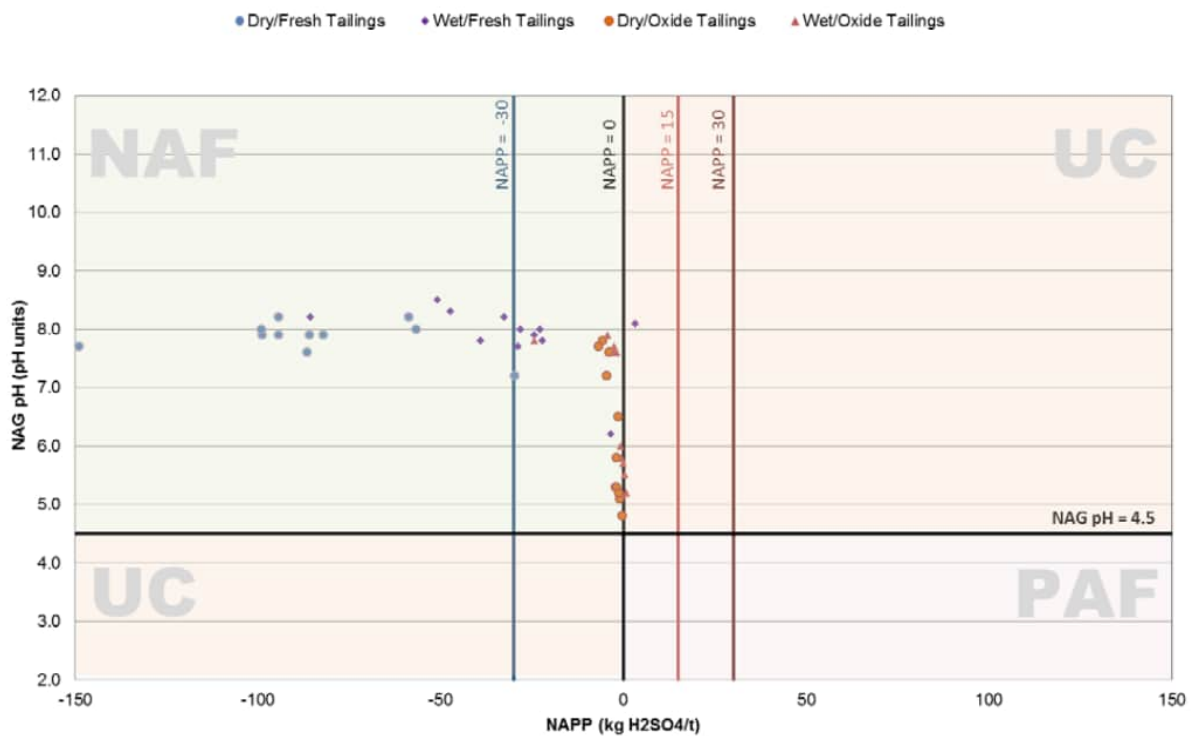


Acid Production Potential (NAPP) value. Note that NAPP is simply the difference between the Maximum Potential Acidity (MPA) and the Acid Neutralising Capacity (ANC).

Chart 5.2 presents the tailings sample classification following the AMIRA method [24].

**Chart 5.2**  
**NAG pH & NAPP Values For Tailings Samples [23]**



All the tailings samples reported negative NAPP value and high ANC/MPA ratios (>3), except one sample which in Golder's opinion was a caveat of the heterogeneity between batches of tailings.

The range of NAPP (-30 to -90 kg H<sub>2</sub>SO<sub>4</sub>/t) and ANC/MPA ratio values (2.4 to 193) placed all the samples in the acid consuming category.

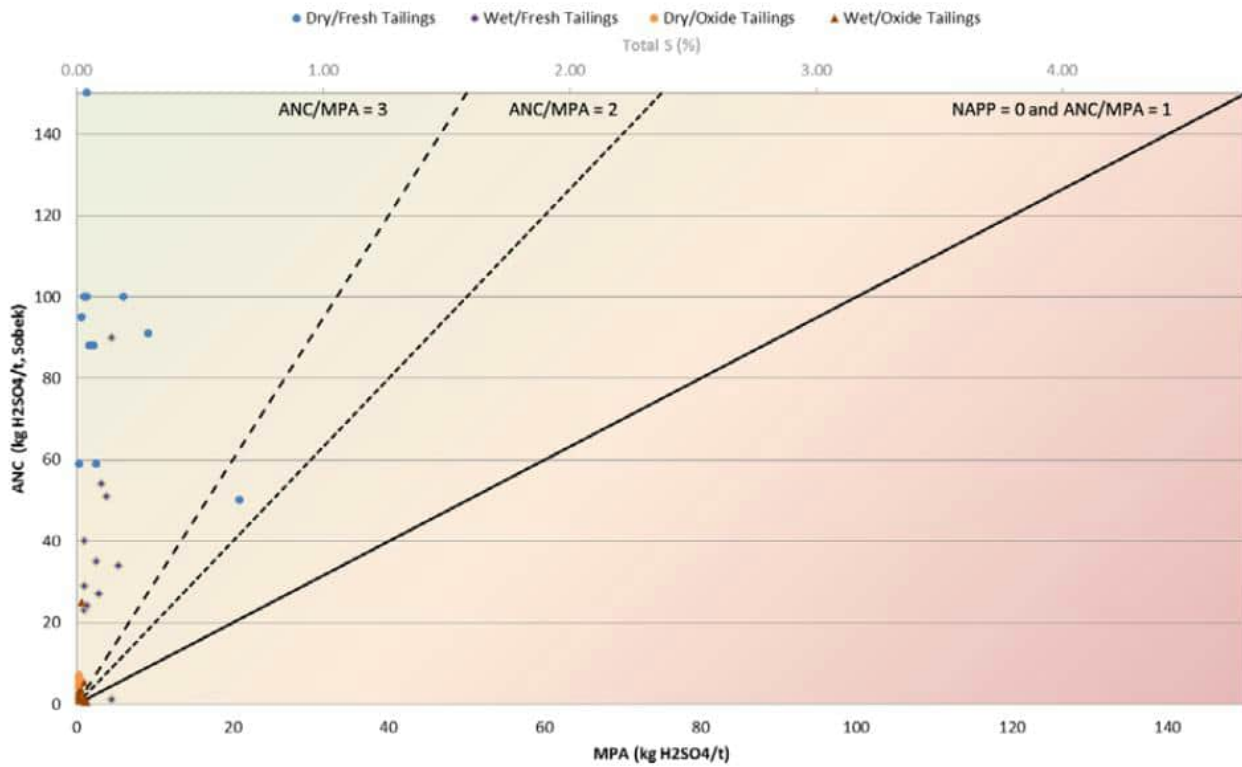
Chart 5.3 shows the ANC versus MPA values calculated for tailings samples.

Golder conducted X-ray diffraction (XRD) analyses to identify the mineralogy of the tailings samples. The oxide tailings were mainly quartz and iron oxide in the form of hematite and goethite. No sulphide or carbonate minerals were identified in the oxide samples. A wide variety of minerals were encountered in the fresh tailings samples. Pyrite or any other sulphide phase was not present in detectable quantities. Significant sulphate mineral content (fibroferrite) was observed in the fresh tailings. As the only sulphate mineral phase identified by XRD, the potential for acid formation from fibroferrite is much lower than pyrite.

RGS Environmental Pty Ltd (RGS) was commissioned in 2017 by IBO to review the works conducted for geochemical characterisation of the waste rock and tailings materials [25]. RGS reviewed the historical studies as well as IBO documents relating to proposed refinement of the waste rock classification. RGS agreed that most of the waste rock materials are likely to be NAF with excess ANC. However, they confirmed that there are materials which are PAF.

RGS recommended further work to be conducted on geochemical characterisation of waste rock materials. It appears RGS is in agreement with the ANC of tailings materials outlined by GHD and Golder.

**Chart 5.3  
ANC & MPA Values For Tailings Samples [23]**



No geochemical characterisation testing has been conducted on decant water to date.

Since there is a possibility of PAF material in the waste rock, it is suggested that routine tests are conducted on the waste rock during all stages of construction with the aim of eliminating the use of any PAF material in the TSF facility.

## 6 TSF CLASSIFICATION AND CONSEQUENCE CATEGORIES

### 6.1 Assessment Criteria

TSF 2A (which incorporates the RWP) has been designed to comply with the:

- Code of Practice for Tailings Storage Facilities in Western Australia issued by the WA Department of Mines, Industry Regulation and Safety (DMIRS, formerly DMP) [26];
- DMIRS (formerly DMP) August 2015 guides on the preparation of a design report for tailings storage facilities (TSFs) [27]; and
- Australian National Committee on Large Dams (ANCOLD) May 2012 guidelines on tailings dams planning, design, construction and closure [28].

Specific design requirements for tailings dams are given by DMIRS and ANCOLD, which relate to the risk classification and consequence category of the facility. These classifications dictate the level of detail applied to design, operation, maintenance and surveillance.

With respect to design, the determined consequence category (equivalent to hazard rating) from ANCOLD [27] provides the required recurrence intervals of rainfall events (for stormwater storage capacity) and earthquakes (for seismic stability analyses).

### 6.2 WA Regulatory Requirements

In accordance with the DMIRS Code of Practice [26], the hazard rating of the proposed TSF 2A arrangement has been assessed as tabulated in Table 6.1.

**Table 6.1  
Hazard Rating for The Proposed TSF 2A (DMIRS)**

Type	Extent or severity of impact or damage	Hazard Rating
<i>Controlled or Uncontrolled Release of Tailings/Water or Seepage</i>		
Loss of human life or personal injury	Loss of life or injury is possible although not expected	Medium
Adverse human health	The potential for human exposure is limited, and temporary adverse health effects are possible	Low
Loss of assets	Limited or no potential for loss of livestock. Limited or no potential for destruction or loss of assets. Loss of TSF storage capacity is possible and repair is practicable.	Medium
Environmental or heritage damage	Temporary damage to the natural environment is possible. Temporary adverse effects on flora and fauna. Limited or no potential for damage of items of heritage or historical value.	Medium
<i>Embankment or Structural Failure</i>		
Loss of human life or personal injury	Loss of life or injury is possible.	Medium
Adverse human health	The potential for human exposure is limited, and temporary adverse health effects are possible.	Medium
Loss of assets	Loss of some livestock is possible. Temporary loss of assets is possible and economic repairs can be made. Loss of TSF storage capacity is possible and repair is practicable.	Medium
Environmental or heritage damage	Temporary damage to the natural environment is possible. Temporary adverse effects on flora and fauna are possible. Temporary damage of items of heritage or historical value is possible.	Medium

A hazard rating of ‘**Medium**’ has been derived based on the highest perceived impact of release of tailings, water or seepage and embankment structural failure.

Based on consideration of hazard rating and maximum embankment height (over 68 m at the end of the life of the facility), TSF 2A is considered as a **Category 1** facility, as per the DMIRS requirements [26].

### 6.3 Consequence Category Assessment Methodology

The ANCOLD guidelines [28] provide a similar method for classification of tailings dams, defining “consequence categories” for (a) embankment failure and (b) for environmental spillage of contained water. The consequence category for embankment failure is considered first, followed by environmental spillage of contained water.

The embankment failure consequence category considers the potential failure modes of the facility and the resulting consequences to the business, the social and natural environment, and the potential for loss of life in a dam break failure situation. Hence an assessment of the downstream receiving environment is required, together with a dam break study to assess the downstream inundation impacts of a critical failure case for the TSF and RWP.

A detailed presentation of the dam break analysis methodologies, inputs and outcomes is provided in the **Appendix D**, Dam Break Study Report. The following sections provide a summary.

### 6.4 Dam Break Analysis

#### 6.4.1 Downstream Environment

Based on the topographic detail in the vicinity of TSF2A area and plant site, it is evident that spillage of tailings or water would not impact the plant area or mine pits, as these features are both upstream from the TSF. The area of impact downstream of the TSF and RWP is largely undeveloped with only road and rail transport infrastructure and potential small mining developments possibly in the path of a tailings and water release.

The direction of flow would be to the north-west along existing drainage channels (Chinnamon Creek) for approximately 45 km until it intersects with the tributary of the Turner River. The tributary crosses the Pilgangoora mining area access road, Pippingarra Road, the Port Hedland - Wittenoom Road and flows beneath three railway bridges (BHP, FMG and Roy Hill) before it flows into the Turner River.

The area along Chinnamon Creek and the tributary of Turner River provides habitat for a number of significant fauna species, and there are also individual specimens of Declared Rare Flora (DRF) in this area.

Regarding major infrastructure in the downstream receiving area, the following is a summary:

- North Star borefield road (immediately downstream of the RWP Embankments);
- The Pilgangoora mining area access road (approximately 28 km north-west);
- Pippingarra Road and the adjacent Port Hedland - Wittenoom Road (approximately 37 km north-west); and
- Three railway crossings (BHP, FMG, Roy Hill, 37 km to 39 km north-west).

The North Star borefield road is an IBO road, with only intermittent use and no public access. The Pilgangoora access road crosses Chinnamon Creek further downstream, which connects the Lithium mines operated by Pilbara Minerals and Altura at Pilgangoora to the Great Northern Highway. Pippingarra Road and the nearby Port Hedland - Wittenoom Road run northwards alongside the BHP rail line and cross Chinnamon creek approximately 10 km downstream of the Pilgangoora access road crossing. Each road has a broad floodway at the creek. About 2 km further downstream, the FMG and Roy Hill railways cross Chinnamon Creek.

The railway crossings over Chinnamon Creek have been designed to high standards to withstand extreme flood conditions, and have clearances from the creek bed ranging from 4.3 m (BHP) to 6 m (FMG and Roy Hill). The BHP crossing is illustrated in **Photo 2**.



**Photo 2 - BHP Rail Crossing At Chinnamon Creek**

#### 6.4.2 Failure Mode Assessment Approach

The risk-based approach to the design of the TSF and RWP embankments are discussed in **Section 2.3**. It is evident from this discussion that plausible failure modes of key elements of the TSF system are addressed in the design. When undertaking a dambreak analysis, it is generally accepted that the plausible failure mode generating the greatest impact to the downstream receiving environment be selected for assessment.

However, in this case there is a degree of uncertainty regarding the downstream receiving environment and the Population At Risk (PAR, discussed further in **Section 6.4.7**). It has therefore been decided to adopt a “worst-case” failure mode in the evaluation of the dambreak impacts. This comprises earthquake-induced deformations of the embankment, in which severe lateral cracking occurs in the embankment crest. Coupled with an end-of-filling timeframe, erosive breaching of the crest initiates, and failure of the TSF embankment occurs. This is the basis of the dambreak assessment provided in **Appendix D**.

#### 6.4.3 Failure Condition

The Sunny Day Failure (SDF) condition was considered to be the critical case based on a comparison of expected incremental effects. Under an “incremental flood” situation, the antecedent conditions in the downstream receiving area due to extreme flooding would include significantly larger flows than what could be expected due to the overtopping failure of the TSF and RWP Embankments. This is due to the TSF2A site being at the head of the catchment, with the catchment area at the location of critical public infrastructure being several times larger.

Therefore, it is considered that a release of the maximum theoretical water volume able to be stored in the system at the final Stage of filling would produce a negligible increase in the existing downstream flood flow depths and velocities. Also, under an extreme flood condition, it

is expected that utilisation of downstream infrastructure would be suspended until flood levels subside eliminating any risk for loss of life.

Conversely, a breach under the SDF condition is expected to induce considerable incremental flows into the downstream receiving area, especially during the dry season due to the ephemeral nature of Chinnamon Creek. The SDF dam break analysis has been conducted for the worst-case theoretical scenario, which comprises “full” storages at the end of Stage 7 TSF filling. The analysis is a two-stage process:

- (i) Firstly, the initial loss of impounded decant pond water (5,100 ML of RWP water, plus 3,700 ML of decant pond water in the TSF); followed by
- (ii) The run-out of the flowable portion of the deposited tailings through the breach.

The above configuration is an extremely conservative case valid only at the very end of TSF operations, and requiring the TSF decants to operate during an extreme rainfall event large enough to fill the RWP then become instantaneously blocked, accumulating the maximum amount of water behind the TSF embankment. In reality, this would be very unlikely to occur, without a significant long-term change to the specified TSF and RWP water management practices. Nevertheless, the adopted configuration is considered valid for the purposes of understanding the “worst case” theoretical incremental effects of a coupled TSF and RWP dam break on the downstream receiving environment.

#### 6.4.4 Summary of Outcomes - Initial Flood Wave

The following is a summary of the key outcomes from the initial flood water release component of the “worst case” dam break analysis. Refer to **Appendix D** for a detailed discussion.

- Key outcomes with respect to the major infrastructure are presented in **Table 6.2**. It can be seen that the maximum depth of inundation and flow velocity reduces along Chinnamon Creek prior to reaching the Railway Bridges, where the topography narrows the inundation width slightly.

**Table 6.2**  
**Dam Break Decant Pond Flood Inundation Results**

Location	Max Inundation Width (m)	Max Depth (m)	Max Velocity (m/s)	Time to Inundation (hrs)
North Star Borefield Road	450	3.3	3.3	0.5
Pilgangoora Access Road	650	2.1	1.6	8.0
Port Hedland - Wittenoorn Road	850	1.3	1.0	11.0
Railway Bridges	600	2.0	0.8	11.5

- The maximum area of inundation within the modelled extents is 4,450 hectares, prior to reaching the model boundary 60 km to the north-west.
- Modelled flood velocities immediately downstream of the RWP Embankment are up to 5 m/s. Such velocities are capable of causing significant scouring and erosion, as well as bulk transport of solid particles such as cobbles and boulders.

- The velocity in Chinnamon Creek further downstream reduces to less than 1.3 - 2.0 m/s, while the velocity for the flood in the Turner River is generally below 1.0 m/s. Such velocities would generally only cause minor scouring and erosion, with limited potential for vegetation stripping or bulk transport of solid particulates.
- Based on the height of the railway bridges (4.3 m and 6 m), the flood will not inundate the railway tracks, nor would the velocities cause damage to these structures.
- It is also noted that the flood wave will take 8 hours to reach the nearest public road, by which time evacuation notifications would have been initiated.

#### 6.4.5 Summary of Outcomes - Tailings Runout

One of the key considerations in the “worst case” dam break modelling is the vertical extent of the breach propagation. This is an engineering judgement, based on construction materials and compaction methodology; and the erosive actions of mobilised materials flowing through the breach.

The TSF embankment will be geomembrane-lined and constructed predominantly of quarried rockfill. The rockfill will be non-erosive, free of excessive weathering, and individual particles will be sound, hard and durable. Compaction and quality control will be to a high standard.

For the adopted initial conditions, a vertical breach depth of approximately 4 m will reach the final tailings beach surface, and will result in the outflow of all stored decant pond water. Beyond that level, it is estimated that the breach will continue to propagate as the flowable proportion of the deposited tailings mobilise, but will reach a point of equilibrium with increased depth as higher strength, more consolidated tailings are exposed. Another factor which will limit the vertical extent of the breach is the increasing width of the embankment rockfill with depth.

For the dam break model, the TSF embankment has therefore been assumed to breach to half of its total height (extending 33 m below the tailings surface level).

Estimating the proportion of deposited tailings likely to be mobilised and released through the embankment breach is the most difficult and subjective analysis step. The adopted approach has been to estimate a post-failure slope, with the gradient being a function of the consolidation density and post-liquefied shear strength profiles within the tailings.

Based on the North Star tailings properties, the estimated tailings failure slope within the TSF resulting from a breach-induced mobilisation of the deposited tailings is approximately 4%. This equates to a released volume in the order of 16 Mm<sup>3</sup>.

In order to estimate the extent of flow of mobilised tailings, a runout analysis was performed using a two-dimensional, non-Newtonian method as detailed in **Appendix D**. The liquefied flow of the tailings material is assumed to follow the Bingham plastic model (i.e. flow commences once the liquefied shear strength is exceeded). The results of the analysis identify the equilibrium gradient of the mobilised tailings, the overall distance the liquefied tailings will flow, the time taken for this to occur and the expected velocity of the flow.

The analyses indicate that the top surface of the tailings runout will most likely conform to an average equilibrium longitudinal gradient of 2.4%. For the adopted breach scenario, the tailings are expected to flow of the order of 1,600 m downstream of the TSF Main Embankment, passing through the fully-breached RWP Embankment area. This equates to an inundation area of 128 ha.

#### 6.4.6 Summary of Outcomes - Discussion

With respect to the decant water pond release and tailings run-out estimates described above, it is important to recognise the following:

- Due to the configuration of the TSF, the deepest point of the TSF decant pond will be at the TSF embankment. Hence, a “worst case” dam break will result in the outflow of practically all stored water prior to mobilisation of the underlying tailings.
- A review of case histories of tailings dam failures does, however, show that whilst all of the water flows out of the dam, there is generally a degree of mixing with the tailings.

As the breach develops and the water outflow reaches its peak discharge rate, the water entrains some tailings, which are carried as suspended solids. The released water flows for many kilometres downstream, to wherever the topography leads it, and the suspended tailings eventually settle out.

This mechanism is not covered in the above analyses.

#### 6.4.7 Estimated Population At Risk (PAR)

The assessment of PAR is largely dependent on a qualitative risk assessment of the temporal-spatial probability of a person or persons being in the path of the flood flow at the time of the breach, and whether they take action to move out of the inundation area prior to the flood flow passing. A qualitative risk assessment has been carried out, taking into the account the following:

- The routine management of the TSF and the subsequent high degree of awareness of the condition of the TSF, and of developing extreme weather conditions;
- The emergency management communications with local authorities (to be developed in the operating manual);
- The fact that there are no permanent dwellings or structures within 45 km downstream of the TSF;
- The likely actions a person would take to move out of the local flow channel and move to higher ground in the period of time for the flow from the facility to fully develop; and
- The general remoteness of the area.

The primary public road users downstream of the TSF and RWP are as follows:

- (i) Borefield road - this is an IBO road, with only intermittent use and no public access.
- (ii) Pilgangoora mining area access roads.

The Altura operated Pilgangoora Lithium Mine access road meets the Pilbara Minerals (PLS) access road (from Great Northern Highway) to the east of Chinnamon Creek. PLS will use the upgraded Pippingarra road to transport concentrate. The Altura Camp is located at the Great Northern Highway, so there is also daily movement of personnel along the Pilgangoora access road.

Based on the above, an approximate estimate of traffic over the Chinnamon Creek crossing on the Pilgangoora access road is:

- Concentrate haulage: 1 x Quad Road Train per two hours in both directions.

- Personnel Movement:
  - PLS - weekly shift change = 3 x buses each way;
  - Altura - daily trip to camp = 4 x buses each way;
  - General LVs - approx. 10 per day; and
  - Freight deliveries - approx. 6 trucks per week each way.

Traffic over the Pippingarra road crossing (near to BHP Rail bridge) will not include the Altura daily buses but could be similar otherwise.

It is considered that the level of risk that people will be in the path of the breach flood flow at the time of the breach is very low, nevertheless, based on the potential for vehicles crossing Chinnamon Creek on the Pilgangoora access road, it is considered that a PAR of 1 to 10 should be conservatively adopted.

## 6.5 Embankment Failure Consequence Category

### 6.5.1 Criteria

The ANCOLD guidelines [28] set out the criteria for selection of the relevant Severity Level Impact due to dam failure, which are replicated below in **Table 6.3**. The ANCOLD guidelines also set out the criteria for selection of the relevant consequence category based on the adopted PAR (**Section 6.4.7**) and Severity Level Impact, which are replicated as **Table 6.4**.

The severity level impact matrix was provided to IBO, to ascertain appropriate severities regarding the likely impacts on the mine operations and other impacts in the event of an embankment failure. The responses from IBO are highlighted in **Table 6.3**.

**Table 6.3**  
**Severity Level Impact - ANCOLD [28]**

Damage Type	Minor	Medium	Major	Catastrophic
Infrastructure (dam, houses, commerce, farms, community)	<\$10M	\$10M - \$100M	\$100M - \$1B	>\$1B
Business importance	Some restrictions	Significant impacts	Severe to crippling	Business dissolution, bankruptcy
Public health	<100 people affected	100 - 1000 people affected	<1000 people affected for more than one month	>10,000 people affected for over one year
Social dislocation	<100 person or <20 business months	100 - 1000 person months or 20 - 2000 business months	>1000 person months or >200 business months	>10,000 person months or numerous business failure
Impact area	<1km <sup>2</sup>	<5km <sup>2</sup>	<20km <sup>2</sup>	>20km <sup>2</sup>
Impact duration	<1 (wet) year	<5 years	<20 years	>20 years

Damage Type	Minor	Medium	Major	Catastrophic
Impact on natural environment	Damage limited to items of low conservation value (e.g. degraded or cleared land, ephemeral streams, non-endangered flora and fauna). Reclamation possible	Significant effects on rural land and local flora & fauna. Limited effects on: A. Item(s) of local & state natural heritage B. Native flora and fauna within forestry, aquatic and conservation reserves, or recognised habitat corridors, wetlands or fish breeding areas	Extensive rural effects. Significant effects on river system and areas A & B. Limited effects on: C. Item(s) of National or World natural heritage D. Native flora and fauna within national parks, recognised wilderness areas, RAMSAR wetlands and nationally protected aquatic reserves.  Remediation difficult	Extensively effects areas A & B. Significantly affects areas C & D. Remediation involves significantly altered ecosystems.

From **Table 6.3** the Severity Impact Level for TSF / RWP Embankment failure is assessed as **Major**. This level is then used in **Table 6.4** along with the PAR to assess the dam failure consequence category.

**Table 6.4**  
**Assessed Consequence Category - ANCOLD [28]**

Population at Risk (PAR)	Severity of Damage and Loss			
	Minor	Medium	Major	Catastrophic
<1	Very Low	Low	Significant	High C
>1 to 10	Significant (Note 2)	Significant (Note 2)	High C	High B
>10 to 100	High C	High C	High B	High A
>100 to 1,000	(Note 1)	High B	High A	Extreme
>1,000		(Note 1)	Extreme	Extreme

*Note 1: With a PAR in excess of 100, it is unlikely damage will be minor. Similarly with a PAR in excess of 1,000 it is unlikely damage will be classified as Medium.*

*Note 2: Change to "High C" where there is the potential of one or more lives being lost. The potential for loss of life is determined by the characteristics of the flood area, particularly the depth and velocity of flow.*

### 6.5.2 Assessment

The North Star TSF2A and RWP embankment failure consequence category, in accordance with **Table 6.4** given a PAR > 1 to 10 and a "Major" severity of damage or loss is **High C**.

## 6.6 Environmental Spill Consequence Category

The environmental spill consequence category as a result of water release is generally lower than the embankment failure consequence category, particularly if no loss of life is expected and water spilling from the facility is controlled by spillways.

For this project, the decant water is not toxic or likely to become acidic (refer to **Section 5.5**), and the impact on the natural environment as a result of unseasonal flow along the Turner River channel is expected to be “Minor “(based on **Table 6.3**).

Hence an environmental spill consequence category of **Low** is considered appropriate for the North Star TSF2A and RWP facility.

## 7 TSF & RWP PLANNING

### 7.1 Tailings Storage and Water Management Concept

An overall site plan of the North Star TSF 2A and RWP is shown in **Figure 114185.14\_002**. The tailings storage concept is relatively simple, and makes use of the inherent characteristics of the selected Site 2A valley. Tailings will be pumped from the process plant to the head of the TSF at the eastern end of the valley, via a tailings pipeline access road which will be constructed around the south-eastern side of the TSF.

The general arrangement for the TSF is shown in **Figure 114185.14\_003**. This comprises down-valley discharge on a rotational basis from a total of six (6) discharge points distributed around the south-eastern side of the valley, such that the tailings beach low point is maintained at the TSF main embankment which divides the valley into the TSF (east) and the RWP (to the west). Confinement for the RWP is achieved by the construction of the RWP Embankments within the valley outlet on the western side.

A decant system, comprising two decant chutes and outlet pipes (the North Decant and the South Decant), will be constructed on the upstream face of the main TSF embankment for the removal of surface water, including decant water and storm runoff, from the TSF impoundment. The decant system will transfer water to the RWP, located immediately downstream of the TSF.

The return water pumping system, consisting of a pontoon-mounted pumping station, will be located within a channel excavated to access the central part of the RWP impoundment from the southern perimeter, near the junction with the TSF where an associated access road will be located. This will provide recovery of decant water and stormwater runoff for reuse at the process plant.

The TSF and RWP extents have been limited to within the current IBO North Star lease tenure. Containment will almost entirely be provided by the valley topography, with the only additional retaining features being the RWP embankments on the west side of the RWP, two small saddle dams on the north-eastern side of the TSF, and the Eastern Limb Sterilisation Dumps which will progressively infill the small gullies along the southern side of the TSF.

The TSF/RWP Site 2A valley has a large combined catchment of over 20 km<sup>2</sup>, with no diversion drains proposed. This catchment is however a benefit of the site, in that it can be used to collect and temporarily store runoff during the wet season for use in the process plant.

### 7.2 TSF Staging Overview

As defined in **Section 2.1**, it is proposed that the TSF embankments will be constructed in seven stages over the 20-year TSF operational life to minimise up-front capital costs. Staging will be 2 years initially, then 3 years thereafter using the “downstream” method of raise construction.

In order to reduce costs in the early stages of the TSF, Stage 1 is divided into two stages (A and B), as detailed in **Section 7.3**. Details of the subsequent TSF stages are described in **Section 7.4**.

The required TSF embankment crest level is a function of the tailings beach toe elevation with time, to which is added the necessary operating, flood and regulatory freeboard requirements. A detailed description of the procedure utilised to define these freeboard requirements and arrive at the adopted embankment stage crest levels is provided in **Section 8**.

### 7.3 Start-Up (Stage 1) Scheme Description

Stage 1 has been divided into Stage 1A and Stage 1B to reduce start-up costs by subdividing the TSF2A valley into a larger, northern and smaller, southern arm by the construction of a small sacrificial bund equipped with a wide crest and flat downstream slope to enable it to be ultimately overtopped.

Stage 1A deposition will be directed into the northern arm, with the sacrificial bund height sized to provide 16 months storage capacity. This reduces the start-up construction activities to the northern component of the main TSF embankment (referred to as Main Embankments A and B), and enables the construction of the large southern embankment component (Main Embankments C and D) and south decant to be delayed by approximately 12 months.

The sacrificial bund also functions as the Stage 1A emergency spillway.

By 16 months, Stage 1B construction of main embankments C and D and the south decant will be completed to Stage 1 height, providing southern valley arm tailings storage capacity once the sacrificial bund overtops as part of Stage 1B deposition.

### 7.4 Subsequent TSF Stages

Beyond Stage 1B, tailings management within the North Star TSF 2A will be relatively straightforward, which is one of the beneficial features of the adopted scheme. The tailings beaching effect discussed in **Section 5.3** will be exploited together with the long, broad configuration of the storage impoundment to create a sub-aerially deposited, essentially consolidated tailings stack.

After year 2, subsequent TSF raises will be completed at 3 year intervals until the TSF embankments have reached the final (i.e. Stage 7) crest height.

Tailings deposition will occur from the southern and eastern sides of the valley via end-point discharge from the nominal locations shown in **Figure 114185.14\_003**. **The multiple discharge points (up to four in total) will be commissioned progressively and rotated over the life of the storage, to allow an even filling distribution and avoid “dead spots” where ponded water would become trapped.**

### 7.5 Eastern Limb Sterilisation Dumps

As shown in **Figure 114185.14\_003**, the southern side of the TSF 2A site consists of six narrow valleys which are aligned north - south, and separate the TSF from the impoundment and the IBO Eastern Limb North Pit. Tailings will naturally deposit in these valley over time, resulting in the formation of isolated decant ponds in close vicinity to the adjacent pit. This situation introduces risks of inrush to the pit, hence IBO have designed a series of six waste rock dumps (Eastern Limb Sterilisation Dumps).

These dumps (indicated as Areas 1 to 6 in **Figures 114185.14\_004 to 114185.14\_011**) will be progressively developed within each valley to provide confinement of tailings and decant water, and maintain a suitably wide sterilisation width between the TSF and the Eastern Limb North Pit. The rate of dump development will need to maintain the dump levels in each area a nominal 3 m above the level of the adjacent tailings. The required dump levels are indicated for the end of each TSF stage in **Figures 114185.14\_004 to 114185.14\_011**.

## 7.6 Water Balance Modelling

Given the site configuration and inherent water management issues, a comprehensive water balance model was developed by ATCW for the TSF 2A scheme, to investigate specific design parameters for the TSF and RWP.

GoldSim modelling software was used to develop and run a daily water balance model. Actual daily climatic records were used to statistically synthesise 1,000 years of climate data. 1,000 realisations were run to stochastically analyse all possible climatic scenarios.

Inputs into the TSF system include tailings bleed and catchment runoff, with outputs being decant delivery to the RWP and evaporation. RWP inputs include TSF decant and catchment runoff, with outputs comprising process water return and evaporation. Following saturation of the impoundment subsoils, seepage from the TSF to the RWP and RWP to the downstream catchment was assumed to be zero.

The following are the key objectives of the water balance study:

- (i) Determine the probable likelihood the RWP embankment spillway will be engaged;
- (ii) Assess the water level fluctuation and hence the statistical range of expected water levels in the RWP due to the regional climate at the project site;
- (iii) Determine the probable likelihood of internal TSF Spillway discharge;
- (iv) Determine the probable performance of the TSF decant system;
- (v) Determine the probable availability of water in the RWP for return to the process plant; and,
- (vi) Determine the required capacity of the RWP to adequately provide sufficient storage for the required design rainfall event, and manage spillway operations to within the project regulatory requirements.

A detailed presentation of the water balance methodologies, inputs and outcomes is provided in the **Appendix E, Rainfall Analysis and Water Balance Report**.

## 8 TSF EMBANKMENT GEOMETRIC DESIGN

### 8.1 TSF Capacity

Based upon a nominal 20 year operational life, a tailings tonnage of 538 million tonnes (refer to **Section 2.1**), and an OID of 1.64 to 1.7 t/m<sup>3</sup> at the end of filling (refer to **Section 5.2**), the required minimum design TSF 2A capacity is 318 million m<sup>3</sup>.

As outlined in the TSF staging schedule in **Chart 8.5**, the actual capacity of the TSF at the end of filling the final Stage 7 is 330 million m<sup>3</sup>, equating to 558 million tonnes (i.e. slightly higher than the minimum requirement).

### 8.2 TSF Embankment Crest Level Design Procedure

The TSF Embankment crest level is a function of the tailings beach toe elevation with time, plus the required pond storage, operating freeboard and spillway depth. To identify the required final crest level for each TSF Embankment stage, the following design process was conducted:

- (i) Assign the design Consequence Category and hence the relevant hydrologic design criteria (refer to **Section 6**);
- (ii) Conduct tailings beach development modelling for deposition within the TSF, in terms of rate of rise at the TSF Embankment, tailings level and volume as a function of time (refer to **Section 8.3**);
- (iii) Estimate the freeboard requirements for the TSF (refer to **Sections 8.4** and **8.5**) to establish the required spillway invert level for each stage; and
- (iv) Complete analyses and design of the TSF Embankment spillways (refer to **Section 8.6**) to establish the TSF Embankment crest level for each stage (refer to **Section 8.7**).

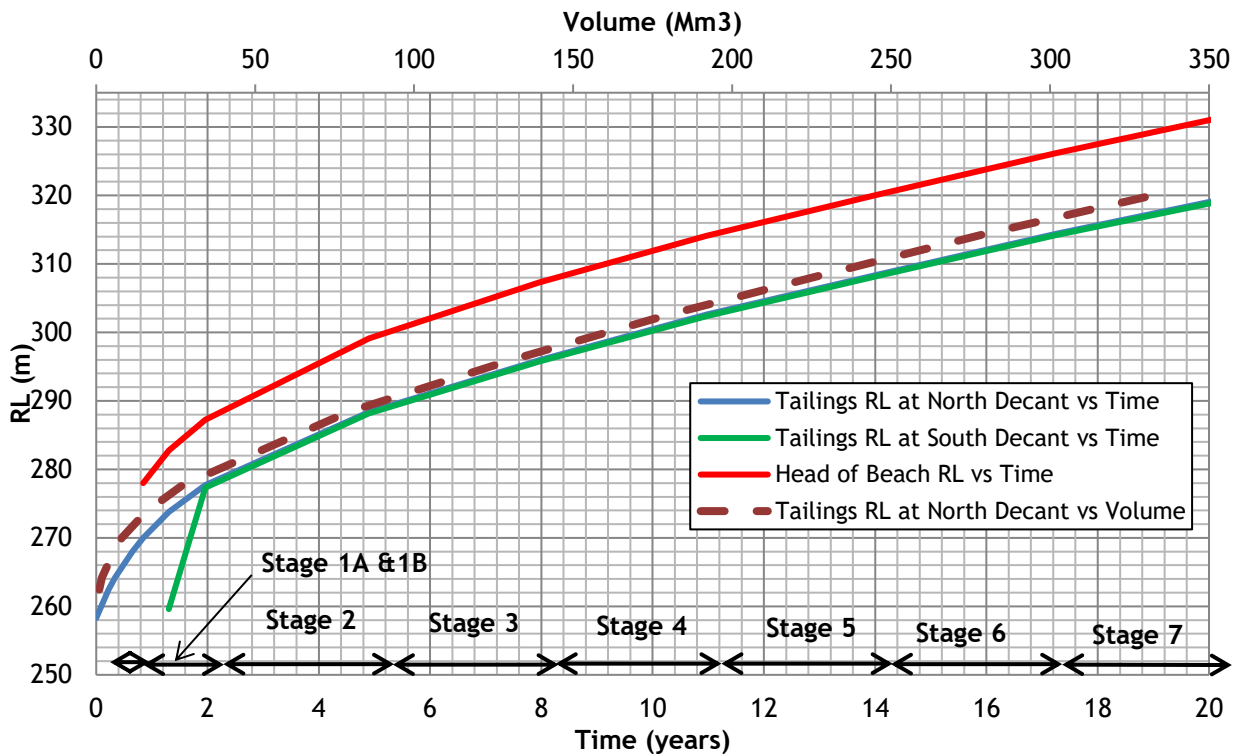
### 8.3 TSF Filling

In order to understand the rate of TSF filling resulting from the various tailings input parameters (detailed in **Section 2**), tailings beach models were developed for incremental head of beach levels covering the full required TSF 2A filling life. These beach models were developed using the tailings beach slopes estimates outlined in **Section 5.3**. The resultant TSF filling curves, in terms of rate of rise, tailings level and volume as a function of time, are depicted in **Chart 8.1**.

From the filling curve presented in **Chart 8.1**, it is evident that the base of the TSF valleys will fill quickly, and the beach toe (at the TSF main embankment) will rise moderately quickly during Stage 1 (average 11.7 m per year). However, due to the topography of the valley, the rate will reduce markedly, to an average of 2.8 m per year from Stage 2 to Stage 4.

The tailings level is predicted to rise, on average, at a rate of 1.9 m per year from Stage 5 to Stage 7 (end of filling). Rates of rise of this order are considered beneficial with respect to achieving optimum density in the deposited tailings (as outlined in **Section 5.2**).

**Chart 8.1**  
**TSF Filling Vs Time Relationship**



## 8.4 TSF Freeboard Design Criteria

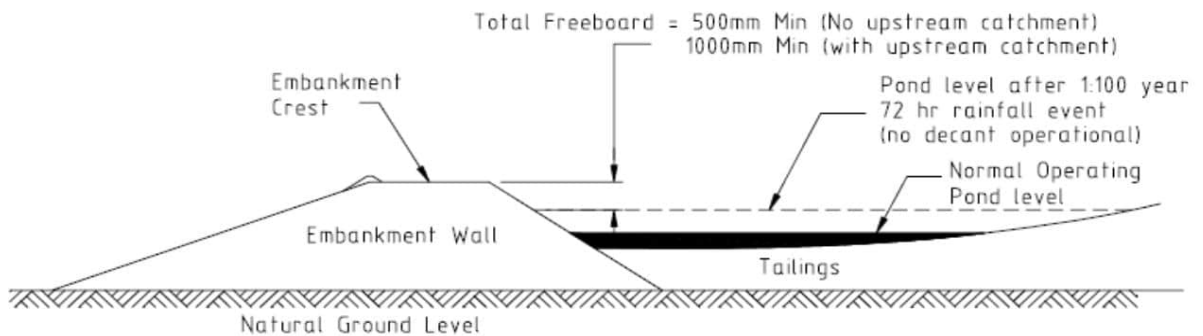
### 8.4.1 Adopted Approach

The specific design requirements for flood freeboard (based on DMIRS [27]) and emergency spillway capacity (based on ANCOLD [28]) for the TSF are presented below.

- Embankment Freeboard (DMIRS):
  - “Normal” Operating Pond - for the adopted decant structures operating under normal conditions (i.e. no extreme rainfall catchment runoff and bleed water is reporting to the decant pond) the design pond depth is 400 mm; plus
  - “Wet” Freeboard - 1:100 Annual Exceedance Probability (AEP), 72 hr run-off superimposed on normal operating pond (assuming that the decant facility is not operational for the duration of the event); plus
  - “Total” (dry) freeboard of an additional 1 m if no spillway is provided. As a spillway is considered an appropriate additional safety measure for down-valley TSFs such as proposed for Site 2A, the Total (dry) freeboard has been incorporated into the spillway depth.

Diagram 8.1 illustrates the DMIRS freeboard definitions.

**Diagram 8.1  
DMIRS Freeboard Definition**



- **Emergency Spillway Depth (ANCOLD):**
  - With respect to emergency spillway design criteria, the ANCOLD guidelines require High C consequence category facilities to be able to pass the maximum runoff from the critical duration storm with a minimum AEP of 1 in 100,000 years.
  - No wave run-up allowance has been included, given the relatively short fetch distance and the fact that the critical design period is only at the end of filling of each stage.

#### 8.4.2 Resultant TSF Freeboard Design Criteria

As explained in **Section 7.2**, the TSF will be constructed and operated in seven stages, with Stage 1 being divided into Stage 1A and 1B. Stage 1B will have an eight-month term of operation. Given the very limited operation life, that Stage 2 construction will be underway during this entire period, and the reduced consequences of failure (relative to subsequent stages), a 1 in 1,000 AEP design flood peak level has been adopted for this stage.

For Stage 1A and Stages 2 to 7, a more conservative 1 in 100,000 AEP design flood peak level has been adopted due to the longer term of operation. A summary of the TSF freeboard design criteria is shown in **Table 8.1**. The dry freeboard has been incorporated within the spillway freeboard allowance, given that this constitutes an internal spill (i.e. a transfer of water to the RWP in a similar manner to the decant operation), with no offsite environmental discharge.

**Table 8.1  
TSF Freeboard Design Criteria**

TSF Stage	“Normal” Operating Pond	Wet Freeboard	Dry Freeboard	Spillway Capacity & Depth
Stage 1A	400 mm	1:100 AEP, 72 hr duration	Included in spillway freeboard allowance	1 in 100,000 AEP, critical duration
Stage 1B				1 in 1,000 AEP, critical duration
Stage 2 to 7				1 in 100,000 AEP, critical duration

## 8.5 Wet Freeboard

Each stage of the TSF embankment has been designed to store the run-off of a 1:100 AEP, 72 hour storm superimposed on the normal operating pond (i.e. 400 mm depth), assuming that the decant facility is not operational for the duration of the event.

The exception to this is Stage 1A, which spills via the sacrificial bund at a crest RL 276 m, which is 5.6 metres lower than the Stage 1A TSF embankment crest height.

The adopted rainfall depth for a 1:100 AEP, 72 hour storm is 394 mm, based on the current design rainfall data derived from BOM and presented in **Chart 8.2**.

The catchment areas for the freeboard calculations are those used in the water balance modelling (refer to **Section 7.5**), and were divided into dry beach area, wet beach area, and natural catchment area. Based on past experience, it has been assumed that the dry beach area comprises 65% of the total tailings beach area.

Long term catchment yields (effectively runoff coefficients) of 0.37 and 0.41 were derived from the water balance modelling (refer to **Section 7.5**) for the dry beach and natural catchment area, respectively. A runoff coefficient of 1 was adopted for the wet beach area, given that it is saturated by definition.

The total wet freeboard for the TSF main embankment has been calculated for Stage 1A, 1B, Stage 2 and Stage 7 (Final), as presented in **Table 8.2**.

**Table 8.2**  
**TSF Wet Freeboard Calculations**

Stage	Catchment	Area (ha)	Wet Freeboard (m)
1A	Dry Beach	136	N/A (Sacrificial Bund in operation)
	Wet Beach	73	
	Natural Catchment	966	
1B	Dry Beach	201	2.2
	Wet Beach	108	
	Natural Catchment	1001	
2	Dry Beach	373	2.0
	Wet Beach	201	
	Natural Catchment	730	
Final	Dry Beach	408	2.0
	Wet Beach	585	
	Natural Catchment	315	

## 8.6 Emergency Spillway Hydrology

### 8.6.1 Establishment of Design Rainfall

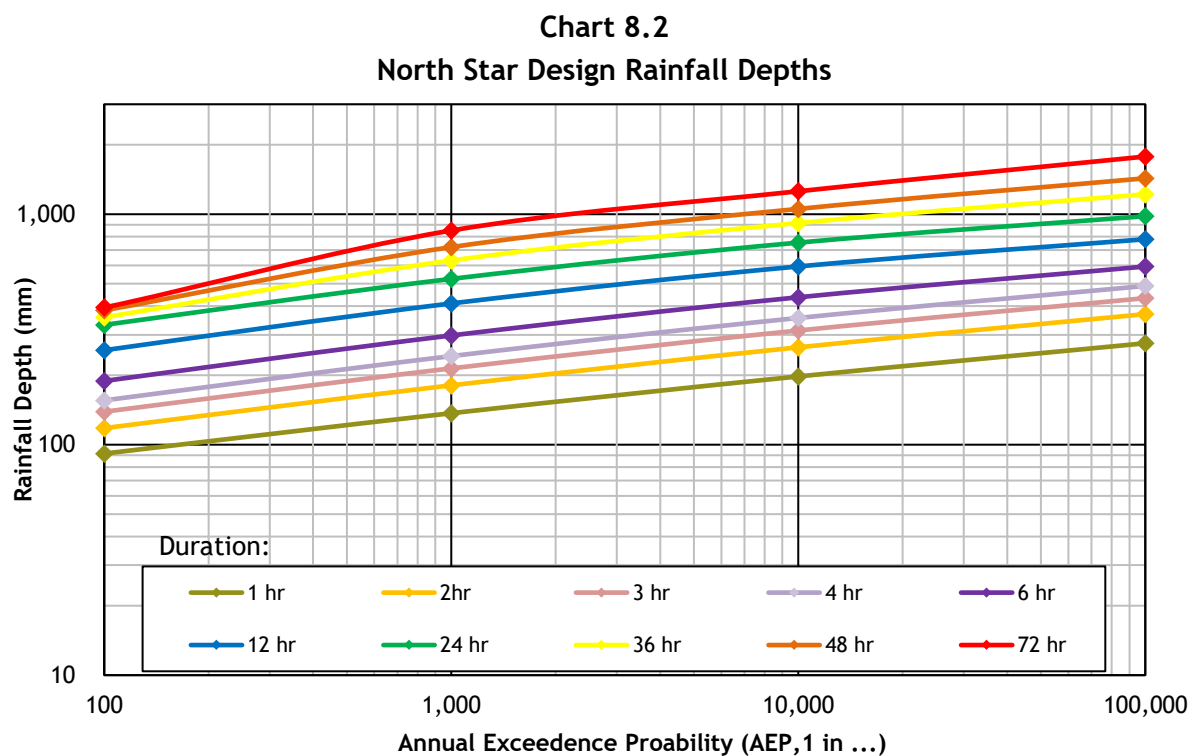
The procedures outlined in Australian Rainfall and Runoff (ARR) [29] were utilised in the calculation of the 1 in 10,000 year and 1 in 100,000 year AEP design rainfall depths for durations

of 1 hour to 72 hours. The procedure involves first calculating 1 in 50 year and 1 in 100 year rainfall, then the Probable Maximum Precipitation (PMP) using the GSDM method [30] and the ARR preliminary prediction equations.

The design rainfall events are then interpolated from the calculated data sets, and temporal distributions applied using the parameters recommended by ARR [29]. In accordance with ARR [29], the following methods have been adopted in assigning temporal patterns to specific rainfall durations:

- (i) GSDM [30] for short durations (i.e. up to 6 hour rainfall durations), and;
- (ii) GSTMR [31] for intermediate durations (i.e. up to 24 hour rainfall durations).

The estimated rainfall depths for various AEPs and durations are shown in **Chart 8.2**.



### 8.6.2 Derivation of Design Flood

The inputs in the derivation of the design flood for a given duration are the design rainfall event and the design inflow hydrograph. The inflow hydrograph for a particular duration storm is the summation of individual component hydrographs from the pond and the surrounding catchment.

The total catchment area of the TSF impoundment and contributing natural catchment is 1,352 ha. As the catchment is ungauged, the flood hydrograph has been modelled using the synthetic unit hydrograph technique. A synthetic unit hydrograph was constructed for each individual contributing catchment using the key parameters of the catchment (time of concentration, area, average slope), in accordance with the method described in ARR [29]. The tailings impoundment flood hydrograph was derived using the time-area histogram method.

### 8.6.3 Spillway Flood Routing

#### 8.6.3.1 General Methodology

Once the design flood for particular storm durations was derived, spillway sizing was undertaken by routing the flood through the storage. This was accomplished using the storage indication method, a direct numerical procedure which is described in ARR [29].

A flood wave passing through the storage is both delayed and attenuated as it enters and spreads over the pool surface. The surcharge storage is gradually released over the spillway. The outflow depends on the spillway configuration, as well as on the surcharge storage characteristics. In order to perform satisfactorily, the spillway configuration must be able to pass the critical duration design AEP flood without overtopping of the embankment crest.

Floods from storms of increasing duration were progressively routed through the storage, until a peak outflow was obtained. The spillway configuration (i.e. depth and width) was considered satisfactory only if the capacity was greater than the critical peak outflow.

#### 8.6.3.2 TSF Embankment Spillway Routing Results

The results of the TSF embankment spillway routing calculations are summarised in **Table 8.3**.

**Table 8.3**  
**Spillway Flood Routing Results For TSF**

Stage	Design Storm (AEP)	Critical Duration Event	Results		
			Parameter	Unit	Outcome
1A	1:100,000	6 hour	Sacrificial Bund Crest Width:	(m)	45
			Peak Flood Depth:	(m)	1.9
			Peak Outflow:	(m <sup>3</sup> /s)	236
1B	1:1,000	18 hour	Spillway Width:	(m)	20
			Peak Flood Depth:	(m)	1.3
			Peak Outflow:	(m <sup>3</sup> /s)	54
2	1:100,000	12 hour	Spillway Width:	(m)	50
			Peak Flood Depth:	(m)	1.4
			Peak Outflow:	(m <sup>3</sup> /s)	147
7 (Final)	1:100,000	18 hour	Spillway Width:	(m)	50
			Peak Flood Depth:	(m)	1.2
			Peak Outflow:	(m <sup>3</sup> /s)	114

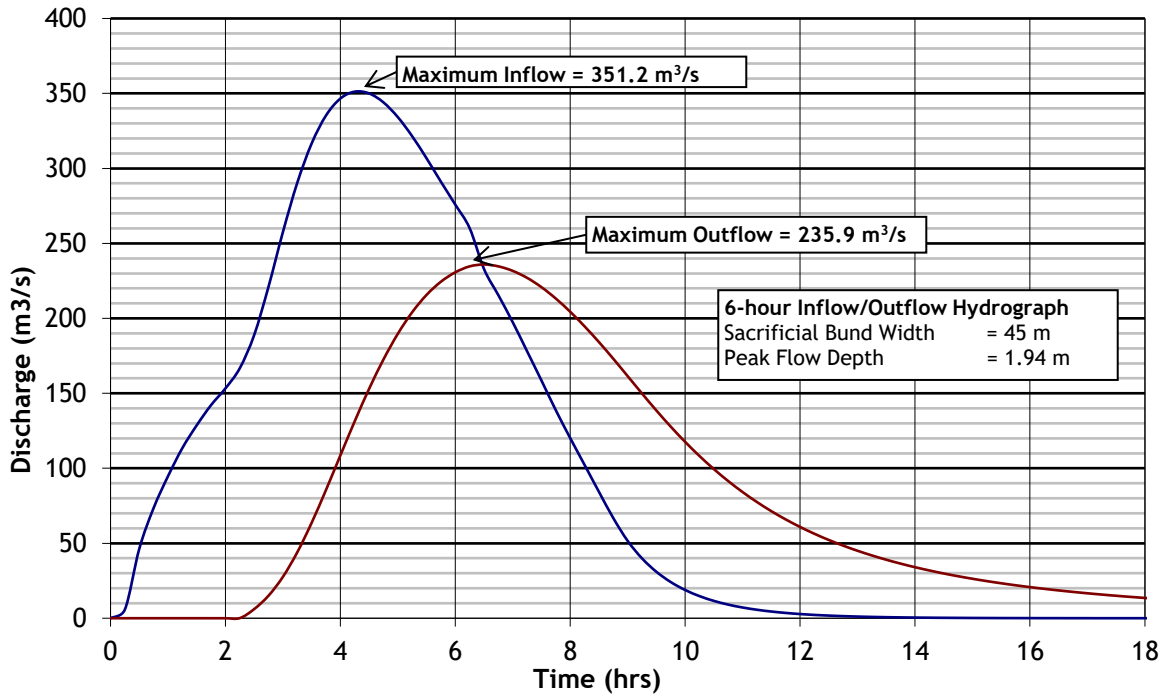
A sacrificial bund, built to crest level RL 276 m, will act as the spillway for Stage 1A. The bund has a crest length (i.e. effectively the spillway width) of 45 m.

Based on the results in **Table 8.3**, a 20 m wide emergency spillway has been adopted for the Stage 1B TSF Embankment.

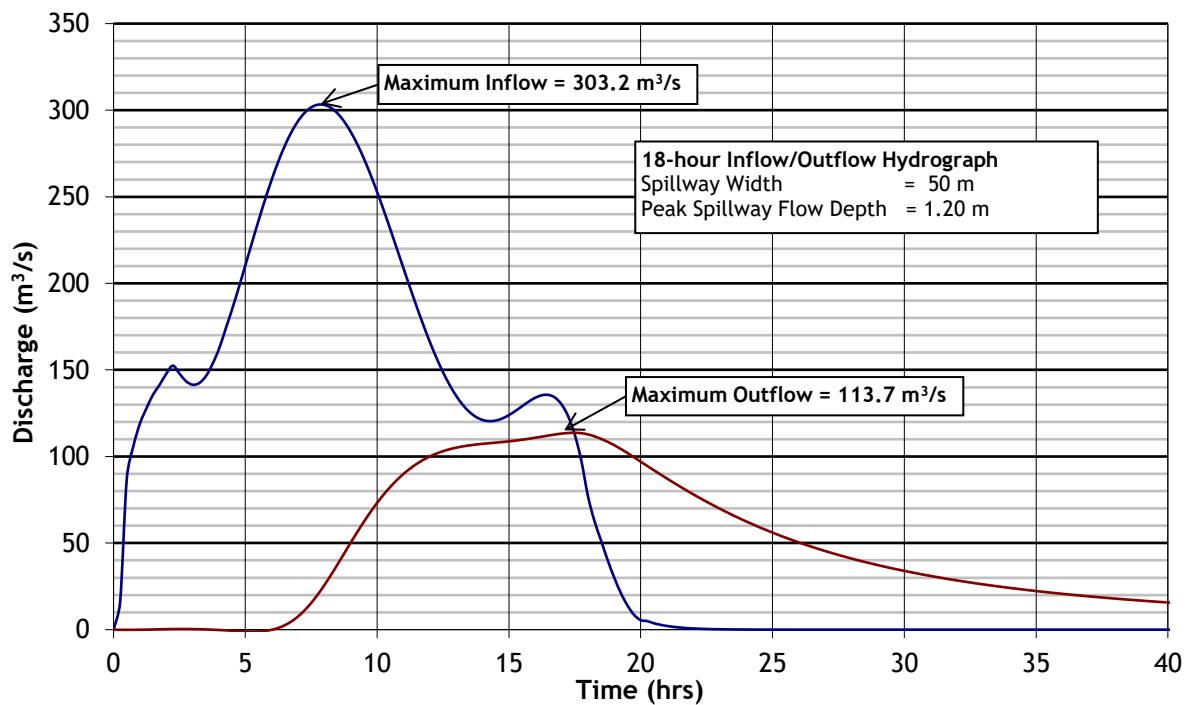
A 50 m wide emergency spillway has been adopted for each subsequent stage of the TSF embankment. No additional freeboard allowance for wave run-up has been considered necessary as the critical design period is only at the end of filling for each stage.

The inflow/outflow hydrograph for the Stage 1A TSF embankment critical duration (6 hour) storm is presented in **Chart 8.3**, and for the Stage 7 (final) TSF embankment critical duration (18 hour) storm event in **Chart 8.4**.

**Chart 8.3**  
**Stage 1A TSF Spillway/Sacrificial Bund - Inflow/Outflow Hydrographs**



**Chart 8.4**  
**Stage 7 TSF Spillway - Inflow/Outflow Hydrographs**



## 8.7 Embankment Crest Levels & Staging

The required TSF embankment crest level is a function of the tailings beach elevation with time, and the requirement to provide flood freeboard protection to the embankment.

As outlined in **Section 8.2**, the procedure to establish the design TSF embankment crest level for each stage is defined as follows:

End of filling stage tailings surface (at main embankment) + “Normal Operating Pond” + Wet Freeboard + Peak Spillway Flood Depth.

Based on the assessments provided previously in this section, the embankment freeboard requirement for each TSF embankment stage is summarised in **Table 8.4**. The total depth between the tailings beach toe and the embankment crest represents the limiting operational pond depth below the embankment crest (this could be demarcated by a line painted on the BGM liner).

**Table 8.4**  
**TSF Freeboard Design Criteria**

TSF Stage	“Normal” Operating Pond	Wet Freeboard	Dry Freeboard	Spillway Depth	Total Depth
Stage 1A	0.4 m	N/A	Included in spillway freeboard allowance	N/A	N/A
Stage 1B		2.2 m		1.3 m	3.9 m
Stages 2 to 4		2.0 m		1.4 m	3.8 m
Stages 5 to 7					

As stated in **Section 8.4.1**, no wave run-up allowance has been made for the TSF Embankment. To account for this, the maximum tailings level at the embankment prior to the commencement of raising must not exceed the allowable levels shown in **Table 8.5**.

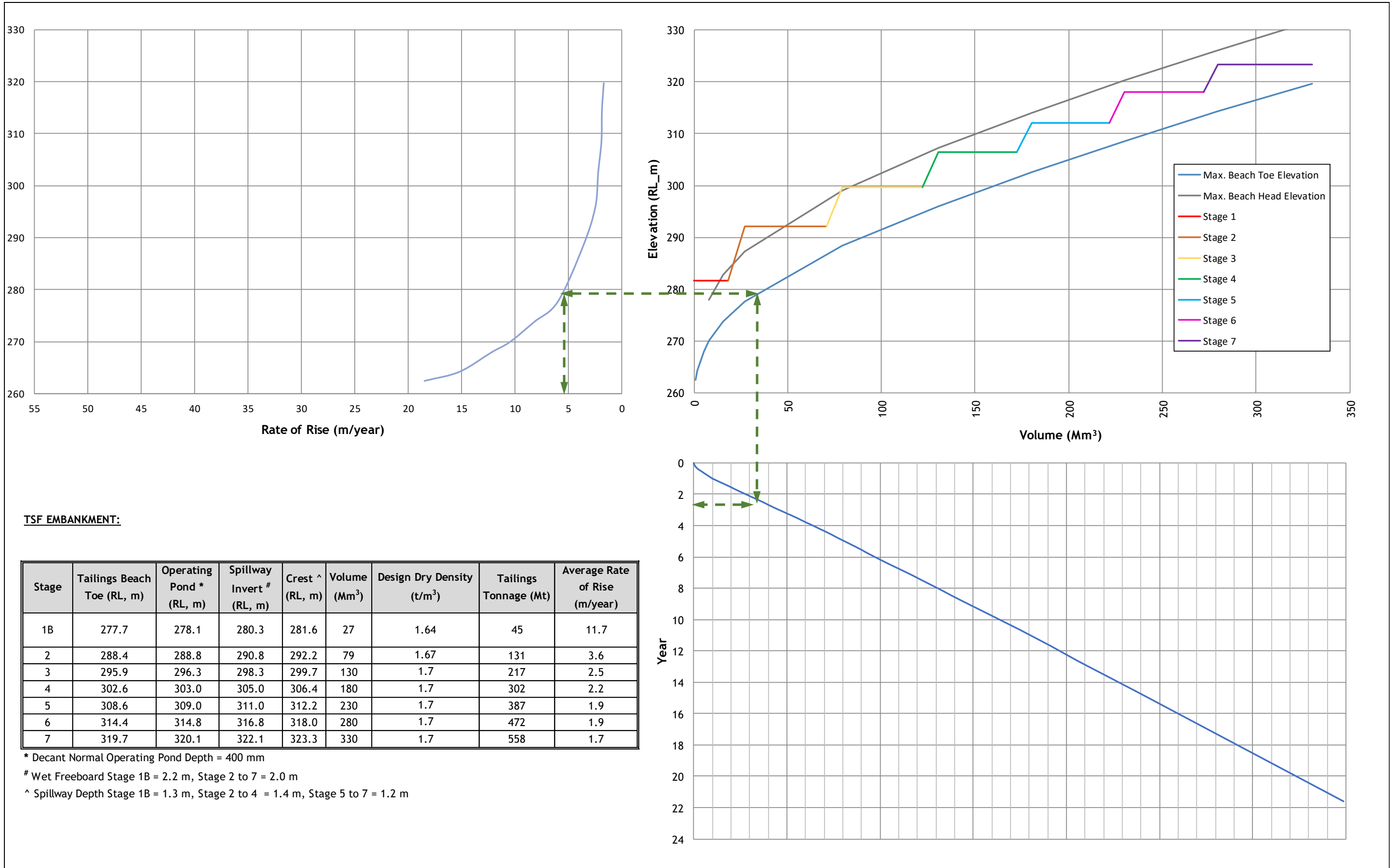
**Table 8.5**  
**Maximum Allowable Tailings Level Prior to Raising**

TSF Stage	Max. Allowable Tailings Level
Stage 1B	279.3
Stage 2	289.9
Stage 3	297.3
Stage 4	304.0
Stages 5	310.0
Stage 6	315.8
Stage 7	321.1

The stage capacity curves (SCC) for tailings deposition within TSF 2A, in terms of rate of rise, tailings level and volume as a function of time, are depicted in **Chart 8.5**. Also shown is the TSF embankment staging schedule.

General layouts of TSF 2A at the end of Stages 1A through to 7 are shown in **Figures 114185.14\_004 to 114185.14\_011** respectively.

**Chart 8.5**  
**TSF Staging - Time - Volume - Filling Relationship**



**TSF EMBANKMENT:**

Stage	Tailings Beach Toe (RL, m)	Operating Pond * (RL, m)	Spillway Invert # (RL, m)	Crest ^ (RL, m)	Volume (Mm³)	Design Dry Density (t/m³)	Tailings Tonnage (Mt)	Average Rate of Rise (m/year)
1B	277.7	278.1	280.3	281.6	27	1.64	45	11.7
2	288.4	288.8	290.8	292.2	79	1.67	131	3.6
3	295.9	296.3	298.3	299.7	130	1.7	217	2.5
4	302.6	303.0	305.0	306.4	180	1.7	302	2.2
5	308.6	309.0	311.0	312.2	230	1.7	387	1.9
6	314.4	314.8	316.8	318.0	280	1.7	472	1.9
7	319.7	320.1	322.1	323.3	330	1.7	558	1.7

\* Decant Normal Operating Pond Depth = 400 mm

# Wet Freeboard Stage 1B = 2.2 m, Stage 2 to 7 = 2.0 m

^ Spillway Depth Stage 1B = 1.3 m, Stage 2 to 4 = 1.4 m, Stage 5 to 7 = 1.2 m

## 9 RWP EMBANKMENT GEOMETRIC DESIGN

### 9.1 Required RWP Capacity

The required RWP storage capacity is a function of the operational intent and regulatory mine water storage requirements. Hence the adopted capacity has been the result of various trade-off studies based on the following considerations:

- Return Water - the combined TSF and RWP catchment of 20.6 km<sup>2</sup> is significant and when combined with the tailings bleed water inflows, will provide IBO with a valuable source of process water;
- Capital Cost - large water retaining embankments represent significant capital expenditure, hence the RWP cannot feasibly be constructed to store and return all inflows; and
- Spillway Discharge Risk and Regulatory Requirements - the RWP capacity cannot be set too low, as the return water pumping capacity would not be able to manage large wet season inflows and relatively frequent spillway discharges would result. It is understood that the current EPA approval conditions for the North Star project require that no environmental spillage into the downstream environment occurs under “normal” operating conditions. Consequently, the spillway for the RWP should only function as an emergency spillway to protect the embankments from overtopping during rare flood events.

### 9.2 RWP Embankment Crest Level Design Procedure

In order to identify the required final crest level for the RWP Embankment, the following design process was conducted:

- (i) Assign the design environmental spill consequence category and hence the relevant hydrologic design criteria (refer to **Section 6**);
- (ii) Undertake water balance modelling and trade-off studies to assign an appropriate return water pumping capacity, develop RWP inflow volume statistics (refer to Section 9.3) and assign a “normal” RWP pond operating level (refer to **Section 9.5**);
- (iii) Estimate the freeboard requirements for the RWP to be applied above the “normal” operating level to establish the required spillway invert level (refer to **Section 9.6**); and
- (iv) Complete hydrological and wave run-up analyses and design of the RWP embankment spillway (refer to **Sections 9.7 to 9.9**) to establish the RWP embankment crest level (refer to **Section 9.10**).

### 9.3 RWP Trade-Off Studies

In order to assess the required RWP capacity and hence embankment crest level, it is first necessary to adopt a “normal” operating pond level, based on trade-off water balance studies. The trade-off studies utilised an iterative water balance approach, with the variables being the spillway invert level (i.e. the maximum storage capacity of the RWP) and the return water pumping rate.

The first trade-off study [32], considered the trade-off between return water pumping capacity and operational spill frequency as a result of concentrated rainfall events to minimise RWP

embankment height. The study outcomes indicated that with allowance for infrequent spillway flows (which would likely have favourable impacts to the downstream environment provided scour potential was minimised), the maximum RWP embankment height could be reduced to 9 m. This was however inconsistent with the perceived regulatory requirement to have zero operational discharge from the overall facility (other than emergency discharge to prevent catastrophic embankment failure).

A second study [33] comprised additional modelling of configurations to achieve a “no operational spill” RWP arrangement in compliance with the DMIRS [27] guidelines for TSFs.

The outcome was that significant embankment construction savings could be made by adding a second pump to the return water system, whilst still meeting the general regulatory objectives outlined in Section 9.1, and a maximum design RWP embankment height of 16 m was determined.

A two-pump return water system was thus implemented, with a combined maximum design capacity of 1,980 m<sup>3</sup>/hr (based on commercial pump specifications provided by IBO).

#### 9.4 Adopted RWP Freeboard Design Criteria

The specific design requirements for flood freeboard (based on DMIRS [27]) and emergency spillway capacity (based on ANCOLD [28]) for the RWP are presented below.

- Embankment Freeboard (DMIRS):
  - “Normal” Operating Pond - a statistically derived mean operating volume based on water balance studies; plus
  - “Wet” Freeboard - 1:100 year AEP, 72 hr run-off superimposed on normal operating pond (assuming that the return water pumps are not operational for the duration of the event); plus
  - “Total” (dry) freeboard of an additional 1 m to assign the required spillway invert level. This is considered conservative given the inclusion of an emergency spillway (discussed below), but is considered warranted given the need to meet the general regulatory objectives outlined in Section 9.1.
- Emergency Spillway Depth (ANCOLD):
  - With respect to emergency spillway design criteria, the ANCOLD guidelines require High C consequence category facilities to be able to pass the maximum runoff from the critical duration storm with a minimum AEP of 1 in 100,000 years.
  - A slightly more conservative approach was adopted for the antecedent operating pond level at the beginning of the 1 in 100,000 year AEP flood, whereby the 1 in 10 year AEP RWP volume from the “Average LOM Rainfall” water balance scenario (refer to Table 9.2 in Section 9.5) was used instead of the mean pond volume. It is noted however that this has had negligible influence on the spillway design depth, given the extreme inflows resulting from 1 in 100,000 year AEP events.
  - Given that the RWP is a water dam of significant surface area, an additional wave run-up resulting from a 1 in 10 year AEP wind was applied to the spillway depth in order to assign the required RWP embankment crest level, in accordance with the ANCOLD guidelines.

- The combination of flood depth passing over the spillway plus wind-induced wave run-up producing the highest total freeboard was adopted as the design RWP embankment freeboard.

The design criteria adopted for the RWP embankment are summarised in **Table 9.1**.

**Table 9.1**  
**RWP Freeboard Design Criteria**

Freeboard to Assign Spillway Invert Level			Spillway Depth to Assign Embankment Crest Level		
Operating Pond	Wet Freeboard	Dry Freeboard	Operating Pond	Spillway Capacity	Wave Run-up
Mean RWP volume from “Average LOM Rainfall” water balance scenario	1:100 AEP, 72 hr duration	1 m	1 in 10 AEP RWP volume from “Average LOM Rainfall” water balance scenario	1 in 100,000 AEP, critical duration	1 in 10 AEP

### 9.5 “Normal” Operating Pond

Based on the outcomes of the trade-off studies (refer to **Section 9.3**), and as agreed with IBO projects and environmental personnel, the following procedure was adopted for the adoption of a “normal” operating pond level:

- Undertake LOM water balance modelling to develop RWP inflow volume statistics for a spectrum of LOM total rainfall scenarios. As described in **Section 7.5** (and detailed in **Appendix E**), 1,000 LOM daily climate scenarios were analysed.
- Select the average (mean) and maximum LOM total rainfall scenarios for further assessment.
- Order, rank and assign a probability to the daily RWP volume data for each scenario.

Presented in **Table 9.2** (reproduced from **Appendix E**) are RWP volumes corresponding to various return periods for the two selected rainfall scenarios.

**Table 9.2**  
**RWP Volume - Probability of Occurrence (Once in the LOM)**

Probability	Pond Volume (ML)	
	Realisation 786 (Average LOM Total Rainfall Scenario)	Realisation 750 (Maximum LOM Total Rainfall Scenario)
Mean	260	886
1 in 10	875	2,967
1 in 20	1,675	3,927

- For the “normal” operating pond level, adopt the average (mean) pond volume from the average LOM total rainfall scenario. From **Table 9.2** this volume is 260 ML, which equates to a water level of RL 246.6 m (based on the RWP filling curve presented in **Section 9.10.2**).

## 9.6 Wet & Dry Freeboard and Adopted Spillway Invert Level

Similar to the TSF embankment spillways, the RWP embankment has been designed to provide a “wet” freeboard due to the run-off from a 1:100 AEP, 72 hour storm superimposed on the “normal” operating pond.

The adopted rainfall depth for a 1:100 AEP, 72 hour storm is 394 mm, based on the design rainfall data presented in **Chart 8.2**. The catchment areas for the freeboard calculations are those used in the water balance modelling (refer to **Section 7.5**). A runoff coefficient of 0.37 has been adopted for the natural catchment area, based on long term catchment yields from the water balance modelling (refer to **Section 7.5**). The resultant wet freeboard for the RWP embankment is presented in **Table 9.3**.

**Table 9.3**  
**RWP Wet Freeboard Calculation**

Catchment	Area (ha)	1:100 AEP, 72 hr Inflow Volume	Wet Freeboard
Normal Pond Natural Catchment	7.4 101.3	4,042 ML	8.2 m (RWP Filling Curve presented in <b>Section 9.10.2</b> )

In order to assign the RWP spillway invert level, an additional “dry” freeboard of 1 m is added to the “wet” freeboard. This process is summarised in **Section 9.10.1**.

## 9.7 Emergency Spillway Hydrology

### 9.7.1 Methodology

The procedure for the derivation of the design flood and the subsequent routing through the RWP emergency spillway are equivalent to those described for the TSF embankment spillways in **Section 8.6**.

The adopted pond level at the beginning of the design flood was RL 250 m (based on the RWP filling curve presented in **Section 9.10.2**), which is equivalent to the 1 in 10 year AEP inflow of 875 ML from the “Average LOM Rainfall” water balance scenario (refer to **Table 9.2** in **Section 9.5**)

### 9.7.2 RWP Embankment Spillway Routing Results

The results of the RWP Embankment spillway routing calculations are summarised in **Table 9.4**. Note that in addition to the surrounding catchment runoff, the inflow hydrograph for the RWP spillway routing included the equivalent duration outflow hydrograph from the TSF spillway.

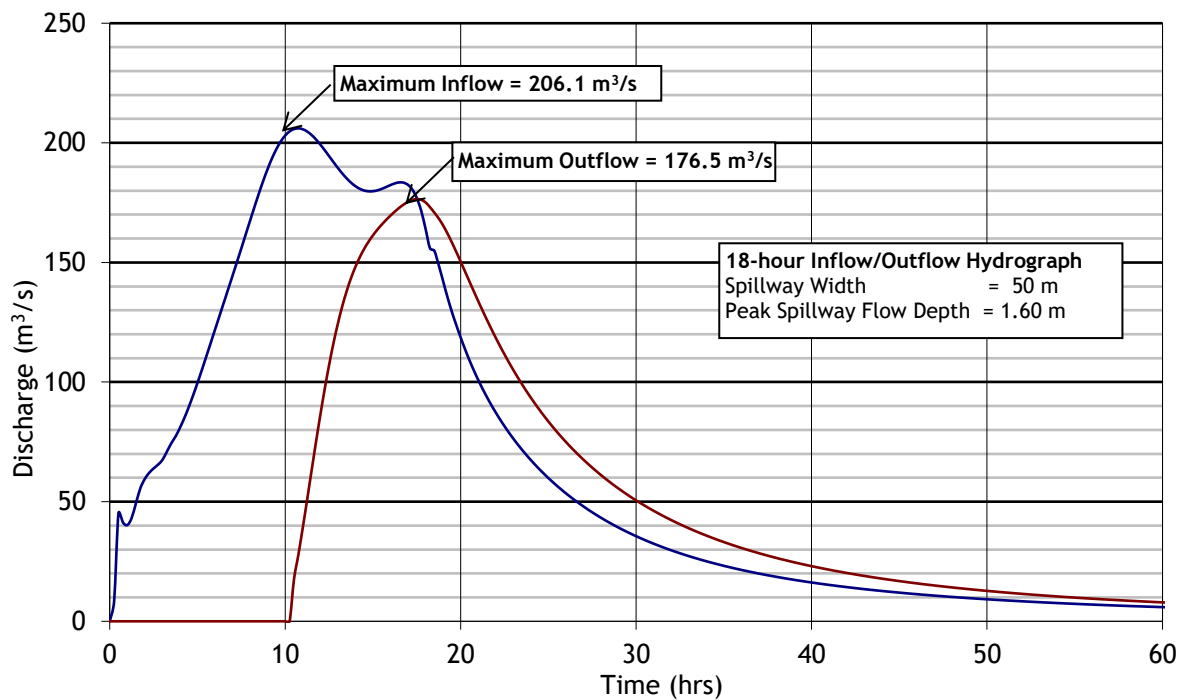
The inflow / outflow hydrographs for the critical duration (18 hour) 1 in 100,000 year storm event are presented in **Chart 9.1**.

A 50 m wide emergency spillway with a peak flood height of 1.6 m has been adopted for the RWP embankment. It is evident that the attenuating effects of the RWP storage have a significant impact on the routing of the flood wave through the spillway, and hence on the required width of the spillway.

**Table 9.4  
Spillway Flood Routing Results For RWP**

Parameter		Unit	Outcomes for 1 in 100,000 yr AEP Storms
Spillway Dimensions:	Spillway Width	(m)	50
	Spillway Depth	(m)	1.6
	Spillway Capacity	(m <sup>3</sup> /s)	176.5
Peak Outflow:	12 hr Storm	(m <sup>3</sup> /s)	160.4
	18 hr Storm		<b>176.5</b>
	24 hr Storm		158.9
Peak Flood Height:	12 hr Storm	(m)	1.58
	18 hr Storm		<b>1.60</b>
	24 hr Storm		1.57

**Chart 9.1  
RWP Spillway - Inflow/Outflow Hydrographs**



## 9.8 RWP Embankment Wave Run-up

### 9.8.1 General Methodology

The wave run-up calculations for the RWP embankment are based on the current best-practice methods detailed by the U.S. Department of the Interior Bureau of Reclamation [34]. The calculations have been completed based on the following assumptions:

- Water level within the RWP is at the spillway invert level;

- RWP embankment will have a “smooth” upstream face consisting of a bituminous geomembrane liner (i.e. wave run-up will be more pronounced than for a rockfill-faced embankment); and
- No infiltration or evaporation losses have been taken into account.

### 9.8.2 Predominant Wind Data

As discussed in **Section 3.2**, wind data from the Marble Bar weather station (BOM Station ID 4016) was analysed in order to determine an appropriate wind speed and predominant wind direction for use in the wave run-up calculations. Wave development is a function of sustained winds rather than short duration gusts, so it is important to distinguish maximum sustained (i.e. at least 10-minute duration) wind speeds from higher speed gusts.

The wind data used was in the form of a wind rose plot of wind direction versus wind speed from a total of 5,907 observations between 26 September 2000 and 12 August 2017.

Due to the orientation of the RWP embankments and the largest fetch distance, it was identified that winds from an easterly to south-easterly direction are likely to result in the largest waves being generated.

The wind rose data showed that the highest percentage of easterly to south-easterly winds occurs in May. Sustained speeds do not exceed 40 km/hour, however the majority of the time do not exceed 20 km/hr. In other months, easterly to south-easterly winds exceed 40 km/hr very rarely. Hence, whilst no statistical analyses were able to be conducted, it was considered reasonable conservative to adopt a 40 km/hr wind speed as being suitably representative of a 1 in 10 AEP wind for the wave run-up calculations.

## 9.9 RWP Spillway Depth Results

The results of the RWP embankment wave run-up and resultant total spillway depth assessment is presented in **Table 9.5**.

**Table 9.5**  
**RWP Embankment Spillway Depth Summary**

Flood AEP	Peak Flood Height (m)	Wind Duration & Approx. AEP	Wind Speed (km/h)	Wave Run-up (m)	Required Spillway Depth (m)
1 in 100,000	1.6	1 in 10, 10 min.	40	0.6	2.2

## 9.10 RWP Embankment Crest Level & Capacity

### 9.10.1 Adopted Crest Level

As outlined in **Section 9.2**, the procedure to establish the RWP embankment crest level is defined as follows:

“Normal” Operating Pond + “Wet” Freeboard + “Dry” Freeboard +  
Peak Spillway Flood Depth + Wave Run-up

Based on the assessments provided previously in this Section 9, the total embankment freeboard requirement for the RWP embankment is summarised in Table 9.6.

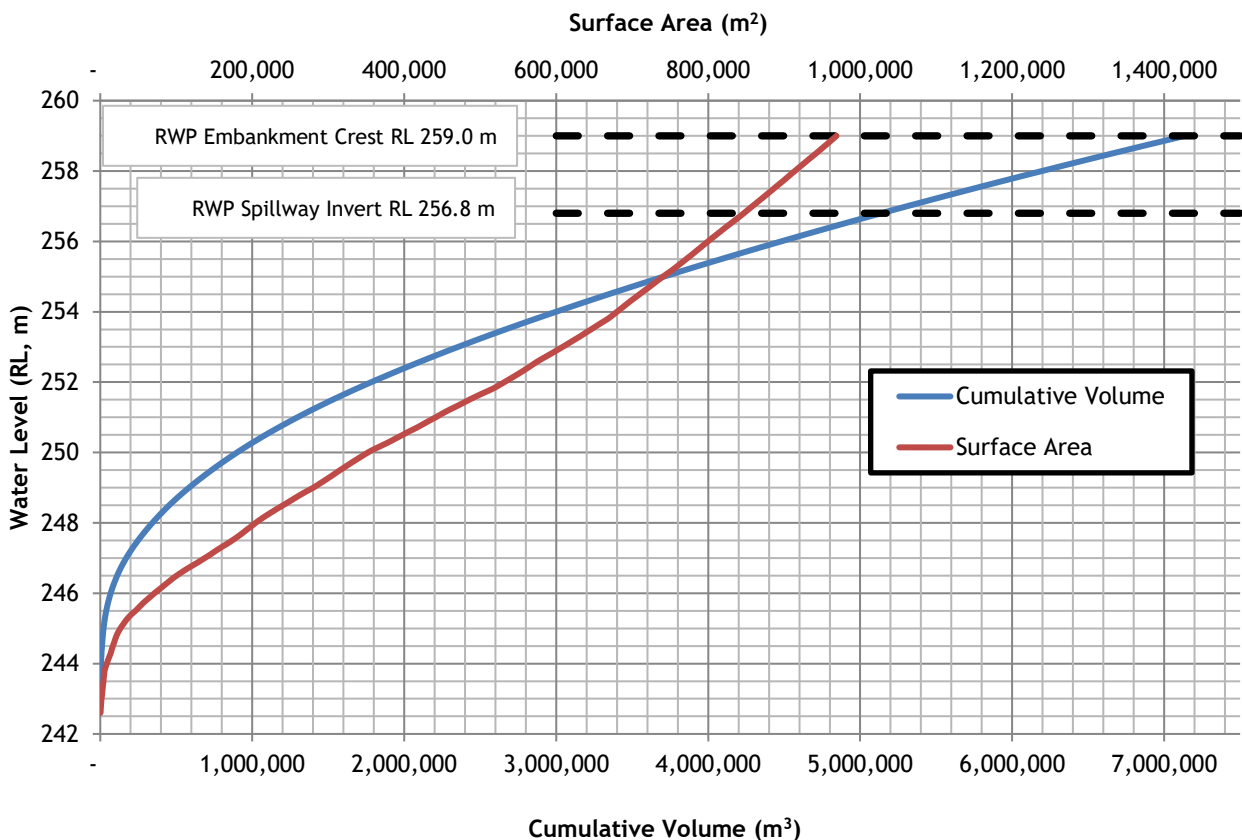
**Table 9.6**  
**RWP Embankment Freeboard and Crest Level**

“Normal” Operating Pond (RL, m)	Wet Freeboard (m)	Dry Freeboard (m)	RWP Spillway Invert (RL, m)	Spillway Depth (m)	RWP Embankment Crest (RL, m)
247.6	8.2	1.0	256.8	2.2 m	259.0

### 9.10.2 RWP Capacity

Based on the spillway and embankment freeboard studies, the RWP embankment will require a spillway with an inlet RL 240.5 m. The filling curve for the RWP is shown in Chart 9.2. The capacity of the RWP at the maximum water level (spillway inlet level) is 5.1 million m<sup>3</sup> (5,100 ML).

**Chart 9.2**  
**RWP Filling Curve (Volume - Area - Water Level Relationship)**



### 9.10.3 Flood Storage Provision and Spill Risk

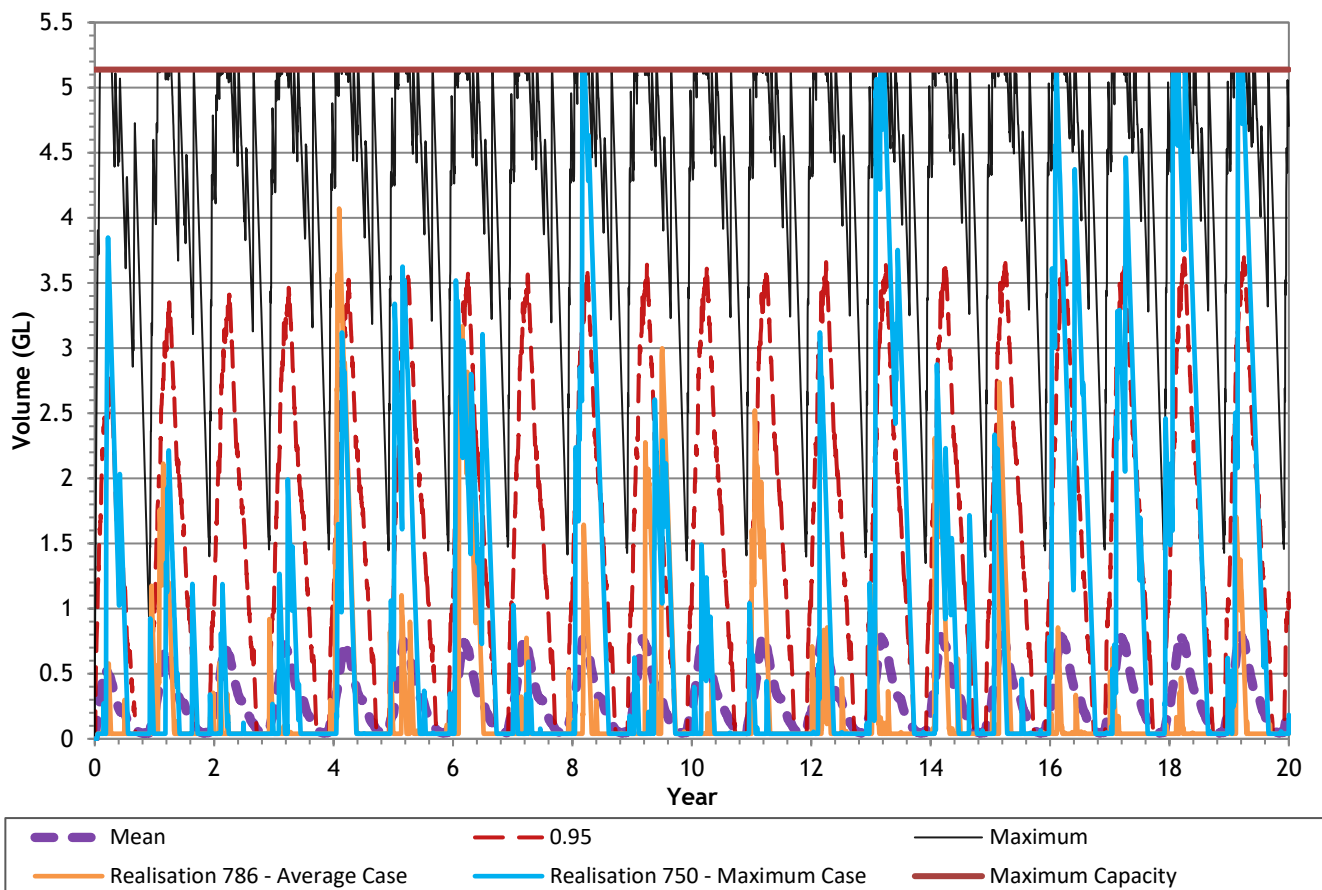
In order to confirm the adopted RWP capacity conforms with the operational and regulatory intent outline in Section 9.1, a detailed evaluation of the LOM water balance model (refer to Section 7.5) has been conducted.

Various daily RWP volume statistical (stochastic) and individual simulated water balance outcomes (referred to in **Appendix E** as “realisations”) are presented in **Chart 9.3** (reproduced from **Appendix E**).

As introduced in **Section 9.5**, the individual realisation results comprise daily output following the unique climate input for average (mean) and maximum LOM total rainfall scenarios. The statistical results depict the percentile values based on a certain day across all realisations, and hence represent a collection of values from all realisations for the model duration rather than representing any specific realisation.

The RWP spillway invert level is also included in **Chart 9.3** to demonstrate the relationship between the pond levels and the spill potential. It can be seen from the stochastic data that the “maximum” percentile graph comprises the peak outcomes of Realisation 750 (the maximum LOM rainfall case) on a number of occasions. However, the significantly lower 95th percentile outcomes, and the mean volume averaging at 305 ML, show the sensitivity of the system to the tropical and monsoonal events within the rainfall database.

**Chart 9.3**  
**RWP Volume Stochastic Results & Selected LOM Rainfall Scenarios**



With respect to spill risk, **Chart 9.3** shows that for the average LOM total rainfall case (Realisation 786), no spillway discharge is predicted as the pond never reaches the maximum capacity.

A statistical analysis of spill events over the 1,000 water balance realisations (refer to **Appendix E**) identified that the number of spilling days is relatively small compared to the 20 year mine life, with a maximum stochastic number of 40 days. For the maximum LOM total rainfall case (Realisation 750), the cumulative number of spilling days is 22, which equates to a 1 in 330 chance of spilling over the 20 year LOM.

In summary, all simulated spillway discharges result from coupled events comprising high antecedent pond volume and extreme rainfall, for which the joint-probability is not able to be directly assessed, but is well beyond what could be considered “normal” operating conditions.

## 10 EMBANKMENT CONSTRUCTION MATERIALS

### 10.1 Construction Materials Summary

Construction of the TSF and RWP embankments involves the placement of five zones of rockfill and granular filter materials, categorised below:

- (i) Granular and filter materials - Zone 2A and 2B
- (ii) Rockfill embankment and drainage materials - Zone 3A, 3B and 3C

The geotechnical investigations (**Section 3.7**) have identified a number of hill areas within the TSF and RWP impoundments from which Zone 3A and Zone 3B rockfill materials can be sourced. Additional rockfill material will be available from waste rock dumps generated during Stage 2 mining operations.

Laboratory testing (**Section 3.7**) indicated that the existing coarse reject stockpiles at the Stage 1 North Star Mine Site provide a suitable source for Zone 2A and Zone 2B materials for initial construction. Particle size and production schedule data provided by IBO also indicates that the Stage 2 coarse rejects will be suitable and available for Zone 2A and 2B materials.

In addition, the following materials will be supplied and installed within the TSF and RWP embankments and associated civil works:

- (i) Geosynthetics - Bituminous Geomembrane (BGM) liner, non-woven geotextile, concrete canvas.
- (ii) Concrete - Reinforced concrete toe plinth, reinforced concrete slab and steps, precast concrete decant culverts and headwalls, reinforced decant pipe encasement.
- (iii) Other Items - Corrugated metal decant pipe, aluminium decant stoplogs and frames.

Design details for the primary construction materials (embankment fill and BGM liner) are presented in the following sections, however refer also to **Sections 11** and **12**.

### 10.2 Zone 2 Granular and Filter Materials

#### Zones 2A, 2B (Granular and Filter materials)

Zone 2A material forms the granular supporting layer for the BGM liner on the upstream face of the TSF and RWP Embankments. Zone 2B will provide the trench backfill material along the alignment of the TSF decant pipe encasements.

Zones 2A and 2B are expected to be sourced from the existing coarse reject stockpile located at the Stage 1 North Star process plant site and subsequently from Stage 2 production of similar material. Particle size distributions for recent (2018 geotechnical investigation) and proposed North Star Stage 2 coarse rejects (as advised by IBO) are plotted in **Chart 10.1**.

It is evident that the rejects are generally blended silt-sand-gravel materials with fines contents (percent passing 75  $\mu\text{m}$ , i.e. silt and/or clay) ranging from 5% to 10%. Based on laboratory testing, the fines are non-plastic.

Zone 2A materials will form the subgrade for the BGM liner, for which a maximum asperity (particle size) of 25 mm is recommended by the manufacturers to prevent the liner being punctured. Hence it is possible that some of the coarser existing rejects stockpile sources (if utilised) will need to be screened to remove particle sizes larger than 25 mm.

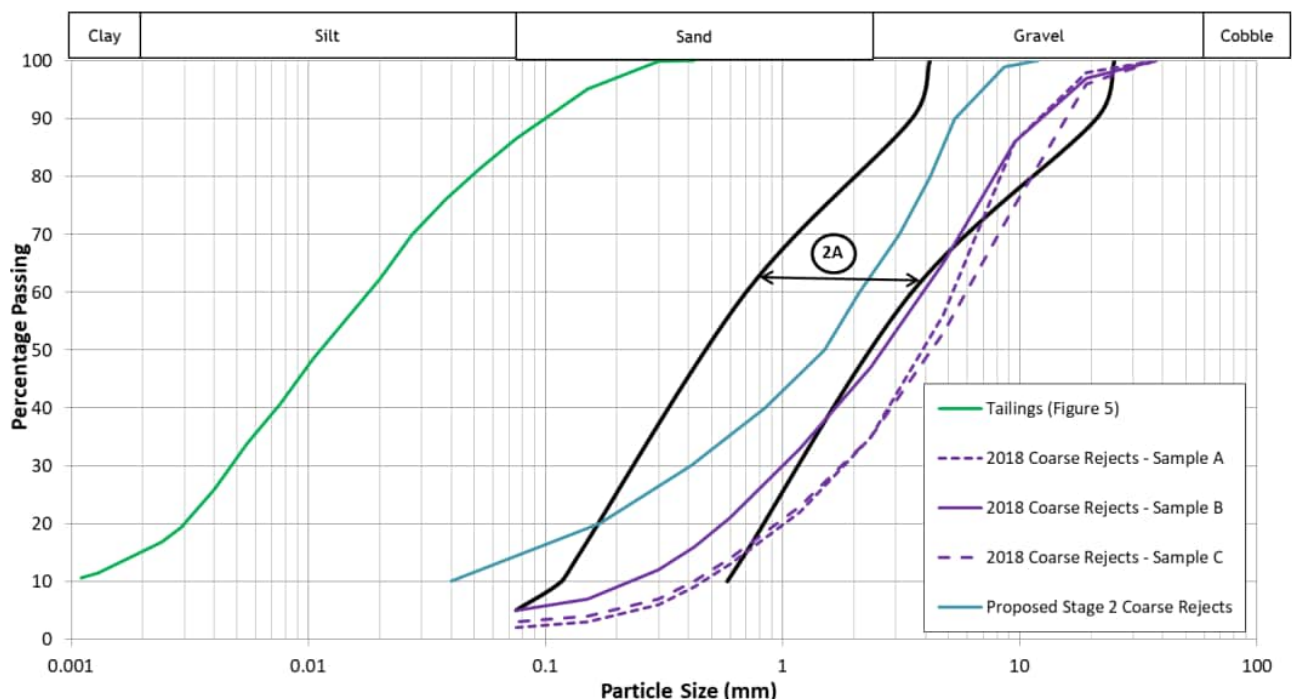
In order to ensure that any concentrated leak in the BGM Liner will not result in the migration of tailings particles, Zone 2A has been designed in accordance with the filter design criteria established by Sherard & Dunnigan [35], which essentially involves the comparison of  $D_{15}$  of the “filter” (where  $D_{15}$  is the particle diameter for which 15% of the filter is finer than) and  $D_{85}$  of the “base” soil (the soil which is being protected by the filter).

The North Star tailings will be the “base” soil, from which the “filter” design criteria require a  $D_{15F}$  not greater than 0.7 mm.

The North Star “Figure 5” tailings (refer to Section 5.1), and the resultant Zone 2A filter design criteria are also plotted in Chart 10.1. It is evident that the tested coarse rejects all essentially fit the key filter criteria, although it is recognised that the slightly gap-graded nature of the existing stockpile materials means they will be prone to segregation during placement.

The effect of this has been reduced by the minimum design Zone 2A width of 3.5 m, and the specification of minimum acceptable moisture conditioning and compaction standards.

**Chart 10.1**  
**Particle Size Distribution - Coarse Rejects & Zone 2A Filter Criteria**



Zone 2B will effectively be the same materials to Zone 2A, being sourced from the same coarse rejects stockpiles, however the material does not require a minimum particle size of 25 mm so will not require screening.

### 10.3 Zone 3 Rockfill Materials

Zone 3 Rockfill materials form the major rockfill zones within the TSF and RWP embankments, and will generally consist of clean (i.e. low fines) sound, durable rockfill conforming to the rock weathering classifications and maximum particle sizes defined in **Table 10.1**.

The zones will be sourced from the following locations:

- Quarries - blasted, quarried rockfill from designated quarry areas within the TSF and RWP impoundments (Zones 3A, 3B, 3C);
- Required excavations - BGM anchor trench and decant pipe excavations, spillways, approved foundation strip materials (Zones 3A, 3B, 3C);
- Existing and future Stage 2 Mine Waste Rock Stockpiles (Zones 3A, 3B, 3C); and
- Coarse Rejects (Zone 3A only).

**Table 10.1**  
**Zone 3 - Allowable Weathering Classifications & Particle Sizes**

Rockfill Type	Allowable Rock Particle Weathering Classifications	Maximum Particle Size (mm)	Maximum % Passing			
			26.5 mm	4.75 mm	0.6 mm	0.075 mm
3A -Transition Rockfill	Highly Weathered to Slightly Weathered	400	55	30	10	5
3B - General Rockfill	Moderately Weathered to Fresh	800	45	15	-	5
3C - Rockfill Drain	Slightly Weathered to Fresh	600	20	5	-	0

Zone 3A material has been included in the design to allow for placement of more weathered and/or finer-grained rockfill (shale) encountered in the quarry areas.

Similar to Zone 2A material, in order to ensure that any concentrated leak in the BGM Liner will not result in the migration of Zone 2A particles, Zone 3A has been designed in accordance with the filter design criteria established by Sherard & Dunnigan [35], as described in **Section 10.2**.

In this case, the Zone 2A material will be the “base” soil, from which the “filter” design criteria require a  $D_{15F}$  not greater than 2.8 mm.

Zone 3B and 3C materials will generally be restricted to slightly weathered to fresh rockfill, whereas highly and moderately weathered rockfill will be allowable for Zone 3A materials.

The proposed rockfill quarry area for Stages 1A and 1B is located within the TSF containment area, directly to the east of Main Embankment A (refer to **Figure 114185.10\_002**). Many hills within the TSF containment area are composed of suitable rock for use as Zone 3 material however will eventually become inundated with tailings. For this reason, subsequent stages will be sourced from mine waste stockpiles.

## 10.4 BGM Liner

### 10.4.1 Background

Over the course of the various studies and investigations for the project, it has become clear that the absence of suitable low permeability clay materials in the vicinity of the site is likely to necessitate the utilisation of alternative water retaining elements in the design of the embankments.

There is a limited availability of natural clayey soils and although the use of tailings as low permeability material is feasible, this would require further upstream embankment zones to protect the tailings from erosion and dispersion. Because of the lack of natural clayey soils and because the use of tailings is problematic, the adoption of a geosynthetic lining system is proposed for the TSF and RWP embankments.

### 10.4.2 Liner Options Review

A number of geosynthetic lining systems were compared with each other from the perspectives of performance, durability and cost. The liners included:

- LLDPE and HDPE (Linear Low, and High Density Poly Ethylene);
- PVC Geomembrane liners; and
- Bituminous Geomembrane (BGM) liners.

LLDPE and HDPE have good chemical and acceptable UV resistance, and may be left exposed for a number of years, pending weather conditions. They are significantly cheaper than PVC and BGM (to a lesser degree). Nonetheless they were rejected because they must be anchored at 10 m to 15 m height intervals, resulting in 5 m to 10 m wide berms at 10 m to 15 m height intervals. This results in flattening of the overall upstream slope and higher earthworks volumes and costs.

PVC and BGM liners on the other hand were considered to be more suitable than LLDPE and HDPE because they:

- Are very flexible and conform very well to their underlying foundations; and
- Have better dimensional stability and deformation characteristics compared to HDPE.

Comparing PVC with BGM liners:

- Seaming of the BGM and PVC panels are similar in their ease of application and the resultant strength and durability of the seams are similar;
- Perimeter anchorage requirements for the two liner types are similar, with:
  - mechanical tie-down anchors to a concrete plinth at the toe, and
  - trench ballasting at the toe and the crest being appropriate;
- BGM has a much higher unit weight than PVC, making it more resistant to wind uplift, especially during installation. In exposed applications, this simplifies the face anchorage requirements for the BGM liner compared to PVC, which must be anchored to the face either in continuous vertical lines or in a regular grid pattern.
- BGM can be manufactured with a high friction underside, such that the drag-down forces developed by consolidating tailings can be resisted without intermediate anchorage. This is because the interface friction between the BGM liner and the embankment

subgrade is greater than the friction developed on the liner face by the tailings. Although the friction developed between the subgrade and the underside of the PVC geocomposite is sufficient to resist the drag-down forces, the wind uplift issues necessitate intermediate anchors.

- PVC is significantly more expensive than BGM.

#### 10.4.3 Liner Selection

Based on the foregoing comparison and assessment of the alternative geosynthetic lining systems, the reinforced elastomeric BGM liner has been selected for use in the TSF and RWP embankments for the following reasons:

- (i) It is cheaper than PVC whilst having similar deformation characteristics; and
- (ii) It requires a simpler anchorage system compared to PVC, with no intermediate face anchors due to the higher unit weight and enhanced frictional resistance provided by the high friction underside.

There are two BGM products on the market considered suitable for this liner application. Both of which are manufactured in Europe. They are referred to as “reinforced elastomeric BGM”, these lining systems are manufactured from SBS (Styrene Butadiene Styrene) elastomeric bitumen reinforced with a non-woven polyester geotextile.

Reinforced elastomeric BGMs are bonded by heat, employing a relatively simple procedure requiring a lesser input of skilled labour compared to HDPE seaming. They are robust and have excellent UV stability and weathering properties necessary to survive for up to 40 years uncovered. These materials have a density greater than 1 t/m<sup>3</sup>, hence will not float in water and are less susceptible to wind uplift than traditional polyethylene (LDPE or HDPE) or PVC geomembranes.

The surface is coated with silica sand. Both products are considered viable alternatives in exposed installations and demanding environments.

The use of BGM liners has significantly increased in the past two decades mainly due to major improvements in the application of elastomeric polymers and a higher manufacturing quality control, overcoming historic quality control issues. BGM waterproofing systems have been successfully installed on a number of water and tailings dams around the world, and ATCW have had recent successful experience with a large TSF in a very demanding environment in north-west Queensland.

#### 10.4.4 BGM Liner Specification

The selected BGMs for the TSF and RWP embankments shall consist of either:

- (i) Siplast TERANAP 531 TP elastomeric modified bituminous geomembrane, nominal thickness 4.6 mm; or
- (ii) Axter COLETANCHE ES3 SBS elastomeric modified bituminous geomembrane, nominal thickness 4.8 mm.

The interface friction design assessment has shown that the high friction underside variant of the BGM will be required. In the case of the COLETANCHE product, this product is referred to as “HFA”, whereby the root barrier film is omitted from the liner underside during manufacture.

## 11 TSF EMBANKMENT OVERVIEW

### 11.1 General Description

The TSF embankment has been designed as a zoned rockfill embankment, and will be constructed in seven stages. The TSF embankment layouts for Stages 1 to 7 are shown in **Figures 114181.14\_004 to 114181.14\_011**.

General Arrangements for Stages 1A and 1B are provided in **Figures 114181.14\_012 to 114181.14\_013**.

Typical embankment sections and details are presented in **Figures 114185.14\_014 to 114185.14\_017**.

As the Figures show, the TSF embankment comprises a total of eight (8) individual embankments, some of which become merged as the staged raising progresses. The embankments are referred to in the Figures as follows:

- TSF Main Embankment, comprising Main Embankments A, B, C & D;
- West Embankments 1 & 2; and
- East Saddle Dams 1 & 2.

The TSF embankments will have a maximum (Stage 7) height of approximately 68 m, a minimum crest width of 15 m and side slopes of 2 : 1 (H : V) upstream and downstream.

Upon completion of TSF filling, the embankment downstream profile will need to be buttressed and regraded to form a stable, erosion resistant long-term landform as part of the closure works, presented in **Section 17**. In order to reduce start-up and raising costs during the life of the TSF, the embankment downstream profile has been designed on the basis of structural integrity, without taking this closure profile into account. If minimisation of closure earthworks becomes a priority in later stages of the TSF, IBO may elect that the downstream slopes of the TSF Embankment raises be formed to integrate the final closure profile.

### 11.2 Embankment Zoning & BGM Liner

The design criteria for the TSF embankment is to primarily retain tailings, with seepage of water through the embankment not a strict operating issue, as it will be collected in the downstream RWP. However, given the ultimate size of the embankments, it is desirable from stability and erosion perspectives to minimise seepage, and also to reduce the potential for tailings migration to the RWP.

Hence, the BGM liner has been included in the embankment geometry design as a seepage reduction measure. Where seepage is not considered a viable design issue for particular embankment stages, the BGM liner has been omitted. This is the case for Stages 1 and 2 of Main Embankments B and D, where the natural topography of the area creates a closed boundary downstream, and the early stages are effectively only required to provide a platform for subsequent raises.

The BGM will be installed exposed on the upstream face upon a 3.5 m wide Zone 2A support layer of granular fill. The Zone 2A support layer also acts as a filter to minimise the ingress of tailings fines which would potentially migrate through the embankment if the liner were to be punctured.

Zone 2A is placed directly upstream of the 20 m wide Zone 3A transition rockfill, in order to separate the finer grained Zone 2A from the coarser Zone 3B rockfill.

The remainder, and bulk of the embankment, will consist of Zone 3B quarried rockfill. The rockfill provides strength and stability to the structure and supports the upstream waterproofing BGM liner.

### 11.3 Downstream Staging Benches

The TSF embankment will be raised downstream and will thus require downstream benches to be constructed at strategic stages of embankment raises to protect subsequent embankment stage foundations from inundation in the event of temporary expansion of the RWP pond due to extreme rainfall events.

The elevation of the staging benches has been set at the RWP maximum water level (RL 256.8 m). The timing of bench construction will be based on when the downstream toe of a particular stage encroaches upon the RWP maximum water level. Hence, staging benches will be included in the construction of Stages 1A, 1B and 4 of the TSF embankment raises, as shown in **Figures 114181.14\_004 to 114181.14\_011**.

### 11.4 BGM Anchorage

As shown on **Figure 114185.14\_017**, the BGM liner will be anchored at the lower perimeter in a trench excavated to refusal at the upstream toe of the TSF embankments. Based on the geotechnical investigations, the depths of this trench will range between 0.5 m and 2.5 m deep.

To protect the BGM from damage during backfilling of the anchor trench, the BGM will be covered by a non-woven geotextile. In trench areas containing particularly rough asperities, a geotextile will also be placed beneath the BGM. Whilst geotextiles can provide planar leakage (i.e. flow parallel to the geotextile layer) in these situations, the flow rate in terms of equivalent permeability is similar to the adjacent embankment and foundation materials.

At the crest of each stage, the BGM will be anchored by embedment in a 0.4 m deep trench excavated along the upstream shoulder.

Due to the adoption of a BGM product with a high interface friction angle (Refer to **Section 10.4**), internal anchorage to the embankment face are not required.

### 11.5 Embankment Construction Aspects

#### 11.5.1 Impoundment and Embankment Subgrade Preparation

Prior to the construction of each embankment stage, all topsoil from within the TSF embankment footprints will need to be stripped. It is recommended that where possible, topsoil be stripped from the TSF impoundment area as well. Stripped topsoil will need to be stockpiled near to the TSF for re-use in capping the tailings surface as part of the TSF rehabilitation. Trees within the impoundment area will also need to be cleared.

The footprint beneath the TSF embankment will be stripped to a nominal 500 mm depth in dense alluvial deposits, and a nominal 200 mm depth in other areas, or to weathered rock. Following

stripping, the foundation material is generally expected to be weathered rock. Rock foundations will need to be generally clean, removing loose soils and all soft or weak material if encountered.

### 11.5.2 Compaction of Embankment Fill

Due to the confined widths, Zone 2A and Zone 2B granular materials, and Zone 3A transition rockfill will be compacted using conventional vibratory rollers adopting a method specification developed on the basis of compaction trials (as discussed in **Section 4.3.2**).

The option of utilising dynamic compaction (impact rolling) for the compaction of the large Zone 3B rockfill quantities has been considered because of the high production capacity achievable using such methods. For example, some roller manufacturer's output data claims up to 2,500 m<sup>3</sup>/hour at 8 passes on a 1 m thick layer (based upon a 45 minute working hour).

### 11.5.3 Upstream Face Constructability

It is important to achieve a properly compacted upstream (Zone 2A) face, as this forms the BGM subgrade. It is possible that this will need to be achieved whilst maintaining a minimum 0.5 m safety bund or barrier during fill placement. To accommodate the requirement for a constructed safety bund, it will be necessary to initially over-place the initial Stage 1 embankment face to achieve the specified compaction standards within the design lines of the embankments, leaving a less compacted zone comprised of the safety windrows on the upstream face.

Once the embankments have been constructed to the full Stage 1 crest elevation, the upstream face must be cut back to the design lines, leaving a properly compacted face suitable for placement of the BGM Liner.

The construction methodology implemented to achieve the necessary BGM liner subgrade condition, lines and levels will need to be developed by the construction contractor, in conjunction with advice from the design engineers and IBO.

## 12 RWP EMBANKMENT OVERVIEW

### 12.1 General Description

The RWP will contain all water released from the deposited tailings, as well as storm water runoff from the 1,352 ha and 714 ha catchment areas of the TSF and RWP, respectively. Based on regulatory requirements and adopted operating criteria, the RWP will have a design water storage capacity of 5,100 ML (refer **Section 9.10**).

The RWP consists of two embankments, the RWP South embankment and North embankment. The 16 m high RWP South embankment and 10 m high RWP North embankment (both RL 259 m) are water retaining structures required to close the RWP valley outlet on its western side.

The geometry of the RWP embankments will comprise a minimum crest width of 10 m and side slopes of 2 : 1 (H : V) upstream and downstream. A general arrangement of the RWP embankments is shown in **Figure 114185.14\_022**, with sections and details in **Figures 114185.14\_023 & 024**.

### 12.2 Embankment Zoning, Liner and Foundation Arrangements

The internal zoning of the RWP embankments adopts the same arrangement as described for the TSF embankment in **Section 11.2**. Both RWP embankments will require the installation of an impermeable BGM liner on the upstream face, as per the TSF embankment design.

The RWP South embankment BGM liner will be anchored at the upstream toe to a reinforced concrete plinth, which will also act as the foundation curtain grout cap (refer to **Section 12.4**). The South embankment will be the primary water retaining structure, as it is located in the lowest part of the valley through which the drainage paths traverse. Hence to control foundation seepage, a grout curtain will be installed along the length of the upstream toe plinth of the RWP South embankment.

A more simplified approach has been taken with the RWP North embankment, due to its higher base elevation, and hence more limited contact with ponded water. Referring to the statistical water balance outcomes discussed in **Sections 9.5 to 9.7**, the 1 in 10 AEP RWP water level of RL 250 m does not reach the North embankment toe, hence expensive seepage minimisation measures (such as plinth construction or foundation curtain grouting) to counteract sustained water retention are not warranted. Instead the BGM Liner will be anchored in an upstream key trench, as per the TSF embankment design described in **Section 11.4**. The implications of this with respect to estimated seepage rates are discussed in **Section 13**.

Both embankments will include additional seepage control measures comprising:

- A rockfill seepage collection drain in the base of the embankment foundation. This will discharge into a seepage monitoring sump directly downstream of each embankment;
- A circular concrete seepage monitoring sump (soak well), installed in a rock-filled interception trench; and
- Downstream groundwater monitoring bores (nominally six in number).

During operations, the monitoring bores and sump will all be capable of being fitted with submersible, solar-powered pumps to return to the RWP any seepage considered to be environmentally unsuitable to be released downstream.

### 12.3 BGM Anchorage

The upstream BGM anchor trench for the RWP North embankment will be constructed as per the arrangement described for the TSF key trench in **Section 11.4**. Based on the geotechnical investigations, the depths of this trench will range between 0.5 m at the abutments, and 2.0 m at the lowest point.

As shown in **Figure 114185.14\_024**, a plinth trench terminating on moderately to slightly weathered rock will be excavated at the upstream toe of the RWP South embankment footprint. The trench will be up to 3.0 m deep at the lowest point of the embankment, in order to fully penetrate the alluvial and colluvial soils, as well as the near surface highly fractured rock horizon, thus minimising the volume of near-surface foundation grouting to be conducted.

The 3 m wide, 0.4 m deep reinforced concrete plinth will be cast in the base of the trench, and held in place with anchor dowels. The plinth will form a water-tight transition from the BGM liner to the foundation grout curtain.

At the crest of each stage, the BGM will be anchored by embedment in a 0.4 m deep trench beneath a mass-concrete anchor beam cast along the upstream shoulder.

Due to the adoption of a BGM product with a high interface friction angle (Refer to **Section 10.4**), internal anchorage to the embankment face is not required.

### 12.4 Foundation Grouting

The geotechnical investigation (**Section 3.7**) has revealed high foundation rock permeability in the vicinity of the RWP North embankment and part of the South embankment to depths of up to 20 m.

As discussed in **Section 12.2**, a program of foundation grouting will be required prior to the RWP South embankment construction. The foundation grouting will comprise both shallow contact and deep curtain grouting, and will be conducted in two lines at regular intervals along the length of the plinth. Curtain grout hole depths will be at least 15 m, in order to create (in combination with the toe plinth trench excavation) a total interception depth of the order of 20 m.

### 12.5 Embankment Construction Aspects

#### 12.5.1 Impoundment and Embankment Subgrade Preparation

The impoundment and embankment subgrade preparation requirements will be as described for the TSF embankment in **Section 11.5.1**. The only additional works will comprise the localised, deeper excavation of alluvial deposits from the creek bed within the RWP South embankment footprint.

#### 12.5.2 Compaction of Embankment Fill

Due to the confined widths, Zone 2A and Zone 2B granular materials, and Zone 3A transition rockfill will be compacted using conventional vibratory rollers adopting a method specification developed on the basis of compaction trials.

## 12.6 Downstream Erosion Protection

### 12.6.1 Overview

Considering the significant spillway peak outflow during the design storm event determined in **Section 9.7**, the RWP northern embankment toe, seepage interception trench and monitoring chamber are likely to be damaged due to discharge without proper protection. Hence, hydraulic modelling and analyses were completed to understand the potential impact from the maximum spillway discharge and determine appropriate erosion protection measures at the impacted areas. This modelling and analyses were reported in a Technical Memorandum [36], and are summarised in the following sections.

### 12.6.2 Methodology

Numerical models were created using the hydraulic and hydrological modelling program HEC-RAS [37]. The outflow hydrograph presented in **Section 9.7** was applied as the inflow boundary condition to the RWP spillway. A normal depth boundary condition was applied to the creek downstream extremity of the model, far enough from the RWP to withdraw water from the model domain. Based on previous hydraulic studies, a Manning's roughness coefficient ( $n$ ) of 0.05 was assigned to the downstream flood plain. The HEC-RAS model was run for a duration of 60 hours, following the duration of the spillway outflow hydrograph using dynamically controlled timesteps to maintain suitable modelling accuracy. Full momentum equation sets were utilised to better simulate the unsteady flow conditions during the spillway discharge.

### 12.6.3 Results

The results showed a maximum flow depth and velocities of approximately 2.0 m and 5.6 m/s, respectively. The RWP northern embankment downstream toe will likely be inundated during the spillway discharge under the design storm event, with maximum flow depths and flood velocities of 1 to 1.5 m and 2.0 to 3.0 m/s, respectively. Similarly, the downstream seepage interception and monitoring system will be inundated to a maximum depth and velocity of 1.0 to 2.0 m and 1.0 to 2.0 m/s, respectively.

### 12.6.4 RWP Embankment Protection

Based on the above results, an outlet channel at the RWP northern embankment toe has been included to divert flow away from the downstream toe. Also, a flood protection bund has been included to prevent high velocity flows from damaging the embankment and seepage monitoring system. The  $D_{50}$  for the required bund construction material is 400 mm, which complies with embankment Zone 3B/3C materials. The arrangement of the flood protection bund is presented in **Figure 114185.14\_022**.

## 13 EMBANKMENT SEEPAGE

### 13.1 Overview

Seepage estimates at the TSF and RWP embankments have been conducted for various scenarios using a computer simulation. For the TSF, the purpose of these analyses has been to assess the likely range of decant water seepage rates reporting to the RWP, downstream of the TSF.

The purpose of the RWP analyses was to estimate the seepage rate likely to pass through the central abutment hill and embankment foundations, and if any seepage will surface downstream of the RWP embankments. RWP cases were also used to assess the improvement in seepage rates resulting from the inclusion of a grout curtain at the upstream toe of the embankment.

The seepage through the TSF embankment have been estimated using the finite element program SEEP/W [38], assuming the flow originates from the surface of the tailings and the upstream face of the embankment that is directly exposed to ponded water. A simplified approach has been used, consisting of steady-state modelling with saturated permeability conditions assumed.

Also using SEEP/W [38], transient analyses were performed for the RWP embankment along the creek line and through the central abutment hill separating the two embankments. Transient analyses were conducted to estimate the time required for steady state conditions to develop. For transient analyses, saturated permeability conditions were assumed. Soil Water Characteristic Curves (SWCC) were derived for the foundation soil and rock units using inferred saturated porosity, particle size and database methods. Analyses were also performed to assess the impact of installing a grout curtain at the RWP embankment upstream toe and the central abutment.

### 13.2 TSF Embankment Analyses

Due to the varying foundation permeabilities observed during the geotechnical investigation, two scenarios were modelled for the final Stage 7 TSF embankment configuration.

The first represented a cross section taken through Main Embankment A in the northern stream bed. The second cross section was taken through Main Embankment C, reflecting foundation conditions within the southern stream bed.

Both scenarios were modelled to the final stage TSF embankment, with the tailings at the end of filling level, and a normal operating pond of 400 mm. These scenarios represent the long-term seepage scenario beneath the TSF embankment.

### 13.3 RWP Embankment Analyses

#### 13.3.1 South Embankment

Analyses conducted for the RWP South embankment have been performed using a conservatively adopted “normal” operating pond level of RL 250 m. This level equates to a storage volume of approximately 900 ML which, based on the water balance data presented in **Section 9.5**, represents somewhere between the mean and the 1 in 10 AEP recurrence, depending on the LOM rainfall scenario.

The analyses incorporates the BGM liner and grout curtain, and has been performed for the worst case foundation permeabilities (refer to **Section 3.7**) observed in the area.

### 13.3.2 North Embankment

The RWP North embankment sits at a higher elevation than the RWP South embankment, with the minimum toe elevation being RL 250.25 m. This means that the normal operating pond level of RL 250 m will not be in direct contact with the embankment. Hence, seepage analyses for the RWP North embankment have been conducted to assess the necessity of incorporating a grout curtain into the design.

Seepage analyses were conducted using an RWP level of RL 250 m, and a separate case using an elevated RWP level of RL 252 m, (representing a 1 in 10 AEP recurrence for the maximum LOM total rainfall scenario, based on the water balance modelling introduced in **Section 7.5**) to assess the likely seepage rates that may occur throughout the LOM. It is recognised that it is questionable whether the second scenario would generate sustained ponding periods for the transient analyses, but were assessed as a worst-case scenario.

### 13.3.3 Central Abutment Hill

Transient seepage analyses were conducted for the central abutment hill located between the two RWP embankments. The purpose of these analyses was to determine the amount of seepage passing through the hill and if any water will appear at the surface downstream of the RWP.

Analyses were also run to assess the necessity to install a BGM Liner and grout curtain on the upstream side of the hill.

A normal operating pond of RL 250 m has been adopted for the analyses.

## 13.4 Material Properties

### 13.4.1 Embankment Materials

**Table 13.1** shows the saturated permeability estimates that have been adopted for the embankment materials (and tailings), based on the geotechnical investigations (refer to **Section 3.7**), tailings laboratory testing (**Section 5.1**) and ATCW previous experience (regarding embankment Zone 2 and Zone 3 materials).

**Table 13.1**  
**Saturated Permeability Estimates for TSF & RWP Embankment Materials**

Material / Embankment Zone	Saturated Permeability ( $k_{sat}$ , m/s)
Tailings :	Refer Section 13.4.2
Zone 2 - BGM Liner Subgrade :	$1 \times 10^{-4}$
Zone 3A and Zone 3B - Embankment Rockfill :	$1 \times 10^{-5}$
BGM Liner :	$1 \times 10^{-15}$
Grout Curtain (RWP Only) :	$1 \times 10^{-8}$

### 13.4.2 Tailings

Tailings saturated hydraulic conductivities have been adopted based on the profiles determined from consolidation modelling (refer **Appendix C3**).

The tailings have been modelled using saturated/unsaturated hydraulic conductivity curves by estimating Soil Moisture Characteristic Curve (SMCC) functions using the Fredlund and Xing method [39].

Hydraulic conductivity curves were then estimated from the SMCC function using saturated hydraulic conductivities determined from the profiles presented in **Appendix C3**.

Varying saturated hydraulic conductivity values were used based on the deposited depth of tailings and respective hydraulic conductivity profiles.

The estimated SMCC is presented in **Figure F1** and saturated hydraulic conductivities for the tailings are presented graphically in **Figures F2 and F3**.

### 13.4.3 TSF Foundation Materials

Adopted saturated permeabilities for the foundation materials beneath the TSF main embankment have been inferred from in situ permeability testing (refer **Section 3.7**) conducted within the TSF embankment footprint area. Varying foundation permeabilities were selected based on the location being modelled, as summarised in **Table 13.2**.

**Table 13.2**  
**Saturated Permeability Estimates for TSF Foundation Materials**

Section	Depth	Saturated Permeability ( $k_{sat}$ , m/s)
Main Embankment A	Anchor Trench (0 to 3 m) :	Zone 2 - <b>Table 13.1</b>
	Shallow Foundation (3 to 7 m) :	$1 \times 10^{-5}$
	Deeper Foundation (7 to 11 m) :	$1 \times 10^{-7}$
	Deep Foundation (11 to 15 m) :	$1 \times 10^{-6}$
	Foundation (>15 m) :	$1 \times 10^{-7}$
Main Embankment C	Anchor Trench (0 to 3 m) :	Zone 2 - <b>Table 13.1</b>
	Shallow Foundation (3 to 7 m) :	$1 \times 10^{-7}$
	Deeper Foundation (7 to 10 m) :	$5 \times 10^{-7}$
	Deep Foundation (10 to 15 m) :	$5 \times 10^{-6}$
	Foundation (>15 m) :	$1 \times 10^{-7}$

### 13.4.4 RWP Foundation Materials

Adopted saturated permeabilities for the foundation materials have been inferred from in situ permeability testing (refer **Section 3.7**) conducted within the RWP embankment footprint area. Varying foundation permeabilities were selected based on the location being modelled, as summarised in **Table 13.3**.

**Table 13.3**  
**Saturated Permeability Estimates for RWP Foundation Materials**

Location	Depth	Saturated Permeability ( $k_{sat}$ , m/s)
Central Abutment Hill	*Rock A (0 to 15 m) :	$4 \times 10^{-5}$
	*Rock B (15 to 20 m) :	$2.5 \times 10^{-5}$
	*Rock C (20 to 25 m) :	$3 \times 10^{-6}$
	*Rock D (>25 m) :	$2 \times 10^{-7}$
Embankment	Creek Sand (0 to 5 m) :	$2 \times 10^{-4}$
	Rock A (5 to 12 m) :	$4 \times 10^{-5}$
	Rock B (12 to 17 m) :	$2.5 \times 10^{-5}$
	Rock C (17 to 22 m) :	$3 \times 10^{-6}$
	Rock D (>22 m) :	$2 \times 10^{-7}$

\*Depths shown are below the level of the toe of the central abutment hill.

### 13.5 TSF Embankment Results

The steady state seepage analyses result for the Stage 7 TSF Main Embankments A and C are summarised in **Table 13.4**. The graphical outputs of the seepage analysis results are presented in **Figures F4 and F5** in Appendix F.

**Table 13.4**  
**Seepage Rates for The TSF Embankment**

Analysis Scenario	Decant Pond Level (RL, m)	Seepage Flow Rate ( $m^3/day$ )	Seepage Flow Rate (L/sec)	Figure #
Final Stage (Stage 7) Main Embankment A	320.1	17.3	0.2	<b>Figure F4</b>
Final Stage (Stage 7) Main Embankment C	320.1	90.0	1.0	<b>Figure F5</b>

The seepage rate for the Stage 7 Main Embankment A was calculated for a length of 950 m. The seepage rate for the Stage 7 Main Embankment C was applied to the remaining length of the Stage 7 embankment, which is of the order of 1,500 m. Considering the daily throughput of the TSF decants (bleed water only is approximately 16,000  $m^3/day$ ) and the available RWP pumping capacity of 1,980  $m^3/hour$  (47,500  $m^3/day$ ), the estimated seepage rates from the TSF to the RWP presented in **Table 13.4** are considered to be acceptable and of no detriment to operations.

### 13.6 RWP Embankment Results

The results of seepage analyses for the RWP embankment and central abutment hill are presented in **Table 13.5**. The graphical outputs are presented in **Figure F6 to F10**.

**Table 13.5**  
**Seepage Rates for RWP Embankments and Central Abutment Hill**

Analysis Scenario	RWP Pond Level (RL, m)	Seepage Flow Rate (m <sup>3</sup> /day)	Seepage Flow Rate (L/sec)	Figure #
Central Abutment Hill (no grout or liner)	250	390	4.5	Figure F6
Central Abutment Hill (with grout and liner)	250	65	0.8	Figure F7
RWP North Embankment (with liner, no grout)	250	65	0.8	Figure F8
	252	225*	2.6*	Figure F9
RWP South Embankment (with grout and liner)	250	33	0.4	Figure F10

\*Result for transient analysis after 1 month to represent worst case for short term RWP pond elevation.

The seepage analyses through the RWP central abutment hill indicate that surficial flows could potentially daylight at the downstream side of the hill if the RWP pond level was to be sustained at an elevation greater than RL 250 m, resulting in an estimated flow rate of 4.5 l/sec. With the inclusion of a grout curtain and BGM Liner, the analyses indicate that seepage is unlikely to reach the surface downstream of the RWP, and the flow rate would reduce to 0.8 l/sec.

Analyses for the RWP North embankment indicate that surface flows would be observed downstream of the RWP if the water level rose to RL 252 m for at least 1 month, however this is not predicted to occur for the normal operating pond of RL 250 m. Nonetheless, seepage rates would be relatively low in both instances, with the maximum flow rate observed in the transient analysis being 2.6 l/sec.

Similarly, the case for the RWP South embankment (with grouting and BGM liner) indicates that surface seepage flows are not anticipated downstream of the RWP.

## 13.7 Conclusions

### TSF Embankment

The results for the TSF embankment indicate that a comparatively insignificant seepage flow rate reports from the TSF to the RWP. The low seepage rates and hence velocities are not expected to result in migration of tailings particles through the foundation. Seepage or development of a steady state phreatic surface through the TSF embankments is not anticipated since the embankments incorporate an upstream face BGM liner anchored into a trench in the foundation.

### RWP Embankments

Water balancing modelling indicates that water will consistently be ponded against the upstream lower portions of the RWP South embankment. It is therefore considered necessary to incorporate a grout curtain at the upstream toe of the embankment, which ties into the BGM liner, to prevent excessive seepage downstream of the RWP. The seepage modelling results indicate that with grouting, no surface emergence of seepage occurs downstream of the RWP South embankment, and very low seepage rates of less than 0.4 l/sec predicted for a sustained

pond elevation less than RL 250 m. The “target” minimum pond elevation is 246 m AHD (at the RWP pump inlet).

The seepage analyses for the RWP central abutment hill indicate that without lining or grouting, seepage rates of 4.5 L/sec may occur under sustained ponding at RL 250 m, however only localised surface expressions are predicted at the downstream toe of the hill. The assessment of seepage through the central abutment hill is generally considered conservative since it is based on the highest rock mass permeability derived in the upper 20 m of boreholes drilled around the base of the hill. Since opening of joints due to stress relief is likely the main contributor to the observed results, it is likely that the permeability of the rock mass in the “core” of the hill is several orders of magnitude lower, in which case seepage to the downstream side would be effectively impeded. To investigate this further, it is proposed to drill additional boreholes through the hill once access tracks are created for Stage 1A construction.

The maximum temporary seepage rate beneath the RWP North embankment, assuming a “worst case” sustained pond level of RL 252 m, is 2.6 L/sec.

It is concluded that any potential seepage beneath or around the RWP embankments can be most economically managed by observation and monitoring via sumps and monitoring/recovery bores installed downstream of the embankments as outlined in **Section 12.2**, rather than installation of a plinth and grout curtain at the RWP North embankment or extension of a liner between the embankments.

## 14 EMBANKMENT STABILITY & DEFORMATION

### 14.1 Overview

The following sections detail the static stability, seismic coefficient screening and deformation analyses conducted for the proposed TSF and RWP embankment designs. The governing design principle is that the embankments must be designed such that the integrity of the structures with respect to stability under static and seismic loading conditions is preserved.

#### General Methodology - Stability

Analyses were performed using SLOPE/W software [40] and employing a General Limit Equilibrium (GLE) approach, which satisfies both force and moment equilibrium. Both upstream and downstream slope failures were considered.

The static stability of the RWP embankments has been assessed using the maximum height embankment section, which is considered to be the critical case in terms of stability.

Upstream analyses were conducted for Stage 1 of the TSF embankment at an intermediate filling stage with an elevated phreatic surface level, which was considered a 'worst case' short term scenario. The final tailings level with a normal operating pond was considered for the Stage 7 TSF embankment downstream slope.

The analyses have conservatively adopted an elevated phreatic surface level in the TSF embankment model, representing two possible scenarios:

- (i) Variable rockfill material consistencies in the downstream raising process has resulted in horizontal laminations of lower permeability materials; or
- (ii) a long-term, concentrated leak has formed in the BGM Liner.

Under normal conditions, the BGM Liner would prevent the development of an elevated phreatic surface. The analyses presented are therefore considered worst case scenarios.

#### Seismic Coefficient Screening of Downstream Slope

The Hynes-Griffin and Franklin [41] pseudo-static seismic coefficient method was utilised in a **Safety Evaluation Earthquake (SEE)** pseudo-static analysis to adopt a design embankment downstream slope. Although it is accepted that the use of pseudo-static analyses should not be relied on in the assessment of seismic stability, it is considered a suitable initial screening tool for determining a "safe" slope for an embankment.

The method is applicable to well compacted embankments not susceptible to liquefaction, as is the case with the TSF embankment. It recommends use of a seismic coefficient equal to one-half of peak ground acceleration (PGA) using 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 (with respect to minimum available embankment freeboard) predicted deformations under the **SEE** (safety) criteria. This determination was based upon screening resultant Factors of Safety (FS) being greater than 1.0 (when using seismic coefficient method, an FS of 1.0 indicates deformations of less than 1 m along the nominal failure "plane").

## Deformation Assessment

Empirical database methods have been used to assess the seismic deformation of the TSF embankment profile. In accordance with ANCOLD [19], empirical database methods are considered applicable in cases where there is no potential for liquefaction or significant strain softening, as is the case for the North Star TSF.

Once a satisfactory TSF embankment downstream slope was arrived at using the seismic coefficient screening approach, deformation analyses were carried out to assess that the amount of crest settlement under SEE loading was tolerable.

Provided that the degree of deformation under SEE loading was tolerable, it was not considered necessary to perform similar analysis using the Operating Basis Earthquake (OBE).

## 14.2 Seismicity

For the purposes of the seismic analysis of the TSF2A and RWP embankments, a deterministic, prescribed approach to design has been adopted, which is the method recognised as the most widely used to date. A Probabilistic Seismic Hazard Assessment (PSHA) of the region was conducted by ATCW for the IBO North Star site, with all identified faults added to the seismotectonic model. The PSHA [42] presents an ARI versus ground motion relationship determined for the seismotectonic province in which the North Star project is also located. The ANCOLD Guidelines [28] define this approach to design, with two levels of earthquake motion as follows:

- **Operating Basis Earthquake:** the OBE 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.
- **Safety Evaluation Earthquake:** the SEE will produce the maximum level of ground motion for which the dam should be designed or analysed. It is a minimum requirement that the impounding capacity of the dam be maintained when subjected to that seismic load.

For a “High C” dam failure consequence category, the ANCOLD Guidelines [28] recommend the design earthquake Annual Exceedance Probabilities (AEPs) listed in Table 14.1. From the PSHA [42] the corresponding horizontal peak ground acceleration (PGA) inferred for the OBE and SEE events are also presented in Table 14.1, where ‘g’ refers to the acceleration due to gravity.

**Table 14.1**  
**Summary of Seismic Design Criteria**

Design Event	OBE	SEE
Annual Exceedance Probability (AEP)	1 in 500	1 in 2,000
Peak Ground Acceleration	0.05 g	0.13 g

### 14.3 Embankment Stability Criteria

Embankment stability has been assessed using the following minimum FS as recommended in the ANCOLD [28] guidelines:

- Long term (drained) - FS  $\geq$  1.5
- Short term (no potential loss of containment) - FS  $\geq$  1.3
- Downstream slope (seismic coefficient screening) - FS  $\geq$  1.0

### 14.4 Material Properties

#### 14.4.1 Summary

The material properties used in the stability analyses are summarised in Table 14.2.

**Table 14.2**  
**Materials Parameters for Main Embankment Stability and Screening Analyses**

Zone	Static		Seismic Coefficient Screening		Unit Weight, Y (kN/m <sup>3</sup> )
	c' (kPa)	$\phi'$ (Degrees)	c' (kPa)	$\phi'$ (Degrees)	
2A	Douglas (2003) [43] Shear Stress-Normal Stress Function (refer Figure G1) Note: 20% reduction factor applied for Seismic Coefficient Screening				20
2A (Sensitivity)	0	43	-	-	20
3A	Leps (1970) [44] Lower Bound Shear Stress-Normal Stress Function (refer Figure G2). Note: 20% reduction factor applied for SEE Analyses <sup>1</sup>				20
3B	Douglas (2003) [43] Shear Stress-Normal Stress Function (refer Figure G1). Note: 20% reduction factor applied for Seismic Coefficient Screening				20
3A / 3B (Sensitivity)	0	40	-	-	20
Tailings	0	0	0	0	16
Foundation Gravels	5	32	3	27	
Foundation Rock	125	28	125	28	22

It is well established that rockfill shear strength is a function of the normal effective stress, dry density, particle roughness, particle crushing strength, grain size angularity and uniformity of grading. To reflect this, a number of rockfill shear strength functions are available which results in a curved strength envelope, which best represents the higher frictional strengths at low confining pressures, and the lower strengths at high overburden pressures due to suppressed dilation and particle crushing. These methods are preferred by ATCW as they allow the development of a more realistic model. To justify the use of rockfill shear strength functions, sensitivity analyses have been conducted on the TSF Embankment stability, adopting a constant friction angle with overburden stress, derived from laboratory testing.

The Douglas (2003) [43] and Leps (1970) [44] shear stress-normal stress functions were both considered in assigning material parameters to the embankment granular and rockfill materials.

Leps (1970) is a database method which adopts upper, average and lower bound strength functions based on varying materials of different grading, density and strength. Alternatively, Douglas (2003) adopts a more empirical approach, quantifying strength parameters based on the angularity, grading and measured Unconfined Compressive Strength (UCS) of rock material, which is discussed in more detail in **Section 14.4.2**.

The Douglas (2003) method was selected for Zone 2A and Zone 3B due to the availability of known values to correlate to shear strength. As mentioned in **Section 10.3**, Zone 3A has been incorporated into the design to accommodate blasting product from more weathered and/or finer grained rock bands encountered in quarry areas. Therefore, it was deemed suitable to adopt the more conservative Leps (1970) lower bound function, which is based on a database of comparatively lower density, more poorly graded, less durable particles.

Unit weight values for the compacted rockfill have been adopted based on past experience with compacted, well graded sedimentary rockfill.

#### 14.4.2 Zone 2A and Zone 3B (Shear Stress-Normal Stress Function)

The Douglas (2003) [43] method uses the following relationships:

$$\varphi'_{sec} = a + b \cdot \sigma'_n{}^c$$

$$a = 36.43 - 0.267ANG - 0.172FINES + 0.756(Cc-2) + 0.0459(UCS-150)$$

$$b = 69.51 + 10.27ANG + 0.549FINES - 5.105(Cc - 2) - 0.408(UCS - 150) - 0.408$$

ANG	=	Angularity rating determined by particle shape
FINES	=	Percent passing 0.075 mm
Cc	=	Coefficient of Curvature ( $d_{30}^2 / (d_{10} \cdot d_{60})$ )

Particle size distribution (PSD) results from geotechnical investigations (refer to **Section 3.7**) have been used for selecting parameters for Zone 2A, together with information provided by IBO. For Zone 3B, estimations were made for material PSD based on target material specifications and observations made by ATCW during geotechnical investigations and geological mapping in the area.

**Figure G1 of Appendix G** shows the relationship for Zone 2A and Zone 3B for the static loading condition.

#### 14.4.3 Zone 3A (Shear Stress-Normal Stress Function)

Leps (1970) [44] has shown that shear strength, as expressed by its friction angle, varies markedly as a function of the effective normal stress.

Zone 3A rockfill has been assessed as being comparable to the Leps (1970) lower bound function. The rockfill which will be used in the embankment is potentially of higher quality than these estimates; however, the adopted parameters have been derived to make allowance for areas of lower strength, more weathered materials being present in the proposed rockfill quarries.

**Figure G2 of Appendix G** shows the relationship derived for Zone 3A rockfill.

#### 14.4.4 Tailings & Foundations

The geotechnical investigations have indicated that the alluvial and colluvial deposits in the maximum height (creek) sections of the TSF Main embankment and RWP South embankment consisted of medium to high density sandy gravels. These materials overly weathered rock.

The strength parameters adopted in the stability models are based on the results of the geotechnical investigations and ATCW past experience with similar materials. The TSF embankment foundation rock mass is less fractured than the rock mass at the RWP embankments, hence higher strength parameters have been adopted.

Since the TSF embankments will be downstream raised, the strength parameters for the tailings will not impact on the stability results, hence zero strength has been assigned, although a unit weight of 16 kN/m<sup>3</sup> has been adopted.

#### 14.4.5 Sensitivity Analyses - Zone 2A (Constant Friction Angle)

Single stage Consolidated Drained (CD) and Consolidated Undrained (CU) triaxial tests were conducted on proposed Stage 2 coarse rejects material by E-Precision Laboratory in December 2018. Results of these tests were provided to ATCW by IBO in January 2019, and are summarised in **Table 14.3**.

**Table 14.3**  
**Summary of Coarse Rejects Triaxial Data (Zone 2A)**

Test Type	CU	CD
Cohesion $c'$ (kPa)	61	23
Friction Angle $\phi'$ (deg)	40	44

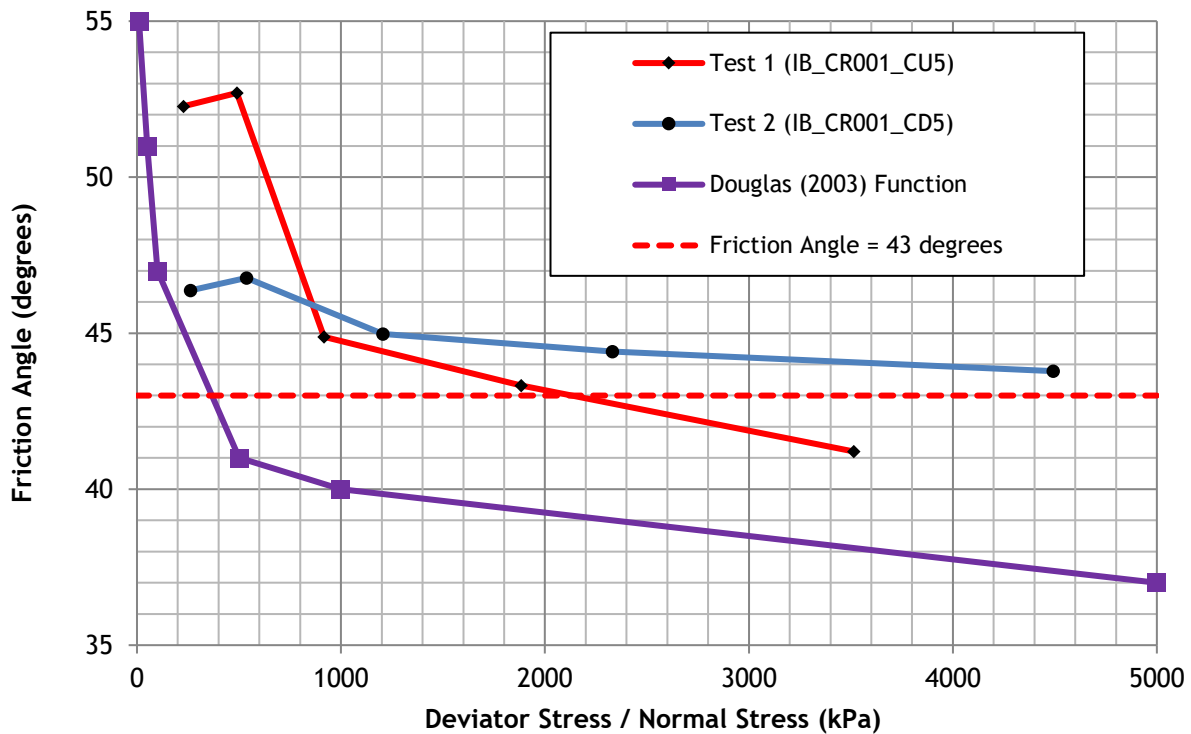
It is commonly accepted practise for granular materials to correct the reported shear strength for zero apparent cohesion, which results in a higher friction angle, with a cohesion of 0 kPa.

For simplicity, upon review of the results presented in **Table 14.3**, it was decided to conservatively apply a constant friction angle of 43 degrees with 0 kPa cohesion (as summarised in **Table 14.2**) to conduct the sensitivity analyses.

To demonstrate the correlation between the non-linear rockfill shear strength function and the above test results, **Chart 14.1** plots the friction angle for each triaxial stage with the deviator stress, together with the Zone 2A Douglas (2003) functions defined in **Section 14.4.2**.

As evident in **Chart 14.1**, the Douglas (2003) function is appropriately conservative when compared to triaxial data, however at shallow depths the rockfill shear strength function estimates higher theoretical shear strengths. Therefore, it is most important that the sensitivity analyses assess shallow upstream failure circles that may result in a localise sliding failure of the BGM liner.

Chart 14.1  
Particle Size Distribution - Coarse Rejects & Zone 2A Filter Criteria



#### 14.4.6 Sensitivity Analyses - Zone 3A & Zone 3B (Constant Friction Angle)

A conservative constant frictional angle of 40 degrees has been applied to both Zone 3A and Zone 3B rockfill zones to provide comparison with the rockfill strength functions adopted.

Similarly to the Zone 2A analysis, this is less conservative than the rockfill strength function at depth, however more conservative when applied to shallow slip circles, which for the downstream slope are considered less significant as they will not result in loss of containment or failure of the BGM Liner.

### 14.5 Embankment Stability Results

#### 14.5.1 Static

A summary of the stability analyses results for the TSF embankment under static loading are presented in **Table 14.4**. The figure numbers in the table refer to graphical output of the critical failure surfaces, which are presented in **Appendix G**.

The RWP embankment stability analyses results are presented in **Table 14.5**.

FS values for each scenario modelled satisfy the stability criteria outlined in **Section 14.3**.

As shown in **Table 14.4**, the FS values for the sensitivity analyses have reduced slightly, however all still exceed the minimum required FS. Based on the information presented in this section, ATCW considers the adoption of rockfill shear strength functions to be appropriately conservative.

**Table 14.4**  
**Stability Analysis Results - TSF Embankment**

Stage	Scenario	Description	Failure	FS		Appendix G Figure No.
				Required	Modelled	
1	Static	Intermediate filling stage, Normal operating pond with elevated phreatic surface.	Upstream	1.5	2.01	Figure G3
1	Static	Sensitivity analyses As above, with constant friction angles	Upstream	1.5	1.81	Figure G4
Final	Static	Final tailings level, Normal operating pond with elevated phreatic surface.	Downstream	1.5	1.67	Figure G5
Final	Static	Sensitivity analyses As above, with constant friction angles	Downstream	1.5	1.62	Figure G6
Final	Static	Sensitivity analyses Empty, constant friction angles, liner stability	Upstream	1.5	1.72	Figure G7
3	Static	Unlined to RL 289 m. Tailings at final Stage 2 level, normal operating pond.	Upstream	1.5	2.11	Figure G8

**Table 14.5**  
**Stability Analysis Results - RWP Embankment**

Water Level	Scenario	Description	Failure	FS		Appendix G Figure No.
				Target	Modelled	
Normal Operating Pond (RL 250 m)	Static	Long term, normal operating conditions	Downstream	1.5	2.09	Figure G9
	Static	Long term, normal operating conditions	Upstream	1.5	1.92	Figure G10
Spillway Invert Level (RL 256.8 m)	Static	Short term, flood conditions	Downstream	1.3	1.93	Figure G11
	Static	Short term, flood conditions	Upstream	1.3	2.10	Figure G12

#### 14.5.2 Pseudo-Static (Screening of Downstream Slope)

As discussed in Section 14.1, pseudo-static analyses have been used as an initial screening tool to assign and check the safety of the downstream slope of the TSF embankment, under the **SEE** loading case. Results of the **SEE** seismic coefficient screening assessment satisfy the target FS, as presented in Table 14.6.

**Table 14.6**  
**Seismic Coefficient Screening Results - TSF Embankment Downstream Slope**

Stage	Scenario	Description	Failure	FS		Appendix G Figure No.
				Required	Modelled	
Final Stage 2 : 1 (H : V) downstream slope	SEE	Final tailings level, Normal operating pond with elevated phreatic surface.	Downstream	>1.0	1.20	Figure G13

The screening result for the 2 : 1 (H : V) downstream slope satisfies the target stability criteria (FS ≥ 1.0) outlined in Section 14.3. Hence, based on the empirical interpretation of this result described in Section 14.1, is estimated from this outcome that deformations resulting from the SEE seismic event would be negligible for the TSF embankment. It can be inferred that this outcome would also apply to the RWP embankment, given the similarity of the material properties, geometry and the lesser height of the RWP embankments.

#### 14.6 TSF Embankment Seismic Deformation Assessment

Although, significant deformation is unlikely based on the outcome of pseudo-static screening analyses, in accordance with ANCOLD guidelines [19] a deformation analysis has been conducted to estimate the TSF embankment deformation (crest settlement) associated with an SEE event.

The embankment foundations or materials are not susceptible to liquefaction or significant strain weakening. Hence the following simplified empirical database methods have been utilised for this assessment:

- Swaisgood (1998, 2003) [45][46]
- Pells and Fell (2002, 2003) [47][48]

The methods outlined above are considered sufficient, provided that the dams are well constructed and there is a large margin between estimated settlement and available freeboard.

The Swaisgood (1998) method utilises a database on embankment dams which have been subject to earthquake, and relates the amount of deformation and cracking to the earthquake loading experienced by the dam. It requires the following inputs:

- Embankment type and height (including soil foundation thickness),
- Earthquake Magnitude and Peak Ground Acceleration (PGA).

Crest settlement (as a percentage of the embankment height) is estimated by:

$$CS = SEF \times RF$$

$$RF = 0.12D^{-.61} \text{ (rockfill dams only)}$$

$$SEF = e^{(0.72M+6.28PGA-9.1)}$$

Where:

*CS* = Vertical Crest Settlement (as % of Dam Height)

*RF* = Resonance Factor

*SEF* = Seismic Energy Factor

*D* = Distance between seismic energy source and dam (km)

*M* = Magnitude of the earthquake

The Swaisgood (2003) [45] method utilises an expanded database on embankment dams and simplifies the Swaisgood (1998) [46] formula as follows:

$$\% \text{ Settlement} = e^{(6.07 \text{ PGA} + 0.57 \text{ M} - 8.00)}$$

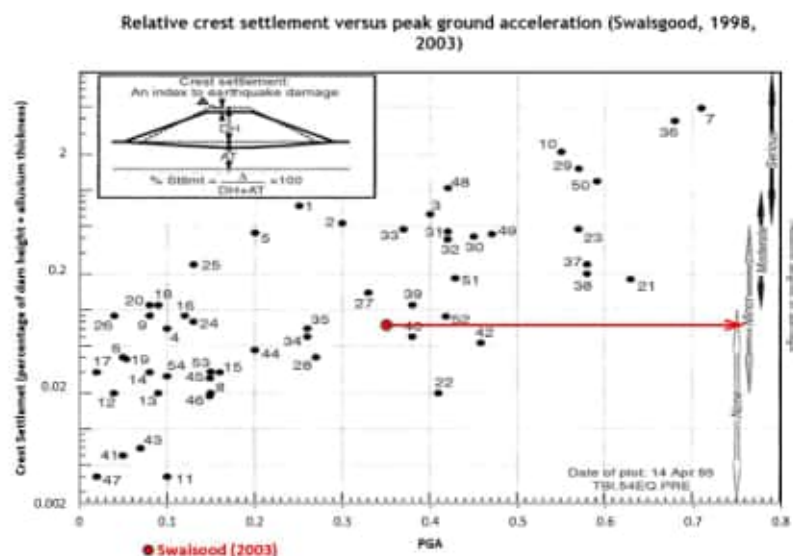
The earthquake data used in the calculations was based on the site-specific seismic hazard assessment (refer to Section 14.2). Input and results are presented in Table 14.7.

**Table 14.7**  
**Swaisgood (1998, 2003) Method for Estimating Seismic Embankment Deformations**

Swaisgood Parameter	Swaisgood (1998)	Swaisgood (2003)
D	25.1	
M	5.6	
PGA	0.35	
RF	0.017	N/A
SEF	0.019	N/A
Embankment Height (m)	65	
CS (%)	0.001	0.07
Estimated Settlement (mm)	0.6	44.2

The results of Swaisgood (1998, 2003) estimates crest settlements in the range of 1 to 44 mm, which indicates a relative degree of damage of “None” to “Minor” based on Chart 14.2.

**Chart 14.2**  
**Swaisgood (2003) Relative Crest Settlement Vs. Peak Ground Acceleration**



The second empirical database method was developed by Pells and Fell (2002, 2003) [47][48], which uses a larger database than Swaisgood (1998, 2003). The method uses two correlations, one for earthfill dams and one for earth and rockfill dams. The latter correlation has been adopted for this analysis. Chart 14.2 shows that the TSF Embankment Damage Class Number plots as Class 0 to 1. Table 14.8 provides an interpretation of the Damage Class Number.

Chart 14.3

Contours of Damage Class Vs. Earthquake Magnitude and PGA (Pells and Fell, 2003)

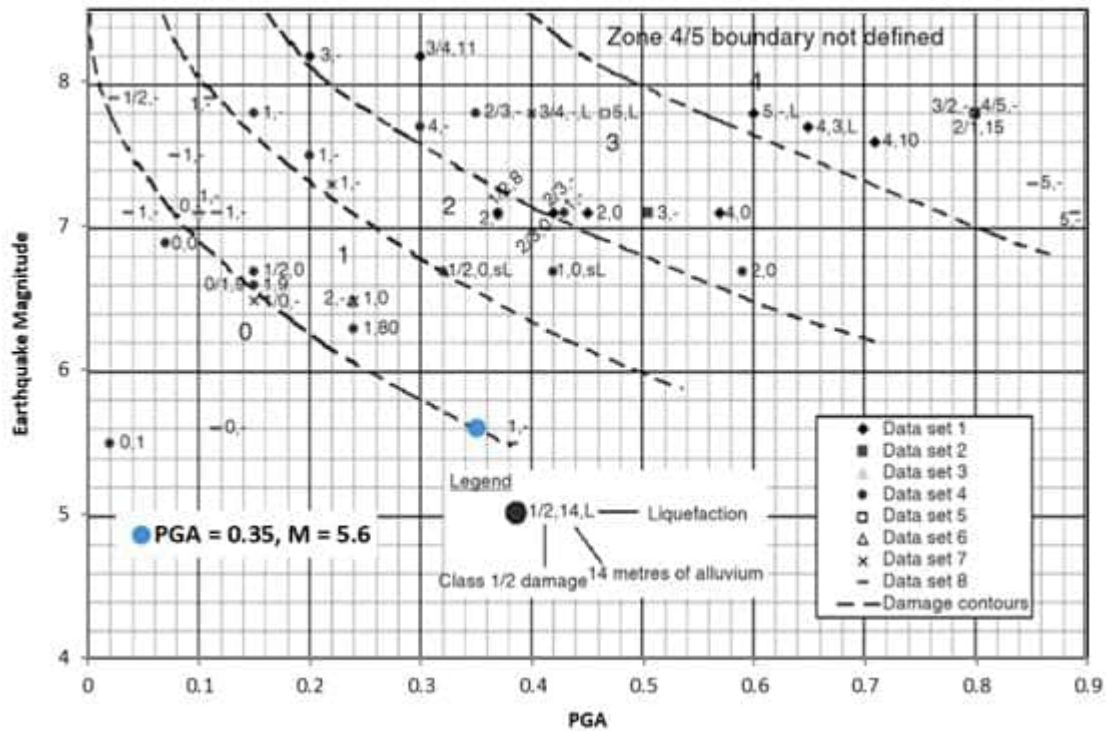


Table 14.8

Damage Classification System (Pells and Fell, 2003)

Damage Class Number	Description	Max. Longitudinal Crack Width (mm)	Max. Relative Crest Settlement (%)	Estimated Settlement (mm)
0	No or Slight	< 10	< 0.03	0 to 19.5
1	Minor	10 to 30	0.03 to 0.2	19.5 to 130
2	Moderate	30 to 80	0.2 to 0.5	130 to 325
3	Major	80 to 150	0.5 to 1.5	325 to 975

The Pells and Fell (2002, 2003) [47][48] method appears to produce a similar result to Swaisgood (1998, 2003) [45][46], with a maximum estimated settlement is in the vicinity of 20 mm, which would not impact on the operation of the TSF.

The Stage 7 TSF embankment has a minimum freeboard to spillway invert level of 2.4 m. Evidently, there is a significant margin between the amount of seismic deformation predicted by the empirical database methods (maximum 44 mm) and the embankment freeboard.

Hence, in accordance with ANCOLD [19], the use of empirical database methods is considered a sufficient level of study and it is not considered necessary to apply more complex numerical methods for estimating deformations during earthquakes.

## 15 WATER MANAGEMENT

### 15.1 TSF Spillways

#### 15.1.1 Overview

The function of the TSF emergency spillways will be to prevent the TSF embankments from overtopping in times of extreme rainfall, particularly as each TSF stage nears capacity and the available flood storage capacity in the decant pond diminishes.

Location of the TSF spillways vary with each stage of construction. The location of the TSF spillways have been based on identifying a natural rock foundation, minimising the cut volume and being located within the footprint of one of the embankments in the subsequent stage of construction. i.e. The Stage 3 spillway will be situated at a location within the footprint of the Stage 4 embankment. The purpose of this is to minimise the number of construction areas for each stage and thereby avoid additional haulage and relocation of construction equipment.

With the exception of the Stage 1A spillway (sacrificial bund), the spillways will be constructed in cut. No erosion protection works have been specified, as the floor and sides of the channels will be in excavated rock.

Water balance modelling has indicated that the TSF spillways are unlikely to flow during the life of mine. Nevertheless, they may be required during sustained, extreme rainfall events at the end of filling of a particular stage.

All spillways (with the exception of Stage 1A) are excavated into competent rock and have been designed with side slopes of 1 : 1 (H : V).

#### 15.1.2 Stage 1A and 1B Spillway

For Stage 1A (0-16 months), the sacrificial bund (refer **Section 7.3**) separating the TSF into a northern and southern valley has been designed to overtop and act as the Stage 1A spillway.

Depending on material placement and BGM liner installation rates, construction of the Stage 1B Main Embankment C will need to commence between 10 and 12 months of commencing operations. At this point, construction of Main Embankment C will take place downstream of the Stage 1A spillway, which will thereby become inactive.

The Stage 1B spillway will provide protection during construction of the Stage 2 embankments, and will ultimately be backfilled to form part of the Stage 2 embankments prior to the commissioning of Stage 2. Excavation of the Stage 2 spillway must be completed by 24 months after commencement of filling.

As detailed in **Section 8.4**, the Stage 1B spillway will have a limited operational life before it has to be backfilled to form part of the Stage 2 TSF embankment. The Stage 1B spillway will have a base width of 20 m and a depth of 1.3 m.

#### 15.1.3 Stage 2 Spillway

Two spillways have been designed for the Stage 2 TSF, with base widths of 25 m. The purpose of this is to position the spillways within the footprint of the Stage 3 embankments, at a suitable

distance from the Stage 1 embankment crests, and to minimise cut volume whilst maintaining spill flow capacity.

Additionally, the location of the Stage 2 spillways allows for optimal positioning of the subsequent stage spillways. The Stage 2 Spillways will have base widths of 25 m (a combined 50 m width) and depths of 1.4 m.

#### 15.1.4 Stage 3 to Stage 7 Spillways

The spillways for Stage 3 to 7 are all located on ridges at the west side of the TSF containment area, as shown in **Figures 114185.14\_018 & 019**. Base widths will be 50 m and depths will be 1.4 m (Stages 3 & 4) and 1.2 m (Stages 5 to 7).

The spillway channel outlets have been positioned to divert any flows into a stream located directly downstream of the main and west embankments, and away from the embankment toe areas to minimise the potential for erosion.

## 15.2 RWP Spillway

The emergency spillway for the RWP will be located at the north abutment of the RWP North embankment and constructed entirely in cut (through competent rock), with the excavated material utilised as construction material for the RWP embankments.

The RWP spillway will have a base width of 50 m, side slopes of 0.5 : 1 (H : V), and an invert level 2.2 m below the RWP embankment crest level of RL 259.0 m.

## 15.3 TSF Decant System

### 15.3.1 Initial Options Assessment

ATCW submitted a High Level Options Study [49] in March 2018, which discussed various options for the TSF Decant Structure and to confirm the most suitable decant system for the North Star TSF.

Options considered were:

- (i) Upstream face inclined decant structure;
- (ii) Decant tower;
- (iii) Syphon; and
- (iv) Floating pontoon.

The alternate decant structure options were compared using a weighted ranking system which accounted for capital and operating costs, ease of raising, power requirements, manual operation and OH&S, operating depths, flood capacity, safety, reparability, manoeuvrability and effectiveness.

Ultimately the option for upstream face inclined decant structure was determined to be the most suitable decant system for the North Star TSF.

### 15.3.2 Description

Two decant systems (North and South decants) will be constructed on the upstream slope of the TSF embankment, as shown in **Figures 114185.14\_012 and 013**. The objective of the decant system is to remove the maximum practicable amount of decant and storm water runoff from the surface of the tailings, and pass this water to the downstream RWP.

Details of the decant system are shown in **Figures 114185.14\_020 and 021**. The system consists of the following components:

- (i) Decant inlet chute system - comprising the decant chute, aluminium stoplogs, safety grates and trash racks, access stairway, handrails, and connection with the decant outlet.
- (ii) Decant outlet - comprising reinforced concrete-encased steel outlet pipe (NB700, API 5L X52, STD wall thickness 9.5 mm) pipe trench backfill, emergency isolation gate valve, precast concrete headwall and downstream stilling basin.
- (iii) Decant operations system - comprising decant access pontoon and stoplog installation arrangements.

The decant inlet chute will be constructed of precast concrete box culvert sections (made to suit), within a reinforced concrete base slab. The decant inlet chute will allow fine control of the pond level (to nominal 200 mm increments) by the placement of heavy duty aluminium stoplog sections inside a frame attached to the decant inlet chute.

For the purposes of designing the decant system, the “normal” operating decant pond depth is considered to be 0.4 m.

The culvert sections will be constructed to the maximum height of each stage. Each pre-cast unit will be placed on a concrete slab, and will have a water-tight seal between each unit as well as having a commercial joint sealing tape on the outside of the joint. Cast in-situ steps and safety hand-rails will allow access full length of the decant chute.

Depending on water level in the tailings storage pond and the water level in RWP, the discharge capacity of the decant system can be controlled by one of the following mechanisms:

- (i) Weir flow control (inlet control);
- (ii) Orifice flow control (inlet control); or
- (iii) Pipe flow control (outlet control).

Two decant systems have been allowed for in the TSF design. The first (North decant) will be constructed as part of the Stage 1A works, whilst the second (South decant) will be constructed with the Stage 1B works. The decant outlet pipelines beneath the embankments will need to be constructed to the full length required for the Stage 7 (final) embankments.

Routine monitoring of both tailings and decant pond levels will determine the actual requirements for decant stoplog placement, however initial estimates are provided in **Section 15.3.4**.

Operational protocols will need to be established for decant management during high rainfall conditions to protect the decant system from potential tailings inundation.

### 15.3.3 Hydraulic Analysis and Design

#### 15.3.3.1 Overview

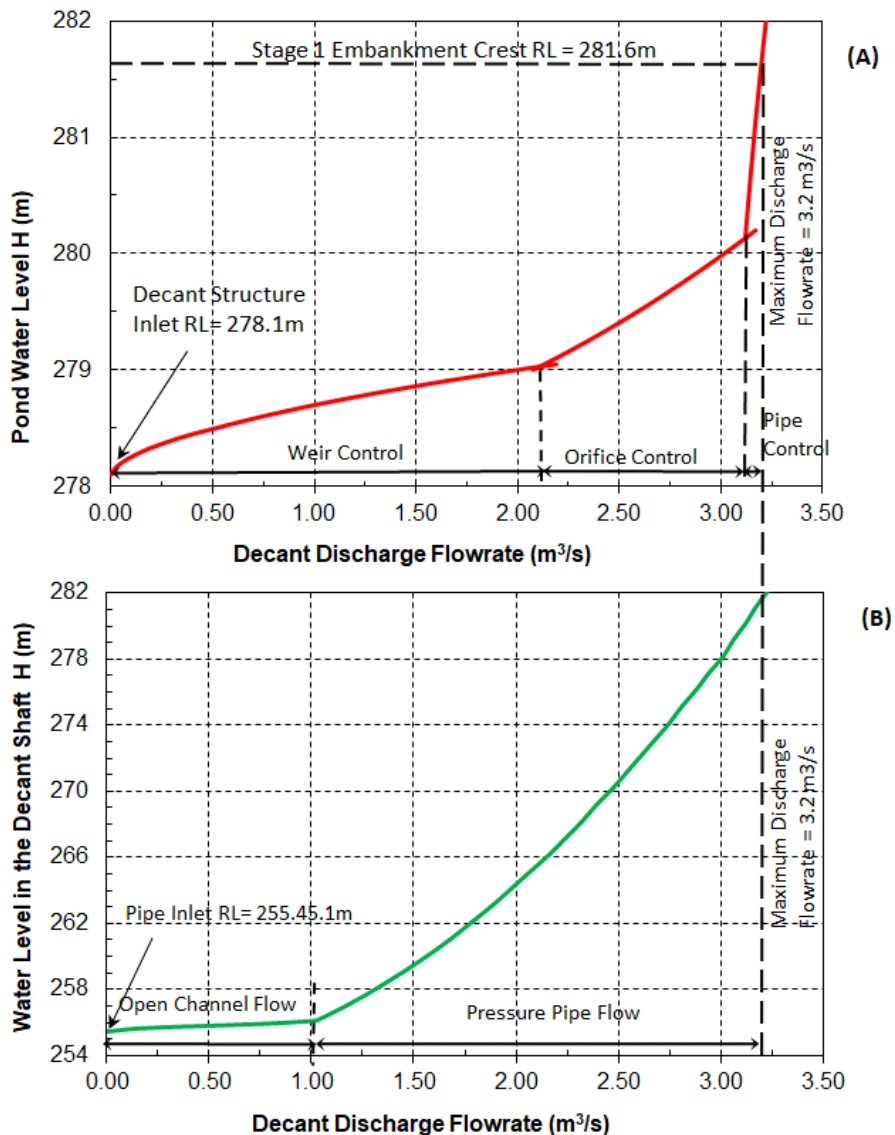
The hydraulic design of the decant system has been based on the removal of a 1 in 100 year ARI storm inflow over an estimated period of time for the respective catchment areas associated with each stage. No losses have been assumed in order to estimate a worst case scenario.

The detailed hydraulic analysis of the decant structures under different flow conditions has been undertaken for the Stage 1 and Stage 7 (Final Stage) embankments to identify and mitigate the associated potential operational risk.

#### 15.3.3.2 Stage 1

The discharge capacity of the decant structure versus water level in the TSF for the Stage 1 embankment is presented in Chart 15.1 (A) & (B).

**Chart 15.1**  
**Decant Structure Capacity vs. Pond Water Level (Stage 1 Embankment)**



As evident in the charts, the maximum discharge capacity of the decant system for the Stage 1 TSF Embankment configuration is  $3.2 \text{ m}^3/\text{s}$ .

**Chart 15.1 (B)** also shows that the discharge pipe operates in open channel flow regime for flowrates less than  $1 \text{ m}^3/\text{s}$ . For flow rates higher than  $1 \text{ m}^3/\text{s}$ , the head water gradually builds up in the decant shaft (chute) and the outlet pipe operates in a pressure pipe flow regime, but as evident in **Chart 15.1 (B)** the water level in the decant shaft does not reach the decant pond water level for the majority of the operating condition.

In other words, there is nearly always a discontinuity between the acting hydraulic head on the decant pipe and the decant water pond within the TSF.

From the hydraulic analysis carried out for the decant structure for the Stage 1 Embankment, the following flow control conditions are identified:

- (i) Weir flow control for flowrates between 0 to  $2.1 \text{ m}^3/\text{s}$  (decant shaft partially full);
- (ii) Orifice flow control for flowrates between  $2.1 \text{ m}^3/\text{s}$  to  $3.1 \text{ m}^3/\text{s}$  (decant shaft partially full); and
- (iii) Pressure pipe flow control for flowrates between  $3.1 \text{ m}^3/\text{s}$  to  $3.2 \text{ m}^3/\text{s}$  (decant shaft full).

Considering the above control conditions (which can occur during the operation of the decant structures), the hydraulic analysis of the system for the transitional states is discussed below.

#### Transition between “Open Channel Flow” and “Full Pipe Flow”

Because the transition between open channel flow to pressure pipe flow occurs at flowrates as low as  $1 \text{ m}^3/\text{s}$  (flow velocity of  $2.5 \text{ m/s}$ ) the dynamic surface impact on the hydraulic performance of the system and structural integrity is negligible during this transition and a smooth transition between the two regimes of flow is expected.

#### Transition from Weir Flow to Orifice Flow

As seen in **Chart 15.1(A)**, the transition between “Weir Flow” and “Orifice Flow” occurs at a flowrate of  $2.1 \text{ m}^3/\text{s}$  while the decant shaft is still partially full and there is no hydraulic connection between the pond water level and the water level within the decant shaft. Under this condition the discharge capacity of the entire system is controlled by the orifice flow which forms at the entrance of the structure. Under this condition the inlet of the decant structure is fully submerged and a local depression will form on the water surface above the decant inlet.

This condition (known as “Slack Flow” in the hydraulic design of pipelines) is encountered in a lot of pipelines designed for transport of water or slurry and is not considered as a risk for the operation.

Slack flow by definition is a phenomena in which a pipeline transporting a liquid develops vapour bubbles at points at which the pipeline pressure falls below the vapour pressure of that liquid. In simple hydraulic terms, what happens in the decant structure in this condition is a jet of high velocity water from the decant chute enters into the static water column formed at the upstream entrance of the outlet pipe, and a submerged hydraulic jumps forms in the decant chute. For the range of flowrates that the decant structure will be operating, this hydraulic jump is not considered to be a concern as the negative pressure from slack flow would not impose any risk to the structure, given that both the reinforced concrete culvert and the steel pipe can withstand the absolute vacuum without detrimental effects.

### Risk of Vortex formation during Orifice Flow control condition

One of the potential risks during the orifice control flow condition is vortex formation at the inlet of the decant structure. Strong vortices can induce air and trash into the decant system and reduce performance of the system [50]. To prevent strong vortex formation a minimum (critical) submergence depth ( $S_c$ ) is recommended [51][52], which is defined as follows (with D being the diameter of the intake):

$$S_c = 1.5 \times D$$

Considering the equivalent diameter of the precast concrete culvert used as the inlet of the decant structure (0.9 m x 0.9 m), the estimated critical submergence depth is 1.5 m, which means for flow depths exceeding 1.5 m over the inlet (i.e. pond level higher than RL 279.6 m for the Stage 1 embankment) the risk of vortex formation is minimum.

However, referring **Chart 15.1(A)** it is identified that when the pond water level is between RL 279 m (start of the orifice control condition) and RL 279.6 m (critical submergence depth), there is a risk of vortex formation at the intake. Based on further assessment, for the following reasons this situation is only expected to produce a weak vortex:

- During the orifice flow condition (when a vortex can potentially form) the decant shaft is partially full, hence the air core of the vortex will not be connected to the water column in the decant shaft; and
- The maximum height of the vortex air core will be limited to 0.6 m (RL 279.6 m - RL 279.0 m).

One of the effective methods to prevent (or reduce) the risk of vortex formation is to use “Anti-Vortex” devices [50][53]. One of the commonly used Anti-Vortex devices is an horizontal perforated plate at the intake. The proposed steel trash rack at the inlet of the decant structure (refer to **Section 15.3.7**) is expected to act as an effective Anti-Vortex device to break the weak vortices that can potentially form at the inlet under some flow conditions.

### Risk of water hammer and dynamic loading impact on the decant outlet pipe during sudden closure of emergency valve

Water hammer and the associated dynamic impact on the hydraulic structures are usually considered a potential risk for operation if there are elements such as valves or pumps etc. in the system that can cause sudden changes in the flow condition. The only such element on the proposed decant structures are the emergency isolation gate valves at the downstream end of the decant pipes.

For the Stage 1 decants, the maximum acting static pressure on the decant outlet pipe is 3 bar. The maximum theoretical dynamic pressure rise in the system due to instant closure of the emergency isolation valve (refer to **Section 15.3.5**), assuming a closure time of 0 seconds, is estimated to be 82 bar (pipe discharging a flowrate of 3.2 m<sup>3</sup>/s at operating velocity of 8.4 m/s).

However, the risk of any dynamic pressure surge will be totally eliminated if the valve closure time is set to longer than the “Critical Closure Time”. Considering the length of the outlet pipes of the two decant structures (L = 350 m), the estimated Critical Closure Time is less than 1 second. In other words, no dynamic pressure surge will form in the system if the valve closure time is set to longer than 1 second.

The hydraulic actuator selected for the valve (refer to **Section 15.3.5**) is set to close the valve in 30 seconds (the closure time is adjustable on the Hydraulic Power Pack Unit). The selected closure time for the valve will totally eliminate the possibility of water hammer and dynamic surge impact on the pipe. It should also be noted that the selected steel pipe for the decant structures can accommodate a Maximum Allowable Operating Pressure (MAOP) of 42 bar and an ultimate pressure of 100 bar.

**Table 15.1** summarises the performance characteristics of the decant structures for the Stage 1 Embankments.

**Table 15.1**  
**Summary of Decant Structure Hydraulic Characteristics (Stage 1 Embankment)**

Embankment Crest RL (m)	Decant Inlet RL(m)	Spillway RL (m)	Maximum Discharge Flowrate (m <sup>3</sup> /s)	Maximum Water Level Acting on the Decant Shaft RL(m)	Maximum Discharge Velocity (m/s)	Maximum Static Head on the Outlet Pipe (m)	MAOP (m)
281.6	278.1	280.3	3.2	281.5	8.4	29	430

### 15.3.3.3 Stage 7

Similarly to the Stage 1 decant, the hydraulic analysis of the decant structure performance has been repeated for the Stage 7 (Final Stage) embankment configuration. The discharge capacity of the decant structure versus water level in the tailings impoundment for Stage 7 is presented in **Chart 15.2 (A) & (B)**.

As seen in these charts, the maximum discharge capacity of the decant structure at its final stage (i.e. Stage 7 Embankment) is about 4.1 m<sup>3</sup>/s. Under the maximum discharge condition, the maximum velocity in the discharge pipe is about 10.8 m/s.

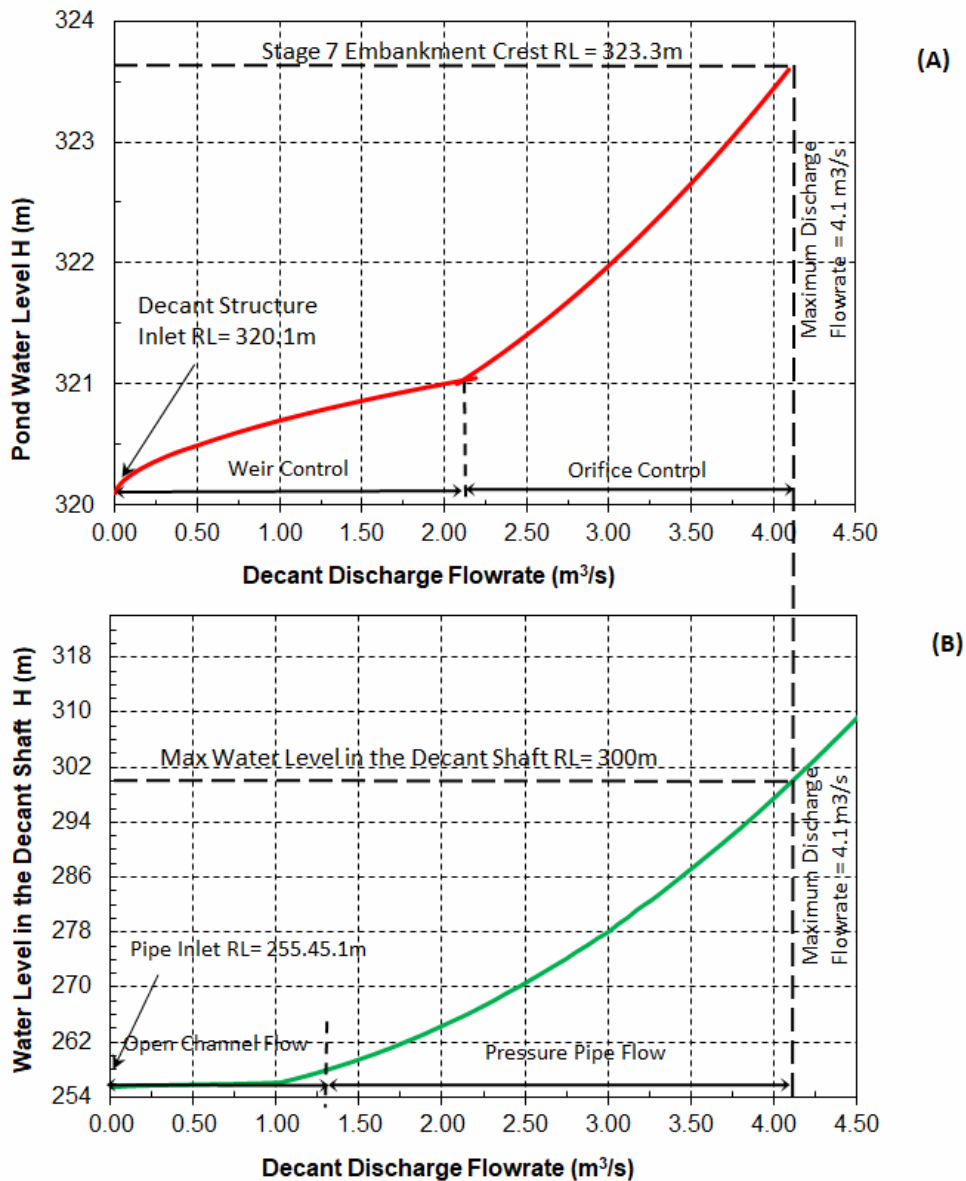
As shown in **Chart 15.2 (B)** under this condition the decant shaft will only be partially full and the discharge flowrate is controlled by the orifice flow condition formed at the inlet of the decant structure. In other words at the final stage of the embankment, when the decant shaft is raised to its final height there will always be a discontinuity between the water head in the storage pond and water head formed in the decant shaft which is acting on the decant outlet pipe.

All of the discussions and analyses regarding the transition flow, risk of vortex formation and dynamic pressure surge and mitigation/elimination of these potential risks for the decant structure operation at Stage 1 embankment remains valid for the Stage 7 embankment. The only difference is that during operation of Stage 7 embankment, a greater hydraulic head (48 m instead of 29 m) will act on the exit pipes of the decant structures.

This means that if the emergency isolation valves are closed instantly (Valve Closure Time 0 seconds) during the Stage 7 operation, the maximum theoretical pressure surge in the system can be about 100 bar. As stated earlier, by selecting a valve closure time of 30 seconds the risk of any dynamic pressure build up in the system is totally eliminated.

Chart 15.2

Decant Structure Discharge Capacity vs. Pond Water Level (Stage 7 Embankment)



The hydraulic characteristics and performance of the decant structures during Stage 7 embankment operation are summarised in Table 15.2 below.

Table 15.2

Summary of Decant Structure Hydraulic Characteristics (Stage 7 Embankment)

Embankment Crest RL (m)	Decant Inlet RL(m)	Spillway RL (m)	Maximum Discharge Flowrate (m <sup>3</sup> /s)	Maximum Water Level Acting on the Decant Shaft RL(m)	Maximum Discharge Velocity (m/s)	Maximum Static Head on the Outlet Pipe (m)	MAOP (m)
323.3	320.1	322.1	4.1	300	10.8	48	430

### 15.3.3.4 Nominal Discharge Rates

The nominal discharge rate under normal operating conditions (i.e. no rainfall and accounting for evaporation) is approximately 5,200 m<sup>3</sup>/day. Nominal decant structure discharge rates (average) are summarised for Stages 1A, 2 and 7 in **Table 15.3**. The time taken to remove water represents the time until the decant pond returns to the normal operating level (i.e. 400 mm).

**Table 15.3**  
**Nominal Decant Structure Discharge Rates**

Stage	1 in 100 ARI Storm Duration (hours)	Run-off Volume (ML)	Assumed Days to Remove Water	Required Discharge Rate (L/s)
1A	2	612	3.0	2,350
	6	980	3.5	3,200
	12	1,332	4.8	3,200
2	2	793	7.0	1,300
	6	1,270	7.3	2,000
	12	1,727	7.6	2,650
7	2	885	6.0	1,700
	6	1,417	6.5	2,500
	12	1,927	7.0	3,200

Flow calculations have been completed to determine the optimum sizing of the various components. The decant chute weir flow capacity is related to the depth of water in the TSF decant pond, whilst the hydraulic discharge capacity of the outlet pipe is a function of the acting head.

Based on these calculations, the decant system will be able to return the decant pond depth back to a “normal operating level” after a 1 in 100 year ARI, 2 hour storm in approximately 3 days during Stage 1A operation and between 6 to 7 days for Stages 2 to 7.

The main components of the decant system are discussed in the following sections.

#### 15.3.4 Decant Inlet Chute System

##### (i) Formed Concrete Chute

The tailings rate of rise during the first two months of operation reduces from 50 m/year to 20 m/year, until the tailings beach develops and the rate of rise steadies. With such a high rate of rise and associated turbulence in the pond, the tailings are unlikely to fully settle and it is expected that no notable decant pond will be able to form. Therefore, the bottom part of the decant chute will be constructed with a reinforced concrete lid to RL 264 m (i.e. the tailings level at approximately 2 months after start of deposition).

This effectively means that the decant system will not operate for the first two months after deposition commences, after which point the tailings rate of rise decreases and placement of aluminium stoplogs must commence to manage the decant pond.

## (ii) Precast Concrete Culverts

As the TSF embankment will be incrementally raised, the decant inlet chute has been designed to be upgraded and extended along with the embankment raises.

The decant chute consists of upturned, precast concrete culverts constructed on a reinforced concrete base positioned on the upstream face of the TSF main embankment. The culverts will be nominally 0.9 m square (internal dimensions), and 1.2 m long.

The Stage 1 culverts will be manufactured to withstand an estimated overall LOM vertical loading imposed by the tailings of approximately 1,150 kPa (unfactored). The culverts have been designed in accordance with AS 1597.2 (Pre-cast reinforced concrete box culverts Large culverts). AS 1597.2 applies load factors based on the installation conditions, which in this scenario is 'embankment installation'. Embankment installation accounts for the effects of "positive projection" and factors the design loads accordingly.

AS 1597.2 adopts maximum loading factors of 1.4 for uncompacted backfill, and 1.15 for compacted backfill (for embankment installations). Although the decant systems will effectively be backfilled by tailings, which are hydraulically placed, it is expected that by the time the culverts are subjected to the worst case loading conditions, the tailings around the culverts will be normally consolidated and hence the system will behave as though compacted backfill has been used.

To confirm this assumption, consolidation testing performed as part of the basis of design verification (**Section 16.4**) during Stage 1A will be used to model consolidation over time.

Therefore a loading factor of 1.4 has been adopted in accordance with AS 1597.2 for embankment installations for uncompacted backfill for Stage 1A only. For subsequent stages a loading factor of 1.15 has been adopted, which is to be verified prior to construction of Stage 1B as described above.

Design loads for the box culverts will incrementally reduce with increased elevation, which will reduce the size and cost of each unit. Culvert structural design has been completed by Humes Holcim of Welshpool, WA (Project Reference Q18-261).

The estimated number of precast concrete culverts (1.2 m units) required for each construction stage of the TSF embankment are summarised in **Table 15.4**.

## (iii) Aluminium Stoplogs

Although equivalent concrete lids are less expensive from a manufacturing perspective, the aluminium stoplog system has significant advantages when compared to concrete, as follows:

- The aluminium stoplog can provide a more reliable watertight seal, exceeding the requirements of the Australian Technical Specification for Fabricated Water Control Infrastructure. The seal is created by an automatic interlocking of the stoplog segments, with no manual intervention required.
- Aluminium stoplogs are significantly lighter with a maximum weight of approximately 150 kg, compared to approximately 520 kg for equivalent concrete lids. Installation of concrete lids would require the use of cranes or additional infrastructure. The lighter weight of the aluminium stoplogs facilitates a much more practical installation procedure.

Placement of aluminium stoplogs to cover the precast culvert sections will be required as the tailings level raises within the TSF. The aluminium stoplogs will be 450 mm wide, constructed of high corrosion resistance marine grade aluminium, and will be sealed in place to minimise ingress of tailings material through the decant system.

The aluminium stoplogs will be manufactured to withstand the estimated overall vertical loading of 1,150 kPa (as per the box culvert design loads).

As with the box culverts, the design load for the aluminium stoplogs will incrementally reduce as the elevation of the tailings at the decant structure raises.

Preliminary stoplog and frame designs have been provided by AWMA Pty Ltd ([www.awmawatercontrol.com.au](http://www.awmawatercontrol.com.au)), with typical details presented in **Figure 114185\_021**.

The estimated number of precast concrete culverts (1.2 m units) and aluminium stoplogs required for each construction stage of the TSF embankment are summarised in **Table 15.4**.

**Table 15.4**  
**Precast Culvert & Aluminium Stoplog Requirements (Total for Both Decants)**

Construction Stage	Culvert	Aluminium Stoplogs
1A	35	88
1B	35	88
2	40	106
3	30	76
4	26	68
5	22	58
6	22	60
7	20	54

During operations, the aluminium stoplogs should be placed in such a way that the vertical height difference between the deposited tailings and the top of the aluminium stoplog is at a minimum of 0.4 m.

Based on advice by AWMA Pty Ltd, the recommended installation method would be to use a light vehicle based crane/hoist to lower the segment into the top of the frame then slide the segment downslope inside the frame rails until it is in position. The inside of the frame may need to be wetted down to overcome the natural resistance between the seal rubber and the frame. However, at all times, the operator is out of the way of the segment.

Average rates of rise and aluminium stoplog placement rates are summarised in **Table 15.5**.

**Table 15.5  
Aluminium Stoplog Placement Rate Requirements**

Construction Stage	Tailings Rate of Rise at Decant Structure <sup>(1)</sup> (m/year)	Aluminium Stop Logs <sup>(1)</sup> (units per month)
1A (2 to 16 months)	14.1	5.9 <sup>(2)</sup>
1B (16 to 24 months)	7.1	5.9 <sup>(3)</sup>
2	3.6	2.9 <sup>(3)</sup>
3	2.5	2.1 <sup>(3)</sup>
4	2.2	1.9 <sup>(3)</sup>
5	1.9	1.6 <sup>(3)</sup>
6	1.9	1.6 <sup>(3)</sup>
7	1.7	1.5 <sup>(3)</sup>

<sup>(1)</sup> Displayed rates of rise and placements rates are averaged over each stage. Actual monthly placement rates will vary, generally reducing with time as the tailings beach develops.

<sup>(2)</sup> Only 1 decant structure is operational during Stage 1A deposition, with the southern decant structure becoming operational at the commencement of Stage 1B.

<sup>(3)</sup> Average aluminium stoplog placement rate includes both decant structures.

### 15.3.5 Decant System Emergency Isolation Valve

An isolation slurry knife gate valve with hydraulic actuator has been considered at the downstream end of the outlet pipes for each decant structure. The valve should only be closed in an unlikely emergency situation such as a decant structural failure whereby tailings enters the decant structure, otherwise the valves should be fully opened at all times.

The selected valves for this application are slurry knife gate valves with hydraulic oil cylinder actuator (ANSI 150 end connection flanges). Because the valves are installed at the downstream end of the outlet pipes, there is a possibility that the valves may become inundated by water level rise in the RWP during severe flood conditions. Hence, the valves should be operable in an inundated condition.

To overcome this issue, the Hydraulic Power Unit (HPU) of the valve needs to be separated from the valve and the actuator cylinder. Two hydraulic rubber hoses that are used to inject the oil into the valve actuator cylinder will be extended from the location of the valve to a platform at a level higher than the maximum RWP inundation level (i.e. higher than RL 259m).

A mobile diesel powered HPU unit on a skid frame (approximate weight 500 kg), which will fit in the back of a Light Vehicle, will be used to inject oil to the actuator cylinder and actuate the valve. Because the oil injection hoses are extended to the above maximum inundation water level, the mobile HPU can be used to actuate the valves under water (if needed) when the valves are inundated in a severe flood condition.

Although the selected valve can operate under water, the outside surface of the valve and the actuator cylinder will need to be protected from water and associated debris during the inundation period. Minor modifications and improvements to the valve standard specifications (i.e. valve and actuator cylinder coating, paint, seals and cover shell) have been recommended by the supplier.

### 15.3.6 Decant Outlet

The decant outlet pipes have been designed to function as a gravity system which will remove decant from the decant chutes for discharge in the downstream RWP. The selected pipe is steel pipe of NB700, API 5L X52, STD wall thickness 9.5mm with internal diameter of 692mm.

The pipe will be completely encased in reinforced concrete. Concrete encasement could not be avoided in the design, due to the estimated overall loading of approximately 1,150 kPa.

A conventional precast outlet headwall will be installed at the outlet, which will discharge into a Zone 3C rockfill - lined stilling basin to dissipate the excess energy which would otherwise scour the RWP base between the RWP operating pond and the outlet points.

The top of decant outlet pipe must be installed at a level lower than the maximum water level of the stilling basin. This ensures that the outlet of the pipe will be fully submerged at high flowrates to obtain better energy dissipation and to reduce the risk of downstream erosion. It also prevents the formation of air locks which could potentially clog the downstream outlet.

### 15.3.7 Decant Access Pontoon

From time to time it will be necessary for operations personnel to gain access to the decant chute inlet to measure the decant pond depth, and to measure the tailings freeboard. In order to provide a safe working environment, some form of floating pontoon securely tethered to the decant system stairway will be required.

The design of a suitable access pontoon has not been undertaken at this stage, but will be required prior to the commissioning of the decant system.

It will be necessary for operations personnel to gain access to the decant chute inlet to measure the decant pond depth, measure the tailings freeboard, or to place a decant lid. In order to provide a safe working environment, a removable safety grate will be installed at the top of the decant inlet chute for each stage. The safety grate will be removed prior to each stage raise, and repositioned at the top of the decant inlet chute for each subsequent raise.

A removable trash rack will also be maintained at the last placed aluminium stoplog. The trash rack will be attached to the most recently installed aluminium stoplog, with the grate positioned inside the box culvert, such that no objects greater than the mesh size can fall down the chute. The trash rack is to be removed prior to placement of each additional stoplog, and replaced onto the new aluminium stoplog. Prior to removal of a trash rack, the operations personnel should attach themselves via a harness and rope to the adjacent hand rail at length that prohibits entry to the open culvert. The operations personnel should not remove the harness until the trash rack has been reinstated in its new position.

## 15.4 TSF Decant Pond Management

### 15.4.1 Decant Pond Operation

The natural topography of the TSF 2A valley, coupled with the tailings deposition methodology, will allow decant water management to occur via the decant system from fixed locations adjacent to the TSF embankment. This is demonstrated in the beach development layouts shown in **Figures 114185.14\_004 to 114185.14\_011**.

Due to the topography of the TSF embankment alignment, the base of the TSF is significantly higher than the RWP base level, which allows decant water to freely flow through the decant structure. Even in extreme rainfall events, there is always sufficient freeboard to transfer catchment runoff to the RWP.

As such a head difference will always be maintained, allowing the TSF gravity decant structure to maintain the decant pond in a minimum condition. The filling of the TSF (i.e. the rising tailings surface) with time is an input to the water balance model. This allows the relative water levels of the TSF decant pond and the RWP pond to be compared.

After significant rainfall events (e.g. 1 in 100 ARI events), the TSF decant pond will be temporarily elevated, however will return to “normal” operating conditions after a number of days, as detailed in **Section 15.3.3**.

#### 15.4.2 Performance of the TSF Decant System

Typical and extreme wet season TSF decant pond level responses were evaluated in the water balance modelling (refer to **Section 7.5**). It is evident that the decant system capacity is adequate, and that the risk of sustained flooding of the tailings beach during the life of mine is very low.

From the statistical analyses of the water balance results, the rate of bleed water escaping from the tailings exceeds the rate of evaporation in the Pilbara region. Hence the decant pond does not reduce below the “normal” operating level (i.e. 400 mm) during the operation of the TSF. Over the LOM, there are not expected to be significant periods (other than extended shut downs) where the decant system is not operational.

### 15.5 RWP Return Water System

#### 15.5.1 Overview

The return water system within the RWP will consist of a pontoon-mounted pumping station. The pumping station will comprise two return water pumps with a combined pumping rate of 1,980 m<sup>3</sup>/hr.

#### 15.5.2 Availability of RWP Return Water

Return water will be re-utilised within the process plant and for various other mine activities. Typical and extreme wet season RWP return water pumping responses were evaluated in the water balance modelling (refer to **Section 7.5**).

In order to avoid the pumps running dry and being damaged, a nominal minimum pond depth of 3.0 m against the RWP South embankment has been modelled in the water balance. The corresponding minimum pond elevation is RL 246 m. The pumps will shut down when this depth is reached, hence maintaining a minimum RWP depth of 3 m throughout the LOM. At times when the RWP is at the minimum level, return water is not available under the modelled scenario.

Evaporation at the North Star site is significantly higher than rainfall but large storms have a significant impact on the RWP size. From the water balance modelling results, both the TSF decant pond and the RWP water level can be expected to exhibit large fluctuations over the LOM, and the TSF and RWP may periodically be at their minimum operating levels. In general, both the

decant pond and the RWP will operate under “normal” operating conditions due to the daily inflow of tailings bleed water (approximately 15,000 m<sup>3</sup>/day).

Statistical analysis has been undertaken of the number of days in the LOM in which return water pumping drops to a very “low” rate (defined in the water balance as a return water pumping rate less than 1,980 m<sup>3</sup>/day, or approximately 4% of the adopted pump capacity of 1,980 m<sup>3</sup>/hr). It can be identified that on average, the number of days with such significantly reduced water return over the LOM can be expected to be in the range of 75 to 165 days. This equates to 1% to 2% of the total mine life.

Statistical analysis of the cumulative volume of water returned back to the process plant has also been undertaken. These results demonstrate that the range of the total availability of return water through LOM for the process plant is expected to be 120 GL to 210 GL, which equates to an annual return water rate of 6 GL/yr to 10.5 GL/yr.

## 16 OPERATIONAL REQUIREMENTS

### 16.1 Management of Tailings Deposition and Water

#### 16.1.1 Design Operational Conditions

Once Stage 1A construction is completed, the slurry distribution pipework will be assembled at the head of the central discharge valley and will extend around the southern and eastern perimeter of the facility to the tributary valley on the northern side of the TSF.

Initially deposition will be from two discharge points (DP1 and DP2) located on the TSF access road around the south-eastern side of the valley. Four additional discharge points will be commissioned along the alignment of the TSF access road during subsequent stages of the TSF construction, and operated on a rotational basis.

Site topography indicates that there are two main flow paths from the south eastern deposition area. For flexibility in operation, initial deposition will be split into two discharge points (DP1 and DP2) to feed both sub-valleys (flow paths), once a sustained return water feed is established at the decant pond.

The optimal discharge points will vary with each stage of the TSF facility development. The stage layouts (**Figures 114185.14\_004 to 114185.14\_011**) each present the optimal discharge points for the respective stage.

The discharge points can be used concurrently or alternated, and it is a design expectation that flow rates between the discharge points be optimised during operations to achieve the design beach profile as presented in the stage layouts and maintain deposition with minimum disruptions.

Pipe flushing, shut-downs, pond control and beach control will all play a part in optimising the deposition regime.

Based on an adopted typical channel width of 50 m and 1% gradient and a channel length of 4.6 km, stream flow along established drainage lines is expected to result in initial water discharge reaching the TSF valley embankment within a period of approximately 6 hours. Nevertheless, initially, there will be a lag in water recovery due to the time it will take to saturate the in-situ soils and for water to accumulate to form a pond of sufficient size to initiate decant chute operation.

The North decant is not expected to be active during the first two months of Stage 1A operation, in order to allow the tailings beach to develop, and the decant water pond to form. Hence, no process water return will be available from the facility (unless rainfall accumulates) during this initial period of operations.

Directional discharge control at the main spigots should be managed on a daily basis to maintain an overall drainage gradient towards the decant area, prevent long term ponding in areas other than the designated decant area and minimise drying beach thickness.

As the tailings beach develops, control of beach slopes will be facilitated by adjusting the number of active discharge spigots and the direction of discharge. To maximise beach slope, at least two spigots should be in operation; however, as far as possible, channel flows originating from the discharge points should not be permitted to intersect on the beach.

The facility Operating Manual (refer to **Section 16.3**) should incorporate a deposition schedule showing the planned sequence of cycling of deposition locations which should be updated once operational trials have been completed and operational experience has been gained.

### 16.1.2 Non-operational Conditions

Non-operational discharge arrangements are shown on **Figures 114185.14\_025** and **114185.14\_026**, representing scenarios for which only one deposition point (DP1) is active during Stage 1B and Stage 2. The plans show that if only DP1 is active, the normal operating decant pond will still distribute along the majority of the embankment length, hence both decant structures will remain operational.

However, substantial ponding will occur at the northern extent of the tailings beach where DP2 would be discharging from under normal circumstances. This pond will be isolated from the decant structures, hence will remain stagnant. Although this is not a dam safety issue and does not affect TSF operations, it is preferable to make available as much decant water return as possible.

## 16.2 Erosion Control

Generation of excessive dust from drying tailings beaches presents a potential environmental and health risk.

The principal mitigating measure to minimise the likelihood of dust generation for the TSF is to operate a cyclic deposition system, whereby fresh wet tailings are discharged over the previously deposited layer at a frequency such that complete drying does not occur. With single point discharge systems, this can be achieved by adjusting the orientation of the discharge outlet.

Other factors that will assist in minimisation of dust generation include:

- The high degree of moisture entrainment within the tailings;
- The presence of strong inter-particle forces due to the fine grain size and cohesive nature of the tailings;
- The anticipated generation of shrinkage and desiccation cracks and cementation of tailings on the surface of desiccation polygons which will resist further breakdown into silt and clay particles (unless disturbed or very wide desiccation cracks form) that may lead to dusting; and
- The presence of continuous containment embankments elevated above the tailings surface which limits the potential for dust transportation across the tailings surface and off the surface.

Provided the facilities are operated in accordance with the design intent and contingency measures utilised if required, the likelihood of dust generation that could adversely affect the surrounding environment is low. Contingency measures could include recycling of water from the RWP to irrigate the dry and dusty areas of tailings as necessary.

## 16.3 Operating Manual

The existing North Star TSF 1 operating manual will be updated prior to commissioning of the much larger TSF 2A. The principal objective of the manual is to provide a documented operation procedure to assist in the safe and efficient storage of tailings and water management in the TSF.

The manual will set out how the TSF can be operated in a way that is in line with the assumptions and principles adopted in the TSF design.

Operating manuals must be prepared in accordance with the DMIRS guidelines [27] to address departmental requirements for the management and closure of TSFs. Operating manuals are to be updated regularly, particularly when design or operational changes occur.

#### 16.4 Basis of Design Verification

As outlined in **Section 2.1**, it is a requirement that verification of the fundamental TSF design parameters be undertaken at defined intervals to reconcile the performance of the TSF with design expectations.

It is considered that annual intervals coinciding with the annual surveillance audits would be appropriate (refer **Section 16.5**). Parameters to be assessed should include:

- Sampling and laboratory characterisation of tailings particle size distribution, specific gravity, Atterberg limits, initial settled density and shrinkage limit density;
- Consolidation (Rowe Cell) laboratory testing and associated consolidation modelling;
- Tailings beach slope and insitu dry density (utilising the data collected as described in the following sections);
- Tailings production rates (to enable calibration of TSF filling rates and ultimate filling level); and
- Water inputs (rainfall runoff, paste thickener overflow) and outputs (evaporation, return water pumping) to enable calibration of water balance parameters.

During Stage 1 deposition a tailings sample should be taken for Rowe Cell consolidation testing to confirm the tailings parameters and enable calibration of TSF filling rates and ultimate filling level. It is likely that this testing would be one off, however if tailings characterisation changes throughout the LoM, additional consolidation testing may be necessary during subsequent verifications.

#### 16.5 Performance Monitoring and Instrumentation

##### 16.5.1 Overview

A program of monitoring and surveillance will be implemented to evaluate the performance of the TSF with respect to the original design expectations. The data obtained will:

- Form the basis of annual surveillance (quarterly in the first year);
- Be used to assess and integrate required maintenance programs;
- Be used in the detailed design of subsequent stage raises; and
- Be used for calibration of the site water balance model.

Some of the key monitoring items will include the following:

- Routine reconciliation of tailings discharge tonnage and solids concentration;
- Routine monitoring of embankment crest elevation;
- Routine monitoring of tailings beach head and beach toe level;

- Routine monitoring of pond water levels and process plant return water rates;
- Routine monitoring of groundwater level fluctuations;
- Routine assessment of groundwater and decant pond water quality; and
- Annual field evaluation of tailings beach density and shear strength profiles.

#### 16.5.2 Instrumentation

There are existing groundwater monitoring bores installed around the proposed TSF and RWP area to monitor groundwater level and quality. As the distribution of bores only partially includes the TSF area, additional bores are proposed, with their final locations to be agreed by IBO. The bores will be sampled on a quarterly basis to provide data throughout the life of the project.

An embankment piezometer network is also proposed for the facility. At the TSF embankment, three sets of Vibrating Wire Piezometers (VWP) will be installed, two sets in the foundations of TSF Main Embankment A, and one set in the foundations of TSF Main Embankment C. The VWP sets will each be comprised of 3 x VWP's located within the embankment footprint. The two sets located within TSF Main Embankment A will each include an additional VWP located directly upstream of the Key Trench. The purpose of the upstream VWP's will be to monitor excess pore pressures in the tailings beach, and hence enable evaluation of drainage and consolidation performance.

One set of VWP's will be installed at each of the RWP embankments, with two at each location.

A series of groundwater monitoring bores will be installed downstream of the RWP embankments. Four monitoring bores will be installed in an arc shape downstream of the RWP North embankment and two monitoring bores will be installed in the narrower creek bed downstream of the RWP South embankment. The bores will be drilled to sufficient diameter to enable placement of 125 mm diameter screen and riser pipe, which will allow the bores to be used for seepage recovery if required.

In addition, a seepage monitoring trench and contingency recovery sump will be provided directly downstream of each RWP embankment.

The proposed arrangement, and sections and details of the VWP's and the RWP downstream monitoring bores are shown on **Figures 114185.14\_025** and **114185.14\_026**, respectively. The data records from piezometer and bore monitoring and sampling will provide timely information on changes in water (phreatic) level within or beneath the embankments and will be used for assessing the operational performance of the TSF and RWP.

A series of survey movement monitoring prisms will be installed on the crests of the RWP North and RWP South embankments. The monitoring prisms will be used to monitor crest settlement of the embankments over time. A total of four monitoring prisms will be installed on the crest of the RWP North embankment at 50 m centres. Five monitoring prisms will be installed on the crest of the RWP South embankment at 30 m centres. The monitoring prisms will be installed in a mass concrete anchor beam, the details of which are shown in **Figure 114185.14\_024**. The proposed arrangement of the RWP monitoring prisms is presented on **Figure 114185.14\_025**.

Survey monitoring prisms may also be installed on the crest of the TSF embankments at selected locations (although at this stage non have been shown in the Figures). Such installations would require relocation after each stage, and hence would only provide short term (2 to 3 year) sequences of cumulative movements.

### 16.5.3 Inspections

Routine inspection of the TSF and RWP will be carried out to confirm that the design strategy is being implemented and to identify any maintenance requirements or any circumstances associated with TSF performance that need further attention.

IBO plant operators will carry out regular inspections of the TSF and its appurtenant features. The operating manual will provide procedures and protocols for inspection requirements in accordance with the facility consequence category. Observations made during the inspections will be recorded and appropriately reported. Any incidents that may occur will be reported in accordance with the facility operating licence conditions and statutory regulations.

In accordance with DMIRS [27] and ANCOLD [28] Guidelines, surveillance requirements for the TSF will generally involve routine daily, weekly and monthly inspections, as well as mandatory annual audits given the TSF is a **Category 1 / High C** consequence category facility. The focus of such surveillance will be as follows:

#### Daily Inspections -

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

#### Weekly Inspections -

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

#### Monthly Inspections -

Focus on issues that may develop over time and impact on the safety of the TSF or environment. These include detailed inspection of the embankments, and all appurtenant structures, tailings characteristics, tailings beach development, decant pond level, decant and return water system operation, tailings and return water pipeline, and surveillance of all monitoring installations.

#### Annual Audits -

These are conducted by a qualified 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. These audits are required to be carried out to ensure compliance with legislation and tenement conditions.

## 17 CLOSURE AND REHABILITATION

### 17.1 Overview

The following general objectives relate to the closure and rehabilitation of the TSF:

- Protect public health, safety and property;
- Ensure long term physical stability of the facility;
- Ensure long term chemical stability of the facility;
- Design for a sustainable ecosystem and land use;
- Employ rehabilitation methods that are technically effective and cost efficient; and
- Implement standard and proven engineering practices to minimise on-going maintenance.

DMIRS guidelines [27] recommend the following process for TSF closure design:

- Decommissioning: to be commenced near or at the cessation of deposition;
- Rehabilitation: to progressively return the disturbed land to a self-sustaining condition; and
- Performance monitoring: to demonstrate the proposed closure design outcomes are achieved.

As required by the guidelines, a Mine Closure Plan incorporating the North Star TSF and RWP has been prepared, and is presented as **Appendix H**. This plan will form part of the Mining Proposal submission.

### 17.2 Closure Design Concepts

Closure design for the RWP area will comprise draining of the residual pond, removal of the RWP embankments (for use in buttressing the TSF main embankment) and grading of the RWP impoundment to remove any transported tailings residue on the ground surface. Vegetation over the majority of the RWP impoundment is unlikely to have been significantly impacted during facility operation (other than during temporary flooding events) and re-establishment of vegetation in the drained operating pond area is likely to occur rapidly with the aid of targeted topsoil placement and nutrient addition.

The downstream slopes of the LOM TSF embankments will be buttressed and regraded to form a stable, erosion resistant long-term landform. Based on current best practice for landform rehabilitation in the Pilbara region, the regrading is likely to comprise either a single concave slope with upper and lower slope angles of 20° and 15° respectively, or a benched slope with bench height of 10 m and berms of sufficient width to contain run off from a range of design storm events. Nominal bench slope angles up to 1V:1.3H (equivalent to a nominal angle of repose of 37°) may be considered, subject to validation of bench stability under post closure conditions (including impact of a Maximum Credible Earthquake).

It is not anticipated that the TSF Embankments will be constructed to the final closure profile during the operational life of the TSF. The key driver for adopting the closure profile at the end of the TSF life span is to reduce upfront capital costs throughout the LoM. Nonetheless, if minimising closure costs becomes a priority in the future it will be possible to adopt the closure profile at an earlier stage of TSF Embankment raises.

Although the tailings beach is likely to have growth medium characteristics similar to the stockpiled 20-year old topsoil, a nominal cover (300 mm - 500 mm) to the beach is likely to be required to reduce the long-term likelihood of dust generation and scour due to concentration of rainfall run off.

As there will be insufficient topsoil recovered from the TSF impoundment to fully cover the LOM beach, the cover material is likely to predominantly comprise waste material from the Stage 2 mining area, with localised areas designated for topsoil spreading, fertilisation and establishment of vegetation.

The available waste materials will include oxide waste rock, fresh waste rock and coarse rejects material. The oxide waste is likely to be reasonably well graded and a potential foundation for growth medium. The fresh waste is expected to predominantly comprise cobble and boulder size material of extremely hard and durable material, suitable for long term erosion protection to design drainage pathways on the landform surface. Coarse rejects may be used to reshape strategic areas of the landform surface prior to erosion resistant cover placement.

Towards the end of deposition, progressive rehabilitation of the tailings beach head and strategic areas of the LOM beach may be implemented by extending the spigot lines across the beach head. Targeted tailings discharge to strategic parts of the lower beach may also be implemented to reshape the tailings surface prior to cover placement. Progressive rehabilitation in a mine scale context may include creation of a new waste dump at the beach head area, thus removing the need to clear virgin ground for waste dump siting.

Closure design will incorporate hydrological flood routing across the final design landform, up to and including probable maximum flood (PMF) conditions. The LOM TSF spillway suitability to cater for PMF conditions will be evaluated as an outcome of such modelling and if required, the spillway will be widened, deepened or relocated. Relocation may be the ultimate outcome depending on the extent to which the main embankment reprofiling impacts the LOM spillway discharge route to the RWP area.

Reshaping and contouring of the cover surface will be optimised to reduce the likelihood of high velocity, erosive flow concentrations occurring and to provide localised water "traps" where topsoil placement and re-vegetation is proposed.

## 18 CONSTRUCTION QUANTITIES AND TSF EXPANSION OPTIONS

### 18.1 Civil Works Quantities

Schedules of estimated quantities for the RWP embankments and the seven stages of TSF embankment construction works are presented in **Appendix I**. The schedules include all embankments, civil works and mechanical works items described in this report.

A summary of quantities relating to civil earthworks construction is presented in **Table 18.1**.

**Table 18.1**  
**Summary of Embankment Civil Earthworks Quantities**

Storage	Stage	Zone 2A m <sup>3</sup>	Zone 3A m <sup>3</sup>	Zone 3B m <sup>3</sup>	Stage Volume m <sup>3</sup>	BGM Liner m <sup>2</sup>
RWP	1A	13,500	22,000	60,000	95,000	10,000
TSF	1A	50,000	240,000	440,000	730,000	30,500
	1B	16,000	90,000	300,000	406,000	12,000
	2	50,000	270,000	685,000	1,005,000	38,000
	3	45,000	230,000	970,000	1,245,000	33,000
	4	45,000	235,000	1,215,000	1,495,000	33,100
	5	45,000	240,000	1,420,000	1,705,000	34,500
	6	45,000	245,000	1,710,000	2,000,000	35,500
7	45,000	235,000	1,950,000	2,230,000	34,000	
<b>Total</b>		354,500	1,807,000	8,750,000	10,911,000	260,600

Note: Quantities shown are incremental values.

It is evident that the start-up (Stage 1A) construction of the TSF and RWP embankment works will require approximately 825,000 m<sup>3</sup> of rockfill and granular fill materials, and installation of 40,500 m<sup>2</sup> of BGM liner. The volumes of the existing Stage 1 mine coarse rejects stockpile and waste dumps are understood to be approximately 140,000 m<sup>3</sup> and 330,000 m<sup>3</sup> respectively. This indicates that there are enough coarse rejects available to supply Zone 2A material up to Stage 2 TSF construction; but insufficient waste rock to complete Stage 1A construction without blasting and recovering quarried material from the TSF impoundment.

Total quantities to complete the embankments to the final, Stage & TSF configuration involve the placement of approximately 10,911,000 m<sup>3</sup> of material, and installation of 260,600 m<sup>2</sup> of BGM Liner.

## 18.2 Contingency TSF Stage 8 Embankment

As outlined in Section 2.1, the North Star Magnetite - Stage 2 Project has a nominal maximum LOM of 24 years, However IBO have advised that the TSF have a design life of 20 years. As a contingency item, a TSF Stage 8 embankment has been designed with an additional 4 year life span, to extend the design life of the TSF to 24 years.

Based on projected annual tailings production rates, the contingency TSF Stage 8 Embankment would require an additional capacity to store 107 Mt, or 63 Mm<sup>3</sup> of tailings at the adopted overall tailings dry density of 1.7 t/m<sup>3</sup>.

To accommodate this additional volume, the crest level of the TSF embankments must be raised by 6.5 m, from RL 323.3 m (Stage 7) to RL 329.8 m (Stage 8).

The Stage 8 raise will require an additional embankment volume of approximately 3,089,000 m<sup>3</sup>, and the installation of a further 35,000 m<sup>2</sup> of BGM Liner.

## 19 CLOSURE

An Executive Summary is provided at the front of this report.

The reader's attention is also drawn to the "Conditions of Investigation and Report" which appear on the title page of this report.

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# IRON BRIDGE OPERATIONS PTY LTD NORTH STAR MAGNETITE - STAGE 2

## TAILINGS STORAGE FACILITY 2A & RETURN WATER POND (TSF2A & RWP) MINING PROPOSAL DESIGN FIGURES

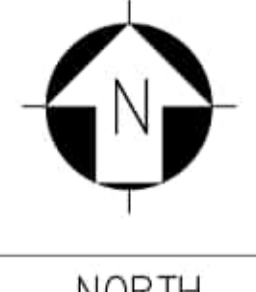


FIGURE NUMBER	FIGURE TITLE
114.185.14_001	COVER PAGE
114.185.14_002	OVERALL MINE PLAN
114.185.14_003	SITE LAYOUT
114.185.14_004	TSF STAGE 1A (MONTH 16 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_005	TSF STAGE 1B (MONTH 24 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_006	TSF STAGE 2 (YEAR 5 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_007	TSF STAGE 3 (YEAR 8 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_008	TSF STAGE 4 (YEAR 11 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_009	TSF STAGE 5 (YEAR 14 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_010	TSF STAGE 6 (YEAR 17 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_011	TSF STAGE 7 (YEAR 20 TAILINGS DEPOSITION) - GENERAL LAYOUT
114.185.14_012	TSF STAGE 1A - GENERAL ARRANGEMENT
114.185.14_013	TSF STAGE 1B - GENERAL ARRANGEMENT
114.185.14_014	TSF EMBANKMENTS SECTIONS & DETAILS (SHEET 1 OF 4)
114.185.14_015	TSF EMBANKMENTS SECTIONS & DETAILS (SHEET 2 OF 4)
114.185.14_016	TSF EMBANKMENTS SECTIONS & DETAILS (SHEET 3 OF 4)
114.185.14_017	TSF EMBANKMENTS SECTIONS & DETAILS (SHEET 4 OF 4)
114.185.14_018	TSF SPILLWAYS - SECTIONS & DETAILS (SHEET 1 OF 2)
114.185.14_019	TSF SPILLWAYS - SECTIONS & DETAILS (SHEET 2 OF 2)
114.185.14_020	TSF DECANTS - SECTIONS & DETAILS (SHEET 1 OF 2)
114.185.14_021	TSF DECANTS - SECTIONS & DETAILS (SHEET 2 OF 2)
114.185.14_022	RWP EMBANKMENTS - GENERAL ARRANGEMENT
114.185.14_023	RWP EMBANKMENTS - TYPICAL SECTIONS & DETAILS (SHEET 1 OF 2)
114.185.14_024	RWP EMBANKMENTS - TYPICAL SECTIONS & DETAILS (SHEET 2 OF 2)
114.185.14_025	STAGE 1B END OF YEAR 2 TAILINGS DEPOSITION - SINGLE POINT DEPOSITION (DP1)
114.185.14_026	STAGE 2 END OF YEAR 5 TAILINGS DEPOSITION - SINGLE POINT DEPOSITION (DP1)
114.185.14_027	TSF & RWP INSTRUMENTATION - LAYOUT
114.185.14_028	TSF & RWP INSTRUMENTATION - DETAILS

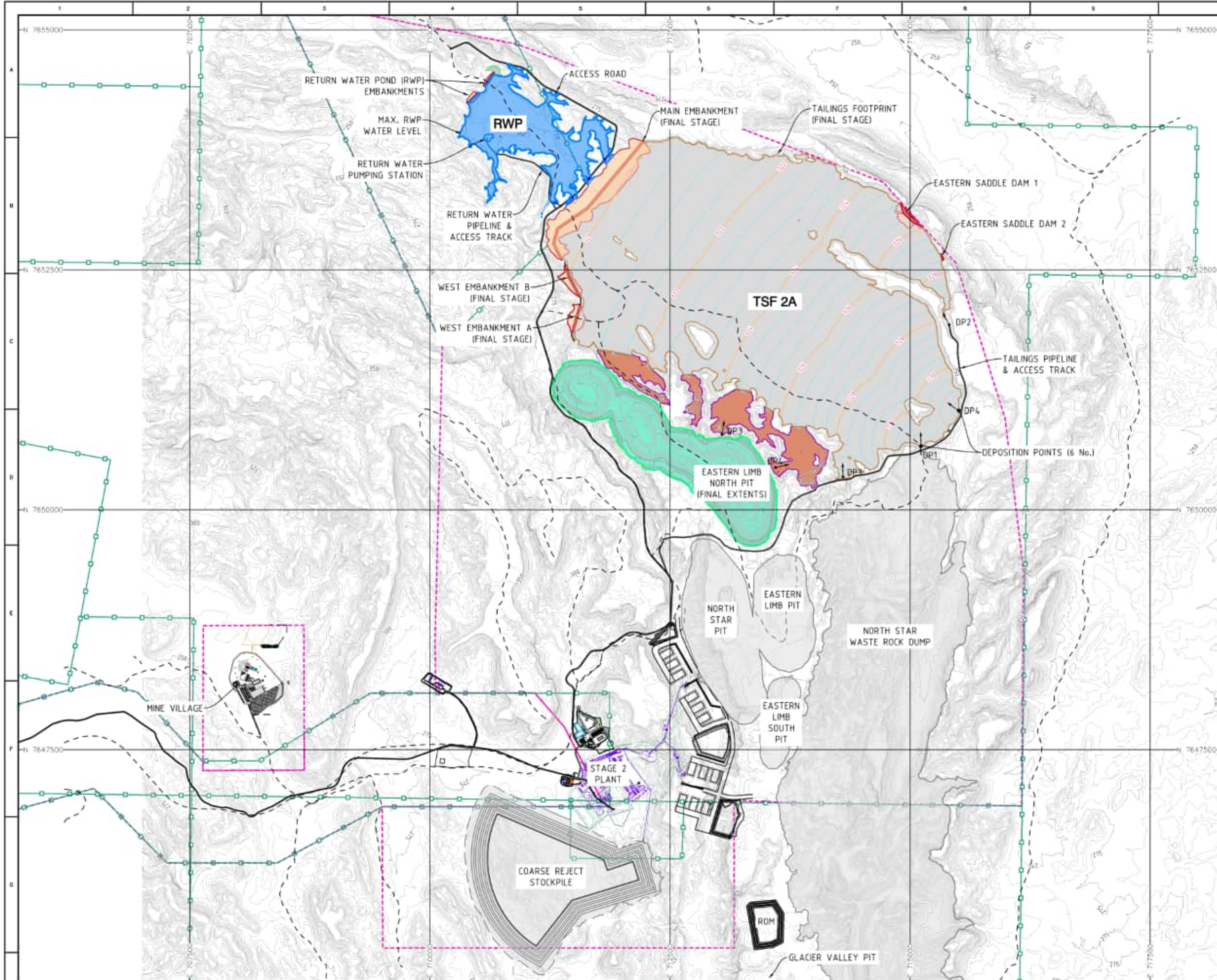
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			A	26.10.18	ISSUED FOR CLIENT REVIEW	HR	CN					



PROJECT NUMBER	114185.14		
Fortescue Metals Group Ltd			
NORTH STAR MAGNETITE STAGE 2			
TSF 2A & RWP			
COVER PAGE			
SCALE	DRG No:	REV:	
A3	NTS		



- LEGEND**
- EXISTING ACCESS ROADS
  - TENEMENT BOUNDARIES
  - PART IV BOUNDARY
  - CONTOUR 25m
  - CONTOUR 5m
  - DP# TAILINGS DISCHARGE POINT
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**NOTES:**

- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50

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VENDOR/DESIGNER DRG No. 114185.14\_002 REV: C

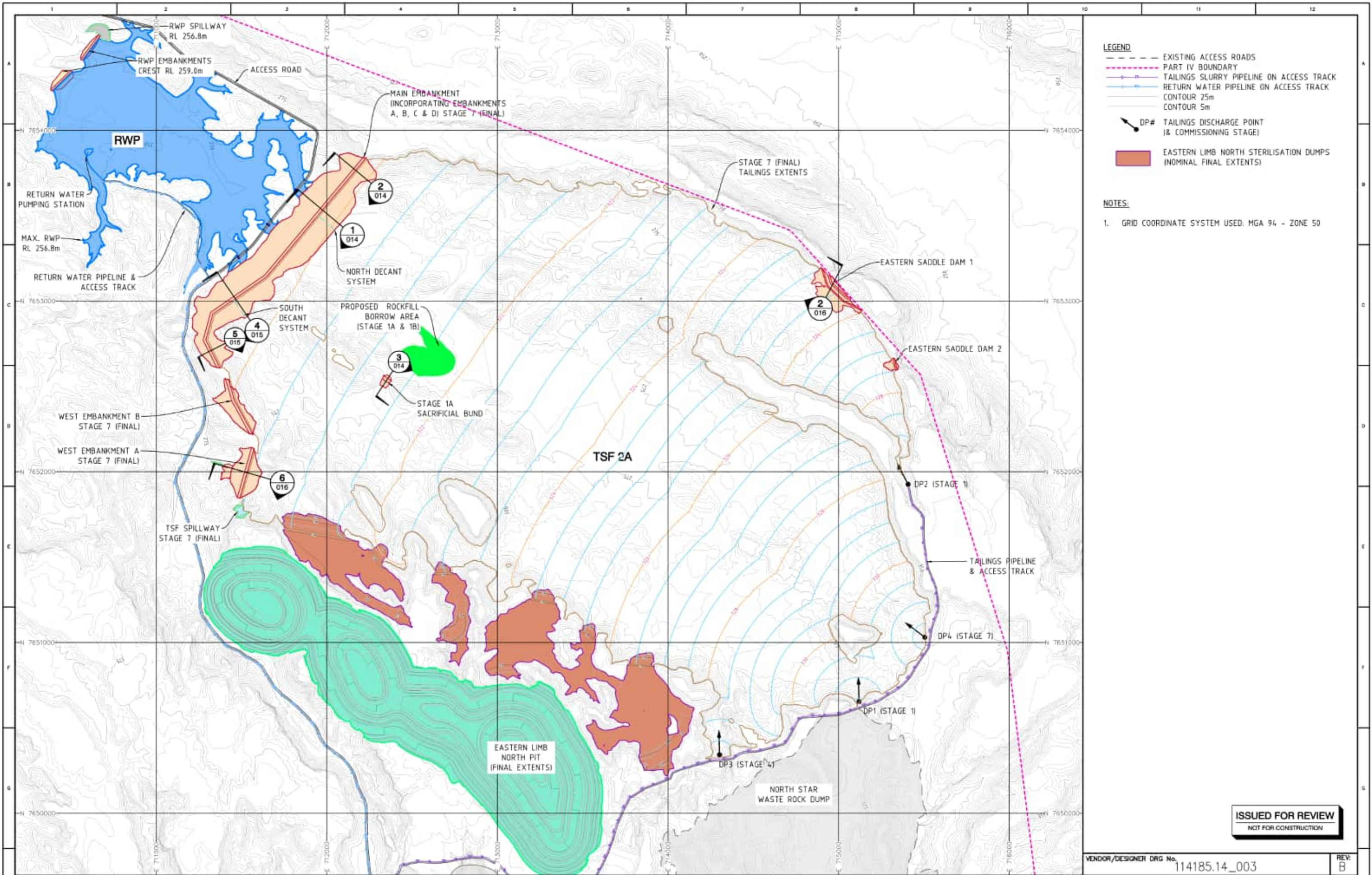
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			A	26.10.2018								



PROJECT NUMBER  
114185.14  
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Fortescue Metals Group Ltd  
NORTH STAR MAGNETITE STAGE 2  
TSF 2 & RWP  
OVERALL MINE PLAN

SCALE 1:40000 DRG No. REV:



- LEGEND**
- EXISTING ACCESS ROADS
  - - - PART IV BOUNDARY
  - TAILINGS SLURRY PIPELINE ON ACCESS TRACK
  - RETURN WATER PIPELINE ON ACCESS TRACK
  - CONTOUR 25m
  - CONTOUR 5m
  - DP# TAILINGS DISCHARGE POINT (A COMMISSIONING STAGE)
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**NOTES:**

- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50

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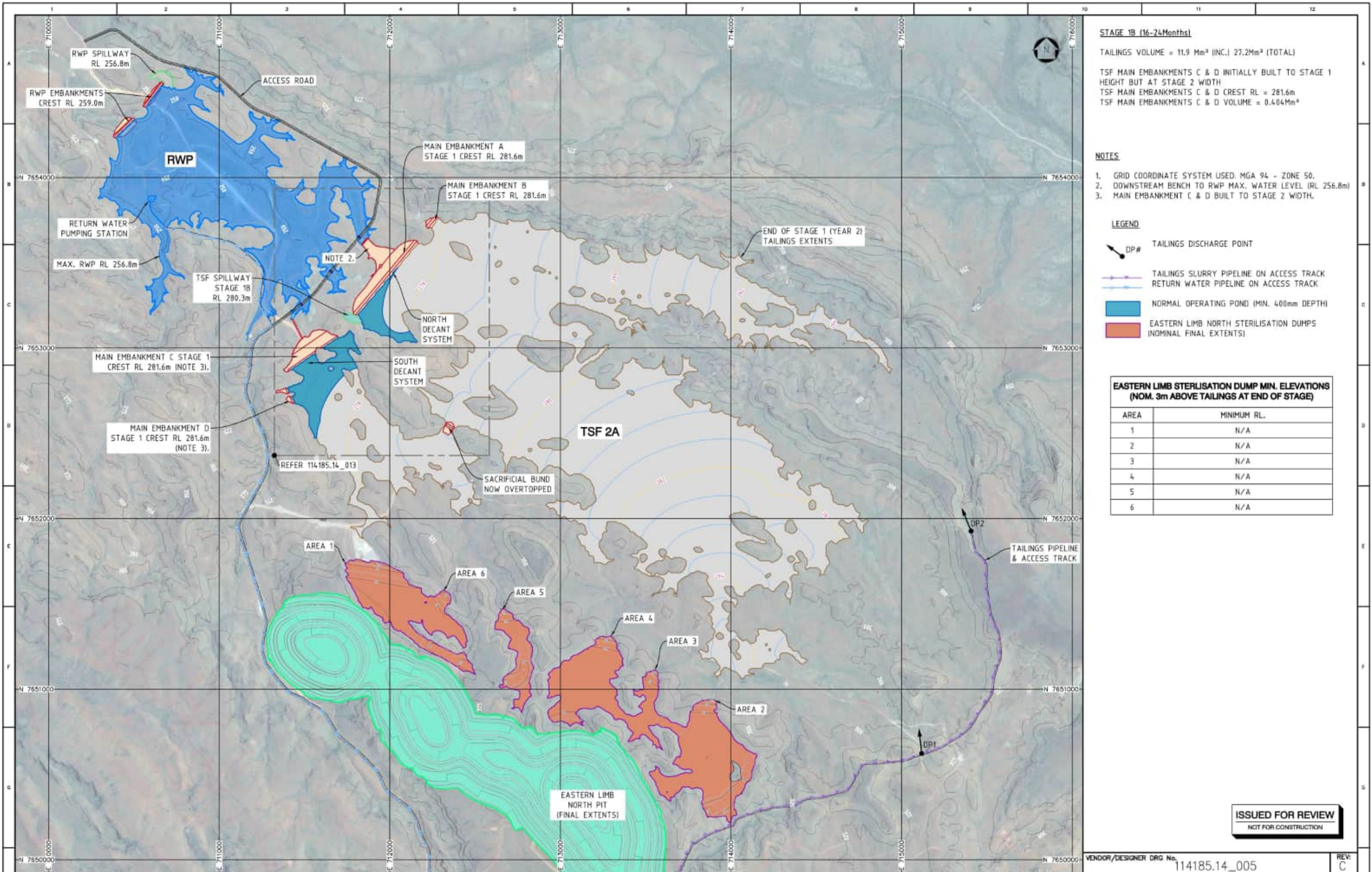
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PROJECT NUMBER 114185.14	Fortescue Metals Group Ltd	
	NORTH STAR MAGNETITE STAGE 2 TSF2A & RWP SITE LAYOUT	
SCALE 1:20000	DRG No:	REV:
	A3	

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		B	27.04.2024								
		A	26.10.2018								







**STAGE 1B (16-24Months)**  
 TAILINGS VOLUME = 11.9 Mm<sup>3</sup> (INC.) 27.2Mm<sup>3</sup> (TOTAL)  
 TSF MAIN EMBANKMENTS C & D INITIALLY BUILT TO STAGE 1 HEIGHT BUT AT STAGE 2 WIDTH  
 TSF MAIN EMBANKMENTS C & D CREST RL = 281.6m  
 TSF MAIN EMBANKMENTS C & D VOLUME = 0.404Mm<sup>3</sup>

- NOTES**
- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.
  - DOWNSTREAM BENCH TO RWP MAX. WATER LEVEL (RL 256.8m)
  - MAIN EMBANKMENT C & D BUILT TO STAGE 2 WIDTH.

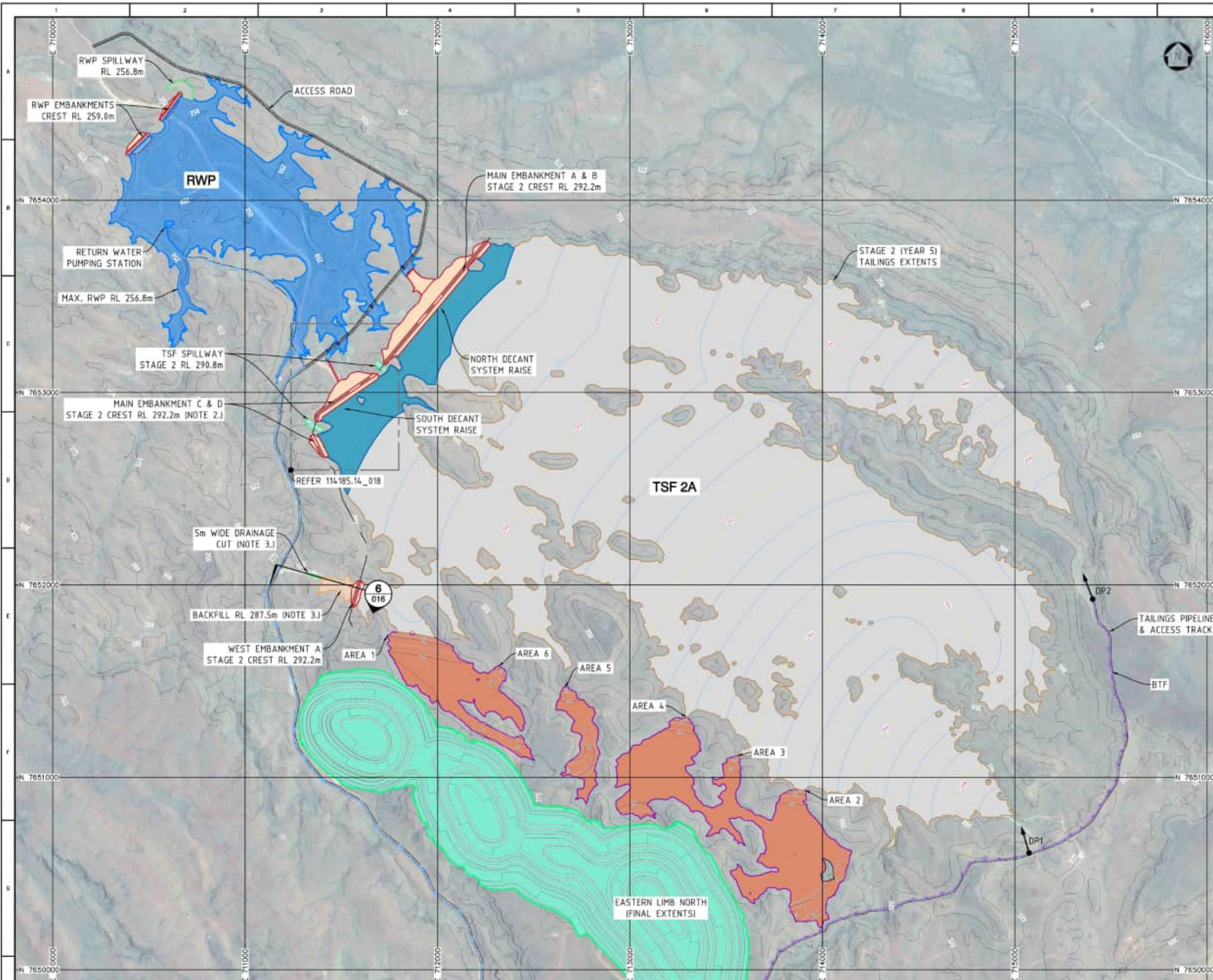
- LEGEND**
- DP# TAILINGS DISCHARGE POINT
  - TAILINGS SLURRY PIPELINE ON ACCESS TRACK
  - RETURN WATER PIPELINE ON ACCESS TRACK
  - NORMAL OPERATING POND (MIN. 400mm DEPTH)
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

AREA	MINIMUM RL.
1	N/A
2	N/A
3	N/A
4	N/A
5	N/A
6	N/A

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REF. DRG. No.		TITLE		REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG	PROJECT NUMBER	114185.14		Fortescue Metals Group Ltd		
														NORTH STAR MAGNETITE STAGE 2		TSF2A & RWP		STAGE 1B END OF YEAR 2 TAILINGS DEPOSITION	
														SCALE		DRG No:		REV:	
														A3		1:20000			



**STAGE 2 (YEARS 2-5)**  
 TAILINGS VOLUME = 52.1 Mm<sup>3</sup> (INC.) 79.3Mm<sup>3</sup> (TOTAL)  
 TSF MAIN EMBANKMENTS A & B CREST RL = 292.2m  
 TSF MAIN EMBANKMENTS A & B VOLUME = 0.792Mm<sup>3</sup>  
 TOP HAT RAISE TO FINAL STAGE 2 HEIGHT  
 TSF MAIN EMBANKMENTS C & D CREST RL = 292.2m  
 TSF MAIN EMBANKMENTS C & D VOLUME = 0.181Mm<sup>3</sup>

- NOTES**
- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.
  - MAIN EMBANKMENT C & D TOP-HAT RAISED TO STAGE 2.
  - THE LOW-LYING AREA DOWNSTREAM OF THE STAGE 2 EMBANKMENT SHALL BE BACKFILLED, WITH AN ACCOMPANYING DRAINAGE CUT, TO AVOID PONDED WATER. REFER TO SECTION **6 016**

- LEGEND**
- DP# TAILINGS DISCHARGE POINT
  - TAILINGS SLURRY PIPELINE ON ACCESS TRACK
  - RETURN WATER PIPELINE ON ACCESS TRACK
  - NORMAL OPERATING POND (MIN. 400mm DEPTH)
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

AREA	MINIMUM RL.
1	292.0m
2	297.8m
3	297.8m
4	295.8m
5	N/A
6	N/A

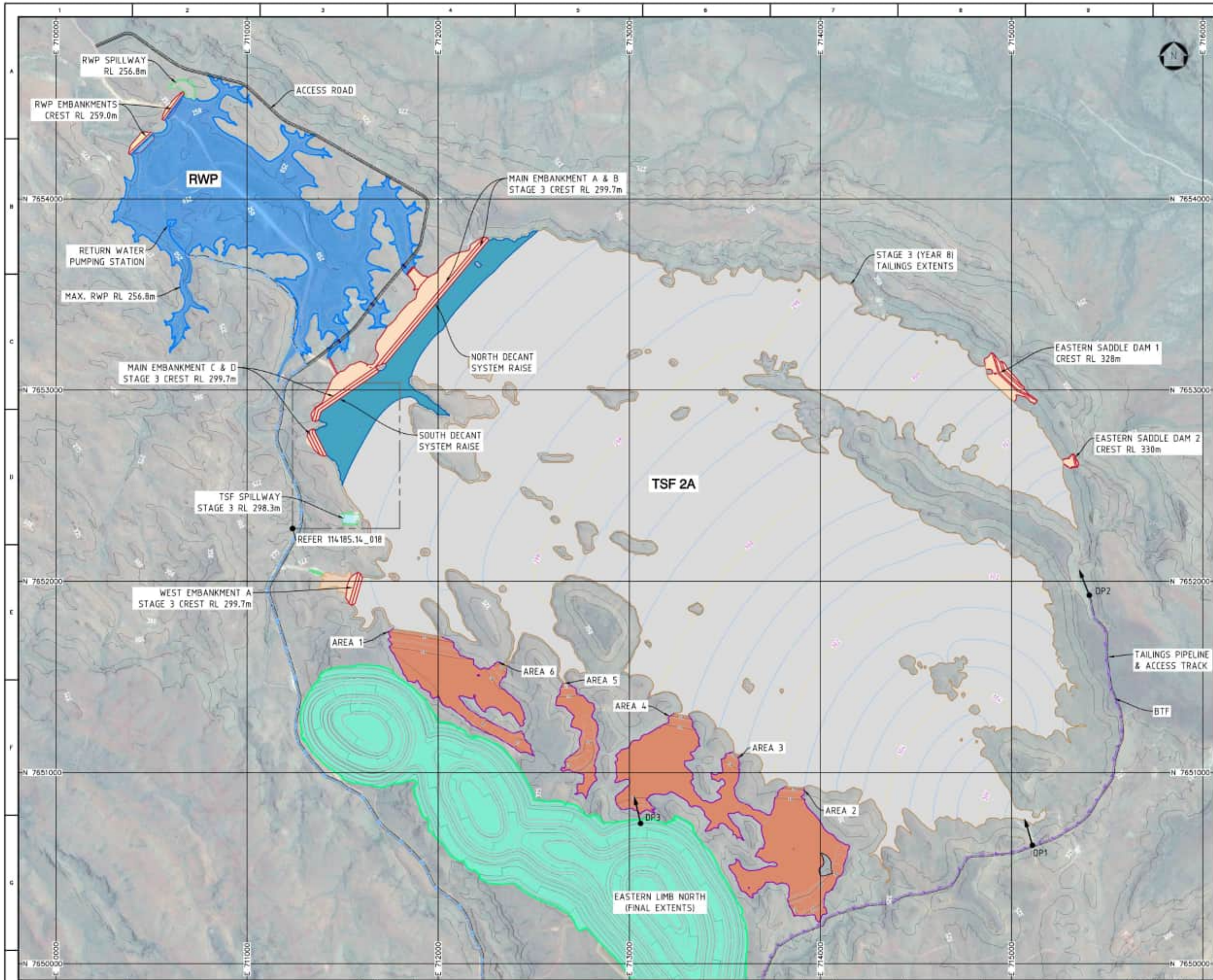
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PROJECT NUMBER 114185.14  
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 NORTH STAR MAGNETITE STAGE 2  
 TSF2A & RWP  
 STAGE 2 END OF YEAR 5 TAILINGS DEPOSITION  
 SCALE 1:20000 DRG No: A3 REV:

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		C	26.03.2024								
		B	18.02.2019								
		A	26.10.2018								





**STAGE 3 (YEARS 5-8)**

TAILINGS VOLUME = 51.1 Mm<sup>3</sup> (INCJ 130.4Mm<sup>3</sup> (TOTAL))

TSF MAIN EMBANKMENT CREST RL = 299.7m  
 TSF MAIN EMBANKMENT VOLUME = 1.27Mm<sup>3</sup>

EASTERN SADDLE DAM 1 CREST RL = 328m  
 EASTERN SADDLE DAM 1 VOLUME = 0.1Mm<sup>3</sup>

EASTERN SADDLE DAM 2 CREST RL = 330m  
 EASTERN SADDLE DAM 2 VOLUME = 0.009Mm<sup>3</sup>

**NOTES**

1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.

**LEGEND**

- DP# TAILINGS DISCHARGE POINT
- TAILINGS SLURRY PIPELINE ON ACCESS TRACK
- RETURN WATER PIPELINE ON ACCESS TRACK
- NORMAL OPERATING POND (MIN. 400mm DEPTH)
- EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

AREA	MINIMUM RL
1	299.6m
2	305.5m
3	304.4m
4	303.5m
5	300.8m
6	300.5m

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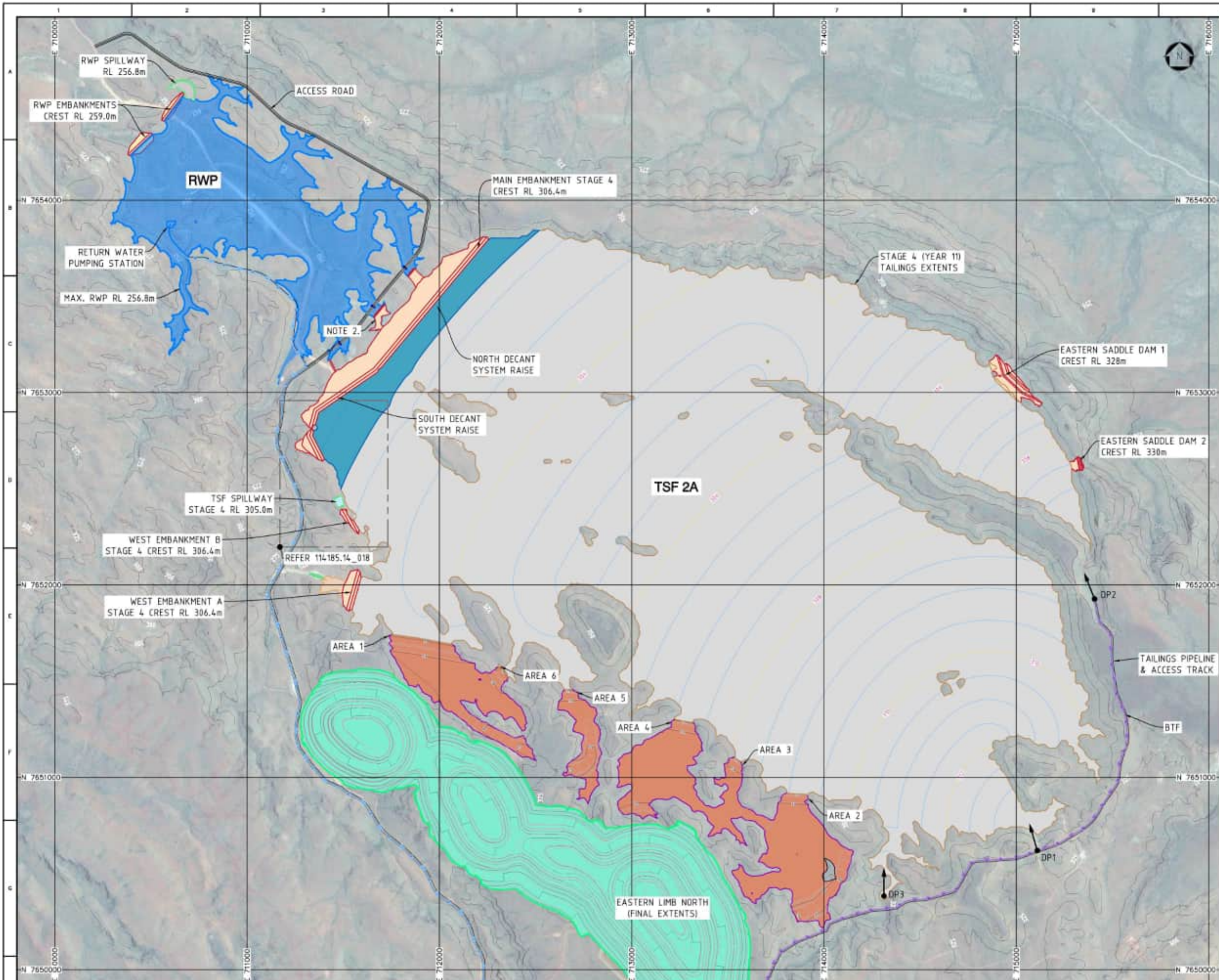
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**NORTH STAR MAGNETITE STAGE 2  
 TSF2A & RWP  
 STAGE 3 END OF YEAR 8 TAILINGS DEPOSITION**

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**A3** SCALE DRG No: 1:20000 REV:





**STAGE 4 (YEARS 9-11)**  
 TAILINGS VOLUME = 50.0 Mm<sup>3</sup> (INC.) 180.4Mm<sup>3</sup> (TOTAL)  
 TSF EMBANKMENT CREST RL = 306.4m  
 TSF EMBANKMENT VOLUME = 1,48Mm<sup>3</sup>

- NOTES**
- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50,
  - DOWNSTREAM BENCH TO RWP MAX. WATER LEVEL (RL 256.8m)

- LEGEND**
- DP# TAILINGS DISCHARGE POINT
  - TAILINGS SLURRY PIPELINE ON ACCESS TRACK
  - RETURN WATER PIPELINE ON ACCESS TRACK
  - NORMAL OPERATING POND (MIN. 400mm DEPTH)
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

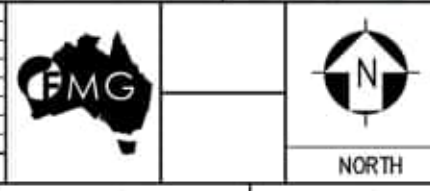
**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

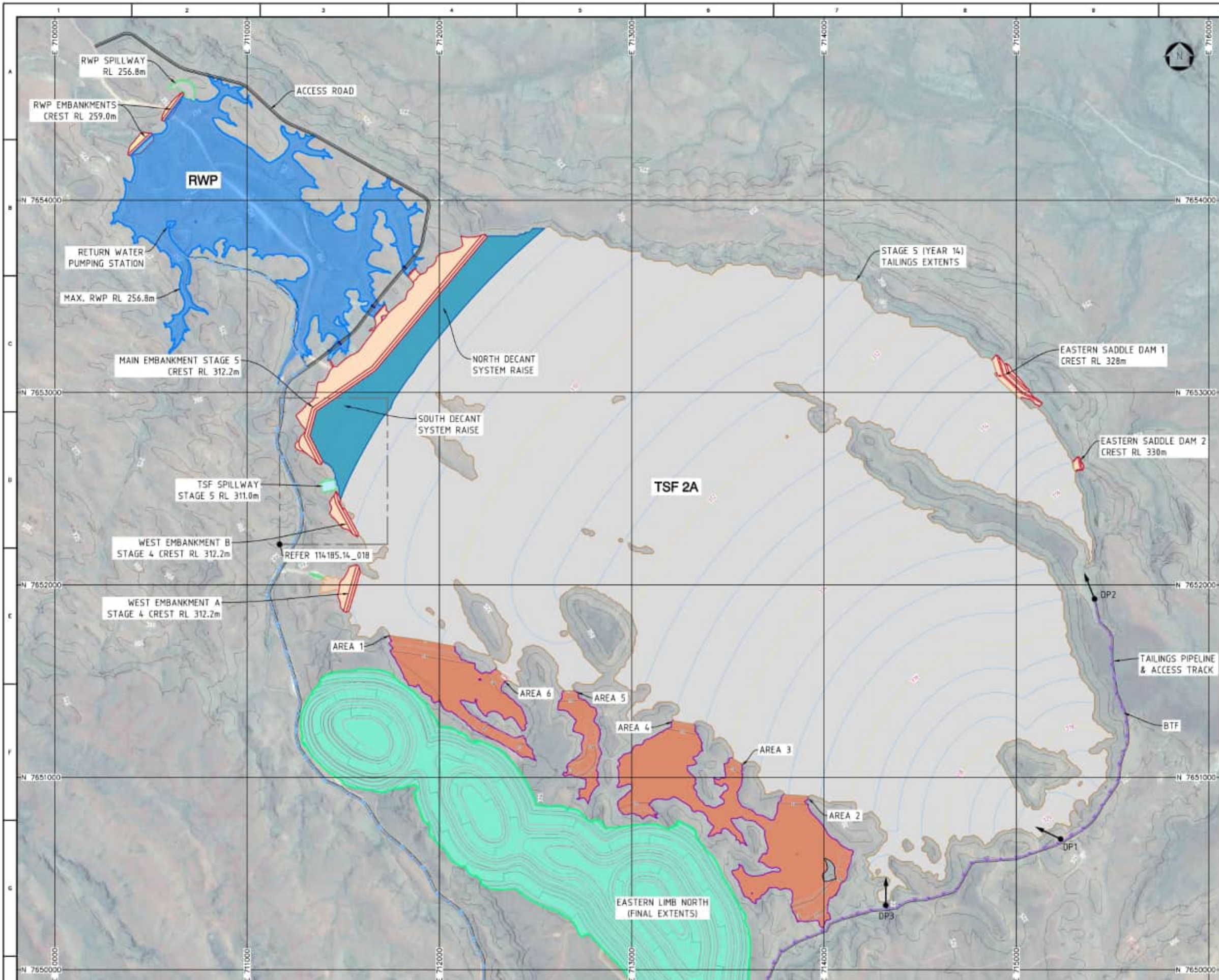
AREA	MINIMUM RL.
1	306.3m
2	312.3m
3	311.2m
4	310.2m
5	308.5m
6	307.0m

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REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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		B	18.02.2019								
		C	26.03.2020								

VENDOR/DESIGNER DRG No. 114185.14\_008  
 Fortescue Metals Group Ltd  
 PROJECT NUMBER 114185.14  
 NORTH STAR MAGNETITE STAGE 2  
 TSF2A & RWP  
 STAGE 4 END OF YEAR 11 TAILINGS DEPOSITION  
 SCALE 1:20000  
 DRG No. A3  
 REV: C





**STAGE 5 (YEARS 12-15)**  
 TAILINGS VOLUME = 49,6 Mm<sup>3</sup> (INC.) 230.0Mm<sup>3</sup> (TOTAL)  
 TSF EMBANKMENT CREST RL = 312.2m  
 TSF EMBANKMENT VOLUME = 1,71Mm<sup>3</sup>

**NOTES**  
 1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.

**LEGEND**

- DP# TAILINGS DISCHARGE POINT
- TAILINGS SLURRY PIPELINE ON ACCESS TRACK
- RETURN WATER PIPELINE ON ACCESS TRACK
- NORMAL OPERATING POND (MIN. 400mm DEPTH)
- EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

AREA	MINIMUM RL.
1	312.3m
2	318.3m
3	317.2m
4	316.3m
5	314.5m
6	314.3m

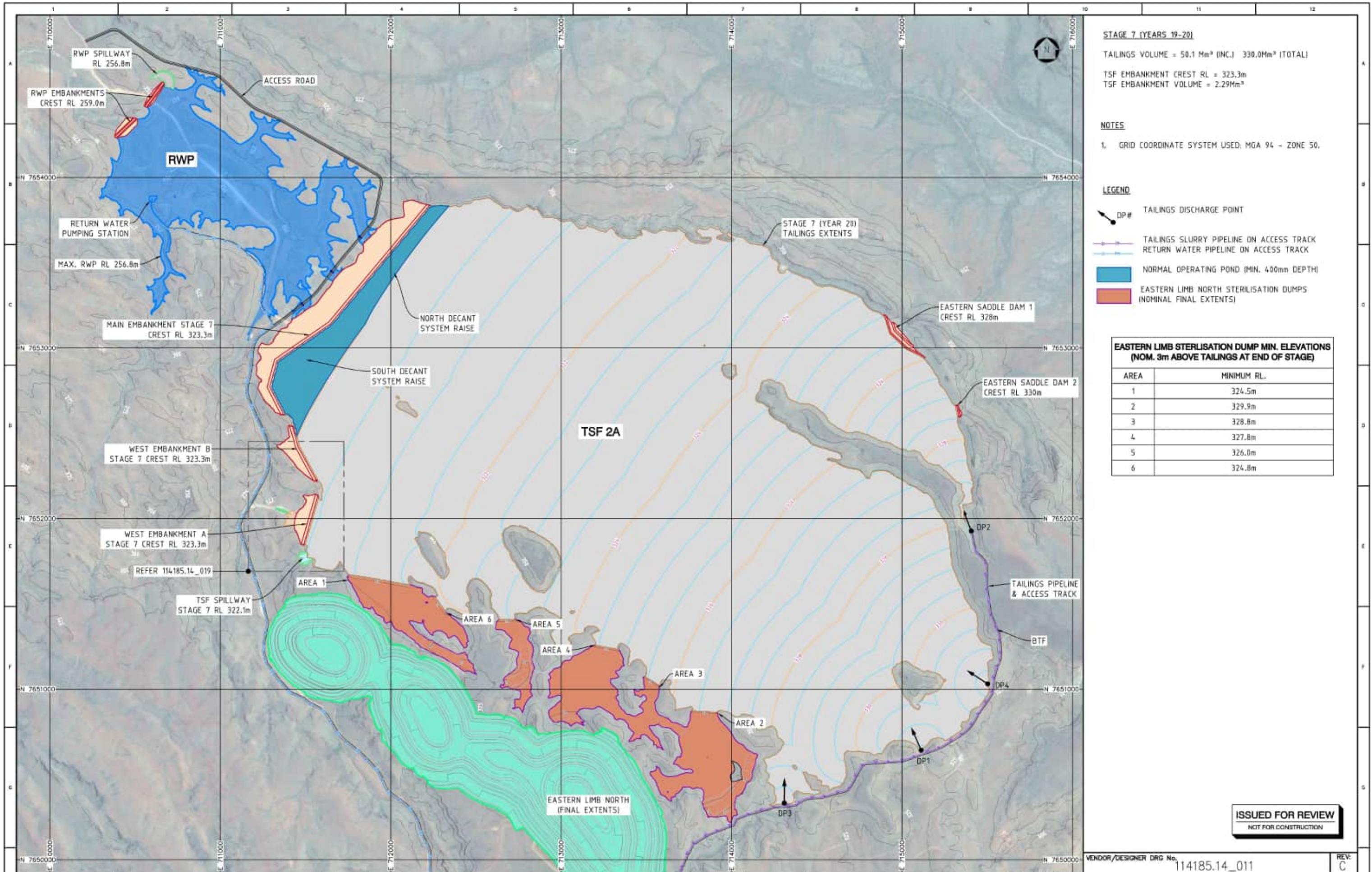
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			A	26.10.2018								

VENDOR/DESIGNER DRG No. 114185.14\_009  
 PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF2A & RWP  
 STAGE 5 END OF YEAR 15 TAILINGS DEPOSITION  
 SCALE 1:20000  
 DRG No. A3  
 REV: C







**STAGE 7 (YEARS 19-20)**  
 TAILINGS VOLUME = 50.1 Mm<sup>3</sup> (INC.) 330.0Mm<sup>3</sup> (TOTAL)  
 TSF EMBANKMENT CREST RL = 323.3m  
 TSF EMBANKMENT VOLUME = 2.29Mm<sup>3</sup>

**NOTES**  
 1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.

**LEGEND**

- DP# TAILINGS DISCHARGE POINT
- TAILINGS SLURRY PIPELINE ON ACCESS TRACK
- RETURN WATER PIPELINE ON ACCESS TRACK
- NORMAL OPERATING POND (MIN. 400mm DEPTH)
- EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**EASTERN LIMB STERILISATION DUMP MIN. ELEVATIONS (NOM. 3m ABOVE TAILINGS AT END OF STAGE)**

AREA	MINIMUM RL.
1	324.5m
2	329.9m
3	328.8m
4	327.8m
5	326.0m
6	324.8m

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REF	DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
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PROJECT NUMBER 114185.14

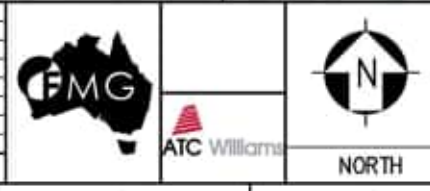
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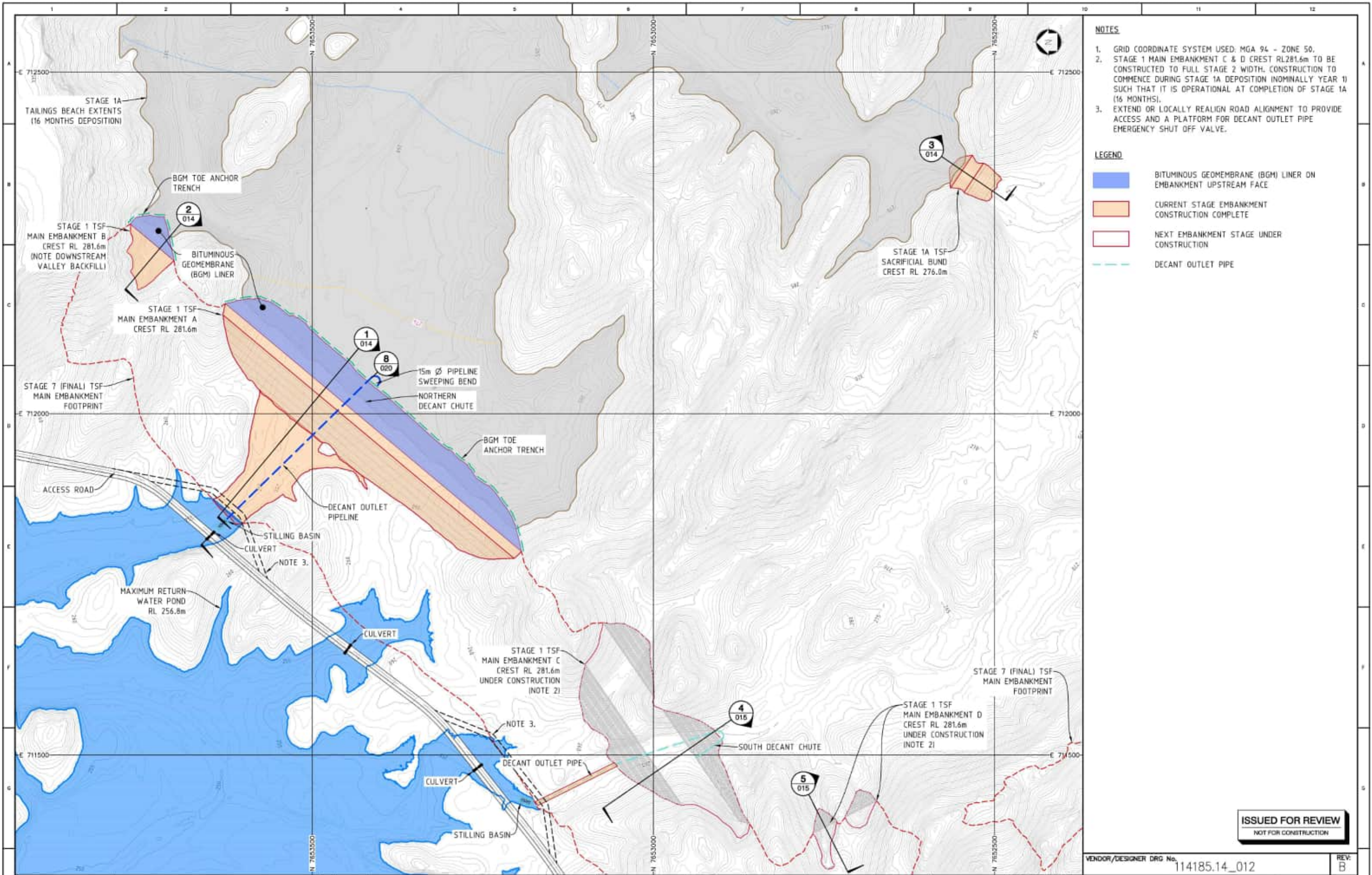
NORTH STAR MAGNETITE STAGE 2  
 TSF2A & RWP  
 STAGE 7 END OF YEAR 20 TAILINGS DEPOSITION

SCALE 1:20000

DRG No. A3

REV: C





- NOTES**
- GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.
  - STAGE 1 MAIN EMBANKMENT C & D CREST RL 281.6m TO BE CONSTRUCTED TO FULL STAGE 2 WIDTH. CONSTRUCTION TO COMMENCE DURING STAGE 1A DEPOSITION (NOMINALLY YEAR 1) SUCH THAT IT IS OPERATIONAL AT COMPLETION OF STAGE 1A (16 MONTHS).
  - EXTEND OR LOCALLY REALIGN ROAD ALIGNMENT TO PROVIDE ACCESS AND A PLATFORM FOR DECANT OUTLET PIPE EMERGENCY SHUT OFF VALVE.

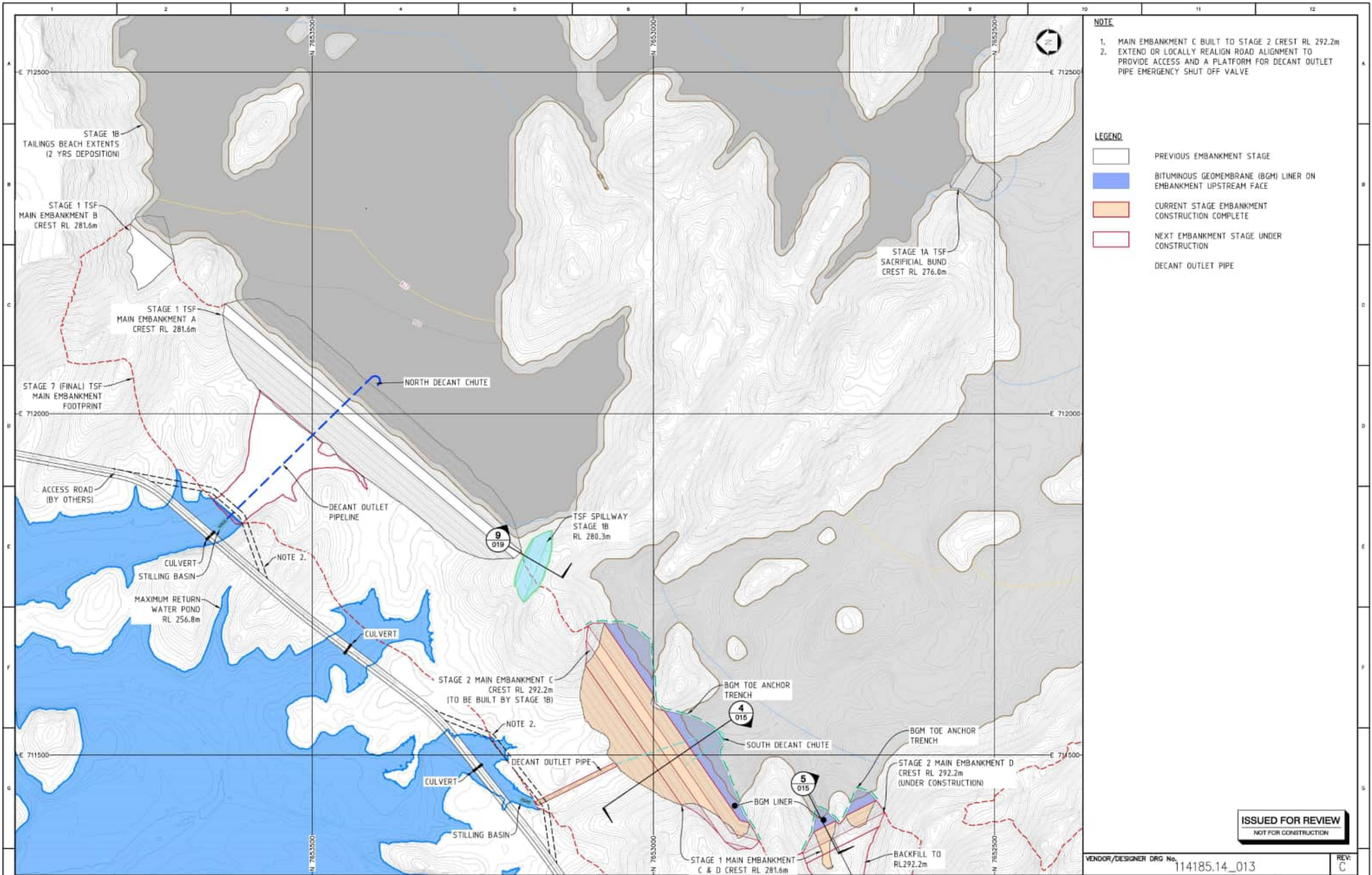
- LEGEND**
- █ BITUMINOUS GEOMEMBRANE (BGM) LINER ON EMBANKMENT UPSTREAM FACE
  - █ CURRENT STAGE EMBANKMENT CONSTRUCTION COMPLETE
  - █ NEXT EMBANKMENT STAGE UNDER CONSTRUCTION
  - DECANT OUTLET PIPE

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_012 REV: B

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		B	21.04.2020								
		A	26.10.2019								

PROJECT NUMBER 114185.14  
**Fortescue Metals Group Ltd**  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 TSF STAGE 1A - GENERAL ARRANGEMENT  
 SCALE 1:5000 DRG No. A3  
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- NOTE**
1. MAIN EMBANKMENT C BUILT TO STAGE 2 CREST RL 292.2m
  2. EXTEND OR LOCALLY REALIGN ROAD ALIGNMENT TO PROVIDE ACCESS AND A PLATFORM FOR DECANT OUTLET PIPE EMERGENCY SHUT OFF VALVE

- LEGEND**
- PREVIOUS EMBANKMENT STAGE
  - BITUMINOUS GEOMEMBRANE (BGM) LINER ON EMBANKMENT UPSTREAM FACE
  - CURRENT STAGE EMBANKMENT CONSTRUCTION COMPLETE
  - NEXT EMBANKMENT STAGE UNDER CONSTRUCTION
  - DECANT OUTLET PIPE

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_013 REV: C

REF	DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
			C	21.04.2020								
			B	26.10.2018								
			A	26.10.2018								

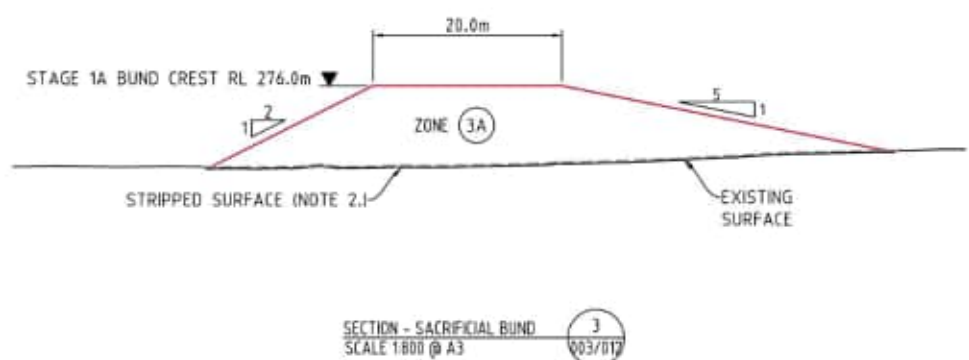
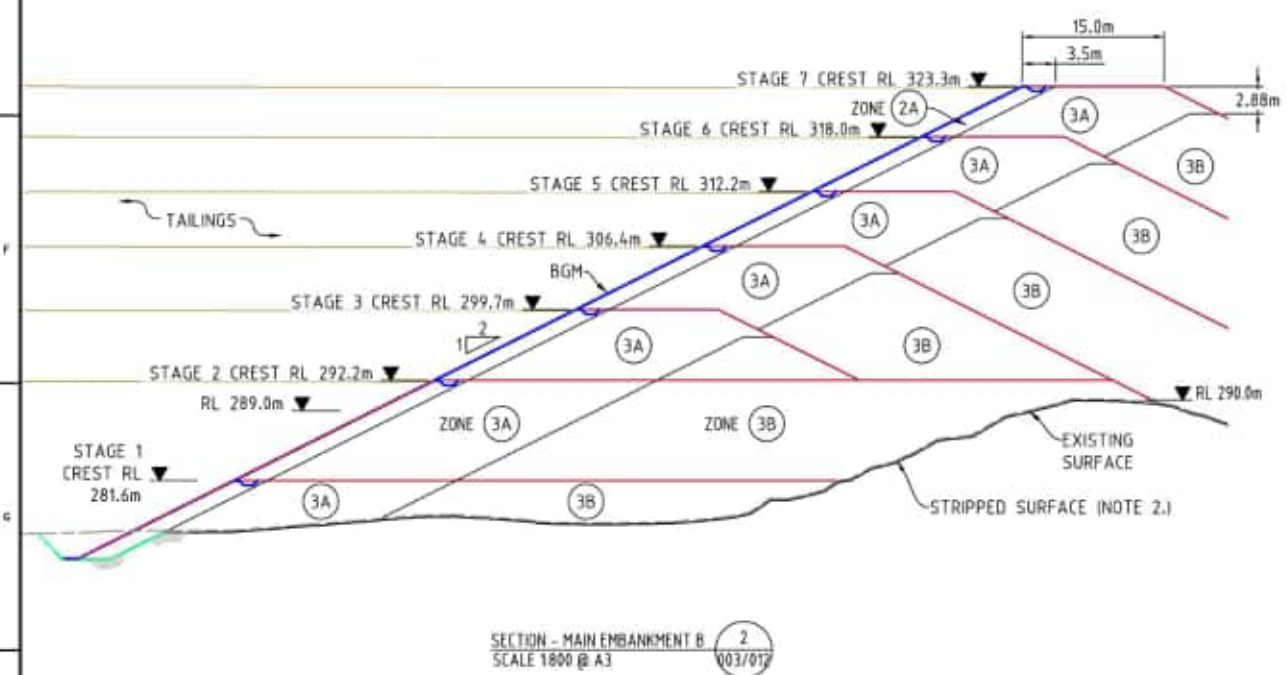
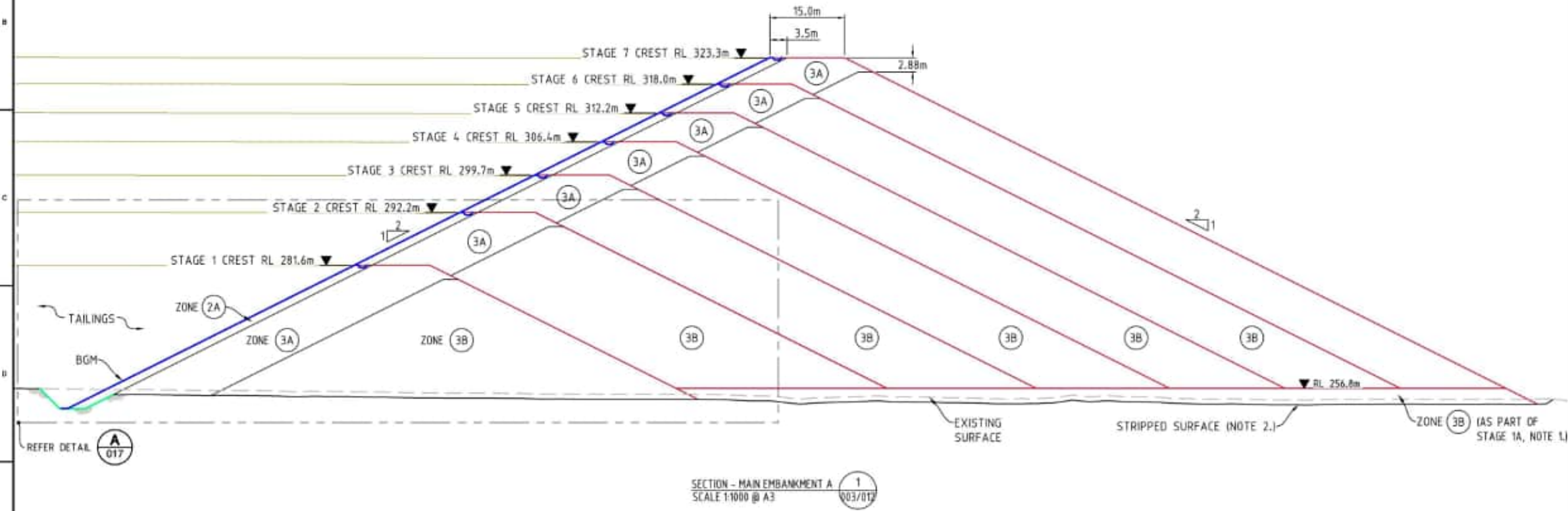


PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 TSF STAGE 1B - GENERAL ARRANGEMENT  
 SCALE 1:5000 DRG No. A3  
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ZONE	MATERIAL DESCRIPTION	PLACEMENT	COMPACTION
BGM	BITUMINOUS GEOMEMBRANE LINER	PLACE ON PREPARED SURFACE AND ANCHORED AT PERIMETER	N.A.
2A	BGM SUPPORT LAYER - 30 mm MINUS COARSE REJECTS	HORIZONTAL LAYERS - LOOSE THICKNESS 400mm	6 PASSES OF MINIMUM 10 TONNE STATIC MASS VIBRATING ROLLER
3A	TRANSITION ROCKFILL - COARSE REJECTS/APPROVED MINE SPOIL/DESIGNATED QUARRIES & EXCAVATIONS	HORIZONTAL LAYERS - LOOSE THICKNESS 400mm	6 PASSES OF MINIMUM 10 TONNE STATIC MASS VIBRATING ROLLER
3B	GENERAL ROCKFILL - APPROVED MINE SPOIL/DESIGNATED QUARRIES & EXCAVATIONS	HORIZONTAL LAYERS - LOOSE THICKNESS 800mm OR HORIZONTAL LAYERS - LOOSE THICKNESS 1600mm	6 PASSES OF MINIMUM 10 TONNE STATIC MASS VIBRATING ROLLER OR 10 PASSES OF 4 SIDED 8 TONNE IMPACT ROLLER

**LEGEND**  
 ( # ) MATERIAL ZONE NUMBER (REFER TO 114185.14\_014 FOR DESCRIPTIONS).

- NOTES:**
- EMBANKMENT DOWNSTREAM BENCH TO BE CONSTRUCTED TO FULL RWP MAX. WATER LEVEL (RL 256.8m) AT FULL WIDTH OF STAGE 7 EMBANKMENT AS PART OF STAGE 1A WORKS TO PROTECT RAISE FOUNDATIONS FROM RWP INUNDATION.
  - EMBANKMENT FOUNDATION STRIPPING TO BE NOMINALLY:
    - 500mm (ALLUVIAL AREAS)
    - 200mm (OTHER AREAS)



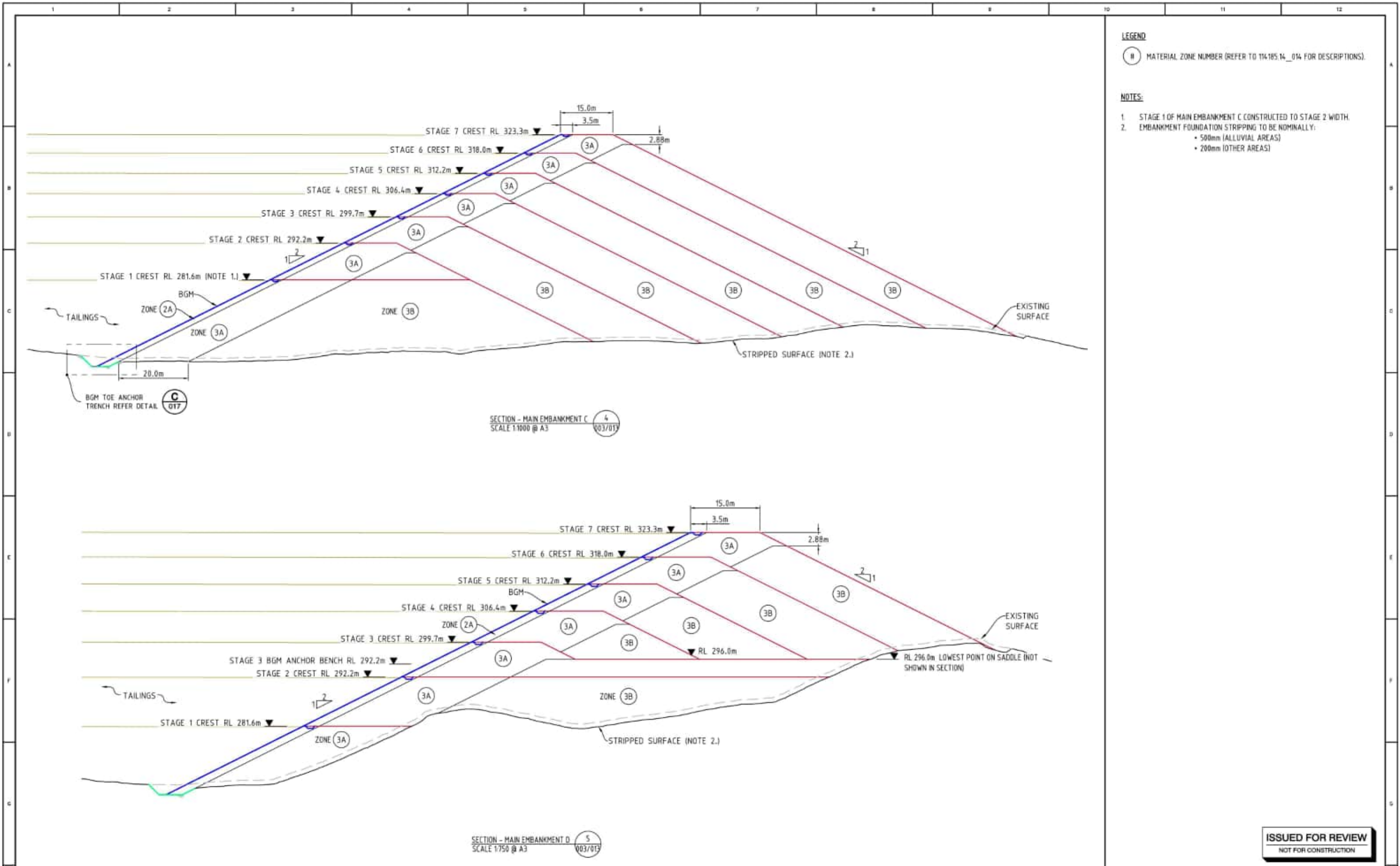
**ISSUED FOR REVIEW**  
 NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_014 REV: B

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		B	21.04.2020								
		A	26.10.2019								



PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 1  
 TSF 2A & RWP  
 TSF EMBANKMENTS  
 SECTIONS & DETAILS (SHEET 1 OF 4)  
 SCALE AS SHOWN  
 DRG No. A3  
 REV: B



- LEGEND**
- ① MATERIAL ZONE NUMBER (REFER TO 114185.14\_014 FOR DESCRIPTIONS).
- NOTES:**
- STAGE 1 OF MAIN EMBANKMENT C CONSTRUCTED TO STAGE 2 WIDTH.
  - EMBANKMENT FOUNDATION STRIPPING TO BE NOMINALLY:
    - 500mm (ALLUVIAL AREAS)
    - 200mm (OTHER AREAS)

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_015 REV: B

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		B	21.04.2020								
		A	26.10.2019								

PROJECT NUMBER 114185.14

Fortescue Metals Group Ltd

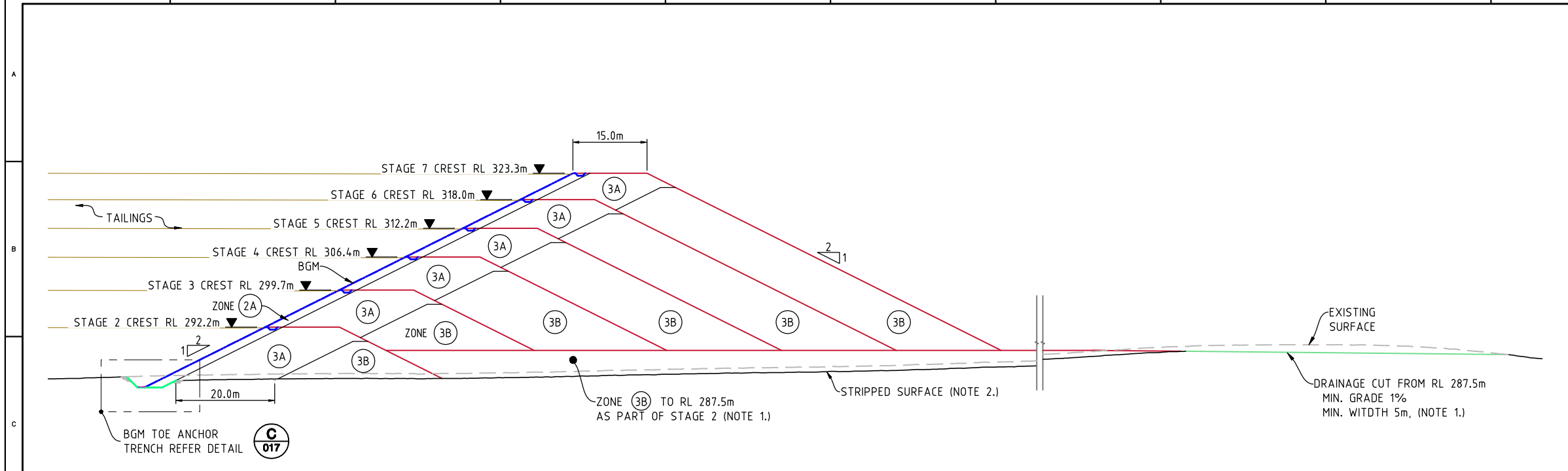
NORTH STAR MAGNETITE STAGE 2  
TSF 2A & RWP  
TSF EMBANKMENTS  
SECTIONS & DETAILS (SHEET 2 OF 4)

SCALE AS SHOWN DRG No. A3

REV: B

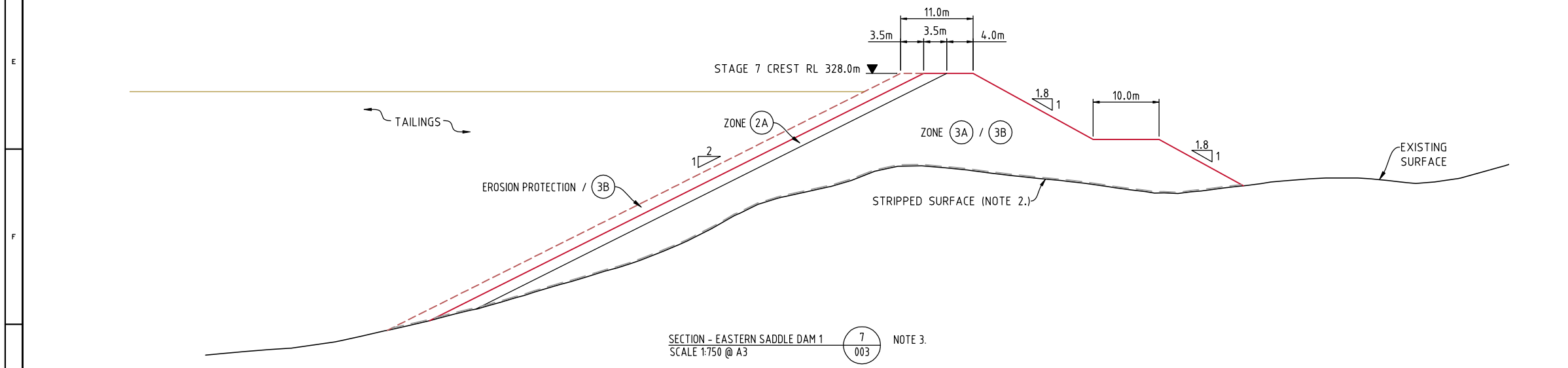
ATC Williams

NORTH



SECTION - WEST EMBANKMENT A 6  
SCALE 1:1000 @ A3 003

- LEGEND**
- # MATERIAL ZONE NUMBER (REFER TO 114185.14\_014 FOR DESCRIPTIONS).
- NOTES:**
- THE LOW LYING AREA DOWNSTREAM OF THE STAGE 2 WEST EMBANKMENT A SHALL BE BACKFILLED TO RL 287.5m, WITH AN ACCOMPANYING DRAINAGE CUT, TO AVOID PONDED WATER.
  - EMBANKMENT FOUNDATION STRIPPING TO BE NOMINALLY:
    - 500mm (ALLUVIAL AREAS)
    - 200mm (OTHER AREAS)
  - EASTERN SADDLE DAM 1 DETAILS ARE TYPICAL FOR EASTERN SADDLE DAM 2.



SECTION - EASTERN SADDLE DAM 1 7 NOTE 3.  
SCALE 1:750 @ A3 003

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_016 REV: A

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2018								

NORTH

**PROJECT NUMBER**  
114185.14

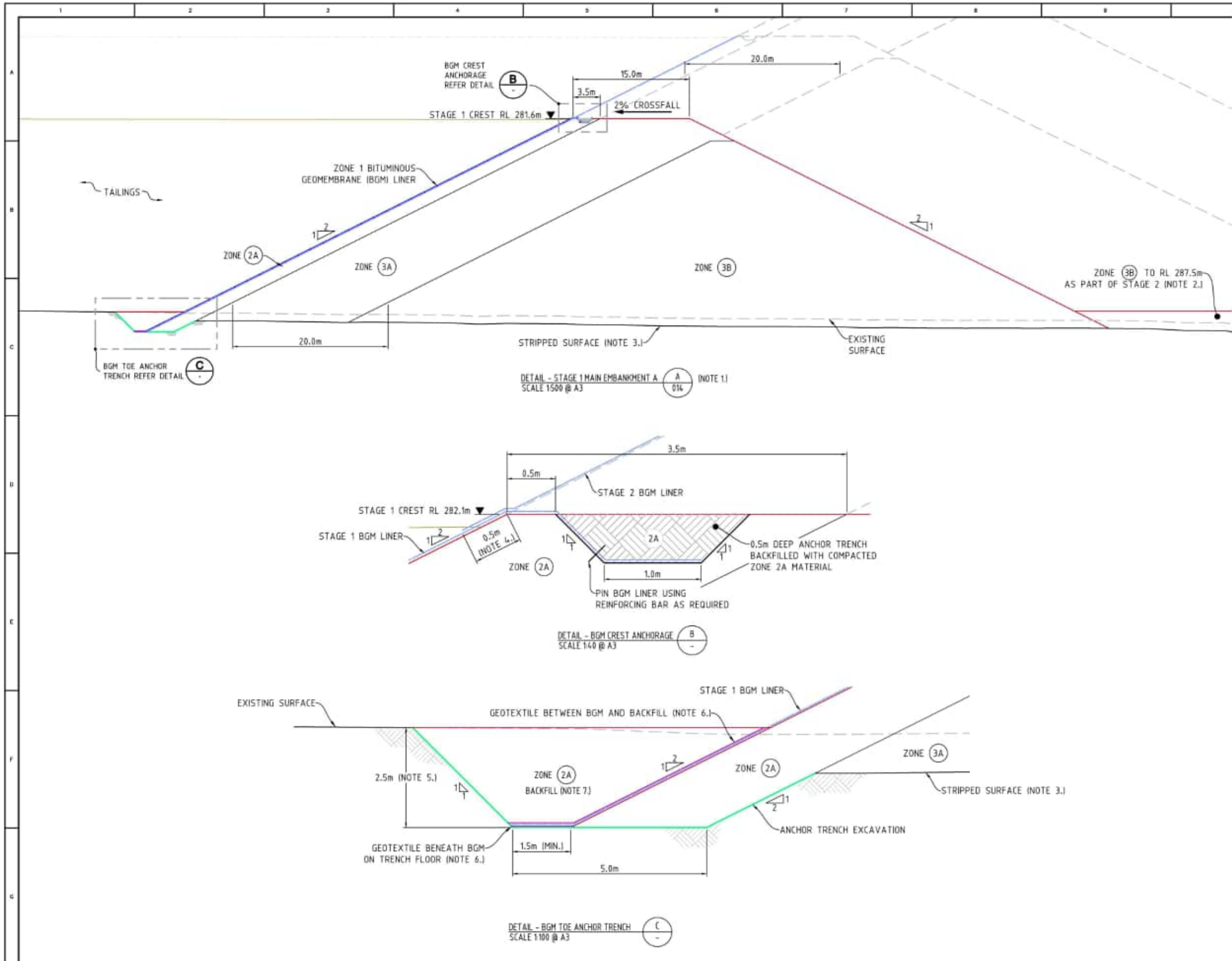
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**SCALE** DRG No: A3 AS SHOWN

**Fortescue Metals Group Ltd**

NORTH STAR MAGNETITE STAGE 2  
TSF 2A & RWP  
TSF EMBANKMENTS  
SECTIONS & DETAILS (SHEET 3 OF 4)

REV: A



**LEGEND**

⊙ MATERIAL ZONE NUMBER

**NOTES:**

1. STAGE 1 MAIN EMBANKMENT A, DETAILS ARE TYPICAL FOR STAGE 1 MAIN EMBANKMENT C.
2. EMBANKMENT DOWNSTREAM BENCH TO BE CONSTRUCTED TO FULL RWP MAX. WATER LEVEL (RL 256.0m) AT FULL WIDTH OF STAGE 7 EMBANKMENT AS PART OF STAGE 1A WORKS TO PROTECT RAISE FOUNDATIONS FROM RWP INUNDATION.
3. EMBANKMENT FOUNDATION STRIPPING TO BE NOMINALLY:
  - 500mm (ALLUVIAL AREAS)
  - 200mm (OTHER AREAS)
4. NEXT STAGE BGM LINER WELDED HORIZONTAL SEAM 0.5m OVERLAP.
5. MINIMUM DEPTH OF ANCHOR TRENCH TO BE 2.5m OR TO REFUSAL OF EXCAVATION EQUIPMENT.
6. GEOTEXTILE SHALL BE NON-WOVEN POLYPROPYLENE WITH MINIMUM MASS 500g/m<sup>2</sup>.
7. SUITABLE TRENCH BACKFILL ALTERNATIVES INCLUDE FOUNDATION SPOIL WITH MIN. PARTICLE SIZE 50mm.

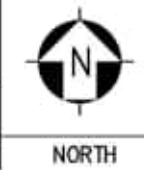
**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

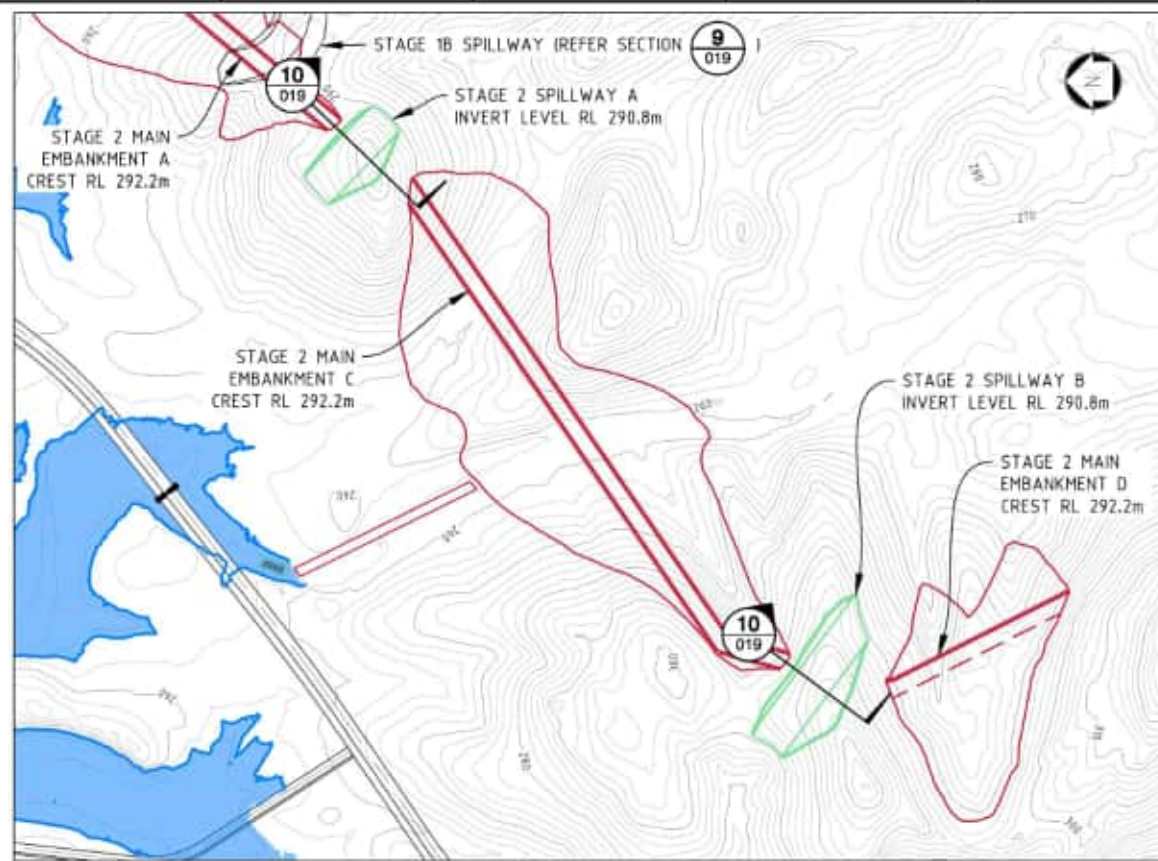
VENDOR/DESIGNER DRG No. 114185.14\_017 REV: A

PROJECT NUMBER 114185.14  
Fortescue Metals Group Ltd  
NORTH STAR MAGNETITE STAGE 2  
TSF 2A & RWP  
TSF LIFE OF MINE EMBANKMENTS  
SECTIONS & DETAILS (SHEET 4 OF 4)

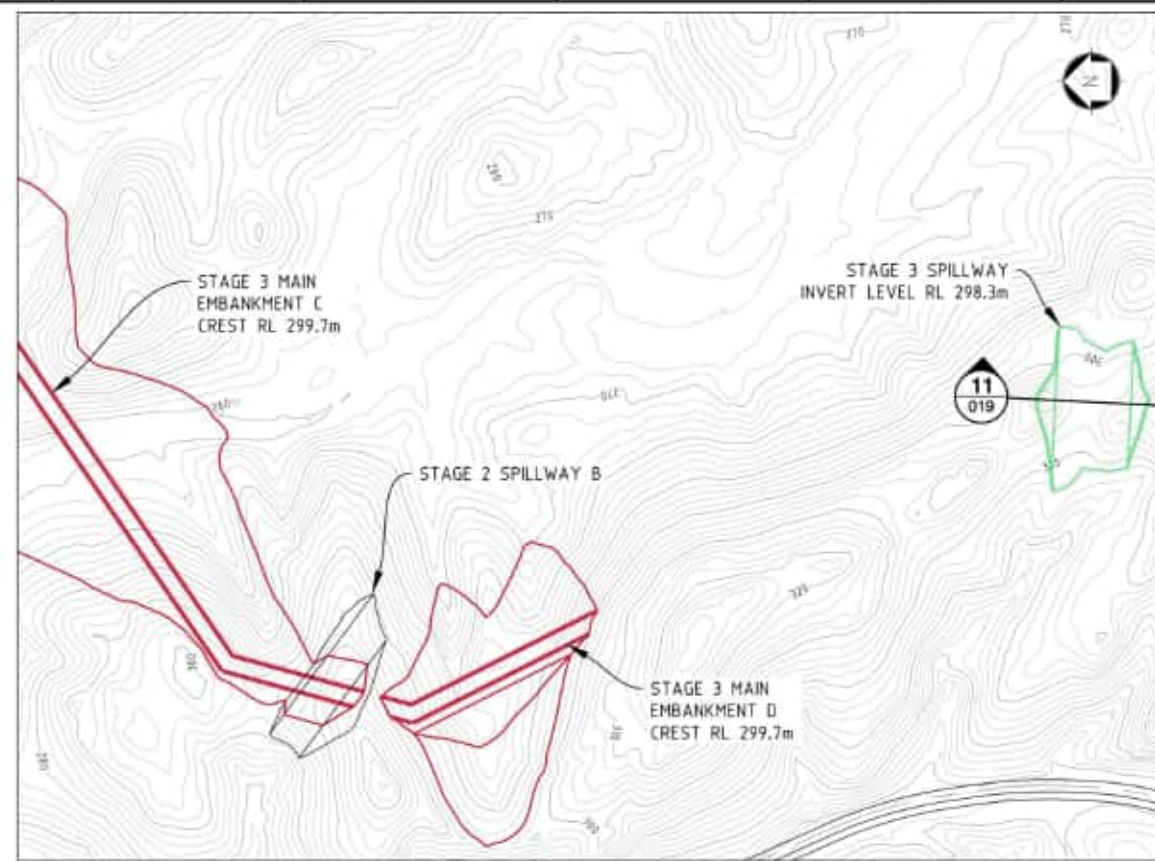
SCALE AS SHOWN DRG No. REV:

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2019								

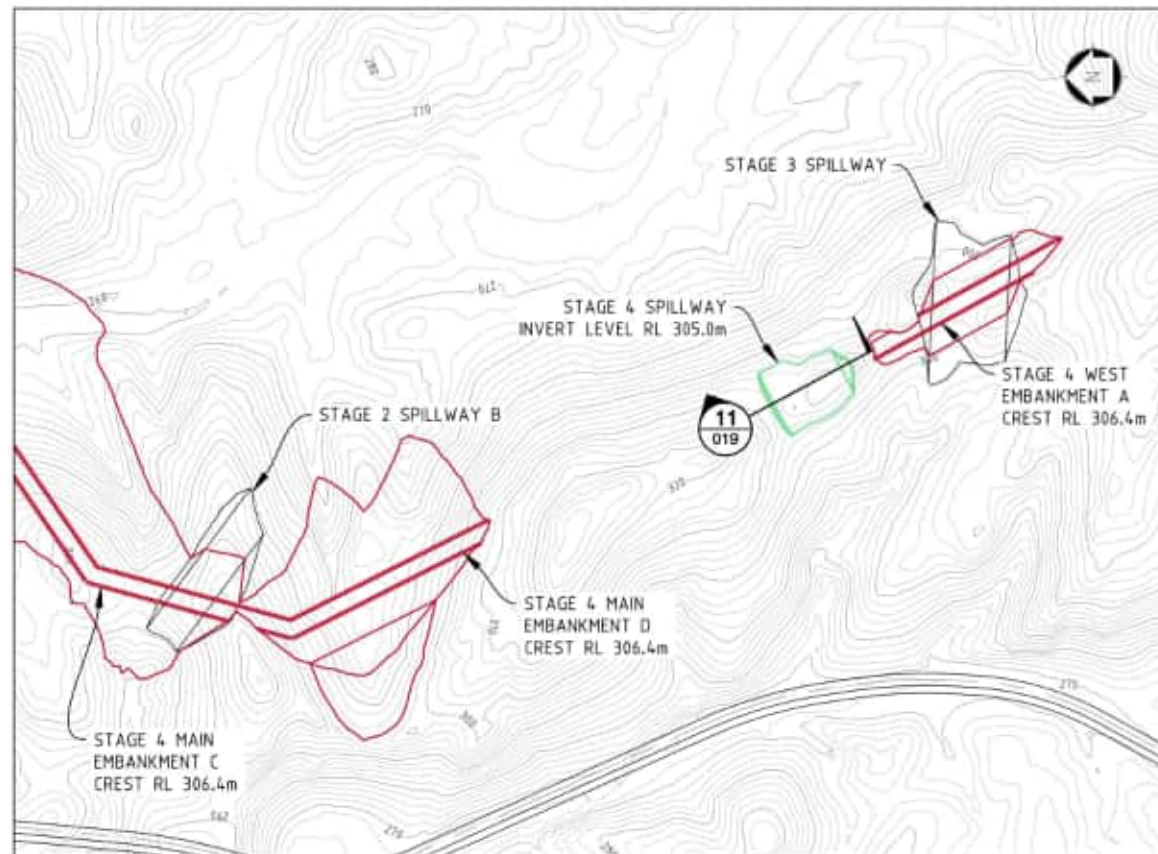




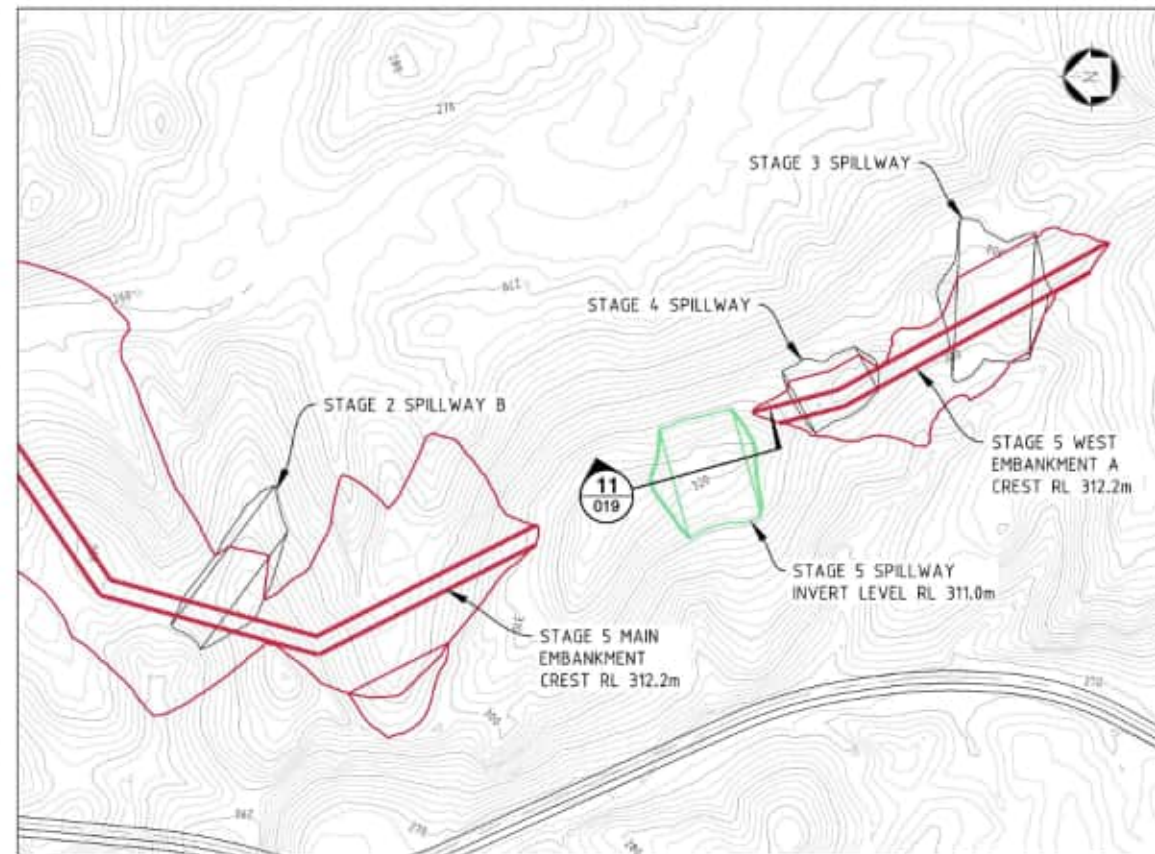
SPILLWAY LAYOUT STAGE 2  
SCALE 1:5000 @ A3



SPILLWAY LAYOUT STAGE 3  
SCALE 1:5000 @ A3



SPILLWAY LAYOUT STAGE 4  
SCALE 1:5000 @ A3



SPILLWAY LAYOUT STAGE 5  
SCALE 1:5000 @ A3

**NOTES**

- LAYOUTS ARE PROVIDED TO DEMONSTRATE STAGING OF SPILLWAY EXCAVATIONS AND SUBSEQUENT BACKFILLING. INDIVIDUAL EMBANKMENT STAGES HAVE BEEN OMITTED FOR CLARITY.
- SPILLWAY EXCAVATION QUANTITIES:  
 STAGE 1B = 14,250m<sup>3</sup>  
 STAGE 2 = 36,320m<sup>3</sup>  
 STAGE 3 = 19,800m<sup>3</sup>  
 STAGE 4 = 6,600m<sup>3</sup>  
 STAGE 5 = 27,100m<sup>3</sup>  
 STAGE 6 = 20,900m<sup>3</sup>  
 STAGE 7 = 17,800m<sup>3</sup>

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_018 REV: A

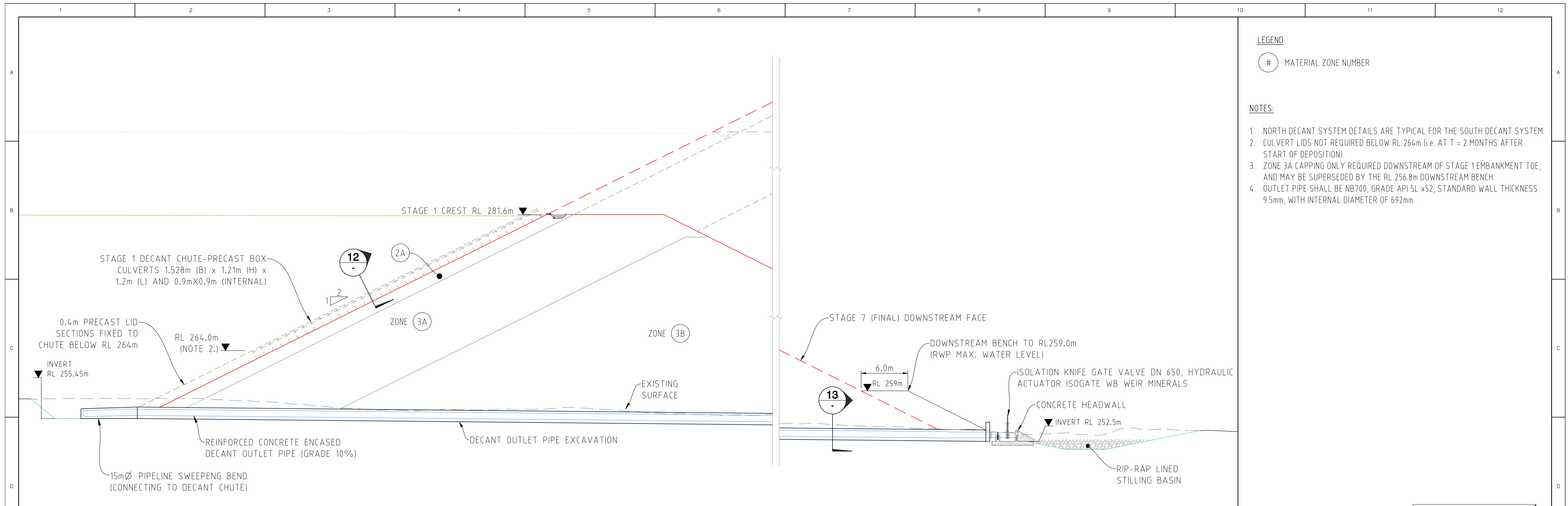
PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 TSF SPILLWAYS LAYOUT & SECTIONS  
 SHEET 1 OF 2

SCALE 1:5000 DRG No. A3 REV:

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2018								





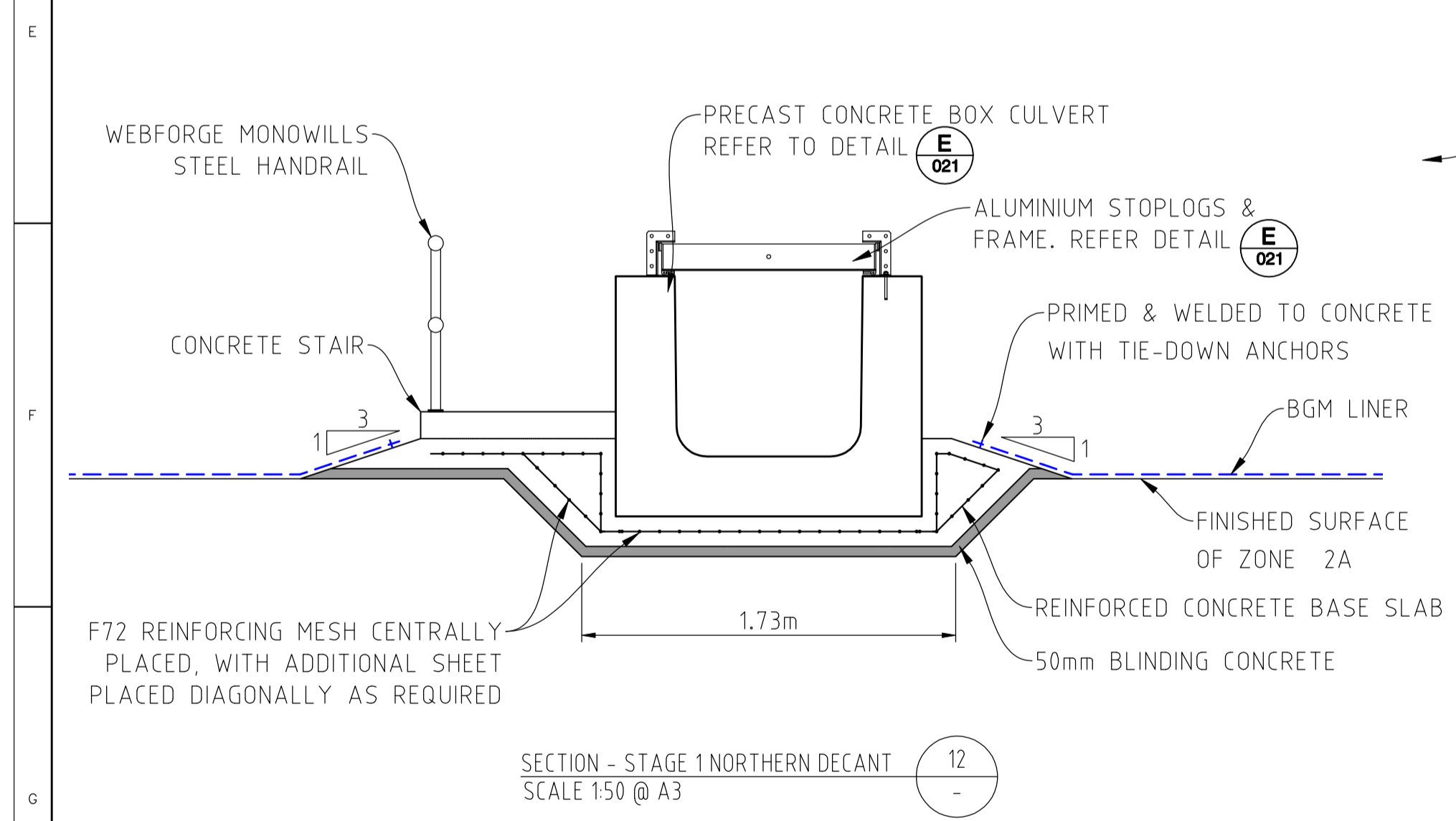


**LEGEND**  
 # MATERIAL ZONE NUMBER

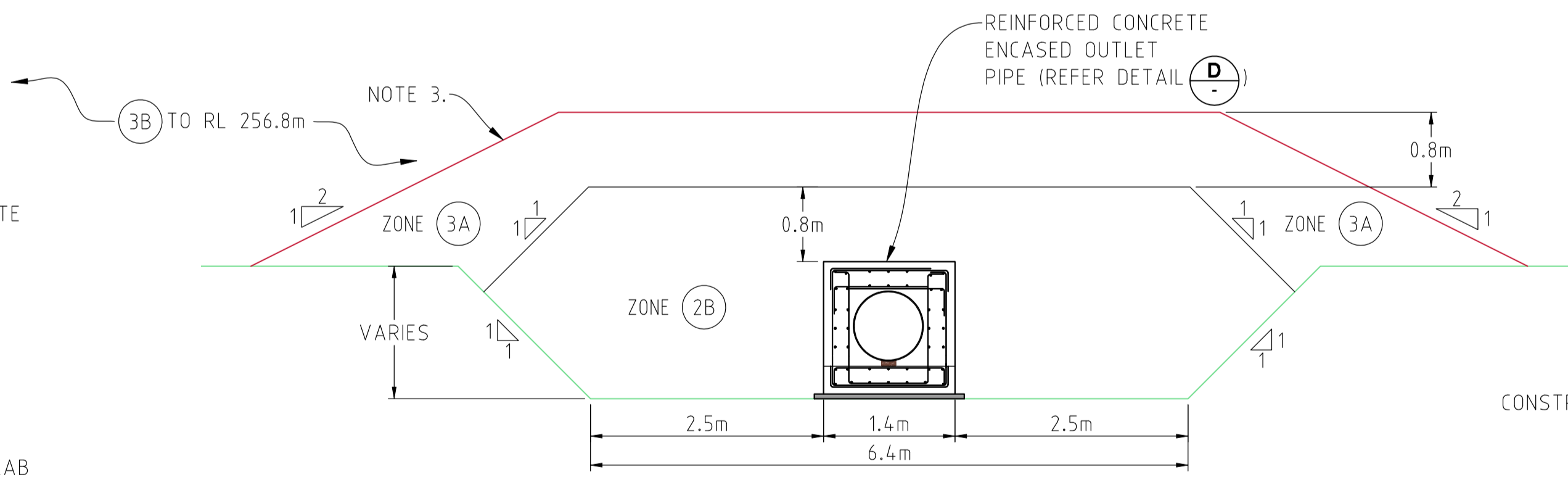
- NOTES:**
- NORTH DECANT SYSTEM DETAILS ARE TYPICAL FOR THE SOUTH DECANT SYSTEM.
  - CULVERT LIDS NOT REQUIRED BELOW RL 264m (i.e. AT T = 2 MONTHS AFTER START OF DEPOSITION).
  - ZONE 3A CAPPING ONLY REQUIRED DOWNSTREAM OF STAGE 1 EMBANKMENT TOE, AND MAY BE SUPERSEDED BY THE RL 256.8m DOWNSTREAM BENCH.
  - OUTLET PIPE SHALL BE NB700, GRADE API 5L x52, STANDARD WALL THICKNESS 9.5mm, WITH INTERNAL DIAMETER OF 692mm.

SECTION - STAGE 1 NORTH DECANT SYSTEM (NOTE 1) 8/012  
 SCALE 1:500 @ A3

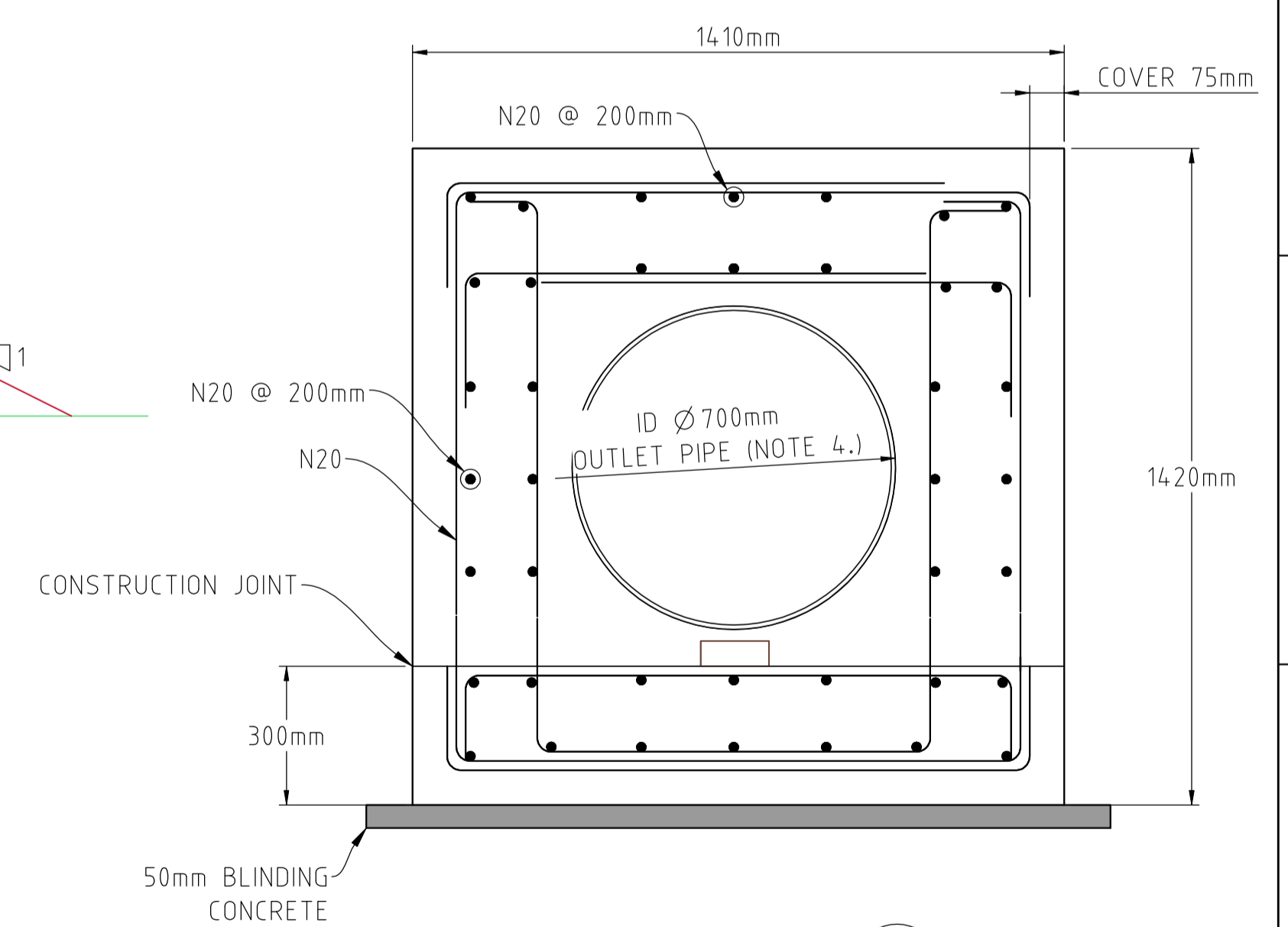
**ISSUED FOR REVIEW**  
 NOT FOR CONSTRUCTION



SECTION - STAGE 1 NORTHERN DECANT 12/-  
 SCALE 1:50 @ A3



SECTION - STAGE 1 NORTH DECANT OUTLET PIPE TRENCH 13/-  
 SCALE 1:100 @ A3



DETAIL - DECANT OUTLET PIPE D/-  
 SCALE 1:25 @ A3

REF DRG No.										TITLE										REV: A										DATE: 17.08.18										DESCRIPTION										DRN										CHK										SUP										DES										DAP										PAP										FMG									
1										2										3										4										5										6										7										8										9										10										11										12									

NORTH

VENDOR/DESIGNER DRG No. 114185.14\_020

PROJECT NUMBER 114185.14

Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2

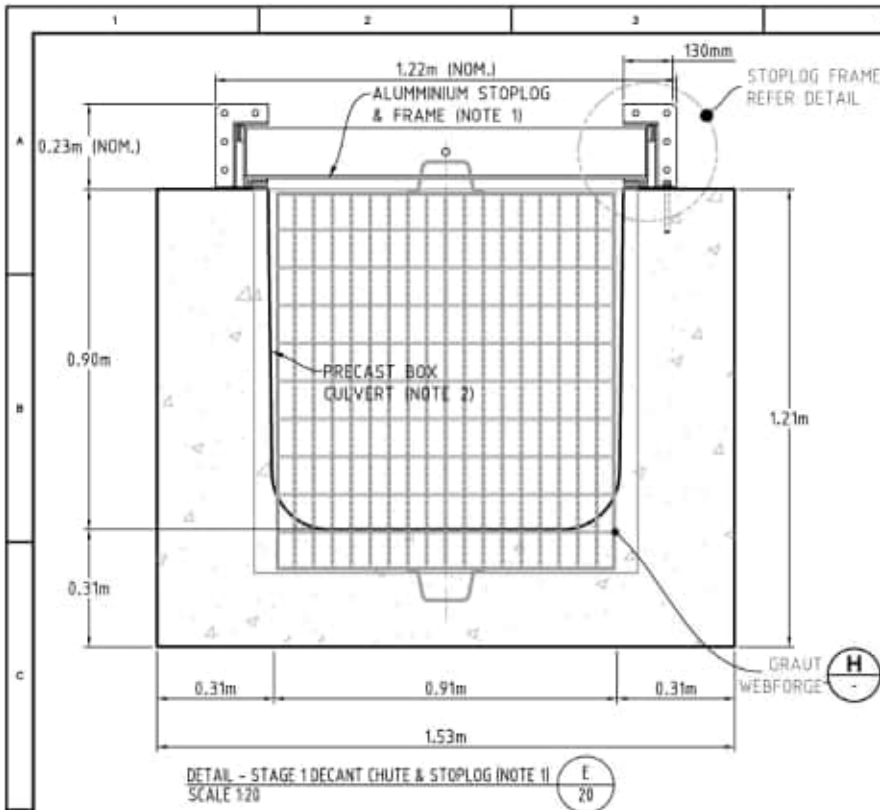
TSF 2A & RWP

TSF DECANTS

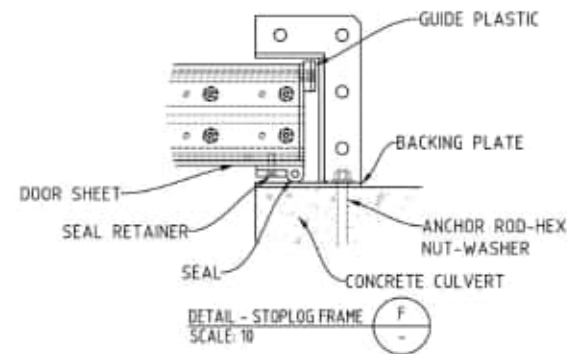
SECTIONS & DETAILS (SHEET 1 OF 2)

SCALE DRG No: A3

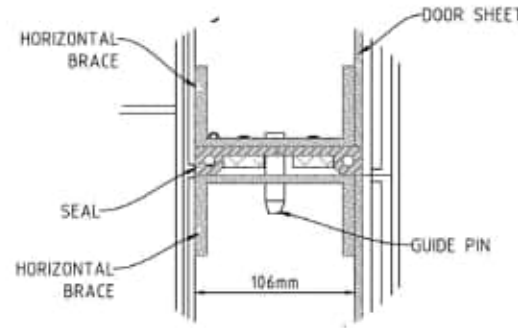
REV: A



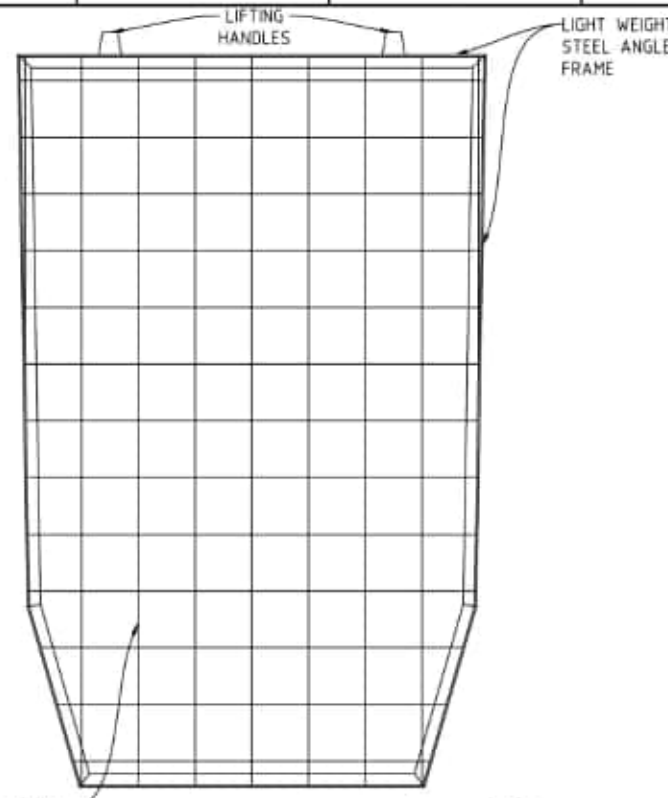
DETAIL - STAGE 1 DECANT CHUTE & STOPLOG (NOTE 1)  
SCALE 1:20



DETAIL - STOPLOG FRAME  
SCALE 1:10



DETAIL - STOPLOG SEAL  
SCALE 1:5 @ A3

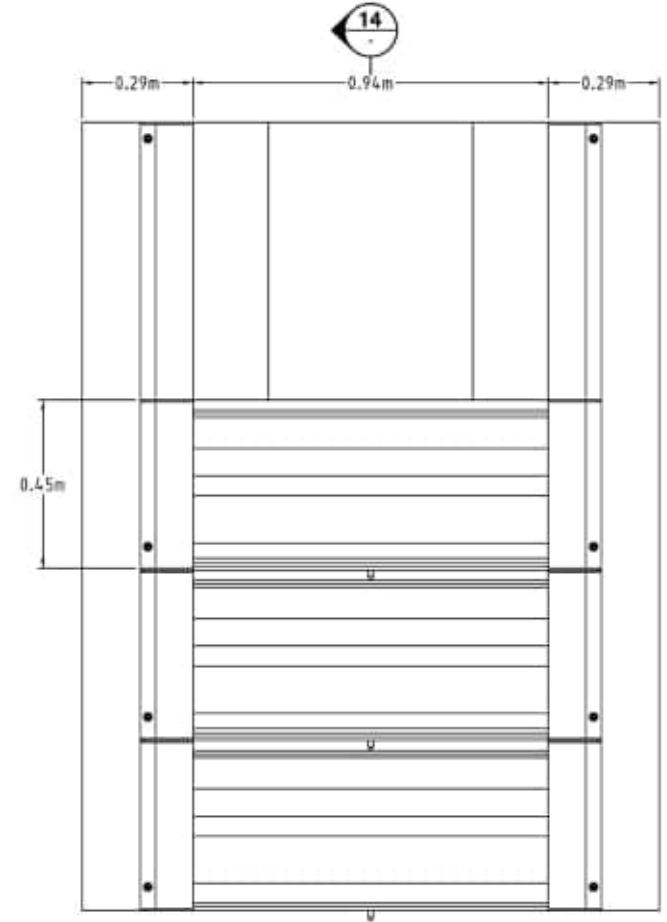
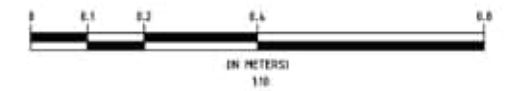


DETAIL - REMOVABLE TRASH RACK  
SCALE 1:20 @ A3

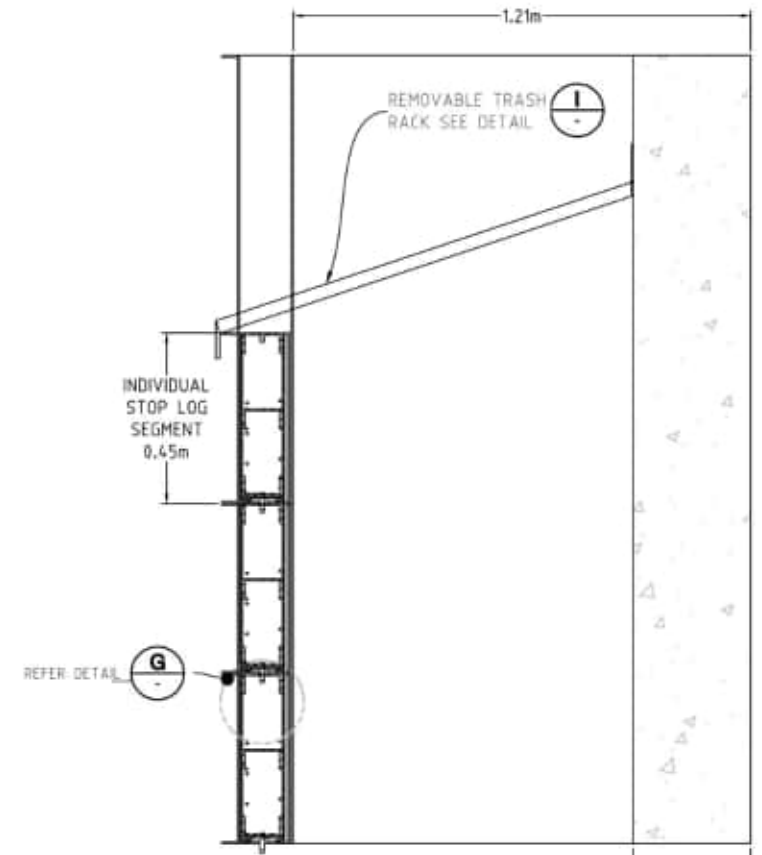
LIGHT WEIGHT STEEL MESH WELDED TO RIGID STEEL FRAME (MINIMUM DIMENSIONS OF MESH 150x150mm)

- NOTES:**
- ALL ALUMINIUM STOPLOG DIMENSIONS AND SPECIFICATIONS ARE SUBJECTED TO DETAILED DESIGN BY AWMA PTY. LTD. (WWW.AWMAWATERCONTROL.COM.AU)
  - PRECAST BOX CULVERT DIMENSIONS RELATE TO STAGE, LOADING, AS ADVISED BY HUMES, HOLCIM, WELSHPOOL WA. LEG THICKNESSES MAY BE REDUCED AS DESIGN LOADS REDUCE FOR SUBSEQUENT STAGES.

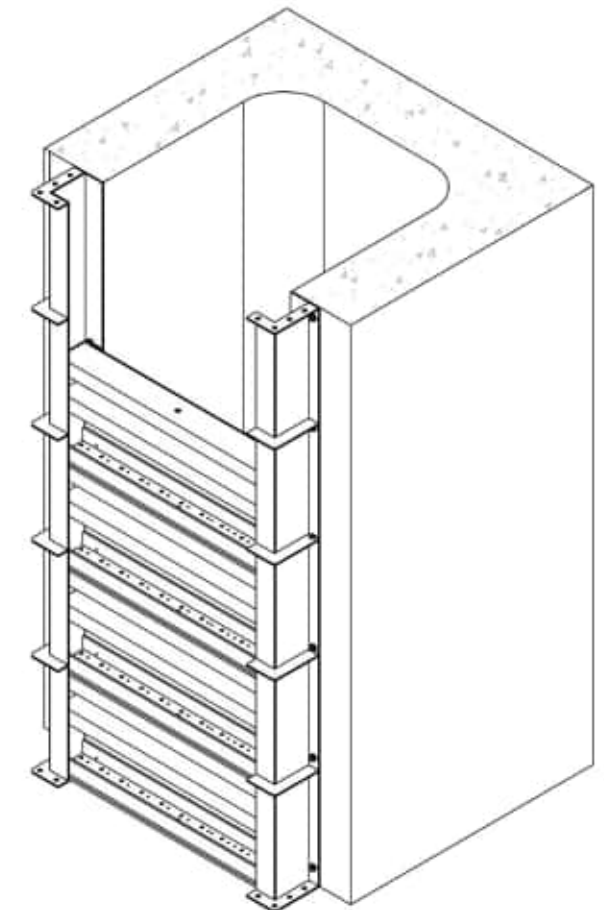
**ISSUED FOR REVIEW**  
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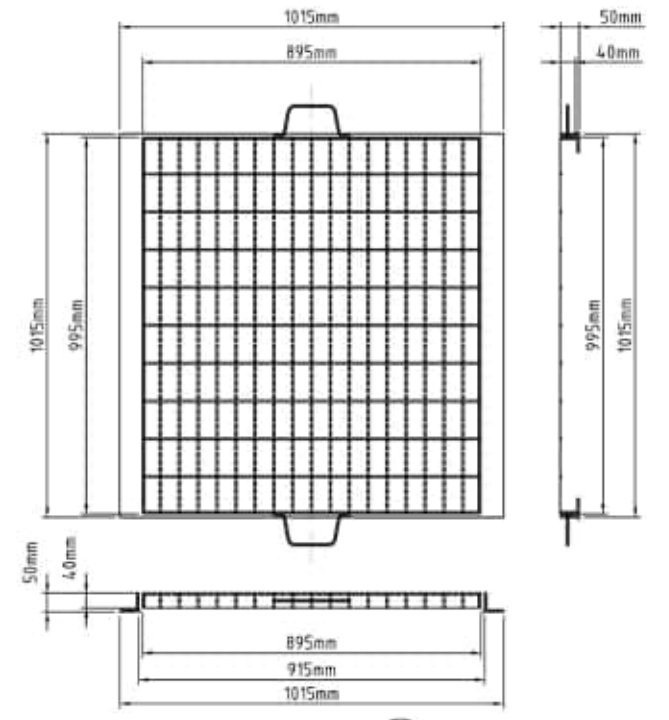
PLAN VIEW - STAGE 1 DECANT CHUTE & STOPLOGS  
SCALE 1:20 @ A3



SECTION - STAGE 1 DECANT CHUTE & STOPLOGS  
SCALE 1:20 @ A3

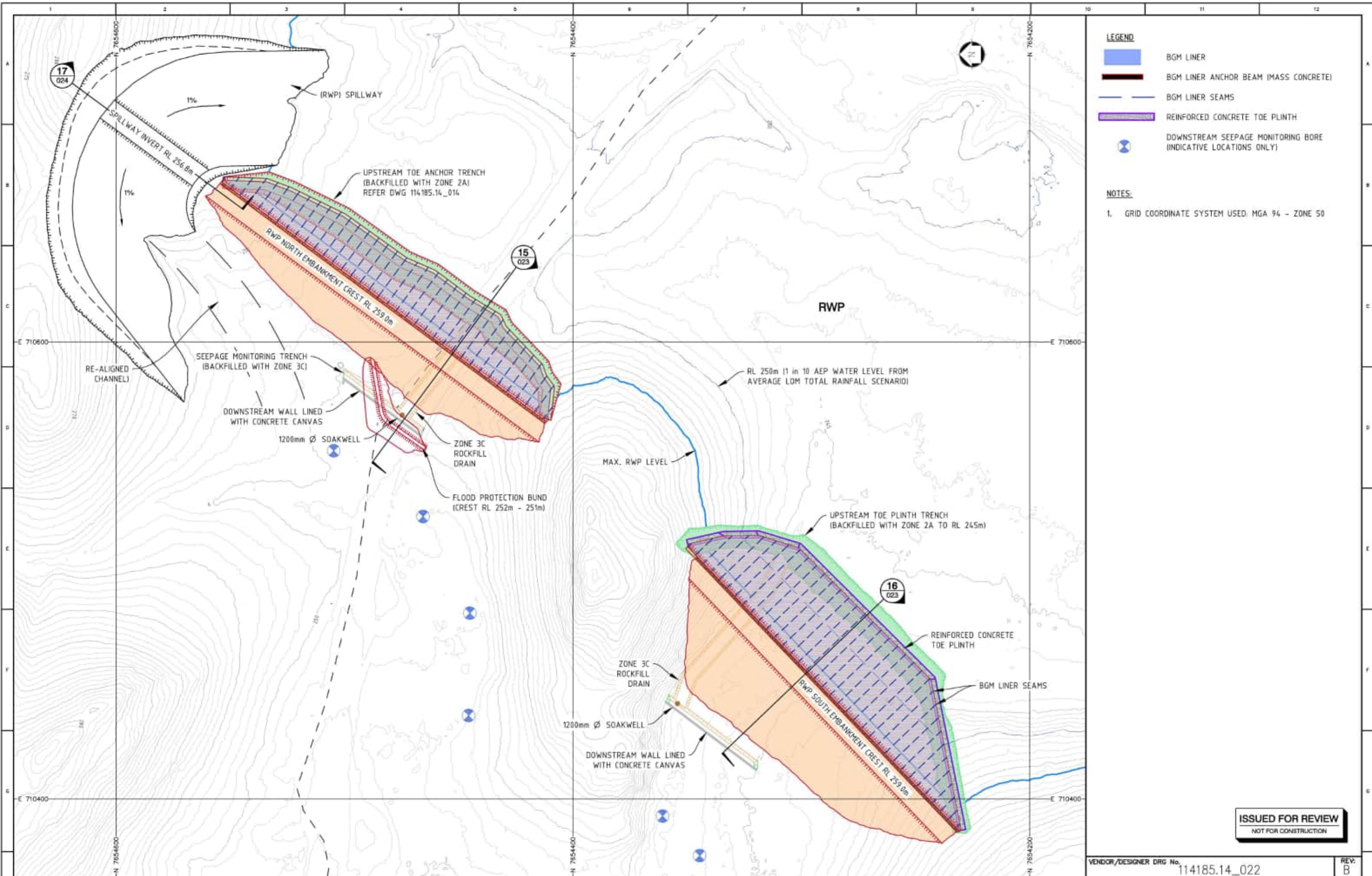


ISOMETRIC VIEW - STAGE 1 DECANT CHUTE & STOPLOG  
SCALE: NTS

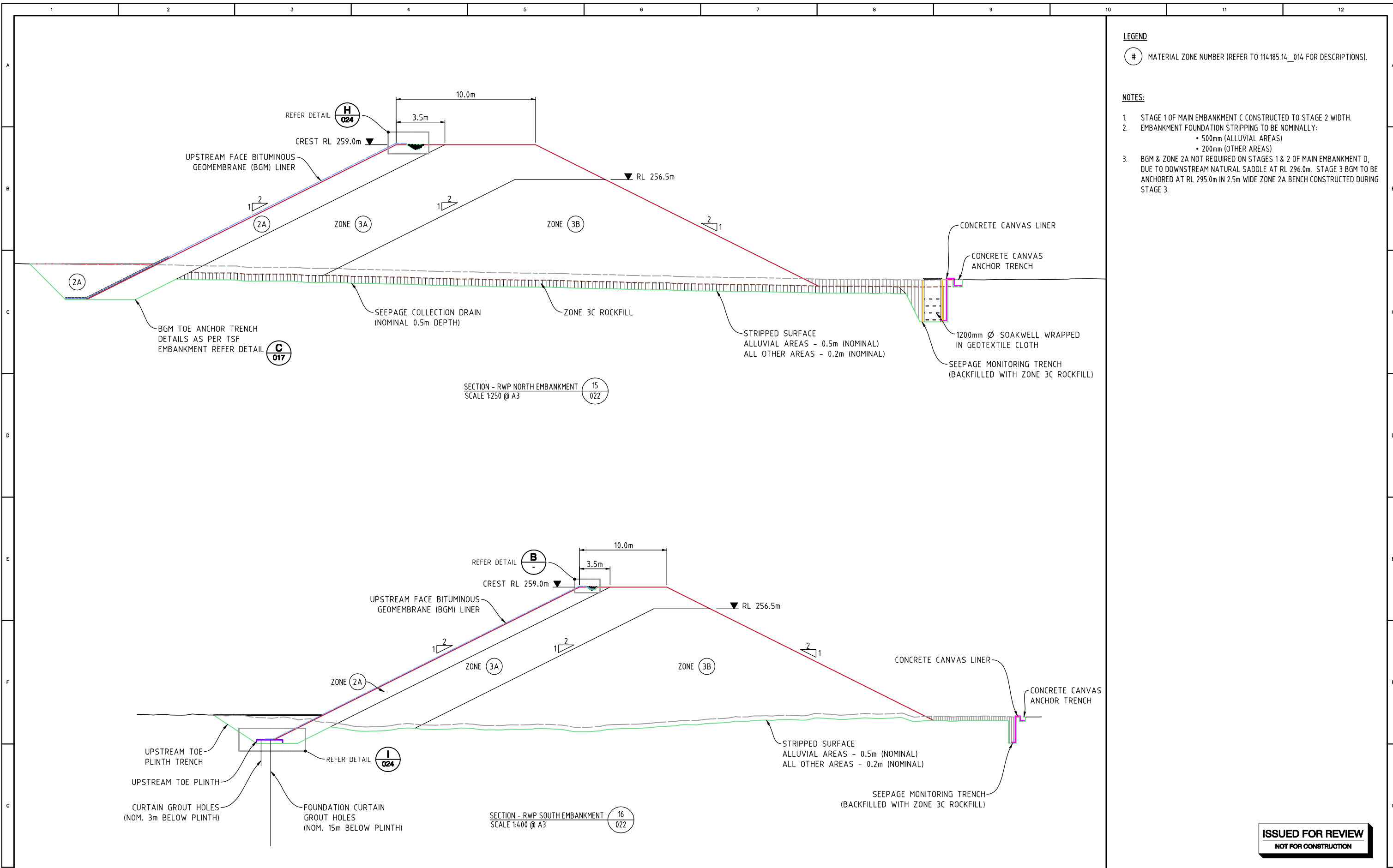


DETAIL - GRAUTING  
SCALE 1:5 @ A3

REF DRG No.		TITLE		REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG			PROJECT NUMBER 114185.14 This drawing must not be copied or reproduced in any form or used for any purpose other than originally intended without written approval of Fortescue Metals Group Ltd.	VENDOR/DESIGNER DRG No. 114185.14_021 Fortescue Metals Group Ltd NORTH STAR MAGNETITE STAGE 2 TSF 2A & RWP TSF DECANTS SECTIONS & DETAILS (SHEET 2 OF 2)	REV: A SCALE: A3 DRG No:
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REF DRG No.		TITLE		REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG			PROJECT NUMBER 114185.14 <small>This drawing must not be copied or reproduced in any form or used for any purpose other than originally intended without written approval of Fortescue Metals Group Ltd</small>	SCALE 1:1500	DRG No: A3	REV: B				
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**ISSUED FOR REVIEW**  
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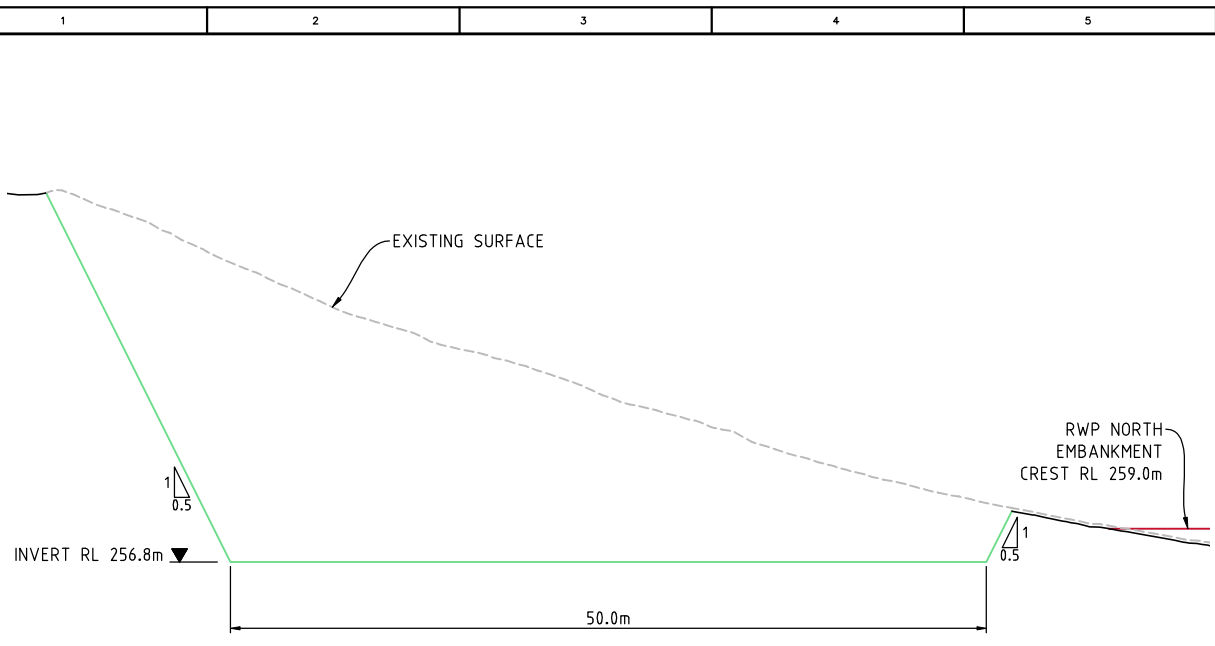
VENDOR/DESIGNER DRG No. 114185.14\_023 REV: A

PROJECT NUMBER Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 RWP GENERAL ARRANGEMENT

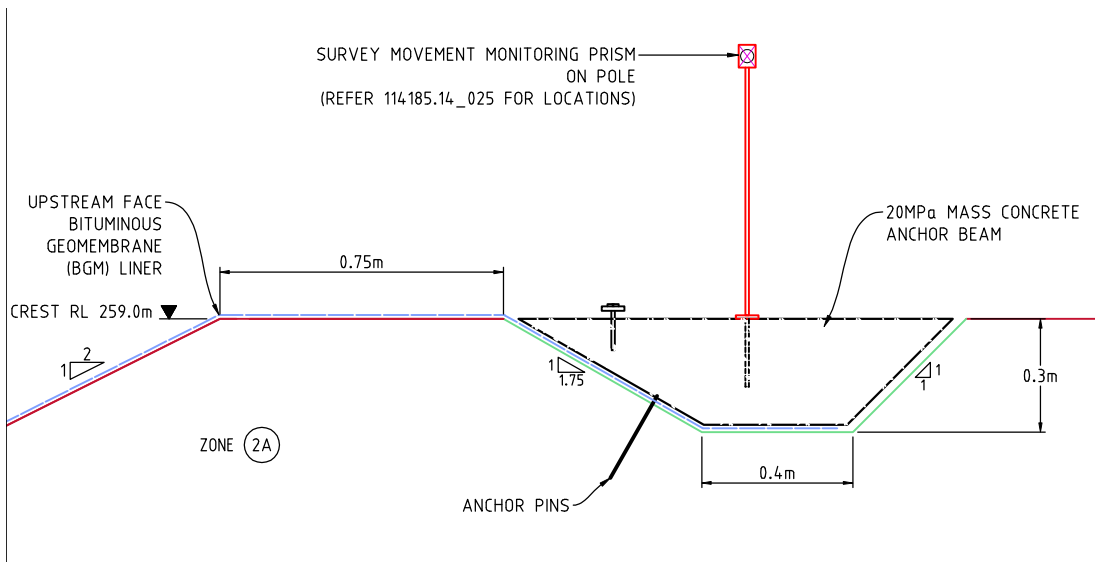
SCALE 1:500 DRG No. A3 REV:

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	CN	HR	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2018	ISSUED FOR CLIENT REVIEW									





SECTION - RWP SPILLWAY  
SCALE 1:500 @ A3

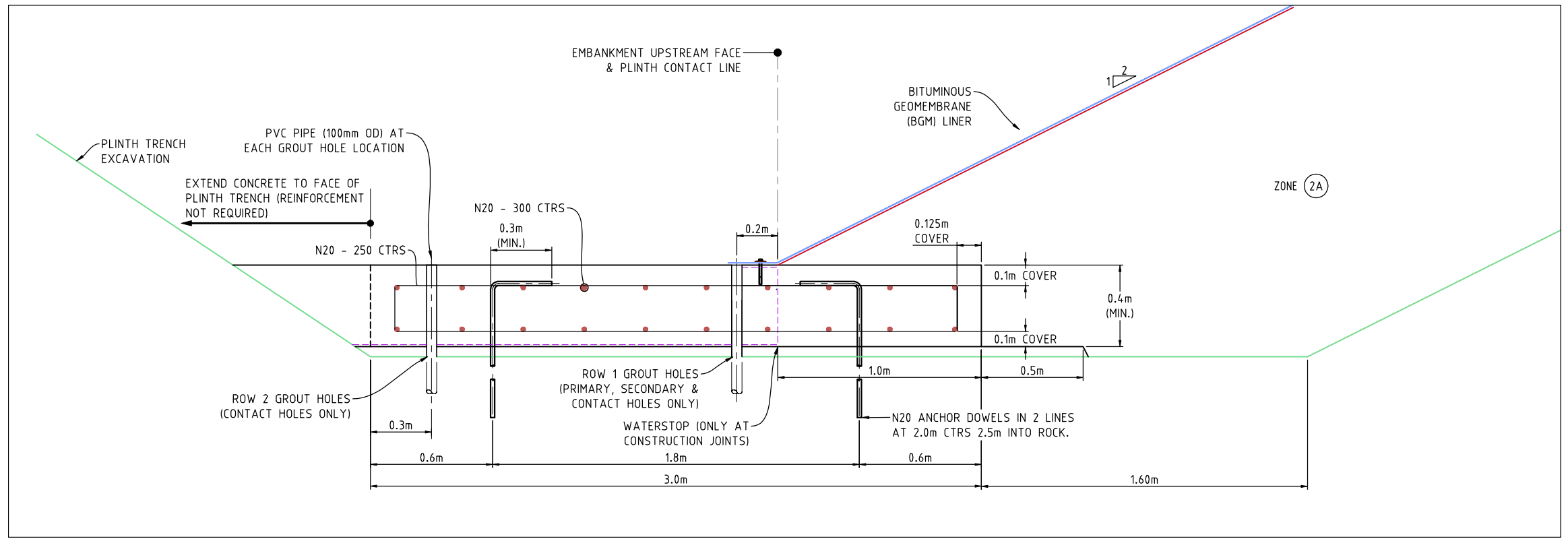


DETAIL - BGM CREST ANCHOR  
SCALE 1:20 @ A3

**LEGEND**

— BGM LINER

□ BGM LINER ANCHOR BEAM



DETAIL - REINFORCED CONCRETE TOE PLINTH  
SCALE 1:20 @ A3

**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_024 REV: A

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2018	ISSUED FOR CLIENT REVIEW							

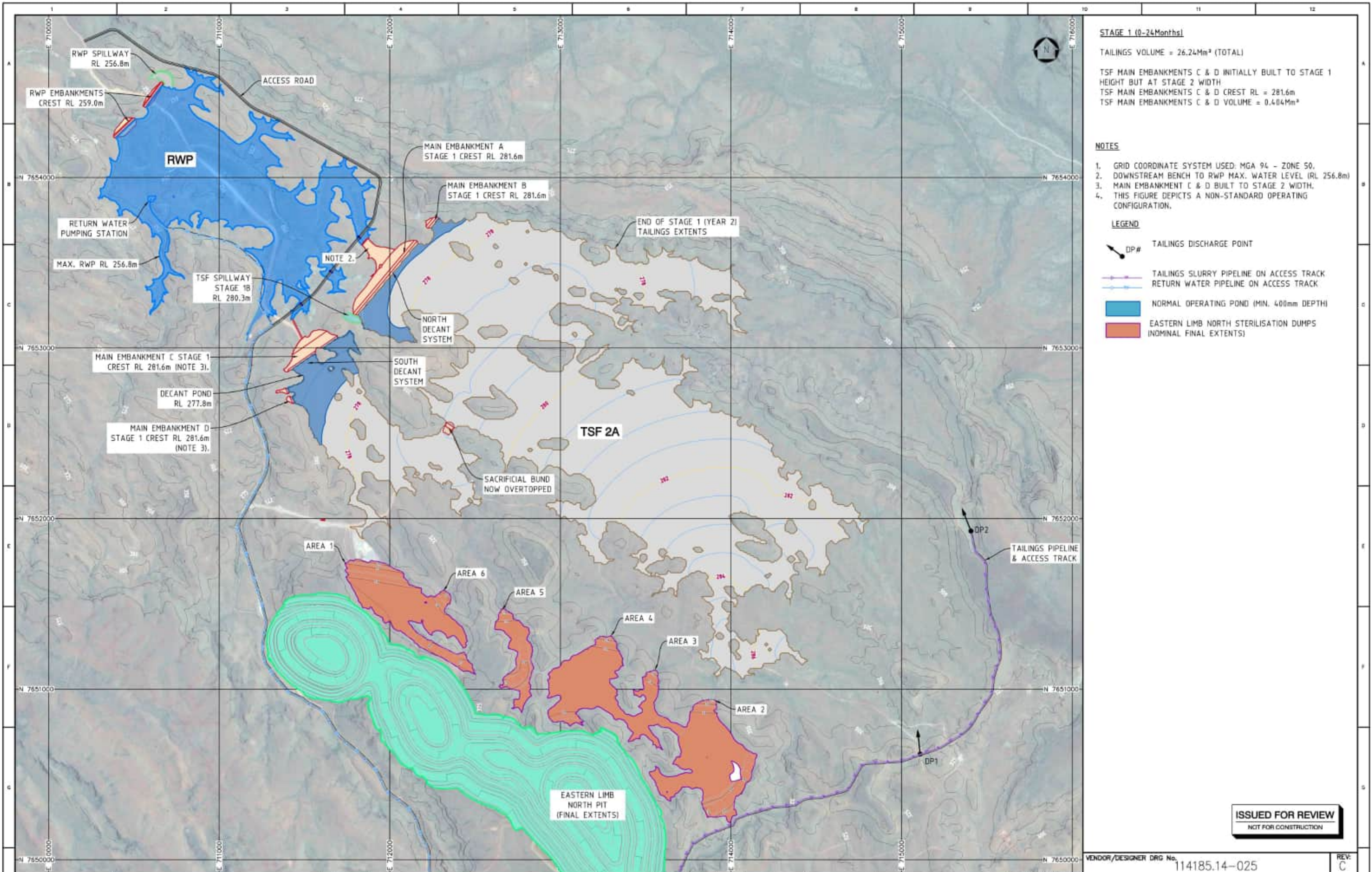
PROJECT NUMBER: Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2  
TSF 2A & RWP  
RWP GENERAL ARRANGEMENT

SCALE: A3 1:500 DRG No: REV: A

Logos for FMG and ATC Williams.

North arrow pointing up.



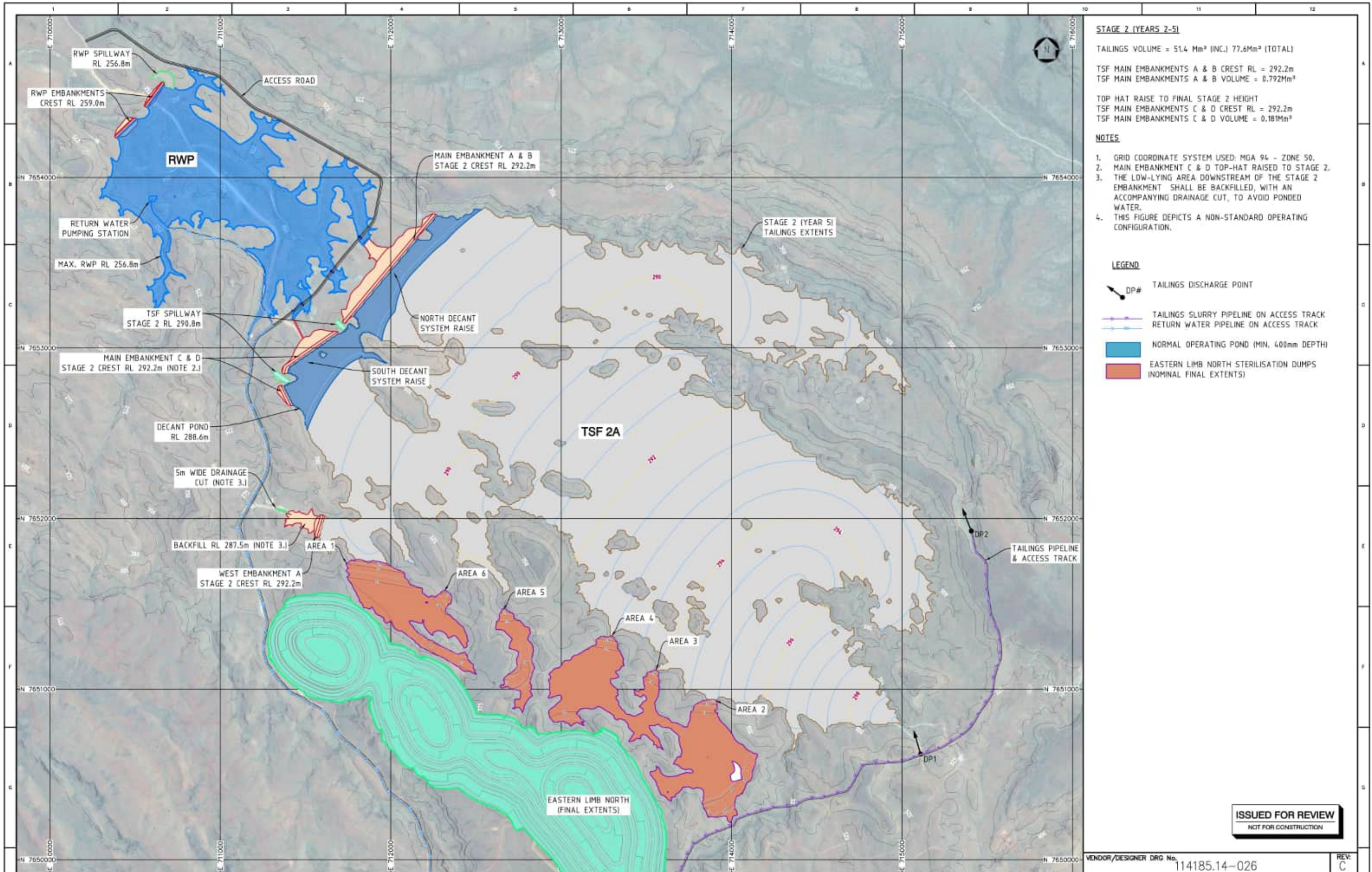
**ISSUED FOR REVIEW**  
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VENDOR/DESIGNER DRG No. 114185.14-025 REV: C

PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 STAGE 1B END OF YEAR 5 TAILINGS DEPOSITION  
 - SINGLE DISCHARGE POINT (DP1)  
 SCALE 1:20000 DRG No. A3

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		C	30.04.2020								
		B	18.02.2019								
		A	26.10.2018								





**STAGE 2 (YEARS 2-5)**  
 TAILINGS VOLUME = 51.4 Mm<sup>3</sup> (INC.) 77.6Mm<sup>3</sup> (TOTAL)  
 TSF MAIN EMBANKMENTS A & B CREST RL = 292.2m  
 TSF MAIN EMBANKMENTS A & B VOLUME = 0.792Mm<sup>3</sup>  
 TOP HAT RAISE TO FINAL STAGE 2 HEIGHT  
 TSF MAIN EMBANKMENTS C & D CREST RL = 292.2m  
 TSF MAIN EMBANKMENTS C & D VOLUME = 0.181Mm<sup>3</sup>

- NOTES**
1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50.
  2. MAIN EMBANKMENT C & D TOP-HAT RAISED TO STAGE 2.
  3. THE LOW-LYING AREA DOWNSTREAM OF THE STAGE 2 EMBANKMENT SHALL BE BACKFILLED, WITH AN ACCOMPANYING DRAINAGE CUT, TO AVOID PONDED WATER.
  4. THIS FIGURE DEPICTS A NON-STANDARD OPERATING CONFIGURATION.

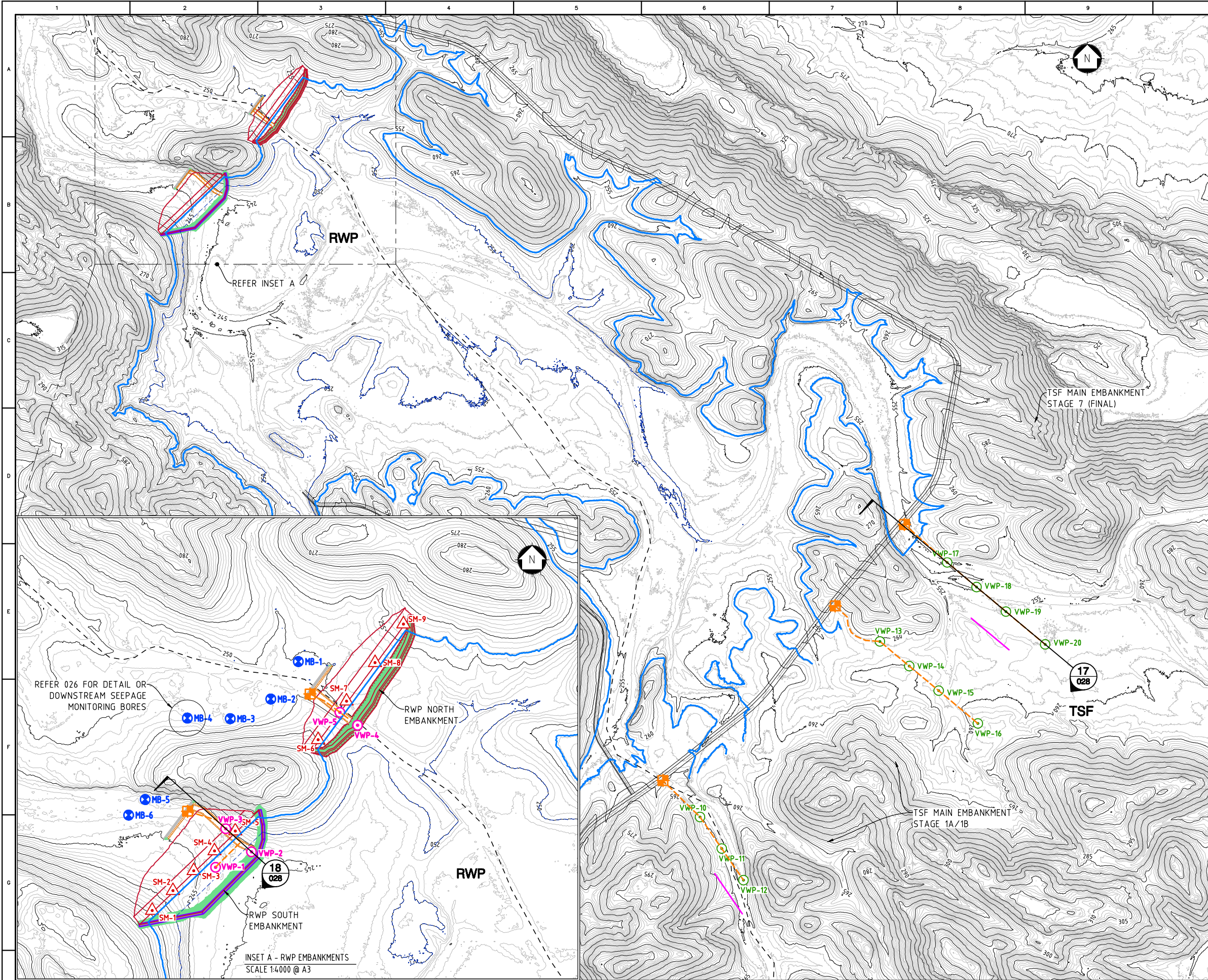
- LEGEND**
- DP# TAILINGS DISCHARGE POINT
  - TAILINGS SLURRY PIPELINE ON ACCESS TRACK
  - RETURN WATER PIPELINE ON ACCESS TRACK
  - NORMAL OPERATING POND (MIN. 400mm DEPTH)
  - EASTERN LIMB NORTH STERILISATION DUMPS (NOMINAL FINAL EXTENTS)

**ISSUED FOR REVIEW**  
 NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14-026 REV: C

REF	DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
			C	30.04.2020								
			B	18.02.2019								
			A	26.10.2018								

PROJECT NUMBER 114185.14  
 Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 STAGE 2 END OF YEAR 5 TAILINGS DEPOSITION  
 - SINGLE DISCHARGE POINT (DP1)  
 SCALE 1:20000  
 DRG No. A3  
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- LEGEND**
- EXISTING ACCESS ROAD
  - △ SM-# SURVEY MONUMENTS RWP
  - VWP-# VIBRATING WIRE PIEZOMETERS RWP (REF. NOTE 2)
  - VWP-# VIBRATING WIRE PIEZOMETERS TSF (REF. NOTE 2)
  - ⊗ MB-# DOWNSTREAM SEEPAGE MONITORING BORE (REF. NOTE 2)
  - ⊕ VWP TERMINAL BOX
- NOTES:**
1. GRID COORDINATE SYSTEM USED: MGA 94 - ZONE 50
  2. INSTRUMENTATION COORDINATES ARE INDICATIVE ONLY, TO BE CONFIRMED IN CONSTRUCTION DOCUMENTATION.

**SURVEY MONITORING RWP**

PT#	EASTING	NORTHING
SM-1	710400	7654245
SM-2	710422	7654266
SM-3	710443	7654286
SM-4	710465	7654307
SM-5	710487	7654328
SM-6	710573	7654423
SM-7	710603	7654463
SM-8	710632	7654504
SM-9	710662	7654544

**BOREHOLES RWP**

PT#	EASTING	NORTHING
VWP-1	710466	7654290
VWP-2	710504	7654306
VWP-3	710477	7654330
VWP-4	710614	7654438
VWP-5	710596	7654452

**BOREHOLES TSF**

PT#	EASTING	NORTHING
VWP-10	711441	7653090
VWP-11	711484	7653029
VWP-12	711527	7652967
VWP-13	711794	7653434
VWP-14	711851	7653386
VWP-15	711909	7653338
VWP-16	711985	7653273
VWP-17	711925	7653588
VWP-18	711983	7653540
VWP-19	712040	7653492
VWP-20	712117	7653428

**MONITORING BORE**

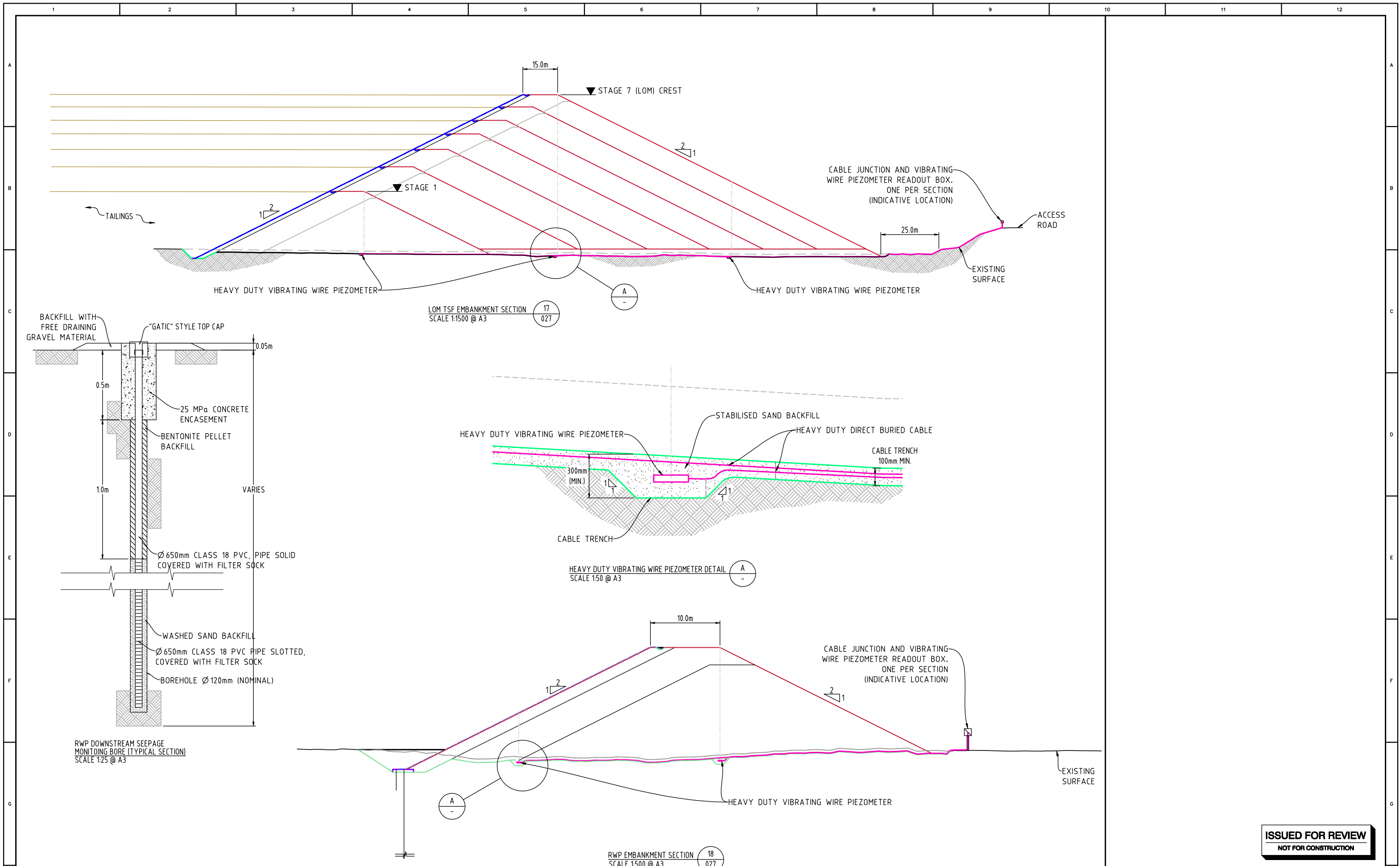
PT#	EASTING	NORTHING
MB-1	710552	7654505
MB-2	710524	7654466
MB-3	710481	7654445
MB-4	710437	7654446
MB-5	710393	7654361
MB-6	710375	7654344

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VENDOR/DESIGNER DRG No. 114185.14\_027 REV: A

PROJECT NUMBER: Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TSF 2A & RWP  
 TSF & RWP EMBANKMENT INSTRUMENTATION  
 GENERAL LAYOUT  
 SCALE: 1:7500  
 DRG No: A3  
 REV: A

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
A	18.02.2019 ISSUED FOR CLIENT REVIEW				JS	CN	HR	CN	JL	CN	



**ISSUED FOR REVIEW**  
NOT FOR CONSTRUCTION

VENDOR/DESIGNER DRG No. 114185.14\_028 REV: A

REF DRG No.	TITLE	REV	DATE	DESCRIPTION	DRN	CHK	SUP	DES	DAP	PAP	FMG
		A	26.10.2018	ISSUED FOR CLIENT REVIEW	JS	PR	HR	PR	GB	CN	



NORTH

PROJECT NUMBER  
114185.14  
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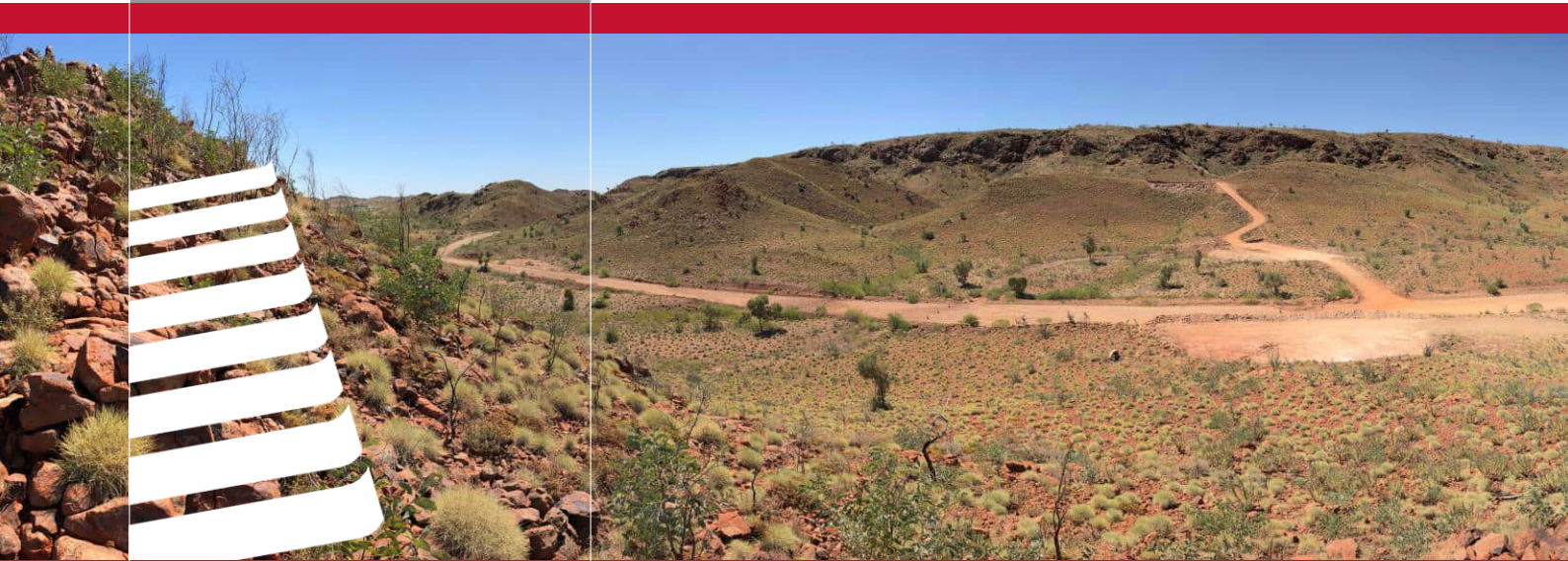
Fortescue Metals Group Ltd  
NORTH STAR MAGNETITE STAGE 1  
TAILINGS STORAGE FACILITY 2A  
TSF & RWP EMBANKMENT INSTRUMENTATION  
DETAILS  
SCALE DRG No: REV:



## TAILINGS STORAGE DATA SHEET

PROJECT OPERATOR			
1. PROJECT NAME: <b>IRON BRIDGE MINE</b>			1.2 Date: October 2018
1.3 TSF Name: <b>TSF 2A</b>	1.4 Commodity: <b>Iron Ore</b>		
1.5 Name of data provider: <b>Mr. Justin Dixon, Senior Geotechnical Engineer, FMG Perth</b>		Phone: <b>08 6218 8327</b>	
1.6 TSF centre co-ordinates (MGA)			
<b>7,649,597m North</b>		<b>713,696m East</b>	
1.7 Mining Tenement and Holder(s) Details: <b>M45/1226</b>			
2. TSF DATA			
TSF Status:    Proposed <input checked="" type="checkbox"/> Active <input type="checkbox"/> Non Active <input type="checkbox"/> Decommissioned <input type="checkbox"/> Rehabilitated <input type="checkbox"/> Closed <input type="checkbox"/>			
2.2 Type of TSF: <b>Down-valley</b>	2.2.1 Number of cells: <b>1</b>		
2.3 Hazard rating: <b>MEDIUM</b>	2.4 TSF category: <b>CATEGORY 1</b>		
2.5 Catchment area: <b>1,308 ha</b>	2.6 Nearest watercourse: <b>Turner River</b>		
2.7 Date deposition started (mm/yy): <b>NA</b>	2.7.1 Date deposition completed (mm/yy): <b>NA</b>		
2.8 Tailings discharge method: <b>Multi-spigots</b>	2.8.1 Water recovery method: <b>Two Decant Towers</b>		
2.9 Bottom of facility sealed or lined? <b>Y/N</b>	2.9.1 Type of seal or liner: <b>NA</b>		
2.10 Depth to original groundwater level <b>2.0 to 21.5 m</b> Current groundwater level <b>2.0 to 21.5 m</b>	2.10.1 Original groundwater TDS/pH <b>1000 mg/l                      8.0</b>		
2.11 Ore process: <b>Cushing &amp; screening, Magnetic Separation</b>	2.12 Material storage rate: <b>NA</b>		
2.13 Impoundment volume (present) <b>0 m<sup>3</sup></b>	2.13.1 Expected starter maximum <b>8 x 10<sup>6</sup> m<sup>3</sup></b>		
2.14 Mass of solids stored (present) <b>0 tonnes</b>	2.14.1 Expected starter maximum <b>12.5 x 10<sup>6</sup> tonnes</b>		
3. ABOVE GROUND FACILITIES			
3.1 Foundation soils <b>silty and gravel sand (SP-SM)</b>	3.1.1 Foundation rocks <b>Weathered BIF, Sandstone and Chert</b>		
3.2 Starter bund construction materials: <b>Bituminous Geomembrane Liner and Coarse Rejects and Rockfill from Mine Waste Rock or Quarry within TSF Impoundment</b>	3.2.1 Wall lifting by: Upstream <input type="checkbox"/> Downstream <input checked="" type="checkbox"/> Centre Line <input type="checkbox"/>		
3.3 Wall construction by: <b>NA</b>	3.3.1 Wall lifting by: Mechanically <input checked="" type="checkbox"/> Hydraulically <input type="checkbox"/>		
3.4 Present maximum wall height agl: <b>0 m</b>	3.4.1 Expected starter maximum <b>27 m</b>		
3.5 Crest length (present) <b>0 m</b>	3.5.1 Expected starter maximum <b>750 m</b>		
3.6 Impoundment area (present) <b>0 ha</b>	3.6.1 Expected starter maximum <b>1200 ha</b>		
4. BELOW GROUND (IN-PIT) FACILITIES			
4.1 Initial pit depth (maximum) <b>N/A m</b>	4.2 Area of pit base <b>N/A ha</b>		
4.3 Thickness of tailings (present) <b>N/A m</b>	4.3.1 Expected maximum <b>N/A m</b>		
4.4 Current surface area of tailings <b>N/A ha</b>	4.4.1 Final surface area of tailings <b>N/A ha</b>		
5. PROPERTIES OF TAILINGS AND RETURN WATER			
5.1 TDS <b>1,000 mg/L</b>	5.2 <b>7.5-8.5 pH</b>	5.3 Solids content <b>65%</b>	5.4 Deposited density <b>1.5 t/m<sup>3</sup></b>
5.5 Potentially hazardous substances: <b>NA</b>		5.6 WAD CN <b>N/A g/L</b>	5.7 Total CN <b>N/A mg/L</b>
		5.8 Any other NPI listed substances in the TSF? <b>Y/N</b>	





**REPORT**

**IRON BRIDGE OPERATIONS PTY  
LTD**

**North Star Magnetite - Stage 2  
Pilbara, Western Australia**

**Tailings Storage Facility &  
Return Water Dam**

**Mining Proposal Design Report**

**Appendix B -  
Geological Mapping &  
Geotechnical Investigations**

**November 2018**

**Doc. No: 114185.14-R04**

## Document History and Status

**Title:** North Star Magnetite - Stage 2  
 Pilbara, Western Australia, Tailings Storage Facility &  
 Return Water Dam, Geological Mapping & Geotechnical  
 Investigations  
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 Please refer to our Conditions of Investigation and Report



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## CONDITIONS OF INVESTIGATION AND REPORT

### Conditions of Report

1. This report has been prepared by us for the purposes stated herein. We do not accept responsibility for the consequences of extrapolation, extension or transference of the findings and recommendations of this report to different sites, cases or conditions.
2. This report is based in part on information which was provided to us by the client and/or others and which is not under our control. We do not warrant or guarantee the accuracy of this information.
3. We believe the conclusions and recommendations contained herein were reasonable and appropriate at the time of issue of the report. However, the user is cautioned that fundamental input assumptions upon which this report is based may change with time. It is the user's responsibility to ensure that input assumptions remain valid.
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### Geotechnical Investigation

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.

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## DRAWINGS

Drawing 114185.14R04\_001  
Drawing 114185.14R04\_002

Site Layout and Test Locations  
RWP Site Layout and Test Locations

Drawing 114185.14R04\_003  
Drawing 114185.14R04\_004

TSF Site Layout and Test Locations  
Stage 1 Stockpiles Site Layout and Test Locations

### APPENDICES

Appendix A	Site Geological Mapping Summary
Appendix B	Test Pit Logs and Photographs
Appendix C	Borehole Logs and Core Photographs
Appendix D	In-situ Permeability Testing Data Sheets
Appendix E	Laboratory Test Certificates

## 1 INTRODUCTION

On behalf of Iron Bridge Operations (IBO), ATC Williams (ATCW) has conducted a geotechnical site investigation for the proposed Tailings Storage Facility 2A (TSF 2A) at the North Star Magnetite Project (the Project) located in the Pilbara region of Western Australia.

The objectives of the geotechnical investigation were to:

- Identify the principal geological units present in the TSF 2A site area;
- Undertake field investigations and laboratory testing of soil and rock samples to an extent sufficient to provide data and relevant geotechnical parameters to confirm TSF design assumptions;
- Assess the availability and suitability of site won soil and rock materials for earthworks construction;
- Provide quantitative parameters for design of the tailings and water containment embankments, and assessment of seepage potential; and
- Characterise the surface and subsurface soils within the TSF impoundment for potential use as growth medium during TSF closure.

## 2 SITE DESCRIPTION

The proposed TSF site is located in an area consisting of a series of flat-topped elongated plateaus (mesas) of banded iron formation sediments (BIF) and intervening wide valley areas containing localised hills of erosion resistant sandstone generally between 10 m and 70 m high (in the western part of the TSF impoundment) and relatively flat areas typically underlain by weathered shale.

The mesas and valleys in the TSF area are aligned in an east-west direction with approximately 1% westward gradient in drainage channels which meander across the valley floor. Ground elevation in the area ranges between 270 m AHD in the valley floor at the western end of the TSF area and 400 m AHD in the mesa crest at the eastern end of the TSF area. Generally, the mesas are approximately 90 m above the valley floor. The TSF valley is approximately 2.5 km wide and 4.4 km long and is located approximately 5 km north of the proposed Stage 2 process plant.

The eastern part of the TSF valley area is split into two sub-valleys by an elongated mesa ridge extending approximately 2 km in a north westerly direction from the eastern side of the TSF impoundment.

The proposed return water pond (RWP) area is located immediately downstream of the TSF where the valley is approximately 600 m wide and 1.5 m long.

Vegetation in the study area is sparse. The creek and valley floor areas support small bushes and gum trees; whereas the hills and elevated areas are less vegetated, having patchy spinifex cover.

### 3 REVIEW OF AVAILABLE INFORMATION

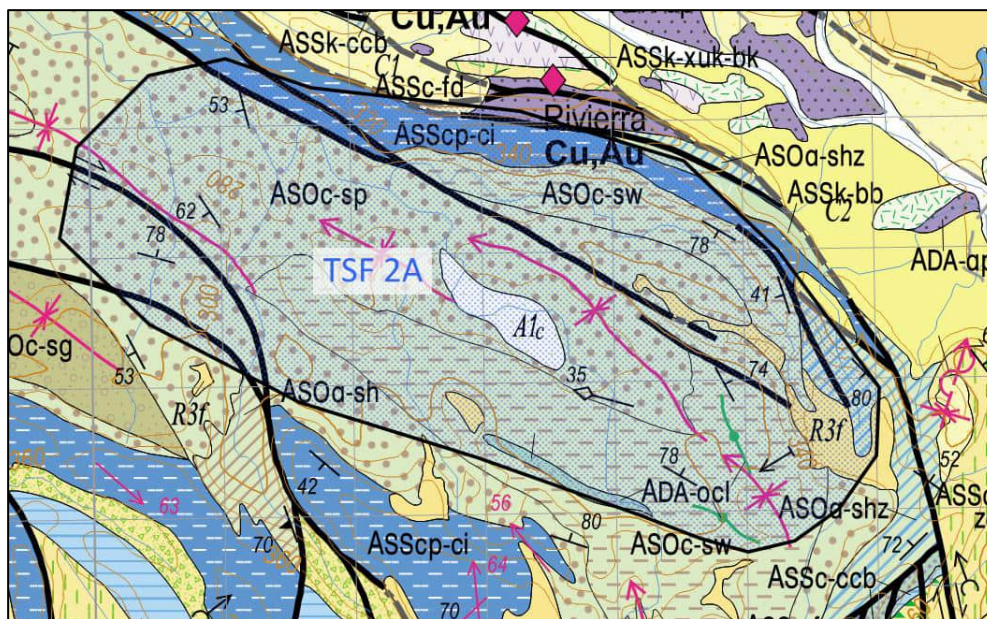
#### 3.1 Regional Geology

The North Shaw 1:100,000 geological maps cover the project area and the regional geology interpretation is largely based on the work of Van Kranendonk (2000) [1].

The Project is situated within the North Pilbara terrane of the Pilbara Craton which is Archaean in age (2.5-3.5 billion years), and contains volcanic and volcanoclastic rocks of the Pilbara Super Group (greenstone belts) intruded by Archaean age felsic granitoids, to form dome and keel type structures.

The area has generally been regionally metamorphosed to greenschist facies, and folded into sub vertical tight to isoclinal folds that have wrapped around granitoid contacts during regional deformation and now are parallel to the granite contacts. This regional deformation was also accompanied by strike slip and reverse faulting which has dislocated the main formations in the project area.

An extract from the North Shaw map is shown in **Figure 1**.



**Figure 1: Site Geology**

Extensive rock exposures are present in the TSF 2A area and superficial cover is typically very thin other than within the confines of current alluvial channels. The area is characterised by metamorphosed sediments of the Soanesville and Sulphur Springs groups of mid Archean age.

The Soanesville group is dominant in the TSF valley area and is represented by rocks of the Corboy and Cardinal formations. The Corboy formation (ASOc) rocks comprise sandstones, wackes (clayey sandstone) and lithic arenites (sandstone with > 50% included rock fragments, typically of fine and medium gravel size). Local interbeds of conglomerate, siltstone and shale are also present. The Cardinal formation (ASOa) predominantly comprises ferruginised black shale with local siltstone, banded iron formations (BIF), sandstone and conglomerate.

Rocks of the older Sulphur Springs group are localised, poorly represented and generally confined to the perimeter of the TSF area. They include components of the Kangaroo caves formation (ASSc) including the Pincuna BIF member, layered cherts, dacite and rhyolite volcanics and the Kunagunarrina formation (ASSk) comprising Basalt and Komatiite.

### **3.2 Previous Investigations**

As part of the North Star project feasibility study, Golder Associates completed a geotechnical and hydrogeological investigation in parts of the TSF 2A area in December 2011 [2].

A total of forty-five test pits were excavated to depths between 0.4 m and 4.0 m (average 1.5 m) and nine groundwater monitoring bores were installed (at six sites) by reverse circulation drilling to depths between 6.5 m and 102 m. As the main embankment at feasibility study stage was located approximately 1 km west of the current design alignment, most of the test pits are located in the currently proposed RWP impoundment area. Two of the monitoring bore sites (BH11 A/B and BH17) coincide with the west and east abutments of the proposed TSF main embankment and the remainder are in the vicinity of the proposed RWP embankments. The locations of the exploratory holes are included on Drawings 114185.14 R04\_001 to 114185.14 R04\_005.

All of the test pits terminated on refusal within gravel deposits (or on rock) and most refused at depths less than 1 m. Minor amounts of clayey gravel and clayey sand were locally identified and a thin (typically 0.6 m) surficial layer of low plasticity sandy clay / clayey sand was encountered at the current proposed RWP location.

Five test pits (TP 41 to TP 45) were located in the area upstream of the TSF main embankment. These pits encountered sandy gravel alluvial deposits (with minor clayey gravel layers) and terminated at depths between 0.9 m and 2.5 m (average 1.75 m).

The rocks penetrated by the boreholes are invariably described as interbedded siltstone and sandstone of medium or high strength except at BH 17 where the entire profile from ground level to 102 m is described as chert. Groundwater was encountered in most bore locations at between 236 m AHD and 242 m AHD; at depths between 4.5 m and 35 m depending on surface elevation. At BH 11A the groundwater level was 265.5 m AHD (20 m depth). BH 17 was recorded as dry but its base (at 102 m depth) was 232 m AHD so it may have just missed intersecting with groundwater.

One test pit, TP-29, had some water ingress overnight to 242.7 m AHD (1.5 m deep). This test pit was near the centre of the stream bed at the current RWP embankment.

The test pit and borehole logs and laboratory test results are included in the relevant appendices of this report.

Two additional groundwater observation bores have subsequently been drilled in the TSF vicinity (NS-Obs24 and NS-Obs25). Groundwater was measured at approximately 8 m depth in both of these wells in November 2014 at elevations of approximately 261 m AHD and 246 m AHD respectively.

## 4 FIELD MAPPING

A site walkover was conducted by ATCW between 19 and 22 January 2015. The purpose of the walkover was to identify:

- Geological conditions at the TSF perimeter embankment sites,
- Sources of potential borrow material for the construction of the starter embankments,
- Likely geotechnical constraints at embankment locations, such as zones of hard excavation, zones of potential high seepage.

Schematic geological maps and photographs for the mapped embankment areas are given in **Appendix A**. The proposed embankment footprints shown on the maps were current at the time of the site visit; however most of these designs have subsequently been modified.

At the main TSF embankment location (**Figure 2**), steeply dipping, interbedded siltstone/shale (pink) and sandstone (yellow) was identified, with banded iron formation (blue) at the right abutment. The strata strike almost perpendicular to the embankment axis and generally dip at 70° northwards.

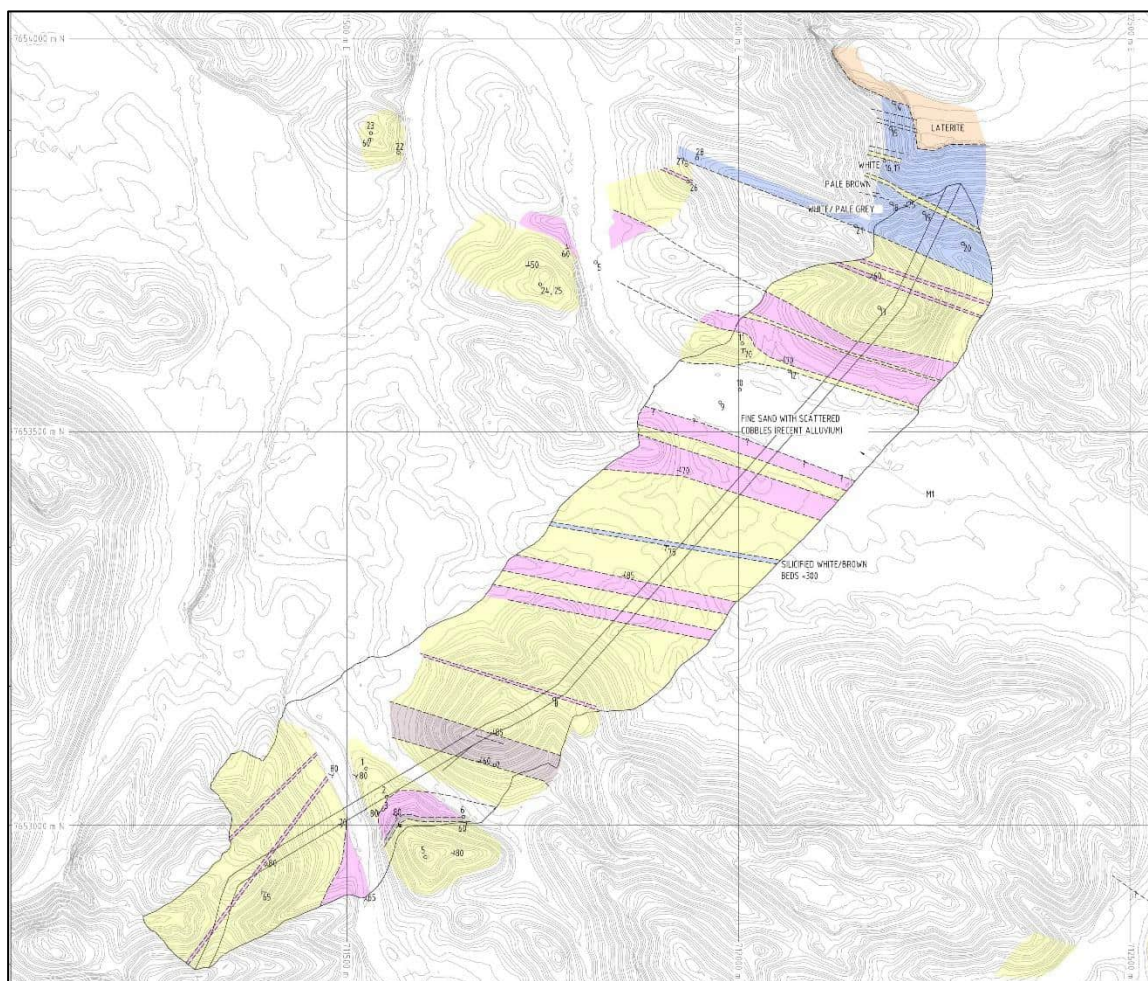


Figure 2: TSF Main Embankment Geology

The rock units have been subject to low grade metamorphism by compression in an east - west direction and contacts between sandstone and interbedded shale units are frequently sinuous. Small scale, tight isoclinal folding is frequently observed in the relatively deformable shale units.

The sandstone outcrops are blocky in nature with typical block dimensions of a few hundred millimetres. The rock mass is discoloured red brown on the surface; however, this is superficial in nature, being a few millimetres thick. Breaking of blocks with sturdy blows of a geological hammer reveals grey, relatively fresh rock material of moderate to high strength beneath.

The shales and siltstones are thinly bedded, typically with alternating purple grey and pale-yellow grey bands. Contacts between individual beds are tight. The rock is typically of low and medium strength.

Superficial deposits are restricted in depth and lateral extent, generally being confined to the valley floor area and the drainage channels that meander westwards between the sandstone hills.

Supplementary geological mapping was also undertaken at the RWP embankment locations in July 2018 **Figure 3**. Details are also included in **Appendix A**.

Moderately weathered fine to coarse grained sandstones of high strength (yellow) are the dominant rock type in the south and central abutment areas. This rock type forms the low hills throughout the RWP and TSF impoundment areas. Moderately weathered shales (pink) of medium strength and siltstones with interbedded shale (blue) of high strength are also prevalent at the north embankment and abutment.

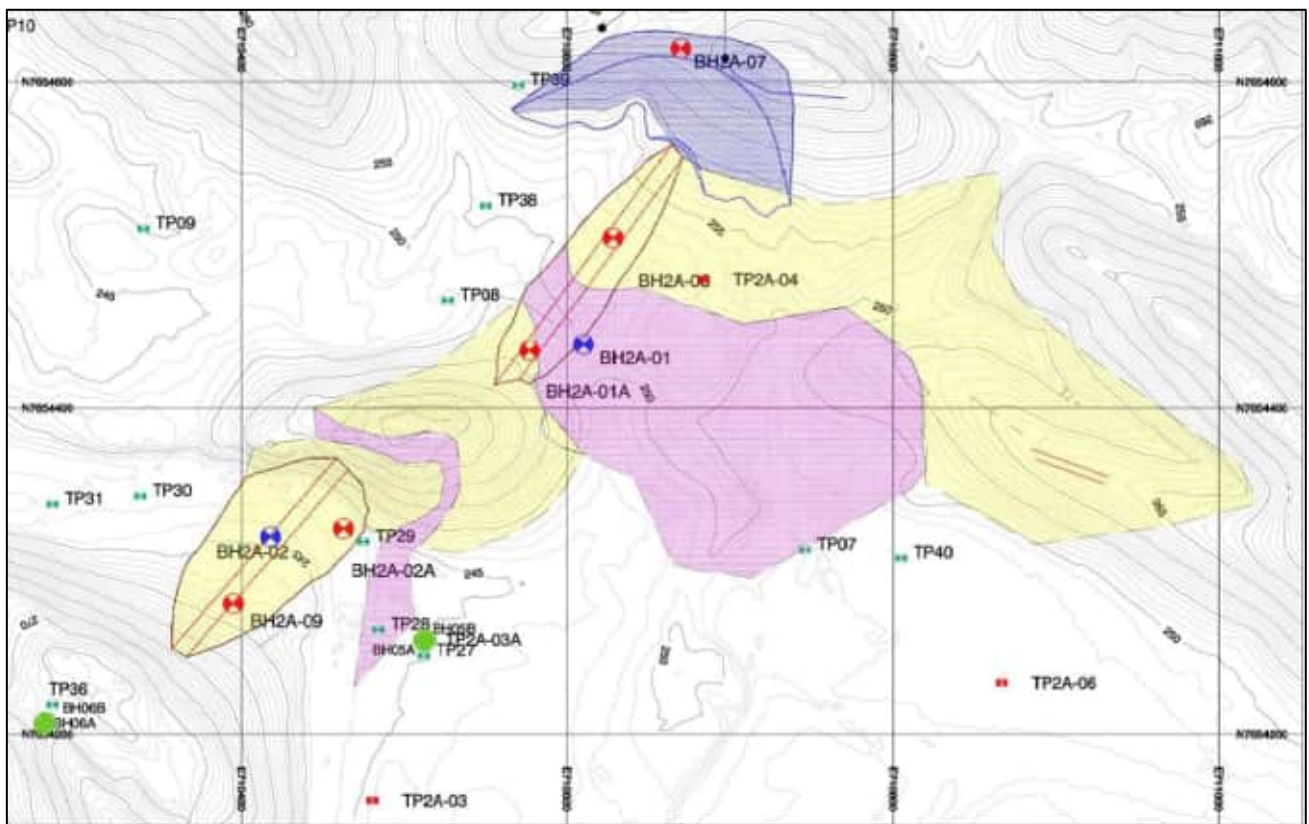


Figure 3: RWP Embankment Geology

## 5 GEOTECHNICAL SITE INVESTIGATIONS (2017- 2018)

### 5.1 General

The ATCW geotechnical field investigation was conducted in three stages between December 2017 and February 2018 which involved the following:

- Stage 1 Investigation - carried out between 11 December and 17 December 2017 including excavation of 40 test pits using a 25 t excavator to depths between 0.5 m and 2.6 m. Samples were also obtained from the existing TSF 1 tailings beach, Stage 1 coarse rejects stockpile and waste rock dump. Sixteen topsoil samples were recovered from the TSF impoundment for material characterisation;
- Stage 2 Investigation - conducted in January and February 2018 including the drilling of six HQ diamond core boreholes to depths between 6 m and 19.5 m in the TSF and RWP embankment footprint areas. Two in-situ falling head permeability tests and four down-hole packer tests were conducted in three of the boreholes. Three boreholes were completed as standpipe piezometers.
- Stage 3 Investigation - conducted in August and September 2018 including the excavation of 52 test pits using a 20 t excavator (to depths between 0.45 m and 4.8 m) and the drilling of eight PQ diamond core boreholes (to depths between 9.0 m and 26 m) within the TSF and RWP areas. In-situ permeability testing was conducted in the area of the RWP embankments and central abutment hill, comprising four falling head tests in two boreholes and 28 straddle packer tests in six boreholes.

The locations of the test pits and boreholes near to existing roads and tracks at the TSF site were generally accessible. However, most of the test pit locations in the north east and centre of the TSF targeted during Stage 1 of the investigation were inaccessible due to the terrain encountered. Minor tracks were made with an excavator to provide access where required during test pit work. Clearing was kept to a minimum to reduce environmental damage.

The fieldwork and logging were carried out by an experienced ATCW Engineer, who logged and sampled the test holes.

The locations of the boreholes and test pits are presented on Drawings **114185.14R04\_001** to **114185.14R04\_005**.

The test pit logs and photographs are presented in **Appendix B**.

The borehole logs and core photographs are presented in **Appendix C**.

In-situ permeability testing data sheets are presented in **Appendix D**.

Laboratory test certificates for soil sample and rock core testing subsequently undertaken are presented in **Appendix E**.

## 5.2 Scope of Investigations

### 5.2.1 Field Investigation Locations

A summary of borehole and test pit information is presented in Table 1 and Table 2.

**Table 1 - TSF Borehole positions, depths and dates**

Hole ID	Easting	Northing	RL (m)	Dip	Depth (m) bgl	Date Started	Date Completed	Hole Type
	GDA94	GDA94						
BH2A-01	710,604	7,654,460	251.1	-90	15	9/02/2018	9/02/2018	HQ
BH2A-01A	710,576	7,654,435	-	-90	26.0	3/09/2018	5/09/2018	PQ
BH2A-02	710,410	7,654,324	244.7	-90	6	9/02/2018	10/02/2018	HQ
BH2A-02A	710,463	7,654,326	-	-90	26.1	31/08/2018	2/09/2018	PQ
BH2A-03	712,077	7,653,499	256.1	-90	19.5	5/02/2018	8/02/2018	HQ
BH2A-04	711,920	7,653,335	257.4	-90	15	11/02/2018	12/02/2018	HQ
BH2A-05	711,489	7,653,037	258.9	-90	15	3/02/2018	5/02/2018	HQ
BH2A-06	711,781	7,652,096	272.7	-90	10	10/02/2018	10/02/2018	HQ
BH2A-07	710,670	7,654,621	-	-90	26.1	8/09/2018	10/08/2018	PQ
BH2A-08	710,628	7,654,504	-	-90	26.1	5/09/2018	7/09/2018	PQ
BH2A-08(A)	710,625	7,654,503	-	-90	12.6	7/09/2018	8/09/2018	PQ
BH2A-09	710,395	7,654,280	-	-90	26.1	28/08/2018	31/08/2018	PQ
BH2A-10	710,659	7,654,067	-	-90	15.2	12/09/2018	13/09/2018	PQ
BH2A-11	711,176	7,654,067	-	-90	9.0	13/09/2018	14/09/2018	PQ
BH2A-12	711,666	7,653,021	-	-90	15.7	11/09/2018	12/09/2018	PQ

**Table 2 - TSF Test Pit positions and depths**

Hole ID	Easting GDA94	Northing GDA94	Depth (m) bgl
TP2A-02	710,602	7,653,858	0.35
TP2A-03	710,481	7,654,159	2.4
TP2A-03A	710,510	7,654,258	0.4
TP2A-04	710,684	7,654,479	0.8
TP2A-06	710,867	7,654,232	2.6
TP2A-07	710,926	7,654,059	0.5
TP2A-08	711,419	7,653,644	1.75
TP2A-09	711,693	7,652,376	0.3
TP2A-11	711,565	7,652,573	0.5
TP2A-15	711,654	7,652,808	0.3
TP2A-16	711,527	7,652,916	1.1
TP2A-18	711,507	7,652,998	0.5
TP2A-19	711,552	7,653,126	0.4
TP2A-20	711,635	7,653,324	0.4
TP2A-21	711,898	7,653,192	0.3
TP2A-22	711,844	7,653,340	0.8
TP2A-23	712,012	7,653,362	1.5
TP2A-24	711,987	7,653,527	1.7

Hole ID	Easting GDA94	Northing GDA94	Depth (m) bgl
TP2A-25	712,236	7,653,500	1.2
TP2A-28	712,339	7,653,343	0.4
TP2A-30	712,068	7,651,775	1.0
TP2A-31	711,808	7,651,935	0.9
TP2A-32	712,000	7,651,896	0.4
TP2A-34	711,967	7,651,986	0.3
TP2A-35	711,868	7,652,058	0.6
TP2A-41	712,404	7,652,743	1.5
TP2A-42	712,563	7,652,988	1.2
TP2A-43	712,746	7,652,840	1.0
TP2A-45	712,775	7,653,252	2.2
TP2A-46	712,850	7,652,620	2.5
TP2A-47	713,048	7,652,671	0.6
TP2A-48	713,068	7,652,895	0.6
TP2A-50	713,153	7,652,511	1.6
TP2A-51	713,481	7,652,471	0.6
TP2A-54	713,543	7,652,153	0.7
TP2A-54A	713,617	7,652,102	0.9
TP2A-57	713,709	7,651,931	0.5
TP2A-61	713,951	7,651,740	0.4
TP2A-69	712,797	7,653,615	0.7
TP2A-70	713,022	7,653,574	1.3
TP2A-R01	710,383	7,654,257	1
TP2A-R02	710,400	7,654,265	2
TP2A-R03	710,415	7,654,275	0.55
TP2A-R04	710,429	7,654,288	0.5
TP2A-R05	710,443	7,654,300	2.1
TP2A-R06	710,448	7,654,303	1.2
TP2A-R07	710,454	7,654,308	2.2
TP2A-R08	710,463	7,654,317	2.8
TP2A-R09	710,468	7,654,323	2.1
TP2A-R10	710,578	7,654,420	0.6
TP2A-R11	710,595	7,654,435	0.7
TP2A-R12	710,612	7,654,453	1
TP2A-R13	710,622	7,654,476	1.4
TP2A-R14	710,639	7,654,496	2.1
TP2A-R15	710,650	7,654,515	1.7
TP2A-R16	710,658	7,654,543	1
TP2A-R17	710,593	7,654,559	0.45
TP2A-R18	710,707	7,654,517	1.2
TP2A-R19	710,457	7,654,233	1.9
TP2A-R20	711,420	7,652,881	1.8
TP2A-R21	711,489	7,652,899	1.2
TP2A-R22	710,423	7,654,330	2.5
TP2A-R23	710,420	7,654,339	2.7

Hole ID	Easting GDA94	Northing GDA94	Depth (m) bgl
TP2A-R24	710,424	7,654,345	2.8
TP2A-T20	711,427	7,652,866	0.6
TP2A-T21	711,489	7,652,899	0.6
TP2A-T22	711,488	7,653,083	1.9
TP2A-T23	711,433	7,653,191	2.5
TP2A-T24	711,661	7,653,016	1
TP2A-T25	711,853	7,653,220	0.7
TP2A-T26	711,911	7,653,268	1.1
TP2A-T27	711,947	7,653,298	1.2
TP2A-T28	712,055	7,653,428	2
TP2A-T29	712,089	7,653,463	2.6
TP2A-T30	712,114	7,653,491	2.4
TP2A-T31	712,134	7,653,513	4.8
TP2A-T32	712,155	7,653,548	2.3
TP2A-T33	712,276	7,653,723	0.6
TP2A-T34	712,006	7,653,479	2.9
TP2A-T35	711,928	7,653,529	2.8
TP2A-T36	711,920	7,653,570	3.5
TP2A-T37	711,841	7,653,640	1.9
TP2A-T38	712,326	7,652,489	1.2
TP2A-T39	712,339	7,652,533	1.7
TP2A-T40	712,358	7,652,559	1.9
TP2A-T41	712,371	7,652,553	2.1
TP2A-T42	712,139	7,653,616	0.5
TP2A-T43	712,257	7,653,705	0.7
TP2A-T44	712,281	7,653,742	0.5
TP2A-T45	711,803	7,653,234	0.5
TP2A-T46	711,622	7,653,071	0.7
TP2A-T47	711,595	7,653,038	0.7

The locations of samples acquired from various stockpiles around the Stage 1 plant and TSF 1 are summarised in **Table 3**.

**Table 3 - Stage 1 Plant area samples**

Hole ID	Easting GDA94	Northing GDA94
SP01 (Coarse Rejects 1)	713,568	7,650,090
SP02 (Coarse Rejects 2)	713,556	7,650,135
SP03 (Coarse Rejects 3)	713,595	7,650,081
Sample A (Coarse Rejects)	713,625	7,650,130
Sample B (Coarse Rejects)	713,624	7,650,125
Sample C (Coarse Rejects)	713,628	7,650,115
SP04 (TSF 1 Tailings)	713,598	7,649,748
SP05 (Waste dump 1)	713,262	7,649,532

Hole ID	Easting GDA94	Northing GDA94
SP06 (Waste dump 2)	713,298	7,649,468
SP07 (Waste dump 3)	713,297	7,649,440
SP08 (ROM pad)	712,965	7,650,206

**Table 4** lists the test pits in the proposed field program that were not excavated. These test pit locations were on steep slopes and in narrow valleys which were not readily accessible to the excavator. Photographs of the ground surface were taken at locations which were readily accessible by foot.

**Table 4 - Unexcavated Test Pits**

Test pit locations with photos only	Inaccessible test pits (No photos)
TP2A-05	TP2A-01
TP2A-12	TP2A-10
TP2A-13	TP2A-14
TP2A-17	TP21-29
TP2A-26	TP2A-44
TP2A-27	TP2A-49
TP2A-33	TP2A-52
TP2A-36	TP2A-53
TP2A-37	TP2A-55
TP2A-38	TP2A-58
TP2A-39	TP2A-59
TP2A-40	TP2A-62
TP2A-56	TP2A-64
TP2A-60	TP2A-65
TP2A-63	TP2A-66
TP2A-71	TP2A-67
	TP2A-68
	TP2A-72
	TP2A-73
	TP2A-74
	TP2A-75
	TP2A-76
	TP2A-77
	TP2A-78

## 5.2.2 Laboratory Testing

Following the fieldwork campaigns, selected soil samples recovered from the test pits, boreholes and stockpiles were submitted to a NATA accredited soils laboratory for a range of testing. The test samples were chosen to determine the design parameters of each material unit encountered at the TSF site. **Table 5** summarises the types and quantities of tests undertaken.

The test certificates are presented in **Appendix E**.

**Table 5 - Laboratory Testing Scope**

Test	Test Output	Quantity
PSD: Sieve Only	PSD Chart	24
PSD: Hydrometer	Fines PSD Chart	6
Atterberg Limits	PI, LL, PL	19
In-situ Moisture Content	% Moisture	19
Standard Maximum Dry Density (SMDD)	Moisture density curve	6
Modified Maximum Dry Density (MMDD)	Moisture density curve	3
Emerson Class (erodibility)	Dispersion numbers	3
Permeability constant head: Re-moulded	Permeability	6
Triaxial : Comprehensive multistage CU	Shear strength & cohesion	3
UCS testing	Uniaxial compressive strength	5
Point Load testing	Point load strength index	10
Slake Durability (Shale)	Slake durability index	2
Water Quality	-	4
Soil pH, EC	-	16
Cation Exchange Capacity	-	16
Organic Carbon	-	16
Total Nitrogen	-	16
Available P, K, S	-	16
NAPP/NAG (Acid Base Accounting)	-	16

## 6 RESULTS OF INVESTIGATION

### 6.1 Subsurface Conditions - Test Pits

Detailed descriptions of the surficial and subsurface conditions encountered are presented on the test pit and borehole logs in **Appendix B**.

The near surface stratigraphy encountered in the test pits is summarised in **Table 6**. Surficial silty, sandy or gravelly material, containing organic matter has generically been described as topsoil.

**Table 6 - Summary of Subsurface Conditions - Test Pits**

Test pit ID	Topsoil	SM	SC	GP/ GP-GM	GC/GC-GP	GW /SW	BOULDERS / COBBLES	SANDSTONE	SHALE/ SILTSTONE
TP2A-02	0.2							0.4	
TP2A-03	0.2					2.2		2.4	
TP2A-03A	0.2		0.3					0.4	
TP2A-04	0.2		0.5					0.8	
TP2A-06	0.2					2.5		2.6	
TP2A-07	0.1								0.5
TP2A-08	0.4					1.8			
TP2A-09	0.1							0.3	
TP2A-11	0.2							0.5	
TP2A-15	0.2								0.3
TP2A-16	0.1				1.0	0.6		1.1	
TP2A-18						0.3		0.5	
TP2A-19	0.2								0.4
TP2A-20	0.2							0.4	
TP2A-21	0.2								0.3
TP2A-22	0.2								0.8
TP2A-23	0.2					0.9		1.5	
TP2A-24	0.3		0.8			1.6		1.7	
TP2A-25	0.3					1.0			1.2
TP2A-28	0.2							0.4	
TP2A-30	0.3					0.7		1.0	
TP2A-31						0.4		0.9	
TP2A-32	0.3							0.4	
TP2A-34	0.2							0.3	
TP2A-35	0.2					0.4		0.6	
TP2A-41	0.3				1.4			1.5	
TP2A-42	0.3		1.1					1.2	
TP2A-43	0.2				0.9				1.0
TP2A-45	0.4				2.0	1.6			2.2
TP2A-46	0.3				1.1	2.4		2.5	
TP2A-47	0.2					0.5		0.6	
TP2A-48	0.2							0.6	
TP2A-50	0.3				1.5			1.6	
TP2A-51	0.1							0.6	
TP2A-54	0.2							0.7	
TP2A-54A	0.2		0.8					0.9	
TP2A-57	0.2								0.5
TP2A-61	0.1							0.4	
TP2A-69	0.2					0.4		0.7	
TP2A-70	0.2		1.2			1.0		1.3	
TP2A-R01	-							1.0	-
TP2A-R02	0.05							2.0	
TP2A-R03	0.05				0.45			0.55	
TP2A-R04	0.05							0.5	

Test pit ID	Topsoil	SM	SC	GP/ GP-GM	GC/GC-GP	GW /SW	BOULDERS / COBBLES	SANDSTONE	SHALE/ SILTSTONE
TP2A-R05	0.05				1.7				2.1
TP2A-R06	0.05			0.8					1.2
TP2A-R07						1.65		2.2	
TP2A-R08						2.2		2.8	
TP2A-R09						1.65		2.1	
TP2A-R10	0.05			0.4				0.6	
TP2A-R11	0.1								0.7
TP2A-R12	0.05				0.95				1.0
TP2A-R13	0.05				0.25				1.4
TP2A-R14	0.05								2.1
TP2A-R15	0.05								1.7
TP2A-R16	0.05								1.0
TP2A-R17	0.05				0.2			0.45	
TP2A-R18	0.05			0.3				1.2	
TP2A-R19	-					1.3		1.9	
TP2A-R20	0.05								1.8
TP2A-R21	0.05								1.2
TP2A-R22							1.9		2.5
TP2A-R23							2.1		2.7
TP2A-R24							2.6		2.8
TP2A-T20	0.15								0.6
TP2A-T21	0.05								0.6
TP2A-T22	0.05	0.25					1.0		1.9
TP2A-T23	-						1.3		2.5
TP2A-T24	0.05				0.5				1.0
TP2A-T25	0.05								0.7
TP2A-T26	0.05				0.15				1.1
TP2A-T27	0.05						0.3		1.2
TP2A-T28	0.05								2.0
TP2A-T29	0.05	0.5					2.6		
TP2A-T30	0.1			1.9				2.4	
TP2A-T31	-						0.55		4.8
TP2A-T32	0.1			2.3					
TP2A-T33	0.05				0.25				0.6
TP2A-T34	0.05	0.4				2.6			2.9
TP2A-T35	0.1	0.6				2.7			2.8
TP2A-T36							2.1		3.5
TP2A-T37							0.75	1.9	
TP2A-T38	0.05			0.2				1.2	
TP2A-T39	-						0.4	1.7	
TP2A-T40	0.05			0.8					1.9
TP2A-T41	0.05				1.1				2.1
TP2A-T42	0.05								0.5
TP2A-T43	0.05				0.2				0.7
TP2A-T44	0.1								0.5
TP2A-T45	0.05							0.5	

Test pit ID	Topsoil	SM	SC	GP/ GP-GM	GC/GC-GP	GW /SW	BOULDERS / COBBLES	SANDSTONE	SHALE/ SILTSTONE
TP2A-T46	0.05								0.7
TP2A-T47	0.05			0.2				0.7	

Topsoil generally comprised red brown clayey or silty gravel with variable organic and root content. The topsoil was typically 0.1 to 0.4 m thick in the valley floor areas, and generally not more than 0.1 m thickness on slopes and hills.

Colluvium was identified in elevated areas at the foot of hillslopes and intersected to a maximum depth of 2.0 m in test pits. The colluvial soils mainly comprised red brown clayey gravels of low plasticity or sandy gravels, with minor silt and clay content. Cobbles and boulders of local sandstone and siltstone were also abundant in the colluvium, which was mostly medium dense and dense.

Alluvial soils comprising well graded brown, red and grey sand/gravel mixtures, or rounded cobbles with boulders were encountered in the creeks to depths up to 2.6 m before reaching weathered rock. The alluvial soils were visually assessed as medium dense to very dense. Boulders were recorded to a maximum of 800 mm diameter at creek locations.

The majority of test pits were dug to refusal in rock, with the exception of test pits TP2A-08, TP2A-T29 & TP2A-T32 which refused in cobble and gravel layers.

The rock mass exposed in the base of test pits comprised extremely to moderately weathered sandstone or interbedded siltstones and shales. On hills and slopes, a thin mantle (100 mm - 300 mm) of extremely weathered material was typically encountered, which broke to a gravelly matrix upon excavation. Highly weathered sandstone was generally closely jointed, grey with red brown staining and low to high strength. The shales and siltstones were thinly bedded, generally grey and white, and had very low to medium strength, with an apparent vertical to subvertical bedding dip.

## 6.2 Subsurface Conditions - Boreholes

The subsurface conditions encountered in the boreholes are summarised in Error! Reference source not found..

Superficial materials were encountered from surface at the majority of borehole locations, except BH2A-01, BH2A-01A and BH2A-07 where rock was present from surface. The superficial materials comprised clayey gravels, clayey sands with alluvial sands, gravels and cobbles encountered at several boreholes located near to creeks. Alluvial gravels extended to 2.9 m depth at BH2A-02, located in the creek at the RWP south embankment.

**Table 7 - Summary of Subsurface Conditions - Boreholes**

Borehole ID	GW, SW /COBBLES	SC	GC	SHALE	SILTSTONE	SANDSTONE
BH2A-01					0 - 10.0 14.1 - 15.0	10.0 - 14.1
BH2A-01A					21.5 - 22.7	0 - 21.5 22.7 - 26.0
BH2A-02	0.0 - 2.9		5.4			5.4 - 6.0
BH2A-02A				1.4 - 2.8	2.8 - 4.0 9.4 - 22.4	0 - 1.4 4.0 - 9.4 22.4 - 26.1
BH2A-03	0.0 - 2.8					2.8 - 19.5
BH2A-04		0.0 - 1.8		1.8 - 15.0		
BH2A-05			0.0 - 2.9	2.9 - 15.0		
BH2A-06				0.0 - 10.0		
BH2A-07				2.4 - 5.4 12.7 - 14.9 24.7 - 26.1	0.0 - 2.4 5.4 - 12.7 14.9 - 24.7	
BH2A-08			0 - 0.6	17.4 - 21.0		0.6 - 17.4 21.0 - 26.1
BH2A-08(A)			0.0 - 0.4	7.2 - 12.6		0.4 - 7.2
BH2A-09			0.0 - 0.3			0.3 - 26.1
BH2A-10	0.0 - 2.4			2.4 - 6.1		6.1 - 15.2
BH2A-11	0.0 - 2.6			8.0 - 9.0	4.6 - 8.0	2.6 - 4.6
BH2A-12	0.0 - 1.1				1.1 - 15.7	

The rocks encountered below the superficial materials comprised sandstone, siltstone and shales. Rock profiles in the boreholes may be summarised as follows:

#### *RWP Embankment*

At the RWP embankment location, sandstones were the dominant rock type encountered in BH2A-01A, BH2A-02, BH2A-02A, BH2A-08 and BH2A-09. Shales and siltstones were prevalent in BH2A-01 and BH2A-07 located at the low point in the north abutment and at the spillway location respectively.

#### *TSF Main Embankment*

At the TSF main embankment, shales were intersected to depths of 10 m in boreholes BH2A-04, BH2A-05 and BH2A-06 located in gullies and low-lying areas. Sandstone was present to at least 20 m depth at BH2A-03 located at the foot of the north abutment area. Siltstone was encountered mostly throughout BH2A-12 to approximately 16 m depth.

#### *RWP Impoundment*

Two boreholes in the RWP impoundment area (BH2A-10 and BH2A-11) encountered alluvial gravels and cobbles to approximately 2.5 m depth overlying interbedded Siltstone, Sandstone and Shale.

Sandstone was generally medium and coarse grained and massive, with few bedding sets apparent in the fabric of the rock. The rock was predominantly extremely to moderately weathered, low to high strength and was mainly grey at depth but stained red brown and brown through oxidation near to surface. Freshly to slightly weathered sandstone, intersected in several boreholes from below 17 m, was generally of high and very high strength.

Siltstone was typically laminated with grey, purple and red layering and was typically discoloured red brown near to surface. Shale interlayers were also abundant within the siltstone. The siltstones were mostly highly to moderately weathered and medium to very high strength and overall less susceptible to weathering and degradation than the sandstones and shales.

Shale was variably weathered, and coloured pale grey and white where not stained purple brown from oxidation. The structure of the rock was laminated and thinly bedded with some foliation of the fabric evident in places. The bedding dip of the siltstones and shales was generally between 70° and vertical.

### 6.3 In-situ permeability testing

The results of permeability tests completed in the boreholes at the TSF site are summarised in Table 8 - 9. Falling head permeability test results are provided in Table 10.

**Table 8 - RWP Packer Test Results**

ID	Depth (m bgl)	Formation (Typical)	Lugeon Value	Perm. k (m/s)	Behaviour	Condition of rock mass discontinuities
BH2A-01A	4.0 - 6.0	Sandstone HW	359	$3.59 \times 10^{-5}$	Void Filling	Open, closely spaced, or voids
	9.5 - 11.5	Sandstone HW	430	$4.30 \times 10^{-5}$	-	Open, closely spaced, or voids
	12.0 - 15.0	Sandstone HW	250	$2.50 \times 10^{-5}$	-	Open, closely spaced, or voids
	17.5 - 19.5	Sandstone HW	30	$3.00 \times 10^{-6}$	Wash out	Some open
	23.0 - 25.0	Sandstone HW-MW	2	$2.00 \times 10^{-7}$	Void Filling	Tight
BH2A-02A	3.0 - 6.0	Shale/Sandstone EW-HW	50	$5.00 \times 10^{-6}$	Dilation/void filling	Many Open
	6.5 - 10.0	Sandstone MW	61	$6.10 \times 10^{-6}$	Dilation/void filling	Many Open
	12.0 - 15.0	Siltstone MW-SW	30	$3.00 \times 10^{-6}$	Void Filling	Some open
	16.5 - 20.5	Siltstone FR	10	$1.00 \times 10^{-6}$	Laminar	Few partly open
	22.5 - 24.5	Sandstone FR	2	$2.00 \times 10^{-7}$	Wash-out	Tight
BH2A-07	7.5 - 9.5	Siltstone MW-HW	11	$1.10 \times 10^{-6}$	Dilation	Few partly open
	10.0 - 12.0	Siltstone SW	1	$1.00 \times 10^{-7}$	Dilation	Very tight
	15.0 - 17.0	Siltstone MW	3	$3.00 \times 10^{-7}$	Laminar	Tight
	18.5 - 20.5	Siltstone MW	1	$1.00 \times 10^{-7}$	Turbulent	Very tight
	21.5 - 23.5	Siltstone MW	2	$2.00 \times 10^{-7}$	Dilation	Tight
BH2A-08A	7.2 - 9.2	Shale/Siltstone HW-MW	40	$4.00 \times 10^{-6}$	Turbulent	Some Open
	9.0 - 11.0	Shale/Siltstone HW-MW	70	$7.00 \times 10^{-6}$	Turbulent	Many Open
BH2A-08	13.0 - 15.0	Sandstone MW	3	$3.00 \times 10^{-7}$	Void Filling	Tight
	19.0 - 21.0	Shale HW	50	$5.00 \times 10^{-6}$	Void Filling	Some Open
	22.5 - 23.5	Shale SW	2	$2.00 \times 10^{-7}$	Laminar	Tight

ID	Depth (m bgl)	Formation (Typical)	Lugeon Value	Perm. k (m/s)	Behaviour	Condition of rock mass discontinuities
BH2A-09	5.5 - 7.5	Sandstone/Siltstone HW	80	$8.00 \times 10^{-6}$	Dilation	Many Open
	7.5 - 10.5	Sandstone HW	110	$1.10 \times 10^{-5}$	Dilation	Open, closely spaced, or voids
	13.5 - 16.5	Sandstone/Siltstone HW-MW	100	$1.00 \times 10^{-5}$	Wash-out	Many Open
	19.0 - 22.0	Sandstone MW	66	$6.60 \times 10^{-6}$	Laminar	Many Open
	22.0 - 25.0	Sandstone SW	10	$1.00 \times 10^{-6}$	Wash-out	Few partly open

**Table 9 - TSF Packer Test Results**

ID	Depth (m bgl)	Formation (Typical)	Lugeon Value	Perm. k (m/s)	Behaviour	Condition of rock mass discontinuities
BH2A-03	7.0 - 10.5	Sandstone MW	1	$1.00 \times 10^{-7}$	Dilation	Tight
BH2A-03	11.5 - 15.0	Sandstone HW-SW	10	$1.00 \times 10^{-6}$	Void Filling	Few partly open
BH2A-05	7.0 - 10.5	Shale HW-MW	83	$8.3 \times 10^{-6}$	Dilation	Many Open
BH2A-05	10.0 - 15.0	Shale MW	109	$1.09 \times 10^{-5}$	Turbulent/Void Filling	Many open or voids
BH2A-12	3.0 - 4.0	Siltstone HW	8	$8.00 \times 10^{-7}$	Turbulent/laminar	Few partly open
	6.3 - 9.3	Siltstone HW	3	$3.00 \times 10^{-7}$	Void Filling	Tight
	9.5 - 12.5	Siltstone SW	40	$4.00 \times 10^{-6}$	Void Filling	Some open

**Table 10 - Falling Head Permeability Testing**

Borehole ID	Depth (m bgl)	Formation (Typical)	Permeability, k (m/s)
BH2A-01	0.0 - 7.5	Siltstone	$4.31 \times 10^{-6}$
BH2A-02	0.0 - 6.0	Gravelly Sand (Alluvial)	$3.09 \times 10^{-5}$
BH2A-10	0.0 - 2.1	Cobbles (Alluvial)	$3.50 \times 10^{-6}$
BH2A-10	7.4 - 8.0	Sandstone	$1.69 \times 10^{-5}$
BH2A-11	0.0 - 2.3	Cobbles	$8.30 \times 10^{-5}$
BH2A-11	8.3 - 9.0	Siltstone / Shale	$1.34 \times 10^{-5}$

The permeability results may be summarised as follows:

#### *RWP Embankments*

- High permeability (250 - 430 Lugeons) was indicated for the highly weathered sandstone formation in BH2A-01A to approximately 15 m. Pump flow rates observed during tests were indicative of an open, closely fractured rock mass which is consistent with core samples retrieved from the borehole and observations of outcrops on the hills;
- Moderately permeability (50 - 110 Lugeons) was indicated for the extremely weathered shales, siltstones and sandstones to depths of approximately 20 m in BH2A-02A, BH2A-08(A), BH2A-09;
- Below approximately 20 m depth, low permeability values (< 10 Lugeons) were recorded in the sandstones; and
- The rock mass at the north eastern part of the north embankment (Sandstone and shale interbeds) was of low to moderate permeability (2 - 50 Lugeons).

- The rock mass at the spillway location (BH2A-07) generally comprised of shale and siltstone, was of low permeability (1 - 11 Lugeons).

#### *TSF Main Embankment*

- The shales in BH2A-05 at depth of 7.0 to 15.0 m were moderately permeable (83 - 109 Lugeons); and
- Siltstones and sandstones in BH2A-03 and BH2A-12 were of low permeability (<10 Lugeons).

## 6.4 Groundwater

Groundwater was encountered in all the boreholes between 2.3 m and 6.6 m bgl. The inferred groundwater elevation ranges from RL 241.7 m to RL 270.4 m at the TSF location, with an inferred hydraulic gradient of approximately 1:100 towards the North West.

The groundwater levels are summarised in **Table 11** .

**Table 11 - TSF GROUNDWATER LEVELS**

Borehole ID	Surface RL (m)	GWL bgl (m)	Groundwater RL (m)
BH2A-01	251.1	6.6	244.5
BH2A-02	244.7	3.0	241.7
BH2A-03	256.1	4.0	252.1
BH2A-04	257.4	6.4	251.0
BH2A-05	258.9	4.2	254.8
BH2A-06	272.7	2.3	270.4
BH2A-01A	251.1	9.7*	241.4
BH2A-02A	242.2	2.0*	240.2
BH2A-07	251.4	21.5*	229.9
BH2A-08	253.6	9.0*	244.6
BH2A-08 (A)	253.6	9.0*	244.6
BH2A-09	256.5	5.0*	251.5
BH2A -10	245.8	3.0*	242.8
BH2A-11	249.4	4.0*	246.4
BH2A-12	263.0	7.0*	256.0

\* Groundwater depth immediately after drilling

## 6.5 Laboratory Test Results

### 6.5.1 Soils

The result of the soil laboratory analyses for the TSF site are summarised in **Table 12 and 13**.

Laboratory results for natural materials sampled from test pits at the TSF area indicate the materials to comprise sand and gravel mixtures with between 1% and 34% fines. The majority of the samples were classified as clayey gravel or clayey sand. The clayey materials were mostly low plasticity with the exception of TSF-TP2A-45 1.7 - 1.9 m and TSF-TP2A-16 0.7 - 0.9 m which were medium plasticity and high plasticity respectively. Samples TSF-TP2A-46 0.4 - 0.6 m and TSF-TP2A-23 0.4 - 0.6 were classified silty/clayey sand with borderline silt-clay fines.

Samples of near surface creek bed materials varied between fine and medium sand with 11% fines and well graded fine to coarse gravel and cobbles with between 20% and 50% medium and coarse sand.

**Table 12 - Soil Classification Test Results**

Test pit ID	Depth (m bgl)	PSD (%)			ATTERBERG LIMITS				MC (%)	EMERSON CLASS	USCS
		Gravel	Sand	Fines	LL	PL	PI	LS			
Coarse Rejects 1 - SPO1	-	35	36	29	16	11	6	3.6	0.5	-	SC-SM
Coarse Rejects 2 - SPO2	-	97	3	0	-	-	-	-	0.2	-	GP
Coarse Rejects 3 - SPO3	-	44	50	6	18	NP	NP	0.4	0.3	-	SP-SM
Coarse Rejects Sample A	-	65	33	2	-	-	-	-	-	-	GW
Coarse Rejects Sample B	-	53	42	5	-	-	-	-	-	-	GW
Coarse Rejects Sample C	-	65	32	3	-	-	-	-	-	-	GW
Coarse Rejects Samples A, B, C	-	63	33	4	-	NP	NP	0.5	-	-	GW
ROM PAD - SP08	-	72	18	10	-	-	-	-	0.8	-	GP-GM
Waste dump 1 - SP05	-	90	7	3	-	-	-	-	1.7	-	GP
Waste dump 2 - SP06	-	82	13	5	-	-	-	-	2.1	-	GP-GM
Waste dump 3 - SP07	-	54	32	14	-	-	-	-	0.9	-	GM
Tailings Stage 1 - SP04	-	0	12	88	18	13	5	3.6	3.2	6	CL-ML
TSF-TP2A-03	0.5 - 0.7	45	53	2	29	NP	NP	0	1.7	-	SP
TSF-TP2A-04	0.2 - 0.4	36	30	34	28	13	15	6.7	2.6	5	GC
TSF-TP2A-06	1.0 - 1.2	50	47	3	25	NP	NP	0	1.9	-	GW
TSF-TP2A-16	0.2 - 0.4	57	35	8	25	15	11	3.6	5	-	GP-GC
TSF-TP2A-16	0.7 - 0.9	57	30	13	52	28	24	10.4	12.3	-	GC
TSF-TP2A-23	0.4 - 0.6	9	70	21	28	22	6	2	2.3	-	SC-SM
TSF-TP2A-24	0.5 - 0.7	15	63	22	23	14	9	1.2	5	-	SC
TSF-TP2A-42	0.5 - 0.7	57	30	13	28	13	15	6.4	4.3	-	GC
TSF-TP2A-45	1.7 - 1.9	54	33	13	47	21	27	12	10.1	-	GC
TSF-TP2A-46	0.4 - 0.6	29	55	16	22	15	7	2.4	2.8	-	SC-SM
TSF-TP2A-54a	0.4 - 0.6	7	64	29	25	12	14	4	4.6	5	SC
TP2A-T35	0.2 - 0.4	0	89	11	-	-	-	-	-	-	SP-SM
TP2A-R08	0.8 - 1.2	80	19	1	-	-	-	-	-	-	GW

The Emerson Class number tests did not indicate soil dispersion. Remoulded balls of soil did not exhibit dispersion and the presence of calcite or gypsum was not inferred in the samples tested. The samples tested were classified as non-dispersive with Emerson Class 5 and 6.

Coarse rejects samples comprised fine and medium sandy gravels with less than 10% fines and a maximum particle size of approximately 20 mm.

Samples recovered from the Stage 1 waste rock dump comprised cobbles and fine to coarse gravel with sand.

The TSF 1 tailings classify as low plasticity silt with trace sand.

**Table 13 - Soil Density, Compaction and Permeability Test Results**

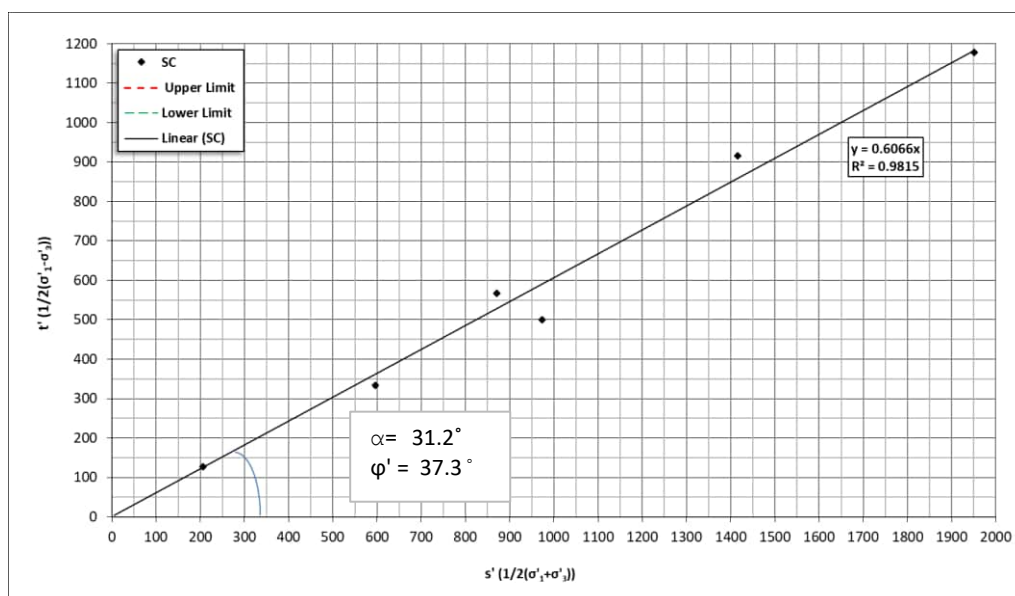
Test pit ID	Depth (m bgl)	Application	Particle Density	SMDD		MMDD		PERMEABILITY (m/s)
				(t/m <sup>3</sup> )	OMC (%)	(t/m <sup>3</sup> )	OMC (%)	
TSF-TP2A-04	0.2 - 0.4	Zone 1	2.81	1.955	11.5	-	-	$6.0 \times 10^{-10}$ @ 98% SMDD
TSF-TP2A-54a	0.4 - 0.6	Zone 1	2.68	2.023	9.7	2.144	7.7	$1.0 \times 10^{-9}$ @ 98% SMDD
Tailings Stage 1 - SP04	-	Zone 1	3.93	2.51	9.7	2.637	8.8	$4.0 \times 10^{-8}$ @ 98% SMDD
TSF-TP2A-03	0.5 - 0.7	Filter	2.73	1.827	11.9	-	-	$3.1 \times 10^{-4}$ @ 90% SMDD
Coarse Rejects Samples A, B, C	-	Filter	2.87	2.44	3.0	-	-	$2.76 \times 10^{-4}$ @ 95% SMDD
Coarse Rejects 3 - SPO3	-	Filter	2.71	2.506	6.4	-	-	$1.1 \times 10^{-5}$ @ 90% SMDD

Measured laboratory permeabilities for samples of clayey sandy gravel and clayey sand remoulded to 98% SMDD were  $6.0 \times 10^{-10}$  m/s and  $1.0 \times 10^{-9}$  m/s respectively. The permeability of similarly compacted tailings was  $4.0 \times 10^{-8}$  m/s. These materials would generally be considered suitable for low permeability embankment fill.

Permeability testing conducted for potential filter materials indicated the coarse rejects to have a permeability in the range of 0.1 to  $3 \times 10^{-4}$  m/s. A sample of sand and gravel recovered from the creek bed and lightly compacted had a permeability of approximately  $3 \times 10^{-4}$  m/s.

The results of two triaxial tests performed on samples of clayey sand re-compacted to 98% SMDD are shown on **Figure 4** as a combined s'-t' plot. The results indicate that the material when compacted would have effective shear strength parameters of  $c' = 0$ ,  $\phi' = 37^\circ$ .

A triaxial test on a sample of tailings from TSF 1, also compacted to 98% SMDD inferred effective shear strength of  $c' = 0$ ,  $\phi' = 38^\circ$ .



**Figure 4: Triaxial Test Results**

## 6.5.2 Rock

Table 14 summarises the results of uniaxial compressive strength and point load index tests carried out on selected rock core samples.

**Table 14 - Rock Strength Test Results**

Borehole ID	Depth (m)	Weathering	Rock type	UCS			Point Load		Strength Class (AS1726-2017)
				Max UCS (MPa)	Modulus (GPa)	Failure Mode	Test Type	I <sub>50</sub> (MPa)	
BH2A-01	13.80 - 13.90	EW/HW	Sandst.	-	-	-	Axial	1.40	High
BH2A-01A	3.20 - 3.62	HW	Sandst.	57.0	49.40	Shear	Axial	2.05	High
BH2A-02A	3.30 - 3.45	HW/EW	Shale	-	-	-	Axial	0.29	Low
BH2A-02A	6.30 - 6.68	MW/HW	Sandst.	60.7	-	Shear	Axial	3.33	Very High
BH2A-03	4.70 - 4.90	EW/HW	Sandst.	8.17	-	Shear	-	-	Medium
BH2A-03	9.80 - 10.10	MW	Sandst.	-	-	-	Axial	0.87	Medium
BH2A-03	11.10 - 11.25	MW	Sandst.	10.44	-	Shear	-	-	Medium
BH2A-03	15.15 - 15.35	MW/SW	Sandst.	-	-	-	Axial	1.14	High
BH2A-04	3.85 - 4.00	EW/HW	Shale	-	-	-	Axial	0.55	Medium
BH2A-04	6.60 - 6.90	MW	Shale	8.02	-	Shear	-	-	Medium
BH2A-04	11.80 - 11.95	MW	Shale	-	-	-	Axial	0.90	Medium
BH2A-04	13.70 - 13.90	MW/SW	Shale	27.8	-	Shear	-	-	High
BH2A-07	1.46 - 1.63	HW	Siltst.	-	-	-	Axial	2.39	High
BH2A-07	6.00 - 6.50	MW	Siltst.	20.0	6.81	Shear	Axial	1.91	High
BH2A-08	3.00 - 3.38	EW/HW	Sandst.	-	-	-	Axial	0.65	Medium
BH2A-09	2.24 - 2.40	EW/HW	Sandst.	-	-	-	Axial	0.15	Low
BH2A-09	23.00 - 23.35	SW	Sandst.	79.9	-	Shear	Axial	4.94	Very High
BH2A-12	1.50 - 1.68	HW	Siltst.	-	-	-	Axial	2.30	High
BH2A-12	5.70 - 6.06	MW	Siltst.	134.7	-	Shear	Axial	1.56	Very High

\* Based on USC strength value

Samples of extremely weathered to highly weathered sandstone were mostly medium and high strength, with moderately and slight weathered specimens having high and very high strengths.

The shale was comparatively weaker. Highly and moderately weathered shale samples were low to medium strength and moderately and slightly weathered specimens were medium strength.

The compressive strength of the siltstone was less sensitive to the effects of weathering. Samples of highly to slightly weathered core were classified as either high or very high strength.

## 6.5.3 Topsoil

Where encountered, topsoil was generally dry, brown or pale brown silty gravel or silty sand with rootlets. The topsoil is estimated to be present and recoverable over less than 30% of the TSF and RWP area.

Laboratory testing of sixteen topsoil samples was undertaken in accordance with the draft DMIRS guidelines for materials characterisation [3]. The results are given in **Appendix E**.

The WA soil group is classified according to Schoknecht and Pathan [4] and has been assigned as Soil Group 203 Stony Soil with a qualifier of LMM (Loamy Matrix). This is based on the relatively neutral pH values and description of the topsoil as either silty gravel or silty sand. Soils within this category are considered to have a moderate inherent fertility level.

Soil pH was near neutral (6.7 - 8.5) and conductivity generally ranged from 0.01 ms/cm to 0.08 ms/cm. Total organic carbon ranged from 0.2% to 0.9%.

Table 15 provides an overview of the soil characterisation.

**Table 15 - Topsoil Characterisation**

Test pit ID	Lab ID	Nutrient Status					AMD Potential	Salinity (ECe) 23 x ms/cm (ms/cm = ds/m)	Sodicity (ESP %)	Dispersion Risk Class	Erosion Hazard Wind	Erosion Hazard Water
		NOx	N	P	K	S						
TSF-TP2A-03	LLS18/27	3	42 0	4	20 0	1.7	NAF MPA < 2 Kg H <sub>2</sub> SO <sub>4</sub> /t	0.92	8.6	Class 1	Low to Moderate	High
TSF-TP2A-06	LLS18/28	1	25 0	4	18 0	1.4		0.69	16.3	Class 1		High
TSF-TP2A-08	LLS18/29	1	36 0	5	21 0	1.3		0.69	6.7	Class 2A		Moderate
TSF-TP2A-11	LLS18/30	<1	23 0	3	16 0	1.5		0.69	7.5	Class 2A		Moderate
TSF-TP2A-19	LLS18/31	1	45 0	4	17 0	2.1		1.61	7.8	Class 1		High
TSF-TP2A-22	LLS18/32	<1	36 0	3	16 0	1.0		1.15	8.0	Class 1		High
TSF-TP2A-32	LLS18/33	<1	27 0	<2	96	1.5		0.23	11.4	Class 1		High
TSF-TP2A-37	LLS18/34	<1	24 0	4	13 0	0.5		1.84	1.4	Class 2A		Moderate
TSF-TP2A-39	LLS18/35	23	28 0	6	31 0	110.0		29.9	31.7	Class 1		High
TSF-TP2A-41	LLS18/36	1	53 0	6	33 0	1.9		0.92	6.3	Class 2A		Moderate
TSF-TP2A-45	LLS18/37	2	35 0	4	22 0	1.0		0.69	11.3	Class 1		High
TSF-TP2A-50	LLS18/38	<1	20 0	<2	25 0	0.9		0.46	10.0	Class 1		High
TSF-TP2A-56	LLS18/39	<1	16 0	3	17 0	<0.5		0.23	8.9	Class 1		High
TSF-TP2A-57	LLS18/40	<1	50 0	<2	19 0	<0.5		0.92	4.0	Class 2A		Moderate
TSF-TP2A-65	LLS18/41	<1	15 0	3	83	1.0		0.23	15.0	Class 1		High
TSF-TP2A-69	LLS18/42	18	33 0	3	16 0	3.3		1.84	10.0	Class 1		High

The intended action is the removal of the topsoil in the TSF area and stockpiling for future re-use, consequently the bulk density and volumetric water content of the material will depend on the level of compaction and moisture conditioning applied during spreading. These parameters together with the species of vegetation proposed for rehabilitation will influence the plant available water capacity (PAWC) of the soil. As a general rule for sandy soils, PAWC may be in the region of 0.5 mm/cm.

Nutrient levels are towards the low end of the range generally considered desirable to support plant growth.

The topsoil samples may generally be regarded as non-saline ( $EC_e < 2$  ds/m, where  $EC_e$  is based on a multiplier of 23 to measured EC values for sandy soils). An exception is test pit TSF-TP2A-39, where a higher EC level classes the material as being highly saline.

The majority of topsoil samples fall within the marginally sodic to sodic category [5] (ESP 6-14), while three samples (TSF-TP2A-06, TSF-TP2A-39, TSF-TP2A-65) classify as strongly sodic (ESP>14).

The dispersion risk class has been assessed using the classification method of Hazelton and Murphy [6]. Class 1 materials are considered to be materials that disperse spontaneously in water. They are unstable, sodic soils that can have severe management and erosion problems. Class 2A materials are soils with few structural problems if there is no mechanical stress from earthworks present.

The topsoil materials have similar particle size distribution to the underlying subsoils. Based on analyses of these materials, the proportion of aggregates > 0.85 mm is generally > 30% where gravelly material is present in the subsoil, giving a low wind erodibility rating. Where less gravelly subsoil is present, the proportion of aggregates > 0.85 mm is between 20% and 30%, giving a moderate wind erodibility rating.

As the topsoil material at all locations may be considered as sodic, consists of silty gravel or silty sand, and has low total organic carbon content, it should be considered as highly susceptible to water erosion. This is further reinforced by the fact that the majority of locations have been assigned a Class 1 dispersion risk. Due to this, a conservative erosion hazard for water of “High” is suggested for locations with Class 1 dispersion risk and “Moderate” for locations with Class 2A dispersion potential.

DMIRS guidelines note that for materials with low clay content (<10 % clays) sodicity is of a low risk and subsequently the dispersion risk is lower.

Overall; the soil characteristics indicate that if properly managed, the topsoil would provide a suitable medium for growth during rehabilitation of the TSF.

#### 6.5.4 Groundwater

Samples of groundwater were recovered from boreholes 2A-01, 2A-04 and 2A-05 for water quality analysis (inorganic analytes).

The results indicate the groundwater to be of livestock drinking water quality [7].

TDS was between 500 mg/l and 600 mg/l and pH was slightly alkaline (8.08 - 8.31).

Most analytes were present in concentrations less than the trigger levels for human drinking water quality [8], except for Arsenic which marginally exceeded the Groundwater Investigation Level (GIL) of 0.01 mg/l (0.009 mg/l - 0.034 mg/l). The livestock drinking water guideline Arsenic concentration is 0.5 mg/l.

## **7 CONCLUSIONS AND RECOMMENDATIONS**

### **7.1 Key Results**

The key results from the site investigation are summarised as follows:

- The surficial sands and gravels present in creek channels are of high permeability. Excavation to remove the surficial sands and gravels will be required beneath the TSF embankments and at the Return Water Pond (RWP).
- The formation below the south west RWP embankment consists of highly to moderately permeable sandstone typically to 20 m depth and it is envisaged that grouting will be necessary to seal the foundation. Seepage control measures will also be required to mitigate the seepage transmission through the sandstone hill between the two RWP embankments.
- The foundation underling the north east RWP embankment and the spillway footprint generally consists of low permeability shales and siltstones. Relatively low permeability rocks are also present beneath the TSF embankments.
- There is likely to be insufficient low permeable clayey soils for conventional upstream low permeability zones or central cores for the embankments. For the TSF dam the selective use of the available natural material and tailings material is a potential solution for the smaller sacrificial embankments. It is envisaged that the main embankments for the TSF and RWP will utilise locally blasted sandstone rock or mine waste for bulk fill and given the shortage of soils for borrow material, the use of a geosynthetic liner, such as a bituminous membrane on the upstream face of the embankments is likely to be required. The Stage 1 coarse rejects material is considered a suitable material to use as a cushion and filter layer for the geomembrane.

### **7.2 Site Preparation and Construction Considerations**

Prior to bulk earthworks and tailings embankment construction, the TSF impoundment and starter embankment footprint should be cleared, grubbed and stripped of topsoil (where present). The topsoil should be stockpiled for subsequent use in rehabilitation of the TSF.

Embankment foundation preparation will include excavation of a cut off trench through the shallow superficial materials and backfilling with low permeability material (lean mix concrete, clayey sand or tailings) to minimise potential for lateral seepage beneath the embankments. The cut off trench will typically be excavated to competent rock. At the RWP south embankment, groundwater is close to surface and temporary dewatering may be required.

At the RWP, placement of available clayey soils from within the impoundment and/or a combination of a geomembrane may be considered to form a composite barrier to seepage.

Excavation conditions in the vicinity of the TSF are expected to be variable. The superficial clayey sand, clayey gravel and sandy materials will excavate easily, but ripping will be required to excavate indurated, near surface highly to moderately weathered rock materials.

The in-situ moisture content of the clayey borrow materials is generally expected to be lower than optimum moisture content at the time of construction (typically 6%-9% dry of OMC), consequently

moisture conditioning by adding water is anticipated to optimise compaction if these materials are used for construction.

### **7.3 Foundations**

Foundation conditions under the embankments are anticipated to comprise dense superficial soils and highly to moderately weathered rock at shallow depth. The rock mass is considered to have sufficient strength to support embankment loading without undue consolidation and settlement.

The embankments will be founded on rock therefore, liquefaction of saturated foundation materials is not expected in the event of an earthquake.

### **7.4 Embankment Construction Materials**

The hills within the impoundment area predominantly consist of jointed durable sandstone and as such may be considered a potential source of rock fill for embankment construction. Sandstone outcrops will require blasting prior to removal with heavy mine plant. Mine waste rock from the existing waste rock dump and future operations is likely to be required for ongoing construction stages as access to the sandstone hills becomes impacted by the developing tailings beach.

Areas along the creek line between TP2A-42 and TP2A-54 are potential sources for Zone 1 clayey material with approximate thickness of material estimated to be between 0.5 m to 1.0 m. The total volume of Zone 1 clayey material likely to be available from these areas is of the order of 20,000 m<sup>3</sup> to 30,000 m<sup>3</sup> only.

Tailings from the existing TSF 1 are limited in volume but could also be used for construction of Zone 1; however, for consistency during construction, natural clayey materials should not be mixed with tailings on any single embankment.

Filter material, if required for Zone 2 could potentially be acquired from existing coarse rejects piles at the plant site, or from excavation of the creek sands inside the TSF footprint.

### **7.5 Seepage Control Measures**

The tailings materials are expected to be benign and decant water is expected to be of similar quality to groundwater, consequently the consequences of seepage from the TSF are likely to be minor.

Lateral seepage beneath the embankments will be minimised by the provision of a perimeter cut off trench (and/or grout barrier) on the upstream side of the starter embankments and subsequent extensions.

### **7.6 TSF Design**

The results of the geotechnical investigations will be used for various studies during the design of the TSF and RWP embankments and infrastructure. Details of embankment slope stability analyses, seepage analyses, and material suitability will be presented in the TSF design report and are not discussed in detail within this geotechnical report.

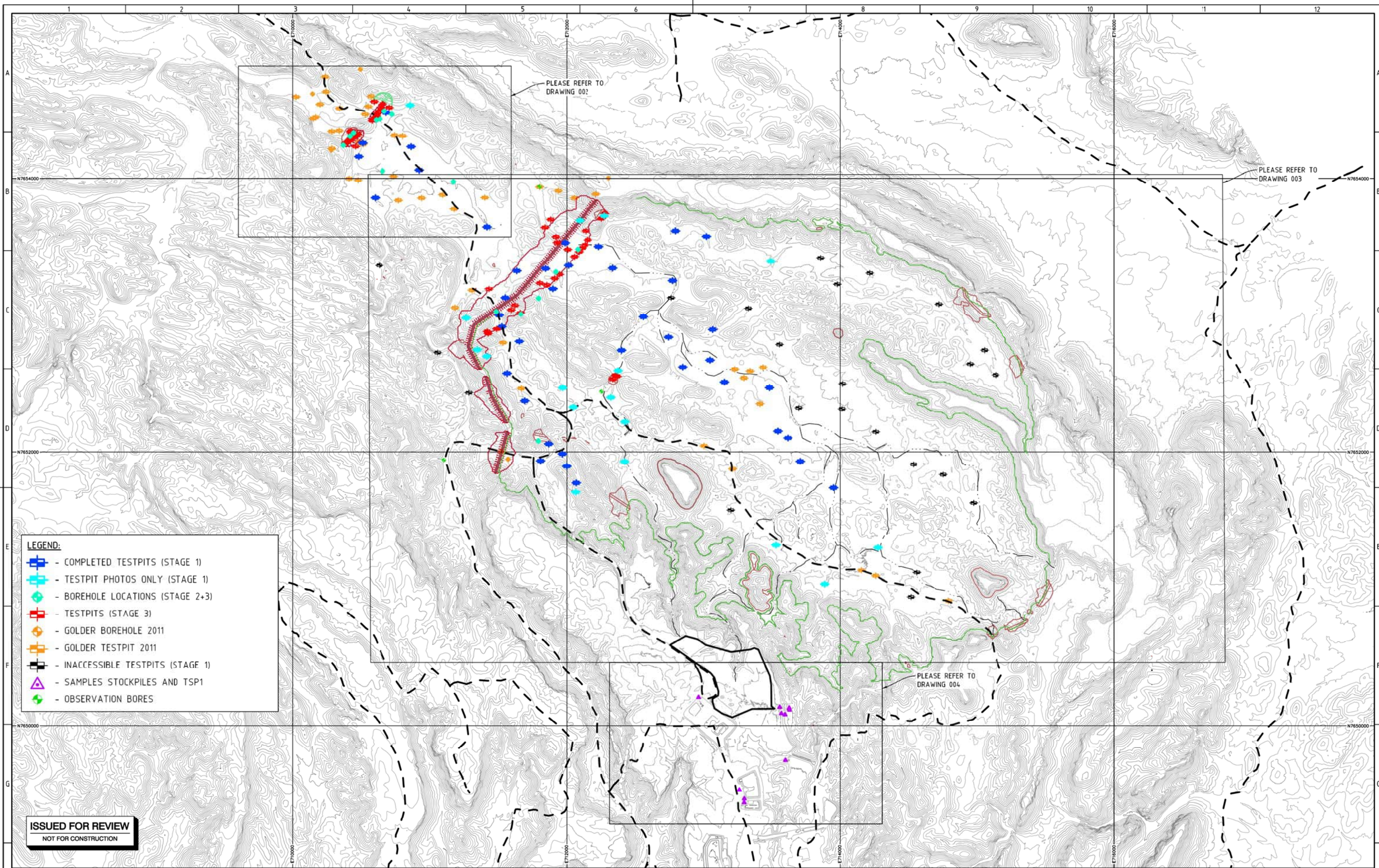
## 8 CLOSURE

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

## REFERENCES

- [1] Van Kranendonk, M.J. (2000): - Geology of the North Shaw 1:100 000 Sheet Geological Survey of Western Australia. Explanatory Notes 1v 86p.
- [2] Golder Associates (2012): Results of Preliminary Geotechnical and Hydrogeological Investigations for the North Star Magnetite Tailings Storage Facility. Technical Memorandum 117645032-032-M-Rev 0.
- [3] Department of Mines and Petroleum (2016): Draft Guidance Materials Characterisation Baseline Data Requirements for Mining Proposals, March 2016
- [4] Schoknecht N Pathan S (2013): Soil Groups of Western Australia: edition 4
- [5] Northcote, KH; Skene, JKM. (1972): Australian soils with saline and sodic properties. Report No.:(27): 62p. procite: baacdecf-4d9d-482b-b4b1-a401fe69b4b5
- [6] Hazelton P Murphy B. (2007): Interpreting Soil Test Results - what do all the numbers mean? 2007
- [7] National Environment Protection (Assessment of Site Contamination) Measure 1999 as amended 2013. Schedule B1; Investigation Levels for Soil and Groundwater.
- [8] ANZECC/ARMCANZ (2000): National Water Quality Management Strategy, Paper 4; Australian and New Zealand Guidelines for Fresh and Marine Water Quality.

DRAWINGS



- LEGEND:**
- COMPLETED TESTPITS (STAGE 1)
  - TESTPIT PHOTOS ONLY (STAGE 1)
  - BOREHOLE LOCATIONS (STAGE 2+3)
  - TESTPITS (STAGE 3)
  - GOLDER BOREHOLE 2011
  - GOLDER TESTPIT 2011
  - INACCESSIBLE TESTPITS (STAGE 1)
  - SAMPLES STOCKPILES AND TSP1
  - OBSERVATION BORES

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No.	DESCRIPTION	DATE	DRAWN	CHECKD	APPRD

SCALE:	1:25,000
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DATE	09/11/2010
DESIGN	AP
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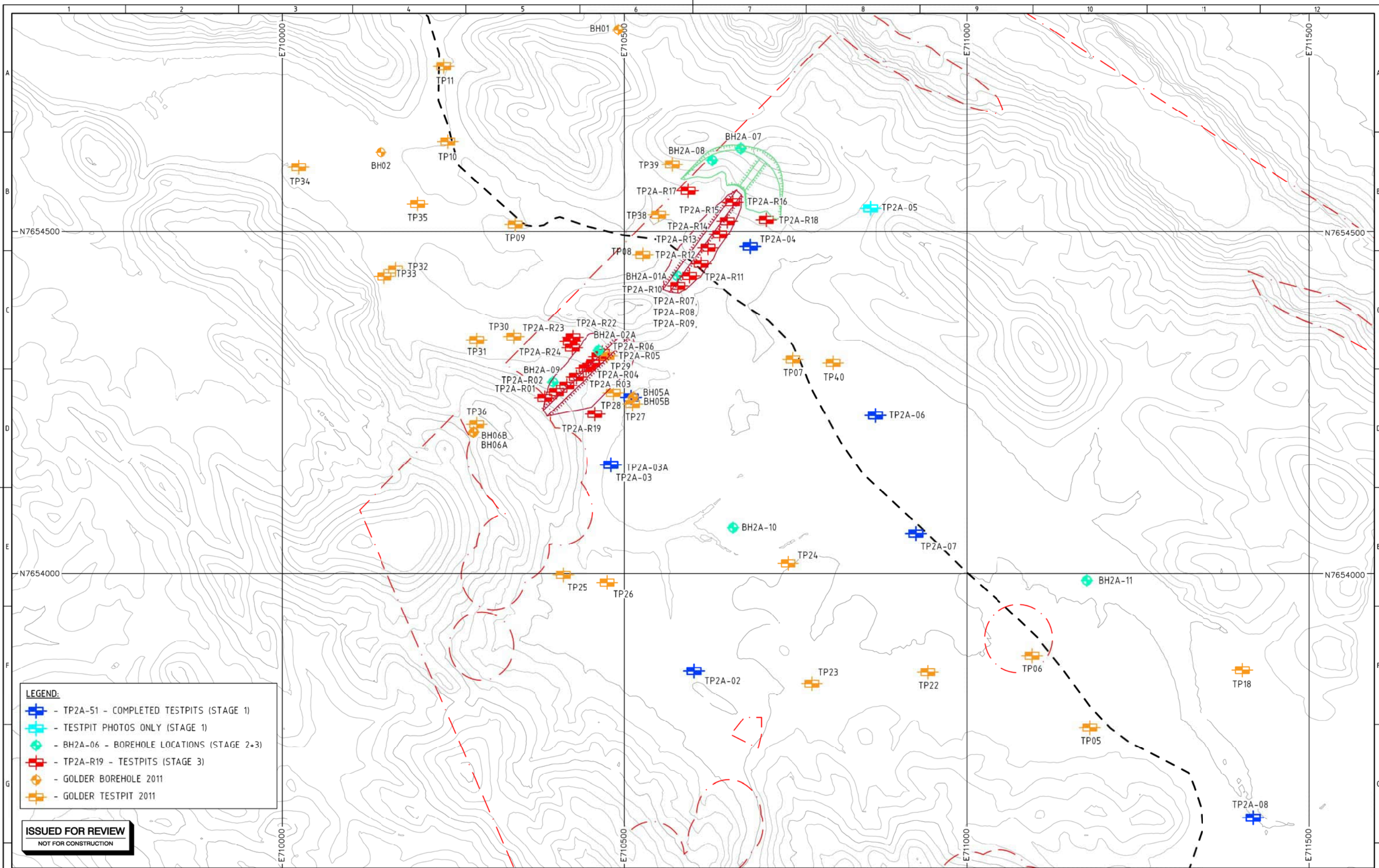
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**IBO  
NORTH STAR MAGNETITE STAGE 2**

---

**TAILINGS STORAGE FACILITY 2A  
SITE LAYOUT AND TEST LOCATIONS**

DWG. No.	114185.10_001	
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SHEET 1 OF 1		



- LEGEND:**
- - TP2A-51 - COMPLETED TESTPITS (STAGE 1)
  - - TESTPIT PHOTOS ONLY (STAGE 1)
  - ◆ - BH2A-06 - BOREHOLE LOCATIONS (STAGE 2+3)
  - - TP2A-R19 - TESTPITS (STAGE 3)
  - ◆ - GOLDER BOREHOLE 2011
  - - GOLDER TESTPIT 2011

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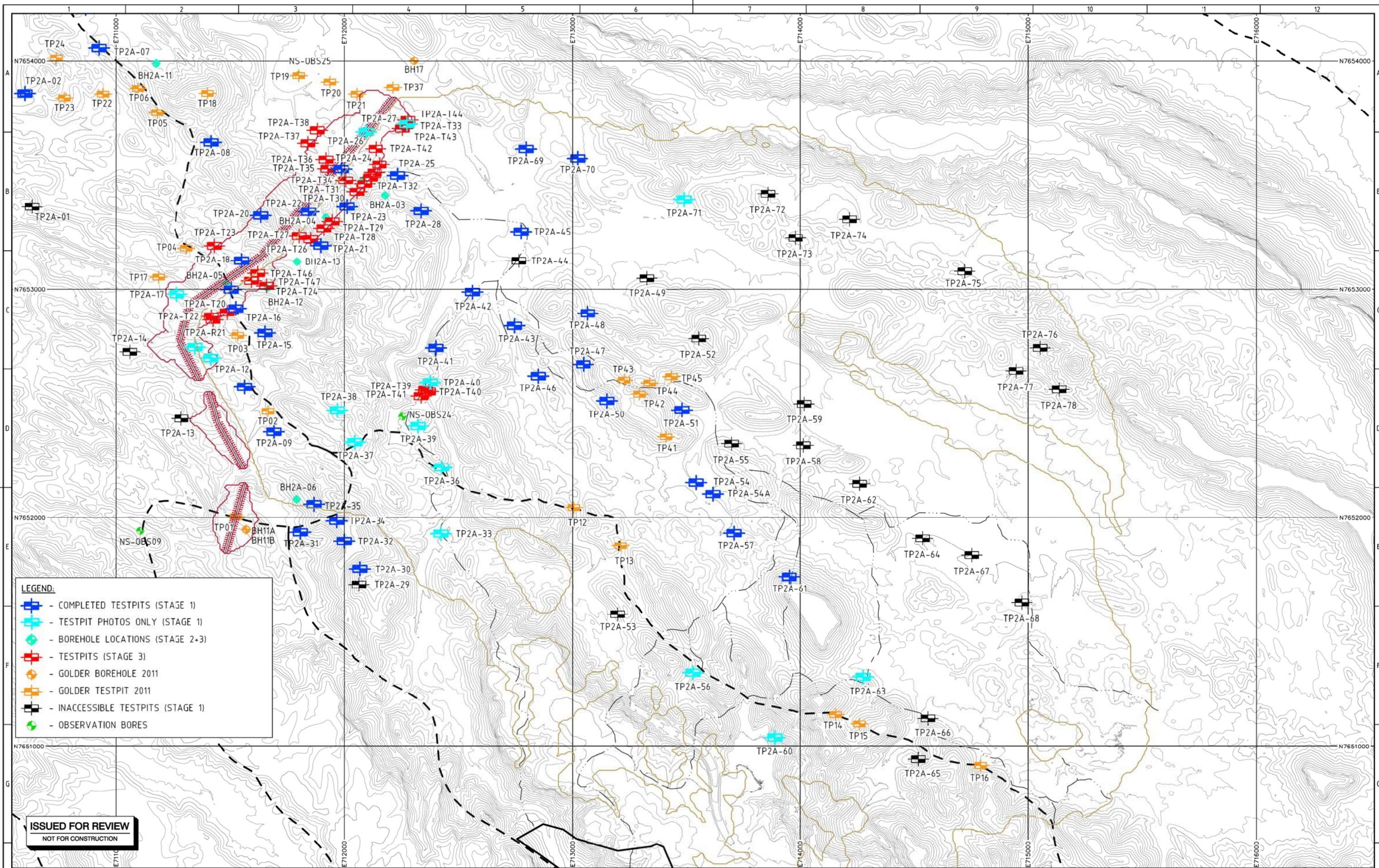


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**IBO**  
**NORTH STAR MAGNETITE STAGE 2**

**TAILINGS STORAGE FACILITY 2A**  
**RWP SITE LAYOUT AND TEST LOCATIONS**

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- LEGEND:**
- - COMPLETED TESTPITS (STAGE 1)
  - - TESTPIT PHOTOS ONLY (STAGE 1)
  - - BOREHOLE LOCATIONS (STAGE 2+3)
  - - TESTPITS (STAGE 3)
  - - GOLDER BOREHOLE 2011
  - - GOLDER TESTPIT 2011
  - - INACCESSIBLE TESTPITS (STAGE 1)
  - - OBSERVATION BORES

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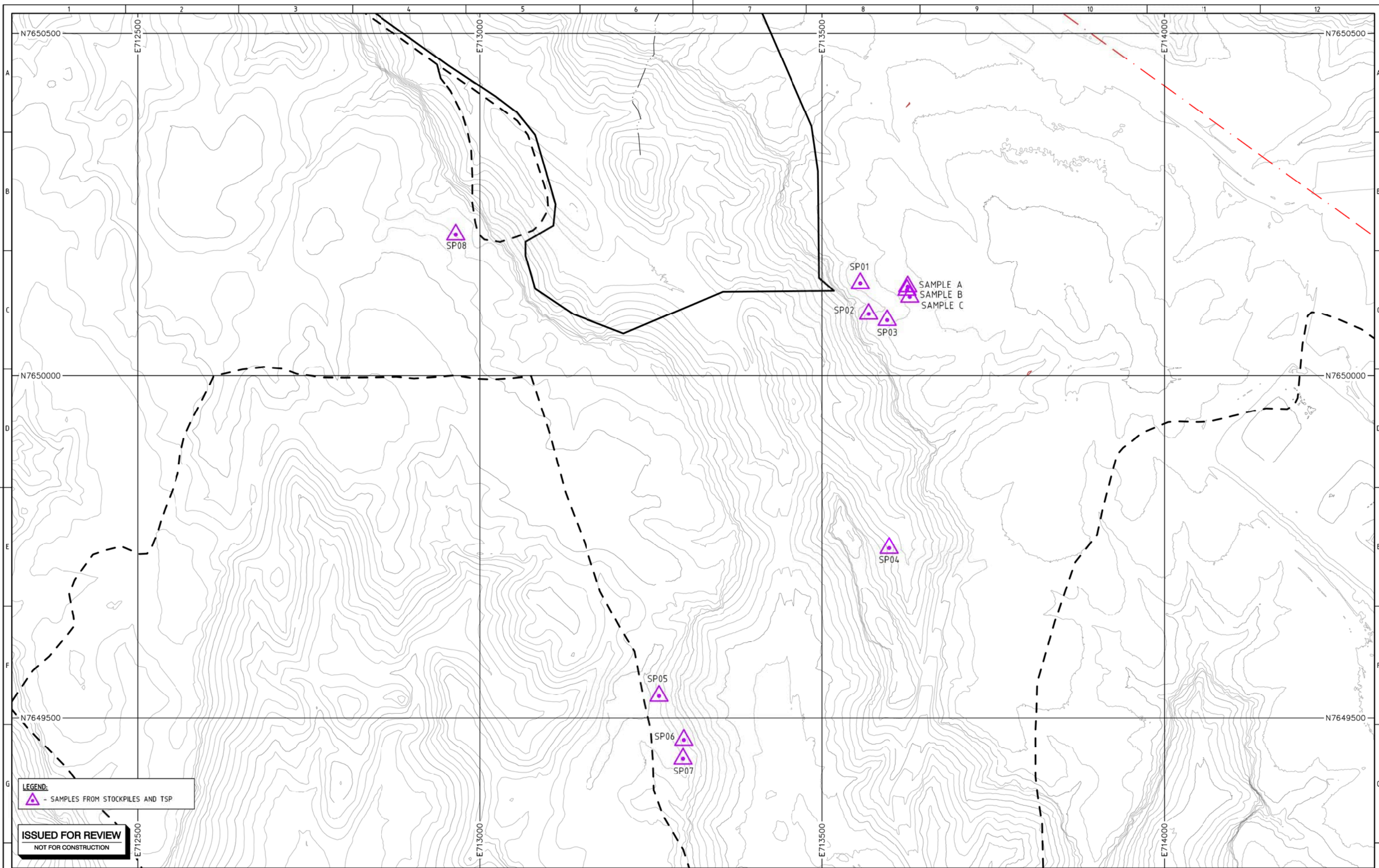
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**IBO**  
**NORTH STAR MAGNETITE STAGE 2**

**TAILINGS STORAGE FACILITY 2A**  
**TSF SITE LAYOUT AND TEST LOCATION**

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SHEET 1 OF 1	



**LEGEND:**  
 - SAMPLES FROM STOCKPILES AND TSP

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DATE	09/11/2010
DESIGN	AP
DRAWN	AH
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APPROVED	JL



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**IBO**  
**NORTH STAR MAGNETITE STAGE 2**

**TAILINGS STORAGE FACILITY 2A**  
**STAGE 1 STOCKPILES SITE LAYOUT AND TEST LOCATIONS**

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SHEET 1 OF 1		



## APPENDIX A

### General findings of the Geological Mapping Exercise January 2015

#### 1 TSF MAIN EMBANKMENT

Slightly weathered, fine and medium grained, pale grey / grey sandstone of high and very high strength, with occasional siltstone beds is predominant over the southern and central part of the main embankment footprint, with at least five units of pale grey and red brown banded shale up to 30 m thick also identified. Bed thickness in the shales is typically 10 mm to 40 mm. The sandstone is generally massive although discrete beds between 0.5 m and 2 m thick were locally observed.

The surface outcrops are blocky and closely jointed with a joint frequency (joints per m<sup>3</sup>) of approximately 6 to 10. Block edges are typically sub rounded as a result of physical weathering and the rock surfaces are discoloured orange brown (**Photograph 1**).

Dominant strike is WNW to ESE with dip between 60° and sub-vertical to the north. Local beds of fine to coarse grained lithic arenite (with fine to coarse gravel clasts) are present through the hills at the western abutment and the rock units (particularly the shales) are tightly folded in this area (**Photograph 2**).

Rock subcrops are visible across most of the embankment footprint, including the low lying gentle slopes, except at the northern side of the valley where deeper alluvial deposits are present. Soil cover in this area is fine and medium brown sand with occasional gravel and cobbles of both shale and sandstone.



PHOTOGRAPH 1 - TYPICAL SANDSTONE OUTCROP





**PHOTOGRAPH 2 - FOLDED SHALE AT TSF EMBANKMENT**

Slightly weathered, pale grey and red brown shale of high strength forms the lower part of the hill at the northern abutment and fine to coarse sandstone forms the central part.

BIF (Pincuna member) is exposed in the upper slopes of the valley and comprises slightly to moderately weathered, white grey and red brown shale with ferruginised, dark grey bands (**Photograph 3**). Bed thickness is typically 10 mm to 50 mm although local beds of siltstone / fine sandstone up to 2 m thick are present. The shales and BIF are frequently intensely folded. The perimeter ridge above the north abutment is capped by ferruginised laterite caprock of very high strength.



PHOTOGRAPH 3 - PINCUNA BIF

## 2 WEST EMBANKMENT

The predominant rock type in the West Embankment footprint is slightly weathered, fine to coarse, grey sandstone of high strength. One discrete bed of slightly weathered, pale grey and red brown shale of high strength was observed within the footprint, striking parallel to the embankment and dipping at approximately 70° to the south west (downstream side).

Local beds of lithic arenite, generally less than 5 m thick were also observed; however, west of the existing road junction, pale grey, fine sandstone / siltstone is predominant. Both materials are slightly weathered and of moderate to high strength.

The siltstone contacts brown, moderately weathered, intensely folded BIF approximately 200 m to the south of the junction. To the west, thinly bedded (<50 mm) pale grey shale with occasional red brown bands and infrequent sandstone beds was observed to a distance of approximately 400 m from the junction, where fine and medium sandstone of high strength was identified.

## 3 SOUTH DEPOSITION EMBANKMENTS

The proposed location of the southern embankments has moved north by approximately 400 m since the field mapping was completed; however, it is possible to broadly extrapolate the likely stratigraphy based on field observations.

The central ridges spanned by the embankments are anticipated to consist of fresh to slightly weathered, grey and brown shale of high strength. The shale is flanked to the west by alternating units of fine and medium, grey sandstone and fine to coarse lithic wacke / arenite, both slightly weathered and of high strength. (Photographs 4 and 5).



**PHOTOGRAPH 4 - LITHIC ARENITE**



**PHOTOGRAPH 5 - WACKE**

The rock units strike generally north to south, dipping at approximately 70° eastwards.

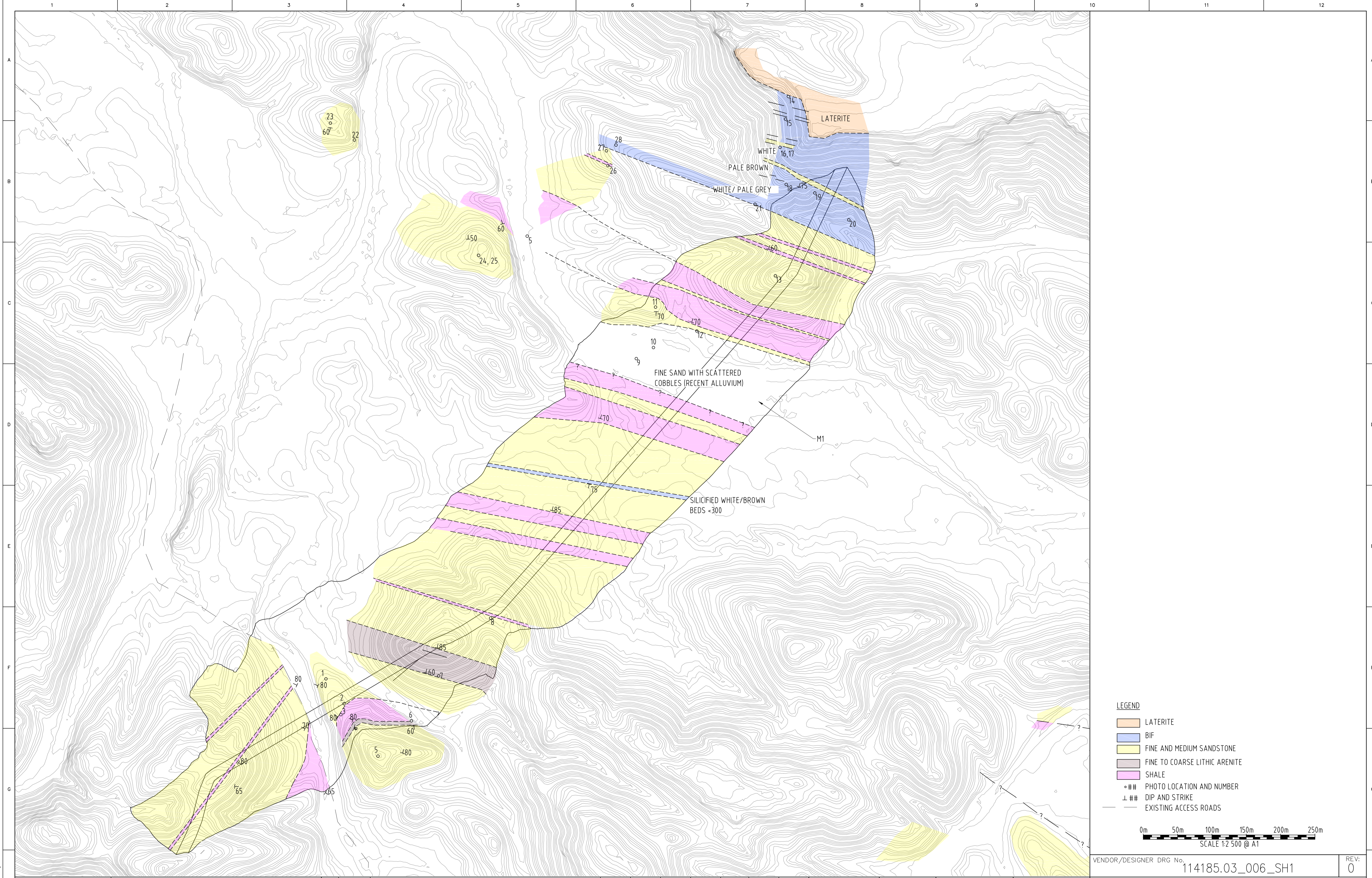
The central shale ridge is also flanked by similar sandstone and wacke units to the east, with shale possibly present below the creek alluvial deposits. At the eastern abutment, the sedimentary sequence of the Corboy formation unconformably overlies the silicified shale of the Cardinal formation which is of very high strength and dips at 60° to the north west (**Photograph 6**).



**PHOTOGRAPH 6 - SILICIFIED SHALE**

#### **4 NORTH EMBANKMENTS**

The ridge along which the north saddle embankments will be constructed is predominantly composed of slightly weathered, white and dark grey banded, silicified shale of high and very high strength. Localised beds of silicified siltstone / fine sandstone are present. The strata dip at approximately 60° to the north.



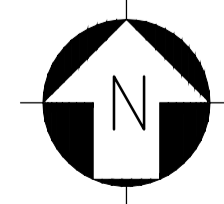
**LEGEND**

- LATERITE
- BIF
- FINE AND MEDIUM SANDSTONE
- FINE TO COARSE LITHIC ARENITE
- SHALE
- PHOTO LOCATION AND NUMBER
- DIP AND STRIKE
- EXISTING ACCESS ROADS

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	LAYOUT OF GEOTECHNICAL FINDINGS MAIN EMBK	A	16.02.2015	DRAFT ISSUE	PC	GB	BM	BM	GB		KDS



NORTH

PROJECT NUMBER: Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TAILINGS STORAGE FACILITY 2A  
 LAYOUT OF GEOTECHNICAL FINDINGS MAIN EMBK  
 SHEET 1 OF 5

SCALE: A1 DRG No: REV: 0

\$DATE\$ \$TIME\$ \$FILE\$



1. F-M Sandstone



2. Sandstone



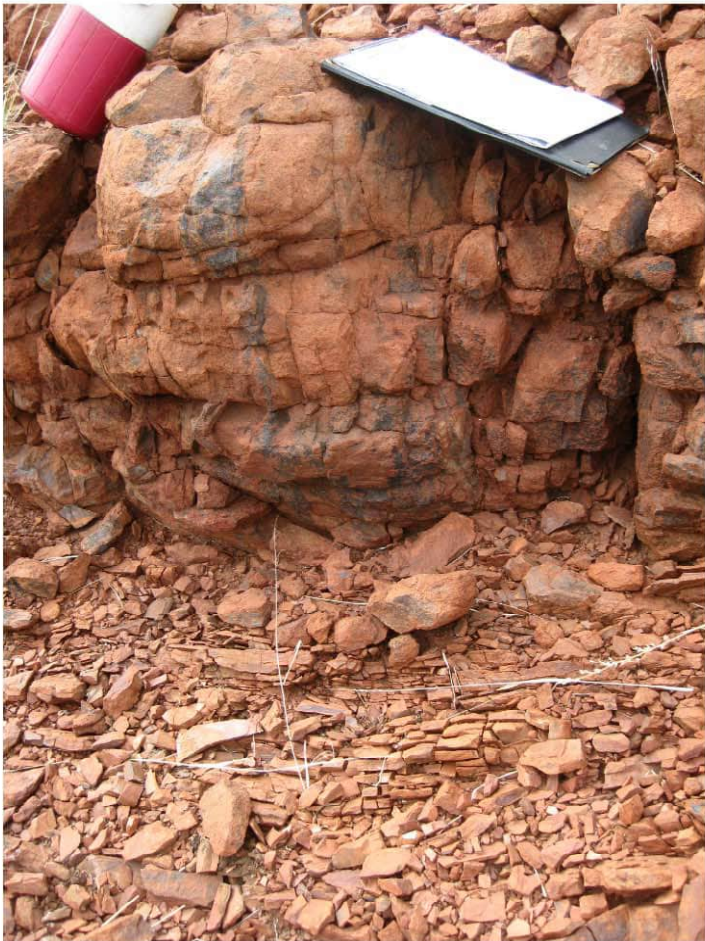
3. Folded Shale



4. Lithic Arenite



5. M3 Valley Crossing



6. Sandstone / Shale contact



7. Lithic Arenite / Conglomerate



8. Shale bed (2 m)



9. Creek bed



10. Main Creek bed



11. Sandstone



12. Sandstone / Shale contact



13. M2 abutment



14. Laterite caprock over BIF



15. BIF



16. BIF



17. Folded BIF



18. BIF outcrop



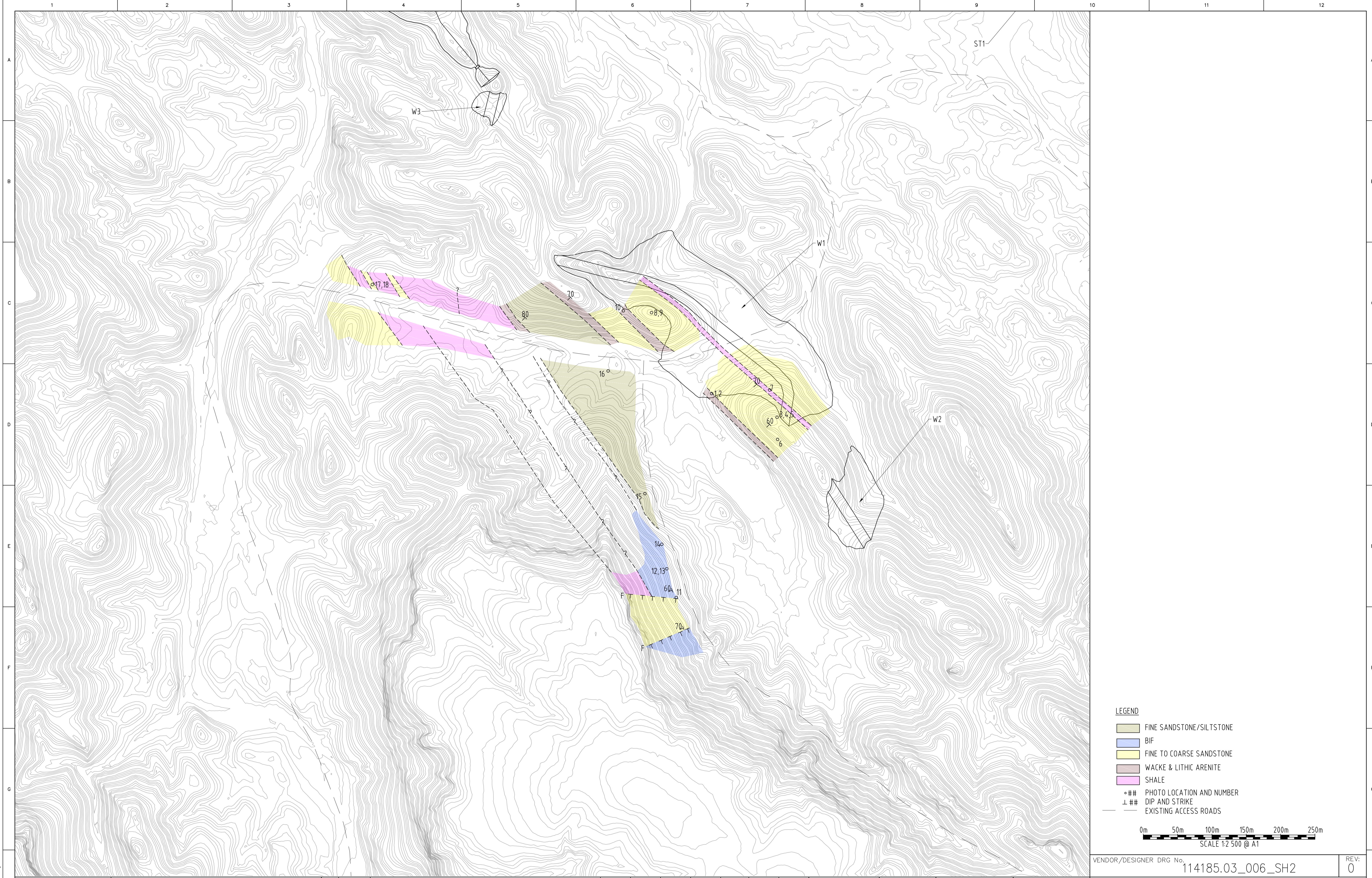
19. Iron rich Siltstone beds in BIF



20. Folded BIF

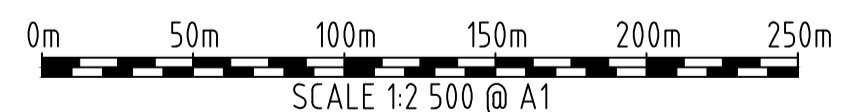


21. BIF / Sandstone contact



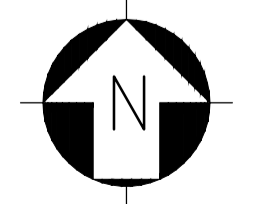
**LEGEND**

- FINE SANDSTONE/SILTSTONE
- BIF
- FINE TO COARSE SANDSTONE
- WACKE & LITHIC ARENITE
- SHALE
- ### PHOTO LOCATION AND NUMBER
- ⊥ ## DIP AND STRIKE
- EXISTING ACCESS ROADS



VENDOR/DESIGNER DRG No. 114185.03\_006\_SH2 REV: 0

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NORTH

PROJECT NUMBER Fortescue Metals Group Ltd  
 NORTH STAR MAGNETITE STAGE 2  
 TAILINGS STORAGE FACILITY 2A  
 LAYOUT OF GEOTECHNICAL FINDINGS WEST EMBK  
 SHEET 2 OF 5

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SCALE DRG No: A1 REV: 0



1. Sandstone



2. Sandstone: Onion skin weathering



3. Interbedded fine sandstone & siltstone



4. West Dam alignment to NW



5. Borrow pit & road embankment to south



6. 75/245 joint set in sandstone



7. Shale bed



8. West Dam alignment to SE



9. Creek to North



10. Altered Wacke



11. BIF with chert



13. BIF in base of cut



12. Folded BIF



14. BIF / Siltstone contact



15. Shale / siltstone in cut



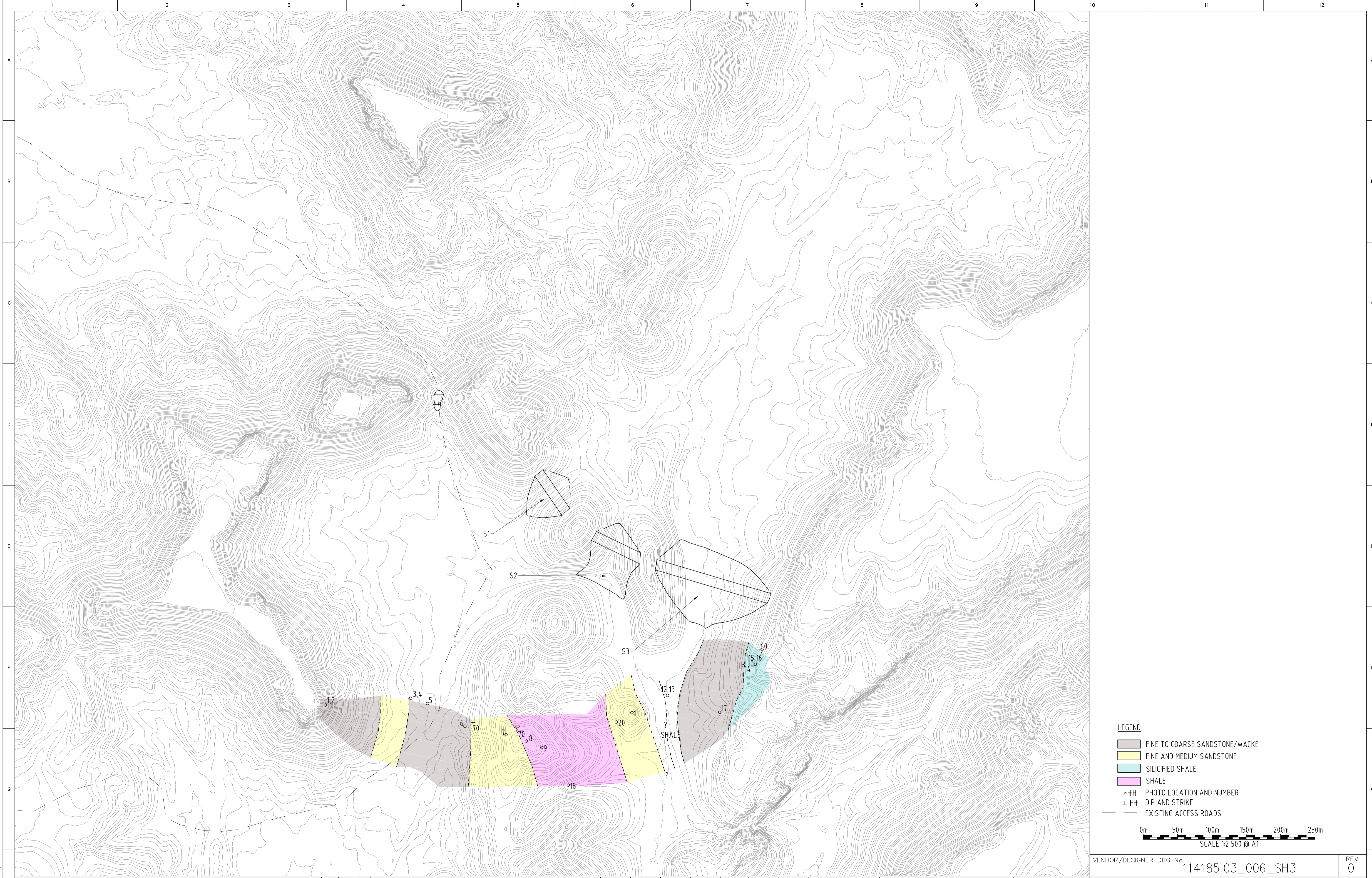
16. Cut shale / siltstone  
on South road side



17. Shale / Sandstone contact above creek



18. Cut shale on North road side



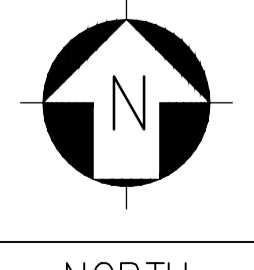
**LEGEND**

- FINE TO COARSE SANDSTONE/WACKE
- FINE AND MEDIUM SANDSTONE
- SILICIFIED SHALE
- SHALE
- ## PHOTO LOCATION AND NUMBER
- ┌ ## DIP AND STRIKE
- EXISTING ACCESS ROADS

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SCALE 1:2500 @ A1

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	LAYOUT OF GEOTECHNICAL FINDINGS SOUTH EMBK	A	16.02.2015	DRAFT ISSUE	PC	GB	BM	BM	GB	KDS	



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NORTH STAR MAGNETITE STAGE 2  
TAILINGS STORAGE FACILITY 2A  
LAYOUT OF GEOTECHNICAL FINDINGS SOUTH EMBK  
SHEET 3 OF 5

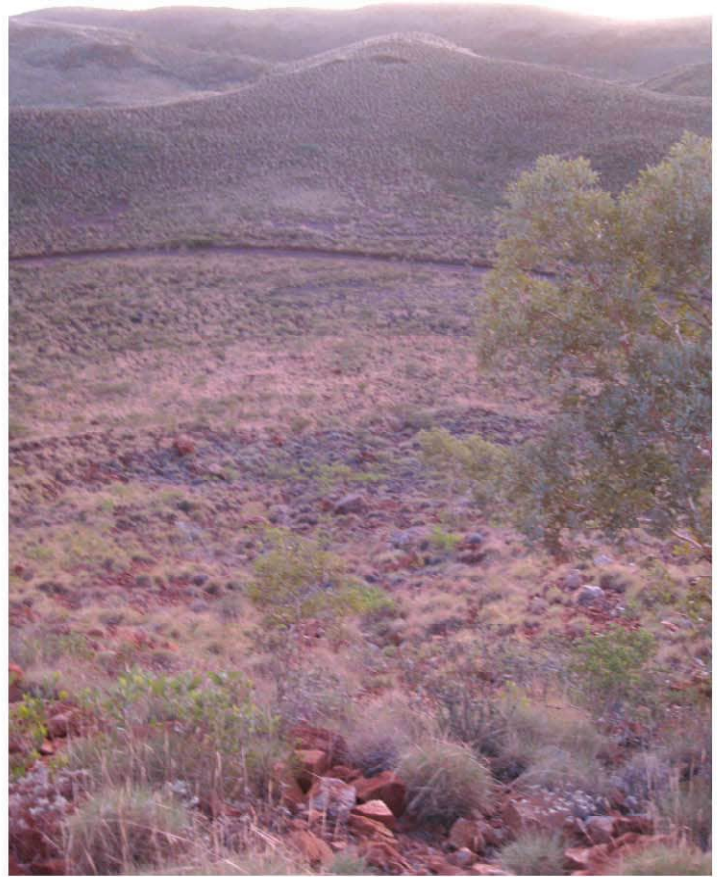
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A1 SCALE DRG No: REV:

DATE\$ TIME\$ FILE\$



1. Above S1 west abutment



2. S1 alignment to east



3. S1 west abutment



4. Wacke



5. Wacke



6. Bedded Sandstone



7. Fine and medium Sandstone



8. Shale



9. Shale



10. Fine Sandstone



11. Shale capped ridges to south



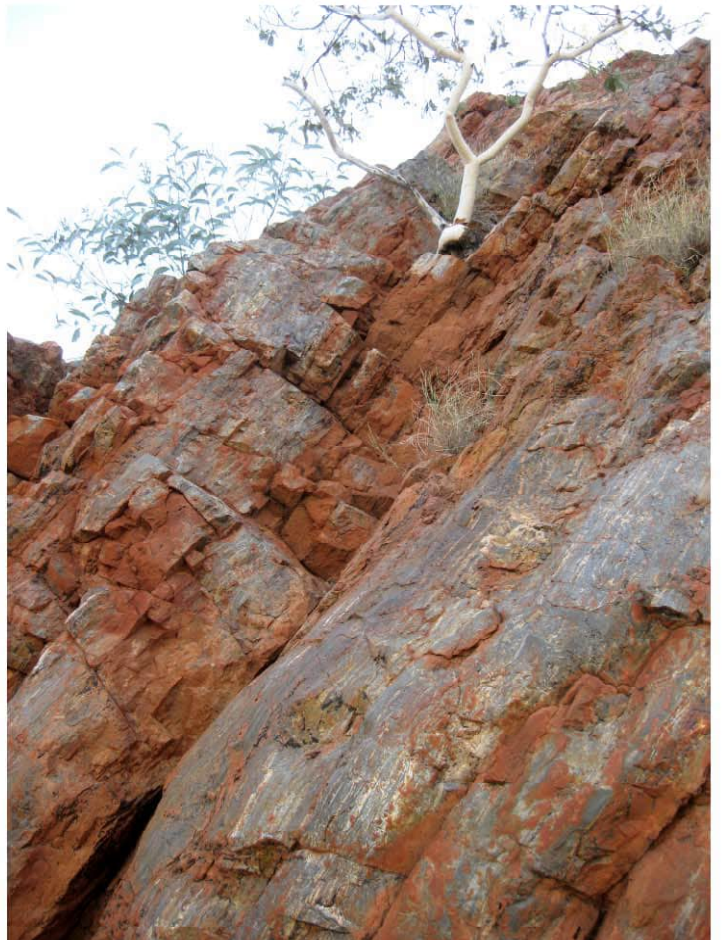
12. Creek bed south



13. Creek bed north



14. Silicified Shale



15. Silicified Shale



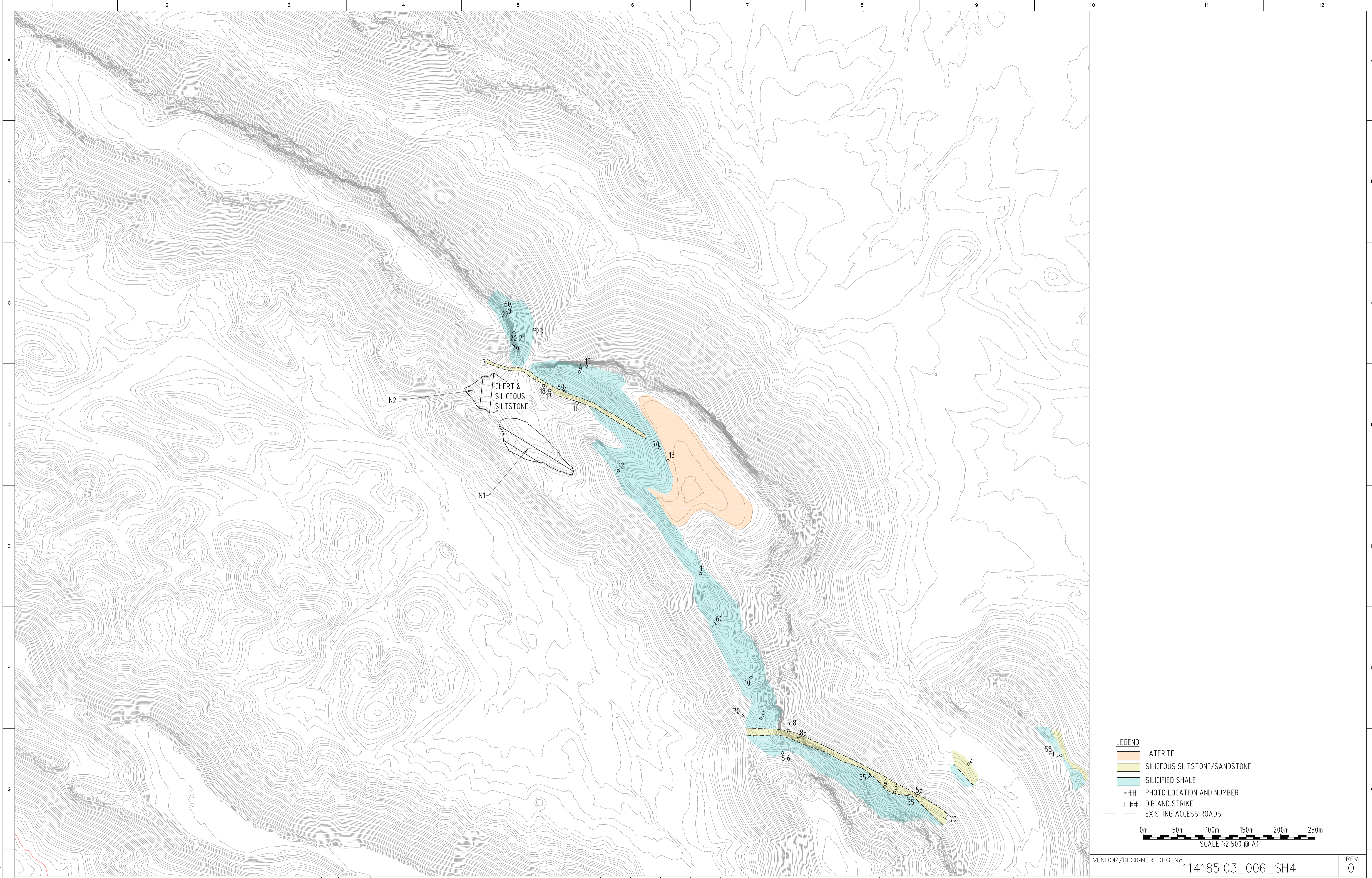
16. S2 alignment to west



17. Wacke



18. Shale waterfall



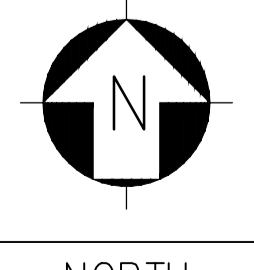
**LEGEND**

- LATERITE
- SILICEOUS SILTSTONE/SANDSTONE
- SILICIFIED SHALE
- ## PHOTO LOCATION AND NUMBER
- ┌ ## DIP AND STRIKE
- EXISTING ACCESS ROADS

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NORTH STAR MAGNETITE STAGE 2  
TAILINGS STORAGE FACILITY 2A  
LAYOUT OF GEOTECHNICAL FINDINGS NORTH EMBK  
SHEET 4 OF 5

SCALE: A1 DRG No.: 114185.03\_006\_SH4 REV: 0

DATE\$ FILE\$



1. Interbedded dark grey silicified shale and fine sandstone



2. Silicified fine sandstone / siltstone: onion skin weathering



3. Silicified shale



4. Silicified shale with fine sandstone / siltstone beds



5. Silicified Shale



6. Silicified shale & fine sandstone



7. N2 Dam alignment to North



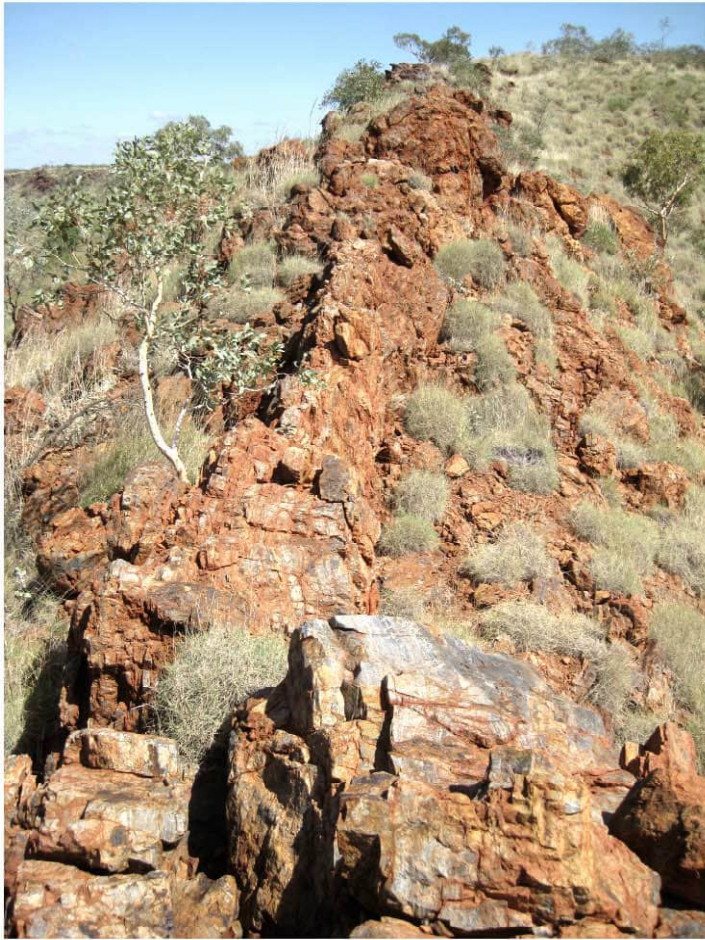
8. N2 Dam Alignment to north



9. Folded silicified shale



10. N2 Dam alignment to south



11. Ridge of silicified shale



12. View to N1 Dam site



13. Silicified shale



14. Chert & microfaulting in metamorphosed shale



15. N1 Dam alignment to north



16. Silicified siltstone



17. Silicified Wacke bed



18. Silicified Wacke



19. Microlaminae in silicified siltstone



20. N1 Dam alignment to south



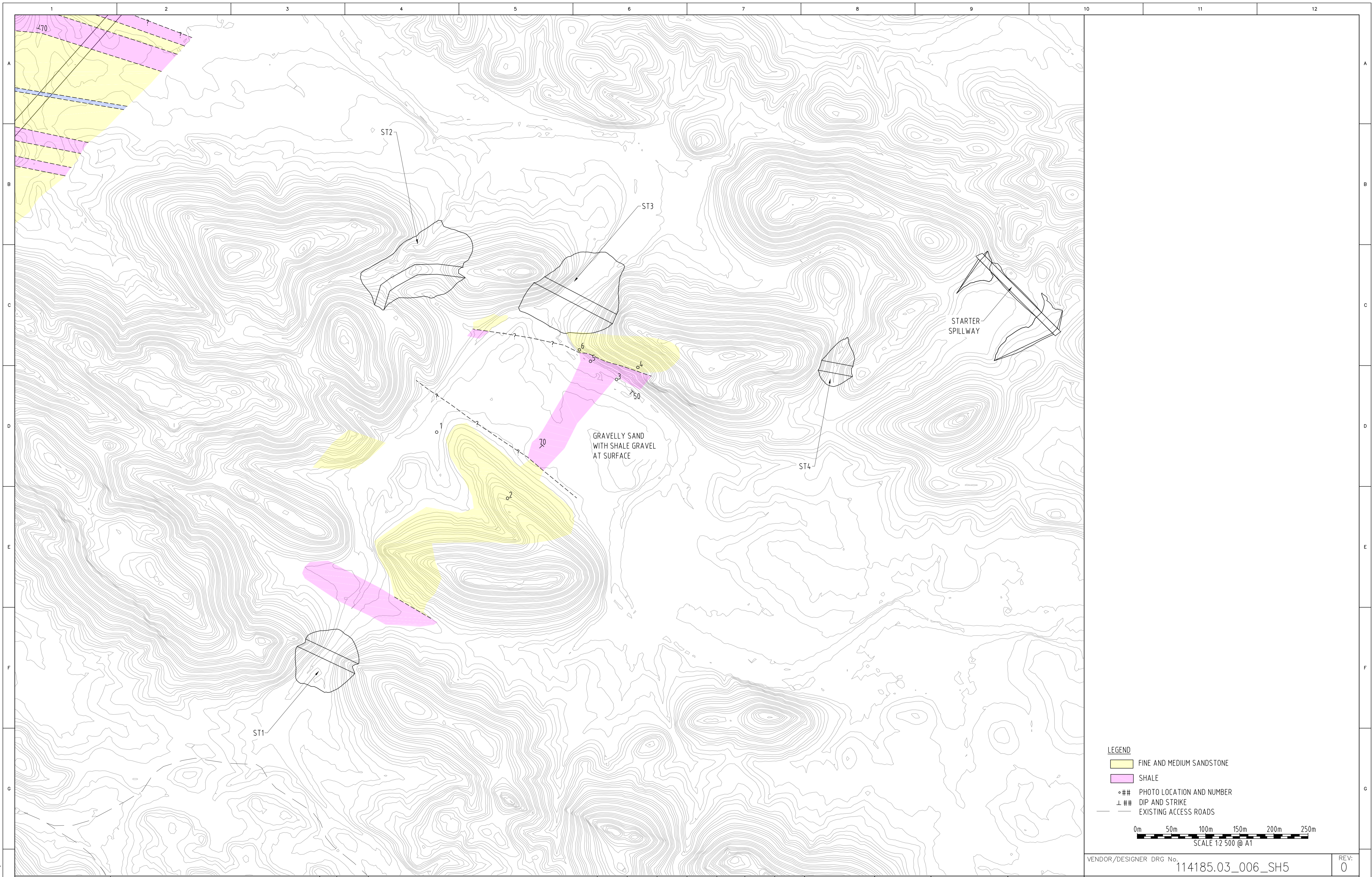
21. Silicified siltstone in cave



22. Cave



23. Silicified shale



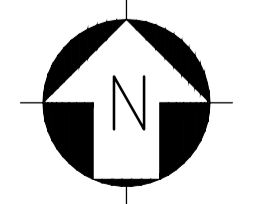
**LEGEND**

- FINE AND MEDIUM SANDSTONE
- SHALE
- ## PHOTO LOCATION AND NUMBER
- ⊥ ## DIP AND STRIKE
- — — EXISTING ACCESS ROADS

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PROJECT NUMBER Fortescue Metals Group Ltd

NORTH STAR MAGNETITE STAGE 2  
TAILINGS STORAGE FACILITY 2A  
LAYOUT OF GEOTECHNICAL FINDINGS STARTER EMBK  
SHEET 5 OF 5

SCALE DRG No: A1 REV: 0

DATE\$ FILE\$



1. Thin calcrete



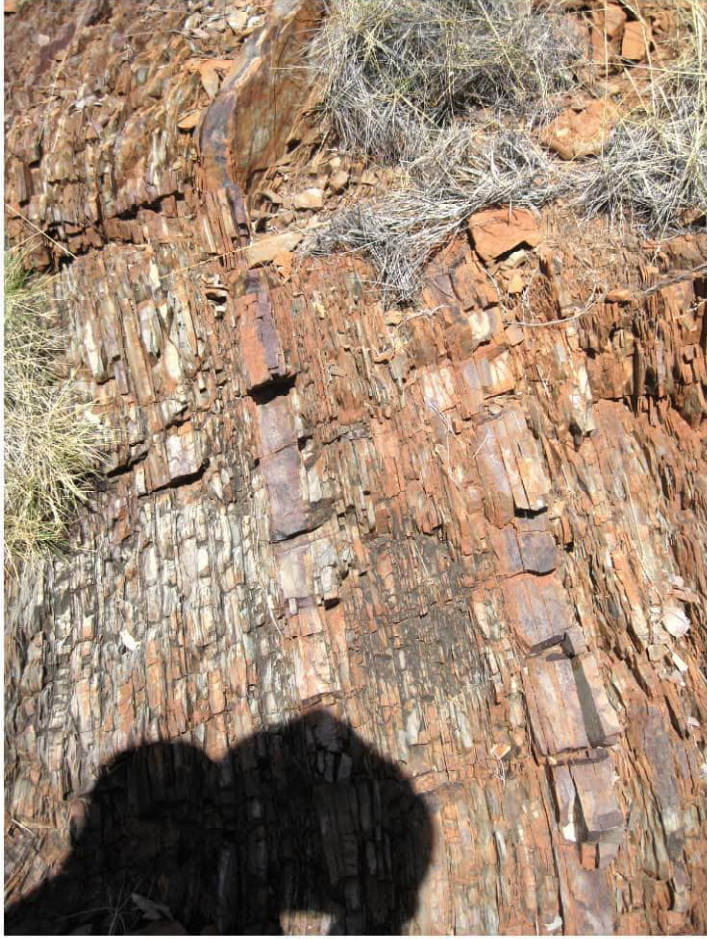
2. Starter Dam Alignment to North



3. Folded Shale in Creek bed



4. Starter Dam Alignment to South



5. Shale in Creek bed



6. Sandstone / Shale contact

## APPENDIX A

### General findings of the Geological Mapping Exercise July 2018

#### 1 RWP EMBANKMENTS

Slightly weathered, fine and medium grained, pale grey / grey sandstone of high and very high strength, with rare shale beds is predominant in the hills forming the left and central. Bed thickness in the shales is typically 10 mm to 40 mm. The sandstone is generally massive although discrete beds between 0.5 m and 2 m thick were locally observed.

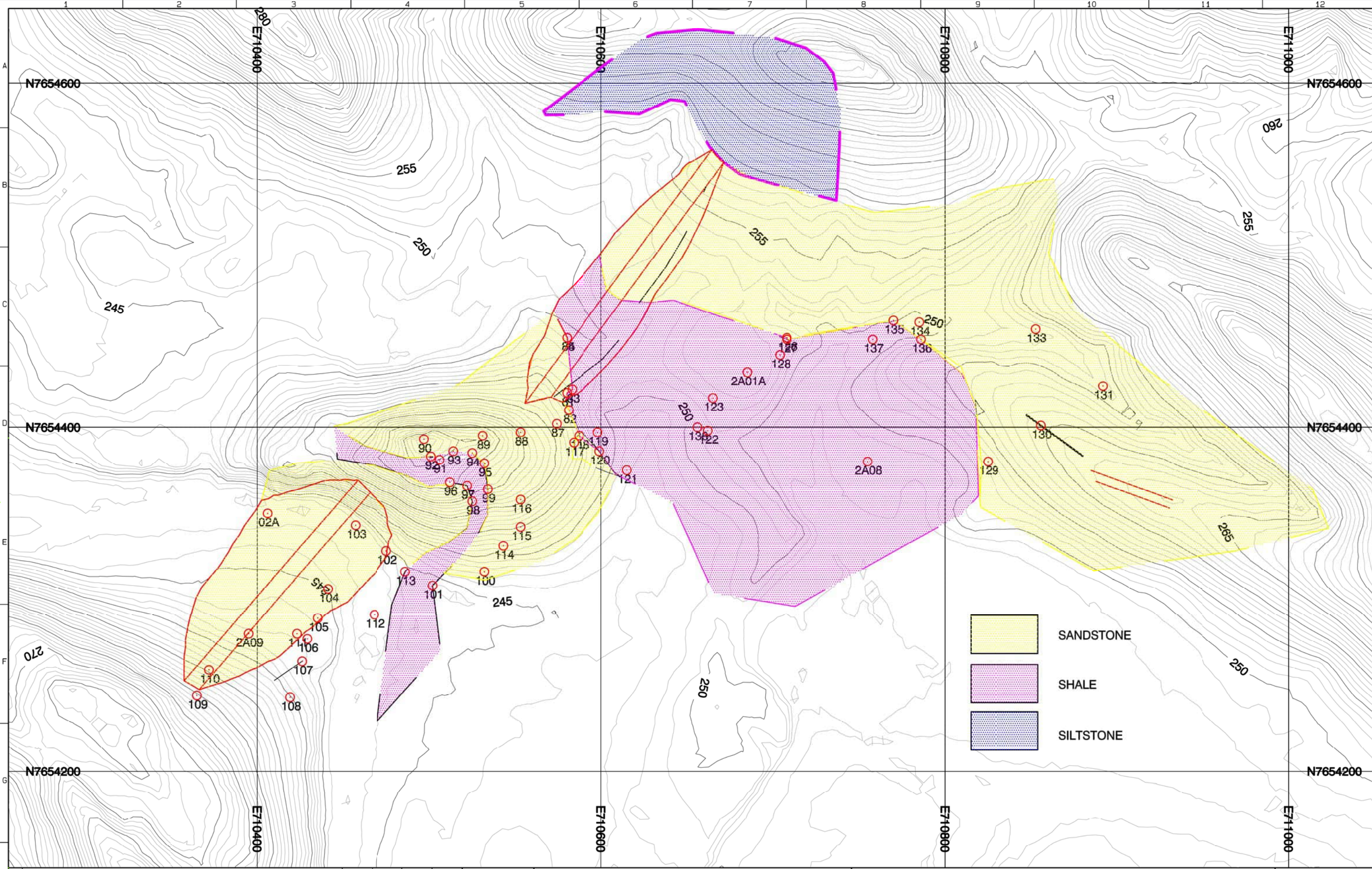
The surface outcrops are blocky and closely jointed with a joint frequency (joints per m<sup>3</sup>) of approximately 6 to 10. Block edges are typically sub rounded as a result of physical weathering and the rock surfaces are discoloured orange brown.

Dominant strike is WNW to ESE with dip between 65° and sub-vertical to the north.

Moderately weathered Shale is predominant in the central part of the north east RWP embankment and moderately to slightly weathered Siltstone with occasional shale interbeds forms the right abutment and spillway footprint area.

Rock subcrops are visible across most of the embankment footprint, including the low lying gentle slopes, except in the creek bed passing through the south west embankment where deeper alluvial deposits are present. Soil cover in this area is fine and medium brown sand with occasional gravel and cobbles of both shale and sandstone.





- SANDSTONE
- SHALE
- SILTSTONE

No.	DESCRIPTION	DATE	DRAWN	CHECKD	APPRD

SCALE 1:20000

JOB No. 114185.14

DATE 18/03/18

DESIGN JL

DRAWN

CHECKED

APPROVED

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**FMG**  
NORTH STAR STAGE 2

RWP EMBANKMENTS GEOLOGY

**FIGURE X**

CLIENT No. 114185

REV. No. A    SHT SIZE A3

SHEET 1 OF 1



81 Silcrete on sandstone surface



87 Sandstone o/c on Hill



87 F-C Grey Sandstone with brown oxidised surface



88 Sandstone o/c



89 Sandstone Bedding



90 Sandstone Bedding