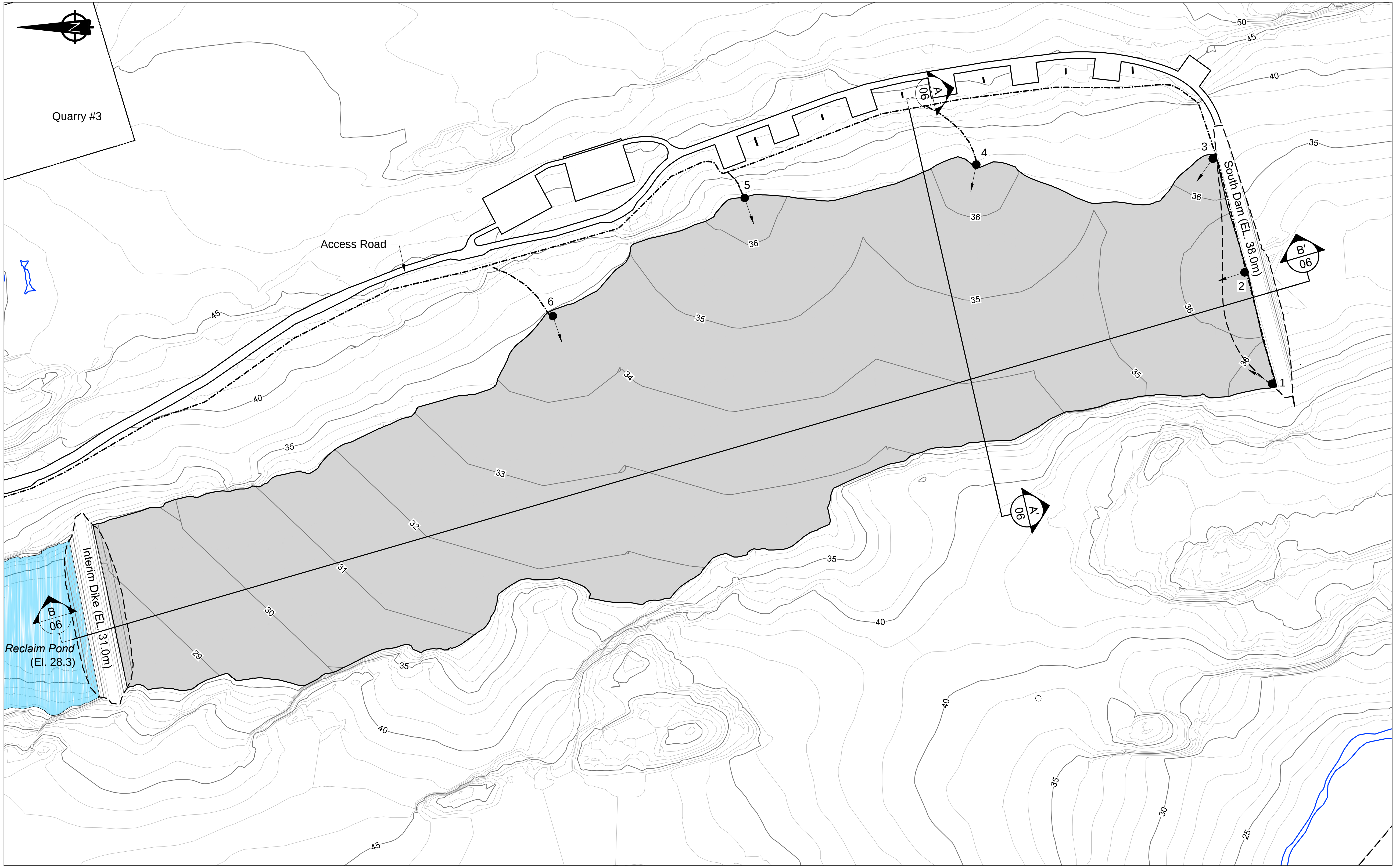

LEGEND

- Deposition Location
- Major Contour (5m)
- Minor Contour (1m)
- Approximate Tailings Line
- Proposed Tailings Deposition
- Proposed Dam / Dike

NOTES

1. Assumed an average deposited tailings beach slope of 1.0%.
2. A deposited tailings dry density of $1.29 t/m^3$ was used (based on laboratory testing).
3. Ice entrainment was assumed at 20% of production.
4. Dam and dike elevations shown were assumed constant for all deposition stages.
5. Total storage requirement is $2.32 Mm^3$ (tailings $1.93 Mm^3$ + ice entrainment $0.39 Mm^3$).

REFERENCE

NAD83 UTM Zone 13.

0m 50 100 150 200

SCENARIO 3

Max. Tailings Volume: $2.34 Mm^3$
Spigot Elevation: 36.5m (Locations 1-5), 35.0 (Location 6)
Tailings Surface Area: $0.44 km^2$
Approx. Interim Dike Volume: $42,000 m^2$
Approx. Tailings Line Length: 5,580m

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MAC RESOURCES

TAILINGS DEPOSITION PLAN

DEPOSITION SCENARIO 3

DORIS NORTH PROJECT

SRK JOB NO.: 1CT022.002.200.500	FILE NAME: 1CT022.002 - Scenario 3.dwg	DATE: 2015/02/19	APPROVED: TPP	FIGURE: 03

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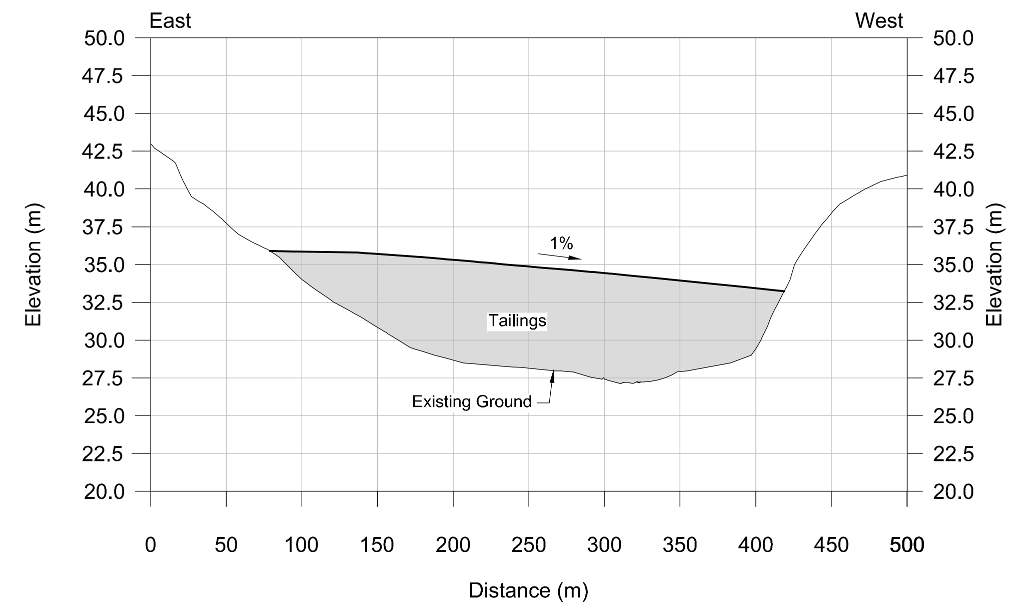
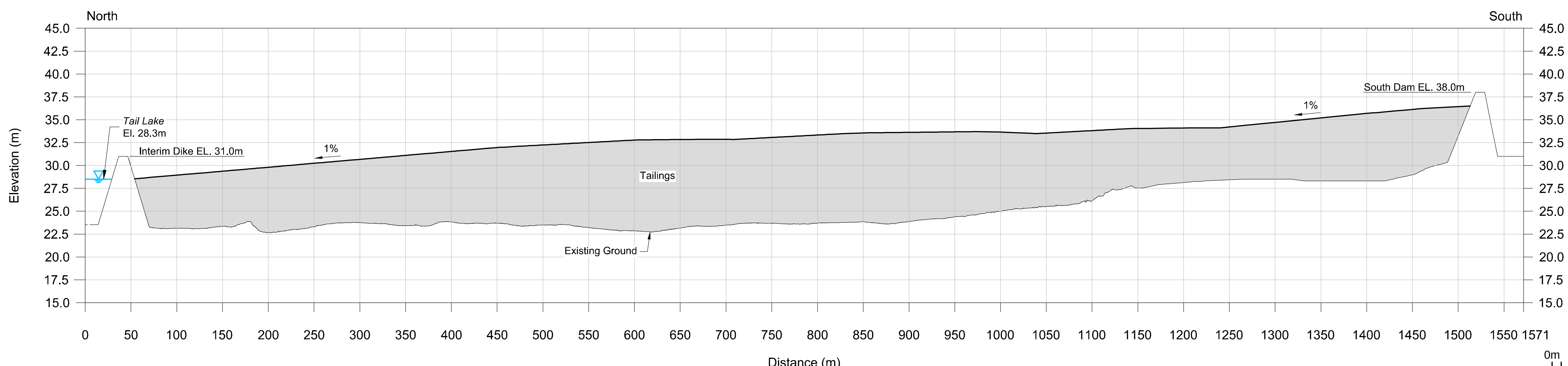
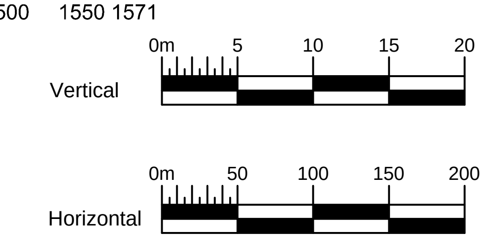


A
05
CROSS SECTION A-A'



B
05
CROSS SECTION B-B'



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SRK JOB NO.: 1CT022.002.200.500

FILE NAME: 1CT022.002 - Scenario 3.dwg

MAC RESOURCES

DORIS NORTH PROJECT

TAILINGS DEPOSITION PLAN

DEPOSITION SCENARIO 3
CROSS SECTIONS

DATE:	APPROVED:	FIGURE:
2015/02/19	TPP	04

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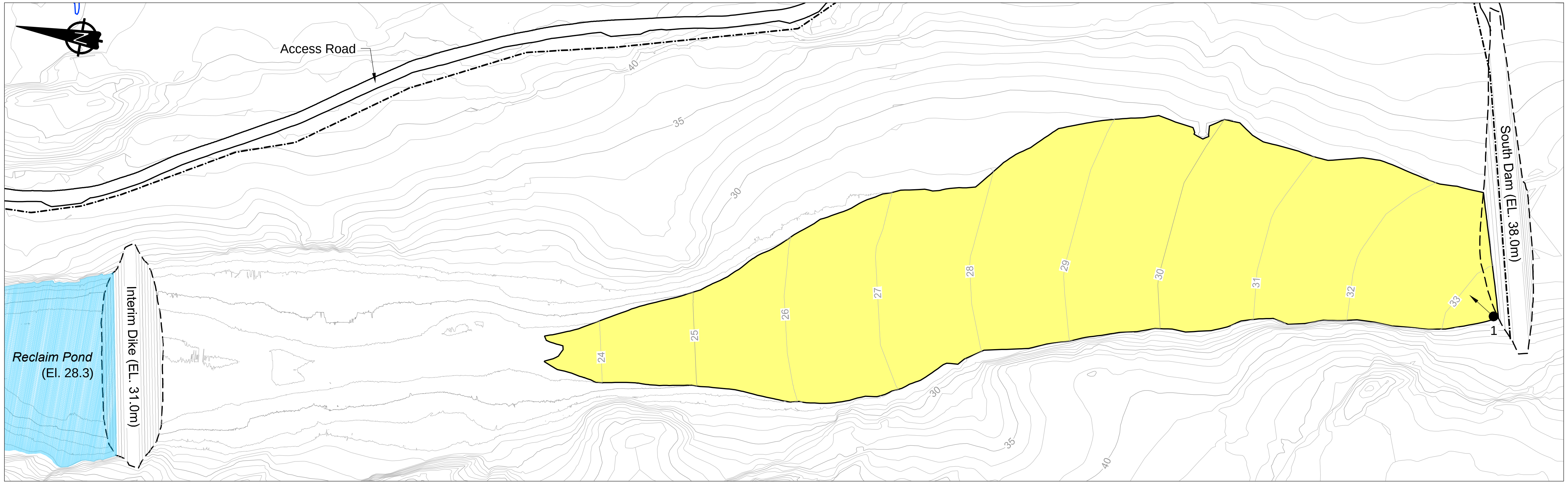
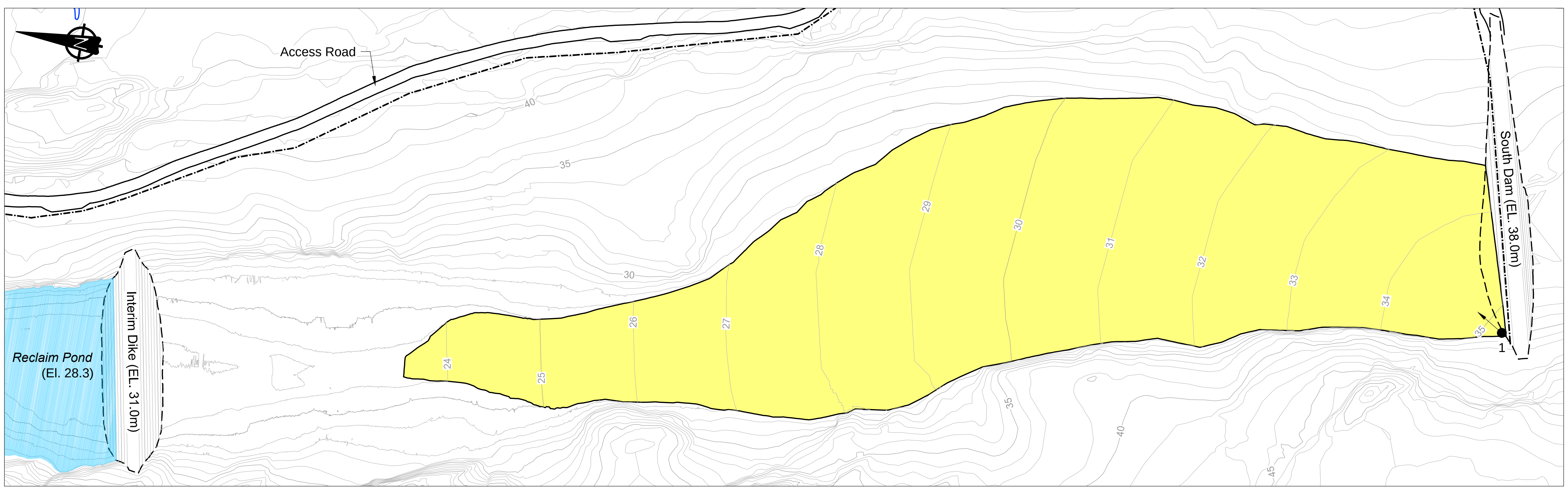


TAILINGS DEPOSITION - YEAR 1

Spigot Elev.: No.1: 33.5m
Deposited Tailings: 0.34Mm³
Duration: 1 Year
Production Rate: 773.4m³/day (1,000tpd)
Deposited Tailings Surface Area (cumulative): 0.17km²



TAILINGS DEPOSITION - YEAR 2

Spigot Elev.: No.1: 35.25m
Deposited Tailings (Cumulative): 0.68Mm³
Duration: 1 Year
Production Rate: 773.4m³/day (1,000tpd)
Deposited Tailings Surface Area (cumulative): 0.23km²

LEGEND

- Deposition Location
- Major Contour (5m)
- Minor Contour (1m)
- Approximate Tailings Line
- Current Deposition
- Proposed Dam / Dike

NOTES

1. Deposition durations are approximate and were based on an average production rate of 1,000tpd for years 1 and 2 and 2,000tpd for years 3 and 4.
2. Assumed an average deposited tailings beach slope of 1.0%.
3. A deposited tailings dry density of 1.29 t/m³ was used (based on laboratory testing).
4. All tailings volumes presented include ice entrainment, which was assumed at 20% of production.
5. Dam and dike elevations shown were assumed constant for throughout deposition.
6. Total storage requirement is 2.32Mm³ (tailings 1.93Mm³ + ice entrainment 0.39Mm³).

0m 50 100 150 200

srk consulting

MAC RESOURCES

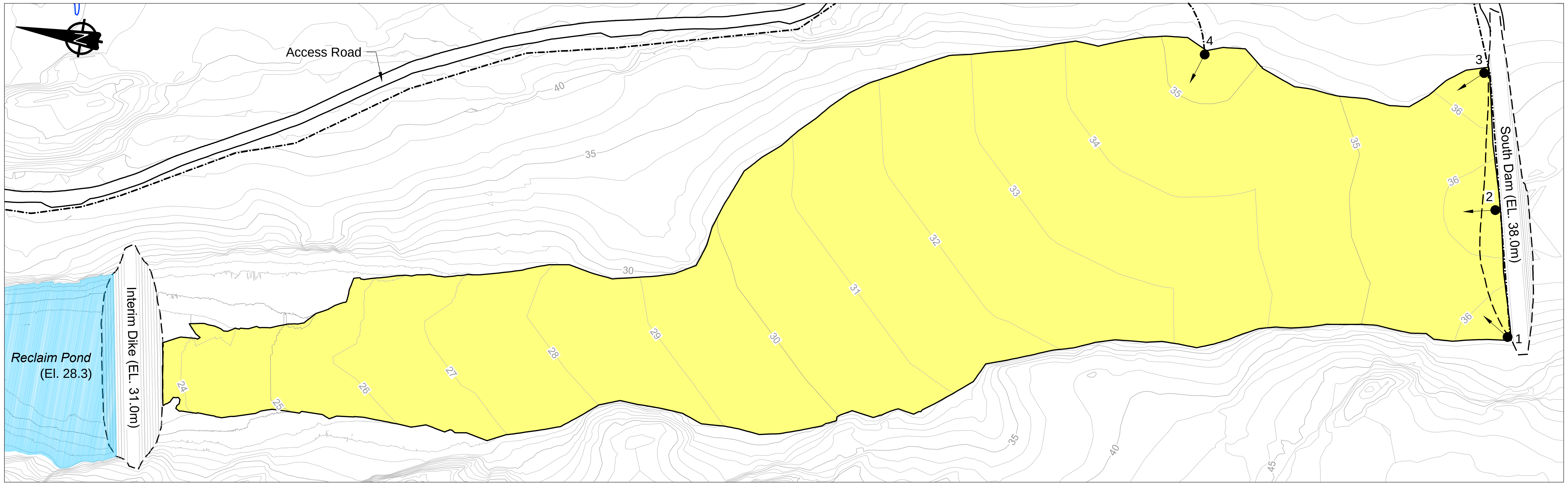
SRK JOB NO.: 1CT022.002
FILE NAME: 1CT022.002 - SC4 -staged.dwg

DORIS NORTH PROJECT

TAILINGS DEPOSITION PLAN

TAILINGS DEPOSITION PLAN (YEARS 1 & 2)

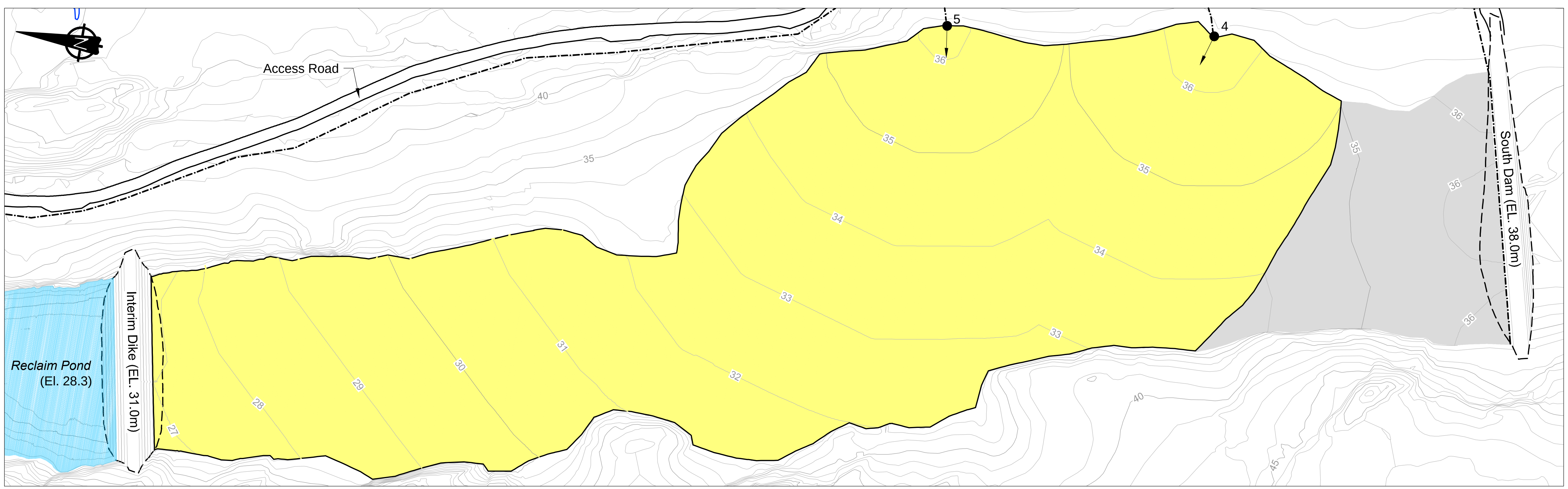
DATE:	APPROVED:	FIGURE:
2015/04/07	TPP	05



TAILINGS DEPOSITION - YEAR 3

Spigot Elev.: No.'s 1 to 3: 36.5m
No. 4: 35.5m
Deposited Tailings (Cumulative): 1.35Mm³
Duration: 1 Year

Production rate: 1,546.8m³/day (2,000tpd)
Deposited Tailings Surface Area (cumulative): 0.34km²



TAILINGS DEPOSITION - YEAR 4

Spigot Elev.: No. 4: 36.5m
No. 5: 36.25m
Deposited Tailings (Cumulative): 2.03Mm³
Duration: 1 Year

Production Rate: 1,546.8m³/day (2,000tpd)
Deposited Tailings Surface Area (cumulative): 0.36km²
Previous Tailings Surface Area: 0.06km²

LEGEND

- Deposition Location
- Major Contour (5m)
- Minor Contour (1m)
- Approximate Tailings Line
- Current Deposition
- Previous Deposition
- Proposed Dam / Dike

NOTES

1. Deposition durations are approximate and were based on an average production rate of 1,000tpd for years 1 and 2 and 2,000tpd for years 3 and 4.
2. Assumed an average deposited tailings beach slope of 1.0%.
3. A deposited tailings dry density of 1.29 t/m³ was used (based on laboratory testing).
4. All tailings volumes presented include ice entrainment, which was assumed at 20% of production.
5. Dam and dike elevations shown were assumed constant for throughout deposition.
6. Total storage requirement is 2.32Mm³ (tailings 1.93Mm³ + ice entrainment 0.39Mm³).

0m 50 100 150 200

srk consulting

MAC RESOURCES

TAILINGS DEPOSITION PLAN

TAILINGS DEPOSITION PLAN (YEARS 3 & 4)

SRK JOB NO.: 1CT022.002	DORIS NORTH PROJECT	DATE: 2015/04/07	APPROVED: TPP
FILE NAME: 1CT022.002 - SC4 -staged.dwg		FIGURE: 06	

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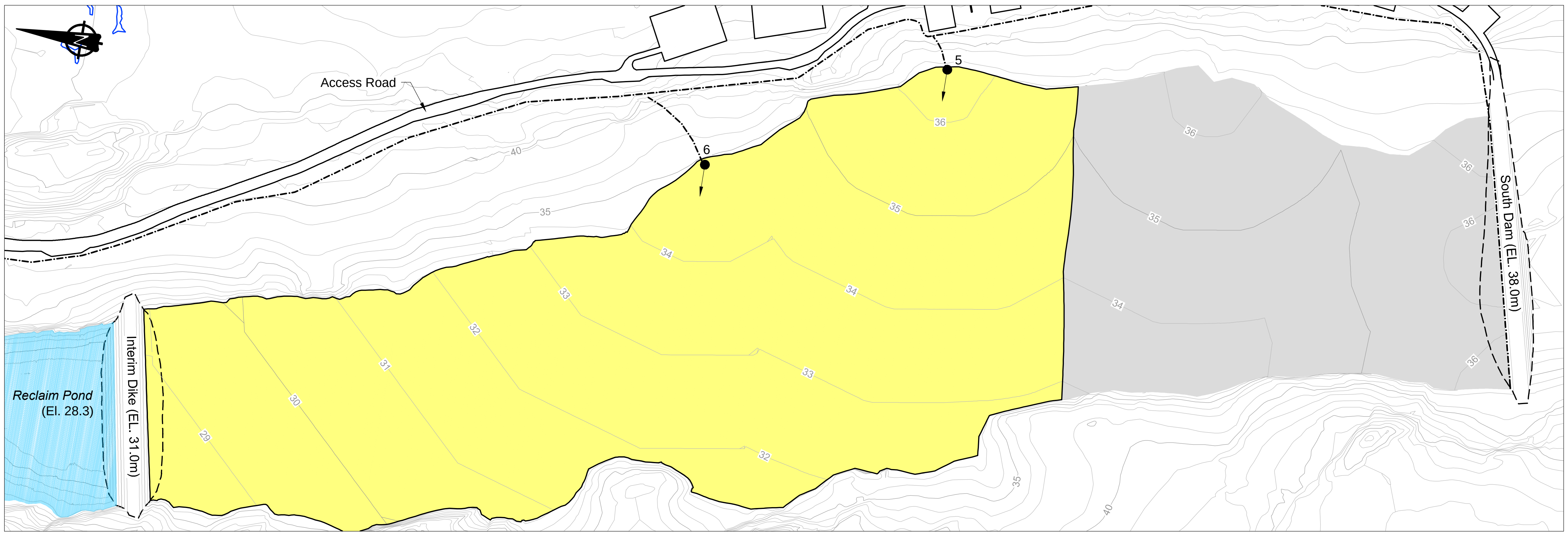
LEGEND

- Deposition Location
- Major Contour (5m)
- Minor Contour (1m)
- Approximate Tailings Line
- Current Deposition
- Previous Deposition
- Proposed Dam / Dike

NOTES

1. Deposition durations are approximate and were based on an average production rate of 1,000tpd for years 1 and 2 and 2,000tpd for years 3 and 4.
2. Assumed an average deposited tailings beach slope of 1.0%.
3. A deposited tailings dry density of 1.29 t/m³ was used (based on laboratory testing).
4. All tailings volumes presented include ice entrainment, which was assumed at 20% of production.
5. Dam and dike elevations shown were assumed constant for throughout deposition.
6. Total storage requirement is 2.32Mm³ (tailings 1.93Mm³ + ice entrainment 0.39Mm³).



TAILINGS DEPOSITION - END OF MINE (YEAR 4, MONTH 5)

Spigot Elev.: No. 5: 36.5m
No. 6: 35.0m
Deposited Tailings (Cumulative): 2.32Mm³
Duration: 5 Months

Production Rate: 1,546.8m³/day (2,000tpd)
Deposited Tailings Surface Area (cumulative): 0.30km²
Previous Tailings Surface Area (cumulative): 0.14km²

0m 50 100 150 200

srk consulting

SRK JOB NO.: 1CT022.002
FILE NAME: 1CT022.002 - SC4 -staged.dwg

MAC RESOURCES

DORIS NORTH PROJECT

TAILINGS DEPOSITION PLAN

TAILINGS DEPOSITION PLAN
(COMPLETE AT YEAR 4 + 5 MONTHS)

DATE: 2015/04/07
APPROVED: TPP
FIGURE: 07

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Appendix D – Interim Dike Wave Run-up, Freeboard and Riprap Hydro-Technical Assessment

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Memo

To:	Project File	**Client:**	TMAC Resources Ltd
From:	Murray McGregor, EIT	**Project No:**	1CT022.002.200.530
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	May 29, 2015
Subject:	Doris North Project: Interim Dike Wave Run-up, Freeboard and Riprap Hydrotechnical Assessment		

1 Introduction

TMAC Resources Ltd. (TMAC) plan to revise their tailings management plan to accommodate a greater volume of tailings at the Doris North Project (Project), located in Nunavut, approximately 160 km southwest of Cambridge Bay. The revised volume of tailings exceeds the amount that can sub-aqueously be deposited in the Tailings Impoundment Area (TIA) with a permanent water cover after the North Dam gets breached.

The tailings management system has subsequently been redesigned to incorporate a sub-aerial deposition strategy starting at the south end of the TIA. The approximately 2.5 Mt of tailings will be deposited along the southern end of the TIA and will be contained by a new Interim Dike about 1,500 m north of the South Dam. The remaining portion of the TIA between the Interim Dike and the existing North Dam will not contain any tailings, and will act as a Reclaim Pond. Tailings will be spigotted from a number of points along the eastern perimeter of the TIA and from the South Dam creating a landscape that drains towards the Interim Dike at an average slope of about 1%.

Upon closure, the tailings surface will be covered with a nominal waste rock cover 0.3 m thick. The function of the cover is to prevent dust and to minimize direct contact by terrestrial animals. The cover will terminate at the Interim Dike, and the Interim Dike will be levelled to match the elevation of the cover. Once the water quality in the Reclaim Pond has reached the required discharge criteria, the North Dam will be breached as originally intended. Should there be any exposed shoreline erosion these areas will be covered as described in the original Project closure plan.

This memo summarizes the wave run-up and freeboard calculations completed for the Interim Dike, as well as the riprap requirements.

2 Wave Run-up Freeboard

2.1 Definition

Wave run-up freeboard is the sum of the wind setup and the wave run-up. The following subsections describe how these values have been calculated for the Interim Dike.

2.2 Wind Setup

2.2.1 Calculation Procedure

The wind set-up is defined as the vertical water height above the static water level which may result from wind stress over the water surface. The US Army Corps of Engineers (USACE 1989) estimates the wind setup relative to the supply water level (SWL) using the following expression:

$$S = \frac{U^2 F}{1400d} \qquad \text{(Eq. 1)}$$

Where:

S is the wind setup relative to the SWL (ft);
U is the wind speed (mph);
F is the effective fetch length (miles); and,
d is the average water depth over the fetch (ft).

2.2.2 Wind Speed

Estimation of wind speed and frequency duration has been performed for the initial run-up calculation completed in SRK (2005). The estimated hourly wind speeds at Cambridge Bay for various return periods produced from the 2005 report are presented in Table 1 below.

Table 1: Estimated Annual Maximum Hourly Wind Speeds (km/h) at Cambridge Bay for Various Return Periods

Return Period (years)	Frequency Distribution Used to Predict Extreme Hourly Wind Speeds		
	Generalized Extreme Value	3-Parameter Lognormal	Log-Pearson Type III
2	75	75	74
5	84	83	83
10	89	89	89
20	95	95	96
50	102	104	105
100	107	111	112
200	112	118	120
500	118	128	131

Hourly wind speeds were selected using the Log-Pearson Type III method for 1:500 year return period providing wind speeds of 131 km/hr or 36.4 m/s. Although site specific wind speed data exist, the data record is too short to evaluate long-term statistical trends required for the hydro-technical analysis presented herein.

2.2.3 Fetch Length

The fetch length for the Interim Dike is assumed to be equal to the maximum fetch length between the North Dam and the Interim Dike. This has been estimated at 2,300 m.

2.2.4 Average Water Depth over Fetch

The average depth over the fetch was estimated using topography and bathymetry provided by TMAC. Using 25 m grid, the average was calculated from the data presented in Figure 1.

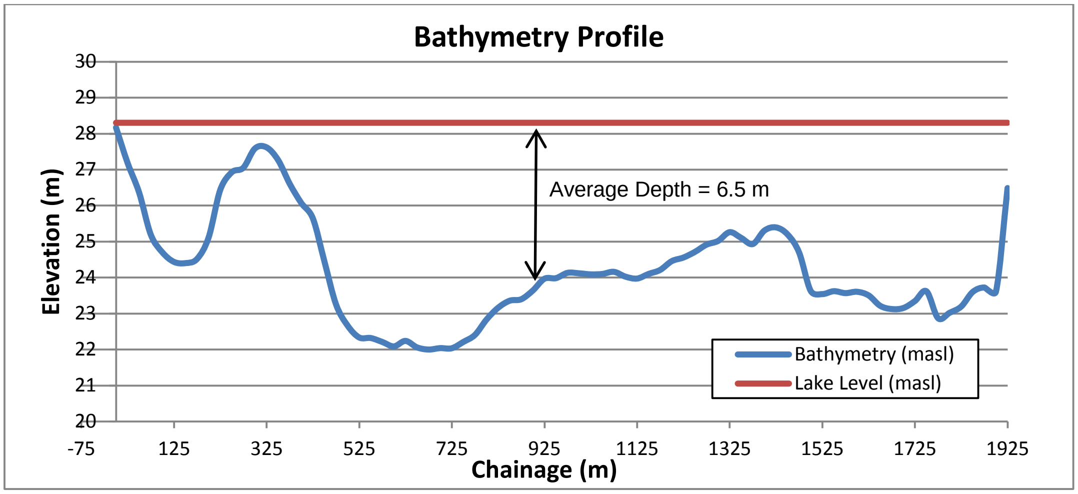


Figure 1: Average Water Depth along the Maximum Fetch

2.2.5 Design Wind Setup

Sensitivity was performed by varying the depth of water between the natural lake level (28.3 m) and the TIA (and North Dam) Full Supply Level (33.5 m). The resultant wind setup changes between 0.07 m and 0.16 m. The design wave setup has been selected to be the maximum value of 0.16 m

2.3 Wave Run-up

2.3.1 Calculation Procedure

Wave run-up is defined as the maximum vertical extent of a wave uprush on a beach or structure. The estimation of the wave run-up requires the estimation of the wave height. This analysis will use the method presented by the South African National Committee on Large Dams (SANCOLD 1990).

2.3.2 Wave Height Estimation

SANCOLD (1990) further suggests the estimation of significant wave height using a relationship between effective fetch length and over-water wind speed, as shown in Figure 2.

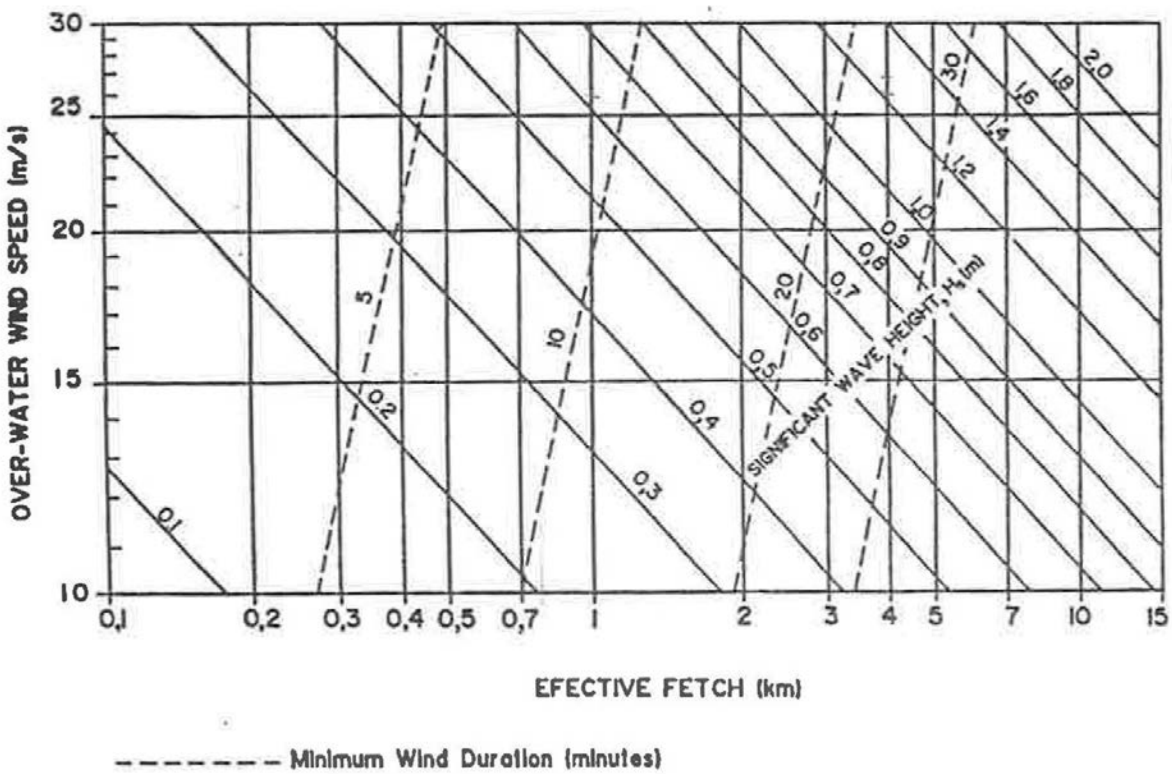
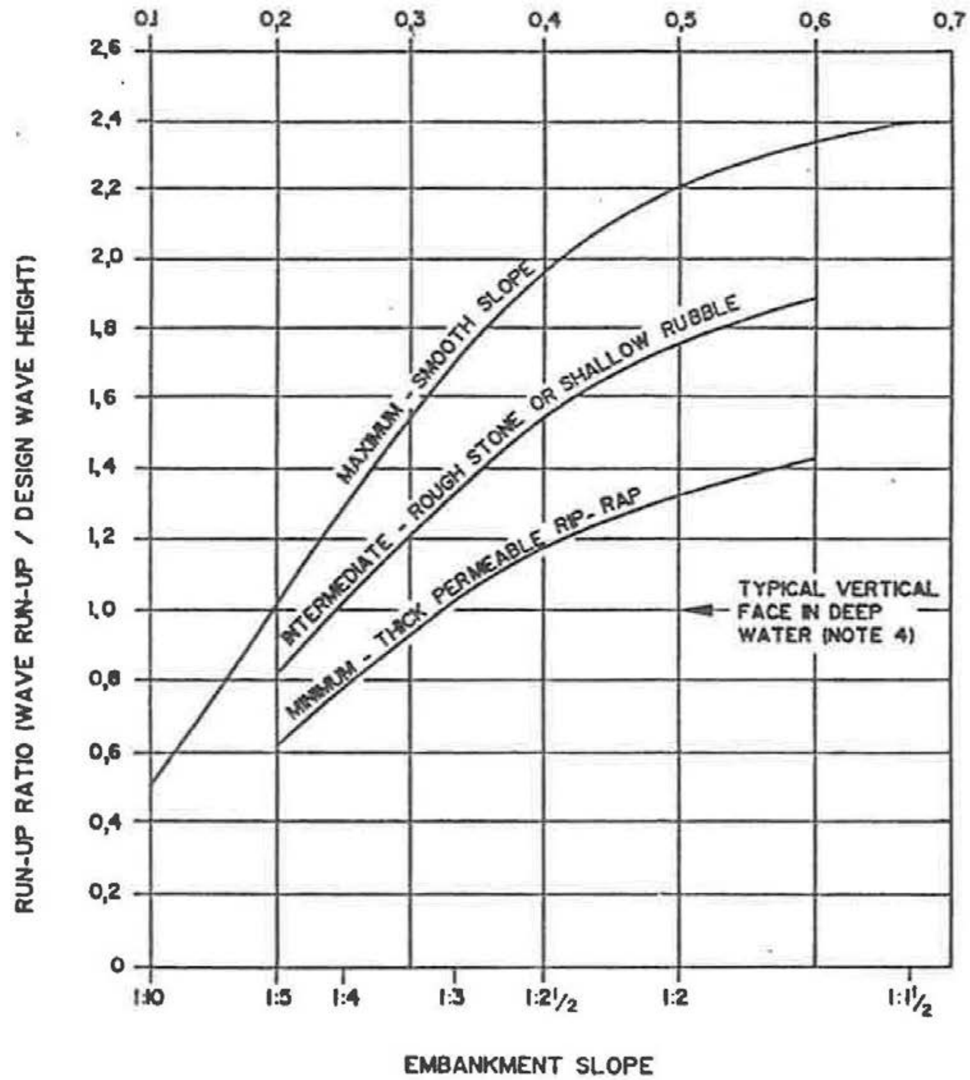


Figure 2: Significant wave height from SANCOLD (1990)

Based on the wind speed and fetch distance presented in Section 2.1, the significant wave height as interpolated from Figure 2, is approximately 1.2 m (note that this estimate does not actually plot on Figure 2, but was extrapolated).

2.3.3 Wave Run-up Estimation

SANCOLD (1990) presents a relationship between the wave height, the embankment slope and the embankment protection. The wind run-up ratio is obtained from Figure 3.



NOTES:

1. MAXIMUM LINE FROM SAVILLE ET AL (1962) FOR TYPICAL WAVE STEEPNESS (SIGNIFICANT WAVE HEIGHT/LENGTH) = 0,05.
2. INTERMEDIATE LINE IS 0,8 x MAXIMUM.
3. MINIMUM LINE IS 0,6 x MAXIMUM.
4. FOR FACES OFF-VERTICAL THE RUN-UP RATIO RISES ABOVE UNITY AND CAN APPROACH 2 IN SOME CIRCUMSTANCES WHERE THE DEEP WATER CONDITION IS NOT FULFILLED.

Figure 3: Wave run-up estimation from SANCOLD (1990)

The maximum wave run-up was calculated to be 1.56 m based on the SANCOLD (1990) method with significant wave height of 1.2 m and run-up ratio of 1.3.

2.4 Freeboard Design

The design freeboard is the sum of the wind setup (0.16 m), and the wave run-up (1.56 m), which equates to 1.72 m.

3 Riprap

The Interim Dike will be constructed from run-of-quarry rock and therefore no specific riprap is deemed necessary.

4 References

South African National Committee on Large Dams (SANCOLD), 1990. Report N°3: Interim guidelines on Freeboard for Dams.

SRK Consulting (Canada) Inc. 2005. Wave Run-up Calculation to Determine Hydraulic Freeboard for Doris North Project Tailings Dam. Project 1CM014.006. September 6, 2005.

US Army Corps of Engineers (USACE), 1989. EM 1110-2-1414 Engineering and Design – Water Levels and Wave Heights for Coastal Engineering Design.

Appendix E – South Dam and Interim Dike Stability Assessment

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Memo

To:	Project File	**Client:**	TMAC Resources Ltd.
From:	Peter Luedke, EIT Murray McGregor, EIT	**Project No:**	1CT022.002.510
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	May 29, 2015
Subject:	Doris North Project: South Dam and Interim Dike Stability Assessment		

1 Introduction

1.1 Tailings Management Concept

TMAC Resources Ltd. (TMAC) plan to revise their tailings management system to accommodate a greater volume of tailings at the Doris North Project, located in Nunavut, approximately 160 km southwest of Cambridge Bay. The revised volume of tailings exceeds the amount that can sub-aqueously be deposited in the Tailings Impoundment Area (TIA) with a permanent water cover after the North Dam gets breached.

The tailings management plan has subsequently been redesigned to incorporate a sub-aerial deposition strategy starting at the south end of the TIA. Approximately 2.5 Mt of tailings will be deposited along the southern end of the TIA and will be contained by a new Interim Dike about 1,500 m north of the South Dam. The remaining portion of the TIA between the Interim Dike and the existing North Dam will not contain any tailings, and will act as a Reclaim Pond. Tailings will be spigotted from a number of points along the eastern perimeter of the TIA and from the South Dam creating a landscape that drains towards the Interim Dike at an average slope of about 1%. Figure 1 provides a general layout of the TIA.

The South Dam was originally designed as a frozen core dam (SRK 2007), as it was intended to retain water for a period of up to 20 years. With the proposed revised tailings deposition strategy, the South Dam is not required to retain water since the tailings will be beached from the face of the dam from the start of operations. As a result, the South Dam design has been changed to a frozen foundation dam consisting of a compacted rock fill dam with a geosynthetic clay liner (GCL) keyed into the permafrost overburden foundation.

Upon closure, the tailings surface will be covered with a nominal waste rock cover of 0.3 m thickness. The function of the cover is to prevent dust and to minimize direct contact by terrestrial animals. The cover will terminate at the Interim Dike, and the Interim Dike will be levelled to match the elevation of the cover. Once the water quality in the Reclaim Pond has reached the required discharge criteria, the North Dam will be breached as originally intended.

1.2 Stability Assessment Objectives

The minimum factors of safety (FOS) that are required to be achieved for the revised South Dam and the Interim Dike, are defined by the Canadian Dam Safety Guidelines, under the mandate of the Canadian Dam Association (CDA 2014), and are reproduced in Table 1. This memorandum provides details of the stability assessment that has been carried out to confirm that these conditions can be attained.

Table 1: Minimum Required Factors of Safety for the South Dam and Interim Dike in Accordance with CDA (2014)

Stability Condition	Minimum Factor of Safety	Slope
Static Assessment		
During, or at end of construction	Greater than 1.3 depending on risks assessed during construction	Typically downstream
Long-term (steady-state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream slope where applicable
Seismic Assessment		
Pseudo-static	1.0	Downstream
Post-earthquake	1.2	Downstream

Note: This table is summarized from Tables 3-4 and 3-5 in the 2014 CDA Dam Safety Guidelines

2 Slope Stability Assessment

2.1 Methodology

Slope stability analysis was carried out using the commercial RocScience Slide (Version 6) (RocScience 2014) software package. The Spencer Limit Equilibrium Method was initially used to assess stability, and the results checked using the Morgenstern-Price Limit Equilibrium Method. These methods satisfy all limit equilibrium conditions, but each differs in its assumptions. Spencer's method makes the least static assumptions while the Morgenstern-Price method assumes side forces follow a prescribed function and can vary from slice to slice (ASCE 2002). Generally the Morgenstern-Price method is considered to be more conservative.

The analyses were carried out using the built in Non-circular Auto-Refine Search Function. Additional refinement to the search included optimized searches, slope limits and direction of failure.

2.2 Configuration

Figure 2 provides a plan layout of the portion of the TIA that will be used for sub-aerial tailings deposition. Typical sections of the South Dam and Interim Dike are shown in Figures 3 and 4.

The South Dam is a frozen foundation dam which has been designed with a crest width of 10 m and an upstream slope of 4:1 H:V and downstream slope of 2:1 H:V. The crest elevation is set at 38.0 m and results in a maximum dam height of 6 m. The key trench will be about 4 m deep, have a base width of 4 m with 2H:1V, and 1H:1V upstream and downstream slopes respectively. The GCL will be placed along the entire base of the key trench, along the upstream face of the key trench and then slope back within the center of the dam at a slope of 3H:1V.

The Interim Dike is a homogeneous run of quarry (ROQ) rock fill dike placed in the TIA, directly on the existing lake bed sediments, without dewatering the TIA. The Interim Dike will have a crest elevation of 31.0 m, a crest width of 10 m, and upstream and downstream slopes of 3H:1V. The maximum height of the Interim Dike will be about 7.5 m, with the bottom 4.8 m being below the pre-existing water level in the TIA of 28.3 m. The Interim Dike is not required to hold back tailings supernatant water, but is expected to retain tailings solids.

At the time this stability assessment was completed, the Interim Dike was to be 1.5 m higher, with a resultant crest elevation of 32.5 m as illustrated in Figure 4. Subsequently the tailings deposition plan (SRK 2015c) was optimized which resulted in lowering of the crest. However, this stability assessment was not redone since the necessary stability criteria, as defined in Table 1, is met at the higher dike crest, and therefore the results as presented is deemed conservative.

2.3 Foundation Conditions

2.3.1 South Dam

Rigorous foundation characterization has been carried out at the proposed South Dam alignment (SRK 2003, 2007). The foundation conditions are variable, with the overburden thickness thinning drastically towards the abutments, as illustrated in Figure 3. Towards the center of the proposed alignment, the overburden profile is at its maximum thickness. The upper approximately 5.5 m of the profile consists of marine silt, which transitions to marine silt and clay to a depth of about 24 m below ground surface (i.e. about 18.5 m thick). Beneath these sediments is a layer of gravelly till of about 10 m thickness overlying the host basalt bedrock. The entire profile is cold permafrost (-8ºC surface temperature), with an active layer thickness of about 1 m. The marine silts and clays are ice rich, and there are clear ice lenses present. Salinity results from samples collected in the footprint of the South Dam foundation indicate salinity ranges from 30 to 46 parts per thousand, with an average of about 41 parts per thousand. This results in a depressed freezing point of about -2.3ºC in accordance with Velli and Grishin's empirical formulation (Andersland 2004).

2.3.2 Interim Dike

There has been no geotechnical characterization of the foundation conditions beneath the proposed alignment of the Interim Dike. A series of condemnation holes were drilled in Tail Lake, approximately 1,700 m north of the proposed Interim Dike location (SRK 2003). These holes suggest that the overburden thickness is likely to be in the order of 5 to 22 m thick, but provide little detail of the material. Detailed characterization of lake bed sediments in nearby Doris and Patch Lakes (SRK 2009), suggest lake bed thickness (i.e. overburden thickness) of between 4.8

and 35.8 m would not be unreasonable to expect. Foundation conditions beneath the North Dam (SRK 2007, 2012) are similar to those under the proposed South Dam alignment; however, the overall overburden thickness is shallower beneath the North Dam since there is no evidence of the gravelly till zone being present.

All of this anecdotal evidence, together with the fact that Tail Lake, the original lake forming the TIA is a much smaller lake than either the Doris or Patch Lakes, and is geomorphologically different, was used to conservatively adopt foundation conditions beneath the Interim Dike similar to those observed under the South Dam, but with the following modifications. The upper 5 m of the profile was assumed to be unconsolidated lake bed sediments overlying about 18.5 m of marine silt and clay. This once again overlies about 10 m of gravelly till over in-situ basalt bedrock. The TIA is believed to host a closed talik underneath it; however, this has not been conclusively demonstrated (SRK 2005). For the purpose of this assessment the entire profile underneath the proposed Interim Dike was assumed to be within this talik. This profile is considered very conservative.

Based on the assumed soft, unconsolidated nature of the lake bed sediments, the top 2.5 m of sediments under the Interim Dike footprint is initially assumed to have mixed with, or be displaced by the rock fill material, improving the material parameters in the zone of disturbance (i.e. mixing zone). This is later on referred to as Stage 1 construction.

2.4 Material Properties

2.4.1 Summary

Material properties adopted for the foundation, dam and dike materials stability assessment presented in this memorandum is generally consistent with the original North and South Dam design properties (SRK 2007). Where available, these properties have been updated to reflect additional characterization data that has become available subsequent to the original design. Tables 2 and 3 summarize these properties for the South Dam and Interim Dike respectively. More discussions about these properties are provided in the sections below.

Table 2: South Dam Foundation and Material Properties

Parameter		Marine Silt and Clay	Marine Silt	Tailings	Run of Quarry (Dam Fill)	GCL Liner
Moist Unit Weight (kN/m^3)		17 [2]	18	17.5 [1]	20 [2,4]	18 [2,3]
Undrained Shear Strength s_u (kPa)		40 [2]	40 [1]	-	-	-
Non-frozen	Apparent Cohesion c' (kPa)	0 [1]	0 [2]	0	0 [2,4]	0 [2,3]
	Friction Angle, φ ^{0}	30 [1]	32 [2]	40	40 [2,4]	15 [2,3]
Frozen	Apparent Cohesion c' (kPa)	112 [4]	112 [4]	-	5	-
	Friction Angle, φ ^{0}	26 [4]	26 [4]	-	40 [2]	-

(1) SRK (2003)
(2) EBA (2006)
(3) SRK (2007)
(4) SRK (2011)

Table 3: Interim Dike Foundation and Material Properties

Parameter		Marine Silt and Clay	Lake Bed Sediments	Mixing Zone	Tailings	Run of Quarry (Dam Fill)
Moist Unit Weight (kN/m^3)		17 [2]	16	19	17.5 [3]	20 [2,4]
Undrained Shear Strength s_u (kPa)		40 [2]	12	-	-	-
Non-frozen	Apparent Cohesion c' (kPa)	0 [1]	-	0	0	0 [2,4]
	Friction Angle, φ ^{0}	30 [1]	-	32	40	40 [2,4]

(1) SRK (2003)
(2) EBA (2006)
(3) SRK (2007)
(4) SRK (2011)

2.4.2 Marine Silt / Marine Silt and Clay

The adopted marine silt/marine silt and clay properties are generally consistent with the original North and South Dam design (SRK 2007) properties, but have been supplemented by earlier and later overburden characterization data (SRK 2003, SRK 2009, SRK 2011). Specifically, the following logic was applied:

- The unit weight and undrained shear strength parameters were adopted from (EBA 2006). An adjustment was made to the unit weight of the marine silt layer beneath the South Dam. The unit weight was increased based on the decreased clay content as noted in the drill logs (SRK 2007).
- The non-frozen friction angle and cohesion values are adopted from (SRK 2003) and compared with the triaxial test results of the silt and clay material below Doris and Patch Lakes (SRK 2009).
- Published correlations were used to determine the approximate frozen soil properties (Andersland 2004) which reduces the friction angle and increases the cohesion with increasing ice content. These correlations align well with previously identified properties for typical overburden at Hope Bay (SRK 2011). The typical frozen properties identified in 2011 are marginally more conservative than the calculated properties and were subsequently used in this assessment. The frozen material properties are stronger than the thawed material, thus the thawed material is expected to dominate the stability behaviour.

2.4.3 Lake Bed Sediments

Field and laboratory test results of the lake bed sediments indicate a range of low in-situ strength characteristics and relatively high void ratios (SRK 2009). These characteristics may be interpreted as normally consolidated or possibly under consolidated lacustrine sediments. The ratio of undrained shear strength to vertical effective stress was used to determine the average undrained shear strength for the entire 5 m layer. This value was subsequently used as the characteristic property.

2.4.4 Mixing Zone

Properties for the mixing zone (shallow and deep, i.e. Stage 1, 2 and 3 construction referred to later on) was based on engineering judgement, understanding that the mixed material will behave similar to the run of quarry rock; however, the introduction of the low strength lake bed sediments will result in a lowering of the friction angle.

2.4.5 Gravelly Till and Bedrock

Material properties for the gravelly till layer and the basalt bedrock were assumed based on engineering judgement and previous values used in the North and South Dam design (SRK 2007). These values are listed in in Table 4.

In actual fact, the thick upper low strength foundation soils dominate the stability behavior, and there are no conceivable critical deep seated failure mechanism that would be a function of the gravelly till or bedrock. This is demonstrated by the results.

Table 4: Gravelly Till and Basalt Bedrock Material Properties

Parameter	Gravel Till	Bedrock
Moist Unit Weight (kN/m^3)	20	16
Shear Strength (kPa)	-	1,000
Friction Angle (°)	34	-

2.4.6 Run-of-Quarry Properties

The ROQ material properties for the Project have not been measured, but are based on comparison of similar materials as reported in the literature and SRK's internal database (SRK 2011).

2.4.7 Geosynthetic Clay Liner

The GCL properties were similar to those adopted in the previous stability assessment for the South Dam (SRK 2007). These values are based on published laboratory residual shear strength test data.

2.4.8 Tailings

The tailings geotechnical properties are consistent with the test data reported in SRK (2007). Although the sub-aerial deposition method will result in some segregation, and subsequently different gradation of tailings along the surface of the TIA, homogeneous properties have been assumed for the purpose of the stability assessment.

The test data friction angle of 43.2 degrees was deemed too high when compared to similar tailings from other projects, and was reduced to 40 degrees based on SRK's engineering judgement.

2.4.9 Massive/Clear Ice

Massive/clear ice was not included in the analysis. Such ice is not expected to be present under the proposed Interim Dike alignment, and although there is excess ice under the proposed South Dam alignment, any shallow ice will be excavated as part of the key trench excavation, and deep ice will not impact the short term stability.

Based on creep analysis of the previous South Dam design (EBA 2006), creep is expected to occur at a high rate in the thawing foundations upstream and downstream of the dam and at a lower rate within the dam and frozen foundation material. These strains are predicted to occur slowly and in a ductile manner. This analysis was however assuming the dam was behaving as a water retaining structure, which it no longer is. Long term deformation will not compromise the integrity or performance of the structure and as a result additional creep analysis was not carried out.

2.5 Model Setup

For both the South Dam and the Interim Dike, a single typical cross section was analysed. This critical section was conservatively assumed to be the zone where the foundation overburden soils were at its maximum thickness and the structure was at its maximum height. Figures 5 and 6 respectively present the model cross-sections for the South Dam and Interim Dike.

The South Dam stability analysis was carried out for two scenarios; partially thawed foundation conditions and fully thawed foundation conditions. Thermal modeling of the South Dam has confirmed that fully thawed foundation conditions are not likely to occur (SRK 2015a); however, the analysis represents a conservative case demonstrating the system sensitivity. For the partially thawed conditions, the -2.3 degrees Celsius isotherm, as developed from thermal modeling (SRK 2015a), was adopted to define the extent of thaw as illustrated in Figure 5.

For the initial stability assessment, the water level between the South Dam and the Interim Dike was set at an elevation of 31.0 m, while downstream of the Interim Dike the elevation was 30.5 m. In actual fact, water levels will fluctuate and a sensitivity analysis was carried out to confirm the system behaviour.

3 Modeling Scenarios

3.1 During, or at End of Construction

This scenario considers the short-term period during construction, and immediately upon completion of the South Dam and the Interim Dike structures. In both cases, the structures are small in scale and will be constructed in a single construction season, and are likely to be completed within a period of three to four months. For both structures, since tailings deposition will not commence immediately following dam construction, upstream and downstream slopes were assessed under these conditions.

The frozen foundation requirement of the South Dam requires that it be constructed in the winter season, and therefore construction will occur during a period when the foundation is fully frozen. For the conservative scenario, where a partially thawed foundation was assessed, the unthawed materials are considered to be undrained during and immediately following the construction stage.

The Interim Dike will be constructed directly on saturated unfrozen soft unconsolidated lake bed sediments, irrespective of the season of construction. It is therefore reasonable to assume that the lake bed sediments will be in an undrained state during this period since pore pressures may not fully dissipate in the short term.

3.2 Long Term

Overall long term stability for the South Dam was assessed for both fully thawed and partially thawed foundation conditions. The fully thawed scenario was evaluated for both drained and undrained conditions.

For the Interim Dike, long-term stability was assessed with undrained foundation conditions. This may seem counterintuitive; however, due to the long time to consolidate these foundation materials (SRK 2015b) this was deemed the appropriate conservative assumption.

3.3 Full or Partial Rapid Drawdown

No rapid drawdown case was completed for the South Dam. In the long term this dam will have an extensive tailings beach developed adjacent to it, effectively alleviating any rapid drawdown risk. During the period prior to a beach being developed against the dam, the available storage capacity within the TIA, before water will be retained behind the South Dam is very significant, and therefore the likelihood of a full or rapid drawdown scenario occurring is extremely remote.

Rapid drawdown scenarios for the Interim Dike were assessed by increasing the hydraulic gradient across the dike through maintaining a high water level on the upstream side while lowering the downstream water level.

3.4 Pseudo-static

The Project site is located in an area of very low seismicity (SRK 2007). Both the South Dam and the Interim Dike have been assigned a hazard classification of LOW in accordance with CDA guidelines (2014). In accordance with this, the design earthquake is the 1 in 2,475 year recurrence interval event. For the Project site, the peak ground acceleration (PGA) for this return period is 0.036 g, and this value was used for the pseudo-static analysis of the South Dam and Interim Dike.

3.5 Post-earthquake

Given the low seismicity of the Project area and the results pf the pseudo-static stability analysis, significant deformation of either the South Dam or the Interim Dike is not expected during the design earthquake. As a result, further numeric analysis of the stability of these structures to assess post-earthquake stability was not deemed necessary.

4 Stability Assessment Results

4.1 Summary Findings

Attachments 1 and 2 provide details on the results of each individual stability analysis completed. The overall combined results are summarized in Table 5 and Table 6 below, and a more comprehensive explanation of the analysis results are provided in the sections that follow.

Table 5: Interim Dike Slope Stability Results Summary

Stability Condition	Required Minimum FOS (CDA 2014)	Resulting FOS	Run #	Description
Short Term (Construction)	Greater than 1.3	0.9 to 1.5	1 to 6	As construction progresses, the mixing zone depth is expected to increase from the initial shallow (0-2.5 m) zone to a final deep (2.5-5 m) zone.
Long Term	1.5	1.4 to 1.5	7 and 8	This assumes the water level in the TIA reverts to its pre-mining elevation of 28.3 m, with a full tailings beach deposited behind the Dike. The phreatic level in the tailings is varied in the two runs.
Full or Partial Rapid Drawdown	1.2 to 1.3	1.0 to 1.3	9 to 11	Evaluating the effect of a rapid drawdown effect downstream of the Interim Dike after the tailings beach has formed along the upstream side.
Pseudo-static	1	1.2	12	Base case, but with seismic load applied.
Side Slope Sensitivity	-	1.0 to 1.6	13 to 15	Sensitivity analysis of changing the upstream and downstream slope angles of the Interim Dike between 3H:1V and 5H:1V.
Shear Strength Sensitivity	-	1.0 to 1.5	16 to 21	Sensitivity analysis considering different shear strength parameters of the lakebed sediments from 12kPa to 30kPa.

Table 6: South Dam Slope Stability Results Summary

Stability Condition	Required Minimum FOS (CDA 2014)	Resulting FOS	Run #	Description
Short Term (Construction)	Greater than 1.3	1.7 to 2	1 to 4	Changing foundation properties of the thawed zone from undrained to drained.
Long Term	1.5	1.6 to 1.7	5 and 6	Similar to short-term analysis, but tailings is added to the model.
Full or Partial Rapid Drawdown	1.2 to 1.3	-	-	Not analyzed.
Pseudo-static	1	1.5 to 1.6	7 and 8	Similar to base case, but seismic load is added.
Fully Thawed Foundation	-	1.6 to 1.8	9 and 10	Consideration of a fully thawed foundation in the very long term.

4.2 During, or at End of Construction

4.2.1 Interim Dike

The Interim Dike was modelled assuming the ROQ material will be placed directly onto the lake bed sediments, and therefore the lake bed sediments was assumed to be undrained. From experience we know that a mixing zone will occur as rock is dumped onto the soft sediments, and the thickness of this initial layer was assumed to be 2.5 m as previously described. The development of this initial mixing zone is considered as Stage 1 construction in the context of this discussion.

The stability analysis results confirm that even with this Stage 1 mixing zone, large bearing capacity failures are expected (Run #1, Attachment 1). The failures will happen progressively as construction progresses, with the resultant effect of increasing the mixing zone (Stage 2 construction). This increases the FOS (Run #2, Attachment 1). Ultimately by the time construction is complete, the mixing zone is expected to be throughout the entire lake bed sediment layer (Stage 3 construction) with a resultant increased FOS (Run #5, Attachment 1). Between the Stage 2 and Stage 3 construction, smaller failures are expected (Runs #3 and #4, Attachment 1).

The properties used for the lake bed sediments were conservatively based on the in-situ shear strength of the lake bed sediments. Once the load of the Interim Dike has been placed on the material, it is expected that the sediments will consolidate. While the time for complete consolidation is long, early primary consolidation will occur during the construction phase with a resultant increase in the FOS.

The conclusion is that the minimum required FOS of 1.3 during construction will be attained; however, there will be a period where controlled failures are expected and will have to be planned for, and managed. Appropriate work procedures must be developed to ensure that crew and equipment are not put at risk.

The scenario modelled in Run #6 (Attachment 1) demonstrates the buttressing effect of the TIA water. Should the TIA be fully drained during the construction phase, the required FOS may not be attained. This is however not a realistic scenario, since the TIA needs to contain sufficient water to provide reclaim water for the mill.

4.2.2 South Dam

The foundation conditions under the South Dam is generally competent, and the results in a FOS of at least 1.7 for the structure, even when considering undrained properties in the thawed zone (Runs #3 and #4, Attachment 2). In fact, the critical slip surface occurs along the GCL as opposed to through the structure or the foundation.

4.3 Long Term

4.3.1 Interim Dike

Once Stage 3 construction conditions have been reached, and assuming a phreatic level differential between the upstream and downstream sides of the Interim Dike (Run #7, Attachment 1), the FOS is below the required value of 1.5. This is however not a plausible scenario because the dike is free draining and such a phreatic condition cannot occur. The worst case actual condition would be represented by Run #8 (Attachment 1) where there is a high phreatic level in the tailings which dissipates through the dike. Under this scenario the required FOS is attained. In actual fact, thermal modeling has demonstrated that in the long term the tailings will freeze, and as a result the long-term stability is expected to increase. This condition has not been simulated.

Once again the analysis is conservative in the sense that it does not take into consideration the long-term consolidation strengthening of the lake bed sediments.

4.3.2 South Dam

Not surprising, the FOS for the long-term conditions when a tailings beach is developed remain high for all conditions assessed (Runs # 5 and #6, Attachment 2). In this case the critical slip surface is no longer on the upstream slope along the GCL interface, but superficial failure along the downstream slope. In any event the minimum FOS of 1.6 exceeds the required value of 1.5.

Although complete foundation thaw beneath the South Dam is not expected, even in the very long term after consideration of climate change (SRK 2015a), analysis of a fully thawed foundation confirms that it does not change the outcome of the long-term dam stability (Runs #9 and #10, Attachment 2).

4.4 Full or Partial Rapid Drawdown

As discussed previously, drawdown was only considered for the Interim Dike. Drawdown of the Reclaim Pond would result in an increased hydraulic gradient through the Interim Dike and subsequently a reduced FOS depending on the initial and final water level and the rate of drawdown. This is however not a typical case where rapid drawdown causes high internal seepage forces within the structure causing pore pressure dissipation failures because the ROQ material will drain rapidly. Instead, a lowering of the downstream reservoir level would decrease the hydrostatic load provided by the lake, acting as a resisting force in the moment equilibrium calculation. This ensues when the center of rotation occurs upstream of the critical slip surface toe.

In accordance with the tailings deposition plan, the maximum tailings elevation at the Interim Dike is 28.3 m; however, at the time this analysis was completed it was assumed this elevation would be 31 m, 1.5 m below the crest of the Interim Dike. The water level in the Reclaim Pond, when the North Dam is breached, would be the pre-mining elevation of 28.3 m. It was therefore conservatively assumed that there could be a phreatic level in the tailings at 31 m, and should there be a rapid drawdown of the Reclaim Pond to elevation 28.3 m, the resulting FOS is 1.3 (Run #9, Attachment 1). Should the drawdown equivalent to draining the entire TIA occur (i.e.

elevation 23.5 m), the FOS drops to just over 1 (Run #10, Attachment 1). This is however not a realistic scenario since the natural TIA outlet is 28.3 m, and no failure condition would result in a lowering of the water level beyond this point. Should the TIA water level be lowered though pumping, the maximum drawdown elevation to ensure a FOS of 1.3 would be 27.3 m (Run #11, Attachment 1).

4.5 Pseudo-static Analysis

The pseudo-static cases analysed with a PGA of 0.036 g resulted in acceptable FOS for both structures. The South Dam FOS is 1.5 (Runs #7 and #8, Attachment 2) and the Interim Dike FOS is 1.2 (Run #12, Attachment 1).

4.6 Sensitivity Analysis Results

4.6.1 Reclaim Pond Water Level Variability

As described earlier, the water level in the Reclaim Pond acts as a buttress for the Interim Dike and therefore the stability of the structure is quite sensitive towards the water level. A sensitivity analysis to demonstrate this was completed and the results are presented in Figure 7. The results confirm that with the exception of short period of pumped drawdown, the water level behind the Interim Dike should ideally be at about 28.3 m. Pumped drawdown to 27.3 m for short periods would be acceptable.

4.6.2 Interim Dike Side Slope Variability

A practical mitigation strategy to improve the FOS of the Interim Dike would be to flatten the structure side slopes. Figure 8 demonstrates the results of this analysis. These results also assume that only the Stage 1 construction mixing zone is present. These results (Runs # 13, #14 and #15, Attachment 1) clearly demonstrate that at slopes of 5H:1V all required FOS limits can be met, even without accounting for the deep mixing zone (Stages 2 and 3 construction). Increasing the Interim Dike side slopes from 3H:1V to 5H:1V would result in an increase in the Interim Dike construction volume of about 30 to 40%, which is considered to be reasonable and practical as a mitigation strategy.

4.6.3 Interim Dike Lake Bed Sediment Undrained Shear Strength Variability

The stability of the Interim Dike is most sensitive to the undrained shear strength of the lake bed sediments. To demonstrate this, a sensitivity analysis was completed for the case when only Stage 1 construction conditions have been met, but with varying shear strength properties for the sediments (Runs # 16 through #21, Attachment 1). The base case run assumes a value of 12 kPa for the undrained shear strength of the lake bed sediments, based on in situ test data as described earlier. The sensitivity analysis results presented in Figure 9 demonstrate that a 30% increase in shear strength to 16 kPa would be sufficient such that the critical slip surface would no longer be at the base of the lake bed sediments, with a subsequent rapid increase in FOS. In order to meet the long-term stability requirements, without the deep mixing zone (Stages 2 and 3 construction) assumed in the initial analysis, the undrained shear strength of the lake bed sediments would need to exceed 20 kPa.

5 Discussion and Conclusions

The South Dam readily meets all the required minimum slope stability FOS as prescribed by the CDA (2014). This applies even for the most conservative assumption of a fully thawed foundation condition. Detailed site characterization, and associated material property testing have been carried out under the proposed South Dam alignment, and as a result there is a high level of confidence in the results as presented in this memorandum.

Site characterization under the proposed Interim Dike alignment has not been carried out. A foundation profile has been assumed based on a general understanding of the site geomorphology. The assumed foundation profile is however heavily biased towards conservatism. In addition, the Interim Dike crest will be at an elevation of 31 m; however, the stability analysis results presented in this memorandum are for an Interim Dike with a crest elevation of 32.5 m which is 17% higher and therefore adds additional conservatism.

The Interim Dike stability assessment results clearly demonstrate that initial bearing capacity failure is expected in the very weak layer of surficial lakebed sediments. As construction progresses, a mixing zone will develop which will serve to buttress the Interim Dike, with the resultant effect of achieving the required CDA (2014) FOS. Sensitivity analysis has also demonstrated that by flattening the Interim Dike side slopes to 5H:1V would have an effect of increasing the required FOS without the need for development of a deep mixing zone. The approximate 40% additional ROQ material required to construct this requirement is not considered significant; therefore this potential mitigation strategy is very realistic.

Sensitivity analysis results also demonstrate that a 30% increase in undrained shear strength would result in a dramatic increase in the Interim Dike FOS.

The results presented in this memorandum demonstrate that both the South Dam and the Interim Dike can be constructed to ensure that all the necessary minimum FOS in accordance with CDA (2014) can be met. To facilitate construction planning, it is recommended that foundation characterization be carried out along the proposed Interim Dike alignment to confirm the overall foundation profile, specifically the lake bed sediment thickness and material properties. This characterization should be completed as part of the detailed engineering phase of the Project.

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Figures