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17 June 2025

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Dear Maxime,

Calliope Progressive Rehabilitation and Closure Plan (PRCP) - Pits and WRD: Geotechnical

The attached report addresses geotechnical issues relevant to the PRCP for the Calliope Mine. I trust it provides the information you require. Please contact this office if you wish to discuss any aspect of it or require further assistance.

For and on behalf of
ROCKTEST

Tony Meyers

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Principal Rock Mechanics Engineer



**CALLIOPE MINE PRCP
PITS AND WRD:
GEOTECHNICAL
V1.5**

for



GRAYMONT

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Approval

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ACRONYMS

AQP	Appropriately Qualified Person (e.g., qualified geotechnical consultant)
BGL	Below Ground Level
FoS	Factor of Safety
EOML	End of Mine Life
GCMP	Ground Control Management Plan
GLE	General Limit Equilibrium method of slope stability analysis
PGA	Peak Ground Acceleration
PoF and P_f	Probability of Failure
PRCP	Progressive Rehabilitation and Closure Plan
RL	Relative Level
SWL	Standing Water Level
WRD	Waste Rock Dump

1 INTRODUCTION

1.1 Background

This report considers geotechnical issues¹ applicable to the Progressive Rehabilitation and Closure Plan (PRCP) for the final landforms at the Calliope Mine. The landforms considered are two open-cut pits and a waste rock dump (WRD). The requirements of the PRCP, as outlined by the Guideline prepared by the QLD Department of Environment and Science², are that this report provides details of how the landforms will achieve post-closure stabilities.

The project was undertaken by Rocktest at the request³ of Graymont Australia Pty Ltd (Graymont).

1.2 Requirements in the PRCP

Consistent with the requirements of the PRCP in regard to the pits, this document discusses the:

- locations and specifications i.e., depths, lengths, widths, overall slope angles, bench specifications,
- methods of construction, with 3D design plans,
- soil/rock characteristics,
- geotechnical conditions (i.e., soil/rock characteristics),
- geotechnical stabilities of the walls, considering the effects of long-term erosion and weathering and the effects of significant hydrological events,
- rehabilitation strategies.

Consistent with the requirements of the PRCP in regard to the WRD, this document discusses the:

- location and specification i.e., height, length, width, overall slope angles, bench specifications,
- method of construction with 3D design plans,
- soil/rock characteristics,
- geotechnical stabilities of the slopes,
- settling and subsidence over time,
- rehabilitation strategy.

1.3 Terminology

Time periods referred to in this report are:

- Short-term < Five years
- Medium-term Five years to EOML
- Long-term > 100 years

¹ This report considers geotechnical issues only. Surface and subsurface hydrology issues and environmental issues are considered only where relevant.

² Guideline – Progressive rehabilitation and closure plans (PRC plans) ESR/2019/4964, Version 2.00, Last reviewed: 17 March 2021.

³ Phone call to T. Meyers of Rocktest from E. Ashong of Graymont, 28 February 2022.

2 LOCATION AND GENERAL LANDFORM CHARACTERISTICS

The landforms at the Calliope mine at the EOML will comprise two pits and a waste rock dump⁴ (WRD). Figure 1 shows their final footprints. Pit 2 is the small northernmost pit, which will not be mined between the current date and the EOML. The large pit, Pit 3-4, will be mined prior till then. This pit comprises existing Pits 3 and 4.

The landforms are on a low north-south trending ridge separating Awoonga Dam on the Boyne River to the east from its tributary, Futter Creek, to the southwest, and its tributary Ragotte Creek to the west (Figures 2 and 3). Ragotte Creek is ephemeral. Futter Creek has permanent water. Its maximum depth, and that of the water in the lake, is controlled primarily⁵ by the height (i.e., 40 mRL) of the spillway on the dam wall.

Pit 2 is within a natural contour high that prevents it from being inundated during floods.

The likelihood for Pit 3-4 to flood is reduced by two levees, the Western and Eastern levees (Figures 4 and 5), and natural contour highs to the north and east of the pit. The lowest point on the upper surfaces of the levees is at 52 mRL, which is significantly higher than the spillway.

3 GEOLOGY

3.1 General Characteristics

The Taragoolla limestone deposit occur within the Early Devonian age (419-Ma) Calliope Beds. The Beds occupy a north-northwest striking belt about 40 km long and 3 to 4 km wide. They comprise predominantly volcanic rocks with relatively minor inter-bedded sedimentary rocks, including limestone. The rocks have been subjected to folding and faulting.

Overburden consists of a shallow cover of soil. Boulders and cobbles of limestone and volcanics occur in much of the layer.

Underlying the overburden is predominantly limestone. Discontinuities within the limestone are clay filled to a depth up to approximately 15 mBGL. Karst features are prevalent to approximately this depth. Below this depth, karst and infill becomes less prevalent and the apertures of the discontinuities become tight.

Bedding surfaces within the limestone generally dip very steep towards the west. This characteristic is consistent with the regional structure. The major axes of the pits are parallel to the strikes of the beds.

The volcanics exposed in the walls of the pits include predominantly basic basalt and intermediate olive green andesite (Figure 6).

- The basalt occurs as lava flows. It is exposed primarily in the upper benches of the east and south walls of Pit 4. It is dark grey, has high to very high strength and is slightly weathered.
- The andesite generally occurs as steep dipping dykes that follow vertical to sub-vertical bedding and joints within the limestone. The two dominant systems of the dykes strike north-north-west and east. The widths of the dykes range from stringers to a few metres. Reduced vertical stresses at shallower depths

⁴ Two WRDs are on site at the current date, a northern and a southern dump. The northern dump will be removed during the medium to long term, prior to the EOML. Hence, at the EOML, only the southern dump will be a component of the final landform. As this report considers the landforms at the EOML, the terms WRD and dump refer to the southern dump only.

⁵ The width of the spillway is also relevant under flood conditions.

have allowed the andesite to extrude along shallow dipping joints. The andesite has high to very high strength and is moderately to slightly weathered.

The volcanics comprise less than 5% of the total rockmass below the second bench (i.e., <27 mRL). Combined with their high strengths, and discontinuity characteristics no less favourable to wall stabilities than those within the limestone, the volcanics have insignificant influence on the stabilities of the walls. Their influence is not therefore considered specifically in this report.

3.2 Soil

3.2.1 Characteristics

The thickness of the soil generally ranges between 1 m and 4 m (Figure 7).

- The upper horizon is ≈ 0.2 m thick. It comprises red to red-brown, Terra-Rosa, clay loam, medium plasticity, moist, soft, medium dense, weakly cemented. It contains some to many smooth to angular gravels to boulder sized rocks. The rocks comprise weathered limestone and volcanics from 10 mm to 100 mm.
- The lower horizon comprises light to medium, yellowish brown, silty clay, low plasticity, dry, firm to stiff, medium dense, weakly to moderately cemented. It contains many angular gravels to boulder sized rocks. The rocks comprise weathered limestone and volcanics from 10 mm to 300 mm.

The average shear strength of the two horizons is defined⁶ in the stability analyses in terms of a conservative friction angle of 25° and a cohesion of 50 kPa.

3.2.2 Soil Management

As the depths of in-situ soil is generally shallow, soil stripping and handling prior to the EOML requires:

- retaining the depth of soil where possible,
- minimising the total area from which soil is stripped,
- stockpiling all soil stripped.

The stability of the stockpiled soil will be maintained by:

- maximising the height of the stockpile to below 3 m,
- maintaining the slopes of the stockpile at less than 3H:1V,
- vegetating the slopes,
- installing sediment fences and/or up contour diversion bunds,
- re-using the soil as soon as practical.

3.3 Limestone

3.3.1 Intact Rock Characteristics

The intact limestone is fine grained and recrystallised due to regional metamorphism. It is light to dark grey but can be various colours (e.g., pink, red, pale brown). It can be vitreous or mottled. It is even textured. It can have occasional ferruginous bands. It can be iron stained on discontinuities.

⁶ The shear strength of the soil is estimated, based on the characteristics of the materials.

At depths exceeding ≈ 15 mBGL, the limestone is moderately-weathered to fresh with high to very high strength (ave. 65 MPa) and an average density of 25.8 t/m^3 (refer Appendix 1). At shallower depths, and adjacent to faults and karst structures, the surfaces of individual rocks tend to be highly weathered. However, their cores may be only slightly weathered, hence their strengths can be similar to those of the unweathered⁷ rocks. The strengths progressively decrease as the depths of weathering into the cores increase. The rocks eventually become extremely weathered gravels of medium to high strength, then become residual sands and silts. The shear strength of the sediments is defined in the stability analyses in terms of a conservative friction angle of 34° ⁸.

Most of the weathered⁹, and all of the unweathered, limestone has sufficient strength and durability for it to be suitable for use as erosion control cover. Only the extremely weathered rocks are not suitable for use as cover, although these rocks are non-typical.

3.3.2 Discontinuities

The surfaces of the discontinuities (i.e., joints, bedding) are generally smooth. At depths exceeding ≈ 20 mBGL, they are tight, clean, and unfilled. At shallower depths in the upper benches, they can be dilated due to stress relaxation, karst, and weathering. When dilated, they are generally filled with either lava or hard clay. The clay can influence the stabilities of the upper benches. Maintaining stability over the long-term will require reducing the batter angles of the applicable sections of the benches.

Analysis of data obtained from discontinuity mapping (Figure 8) indicates that most discontinuities can be assigned to one of 6 sets¹⁰ (Table 1). Set 1 contains bedding-parallel cleavage, which is the dominant fabric. Sets 2 to 6 contain joints. Based on the mean orientations of each set, discontinuities within:

- Set 1 dip very steep¹¹ generally towards the west but can be overturned and dip east,
- Set 2 dip very steep generally towards the southeast but can be overturned and dip northwest,
- Set 3 dip very steep generally towards the south but can be overturned and dip north,
- Set 4 dip moderate towards the southwest,
- Set 5 dip shallow to moderate towards the northeast and east,
- Set 6 dip very steep generally towards the southwest but can be overturned and dip northeast,
- Set 0 are random discontinuities having no preferential orientations.

The average of the six Fisher's constant¹² values (i.e., $F_{\text{ave}} = 35$) applicable to Sets 1 to 6 is moderate. This characteristic is generally indicative of a rockmass not having been significantly deformed. However, the high proportion of random discontinuities¹³ (i.e., $R_p = 28\%$) is typically indicative of a rockmass having been

⁷ "Unweathered" rocks that shows little of no change in strength from fresh rocks. The term includes fresh rocks and rocks that are moderately weathered and slightly weathered.

⁸ The friction angle of the sediments is based on the angle of repose of fines and gravels in the WRD.

⁹ "Weathered" rocks have strengths that are significantly reduced from what they were when the rocks were unweathered. The term includes the weathering grades of highly weathered and extremely weathered.

¹⁰ Characteristics based on the orientations of 726 discontinuities within fresh limestone.

¹¹ Dip terms: Very shallow $0-20^\circ$; Shallow $20-40^\circ$; Moderate $40-60^\circ$; Steep $60-75^\circ$; Very steep $75-90^\circ$.

¹² Fisher's constant defines how tightly the orientation data within a group are clustered around the mean orientation of that group. Values near 0 imply parallelism. Values less than 20 imply a highly deviated set of data.

¹³ Random discontinuities are those that have no preferential orientation and hence are not assigned to a set. They are sometimes referred to as the isotropic component.

significantly deformed, which is consistent with the tectonic history of the rockmass. Understanding the tectonic history that resulted in this inconsistency¹⁴ is beyond the scope or requirement of this PRCP.

The mean and maximum trace lengths¹⁵ of the discontinuities within each set are listed in Table 1. Based on these values, the persistences¹⁶ of the discontinuities are generally medium¹⁷ although a small proportion have high persistences. The general characteristic has contributed to the instabilities on all walls in Pit 4 having been of bench scale or smaller (e.g., Figures 9, 10 and 11).

The shear strength characteristics of the discontinuities is described in terms of an average peak friction angle of 32° for unweathered limestone, reducing to 18° over the long-term (Appendix 1.4).

3.3.3 Karst

Karst does not generally occur below 20 mBGL. Karst has not therefore influenced the strengths of the lower benches in any of the pits. However, karst is exposed extensively in the batters of the upper benches of all walls of the pits. Although it can extend to 20 mBGL, it is most prevalent to 10 mBGL. The most obvious karst features are clay filled solution widened discontinuities and caverns (e.g., Figure 12). The widths of these features are generally less than 2 m. Unfilled features are rare.

Prior to the EOML, the batter angles of sections of the upper benches that contain karst will be reduced to <18°. Soil exposed within karst will be covered with broken limestone to reduce the rate the soil erodes.

Table 1. Characteristics of the sets of discontinuities in the limestone.

<i>Set</i>	<i>Dip</i>	<i>Dip Direction</i>	<i>Fisher's constant, F</i>	<i>Weighted # logged (% of total)</i>	<i>Min/Mean/Max spacing (m)</i>	<i>Mean/Max trace lengths (m)</i>
1 (cleavage)	89	269	25	438 (25)	0.01/0.31/10.3	1.8/10
2	82	146	51	243 (14)	0.01/0.43/3.0	1.6/8
3	85	175	48	151 (9)	0.01/0.58/2.1	1.4/10
4	44	247	42	128 (7)	0.01/0.25/1.1	0.8/3
5	14	031	19	107 (6)	0.03/0.90/5.4	4.5/10
6	85	222	28	178 (10)	0.01/0.69/4.3	1.5/10
0		n/a	0	486 (28)	0.01/0.35/3.9	1.2/10
<i>Total</i>				1731 (100)		

¹⁴ The inconsistency between F_{ave} and R_p may be due to insufficient data having been sampled. The data set is however sufficiently large to characterize the orientation and spacing characteristics of the discontinuities.

¹⁵ When the 2D surface of a discontinuity intersects a batter, the line of intersection on the batter is referred to as the trace of the discontinuity.

¹⁶ The persistence is the two-dimensional areal extent of a discontinuity.

¹⁷ Persistence classification: 1-m ≤ Low ≤ 3-m ≤ Medium ≤ 10-m ≤ High ≤ 20 m, Very high >20 m.

3.4 Earthquakes

Geoscience Australia (2018)¹⁸ places Calliope in a zone of low earthquake hazard. The estimated peak ground acceleration (PGA) having a 10% probability of being exceeded in a 50-year period is 0.005g (0.05 m/s²). The probability corresponds to a 1 in 500 annual return period. There is therefore a 10% probability that an acceleration (A_e) of 0.001g (100/500ths of 0.005g) will be exceeded during a 100-year period.

Several studies have reported on PGA values of 0.2g being required to generate falls of individual rocks from pit walls¹⁹ in low stress conditions as apply at this site. On this basis, PGA values of 0.001g are unlikely to generate many rockfalls. On this basis, a 100-year event is unlikely to significantly increase the risk to persons within or upon the crest of a wall when an earthquake of this intensity occurs.

Wall scale instabilities being triggered by earthquakes within any pit are rare. Although they are unlikely to do so at this site, the influence of earthquake loading on the stabilities of the walls is considered in the stability analyses (Section 6.3 and 7.1).

4 PIT CHARACTERISTICS AND RISK MANAGEMENT

4.1 Background

4.1.1 Pit 1

Pit 1 (Figure 13) was mined prior to 1960. The final floor of the pit was at 46 mRL, 6 m above the spillway of the dam. Hence, the floor does not saturate with groundwater.

The pit was fully backfilled to surface in 2024. The backfill material comprises well-graded²⁰ blasted material. It was placed in thin layers (i.e., <30 cm), moisture conditioned then well-compacted with a roller prior to placing the next layer. Due to this procedure, the maximum displacement of the final upper surface would unlikely have exceeded 0.1 m²¹ at the EOML. However, to accelerate consolidation prior to then, the surface will be used as a laydown crushing area. If any consolidation occurs, the surface will be filled back to the level of the natural surface. The rate of consolidation during the PMLU will be insignificantly low and the area will be safe and stable.

Prior to the EOML, the surface will be ripped to encourage natural revegetation.

4.1.2 Pit 2

Pit 2 (Figure 14) was mined from the mid-1960s until the mid-1970s. It is currently used for water storage and will continue to be used for that purpose prior, and subsequent, to the EOML.

4.1.3 Pits 3 and 4

Pit 3 was mined from the mid-1960s until the mid-1970s. Pit 4 begun in 1976 and has been mined since then. The two pits will be combined into a single pit, Pit 3-4, prior to 2030. This pit will be mined until the EOML using a standard drill/blast/dig/scale method. As currently occurs, ore-grade limestone is hauled to the ROM

¹⁸ Geoscience Australia (2018) The 2018 National Seismic Hazard Assessment for Australia. (Editor. Allen, T.) Page 10.

¹⁹ Rockfalls have been reported for PGA values of 0.16g in pits where discontinuities contain significant joint pressures.

²⁰ The sizing of the backfill material ranges from sand to boulder sized rocks up to 30cm.

²¹ Consolidation is estimated using the method of Kermani, M. and Konrad, J.M. (2018) *In-situ short-term and long-term rockfill compressibility as a function of void ratio and strength of parent rock*. Jnl. Geotechnical and Geoenvironmental Engineering, April.

pad north of the pit from where it is transported off site. Low-grade limestone and limestone contaminated with clay or volcanics are hauled to the WRD south of Pit 3.

4.2 Specifications

4.2.1 Pit 2

Figure 15 shows cross sections through the highest section of each wall of Pit 2. Their specifications are listed in Tables 2 and 3. The pit is shallow, the walls being less than 17 m high. Their stabilities are considered in Section 6.3.

4.2.2 Pit 3-4

Figures 2 and 3 show the elevations of Pit 3-4 at the EOML. It will have an approximately rectangular circumference with the major axis trending north-south. Figure 16 shows cross-sections through the highest section of each wall. Their specifications are listed in Tables 2 and 3.

Water seeps from the western and eastern walls of Pits 3 and 4 respectively. These locations, highlighted in Figures 17 and 18, are listed in Table 4. It will do so from the same locations in Pit 3-4. A hydrogeological study²² concluded that the water originates in Awoonga Dam and its tributaries. The flows likely occur within permeable sediments between the sources and the walls and within karst voids. They may also occur along east-west striking joints with solution widened apertures. The rates of seepage are functions of the heights of water in the sources (Table 5). No evidence has been observed to indicate the flows have contributed to hydrostatic pressures sufficiently large to reduce the stabilities of the respective sections of either walls. Nonetheless, the influence of groundwater on the stabilities of the walls is considered in stability analyses in Section 6.3.

Table 2. Specifications of the pits at the EOML.

<i>Dimension</i>	<i>Pit 2</i>	<i>Pit 3-4</i>
Trend of major axis	168°	178°
Length of major axis at crest	418 m	1550 m
Average length of minor axis at crest	200 m	584 m
Maximum heights of walls	34 m	110 m
Heights of benches	none	12 m
Maximum number of benches	1	9
Widths of berms	n/a	10 m
Height of edge bund on berms	n/a	1 m
Batter angles in unweathered rocks	n/a	65°
Batter angles in weathered rocks	n/a	<18°
Inter-ramp angles	n/a	38°
Maximum overall slope angles (north/south/east/west)	34°	32° to 38°
Ramp type / Minimum width	linear / 17 m	linear / 30 m

²² Graymont Mining Services (2021) Calliope: Hydrogeological Investigation of Excess Water in Pit 3 2018-2020. 27 pages.

Table 3. Dip directions of the walls of the pits at the EOML.

<i>Pit</i>	<i>North (South dip)</i>	<i>South (North dip)</i>	<i>East (West dip)</i>	<i>West (East dip)</i>
2	139°	303°	243°	086°
3-4	151°	306° to 355°	240° to 295°	082°

Table 4. Locations of water seeps in Pit 3-4.

<i>Wall</i>	<i>Easting</i>	<i>Northing</i>	<i>RL</i>
West	320785	7331969	32-35
East	321284	7332811	28-40

Table 5. Likelihood of seepage for various RLs of water in Awoonga Dam.

<i>Dam Level (mRL)</i>	<i>West Wall</i>	<i>East Wall</i>
<40	negligible	none
40-42.5	low	none
>42.5	progressively increasing	

4.3 Risk Management Strategies for the Pits at EOML

After the EOML, the key geotechnical related considerations that will be managed will be:

- Pit wall stability, particularly failure of multiple benches or an overall slope, actions which could result in retreat of a crest.
- Erosion of a crest of a pit wall due to typical surface flows or flows due to an extreme flood event.
- Restricting public access to, and around the site.

Achieving these outcomes will require implementing the following strategies:

- Install abandonment bunds behind the crests of the pits. The bunds are to be offset the appropriate distances behind the crests (Section 4.7).
- Reduce to <18° batter angles of sections of the upper benches that comprise extremely weathered rocks and soil filled karst. Cover the stripped ground with broken rocks.
- Restrict public access to the site.
- Rip all unrequired access routes to and within the pits.
- Install warning signs around the pits.
- Adhere to the requirements of the completion criteria (refer Section 8).
- Visually monitor and audit the performances of the walls (refer Section 9).
- Be prepared to modify the designs of the as-constructed pits if they deviate from the specified designs and/or their performances do not comply with the completion criteria.

4.4 Pit Water Management During Typical Conditions

Under typical conditions, inflows to Pit 3-4 comprise predominantly runoff from the walls, groundwater inflows, and direct impact by rainfall.

After the EOML, inflows will no longer be pumped out of the pit. As the pit is a passive hydraulic sink, the level of water within it will subsequently increase progressively to form a pit lake. The upper surface of the lake will stabilise at an elevation at which the surface area is large enough for losses due to evaporation and other outflows to equate to the inflows. The level of the surface will be similar to those of the water in Awoonga Dam and its tributaries, although there will be time lags for the levels to equate.

The lake will be contained within predominantly unweathered limestone exposed on the lower batters. These rocks are durable²³. Hence, repeated cycles of wetting and drying of the rocks will not significantly reduce their strengths. Hence, progressive loss of the support these rocks provide to the toes of the walls is unlikely to occur.

As indicated in analyses in Section 6.3, water in the pit will not reduce the stabilities of the walls. The water will negate any deleterious effects²⁴ of excess hydrostatic pressures on stability due to flows of water through the walls into the pit. The exception could be if the level of the water in the pit is reduced (i.e., drawn down) rapidly. The resulting pressure differentials either side of rocks that were below the original level of the water could result in some rocks displacing towards the pit. However, this action is unlikely as there is no conceivable method by which a rapid draw-down will occur.

4.5 Overland Flows Around Pits

The areas around the pits, between the two levees and the natural contour highs to the north and east of the pits, are relatively flat. Most rainwater that falls during heavy intensity and/or long duration rainfall events pools in low points on the surfaces. Some of the water flows down ramps into the pits.

After the EOML, the abandonment bunds that will encircling the pits (refer Section 4.7) will modify the rates and directions of flows into the pits. Ripping the surfaces of the ramps and the surfaces between the abandonment bunds and the crests of the pit walls will also do so. The volume of water that will flow into the pits during a rainfall event will be less than the volume that flows into them pit prior to the EOML.

4.6 Management of Flood Events

Historical evidence indicates the eastern and western levees and natural contour highs north and east of the site prevent the walls of the pits being overtopped by flood water during most flood events originating from Awooga Dam and its tributaries. However, a rare Peak Maximum Flood event could result in some sections of some walls being overtopped (Figure 19). Experience in Pit 4 provides evidence that such an event does not cause significant instabilities of rocks exposed in the batters. However, soil within karst exposed in the upper batters can erode, resulting in gully erosion to the extent of the karst. Preventing this occurrence after the EOML will require earthworks in areas of soil and soil filled karst:

1. Reduce, to less than 18°, the batter angle in sections of the upper benches that comprising soil.
2. In areas of extensive and/or deep soil filled karst, create a shallow V-drain that will cause water to migrate down the centreline of the karst (Figure 20).

²³ Slake durability tests indicate the limestone has a 2nd stage durability index exceeding 95%.

²⁴ No deleterious effects have been observed, and none are expected after the EOML.

3. Sheet the soil with a minimum 0.5 m thickness of well graded durable rocks of +0.25 mm size.
4. Spray the rocks with a cementitious binder of sufficient strength and quality to reduce the likelihood of the rocks that form the bed of the drain from detaching during flows.
5. Inspect drains and, if necessary, repair damage after flood events.

As flood water subsides, the level of water in the pits will reduce as water evaporates or dissipates into the walls through the strata. The levels will eventually stabilise at the level of water in the dam and its tributaries.

Additional flood modelling prior to the EOML will ascertain whether additional strategies may be required to further reduce the likelihood of water overtopping of the walls.

4.7 Abandonment Bund

4.7.1 General Requirements

The final landform will include an abandonment bund (Figure 21) behind the crest of each wall of each pit to:

- prevent inadvertent access to the crests by vehicles, and restrict access by motorbikes and persons on foot,
- provide areas between the crests and the bunds that are sufficiently clear and flat to enable evidence of crest retreat to be observed during inspections and to enable access by equipment for remediation.

In lieu of Queensland specific guidelines on abandonment bunds, the recommendations in the Western Australian Guideline (1997)²⁵ will be applied. Their intentions are for a bund to:

- be installed behind the crest of each wall beyond potential failure zones,
- encircle the pits with no gaps other than where locked gates provide restricted access through a bund,
- be obvious,
- have sufficient height to restrict access,
- have sufficient longevity to remain functional for the long-term²⁶.

The characteristics of the bunds are indicated in Figure 22. They have a 1 m high core of compacted clayey soil. They are sheeted in unweathered crushed limestone to a minimum height of 2 m. They are not vegetated to aid their inaccessibility. Their specifications are:

- Total height 2 m (minimum)
- Footing width 4.3 m (minimum)
- Slope angles Cover: 1.1H:1V (45°)
Core: 1.3H:1V (38°)
- Cover Well-graded coarse unweathered limestone of +25 mm
- Core Well-compacted, well graded, clayey sand and/or sandy clay.

²⁵ Government of Western Australia Department of Industry and Resources (1997) *Safety Bund Walls Around Abandoned Open Pit Mines: Guideline*. MOSHAB Approved. Document No: ZMA048HA. 14 pages.

²⁶ A bund remains in place for the long-term with negligible maintenance whereas a fence can deteriorate within 10 to 15 years if not maintained.

4.7.2 Offsets from the Crests

The Guideline recommends bunds be offset a minimum of 10 m behind the point at which a construction line from the toe of a wall intersects the natural ground surface behind the wall. The line is to be orientated at 45° in unweathered rocks and 25° in weathered rocks. On this basis, the minimum offsets are those listed in Tables 6 and 7.

Stability analyses (Section 6.3) indicate that the scarp of a large scale instability is unlikely to develop more than a few metres behind a crest. This understanding is consistent with the historical performances of the walls in Pits 2, 3 and 4. On this basis, the offsets in the table are the minimum distances the bunds are to be placed behind the crests albeit within the lease boundaries. Their actual locations will be established prior to the EOML, informed by the performances of the walls prior to then and the locations of the lease boundaries.

4.7.3 Construction

The cores of the bunds will be constructed by placing the specified material in a thin layer (i.e., <30 cm), moisture conditioning, then well-compacting the layer prior to placing the next layer. The process will be repeated until a height of approximately 1 m is achieved. Doing so will aim to achieve the maximum practical compaction. The crushed rock sheeting will be end dumped or placed over the core to achieve an approximate height of 2m.

Table 6. Minimum abandonment bund offsets from the crests of the walls of Pit 2.

Wall	Height of Wall (m)	Overall Slope Angle	Weathering Depth (m)	Minimum Offset (m)
North	31	36°	31	34
South	16	38°	16	24
East	21	35°	21	25
West	37	36°	37	38

Table 7. Minimum abandonment bund offsets from the crests of the walls of Pit 3-4.

Wall	Height of Wall (m)	Overall Slope Angle	Weathering Depth (m)	Minimum Offset (m)
North	105	38°	20	10
South	107	30°	20	10
East	105	26°	20	10
West	116	38°	20	10

5 WASTE ROCK DUMP (WRD) CHARACTERISTICS AND RISK MANAGEMENT

5.1 Background

The current and final WRD is the sidehill fill type. The materials within the dump comprise low-grade limestone, limestone contaminated with clay, volcanics, and crusher fines.

The footing of the dump was constructed on a shallow (57H:1V) natural slope dipping towards the west. A thin layer of clay loam topsoil was stripped from the surface to achieve a relatively flat footing comprising bouldery silty clay having a shear strength greater than that of the topsoil. Subsequent lifts use the top down method whereby trucks end dump materials near the crests. A dozer then spreads and compacts some of the materials on the upper surface and pushes the remainder of the materials over crests where they are compacted and shaped. The slopes are advanced in the same directions as the topography. No significant drainage paths are blocked by the dump.

5.2 Specifications

Figure 2 shows the proposed final elevation of the dump at the EOML. It will be approximately rectangular, with the specifications listed in Table 8 and highlighted in Figure 25.

Table 8. Specifications of the WRD at the EOML.

<i>Dimension</i>	<i>Value</i>
Trend of major axis	105°
Length of major axis along footing	564 m
Length of minor axis along footing	362 m
Maximum height of dump	23 m
Maximum overall slope angles	7°
Minimum distance of the toe of a slope from a pit crest	58 m
Concave grade on upper levels for drainage	3°

5.3 Managing and Assessing the WRD Slopes at the EOML

After the EOML, the key geotechnical related considerations that will be managed will be:

- Geotechnical stability, particularly slumping of a slope potentially resulting in loss of material to the surrounding area (Section 7.1).
- Erosion, due possibly to failure of vegetation to establish and/or ineffective management of surface flows.
- Settlement of the upper surfaces (Section 5.4).
- Restricting public access to, and around the site.

Achieving these outcomes will require implementing the following strategy:

- Prior to the EOML, construct the cores of the outer slopes of the dump in a series of lifts from the bottom up, beginning on the natural surface. Doing so will compact the materials, so they achieve their greatest practical strength and lowest practical permeability:
 1. Strip topsoil and vegetation from the natural surface.
 2. Trucks dump waste materials in a pile.
 3. Spread the material in a near horizontal layer as thin as practical.
 4. Moisture condition (i.e., damp) the material.
 5. Use repeated passes of equipment to compact the material until no further compaction occurs with additional passes.
 6. Repeat stages 2 to 5 for subsequent lifts.
- Prior to the EOML, push down the slopes to 8.3H:1V (12%). Begin the work from the upper level. Recompact the material.
- Grade the upper surface at approximately 19H:1V (5.2%) towards the centre of the dump to direct water away from the slopes.
- Place a minimum 1 m high edge bund of unweathered rock approximately 2 m behind the crests of the slopes to provide an additional barrier to prevent water shedding over the crests.
- Rip all unrequired access routes to the dump.
- Visually monitor the performance of the dump over the medium to long-terms (Sections 8 and 9) to ascertain whether the design continues to perform acceptably.
- Be prepared to modify plans, designs and/or site works if the dump deviates from the design and/or its performance does not comply with the completion criteria.

The performances of the upper surface and the slopes of the dump will be visually monitored and assessed against the indicators and completion criteria outlined in Section 8 to demonstrate the success of the strategies.

5.4 Settlement of the WRD

Settlement of the upper surface of the dump will occur over the long-term as the materials within the dump spatially rearrange to minimise the inter-granular void spaces. This action will be exacerbated by the weight of the materials, by water infiltrating through the dump and by the water causing rocks to degrade. Crushing of grains is unlikely to occur at the low vertical pressures that will apply within this relatively low dump (i.e., <23 m).

Settlement will occur slowly as >90% of the materials within the dump are high strength and durable; a moderate proportion of which are oversize. Based on a relationship by Kermani (2018)²⁷, the maximum settlement ΔH over the long-term will be relatively small (i.e., $21 \text{ mm} \leq \Delta H \leq 42 \text{ mm}$). This range is insignificant, particularly because no infrastructure will be constructed on any surface of the dump. The associated increases in the bulk densities of the materials due to consolidation will reduce their permeabilities and enhance their stabilities. Both actions will reduce the rates of subsequent settlement.

²⁷ Kermani, M., and Konrad, J.M. (2018) In-situ short-term and long-term rockfill compressibility as a function of void ratio and strength of parent rock. In *Jnl. Geotechnical and Geoenvironmental Engineering*, April. Figure 3.

5.5 Water Management

A bund will be placed up contour of the dump on the western levee to reduce the volume of water that flows off the levee on to the upper surface of the dump.

Water on the upper surface will migrate to its centre. Some of this water will evaporate. Some will be retained within voids and pores in the near surface materials from where it will eventually evaporate. Some water will migrate into and be retained within the body of the dump. The small volume of this water that subsequently migrates to the footing of the dump will seep from the toe of the eastern slope. A drain installed along the toe will direct this water to a natural contour low thereby preventing erosion, which could result in instability at the toe of the dump. The drain will be a minimum 1.5 m wide and 0.5 m deep. It will be lined with well graded unweathered rocks.

6 STABILITIES OF THE WALLS OF THE PITS

6.1 Rockfalls and Bench Scale Stabilities

The final berms in Pit 3-4 will be 10 m wide, sufficient to contain 1 m high edge bunds (Table 2). The widths of the berms exceed the minimum 7 m width recommended by Ryan and Prior (2000²⁸) for retention of individual rocks that fall from 12 m high benches. A series of rockfall simulations²⁹ (Appendix 5) were undertaken to confirm this theory and to inform an understanding of the trajectories of rocks that release from, and traverse down, a batter and the likelihoods for these rocks to traverse over the crest of a berm. The simulations confirmed (Figure 23) that rocks are unlikely to traverse beyond the edge bunds.

Theoretically³⁰, the largest bench height wedge³¹ that could develop in a bench will have a volume and a length along the crest of approximately 265 m³ and 13 m respectively (Figure 24). Based on relationships of Gibson et. al. (2006³²), a 10 m wide berm is likely to be sufficient to prevent almost all rocks involved in detachment of a wedge of this volume from traversing beyond the edge bund³³. Those rocks that do so will be retained on the berm below.

On the bases of the empirical evidence, the results of the simulations and observations of rockfalls in Pit 4, the specified final berms are likely to have sufficient retention capacities to control and retain individual rocks and masses of rocks up to bench scale. In addition, the likelihood of a person being below a rock that detaches at the time it detaches is extremely low as access into the pit will be restricted, the pit floor will be a lake and access on to a berm will not possible. For these reasons, the risk applicable to rockfall hazards will be acceptable.

²⁸ Ryan TM and Pryor PR (2000). Designing catch benches and interramp slopes. In *Slope Stability in Surface Mining* (eds WA Hustrulid, MK McCarter & DJA Van Zyl), pp. 27-38. SME. Colorado.

²⁹ The simulations used the software Rocfall2.

³⁰ Historically, a wedge of the indicated volume has never formed on any bench in any of the three pits.

³¹ A large wedge could comprise many individual rocks.

³² Gibson, W, de Bruyn, I A and Walker, D J H, 2006. Considerations in the optimisation of bench face angle and berm width geometries for open pit mines, in *Proc. South African IMM Int Symp on Stability of Rock Slopes*, Symp. Series S44 Stability of rock slopes in open pit mining and civil engineering situations, pp. 557 – 579.

³³ The berm width is based on two equations for calculating the required berm width from the wedge failure volumes, which depend on the geometry of the failed material on the berm (conical vs pyramid shape). Assumed is a swell volume of 1.2, volume of wedge of 265 m³, wedge width along crest of 13 m and 38° angle of repose of the failed rock.

6.2 Inter-ramp Scale Stabilities

No inter-ramp (IR) scale instabilities have occurred in any of the pits at this site since mining commenced. None is expected to occur after the EOML as the IR angle of the final walls (i.e., 38°) will be less than it is for the current walls (i.e., 40° to 48°). Nonetheless, a series of analyses (Appendix 2) were undertaken for Pit 3-4 to confirm this understanding. An analysis was not undertaken for Pit 2 as this pit is not benched. The analyses indicate:

- Toppling will be the most likely mode of instability in unweathered rocks on all walls (e.g., Figure 11). The mode will involve masses of rocks toppling from discontinuities that dip steeply into the walls. This result is consistent with occurrences in Pit 4.
- Instabilities involving masses of rocks sliding along underlying discontinuities (i.e., planar sliding) or pairs of intersecting discontinuities (i.e., wedge failure) will be less prevalent.

Over the long-term, as weathering reduces the shear strengths of the discontinuities, toppling may become less prevalent and wedge sliding (e.g., Figure 10) may become increasingly more prevalent. These modes will be most apparent in the upper benches, within which will be the most weathered discontinuities. They will be less frequent on the lower benches where discontinuities will be less weathered. The relationships between weathering and the modes of instability are consistent with occurrences in Pit 4.

The results of the analyses are listed in Tables A2.1 and A2.2 in terms of likelihood for instability (P_f) values³⁴. The acceptabilities of these values can be considered by comparing them to values typically considered acceptable in the mining industry.

Read and Stacey (2009)³⁵ recommend maximum P_f values of 20% as a general guide for inter-ramp scale instabilities where the consequence of instability could be “low” to “medium”. These classifications are appropriate due to access into hazardous area by persons during the PMLU being significantly restricted, interramp scale breakback at a crest being within the area bounded by the abandonment bunds and no inter-ramp scale instability having occurred in any pit since mining commenced even though the inter-ramp angles have exceeded those that will be applicable after the EOML.

The results from the analyses indicates that the P_f values are approximately 20% or less, hence are within the range of acceptability. On this basis, the proposed bench specifications are likely to be to be acceptable.

6.3 Wall Scale Stabilities

No instability of wall scale (i.e., wall crest to toe) has occurred in any pit at this site since mining commenced. However, if such an instability occurs during the PMLU it would likely result in a section of the crest of a wall retreating. A series of analyses were therefore undertaken to quantify the likelihood of such an event and to highlight the likely maximum offsets from the crests of potential surfaces of instability. These offsets inform the area that should be inspected for evidence of instability during the PMLU. They also inform the abandonment bund set back to ensure a section of a bund is not subsumed by crest retreat.

³⁴ The likelihood of sliding quantifies the probability that a particular location on a wall will experience a sliding instability.

³⁵ Acceptance criteria are listed in Read, J. and Stacey, P. (2009) Chapter 9: Acceptance Criteria. *Guidelines for Open Pit Slope Design*. CSIRO Publications.

Details of the analyses are discussed in Appendix 3. The results indicate:

- A wall scale instability is unlikely during the PMLU. This result is consistent with the performances of the walls in all pits to date.
- The largest surfaces likely to develop have toes that daylight within weathered limestone in the upper two benches. If large instabilities develop, they will likely occur in, and be limited to, these benches with crest retreat being less than 3 m. Retreat is therefore unlikely to extend to an abandonment bund.
- Water within either pit does not have a significant influence on the stabilities of the walls.

The results of the analyses (Figures A3.4 to A3.8) are quantified in terms of Factor of Safety (FoS)³⁶ values. Read and Stacey (2009) recommend a minimum value of 1.3 under static loading conditions and 1.05 under dynamic (earthquake) loading conditions where the consequence of instability could be “medium”. This classification is appropriate for the reasons noted in Section 6.2. The results from the analyses indicate that no wall scale surface in either pit has a FoS value less than 1.5 for both loading states, hence are within the ranges of acceptability. On this basis, the wall specifications are deemed to be acceptable.

7 STABILITIES OF THE SLOPES OF THE WRD

7.1 Slope Scale Stabilities

The current slopes of the WRD dip at approximately 33°. No significant instability has occurred within these slopes to date. At the EOML, the slope angles will be reduced to 7°. The likelihood of an instability developing at these angles is significantly less than at the current date. A series of analyses were undertaken to quantify the stabilities of the highest of the final slopes during the long-term as the rocks within the dump weather. The analyses also highlight the likely offset of the lowest strength surfaces within the slope. This offset informs the extents of the areas behind the crests of the slopes that should be inspected for evidence of instability. It also informs the locations where edge bunds should be placed. Details of the analysis are discussed in Appendix 4.

The results of the analysis are quantified in terms of Factor of Safety (FoS) values for each of hundreds of surfaces assessed within the slope. The surfaces with the lowest capacities are highlighted in Figures A4.2. The results indicate that no overall slope scale surface has a FoS value less than 4.0. The acceptability of this lower bound value is considered by comparing it to values typically considered acceptable in the mining industry. Subsummation within flowing rocks at the toe of a slope or within a collapsing crest are high consequence events. However, these events are very unlikely to occur due to the risk mitigation strategies that will be implemented (Section 5.3). They are therefore medium risk events³⁷. Hawley and Cuning (2017)³⁸ recommend a minimum FoS of 1.5 under static loading and 1.15 under dynamic loading be applied for these events. A comparison of the results from the analysis with these criteria indicates the FoS values exceed significantly the guideline values. Hence, the slopes are likely to be acceptably stable during the long-term post EOML.

³⁶ The Factor of Safety (FoS) is the ratio of the average shear strength of a surface to the average shear stress applied to the surface. A FoS value of less than unity (1.0) indicates a 50% or greater likelihood for an unstable scenario.

³⁷ Risk classification terms are consistent with AS/NZS ISO 31000-2009 Risk management— Principles and guidelines. Standards Australia

³⁸ Acceptance criteria are listed in Hawley, M. and Cuning, J. (2017) Chapter 8: Acceptance Criteria. *Guidelines for Mine Waste Dump and Stockpile Design*. CSIRO Publications.

During the long-term, weathering of the materials within the dump will create additional fines which will increase the bulk densities of the materials. They will also reduce the permeabilities of the materials which could enable pore water pressures to develop. These actions may result in the angles of the slopes reducing progressively. As they do, the slopes will become more stable. Extension of any toe will be negligible and will not result in any additional geotechnical hazard. A drain and bunds placed beyond the toes of the slopes will prevent any material from traversing beyond their locations.

8 COMPLETION CRITERIA

Table 9 lists the general objectives relevant to geotechnical aspects of the pits and the WRD during the short- to long-terms. Milestone criteria relevant to these aspects are listed in Table 10. The respective landforms will be considered suitable for relinquishment when these objectives and criteria have been met.

Table 9. General Objectives Relevant to Geotechnical Aspects of the Landform

<i>Short-term</i>	<i>Medium to Long-term</i>
<ul style="list-style-type: none"> • Minimise the areas disturbed by limiting clearing to the minimum area required for the operation and by progressively rehabilitating areas as they become available. • Install erosion control measures prior to the commencement of soil stripping and rehabilitation activities. • Strip soil from areas of disturbance only as, when, and where required to reduce the potential for erosion and to minimise the volumes of soil stockpiled and the period of storage. • In preference to stockpiling soil, replace stripped soil directly on to completed sections of the final landform where practical. • Reshape sections of the batters of the upper benches to their final landform so they can be progressively rehabilitated. • Stabilise haulroads, batters of the upper benches of the pits and the slopes of the WRD as soon as practical. • Visually monitor the performances of the walls of the pits and the slopes of the WRD and the surrounding landforms. 	<ul style="list-style-type: none"> • Provide self-maintaining, geotechnically stable and safe landforms that compliment with the surrounding landforms in terms of slopes, geomorphological characteristics, and postmining land use. • Remediate hazards associated with the pits and the WRD that may increase the risks applicable to persons and/or animals. • Construct the upper surfaces on the WRD to ensure rainfall runoff drains away from the crests of the slopes towards the centres of the surfaces. • Minimise the catchment areas applicable to the pits and the toes of the WRD with up contour bunds and the abandonment bunds.

Table 10. Proposed Geotechnical Rehabilitation Milestone Criteria for the Residual Voids and WRD.

<i>Landform</i>	<i>Rehabilitation milestone reference</i>	<i>Rehabilitation milestone</i>	<i>Milestone criteria</i>
Residual voids (Pit 2 and Pit 3-4)	RM3	Landform development and reshaping / re-profiling of residual void	<ul style="list-style-type: none"> • An AQP³⁹ has confirmed that geotechnical stability of the residual void walls (i.e., a factor of safety value of ≥ 1.2) has been achieved. • Pit 3-4 batter slopes meet final design specifications (18° in weathered rock, 65° in unweathered rock) • Pit 2 maximum overall slope angle meets final design specifications (34°) • Perimeter abandonment bund and safety signage installed offset, as specified by an AQP, from the crest of the voids • Locked gates and signage installed at access points to the residual voids to prevent unauthorised access • AQP certifies that void final landform design intent has been achieved
	RM10	Achievement of post-mining land use to stable condition (water storage)	<ul style="list-style-type: none"> • Visual inspection by an AQP provides evidence that the stabilities of the residual void crests and walls have remained constant or increased following cessation of mining and rehabilitation activities
WRD	RM4	Landform development and reshaping / re-profiling of waste rock dump.	<ul style="list-style-type: none"> • As built inspection by an AQP confirms that the final landform design intent has been achieved, and batter slopes do not exceed 12%
	RM11	Achievement of post-mining land use to stable condition (grazing)	<ul style="list-style-type: none"> • Visual inspection by an AQP provides evidence that the geotechnical stability of the WRD has remained constant or increased following completion of rehabilitation activities

³⁹ AQP = Appropriately qualified person (such as a qualified geotechnical consultant)

9 GEOTECHNICAL MONITORING

Calliope's Ground Control Management Plan (GCMP) requires the following geotechnical monitoring and management strategies be implemented until the EOML:

- Profiles of the walls of the pits and the slopes of the WRD will be collected annually using drone based photogrammetry. Surfaces will be created from the point clouds. The surfaces will be compared to previous surfaces to highlight changes. Excessive deformations will require strategies to be implemented to ensure the deformation mechanisms are understood and, if necessary, ensure the risk to persons and the operation remain acceptable.
- Data quantifying the orientations of the discontinuities most recently exposed in Pit 3- 4 will be collected using discontinuity mapping techniques if changes to the geology occurs. The data will be compared to data collected elsewhere to highlight whether the characteristics of the discontinuities are consistent with those recorded previously. If different, the data will be used as input to kinematic analyses to ascertain whether the specifications of the walls are still appropriate or require modification.
- Visual inspections of the walls of the pits and the slopes of the WRD will be undertaken weekly. Hazards and strategies for managing any increase in risk will be recorded in the daily log.
- An audit of geotechnical risk issues will be undertaken by an independent geotechnical consultant every 18 months.

A programme for visually monitoring and auditing the walls, slopes, and the landforms after the EOML will be established prior to then.

10 SUMMARY

10.1 Conclusions

- The proposed landform designs satisfy the geotechnical related requirements of the PRCP Guideline.
- The long term stabilities of the walls of Pit 2 and Pit 3-4 and the slopes of the southern WRD will be within the ranges typically considered acceptable in the mining industry.
- After the EOML, the water level in Pit 3-4 will increase. A pit lake will form with a final level being controlled by the heights of water in Awoonga Dam and its tributaries. The water will not significantly reduce the performances of the pit walls unless rapid drawdown occurs, which is unlikely.

10.2 Risk Management Strategies

After the EOML, the principal strategies to reduce the risks applicable to the walls of the pits and the slopes of the WRD will be:

- restricting public access to the site,
- ripping access routes within and into the pits and to the upper surfaces and toes of the WRD,
- installing abandonment bunds behind the crests of the pits and edge bunds behind the crests of the WRD and ensuring the offset of the abandonment bunds are as specified in Section 4.7.2,
- visually monitoring and auditing the performances of the walls of the pits and the slopes of the WRD,
- routinely inspecting the areas behind the crests of the pit walls and the slopes of the WRD for evidence that may be indicative of large scale instabilities developing (e.g., cracking, subsidence etc.),

- adherence to the requirements of the completion criteria (Section 8),
- prior to the EOML, preparedness to modify plans and/or designs and/or site works if any landform deviates from the design and/or its performance does not comply with the criteria.

10.3 Limitations in the Stability Analyses

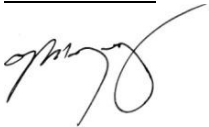
The stability analyses for the walls of the pits and the slopes of the WRD and the risk assessment process are based on limited data. For example, no information is available on the locations, extents, or characteristics of any very persistent structures (e.g., faults, shear zones etc.) that may occur behind a pit wall prior to the EOML and how these structures may influence the performance of the wall during the PMLU. The implications of these limitations are:

- the geotechnical model will be updated if information becomes available prior to the EOML that could influence the conclusions and recommendations,
- adherence to the recommended strategies will be necessary to ensure the risks to persons and the post-closure landform remain acceptable.

11 CLOSURE

This report addresses the geotechnical related issues required in a PRCP as listed in Section 1. I trust that it provides the information you require. Please contact the undersigned if you have any queries on any matter discussed within it or wish to discuss any aspect of the report in more detail.

For and on behalf of
ROCKTEST



Tony Meyers
Principal Rock Mechanics Engineer

FIGURES



Figure 1. Current view of the site highlighting the approximate footprints of the final landforms of Pit 2, Pit 3-4 and the WRD .

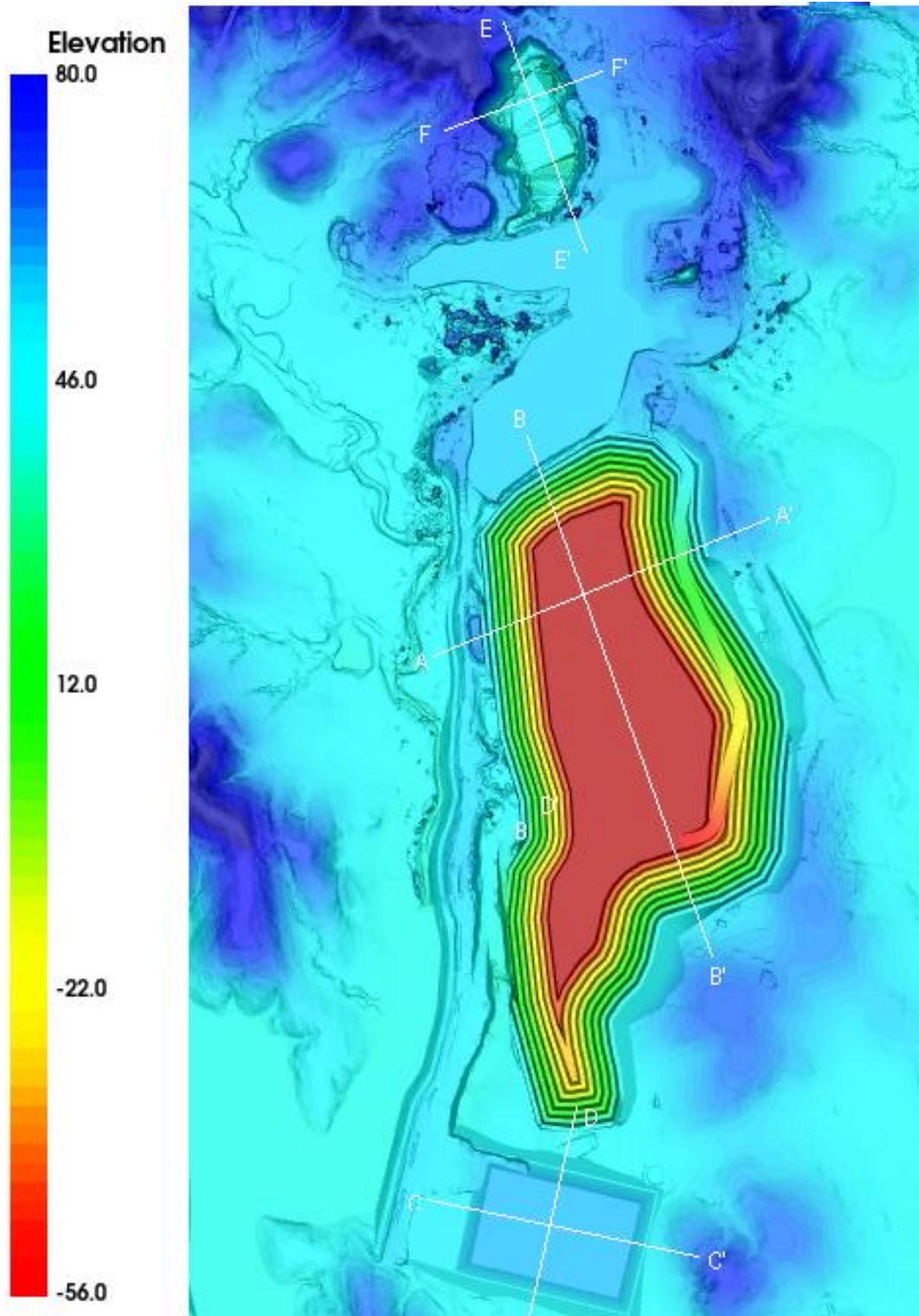


Figure 2. Elevations of landforms around Pit 3-4 and locations of cross sections.

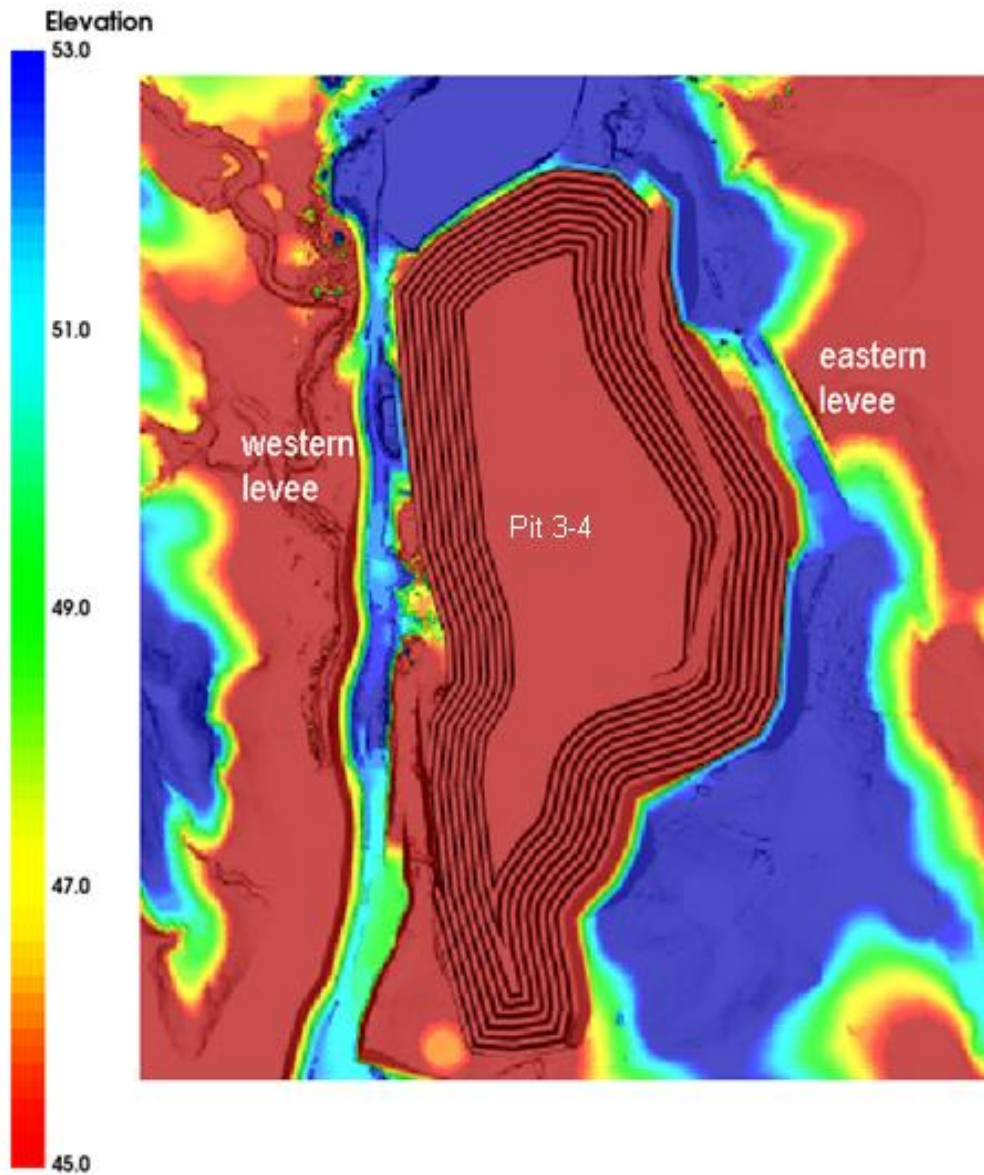


Figure 3. Elevations of landforms around Pit 3-4 highlighting the RLs of the eastern and western levees.



Figure 4. Eastern levee looking south. Awoonga Dam is to the left and Pit 4 is to the right.



Figure 5. Western levee looking north. Ragotte Creek is to the left and Pit 4 is to the right.



(a)



(b)

Figure 6. Basalt lava flows exposed in the upper benches of the east wall of Pit 4 and andesite dykes daylighting within the (a) floor and (b) south wall.



(a)



Figure 7. Bouldery clay overlying limestone on the (a) east wall and (b) north wall of Pit 4.

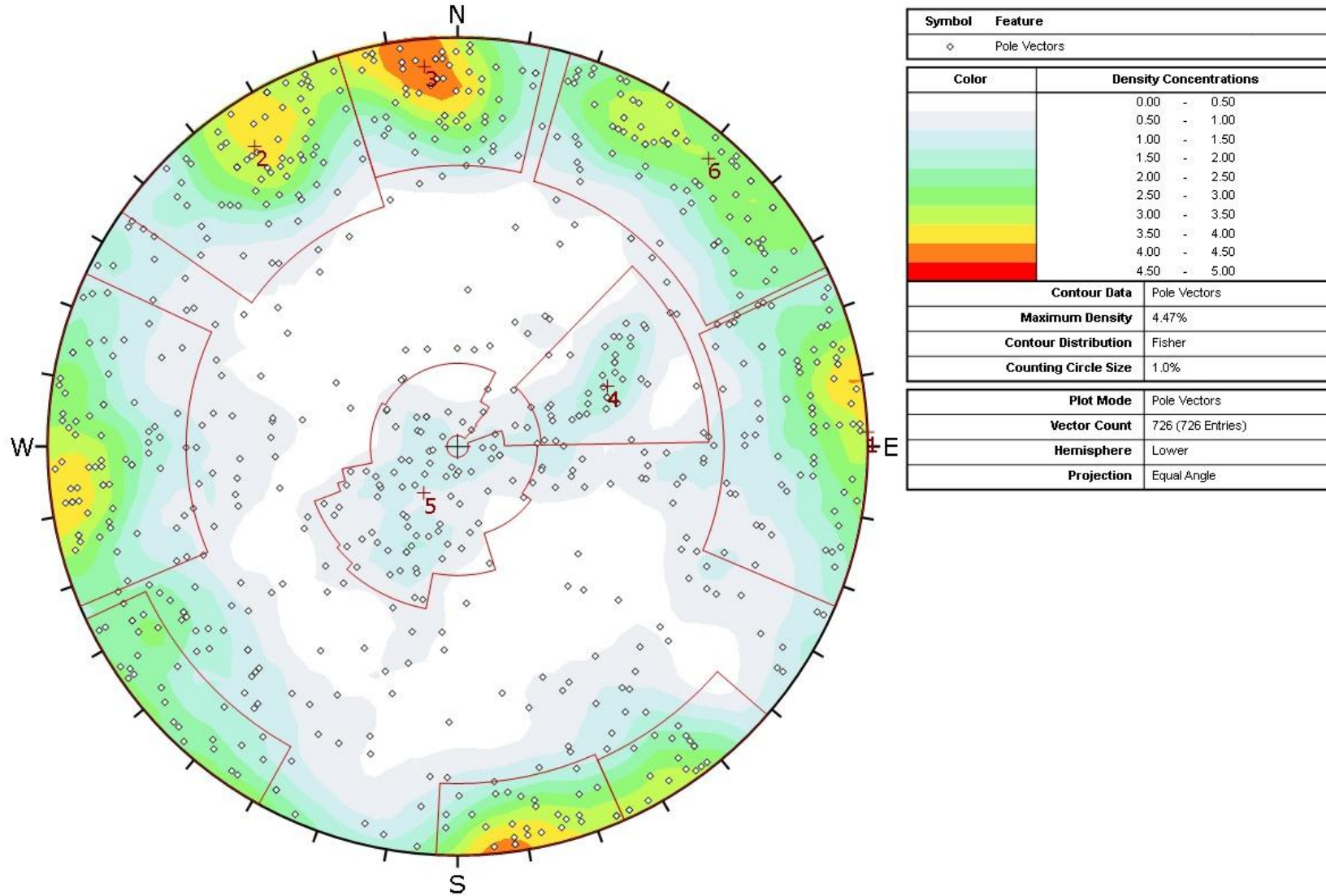


Figure 8. Stereonet highlighting the orientations of poles to 726 discontinuities within the fresh limestone, the boundaries of the 6 sets into which the discontinuities have been assigned and the mean orientations of the sets.



Figure 9. Examples of planar sliding on the west wall of Pit 4 involving bench scale masses of rocks sliding along bedding/cleavage from Set 1 and joints from Set 5.

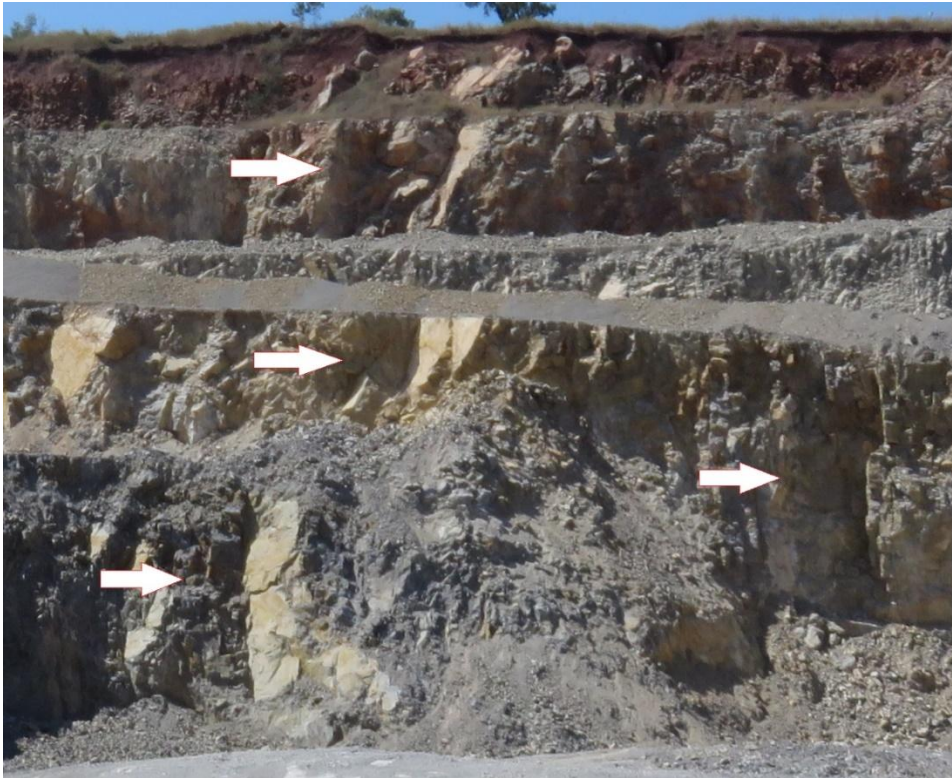


Figure 10. Examples of wedge sliding on the east wall of Pit 4 involving bench scale masses of limestone sliding along steep dipping joints from Sets 3 and 4 having medium to high persistences.



Figure 11. Example of toppling on the north wall of Pit 4 involving bench scale masses of rocks bounded to the rear by steep dipping joints from Set 3 having high persistences..



(a) Apertures within discontinuities daylighting in the north wall of Pit 4.



(b) Cavern on the south wall.

Figure 12. Examples of clay filled karst structures within the upper benches of Pit 4.



Figure 13. Pit 1 (upper) was fully backfilled in 2024 (lower) with well compacted blasted rocks and fines.



Figure 14. Pit 2 has not been mined since the 1970s and will not be mined prior to the EOML.

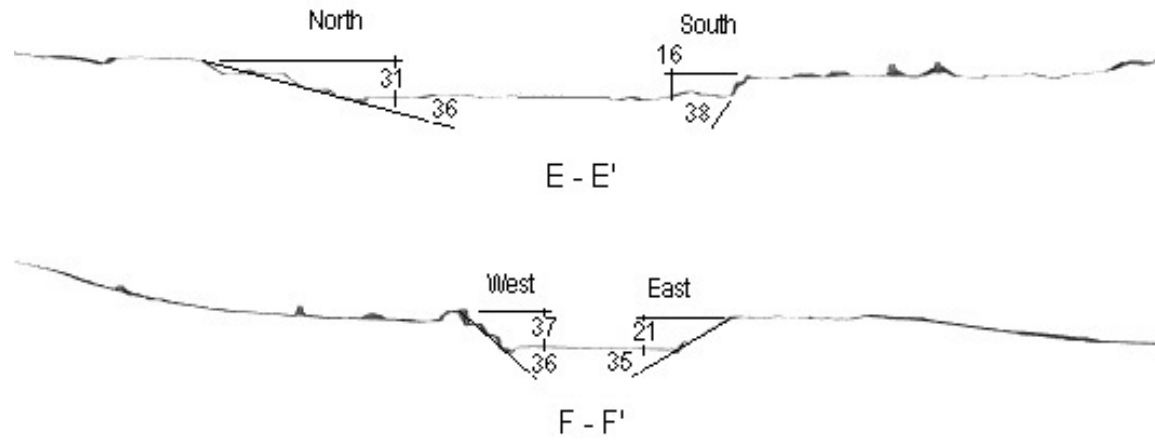


Figure 15. Cross-sections through the walls of Pit 2 at the EOML. Refer Figure 2 for locations.

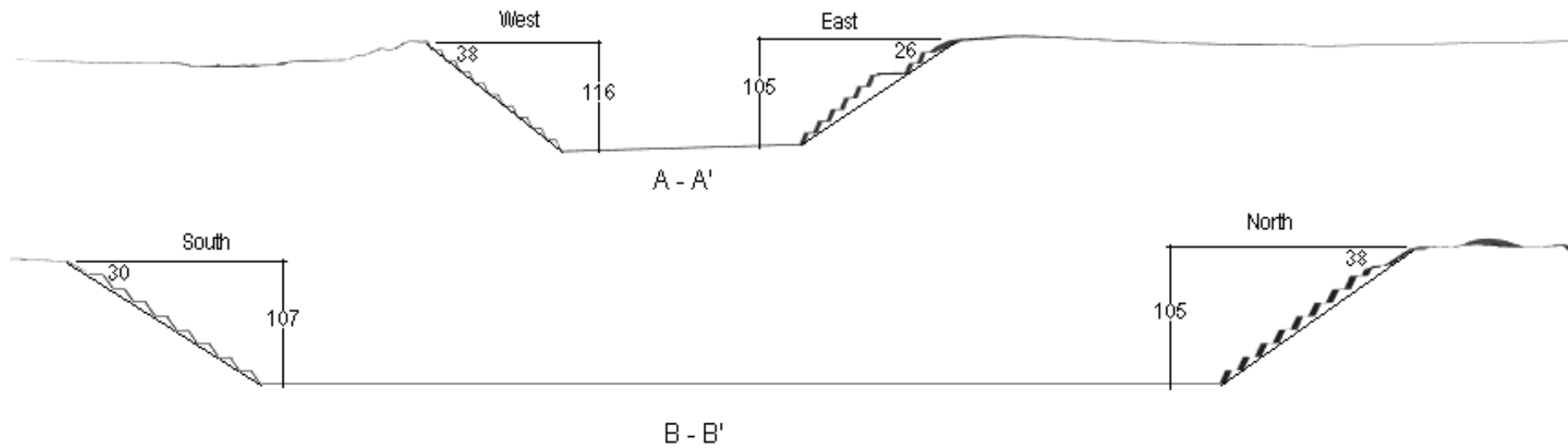


Figure 16. Cross-sections through the walls of Pit 3-4 at the EOML. Refer Figure 2 for locations. .



Figure 17. Water can seep from the batter of the upper bench on the east wall of Pit 4 when the level of water in Awoonga Dam exceeds ≈ 40 mRL.

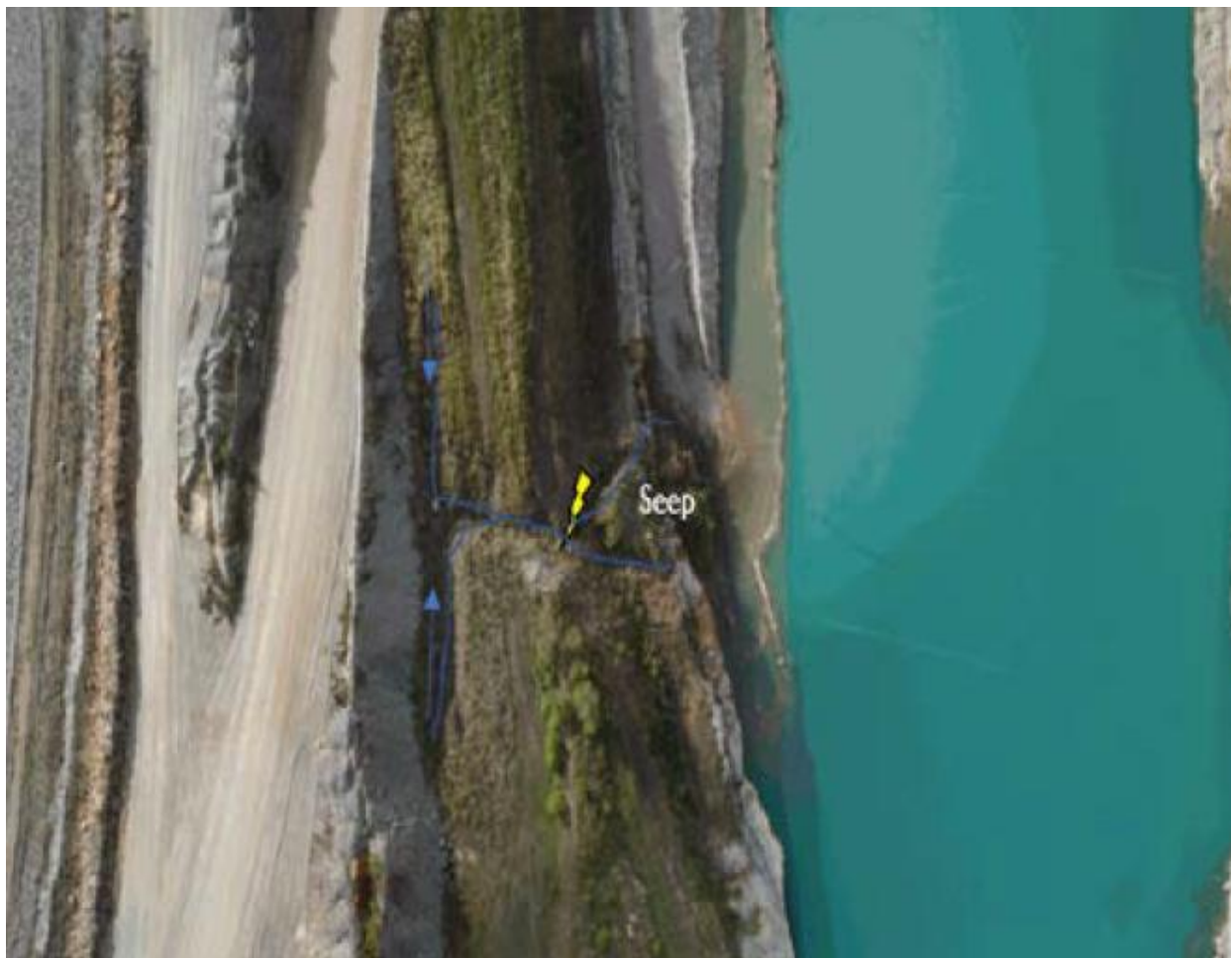


Figure 18. Water can seep from the upper batter on the west wall of Pit 3 when the level of water in Awoonga Dam exceeds ≈ 40 mRL.

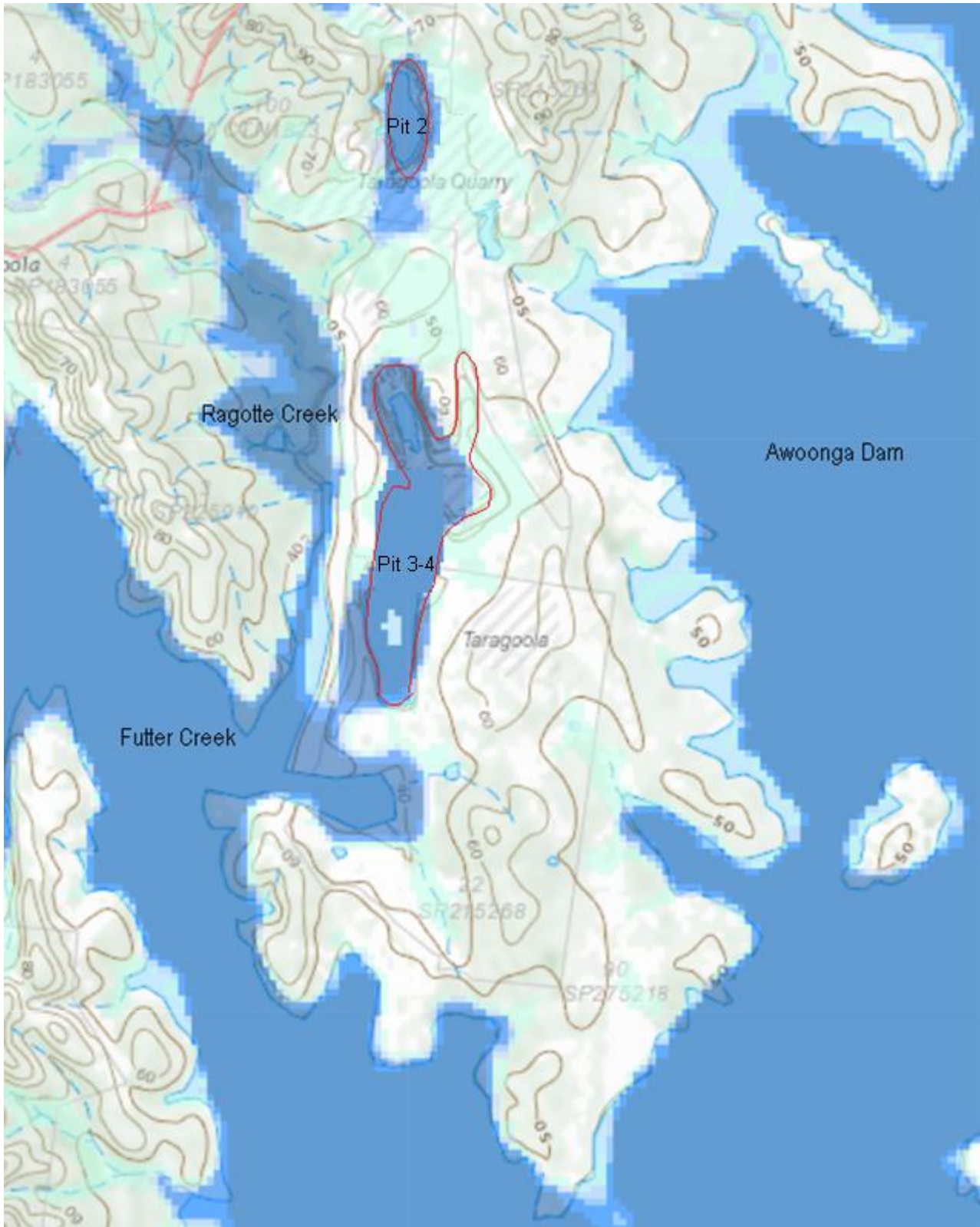


Figure 19. Estimated water level for an extreme flood event.
(Source: <https://floodcheck.information.qld.gov.au>).



Figure 20. Example of a shallow rock lined v-drain constructed down a slope comprising erodible soil to prevent gully erosion. Similar drains will be required in sections of final upper benches comprising soil filled karst.



Figure 21. Example of an abandonment bund similar to those to be placed around Pits 2 and 3-4.

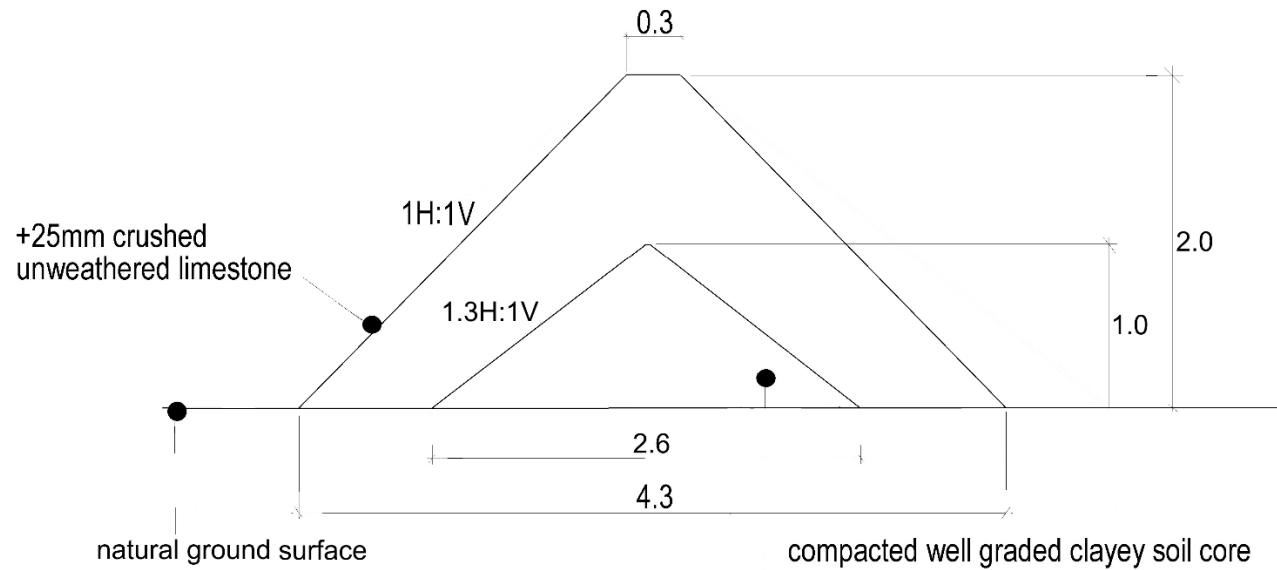


Figure 22. Nominal specifications of the abandonment bunds to encircle Pits 2 and 3-4.

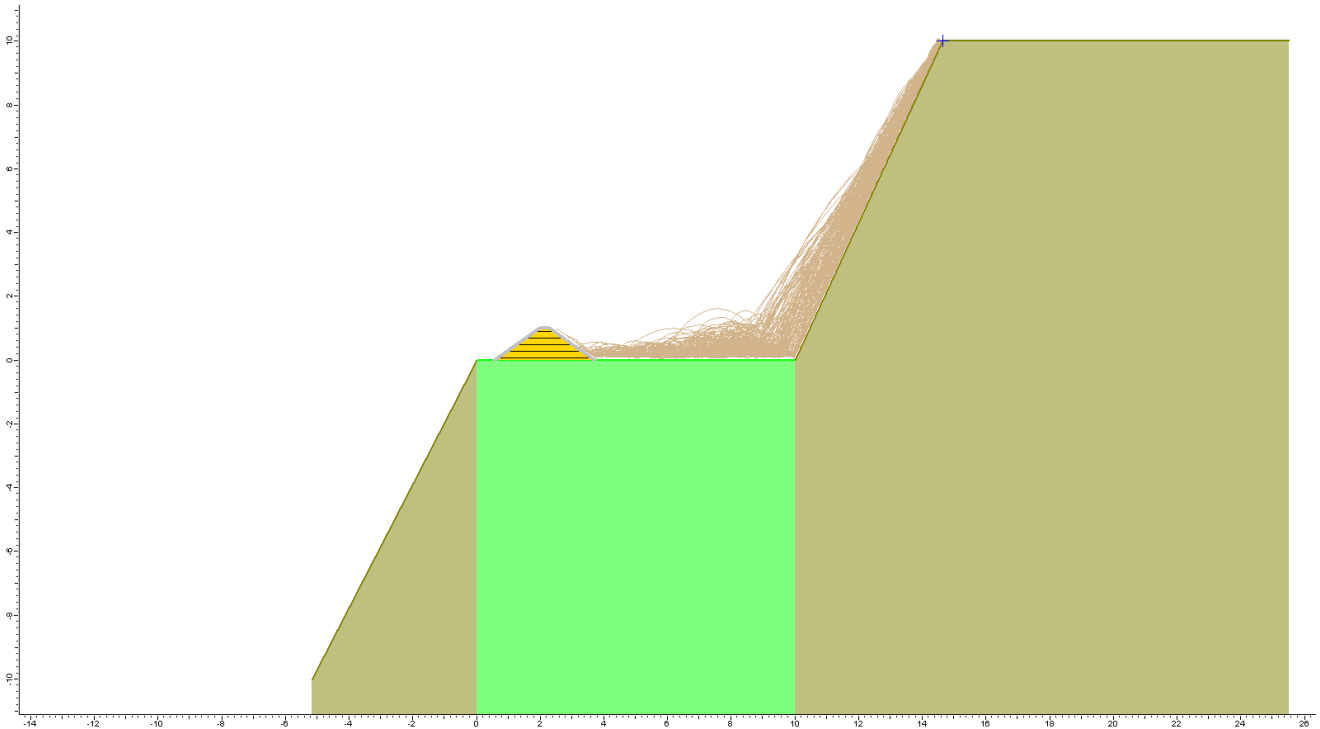


Figure 23. Rockfall simulations of 1000 rocks detaching from the crest of a batter indicate that no rocks are likely to traverse beyond the edge bund.

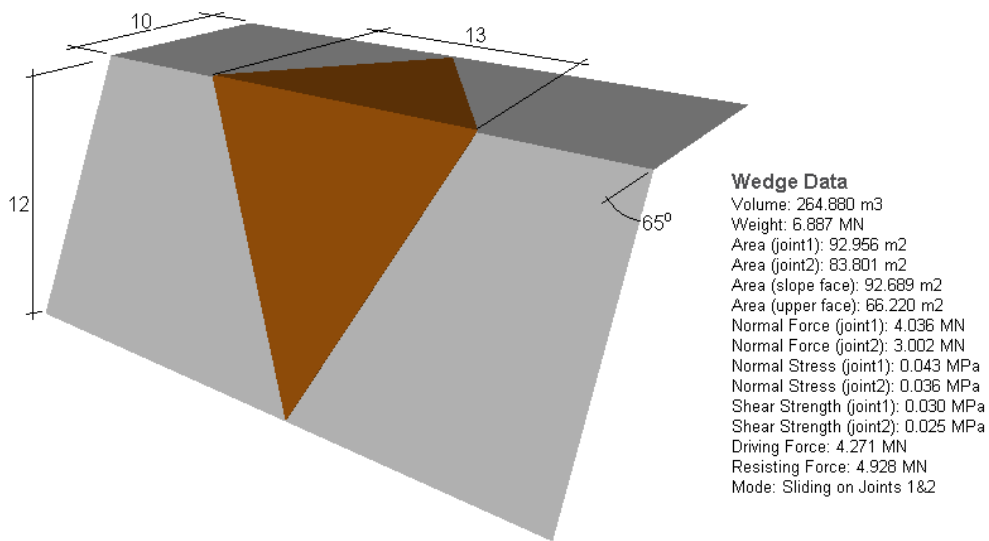


Figure 24. Largest wedge that could develop within a bench. Spill volume analyses indicate that almost all rocks will be retained on the specified 10 m wide berms.

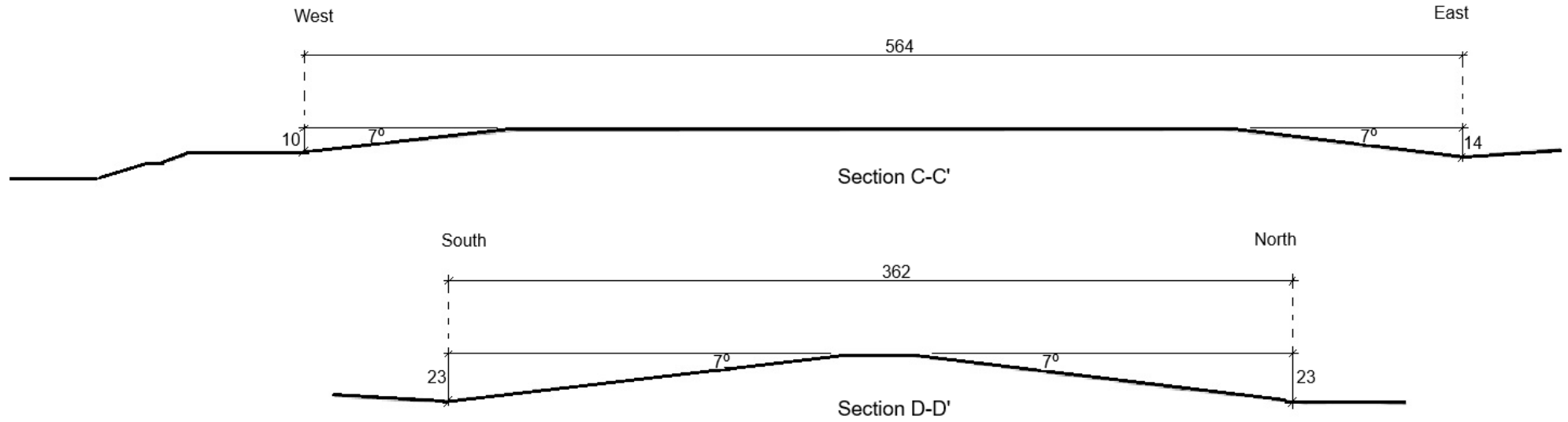


Figure 25. Cross-sections through the steepest slopes of the WRD at the EOML. Refer Figure 2 for locations of cross sections.

ADDITIONAL INFORMATION

- The recommendations in this report have been based on certain assumptions and very few observations. It would be very rare if the conditions at the site did not vary from those observed even if exhaustive investigations had been undertaken. Hence, it is unlikely that the conditions observed will represent the extremes of conditions that exist. As such, even though the aim of this report is to address issues influencing geotechnical related risks applicable to the landforms at the Calliope mine during the PMLU, it is possible for events to occur or conditions to exist that were not planned for and as such should not be unexpected.
- The report was prepared on completion of the field work and is based on conditions encountered and reviewed at the time of preparation. Natural developments in, and changes to, or surrounding a volume of geomaterials over time, which are out of the control of the Rocktest Engineer (“Engineer”), will change the stresses that act through the geomaterials and their engineering characteristics. Changes in these conditions may significantly alter the way in which the ground conditions behave. As such, the conclusions reached in this report will be less applicable over time and may not be applicable or relevant at a later point in time. Rocktest disclaims responsibility for any changes that might have occurred or the implication of these changes on the assessments or recommendations in this report.
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- It is not possible to make a proper assessment of the report without a clear understanding of the terms of engagement upon which it was prepared, including the scope of instructions, directions and information provided by the client to those who prepared the report and the assumptions made in the report. The report may not address issues that would need to be addressed if the intention, environment, circumstances, time requirements and brief were different from those for which the report was prepared. Only the Engineer has a true understanding of the effects that these limitations and the assumptions made in the report may have on the conclusion reached within the report. As such, no party other than the Client should use the data, contents of the report or the techniques used in it for application to conditions different from those for which the report was intended without an independent and detailed review being carried out as to its suitability, relevance, and accuracy.
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- The Engineer will not accept any responsibility for negligence, nor any loss or damage suffered by the Client due to the occurrence of events or the existence of any factors which are out of the Engineer’s control. The Engineer will indemnify the Client for claims or loss arising from a breach of professional duty in the provision of the professional services. The liability of the Engineer is reduced to the extent that the Client or other person(s) caused or contributed to the loss or occurrence which gave rise to the claim.
- This report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners. The above clauses do not in any way attempt to absolve the Engineer from any professional responsibilities in relation to the diligent and careful performance of duties in providing this report. Rather, they are intended to be a guide as to where the Engineer’s responsibilities end, and where forces of nature assume responsibility for the composition of a volume of geomaterials and its performance.

APPENDIX 1 ROCKMASS CHARACTERISTICS

A1.1 Compressive Strengths

The compressive strengths of weathered and unweathered limestone were defined in terms of Point-Load Strength Indices, $I_{s(50)}$.

Samples of limestone were collected at random locations within the pit and from areas nearby. No preparation of the rocks was required. They were tested at their moisture content at the time of testing.

The tests were carried out in accordance with ISRM (1985⁴⁰) and AS 4133.4.1 9 (1993⁴¹).

1. A rock was inserted into an ELE point-load test apparatus. The rock was aligned perpendicular to the dominant fabric.
2. The platens were closed to contact the smallest dimension and away from the edges.
3. The load was steadily and slowly increased until failure. Tests for which the failure plane did not fully intersect the rock were rejected. The pressure at failure was recorded.
4. The distance between the platens at failure and the width of the failure plane were recorded with a Vernier calliper. The width was calculated as $(W1+W2)/2$ ⁴² if opposite sides of the failure plane were not parallel.

The results are summarised in Table A1.1.

Table A1.1. Summary of point load strength indices $I_{s(50)}$ for limestone.

<i>Grade</i>	<i>Number of tests</i>	<i>Minimum (MPa)</i>	<i>Mean (MPa)</i>	<i>Maximum (MPa)</i>
weathered	4	1.5	2.0	2.6
unweathered	11	2.1	3.6	6.3

A1.2 Specific Gravities

The densities of weathered and unweathered limestone were estimated as functions of their specific gravities.

1. The mass (M_d) of a dry rock was measured with a Nuweigh KK5000 electronic balance.
2. The rock was suspended in water and its saturated mass (M_s) was measured.
3. The rock was removed from the water and its wet mass (M_w) was measured.
4. The density of the rock was estimated from the following equation.

The results are summarised in Table A1.2.

$$\text{Density} = \frac{1000 M_d}{M_w - M_s}$$

⁴⁰ ISRM (1985) International Society for Rock Mechanics Commission on Testing Methods. Suggested Methods for Determining Point Load Strength, Int. Jnl. Rock Mech., Min. Sci. & Geomech. Abstr. Vol. 22 No. 2 pp.51-60

⁴¹ AS 4133.4.1 (1993) Australian Standard Methods of Testing Rocks for Engineering Purposes. Method 4.1 Rock Strength tests – Determination of Point Load Strength Index.

⁴² ISRM (1985) and AS4133.4.1 suggest that the dimension W should be the minimum dimension of the sample perpendicular to the loading direction prior to testing due to the simplicity of obtaining this dimension. However, researchers and the German Standards argue in favour of measuring W as the minimum dimension of the failure surface after testing although obtaining this dimension is more time consuming than the former method. Rocktest adopts the latter method.

Table A1.2. Summary of density values for limestone.

<i>Grade</i>	<i>Number of tests</i>	<i>Minimum (kg/m³)</i>	<i>Mean (kg/m³)</i>	<i>Maximum (kg/m³)</i>
weathered	8	2511	2576	2617
unweathered	10	2575	2631	2679

A1.3 Basic Friction Angles

The basic friction angles⁴³ (ϕ_b) of discontinuities within unweathered and weathered limestone were estimated using the procedure:

1. A diamond tipped saw blade was used to cut through the sample. The smooth cut surfaces were manually abraded to remove all roughness features. The halves were then reassembled.
2. The lower half of the sample was tilted slowly until the top half began to slide. The tilting was stopped.
3. The tilt angle was recorded with a digital M&D “SmartTool” level.
4. The halves were realigned and the test repeated a minimum of 12 times.

Table A1.3. summarises the results from the tests. Pitting of the surfaces contributes to the values for the weathered rocks being higher than those for the unweathered rocks.

Table A1.3. Summary of basic friction angles test data.

<i>Grade</i>	<i>Number of tilts</i>	<i>Minimum</i>	<i>Mean</i>	<i>Maximum</i>
weathered	50	23°	25°	27°
unweathered	54	16°	21°	24°

A1.4 Discontinuity Shear Strengths

Bench scale instabilities within the pits are dominated by gravity driven sliding and toppling of rocks. Displacements occurs when the driving forces applied to individual rocks and masses of rocks exceed the shear strengths of the underlying discontinuities⁴⁴. In this study, these strengths are quantified in terms of minimum and mean peak friction angles⁴⁵. The discontinuities are assumed to have negligible cohesive strengths as:

- the rocks within the benches have been vibrated during blasting,
- most discontinuities have undergone some shear displacements during tectonic deformations and creep which destroyed cementation,
- relatively low stresses are applied to the discontinuities within the benches.

⁴³ The basic friction angle quantifies the inter-granular friction after all roughness asperities have been abraded from opposing surfaces of a discontinuity.

⁴⁴ Assuming rocks are free to move out from a bench.

⁴⁵ Any movement along a discontinuity destroys any cohesive bonds leaving a surface that is purely frictional.

The “shape” of the shear strength/normal stress curve is described by the non-linear Barton-Bandis (BB) shear strength criterion⁴⁶. The criterion is defined in terms of the following parameters. Their assumed values are listed in Table A1.4:

- Joint Wall Compressive Strength (JCS) is assumed to be equivalent to the compressive strengths of the intact rocks. These strengths were quantified as functions of the point load strength indices listed in Table A1.1. A conservative multiplication factor K (i.e., $UCS = K \times I_{s50}$) of 18 was assumed.
- Normal stress (σ_n) acting perpendicular to a discontinuity. This stress is a function of the unit weight of the overlying rocks. This value was quantified using the data obtained from the specific gravity tests listed in Table A1.2.
- Residual friction angle (ϕ_r) of the discontinuities was quantified as a function of the basic friction angles obtained from the tilt tests listed in Table A1.3.
- Joint Roughness Coefficients⁴⁷ (JRC) was quantified by comparing the surfaces of in-situ discontinuities to standard roughness profiles. The values are listed in Table A1.5.

The estimated peak friction angles are listed in Table A1.6. The range of values is typical for limestone. The values for the unweathered limestone are lower than those for the weather limestone as a pitted surface due to weathering increases the shear strength of the surface. The lowest value, 18°, is used in stability analyses as a lower bound shear strength.

Table A1.5. Summary of Joint Roughness Coefficient (JRC) values.

<i>Number of observations</i>	<i>Minimum</i>	<i>Mean</i>	<i>Maximum</i>
532	1	4	10

Table A1.6. Summary of peak friction angles for limestone based on B-B model.

<i>Grade</i>	<i>Minimum</i>	<i>Mean</i>
weathered	25°	35°
unweathered	18°	32°

⁴⁶ Barton, N. and Bandis, S (1990) Review of predictive capabilities of JRC-JCS model in engineering practice. In *Proc. Rock Joints*. Balkema pp.803-810.

⁴⁷ The Joint Roughness Coefficient (JRC) quantifies the roughness of the surface of a discontinuity. It is estimated by comparing the surface of a discontinuity with standard roughness profiles. The roughness is scaled from one for a very smooth (slickensided) surface to 20 for a very rough stepped surface.

A1.5 Rock Mass Strengths

The shear strength of the limestone rockmass was estimated using the Generalised Hoek-Brown Rock Mass Strength criterion⁴⁸. The criterion is defined in terms of the parameters listed in Table A1.8.

- The Hoek-Brown intact rock constant m_i for limestone.
- The disturbance factor (D) which depends upon the degree of disturbance to which the rock mass at the depth of a potential surface of instability has been subjected by blast vibration and stress relaxation⁴⁹. A value of 0.5 is applied as controlled blasting is not used and hence benches undergo some disturbance.
- The Geological Strength Index (GSI)⁵⁰ is a quantitative value that defines the quality of the rockmass in terms of the parameters⁵¹:
 - Rock volume (Table A1.7) is a function of the set spacing values listed in Table 1. Characteristics of the sets of discontinuities in the limestone. Only the minimum and mean values are calculated. A maximum value of 1 m^3 is based on observation of oversize rocks⁵².

Minimum	Mean	Maximum
$3 \times 10^{-14} \text{cc}$	$4.2 \times 10^3 \text{cc}$	$1.0 \times 10^6 \text{cc}$

- Joint Condition Factor (J_c) is the ratio of the following qualitative parameters from the Q^{53} rock mass classification system:
 - Joint Roughness number (J_r) is a function of the surface of the discontinuities. Most surfaces are smooth planar resulting in a value of 1.0/
 - Joint Alteration number (J_a) ranges from 2 to 3 for the weathered limestone and 0.75 to 1.0 for the unweathered limestone.

Figure A1.1 shows the likely ranges in the GSI values based on the data described above. The average values decrease from 43 to 24 as the limestone weathers. These values are low, reflecting the highly discontinuous nature of the rock mass. Although the values are not definitive, they still provide an indication of the decrease in the quality of the limestone as the degree of weathering increases during the long-term.

⁴⁸ Hoek, E. Carranza-Torres, C. & Corkum, B. (2002) Hoek-Brown failure criterion, 2002 edn. In *Mining and Tunnelling Innovation and Opportunity. Proc. 5th North American Rock Mech. Symp.* Toronto, vol. 1 pp.267-273 Uni. Toronto Press, Toronto.

⁴⁹ D varies from 0 for undisturbed rockmasses to 1 for very disturbed rockmasses.

⁵⁰ The geological strength index (GSI) produces quantitative values used to calculate the strength properties of a rockmass.

⁵¹ Cai, M., Kaiser, P.K., Uno, H., Tasaka, Y., Minami, M. 2004. Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI system. *Int. J. Rock Mech. & Min. Sci.* 41:3–19

⁵² A maximum value is not calculated as a lack of wide spacing value data produces erroneous results.

⁵³ Barton and Choubey (1977) The shear strength of rock joints in theory and practice. *Rock Mechanics* 6 183-236.

Table A1.8. Rock mass shear strength parameters for limestone.

Characteristic		Grade	Minimum	Mean
Intact rock constant, m_i .		all	9	12
Unit weight, (kN/m ³)		weathered	24.6	25.3
		unweathered	25.3	25.8
Disturbance factor, D.		all	0.5	
Uniaxial compressive strength, UCS (MPa)		weathered	27	36
		unweathered	38	65
Rock volume (cm ³)		weathered	100	2000
		unweathered	1000	4200
Joint condition factor, $J_c = J_r/J_a$		weathered	0.3	0.5
		unweathered	1.0	1.3
Geological Strength Index, GSI. (Figure A1.1)		weathered	24	33
		unweathered	38	43
Rock mass shear strength	friction angle	weathered	29°	35°
	cohesion (MPa)		0.29	0.60
	friction angle	unweathered	29°	37°
	cohesion (MPa)		0.41	1.31

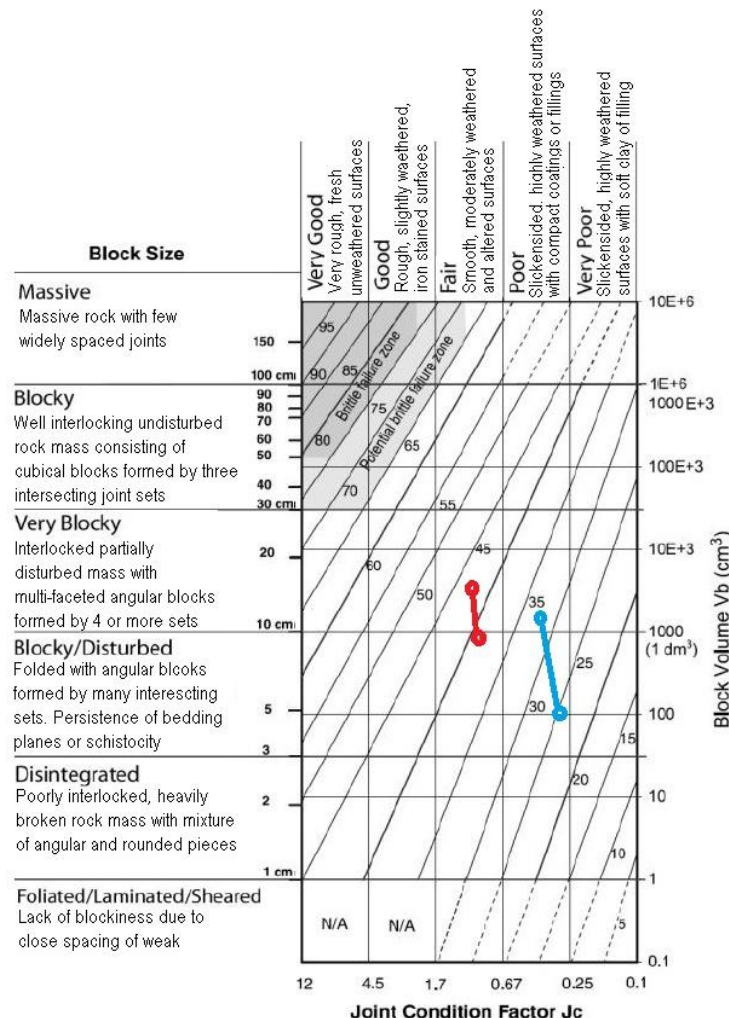


Figure A1.1. Ranges of GSI values for (red) unweathered and (blue) weathered limestone.

A1.6 Rockfill Shear Strength

Table A1.9 list the range of angles of repose of the materials exposed in the slopes of the WRD. The rocks within the dump are in various stages of decomposition.

The shear strength of the materials within the dump can be defined in terms of a friction angle. This angle is typically a few degrees higher than the angle of repose. Based on the values in Table A1.9, the friction angle likely ranges from approximately 34° to 40°. As the rocks in the WRD are generally free-draining, the values when saturated will differ little from these values when dry.

Table A1.9. Observed angle of repose and estimated friction angles of weathered rocks in the WRD.

<i>Parameter</i>	<i>minimum</i>	<i>average</i>	<i>maximum</i>
Angle of repose	32°	35°	37°
Estimated friction angle	34°	37°	40°

The friction angle of the materials within the dump can also be estimated by applying a non-linear function proposed by Barton (2008)⁵⁴. The function is defined in terms of the parameters listed in Table A1.10. The values listed in the table are lower bound values. The estimated friction angles listed in the table are similar to the values in Table A1.9. On this basis, a lower bound value of 31° is assumed for the long-term friction angle of the materials.

Table A1.10. Estimated parameters for the weathered waste rock.

<i>Parameter</i>	<i>Source</i>	<i>Value</i>
Basic friction angle, ϕ_b	A1.2	23°
Compressive strength of intact rock, UCS	A1.1	27MPa
Mean size of rocks, D_{50}		100mm
Compacted porosity, n	typical	29%
Density of materials in WRD, d	estimated	1.9t/m ²
Size dependent equivalent strength, $S = f[UCS, D50]$ (MPa)	chart	6.8MPa
Equivalent roughness, $R = f[n]$	chart	7.3
Friction angle = $f[S, R, \phi_b]$	calculated	31° to 39°

⁵⁴ Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design. N.R. Barton. Rock Dump 2008, Perth, Australia 15 pages

APPENDIX 2 PIT INTER-RAMP SCALE STABILITY

Stability analyses were undertaken to quantify the likelihoods⁵⁵ for discontinuity controlled modes of instabilities at inter-ramp scales⁵⁶ in Pit 3-4. The analyses used the stereographic method, implemented in the software Dips⁵⁷, to assess whether the kinematics⁵⁸ applicable to the benches are conducive to instabilities. The method calculates the proportions of the logged discontinuities that are conducive to masses of rocks:

- sliding from the walls along underling single (planar) or multiple (wedge) discontinuities
- or toppling from the walls bounded to the rear by single discontinuities.

Although significant limitations in the analysis method limit the veracities of the results, they are of a standard suitable for highlighting the possible dominant modes of instabilities and the relative likelihood for the modes to occur.

The assessments assumed the following:

- The six sets of discontinuities have the orientation characteristics listed in Table 1.
- An inter-ramp angle of 38° as listed in Table 2.
- The slopes dip towards the medians of the ranges of directions listed in Table 3.
- The shear strengths of the discontinuities transition from a friction angle $\phi = 32^\circ$ to $\phi = 18^\circ$ over the long-term as listed in Table A1.5.

The percentage of discontinuities having orientations conducive to sliding is defined as:

- For planar sliding and flexural and direct toppling, the percentage (P_{fp}) of the total number of poles⁵⁹ to discontinuities that are within “critical zones⁶⁰”.
- For wedge sliding and oblique toppling, the percentage (P_{fw}) of the total number of intersections between pairs of discontinuities that are within “critical zones”.

The total percentage of discontinuities having orientations conducive to sliding or toppling (P_u) is a function of P_{fp} and P_{fw} based on decision tree theory.

The likelihood for failure (P_f) is defined as $P_u \times P_l$ where P_l is the proportion of discontinuities sufficiently persistent to underlie or bound to the rear an inter-ramp scale instability.

As discussed in Section 3.3.2, discontinuities are likely to generally have medium persistences. A conservative value for P_l of 0.7 is assumed to characterise this feature i.e., 70% of discontinuities may have sufficient persistences to influence inter-ramp scale instabilities.

Figures A2.1 to A2.4 shows example of the stereonet applicable to planar and wedge sliding. The results of the analyses are listed in Tables A2.1 and A2.2 and discussed in Section 5.1.

⁵⁵ The likelihood (P_f) is not equivalent to the probability of failure (PoF). A stereonet method cannot provide a definitive assessment of PoF due to fundamental limitations in the analysis method.

⁵⁶ After the EOML, loss of one or more benches is a significant hazard. Loss of width from an individual berm is not a significant hazard.

⁵⁷ The kinematic analysis software, Dips, is produced by Rocscience Inc, Toronto.

⁵⁸ Kinematics describes the motion of rocks without considering their masses nor the forces that may cause the motion. It is primarily concerned with the orientations of discontinuities bounding rocks with respect to the orientations of slopes from which rocks may move.

⁵⁹ In a stereonet analysis, a discontinuity is assumed to be a planar feature represented on a stereonet as a “great circle”. The “pole” to a discontinuity is a downward directed line orientated perpendicular to the plane. It is represented on a stereonet as a point.

⁶⁰ “Critical zones” are areas on a stereonet within which a pole is deemed to have the orientation characteristics conducive to sliding.

Table A2.1. Results of analyses in unweathered limestone ($\phi = 32^\circ$).

Failure Mode	Wall (dip direction)			
	North (151°)	South (330°)	East (267°)	West (082°)
	Percentage of poles in critical area/s (%)			
Planar sliding, P_{fp}	0.0	0.0	0.3	0.1
Wedge sliding, P_{fw}	0.2	0.2	0.3	0.3
Flexural toppling, P_{ff}	1.9	2.2	1.5	1.5
Direct toppling, P_{fd}	3.2	6.4	5.4	5.9
Oblique toppling, P_{fo}	14.3	19.2	17.0	15.4
Total, $P_u = f(P_{fp}, P_{fw}, P_{ff}, P_{fd}, P_{fo})$	18.8	26.2	23.1	21.9
Likelihood for failure, $P_f = P_u \times P_i$	13.1	18.4	16.2	15.3

Table A2.2. Results of analyses in weathered limestone ($\phi = 18^\circ$).

Failure Mode	Wall (dip direction)			
	North (151°)	South (330°)	East (267°)	West (082°)
	Percentage of poles in critical area/s (%)			
Planar sliding, P_{fp}	0.6	0.3	2.3	1.0
Wedge sliding, P_{fw}	4.6	6.0	6.5	4.9
Flexural toppling, P_{ff}	6.1	9.8	8.0	7.6
Direct toppling, P_{fd}	3.2	6.4	5.4	5.9
Oblique toppling, P_{fo}	7.4	10.6	9.5	8.3
Total, $P_u = f(P_{fp}, P_{fw}, P_{ff}, P_{fd}, P_{fo})$	20.1	29.3	28.1	24.9
Likelihood for failure, $P_f = P_u \times P_i$	14.1	20.5	19.7	17.4

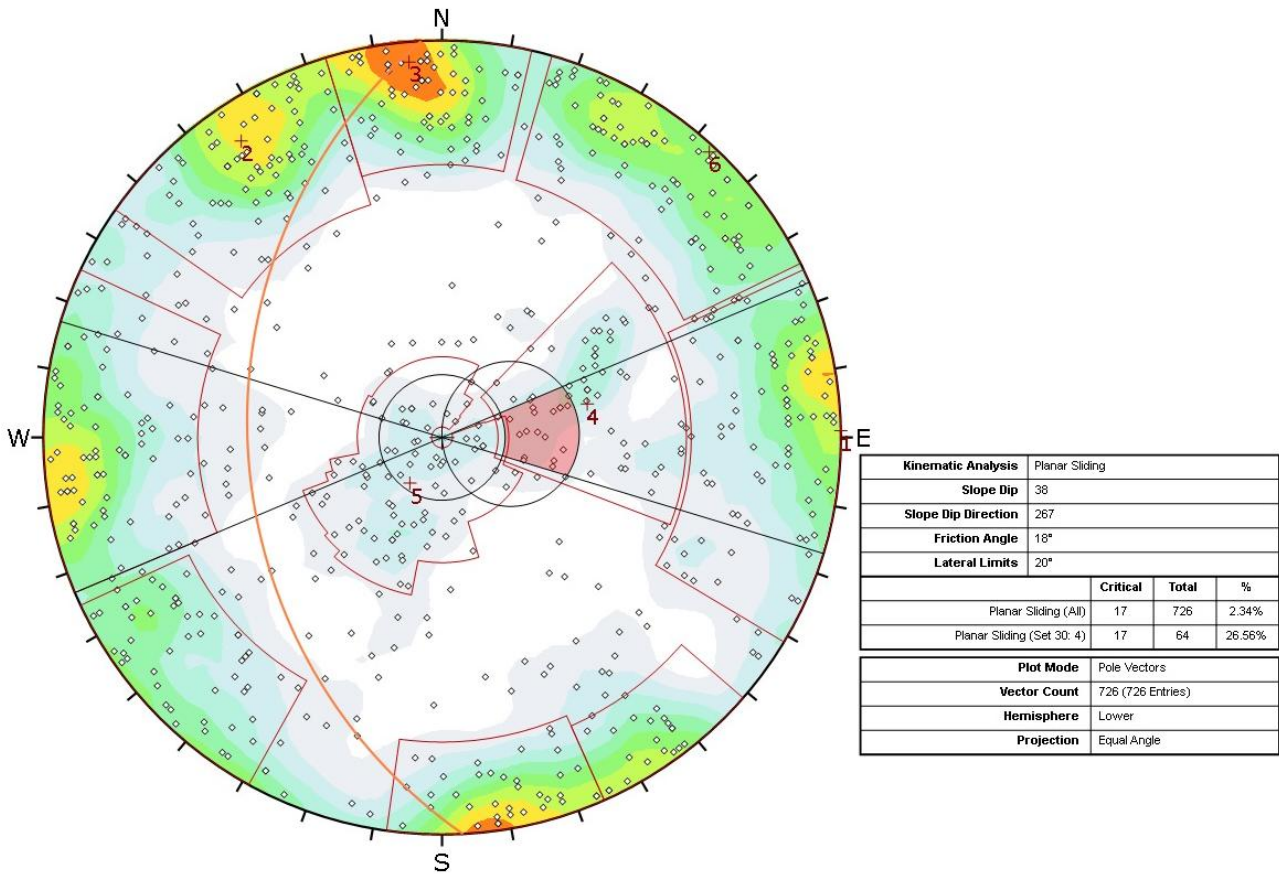


Figure A2.1. Stereonet applicable to planar sliding on the east wall during the long-term.

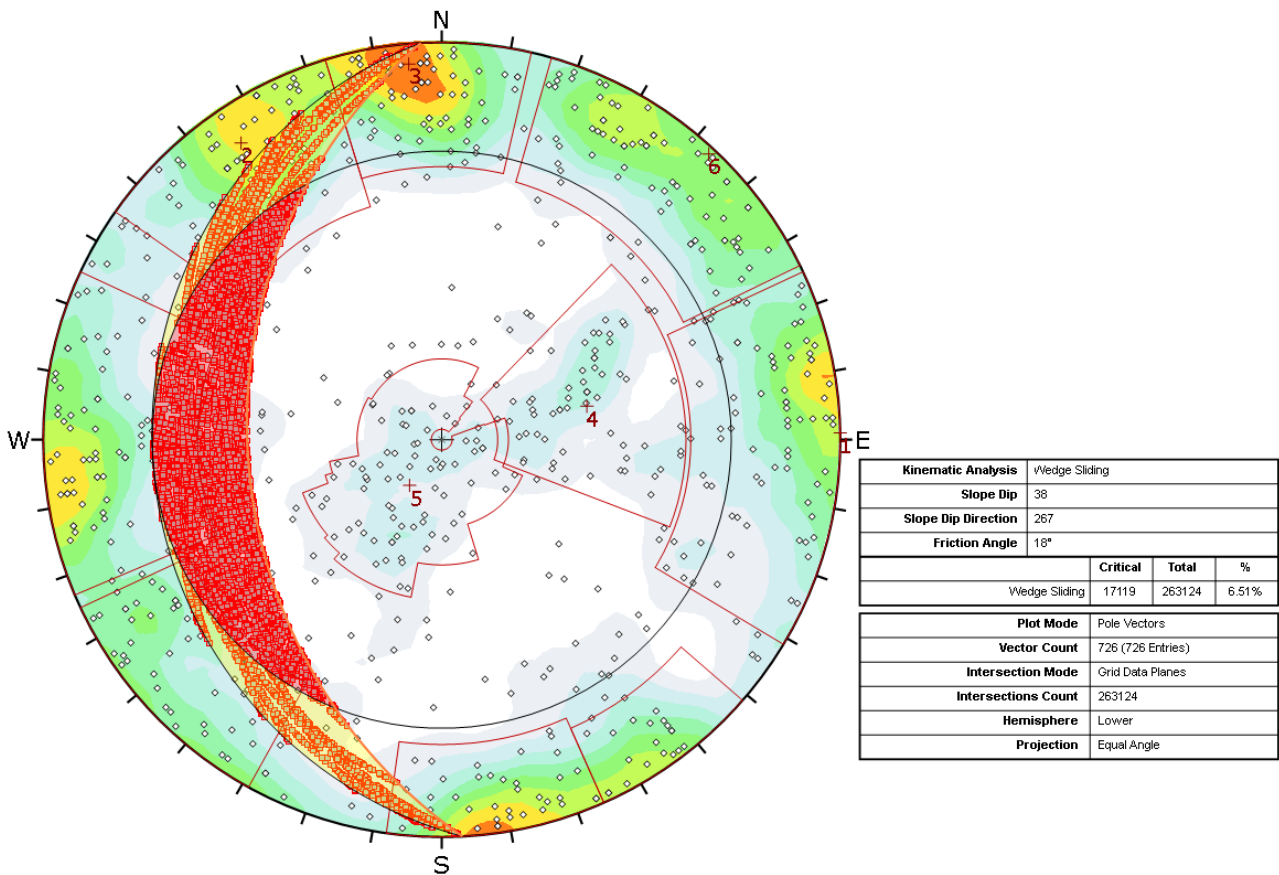


Figure A2.2. Stereonet applicable to wedge sliding on the east wall during the long-term.

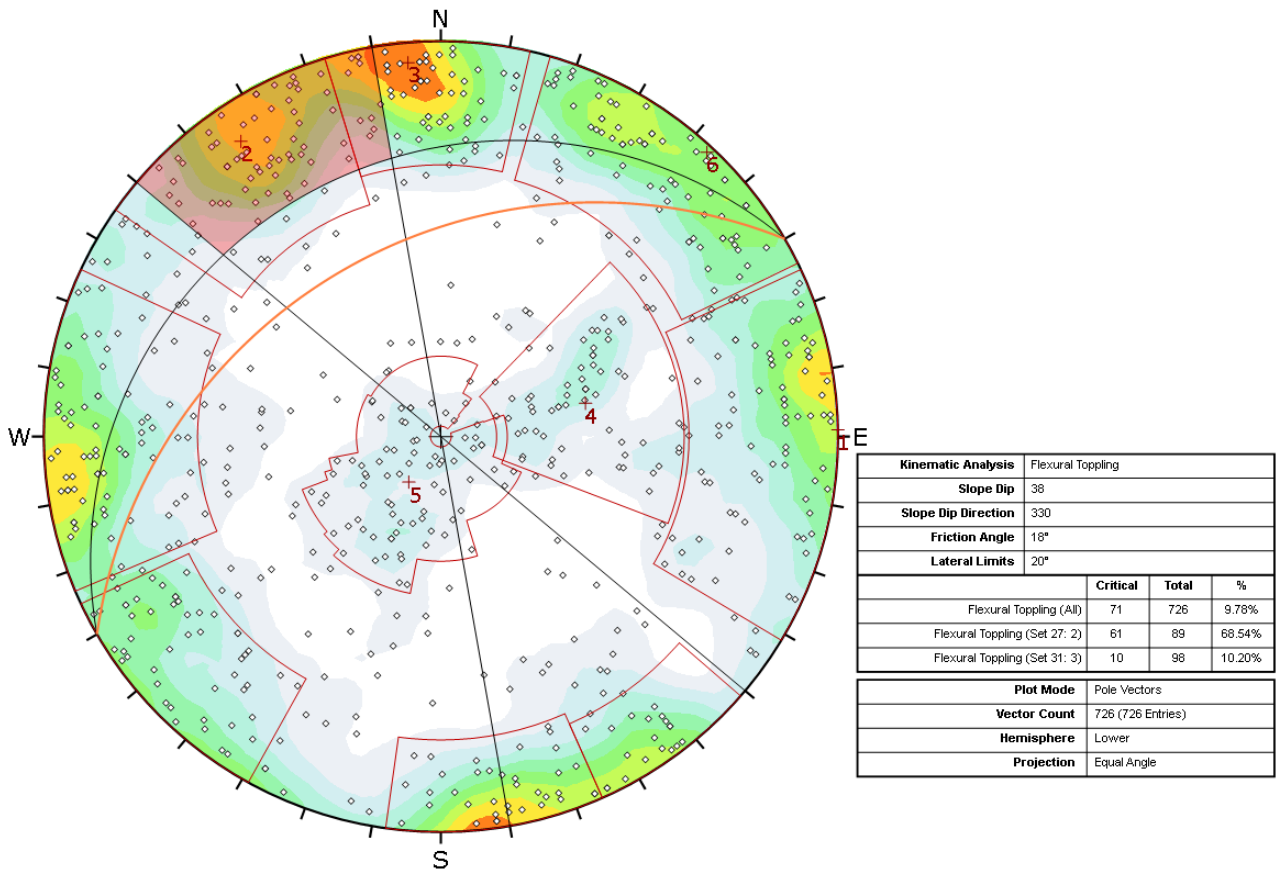


Figure A2.3. Stereonet applicable to flexural toppling on the south wall during the long-term.

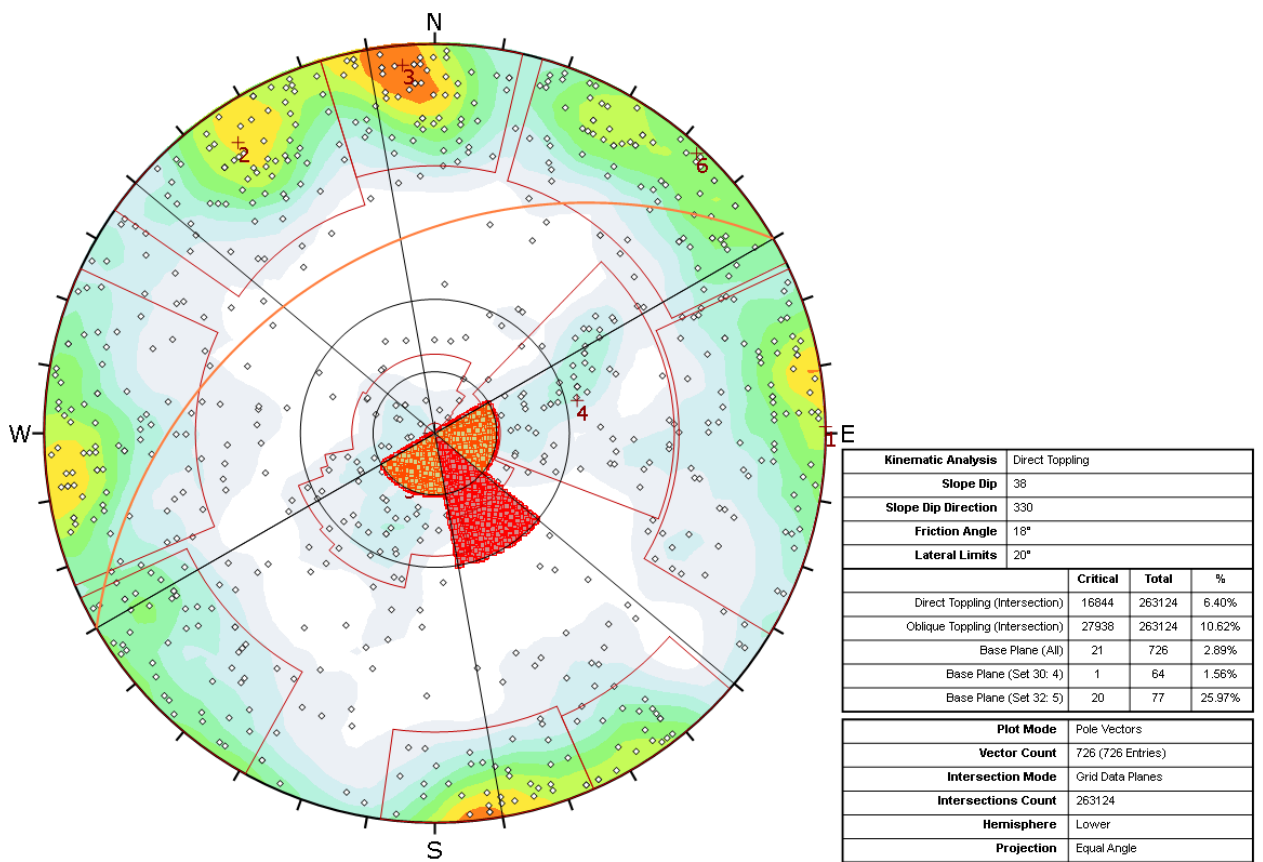


Figure A2.4. Stereonet applicable to direct and oblique block toppling on the south wall during the long-term.

APPENDIX 3 PIT WALL SCALE STABILITY

The stabilities of wall scale surfaces through the highest and steepest walls⁶¹ in both pits and through the north wall of Pit 3-4 were assessed using a limit-state equilibrium method implemented in the software *Slide2*⁶². The two-dimensional analysis code is applicable for these walls as they are wide relative to their heights and they are relatively straight.

The following characteristics were assumed:

- The walls have the specifications listed in Table 2. Figures A3.1 to A3.3 show the specifications. In both pits weathered limestone is underlain by less weathered limestone.
- The shear strength characteristics of the limestone rockmass is described by Generalised Hoek-Brown shear strength models discussed in Appendix A1.5. Lower bound values are assumed as these values become progressively more applicable over the long-term.
- An anisotropic model is applied to allow for cleavage in the limestone. The shear strength characteristics of the cleavage are applied to sections of potential sliding surfaces that dip at $90^\circ \pm 20^\circ$. This strength is described by the Barton-Bandis (B-B) discontinuity shear strength criterion defined in terms of the parameters listed in Table A1.4. Conservative values applicable to weathered limestone are assumed (i.e., $\phi = 18^\circ$).
- A PGA of 0.001g is represented as a pseudo-static horizontal force directed out of the wall. The horizontal component (A_H) of the acceleration is conservatively⁶³ estimated by $A_H = k_H \times A_e$ where $k_H = 0.45$. On this basis, $A_H = 0.0005g$.
- Two groundwater scenarios are considered for both pits: no water and water at 40 mRL, the level of the spillway on the dam wall.

The results of the analyses are quantified in terms of Factor of Safety (FoS) values for each of hundreds of surfaces assessed within the walls. The surfaces with the lowest capacities are highlighted in Figures A3.3 to A3.7. The results of the analyses are discussed in Section 6.3.

⁶¹ The east wall is not considered as its height is insignificant. The west wall is not considered as its performance will be an average of the performances of the north and south walls.

⁶² *Slide2 Ver 9.02* is developed by RocScience Inc. Toronto, Ontario.

⁶³ Melo and Sharma (2004) suggest (*Seismic Coefficients for pseudo-static Slope Analysis*, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada August 1-6, 2004) that k_H values ranging from 40% to 45% of A_e should be used in slope stability design rather than the 50% as proposed by Hynes-Griffin and Franklin (*Rationalizing the seismic coefficient method*. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi, 1984, Miscellaneous Paper GL-84-13).

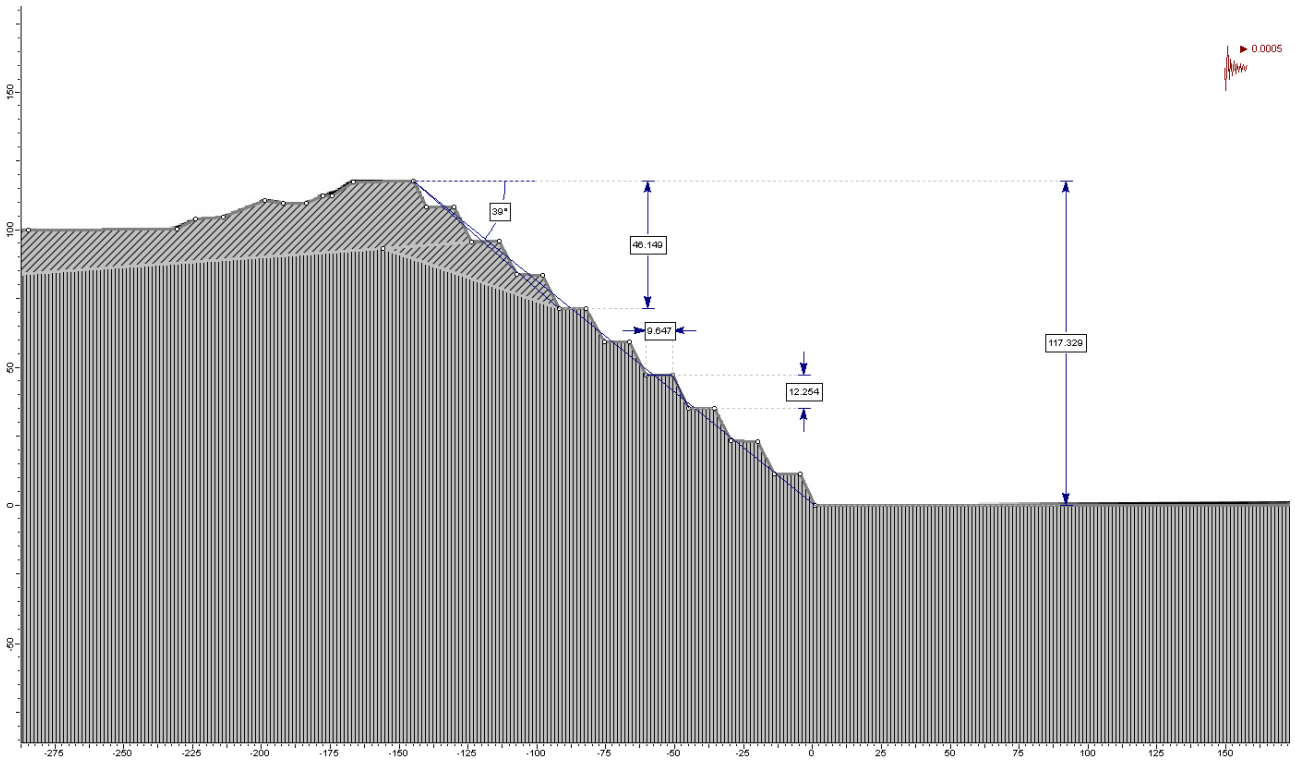


Figure A3.1. Cross-section through the west wall of Pit 3-4.

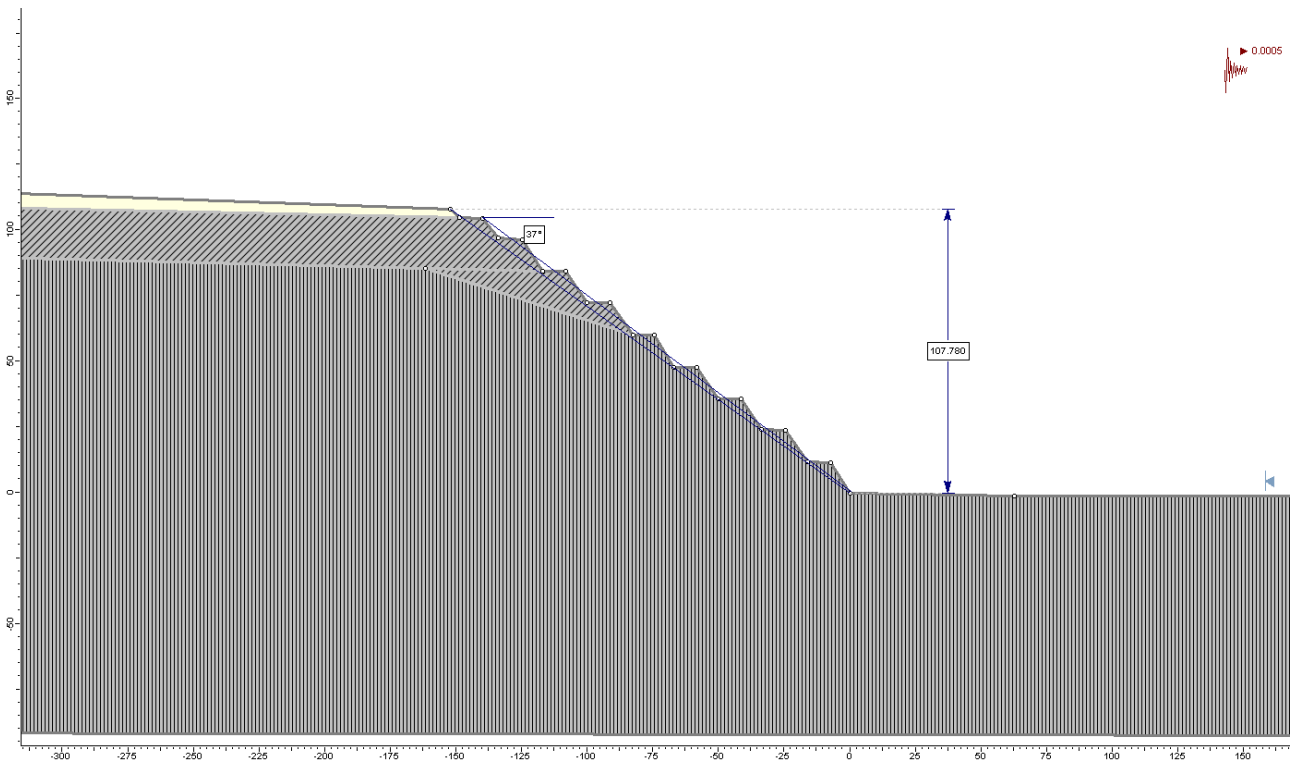


Figure A3.2. Cross-section through the north wall of Pit 3-4.

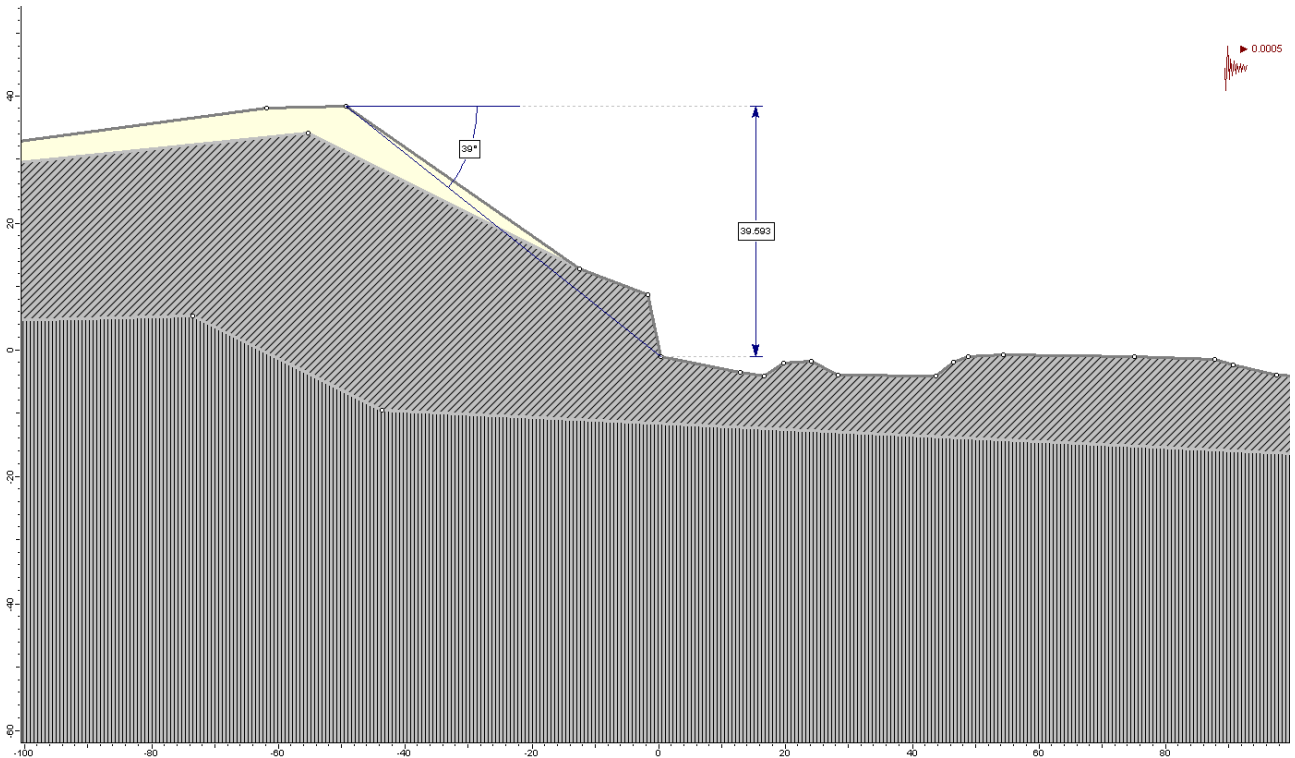


Figure A3.3. Cross section through the west wall of Pit 2.

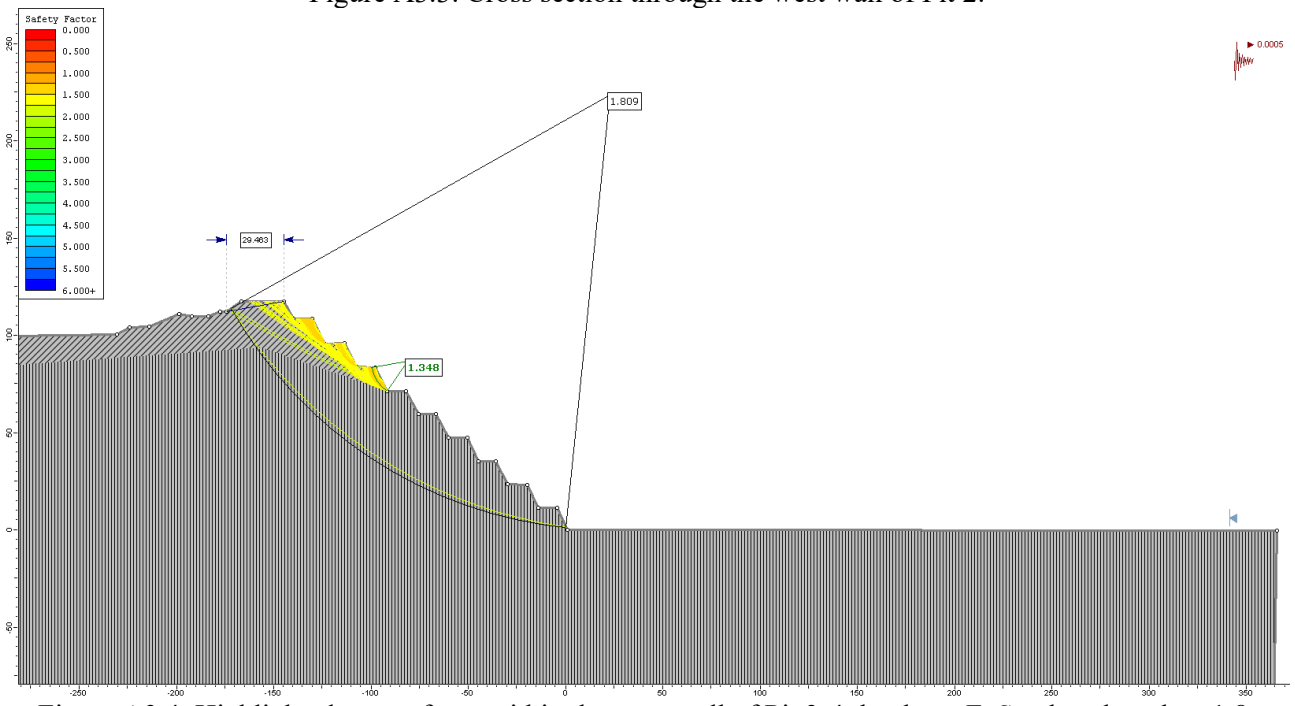


Figure A3.4. Highlighted are surfaces within the west wall of Pit 3-4 that have FoS values less than 1.8.

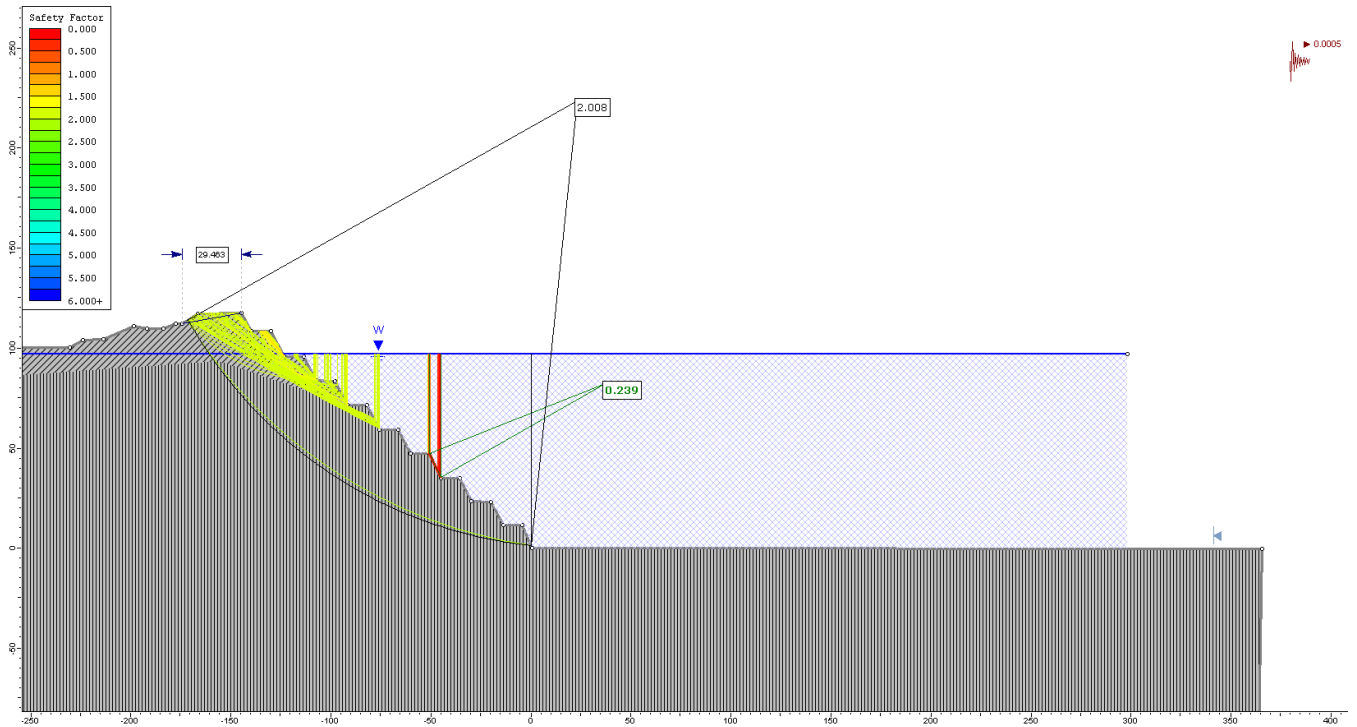


Figure A3.5. Highlighted are surfaces within the west wall of Pit 3-4 as a lake having FoS values less than 2.0.

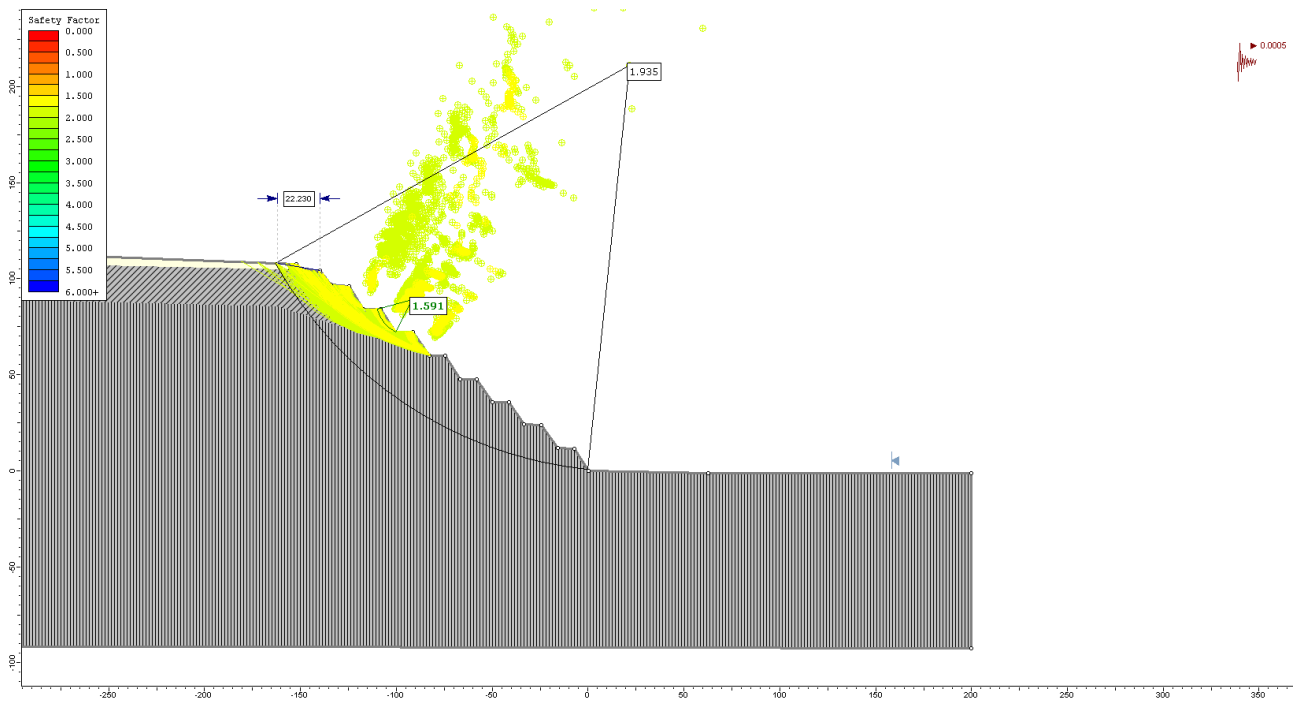


Figure A3.6. Highlighted are surfaces within the north wall of Pit 3-4 having FoS values less than 1.9.

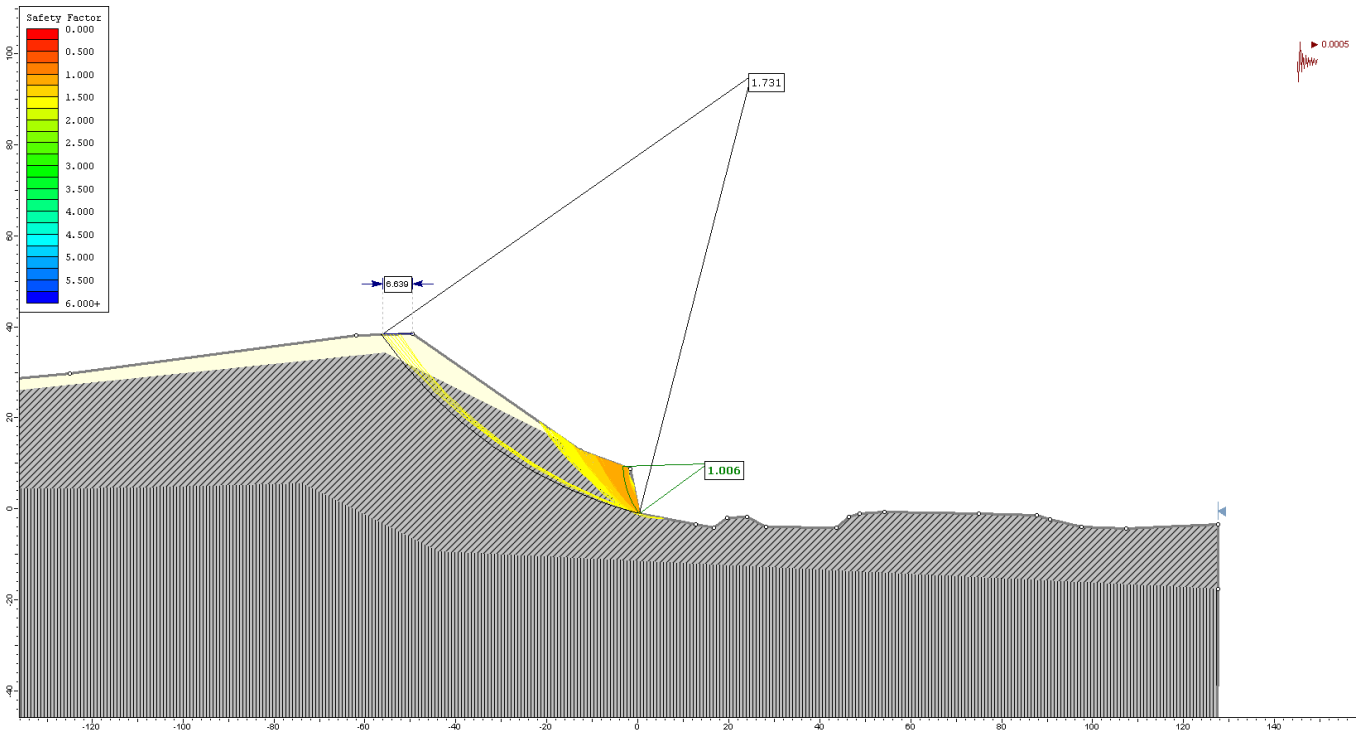


Figure 3.7. Highlighted are surfaces within the west wall of Pit 2 that have FoS values less than 1.7.

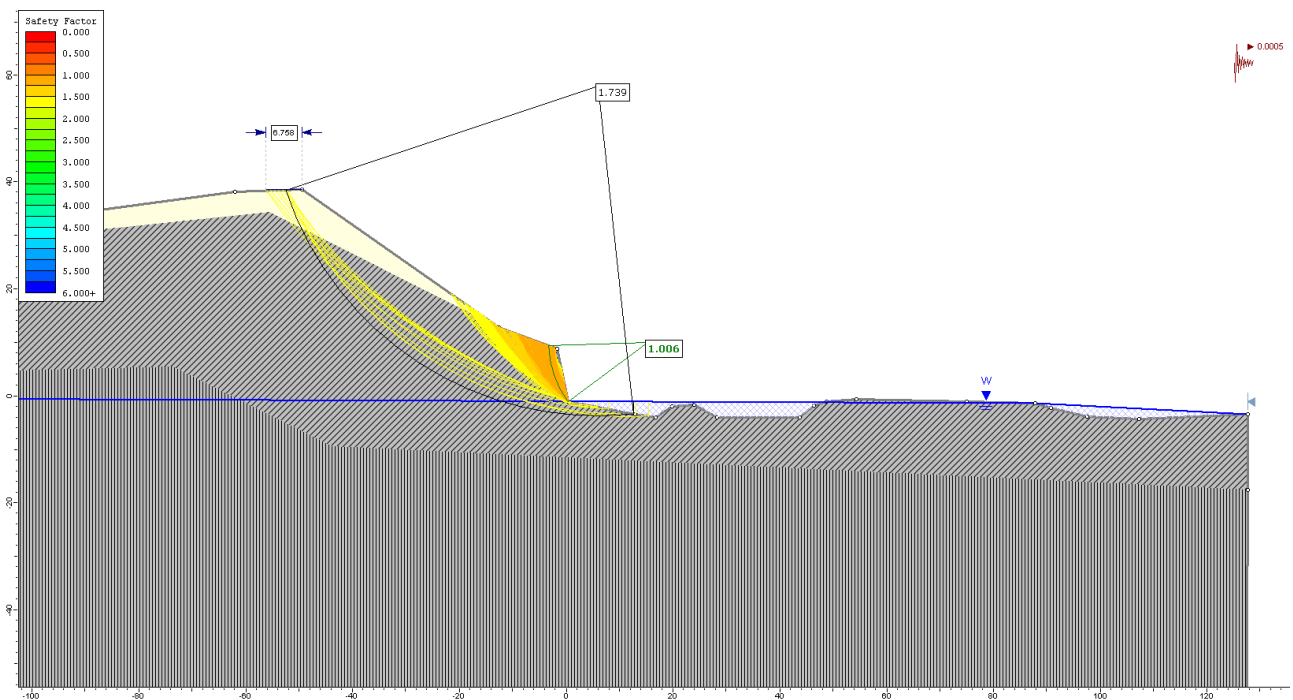


Figure A3.8. Highlighted are surfaces within the west wall of Pit 3-4 with water at floor level having FoS values less than 1.74.

APPENDIX 4 WRD SLOPE STABILITY

The stabilities of surfaces through the highest slope of the WRD was assessed using the limit-state equilibrium method implemented in *Slide2*.

The following characteristics were assumed:

- The slope, shown in Figure A4.1, has the dimensions listed in Table 8.
- The shear strength characteristics of all materials are described using a linear Mohr-Coulomb model defined in terms of a friction angle and a cohesion:
 - The footings comprise clayey silt defined in terms of a friction angle of 25° and a cohesion of 45 kPa, as discussed in Section 3.2.1.
 - The materials within the dump are defined in terms of a conservative 25° friction angle, as discussed in Section A1.6 in Appendix 1. This value is likely to be appropriate during the long-term.
- As evidenced by observation, the materials within the dump are assumed to be permeable. Water that infiltrates the surfaces and does not subsequently evaporate, seeps from the down contour toe. Excess hydrostatic pressures do not develop within the materials.
- Earthquake loading applies an acceleration of 0.0005, as discussed in Appendix 3.

The results of the analyses are quantified in terms of Factor of Safety (FoS) values for each of hundreds of surfaces assessed within the slope. The surfaces with the lowest capacities are highlighted in Figures A4.2. The results of the analyses are discussed in Section 7.1.

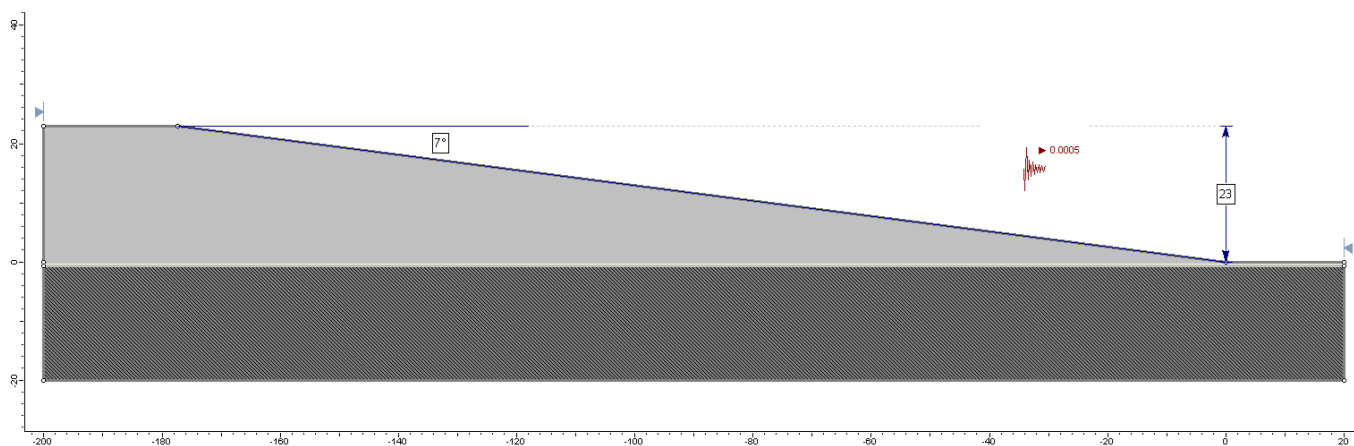


Figure A4.1. Cross-section through the highest slope of the WRD.

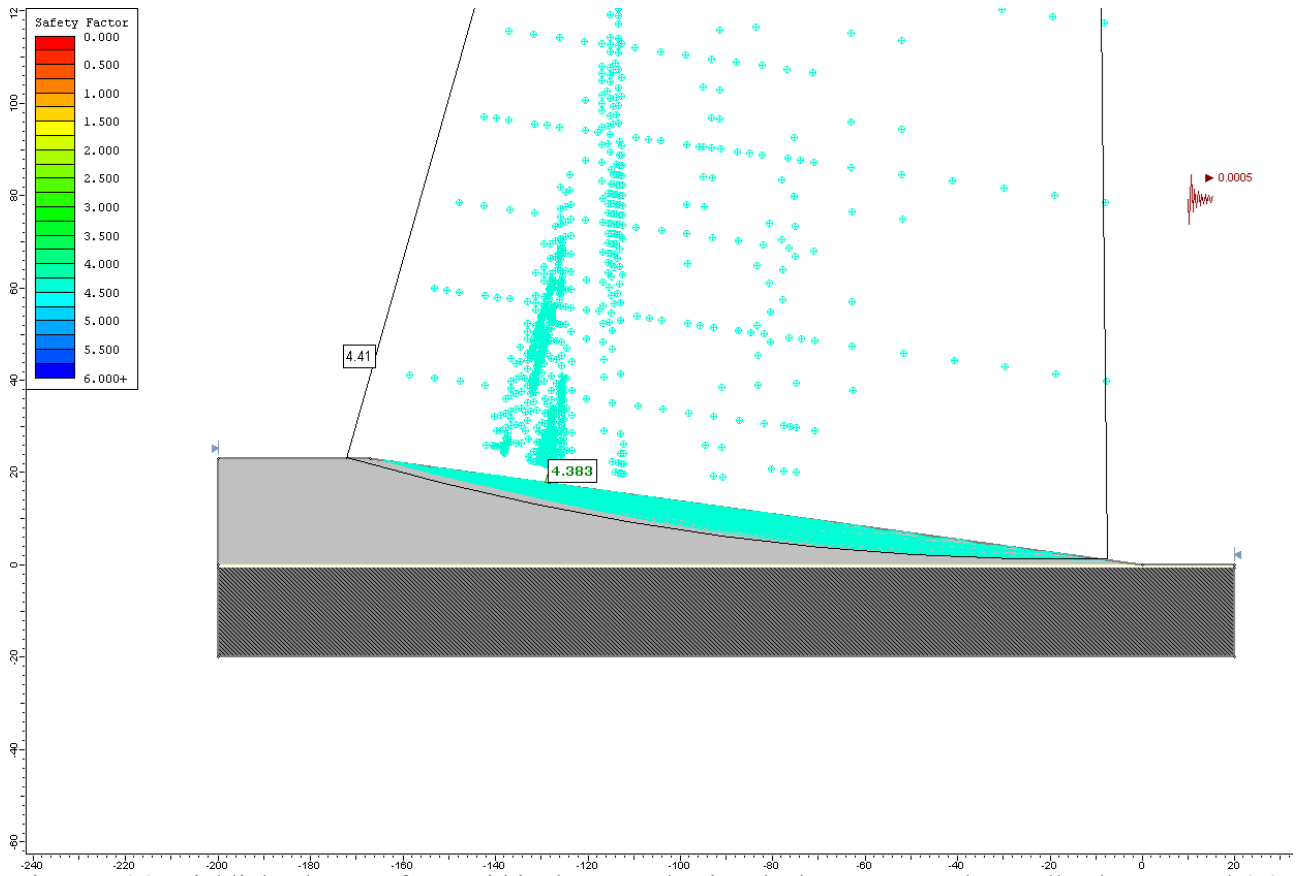


Figure A4.2. Highlighted are surfaces within the WRD having the lowest FoS values. All values exceed 4.4.

APPENDIX 5 BENCH SCALE ROCKFALL ANALYSES

A series of rockfall simulations⁶⁴ were undertaken to inform an understanding of the trajectories of rocks that release from and traverse down a final batters in Pit 3-4 and the likelihoods for these rocks to traverse over the crest of the berm. The simulations used the rigid block method as implemented in the software *Rocfall2*⁶⁵. This two-dimensional analysis code is applicable as no evidence has been observed indicating that falling rocks traverse significant distances in the third dimension i.e. along strike of a batter.

- Figure 23 is the cross section of the bench. It is 12 m high with a 65° batter angle. The berm is 10 m wide and contains a 1 m high edge bund. These values are consistent with the values listed in Table 2.
- Table A5.1 lists the values of the parameters assumed for the rebound characteristics of the batter and the berm⁶⁶. The batter is assumed to comprise exposed rocks; the berm to comprise fines and rockfall debris.
- Tables A5.2 and A5.3 summarise the sizes and shapes assumed for the rocks based on observation (Figure A5.1). The shapes are typically rounded and non-rounded blocks (i.e., uniform side lengths) and rounded and non-rounded rectangles (i.e., non-uniform side lengths).

The source for the rocks is assumed to be the crest; rockfalls being triggered by overland flows. This assumption is conservative as rocks typically fall from anywhere on a batter. A simulation involved generating a thousand rocks.

Figure 23 shows the trajectories of 1000 rocks generated. None of the rocks traverses beyond the edge bund. The combination of a 10 m wide berm and 1 m high edge bund is likely to be sufficient to terminate the trajectories of rocks that detach from the adjacent batters.

Table A5.1. Normal distribution parameters assumed for the batter.

<i>Parameter</i>	<i>Normal restitution</i>	<i>Tangential restitution</i>	<i>Dynamic friction</i>	<i>Rolling friction</i>
Mean	0.45	0.85	0.75	0.40
Standard deviation	0.04	0.04	0.04	0.04
Relative minimum	0.12	0.12	0.12	0.12
Relative maximum	0.12	0.12	0.12	0.12

⁶⁴ The simulations used the software Rocfall2.

⁶⁵ The proprietary software Rockfall 2 Version 8.024 is developed by Rocscience.

⁶⁶ Some values of the parameters listed in Table A5.1 are different to the default values suggested by Rocscience. The four mean values "suggested" by Rocscience for the normal and tangential coefficients of restitution (0.35 and 0.85) and the coefficients of dynamic and rolling friction (0.75 and 0.40) are default values. Some, or all, values rarely reflect the actual values at any site as the geology, surface characteristics and profile and weathering characteristics differ between sites. Rocscience provides tables of values that have been used by other practitioners. In these tables, the ranges of coefficients and the mean value assumed in the report are approximately those as follows: normal restitution: 0.25 to 0.60 (0.45), tangential restitution: 0.55 to 0.95 (0.85), dynamic friction: 0.5-inadequate data (0.75) and rolling friction: 0.4 to 0.9 (0.4). The values used in the current study are within the ranges applied by other practitioners. The upper bound value for dynamic friction seeks to quantify the significant braking effect the roughness of the weathered surface and rocks have on sliding. The values assumed in this study are deemed appropriate based on observations of the trajectories of rocks that have fallen within the pits at this site. The end locations of the rocks in the simulations are acceptably similar to the actual end locations of the rocks that have ended up on the berms, ramps and pit floors, which provides evidence that the values assumed for the parameters are "in the ballpark".

Table A5.2. Normal distribution parameters assumed for the berm.

<i>Parameter</i>	<i>Normal restitution</i>	<i>Tangential restitution</i>	<i>Dynamic friction</i>	<i>Rolling friction</i>
Mean	0.32	0.80	0.50	0.30
Standard deviation	0.04	0.04	0.04	0.04
Relative minimum	0.12	0.12	0.12	0.12
Relative maximum	0.12	0.12	0.12	0.12

Table A5.2. Parameters assumed for the rocks.

<i>Parameter</i>	<i>Edge length (m)</i>	<i>Volume (cc)</i>	<i>Mass (kg)</i>	<i>Density (kg/m³)</i>
Mean	0.20	7,990	21.1	2,650
Standard deviation	0.05	3,560	9.4	50
Relative minimum	0.19	7,391	19.5	200
Relative maximum	0.80	22,230	56.7	50

Table A5.3. Shapes assumed for the rocks.

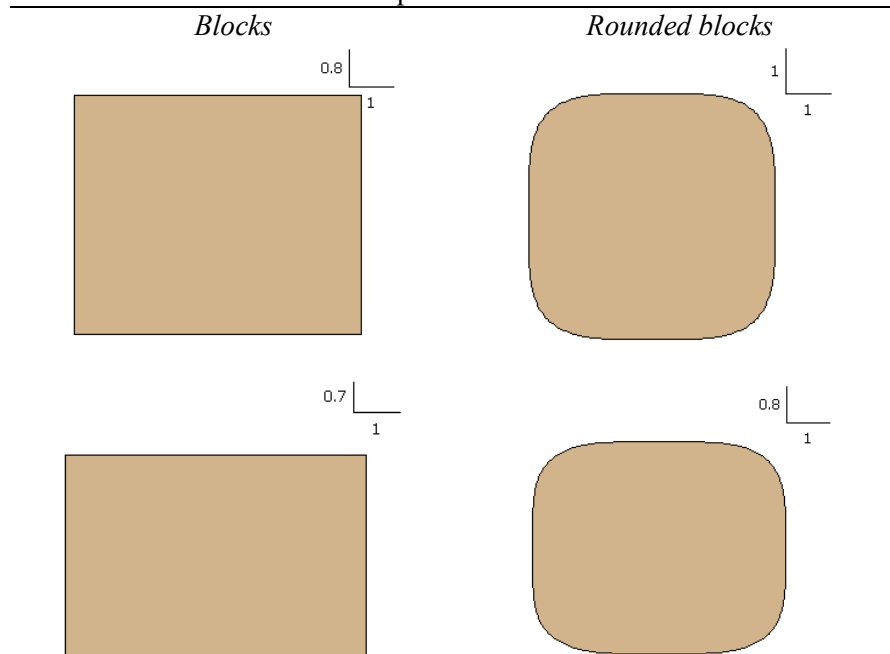




Figure A5.1. Typical muckpile highlighting the range of rock shapes and sizes generally exposed on the benches. The sizes and shapes generated in the simulation are consistent with these observations.