REPORT

BOWDENS SILVER PTY LIMITED

Bowdens Silver Project

Lue, N.S.W.

Tailings Storage Facility Preliminary Design

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CONDITIONS OF INVESTIGATION AND REPORT

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7. Geotechnical site investigation necessarily involves the investigation of the subsurface conditions at a site at a few isolated locations, and the interpretation and extrapolation of those conditions to elsewhere on the site not so investigated. This procedure has been adopted at the site that is the subject of this report and due care and skill has been applied in carrying out and reporting on the work. Thus the findings, conclusions and comments contained in this report represent professional estimates and opinions and are not to be read as facts unless the context makes it clear to the contrary. In general, statements of fact are confined to statements as to what was done and/or what was observed. Other statements have been based on professional judgement.

8. The scope of the work has been planned in the absence of any fore-knowledge of the site other than that stated in the report. Unless otherwise stated we consider that the number of locations investigated and the depths to which they have been investigated are reasonable bearing in mind the scale and nature of the project, and the defined purpose for which the investigation was undertaken.

9. We do not accept any responsibility for any variance between the interpreted and extrapolated conditions and those that are revealed by any means subsequently. Specific warning is also given that many factors, either natural or artificial, may render ground conditions different from those which pertained at the time of the investigation. Should there be revealed during the construction or at any other time any apparent difference from subsurface conditions described or assessed in this report, it is strongly recommended that such differences be brought to our attention so that its significance may be assessed and appropriate advice given.

ATC Williams

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COMMONLY USED ACRONYMS

ACRONYM	NAME
AEP	Annual Exceedance Probability
AHD	Australian Height Datum
ANCOLD	Australian National Committee on Large Dams
ARI	Annual Recurrence Interval
ATCW	ATC Williams
BGM	Bituminous Geomembrane
BGW	Borehole Groundwater Identification
BH	Borehole Identification
BOM	Bureau of Meteorology
BP	Bingham Plastic (a model of fluid viscosity)
CH	High Plasticity Clay
CT	Crystal Tuff
D/S	Downstream
EPA	Environment Protection Authority
FS	Factor of Safety
GLE	General Limit Equilibrium (method of stability analysis)
IFD	Intensity Frequency Duration (Rainfall)
ISD	Initial Settled Density
LOM	Life of Mine
MDD	Maximum Dry Density
MDE	Maximum Design Earthquake
NAF	Non-Acid Forming
NSW DSC	New South Wales Dams Safety Committee
OBE	Operating Basis Earthquake
OMC	Optimum Moisture Content (for compaction of clays)
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
Project	Bowdens Silver Project
SG	Soil Particle Density (Specific Gravity)
SV	Soil Vision (Software)
TSF	Tailings Storage Facility
U/S	Upstream
USACE	US Army Corps of Engineers
VB	Volcanic Breccia
WRE	Waste Rock Emplacement

EXECUTIVE SUMMARY

The Project

The Bowdens Silver Project (Project) comprises an open cut Silver Mine on a greenfield site located 2 km to 3 km north-east of Lue. The distance between Lue and the edge of the main open cut pit is approximately 2.8 km. The closest point of the TSF embankment to Lue is 2km, however, a substantial ridge is present between the TSF and Lue. The TSF is located in a valley that has its confluence with Lawsons Creek downstream of Lue and approximately 26 km south-east of Mudgee in NSW.

The Project would have an operational life of approximately 15 years, with a total dry tonnage of approximately 30 Mt and a throughput of 2 Mtpa.

Tailings Storage Facility

The Tailings Storage Facility (TSF) for the Project has been designed on the philosophy of a down valley tailings discharge TSF for its practicality and as a cost-effective solution. As the decant pond on the tailings is at the embankment, the embankment has been designed to be a full water retaining structure with both an operating pond of water and tailings against the embankment. The embankment is to be constructed in three stages; the two raises adopt the downstream raise method.

After consideration of three options, the TSF is to be located to the west of the main open cut pit and processing plant. The embankment for this TSF option lies across the valley of an intermittent watercourse. Thickened tailings are to be discharged from the head of three valleys forming the upper portion of the TSF impoundment. The decant pond would form against the embankment at the south-west side of the TSF. The catchment for the TSF is relatively small at approximately 3.0 km^2 with the impounded tailings taking up approximately 1.0 km^2 of this area.

As the Project is a greenfield project, the derivation of suitable parameters for this feasibility study has been based on limited data, and accordingly further studies and testing would be carried out during detailed design. Following the assessment of two tailings samples (Ignimbrite and Crystal Tuff (CT)) the CT was adopted to provide the best representation of the tailings to be discharged into the TSF. The tailings characterisation test work showed that this material is a low plasticity clay with a particle density of 2.7 t/m^3. Later in the design process further test work was carried out on a Volcanic Breccia (VB) manufactured tailings sample to provide further parameters for a more detailed design of the impoundment clay liner and an estimate of the seepage beneath the TSF embankment. The test work on the CT showed that a high-rate thickener would deliver tailings with a solids concentration in the order of 56% solids. Testing by Graeme Campbell & Associates (2019) indicated that the tailings would be potentially acid generating.

At the start of deposition, when the rate of rise of the tailings is high, an in-situ density of around 1.35 t/m^3 has been estimated. The overall consolidated density of the tailings at the LOM has been estimated at 1.5 t/m^3.

Tailings Beach Slope

The tailings beach slope varies with the number of tailings discharge points as well as other parameters such as solids content, rheology and discharge rates. Based on experience, the beach slope would be concave, and the main tailings beach would divide into four parts; upper, middle, lower and runout. For the start of deposition, assuming a consistent rate of deposition, it is expected one effective tailings stream would be maintained and the beach slopes are estimated to vary from upper, middle and lower as 1.4 %, 1.0 % and 0.7 % respectively with a run-out of 0.2%. Considering two tailings streams later in the LOM, the later slopes are estimated to be 1.8%, 1.35% and 1.0% with a runout of 0.7%.

TSF Embankment

The 56 m maximum height TSF embankment would mainly comprise a combination of rock and earthfill and would be constructed in three stages. The embankment would include a low permeability Bituminous Geomembrane (BGM) Liner fitted to the upstream face, a zone of curtain grouting of the foundations along the upstream toe of the embankment to a nominal depth of 40 m and a concrete plinth connecting the BGM to the foundation grouting also along the upstream toe of the embankment.

The starter embankment would provide approximately 3 years of storage, with the Stage 2 filling in 5 years and Stage 3 a further 7 years. The TSF embankment and storage details are provided in **Table E1** and a summary of the embankment earthwork and geomembrane quantities is presented in **Table E2**.

TABLE E1

TSF EMBANKMENT AND STORAGE DETAILS

<table>
<tr><th rowspan="3">Stage</th><th rowspan="3">Cumulative Storage Capacity (Mt)</th><th rowspan="3">Tailings Elevation at Decant (m, AHD)</th><th colspan="5">Embankment</th></tr>
<tr><th rowspan="2">Maximum Embankment Height (m)</th><th rowspan="2">Crest Elevation (m, AHD)</th><th rowspan="2">Crest Width (m)</th><th>D/S slope</th><th>U/S slope</th></tr>
<tr><th colspan="2">(horizontal: vertical)</th></tr>
<tr><td>Stage 1 - Start-up</td><td>6.0</td><td>595.0</td><td>38</td><td>601.5</td><td>20</td><td rowspan="2">1.5 :1 and 2.5:1</td><td rowspan="3">2.25:1</td></tr>
<tr><td>Stage 2</td><td>16.0</td><td>603.7</td><td>47</td><td>611.0</td><td>20</td></tr>
<tr><td>Stage 3</td><td>30.0</td><td>613.1</td><td>56</td><td>620.0</td><td>20</td><td>2.5:1</td></tr>
</table>

TABLE E2

SUMMARY OF EMBANKMENT EARTHWORK AND GEOMEMBRANE QUANTITIES

Stage	Clay Zone 1 (m^3)	Filters (m^3)		Rockfill (m^3)		Bituminous Geomembrane (BGM) liner (m^2)
		Zone 2A	Zone 2B	Zone 3A	Zone 3B	
Stage 1	134,000	29,000	31,000	180,000	689,000	44,000
Stage 2	78,000	16,000	16,000	117,000	833,000	27,000
Stage 3	78,000	16,000	16,000	109,000	1,254,000	28,000
Total	**290,000**	**61,000**	**63,000**	**406,000**	**2,776,000**	**99,000**

Consequence Category
Based on the potential for acid generation and as the TSF is located in a rural environment, the TSF has been assessed at a Consequence Category of "High C" in accordance with the NSW Dam Safety Committee Tailings Dam Guidelines.

The embankment crest level has been designed on the basis of a Consequence Category of High C to contain the expected pond levels obtained from the water balance together with wave run-up and additional contingency freeboard.

The emergency spillway has been designed for the High C Consequence Category with a 1 in 100,000 AEP for the critical flow.

Decant Pond and Return Water
The decant water accumulating within the TSF would be utilised for processing, with water returned via a pontoon-mounted pumping station installed within the TSF.

The water balance shows the decant pond level (with a 99% probability of non-exceedance) provides a volume of around 1.3 Mm^3 towards the end of Stage 1, 1.4 Mm^3 towards the end of Stage 2 and 1.5 Mm^3 two years before the end of LOM. All these volumes are less than the storage capacity of the decant on the tailings at each stage. The Stage 3 also allows for the storm storage, plus allowances for wave run-up and additional freeboard based on the ANCOLD guidelines (2012) on top of the expected maximum operating pond (with 50% probability of non-exceedance) for Stage 3.

The decant pond is however not expected to have capacity to supply all the water required for the processing plant.

Seepage Management
Within the TSF impoundment, seepage management includes foundation treatment to reduce seepage within the area below the maximum inundation level. This includes using in-situ clay where the existing thickness is more than 0.5 m or importing and placing clay where the existing thickness is less than 0.45 m thick, so as to achieve a compacted clay liner of a minimum depth of 0.45 m. There would be no additional clay placed outside the area of the maximum inundation level, with the in-situ clay considered to be adequate.

At the TSF embankment, seepage is constrained by a reinforced Bituminous Geomembrane (BGM) Liner fitted to the upstream face, a concrete plinth connecting to the nominal 40 m deep foundation curtain grouting along the upstream toe, as well as a drainage system in the downstream foundation area of the embankment, draining to a seepage collection system at the embankment downstream toe. Steady state seepage at the end of LOM is estimated to be less than 3 L/sec. This would be collected by the seepage collection system, drained to the seepage collection ponds and subsequently pumped back to the TSF. Further work will be carried out to confirm the assumptions made in this system during detailed design.

A contingency has been allowed for further foundation treatment and an underdrainage (seepage interception) system in the area up to 150 m upstream of the embankment beneath the area of the operating pond. Further work will be carried out to assess the merits of the underdrainage system during detailed design.

Performance Monitoring

Instrumentation would be installed to monitor groundwater pressures and levels, groundwater quality, seepage volumes and embankment settlements, along with the routine recording of tailings discharge tonnages, decant pond levels, and return water volumes.

Seepage monitoring would be undertaken using a combination of vibrating wire piezometers to measure the pressure gradient across the grout curtain beneath the embankment and standpipes to allow the collection of water quality samples beyond the seepage collection ponds. Settlement monitors would also be installed during each raise of the embankment.

Seepage monitoring would be undertaken on a weekly basis along with a record of tailings discharge tonnages, return water volumes from the seepage pond and the decant as well as water levels on the tailings and in the seepage collection ponds. Settlement monitors would be surveyed once a month.

Regular inspections would be made of the TSF facility and an annual audit would be carried out by a Dams Engineer. Every second year, the audit by the Dams Engineer would be comprehensive commencing one year after the start of filling.

Closure and Rehabilitation

Closure and rehabilitation studies have been undertaken separately. Generally, the rehabilitation would likely consist of a low flux store and release cover over the tailings. To achieve a self-shedding profile, the final deposition of the tailings would be modified to shift the low point towards the final spillway location. For closure, the spillway invert would be lowered to match the top of the cover. Any material cut from the closure spillway would be used for rehabilitation works such as flattening the embankment or for cover. The closure spillway will be designed for the Probable Maximum Flood (PMF), with suitable erosion protection and energy dissipators. Once tailings deposition ceases and the decant water is removed it is expected that the quantity of seepage would reduce, and the quality would improve. Seepage would be pumped back to a small lined pond on the TSF and then the main open cut pit until the quality has improved to acceptable levels

It is expected that the TSF embankment would be flattened in sections on the downstream slope and both this slope and the covered tailings would be revegetated to reduce the potential for erosion.

1 INTRODUCTION

This report presents the results of the preliminary design for the tailings storage facility (TSF) proposed for the Bowdens Silver Project (Project). ATC Williams (ATCW) were engaged in October 2016 by Anthony McClure of Bowdens Silver Pty Limited (Bowdens Silver) to undertake a TSF study.

This work has been undertaken to provide a practical and cost-effective solution for tailings deposition and long-term storage of tailings generated by the Project. The work has been undertaken to a level commensurate with the information available at the time of the study.

2 PROJECT BACKGROUND

2.1 General

The Project consists of an open cut silver mine on a greenfield site located 2 km from the north-east corner of Lue to the closest activity within the Mine Site i.e. the southern barrier. The distance between Lue and the edge of the main open cut pit is approximately 2.8 km. The closest point of the TSF embankment to Lue is 2 km, however, a substantial ridge is present between the TSF and Lue. The TSF is located in a valley that has its confluence with Lawsons Creek downstream of Lue and is approximately 26 km east of Mudgee in NSW. This study has been based on a life of mine (LOM) tonnage of 30 Mt with an annual throughput of approximately 2 Mtpa.

2.2 Option Study

Three options for the location of the TSF in the two valleys adjoining the main open cut pit were considered during the initial evaluation of options. Various discharge arrangements, including perimeter discharge, down valley discharge and degrees of tailings thickening up to and including paste and dry stacking were considered in the early stages of the project.

Once the LOM dry tonnage of 30 Mt was adopted by Bowdens Silver in late February 2018 and following further discussions, three options were considered for a high-level option study:

- down valley tailings discharge into the two valleys (two options) to the west of the main open cut pit; and
- perimeter tailings discharge in the eastern valley.

The results of this high-level option study were provided to Bowdens Silver on 22 March 2018, and the down valley tailings discharge within Walkers Creek valley to the west of the main open cut pit was adopted.

2.3 Scope of Study

As initial capital expenditure was a critical factor for the Project, a TSF with down valley discharge, high solids content discharge from a high rate thicker, and an embankment at a location with more favourable foundation conditions was chosen by Bowdens Silver for the preliminary design. The final location of the TSF is shown in **Figure 1**, together with the final tailings surface, which is discussed in **Section 13**.

3 SITE CONDITIONS

3.1 Site Description

The site chosen for the TSF is to the west of the main open cut pit and processing plant. The proposed TSF is located in the Walkers Creek valley formed by undulating lower slopes and steeper high slopes varying in grade between 10% and 25%. The natural catchment is around 3.0 km^2 and has an intermittent creek on the valley floor with a bed slope at about 3% to the west. The catchment and surrounding area are presented in **Figure 1,** the site locality plan. The upper higher areas of the catchment are covered with trees, whereas the lower area in the valley and on the lower slopes are covered with grass, the occasional tree, and frequent rock outcrops.

3.2 Regional Geology

The Mudgee Regional Geology 1:100,000 scale map indicates that the subsurface materials at the TSF site comprise Permian Age, Rylstone Volcanics comprising dacitic and rhyolitic pyroclastic and epiclastic rocks with tuffaceous sandstone. These are overlain by Sydney Basin, Shoalhaven Group, conglomerate, and sandstone. The maps also indicate limited areas of overlying Illawarra Coal Measures.

Preliminary geological plans and sections were also provided by Bowdens Silver. This information is consistent with the regional geology described above but provides more detail at a site scale; including the location of some discontinuities such as faults (refer to **Section 7**).

4 START-UP SCHEDULE

As the start-up schedule has varied throughout the study, all time for this report is from the commencement of tailings deposition unless otherwise stated.

5 GEOTECHNICAL INVESTIGATION

5.1 General

A preliminary geotechnical investigation was carried out during May and June 2017 and the results are presented in ATCW report "Tailings Storage Facility Dam and Water Storage Dam, Preliminary Geotechnical Investigation" 116217.05R01, October 2017 [Ref. **1**]. The results of the preliminary investigation of the TSF embankment foundation and the impoundment are summarised below. It should be noted when reading the Geotechnical Investigation report [Ref. **1**] and comparing figures that the tailings deposition and the TSF embankment height and footprint have altered between the time of the geotechnical investigation and this preliminary design. The geotechnical investigation locality plan has been updated to include the current final (Stage 3) preliminary design and is presented in **Figure 2.**

5.2 Fieldwork

Field investigations included borehole drilling, in-situ permeability testing, installation of groundwater monitoring bores, test pit excavations and bulk sampling for subsequent laboratory testing. The main aims of the geotechnical investigation were to:

- Investigate condition and type of the foundation materials beneath the TSF Embankment footprint;

- Estimate the rock mass permeability and identify any high permeability zones where possible;
- Assess topsoil and potential foundation stripping requirements;
- Identify surface conditions within the TSF impoundment; and
- Locate any potential clay borrow areas within the impoundment.

The following works located in **Figure 2**, were carried out:

- Three (3) boreholes were drilled within the footprint of the TSF Embankment to depths that ranged from 25 m to 33 m together with permeability testing (9 packer tests and falling head tests) and the installation of 2 standpipe piezometers at BH2 location;
- Six (6) test pits in the TSF Embankment footprint area were excavated to refusal using a 20 t excavator; and
- Eighteen (18) test pits inside the tailings impoundment area, which could also be potential borrow areas, were excavated to refusal using a 20 t excavator.

During the investigation, samples were extracted for laboratory testing to further identify foundation and potential construction material properties.

5.3 Subsurface Conditions TSF

The investigation for the TSF shows that the topsoil covers the valley in varying thickness between 0.2 m to 0.45 m but generally around 0.25 m. The soils observed beneath the topsoil overlie rock to depths between 0.55 m and 6.8 m. The subsoils below the topsoil vary in general from high plasticity Clay to Clayey Sand and Sandy Clay. The thickness of clay varies. A significant area of Clay, Sandy Clay, and Clayey Sand was found in the northern part of the impoundment in the valley. Some of these clays, the darker brown alluvials/colluvials, are low plasticity and potentially dispersive whereas most of the clay observed on site is medium to high plasticity and non-dispersive. In the Walkers Creek valley, the soil transitions at depth from Clayey materials to more Gravelly Clay and Clayey Gravel, residual to extremely weathered through to moderately weathered rock.

It should be noted that the definition of Clay used in this body of work is that outlined in AS1726 (1993), not the recent revision AS1726 (2017), with the intent that the description of Clay in this and any previous site investigations match. The definition of fine-grained material in AS1726 (1993) is 50 % passing 75µm whereas the current standard, AS1726 (2017), is 35 % passing 75µm. This has had little effect on the definition of soils from the TSF site.

The hills forming the valley have very little soil cover over the weathered rock.

The rock in the footprint of the TSF comprises Tuff and Volcanoclastic Breccia on the west side of the valley with Rhyolite overlying Tuff in the valley and to the east. The rock has some jointing and staining but is generally massive. The rock strength varies from weak to moderate strength in the Rhyolite and moderate to very high strength in the tuff and Volcanoclastic Breccia.

5.4 Foundation Permeability

The foundation conditions at the northern abutment of the TSF embankment include volcanic breccia. The permeability of this section, based on BH1, decreases with depth from 1.7×10^{-6} m/s at 6.0 m to as low as 1.6×10^{-10} m/s between 18.0 m and 28.0 m with an overall average of 2.4×10^{-8} m/s.

In the valley and the southern abutment of the TSF embankment, the foundations consist of rhyolite and welded tuff which displayed higher permeability. Based on BH2 and BH3, there was no distinct pattern in the permeability changes with depth or position. The permeability varies between 6.9 x 10^{-6} m/s and less than 1 x 10^{-10} m/s in this area.

5.5 Groundwater

The preliminary geotechnical investigation included only one borehole, BH02 (BGW60) in the base of the valley of the proposed TSF embankment drilled to approximately 33 m and completed as a groundwater monitoring bore. The monitoring of the borehole indicated a water level within 1.5 m to 2 m of the surface. To check whether the water was possibly locally perched water in the floor of the valley a second shallow (approximately 5 m deep) borehole, BH 02-2 (BGW61), was drilled beside BH 02. The water levels in BH 02-2 were consistently 1.0 m to 1.5 m lower than the levels in BH 02 up to March 2018 and again from April 2019 onwards. In the intervening period, BH 02-2 on occasion had a higher ground water level than BH02 but most of the time remained lower. This suggests that while the groundwater level is close to the surface in the area, that the higher groundwater levels in BH 02 are possibly associated with a confined aquifer at depth. Further discussions on ground water are discussed in Ref. **2**.

The results of monitoring these boreholes are presented in Figure **3.**

The relatively high groundwater level may cause some challenges during construction and some local dewatering may be required.

5.6 Construction Materials

5.6.1 Clay

High plasticity Clay encountered to the north of the impoundment and within the TSF embankment foundation would be used for low permeability (Zone 1) elements in the embankment and Foundation Treatment B material in the impoundment. Other medium to low plasticity Clay, Sandy Clay, Gravelly Clay, Clayey Sand and Clayey Gravel may be identified as suitable for Zone 1 in the embankment. Further investigation and testing would be undertaken to assess the borrow materials and to avoid the use of dispersive clays for borrow.

5.6.2 Rockfill

It is understood that Non-Acid Forming (NAF) rockfill from waste rock would be made available for the construction of the rockfill and filter zones of the TSF. The rockfill would be tested for durability and strength properties during the geotechnical investigation for detailed design. Check testing to confirm that the rock is NAF would also be carried out.

6 TAILINGS TESTING

Tailings testing was carried out in the ATC Williams laboratory on two samples provided in February 2017, Ignimbrite and Crystal Tuff (CT) as well as on a third sample, Volcanic Breccia (VB), in 2019. These samples had been manufactured for laboratory testing by crushing and grinding exploration core. The results are provided in our Laboratory Testing Report 116217.04R02 [Ref. **3**] and 116217.08R01 [Ref. **4**], respectively. Following discussions with Neville Bergin of Neville Bergin and Associates, and Tony Mathwin of GR Engineering Services, it was agreed that the CT and VB samples provided the best representation of the tailings to be discharged into the TSF of the testing currently available. The following laboratory tests were undertaken:

- Particle Size Distribution (Sieve and Hydrometer)
- Atterberg Limits
- Particle Density (Specific Gravity)
- Segregation Threshold
- Minimum and Maximum Density
- Initial Settled Density
- Shrinkage Limit Density
- Moisture Content / Shear Strength Relationship
- Rheology
- pH
- Conductivity
- Rowe Cell Consolidation

7 TAILINGS TONNAGE AND DEPOSITION RATE

The preliminary design has been based on the deposition of 2 Mt of tailings per annum with a total of 30 Mt deposited over the 15-year LOM. A discharge solids content of 56 % for the tailings has been adopted on the basis of the thickener study carried out by mining equipment supplier Outotec on the CT tailings sample produced for testing in February 2017 [Ref. **5**].

8 DENSITY

Based on the initial tailings testing [Ref. **3**], the tailings parameters adopted for this study are presented in **Table 1**.

TABLE 1

TAILINGS DENSITY PARAMETERS ADOPTED FOR DESIGN

Discharge Solids Content (%)	Initial Settled Density (t/m^3)	Final Settled Density for Start-up Deposition (t/m^3)	Final Settled Density for after Start-up Deposition (t/m^3)	Over-all Tailings Density at the End of LOM (t/m^3)	Soil Particle Density (t/m^3)
56	1.04	1.35	1.6	1.5	2.7

The initial settled density value adopted was the result of undrained settled density laboratory testing on the CT sample [Ref. **3**] and this was confirmed by the initial settled density of the VB sample [Ref. **4**].

The final settled densities adopted have been varied to reflect the staging/ filling rates for the TSF. The initial rate of rise of the tailings is usually high as a result of the impoundment terrain and this results in lower densities. Consequently, the final density at startup and partway through filling the first raise was chosen as 1.35 t/m^3 which is slightly less than the CT drained settled density of 1.44 t/m^3 [Ref. **3**]. This was confirmed by the drained settled density of 1.39 t/m^3 for the VB sample [Ref. **4**] and the results of the consolidation modelling for Stage 1, reported in **Section 16.7** with an average dry density of 1.36 t/m^3. The final settled density for the remainder of the LOM has been adopted at 1.6 t/m^3 which is slightly higher than the shrinkage limit density test result for CT sample of 1.56 t/m^3 [Ref. **3**] to allow for some consolidation of the start-up tailings which would then be covered with further tailings. Based on the more recent shrinkage limit density test result for CT sample of 1. 60 t/m^3 [Ref. **4**], the final density after startup may be conservative.

9 DESIGN STANDARDS

The following design standards from both the Australian National Committee on Large Dams Incorporated (ANCOLD) and the New South Wales Dam Safety Committee (NSW DSC) have been considered for the preliminary design of the TSF.

- ANCOLD, "Guidelines on the Consequence Categories for Dams", October 2012 and revised October 2015 [Ref. **6**];
- ANCOLD, "Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure", May 2012 [Ref. **7**];
- ANCOLD, "Guidelines on Design of Dams for Earthquake, 1998 [Ref. **8**];
- ANCOLD, "Guidelines on Selection of Acceptable Flood Capacity for Dams", 2000 [Ref. **9**];
- NSW DSC, "Consequence Categories for Dams", DSC3A, May 2014 [Ref. **10**];
- NSW DSC, "Acceptable Flood Capacity for Dams", DSC3B, June 2010 [Ref. **11**];
- NSW DSC, "Acceptable Earthquake Capacity for Dams", DSC3C, June 2010 [Ref. **12**];
- NSW DSC, "Tailings Dams", DSC3F, June 2012 [Ref. **13**]; and
- NSW DSC, "General Dam Safety Considerations", DSC3G, June 2010 [Ref. **14**].

10 DESIGN PHILOSOPHY

The TSF for the Project has been designed on the philosophy of a down valley tailings discharge TSF for its practicality for rehabilitation and as a cost-effective solution. As the decant pond on the tailings is at the embankment, the embankment has been designed to maintain both an operating pond of water and tailings against the embankment. The embankment has been designed to be constructed in three stages, the two raises of which both are downstream.

As testing by Graeme Campbell & Associates (2019) [Ref. **15**] shows that the tailings are potentially acid generating, the deposition of tailings will be controlled in such a way as to only allow the tailings to be exposed for short periods of time before being covered by subsequent deposition. Thus, it is expected that there would be no acid generated from the tailings bleed or runoff during operations.

11 CONSEQUENCE CATEGORY

The consequence category according to the NSW DSC Guidelines for Tailings Dams [Ref. **10**] considers any acid-forming tailings to have a consequence category of **High C** as a minimum where the receiving environment is rural. The ANCOLD guidelines are not as prescriptive. This rating is also considered realistic on the basis that non-itinerate loss of life is not expected and as the emergency spillway has been designed to discharge into Walkers Creek, downstream of the TSF and it is anticipated that any flow from the spillway would enter Walkers Creek and subsequently Lawsons Creek as would any release during a dam break. Therefore, no buildings/residences are known to be situated in the flow path. It is also noted that the TSF is downstream from the village of Lue, approximately 2.0 km in a direct line from the closest point of the TSF embankment.

Non-itinerate loss of life is not expected. However, a dam-break tailings run-out study would be carried out during detailed design following the receipt of development consent and once the full site-wide water balance has been confirmed.

12 DESIGN CRITERIA

12.1 Introduction

Based on the **High C** consequence category, the accepted fall-back design criteria are as outlined in the sections below.

12.2 Storm Storage

The allowance for the storage of the runoff from a design storm and additional freeboard was estimated using both the ANCOLD (2012) and NSW DSC guidelines as follows:

ANCOLD (2012)

1. Storm Storage over the maximum operating pond:
    a. Minimum Extreme Storage: 1:100 Annual Exceedance Probability (AEP) 72 Hr storm; and
    b. Contingency Freeboard: 0.5 m plus 1:10 AEP wave run-up from wind to spillway invert.

Or

NSW DSC (Guideline DSC3F)

2. Storm Storage over the maximum operating pond:
    a. Flood Storage 1:100 AEP 72 hr storm; and
    b. Operational freeboard of 0.5 m to the crest of the embankment.

   Or
3. Total freeboard with critical duration storm with an AEP of 1:100,000 to the crest of the embankment.

   Or
4. Environmental Freeboard of 1:100 AEP 72 hr storm to spillway invert.

The most stringent of the above design criteria, i.e. ANCOLD (2012) has been chosen for this preliminary design for each stage of the TSF.

The ability to reinstate 1 in 100 AEP 72 hr storm capacity within 7 days of a design storm is also required.

12.3 Emergency Spillway Design

In accordance with the ANCOLD (2012) guidelines, the TSF spillway has been designed for a 1 in 100,000 AEP with an additional allowance for wave run-up.

12.4 Seismic Analysis for Embankment Stability

The following criteria apply:

Operating Basis Earthquake (OBE):
1:1,000 AEP earthquake (ANCOLD)

Maximum Design Earthquake (MDE):
1:10,000 AEP (ANCOLD)
1:1,000 AEP (NSW DSC – DSC3C)

The most stringent of the above design criteria have been chosen for this preliminary design.

During the process of finalising this report, ANCOLD released the updated Guidelines on Design of Dams for Earthquake (2019) [Ref. **16**] and also Revision 1 to the ANCOLD (2012) guidelines [Ref. **17**]. This revision allows the maximum design earthquake for **High C** dams to be reduced to 1:2000 provided a risk assessment is undertaken. This and other changes to the ANCOLD (2012) Tailings Dam guidelines would be adopted in the final design.

12.5 Embankment Stability Factors of Safety

The acceptable factors of safety for stability analysis based on the ANCOLD Guidelines on Tailings Dams [Ref. **7**] and the stress state adopted are presented in **Table 2** below:

TABLE 2

FACTORS OF SAFETY FOR EMBANKMENT STABILITY

Loading Condition	Recommended Minimum for Tailings Dams	Stress State Adopted for Analysis
Long-term Drained	1.5	Effective Strength
Short-term undrained end of construction upstream	1.3	Consolidated Undrained Strength
OBE*	1.1	Consolidated Undrained Strength
MDE*	1.0-1.1	USACE method of 80% of Consolidated Undrained Strength and 50% of earthquake magnitude*

* Where OBE is the Operating Basis Earthquake and MDE is the Maximum Design Earthquake as outlined in Section 11.4 and USACE is the US Army Corps of Engineers. The use of the pseudo-static analysis for the OBE and MDE along with USACE provision for 80% of the undrained strength and 50% of the earthquake load is appropriate for this preliminary analysis given the level of the study and the information available. This would be reanalysed during detailed design after a site-specific seismic study has been undertaken. A detailed seismic deformation analysis would also be completed.

12.6 EPA Liner Guidelines

It is understood from the correspondence forwarded to Department of Planning by the EPA on 12 December 2016 titled "Tailings Dam Liner Policy", that the benchmark position for the EPA with regards to protection from seepage is a prescribed clay liner of minimum 1 m thickness and with a maximum permeability of 1 x 10^{-9} m/sec, at the base of the storage.

13 BEACHING PROFILE DETERMINATION

13.1 Tailings Beach Slope Prediction

The shape and slope of the tailings beach from the deposition point to the decant pond is referred to as the Beaching Profile and is a fundamental part of the estimation of capacity, embankment sizes, and the way tailings are expected to behave.

13.2 Tailings Discharge Rate

The preliminary design is based on the discharge rate of 2 Mtpa with 8000 hrs of discharge each year, allowing for 10% downtime. For beach slope prediction, a further adjustment has been made for some losses with removal of concentrate giving a design discharge flow rate of 246 t/hr.

13.3 Prediction Criteria

The evaluation of beach slopes for the Project has been based on various process input parameters in addition to the Bingham Plastic(BP) rheological parameters which have been obtained from the laboratory testing on the CT and VB tailings sample provided by Bowdens Silver and conducted at the ATCW's laboratory in Melbourne.

The input parameters to beach slope prediction model for a varying number of discharge points are listed in **TABLE 3.**

TABLE 3

INPUT PARAMETERS FOR BEACH SLOPE PREDICTION USING CT AND VB TAILINGS

Case No.	Dry Tonnage[1] (tph)	d85 (μm)	Solids (%)	Rheology (BP Parameters)		No. of Discharge Points	Slurry Density (kg/m^3)	Flow rate Per Discharge Point (L/s)
				Yield stress (Pa)	Plastic Viscosity η (Pa.S)			
PARAMETERS FOR CT TAILINGS								
1	246	200	56	12.7	0.055	1	1,545	79.0
2	246	200	56	12.7	0.055	2	1,545	39.5
3	246	200	56	12.7	0.055	3	1,545	26.3
PARAMETERS FOR VB TAILINGS								
4	246	200	56	14.0	0.048	1	1,545	79.0
5	246	200	56	14.0	0.048	2	1,545	39.5
6	246	200	56	14.0	0.048	3	1,545	26.3

[1]Assuming pumping for 8000 hrs per year.

13.4 Methodology

The beach slopes have been calculated in accordance with the methodology set out in Pirouz et al. (2014) [Ref. **18**]. The beach slope method described in this technical paper is based on the head loss in a self-formed channel (formed by deposited tailings) flowing at its equilibrium slope carrying tailings at a minimum transport velocity (minimum turbulence needed to keep the solids particles within the flow).

The model is founded on the assumption that it is the slope of the self-formed channel that dictates the overall slope of a tailings beach.

13.5 Results of Prediction

All beach slopes exhibit concavity due in part to thickener underflow variability and segregation of the tailings if this occurs. To determine the concavity, the slopes are divided into four parts; the upper section, middle section, lower section and runout. The beach slope predictions of these top three parts are determined at mean minus standard deviation slurry concentration, mean slurry concentration and mean plus standard deviation slurry concentration values. **TABLE 4** summarises the findings of the beach slope prediction calculations for a number of discharge points.

TABLE 4

PREDICTED BEACH PROFILE

Case No.	Predicted Beach Slope (%)		
	Upper	Middle	Lower
USING CT BP PARAMETERS			
1	1.87	1.40	1.05
2	2.63	1.97	1.48
3	3.21	2.41	1.81
USING VB BP PARAMETERS			
4	1.92	1.44	1.08
5	2.71	2.03	1.52
6	3.31	2.48	1.86

13.6 Design Beach Slope

The analysis above shows the sensitivity of the beach slope to discharge rates. It is proposed that there would be three discharge points in operation, but the facility would have one effective discharge stream for the first 5.4 years when the rate of rise is high. However, for the remainder of the LOM, it is assumed that as generally two separate tailings streams would be maintained at any one time and higher beach slopes are achieved. The design slopes adopted for the two periods of time are as presented in **TABLE 5**.

It is noted that the beach slopes are slightly conservative for the down-valley discharge and it is important that these beach slope predictions be revisited during detailed design when there are more samples of a wider variety of expected tailings type available for testing. Rheology testing along with beach slope prediction should also be revisited at the start of deposition once the grind and tailings thickener solids concentration becomes consistent.

TABLE 5

DESIGN BEACH PROFILE

Case No.	Design Beach Slope (%)			
	Upper	Middle	Lower	Run out
Start-up to 5.4 years (10.8 Mt)	1.4	1.0	0.7	0.2
5.4 to 15 years (19.2 Mt)	1.8	1.35	1.0	0.7

14 TAILINGS DEPOSITION STAGING

14.1 Timing

For the preliminary design, three stages of embankment construction and tailings deposition have been considered practical in a 15-year period. The capacity of the first stage was chosen as three years and 6 Mt as a balance between start-up cost and ease of operation. The second stage is more arbitrary but has been selected so that Stage 2 and Stage 3 are approximately similar in raise height and capacity. It is recommended that this be revisited during detailed design when more investigation work for the foundation area, borrow area in the impoundment and the overburden has been undertaken as well more information on tailings production rate, solids content and the tailings properties and scheduling for NAF overburden would be available.

TABLE 6 presents the adopted staging information for the TSF.

TABLE 6

TSF STAGING TIMING

Stage	Incremental Capacity (Mt)	Filling Duration (years)	Preliminary Design Filling Times (years from start of TSF filling)	TSF Construction Timing (Preliminary Design Times)
Stage 1 (Start-up)	6.0	3	Year 1 to Year 3	Prior to Year 3
Stage 2	10.0	5	Year 4 to Year 8	Prior to Year 4
Stage 3	14.0	7	Year 9 to Year 15	Prior to Year 9

14.2 TSF Layouts

Figure **4, Figure 5**, and **Figure 6** show the layouts for the TSF embankment and the tailings when they are at capacity for each stage as well as the inundation of the maximum operating pond size expected during the stage.

14.3 TSF Filling curves

The TSF tailings filling curve is presented in **Figure 7** along with the Stage raising levels.

15 TAILINGS DEPOSITION AND LINER REQUIREMENTS

15.1 General

As the tailings are to be deposited down-valley against the embankment, the rate of rise of tailings would be greatest in the first three years (Stage 1). At its deepest point, approximately 20 m of tailings are to be deposited near the embankment. Because of the rapid rate of rise, consolidation would not be complete at the end of the three years and consequently the permeability would be greater in these years than later when there has been time for further consolidation. Consequently, the impoundment liner has been designed based on the equivalent flux over these three years.

15.2 Liner Requirements

In order to design a clay liner system with equivalence to the EPA 1 m of clay liner with a permeability of 1×10^{-9} m/sec the flux of water through 3 to 20 m of tailings was compared to the flux of water over the EPA liner. The permeability of the tailings was determined by consolidation modelling as provided in **Section 16**.

16 CONSOLIDATION MODELLING FOR EPA PERMEABILITY EVALUATION

16.1 General

Consolidation modelling was undertaken to determine the permeability and density of the tailings during the first three years of deposition. These parameters were subsequently used to provide a reasonable estimate of flux through the tailings and the proposed clay liner below.

The first three years of tailings deposition were modelled as a one-dimensional column using the SV Consolidation software [Ref. **19**].

The model was generated using the material properties, in particular, the Settled Density and Rowe Cell test results from the manufactured Volcanic Breccia tailings sample, presented in the Tailings Testing Consolidation and Permeability, Laboratory Testing Report [Ref. **4**], **TABLE 7** and **Figure** 8.

Rise rates for the consolidation model were developed using an iterative procedure based on the TSF filling model of the predicted relationships between tonnage, beach elevation, tailings volumes and time.

16.2 Initial Conditions

As part of the laboratory testing program [Ref. **4**], Initial Settled Density (ISD) testing and particle density (SG) testing was conducted. ISD results were used to describe the initial conditions in the model. The adopted values used in the model are presented in **TABLE 7**.

TABLE 7

INITIAL CONDITIONS – MATERIAL PROPERTIES

Material Property	Unit	Adopted Value
SG	t/m^3	2.72
ISD Void Ratio (e_0)	-	1.61
ISD Dry Density (ρ_{d0})	t/m^3	1.04

16.3 Material Compressibility and Permeability Functions

The program SV Consolidation [Ref. **19**] couples SV Solid [Ref. **20**] and SV Flux [Ref. **21**]. The individual software packages require separate input properties to define a material in terms of compressibility (SV Solid [Ref. **20**]) and permeability (SV Flux [Ref. **21**]). Both properties are defined by Rowe Cell testing.

Material compressibility and permeability are defined by power functions, as follows:

Material compressibility power function (Somogyi, 1980) $\boldsymbol{e = A \cdot \sigma'^{B}}$ *Equation 1*

Material permeability power function (Somogyi, 1980) $\boldsymbol{k_{sat} = C \cdot e^{D}}$ *Equation 2*

Where:

$e = void\ ratio,$
$\sigma' = effective\ stress\ (kPa),$
$k_{sat} = saturated\ hydraulic\ conductivity\ (m/s), and$
$A, B, C, D = experimental\ curve\ fitting\ parameters\ (refer\ to\ \boldsymbol{Table}\ \boldsymbol{3}).$

The experimental parameters; A, B, C and D are derived from the Rowe cell test by applying power functions to the data. These fits are presented in **Figure 8** and the resulting experimental parameters are summarised in **TABLE 8**.

TABLE 8

EXPERIMENTAL PARAMETERS FOR TAILINGS COMPRESSIBILITY AND PERMEABILITY

Tailings Compressibility		Tailings Permeability	
A	B	C	D
1.3148	-0.128	0.0024	3.2481

16.4 Boundary Conditions

SV Consolidation [**Ref. 19**] allows the user to define different boundary conditions for the tailings compressibility and permeability. SV Solid [**Ref. 20]** boundary conditions control the degree of vertical movement of water allowed in the tailings column. SV Flux **[**Ref. **21**] boundary conditions control the drainage characteristics of the tailings and can apply pressures as required.

Based on the assumption that the clay liner would be less permeable than tailings, the column of tailings has been modelled assuming one-way drainage (upwards) with a 'Zero Flux' boundary condition (SV Flux [**Ref. 21**]) at the base. To replicate the normal operating pond, a 'Constant Pressure Head' boundary condition (SV Flux [**Ref. 21**]) was used to apply a 2 m head of water to the surface of the tailings column.

Similarly, "No Boundary Condition" was applied at the surface of the tailings column in SV Solid [**Ref. 20**], to allow free vertical movement as the tailings consolidate. A "Fixed" boundary condition was applied at the base of the column, so the only consolidation in the tailings is being considered. The applied boundary conditions are summarised in **TABLE 9**.

TABLE 9

TAILINGS SV CONSOLIDATION MODEL – BOUNDARY CONDITIONS

Boundary	Boundary Conditions			
	SV Solid [Ref. 20]	Description	SV Flux [Ref. 21]	Description
Tailings Surface	Free	Allows free movement in the vertical direction	Pressure Head Constant = 2 m	Maintains a constant head of 2 m to replicate the operating pond.
Base of Tailings	Fixed	Does not allow movement in any direction	Flux = 0	No drainage is allowed to occur from the base of the tailings column.

16.5 Rate of Rise

The rate of rise was based on a target tonnage of 2 Mtpa over the TSF impoundment taking into consideration the varying depths of deposition over the impoundment. The outcome of the tonnage balance from the modelling is presented in **TABLE 10**.

TABLE 10

RATE OF RISE – TONNAGE BALANCE

Consolidated Tailings Column Depth (m)	Tailings Column Tonnage (t/m^2)	Beach Area Corresponding to Tailings Column (m^2)	Tailings Tonnage for each Column Depth (t)
23	33.3	10,360	344,990
20	29.1	30,860	898,030
15	22.9	58,100	1,330,490
10	13.7	103,810	1,422,200
5	6.2	324,870	2,014,190
		Total Tonnage	6,009,900
		Tonnage per Year	2,003,300
		Target Tonnage	2,000,000
		Difference	0.16%

As presented in **TABLE 10**, the modelled deposition rates closely reflect the target conditions of 2 Mtpa. Based on this, ATCW is satisfied that the adopted filling rates were suitable for developing the model. The adopted filling relationship with time for the first three years of deposition is provided in **Figure 9.**

16.6 Model Results

The consolidation modelling results are provided in **Figure 10** and **Figure 11.**

Figure 10 provides the dry density and permeability results required for seepage modelling and the density check for design.

Excess pore water pressure results provided in **Figure 11** show that in the first three years of deposition the excess pore water pressure continues to build up as a result of the high rate of rise of the deposited tailings.

16.7 Consolidation during Tailings Deposition

The model shows that the materials consolidate to an average dry density of 1.42 t/m^3 by the end of Stage 1 (i.e. after 3 years). After 1 year and 2 years of deposition, the average dry density in the column is 1.27 t/m^3 and 1.35 t/m^3, respectively.

The approximate rate of filling from the consolidation model is also presented in **Figure 9**. The filling curves presented in **Figure 9** indicate that approximately 7.6 m of consolidation occurs during the first 3 years of filling, resulting in a consolidated depth of 20.6 m.

16.8 Permeability during Tailings Deposition

The initial hydraulic conductivity of the tailings at the time of deposition is approximately 1.3 x 10^{-7} m/sec.

Hydraulic conductivity profiles presented in **Figure 10** indicate that the permeability of the tailings at the base of the column would reduce to approximately 1.0 x 10^{-8} m/sec after 3 years of the deposition. This is a function of the dry density (or conversely, reduced void ratio) of the tailings increasing with depth as the tailings consolidate. Therefore, the permeability of the tailings gradually decreases from the surface of the tailings (1.3 x 10^{-7} m/sec) to the base of the tailings (1.0 x 10^{-8} m/sec).

17 IMPOUNDMENT LINER DESIGN

17.1 General

In order to determine an equivalent liner system, the target flux through the equivalent liner was set to be equal to or less than that achieved by the EPA benchmark liner under 20 m of water with free draining material beneath the liner. This depth has been adopted as 20 m of tailings would be deposited in the first three years and this is when the maximum rate of rise of tailings occurs and consequently, the lowest tailings permeabilities are expected.

17.2 EPA Benchmark Seepage

Based on this EPA benchmark, the seepage rate through the prescribed 1 m clay liner with a permeability of 1 x 10^{-9} m/sec was calculated. A one-dimensional steady-state analysis was undertaken with varying depths of water standing over the prescribed liner, and the seepage flux per square metre of the liner was calculated. It has been assumed that the material below the liner is relatively free draining and has zero head. The calculated unit seepage rates for various heads of water are shown in **TABLE 11**.

TABLE 11

EXPECTED SEEPAGE RATES ($m^3/sec/m^2$)

Thickness of Clay Liner (m)	Head of Water (m)		
	3	6	20
1.0	4.0×10^{-9}	7.0×10^{-9}	2.1×10^{-8}

The resulting seepage rate for a 20 m head of water has been considered as the maximum allowable seepage rate for any proposed alternative liner arrangement.

17.3 Seepage through the tailings and proposed liner

17.3.1 General

Seepage through the proposed liner (including a minor contribution from the placed tailings) has been calculated to enable a comparison with the EPA maximum limit of 2.1×10^{-8} $m^3/sec/m^2$ **(TABLE 11).**

As outlined previously, the first stage of TSF filling is expected to take around 3 years, resulting in a maximum depth of tailings of about 20 m at the embankment.

17.3.2 Tailings and Clay Liner Properties

For the purpose of this additional analysis, the parameters for both the proposed liner and the tailings have been considered in more detail.

As discussed in **Section 16**, based on the recent Rowe Cell laboratory testing on the VB sample and the subsequent consolidation analysis, permeabilities for the various depths of tailings have been derived.

The permeability adopted for the proposed clay liner was based on the results of permeability tests on compacted clay samples. These samples were taken from test pits excavated in the proposed TSF impoundment area [Ref. **1**].

The hydraulic properties adopted for the seepage assessment are summarised in **TABLE** 12**.**

TABLE 12

HYDRAULIC PARAMETERS

Material	Average Saturated Hydraulic Conductivity, k (m/sec)	Source
Foundation Clay - Compacted	5×10^{-10}	Based on in-situ and laboratory testing
Tailings 0-3 m	8×10^{-8}	Consolidation analysis and Rowe Cell test results for VB sample
Tailings 3 -10 m	4×10^{-8}	
Tailings 10 – 20 m	2×10^{-8}	

17.3.3 Liner Seepage Analysis and Results

For the seepage analysis, it has been assumed that the water level in the tailings is at the top of the tailings which is the same water depth as analysed for the EPA benchmark liner. It has also been assumed, conservatively, that there is no water pressure under the liner, i.e. the pressure head on the underside of the liner is zero.

The estimated unit seepage for a range of thicknesses of foundation clay and depth of tailings are summarised below in **TABLE 13**.

TABLE 13

EXPECTED LINER SEEPAGE RATES ($m^3/sec/m^2$)

Thickness of Clay (m)	Depth of Tailings (m)		
	3	6	20
0.45	3.68×10^{-9}	6.37×10^{-9}	1.29×10^{-8}
0.70	2.57×10^{-9}	4.43×10^{-9}	9.92×10^{-9}

Figure 12 presents the estimated seepage rates for the TSF and shows a comparison with the allowable maximum derived from 1 m clay with a permeability of 1×10^{-9} m/sec, the EPA benchmark liner (**TABLE 11**).

17.4 Impoundment Liner Design

It is evident from **Figure 12** that the expected seepage rates from the TSF are lower than the maximum allowable seepage rates and consequently it has been proposed to line the impoundment with 0.45 m of clay in all areas of the storage below the maximum possible water level.

For comparison, the seepage analysis undertaken, as outlined below in **Section 23**, indicates that the seepage with the TSF full is equivalent to a unit flux of 6×10^{-9} m^3/sec per m^2. This is significantly lower than the target unit flux of 2×10^{-8} m^3/sec per m^2. This is because the seepage analysis includes the permeability of the bedrock below the liner.

The flux from the preliminary design is less than both the target seepage rate for 20 m of head of water based on the EPA requirements (**TABLE 11**), the flux expected on the EPA 1 m liner with 6 m of head and the calculated 1D liner seepage with 6 m of tailings and water (TABLE 12**)** or more. The implication of this is that the overall site (including the effects of low permeability rock foundations) is actually less transmissive than the liner as a stand-alone (which implicitly assumes a permeable underlying layer).

18 WATER MANAGEMENT

18.1 General

The capture and control of water flowing into the TSF and water being taken from the TSF have been considered in a site-wide water balance study undertaken by WRM [**Ref. 22**].

The results of the water balance together with the runoff from design storms have provided the total storm storage requirement over the tailings, and hence the spillway invert level for the TSF. The spillway routing for the design storm provides the depth of spillway and hence the crest elevation. These designs have been undertaken in accordance with the criteria outlined in **Section 12** and are presented in **Section 19** and **20.**

18.2 Storm Event Data

Based on the IFD curve analysis provided by BOM closest to the TSF site at Latitude -32.6375 degrees and Longitude 149.8375 degrees, the 1:100 AEP, 72 hr storm has a depth of 211 mm of rainfall.

18.3 TSF Decant Pond Elevation Results

The decant pond is situated on the tailings, and as a consequence, the storage rating curve for the decant pond changes with time. The storage rating curves for the decant pond within the embankment are presented in **Figure 13** for the following times:

- Immediately following construction and no tailings;
- 3 years and storage of 6 Mt of tailings deposition;
- 8 years and storage of 16 Mt of tailings deposition; and
- 15 years and storage of 30 Mt of tailings deposition.

The TSF's decant pond level would fluctuate both seasonally and annually as the decant level is affected by the requirements of the plant, rainfall and evaporation. The floating pontoon would allow water to be returned to the process plant on an as needs basis when the decant pond is deeper than 2 m. However, towards the end of LOM, an alternate pumping arrangement would be introduced to lower the decant pond to a minimum of 1.0 m and hence optimise the embankment height and reduce the water volume needing to be pumped off the TSF during rehabilitation.

With respect to the design of the TSF, the primary use of the water balance results has been to provide water levels in the decant pond. The first of the two design levels have been adopted as follows:

The maximum water level estimated in the water balance at each stage of filling (the 99% non-exceedance water level). These levels are provided in **TABLE 14**.

TABLE 14

DECANT POND ELEVATION PREDICTED BY WATER BALANCE

Design Case	Maximum Decant Pond Elevation (m, AHD)		
	Stage 1	Stage 2	Stage 3
99% probability of non-exceedance for the Stage	598.5	608.6	616.7

The levels provided in **TABLE 14** are the minimum levels acceptable for the spillway invert. Consequently, by adopting a minimum of these levels for the spillway invert at the three stages, there is 99% probability that the spillway will not flow during the LOM.

The second design level is the 50% non-exceedance water level. For the final TSF embankment stage and the elevation of this water level is 615.1 m and this represents the maximum operating expected pond level when the TSF is full.

The water balance **[Ref. 22]** reports a potential maximum decant pond volume of around 1.3 Mm^3 towards the end of Stage 1, 1.4 Mm^3 towards the end of Stage 2 and 1.5 Mm^3 two years before the end of LOM. It should be noted that as the predicted tailings slope are expected to change so does the volume of water stored for a given depth of decant pond as shown in **Figure** 13.

19 STORM STORAGE

19.1 General

The embankment crest and spillway levels have been estimated based on the tailings levels, the decant pond levels, and allowances for stormwater storage in accordance with both the ANCOLD and NSW DSC fall back designs (See **Section 12.2**). All criteria in **Section 12.2** were analysed, and it was found that the ANCOLD guidelines [**Ref. 7**] gave the highest design water levels, providing the most stringent criteria. Consequently, these have been adopted for the final stage design with the basis of this estimate outlined below.

19.2 Design Storm Storage Allowances

Based on the ANCOLD Tailings Dam Guidelines [**Ref. 7]** the storm storage allowances for the final stage are provided in **TABLE 15.**

TABLE 15

DESIGN STORM STORAGE ALLOWANCE

Stage	Design Maximum Operating Pond		1:100 AEP, 72 hr Storm Flood Volume (m^3)	Additional Freeboard (m)	Wave run-up (m)	Spillway Invert Elevation (m, AHD)	Storage Freeboard[2] (m)
	Volume[1] (m^3)	Elevation (m, AHD)					
Stage 3	150,000	615.1	648,000	0.5	0.5	618.2	2.9

[1]The maximum decant pond water level estimated in the water balance has been used as one of the design checks when estimating the storage allowance on the tailings as shown in **TABLE 14.**
[2] Storage freeboard is the freeboard between the expected maximum operating pond and the spillway invert.

During detailed design on the basis of an updated site-wide water balance and beach slopes, using the most stringent criteria which is expected to remain as the ANCOLD guidelines, the timing for raising the embankment will be confirmed as well as the design levels for Stage 3. The spillway inverts adopted for the Stage 1 and Stage 2 are provided in **TABLE 18**.

20 SPILLWAY DESIGN

Based on the ANCOLD Tailings Dam Guidelines **[Ref. 7**] the spillway has been designed for a 1:100,000 AEP critical flow as outlined in **Section 7**. The design assumes very conservatively storage full to spillway level at the start of the rainfall event. The spillway is located at the right/ northern abutment as shown in **Figure 3, Figure 5** and **Figure 6** with a width of 20 m. The topography of this abutment means that a portion of the lower spillway needs to be filled for the next stage and would require careful consideration during detailed design. The spillway is flat initially and falls away steeply down the abutment. The spillway chute runs into a stilling basin to protect the toe of the embankment. It should be noted that the spillways are for emergency only and not expected to flow during the LOM.

The spillway details are provided in **TABLE 16.**

TABLE 16

SPILLWAY DESIGN

Stage	Maximum 1:100,000 AEP critical flow		Spillway Invert	Embankment Crest Elevation
	Discharge (m^3/sec)	Depth (m)	(m, AHD)	(m, AHD)
Stage 1 (Start-up)	62	1.43	599.5	601.5
Stage 2	53	1.30	609.2	611.0
Stage 3	49	1.22	618.2	620.0

The discharge rates for each stage vary with the storage area available for routing the flood through the decant pond.

It should be noted that for mine closure, the spillway would be designed for the Probable Maximum Flood (PMF).

It is understood that rehabilitation would consist of a low flux store and release cover. To achieve the self-shedding profile, final deposition of the tailings would be modified to shift the low point towards the spillway location. The spillway invert would then be lowered to the top of the cover. Any additional material cut from the spillway would be used for rehabilitation works such as flattening the embankment. The spillway would be designed with suitable erosion protection and energy dissipaters for the long term.

21 EMBANKMENT DETAILS

The basic embankment details and the design levels resulting from the storm storage allowance and spillway design are provided in **TABLE 17** and the typical section presented in **Figure 14.**

A toe drain and a seepage collection drain terminating at the seepage collection ponds have been included as shown in the plan **Figure 3** and the section in **Figure 15**. This system would collect the seepage from the TSF impoundment (including natural infiltration higher in the catchment) and runoff from the TSF embankment. This would then be pumped back to the TSF.

TABLE 17

TSF EMBANKMENT DESIGN DETAILS

Stage	Incremental Capacity (Mt)	Tailings Elevation at Decant (m, AHD)	Embankment				
			Maximum Embankment Height (m)	Crest Elevation (m, AHD)	Crest Width (m)	D/S slope (horizontal: vertical)	U/S slope (horizontal: vertical)
Stage 1 - Start-up	6.0	595.0	38	601.5	20	1.5 :1 and 2.5:1	2.25:1
Stage 2	10.0	603.7	47	611.0	20		
Stage 3	14.0	613.1	56	620.0	20	2.5:1	

22 EMBANKMENT DESIGN AND MATERIALS

22.1 General

The down valley discharge of tailings to the TSF means that water is stored on the tailings at the embankment. The TSF is to be designed as a water retaining structure for the life of its operation.

The embankment would comprise a zoned rockfill embankment with a low permeability geomembrane/clay zone on the upstream face and a low permeability (1×10^{-13} m/s) Bituminous Geomembrane (BGM) liner. Curtain grouting to a nominal depth of 40 m of the rock foundations along the upstream toe and partial clay lining of the decant pond area inside the storage would also be included as part of seepage control measures.

22.2 Construction Materials

The construction material proposed for the TSF embankment, as shown in **Figure 14**, are a BGM and low permeability clay/gravelly clay upstream zone, two filter zones, and two general waste rock zones as follows:

- Bituminous Geomembrane (BGM) liner
- Clay/Gravelly Clay/Clayey Gravel- Zone 1
- Granular Filter Gravels and Sands- Zone 2A and Zone 2B
- Slightly weathered to fresh Rockfill- Zone 3A and Zone 3B

It is expected that the Zone 1 low permeability clayey material would be won from within the TSF embankment foundation or in the upstream portion of TSF impoundment and contain at least 30 % clay fines and a maximum particle size of 50 mm. Zone 2A and Zone 2B would be manufactured from crushed and screened rock or imported to the required particle size distribution for primary and secondary filters respectively. Zone 3A and 3B would be a maximum particle size of 0.15 m and 0.3 m respectively supplied from the NAF waste rock available from the main open cut pre-strip and any suitable material won during the excavation of the spillway and access road. If any crushing and screening of rock is required it is expected to be undertaken in the 10 ha area immediately southwest of the TSF embankment.

The upstream face of the TSF embankment would be lined with a BGM liner which would then be tied into a 40 m deep grout curtain using a concrete plinth to reduce any seepage. The site investigation identified variable rock permeability, so it is expected that it would be necessary to grout the foundations to around 40 m depth with primary, secondary and possibly tertiary grouting to achieve a target permeability of around 10^{-7} m/sec.

A summary of the earthfill materials required for the embankment zones is provided in **TABLE 18** below along with the expected source.

TABLE 18

TSF EMBANKMENT EARTH AND ROCKFILL ZONE MATERIALS

Zone	Name	Materials	Placement and Compaction	Source expected
1	Clay	CLAY, sandy CLAY, Clayey SAND (more than 30% passing 75um) and gravelly sandy CLAY	0.2 m (final thickness) horizontal layers, Moisture conditioned to OMC+/- 2% and compact to 98% MDD of standard compaction Maximum particle size 50 mm	Won from embankment foundation excavation and impoundment
2A	Filter 2A	Gravelly SAND and SAND	0.3 m (final layer thickness) horizontal layers, compact with 3 passes of a 10 t vibrating smooth drum roller Not more than 10% non-plastic fines	External supply or crushed, washed and screened from available NAF waste rock to provide suitable grading.
2B	Filter 2B	GRAVEL	0.3 m (final layer thickness) horizontal layers, compact with 3 passes of a 10 t vibrating smooth drum roller	External supply or crushed, washed and screened from available NAF waste rock to provide a suitably graded material.
2C	Toe Drain	GRAVEL	Place loose over geofabric and wrap Maximum particle size 0.05 m	External supply or crushed, washed and screened from available NAF waste rock to provide suitable grading.
3A	Rockfill	Rock fill	0.3 m (final thickness) horizontal layers. Maximum particle size 0.25 m compact with 6 passes of a 10 t vibrating smooth drum roller	Selected from NAF overburden stripping and WRE 4 km from the site.
3B	Rockfill	Rockfill	Doze to 0.6 m (final thickness) horizontal layers. Maximum particle size 0.40 m compact with 6 passes of a 10 t vibrating smooth drum roller	Stage 1 supplied in place by 777 mine trucks. Stage 2 and 3 won from WRE 4 km from embankment.

22.3 Foundations

The foundation treatment for the TSF embankment (Foundation Treatment A), as depicted in **Figure 14** and **Figure 16**, involves stripping the entire footprint to moderately to slightly weathered rock and cleaning the surface of any fine or loose material. The embankment would be grouted to seal up any permeable zones in the embankment foundation, and this would be tied into the BGM liner placed up the upstream face of the embankment.

The foundation treatment within the tailings impoundment is divided into the area where the impoundment is above and below the maximum water level as set out in **Figure 16**. The area where the impoundment is below the maximum water level defines the Foundation Treatment B area.

The areas of the impoundment below the maximum water levels (Foundation Treatment B) are to be lined as follows:

- In areas where an adequate thickness of clay exists, a depth of 0.45 m of compacted clay liner would be provided as follows:
    - Remove topsoil and any clay required for borrow;
    - Remove the top 0.3 m of exposed clay;
    - Rip to a depth of 0.15 m in the natural clay, moisture condition, and compact to 98% of Standard Compaction; and
    - Replace the 300mm of clay in two further layers (2 x 150mm). Moisture condition the clay and compact as previously.

- In areas where clay is a total depth of less than 0.45 m, a total thickness of 0.45m of liner would be provided as follows:
    - Remove topsoil and any unsuitable material;
    - Rip to a depth of 0.15 m in the natural clay (if available), moisture condition, and compact to 98% of Standard Compaction; and
    - Place up to three layers (150 mm thick each) of moisture conditioned clay and compact to 98% of Standard Compaction.

Finally, place protective material over the clay to reduce shrinkage cracks from forming, until covered by tailings.

This procedure may be carried out in stages ahead of the filling of the storage.

The area of the impoundment above the maximum water level would be treated as Foundation Treatment C. This involves ripping, conditioning and compacting the surface stripped of topsoil and any borrow.

TABLE 19 summarises these foundation treatments.

TABLE 19

TSF FOUNDATION TREATMENT

Type	Name	Materials	Treatment	Source expected
A	Embankment Foundation		Strip to moderately weathered rock and clean off any loose material.	
B	Tailings Impoundment Treatment beneath the Maximum Expected Decant Pond	CLAY (CH)	1. Strip topsoil and place in stockpile located southwest of the TSF embankment. 2. Where clay is more than 0.5 m deep: Remove top 0.3 m of exposed clay, rip, moisture condition and compact. Moisture condition and replace clay in 0.15 m layers. Final total thickness of compacted clay 0.45 m. 3. Or place moisture conditioned clay in three 0.15 m layers to a final total (natural clay plus fill) thickness of 0.45 m. Moisture condition all worked clay to OMC+/-2% and compact to 98% of MDD at standard compaction. 4. Cover Clay media to prevent loss of moisture from the clay (e.g., Builders plastic weighted down with spoil)	Materials won from impoundment as required
C	Tailings Impoundment Treatment outside area of Decant Pond		1. Strip topsoil and place in stockpile located southwest of the TSF embankment. 2. Proof roll surface	

23 SEEPAGE

23.1 TSF Foundation Seepage Treatment

The TSF would store both tailings and water adjacent to the embankment. Seepage through the embankment and foundations is expected to report to the depressions of the natural surface located in the valley, along the seepage collection drains towards the toe drains and seepage collection ponds as shown in Figure **15**.

A depth of 40 m of grouting has been proposed to control seepage through the more permeable zones in the bedrock. This would be confirmed during the foundation investigation for detailed design.

23.2 Seepage Modelling and Analysis

Seepage modelling and analysis have been carried out using the finite element computer software package SEEP/W [Ref. **23**] to assess the quantity of seepage under the embankment. For this preliminary work, the steady state case has been analysed when the TSF is full and the decant pond is at the maximum expected operating level with no under drainage beneath the tailings. This was chosen as the greatest long-term seepage is expected when the embankment is at capacity with tailings and the elevation of the decant pond is at its maximum as determined by the water balance. For this assessment, this equates to a decant pond elevation of 615.3 m AHD.

The permeabilities adopted for the clay, subsoil and rock foundations are based on permeability testing of both in-situ and on remoulded samples in the case of the clays. The permeabilities of the rockfill and sand filters are based on generally accepted values.

The permeability of the tailings was based on the VB Sample Rowe Cell test results and consolidation modelling. As the permeability of the tailings is important in the estimation of seepage, particularly in the later years of the LOM and as the tailings tested for this study comprised limited laboratory samples provided by the Project, further testing may be carried out during detailed design. Consequently, the results of the current seepage analysis are subject to confirmation by further work. The permeability coefficients adopted for the analysis based on the results of the geotechnical investigation and typical values for rockfill are set out in **TABLE 20**.

TABLE 20

PERMEABILITY COEFFICIENTS

Material Type (Zone)	Permeability (m/s)
Clay placed/worked beneath the Decant Pond (0.45 m thick)	$5x10^{-10}$
Sub-soil (adopt 0.15 m thick) Clay (Zone 1)	$1x10^{-8}$ $1x10^{-8}$
Sand Filters (Zone 2A/2B)	$1x10^{-4}$
Rockfill (Zone 3A/3B)	$1x10^{-5}$
Tailings (Depth 0-3 m)	$8x10^{-8}$
Tailings (Depth 3-10 m)	$4x10^{-8}$
Tailings (Depth 10-20 m)	$2x10^{-8}$
Tailings (Depth 20-45 m)	$8x10^{-9}$
Rock Foundations (0 - 50m)	$2x10^{-6}$
Rock Foundations (50m – 100m)	$4x10^{-7}$
Bituminous Geomembrane	$1x10^{-13}$
Grout Curtain	$5x10^{-7}$

As outlined above, the aim and assumption are that the grout curtain would reduce the higher permeability associated with the zone of fractured rock. Further geotechnical investigation drilling would be required during detailed design to confirm the extent of the fractured rock for the detailed design of the grout curtain.

23.3 Seepage Analysis Results and Discussion

Based on the limited tailings testing and the preliminary geotechnical investigation, the seepage rate based on a 900 m wide flow beneath the Stage 3 Embankment is estimated at about 2 L/s (160 m^3/day). The graphical output from SEEP/W is shown in **Appendix A.**

This seepage would report to the seepage collection system below the embankment rockfill and be collected in two small lined ponds some 50 m by 50 m by 2 m deep. The seepage water together with stormwater runoff from the toe drain would be pumped back to the TSF. The seepage through the foundations would also be monitored as set out in **Section 25.1.2**. The monitoring would provide data on the presence, depth, and flow direction of groundwater beneath the embankment.

The standpipe piezometers would also be used to measure the quality of the groundwater alongside the measurement of the quality of the water in the decant to ascertain the extent of seepage from the TSF beyond the seepage collection ponds. If seepage from the decant is detected beyond the seepage collection ponds a seepage interception system would be installed downstream of the seepage collection ponds.

As the parameters adopted for the analysis are dependent on limited testing, further permeability assessment would be carried out during detailed design as follows:

- Detailed tailings testing on a range of typical tailings;
- Detailed geotechnical site investigation as follows:
    - Embankment foundations to confirm the variation in stripping and the depth of grouting required; and
    - Impoundment for clay borrow and seepage conditions.

Because of the limited testing at this preliminary stage of design, it is recommended that a contingency is allowed, in the area of the nominal 2 m decant pond, for further impoundment floor treatment (foundation treatment). The contingency recommended is further foundation treatment in the zone within 150 m of the embankment to reduce the permeability of that area and the inclusion of underdrainage over that foundation treatment and beneath the tailings. The requirement or not for this contingency would become apparent during detailed design.

24 STABILITY ANALYSIS

24.1 Overview

A preliminary design was undertaken for embankment stability under both static and seismic loading conditions. Both upstream and downstream failures were considered. The analyses were conducted for each of the embankment stages, i.e., Stage 1, Stage 2 and Stage 3 embankment sections where the embankment is the highest, which is considered critical for stability.

Analyses were carried out using SLOPE/W software [Ref. **24**] and employing a GLE (General Limit Equilibrium) approach, which satisfies both force and moment equilibrium.

24.2 Loading Cases

The following loading cases were considered for assessment of embankment stability:

1) End of Construction: Upstream analyses were undertaken for the case when the construction of each of the stages had been completed.

2) Long-term: Downstream analyses were considered the critical case for the long-term stability for each stage with the phreatic surface at the maximum expected pond level and the tailings at the end of filling.

3) Seismic analysis: Both the MDE and the OBE seismic loadings considered for the downstream embankment slope with the same loading conditions as the long-term. The OBE seismic loading was considered for the upstream slope end of construction case for Stage 1, the critical condition.

24.3 Seismicity Parameters

As outlined in **Section 12.4**, the ANCOLD tailings dam guidelines [Ref. **7**] defines seismic design for the HIGH C Consequence Category Dam, with two levels of earthquake motion as follows:

- Operating Basis Earthquake: the OBE of a 1:1,000 AEP earthquake is for the purposes of evaluating the serviceability of the dam, rather than its safety. It is an earthquake which could reasonably be expected to occur during the life of the dam, and should only result in minor, easily repairable damage. The dam and appurtenant structures should remain functional after the occurrence of earthquake shaking not exceeding the OBE.
- Maximum Design Earthquake: the MDE 1 in 10,000 AEP earthquake would produce the maximum level of ground motion for which the embankment should be designed or analysed. At the time the design was undertaken this was the minimum requirement that the impounding capacity of the embankments be maintained when subjected to that seismic load. As the ANCOLD guidelines have been updated [Ref. **16**] and the minimum requirement reduced, this design may be conservative and would be revisited during detailed design.

No site-specific seismic study has been carried out to date for this preliminary design, but this would be undertaken during detailed design. The earthquake accelerations summarised in the Australian Earthquake Hazard Map [Ref. **25**] were adopted for this study. The adopted horizontal peak ground acceleration (PGA) for both the OBE and MDE events are as presented in **TABLE 21**, where 'g' refers to the acceleration due to gravity.

TABLE 21

SUMMARY OF SEISMIC DESIGN CRITERIA AND EARTHQUAKE PARAMETER

Design Event	OBE	MDE
Average Recurrence Interval	1 in 1,000 years	1 in 10,000 years
Peak Ground Acceleration	0.105	0.30

24.4 Pseudo-Static Stability Methodology

For this preliminary design, pseudo-static analyses were conducted using the specified OBE (serviceability) and MDE (safety) loading cases. MDE analyses were performed using the US Army Corps of Engineers (USACE) screening method [Ref. **26**]. For this level of study, the USACE method is an internationally recognised screening tool for seismic instability, applicable to well-constructed embankments not susceptible to liquefaction.

The USACE method recommends the use of a seismic coefficient equal to one-half of the peak ground acceleration (PGA) using undrained strength for cohesive materials and drained conditions for free draining granular materials, with a 20 percent strength reduction to allow for strain weakening during the earthquake loading.

The design objective was for tolerable predicted deformations under the MDE (safety) criteria. This determination was based upon USACE Screening resultant Factors of Safety (FS) being greater than 1.0, where a USACE screening FS of 1.0 indicates negligible deformations of less than 1 m along the nominal failure "plane" [Ref. **26**].

OBE pseudo-static analyses were conducted using conventional, peak strength properties and no reduction factor on the earthquake acceleration, in order to assess the serviceability of the embankments when subjected to an earthquake load which could reasonably be expected during their operational lives. In these analyses, an FS greater than 1.0 indicates that there are theoretically no moments during shaking in which deformations occur.

24.5 Material Properties

The soil and rock properties adopted for the analysis were based on the results of the geotechnical investigation [**Ref. 1]**, published literature, established correlations, and previous experience. The properties used in the stability analysis are summarised in **TABLE 22**.

Zone 1 Clay

No triaxial test has yet been carried out to evaluate the shear strength of Zone 1 clay. Hence, a preliminary estimate of shear strength for clay has been based on the soil index parameters particle size and plasticity.

The undrained shear strength for the clay has conservatively been derived from the curves relating remoulded undrained shear strength to liquidity index established by Skempton and Northey, 1952 **[**Ref. **27]**.

The drained shear strength was estimated from the curves relating to friction angle, liquid limit and clay size fraction established by Stark and Eid **[**Ref. **28]**.

Zone 2A/2B Filters

Significant differences between drained and undrained strength response are not expected for the compacted granular filter materials, nor are they considered susceptible to liquefaction. Consequently, drained strength parameters have been used for Zone 2 filter materials in all stability analyses.

The drained friction angle of 30º for the filters has been conservatively adopted from numerous data sources such as **[**Ref. **29]**, which correlate test results for loose to dense, angular gravels and sands.

Zone 3A/3B Rockfill

Because of the large particle sizes and it is expected essentially well-graded nature of the Zone 3 rockfill materials, significant differences between drained and undrained strength response are not expected.

With regard to the shear strength of the rockfill, Leps (1970) **[**Ref. **30]** has shown that the shear strength, as expressed by its friction angle, varies noticeably as the function of the effective normal stress. Leps indicates that the lower bound friction angle for rockfill varies from 36^0 – 50^0 for the effective normal stress rage of 10 – 1000 kPa. Conservatively a friction angle of 38^0 has been adopted for Zone 3A and 3B.

TABLE 22

ADOPTED SOIL/ROCK STRENGTH PARAMETERS FOR STABILITY ANALYSES

Zone	Material	Unit Weight γ (kN/m^3)	Shear Strength			
			Drained Parameters		Undrained Parameters	
			c' (kPa)	Φ' (degrees)	Su (kPa)	Φ (degrees)
1	Compacted Clay	19	2	28	50	-
2A/2B	Sand Filters	18	0	30	0	30
3A/3B	Rockfill	22	0	38	0	38
Tailings	-	13.2			Strength Ratio (Su/σ') = 0.25	

24.6 Phreatic Surface

It should be noted for the stability analysis it has been assumed that the BGM liner on the upstream surface has a leak causing a phreatic surface to develop in Zone 1. Subsequently, the position of the phreatic surface adopted for the analysis has been estimated on the basis that the embankment Zones 2 and 3 sand and rockfill is relatively free draining. Consequently, the phreatic surface falls sharply away from the active pond level and runs along close to the bedrock surface.

24.7 Stability Analysis Results

24.7.1 Design Criteria

As outlined in **Section 12.5**, for the purposes of assessing satisfactory performance, the minimum acceptable factor of safety (FS) for the various loading cases are as follows:

- End of Construction FS≥ 1.3
- Static loading for maximum operating pond FS≥ 1.5
- Seismic Loading OBE for serviceability FS≥ 1.1
- Seismic Loading MDE for safety FS≥ 1.0

24.7.2 Results of Analysis

A summary of the stability analysis results for the TSF static and seismic stability analyses are presented in **TABLE 23.** The figure numbers in the table refer to the graphical output from SLOPE/W, which are presented in **Appendix B**.

As the embankment crest is 20 m in width, the non-trivial case for failure of the downstream slope was considered to be when over a quarter of the downstream embankment crest would be displaced. However, as these resulting failure surfaces were shallow, a global downstream failure surface was also analysed where at least 50% of the embankment crest and a significant proportion of the downstream face were within the failure zone.

TABLE 23

SUMMARY OF STABILITY ANALYSES

Load Case	Failure Type	Factor of Safety	Allowable Factor of Safety	Figure
Stage 1- Crest Elevation 601.5 m				
End of Construction	Upstream	1.63	1.3	**B1.1**
Seismic OBE (pseudo-static analysis)	Upstream	1.24	1.1	**B1.2**
Long-term Static – End of Filling	Downstream	1.78	1.5	**B1.3**
Long-term Static – End of Filling	Global Downstream	1.99	1.5	**B1.4**
Seismic OBE (pseudo-static analysis)	Downstream	1.38	1.1	**B1.5**
Seismic OBE (pseudo-static analysis)	Global Downstream	1.51	1.1	**B1.6**
Seismic MDE - (USACE Method)	Downstream	1.02	1.0	**B1.7**
Stage 2- Crest Elevation 611.0 m				
End of Construction	Upstream	1.96	1.3	**B2.1**
Long-term Static – End of Filling	Downstream	1.78	1.5	**B2.2**
Long-term Static – End of Filling	Global Downstream	1.98	1.5	**B2.3**
Seismic OBE (pseudo-static analysis)	Downstream	1.38	1.1	**B2.4**
Seismic OBE (pseudo-static analysis)	Global Downstream	1.50	1.1	**B2.5**
Seismic MDE - (USACE Method)	Downstream	1.01	1.0	**B2.6**
Stage 3 - Crest Elevation 620.0 m				
End of Construction	Upstream	1.39	1.3	**B3.1**
Long-term Static – End of Filling	Downstream	1.99	1.5	**B3.2**
Seismic OBE (pseudo-static analysis)	Downstream	1.51	1.1	**B3.5**
Seismic MDE - (USACE Method)	Downstream	1.03	1.0	**B3.6**

The results presented in **TABLE 23** indicate that all stages of the Embankment are considered to have adequate FS against static and seismic failures.

25 SURVEILLANCE AND MONITORING

25.1.1 TSF Performance Monitoring

Monitoring the TSF operation would be undertaken to collect the data necessary to evaluate the performance of the TSF with respect to the original design expectations. The data would form the basis of annual surveillance audits, would be used to assess and instigate required maintenance programs, would be used in the detailed design of subsequent stage raises and calibration of the site water balance model.

Some of the key monitoring items would include the following:

- Routine reconciliation of tailings discharge tonnages and solids concentrations;
- Routine monitoring of tailings beach head and beach toe levels;
- Routine monitoring of water levels and process plant return water rates; and
- Annual field evaluation of tailings beach density and shear strength profiles.

25.1.2 Monitoring – Groundwater and Embankment Settlement

A system of vibrating wire and standpipe piezometers would be installed as set out in Figure **17** and **Figure 18**, upstream and downstream of the foundation grouting, beneath the embankment, at the toe of the embankment and downstream of the seepage collection ponds. It is proposed that these would be read on a weekly basis.

As outlined in **Section 23.3**, the purpose of these bores would be to provide data on the presence, depth, and flow direction of groundwater beneath the embankment. Water quality testing of the water in the standpipe piezometers and the decant pond would be undertaken on a weekly basis to ascertain the extent of seepage from the TSF beyond the seepage collection ponds.

Survey monuments would be installed on the crest of Stage 1, 2 and 3 TSF embankments to monitor settlement of the fill materials. The monuments would be surveyed on a monthly basis.

25.1.3 Surveillance

Surveillance requirements for the TSF would involve routine daily and weekly inspections, as well as mandatory annual audits. The focus of such surveillance would be as follows:

<u>Daily Inspections</u> –

Focus on operational issues to do with the TSF, including inspections of the tailings and return water pipelines, tailings discharge point management, decant pond location and decant and return water system operation as well as seepage collection ponds.

<u>Weekly Inspections</u> –

Focus on issues that may develop over time and may impact on the safety of the TSF or the environment. These include detailed inspections of the TSF Embankment, all associated structures, tailings beach development and decant pond level, and surveillance of all monitoring installations.

<u>2 Yearly and Annual Audits</u> –

These are conducted by a Dams Engineer and focus on the identification of deficiencies by visual examination of the embankments and all appurtenant structures, as well as a review of all surveillance and monitoring data. Every second year, commencing at the end of the first year of filling, the annual audit is replaced by a comprehensive audit. The comprehensive audit is essentially an annual audit with the addition of the operation of all TSF related equipment for the auditor to observe.

26 DESIGN FOR CLOSURE

The design for closure has being prepared as a separate study (Advisian (2020) [Ref. **31**]). Generally, the rehabilitation would likely consist of a low flux store and release cover over the 1.0 km^2 of tailings. Once in place, the cover would be revegetated. To achieve a self-shedding profile, final deposition of the tailings would be modified to shift the low point towards the spillway location.

At the end of LOM, any additional decant water on the tailings would be pumped back to the main open cut pit. For closure, the spillway invert would be lowered to the top of the cover. Any additional material cut from the spillway would be used for rehabilitation works such as flattening the embankment. The spillway would be designed for the Probable Maximum Flood (PMF), with suitable erosion protection and energy dissipaters. Once tailings deposition ceases and the decant water is removed, it is expected that the quantity of seepage would reduce, and the quality would improve. Seepage would be pumped back to a small lined pond on the TSF and then to the main open cut pit until the quality has improved.

It is proposed after the construction of the third raise that the TSF embankment would be flattened on the downstream slope and revegetated to reduce the potential for erosion and the visual impact of the embankment.

27 QUANTITIES

Summary schedules of estimated quantities for the three stages of the TSF construction are presented in **Appendix C**. This schedule includes the construction of the embankment and does not include any pumps or pipes for tailings delivery or water return.

It should also be noted that some major items within the schedule may change during detailed design when more certainty is provided for the following:

- Tailings properties and thickener arrangements to confirm:
    - Discharge solids content
    - Initial settled density
    - Final density
    - Permeability

- Site-wide water balance to confirm the maximum operating pond expected.

Both the tailings properties and the site wide water balance have a significant impact on the embankment crest and spillway invert levels as well as the potential foundation treatment and underdrainage beneath the area of the decant pond.

A summary of quantities relating to the civil earthworks construction is presented in **TABLE** 24.

TABLE 24

SUMMARY OF EMBANKMENT EARTHWORK AND GEOMEMBRANE QUANTITIES

Stage	Clay Zone 1 (m^3)	Filters (m^3)		Rockfill (m^3)		Total Filter and Rockfill (m^3)	Bituminous Geomembrane (BGM) liner (m^2)
		Zone 2A	Zone 2B	Zone 3A	Zone 3B		
Stage 1	134,000	29,000	31,000	180,000	689,000	929,000	44,000
Stage 2	78,000	16,000	16,000	117,000	833,000	982,000	27,000
Stage 3	78,000	16,000	16,000	109,000	1,254,000	1,395,000	28,000
Total	**290,000**	**61,000**	**63,000**	**406,000**	**2,776,000**	**3,306,000**	**99,000**

28 SUMMARY AND CONCLUSIONS

The preliminary TSF design for 30 Mt of tailings consists of a robust downstream rock and clay fill embankment founded on rock. The TSF embankment would be constructed in three stages. The upstream face of the embankment is covered with a bituminous geomembrane liner connected to a 40 m deep grout curtain to reduce seepage under the TSF embankment. Seepage control is augmented with a 0.45 m clay liner on the impoundment floor to the level of the spillway as well as a seepage collection system beneath the embankment. This seepage collection system discharges to into seepage collection ponds at the downstream toe of the embankment. Any water in the seepage collection pond would then be pumped back to the TSF impoundment.

Water in the decant pond on the tailings is returned to the plant for reuse using a floating pontoon pump. An emergency spillway is provided at the TSF abutment for each stage of construction.

The preliminary design also includes performance monitoring which entails the installation of instrumentation to monitor groundwater pressures and levels, groundwater quality, seepage volumes and embankment settlements, along with the routine recording of tailings discharge tonnages, decant pond levels, and return water volumes.

29 RECOMMENDATIONS

The following work would need to be undertaken during detailed design:

- Additional Tailings testing;
- Detailed site investigation including borrow investigation for Zone 1, Zone 2 and Zone 3 material;
- Site-specific Seismic Risk study;
- Site-wide water balance;
- Tailings deposition and staging;
- Update Flood storage requirements;
- Dam break analysis;
- Seepage analysis;
- Stability analysis;
- Foundation preparation design;
- Grouting design
- Seepage collection design;
- Underdrainage design (if required); and
- Water recovery system design.

30 CLOSURE

Your attention is drawn to the “Conditions of Report” which appear after the document and history page of this report.

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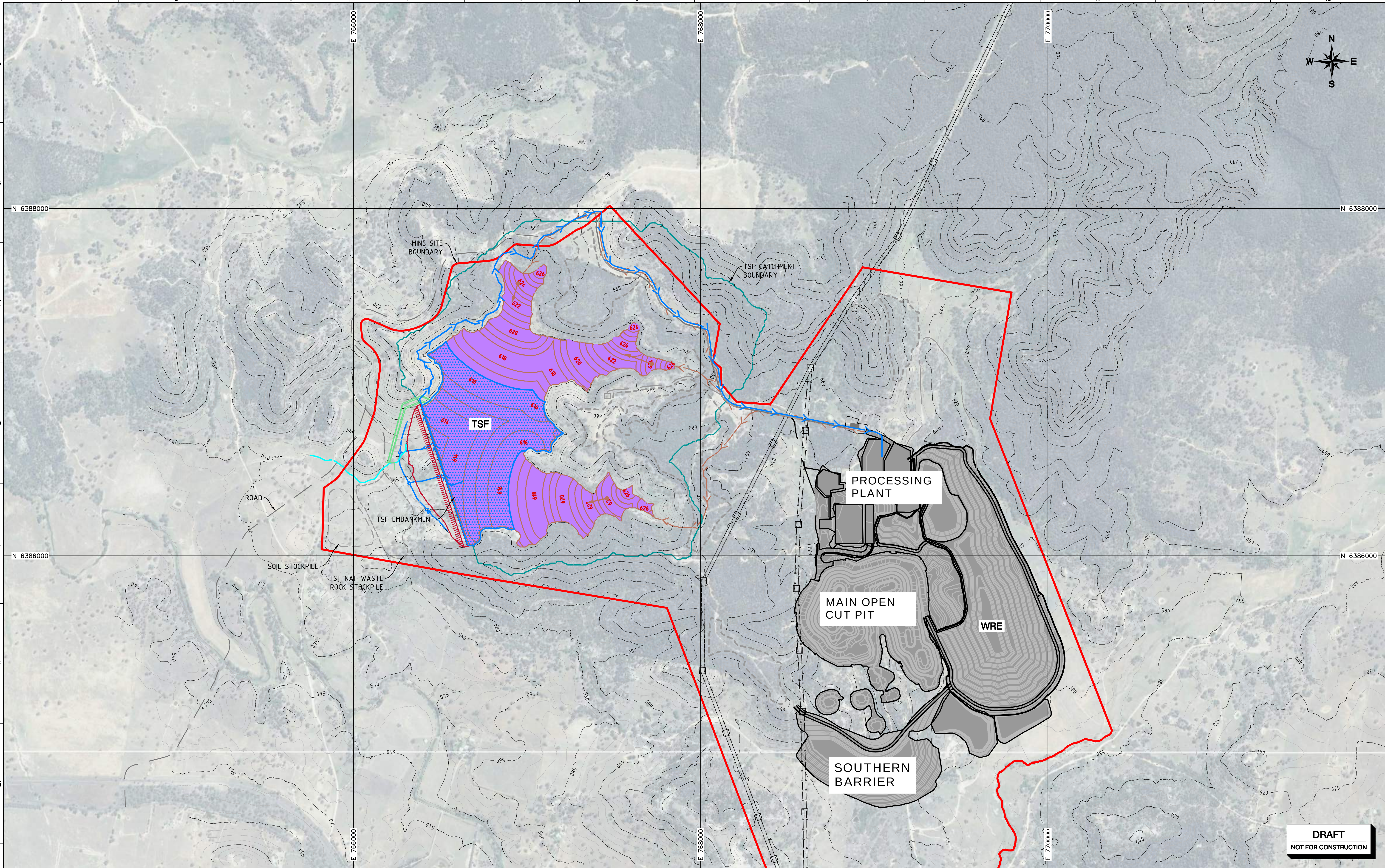
FIGURES



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TSF PRELIMINARY DESIGN
SITE PLAN

DWG. No.

FIGURE 1

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SHEET 1 OF 1

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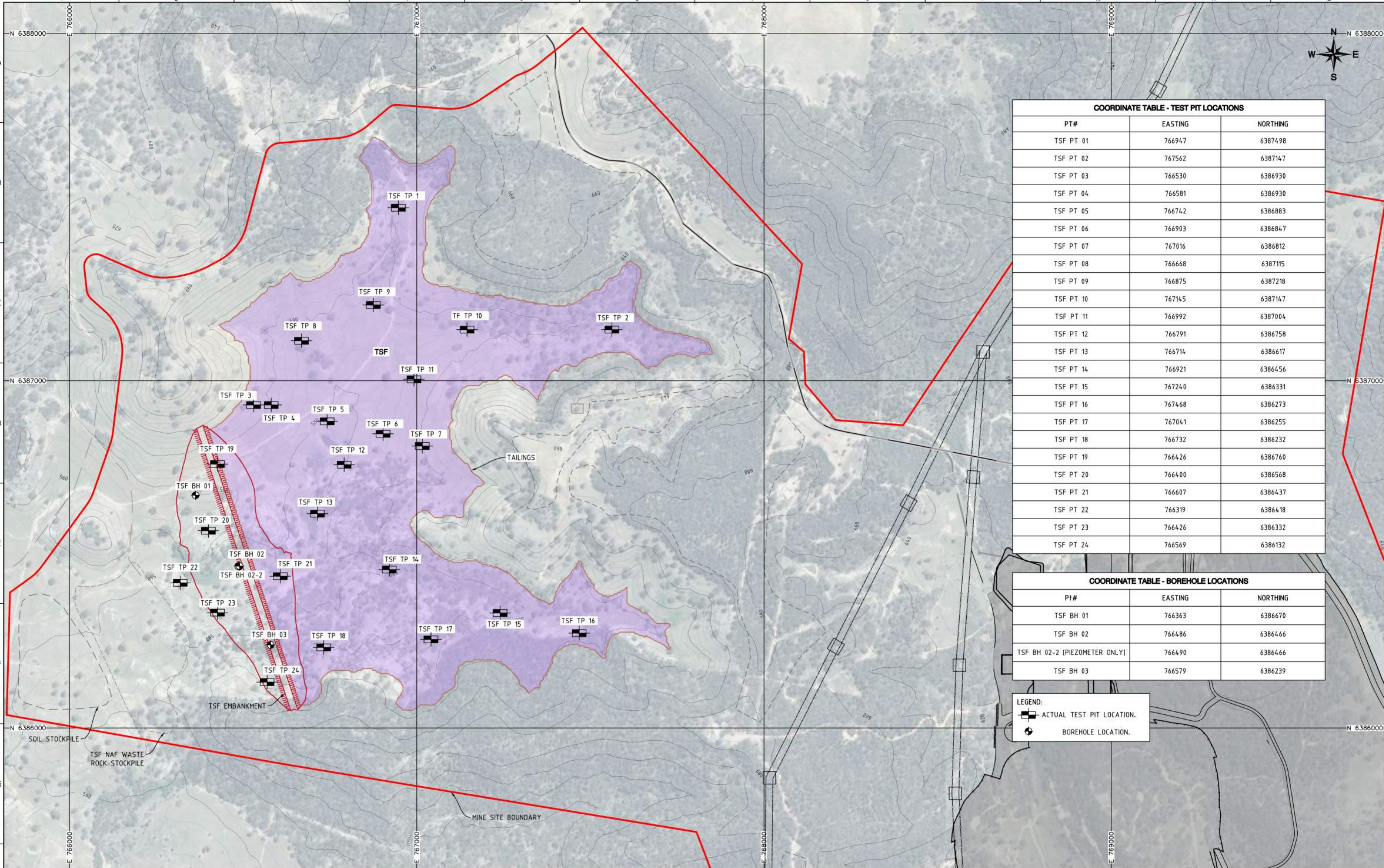

COORDINATE TABLE - TEST PIT LOCATIONS

PT#	EASTING	NORTHING
TSF PT 01	766947	6387498
TSF PT 02	767562	6387147
TSF PT 03	766530	6386930
TSF PT 04	766581	6386930
TSF PT 05	766742	6386883
TSF PT 06	766903	6386847
TSF PT 07	767016	6386812
TSF PT 08	766668	6387115
TSF PT 09	766875	6387218
TSF PT 10	767145	6387147
TSF PT 11	766992	6387004
TSF PT 12	766791	6386758
TSF PT 13	766714	6386617
TSF PT 14	766921	6386456
TSF PT 15	767240	6386331
TSF PT 16	767468	6386273
TSF PT 17	767041	6386255
TSF PT 18	766732	6386232
TSF PT 19	766426	6386760
TSF PT 20	766400	6386568
TSF PT 21	766607	6386437
TSF PT 22	766319	6386418
TSF PT 23	766426	6386332
TSF PT 24	766569	6386132

COORDINATE TABLE - BOREHOLE LOCATIONS

Pt#	EASTING	NORTHING
TSF BH 01	766363	6386670
TSF BH 02	766486	6386466
TSF BH 02-2 (PIEZOMETER ONLY)	766490	6386466
TSF BH 03	766579	6386239

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B					
A	FROM 116217.01 R02				

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FIGURE 2

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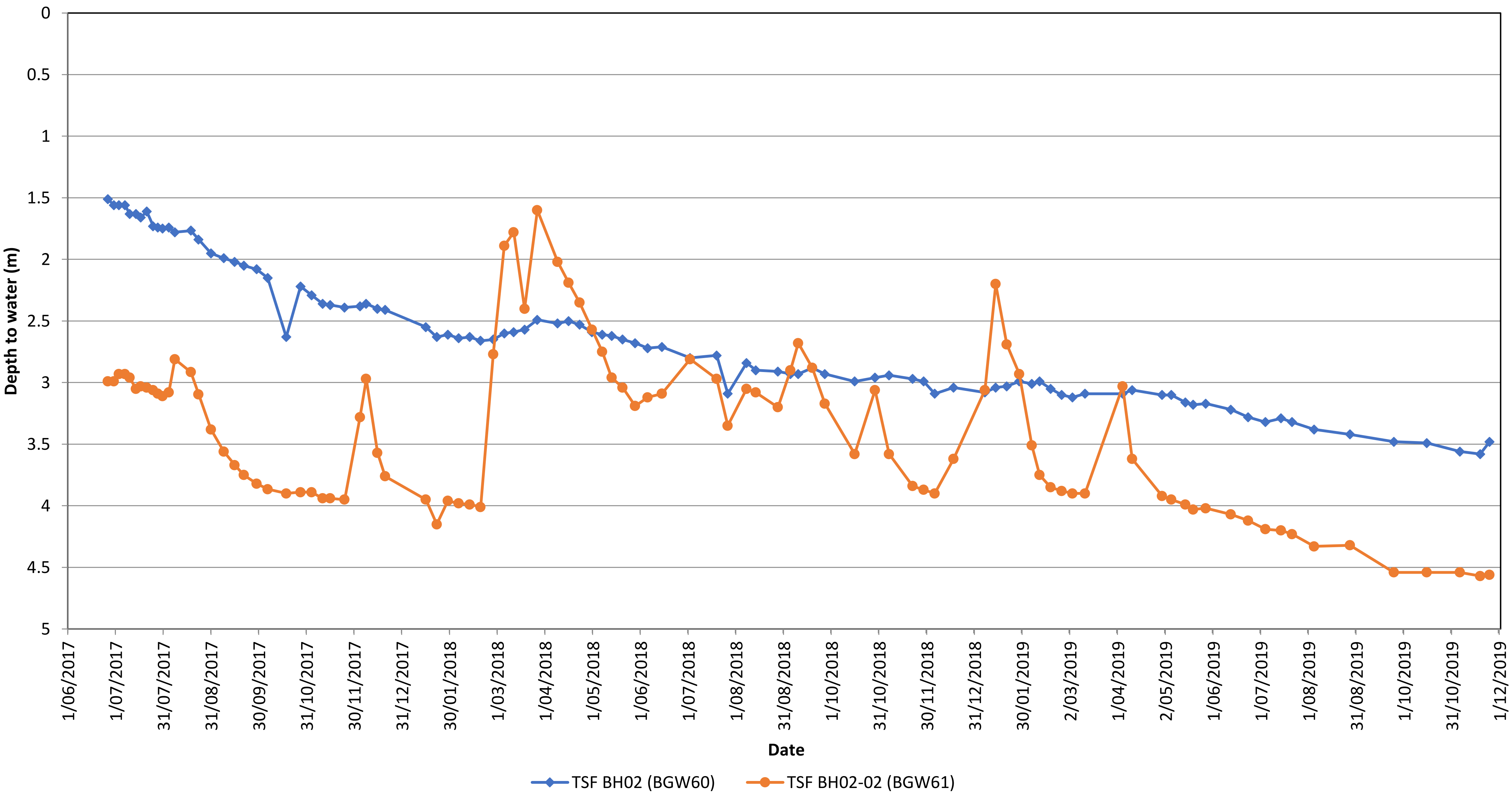

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Ground Water Monitoring at TSF

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FIGURE 3

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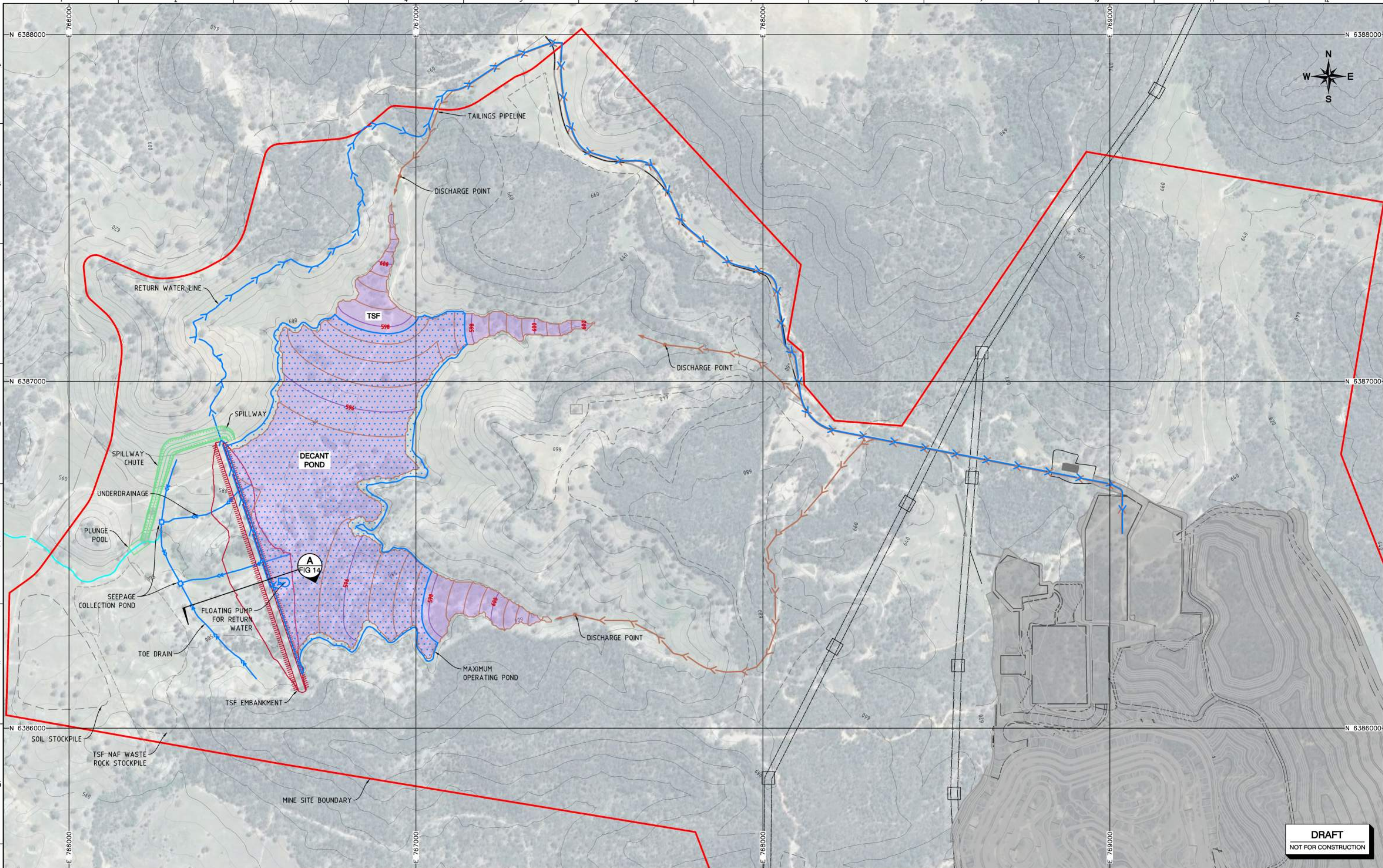

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A	ISSUED	07.05.18	AC	HJW	

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STAGE 1 PLAN

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FIGURE 4

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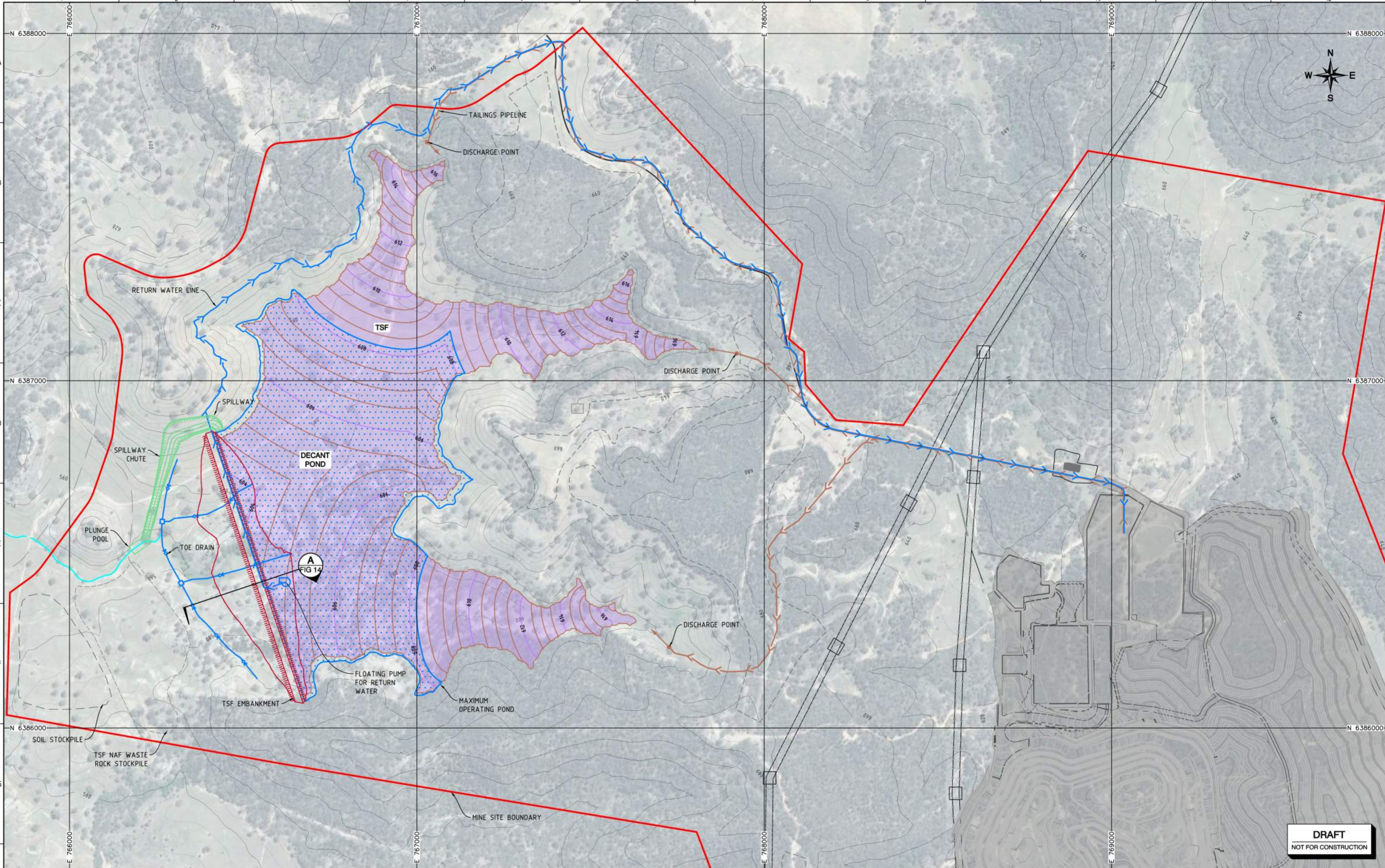

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FIGURE 5

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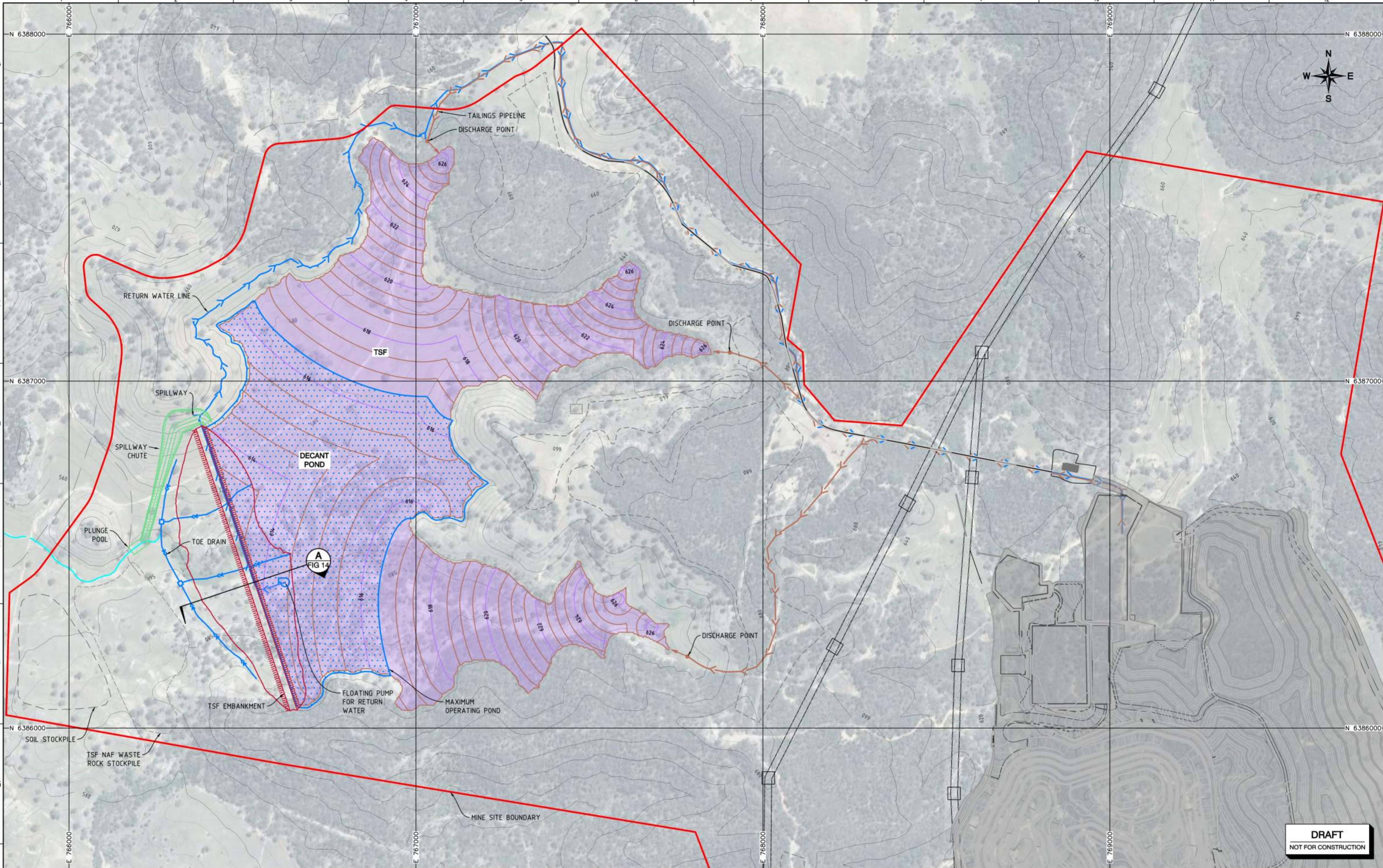

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STAGE 3 PLAN

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FIGURE 6

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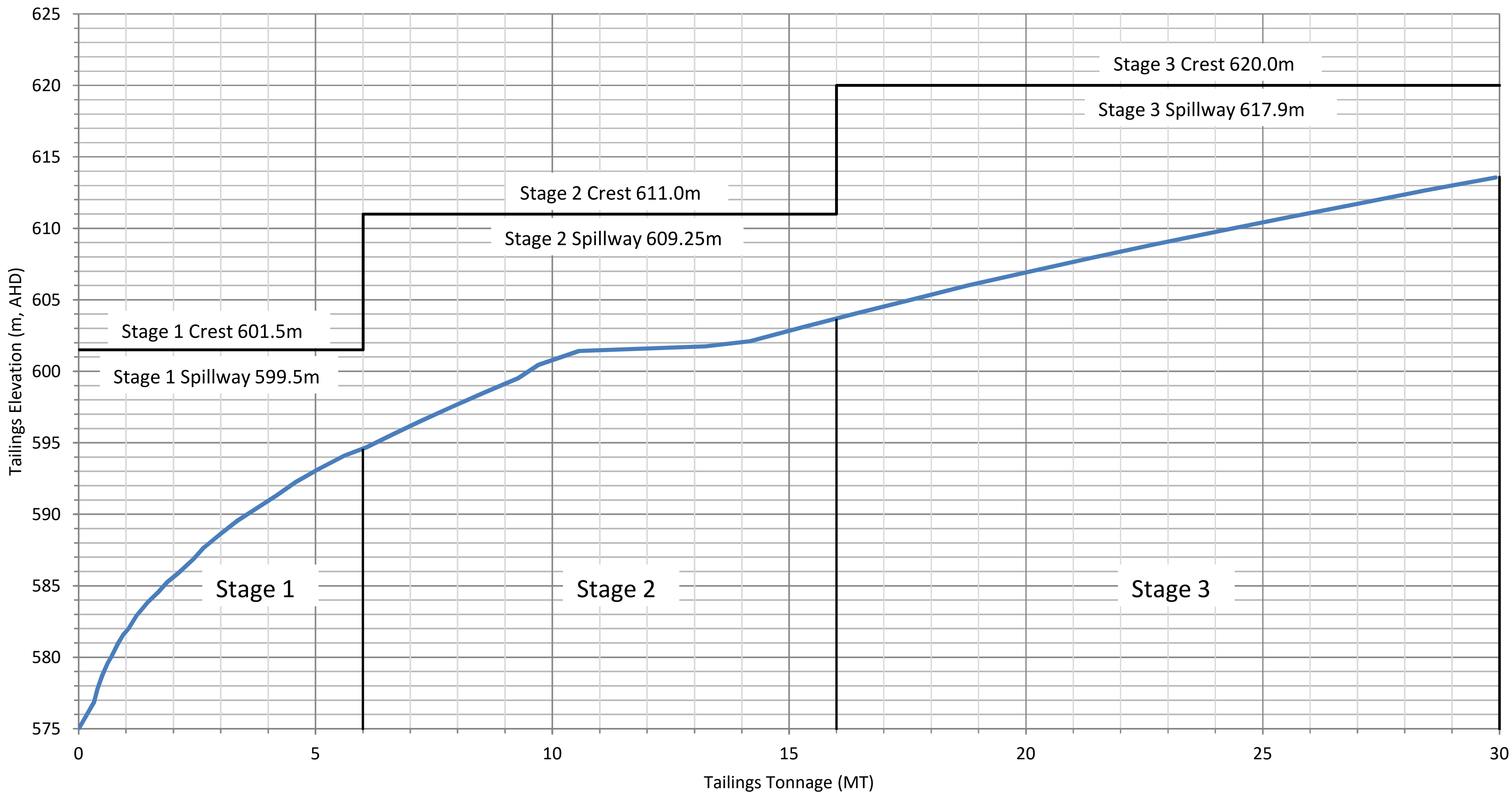

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TSF Tailings Filling Curve

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FIGURE 7

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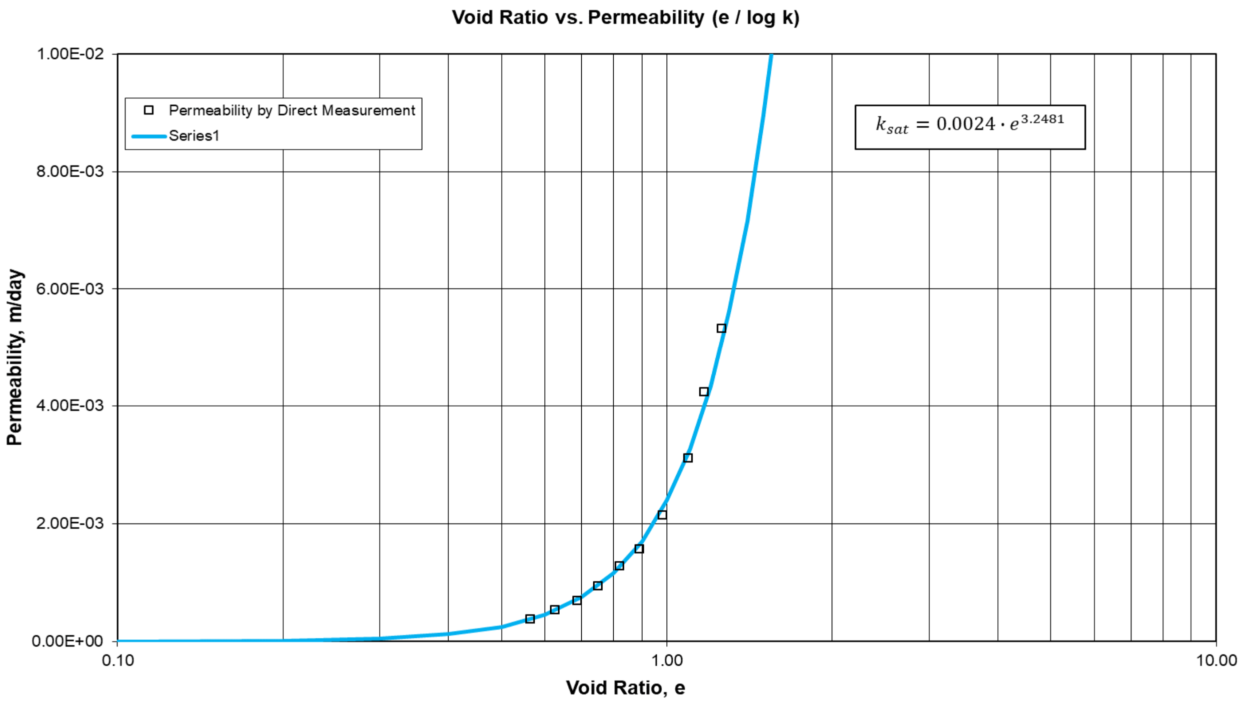

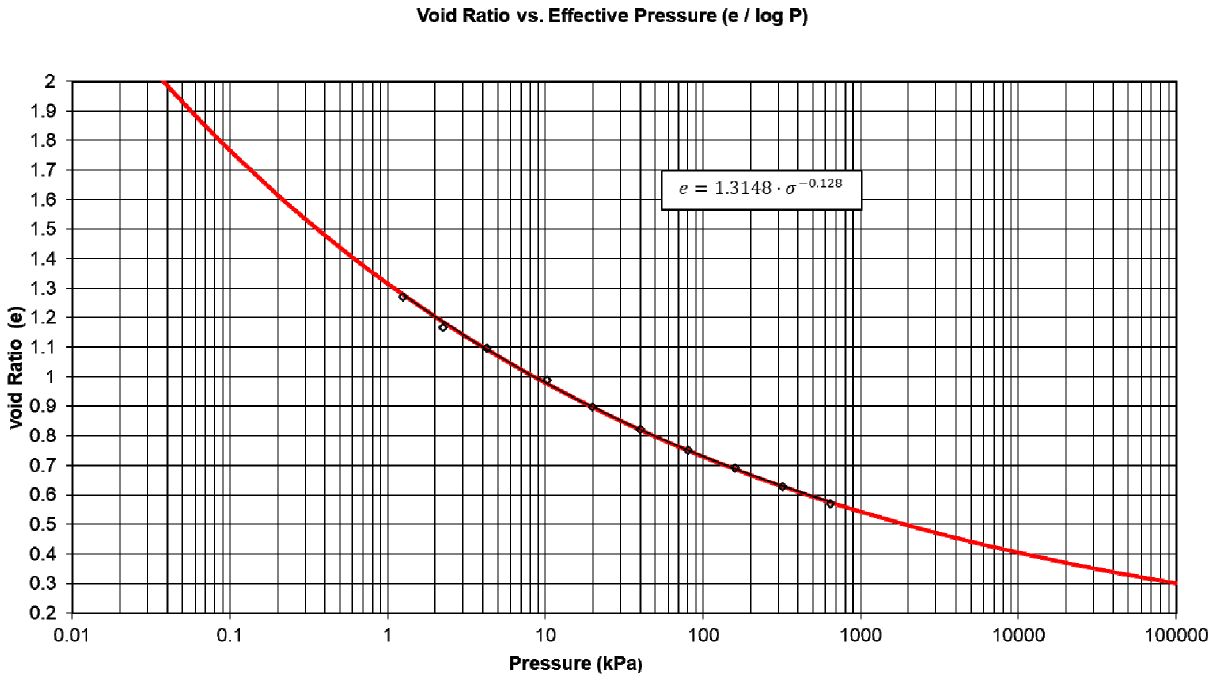

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ATC Williams www.atcwilliams.com.au	**Job No:** 116217.08 **Date:** 28/08/2019 **FIGURE 8**

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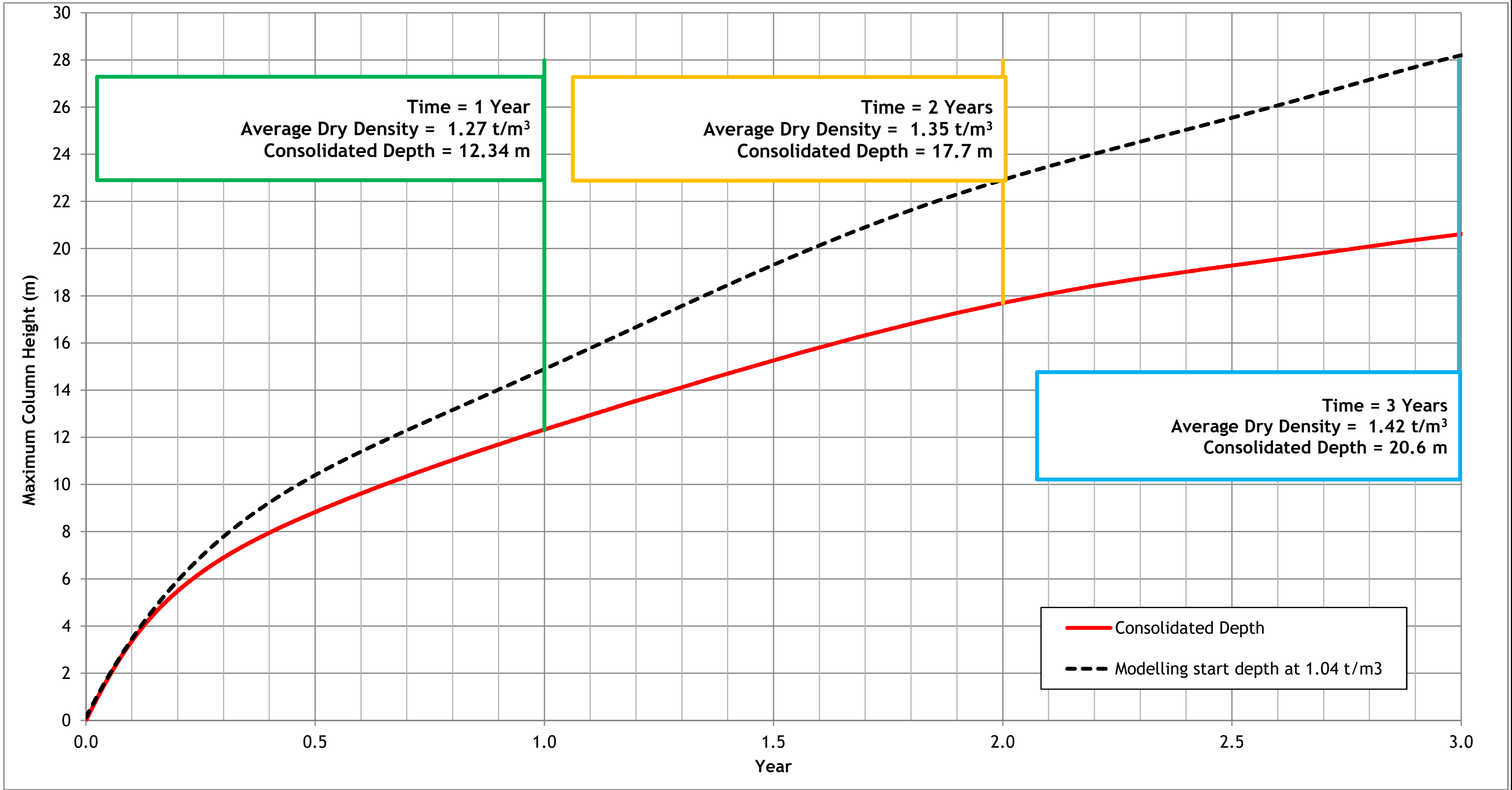

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Consolidation Modelling - Tailings Depth Versus Time

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FIGURE 9

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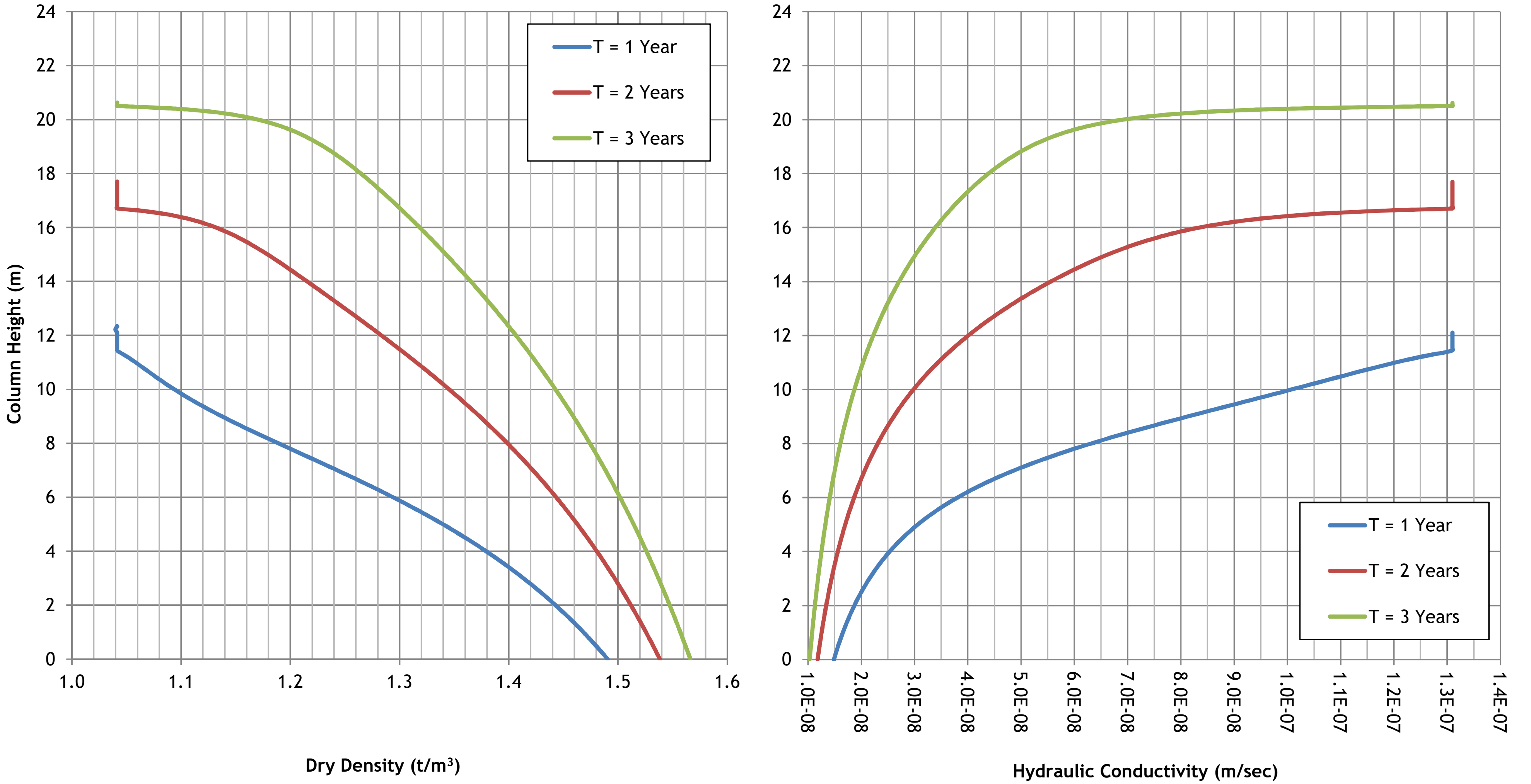

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Consolidation Modelling - Dry Density and Permeability Profiles

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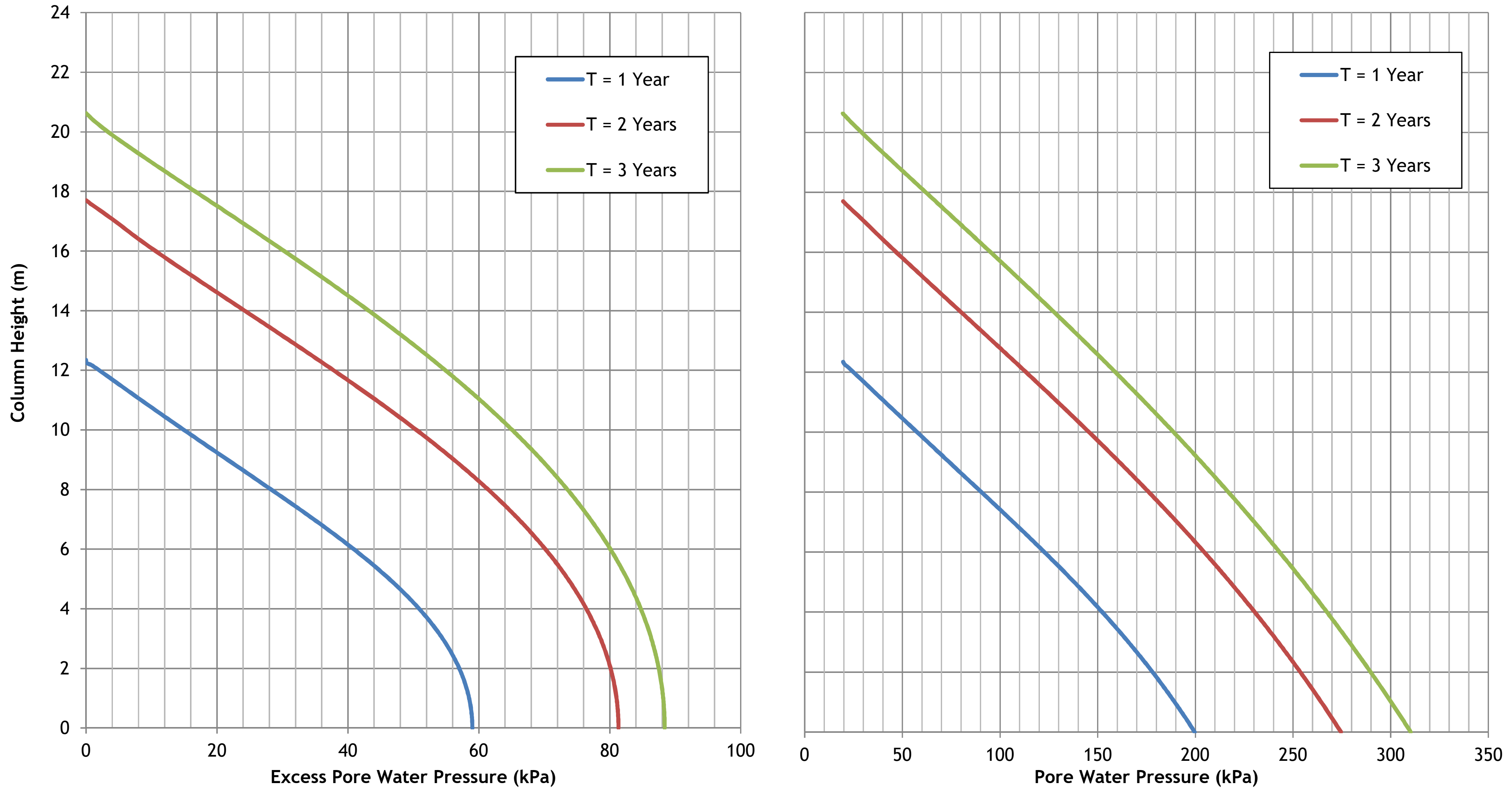

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Consolidation Modelling - Excess and Total Pore Water Pressure Profiles

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FIGURE 11

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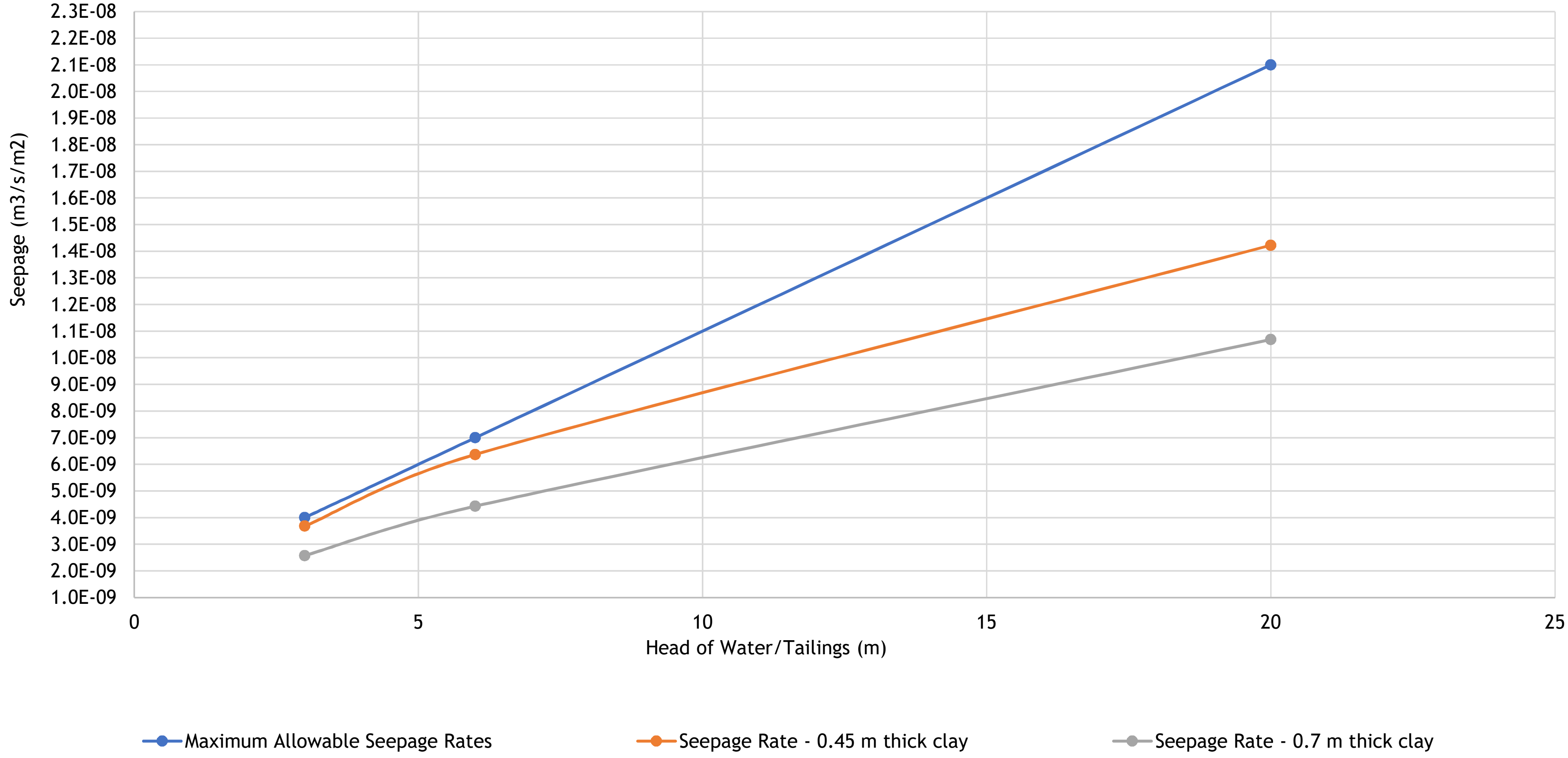

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TSF Preliminary Design

Expected Impoundment Liner Seepage Rates

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FIGURE 12

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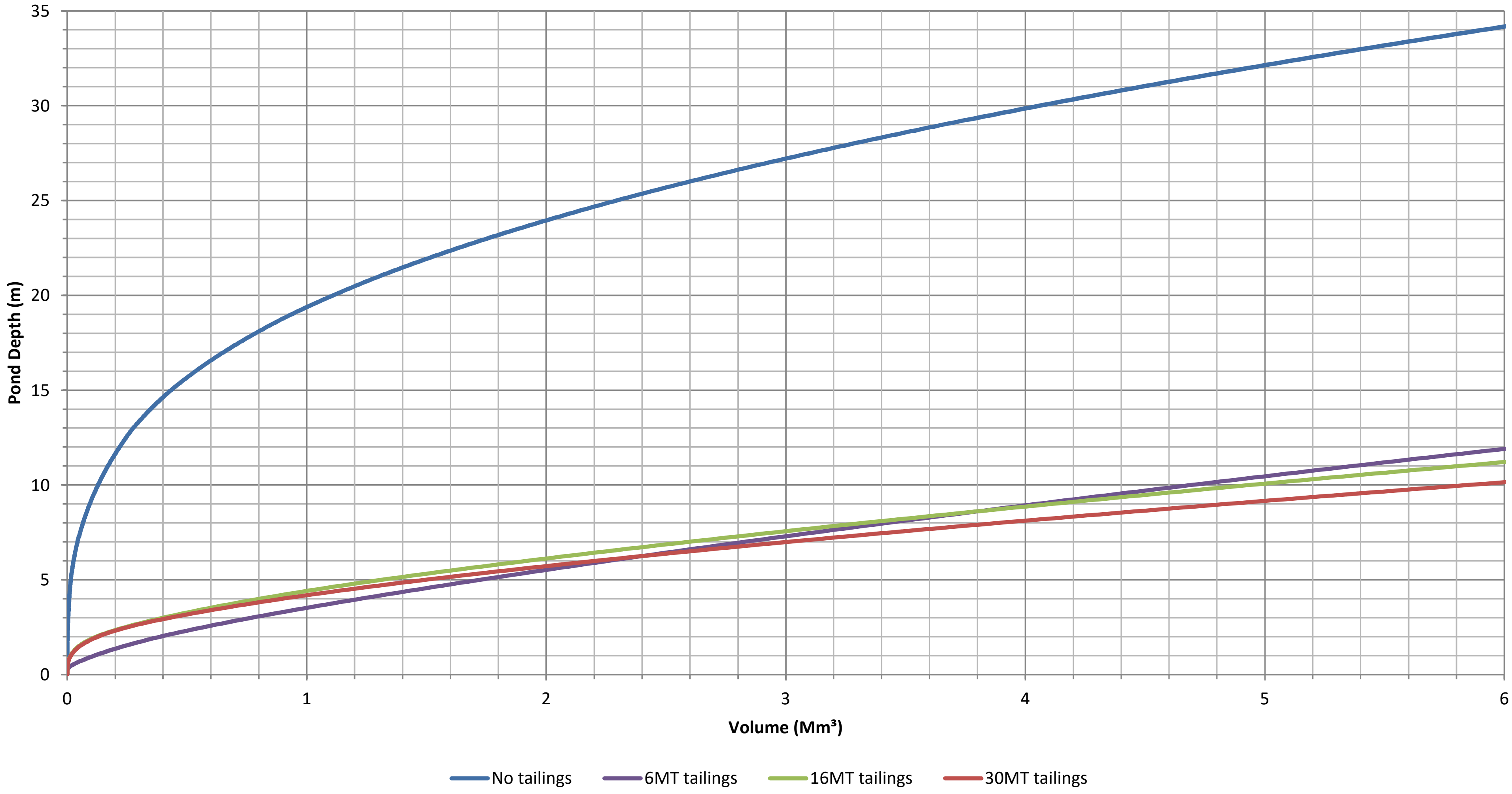

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TSF Decant Pond Storage Capacity

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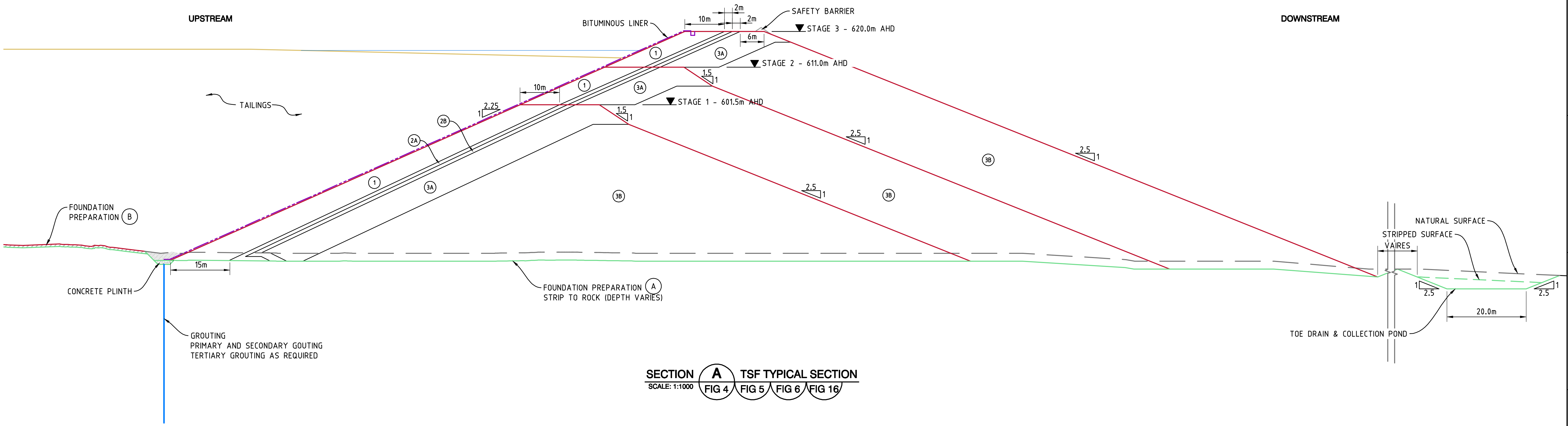
FIGURE 13

K:\Projects\116\116217 Bowdens Silver Project, Bowdens Silver Pty Limited\09 Feasibility Update\Data and Calcs\Figure update for Rev 3 Preliminary design report\TSF Site 2 (2018) - Storage profile (Detailed).xlsx Fig14 - Pond Rating curves



SECTION A TSF TYPICAL SECTION
SCALE: 1:1000
FIG 4 FIG 5 FIG 6 FIG 16

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TSF PRELIMINARY DESIGN
TYPICAL EMBANKMENT SECTION

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FIGURE 14

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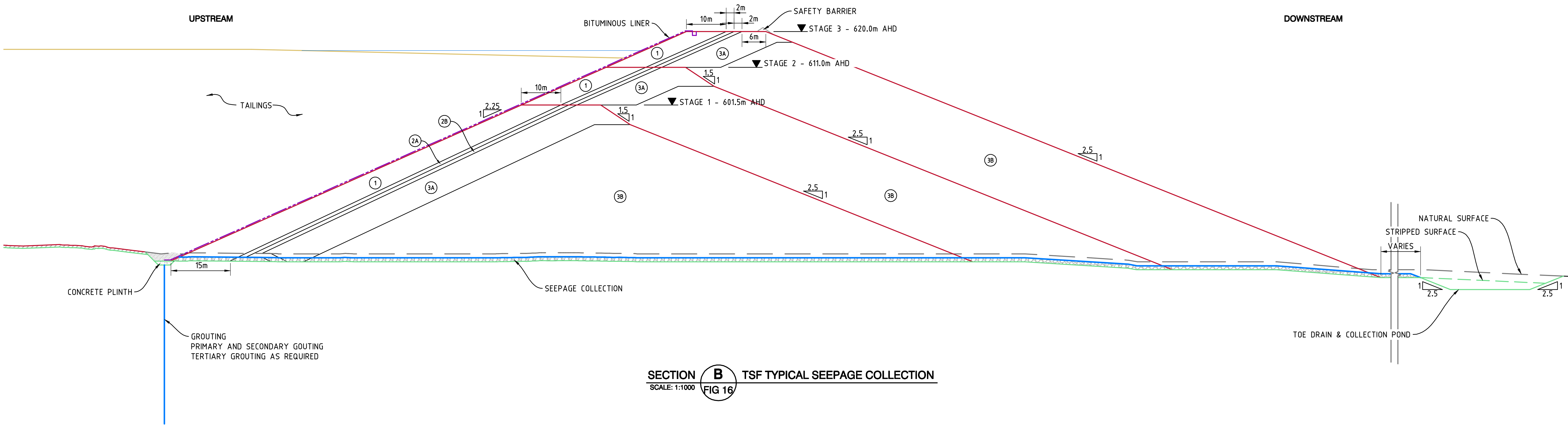

SECTION B TSF TYPICAL SEEPAGE COLLECTION
SCALE: 1:1000 FIG 16

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TSF PRELIMINARY DESIGN
TYPICAL SEEPAGE COLLECTION SECTION

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FIGURE 15

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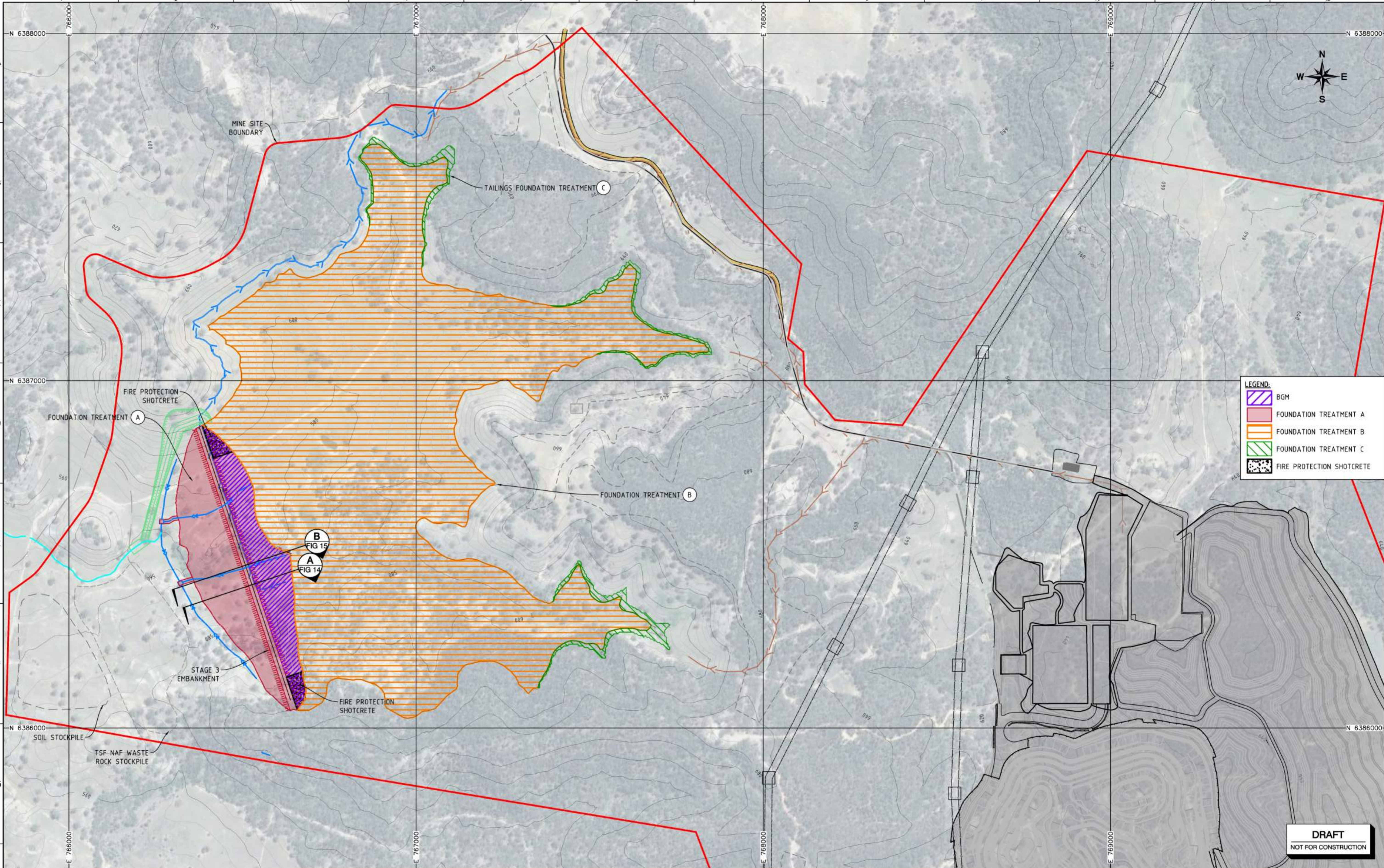

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B	SPILLWAY ALTERATION	22.10.18	HR	HJW	
A	ISSUED	07.05.18	AC	HJW	

SCALE:	1:10,000
JOB No.	116217.09
DATE	04.07.19
DESIGN	HW
DRAWN	HR
CHECKED	PR
APPROVED	

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TSF PRELIMINARY DESIGN
EMBANKMENT LAYOUT PLAN AND FOUNDATION PREPARATION

DWG. No.
FIGURE 16

SHEET SIZE A3 Rev. D

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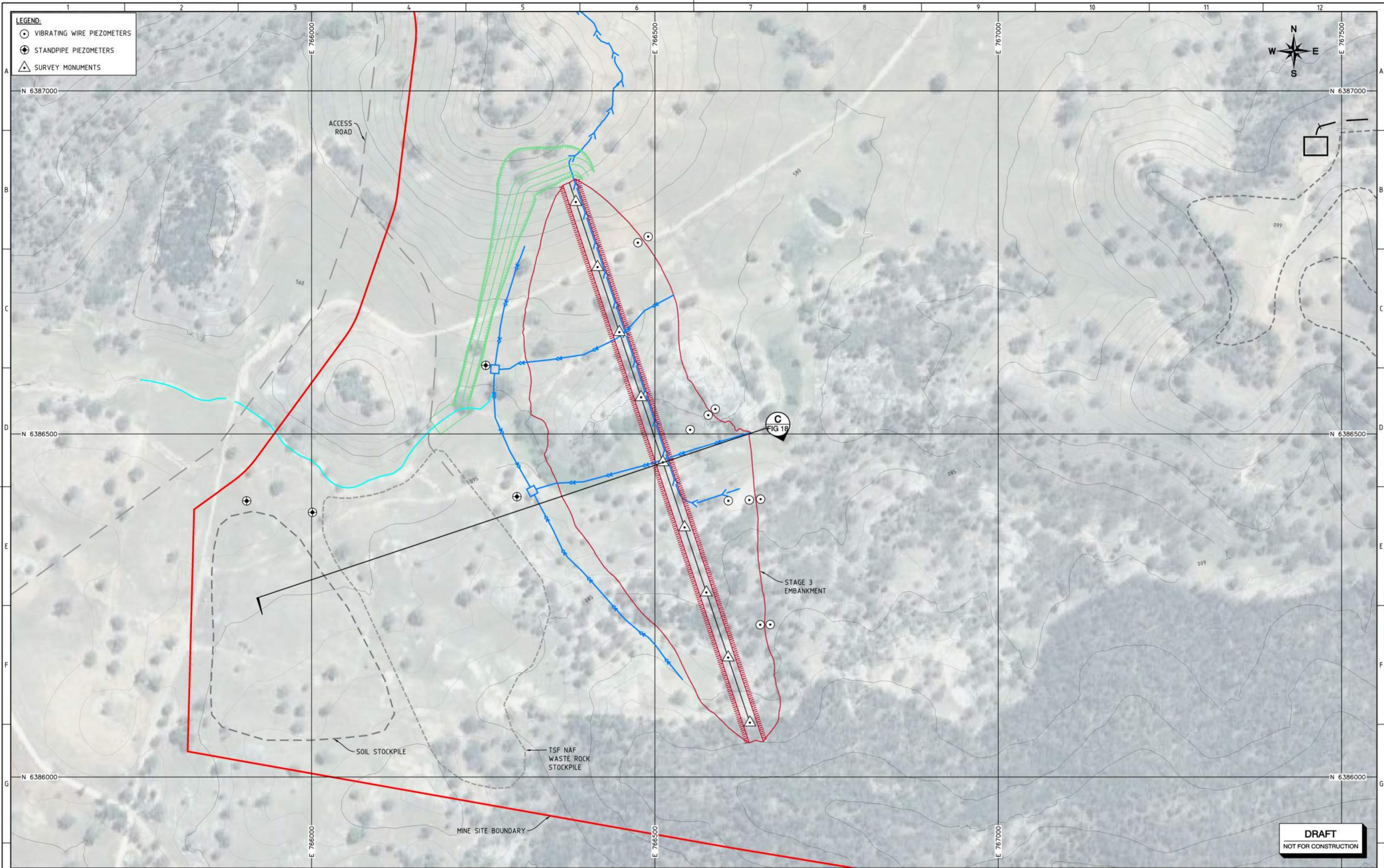
SHEET 1 OF 1

K:\Projects\116\116217 Bowdens Silver Project, Bowdens Silver Pty Limited\09 Feasibility Update\CADD\CAD\Figure 16 - Embankment Layout Plan and Foundation Preparation.dwg Friday, 29 November 2019 11:14:07 AM



No.	DESCRIPTION	DATE	DRAWN	CHECK'D	APPR'D
D	SPILLWAY ALTERATION & UPDATED STANDPIPE PIEZOMETER LOCATIONS	03.09.19	SC	HJW	
C	PLANT DETAILS REVISED	04.07.19	HR	PR	
B	SPILLWAY ALTERATION	22.10.18	HR	HJW	
A	ISSUED	07.15.18	AC	HJW	

SCALE:	1:5,000
JOB No.	116217.01
DATE	07.05.18
DESIGN	HW
DRAWN	HR
CHECKED	
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TSF PRELIMINARY DESIGN
TYPICAL MONITORING PLAN

DWG. No.
FIGURE 17

SHEET SIZE A3 | Rev. D

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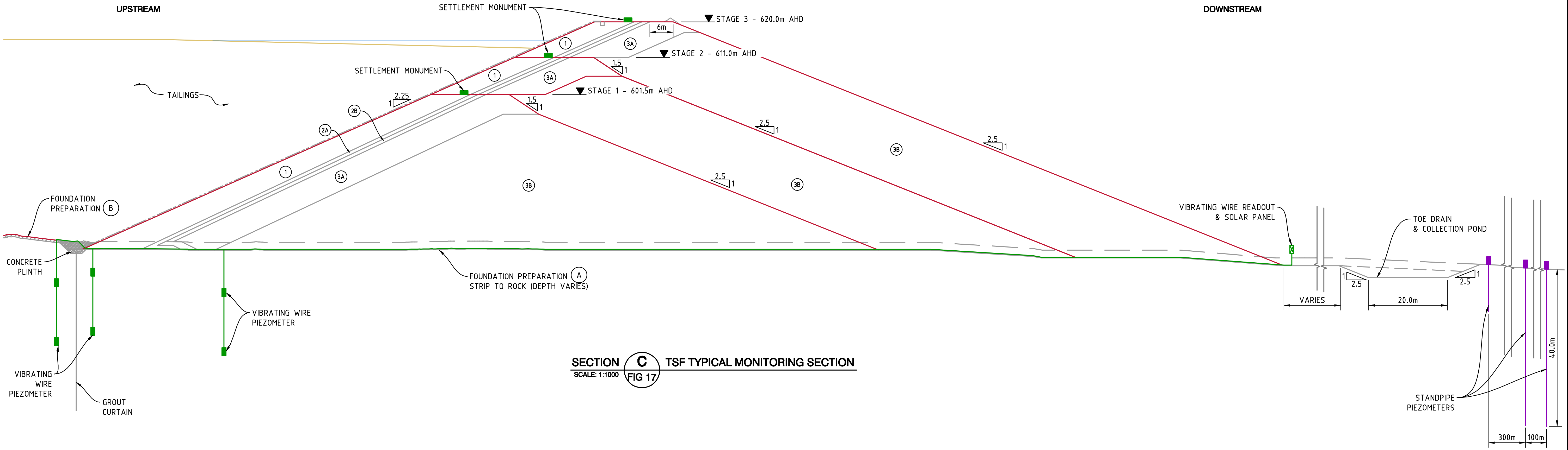
SHEET 1 OF 1

K:\Projects\116\116217 Bowdens Silver Project, Bowdens Silver Pty Limited\09 Feasibility Update\CADD\CAD\Figure 17 - Monitoring Plan.dwg Monday, 7 May 2018 11:55:33 AM



SECTION C FIG 17 TSF TYPICAL MONITORING SECTION
SCALE: 1:1000

DRAFT
NOT FOR CONSTRUCTION

No.	DESCRIPTION	DATE	DRAWN	CHECK'D	APPR'D
A	FIGURE NUMBER UPDATE AND ADDITIONAL STANDPIPE	03.09.19	SC	HW	-

SCALE:	1:1,000
JOB No.	116217.01
DATE	23.04.18
DESIGN	HW
DRAWN	AC
CHECKED	-
APPROVED	-

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TSF PRELIMINARY DESIGN
MONITORING SECTION

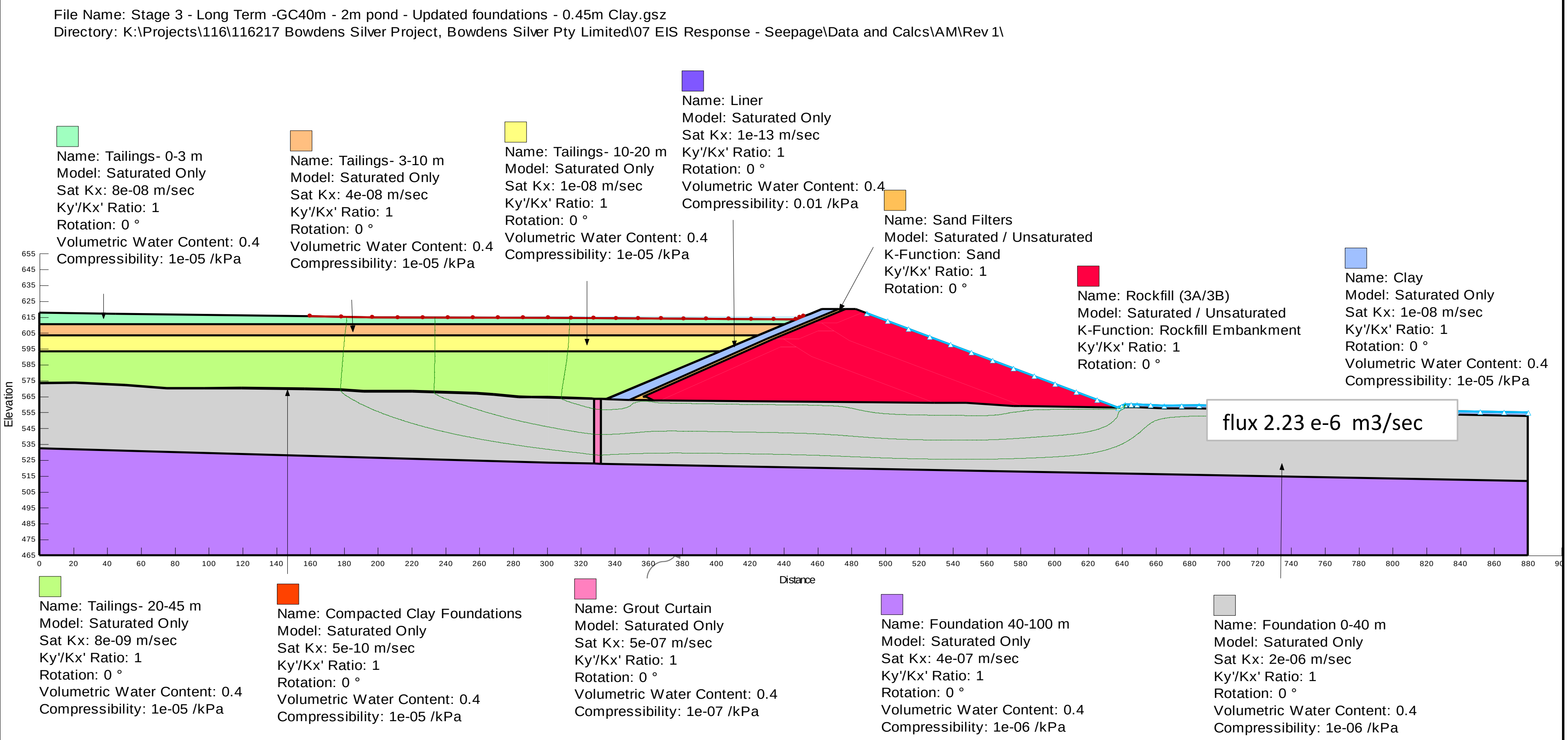
DWG. No.
FIGURE 18

SHEET SIZE A3 Rev. A

SHEET 1 OF 1

APPENDICES

APPENDIX A



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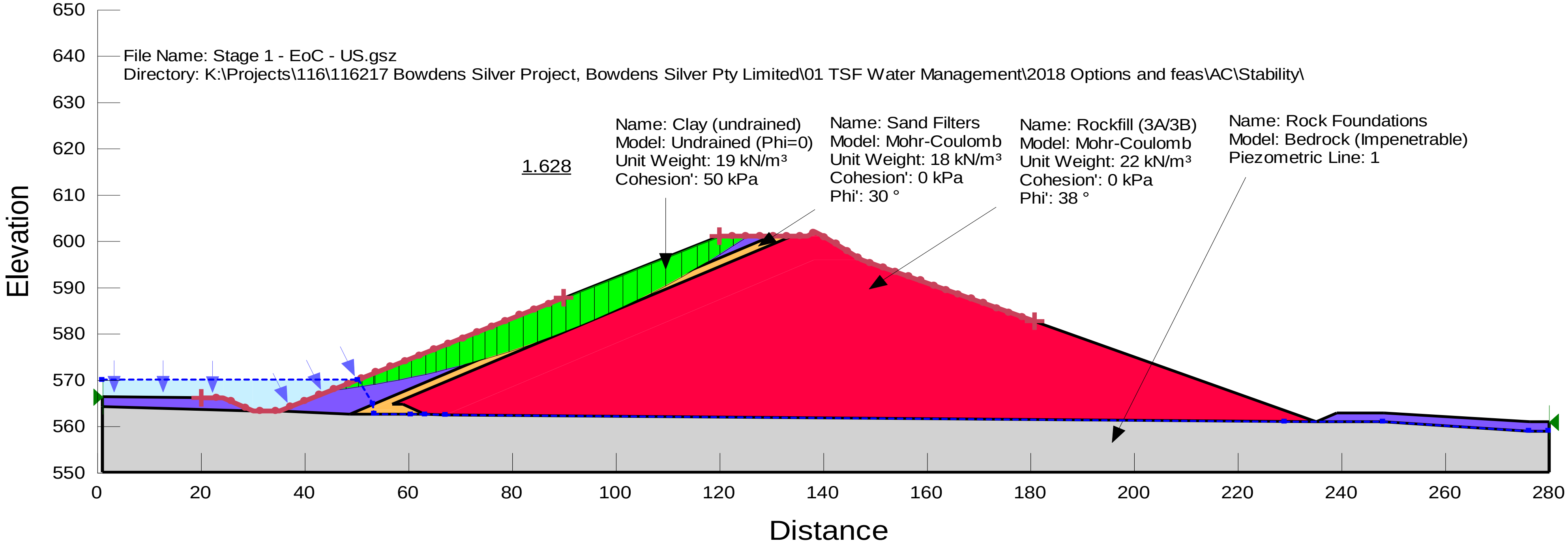
Stage 3 Embankment Crest at Elevation 620 m AHD

Steady State Seepage Analysis - 2m Operating Pond 0.45 m Compacted Clay Foundations

Date:	28/08/2019	Job No:	116217.08	FIGURE A.1

APPENDIX B



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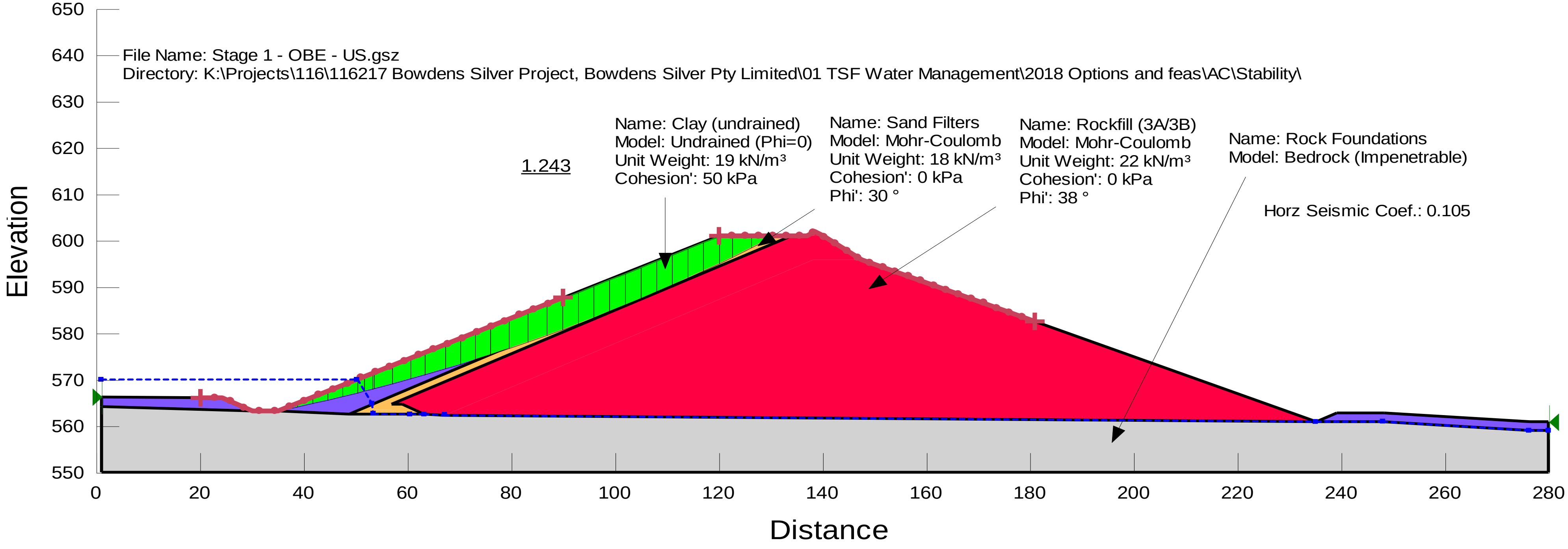
PRELIMINARY DESIGN

Stage 1 Embankment - End of Construction - Upstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B1.1

Y:\2016\116217 Bowdens Project, Silver Mines Limited\01 TSF Water Management\2018 Options and feas\AC\Stability\Stability summary Fig B1.1-Stg 1 EoC US



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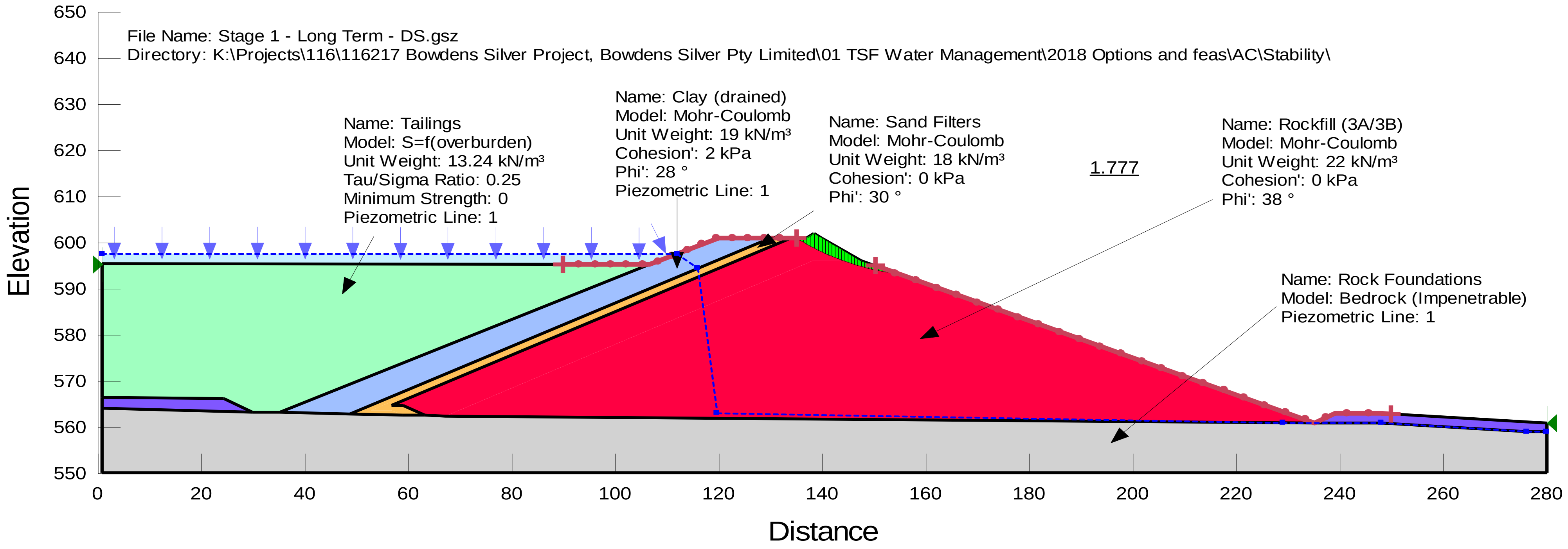
Stage 1 Embankment - OBE - Upstream Failure

Date:	18/014/2018	Job No:	116217.01	FIGURE B1.2



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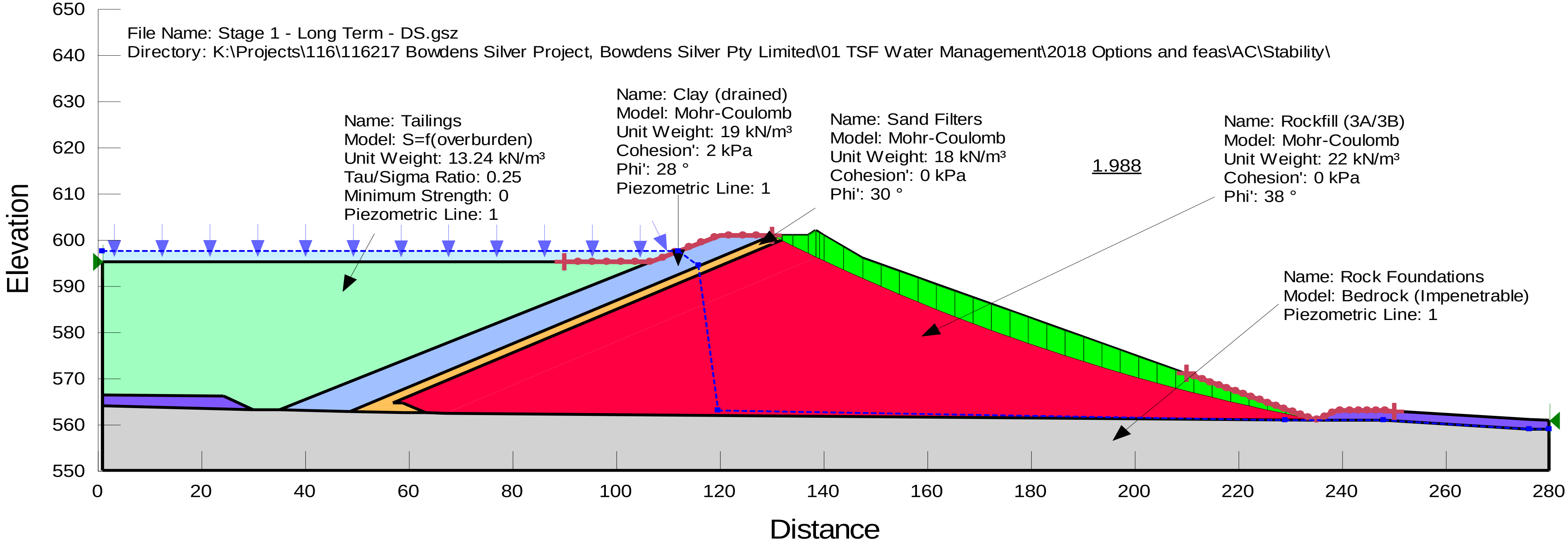
PRELIMINARY DESIGN

Stage 1 Embankment - Long Term Static Conditions - Local Downstream Failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B1.3**



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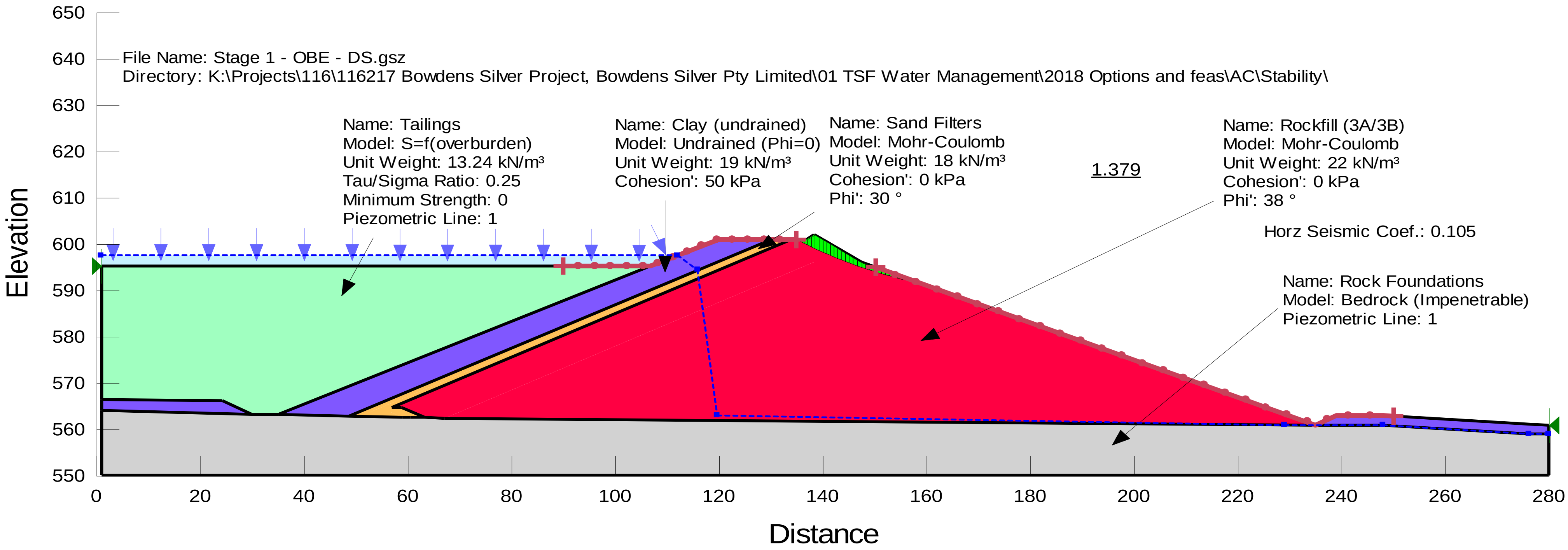
PRELIMINARY DESIGN

Stage 1 Embankment - Long Term Static Conditions - Global Downstream Failure

Date:	18/014/2018	Job No:	116217.01	FIGURE B1.4



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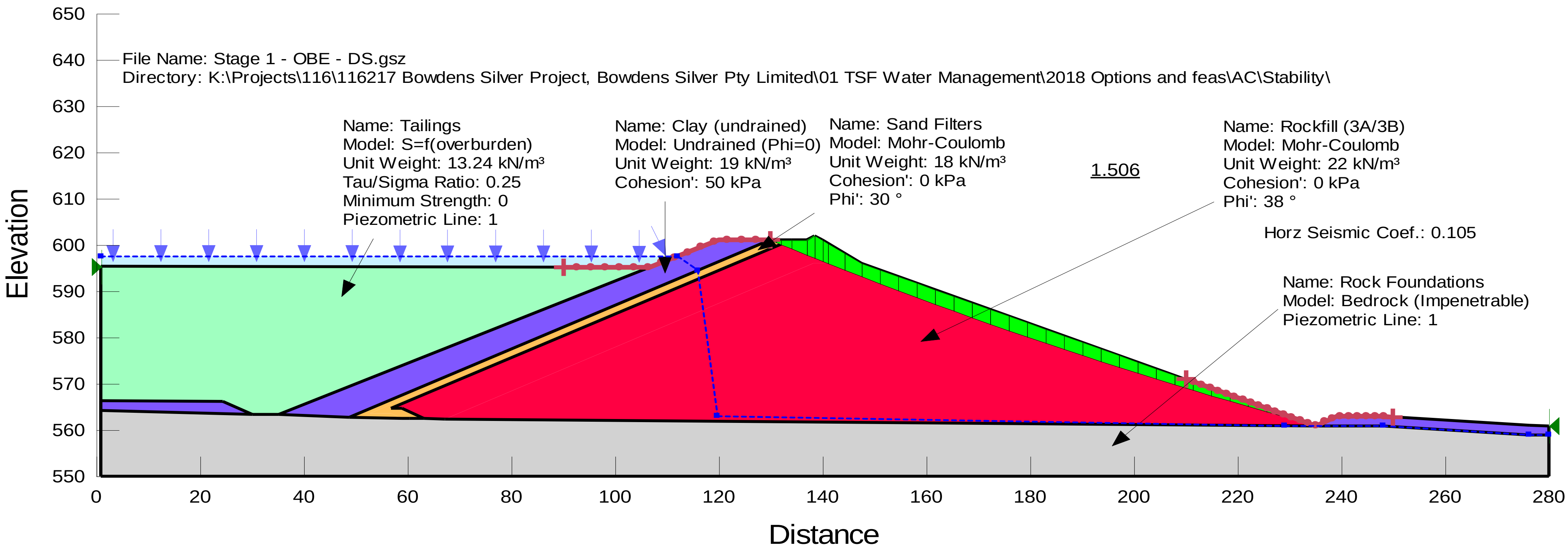
PRELIMINARY DESIGN

Stage 1 Embankment - Long Term Conditions - OBE - Local Downstream Failure

Date:	18/014/2018	Job No:	116217.01	FIGURE B1.5

Y:\2016\116217 Bowdens Project, Silver Mines Limited\01 TSF Water Management\2018 Options and feas\AC\Stability\Stability summary Fig B1.5 - Stg 1 LT OBE LDS



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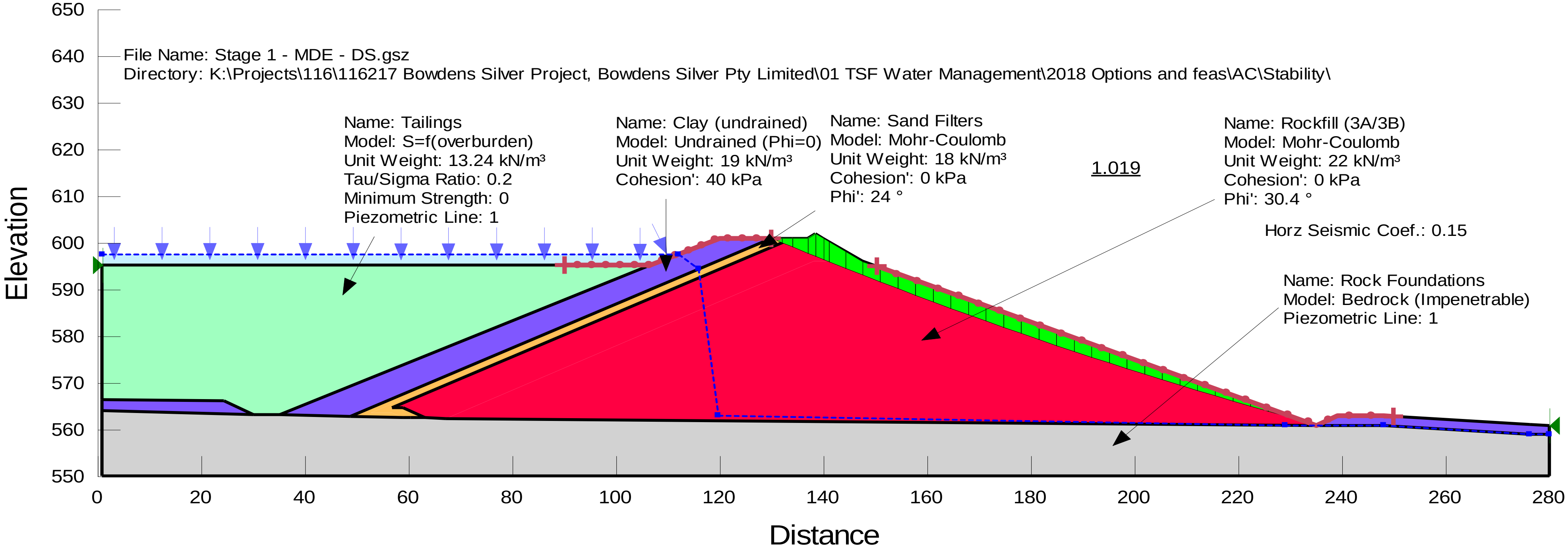
PRELIMINARY DESIGN

Stage 1 Embankment - Long Term Conditions - OBE - Global Downstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B1.6



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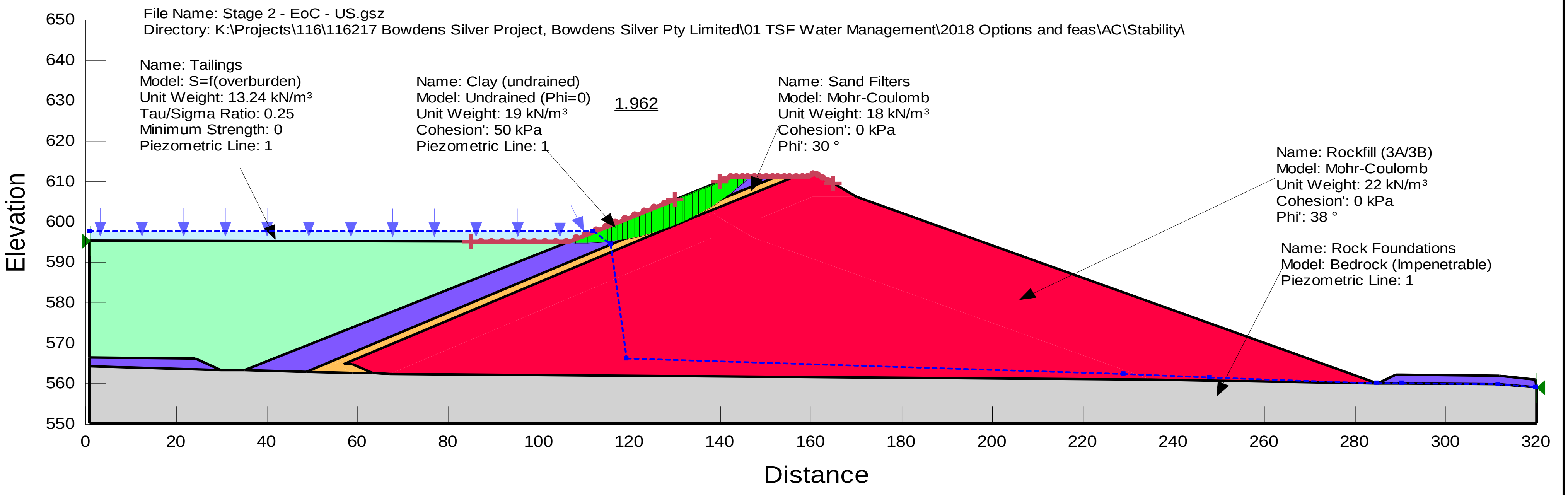
PRELIMINARY DESIGN

Stage 1 Embankment - Long Term Conditions - USACE MDE (20% Strength Reduction, 50% PGA) - Downstream Failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B1.7**



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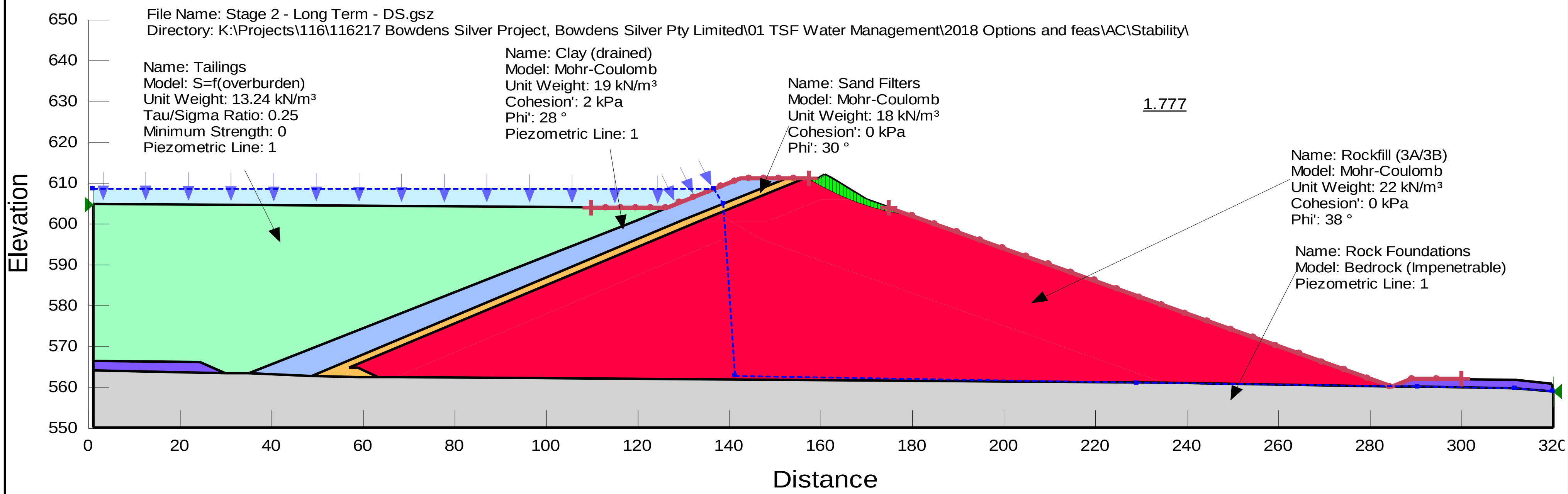
BOWDENS SILVER PROJECT

PRELIMINARY DESIGN

Stage 2 Embankment - End of Construction - Upstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B2.1



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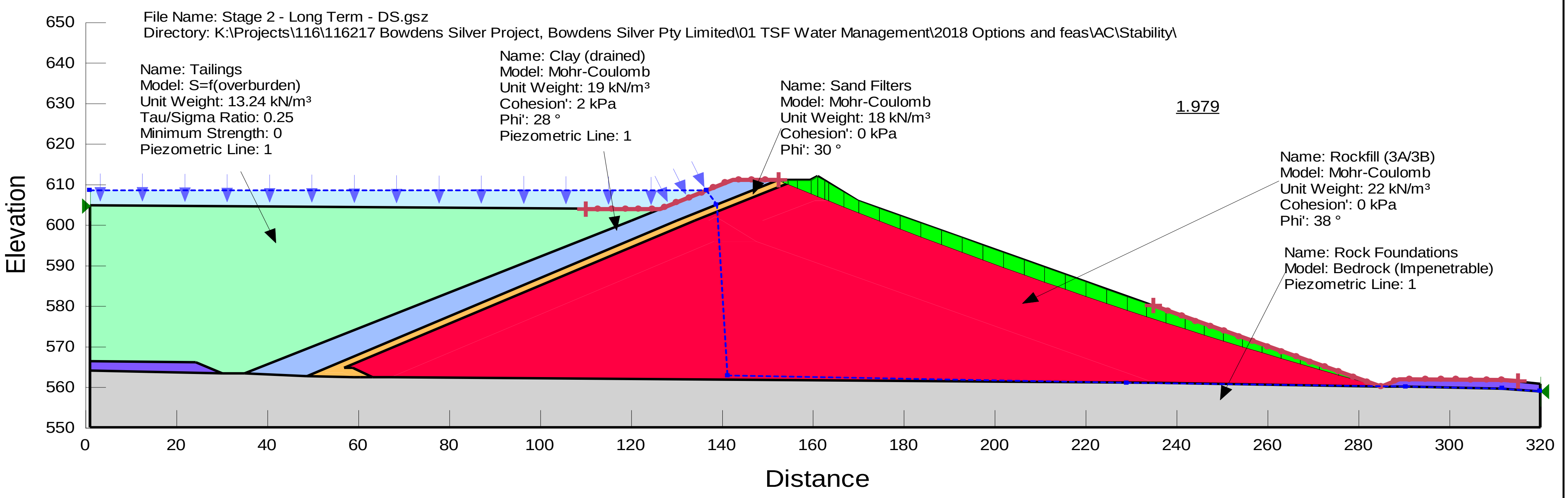
PRELIMINARY DESIGN

Stage 2 Embankment - Long Term Static Conditions - Local Downstream Failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B2.2**



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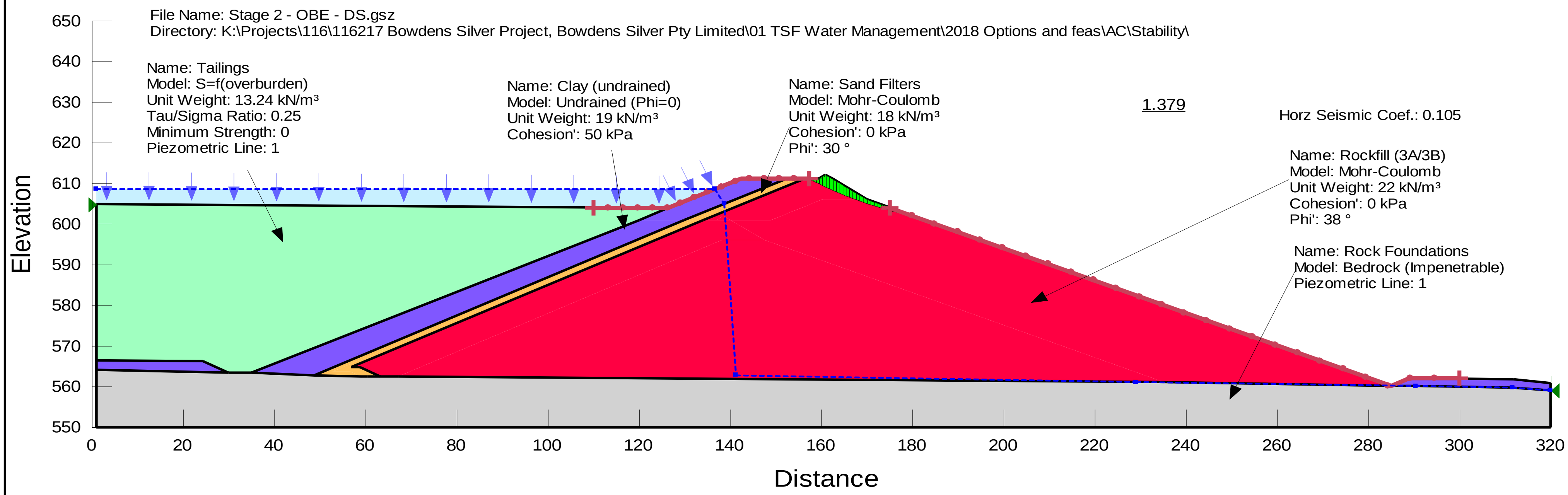
PRELIMINARY DESIGN

Stage 2 Embankment - Long Term Static Conditions - Global Downstream Failure

Date:	18/014/2018	Job No:	116217.01	FIGURE B2.3

Y:\2016\116217 Bowdens Project, Silver Mines Limited\01 TSF Water Management\2018 Options and feas\AC\Stability\Stability summary Fig B2.3 - Stg 2 LT GDS



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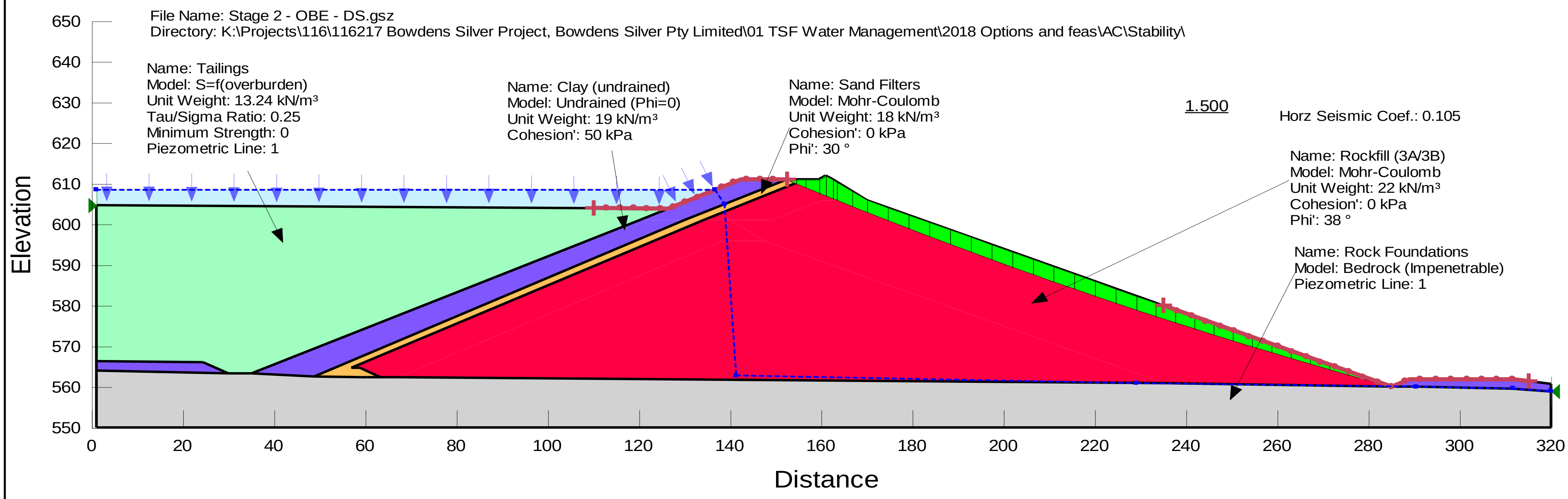
BOWDENS SILVER PROJECT

PRELIMINARY DESIGN

Stage 2 Embankment - Long Term Conditions - OBE - Local Downstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B2.4



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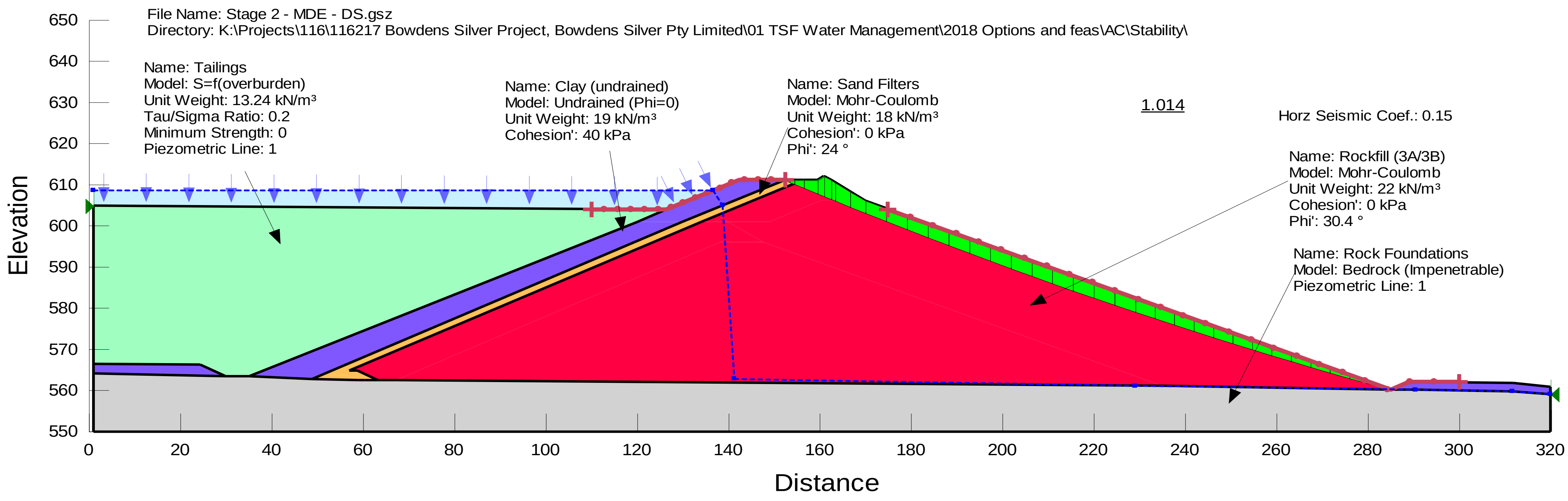
PRELIMINARY DESIGN

Stage 2 Embankment - Long Term Conditions - OBE - Global Downstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B2.5



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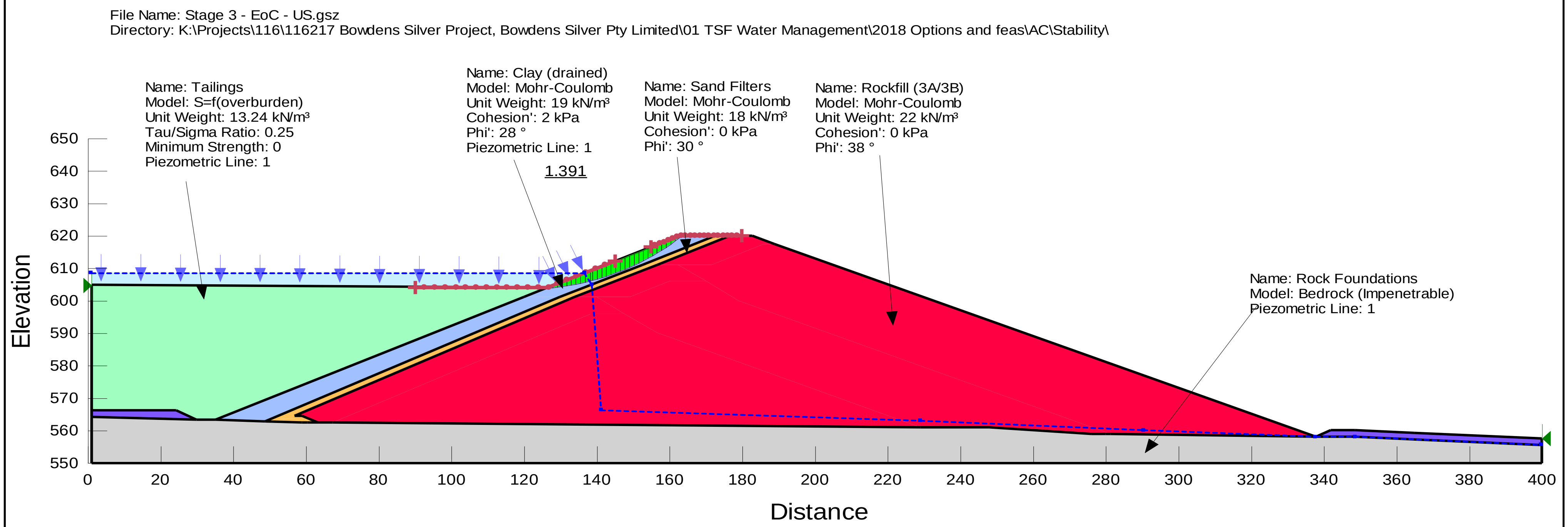
PRELIMINARY DESIGN

Stage 2 Embankment - Long Term Conditions - USACE MDE (20% Strength Reduction, 50% PGA) - Downstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B2.6



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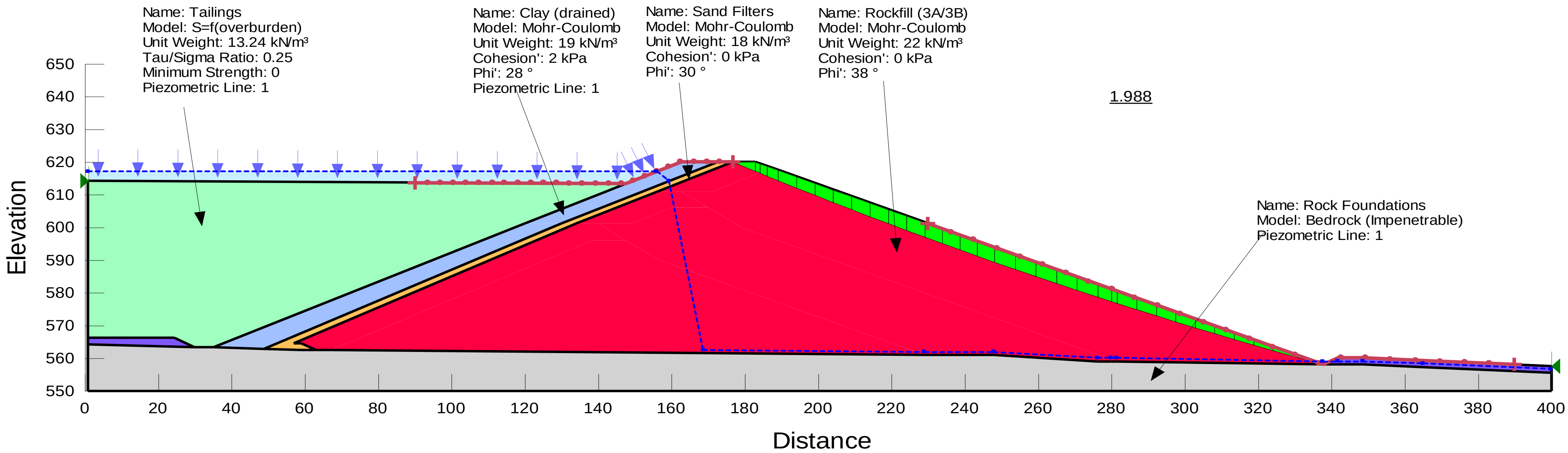
PRELIMINARY DESIGN

Stage 3 Embankment - End of Construction - Upstream failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B3.1**

File Name: Stage 3 - Long Term - DS.gsz
Directory: K:\Projects\116\116217 Bowdens Silver Project, Bowdens Silver Pty Limited\01 TSF Water Management\2018 Options and feas\AC\Stability\



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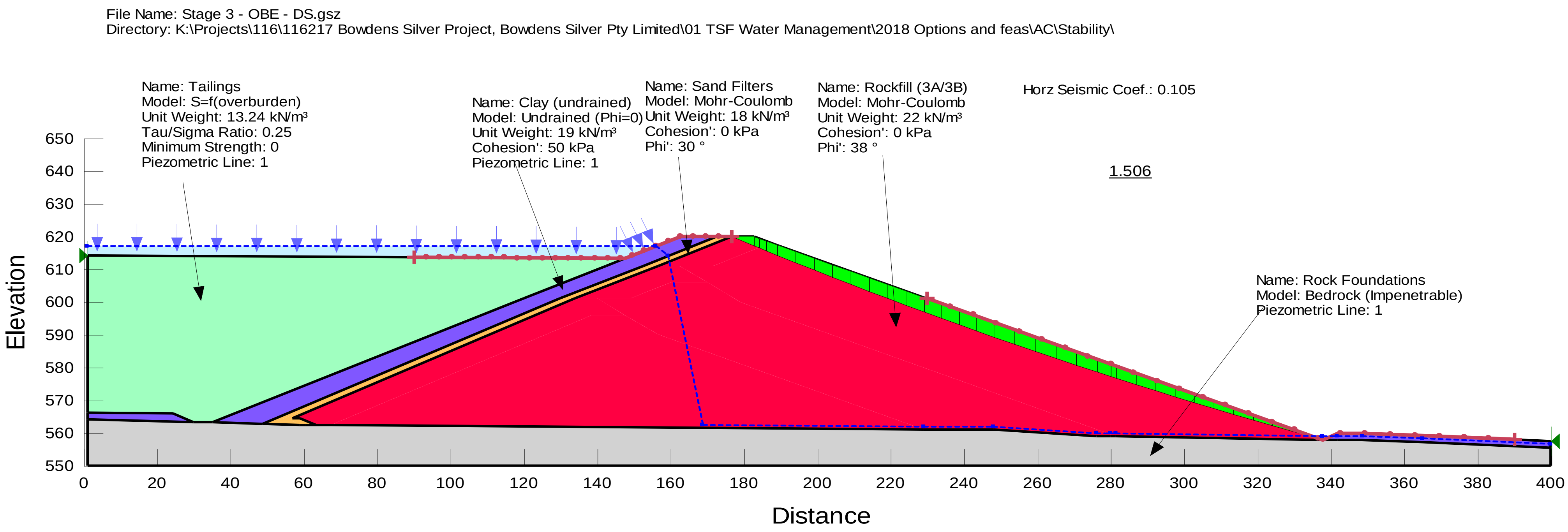
PRELIMINARY DESIGN

Stage 3 Embankment - Long Term Static Conditions - Downstream Failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B3.2**



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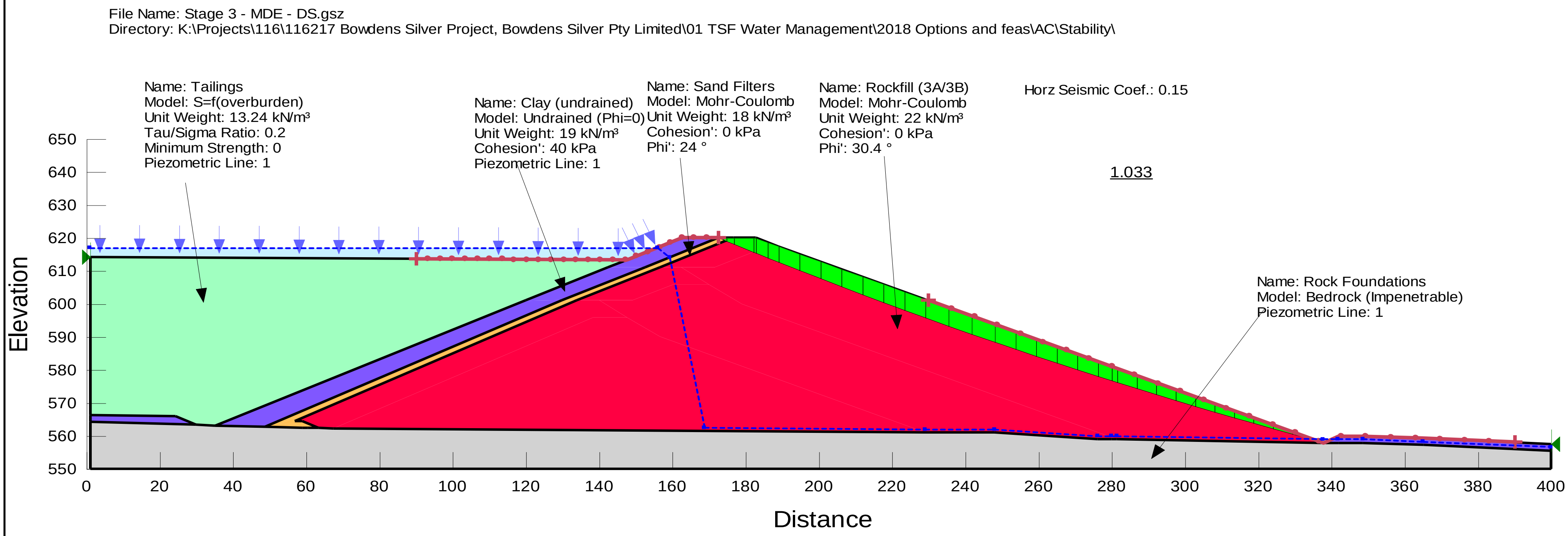
PRELIMINARY DESIGN

Stage 3 Embankment - Long Term Conditions OBE - Downstream Failure

Date:	18/014/2018	**Job No:**	116217.01	**FIGURE B3.3**



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PRELIMINARY DESIGN

Stage 3 Embankment - Long Term Conditions - USACE MDE (20% Strength Reduction, 50% PGA) - Downstream Failure

Date: 18/014/2018 **Job No:** 116217.01

FIGURE B3.4

APPENDIX C

APPENDIX C

SCHEDULE OF QUANTITIES – STAGE 1

Description	Unit	Estimated Quantity
SITE PREPARATION		
Establishment	Item	1
Clearing and Grubbing	m^2	1,087,000
Stripping topsoil & placement within 1km	m^3	326,000
EMBANKMENT CONSTRUCTION		
Foundation Preparation B - Borrow and Compact clay	m^3	204,000
Foundation Preparation B - Rip, Condition and Compact clay	m^2	542,000
Foundation Preparation B2 - Foundation Treatment (Borrow and Compact Clay for B not required)	*m^2*	*193,000*
Foundation Preparation B2 - Underdrainage	*m*	*3,900*
Foundation Preparation A - Stripping unsuitable subsoil to stockpile within 500m	m^3	5,000
Foundation Preparation A - Stripping to Bedrock to stockpile within 500m	m^3	117,000
Foundation Grouting: 40m primary, 30m secondary, 20m Tertiary, 15m Blanket	m	22,000
Seepage Collection system (includes Geotextile and Rockfill)	m	1,350
Seepage Collection Pond and Toe drain	m^3	12,000
Zone 1 (From subsoil stripping)	m^3	20,000
Zone 1 (Borrow from within 500m of foundation excavation)	m^3	114,000
Zone 2 (Filter placement)	m^3	59,000
Zone 3A (Haul from WRE)	m^3	180,000
Zone 3B (Haul from Stockpile within 500m)	m^3	638,000
Zone 3A + 3B (Spread and Compact)	m^3	869,000
LINING		
Bituminous Lining	m^2	44,000
Concrete Fire Protection	m^2	4,000
SPILLWAY		
Spillway Excavation including embankment protection	m^3	69,000
Excess Material Haul	m^3	67,000
MONITORING		
Vibrating Wire Piezometers - Unit	Item	23
Vibrating Wire Piezometers - Cabling	m	1,350
Standpipe Piezometers - Total Depth	m	100
Settlement Monuments	No.	9

SCHEDULE OF QUANTITIES – STAGE 2

Description	Unit	Estimated Quantity
SITE PREPARATION		
Establishment	Item	1
Clearing and Grubbing	m^2	33,000
Stripping topsoil & placement within 1km	m^3	10,000
EMBANKMENT CONSTRUCTION		
Foundation Preparation B - Borrow and Compact clay	m^3	98,000
Foundation Preparation B - Rip, Condition and Compact clay	m^2	260,000
Foundation Preparation A - Stripping unsuitable subsoil to stockpile within 500m	m^3	2,000
Foundation Preparation A - Stripping to Bedrock to stockpile within 500m	m^3	46,000
Zone 1 (From subsoil stripping)	m^3	8,000
Zone 1 (Borrow from within 500m of foundation excavation)	m^3	68,000
Zone 2 (Filter placement)	m^3	31,000
Zone 3A (Haul from WRE)	m^3	114,000
Zone 3B (Haul from Stockpile within 500m)	m^3	777,000
Zone 3A + 3B (Spread and Compact)	m^3	942,000
LINING		
Bituminous Lining	m^2	27,000
Concrete Fire Protection	m^2	7,000
SPILLWAY		
Spillway Excavation including embankment protection	m^3	70,000
Excess Material Haul	m^3	35,000
MONITORING		
Settlement Monuments	No.	9

SCHEDULE OF QUANTITIES – STAGE 3

Description	Unit	Estimated Quantity
SITE PREPARATION		
Establishment	Item	1
Clearing and Grubbing	m^2	44,000
Stripping topsoil & placement within 1km	m^3	14,000
Foundation Prep C - Proof Rolling of Tailings Impoundment outside decant area	m^2	27,000
EMBANKMENT CONSTRUCTION		
Foundation Preparation B - Borrow and Compact clay	m^3	74,000
Foundation Preparation B - Rip, Condition and Compact clay	m^2	195,000
Foundation Preparation A - Stripping unsuitable subsoil to stockpile within 500m	m^3	3,000
Foundation Preparation A - Stripping to Bedrock to stockpile within 500m	m^3	62,000
Zone 1 (From subsoil stripping)	m^3	11,000
Zone 1 (Borrow from within 500m of foundation excavation)	m^3	6,000
Zone 2 (Filter placement)	m^3	31,000
Zone 3A (Haul from WRE)	m^3	106,000
Zone 3B (Haul from Stockpile within 500m)	m^3	1,196,000
Zone 3A + 3B (Spread and Compact)	m^3	1,355,000
LINING		
Bituminous Lining	m^2	28,000
Concrete Fire Protection	m^2	8,000
SPILLWAY		
Spillway Excavation including embankment protection	m^3	47,000
Excess Material Haul	m^3	17,000
MONITORING		
Settlement Monuments	No.	9