influences of such changes in structural geometry, whether beneficial or deleterious, are not and cannot be considered in singular analyses.

Minimum FOS returned for all loading and slope saturation conditions on sections with similar dip direction to the TGF and which undercut the TGF were ≤ 1.0, with theoretical failure occurring via sliding along the core of the TGF. This influence is locally evident for all considered pit designs.



Dependent on the findings of recommended 3D analysis, pit configuration adjustment may be needed to remove such undercuts, either by avoidance in design or excavation during mining.

Table 1 Results of LE analyses for RAS open pit design *ras_hg_cone1.dtm* (Figure 28)

Section	**Slope condition**	**Loading**	**Minimum FOS** Morgenstern-Price	**Comment**
EAST	Dry	Static	1.59	Completely dry conditions highly unlikely
	Dry	Pseudo-static	1.27	Seismic load 0.13g lateral acceleration
	Saturated	Static	1.20	
	Saturated	Pseudo-static	0.93	0.13g disturbance
	Semi-saturated	Static	1.21	
	Semi-saturated	Pseudo-static	1.04	
N-EAST	Dry	Static	1.79	Completely dry conditions highly unlikely
	Dry	Pseudo-static	1.54	0.13g disturbance
	Saturated	Static	1.39	
	Saturated	Pseudo-static	1.12	
	Semi-saturated	Static	1.44	
	Semi-saturated	Pseudo-static	1.23	
WEST	Dry	Static	1.00	TGF dips out of wall & is undercut
	Dry	Pseudo-static	0.99	0.13g disturbance
	Saturated	Static	<< 1	
	Saturated	Pseudo-static	<< 1	0.13g disturbance
	Semi-saturated	Static	0.77	SWL at TGF
	Semi-saturated	Pseudo-static	0.75	0.13g
S-WEST	Dry	Static	1.31	TGF dips out of wall & is undercut
	Dry	Pseudo-static	0.94	0.13g disturbance
	Saturated	Static	0.62	
	Saturated	Pseudo-static	<< 1	0.13g disturbance
	Semi-saturated	Static	1.12	
	Semi-saturated	Pseudo-static	0.84	0.13g disturbance

Table 2 Results of LE analyses for RAS open pit design *ras_phase7.dtm* (Figure 29)

Section	Slope condition	Loading	Minimum FOS Morgenstern-Price	Comment
NE 216.6m high 46° overall slope angle	Dry	Static	1.88	Completely dry conditions highly unlikely
	Dry	Pseudo-static	1.54	Seismic load 0.13g lateral acceleration
	Semi-saturated	Static	1.33	
	Semi-saturated	Pseudo-static	1.07	
SE 316m high 44° overall slope angle	Dry	Static	1.79	Completely dry conditions highly unlikely
	Dry	Pseudo-static	1.46	0.13g disturbance
	Semi-saturated	Static	1.40	
	Semi-saturated	Pseudo-static	1.13	
WEST 273m high 31° overall slope angle	Dry	Static	1.21	TGF dips out of wall & is undercut Minimum FOS restricted to undercut TZ3 & lies within the upper ~ 155m of the wall
	Dry	Pseudo-static	0.84	0.13g disturbance Minimum FOS restricted to undercut TZ3 in the upper ~ 155m of the wall
	Semi-saturated	Static	1.12	SWL at TGF Minimum FOS restricted to undercut TZ3 in the upper ~ 155m of the wall
	Semi-saturated	Pseudo-static	0.84	0.13g, failure is above groundwater level Minimum FOS restricted to undercut TZ3 in the upper ~ 155m of the wall
	Semi-saturated	Static	2.00	Analysis forced to assess overall slope Minimum FOS surface includes ~ 34m TZ4
WEST East wall cut by W section 238m high 43° overall slope angle	Dry	Static	1.84	TGF dips into wall
	Dry	Pseudo-static	1.49	0.13g disturbance
	Semi-saturated	Static	1.31	SWL at TGF
	Semi-saturated	Pseudo-static	1.00	0.13g
SOUTHERN 222m high upper wall segment 39° segment overall angle	Dry	Static	1.35	TGF dips out of wall & is undercut
	Dry	Pseudo-static	1.07	0.13g disturbance
	Semi-saturated	Static	1.15	
	Semi-saturated	Pseudo-static	0.98	0.13g disturbance

Results of 2D LE analyses return results for varied groundwater and loading conditions as follows:

Dry slope, static loading

TGF dipping into slope
Acceptable FOS, between ~ 1.6 & 2.0 (includes FOS values not listed in the Table 1 summary)

TGF undercut by slope
FOS ~1.3 to ≤ 1.0, stable to metastable / unstable, depending on TGF dip & undercut block mass

Dry slope, pseudo-static loading (seismic disturbance)

TGF dipping into slope
Acceptable FOS, between ~ 1.3 & 1.6

TGF undercut by slope
FOS ≤ 1.0, metastable to unstable

Saturated slope, static loading

TGF dipping into slope
Acceptable FOS, 1.2 to 1.4

TGF undercut by slope
FOS ≤ 1.0 unstable

Saturated slope, pseudo-static loading

TGF dipping into slope
FOS ≤ 1.0 to ~ 1.1, metastable to unstable

TGF undercut by slope
FOS ≤ 1.0 unstable

Semi-saturated slope, static loading

TGF dipping into slope
Acceptable FOS, between 1.2 & 1.50

TGF undercut by slope
FOS ≤ 1.0 to ~ 1.1, unstable to metastable

Semi-saturated slope, pseudo-static loading

TGF dipping into slope
FOS ~ 1.1 to 1.3, dependent on slope height & angle & apparent dip of TGF

TGF undercut by slope
FOS ~ 0.8 unstable

5.2.3 Numerical Analyses

Numerical (finite element (FE)) analysis of RAS slope stability has been performed using the Rocscience program RS2 [11]. This approach provides a check on results from LE analyses using a shear strength reduction routine to find the critical point at which the modelled slope fails (results fail to converge). A critical strength reduction factor (SRF) based on the ratio of the reduced strength at which failure occurs versus the adopted/ actual strength is fundamentally equivalent to the LEA FOS.

The FE analysis process includes the entire slope and surrounds and identifies the location and shape of the critical shear surface (whereas the LEA search range is identified by the user).

The RS2 FE analyses returned results that essentially coincide with and confirm the LEA results.

5.3 Comment on Stability Analyses

LE analyses do not explicitly include structural features; however, directional bias in rock mass strength due to structural influences can be incorporated via anisotropic strength functions, for example, to represent the influence of (typically weaker) bedding or foliation fabrics. An anisotropic function was assigned to TZ3 to reflect the anticipated lesser shear strength parallel to the dip of the bedding/ foliation fabric.

The lowest FOS thus apply as the pit approaches or exposes the TGF, that is, as open pit mining nears full depth.

Regardless of the pit design assessed, results from 2D stability analyses highlight a high potential for the TGF to be an adverse influence on wall stability. In RAS pit designs most of the TGF-pit wall intersection length is located on the eastern and western sidewalls, with the TGF ultimately dipping into the north wall. It is theoretically possible that undissipated groundwater pressures could enable the ‘uphill’ shear failure and displacement of the toes of these walls predicted by analyses. The forces produced by the mass of the overlying highwall materials are such that uphill displacements (along the TGF dipping into the wall) could be initiated and lead to collapse.

The LE analyses results presented are generally based on unrestricted search ranges. Lowest FOS surfaces typically span from surface to the TGF, following a quasi-parabolic arc from surface to the TGF. These surfaces breakout along the weakest sliding path which is defined by the core of the TGF. The surfaces can follow the TGF several tens of metres to ≥ 100m (prior to daylighting at the wall). At this stage it is unknown whether this theoretical propensity for shearing to occur over such distances on/ within the core of the TGF is real.

Where groundwater pressures remain undissipated, the mass of the typical volume overlying these theoretical slip surfaces is sufficient to drive displacement along the TGF, even where the fault dips into the wall (sometimes at moderately steep inclination).

The footwalls of the inspected RAS pit designs/ shells variously undercut the TGF in the upper southern and south-western sectors. The floor then tracks beneath the TGF to the toe of the northern highwall. These directly undercut zones (where slope dip ≈ TGF true dip) are theoretically unstable.

Direct undercutting of the TGF should be avoided wherever practicably possible.

Exposure of the TGF by mining stage as defined by the May 2025 pit design is shown in Figure 32.

Outside the zone in which planar sliding could occur directly out of the face (block movement sub-perpendicular to the slope) the TGF intersection is inclined and direct down-dip movement of the block(s) in the hangingwall of the fault zone is ‘buttressed’ by the pit walls (eastern and western walls in this case). Movement into the pit must therefore occur obliquely across the dip of the fault zone sliding obliquely, on an apparent dip and greater driving forces are required to initiate movement.

In some instances, in RAS 2D LE analysis, however, the two-dimensional geometry of geological units on cut sections is misleading with respect to the intersection between the TGF and pit walls because wall curvature limits the span over which simple planar sliding could occur.

Figure 32 RAS open pit intersection with TGF Stages 1 to 7

Additionally, as noted, 2D LE stability analysis assumes the geometry of the geology and slope (represented by the model section) are infinite. As such, the method cannot account for the influence of the of lateral restraint against sliding (provided by the third dimension) and 2D analysis results, while typically conservative, may well be misleading.

Future three-dimensional numerical analysis is strongly recommended.

Avoiding the potentially adverse influence of the TGF on slope stability is not straightforward. The toe of an adjusted wall design would need to be offset from the TGF (daylighting in the floor) by something in the order of 30m. That disadvantage can, however, be at least partially offset by the ability to increase the inclination of the wall, no longer subject to the adverse influence of the TGF. Figure 33 schematically illustrates the possible offset.

Ore loss may be compensated by selectively mining into the pit floor on retreat under real time monitoring. For example, local excavation into the pit floor, approaching the toe of the wall between a series of buttressing pillars. Dependent on monitoring results, the pillars might be partially scavenged.

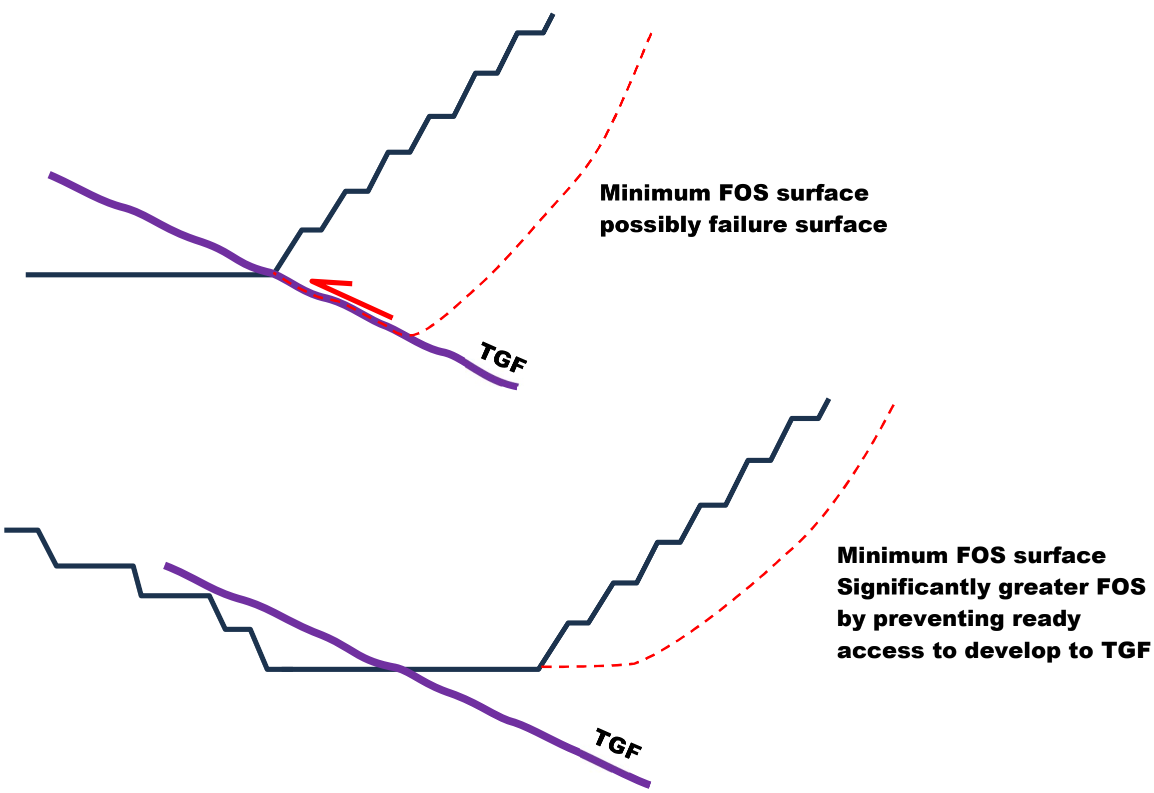


Figure 33 Avoidance of adverse TGF influence on wall stability

Additional hydrogeological assessment is recommended.

Further hydrogeological assessment is required to identify the potential for dewatering at RAS and the time(s) and methods by which adequate drawdown and depressurisation can be achieved.

The hydraulic pressurisation suggested in the fully saturated condition are unlikely to be encountered (at least initially) – the rock mass will relax on unloading (excavation) and reduce the active pressures in pit walls (at least temporarily). To augment these effects, installation of sub-horizontal depressurisation boreholes is recommended. Holes of up to ~ 60m long are anticipated to be necessary.

5.4 Recommended Open Pit Wall Design Parameters

The wall design parameters recommended for the RAS open pit are listed in Figure 34 and Table 3.

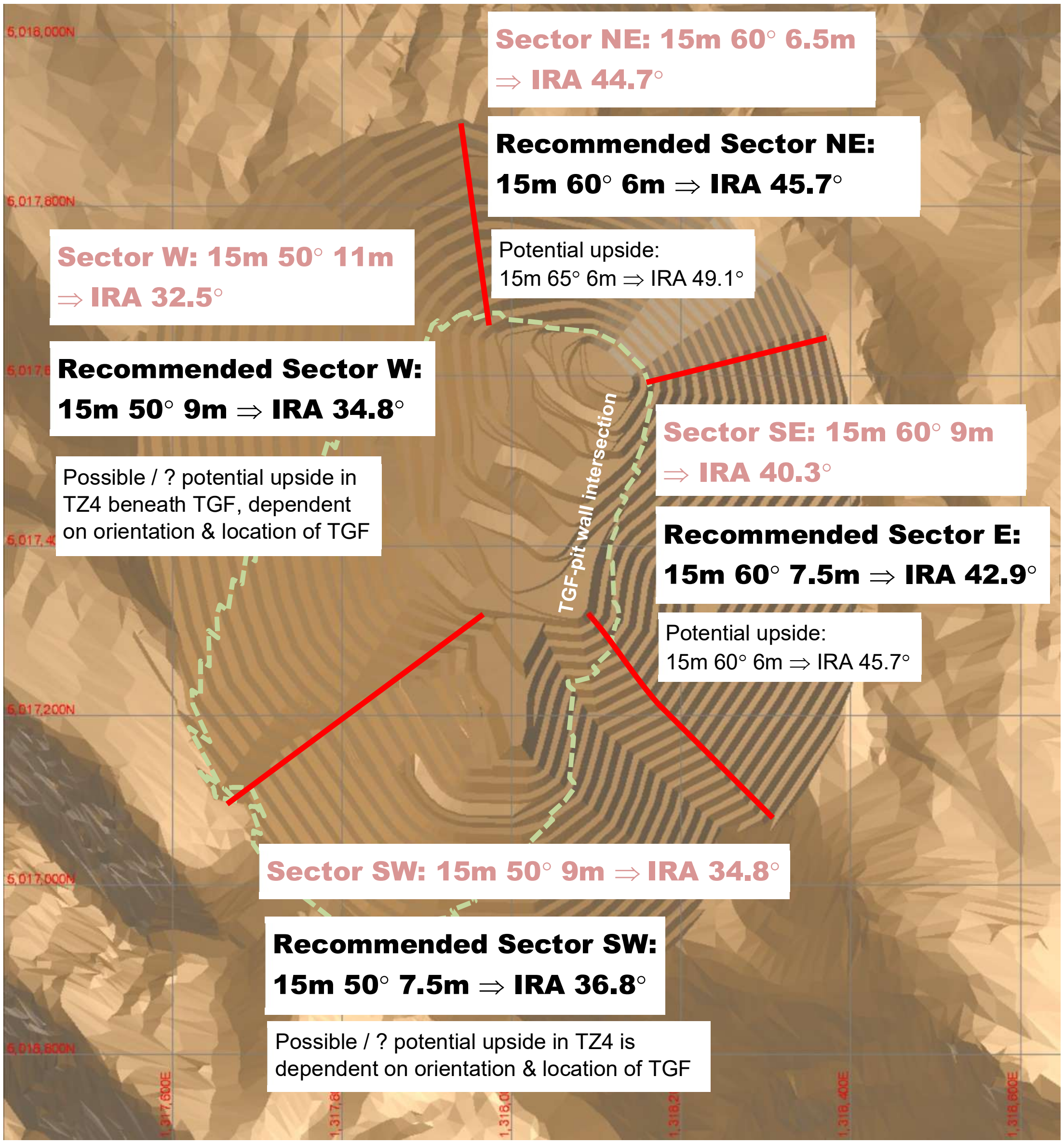


Figure 34 RAS recommended open pit wall design parameters (on Phase 7 pit configuration)

Pit design *ras_hg_cone1* is used to define design domain parameters (PSM [1]) are shown as:

Sector N: Batter ht., face angle, berm width ⇒ IRA

Recommended bases case design parameters for each sector are shown as:

Recommended Sector N: Batter ht., face angle, berm width ⇒ IRA

Recommended inter-ramp profiles for the above design domains are:

Sector W:	15m high, 50° face angle batters, separated by 9m wide berms	⇒ for inter-ramp angle (IRA) 34.8°
Sector NE:	15m high, 60° face, 6m wide berm	⇒ IRA 45.7°
Sector E:	15m high, 60° face, 7.5m wide berm	⇒ IRA 42.5°
Sector SW:	15m high, 50° face, 7.5m wide berm	⇒ IRA 36.8°

Table 3 Recommended RAS open pit *base case* wall design parameters

SECTOR	FACE HEIGHT	FACE ANGLE	BERM WIDTH	INTER-RAMP ANGLE	Potential upside
WEST	15m	50°	9m	34.8°	Unknown
NORTH-EAST	15m	60°	6m	45.7°	? 49.1° IRA
EAST	15m	60°	7.5m	42.9°	? 45.7° IRA
SOUTH-WEST	15m	50°	7.5m	36.8°	Unknown

5.5 Comment on Open Pit Wall Design Recommendations

The following comments are considered to be applicable to the recommended *base case* design parameters for proposed open pit mining at RAS:

- **The recommended parameters are neither conservative nor overly aggressive.**

  Mining to the recommended wall parameters is expected to be accompanied by some local batter scale wall failures. Careful slope monitoring will be required throughout all stages of mining (including stability monitoring of interim slopes).

  The parameters are recommended with an expectation that initial mining will allow use of observational techniques (Section 5.8) to refine slope parameters for final walls. That is, assessment of interim slopes will permit confirmation and/or amendment of the parameters.
- Successful use of appropriate mining techniques, particularly in drilling and blasting and excavation during development of final walls, will be critical to the achievement of the design and maintenance of wall stability.
- A key performance indicator for pit wall development should be for ≥ 85% of berms to be formed at design width.
- Note that inclusion of access ramps will reduce the overall angles achieved within the pit.
- The recommended parameters assume that stable wall conditions are required for the life of the open pit only. Further geotechnical assessment would be required if longer term pit access is required, for example, for access to future underground operations.
- Local adjustments to design parameters may be necessary to satisfy stability requirements. Few data are known regarding the persistence of geological structures which could contribute to instability. Flattened batters and/or wider berms may be necessary locally. Conversely, there may be opportunity for local wall steepening.
- Convex, unconfined slope sectors (bullnoses) must be expected to be prone to failure. While it is reasonable to include such shapes in pit plans, (rather than committing directly to remove large ‘additional’ volumes of waste) Matakanui must remain aware of the potential for instability and the possible need to adjust (‘smooth out’) bullnose areas. Ideally, bullnoses should be avoided as far as practicable during final design.

Local factors which could run counter to successful implementation of these parameters could arise where/ if:

⇨ Highly structured zones compromise the inherent strength of the fresh rock mass

⇨ Persistent faults/ shears are oriented and located such that slope stability could be compromised. For example, via the structure in question being undercut by the wall, or where the structure could provide lateral or rear release for movement of adjacent blocks.

Accommodating TGF Exposure

As Matakanui has noted, there will be potential for batter fretting in and around areas where the TGF is exposed in pit walls. Damage or disturbance to the TGF could result in batter undercutting and promote propagation of deterioration upslope and laterally. Performance of the core of the TGF on exposure will depend on local thickness and moisture content. TGF materials will vary from competent stiff to firm clay to soft clay or gouge each ± rock fragments (of various competencies) and may be highly fractured disintegrated rock locally.

Measures needed to protect/ retain the TGF where exposed are expected to vary across the site. To retain TGF it is likely to be necessary locally to seal the exposure using sprayed concrete (simplest approach), which will need to be pinned in place. Fibre-reinforced shotcrete (FRS) and bolts or shotcrete over pinned mesh or pinned mesh over shotcrete should provide an acceptable long-term solution.

Blasting through or adjacent to positions where the TGF will be exposed in the final wall should be avoided. Ideally, batters in the vicinity of the TGF will be mechanically excavated/ profiled. Conditions observed in cores suggest (usually strongly) that the TGF and immediately overlying TZ3 will be amenable to mechanical excavation, which should reduce/ avoid dilution and enable selective loading of ore and waste rocks. Resistance to mechanical excavation will increase with depth below the TGF and resort to drill and blast will be necessary within TZ4 rock outside the RSSZ. Blasting may be necessary at some locations within the interpreted RSSZ boundaries.

Previous Wall Parameters

Variations to the PSM recommendations are derived from:

- Assumed successful active depressurisation – to the drawn-down phreatic profiles modelled
- Analyses including selective application of anisotropic shear strength in TZ3 provide improved FOS results. These analyses restrict the applicability of the low shear strength to those directions sub-parallel to the TGF.
- A different approach was used in modelling hydrostatic pressures. (The approach used is inferred to be more realistic.)

Note that the analyses (by PSM & ourselves) assumed no wall damage from blasting.

Upside in wall design/ wall profile is inferred based on consideration of:

- Relatively high FOS obtained in the nominated sectors
- The unrealistic assumption the analyses make concerning geometry into the third dimension
- That in the real situation there would be lateral confinement.

It is possible that upside exists for those segments of pit walls mined in TZ4 where sufficiently beneath the TGF and out of the influence of the RSSZ. This potential should be examined/ evaluated as part of the detailed pit design process.

Upper batters were originally recommended to be limited to 10m height to 30m depth to accommodate inferred lesser competence in near-surface materials. This recommendation should be reinstated in final design, with the extent and height of its application based on findings of proposed geotechnical drilling investigations.

The universal use of 15m high batters in preliminary design work was conceded after requests from MineComp (Mine Planning Consultants) to avoid difficulties in matching pit excavation to topography. It is recommended that reduced batter height in the upper walls be considered and adopted, wherever practicably possible, in detailed pit design.

5.6 Long Term Pit Wall Stability

Assumptions concerning degradation are subjective and the manner and rate of degradation of rock mass conditions over time are difficult to quantify. There is a paucity of relevant examples.

Pit walls inevitably deteriorate with time, due to stress conditions and/ or ongoing relaxation (surficial loosening and unravelling); weathering/ alteration causing shear strength changes in wall materials (from reduction in intact material strength and reduction in structural defect shear strength); physical wastage via wind and rainfall run-off (or flooding) erosion; re-establishment of hydrostatic pressures in pit walls; influence of underground voids; and potentially intermittent transient seismic disturbance.

Degradation of pit walls, however, does not necessarily lead to wall collapse. At RAS it is expected that debris derived from slope degradation would be contained within the pit, largely on the pit slopes.

The expected pattern of deterioration at RAS is that surficial fretting within TZ3 rocks will lead to accumulation of debris against batter toes. Berm allowances will be sufficient to contain this fretting debris. Tenuously stable blocks or wedges exposed by mining would be expected to fail/ displace during operation, typically shortly following exposure allowing timely clean-up and repair. Block instability following completion of mining could conceivably occur and while this may be localised, with debris largely contained on the underlying slope, it is possible that such events could promote propagation of instability, and it remains possible that large scale instability could develop with time.

There are, however, no standard/ industry-accepted methods (empirical or deterministic) to identify or predict the volume of debris that may be generated by long-term slope deterioration.

Modification/ downgrading strengths of near surface zones in numerical models or performing multiple cycle block stability analyses could provide a reasonable (realistic?) estimate of the possible extent of deterioration and slope retreat. It is inferred the limit of slope regression would be dependent on the angle of repose of the disintegrated rock mass and the volume and distribution of rock debris, beyond which point erosion and seismicity would be the dominant disturbance factors.

A pit lake will develop following suspension of pit dewatering at RAS. Wave action can lead to in weak or intensely jointed/ fractured materials at/ near lake surface levels. Deterioration and shallow slumping could occur due to saturation in weak clay-rich or loosened materials at/ near the wall surface. Undercutting via such wave action could cause minor upslope propagation of fretting.

The presence of a pit lake reduces the potential for destabilisation from hydrostatic pressures within wall rocks. The pressure exerted by the body of water in the lake acts against the groundwater hydrostatic pressure acting within the pit walls below the lake level.

Given the known statistical rate of recurrence and magnitudes of seismic activity possible at RAS, earthquake-induced disturbance could conceivably be a major adverse influence on pit wall stability. The extent to which pit slope angles would need to be reduced to negate entirely the theoretical possibility of earthquake destabilisation, would likely not be economically viable.

A well-documented database and record of slope stability performance during mining operations will aid prediction/ identification of the manner of development of wall instability over the long-term. Failure may be structurally controlled by the general shear strength, and potentially the tensile characteristics, of the rock mass; by bedding/ ubiquitous foliation, or combination(s) of influences. All open structures may promote groundwater movement and facilitate/ accelerate slope deterioration.

6. Underground Mining Assessment

Underground development within TZ3 is anticipated to require general adoption of high capacity support schemes, based on floor to floor application of fibrecrete and closely spaced rockbolts. Drive advance should be based on short ends. It is possible that spiling/ forepoling of some manner will be required locally.

Initial access development into and within TZ4 will likely require continued use of TZ3 support; however, expected improvement of ground conditions away from the TGF 'contact' zone should allow application of less intensive support, eventually to be based on 'standard' rockbolt and mesh schemes.

Underground production will likely be based variously on drift and fill (D&F immediately beneath the TGF) and primary-secondary longhole open stoping (LHOS) with engineered (paste) backfill.

High RQD cannot be used as a singular indicator of rock mass conditions/ quality or competence.

The separation between the nearest point of the geotechnical borehole nearest to proposed underground development is ~ 170m (the toe of MDD287 to the southernmost drive at 380mRL) (Figure 35). The 'average' separation distance between geotechnical boreholes and an approximate stoping mid-point is ~ 600m.

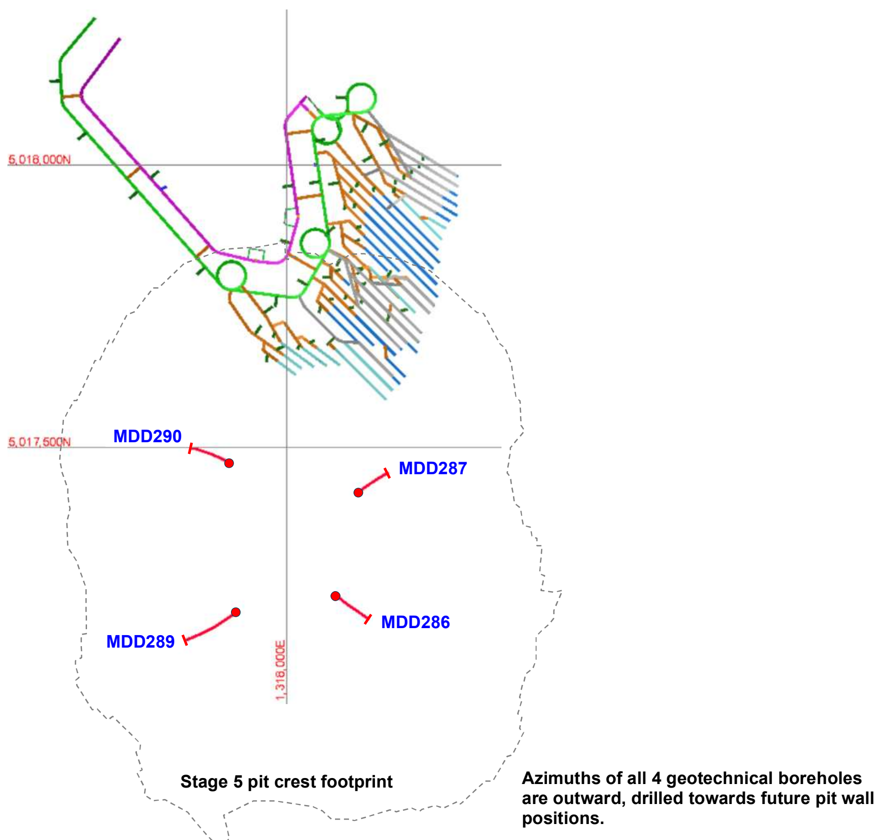


Figure 35 Geotechnical borehole locations relative to underground workings

6.1 Portal & Decline Development

Underground access is planned to be via a main decline with a portal developed from a cut into the southern flank of Shepherds Valley (Figure 8). A ventilation decline would be developed from a point ~ 50m south-east of the main access decline.

Figure 36 illustrates a general cut and fill arrangement for portal development and decline access.

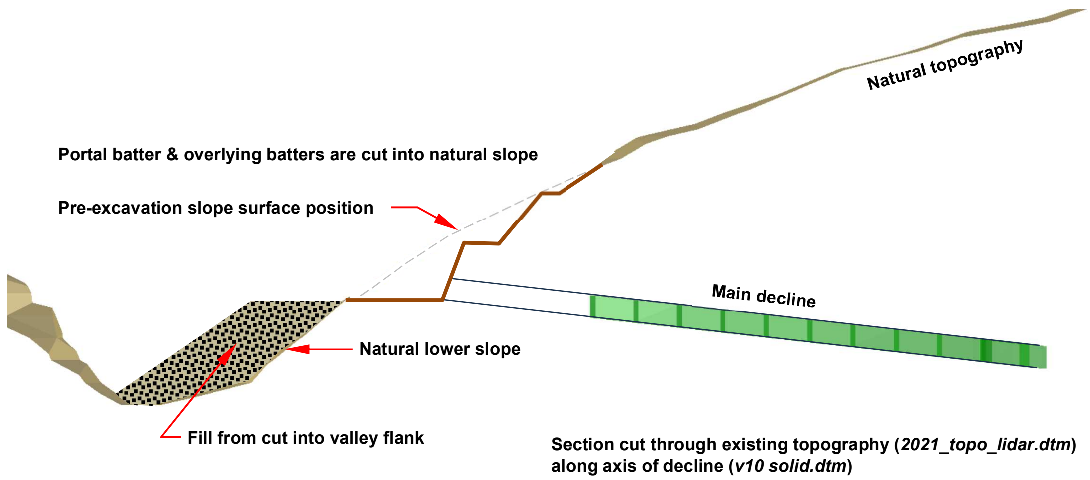


Figure 36 Hillside cut for portal batter (schematic only)

The notional portal sites have not been inspected or investigated (to our knowledge); however, it is anticipated that ground conditions will be *poor*.

The profile of the hillside cut:

Upper batters ≤ 10m high, top batter face angle 35°, mid-batter(s) 50°
Portal batter ≥ 15m high, face angle 70°
Upper berm(s) ≥ 5m wide
Berm over portal ≥ 10m wide.

Figure 37 illustrates a generic configuration portal batter support in weathered/ fractured rock. The portal batter is to be ≥ 15m high, with the face at ~ 70° to facilitate initial decline development. Cover over the portal area will be ~ 2 × decline height.

The batter in the portal area to ≥ 5m away from the sidewalls of the opening is to be supported with galvanised high tensile steel woven wire mesh suspended from bolts in the overlying berm and pinned to the face where required to maintain intimate/ close contact to the face. Alternatively, the batter in this area is to be supported with bolt-pinned weldmesh sheets with 300mm overlap.

The entire bolted and meshed face is to be sprayed with a 40mm thick layer of shotcrete/ fibrecrete (≥ 32 MPa 28-day compressive strength).

The periphery of the portal is to be rock bolted (solid bolts, full-column resin or cement grouted) and cable bolted as shown (for example) in Figure 39. Direct inspection of the excavation is required to refine/ confirm support requirements.

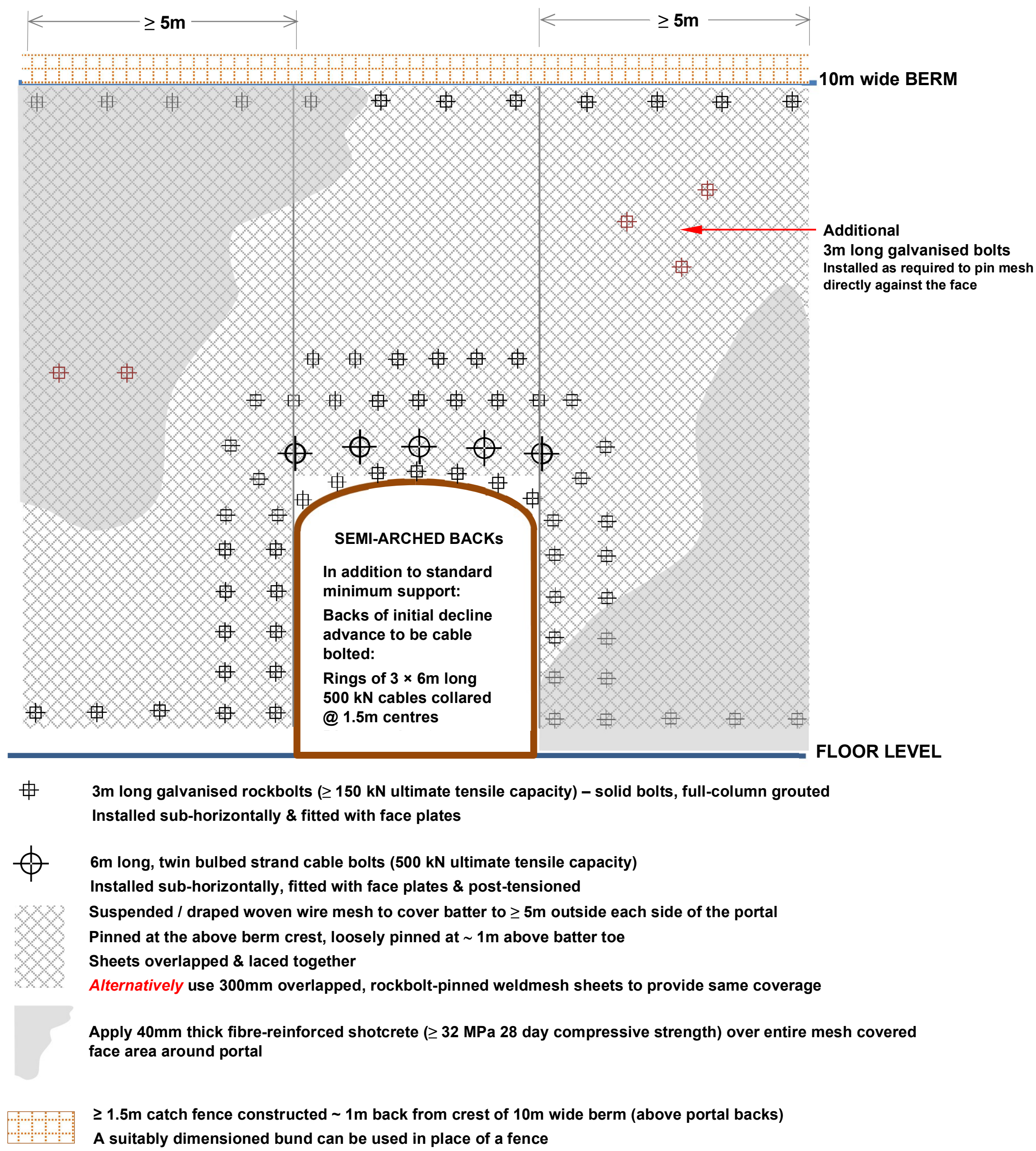


Figure 37 Example of suitable portal batter face reinforcement & surface support

Initial main decline development would be in TZ3 for ~ 500m and for ~ 570m in the ventilation decline. After developing through the core of the TGF most remaining access development would be in TZ4; however, the conceptual access arrangement indicates some decline segments and local cross-cut access, and some portions of production drifts must be developed in TZ3 (Figure 9).

A semi-arched decline profile of ~ 5.5m width × 5.0 to 5.5m height is assumed.

Within TZ3 and other similarly *poor* ground access opening support schemes are to be based on a 75mm thick fibrecrete (FRS) layer sprayed floor to floor and pinned with 2.4m long full-column grouted rockbolts installed on a 1.2m × 1.2m grid. FRS is to be applied as soon as is practicable following firing. Ideally, drive backs and shoulders will be sprayed as soon as mucking provides sufficient space to operate a boom over the muckpile. Sidewalls are sprayed and all bolts installed following completion of mucking.

It is anticipated that shortened development ends will be necessary in TZ3 (and other *poor* ground) to avoid deterioration of stability conditions prior to installation of support.

Where underground access is developed in the more competent ground in TZ4 (beyond the adverse influence of the TGF and RSSZ) it is expected that conditions will be such that support may be reduced to a scheme/ schemes based on rockbolts and mesh. The extent to which such a change may be applied and the manner in which it could be managed (gradual or a simple step-change) are unknown at present.

6.2 Stability & Support of Access Development

Underground opening stability will be governed by the general quality and the geological structures within the local rock mass, which is anticipated to vary from *very poor* through *poor*, *fair/ good* to *good* with rock/ textural zone type and depth.

Decline portal development: surface box-cut excavation into hillside in weathered ground and TZ3 schists.

Decline development in TZ3: decline development will be in varied ground conditions, ranging from *fair* to *very poor*. Support schemes must be based on floor to floor surface support (fibrecrete and rockbolts), possibly also requiring mesh if squeezing conditions are encountered. It is inferred that accesses in TZ3 schists could be mined mechanically, for example, developed using a suitably configured road header.

Decline development in TZ4: once development has advanced well into TZ4, (away from TGF and RSSZ influence), support schemes based on bolts and mesh to either the grade line or within ≤ 3.5m of floor level will be adequate.

Cross-cut development: cross-cut development in TZ4 can be based on bolts and mesh. Any cross-cut development in TZ3 or upper 'disturbed' TZ4 will require a support scheme based on fibrecrete and bolts.

Drive intersections: intersections will require installation of standard support for the local rock type plus cable bolts (nominally 6m long) installed on a 2m × 2m grid within the central two-thirds of the intersection area. All cables with borehole collars accessible from the initially developed drive are installed prior to commencement of stripping for the intersection.

Stope drive development: top sill development will require support schemes based on fibrecrete and bolts plus cable bolts.

General experience indicates minimum ground support for declines, decline stockpiles, sub-level access and ore drives can be based on friction bolt reinforcement in concert with weldmesh supporting opening surfaces.

It is recommended that allowance be made to augment standard support schemes in the initial advance of the decline and for wide spans (for example, drive intersections, with cable bolt reinforcement).

Use of galvanised reinforcing and support fixtures is recommended for all capital development.

6.2.1 Decline & Access Drive Support

Decline – Upper: 5.5m W × 5.5m H, semi-arched back

Support schemes must be based on floor to floor surface support (100mm fibrecrete (FRS) (32 MPa 28 day UCS) and solid resin grouted rockbolts (≥ 18mm diameter, 2.4m long) on a ≤ 1.2m × 1.2m grid). Mesh over FRS may be required if squeezing conditions are encountered in development.

For the initial ~ 20m advance, decline support is to include:

6m long, twin bulbed strand cable bolts are to be installed in rings of three (3) cables at 1.5m collar spacings, with the central cable installed on the centreline of the decline back.

The centreline cable is vertical; outer cables are to be dumped outward (in-plane) by ~ 15°.

Inter-ring spacing is to be 2m.

Remainder of decline development 5.5m W × 5.5m H, arched back

Retain use of floor to floor FRS plus solid resin grouted bolts. Reduction to ≥ 75mm thick FRS may be possible in TZ3 and upper TZ4 with increased depth below surface. Geotechnical/ structural mapping will be required to identify the actual distributions and larger scale characteristics of rock defect sets and, in turn, optimise support scheme parameters.

Lower level decline development in TZ4 5.5m W × 5.5m H, arched back

Weldmesh surface support (nominal 6mm diameter wire in 100mm × 100mm grid) installed grade line to grade line (to ≤ 3.5m from floor level)

2.4m long, 46mm diameter friction bolts on a nominal 1.4m (intra-ring) × 1.4m grid (inter-ring).

Use galvanised mesh and bolts in capital development.

6.2.2 Cross-cuts

5.5m W × 5.5m H, arched back in TZ3 & upper TZ4

Floor to floor ≥ 75mm layer FRS (32 MPa 28 day UCS).

2.4m long, 18mm diameter solid bolts on a nominal 1.2m (intra-ring) × 1.2m grid (inter-ring).

5.5m W × 5.5m H, arched back in lower TZ4

Weldmesh surface support (nominal 6mm diameter wire in 100mm × 100mm grid) to ≤ 3.5m from floor level.

2.4m long, 46mm diameter friction bolts on a nominal 1.4m (intra-ring) × 1.4m grid (inter-ring).

6.2.3 Ore & Ventilation Drives

5m W × 5m H, semi-arched back in TZ3 & upper TZ4

Floor to floor ≥ 75mm layer FRS (32 MPa 28 day UCS).

2.4m long, 18mm diameter solid bolts on a nominal 1.2m (intra-ring) × 1.2m grid (inter-ring)

5.0m W × 5.0m H, arched back in lower TZ4

Weldmesh surface support (nominal 6mm diameter wire in 100mm × 100mm grid) to ≤ 3.5m from floor level.

2.4m long, 46mm diameter friction bolts on a nominal 1.2m (intra-ring) × 1.4m grid (inter-ring)

While the recommended bolting pattern matches that for the wider openings, intra-ring bolt positioning can be adjusted to reduce ring requirements by one bolt.

6.3 Wide Span Support

In addition to the above-noted standard support scheme, it will be necessary to assess requirements for additional reinforcement in wide spans/ intersections.

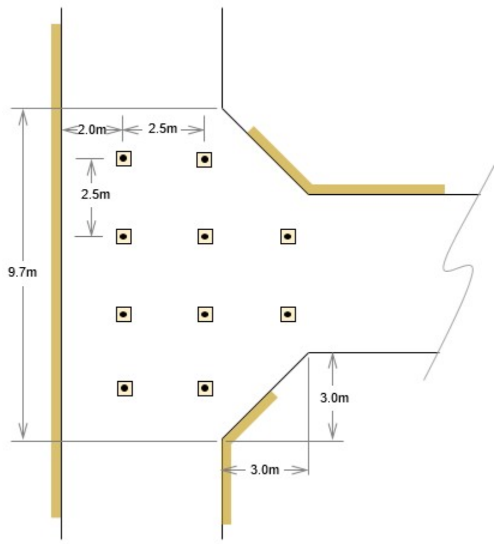


Figure 38 T-intersection ground support
Schematic example only

Two approaches are possible in this regard; the first is to assess each intersection/ wide span area (before development of the wide span) for suitability of the location for development and design of specific local requirements; or adopting wide span reinforcement as a standard practice.

Intersections are to be reinforced as standard practice, use of full-column cement grouted and post-tensioned, 6m long twin bulbed strand cables is recommended. The cable would be installed sub-vertically on a nominal 2m × 2.5m pattern (maximum cable collar grid to be ≤ 2.5m × 2.5m) within at least the central two thirds of the wider intersection span area. Figure 38 shows an example of an appropriate cable bolt configuration to reinforce a T-intersection of two 5.5m wide drives.

Support design needs to consider structural conditions assessed directly from the areas/ zones in which access development and production will be carried out and not be based only/ simply on empirical assessment and/ or results from proposed numerical analysis.

The proposed approach to exposing longhole open stope (LHOS) backs is inferred to be non-viable. Cable bolt reinforcement into the lower TZ3 zone (from a single top sill drive) has a low likelihood of success, and (the alternative of) allowing the backs to cave would introduce excessive dilution which could become uncontrollable.

7. Stoping

7.1 Stoping Methods

The assessed quality of the fresh rock mass at RAS is such that stoping must be based on modestly dimensioned openings; however, it is inferred that this can be managed using bulk mining techniques, albeit with requirements to mine at a small scale to stabilise the lode hangingwall with the aim of establishing sustainable stope backs.

The RAS lode dips/ plunges shallowly to the north north-east. It is inferred that limited height sub-level longhole open stoping (LHOS) can be used successfully provided measures are in place to stabilise the lode hangingwall and engineered backfill provides resistance against overbreak.

Fundamentally the problems relate to the poor rock mass quality within the TZ3, TGF and RSSZ and in the upper horizon of TZ4, where there is potential for overbreak into and dilution from the hangingwall and stope backs. It is conceivable that caving could be initiated. The extent to which caving could propagate would be dependent on local ground conditions, presence/ absence of major faults and stoping geometry, that is, effective hangingwall spans and available stope void. Specific potentials for caving are unknown; however, given limited stope volumes it is expected that ore draw would be terminated, and caving would be choked well before overbreak could reach natural surface.

The ability to successfully support the hangingwall/ stope backs is contingent on the rock mass quality within the zone being sufficient to enable reinforcing elements to a) work and b) interact effectively and that access development can be configured to allow adequate quantities and areal coverage of reinforcement and support to be installed.

It is possible that some areas will be difficult/ impossible to reinforce effectively. In some poorer areas it is conceivable that the rock mass could unravel around installed reinforcing elements.

Deeper into TZ4, rock mass quality is significantly greater and amenable to more conventional LHOS.

Backfill reticulation must be devised such that the fill makes intimate contact with (most of) the hangingwall/ stope backs. The passive support provided by the fill limits the extent to which overbreak can propagate; however, it cannot be expected to prevent all displacement within and dilation of the overlying rock mass. Even engineered (cemented) fill must experience high strains before significant resistance.

Backfill must be placed as soon as is practicably possible after mucking is completed. There is an inter-dependence between stope back support and (eventual) backfill support. The stope back support scheme (reinforcement plus support elements such as mesh and fibrecrete) must work until the stope is ‘tight’ filled. If the support fails before completion of backfilling, complete backfilling may not be possible. Without complete backfilling it is possible that the mining or stability of adjacent stopes or stoping panels could be compromised.

The effective/ cumulative span of the hangingwall must also be considered. Barrier pillars will likely be needed (at least temporarily) to separate stoping panels and thus divide hangingwall exposure into a number of sustainable spans. Recovery or partial recovery of these pillars will depend on the effectiveness and areal coverage of tight fill and monitored hangingwall displacements (measured via borehole extensometers and ground surface survey).

It is recommended that capital development be located ≥ 40m from planned stoping. More specific guidance can be provided when results of recommended numerical modelling are interrogated.

7.2 Stoping Parameters

Assessment of sustainable stoping spans has used the empirical Modified Stability Graph method, initially devised by Mathews *et al* (1981) [12] for deep bulk stoping, and subsequently modified by other workers, notably Potvin (1988) [13] and Mawdesley *et al* (2001) [14] for more general application. It is important to note that the method is based on data from numerous locations and settings and thus should be used cautiously until experience at RAS provides information for site specific stope design.

At this stage of assessment, the rock mass ratings have been based on data from the four (4) geotechnically logged and televiewed boreholes (MDD286, 287, 289 and 290).

Derivation of stoping parameters has been based on use of a nominal 1st quartile values of rock mass quality Q′ = 50 for country rocks and 93 for ore zones, (adopting Q′ = Q, which is based on assumptions that groundwater conditions are benign, and that rock mass competence is not significantly influenced by ambient stresses).

$$\mathbf{Q = RQD / J_n \times J_r / J_a \times J_w / SRF}$$

$$\mathbf{Q' = RQD / J_n \times J_r / J_a\ (\times 1 / 1)}$$

Where RQD is Rock Quality Designation, based on logged measurements

J_n is a rating based on the number of defect sets identified from core photographs

J_r is rated according to logged defect surface conditions

J_a is rated according to logged defect wall alteration

J_w is a rating for groundwater

SRF is a stress reduction factor

In the absence of data in manipulative format, estimates of Q have been based on empirical relationships with the ‘typical’ (inferred estimated mean or median) GSI (Geological Strength Index) and RMR (Rock Mass Rating) values reported by PSM [1]. The derived mean/ median Q values have been used in this assessment whereas it is usual to use lesser values (for example, 1st quartile) is assessing dimensions for bulk stoping. It has been assumed that Q′ = Q.

The Modified Stability Graph method involves calculation of a Stability Number (N) which is the product of the assessed Q′ values and three factors (A, B and C Factors):

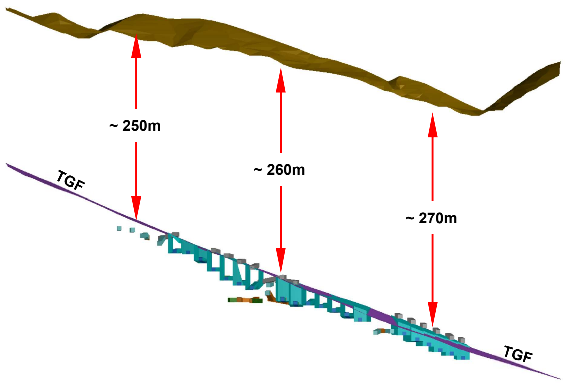
$$\mathbf{N = Q' \times A \times B \times C}$$

The factors aim to account for the influences of induced rock stress relative to intact rock strength (A); critical structure orientation relative to the surface being considered (B); and possible structural failure mechanisms relative to stope surfaces (C).

Note that the method does not and cannot account for the individual influence(s) of major geological structures. The potential for adverse influence on stability due to the presence of major structures, when/ if identified as proximal to stoping, should be assessed on a case by case basis.

Factor A is based on the ratio σ_C / σ_1, where σ_C is the mean intact rock strength, relying on logging estimates and laboratory testing performed on geotechnical diamond cores for rock substance strength. Mean UCS values are ~ 20 MPa for TZ3 and ~ 70 MPa for TZ4.

Mining depths are shown in long section in Figure 39.



Depths from surface to TGF = stope backs vary from ~ 250m to ~ 270m

Figure 39 Section cut on approximate centreline of stoping panels

For backs and stope sidewalls and endwalls infer average maximum mining-induced σ_1 magnitude $\approx 2.21 \times \sigma_V$ [1] (which is $2.21 \times 0.027 \times 260$mbs (~ mean stoping elevation)), say 20 MPa with minor mining-induced stress concentration. At this stage (and level of awareness of stress magnitudes) apply this stress to all stope surfaces.

Thus, $\sigma_C / \sigma_1 = 20 / 20 \approx 1.0$ for stope backs in TZ3 (hangingwall) @ 260m (average depth)

$\sigma_C / \sigma_1 = 70 / 20 \approx 3.5$ for walls in TZ4 (footwall)

$\Rightarrow$ **Factor A** = 0.10 for backs in TZ3 $\Rightarrow$ **Factor A** = 0.75 for walls in TZ4

Factor B

Backs

The critical defects to stope back stability will be pervasive bedding/ foliation/ shear partings sub-parallel to the back orientation.

Difference between dips of backs and local critical defects is ≤ 5°, hence B_{backs} is 0.3

Stope walls

The critical defects for sidewalls are those associated with lode-parallel / sub-parallel defects, with release able to be provided by various cross-cutting joints and sub-horizontal joints.

Difference between dip of cross-dip sidewall and critical defect ~ 70°, hence $B_{x\text{-dip sidewall}}$ is 0.85

Difference between dip of dip-parallel endwall and critical defect ~ 90°, hence $B_{dip\text{-para endwall}}$ is 1.00

Factor C

Backs

Critical defects are sub-horizontal joints, with release available from lode-parallel defects/ faults and cross-cutting joints; the possible failure mechanism is slabbing/ falling

Backs assumed ~ 25°dip lode-parallel, hence C_{backs} is 2.5

Sidewalls & Endwalls

Critical defects are lode-parallel defects/ faulting @ ~ 25° (mean) release from cross-cutting joints and sub-horizontal joints (± moderately steep outliers)

Walls assumed vertical, hence $C_{x\text{-dip endwall}} = C_{dip\text{-para sidewall}} = 8$

Stability Numbers & Hydraulic Radii

A generic experienced-based nomogram enables the Hydraulic Radius (HR = stope surface area ÷ stope surface perimeter) for a given stope surface to be identified, variously for unsupported and uniformly supported conditions.

Using a 1st quartile rating of 50 for sidewalls and 93 for backs (Tables 6.1 and 6.2 respectively) the following calculations provide N values, which in turn provide HR values:

Backs

$N_{backs} = 1.1 \times 0.1 \times 0.3 \times 2.5 \approx 0.1$ ⇒ Hydraulic Radius (HR) ≈ 1.4m unsupported
⇒ HR ≈ 3.3m to 5.0m supported (if supportable)
e.g., 20m × 10m (HR 3.m) to 20m × 20m

Sidewalls

$N_{x\text{-dip sidewall}} = 6.0 \times 0.75 \times 0.85 \times 8 = 30.6$ ⇒ HR ≈ 8.0m unsupported ⇒ 20m × 80m

$N_{x\text{-para endwall}} = 6.0 \times 0.75 \times 1.00 \times 8 = 36.0$ ⇒ HR ≈ 12.0m unsupported ⇒ 20m × unlimited

It is not practicably/ economically possible to provide support to exposed LHOS sidewalls.

Note that adherence to these (and all other listed) hydraulic radii does not guarantee stability, as the behaviour of rock masses is dictated by the unit/ feature under greatest stress (relative to capacity) rather than the calculated median/ mean/ typical rock mass condition.

The assessment of RAS ground conditions indicates *poor* to *good* rock mass quality, without full appreciation of the potential adverse influence of major structural defects. It is conceivable that a single structure or local interaction of multiple structures could disrupt an otherwise stable span (in the backs or hangingwall of a stope). Accordingly, it will be prudent to moderate stoping spans.

7.3 Stope Overbreak / Dilution

Assessment of dilution has used the empirical Dilution Design Graph method, devised by Pakalnis and Clark (1997) [15] for assessment of the degree of stope instability in terms of metres of slough or dilution. The method is based on the modified stability number N' and the HR (derivation outlined above) and presents the degree of stability as average metres of slough (ELOS) that can be expected to fail from a stope surface (Figure 40) and expected instability (Figure 41).

Plotting relevant, albeit roughly estimated, parameters for RAS stoping (marked in Figure 41) indicate potential for high levels of sloughing from stope backs but acceptably low overbreak from stope walls.

Inference drawn from observation of TZ3 cores obliges agreement with predicted potential for overbreak from extensive shallowly dipping to sub-horizontal stope backs. Inopportunely, the TZ3 schist and the immediately underlying even poorer quality/ weaker TGF (particularly the clay-rich gouge core of the TGF), identified as the weakest units at RAS, form the hangingwall to mineralisation over the entire deposit. As such there is a compelling reason to expose, rather than avoid, these units during stoping.

Additionally, it is pertinent to note the ratings applied to the TZ3 rock mass are identified as 'typical' (inferring mean or median), so half the rock mass will be more competent and less prone to sloughing whereas the other half will be of equally *poor* or *poorer* quality. A critical issue is that local loss of integrity in a stope back, whether due to poorer quality or structural influence, will, almost inevitably, extend laterally regardless of the quality of unconfined ground adjacent to the initial overbreak.

Comprehensive back reinforcement and support for any opening exposing the TGF and/ or TZ3 schist is thus critical; however, effective reinforcement may be difficult given the restricted access resulting from mining geometry and the limits of rock reinforcing mechanics.

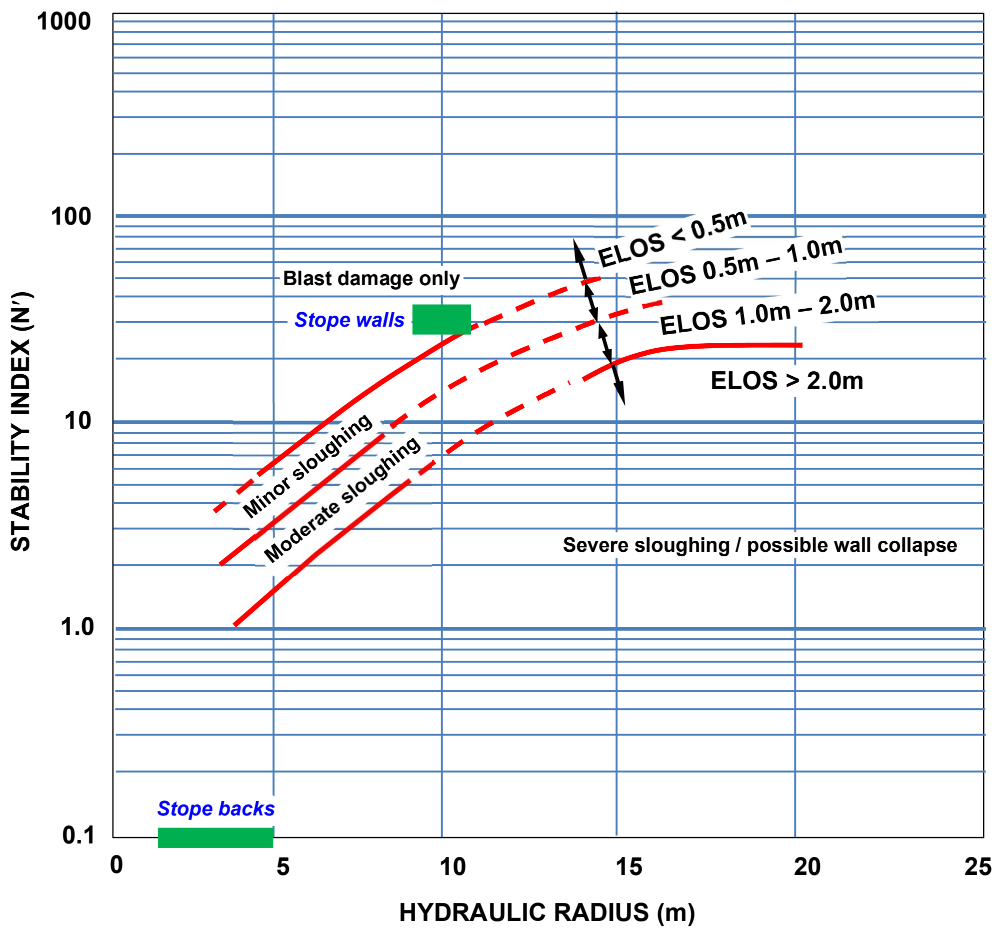


Figure 40 Estimation of overbreak / slough for non-supported hangingwalls/ footwalls (after Clark & Pakalnis, 1997)

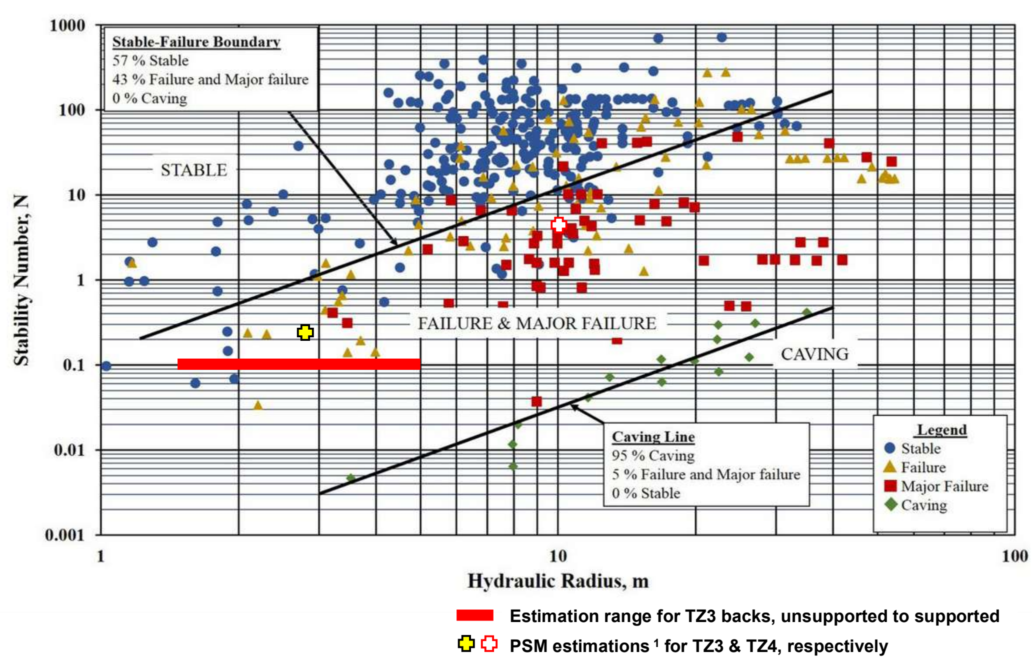


Figure 41 Extended Mathews Stability Graph (after Trueman & Mawdesley, 2003)

7.4 Pillars

Assessment of needs for pillars and pillar design have not been performed. Absence of specific data regarding ground condition and major structures and the relative complexity of the mining configuration means analysis results could be meaningless or misleading. Information to identify ground conditions can be gleaned from existing resource drilling data (but is beyond the remit of this review). Further investigation and interpretation are required to develop/ refine the structural model of the orebody and immediate country rocks.

Progressive placement of engineered (presumably cemented paste) backfill limits needs for pillars; however, it is anticipated that pillars will be necessary to divide the hangingwall span. The preliminary underground mining plan is divided into three (3) stoping panels separated by ~ 30m barrier pillars, Figure 42.

The LHOS (tall) panels are extensive at (notionally) ~ 115 × 135m; 120 × 155m; and 95 × 90m.

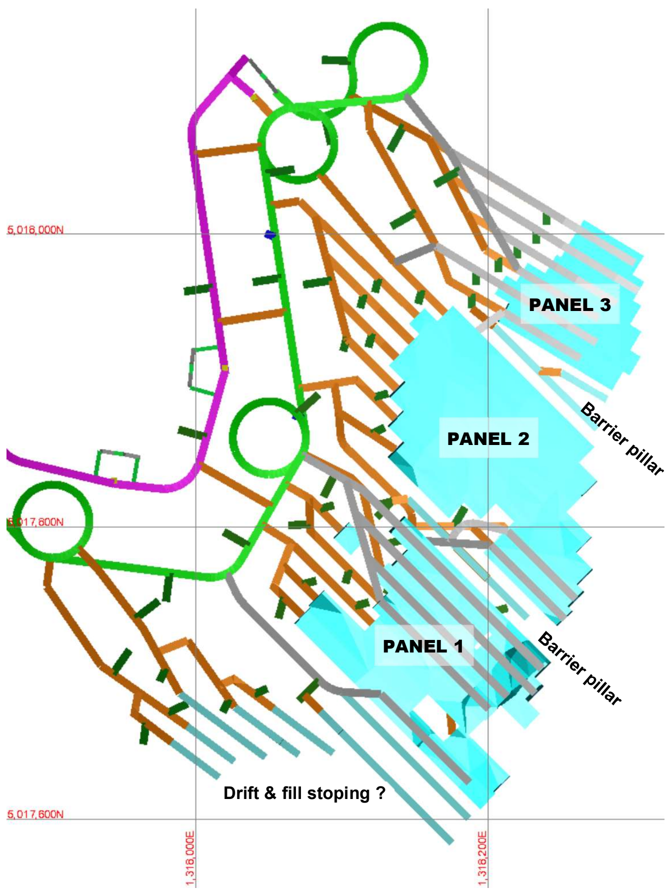


Figure 42 Conceptual stoping panel configuration

Stress magnitudes are moderate and concentrations from stoping-induced stress re-distribution should be mild and development of a low stress/ tensile relaxed zone above the stoped areas may be restricted; however, the capacity of the hangingwall rocks to arch over the stoped areas will limit sustainable (cumulative) spans.

The presence of faults intersecting stoping areas is acknowledged; however, the specific locations and characteristics of these structures/ zones are not known/ not well understood. The structural model of the area must be refined to inform more detailed assessment (three-dimensional numerical modelling) of potential influences on stoping and associated pillar support requirements.

7.5 Stope Backfilling

Engineered stope (cemented paste) backfill will be required to maximise ore extraction and provide a level of hangingwall support via restraint against overbreak and limitation of hangingwall relaxation.

It is inferred Matakanui has engaged or will engage suitably qualified and experienced mine fill engineers to assess the suitability of and strengths attainable from available processing tails and waste rock and the possible means of transporting/ reticulating and placing a preferred backfill medium.

Fresh cemented paste backfills flow (under gravity or by pumping) have a modest beach angle so tight backfilling can be achieved where stope geometry is favourable and suitable reticulation/ delivery can be arranged. Matakanui indicates that stope sequencing, paste delivery points and stope filling sequences have been planned to maximise tight filling (intimate contact between the paste and stope backs).

Paste fill is incapable of transmitting meaningful loads until very high strains have developed which has influence on hangingwall relaxation/ dilation even where tight filling is achieved. Likely strains need to be examined via physical properties testing of representative paste fill samples and numerical modelling given potential implications/ limitations on the stability of cumulative hangingwall spans.

7.6 Exposing the TGF / TZ3 Hangingwall

Inherently *poor* (to locally *very poor*) quality of the rock mass at the hangingwall of the stoping zone; the TGF and immediately overlying base of the TZ3 schist ± fractured ground at the top of the TZ4 complicate recovery of mineralisation at/ near the lode hangingwall.

Mineralisation lies in the RSSZ directly beneath the TGF, hence stope backs mined to the ‘contact’ expose a rock mass of low competence, so potentials for excessive dilution and initiation of caving must be considered. Scope to prevent/ curtail overbreak, once initiated is limited.

Reinforcement of TGF and TZ3 will be problematic in that effective load transfer from *poor* quality rock to reinforcement and from reinforcement to more competent rock (if such exists within reach) cannot be guaranteed and potential for unravelling between bolts would be probable (at least locally).

Accordingly, initial exposure of the TGF-affected zone must be at small scale and slow pace to gradually expose and reinforce/ pre-reinforce stope backs.

Stoping methods and sequencing strategies must aim to prevent/ limit overbreak/ dilution. Three basic approaches are available for stoping to the hangingwall (TGF or base of TZ3):

1. Exposing and supporting the hangingwall as the initial stoping stage, forming stable backs prior to stoping underlying ore.
2. Developing top access in TZ3 above/ intersecting the TGF, supporting backs in TZ3 with acceptance of increased ‘planned’ dilution.
3. Perform initial stoping beneath a temporary sill pillar, with subsequent blind stoping to the hangingwall operating on/ from the top of backfill in the previously mined underlying stopes.

Option 1 could incorporate construction of an artificial sill pillar by mining and tightly backfilling a contiguous series of drifts which mine to or mine out the core of the TGF. The backs of each drift would be intensely reinforced with full-column grouted rockbolts and backs and wall supported floor to floor with fibre-reinforced shotcrete (FRS). A drift and fill development sequence is shown schematically in Figure 43. A 1m thick higher strength (high-percentage cement content) slab would be formed on the floor of each drift. These higher strength slabs would form the backs to underlying LHOS which would be mined blind to the slabs.

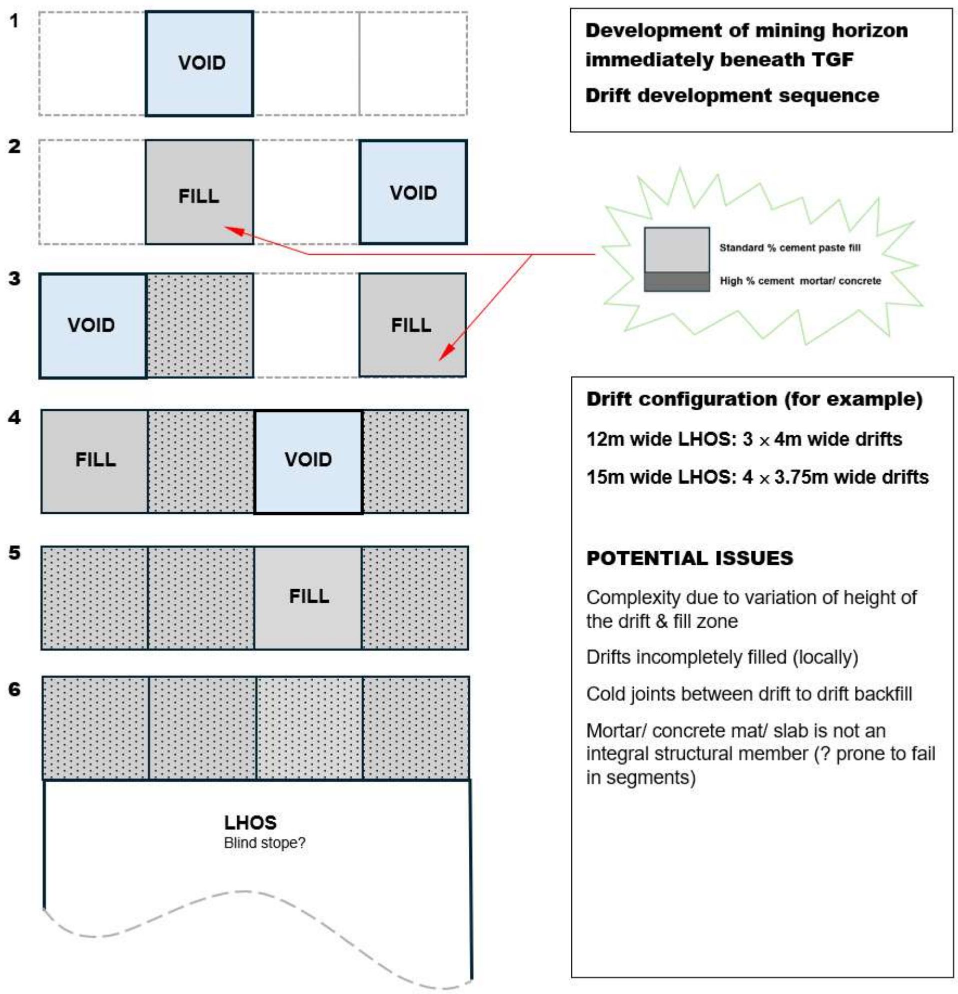


Figure 43 Option 1: Drift & fill mining / artificial sill pillar construction sequence

A significant disadvantage of the approach is the need for intensive and likely slow development (high number of drifts) in difficult ground. Potential difficulties with this approach relate to the dip of the lode and variation in height of the lode and drift and fill zones, achieving true tight fill, possible cold joints between slabs (and backfill generally), and that the slabs do not form an integral structure as LHOS backs would typically span multiple drifts.

Option 2 is based on developing stope top access at least partially in waste rock (into the TZ3 zone) and effectively supporting the backs with intensive reinforcement and back and wall support (FRS).

Disadvantages of the approach include the requirement to mine waste and the difficulty associated with effectively reinforcing drive backs and walls in the poor quality TZ3. Reinforcing the backs of a 12m or 15m wide LHOS will be difficult to unachievable where access is available from only a single 5m or 5.5m wide top sill drive.

Figure 44 illustrates the zones able to be reinforced effectively (and less effectively) and those inferred unable to be reinforced effectively. In some very poor TZ3 zones it is possible that the theoretically reinforceable zones cannot be reinforced effectively and would be prone for unravelling to occur around the reinforcing elements.

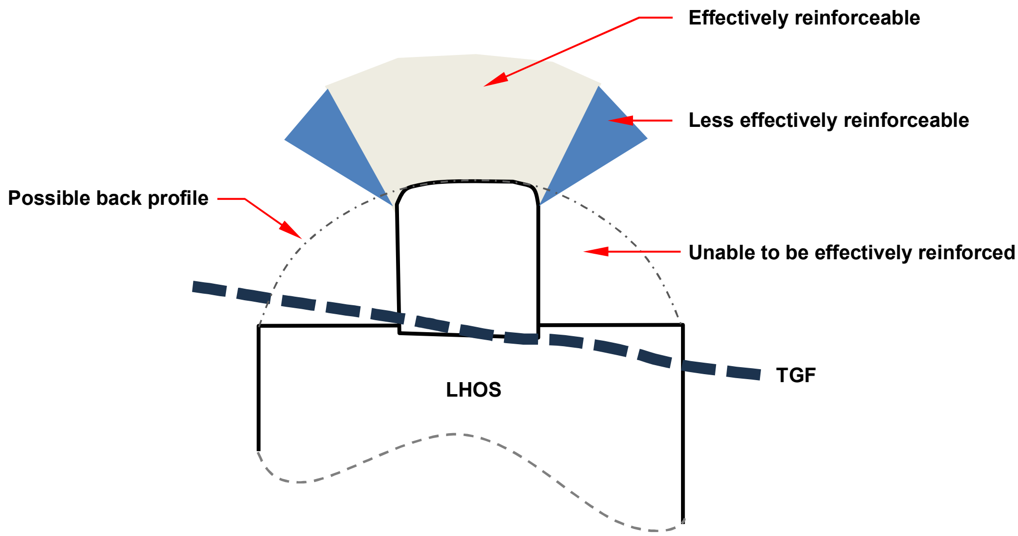


Figure 44 Option 2: Top sill above TGF

Option 3 involves delaying exposure of the hangingwall. In zones where ore abuts the TGF core, mine and backfill all primary (P) and secondary (S) LHOS beneath a 'substantial' (stable) sill/ crown.

Backfill LHOS as tightly as is practicable and conduct tertiary/ remnant mining using a front caving method, retreating down dip. When recovering the sill pillar, drill/ haulage drives are developed through the base of the pillar, operating on backfill in LHOS (to avoid exposure to backs of LHOS).

Drives need to be closely spaced to effectively cave the upper portion of the sill pillar.

Figure 45 is schematic illustration for a northerly-looking section through ~ 5,017,500N; however, it would be possible to arrange mining either transversely or longitudinally, dependent on local lode geometry and the preferred backfilling sequence to maximise tight fill placement.

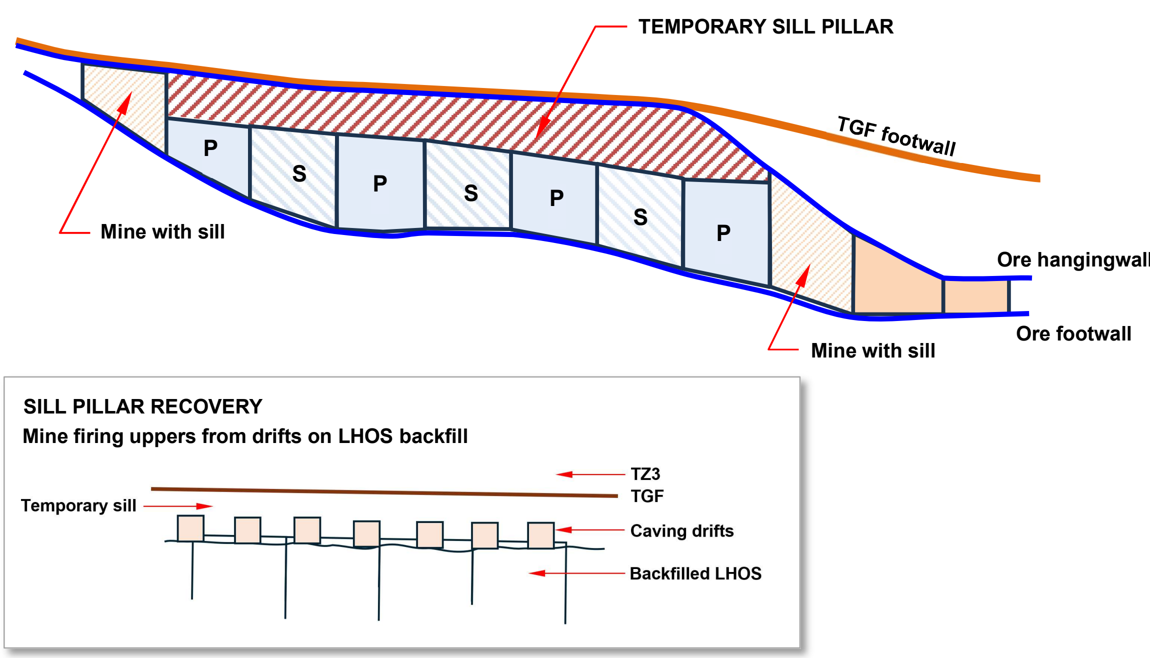


Figure 45 Option 3: Temporary sill pillar & cave sequence

Disadvantages of the method are extensive development for the stoping return; potential for high dilution during caving; and that the caving process may make the hangingwall more susceptible to subsidence. It is inferred, however, that cave overbreak would choke-off at or beneath a point ~ 20m above the sill.

7.7 Mining-Induced Subsidence

Underground mining generates potential for caving through the TGF and into TZ3 schists. The potential exists regardless of stoping and sequencing strategies (though to varying degrees).

The extent to which caving, and in turn subsidence, can develop is a function of the size of the undercut area(s), the void available above backfill and the volume of bulked debris required to choke that void and arrest caving.

Potential for surface subsidence is a function of depth of mining beneath surface; the total footprint and geometry of the mined out area(s); pillar sizes and locations; pillar strengths and the potential for caving (as noted above).

Division of the mining area into a series of stoping panels separated by ≥ 30m wide regional/ barrier pillars is expected to be necessary to negate or at least limit the potential for subsidence.

Required pillar sizes will depend on the quality/ competence of the ore zone in which the pillars will be developed, of the hangingwall rock mass and the rock mass providing the foundation to the pillars.

Selection of pillar locations will need to be guided by assessed local hangingwall competence, identified structural conditions and opening geometry and ore grade and ore distribution.

It is inferred that further assessment will be performed in advance of underground mining. In that respect, further information on the abovementioned factors is required.

Initial indications regarding subsidence potential would be provided by recommended 3D modelling; however, more detailed fault modelling and refined underground mining plans will be required as input to specific subsidence assessment. Future modelling will also benefit from improved rock mass strength data, particularly for TZ3 rocks and rock mass, expected to be obtained from monitoring open pit wall stability performance and associated back-analysis.

8. Further Geotechnical Investigation

This assessment has been based on information derived from data obtained from exploration and geotechnical cores.

Ongoing geotechnical assessment of actual conditions, observed wall stability performance and mining experience will be required to refine geological and geotechnical models and hence pit and underground design parameters.

The amount of information available varies across the deposit. In most cases, there are needs for further pre-mining investigation to confirm/ refine or adjust the inferences/ assumptions made to date. A program of additional drilling was discussed with Matakanui and is being implemented.

8.1 OP Mining Assessment

The primary requirement for pre-mining assessment of open pit mining and revision/ refinement of wall design parameters is for three-dimensional numerical modelling of proposed open pit and underground mining. As noted, results from two-dimensional modelling are not strictly representative of actual situations and can therefore be misleading.

Operationally, ongoing geotechnical assessment of actual conditions, observed wall stability performance and mining experience will be required to re-assess/ refine geological and geotechnical models, confirm the applicability of wall design parameters and/ or derive appropriate local design adjustments, including identification of possible opportunity for reduction of waste removal requirements or improved ore recovery.

8.2 UG Mining Assessment

More detailed geotechnical assessment of existing and additional data is required to review and revise preliminary recommendations for underground access development and production mining, particularly in relation to details of access development and support schemes in LHOS and other stoping areas.

The same methods applied in the preliminary assessment reported herein would be used, though to increased detail and with greater coverage of the planned access and production areas. Data sources need to provide uniform coverage across all proposed decline/ access routes and stoping panels.

Specific assessment(s) may be required to identify means of development and needs for support in capital vertical development for mine ventilation.

Numerical modelling of proposed mining is recommended, the results from which are seen to be a highly important in identifying appropriate extraction sequences (and possible limits to extraction) and potential problematic areas/ aspects of the mining plan. These analyses can be coupled with the open pit modelling recommended in Section 7.1.

Should underground mining extend more than 500 metres below surface, measurement of *in situ* rock stress at RAS would be required and is a critical input to proposed numerical modelling.

The viability of establishing and maintaining a suitable backfilling system or systems must be examined and the implications of the opportunities/ implications/ limitations applicable to geotechnical aspects must be considered to confirm the viability of the proposed stoping system and sequences.

Dedicated investigations will be required to identify ground conditions influencing development for ventilation, whether via vertical shafts to surface or to in-pit collars, or for horizontal development into the open pit. Dedicated cored drilling will be required for assessment of vertical development (expected to be raisebored). Detailed structural/ geotechnical mapping is likely to be adequate in assessment of drive break-throughs to the pit or adit development from within the pit.

While this report has considered stope backfilling only briefly, requirements and limitations of backfilling must be thoroughly examined from logistics and reticulation/ placement viewpoints as well as confirming the viability and capacity of the medium/ media to be used.

8.3 Hydrogeology & Groundwater Assessment & Monitoring

The presence of groundwater pressures within pit walls is inevitably a destabilising influence. The buoyant effects generated by hydrostatic pressures will exacerbate the potential for all possible failure mechanisms. It is crucial, therefore, that steps are taken to identify and monitor hydrogeological conditions as mining extends below the groundwater level and, where necessary, derive a program of dewatering/ depressurisation.

Monitoring groundwater levels will be crucial to assessing the effectiveness of depressurisation measures and will require piezometers to be constructed at the pit crest, and progressively on selected berms as the pit is developed.

Even with dewatering/ depressurisation measures in place it remains possible that transient water pressures could develop within wall rocks due to infiltration of rainfall run-off. Appropriate management of surface flows is thus also important to slope stability.

8.4 Operational Issues

8.4.1 Stability Monitoring

Slope stability monitoring is required to confirm that stable wall conditions are maintained and identify any potential for changes in conditions that could indicate potential onset of large scale instability. Acceptable results from ongoing assessment of the monitoring data are required to justify continued access.

Provided pit walls are being monitored appropriately signs of destabilisation will be detected in advance of development of collapse. To enable detection of onset of wall destabilisation, should such develop, use of qualitative visual and quantitative electro-optical distance measurement (EDM) stability monitoring methods are recommended for assessment of pit wall slope stability conditions.

In the first instance electro-optical distance measurement (EDM) survey methods, preferably automated with real-time data acquisition, should be adopted to measure point displacements on all walls. Arrays of prisms must be established on all walls as they are developed. Prisms should initially be spaced at ≤ 50m intervals at 15m vertical intervals in weathered and transitional ground, staggering prism positions on alternate (horizontal) rows.

Adjustments to prism locations will be needed to adequately monitor expected local variations in displacement around geological structures and across major cracks. Additional prisms may be required locally. Where non-automated surveying is used the frequency of prism surveys after identification of movement trends immediately following installation can be based on measured displacement rates but should not be less frequently than weekly.

LiDAR scanning can be used for timelapse comparison of wall positions and is also highly useful in compliance checks of as-built slopes against designed profiles.

Frequent visual inspection of the pit walls, including walking over all safely accessible berms, must be regarded as an integral aspect of pit mining. Observations should be recorded in a written log, and regularly updated photographic record can aid in qualitative assessment. The need or otherwise for further action (more intensive monitoring) and/ or design adjustment will be dependent on the results obtained from the proposed monitoring.

Underground, monitoring of opening geometry and stability will be required in a number of areas:

- It is assumed that cavity monitoring survey (CMS) methods will be used to reconcile as-mined against as-designed stope shapes. Findings from stope performance assessments should be used to assist/ enable development of a site-specific nomogram for refinement of Modified Stability Graph assessment of achievable stope dimensions.
- Use of borehole extensometers may be warranted to follow stope sidewall displacement/ deformation.
- Instrumentation to confirm absence of or detect surface subsidence/ settlement will be required.

  The approach to subsidence monitoring should be based variously on precise levelling of the overlying surface area; InSAR detection of surface settlement (intermittent use of satellite-acquired data); time-domain reflectometry (TDR) attrition meters to detect fractures in the rock mass above stoping areas; and multi-point borehole extensometers to measure strains within the hangingwall rock mass.

8.4.2 Ground Control Management Plans

Formal Ground Control Management Plans (GCMP) must be developed for proposed open pit and underground mining at RAS. The GCMPs will need to either be a part of or be specifically referenced by the RAS Principal Mining Hazard Management Plan.

The GCMPs describe the ground conditions encountered and/or anticipated in proposed open pit walls and underground development and describe/ justify the mine design parameters in use (or proposed). It would identify expected open pit and underground stability conditions, possible mechanisms of instability and the means by which these would/ could be precluded or avoided to permit safe development and production.

The physical and management procedures to be used to ensure appropriate mine design and use of safe mining practices would also be described.

8.5 Open Pit Abandonment Barriers

No assessment of very long term open pit slope deterioration and destabilisation or of requirements for abandonment have been made in this review

8.6 Geotechnical Review

Regular geotechnical review of ground conditions during operations is recommended.

For open pits initial review should be conducted relatively early in the life of mining, say, once mining has reached a depth of ~ 30m. The timing of subsequent reviews would depend on the findings of the initial review, current mining area and future mining proposals, and/ or according to assessment of actual conditions by Matakanui mining personnel.

For underground operation initial review will be required for assessment of the hillside cut for decline portal development. Ongoing review will be required for decline portal development and continued decline, level access and ore drive development.

Specific review and assessment are expected to be necessary for vertical development (ventilation shaft design and development).

9. Closure

We trust that the information provided in this report is adequate for your current requirements.

We stress the need for further investigation and analysis for feasibility assessment, detailed mine design and ongoing geotechnical assessment and review during mining once operations are established.

Please contact this office if there is any need for clarification or further information.

PETER O'BRYAN *& Associates*

per:

Peter O'Bryan

BE (Mining) MEngSc MAusIMM (CP)

Principal

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Clause 71: Principal hazard management plans for ground or strata instability

Institute of Geological & Nuclear Sciences (GNS), various
General geological background information

11. Attachment

Seismic Disturbance & Slope Stability Analysis

Excerpts from: Read & Stacey 2009 (eds)
Guidelines for Open Pit Slope Design
CSIRO Publishing, CRC
ISBN 9780415874410

3.5 Regional seismicity

3.5.1 Distribution of earthquakes

There are a number of instances where earthquakes have triggered landslides in natural slopes, but there are no positively identified instances of earthquakes triggering slope failures in large open pit mines. This situation has created considerable debate about the need to perform seismic analyses for open pit slopes and is the main reason why seismic loading is often ignored in open pit slope design. However, if a property was located in a seismically active region and a large earthquake were to occur near a mined slope the effects could be significant, particularly if weak soil-like materials are involved.

Additionally, other mine infrastructure, especially tailings dams, may be and have been affected by earthquakes. For this reason, documenting the regional seismicity and integrating it with the geological mode is important.

Most earthquakes are caused by interactions between two crustal plates and are concentrated in narrow geographic belts defined by the plate boundaries.

There are four basic types of plate boundaries:

Divergent plate boundaries such as the mid-Atlantic ridge or the Great African Rift Valley exhibit narrow zones of shallow earthquakes along normal faults.

Transform boundaries such as the San Andreas Fault in California exhibit shallow earthquakes caused by lateral strike-slip movement along the fault.

Convergent boundaries where continents collide, such as in the Himalayas, exhibit broad zones of shallow earthquakes associated with complex fault systems along the line of collision.

Subductive convergent boundaries, such as in the Andes where the oceanic Nazca plate is sinking beneath the continental South American plate, exhibit shallow, intermediate and deep-focus earthquakes caused by tension, underthrusting and compression.

3.5.2 Seismic risk data

Earthquakes create four types of ground motion:

1. Ground motion that can trigger landslides or similar surface movements, which may destroy structures by simply removing their foundations;
2. Sudden fault displacements that may occur at the ground surface and disrupt structures such as roads and bridges;
3. Ground motion that results in soil and subsoil consolidation or settling, which may damage structures through excessive foundation deformation;
4. Ground accelerations that may induce inertial forces in a structure sufficient to damage it.

The first two effects are static effects. The third effect may be static or dynamic and the fourth is dynamic.

For risk assessment purposes, the following data should be included in the model.

The locations and magnitudes of all historic and recent earthquakes in the region of interest. The US Geological Survey (USGS) is the most common source of these data.

- The occurrence history of the earthquakes, with magnitude plotted against the number of events
- The magnitude, return period, peak ground acceleration and distance from the project site of the maximum credible earthquake and the maximum earthquake likely to occur during the project lifetime. This information should be supplemented by the likelihood of each of these events and their corresponding peak ground accelerations being exceeded during the project life.
- Ground acceleration curves for the maximum credible and the project earthquakes, from which velocity and displacement curves can be obtained.

10.3.3.5 Seismic analysis

There is considerable debate about the need for seismic analyses for open pit slopes. There are few, if any, recorded instances in which earthquakes have been shown to produce significant slope instabilities in hard rock conditions, a statement supported by evidence from a number of mines in highly active seismic zones. These include the Bougainville and Ok Tedi mines in Papua New Guinea, and a number of mines in Chile and Peru.

Earthquakes have produced small shallow slides and rockfalls in open pits but none on a scale sufficient to disrupt mining operations. The Bougainville mine, which is located within 60 km of the Pacific plate subduction zone and experiences a level of seismic activity three times that of California, was subjected to the design earthquake (magnitude 7.6) in July 1975. Although there were a number of spectacular tailings liquefaction failures, small face failures on the waste dumps and some overburden failures on ridge crests around the pit, there were no incidents on the pit slopes.

Because of the high level of seismicity in the Bougainville Island region, in 1981 Bougainville Copper Ltd commissioned a consultant to collate the record of open pit behaviour under earthquake loading in Chile, selected because of its mining history and its tectonic similarity to Bougainville. The west coast of Chile is underlain by the Peru-Chile subduction zone. The basic conclusion of the report (Hoek & Soto 1981) was that the Chilean open pit mines exhibited a high degree of stability, with the 1969 East Wall (in pit crusher) slide at the Chuquicamata mine noted as a possible exception (still debated). At that time the Chuquicamata mine slopes were about 550 m high and had successfully withstood an earthquake of magnitude 7.0 at a hypocentral distance of 30 km, which produced a maximum horizontal acceleration of the order of 0.25 g. At the Penoso iron ore mine near Vallenar, which is situated near the coast midway between Santiago and Antofagasta, slopes up to 320 m high had withstood an earthquake of magnitude 7.0 at a hypocentral distance of 20 km and a ground acceleration of 0.31 g.

Most recently (14 November 2007), the northern regions of Chile experienced a magnitude (Mw) 7.7 earthquake. Although there was considerable damage to property and loss of life, rock slopes at the many open pit mines in the region were not damaged by the main earthquake or two major (Mw 6.8 and Mw 6.2) events that occurred the next day. It was noted that slope monitoring prisms and piezometers reacted to the earthquake, but no incidents of slope instability were reported.

These incidents contrast experiences with natural slopes, where earthquakes have produced numerous landslides, large and small. The process responsible for earthquake induced landslides in natural rock slopes is generally considered to be topographic amplification, which is an increase in shaking associated with ridges and topographic changes. It is believed to be a function of slope geometry and seismic wavelength resulting from at least two interacting processes: focusing and de-focusing of seismic waves from the free surfaces of hills and canyons; and excitation of whole topographic edifices, which occurs when the wavelength of the incoming seismic wave is similar to the width of the topographic feature (Murphy 2008).

Ashford and Sitar (1997) noted that maximum topographic amplification, which may result in a two to five-fold increase in the peak ground acceleration (Faccioli et al. 2002), occurs when the wave propagates downwards into the slope. Meunier et al. (2008) demonstrated that S waves are significantly more important to topographic amplification than P waves and Ashford et al. (1997) noted that the effect of topographic amplification decreases significantly within even one slope height distance behind the slope crest. Given these descriptors, it does not seem unreasonable to suggest that topographic amplification and consequent slope failure does not occur in large open pit slopes because the slopes are outside the range of geometries that would experience topographic amplification or, if amplification does occur, the slope geometries are such that the amplification is too weak to promote slope failure. Another possibility, of course, is that the rock mass is simply too strong. Whichever is true, in some jurisdictions it may be that executive management and/or the regulatory authorities will raise the familiar maxim ‘absence of evidence is not necessarily evidence of absence’ and declare that an analysis is necessary.

If it is decided that earthquake effects should be considered, designers can use limit equilibrium models that consider seismic loading pseudo-statically by specifying a horizontal static acceleration, similar to gravity, which is meant to be representative of the design earthquake. There are at least three important assumptions inherent in the pseudo-static approach:

- Earthquakes can be modelled as a static force acting on the mass of a potential slide;
- No dynamic water pressures are generated;
- Materials show no significant loss of strength as a result of cyclic loading.

Selection of the design earthquake is usually left to experts in seismology. Choice of an appropriate horizontal acceleration (or seismic coefficient) is the main difficulty with the pseudo-static approach. It is an approach that has no physical basis but relies on a fictitious parameter (the seismic coefficient) which cannot be derived using logical or physical principles.

There is no simple, universally accepted way to determine an appropriate seismic coefficient since earthquakes involve different durations and frequency contents. In almost all cases the horizontal acceleration is less than or equal to half the maximum acceleration of the design earthquake (Pyke 1997). The horizontal acceleration acts to produce an inertial force out of the slope, therefore the determination of the safety factor using the limit equilibrium method proceeds as usual. Since earthquakes are not static, the analysis with constant horizontal acceleration is usually considered to be conservative. If the seismic coefficients in Figure 10.24 are used, resultant FOS greater than 1.0 (Pyke 1997) to 1.15 (Seed 1979) usually indicate that seismic displacements will be acceptably small.

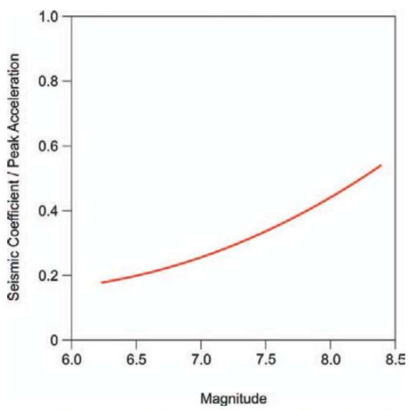

Figure 10.24: Recommendations for seismic coefficient based on earthquake magnitude and peak acceleration
Source: Pyke (1997)

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Note: Figures and references to Figures, (except Figure 10.24) have been omitted from these excerpts.