Appendix 21.

SLR Consulting Australia (2020l) *Rum Jungle Rehabilitation – Stage 2A Detailed Design – Main Pit Backfill Strategy, Geotechnical Considerations*. Report to the Department of Mines and Energy, Northern Territory. PART A.

NORTHERN TERRITORY GOVERNMENT

Department of **Primary Industry and Resources**

RUM JUNGLE REHABILITATION - STAGE 2A
DETAILED DESIGN

Main Pit Backfill Strategy
Geotechnical Considerations
Issued for Client and External Peer Review

Prepared for:

NT DPIR - Mines Division
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BASIS OF REPORT

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DISCLAIMER

The following Main Pit Backfill Strategy has been based on limited available data at the time of design. A reference backfill strategy design involving barge subaqueous backfilling has been prepared. A suitably qualified Contractor will be required to backfill the Main Pit with waste rock materials and cap to the satisfaction of Rum Jungle remediation objectives.

EXECUTIVE SUMMARY

Scope / Objectives

This report provides a remediation strategy and preferred design option for the Stage 2A Detailed Engineering Design for the rehabilitation of the Main Pit at the former Rum Jungle mine in the Northern Territory.

This report addresses the geotechnical considerations for the backfill methodology. Geochemical considerations have been addressed by others [1], and have been subsequently incorporated with this report into an overall backfill design, which is outlined in SLRs Construction Methodology, Technical Specifications and Design Drawings that accompany this report.

Full details over the overarching rehabilitation strategy for Rum Jungle are outlined in the Draft Environmental Impact Statement [2]. SLR has undertaken the design for the rehabilitation and this is presented in SLRs Detailed Engineering Design Report [3]. It is recommended that this report be read in conjunction with those references.

Assessment Process

A review of the site history and a summary of previous reports and investigations is documented with assessment of in-situ testing and laboratory test results to define the ground conditions and geotechnical parameters for backfilling design. A conceptual model of the mine ground conditions is presented along with ground engineering characteristics of existing soil and rock materials and proposed backfilling materials (sand bedding layers and waste rock backfill).

Options for backfilling of the Main Pit are discussed and a preferred approach is outlined along with methodology, construction sequence, program and monitoring requirements for placement of backfilling materials.

Backfilling Strategy and Preferred Solution

Various backfilling options have been assessed, considering site-specific constraints such as disturbance of the contaminated basal waters (chemocline), soft tailings deposits, treatment of displaced pit lake water and available backfilling materials and placement processes. A reference backfill design methodology involving barge backfilling has been prepared. The preferred backfilling strategy may be summarised as follows:

- Geochemical precautions are required to manage the risk of disturbing the existing lens of contaminated basal pond water (chemocline) which is estimated as 3-4m thick, based on the last profile done in 2014. Risk management measures include preliminary field scale sand placement trials with water quality testing, addition of granular lime to backfill as an acid mitigation measure, aerially broadcast neutralant.
- To reduce the risk of disturbing sediments and chemocline, a layered backfill profile is desirable consisting initially of sand bedding carefully placed over the existing chemocline and soft tailings. Options involving the use of a geofabric separation layer have been considered and dismissed. As successive backfill layers are placed, the thickness and grading of subsequent fill layers can increase to increase the productivity of backfilling operations. The geometry of backfilling (slopes and lift

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heights) would be carefully controlled and validated by survey to maintain stability of the placed materials.

- Similar to the proposed overwater dumping approach in RGC (2016) report, a suitable borrow and placement technique would utilise bulk (dry) excavation of sand borrow material, processing (screening), stockpiling, then fluidisation by end tipping into receivals bin, pumping using a wet placement process through floating line and discharge from near water surface via spreader pontoon and diffuser. An alternative to this would be to utilise a dry transport process involving land and floating conveyor systems instead of a slurry filled pipeline to transport material to a spreader pontoon (barge) for controlled sub-aqueous placement.
- The rate of placement is effectively governed by the capacity of the water treatment plant which extracts and treats pit lake water as it is displaced by capping and backfill. The maximum water treatment (and therefore fill placement) is about 80-90L/s. The resulting backfilling process is programmed to take about 26 months.
- Target filling levels are such that that Potentially Acid Forming (PAF) waste rock must remain submerged under the lowest expected dry season water level of RL 59 m AHD. Adding the nominal 2m thickness of inert (non-PAF) surface capping layer material, results in a target finished surface level of RL 58 m AHD. The surface level design recognises that post construction total and differential settlements will take place, resulting shallow ponding over much of the site.
- Construction stage monitoring includes chemical testing of pit lake waters to confirm that chemocline disturbance is managed to achieve acceptable groundwater quality levels, regular multibeam survey (intense initially) to confirm backfilling placement coverage, check for instability and mud-waving. Periodic CPT validation testing will also be undertaken to confirm settlement of the capping/tailing interface and confirm tailings strength and settlement behaviour at critical staging.

During placement, it will be necessary to undertake construction quality assurance (CQA) testing to confirm design assumptions regarding the density, volume, stability and settlement of backfill materials and associated geochemical conditions.

Design Considerations

Development of the preferred sub-aqueous backfilling strategy has taken into consideration the following design issues which are documented herein:

- **Chemocline Disturbance** – A range of measures to prevent partial or complete mixing of the chemocline lens which have been considered, which would otherwise have adverse consequences for water quality in the bulk of the pit. Assessed options include preliminary decanting, initially raining in fine-grained sand fill, using aerially broadcast neutralant and dosing backfill with crushed limestone reagent. The broadcast neautralant option would need to be done as early as possible commencing prior to backfilling to stabilise the chemocline prior to sand placement. Water treatment design is reported separately (Ref. WTP Design Report 680.10421.90060-R01-v1.0).
- **Availability and suitability of Backfilling materials** suitability– Investigations, testing, borrow and mass haul design has been undertaken to identify suitable sand borrow material (placed as bedding layer material), PAF waste rock material (bulk backfilling) and inert waste rock capping

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material. Design considerations include: stability, separation and filtration behaviour, advection storage capacity, compressibility and bulking, handling and placement techniques.

- **Pit Wall Stability** – Characterisation of pit rim and pit wall materials has been undertaken to develop a conceptual ground model and assess material properties for subsequent stability analyses. Slope stability models have been undertaken for appropriate temporary conditions (during backfilling) and where permanent slopes remain after backfilling. Additionally, risk assessments have been completed considering risk to property and risk to life for construction operations which may be affected by slope stability risk. Risk controls are presented, such as minimum crest offset for construction operations, and suitable slope angles for design of temporary and permanent slope in the adopted geotechnical units.
- **Tailings Stability** – The type, distribution and engineering properties of the in-situ tailings (also called slimes) and related slope-wash and backfill soils which exist in some perimeter areas have been assessed based on a review of historical investigations. Geotechnical units have been developed and strength and compressibility properties have been assigned based on laboratory and field testing results. Behaviour of the very soft and normally consolidated soft, cohesive tailings under backfilling has been assessed to develop placement strategies and controls which maintain suitable factors of safety against bearing and slope failure.
- **Settlement Summary** – The settlement behaviour of existing tailings and overlying soil and beach deposits have been assessed during and after backfilling operations. Consideration of time-dependant consolidation and creep behaviour of cohesive in-pit materials and introduced waste rock has been assessed to predict long-term settlement behaviour and assist in the refinement of borrow demand for backfill materials.
- **Acid Rock** - Target filling levels are such that that PAF waste rock will remain submerged under the lowest expected dry season water level of RL 59 m AHD. Adding the nominal 2m thickness of inert (non-PAF) surface capping layer material, results in a target landform surface level of RL 58m AHD on completion. Post-backfilling settlements in the order of 3.5 to 4m are expected over 100 years)
- **Final Landform** – Final Landform adhering to seasonal Main Pit water levels and ensuring constant water coverage over backfilled materials as well as final Main Pit crest shape and batters allowing the flow of the Eastern Finniss River, revegetation of crest and battered access points facilitating access and mitigating erosional forces.
- **Monitoring requirements -** will be incorporated into the earthworks specifications and Instrumentation and Monitoring Plan as follows:
  - Regular chemical testing of pit lake water and chemocline to manage disturbance risk
  - Bench scale and field scale trials to validate sedimentation placement processes
  - Grading tests to check the conformity of bedding layers and verify screening elimination control of oversize particles
  - Progressive CPT testing to assess material thickness, density, segregation effects, tailings strength gain and/or excess porewater pressures (to confirm consolidation and settlement behaviour)
  - Regular survey to confirm conformance to placement stability ‘rules’ including limiting the height difference and slope angle of backfill materials at successive stages of backfilling.

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Limited Information

The Main Pit backfill strategy has been developed using limited available data. The reader is made aware, assessment of in-situ conditions has been derived from investigations that may not constitute traditional level of detail required for detailed design. The following limitations are acknowledged:

Investigation	Limitations	SLR Acknowledgement
CPTs within Main Pit tailings	CPTs do not penetrate entire tailings layer.	CPT profiles extrapolated to base of tailings based on existing data. Base of tailings based on literature references.
Tailings Characteristics	No in-situ testing performed in CPTs (shear vane, dissipation or similar)	Characteristics of tailings based off CPT profiles and cross checked against known tailings behaviour (academic literature and experience). Tailings parameter sensitivities and screening also performed for strength and liquefaction assessments.
Main Pit Side Walls	Limited boreholes and information on structural aspects of Main Pit geology	Characteristics based off available data (3 x boreholes), qualitative observations and model sensitivity analysis.
Chemocline Characteristics	No recent information on extents or density of layer.	The design allows for the chemocline layer as observed in the most recent studies.
Main Pit Geometry	No recent survey.	The design has been based on the most recent, Main Pit 2014 bathymetry survey.

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1 Introduction

As part of the Stage 2A Detailed Engineering Design of the rehabilitation of the former Rum Jungle mine remediation options for the Main Pit require assessment. The purpose of this report is it provide a conceptual model of the Main Pit (hereafter referred to as the "site") to identify the various elements which require consideration in order to inform a remediation strategy and to develop a backfill design methodology.

This report addresses the geotechnical considerations for the backfill methodology. Geochemical considerations have been addressed by others [1], and have been subsequently incorporated with this report into an overall backfill design, which is outlined in SLRs Construction Methodology, Technical Specifications and Design Drawings that accompany this report.

Full details over the overarching rehabilitation strategy for Rum Jungle are outlined in the Draft Environmental Impact Statement [2]. SLR has undertaken the design for the rehabilitation and this is presented in SLRs Detailed Engineering Design Report [3]. It is recommended that this report be read in conjunction with those references.

1.1 Objectives

At the end of its mine life, the Main Pit was reported to be approximately 105 m deep with a base at RL -35m. Following rehabilitation works in the 1980s, uranium tailings were disposed of within the pit to an elevation of +14.18m RL. Groundwater has naturally stabilised within the Main Pit at levels ranging from +58.95m RL (Minimum dry season level) to RL +61.59m (Maximum wet season level). An impacted water layer, or chemocline, is present from RL 22.0m (as per 2008 monitoring data).

The aim of the remediation strategy is to allow the:

1 Backfill of the Main Pit (also referred to here as the Pit or the site) with the highest-grade potentially acid forming (PAF) materials currently stored in various waste rock dumps (WRD) across Rum Jungle mine site. The backfilling strategy is to be aimed at:

   a. Preventing bearing capacity failure of the in-situ tailings (i.e. ensure that placement of fill on tailings do not cause the tailings to displace, which can mobilise further contaminants);

   b. Minimising disturbance of the chemocline layer, which can mobilise further contaminants;

   c. Mitigating and minimising slope instability risk inherent within the Pit walls during and following construction;

   d. Backfilling at rates conducive to the treatment of displaced Main Pit water by Water Treatment Plant (WTP);

   e. Ensuring the safety of people and equipment during construction.

2 Development of a long-term shallow pit lake. To allow this backfill approach should allow for:

   f. PAF placed to a final level no greater than RL 56.0m AHD (after allowing for potential settlement of the in-situ tailings and placed backfill)

   g. Finished capping level placed to a level no greater than RL 58m AHD;

3 Realignment of East Branch Finnis River (EBFR) back to its original course. (out of scope of this report).

4 Stabilisation and amelioration of the Pit crest and upper batters to a landform suitable for revegetation and safe future access.

1.2 Performance Requirements and Design Criteria

Key performance requirements applicable to the preferred rehabilitation strategy for the wider Rum Jungle site are set out in Section 4.4 - *Scope of Works)* [4], with specifics to the Main Pit backfilling provided in Section 4.4.2.1 – *Pit Backfilling Strategy.* The objectives are generally qualitative and are summarised below:

- Backfill the Main Pit with the most sulphatic rock possible; and
- Re-evaluate any preliminarily proposed backfilling methodologies based on further information and evaluate any alternatives, inclusive of:
    - Consideration to costs and impacts to the wider referred rehabilitation strategy.

Quantitative performance requirements and design criteria have been developed in conjunction with previous Main Pit backfill proposed strategies including:

i. Technical specifications - O'Kane's Consultants: *Rehabilitation of the Former Rum Jungle Mine: Stage 2 Works Specification, 2016* [5];

ii. Conceptual alternative approaches - Robertson GeoConsultants (RGC): *Main Pit Backfilling Concept Approaches, 2016* [6]; and

iii. Completion Criteria, Completion and Framework Presentation – Department of Mines and Energy, Northern Territory Government, 2016 [4].

The overarching performance requirements as defined by SLR are summarised below:

- Safe and acceptable stability, considering short-term and long-term for backfilling and final landform with additional long term seismic considerations – Sections 7 and 9.
- Serviceability performance relating to target filling levels to aid in the re-alignment of the East Branch of the Finniss River (EBFR) within the Main Pit's footprint – Section 9;
- Technically feasible, cost effective methodology which adheres to anticipated wider project timelines with a high probability of success – Section 9;
- Environmental criteria [4]:
    - safe for people, flora and fauna (short and long-term);
    - chemically, radiologically and physically stable (short and long-term); and
    - significantly reduces acid and metalliferous (AMD) contaminant loads and concentrations travelling beyond the mine boundaries through placement of PAF waste rock below groundwater level.
- Culturally appropriate including the protection and preservation of Aboriginal cultural heritage;
- Design Life in keeping with the wider Rum Jungle remediation specifications.

1.3 Deliverables

The proposed strategy and design deliverables are considered to be suitable for construction, and include:

- This detailed design report;
- Construction methodology report;
- Technical Specifications outlining placement strategies including, but not limited to:

    - Maximum placement rates for the various backfill materials;
    - Placement methods;
    - Instrumentation and monitoring; and
    - A trigger-action-response Plan (TARP)
- Issued for Construction Drawings detailing:
    - Suitable laydown areas;
    - Anchor points;
    - Stockpiling areas;
    - Access to the Pit;
    - Backfill Staging and Stability Requirements; and
    - Final Landforms.

2 Outline of Proposed Backfill Strategy

The conceptual backfill strategy developed by SLR in consultation with DPIR, aimed at meeting the objectives in **Section 1.1**, includes:

- Sub-aqueous (i.e. placement below water) placement of a bedding layer to facilitate the backfilling of waste rock materials and prevent the remobilisation of contaminate materials within the tailings and chemocline;
- Sub-aqueous Placement of waste rock with priority to higher risk potentially acid forming (PAF) waste rock materials located within Intermediate Waste Rock Dump (WRD) and Dysons Overburden WRD
- Consideration of placement methods and their impacts on contaminate mobilisation risk, Water Treatment Plant rates, Main Pit geological features, Rum Jungle construction scheduling and final landform requirements;
- Placement of clean, inert cover materials over backfilled waste rock;
- Placement of erosion protections;
- Recontouring of Main Pit crest to meet final landform objectives and facilitate revegetation.

3 Previous Reports and Data Sources

A desktop study of previously published reports has been undertaken, and relevant information extracted for use in this study. **Table 1** provides a summary of the most relevant reports referenced.

Table 1 Summary of Documents Reviewed

Type	Author	Title	Year	Ref.
Reference	Zinc Corporation Limited	Geology of the Rum Jungle District, with Particular Reference to the origin of the Uranium Orebodies, P.F. Williams	1963	[7]
Reference/Photographs	National archives of Australia	Historical photographs form mining operations from national archives of Australia: A1200, L25494	1960's	[8]
Report	Cameron McNamara	Rum Jungle Rehabilitation Stage 3: Tailings Dam, Copper Heap Leach Pile and Dysons Open Cut, Options and Design Criteria Report, Dec 1983	1983	[9]
Report	Northern Territory Department of Mines and Energy	Final Project Report – The Rum Jungle Rehabilitation Project	1986	[10]
Report	Northern Territory Government	Rum Jungle Rehabilitation Project Monitoring Report 1993-1998	2002	[11]
Report	M4K Environmental	Environmental Issues and Considerations for Future Management, February 2004	2004	[12]

Type	Author	Title	Year	Ref.
Report	Robertson GeoConsultants Inc.	Phase 3 (Stage 2 Report): Report on Numerical Groundwater Flow Modelling at the Rum Jungle Mine Site, NT, April 2012	2012	[13]
Report	Robertson GeoConsultants Inc.	Phase 2 Report: Detailed Water Quality review and Preliminary Contaminant Load Estimates at the Rum Jungle Mine Site, NT, Feb 2011	2012	[14]
Report	Robertson GeoConsultants Inc.	2014 Hydrogeology Drilling Program, Old Tailings Dam Area, Rum Jungle, April 2015	2015	[15]
Report	Miloshis, M. & Fairfield, C.	Rum Jungle 14-253-RFQ-RS Bathymetric Survey	2015	[16]
Report	O'Kane Consultants	Detailed Civil Works Project for Rehabilitation of the Former Rum Jungle Mine: Potential Borrow Material Assessment	2016	[17]
Presentation	Department of Mines and Energy NTG	Rum Jungle Mine Rehabilitation Project Stage 2 Detailed Design. Completion Criteria and Framework.	2016	[4]
Report	Robertson GeoConsultants Inc.	Physical and Geochemical Characteristics of Waste Rock and Contaminated Materials, Rum Jungle	2016	[18]
Report	Robertson GeoConsultants Inc.	Groundwater Flow and Transport Model for Current Conditions, Rum Jungle	2016	[19]
Report	Robertson GeoConsultants Inc	Main Pit Backfilling Concept Approaches, Rum Jungle, June 2016	2016	[6]
Report	SLR Consulting Australia Pty Ltd	Rum Jungle Rehabilitation - Borrow Pit Identification, Geotechnical Field Investigation	2016	[20]
Report	Robertson GeoConsultants Inc.	Waste Storage Facility Investigations, Rum Jungle	2016	[21]
Report	SRK Consulting (Canada) Inc.	Rum Jungle Pit Backfill Investigations: Factual Data Report	2018	[22]
Report	ATC Williams Pty Ltd.	Rum Jungle - Stage 1, Tailings and Soil Testing, Laboratory Testing Report, May 2019, Ref. 117213.01-R01	2019	[23]
Memorandum	ATC Williams Pty Ltd.	Rum Jungle – Main Pit (White's Open Cut) Backfill Strategy – Tailings Consolidation Modelling – Preliminary Results, Ref. 117213.01-M002	2019	[24]
Report	NT EPA	Draft Environmental Impact Statement – Rehabilitation of the Former Rum Jungle Mine Site	2019	[25]
Report	SLR Consulting	Rum Jungle Waste Rock Storage Facility and Borrow Area Geotechnical Investigation	2020	[26]

In particular, the background historical and more recent Robertson GeoConsultants (RGC) works, [6], [18], [21] have been used to build up an understanding of the main soil and rock types, spatial relationships, geological structures [22] and material properties relevant to rehabilitation of the Main Pit. Plans and geological sections taken through the Main Pit are discussed in **Section** 7 and show the pit geometry and interpreted subsurface conditions.

4 Site Description

4.1 Past Mining Operations

The Rum Jungle mineral field is located in Northern Territory, Australia (approximately 105 km south of Darwin) and contains numerous polymetallic ore deposits, such as the Ranger and Woodcutters ore deposits and the ore deposits associated with the Rum Jungle Mine (i.e. the Main, Intermediate, Dyson's, and Browns Oxide ore deposits) [7].

The Rum Jungle Main Open Pit is located in the central mine reach along the pre-mining course of the East Branch of the Finniss River (EBFR). The pit was mined out in the 1950s and 1960s and became flooded with contaminated groundwater and seepage when mine de-watering ceased [27]. Rum Jungle's Main Pit was an open cut uranium and copper mine which was roughly circular and about 350 m in diameter. The pit was mined to about 105 m below ground level (bgl), approximately RL -35 m RL with steep side walls and a spiralling haul road leading from the surface to the base of the pit [8], [16], [27].

Once mining had ceased in this area of the wider mine site, the Main Pit was partially backfilled with tailings with some side-cast soil and waste rock in the 1960s. Uranium ore tailings were historically deposited into the Main Pit between 1965 and 1971 from an adjacent processing plant, which at the time was processing ore likely originating from the Rum Jungle Creek South (RJCS) open cut, located approximately 6 km south of the Main Pit. The original thickness of backfill material was estimated to be about 60 m [6]. The remaining void was flooded to an approximate level of +61 m RL (40 m head of water) by overflow water from the Finniss River and groundwater from the local bedrock aquifer.

4.2 Past Rehabilitation Activities

Mining and mineral processing at the site created significant environmental impacts, primarily elevated dissolved copper from AMD which polluted the EBFR. In the early 1960s, the significant environmental impacts were recognised in correspondence between the AAEC and the NT Administration (NAA: F1, 1962/1824). The Commonwealth initiated an aesthetic clean-up of the mine site in 1977. The outcome of this technical assessment and planning effort was a 4-year rehabilitation project funded by the Commonwealth and implemented by the NTG between 1982 and 1986.

On 4 March 1983 a $16.2 million agreement between the Commonwealth and NTG established the 1983 Agreement. The site was rehabilitated between 1983 and 1986, and the major proportion of funding was spent treating highly contaminated water in Main Pit. The Final Project Report (Allen and Verhoeven, 1986) provided a full description of the rehabilitation project, including the rationale for works and the results of preliminary monitoring. At the time, the rehabilitation was deemed to have achieved its objectives [28].

The rehabilitated site was considered to have successfully achieved its set engineering and environmental criteria based on the results of a 12-year monitoring program undertaken between 1986 and 1998, funded jointly by the Commonwealth and NTG. The rehabilitation of the Rum Jungle site was recognised as being world-leading practice at the time, especially the installation of a multi-layer cover system. Cover system design and construction technologies were then in their infancy, so the site attracted international attention as one of the first implementations of a cover system for rehabilitation of sulfidic waste rock dumps.

According to Allen and Verhoeven [28], the objectives of the 1980s Rum Jungle Rehabilitation Project were to:

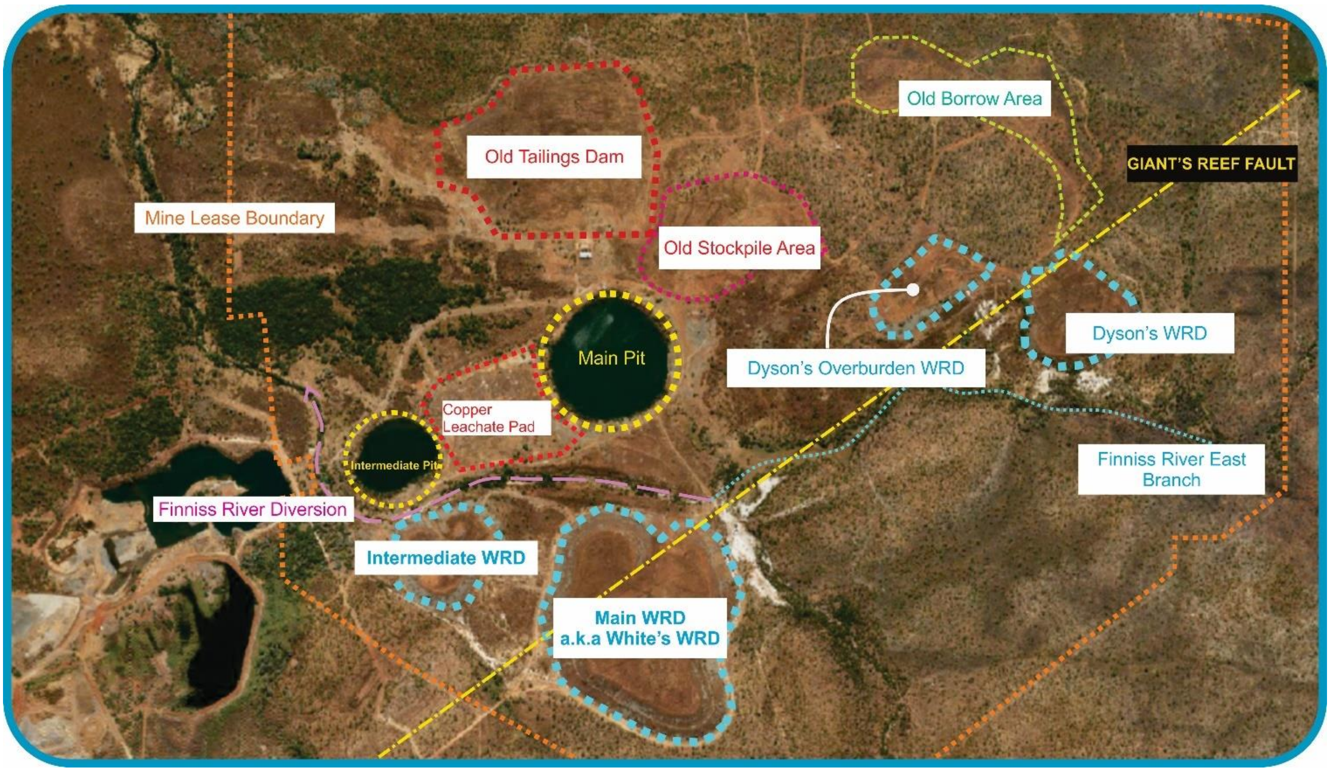
1. Achieve a major reduction in surface water pollution, aimed at reducing the average quantities of copper (by 70%); zinc (by 70%); and manganese (by 56%) as measured at the confluence of the East Branch and the Finniss River;
2. Reduce pollution levels in the Main and Intermediate Pits;
3. Reduce public health hazards, including radiation levels at the site to at least the standards set by the Code of Practice on Radiation Protection in the Mining and Milling of Radioactive Ores (Commonwealth of Australia, 1980); and
4. Implement aesthetic improvements, including revegetation.

According to Allen and Verhoeven [28], four primary rehabilitation treatments were undertaken:

- A three-layer cover system was constructed over the WRDs to reduce infiltration to less than five percent of rainfall. The WRDs were also reshaped and drainage structures installed to mitigate erosion and maintain the integrity of vegetation cover. A mix of introduced pastures and legumes were used for rapid revegetation. Grass cover was the specified revegetation condition for the WRDs.
- A water treatment plant was constructed to treat heavily contaminated water from the Main Pit. Water was withdrawn from depth, with lower density treated water returned to the surface of the pit where it formed a layer of clean water overlying the untreated water at depth. Water in the Intermediate Pit was treated in situ with lime to remove heavy metals and neutralise pH. Wet season flows were then re-instated through both pits so that the system would be flushed each Wet season. Based on the results from limnological modelling, it was anticipated this process would slowly cleanse the contaminated water that remained at depth in the pits by a combination of seasonal partial vertical mixing and Wet season flushing of the surface layers. Filter cake from the water treatment process was buried in Borrow Area 5, to the north of the site and capped with a three-layer cover system.
- Dyson's Pit was partially backfilled with tailings from the tailings area and Tailings Creek. The surface of the tailings was covered with a coarse geotextile and an approximately one metre thick rock blanket drainage layer. The drainage layer was overlain with low-grade copper ore, copper launders from the Copper Extraction Pad and contaminated soils from both sites. A moisture barrier, a moisture retention zone and an erosion resistant cover were installed on top and the final surface revegetated in the same way as the WRDs.
- After the tailings were removed to Dyson's Pit, the tailings area footprint was reshaped to control drainage, limed and covered with a one-layer system (of soil) to enable revegetation with introduced pastures and native trees and shrubs.
- A sub-surface drainage system and a four-layer cover system were installed over the copper extraction area to address residual surface and sub-surface contamination. The surface was revegetated with the same methodology as the WRDs

Significant mining and rehabilitation features are highlighted in **Figure 1**.

Figure 1 Rum Jungle Mine Layout



4.3 Geology

The local mineralised complex comprises a faulted and folded series of units including meta-sediments, consisting of dolostones, mudstones, schists and slates and intrusive igneous rocks (Coomalie and Golden Dyke Formations). Previous investigations (Robertson GeoConsultants, 2016 [21]) at the surface have encountered highly weathered rock forming clays to sandy gravelly clays and references to the Main Pit side walls describe "soft slates and intensely fractured rock" prior to backfilling. Recently observed conditions of the side walls in the uppermost exposed zone record very low to medium strength weathered rock to clay soil.

4.4 Hydrology and Hydrogeology

Between April 2015 and August 2019 flows from the East Branch Finniss River (EBFR) were blocked from entering the Main Pit and all flow were diverted south of the Main and Intermediate Pits via the man-made Main Diversion drain. The inlet to the Main Pit has been sealed by means of a steel plate and loose earth placed over a DN1000 HDPE stub pipe protruding from the upstream headwall of a 1.5 m x 0.75 m reinforced concrete box culvert (RCBC) (refer **Figure 2**). Blockout material (most likely steel) has been used to seal the void between the two conduits. Prior to April 2015, low flows in the EBFR entered the Main Pit when water depths exceeded RL 60.64m AHD at the confluence of the Main Diversion drain and Main Pit spur channel to the south east of the Main Pit.

Figure 2 Main Pit Entrance April 2015 (left) and August 2019 (right). Steel Plate under rubble blocking pipe entrance visibility

Main Pit Entrance April 2015

Main Pit Entrance August 2019

Flows into the Main Diversion drain commenced when water levels exceeded RL 62m AHD at the Main Pit entrance.

The man-made embankment above the culvert entrance to the Main Pt has a crest level of RL 66m AHD which is 5.4 m above the invert of the pipe. During flooding, water borne sediment and silt eroded from the EBFR is washed into the Main Pit.

The water level in the Main Pit is controlled by a 2.4 m x 0.75 m RCBC outlet in north western embankment of the Main Pit. The invert of the culvert is 59.98m AHD. Sometime between April 2015 and August 2019 it appears the outlet of the Main Pit was blocked (**Figure 3**).

Figure 3 Outlet of the Main Pit. April 2015 (left) and August 2019 (right)

Main Pit outlet April 2015

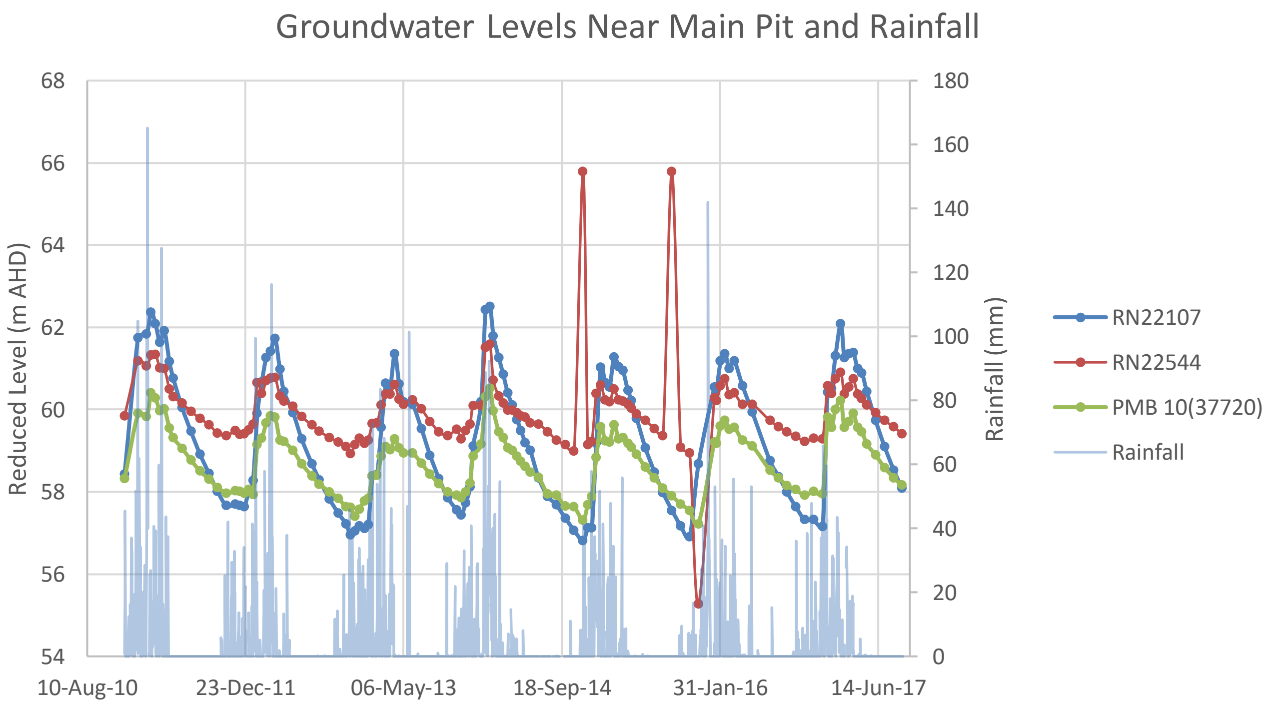
Main Pit outlet August 2019

Overflood water from the east branch of the Finniss River, located to the south of the Main Pit, is fed into the Main Pit via a culvert which passes through an embankment with a maximum elevation of +66 m RL. During the flooding, sediment from the surface is eroded and washed into the Main Pit.

The culvert sits at RL+61.5 m RL to +62.5 m RL and is located on the south east of the Main Pit perimeter. An over flood drainage for the Main Pit is present in the north west of the pit at a level of +62.5 m RL and represents the maximum water level in the Main Pit.

Prior to being backfilled, groundwater ingresses into the Main Pit were recorded at several elevations from the local principal aquifer, the Coomalie Dolostone; approximately at 27 m (RL 33m AHD) in the south east of the pit, 70 m (RL 10m AHD) in the north and 76 m (RL -16m AHD) below the surface level in the west [19]. The backfilled tailings and waste rock in the Main Pit have reduced the hydraulic connectivity to the deeper bedrock aquifer, due to the low permeability of the tailings.

Figure 4 Annual Main Pit Surface Water and Groundwater Levels (DPIR Monitoring Data Dec-2010 to Aug-2017)



Seasonal fluctuations in the Main Pit's water level have been recorded based on Main Pit proximal groundwater monitoring wells. Main Pit water level has been observed to be well connected to Monitoring Well RN22544. Groundwater levels in RN22544 vary between +58.95 m RL and +61.59 m RL and are presented in **Figure 4** along with proximal ground monitoring wells (RN22107 and PMB 10) and rainfall data collected from Batchelor Airport (Station No. 14272) (Bureau of Meteorology) located approximately 8 km south of the Rum Jungle Mine Site. It is noted the graphed maximum and minimum results recorded on 24-11-14, 01-09-15 and 24-11-15 are considered anomalous, and are ignored in the seasonal variation consideration.

The highest Main Pit water levels are recorded in the wet season and appear to be influenced by the local groundwater within the bedrock aquifer. During the dry season the recorded groundwater is up to 4.0 m lower than the Main Pit which suggests the water within the pit is discharging into the bedrock aquifer [19].

4.5 Bathymetry Surveys

The most recent bathymetric survey completed in 2015 [16] shows that the greatest depth to the base of the pit is +14.18 m RL near the it’s centre. At that time (summer 2015/2016) the surface water level was recorded at an elevation of +58.92 m RL. A previous bathymetric survey performed in the winter of 2008 [29] also estimated the full capacity level to be approximately +60 m RL. Based on the available data, the pit’s current maximum depth is estimated to be approximately 46 m below water surface.

The deepest area of the Main Pit is relatively flat with a slight slope to the south. The side walls along the eastern, southern and western edges are steep to near vertical with evidence of the old haul road, showing little variation from the original open pit shell. The northern and north eastern edge of the pit is assumed to be where the tailings and waste rock where deposited around edge areas, forming a beach-like feature with an approximate 1V : 3H slope. **Figure 5** shows the pit shell dimensions and topography [16] as recorded in 2015. Obliques of the Main Pit surface interpolated from the 2015 bathymetry contours are presented in **Figure 6**.

Figure 5 2015 Main Pit Bathymetric Survey (Miloshis and Fairfield [16])

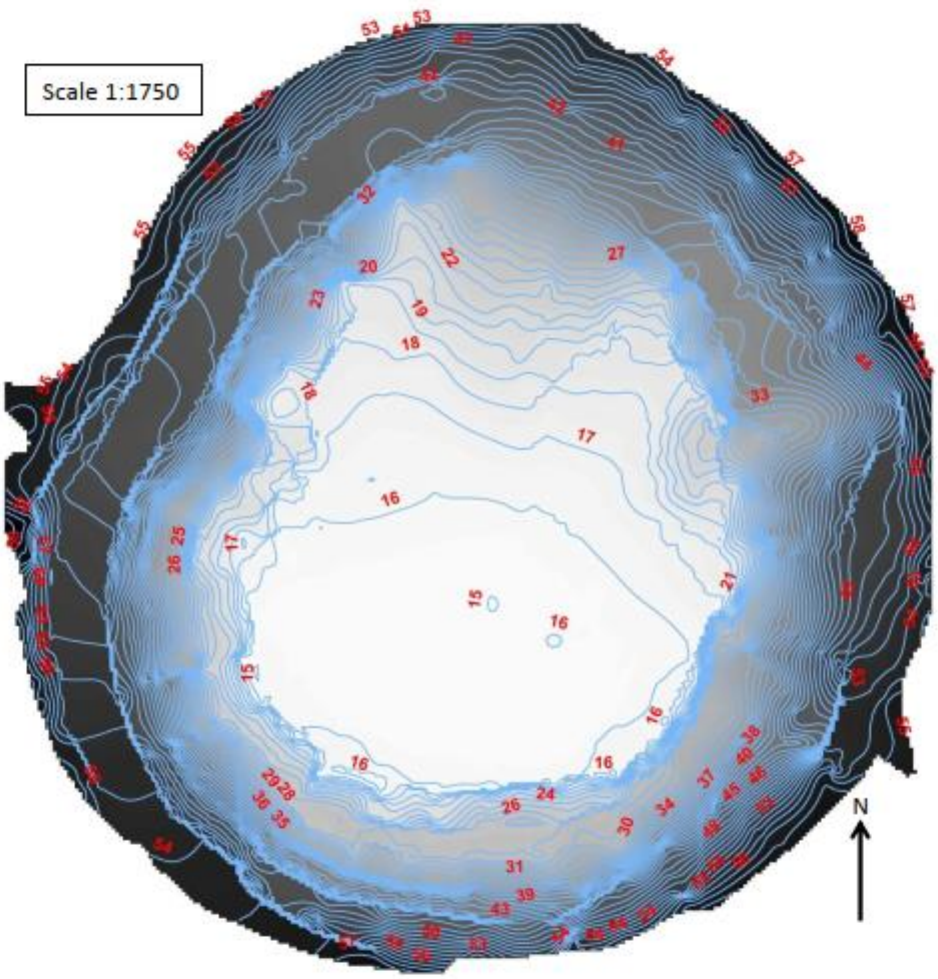
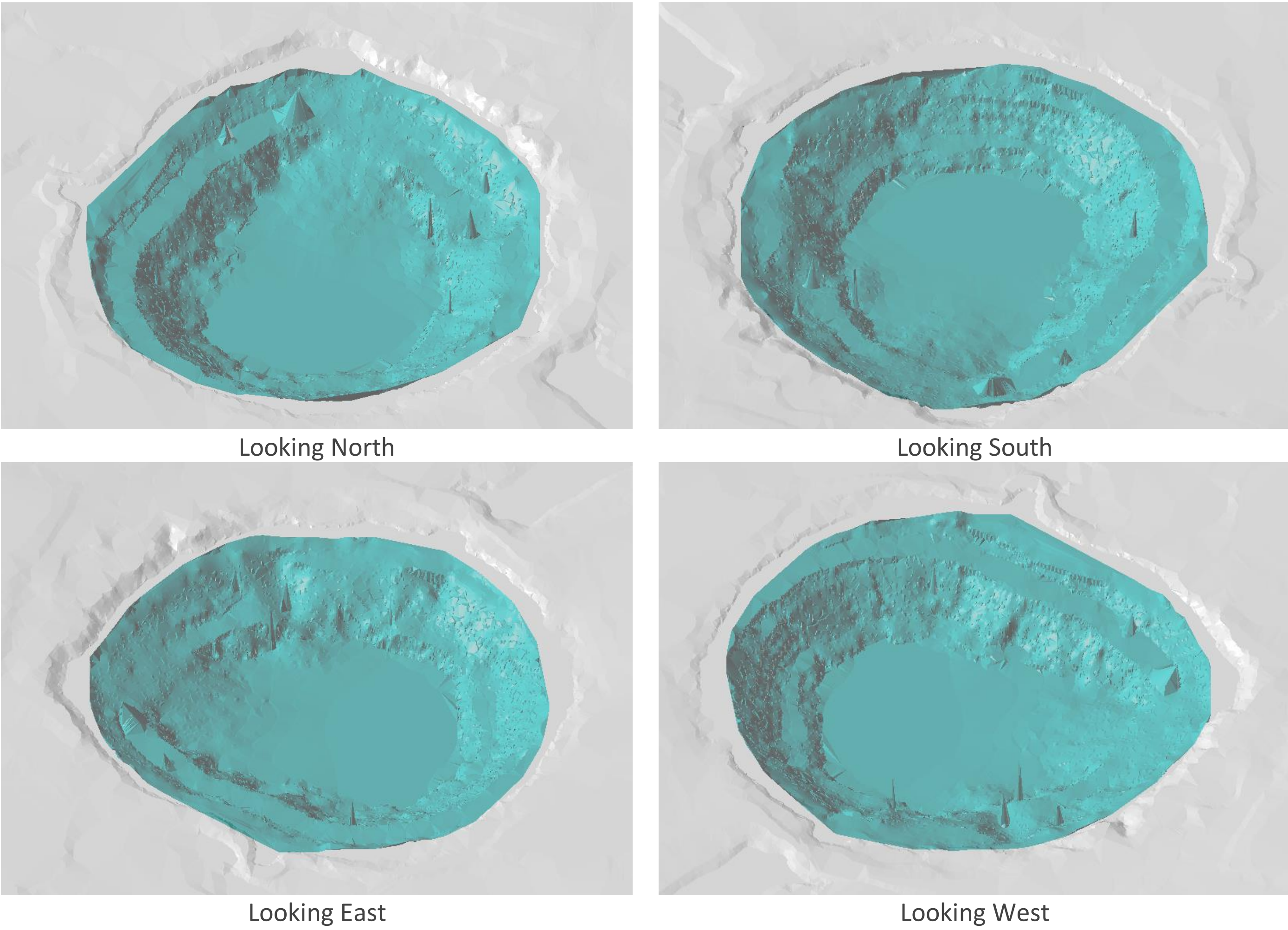


Figure 6 3D Obliques of Main Pit 2015 Surface (2015 Bathymetry Survey)



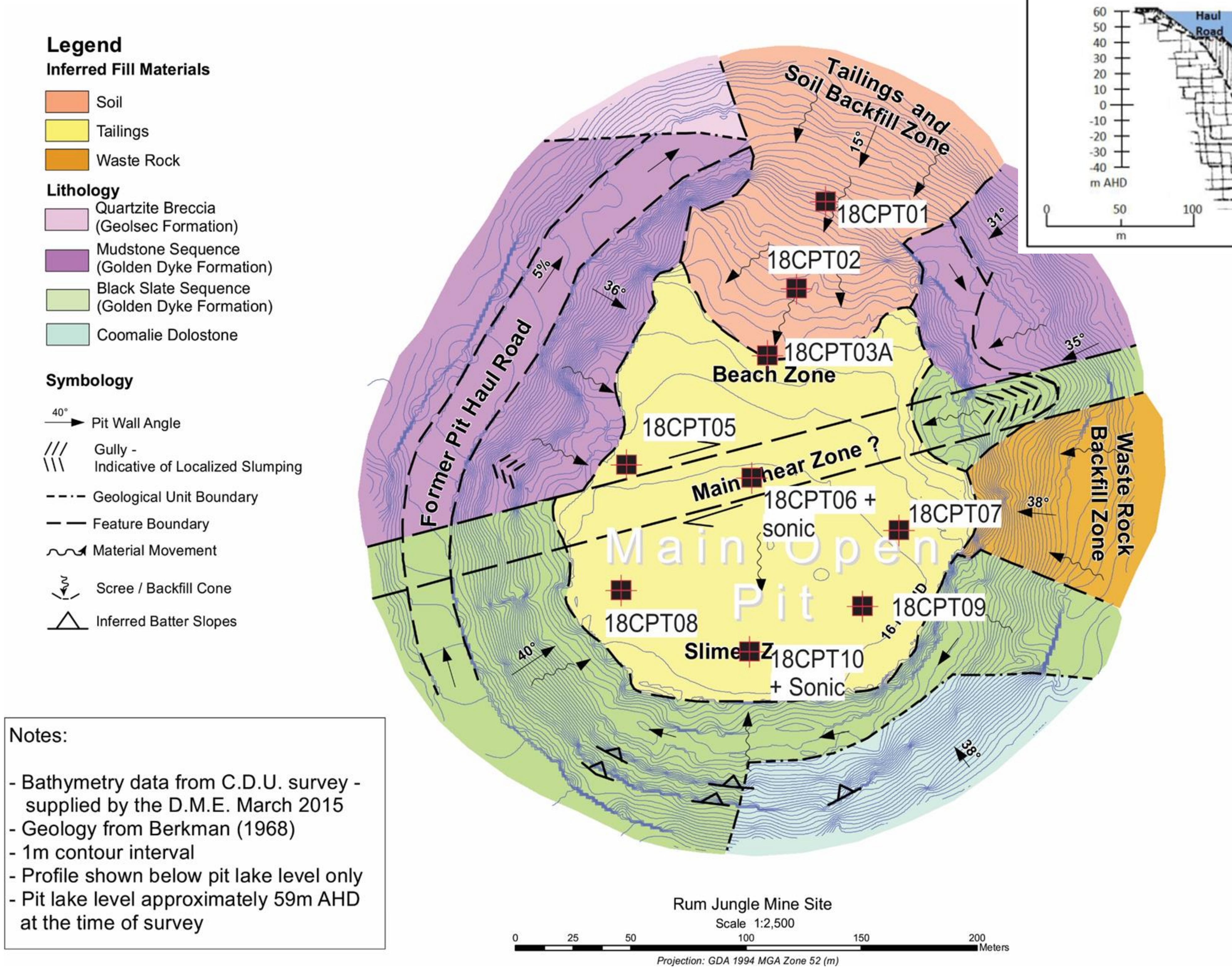
4.6 Water Quality and Contamination

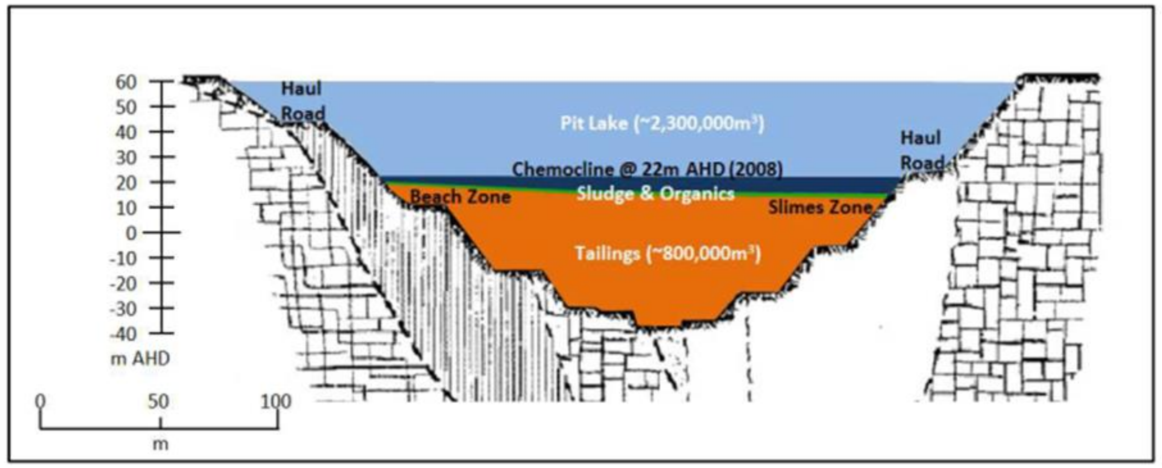
The groundwater which flows into the Main Pit is highly impacted by contaminants sourced from the old leaching pads, rock dumps and other mining activities. A "chemocline" has formed at the base of the pit, comprised of a dense liquid with considerably higher concentrations of heavy metals and other pollutants. This is a consequence of the water body being largely stagnant, allowing for settlement of more dense constituents [6]. Most recent groundwater monitoring data [19] suggests the top of the chemocline is at an approximate elevation of +19 m RL and estimated to be 4 m thick across the base of the pit.

The chemocline layer is thought to have lowered over time with measurements in 1990 indicating a level of +38 m RL and levels in 2008 indicating a level of about +22 m RL. The decrease in level is thought to be due to the seasonal 'flush' of the Main Pit from flood waters. The groundwater quality within the pit is very poor but has been shown to improve with elevation over time, due to rehabilitation efforts in the 1980s and from the inflowing surface water.

Figure 7 below shows an interpretation of the geology and backfill material distribution as interpreted by SRK Consulting [22] based on 2018 geotechnical field program (in pit Cone Penetration Tests (CPT) and Pit Rim Boreholes).

Figure 7 Historic Findings and Assessments





Robertson GeoConsultant Assumptions:

- Slimes are typically normally consolidated;
- Excess pore pressure has dissipated due to 30yr period;
- High void ratio and low undrained shear strength (Su) near surface;
- Highly compressible and will generate excess pore pressures during compression;
- From sonic boreholes, appears to be 0.50 – 1.00m of naturally deposited silt/organics;
- Scree slope on top of tailings. Failure of shear zone inferred post tailings deposition;
- Tailing of 8.00m to 12.00m thick sitting on top of waste rock lens in 18CPT09 + 18CPT07. Waste rock and tailings well consolidated;
- Slight slope to the south in central and southern base areas, becoming steeper in the north ~15°

5 Previous Investigations

5.1 Main Pit Investigations

5.1.1 SRK, 2018 – Main Pit Investigations

As part of Stage 2 works, SRK Consulting (SRK) undertook a geotechnical investigation comprising of boreholes program along the Main Pit Rim (3 No. Boreholes) using both sonic drilling techniques and HQ diamond core drilling techniques angled to vertical to target Main Pit tailings deposits and geological structural features along the Main Pit Rim. Boreholes were accompanied with in-situ testing including cone penetration tests (CPT), standard penetration tests (SPT), ball cone penetrometer testing (BCPT) and collection of disturbed and core samples for laboratory testing.

The sonic drilled boreholes (18CPT06 and 18CPT10) were orientated at Pit Rim to intercept tailings deposits within the base of the Main Pit. CPTs within the Main Pit were undertaken using a barge over water to intercept at Sonic borehole locations to compare drilling to CPT results. An additional seven CPTs only investigation locations were undertaken within the Main Pit over water (18CPT1 – 3a, 18CPT05 and 18CPT07 – 18CPT09) with four locations also including ball CPT in-situ testing. CPTs were also conducted at Dyson's Overburden Pit (18CPT32, 18CPT33 and 18CPT34) from the surface level. The diamond drilled boreholes were located along the western rim of the Main Pit.

A factual report was produced [20], alongside in-situ testing data results and borehole logs for the sonic drilling program however the diamond drill program was not reported on except for preliminary photo and borehole logs.

Table 2 below details the exploratory locations undertaken and **Figure 7** above shows their locations.

Table 2 SRK Intrusive Investigation Strategy

Area	Location ID	Method	Easting	Northing	Initial Backfill Elevation (m AHD)	Completed Elevation (m AHD)	Additional Comments
Main Pit Over Water	18CPT01	CPT	717875	8563498	38	29	Top of soil backfill zone. Cohesive below 3.5m depth
		BCPT	717872	8563498	38	37.5	
	18CPT02	CPT	717873	8563462	24	18	Mid slope in soil backfill zone Mainly granular. Unreliable test in top 1.5m – inferred CPT sleeve incorrectly reading
	18CPT03A	CPT	717864	8563428	18	14.4	Toe of slope in soil backfill zone. Unreliable test – inferred CPT sleeve incorrectly reading
	18CPT05	CPT	717809	8563377	16	-1.8	Tailings / slimes
	18CPT06	CPT	717856	8563371	16	-4	Tailings / slimes
		Sonic Drilling	717850	8563366	16	-8	Tailings / slimes
	18CTP07	CPT	717908	8563362	15	1	Tailings / slimes with granular zone below RL 7 (inferred soil debris flow)

Area	Location ID	Method	Easting	Northing	Initial Backfill Elevation (m AHD)	Completed Elevation (m AHD)	Additional Comments
	18CPT08	CPT	717808	8563333	15	-4	Tailings / slimes
		BCPT	717808	8563331	15	-2	
	18CPT09	CPT	717898	8563328	15	-6	Tailings / slimes with interbedded granular zones below RL 4 (inferred soil debris flow)
		BCPT	717896	8563328	15	4	
	18CPT10	CPT	717853	8563303	15	-2	Tailings / slimes
		BCPT	717855	8563303	15	-1	
		Sonic Drilling	717853	8563299	15	-14	
Main Pit Over Water	18DH01	Diamond	717769	8563224	69	18.3	
	18DH02	Diamond	717691	8563286	67	15.8	
	18DH03	Diamond	717709	8563463	71	-15.7	
Main Pit Rim	18CPT32	CPT	718664	8563584	91.5	71.5	
		Sonic Drilling	718669	8563579	91.5	68.5	
	18CPT33	CPT	718779	8563593	80.5	41.3	
		Sonic Drilling	718791	8563586	80.5	47.5	
	18CPT34	CPT	718819	8563665	79.5	47.9	
		Sonic Drilling	718813	8563654	79.5	68.5	

The raw data from the CPTs in **Table 2** above has been reprocessed using the proprietary CPeT-IT® software to assist in the characterisation of materials and development of intrinsic soil parameters for backfilling design.

Historical evidence indicates that Dyson's Overburden Pit was also backfilled with tailings [9]. To assist in assessment Main Pit tailings CPT results, comparison to Dyson's Overburden Pit CPTs results were made to support the analysis.

A series of overlay reports has been prepared by grouping the data into two main groups of materials and is presented in **Appendix A.**

5.1.1.1 Sonic Drilling

The sonic drilling program was split into two areas; the Main Pit and Dyson's Overburden Pit. For the Main Pit CPT program, the following CPT locations were undertaken along the northern tailings and soil backfill zone beach shelf and across the central and southern Main Pit floor, summarised as:

- 18CPT01, 02, 03A undertaken in the soil backfill zone above the sloping pit wall and tailings; and
- 18CPT05, 06, 07, 08, 09 and 10 undertaken in tailings above the pit floor.

Two boreholes were completed within the Main Pit, drilled from a barge over approximately 45 m of water. Both boreholes were located centrally and off the tailings and soil backfill zone beach shelf.

Boreholes drilled at locations of 18CPT06 and 18CPT10 encountered very soft to silty clay and clayey silt from the base of the pit (Interpreted backfilled tailings surface) to 0 m RL (16m below interpreted tailings surface) and +9.5 m RL (4.5m below interpreted tailings surface) respectively. The upper 0.5 m to 1.0 m clayey material was observed to have an organic odour (SRK daily diary field observations [22]) which is thought to be associated with recent sediments derived from the inflowing river water.

Below this the encountered ground conditions comprise mainly cohesive normally consolidated tailings with interbedded clays and silts with variable amounts of clay, silt and some minor sand interbeds. Within borehole 18CPT06 this interbedded unit remains very soft to soft and in borehole 18CPT10 from very soft to firm. From approximately +1 m RL in borehole 18CPT10 (14 m below interpreted tailings surface) a more homogenous unit of silty, sandy clay is encountered and extends to the base of the borehole -13.5 m RL (29m below interpreted tailings surface). All the fined grained deposits are described as high plasticity.

Three sonic drill boreholes were completed with CPTs within the backfilled Dyson's Pit area. CPT data was generally heterogenous throughout, with some indications of more cohesive material at depth (increasing pore water pressure and less variability in other parameters recorded). Borehole logs recorded variable soil materials which were summarised as consisting of cover material, waste rock and a drainage layer overlying backfilled tailings. This is in line with proposed engineered design from the rehabilitation works undertaken in the 1980s [10].

Review of sonic core photographs indicate a significant amount of drilling disturbance has occurred due to the nature of the material and drilling technique used.

5.1.1.2 Main Pit Laboratory Testing

Geotechnical lab testing has been undertaken on samples collected during the SRK investigation. Samples were transported to ATC Williams Pty Ltd laboratory to undertake testing on disturbed soil and tailings from the sonic and diamond drilling programs and on weathered rock core from the diamond drilling program.

The following tests were undertaken on disturbed tailings slimes samples collected during the sonic drilling program:

- Atterberg limits;
- Moisture content;
- Consolidation (Rowe Cell);
- Solids concentration;
- Specific gravity;
- Bulk dry density; and,
- Particle size distribution with hydrometer.

The following tests were undertaken on disturbed soil samples collected during the diamond drilling program:

- Atterberg limit;
- Moisture content;
- Solids concentration; and,
- Particle size distribution with hydrometer.

The following tests were undertaken on rock core samples collected during the diamond drilling program:

- Point load test; and,
- As-received density (submersion)

Table 3 to **Table 5** below present the results of the testing.

Table 3 Sonic Drilling Program – Tailings Slimes Laboratory Testing Results

Sample	Depth (m below tailings surface)	Type	Moisture Content (%)	Atterberg Limits			Classification	Particle Size Distribution			Solid Concentration (%)	Field Dry Density (t/m³)	Field Bulk Density (t/m³)	Specific Gravity (t/m³)
				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)		Gravel + Sand (%)	Silt (%)	Clay (%)				
CPT10 P1	1.0 - 1.5	Piston	51	-	-	-	-	20.6	56.4	23	66	0.94	1.42	-
CPT10 P2	4.0 - 4.5	Piston	48	55	32	23	High Plasticity	7.5	66.5	26	68	0.93	1.38	-
CPT10 P4	9.0 - 9.5	Piston	53	-	-	-	-	15	53.0	32	65	1.09	1.67	-
CPT10 1034	18.0 - 18.5	Bulk	43	42	25	17	Intermediate Plasticity	1.3	65.7	33	70	-	-	-
Composite Tailings CPT10	1.0 - 18.5	Composite	-	45	27	18	Intermediate Plasticity	4.3	67.7	28	-	-	-	2.79

Table 4 Diamond Drilling Program – Soil Laboratory Testing Results

Sample	Depth (m)	Type	Moisture Content (%)	Atterberg Limits			Classification	Particle Size Distribution			Solid Concentration (%)
				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)		Gravel (%)	Sand (%)	Fines (%)	
DH01 10	0.00 - 1.50	Soil	2.1	-	-	-	-	35	47	18	98
DH01 11	1.50 - 3.40	Soil	6.0	-		-	-	87	9	4	94
DH02 5	1.40 - 1.70	Soil	2.8	41	19	22	Intermediate Plasticity	35	23	42	97
DH03 16	4.70 - 4.90	Soil	1.4	-	-	-	-	36	43	21	99
DH03 17	6.90 - 7.20	Soil	8.1	47	22	25	Intermediate Plasticity	42	18	40	93
DH03 19	7.80 - 8.20	Soil	9.0	-	-	-	-	71	17	12	92
DH03 20	9.10 - 9.30	Soil	15	-	-	-	-	35	34	31	87

Sample	Depth (m)	Type	Moisture Content (%)	Atterberg Limits			Classification	Particle Size Distribution			Solid Concentration (%)
				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)		Gravel (%)	Sand (%)	Fines (%)	
DH03 21	10.40 - 10.70	Soil	8	-	-	-	-	69	22	9	93

Table 5 Diamond Drilling Program – Rock Core Laboratory Testing Results

Sample	Depth (m)	Type	Dry Density (t/m3)	Point Load Is50 (MPa)	Strength Class (AS 1726:2017)
DH02 6	2.7 – 3	Core	1.9	0.05	Very Low
DH03 18	6.9 - 7.2	Core	2.2	0.49	Medium

5.1.2 ATC Williams, 2019 – Main Pit Settlement Assessment

In April 2019, as part of the early Stage 2A work, ATC Williams (Ref: 117213.01-M002, dated 17 April 2019 [24]) analysed the potential consolidation of the in-situ tailings based on preliminary backfill modelling strategies.

Compressibility parameters for tailings were assessed based on testing of samples from the investigation campaign described above. Specifically, consolidation properties were based on a single Rowe Cell consolidation test undertaken on a composite sample derived from sub-samples obtained at several depth intervals in the sonic borehole drilled at the location of 18CPT10.

Key assumptions included in the ATC William settlement analysis include:

- Filling to a final design level of RL 56.5 m RL;
- Filling over a period of 22 months at a rate of 360 t/hr; and
- Backfill density of 18 kN/m^3.

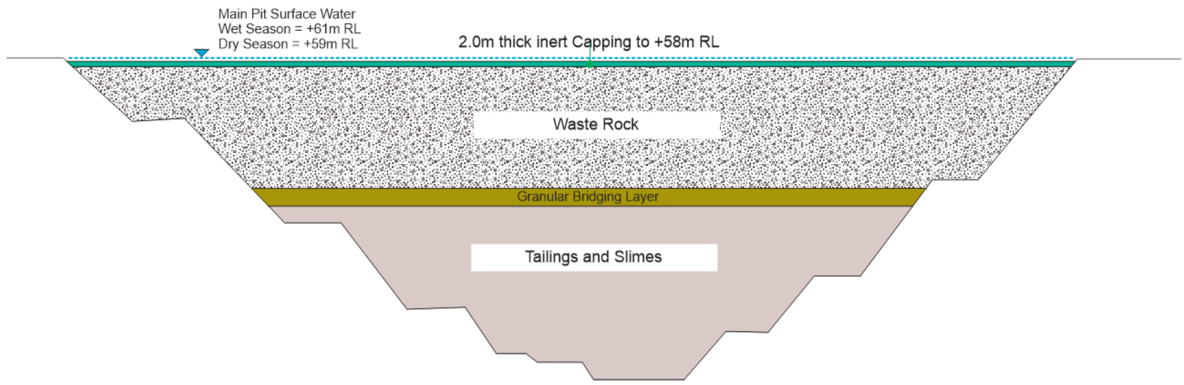
A resulting settlement prediction indicated that up to 6.6 m of settlement could occur over a period of 170 years following backfilling, requiring pre-emptive 'doming' of the finished surface to offset the expected 'dishing' of the backfill surface over time.

6 Geotechnical Investigation of Backfill Materials

The following section details the characteristics of materials proposed to backfill the Main Pit. The backfill materials can be broadly separated into three layers as defined below and shown in **Figure 8**.

- Granular Bridging Layer
- Waste Rock Backfill
- Inert Capping Layer

Figure 8 Generalised Main Pit Backfilled Profile

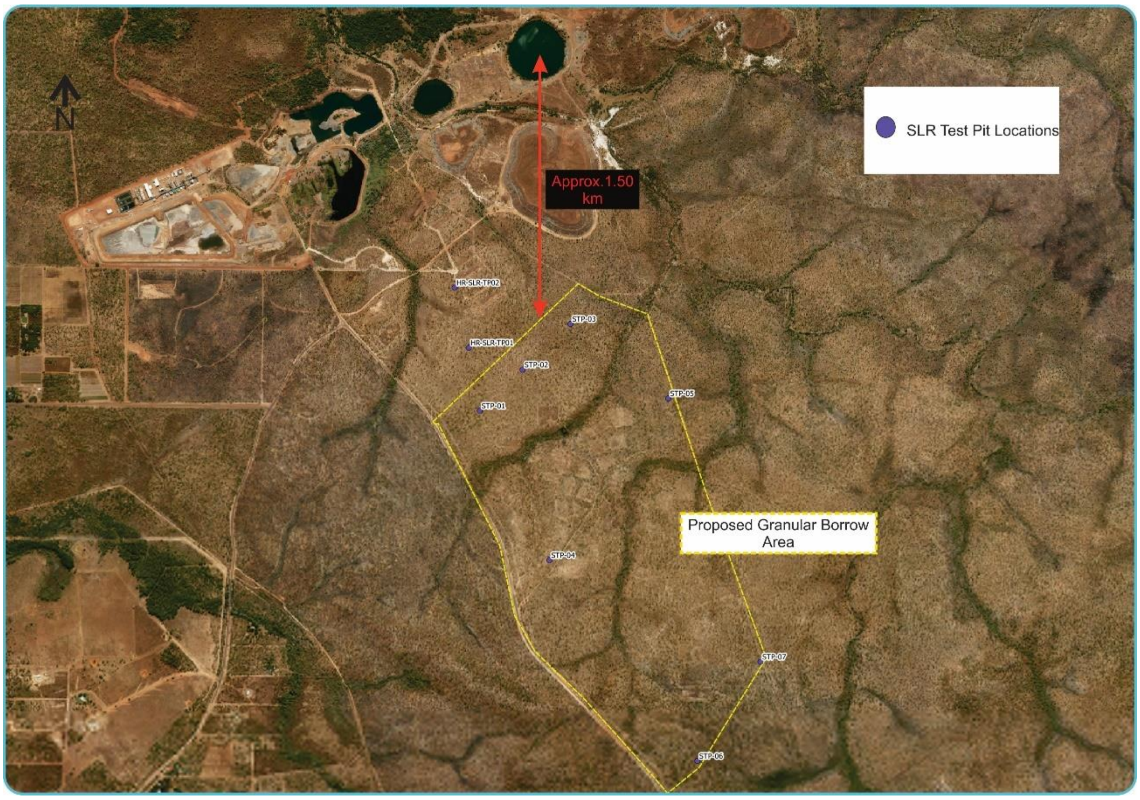


Further detail on the granular bridging layer material and inert capping layer material characteristics can be found in the 2020 SLR Geotechnical Report [26]. Details on the waste rock material characteristics, can be found in the Robertson GeoConsultants 2016 Physical and Geochemical Characteristics of Waste Rock and Contaminate Material Report [18]. The following provides a summary of the material characteristics as presented in the aforementioned reports.

6.1 Granular Bridging Layers

The granular borrow (Borrow Area B), located approximate 1.50 km south of the Rum Jungle Mine site was investigated by SLR in July and October 2019. The ground conditions encountered included thin and discontinuous topsoil and alluvium over residual soil derived from the underlying granite bedrock. The residual soil descriptions ranged from clayey SAND, gravelly SAND to sandy GRAVEL. The layer was found to be dense to very dense comprising fine to coarse sand and fine to coarse, sub-rounded to sub-angular gravel with occasional pale grey quartzite inclusions and granite derived cobbles. This material will be the main source of the bedding and inert capping backfill layers.

Figure 9 Granular Borrow Location



6.1.1 Granular Borrow Laboratory Testing

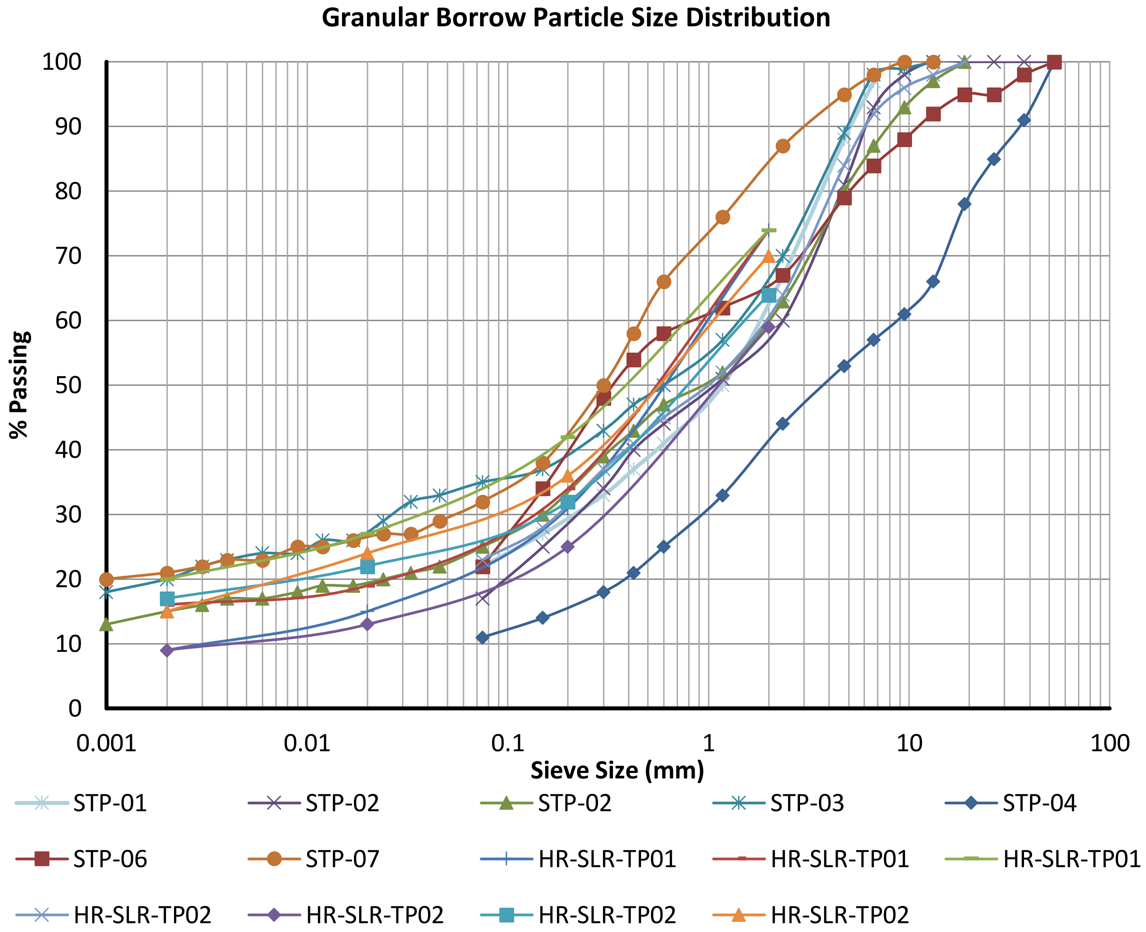
Testing conducted on samples collected within and proximal to the borrow area comprised particle size distribution (PSD), particle density, shear box, Atterberg limits, compaction and Emerson classifications. Regarding the backfilling strategy, the PSD, particle density and shear box testing results are significant in selecting physical properties for this material during the backfilling assessment. A summary of test results on borrow materials are provided below. Laboratory certificates are presented in SLR Geotechnical Report [26].

Compilation of PSD data relevant to the granular backfill is provided below in **Table 6**. Compiled PSD curves are shown in **Figure 10.** All samples tended to plot within the sandy GRAVEL to gravelly SAND brackets as shown in the curves and statistical breakdown below. **The D_{50} value (50% passing diameter) is considered to be 1 mm.**

Table 6 Particle Size Distribution - Granular Borrow (Borrow Area B)

Analyte	% Gravel	% Sand	% Fines
Count	14 Samples (14 x SLR)		
Mean	33	43	24
Maximum	56	55	35
Minimum	13	33	11
Standard Deviation	9.8	6.2	6.6

Figure 10 Compiled PSD Curves - Granular and Growth Borrow (Borrow Area B)



Nine Particle density tests were performed on un-sieved material, and three tests were performed on samples passing 2.36 mm. Results of the Particle Density tests are provided in **Table 7** below.

Table 7 Particle Density Results

Analyte	Particle Density (g/cm^3)		
	Particle < 2.36 mm	Particle > 2.36 mm	Unscreened
Count	7	3	1
Mean	2.58	2.61	2.64
Maximum	2.68	2.63	-
Minimum	2.41	2.59	-
Standard Deviation	0.09	0.02	-

Eleven Emerson Class dispersity tests on samples collected from the Granular Borrow (Borrow Area B) retuned values ranging from Class 4 to Class 6, indicating the material as non-dispersive.

One sample (HR-SLR-TP02) was tested for shear strength properties using the direct shear box method to assess the material strength characteristics in its loose, saturated state. The same sample was tested twice for material passing 3.00 mm particle size and 1.00 mm particle size. Each test was confined at 50kPa, 75kPa and 150kPa. Each sample was poured into the shear box and saturated with no mechanical compaction. Given the test pit location and particle size distribution, the sample is considered representative of material within the Granular Borrow area (Borrow Area B).

Table 8 Shear Box Results

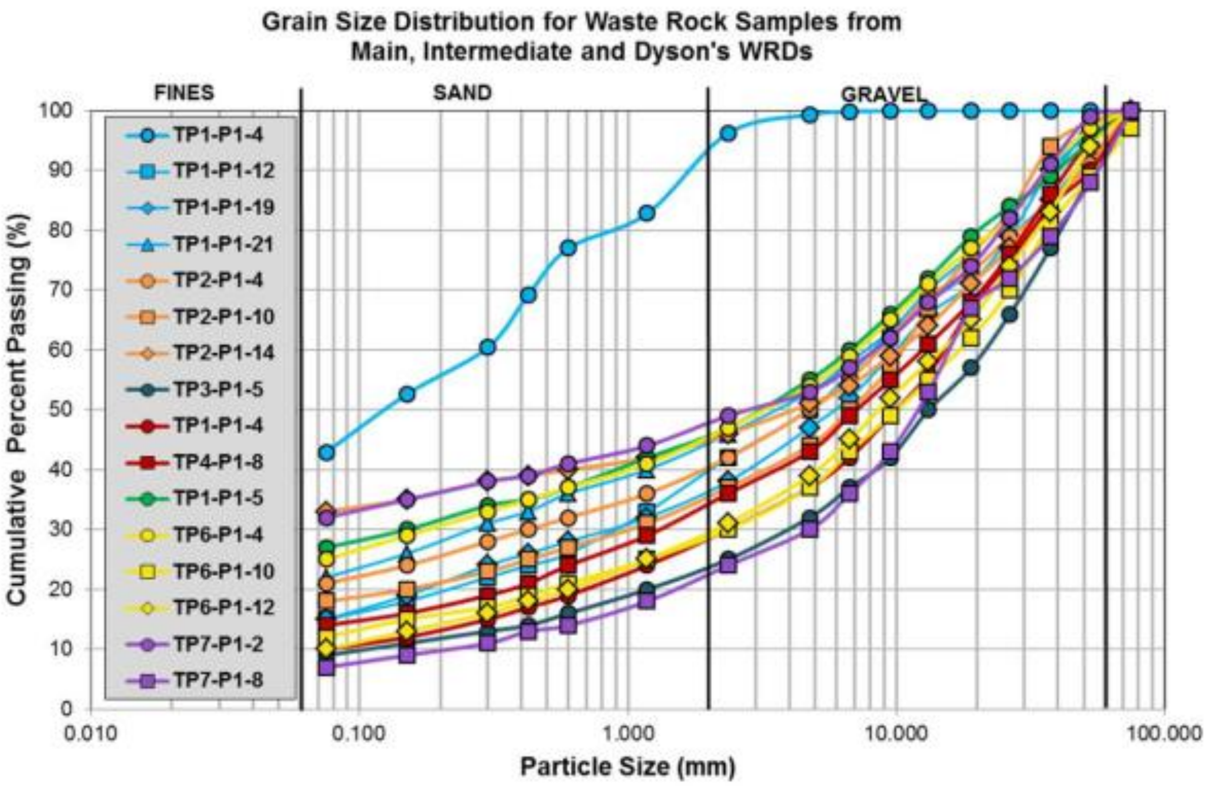
Sample ID	Dry Density (t/m^3)	Cohesion (c')	Friction Angle: Peak (ϕ')	Friction Angle: Ultimate (ϕ')	% Fines	Maximum Sieve Size Material Passing
HR-SLR-TP02 0.80 m -1.30 m	1.30	0 kPa	43	43	23 %	3.15 mm
HR-SLR0TP02 0.80 m – 1.30m	1.14	0 kPa	40	37	23 %	1.18 mm

6.2 Waste Rock Backfill Characterisation

SLRs investigations in July and October 2019 within the waste rock material generally described the material as gravelly SAND, sandy GRAVEL and GRAVEL and COBBLES with minor silt and clay. Coarse particles comprised mainly of sub-angular to sub-rounded shale which generally broke down easily. No testing was conducted on the waste rock during this investigation.

In 2014, RGC performed a detailed classification investigation into the characterisation of waste rock from the waste rock dumps through large test pit excavation and sampling. In-field particle size sieving was conducted on samples to remove any particles greater than 75 mm. The remainder of sample passing 75 mm sieve then underwent laboratory PSD testing. Generally, the percentage of sample passing the 75 mm sieve ranged from 95% to 20% of total sample quantity. The large range suggests a highly variable material consistent with the description SLR produced in 2019. The laboratory PSD curves for waste rock samples passing the 75 mm sieve are presented in **Figure 11.**

Figure 11 Waste Rock PSD Curves for Material <75mm (From RGC [18])



6.3 Summary of Key Material Characteristics and Parameters

Table 9 Summary of material backfill characteristics

Parameter	Granular Borrow (Borrow Area B)	Main Waste Rock Dump	Intermediate Waste Rock Dump	Dyson's Waste Rock Dump
% Material Particle Size > 75 mm	0 %	40%	33%	30%
D_{50}*	1.00 mm	6.00 mm#	10.00 mm#	8.00 mm#
Particle Density	2.61 (g/cm^3)	2.65 g/cm$^{3\Delta}$		
Material Type	Decomposed Granite Sands/Gravels	Waste Rock (pyritic shales, argillite, dolostone)		

Notes:

[*] = 50% of material passing particle size

[#] = D_{50} of Screened Sample < 75mm particle size

[Δ] = Based off G.E., Manger, *Porosity and Bulk Density of Sedimentary Rocks,* US Atomic Commission, 1963.

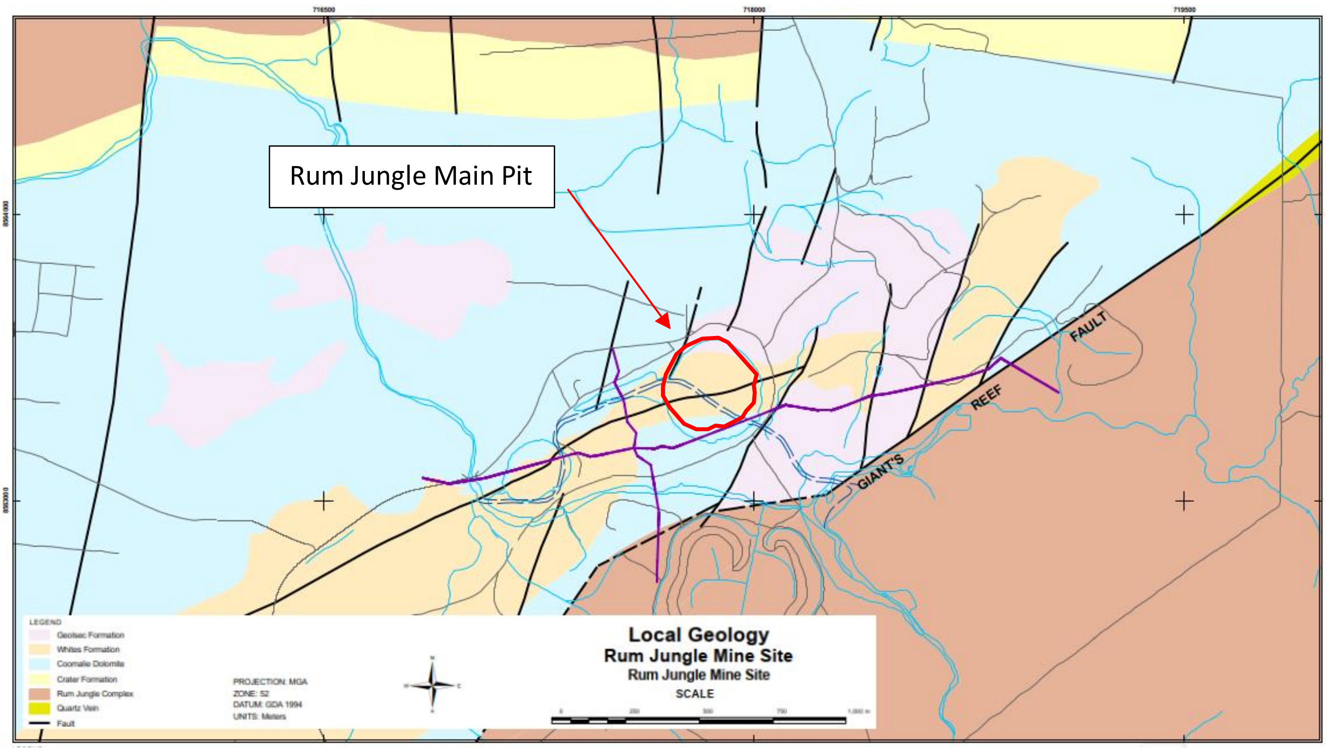
7 Main Pit Rock Mechanics Assessment

7.1 Geology

The Rum Jungle Mine Site is situated in a triangular area of the Rum Jungle mineral field that is bounded by the Giant's Reef Fault to the south and a series of east-trending ridges to the north. This triangular area is known as The Embayment and it transects a northeast-trending, southwest plunging asymmetric syncline that has been cut by northerly dipping faults (**Figure 12**).

The main lithologic units in The Embayment are the Rum Jungle Complex and meta-sedimentary and subordinate meta-volcanic rocks of the Mount Partridge Group. The Rum Jungle Complex consists mainly of granites and occurs primarily along the south-eastern side of the Giant's Reef Fault, whereas the Mount Partridge Group occurs north of the fault and consists of the Crater Formation, the Geolsec Formation, the Coomalie Dolostone, and the Whites Formation (**Figure 12**).

Figure 12 Rum Jungle Geological Setting – Reproduced from Robertson GeoConsultants (2012) [13]



The site is variably overlain by a residual soil / extremely weathered rock profile to depths of up to 15 m consisting of:

- Laterite, developed by intensive and long-lasting weathering of the underlying bedrock into an iron-rich oxidized pelloids profile, clayey silt to silty gravel and;
- Saprolite, decomposed and chemically weathered (less than laterite) bedrock, clay and silt rich, containing trace structure and texture that were present in the original rock.

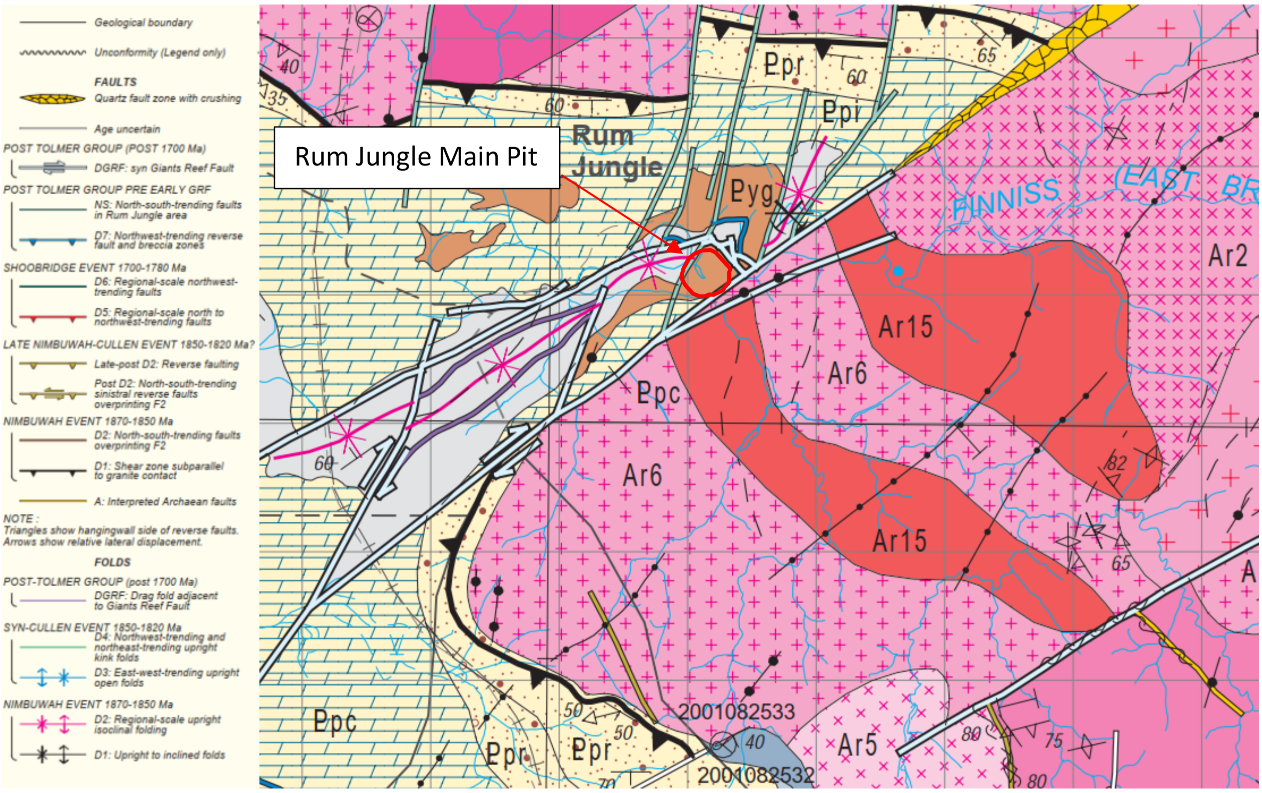
The bedrock sequence within the main pit form part of the Partridge Group and include (in order of increasing age):

- Geolsec Formation, sedimentary deposit of hematite-quartz breccia (HQB); mainly quartz clasts in an amorphous hematite matrix;
- White's Formation (aka Golden Dyke Formation), sedimentary carbonaceous shale to metamorphosed schist with tremolite, graphite and pyrite. In places it is described as a quartz-sericite material with a strong foliation which has been subjected to at least two generations of later micro folding; and
- Coomalie Dolostone, sedimentary carbonate rock, karstic in places, comprising mainly dolomite, magnetite and calcite; can be in a saccharoidal or crystalline form to a more hematized and silicified or even brecciated form closer to the thrust fault zones.

1.1.1 Structural Geology

The main pit ore body was located on the northern limb of a tightly folded syncline (**Figure 13**), on the contact between the Coomalie Dolostone and the Whites Formation. The syncline is juxtaposed against the Giants Reef Fault south east of the main pit which defines the boundary between the Partridge Formation and the Rum Jungle Complex Intrusive Suite.

Figure 13 Rum Jungle 1:100:000 Geological Interpretation [30]

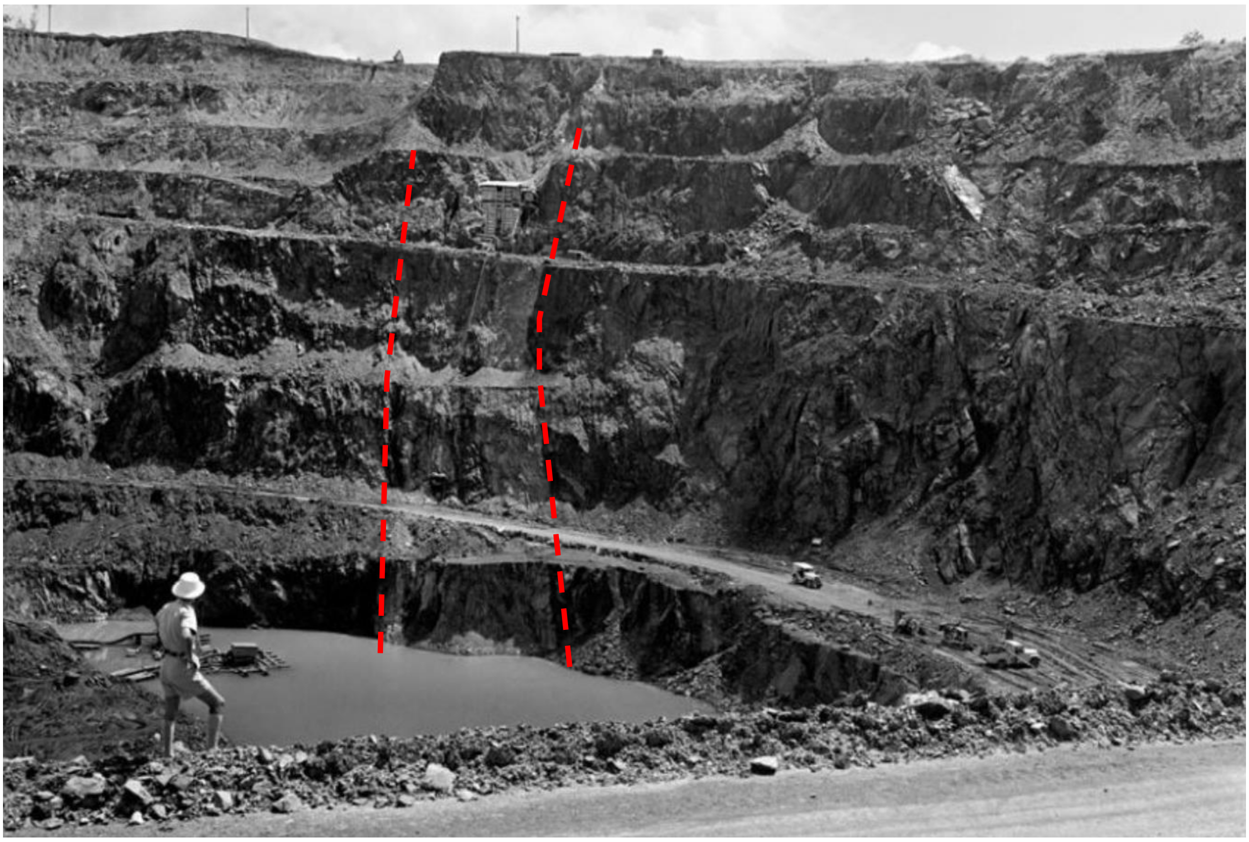


A series of north-north-east trending, steep and inferred westerly dipping faults associated with tectonic shattering cross-cut the syncline. A number of the contacts between the rock units of the Partridge Group are bound by these faults.

The northern limb of the syncline (and possibly the axial plane) is characterised by an intensely sheared east-north-east trending zone, known as the main shear zone (**Figure 14**).

A series of north east trending shear zones associated with this shearing are thought to be associated with the Giants Reef Fault and cross-cut the syncline.

Figure 14 Photograph of White's Open Cut (Main Pit) in 1958 looking north-east along strike of the main shear zone (photo courtesy of National Archives of Australia:A1200,L25494 [8]



According to Williams (1963) [7], no detailed geological structural analysis was carried out in the Main Pit at the time of operations. The structure around the periphery is very complex, having been subjected to at least four generations of movement with brecciation in the later stages.

Williams identified the predominant foliation in the Whites Formation (referred to as the Black Slate of the Golden Dyke Formation) is characterised by a strong foliation (S1) which is folded by at least two generations of micro-folding, one set which is isoclinal and transposes S1 and the other much more open with an incipient axial plane cleavage. In parts the formation is brecciated, and fragments re-cemented by quartz and sulphides. Observations within the pit suggest the foliation had been rotated about the microfolds with up to three generations of folding and associated over-printing of cleavage creating a complex structural relationship.

Boulders supposedly from the main shear were also described by Williams [7] as a green, very well foliated schist, with foliation crenulated as a result of multiple stages of deformation. Williams interpreted the zone as characteristic of a zone of movement which is potentially parallel to the axial surface of a generation of folds and that the main shear is over printed by younger stage deformation events (i.e. the north trending faults).

The dolostone was noted to be well foliated in places and contained bands of opaque minerals which were isoclinally folded. Elsewhere it is re-crystallised into a coarse-grained, granoblastic marble.

Later interpretation of the Rum Jungle Region Structural Geology by Lally (2003) [30], details the interpreted deformation sequence and is summarised relevant to the Rum Jungle Main Pit below:

- D1 – Shearing sub-parallel to granite contact
- D2 – Regional scale upright isoclinal folding (potential development of S1 foliation), formation of main Rum Jungle Syncline
- D3 – North-South trending faults and North West trending Reverse Faults and breccia Zones in Rum Jungle area
- D4 – Giants Reef Fault, and associated north east trending shear zones and drag folds

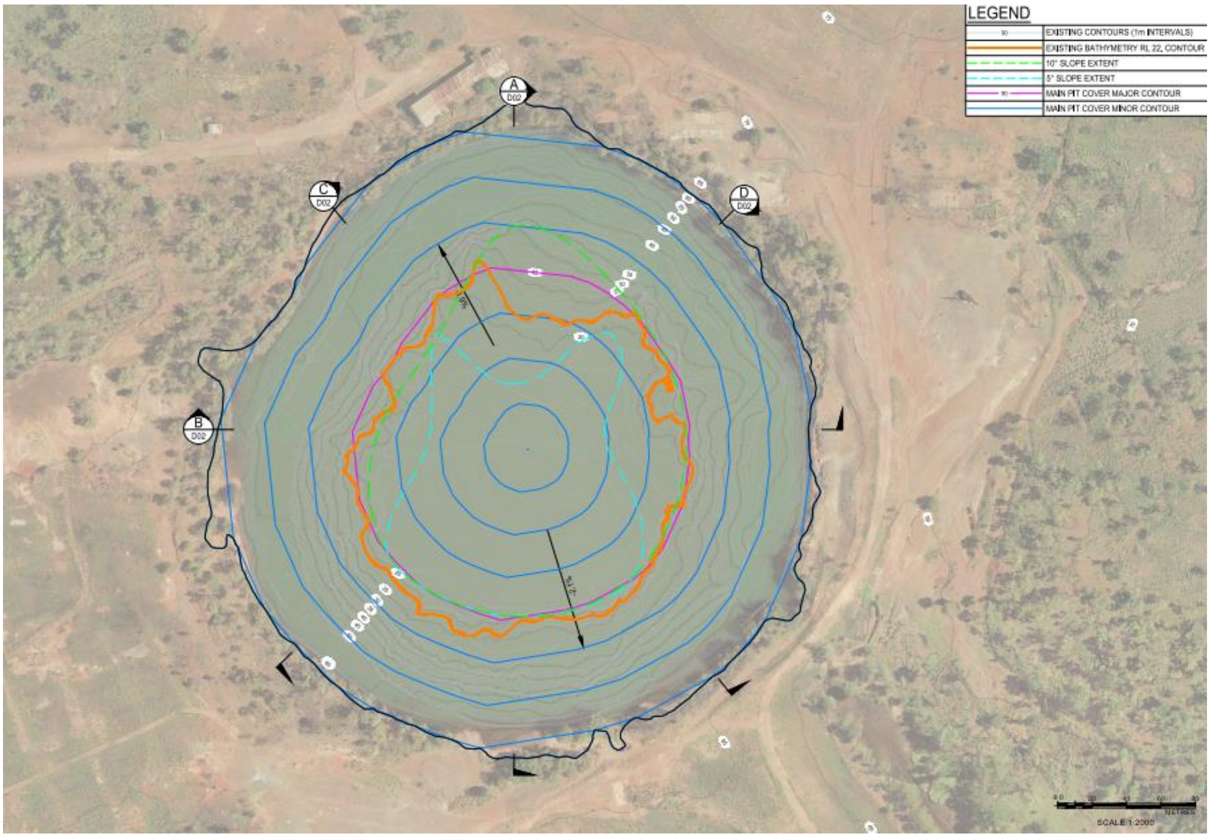
7.2 Pit Geometry

A bathymetrical survey of the main pit was completed as part of the Main Pit Backfill remediation concept assessment in 2015 [16] and was interpreted by SRK [22] (**Figure 7**). From the survey data, the upper half-spiral of the former haul road appears to be relatively intact. Below this, the original haul road is covered by backfill and scree.

The former benches are also mostly indistinguishable now, due to filling-in resulting in relatively uniformly sloped pit walls with overall slope angles ranging from between 25° to 30° in the mudstone and 28° to 38° in the slate. Flattening of the pit walls since mining suggests that the slope has deteriorated to slopes which are indicative of the angle of repose of degraded host materials.

The current surface of the submerged backfill is deepest in the southern areas and is mapped at an approximate RL 16 m RL. Bathymetric survey shows the central backfilled pit surface dips gently towards the south (<5 degrees) and everywhere towards the submerged perimeter becomes progressively steeper towards the former pit walls, up to about 15°. The slight slope increase over the central and southern areas is consistent with fine-grained tailings sedimentation, whereby progressively finer components drop out of suspension with increasing distance from the discharge point and associated beach deposits flanking the northern portion of the pit. Using similar principal to fine grained soil subaqueous depositional environments, a boundary of where the bathymetric slope becomes 5° to 10° has been mapped outwards from the deepest point of the main pit bathymetric surface to reflect the possible extent of the tailings (**Figure 15**).

Figure 15 5° and 10° Slope Extent from Main Pit Centre



The geometry of the beach fan in the north (**Figure 7**) indicates coarse-grained material rather than tailings, which is consistent with test results in this area.

According to the Department of Transport and Works, un-neautralised tailings were sub-aqueously discharged at the northern perimeter of the Main Pit from 1965 to 1971 [6]. There is anecdotal evidence that soil from the Old Tailings Dam and waste rock was end-dumped into the pit during the rehabilitation works which likely accounts for all or some of the depositional cones observed from survey data [6]. There is evidence of an end-dumped fill zone on the eastern side of the pit which pre-dates the tailings deposition (covered by tailings at the toe). A scree cone on top of the tailings to the east is inferred to be due to failure of materials in the main shear zone after tailings deposition. The bathymetrical survey 3D surface reproduced against the existing surface model is shown in **Figure 16**. Interpretation of the buried pit walls has been made based on review on historical photographs (**Figure 17**) and the assumption inter-ramp benches are relatively indistinguishable, whilst the upper ramps (haul roads) remain relatively intact.

Figure 16 Oblique View of Main Pit 2015 Bathymetry Survey

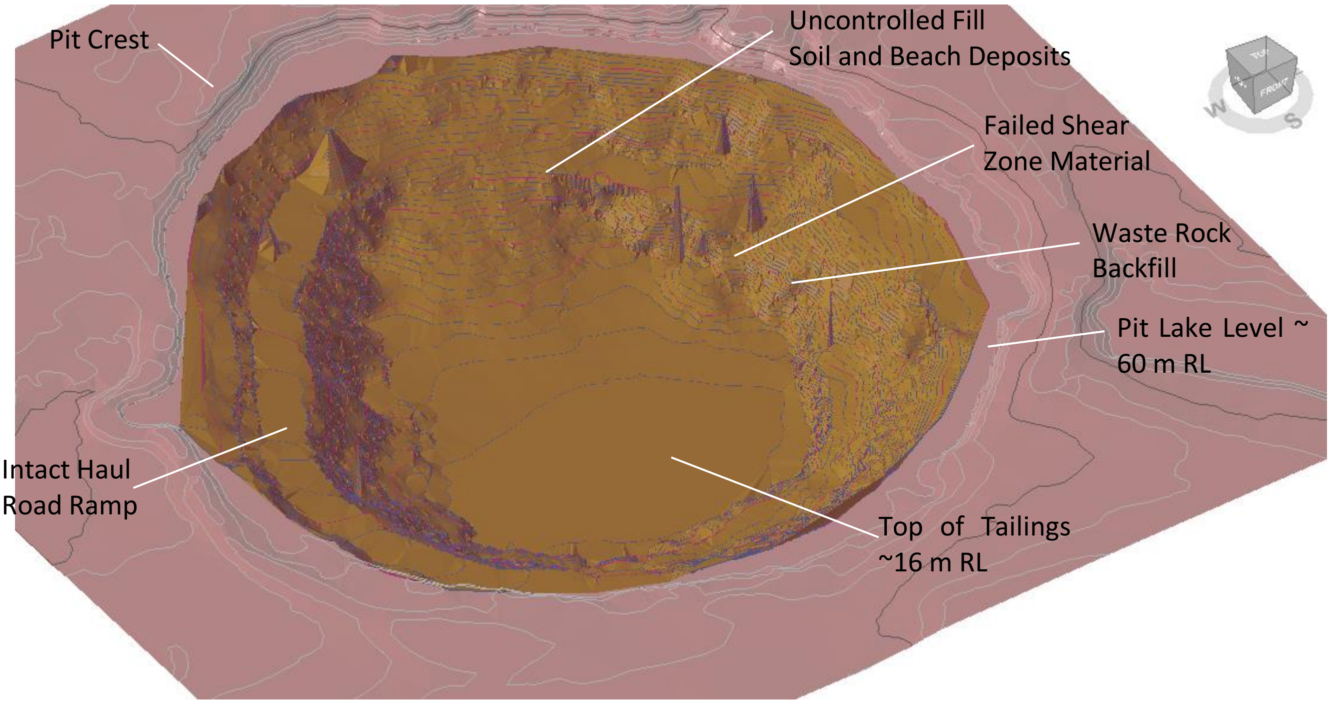
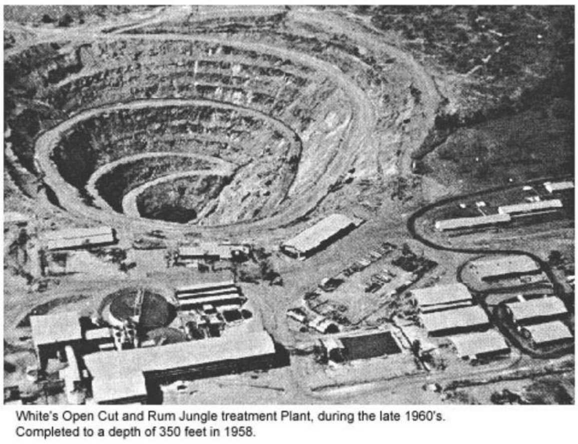


Figure 17 Historical Photographs

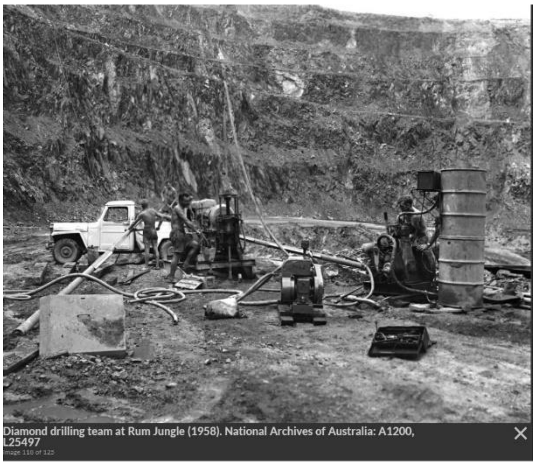


Completed Main Pit 1960s Looking South East

Image Courtesy: World Nuclear Association, https://www.world-nuclear.org/information-library/country-profiles/countries-a-f/appendices/australia-s-former-uranium-mines.aspx, Accessed 19 Feb 2020

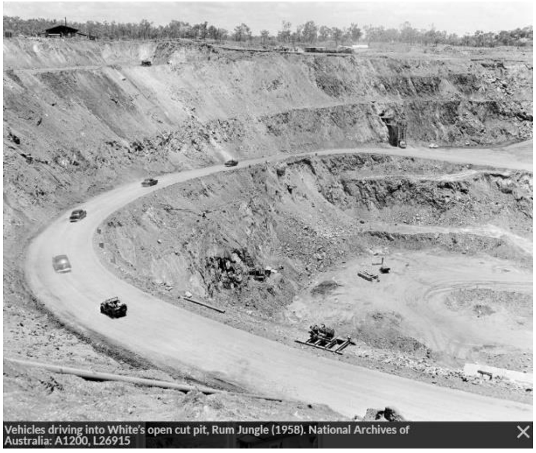
Main Pit 1958 Looking South East

Image Courtesy: DPIR, https://dpir.nt.gov.au/mining-and-energy/mine-rehabilitation-projects/rum-jungle-mine/photo-gallery. Accessed 19 Feb 2020.



Pit Wall Conditions Near Pit Completion in 1958

Image Courtesy: DPIR, https://dpir.nt.gov.au/mining-and-energy/mine-rehabilitation-projects/rum-jungle-mine/photo-gallery. Accessed 19 Feb 2020.



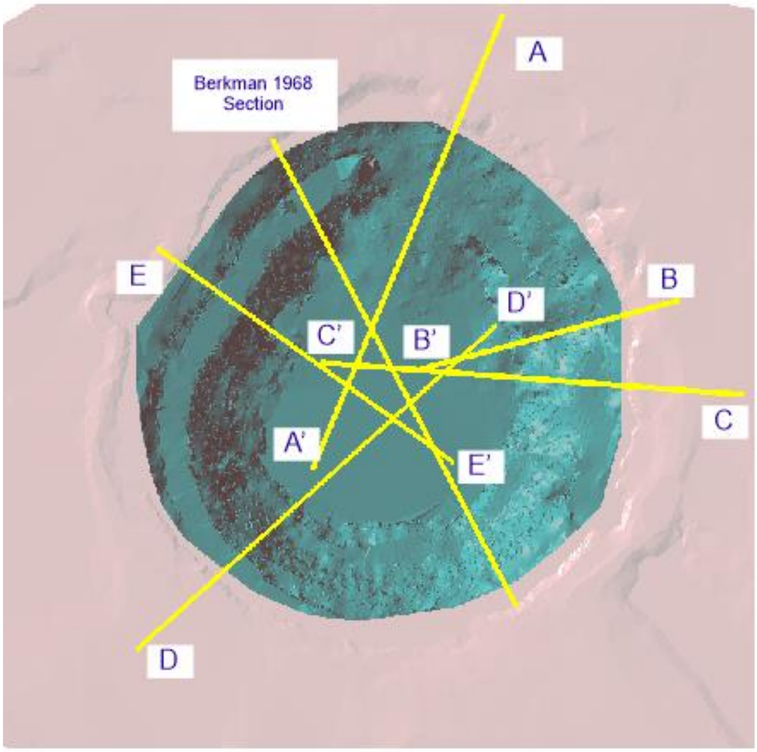
Pit Wall Conditions Looking West (estimated)

Image Courtesy: DPIR, https://dpir.nt.gov.au/mining-and-energy/mine-rehabilitation-projects/rum-jungle-mine/photo-gallery. Accessed 19 Feb 2020.

7.3 Geotechnical Model

In order to assess the stability aspects of the main pit rehabilitation activities, a geotechnical model was derived from the available information. A series of geotechnical sections (characterising the various rock types and their orientation with regard to the Pit Geometry have been developed. The sections **(Figure 18**), are included in **Appendix B.**

Figure 18 Main Pit Section Lines



7.4 Rock Mass Units

The majority of geological and rock mass information available is in the form of historical reports, field mapping and observations and more recently, intrusive investigations for rehabilitation and monitoring purposes (i.e. in the form of groundwater investigations and well installations). As mentioned in **Section 5.1.1,** three HQ diamond core boreholes were drilled around the western rim of the main pit to depths between 50.7 and 86.7 m. The boreholes were supervised by SRK in 2018 [22] and provide some detail on the rock mass type, strength, weathering, alteration and discontinuity (defect) orientation and character. Orientation of the core was limited to the more competent Dolostone and generally not achieved within the Whites Formation of Shear Zone due to poor rock core condition. The boreholes were drilled at inclinations of 60 – 70^{o} south-east and north-west, presumably targeting geological structures.

7.4.1 Whites Formation

Generally, where the Whites Formation was encountered during investigations, the rock consisted of Schist, Shale, low grade Meta-Sediments and minor Dolerite. Intact rock strength estimation and point load tests indicate the rock strength is variable, ranging from <5 MPa to 40 MPa, and locally up to 70 MPa. The recovered core appears highly fractured, sheared and was often recovered as rubble. The intense fracturing of the core is likely to have resulted in no core orientation within these zones (e.g. 18DH02). Description of the defect alpha angle (relative dip), roughness and nature of infill has been recorded in most cases which allows for some structural interpretation.

The predominant defect feature apparent within the Whites Formation rock is the presence of a well-defined foliation, with surfaces generally described as smooth, slightly weathered with minor infill. The foliation partings are often broken by drilling indicating little to no tensile strength. Secondary joints, veins, shears and brecciated zones are also common, specifically within the main shear zone where the rock mass is highly disintegrated and a number of 'major structures' are noted.

Given the generally well-developed foliation within the Whites Formation, in sections where the rock mass is more 'intact' (i.e. away from the main shear zone / faults, slope stability is likely to be dictated by kinematic mechanisms. The predominant mechanism would be expected to consist of plane failure along foliation surfaces which dip out of the face, with fretting and toppling likely where high angle foliation planes dip into the face. Additionally, wedge style failures would be expected where the intersection line of defects dip out of the face.

As there is no orientation data from the drill core, and limited description from historical records of the foliation dip and dip direction available, it is difficult to assess these mechanisms with much accuracy. In the drill core, the alpha angles of the main foliation partings vary between 30 – 70^{o}, but on average are generally recorded as 40° – 50^{o}, which could indicate a true dip of 60° – 70^{o} assuming the foliation is steeply dipping.

Historical observations and reports suggest the foliation may be sub-parallel to the axial hinge of regional folds and possibly remnant of original bedding planes. This suggests the S1 foliation is likely to be steeply inclined and sub parallel to fold limbs. Later stage micro-folding events (as described by Williams, 1963 [7]) may then be responsible for open folding of the S1 foliation which creates the variation in dip angle.

The behaviour of the Whites Formation is also complicated by the frequent, intense shearing and fracturing associated with late stage brittle deformation. In these zones, rock mass style failure is expected, whereby failure occurs 'through' the rock mass rather than on well-defined defect surfaces. As such, rock mass strength parameters have been estimated using the Hoek & Brown [31] failure criterion to provide equivalent Mohr Coulomb parameters for analysis, as outlined in **Section 7.7**.

7.4.2 Coomalie Dolostone

The dolostone was encountered in 18DH01 and 18DH03, in the north west and south west pit rim, respectively. The rock is described as high to very high strength with intact rock strength from field observations and point load tests from 50 - >150 MPa. Given the competent nature of the Dolomite, the orientation of the recovered core from boreholes was able to be measured and hence the alpha (relative dip) and beta (relative dip direction) were recorded for most defects. Additionally, defect condition including roughness, weathering and nature of infill has been recorded.

Generally, the rock mass was described as having poorly defined to indistinct fabric, with high quartz content and was largely crystalline, medium to coarse grained and in 18DH03 was logged as dolomitic quartzite. The rock appeared to display a number of joint sets, with surfaces described as slightly rough to very rough, with slight to no weathering. Locally, the micro-defects and healed breccias were noted. Slope instability within the Dolomite is therefore interpreted to be predominantly structurally controlled where the rock is not significantly sheared or brecciated. Refer to **Section 7.6** for the Kinematic Analysis on the Dolostone unit.

7.5 Stability Analysis

7.5.1 Previous Main Pit Stability Assessment

Robertson GeoConsultants (RGC) [6] completed an options assessment in 2016 relative to the complete backfilling of the main pit. A brief assessment was provided of the Main Pit Wall stability and potential geohazards introduced as a result of dewatering, backfilling and construction works in and around the main pit crest.

RGC outlined in report 183006/3 [6] *"Berkman (1968) states that the material in the pit is generally soft slates or intensely fractured chloritic rock. Such argillaceous rocks are particularly susceptible to swelling and softening when saturated. Observations of the walls around the perimeter of the pit by Andy Thomas of RGC in July 2015 support this; in wetted zones of the walls it was evident that the rock had softened, in some cases to a clayey soil-strength material. The exposed wall material strength ranged from very low (Coomalie Dolostone) to medium in the other units."*

A number of geohazards were identified to be present with risk relevant to the various backfill concept approaches assessed. The hazards relevant to the pit wall stability included:

- Steep backfill/scree cones founded on tailings in a meta-stable condition which could potentially be exacerbated by dewatering;
- Compressible materials which could take many years to consolidate;
- Tailings susceptible to liquefaction and sudden loss of strength;
- Unstable, low strength and highly fractured (in shear zones) pit wall materials since softened from pit flooding, susceptible to sliding and slumping;
- Clayey pit wall skin impeding dewatering of the walls leading to wall pressurization and instability; and
- High decay rate and possible solution channelling/undercutting in the Coomalie Dolostone exposed in the pit.

Construction activities, specifically in relation end dumping over the pit crest were identified as likely to present instability created by the backfill material sliding on the pit wall as well as basal failure of the tailings. Additionally, surcharge loading of the pit crest from waste rock piles and earthworks machinery would act to destabilise the crest and could cause failure of the walls. This is particularly a problem if the walls have softened or weathered.

From review of the historical photos and the bathymetry survey, it is evident that localized pit wall instability has previously occurred. The possibility of sudden crest failure near the current rim would present an unacceptable risk to operators and machinery involved in rehabilitation.

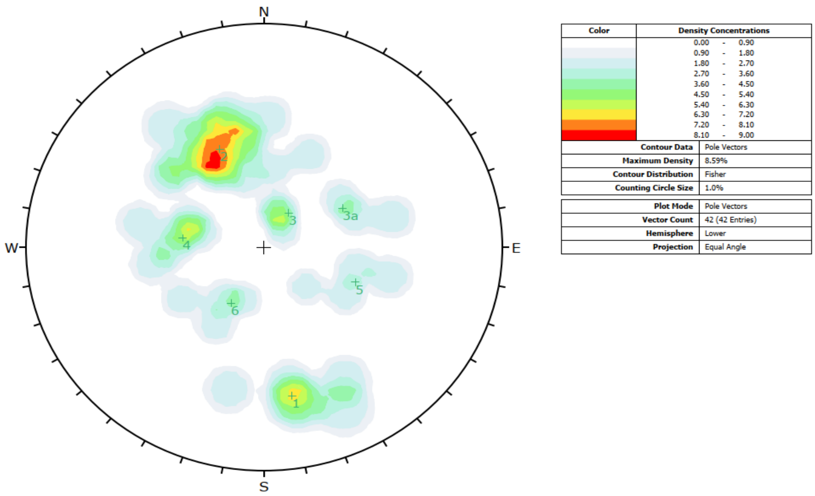
A stability analysis has been completed herein to assess the pit rim stability and inform the need for, and type of risk mitigation measures. This assessment utilises recent borehole information drilled behind the crest of the main pit and will also be used to inform construction considerations for any main pit remediation activities and backfill methodology design.

7.6 Kinematic Analysis

The orientated defect data was entered into the Rocscience software Dips 7.0 to convert the alpha and beta defect measurements into true dip and dip direction to allow for identification of the main defect sets and facilitate kinematic analysis. The development of stereo-plots using the Dips software to identify the major and minor joint sets and discontinuities (as shown in **Figure 19**), including:

- Poles
- Defect type (i.e. joints / bedding / shear zone)
- Contour – pole density Concentrations
- Assign mean sets

Figure 19 Pole Plot of Orientated Defect Data for Dolostone



A summary of the converted defect orientation data extracted from SRK boreholes is presented in **Appendix C** and a summary of the main joint sets (+/- 20°) identified is included in **Table 10** below.

Table 10 Dolostone Main Joint Sets

Name	Dip	Dip Direction	Type	Condition	Spacing (m)*	Extent
Joint Set 1	70	350	Fabric	Slightly Rough to Rough, Slightly Weathered	0.2 – 1.7	Dominant in 18DH01
Joint Set 2	50	160	Joint, breccia fracture	Slightly Rough to Very Rough, slight to no weathering	0.1 – 0.5, >4	Dominant in 18DH03
Joint Set 3 (3a)	20 - 40	210 – 240	Joint	Slightly rough to rough, slight to no weathering, no fill, <1mm	2 - 5	18DH01 & 18DH03
Joint Set 4	40	95	Joint	Rough to very Rough, slightly weathered, <5 mm, little to no fill	1.3 - >3	18DH01 & 18DH03

Name	Dip	Dip Direction	Type	Condition	Spacing (m)*	Extent
Joint Set 5	45	290	Joint	Smooth to slightly rough, moderate to slightly weathered, <1mm, minor soft to hard fill (clay, sand, chlorite)	Varies, generally widely spaced and un-common	18DH01 & 18DH03
Joint Set 6	30	30	Micro-Fracture	Slightly rough to rough, slightly weathered, <1mm little to no fill	2.8 – 3.1	Minor at base of 18DH01 and deep (~80 m bgl) in 18DH03

*based on limit number of defects within group and hence spacing may vary considerably. Data is not corrected for multiple defects per entry.

The main defect sets are used to characterise geotechnical domains related to structural features to be analysed within various aspects of the pit geometry by kinematic analysis (i.e. the exposed dolerite in the upper south-east quadrant of the pit), whereby the stability within the dolerite is governed by structures.

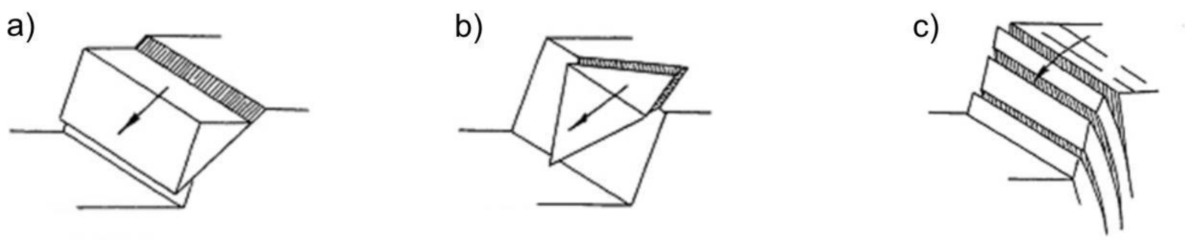
Kinematic analysis of the “Dips” data is used to identify potential rock block fall-out mechanisms in relation to pit wall geometry. Identification of the potential rock blocks and inferred shear strength characteristics of the rock mass from laboratory test data are used to determine the likely failure mechanisms and factor of safety against failure (see **Figure 20**):

Figure 20 Failure Mechanism Types

(a) Planar

(b) Wedge

(c) Toppling



The analyses take into consideration the likely failure mechanisms in relation to the proposed pit wall geometry analysis, including Factor of Safety – Risk and Likelihood of failure for:

- Individual benches;
- The overall stability of the pit walls, from crest to base; and
- The potential for localised instability within benches.

Generally, as discussed in **Section 7.7**, structurally controlled instability is expected within the competent rock mass units whereby potential slope failure occurs along unfavourably orientated defects (and/or fabric).

Therefore, the kinematic analysis is most relevant to the lower portion of the pit walls (noting that the upper pit walls i.e. 15m - 20m from the crest is generally expected to experience rock mass (rotational) style instability as outlined in **Section 7.10** and as such is not expected to present an immediate risk to remediation activities. Further detail on anticipated levels of risk in the form of a semi-quantitative risk assessment is included in **Section 7.11**.

7.6.1 South East Quadrant

Kinematic analysis of a north west facing pit slope (i.e. south-east quadrant) dipping at 60° is completed to assess the likely failure mechanisms as summarised in **Table 11** and **Figure 21** to **Figure 23** below. A conservative defect friction angle of 30° is used to represent the worst combination of surface conditions based on the borehole logs i.e. slightly rough, slightly weathered defects with minor clay/chlorite infill.

Table 11 Summary of Kinematic Analysis for South East Main Pit Slope

Mechanism	Likelihood (%)	Contributing Joint Set	Comment
Planar	7.14	Joint Set 5, possibly Joint Set 3	Potentially thick 2 – 5 m rock blocks produced, although other closely spaced cross cutting joint sets would limit blocks size. Relatively uncommon.
Wedge	10.69	Intersection of Joint Sets 1, 3, 5 & 6	Expected to be a relatively common failure mechanism, blocks generally < 1.5 – 3 m diameter
Flexural Toppling	0.00	-	Unlikely
Direct Toppling	Direct 8.25% Oblique 0.35% Base Plane 14.29%	Intersection of Joint Set 2 & 4 Intersection of Joint Set 3 & 6 Joint Sets 3, 5, 6	Potentially larger thin blocks up to 3m length

Figure 21 Planar Sliding Failure Analysis for South East Quadrant Slope

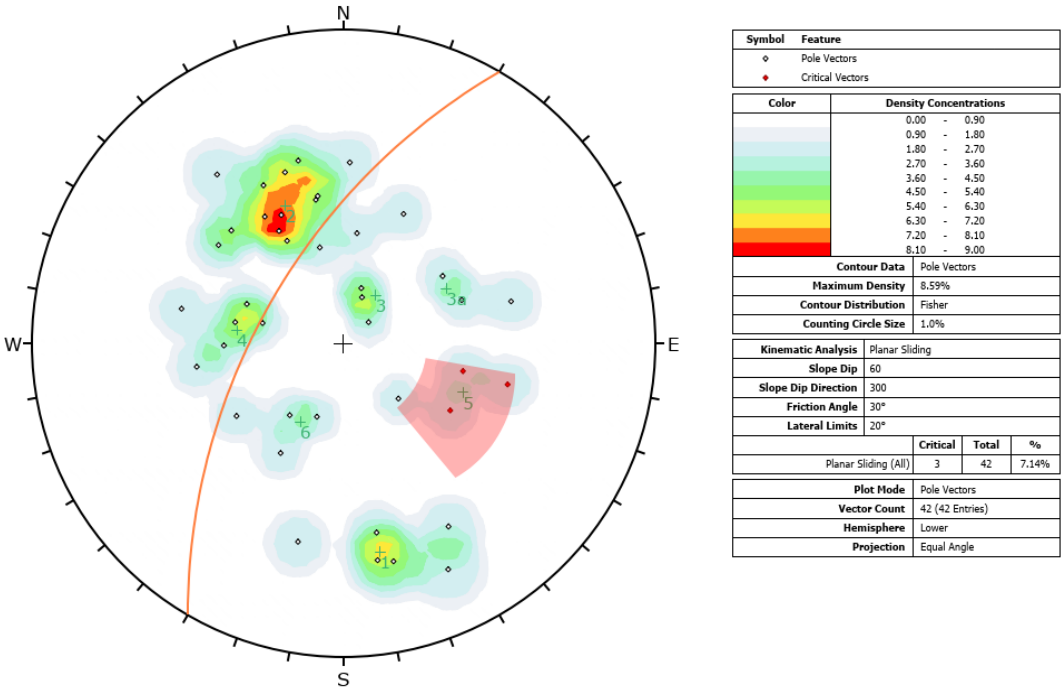


Figure 22 Wedge Failure Analysis for South East Quadrant Slope

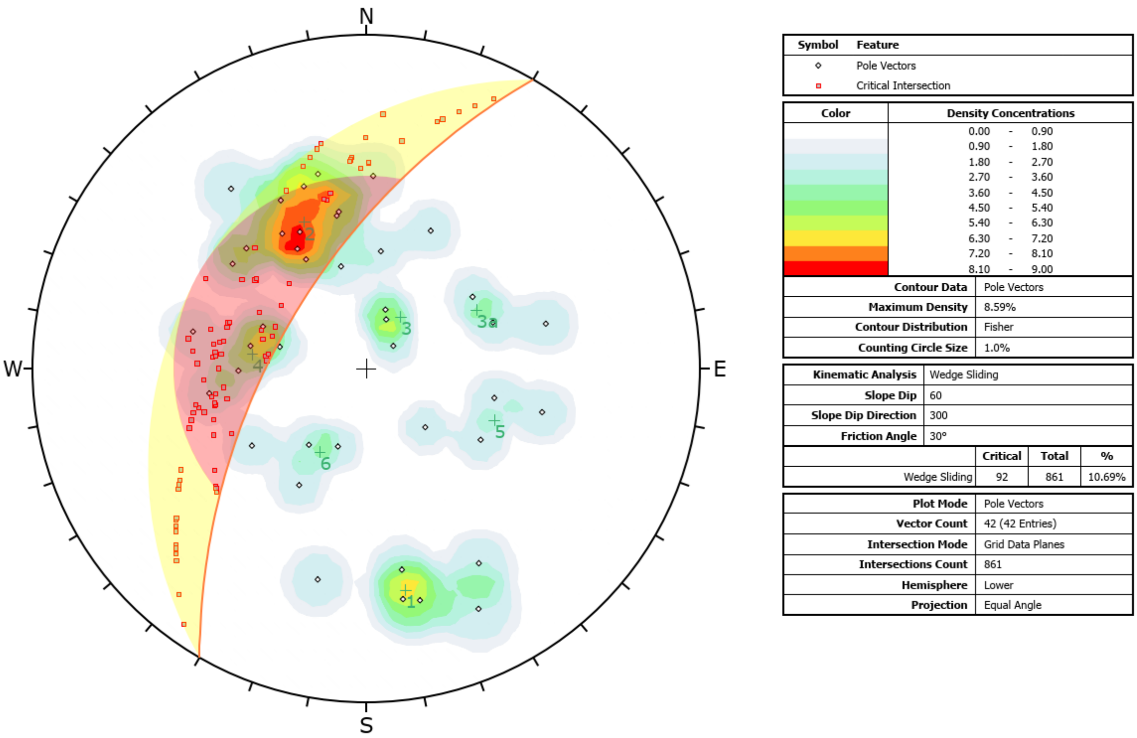
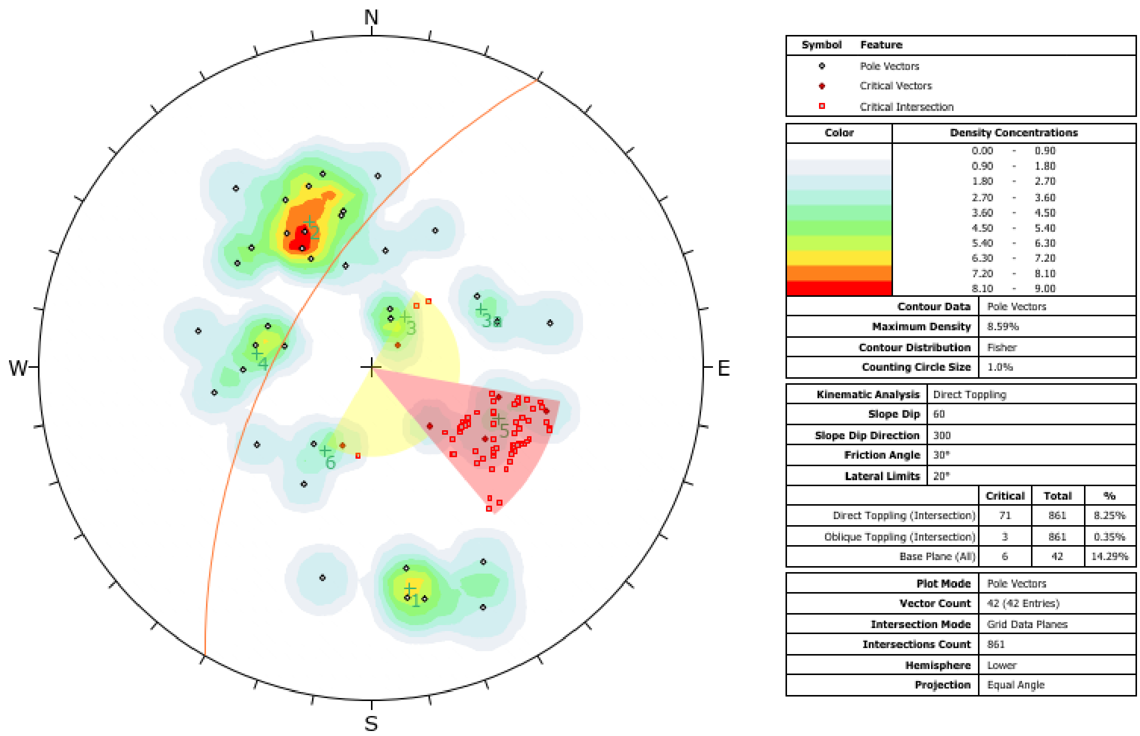


Figure 23 Direct Toppling Failure Analysis for South East Quadrant Slope



7.6.2 Summary

Specific aspects of the main pit edge stability have been assessed based on the available information (i.e. orientated defect data from the Dolostone unit) with the analysis targeting aspects where the Dolostone may be expected to form the pit wall geology, i.e. the south east portion of the pit.

The analysis indicates all three of the kinematic failure mechanisms are possible due to intersection of the various joint sets (wedge failure), and unfavourable orientation (planar and toppling failure). Defect spacing for the dominant defect sets is generally closely spaced and <1 m – 3 m and hence potential block size is expected to be limited to a maximum diameter in the order of the spacing of the relevant defect set. A summary of the typical failure blocks is included below:

- Planar: potentially 2 – 5 m thick rock blocks produced, although other closely spaced cross cutting joint sets would limit block size to less than 2 m length;
- Wedge: expected to be a relatively common failure mechanism, blocks generally < 1.5 – 3 m diameter; and
- Direct Toppling (Base Plane): potentially longer thin blocks up to 3 m length.

It should be noted the potential for large scale failure along structures (i.e. shear zones or faults) has not been completed as no detail on the orientation and nature of these structures is currently available.

Similarly, limited orientation data of defects or fabric from the White's Formation is available and hence kinematic analysis of these rock units has not been completed. As discussed in **Section 7.4.1,** given the generally well-developed foliation within the Whites Formation, in sections where the rock mass is more 'intact' (i.e. away from the main shear zone / faults), slope stability is likely to be dictated by kinematic mechanisms.

Assuming the foliation is steeply dipping i.e. >60° (and steeper than the generally assumed pit wall slope angles) the predominant failure mechanism would be expected to consist of fretting and toppling likely where high angle foliation planes dip into the face.

Further details on the potential risks associated with kinematic failures is outlined in **Section 7.13**.

7.7 Rock Mass Strength

Historical observations and interpretation of the SRK 2018 borehole data [22] indicate the main rock units making up the main pit walls are characterised by a fractured, sheared and locally disintegrated rock mass with varying degree of weathering and infill along discontinuities. Subsequent flooding of the pit appears to have contributed to weathering and softening of the exposed rock surface as evidenced by sidewall degradation although it is unclear how far such softening penetrates behind the pit wall face. Borehole data indicates the upper 15 m – 20 m of rock is variably weathered and generally overlain by up to 7 m of saprolite.

Rock mass parameters for the low strength, deformed and/or weathered rock have been estimated using the Hoek & Brown (2018) failure criterion [31] to provide equivalent Mohr Coulomb parameters.

The Generalized Hoek-Brown criterion is non-linear and relates the major and minor effective principal stresses (σ_1 and σ_3) according to the following equation:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

where:

- σ_1 and σ_3 are the axial (major) and confining (minor) effective principal stresses, respectively;
- σ_{ci} is the uniaxial compressive strength (UCS) of the intact rock material (estimated from field descriptions and point load tests on rock samples by SRK);
- mb is a reduced value (for the rock mass) of the material constant mi (for the intact rock); and
- s and a are constants which depend upon the characteristics of the rock mass.

In most cases it is difficult to carry out triaxial tests on rock masses at a scale which is necessary to obtain direct values of the parameters in the Generalized Hoek-Brown equation. Therefore, some practical means of estimating the material constants mb, s and a is required. According to the latest research, the parameters of the Generalized Hoek-Brown criterion [31], are given by the following equations:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

where:

- **GSI** (the Geological Strength Index) relates the failure criterion to geological observations in the field;
- **mi** is a material constant for the intact rock; and
- the parameter D is a "disturbance factor" which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and/or stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. D was taken as 0.5 in all cases to represent some disturbance to the rock mass from historical blasting.

It is important to acknowledge the impact of defect orientation with respect to pit slope geometry. Given the very limited structural information in all rock mass units except for the Dolostone this is inherently difficult to assess. As such, the Hoek-Brown interpretation has been utilised without specific consideration on defect orientation, rather that the stability of the majority of the rock mass within the main pit is expected to be controlled by rock mass properties, due to the interpreted low geological strength index rather than along specific defect sets whereby the later would be analysed using a kinematic analysis.

It is apparent that the Whites Formation contains a distinct fabric (foliation), which is considered steeply inclined. Where the fabric dips out of the face of a slope sliding along foliation planes would be the most likely failure mechanism. Similarly, where the foliation dips steeply into the slope toppling of rock blocks along foliation would be the most likely failure mechanisms.

In this case, a kinematic analysis has only been possible for the Dolostone unit (refer **Section 7.6**) due to the lack of any reliable orientation within the Whites Formation.

The borehole logs, defect descriptions and engineering judgement from review of historical photographs of the exposed rock mass to develop the rock mass strength profiles, which are summarised in **Table 12**. The Hoek Brown classification outputs are included in **Appendix D.**

Table 12 Rock Mass Strength Parameters

Rock Mass Unit	Typical Intact Rock UCS (MPa)	Geological Strength Index	Rationale	Mi (based on rock type)	Intact Modulus (based on modulus ratio) [MPa]	Mohr Coulomb Fit	
						C′ (kPa)	Φ (°)
Whites Formation Shale, Schist	25 (3)	25 (20)	Laminated/sheared rock mass with fair to poor surface quality (smooth/slickensided to slightly rough defect surfaces) Foliated/sheared rock mass with poor surface quality (smooth to slightly rough, weathered defect surfaces)	6 -shale	5,000 (600)	48 (12)	37 (24)
Main Shear Zone	1.5	18	Sheared rock mass with closely spaced shear planes, poor surface quality, (slickensided to smooth defect surfaces)	10 - schist	1,012	10	22.5
Geolsec Formation Quartz Breccia	15 (1)	30 (25)	Hematite Quartz Breccia, brecciated, disintegrated rock mass with fair to poor surface quality (weathered, clay filled)	14 - breccia	7250 (290)	61 (13)	43 (26)
Coomalie Dolostone	(3)	(30)	Highly weathered, softened dolomite	9 - Dolomite	1275	(20)	(32)

() denotes highly weathered/softened rock mass parameters

7.7.1 Model Limitations

Given the overall lack of geotechnical data, specifically with regard to laboratory testing (triaxial and unconfined compressive strength), cautious strength parameters have been adopted. Furthermore, the relatively weak, weathered and shallow depth of rock masses near the critical pit rim areas, are near the lower limit of reliable strength derivation using the Hoek-Brown criteria, requiring additional cross-checking of the above shear strength parameters with back analysis of the existing pit slopes.

7.8 Seismic Conditions

Guidance on the selection of appropriate seismic events for slope stability applications s provided by AS1170.4 (2007) Structural Design Actions – Earthquake Actions in Australia and by the Australian National Committee on Large Dams (ANCOLD) [32].

Figure 24 below shows the published Hazard Factor (Peak Ground Acceleration, PGA) map for Northern Territory, and indicates that a PGA of 0.085 is appropriate for a 1 in 500 return interval earthquake event which is commonly adopted for design of Australian infrastructure assets.

Figure 24 Hazard Factor (PGA) Map for Northern Territory [32]

AS 1170.4—2007 24

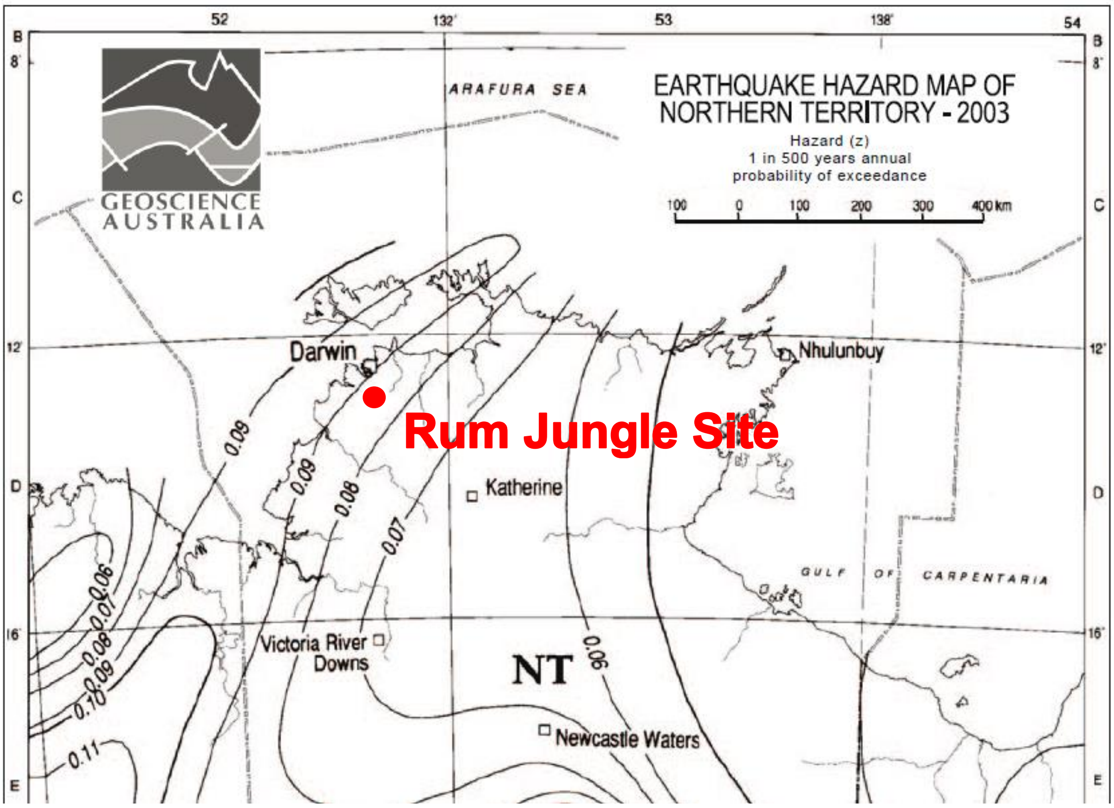


Image courtesy of Australian Standards AS1170.4-2007.

Based upon the above and SLR's experience of carrying out slope stability and dam hazard assessments and taking into account the proximity of the site to seismically active regions whilst considering that a slope failure would not create a flood hazard that could travel large distances downstream, a 1 in 500-year return period for the design event has been selected.

Destabilising seismic acceleration forces are added to the Limit Equilibrium (LE) analyses, these forces being based upon the predicted PGA for the site for the design seismic event. During a seismic event, both destabilising and stabilising forces will be present since the ground motions are both positive and negative. It is therefore standard practice for the PGA to be reduced within a pseudo-static analysis using a ratio known as the seismic coefficient. According to Hynes-Griffin and Franklin, considering the inertial effects, half of the PGA value should be used as the recommended horizontal seismic coefficient in a limit equilibrium analysis. Vertical acceleration is taken as zero as per normal practice.

The horizontal seismic coefficient value adopted for the purposes of the current assessment is shown below:

1:500 AEP event correlates to a PGA of 0.085 [33] with a corresponding horizontal seismic coefficient of 0.0425 adopted for pseudo-static analyses.

7.9 Performance Criteria

SLR is not aware of any set guidance requirement in the Northern Territory for minimum factors of safety (FOS) for slope stability applications, factors of safety have been established based on internationally accepted guidance and similar stability assessments of projects.

In relation to the FOS under seismic loading, reference can be made to standard practices for dams and reservoirs. ANCOLD Guidelines on Tailings Dams (2012) [34] indicates recommended minimum factors of safety for tailings dams as 1.0 to 1.2 for pseudo-static loading conditions.

For the purposes of this report, the following FoS design criteria have been adopted:

- Permanent slopes and static analyses: FoS ≥ 1.5
- Temporary slopes and transient surface loading: FoS ≥ 1.2
- Earthquake analyses and liquefaction events: FoS ≥ 1.1

Given the temporary condition of the slopes during remediation activities seismic induced pit wall instability has not been considered in this analysis. A liquefaction assessment under the guidelines outlined above is not required for the Main Pit side slopes but has been considered in the backfilling strategy hazard management.

7.10 Limit Equilibrium

Limit equilibrium stability analyses of the upper section of the Main Pit wall has been undertaken near the elevation of pit rim areas which are partially submerged but not currently covered with backfill section. Relevant cross-sections shown in **Appendix B** have been assessed using the GeoStudio Slope/w software program. The geotechnical model presented in **Section 7.3** forms the basis for analysis, with conservative parameters developed based on interpretation of the rock units described in historical reports and that recovered from more recent geotechnical drilling (SRK [22]). The material parameters used in the analyses are presented in **Section 7.10.2** and other relevant assumptions are discussed below.

Within this report, the two predominant slope stability scenarios considered are the stability of the submerged upper pit slopes prior to and during backfilling and the partially submerged upper pit wall slopes during and post back filling which may be subject to seasonal or operational variation in pit lake levels, including possible rapid draw down events.

It should be noted that the rock mass stability of the lower pit wall slopes below +15 m RL (i.e. the tailings backfill level) were screened out of the analysis as part of the risk assessment process. As such, instability of backfilled pit wall sections is not expected to present an un-acceptable level of risk to the remediation objective. Settlement and stability of the in-situ tailings under backfill loading and fill slopes are discussed separately in **Sections 9.5** and **9.6** respectively.

7.10.1 Modelling Method and Assumptions

The stability assessment undertaken represents the scenarios and the different geotechnical units of the upper main pit slopes under both short and long-term conditions.

Methods used in this Stability Assessment include:

- Limit Equilibrium (LE) stability analyses for the derivation of factors of safety for the existing 'unconfined' pit wall side slopes.

The modelling has been undertaken using conventional LE stability methods within the package Slope/W (GEO-SLOPE International, 2018) and adopting the Bishop and Morgenstern-Price methods for circular and non-circular forms of analysis.

The programme requires the input of unit weight (y) and shear strength characteristics of the materials present within the analytical cross section(s). In addition, a definition of pore water pressure conditions is required. In this case, the pore water pressures conditions are represented by the pit lake level represented in the analysis.

The following assumptions have been made:

- The shear strength of the soil and weathered rock mass can be described using the Mohr-Coulomb shear strength parameters of effective cohesion, c' (in kPa) and effective angle of shearing resistance, ø' (in degrees) for static and long-term conditions.
- Shear strength of residual soil (Saprolite / Laterite) is based on triaxial testing completed by O'Kane [17] on the Saprolite properties for re-use as engineered fill of waste rock dumps in addition to recent sampling and characterisation by SLR [26]. A cautious value has been selected, given the very low anticipated strength characteristics of pit walls.
- For transient loading such as plant loading, earthquakes and rapid draw down conditions in cohesive soil materials, undrained shear strength (Su) is adopted.
- Rapid draw down cases consider effective stress conditions in granular soil materials and undrained strength parameters in cohesive materials.

Rock slope stability is typically controlled by dominant structures/fabric within the rock mass, however, given the soil mantle and relatively deep weathering profile with secondary softening, a rock mass (rotational) style failure is considered feasible.

7.10.2 Design Parameters

The following have been assessed as inputs for the analyses undertaken for this Stability Assessment:

- Material unit weight.
- Drained shear strength of soil and rock.
- Undrained shear strength of cohesive soil.

The adopted material strength parameters are summarised in **Table 13**, noting that further discussion of tailings properties is also provided in **Section 8**.

Table 13 Material Parameters

Material	Bulk Unit Weight, Y (kN/m3)	Effective Cohesion, c' (kPa)	Friction Angle, ϕ (°)	Undrained Shear Strength, Su	Material Type, Consistency or Density
Uncontrolled Fill (Soil & Beach Deposits)[2]	18	1	28	35	Cohesive, fine grained, firm
	20	0	30	-	Granular, coarse grained, very loose to loose
Uncontrolled Fill (Tailings)[2]	16	0	26	Su = 0.5+ 0.25 x P'	Cohesive, normally consolidated
Backfilled Waste Rock[2]	18	0	32	-	Waste Rock (WR) placed as backfill during past mining / remediation activities. Future WR backfill and Capping materials assumed to have same geotechnical properties.
Saprolite / Laterite	18	5	25	30	Clay rich decomposed material
Highly Weathered Geolsec Formation Quartz Breccia	22	13	26	-	Highly weathered hematite – quartz breccia, with clay
Geolsec Formation Quartz Breccia	24	61	43	-	Intact Hematite Quartz Breccia
Highly Weathered Whites Formation	22	12	24	-	Shale, Schist, minor Quartzite, some shearing and brecciation, weathered / softened with clay infill
Whites Formation	24	48	37	-	Shale, Schist, Meta-Sandstone, some shearing and brecciation
Main Shear Zone [1]	22	10	22.5	-	Highly sheared, disintegrated Shale, Schist and Meta-Sandstone
Highly Weathered Dolostone	22	20	32	-	Highly weathered / softened Dolomite
Coomalie Dolostone	25	High Strength Material			Dolomite

[1] Determined from back analysis

[2] Refer to **Section** 8 for derivation and discussion of soil parameters.

7.10.3 Stability Back Analysis

A back analysis has been undertaken on the assumed pre-failure geometry where the large potential slope failure identified by SRK [22] is intersected. The geometry assumes an overall slope angle of 40° based on interpretation of the adjacent pit wall geometry. Reiterations of the analysis using assumed shear strength parameters are completed until a FOS of 1 for failure along a large full batter slip plane similar to that identified from the pit bathymetrical survey [16].

The shear strength parameters determined by back analysis suggest the initially interpreted parameters determined using the Hoek Brown method for the shear zone material appeared conservative, producing an initial FOS < 1. Calibration to the stable existing field condition (FOS ≥ 1) was achieved by increasing the friction angle from an initially predicted 16° to 22.5°, whilst keeping the cohesion and unit weight consistent.

The parameters determined from back analyses at representative locations were similarly used to calibrate the Hoek Brown as shown in **Table 13**.

Table 14 Summary of Calibrated Back Analysis for Static Conditions

Scenario	PWP Conditions	Critical Rock Mass Units	Approximate Overall Slope Angle (°)	Analysis	Factor of Safety	Comparison with adopted performance criteria
Section B-B', Static (Back Analysis)	Pit Lake at RL 60 m	Main Shear Zone	40	Drained	1.00	N/A

7.10.4 Static Long-term Results

The limit equilibrium analyses for the Main Pit wall(s) has been undertaken using the Morgenstern-Price half-sine form of analysis which considers the various wall geometry and geology comprising the geotechnical model outlined in **Section 7.3**.

The inferred 'normal' groundwater level representative of typical wet/dry season levels is taken as approximately 60 m RL. It should be noted that typical seasonal wet/dry variation and expected fluctuations of groundwater (and pit lake) levels are to be expected (ref: **Section 4.4**). The long-term condition has been assessed assuming a static water level within the pit prior to any dewatering or construction activity.

By using the back analysis completed for the main shear zone failure as a calibration the material shear strength parameters determined using the Hoek Brown method appear to under estimate the materials effective angle of shearing resistance, however the resulting conservative assessment of pit wall stability is considered reasonable given the limited geotechnical data at hand.

The summary of the stability analysis for the pit wall side slopes under static conditions are presented in **Table 15**. Selected Slope/W stability plots are presented in **Appendix E**.

Table 15 Summary of Stability Analysis for Static Condition

Scenario	PWP Conditions	Critical Rock Mass Units	Approximate Overall Slope Angle (°)	Analysis	Factor of Safety	Comparison with Adopted Performance Criteria
Berkman 1968 Section, Static	Pit Lake at RL 60 m	Saprolite overlying HW Geolsec Fm	29	Drained	1.59	Acceptable >1.5
Section A-A', Static	Pit Lake at RL 60 m	Saprolite overlying HW Geolsec Fm – Buttressed at toe by Beach Deposits	35	Drained	1.09	Unacceptable < 1.2
Section B-B', Static	Pit Lake at RL 60 m	Saprolite overlying Main Shear Zone	35	Drained	1.14	Unacceptable < 1.2
Section C-C', Static	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm – Buttressed at toe with waste rock backfill	66	Drained	1.14	Unacceptable < 1.2
Section D-D', Static	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm /Coomalie Dolostone	20 – 30	Drained	1.45	Tolerable > 1.2
Section E-E', Static	Pit Lake at RL 60 m	Saprolite overlying HW Geolsec Fm	30	Drained	1.58	Acceptable >1.5

Under the adopted factor of safety criteria, some existing pit rim slopes which are near natural angle of repose are below the long-term target FOS (1.5) by some margin. Generally, where highly weathered material is exposed in slopes of less than 35° a tolerable FOS under short term conditions is achieved, with the exception of potentially semi-stable shear zone material which presents an unacceptable FOS under short term conditions even within relatively shallow slope angles.

Where overall slope angles are steeper than 35° an unacceptable FOS is generally attained/expected, and the FOS generally approaches or is less than 1.1.

7.10.5 Transient Loading Results

The summary of the stability analysis for the pit wall side slopes under transient loading conditions are presented in **Table 16**. The model results for the analysis are presented in **Appendix E**.

Table 16 Summary of Stability Analysis for Transient Loading Conditions

Scenario	PWP Conditions	Critical Rock Mass Units	Approximate Overall Slope Angle (°)	Analysis	Factor of Safety	Comparison with Adopted Performance Criteria
Section C-C', Rapid Draw Down	Pit Lake at RL 60 m rapidly drained to 54 m	Saprolite	66	Undrained	1.005	Unacceptable < 1.2, however occurrence unlikely
Section D-D' Rapid Draw Down + 20 kPa Surcharge	Pit Lake at RL 60 m rapidly drained to 54 m	Saprolite	30	Undrained	1.30	Tolerable > 1.2
Section D-D' Stockpile Surcharge – 20 m offset	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm /Coomalie Dolostone	30	Undrained	1.696	Acceptable >1.5
Section D-D' Crane Surcharge– 40 m offset	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm /Coomalie Dolostone	30	Undrained	1.696	Acceptable >1.5
Section D-D' Small Vehicle Surcharge – no offset	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm /Coomalie Dolostone	30	Undrained	1.263	Tolerable > 1.2
Section D-D' Hypothetical Access Ramp Construction	Pit Lake at RL 60 m	Saprolite overlying HW Whites Fm /Coomalie Dolostone	30	Undrained	1.572	Acceptable >1.5

Where Saprolite remains in steep slopes at the pit crest and is subjected to short term, rapid draw down conditions; an unacceptable FOS is achieved. A FOS of ~1 in the case of Section C-C' suggests slope failure may occur for slopes steeper than 65° under such conditions.

Where flatter slope angles (<35°), are assessed, a tolerable FOS of >1.2 is achieved, inclusive of when a surcharge load of 20kPa behind the slope crest is applied to represent construction activities in conjunction with rapid draw down conditions, considered a worst-case scenario. The relative risk level of this hazard considering likelihood and consequence is discussed in the following sections.

Considering a case where heavy machinery (assumed to be 120 tonne crane) and stockpiling (assumed to comprise waste rock material to a maximum height of 20 m) occurs, a tolerable FOS is achieved when the crane is offset from the pit rim by >40 m and the nearest toe of stockpiles are offset >20 m from the pit edge. Given this, a simplified minimum offset from the Main Pit edge for stockpiles and large machinery of 40 m has been adopted for operational planning purposes. Smaller vehicles (< 30 tonne) are able to operate within a closer proximity to the pit edge and ramp (with in a nominal 5m offset), achieving a tolerable FOS.

7.11 Pit Slope Stability Risk Assessment

A qualitative risk assessment has been conducted to assess potential risk of pit slope instability to construction works in the vicinity of the main pit crest. This has then been used to determine acceptability and appropriate treatments / monitoring requirements for the identified critical stability hazards.

The risk assessment procedure adopted below is based on the Australian Geomechanics Society, 2007 Practice Note Guidelines for Landslide Risk Management [35] and the AS/NZS ISO 31000:2009 Risk Management – Principles and guidelines [36].

It is important to distinguish between "acceptable risks" and "tolerable risks". Tolerable Risks are risks within a range that society can live with so as to secure certain benefits. It is a range of risk regarded as non-negligible and needing to be kept under review and reduced further if practicable.

Acceptable Risks are risks which everyone affected is prepared to accept. Action to further reduce such risk is usually not required unless reasonably practicable measures are available at low cost in terms of money, time and effort. Acceptable risks are usually considered to be one order of magnitude lower than the Tolerable Risks.

The AGS risk framework presents tolerability criteria for different types of developments, recognising the trade-off between the risks, the benefits of development and the cost of risk mitigation. Given the relatively short-term nature of the risk period during construction, a risk of moderate or low is considered tolerable level, along with appropriate risk mitigation measures to reduce risk as low as reasonably practical.

7.12 Slope Risk Assessment Criteria

The AGS 2007c guidelines [35], outline an approach that includes a qualitative risk assessment for risk to property and a 'semi-quantitative' assessment for risk to persons. An assessment of risk from site-specific hazards is presented below and measures are proposed to meet the relevant Tolerability Criteria for risk to persons. The conditions that may result in low-consequence nuisance slumps (minor slumps) or erosion are not included in this assessment.

As presented in AGS 2007c [35], the qualitative level of risk to property resulting from a landslide event is based on a measure of the likelihood of occurrence (**Table 17**) combined with the consequence to property (**Table 18**).

Table 17 Qualitative Measures of Likelihood

Level	Descriptor	Description	Approximate Annual Probability
A	Almost certain	The event is expected to occur over the design life	10^{-1}
B	Likely	The event will probably occur under adverse conditions over the design life	10^{-2}
C	Possible	The event could occur under adverse conditions over the design life	10^{-3}
D	Unlikely	The event might occur under very adverse circumstances over the design life	10^{-4}
E	Rare	The event is conceivable but only under exceptional circumstances over the design life	10^{-5}
F	Barely credible	The event is inconceivable or fanciful over the design life	10^{-6}

Table 18 Qualitative Measures of Consequence to Property

Level	Descriptor	Consequence to Property	Consequence to Person (s) at Risk
1	Catastrophic	Structure(s) completely destroyed and/or large-scale damage requiring major engineering works for stabilisation. Could cause at least one adjacent property major consequence damage.	Multiple Fatalities to person(s) most at risk
2	Major	Extensive damage to most of structure, and/or extending beyond site boundaries requiring significant stabilisation works. Could cause at least one adjacent property medium consequence damage.	Single Fatalities to person most at risk
3	Medium	Moderate damage to some of structure, and/or significant part of site requiring large stabilisation works. Could cause at least one adjacent property minor consequence damage.	Significant Injury to person most at risk
4	Minor	Limited damage to part of structure, and/or part of site requiring reinstatement stabilisation works.	Minor injury to persons most at risk, ample time to escape
5	Insignificant	Little damage	No significant injury

Likelihood and consequence are combined in the matrix shown in **Table 19**, resulting in risk level that can range from very low (VL) to very high (VH).

Table 19 Qualitative Risk Analysis Matrix

Likelihood		Consequence				
		Catastrophic	Major	Medium	Minor	Insignificant
Almost Certain	10^{-1}	VH	VH	VH	H	M or L
Likely	10^{-2}	VH	VH	H	M	L
Possible	10^{-3}	VH	H	M	M	L
Unlikely	10^{-4}	H	M	L	L	VL
Rare	10^{-5}	M	L	L	VL	VL
Barely Credible	10^{-6}	L	VL	VL	VL	VL

The standard definition of the risk levels from AGS 2007c [35] are presented in **Table 20.**

Table 20 Risk Level Implications

Risk Level		Example Implications
VH	Very High	Unacceptable without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low; may be too expensive and not practical. Work will likely cost more than the value of the property
H	High	Unacceptable without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to Low. Work would cost a substantial sum in relation to the value of the property.
M	Moderate	May be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce risk to Low.

Risk Level		Example Implications
L	Low	Usually acceptable to regulators. Where treatment has been required to reduce the risk to this level, ongoing maintenance is required.

7.13 Slope Hazard Identification and Assessment Process

The following landslide hazards as defined in AGS 2007c [35] have been considered for a landslide risk assessment based on the potential site disturbances associated with remediation of the Rum Jungle Main Pit.

- Shallow earth slide within soil profile;
- Deep-seated slide (rock and soil); and
- Slide or topple (detachment of boulder) within pit walls.

7.14 Qualitative Slope Risk Assessment

For the purposes of this study, the elements at risk are considered to be the infrastructure associated with backfilling operations and the inherent risk to construction workers working in close proximity to the pit crest.

The qualitative risk assessment considers the probability and consequences of material being displaced and the probability that the displaced material, once mobilised, will impact or undermine the element at risk.

The qualitative level of risk resulting from a landslide event is based on a measure of the likelihood of occurrence combined with the consequence to property. **Table 21** summarises the qualitative assessment of slope instability risk to property prior to any remedial works or engineering controls.

Table 21 Results of Qualitative Risk Assessment

Potential Hazard	Likelihood	Consequence	Qualitative Risk	Discussion
Shallow slide within soil profile.	Possible	Minor	Moderate	Minor, <2m crest loss expected, recommended to be managed by implementing offset behind the pit crest and building suitable access to pit crest
Deep-seated slide (extremely weathered/sheared rock and soil).	Unlikely	Major	Moderate	Major, <5 m crest loss, recommended to be managed by implementing offset behind the pit crest, constructing laydown areas in demarcated zones and building suitable access to pit crest
Slide or topple (detachment of boulder) within pit walls	Likely	Insignificant	Low	Kinematic instability is likely within submerged blocky sections of more intact rock mass e.g. Dolostone although the risk of such instability is considered insignificant due to depth below crest not considered to impact on surface works. Slopes will be buttressed by pit backfill.

7.15 Risk to Life

Considering the moderate risk of crest failure summarised above, this is may be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce risk to Low.

A semi-quantitative assessment of risk to life has been undertaken cross-check the risk level of these hazards as it relates to persons at risk (principally construction personnel inside a vehicle). This assessment has been done for representative 'moderate risk' hazards tabulated above, as follows:

Shallow slide within soil

- The likelihood of failure is considered 'Possible' (P=0.001);
- The likelihood of that failure affecting a construction activity occurring in the vicinity of the slope crest or toe may be 'Likely' (P=0.01);
- The vulnerability (V) to personnel inside a vehicle impacted by the landslide. Outcome; Unable to escape the vehicle or plant they occupy. Vehicle falls into the pit lake or is buried, likely be killed (V = 1.0); hence
- The annualised risk to life is P x V = $1E^{-5}$ which is at the limit of tolerability for newly constructed slopes under the AGS 2007c guidelines but potentially acceptable for existing slopes.

Large scale slip failure at the crest

- The likelihood of failure is considered 'Unlikely' (P=0.0001);
- The likelihood of that failure affecting a construction activity occurring in the vicinity of the slope crest or toe may be 'Almost certain' (P=0.1);
- The vulnerability (V) to personnel inside a vehicle impacted by the landslide. Outcome; Unable to escape the vehicle or plant they occupy. Vehicle falls into the pit lake or is buried, likely be killed (V = 1.0); hence
- The annualised risk is P x V = $1E^{-5}$ which is also at the limit of tolerability for newly constructed slopes under the AGS 2007c guidelines but potentially acceptable for existing slopes.

The semi-quantitative risk assessment is generally in line with the qualitative risk level and indicates that additional investigation, planning and implementation of risk mitigation measures are required at construction to reduce the overall level of risk to low (acceptable) levels. As discussed further below, appropriate measures to mitigate risk would typically involve appropriate offset of crest loading, progressive geological mapping of existing and new cut slopes at intermediate bench cuts, refinement of location-specific hazards, re-assessment of risk to property and risk to life and where required development of detailed remedial measures (exclusion zones, scaling, netting, meshing, bolting, monitoring) as appropriate to the location-specific conditions.

7.16 Slope Risk Management

As highlighted in **Section 7.11**, there is an estimated low to moderate level of risk to construction activities expected to occur in the vicinity of the main pit crest, which is may be tolerated in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment/mitigation options to reduce risk to Low.

As such a number of risk mitigation strategies are recommended to lower the risk to as low as reasonably practical (ALARP).

A number of risk mitigation principals as outlined in AGS 2007c [35] are considered appropriate and should be implemented during construction, including;

- Avoid the risk;
- Reduce the frequency of land sliding;
- Reduce the consequences; and
- Manage the risk by establishing monitoring and early warning systems.

7.16.1 Avoid the risk

This may include re-location of construction infrastructure to avoid the zone of influence from identified 'higher' risk zones and targeting more stable, geometrically feasible sections of the pit to provide access for over-water construction activities. Refer to **Section 7.5** and **Figure 15** below**.**

7.16.2 Reduce the Frequency of Land Sliding

Given the access constraints this approach is considered limited, although some improvement may be achieved by stabilisation measures to control the initiating circumstances, such as by re-profiling the surface geometry where existing slopes are 'over steep' and by provision of improved surface water drainage measures.

This might be completed at the 'targeted' locations identified from mapping above, following a detailed risk assessment and investigation to determine the minimum set back distances, construction methodology, safe batter angles for cut and fill to facilitate safe construction and access requirements.

7.16.3 Reduce the Consequences

This may be achieved in conjunction with reducing the frequency by provision of defensive stabilisation measures or by relocation of construction infrastructure and establishing pit access points away from designated higher risk locations and implementing minimum safe set back distances and designing stable access points. Refer to **Section 9.5** and **Figure 25** below**.**

7.16.4 Manage the Risk by Establishing Monitoring and Early Warning Systems

Managing the risk by regular monitoring will be imperative during construction to assess the performance of the risk mitigation measures outlined above and to detect a change in conditions which may highlight increased risk.

This could include regular site visits, inspections, mapping and/or survey (including bathymetry) during the construction period, which may enable the risks to be managed as an interim measure in the short term or as a permanent measure for the long term by alerting persons potentially affected to a change in the surrounding ground condition.

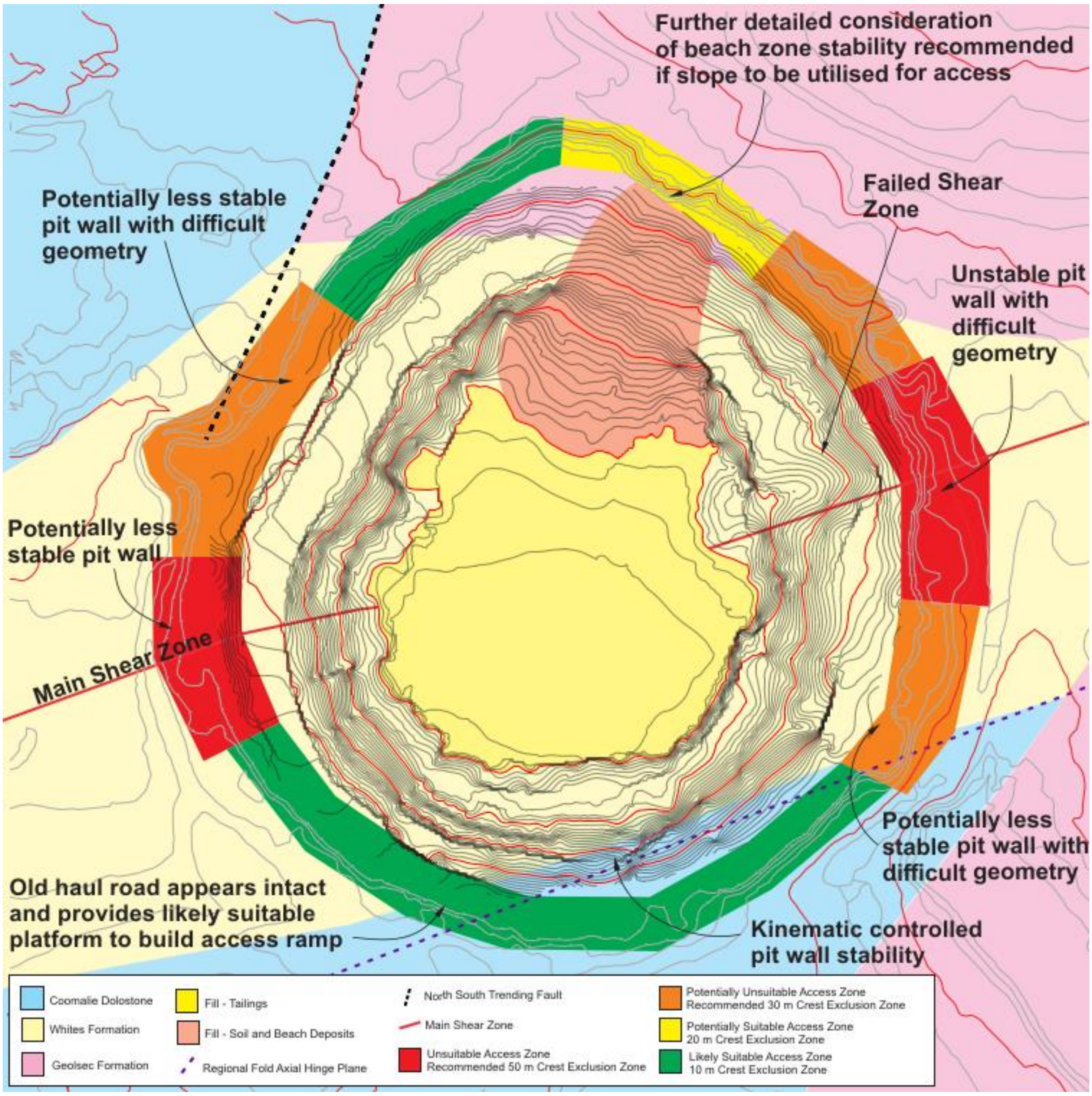
Such systems may be regarded as a method of reducing the consequences provided it is feasible for sufficient time to be available between the alert being raised and appropriate action being implemented.

7.17 Slope Risk Construction Considerations

7.17.1 Risk Demarcation

The risk mitigation recommendations outlined above have been summarised into a site plan to be used to inform construction activities and future works in the vicinity of the main pit, as shown in **Figure 25** below.

Figure 25 Rum Jungle Main Pit Stability Observations



As discussed in **Section 7.6**, the critical soil and rock components which pose the greatest risk of instability in the vicinity of the pit crest include over-steepened, potentially saturated exposures of saprolite and extremely weathered or disintegrated rock masses. Further, deep seated instability is noted to have occurred within the submerged profile of the main shear zone in the eastern wall of the main pit, based on interpretation of bathymetry indicating scree slope debris at susceptible locations.

As such, these areas have been highlighted as potentially high-risk zones which should be suitably managed by utilising the controls discussed above, including establishment of exclusion zone set back distances for activities such as stockpiling of back fill materials, storing of plant and equipment, laydown areas and pit lake access points.

Where pit access points are required, these are recommended to be placed within the likely suitable access zones highlighted in **Figure 25**. Access points (ramps, pontoons etc) should be suitably designed following additional detailed investigation and assessment at preferred locations.

Detailed design drawings are to be developed showing set out of setback distances, recommended layouts for laydown areas, stockpiling, access to pit and controls to be implemented during construction activities.

7.17.2 Limiting Slope Batter Angles

The stability analysis completed herein generally indicates that slopes within Saprolite / Extremely Weathered Rock flatter than 30^{o} (1V:2H) are expected to provide a tolerable FOS under short term conditions, although it is envisioned that significantly flatter slopes (ramps) would be required at pit lake access points (i.e. 1V:10H). A summary of the recommended geometry for slopes in vicinity of activities with relation to the zones outlined in **Figure 25** (above) are shown in below.

Limiting batter slope angles for the expected range of cut and fill materials are provided in **Table 22**. Global stability analysis at key locations has been undertaken and results presented in **Appendix E** to confirm the performance of permanent batter slope angles and treatment requirements.

Table 22 Recommended General Batter Slope

Slope Material Type	Limiting Slope Geometry (temporary)[1,2,3]	Limiting Slope Geometry (permanent) [2,3]
Saprolite	2H:1V	3H:1V
Soil and Beach Deposits	2H:1V	3H:1V
Waste Rock Backfill	1.5H:1V	2H:1V
Main Shear Zone material	Required individual detailed assessment	
Variably weathered Whites Formation	1.5H:1V	2H:1V
Variably weathered Geolsec Formation	1.5H:1V	2H:1V
Variably weathered Coomalie Dolostone	1.5H:1V	2H:1V

Notes:

1. Generic slope recommendations exclude consideration of surface and traffic loading near the slope crest, high groundwater (within depth of cut or fill), submerged or soft ground areas. Where these conditions exist, location-specific stability analyses and site controls will be required.
2. Slope ends to transition into landform; slope drainage requirements as per Landcom, *Soils and Construction* 'Blue Book', 2004.
3. Location-specific individual assessment required for slopes ≥ 3.0 m high, or steeper than 2H:1V.
4. Erosion protection to be provided for soil slopes no steeper than 2H:1V (Topsoil/ Jutemesh/ geotextile matting planted with indigenous plants and grasses)

7.17.3 Constructability in areas exposed to slope risk

Earthworks undertaken in the vicinity of the crest should be in accordance with the specifications and recommendations outlined in the Detailed Design Drawings, relevant technical specification and SLR Geotechnical Report [26]. Where slope batters are to be cut at the pit crest, works should be completed using long reach excavation techniques set back behind the minimum set back distance detailed in design drawings to trim batters to the recommended profile by pulling back and pushing forward the crest.

Where works (i.e. earthworks, unloading equipment, crane operation etc) are required within the recommended set back distance, a detailed assessment of the condition of the pit crest should be completed with appropriate risk assessment to assess the suitability of machinery accessing the pit crest within the recommended set back distance (40 m). Alternatively, works could be completed by barge to access unstable / inaccessible areas of the pit crest requiring treatment.

7.18 Summary of Key Parameters

7.18.1 Geology Profile

Soils:

- Laterite, developed by intensive and long-lasting weathering of the underlying bedrock into an iron-rich oxidized pelloids profile, clayey silt to silty gravel and;
- Saprolite, decomposed and chemically weathered (less than laterite) bedrock, clay and silt rich, containing trace structure and texture that were present in the original rock.

The bedrock sequence within the main pit form part of the Partridge Group and include (in order of increasing age):

- Geolsec Formation, sedimentary deposit of hematite-quartz breccia (HQB); mainly quartz clasts in an amorphous hematite matrix;
- White's Formation (aka Golden Dyke Formation), sedimentary carbonaceous shale to metamorphosed schist with tremolite, graphite and pyrite. In places it is described as a quartz-sericite material with a strong foliation which has been subjected to at least two generations of later micro folding; and
- Coomalie Dolostone, sedimentary carbonate rock, karstic in places, comprising mainly dolomite, magnetite and calcite; can be in a saccharoidal or crystalline form to a more hematized and silicified or even brecciated form closer to the thrust fault zones.

7.18.2 Pit Geometry

Based on bathymetrical survey of the main pit was completed as part of the Main Pit Backfill remediation concept assessment in 2015 [16].

- upper half-spiral of the former haul road appears to be relatively intact, below this, the original haul road is covered by backfill and scree.
- The former benches are also mostly indistinguishable now, due to filling-in resulting in relatively uniformly sloped pit walls with overall slope angles ranging from between 25° to 30° in the mudstone and 28° to 38° in the slate;

- The current surface of the submerged backfill is deepest in the southern areas and is mapped at an approximate RL +16 m RL;
- Pit crest is approximately +61m RL.

7.18.3 Material Parameters

Table 23 Summary of Material Parameters

Material	Bulk Unit Weight, γ (kN/m³)	Effective Cohesion, c' (kPa)	Friction Angle, ø' (º)	Undrained Shear Strength, Su	Material Type, Consistency or Density
Uncontrolled Fill (Soil & Beach Deposits)[2]	18	1	28	35	Cohesive, fine grained, firm
	20	0	30	-	Granular, coarse grained, very loose to loose
Uncontrolled Fill (Tailings)[2]	16	0	26	Su = 0.5+ 0.25 x P'	Cohesive, normally consolidated
Backfilled Waste Rock[2]	18	0	32	-	Waste Rock (WR) placed as backfill during past mining / remediation activities. Future WR backfill and Capping materials assumed to have same geotechnical properties.
Saprolite / Laterite	18	5	25	30	Clay rich decomposed material
Highly Weathered Geolsec Formation Quartz Breccia	22	13	26	-	Highly weathered hematite – quartz breccia, with clay
Geolsec Formation Quartz Breccia	24	61	43	-	Intact Hematite Quartz Breccia
Highly Weathered Whites Formation	22	12	24	-	Shale, Schist, minor Quartzite, some shearing and brecciation, weathered / softened with clay infill
Whites Formation	24	48	37	-	Shale, Schist, Meta-Sandstone, some shearing and brecciation
Main Shear Zone[1]	22	10	22.5	-	Highly sheared, disintegrated Shale, Schist and Meta-Sandstone
Highly Weathered Dolostone	22	20	32	-	Highly weathered / softened Dolomite
Coomalie Dolostone	25	High Strength Material			Dolomite

7.18.4 Construction Considerations

7.18.4.1 Exclusion Zones

- Observe typical 40m exclusion zone around Main Pit rim for non-essential Main Pit activities e.g. haulage, stockpiling, laydown etc.

- For Main Pit activities, observe exclusion zones recommendations where practical (ref: **Figure 25**).
- For heavy loading works within or proximal exclusion zones, activity specific stability assessment is recommended (e.g. crane works adjacent to pit rim).

7.18.4.2 Limiting Slope Batter Angles

Slopes within Saprolite / Extremely Weathered Rock flatter than 30^{o} (1V:2H) are expected to provide a tolerable FOS under short term conditions, although it is envisioned that significantly flatter slopes (ramps) would be required at pit lake access points (i.e. 1V:10H). A summary of the recommended geometry for slopes in vicinity of activities with relation to the zones outlined in **Figure 25** (above) are shown in below in **Table 22**.

Table 24 Recommended General Batter Slope

Slope Material Type	Limiting Slope Geometry (temporary)[1,2,3]	Limiting Slope Geometry (permanent)[2,3]
Saprolite	2H:1V	3H:1V
Soil and Beach Deposits	2H:1V	3H:1V
Waste Rock Backfill	1.5H:1V	2H:1V
Main Shear Zone material	Require individual detailed assessment	
Variably weathered Whites Formation	1.5H:1V	2H:1V
Variably weathered Geolsec Formation	1.5H:1V	2H:1V
Variably weathered Coomalie Dolostone	1.5H:1V	2H:1V

Notes:

1. Generic slope recommendations exclude consideration of surface and traffic loading near the slope crest, high groundwater (within depth of cut or fill), submerged or soft ground areas. Where these conditions exist, location-specific stability analyses and site controls will be required.
2. Slope ends to transition into landform; slope drainage requirements as per Landcom, *Soils and Construction* 'Blue Book', 2004.
3. Location-specific individual assessment required for slopes ≥ 3.0 m high, or steeper than 2H:1V.
4. Erosion protection to be provided for soil slopes no steeper than 2H:1V (Topsoil/ Jutemesh/ geotextile matting planted with indigenous plants and grasses)

8 Main Pit Backfilling Strategy

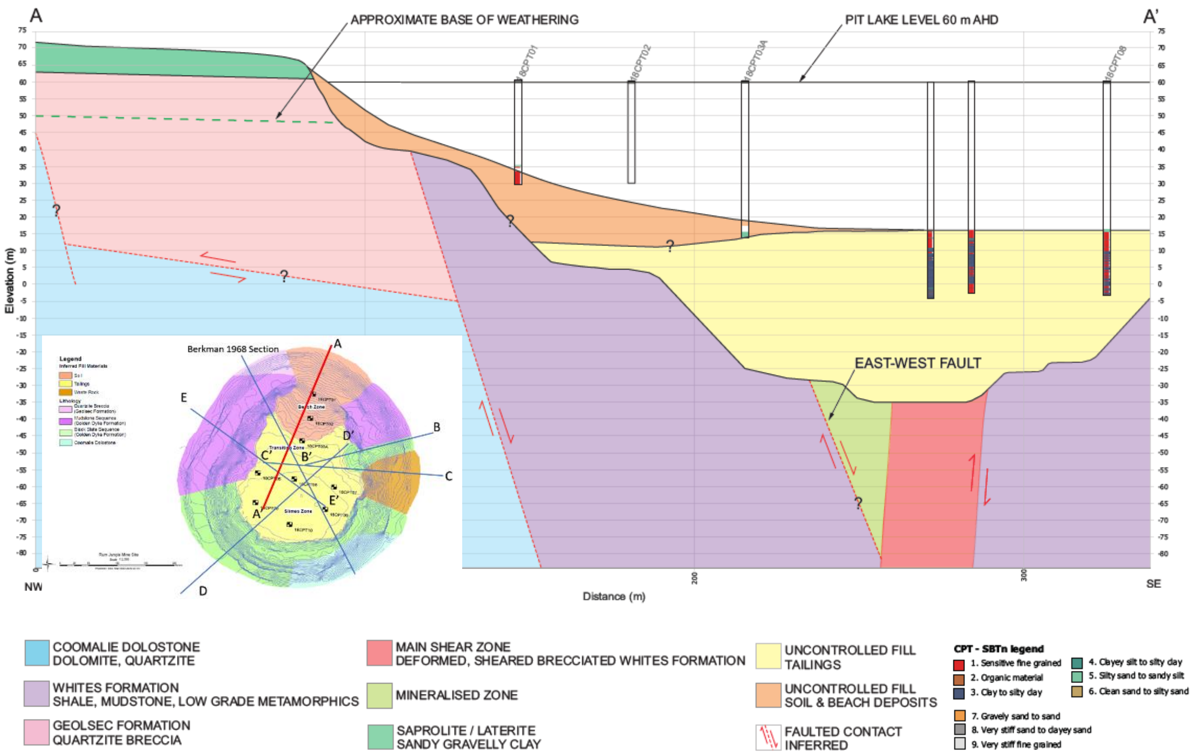
8.1 Soil, Tailings and Backfill Material Assessment

8.1.1 Geotechnical Units

As described in introductory Sections of this report, The Main Pit has been partially filled with approximately 800,000 m^3 of un-neutralised tailings overlain by approximately 46m of pit water. Flanking these deposits and locally interspersed within the tailings are scree slopes of inferred failed pit-wall material and end-dumped and washed-in zones of transported soil backfill and waste rock.

The schematic distribution of these materials at surface is shown in plan on **Figure 7**. A section showing the indicative profile of subsurface materials at one location is also shown below at Section A-A’ in **Figure 26** below.

Figure 26 Generalised Section A-A’ Showing Geological Units



The in-situ soil materials and proposed backfill materials have been defined as a set of geotechnical units to inform engineering design of the project as shown in **Table 25** below.

Table 25 Main Pit Geological Soil Units

Unit	Geological origin	Geological description	Predominant material type	Sub-unit	Consistency / Density / Inferred strength
SB	Soil & Beach deposits (In-Situ)	End dumped soil and reconstituted waste rock forming submerged soil slopes and beaches	Cohesive / fine grained	SB-C	Generally stiff (consolidated under overburden)
			Granular / coarse grained	SB-G	Very Loose to Loose, generally uncompacted
T	Tailings (In-Situ)	Hydraulically placed tailings and slimes	Cohesive / fine grained	T	Very soft to stiff, normally consolidated. Interbedded with SB-G in places.
BL	Bedding Layer (Site-won granitic sand)	Hydraulically placed sand	Granular / coarse grained	BL	Very Loose to Loose
WR	Waste Rock (Site-won waste rock fill, PAF)	Hydraulically placed waste rock conditioned with lime	Granular / coarse grained	WR	Very Loose to Loose sandy Gravel, comprising DW-XW weathered shale
CAP	Capping Rock Fill (Site-won waste rock fill, NAF)	Sidecast waste rock	Granular / coarse grained	CAP	Nominally compacted site-won fill

8.1.1.1 Soil & Beach deposits

CPT data and bathymetric data indicate that the excavated pit walls in some areas have since been overlain by more recent fill which now form a mantle of soil and beach deposits extending between the pit rim and the deeper tailings forming the current pit lake bed in central areas.

The interpreted distribution of these materials at surface in plan is shown in **Figure 7**.

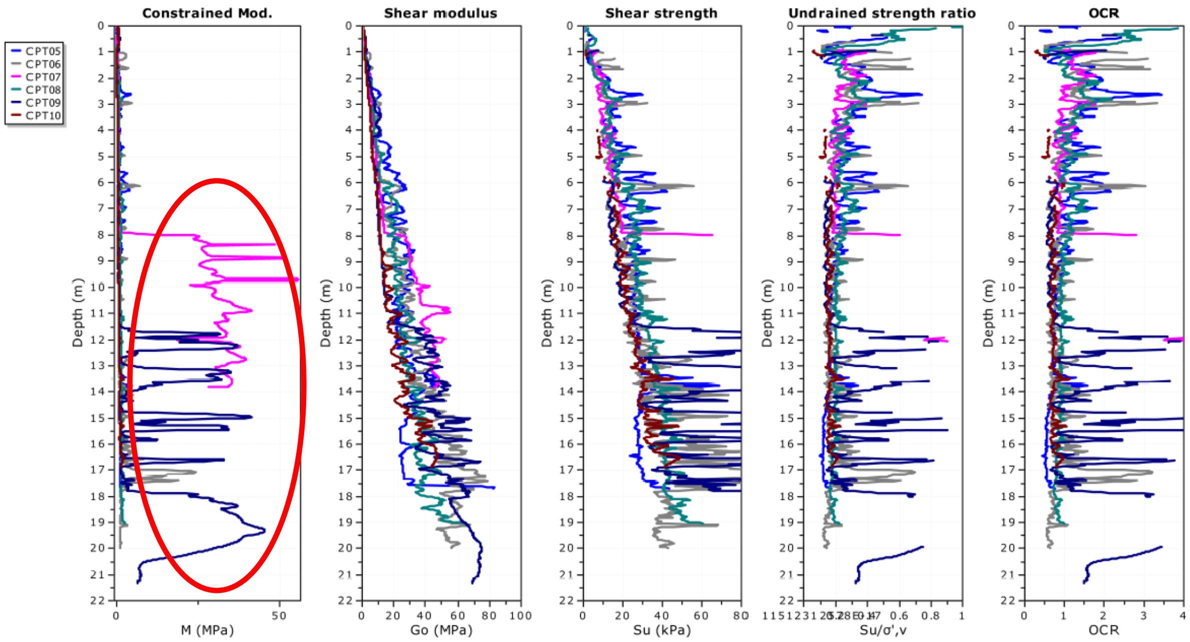
Anecdotal and reported evidence indicates that the origin of these side-slope materials may comprise a mixture of materials formed at different times and comprising:

- End dumped, uncontrolled backfill comprising 'Sandy Clay', 'Clayey Sand' or 'Clayey Gravel';
- Waste Rock;
- Fluvial and/or debris flow deposits washed in by river systems and/or flood events (sometimes occurring as interbedded deposits at depth within the main body of tailings); and
- Landslip material and scree slopes from unstable sections of pit wall (particularly near the shear zone area).

Introduced fill materials are likely associated with different phases of partial backfilling, placed by end-dumping from the pit rim with slopes formed near their natural angle of repose. In some areas, introduced filling and debris flows or tailings segregation has been inferred to occur and in edge areas is likely mixed with older, underlying tailings. In some areas as discrete coarser layers occur at depths greater than 8m below pit lake floor (CPTs 06, 07 and 09 in– See circled area in **Figure 27** and expanded plots in **Appendix F, Figures F1.2 to F1.5**).

CPTs probed through shallower side-slope soil and beach deposits (CPTs 06, 07 and 09 and expanded plots in **Appendix F**) penetrated to between 3.5m to 10.5m below the slope surface and indicated the fill slopes mainly comprise firm to stiff clay and silt-rich sensitive soils with a minor proportion of loose sand.

Figure 27 Inferred slopewash soils within tailings (circled)



As the shallow side-slope deposits are generally to be progressively buttressed by fill placed from the bottom up, and dewatering of the pit lake is not planned, it is anticipated that stability will generally be improved through the process of backfilling.

8.1.1.2 Tailings (In Situ)

Subaqueous deposition of tailings within the Main Pit is known to have occurred from about 1965 to 1971. Tailings with a high proportion of fines are also referred to as 'slimes' and these silt-rich deposits are easily disturbed and are typically dispersive when mobilised in water.

As indicated in the previous Section, in some areas, tailings segregation (or hydraulic sorting of particles) may have occurred during placement, resulting localised sandy layers at depth. However, the great majority of the CPT probed tailings below the pit lake floor are classifiable as medium to high plasticity silt and clay mixtures which are very soft near surface (shear strength <12.5kPa), increasing to typically firm (shear strength 25-50kPa) over about 20m depth.

Characteristics of the tailings are shown in Figures below.

Figure 28 Tailings and Pit Rim Soils Atterberg Limits

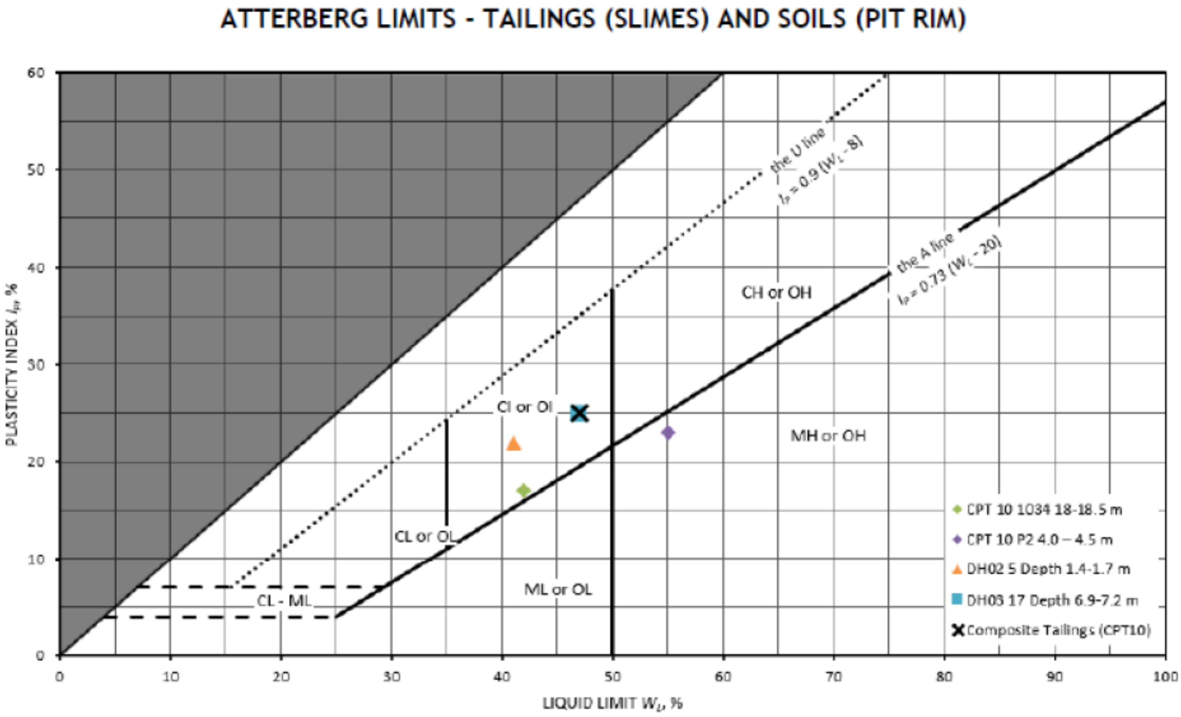
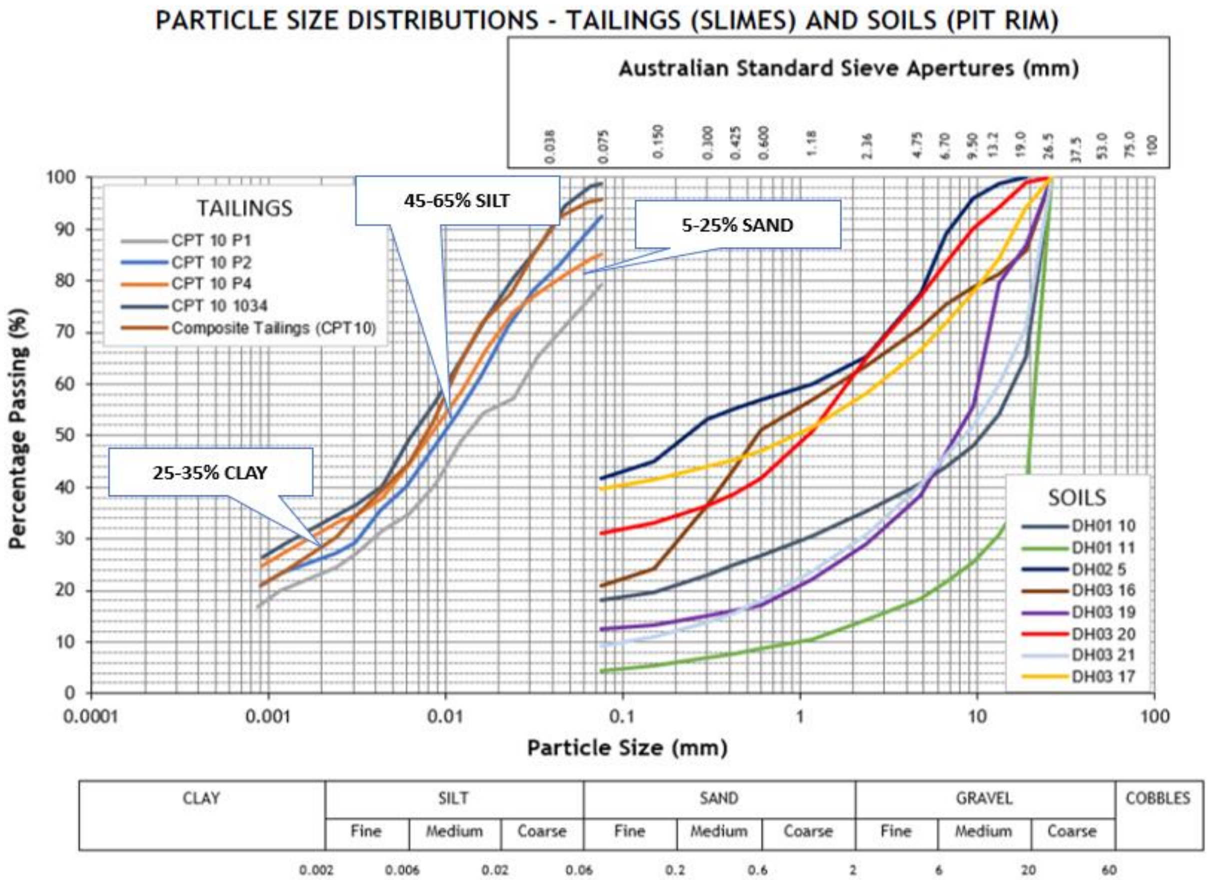


Figure 29 Particle Size Distribution Tailings and Pit Rim Soils



As expected, CPT and BCPT plots indicate the thick tailings deposits of cohesive material exist in a near-normally consolidated condition and therefore loading of these soils will result in significant consolidation and creep settlements as pore pressures dissipate over time. In order to assess settlement behaviour, compressibility properties including consolidation and creep indices and rate (coefficient) of consolidation have been assessed by reviewing laboratory test results (Rowe Cell), by correlation with plasticity test data and from CPT data.

Assessment of shear strength has been undertaken to enable stability analysis of backfilling materials which are to be supported by the tailings. In the absence of borehole or laboratory derived strength data, reliance has been placed on interpreting tailings strength profiles using CPT and BCPT data. In absence of shear vane or triaxial shear strength data, Nkt for cone has been derived by matching CPT shear strength to Ball cone assuming: Nball = 10 (Typ range 10 to 13) – to give near-normally consolidated strength profile Nkt varies from 13.5 to 18 to match ball interpretation.

A summary of key design parameters derived using the approaches above are shown below in **Figure 30** and **Figure 31**. Correlations used in the derivation of these properties are presented in **Appendix F Figures F2.1 to F2.5** along with additional tailings parameter plots overlaid with corresponding design lines – **Appendix F Figures F.3.1 to F3.11**.

Figure 30 Tailings Parameter Plot (1)

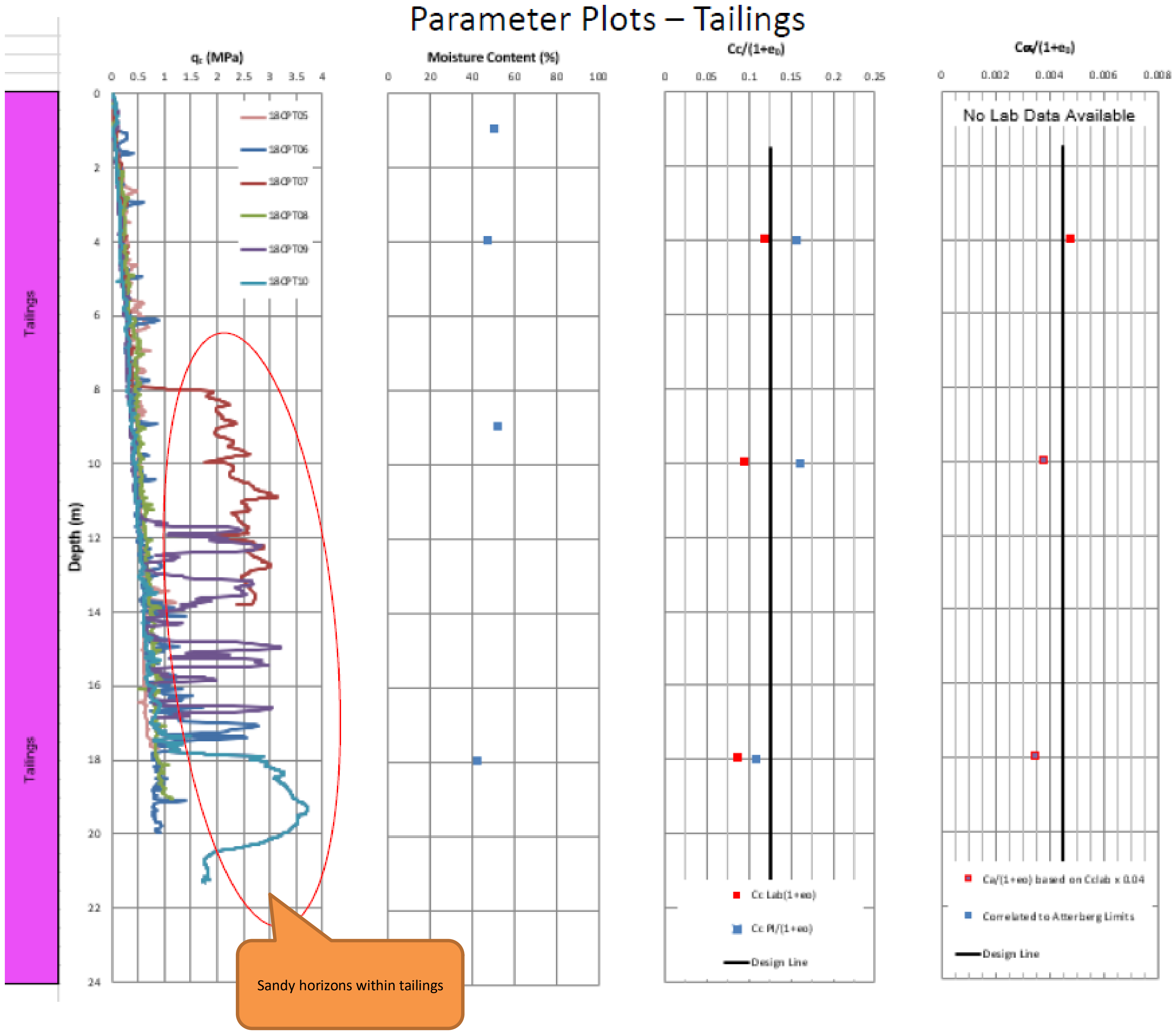
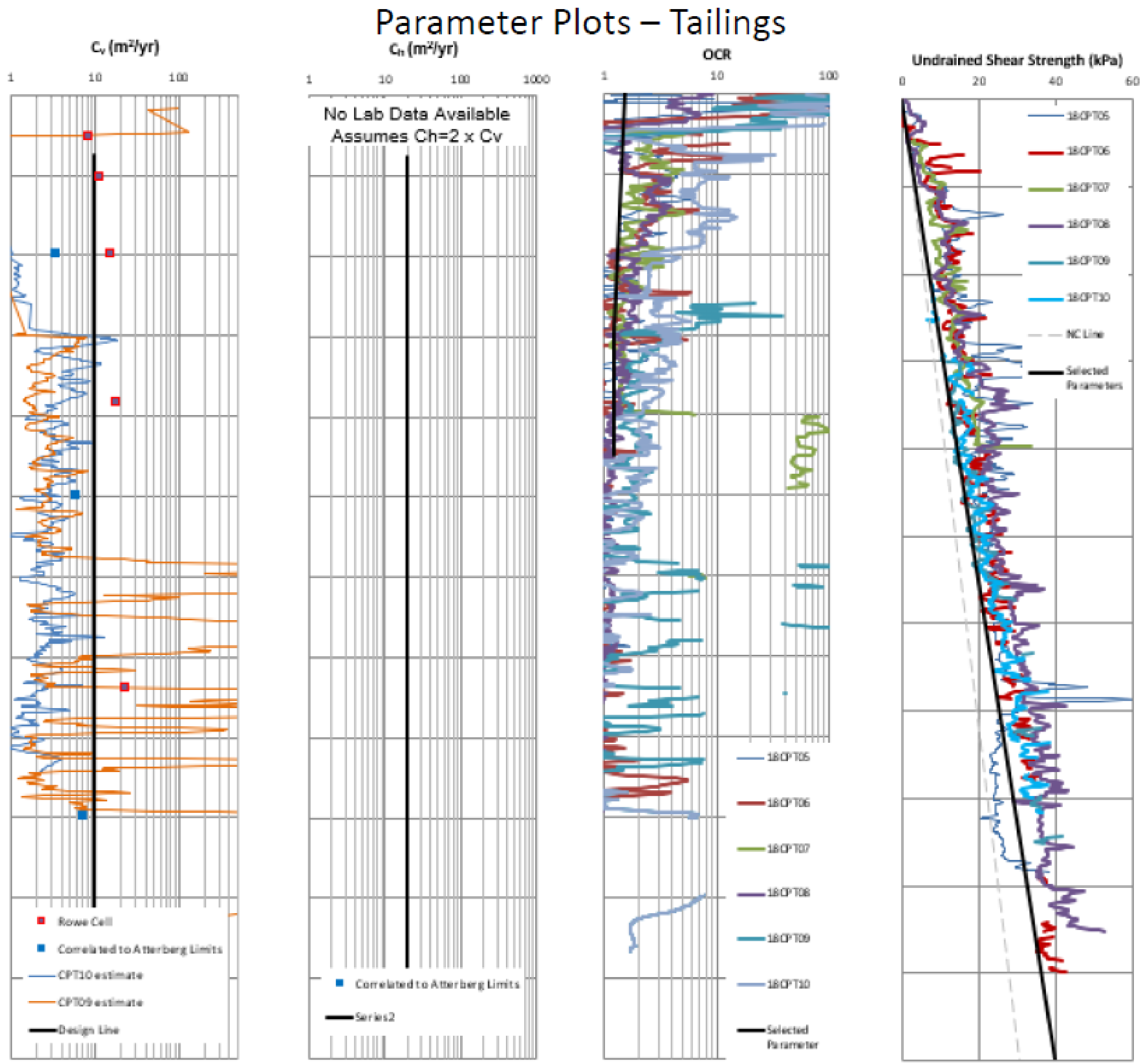


Figure 31 Tailings Parameter Plot (2)

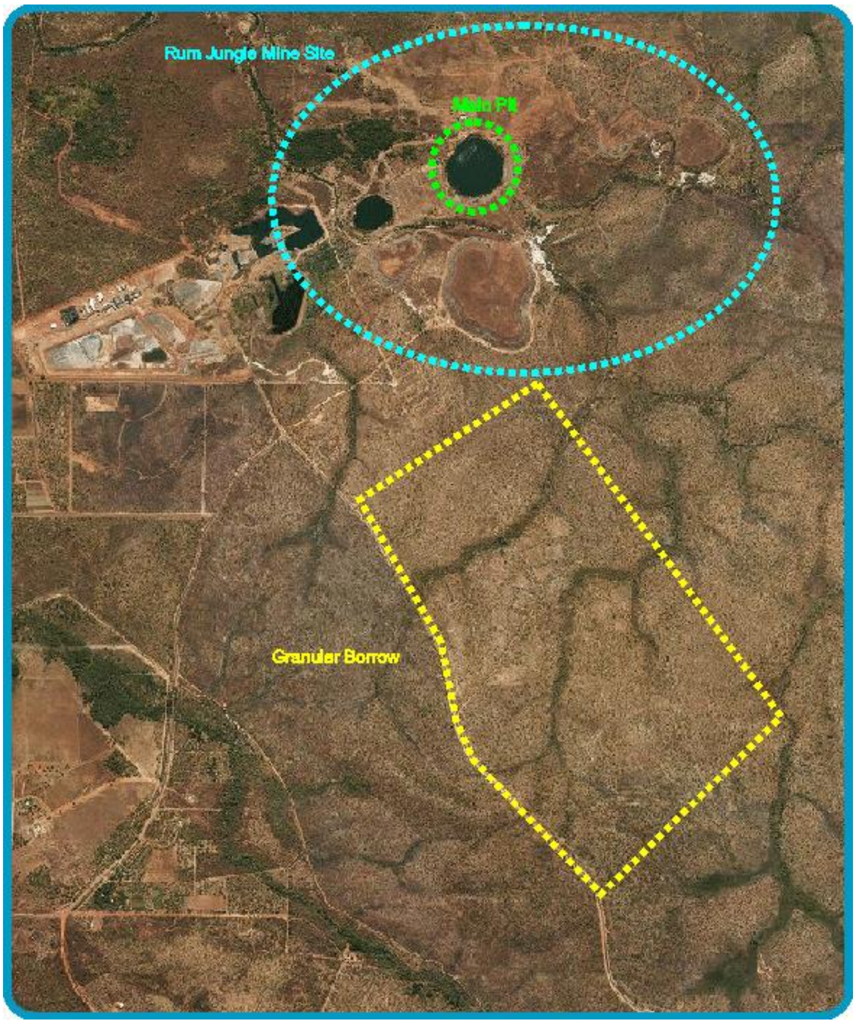


8.1.1.3 Sand Bedding Layer (Proposed Backfill)

The sand bedding layer will be sourced from the nearby granular borrow pit, approximately 1.50km south west of the Rum Jungle Mine Site. A haul road will be constructed from the borrow area to the mine site, leading to a stockpiling area near the Main Pit. **Figure 32** below shows the location the granular borrow relative to the mine site.

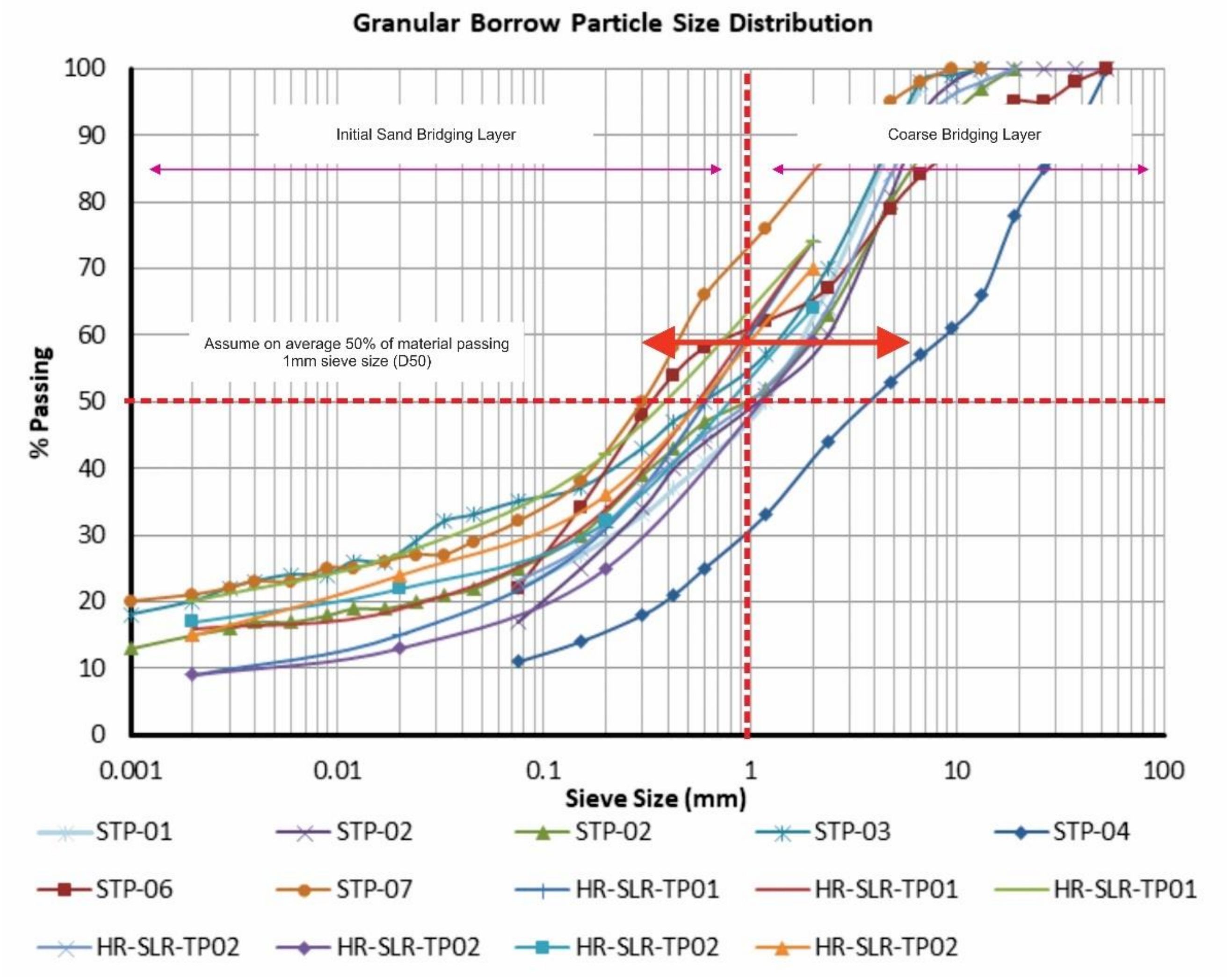
A recent investigation by SLR in July 2019 recorded "*topsoil overlying alluvial and residual soils which is underlain by extremely weathered granite bedrock and/or competent granite bedrock*" [26]. Descriptions of the encountered strata included loose to very dense sands and gravels with minor amounts of silt and clay.

Figure 32 Location of Granular Bridging Layer Borrow



The SLR investigation [26] recorded granular residual soils, from clayey SAND to sandy GRAVEL, largely from the weathering of the underlying granite bedrock. Samples of the residual soils underwent Particle Size Distribution (PSD) analysis and shear box testing. The results are provided in the grading plots in **Figure 33** below.

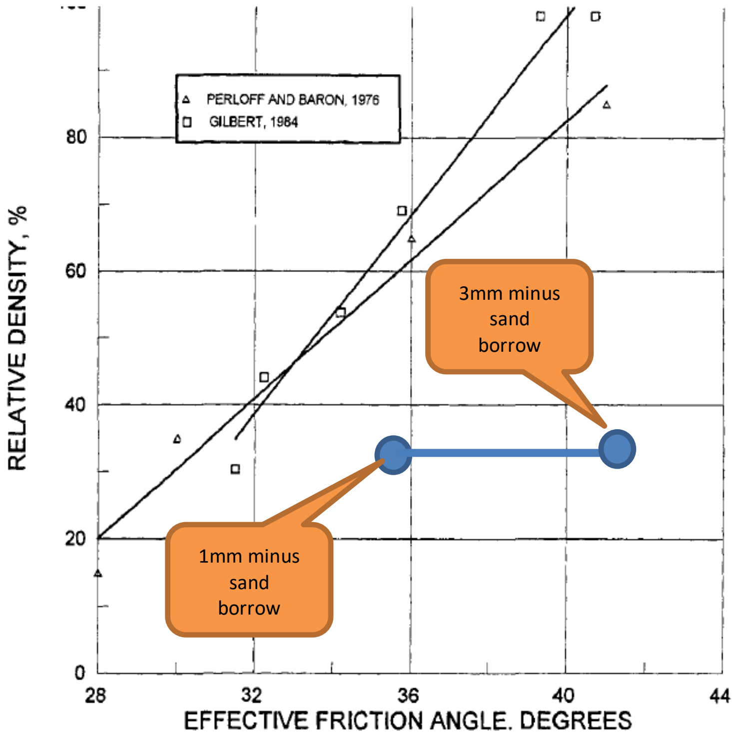
Figure 33 Sand Bedding Layer Grading Summary



PSD results on average have a D_{50} of about 1 mm particle diameter and the majority of gravel found to be fine and medium grained. This correlates to a sandy GRAVEL or gravelly SAND with up to 23% fines, as observed. Results of shear box testing undertaken on screened samples (1mm minus) indicated a constant volume friction angle of 37° and coarser fraction (3 mm minus) 43°.

Data relating the effective friction angle of sand with the relative density is also shown in **Figure 34** [37], [38]. For cap materials consisting typically of clean sands that are loosely deposited by pluviation (settling of material through water), the relative density is typically up to 20% and, using **Figure 34** and guidance from BS6349 Table 11 [39], the corresponding limiting effective friction angle is about 28 to 30 degrees. Also shown are the measured shearbox test results for site soils (tested as Loose which is about 30% relative density).

Figure 34 Typical and Measured Friction Angle for Sands



A cautious design value of 30° has been adopted for design, in addition to recognising also that the angle of repose of sub-aqueously placed sand in calm water is likely to be flatter at about 1V : 3H to 4H as follows:

Figure 35 Typical angle of repose for underwater slopes (BS6349, Part 5, 1991)

SOIL TYPE	SIDE SLOPE ANGLE			
	Still Water		Active Water	
	(degrees)	(V:H)	(degrees)	(V:H)
Rock	Nearly vertical		Nearly vertical	
Stiff clay	45	1:1	45	1:1
Firm clay	40	1:1.2	35	1:1.4
Sandy clay	25	1:2.1	15	1:3.7
Coarse sand	20	1:2.7	10	1:5.7
Fine sand	15	1:3.7	5	1:11.4
Mud and silt	10 – 1	1:5.7 – 1:57	5 or less	1:11.4 or less

The sand borrow material is considered to be appropriate for use as forming a bedding layer on top of the soft tailings. Placement would be sub-aqueous using a hydraulic pumping to carefully spread the bedding layer across the surface. The material would be screened to form a fine Bedding Layer (<1 mm diameter particles) which would be placed initially and then the coarser fraction (>1 mm diameter) would form an Intermediate Bedding Layer.

The sand sized particles are likely to be suspended in the water column and so disperse and settle out onto the base of the pit gently, reducing the potential disturbance of the chemocline and tailings. The thickness of emplaced granular material may be monitored using sonar to avoid overloading small areas with large volumes of material. By doing so the expectation is that the bearing capacity of the underlying soft sediments will not be exceeded and cause instabilities and failures. Through placement of the granular material, the underlying normally consolidated tailings and organic sediment may also be surcharged, improving bearing capacity.

The bedding layers material also has material properties which would be suitable for a flocculent agent to be added to, in order to minimise potential agitation of the chemocline.

As a mitigation measure the neutralise the acidic nature of the chemocline, a 1.7% w/w hydrated lime to sand bedding is to be incorporated in with the Sand Bedding materials. It is anticipated the relatively small addition of lime will not be detrimental to the geotechnical properties of the materials.

8.1.1.4 Waste Rock Fill (Proposed Backfill)

Waste rock backfill is planned to come from the following sources (in placement order): the Dyson's Overburden Waste Rock Dump (WRD), Intermediate Waste Rock Dump (WRD) and the Main Waste area. The waste rock is to be conditioned with lime at a rate presented by RGC & Jones [40] and placed to a maximum elevation of 56m RL. Liming rates extracted from RGC & Jones are presented below.

- Dyson's Overburden WRD Lime Conditioning Rate: 24 kg $CaCO_3$ per tonne of Waste Rock
- Intermediate WRD Lime Conditioning Rate: 24 kg $CaCO_3$ per tonne of Waste Rock
- Main WRD Lime Conditioning Rate: 15 kg $CaCO_3$ per tonne of Waste Rock

Above 56m RL, clean backfill is proposed to consist of rock capping and clayey laterite material. Typical properties for these units are provided in **Table 12** and **Table 13.** Waste rock compacted density was obtained using data from site compaction trials undertaken by RGC in 2014:

- Loose Density (t/m^3): 1.50 to 1.60
- Compacted Density (t/m^3): 1.65 to 1.76 (i.e. typical bank to truck bulking factor 1.1-1.2)
- Water-placed Density (t/m^3): 1.5 to 1.66 (i.e. typical bank to underwater bulking factor 1.0-1.2)
- Void Space (%): 15

The data indicates that the waste rock from the Main WRD has a compaction factor ranging from about 5% to 20% depending upon the thickness of lift (800mm and 200mm respectively). From the Intermediate WRD, the compaction factors range from about 8% to 25%.

The water-placed relative density of the waste rock is estimated by [41]such that '*Relative densities of cohesionless materials (sands and gravels) placed through a substantial depth of water may vary from 40% to 60%*'.

Grading tests on waste rock were undertaken following screening to remove oversize (75 mm), the proportion of which varied from about 5 to 90% on samples processed for grading. Typical D_{50} of screened WR is 6mm i.e. mainly fine-med gravel (**Figure 36**). Review of photographs presented within the Robertson GeoConsultant 2016 *Physical and Geochemcial Characteristics of Waste Rock and Contamined Materials report* [18] (**Figure 37 –** additional photographs provided in RGC report) indicate that, visually, up to about 75% by volume may be up to about 150 mm particle size, with an effective upper size of about 500 mm.

The waste rock borrow is known to be PAF material and therefore requires burial underwater (design dry season Main Pit Water Level = 59m RL, max level of waste rock to be at least 2.0m below low season water level and shallow pit lake as final landform).

Figure 36 Waste Rock Grading for Sample Particle Size Passing 75mm

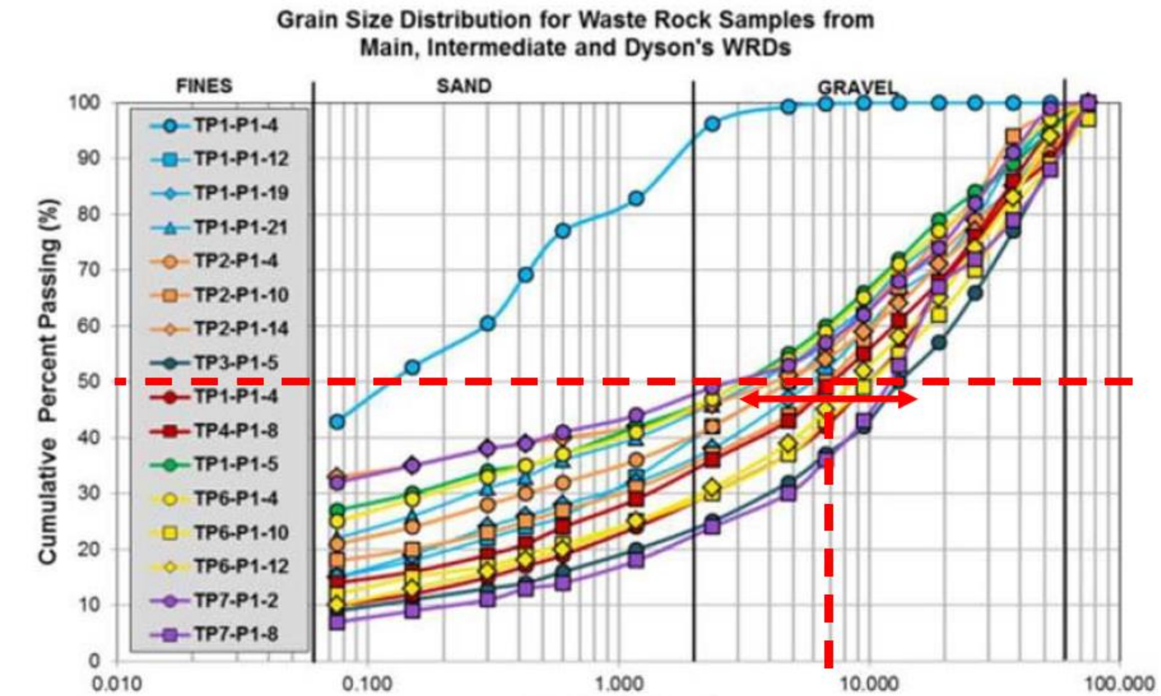


Figure 4-1. Particle Size Distributions of Waste Rock.

Figure 37 General Waste Rock Sample Photos Extracted from Robertson GeoConsultant Investigation [18]

'Photo A 24: TP1-P1 profile from 15 to 18m'

'Photo A 59: TP4-P1 sample collected from 5 m'

'Photo A 67: TP5-P1 sample collected from 2 m'

'Photo A 104: TP7-P1 on Intermediate WRD facing Main WRD'

8.1.2 Capping Fill (Proposed Backfill)

A minimum of 2 m thick capping layer is required to cover the Waste Rock material to prevent its exposure to Finniss River and hence mitigate mobilisation of PAF leachate. The capping is to have a surface level approximately 1.0 m below the low season surface water level of the Main Pit. Material won from site excavations and from the granular borrow are envisioned to form this capping layer and consequently the base of the re-aligned Finniss River. Details of the granular material properties are given the Bedding Layer section above and details regarding general site materials likely to be used as capping fill are provided in SLR Geotechnical Report [26]. Preference of capping material will be given to >30° friction angle and non-dispersive materials and relatively low fines (particle size <0.075mm) content (≤30% by weight).

8.2 Geotechnical Design Parameters

Based on a review of the available information described in preceding sections, a set of engineering design parameters have been developed to enable in-pit stability and settlement relating to the in-situ soil and fill units defined in **Table 26**. Graphical plots showing the distribution of material properties and supporting the selection of representative design values are presented in **Appendix F**. These properties are representative values typical of the project-wide geotechnical conditions along the proposed alignments. The interpreted distribution of soil units are shown on representative sections in **Appendix B** and together with material parameters form the basis of backfilling design, performance assessment and controls.

For Tailings materials (Unit T), the proposed consolidation parameters are presented in **Table 27**.

Table 26 Backfill Materials Geotechnical Design Parameters

Unit ID	Description	Material type, consistency or density	Saturated Bulk Unit Weight, γ (kN/m³)	Undrained shear strength, cu (kPa)	Effective shear strength parameters		Poissons Ratio (ν)[3]	Drained (undrained) elastic modulus E' (MPa) [2,5]
					Cohesion c' (kPa)	Friction Angle, ø' (°)		
SB-C		Cohesive / fine grained, Firm	17-19 (18)	35	1	25-30 (28)	0.3	10 (Eu = 12)

Unit ID	Description	Material type, consistency or density	Saturated Bulk Unit Weight, γ (kN/m³)	Undrained shear strength, cu (kPa)	Effective shear strength parameters		Poissons Ratio (v)³	Drained (undrained) elastic modulus E' (MPa) ²,⁵
					Cohesion c' (kPa)	Friction Angle, ø' (°)		
SB-G	Uncontrolled Fill (Soil & Beach deposits)	Granular / coarse grained Very Loose to Loose	18-20 (19)	-	0	25-35 (30)	0.3	0.5 + 0.6 x Z below surface (Max 30MPa)
T	Uncontrolled Fill (Tailings)	Cohesive / normally consolidated	14-18 (16)	Su = 0.5+ 0.25 x P' (Su = 0.5+1.5 x Z below surface) ⁴	0	25-27 (26)	0.35	Refer to consolidation properties
BL	Hydraulically placed sand Bedding Layer (Site-won granitic sand)	Granular / coarse grained, V Loose to Loose	17-20 (18)	-	0	28-32 (30)	0.3	20
WR	Hydraulically placed Waste Rock (Site-won waste rock fill)	Granular / coarse grained, V Loose to Loose	18-20 (19)	-	0	30-35 (32)	0.3	0.5 + 1 x Z below surface (Max 30MPa) Creep @ 0.7% per log time cycle

(1) Parameters reported as range and recommended design value, i.e. Min-Max (Selected).
(2) If required, unload/reload elastic modulus (E_{ur}) can be taken as 5 times the loading elastic modulus (E) for sands and 3 times the virgin loading elastic modulus (E) for clays.
(3) Poisson's ratio (v) shown for drained materials. Use v_u ~ 0.5 for the undrained condition in cohesive materials.
(4) Undrained shear strength profile presented above is pre-filling condition. Strength gain due to consolidation is assessed based on the amount of consolidation occurring with time (Refer to Section 7 for discussion)
(5) Relationship between Eu and E2 taken as Eu = 3E'/2(1+v_u).

Table 27 Consolidation Parameters for Tailings

Geotechnical Unit	Coefficient of consolidation Cv (m²/yr)	Coefficient of consolidation Ch (m²/yr)	Compression Ratio $C_c/(1+e_0)$	Recompression ratio $Cr/(1+e_0)$	Secondary compression ratio, $C\alpha\varepsilon = C_\alpha/(1+e_0)$
T (Tailings)	5-25 (10)	2-40 (20)	0.08-0.2 (0.125)	-	0.003-0.007 (0.0045)

(1) Parameters reported as range with selected value i.e. Min-Max (selected)
(2) Note: Laboratory test data from Rowe Cell indicates Cv range varies from 5m²/yr (low stress) to 25 m²/yr (high stress)

A review of published data regarding the long-term creep behavior of mine backfills has been undertaken to characterize predicted settlements within backfill layers which comprise a relatively thin bedding sand layer (4m) and about 40m of waste rock fill. The modelled behaviour is based on the work of [42]) who proposed that the creep behavior of mine waste rock fills generally follows a log-time relationship in the form:

$$S = \alpha(\log t_1 - \log t_2)$$

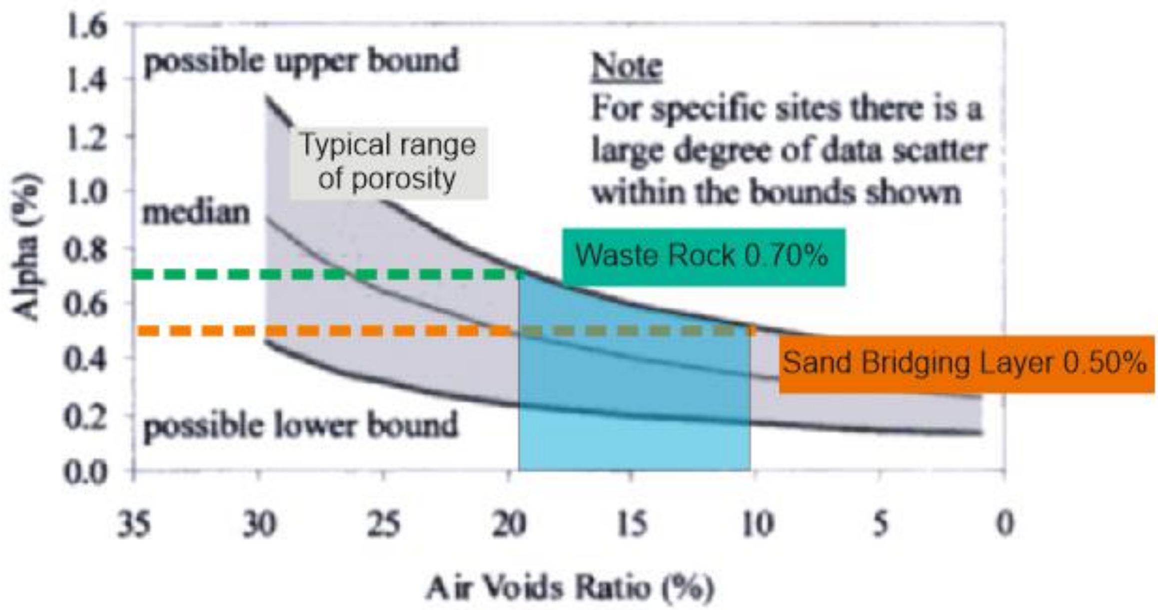
Where:

S = Strain expressed as a percentage of backfill depth
α = creep compression rate parameter
t_1 = beginning of time step (where settlement begins at t_0)
t_1 = end of time step

In practice, definition of t_0 is difficult as the lowest layers of fill will have been placed and started to settle before the upper layers are even placed. Sowers [42] overcame this difficulty by proposing that t_0 be taken as the time when the fill placed had reached half of its ultimate height. The approach adopted for the Main Pit settlement assessment achieves a similar outcome and involves calculating the onset of creep in each individual layer of backfill using the estimate program derived from assumed filling rates then summing the individual creep of placed layers at any point in time.

Typical creep compression rate parameters (α) published by Goodwin and Holden [43] are shown in **Figure 38** below, alongside the parameters adopted in backfilling design.

Figure 38 Published and Selected Mine Backfill Creep Rates



9 Backfilling Strategy and Design Approach

9.1 Backfilling Strategy

9.1.1 Overview and objectives of capping and backfilling design

Objectives of sub-aqueous capping and backfilling above tailings at Rum Jungle Main Pit are to reduce environmental risk, by providing:

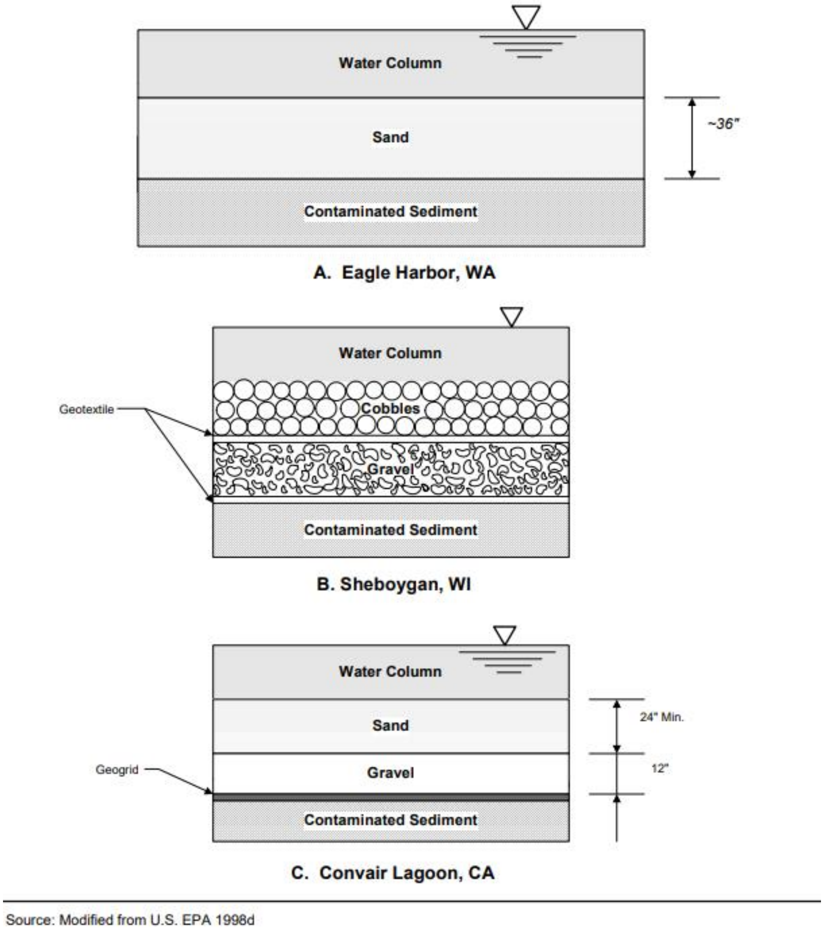
- Physical isolation and stabilisation of existing tailings and sediments;
- Mitigation of chemocline disturbance and mobilisation;
- Containment of tailings porewater due to consolidation of underlying tailings; and
- A bedding layer to enable subsequent placement of waste rock.

Remedial works will include the placement of a sub-aqueous initial capping (or bedding) layer of clean granular material over existing tailings and washed-in sediments which are to remain in place. It is proposed to use capping material consisting of site-won sand borrow material to achieve a suitable capping layer thickness, before introducing waste rock sub-aqueously as bulk backfill.

An option to assist with separation and stability of placed capping at the interface between sand bedding and in-situ tailings involves placement of a geotextile layer prior to bedding layer backfilling. Options to assist with geochemical management of potential chemocline or sediment disturbance include the potential use of flocculants and/or lime to manage water quality.

Typical capping systems used to remediate contaminated sediments are shown schematically below in **Figure 39**. Option A is the preferred concept design for Main Pit capping design, optionally with a geotextile separation layer at the sand capping layer / in-situ sediment interface.

Figure 39 Typical Capping Types (Palermo [44])



9.1.2 Backfill Design process

Design of the Main Pit capping and backfill has been undertaken in general accordance with the following guidelines, to the extent that they are applicable to the proposed works:

- Guidelines for the Environmental Assessment of Marine Dredging in the Northern Territory Version 2.0 [45];
- US EPA Contaminated Sediment Remediation guidance for hazardous waste sites [46]. The latter guidelines include the following key steps in the design process for development of sub-aqueous capping systems:
- Identifying candidate capping materials physically and chemically compatible with the environment in which they will be placed;
- Evaluating geotechnical considerations including consolidation of compressible materials and potential interactions and compatibility among cap components;
- Assessing placement methods that will reduce short-term risk from release of contaminated pore water and resuspension of contaminated sediment during cap placement; and
- Identifying performance objectives and monitoring methods for cap placement and long-term assessment of cap integrity

Each of these design aspects is considered further below.

9.1.1 Capping material suitability

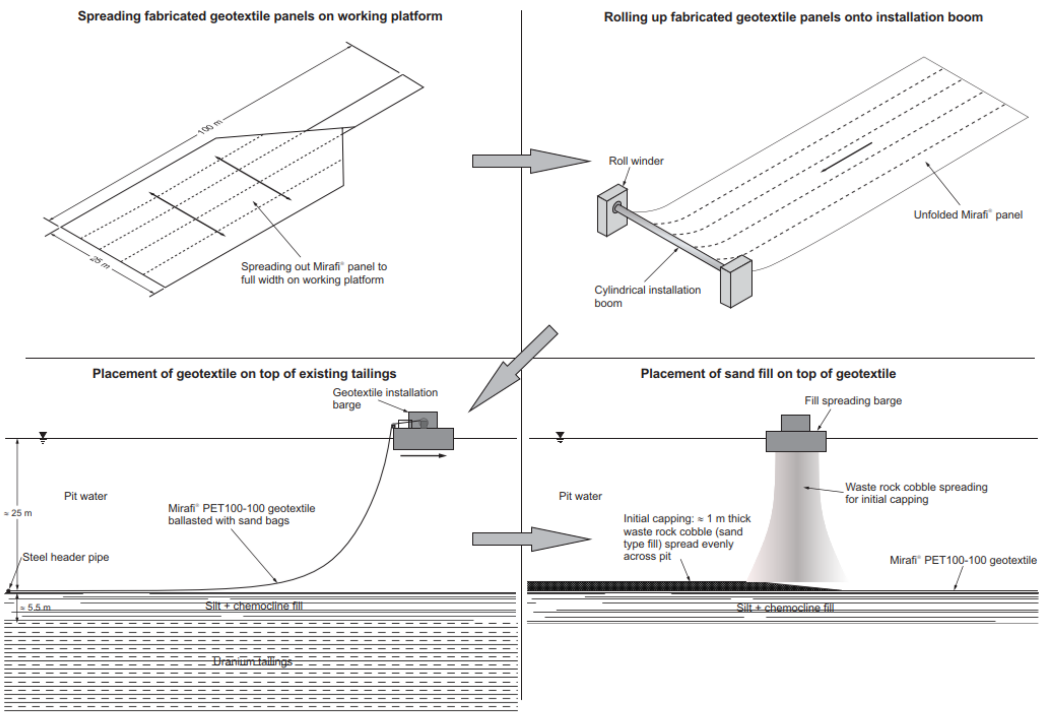
Capping materials used for sub-aqueous remediation by backfilling are generally selected to be clean granular materials, such as sand. The proposed site-won borrow material which proposed to be used for initial capping layers to be placed on tailings may typically be described as a well graded clayey to gravelly SAND with a minor proportion of sandy GRAVEL (**Section 8.1.1.3)**.

Some processing of site-won borrow material by screening is proposed to harvest the finer grade portion of for the initial bedding material (D_{50}≤1mm) and filter out gravel to reduce segregation and chemocline/sediment disturbance. Further discussion of this process is provided in the following sections.

There is no requirement for sand bedding layer to be specifically designed to withstand significant water velocities as the Main Pit lake is a relatively static hydrodynamic environment. As the capping system is designed to be chemically passive, there is also no requirement to use specialized capping materials with enhanced chemical isolation capacity or reactive/adsorptive characteristics (such as activated carbon, apatite, coke, organoclay, zero-valent iron or zeolite). However, options to assist with geochemical management of potential chemocline or sediment disturbance include the potential use of flocculants and/or lime to promote sedimentation of suspended solids and to manage water quality.

The option to include geotextile at the sand capping /sediment interface has been assessed as shown schematically in **Figure 40**.

Figure 40 Concept Design of Geotextile Option



Use of a geotextile separation layer would potentially result in the following advantages and disadvantages:

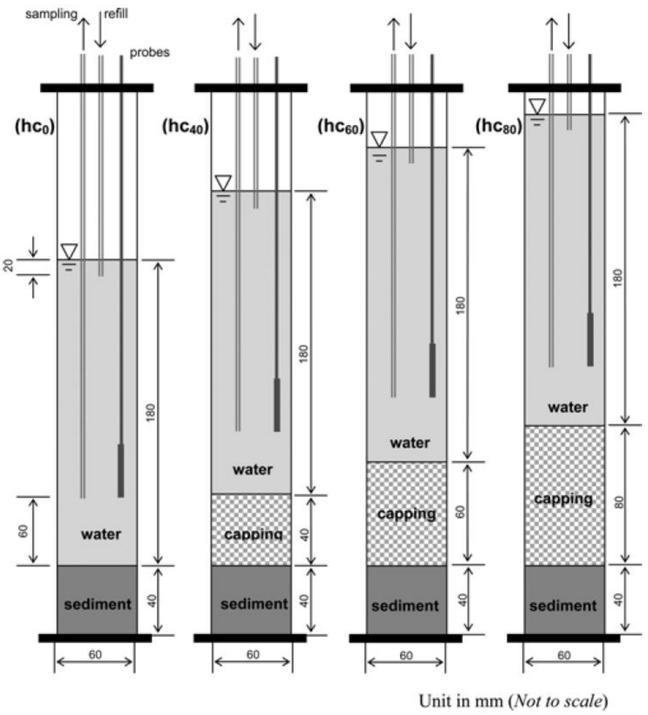
Table 28 Advantages and disadvantages of a geotextile separation layer

<table>
<tr><th>Advantage</th><th>Disadvantage</th></tr>
<tr><td>Tensile reinforcement would potentially enable thicker / more rapid capping layer lifts.</td><td>Limitations in the rate of treatment of displaced lake pit water offset the advantage of increased placement productivity.</td></tr>
<tr><td>Geotextile layer would provide separation between sand capping and underlying chemocline. The degree to which geotextiles would mitigate chemocline disturbance is uncertain but could be estimated from trials (below), noting that the water treatment cost impact of widespread chemocline disturbance could be significant.</td><td>To an extent, the grading sand capping material can be controlled by screening to meet granular filter design rules to achieve separation with tailings (but not chemocline). Disturbance effects could be largely managed by careful sand placement and validated through monitoring.</td></tr>
<tr><td rowspan="3">Geotextile layer would reduce loss of backfill into underlying materials.</td><td>Loss of backfill by penetration and mixing without geotextile is uncertain and would require trials to verify, but based on reclamation experience, about 50% of the first 1m placed may be lost into the in-situ pit lake floor depending on the gradation of the chemocline/sediment boundary. Given the substantial volumes of material involved and aiming for a 3m thick capping layer, this would be a relatively small loss.</td></tr>
<tr><td>Generally, there is increased time, expense and uncertainty associated with sub-aqueous geofabric placement.</td></tr>
<tr><td>Potential cultural considerations relating to the introduction of synthetic materials into backfilling of native lands.</td></tr>
</table>

Discounting the benefits of geotextile reinforcement and material loss mitigation as relatively minor, the business case for using a geotextile separation layer will largely depend on the amount of disturbance that sand capping placement causes to the chemocline and in-situ sediments and the extent to which this can be mitigated by using a geotextile.

It is proposed to use a pre-construction pilot scale sedimentation trial similar to that undertaken by Kim and Jung [47] as pictured conceptually below in **Figure 41**. The outcome of this trial showed that for capped sediment, the concentration of sediment contaminants in the overlying water was very low compared to that of the columns without capping. The Main Pit sedimentation trial could potentially be adapted for site use using pallet mounted intermediate bulk container (IBC) with carefully reconstructed tailings/chemocline/lake pit water profiles progressively placed by tremie using controlled filling tubes. Representative chemocline and upper sediment samples could be obtained from the pit floor using suitable bottom grab samplers, with deeper tailing samples obtained using a piston sampler if required.

Figure 41 Schematic of bench scale sediment column testing



US EPA [46] also report that Environment Canada has performed tank tests on sediments from Lake Ontario to qualitatively investigate the interaction of capping sand and compressible sediments. The tests were carried out in 3.6 x 3.6 x 3. 7 metre observation tanks in which the compressible sediments were placed and allowed to consolidate and sand was placed through the water column onto the sediment surface. In the initial tests, physical layering and consolidation behaviour were observed.

9.2 Geotechnical Considerations

Pit floor investigations have shown that the in-situ Main Pit Tailings and washed in sediments are predominately fine-grained silt/clay fraction materials with moderate to high water content (43-53%) and low shear strength (near-normally consolidated). These materials are relatively compressible and unless appropriate controls are implemented, can be easily displaced or resuspended during backfill placement.

Following placement, bridging layer stability and settlement due to consolidation can become two additional geotechnical issues that may be important for bridging layer effectiveness. The shear strength of the tailings and sediment will influence its resistance to localized bearing capacity or sliding failures, which could cause localized mixing of placed granular materials and contaminated materials. Bridging layer stability immediately after placement is critical, before any excess pore water pressure due to the weight of the backfill has dissipated.

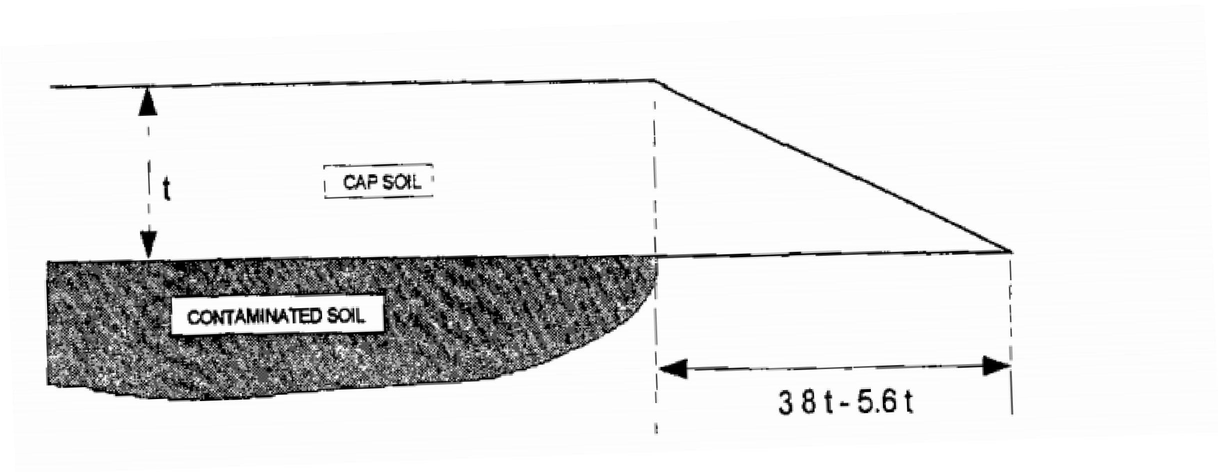
9.2.1 Stability

Gradual placement of sand bridging layer materials in thin, controlled layers over the entire area to be bridged will reduce the potential for localized bearing failures. Site specific stability analyses reported in **Section 9.6** which adopt suitable strength properties for sand backfill and near-normally consolidated sediments (as reported in Section 8.1.1) have shown that formation of an initial 0.5m thick sand capping layer will result in a satisfactory factor of safety against failure near the fill edge provided the face advancing sand fill is formed no steeper than 4H:1V.

This is consistent with the findings of a case study review by US EPA [46] of capping projects where shear strengths of the in-situ sediments were measured was conducted for the ARCS program, and is provided as Appendix C of the original report which referenced at the end of this document. The EPA report showed that conventional slope stability analysis using the measured shear strengths indicated stable conditions for most of the capping projects evaluated (all of which used a sand cap).

As documented in US EPA [46], to achieve a corresponding factor safety, the recommended limiting end slope geometry becomes 1V:3.8H to 5.6H (or 3.8 to 5.6 times the thickness of the cap) as shown in **Figure 42**. Regular bathymetric survey by multibeam survey methods is recommended to confirm that the side slope geometry and limiting lift heights are achieved and that mud-waving of chemocline or sediments are being appropriately managed.

Figure 42 Typical Limiting Fill Slope for Stability



Placement of the initial sand bedding layer particles ($D_{50} \leq 1mm$) by sub-aqueous discharge would result in raining down of quartz sand particles into the basal chemocline profile consisting of increasingly dense and viscous fluid. It is anticipated that as individual sand grains rain down that they would penetrate and displace lighter fractions of chemocline before coming to rest once the frictional resistance of the receiving body exceeds the gravitational force exterted on each grain. As subsequent capping layer material is rained down, a degree of mechanical interlock is likely to form between sand grains and the displaced chemocline will occupy pore space of subsequently placed material.

9.2.2 Consolidation

Consolidation of underlying sediments due to placement of a cap will result in advection of pore water upward into the cap. In addition to settlements arising from the dissipation of pore pressures under backfill loading, there may be a potential advective flux of contaminants up into the cap, laterally and/or down into host rock mass defects. A consolidation evaluation has been undertaken to assess these effects.

Contaminant migration associated with the movement of sediment particulates can be controlled by appropriate filtration design compatibility between the cap and underlying materials. Most contaminants of concern also tend to remain tightly bound to sediment particles. However, the movement of contaminants by advection (movement of porewater) upward into the cap is possible, while movement by molecular diffusion over long time periods is inevitable.

Advection refers to the movement of porewater. Advection can occur as a result of compression or consolidation of the contaminated sediment layer or other layers of underlying sediment. Movement of porewater due to consolidation would be a finite, short-term phenomena, in that the consolidation process slows as time progresses and the magnitude of consolidation is a function of the loading placed on the compressible layer. The weight of the cap will "squeeze" the sediments, and as the porewater from the sediments moves upward, it displaces porewater in the cap. The result is that contaminants can move part or all the way through the cap and into overlying backfill layers over time. This advective movement can cause a short-term loss, or it can reduce the breakthrough time for long-term diffusive loss. Due to the overall thickness of backfill at the Main Pit, it is anticipated that the available pore space will be sufficient to accommodate the consolidation tail water arising from underlying sediments.

For example, as reported in the US EPA Report [46], Bokuniewicz [48] has estimated that the pore waterfront emanating from a consolidating two-meter-thick mud layer would only advance 24 cm into an overlying sand cap [49]. Factoring this up for the Rum Jungle Main Pit (max. 56 m thick of tailings) would result in about 6.5m of consolidation tailwater advection into overlying sand and waste rock.

This empirical example can be cross checked by comparing the volume of tailwater expected to be generated by a column of tailings based on predictive settlement analyses undertaken by SLR (**Section 9.5**). This shows that about 4 to 5 m of long-term settlement is expected within a 56 m deep profile (i.e. approximately 4,500 litres of tailwater per m^2 of tailings area). Comparing this with the expected porosity of sand capping (porosity (v) = 0.3) and overlying well graded gravelly waste rock (v = 0.2), then the total thickness of overlying materials required to accommodate upwards advection due to consolidation would be the 4 m of planned bedding layer sand and 16.5 m of waste rock, for a total backfill thickness of 20.5 m. With an average backfill thickness of about 40 m above the pit lake floor, this demand is easily satisfied with a factor of safety of between 2 to 3.

Diffusion is the process whereby ionic and molecular species in water are transported by random molecular motion from an area associated with high concentrations to an adjacent area associated with a low concentration (Fetter [50] as reported in US EPA [46]). Diffusional mass transport assumes that the rate of transport is directly proportional to the concentration gradient. In an isotropic medium, this occurs in a direction perpendicular to the plane of constant concentration at all points in the medium. If the diffusional flux is steady-state, mass transport by diffusion is described by Flick's first law (Fetter [51]). Fick's second law is used to describe systems in which the contaminant concentrations are dependent upon time.

From an environmental perspective, diffusion is as slow as contaminant transport processes can become in a porous medium. However, although diffusion is notoriously slow, diffusional driven mass transport will always occur if concentration gradients are present. Consequently, diffusion can transport contaminants through a saturated porous media in the absence of advection. Advection and/or diffusion transport processes can be viewed as end-members of a continuum. Based upon random molecular motion attempting to equalize contaminant concentrations, diffusion is commonly the slower of these two processes (Fetter [51]). In contrast, advection as the bulk movement of ground water due to differences in hydraulic head is generally a much more rapid transport process. In many/most geologic settings, mass transport is driven by advection (Fetter [51]; Bear and Verruijt [52]).

Generally, predictions of contaminant transport based upon diffusion alone would only become appropriate for geologic settings and/or cap designs which incorporate a porous layer associated with a very low hydraulic conductivity value, or in the absence of hydraulic gradients (the hydrostatic case) (Fetter [51]). Even if contaminant concentrations are high in the pore water, a granular cap component would act as both a filter and buffer during advection and diffusion. As pore waters move up into the relatively uncontaminated granular cap material, these cap materials can be expected to remove contaminants (through sorption, ion exchange, surface complexation, and redox mediated flocculation) so that pore water that travelled completely through the cap would theoretically have a reduced contaminant concentration. The extent of the contaminant removal in the cap is very much dependent upon the nature of the cap materials. For example, a cap composed of quarry run sand would not be as effective as a naturally occurring sand with an associated fine fraction and organic content.

In summary, based on the significant advection capacity of the Main Pit backfill, it is anticipated that this would also be easily sufficient to accommodate potential upward migration of contaminants via the slower diffusion process. However, in the event that bench and field trials and production water quality monitoring near the pit floor show that the chemocline and/or slimes are displaced rather than absorbed into the lime-dosed backfill, then contingency measures of collecting and decanting geo-chemically problematic layers should be explored.

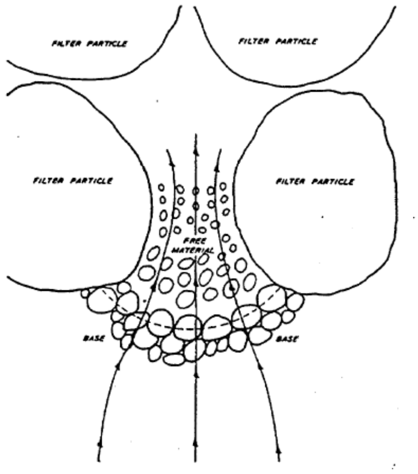
9.2.3 Separation and filtration at capping / sediment interface

To inform the decision process around potential use of a geotextile separation layer, an assessment of the natural filtration compatibility between tailings/sediments and sand borrow material has been undertaken. As mentioned in above, this assessment does not explicitly examine the filtration/separation compatibility between sand bedding and chemocline, which could only reasonably be assessed using a site trial or through advanced modelling using appropriate constitutive models which appropriately model sedimentation and pore fluid mechanics.

A simplistic assessment of the grading compatibility between tailings/sediments and sand borrow material has been undertaken using published empirical data commonly used for the graded design of filters for foreshore works, drainage, dam cores, etc. The typical grading curves for main pit tailings and washed in / end-dumped soils is shown in **Figure 29**.

Granular filter design rules governing the selection of grading to prevent piping or migration of soils under hydraulic gradients have been developed and published by empirical experimentation. The process is shown schematically below in **Figure 43**.

Figure 43 Granular Filter at Capping/Tailings Interface (Image: USACE [53])



Filter design rules published by Unites States Army Corps of Engineers [53] are tabulated below and fall into two groupings comprising uniform and graded filters. As the Rum Jungle sand borrow material is relatively well graded, the graded filter rule is considered to be appropriate, however the uniform filter rule is also shown for completeness.

Table 29 Typical filter grading rules

Source	USACE Uniform Filter D_{50}/d_{50}	USACE graded filter D_{50}/d_{50}
USACE Criteria Recommendation	5 to 10	12 to 40
Rum Jungle Main Pit Sand Bedding/Tailings particle size from grading curves	0.25 / 0.01 = 25	0.25 / 0.01 = 25

Note:
Terminology: Underlying tailings(d), Sand bedding (D) – corrected for screening of 50% volume to D_{50}≤1mm

Based on the above, it can be seen that use of the screened site-won sand (D_{50}≤1mm) placed directly on the silty tailings would, on average, satisfy filter grading rules and limit the upwards migration of fine material into overlying capping material.

9.3 Placement Methods

Important considerations in selection of placement methods at the subject site include the need for controlled, accurate placement of capping materials. Slow, uniform application that allows the capping material to accumulate in layers is necessary to avoid displacement of or mixing with the underlying sediment and if possible, chemocline. Uncontrolled placement of the capping material can also result in the resuspension of adverse suspended solids and/or contaminants into the water column and the creation of a fluid mud wave that results in problematic build up.

Examples of typical capping layer placement methods are shown in **Figure 44** and **Figure 45** below (from US EPA 2005 [46]).

Granular cap material can be handled and placed in a number of ways. Mechanically excavated materials and soils from the proposed borrow areas are relatively dry can be handled mechanically in a dry state until released into the water over the contaminated site. Mechanical methods (e.g., clamshells or controlled release from a barge) rely on gravitational settling of cap materials in the water column and can be limited by depth in their application, especially if well graded when segregation in the water column can change the intended performance of the capping.

Granular cap materials can also be entrained in a water slurry and carried to the contaminated site wet, where they can be discharged by pipe into the water column at the water surface or at depth. These hydraulic methods offer the potential for a more precise placement, although the energy required for slurry transport could require dissipation using an underwater diffuser to prevent resuspension of contaminated sediment (**Figure 45**). Placement of the optional geotextile layer would require special equipment.

Figure 44 Examples of Typical Capping Layer Placement Methods (Image: US EPA [46])

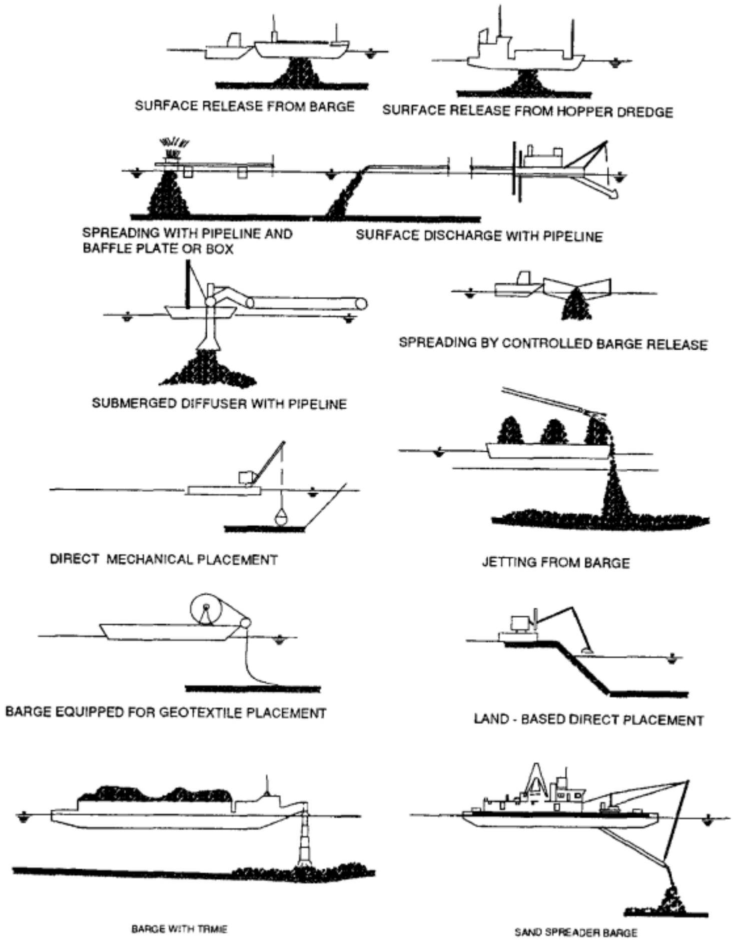
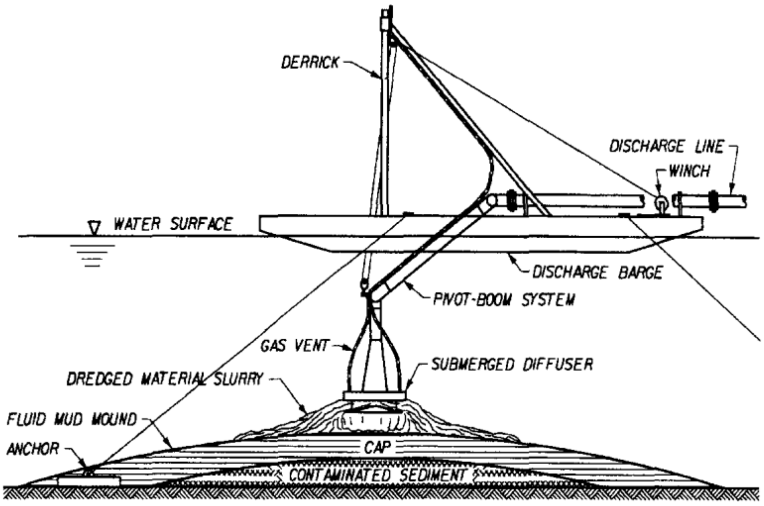


Figure 45 Example of Spreader Pontoon with Diffuser



During reference design development, a literature review was undertaken and discussions were held with a specialist marine contractor to identify suitable material handling and placement techniques.

Key constraints considered when assessing a preferred placement methodology include:

- Limiting rate of water treatment plant treating pit lake water which is displaced by backfill materials: The maximum filling rate is based on a limiting water treatment capacity of 80 L/S which governs the fill placement rate. This equates to a limiting maximum placement rate of 6,900m^3 per day.
- Material supply rates (barge, conveyor, pipeline): Discussions with specialist contractors have been undertaken to verify appropriate placement rates at various stages of filling. Typical rates are as follows:
  - Floating line to barge with sub-aqueous placement of fluidized sand slurry by spreader pontoon with diffuser for careful and controlled placement of basal sand and intermediate sand bedding layers: 60-80 m^3 per hour (600-800 m^3 per day considering 10 hours production)
  - Split Barge or pontoon for careful and controlled placement of wet or dry basal sand bedding layer and waste rock: 120-150 m^3 per hour (1,200-1,500 m^3 per day considering 10 hours production). Multiple barges may be used.
  - Conveyor: Up to 1,000 m^3 per hour (10,000 m^3 per day) for land-based transport of dry basal sand bedding layer and waste rock. The system could either deliver to receiving stockpiles at pit rim (for to hoppers for loading on to barges) or be connected to floating pontoons for use over water.
  - Side cast by Dozer: Placement rates of up to 1,000 m^3 per hour are achievable for a large (CAT D10+) dozer operating short haul distances.
- There is a need to place backfill material into the pit lake very carefully to avoid disturbance of the chemocline: Refer to **Section 9** for further discussion of fill-chemocline interaction.
- Stability of capping: Sub aqueous placement of predominantly granular materials will result in a very loose to loose matrix which may exhibit creep settlement over time (See **Section 9.5**) could undergo bearing or slope stability failure on very soft tailings (See **Section 9.6**) and may be prone to liquefaction (See **Section 10.1**).

- Chemical suitability of backfill materials: Potentially Acid Forming (PAF) waste rock must be kept submerged below the lowest (dry season) water level which is taken to be RL 59 m AHD. For this reason, the final layer of PAF waste rock is placed no higher than **RL +56m AHD** below a 2.0m thick inert capping material (**RL 58m AHD**) with a minimum **1.0 m** water cover at the peak of dry season. Settlement analyses show that the upper surface of the PAF material will settle several metres over a period of decades which provides risk mitigation against oxidization of these materials (i.e. the amount of favourable water coverage over oxidizable materials will increase with time).
- The final landform of the backfill will be profiled to maintain stream flow, minimum channel depth, operational surface drainage (to avoid ponding), appropriate landscaping and erosion management.

Taking into consideration these constraints, various placement methods and strategies have been considered and the following key outcomes of the design development are summarised below:

- Similar to the proposed overwater dumping approach in RGC report [6], a suitable borrow and placement technique could utilise bulk (dry) excavation of sand borrow material, processing (screening), stockpiling, then fluidisation by end tipping into receivals bin, pumping using a wet placement process through floating line and discharge from near water surface via spreader pontoon. An alternative to this would be to utilise a dry transport process involving land and floating conveyor systems instead of a slurry filled pipeline to transport material to a spreader pontoon (barge) for controlled sub-aqueous placement.
- For wet placement methods (involving slurry/pipeline) a relatively fine grading of bedding layer material is preferred (D_{50}≤1mm) to reduce impeller wear and pump maintenance. On this basis, it would be necessary to screened and stockpile oversize material for later barge placement as intermediate grade fill before placing waste rock. As shown in grading plots, the approximate split of borrow material between sand bedding (D_{50}≤1mm) and intermediate sand fill (D_{50}≥1mm) is about 50/50, and so this can be used to balance stockpile volumes for 1mm minus, unscreened and 1mm plus bedding materials.
- The rate of placement is effectively governed by the capacity of the water treatment plant which extracts and treats pit lake water as it is displaced by capping and backfill. The maximum water treatment (and therefore fill placement) rate is understood to be 80-90L/s.
- Target filling levels are such that that PAF waste rock must remain submerged under the lowest expected dry season water level of RL 59 m AHD. Adding the nominal 2m thickness of inert (non-PAF) surface capping layer material, results in a target landform surface level of **RL +58m AHD**. Further consideration of the landform design is required which needs to take into consideration post construction total and differential settlements, grade of submerged waterways and finished landform (considering erosion, etc).
- Placement of first 1000 mm of bedding sand may result in 500mm lost into surface of tailings (to be factored into program and volumes – see **Section 9.4**).
- Hydraulic fill placement has the potential to disturb and displace a liquid / slurry chemocline layer where this exists. Further assessment of the type and thickness of potential chemocline layer is discussed above.
- Following sampling, testing and characterisation of chemocline baseline conditions, then regular construction stage monitoring should be undertaken to confirm that chemocline disturbance is being managed to achieve acceptable groundwater quality levels.

- Regular multibeam survey (intense initially) is required to confirm placement coverage, check for instability and mud-waving. Periodic CPT validation testing should also be undertaken to confirm settlement of the capping/tailing interface and confirm excess pore pressure and strength gain assumptions at critical staging.

During placement, it will be necessary to undertake construction quality assurance (CQA) to confirm design assumptions regarding the density, volume, stability and settlement of backfill materials and associated geochemical conditions. These requirements will be incorporated into the earthworks specifications as follows:

- Grading conformity of bedding layers and elimination of oversize particles;
- Progressive CPT testing to confirm material density and absence of segregation, tailings strength gain and/or excess porewater pressures (to confirm consolidation and settlement behaviour);
- Regular chemical testing of pit lake water and chemocline to manage disturbance.
- Requirements for Instrumentation and Monitoring (I&M) and production of an I&M Plan as discussed further in **Section 11**, below.

Based on literature review, discussions with specialist marine and conveyor contractors and consideration of the unique constraints affecting backfilling operations at this site as described above, the preferred backfilling methods and sequence is outlined in **Figure 46** below and a conceptual layout of operations required for underwater fill placement are shown on

Figure 47.

The layering of proposed fill materials is illustrated in **Figure 48** and further detailed in design drawings, noting that the design intent is to spread the initial bedding layers to provide complete coverage of the tailings and chemocline. This extent has been assessed based on the estimated thickness or chemocline (approximately 2m), noting a pit floor of RL 16.5m, and taking the lateral extent of the tailings as being within an area where pit floor slopes are within the estimate angle of repose of tailings (5-10°). These combined assumptions require that a minimum of 3 m of sand bedding is placed above the existing pit floor across an area defined by the RL 19 m bathymetric contour. The method of placement shall be from the centre of the Main Pit outwards to buttress the fill layers and mitigate against slope instabilities.

Figure 46 Schematic Backfill Sequence

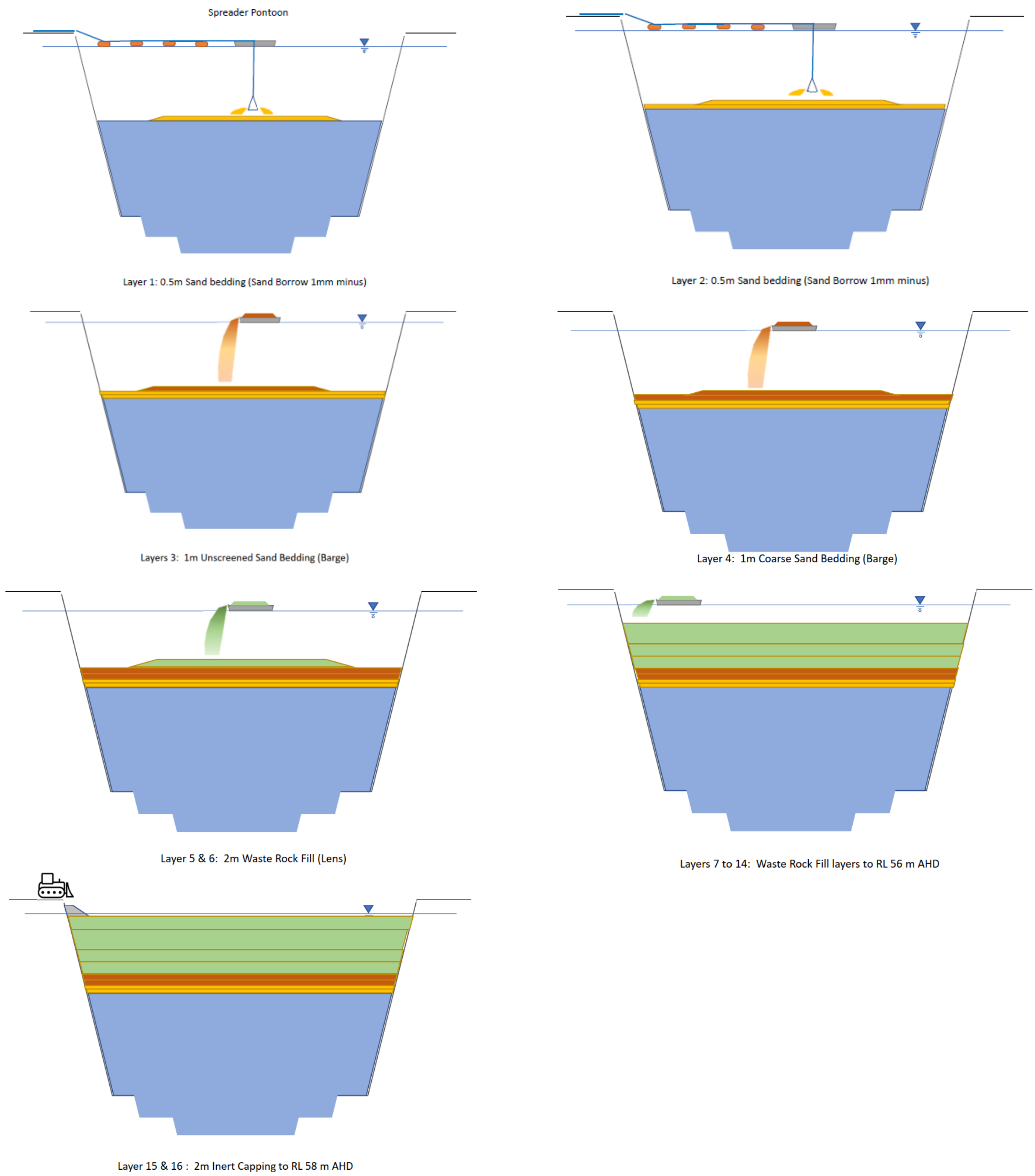


Figure 47 Anticipated Layout of Typical Backfilling Operations

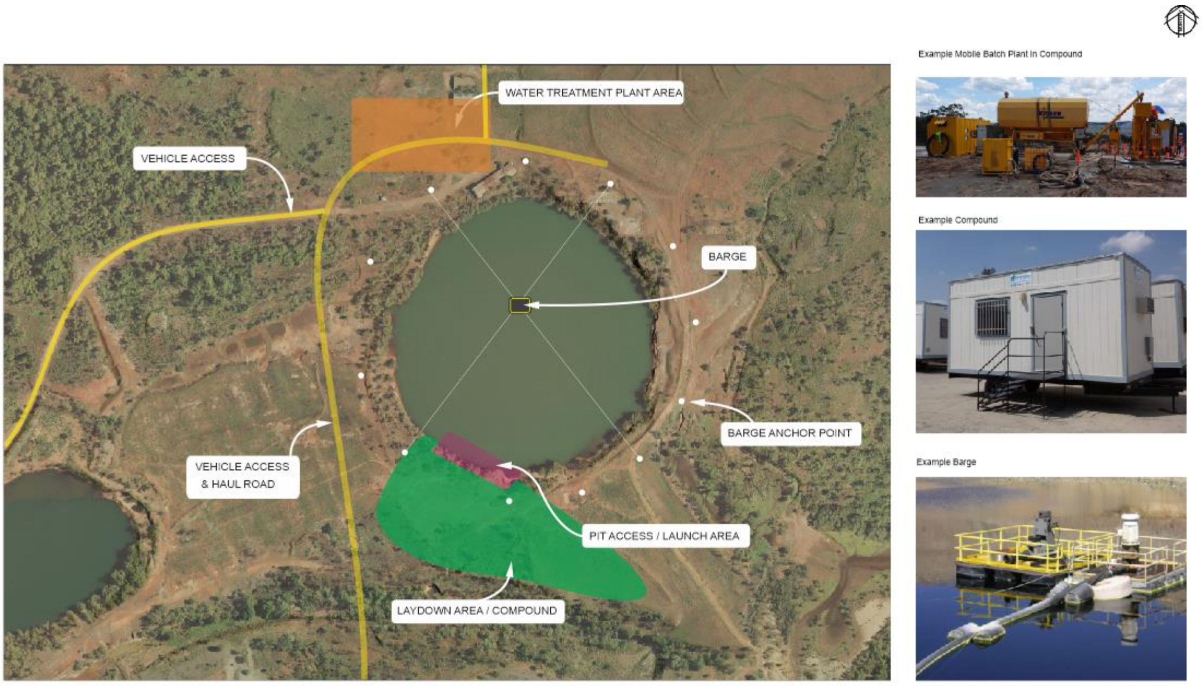
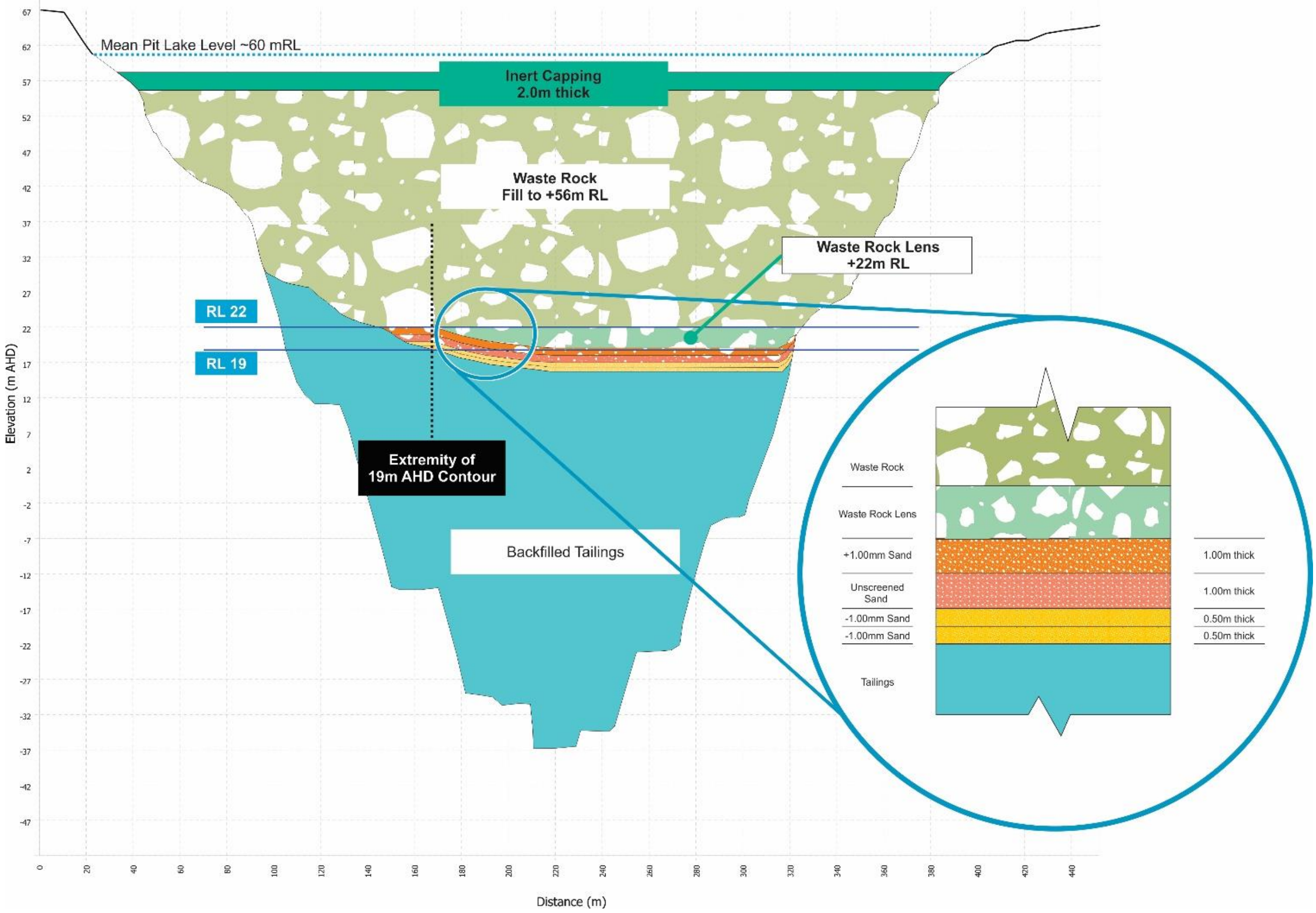


Figure 48 Proposed Layering of Fill Materials

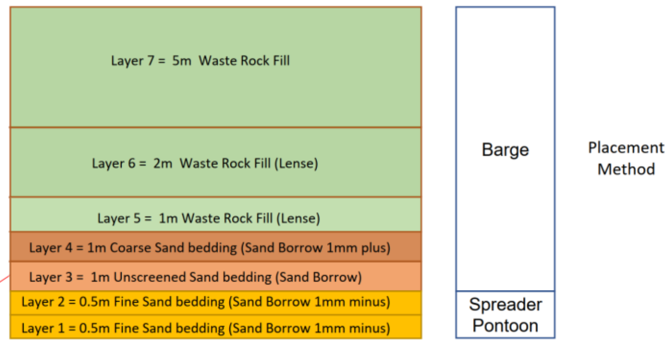


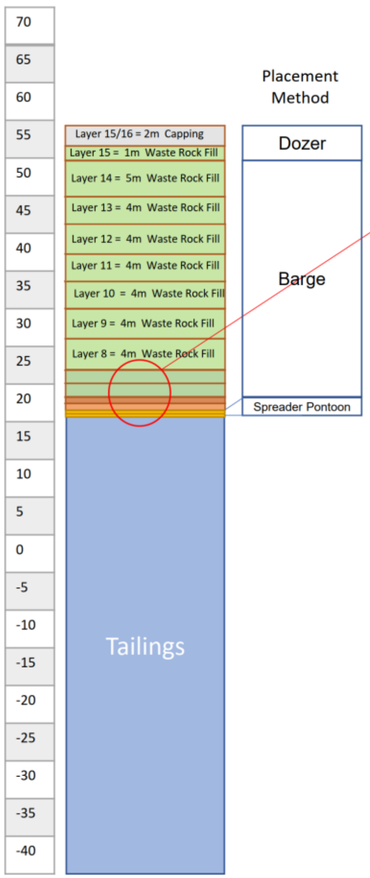
Based on the constraints and adopted methodology above, it is possible to estimate the volume and the estimated placement timeframes for individual backfill layers and the pit backfilling as a whole, as shown in **Figure 49**.

The corresponding backfilling program identifying the time and volume of each backfill layer is presented in **Table 30.** It should be noted that the volumes presented in the table are calculated from a 3D survey model of the pit shell and further adjustments to fill demand and fill placement estimates are required in consideration of tailings settlement, bulking (bank-to-truck/barge and barge/truck-to-placed, fill compression during placement, fill creep after placement and general earthworks losses).

The chosen placement process described above has been used as a basis for stability and settlement calculations presented in the following sections.

Figure 49 Backfilling Strategy and Indicative Program





Time Inc.	Water Table RL	Surface RL	Load [kPa] Change	Load [kPa] Total	Start Date	Finish Date	Time [years] Increment	Time [years] Elapsed	Description	Months
Start:	61.0	16.0		0.0				0		
1	61.0	16.5	4.6	4.6	01/06/2020	18/07/2020	0.129	0.13	0.5m Fine Sand Bedding (Pontoon)	1.6
2	61.0	17.0	4.6	9.2	18/07/2020	02/09/2020	0.126	0.26	0.5m Fine Sand Bedding (Pontoon)	1.5
3	61.0	18.0	9.2	18.4	02/09/2020	10/10/2020	0.103	0.36	1m Unscreened Sand Bedding (Barge)	1.2
4	61.0	19.0	9.2	27.6	10/10/2020	14/11/2020	0.096	0.45	1m Unscreened Sand Bedding (Barge)	1.1
5	61.0	20.0	9.2	36.8	14/11/2020	25/11/2020	0.032	0.49	1m Coarse Sand Bedding (Barge)	0.4
6	61.0	22.0	18.4	55.1	25/11/2020	12/12/2020	0.047	0.53	2m WR Fill (Barge) (Lense Max 2m thick)	0.6
7	61.0	27.0	46.0	101.1	12/12/2020	07/03/2021	0.232	0.77	5m WR Fill (Barge/Conveyor)	2.8
8	61.0	31.0	36.8	137.9	07/03/2021	16/04/2021	0.110	0.87	4m WR Fill (Barge/Conveyor)	1.3
9	61.0	35.0	36.8	174.6	16/04/2021	31/05/2021	0.124	1.00	4m WR Fill (Barge/Conveyor)	1.5
10	61.0	39.0	36.8	211.4	31/05/2021	21/07/2021	0.139	1.14	4m WR Fill (Barge/Conveyor)	1.7
11	61.0	43.0	36.8	248.1	21/07/2021	16/09/2021	0.155	1.29	4m WR Fill (Barge/Conveyor)	1.9
12	61.0	47.0	36.8	284.9	16/09/2021	20/11/2021	0.177	1.47	4m WR Fill (Barge/Conveyor)	2.1
13	61.0	51.0	36.8	321.7	20/11/2021	02/02/2022	0.203	1.67	4m WR Fill (Barge/Conveyor)	2.4
14	61.0	56.0	46.0	367.6	02/02/2022	18/05/2022	0.288	1.96	5m WR Fill (Barge/Conveyor)	3.5
15	61.0	59.5	32.2	399.8	18/05/2022	02/07/2022	0.125	2.09	3.5m WR (Barge/Conveyor)	1.5
16	61.0	61.5	22.8	422.6	02/07/2022	25/07/2022	0.06	2.15	2m Capping (Dozer) i	0.8
17	61.0	61.5	0.0	422.6	25/07/2022	25/07/2032	10.00	12.15	10yr Post Filling	120.0
18	61.0	61.5	0.0	422.6	25/07/2032	25/07/2072	40.00	52.15	50yr Post Filling	480.0
19	61.0	61.5	0.0	422.6	25/07/2072	26/07/2122	50.00	102.15	100yr Post Filling	600.0

Table 30 Backfilling Layer Sequence and Volume Summary

Time Step	Water Level	Surface RL	Load Change	Total	Description	Time to Place	Cumulative Thickness	Top RL	Volume	Cumulative Volume
	(m AHD)	(m AHD)	(kPa)	(kPa)		(months)	(m)	(m AHD)	(m^3)	(m^3)
0	61	16	0	0	Tailings					
1	61	16.5	4.595	4.595	0.5m Fine Sand Bedding (Pontoon)	1.6	0.5	16.5	14,409	14,409
2	61	17	4.595	9.19	0.5m Fine Sand Bedding (Pontoon)	1.5	1	17	14,072	28,481
3	61	18	9.190	18.38	1m Unscreened Sand Bedding (Barge)	1.2	2	18	26,834	55,315
4	61	19	9.190	27.57	1m Coarse Sand Bedding (Barge)	1.1	3	19	24,881	80,196
5	61	20	9.190	36.76	1m WR Fill (Barge) Lens	0.4	4	20	22,340	102,536
6	61	22	18.38	55.14	2m WR Fill (Barge) Lens (Max 2m thick)	0.6	8	22	33,485	136,021
7	61	27	36.76	101.09	5m WR Fill (Barge/Conveyor)	2.8	11	27	164,372	300,393
8	61	31	36.76	137.85	4m WR Fill (Barge/Conveyor)	1.3	15	31	155,894	456,287
9	61	35	36.76	174.61	4m WR Fill (Barge/Conveyor)	1.5	19	35	175,795	632,082
10	61	39	36.76	211.37	4m WR Fill (Barge/Conveyor)	1.7	23	39	197,630	829,712
11	61	43	36.76	248.13	4m WR Fill (Barge/Conveyor)	1.9	27	43	220,092	1,049,804
12	61	47	36.76	284.89	4m WR Fill (Barge/Conveyor)	2.1	31	47	251,341	1,301,145
13	61	51	36.76	321.65	4m WR Fill (Barge/Conveyor)	2.4	35	51	287,360	1,588,505
14	61	56	45.95	367.60	5m WR Fill (Barge/Conveyor)	3.5	40	56	408,031	1,996,536
15	61	58	18.38	385.98	2.0m Clean Capping Fill (Barge/Conveyor)	1.5	42	58	176,692	2,173,228
									TOTAL VOLUME	**2,173,228**
									Waste Rock Only	**1,916,340**
									-1.0mm Fine Sand	**28,481**
									Unscreened Sand	**26,834**
									+1.00mm Coarse Sand	**24,881**
									Clean Capping	**176,692**

9.4 Volume Adjustments

The theoretical CAD generated volumes shown in **Table 30** are a 'lower bound' estimate of volume which does not take into consideration settlements and losses, summarised in **Table 31**.

It will be necessary to account for the following adjustments when assessing fill demand and placement volumes:

- Bulking Factors: Net bulking (bank-to-truck-to-placed underwater), assume:
    - 1.5 for rock (Waste Rock (WR) mined from intact rock mass),
    - 1.2 (Waste Rock (WR) excavated from existing Waste Rock fill areas)
    - 1.1 for granular borrow.
- For intermediate haulage calculations requiring estimation of truck and barge movements, add approx. 10-20%, to the above net bulking values for cartage volume.
- Immediate compression during filling under self-weight would reduce the net bulking by up to 2% to 4%, say 3% (separate to time-dependent creep which is also identified below).

Table 31 Backfilling Volume Adjustments

	Volume Case	Sand Bedding (SB) (1mm minus, unscreened, 1mm plus)	Waste Rock (WR)	Comment
Material lost into surveyed surface of tailings below RL 19 contour	Lower bound	0 mm of first layer (i.e. 0% 1st layer)	N/A	Affected by interaction with chemocline – to be confirmed through field trials
	Base Case	250 mm of first layer (i.e. 50% 1st layer)	N/A	
	Upper bound	500 mm of first layer (i.e. 100% 1st layer)	N/A	
Immediate fill compression loss during placement (under self-weight)	Lower bound	0% SB Layer Thickness	2% WR Layer Thickness	Assume +/- variability is normally distributed
	Base Case	1% SB Layer Thickness	3% WR Layer Thickness	
	Upper bound	2% SB Layer Thickness	4% WR Layer Thickness	
Compensation for creep settlement within fill (See Section 9.5)	Lower bound	N/A*	0.5 % WR Layer Thickness	Assume +/- variability is normally distributed
	Base Case	N/A*	0.8 % WR Layer Thickness	
	Upper bound	N/A*	1.2 % WR Layer Thickness	
Handling Losses	Lower bound	N/A*	0.1 % WR Layer Thickness	Estimated due to screening, crushing, loss of fines, unsuitable
	Base Case	N/A*	0.3 % WR Layer Thickness	
	Upper bound	N/A*	0.5 % WR Layer Thickness	

	Volume Case	Sand Bedding (SB) (1mm minus, unscreened, 1mm plus)	Waste Rock (WR)	Comment
Compensation for settlement of tailings (See Section 9.5)	Lower bound	N/A	1.4% Tailings Layer Thickness	Adjustment based on 1m settlement at completion of filling in centre of pit (56m of tailings). To be adjusted pro-rata according to depth of tailings/soil thickness (i.e. nil settlement loss at edge of pit)
	Base Case	N/A	1.8% Tailings Layer Thickness	
	Upper Bound	N/A	2.3% Tailings Layer Thickness	

*Small enough to ignore

9.5 Settlement Analysis

9.5.1 Previous Settlement Assessment (ATC Williams, 2019)

A memo prepared by ATC Williams (Ref. 117213.01-M002, dated 17 April 2019 [24]) was provided to SLR with preliminary results of consolidation modelling to assist with estimating backfill volume requirements and settlement behaviour of tailings under backfill overburden.

Compressibility parameters for tailings were assessed based on testing of samples from the investigation campaign described **Section 4.1**. Specifically, consolidation properties were based on a single Rowe Cell consolidation test undertaken on a composite sample derived from sub-samples obtained at several depth intervals in the sonic borehole drilled at the location of 18CPT10.

Key assumptions included in the ATCW settlement analysis include:

- Filling to a final design level of RL 56.5 m AHD (SLR base case assumes filling to RL 58 m AHD)
- Filling over a period of 22 months at a rate of 360 t/hr (SLR base case assumes 25.8 months filling program at an average rate of 222 t/hr, assuming 24/7 operations in calculation)
- Backfill density of 18 kN/m^3 (SLR base case assumes same).

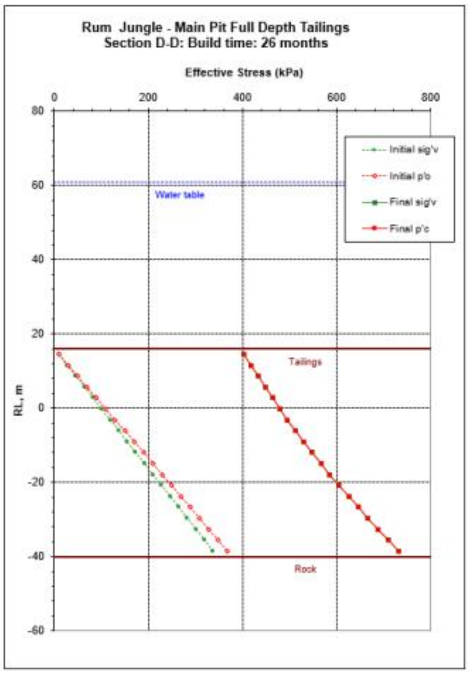
The resulting ATC Williams settlement prediction indicated that up to 6.6 m of settlement could occur over a period of 170 years following backfilling, requiring pre-emptive ‘doming’ of the finished surface to offset the expected ‘dishing’ of the backfill surface over time. The equivalent SLR base case tailings settlement at 170 years (by extrapolation of **Figure 54**) indicates about 1.1m of tailings settlement during filling plus 4.5 m of post-filling settlement giving a total comparable equivalent of 5.6m settlement at 170 after commencement of filling (within about 20% of the ATC estimate).

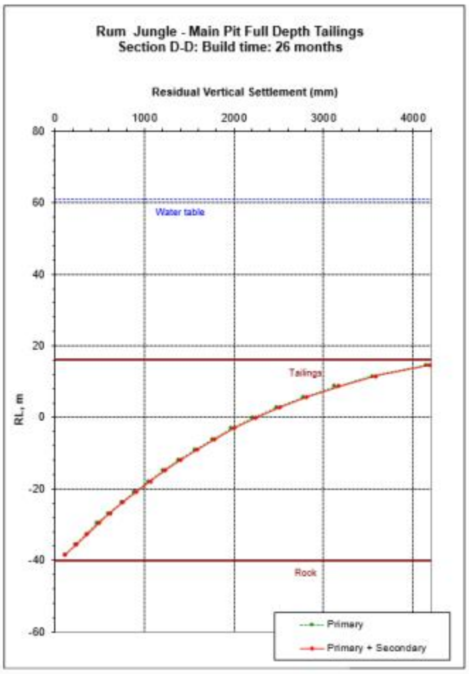
9.5.2 Settlement Input Assumptions

Base Case compressibility values for each soil and fill unit are shown in **Table 26** and are also summarised in settlement outputs presented in **Appendix G - Figure G10** in the parameter plots in the settlement spreadsheet input table below.

Figure 50 Settlement Input Summary

INPUT PROFILE:					Total Unit Weight		Layer		Normally Consolidated			Cc/Cr	Initial Stress Conditions (below initial surface RL for loading)					
Layer No.	Description	Depth to Top	RL Top	Thickness (m)	[not buoyant] below WT (kN/m³)	above WT (kN/m³)	Drainage Code*	$\bar{B}$	$\frac{Cc}{1+e_0}$	Cv [initial] (m²/yr)	Cα [% strain] (%)	[ratio of NC to OC slopes]	Sig ' v Layer Top (kPa)	Over-consolidation Factor (ratio)	Offset (kPa)	Maximum Past Pressure Layer Top (kPa)	Bottom (kPa)	Not less than (kPa)
1	Tailings	0.00	16.00	56.00	16.00	16.00	2	1	0.13	10.00	0.450%	6.00	0.00	1.09	0.5	0.50	377.28	0.50
2	Rock	56.00	-40.00				0						346.64			0.00		
3																		





9.5.3 Settlement Design Methodology

Settlement has been estimated using general one-dimensional consolidation theory for cohesive tailings (primary and secondary consolidation) and estimating creep within granular backfill according to the log-time relationship described in Section Settlement Design Methodology **Section 9.5.3**.

Other simplifying assumptions used in the settlement analysis include:

- Simplified geometry of centre pit profile containing granular back fill over consolidating tailings below RL 16 m RL.

- Two-way drainage (up into bedding layer and down into host rock). The drainage potential of interbedded sand layers or laterally into the pit walls has been conservatively ignored (from a timing perspective). However, from a borrow demand perspective, faster drainage would result in accelerated consolidation and increased borrow demand to compensate for settlements during backfilling. Slower drainage may also occur due to reducing permeability as the tailings consolidate and so to cover a range of outcomes, sensitivity cases of faster consolidation (Average Cv = 20m^2/yr) and slower consolidation (Average Cv = 5m^2/yr), have been compared to a base-case time-dependent settlements associated with an average Cv = 10m^2/yr. Effect of localised interbedded granular materials within tailings (inferred slope wash) has been ignored. This is generally conservative as such layers will improve the rate of settlement of tailings by acting as drainage layers and will reduce the overall settlement due to their relatively high stiffness compared to tailing. Differential settlements at surface are expected to be non-problematic due to the depth of overlying backfill which will tend to mask variability within tailings.
- Effect of localised interbedded granular materials within tailings (inferred slope wash) has been ignored. This is generally conservative (for timing) as such layers will improve the rate of settlement of tailings by acting as drainage layers and will reduce the overall settlement due to their relatively high stiffness compared to tailing. Differential settlements at surface are expected to be non-problematic due to the depth of overlying backfill which will tend to mask variability within tailings.
- Based on feedback received from NT Government no pre-emptive doming of fill has been allowed for in the settlement analysis. This would potentially result in fill placed to RL 58 m AHD settling by about 3.5 to 4 m over 100 years (i.e. to RL 54 to 54.5 m RL) which would be below the dry season low water level of RL 59 m AHD. Correspondingly, this would result in formation of a large, shallow pit lake over a significant area of the Main Pit as the surface settles below the seasonal average groundwater level over time.

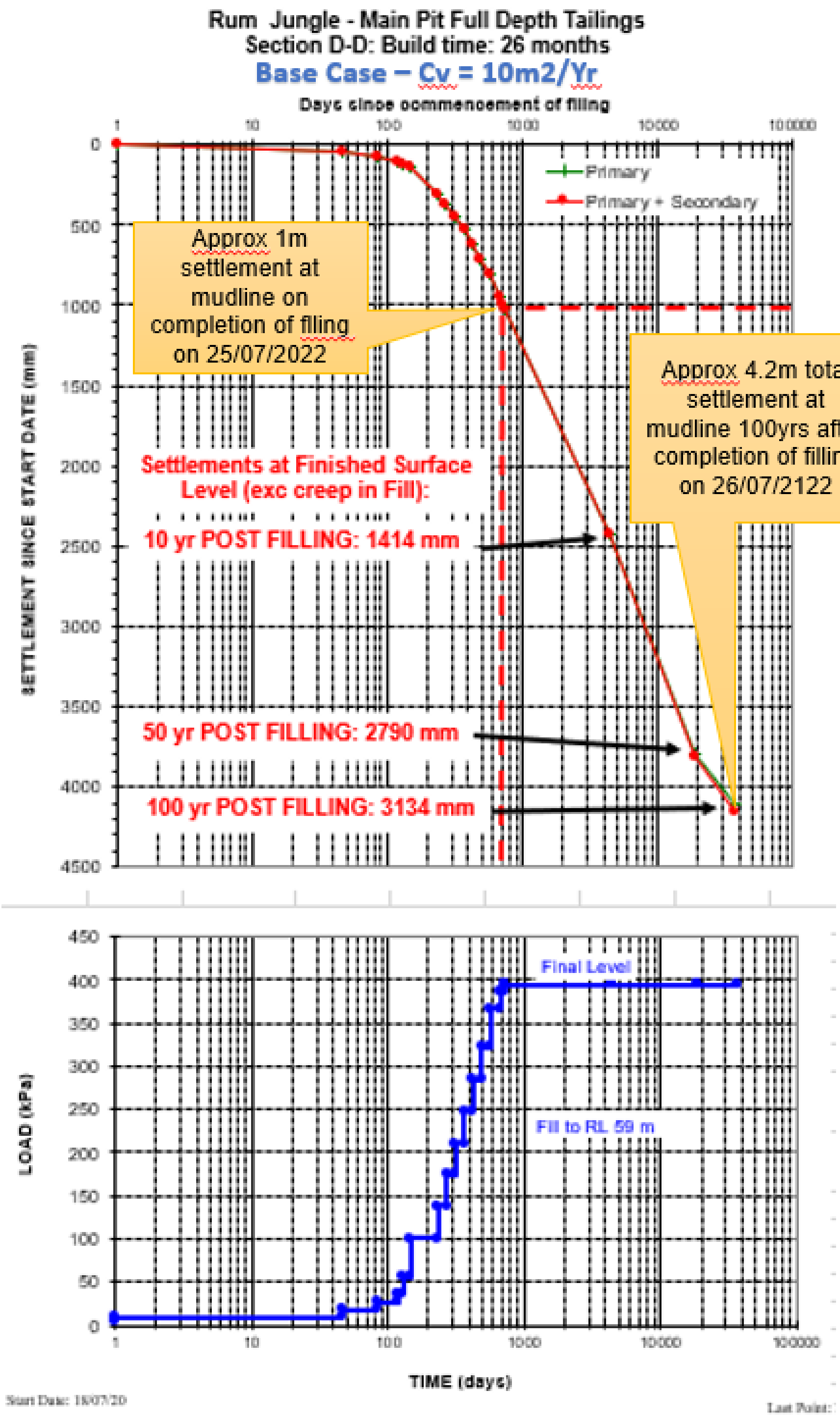
9.5.4 Settlement Outcomes

A Base Case Settlement analysis at the deepest (central) pit area indicates that about 1m of settlement would occur with tailings and within the backfill materials during backfilling, then about 3.5 m to 4 m of ongoing settlements over 100 years as shown in **Figure 51** and **Figure 54 and Appendix G**. Additional plots showing the modelled excess pore pressure in tailings and stress and settlement profiles within the tailings are also included in **Figure 52** and **Figure 53** respectively and **Appendix G**.

By comparison, settlements observed at Dysons Pit, reports estimated 6% of thickness of tailings settlement has occurred under backfill, with 0.8m in first 18months and approximately 2m in next 22 years. For the modelled main pit tailings thickness of 58m, this would translate to 0.06 x 61 = 3.48m in 22 years, which is very similar to the equivalent SLR estimate for Main Pit from **Figure 54** of 1.1m (during filling) plus 2.5m at 22 years post-filling = 3.6 m. As also mentioned previously, the SLR prediction is also within 20% of the ATC Williams prediction.

The rate of settlement is potentially significantly affected by the assumed coefficient of consolidation which can vary significantly spatially and with time as consolidation occurs and the permeability of the tailings changes. For this reason, two sensitivity cases have been undertaken to explore the potential effects of a lower and higher C_v values. The base cases uses 10 m^2/year, with sensitivity cases of 5 and 20 m^2/year.

Figure 51 Settlement of Tailings



Above settlements are at mud-line and excludes post-construction creep settlement within fill

Figure 52 Excess Pore Pressure in Tailings

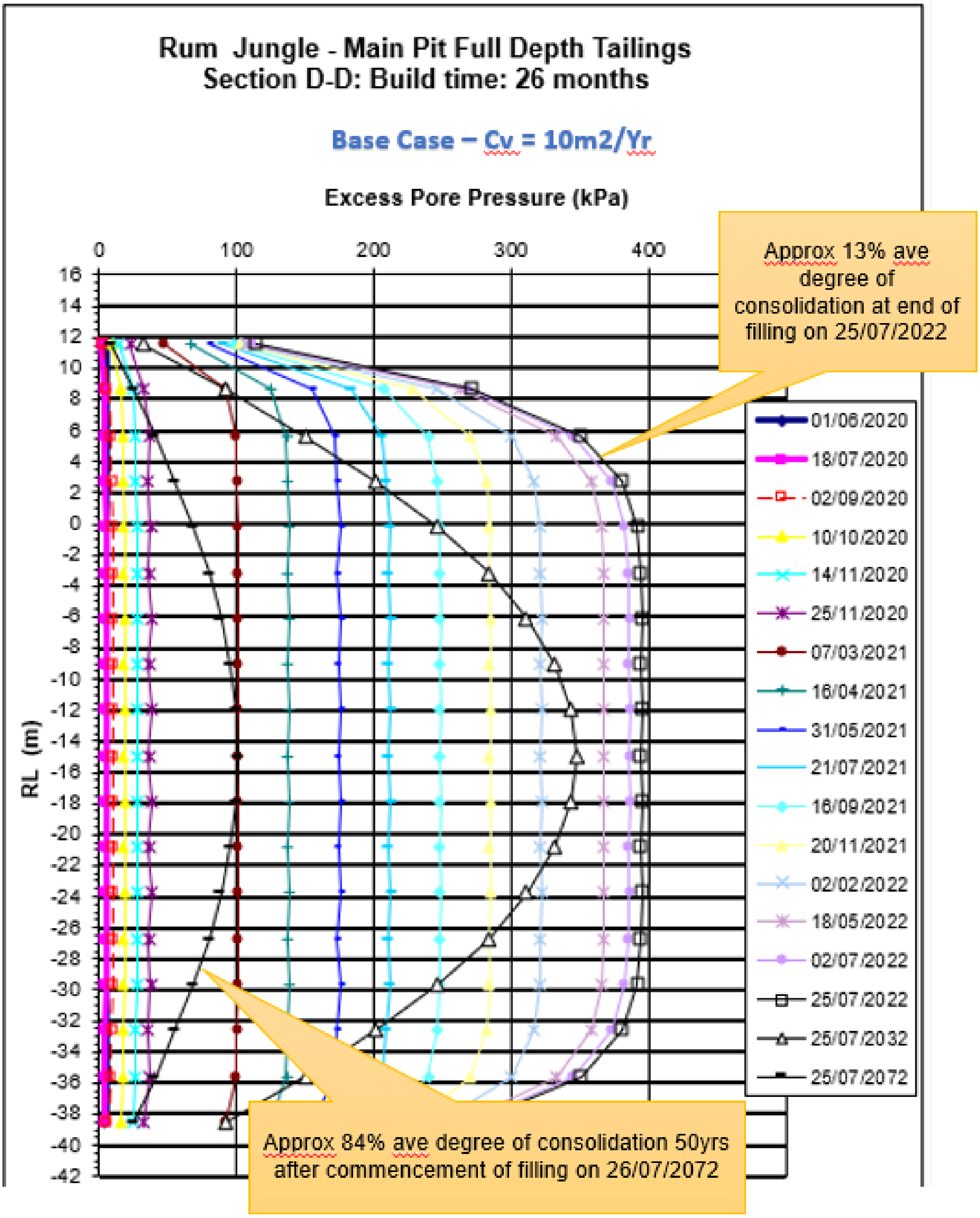
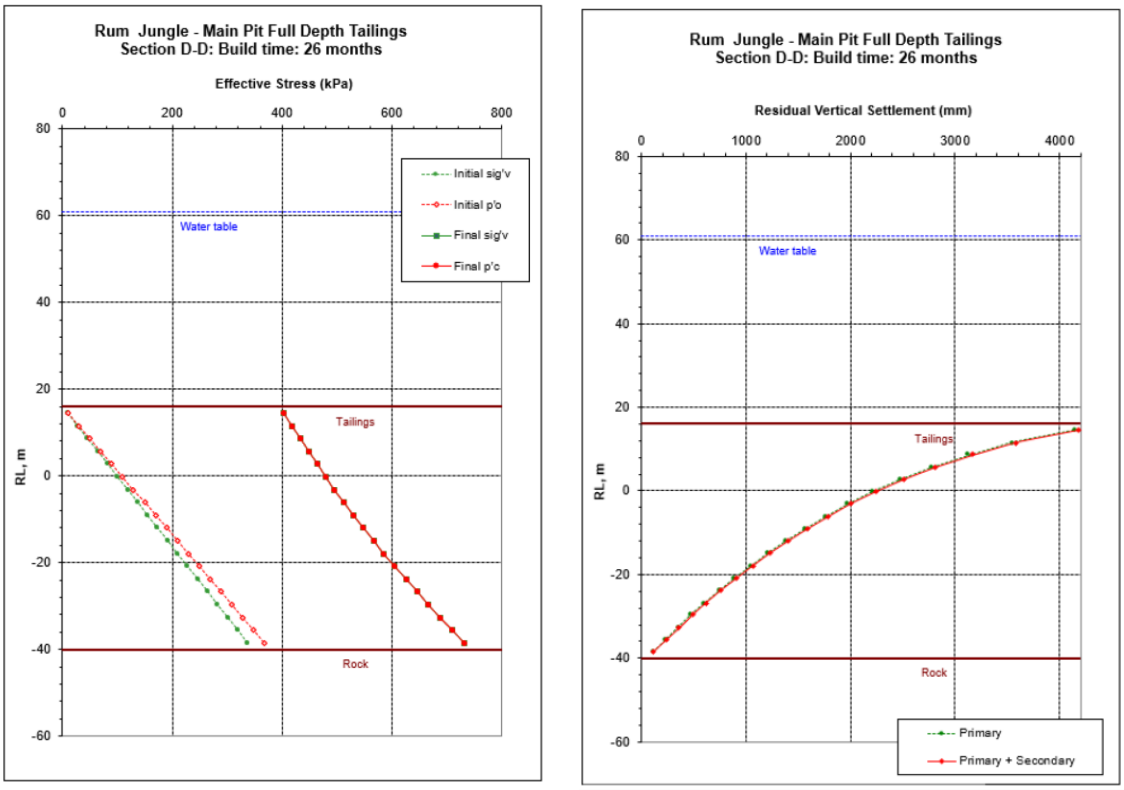
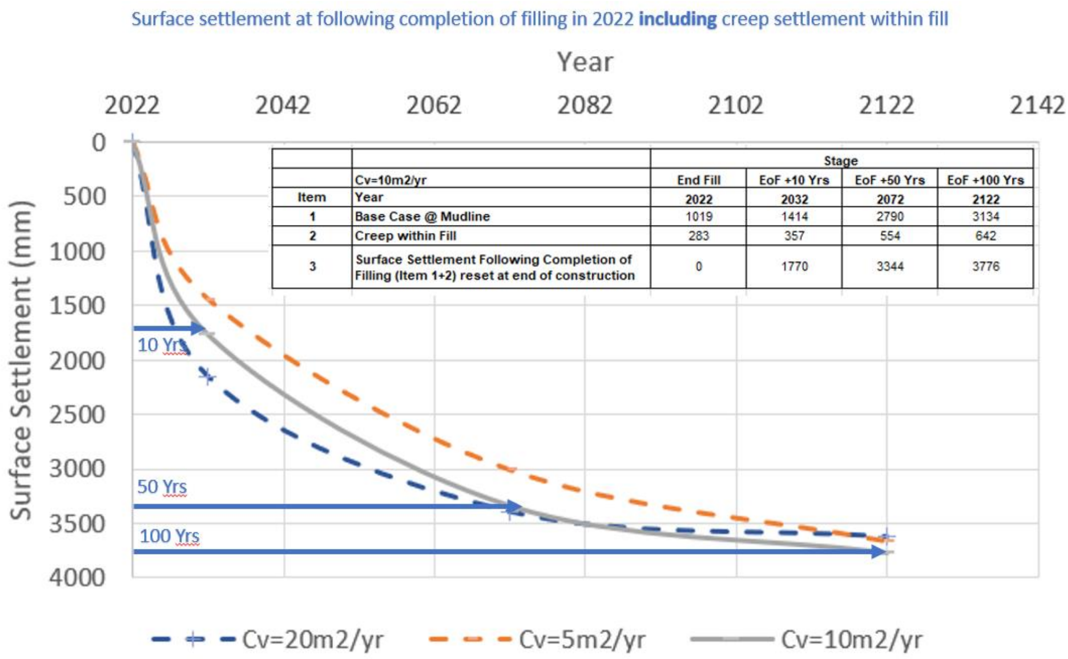


Figure 53 Stress and Settlement in Tailings Layer



Settlement summary showing Cv sensitivity analysis.

Figure 54 Settlement Summary



9.6 Stability Analysis

9.6.1 Backfilling Methodology

The following summary (**Table 32**) is based on the proposed backfilling strategy described in detail in the preceding sections.

Table 32 Backfilling Methodology Summary

Backfill Material / Layer	Sub – Layer Thicknesses (m)	Total Layer Thickness (m)	Maximum Elevation (m RL)	Method of Placement	Purpose
Bedding Layer: gravelly SAND (< 1 mm particle size	0.5	1.0	19.0	Produce a water/sand slurry to be piped to the base of the Main Pit gently and carefully via spreader pontoon with diffuser	Improve stability and bearing capacity conditions of the uppermost tailings without significant displacement of the tailings or chemocline
Intermediate bedding Layer: gravelly SAND and sandy GRAVEL (> 1 mm particle size)	1.0	3.0	22.0	Barges dispersing sand overboard from surface water level by diffuser (above) or in a controlled manner (water resistance on particles reduces deposition rates)	Improve stability and bearing capacity conditions of the uppermost tailings without significant disturbance of the tailings or chemocline. Form protective layer between waste rock and tailings
Waste Rock: Layers – sandy GRAVEL	2.0	2.0	22.0	Barges cascading screened waste rock overboard from surface water level in a controlled manner (water resistance on particles reduces deposition rates). Grading control may be required for initial layers to manage dynamic impact effects of initial layers (sub-150mm screening)	Long term safe, storage of PAF waste rock.
	4.0 – 5.0	34.0	56.0		
Capping Layer: non-dispersive, inert, gravelly SAND	2.0	2.0	Approx. 58.0	Barges cascading suitable clean fill overboard from surface water level in a controlled manner (water resistance on particles reduces deposition rates.	Forms a protective barrier over the PAF waste rock, preventing exposure at the surface and subsequent negative impacts on the environment. Formation of a shallow pit lake as final landform.

9.6.2 Stability Analysis

Base on the above backfilling approach, the main geotechnical components within the Main Pit backfilling process which require consideration in regards to stability include the pit side slopes, the basal subgrade of the pit (including tailing and beach soil deposits) and thee backfill material itself (granular bedding layers and waste rock). A screening exercise, presented in **Table 33**, was undertaken to determine the critical components to be analysed.

Table 33 Risk Screening of Backfilling Stability

Component	Risk Assessment
Main Pit Side Slopes above tailings (including various rock and soil units)	Stability of the Main Pit walls and side slopes has been undertaken in Section 7.11 and considers the current slopes as unconfined and submerged as being marginally stable. Once in a confined state, as filling and buttressing occurs, the side slopes will have greater stability. Therefore, this component does not require any further assessment.
Pit Floor Surface – Tailings and Slimes unit	Tailings surface at the lake pit floor shows a very slight gradient (typically <2°) towards the south and southeast of the Main Pit. It has been consolidating under its own weight since it was deposited in the Main Pit with the uppermost tailings material considered to be extremely weak, normally consolidated silty CLAY to clayey SILT becoming over-consolidated at depth. Therefore, both compressibility and strength of the tailings is considered a risk. Once backfilling begins, temporary slopes will develop within the backfill material which may cause instabilities. Since this issue is largely dependent upon the geometry of the backfill material, this aspect of the stability review is considered in the Backfill Soil mass component, below.
Pit Side Slopes – Soil and Beach Deposits unit	A mixture of granular and cohesive materials is found along segments of the northern and eastern edges of the Main Pit. These beach deposits form slopes up to 10° nearest the Main Pit side slopes towards the centre of the Main Pit and are generally considered to comprise granular soils interbedded with compacted tailings material. Given their proposed material properties these peripheral beach soils are not considered representative of the worst-case scenario (i.e. soft tailings). Furthermore, they are currently stable in the unconfined condition and so it is given that their stability in the confined condition will be greater due to the buttressing effect of the backfill material and so does not require further assessment. An underlying assumption is that no significant end dumping is undertaken from the pit rim below a backfill elevation of about RL 54 m AHD.

Component	Risk Assessment
Pit Floor Surface – Engineered Geosynthetic Liner Option	A geosynthetic liner is being considered in the backfill methodology to improve bearing capacity of the basal surface on to which the backfill material will be deposited. It will not serve as a barrier to potential contaminants released by the tailings but may limit disturbance. The liner would be placed sub-aqueously on top of the soft tailings at a very shallow gradient. Potential instability of the liner may occur as over lying backfill material develops temporary slopes. Since this issue is largely dependent upon the geometry of the backfill material, this aspect of the stability review is considered in the Backfill Soil mass component, below. The purpose of the liner is to provide additional strength to the uppermost tailings so that the risk of failures through the tailings is reduced. The loading of the backfill material will progress slowly and provide strength gain in the tailings and a stronger founding surface for further backfilling. As such the liners long term integrity is not considered significant and the short term loading is not considered adequate to cause significant strain. Therefore, the integrity of the geosynthetic liner requires no further assessment.
Backfill Soil mass (including all backfill material types)	A sequence of backfilling soils will be deposited sub-aqueously into the pit, either onto the tailings, beach deposits, geosynthetic liner or other backfill material. During this process temporary slopes will develop int eh backfill materials which may cause instabilities within some or all of the components within the Main Pit. Three granular material types are proposed within the backfill and limited testing has shown that they have the potential to develop slope at 1V:2H or greater. However, it is assumed that they will form gentler slopes as they fall through the pit lake water column. The deposition of backfill material requires assessment. As backfilling progresses the impact of placing more backfill waste rock upon the developing thick column of backfill material decreases and the risk of instability reduced. Given the geometry of the Main Pit, minor instabilities within the backfill material (including shallow slips and slumps) have little impact. The critical conditions are when large scale instabilities, with failure slip surfaces which cut deep into the soft tailings as this is likely to impact the groundwater quality in the Main Pit. The stability assessment will focus on this failure mode.

The stability analysis considers only the short-term condition of the backfilled pit. Over time further consolidation will occur and differential settlement has the potential to cause undulations in the surface of the backfill material. However, the consequences of these are unlikely to have a significant impact on any assets within the Main Pit and as such are not considered any further.

9.6.3 Stability Input Assumptions

The following data are required as input for the analyses undertaken for the stability assessment of the soil and tailings backfilling:

- Material unit weight
- Undrained and effective shear strength of tailings and soils within the system
- Properties of structural elements, if used, to represent geosynthetics along the base of the Main Pit

The design material parameters are based on the review of information and data available to SLR.

Strength gain is also considered based on the change of pore water pressure calculated within the tailings during the backfilling process. A conservative increase in undrained shear strength of the tailings has been applied to the upper 6.0 m of tailings after the required loading and time has been applied, in line with the proposed backfilling methodology. The degree of pore pressure dissipation and therefore tailings strength gain is presented in **Figure 52** and shows that due to the relatively slow-draining tailings, a relatively shallow zone of improved strength occurs in the timeframe of filling.

A summary of the design material properties is given in **Table 34** below.

Table 34 Material Design Parameters for Soil and Tailings Stability Analysis

Unit Name	Saturated Bulk Unit Weight (kN/m³)	Undrained Shear Strength, Cu (kPa)	Effective Shear Strength	
			Cohesion c' (kPa)	Friction Angle, ϕ (°)
Tailings and Slimes (normally consolidated)	16	0.5 + 1.5(z)*	-	-
Tailings and Slimes after strength gain** (tending towards over consolidated)	16	6.25 + 0.6786(z)	-	-
Tailings and Slimes' *** (normally consolidated)	16	11.0 + 1.5(z)	-	-
Soil & Beach Granular Deposit	19	-	0	30
Fine Sand Bedding Layers	18	-	0	30
Intermediate / Coarse Sand Bedding Layer	19	-	0	34
Waste Rock	19	-	0	32
Geosynthetic Fabric^ Interface	-	-	0	22

* Where z is the depth below the surface of tailings **Tailings and Slimes showing strength gain from loading during backfilling between 0.0 m and 6.0 m depth ***Tailings and slimes underlying 6.0 m of over consolidated strength gain after consolidation from loading ^Geosynthetic fabric with a tensile capacity of 100 kN/m

9.6.4 Modelling Approach and Software

Stability of Main Pit backfilling has been considered at each critical layer stage of the backfilling lifecycle. Where appropriate, the methodology and the software used also enables interrogation of output parameters such as limit equilibrium factor of safety and calculation of tension within geotextile components.

The analytical methods used in this stability assessment include limit equilibrium stability analyses for the derivation of factors of safety for the unconfined side slope subgrade, side slope liner and the temporary waste slopes. The limit equilibrium analyses have been undertaken using the package SLOPE/W Version 6.21 (Geo-Slope International). The Bishop[1] slip-circle and Morgenstern-Price[2] non-circular methods of analysis have been used.

9.6.5 Selection of Factor of Safety

The Factor of Safety (FOS) is the numerical expression of the degree of confidence that exists, for a given set of conditions, against a particular failure mechanism occurring. It is commonly expressed as the ratio of the load or action which would cause failure against the actual load or actions likely to be applied during service. This is readily determined by limit equilibrium slope stability analyses.

Prior to determining appropriate factors of safety for the various components of the model, it is necessary to identify key ‘receptors’ and evaluate the consequences in the event of a failure (relating to both stability and integrity). The main receptor which requires consideration is groundwater and surface water contamination. Further contaminant release could impact the larger catchment area with associated environment risks and also the impact on the construction works with implications to the timeframe and costs.

As described previously, a suitable minimum temporary factor of safety (FOS) considered to be appropriate for this assessment is 1.2.

9.6.6 Stability Design Methodology

The analyses undertaken follow a sequence of deposited backfill material types with various thickness and temporary slope gradients. Considering the mode of deposition, there is a limit to how much control over developing layers and slope gradients form. In order to minimise the risk of instability arising from over steep temporary slopes or exceeding bearing capacity a number of criteria need to be established in regards to the following:

- minimum offsets between crest and toe of consecutive layers;
- maximum height difference between the lowest elevation in the Main Pit and the highest elevation of the backfill material; and
- maximum overall gradient from toe to crest of all backfill layers.

During backfilling different types of material and layer thicknesses will be deposited, plus the rates of backfilling will also vary with time. As such, different sets of criteria will apply to specific stages of backfilling.

[1] Bishop, A.W., (1965), ‘The use of the slip-circle in the stability analysis of slopes’ Geotechnique
[2] Morgenstern, N.R and Price, V.E. (1965), ‘The analysis of stability of general slip surfaces’ Geotechnique.

Based on the settlement analysis, consolidation induced by the deposition of backfill material will improve strength in the upper, weakest tailings. A conservative strength gain in the top 6.0 m of tailings will be applied after 7.0 m of backfill material has been placed to assess the impact on tailings stability. The impact of placement of geosynthetic liner on top of the tailings will also be analysed.

9.6.7 Stability Outcomes

Two scenarios have been modelled:

- Base Case backfilling onto tailings directly (Models 1A – 1H); and
- Optional Case of backfilling onto geosynthetic liner covering the surface of the tailings (Models 2A – 2H).

A summary of the results and the associated backfilling controls required to maintain stability are given in **Table 35** with Slope/W Models presented in **Appendix H**. The results are discussed below.

Table 35 Summary of Stability Modelling Results

Model Description	Total Slope Height (m)	Total Slope Gradient	Minimum Offset Between Sub Layers. Crest to Toe	FoS without reinforcement – Model 1	FoS with reinforcement – Model 2
A - 0.5 m Fine Bedding Layer	0.5	1V:3H	n/a	1.497	6.494
B -Two 0.5 m Fine Bedding Layers	1.0	1V:3H	0.0	**0.930**	4.530
	1.0	1V:10H	10.0	1.418	5.657
C - Two 0.5 m -1mm Bedding Layers One 1.0 m Unscreened Bedding Layer	2.0	1V:10H	10.0	1.742	4.303
D - One 1.0 m Unscreened Bedding Layer One 1.0 m -1mm Bedding Layer	2.0	1V:10H	20.0	1.340	3.384
E - One 1.0 m -1mm Bedding Layer Half 2.0 m Waste Rock Layer	2.0	1V:10H	20.0	2.057	3.249
F - Two 2.0 m Waste Rock Layers	4.0	1V:4H	7.0	1.198	1.679
G – Two 4.0 m Waste Rock Layers	8.0	1V:3H	7.0	1.226	1.351
H - Two 4.0 m Waste Rock Layers with strength gain in the upper 6.0 m of tailings	8.0	1V:3H	7.0	1.343	1.450

Bedding Layers

Bedding layer material, comprising screened gravelly sand 1mm (minus) from the granular borrow area will be placed first followed by the intermediate layer, comprising the coarse gravelly sand to sandy gravel material (unscreened or 1mm plus) also recovered from the granular borrow. Given the sub-aqueous deposition methodology, a slope gradient of 1V:3H was adopted for this material, however based on the available shear box testing of the granular borrow material, the potential for far steeper slopes to develop requires consideration.

Stability modelling of the bedding layers determined that the weakest of the tailings requires very careful and spread distribution of material initially and confirmed the first layer should not exceed 0.5 m thickness. Within the lower bedding layers the failure mechanism is shallow rotational slips near the slope crest which is indicative of mud waving, and produces FOS of 1.497 (Model 1A) and 1.418 (Model 1B).

Figure 55 shows Model 1C, with 10.0 m lateral offsets between subsequent layers of the lower bedding layer and intermediate bedding layer, and Model 1D, with a single 20.0 m offset between layers of intermediate bedding material. These offsets maintain a total overall slope gradient of 1V:10H.

A conservative gradient of 1V:10H was selected as it produced FOS > 1.2 for all stages during the bedding layer deposition and minimises the risk of instabilities from over steep temporary slopes. Additionally, to further reduce the potential of over steep slopes a limit of 2.0 m maximum height difference between the lowest point in the Main Pit and highest point of backfilled material was adopted

The following placement criteria are necessary to achieve acceptable FOS values for all initial sand bedding layers and waste rock lens infill layers up to RL 22m AHD:

- A maximum overall gradient of 1V:10H within any 20 m x 20 m area (using a 2m survey grid); and
- A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed sand bedding and waste rock lens backfill of 2.0 m.

Bulk Waste Rock Layers

Waste rock backfill material, generally comprising gravelly sand to sandy gravel, will be placed on top of the sand bedding layer and central waste rock lens above RL 22m AHD.

The first stage of waste rock backfilling, modelling as two 2.0 m thick layers, recorded a FOS of 1.198 for a deep rotational base failure. The maximum slope of the individual 2.0 m high sub-layer slopes was limited to 1V:2H and the overall geometry of the waste rock was a gradient of 1V:4H with a 7.0 m offset between the crest and toe of the layers. This is presented in **Figure 56** as Model 1F.

Following completion of this 4.0 m thick waste rock (top elevation of 26 m RL) a more competent founding layer (comprising bedding and waste rock material) is in place which reduces the risk of disturbing the tailings and chemocline. The next two layers of waste rock proposed in the backfilling methodology are two 4.0 m thick layers, again modelled with a maximum layer slope of 1V:2H. A FOS of 1.226 was achieved modelling a slope of total height of 8.0 m and gradient of 1V:3H, with a 7.0 m offset between the layers crest and toe. The failure mechanism is a large rotational failure of nearly 40 m radius, cutting deep into the backfill and tailings.

A strength gain in the upper 6.0 m of tailings was applied at this stage of backfilling, analysed as Model 1H. A FOS of 1.343 was recorded showing a slight increase from Model 1G, showing the same deep rotational failure mode. Model 1G and 1H are presented in **Figure 57.**

To reduce the risk of over steep slopes developing, the following criteria must be complied with to achieve acceptable FOS values in the waste rock fill stages:

- No waste rock filling prior to completion of all sand bedding layers to within a minimum distance of 100m from the nearest location of proposed waste rock filling;
- Below RL 22 m AHD:
  - A maximum overall gradient of 1V:10H within any 20 m x 20 m area (using a 2m survey grid); and
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed sand bedding and waste rock lens backfill of 2.0 m.
- Between RL 22m and 26m AHD:
  - A maximum overall gradient within any 20 m x 20 m area (using a 2m survey grid) of 1V:4H; and
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 4.0 m.
- Between RL 26m and 34m AHD:
  - A maximum overall gradient within any 20 m x 20 m area (using a 2m survey grid) of 1V:3H; and
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 8.0 m.
- Above RL 34m AHD:
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 8.0 m.

Reinforcing Geosynthetic Liner

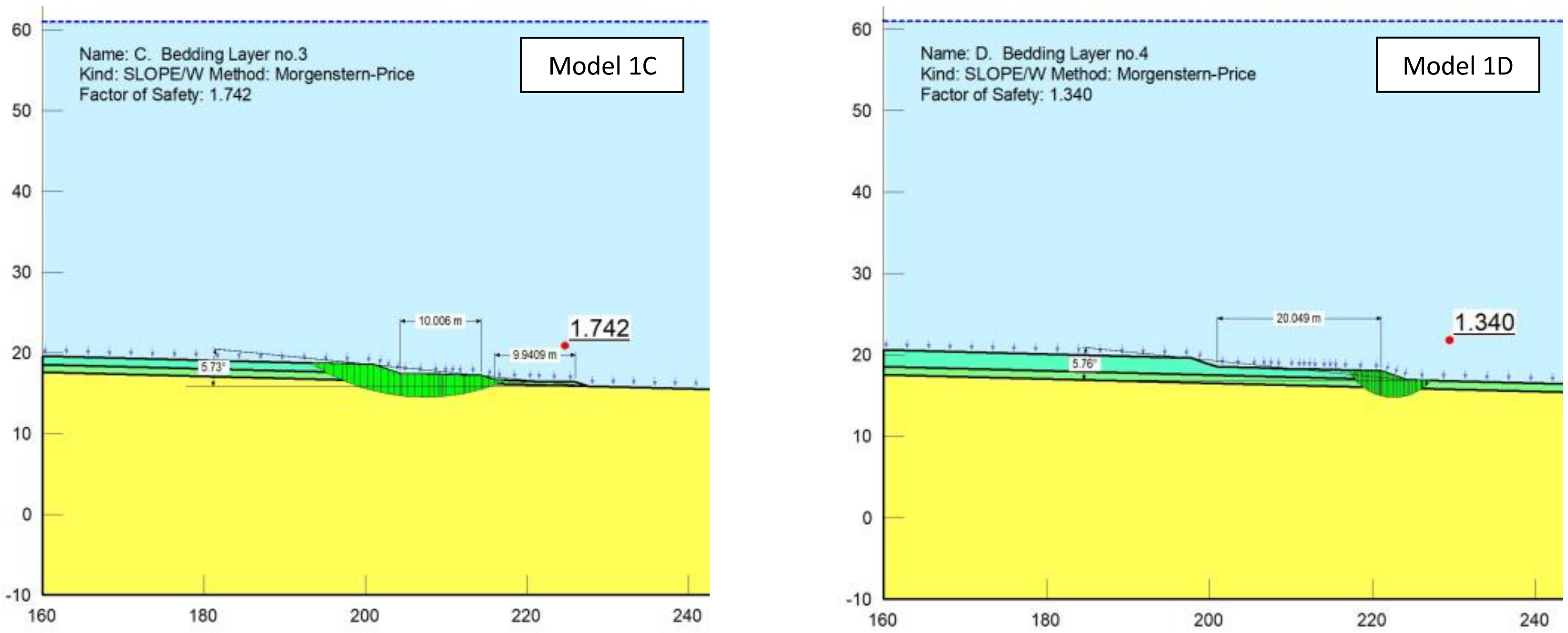
All the Base Case models discussed above were also analysed with a geotextile liner (Models 2A to 2H) to compare the FOS values and potential reduction in risk. Failure mode considered is that of a deep slip surface which shears through the liner and tailings.

All FOS values recorded were greater than 1.2. The trend, as presented in **Table 35**, above, shows a general decrease in FOS as the thickness of backfill material increases. FOS values range from 6.494 in Model 2A to 1.351 in Model 2E. Comparison of Model 1 and Model 2 FOS values shows an increase by a factor of between 4.8 and 2.3 during deposition of the bedding layers.

Model 2F, showing the analysis results for the placement of two 2.0 m thick waste rock layers, returned a FOS of 1.679, approximately 0.5 greater than the FOS in Model 1F. Once 7.0 m of backfill material has been placed the analyses show only slight increases in FOS. From these analyses the benefits of the presence of a geotextile liner are significant during the initial placement of material, becoming insignificant once sufficient backfill material has been deposited, approximately 7.0 m thick.

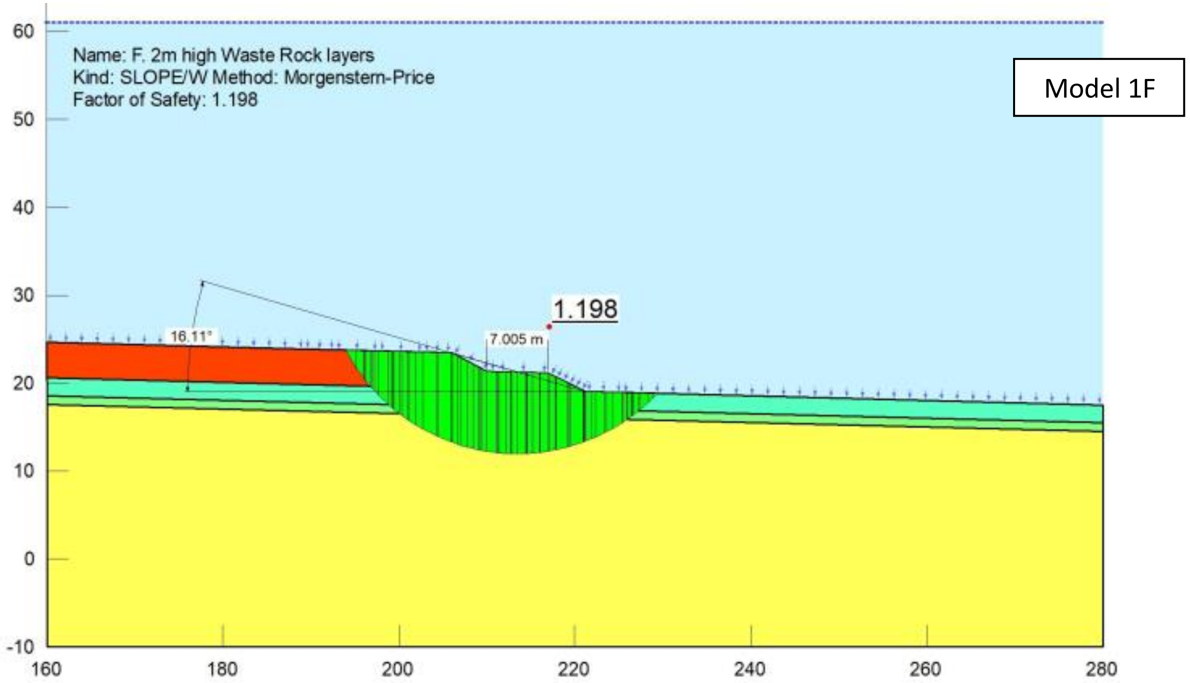
Figure 58 shows Model 2A, the maximum FOS recorded, and Model 2F, exhibiting the last significant increase in FOS due to the presence of the geotextile liner.

Figure 55 Models 1C and 1D



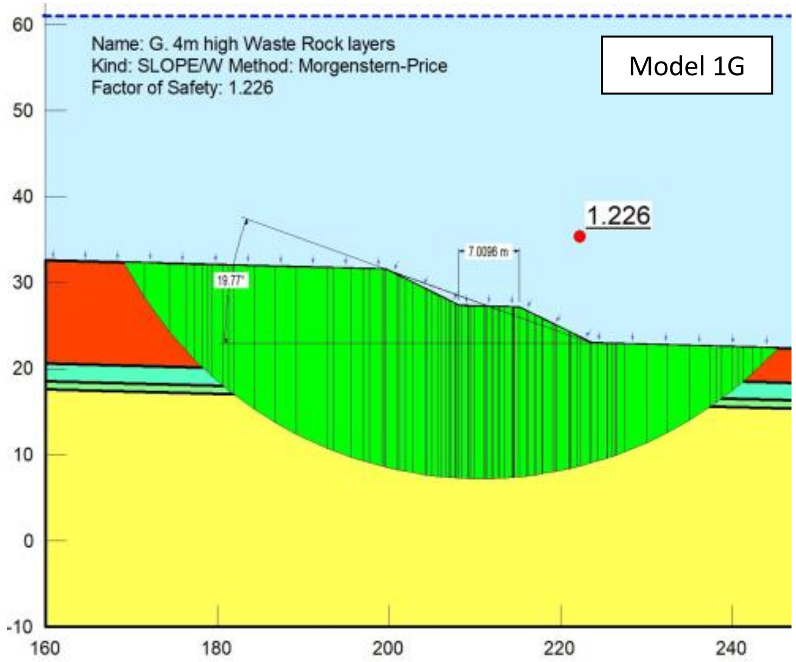
Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)
	Bedding Layer	Mohr-Coulomb	18				0	30
	Intermediate Bedding Layer	Mohr-Coulomb	19				0	34
	Tailings and Slimes	S=f(depth)	16	0.5	1.5	0		

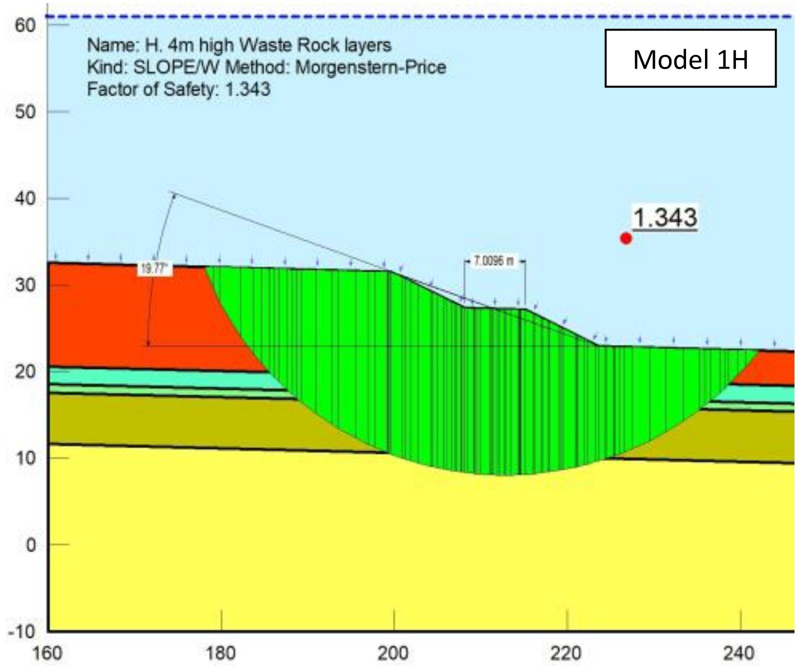
Figure 56 Models 1F



Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Bedding Layer	Mohr-Coulomb	18				0	30	0	1
	Intermediate Bedding Layer	Mohr-Coulomb	19				0	34	0	1
	Tailings and Slimes	S=f(depth)	16	0.5	1.5	0				1
	Waste Rock	Mohr-Coulomb	19				0	32	0	1

Figure 57 Models 1G and 1H without geotextile

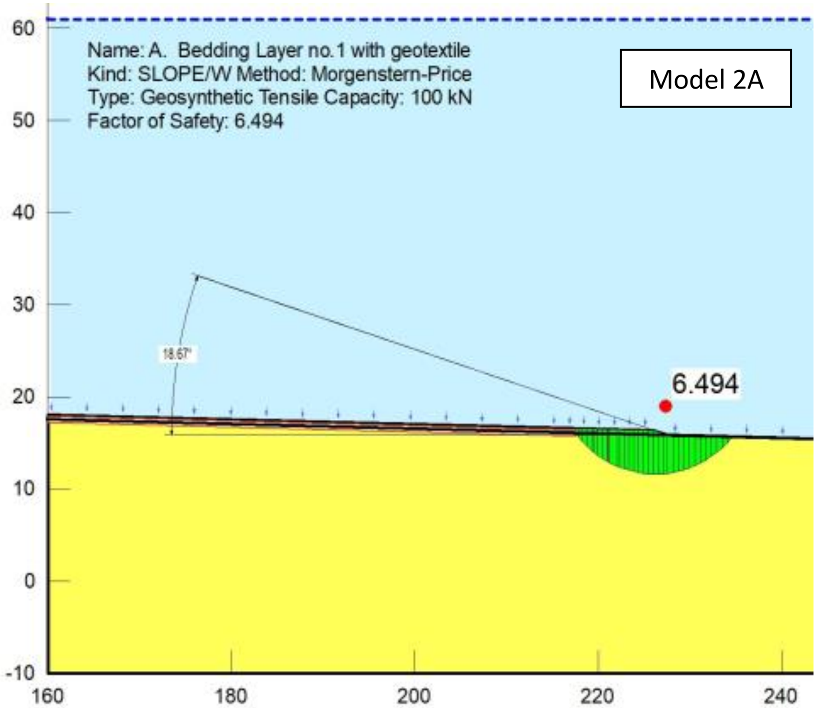


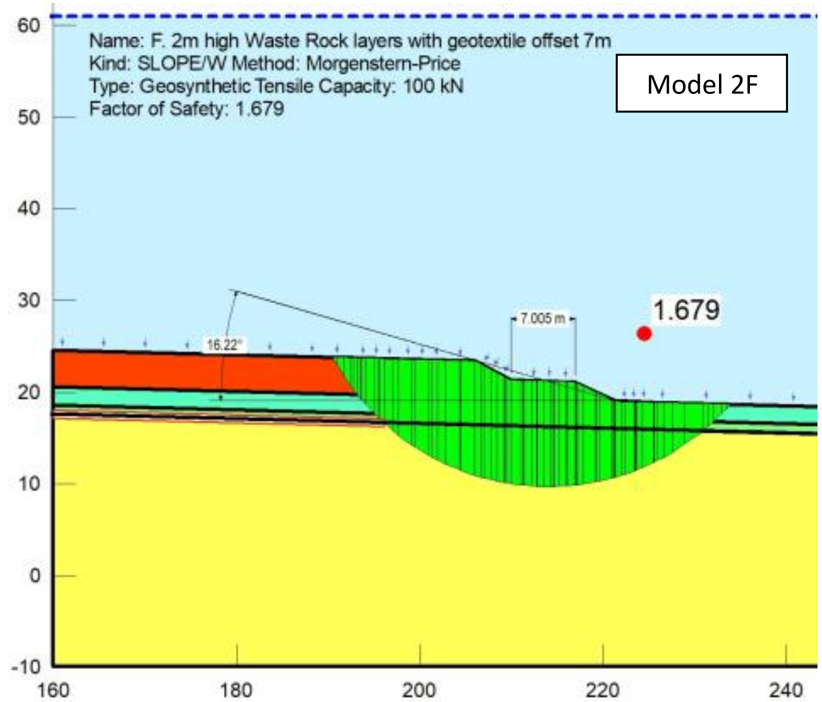


Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Bedding Layer	Mohr-Coulomb	18				0	30	0	1
	Intermediate Bedding Layer	Mohr-Coulomb	19				0	34	0	1
	Tailings and Slimes	S=f(depth)	16	0.5	1.5	0				1
	Waste Rock	Mohr-Coulomb	19				0	32	0	1

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m³)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Bedding Layer	Mohr-Coulomb	18				0	30	0	1
	Intermediate Bedding Layer	Mohr-Coulomb	19				0	34	0	1
	Tailings and Slimes'	S=f(depth)	16	11	1.5	0				1
	Tailings and Slimes Strength Gain	S=f(depth)	16	6.25	0.6786	0				1
	Waste Rock	Mohr-Coulomb	19				0	32	0	1

Figure 58 Models 1G and 1H with geotextile





Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Bedding Layer	Mohr-Coulomb	18				0	30	0	1
	Tailings and Slimes	S=f(depth)	16	0.5	1.5	0				1

Color	Name	Model	Unit Weight (kN/m³)	C-Top of Layer (kPa)	C-Rate of Change ((kN/m²)/m)	C-Maximum (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Bedding Layer	Mohr-Coulomb	18				0	30	0	1
	Intermediate Bedding Layer	Mohr-Coulomb	19				0	34	0	1
	Tailings and Slimes	S=f(depth)	16	0.5	1.5	0				1
	Waste Rock	Mohr-Coulomb	19				0	32	0	1

9.6.8 Summary of Stability Analysis

The critical stages of backfilling are the first deposits which are in contact with or in close proximity to the underlying soft Tailings and Slimes and, as such, are more likely to disturb the tailings (stages A to F). The careful placement of the bedding layers is significant in improving the bearing capacity of the underlying Tailings to enable deposition of thicker Waste Rock layers (stage G onwards).

To maintain a minimum acceptable FOS >1.2 whilst placing the first two bedding layers, strict controls of the absolute fill height and gradient are required as described in **Section 9.6.7**.

After the initial 3.0 m of sand bedding layers have been placed, the first internal waste rock 'lens' infill layers must be placed carefully and with strict placement controls to avoid instability as described in **Section 9.6.7**. In this way, successive layers will provide founding platform for further waste rock deposition providing benefits of confinement and strength gain over time in the upper most soft tailings. The strength gain increases the FOS values slightly, however analyses in Model 2 indicate the difference in FOS caused by the absence of a strength gain in not considered significant.

Each sand bedding and waste rock layer must be completed over a wide area before beginning the next fill layer as specified above (minimum 100m x 100m area surrounding the point of next filling). FOS values generally increase as the thickness of waste rock backfill increases and the likeliness of disturbing the tailings reduces. The final layers can be placed without geometric controls (but must consider Water Treatment Plant rates for displaced water), once the whole footprint has achieved a minimum level of RL +54 m RL.

Addition of a geosynthetic reinforcement layer (Model 3) between the top surface of the Tailings and Slimes and the base of the first Bedding layer will increase FOS values and reduce the risk of failures occurring through the tailings. As the thickness of waste rock increases the effect of the reinforcement reduced to have little or no impact on the FOS values recorded. Therefore, the use of geosynthetic reinforcement could be justified to reduce the risk of deep failure within the tailings however, adequate FOS values have also been demonstrated without reinforcement and placement rates are generally controlled by water treatment.

Similarly, any granular deposits within the soft tailings (Models 4 and 5) improved the ground conditions for depositing backfill material and generally increases the FOS values recorded. However, the accuracy of locating these intermittent discrete deposits is limited and so the worst case of a full column of Tailings should be assumed

Table 36 Summary of Backfilling Criteria

Backfill Material / Layer	Sub-Layer Thickness (m)	Number of Sub-Layers	Total Layer Thickness (m)	Maximum Elevation (m AHD)	Minimum Offset Between Sub-Layers: Crest to Toe	Maximum Total Slope Height (m)	Maximum Total Slope Gradient
Bedding Layer – gravelly SAND (< 1 mm particle size	0.5	2	1.0	20.0	10.0	2.0	1V:10H

Backfill Material / Layer	Sub-Layer Thickness (m)	Number of Sub-Layers	Total Layer Thickness (m)	Maximum Elevation (m AHD)	Minimum Offset Between Sub-Layers: Crest to Toe	Maximum Total Slope Height (m)	Maximum Total Slope Gradient
Intermediate Bedding Layer – gravelly SAND and sandy GRAVEL (> 1 mm particle size)	1.0	2	2.0	22.0	10.0*		
					20.0**		
Waste Rock Layers – gravelly SAND to sandy GRAVEL	2.0	2	4.0	26.0	7.0	4.0	1V:4H
	4.0	2	8.0	34.0		8.0	1V:3H
	3.0 – 5.0	8 - 10	Approx. 28.0	59.0	n/a	8.0	1V:3H
Capping Layer – gravelly SAND	2.0	1	2.0	Approx. 61.0	n/a	2.0	Natural angle of repose

* offset between 0.5 m Bedding Layer and 1.0 m Intermediate Bedding Layer

** offset between 1.0 m Intermediate Bedding Layers

9.6.9 Bearing Capacity Failure

Separate to the slope stability but also important to assessing if the tailings will be disturbed during backfilling is the tailings bearing capacity when subjected to isolated loading, as backfilling occurs.

A worst-case scenario has been considered where a full barge load of backfill material with a volume of 4,000 m^3 is deposited at once, forming a conical, discrete pile to form at the base of the Main Pit. To give the smallest basal surface area it is assumed to form a peak of 8.0 m height. This produces an approximate 22.0 m radius and 800 m^2 basal surface area. An average load for this form was applied based on half the peak height (4.0m high cylinder) to calculate FOS from the allowable working load at the base of the Main Pit during backfilling. The average load applied was 3,800 kN.

The same stages A to D used in the stability modelling are used here. A summary of the bearing capacity FOS values recorded alongside the stability Models 1A to 1D are presented in **Table 37** below.

Table 37 Summary of Bearing Capacity and Stability Model FOS

Model	Allowable Working Load (kN)	Bearing Capacity FoS	Stability Analysis FoS
1A - tailings	6000	1.6	1.497
1B – 0.5 m backfill	5500	1.5	1.418
1C – 1.0 m backfill	7800	2.0	1.742
1D – 2.0 m backfill	16800	4.4	1.340

The results show bearing capacity calculation FOS greater than the stability modelling FOS with a general increase as backfill material thickness increases. Therefore, static bearing capacity is not the critical failure mode and the stability governs. It is possible that the dynamic load of falling fill would exceed the available bearing capacity, although calculations have not been undertaken to confirm this.

Regardless of the bearing capacity assessment above, the limiting geometric filling controls described in **Section 9.6.7** must be adhered to.

9.7 Summary of Backfill Strategy and Design Approach

9.7.1 Geotechnical Considerations

Pit floor investigations have shown that the in-situ Main Pit Tailings and washed in sediments are predominately fine-grained silt/clay fraction materials with moderate to high water content (43-53%) and low shear strength (near-normally consolidated). These materials are relatively compressible and unless appropriate controls are implemented, can be easily displaced or resuspended during backfill placement.

Following placement, bridging layer stability and settlement due to consolidation can become two additional geotechnical issues that may be important for bridging layer effectiveness. The shear strength of the tailings and sediment will influence its resistance to localized bearing capacity or sliding failures, which could cause localized mixing of placed granular materials and contaminated materials. Bridging layer stability immediately after placement is critical, before any excess pore water pressure due to the weight of the backfill has dissipated.

9.7.2 Volume Adjustments

It will be necessary to account for the following adjustments when assessing fill demand and placement volumes:

- Bulking Factors: Net bulking (bank-to-truck-to-placed underwater), assume:
  - 1.5 for rock (Waste Rock (WR) mined from intact rock mass),
  - 1.2 (Waste Rock (WR) excavated from existing Waste Rock fill areas)
  - 1.1 for granular borrow.
- For intermediate haulage calculations requiring estimation of truck and barge movements, add approx. 10-20%, to the above net bulking values for cartage volume.
- Immediate compression during filling under self-weight would reduce the net bulking by up to 2% to 4%, say 3% (separate to creep).

Table 38 Backfilling Volume Adjustments

	Volume Case	Sand Bedding (SB) (1mm minus, unscreened, 1mm plus)	Waste Rock (WR)	Comment
Material lost into surveyed surface of tailings below RL 19 contour	Lower bound	0 mm of first layer (i.e. 0% 1st layer)	N/A	Affected by interaction with chemocline – to be confirmed through field trials
	Base Case	250 mm of first layer (i.e. 50% 1st layer)	N/A	
	Upper bound	500 mm of first layer (i.e. 100% 1st layer)	N/A	
Immediate fill compression loss during placement (under self-weight)	Lower bound	0% SB Layer Thickness	2% WR Layer Thickness	Assume +/- variability is normally distributed
	Base Case	1% SB Layer Thickness	3% WR Layer Thickness	
	Upper bound	2% SB Layer Thickness	4% WR Layer Thickness	
Compensation for creep settlement within fill (See Section 9.5)	Lower bound	N/A*	0.5 % WR Layer Thickness	Assume +/- variability is normally distributed
	Base Case	N/A*	0.8 % WR Layer Thickness	
	Upper bound	N/A*	1.2 % WR Layer Thickness	
Handling Losses	Lower bound	N/A*	0.1 % WR Layer Thickness	Estimated due to screening, crushing, loss of fines, unsuitable
	Base Case	N/A*	0.3 % WR Layer Thickness	
	Upper bound	N/A*	0.5 % WR Layer Thickness	
Compensation for settlement of tailings (See Section 9.5)	Lower bound	N/A	1.4% Tailings Layer Thickness	Adjustment based on 1m settlement at completion of filling in centre of pit (56m of tailings). To be adjusted pro-rata according to depth of tailings/soil thickness (i.e. nil settlement loss at edge of pit)
	Base Case	N/A	1.8% Tailings Layer Thickness	
	Upper Bound	N/A	2.3% Tailings Layer Thickness	

* Where z is the depth below the surface of tailings **Tailings and Slimes showing strength gain from loading during backfilling between 0.0 m and 6.0 m depth ***Tailings and slimes underlying 6.0 m of over consolidated strength gain after consolidation from loading ^Geosynthetic fabric with a tensile capacity of 100 kN/m

Further to the volumes provided above, particularly for the lower layers sand bedding and waste rock lens, the nature of project specification dictates that the design reduced levels and thicknesses are "minimums" in order to achieve required geotechnical and geochemical stabilities. To this end, it is anticipated that some 'over filling' volume will be incurred during construction due to the limited control allowed in subaqueous placement. It is noted a key objective is to maximise the waste rock placed within the Main Pit, overfilling above design levels should be minimised with practicable, however there should be no comprise to achieving the specified fill levels, especially for the initial bedding and lens layers.

9.7.3 Settlement Analysis

A Base Case Settlement analysis at the deepest (central) pit area indicates that about 1m of settlement would occur with tailings and within the backfill materials during backfilling, then about 3.5 m to 4 m of ongoing settlements over 100 years as shown in **Figure 51** and **Figure 54**. Additional plots showing the modelled excess pore pressure in tailings and stress and settlement profiles within the tailings are also included in **Figure 52** and **Figure 53** respectively.

By comparison, settlements observed at Dysons Pit, reports estimated 6% of thickness of tailings settlement has occurred under backfill, with 0.8m in first 18months and approximately 2m in next 22 years. For the modelled main pit tailings thickness of 58m, this would translate to 0.06 x 61 = 3.48m in 22 years, which is very similar to the equivalent SLR estimate for Main Pit from **Figure 54** of 1.1m (during filling) plus 2.5m at 22 years post-filling = 3.6 m. As also mentioned previously, the SLR prediction is also within 20% of the ATC Williams prediction.

The rate of settlement is potentially significantly affected by the assumed coefficient of consolidation which can vary significantly spatially and with time as consolidation occurs and the permeability of the tailings changes. For this reason, two sensitivity cases have been undertaken to explore the potential effects of a lower and higher C_v values. The base cases uses 10 m^2/year, with sensitivity cases of 5 and 20 m^2/year.

9.7.4 Stability Analysis

Table 39 Material Design Parameters for Soil and Tailings Stability Analysis

Unit Name	Saturated Bulk Unit Weight (kN/m^3)	Undrained Shear Strength, Cu (kPa)	Effective Shear Strength	
			Cohesion c' (kPa)	Friction Angle, ϕ (°)
Tailings and Slimes (normally consolidated)	16	0.5 + 1.5(z)*	-	-
Tailings and Slimes after strength gain** (tending towards over consolidated)	16	6.25 + 0.6786(z)	-	-
Tailings and Slimes' *** (normally consolidated)	16	11.0 + 1.5(z)	-	-
Soil & Beach Granular Deposit	19	-	0	30
Fine Sand Bedding Layers	18	-	0	30
Intermediate / Coarse Sand Bedding Layer	19	-	0	34

Unit Name	Saturated Bulk Unit Weight (kN/m³)	Undrained Shear Strength, Cu (kPa)	Effective Shear Strength	
			Cohesion c' (kPa)	Friction Angle, ϕ (°)
Waste Rock	19	-	0	32
Geosynthetic Fabric^ Interface	-	-	0	22

Placement of Sand Bedding Requirement

The following placement criteria are necessary to achieve acceptable FOS values for all initial sand bedding layers and waste rock lens infill layers up to RL 22m AHD:

- A maximum overall gradient of 1V:10H within any 20 m x 20 m area (using a 2m survey grid); and
- A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed sand bedding and waste rock lens backfill of 2.0 m.

Placement of Waste Rock Bedding Requirement

To reduce the risk of over steep slopes developing, the following criteria must be complied with to achieve acceptable FOS values in the waste rock fill stages:

- No waste rock filling prior to completion of all sand bedding layers to within a minimum distance of 100m from the nearest location of proposed waste rock filling;
- Between RL 22m and 26m AHD:
  - A maximum overall gradient within any 20 m x 20 m area (using a 2m survey grid) of 1V:4H; and
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 8.0 m.
- Between RL 26m and 24m AHD:
  - A maximum overall gradient within any 20 m x 20 m area (using a 2m survey grid) of 1V:3H; and
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 8.0 m.
- Above RL 34m AHD:
  - A maximum relative height difference within any 100 m x 100 m area (using a 2m survey grid) between the lowest and highest elevation of the placed waste rock backfill of 12 m.

10 Seismic Design

In accordance with common engineering practice, seismic design cases are considered only for permanent design cases (and not temporary design), as follows.

- Potential liquefaction of backfill materials after completion of filling.

Seismic design cases which are excluded from consideration in this report are as follows:

- Temporary stability of un-submerged pit rim slopes during backfilling under earthquake conditions;
- Temporary stability of submerged slopes during backfilling under earthquake conditions; and
- Potential liquefaction of backfill materials during backfilling.

10.1 Liquefaction Assessment

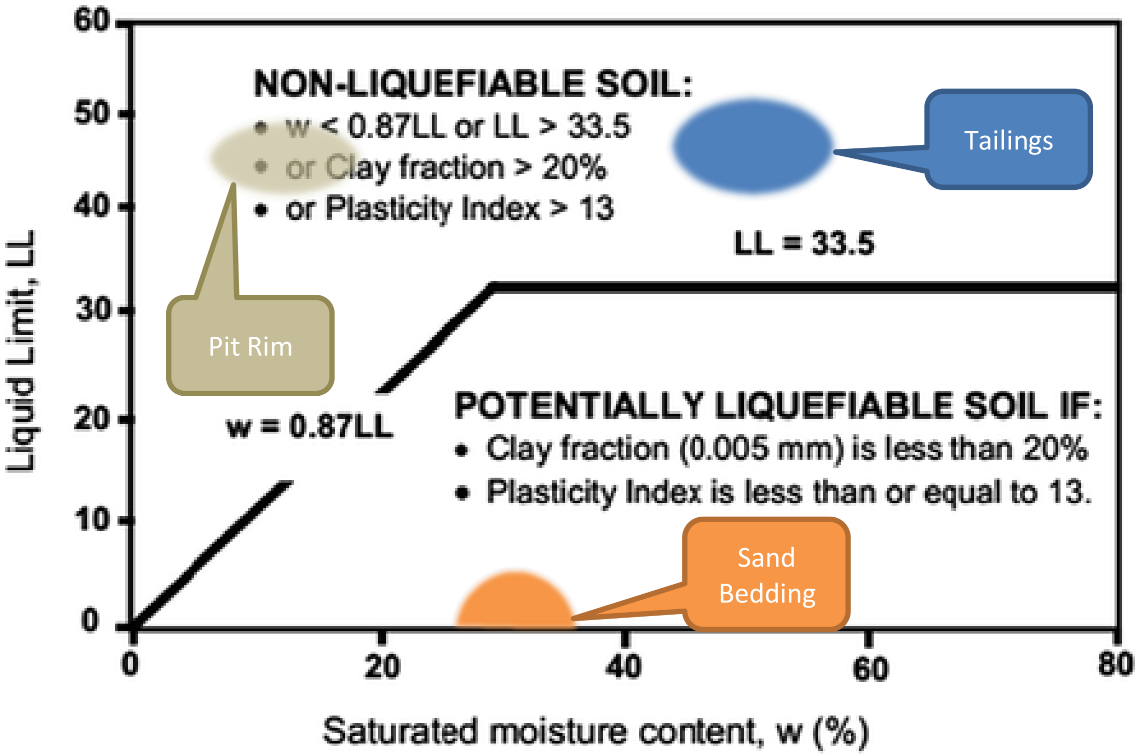
Cyclic behaviour of saturated soils during strong earthquakes is characterised by development of excess pore water pressures and consequent reduction in the effective stress. In the extreme case, the effective stress may drop to zero and the soil would liquefy.

Liquefaction is associated with significant loss of stiffness and strength and consequent large ground deformation. Adverse outcomes include excessive surface settlement, sand boils and lateral spreading. Materials that are typically susceptible to liquefaction during an earthquake are usually:

- Geologically-young;
- Granular materials with low fines content (however very loose silty sands can also be susceptible); and/or
- Materials in a relatively loose condition below the water table.

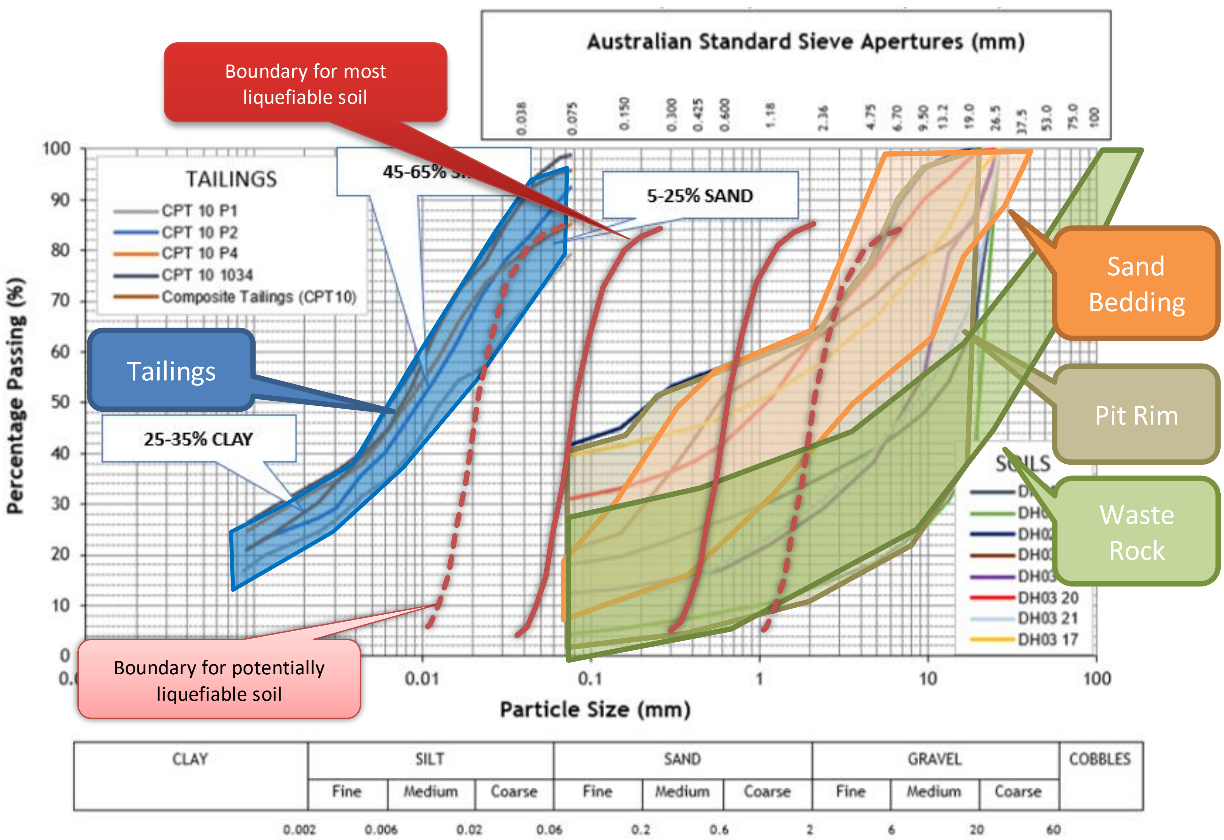
The liquefaction potential for existing Main Pit backfill, Tailings and Pit Rim soils has been assessed using empirical screening processes, considering published guidelines for grading and plasticity characteristics. **Figure 59** shows site soils in the context of Chinese liquefaction screening criteria adapted to ASTM Definitions of soil properties [54]. This screening test indicates that tailings and pit rim soils typically contain enough fines material so as to be non-liquefiable, however sand bedding material is potentially liquefiable.

Figure 59 Liquefaction Screening Based on Material Classification



An alternative screening method based on grading of soil materials published by Tsuchida [55] was also used to examine liquefaction potential as shown in **Figure 60** below.

Figure 60 Liquefaction screening based on grading (after Tsuchida 1970 [55])



The screening process shown above indicates the following:

- Tailings are generally sufficiently cohesive such that there is a very low risk of liquefaction.
- A minor fraction of pit rim soils are potentially liquefiable, however their in-situ condition is typically mixed and well graded (containing both material coarser and finer than liquefiable material) and sufficiently dense / stiff in their natural state such that there is a low liquefaction potential. It is also noted that once backfilled, the pit rim soils are fully buttressed and so slope stability or of these materials in their long term (post-construction) condition is not of concern.
- The majority of sand bedding material has a potentially liquefiable grading range and in particular, the basal sand fill (screened to be 1mm minus) is the most susceptible. However, given the depth that this material exists at on completion of backfilling, liquefaction is unlikely, as demonstrated in **Figure 61** below.
- A minor fraction of waste rock is potentially liquefiable, however it is typically well graded (containing mainly material coarser than liquefiable material). Given also it has a relatively high friction angle and is relatively deep (typically confined under high overburden stress), then

A liquefaction assessment has been undertaken to assess the potential loss of shear strength of foundation soils when they are subjected to cyclic loading under earthquake conditions.

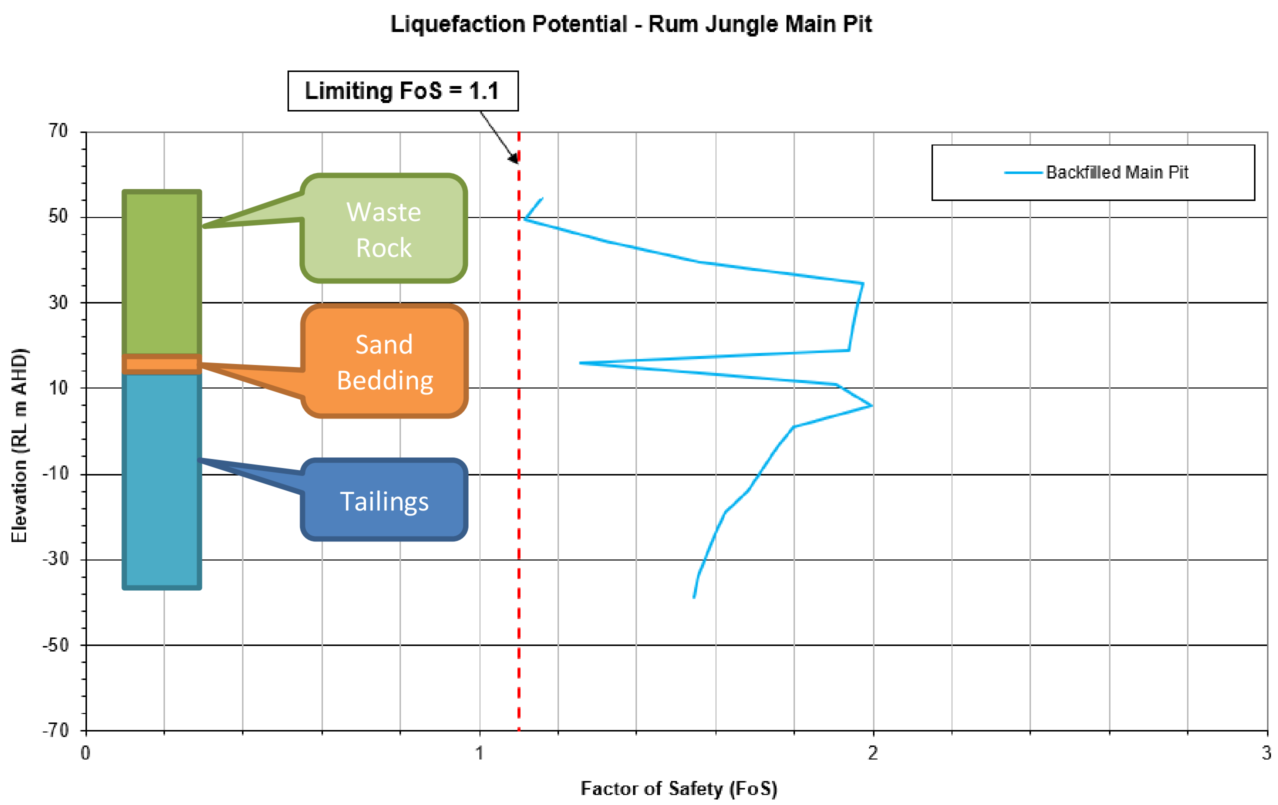
A liquefaction assessment has been undertaken using SPT data, based on the empirical method presented by Kramer [56] with due consideration of final filling levels and overburden stress. This simplified approach requires estimation of two variables (1) the Cyclic Stress Ratio (CSR), which represents the seismic demand on a soil layer caused by the adopted design earthquake, and (2) the Cyclic Resistance Ratio (CRR), which represents the capacity of the soil to resist liquefaction. The liquefaction triggering factor (FSLiq) - or factor of safety - is computed using:

$$FS_{Liq} = \frac{CRR}{CSR}$$

Theoretically, liquefaction will be triggered if $FS_{Liq} \leq 1.0$. The triggering factor FS_{Liq} is calculated (for liquefiable soils) throughout the depth of the deposit. The peak horizontal ground acceleration (a_h) adopted in the liquefaction assessment is a_h = 0.08g.

The results of the liquefaction assessment based on modelled SPT data has been undertaken using the method published by Kramer [56] for potentially liquefiable granular materials. Results for the simulated backfill profile are presented in as a plot of calculated factor of safety versus depth **Figure 61**. The results of this liquefaction assessment indicate that the calculated factor safety against liquefaction is generally greater than 1.1 which is conventionally classified as non-liquefiable and requiring no special precautionary measures.

Figure 61 Liquefaction FoS Assessment (after Kramer 1995 [56])



It is noted, whilst a peak ground acceleration of 0.08g has been used for assessment, there could be argument to consider longer return period intervals for peak ground acceleration. For larger peak acceleration, it may give rise to discrete layered liquefaction with potential for localised sand boils and small-scale settlements at the surface. Evaluated in consideration of possible project long-term consequences; with no planned infrastructure or services within the area, for a waste rock and tailings mass contained within a pit, the risk at the surface is deemed negligible.

11 Design Control and Validation

11.1 Additional Investigations and Field Trials

Several aspects of the adopted design approach will need to be further assessed prior to construction, including:

- Investigations to include:
  - Grab samples for chemocline to establish baseline conditions (thickness, extent, physical and geochemical properties);
  - Preconstruction verification of in-situ shear strength using in-situ vane testing and collection of undisturbed samples for laboratory triaxial and shear vane strength testing; and
  - Collection of undisturbed tailings samples for additional consolidation testing.
- Field Trials to include:
  - Field compaction trials to confirm bulking factors (bank-to-truck/barge and barge/truck-to-placed underwater)
- Modelling the sedimentation of the material and the impact of particle size, method and rate of dumping the material and the interaction with the chemocline.
- Verification and validation of initial filling stages for the weak sediments and tailings, to confirm the limiting geometric filling controls potential for slips or slumps and maximum safe load of granular material to control instability and mud-waving.

11.2 Monitoring

Monitoring of backfilling operations is essential for risk management and quality assurance. An instrumentation and monitoring (I&M) plan is to be developed as part of the design documentation for the backfilling operations. Key components to be addressed in the plan are described below.

11.2.1 Bathymetric Survey

Precision bathymetric surveys are a critical monitoring tool which enable determination of the location, size, and thickness of the contaminated material deposit and cap. For the main pit rehabilitation, a series of surveys should be taken immediately prior to placement of the cap, periodically during placement, and at the completion of placement. The differences in bathymetry as measured by the consecutive surveys yields the location and thickness of the deposits and onset of mud-waving or instability near the filling front.

Contractors should make bathymetric measurements on a daily basis initially to keep track of their progress and plan work for the following days.

Acoustic instruments such as depth sounders (bottom elevations accurate to +/- 200 mm under favourable conditions), side scan sonar (mapping of areal extent of sediment and bedforms), and sub-bottom profilers (measures internal mound and seafloor structure) are used for these physical measurements. Survey track spacing can be 15 to 75 m depending on the areal coverage of the cap.

Multi-beam depth sounding systems provide 100 percent coverage of the bottom. Their additional expense may be justified for some projects. Contract criteria for limiting sediment resuspension during capping placement may require monitoring by a combination of survey and sampling for suspended solids.

Water samples are to be collected around the placement operation and analysed for total suspended solids and selected Chemocline analytes and physical parameters (such as dissolved oxygen, total suspended solids, ammonia and total sulphides). The colour of particulates on filter paper can also indicate the type of suspended solids in the plume around the capping operation and identify whether they are fines washed off the sand during placement, or resuspended bottom sediments (Zeman and Patterson [57]). In addition, sediment may be deployed near the deposition areas to collect and measure resuspended bottom sediments.

The interpretation of bathymetric data needs to be coupled with an understanding of consolidation processes. Consolidation that occurs in the cap, contaminated sediment, and the original base material can result in substantial changes in bathymetry (Silva et al. [58], Poindexter-Rollings, [59]) that could mistakenly be considered as an indication of inadequate cap thickness. The ability to measure or predict consolidation can limit the utilization of bathymetric data for monitoring the total cap thickness.

Post-completion survey of the finished landform is necessary at regular intervals using mass concrete survey monoliths to confirm settlement behaviour prior to site hand-over.

11.2.2 Physical sampling

The thickness of granular cap components and the presence of sediment contaminants or chemocline at any level within backfill can also be determined from post filling grab samples (or boreholes if necessary). Regular sediment, chemocline and capping grab sampling during backfilling can assist with:

- Validating effects of segregation, fines migration
- Validating mixing / advection / diffusion / settlement of mudline
- Enabling controlled water sampling and testing of basal pit lake water. This should be undertaken for comparison with trigger/action/response criteria as part of the monitoring plan for selected analytes to manage and respond to contamination risk and potential chemocline disturbance.

11.2.3 Probing

Due to the depth of the pit lake floor it may not be possible to use settlement plates placed on the tailings surface to validate consolidation of the tailings surface, which can otherwise provide a means of measuring the absolute level of the cap/tailings interface and consolidation behaviour of the underlying sediments. An alternative system would be to undertake probing with carefully surveyed CPTs prior to placement of waste rock fill to verify the absolute level of the cap/tailings interface to validate settlement behaviour. To improve delineation of the cap/tailings interface, a localised sheet of geofabric may be carefully positioned over the future CPT test areas prior (weighted down in corners).

Undertaking dissipation testing during CPT testing can also confirm excess pore pressure and strength gain assumptions at critical staging. Optional installation of VWP's within grouted boreholes installed into tailings can also enable validation of pore pressure dissipation to assist with validation of settlement behaviour.

A suitable grid and frequency of construction stage CPT validation testing is to be developed and included in the design and construction documentation.

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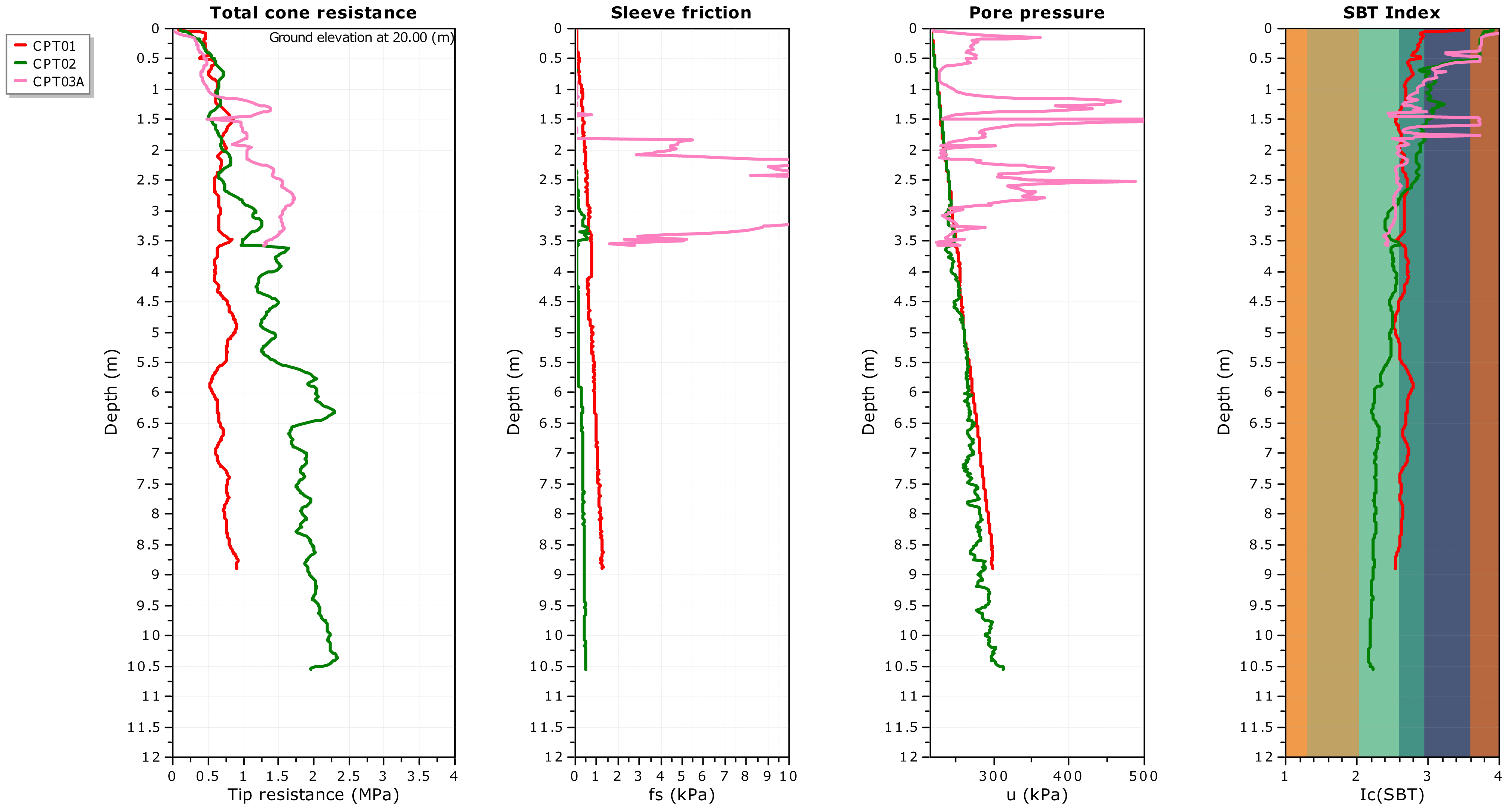
APPENDIX A

CPT Data and Interpretation Outputs

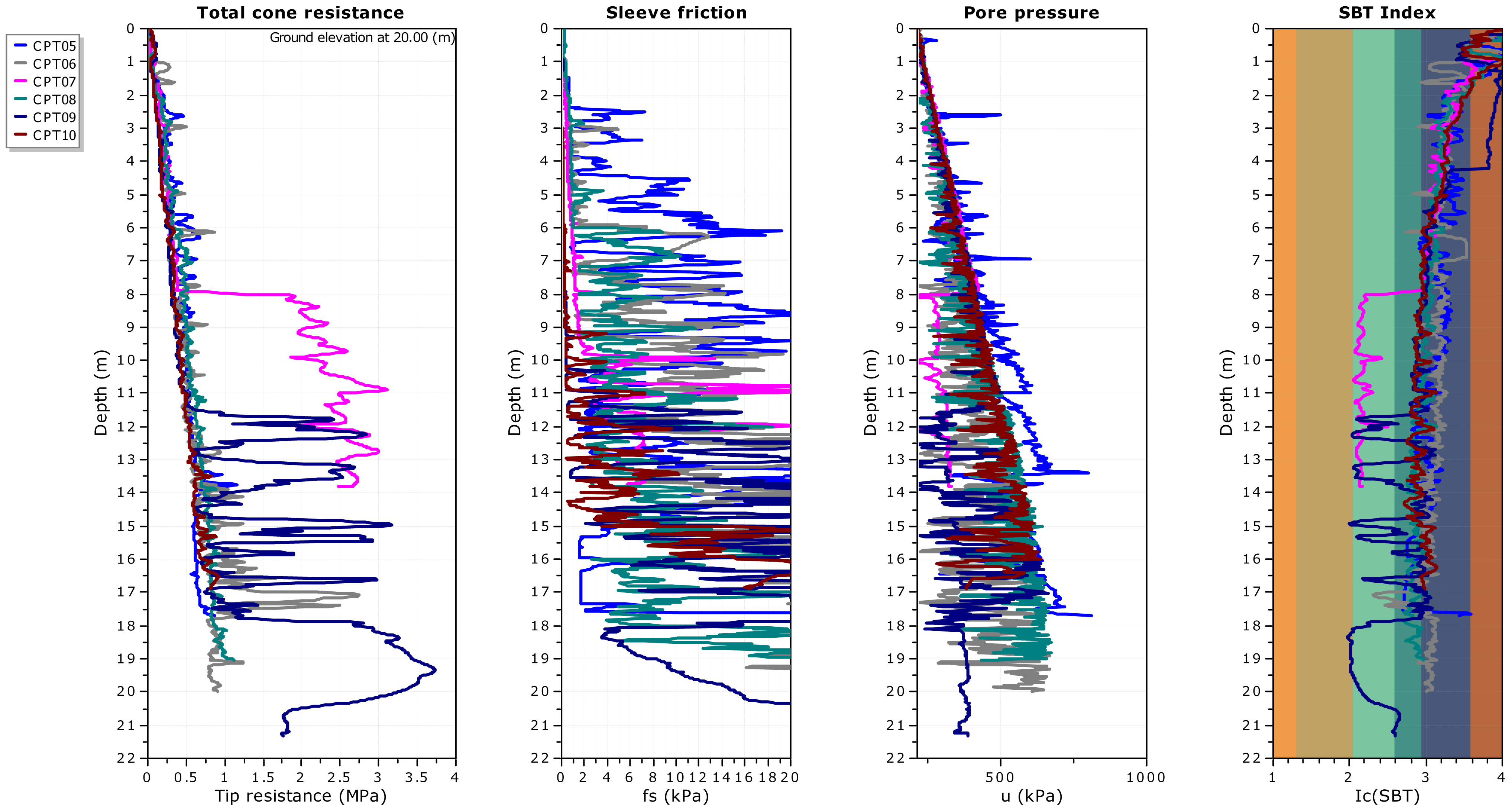
Project: Rum Jungle
Location: NT

Overlay basic interpretation plots



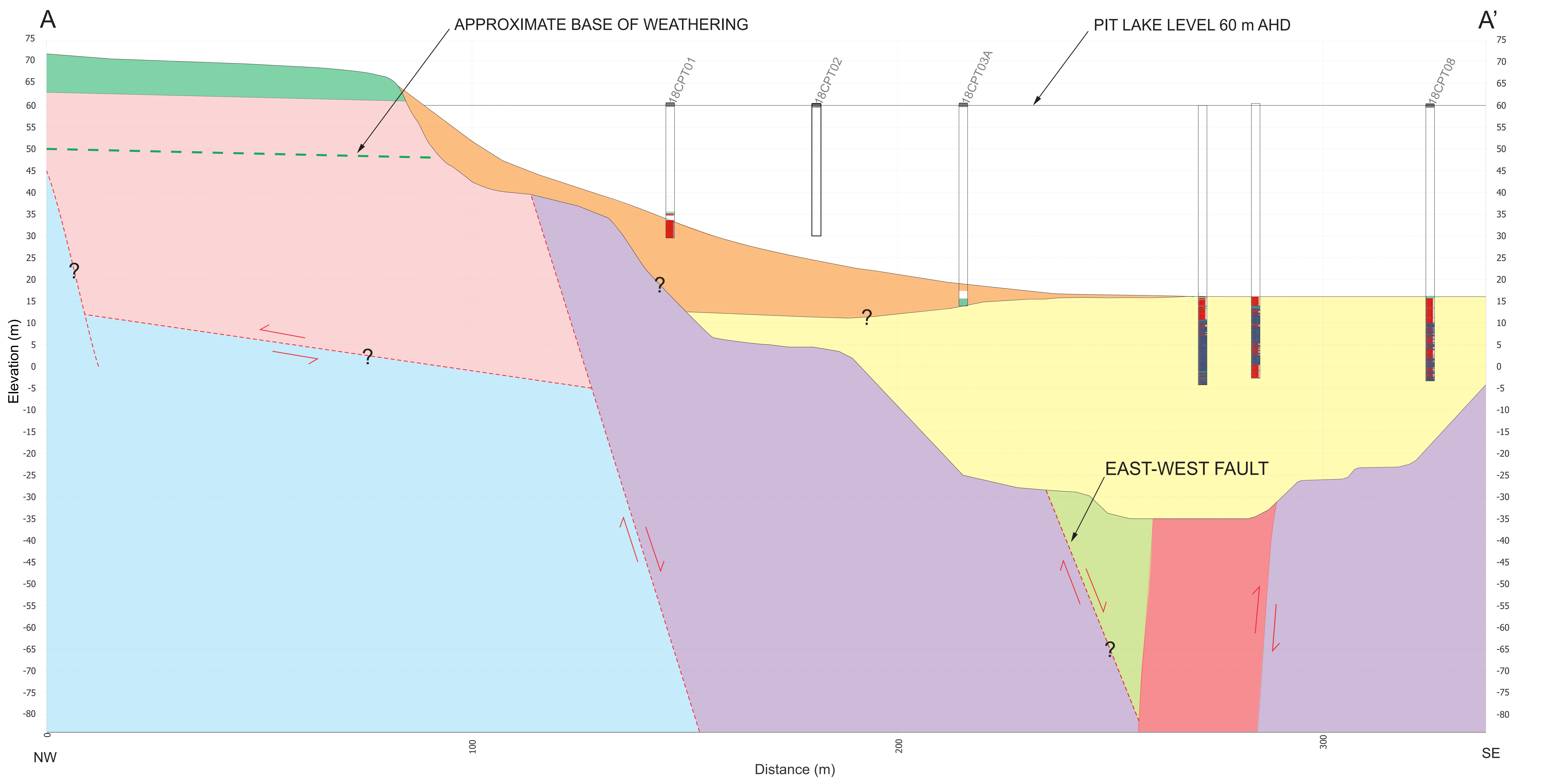
Project: Rum Jungle
Location: NT

Overlay basic interpretation plots



APPENDIX B

Geotechnical Main Pit Sections



COOMALIE DOLOSTONE
DOLOMITE, QUARTZITE

WHITES FORMATION
SHALE, MUDSTONE, LOW GRADE METAMORPHICS

GEOLSEC FORMATION
QUARTZITE BRECCIA

MAIN SHEAR ZONE
DEFORMED, SHEARED BRECCIATED WHITES FORMATION

MINERALISED ZONE

SAPROLITE / LATERITE
SANDY GRAVELLY CLAY

UNCONTROLLED FILL
TAILINGS

UNCONTROLLED FILL
SOIL & BEACH DEPOSITS

FAULTED CONTACT
INFERRED

CPT - SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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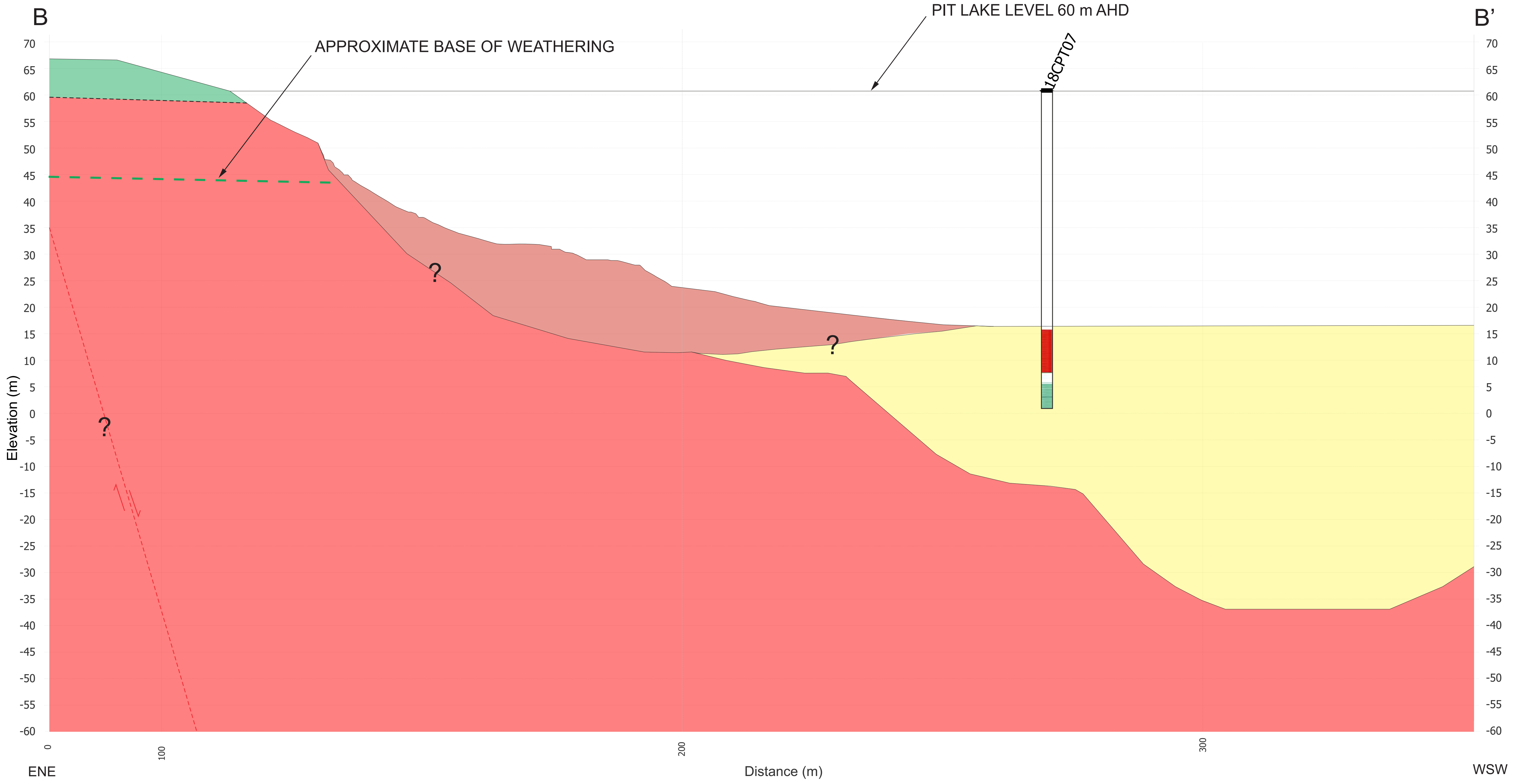

MAIN SHEAR ZONE
DEFORMED, SHEARED BRECCIATED WHITES FORMATION

FAILED SHEAR ZONE MATERIAL

SAPROLITE / LATERITE
SANDY GRAVELLY CLAY

FAULTED CONTACT
INFERRED

UNCONTROLLED FILL
TAILINGS

CPT - SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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Date:	06/02/2020		

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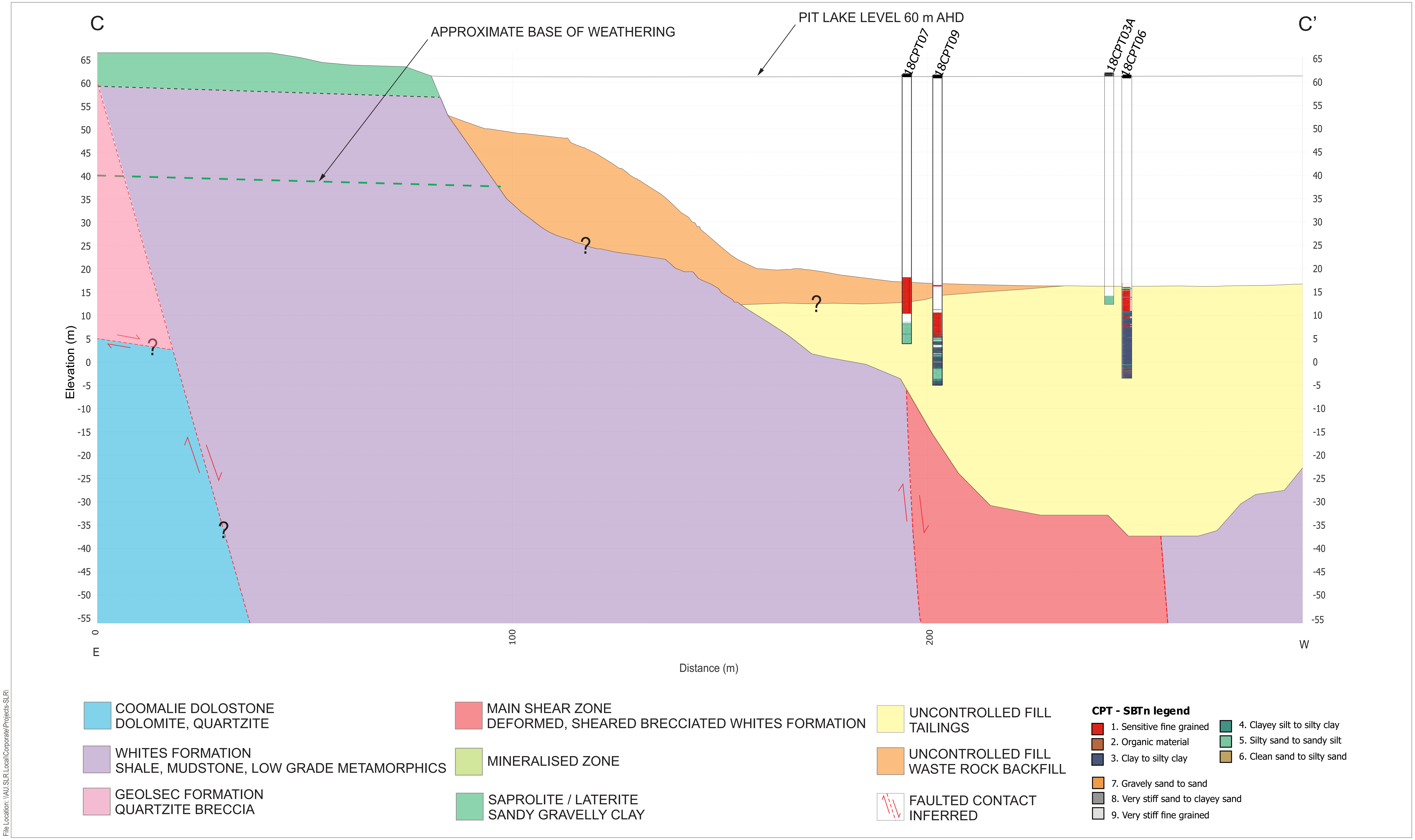

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Status:	Final	Version:	1.0
Date:	06/02/2020		

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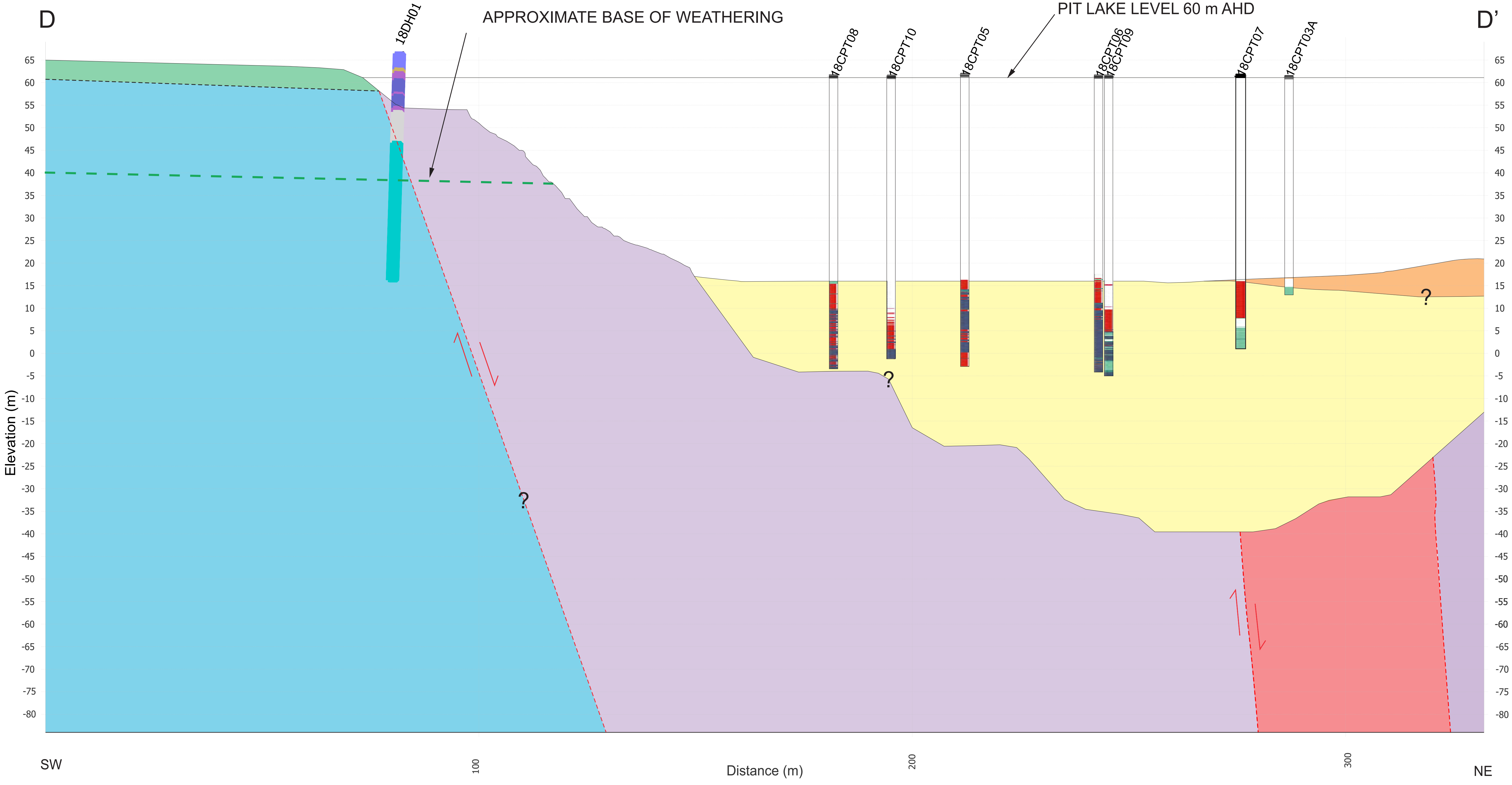

COOMALIE DOLOSTONE
DOLOMITE, QUARTZITE

WHITES FORMATION
SHALE, MUDSTONE, LOW GRADE METAMORPHICS

MAIN SHEAR ZONE
DEFORMED, SHEARED BRECCIATED WHITES FORMATION

SAPROLITE / LATERITE
SANDY GRAVELLY CLAY

UNCONTROLLED FILL
TAILINGS

UNCONTROLLED FILL
SOIL & BEACH DEPOSITS

FAULTED CONTACT
INFERRED

CPT - SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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Status:	Final	Version:	1.0
Date:	06/02/2020		

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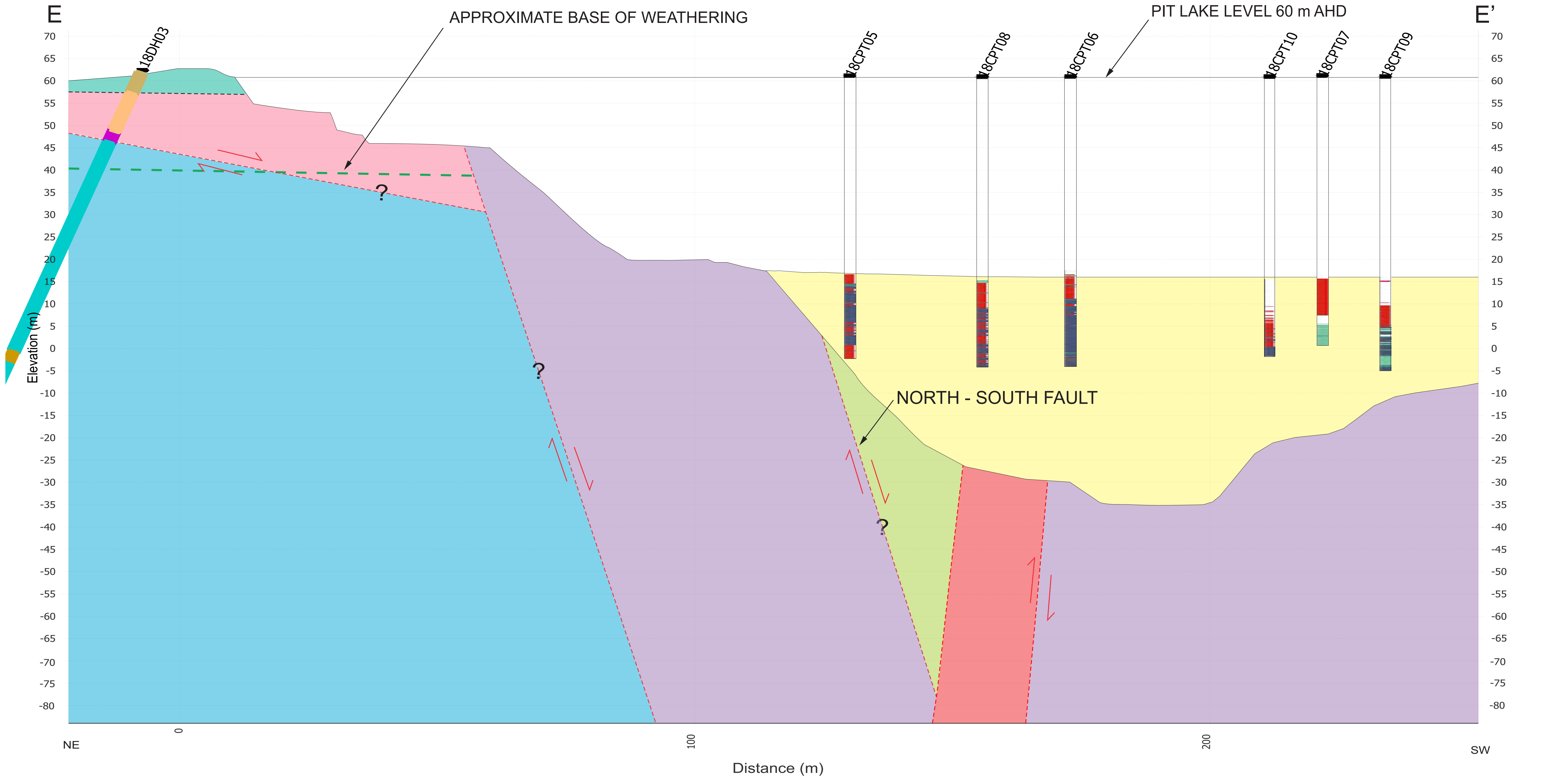

COOMALIE DOLOSTONE DOLOMITE, QUARTZITE	MAIN SHEAR ZONE DEFORMED, SHEARED BRECCIATED WHITES FORMATION
WHITES FORMATION SHALE, MUDSTONE, LOW GRADE METAMORPHICS	MINERALISED ZONE
GEOLSEC FORMATION QUARTZITE BRECCIA	SAPROLITE / LATERITE SANDY GRAVELLY CLAY
UNCONTROLLED FILL TAILINGS	UNCONTROLLED FILL SOIL & BEACH DEPOSITS
FAULTED CONTACT INFERRED	

CPT - SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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Status:	Final	Version:	1.0
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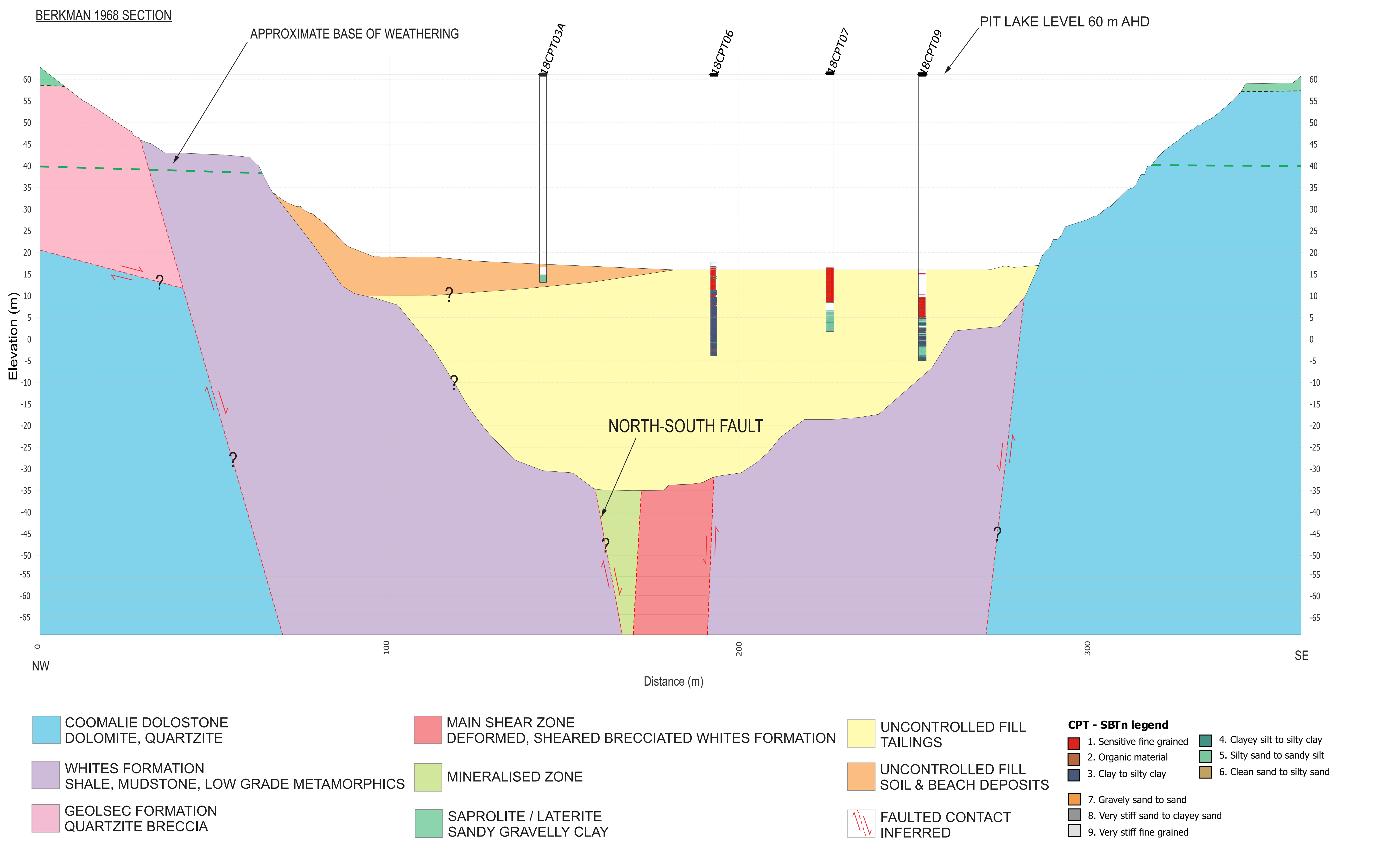

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Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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APPENDIX C

SRK Extracted Defect Orientation Data

Depth (borehole sting depth m)	Borehole	alpha	beta	dip	dip direction	set
25.03	18DH01	44	35	63	350	1
25.21	18DH01	32	15	77	335	1
25.43	18DH01	35	60	66	13	1
25.5	18DH01	34	140	42	89	4
35.32	18DH01	60	300	43	283	5
35.44	18DH01	38	280	58	256	5
37.8	18DH01	28	130	51	81	4
44.56	18DH01	57	210	18	202	3
46.38	18DH01	54	210	21	198	3
42.65	18DH01	50	90	44	30	6
28.6	18DH01	37	30	71	347	1
27.35	18DH01	42	10	68	330	1
27.16	18DH01	37	35	70	351	1
26.2	18DH01	48	310	57	284	5
33.48	18DH03	45	40	62	166	2
33.97	18DH03	70	60	35	166	2
34.88	18DH03	50	315	56	102	4
36.76	18DH03	25	170	44	302	1
38.79	18DH03	55	50	51	170	2
41.92	18DH03	65	150	12	230	3
47.53	18DH03	40	120	42	236	3
49.45	18DH03	35	130	44	250	3
49.6	18DH03	60	30	49	154	2
56.9	18DH03	65	25	45	150	2
62.73	18DH03	45	90	49	205	3a
65.34	18DH03	60	80	39	187	3a
71.88	18DH03	69	30	41	151	2
73.3	18DH03	41	180	28	315	5
73.45	18DH03	51	25	59	153	2
73.33	18DH03	57	0	54	135	2
73.8	18DH03	57	350	54	128	2
73.9	18DH03	43	60	60	182	2
77.59	18DH03	74	290	30	104	4
78.67	18DH03	56	50	50	169	2
78.82	18DH03	66	300	39	101	4
80.18	18DH03	71	315	37	112	4
80.36	18DH03	50	235	32	37	6
83.18	18DH03	49	220	28	20	6
84.39	18DH03	43	10	68	143	2
84.79	18DH03	59	20	51	148	2
85.13	18DH03	48	35	60	161	2
86.26	18DH03	45	260	45	56	6

APPENDIX D

Hoek Brown Criterion

Whites Fm Shale

Analysis of Rock Strength using RocLab

Hoek-Brown Classification

intact uniaxial comp. strength (sigci) = 25 MPa
GSI = 20   mi = 6   Disturbance factor (D) = 0.5
intact modulus (Ei) = 5000 MPa
modulus ratio (MR) = 200

Hoek-Brown Criterion

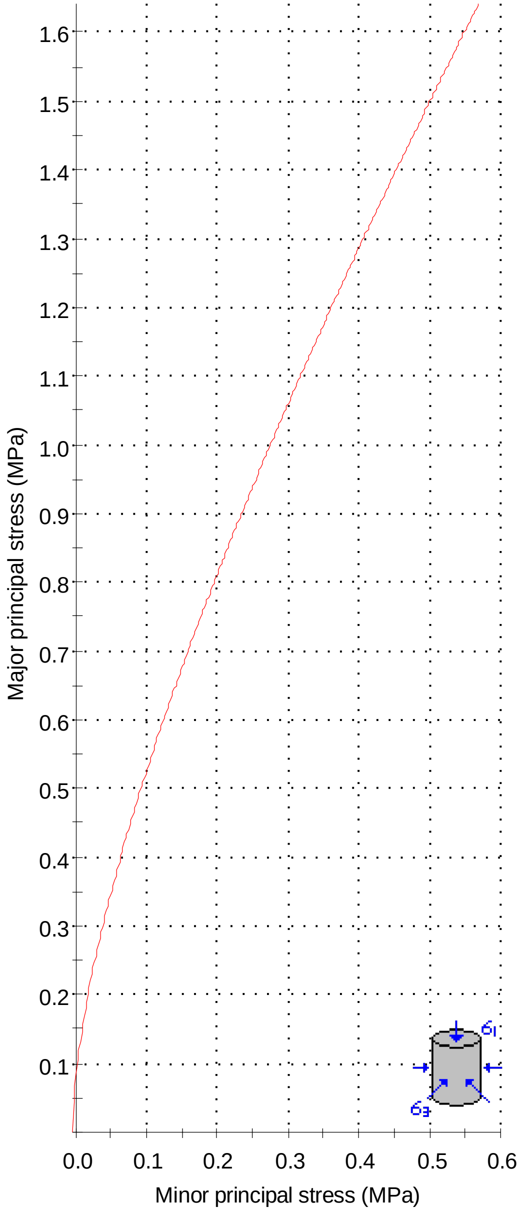
mb = 0.133   s = 2.33e-5   a = 0.544

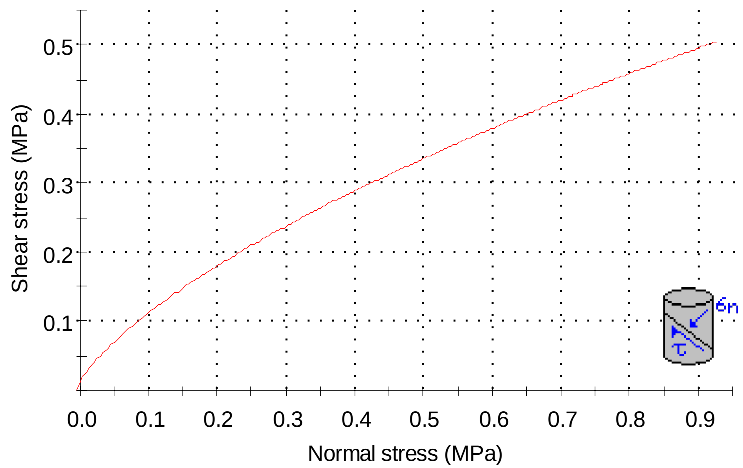
Mohr-Coulomb Fit

cohesion = 0.080 MPa   friction angle = 25.91 deg

Rock Mass Parameters

tensile strength = -0.004 MPa
uniaxial compressive strength = 0.076 MPa
global strength = 0.915 MPa
deformation modulus = 149.31 MPa





Whites Fm Shear Zone

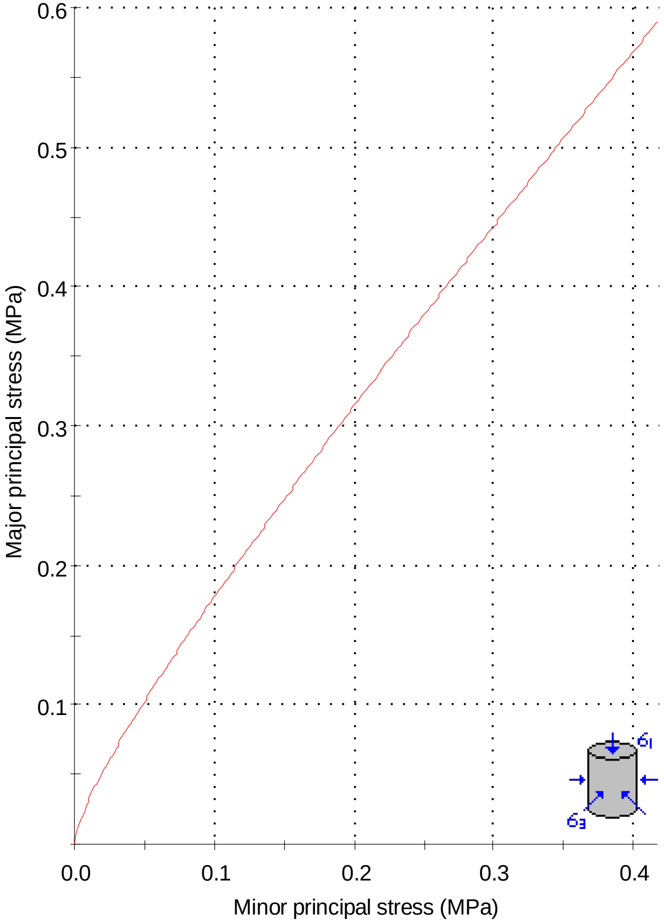
Analysis of Rock Strength using RocLab

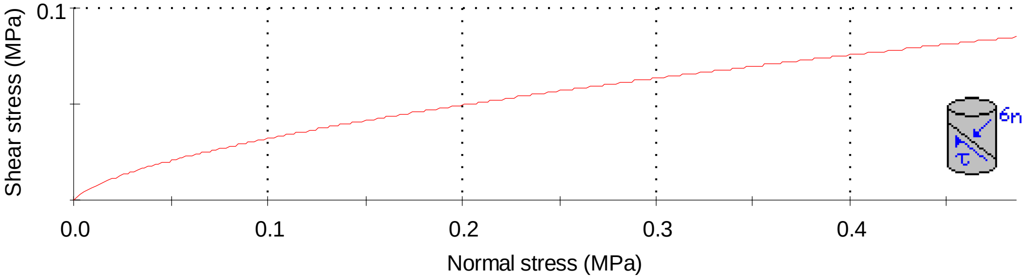
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 1 MPa
GSI = 15   mi = 6   Disturbance factor (D) = 0.5
intact modulus (Ei) = 200 MPa
modulus ratio (MR) = 200

Hoek-Brown Criterion
mb = 0.105   s = 1.2e-5   a = 0.561

Mohr-Coulomb Fit
cohesion = 0.016 MPa   friction angle = 8.54 deg

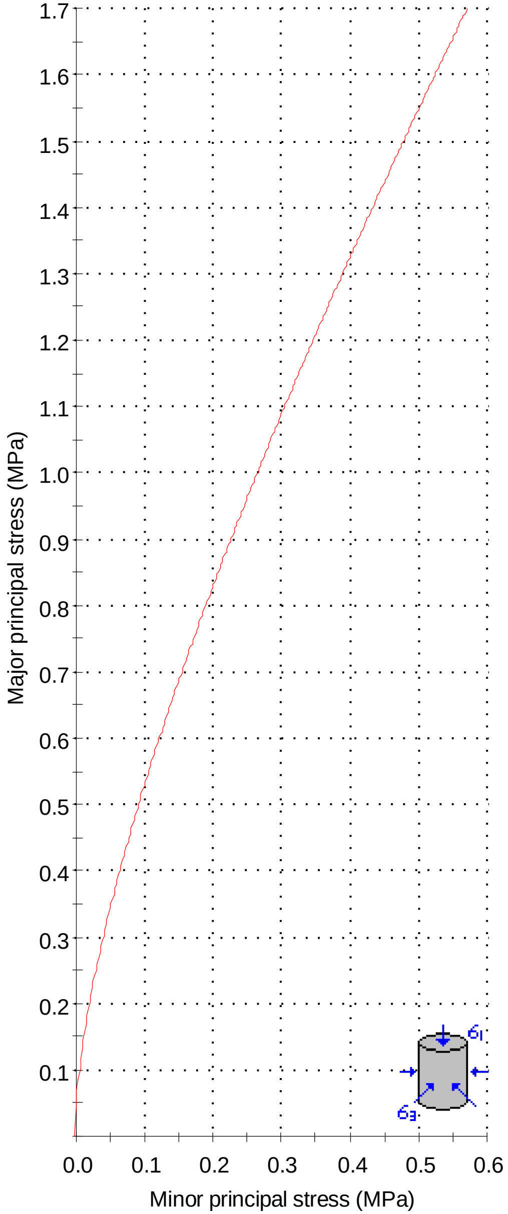
Rock Mass Parameters
tensile strength = -0.000114 MPa
uniaxial compressive strength = 0.002 MPa
global strength = 0.029 MPa
deformation modulus = 5.26 MPa





Whites Fm Schist

Analysis of Rock Strength using RocLab

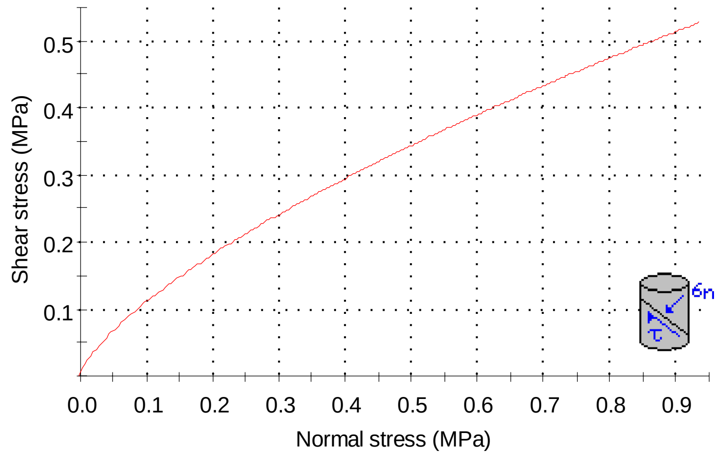


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 25 MPa
GSI = 15 mi = 10 Disturbance factor (D) = 0.5
intact modulus (Ei) = 16875 MPa
modulus ratio (MR) = 675

Hoek-Brown Criterion
mb = 0.175 s = 1.2e-5 a = 0.561

Mohr-Coulomb Fit
cohesion = 0.077 MPa friction angle = 26.96 deg

Rock Mass Parameters
tensile strength = -0.002 MPa
uniaxial compressive strength = 0.043 MPa
global strength = 0.948 MPa
deformation modulus = 443.64 MPa



Whites Fm Meta Sandstone

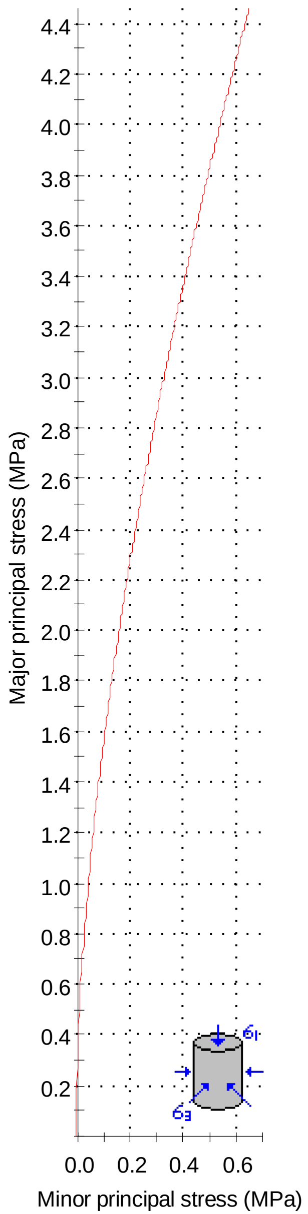
Analysis of Rock Strength using RocLab

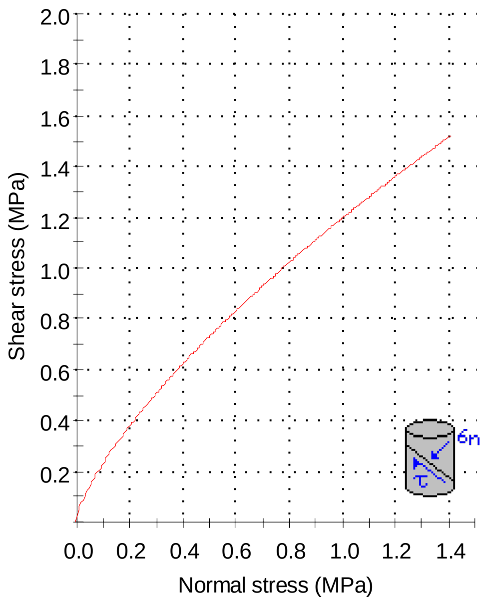
Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 40 MPa
GSI = 30 mi = 19 Disturbance factor (D) = 0.5
intact modulus (Ei) = 10000 MPa
modulus ratio (MR) = 250

Hoek-Brown Criterion
mb = 0.678 s = 0.0001 a = 0.522

Mohr-Coulomb Fit
cohesion = 0.203 MPa friction angle = 44.72 deg

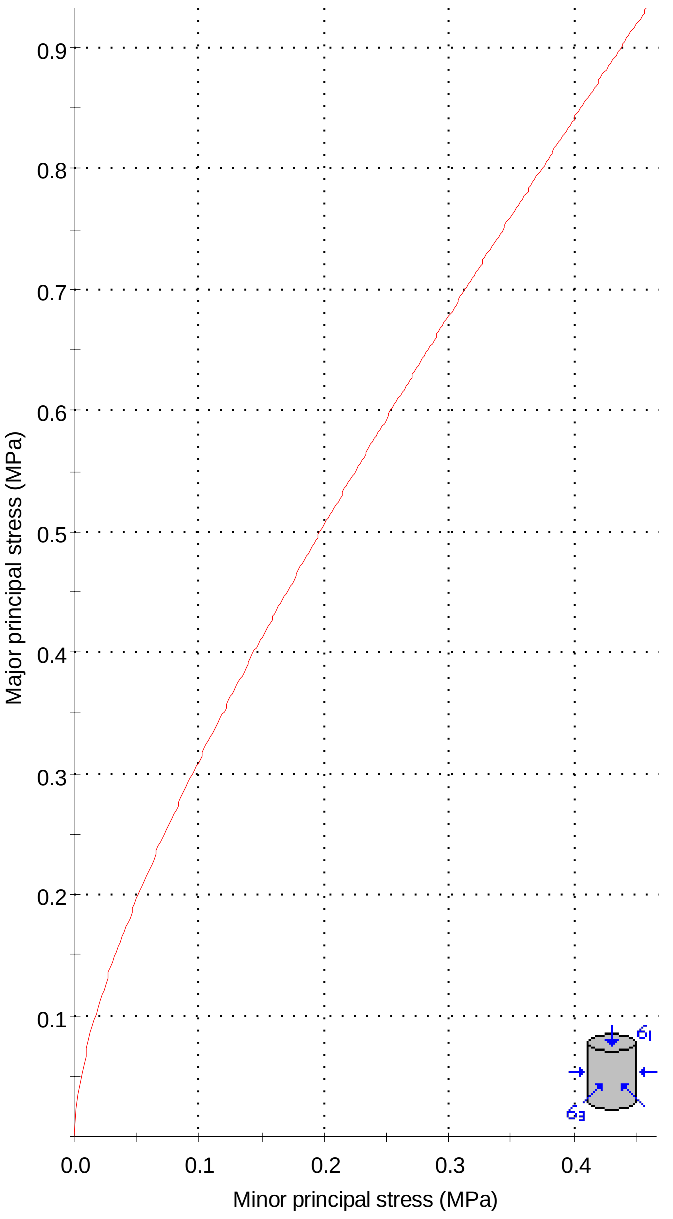
Rock Mass Parameters
tensile strength = -0.005 MPa
uniaxial compressive strength = 0.305 MPa
global strength = 3.945 MPa
deformation modulus = 440.09 MPa





HW Quartz Breccia

Analysis of Rock Strength using RocLab

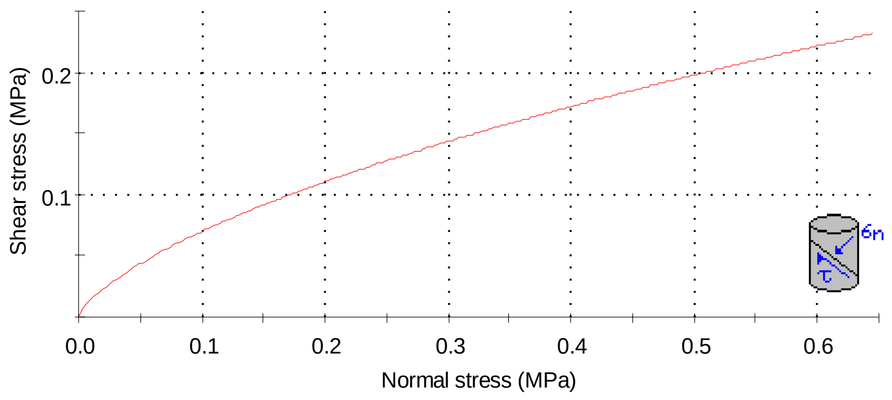


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 1 MPa
GSI = 25 mi = 19 Disturbance factor (D) = 0.5
intact modulus (Ei) = 500 MPa
modulus ratio (MR) = 500

Hoek-Brown Criterion
mb = 0.534 s = 4.54e-5 a = 0.531

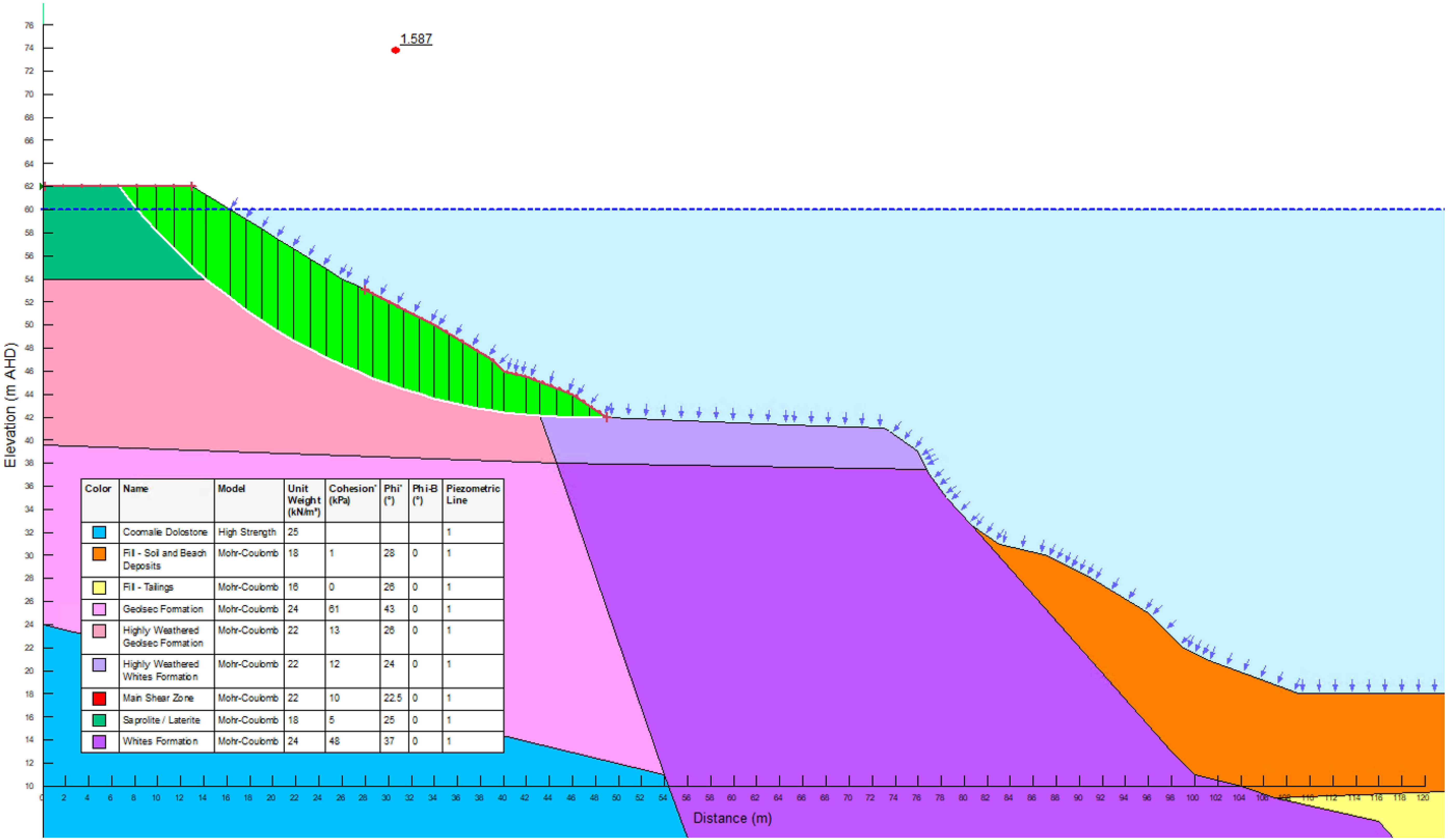
Mohr-Coulomb Fit
cohesion = 0.042 MPa friction angle = 17.34 deg

Rock Mass Parameters
tensile strength = -8.5e-5 MPa
uniaxial compressive strength = 0.005 MPa
global strength = 0.083 MPa
deformation modulus = 17.71 MPa



APPENDIX E

Geostudio® Slope/W Main Pit Wall Output



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25				1
	Fill - Soil and Beach Deposits	Mohr-Coulomb	18	1	28	0	1
	Fill - Tailings	Mohr-Coulomb	16	0	26	0	1
	Geolsec Formation	Mohr-Coulomb	24	61	43	0	1
	Highly Weathered Geolsec Formation	Mohr-Coulomb	22	13	26	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22	12	24	0	1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	1
	Saprolite / Laterite	Mohr-Coulomb	18	5	25	0	1
	Whites Formation	Mohr-Coulomb	24	48	37	0	1

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Title:	Appendix E -Berkman 1968 Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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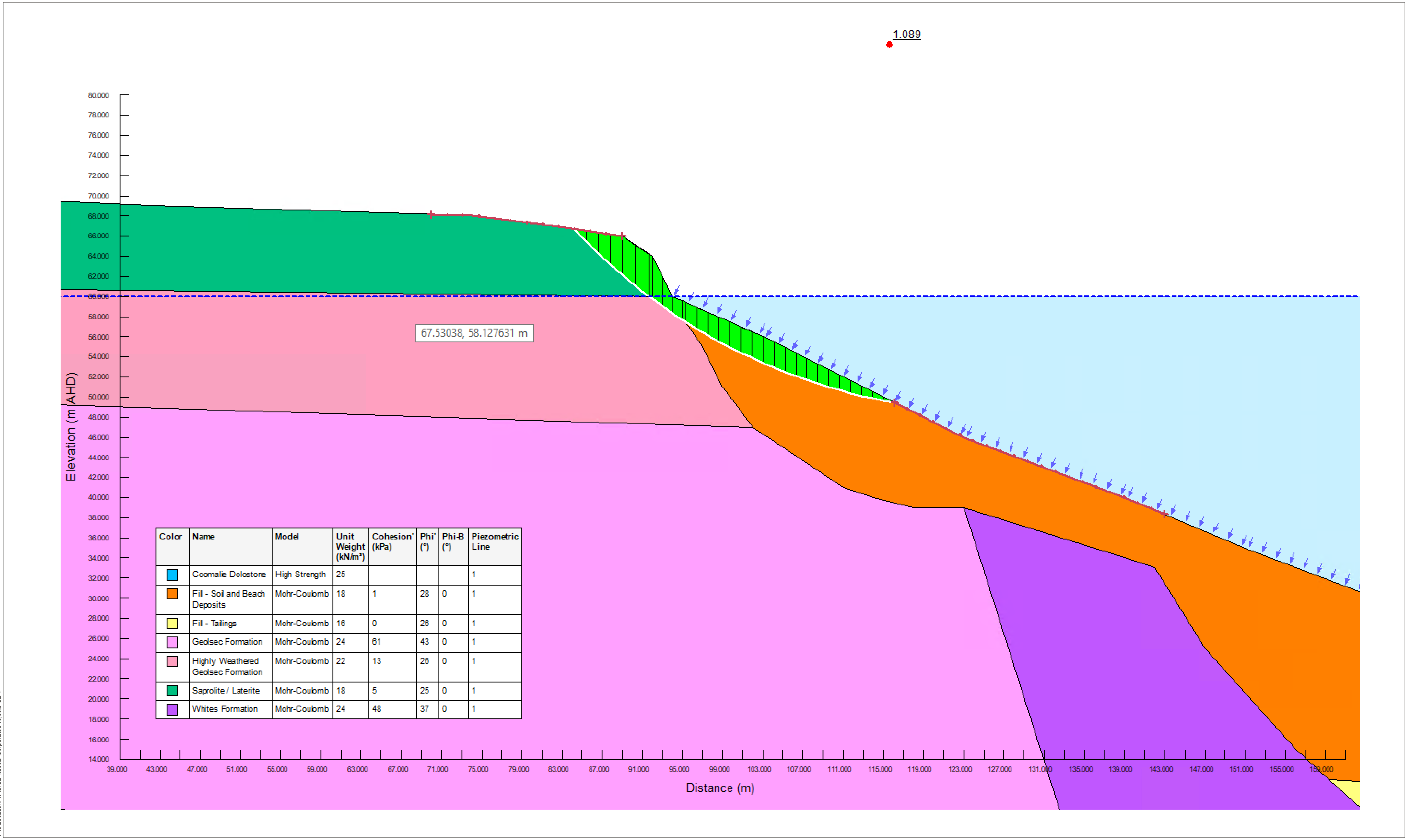

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25				1
	Fill - Soil and Beach Deposits	Mohr-Coulomb	18	1	28	0	1
	Fill - Tailings	Mohr-Coulomb	16	0	26	0	1
	Geolsec Formation	Mohr-Coulomb	24	61	43	0	1
	Highly Weathered Geolsec Formation	Mohr-Coulomb	22	13	26	0	1
	Saprolite / Laterite	Mohr-Coulomb	18	5	25	0	1
	Whites Formation	Mohr-Coulomb	24	48	37	0	1

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Title:	Appendix E - A-A' Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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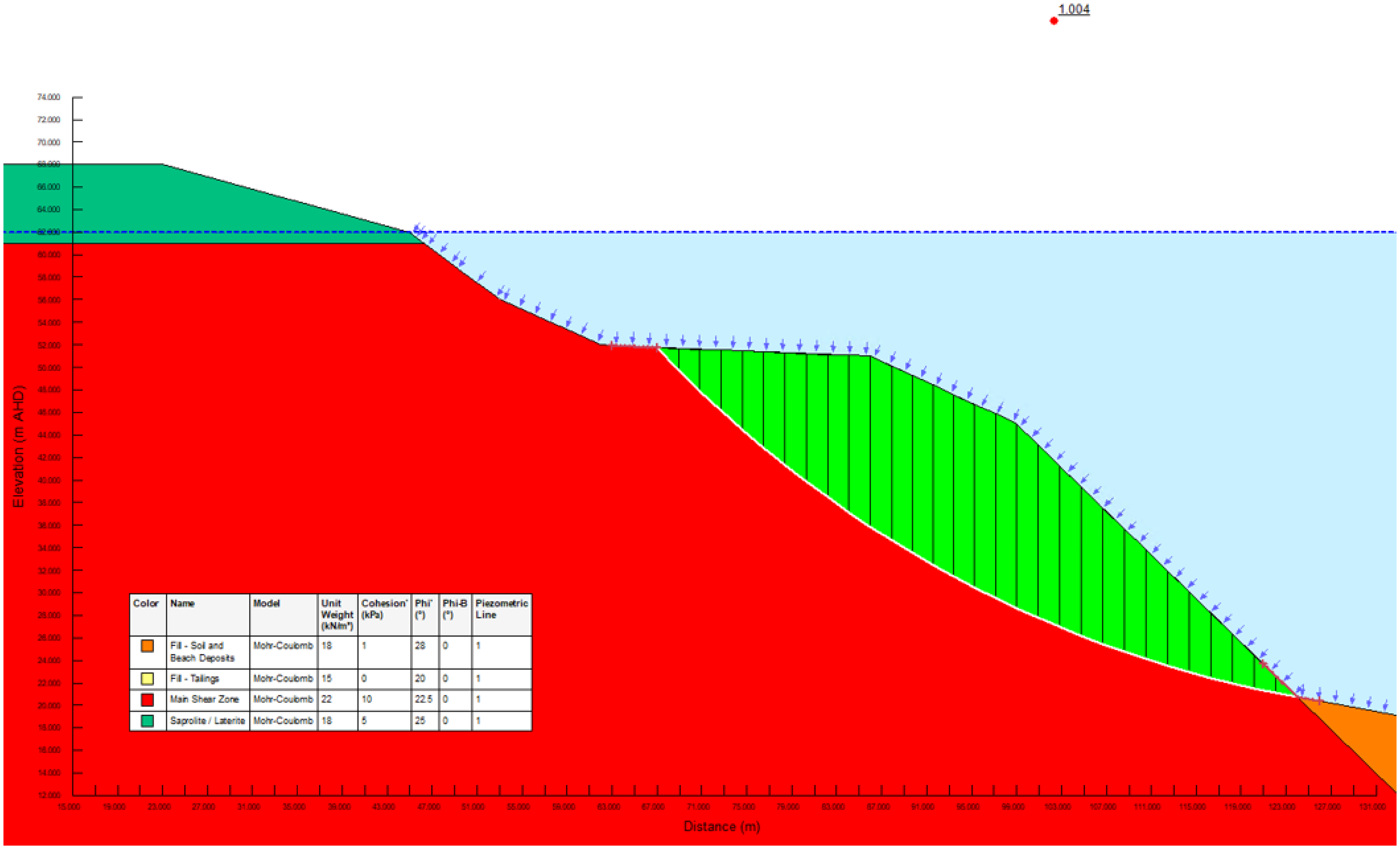

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Fill - Soil and Beach Deposits	Mohr-Coulomb	18	1	28	0	1
	Fill - Tailings	Mohr-Coulomb	15	0	20	0	1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	1
	Saprolite / Laterite	Mohr-Coulomb	18	5	25	0	1

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Title:	Appendix E - B-B' Back Analysis	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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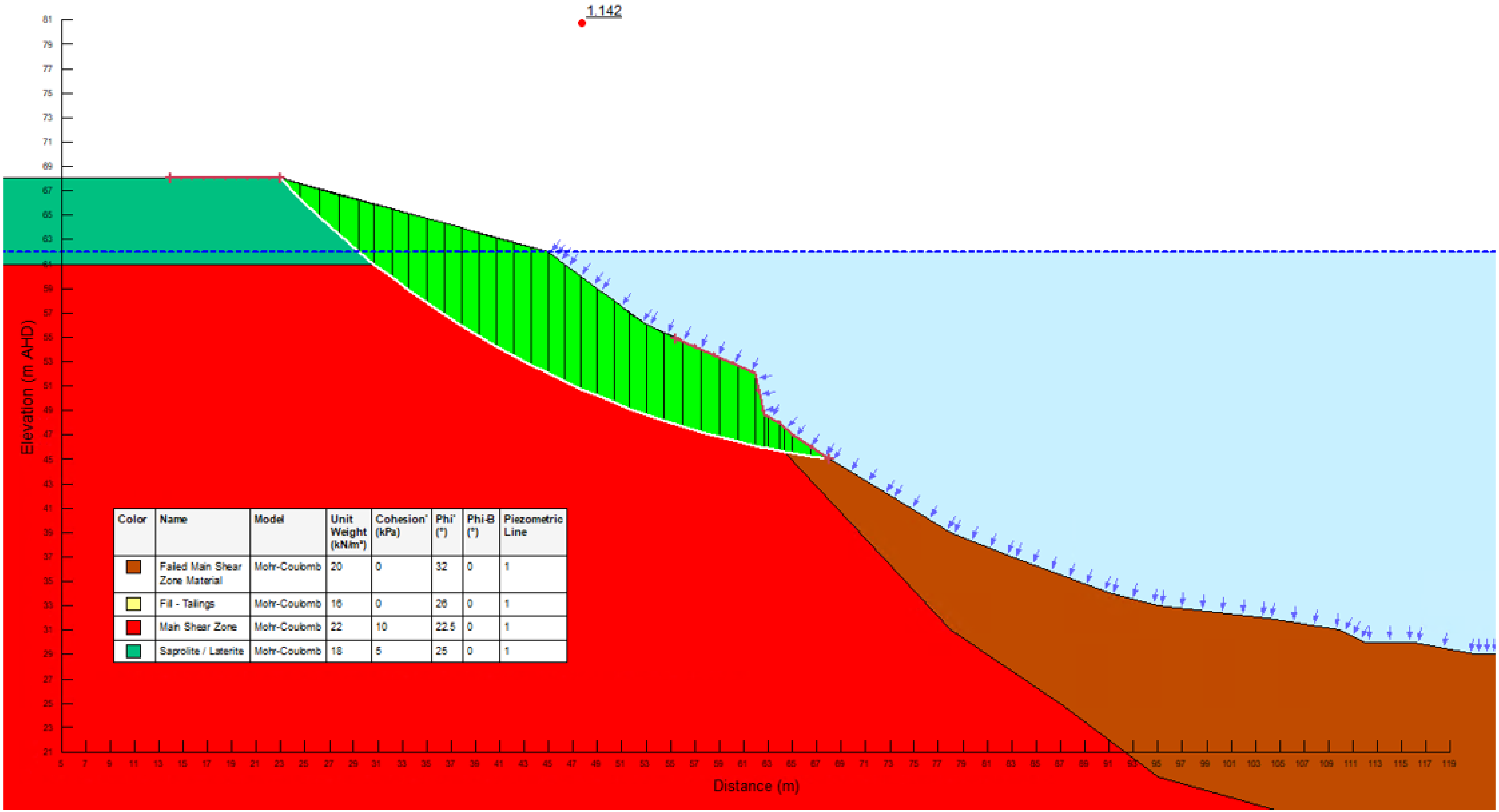

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Failed Main Shear Zone Material	Mohr-Coulomb	20	0	32	0	1
	Fill - Tailings	Mohr-Coulomb	16	0	26	0	1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	1
	Saprolite / Laterite	Mohr-Coulomb	18	5	25	0	1

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Title:	Appendix E - B-B' Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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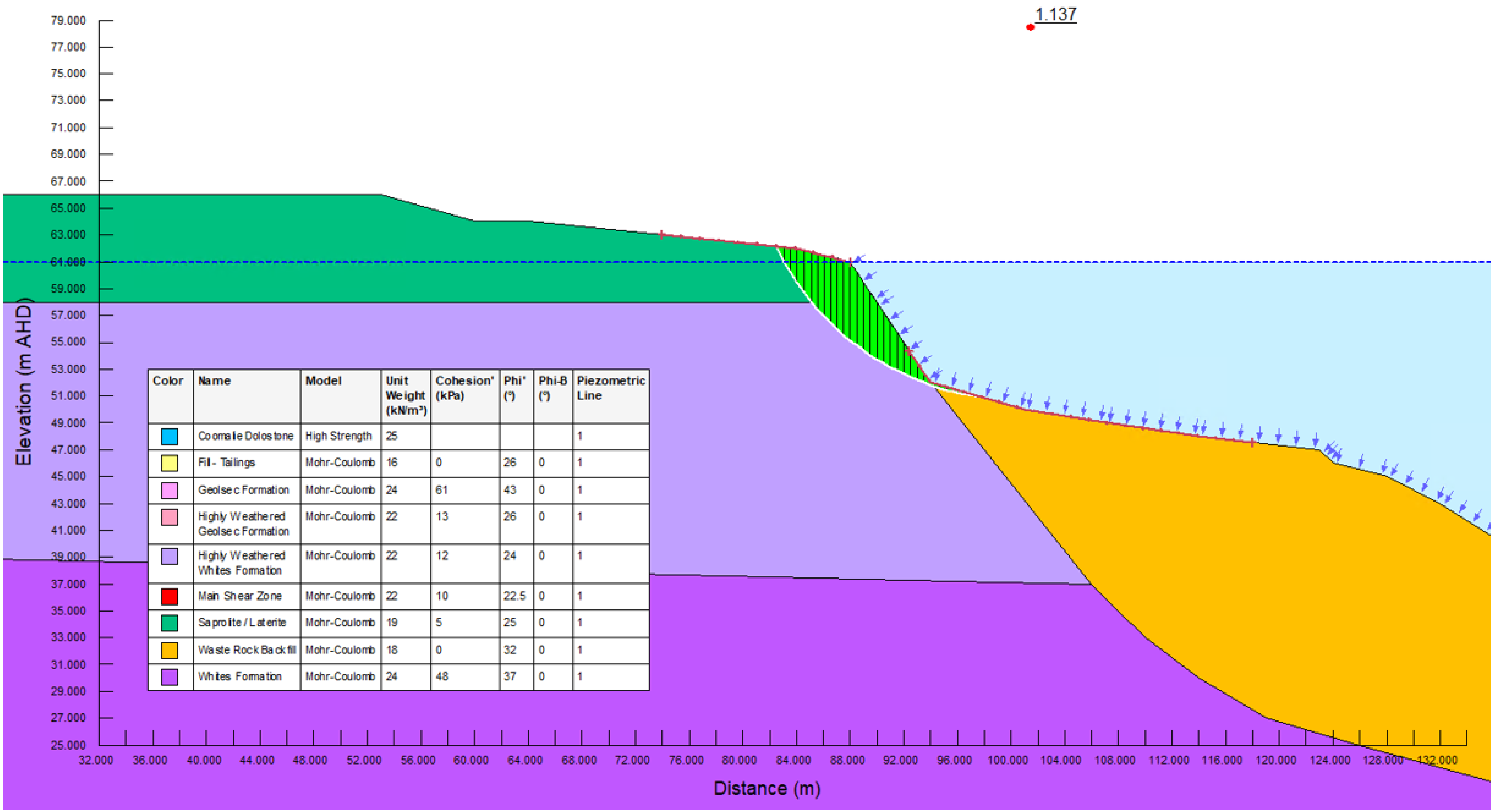

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25				1
	Fill - Tailings	Mohr-Coulomb	16	0	26	0	1
	Geolsec Formation	Mohr-Coulomb	24	61	43	0	1
	Highly Weathered Geolsec Formation	Mohr-Coulomb	22	13	26	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22	12	24	0	1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	1
	Saprolite / Laterite	Mohr-Coulomb	19	5	25	0	1
	Waste Rock Backfill	Mohr-Coulomb	18	0	32	0	1
	Whites Formation	Mohr-Coulomb	24	48	37	0	1

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Title:	Appendix E - C-C' Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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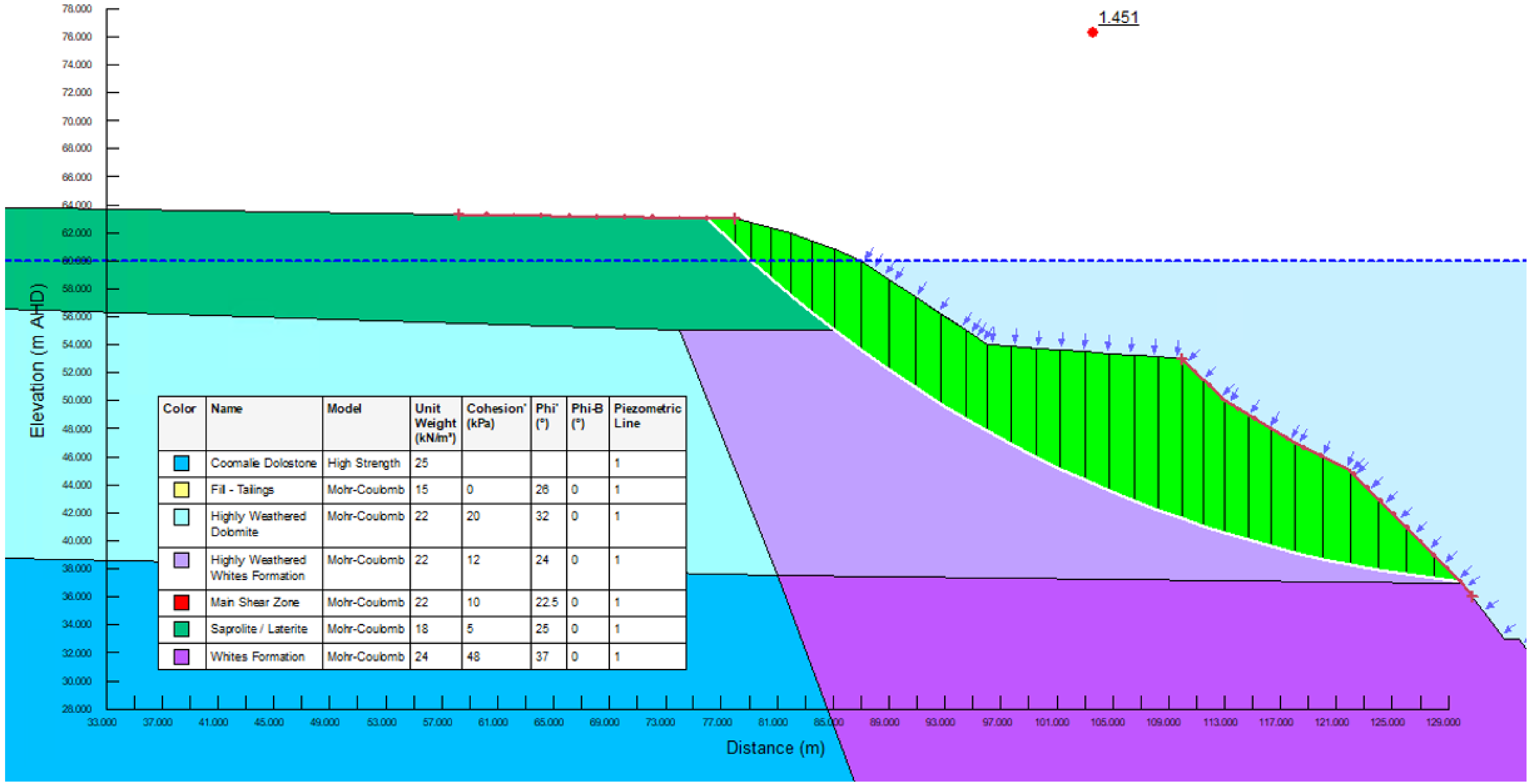

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25				1
	Fill - Tailings	Mohr-Coulomb	15	0	26	0	1
	Highly Weathered Dolomite	Mohr-Coulomb	22	20	32	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22	12	24	0	1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	1
	Saprolite / Laterite	Mohr-Coulomb	18	5	25	0	1
	Whites Formation	Mohr-Coulomb	24	48	37	0	1

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Title:	Appendix E - D-D' Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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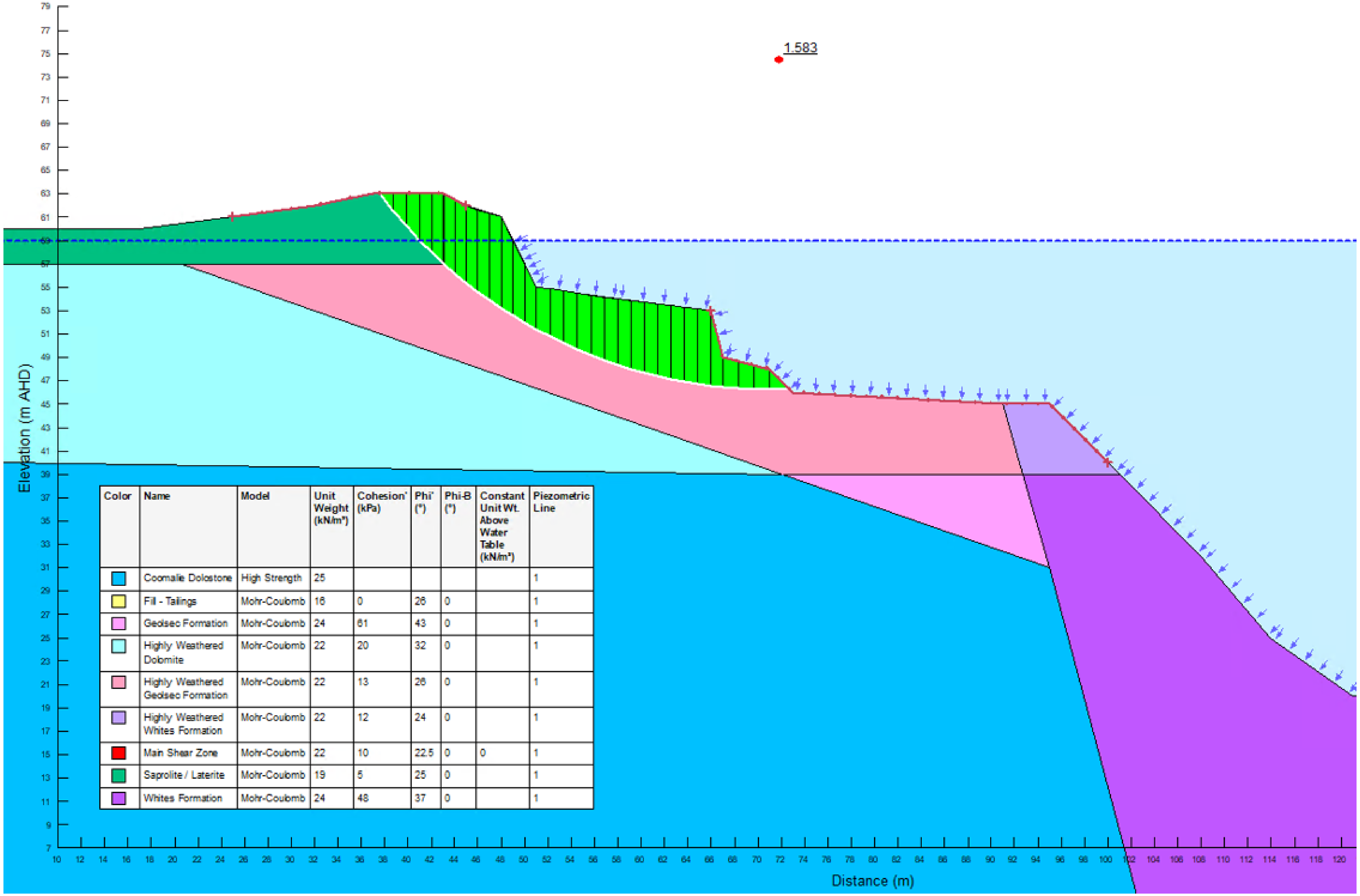

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Constant Unit Wt. Above Water Table (kN/m³)	Piezometric Line
	Coomalie Dolostone	High Strength	25					1
	Fill - Tailings	Mohr-Coulomb	16	0	26	0		1
	Geolsec Formation	Mohr-Coulomb	24	61	43	0		1
	Highly Weathered Dolomite	Mohr-Coulomb	22	20	32	0		1
	Highly Weathered Geolsec Formation	Mohr-Coulomb	22	13	26	0		1
	Highly Weathered Whites Formation	Mohr-Coulomb	22	12	24	0		1
	Main Shear Zone	Mohr-Coulomb	22	10	22.5	0	0	1
	Saprolite / Laterite	Mohr-Coulomb	19	5	25	0		1
	Whites Formation	Mohr-Coulomb	24	48	37	0		1

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Title:	Appendix E - E-E' Static	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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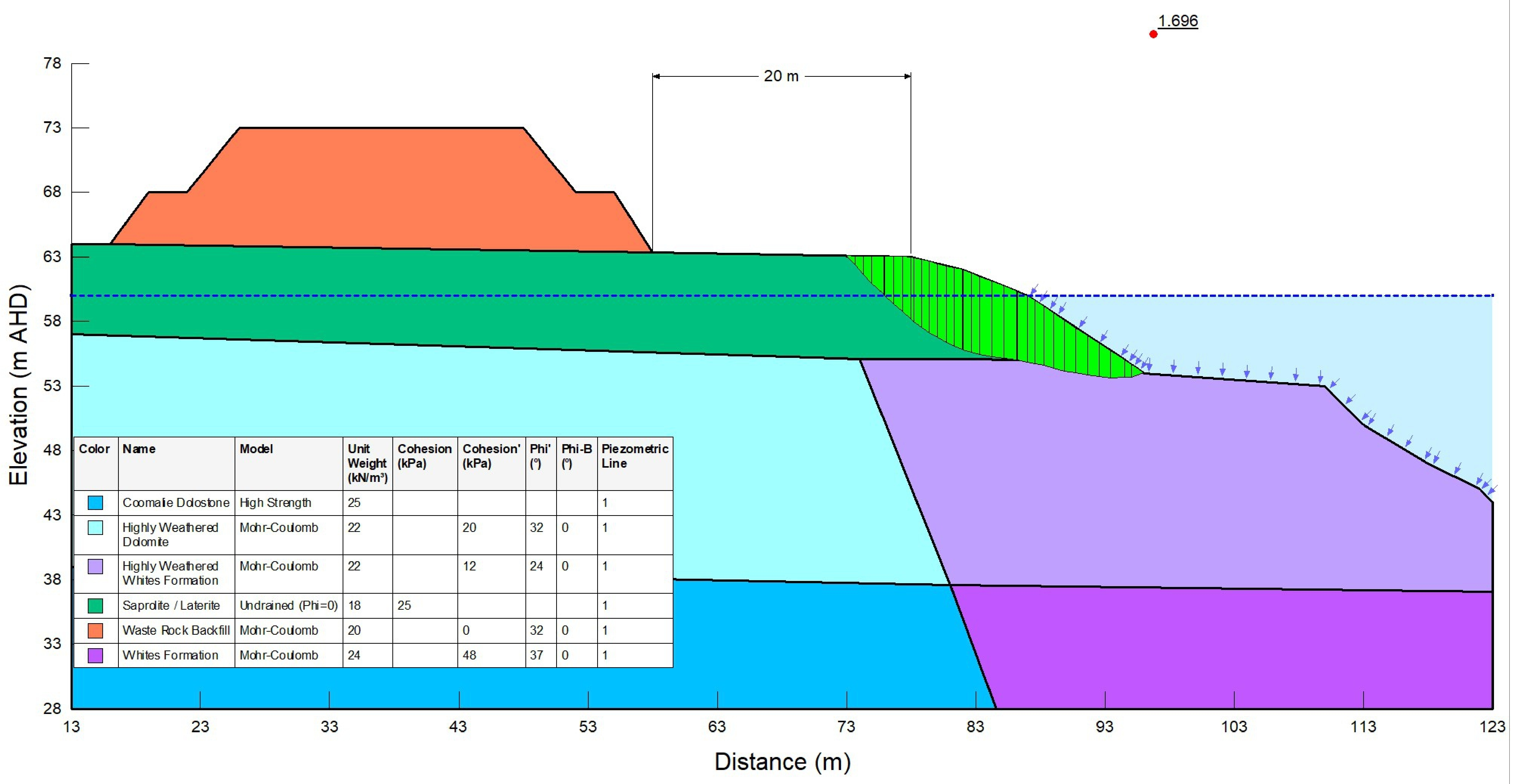

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25					1
	Highly Weathered Dolomite	Mohr-Coulomb	22		20	32	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22		12	24	0	1
	Saprolite / Laterite	Undrained (Phi=0)	18	25				1
	Waste Rock Backfill	Mohr-Coulomb	20		0	32	0	1
	Whites Formation	Mohr-Coulomb	24		48	37	0	1

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Title:	Appendix E - Stockpile Undrained	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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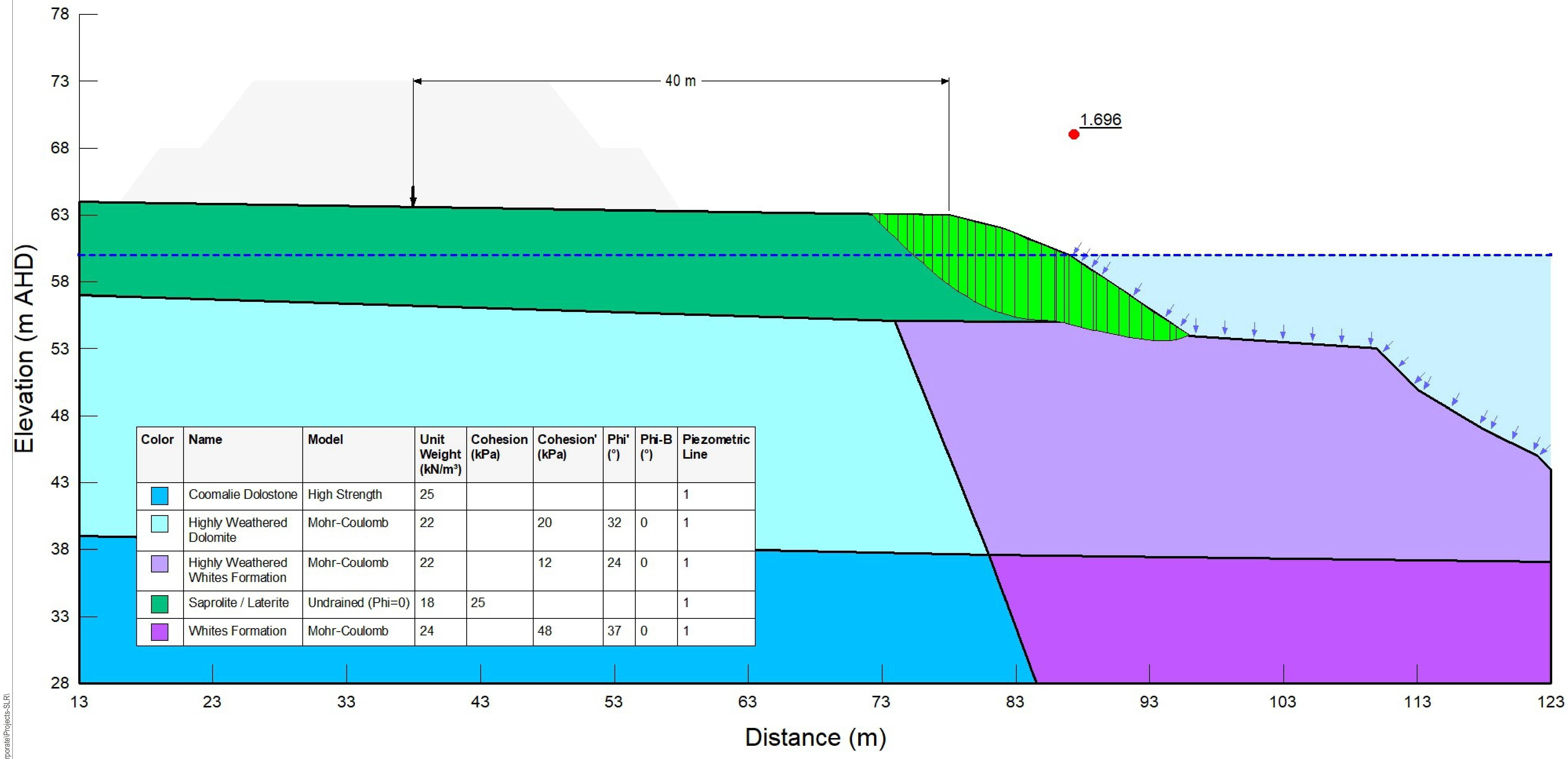

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25					1
	Highly Weathered Dolomite	Mohr-Coulomb	22		20	32	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22		12	24	0	1
	Saprolite / Laterite	Undrained (Phi=0)	18	25				1
	Whites Formation	Mohr-Coulomb	24		48	37	0	1

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Title:	Appendix E - Crane Undrained	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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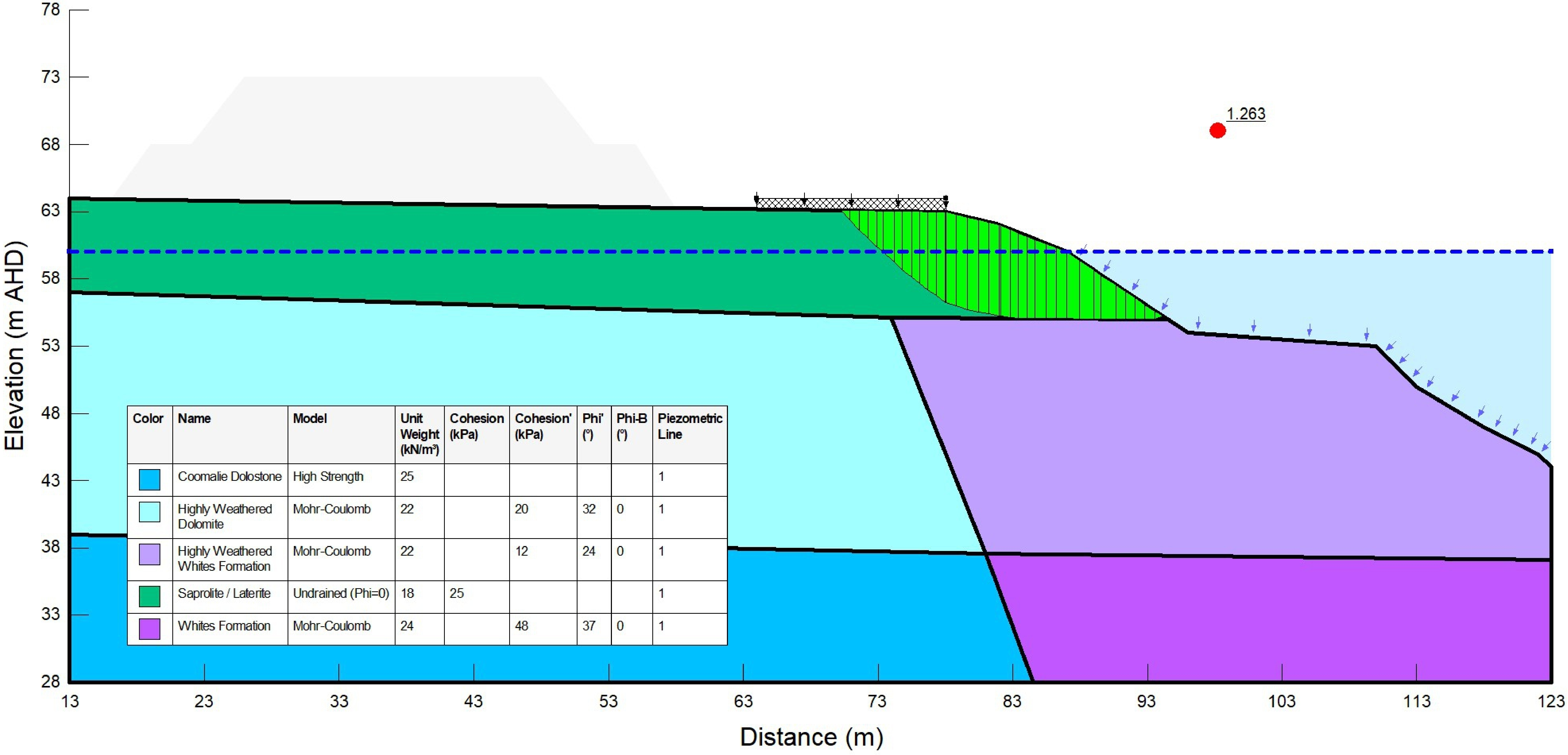

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25					1
	Highly Weathered Dolomite	Mohr-Coulomb	22		20	32	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22		12	24	0	1
	Saprolite / Laterite	Undrained (Phi=0)	18	25				1
	Whites Formation	Mohr-Coulomb	24		48	37	0	1

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Title:	Appendix E - Light Vehicles Undrained	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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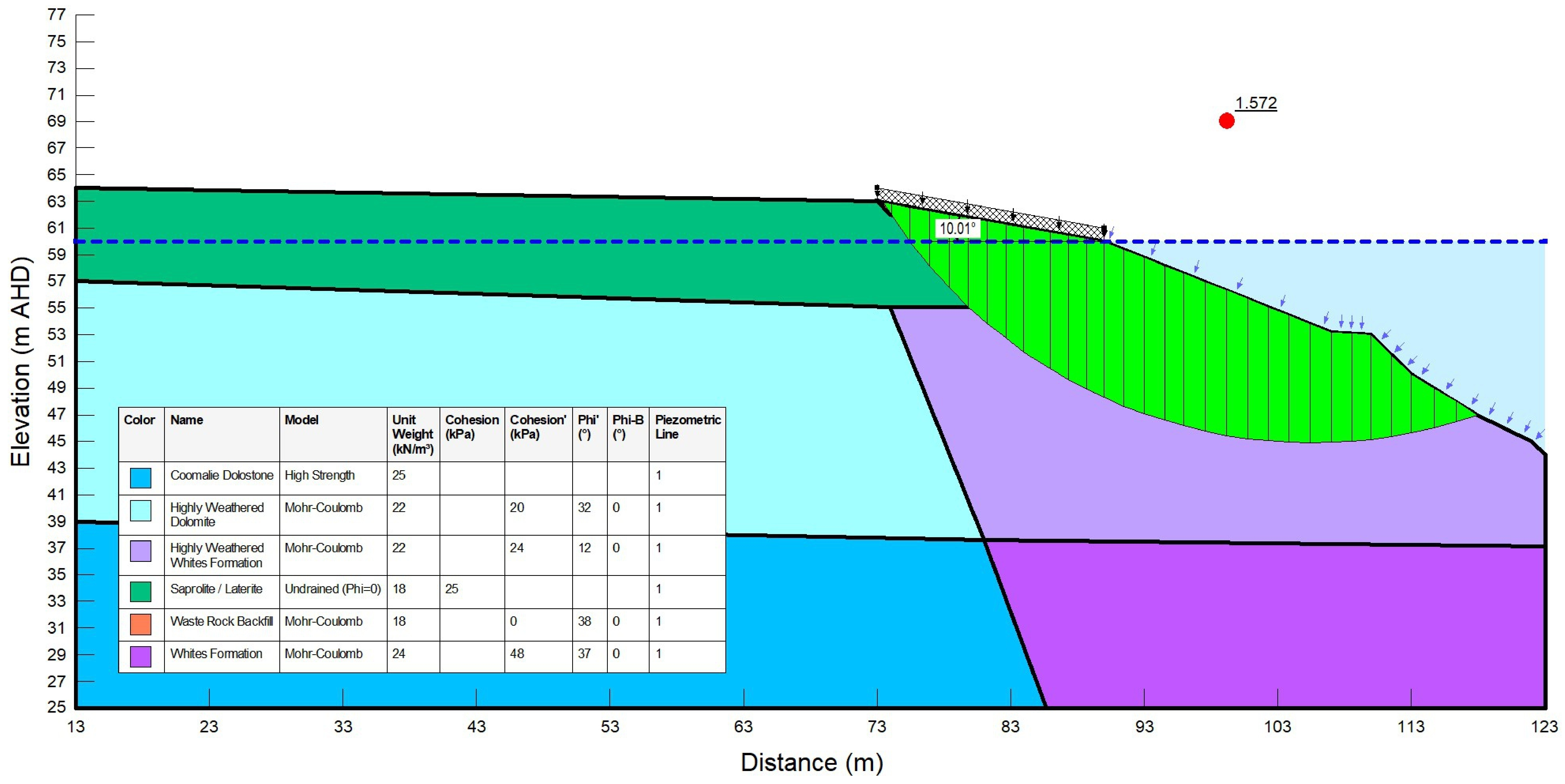

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Coomalie Dolostone	High Strength	25					1
	Highly Weathered Dolomite	Mohr-Coulomb	22		20	32	0	1
	Highly Weathered Whites Formation	Mohr-Coulomb	22		24	12	0	1
	Saprolite / Laterite	Undrained (Phi=0)	18	25				1
	Waste Rock Backfill	Mohr-Coulomb	18		0	38	0	1
	Whites Formation	Mohr-Coulomb	24		48	37	0	1

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Title:	Appendix E - Ramp Undrained	Drawn:	AR
Client:	Department of Primary Industries and Resources	Reviewed:	DO
Project:	Rum Jungle Rehabilitation	Size:	A3
Project No.:	680.10421	Datum:	m AHD
Status:	Final	Version:	1.0
Date:	06/02/2020		

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APPENDIX F

Soil Parameters