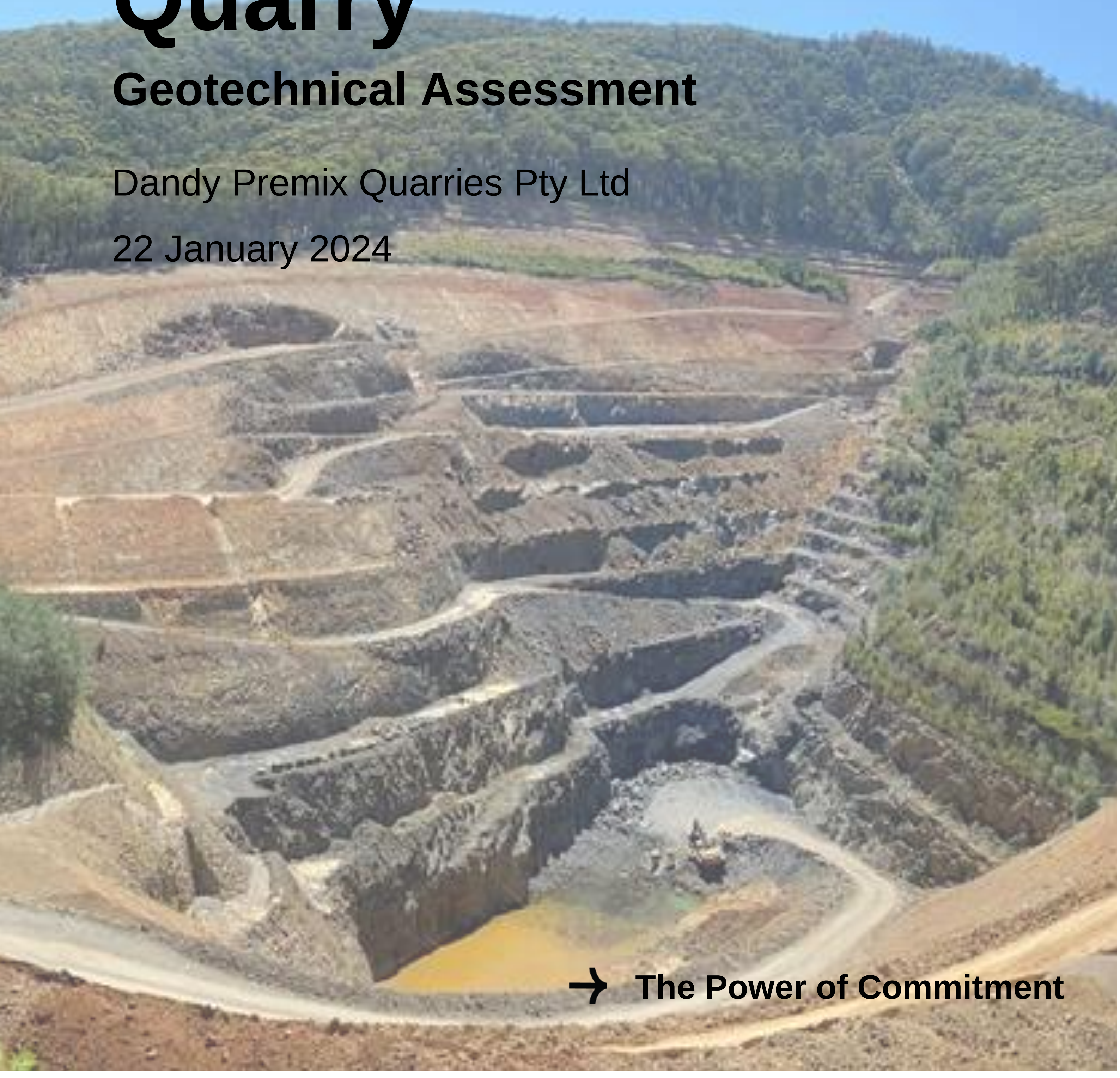
Woori Yallock Quarry

Geotechnical Assessment

Dandy Premix Quarries Pty Ltd

22 January 2024



The Power of Commitment

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1. Introduction

1.1 General

GHD has been requested by Mr Garry Cranny of Dandy Premix Concrete Pty Ltd (the Client) to undertake a geotechnical assessment for the proposed development of the Woori Yallock Quarry, Work Authority 375 (WA375), located in Launching Place, Victoria. It is understood that the Client is preparing a Work Plan Variation (WPV) to the Earth Resources Regulation (ERR) division of the Department of Energy, Environment and Climate Action (DEECA) to expand their current quarrying operations. The WPV proposes to extend their current quarrying operations by expanding the current WA boundary into 'Lot 50c'. The proposed expansion area is depicted in Figure 1.

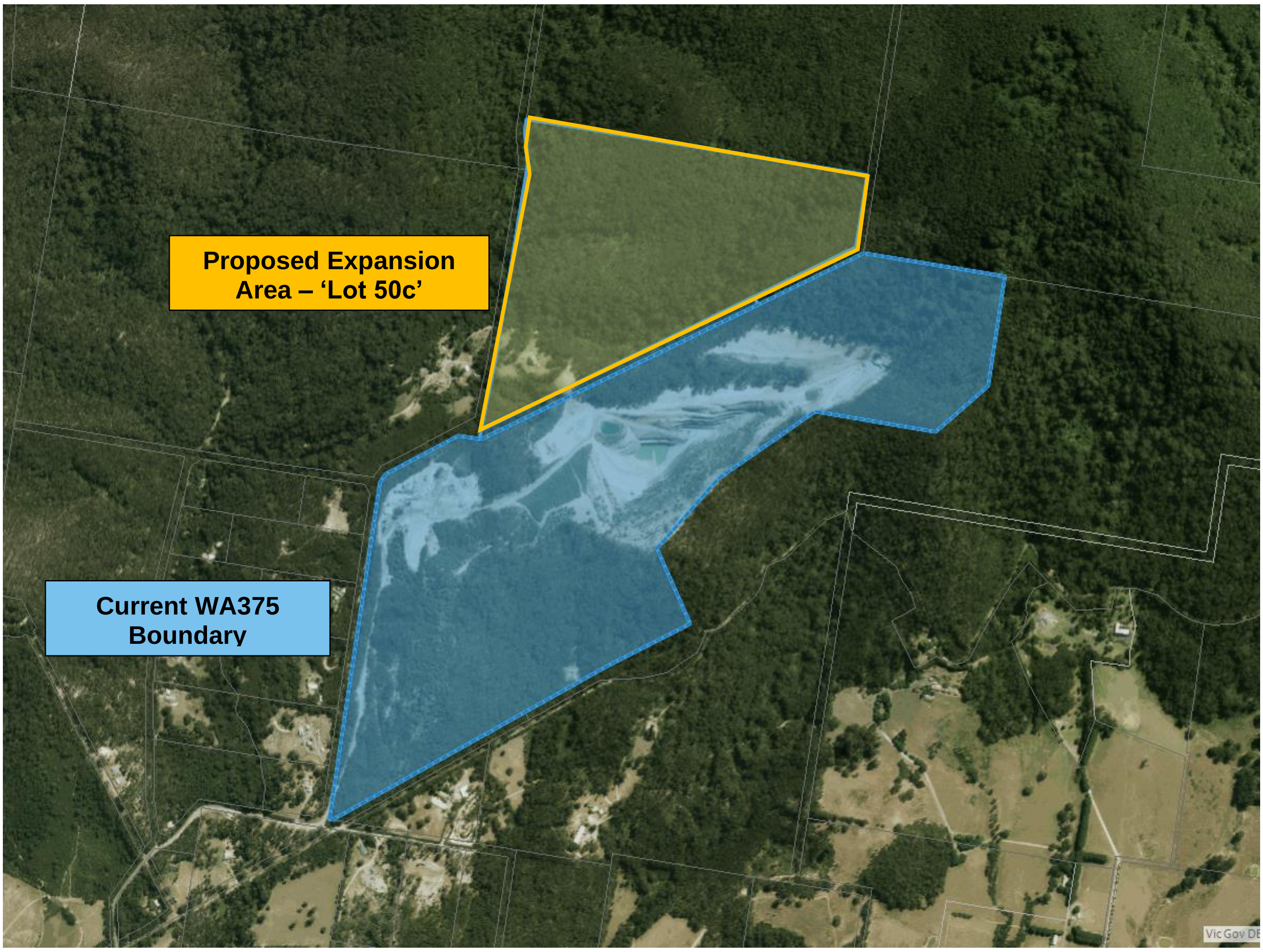


Figure 1 Plan View of Woori Yallock Quarry Depicting Proposed Expansion Area

1.2 Client Objectives

GHD understands that the Client's primary objective is to obtain WPV approval from ERR to proceed with further development of the Woori Yallock site. The WPV conditions and geotechnical requirements that are anticipated to be required for approval include:

– Development of shear strength parameters appropriate for the final depth of the pit.
– Long-term stability performance of terminal and rehabilitated slopes considering the full depth development as well as:
  - The influence of major and minor geological structures on batter stability.
  - The influence of groundwater on batter stability.

1.3 Scope of Work

As outlined in the GHD proposal document titled '*Woori Yallock Quarry – Work Authority WA375 – Proposal for Geotechnical Assessment' dated 04 February 2021* (GHD Reference no.: 12545408-52564-1), which was accepted by the Client on 13 February 2022, the agreed scope of work is as follows:

1.3.1 Site Inspection

GHD will undertake a site inspection at WA375 to:

– Visually assess the geotechnical conditions of the existing quarry batter and stability performance.
– Undertake high level mapping of major defects including any faults that may intersect the site and those observed to be exposed at the time of the inspection.
– Gain a visual appreciation of key operational procedures and potential hazards at the site.

1.3.2 Geotechnical Assessment

Following completion of the site inspection component of the geotechnical assessment, GHD would:

– Develop a site geological and geotechnical model based on the results of the desktop review, site inspection and the available resource definition drilling information.
– Undertake slope stability assessments using Client supplied batter geometry profiles for the proposed development at the site.
  - Limit-equilibrium modelling (LEM) analyses would be undertaken to calculate factors of safety to assess the stability of the proposed design batter geometries (for long term stability) where required.
  - Kinematic assessment of the mapped geological structures (i.e., from field mapping).
  - Undertake sensitivity assessments for seismic, elevated phreatic conditions and weathered material strengths.
– Based on the outcomes of the stability analyses, with regards to the proposed quarry design, undertake a geotechnical risk assessment which would identify, where necessary, suitable risk treatment protocols.
– GHD would undertake preliminary erodibility assessments, based on the proposed rehabilitation concepts including the slope design, using the revised universal soil loss equation (RUSLE). The findings would assist the Client in understanding potential long-term average annual soil loss volumes. Application of the RUSLE equation considers the following factors:
  - Rainfall erosivity
  - Soil erodibility
  - Topography
  - Cropping management factors
– Prepare a geotechnical assessment report for WA375 outlining the findings and recommendations, which can be subsequently submitted to the ERR as part of the Client's WPV submission. The geotechnical assessment report would include:
  - A summary of the methodology.
  - A summary of the site observations.
  - Limit equilibrium stability analysis results.
  - Kinematic assessment from the field mapping.
  - Soil erodibility assessment results.
  - An assessment on the long-term stability of the proposed rehabilitation.
  - Recommendations on the safe and stable batter profiles / geometries within the overburden (if any) and resource units.
  - Outline of recommendations as applicable for any requirements in relation to slope / batter movement monitoring during profiling works to the proposed design.

- Risk Assessment Matrix with controls outlined.

1.4 Limitations

This report has been prepared by GHD for Dandy Premix Quarries Pty Ltd and may only be used and relied on by Dandy Premix Quarries Pty Ltd for the purpose agreed between GHD and Dandy Premix Quarries Pty Ltd as outlined in this report.

GHD otherwise disclaims responsibility to any person other than Dandy Premix Quarries Pty Ltd arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the bases of information provided by Dandy Premix Quarries Pty Ltd and others who provided information to GHD (including government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.

1.5 Information Relied Upon

We have relied upon the following sources of information for the conducted analyses, detailed in this report. Relevant information was extracted from the following documents:

– Bell, Cochrane & Associates (BCA) Extractive Industries – Drilling Data Summary
  - Portable Document Format (.pdf) of drilling logs titled '*WA375_DrillDataSummary*', dated 14 July 2010
  - Excel Spreadsheet (.xls) of drilling logs titled '*WA375_RawDrilLData_0322.xls',* dated 22 March 2022

– BCA (2009) report titled '*Work Plan for Extractive Industry Work Authority No.375 – Woori Yallock Quarry',* dated 15 December 2009, (BCA Reference No. D10-1)

– GHD (2007) report titled, '*Report on Woori Yallock Quarry – Geotechnical Assessment',* dated 16 October 2007 (GHD Reference No. 31/21728/139978, Rev 0)

– GHD (2009) report titled '*Report on YVQ Launching Place – Geotechnical Review of Proposed Quarry Extension (Design 2009-10)'*, dated 10 August 2009 (GHD Reference No. 31/24592/168663, Rev 0)

– GHD (2021) letter report titled '*Yarra Valley Quarry Desktop Review and Gap Analyses',* dated 8 January 2021 (GHD Reference No. 12535505-10052-1, Rev 0)

– Groundwater monitoring summary provided in excel spreadsheet format titled '*Summary of GWater Bores Info_28 Feb 2022.xls'*, dated 28 February 2021

– Groundwater borehole plan titled, ‘*WA375-YVQ_GWater Boreholes Survey A3 Plan_22 Feb 22.pdf*’ dated 22 February 2022
– John Leonard Consulting Services Pty Ltd (JLCS), report titled ‘*Hydrogeological Assessment – Proposed Extension to Yara Valley Quarries Hard Rock Quarry – McMahon Road, Launching Place’*, dated July 2009 (JLCS Reference No. GW-09/005)
– BCA WA 375 terminal pit design strings titled ‘*WA375_FinalPit_Sep2022.DWG*’, dated September 2022
  - BCA WA 375 SITE LAYOUT PLAN (Rev E), dated 16 Jun 2023
  - BCA WA 375 REHABILITATED LANDFORM PLAN (Rev A), dated 8 June 2023.

2. Background

2.1 General

The Woori Yallock Quarry, WA375 (the site) is located in Yarra Valley region, Victoria, approximately 53 km east north-east of Melbourne City and approximately 5 km to the north north-east of the Woori Yallock township. The location of the site is presented in plan view in Figure 2.

Figure 2 Approximate Site Location

The Woori Yallock Quarry site, originally owned and operated by Hornsfeld Resources Pty Ltd, has been operational since the mid-1980's. Circa 2007, the Woori Yallock Quarry was acquired by Dandy Premix Quarries Pty Ltd (t/a Yarra Valley Quarries Pty Ltd), which are the current owner-operators. The primary resource being extracted at the site is hornfels, which is suited to the production of high-quality crushed rock, concrete, asphalt and sealing aggregates, and are primarily sold to private customers, Local Government and Government authorities at rate of approximately 250,000 t/yr. Resource definition drilling campaigns have indicated a reserve of 45 Mt of quality hornfels plus up to several million tonnes (approximately 5 Mt) of low quality saleable material, dependent on the market need, giving a total of 50 Mt of saleable material.

The currently approved WA 375 is approximately 90 Ha, with operations occurring along the north wall and progresses in a northerly direction. Extraction of the resource is undertaken using conventional earth moving practices with clearing, grubbing, and stripping of softer 'diggable' units using digger and dozer operations, and conventional drill and blast followed by digger and truck operations in the harder 'blastable' units. Extracted material is transported on site via haul trucks to the onsite fixed crushing and screening plant. Depicted in Figure 3 is the typical working face treatment for the Woori Yallock Quarry site.

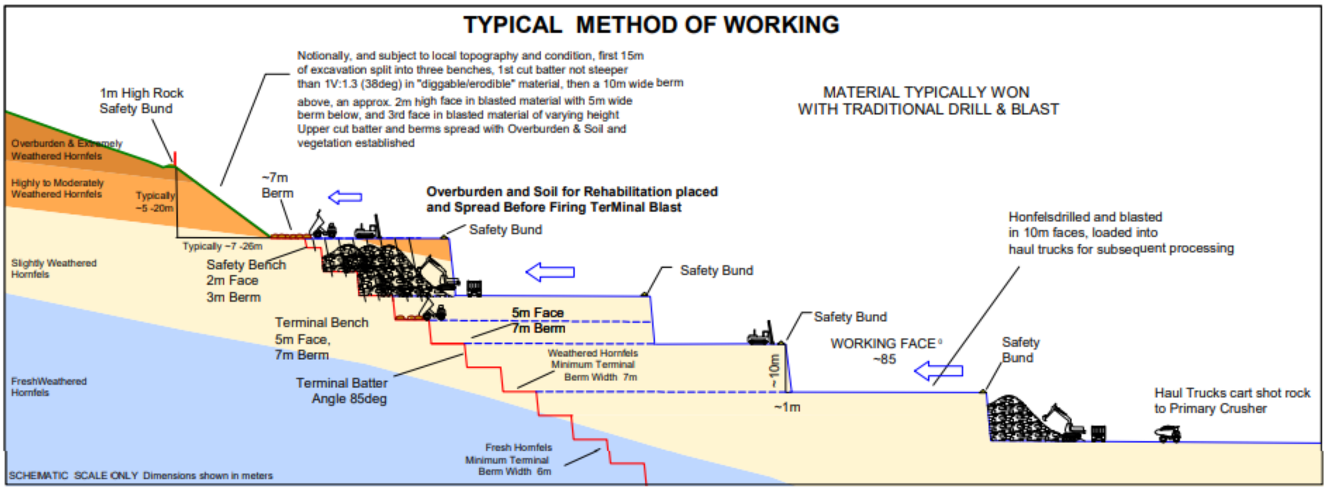


Figure 3 *Schematic Depicting Typical Working Face Treatment, after BCA (2023)*

Contour maps show that the current extraction footprint is situated in a valley, north of an east-west ridge, which intersects a small perennial drainage course. The natural surface of the quarry area generally slopes north-east to south-west with an approximate 48 m (~5°) cross-fall.

Surrounding the WA375 boundary to the north is the Mount Toolebewong State Forest. To the east is the Yarra Ranges National Park. The Oshannessy Aqueduct Trail can be found to the south of WA 347, along with privately owned property on the other side of the trail. The western boundary of WA 347 is bounded by McMahons Road, with Ure Creek and more private properties located on the opposite side of the road.

Receptors within 1 km of the site is presented in Figure 4.

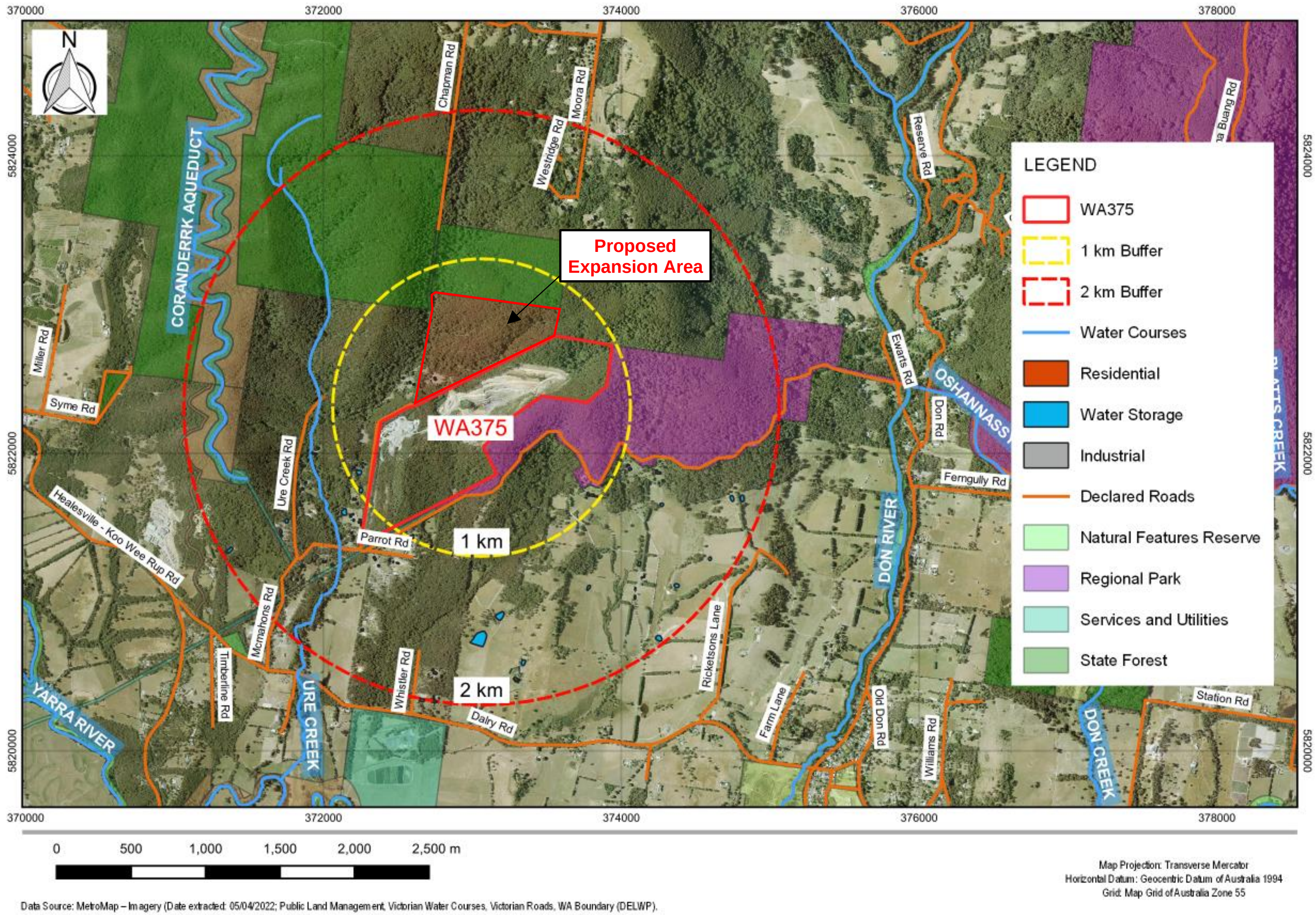


Figure 4 *Plan View of the Woori Yallock Quarry (WA375) Site Depicting Nearby Receptors.*

2.2 Regional and Local Geology

2.2.1 Regional Setting and Geological History

WA375 is located within the Melbourne Structural Zone, the easternmost zone of the Whitelaw Terrane of the Lachlan Fold Belt (Figure 5) (Willman, 2002). The Melbourne Zone is a complexly deformed tectonic zone which has undergone multiple deformation events, resulting in a series of large-scale north-south trending structures combined with the intrusion and extrusion of igneous rock units (VandenBerg et al., 2000).

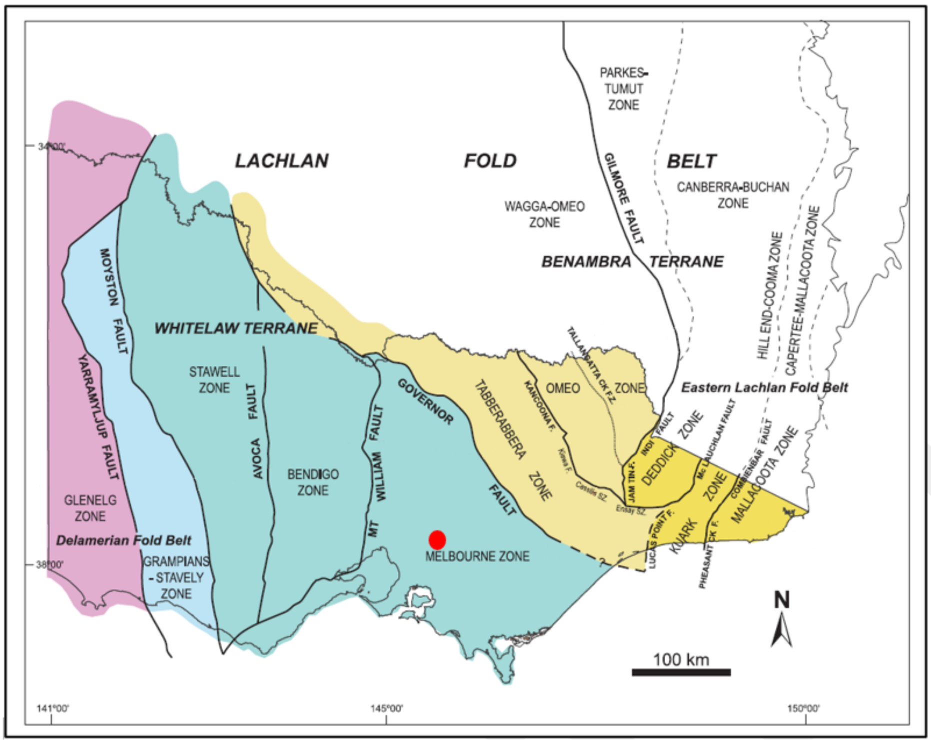


Figure 5 The zone and terrane subdivisions of the Lachlan Fold Belt (VandenBerg et al., 2000), and the approximate location of the Woori Yallock Quarry (red)

The origins of the Melbourne Zone geology commenced in the late-Cambrian to the Ordovician (495-455 Ma), with the deposition of deep marine turbidites in an oceanic setting between the then-Australian continental margin and the offshore Selwyn Block (VandenBerg et al., 2000). In the late-Ordovician to the early-Silurian (455-430 Ma), the onset of the Western Lachlan Orogen commenced, caused by the east-west convergence between the Selwyn Block and the Australian continental margin. In the early-Silurian through to the Devonian (430-410 Ma), the Murrindindi Supergroup (including the Whitelaw Siltstone (*Sjw*)) was deposited in a submarine foreland basin setting (Willman, 2002).

Convergence of the Selwyn Block continued into the early-Devonian (410-395 Ma), with the Melbourne Zone transitioning into a shallow marine facies leading to the subsequent deposition of shoreface sediments (VandenBerg, 2000).

The middle-Devonian (385 Ma) marked the Tabberabberan Orogeny, signifying the accretion of the Melbourne Zone and the Selwyn Block onto the Australian continental margin. Continued east-west compression of the Melbourne Zone resulted in predominately north-south trending structures (VandenBerg, 2000).

Following the Tabberabberan Orogeny, in the late-Devonian, post-tectonic granitic intrusions occurred in the Melbourne Zone and the Bendigo Zone to the west. During this period, the Toole-be-wong Granodiorite (*G226*) was emplaced (Vandenberg *et al*., 2000). This event resulted in the contact metamorphism of the intruded marine sediments surrounding the granite body, producing a metamorphic aureole, within which WA375 sits. Typically, the outer aureole consisted of a spotted hornfels, while the inner aureole is defined as a coarse-grained cordierite hornfels (VandenBerg *et al*., 2000). By the end of the Devonian, the various crustal blocks and associated geological units were more or less in their present day positions with relative to each other. This series of deformation events commencing in the Cambrian and progressing all the way to the middle-Devonian is summarised as a schematic diagram in Figure 6.

Immediately to the northeast of WA375 lies the Acheron Cauldron, one of three main components of the Marysville Igneous Complex (VandenBerg *et al*., 2000), which are a series of subaerial caldera volcanics.

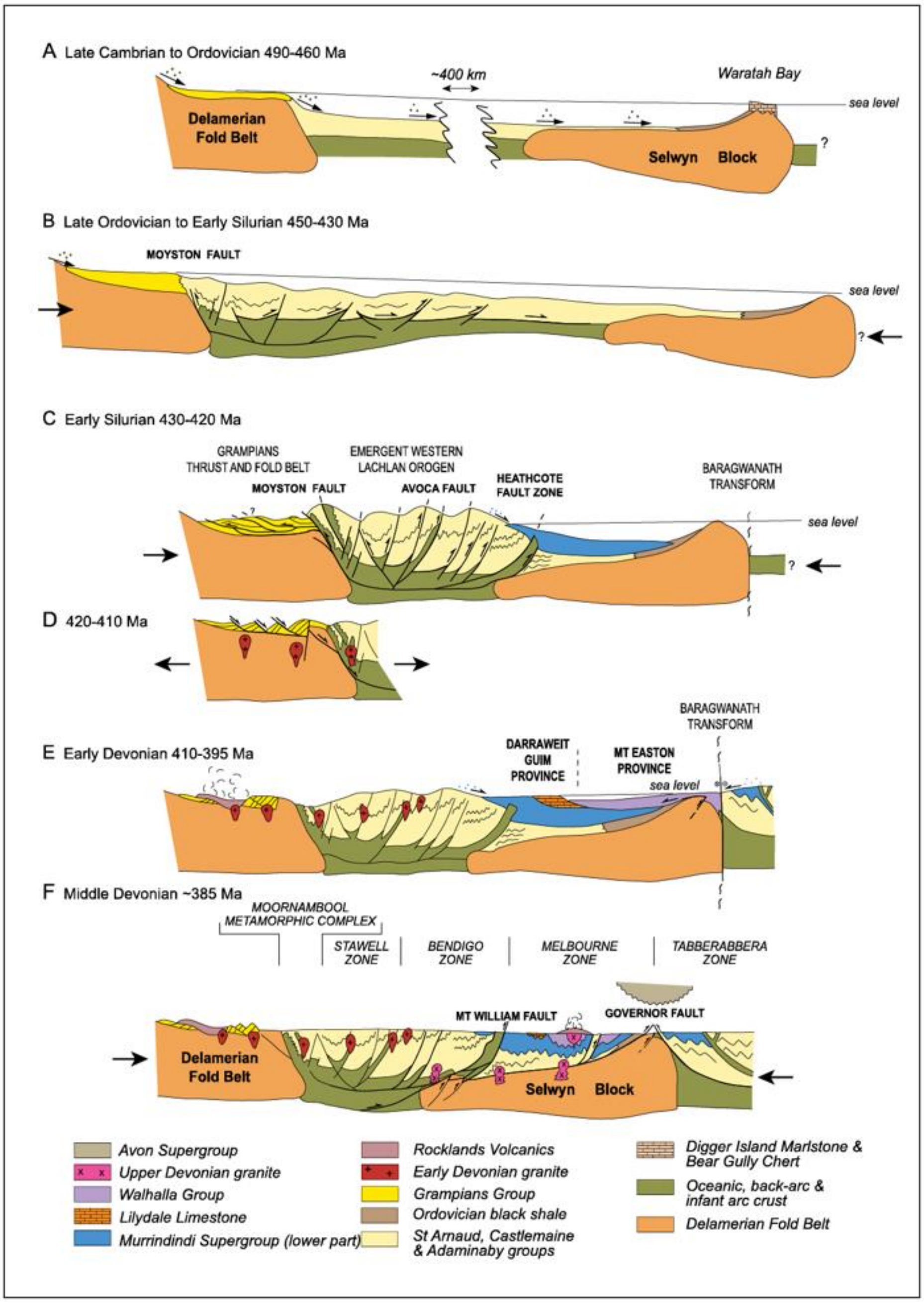


Figure 6 ***A series of schematic diagrams Depicting the Cambrian-middle-Devonian evolution of the Lachlan Fold Belt in Western Victoria (VandenBerg et al., 2000)***

Unconformably overlying the sediments of the Melbourne Zone in the Woori Yallock area is the Monbulk Volcanic Group (Nuo) which was deposited around 22 Ma (McKenzie *et al*., 1984). This is in turn overlain by unconsolidated Neogene-aged colluvial sediments.

2.2.2 Outcrop Geology

The outcropping geology in the vicinity of the Woori Yallock Quarry consists predominately of Humevale Siltstone (*Dxh*) and overlain by Neogene-aged incised colluvium (*Nc1*) towards the west of the quarry. While the GeoVic database (2014) indicates that the Humevale Siltstone in the area is characterised by a series of laminated brown siltstones, with minor very fine to fine grained sandstone laminae, site observations indicate that this area has been subject to contact metamorphism caused by the intrusion of the Toole-be-wong Granodiorite (*G226*), resulting in the rock within the contact aureole being altered to hornfels.

Other units within the area but outcropping outside of the WA375 boundary include the Melbourne Formation (*Sxm*), the Taggerty Subgroup (*Dyt*) and the Donna Buang Rhyodacite (*Dyad*). Figure 7 displays a simplified geological map of the Woori Yallock region.

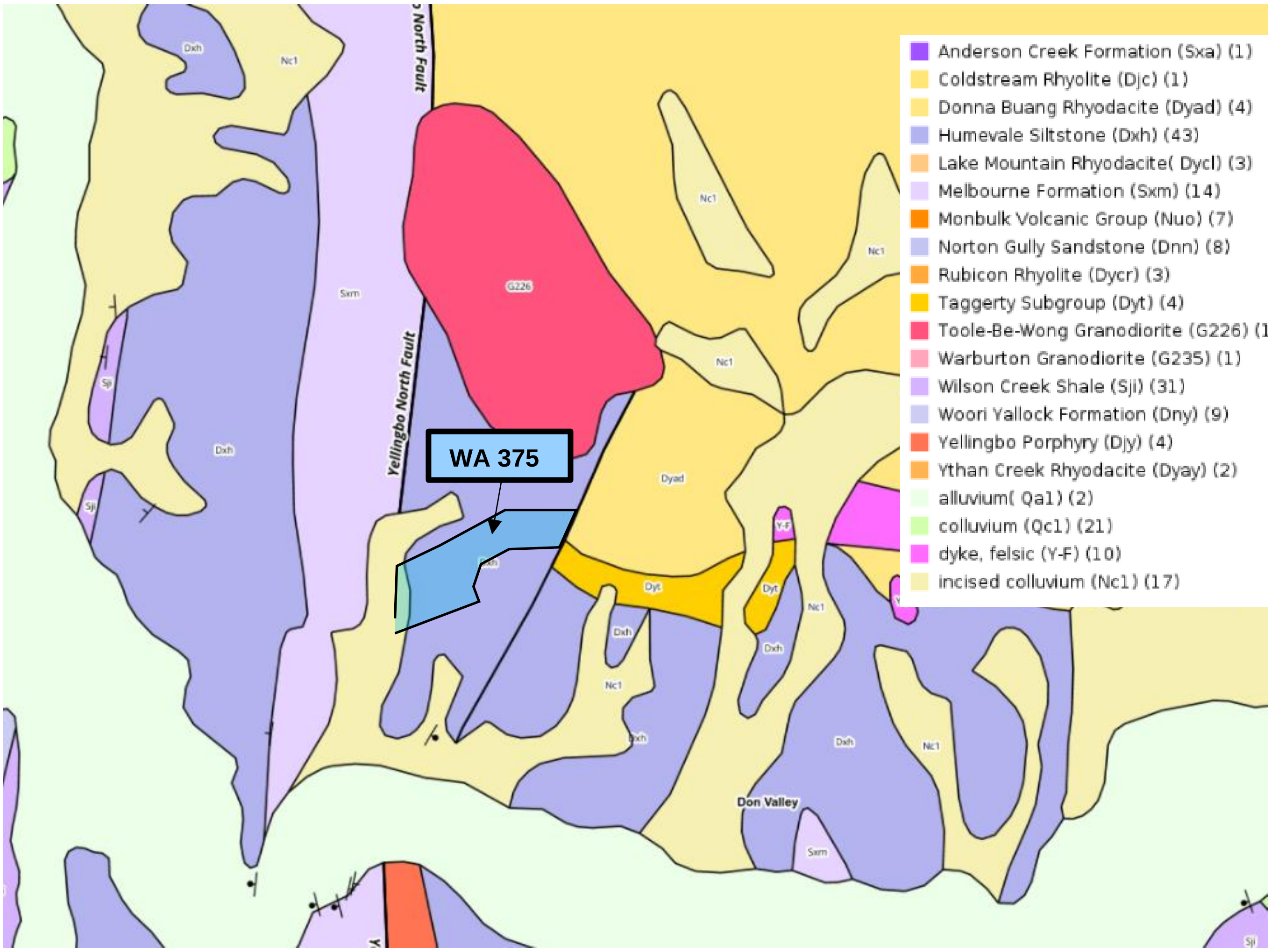


Figure 7 Simplified Geological Map of the Woori Yallock Quarry Site

2.2.3 Site Stratigraphy

The area in the vicinity of the of the Woori Yallock Quarry consists of six main stratigraphic units (youngest to oldest), which are summarised in Table 1.

Table 1 Stratigraphic Sequence of Lithological Units

Neogene		Incised Colluvium (Nc1)	Sedimentary, non-marine; gravel sand and silt. Coarse fraction generally poorly graded and sub-angular
Devonian		Toole-be-wong Granodiorite (G226)	Igneous, intrusive (granite S-type)
		Donna Buang Rhyodacite (Dyad)	Igneous, extrusive; biotite -hypersthene rhyodacite ignimbrite, recrystalised
		Taggerty Subgroup (Dyt)	Igneous, extrusive; fluvial: felsic ignimbrites, basalt and andesite lavas, conglomerate, sandstone
		Humevale Siltstone (Dxh)	Sedimentary; marine siltstone, minor sandstone
Silurian		Melbourne Formation (Sxm)	Sedimentary; marine sandstone, mudstone, medium to thin bedded

Melbourne Formation (*Sxm*)

The oldest rocks found within the Woori Yallock Quarry is the Melbourne Formation. The Melbourne Formation is defined by Welch *et al*. (2011) as a series of thin-bedded siltstones and sandstones with plane parallel and ripple drift cross-lamination, with bioturbation and occasional hummocky cross lamination indicative of reworking (VandenBerg *et al*., 2000).

Humevale Siltstone (*Dxh*)

The Humevale Siltstone is the main extractive resources within WA375. This unit is a sequence of marine mudstones, with minor sandstone and marlstone. Overall, the Humevale Siltstone is a thick unit, reaching a thickness of up to 3800 m (Sandiford, 2004), however lithologies within the siltstone are generally thin-bedded with mostly continuous lamination. Fossil evidence within this unit indicate a depositional age of around Silurian to early-Devonian (Earp, 2015).

Taggerty Subgroup (*Dyt*)

The Taggerty Subgroup is a series of deposits representing the pre-collapse phase of the Marysville Igneous Province and consist of volcanics (ignimbrites, andesites and basalts), volcanogenic sandstone, siltstones and conglomerates. The Taggerty Subgroup outcrops around the igneous province in an irregular and discontinuous manner and does not outcrop within the WA375 footprint.

Donna Buang Rhyodacite (*Dyad*)

The Donna Buang Rhyodacite is a unit deposited during the collapse phase of the southern Marysville Igneous Province known as the Acheron Cauldron. This unit is up to 1000 m thick and is remarkably uniform in both petrography and chemistry, with a slight coarsening of groundmass towards the top of the unit (VandenBerg *et al*., 2000). This unit appears light to dark grey, with phenocrysts of plagioclase, biotite, enstatite, rare quartz, and K-feldspar, and formed as a single cooling unit.

The Donna Buang Rhyodacite does not appear to outcrop within the WA375 footprint, however GeoVic (2014) indicates that this unit may be present at the far eastern end of the WA boundary.

Toole-be-wong Granodiorite (*G226*)

In the late-Devonian, following the deposition of the Humevale Siltstone, the Toole-be-wong Granodiorite was emplaced into the surrounding country rock. This intrusion was just one of many shallow-crustal intrusions of batholiths and plutons of granodioritic and granitic compositions within Victoria during the late-Devonian.

The Toole-be-wong Granodiorite is a sub-equigranular medium grained S-type biotite granodiorite, with abundant xenoliths. Potassium-Argon (K-Ar) dating of the Toole-be-wong Granodiorite conducted by Richards & Singleton (1981) yielded an age of 371±13 Ma.

During the emplacement process of the Toole-be-wong Granodiorite, the extreme temperatures generated by the body of magma is likely to result in the contact metamorphism of the surrounding country rock. While not mapped by GeoVic (2014), metamorphism of the Humevale Siltstone (to hornfels) was observed on site, placing the WA375 footprint within the metamorphic aureole of the Toole-be-wong Granodiorite.

The Toole-be-wong Granite does not outcrop within the Woori Yallock Quarry footprint.

Neogene Colluvium (*Nc1*)

The youngest unit found within the Woori Yallock Quarry area is a series of Neogene-aged incised colluvial deposits, consisting of generally unconsolidated silt, sand and gravel. Sediments within this unit are generally sub-angular and poorly sorted and dissected to variable degrees.

2.2.4 Major Structures

As identified on the map in Figure 7 the Yellingbo Fault located to the north-west of the current WA375 boundary strikes approximately north-south. Currently the Yellingbo North Fault is not exposed along excavated pit walls, however, a number of fault and shear structures, which may be splays associated with the Yellingbo North Fault, have been identified as part of the GHD (2007) and GHD (2009) inspections, which are discussed further in the next section. Located to the east of the WA 375 boundary is a contact zone, however, no significant alteration zones associated with the contact have been observed along excavated pit walls. During the GHD (2007) site inspection a granitic dyke was observed in the southwestern area of the quarry.

2.2.5 Minor Structures

Defect mapping of geological structures was undertaken during site inspection by GHD (2007) '*Report on Woori Yallock Quarry – Geotechnical Assessment'* dated 16 October 2007 (GHD Reference No. 31/21728/139978, Rev 0), and GHD (2009) report titled '*Report on YQV Launching Place – Geotechnical Review of Proposed Quarry Extension (Design 2009-10'*, dated 10 August 2009 (GHD Reference No. 31/24592/168663, Rev 0). Across the two site inspections a total of 187 discontinuities were mapped. Summarised in the Table 2 are the primary defect characteristics obtained from the two site inspections and presented in Figure 8 is the corresponding stereographic projection.

Table 2 Summary of Defect Characteristics, after GHD (2007) and GHD (2009)

Discontinuity Set	Mean Dip/Dip Direction (Mine Grid)	Description
D1	88°/125°	Bedding
D2	80°/083°	Bedding
D3	36°/030°	Major structural set
D4	30°/231°	Major structural set
D5	63°/318°	Bedding

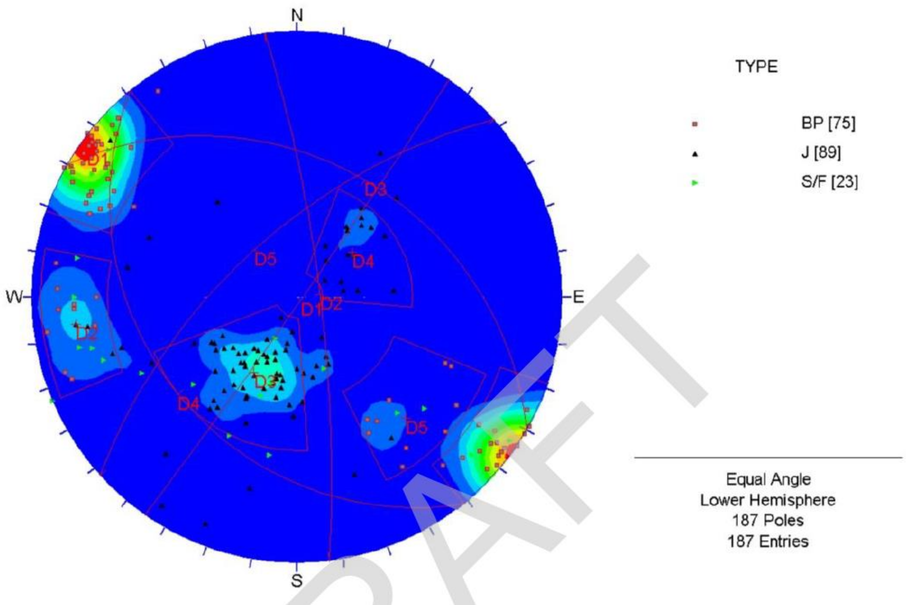


Figure 8 *Stereographic Projection of Identified Defect Sets after GHD (2007) and GHD (2009)*

Further discussion on the minor structures obtained from the GHD (2007) and GHD (2009) site inspections is provided in Section 2.6.

2.3 Hydrogeology

A hydrogeological assessment of the Woori Yallock Quarry site was undertaken by John Leonard Consulting Services Pty Ltd (JLCS), report titled '*Hydrogeological Assessment – Proposed Extension to Yara Valley Quarries Hard Rock Quarry – McMahon Road, Launching Place'*, dated July 2009 (JLCS Reference No. GW-09/005). In May 2023, an update to this assessment was undertaken by JLCS, with the outcomes of the assessment summarised in the report titled '*Hydrogeological Assessment, Proposed Extension, Yarra Valley Quarries Hard Rock Quarry, McMahon Road, Launching Place'*, dated May 2023 (JLCS Reference No. GW-25/002).

At the time of the JLCS (2009) report, groundwater levels within the extraction footprint were inferred from regional bore locations, and the groundwater level was interpreted to occur at an approximate Reduced Level (RL) of 198 m relative to the Australian Height Datum (AHD). More recently a series of groundwater bores have been installed around the Woori Yallock Quarry site to establish a groundwater network for monitoring purposes. A total of three (3) new bores were constructed by Matthew & Sons Drilling Services Pty Ltd early 2022, and a fourth registered stock and domestic stock (State Bore ID No.: 66222) form the current network. Summarised in Table 3 are the bore survey details, with the locations of the groundwater observation bores shown in Figure 9. The groundwater monitoring results are summarised in Table 4 below.

Table 3 *Summary of Groundwater Observation Bores*

Groundwater Bore ID	WMIS Bore Identifier	Collar RL (m)	Easting	Northing
A (GW1)	WRK130817	237.83	372704.87	5822492.33
B (GW2)	WRK130816	248.69	372892.74	5822418.70
GB (GW3)	WRK130818	229.00	372423.03	5822095.83
GW (GW4)	66222	199.77	372268.20	5822171.78

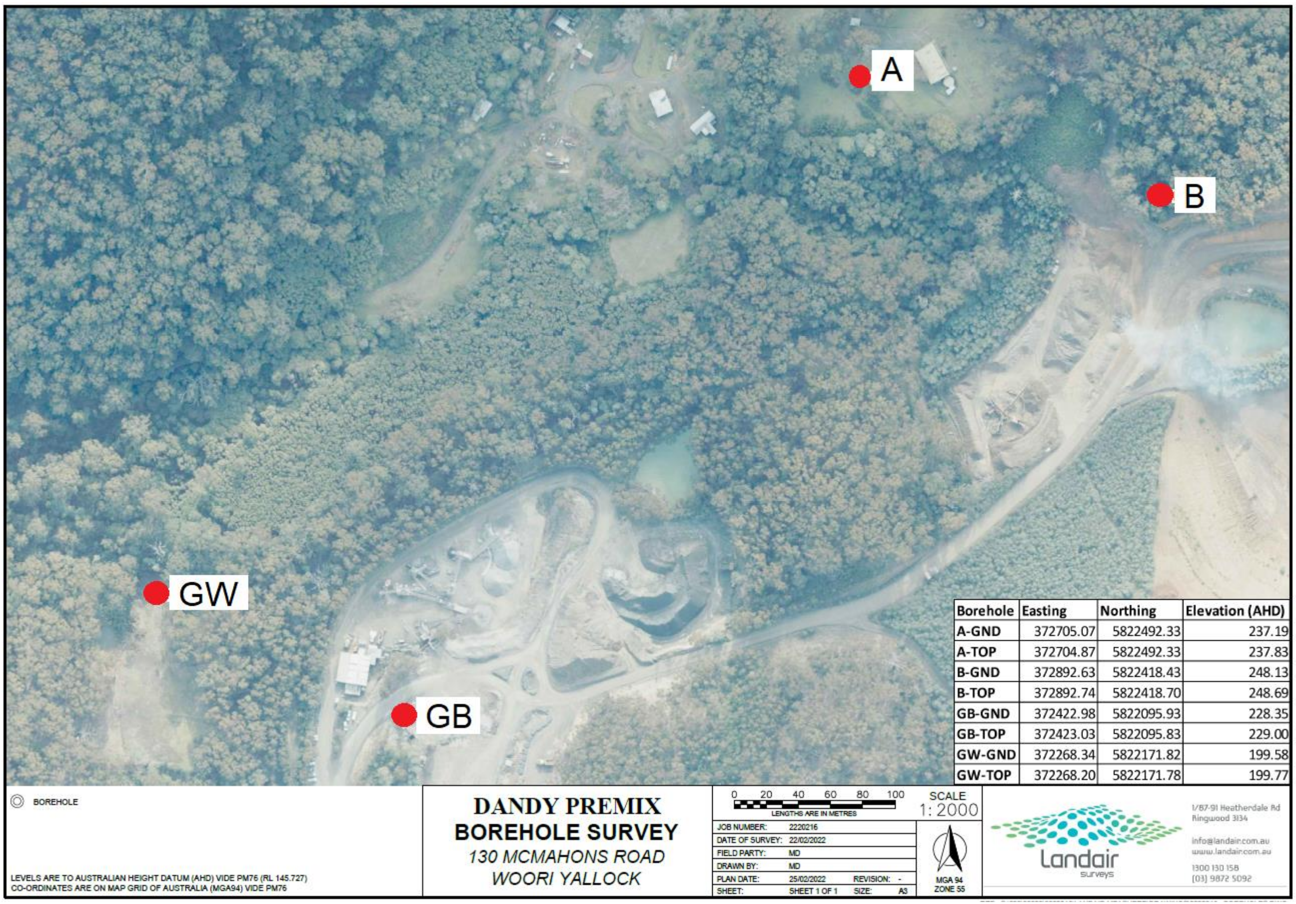


Figure 9 *Approximate Location of Groundwater Observation Bores at WA375, after Landair (2022)*

Table 4 *Groundwater Monitoring of Observation Bores*

Groundwater Bore ID	Water Level Elevation (m AHD)			
	18 Feb 2022	16 Jun 2022	26 Sept 2022	12 Dec 2022
A (GW1)	217.52	216.63	218.96	220.37
B (GW2)	224.52	226.86	229.93	229.76
GB (GW3)	198.61	198.60	198.89	199.65
GW (GW4)	185.92	186.00	186.64	187.18

The measured groundwater data indicates that the groundwater profile generally slopes from the east-north-east to the west-south-west. Figure 10 below depicts the overall groundwater flow lines modelled by JLCS with respect to the July 2022 pit and the proposed Stage 4 (final) pit respectively. It is noted that Figure 10 does not depict the drawdown / cone of depression into the pit, and only provides an indication of the far field groundwater profiles.

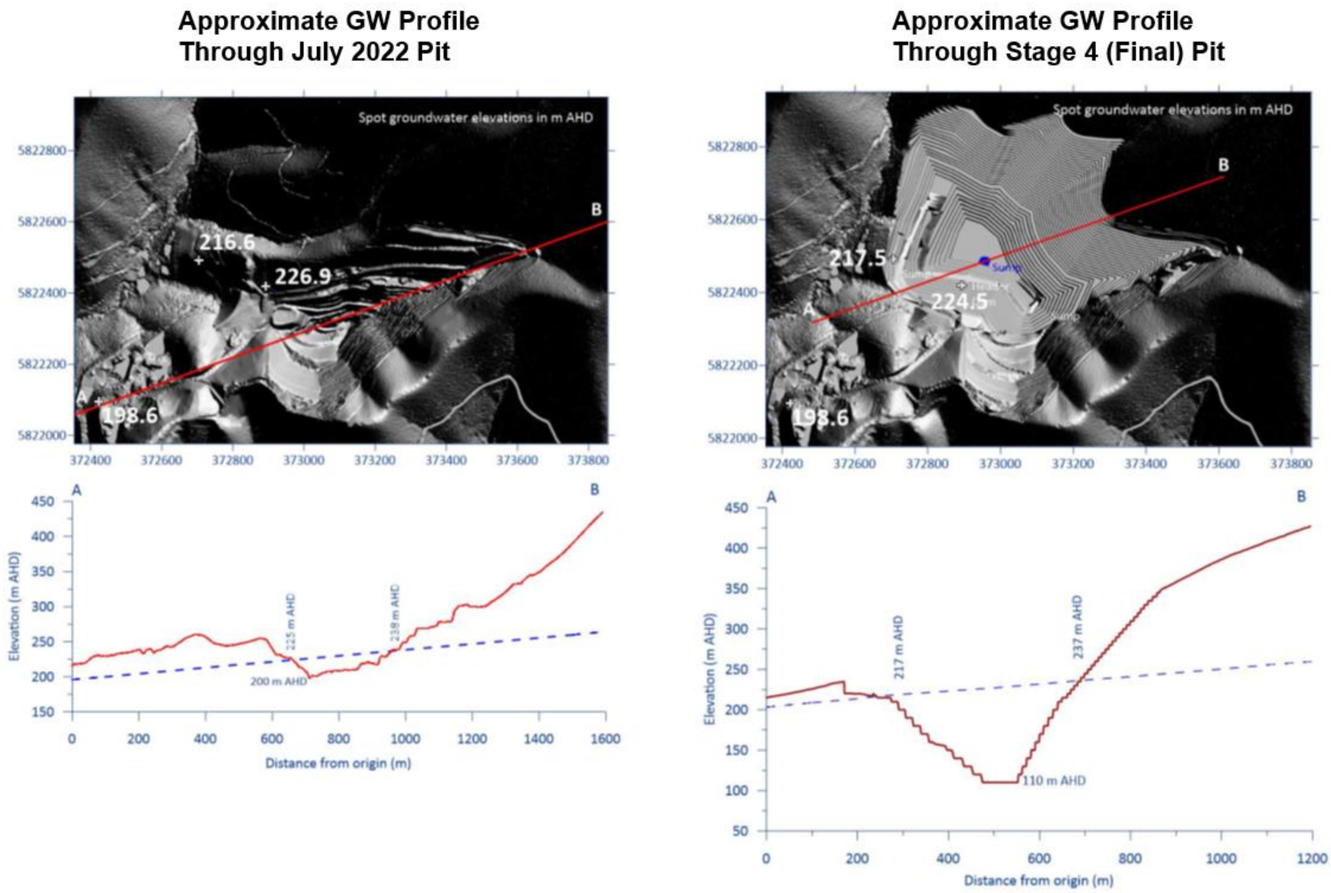


Figure 10 ***Approximate Far-field Groundwater Profiles (ENE to WSW direction) relative to the July 2022 and proposed Stage 4 pit design (not considering cone of depression) (JLCS, 2023).***

The final pit floor will be located > 120 m below the water table, with groundwater expected to drain into the pit through local discontinuities in the host rock mass (i.e., Humevale Siltstone) as it is dewatered. It is noted that due to the *exceedingly low transmissivity* of the Humevale Siltstone bedrock, a steep and localised cone of depression (i.e., steep groundwater profile around the pit) is expected.

When quarrying of the site ceases, the pit will be predominately filled by surface water, forming a final pit lake. The final pit lake level will be controlled by the elevation of the 'pit lake spill point' (RL 217 m), which is located at the west of the site (see Figure 11). As noted by JLCS (2023) '*when the lake water level reaches the spill point elevation, it will become a surface water dominated groundwater throughflow lake with groundwater flowing into the lake from up-hydraulic gradient to the northeast and out of the lake to the southwest*' (see Figure 12).

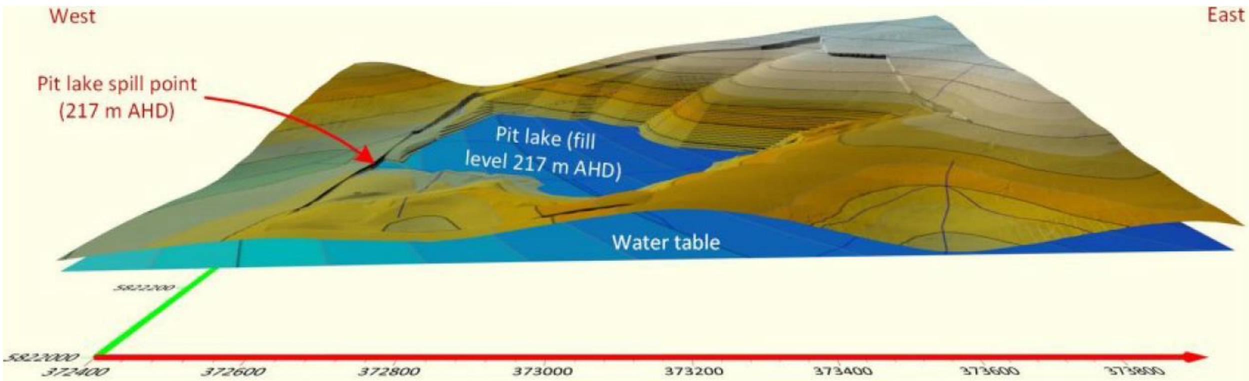


Figure 11 ***WA375 Terminal Pit Lake Visualisation (JLCS, 2023)***

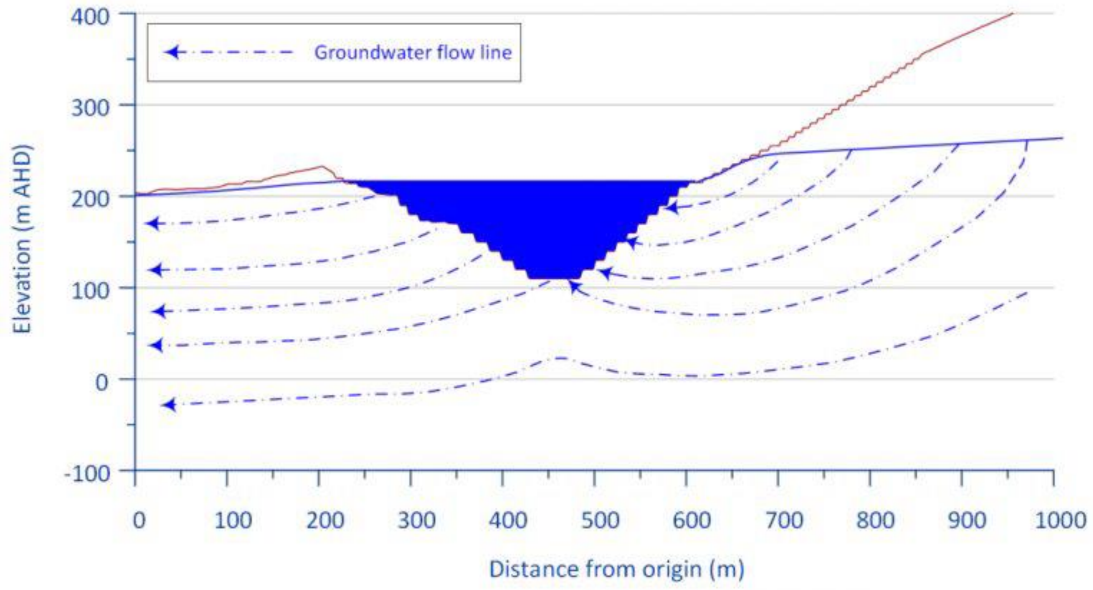


Figure 12 Groundwater Throughflow Pit Lake Visualisations (JLCS, 2023)

The final pit lake is expected to be achieved ~15 to 20 years after quarrying ceases (see Figure 13), due to surface water capture and groundwater inflow.

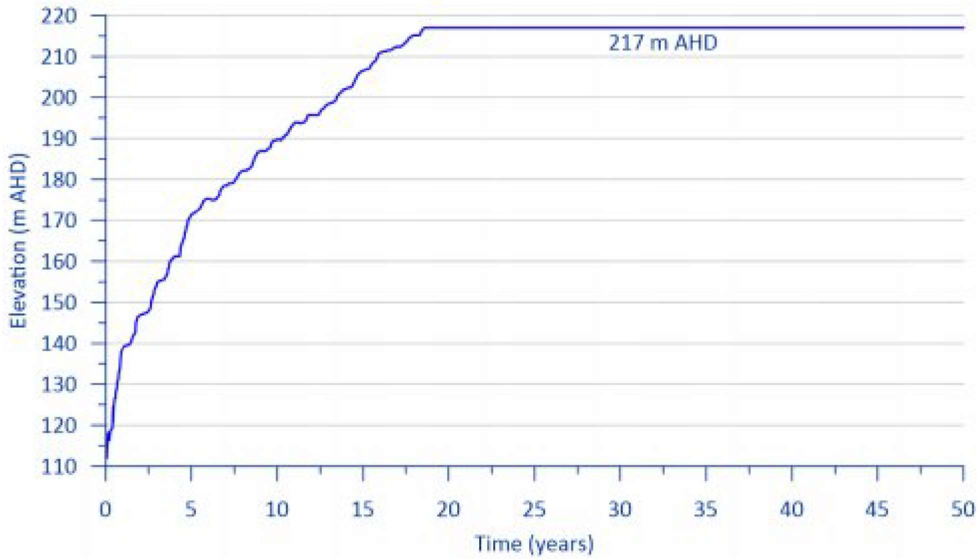


Figure 13 Estimated Stage 4 Pit Lake Filling Times due to Surface Water (extracted from JLCS, 2023)

2.4 Hydrology

Surface water run-off generated within the current extraction footprint and from the quarry's upper tributary catchments that have been 'intersected' by extraction activity, are channelled down the excavated slopes to a sump located in the quarry floor. Water collected in the sump is pumped to an upper level 'Header Dam', which also functions as a bio-retention basin, filtering any fines.

The quarry's water cart is filled from the Header Dam as the primary source of water used for dust suppression of haul roads, the crushing and stockpile pad traffic areas and other vehicle access roads, including the sealed quarry access road, from McMahons Road. Higher seasonal inflows that exceed the fill capacity of the Header Dam are discharged in a south-west direction, via an installed spillway discharge pipe to a tributary colloquially referred to as the 'Moora Creek' and designated as 'Main Tributary' in Figure 14. The Main Tributary is located in the adjoining Dandy Premix land known (Lot 49A) and is the major north-east tributary of Ure Creek. The confluence of these two waterways occurs at the southern boundary of Lot 49A. The Main Tributary also receives inflows from a number of smaller tributary and gully inflows in the upper reaches of Lot 49A and from several in the proposed quarry expansion area of Lot 50C.

The ‘Main Dam’ receives surface water inflows from the vegetated patch to its elevated north-east and the northern extent of the sales loading and stockpiles pad area. Water stored in the Main Dam is plumbed to service the spray-bar dust suppression equipment installed on the primary and secondary crushing plant operations, including product conveyors and the sales loader concrete aggregates stockpiles.

Any surplus surface water captured in the ‘Main Dam’ is discharged via a spillway pipe to the same tributary in Lot 49A to its west, which receives overflow discharges from the Header Dam. As previously outlined, the tributary joins the Main Tributary and thereafter, Ure Creek.

The key site surface water infrastructure is depicted below in Figure 14. It is understood that a hydrological report has been prepared by Water Technology Pty Ltd, which address the surface water infrastructure assets and the interaction with mining operations in more detail.

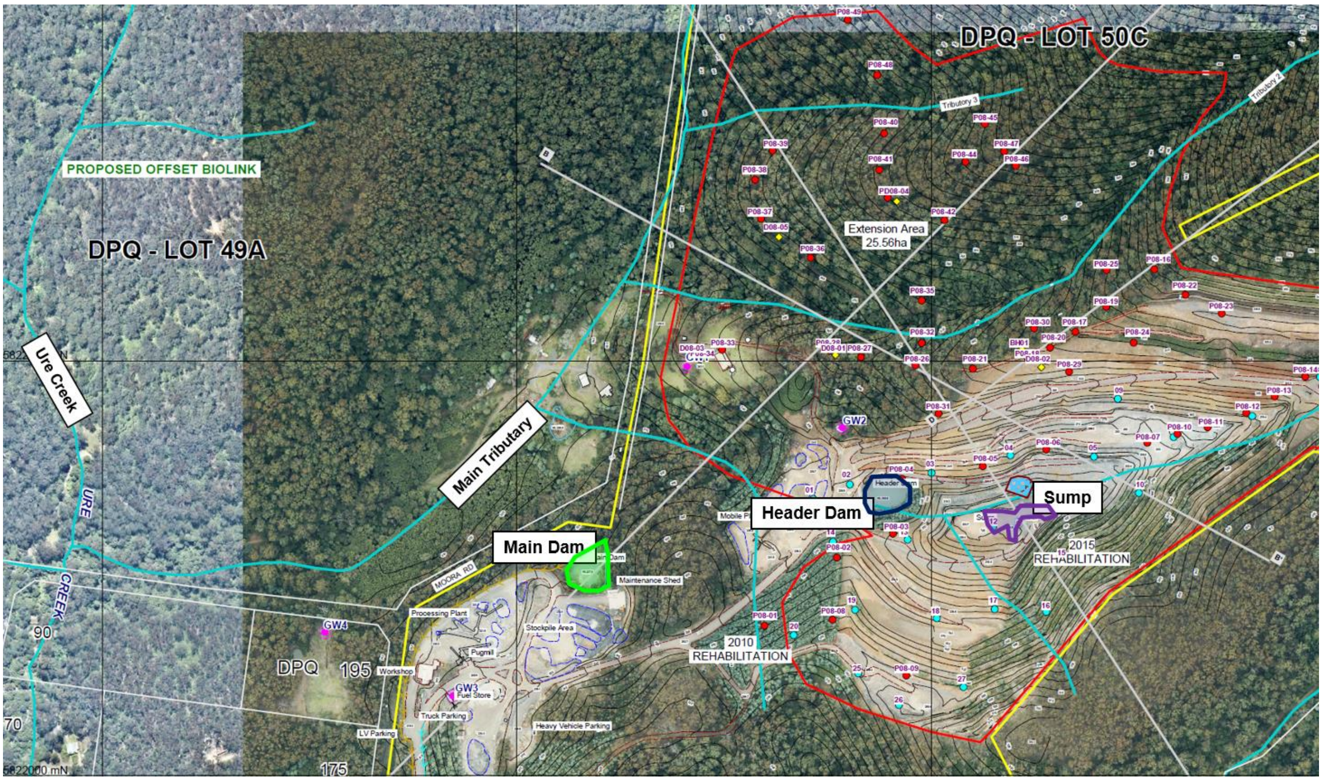


Figure 14 *Plan View Depicting Key Surface Water Infrastructure*

2.5 Drilling Campaign

A resource definition (res-def) drilling campaign was undertaken by the client’s resource consultants BCA in the report titled ‘*Drilling Data Summary – Woori Yallock Quarry*’, dated 14 July 2010 (BCA Reference No. D10-001) to delineate the site’s resource material. The res-def drilling campaign consisted of 49 percussion drill holes and 5 diamond drill holes. In late 2022, an additional 161 m deep diamond drillhole was drilled and logged by Macquarie Geotech. The locations of the drill holes are presented in Figure 15. A summary of the drilling coordinates and details are provided in Table 5.

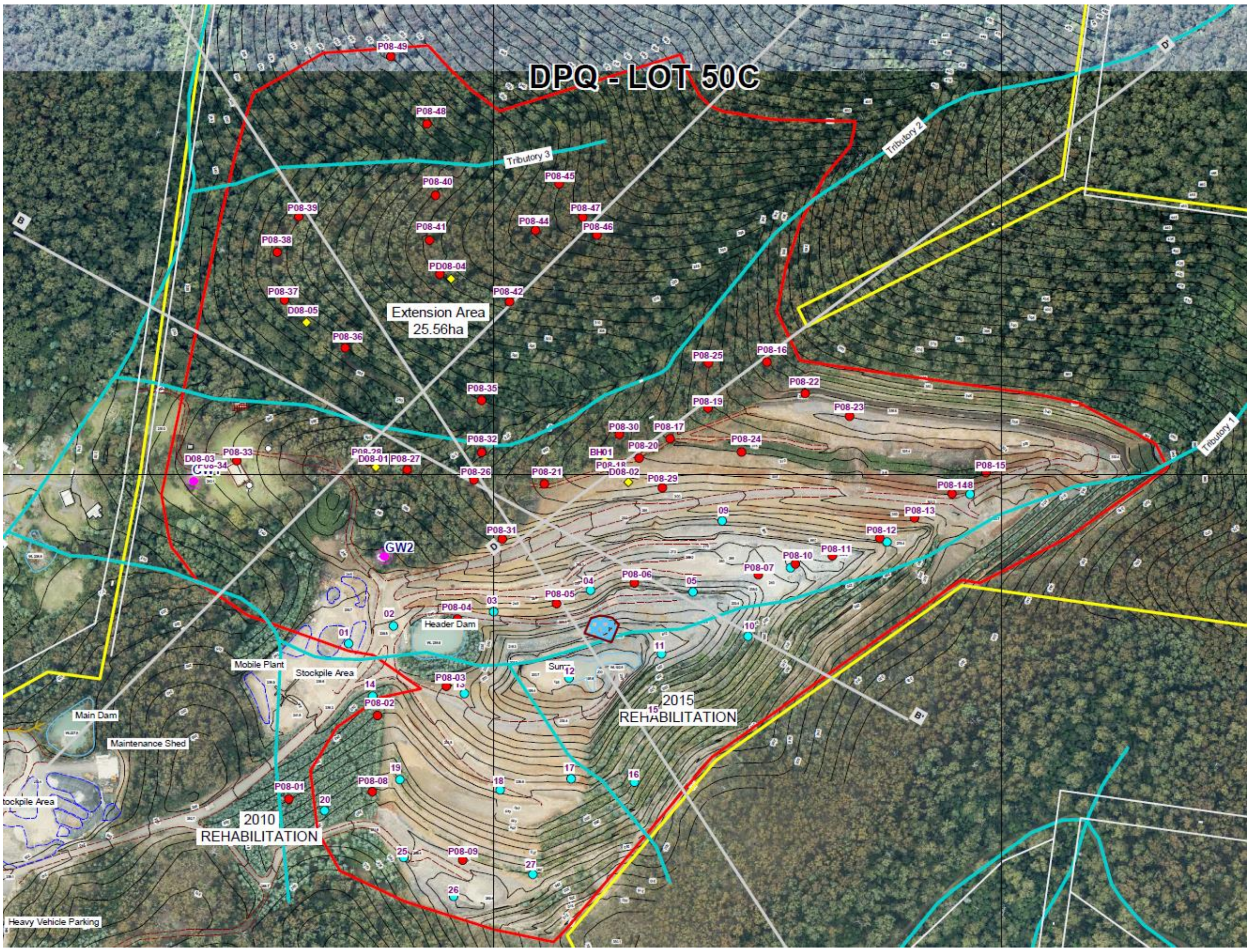


Figure 15 Plan View Depicting Drillhole Locations, after BCA (2023)

Table 5 Summary of Res-Def Drilling Details, after BCA (2010)

Bore ID	Easting	Northing	Collar RL (m AHD)	Drill Depth (m)	Azimuth (°)	Inclination (°)
P08-01	372686	5821997	254.5	27	0	90
P08-02	372773	5822079	250	25.2	0	90
P08-03	372869	5822105	250	23.4	0	90
P08-04	372852	5822174	248	21.6	0	90
P08-05	372949	5822189	259	25.2	0	90
P08-06	373025	5822209	267	25.2	0	90
P08-07	373147	5822217	280	25.2	0	90
P08-08	372767	5822004	264	25.2	0	90
P08-09	372857	5821937	274	23.4	0	90
P08-10	373184	5822227	284	23.4	0	90
P08-11	373220	5822236	288	25.2	0	90
P08-12	373267	5822253	295	21.6	0	90
P08-13	373301	5822273	302	19.8	0	90
P08-14	373338	5822297	310	14.4	0	90
P08-15	373371	5822317	320	21.6	0	90
P08-16	373156	5822426	339.9	25.2	0	90
P08-17	373061	5822351	311	25.2	0	90

Bore ID	Easting	Northing	Collar RL (m AHD)	Drill Depth (m)	Azimuth (°)	Inclination (°)
P08-18	373004	5822318	300.6	25.2	0	90
P08-19	373098	5822381	320.3	25.2	0	90
P08-20	373030	5822332	304.8	25.2	0	90
P08-21	372937	5822306	292.5	25.2	0	90
P08-22	373193	5822395	337	25.2	0	90
P08-23	373237	5822373	333.9	25.2	0	90
P08-24	373131	5822338	313	25.2	0	90
P08-25	373098	5822425	320.7	25.2	0	90
P08-26	372867	5822310	278.6	25.2	0	90
P08-27	372802	5822320	267.4	25.2	0	90
P08-28	372764	5822328	265.4	25.2	0	90
P08-29	373053	5822303	300.2	25.2	0	90
P08-30	373010	5822355	302.9	25.2	0	90
P08-31	372895	5822252	276.3	25.2	0	90
P08-32	372875	5822337	275.1	25.2	0	90
P08-33	372634	5822329	244.5	25.2	0	90
P08-34	372607	5822318	239.7	25.2	0	90
P08-35	372875	5822389	266.7	14.4	0	90
P08-36	372741	5822440	264.2	25.2	0	90
P08-37	372681	5822487	262.2	25.2	0	90
P08-38	372674	5822534	262.2	25.2	0	90
P08-39	372695	5822569	265.2	25.2	0	90
P08-40	372830	5822590	295.1	25.2	0	90
P08-41	372824	5822546	299.2	25.2	0	90
P08-42	372903	5822486	298.8	25.2	0	90
P08-43	372834	5822512	296.9	25.2	0	90
P08-44	372928	5822556	328	25.2	0	90
P08-45	372951	5822601	332.1	25.2	0	90
P08-46	372988	5822551	336.1	25.2	0	90
P08-47	372975	5822569	337.6	25.2	0	90
P08-48	372821	5822660	289.8	25.2	0	90
P08-49	372786	5822727	311.9	25.2	0	90
D08-01	372771	5822323	266	147	90	60
D08-02	373019	5822308	299	64	90	60
D08-03	372602	5822321	238	51	90	60
D08-04	372845	5822508	297	70.2	90	60
D08-05	372845	5822508	263	56.2	90	60
BH01	373109	5822514	161.1	301.0	90	90

Information from the res-def drilling campaign was utilised to delineate zones of weathering for downstream stability analyses, which is discussed further in Section 5.

2.6 Past Geotechnical Studies

Summarised in Table 6 are the past geotechnical studies that pertain the Woori Yallock Quarry site.

Table 6 Summary of Past Geotechnical Studies

Author	Reference	Key Outcomes
GHD (2007)	*Report on Woori Yallock Quarry – Geotechnical Assessment,* dated 16 October 2007 (GHD Reference No. 31/21728/139978, Rev 0	Site inspection followed by geotechnical analyses to provide slope geometries according to the conditions exposed, anticipated and known slope performance. – Mapping of defect sets along excavated slopes and interpretation of major defect characteristics – Derivation of rock mass strength parameters using site observations and field estimates – Provided recommendations on batter/berm configurations and overall slope design geometries – Provided recommendations for ongoing slope quarry management activities
GHD (2009)	*Report on YVQ Launching Place – Geotechnical Review of Proposed Quarry Extension (Design 2009 - 10)*, dated 10 August 2009 (GHD Reference No. 31/24592/168663, Rev 0)	Review of previous geotechnical assessments and related studies to provide recommendations on site expansion and ground control during excavation supported by site inspections. – Additional mapping of defect structures along excavated slope faces – Confirmed the GHD (2007) slope design parameters – Identified suitable drainage options from a geotechnical perspective
GHD (2021)	*Yarra Valley Quarry Desktop Review and Gap Analyses,* dated 8 January 2021 (GHD Reference No. 12535505-10052-1, Rev 0)	Provided a desktop review of the available geotechnical information to identify potential gaps in data required to facilitate subsequent geotechnical assessments for future WPV submissions. Based on the gap analyses GHD (2021) recommended the following: – Confirm the geological conditions at the site including persistence of defect structures and depth of weathering – Adoption of contemporary design acceptance criteria – Consideration to contemporary ground and surface water studies – Confirmation of material strengths – Consideration to rehabilitation profiles

The past geotechnical studies outlined in Table 6 are discussed in further detail in the following subsections. It is recommended that these reports are read in conjunction with this report to fully appreciate the works undertaken to date and the context to which gleaned information pertains to this report.

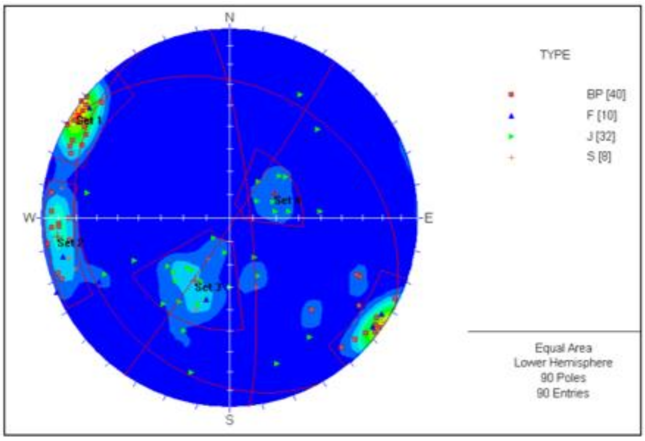
2.6.1 GHD (2007) – Geotechnical Assessment

A geotechnical assessment of excavated quarry slopes was undertaken with the benefit of information obtained during site visits conducted on 31 August and 5 September 2007. The objective of the site visit was to visually assess the stability performance and geotechnical conditions of excavated slopes, and to undertake defect mapping of the quarry exposures.

A total of 90 discontinuities were mapped as part of the site visit. Stereonet projections and cluster analyses indicate that there are 4 primary defect sets, the details of which are summarised in Table 7. The corresponding stereographic projection and a plan view depicting the general locations of the mapped defects relative to the 2007 pit is presented in Figure 16.

Table 7 Summary of Defect Characteristics, after GHD (2007)

Defect Set ID	Mean Dip Angle (°)	Mean Dip Direction (°)	Description
D1	88	124	Bedding
D2	81	84	
D3	31	29	Joint
D4	22	242	



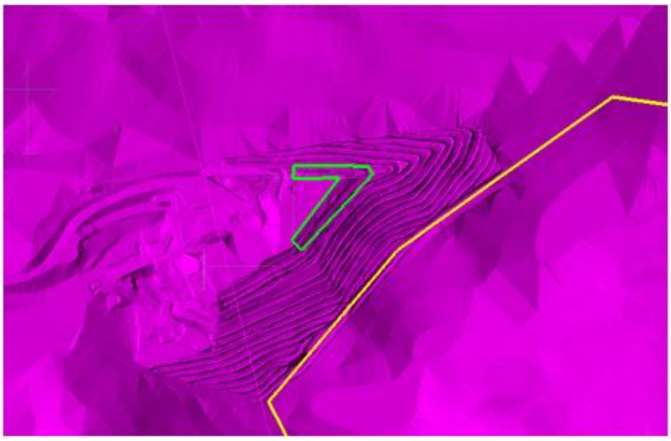


Figure 16 Southern Hemispherical Stereonet Projection of Mapped Discontinuities, after GHD (2007)

Defect plane strengths were derived based on site observations and assumed values of the material properties for the purpose of kinematic limit equilibrium modelling. Defect strengths adopted by GHD (2007) are summarised in Table 8.

Table 8 Summary of Defect Plane Strengths, after GHD (2007)

Cohesion (kPa) – c'	Friction Angle (°) – ϕ'
0	35

Based on the kinematic analyses results, the batter-bench configuration put forward included batter face angles of 75° and 45° for the slightly weathered to fresh rock and moderately weathered rock units respectively, with the adoption of 10 m bench heights and 10 m berm widths within both units.

The slope stability performance considering the recommended batter-bench configuration was undertaken to assess rock mass instability at the overall slope scale. The rock mass parameters were derived based on site observations and field strength estimates for two lithological units as summarised in Table 9.

Table 9 Summary of Rock Mass Strength Parameters, after GHD (2007)

Material Description	Cohesion (kPa) – c'	Friction Angle (°) – ϕ'
Slightly Weathered to Fresh	1800	50
Moderately Weathered to Highly Weathered	140	14

Slope stability performance was measured against a design acceptance criterion of FoS > 1.25 for an overall slope scale instability. The results of the stability analyses indicated that slope stability performance objectives were satisfied with the recommended batter-berm configuration under expected and fully saturated conditions. Recommendations for slope excavation practice were put forward by GHD (2007) following the adoption of the recommended design geometry, which largely focused around maintaining surface water management protocols, ongoing ground control practices and monitoring.

2.6.2 GHD (2009) – Geotechnical Review of Proposed Quarry Extension

A review of the slope design recommendations made by GHD (2007) was undertaken by GHD (2009) to support the expansion of the site. The review was supported by additional defect mapping obtained during a site inspection on 7 June 2009.

An additional 97 discontinuities were mapped along excavated slopes during the GHD (2009) visit, supplementing those previously recorded by GHD (2007) i.e., a total of 187 discontinuities mapped across the two visits. Stereonet projections and cluster analyses on the combined discontinuities indicate that there are 5 major defect sets, which are summarised in Table 10. The corresponding stereonet projection is presented in Figure 17.

Table 10 Summary of Defect Characteristics, after GHD (2009)

Defect Set ID	Mean Dip Angle (°)	Mean Dip Direction (°)	Description
D1	88	125	Bedding
D2	80	83	Bedding
D3	36	30	Joint
D4	30	231	Joint
D5	63	318	Bedding

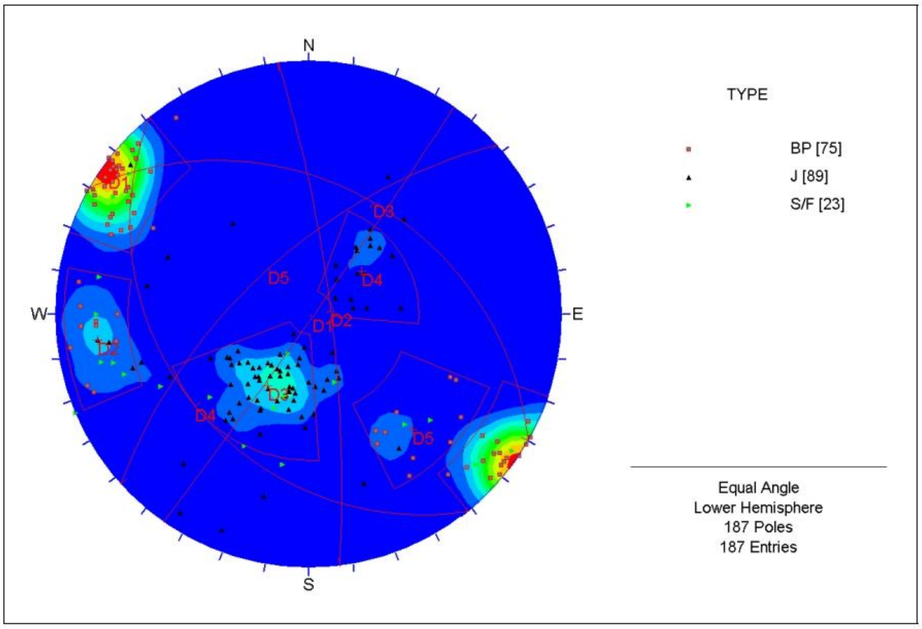


Figure 17 Southern Hemispherical Stereonet Projection of Mapped Discontinuities, after GHD (2009)

Kinematic analyses performed on the revised discontinuity sets performed by GHD (2009) indicated that the GHD (2007) recommended batter-bench configuration was suitable and thus no changes to the overall slope geometry were required for the proposed expansion. It should be noted that there were no changes to the defect plane and rock mass strength between the GHD (2007) and GHD (2009) assessments.

Sensitivity analyses performed as part of this assessment indicated that kinematic instability mechanisms, particularly within the extremely weathered to highly weathered materials are dependent upon groundwater conditions. In line with the results of the sensitivity analyses, recommendations for surface water drainage options were put forward which included drainage down the excavated batter face and diversions around the batter crest.

2.6.3 GHD (2021) – Desktop Review and Gap Analyses

Based on the desktop review and gap analyses performed by GHD (2021), a series of recommendations were made for future geotechnical assessments to inform the WPV submission. The recommendations made broadly encompass:

- Confirmation of continuity and orientation of defect sets D3 and D4 as interpreted by GHD (2009), which are likely to be the critical defect structures controlling kinematic stability.
- Confirmation of weathering depth, where not inadequate definition could "*have an adverse impact on slope stability if an appropriate geometry is not applied".*
- Adoption of contemporary design acceptance criteria published by ERR and review of slope design parameters.
- Confirmation of material strengths with the benefit of contemporary field observations and estimates.
- Incorporate contemporary surface and groundwater information into stability analyses.
- Refine final landform concepts with the benefit of geotechnical inputs.

3. Site Investigations

Dr. Sanjive Narendranathan (Technical Director, Mining) and Mr. Jack Stipcevich (Geotechnical Engineer) of GHD carried out an inspection of the Woori Yallock Quarry on the 14 of February 2022. The objective of the site visit was to:

– Inspect the stability performance of existing quarry slopes, undertake confirmatory structural mapping.
– Gain a visual appreciation of the efficacy of operational procedures.
– Identification of any potential site hazards.

Visual inspections encompassed the entire extraction area, with structural field measurements taken from slopes that were safely accessible by foot and where access was permitted. Summarised below are the relevant key observations pertaining to the stability of the quarry.

3.1.1 Quarrying Operations

Quarry development is currently occurring in a north easterly direction. Five main lithological units were observed on site, which include an overburden, an extremely weathered rock, a highly weathered rock, a slightly to moderately weathered rock, and a fresh rock. Softer 'diggable' units e.g., overburden, are excavated using conventional earth moving equipment (excavator and dozer) the harder 'blastable' rock units are excavated using conventional drill and blast methods with truck and excavator clean-up.

Stripping of topsoil is limited to the area to be 'opened up' in the immediate future to minimise the amount of disturbance at any given time. Topsoil is used immediately for progressive rehabilitation where practicable, and otherwise stockpiled around the site for later use. Progressive rehabilitation of terminal slopes incorporates topsoil bunding along the benches followed by revegetation. Where topsoil cannot be immediately used for progressive rehabilitation, erosion protection measures are employed where required to mitigate the erosion potential of the topsoil stockpiles.

Overburden material is currently being placed to form a continuous slope along the southern portion of the extraction area and will form part of the final rehabilitated landform.

Excavated resource material is transported onsite via haul trucks to the fixed crushing and screening plant, where processed material is stored as stockpiles prior to sale.

3.1.2 Stratigraphic Sequencing – Weathering Profile

The Woori Yallock Quarry deposit is comprised predominantly of metamorphosed Humevale Siltstone (Hornfels) overlain by Quaternary aged soil (thin topsoil overlying regolith). The Humevale Siltstone is characterised by varying degrees of weathering, with each zone of weathering clearly demarcated by a weathering horizon. Accordingly, for this assessment, delineation of the stratigraphy has been done sympathetically to the observed weathering horizon using the ISRM (1981) classification, forming the basis for categorising materials with 'similar' geotechnical characteristics. The ISRM (1981) classification in summarised in Table 11.

Table 11 Weathering Grades of Rock Mass (ISRM, 1981)

Term	Symbol	Description	Grade
Fresh	Fr	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly Weathered	SW	Discolouration indicates weathering of rock material and discontinuity may be somewhat weaker externally than in its fresh condition.	II
Moderately Weathered	MW	Less than half the rock material is decomposed and or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as a corestones.	III
Highly Weathered	HW	More than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as a corestones.	IV

Term	Symbol	Description	Grade
Extremely Weathered	EW	All rock material is decomposed and/or disintegrated to a soil, the original mass structure is still largely intact.	V
Regolith	RS	All rock mass fabric is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported	VI

In line with the weathering grades outlined in Table 11, five weathering grades were observed on site as follows:

– **Unit 1** – Regolith (RS) – up to 2 m, overlying
– **Unit 2** – Extremely Weathered (EW) Hornfels – up to 20 m
– **Unit 3** – Highly Weathered (HW) Hornfels – up to 20 m
– **Unit 4** – Slightly Weathered to Moderately Weathered (SW to MW) Hornfels up to 20 m
– **Unit 5** - Fresh Hornfels – extending below the quarry floor

The stratigraphic units are depicted on an aerial photographic and as a stratigraphic column in Figure 18.

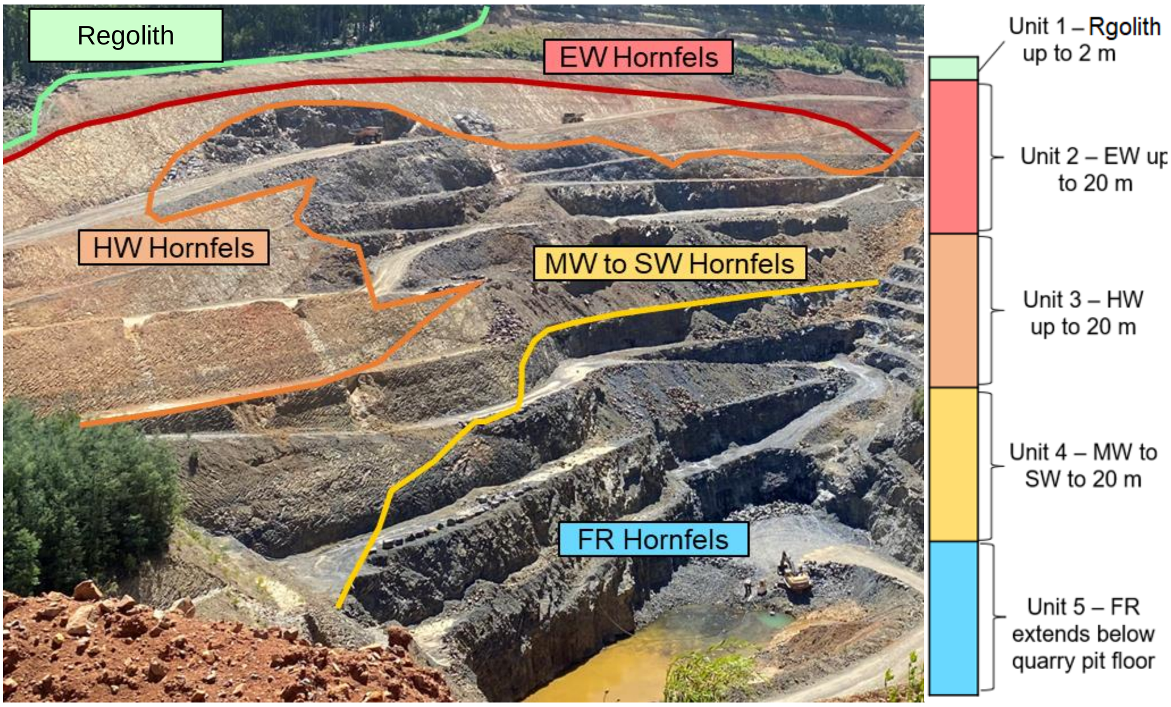


Figure 18 Aerial Image Depicting Stratigraphic Units and Stratigraphic Column

As depicted in Figure 18, the thickness of the weathering zone varies across the site. Such variations in the weathering profile can be attributed to the topography of the site or post-deposition hydrothermal fluid alteration associated with the granodiorite intrusion. Further discussion on the weathering profile with respect to progressive quarry development is outlined in Section 5.3.

3.1.3 Current Stability Performance

Existing quarry slopes were observed to be at sub-vertical batter face angles within the moderately weathered to fresh Hornfels and 45° in the extremely to moderately weathered Hornfels, for bench heights in the order of 5 to 10 m and bench widths in the order of 5 m.

Figure 19 Existing Quarry Wall – Northern Wall

The intact rock mass was observed to be inter-bedded which are typically very thin (< 1 mm) where the spacing between the bedding planes varies in thickness. Along the eastern wall bedding planes are shallow dipping, daylighting along the slope face, as shown in Figure 20 and steepen up along the northern wall, as shown in Figure 21, which is likely a result of the late-stage granite intrusion on the local geology.

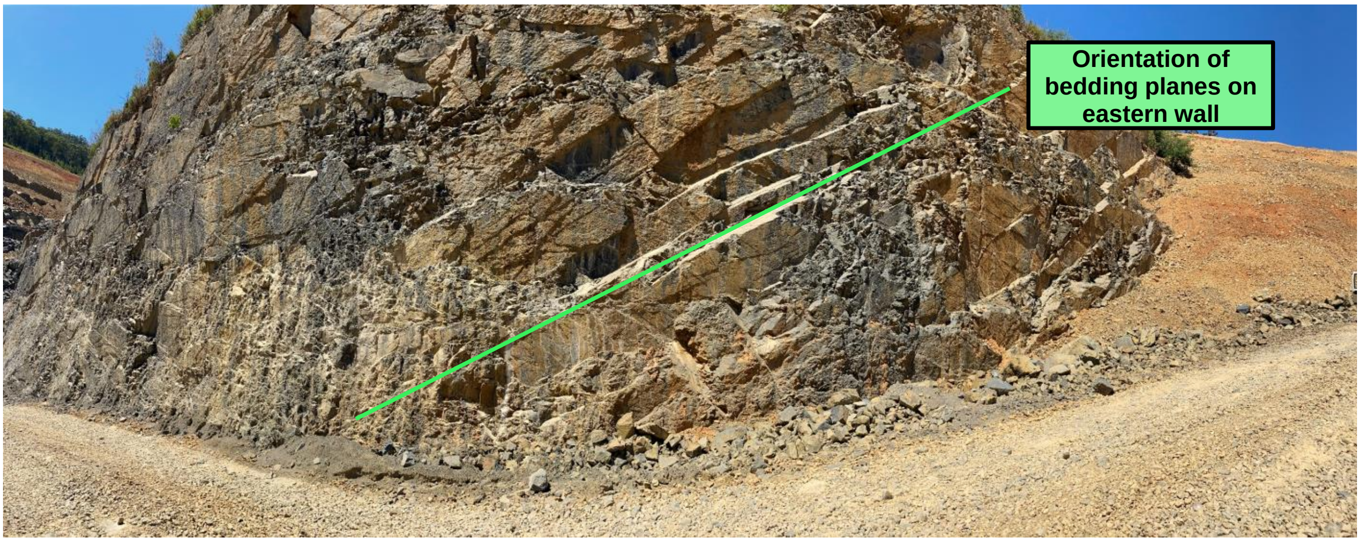


Figure 20 Photograph depicting Interbedded Rock Mass – Eastern Wall

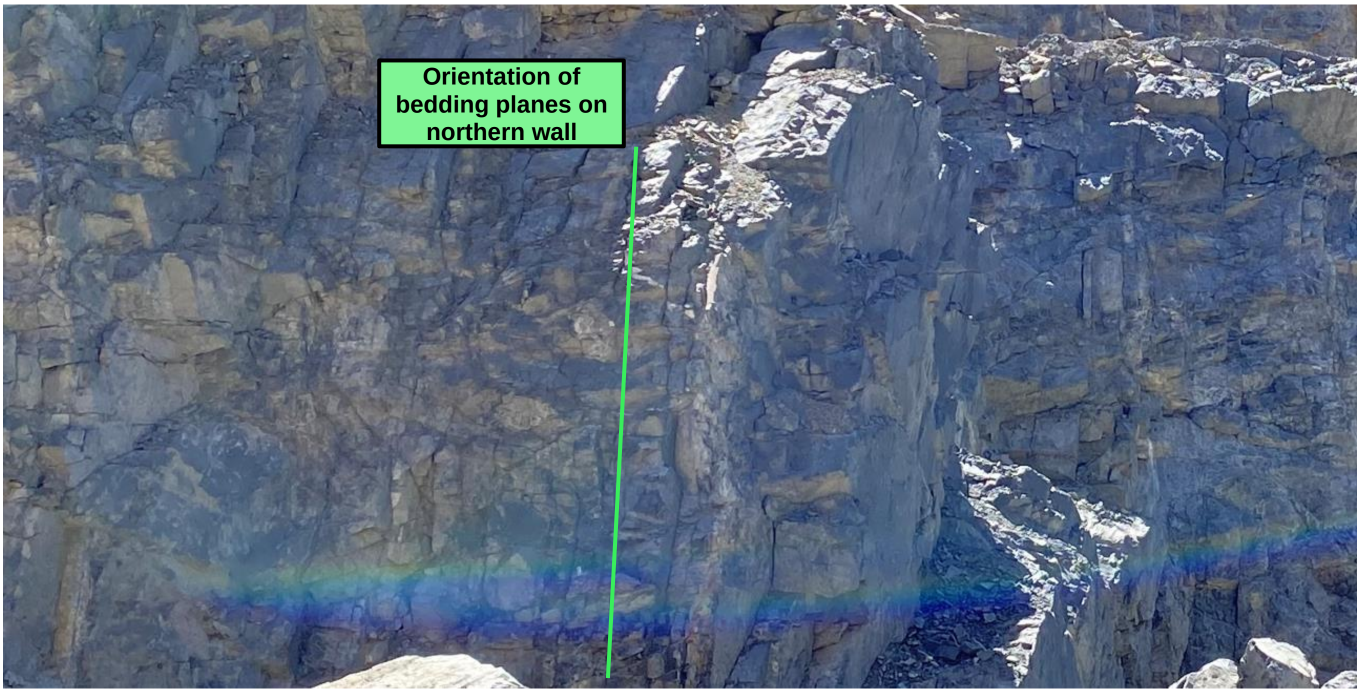


Figure 21 ***Photograph depicting Interbedded Rock Mass – Northern Wall***

There is evidence that suggests that where blasting has occurred along slopes oriented in the same direction as discontinuity planes that planar sliding has occurred as shown in Figure 22.

Figure 22 ***Photograph Depicting Planar Sliding Surface – North East Corner***

Evidence of minor structural instability in the form of tetrahedral wedge sliding was observed along exposed quarry slopes as shown in Figure 23. In certain areas, larger wedge type instabilities were observed as shown in Figure 24 and Figure 25. It should be noted that wedge type instabilities in Figure 25 are likely to have been promoted as a results of water ingress through defect planes.

Figure 23 Photograph of Minor Wedge Formation – Northern Corner

Figure 24 Photograph of Minor to Moderate Wedge Formation – Northern Corner

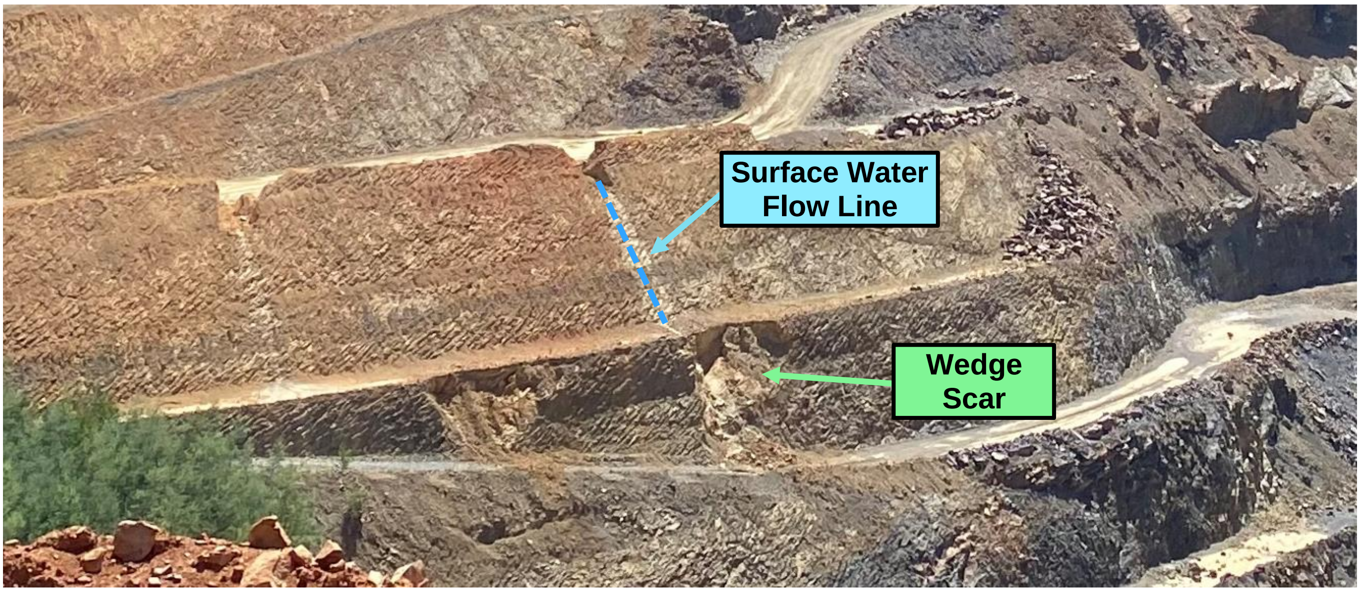


Figure 25 Tetrahedral Wedge Formations Promoted by Water Ingress – Northern Wall

There is evidence to suggest minor instability in the form of flexural block toppling has occurred along excavated quarry slopes. Depicted in Figure 26 are the defect planes that form the flexural toppling mechanism and a zone of instability where toppling has likely occurred.

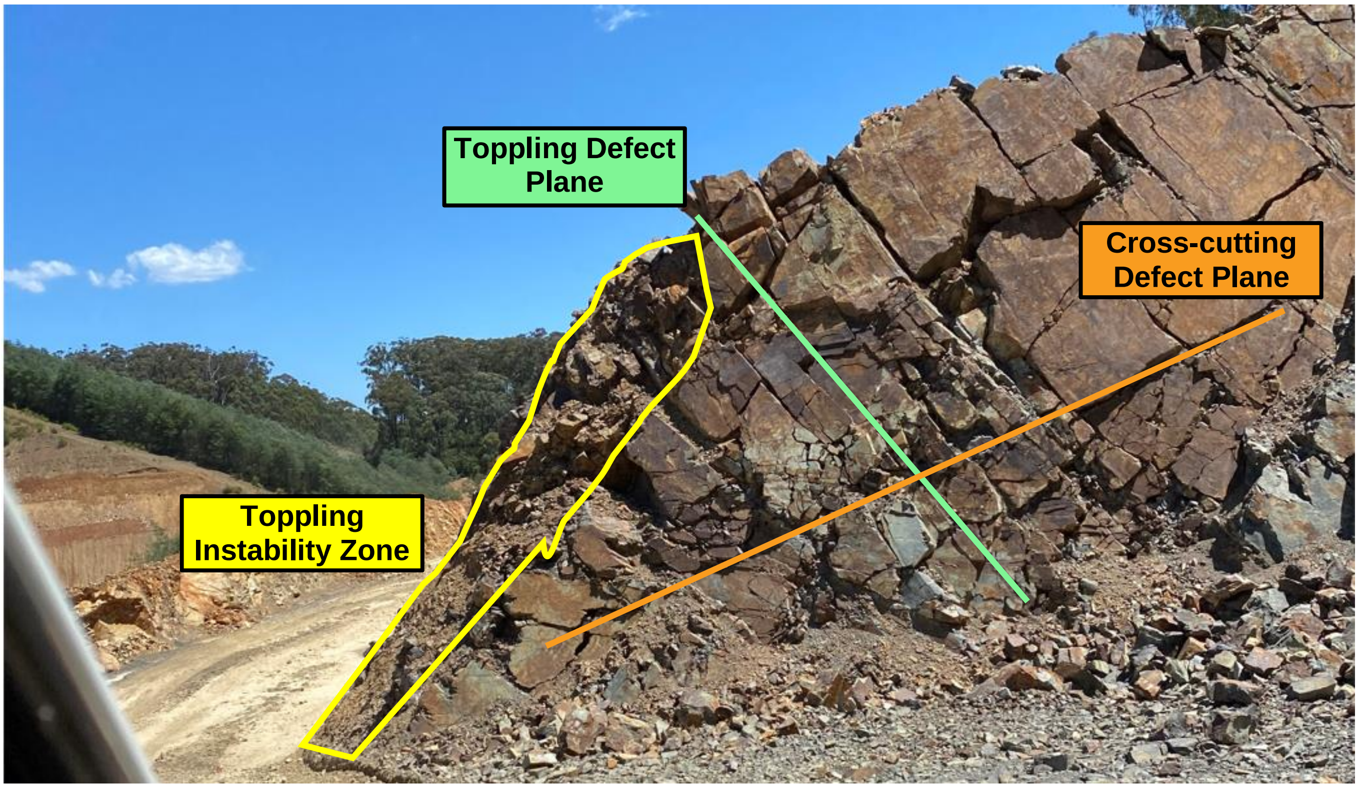


Figure 26 Photograph of Toppling Joints – North Wall

Evidence of minor circular/rotational type instability was observed within the extremely weathered unit along the upper slopes of the northern wall as shown in Figure 27.

Figure 27 ***Photograph Depicting Circular/Rotational Type Instability – Northern Wall***

Seepage flows through the excavated batter face were observed on the upper level and pit floor as shown in Figure 28 and Figure 29 respectively. Seepage flows were estimated to be very low in the order of 0.5 L to 1 L per minute.

Figure 28 ***Photograph Depicting Groundwater Seepage - North East Corner – Upper Level***

Figure 29 ***Photograph Depicting Groundwater Seepage -North East Corner – Pit Floor***

No major signs of major instability were observed with the overburden placement along the south wall for bench face angles in the order of 36° and maximum bench heights of 50 m. However, evidence of erosion gullies was readily observed across the slope face as shown in Figure 30.Table 1

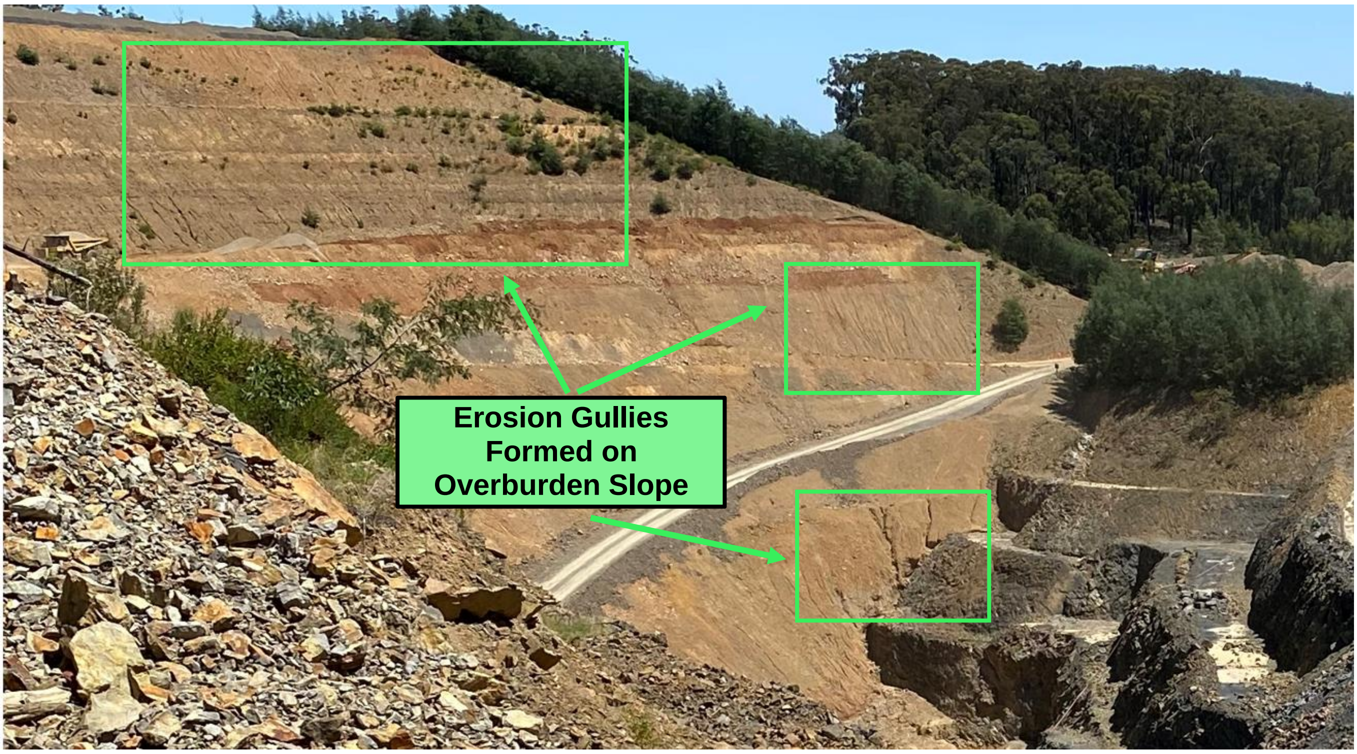


Figure 30 ***Photograph Depicting Erosion Gullies Along Weathered Rock (Overburden) Slope***

3.2 Defect Mapping

Targeted defect mapping was undertaken as a confirmatory exercise to determine whether major defect sets present along exposed slopes were consistent with those previously identified by GHD (2007) and GHD (2009). A total of 10 defect sets were mapped during the February 2022 site visit, the characteristics of which are summarised in Table 12. Further discussion on the mapped defects with respect to the GHD (2007) and GHD 2009) outcomes is presented in Section 5.1.4. Presented in Figure 31 is an example of mapped defects along the currently exposed eastern wall and Figure 32 below presents the location of these defects relative to the current (2022) pit and also historically mapped areas.

Table 12 Summary of Mapped Defect Characteristics – February 2022 Site Visit

Defect ID	Dip Angle (°)	Dip Direction (°)
D1	25	42
D2	63	73
D3	26	299
D4	49	347
D5	56	273
D6	90	107
D7	49	182
D8	56	313
D9	33	346
D10	86	276

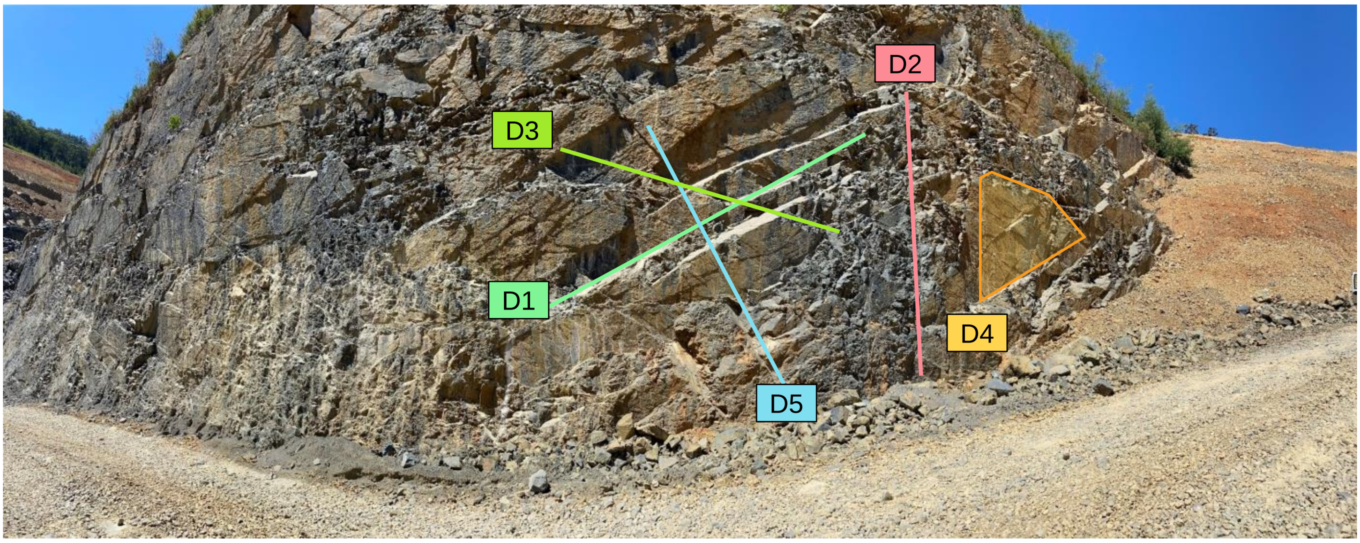


Figure 31 Photograph Depicting Example of Mapped Defect Sets- Eastern Wall

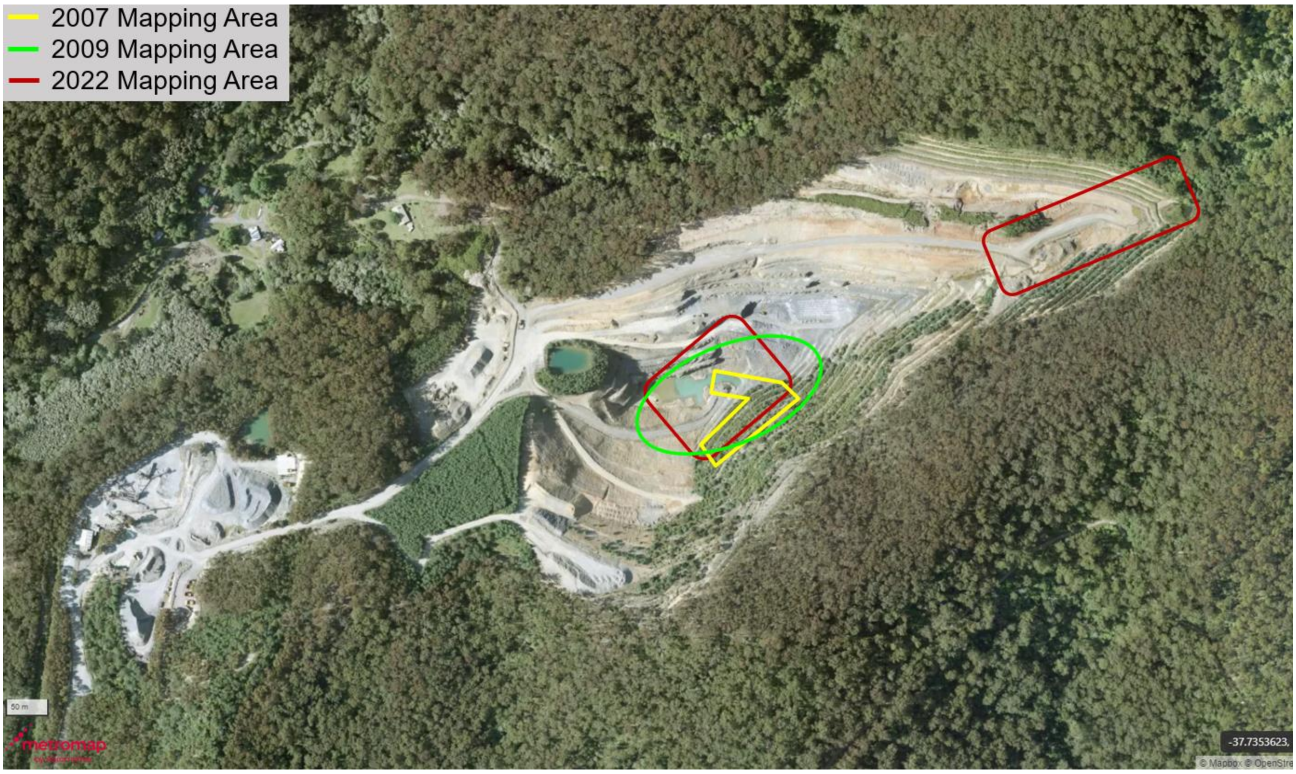


Figure 32 ***Plan View Depicting Mapped Locations relative to the 2022 Pit (Metromap, 2023)***

4. Current and Proposed Pit Geometry

Contour files provided to GHD of the as-excavated geometry, as depicted in Figure 33, were queried to examine the current ‘as constructed’ slope profiles.

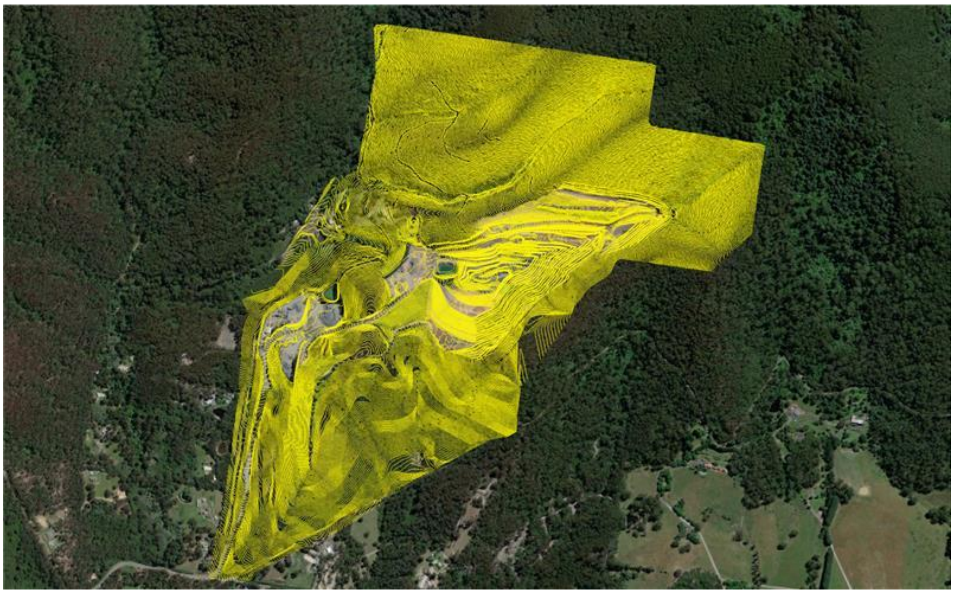


Figure 33 Contour Map Depicting Current Quarry Geometry

The current maximum depth of the quarry pit is in the order of 151 m below the top of crest. Typical slope geometries consist of overall slope angles in the order of 40° along operational slopes, 34° along terminal/rehabilitated slopes with the exception of weathered rock stockpile slope which has an overall slope angle of 24°.

Figure 34shows the model of the quarry profile as proposed for the WPV.

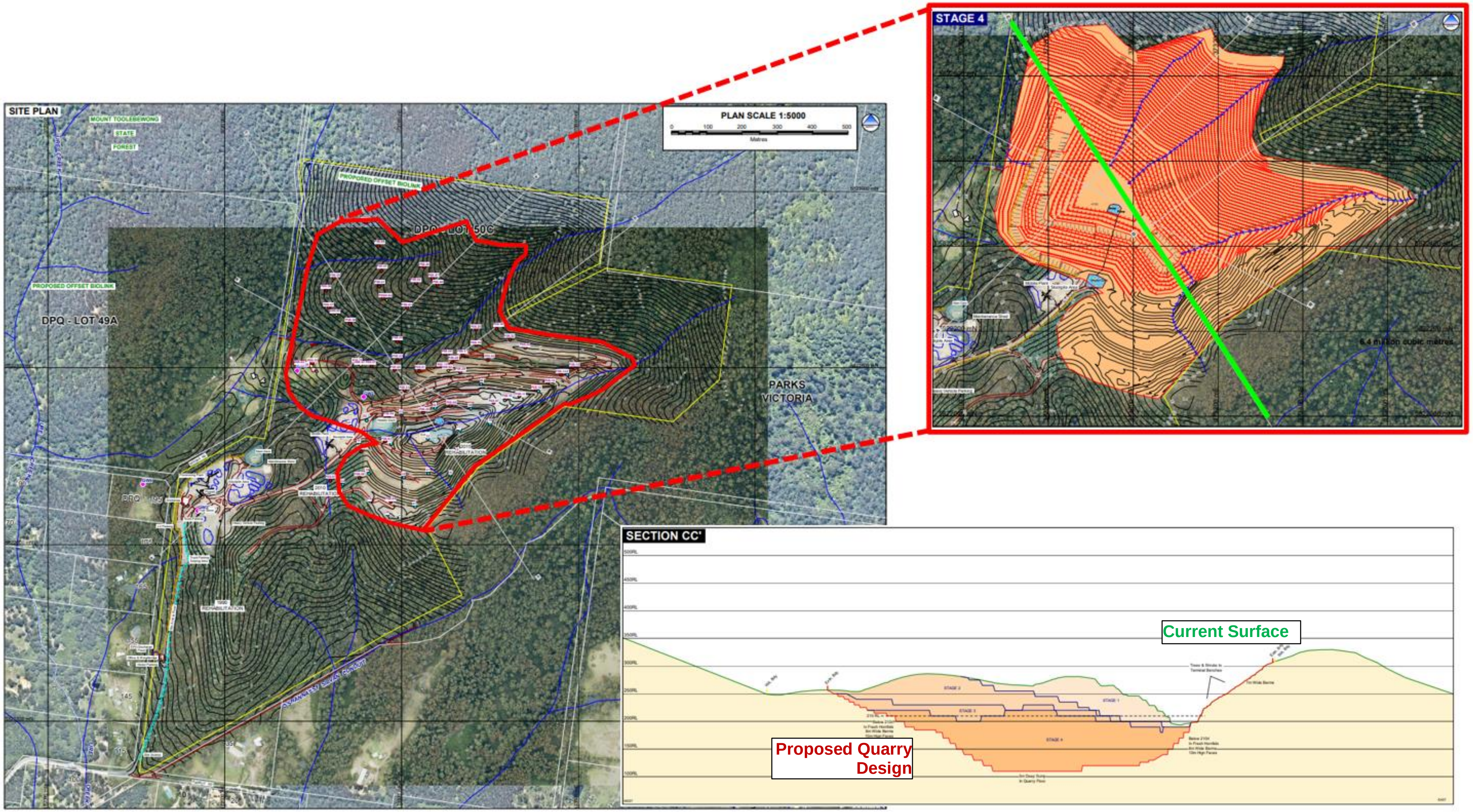


Figure 34 *Contour Map Depicting Proposed Quarry Expansion*

The maximum depth of the proposed quarry site is approximately 260 m below the crest (RL 110 m AHD) and incorporates an overall slope angle equal to 39° for terminal/rehabilitated slopes. A summary of the typical geometry profiles for the current and proposed slopes are summarised in Table 13.

Table 13 Summary of Slope Profile Geometries for Current and Proposed Slopes

Slope Parameter	Operational Slopes		Current Terminal / Rehabilitated Slopes	Proposed Terminal / Rehabilitated Slopes				Rock Slope
	Units 1,2 and 3	Units 4 and 5	All Units	Unit 1 and 2	Unit 3 and 4	Unit 5 (above pit lake level)	Unit 5 (below pit lake level)	
Bench face angle (°)	45	65	50	38	85	85	80	1V:2H
Bench height (m)	10		5	15	5	5	10	15
Bench width (m)	6.5		5	N/A	7	6	8	3
Maximum Overall Slope Height (m)	151			260				
Overall Slope Angle (°)	40		34	39				
*Fresh Hornfels unit (Unit 5) double benched below the pit lake level.								

The proposed final landform design incorporates the application of topsoil and vegetation along the majority of quarry slopes, a pit lake at approximately RL 217 m, and a weathered rock stockpile located along the eastern wall. The rock slope consists of approximately 15 m high benches with 8 m berms sloped to 1V:2H. Reinforcement against erosion is proposed to be undertaken using vegetated grasses. The final pit lake level is noted to be controlled by the 'spill' point.

A schematic depicting the typical terminal face treatment and rehabilitation is presented in Figure 35.

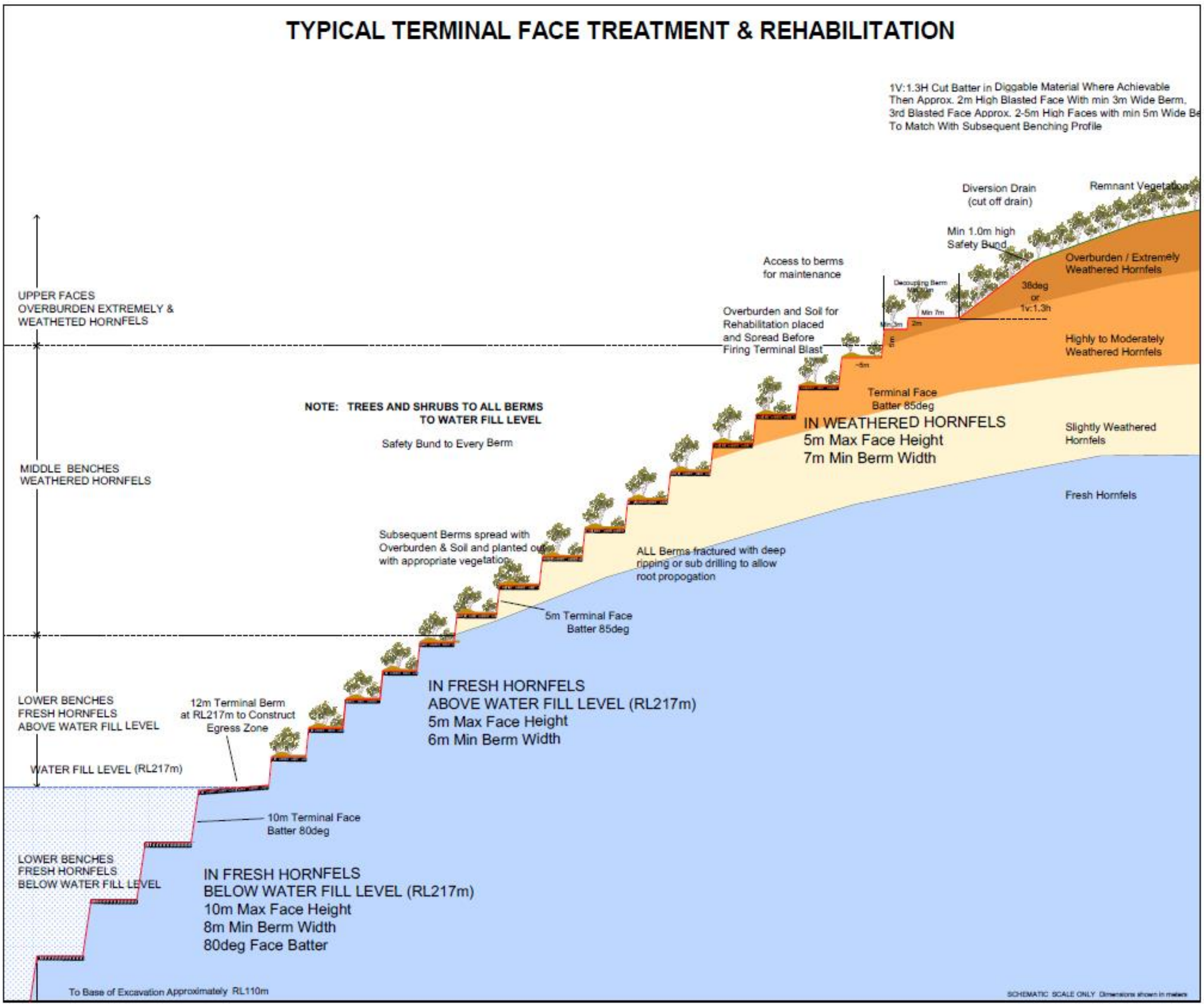


Figure 35 Schematic Depicting Typical Terminal Face Treatment and Rehabilitation (BCA Site Layout Plan, 2023)

5. Geotechnical Domains and Model

The geotechnical domain model forms the basis for any quarry pit slope design. The geotechnical domain model facilitates the segregation of a quarry pit into sectors or zones which have similar geological, structural and material property characteristics, thus modes of instability. In principle, the act of geotechnical domaining allows for multiple optimisation techniques to apply, where the slope design is optimised, in terms of safety and economics, for a given sector rather than applying a single slope design across the entire pit. In essence, geotechnical domaining a quarry pit can be used to inform quarry owners/operators where to focus their time and effort.

The geotechnical domain model is compiled from four component models:

– Geological model
– Structural model
– Hydrogeological model, and
– Rock mass (material properties) model

Geotechnical domaining of the Woori Yallock quarry site has relied upon the philosophy set out by Read and Stacey (2009). Outlined in the Figure 36 are the considerations that are taken into account when formulating site specific geotechnical domains.

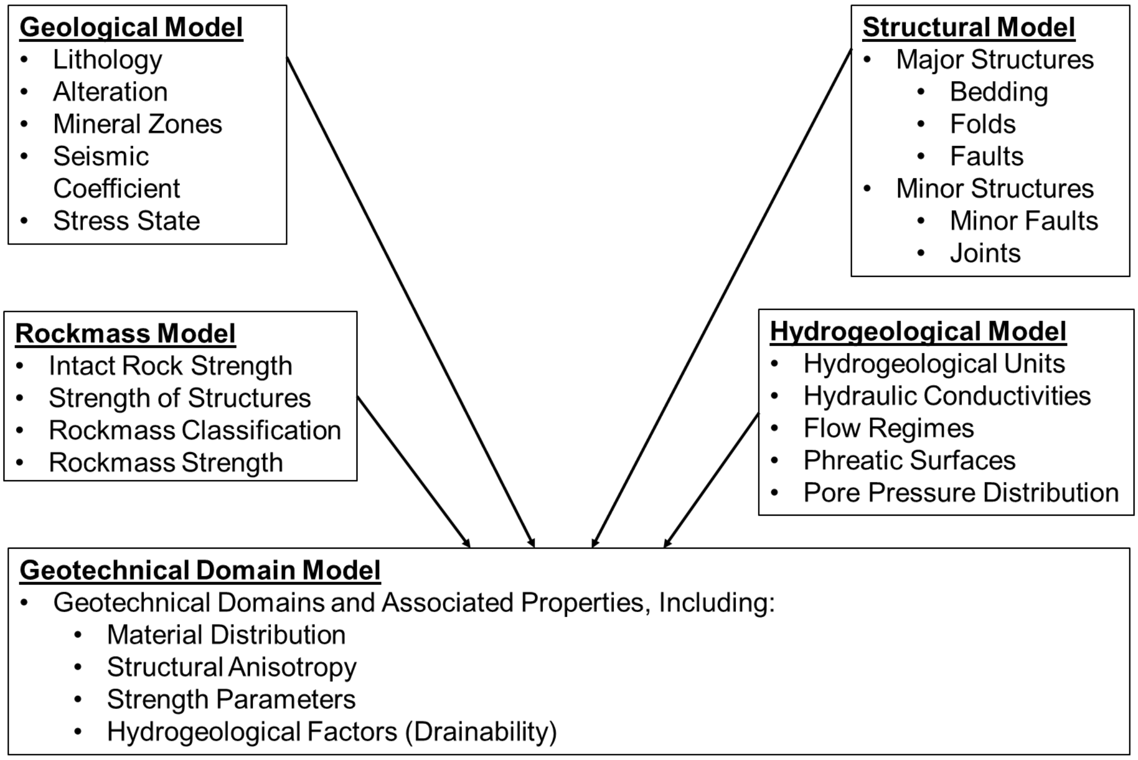


Figure 36 Development of Geotechnical Domain Model after Read and Stacey (2009)

5.1 Anticipated Pit Slope Instability Mechanisms

Understanding the scale and mode of instability forms a crucial component of the geotechnical model. Modes of instability can be classified as either kinematic (structural) or rock mass, and in open cut excavations may occur independently or in combination (multi-modal). The following sub-sections outline the instability mechanisms that are likely to be encountered at the Woori Yallock quarry. The sub sections below outline the instability mechanisms that are likely to be encountered within the Woori Yallock quarry area.

5.1.1 Kinematic (Structural)

Kinematic stability is governed by the characteristics of structural defects (e.g., length of defect) in relation to the orientation and design of corresponding batter geometries. Slopes that contain structural defects are generally susceptible to multiple types of kinematic instability e.g., planar and wedge sliding, however, it is the critical instability mode that will manifest first. The kinematic instability modes likely to be present within the Woori Yallock quarry area are likely to manifest as the following:

– Planar sliding
– Tetrahedral wedge sliding
– Block/flexural toppling

It is common practice to identify the critical instability mode, along with secondary instabilities modes using stereonet projections, the interpretation of which is described in more detail below.

5.1.1.1 Interpretation of Stereonet Projections

The orientation i.e., dip angle and dip direction of structural defects (discontinuities) which have been logged/mapped are plotted as lower hemispheric 'poles' relative to a given slope orientation. Multiple poles with similar orientations i.e., poles that are clustered together, may be represented by a singular mean value (set) from which statistical parameters that define the variability in orientations can be derived. An example of a lower hemisphere stereonet projection highlighting the key features is presented in Figure 37.

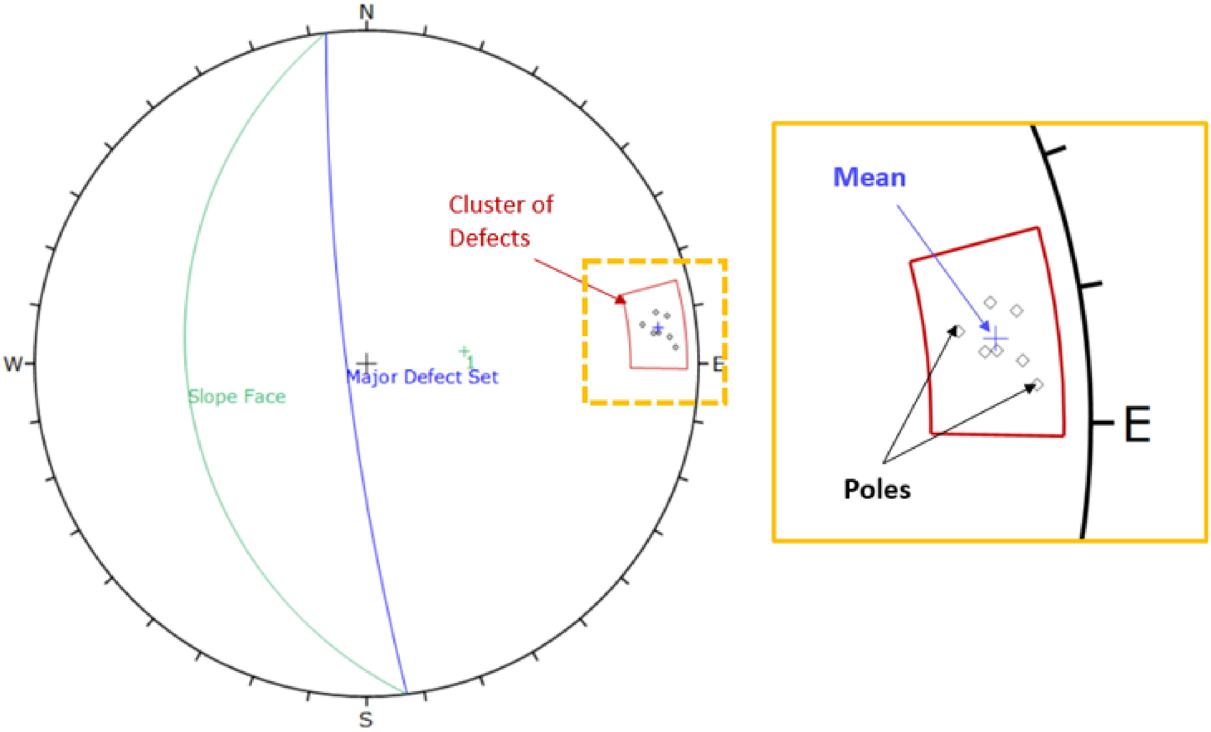


Figure 37 *Stereonet Depicting Key Features*

The Fisher Distribution is commonly used for modelling the distribution of 3-dimensional orientation vectors such as the distribution of defect orientations on a sphere. A Fisher distribution describes the angular distribution of orientations about a mean value and is symmetric about the mean. The Fisher 'K' constant describes the tightness or dispersion of a clustered set, where a larger value implies a tighter cluster and vice versa. The Fisher K constant can be expressed by the following:

$$\theta = \frac{(81^\circ)}{\sqrt{K}}$$

Where:

K = Fisher constant

θ = Angular standard deviation

Rocscience's Dips software used to complete stereographic analysis, automatically calculates the Fisher K constant (or angular standard deviation) for a defined cluster. The Fisher K constant provides a statistical parameter for downstream probabilistic analyses e.g., assessing the likelihood for planar sliding to occur.

5.1.1.2 Planar Sliding

Planar sliding instability occurs when structural defects e.g., bedding planes, which are sub-parallel to the slope face become exposed during mining development and 'daylight' in the slope face. Figure 38 depicts a kinematically feasible instability where the angle of the instability plane (ΨA) dips at a flatter angle than the slope face (Ψf) – plane 'A-A' (ΨA< Ψf).

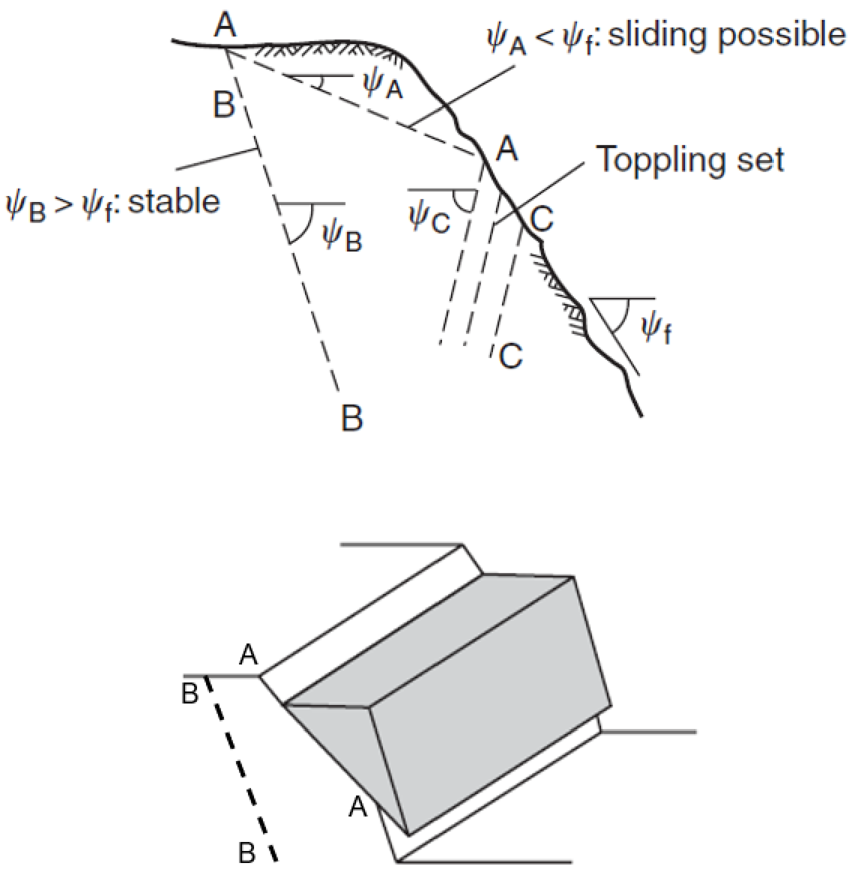


Figure 38 Planar Sliding Instability Schematic

Conversely, planar sliding cannot occur when the structural planes e.g., ‘B-B’ dips at a steeper angle than the slope face (ΨB < Ψf) and does not daylight. Similarly, planes that are not orientated towards the direction of excavation e.g., plane ‘C-C’, are not feasible for planar sliding to occur. In addition to the orientation of structural defects, there are two mechanical principles that also govern the kinematic feasibility for planar sliding, which are:

1. The dip angle of the structural discontinuity must be greater than the angle of friction of the discontinuity i.e., the mobilised shear resistance, referred to as the friction cone (Φ). As the friction angle of the discontinuity surface diminishes the feasibility for planar sliding increases.
2. The dip direction of the structural discontinuity plane cannot differ from the dip direction of the slope face by more than approximately 20°, because under these conditions there will be an increasing thickness of intact rock at one end of the block, which is sufficient for resisting block movement.

The mechanical principles discussed above are used to define a ‘zone of feasibility’ on stereonet projections, which are commonly referred to as the ‘daylight envelope’. The daylight envelope for planar sliding is highlighted for a 60° (red) and 80° (blue) slope face in Figure 39.

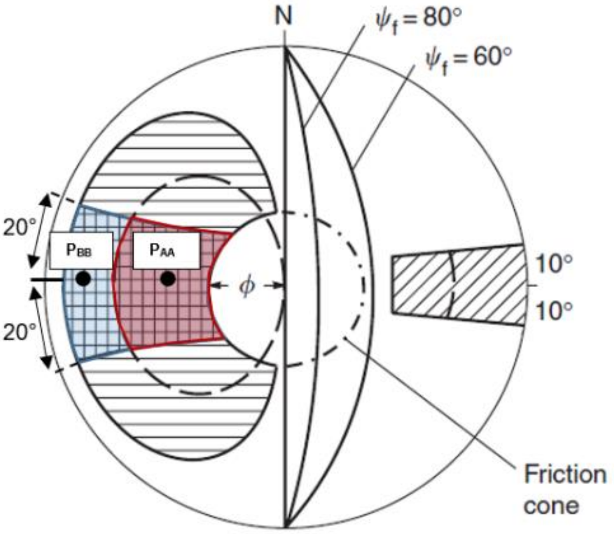


Figure 39 *Stereographic Projection Depicting Daylight Envelope and Friction Cone – Planar Sliding*

The discontinuity plane 'A-A', which was used to depict planar sliding in Figure 38 is represented in Figure 39 as pole 'PAA'. Pole 'PAA' clearly lies within the daylight envelope for the 60° slope face and therefore planar sliding is feasible. Pole 'PBB', used to represent discontinuity plane 'B-B', lies outside of the daylight envelope for a 60° slope face and therefore planar sliding is rendered infeasible. Both pole 'P_{AA}' and 'P_{BB}' lie within the daylight envelope for an 80° slope face and therefore is susceptible to planar sliding along two discontinuity planes i.e., in this example the slope becomes more susceptible to planar sliding when increased from 60° to 80°.

5.1.1.3 Tetrahedral Wedge Sliding

From a kinematic perspective, the formation of wedge type instabilities requires a specific occurrence of structural conditions to become kinematically feasible, which include:

– The dip of the discontinuity planes is flatter than the angle of the slope face.
– The dip of at least one discontinuity plane is greater than friction angle of the discontinuity surface.
– Two or more planes of discontinuity intersect the slope face.

A generalised depiction of tetrahedral wedge sliding is presented in Figure 40 along with the corresponding stereographic projection.

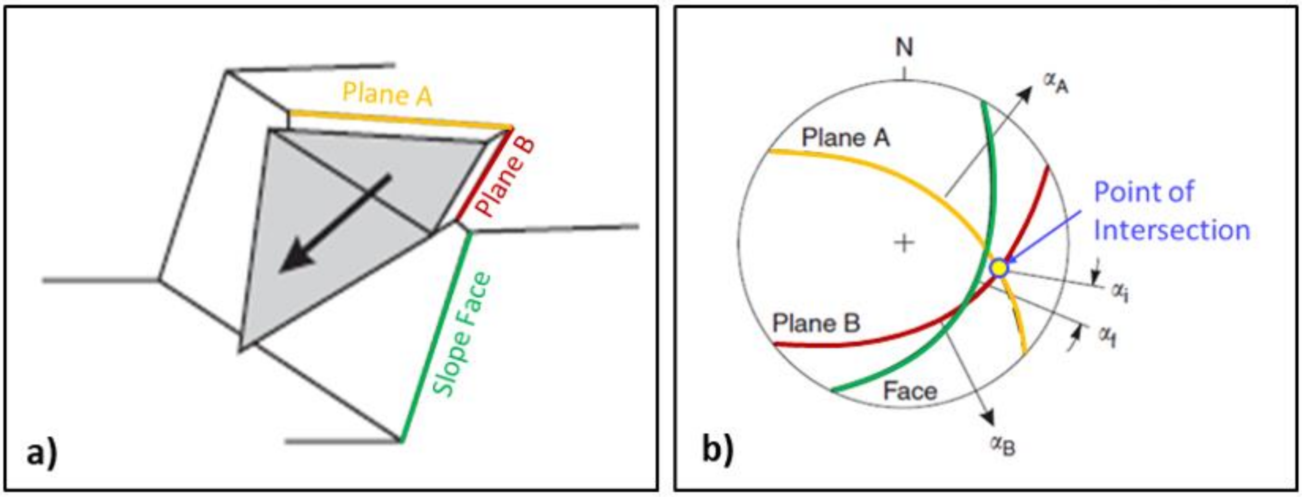


Figure 40 *Schematic of a) Kinematically Feasible Wedge, and b) Corresponding Stereographic Projection*

Given that a wedge is formed by the intersection of at least two discontinuity planes, the direction of sliding i.e., the daylight envelope, is less restrictive than that of planar failures because there are two planes that form release surfaces. The daylight envelope for wedge formations is the locus of all poles representing lines of intersection whose dip directions lie in the plane of the slope face as shown in Figure 41.

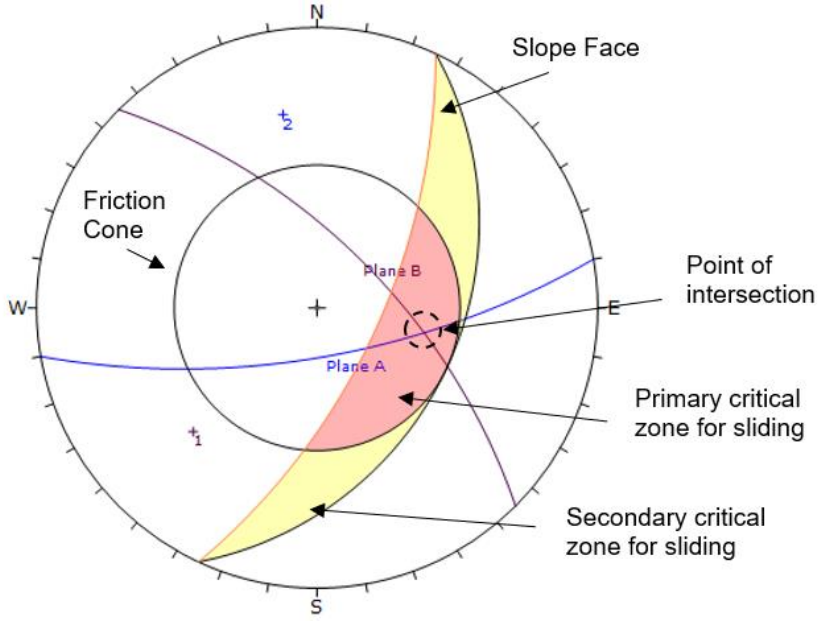


Figure 41 Stereographic Projection Depicting Daylight Envelope – Wedge Sliding

As shown in Figure 41, there are two daylight envelopes, which are the primary (red) and secondary (yellow) critical zones for wedge sliding. The primary difference between the two zones is that in the primary critical zone of sliding, a wedge may slide along the line of intersection i.e., along two defect planes, or on one defect plane; and the critical secondary zone of sliding represents wedges which slide on only one defect plane, noting that the dip vector for sliding on a single plane must be in the primary critical zone.

5.1.1.4 Toppling

There are several kinds of toppling mechanisms that may result in slope instability, which typically include, but are not limited to:

– Flexural
– Block toppling
– Block-flexural toppling

Each of the above toppling mechanisms are pictorially presented in Figure 42.

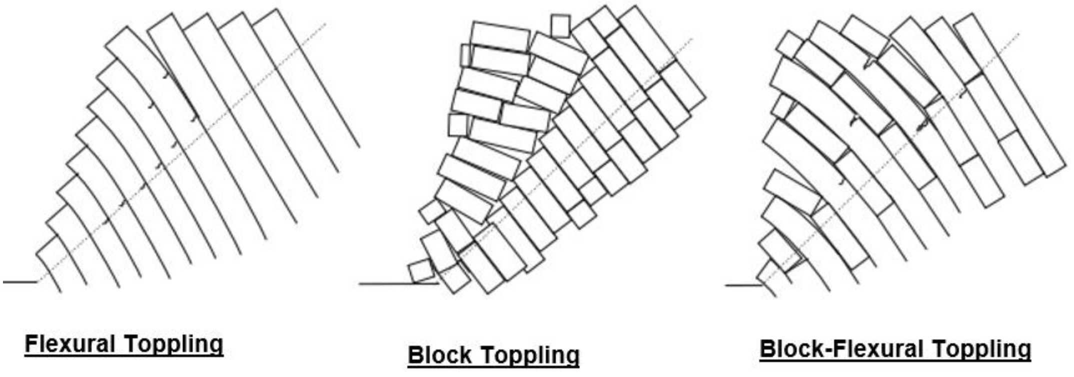


Figure 42 Schematic of Primary Toppling Mechanisms

5.1.1.4.1 Flexural Toppling

Rocks with a pervasive fabric e.g., bedding planes, or a preferred discontinuity system, orientated to present a rock slope with semi-continuous cantilever beams have the potential to result in a flexural toppling type instability.

In such instances, columns formed by sub-vertical structures break in flexure as they bend forward, where characteristics of the defect system and intact rock mass strength govern the potential for toppling to occur. According to Goodman (1980), blocks cannot topple if they cannot slide with respect to one another, and for a slip to occur, the discontinuity system must have a dip angle that is less than a line inclined at an angle equivalent to the friction angle above the slope, which is termed the 'slip limit'. Additionally, flexural toppling cannot occur when the dip direction of the discontinuity system differs from the slope orientation by more approximately 10° to 15°. A daylight envelope showing the zone for which the flexural toppling mechanism is feasible is presented in Figure 43.

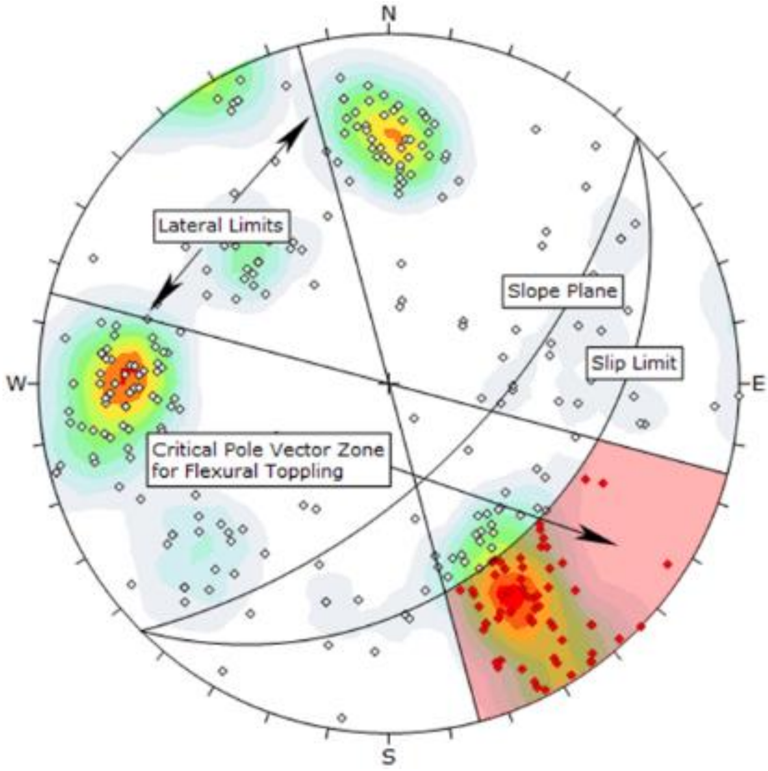


Figure 43 Stereographic Projection Depicting Daylight Envelope – Flexural Toppling

5.1.1.4.2 Block Toppling

Block toppling occurs where rock columns created by sub-vertical defect systems, are intersected by cross-cutting defect systems. At the toe of the slope where the columns are shorter, load from upper (and larger) columns thrust toe columns forward permitting further toppling. In order for block toppling to be kinematically feasible, two discontinuity systems must intersection such that the intersection line dips into the slope and can form discrete toppling blocks, and a third discontinuity system e.g., cross cutting plane, that acts a release plane or release and sliding plane. Similarly, to flexural toppling, the feasibility of block topping to occur requires the dip direction of intersecting discontinuity systems to occur within approximately 10° to 15° of the slope dip direction. A daylight envelope showing the zone for which block toppling instability mechanism is feasible is presented in Figure 44.

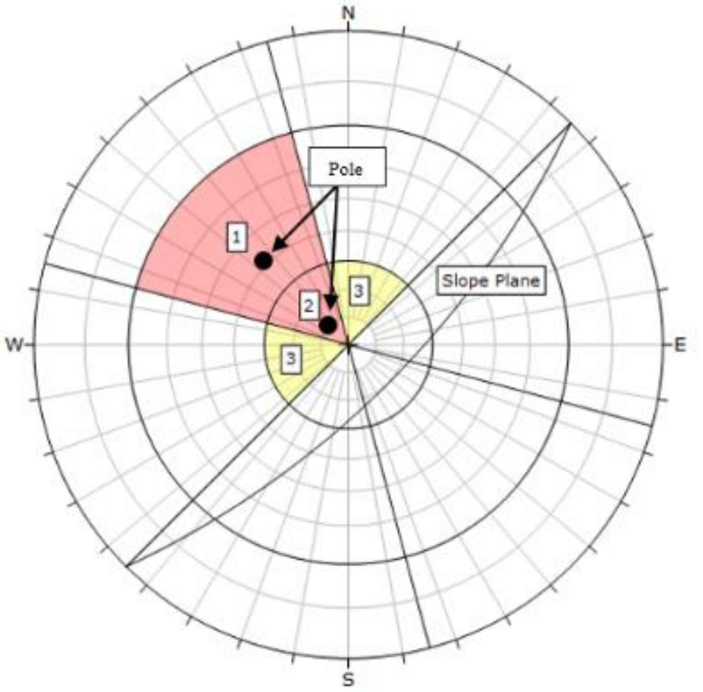


Figure 44 Stereographic Projection Depicting Daylight Envelope – Block Toppling

As shown in Figure 44 above there are three critical zones that make up the daylight envelope. Critical zones 1 and 2, highlighted in red, represent the feasibility of direct block toppling formation. Critical zones marked 3, highlighted in yellow, represent the feasibility of oblique block toppling formation. Any poles which fall into the combined region of zones 2 and 3 represent cross-cutting planes that act as release planes, however, do not permit sliding as the dip angle of the defect plane is less than the mobilised shear resistance. On the other hand, poles that fall into zone 1 represent release planes for which sliding can occur, where a combined sliding and toppling mechanism may manifest.

5.1.1.4.3 Block Flexural Toppling

The two mechanisms presented above, flexural toppling and block toppling, are idealistic mechanisms which require idealistic conditions to occur and are seldom observed in practice. The block-flexural toppling mechanisms is a combination of flexural and block toppling instability modes. In block-flexural toppling, some blocks fail due to tensile bending stress, and some separate from the cross-cutting defect system, at which point all blocks topple together. The block-flexural toppling model accounts for irregularities in the persistence and orientation of cross-cutting joints that serve as release or release and sliding planes. Whilst block-flexural toppling is more readily observed in practice, most methods of toppling assessment do not consider block-flexural toppling and only consider the above idealistic conditions.

5.1.2 Rock Mass Instability

Rock mass stability is governed by the characteristics of both intact rock mass and structural discontinuities. Where kinematic mechanisms typically control the stability at the bench to inter-ramp scale, rock mass mechanisms control the stability at the inter-ramp and global scales. Assessment of the rock mass stability is an essential step in the design process, to check that the rock mass can sustain the proposed design over the full height of the slope. Rock mass models such as the Generalised Hoek Brown Criterion, assume that a rock mass is comprises an isotropic clump of intact rock pieces separated by closely spaced joints for which there is no preferred failure direction. Rock mass that has a pervasive fabric e.g., bedding planes, the strength of the rock mass may be notably lower in the direction of the discontinuity system.

5.1.3 Circular/Rotational Instability

Circular/rotational instability typically occurs in highly disturbed and/or weathered material that typically does not have any remnant structure (see Figure 45).

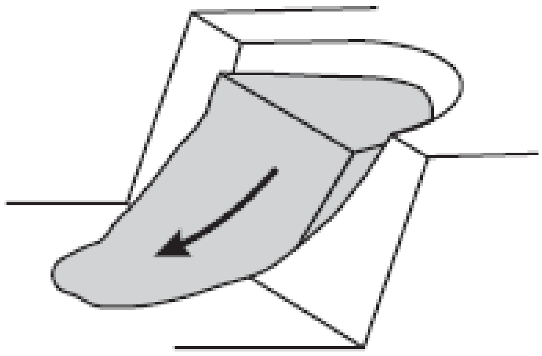


Figure 45 Schematic of a Circular Instability

Circular instability is dependent upon the shear strength characteristic of the highly weathered material and the slope angle of the cut face. Circular instability occurring as potential instability mechanism within the regolith and soft rock units where there is no discernible structure present. Mobilisation of a circular failure is likely to be contained to bench scale instability.

5.1.4 Structural Analyses

A fundamental component of the structural model and thus the geotechnical domain model is the orientation characteristics of critical defect structures relative to pit walls. As outlined in Section 3.2, targeted defect mapping was undertaken as confirmatory exercise to compare defects exposed on current quarry slopes compared to those previously mapped by GHD (2007) and GHD (2009). Presented in Figure 46 below is a stereonet projection depicting the contemporary and past defect orientations mapped at the Woori Yallock Site.

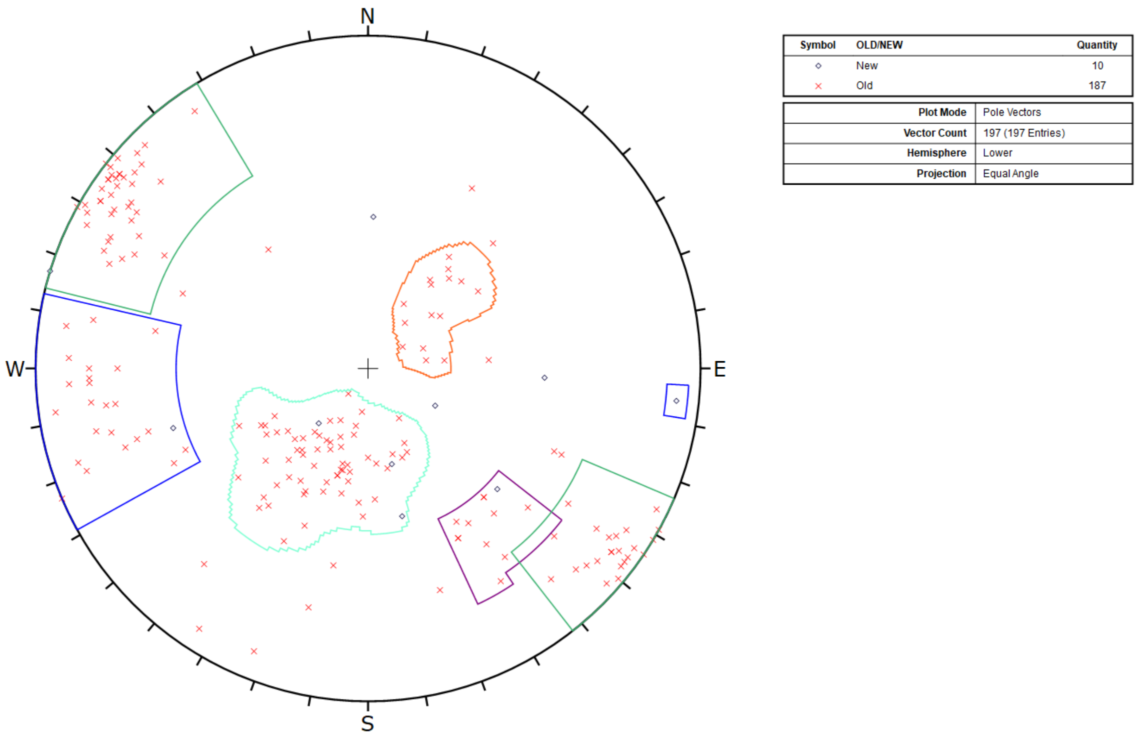


Figure 46 Stereonet Projection Depicting Contemporary and Past Mapped Defect Plane Orientations

As shown in Figure 46, the defects mapped as part of the February 2022 mapping exercise are largely in agreeance with those previously mapped by GHD (2007) and (2009). Accordingly, defect sets were re-interpreted to incorporate the additional information. Summarised in Table 14 are the revised defect set characteristics and presented in Figure 47 is the corresponding stereographic projection.

Table 14 Summary of Revised Defect Set Orientation Characteristics

Discontinuity Set	Dip Angle (°)	Dip Direction (°)	Fisher Constant (K)	Description
S1	88	125	74.25	Bedding
S2	78	82	36.82	Bedding
S3	34	25	24.35	Major structural set
S4	30	228	39.71	Major structural set
S5	61	323	85.25	Bedding

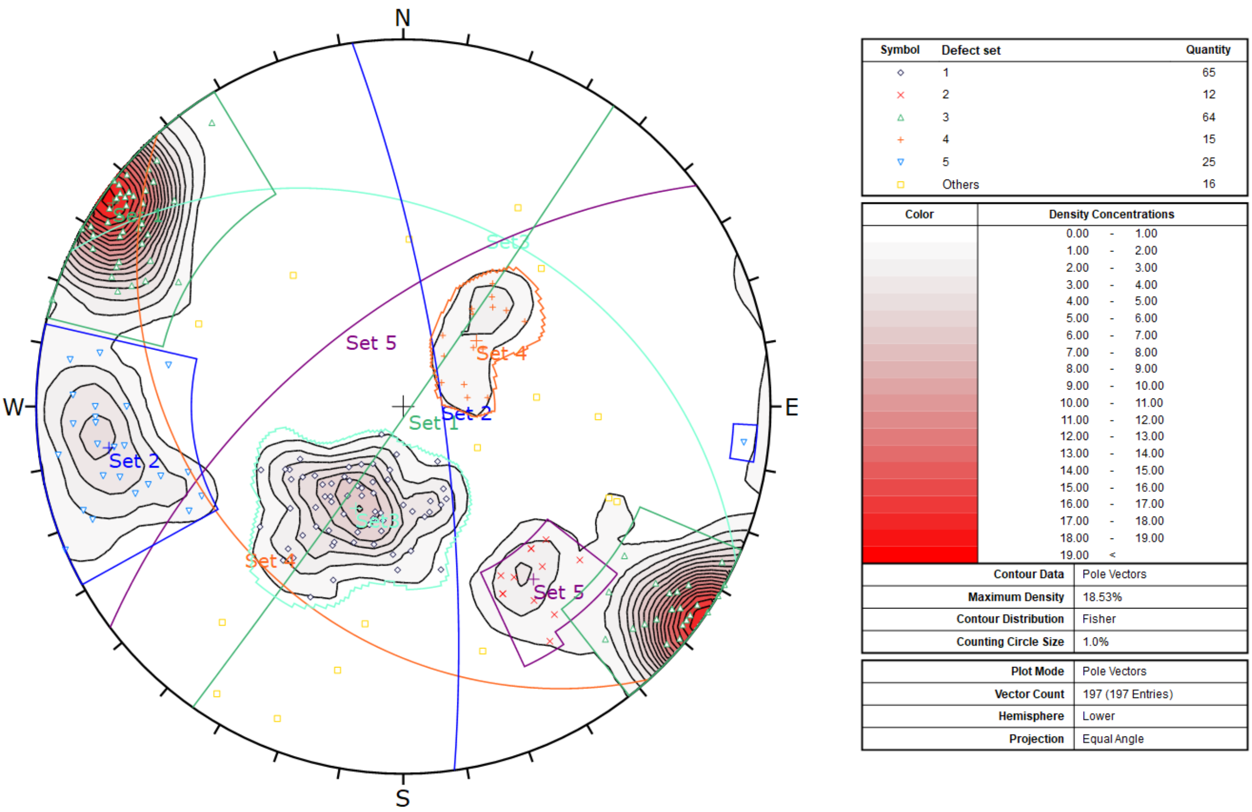


Figure 47 Revised Stereographic Projection Incorporating February 2022 Defect Mapping

5.2 Interpretation of Material Strength Properties

As outlined in 3.1.2 five (5) stratigraphic units have been observed within the Woori Yallock quarry. In order to adequately define the strength characteristics of each unit, a suitable strength criterion should be selected in sympathy to the rock mass characteristics (Hoek and Brown, 2019). Where there is discernible structure within a rock mass, adoption of the Mohr Coulomb criterion can result in an overestimation in the rock mass strength compared to Generalised Hoek Brown (GHB) criterion is adopted, particularly under low normal stresses (or confinement) (Hoek, 1994). Each strength criterion requires unique strength parameters to derive shear strength characteristic curves i.e., the relationship between normal stress and shear stress. Summarised below in Table 15 is the strength criterion type along with the strength parameters required for each material type.

Table 15 Summary of Strength Criterion and Parameters by Material Type

Unit	Criterion	Required Strength Parameters
1	Mohr Coulomb	Cohesion (c') and Friction Angle (ϕ')
2 - 5	Generalised Hoek-Brown	Uniaxial Compressive Strength (σ_c), Geological Strength Index (GSI), Material Constant (m_i) and Disturbance Factor (D)

For each material type and associated strength criterion, strength parameters were obtained through either field estimates, site observations obtained during the December 2021 site inspection, and other available data. Each criterion and the material parameters adopted for downstream stability analyses are discussed further in the following subsections.

5.2.1 Soil Strength

According to the Soil Landscape Grid of Australia (SGLA, 2017) the regolith within the Woori Yallock quarry area is comprised predominantly of clays with minor proportions of silts and sands. Based on observations made on site and experience in dealing with similar materials, the material strength characteristics, represented by MC parameters adopted for stability analyses are presented in Table 16.

Table 16 Summary of Mohr Coulomb Parameters

Unit	Description	Cohesion – c' (kPa)	Friction Angle - ϕ' (°)
1	Overburden	5	28

5.2.2 Rock Mass Strength

Rock mass strength characteristics in the form of the Generalised Hoek Brown criterion were derived for units 2 to and 5 using RocScience's software package RocData v.5003. RocData provides a shear-normal function based on the GHB stress envelope, which is described by a set of empirical equations defined by a series of rock mass failure parameters, which are:

– Geological Strength Index (GSI) - describes the relationship between a rocks structure and surface quality, both of which are related to rock strength. The selection of GSI values for the Woori Yallock Quarry site are outlined in the following sub sections.
– σ_{ci} - Uniaxial Compressive Strength (UCS) – determined from strength indices in accordance with the ISRM (1981).
– m_i – a material constant for intact rock mass which is defined as the ratio between UCS and Uniaxial Tensile Strength.
– D - Disturbance factor, which is related to the degree of disturbance to which the rock mass has been subjected by blast damage. It varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed.

5.2.2.1 Selection of GSI values

Presented below in Figure 48 are the possible GSI ranges for Units 2 to 5 according to observations made on site after Hoek and Marinos (2000).

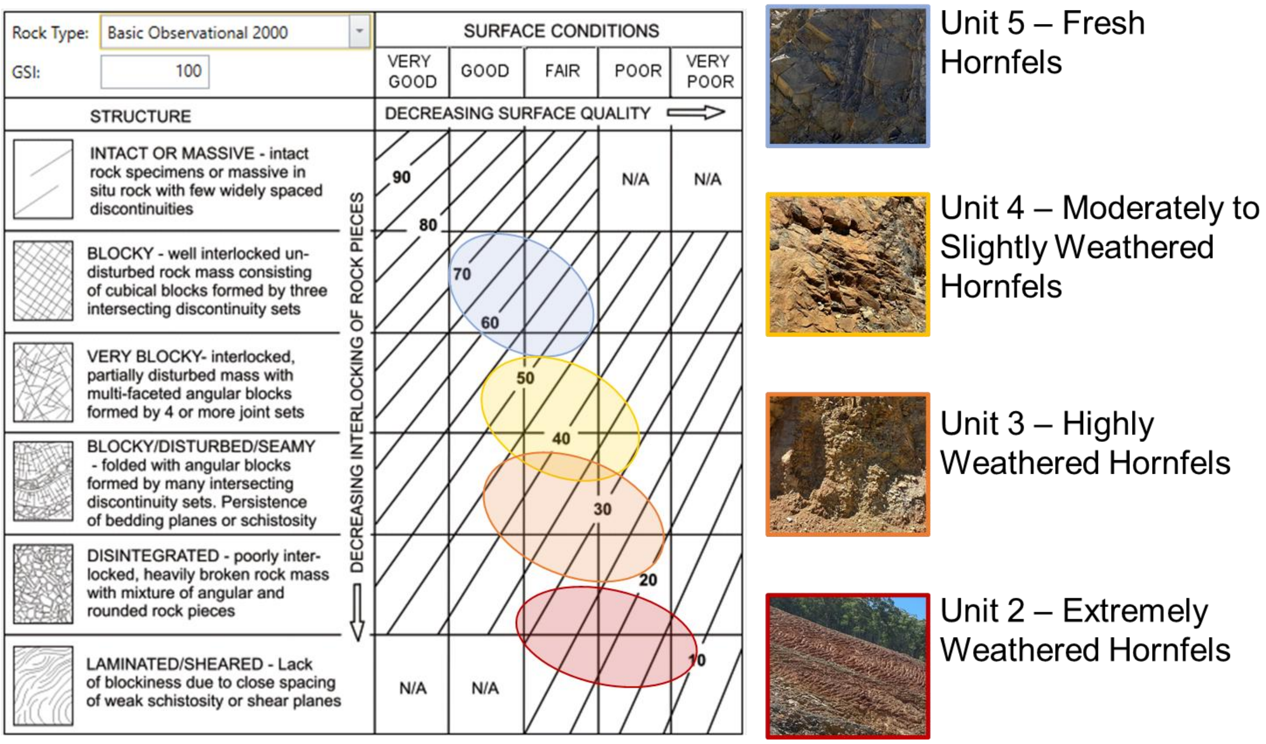


Figure 48 ***GSI Values for Woori Yallock Stratigraphic Units***

For the purpose of this assessment, lower bound GSI values have been conservatively selected for each stratigraphic unit as well as the intense zones of weathering/fracturing, the values of which are summarised in Table 17.

Table 17 ***Summary of GSI Values***

Unit	Description	GSI Value
2	Extremely Weathered (EW) Hornfels	10
3	Highly Weathered (HW) Hornfels	20
4	Moderately to Slightly Weathered (MW to SW) Hornfels	30
5	Fresh (Fr) Hornfels	45

5.2.2.2 Field Strength Index

Field strength index for Units 2 and 5 were obtained during the site visit in line with the ISRM (1981) classification system (Table 18).

Table 18 ***Summary of Intact Rock Mass Properties after ISRM (1981)***

ISRM R Strength	Description	Strength Code	Field Identification	Uniaxial Compressive Strength (MPa)
R0	Extremely Weak Rock	EW	Indented by thumbnail	<1
R1	Very Weak Rock	VW	Crumbles under firm blows with a geological hammer	1 to 5
R2	Weak Rock	W	Can be peeled with a pocketknife, shallow indentations made by firm blow of geological hammer	5 to 25
R3	Medium Strong Rock	MS	Cannot be scraped or peeled with a pocketknife, specimen can be fractured	25 to 50

ISRM R Strength	Description	Strength Code	Field Identification	Uniaxial Compressive Strength (MPa)
			with a single firm blow of a geological hammer	
R4	Strong Rock	S	Specimen requires more than one blow of a geological hammer to fracture	50 to 100
R5	Very Strong Rock	VS	Specimen requires several blows of a geological hammer to fracture	100 to 250
R6	Extremely Strong Rock	ES	Specimen can only be chipped with a geological hammer	>250

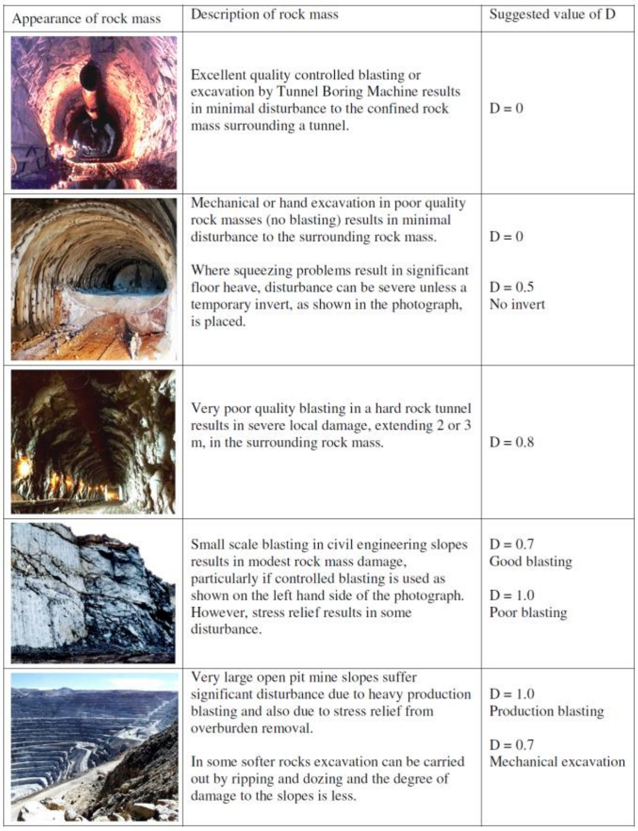
The logged Field Strength Indices for each unit are summarised below in Table 19. It should be noted that corollary Uniaxial Compressive Strengths were conservatively selected for the Woori Yallock Quarry site.

Table 19 ***Summary of Logged Field Strength Estimates***

Unit	Description	Strength Grade (ISRM, 1981)	Field Strength Estimate of UCS (MPa)
2	EW Hornfels	R1	2.5
3	HW Hornfels	R2	15
4	MW to SW Hornfels	R4	75
5	Fr Hornfels	R5	175

5.2.2.3 Disturbance Factor (D)

Guidelines for estimating Disturbance (D) factor is provided in Figure 49 (Hoek *et al*., 2002). A D-factor of 0.7 was adopted for all units, which assumes good blasting practice is maintained, which is considered conservative as the extent of blast damage is spatially dependant and reduces with depth behind the slope.



Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	D = 0 D = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	D = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	D = 0.7 Good blasting D = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	D = 1.0 Production blasting D = 0.7 Mechanical excavation

Figure 49 ***Guidelines for Estimating Disturbance (D) Factor After Hoek et al. (2002)***

5.2.2.4 Generalised Hoek Brown (GHB) Strength Parameters

GHB parameters were derived for each of the rock units to produce characteristic shear strength curves utilising the information in the proceeding sections. Summarised in Table 20 are the interpreted GHB parameters for the in-situ rock units.

Table 20 Summary of GHB Parameters

Unit	Description	UCS (MPa)	mb	s	a
2	EW Hornfels	2.5	0.763	4.54E-05	0.585
3	HW Hornfels	15	0.234	9.22E-06	0.544
4	MW to SW Hornfels	75	0.214	3.93E-05	0.522
5	Fr Hornfels	175	0.341	3.45E-04	0.508
UCS = Uniaxial Compressive Strength					

5.2.2.5 Anisotropy in Rock Mass Strength

To account for the presence of pervasive rock fabric (bedding) and its effects on the strength of intact rock mass, the Snowden Modified Anisotropic Linear Model (SMALM) was adopted. The SMALM allows the user to define non-symmetric anisotropy using the parameters A1, A2, B1 and B2, which are pictorially presented in Figure 50.

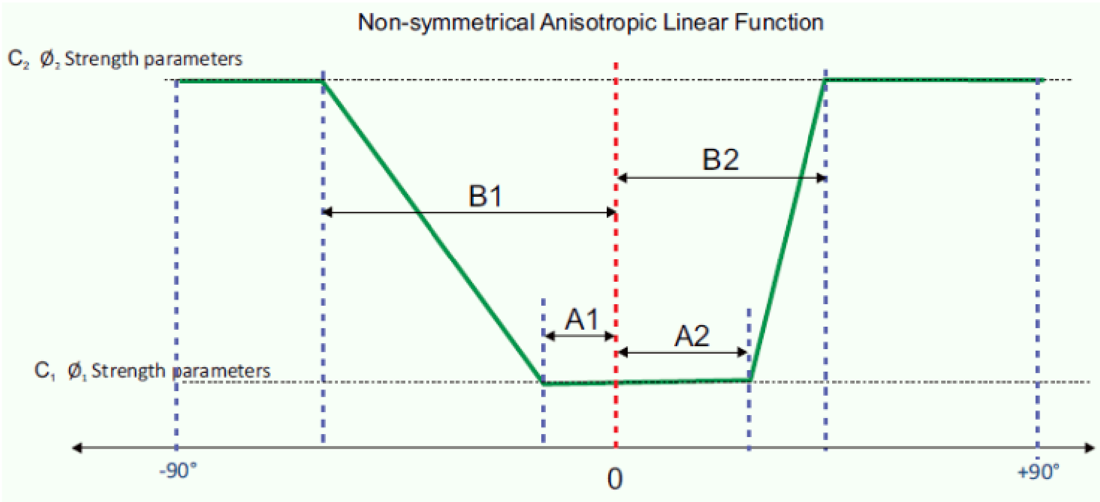


Figure 50 Non-symmetric Anisotropic Linear Function

The model assumes that minimum shear strength of the rock mass occurs in the direction of the discontinuity planes where:

- The parameter ‘A’ defines an angular range on either side of the discontinuity plane orientation, for which the bedding plane shear strength applies.
- The parameter ‘B’ defines the angular range (B-A) over which the increase from discontinuity plane to rock mass shear strength takes place (assumed to be linear).
- For orientations outside of the ‘B’ range only the rock mass shear strength applies.
- In essence, the ‘A’ and ‘B’ parameters allow for the user to describe the transition of material strength between intact rock mass and a discontinuity surface, i.e., bedding plane. The greater the value of the ‘A’ and ‘B’ parameters the greater the effect of the bedding strength on the stability model, where parameter values of 0° for both infer that the strength of the rock mass is entirely dependent on the strength of the intact rock mass.
- For this study the default material parameters of for the ‘A’ and ‘B’ values of 5° and 25° were adopted, respectively.

The dip angle of the discontinuity plane(s) is also accounted for in the SMALM, assuming that the anisotropic layer is constant. The discontinuity plane angle is measured counterclockwise from the horizontal as shown below in Figure 51.

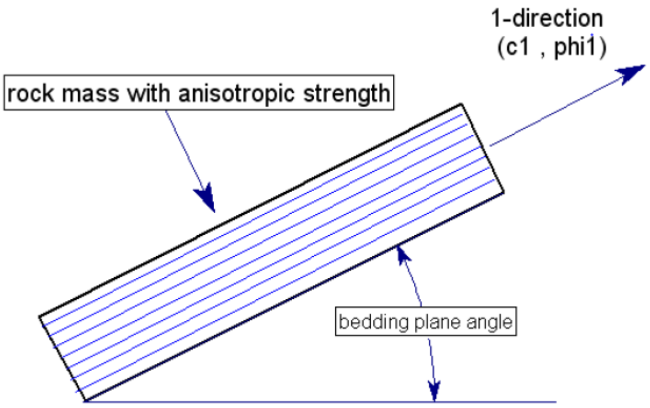


Figure 51 ***Depiction of Discontinuity Plane Angle***

When the primary failure mechanism of a slope is sliding on a discontinuity plane (i.e., the discontinuity angle is upslope of the face), the discontinuity plane angle is positive. On the contrary when the angle of discontinuity is downslope (i.e., toppling) the discontinuity plane angle is negative (Mercer, 2013). Shear strengths adopted for the discontinuity planes are outlined Section 5.2.4.

5.2.3 Shear Strength Characteristic Curves

Shear strength curves for each of the stratigraphic units as observed on site are depicted in Figure 52.

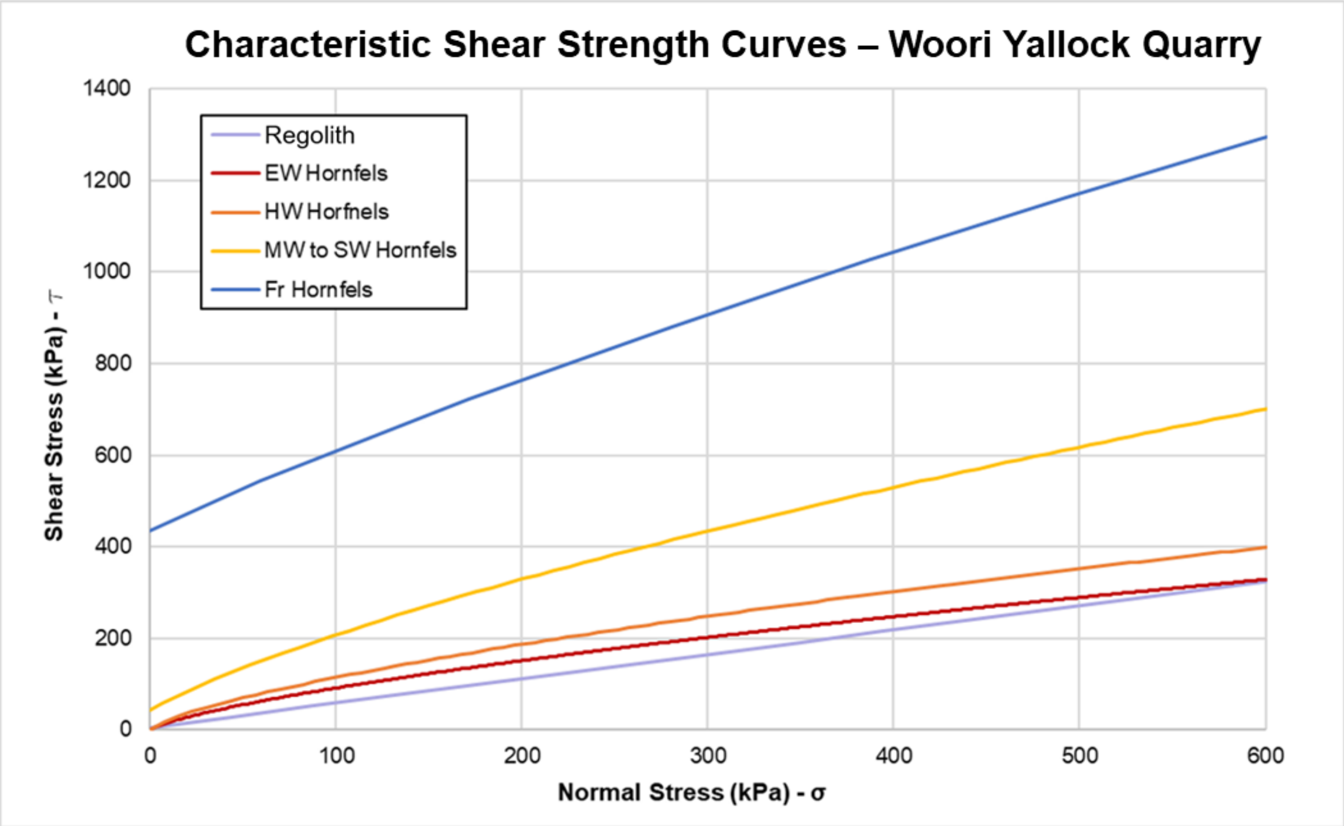


Figure 52 ***Rock Mass Shear Strength Curves – Woori Yallock Quarry***

5.2.4 Defect Plane Strengths

Shear strength parameters for defect planes have been interpreted using data obtained during the February 2022 site investigation. The shear strengths of defect planes were interpreted using the Barton-Bandis (1980) criterion, a non-linear mathematical function specifically derived for modelling the shear strength behaviour of defects in a rock mass. The Barton-Bandis (1980) equation is expressed by the following:

$$\tau = \sigma_n \cdot \tan\left(JRC \cdot \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b\right)$$

Where:

τ = Peak shear strength of a discontinuity

σ_n = Normal stress acting on discontinuity plane

JRC = Joint Roughness Coefficient

JCS = Joint Compressive Strength

ϕ_b = Basic friction angle

5.2.4.1 Joint Compressive Strength (JCS)

The JCS parameter is considered to be the Uniaxial Compressive Strength (UCS) of the rock wall, which is outlined in Table 19.

5.2.4.2 Basic Friction Angle (ϕ_b)

The basic friction angle (ϕ_b) parameter is the friction angle of a flat surface (an intrinsic property of the rock mass). A basic friction angle of 28° has been adopted for the Woori Yallock quarry site, which is based on previous experience in dealing with similar materials.

5.2.4.3 JRC Parameter

The JRC parameter defines the roughness profile of the defect surface which ranges from 0 for smooth, planar, and slickenside surfaces to as much as 20 for rough undulating surfaces. Typical roughness profiles according to Barton and Choubray (1977) and JRC values are presented in Figure 53. Based on the profiles recorded during the site inspection, a JRC value of 4 was considered for the subsequent stability analyses.

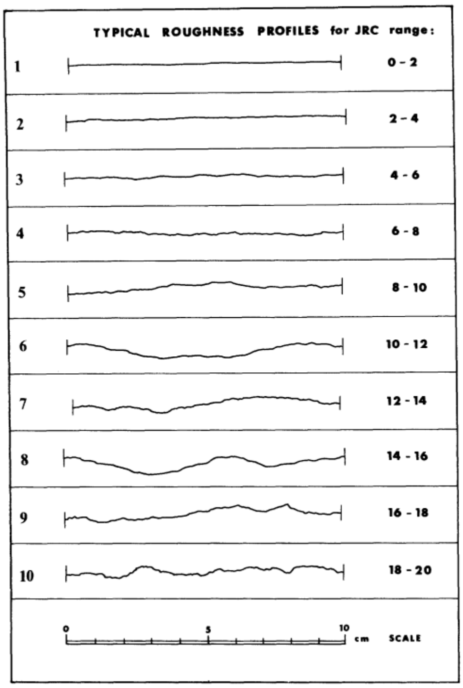


Figure 53 Roughness Profiles and Corresponding Range of JRC Values after Barton and Choubey (1977)

5.2.4.4 Barton and Bandis Parameters

Defect plane strengths only apply to rock mass with discernible structure, and therefore defect plane strengths were derived only for Units 3, 4 and 5. Summarised in Table 10 are the Barton and Bandis parameters adopted for the Woori Yallock quarry site.

Table 21 Summary of Barton and Bandis Defect Plane Strength Parameters

Unit	Description	Joint Compressive Strength – JCS (MPa)	Basic Friction Angle - ϕ_b (°)	Joint Roughness Coefficient - JRC
3	HW Hornfels	15	28	6
4	MW to SW Hornfels	75		
5	Fr Hornfels	175		

5.3 Geological Considerations

Information from the res-def drilling campaign outlined in Section 2.5 was imported into Maptek's Vulcan 2021.5 and using the in-built 'Geology' modelling tool, attribute data including rock type and weathering were used to model 3D surfaces. An example of the modelled surfaces is depicted in Figure 54 for the floor of logged 'Fresh Hornfels'.

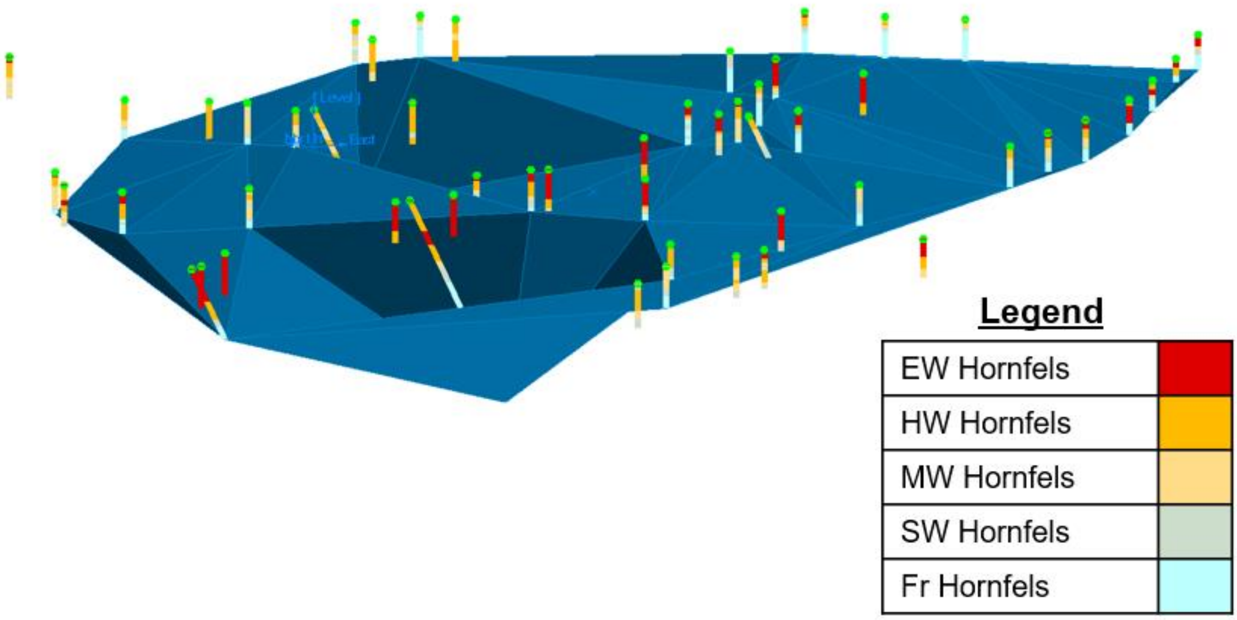


Figure 54 Example Mapped Surface of 3D Geological Surface – Floor Fresh Rock

According to the logged drill data, and as shown in Figure 54, a channel of extremely to highly weathered material exists within the deposit. Such weathering profiles were observed during the February 2022 site inspection as outlined in Section 3.1.2, and are likely to become exposed along terminal wall excavations, particularly along south facing walls, and may occur in the order of 25 m. It is expected that more broadly across the proposed extraction footprint that the exposure of weathered units along terminal walls will occur in the order of 10 m. Geological surfaces interpreted from borehole data have been used to inform downstream stability assessments presented in Section 5.

5.4 Geotechnical Domain Model

A geotechnical domain model has been derived, which is shown below in Table 22, along with the critical instability mechanisms. Table 22 summarises the kinematically feasible instability mechanisms for each of the quarry domains.

Table 22 Summary of Kinematically Feasible Instability Mechanisms for Proposed Woori Yallock Quarry Pit

Domain	Dip Direction (°)	Instability Mechanism
North Domain	220	**Primary:** Planar instability **Secondary:** Wedge, rock mass and composite instability
East Domain	323	**Primary:** Wedge instability **Secondary:** Planar, Toppling, rock mass and composite instability
South Domain	10	**Primary:** Planar instability **Secondary:** Wedge, rock mass and composite instability
West Domain	110	**Primary:** Toppling instability **Secondary:** Composite, planar, wedge, rock mass instability

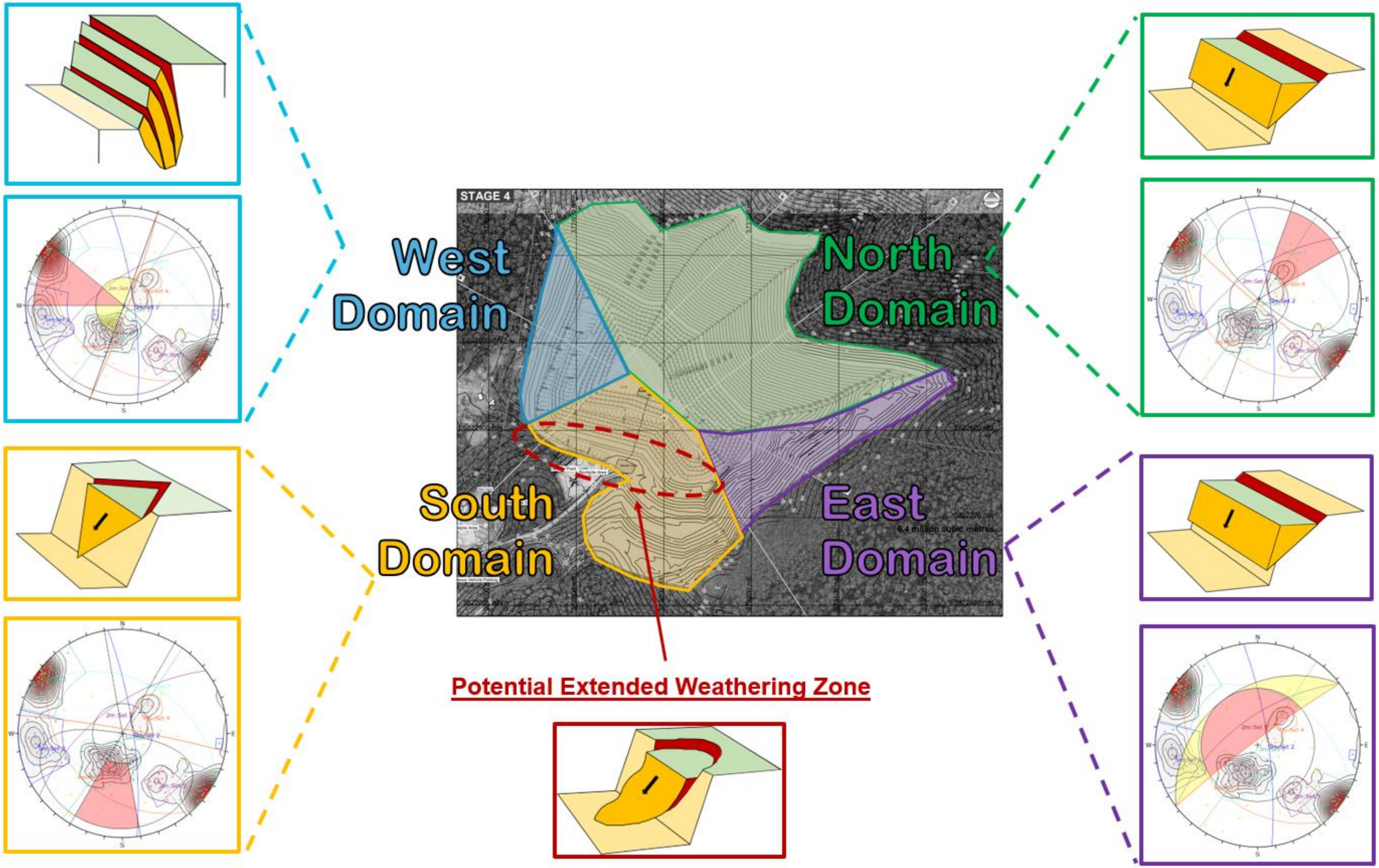


Figure 55 Woori Yallock Geotechnical Domain Model

6. Stability Analyses

Rocscience's limit equilibrium software suite was used to assess the slope stability performance of the proposed Woori Yallock Quarry pit geometry (WA375_FinalPit_Sep2022.DWG). Stability analyses was undertaken to assess:

<u>Terminal Slope Profiles</u>

– The kinematic stability at the individual bench scale, and
– The slope stability at the multi bench and overall slope scales

<u>Rehabilitated Slope Profiles</u>

– The slope stability of rehabilitated slopes at the multi-bench and overall slope scale

Slope stability performance was measured against nominated design acceptance criteria in line with state government guidelines.

Sensitivity analyses was undertaken on terminal and rehabilitated slope profiles to assess the potential stability implications associated with:

– Extended zones of weathering along the slope face for the northern quarry domain
– Seismic loading events
– Elevated phreatic conditions
– Variations in the pit lake level
– Influence of degraded material strengths

Further discussion on each of the above sensitivities is provided in Section 6.6

Table 23 outlines the slope geometrical parameters which were considered as part of the assessment. It is noted that these slope parameters have been iteratively refined based on an iterative design process undertaken in collaboration with BCA. The geometry assessed below has considered double benching below the pit lake level, with the associated stability analyses results presented in Section 6.3.5.

Table 23 Terminal Pit Geometry (Rehabilitated Landform Plan – BCA, 2023)

Slope Design Parameter	Excavated Quarry Batters				Rock Slope
	Regolith / EW Hornfels	HW and MW to SW Hornfels	Fr Hornfels (above pit lake level)	Fr Hornfels (below pit lake level)*	
Bench height (m)	15	5		10	15
Bench width (m)	N/A	7	6	8	3
Bench face angle (°)	38	85	85	80	1V:2H
Geotechnical Decoupling Berm	Yes – Decoupling berm equal to 10 m at the EW to HW Hornfels Horizon				N/A
Beaching Zone	Yes – Beaching Zone equal to 12 m at pit lake shoreline				N/A
Final Pit Lake Level	RL 217 m AHD				
Maximum Overall Slope Height (m)	260				
*Fresh Hornfels unit double benched below the pit lake level					

6.1 Design Acceptance Criteria

The nomination of suitable Design Acceptance Criteria (DAC) is a key part of the assessment process. The basis of nomination for suitable acceptance criteria will need to confirm that the Factor of Safety (FoS), and where applicable the Probability of Failure (PoF) for a particular slope is acceptable. It should be noted that any slope design for open cut extraction necessitates finding a comprise between the risk of slope failure and the economics of mining. Design acceptance criteria for the Woori Yallock quarry site have been nominated in line with accepted industry practice as outlined in the ERR 'Geotechnical Guideline for Terminal and Rehabilitated Slopes', September 2020, and published precedents as outlined in CSIRO's 'Guidelines for Open Pit Slope Design', (Stacey and Read, 2009), as follows:

– Terminal slopes (overall slope scale) – $FoS_{STATIC} > 1.6$
– Rehabilitated slopes (overall slope scale) – $FoS_{STATIC} > 2.0$

In addition to the above the following seismic DAC has been nominated in line with the CSIRO (2009) guidelines:

- $FoS_{SEISMIC} > 1.1$

6.2 Nominated Stability Sections

Three critical stability sections for the proposed quarry pit have been chosen for stability analyses. The bases for nomination are as follows:

– **Section N1** – Represents the maximum overall slope height of the proposed quarry design where wall orientations are in the same or similar the pervasive fabric i.e., rock mass anisotropy.
– **Section S1** – Represents the maximum overall slope of weathered rock stockpile.
– **Section W1** – Intersects and assess the stability of the "Main Dam" located to the west of the pit. For the purpose of the stability analyses, it is considered that the dam is filled.

Figure 56 shows the cross sections in relation to the proposed quarry pit. It should be noted that an additional section (E*) was considered for nomination, however, this stability section was determined to have an overall slope geometry flatter than Section N1 and not orientated in the same direction as the pervasive fabric.

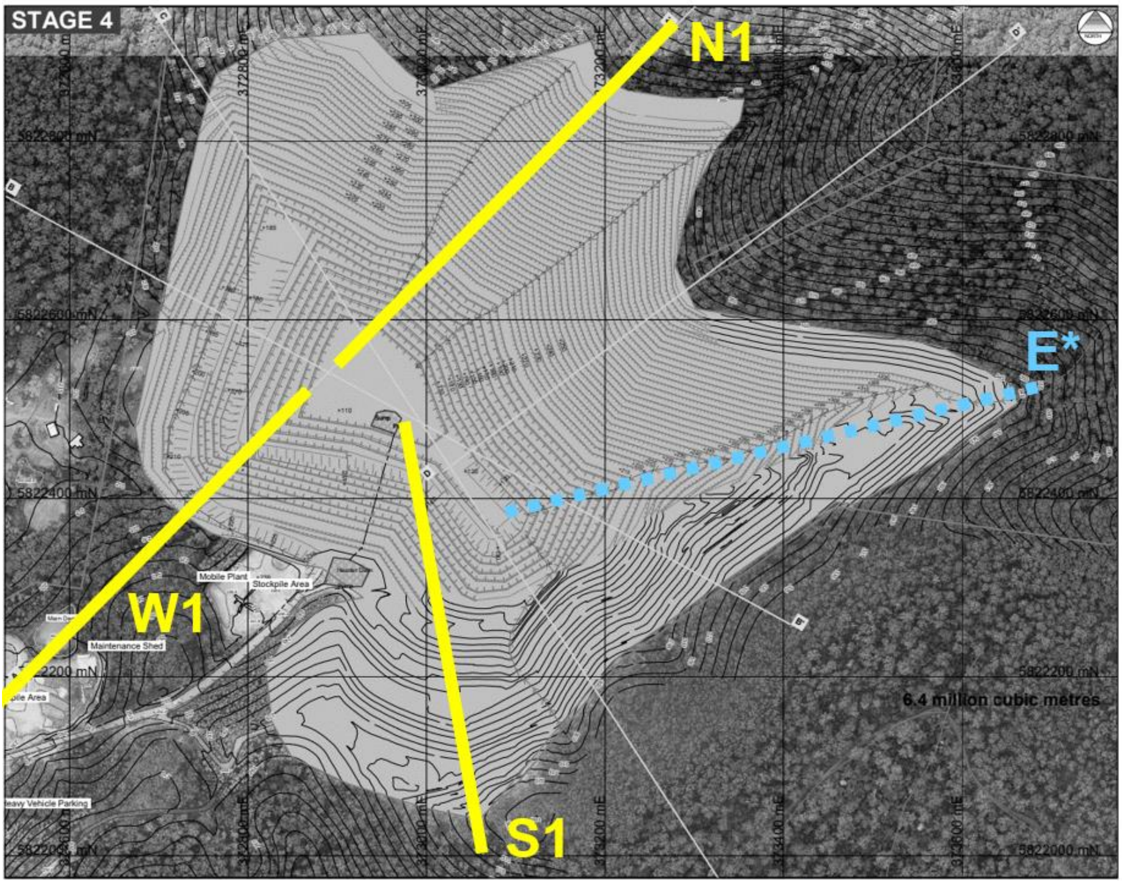


Figure 56 Plan View of Proposed Woori Yallock Pit Depicting Nominated Stability Sections

6.2.1 Slope Stability Model Construction

The construction of a stability model incorporates and combines the elements of the subsurface geology, hydrogeology, material properties and slope design geometry as defined by the geotechnical model. An example cross-section depicting a constructed stability model utilised in the assessment of slope stability is presented in Figure 57, Figure 58 and Figure 59 for Section N1, S1 and W1 respectively.

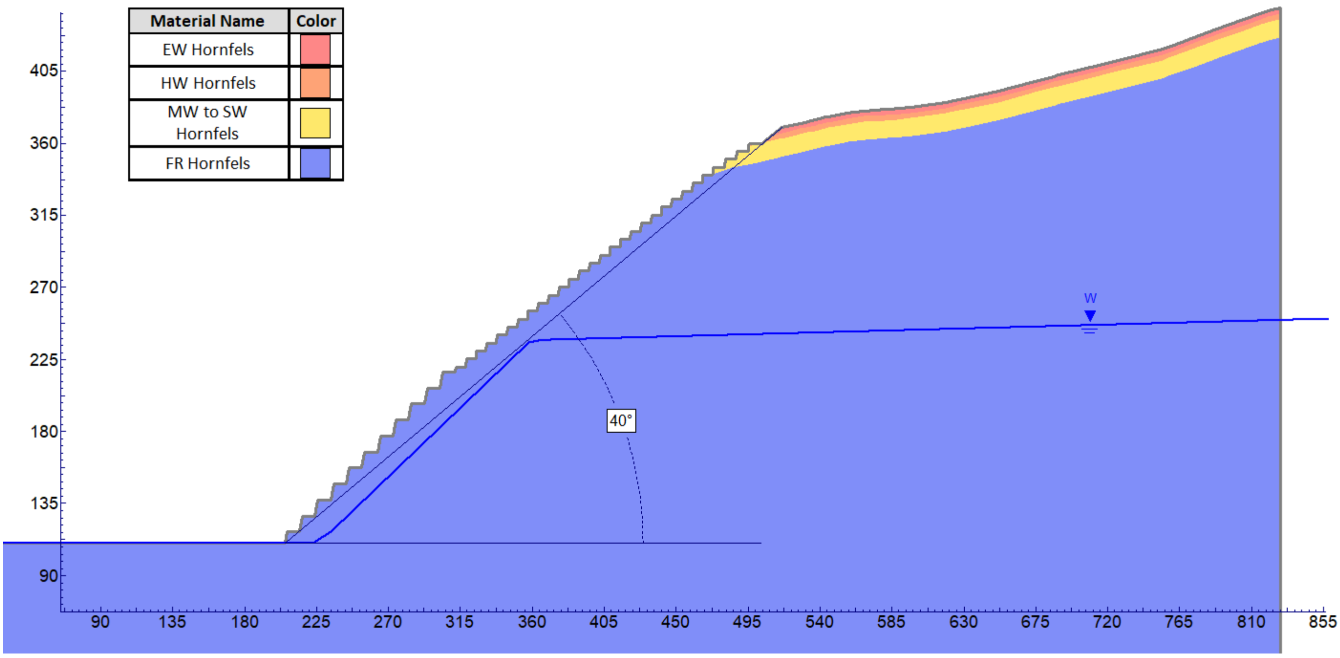


Figure 57 Example of Constructed Slope Stability Model – Section N1

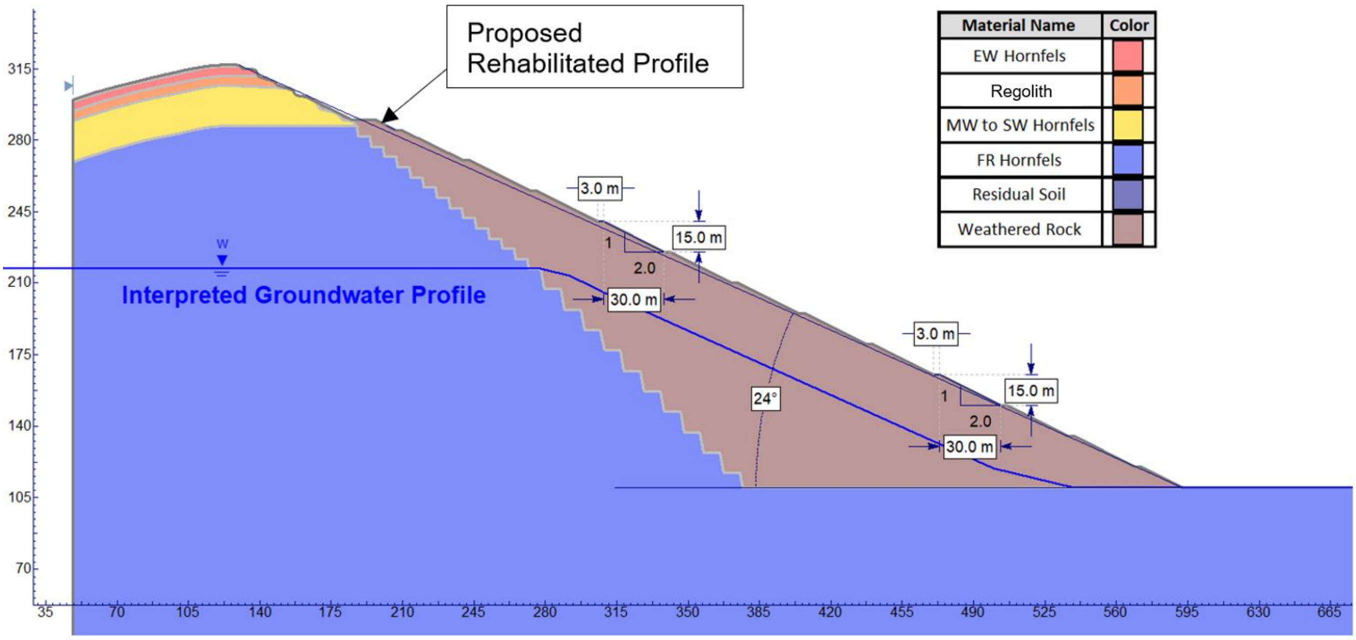


Figure 58 Example of Constructed Slope Stability Model – Section S1

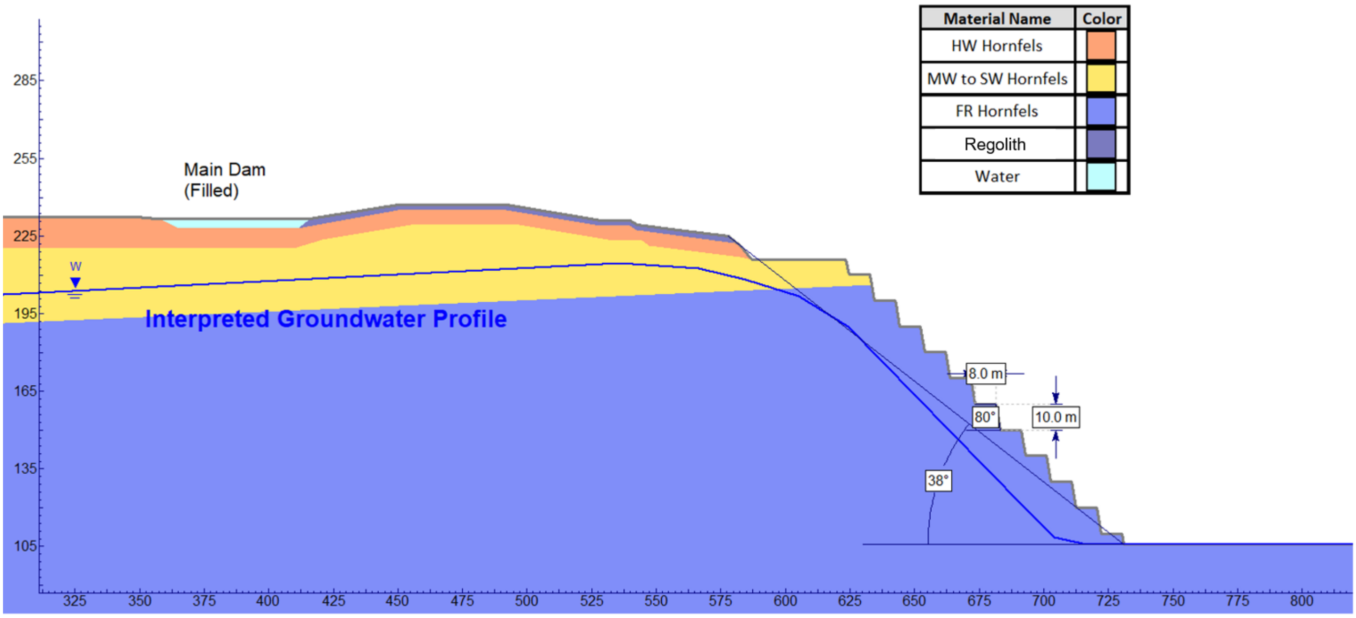


Figure 59 Example of Constructed Slope Stability Model – Section W1

6.3 Kinematic Analyses

When designing slopes in structurally controlled deposits, such is the case at Woori Yallock, it is crucial to ensure that the adopted batter-bench configuration is able to sufficiently catch and retain structural instability, where structural instability is inevitable and must be accounted for in the design process.

Kinematic stability analyses were undertaken to assess the stability performance of the proposed batter-bench configuration using 3D Limit Equilibrium (LE) modelling methods, which takes into account the orientation and design geometry of the slope relative to mapped structural defects. For a given defect set the entire range of possible orientations i.e., angular deviation, can be accounted for facilitating probabilistic analyses. Adopting such an approach allows for both the Factor of Safety (FoS) and Probability of Failure (PoF) to be calculated. Kinematic analyses were undertaken for each stratigraphic unit that has discernible structure i.e., Units 3, 4 and 5.

The following Rocscience LE software was utilised for undertaking individual batter scale assessments:

– RocPlane v 4.007 – Planar sliding.
– Swedge v7.009 – Wedge sliding.
– RocTopple v 2.004– Rock Toppling.

Instability surfaces calculated as part of the kinematic assessment were used to define the critical structure orientations, i.e., dip angle and dip direction, which provide the requisite information to assess minimum bench width requirements, and thus assess the adequacy of the proposed pit geometry. Further details on the calculation of width requirements are outlined below.

6.3.1 Minimum Required Bench Width

The Rocscience kinematic analyses software SWedge and RocPlane, have an in-built bench design tool that allows the user to analyse and assess the suitability of slope design geometry. The bench width design tool is a probabilistic analysis approach making using of the Gibson et al. (2006) equations, which consider both conical- and pyramidal-shaped instabilities, noting that the Rocscience software computes both and chooses the more critical of the two. Presented in Figure 60 is a conical shaped instability and presented in Figure 61 is a pyramidal shaped instability for a tetrahedral wedge sliding instability.

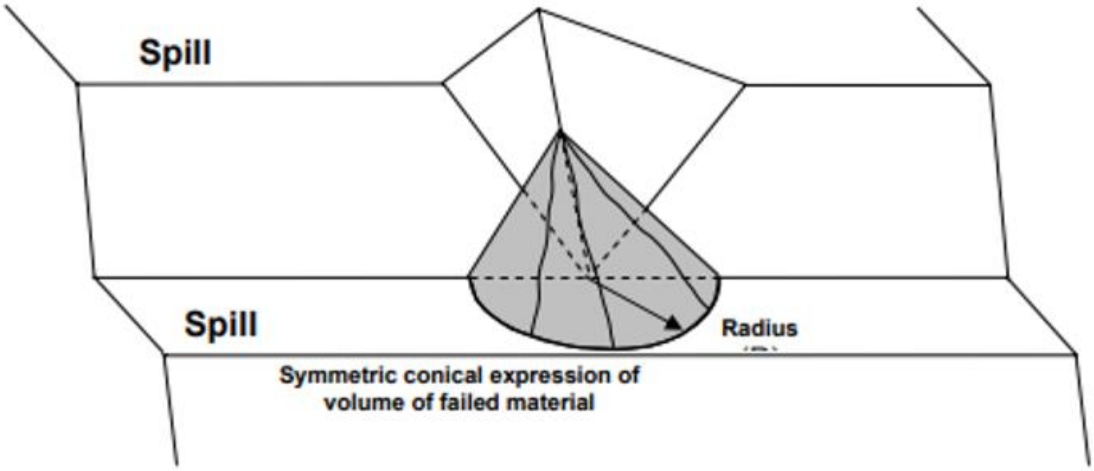


Figure 60 Depiction of Symmetrical Conical Shaped Instability (Gibson et.al, 2006)

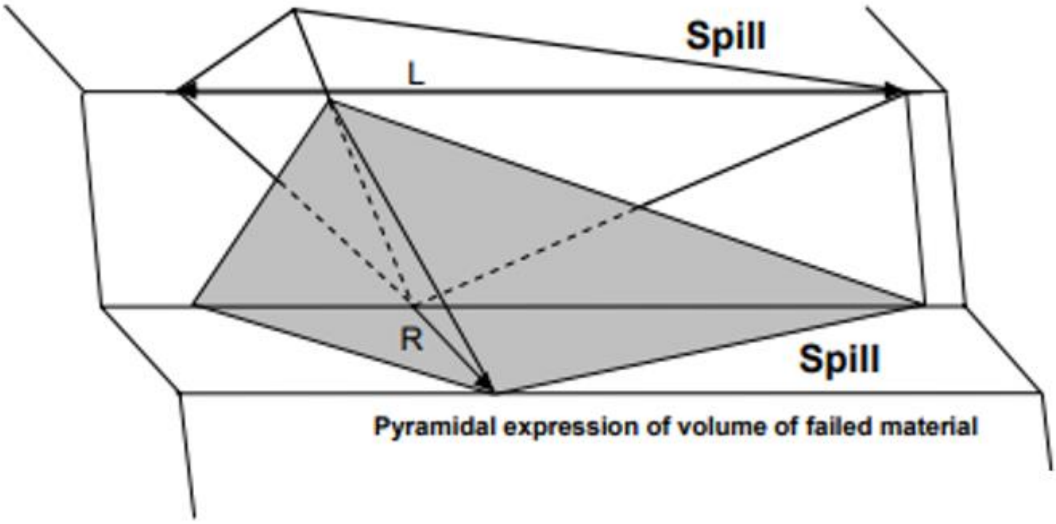


Figure 61 Depiction of Symmetrical Pyramidal Shaped Instability (Gibson et al., 2006)

It should be noted that whilst SWedge and RocPlane both makes use of the Gibson et al. (2006) equations for assessing the minimum bench width requirements for both wedge sliding and planar sliding, the application of the Gibson et al. (2006) equations differ slightly. In SWedge, spill width is calculated using 3-dimensional assumptions as described in Gibson et al. (2006), whereas RocPlane uses a simplified 2-dimensional estimate of spill width.

In order to prevent instabilities impacting lower levels of the slope as a result of spillage, the slope design geometry must be able to sufficiently retain/catch the failed material. The minimum bench width (or spill bench width) required to catch and retain spillage is dependent on a number of factors, which include:

– Slope design geometry.
– Instability geometry.
– Location of back-break.
– Applied bulking (swell) factor.
– Angle of repose.
– Defect plane persistence.

In addition to the above, the minimum bench width is usually measured against a nominated design acceptance criterion for spillage retained, which is reflective of the level of acceptable risk, i.e., the percentage of spillage retained on a single bench. The design acceptance criteria typically range between 70% to 85%, the value of which may reflect the corporate risk profile. By nominating an acceptable level of spillage, steeper slope geometry may be permitted providing that safety is not compromised, which is based on the philosophy that it is acceptable to use steeper bench faces angles, permitting some instabilities to occur providing that safety is not compromised. Whilst there is a greater amount of spilled material on the bench, the economics behind a steeper slope and instituting bench cleaning procedures are far more favourable than a shallower slope with larger benches. It should be noted that with proper scaling and bench clean up following blasting minimises both the likelihood and scale of structural instabilities and therefore the use of the Gibson *et. al.* (2006) equation is considered conservative as it does not reflect the true slope profile after these ground control practices are implemented.

The input parameters required to calculate the minimum required bench width are outlined in more detail below.

6.3.1.1 Slope Design Geometry

The slope design geometry is being assessed for its suitability in catching and retaining spillage, thus remains ‘fixed’, i.e., non-variable.

6.3.1.2 Instability Geometry and Potential Volume of Spill

Defined by the logged defect planes.

6.3.1.3 Backbreak Cells

Backbreak is defined as the horizontal distance between the planned and actual crest after blasting activities have taken place, as shown in Figure 62.

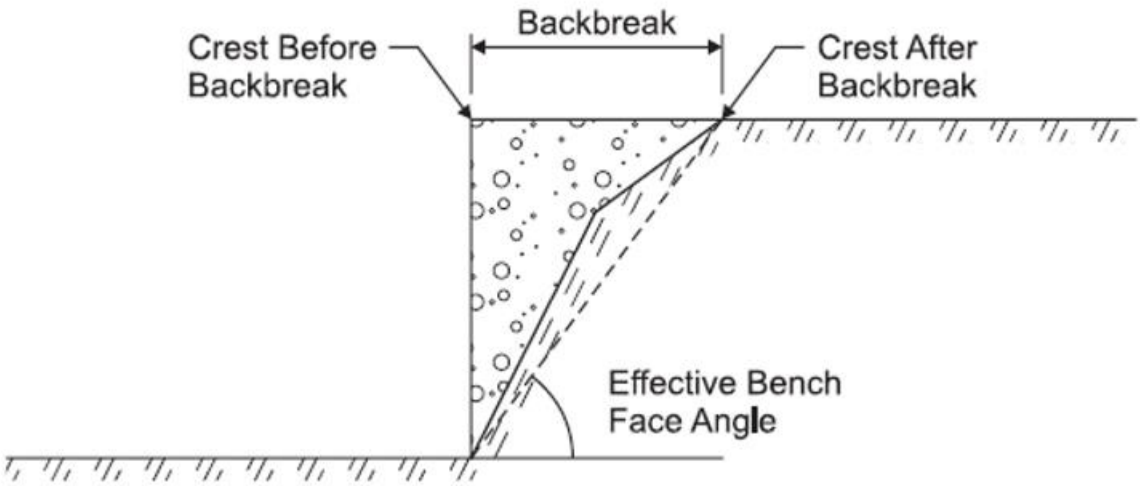


Figure 62 Definition of Backbreak

The location of the backbreak defines the release plane for which instability can occur and thus is a limiting factor of dislodged material, i.e., break breaks located closer to the initial crest position has a lower volume of dislodged mass compared to back breaks located further to the initial crest position. As part of the bench width design analyses the number of backbreak cells, i.e., the number of feasible locations which release planes can form, must be defined. Figure 63 depicts the number of back break cells, with respect to a hypothetical instability and backbreak distance.

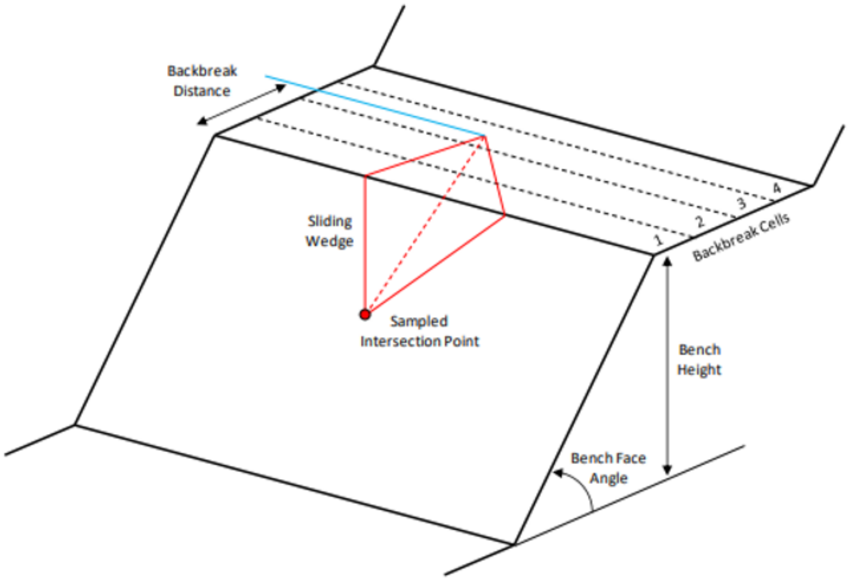


Figure 63 Backbreak Cell Determination

The number of units, or cells that can be analysed for bench width requirements, is dependent on the number and size of the cells, the discontinuity spacing, length and engineering judgment. For the bench width analyses presented within this report, Rocscience's default number of backbreak cells (20 backbreak cells) was adopted, which is considered suitable for materials that are interbedded. It should be noted that where there is a high probability of occurrence for backbreaks to form close to the planned crest, the minimum required bench width tends to be less and vice versa.

6.3.1.4 Bulk (Swell) Factor

The bulk or swell factor is applied to the derived failure volume in order to represent the new volume of space that the failed material will fit into, as it is no longer in situ. This factor is allocated based on the material being assessed. Based on GHD's experience, a bulking factor equal to 1.3 is considered to be suitable.

6.3.1.5 Angle of Repose

The angle of repose can be defined as the maximum possible inclination of a slope of a given mass of cohesionless material. Martin and Piteau (1977) assumed that failed material will rest at angles between 35° to 40°. It should be noted however that large sliding masses of rock may come to rest at angles significantly steeper than this range, where typical values for the angle of repose require for a particular site require back calibration. For the purpose of this assessment an angle of repose equal to 38° has been adopted for calculating the bench width requirements and assessment of the rock slope stability performance as outlined in Section 6.5.

6.3.1.6 Persistence of Defect Planes

Rocscience's software packages SWedge and RocPlane both include two different defect plane spacing options, which are:

- Large defect spacing – where it is assumed that there is only one trace of the defect plane(s) on the slope face e.g., discrete joints. The plane of sliding is randomly located somewhere between the toe and crest of the slope, resulting in a uniform distribution of the instability geometry. The formation of instabilities is limited by the spacing and persistence conditions. This is considered to be the lower bound solution.
- Small defect spacing (ubiquitous) – where it is assumed that defect planes are numerous and may occur at any location e.g., bedding planes. Instabilities formed are scaled down until the persistence conditions are satisfied. The only factors limiting the formation of instabilities is the slope design geometry. This is considered to be the upper bound solution.

Depending on which persistence model is utilised, the outcomes of the minimum bench width analyses can be notably different and requires careful consideration.

Presented in the proceeding sections are the results of the kinematic analyses along with the minimum required bench width calculations.

6.3.2 Planar Sliding

Planar sliding was identified to be a kinematically feasible structural instability mechanism for all wall orientations. Kinematic analyses considering the planar sliding mechanism was undertaken conservatively, where it was assumed that defect planes are orientated in the same direction as the slope face. Where multiple blocks were calculated to fall below unity (i.e., FoS < 1.0), only the critical block formation, i.e., minimum FoS, is reported along with the minimum bench width requirements.

Presented in Figure 64 is an example stability model output for the slopes in South Domain and depicted in Figure 65 is the corresponding minimum berm with plot.

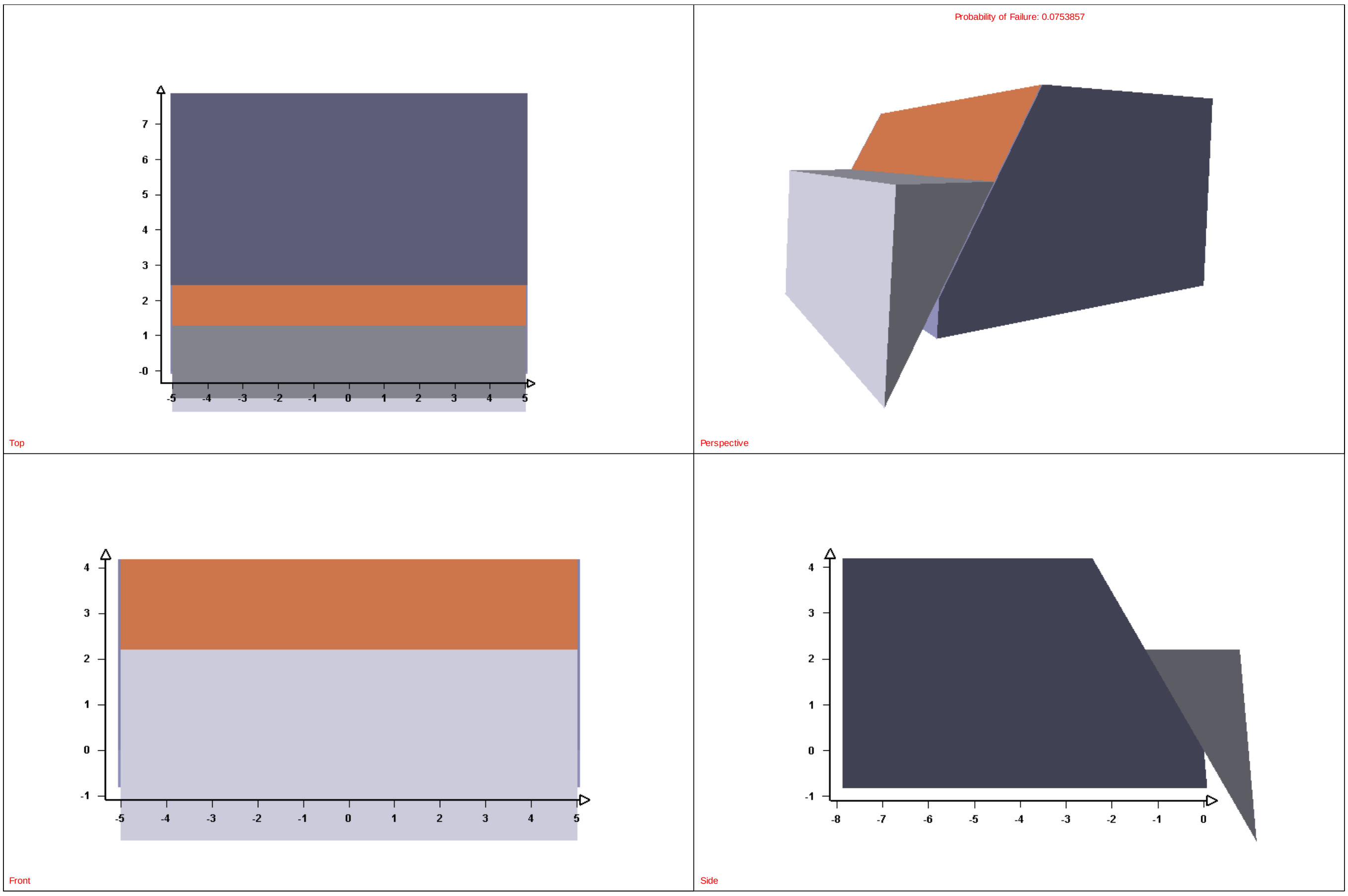


Figure 64 Example Stability Model Output for Planar Sliding – South Domain – Defect Set 3

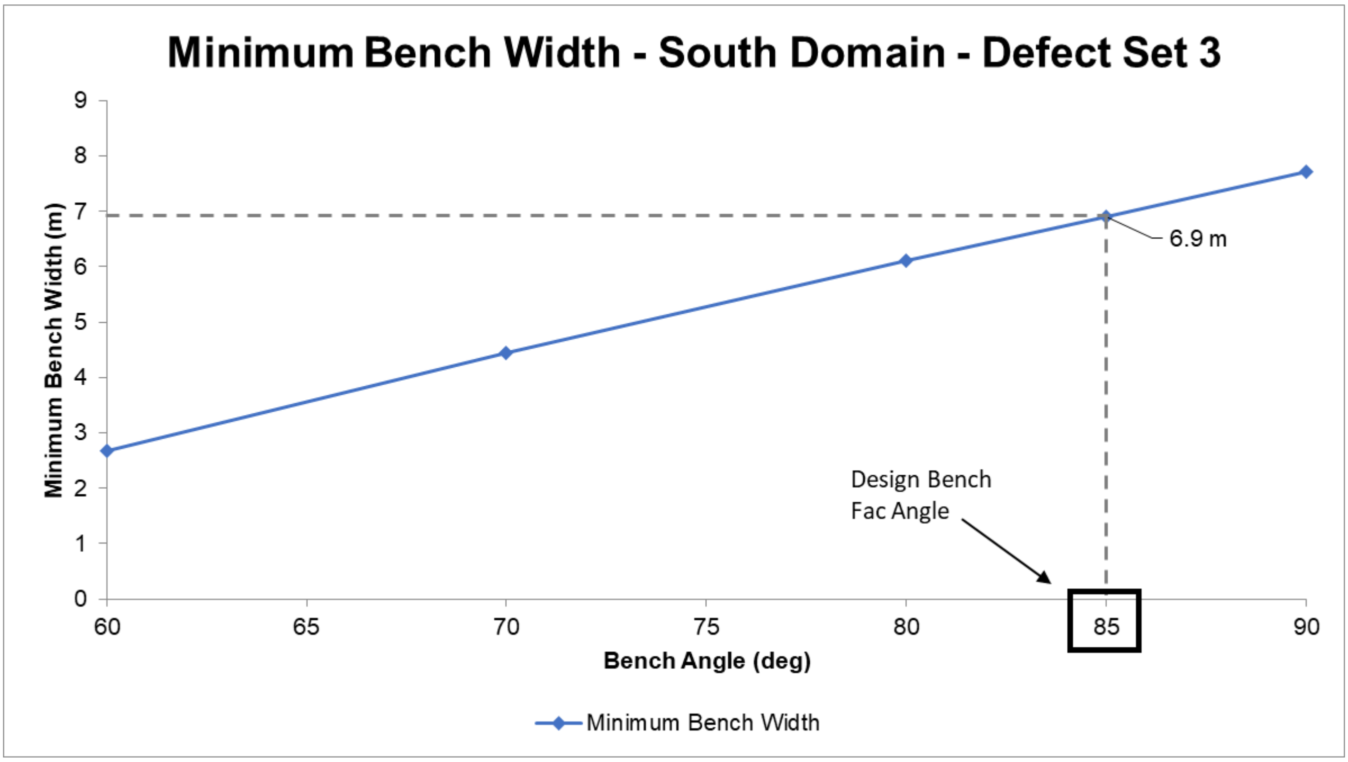


Figure 65 *Example Minimum Bench Width Plot – South Domain – Defect Set 3*

The results of the planar sliding analyses undertaken to assess the stability performance of the proposed batter-bench design configuration are presented below in Table 24.

Table 24 *Summary of Planar Sliding Analyses Results*

Domain	Critical Set ID	Weathering Grade	Min. FoS	PoF	Design Bench Width (m)	Min. Bench Width (m)	Vol. of Instability (m^3/m)
North	Set 4	HW	1.01	0.0%	7	< 1	11.9
		MW to SW	1.17	0.0%		< 1	
		Fr	1.26	0.0%	6	< 1	
East	Set 5	HW	0.34	1.00	7	4.1	2.4
		MW to SW	0.40	97.8%		4.2	
		Fr	0.43	93.9%	6	4.2	
South	Set 3	HW	0.61	7.7%	7	6.8	6.2
		MW to SW	0.70	2.3%		6.2	
		Fr	0.76	1.2%	6	5.9	
West	Set 1	HW	0.17	48.4%	7	1.3	0.1
		MW to SW	0.20	48.7%		1.3	
		Fr	0.22	0.49	6	1.3	

The results of the planar sliding analyses indicate that:

– Under expected conditions, east facing walls i.e., excavations within the eastern domain, are comparatively more sensitive to planar sliding than the other walls.

– The minimum calculated bench width with respect to each weathering grade was calculated (to the nearest metre) to be:

  - HW / MW to SW Hornfels = 7.0 m

- Fr Hornfels = 6.0 m

– Therefore, the nominated design bench widths are suitable for the ‘single’ bench geometry. Analyses pertaining to the double bench design, is presented in Section 6.3.5.

– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.

– The volume of instability was calculated to be of limited scale/volume, with larger volumes calculated along the north (11.9 m^3/m) and south (6.2 m^3/m) walls.

6.3.3 Tetrahedral Wedge Analysis

Tetrahedral wedge sliding was identified to be a kinematically feasible structural instability mechanism for all wall orientations. The potential for wedge instabilities to occur along excavated faces was assessed for ‘representative’ batter orientations, which is conceptually nominated based on the wall extent, e.g., maximum height and length, and the orientation most susceptible to kinematic instability within each domain.

The tetrahedral wedge sliding analyses presented in this section is considered conservative given that:

- Discontinuity systems are ubiquitous.
- Bedding planes are assumed to be infinitely long and linear i.e., does not account for defect waviness (undulations) or significant changes in defect orientation.
- Tertiary discontinuity systems e.g., rock mass fracturing, that can form release planes are assumed to not be present.

Two conditions were assessed as part of this stability analyses which are as follows:

- Managed – Good blasting practice followed by final wall scaling.
- Unmanaged – Poor blasting practice and no final wall scaling.

The unmanaged condition considers the potential retrogressive instability that may occur post blasting as a result of disturbed slope faces and/or where scaling has not been adequately performed. The calculated undamaged volume is considered to be reflective of the potential total volume that may become unstable under such conditions.

Presented below in Figure 66 is an example stability model output for the excavated slopes located within the North Domain and presented in Figure 67 is the corresponding minimum bench width plot.

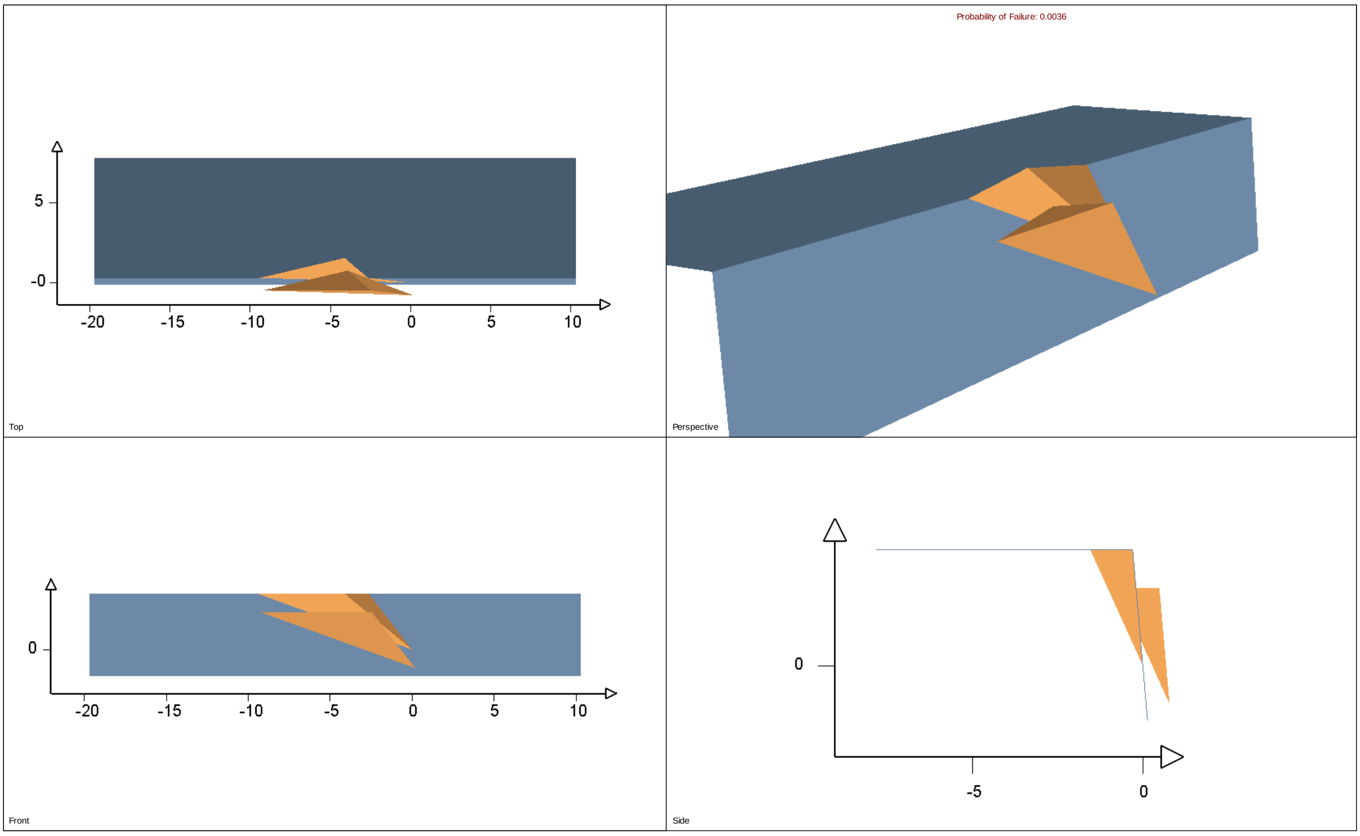


Figure 66 Example Stability Model Output for Planar Sliding – North Domain – Defect Set 2 and 4

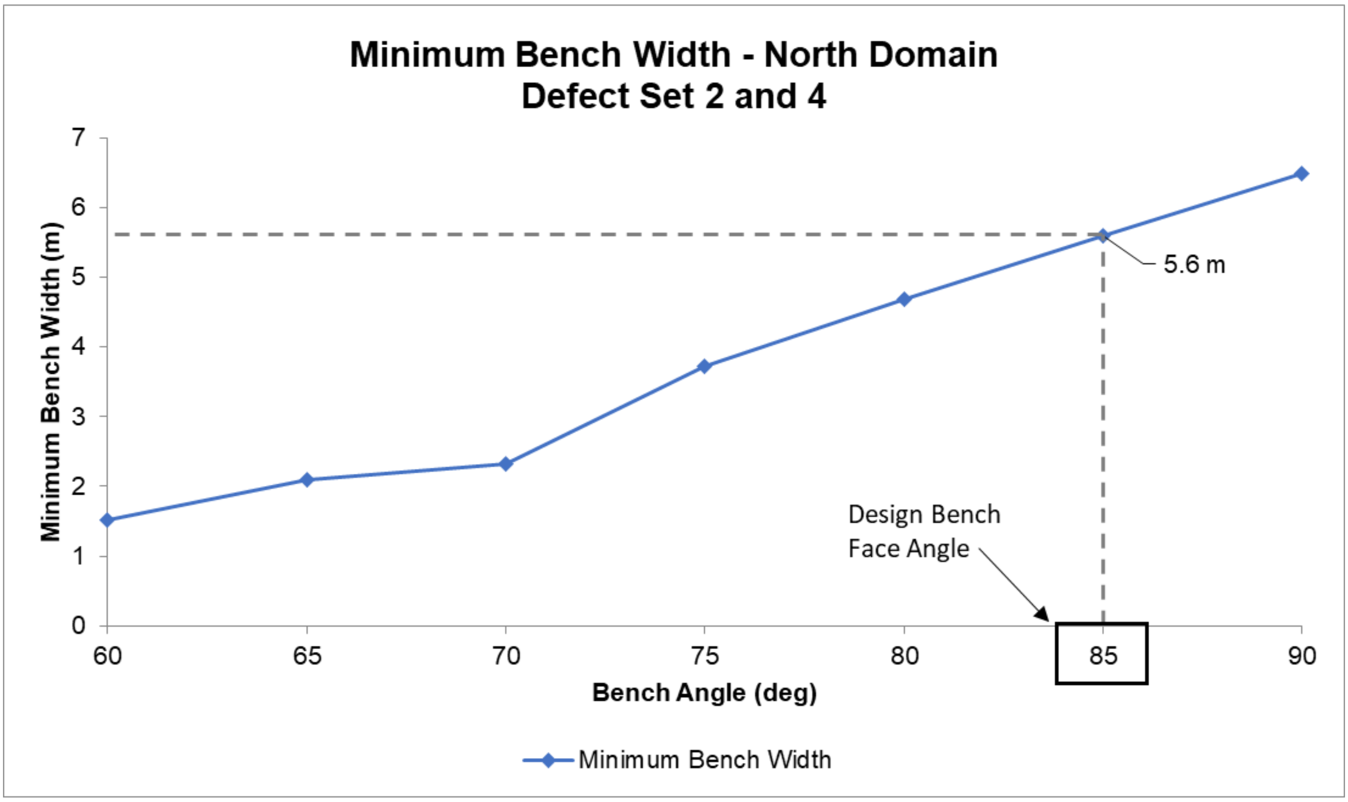


Figure 67 Minimum Bench Width Plot - North Domain – Set 2 and 4

The results of the planar sliding analyses undertaken to assess the stability performance of the proposed batter-bench design configuration are presented below in Table 25.

Table 25 Summary of Kinematic Wedge Sliding Results

Domain	Critical Set ID	Weathering Grade	Min. FoS	PoF	Design Bench Width (m)	Min. Bench Width (m)	Vol (m^3) - Managed	Vol (m^3) – Un-managed
North	S2 & S4	HW	0.80	0.4%	7	5.6	4.765	15.138
		MW to SW	0.93	0.1%		3.4		
		Fr	1.01	0.0%	6	< 1		
East	S2 & S5	HW	0.26	51.8%	7	3.3	0.974	0.238
		MW to SW	0.30	45.7%		3.3		
		Fr	0.33	39.6%	6	3.3		
South	S2 & S5	HW	0.44	21.4%	7	5.4	0.779	12.581
		MW to SW	0.52	9.8%		4.8		
		Fr	0.56	6.5%	6	4.3		
West	S1 & S2	HW	0.18	57.0%	7	2.5	0.037	0.262
		MW to SW	0.22	56.7%		2.5		
		Fr	0.24	56.6%	6	2.5		

The results of the tetrahedral wedge sliding analyses indicate that:

– Under expected conditions, east and west facing walls were calculated to be comparatively more sensitive (PoF > 50%) to tetrahedral wedge sliding compared to the remaining wall orientations.
– The minimum calculated bench width with respect to each weathering grade was calculated (to the nearest metre) to be:
  - HW Hornfels = 6 m.
  - MW to SW Hornfels / Fr Hornfels = 5 m.
– Therefore, the nominated design bench widths are suitable for the 'single' bench geometry. Analyses pertaining to the double bench design, is presented in Section 6.3.5.
– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.
– The volume of instability was calculated to be of limited scale/volume where good blasting and scaling practices are employed. A notable increase was calculated along the north and south walls for the un-managed condition i.e., poor blasting and no scaling.

6.3.4 Toppling Analysis

Block-flexural toppling was identified to be a kinematically feasible structural instability mechanism for east and west wall orientations. Data obtained on site through visual observations were utilised this assessment including persistence and spacing of defect sets. The rock toppling analyses presented within this section are considered to be conservative where the critical defect structures are orientated in the same direction as the slope. The results of the toppling analyses are presented as a series of charts which reflect the stability performance of excavated slopes with respect to the weathering grade, as shown in Figure 68.

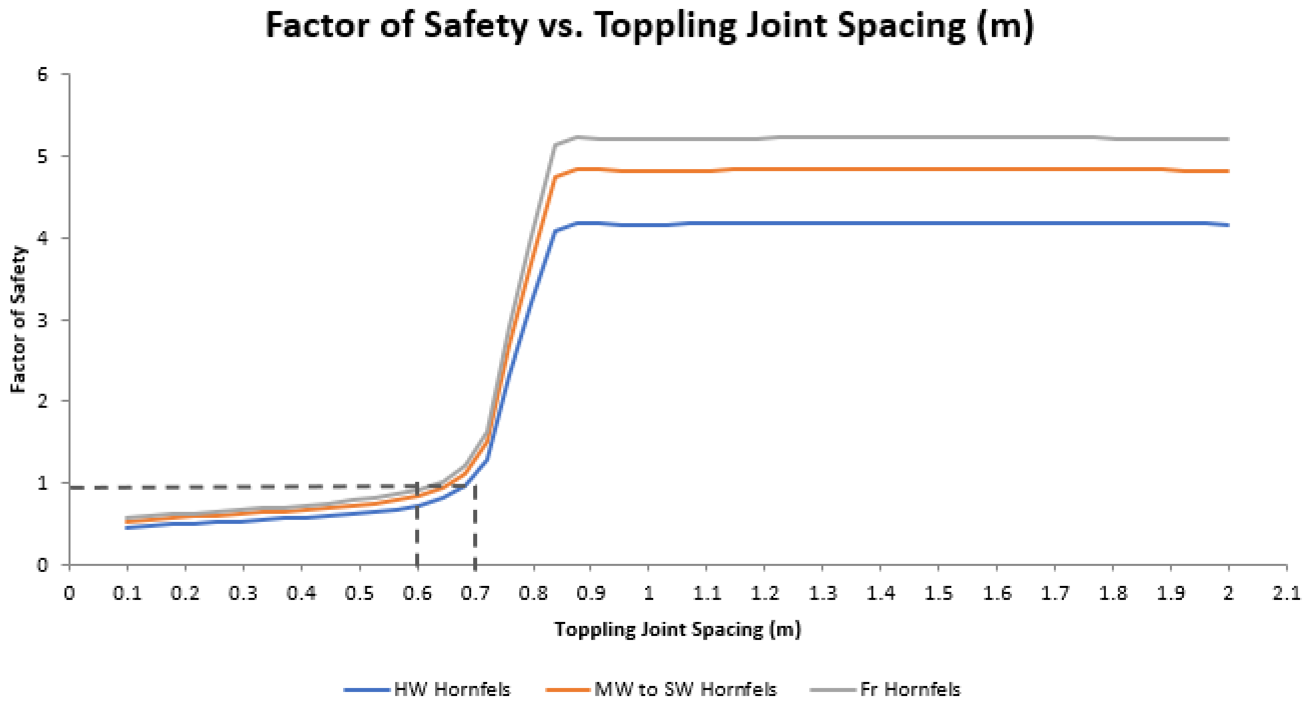


Figure 68 Influence of Defect Set Spacing on Stability Performance – Rock Toppling

The results of the rock toppling analyses indicate that:

– Where the spacing of toppling joints occurs in the range of 0.6 m to 0.7 m or less, the likelihood of rock toppling becomes notably higher as the calculated FoS falls below unity for all stratigraphic units.

As there is no design tool included with the RocTopple software package, rockfall analyses has been undertaken using Rocscience RocFall v8.01, to assess the potential stability implications associated with (direct) block toppling to assess the adequacy of the proposed batter-bench configuration. Rock fall assessment was undertaken using a rigid body model, which accounts for variations in the size, weight and shape of individual rock blocks. Monte Carlo sampling methods were employed to simulate 1000 rock fall events from a point seeder located at the crest of the batter.

The input parameters adopted for the rockfall assessment are summarised in Table 26.

Table 26 Summary of RocFall Input Parameters

Material	Statistical Parameter	Distribution Type	Mean	Standard Deviation	Relative Min.	Relative Max.
Slope Face	Normal Restitution	Normal	0.15	0.060	0.150	0.180
	Dynamic Friction		0.58	0.130	0.390	0.390
	Rolling Friction		0.97	0.184	0.552	0.030
Catch Bench	Normal Restitution	Normal	0.04	0.007	0.021	0.021
	Dynamic Friction		0.58	0.130	0.390	0.390
	Rolling Friction		0.75	0.159	0.351	0.351
Rock Mass Weight (kg)		Log Normal	50	500	50	5600

The results of the rockfall analysis are presented schematically in Figure 69.

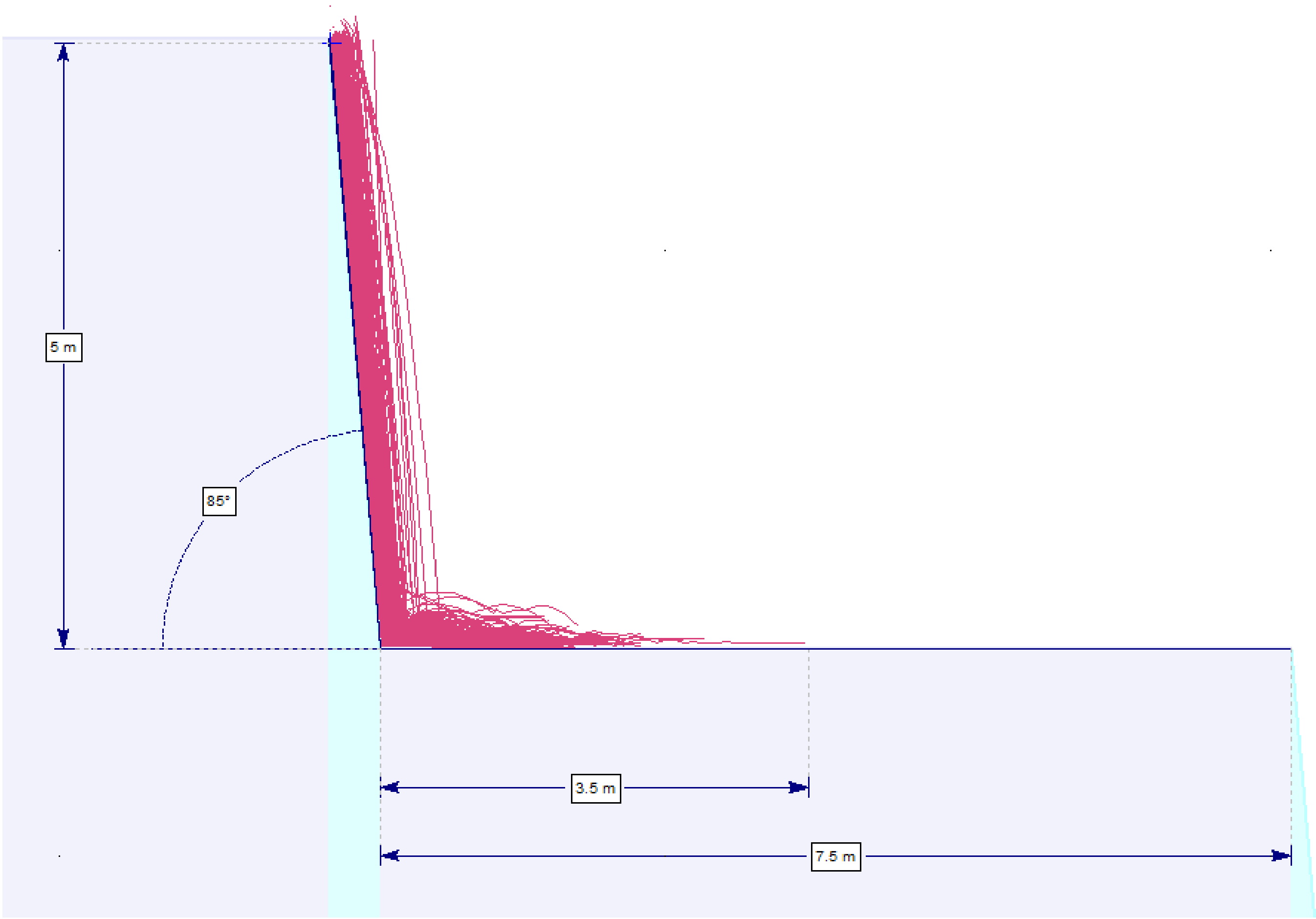


Figure 69 Rock Fall Model Output

The results of the rockfall analyses indicate that in the event of rockfall, the maximum trajectory is likely to occur within 3.5 m from the batter toe, for rock mass instabilities in the order of ~0.5 m^3 to 2 m^3.

6.3.5 Double Benching Design Check

As outlined in Section 6, double benching the fresh Hornfels batters below the anticipated final lake level was considered. Table 27 summarises the results of minimum berm width calculations assessing the critical wedge volumes for the proposed double benched geometry (berm height and width of 10 m and 8 m, respectively).

Table 27 Results of Minimum Berm Width Calculation – Double Benched Fresh Hornfels Batter

Instability Mechanism	Domain	Critical Set ID	Minimum Bench Width (m)
Planar Sliding	South	Set 3	8.0
Tetrahedral Wedge	South	S2 + S5	6.2
Toppling	NA	NA	4.8

The results of the minimum berm width calculation assessing the double benched fresh Hornfels batter geometry indicate that the design geometry is suitable.

6.3.6 Summary of Kinematic Analyses Results

The results of the kinematic analyses indicate that:

Planar Sliding

– Under expected conditions, east facing walls i.e., excavations within the eastern domain, are comparatively more sensitive to planar sliding than the other walls.

– The minimum calculated bench width with respect to each weathering grade was calculated (to the nearest metre) to be:
  - HW / MW to SW Hornfels = 7.0 m
  - Fr Hornfels = 6.0 m
– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.
– The volume of instability was calculated to be of limited scale/volume, with larger volumes calculated along the north (11.9 m^3/m) and south (6.2 m^3/m) walls.

Tetrahedral Wedge Sliding

– Under expected conditions, east and west facing walls were calculated to be comparatively more sensitive (PoF > 50%) to tetrahedral wedge sliding compared to the remaining wall orientations.
– The minimum calculated bench width with respect to each weathering grade was calculated (to the nearest metre) to be:
  - HW Hornfels = 6 m
  - MW to SW Hornfels / Fr Hornfels = 5 m
– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.
– The volume of instability was calculated to be of limited scale/volume where good blasting and scaling practices are employed. A notable increase was calculated along the north and south walls for the un-managed condition i.e., poor blasting and no scaling.

Rock Toppling

– Where the spacing of toppling joints occurs in the range of 0.6 m to 0.7 m or less, the likelihood of rock toppling becomes notably higher as the calculated FoS falls below unity for all stratigraphic units.
– In the event of rockfall, the maximum trajectory is likely to occur within 3.5 m from the batter toe, for rock mass instabilities in the order of ~0.5 m^3 to 2 m^3.

The design slope geometry adopted (as outlined in Section 6) satisfies the requirements for minimum berm widths for all proposed batter-berm configurations, including the double benched fresh Hornfels batters proposed for below the pit lake level.

6.4 Global Slope Stability Analysis Results

Slope stability analyses has been performed under 'expected' conditions which are the in-situ stress conditions known to or likely to exist at the time of the February 2022 site inspection. The stability performance of slopes at the individual, multi-bench and overall slope scale were assessed and compared against the nominated Design Acceptance Criteria (DAC). The following colour coding has been adopted to denote conformance to the DAC.

– Blue – The calculated stability performance satisfies the rehabilitation slope DAC (FoS > 2.0).
– Green – The calculated stability performance satisfies the terminal slope DAC (FoS > 1.6).
– Amber - The calculated stability performance does not satisfy the DAC but remains above unity (FoS > 1.0).
– Red – The calculated stability performance falls below unity (FoS < 1.0).

6.4.1 Terminal Slopes

The results of the stability analyses undertaken on terminal slopes along stability Sections N1, W1 and S1 are summarised in Table 28 and the corresponding stability model output is presented in Figure 70, Figure 71 and Figure 72.

Table 28 Summary of Stability Analyses Results – Terminal Slopes

Stability Section	DAC	Scale of Instability	Calculated FoS*
N1	FoS > 1.6	Multi-bench	N/A
		Overall slope	1.89
W1		Multi-bench	N/A
		Overall slope	2.70
S1		Multi-bench	N/A
		Overall slope	2.65
* N/A = Critical instability plane calculated at the overall slope scale and no multi-bench scale calculated.			

The results of the stability analyses undertaken for terminal slopes under 'expected' conditions indicate that:

– The stability performance at the multi-bench and overall slopes scales satisfy the nominated design acceptance criteria for stability Sections N1, W1, and S1.

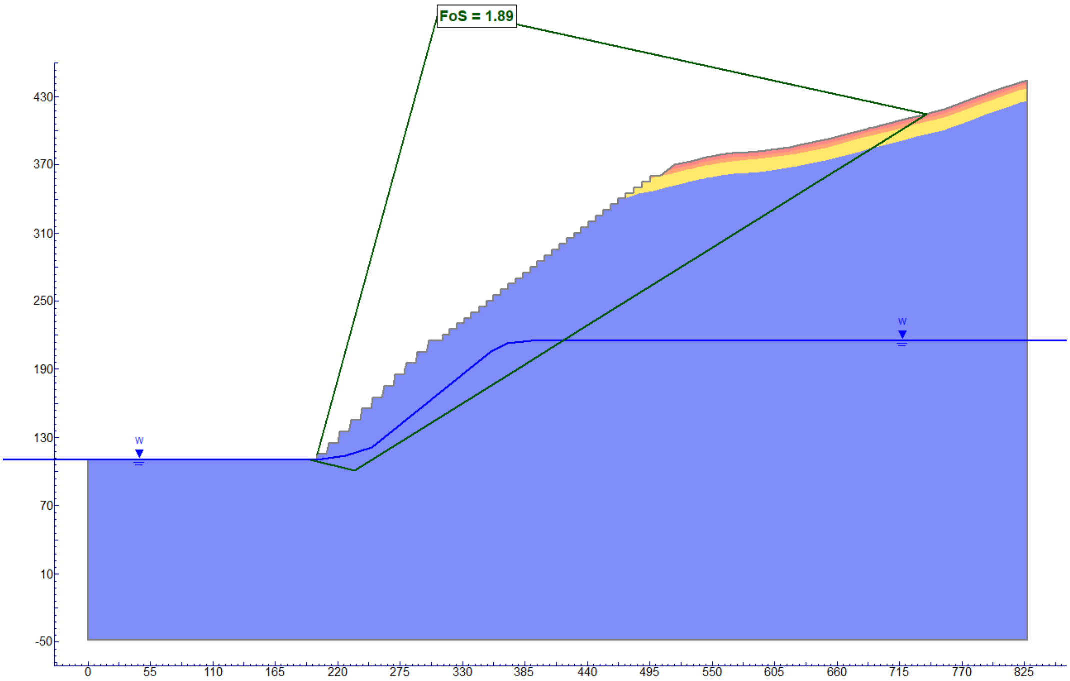


Figure 70 *Stability Model Output – Stability Section N1 – Terminal Slopes*

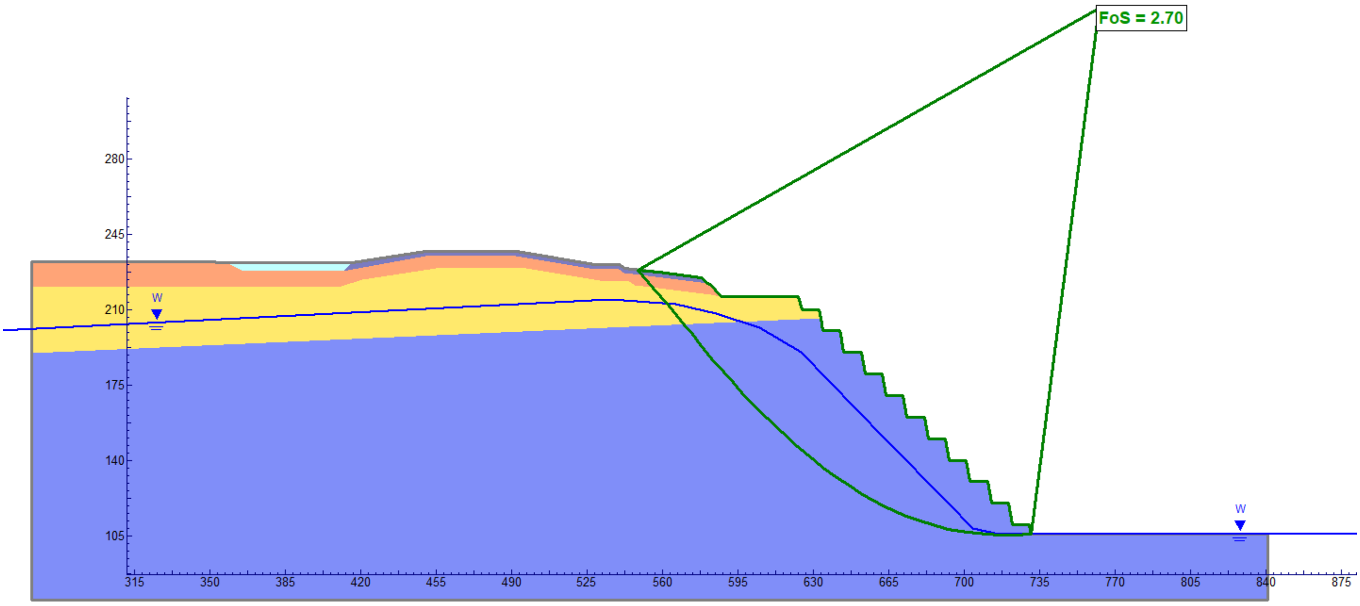


Figure 71 *Stability Model Output – Stability Section W1 – Terminal Slopes*

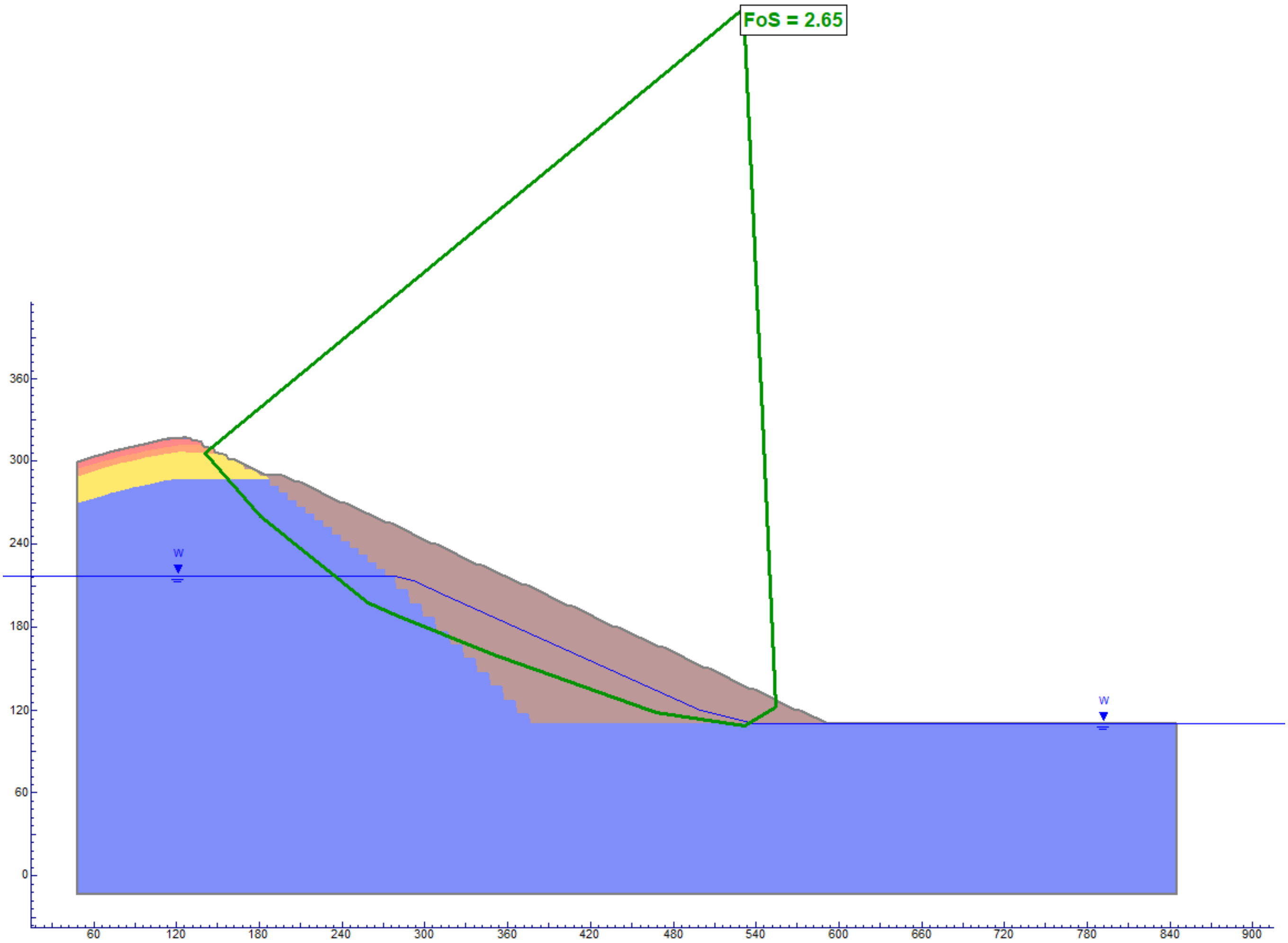


Figure 72 Stability Model Output – Stability Section S1 – Terminal Slopes

6.4.1.1 Internal Dump Stability

Stability analyses along stability Section S1 were undertaken to assess the stability performance of internal dump at terminal stage. The results of the analyses are summarised in Table 29 and the corresponding stability model output is presented in Figure 73.

Table 29 Summary of Internal Dump Stability Analyses Results – Terminal Slope

Stability Section	DAC	Scale of Instability	Calculated FoS*
S1	FoS > 1.6	Multi-bench	1.71
		Overall slope (within dump)	1.72

The results of the internal dump stability analyses undertaken for terminal slope under 'expected' conditions indicate that:

– The internal dump stability performance at the multi-bench and overall slope scales satisfy the nominated design acceptance criteria for internal dump along stability Section S1.

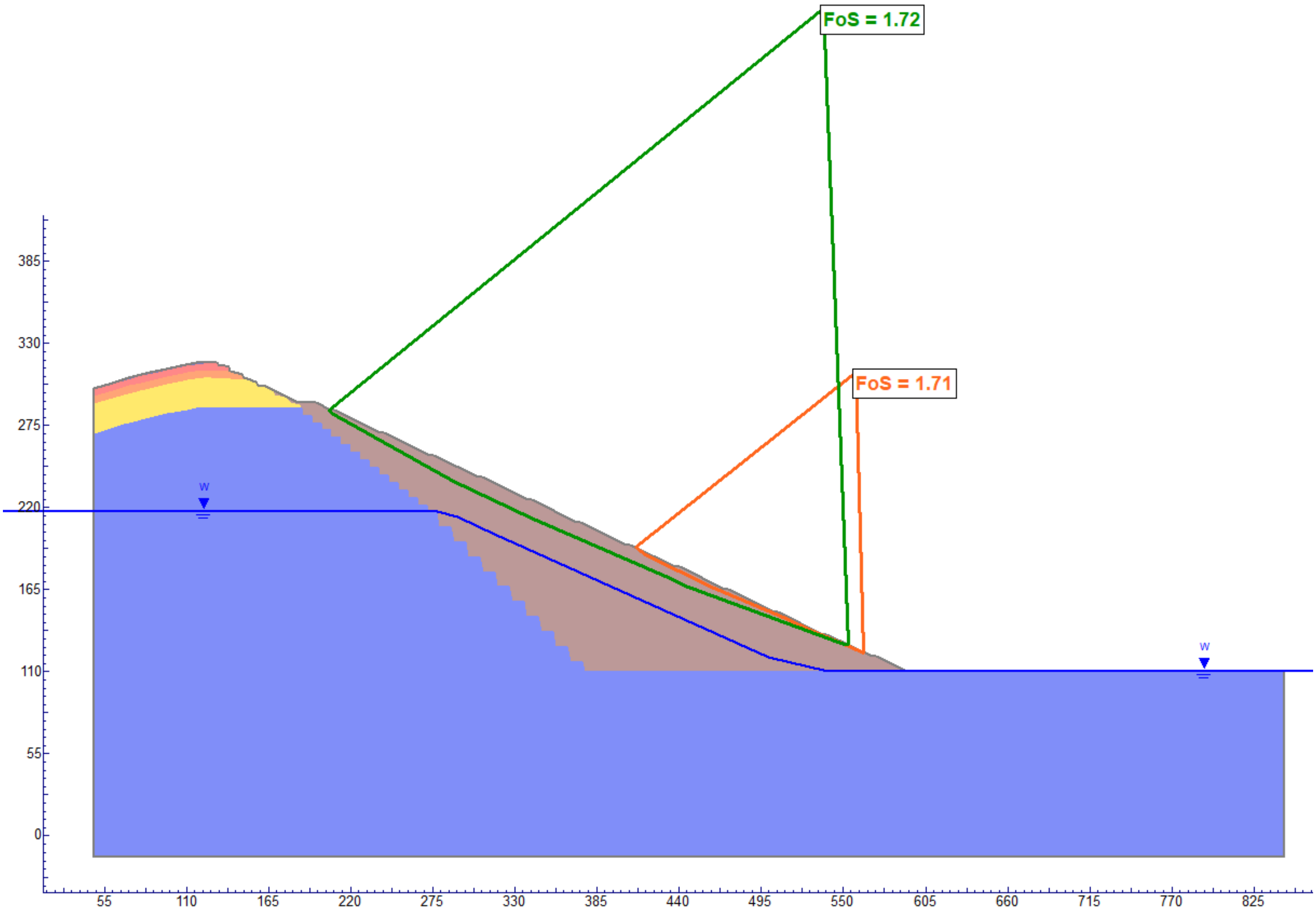


Figure 73 Internal Dump Stability Model Output – Stability Section S1 – Terminal Slope

6.4.2 Rehabilitated Slopes

The results of the stability analyses undertaken on rehabilitated slopes along stability Sections N1, W1 and S1 are summarised in Table 30 and the corresponding stability model outputs are presented in Figure 74, Figure 75 and Figure 76.

Table 30 Summary of Stability Analyses Results – Rehabilitated Slopes

Stability Section	DAC	Scale of Instability	Calculated FoS*
N1	FoS > 2.0	Multi-bench	N/A
		Overall slope	2.01
W1		Multi-bench	N/A
		Overall slope	3.57
S1		Multi-bench	N/A
		Overall slope	2.78
* N/A = Critical instability (minimum FoS) occurs at the overall slope scale.			

The results of the stability analyses undertaken for rehabilitated slopes under ‘expected’ conditions indicate that:

– The stability performance at the multi-bench and overall slopes scales satisfy the nominated design acceptance criteria for all assessed stability sections.

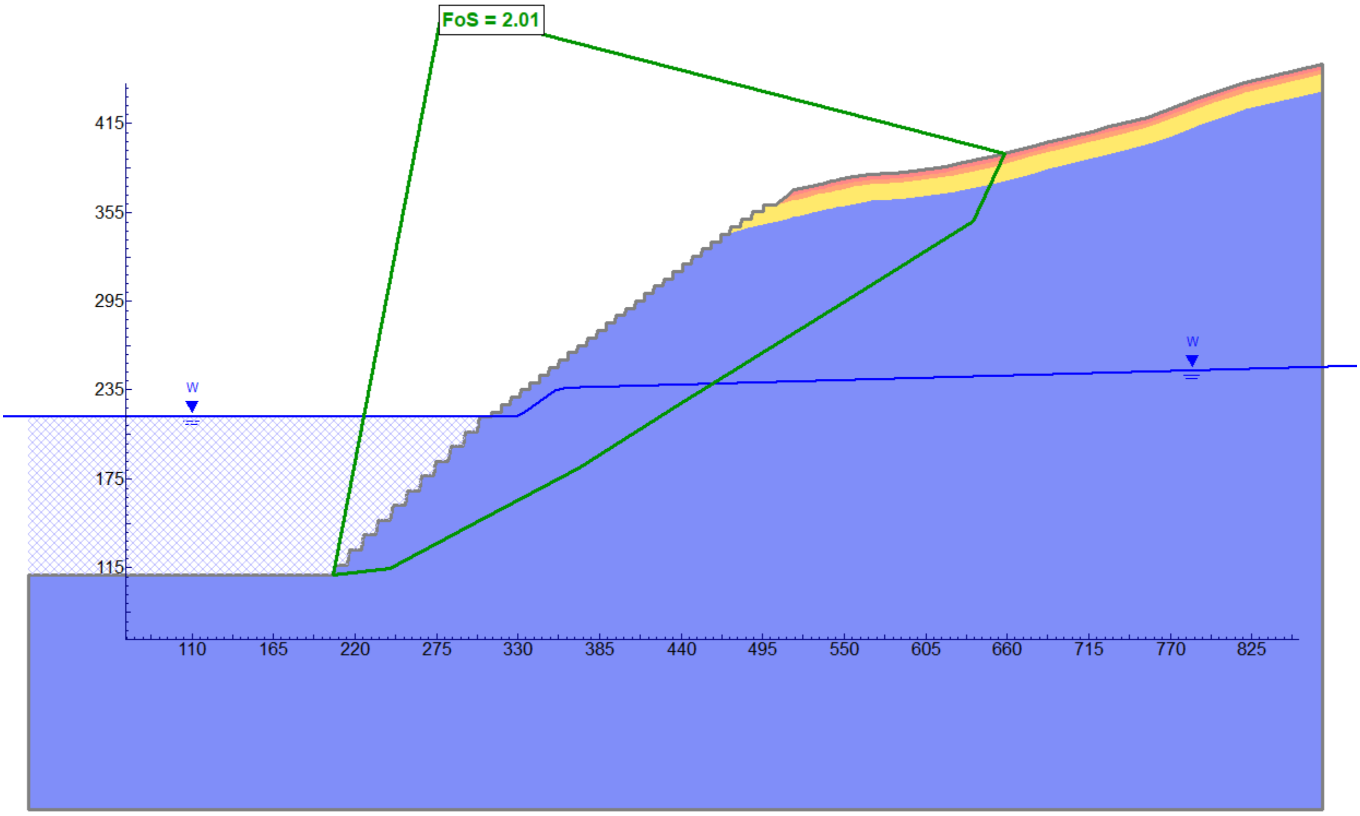


Figure 74 Stability Model Output – Stability Section N1 – Rehabilitated Slopes

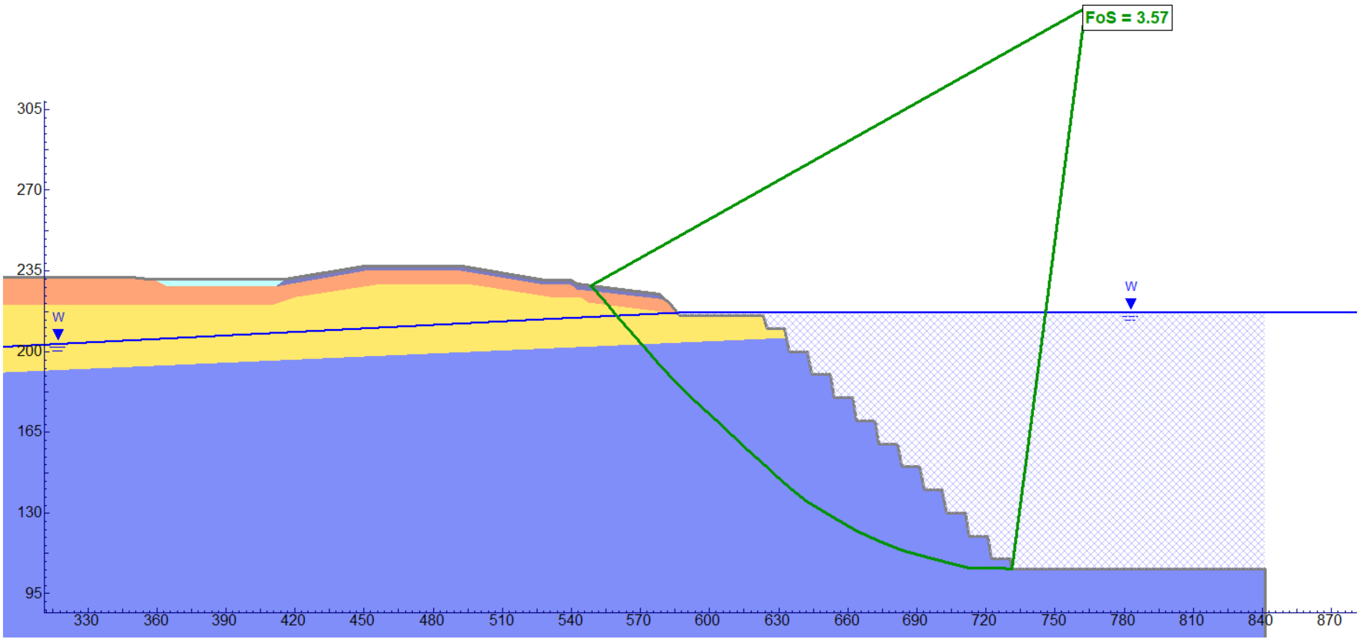


Figure 75 Stability Model Output – Stability Section W1 – Rehabilitated Slopes

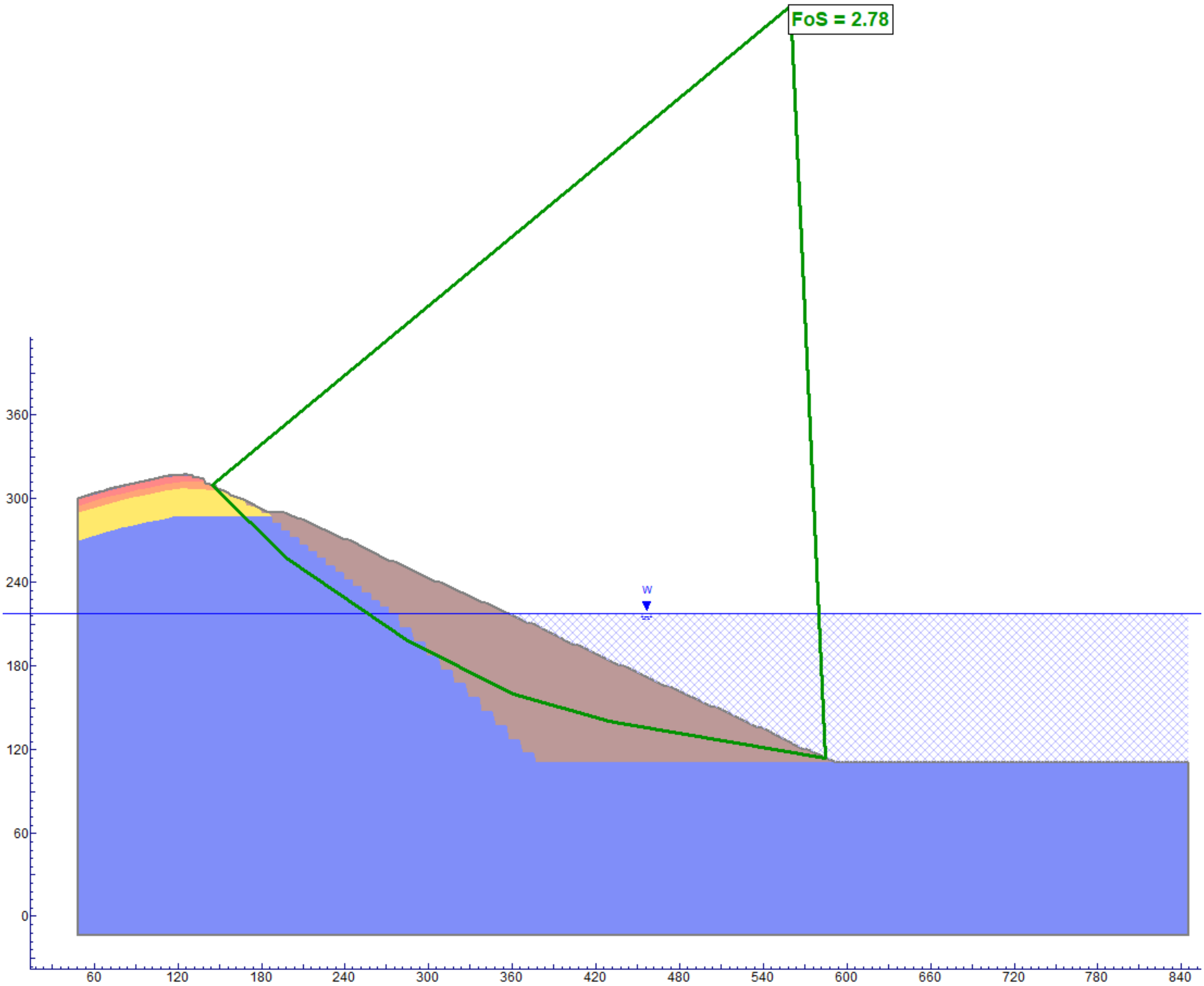


Figure 76 Stability Model Output – Stability Section S1 – Rehabilitated Slopes

6.4.2.1 Internal Dump Stability

The results of the internal dump stability analyses undertaken on rehabilitated slope along stability Section S1 are summarised in Table 31 and the corresponding stability model output is presented in Figure 77.

Table 31 Summary of Stability Analyses Results – Rehabilitated Slopes

Stability Section	DAC	Scale of Instability	Calculated FoS*
S1	FoS > 2.0	Multi-bench	1.66
		Overall slope	1.69

The results of the internal dump stability analyses undertaken for rehabilitated slope under 'expected' conditions indicate that:

– The stability performance at the multi-bench and overall slopes scales does not satisfy the nominated design acceptance criteria for rehabilitation scenario.

– Note that the FoS values indicated in Table 31 represent the stability performance of the internal dump and do not imply any instabilities within the rock slope in the long term.

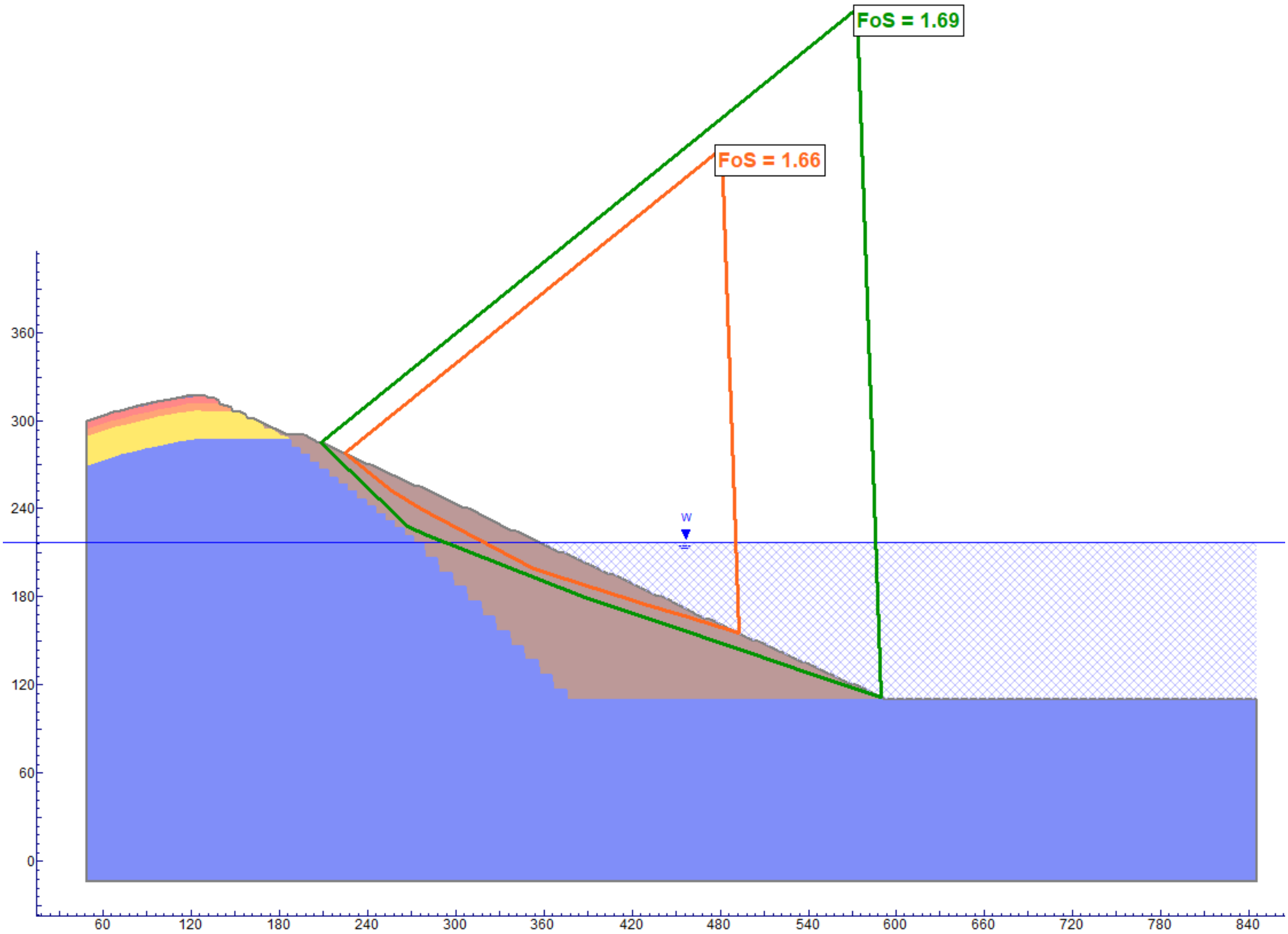


Figure 77 Internal Dump Stability Model Output – Stability Section S1 – Rehabilitated Slopes

6.5 Sensitivity Analyses

Sensitivity analyses has been undertaken to assess the potential stability implications associated with, and the robustness of the revised designs (as outlined in Section 6) against:

– Extended zone of weathering.
– Seismic loading events.
– Elevated phreatic conditions.
– Influence of the final pit lake level.
– Influence of degraded material strengths.

Further detail on each sensitivity assessment along with the stability analyses results are provided in the following subsections.

6.5.1 Extended Zone of Weathering

As outlined in Section 5.3, extended zones of weathering may become apparent along excavated slopes which may persist along terminal and rehabilitated slopes. Accordingly, the potential stability implications associated with an extended weathering zone were investigated. It should be noted for this sensitivity analyses only stability Section N1 was considered owing to the potential long-term exposure of the EW and HW units. The extended zone of weathering profile for assessment was developed utilising information obtained from the drilling campaign as outlined in Section 5.3. The extended weathering profile assessed consists of:

– Unit 1 – Regolith = 1 m, overlying;
– Unit 2 – EW Hornfels = 10 m, overlying;

– Unit 3 – HW Hornfels = 20 m. overlying;
– Unit 4 – MW to SW Hornfels = 30 overlying;
– Unit 5 – FR Hornfels to base of pit.

The extended weathering profile is depicted schematically in Figure 78.

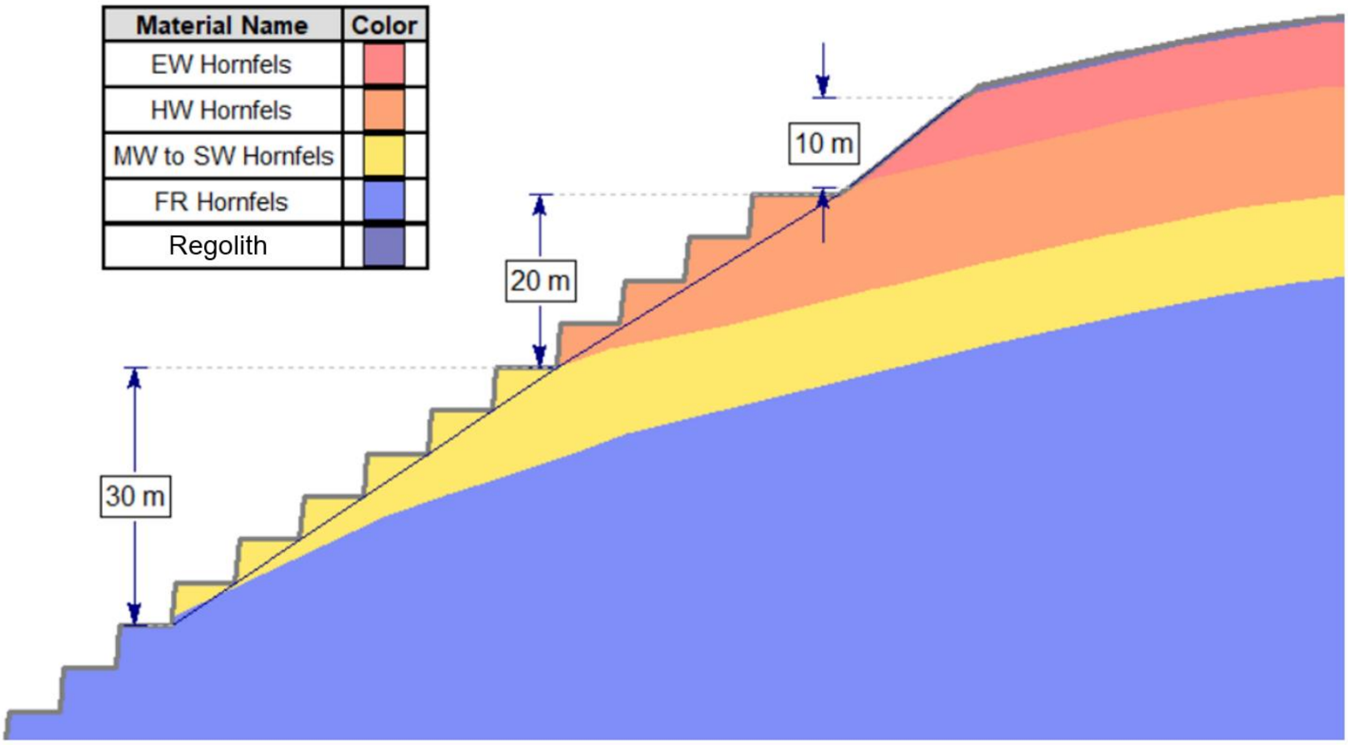


Figure 78 Schematic Depiction of Extended Weathering Profile

The sensitivity analyses results considering the extended zone of weathering are summarised in Table 32 and the corresponding stability model outputs are presented in Figure 79 and Figure 80.

Table 32 Summary of Sensitivity Analyses Results – Extended Zone of Weathering

Stability Section	DAC	Scenario	Scale of Instability	Calculated FoS
N1	FoS > 1.6	Terminal	Multi-bench	1.81
			Overall Slope	2.00
	FoS > 2.0	Rehabilitated	Multi-bench	1.81
			Overall Slope	2.01

The results of the sensitivity analyses undertaken to assess the potential stability implications associated with an extended zone of weathering indicate that:

– The adoption of a flatter slope profile through the upper weathered units acts to mitigate the potential stability implications associated with the extended weathering profiles at the overall slope scales.
– At the multi-bench scale, there is a decrease in the stability performance compared to the results of the 'expected' weathering profile, however, remains above design performance objectives

The results of the extended zone of weathering sensitivity analyses highlight the robustness of the design geometry against varied geological conditions.

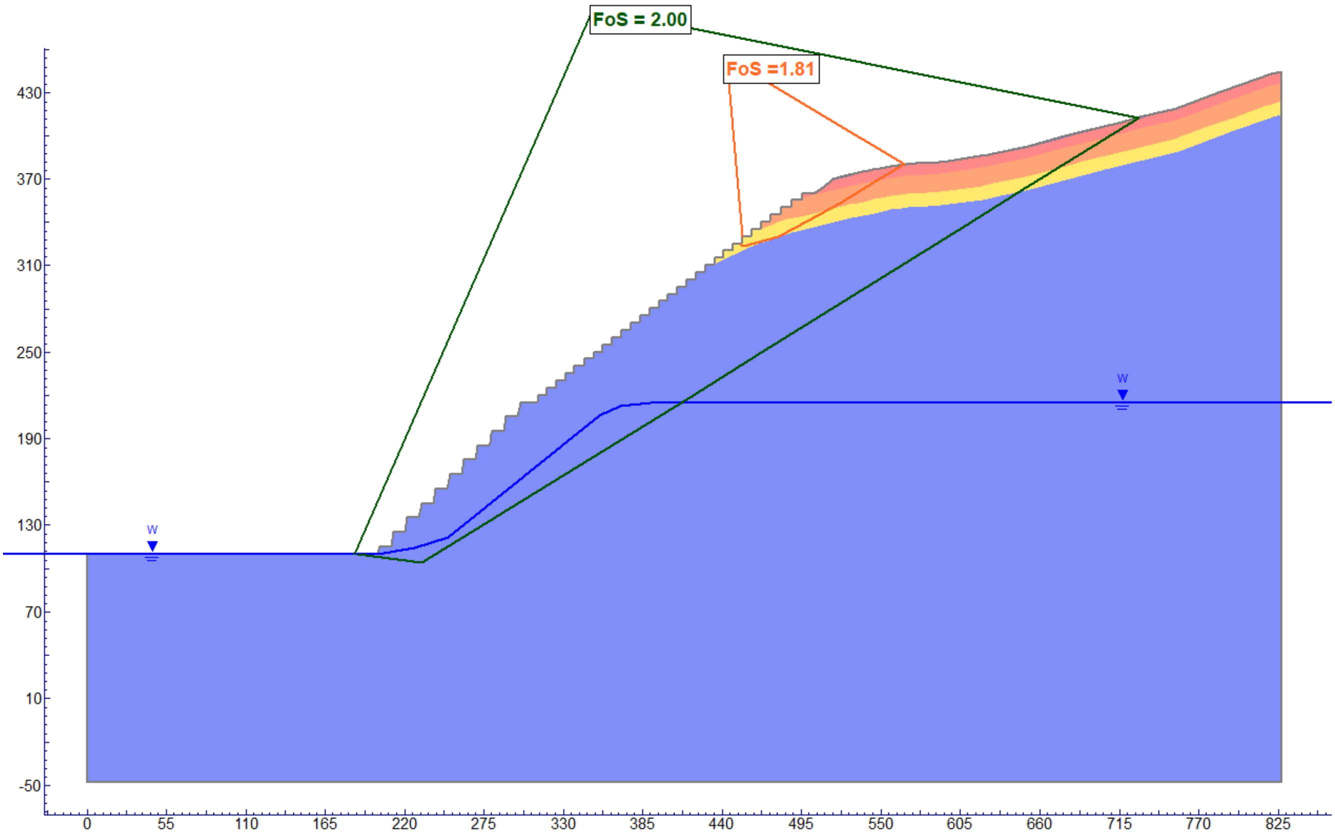


Figure 79 ***Sensitivity Model Output – Extended Zone of Weathering – Section N1 – Terminal Slopes***

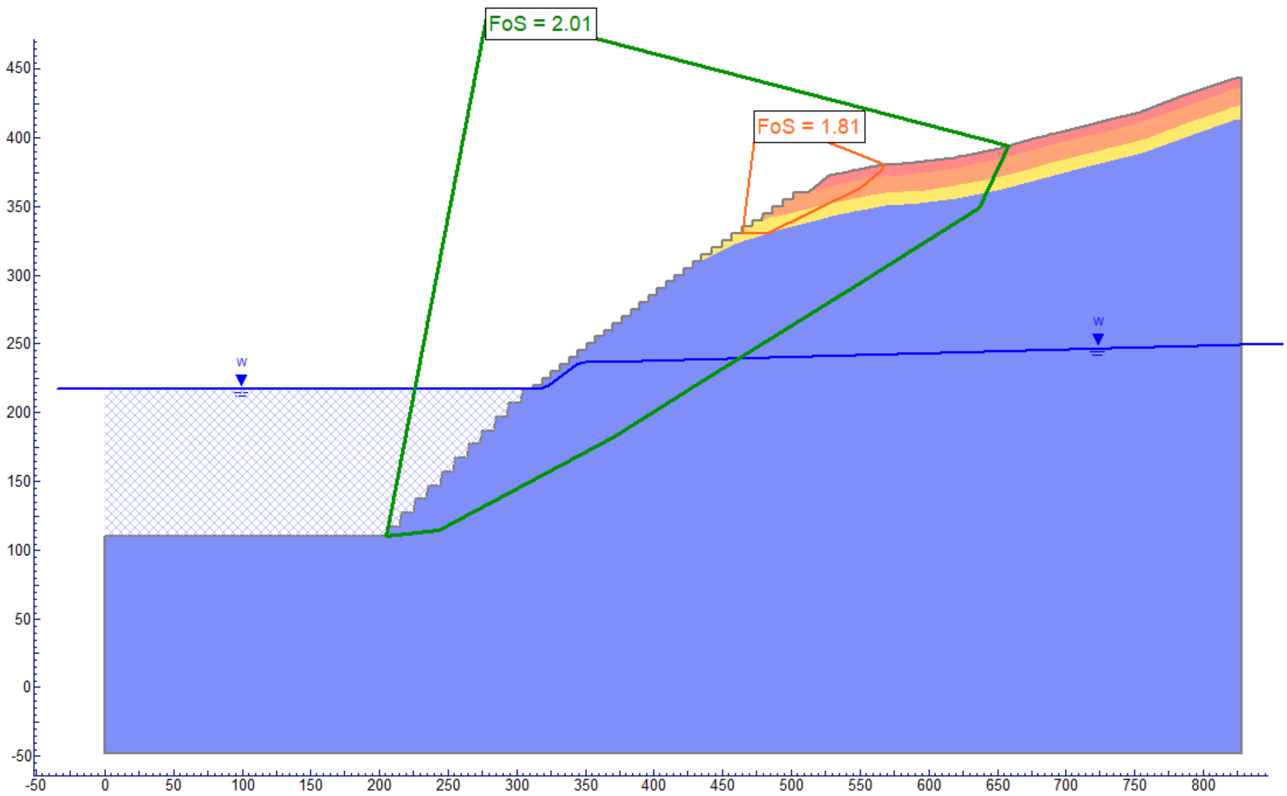


Figure 80 ***Sensitivity Model Output – Extended Zone of Weathering – Section N1 – Rehabilitated Slopes***

6.5.2 Seismic Loading

Sensitivity analyses has been undertaken to assess the potential stability implications associated with seismic loading events on the slope stability performance. A 1:500-year return event or 'expected' seismic loading event was undertaken on the terminal and rehabilitated geometries, and an additional 'worst case' seismic loading event equal to 1:2500-year return event was undertaken on the rehabilitated geometries.

A pseudo-static seismic analysis approach was adopted following the methodology outlined by Hynes-Griffin and Franklin (1984), referred to as the USACE method, which assumes:

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

When undertaking a pseudo-static seismic analyses, Hynes-Griffin and Franklin (1984) recommend using a seismic coefficient equal to half of the Peak Ground Acceleration (PGA). PGA values for a 1:500-year and 1:2500-year return events were obtained from the National Seismic Hazard Assessment (NSHA, 2018) maps. An example of the NSHA (2018) seismic hazard atlas map is depicted in Figure 81 for a 1:500-year return event.

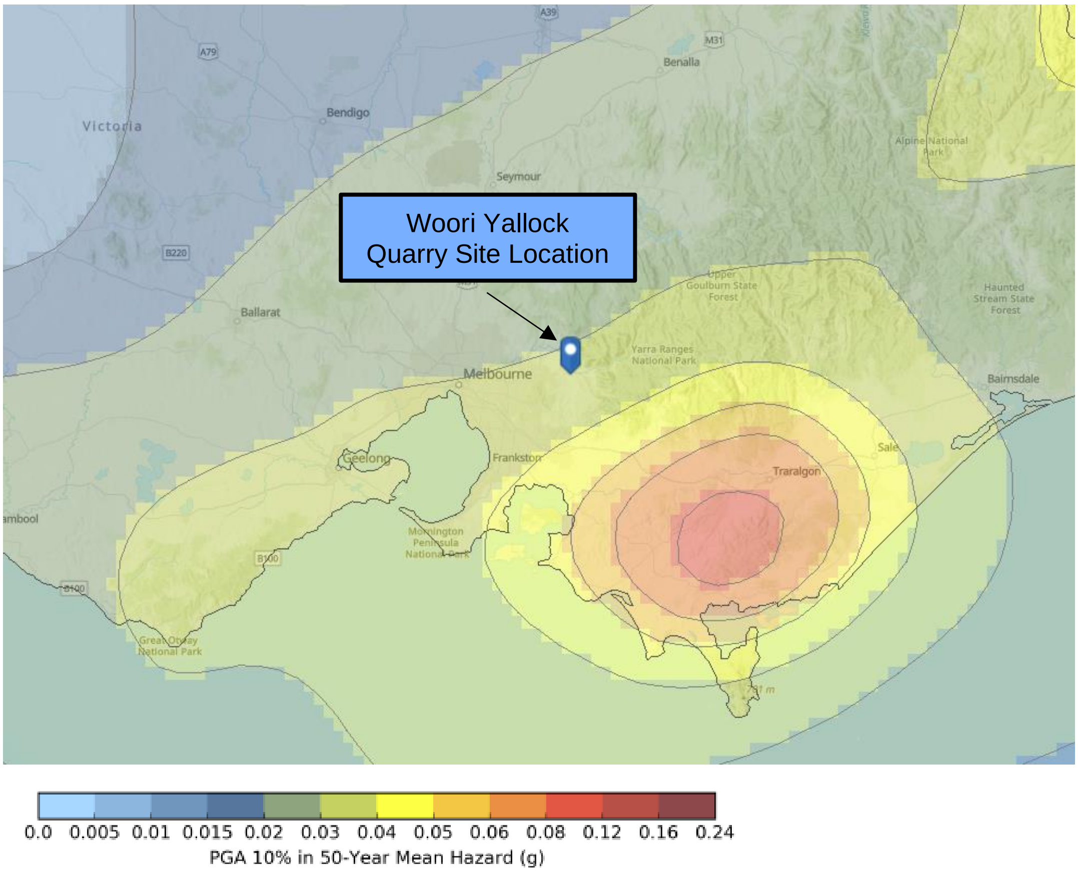


Figure 81 Seismic Hazard Atlas Map (1:500-year) Woori Yallock Quarry, after NSHA (2018)

Summarised in Table 33 are the PGA and seismic coefficient values adopted for this sensitivity analyses. It should be noted that seismic sensitivity analyses considered only the overall scale slope stability performance.

Table 33 Summary of Seismic Input Parameters

Return Interval	Peak Ground Acceleration (g) - PGA	Seismic Coefficient - H_z
1:500	0.035	0.0175
1:2500	0.090	0.0450

The results of the seismic sensitivity analyses are summarised in Table 34 below and the corresponding stability model outputs are presented in Figure 82 to Figure 90.

Table 34 Summary of Sensitivity Analyses Results – Seismic Loading

Stability Section	Return Interval	DAC	Scenario	Calculated FoS
N1	1:500	FoS > 1.1	Terminal	1.81
			Rehabilitated	1.78
	1:2500			1.65
S1*	1:500		Terminal	2.51
			Rehabilitated	2.71
	1:2500			2.20
W1	1:500		Terminal	2.54
			Rehabilitated	3.51
	1:2500			3.21
* Considering overall stability through the placed rock slope and in-situ slope				

The results of the seismic loading sensitivity analyses indicate that:

– A decrease in the slope stability performance was calculated for all stability sections assessed, however, even under extreme ‘worst case’ seismic loading events the stability performance was calculated to satisfy the design performance objectives.

The results of the seismic loading sensitivity analyses highlight the robustness of the design geometry against increased loading conditions.

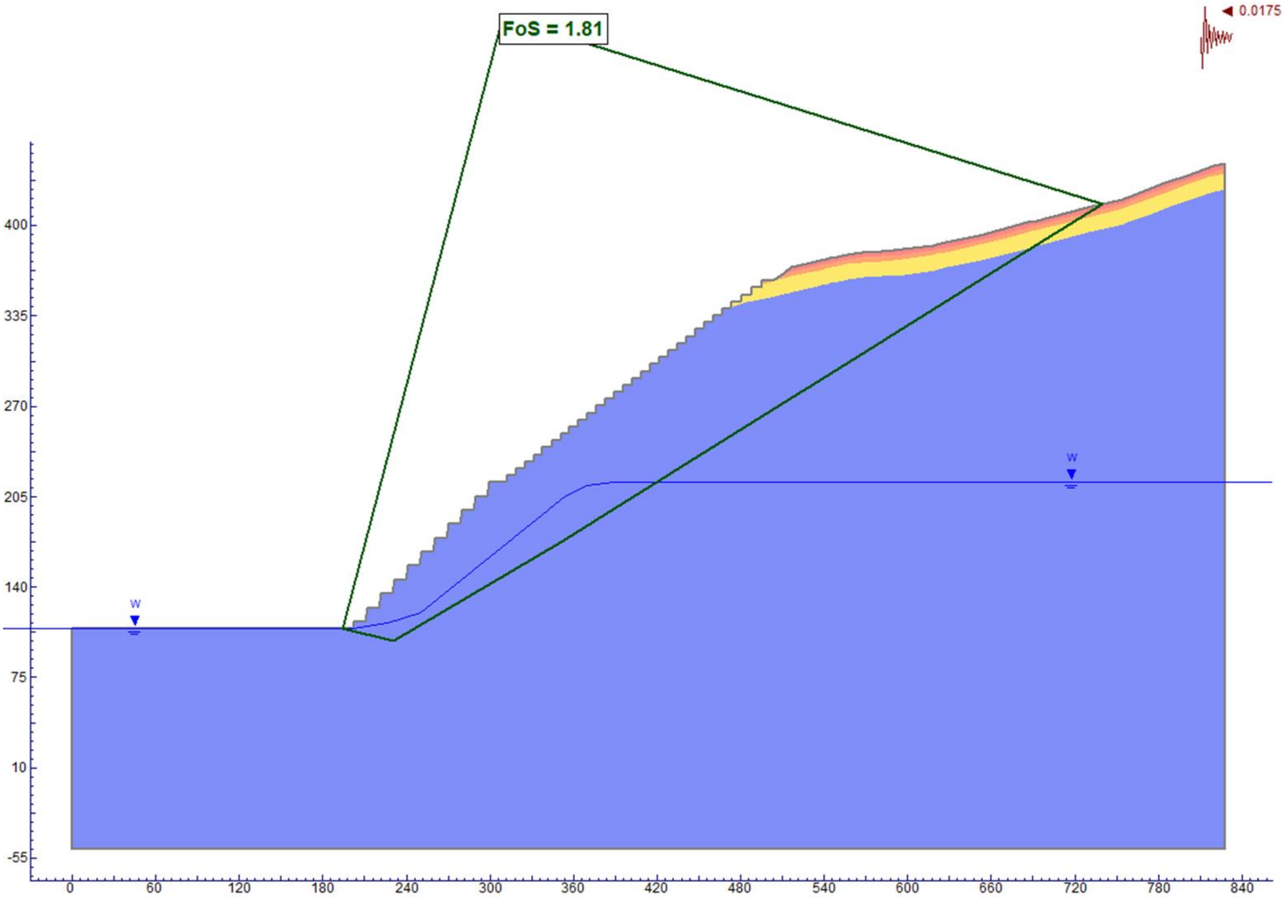


Figure 82 Sensitivity Model Output – Seismic Loading (1:500-year) – Section N1 – Terminal Slopes

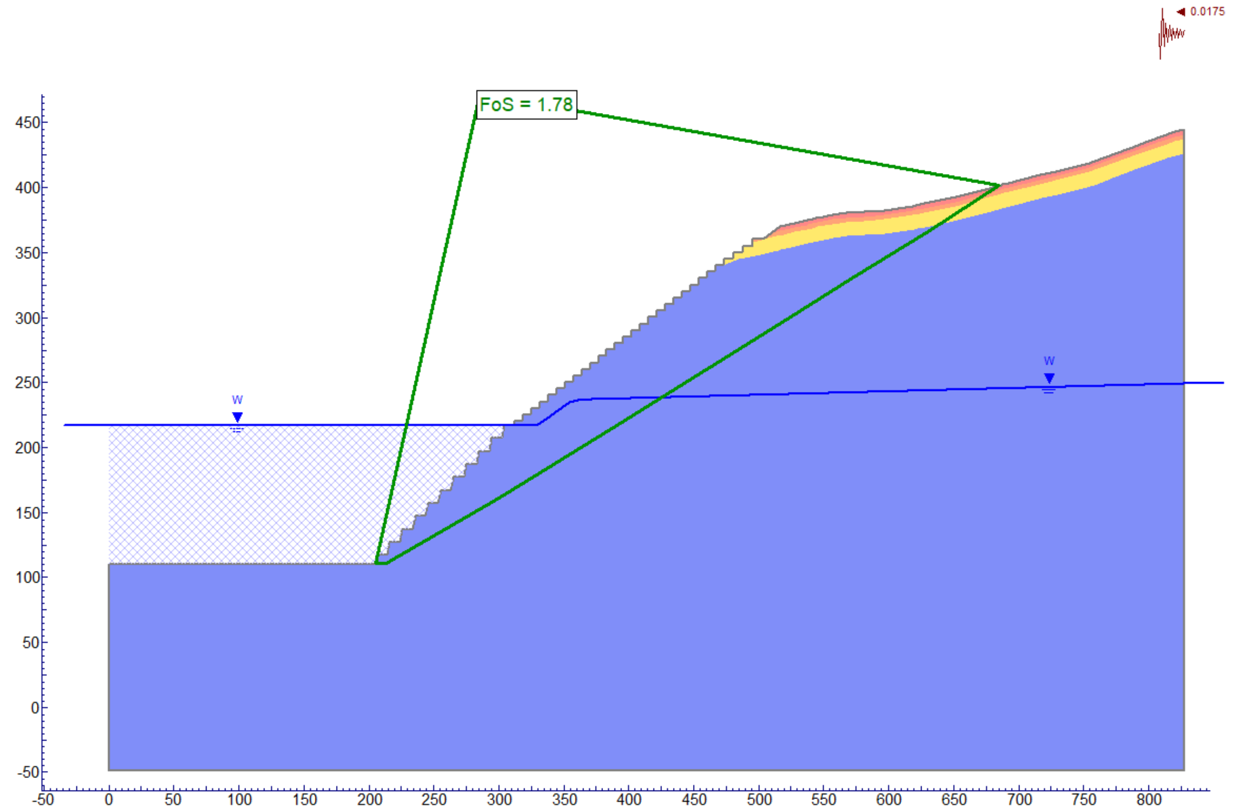


Figure 83 ***Sensitivity Model Output – Seismic Loading (1:500-year) – Section N1 – Rehabilitated Slopes***

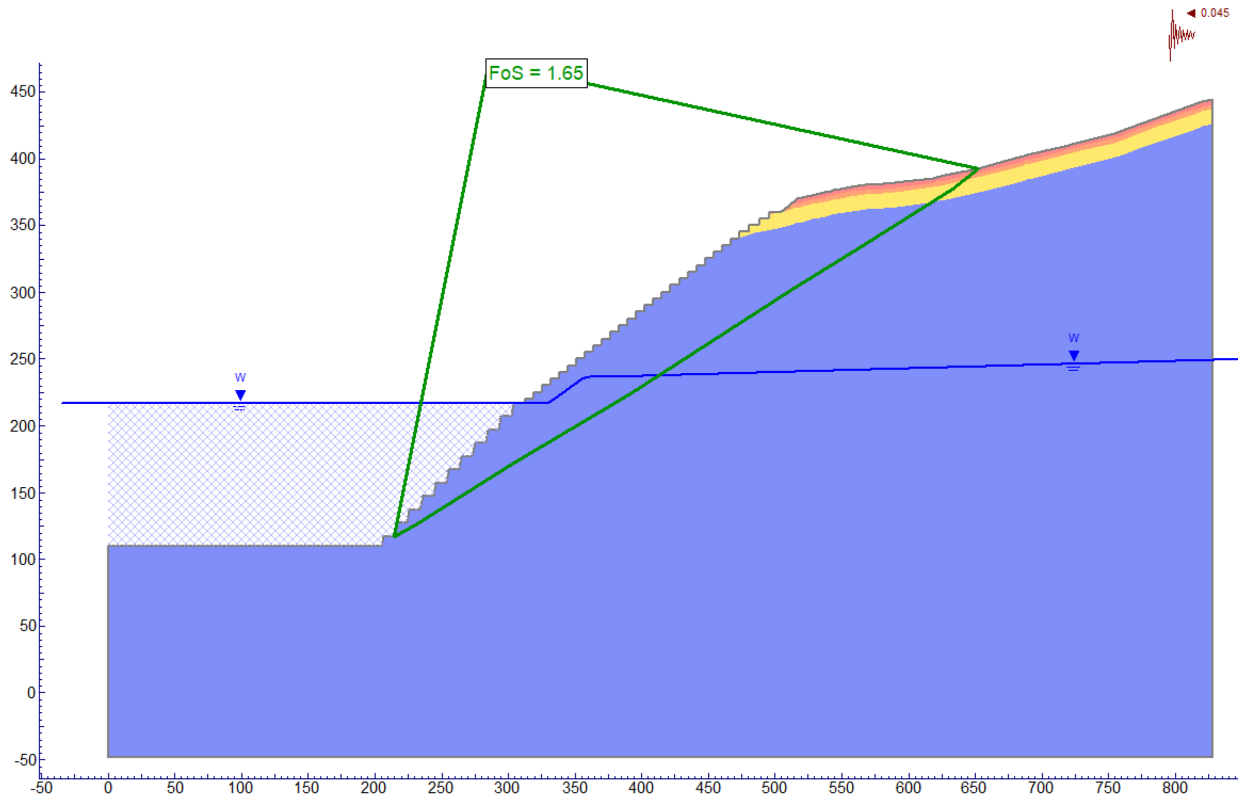


Figure 84 ***Sensitivity Model Output – Seismic Loading (1:2500-year) – Section N1 – Rehabilitated Slopes***

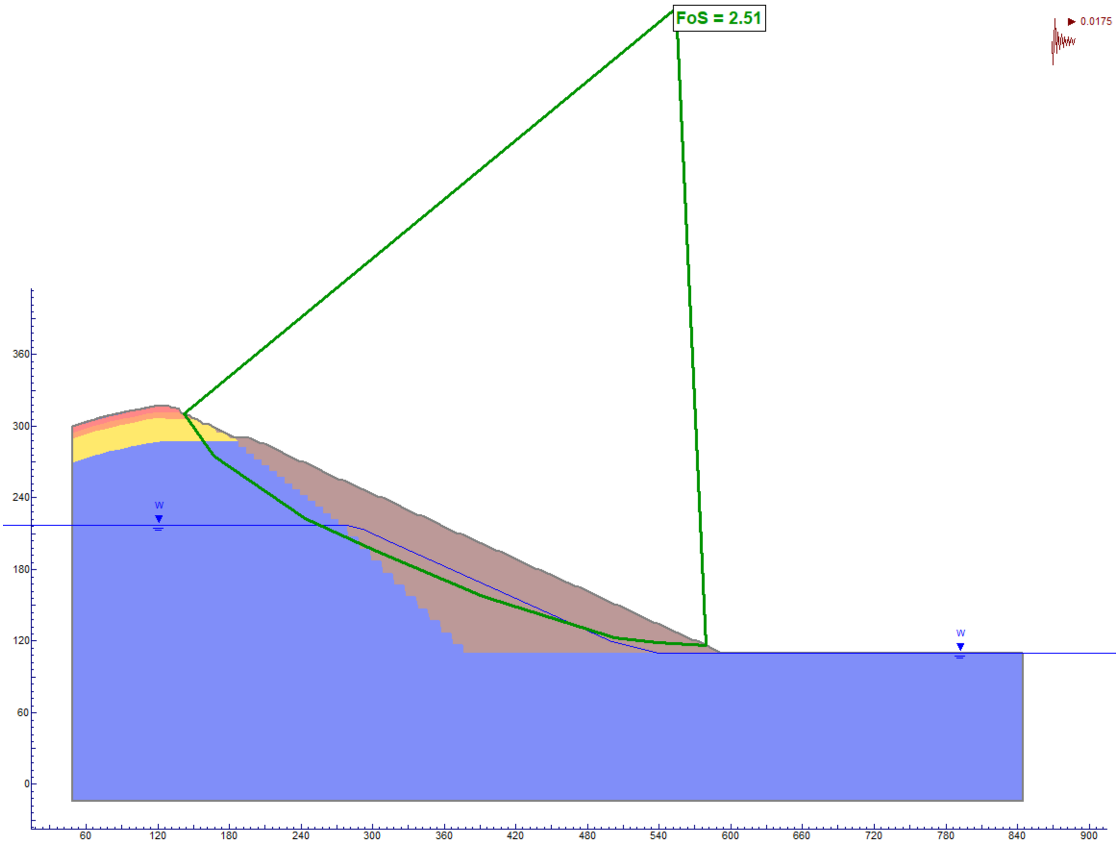


Figure 85 Sensitivity Model Output – Seismic Loading (1:500-year) – Section S1 – Terminal Slopes

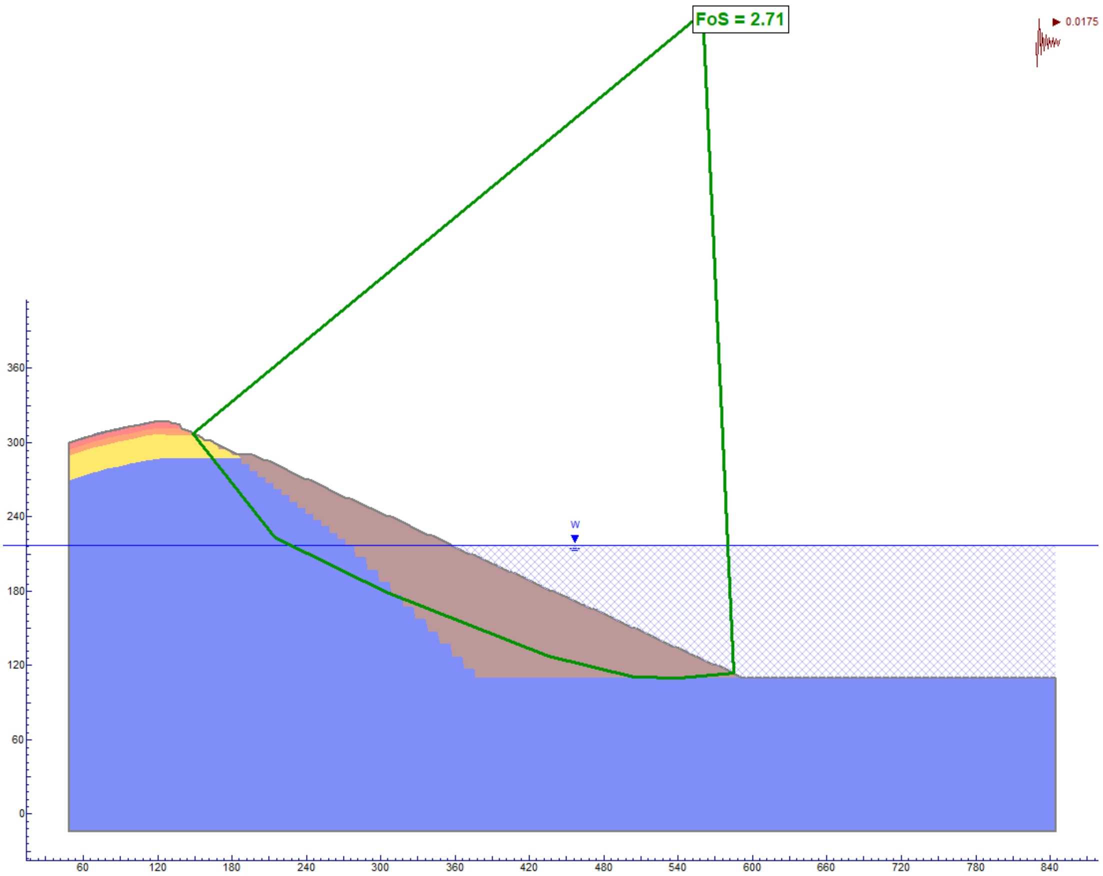


Figure 86 Sensitivity Model Output – Seismic Loading (1:500-year) – Section S1 – Rehabilitated Slopes

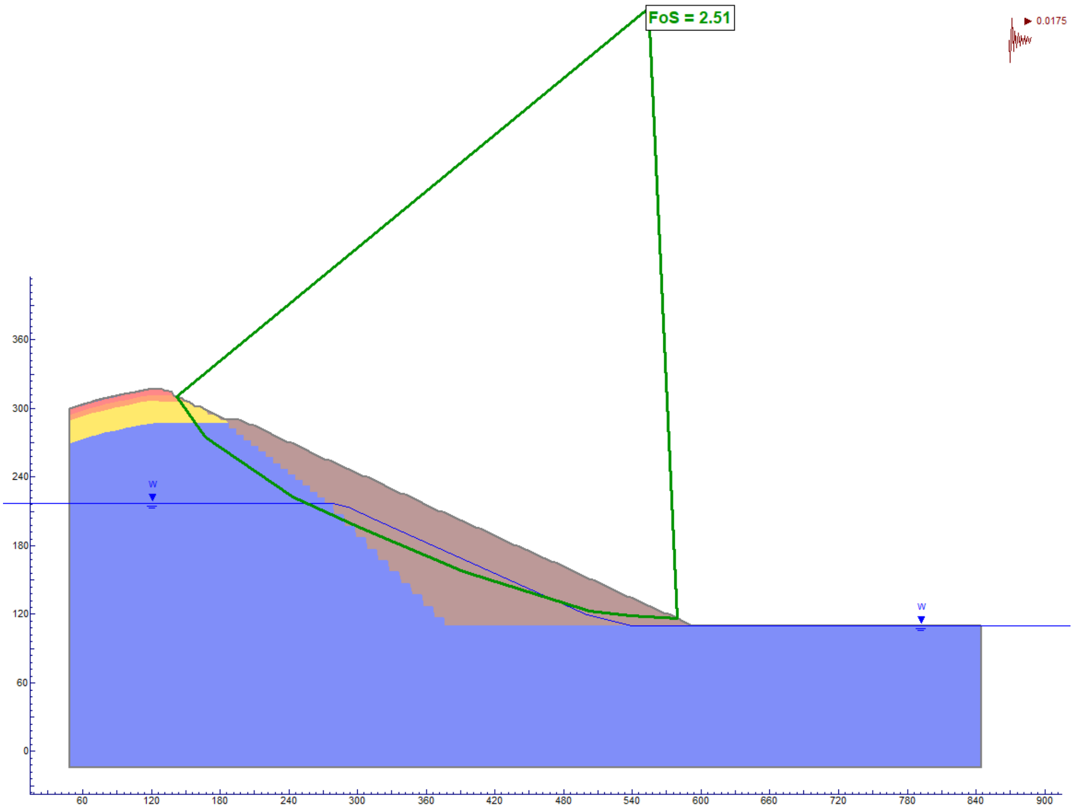


Figure 87 Sensitivity Model Output – Seismic Loading (1:2500-year) – Section S1 – Terminal Slopes

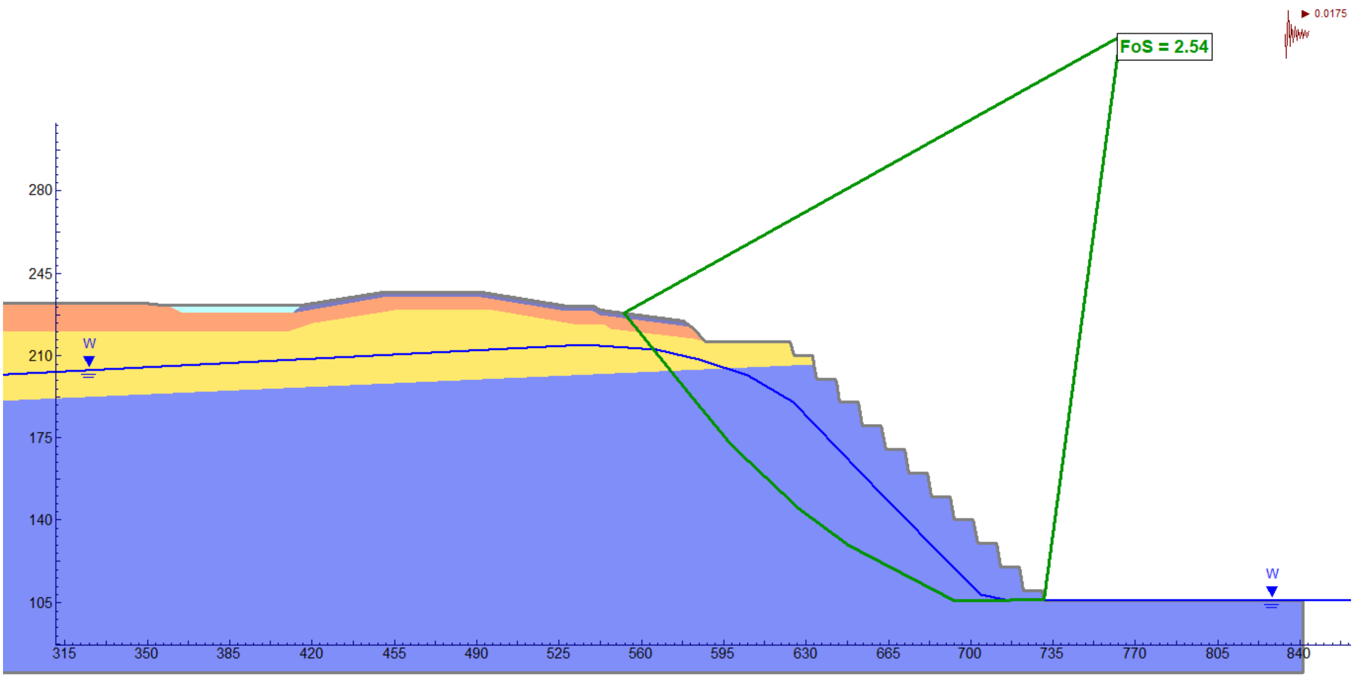


Figure 88 Sensitivity Model Output – Seismic Loading (1:500-year) – Section W1 – Terminal Slopes

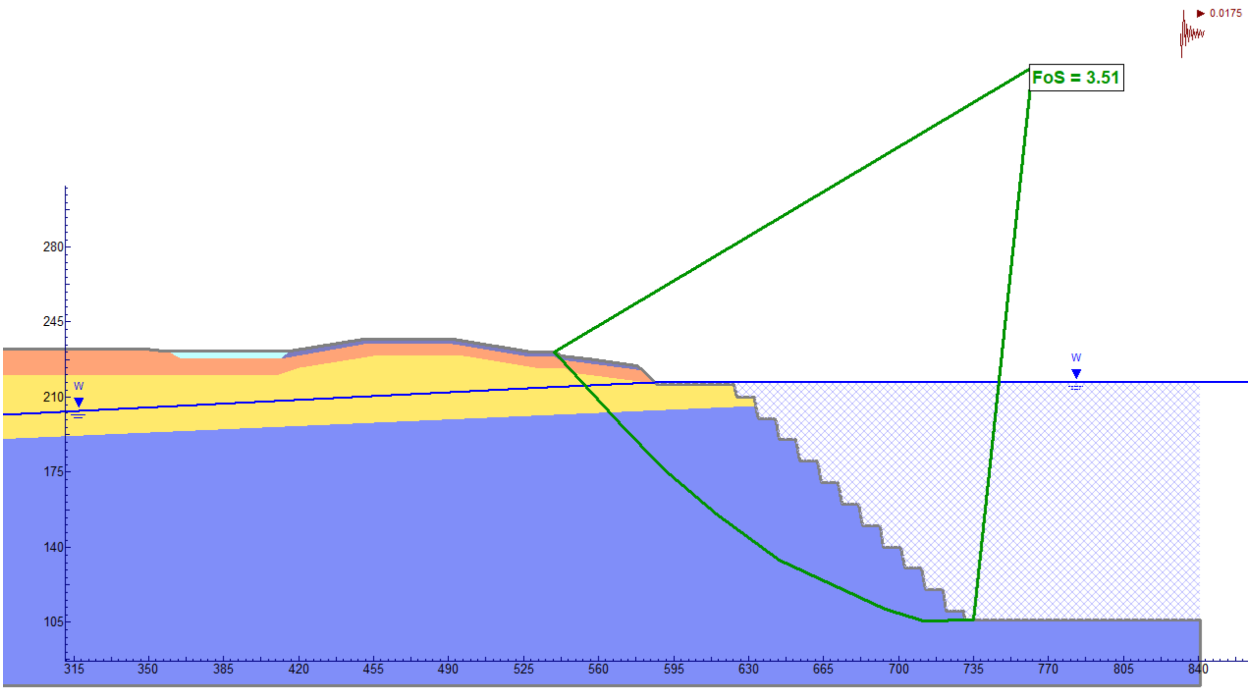


Figure 89 ***Sensitivity Model Output – Seismic Loading (1:500-year) – Section W1 – Rehabilitated Slopes***

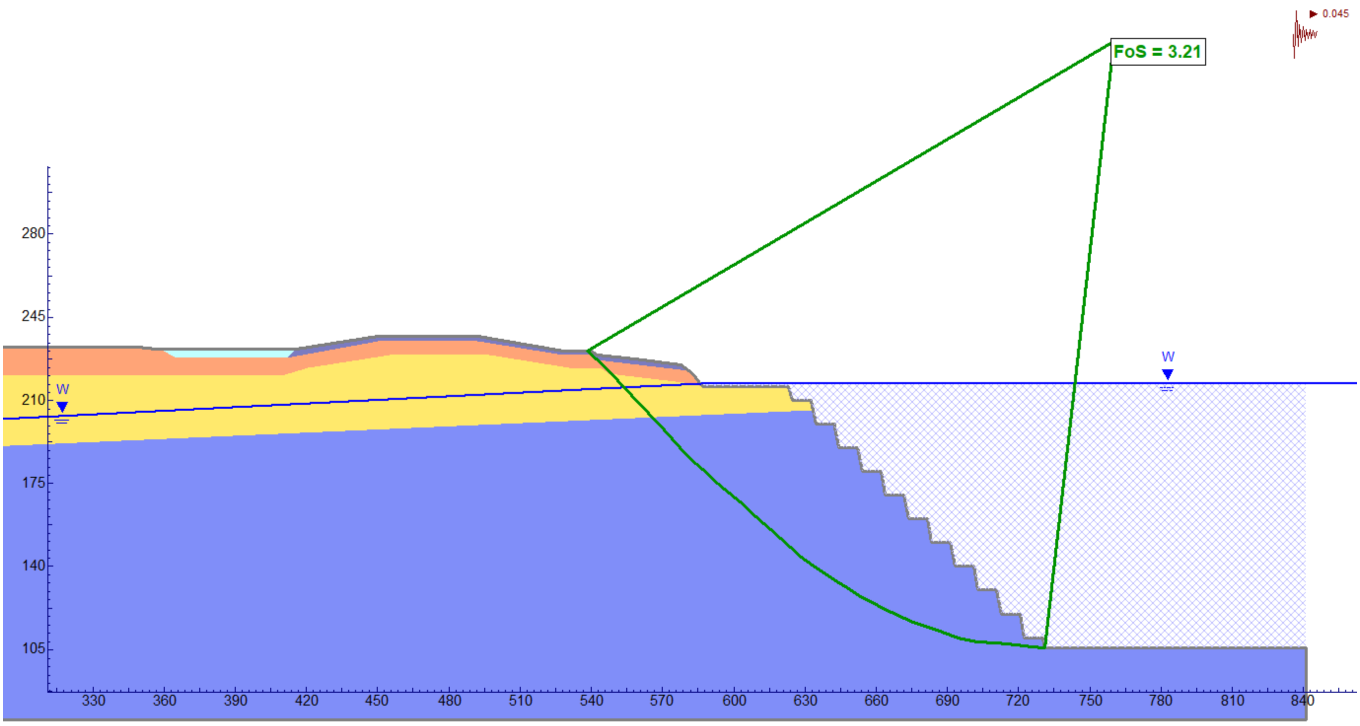


Figure 90 ***Sensitivity Model Output – Seismic Loading (1:2500-year) – Section W1 – Rehabilitated Slopes***

6.5.3 Elevated Phreatic Conditions

Sensitivity analyses has been performed to assess the potential stability implications associated with elevated phreatic conditions, where a fully saturated 'worst case' scenario was adopted. A fully saturated scenario is an extreme event and is unlikely to occur owing to the planned surface and groundwater management protocols that will be installed across the site.

The results of the elevated phreatic conditions sensitivity analyses are summarised in Table 35 and the corresponding stability model outputs are presented in Figure 91 to Figure 96.

Table 35 Summary of Sensitivity Analyses Results – Elevated Phreatic Conditions

Stability Section	Scenario	Scale of Instability	Calculated FoS*
N1	Terminal	Multi-bench	N/A
		Overall slope	1.02
	Rehabilitated	Multi-bench	1.10
		Overall slope	1.16
S1	Terminal	Multi-bench	0.81
		Overall slope	1.00
	Rehabilitated	Multi-bench	0.88
		Overall slope	1.28
W1	Terminal	Multi-bench	N/A
		Overall slope	2.02
	Rehabilitated	Multi-bench	N/A
		Overall slope	3.50
* N/A = Critical instability plane calculated at the overall slope scale and no multi-bench scale calculated.			

The results of the elevated phreatic condition sensitivity analyses indicate that:

– A reduction in the slope stability performance was calculated under 'worst case' fully saturated conditions compared to 'expected' conditions for all assessed stability sections considering both the terminal and rehabilitated slopes.
  - The calculated stability performance was calculated to be below unity (i.e., FoS < 1.0) along Section S1 when considering multi-bench scale stability. However, as depicted in Figure 93 and Figure 94, it is noted that these slip surfaces are typically thin-skinned and will be mitigated where suitable surface water management protocols are in place (as per the surface and groundwater management plan currently being prepared by Water Technology Pty Ltd).
  - However, at the overall slope scale the stability performance was calculated to be above unity (FoS > 1.00) for all stability sections and all scenarios, which indicates a robustness of slope design geometry against varied conditions.

– The slope stability performance of rehabilitated slopes was calculated to be higher than terminal slopes under fully saturated conditions.

The results of the elevated phreatic condition sensitivity analyses highlight the importance of implementing and maintaining adequate surface and groundwater management protocols which are integrated into the final landform design.

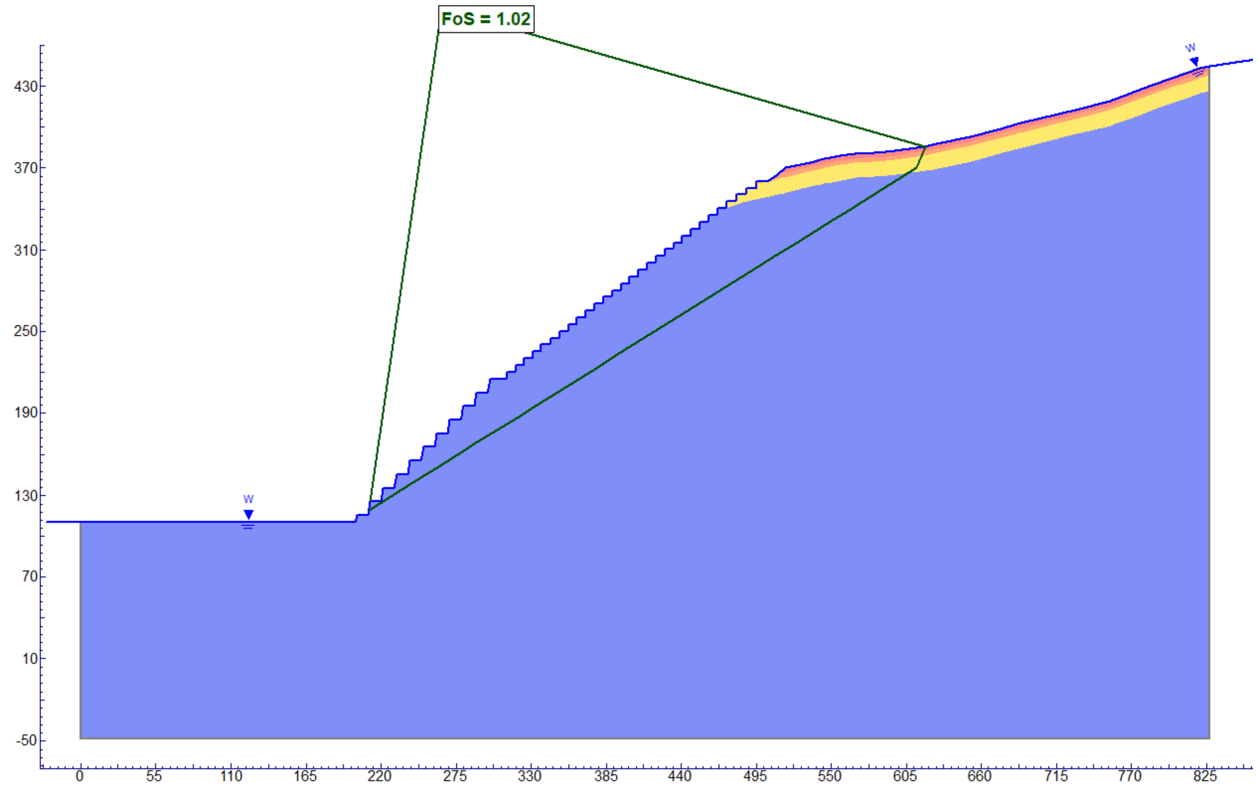


Figure 91 Sensitivity Model Output - Elevated Phreatic Conditions – Section N1 – Terminal Slopes

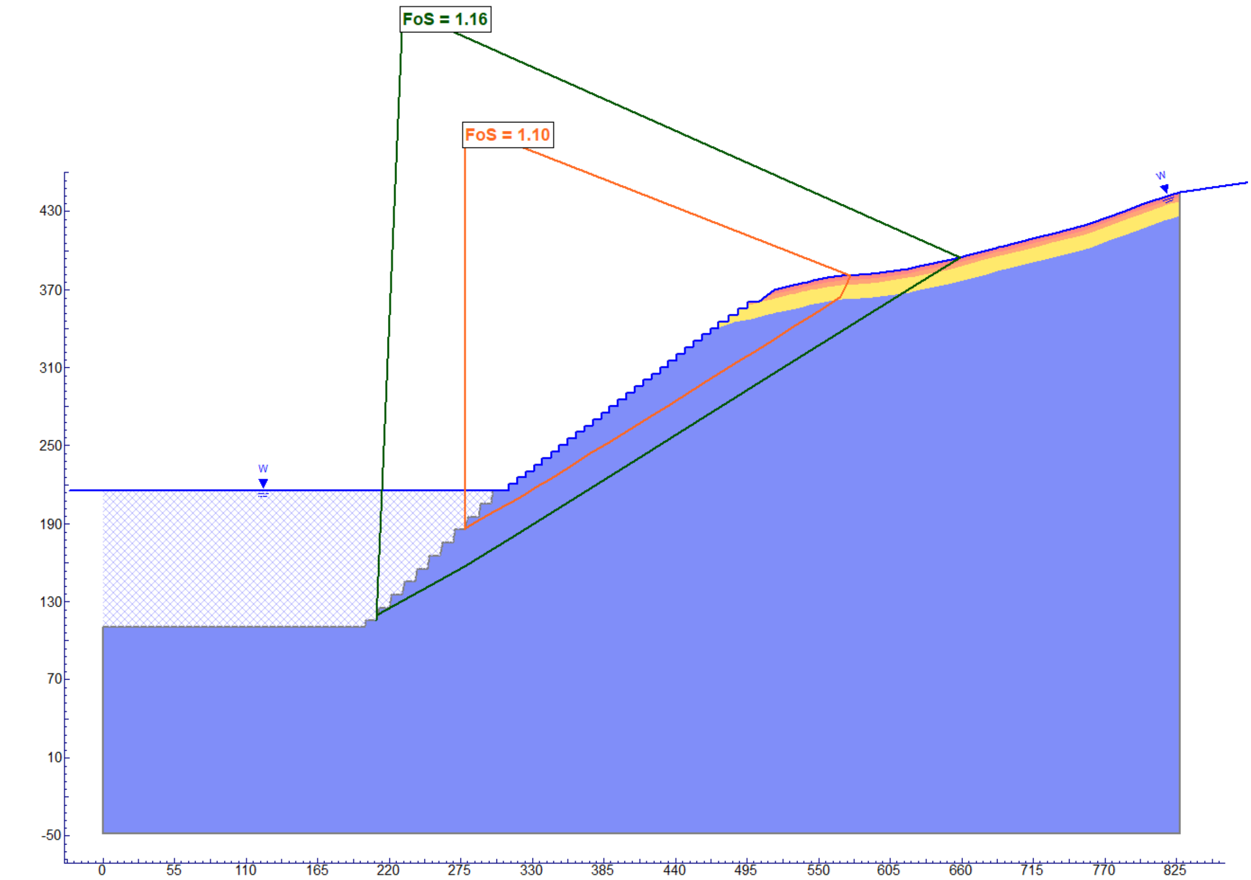


Figure 92 Sensitivity Model Output - Elevated Phreatic Conditions – Section N1 – Rehabilitated Slopes

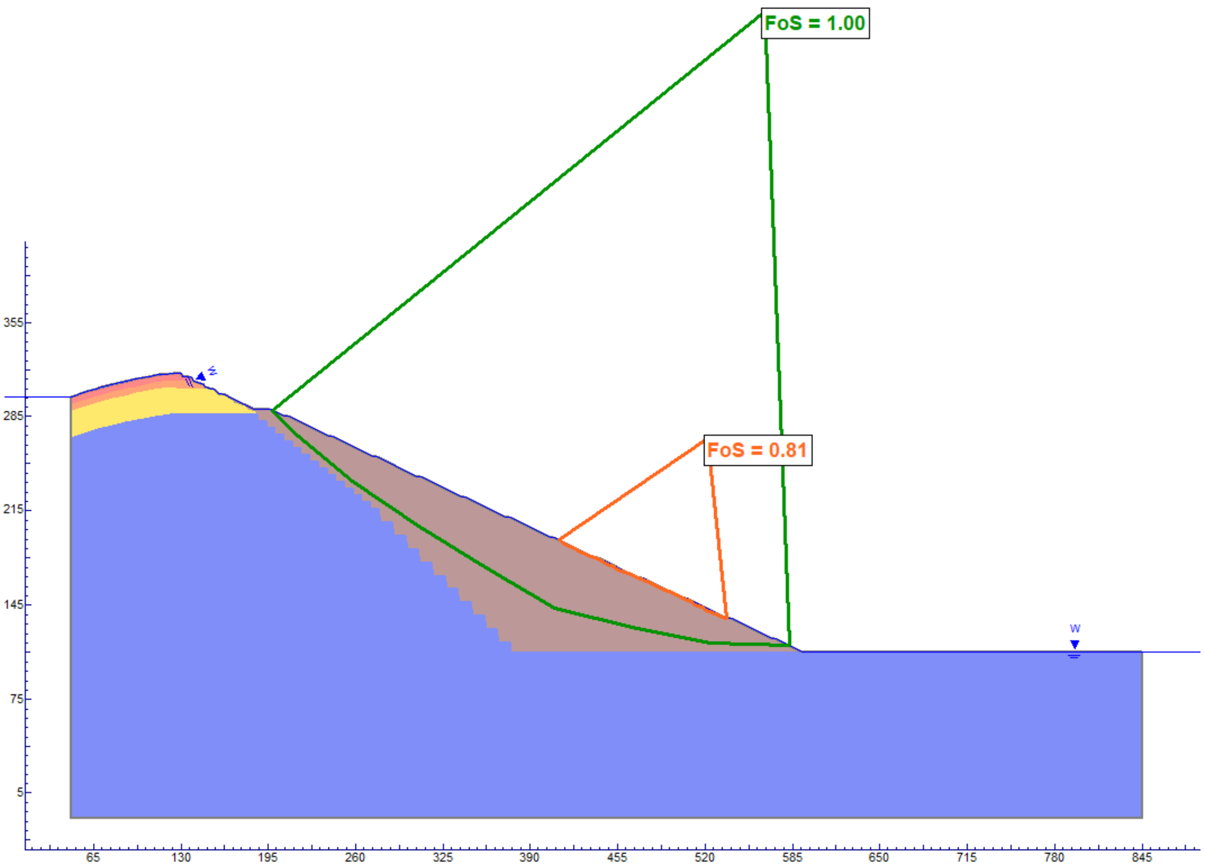


Figure 93 *Sensitivity Model Output - Elevated Phreatic Conditions – Section S1 – Terminal Slopes*

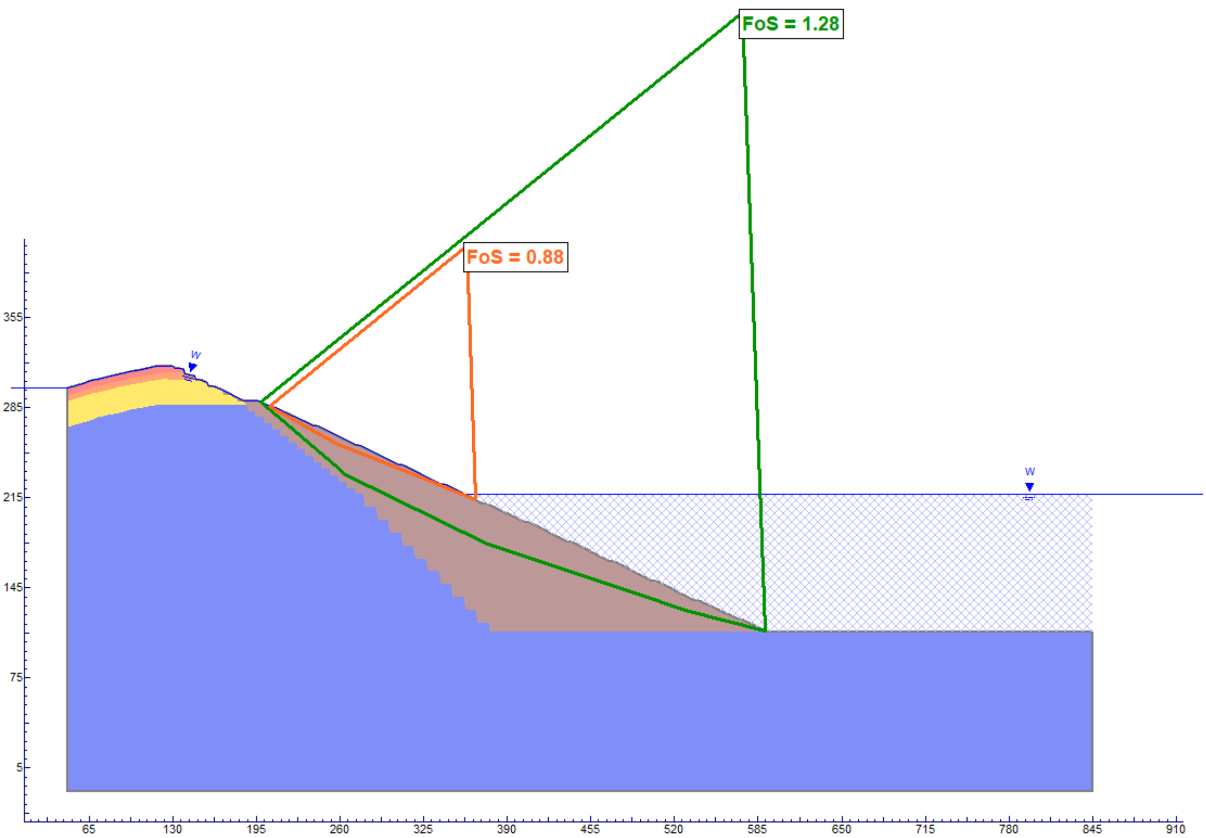


Figure 94 *Sensitivity Model Output - Elevated Phreatic Conditions – Section S1 – Rehabilitated Slopes*

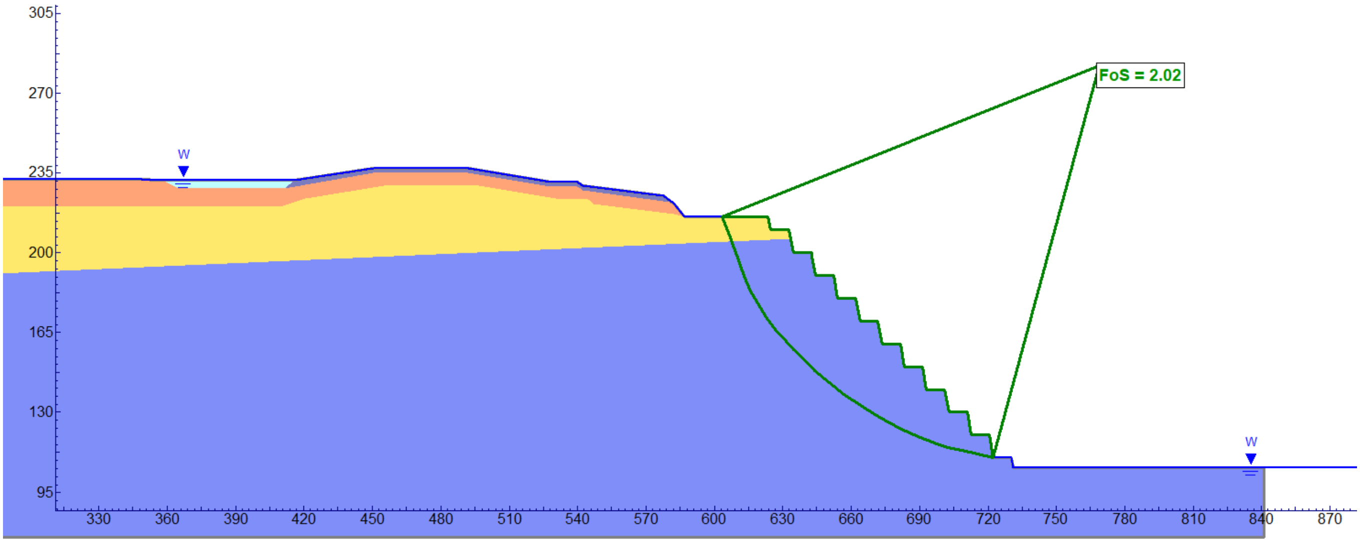


Figure 95 *Sensitivity Model Output - Elevated Phreatic Conditions – Section W1 – Terminal Slopes*

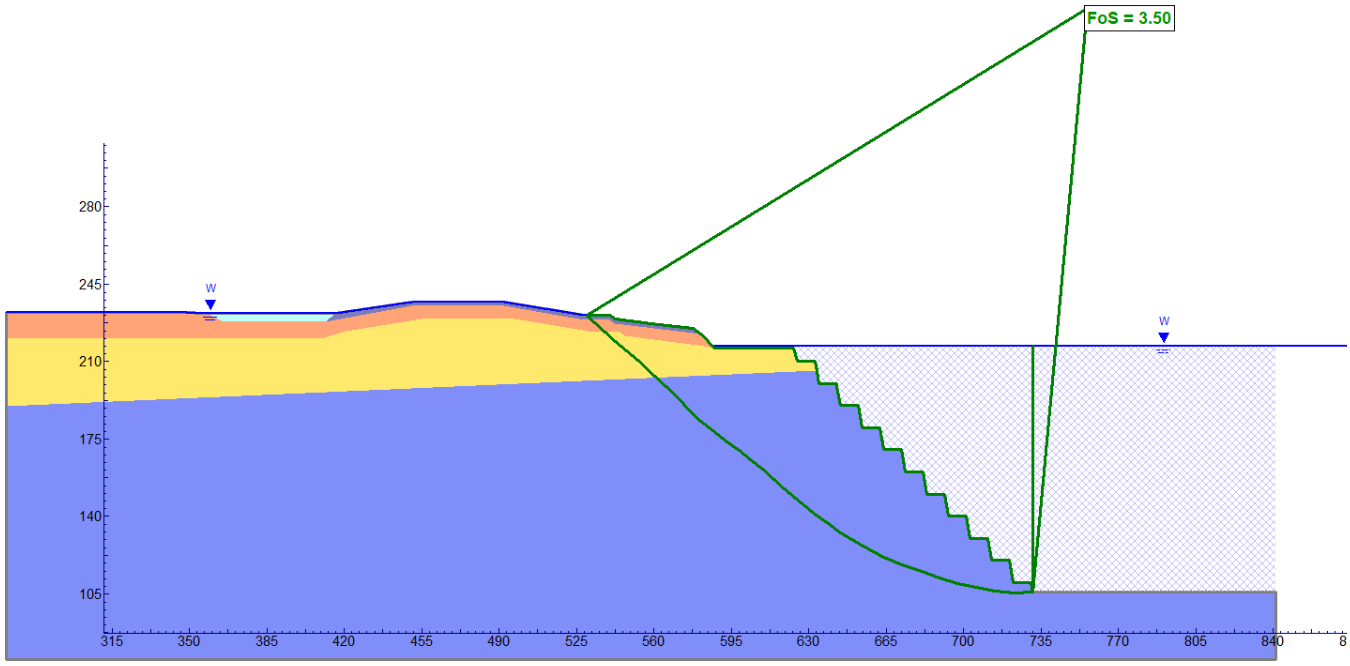


Figure 96 *Sensitivity Model Output - Elevated Phreatic Conditions – Section W1 – Rehabilitated Slopes*

6.5.4 Influence of Final Pit Lake Level

The final pit lake level is expected to vary over time owing natural seasonality cycles, for example. The pit lake level is expected to be lower in summer periods where evaporation is higher, and rainfall is lower. Accordingly, sensitivity analyses were performed to assess the influence of a lower final pit lake level on the stability performance of rehabilitated slopes. For the purpose of this assessment, a lower pit lake level of RL 207 (i.e., 10 m below the final lake level) was considered.

The results of the final pit lake sensitivity analyses are summarised in Table 36 and the corresponding stability model outputs are presented in Figure 97 to Figure 99.

Table 36 Summary of Sensitivity Analyses Results- Influence of Final Pit Lake Level

Stability Section	DAC	Pit Lake Level	Scale of Instability	Calculated FoS
N1	FoS > 2.0	RL +207 m	Multi-bench	N/A
			Overall Slope	1.97
S1			Multi-bench	1.64
			Overall (Rock Slope)	1.67
			Overall Slope	2.61
W1			Multi-bench	N/A
			Overall Slope	3.47
* N/A = Critical instability (minimum FoS) occurs at the overall slope scale.				

The results of the sensitivity analyses undertaken to assess the influence of the final pit lake on the stability performance of rehabilitated slopes indicate that:

- The stability performance of rehabilitated slopes along Sections N1, S1 and W1 were calculated to be slightly lower where lower final pit lakes levels were adopted compared to higher lake levels, however, remain within tolerable thresholds of the design performance objectives at both the multi-bench and overall slope scales.

The results of the final pit lake sensitivity analyses highlight the robustness of the design geometry against varying natural conditions.

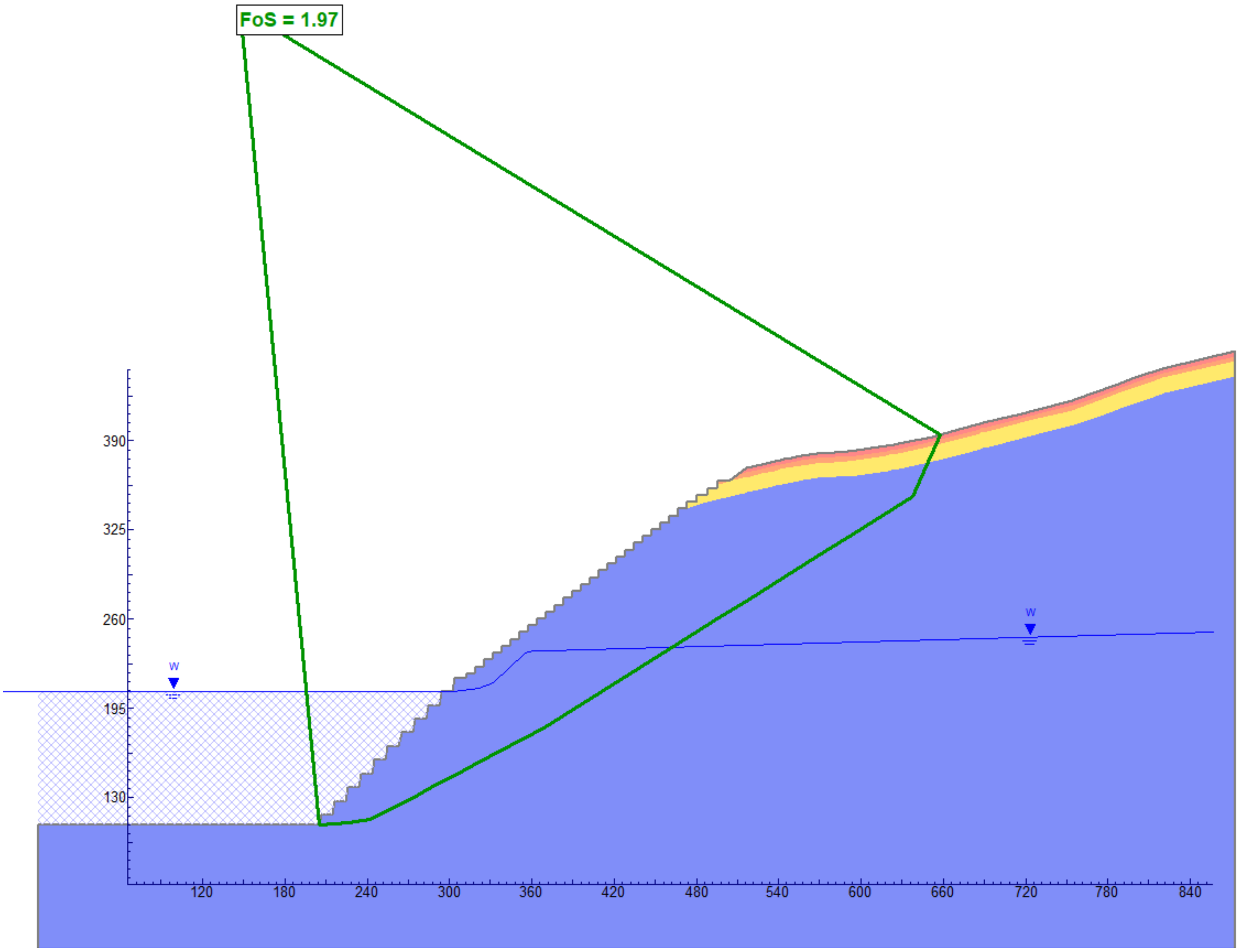


Figure 97 Sensitivity Model Output - Pit Lake Level (RL +207 m) – Section N1 – Rehabilitated Slopes

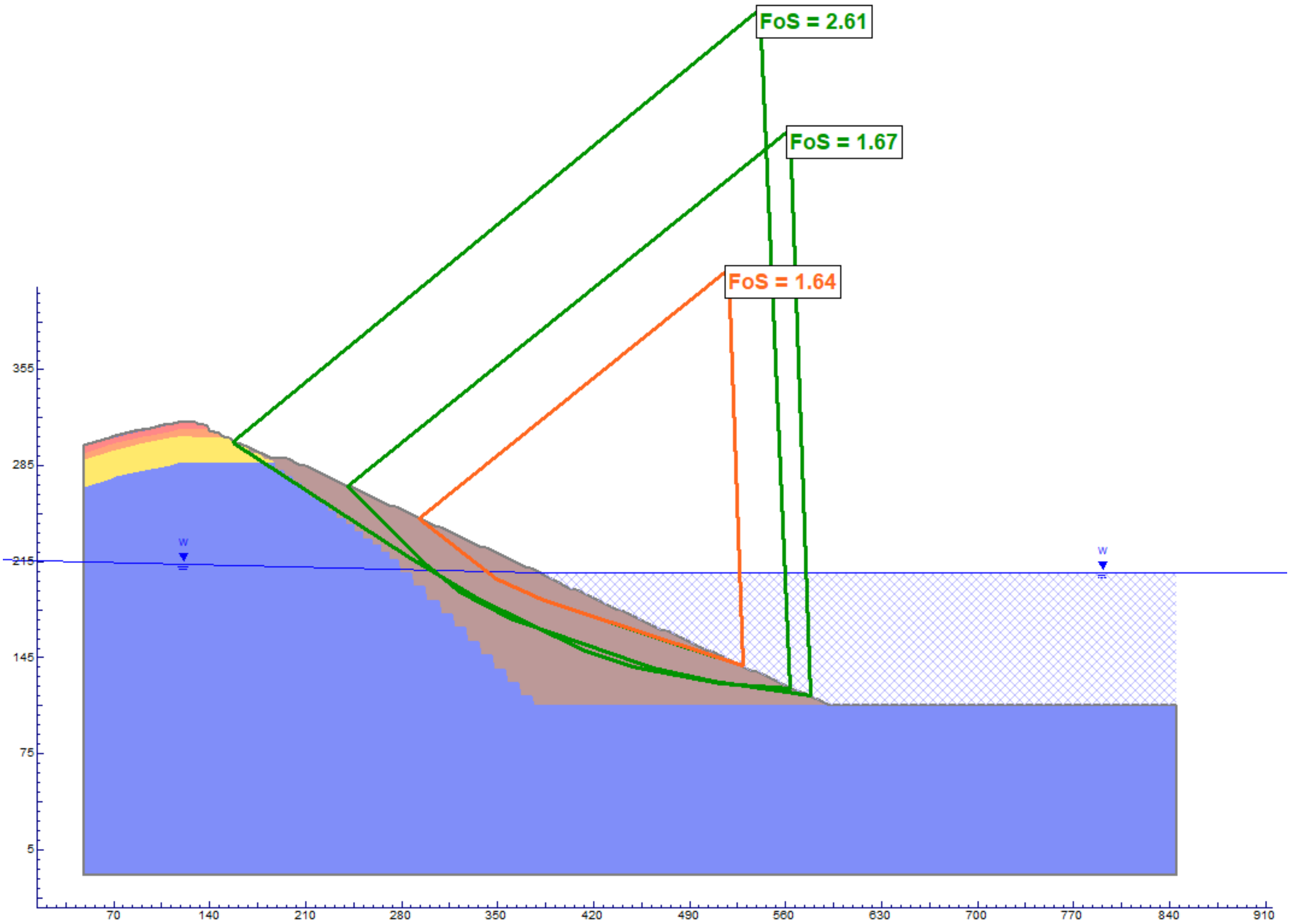


Figure 98 *Sensitivity Model Output - Pit Lake Level (RL +207 m) – Section S1 – Rehabilitated Slopes*

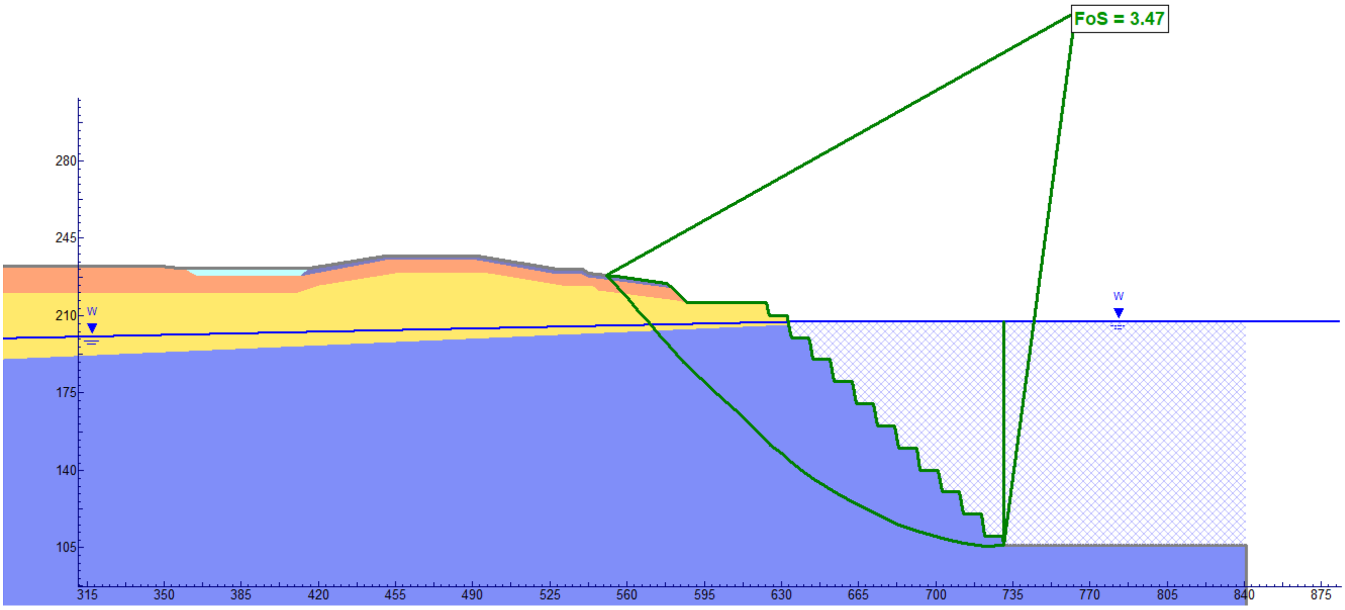


Figure 99 *Sensitivity Model Output - Pit Lake Level (RL +207 m) – Section W1 – Rehabilitated Slopes*

6.5.5 Influence of Degraded Material Strengths

Under long-term closure conditions i.e., rehabilitated slopes, will be subjected to erosion due to natural physical erosion processes which is likely to result in a degradation of slope material strengths. The degree to which a rock is affected by weathering is referred to its durability, which is determined through Slake Durability Index tests. Typically, strong rock, which is the case of Woori Yallock Quarry batters have a high durability i.e., are resistant to weathering. Accordingly, degradation of the slope is likely to occur superficially impacting a thin veneer along the slope face, however, may occur at greater depths behind the slope face due to open fractures say as a result of blast damage. Weathered rock material, as in the case of the weathered rock stockpile may lose strength i.e., loss of interface friction.

Owing to the above, the following shear strength reductions have been applied:

- A 10% decrease in the intact rock UCS and defect plane JCS have been adopted to assess the potential implications of weathering on the stability performance along stability Section N1. It has been assumed that weathering occurs up to 15 m behind the slope face.
- A 10% reduction in shear strength parameters have been adopted for Section W1. It is considered that weathering occurs up to 15 m behind the slope face.
- In addition, a 10% reduction in the interface friction of the weather rock material has been adopted ubiquitously for the weathered rock stockpile to assess the potential stability implications of weathering on the stability performance along stability Section S1.

The results of the sensitivity analyses are presented in in Table 37 and the corresponding stability model outputs are presented in Figure 100 to Figure 102.

Table 37 Summary of Sensitivity Analyses Results – Degraded Material Strengths

Stability Section	DAC*	Scale of Instability	Calculated FoS
N1	FoS > 2.0	Multi-bench	1.36
		Overall	1.76
S1		Multi-bench	1.42
		Overall (Rock Slope)	1.43
		Overall	2.56
W1		Multi-bench	N/A
		Overall	3.54
* Suitability of DAC discussed further below			

The results of the sensitivity analyses undertaken to assess the influence of degraded material strengths indicate that:

- At the overall slope scale the stability performance along Section N1, S1 and W1 were calculated to be lower than under ‘expected’ conditions, though remain within tolerable thresholds of the nominated DAC.
- At the multi-bench scale, instability is more likely to occur along stability Section N1 and S1 compared to ‘expected’ case, noting that the stability performance along both Section N1, S1 and W1 were calculated to be within tolerable thresholds of the nominated DAC.

It is expected that it would take very long time periods for the slope materials to become degraded as a result of weathering 100’s to 1000’s of years. There may a propensity for degradation to occur about the natural lake fill level where fluctuations in the lake occurs as a result of seasonality effects due to fluid penetration and oxidation. It should be noted that there is currently no guidance on assessing stability performance of open cut excavations considering degraded material strengths, though ANCOLD (2019) recommend that where material strengths have become degraded as a result of cyclic loading to adopt a lower DAC. Taking this into account a lower FoS may be acceptable for long-term conditions where materials are likely to become degraded.

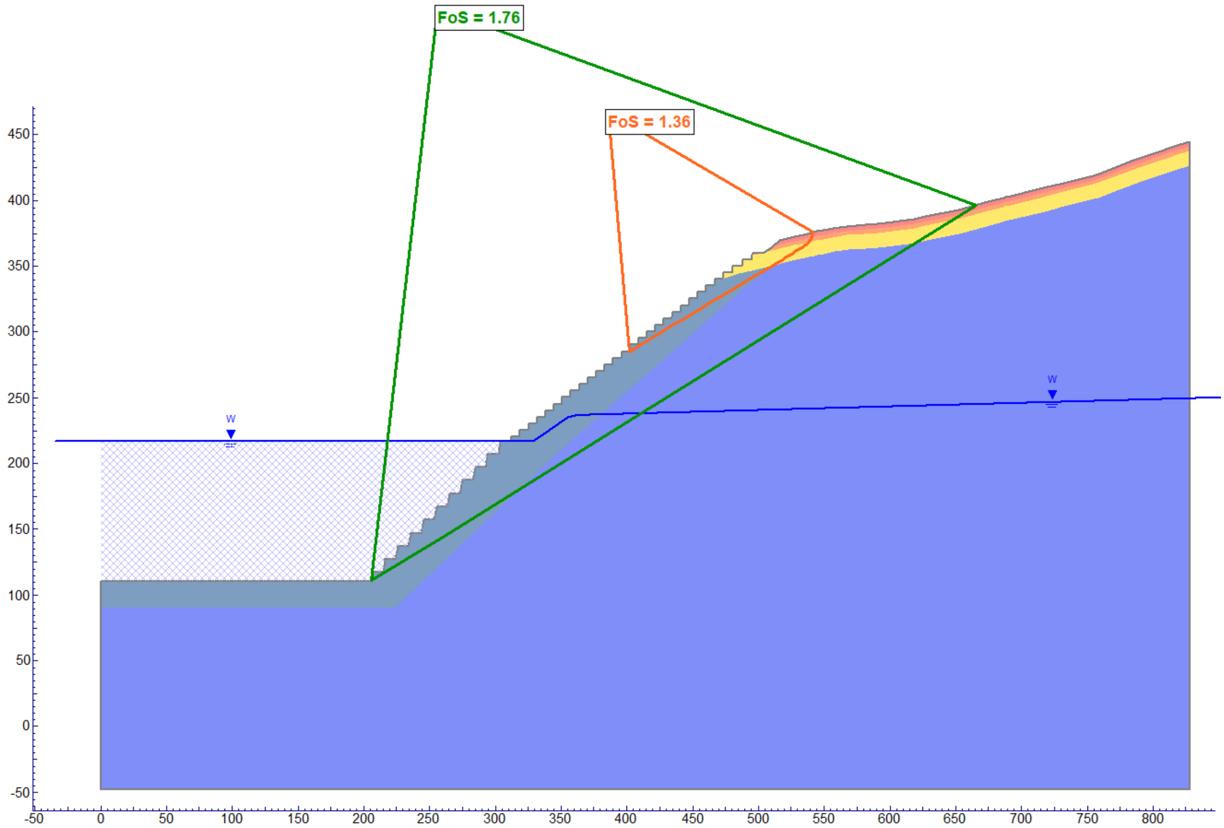


Figure 100 Sensitivity Model Output – Degraded Material Strengths – Section N1 – Rehabilitated Slopes

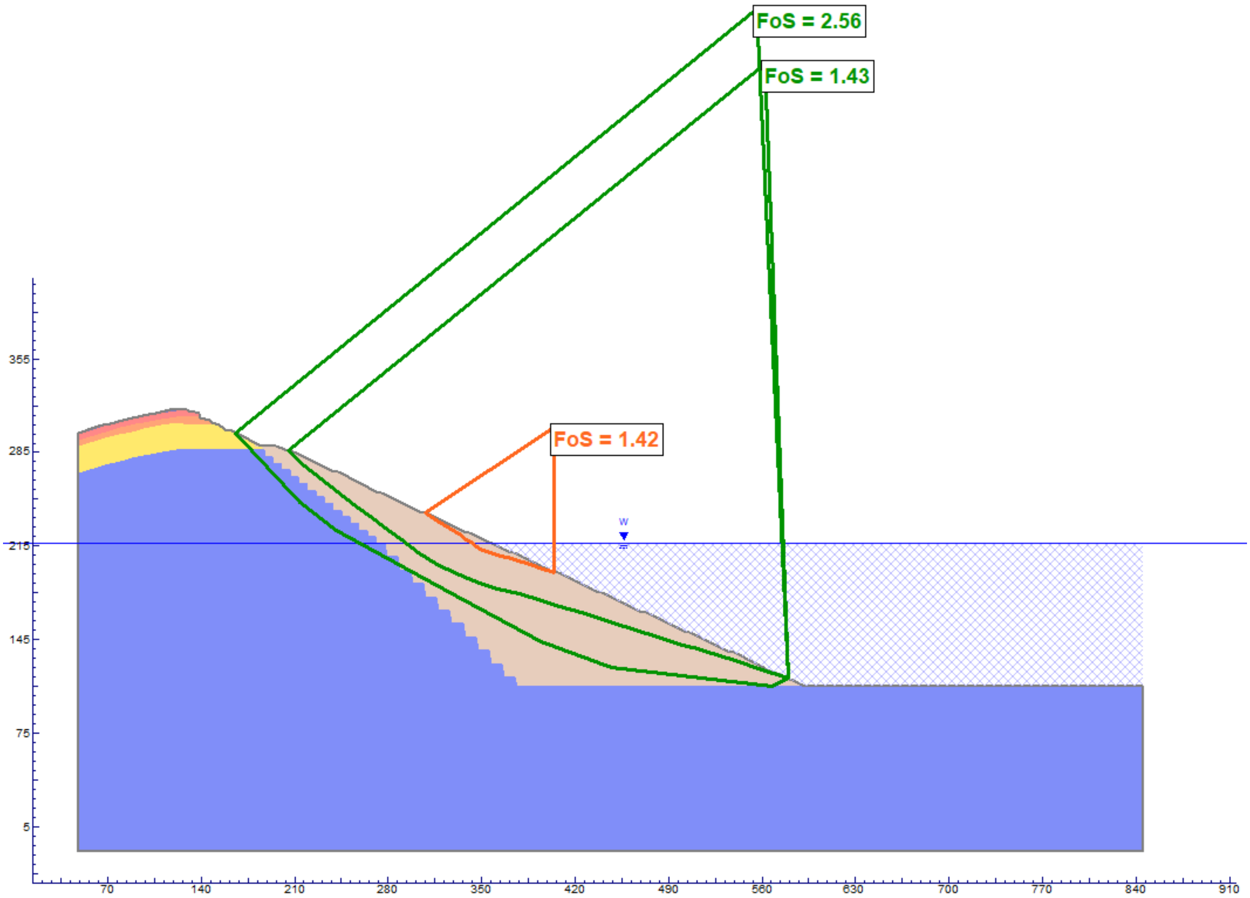


Figure 101 Sensitivity Model Output – Degraded Material Strengths – Section S1 – Rehabilitated Slopes

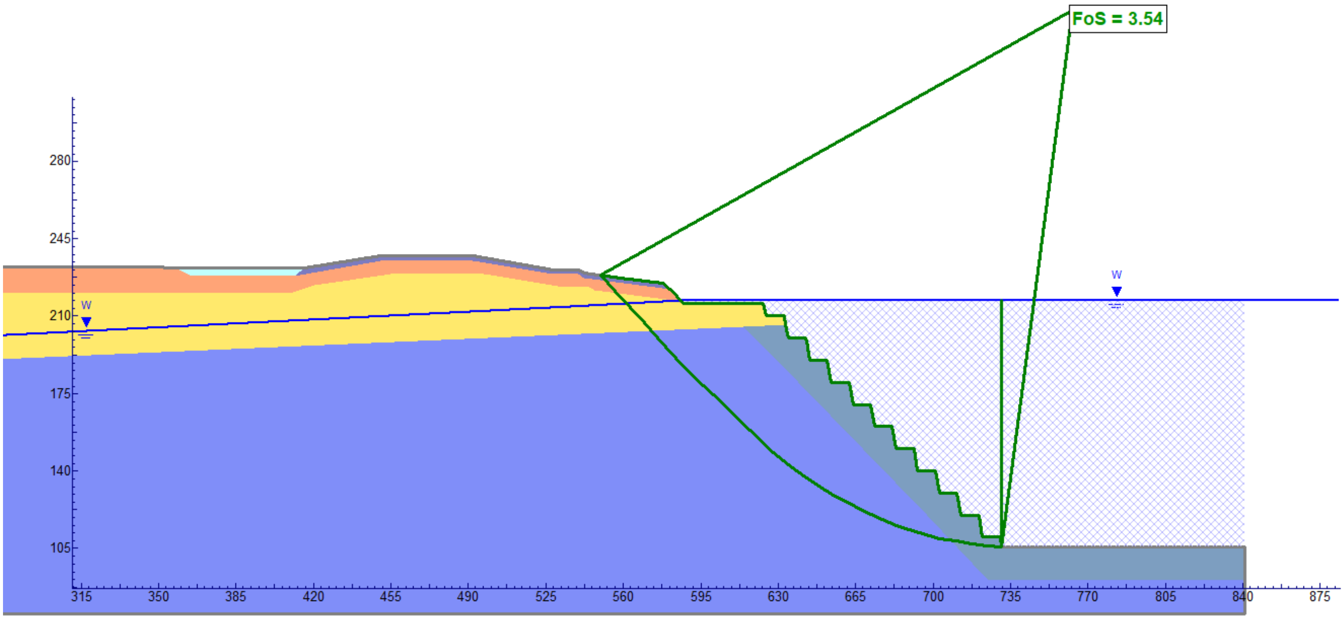


Figure 102 Sensitivity Model Output – Degraded Material Strengths – Section W1 – Rehabilitated Slopes

6.6 Discussion of Stability Analyses Results

The results of the stability modelling indicate that:

6.6.1 Kinematic Analyses

Planar Sliding

– Under expected conditions, east facing walls i.e., excavations within the eastern domain, are comparatively more sensitive to planar sliding than the other walls.

– The minimum calculated bench width for a single bench configuration with respect to each weathering grade was calculated (to the nearest metre) to be:

  - HW / MW to SW Hornfels = 7.0 m
  - Fr Hornfels = 6.0 m

– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.

– The volume of instability was calculated to be of limited scale/volume, with larger volumes calculated along the north (11.9 m^3/m) and south (6.2 m^3/m) walls.

Wedge Sliding

– Under expected conditions, east and west facing walls were calculated to be comparatively more sensitive (PoF > 50%) to tetrahedral wedge sliding compared to the remaining wall orientations.

– The minimum calculated bench width for a single bench configuration with respect to each weathering grade was calculated (to the nearest metre) to be:

  - HW Hornfels = 6 m
  - MW to SW Hornfels / Fr Hornfels = 5 m

– In general, the FoS increases and the PoF decreases where the degree of weathering decreases.

– The volume of instability was calculated to be of limited scale/volume where good blasting and scaling practices are employed. A notable increase was calculated along the north and south walls for the un-managed condition i.e., poor blasting and no scaling.

Rock Toppling and Rockfall

– Where the spacing of toppling joints occurs in the range of 0.6 m to 0.7 m or less, the likelihood of rock toppling becomes notably higher as the calculated FoS falls below unity for all stratigraphic units.

– The maximum trajectory of rockfall is likely to occur within 3.5 m from the batter toe, for rock mass instabilities in the order of ~0.5 m^3 to 2 m^3.

Double Benched Fresh Hornfels Batters

– The results of minimum berm width calculations undertaken for the double benched fresh Hornfels batters (bench height and width of 10 m and 8 m, respectively) located below the anticipated terminal pit lake level indicates the design geometry is suitable.

The design slope geometry adopted (as outlined in Section 6) satisfies the requirements for minimum berm widths for all proposed batter-berm configurations.

6.6.2 Slope Stability Analyses

Slope Stability

– Under 'expected' conditions:
  - The stability performance at the multi-bench and overall slope scales satisfy the nominated DAC for all assessed stability sections (i.e., Section N1, S1 and W1) for terminal slope profiles
  - The stability performance at the multi-bench and overall slope scales satisfy the nominated DAC for the rehabilitated slope profile along all assessed stability sections.
  - Note that addition of access ramps improves the stability performance of slopes as it flattens the overall slope angle.

Extended Zone of Weathering

– The adoption of a flatter slope profile through the upper weathered units acts to mitigate the potential stability implications associated with the extended weathering profiles at the overall slope scales.

– At the multi-bench scale, there is a decrease in the stability performance compared to the results of the 'expected' weathering profile, however, remains above design performance objectives.

The results of the extended zone of weathering sensitivity analyses highlight the robustness of the design geometry against varied geological conditions.

Seismic Loading

– Under extreme 'worst case' seismic loading events the stability performance was calculated to satisfy the design performance objectives.

The results of the seismic loading sensitivity analyses highlight the robustness of the design geometry against increased loading conditions.

Elevated Phreatic Conditions

– At the overall slope scale the stability performance was calculated to be above unity (FoS > 1.00) for all stability sections under fully saturated conditions, which indicates a robustness of slope design geometry against varied conditions.
  - The calculated stability performance of Section S1 at the multi-bench scale is noted to be below unity (i.e., FoS < 1.0). It is however noted that the calculated slip surfaces at the multi-bench scale are typically thin skinned and within the placed rock slope material.

The results of the elevated phreatic condition sensitivity analyses highlight the importance of implementing and maintaining adequate surface and groundwater management protocols which are integrated into the final landform design.

Influence of Final Pit Lake Level

The results of the sensitivity analyses undertaken to assess the influence of the final pit lake on the stability performance of rehabilitated slopes indicate that:

– The stability performance of rehabilitated slopes along Sections N1, S1 and W1 were calculated to be slightly lower when lower final pit lakes levels were adopted compared to final lake level of RL 217 m, however, remain within tolerable thresholds of the design performance objectives at both the multi-bench and overall slope scales.

The results of the final pit lake sensitivity analyses highlight the robustness of the design geometry against varying natural conditions.

Influence of Degraded Material Strengths

The results of the sensitivity analyses undertaken to assess the influence of degraded material strengths indicate that:

- At the overall slope scale the stability performance along Section N1, S1 and W1 were calculated to be lower than under ‘expected’ conditions, though remain within tolerable thresholds of the nominated DAC.
- At the multi-bench scale, instability is more likely to occur along stability Section N1 and S1 compared to ‘expected’ case, noting that the stability performance along both Section N1, S1 and W1 were calculated to be within tolerable thresholds of the nominated DAC.

It is expected that it would take very long time periods for the slope materials to become degraded as a result of weathering 100’s to 1000’s of years. There may a propensity for degradation to occur about the natural lake fill level where fluctuations in the lake occurs as a result of seasonality effects due to fluid penetration and oxidation. It should be noted that there is currently no guidance on assessing stability performance of open cut excavations considering degraded material strengths, though ANCOLD (2019) recommend that where material strengths have become degraded as a result of cyclic loading to adopt a lower DAC. Taking this into account a lower FoS may be acceptable for long-term conditions where materials are likely to become degraded.

7. Erosion Assessment

The Revised Universal Soil Loss Equation (RUSLE) equation is a tool used to estimate the potential soil loss due to direct rainfall on an exposed slope and can provide an indication of the general erosion risk of the surface. It is also useful for quantifying the impact of various factors that contribute to erosion when designing batters, under long term (rehabilitated) conditions. In addition to the above, the area of disturbed land as a result of quarrying activities has been considered.

It is important to note that RUSLE only accounts for soil loss due to direct rainfall on the slope, not concentrated flow from any catchments flowing onto the slope.

7.1.1 Scenarios analysed

The erosion potential of terminal and rehabilitated slopes at the Woori Yallock site have been considered under two scenarios.

– **Erosion Scenario 1 -** Erosion of susceptible materials along exposed batters and the weathered rock stockpile under dry void conditions
– **Erosion Scenario 2 -** Erosion of susceptible materials along exposed batters and the weathered rock stockpile considering the establishment of the final pit lake.

With respect to erosion along the excavated slopes, it is assumed that only the ‘diggable’ units (Unit 1 and 2) are susceptible to erosion and therefore are the only stratigraphic units considered in this assessment. Based on the stratigraphic and geological model, the exposure of Unit 1 (Regolith) is likely to be very low with observed thicknesses up to 2 m, and the exposure of Unit 2 (EW Hornfels) is typically in the order of 5 m, except for in certain areas where an extended zone of weathering was identified, where unit thicknesses may increase up to 10 m. The proposed rock slope along the eastern wall comprised mostly of Unit 2 (EW Hornfels) material.

7.2 Nominated Erosion Potential Criteria

Two conventionally applied erodibility potential criterions have been adopted for this erosion assessment. These design acceptance criterions, as suggested by the Commonwealth of Australia (2016) and by Morse and Rosewell (1996) and Landcom (2004) were adopted to assess the potential volume of soil loss at a site against tolerance levels. In the following subsections, each criterion is discussed in further detail.

Criterion 1

Based on the Erosion Hazard Guidelines (after Morse and Rosewell (1996) and Landcom (2004)), which are summarised in Table 38.

Table 38 Soil Loss Classes after Morse and Rosewell (1996) and Landcom (2004

Soil loss class	Calculated soil loss (t/ha/yr)	Erosion hazard
1	0 to 150	Very Low
2	151 to 225	Low
3	226 to 350	Low-moderate
4	351 to 550	Moderate
5	501 to 750	High
6	751 to 1500	Very High
7	> 1500	Extremely High

Criterion 2

Criterion 2 is based on the tolerable soil loss tolerances which are cited in 'Mine Rehabilitation, Leading Practice Sustainable Development Program for the Mining Industry' (Commonwealth of Australia, 2016). This design acceptance criteria indicates that the soil loss should not exceed 4.5 tonnes per hectare per year (i.e., 4.5 t/ha/yr).

7.3 Potential Erodibility of Terminal and Rehabilitated Slopes

The RUSLE equation calculates an annual erosion rate based on the multiplication of five factors, and is expressed as:

$$\boldsymbol{A = R \cdot K \cdot LS \cdot C \cdot P}$$

Where:

A = Estimated average soil loss in tonnes per acre per year

R = Rainfall erosivity factor

K = Soil erodibility factor

LS = Topographic factor that accounts for slope length and slope gradient

C = Erosion practice control

P = Ground cover factor

The above RUSLE factors are outlined in further detail in the following subsections.

7.3.1 Rainfall erosivity factor 'R'

This factor is determined by the intensity of rainfall in the area and is therefore not a design parameter. Using the aforementioned principal empirical relationships have been established to correlate mean annual precipitation with the Rainfall Erosivity Factor (R). Yu and Roswell (1996) established a relationship to estimate the R-factor based on studies conducted in south-eastern Australia. The relationship had a very good correlation with R^2 = 0.91. The R-Factor and mean annual relationship is expressed as:

$R = 0.0438 \cdot P^{1.61}$

Where:

P = Mean annual precipitation (mm)

Mean annual precipitation for the Woori Yallock were obtained from the Bureau of Meteorology (2022) for the nearby station located approximately 15 km away at the Coranderrk Badger Weir (station number 086219). The mean annual precipitation for the area is 1099 mm, calculated using approximately 136 years of rainfall data. The calculated R-factor is equal to 3447.3

7.3.2 Soil erodibility factor (K)

The soil erodibility factor (K) accounts for the erodibility of the soil based on its composition (e.g., sandy clay). Nomograph equations (and visual representations) are frequently relied upon for deriving suitable K-factors, which is a simple method that makes use of basic soil properties (e.g., particle size distributions). According to the CSIRO publication after Yang et al. (2017) most of the models used to determine suitable K-factors (e.g., Wishmeier *et al*., 1971) have been developed for American soils and may not be representative of Australian Soils. According to Yang et al. (2017) the nomograph developed by Rosewell (1993) referred to as 'K_SOILOSS' yielded comparative results to field measurements and is a preferred method for deriving a suitable K-factor for Australian soils, which contain less than 68% silt content).

The K_SOILOSS nomograph equation is expressed as:

$\mathrm{K_{SOILOSS}} = \left(2.77 \cdot \ \mathrm{M}^{1.14} \cdot 10^{-7} \cdot (12 - \mathrm{OM})\right) + \left(4.28 \cdot 10^{-3} \cdot (\mathrm{SS} - 2)\right) + ((3.29 \cdot 10^{-3} \cdot (\mathrm{PP} - 3))$

Where:

M = Particle Size Parameter = (%Silt + %Very Fine Sand) x (100 – %Clay)

OM = Organic Matter (%)

SS = Soil Structure (ranging from; 1-very fine granular; 2-fine granular; 3-medium to coarse grained; and 4-blocky, platy or massive.

PP = Soil Permeability (ranging from; 1-rapid; 2- moderate to rapid; 3-moderate; 4-slow to moderate; 5-slow; and 6-very slow).

Available soil data required for the input into the above nomograph equation was obtained from the publicly available Soil and Landscape Grid of Australia (SGLA, 2017) database. This data access platform enables the user to query soil data based on the site location with a 95% confidence interval and provides the necessary information to estimate a K-factor. Summarised in below in Table 39 are the adopted overburden soil index parameters obtained from SGLA (2017) utilised to calculate the particle size parameter 'M'.

Table 39 Summary of Soil Properties (after SGLA, 2017)

%Sand (0.05-0.1 mm)	%Silt (0.002-0.05 mm)	%Clay (< 0.002 mm)	M – Particle size Parameter
40	15	45	825

Summarised in Table 40 are the parameters obtained from SGLA (2017) used to calculate the K-Factor for the Woori Yallock Quarry site.

Table 40 Summary of K-Factor Parameters after Rosewell (1933)

M	% Organic matter (OM)	Soil structure (SS)	Permeability (PP)	K-factor (Rosewell, 1993)
825	0.5	2 Fine Granular	4 Slow to Moderate	0.010

Based on the above, a K-factor of 0.01 has been adopted for this assessment.

7.3.3 Topographic Factor (LS)

The topographic factor (LS) accounts for a slopes height (L) and gradient (S) and is used to represent the effect of topography on erosion rates. The equations for calculating the LS in RUSLE are:

$$\mathrm{LS} = \mathrm{L} \cdot \mathrm{S}$$

$$\mathrm{L} = \left(\frac{\lambda}{22.13}\right)^{\mathrm{m}}$$

$$\mathrm{m} = \frac{\beta}{(1+\beta)}$$

$$\beta = \frac{\sin(\theta)}{[3 \cdot \sin(\theta)^{0.8} + 0.56]}$$

$$\mathrm{S} = 16.8 \cdot \sin(\theta) - 0.5\,; \quad \theta \geq 9\%$$

Where:

λ= Slope length (m)

m = Variable length-slope component

β = Variable slope gradient component

θ = Slope angle

The calculated topographic factors for the excavated slopes and overburden emplacement areas are summarised in Table 41. For this assessment, individual rock slope of up to 15 m high were considered.

Table 41 Summary of Topographic Factor Parameters

Geometry	Slope angle (°) - θ	Maximum slope height (m) - L	Topographic factor - LS
Excavated slope (Units 1 and 2)	~38	10	9.16
Rock slope	~27° (1V:2H)	15	7.54

7.3.4 Erosion control factor (C)

The erosion control (C) factor is used to measure the effect of vegetation and management practices on erosion rates. This includes the effects of vegetation, soil cover, soil biomass and soil disturbing activities. Typical cover factors are presented in Table 42.

Table 42 Summary of Typical C-factors

Treatment	Time after application (months)	Assumed grass coverage (%)	C-factor after Landcom (2004)	C-factor after Sprague, (1999)
Untreated	Undefined	0	1.0	1.0
Topsoiled and vegetated	0	0	1.0	0.7
	1 – 3	15	0.55	0.1
	3 – 6	30	0.32	
	6 - 12	45	0.18	0.05
	12 – 18	60	0.1	0.01
	18 – 24	75	0.05	0.01
	> 24	80	0.03	0.01

It is anticipated that rehabilitation of excavated batters and the rock slope will include the application of topsoil and perennial grasses. Accordingly, a temporal overlay to soil erosion has been added, which reflects a reduction in the erosion potential commensurate with increased grass coverage. It should be noted that the correlated C-factors present a conservative approach to reducing soil erosion over time, i.e., C-factor reductions may be quicker than those tabulated. It is also assumed that the ongoing and active maintenance is employed until grass covers reach the desired level and have become 'fully' established (i.e., can maintain grass coverage without active maintenance).

7.3.5 Ground Cover Factor (P)

The erosion control practice factor (P) measures the effect of practices that reduce flow velocity and tendency for water to flow directly downhill (e.g., track-walking or punching straw into the ground). Table 43 presents a summary of typical erosion control practices and the respective P-factor.

Table 43 Ground Cover Factor Scenarios

Surface condition	**Erosion control practice factor, P**
Compacted and Smooth	1.3
Track-walked along contour	1.2
Track-walked up and down the slope	0.9
Punched Straw	0.9
Sacrificial Layer (Loose to 0.3 m in depth)	0.8

Typical 'C' and 'P' factors presented in Table 42 and Table 43 have been adopted from various sources, including Meyer and Ports (1976), Israelson *et al*. (1980), Goldman *et al.* (1980), URS Greiner Woodward Clyde (1999), the North American Green website (2020) and Sprague (1999).

7.4 Results and discussion

The results of the erodibility potential analyses are based on the assumptions outlined above for the respective input factors. The results of the erosion potential analyses are summarised in Table 44 and presented graphically in Figure 103.

Table 44 Summary of Erosion Potential Analyses Results

	Scenario	Calculated erosion loss at 60 months after commence treatment (t/ha/yr)	Erosion criteria	
			Criterion 1	Criterion 2
Excavated slopes	Scenario 1 + 2	2.5	Very Low (0 to 150 t/ha/yr)	Satisfies the Commonwealth (2016) Guidelines
Rock slope	Scenario 1 + 2	2.1		

The results of the analyses indicate that:

– The erosion potential of the excavated batters and rock slope are considered to 'Very Low' (< 150 t/ha/yr) and satisfy the Commonwealth (2016) guidelines after ~3.3 years of perennial grass coverage.
  - The susceptibility to erosion within these units is largely controlled by the topographic factor and ability to apply covers. Erosion in these areas is expected to be ongoing which is likely to result in some minor sloughing of the excavated face and rock slope. It is anticipated that in the long-term, erosion of these faces would not pose a significant geotechnical risk.

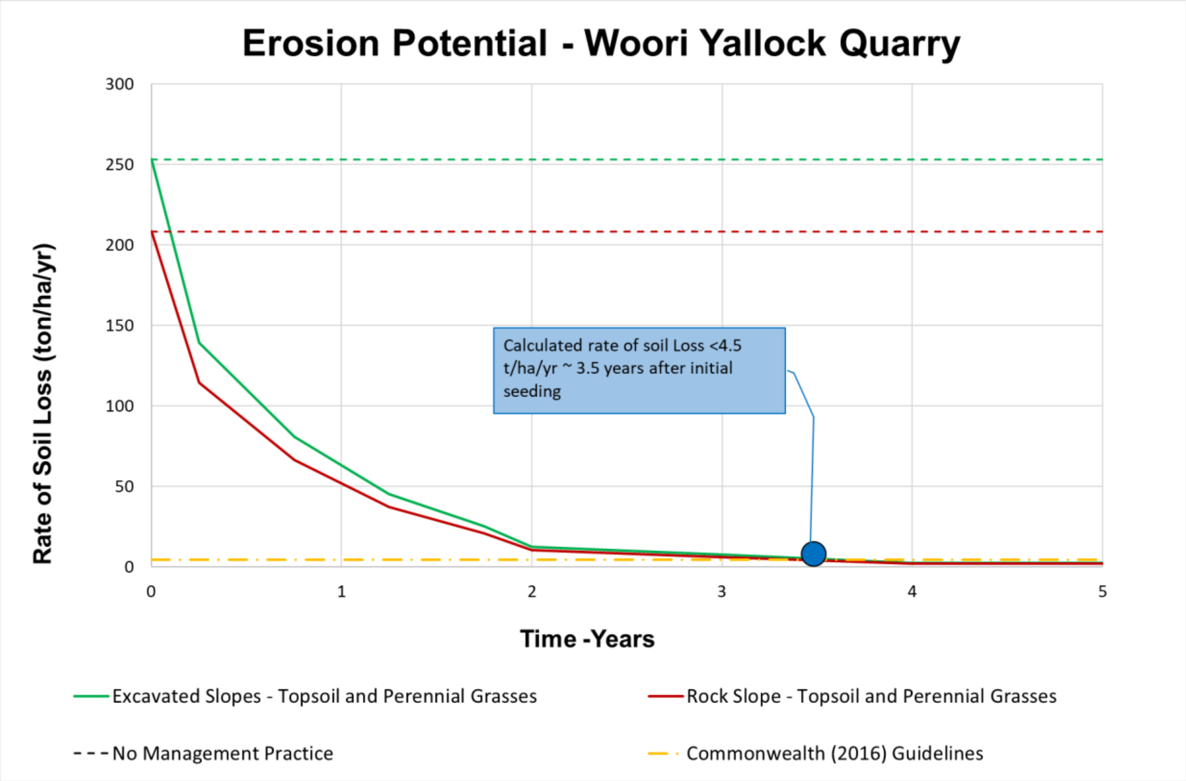


Figure 103 Calculated Rate of Soil Loss – Woori Yallock Quarry

In addition to the above, prompt progressive rehabilitation with ongoing monitoring, maintenance and remediation should be undertaken in-line with the site's Rehabilitation Plan to ensure that a minimum of 80% grass coverage is achieved (NB: this is a design assumption adopted above). Selection of suitable grasses for the site (in line with the final landform use) must be considered along with its applicability to the site-specific soil type(s). Furthermore, it is recommended that work is undertaken to verify the suitability of the erosion input parameters presented within this report. Based on the above, the monitoring criteria outlined in Table 45 is recommended.

Table 45 Proposed Erosion Monitoring Criteria

Rehabilitation / Closure Criteria	Elements to be Monitored	Frequency
Areas of Completed Progressive Rehabilitation Grasses initially established, as soon as practicable, on batters in diggable/erodible materials in progressively rehabilitated areas. <u>In the first 3 years after rehab. batter earthworks:</u> No erosion channels greater than 200 mm deep and/or wide on any progressive rehabilitation. No more than 5 erosion channels greater than 150 mm deep and/or wide within a 20 m wide area on any progressive rehabilitation. <u>By the end of the third year after earthworks:</u> No erosion channels greater than 50 mm deep and/or wide on any progressive rehabilitation. No more than 5 erosion channels greater than 20 mm deep and/or wide within a 20 m wide area on any progressive rehabilitation. Any necessary remedial rehabilitation undertaken as soon as practicable. **Completed Rehabilitation of all Upper Batters / Closure**: No erosion channels greater than 50 mm deep and/or wide on any rehabilitated upper terminal batters. No more than 5 erosion channels greater than 20 mm deep and/or wide within a 20 m wide area on any rehabilitated upper terminal batters.	Visual inspection for erosion channels, recording depth, width and number of any channels and photographed for follow up.	**Ongoing Progressive Rehabilitation** Y1 - 2 Monthly, following completion of earthworks in diggable/erodible materials. Y2 - 3 Monthly until initial grasses established and other measures have limited erosion. Y3 - 6 Monthly, until long-term vegetation established and erosion minimised. Additional inspections after significant rainfall events. Annually review for any need to remediate areas of erosion. **Final Rehabilitation Works for Upper Batters** Y1 - 2 Monthly once rehabilitation works are completed on all upper terminal batters. Y2 - 3 Monthly until initial grasses established and other measures have limited erosion on all upper terminal batters. Y3 - 6 Monthly (and further, if required) until long-term vegetation established and erosion minimised on all upper terminal batters. Additional inspections after significant rainfall events. Annually review for any need to remediate areas of erosion until closure.

8. Geotechnical Risk Assessment

8.1 General

The geotechnical risk assessment is a quantitative assessment based on the 'likelihood' and 'consequence' of a major geotechnical hazard occurring.

8.2 Geotechnical Hazards

Table 46 details the findings of this geotechnical assessment and identified geotechnical hazards relating to the proposed bench design profile at the Woori Yallock Quarry:

Table 46 Geotechnical Hazards at the Woori Yallock Quarry

Mechanism	Description
Hazard 1 Localised rational / composite instabilities along excavated batters within the Regolith and EW Hornfels Units and rock slope	Potential for small scale rotational instability of the final excavated batters and overburden dumps. Potential causes for rotational instability include: – Saturation of the in-situ materials due to intense prolong rainfall and / or improper surface water management. – Generally, these instabilities will occur at the single bench scale, where the extent of instability is governed by the slope geometry. Where continuous slopes are formed without benching, there is an increased likelihood for the scale of this hazard to increase.
Hazard 2 Localised structural instabilities.	Small scale structurally controlled wedge instabilities that are governed by the characteristics of the structural defects. – Generally, these instabilities will occur at the single bench scale. – Inappropriate pit geometry, such as over steepening, can also increase the potential for this hazard to occur. – Water ingress into natural defects from intense and/or prolonged rainfall events or improper surface water management increase the likelihood of occurrence for this hazard. For this hazard to be realised, structures such as jointing must be present within the rock mass and interact with the pit geometry in an unfavourable way.
Hazard 3 Large scale structural instability	Larger slope volume movements that are governed by the discontinuity shear strength characteristics. Whilst it is expected that structural instabilities are contained to a single working bench, there is potential for structural instability to occur across multiple benches as a result of highly persistent defect planes. The consequences of this type of hazard can include partial or full loss of pit crests including pit ramps and haul roads.
Hazard 4 Large scale rock/soil mass instabilities along excavated batters and rock slope	Larger scale slope volume movements that are governed by the shear strength characteristics of in-situ/placed materials. – Slope instability occurs when the driving forces are greater than the resisting forces. – Movement of this hazard occurs in a rotational manner and is dependent upon the slope geometry, material strength and piezometric pressures. – Consequences of this type of hazard can include partial or full loss of pit crests and impacting working benches. In extreme cases the failure zone may migrate some distance from the pit crest which may exceed the work authority boundary.

8.3 Risk Assessment Process

Risk analysis involves the consideration of the source risks, their consequences and the likelihood of those consequences occurring. Risks are typically analysed by combining the likelihood and consequence to determine a category or level for each risk event.

A semi quantitative risk assessment process has been utilised in the risk assessment matrix below (see Figure 104 and Table 48) as suggested by Earth Resources Regulation (ERR).

Table 47 ERR Likelihood Descriptions (DJPR, 2020b)

Likelihood	Description	Probability of event occurring
Almost certain	The risk event is expected to occur in most circumstances	>90%
Likely	The risk event is expected to occur in some common circumstances	70 – 90%
Possible	The risk event might occur in some circumstances	30 – 70%
Unlikely	The risk event could occur in some uncommon circumstances, as this is known to occur at comparable sites	5 – 30%
Rare	Highly unlikely, but the risk event may occur in exceptional circumstances, as may have occurred at comparable sites	<5%

Likelihood		Consequence				
		Insignificant	Minor	Moderate	Major	Critical
	Almost Certain	Medium	High	Very High	Very High	Very High
	Likely	Medium	Medium	High	Very High	Very High
	Possible	Low	Medium	Medium	High	Very High
	Unlikely	Low	Low	Medium	High	High
	Rare	Low	Low	Medium	Medium	High

Figure 104 Risk Assessment Matrix (DJPR, 2020b)

Table 48 Risk Rating Acceptability (DJPR, 2020b)

Risk level	Description
Very High	Totally unacceptable level of risk. Control measures must be put in place to reduce the risk to lower levels.
High	Generally unacceptable level of risk. Control measures must be put in place to reduce the risk to lower levels or seek specific guidance from ERR.
Medium	May be acceptable provided the risk has been minimised as far as reasonably practicable.
Low	Acceptable level of risk provided the risk cannot be eliminated.
Eliminated	The risk is eliminated.

8.4 Risk Assessment Matrix

For the purpose of this geotechnical risk assessment, two categories of risk have been considered:

– Operational / Occupational Health and Safety (OH&S) risks, and
– Risks to external / sensitive receptors (e.g., environment, any member of the public or land, property or infrastructure in the vicinity of the quarry which may be put at risk by the hazard associated with quarrying or rehabilitation activities.

The geotechnical risk assessment for the Woori Yallock Quarry considering these two types of risks are summarised in Table 49 and Table 50 respectively. The risk assessment presented in Table 50 primarily focuses on the with external receptors within a 1 km radius which were previously identified in Section 2.1, noting the 'non-credible' potential for large scale instabilities to extend beyond 1 km of the pit crest (FoS ~2.0 is calculated within ~150 m of the pit crest). .

Based on the risk assessment presented below, the residual risk to external receptors has been assessed to be "Low".

Table 49 OH&S Risk Rating – Proposed Woori Yallock Quarry

Element at risk	Quarry Boundary	Hazard type	Likelihood	Conseq. Category[1]	Risk Rating	Corrective / management Action(s)	Likelihood	Conseq. Category[2]	Residual Risk Rating	Comments
Personnel safety, site assets and infrastructure including haul roads	All domains	**Hazard Type 1** Localised rational / composite instabilities along excavated batters within the Regolith and EW Hornfels Units and rock slope.	Possible	Moderate	Medium	– Implementation and maintenance of surface and groundwater management protocols. – Profiling of slopes within weathered units and rock slope to create stable landforms. Where practicable, vegetate slopes, undertake regular inspections and monitoring.	Unlikely	Insignificant	Low	– Negligible risk should water ingress within the in-situ materials be sufficiently managed in line with the site-specific surface and ground water management plan. – Vegetating slopes reduces the erosion potential and thus the potential for undercutting which may promote instability. – Regular visual observations/monitoring should be undertaken. – Flattening of batter faces results in a more stable landform that is less susceptible to adverse stress states and erosion.
		Hazard Type 2 Localised structural instabilities.	Possible	Moderate	Medium	– Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Unlikely	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting to facilitate adequate retainment of potential structural instability. – Ongoing monitoring for potential loose rocks, debris and crest loose should be undertaken through visual inspections. Adjustments to drill and blast designs should be made where significant crest loss is measured or where high disturbance of the rock face is observed. – Surface and groundwater should be managed in line with site-specific surface and groundwater management plan to minimise uncontrolled water ingress into defect structures.
		Hazard Type 3 Large scale structural instability.	Rare	Moderate	Medium	– Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Rare	Insignificant	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – Regular visual inspections should be undertaken on slopes post blasting and clean-up to identify any major changes in geology or highly persistent defect structures. Where notable features are identified, it is recommended to review the geotechnical modelling and where conditions differ from this report, additional assessments should be undertaken.
		Hazard Type 4 Large scale rock/soil mass instabilities along excavated batters and rock slope.	Rare	Major	Medium	– Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring. – Profiling of slopes within rock slope to create stable landforms.	Rare	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – Vegetating the rock slope and flattening of batter faces results in a more stable landform that is less susceptible to adverse stress states and erosion. – The calculated FoS was calculated to satisfy the design terminal and rehabilitated design performance objectives, which were tested under varying loading scenarios.

[1] Determined on the basis of the critical credible or reasonable outcome, which takes into consideration the temporal exposure of at-risk elements.

Table 50 Risk Assessment for Proposed Woori Yallock Pit – Impact to Sensitivity Receptors during Quarrying and Rehabilitation

Element at risk	Quarry Boundary	Hazard type	Likelihood	Consequ. Category[3]	Risk Rating	Corrective / management Action(s)	Likelihood	Consequ. Category[4]	Residual Risk Rating	Comments
Nearby Residential Property (west of McMahons Rd)	West domain	**Hazard Type 4** Large scale rock/soil mass instabilities along excavated batters.	Rare	Major	Medium	Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Rare	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – The calculated FoS was calculated to satisfy the design terminal and rehabilitated design performance objectives, which were tested under varying loading scenarios. – Overall scale Factors of Safety along the West and South domains are in excess of the nominated design criteria, with slip surfaces corresponding to the global minimums noted to be within the WA boundary along these domains.
Mount Toolebewong State Forest	North domain	**Hazard Type 4** Large scale rock/soil mass instabilities along excavated batters.	Rare	Moderate	Medium	Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Rare	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – The calculated FoS was calculated to satisfy the design terminal and rehabilitated design performance objectives, which were tested under varying loading scenarios.
Yarra Ranges National Park	East domain	**Hazard Type 4** Large scale rock/soil mass instabilities along excavated batters.	Rare	Moderate	Medium	Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Rare	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – The calculated FoS was calculated to satisfy the design terminal and rehabilitated design performance objectives, which were tested under varying loading scenarios.
Local Access Track (Moora Rd)	West domain	**Hazard Type 4** Large scale rock/soil mass instabilities along excavated batters.	Rare	Moderate	Medium	Good final wall scaling techniques, bench clean-up, controlled blasting, regular visual inspections, and monitoring.	Rare	Minor	Low	– Production and final wall blasting should be undertaken in line with the site-specific drill and blast management plan. – Final wall scaling techniques should be employed to ensure loose rocks are removed from the face, with hard scaling back to competent intact rock. Benches should be sufficiently cleared after blasting. – The calculated FoS was calculated to satisfy the design terminal and rehabilitated design performance objectives, which were tested under varying loading scenarios. – Overall scale Factors of Safety along the West and South domains are in excess of the nominated design criteria, with slip surfaces corresponding to the global minimums noted to be within the WA boundary along these domains.
Natural Reserves, Regional Parks and Forests	All domains (area between extraction boundary and WA boundary)	**Hazard Type 1** Localised rational / composite instabilities along excavated batters within the Regolith and EW Hornfels Units and rock slope.	Unlikely	Minor	Low	– Implementation and maintenance of surface and groundwater management protocols. – Profiling of slopes within weathered units and rock slope to create stable landforms. Where practicable, vegetate slopes, undertake regular inspections and monitoring.	Rare	Minor	Low	– Negligible risk should water ingress within the in-situ materials be sufficiently managed in line with the site-specific surface and ground water management plan. – Vegetating slopes reduces the erosion potential and thus the potential for undercutting which may promote regressive and localised instability of the regolith. – Regular visual observations/monitoring should be undertaken. – Flattening of batter faces results in a more stable landform that is less susceptible to adverse stress states and erosion.

[3] Determined on the basis of the critical credible or reasonable outcome, which takes into consideration the temporal exposure of at-risk elements.

9. Post-Closure Considerations

In line with the ERR '*Geotechnical Guideline for Terminal and Rehabilitated Slopes: Extractives Industry Projects*' (Department of Jobs, Precincts and Regions, September 2020) and *Preparation of Rehabilitation Plans: Guideline for Extractive Industry* (March 2021), Work Authority holders are required to develop rehabilitation plans that achieve safe, stable and sustainable rehabilitation outcomes.

The slope stability outcomes of the proposed rehabilitation geometry indicate that large scale slope instability is unlikely to manifest at the site in the long term. Stability analysis of the terminal slopes calculated Factors of Safety in excess of 1.6, with the stability performance of rehabilitated slope calculated to have Factors of Safety in excess of 2.0 with the increasing final pit lake. It is noted that whilst the pit lake reaches an equilibrium level (~10 to 15 years), the stability performance of the pit slopes is anticipated to remain in excess of 1.6 during this transition (rehabilitation) period. The site-specific rehabilitation activities and requirements, including rehabilitation milestones, to achieve a safe, stable and sustainable final landform are documented in the site rehabilitation plan.

Due to the inherent uncertainty in predicting future receptors and their likelihood of being impacted, a post-closure risk rating has not been assigned, however Table 51 summarises the geotechnical considerations of the post-closure design based on the above assessment.

Table 51 Post-Closure Geotechnical Considerations

Hazard Type	Geotechnical Considerations	Long-term Stability Outcomes
Large scale slope instability of rehabilitated quarry slopes	– Geotechnical assessment of rehabilitated slopes undertaken to ensure they meet design for long term stability. • No dangerous features such as high precipices and steep slopes remain without appropriate batter design to protect public safety. – Site meets requirements of ongoing access and use compatible with land use. – Methods of construction and reshaping as well as implementation of rehabilitation is verified by a competent person to ensure construction and rehabilitation is aligned with design life and post-quarrying purpose/land use. – Landform area constructed to design and where variations made, that stability meets objectives (specific erosional stability criteria such as depth and density of rills / gullies in critical areas and the role of vegetation). – Performance monitored, interpreted and reported for at least first 3 years after completion of final rehabilitation works. – Maintenance of final pit walls for at least 3 years if required.	– Outcomes of the stability analyses indicate global stability performance of terminal slopes with FoS > 1.6 and rehabilitated slopes with FoS > 2.0. The stability performance is expected to improve with the increasing final pit lake. – The pit design is not expected to present any difficulties in rehabilitating the slopes as per the site's rehabilitation plan.
Ongoing erosion of rehabilitated upper batters	– Methods of construction and reshaping as well as implementation of rehabilitation is verified by a competent person to ensure construction and rehabilitation is aligned with design life and post-quarrying purpose/land use. – Landform area constructed to design and where variations made, that stability meets objectives (specific erosional stability criteria such as depth and density of rills / gullies in critical areas and the role of vegetation). – Performance monitored, interpreted and reported for at least first 3 years after completion of final rehabilitation works. – Maintenance of final pit walls for at least 3 years if required.	– Erosion studies have been undertaken as part of the geotechnical assessment in support of approval, and erosion reassessed once rehabilitation of all upper batters is complete.
Accessibility of high faces and steep slopes	– Landform area constructed to design and where variations made, that stability meets objectives (specific erosional stability criteria such as depth and density of rills / gullies in critical areas and the role of vegetation). – Performance monitored, interpreted and reported for at least first 3 years after completion of final rehabilitation works. – Maintenance of final pit walls for at least 3 years if required.	– Where the upper batters do erode, there is sufficient capacity in the quarry pit to retain any rockfalls (see Section 6.3.4). – Access ramps for 'egress' at all locations will be incorporated as part of final landform.

10. Recommendations

Based on the information available, and the analyses presented within this report, the following recommendations are made:

– Develop a Ground Control Management Plan (GCMP) which incorporates the following:
  - Surface and groundwater management incorporating the works currently being undertaken by Water Technology Pty Ltd.
  - Drill and blast management, and suitable ground control practices including final wall scaling and bench clean up with regular observations of quarry excavations.
  - Geological mapping campaigns during operations, particularly where drill and blast activities interact with the terminal slopes, and in areas where the pervasive fabric is orientated in the same or similar direction as excavated slopes.
  - Outline the roles and responsibilities of key site personnel to undertake inspections and assessments of quarrying activities.
  - It is noted that a GCMP is currently being prepared by GHD (2023).

Based on the outcomes of the stability analyses it is recommended that the slope design geometry outlined in Section 6 is adopted for terminal and rehabilitated slopes. The key design parameters for the recommended geometry are summarised in Table 52.

Table 52 Summary of Recommended Design Geometry

Slope Design Parameter	Excavated Quarry Batters				Rock Slope
	Regolith / EW Hornfels	HW and MW to SW Hornfels	Fr Hornfels (above pit lake level)	Fr Hornfels (below pit lake level)*	
Bench height (m)	15	5		10	15
Bench width (m)	N/A	7	6	8	3
Bench face angle (°)	38	85	85	80	1V:2H
Geotechnical Decoupling Berm	Yes – Decoupling berm equal to 10 m at the EW to HW Hornfels Horizon				N/A
Beaching Zone	Yes – Beaching Zone equal to 12 m at pit lake shoreline				N/A
Final Pit Lake Level	RL 217 m AHD				
Maximum Overall Slope Height (m)	260				
*Fresh Hornfels unit double benched below the pit lake level					

11. References

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