Volume 1 Annex V1-7 Type A Water Licence Applications

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Doris Tailings Management System Phase 2 Design,
Hope Bay Project

MAC RESOURCES

Doris Tailings Management System Phase 2 Design, Hope Bay Project

Prepared for

TMAC Resources Inc.

Prepared by

SRK Consulting (Canada) Inc.
1CT022.013
November 2017

Doris Tailings Management System Phase 2 Design, Hope Bay Project

November 2017

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Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
ECCC-IR19	6.5	Dust control and management strategy.
INAC-IR43	6.4	Management of TSS in reclaim water
INAC-IR45	4.7.2	Qualitative creep analysis
INAC-IR46	3.5	Tailings storage capacity
KIA-IR168	4.5 4.3.3	Stability analysis Massive ice in foundation
KIA-IR169	4.5 4.4.2	Stability analysis Massive ice in foundation
KIA-IR170	4.8.3	Tailings freeze-back
KIA-IR171	4.9	Tailings settlement
KIA-IR172	4.1.1	Detailed subsurface investigations at a later stage of design
KIA-IR174	4.8.3	No freezing point depression
NRCan-IR4	7.1	No pond left into post-closure
KIA-DEIS-55	4.1.1	Additional surface investigation at later stage of design
INAC-TRC13	4.7.2	Creep deformation of West and South dams

1 Introduction

1.1 Background

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

The Project consists of two phases; Phase 1 (Doris deposit) with an estimated ore reserve of 2.5 million tonnes (Mt), and Phase 2 (Madrid and Boston deposits), which includes an additional ore reserve of approximately 18.7 Mt. The total ore reserve for the combined Phases is 21.2 Mt, which is approximately equal to the total amount of tailings that will be produced. The Doris tailings impoundment area (TIA) has been designed to contain approximately 18 Mt of tailings, with the remaining tailings being deposited in a dedicated Boston tailings management area (TMA) (SRK 2017a). A small fraction of the tailings (detoxified tailings making up about 6%) will be used as underground backfill.

Ore processing includes cyanidation and flotation methods, with two separate streams of tailings being produced, both captured under the tailings management system (TMS). The cyanidation tailings will be detoxified (cyanide destruction) then filtered and blended with waste rock to be returned underground as backfill. At the Doris and Madrid processing facilities, the flotation tailings will be deposited in the Doris TIA, and at the Boston processing facility, the flotation tailings will be filtered and deposited in the Boston TMA (SRK 2017a).

The currently licensed Phase 1 TMS (SRK 2015a) was designed for subaerial deposition of about 2.5 Mt of tailings into the designated Doris TIA. This TIA is a former natural lake (Tail Lake), which has been listed on Schedule 2 of the Metal Mining Effluent Regulations (MMER). Phase 2 development will see the expansion of the TIA to accommodate the increased volume of tailings (Appendix A, drawing DTIA-02).

To ensure environmental containment, the TIA will be impounded by three dams: the North, South, and West dams (Appendix A, drawing DTIA-02). The North Dam, unchanged from the Phase 1 design, will function as a water retaining dam, while the South and West dams will have tailings deposited against their upstream face keeping the Reclaim Pond away from these structures (Appendix A, drawing DTIA-02). The North Dam was constructed in 2012 (SRK 2012) as a water retaining frozen core dam, while the South and West dams are designed as frozen foundation rock fill dams incorporating a geosynthetic clay liner (GCL). The South Dam is part of Phase 1 and will be raised as part of the Phase 2 development. The West Dam is a new structure.

Tailings will be spigotted from a number of points along the eastern perimeter of the TIA, and from the South and West dams, creating a landscape that drains towards the North Dam at an average slope of about 1%.

The TIA will be closed through application of a 0.3 m thick quarry rock isolation cover intended to mitigate tailings dust and prevent direct contact of tailings with terrestrial wildlife. Water quality modeling (SRK 2017b) confirms once the cover has been applied, water discharge from the TIA will meet environmental discharge criteria. Once that has been demonstrated to occur, the North Dam will be breached as originally intended, returning the natural outflow to the pre-mining elevation of 28.3 m.

This report documents TMAC's proposed changes to the currently permitted TMS to accommodate the additional volume of tailings produced as part of Phase 2 development.

1.2 Scope of Work

SRK Consulting (Canada) Inc. was retained by TMAC to carry out the preliminary design of the revised TMS for the Phase 2 Project. The design and related information provided in this report has been prepared in accordance with industry best practice, which includes, but is not limited to the Canadian Dam Safety Guidelines, as documented by the Canadian Dam Association (CDA) (CDA 2007, 2013), the Technical Bulletin on Application of Dam Safety Guidelines to Mining Dams (CDA 2014), various Mining Association of Canada guidelines (MAC 2011a, b, MAC 2017) and publications and bulletins published by the International Commission of Large Dams (ICOLD).

In addition, in response to the 2014 Mount Polley tailings dam failure in British Columbia, and the 2015 Samarco tailings dam failure in Brazil, the design takes into consideration the key recommendations as outlined in the subsequent Independent Expert Engineering Investigation and Review Panel Report (IEEIRP 2015), as well as the recent BC Dam Safety Regulations (B.C. Reg. 40/2016) and the guidelines for Site Characterization for Dam Foundations in BC (APEGBC 2016).

1.3 Report Structure

Section 2 provides a summary of the TMS concept including a comparison of the existing licenced TMS with the proposed revised TMS and a brief discussion of the tailings alternatives evaluation. The TMS design criteria are presented in Section 3, and summary details of the new containment structure design analysis are provided in Section 4. Section 5 list the TMS construction details, including construction material take-off quantities. The TMS operational plan, which includes the deposition plan, is described in Section 6. TMS closure concludes the report in Section 7 and includes a description of monitoring and maintenance activities.

A comprehensive set of appendices, providing details pertaining to the TMS options analysis, tailings deposition plan, hydro-technical design, cover design and engineering analysis (seepage, stability, thermal and consolidation) are included at the end of the report. Engineering drawings for the new containment structures are also included.

A separate Operations, Maintenance, and Surveillance Manual has been prepared for the TIA. (TMAC 2017).

1.4 Reliance on Previous Reports

This report is considered complimentary to reports that have been filed on the public registries as part of the original TMS regulatory approval process, and subsequent compliance reporting. Since much of this information remains valid, the reader is referred to these reports for background information such as general site characterization data which remains unchanged. Table 1 summarizes the key previous reports referred to in this context.

Table 1. Pertinent Previous Reports Relied upon for the Revised TMS Design

Reference	Report Function
SRK Consulting (Canada) Inc. (2005a). *Preliminary Tailings Dam Design, Doris North Project, Hope Bay, Nunavut, Canada*. Report prepared for Miramar Mining Corporation, Project number 1CM014.006, October.	This report was submitted as part of the Final Environmental Impact Assessment for the Doris North Project (MHBL 2005), which ultimately led to issuance of the Project Certificate.
SRK Consulting (Canada) Inc. (2007a). *Design of the Tailings Containment Area, Doris North Project, Hope Bay, Nunavut, Canada*. Report Prepared for Miramar Hope Bay Limited. Project number 1CM014.008.165, March.	This report was submitted as part of the Water Licence Application for the Doris North Project (MHBL 2007), which ultimately led to issuance of the Original Water Licence (2AM-DOH0713).
SRK Consulting (Canada) Inc. (2012). *Hope Bay Project, North Dam As-Built Report*. Report Prepared for Hope Bay Mining Limited. Project number 1CH008.058, October	This report was submitted to the NWB in fulfillment of a Licence Condition after completion of construction of the North Dam.
SRK Consulting (Canada) Inc. (2015a). Doris North Project Tailings Management System Design. Report prepared for TMAC Resources Inc. Project Number 1CT022.002.200.560, May 2015.	This Report was submitted to the NWB as part of the application package for an amendment of the Water License (2AM-DOH1323).
SRK Consulting (Canada) Inc. (2016). Hope Bay Project, Phase 2 Conceptual Closure and reclamation Plan Report prepared for TMAC Resources Inc. Project Number 1CT022.004, November 2016.	This report was submitted as a supporting document to the Draft Environmental Impact Statement.

2 Tailings Management System Concept

2.1 Current Licensed Tailings Management System

The current licensed TMS for the Doris deposit entails subaerial tailings deposition within the designated TIA (SRK 2015a). This TIA was a natural lake, Tail Lake, and was listed on Schedule 2 of the Metal Mining Effluent Regulations (MMER) specifically for use as a tailings facility.

Under the current licence, environmental containment for the TIA is provided through the construction of two dams: a water retaining frozen core dam, the North Dam, and a frozen foundation dam, the South Dam, retaining tailings solids. A third structure, the Interim Dike, is internal to the TIA with the purpose of retaining the tailings solids while allowing tailings water to drain into the Reclaim Pond. About 2.5 Mt (about 2.0 million m^3) of low solids content slurry tailings (49% solids content) will be deposited subaerially between the South Dam and the Interim Dike. The tailings production rate is 1,000 tpd for the first two years, after which it increases to 2,000 tpd for the remainder of Phase 1.

At closure, once environmental discharge criteria have been met, the North Dam will have been breached, allowing the Reclaim Pond to return to its pre-mining elevation of 28.3 m, while the subaerial tailings are to be covered with an isolation cover of a nominal thickness of 0.3 m.

2.2 Phase 2 Tailings Storage Requirements

Mine planning for Phase 2 has resulted in an overall revised mine plan for the Project with a targeted ore production of about 18.7 Mt, of which 15.5 Mt (about 12.0 million m^3) will be deposited in the TIA. This is in addition to the production accounted for under Phase 1, resulting in a total storage requirement in the TIA for Phase 2 of about 18.0 Mt (14.0 million m^3).

About 6% of the tailings are comprised of detoxified cyanide leach tailings, and this tailings stream will be sent underground where it will be mixed with underground waste rock for use as structural mine backfill. The volume of detoxified tailings is small, and therefore storage requirement planning was completed considering the overall total volume without this reduction.

2.3 Selection of Preferred Revised Tailings Management System

Over the life of the Project, several tailings alternatives evaluations were completed at various stages of planning and development (SRK 2005b, 2009a, 2015a). The alternatives evaluation considered different tailings production technologies, different containment system designs, as well as different tailings containment locations.

With respect to tailings containment locations, all previous alternatives evaluations have identified the current Doris TIA as the preferred tailings disposal location. The physical setting of this basin makes it ideally suited to provide superior environmental containment, and its ability to accommodate substantially greater volumes of tailings than currently planned, including the proposed Phase 2 tailings, provides a strong case for continued use of this facility through an expansion design. Finally, since this facility is currently a licenced TIA and Schedule 2 of the MMER has been amended to include the former Tail Lake, the decision was reached to retain the Doris TIA as the preferred tailings disposal location.

The licenced Phase 1 tailings deposition strategy is subaerial hydraulic tailings deposition of a low solids content (49% solids) slurry (SRK 2015a). That tailings technology was adopted as it was demonstrated to be environmentally protective, and economically feasible. For Phase 2, the selected tailings technology was revisited to confirm whether it remained the Best Available Technology (BAT) for the Project.

The first component of that evaluation pertains to the geochemistry of the tailings. Tailings geochemical characterization confirms that the characteristics of the tailings have not changed from what has been evaluated for Phase 1 (SRK 2015b, 2017c). The detoxified tailings are potentially acid generating (PAG) while the flotation tailings contain excess acid neutralization capacity which makes them non-potentially acid generating (non-PAG) and therefore there is no benefit in preventing oxidation of the flotation tailings. To ensure environmental protection associated with both short and long-term exposure of the flotation tailings however, the two tailings streams are managed separately. As a result, tailings disposal technologies requiring full saturation are not necessary. Therefore, it was deemed appropriate to continue to separate these streams and deposit only NPAG flotation tailings in the TIA.

Fresh make-up water for processing is to be minimized as far as practical to minimize any potential environmental impact. As a result, tailings technologies minimizing water loss, maximizing recovery and recycling are appropriate. The TIA is located in an isolated headwater catchment which exceeds the TIA footprint by a factor of 3. Diversion of non-contact water is not practical and as a result, although the climatic water balance at the site is near neutral, the TIA water balance is slightly positive which provides the opportunity for maximizing processing make-up water. This however requires the TIA to have the ability to store contact water, reducing the potential benefits of tailings technologies requiring substantive dewatering.

Tailings containment within the TIA requires the use of containment structures. The height and length of these structures are defined by the assumed design value for tailings density, which will vary in accordance with the adopted deposition technology. Tailings technologies that entail substantive dewatering will result in the need for smaller containment structures which would be advantageous from an overall risk management perspective. However, these risks can also be managed through adopting suitable tailings deposition strategies.

In order to minimize risk to the environment as a result of designing the TIA to contain slurry tailings, the containment structures need to be designed to retain all of the TIA content with a high degree of safety. An evaluation of different tailings deposition strategies to best achieve this was completed (Appendix B), and it was concluded that the preferred deposition strategy entailed developing substantive tailings beaches against the South and West dams, making them solids-retaining structures only (no pond near the dams). There will be no tailings deposition against the North Dam making it a water retaining structure only. This deposition strategy will result in the least amount of environmental risk as tailings discharging from the facility will be prevented. The only environmental release, in case of a containment structure breach, will be supernatant water. This risk is temporary as it will only exist during the operational life of the facility, and can only be eliminated by increasing the fresh water make-up demand from surrounding lakes. Therefore, the best available and preferred tailings technology for Phase 2 is subaerial deposition of 49% solids tailings in the Doris TIA.

2.4 Existing/Unchanged Tailings Management System Infrastructure

2.4.1 North Dam

The North Dam was completed in 2012 (SRK 2012) following two winter construction periods. This structure is a water retaining frozen core dam (Appendix A, drawings DTIA-03 and DTIA-04), with an original design life of 20 years. Additional stability analysis (Appendix C), thermal analysis (Appendix D), and creep deformation analysis (Appendix E) were completed to support extending the design life of the North Dam to satisfy the needs of Phase 2. These analyses confirmed that operational life can be extended with no changes to the dam geometry to about 30 years (additional 10 years on original design life).

In accordance with the currently permitted tailings management plan, the Reclaim Pond elevation will typically vary between 27.3 m and the full supply level (FSL) of 33.5 m under normal operating conditions as predicted by the site wide water and load balance (SRK 2017b).

Since its construction, the North Dam has been subjected to annual geotechnical inspections by a professional geotechnical engineer licensed to practice in Nunavut in accordance with the stipulated Water Licence conditions (SRK 2013, 2014a, 2014b, 2016, 2017d). The inspection reports include a comprehensive analysis of the rigorous dam instrumentation confirming the dam is functioning as designed.

2.4.2 North Dam Spillway

The water level in the TIA will be actively managed through annual discharges to Roberts Bay. Because the North Dam is a frozen core dam, its design requires a substantive thermal freeboard which far exceeds the hydraulic freeboard requirement. As a result, there are no plausible scenarios where the TIA will ever exceed the North Dam hydraulic freeboard level, and therefore an outflow spillway will not be required.

However, as a matter of best practice, an operational spillway has been designed for the TIA at the North Dam at a FSL of 33.5 m. This side-spillway will be 18 m wide and about 180 m long along its centerline with an average gradient of about 0.8%. At the time of its design, the inflow design flood (IDF) was calculated as 3.3 m^3/sec, meaning that flood passing through the spillway would have a maximum flow depth of about 0.2 m at a sub-critical flow velocity of about 1.1 m/sec (SRK 2007a).

As part of Phase 2, the freeboard assessment has been revisited (Appendix F) to allow containment of the probable maximum flood (PMF) such that construction of the spillway will no longer be required.

2.5 Tailings Management System Changes

Table 2 summarizes the proposed changes to the TMS presented in this report and is compared against the existing licensed TMS.

Table 2. Summary of Proposed TMS Changes

Component	Existing Licensed TMS (Phase 1)	Proposed Revised TMS (Phase 2)
Tailings Volume (tonnes)	2.5 million	Additional 15.5 million, for a total of 18.0 million
Tailings Production Rate	1,000 tpd for first two years, ramping up to 2,000 tpd for remaining mine life	1,200 tpd for first year; 2,400 tpd for next 2 years; 3,600 tpd for remaining mine life except last year of mining when production rate drops to 2,400 tpd
Tailings Make-up	Only flotation tailings; detoxified cyanide leach tailings is deposited underground	No change
Deposition Method	Subaerial (single point discharge; multiple locations)	No change (actual spigot locations does differ)
North Dam	Frozen core dam with secondary GCL, construction completed in 2012; FSL at 33.5 m	No change to dam structure; At later stages of the project life when the Reclaim Pond reduces in size the FSL will be lowered to accommodate the IDF
North Dam Spillway	Invert at 33.5 m; designed to pass 1:500 years; 24 hour flood	Not required. Freeboard adjusted to allow containment of IDF of PMF
South Dam	Frozen foundation with GCL liner; Crest elevation 38.0 m	Dam raised by 8 m (downstream method) to a crest elevation of 46.0 m
Interim Dike	New containment structure	Superseded; will be buried by tailings as deposition progresses
West Dam	Did not exist	Frozen foundation with GCL liner; crest elevation of 46.0 m
TIA Discharge Strategy	Seasonal discharge to Roberts Bay; Fixed constant discharge rate; CCME guidelines for the protection of aquatic life (marine) to be met beyond mixing zone in Roberts Bay	No change, with exception of possible year-round discharge at later stages of the project life
Closure Strategy	Breach North Dam; 0.3 m thick dry cover over tailings	No change

The access road to the South Dam will have to be completed in Phase 1 prior to construction of the dam. While not affecting the TIA design directly, the alignment of the road will have to be changed in due time as tailings deposition in Phase 2 will result in sections of the current alignment being submerged in tailings.

3 Tailings Management System Design Criteria

3.1 Dam Hazard Classification

The design, construction, operation and monitoring of dams in Canada have to be completed in accordance with appropriate territorial, provincial, and federal regulations and industry Best Management Practices. The foremost guidance documents are the Canadian Dam Safety Guidelines (CDA 2007, 2013) and the Technical Bulletin on Application of Dam Safety Guidelines to Mining Dams (CDA 2014) published by the CDA.

A key component of the guidelines is classifying the dams into hazard categories (dam class) and establishing appropriate geotechnical and hydro-technical design criteria. Table 3 is a reproduction of the recommended dam classifications as presented in CDA (2013). This classification is based on the incremental consequence of a dam failure (as opposed to total consequence). The incremental consequences of failure are defined as the total damage from an event with dam failure, less the damage that would have resulted from the same event (e.g., a large earthquake or a large flood event) had the dam not failed.

Table 3. Dam Hazard Classification

Dam Class	Population at Risk[1]	Incremental Losses		
		Loss of Life[2]	Environmental and Cultural Values	Infrastructure and Economics
Low	None	0	Minimal short-term loss No long term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary Only	Unspecified	No significant loss of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of important fish or wildlife habitat Restoration or compensation in kind highly possible	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very High	Permanent	100 or fewer	Significant loss or deterioration of critical fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses affecting important infrastructure or services (e.g. highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of critical fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g. hospital, major industrial complex, major storage facilities for dangerous substances)

[1] Definitions for population at risk:
None – There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseen misadventure.
Temporary – People are only temporarily in the dam-breach inundation zone (e.g. seasonal cottage use, passing through on transportation routes, participating in recreational activities).
Permanent – The population at risk is ordinarily located in the dam-breach inundation zone (e.g. as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).

[2] Implication of loss of life:
Unspecified – The Appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

Determination of the appropriate hazard rating is often subjective and is dependent on site-specific circumstances that may require an agreement between the proponent, regulators, and stakeholders. During the dam classification process, each of the four hazard rating components in Table 3 (i.e., population at risk, loss of life, environmental and cultural values, and infrastructure and economics) is considered individually and the overall dam hazard rating is defined by the component with the highest (i.e., most severe) rating. It is important to note that the hazard rating refers to the downstream consequences in the inundation zone of a dam breach.

For all three dams, the "*Population at Risk*" has been generously selected as "*Temporary Only*" due to the very infrequent need for personnel to monitor the dams and areas in the likely dam breach inundation zone. The "*Loss of Life*" has again conservatively been selected as "Unspecified" to reflect that there will be short and infrequent periods of time where people will be present in the in the likely inundation zone.

The "*Environment and Cultural*" impacts associated with a breach of the North Dam will be associated with release of supernatant water (not solids) to Doris Lake and subsequently Doris Creek, which will exceed CCME guidelines for protection of aquatic life (fresh water). A breach of the South Dam could result in release of tailings solids and supernatant water into Ogama Lake, and subsequently supernatant water into Doris Lake. A breach of West Dam could result in release of tailings solids and supernatant water into Doris Lake. Ogama Lake and Doris Lake are considered significant habitat, but restoration of this habitat would be highly possible.

"*Economic*" consequences of a breach of any of the three structures could be significant in terms of direct costs to the proponent, including reputational loss, but would be very minimal in terms of losses to infrastructure or services that might affect other parties.

Based on these factors, the three containment structures are assigned a dam hazard classification as summarized in Table 4.

Table 4. Dam Hazard Classification of TIA Containment Structures

Dam Class	Population at Risk	Loss of Life	Environmental and Cultural Values	Infrastructure and Economics	Overall Hazard Classification
North Dam	SIGNIFICANT	SIGNIFICANT	HIGH	LOW	HIGH
South Dam	SIGNIFICANT	SIGNIFICANT	HIGH	LOW	HIGH
West Dam	SIGNIFICANT	SIGNIFICANT	HIGH	LOW	HIGH

3.2 Design Life

Mining and ore processing for Phases 1 and 2 of the Project is expected to be completed in about 17 years, which is 11 years longer than Phase 1. Once tailings deposition ceases, the North Dam is expected to remain operational for approximately 0.5 years, after which it will be breached. Therefore, the TIA operational life (i.e. the period prior to final closure) is 18 years.

The ultimate design life of the North Dam was set to 22 years, to account for the time passed since the dam was constructed. At closure, the West and South dams will continue to be required to contain tailings solids (but no water). These structures must therefore remain in perpetuity.

The North Dam, which was completed in 2012 (SRK 2012), had an original design life of 20 years as a water retaining structure, assuming it was operating 100% of the time at FSL (33.5 m). Since its completion, the normal water level in the TIA has been about 28.5 m, with a peak of 29.3 m. The most up to date monitoring confirms that the structure is performing in accordance with the design (SRK 2016), and therefore it is reasonable to conclude the structure still has a useable design life of at least 22 years. Additional stability, thermal and creep deformation analysis confirms the expected dam performance can be maintained for a period well over the 22 years of the design life.

3.3 Tailings Physical Properties

Physical properties of the tailings were determined based on three separate geotechnical test campaigns carried out between 2003 and 2009 (Appendix G) and summarized in Table 5.

Table 5. Summarized Tailings Geotechnical Properties

Parameter	Value
Specific gravity	2.85
% Fines (<0.075 mm)	65%
% Silt	52%
% Clay	13%
Void ratio (e) for slurried tailings	1.2
Deposited dry density (Tonnes/m^3) for slurried tailings	1.30
Internal angle of friction (degrees)	40
Cohesion (kPa)	0
Gravimetric Moisture Content (%)	42.6
Hydraulic Conductivity (m/s)	$1.3x10^{-7}$

3.4 Tailings Beach Slope

Subaerial tailings placement will result in beach development (Appendix A, drawings DTIA-10 and DTIA-11). Specific tailings characterization has not been carried out to confirm the expected tailings beach slope angle for the Project; however, typical gold tailings deposited sub-aerially are known to have beach slope angles that range between 0.5 and 2% (Vick 1990). Where the tailings beach transitions to a pond, the slope angle can increase to between 5 and 7%. An overall beach slope angle of 1% has conservatively been assumed for the purposes of tailings deposition modeling for the Project (Appendix B). Should the actual slope be substantially different, additional deposition points will be added into the overall tailings deposition plan to ensure that the final landform can still be developed.

3.5 Tailings Storage Requirement

The revised mine plan for Phase 2 has a targeted ore production of about 18.7 Mt over and above the 2.5 Mt of Phase 1 ore, for a total of 21.2 Mt. The Doris tailings impoundment area (TIA) has been designed to contain approximately 18 Mt of tailings, with the remaining tailings being deposited in the Boston TMA (SRK 2017a).

For the selected TIA tailings deposition strategy, the tailings dry density is about 1.3 tonnes/m^3 (Appendix G). This results in an ultimate tailings volume of about 14.0 Mm^3 deposited at a production rate that ranges between 1,200 and 3,600 tpd.

About 6% of the tailings are comprised of detoxified cyanide leach tailings. These tailings will be sent underground where it will be mixed with underground waste rock for use as structural mine backfill. For the purpose of design, this reduction in volume is not considered material and was not subtracted from the total volume. Ice entrainment during subaerial tailings deposition is a common problem for arctic projects (BGC 2003). An allowance of 20% has been accounted for in the TIA design. Complete Project tailings storage requirements are summarized in Table 6.

Table 6. Tailings Storage Requirements

Component	Value	Source
Tailings storage requirement (Phase 2 only)	12.0 Mm^3 (15.5 Mt)	Quantity based on the mine plan (TMAC 2016a); volume conversion based on dry density listed below in this table. Quantity does not include Phase 1 tailings at 2.5 Mt. Value has been rounded up to nearest million.
Tailings production	1,200 tpd for first year; 2,400 tpd for next 2 years; 3,600 tpd for remaining mine life except last year of mining when production rate drops to 2,400 tpd	Supplied by TMAC.
Tailings production period (Phase 2 only)	17 years	Based on the mine plan (TMAC 2016a).
Ice entrainment allowance	2.4 Mm^3 (20% by volume)	Contingency allowance based on engineering judgement and case studies reported by BGC (2003).
TIA storage requirement (Phase 2 only)	14.4 Mm^3	Sum of tailings storage requirement and ice entrainment allowance.
Run-off and contact water allowance	Not required	Additional storage capacity not required as water will be directed towards the Reclaim Pond. Overall water management will be via the Reclaim Pond.
Deposited tailings dry density	1.3 t/m^3	See Appendix G.

3.6 Stability Criteria

The minimum factors of safety (FOS) that are required to be achieved for the raised South and West dams are defined by CDA (2014) and are reproduced in Table 7.

Table 7: Minimum Required Factors of Safety

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 CDA (2014)

3.7 Design Earthquake

Based on the dam hazard classification of HIGH assigned to the South and West dams the annual exceedance probability (AEP) earthquake design ground motion will be 1:2,475 using both the risk-informed and standards based approaches (CDA 2014). A detailed analysis of the site specific seismic factors was completed (SRK 2017e) with resultant peak ground acceleration (PGA) of 0.021 g and 0.025 g for the South and West dams, respectively.

CDA (2014) specifies for long-term scenarios, i.e. operations and post-closure, the seismic event for a dam with the hazard classification of HIGH must be increased to halfway between the 1:2,475 and 1:10,000-year event. The PGA associated with this condition is 0.036 g and 0.043 g for the South and West dams, respectively. These higher values were used in the pseudo-static analysis.

3.8 Inflow Design Flood

For dams with a HIGH hazard classification, the inflow design flood (IDF) is defined to be 1/3 between the 1,000-year event and the PMF (CDA 2014). The TIA was however designed without a spillway, thus requiring the retention of the full PMF event. The IDF was therefore increased to account for the PMF, which is corresponding to a dam with an EXTREME hazard classification.

A detailed assessment of the inflows was completed (Appendix F) and took into account the local and regional climate models (SRK 2017f). The critical (i.e. most conservative) case was determined to be the snowmelt-dominated spring PMF (i.e. probable maximum snow accumulation + 1:100 year 24 hour rain storm). This translates to a volume of 915,382 m^3 related to snowmelt and 178,718 m^3 related to rainfall for a total IDF volume of 1,094,000 m^3.

3.9 Design Freeboard

The South and West dams are not water retaining structures. Tailings beaches will be developed along the upstream slope of these dams to create a final topography free-draining towards the Reclaim Pond ensuring no water will pond adjacent to these structures. The GCL in both of these dams will terminate at elevation 45.0 m, 11.5 m above the TIA FSL. Tailings deposition will commence from discharge points located near the dam crest at elevation 44.5 m for both the South and West dams, leaving a freeboard of 1.5 m.

The North Dam is a water detaining structure and is subject to wind a wave action. The hydraulic freeboard is the more critical (CDA 2013) of the following two cases:

- Normal freeboard: no overtopping by 95% of the waves caused by the most critical wind with a frequency of 1 in 1,000-year when the reservoir is at its maximum normal elevation; and

- Minimum freeboard: no overtopping by 95% of the waves caused by the most critical wind associated with the AEP event, when the reservoir is at its maximum extreme level immediately following the inflow design flood.

However, the possibility of the extreme high water levels occurring at the same time as the high wind event must also be considered for mining dams (CDA 2014). This was assessed by evaluating a combined freeboard considering the pond elevation following the IDF event as well as 1:1,000-year wind. The calculated values for the combined freeboard were adopted as the design criteria for the TIA.

The increase in the Reclaim Pond elevation at the end of mine life, when the Reclaim Pond is at its smallest, associated with the IDF is about 2.1 m (Appendix F). Therefore, if the Reclaim Pond has a FSL of 33.5, the water level during the IDF will be at elevation 35.6 m.

Consequently the design freeboard is the sum of the wind setup (0.06 m), the wave run-up (1.16 m), and the rise due to the IDF (2.1 m). This equates to 3.3 m of freeboard required above the FSL level of 33.5 m, or an equivalent elevation of 36.8 m. The crest of the North Dam is at a minimum elevation of 37.5 m which exceeds the required freeboard by about 0.7 m.

Based on the current conservative design assumptions, when the Reclaim Pond is at its minimum capacity at the end of the 17 year mine life, the rise in pond elevation required to store the IDF will result in the pond water level exceeding the top elevation of the water retaining component (frozen core) of the North Dam by about 0.3 m. The actual pond stage storage curve will be monitored during operations to update this analyses and if required the FSL will be lowered to accommodate the IDF storage volume below the frozen core elevation in the final years of operations.

3.10 Summary of TMS Design Criteria

A complete summary of the TMS containment dam design criteria are listed below (Table 8), and are consistent with Best Management Practices, including the CDA (2013, 2014) guidelines.

Table 8. Summary of TMS Containment Dam Design Criteria

Component	Criteria
Dam Hazard Classification	North Dam (HIGH) South Dam (HIGH) West Dam (HIGH)
Design Life • Active use period as water retaining structure • Design basis as active water retaining structure • Active use period as solids retaining structure • Design basis as solids retaining structure • Total life until breach	 North Dam (17 years) North Dam (22 years) South Dam (17 years); West Dam (17 years) South Dam (25 years); West Dam (25 years) North Dam (22 years)
Dam staging	North Dam (none, construction completed in 2012) South Dam (single downstream raise planned increasing crest from 38 m to 46 m) West Dam (none)
Tailings production rate	1,200 tpd for first year; 2,400 tpd for next 2 years; 3,600 tpd for remaining mine life except last year of mining when production rate drops to 2,400 tpd
Tailing slurry solids content	37.5% solids (by weight)
Tailings solids specific gravity	2.85
Tailings settled density	1.3 t/m^3
Tailings storage requirement (Phase 2 only) • By mass • By volume	 12.0 Mt 15.5 Mm^3
Ice entrainment allowance • Percentage of tailings capacity • By volume	 20% 2.4 Mm^3
Tailings beach slope • Subaerial tailings • Sub-aqueous tailings	 1% 1%
Tailings deposition method	Single point spigot subaerial discharge (eight locations over the life of mine)
Maximum design earthquake	Halfway between 1:2,475 and 1:10,000 AEP; PGA of 0.036 g (South Dam) and 0.043 g (west Dam)
Inflow design flood	PMF; approximately 1,094,000 m^3
Freeboard requirement	North Dam (1.0 m normal, 3.3 m total) South Dam (0.5 m normal, 1.5 m total) West Dam (0.5 m normal, 1.5 m total)
Stability FOS (Static)	1.3 during construction 1.5 during operation and closure 1.2 to 1.3 partial or rapid drawdown
Stability FOS (Pseudo-Static)	1.0 during earthquake 1.2 post earthquake

4 South Dam and West Dam Design

4.1 Foundation Conditions

4.1.1 South Dam

Rigorous foundation characterizations have been carried out at the proposed South Dam alignment (SRK 2003b, 2005a, 2007a) and are summarized in SRK (2016f). The foundation conditions are variable with the overburden thickness thinning significantly towards the abutments (Appendix A, drawings DTIA-50 and DTIA-06). 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 (-8^{o}C surface temperature), with an active layer thickness of about 1 m. The marine silts and clays are ice rich with clear ice lenses present. Salinity results from samples collected in the footprint of the South Dam foundation indicate salinity ranges from 6 to 86 parts per thousand, with an average of about 47 parts per thousand. This results in a depressed freezing point of about -2.6^{o}C in accordance with Velli and Grishin's empirical formulation (Andersland and Ladanyi 2004).

No subsurface investigations were carried out to date to characterize the foundation conditions downstream of the starter dam. For design purposes it was assumed that the stratigraphy described above extends southward to underlay the footprint of the South Dam raise. Detailed confirmation geotechnical investigations will be required prior to the detailed design stage for the South Dam raise.

4.1.2 West Dam

Minimal subsurface investigations were completed within the footprint of the West Dam, consisting of a single borehole. The borehole log indicates that overburden is consisting of silty clay to a depth of 7 m (Appendix H). Geophysical investigations including ground penetrating radar (GPR) were carried out around the perimeter of Tail Lake (SRK 2006). The nearest GPR line is located about 200 m to the east of the proposed West Dam and indicates depth to bedrock between 3 and 5 m, which is consistent with the thickness indicated in the borehole log.

Regarding the thickness of the overburden profile in the longitudinal direction of the West Dam (parallel to the dam centerline), it was assumed that the overburden thins out gradually and tapers off at the abutments where bedrock is outcropping. The gentle surface topography and the saddle-type topography are a good indicator of the overburden thickness.

4.2 Functionality and Design Parameters

4.2.1 South Dam

The South Dam was originally designed as a frozen core dam (Appendix A, drawings DTIA-05 and DTIA-06) and was intended to retain water for a period of up to 20 years (SRK 2005a, 2007a). With the revised TMS for Phase 1 (SRK 2015a), the South Dam is not required to retain water since the tailings will be deposited as a beach from the face of the dam early on during the mill operations. As a result the South Dam design for Phase 1 has been changed to a frozen foundation dam consisting of a compacted rock fill dam with a GCL keyed into the permafrost overburden foundation. To accommodate the increased tailings quantities in Phase 2, the South Dam will be raised by 8 m in a downstream configuration to reach a crest elevation of 46.0 m.

The South Dam has been designed with a crest width of 10 m and an upstream slope of 4H:1V and downstream slope of 2H:1V. The dam raise will retain the original design parameters for slope grades and crest width. The Phase 1 dam crest elevation is 38.0 m resulting in a maximum dam height of 6 m. The dam raise will increase the maximum height to 14 m (crest elevation 46.0 m).

The key trench configuration and geometry of Phase 1 will be retained in Phase 2 with additional sections to be excavated near the two abutments (Appendix A, drawing DTIA-05). The key trench will be about 4 m deep and will have a base width of 4 m with 2H:1V, and 1H:1V upstream and downstream slopes respectively (Appendix A, drawing DTIA-05). 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 (Appendix A, drawing DTIA-06). The dam raise will also include tying into and raising the GCL liner. The slope of the liner on the raised portion of the dam was however changed to 4H:1V to accommodate geometric constraints of the raise.

4.2.2 West Dam

The frozen foundation West Dam will be built with the same dam cross section as the Phase 1 South Dam, with a key trench and a GCL liner keyed into permafrost. This dam will be constructed in a single raise, and will be about 470 m long with a maximum height of 5 m (crest elevation 46.0 m).

4.2.3 Design Parameters Summary

The key design parameters of the TMS containment structures are summarized in Table 9.

Table 9. Summary of Phase 2 TMS New Containment Structure Design Parameters

Parameter	South Dam Raise	West Dam
Structure type	Downstream raise of the frozen foundation rock fill dam with geomembrane	Frozen foundation rock fill dam with geomembrane
Geomembrane type	GCL	GCL
Geomembrane deployment slope	3H:1V (4H:1V for the raise)	3H:1V
Structure crest centerline length	515 m	470 m
Structure maximum height	14.0 m	5.0 m
Structure crest elevation	46.0 masl	46.0 masl
Full supply level (FSL) (i.e. maximum elevation of tailings)	44.5 masl	44.5 masl
Total freeboard	1.5 m	1.5 m
Spillway	None	None
Structure crest width	10 m	10 m
Upstream structure slope	4H:1V	4H:1V
Downstream structure slope	2H:1V	2H:1V
Key trench depth	4.0 m	4.0 m
Key trench upstream slope	2H:1V	2H:1V
Key trench downstream slope	1H:1V	1H:1V

4.3 South Dam Components

4.3.1 Phase 1 Dam

The Phase 1 dam, approved under the existing Water Licence, has a crest elevation of 38.0 m and incorporates a GCL liner as the seepage control element.

4.3.2 Dam Raise Method

The dam raise will be constructed as part of Phase 2 development and will bring the crest elevation to 46.0 m. The raise must be done in one single step as the tailings deposition strategy requires tailings discharge to be started from the crest of the completed raise.

Dam raises are typically constructed in one of three ways: upstream, downstream, or centerline. The construction method for the upstream and centerline methods require most (upstream), or a part of (centerline) the dam wall to be constructed on previously deposited tailings. In contrast, a downstream configuration requires the raise being built completely outside of the previously deposited tailings footprint.

The upstream configuration is typically the least costly, requiring the least amount of additional fill material; however, a large portion of the raise is founded on the existing tailings. The downstream construction method completely envelops the starter dam (Phase 1 dam) and extends the downstream footprint, requiring the largest amount of dam construction material of the three methods. The centerline method is a compromise between the two previous methods.

It was concluded that founding the dam raise completely or partially onto partially frozen tailings would be a high risk for differential settlement and unacceptable deformation of the dam crest, with possible tear of the GCL liner. Therefore, the downstream configuration is the most appropriate in the case of the South Dam raise.

4.3.3 Key Trench

The seepage control component of the South Dam is the GCL liner, which is keyed into the frozen foundation. The Phase 1 key trench extends to a top elevation of 37.0 m at the abutments and will have to be extended when the dam is raised. This extension cannot be in line with the original key trench, but will turn south towards the abutments of the Phase 2 crest extents. The key trench extension will follow the same design cross section as the Phase 1 dam key trench.

Consistent with the Phase 1 key trench design, the key trench will be excavated in the frozen overburden soils underlying the dam to a depth of about 4 m (Appendix A, drawing DTIA-06). The key trench will terminate in frozen soil; however, should any massive ice be encountered, the key trench will be deepened until all massive ice has been removed. The upstream slope of the key trench will be excavated to 2H:1V to accommodate the deployment of the geomembrane. The downstream slope will be excavated to a grade of 1H:1V to minimize the excavation.

Excavation of the key trench must be done in the winter when the ground is completely frozen. This is necessary to ensure that the ground is as cold as possible before backfilling starts to facilitate the bond between the foundation and the geosynthetic liner. Drill and blast methods will be required to excavate the key trench, and due to the possible high ice content and nature of the soils, a tight drill pattern and high blast load factor will be required. The excavated material will have to be hauled away and disposed of in designated overburden dump, most likely at Quarry #3.

4.3.4 Geosynthetic Clay Liner

The GCL will be the water retaining element of the dam and will be frozen into the key trench to provide the necessary seal. The GCL of the Phase 1 dam was deployed in a chevron shape starting at the base of the key trench, along the upstream 2H:1V key trench slope, sweeping back on a 3H:1V slope to an elevation of 37.0 m, 1 m below the Phase 1 dam crest (Appendix A, drawing DTIA-06).

In preparation for the dam raise, the top of the existing GCL will be exposed by removing the overlying fill and protective cover material. The new GCL sections will be placed in a manner similar to that used in Phase 1. Where the Phase 1 and Phase 2 junction occurs, GCL panels will overlap at least 1 m wide and will include a bead of powdered bentonite. The Phase 2 GCL panels will extend to elevation 45.0 m, 1 m below the ultimate dam crest.

The GCL along the raised portion of the dam will be deployed at a slope of 4H:1V, shallower than the 3H:1V slope of the Phase 1 dam. This change in slope is necessary to accommodate the key trench geometry resulting from the downstream raise. The bedding and protection layers of gravel will be tied-in with the Phase 1 dam zones (Appendix A, drawing DTIA-09).

The top edge of the GCL will be terminated in an appropriately sized anchor trench. The GCL will be deployed in vertical strips (the width of the GCL rolls). Overlaps will be at least 0.5 m wide, and all overlaps will have a seal of powdered bentonite.

In the base of the key trench, the GCL will be placed directly onto the prepared and clean foundation with imperfections filled with granular bentonite. In all other areas, the GCL will be sandwiched between two 0.3 m thick compacted layers of crushed gravel (pea gravel size).

4.3.5 Dam Bulk Fill

The bulk fill of the starter dam, including the key trench, consists of geochemically suitable run-of-quarry (ROQ) material. The bulk fill of the dam raise will consist of the same type of material. The quarry rock size will be limited to material with a maximum size of 600 mm and well graded with a good mix of fines. This material must be placed in lifts no greater than 1 m and must be compacted with a 15 tonne vibratory compactor or using wheel traffic from loaded haul trucks.

4.3.6 Transition Zone

In the Phase 1 dam, the GCL is protected using a fine crushed gravel (pea gravel) produced from ROQ material. To minimize losses of this bedding material, a transition zone of 150 mm minus crushed ROQ material was placed between the bedding and dam bulk fill zones. Prior to raising the dam, the top of this transition zone will be exposed and then extended to reach the design elevation of the dam raise.

The material must be well graded with sufficient fines. This bedding layer will be about 1 m thick, will be placed in a single lift, and compacted using the same means as the dam bulk fill.

4.3.7 Bedding Zone

For the dam raise the GCL will be sandwiched between two 0.3 m thick compacted layers of bedding material for protection. The top of the bedding zone will be removed from over the GCL and once the new GCL extension was installed, the bedding material will be replaced to create a continuous protective bedding layer (Appendix A, drawings DTIA-06 and DTIA-09). This material will be ROQ material crushed to pea gravel size.

4.3.8 Dam Shell

No special dam armouring is required and no special upstream or downstream riprap is required. The dam shell will be constructed using the same ROQ material as the dam bulk fill.

4.3.9 Monitoring Instrumentation

Ground temperature cables installed in the key trench of the Phase 1 dam will be protected during construction and maintained for continuous monitoring of the dam performance. The thermal monitoring will be supplemented by new ground temperature cables located within the dam raise as well as the extended key trench. Some of the survey prisms of the Phase 1 dam will be destroyed during construction, and the overall deformation monitoring will rely on new prisms installed in appropriate locations in the raised portion of the dam.

Vertical ground temperature cables will be installed in boreholes drilled through the dam fill after the completion of the dam raise and the new cables will extend to the original ground level. The portion of the boreholes within the ROQ fill may require temporary casing. Horizontal ground temperature cables will be placed within the liner bedding layer along the upstream side of the key trench.

Survey prisms will be permanently installed in large boulders within the dam shell.

4.4 West Dam Components

4.4.1 Timing

The West Dam was designed using the same typical section as the Phase 1 South Dam. Being a dam with a relatively low height, it will be constructed in a single raise; however, the dam construction could be delayed as it will not become necessary until Year 3 of the Phase 2 tailings deposition. The design elements of the West Dam are detailed below.

4.4.2 Key Trench

Considering the foundation conditions along the proposed alignment of the West Dam, the key trench will be excavated in the frozen overburden soils underlying the dam to a depth of about 4 m (Appendix A, drawings DTIA-07 and DTIA-08). The key trench will terminate on frozen overburden soil; however, should any massive ice be encountered, the key trench must be deepened until all of the massive ice has been removed. The upstream slope of the key trench will be excavated to 2H:1V to accommodate the deployment of the geomembrane. The downstream slope will be excavated to a grade of 1H:1V to minimize the excavation.

Excavation of the key trench must be completed in the winter when the ground is completely frozen. This is necessary to ensure that the ground is as cold as possible before backfilling starts, facilitating the bond between the foundation and the GCL. Drill and blast methods will be required to excavate the key trench, and due to the possible high ice content and nature of the soils, a tight drill pattern and high blast load factor will be required. The excavated material will have to be hauled away and disposed of in designated overburden dumps, most likely at Quarry #3.

4.4.3 Geosynthetic Clay Liner

The GCL will be the water retaining element of the dam and will be frozen into the key trench to provide the necessary seal. The GCL will be deployed in a chevron shape, starting at the base of the key trench, along the upstream 2H:1V key trench slope, and then sweeping back on a 3H:1V slope to an elevation of 45.0 m, 1 m below the dam crest (Appendix A, drawing DTIA-08). The top edge of the GCL will be terminated in an appropriately sized anchor trench. The GCL will be deployed in vertical strips (the width of the GCL rolls). Overlaps will be at least 0.5 m wide, and all overlaps will have a seal of powdered bentonite.

In the base of the key trench, the GCL will be placed directly onto the prepared and clean foundation with imperfections filled with granular bentonite. In all other areas, the GCL will be sandwiched between two 0.3 m thick compacted layers of crushed gravel (pea gravel size).

4.4.4 Dam Bulk Fill

The bulk fill of the West Dam, including the key trench, will consist of geochemically suitable ROQ material. The quarry rock size will be limited to material with a maximum size of 600 mm and well graded with a good mix of fines. This material must be placed in lifts no greater than 1 m and must be compacted with a 15 tonne vibratory compactor or using wheel traffic from loaded haul trucks.

4.4.5 Transition Zone

The GCL is protected using a fine crushed gravel (pea gravel) produced from ROQ material. To minimize losses of this bedding material, a transition zone of 150 mm minus crushed ROQ material will be placed between the bedding and dam bulk fill zones. The material must be well graded with sufficient fines. This bedding layer will be about 1 m thick, placed in a single lift, and compacted using the same means as the dam bulk fill.

4.4.6 Bedding Zone

The GCL will be sandwiched between two 0.3 m thick compacted layers of bedding material for protection. This material will be ROQ material crushed to pea gravel size.

4.4.7 Dam Shell

No special dam armouring is required, and no special upstream or downstream riprap is required. The dam shell will be constructed using the same ROQ material as the dam bulk fill.

4.4.8 Monitoring Instrumentation

A series of ground temperature cables and survey prisms will be installed at the West Dam to monitor the thermal regime of the foundation and overall deformation performance.

Vertical ground temperature cables will be installed in boreholes drilled through the dam fill after the completion of the dam. The portion of the boreholes within the rock fill may require temporary casing. Horizontal ground temperature cables will be placed within the geomembrane bedding layer along the upstream side of the key trench.

Survey prisms will be permanently installed in large boulders within the dam shell.

4.5 Stability Analysis

A comprehensive stability analysis was carried out to confirm whether the raised South Dam and the West Dam meet the appropriate design requirements as stipulated in Section 3.6. Complete details of the analysis are presented in Appendix C and the results are summarized in Tables 10 and 11.

Table 10. South Dam Minimum Factors of Safety

Stability Condition	Required Minimum Factor of Safety (CDA 2014)	Assessed Minimum Factor of Safety	
		Upstream Face	Downstream Face
Short Term (Construction)	Greater than 1.3	1.6	1.3
Long Term	1.5	1.6	1.6
Full or Partial Rapid Drawdown	1.2 to 1.3	Not applicable	Not applicable
Pseudo-static	1.0	1.4	1.2

Table 11. West Dam Minimum Factors of Safety

Stability Condition	Required Minimum Factor of Safety (CDA 2014)	Assessed Minimum Factor of Safety	
		Upstream Face	Downstream Face
Short Term (Construction)	Greater than 1.3	1.8	1.3
Long Term	1.5	2.3	1.5
Full or Partial Rapid Drawdown	1.2 to 1.3	Not applicable	Not applicable
Pseudo-static	1.0	1.4	1.1

Both the South Dam raise and the West Dam meet all the required minimum slope stability FOS as prescribed by CDA (2014). This applies for the expected most conservative condition of a partially thawed undrained foundation. Detailed site characterization, and associated material property testing have been carried out under the Phase 1 South Dam alignment, and as a result there is a high level of confidence in these results.

4.6 Settlement Analysis

Settlement of the South and West Dams could occur as a result of one of two reasons: dam fill consolidation or foundation consolidation. Since the dam's fill is compacted ROQ material and the total dam height is limited, there is no expectation of any appreciable fill settlement.

Foundation settlement beneath the South and West Dams could occur as a result of thaw consolidation. Normal thaw consolidation can also be exacerbated by thaw of massive ice which may be present in the foundation soils. Thermal analysis (Appendix I) has however demonstrated the foundation beneath both dams will remain frozen for the design life of the structure. As a result, thaw consolidation is not expected to be of concern while this facility is in active operation.

4.7 Deformation (Creep) Analysis

4.7.1 North Dam

Creep deformation analysis was completed as part of the original North Dam design (SRK 2007a, EBA 2006). This analysis considered the original design life of the North Dam of 25 years. Since the Phase 2 Project extends the North Dam design life to 2041 (i.e. 30 years from 2011 when construction started), a reassessment of the creep deformation analysis was undertaken. This new timeline assumes a period of nominal water impoundment prior to start of tailings deposition in 2017, active tailings deposition between 2017 and 2036, and a five-year post closure period prior to breaching the dam in 2041.

The objective of the creep deformation analysis is to anticipate if long-term strains, occurring over the dam design life, can affect the performance or compromise the stability of the North Dam. The analysis also confirms whether the integrity of frozen ice-saturated core and underlying saline foundation will be affected by creep deformations occurring in these two zones, and if the level of the core crest remains above the FSL throughout the dam design life.

The detailed analysis is presented in Appendix E and confirms the following:

- Long-term ductile behavior is predicted for the materials in the ice-saturated frozen core and underlying frozen foundation. Creep shear strains in these zones will occur very slowly and will remain below the strain rate for brittle failure modes.

- Vertical creep displacements will not compromise the long-term integrity of the frozen core and underlying foundation. Shear and deviatoric stresses in these zones caused by creep strains will remain well below the expected peak strengths of the materials. No shear strain localization is predicted within and underneath the frozen core.

- Long-term performance of the frozen core is not expected to be compromised throughout the dam design life. Thirty years after dam construction, the total settlement of the core will be around 1.0 m, i.e., 0.5 m above the FSL.

- Shear strains are predicted to localize at the downstream side of the dam. Thirty years after dam completion, high shear strains (~ 60%) with very low strain rates (~1.0E-08 year-1) can be expected in few points within the localization surface. However, shear stresses and principal stress differences will remain well below the expected peak deviatoric stress of the materials (> 1 MPa).

4.7.2 South Dam and West Dam

The South and West dams will be subject to creep deformation over the very long term, assuming ice-rich soils are present. Creep has previously been assessed for the South Dam (SRK 2007a), and no further quantitative analysis was completed.

Rigorous thermal modelling of the dams (Appendix I) demonstrated that once the tailings are frozen back, the foundation will remain frozen in the long term, and therefore deformation and mass instability due to thawing of ground ice is not expected. Ice-rich foundation soils beneath the dams will therefore not affect the bulk stability; but may in the very long-term result in very slow and minor deformation as a result of creep. The tailings deposition plan was designed to eliminate any water ponding against the dams. The anticipated deformations can therefore be accommodated with no risk of losing containment of the tailings solids, especially since the bulk of the tailings will remain frozen for the foreseeable future, as demonstrated by the thermal model (Appendix I). In addition, even under fully thawed conditions, the tailings will be drained and therefore no loss of containment can occur, similar to the Boston dry stack tailings. This determination of low risk of failure due to creep resulted in the decision of not completing a detailed creep analysis for the West and South dams at this stage of design.

Deformation monitoring during the operational phase of the structures will be undertaken, and if required, the downstream dam slopes will be flattened to 4H:1V as was the case for the original South Dam design. However, the revised thermal analysis (Appendix I) suggests this will likely not be required. Notwithstanding this possible mitigation strategy, the South and West dams will not retain saturated tailings, and therefore the structure can tolerate a large amount of long term deformation.

4.8 Thermal Analysis

4.8.1 North Dam

An updated thermal model was completed (Appendix D) to predict the behavior of the North Dam based on the extended design life of the structure. The analysis assumes a very conservative constant water level equal to the FSL of 33.5 m. The modeling was completed for a 48-year design life (ending in calendar year 2050) which extends well beyond the expected period of mining and subsequent closure. The calibrated model, with consideration for climate change and conservative inputs indicate the frozen core will remain below the required -2°C under normal operating conditions.

Towards the end of the design life, the foundation over a portion of the critical section is expected to be warmer than -8°C beneath the thinnest sections of the dam, i.e. near the abutments. The warmer foundation conditions will result in a higher fraction of unfrozen water and a greater potential for creep deformation. The thermal criteria is met over the actual expected design life for the thickest section of the dam which impounds the greatest head of water, indicating that the dam will perform as intended for the expected life of the facility.

4.8.2 South Dam and West Dam

Rigorous thermal modeling was completed to determine whether the foundation of the South and West dams would function as proposed (Appendix I). The modeling confirms, using conservative assumptions, the GCL would remain frozen into the underlying foundation for the required mine life, and in perpetuity. The analysis was completed taking into account the freezing point depression as a result of overburden soil pore water salinity.

4.8.3 Tailings

Freeze-back of the placed tailings was assessed as part of the South and West dams thermal models. Tailings freeze-back was predicted to range from 13 years near the South Dam (17 m thick tailings) to 2 years near the West Dam (3.5 m thick tailings). The maximum thickness of tailings is 20 m and the 3 m difference in thickness is not expected to significantly extend the duration of time for freezeback of the tailings.

The predicted freezeback for the tailings was based on phase change of porewater at 0˚C and an unfrozen water content curve for silt size soil. Process water held within the tailings is not expected to have an appreciable level of dissolved ions which contribute to a freezing point depression, and therefore no allowance was made for such conditions in the thermal model.

This assessment is based on the conservative assumption that the entire volume of tailings and the associated thermal forcing is applied instantaneously at the beginning of tailings deposition and lasts for the entire duration of the deposition. Additional conservatism is introduced by assuming the initial tailings temperature of +6°C. in reality, successive thin layers of tailings will be deposited throughout the year, with some of the layers being frozen and some thawed depending on timing of deposition, therefore complete freeze-back is expected to occur much earlier.

Tailings deposited in water will begin to freeze back in areas where the ice column freezes completely to the bottom (bottom-fast ice) in the winter for a sufficient period of time to allow for a net annual loss of heat from the ground (Stevens *et al*. 2010a; Stevens *et al.* 2010b), or once the tailing surface becomes sub-aerially exposed.

The active layer within the tailings is predicted to be just under 1.5 m for the active design life of the dams. Considering climate change, the active layer is predicted to increase to about 2.1 m by the year 2,100 (Appendix I).

4.9 Tailings Consolidation Analysis

Tailings consolidation will result in a change of storage capacity over the life of the facility, as well as create a change in landform post closure. A rigorous assessment of the consolidation characteristics of the tailings surface was undertaken (Appendix J) to assess the possible range of settlement, assuming thaw of foundation soils to a depth of 7 m. The calculated settlement values represent the displacement at final consolidation, i.e. excess porewater pressures dissipated completely, irrespective of the time it takes to achieve this state.

These results confirmed that consolidation will be about 1.6 m at the expected maximum thickness of tailings of about 20 m. In addition, assuming fully thawed conditions beneath the tailings, an additional settlement of 1.0 to 2.6 m might be experienced as a result of settlement of the underlying foundation soils due to loading of the tailings.

In reality, and as confirmed by the thermal analysis, (Appendix I) tailings will freeze back during the winter following deposition and may thaw in the subsequent summer if the thickness of the overlying tailings is less than the predicted active layer (about 2.1 m). Settlement will progress while the tailings are in a thawed state, but the bulk of tailings are unlikely to achieve final consolidation and maximum settlement before becoming part of the permafrost.

4.10 Dam Break Analysis

In accordance with CDA (2013), a dam break analysis may be required when a dam has a hazard rating of HIGH or greater. The purpose of a dam break analysis is to determine the inundation zone downstream of the dam in question should a catastrophic breach occur.

In determining the dam hazard classification, consideration was given to tailings supernatant water and tailings solids reaching the receiving environment. The North Dam will include Tail Lake outflow, Doris Lake, Doris Creek and further downstream Little Roberts Lake. The South Dam will include Ogama Lake, Ogama Lake outflow and subsequently Doris Lake, while tailings breaching the West Dam will reach Doris Lake.

As previously explained, there is no conceivable chance of tailings solids being released as a result of a breach of the North Dam. Supernatant water; however, could conceivably reach the entire downstream catchment all the way to Roberts Bay.

A breach of the South Dam could result in release of tailings solids into Ogama Lake. There is a remote chance that some solids may find their way into the Ogama Lake outflow, and ultimately Doris Lake. The tailings solids will not be transported any further. Supernatant water will; however, progress all the way along the drainage network to Roberts Bay.

Tailings solids from a breach of the West Dam could reach Doris Lake, but at a location about 3.5 km away from the Doris Lake outflow and it is not expected the solids would migrate any further. Supernatant water could progress all the way along the drainage network to Roberts Bay.

The breach scenarios described above are intuitive, although extremely conservative. Nonetheless, these scenarios were adopted in assigning the dam hazard classification for the structures. A rigorous dam breach analysis will not result in a different conclusion, and therefore was not completed.

4.11 TSF Water Balance

A site wide water and load balance, including the TIA, has been developed for the Project (SRK 2017b). The TIA is designed to contain site-wide contact water, mill process water as well as treated domestic waste water. Reclaim water will be drawn from the TIA Reclaim Pond for re-use in the Process Plant.

There are no non-contact surface water diversions upstream of the TIA. The TIA is located in an isolated catchment, and the benefits of any diversions are outweighed by the relative cost and complexity of constructing them.

4.12 Seepage Analysis

The purpose of the South and West dams is to retain the tailings and supernatant water. Tailings deposition will be done such that a beach will develop upstream of the dams. This will have the resultant effect of ensuring there will never be any water in close proximity of the dams.

While the tailings beach is being developed, there may be short periods when some water may be in close proximity of these dams. However, as it has been demonstrated by the seepage analysis (Appendix C), the maximum amount of seepage that might occur under the worst case scenario is about 50 m^3/day from the South Dam and less than 1 m^3 per day from the West Dam. This result is only predicted if: a) the tailings remain fully saturated in perpetuity, b) the TIA is at its full supply level of 33.5 m, and c) ignoring the fact that most of the tailings and the foundation soils are frozen in the long term.

4.13 Seepage Collection

No seepage collection is planned for either the North, South or West dams since the design analysis confirm that seepage is not expected (SRK 2007a, 2015a, and Appendix C). Should seepage be identified as part of the routine monitoring, a collection system will be implemented and any seepage collected will be pumped back to the TIA.

5 South Dam and West Dam Construction

5.1 Construction Materials

Construction material for the South Dam raise and West Dam consists of bedding, transition and ROQ material. The granular fill will be produced on site from one of many local approved quarries, with Quarry #3 likely being the primary source. Complete geological, mineralogical and geochemical details of these quarry sites are documented in SRK (2007b, 2008).

Other materials to be used to construct these structures include GCL and geotextile. Complete details of these materials are provided in SRK (2011).

5.2 Construction Equipment

Typical construction equipment will be used at the South Dam raise and West Dam. A contractor fleet consisting of 40 tonne trucks will be used for hauling the excavated overburden and the dam fill, with smaller articulated trucks used in the narrower areas near the top of the dam. Bulldozers and smooth drum vibratory compactors will be used to complete the fill placement. Hydraulic excavators may be used for special tasks as required. Drilling and blasting will be completed using conventional tracked blast hole drills.

5.3 Construction Quality Control and Quality Assurance

Complete details of the Quality Assurance and Quality Control (QA/QC) procedures to be followed for the construction activities are provided in SRK (2011). QC will be the responsibility of the Contractor, and/or the equipment and materials manufacturer. The Engineer of Record, a Registered Professional Engineer in the Nunavut Territory, will carry out the QA. Complete documentation of all QA/QC data will be provided in relevant as-built reports.

5.4 Construction Schedule

Extension of the South Dam key trench excavation and backfill must be completed in the winter to eliminate potential issues caused by thawing of the soft overburden soils, as well as to ensure that a thermal blanket is completed to protect the permafrost in the foundation. The bulk of the fill can be completed during any season. Similarly, excavation of the West Dam key trench must be completed in the winter while the bulk fill can be completed during any season.

5.5 Material Quantities

Material quantities for the construction of the South Dam raise and West Dam are summarized in Table 11. All fill and excavation volumes represent neat volumes, i.e. “in place”, with no allowance for swelling and compaction. It was assumed that the key trench excavation will be completed to the full 4 m depth specified, i.e. no termination on shallow bedrock. The liner quantities are neat quantities, with no allowance for seams and waste.

Table 12. South Dam and West Dam Material Quantities

Material	Quantity	
	South Dam Raise	West Dam
Liner Bedding (m^3)	9,300 m^3	6,100 m^3
GCL (m^2)	14,900 m^2	10,000 m^2
ROQ (m^3)	148,500 m^3	33,300 m^3
Transition Fill (m^3)	32,800 m^3	14,700 m^3
Key Trench Excavation (m^3)	8,000 m^3	17,000 m^3

6 Tailings Management System Operation

6.1 Operations, Maintenance and Surveillance Manual

A standalone Operations, Maintenance and Surveillance (OMS) Manual has been prepared for the Doris TIA (TMAC 2017). The OMS Manual is compliant with Part G of the mine's current Water Licence (2AM-DOH1323), the Mining Association of Canada's (MAC) guideline (MAC 2011), as well as CDA (2014). Following issue of the amended water license for Phase 2 and prior to Phase 2 tailings deposition, this OMS Manual will require updating.

6.2 Tailings Deposition

Complete details of the tailings deposition plan, including the options evaluated, are described in Appendix B. The preferred tailings deposition plan ensures that the supernatant pond will be located away from the South and West dams (Appendix A, drawings DTIA-10 and DTIA-11). Deposition will start from the crest of the South and West dams to create beaches that would push the supernatant water away from these structures. Once these beaches were created, the spigot points will be moved to the east flank of the TIA, where deposition will begin from elevation 49.5 m. This will create a long and even tailings surface sloping toward the North Dam, ensuring that the water in the former Tail Lake is displaced towards the north.

6.3 Reclaim Pond Storage Volume

The tailings deposition plan will result in the Reclaim Pond reducing in size over the life of the Project. For all but the last few years of the Project, the Reclaim Pond will have sufficient capacity to allow year-round reclaim water to be drawn from the TIA, including under ice conditions in the winter. However, near the end of the Project life, this condition cannot be satisfied; therefore, increased volumes of fresh make-up water and more TIA discharge will be required, and potentially the FSL may have to be lowered to accommodate the IDF.

6.4 Water Management

Water management in the TIA will be done through the Reclaim Pond. Reclaim water will be drawn for make-up water in the Doris Processing Plant and the Madrid North Concentrator. TSS at the intake will be managed by placing the intake away from the tailings beach and by appropriate screening if necessary.

6.5 Dust Management

A comprehensive assessment of possible dust management practices for the tailings surface is presented in Appendix K. The tailings deposition plan has been developed, as far as practical, to minimize the area of exposed inactive tailings surface that may be prone to dusting. Beyond such mitigation by design, the primary dust control measure of the TIA will be the use of environmentally suitable chemical dust suppressants. The application of these suppressants will be reviewed on an ongoing basis to ensure any areas that may be at risk will be adequately covered. Generally, annual application of chemical suppressants will be completed; however, it is recognized that more frequent applications may be required as discharge locations are changed throughout any given year.

In addition to chemical dust suppressants, natural dust control in the form of packed snow when available, will be used as far as practical. Again, the effectiveness will fluctuate on a year-by-year basis depending on how deposition points vary for any given winter season.

Finally, if for any reason the above dust control methods prove to be temporally ineffective, a suitable water cannon will be available to allow for dust suppression by wetting the areas of concern.

7 Tailings Closure and Reclamation

7.1 Closure Concept

Upon closure, the tailings surface will be covered with a nominal waste rock cover of 0.3 m thickness (Appendix L). The function of the cover is to prevent dust and to eliminate direct contact by terrestrial wildlife. All water from the Reclaim Pond will be pumped to Roberts Bay Discharge System. The North Dam will be breached as originally intended for Phase 1 and the small residual pond will be backfilled to prevent any accumulation of water.

7.2 Closure Components

7.2.1 Landform Design

The tailings deposition plan has been developed to ensure that a free draining tailings surface remains at closure. This tailings surface will be shaped as part of the regular tailings deposition plan (Appendix B), such that there is a primary drainage channel leading from the South Dam towards the Reclaim Pond. This ensures there is no requirement for construction of diversion structures post-closure.

7.2.2 Cover System

As described in the cover design memo (appendix L), the TIA will only contain flotation tailings which are NPAG with abundant neutralization potential and thus buffering capacity. Although several metals in the tailings solids occur at concentrations in excess of crustal abundances, many of these metals are associated with sulphides and as such will primarily partition into the detoxified tailings which means they will not be of concern in the TIA. Long-term humidity cell tests indicate that after the initial flushing of the samples, an increased tendency for neutral pH metal leaching may develop, with arsenic being of particular concern.

The TIA water and load balance (SRK 2017b) suggests that possible neutral metal leaching does not pose a limitation in ensuring that the water quality in the TIA meet site specific closure water quality criteria, and therefore no infiltration reduction cover is required on the exposed tailings surface. The tailings surface will however be susceptible to wind erosion with the resultant effect of dust exposure.

Similarly, although the tailings surface is landscaped to allow free drainage, the tailings are susceptible to hydraulic erosion, which will mobilize tailings towards the Reclaim Pond with a resultant increase in total suspended solids.

Therefore, a tailings cover that functions to prevent wind and water erosion will be constructed over the entire tailings surface.

The minimum thickness of cover that can practically be placed over the tailings surface would be about 0.3 m thick, and therefore the cover design has been set at 0.3 m thick ROQ material.

Part of the tailings will not be trafficable for some years following tailings placement. Therefore, in order to place the cover winter construction over a frozen tailings surface will be required in those areas. Although thermal modeling has demonstrated that the bulk of the tailings mass will freeze back in the long term, and remain frozen (Appendix D), consolidation settlement in the 2 m thick active layer can still be expected (Appendix J). In addition, should ice lenses develop within the active layer of the tailings beach could further contribute towards long term differential settlement of the tailings surface and subsequently any associated cover system. Such differential settlement will not negatively affect the cover performance since localized ponding that might result would not prohibit the cover from ensuring that wind and water erosion is mitigated (Appendix C).

7.2.3 Water Management

Conveyance Channel

Although the tailings will be landscaped to ensure positive drainage of the entire tailings surface area, the resultant effect will be a dedicated primary conveyance channel along the west side of the facility. This channel will be sized and armoured to allow conveyance of the 1 in 500 year, 24-hour duration storm event. Based on the channel geometry, the peak flow is expected to be about 4.3 m^3/second, requiring riprap with a D_{50} of 0.1 m. This rock is similar to the proposed ROQ cover material, and therefore no special requirements for the primary conveyance channel is required other than possible local thickening of the cover. For bonding cost estimates, it has been assumed that this localized cover thickening would occur over a zone about 20 m wide along the entire length of the channel. This section will have an increased cover thickness of 0.6 m.

Discharge Criteria

A key closure objective is to ensure a walk away closure scenario, and therefore the water retaining North Dam must be breached. This can however only occur when the water quality in the Reclaim Pond meets environmental discharge criteria. The TIA, once breached will discharge into Doris Lake which in turn discharges into Doris Creek.

7.2.4 Containment Structures

North Dam

Water in the Reclaim Pond will continue to be managed via active pumping to the Roberts Bay Discharge System, and the North Dam will be breached. The breach design will consist of a slot cut through the dam down to the pre-construction elevation. The cut will measure about 20 m wide, with 4H:1V side slopes on either side. The cut slopes will be covered with a 2.5 m thick layer of ROQ material to ensure physical and thermal stability (Appendix D).

Tail Lake outflow will be re-established along the base of the cut and suitable bedding material will be put in place to ensure channel stability.

South Dam and West Dam

The South Dam and West Dam will be left in place during closure as they will not be retaining any water. No additional closure activities are required.

7.3 Monitoring and Maintenance

7.3.1 Monitoring

Throughout the operational phase of the Project, the containment structures (North, South and West dams) will be subject to rigorous monitoring to evaluate their performance (SRK 2017g). Plant operations staff will carry out daily visual inspections of the TIA structures, taking note of any signs of settlement, unaccounted for drops in water levels, signs of seepage, or any signs of damage or vandalism to instrument clusters. Thermal monitoring and deformation monitoring of the dams will be carried out using geotechnical instrumentation installed within the dams. In addition, water quality monitoring and tailings geochemistry monitoring will be completed (SRK 2017g).

All of the above will be part of the annual reporting in accordance with the relevant water license requirements.

Once the Reclaim Pond has been emptied and the North Dam has been breached, it is expected that routine monitoring will proceed for a period of about five years after which it may be demonstrated that the system is stable and no further action is required.

7.3.2 Maintenance

Throughout the active closure period it is conceivable that some tailings differential settlement could occur. Where necessary maintenance in the form of additional cover or fill material will be allowed for to address any areas of concern.

This report, ***"Hope Bay Project Doris Tailings Management System Phase 2 Design"***, was prepared by SRK Consulting (Canada) Inc.

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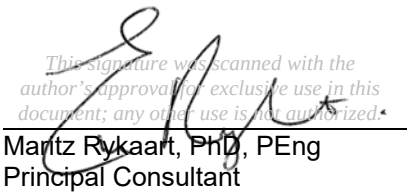
and reviewed by



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All data used as source material plus the text, tables, figures, and attachments of this document have been reviewed and prepared in accordance with generally accepted professional engineering and environmental practices.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

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Appendix A – Engineering Drawings for the Doris Tailings Management System Phase 2 Design

Engineering Drawings for the Doris Tailings Management System Phase 2 Design, Hope Bay Project, Nunavut, Canada

ACTIVE DRAWING STATUS

DWG NUMBER	DRAWING TITLE	REVISION	DATE	STATUS
DTIA-01	Engineering Drawings for the Doris Tailings Management System Phase 2 Design , Hope Bay Project, Nunavut, Canada	A	Nov. 29, 2017	Issued for Discussion
DTIA-02	General Site Plan	A	Nov. 29, 2017	Issued for Discussion
DTIA-03	North Dam Plan View	A	Nov. 29, 2017	Issued for Discussion
DTIA-04	North Dam Profile and Typical Cross Section	A	Nov. 29, 2017	Issued for Discussion
DTIA-05	South Dam Plan and Profile	A	Nov. 29, 2017	Issued for Discussion
DTIA-06	South Dam Typical Section	A	Nov. 29, 2017	Issued for Discussion
DTIA-07	West Dam Plan and Profile	A	Nov. 29, 2017	Issued for Discussion
DTIA-08	West Dam Typical Section	A	Nov. 29, 2017	Issued for Discussion
DTIA-09	South Dam Typical Liner Details	A	Nov. 29, 2017	Issued for Discussion
DTIA-10	Final Tailings Surface	A	Nov. 29, 2017	Issued for Discussion
DTIA-11	Final Tailings Surface Sections FF and GG	A	Nov. 29, 2017	Issued for Discussion

TMAC RESOURCES

srk consulting

PROJECT NO: 1CT022.013-130
Revision A
November 29, 2017
Drawing DTIA-01

N 7556000
N 7557000
N 7558000
N 7559000
E 434000
E 435000
E 436000
Doris-Windy Road
Doris Connector Vent Raise and Acess Road
Doris Camp
Doris Central Vent Raise and Access Road
Doris Creek Bridge
DORIS LAKE
Secondary Road
North Dam
West Dam
Existing Road Alignment
Reclaim Pond
OGAMA LAKE
South Dam
Proposed Road Allignment
Planned Quarry #3

LEGEND

- Proposed Tailings Facility
- Proposed Dam/Dikes
- Existing Facilities
- Existing Doris Camp
- Proposed Road Realignment

NOTES

1. Contours shown at 0.5m Intervals.
2. NAD 83 UTM Zone 13

0m 100 200 300 400 500 600

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
REFERENCE DRAWINGS		A	ISSUED FOR DISCUSSION	IM	EMR	17/11/23
		REVISIONS				

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FILE NAME: 1CT022.004.600.10 - FIGURE 2.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: General Site Plan

DRAWING NO.	SHEET	REVISION NO.
02	2 OF 11	A

P:\01_SITES\Hope Bay\ACAD\2017 Drawings\Doris TIA Phase 2 Design\1CT022.004.600.10 - FIGURE 2.dwg

To Doris Camp

Secondary Road

North Thermosyphon Radiators

North Dam Access Road

Frozen Core Plant Pad

N 7559200

N 7559100

Downstream

Upstream

South Thermosyphon Radiators

North Dam Crest Centerline

FSL at Elev. 33.5m

4H:1V

6H:1V

E 434200

E 434300

E 434400

E 434500

E 434600

0m 10 20 30 40 50

LEGEND

- Dam Area
- North Dam Access Road
- Water

NOTES

1. The co-ordinate system is UTM NAD 83, Zone 13.
2. Contours shown at 0.5 Intervals

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
REFERENCE DRAWINGS						

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FILE NAME: 1CT022.013-130_3.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: North Dam Plan View

DRAWING NO.	SHEET	REVISION NO.
DTIA-03	3 OF 11	A

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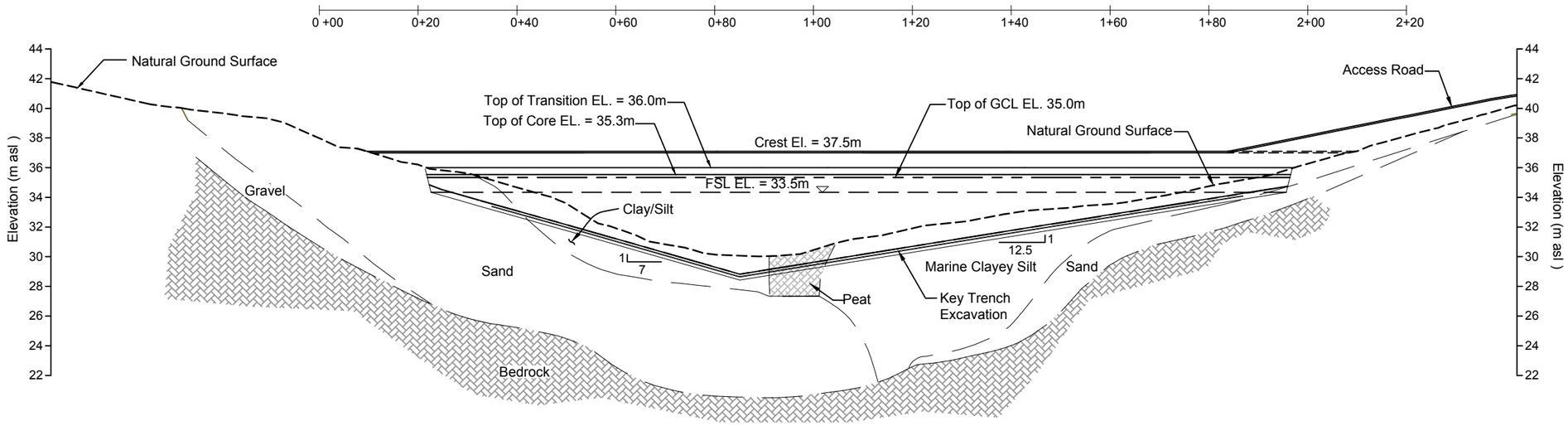
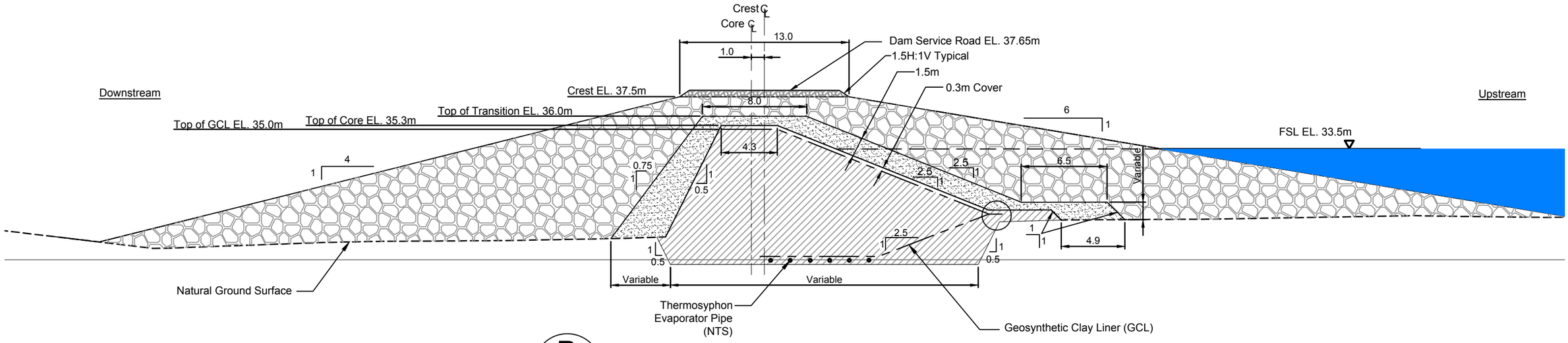


A 04 NORTH DAM PROFILE

(Horizontal Scale) 0m 10 20 30 40

(Vertical Scale) 0m 5 10 15 20



B 03 NORTH DAM TYPICAL SECTION

0m 3 6 9 12

LEGEND

- Natural Ground
- Geosynthetic Clay Liner (GCL)
- Stratigraphic Boundary
- Core Material
- Transition materials
- Run of Quarry (ROQ)
- Surfacing Material
- Bedrock
- Peat
- Water
- Thermosyphon Evaporator Pipe (NTS)

NOTES

1. The subsurface geology has been extrapolated from a series of geotechnical investigations consisting of drill holes, auger holes and shallow test pits. Bedrock contact and geological unit contacts are therefore likely to vary somewhat during final excavation. The Engineer will inspect all excavations and approve the foundation conditions.

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
REFERENCE DRAWINGS		REVISIONS					A	ISSUED FOR DISCUSSION	IM	EMR	17/11/29

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srk consulting

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FILE NAME: 1CT022.013-130_4.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: North Dam Profile and Typical Cross Section

DRAWING NO.	SHEET	REVISION NO.
DTIA-04	4 OF 11	A

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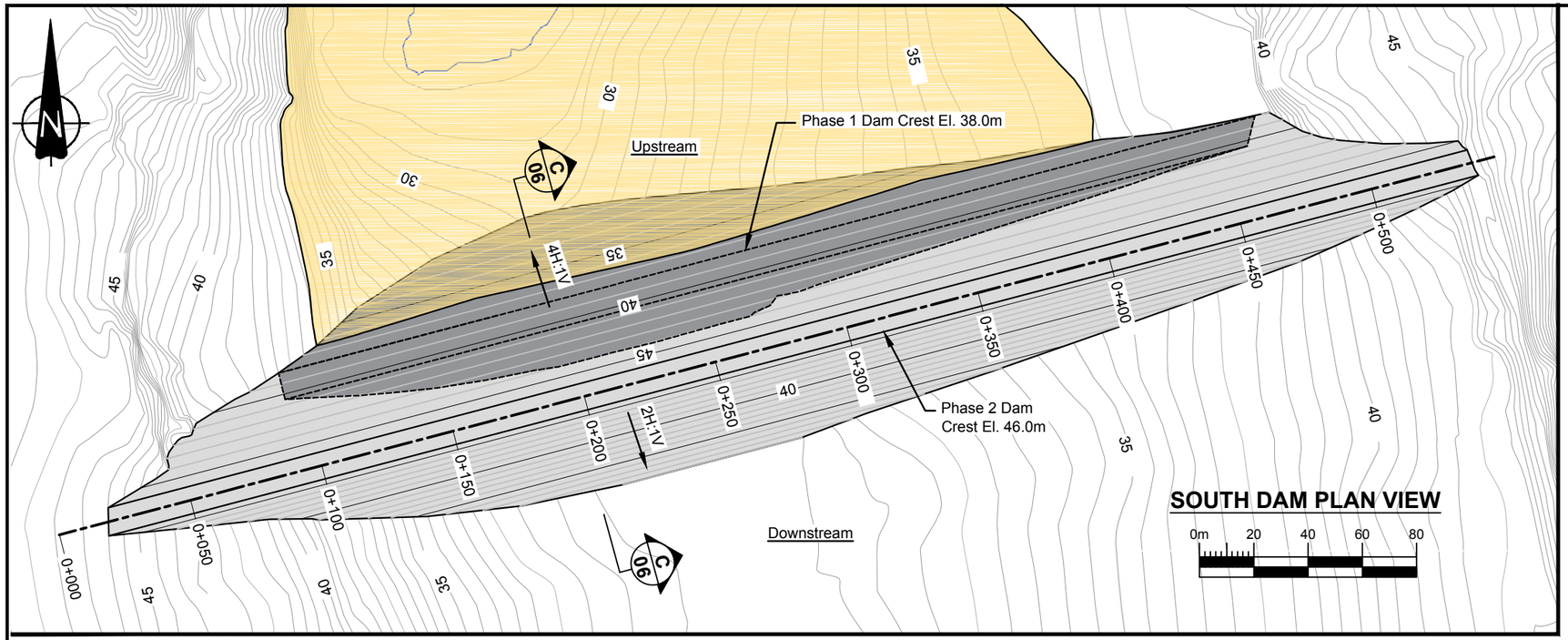


SOUTH DAM PLAN VIEW

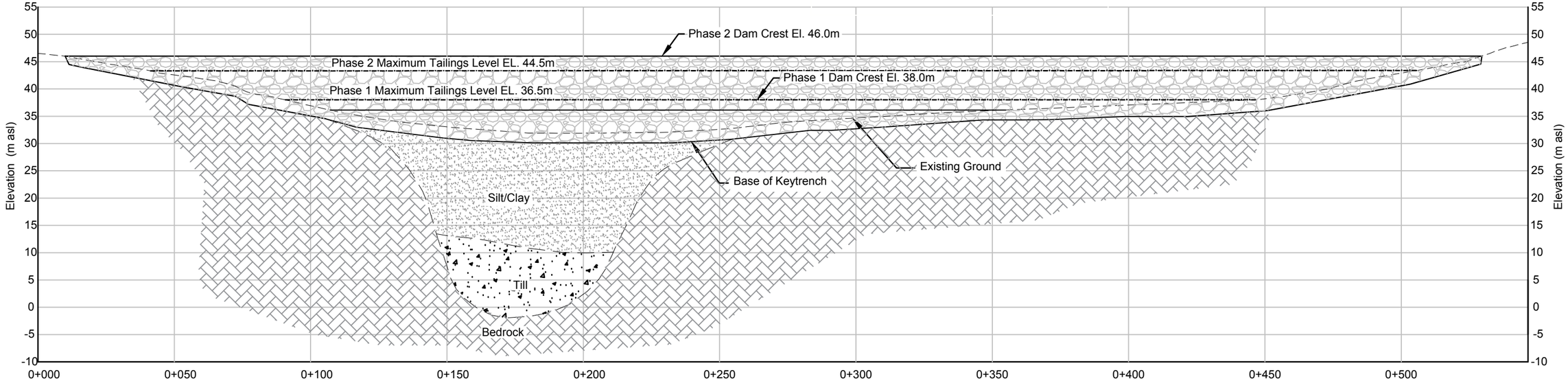
0m 20 40 60 80

LEGEND

- Natural Ground
- Bedrock Contact
- Maximum Tailings Level
- Phase 1 Dam
- Phase 2 Dam
- Run of Quarry (ROQ)
- Silt/Clay
- Till
- Bedrock
- Deposited Tailings (Phase 1)

NOTES

1. All dimensions and elevations are in meters unless stated otherwise.
2. The subsurface geology has been extrapolated from a series of geotechnical investigations. Bedrock contact and geological unit contacts are therefore likely to vary somewhat during final excavation.
3. Contours shown at 0.5m intervals



SOUTH DAM PROFILE

0m 5 10 15 20
Vertical Scale

0m 20 40 60 80
Horizontal Scale

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: South Dam Plan and Profile

DRAWING NO.	SHEET	REVISION NO.
DTIA-05	5 OF 11	A

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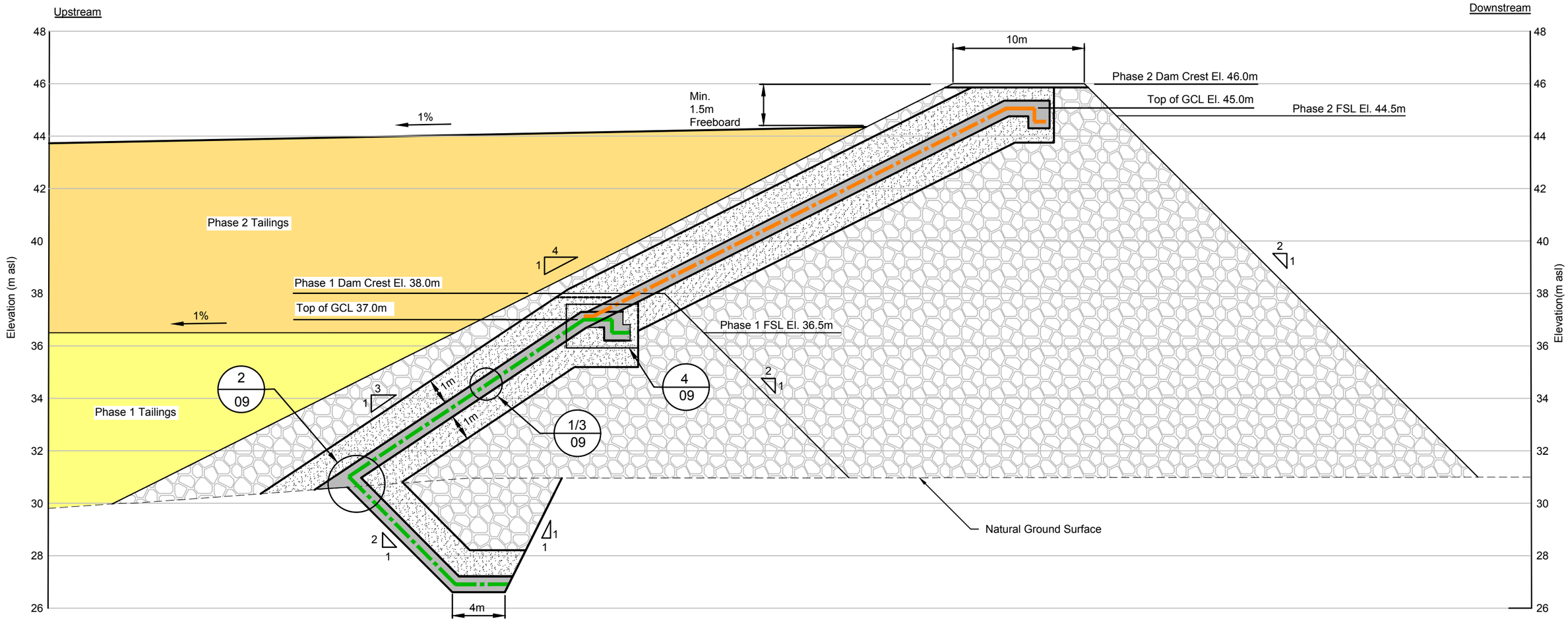
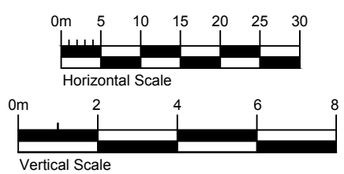


C
05

SOUTH DAM TYPICAL SECTION



LEGEND

- Natural Ground
- Phase 1 Geosynthetic Clay Liner (GCL)
- Phase 2 Geosynthetic Clay Liner (GCL)
- Surface Materail
- Liner Bedding Material
- Transition Material
- Run of Quarry (ROQ)
- Deposited Tailings (Phase 1)
- Deposited Tailings (Phase 2)

REFERENCE DRAWINGS		REVISIONS									
DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
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TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: South Dam Typical Section

DRAWING NO.	SHEET	REVISION NO.
DTIA-06	6 OF 11	A

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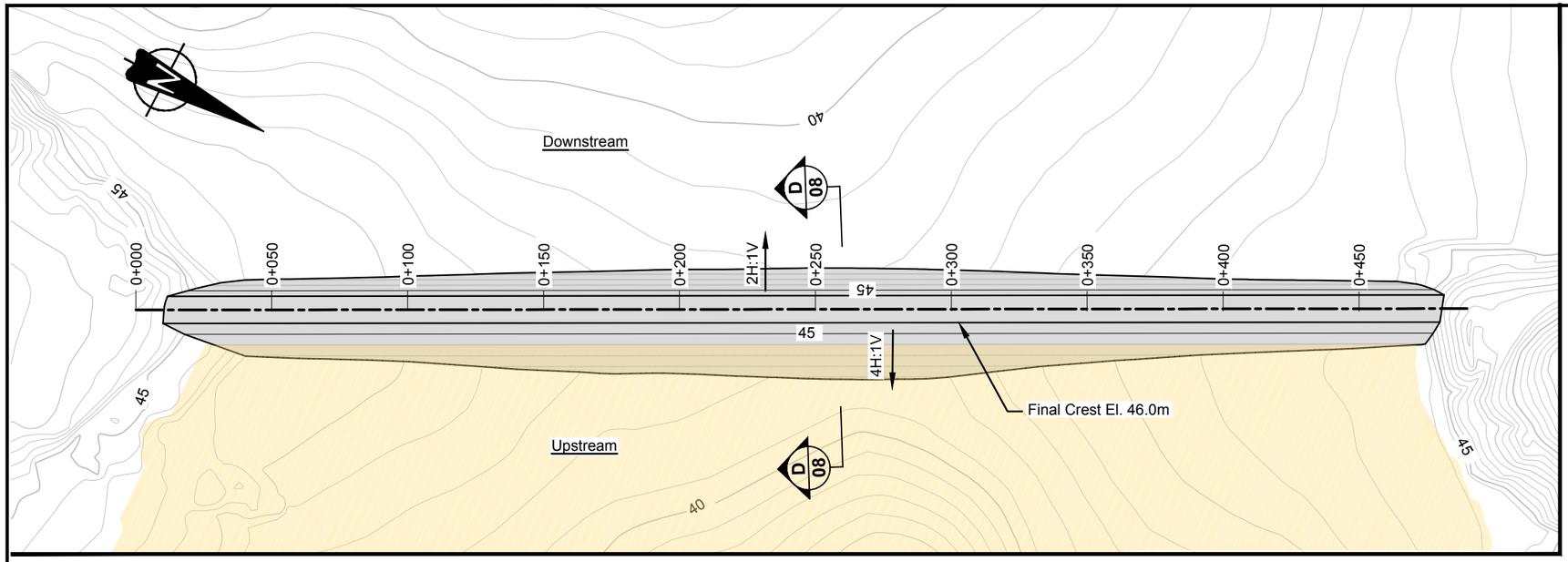
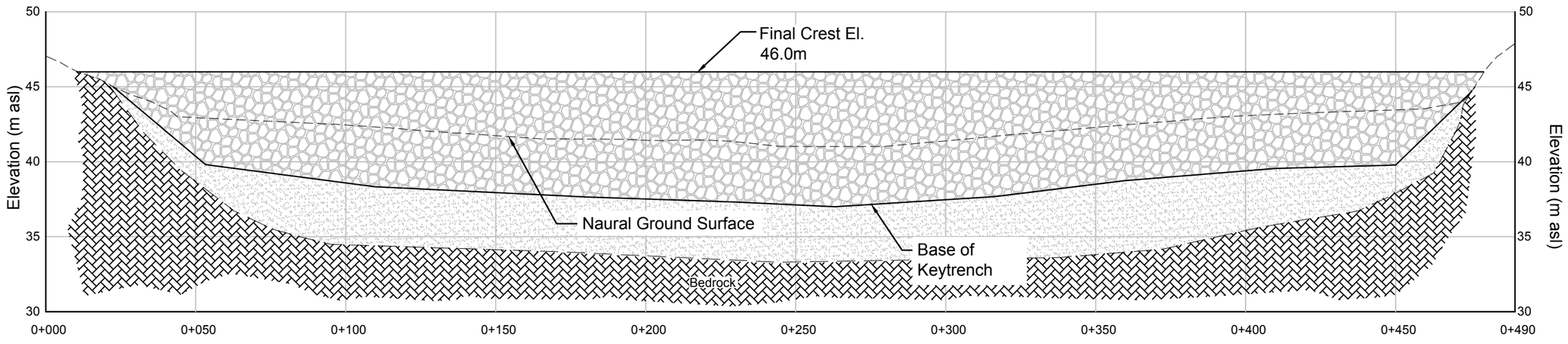


WEST DAM PLAN VIEW

0m 20 40 60 80

1 : 2000



WEST DAM PROFILE

0m 3 6 9 12

Vertical Scale

0m 10 20 30 40 50 60

Horizontal Scale

LEGEND

- Natural Ground
- Inferred Bedrock Contact
- Run of Quarry (ROQ)
- Silt/Clay
- West Dam
- Bedrock
- Deposited Tailings (Phase 2)

NOTES

1. All dimensions and elevations are in meters unless stated otherwise.
2. The subsurface geology has been extrapolated from a series of geotechnical investigations. Bedrock contact and geological unit contacts are therefore likely to vary somewhat during final excavation.
3. Contours shown at 0.5m

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
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FILE NAME: 1CT022.013-130 - FIGURE 7_8.dwg		

MAC RESOURCES

HOPE BAY PROJECT

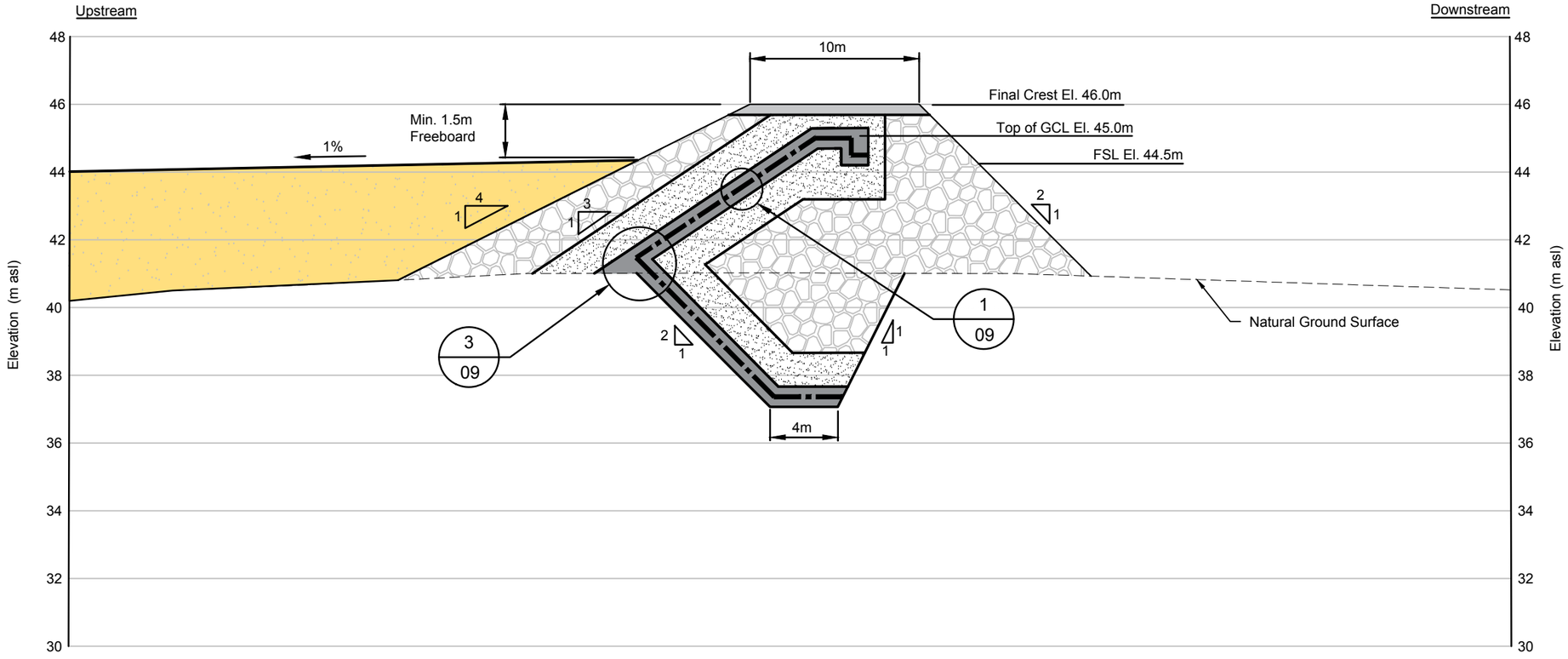
SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: West Dam Plan and Profile

DRAWING NO.: DTIA-07 | SHEET: 7 OF 11 | REVISION NO.: A

P:\01_SITES\Hope Bay\ACAD\2017 Drawings\Doris TIA Phase 2 Design\1CT022.013-130 - FIGURE 7_8.dwg



DD 07

WEST DAM SECTION

0m 1 2 3 4 5 6
Vertical Scale

0m 3 6 9 12
Horizontal Scale

LEGEND

- Natural Ground
- Geosynthetic Clay Liner (GCL)
- Surfacing Material
- Liner Bedding Material
- Transition materials
- Run of Quarry (ROQ)
- Deposited Tailings (Phase 2)

NOTES

1. All dimensions and elevations are in meters unless stated otherwise.

DRAWING NO.	DRAWING TITLE
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NO.	DESCRIPTION	CHK'D	APP'D	DATE
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FILE NAME: 1CT022.013-130 - FIGURE 7_8.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: West Dam Typical Section

DRAWING NO.	SHEET	REVISION NO.
DTIA-08	8 OF 11	A

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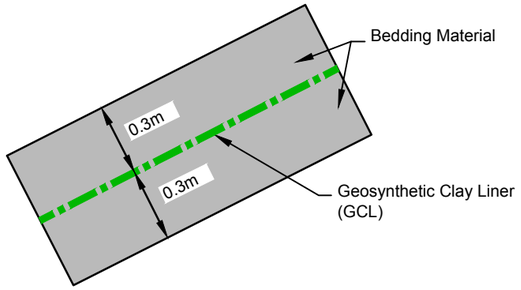
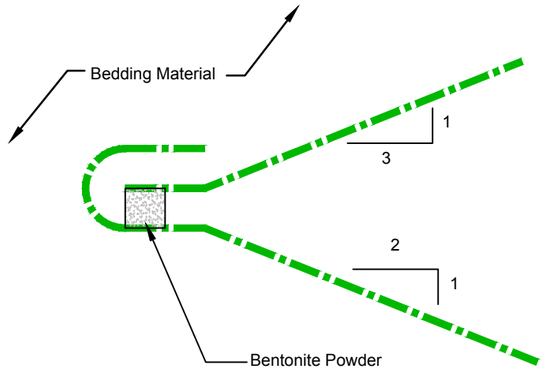


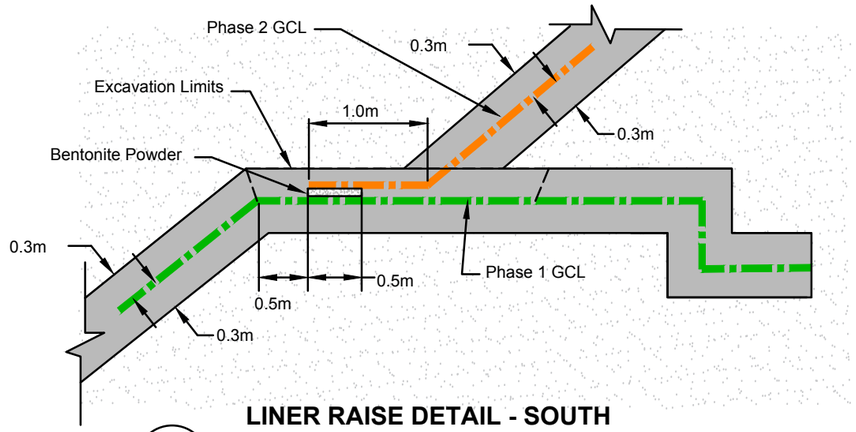
1 / 06/09 **TYPICAL SECTION GCL**

NOT TO SCALE



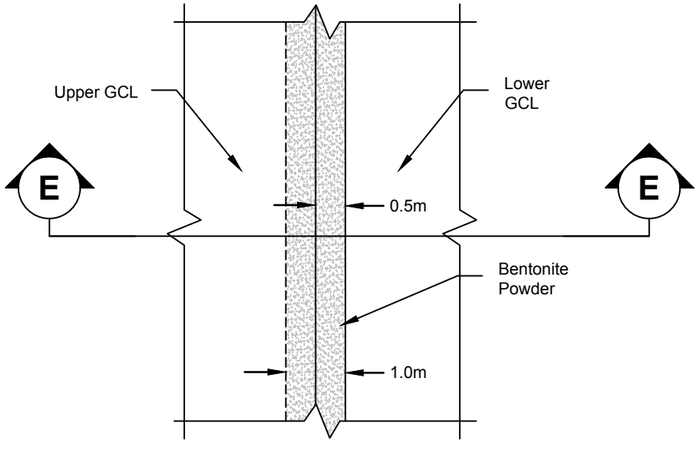
2 / 06 **TYPICAL UPPER AND LOWER GCL JOINT**

NOT TO SCALE



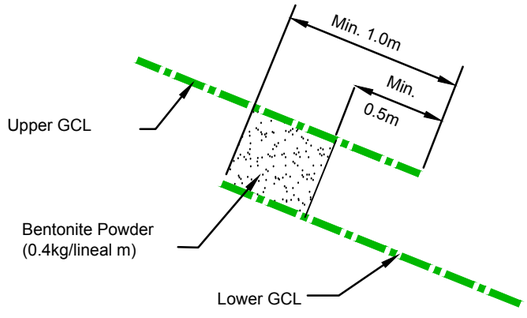
4 / 06 **LINER RAISE DETAIL - SOUTH DAM**

NOT TO SCALE



3 / 06/09 **TYPICAL GCL OVERLAP SEAM DETAIL - PLAN VIEW**

NOT TO SCALE



EE / — **TYPICAL GCL OVERLAP SEAM DETAIL - PLAN VIEW**

NOT TO SCALE

South Dam Quantities

Item	Quantity/Area/Volume	Description
GCL (m²)	14,900 m²	Approximate total surface area(based on Civil 3d surfaces) -no allowance was made for overlaps and wastage
Liner Bedding (m³)	9300 m³	Approximate In-Place Neat-Line Volume (3D volume based on Civil 3d Surfaces) - no allowance has been made for losses and/ or tundra embedment
ROQ (m³)	148,500 m³	
Transition Fill (m³)	32,800 m³	
Key Trench Excavation (m³)	8,000 m³	

West Dam Quantities

Item	Quantity/Area/Volume	Description
GCL (m²)	10,000 m²	Approximate total surface area(based on Civil 3d surfaces) -no allowance was made for overlaps and wastage
Liner Bedding (m³)	6,100 m³	Approximate In-Place Neat-Line Volume (3D volume based on Civil 3d Surfaces) - no allowance has been made for losses and/ or tundra embedment
ROQ (m³)	33,300 m³	
Transition Fill (m³)	14,700 m³	
Key Trench Excavation (m³)	17,000 m³	

LEGEND

- Liner Bedding Material
- Transition Material
- Phase 1 - Geosynthetic Clay Liner (GCL)
- Phase 2 - Geosynthetic Clay Liner (GCL)

NO.	DESCRIPTION	CHK'D	APP'D	DATE
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MAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE: South Dam Typical Liner Details

DRAWING NO.	SHEET	REVISION NO.
DTIA-09	9 OF 11	A

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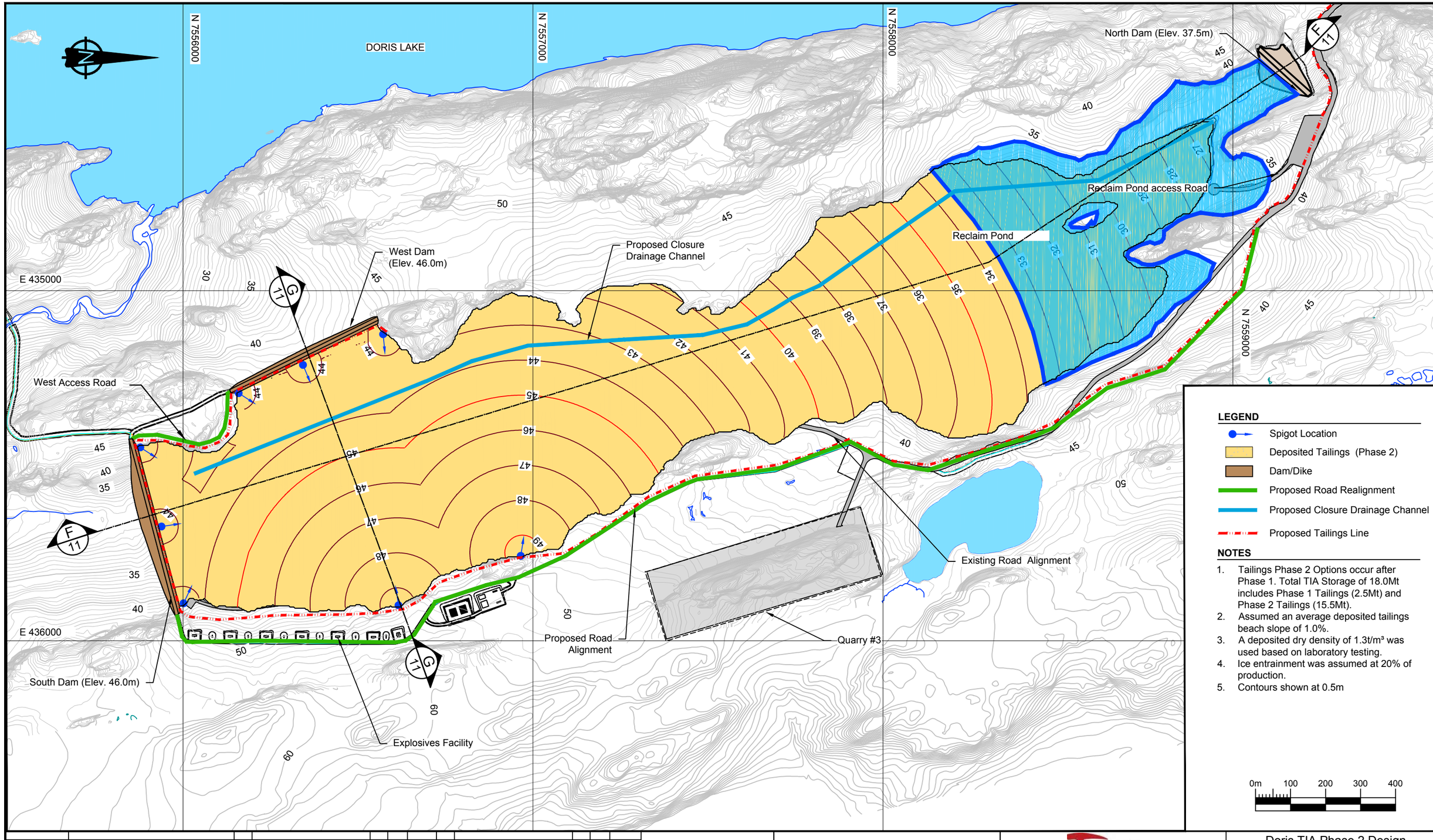

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REVISIONS				

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FILE NAME: 1CT022.013-130_10.dwg		

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE:

Final Tailings Surface

DRAWING NO.	SHEET	REVISION NO.
DTIA-10	10 OF 11	A

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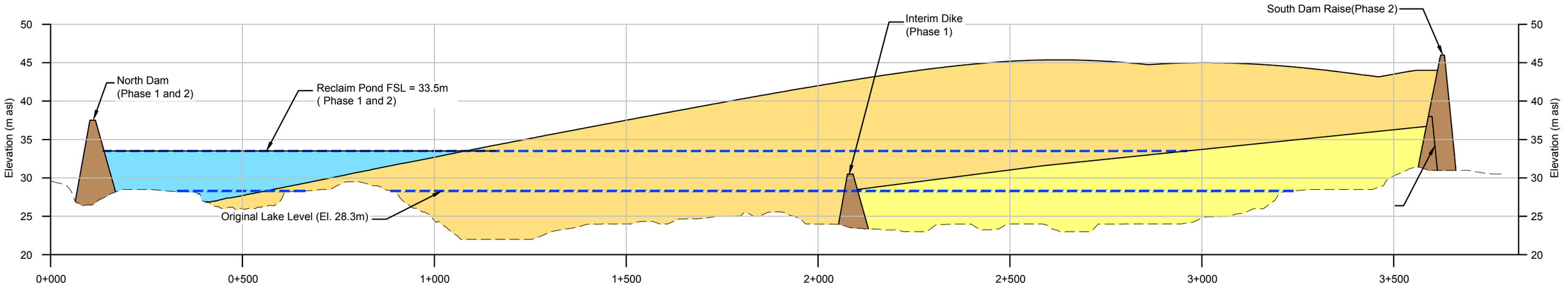
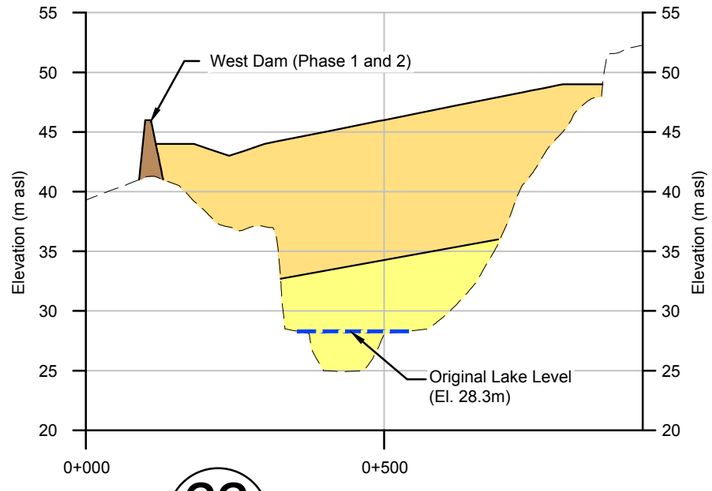


FF
10

SECTION FF

0m 5 10 15 20
Vertical Scale

0m 100 200 300 400
Horizontal Scale



GG
10

SECTION GG

0m 5 10 15 20
Vertical Scale

0m 100 200 300 400
Horizontal Scale

LEGEND

- Natural Ground
- Deposited Tailings (Phase 1)
- Deposited Tailings (Phase 2)
- Dam

DRAWING NO.	DRAWING TITLE	NO.	DESCRIPTION	CHK'D	APP'D	DATE
		A	ISSUED FOR DISCUSSION	IM	EMR	17/11/29

REFERENCE DRAWINGS

REVISIONS

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srk consulting

DESIGN:	DRAWN: NV/TH	REVIEWED: IM
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FILE NAME: 1CT022.013-130 - FIGURE 11.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-130

Doris TIA Phase 2 Design

DRAWING TITLE:
Final Tailings Surface
Sections FF and GG

DRAWING NO.	SHEET	REVISION NO.
DTIA-11	11 OF 11	A

P:\01_SITES\Hope Bay\ACAD\2017 Drawings\Doris TIA Phase 2 Design\1CT022.013-130 - FIGURE 11.dwg

Appendix B – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area Tailings Deposition Plan

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2200–1066 West Hastings Street
Vancouver, BC V6E 3X2

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F: +1.604.687.5532

vancouver@srk.com
www.srk.com

Memo

To:	John Roberts, PEng., Vice President, Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Trevor Podaima, PEng	**Project No:**	1CT002.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Phase 2 Doris Tailings Impoundment Area Tailings Deposition Plan		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix B as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes		

1 Introduction

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

1.1 Phase 1 Tailings Deposition

Phase 1 tailings are deposited sub-aerially in the Doris Tailings Impoundment Area (TIA) approximately 5 km from the Doris Mill (Figure 01). The Doris TIA was formally a natural lake, Tail Lake, but has been delisted in accordance with Schedule II of the Metal Mining Effluent Regulations (MMER).

Tailings containment is provided by three retention structures; a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam); and an interim dike situated at approximately the midpoint of the facility. North Dam construction was completed in 2012 (SRK, 2012) and the South Dam and Interim Dike are scheduled for construction in 2016 or 2017. The final configuration of Phase 1 is provided in Figure 02 and further details associated with this tailings deposition plan are provided in the Doris North Project Tailings Management System Design (SRK, 2015).

1.2 Phase 2 Tailings Deposition

This memo presents an evaluation of different tailings deposition options for Phase 2, assuming a continuation of the use of the Doris TIA and concludes with a presentation of the preferred option. Once the preferred option was chosen, a staged deposition plan at three year intervals (i.e. deposition plans were developed for the end of year 2023, 2026, 2029 and 2032) was developed.

2 Objectives, Operational Criteria and Assumptions

Tailings deposition planning is required to establish the optimal geometric layout of the Doris TIA such that the finished landform is consistent with the operational and final closure objectives of the facility. In addition, the deposition planning provides important operational optimization in terms of establishing where, and for how long, tailings spigots must be situated at specific locations.

Tailings deposition planning was completed using Muck3D - V4.0.3 (MineBridge Software Inc.). Tailings were modeled with an assumed overall average slope for the tailings surface based on past experience with tailings that have similar characteristics and dry density. Hope Bay tailings' properties were based on site specific laboratory test data (SRK, 2016). Tables 1 and 2 summarize the key design criteria adopted for the Phase 2 tailings deposition planning.

Table 1: Tailings Storage Requirement Criteria

Component	Phase	Value	Source
Tailings storage requirement (without ice entrainment)	1	1.9 Mm^3 (2.5 Mt)	Quantity based on TMAC mine plan; volume conversion based on a dry density of 1.29 t/m^3.
	2	11.6 Mm^3 (15.1 Mt)	Quantity based on TMAC mine plan; volume conversion based on dry density listed below in this table. Quantity does not include Phase 1 tailings at 2.5 Mt.
	1 + 2	13.5 Mm^3 (17.6 Mt)	Sum of Phases 1 and 2. (Value used in assessment to determine preferred TIA Option and deposition staging plan Figures 03 to 09)
Tailings production	1 + 2	2017 (1,092 tpd) 2018 (1,852 tpd) 2019 (2,041 tpd) 2020 & 2021 (3,000 tpd) 2022 (3,200 tpd) 2023 to 2031 (3,600 tpd) 2032 (1,597 tpd)	Supplied by TMAC.
Tailings production period	1	3 years, 8 months (to end of August 2020)	Supplied by TMAC.
	2	12 years, 4 months (to end of 2032)	
	1 + 2	16 years	Based on total tailings volume processed at the stated production rates.
Ice entrainment allowance	1	0.4 Mm^3 (20% by volume)	Contingency allowance based on engineering judgement and case studies reported by BGC (2003).
	2	2.3 Mm^3 (20% by volume)	
	1 + 2	2.7 Mm^3	
Total storage requirement (with ice entrainment)	1	2.3 Mm^3	Sum of tailings storage requirement and ice entrainment allowance. (Value used in assessment to determine preferred TIA Option and deposition staging plan, Figures 03 to 09).
	2	13.9 Mm^3	
	1 + 2	16.2 Mm^3	

Table 2: Tailings Deposition Planning Design Criteria

Component	Phase	Value	Source
Minimum pond volume during operations	1	n/a	Reclaim pond always sufficiently large so as to not become a limiting component.
	2	790,000 m^3 plus a 2.0 m pond depth above this volume (Years 2020 to 2030) 270,000 m^3 plus a 2.0 m pond depth above this volume (Years 2031 and 2032)	Quantity based on balance between total reclaim requirement during winter conditions (assumed six month duration) and avoidance of exceeding FSL. An additional 2.0 m pond depth allows for ice entrainment (SRK, 2007). In years 2031 and 2032, the FSL will be lowered to accommodate the IDF and therefore the available reclaim was lowered.
Full Supply Level (FSL)	1	33.5 m	(SRK, 2015)
	2	33.5 m (Years 2020 to 2030) 32.5 m (Years 2031 and 2032	FSL elevation based on final configuration of TIA and the minimum operational pond volume. In the last 2 years of operation, the FSL needs to be lowered to 32.5 m to accommodate the IDF. Overall water management will be required to maintain pond levels (i.e. discharge will be required throughout tailings deposition stages to reduce the initial water volume in Tail Lake, SRK 2017a).
Design storm event	1	182 mm (638,256 m^3)	TIA with “HIGH” Consequence Classification. Inflow Design Flood (IDF) 1/3 between 1:1000 year event and probable maximum flood. The value in the Doris Tailings Management System Design Report has been updated for this study (SRK, 2017b). TIA included a spillway that was sized to safely pass the IDF.
	2	257 mm (838,478 m^3)	TIA with “EXTREME” Consequence Classification. Phase 2 design does not include a spillway and TIA was designed to store the Probable Maximum Flood (PMF) (SRK, 2017b). Value includes climate change. This storage capacity is over and above the operating pond volume.
Maximum pond volume	1	n/a	Reclaim pond sufficiently large so as not to become a limiting component.
	2	1.9 Mm^3 (Elev. 35.2 m)	This is the maximum pond volume at closure, which includes the FSL + PMF.
Deposited tailings dry density	2	1.3 t/m^3	From laboratory testing (SRK, 2016).
Tailings beach slope	1 + 2	1.0%	This is an assumed average slope based on engineering judgement considering SRK’s experience with tailings of similar properties. No allowance for steepened slope as the beach transitions into the pond.
South Dam crest elevation	1	38.0 m	Value set as a function of the deposition planning and is based on the preferred TIA Option.
	2	46.0 m	
West Dam crest elevation	2	46.0 m	Value set as a function of the deposition planning and is based on the preferred TIA Option.
South Dam normal freeboard[1]	1 + 2	1.5 m	Conservative allowance considering best practice and to accommodate a closure cover. No water will ever pond against this dam and therefore the freeboard will never actually be required (SRK, 2015).
South Dam minimum freeboard[2]	1 + 2	n/a	No water will ever pond against this dam and therefore the freeboard will never actually be required.
West Dam normal freeboard[1]	2	1.5 m	Conservative allowance considering best practice and to accommodate a closure cover. No water will ever pond against this dam and therefore the freeboard will never actually be required.
West Dam minimum freeboard[2]	2	n/a	No water will ever pond against this dam and therefore the freeboard will never actually be required.
North Dam normal freeboard[1, 3]	1	FB = 1.8 m FSL = Elev. 33.5 m Final FB = Elev. 35.3 m	Doris Tailings Management System Design Report (SRK, 2015).
	2	FB = 1.2 m FSL = Elev. 33.5 m Final FB = Elev. 34.7 m	Value determined in accordance with Canadian Dam Association Guidelines (CDA, 2014) and is based on wave set-up and run-up calculations (SRK, 2017b).
North Dam minimum Freeboard[2, 3]	1	n/a	Spillway designed to safely pass IDF (SRK, 2015).
	2	FB = 1.0 m FSL = Elev. 33.5 m FSL+IDF = Elev. 35.2 m Final FB = Elev. 36.2 m	Value determined in accordance with Canadian Dam Association Guidelines (CDA, 2014) for TIA with “HIGH” Consequence Classification. Based on IDF, wave set-up and run-up calculations (SRK, 2017b).
	2	FB = 1.0 m FSL = Elev. 33.5 m FSL+IDF = Elev. 35.6 m Final FB = Elev. 36.6 m	Value determined in accordance with Canadian Dam Association Guidelines (CDA, 2014) for TIA with “EXTREME” Consequence Classification. Based on IDF, wave set-up and run-up calculations (SRK, 2017b).
Tailings discharge method	1 + 2	Single point spigot	Goal was to simplify deposition plan as far as practical and still achieve the desired tailings deposition strategy.

Notes:

1. Normal Freeboard = FSL + (wind setup and wave run-up).
2. Minimum Freeboard = FSL + IDF + (wind setup and wave run-up).
3. FSL will be lowered from Elev. 33.5 m to 32.5 m in the last two years of operation (2031 and 2032).

3 Operation Philosophy

3.1 Tailings Deposition

3.1.1 Phase 1

Phase 1 tailings deposition is described in the Doris Tailings Management System Design (SRK, 2015). It entails discharging tailings from six locations starting at the South Dam, progressively moving the tailings beach towards the Interim Dike. The deposition strategy will ensure creation of a landscape that allows for a free-draining surface towards the reclaim pond at closure. Use of a single spigot at any given time was done to simplify as much as practical the operational component of the strategy. This final tailings configuration at the start of Phase 2 is illustrated in Figure 02. This strategy prevents water from ponding against the geosynthetic lined frozen foundation South Dam, and as a result after tailings freeze-back has occurred, there is no risk of seepage from this Dam. The Interim Dike acts to retain tailings solids within a confined area so as to minimize the exposed tailings surface at closure. At all times the frozen core North Dam will act as a water retaining structure holding back a sufficiently large reclaim pond to supply the operational water demands of the Doris and Madrid process plants.

3.1.2 Phase 2

Phase 2 tailings deposition will rest on the same basic principles. As far as possible, discharge should be limited to using a single spigot at any given time, and deposition points should be staged starting from the South Dam raise moving north creating a free-draining landscape. Should any additional containment dams be required, they should be designed and operated in accordance with the same principles as the South Dam.

Due to the geometry of the TIA, construction of further interim dikes is not practical and is therefore not a requirement. Since the tailings volume is sufficiently large, it will occupy most of the TIA surface area between the North and South dams, and therefore are no real advantages to constraining the tailings surface.

Consideration can be given to discharging tailings from the North Dam, to create a beach such that it no longer has to perform as a water retaining structure.

Best Management Practices will be adopted to schedule seasonal changes in discharge locations to minimize ice buildup and possibly permanent entrapment of ice within the tailings mass. The deposition planning presented in this memo does not present complete details of such seasonal operations; seasonal operations will be developed and presented in an Operations, Maintenance and Surveillance (OMS) Manual. This OMS Manual will be prepared by TMAC in accordance with the requirements under the mine's Water Licence.

3.2 Reclaim Pond Management

The key criteria used for reclaim pond management is that:

- The FSL should not be exceed under normal operating conditions;
- Where possible six months of reclaim water should remain available in October to provide water supply through winter; plus
- 2.0 m minimum water cover above the reclaim demand to accommodate ice (SRK, 2007). This ice allowance is considered to be conservative as it is based on historical records of ice thickness for Tail Lake, which is essentially a stagnant water body. During operation of the TIA, the pond will be continually circulated as it will be used as reclaim for the process plant.

This criteria ensures that the North Dam liner at elevation 35.0 m is not overtopped and the water requirement from fresh water sources in minimized by aiming to have enough reclaim water available during winter operations. Since the area has a net positive water balance, there will be a requirement to discharge water annually from the Doris TIA to Roberts Bay. As the pond volume decreases with deposition of tailings, in the last two years of operations there may be a requirement to draw water from Doris Lake to maintain water supply to the operations during winter.

The TIA water level is expected to fluctuate, but will typically vary between the normal water level of 28.3 m and the FSL of 33.5 m.

Tailings deposition modeling has been done assuming constant tailings beach slopes of 1%. It is however recognized that where the beach transitions to the reclaim pond, the beach angle could steepen to between 4% and 8%. By assuming a shallow beach slope, the available reclaim pond is likely oversized (i.e. conservative) since there is less available pond depth.

4 Deposition Options

Five tailings deposition options were considered for Phase 2 tailings disposal into the Doris TIA. The details of these are described in the following sections and are illustrated in Figures 03 through 07. Table 3 also provides a concise summary of the key advantages and disadvantages of these options.

4.1 Option 1

This option aims to minimize the exposed tailings surface area at closure while maximizing the available reclaim pond without requiring any raise to the North Dam. The tailings deposition plan requires construction of three saddle dikes along the west flank of the facility, a new interim dike situated upstream of Tail Lake Bay, a 12.5 m raise to the South Dam to crest elevation 50.5 m, and 19 spigot locations. There will be no change to the North Dam; however, since it will retain water for the life of the Project and will exceed its original design life of 25 years (SRK, 2007), it may have to be retrofitted with vertical thermosyphons to extend its operational design life.

Tailings deposition will initially occur from the raised South Dam and subsequently progress along the east and west flanks, including the saddle dikes of the Doris TIA, forcing the reclaim pond against the new interim dike (Figure 03). The new interim dike will be designed to retain tailings, but not water, which will result in a final reclaim pond situated between the North Dam and the interim dike. To access the spigot locations, approximately 2.4 km of the existing access road along the east flank of the Doris TIA will need to be relocated, and an additional 1 km of an all-weather road will have to be constructed along the west flank of the Doris TIA.

The final tailings landform is a “U Shape” configuration with a slight depression along the center of the TIA. In this option the new interim dike has a crest elevation of 50.5 m, with a freeboard of 1.5 m. The final tailings elevation immediately upstream of the new interim dike will range between 44.0 and 49.0 m. At closure, the North Dam will be breached allowing the water in the reclaim pond to return to its original elevation of 28.3 m. The new interim dike will also be breached at its low spot of 44.0 m, which leaves a transition of 15.7 m for drainage water from the TIA surface to the former reclaim pond, requiring a substantial permanent engineered drop structure.

4.2 Option 2

This option aims to minimize the need to raise any retaining structures, as well as downgrade the need for the North Dam to continue to function as a water retaining dam. To achieve that, tailings deposition is done from the raised North Dam creating a beach which will keep the pond at least 200 m from the dam (Figure 04). Tailings deposition will initially occur from three spigot locations along the raised North Dam, which will push the reclaim pond south and towards the center of the Doris TIA. Subsequently, tailings deposition would occur from the three spigots situated along the raised South Dam and then continue north along the east and west flanks of the facility.

To accommodate this deposition strategy the North Dam has to be raised 4.5 m to a crest elevation of 42.0 m, and the South Dam has to be raised 6.5 m to a crest elevation of 44.5 m. To access the 15 spigot locations, approximately 3.4 km of the existing east access road would require relocation and an additional 1.9 km of an all-weather road will have to be constructed along the west flank of the Doris TIA.

To accommodate closure without a permanent pond, a drainage channel must be constructed through tailings, from the lowest elevation in the reclaim pond of 35.0 m to the North Dam, cutting through a breach in the raised North Dam, for a distance of about 1 km. By restricting the channel to the original Doris TIA spill elevation of 28.3 m, the maximum grade of this channel would be less than 1%, which is not ideal.

Closure with a pond in place in perpetuity would require construction of a PMF spillway with an invert level at elevation 39 m through approximately 200 m of tailings and across the North Dam.

4.3 Option 3

This deposition plan is similar to Option 1; however, the new interim dike is situated downstream of Tail Lake Bay. The intent of this configuration was to lower the height of the new interim dike, reduce the footprint of the reclaim pond, and provide a closure configuration that will be favorable over Options 1 and 2.

This deposition plan requires construction of a new 14 m high interim dike with a crest elevation of 39.5 m, a 2 m raise to the North Dam for a final crest elevation of 39.5 m, a 5.5 m raise to the South Dam to an elevation 43.5 m and 21 spigot locations. Tailings deposition will initially occur from the South Dam and subsequently progress along the east and west flanks of the TIA, forcing the supernatant pond against the new interim dike and ultimately between the interim dike and the North Dam (Figure 05). Approximately 2.4 km of the existing access road along the east flank of the TIA will need to be relocated and an additional 2.45 km of an all-weather road will have to be constructed along the west flank of the TIA.

The final tailings landform will be a relatively flat surface that drains towards the new interim dike. The closure strategy will be similar to that for Option 1, in that the North Dam will be breached. The final tailings elevation immediately upstream of the new interim dike will be at approximately 37.0 m, which means that at closure the elevation difference between the tailings surface and the natural outflow elevation of the TIA will be 8.7 m once the North Dam is breached at elevation 28.3 m. This elevation difference will require construction of a permanent engineered drop structure.

4.4 Option 4

This option builds on the positive components of Option 3, but attempts to lower the height of the new interim dike and eliminate a raise to the North Dam. To achieve this, a dam (West Dam) is required along the west flank of the facility, the South Dam needs to be raised 8.5 m to crest elevation of 46.5 m and the exterior and interior tailings spigots raised to elevations 45 and 44.5 m, respectively (i.e. South Dam and spigot elevations are 3 m higher than Option 3). There are a total of 18 spigot locations, three less than Option 3, as the spigots needed to be shifted further south to free up enough storage capacity to accommodate the required pond storage volumes (Figure 06).

Approximately 3.5 km of the existing access road along the east flank of the TIA will need to be relocated and an additional 1.6 km of an all-weather road will have to be constructed along the west flank of the TIA. In total, Option 4 will require an additional 250 m of road construction compared to Option 3. The new interim dike will have a crest elevation at 35.0 m, which is 1.5 m higher than the FSL at elevation 33.5 m and the final tailings elevation immediately upstream of the interim dike will range between 32 and 33 m. Therefore, at closure the elevation difference between the tailings surface and the natural outflow elevation of the TIA will be between 3.7 m and 4.7 m once the North Dam is breached at elevation 28.3 m.

4.5 Option 5

This deposition plan builds on the benefits of Option 4, but attempts to lower the height of the South Dam and West Dam, eliminate the new interim dike and reduce the amount of tailings spigots. To achieve this, two spigots with a discharge elevation at 49.5 m and 48.5 m will need to be situated along the east flank of the TIA in conjunction with six additional spigots at elevation 44.5 m situated along the raised South Dam and new West Dam (Figure 07).

Tailings deposition will commence from the raised South Dam, then from the West Dam and lastly from the east flank of the TIA. This deposition sequence will generate a small swale-like feature between the West Dam and the tailings deposited from the east flank, which will enable supernatant and runoff to drain towards the North Dam. Such configuration results in a South Dam and West Dam at crest elevation 46.0 m and a total of eight spigots (eight less than Option 4).

Approximately 3.5 km of the existing access road along the east flank of the TIA will need to be relocated and an additional 0.4 km of an all-weather road will have to be constructed along the west flank of the TIA. The toe of the final tailings surface will tie into the original Tail Lake bathymetry at elevation 26.5 m, approximately 300 m upstream of the North Dam. Therefore, at closure there will not be an elevation difference between the downstream toe of the tailings surface and the natural outflow elevation of the TIA once the North Dam is breached at elevation 28.3 m.

For Option 5, the reclaim pond continuously decreases in size as the tailings beach advances north. The resultant effect is that in the final deposition years of the mine life, the minimum pond volume (6 months of reclaim plus 2 m ice allowance) would have an elevation that is above the FSL. Therefore, to ensure that the FSL is not exceeded, the TIA pond will be reduced and if additional water is required over winter, it will be drawn from Doris Lake. It is likely the ice thickness over winter will be less than 2 m thick, which will provide some operational flexibility. This can be monitored and confirmed during previous deposition phases.

The North Dam configuration is considered to have a HIGH consequence classification where the IDF is defined to be 1/3 between the 1,000 year event and the PMF (CDA, 2014). However, since Phase 2 was designed without a spillway, the TIA must contain the full PMF event, which corresponds to a dam with an EXTREME consequence classification. For Option 5, the IDF and final freeboard elevation for both HIGH and EXTREME consequence classifications is 35.2 and 36.2 m, and 35.6 and 36.6 m, respectively (Table 2). The North Dam has a liner to elevation 35.0 m, and cannot accommodate the IDF for either the HIGH or EXTREME consequence classifications as it exceeds the liner crest by 0.2 and 0.6 m, respectively. However, this only becomes an issue during the last two years of tailings deposition and can be accommodated by lowering the FSL from 33.5 to 32.5 m.

The North Dam has a final crest elevation of 37.5 m, so it can readily accommodate the wave set up and run up elevations for both the HIGH and EXTREME consequence classifications. The extra freeboard acts as thermal insulation above the frozen core/liner crest and is an allowance for long-term creep deformation, based on the original 25 year design life of the facility. For the purpose of this assessment it has been assumed that the North Dam can be retrofitted with vertical thermosiphons to extend its design life and arrest creep deformation such that a raise of the North Dam is not required.

Table 3: Advantages and Disadvantages of Phase 2 Tailings Deposition Options

Deposition Option	Advantages	Disadvantages
Option 1	• Smaller tailings footprint than all options. • Additional area downstream for expansion. • More deposition points may provide greater operational flexibility to manage ice entrainment. • Does not require a raise to the North Dam. • Closure consistent with Phase 1	• More spigot locations compared to all other options with the exception of Option 3, thus more complex to operate. • Will require a longer tailings line to reach the west side of the TIA compared to all options. • More containment structures than all options. • Requires a significant fill volume for containment structures and access roads. • High differential elevation between final tailings surface and the normal water level in the TIA at closure. • Largest new interim dike structure of all options. Significant structure with a height of approximately 25 m. May be perceived as an unnecessary risk when compared to other options.
Option 2	• Ranks second out of all options for the least amount of spigots. • Ranks second out of all options for tailings line length. • Is similar to Option 5 in regards to total embankment volume, which is less than all other options. • Does not require a new interim dike or West Dam(s). • No water retaining dams as all containment structures will have substantial frozen tailings beaches.	• Requires a 4.5 and 6.5 m raise to the North Dam and South Dam, respectively. • Largest tailings footprint of all options. • Requires the largest amount of road re-alignment and new road construction of all options. • May be more susceptible to dust generation due to larger tailings footprint. • Very difficult to construction closure cover compared to all options. • Requires a large pond surface and pond volume to achieve water management criteria (Table 1). • Small amount of spigot points offer limited flexibility to manage ice entrainment; however, additional spigots could be added if required.
Option 3	• Is similar to Options 4 and 5 in regards to tailings footprint. • Final tailings surface will be easier to reclaim when compared to Options 1 and 2. • Does not require a west dam(s).	• Requires a 2.0 and 5.5 m raise to the North Dam and South Dam, respectively. • Requires the largest number of tailings spigots. • High differential elevation between final tailings surface and the normal water level in the TIA at closure. • Large interim dike structure (approximately 14.0 m in height).
Option 4	• May be perceived to have a more natural final surface at closure than Option 5. • Final surface may be easier to reclaim when compared to Option 5. • Is similar to Options 3 and 5 in regards to tailings footprint.	• Requires a small west dam along west flank of TIA. • Requires more tailings line than Option 5. • Requires more tailings spigots than Options 2 and 5. • Requires additional embankment construction and fill volume compared to Option 5. • Requires additional road re-alignment and new road construction compared to Option 5. • Moderate differential elevation between final tailings surface and the normal water level in the TIA at closure. • Small amount of spigot points offer limited flexibility to manage ice entrainment.
Option 5	• Does not require a raise to the North Dam. • Does not require a new interim dike. • Requires an 8.0 m raise to the South Dam, which is less than Options 1 and 4. • Requires the least amount of spigots, significantly less than all options. • Requires the least amount of tailings line. • Requires the least amount of embankment fill with the exception of Option 2. The next closest option is Option 4, which requires approximately 100,000 m^3 more fill. • Requires the least amount of road re-alignment and new road construction. • No differential elevation between final tailings surface and the normal water level in the TIA at closure.	• Requires a small west dam along west flank of TIA. • The two spigots situated at the east flank of the TIA is above the crest elevation of the South Dam and saddle dike and stringent monitoring would be required, since spigot elevations are typically lower than the crest elevation of containment structures. • Potential to trap supernatant water between the South Dam and tailings deposition surface, risking the performance of the dam. • May be more susceptible to dust generation due to the higher tailings elevation. • Small amount of spigot points offer limited flexibility to manage ice entrainment. • Smallest supernatant pond at closure, which offers limited operation flexibility during final stages of tailings deposition.

5 Preferred Deposition Plan

Based on the evaluation presented in the preceding section and the summary of advantages and disadvantages listed in Table 3, tailings deposition Option 5 was selected as the preferred solution. It allows for creation of the desired landform using a number of spigot points that enables operational flexibility, while avoiding the need to raise the North Dam and construct a new interim dike. Furthermore, less discharge pipeline and surface infrastructure is required for Option 5.

While the higher tailings surface area along the east flank is a negative, this is offset by the fact that no new interim dike will be required and at closure there will be no differential elevation between the tailings surface and the normal water level in the TIA. The Tailings OMS Manual will have detailed monitoring check lists, triggers and protocols that the mine operations staff can use as a guideline to safely discharge tailings from the higher spigot locations.

Maintaining the North Dam as a water retaining structure beyond its original design life will likely require some retrofitting of the structure using vertical thermosiphons, and ongoing performance monitoring for the life of the structure. If for any reason, the North Dam starts to show undue signs of distress as a result of the extended design life, the deposition strategy can be changed to Option 2, such that a tailings beach is created upstream of the North Dam. The closure strategy will subsequently have to be revised, as described, but that is not considered a fatal flaw and the post-closure environmental effects will be unchanged.

In regards to the potential operational constraint during the last two years of operations (2031 and 2032), where the IDF exceeds the liner crest elevation at the North Dam; contingencies can be put in place well in advance to ensure the safe operation of the facility.

Two sections that illustrate the typical surface profile of the preferred tailings deposition plan is presented in Figure 08. Cross section A-A’ which is a North-South section, shows that the overall tailings surface generally slopes from the South Dam towards the North Dam. Similarly, section B-B’ which is an East-West section, shows that the final drainage swale will be along the western flank of the TIA.

6 Deposition Staging (Every 3 Years)

A staged tailings deposition plan has been prepared for the Phase 2 ore expected to be processed at Doris and Madrid (15.1 Mt). Figure 09 presents the staged tailings depositional surface for every three years of tailings deposition up to year 16 (i.e. 2017 to 2032). The details are summarized in Table 4. Discharge durations were based on the average production rates provided in Table 1 plus ice entrainment (20% of tailings volume) for all years of deposition. The total storage requirement inclusive of ice entrainment equals 13.9 Mm^3.

Table 4: Summary of Tailings Deposition Details per Three-Year Period

Period	Active Spigots (Elevation)	Cumulative Discharge Volume (1)	Active Deposited Tailings Surface Area (km^2)(2)	Previous Tailings Surface Area not Impacted by Active Deposition (km^2)(3)	Total Tailings Surface Area (Cumulative - km^2)
Year 2023	Spigot #1 (44.5 m) Spigot #2 (44.5 m) Spigot #3 (44.5 m) Spigot #4 (44.5 m)	3.1 Mm^3 / 4.0 Mt	0.9	0	0.9
Year 2026	Spigot #5 (44.5 m) Spigot #6 (44.5 m) Spigot #7 (46.5 m)	6.1 Mm^3 / 7.9 Mt	1.16	0.14	1.3
Year 2029	Spigot #7 (49.5 m) Spigot #8 (45.5 m)	9.1 Mm^3 / 11.9 Mt	1.4	0.1	1.5
End of Phase 2 (Year 2032)	Spigot #8 (48.5 m)	11.6 Mm^3 / 15.1 Mt	1.14	0.56	1.7

Notes:

1. Cumulative discharge volumes are tailings only and do not include ice entrainment volumes.
2. The deposited tailings surface area (shown in yellow in Figure 09) is the resultant surface area from active tailings deposition for the period indicated.
3. The previous tailings surface area (shown in grey in Figure 09) is the exposed tailings surface that is not covered by active deposition.

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Figures

N 75□0000
N 7559000
N 7558000
N 7557000
N 755□000
N 7555000
E 434000
E 435000

Doris Creek Bridge
North Dam
Doris Camp
Reclaim Pond
FSL (El. 33.5m)
Doris Connector
Vent Raise and
Acess Road
Doris Central Vent
Raise and Access
Road
□uarry □3
DORIS LAKE
West Dam
South Dam
OGAMA
LAKE

0m 100 200 300 400 500 □00

NOTES

1. Contours shown at 0.5m Intervals.
2. NAD 83 UTM □one 13

LEGEND

- Spigot Location
- Proposed Tailings Facility
- Proposed Dam/Dikes
- Proposed Tailings Topography

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SRK JOB NO.: 1CT022.004.□00.010

FILE NAME: HB PH2 DEPOSITION FIGURE 1.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

GENERAL SITE PLAN
WITH OPTION 5

DATE:	APPROVED:	FIGURE:
201□/10/2□	TPP	01

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 1.dwg

North Dam (Elev. 37.0m FSL 33.5m)

Reclaim Pond
(Elev. 28.3m)

Interim Dike (Elev. 31m)

South Dam (Elev. 38m)

Quarry 3

DORIS LAKE

N 7559000

N 7558000

N 7557000

N 755 000

E 435000

E 43 000

0m 100 200 300 400

NOTES

1. Phase 1 TIA Configuration accommodates Doris North Tailings (2.5Mt).
2. Phase 1 complete at the end of August 2020. Production rates vary from 1,092 tpd up to 3 000 tpd.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road
- Proposed Tailings Line
- Proposed Tailings Topography

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SRK JOB NO.: 1CT022.004. 00.010

FILE NAME: HB PH2 DEPOSITION FIGURE 2.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

TIA CONFIGURATION END OF PHASE 1 (2.5 Mt)

DATE:	APPROVED:	FIGURE:
201 /10/2	TPP	02

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 2.dwg

North Dam (Elev. 37.0m)

Reclaim Pond Access Road

Reclaim Pond
FSL (Elev. 33.5m)

Interim Dike (Elev. 50.5m)

N 7559000

N 7558000

N 7557000

N 755□000

E 435000

E 43□000

DORIS LAKE

□uarry □3

West Access Road

West Dam 3 (Elev. 50.5m)

West Dam 2 (Elev. 50.5m)

West Dam 1 (Elev. 50.5m)

East Access Road

South Dam (Elev. 50.5m)

0m 100 200 300 400

Deposition Option 1
Interim Dike Situated Upstream of Tail Lake Bay

Phase 2 Capacity: 13.9Mm³
Tailings Surface Area: 1.63km²
Spigot Elevation:
- 49.0 around □asin perimeter
- 48.5m in Centre

South Dam Raise Volume: 385,000m³
Interim Dike Volume: 1,140,000m³
Saddle Dike 1 Volume: 147,000m³
Saddle Dike 2 Volume: 8,000m³
Saddle Dike 3 Volume: 62,000m³
Tailings Line: 10□700m
East Road Upgrade: 24,000m³
West Access Road: 10,200m³

NOTES

1. Tailings Phase 2 Options occur after Phase 1. Total TIA Storage of 17.□Mt includes Phase 1 Tailings (2.5Mt) and Phase 2 Tailings (15.1Mt).
2. Assumed an average deposited tailings □each slope of 1.0□.
3. A deposited dry density of 1.3t/m³ was used □ased on la□oratory testing.
4. Ice entrainment was assumed at 20□ of production.
5. Total storage re□uirements to accommodate Madrid and Boston tailings "Phase 2" is 13.9Mm³ (Tailings 11.6Mm³+ ice entrainment 2.3Mm³).
6. Access road volumes are □ased on an assumed configuration of 10 Wide □1.0m thickness.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road Realignment
- Proposed Tailings Line
- Proposed Tailings Topography

srk consulting

SRK JOB NO.: 1CT022.004.□00.010

FILE NAME: HB PH2 DEPOSITION FIGURE 3.dwg

MAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

DEPOSITION OPTION 1
(TOTAL TIA STORAGE 17.□Mt)

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201□/10/2□	TPP	03

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 3.dwg

North Dam (Elev. 42.0m)

Reclaim Pond
FSL (Elev. 39.0m)

West Access Road

East Access Road

South Dam (Elev. 44.5m)

DORIS LAKE

N 7559000

N 7558000

N 7557000

N 755□000

E 435000

E 43□000

□uarry □3

0m 100 200 300 400

Deposition Option 2
Full Basin with Perimeter Spigots

Phase 2 Capacity: 13.9Mm³
Tailings Surface Area: 1.90km²
Spigot Elevation:
- 41.0m North Dam
- 43.0m Basin Perimeter
- 43.0m South Dam

South Dam Raise Volume: 137,000m³
North Dam Raise Volume: 70,000m³
Tailings Line: 8□100m
East Road Upgrade: 34,000m³
West Access Road: 19,000m³

NOTES

1. Tailings Phase 2 Options occur after Phase 1. Total TIA Storage of 17.□Mt includes Phase 1 Tailings (2.5Mt) and Phase 2 Tailings (15.1Mt).
2. Assumed an average deposited tailings □each slope of 1.0□.
3. A deposited dry density of 1.3t/m³ was used □ased on la□oratory testing.
4. Ice entrainment was assumed at 20□ of production.
5. Total storage re□uirements to accommodate Madrid and Boston tailings "Phase 2" is 13.9Mm³ (Tailings 11.6Mm³+ ice entrainment 2.3Mm³).
□. Access road volumes are □ased on an assumed configuration of 10 Wide □1.0m thickness.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road Realignment
- Proposed Tailings Line
- Proposed Tailings Topography

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FILE NAME: HB PH2 DEPOSITION FIGURE 4.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

DEPOSITION OPTION 2
(TOTAL TIA STORAGE 17.□Mt)

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201□/10/2□	TPP	04

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 4.dwg

North Dam (Elev. 39.5m)

Reclaim Pond Access Road

Interim Dike (Elev. 39.5m)

Reclaim Pond
FSL (Elev. 34.8m)

N 7559000

N 7558000

N 7557000

DORIS LAKE

West Access
Road

East Access Road

Quarry 3

South Dam (Elev. 43.5m)

N 755□000

E 435000

E 43□000

0m 100 200 300 400

Deposition Option 3
Interim Dike Situated Downstream of
Tail Lake Bay and Raise North Dam

Phase 2 Capacity: 13.9Mm³
Tailings Surface Area: 1.75km²
Spigot Elevation:
- 42.0m around perimter
- 41.5m in Centre

North Dam Raise Volume: 20,000m³
South Dam Raise Volume: 110,000m³
Interim Dike Volume: 185,000m³
Tailings Line: 10□800m
East Road Upgrade: 24,000m³
West Access Road: 24,500m³

NOTES

1. Tailings Phase 2 Options occur after Phase 1. Total TIA Storage of 17.□Mt includes Phase 1 Tailings (2.5Mt) and Phase 2 Tailings (15.1Mt).
2. Assumed an average deposited tailings □each slope of 1.0□ .
3. A deposited dry density of 1.3t/m³ was used □ased on la□oratory testing.
4. Ice entrainment was assumed at 20□ of production.
5. Total storage re□uirements to accommodate Madrid and Boston tailings "Phase 2" is 13.9Mm³ (Tailings 11.6Mm³+ ice entrainment 2.3Mm³).
□. Access road volumes are □ased on an assumed configuration of 10 Wide □1.0m thickness.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road Realignment
- Proposed Tailings Line
- Proposed Tailings Topography

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FILE NAME: HB PH2 DEPOSITION FIGURE 5.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

DEPOSITION OPTION 3
(TOTAL TIA STORAGE 17.□Mt)

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P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 5.dwg

North Dam (Elev. 37.5)

Reclaim Pond Access Road

N 7559000

Interim Dike (Elev. 35.0m)

Reclaim Pond
FSL (Elev. 33.5m)

Reclaim Pond
FSL (Elev. 33.5m)

N 7558000

DORIS LAKE

West Access
Road

East Access Road

N 7557000

West Dam
(Elev. 4□.5m)

South Dam (Elev. 4□.5m)

E 435000

E 43□000

0m 100 200 300 400

Deposition Option 4
Interim Dike Situated Downstream of Tail Lake Bay and No Raise to North Dam

Phase 2 Capacity: 13.9Mm³
Tailings Surface Area: 1.78km²
Spigot Elevation:
- 45.0m Basin Perimeter and South Dam
- 44.5m Center
- 43.0m in Last 3 North

Saddle Dike Volume: 47□000m3
South Dam Raise Volume: 204,300m³
Interim Dike Volume: 58,200m³
Tailings Line: 9□000m
East Road Upgrade: 34,600m³
West Access Road: 15,800m³

NOTES

1. Tailings Phase 2 Options occur after Phase 1. Total TIA Storage of 17.□Mt includes Phase 1 Tailings (2.5Mt) and Phase 2 Tailings (15.1Mt).
2. Assumed an average deposited tailings □each slope of 1.0□ .
3. A deposited dry density of 1.3t/m³ was used □ased on la□oratory testing.
4. Ice entrainment was assumed at 20□ of production.
5. Total storage re□uirements to accommodate Madrid and Boston tailings "Phase 2" is 13.9Mm³ (Tailings 11.6Mm³+ ice entrainment 2.3Mm³).
□. Access road volumes are □ased on an assumed configuration of 10 Wide □1.0m thickness.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road Realignment
- Proposed Tailings Line
- Proposed Tailings Topography

srk consulting

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FILE NAME: HB PH2 DEPOSITION FIGURE □.dwg

TMAC RESOURCES

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

DEPOSITION OPTION 4
(TOTAL TIA STORAGE 17.□Mt)

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P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE □.dwg

North Dam (Elev. 37.5)

Reclaim Pond Access Road

Reclaim Pond
FSL (Elev. 33.5m)

N 7559000

N 7558000

N 7557000

DORIS LAKE

Quarry Q3

East Access Road

West Dam
(Elev. 4Q0m)

West Access
Road

South Dam (Elev. 4Q0m)

N 755Q000

E 435000

E 43Q000

A 08

A' 08

B 08

B' 08

0m 100 200 300 400

Deposition Option 5
Deposition from Southeast Flank, No Interim Dike and No Raise to North Dam

Phase 2 Capacity: 13.9Mm³
Tailings Surface Area: 1.76km²
Spigot Elevation:
- 49.5 East Flank
- 44.5m South Dam and West Dam

Saddle Dike Volume: 39,000m³
South Dam Raise Volume: 186,200m³
Tailings Line: Q800m
East Road Upgrade: 34,800m³
West Access Road: 4,200m³

NOTES

1. Tailings Phase 2 Options occur after Phase 1. Total TIA Storage of 17.Q Mt includes Phase 1 Tailings (2.5Mt) and Phase 2 Tailings (15.1Mt).
2. Assumed an average deposited tailings Qeach slope of 1.0Q .
3. A deposited dry density of 1.3t/m³ was used Qased on laQoratory testing.
4. Ice entrainment was assumed at 20Q of production.
5. Total storage reQuirements to accommodate Madrid and Boston tailings Phase 2 is 13.9Mm³ (Tailings 11.6Mm³+ ice entrainment 2.3Mm³).
Q. Access road volumes are Qased on an assumed configuration of 10 Wide Q1.0m thickness.

LEGEND

- Spigot Location
- Deposited Tailings
- Dam/Dike
- Proposed Road Realignment
- Proposed Tailings Line
- Proposed Tailings Topography

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TMAC RESOURCES

SRK JOB NO.: 1CT022.004.Q00.010

FILE NAME: HB PH2 DEPOSITION FIGURE 7.dwg

HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN

DEPOSITION OPTION 5 (TOTAL TIA STORAGE 17.Q Mt)

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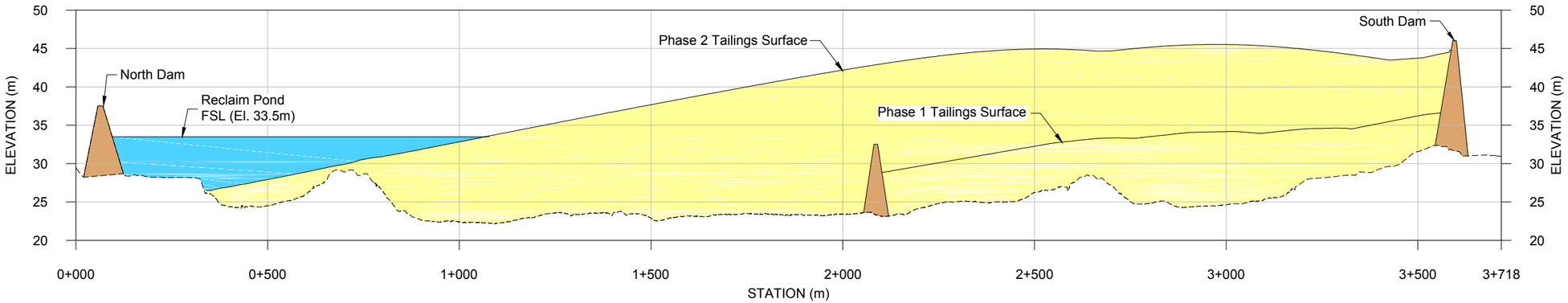
201Q/10/2Q	TPP	07

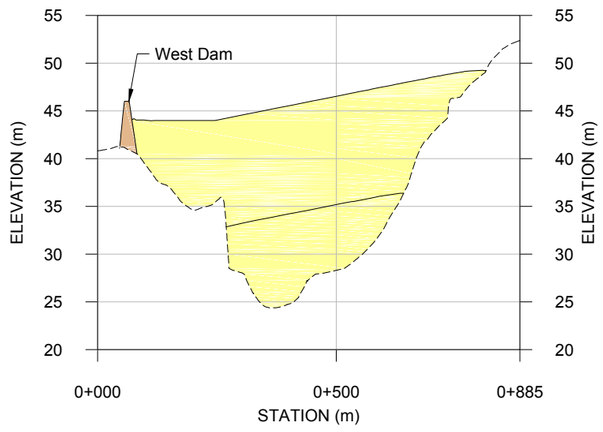
P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 7.dwg

LEGEND

- Existing Ground Surface
- Deposited Tailings
- Dam



AA 07 **SECTION**



BB 07 **SECTION**

20□E□aggeration

0m 100 200 300 400 500

srk consulting	TMAC RESOURCES	PHASE 2 TAILINGS DEPOSITION PLAN
SRK JOB NO.: 1CT022.004.□00.010	HOPE BAY PROJECT	DEPOSITION OPTION 5 (TOTAL TIA STORAGE 17.□Mt) SECTIONS AA AND BB
FILE NAME: HB PH2 DEPOSITION FIGURE 8.dwg		DATE: 201□/10/27 · APPROVED: TPP · FIGURE: 08

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 8.dwg

North Dam
(El. 37.5m)

Reclaim Pond
FSL(El. 33.5m)

DORIS
LAKE

West Dam
(El. 46.0m)

South Dam
(El. 46.0m)

Tailings Deposition - End of 2023
Spigot No.1, 2, 3 and 4: Elev. 44.5 m
Production Rate: Year 2020 & 2021: 3,000 tpd
Year 2020: 3,200 tpd
Year 2023: 3,600 tpd
Deposited Tailings (Cumulative) + Ice: 3.7 Mm³ (4Mt)

Tailings Deposition - End of 2026
Spigot No.5, 6 Elev. 44.5m, Spigot No.7 Elev. 46.5m
Production Rate: 3,600 tpd
Deposited Tailings (Cumulative)+ Ice: 6.1 Mm³ (7.9Mt)

Tailings Deposition - End of 2029 Spigot No.7
Elev. 49.5m, Spigot No.8 Elev. 45.5m
Production Rate: 3,600 tpd
Deposited Tailings (Cumulative) + Ice: 9.1 Mm³ (11.9Mt)

Tailings Deposition - End of 2032
Spigot No.8: Elev. 48.5 m
Production Rate: Year 2030 & 2031: 3,600 tpd
Year 2032: 1,597 tpd
Deposited Tailings (Cumulative) + Ice: 11.6 Mm³ (15.1Mt)

General Notes:
1. Deposition durations are approximate and were based the Mine Plan average production rates provided by TMAC.
2. Assumed an average deposited tailings beach slope of 1.0%.
3. A deposited tailings dry density of 1.3t/m3 was used.
4. All tailings volumes presented include ice entrainment at 20% of production.
5. Dam and dike elevations shown were assumed constant throughout deposition.
6. Total storage requirement includes Madrid and Boston Tailings (Tailings 11.6Mm³ + Ice entrainment 2.3Mm³)

LEGEND
- Spigot Location
- Deposited Tailings
- Dam/Dike
- Previously Deposited Tailings
- Proposed Tailings Topography

0m 200 400 600 800

srk consulting
SRK JOB NO.: 1CT022.004.600.010
FILE NAME: HB PH2 DEPOSITION FIGURE 9.dwg

TMAC RESOURCES
HOPE BAY PROJECT

PHASE 2 TAILINGS DEPSOSITION PLAN
DEPOSITION OPTION 5
STAGING PLAN
(TOTAL TIA STORAGE 17.6 Mt)

DATE:	APPROVED:	FIGURE:
2016/10/27	TPP	09

P:\01_SITES\Hope Bay\ACAD\2015 Deposition Planning\Phase 2 Options\HB PH2 DEPOSITION FIGURE 9.dwg

Appendix C – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area Seepage and Stability Analyses

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Iozsef Miskolczi, MSc, PEng	**Project No:**	1CT022.013
Reviewed:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project — Doris Tailings Impoundment Area Phase 2 Seepage and Stability Analyses		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix C as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
INAC-IR42	Section 3.4 Section 5	a) One triaxial test was completed and properties were derived using engineering judgment and experience. b) critical friction angle is independent of density c) Further testing is required to confirm assumed properties. d) slight changes in shear strength are expected but are inconsequential for the stability of the TIA
INAC – IR44	Section 3.4; Section 4.2 and Section 5	Shear parameter of the frozen soil
INAC-TRC12	Section 3.4; Section 4.3 and Section 5	Effects of long-term creep on Shear strength parameters

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization, plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases: Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

The existing Doris tailings impoundment area (TIA) will be expanded to accommodate the Project tailings, continuing with sub-aerial hydraulic deposition of a thickened slurry. Doris TIA is located approximately 1 km east of Doris Camp and accessed via the existing secondary road. To ensure environmental containment, the TIA would be impounded with three dams: North Dam, South Dam, and West Dam. The North Dam was constructed during the winter months of 2011 and 2012 (SRK, 2012), and will not be modified in any way. The South Dam will be raised 8 m to a new crest elevation of 46.0 m. The new West Dam will have a maximum height of 5 m.

The North Dam will continue to function as a water retaining dam, while the South and West dams will have tailings deposited against them, effectively functioning as tailings solids retaining structures. Over time, tailings freeze back is expected (SRK, 2017a). Tailings will be deposited sub-aerially from eight spigot points along the eastern perimeter of the TIA and from the crest of the South Dam creating a landscape that drains towards the North Dam at an average slope of about 1%.

At closure, once water quality discharge criteria are met within the Reclaim Pond, the North Dam would be breached, returning the pond water level to the pre-mining elevation of 28.3 m.

1.2 Objectives

This memo documents methods, assumptions, and results of the stability analyses completed for the South and West dams. Specifically for each structure, the following analysis were completed:

- Seepage analysis to determine the expected leakage rates, and to establish a phreatic level for use in a coupled stability analysis;
- Overall upstream slope stability and interface stability along the geosynthetic clay liner (GCL) contact zone, under static and pseudo-static conditions; and
- Overall downstream slope stability under static and pseudo-static conditions.

2 Design Criteria

2.1 Minimum Factors of Safety

A factor of safety (FOS) is defined as the ratio of the forces tending to resist failure (i.e., the material's shear strength) over the forces tending to cause failure (i.e., the shear stresses) along a given surface. The selection of a design FOS must consider the level of confidence in the factors that will control stability, i.e., material properties, analysis methods, and consequences of failure.

Design FOSs are generally defined through various industry best practice standards and guidelines, and for dams, including tailings dams, the most notable guideline is the Canadian Dam Association (CDA) Guidelines (CDA, 2014). Table 1 summarizes the recommended minimum design FOS in accordance with CDA (2014).

Table 1: Minimum Factors of Safety Used for Slope Stability Analysis

Loading Condition	Minimum Factor of Safety	Slope
During or at end of construction	>1.3 depending on risk assessment 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
Pseudo-static	1.0	Downstream
Post-earthquake	1.2	Downstream

2.2 Seismic Design Parameters

CDA (2014) provides recommended minimum seismic design criteria, based on the hazard classification assigned to the structure. Assuming a hazard classification of HIGH, the CDA (2014) specifies the design seismic event must be the 1:2,475-year event. For long-term scenarios, i.e., post-closure, the design seismic event must be increased to halfway between the 1:2,475 and 1:10,000-year event.

The Project is located in the lowest category seismic hazard zone of Canada in accordance with the 2015 National Building Code of Canada seismic hazard maps (NRCC, 2015). The seismic hazard is described by spectral-acceleration (Sa) values at different periods, as well as the peak ground acceleration (PGA) and peak ground velocity (PGV). Spectral acceleration is a measure of ground motion that takes into account the sustained shaking energy at a particular period; however, PGA is the parameter considered for foundation design.

These ground motions need to be adjusted for site specific ground type, prior to being used in design. SRK completed a site specific seismic assessment for determining horizontal and vertical seismic parameters to be used in pseudo static slope stability analysis modeling on the Project site (SRK, 2016). This analysis determines the horizontal seismic coefficient by reducing the site-adjusted PGA based on slope height and allowable deformation. The method assumes an allowable deformation of 1 to 2 inches (25 to 51 mm) for a seismic FOS of 1.1. While a larger allowable deformation is unlikely to affect the stability of the facility, this criteria was thought to be appropriately conservative. The horizontal seismic coefficients for the South and West dams during the operational period was determined to be 0.021 g and 0.025 g, resulting from a 1:2,475 year return period earthquake. Post-closure, since the South Dam and West Dam will remain in perpetuity, the design seismic coefficient was 0.036 g and 0.043 g, respectively.

3 Analysis Method

3.1 Modeling Tools

Seepage and stability analyses were carried out using the commercial Seep/W and Slope/W software developed by GEO-SLOPE International Ltd. (GEO-SLOPE, 2012). Seepage values were generated using saturated/unsaturated steady-state limit equilibrium analysis, while stability analysis were evaluated using the Morgenstern-Price method for circular failure modes. Seepage values and stability analysis were coupled.

3.2 Modeling Method

3.2.1 Seepage Model

Porewater pressure boundary conditions for the stability analysis were applied based on the results of the seepage analysis at the TIA full supply level (FSL). The tailings were conservatively assumed to be fully thawed and saturated.

Seepage volumes were calculated by obtaining the unit seepage rate (per unit width of the dam) at the downstream toe of the dams, and multiplying this value by the total length of the dam.
An average dam height was assumed in this calculation, where the average dam height was determined by averaging the height of the dam cross sections created at one meter intervals. Table 2 summarizes the dam dimensions used in this analysis.

Table 2: South Dam and West Dam Geometry used in Seepage Analysis

Location	Total Dam Longitudinal Cross Section Area (m^2)	Total Longitudinal Length (m)	Average Height (m)	Maximum Height (m)
South Dam	5,000	520	9.6	15
West Dam	1,740	470	3.7	5

3.2.2 Stability Model

Slope stability was assessed for both static and pseudo-static conditions. To provide confidence in the results, the models were analyzed using three modes of searching for the failure surface:

- Grid and radius;
- Specified entry and exit locations; and
- Fully specified failure surface.

The following loading conditions, as recommended by CDA (2014), were considered:

- Long-term – static drained (i.e., no excess pore pressure);
- During or at the end of construction – static undrained (with excess pore pressure due to rapid loading);
- During earthquake – pseudo-static undrained condition; and
- Post-earthquake – pseudo-static drained condition.

Since the South and West dams in actual fact have frozen foundations, are expected to remain frozen in the long term, and the area has a low seismic coefficient, post-earthquake deformation is not expected to be material. As such, changes in shear strength parameters in the dam fill and in the foundation soils are expected to be negligible. Therefore, a post-earthquake analysis was not completed.

Considering the loading conditions, the assessment objectives, model geometry and foundation conditions, the following analysis were completed:

- Stability of the upstream slope of the South and West dams respectively, along the interface between the GCL and the dam fill at the end of the dam construction, prior to discharge of any Phase 2 tailings. The following two foundation conditions were considered:
  - Fully thawed – thawed foundation layer thickness of approximately 7 m; and
  - Partially thawed - thawed foundation layer thickness of 1 m.
- Stability of the foundation and downstream slope of the dams, with tailings reaching FSL. Again, the fully thawed and partially thawed foundation conditions were considered.

Partially thawed foundation conditions are supported by thermal analysis (Figures 4 and 5) completed as part of the dam design (SRK, 2017a). Thaw will however be limited to zones near the toe of the dam, while the key trench and the thicker fill zones of the dam will remain frozen. Nonetheless, the partially thawed condition was applied to the full footprint of the dam for added conservatism. For the partially thawed conditions, the -2°C isotherm, as developed from thermal modelling (SRK, 2017a), was adopted to define the extent of thaw (Figure 3).

Fully thawed foundation conditions are not plausible under the expected long term field conditions, and are only used in the analysis as a theoretical absolute limit. Considering an unbounded timeframe, and uncontrolled constant climate change, it could be argued that the complete foundation might thaw (on a millennial scale). However, should that happen, the change would be so slow that pore water pressures would readily dissipate and therefore an assumption of drained conditions is reasonable.

3.3 Model Geometry

The South and West dams are frozen foundation dams with a GCL keyed into the permafrost overburden foundation. The dams will not have ponded water against them as tailings will be beached from the crest of the dams pushing the supernatant pond water, i.e., Reclaim Pond, away from the dams. To accommodate the Phase 2 tailings, the Phase 1 South Dam will be raised, and the West Dam will be a new structure.

The Phase 2 raise of the South Dam will be a downstream raise following the same geometry as the Phase 1 design. The final crest width will be 10 m, the upstream slope 4H:1V, and the downstream slope 2H:1V (Figure 1). The crest elevation is raised 8 m from 38.0 m to 46.0 m, and results in a maximum dam height of 15 m and total dam length of 520 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 previously installed as part of the Phase 1 South Dam will be extended vertically to reach an elevation of 45.0 m at a slope of 3H:1V (4H:1V in the raised portion).

The West Dam was designed with the same cross-section as the South Dam (Figure 2). This dam will be constructed in a single stage, and will be about 470 m long with a maximum height of 5 m.

For both the South and West dams, a single critical cross-section was analyzed. This critical section was conservatively assumed to be the zone where the foundation overburden soils was at its maximum thickness and the dam structure was at its maximum height. Figures 1 and 2 present the model cross-sections for the South and West dams respectively.

3.4 Material Properties

Sub-surface investigations downstream of the Phase 1 South Dam, along the footprint of the Phase 2 raise have not been carried out, and a single drill hole has been completed within the footprint of the West Dam. Material properties for the analysis was therefore based on the North Dam (SRK, 2007), and Phase 1 South Dam (SRK, 2015) designs, supplemented with the site wide geotechnical design properties (SRK, 2017b). Table 3 summarizes the properties used in the analysis.

Table 3: South and West Dam Foundation and Material Properties

Parameter		Marine Silt and Clay	Marine Silt	Tailings	Run of Quarry (Dam Fill)	GCL
Moist Unit Weight (kN/m^3)		18 (1)	18(1)	17.5 (2)	20 (1)	10 (4)
Undrained Shear Strength s_u (kPa)		13 (1)	13 (1)	-	-	-
Non-Frozen	Cohesion c' (kPa)	0 (1)	0 (1)	0 (2)	0 (1)	0 (4)
	Friction Angle, φ ^{0}	30 (1)	32 (1)	40 (2)	40 (1)	15 (4)
Frozen	Cohesion c' (kPa)	112 (1)	112 (1)	-	5 (1)	-
	Friction Angle, φ ^{0}	26 (1)	26 (1)	-	40 (1)	-
Non-Frozen Hydraulic Conductivity (m/s)		4.6x10^{-10} (1)	4.6x10^{-10} (1)	1.3x10^{-7} (2)	5.0x10^{-3} (3)	5.0x10^{-11} (4)
Frozen Hydraulic Conductivity (m/s)		4.6x10^{-10} (1)	4.6x10^{-10} (1)	n/a	1.0x10^{-7} (3)	n/a

Notes:

(1) SRK (2017a)
(2) Knight Piésold (2009)
(3) Engineering Judgement
(4) https://www.layfieldgroup.com/Geosynthetics/Geomembranes/Geosynthetic-Clay-Liner.aspx

The shear strength properties of the tailings materials were derived using engineering judgment from a single three-stage triaxial test completed. The test results represent the residual friction angle, i.e. a value close to the critical state friction angle which does not change as a function of density and saturation. Therefore using this value is appropriate for both filtered tailings and slurry tailings.

The stability analysis results presented in Section 4 show that tailings materials are not involved in the critical failure mechanisms corresponding to low FOS. Further testing of the tailings will be required to confirm the assumed properties for the detailed engineering stage of the design.

Since no specific geotechnical data from the South and West dam foundations were available, the shear strength parameters of the frozen foundation soils (marine silt and clay and marine silt) used in the stability analysis were estimated as c = 112 kPa and φ = 26°. Arenson and Sprigman (2005) proposed an empirical relation between the friction angle of frozen soils and the volumetric ice content w_i. With increasing w_i, the friction angle decreases from an initial friction angle (φ_{init}) at w_i = 0 to a friction angle of 0 at w_i = 1.0 as follows:

$$\varphi = \varphi_{init} - \varphi_{init} w_i^{2.6} \quad (1)$$

Opposed to the friction angle, the cohesion of frozen soils increases with increasing volumetric ice content. Furthermore, the cohesion depends on the temperature and strain rate. According to Arenson and Sprigman (2005), the cohesion increases from a value of c = 0 kPa at $w_i = 0$ with increasing ice content and decreasing temperature T as follows:

$$c_{(T=-2.1°\mathrm{C})} = 534 \text{ kPa } w_{i_1}^{1.91} \quad (2)$$

$$c = -\frac{c_{(T=-2.1°\mathrm{C})}}{2.1} T \quad (3)$$

Salinity and ice content were considered according to Andersland and Ladanyi (2004). Assuming a salinity of 40 ppm and a minimum value of $\varphi_{init} = 30°$ for the initial friction angle at $w_i = 0$ (see Table 3; non-frozen marine silt and clay), the shear parameters of c = 112 kPa and φ = 26° resulted from Equations (1) to (3) for a temperature of T = −4°C, a freeze point at -2°C, and an ice content of w_i = 0.452 (SRK 2017b).

Creep effects are not considered in Equations (2) and (3). As an extreme case, the cohesion of the frozen foundation soils can be assumed to vanish when the frozen soils experience very low strain rates due to long-term creep. In this case the strength of the frozen soils is giving by the strain rate independent friction angle φ = 26° only.

4 Results

4.1 Seepage Analysis

The resultant seepage through the South and West dams is presented in Table 4. These results are the upper limit of seepage considering the conservative assumptions of a fully thawed foundation, and fully saturated tailings. Due to tailings freeze back, these conditions will not occur and seepage is expected to be approaching zero.

Furthermore, post-closure when the North Dam is breached and the system phreatic level returns to the pre-mining value of 28.3 m, the possibility of long-term seepage is all but eliminated because the lowest toe elevation of the South and West dams is 31.0 m at the South Dam.

Table 4: Seepage Analysis Results

Location	Maximum Tailings Elevation (m)	Unit Dimensional Seepage Rate (m^3/sec)	Total Seepage Rate (m^3/day)
South Dam	44.5	1.14×10^{-6}	51.2
West Dam	44.5	8.08×10^{-9}	0.33

4.2 Stability Analysis

The stability of the upstream slope of the South and West dams, at their maximum design height, was analyzed. The results are presented in Table 5.

Table 5: Stability Analysis Results for South and West Dams (Upstream Slope)

Location	Loading Condition	Foundation Condition	Analysis Type	Minimum FOS (CDA 2014)	Computed FOS
South Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.6
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	1.6
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	1.4
West Dam	Long Term	Fully Thawed (Drained)	Static	1.5	2.3
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	1.1
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.8
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.9

Note: Numbers in red mean the FOS does not meet the stability criterion,

The computed FOS for the fully thawed foundation conditions under the South Dam exceed the minimum required values; and therefore, the less conservative partially thawed foundation condition was not analyzed (Table 5). The FOS obtained from a fully thawed foundation and undrained loading condition under the West Dam does not meet the required minimum required values in the static and pseudo static cases; however, based on the thermal analysis (SRK 2017A), the more realistic partially thawed foundation condition for the static and pseudo static cases does satisfy the criteria.

Results of the downstream slope analysis are presented in Table 6.

Table 6: Stability Analysis Results for South and West Dams (Downstream Slope)

Location	Loading Condition	Foundation Condition	Analysis Type	Minimum FOS (CDA 2014)	Computed FOS
South Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.6
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	0.5
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.5
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.2
West Dam	Long Term	Fully Thawed (Drained)	Static	1.5	1.5
	During or at End of Construction	Fully Thawed (Undrained)	Static	1.3	0.9
	During or at End of Construction	Partially Thawed (Undrained)	Static	1.3	1.3
	During Earthquake	Fully Thawed (Undrained)	Pseudo-static	1.0	0.8
	During Earthquake	Partially Thawed (Undrained)	Pseudo-static	1.0	1.1

Note: Numbers in red mean the FOS does not meet the stability criterion,

A fully thawed foundation under undrained loading conditions results in FOS less than the required minimum for the downstream slopes of the South and West dams; however, all FOS exceed the required minimums when the more realistic condition of a partially thawed foundation is imposed based on the thermal analysis.

Factors of safety for the long term static conditions were evaluated for fully thawed foundation; i.e., the shear parameters used in the analysis were c=0 kPa and φ=30°. The minimum FOS was 1.5. In the extreme case that the shear parameters of the frozen soil affected by long-term creep strains were c=0 kPa and φ=26° (Section 3.4), the expected FOS would be a little lower than 1.5 and would not meet the long-term stability criteria. However, a small increment of the cohesion would suffice to achieve the stability criterion.

SRK completed analysis to back-calculate the cohesion required to meet the long-term stability criterion when long-term creep strains occur in the foundation soils. The back-analysis was performed for the lowest factor of safety in Tables 5 and 6 for the long-term condition (Table 6; West Dam). Based on the results, a minimum cohesion of 3 kPa is required to meet the stability criterion when the cohesion of the frozen silt and silt and clays is affected for long-term creep strains.
An additional analysis was performed for a worst-case scenario in which the friction angle is reduced from 26° to 20° due to a hypothetical increase of the volumetric ice-content in the soil. In this case a cohesion of 7 kPa is required to meet a FOS of 1.5.

Attachment 1 includes the critical failures mechanisms for all the loading conditions analyzed, and the results of the back-analysis.

5 Conclusion

South Dam

The upstream slope of the South dam meets the stability criteria for all loading condition analyzed. All the critical failure mechanisms corresponding to low factor of safety presented in the upper part of Table 5 are in the non-frozen ROQ material. The long-term FOS for fully thawed foundation is 1.6.

The downstream slope of the South dam does not meet the stability criteria for short-term static and pseudo-static condition with fully thawed foundation. The fully thawed undrained condition is very unlikely based on the results of the thermal analysis. These FOS are not representative to assess the stability of the downstream slope of the west dam. The long-term FOS for fully thawed foundation is 1.6.

West Dam

The upstream slope of the West dam does not meet the stability criteria for the static and pseudo-static loading condition with fully thawed foundation. As outlined above, these conditions are no representative for the expected frozen condition of the foundation soils. The stability criteria for the long-term condition was met with a FOS of 2.3

The downstream slope of the West dam meet the long term stability criteria with a FOS of 1.5.

Tailings material

In all the analysis, tailings materials were not involved in the critical failure mechanisms. Therefore slight changes in the physical properties and abrasion due to hydraulic transport are inconsequential to the stability of the TIA.

Effect of the long-term creep on the stability

Since a minimum value of about 7 kPa for the cohesion of the frozen foundation soils with a reduced friction angle of 20° is very unlikely based on the results of creep deformation analysis performed for other structures of the Hope Bay project (SRK 2017c and SRK 2017d), the South and West dams meet long-term stability criterion considering the impact of the long-term creep on the shear strengths parameters of the frozen marine silts and silt and clays in the foundation.

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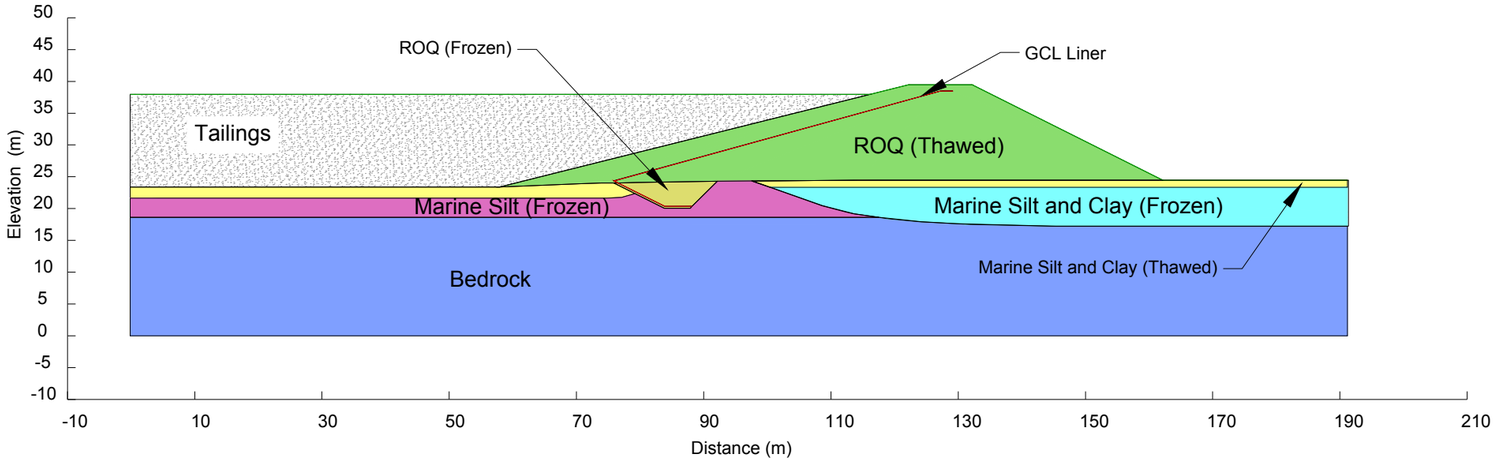
Figures



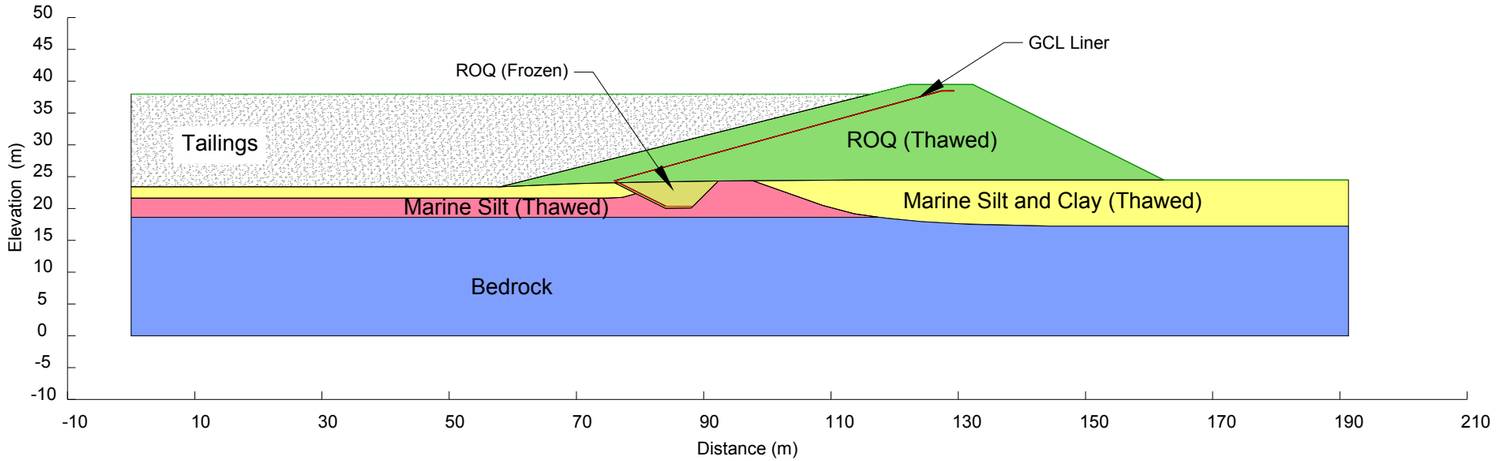
LEGEND

- ROQ (Frozen)
- ROQ (Thawed)
- Marine Silt (Thawed)
- Marine Silt (Frozen)
- Tailings
- Bedrock
- Marine Silt and Clay(Frozen)
- Marine Silt and Clay (Thawed)
- GCL Liner

South Dam
Partially Thawed Foundation Condition

0 10 20 30 40 50

Scale in Metres



South Dam
Fully Thawed Foundation Condition

0 10 20 30 40 50

Scale in Metres

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FILE NAME: HopeBay_1CT022.004_SouthWestDams_SlopeStabilityMemo_Figures_Rev03_so_ms.dwg

TMAC RESOURCES

Hope Bay Project

Doris Phase 2 TIA
Seepage and Stability Analysis

South Dam - Analyzed Section

DATE:	APPROVED:	FIGURE:
April 2016	IM	1

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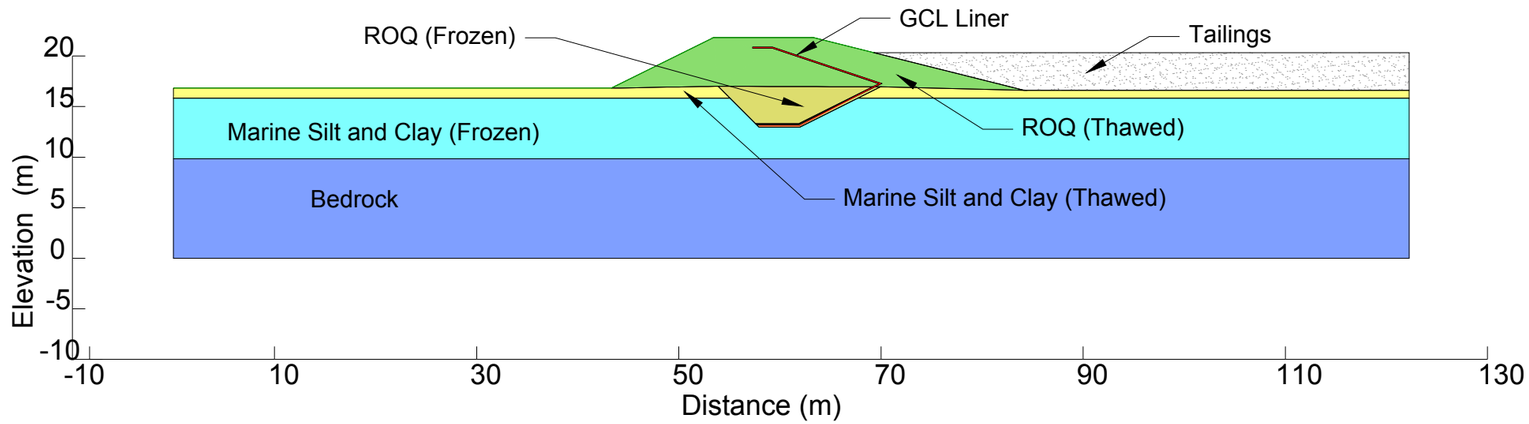
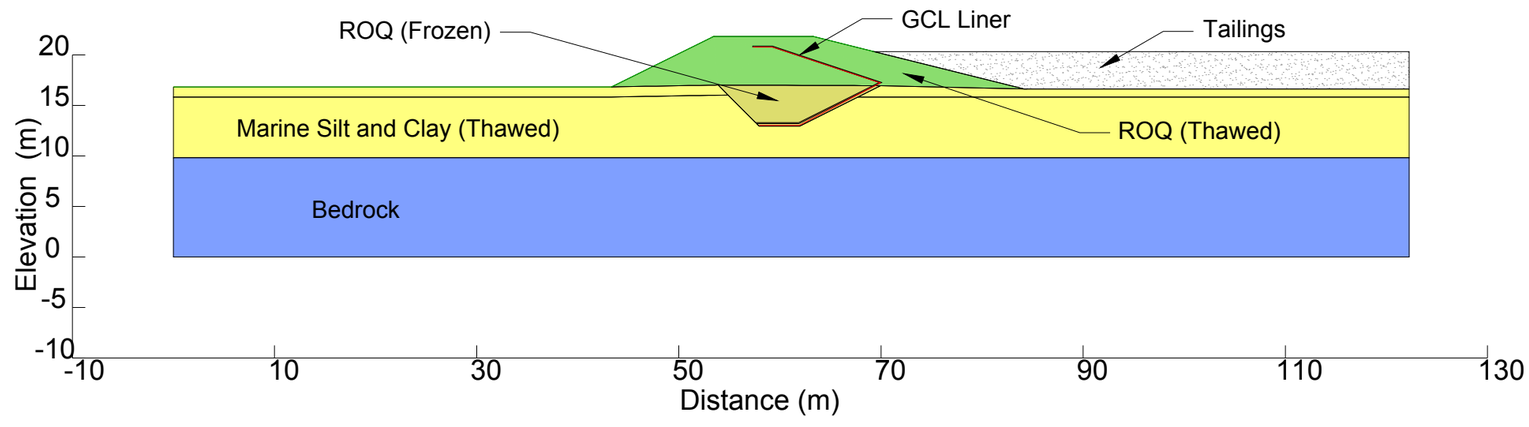


LEGEND

- ROQ (Frozen)
- ROQ (Thawed)
- Marine Silt (Thawed)
- Marine Silt (Frozen)
- Tailings
- Bedrock
- Marine Silt and Clay(Frozen)
- Marine Silt and Clay (Thawed)
- GCL Liner

West Dam
Partially Thawed Foundation Condition

0 5 10 15 20 25
Scale in Metres



West Dam
Fully Thawed Foundation Condition

0 5 10 15 20 25
Scale in Metres

srk consulting

SRK JOB NO.: 1CT022.004.610

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TMAC RESOURCES

Hope Bay Project

Doris Phase 2 TIA
Seepage and Stability Analysis

West Dam - Analyzed Section

DATE:	APPROVED:	FIGURE:
April 2016	IM	2

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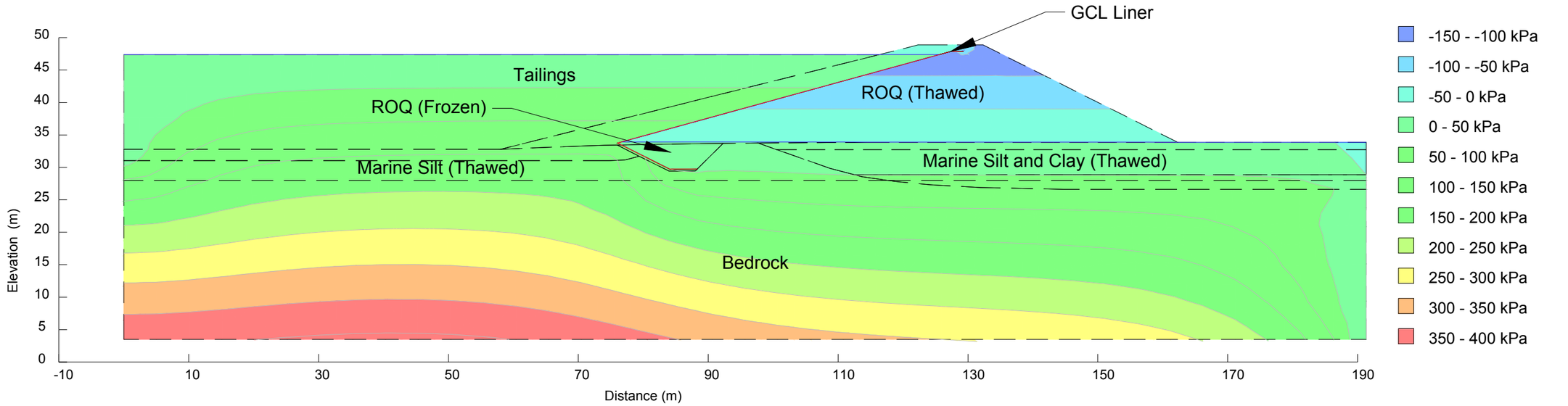


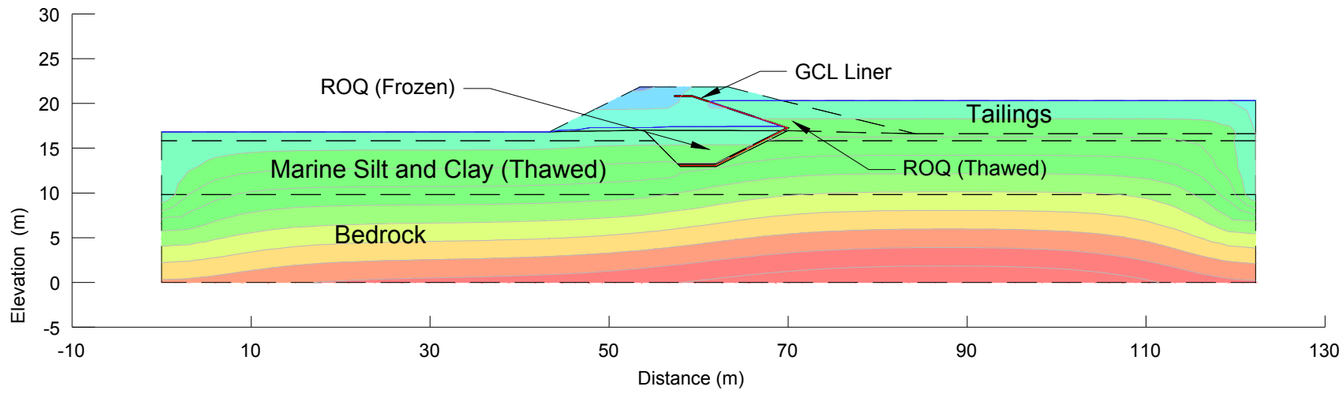
Pore-Water Pressure

- -150 - -100 kPa
- -100 - -50 kPa
- -50 - 0 kPa
- 0 - 50 kPa
- 50 - 100 kPa
- 100 - 150 kPa
- 150 - 200 kPa
- 200 - 250 kPa
- 250 - 300 kPa
- 300 - 350 kPa
- 350 - 400 kPa

South Dam
Pore - Water Pressure Contours

0 5 10 15 20 25

Scale in Metres



Pore-Water Pressure

- -60 - -40 kPa
- -40 - -20 kPa
- -20 - 0 kPa
- 0 - 20 kPa
- 20 - 40 kPa
- 40 - 60 kPa
- 60 - 80 kPa
- 80 - 100 kPa
- 100 - 120 kPa
- 120 - 140 kPa
- 140 - 160 kPa
- 160 - 180 kPa

LEGEND

- Phreatic Surface
- Material Boundary
- Contour Line
- GCL Liner

West Dam
Pore - Water Pressure Contours

0 5 10 15 20 25

Scale in Metres

srk consulting

SRK JOB NO.: 1CT022.004.610

FILE NAME: HopeBay_1CT022.004_SouthWestDams_SlopeStabilityMemo_Figures_Rev03_so_ms.dwg

TMAC RESOURCES

Hope Bay Project

Doris Phase 2 TIA
Seepage and Stability Analysis

Analyzed Seepage Models

DATE:	APPROVED:	FIGURE:
May 2016	IM	3

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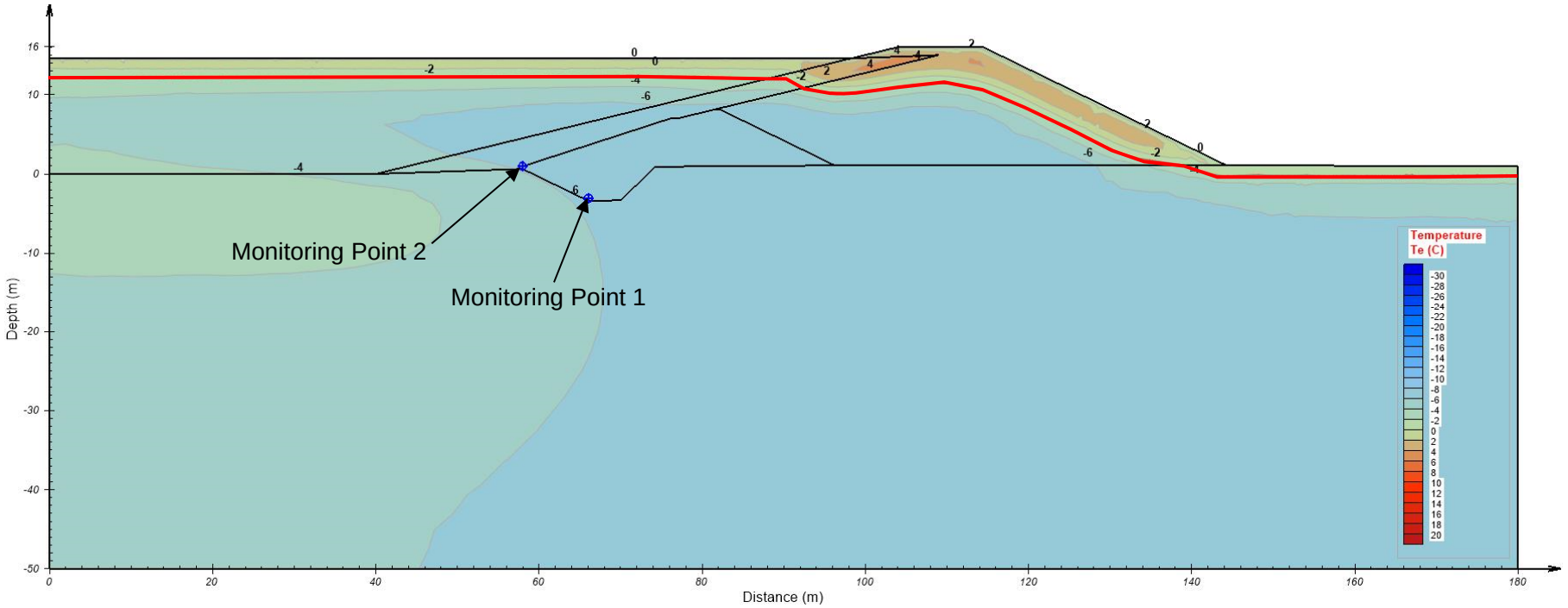

Notes:
1. Model results for maximum position of seasonal thaw at the end of the 25 year design life
2. Solid red line shows -2°C isotherm

srk consulting	TMAC RESOURCES	Doris Phase 2 TIA Thermal Analysis
		Analyzed Thermal Model Results South Dam – End of Design Life
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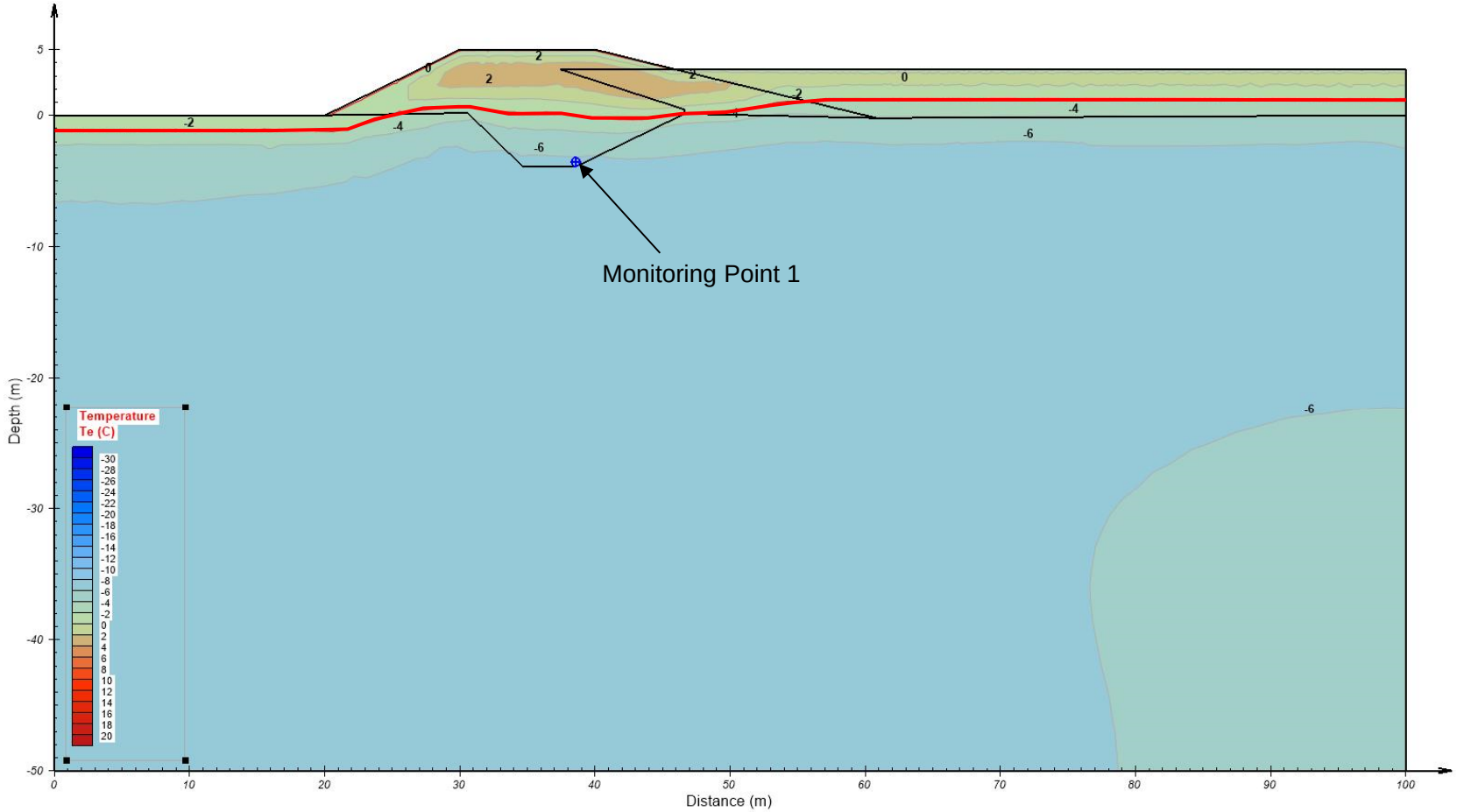
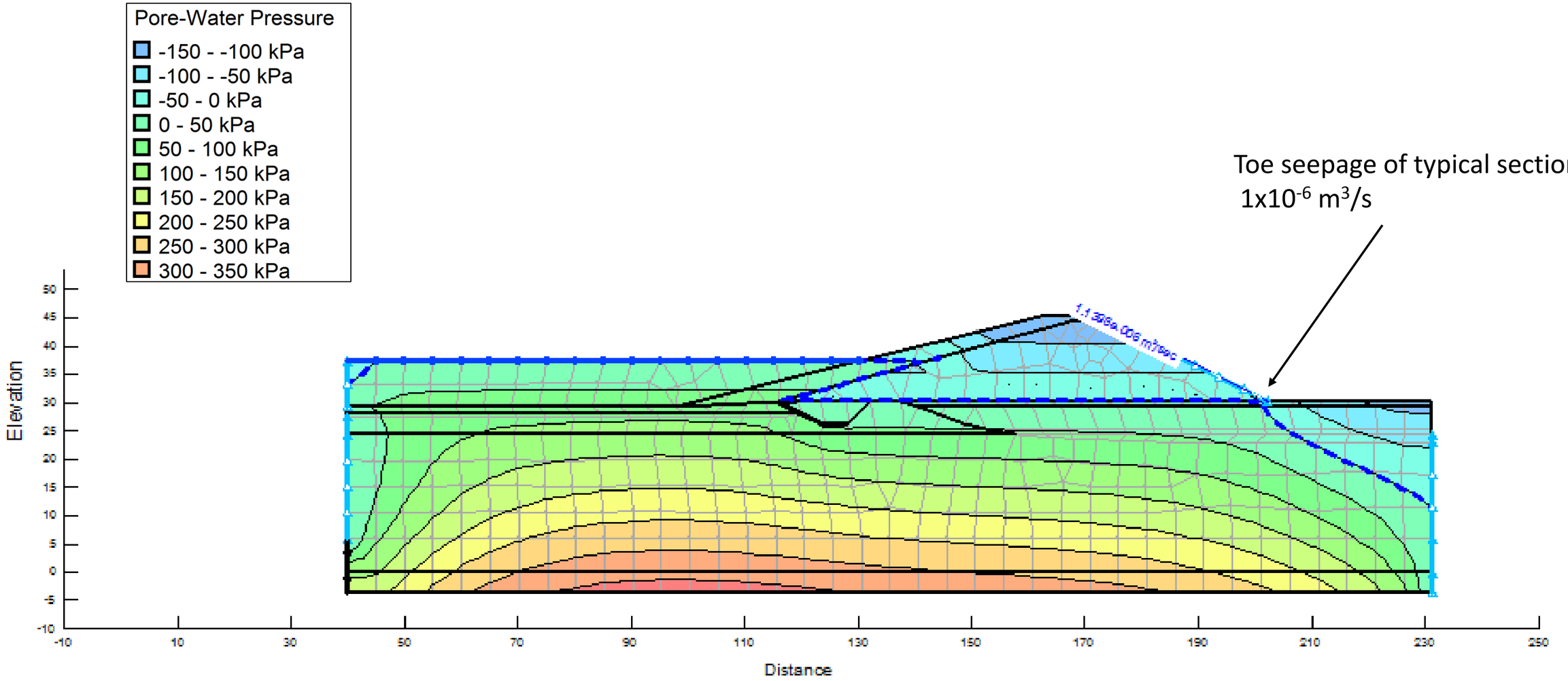


Notes:

1. Model results for maximum position of seasonal thaw at the end of the 25 year design life
2. Solid red line shows -2°C isotherm

srk consulting	TMAC RESOURCES	Doris Phase 2 TIA Thermal Analysis	
		Analyzed Thermal Model Results West Dam – End of Design Life	
Job No: 1CT022.004.610 Filename: Figure3_SlopeStabilityMemo	TMAC Resources Inc.	Date: May 2016	Approved: IM	Figure: **5**

Attachment 1
South and West Dam Seepage Analysis Results



srk consulting	TMAC RESOURCES	DORIS TMS SEEPAGE ANALYSIS
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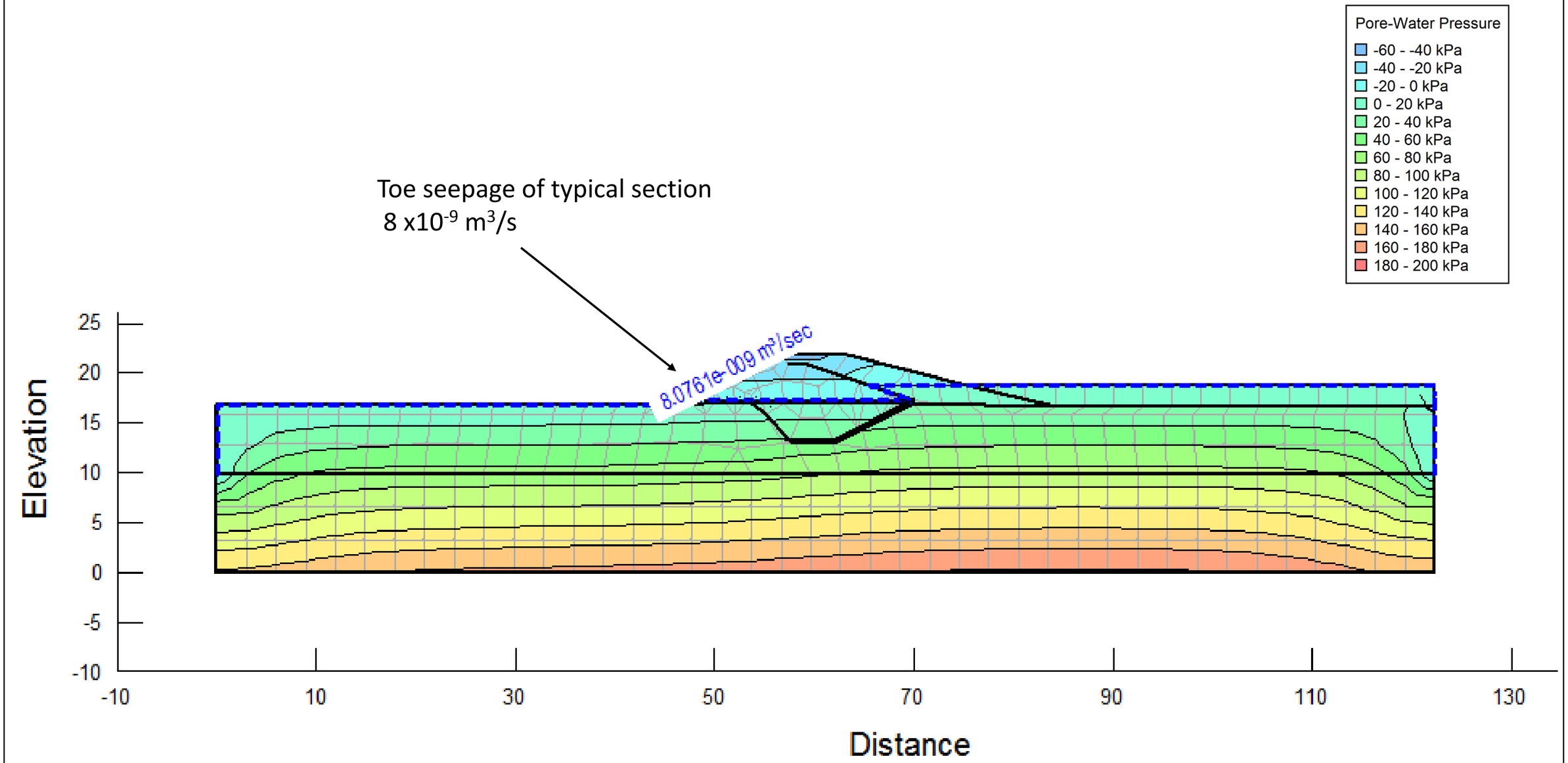

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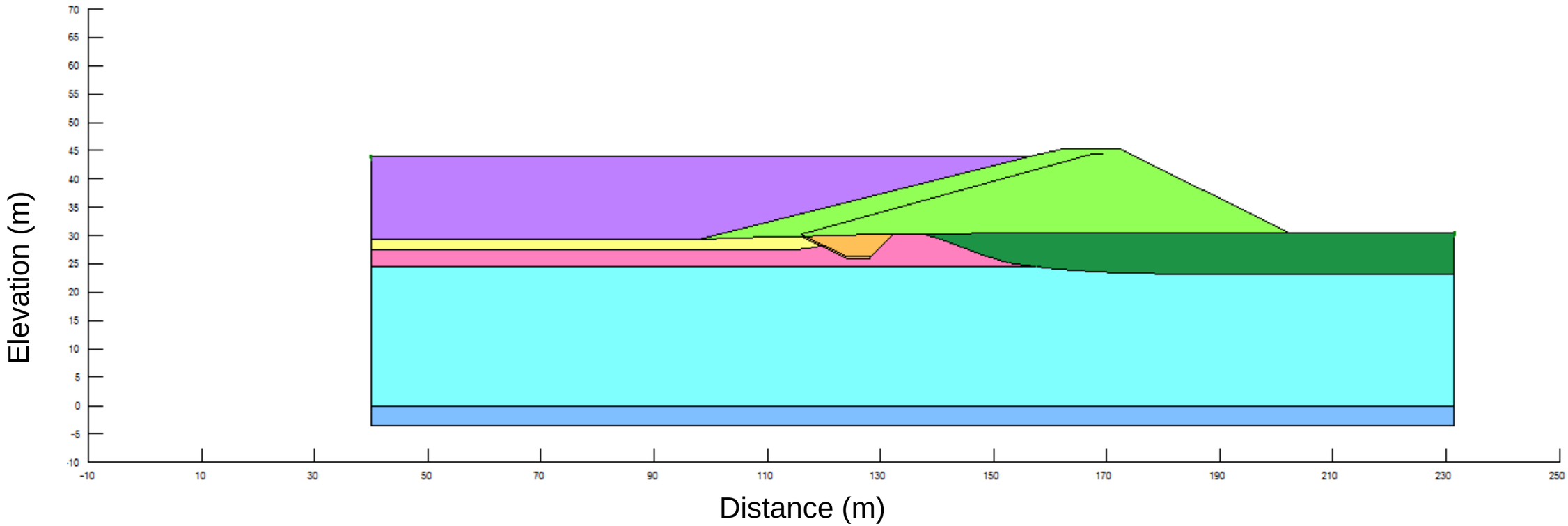
Job No: 1CT22.004.610 Filename: HopeBay_DorisNorth_SouthDam_SlopeStability_160407_sa	HOPE BAY PROJECT	Date: April 2016 · Approved: SA · Figure: **2**

Attachment 2
South Dam Stability Analysis Results



LEGEND

- ROQ (Non-Frozen)
- Marine Silt and Clay (Non-Frozen)
- Marine Silt and Clay (Frozen)
- ROQ (Frozen)
- Marine Silt and Clay (Top 5m)
- Tailings
- Silt (Frozen)
- Bedrock

srk consulting

Job No: 1CT22.004.610

Filename: HopeBay_DorisNorth_SouthDam_SlopeStability_160407_sa

TMAC RESOURCES

HOPE BAY PROJECT

DORIS TIA SLOPE STABILITY

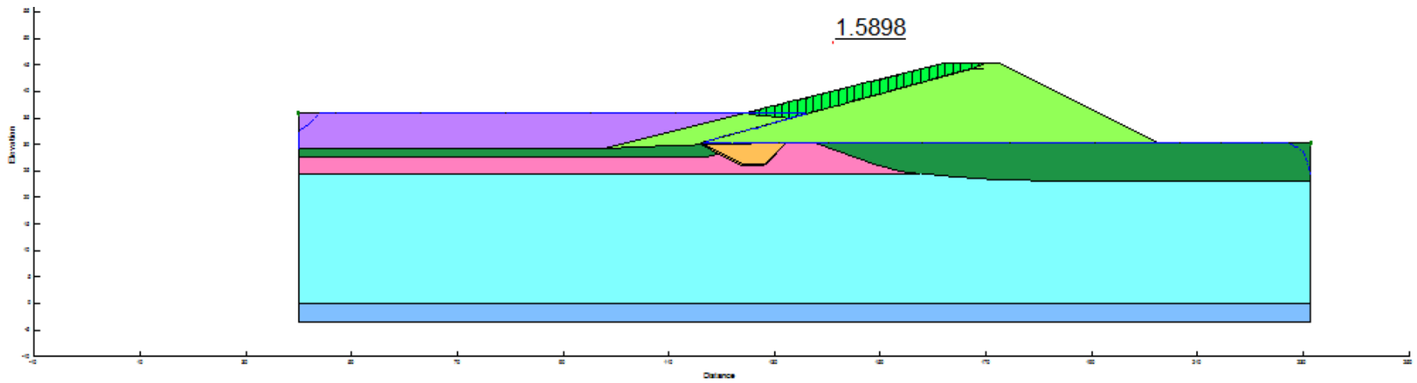
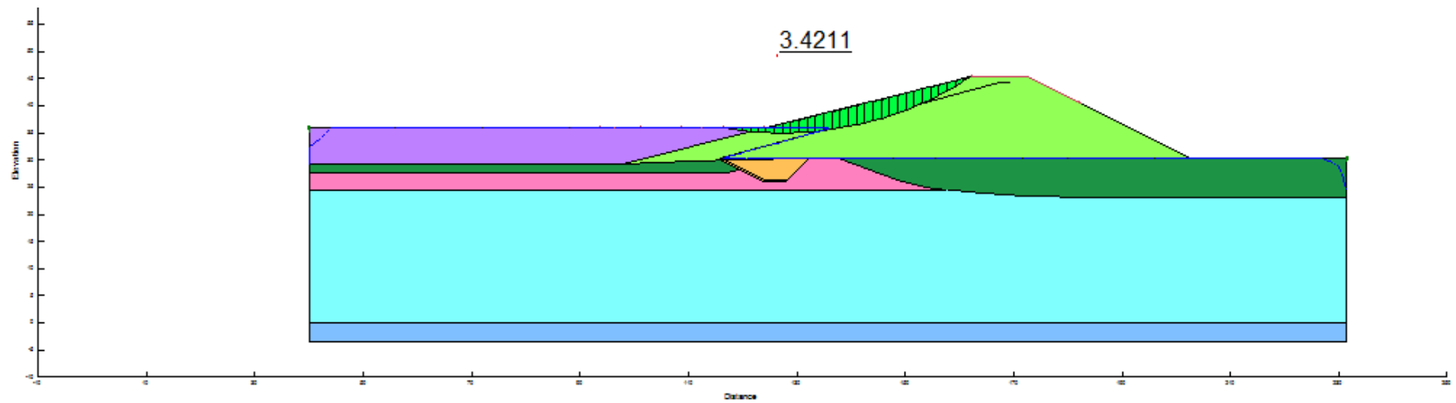
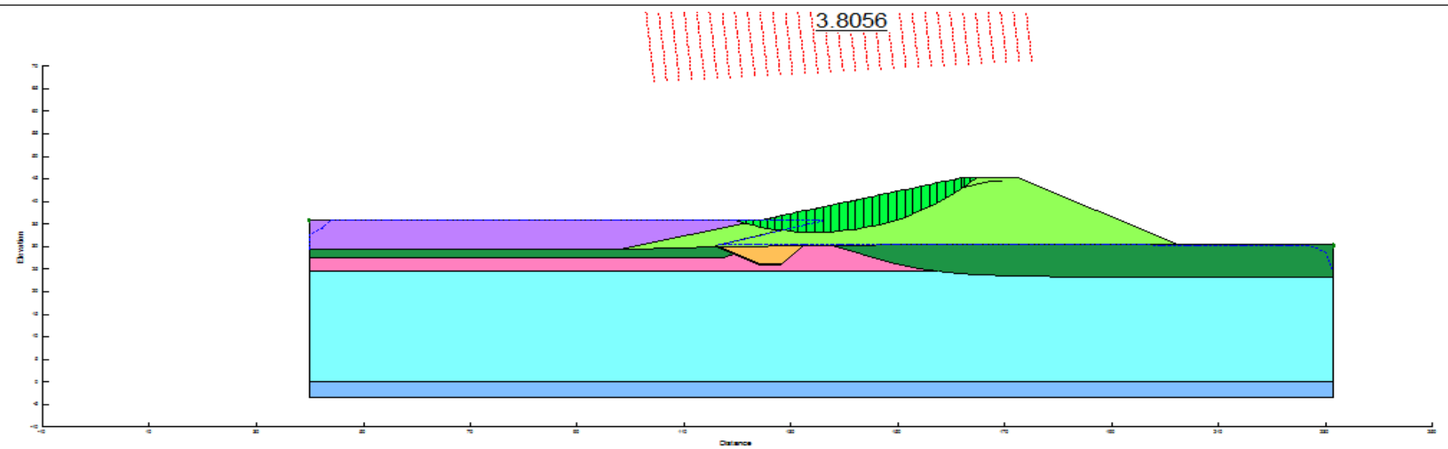
South Dam Analyzed Cross Section

Date: April 2016

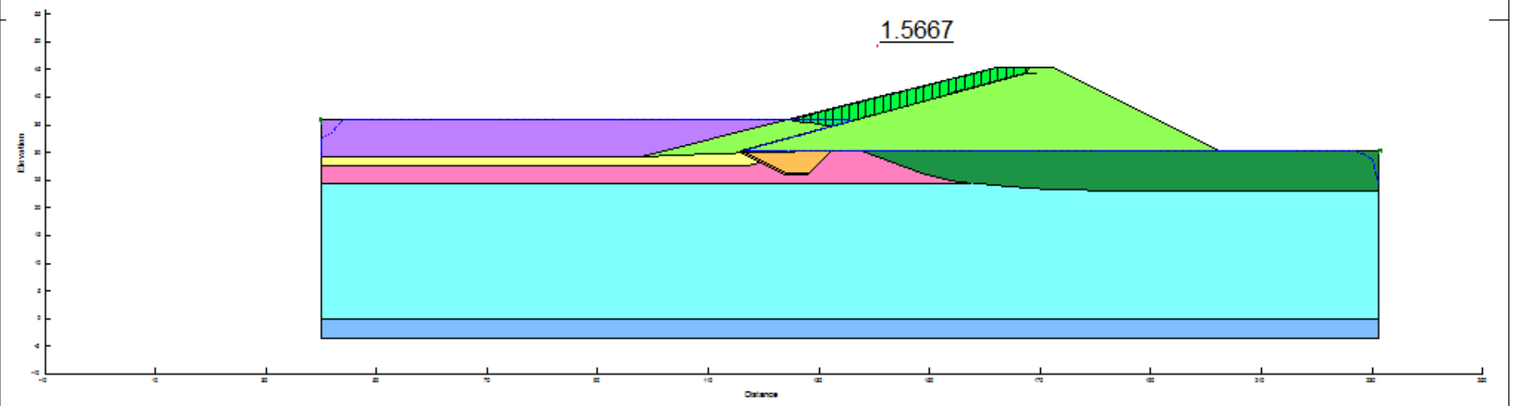
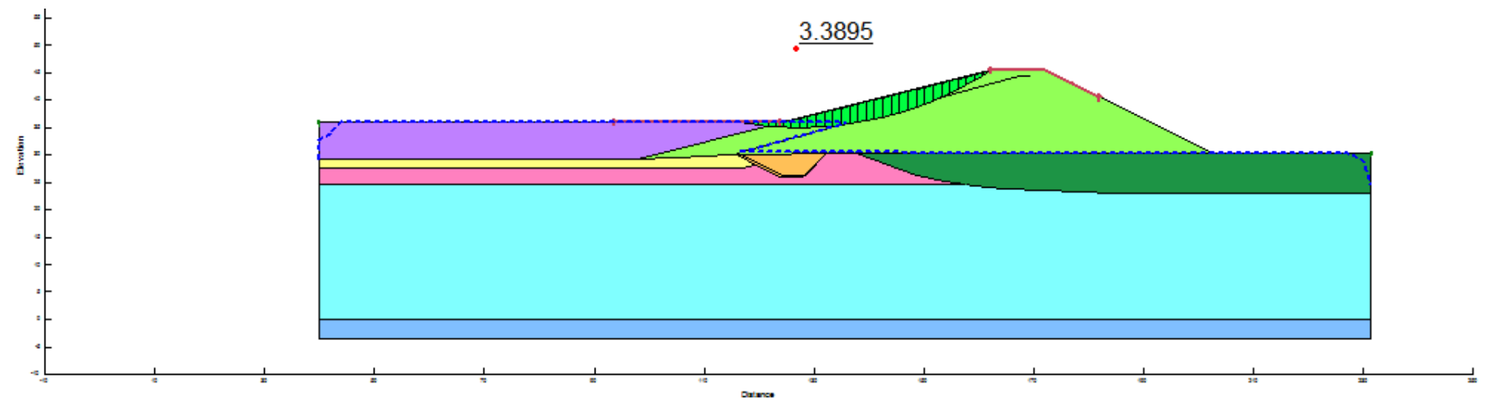
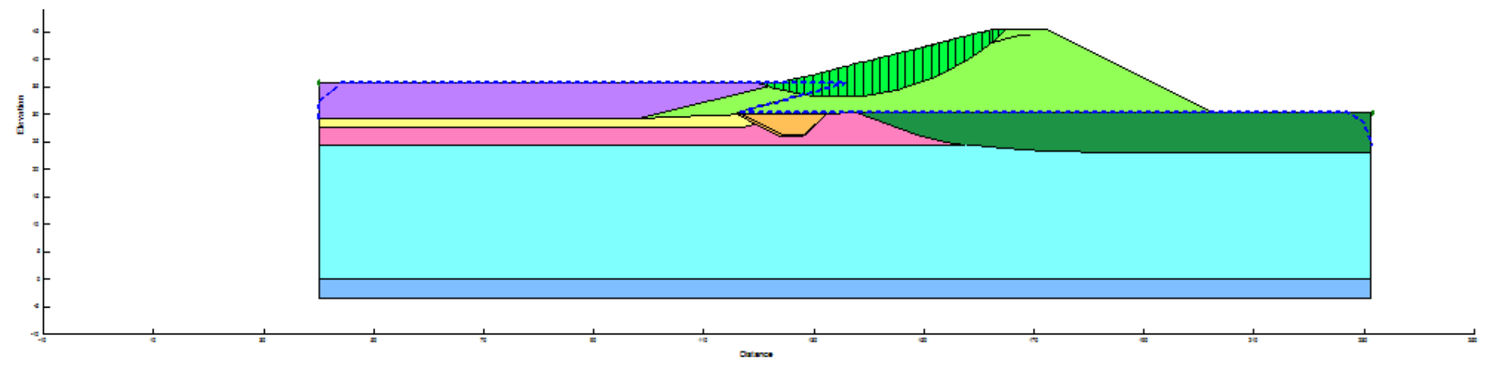
Approved:

Figure: **1**

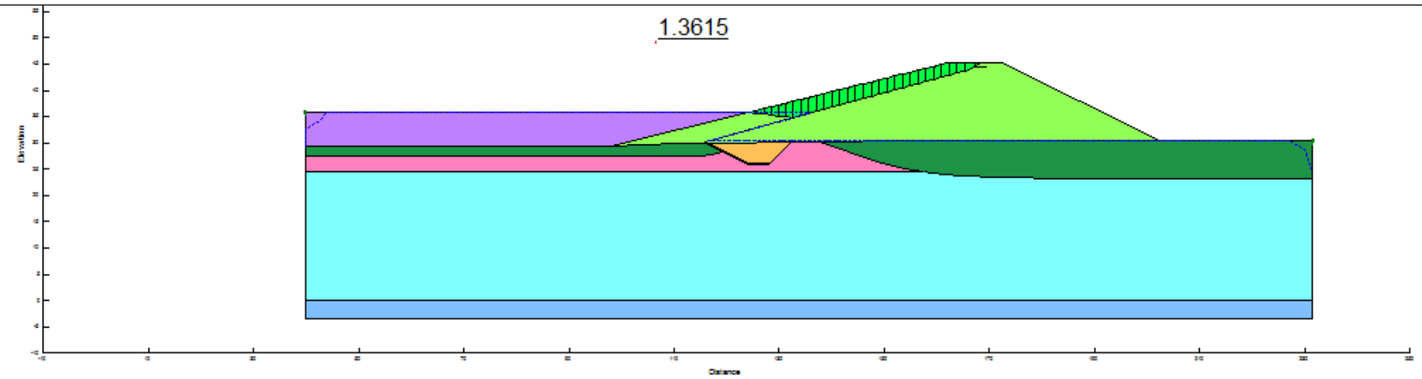
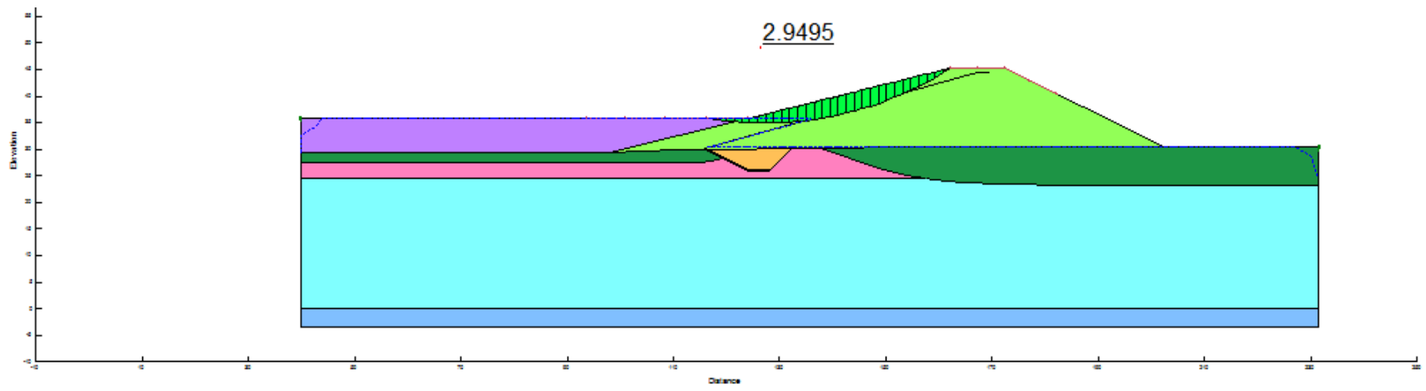
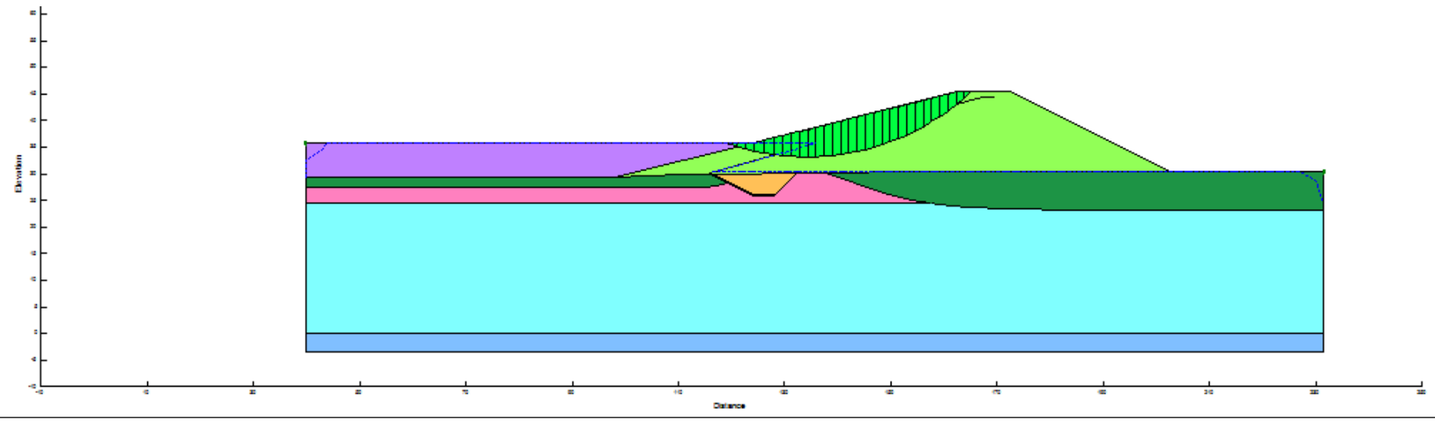
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model A-South Dam (Upstream Slope, Static, Drained, 7m AL)	1.59	Fully Defined	
	3.42	Entry Exit	
	3.81	Grid and Radius	

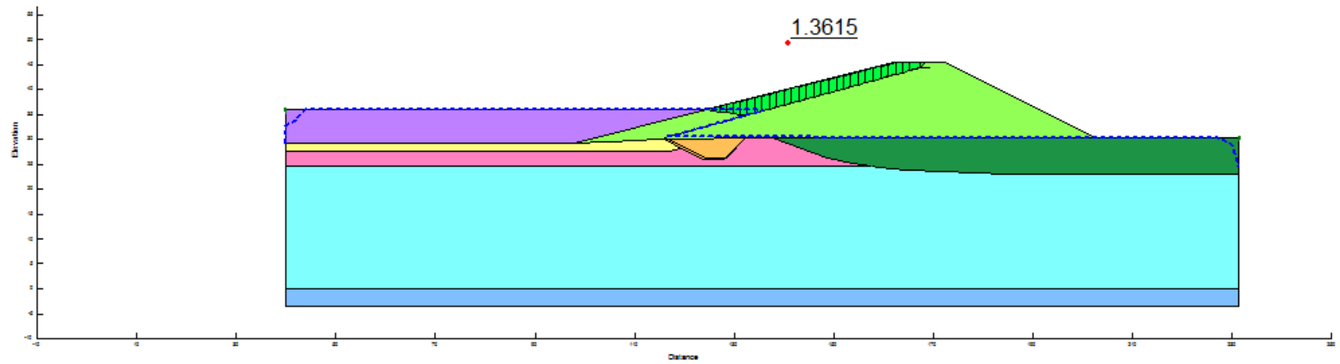
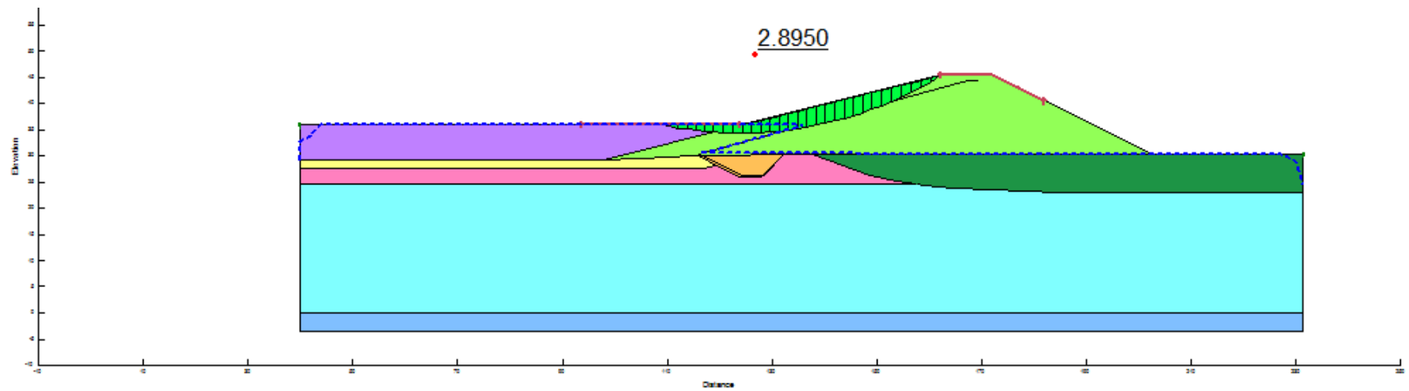
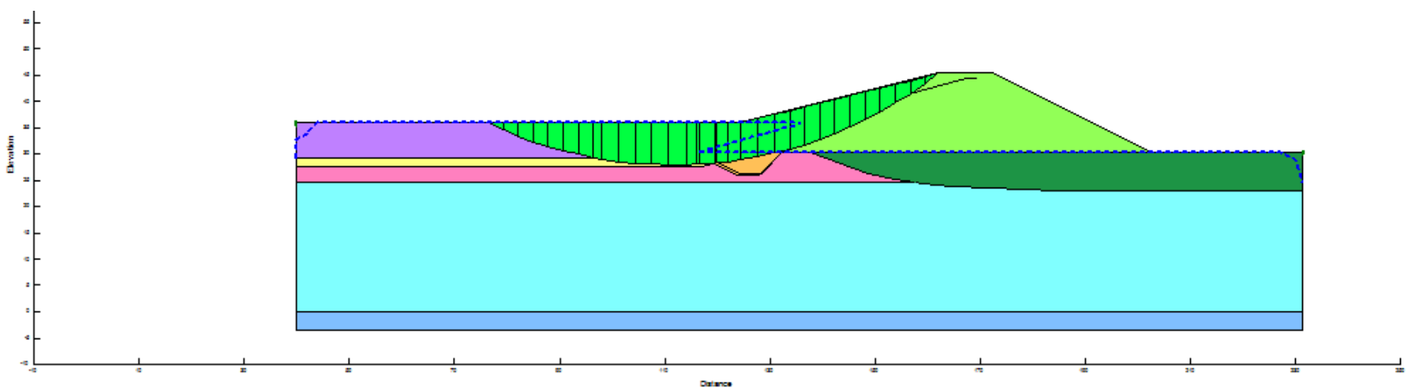
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model A-South Dam (Upstream Slope, Static, Undrained, 7m AL)	1.57	Fully Defined	
	3.39	Entry Exit	
	3.79	Grid and Radius	

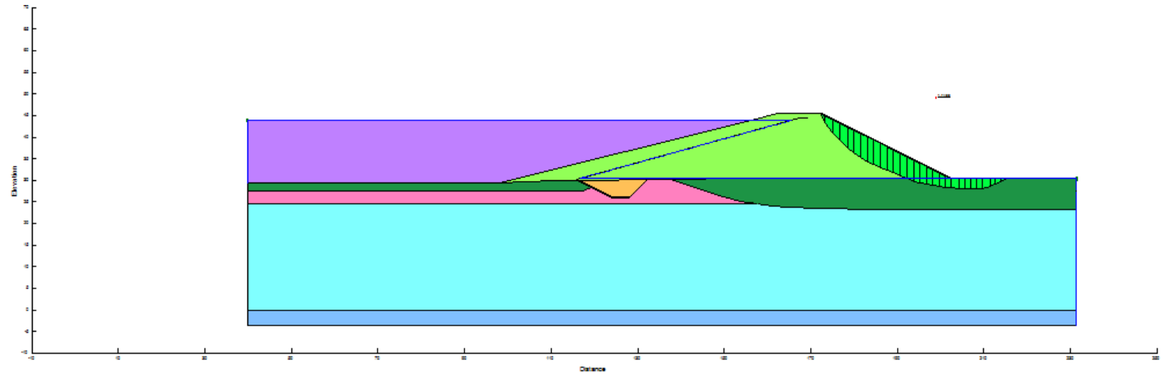
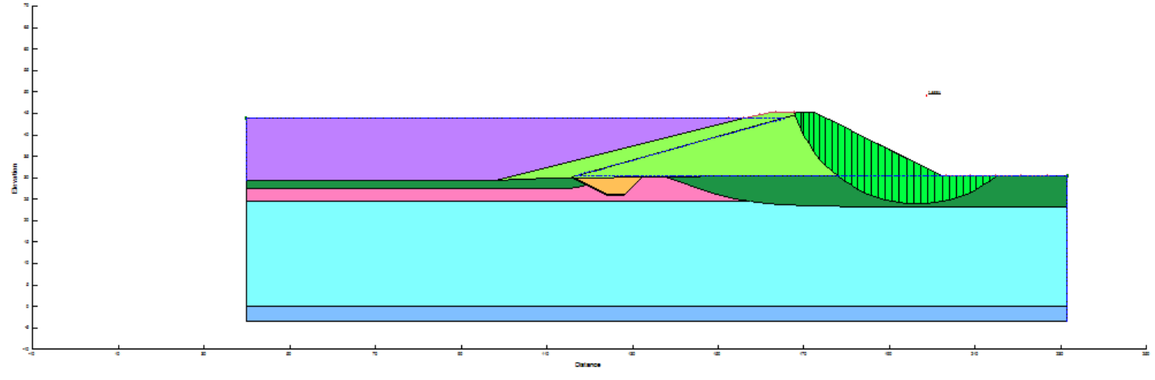
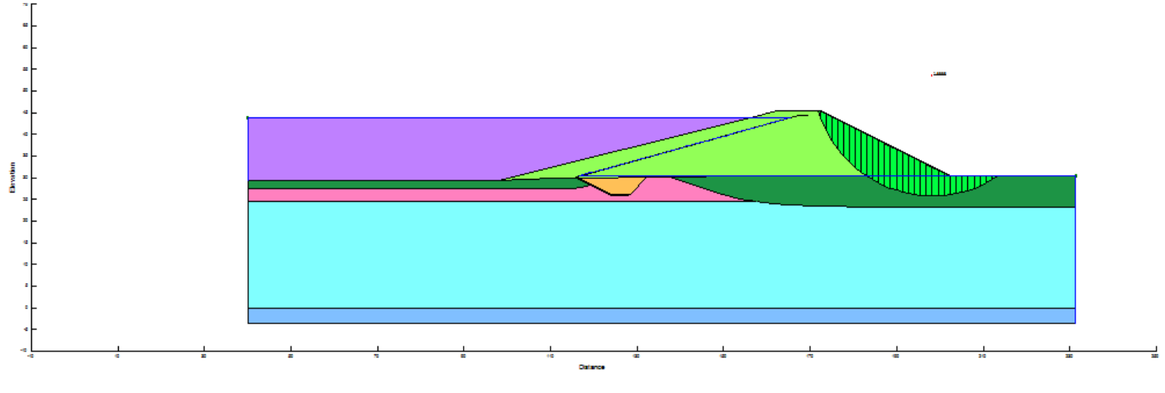
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model A-South Dam (Upstream Slope, Pseudo-Static, drained, 7m AL)	1.36	Fully Defined	
	2.94	Entry Exit	
	3.29	Grid and Radius	

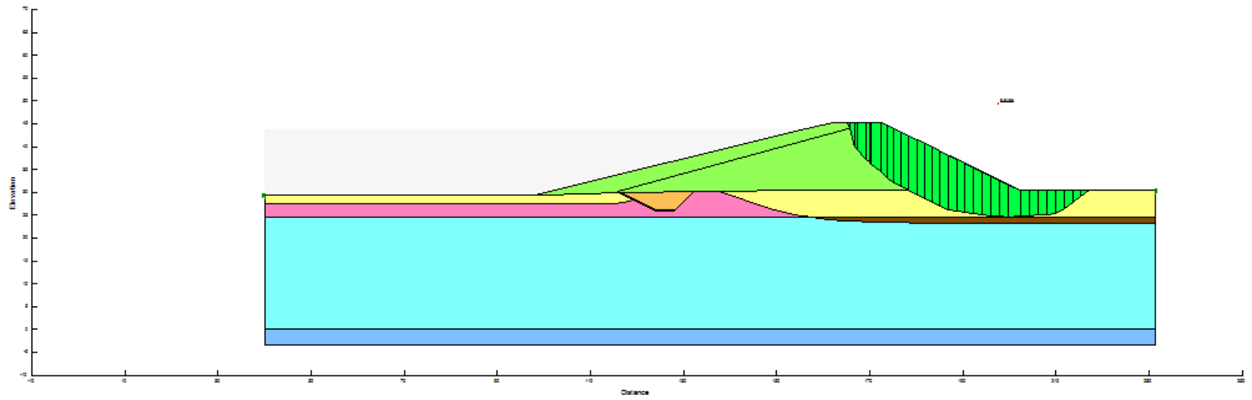
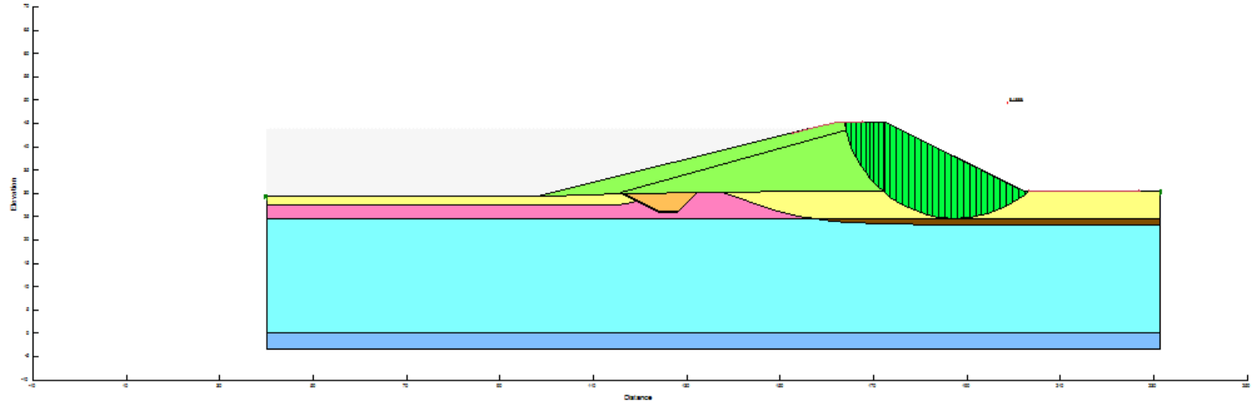
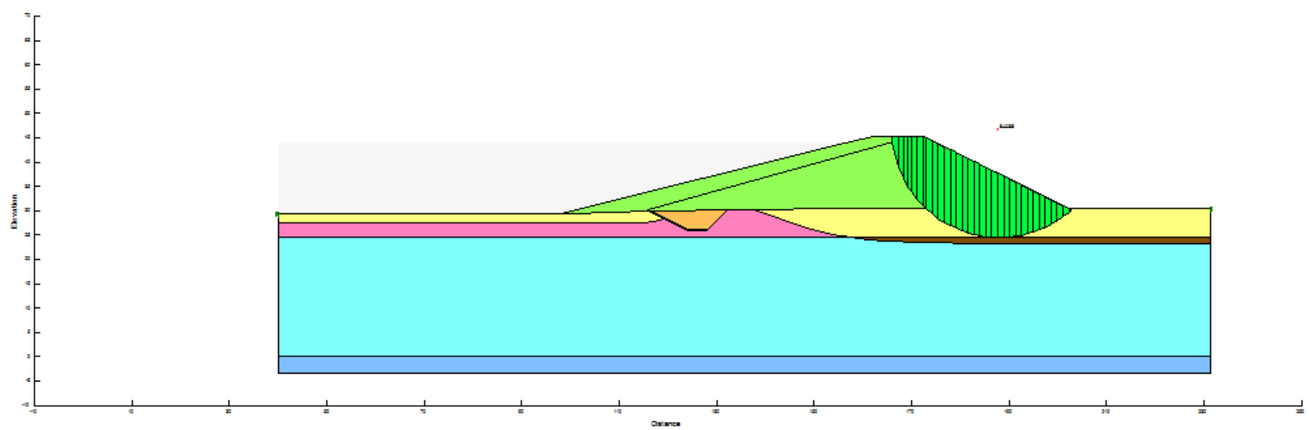
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model A-South Dam (Upstream Slope, Pseudo-Static, undrained, 7m AL)	1.36	Fully Defined	
	2.89	Entry Exit	
	3.04	Grid and Radius	

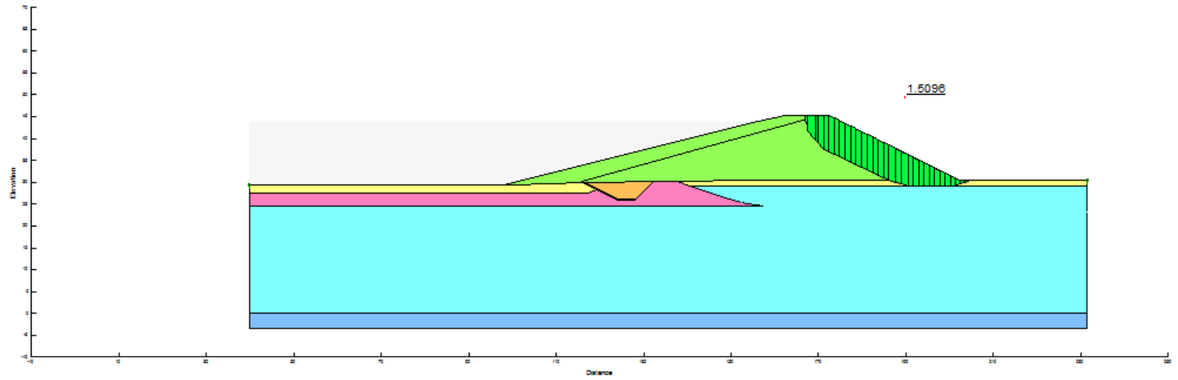
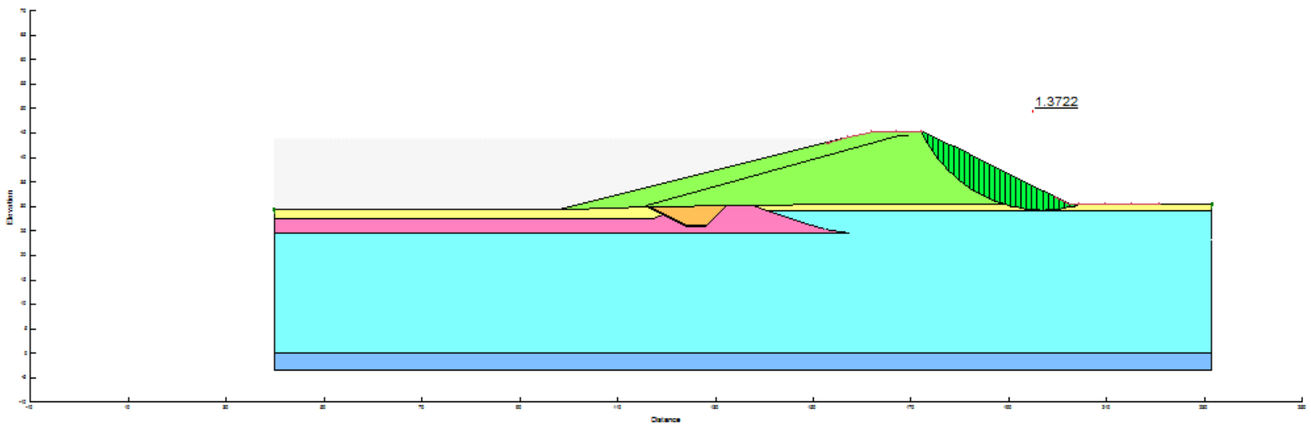
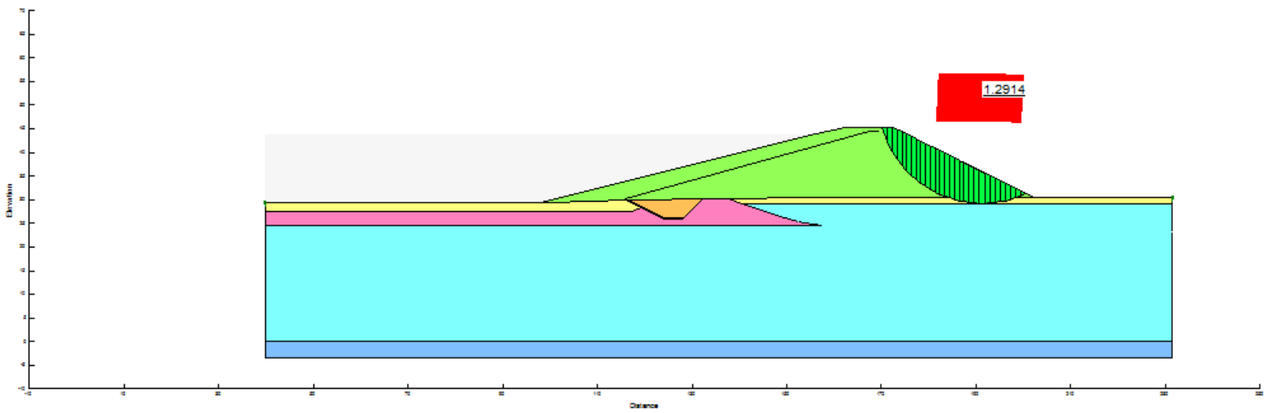
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Downstream Slope, Static, Drained, 7m AL)	1.74	Fully Defined	
	1.62	Entry Exit	
	1.57	Grid and Radius	

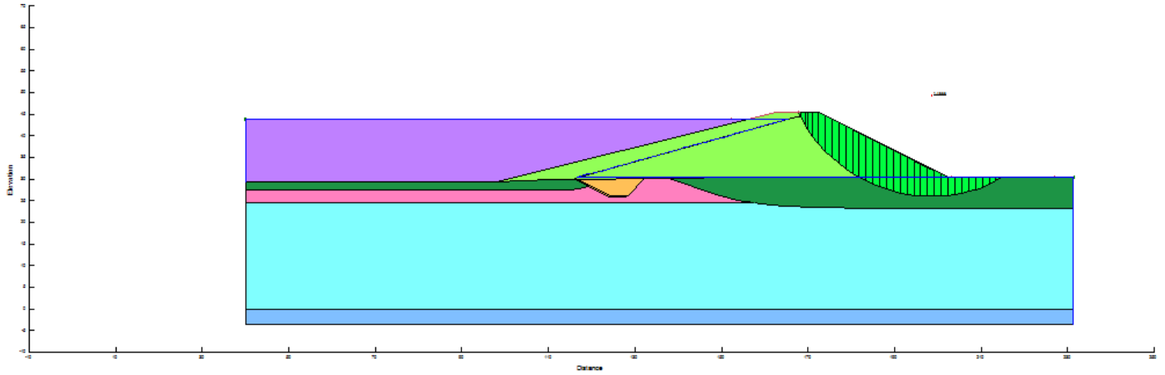
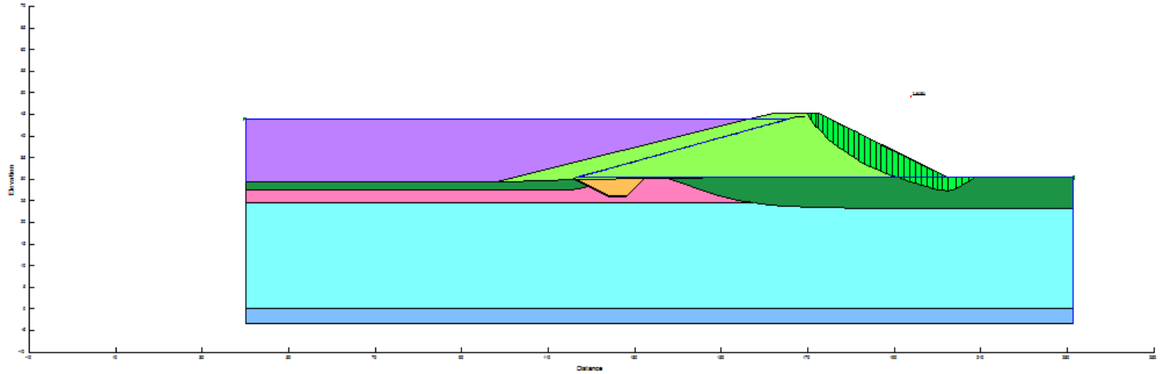
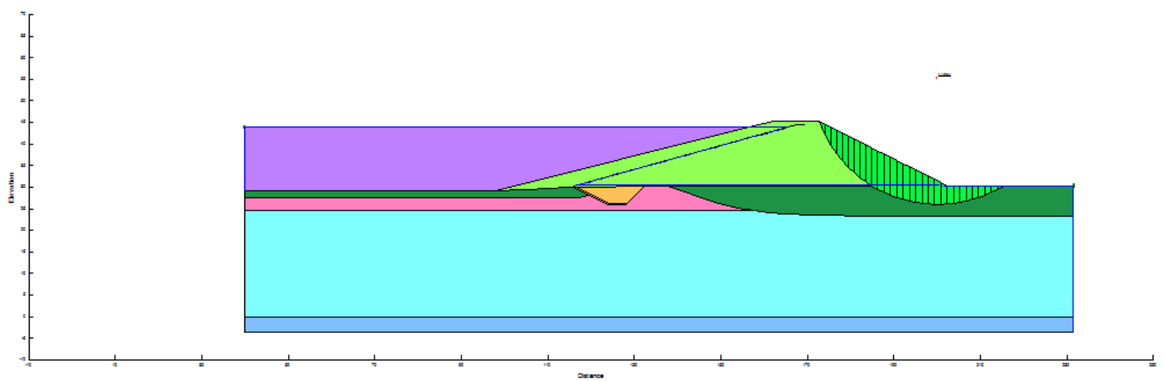
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Downstream Slope, Static, Undrained, 7m AL)	0.84	Fully Defined	
	0.50	Entry Exit	
	0.48	Grid and Radius	

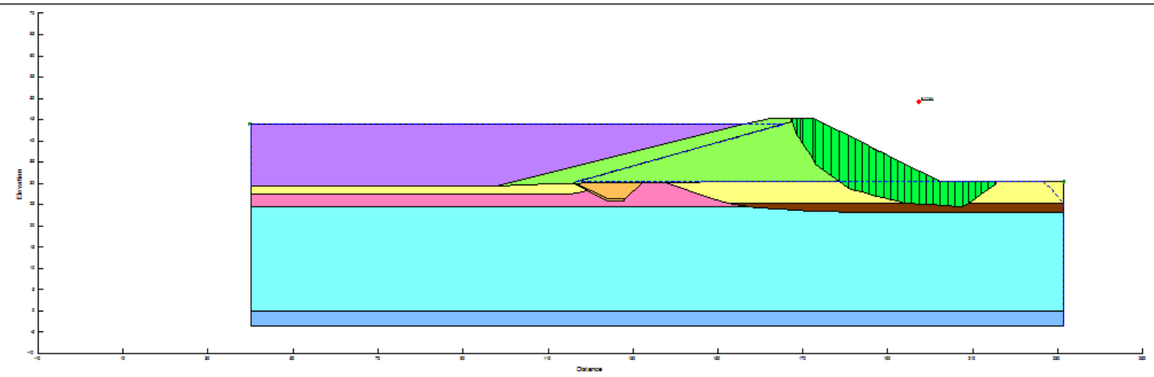
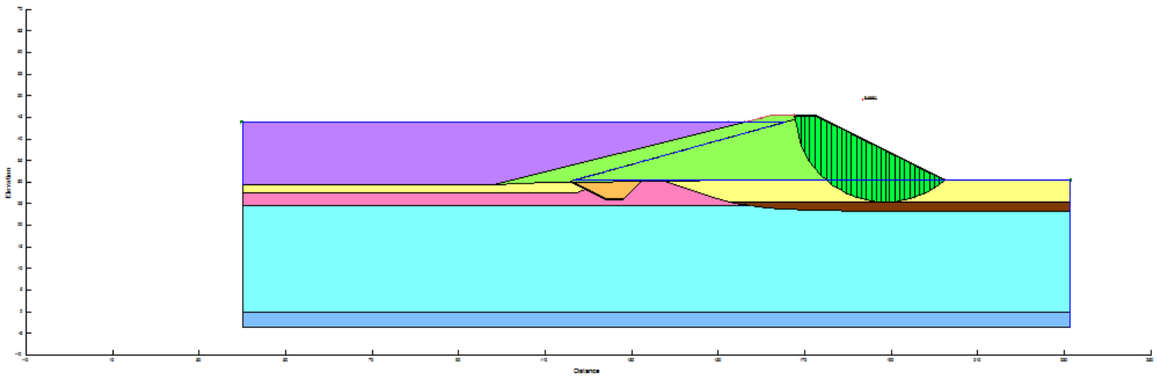
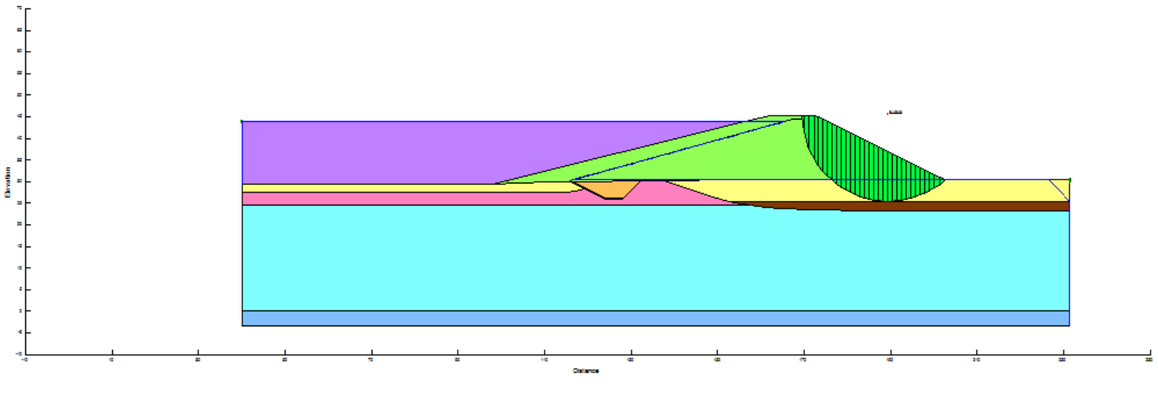
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Upstream Slope, Static, undrained, 1m AL)	1.51	Fully Defined	
	1.37	Entry Exit	
	1.29	Grid and Radius	

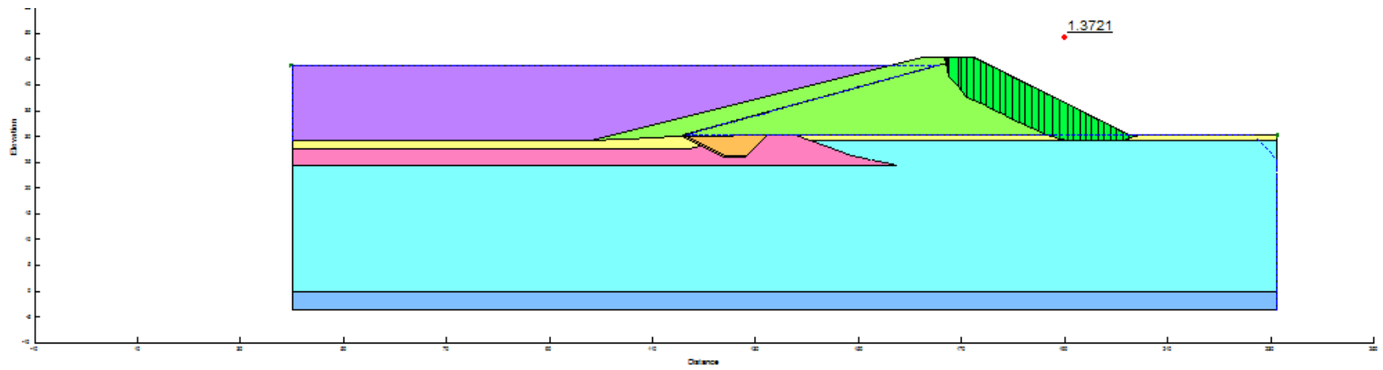
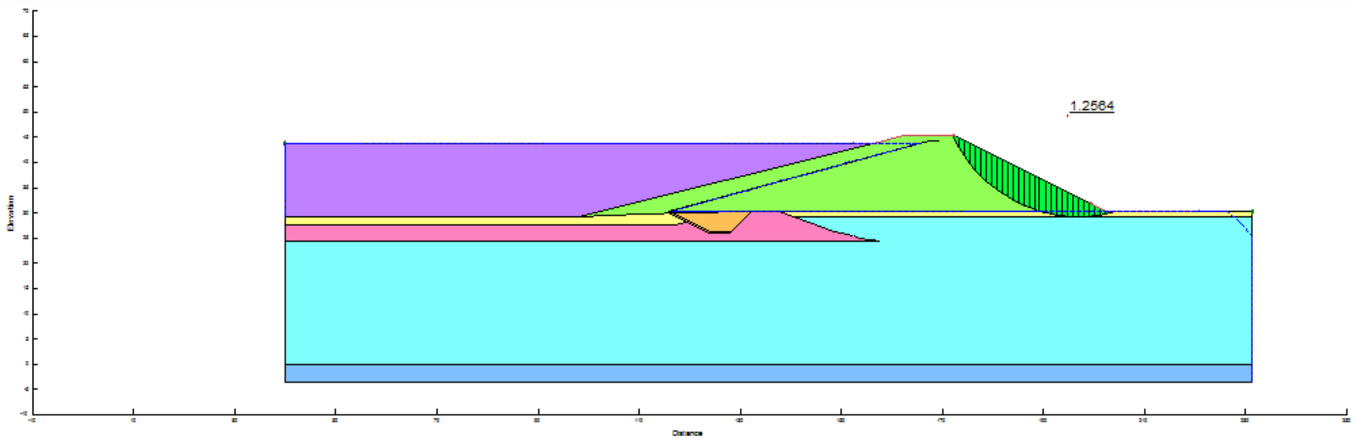
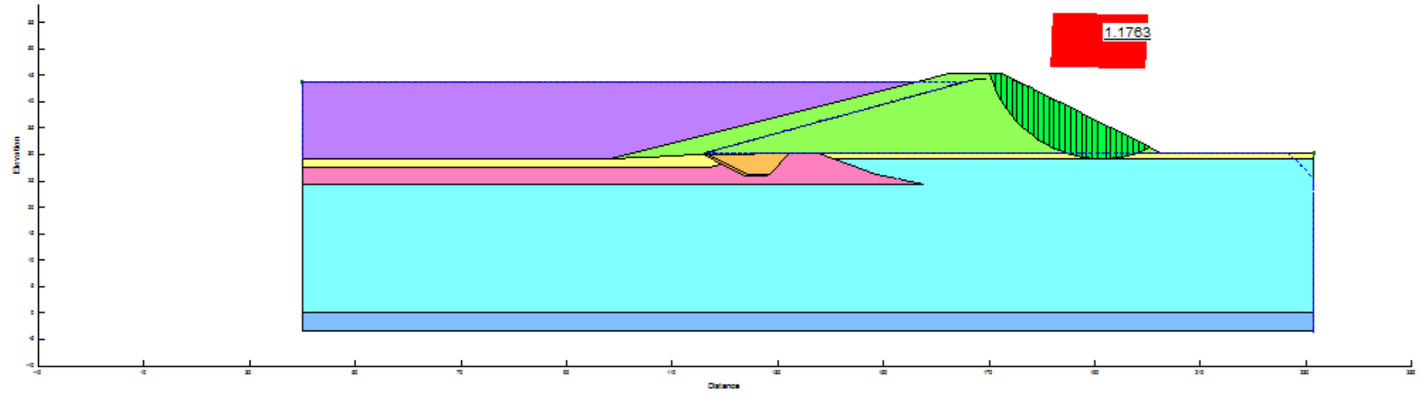
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Downstream Slope, Pseudo-Static, Drained, 7m AL)	1.55	Fully Defined	
	1.46	Entry Exit	
	1.42	Grid and Radius	

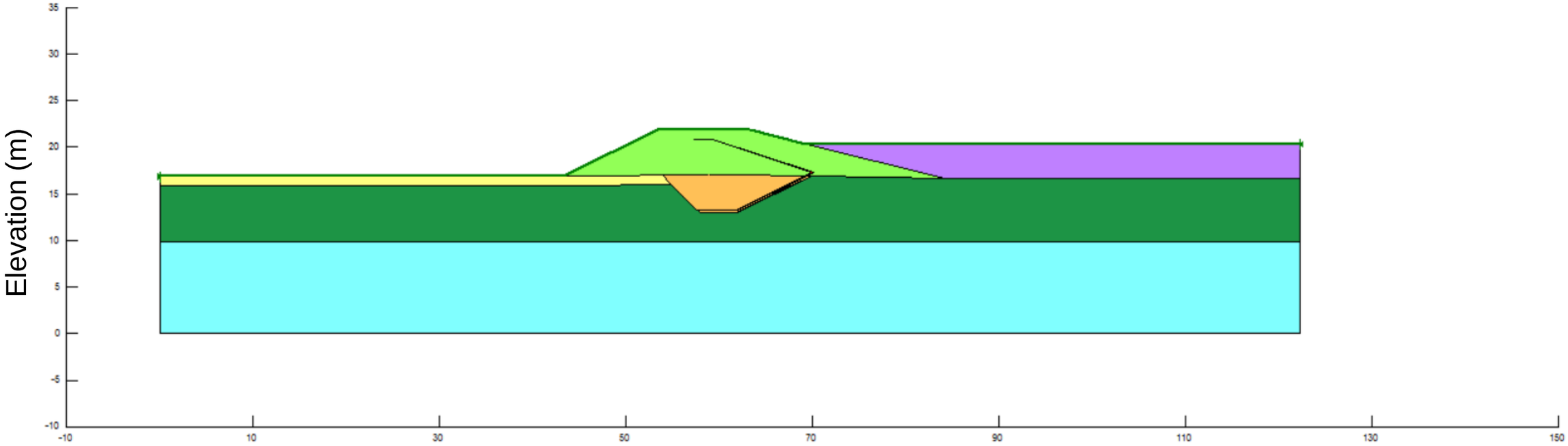
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Downstream Slope, Pseudo-Static, Undrained, 7m AL)	0.78	Fully Defined	
	0.57	Entry Exit	
	0.48	Grid and Radius	

Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model B-South Dam (Downstream Slope, Pseudo-Static, undrained, 1m AL)	1.347	Fully Defined	
	1.25	Entry Exit	
	1.18	Grid and Radius	

Attachment 3
West Dam Stability Analysis Results

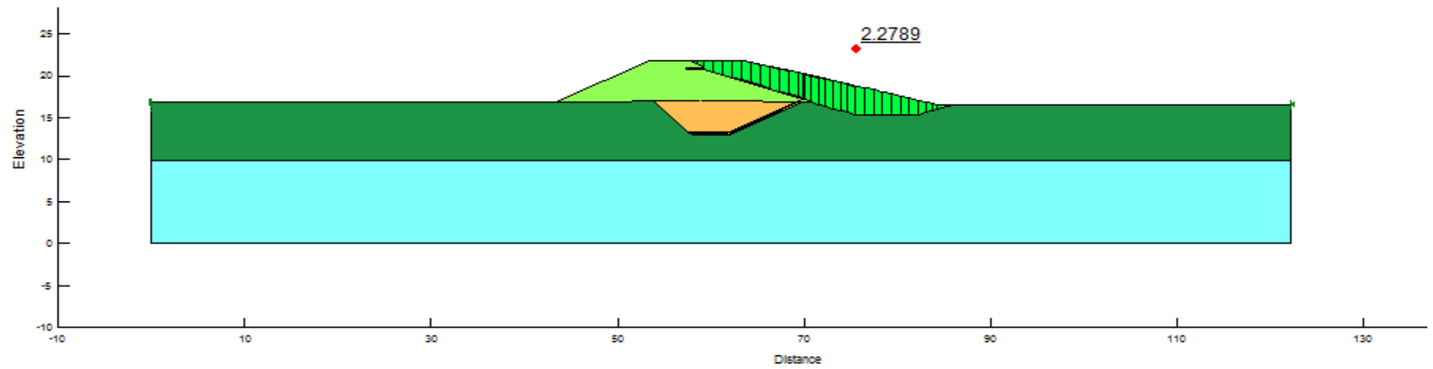
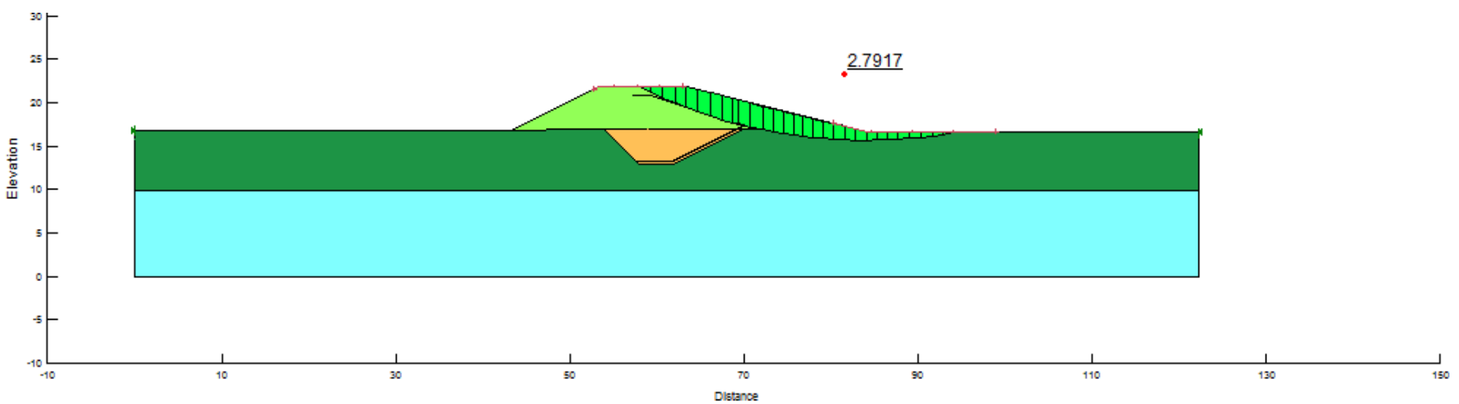
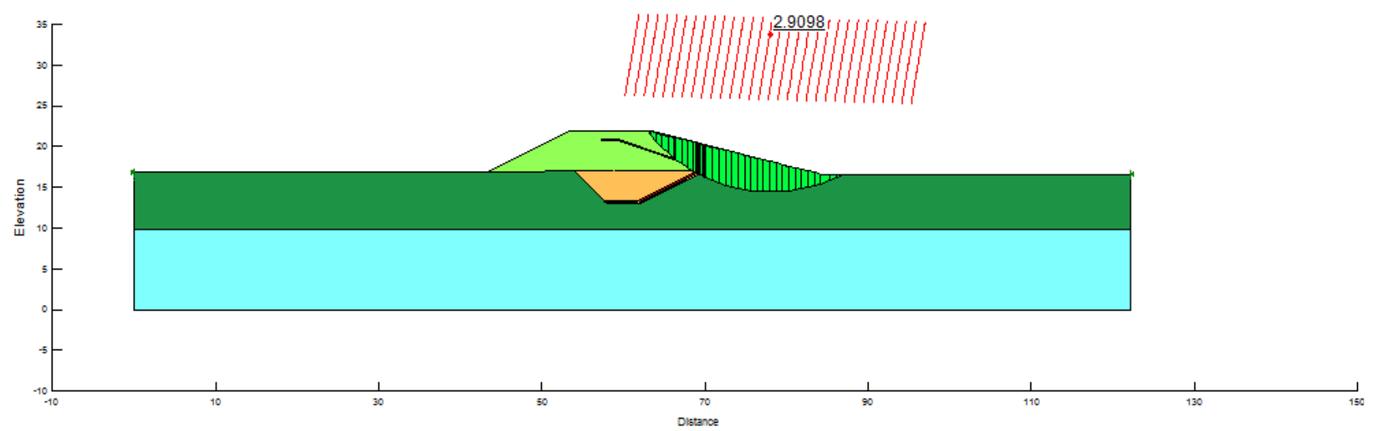


LEGEND

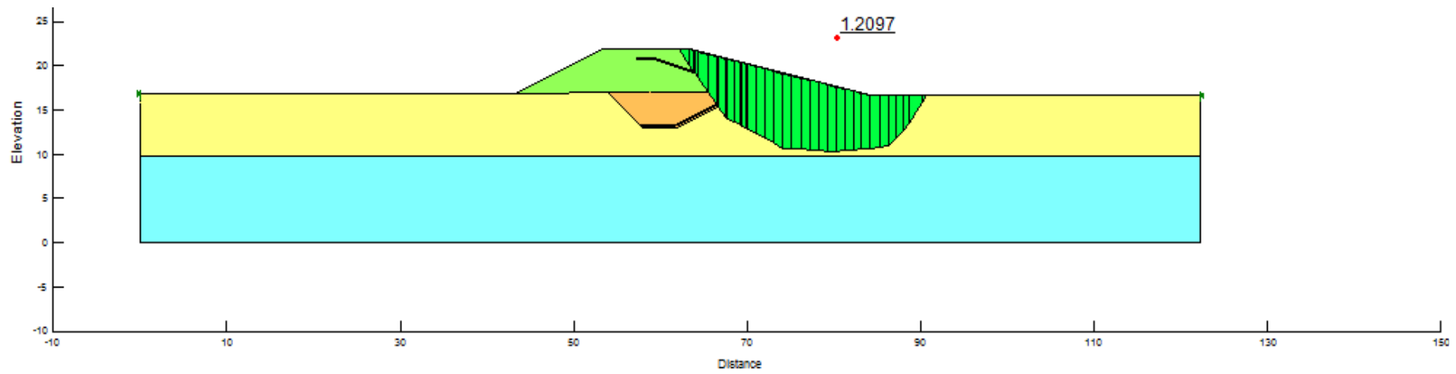
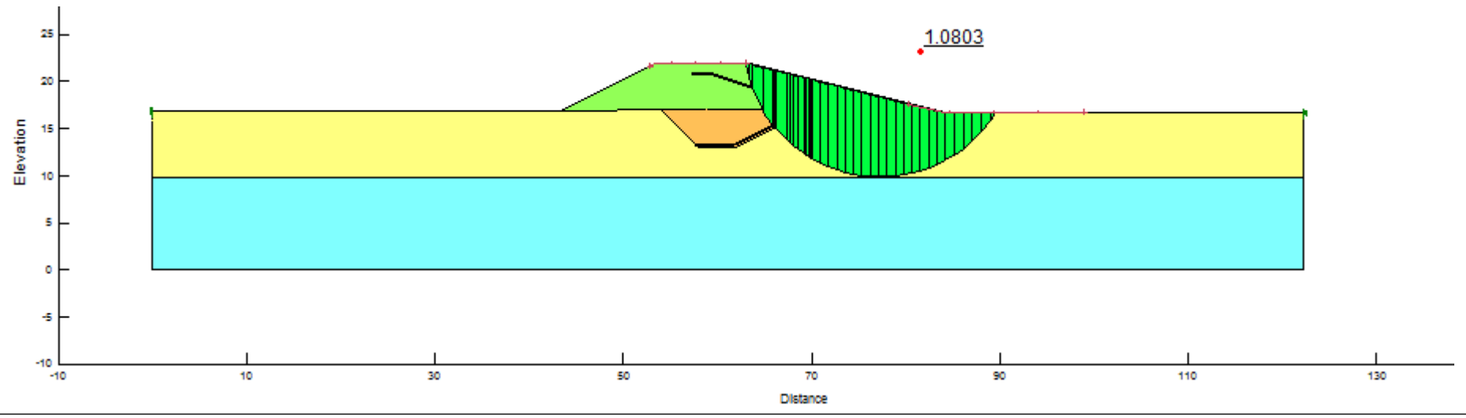
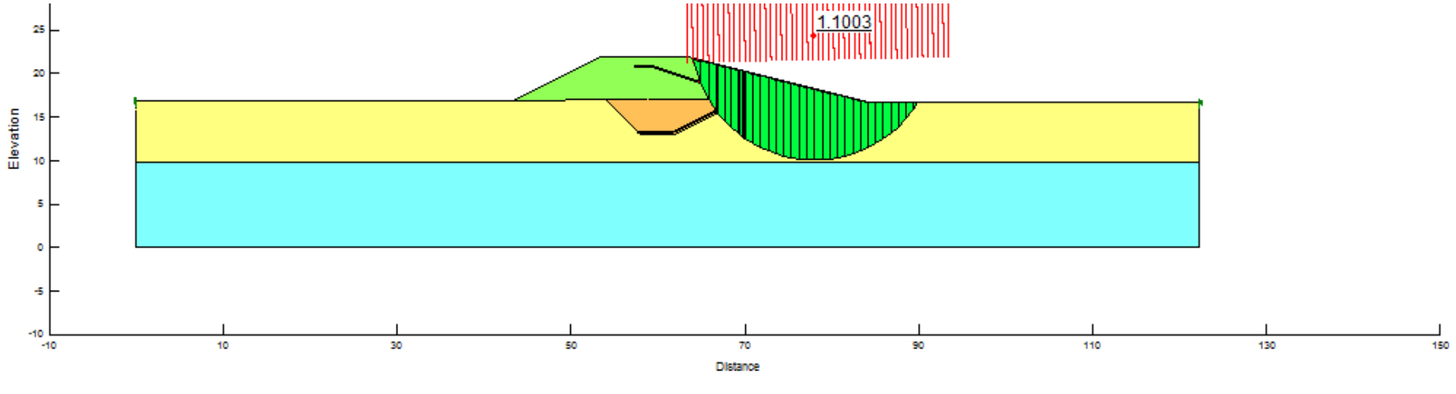
- ROQ (Non-Frozen)
- Marine Silt and Clay (Non-Frozen)
- Marine Silt and Clay (Frozen)
- ROQ (Frozen)
- Marine Silt and Clay (Top 5m)
- Tailings

srk consulting	TMAC RESOURCES	DORIS TIA SLOPE STABILITY
		West Dam Analyzed Cross Section
Job No: 1CT22.004.610 Filename: HopeBay_DorisNorth_SouthDam_SlopeStability_160407_sa	HOPE BAY PROJECT	Date: Dec 2016 / Approved: / Figure: **1**

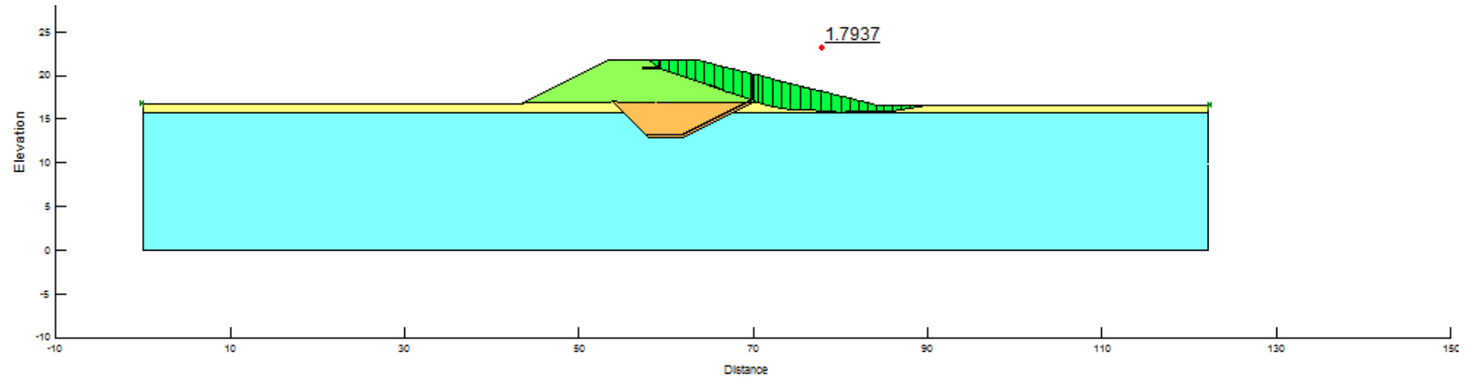
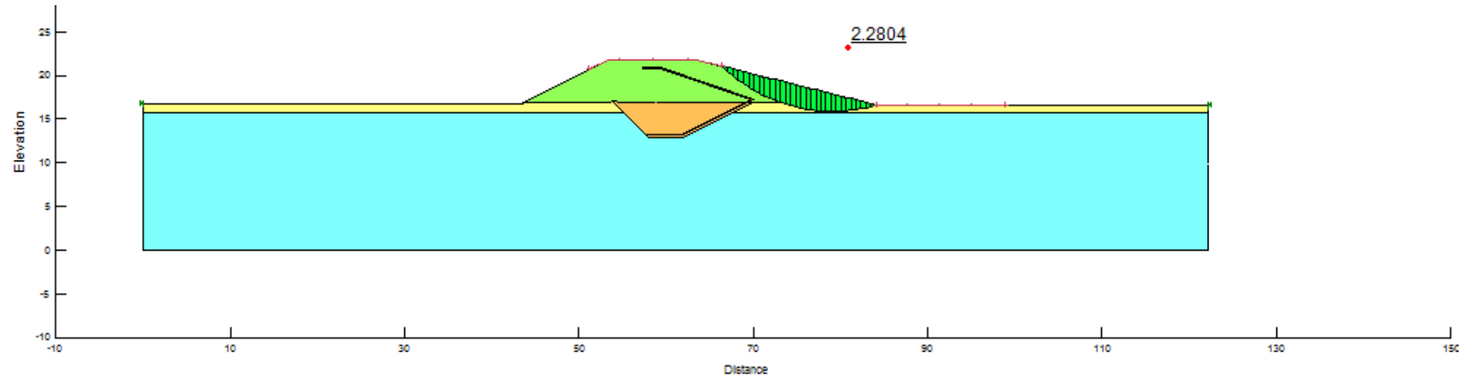
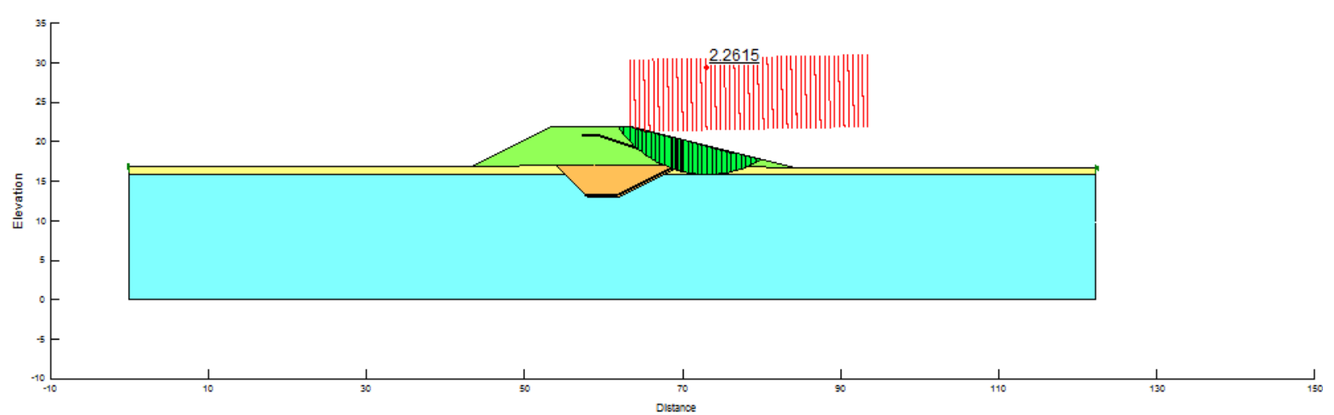
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Static, drained, 7m AL)	2.28	Fully Defined	
	2.79	Entry Exit	
	2.91	Grid and Radius	

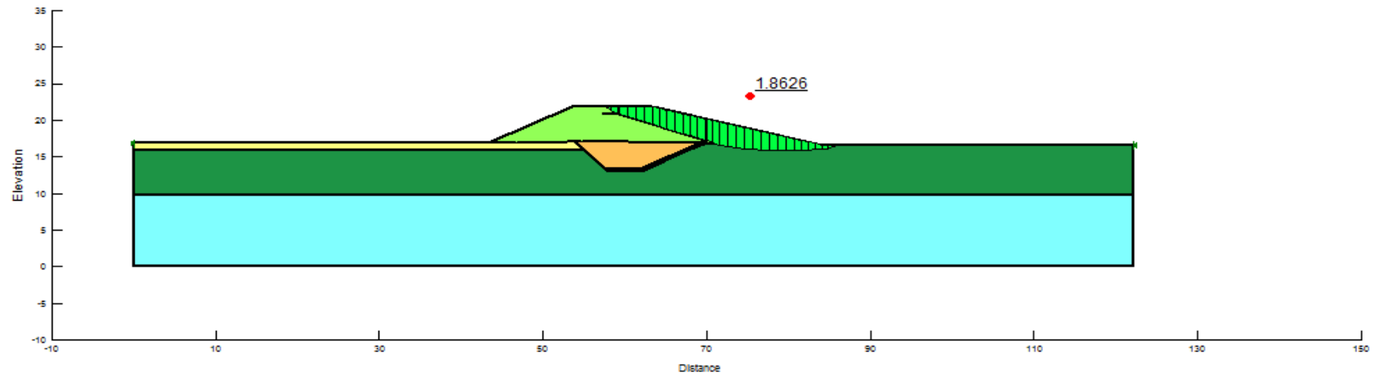
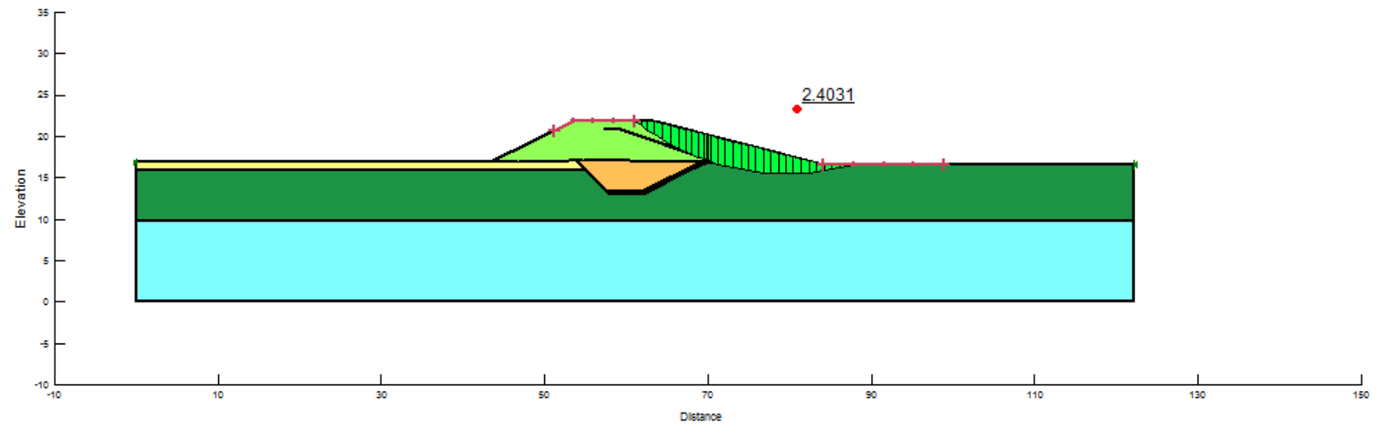
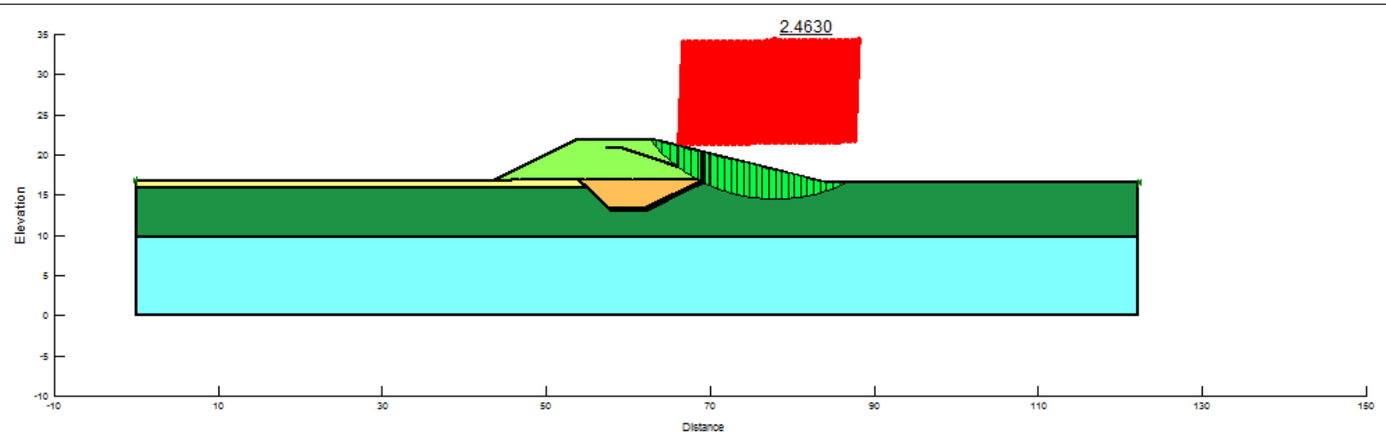
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Static, undrained, 7m AL)	1.2	Fully Defined	
	1.08	Entry Exit	
	1.10	Grid and Radius	

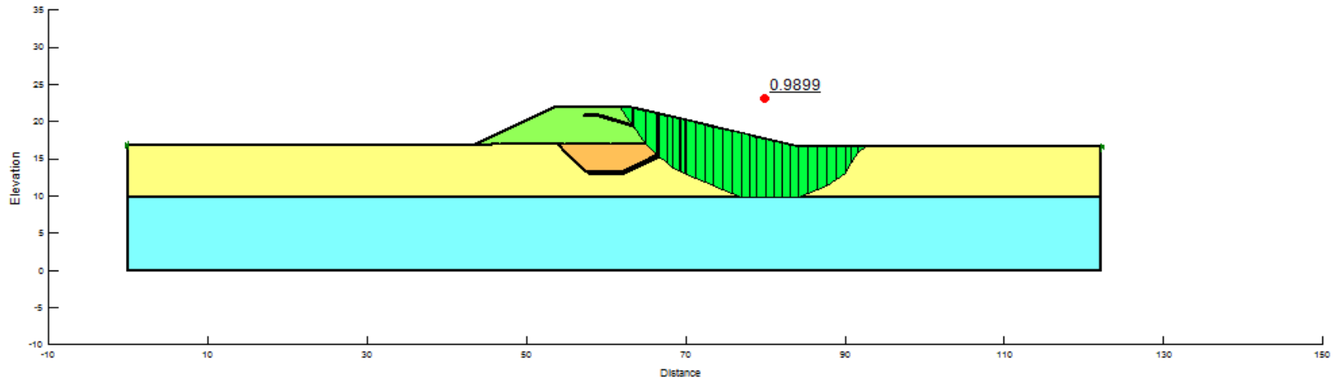
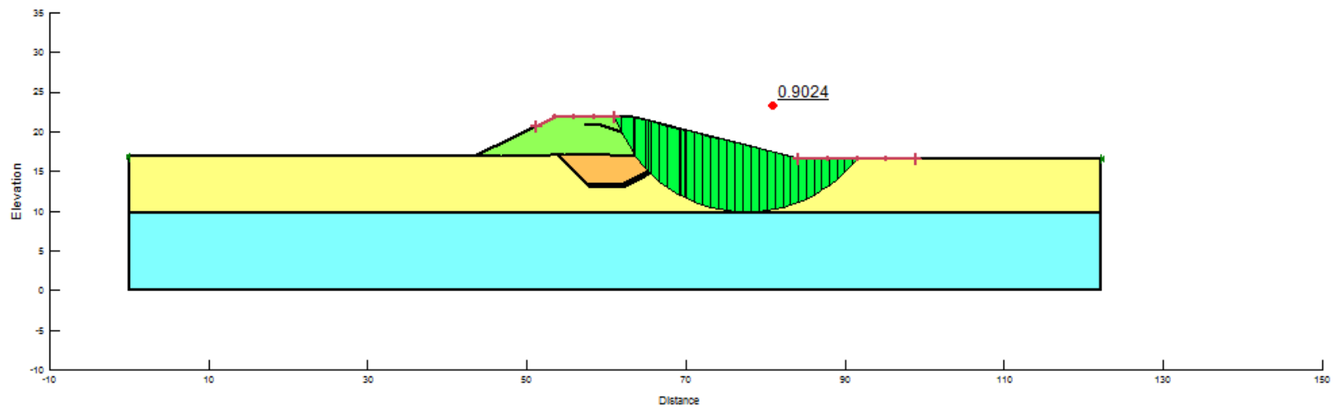
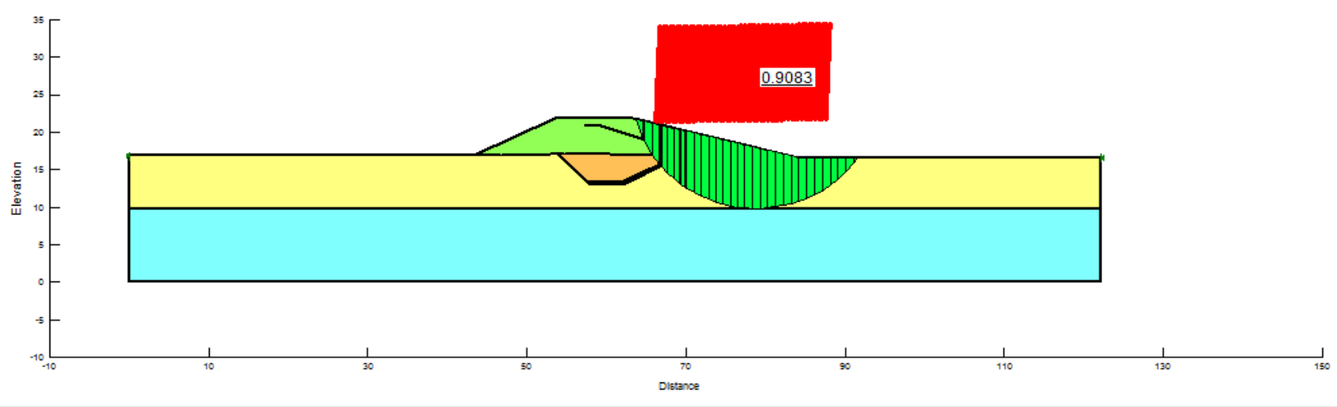
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Static, undrained, 1m AL)	1.79	Fully Defined	
	2.28	Entry Exit	
	2.26	Grid and Radius	

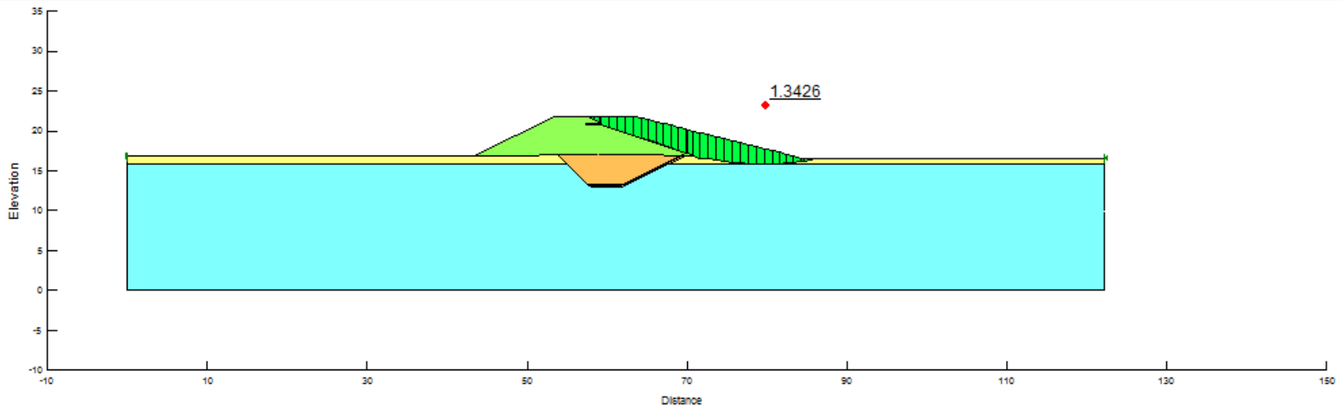
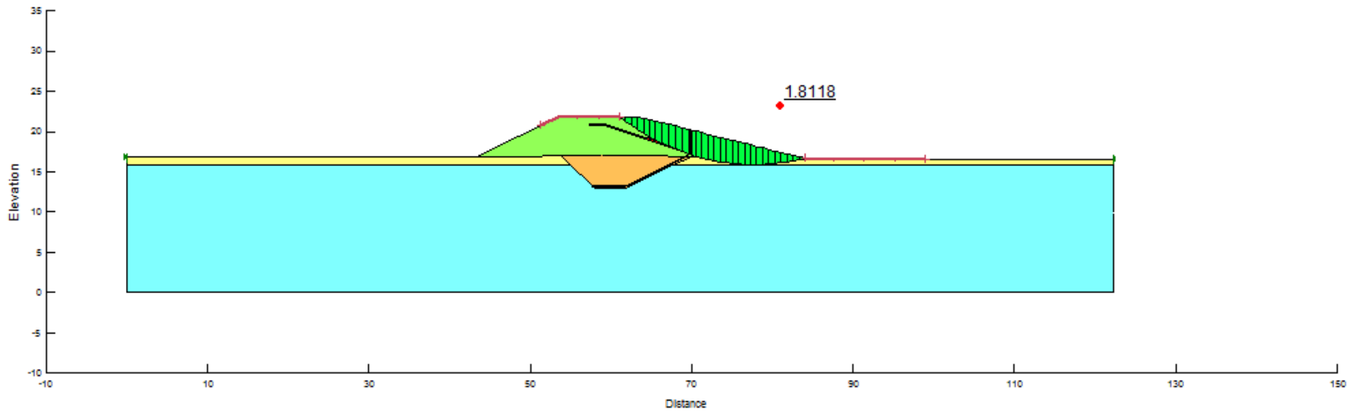
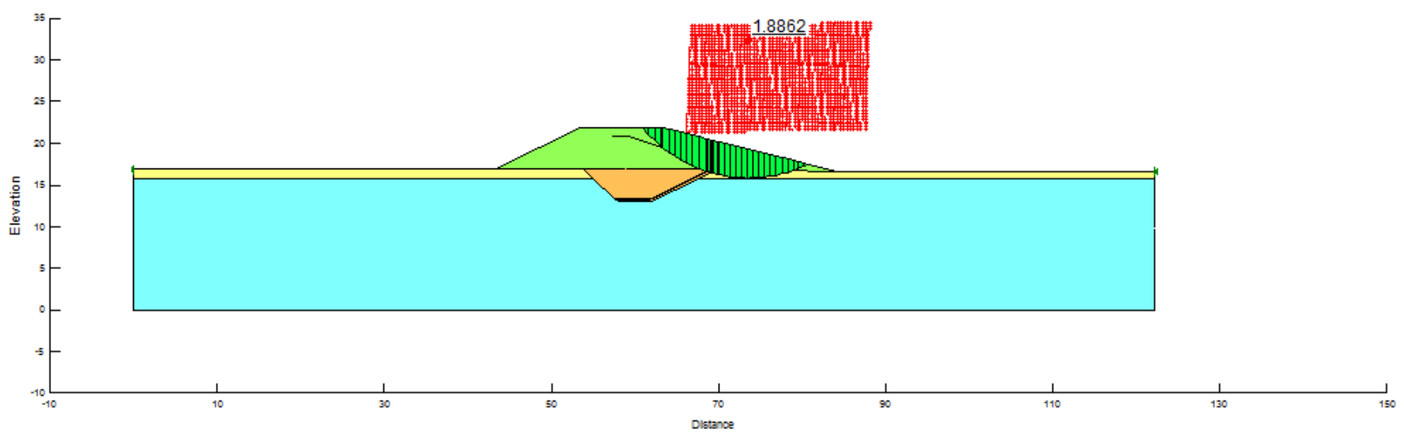
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Pseudo-Static, Drained, 7m AL)	1.86	Fully Defined	
	2.40	Entry Exit	
	2.46	Grid and Radius	

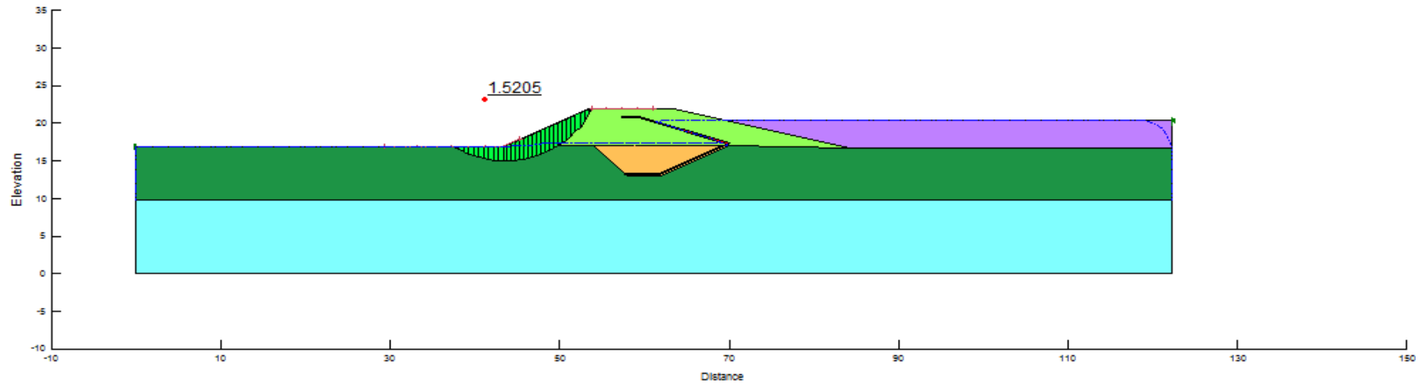
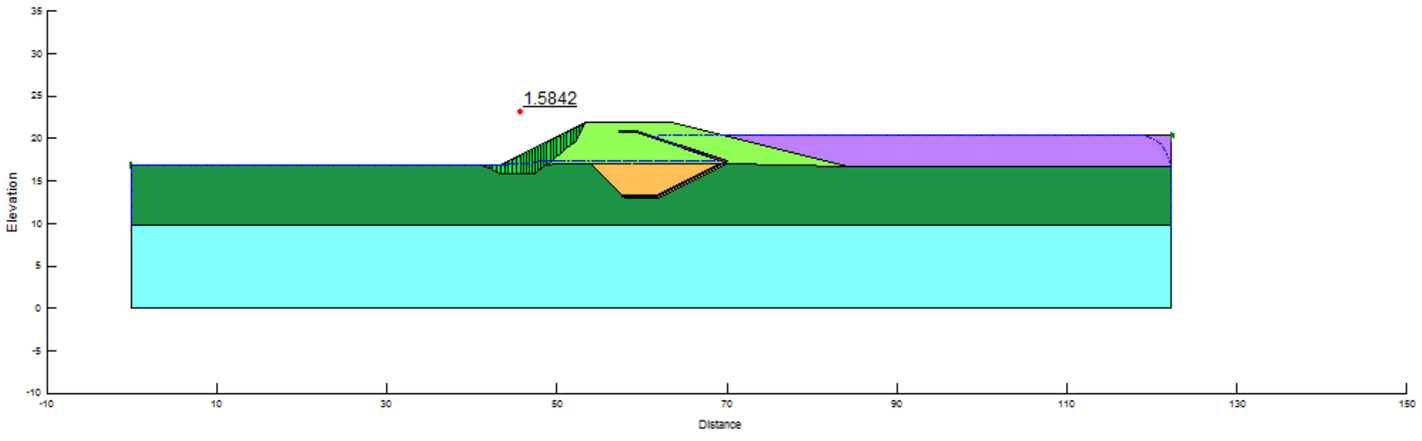
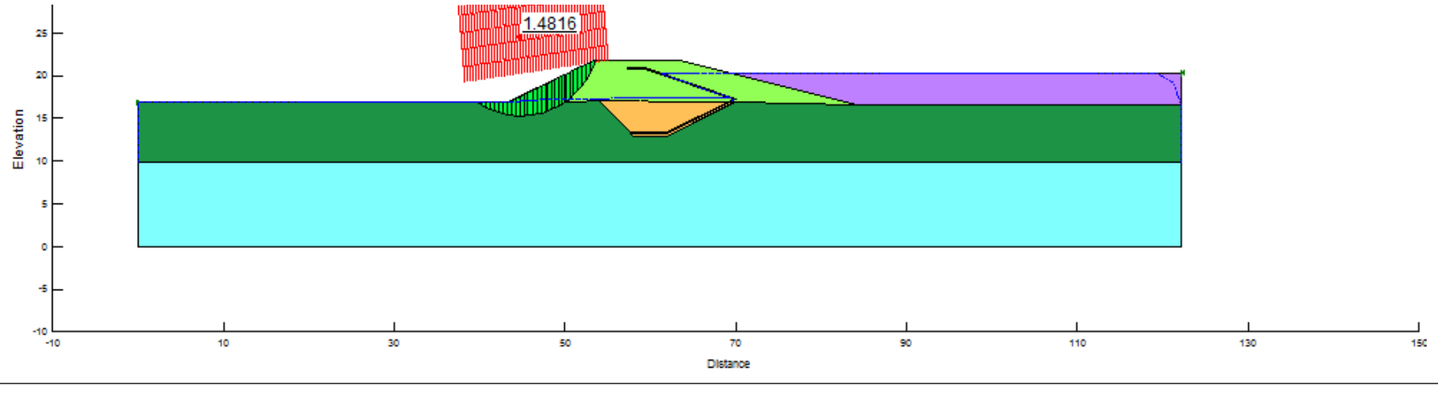
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Pseudo-Static, Undrained, 7m AL)	0.99	Fully Defined	
	0.90	Entry Exit	
	0.91	Grid and Radius	

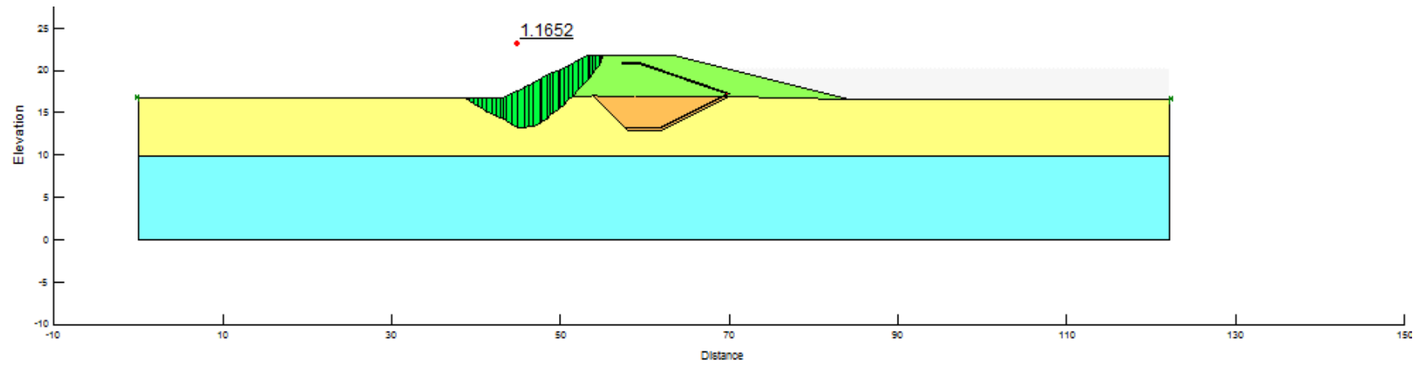
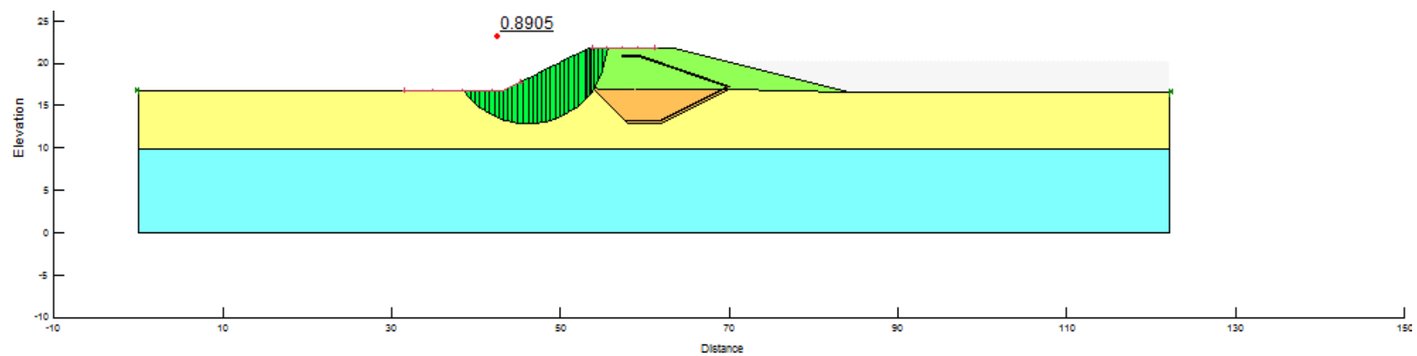
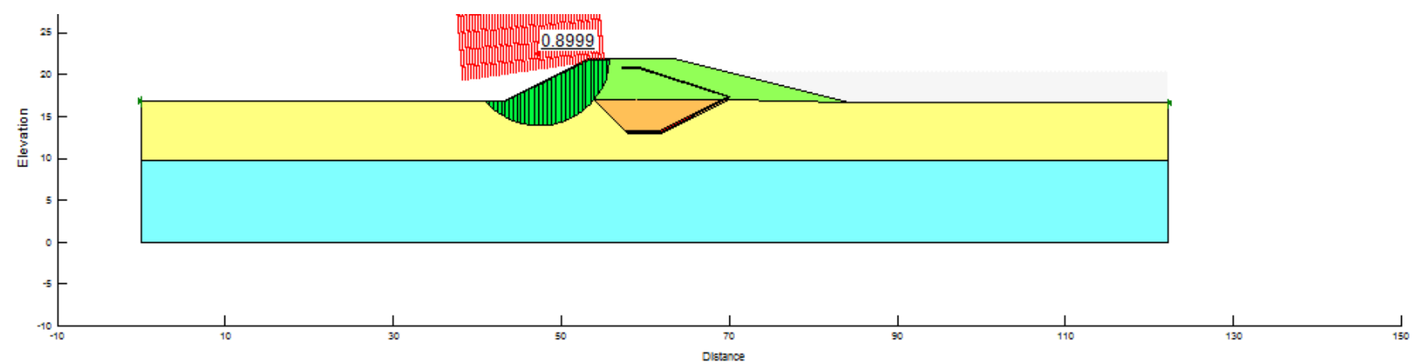
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model C- West Dam (Upstream Slope, Pseudo-Static, Undrained, 1m AL)	1.34	Fully Defined	
	1.81	Entry Exit	
	1.89	Grid and Radius	

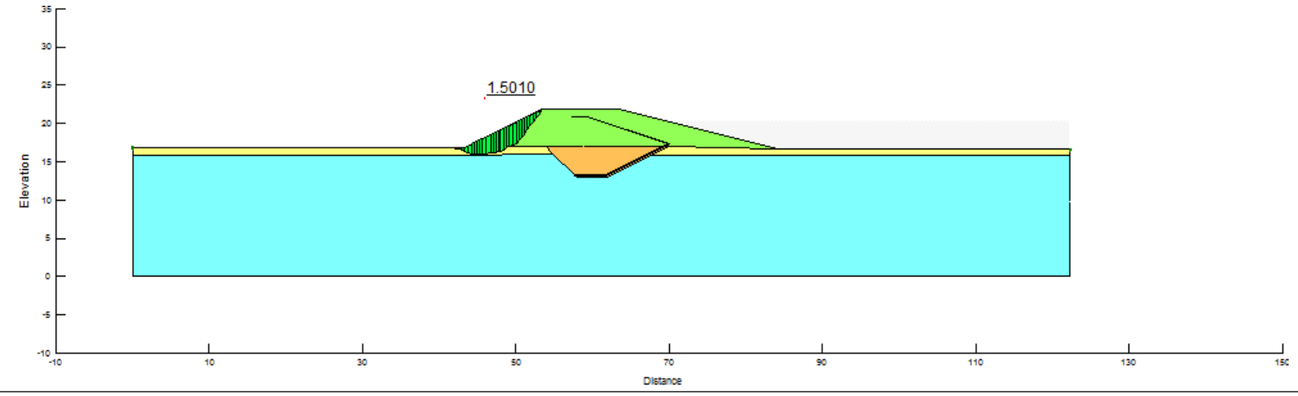
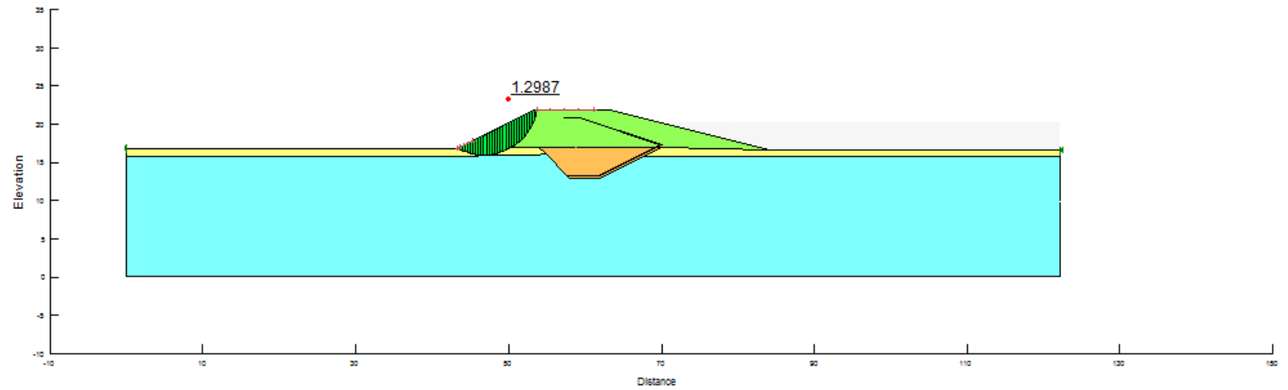
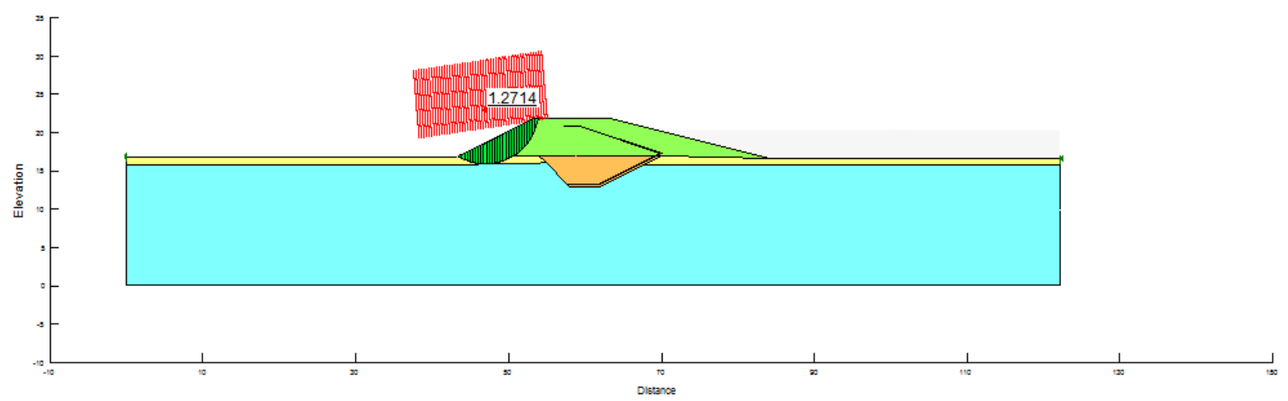
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (downstream Slope, Static, drained, 7m AL)	1.58	Fully Defined	
	1.52	Entry Exit	
	1.48	Grid and Radius	

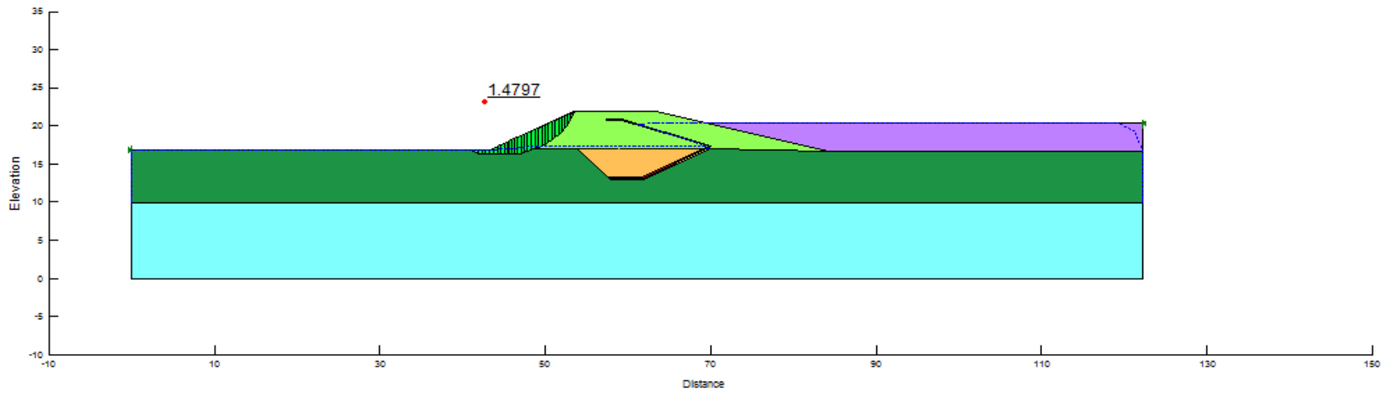
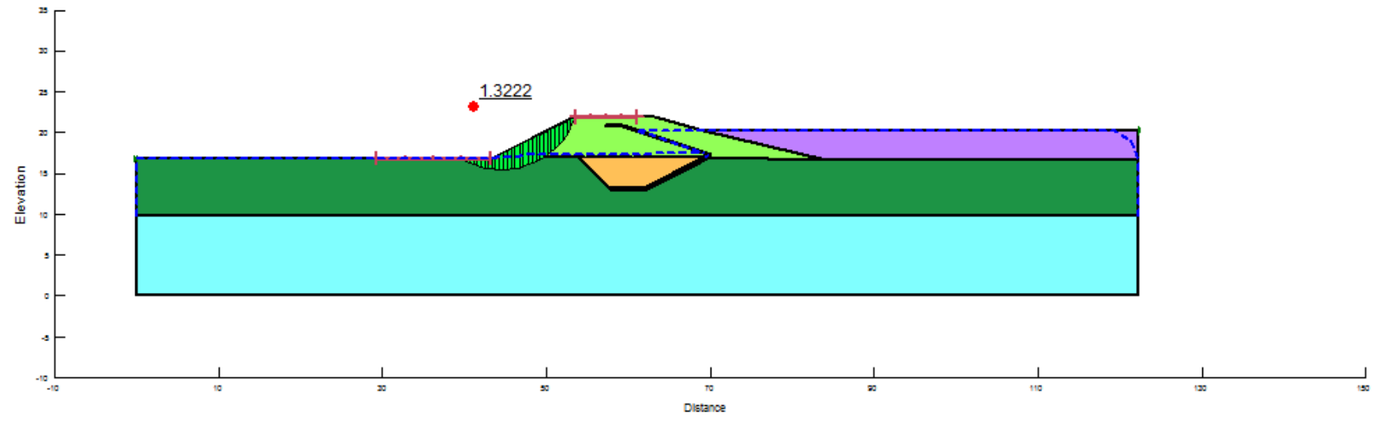
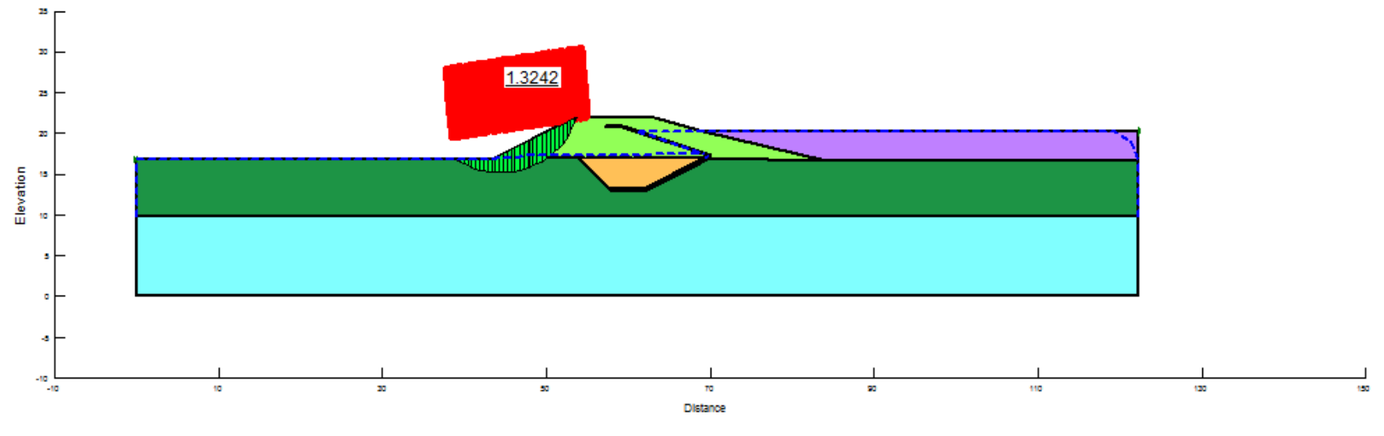
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (downstream Slope, Static, undrained, 7m AL)	1.16	Fully Defined	
	0.89	Entry Exit	
	0.9	Grid and Radius	

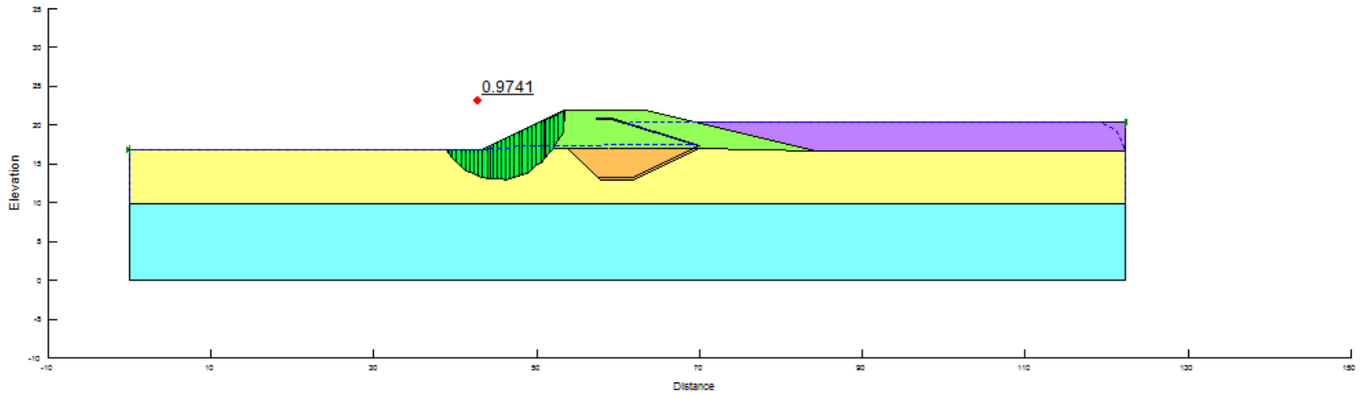
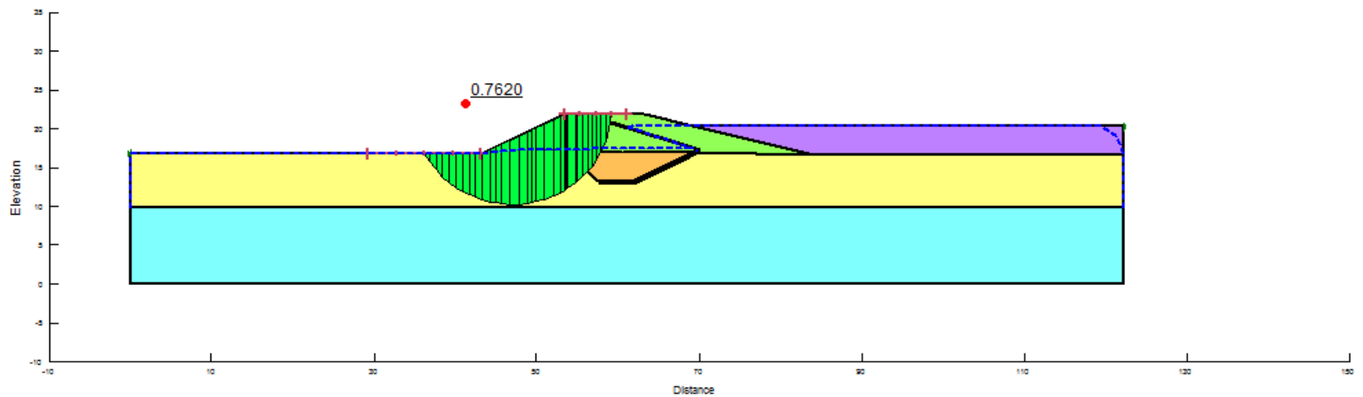
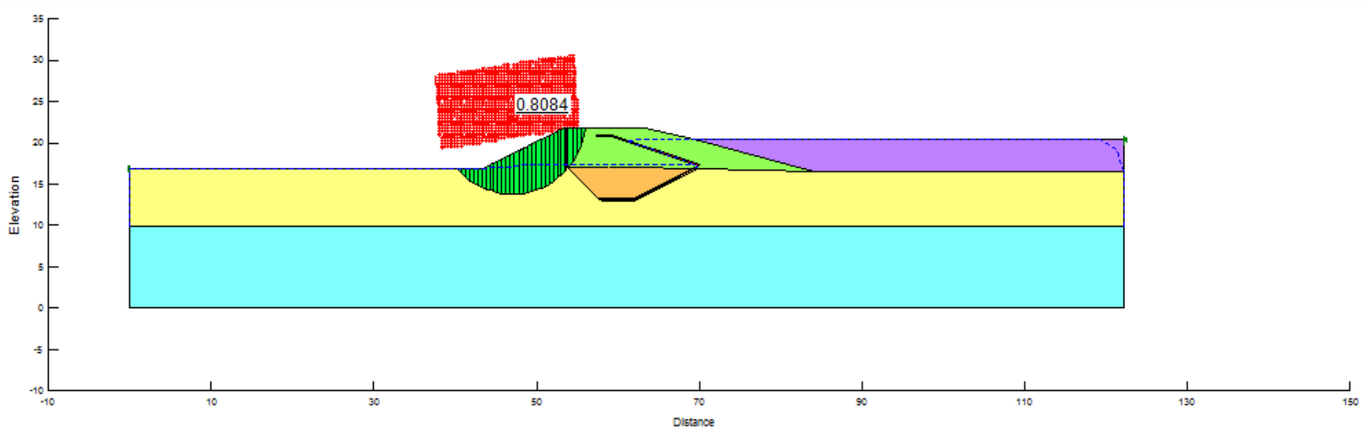
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (downstream Slope, Static, undrained, 1m AL)	1.50	Fully Defined	
	1.30	Entry Exit	
	1.27	Grid and Radius	

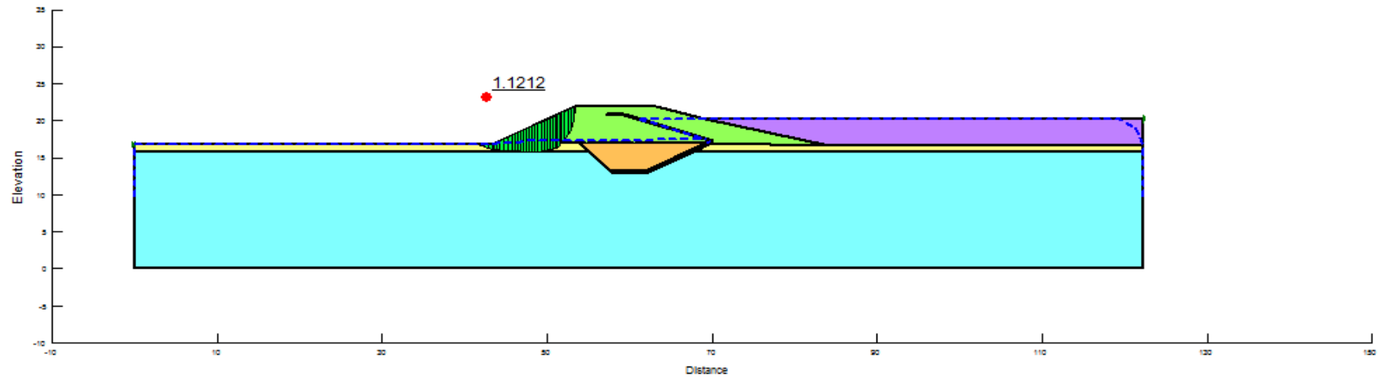
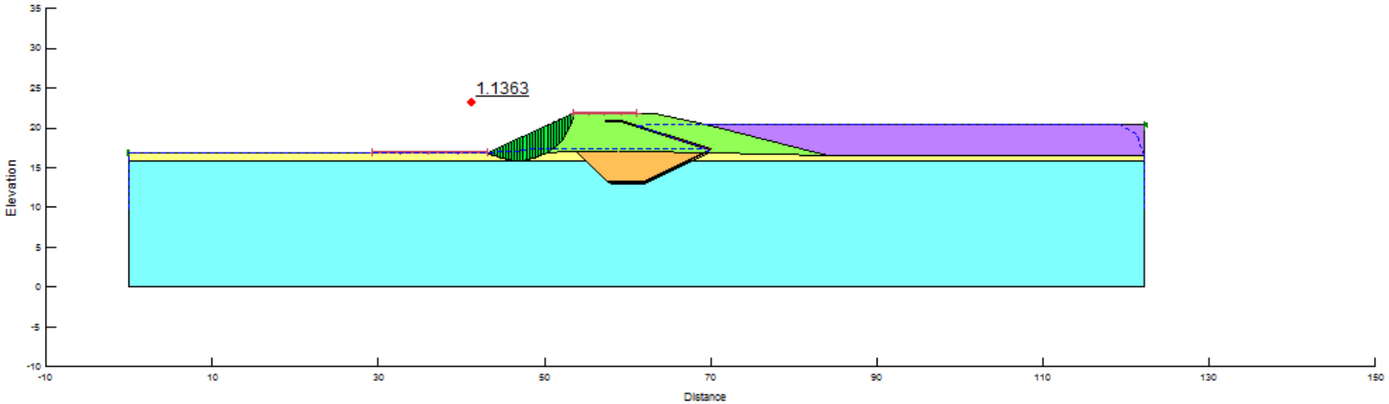
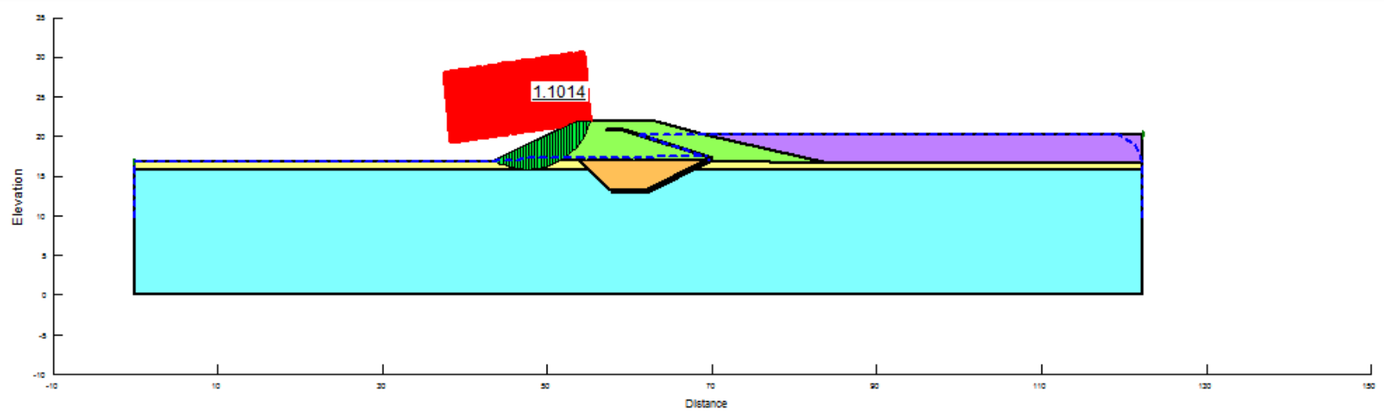
Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (Downstream Slope, Pseudo-Static, Drained, 7m AL)	1.48	Fully Defined	
	1.32	Entry Exit	
	1.32	Grid and Radius	

Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (Downstream Slope, Pseudo-Static, undrained, 7m AL)	0.97	Fully Defined	
	0.76	Entry Exit	
	0.81	Grid and Radius	

Slope Stability Results

Model	Factor of Safety	Slip Surface Option	Figure
Model D- West Dam (Downstream Slope, Pseudo-Static, undrained, 1m AL)	1.12	Fully Defined	
	1.14	Entry Exit	
	1.10	Grid and Radius	

Appendix D – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area North Dam Thermal Modeling

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Christopher W. Stevens, PhD	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Doris Tailings Impoundment Area North Dam Thermal Modeling		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix D as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes		

1 Introduction

1.1 General

The Madrid-Boston Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris only, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Phase 1 tailings are deposited sub-aerially in the Doris TIA, formerly Tail Lake, located approximately 5 km from the Doris mill. Containment would be provided by three retention structures; a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam); and an Interim Dike situated at approximately the midpoint of the facility. Tailings would be deposited sub-aerially between the South Dam and Interim Dike, and the Reclaim Pond will be contained between the Interim Dike and the North Dam.

The North Dam was constructed over two winters (2011 and 2012) and has impounded water since 2011 (Figure 1). The South Dam and Interim Dike are scheduled for construction in 2017. Phase 2 tailings deposition would include a continuation of the Doris TIA with raising of the South Dam and construction of a new West Dam (SRK 2017a). Thermal analysis completed for the South Dam and West Dam is reported in SRK (2017b).

The North Dam relies on a frozen ice-saturated core and foundation to achieve the required water retention properties. A geosynthetic clay liner (GCL) was installed along the upstream side of the frozen core to provide secondary water-retaining capability in case cracks develop in the core caused by thermal expansion or creep deformation. In order to ensure adequate performance of the dam, it is imperative to maintain the frozen state of the core and foundation over the design life.

The original design life of the North Dam was 25 years (SRK 2007). As part of Phase 2 tailings deposition, the North Dam design life would be extended to 2041 (30 years from 2011). This timeline assumes a period of nominal water impoundment prior to start of tailings deposition in 2017, active tailings deposition between 2017 and 2036, and a five-year post closure period prior to breaching of the dam in 2041.

1.2 Modeling Objective

The objective of the North Dam modeling was to determine if the current configuration of the North Dam is suitable to maintain the thermal design criteria over a 40-year design life, which is ten years greater than the actual planned design life of 30 years. The thermal conditions at the end of the planned design life is also used to conduct a creep analysis for the North Dam (SRK 2017c).

The North Dam was modeled along two critical cross sections; Section 0+85, located at the thickest section of the dam, and Section 0+40 near one of the thinnest sections of the dam (Figure 2). Ground temperature measurements are collected at both these sections, which allows for model calibration.

2 North Dam Details

2.1 As-built Overview

2.1.1 General

The North Dam is located across the Tail Lake outlet and extends approximately 200 m long and 10 m high, with upstream and downstream slopes of 6H:1V and 4H:1V, respectively (Figure 1 through Figure 3). The dam as-built report, drawings, and quality control and quality assurance documentation is provided in SRK (2012).

2.1.2 Foundation

The overburden soils are up to 20 m thick at the base of the valley and thin out at the dam abutments. About two-thirds of the dam longitudinal section is characterized by ice-saturated sand of approximately 10 m to 15 m thick. The sand deposit is overlain by a silt and clay layer that is less than 3 m thick. The remaining one-third portion of the dam alignment is characterized by marine clayey silt that is up to 15 m thick. The fine-grained materials are also ice-saturated and contain excess ground ice. At the North Dam, the average porewater salinity is 39 ppt, with a freezing point depression of -2.2°C (Geometric mean of 30 ppt. and -1.7°C). The average site-wide freezing point depression is -0.8°C for sand and -2.2°C for silt and clay, with an average of -2.1°C for all samples collected at the Project site (SRK 2017d). Bedrock is generally competent basalt.

2.1.3 Dam Construction

The North Dam was constructed over the winters of 2011 and 2012, and consists of three major regions; the frozen core, transition zone, and dam shell (Figure 3).

The key trench was excavated in 2011 using drill and blast methods. A hyper-saline zone comprised of clayey silt with an average porewater salinity of 45 ppt. and a freezing point of -2.6°C was encountered between key trench Station 1+00 and Station 1+20. The trench was over-excavated to remove as much hyper-saline material as practical. Further detail of key trench excavation, testing, and conditioning of the surface for material placement is provided in SRK (2012).

The central frozen ice-saturated core was constructed of a 2:3 blend of 20 mm minus material to fines (SRK 2012). This blend was tested on-site to obtain the moisture retention required for placement during construction. The blend material was moisture conditioned in the frozen core mixing plant using freshwater sourced from Doris Lake, with routine testing to ensure no elements in the water would affect the frozen material. A Geosynthetic Clay Liner (GCL) was installed over the upstream side of the frozen core to function as a secondary water retention system.

The transition zone was constructed of 150 mm (6 inch) minus crushed material and placed over the top of the frozen core and GCL (SRK 2012). The transition material was observed to be clean with little fines and no sand and gravel. The external dam shell (Shell) was constructed of run-of-quarry (ROQ) material. Finer crushed rock transition material was placed over a portion of the dam crest to serve as an access road.

2.1.4 Ground Temperature Cables

A total of 24 ground temperature cables (GTCs) (*aka* thermistor strings) have been installed within the North Dam to monitor temperature every six hours. GTCs were installed to ensure the dam core and foundation remain within the design operating temperature (Section 2.2).

GTCs were installed during construction and include; horizontal thermistor strings (HTS) installed in the upper (Upper Core), middle (Middle Core), and lower (Lower Core) region of the frozen core (Figure 4). The horizontal Lower Core GTCs also measure ground temperature near the buried evaporator pipes. For practical purposes, thermistor nodes (sensors or beads) near the buried pipes are used to measure the evaporator pipe temperature. Vertical temperature strings (VTS) were installed in the foundation below the key-trench (KT), and upstream (US) and downstream (DS) of the dam toe (Figure 4).

2.1.5 Thermosyphons

Thermosyphon evaporator pipes located at the base of the key trench provide passive cooling during the winter to ensure that the core and foundation remain frozen throughout the year.

Thermosyphons are pressurized sealed pipes, charged with a two-phase working gas that vaporizes and condenses to move heat without the need of a mechanical pump. A typical passive thermosyphon consists of an evaporator pipe buried in the ground and radiator exposed at the surface. The radiator section is manufactured with fins attached to the radiator pipe to enhance heat transfer with the atmosphere.

Heat is extracted from the ground when the air temperature at the radiator is colder than the ground temperature adjacent to the evaporator pipe. The temperature differential allows for the pressurized gas to condense within the radiator section of the pipe and lowers the pressure-boiling point within the evaporator which causes the lower gas to vaporize. The condensed fluid flows under gravity to the bottom of the evaporator pipe, and the process is repeated until the air temperature becomes warmer than the ground temperature.

Thermosyphon heat transfer is a function of composition and physical properties of the working gas, radiator and evaporator design, temperature difference between the upper and lower sections of pipe, ground thermal properties, and exposure of the radiators to advective cooling from the wind.

North Dam thermosyphons were procured and installed by Arctic Foundations of Canada Inc. Thermosyphon installation included one series of six evaporator pipes installed from the north end of the key trench and another six installed from the south end. The evaporator pipes extend to Section 0+85 which is the lowest point of the key trench. The north and south evaporator pipes are sloped at 4.6° and 8°, respectively.

Two thermosyphon radiators were attached to each evaporator pipe, with a total radiator surface area of 39 m^2. The North and South radiators are exposed at the surface and unobstructed by surface infrastructure to allow for effective heat loss from the radiator (Figure 1). Each pipe is charged with a carbon dioxide working gas and considered to have similar performance. General function of the thermosyphons is assessed in the winter by comparing the temperature differential between the air and the evaporator pipe directly below the ground surface.

2.2 Thermal Design Criteria

The thermal criteria for the extended design life is based on the original criteria proposed by EBA (2006), and requires:

- The top of the frozen core remain higher than the maximum operating level of the water within the TIA;
- The frozen core maintain a temperature at or below -2°C with a width that is at least twice the head of water impounded against the dam. For the Phase 2 tailings deposition a maximum head is approximately 7.9 m, resulting in a required frozen core width of 15.8 m; and
- The frozen foundation maintain a temperature at or below -8°C for a width equal to the required width of the frozen core (15.8 m) and extend to the base of the overburden soil (i.e. top of bedrock).

The critical section for applying the thermal criteria is shown as a yellow bounding box in Figure 3. The section is to ensure a sufficiently wide area of the core and foundation remain nearly ice-saturated impervious barrier to seepage over the design life.

The thermal criteria for the critical section of foundation was selected to reduce the unfrozen water content of the saline marine clayey silt and thus decrease creep of the dam. The low porewater salinity of sand beneath the dam alignment suggests a lower potential for creep at the equivalent temperature to that of the marine clayey silt. The thermal modeling is therefore based on a marine clayey silt foundation which presents the greatest sensitivity to deformation.

2.3 Current Conditions

Annual inspections and review of monitoring data suggest the dam is performing in accordance with the design expectations. The North Dam has impounded water since the first winter of construction in 2011. The operating water level impounded against the upstream face of the dam has averaged 29 m, with a maximum level of 29.5 m over the period from September 2011 to September 2015. The original water level of Tail Lake prior to construction of the North Dam was 28.3 m.

Annual review of ground temperature monitoring data collected at the North Dam indicates (SRK 2016):

- All horizontal GTCs were measuring temperatures well below the core design temperature of -2°C.
- Foundation temperatures were less than the foundation design temperature of -8°C, as measured by all nodes of GTC ND−VTS−085−KT.
- Foundation temperatures for the top six meters below the key trench at Section 0+175, as measured by GTC ND-VTS-175-KT, were warmer than the foundation design temperature of -8°C. However, a cooling trend is observed, with the maximum measured temperature decreasing approximately 1°C between 2014 and 2015.
- The maximum measured temperatures of near surface thermistor nodes for vertical GTCs on the upstream and downstream sides of the North Dam were between -1°C and -2°C.
- All thermosyphons are currently working with the exception of North 2 located along the north panel of thermosyphons. The evaporator pipe for North 2 is the second pipe from the downstream side of the core.

Figure 5 shows the thermosyphon evaporator temperature of one pipe measured at Section 0+85. The temperature record shows the heat extraction (cooling) period extends from mid-to-late October to late March. The thermosyphon ceases to extract heat from the spring when air temperatures are equal or greater than the ground temperature. During this period of time, the ground gradually warms near the evaporator pipe until the air temperature becomes colder than the ground in the fall.

Figures 6 and 7 show the thermosyphon evaporator temperature for Section 0+85 and 0+40, respectively. Evaporator temperature at Section 0+40 is observed to have a greater amplitude (minimum and maximum value). Section 0+40 is characterized by a thinner section of the dam with less thermal mass, which results in greater magnitude of warming and cooling over one year. However, similar average annual evaporator pipe temperature is measured at these two sections of the dam.

Figure 8 shows the foundation temperature directly beneath the key trench at Section 0+85. Ground temperatures are observed to be consistently less than -8°C with a decreasing trend in temperature. Figures 9 and 10 show the foundation temperature beneath the upstream and downstream toe of the dam, respectively. Warmer foundation temperatures are expected beneath the toe due to decreased fill thickness and impoundment of water against the upstream face of the dam.

3 Methods

3.1 Model Setup

Modeling was completed in a two-dimensional domain by solving for conductive heat movement using SoilVision's SVHeat (SoilVision 2011) software package in combination with FlexPDE (FlexPDE 2014). SVHeat was utilized for the problem setup, while FlexPDE 6.35 solver was used to complete the calculation.

As-built survey information was used for the 2D model sections of the dam. Section 0+85 was modeled with a 14 m wide crest, 11 m height, upstream slope of 6H:1V, and downstream slope of 4H:1V. Section 0+40 was modeled with a 13 m wide crest, 4 m height, upstream slope of 4H:1V, and downstream slope of 2H:1V. The model geometry for Section 0+85 and Section 0+40 is presented in Figure 11 and Figure 12, respectively.

Thermosyphon evaporator pipes and thermistor node locations were included in the model using available as-built survey information. Thermosyphon evaporator pipes act as a location for heat extraction in the model and thermistor nodes represent locations for comparing measured and modeled temperature.

3.2 Model Inputs

3.2.1 Material Properties

Five material regions were considered in the model: shell, transition zone, core, natural clay foundation, and bedrock. Table 1 presents a summary of the materials and thermal properties for each material region. The GCL was not physically represented in the model, which is a reasonable omission considering the nominal thickness of the liner (approximately 15 mm).

The thermal properties for ROQ material were taken from previous work completed by SRK for granular pad design (SRK 2017e). The thermal conductivity and heat capacity of the ROQ and transition materials were also calculated for 100% saturation to simulate infiltration of water on the upstream side of the frozen core and liner (Table 1).

The thermal properties for foundation-soil was based on natural clayey silt located beneath a portion of the dam. The average physical properties of the clayey silt was based laboratory measurements from field samples at the site (SRK 2012). A porewater freezing point depression of -2°C was based on average site-wide conditions which is reported to be -2.1°C (SRK 2017d). There is a negligible difference between the position of these two isotherms, and -2°C is presented for clarity of the results. The material property includes an average unfrozen water content curve for natural clay (Andersland and Ladanyi 2004) which has been adjusted for the freezing point depression in accordance with Banin and Anderson (1974). The thermal conductivity and head capacity were calculated in accordance with Cote and Konrad (2005) and Newman (1995), respectively.

Average physical properties of the frozen core material and transition zone material (SRK 2012) were used to calculated representative thermal properties. The core is not expected to have an appreciable level of dissolved ions within the porewater and no allowance was made in the model for a freezing point depression.

Table 1: Material Thermal Properties

Region of Model	Material	Degree of Saturation (%)	Porosity	Thermal conductivity, kJ/(m·day·°C)		Volumetric Heat Capacity, kJ/(m³·°C)	
				Unfrozen	Frozen	Unfrozen	Frozen
Shell	ROQ	30	0.30	104	117	1,697	1,509
Saturated Shell	ROQ	100	0.30	142	211	2,776	2,147
Transition	150 mm (6 inch) minus	40	0.21	172	174	1,821	1,646
Saturated Transition	150 mm (6 inch) minus	100	0.21	208	274	2,347	1,911
Core	20 mm minus: 5 mm minus (2:3 blend by volume)	88	0.26	184	231	2,827	2,351
Clay Foundation[1,2]	Clayey Silt	85	0.52	112	187	2,842	2,038
Bedrock	Basalt	100	0.05	260	260	2,380	2,133

Notes:
1. Overburden clayey silt includes a porewater freezing point depression of -2°C
2. Unfrozen water content curve based on grain size

3.2.2 Climate Boundary Conditions

A ground surface temperature curve was developed for the Project site to represent the ground temperature immediately below surface. The boundary was defined by sinusoidal function of temperature and time based on Equation 1 and the parameters shown in Table 2.

$$T = \max(nf * \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi+(t+\alpha)}{365}\right)\right], \text{nt} \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi+(t+\alpha)}{365}\right)\right] \quad \text{Eq.1}$$

Where:

- T *is the ground temperature measured in °C*
- nf *is the surface freezing n-factor*
- nt *is the surface thawing n-factor*
- $MAAT$ *is the mean annual air temperature measured in °C*
- Amp *is the air temperature amplitude measured in °C*
- C_A *is the air climate change factor in °C d*$^{-1}$
- α *is phase lag of the sine wave*
- t *is time measured in days*

Mean annual air temperature and amplitude are based on average values for the baseline period of 1979-2005 (SRK 2017f). Seasonal n-factors were applied as multipliers of air temperature to estimate the temperature at the ground surface. The upstream and downstream face of the dam (dam face) was based on a freezing n-factor (*nf*) of 0.86 and thawing n-factor (*nt*) of 1.52. These values are based on average published values for crushed rock and gravel (SRK 2017e), and considered to be reasonable base case conditions for the Project site. The n-factors applied to the crest of the dam were calibrated to match measured temperature for the upper core. The calibration reduced the thaw n-factor and increased the freezing n-factor which results in cooler ground temperatures when compared to the average published values applied to the side slopes. The n-factors for natural overburden was applied downstream of the dam using values calibrated to ground temperatures measured at the Project site (SRK 2017g).

Table 2: Current Climate Boundary Parameters

Parameter	Value
Mean Annual Air Temperature *(MAAT)*	-10.7°C
Air Temperature Amplitude *(Amp)*	21°C
Dam Crest, Thawing n-factor *(nt)*	1.30
Dam Crest, Freezing n-factor *(nf)*	0.90
Dam Face[1], Thawing n-factor *(nt)*	1.52
Dam Face[1], Freezing n-factor *(nf)*	0.86
Natural Overburden Downstream, Thawing n-factor *(nt)*	0.55
Natural Overburden, Freezing n-factor *(nf)*	0.65
Water Temperature Boundary	Figure 8

Notes:

1. Dam face refers to both the downstream side and upstream side of the dam above the water level

Climate change is considered in Equation 1 using the air climate change factor. This factor allows for a daily increase in air temperature within the model which is based on the work of SRK (2017f). Table 3 shows the daily increase in air temperature in the model applied to the thermal models.

Table 3: Summary of Doris Air Climate Change Factors Applied to Thermal Models

Year	Rate (°C decade^{-1})	Air Climate Change Factor (°C day^{-1})
2011 – 2040	0.74	0.000203
2041 – 2051	0.71	0.000195

3.2.3 Water Boundary Condition

The thermal effect of water against the upstream face of the dam was simulated using a time dependent temperature boundary (Figure 13). The temperature is similar to average monthly lake bottom temperature for Arctic waterbodies (Burn 2002; Ensom *et al*. 2012) and are conservative when compared to the average lake water temperatures measured at lakes located within the Project site (SRK 2017h). The potential change in water temperature due to climate change was not consider due to conservatism considered in the boundary.

3.2.4 Thermosyphon Boundary Condition

Thermosyphons were included as a heat flux boundary in the model:

$$Q = Hq\ (Tg - Ta) for\ Ta < Tg \quad \text{Eq.2}$$

$$Q = 0\ for\ Ta > Tg \quad \text{Eq.3}$$

$$h = (A + B \cdot V^C) A_{rad} \quad \text{Eq.4}$$

$$Hq = (h \cdot S) \,/\, A_{Evap} \quad \text{Eq.6}$$

$$Ta = \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi + (t+\alpha)}{365}\right)\right] \quad \text{Eq.7}$$

Where:

Q *is the total heat flux (J s^{-1})*
h *is the thermosyphon performance (J s^{-1} °C^{-1})*
Tg *is the evaporator temperature in the ground (°C)*
Ta *is the ambient air temperature (°C)*
V is the wind speed (m s^{-1})
$A, B, and\ C$ are heat transfer coefficients (Table 4)
A_{rad} is the surface area of the radiator
A_{Evp} is area of the evaporator pipe in the model
S is the number of second per day
Hq is the daily heat flux per metre of pipe

Heat transfer coefficients for a sloped evaporator were based on experimental results provided by Zarling and Haynes (1985) (Table 4). The heat flux boundary condition in the model extracts heat for the period of time when the air temperature is less than the ground temperature at the location of the evaporator pipe in the model (Eq. 2). The heat flux is prescribed to be zero for periods of time when the air temperature is greater than the ground temperature (Eq. 3) which agrees with the function of a thermosyphon (Figure 5).

Table 4: Passive Thermosyphon Heat Transfer Coefficients

Parameter	Value	Units
A	2.72	W m^{-2} C^{-1}
B	7.04	W m^{-2} C^{-1}
C	0.273	-
V	4.81	m s^{-1}

Notes:

1. Average wind speed (V) for climate change baseline period (SRK 2017f)

Climate change was applied to the air temperature boundary used to determine thermosyphon heat extraction. The average wind speed based on baseline climate conditions was used in the model, which is predicted to increase by 0.2 m s-1 over the design life (SRK 2017f). The predicted increase in wind speed would result in an increase in thermosyphon heat extraction, and for conservatism the constant baseline wind speed was used (Table 4).

3.2.5 Initial Conditions

The initial conditions were defined by each material region in the model. The frozen core temperature was set to -11°C which is the average temperature measured within the middle of the core (GTC ND-HTS-085-294). The transition material and shell were set to a constant temperature of -8°C and -7°C, respectively.

An initial temperature of -7.6°C was applied to overburden and bedrock which is representative of average permafrost temperatures at the Project site (SRK 2017e). The applied temperature is consistent with the average annual ground temperature measured in 2012 from the deepest thermistor node installed within the foundation at ND-VTS-085-KT (-7.6°C), ND-VTS-085-US (-7.6°C), and ND-VTS-085-DS (-6.2°C).

The vertical sides of the model space were set to a zero flux boundary and the lower boundary set to a constant flux 5.46 kJ/(m^2·day·°C) which was calculated from the average geothermal gradient of 0.021°C m^{-1} and the thermal conductivity of the bedrock (SRK 2017h).

3.3 Model Scenarios

3.3.1 Calibration Model

North Dam ground temperature measurements between August 12, 2012 and April 11, 2016 was used for model calibration of the two Sections 0+40 and 0+85. Table 5 summarizes the calibration model objectives and applied boundary conditions.

Table 5: Summary of Calibration Model Objectives and Boundary Conditions

Model	Objectives	Boundary Conditions
1	Confirm: • Thermosyphon heat transfer coefficients used to calculate heat extraction (Eq. 4)	• Measured Doris Meteorological Station daily air temperature and wind speed • Thermosyphon applied using SVHeat's thermosyphon option built into modeling package with calculated heat transfer coefficients
2	Confirm: • Climate boundary (Eq. 1). • Thermosyphon flux boundary (Eq. 2)	• Generalize climate boundary with average wind speed • Thermosyphon flux boundary

Notes:

1. Calibration models were run from Aug 12 of 2012 to April 11 of 2016

Model 1 was run to confirm reasonable heat extraction from the thermosyphons based on the calculated heat transfer coefficients and thermal properties. Doris Meteorological Station air temperature and wind speed measured over the calibration period was applied to the model (Figures 14 and 15).

Model 2 was run to confirm that reasonable results were obtained using the generalized climate boundaries for air temperature and ground surface temperature, and the flux boundary used to simulate heat extraction from the thermosyphon evaporator pipes.

For the calibration models, the water temperature boundary was applied to upstream face of the dam and based on the average water level (29 m) measured during the calibration period. The calibration models used six working thermosyphons for direct comparison with the current conditions at the south panel of thermosyphons.

3.3.2 Thermal Performance Model

A long-term thermal performance of the dam and foundation were predicted over the design life using:

- Climate boundary with consideration for climate change increase in air temperature (Eq. 1 and Table 2 and 3);
- Thermosyphon flux boundary (Eq. 2) with consideration of climate change (Table 4) and five working thermosyphons to meet conservative conditions along the north panel of thermosyphons;

- Water temperature boundary conditions applied along the upstream dam face to the top of the frozen core over the entire design life (Figures 13 and 14). This represents a conservative input to the model as the full supply level will be 34 m (SRK 2017a);
- Clayey silt foundation to represent thermal conditions for which the dam is physically sensitive;
- Average geothermal heat flux and calculated thermal properties; and
- Design life of 40 years from 2011 to 2051, which extends beyond the expected period of mining and subsequent closure, for at least ten years.

4 Results

4.1 Calibration Period

Model results for the calibration period are shown for Section 0+85 (Figures 16 through 25) and Section 0+40 (Figures 26 through 32).

Calibration Model 1 uses measured daily air temperature and wind speed to confirm the thermal behavior and heat extraction of the thermosyphon is achieved in the model. For Section 0+85 (Figures 16 through 21) and Section 0+40 (Figures 26 through 31), the modeled evaporator pipe temperature closely match the measured values. In particular, the modeled temperature agrees with the timing of heat extraction from the thermosyphon evaporator which is observed as a rapid decrease in temperature, and the timing of warm and maximum temperature reached when the thermosyphons is seasonally shutdown. Short term fluctuation in temperature is also captured when using daily measured air temperature and wind speed. In general, the thermosyphon heat extraction and resulting change in temperature does not exceed the measured minimum temperature or maximum temperature.

Calibration Model 2 uses the modeled climate boundary and average wind speed to confirm the generalized boundary conditions and the thermosyphon heat flux boundary produce comparable results to Model 1 and measured temperatures. A similar change in evaporator temperature results from Model 2 when compared to the more specific inputs used in Model 1; i.e. the input of measured daily air temperature and wind speed. Figures 16 through 21 show the Model 1 and Model 2 comparison for Section 0+85. Equivalent figures for Section 0+40 are shown in Figures 26 through 31. Results from Model 2 confirm the generalized climate boundary and thermosyphon flux boundary are suitable and conservative for predicting long-term thermal performance of the dam over the extended design life.

Measured and modeled temperature for the upper core (Figure 22), middle core (Figure 23), and foundation (Figures 24 and 25) all show good agreement for Section 0+85. Figure 32 shows modeled and measured temperatures for the middle core of Section 0+40. Measured ground temperatures are not available for the upper core of Section 0+40. Modeled temperature below the key trench in the foundation also shows good agreement with measured temperature over the calibration period (Figures 24 and 25).

4.2 Thermal Performance Period

The North Dam thermal regime was modeled over the 40-year design life to evaluate its thermal performance based on the thermal design criteria described in Section 2.2. The critical section of the dam used to assess performance is shown as a yellow bounding box in Figure 3. The critical section defined by the box is 15.8 m wide and extends from the FSL (34 m level) to the top of bedrock.

Figures 33 through 37 show the modeled temperature for Section 0+85 at years 2, 10, 20, 30, and 40, respectively. For Section 0+85, the thermal criteria is met over the design life, with the core remaining below -2°C and the foundation below -8°C (Figure 37). The core and foundation temperatures are observed to increase in response to the surface climate boundary and the water temperature boundary applied to the upstream face of the dam. The warming of the foundation directly below the key trench is mainly caused by the length of time water is applied to the upstream face of the dam. Temperature at the base of the key trench and within the foundation is shown in Figure 38.

Figures 39 through 43 show the modeled temperature for Section 0+40 at years 2, 10, 20, 30, and 40, respectively. The core temperature over the critical section remains below -2°C for the design life and most, but not all of the foundation remains at or below -8°C. A portion of the critical section warms to -6°C by the end of the design life (28% of section) while the remaining 72% is colder than the -8°C. A smaller portion of the foundation (8%) is warm -8°C at the end of 30 years which is the actual expected design life of the dam.

Temperature at the base of the key trench and within the foundation is shown in Figure 44. The predicted warming for the foundation would result in a higher unfrozen water content of the frozen clay. However, little change in the unfrozen water content over the same range in temperature (-8°C to -6°C) would be expected for the foundation consisting of sand; i.e. coarser-grained soils, such as sand with fresh porewater exhibit a small change in the unfrozen water content for this temperature range.

5 Conclusions

Thermal modeling of the North Dam has been completed for a 40-year design life, which is 10 years greater than the actual planned design life of 30 years. The model was validated with measured temperature from the dam, and long-term performance modeling was completed with consideration for climate change and the use of conservative inputs. The model results indicate the frozen core will remain below -2°C. Towards the end of the 40-year design life, the foundation over a portion of the critical section is expected to exceed the -8°C beneath the thinnest sections of the dam. The warmer foundation conditions will result in a higher fraction of unfrozen water and a greater potential for creep deformation. The thermal criteria is met over the 40-year design life for the thickest section of the dam which impounds the greatest head of water.

At the end of the planned design life of 30 years, the thermal criteria of the foundation is met beneath the thickest section of the dam. A small portion of the foundation does not meet the thermal criteria beneath the thinnest sections of the dam (8% of the foundation), with 92% of the area below -8°C. Over this period of time the frozen core temperature is below -2°C.

These results are based on conservative inputs to the model with consideration for increasing air temperature due to climate change, a constant wind speed which is expected to increase and improve thermosyphon performance, a constant head of water applied to the full supply level for the entire design life, a clay foundation, and five working evaporator pipes.

Annual inspections and monitoring to date indicate the North Dam is performing in accordance with the design expectations. Continued monitoring of the North Dam will allow for assessment of its thermal performance over time and the need for any mitigation, such as maintaining the downstream dam face clear of snow to enhance cooling, construction of a coarse rock convective cover over the downstream dam face, placement of added thermal protection at the upstream dam surface, or installation of vertical thermosyphons to reduce foundation temperatures. The existing thermosyphons may also be retrofitted with chilling coils on the surface radiators to increase heat extraction. Thermal responses in the dam will be very slow and trends will be easily identified through monitoring, allowing for ample time to implement any of the mitigation strategies.

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The opinions expressed in this document have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. While SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

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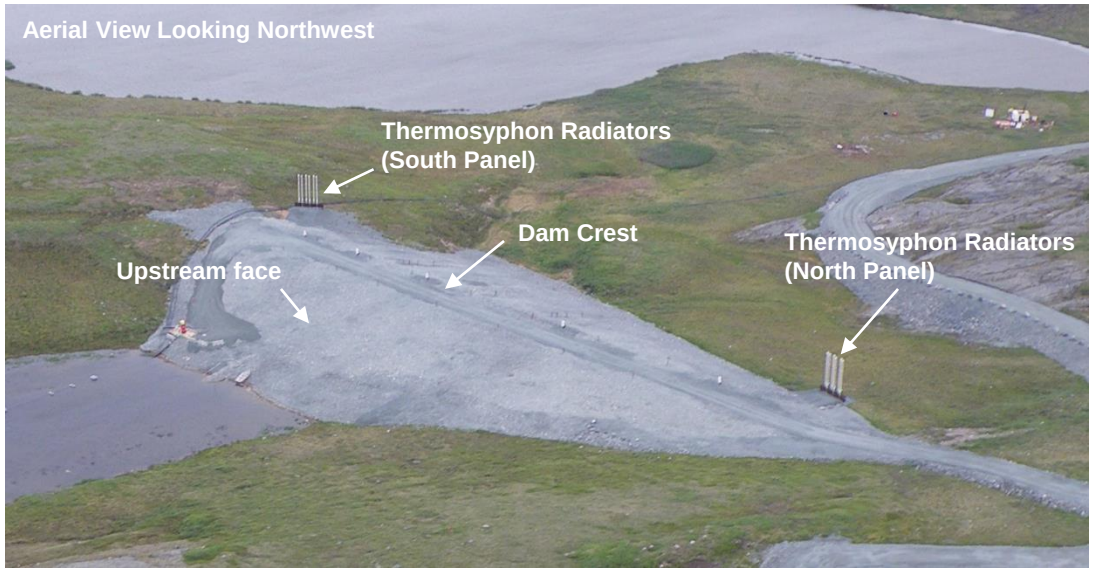
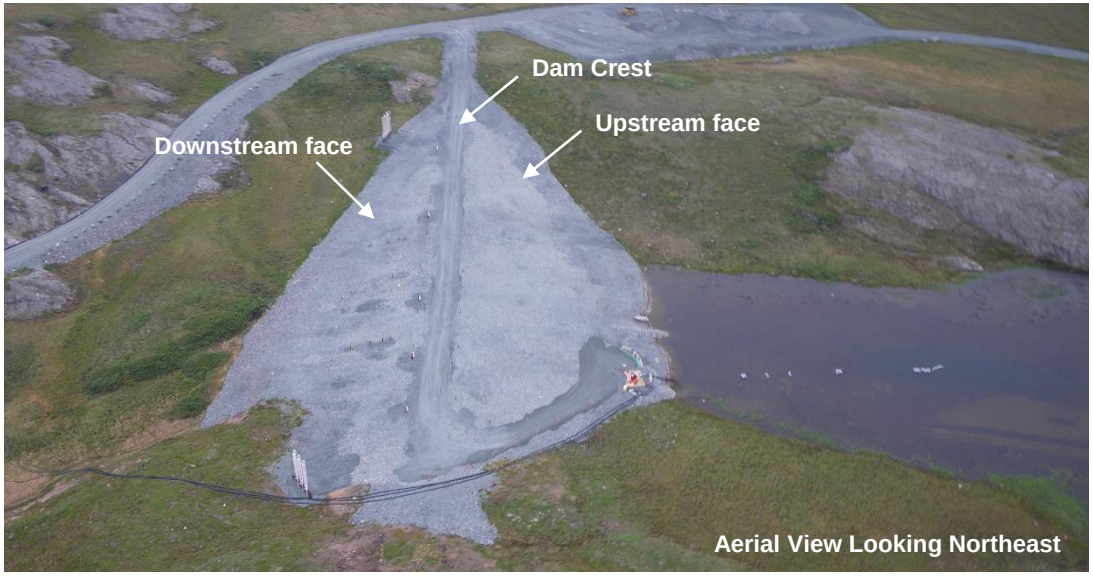
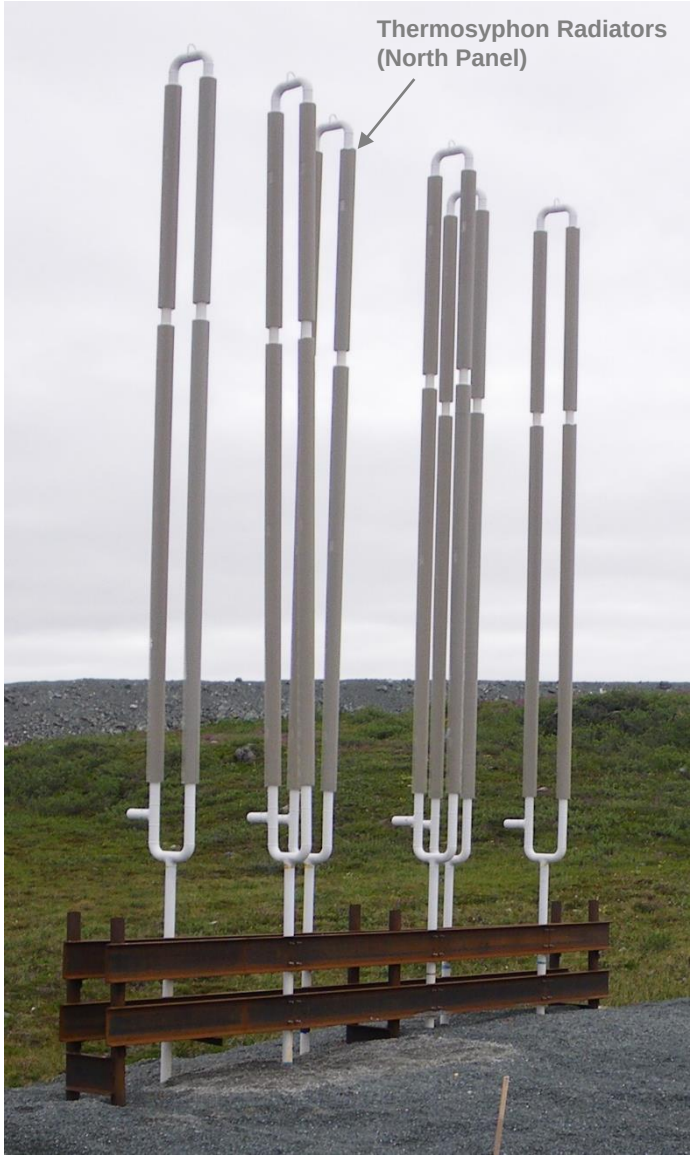
Figures

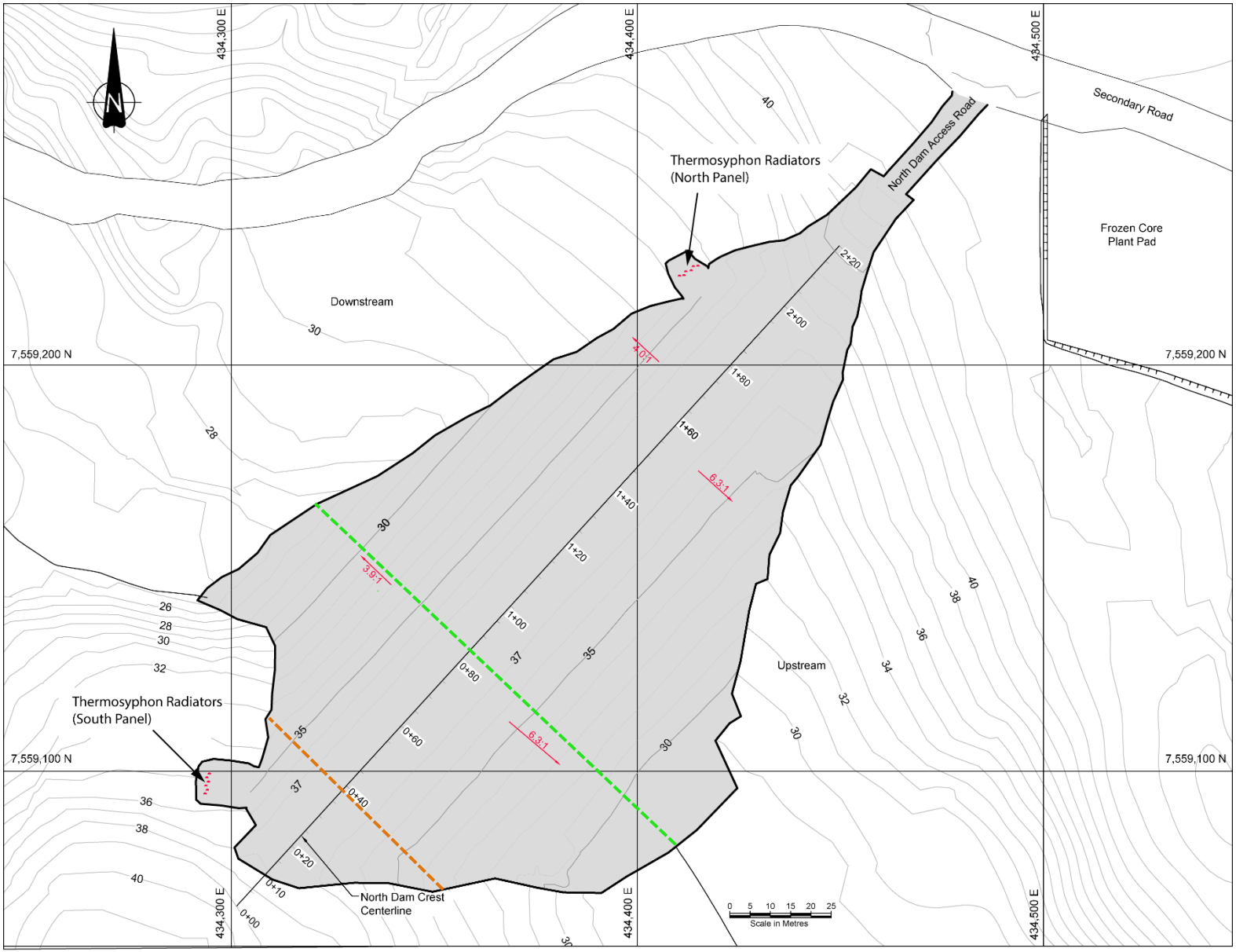






Notes:
1. Field photographs taken July 18th and July19th of 2014

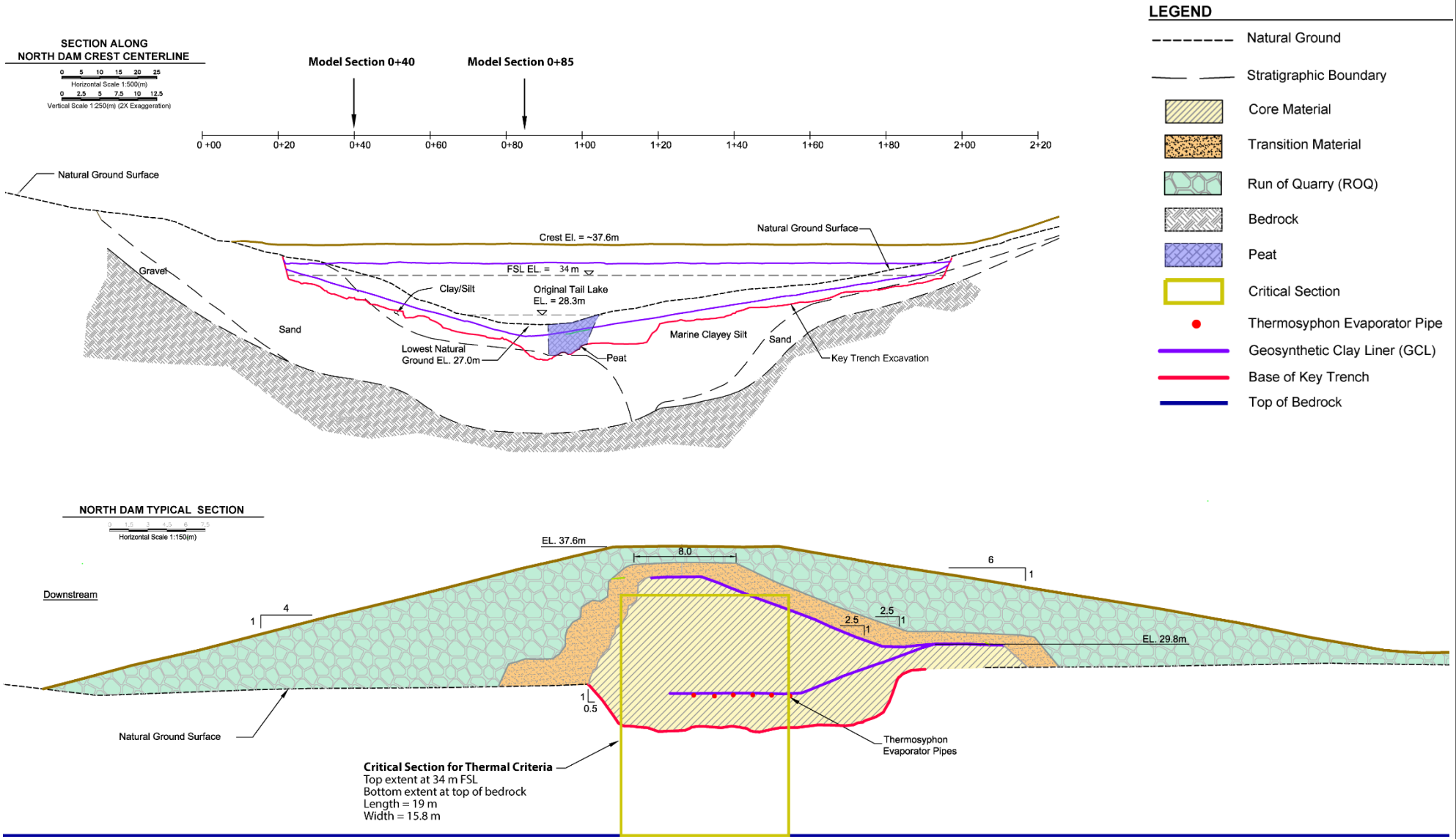
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Field Photograph
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **1**



Notes:

1. Dashed orange line indicates Dam Section 0+40
2. Dashed green line indicates Dam Section 0+85

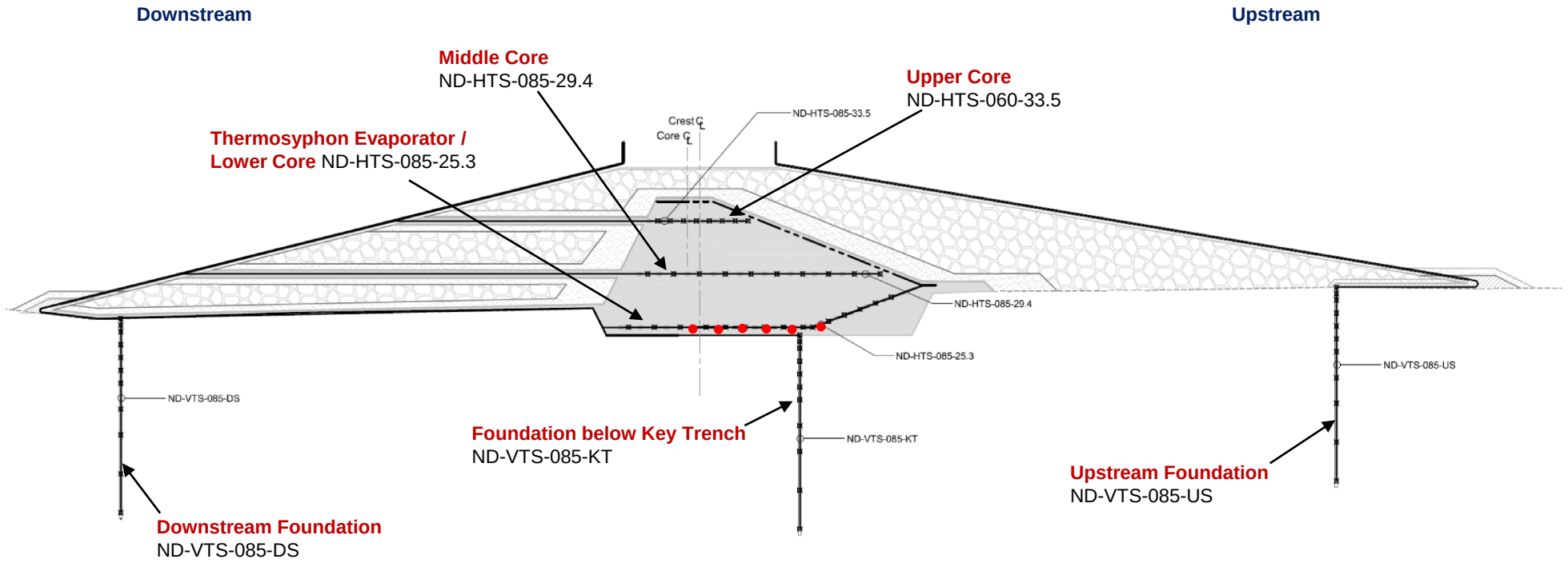
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam General Arrangement
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **2**



Notes:

1. The subsurface geology has been extrapolated from a series of geotechnical investigations and geological unit contacts are therefore likely to vary somewhat.

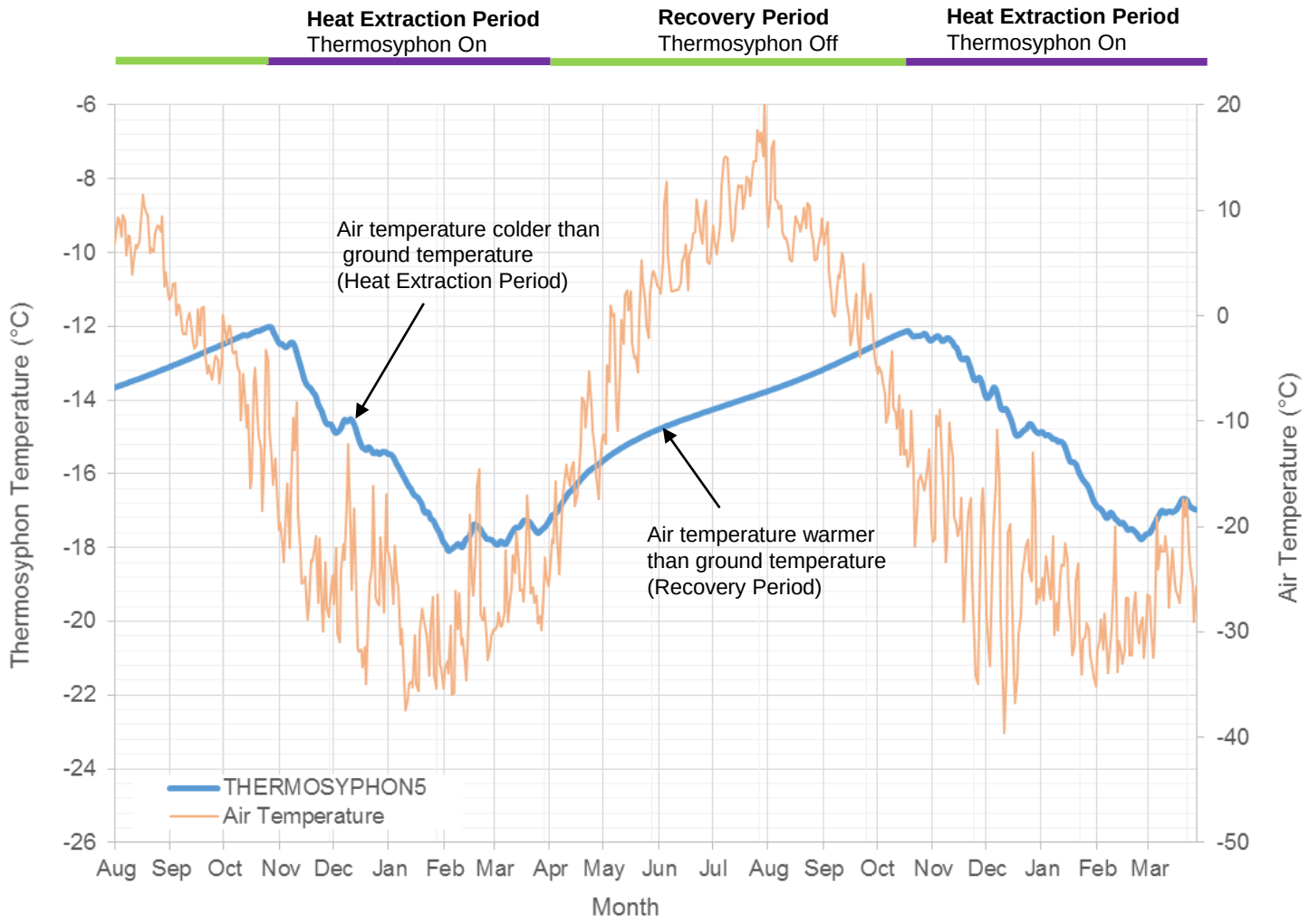
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**North Dam** **Typical As-Built Section**
		Date: 8/15/2017 | Approved: cws | Figure: **3**



Notes:

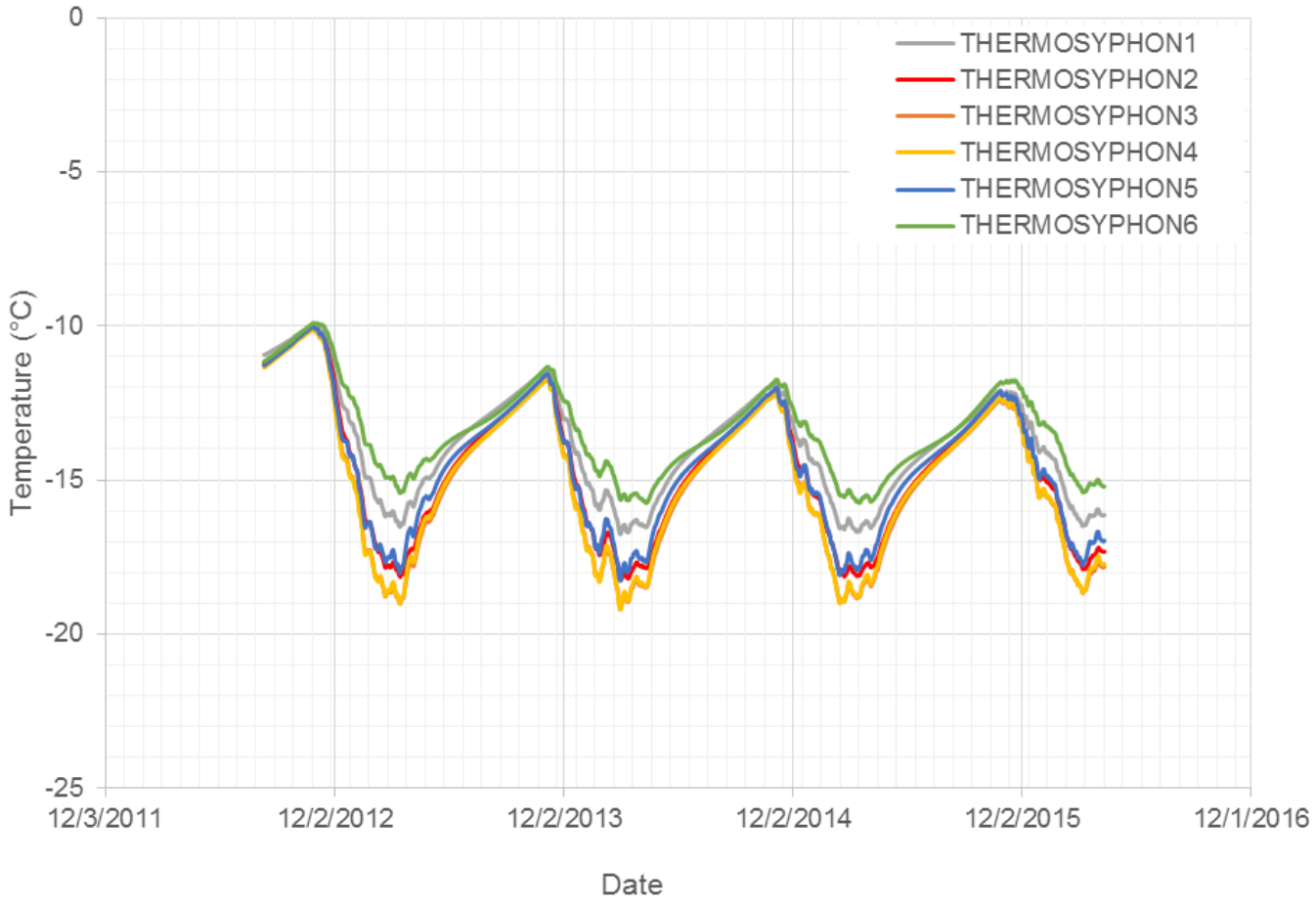
1. Red dots indicate evaporator pipes
2. Black dots indicate thermistor nodes
3. Horizontal thermistor strings (HTS)
4. Vertical thermistor strings (VTS)
5. Key trench (KT)
6. Upstream (US) and downstream (DS)

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**North Dam** **Temperature Cable Arrangement**
		Date: 8/15/2017 | Approved: cws | Figure: **4**



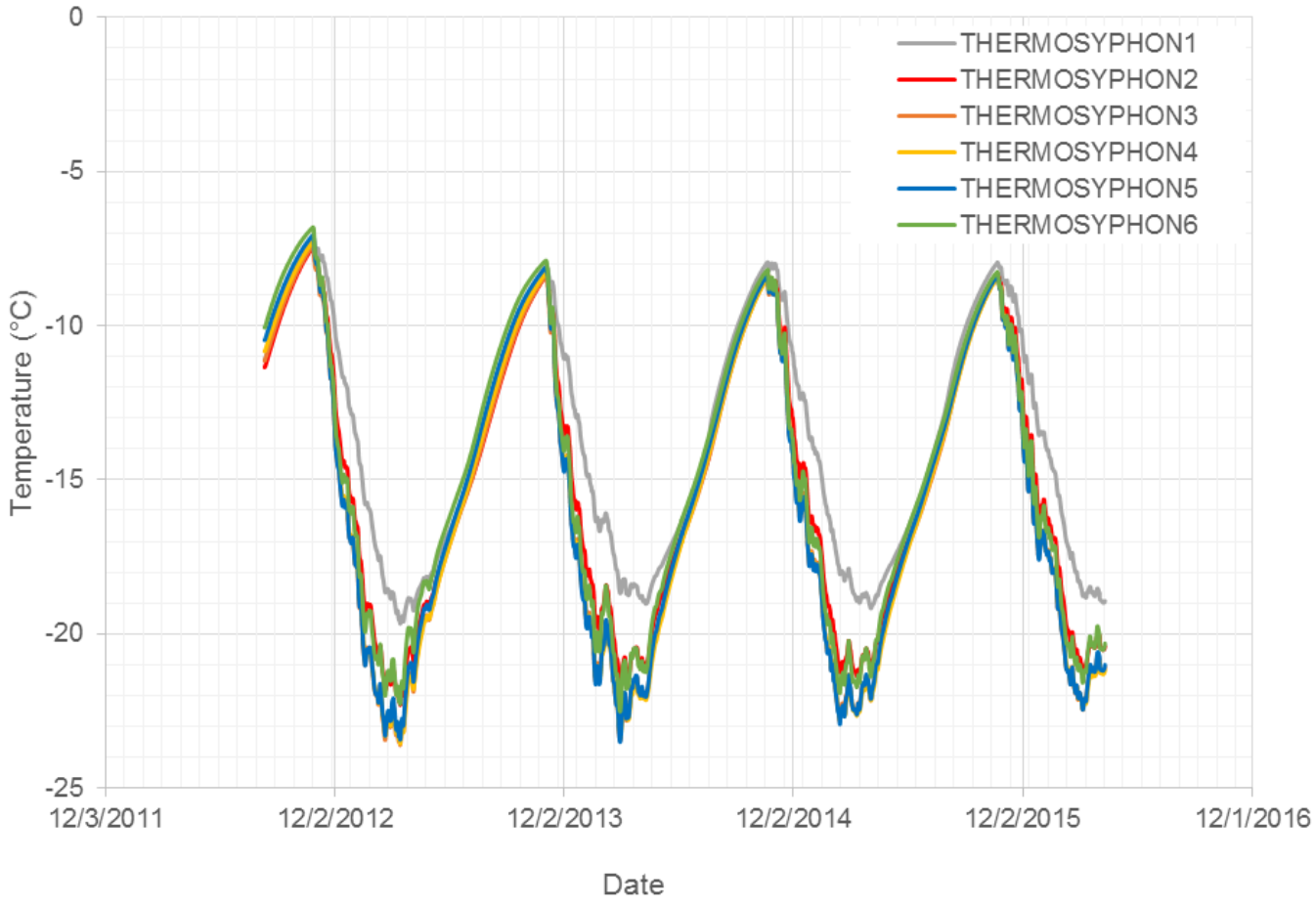
Notes:
1. Blue line represents thermosyphon evaporator temperature
2. Orange line represents Doris Metrological Station air temperature

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Thermosyphon Evaporator – Thermal Behavior
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **5**



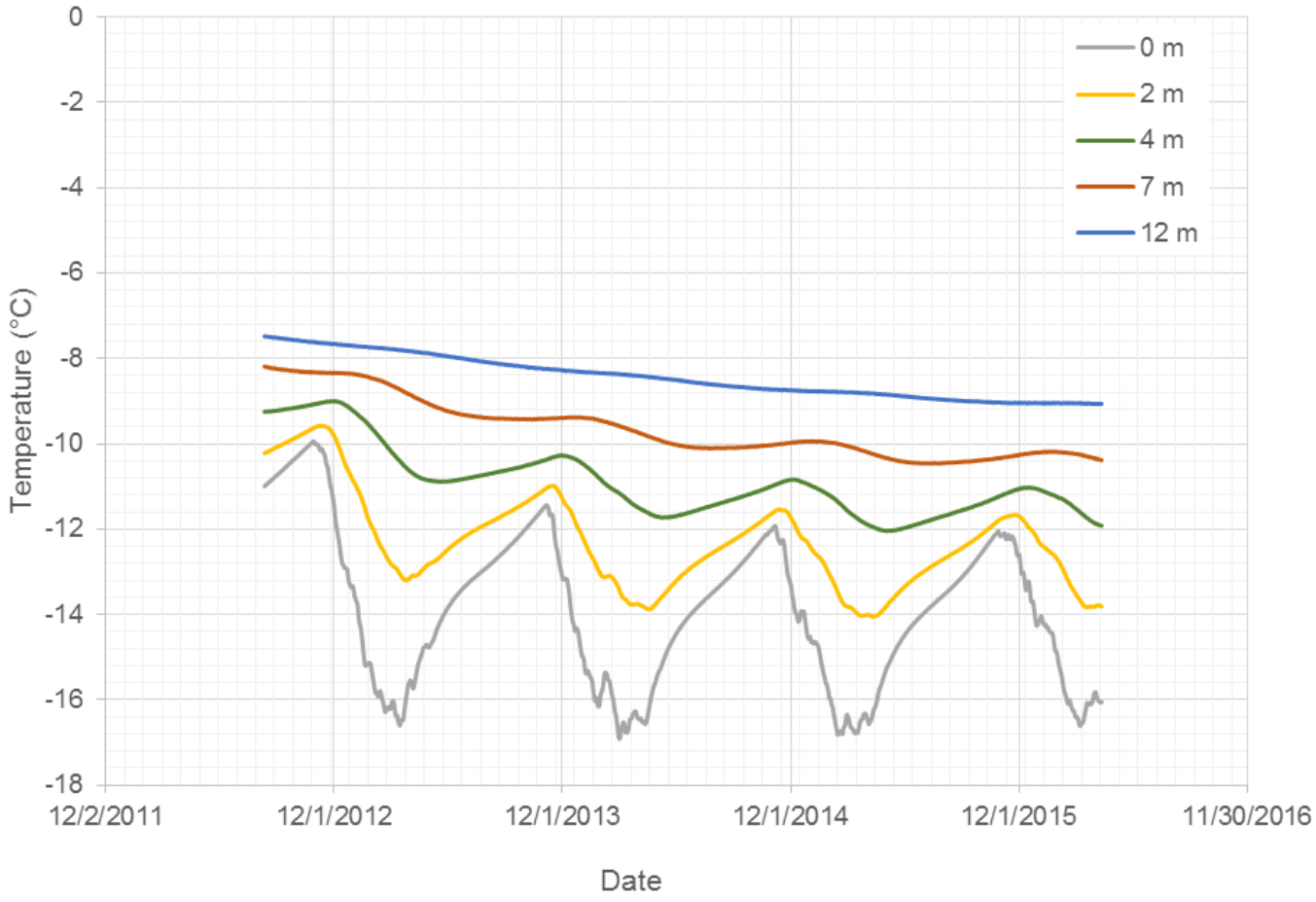
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3 (see Figure 4)
2. Thermistor nodes located near evaporator pipe
3. Data from Section 0+85

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Measured Temperature – Thermosyphon Evaporator (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **6**



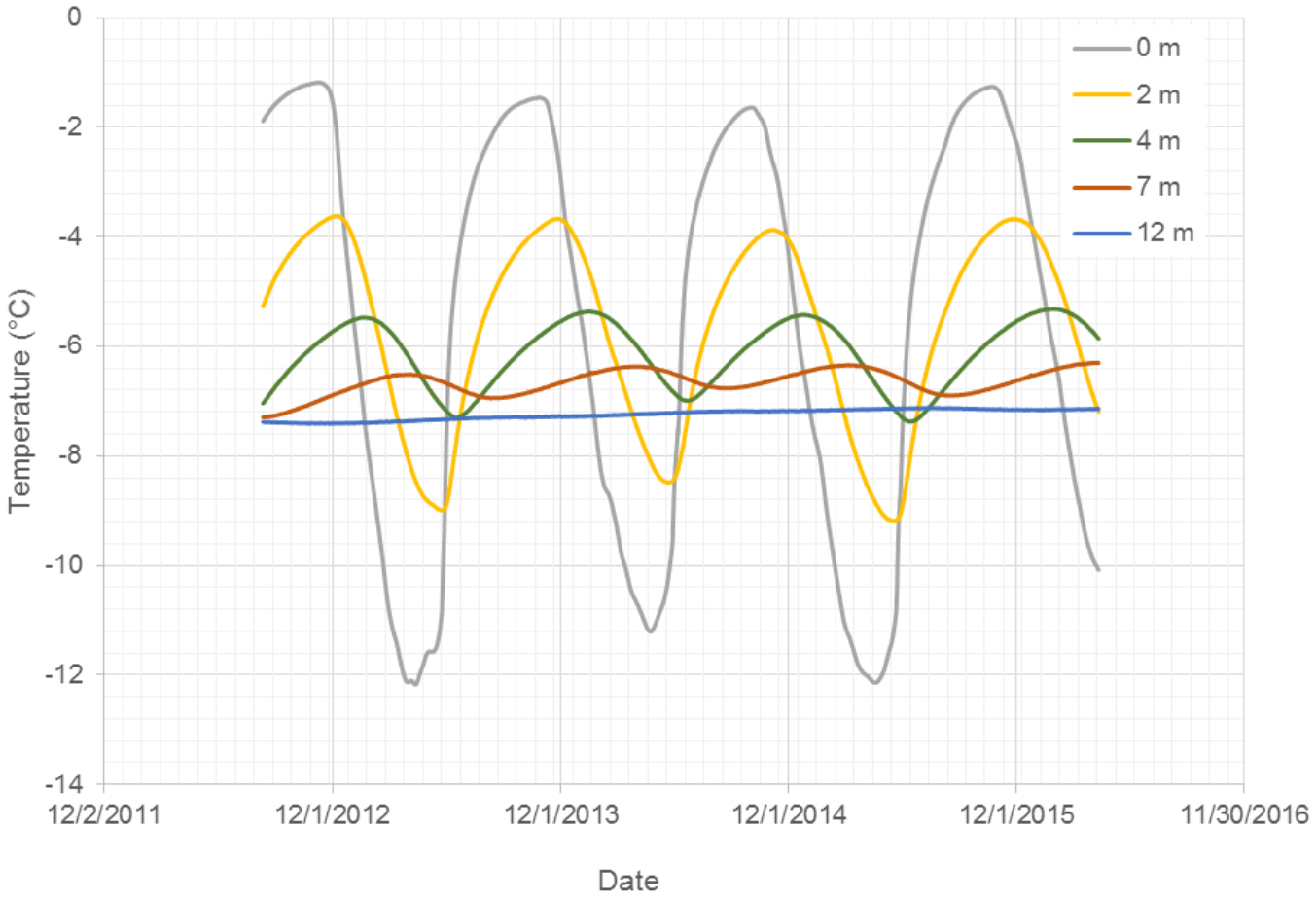
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5 (see Figure 4)
2. Thermistor nodes located near evaporator pipe
3. Data from Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Measured Temperature – Thermosyphon Evaporator (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **7**



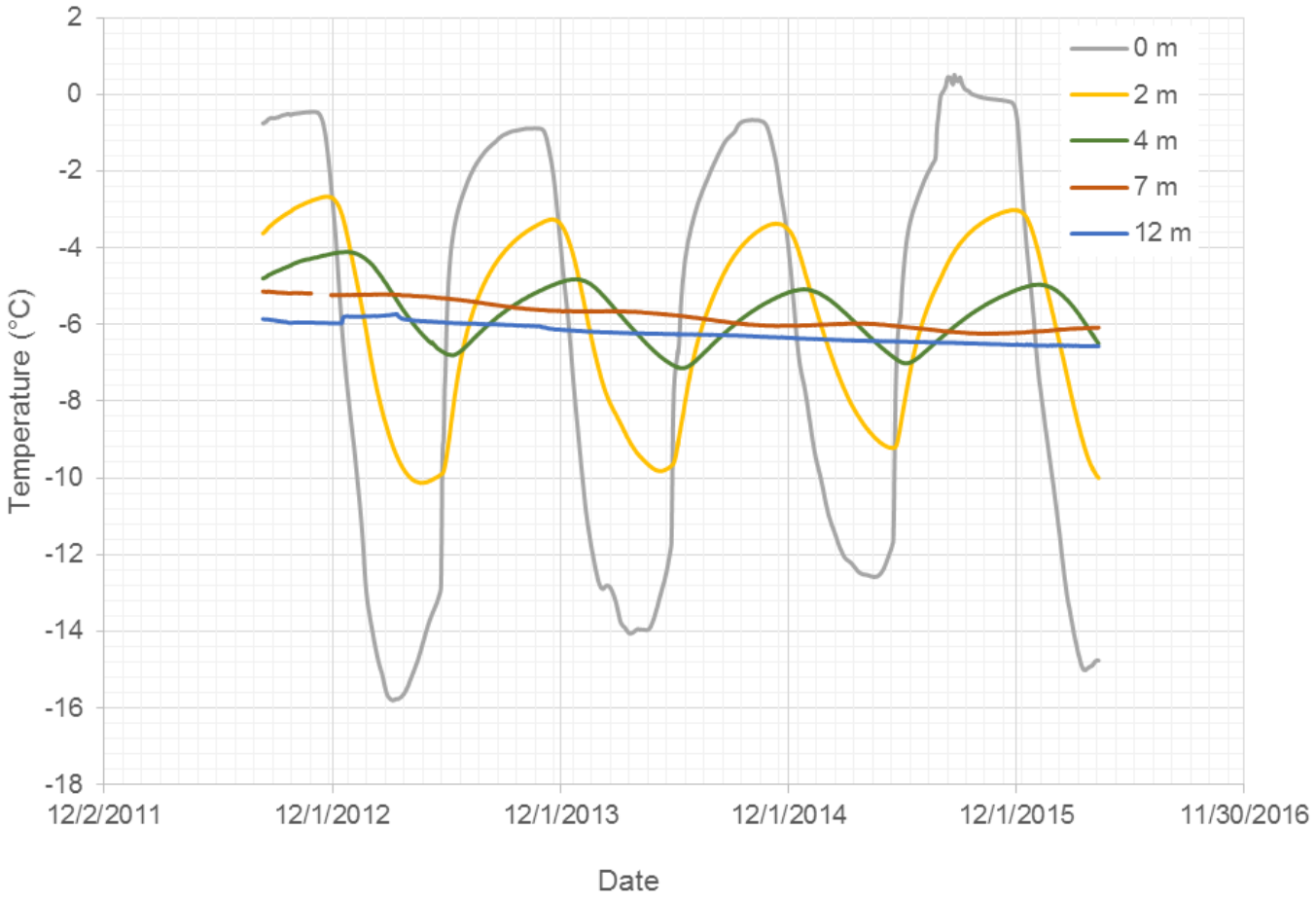
Notes:
1. Ground temperature measurements from thermistor string ND-VTS-085-KT (see Figure 4)
2. Data shown for select thermistor nodes for graphical clarity
3. Depth referenced from top thermistor node

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Measured Temperature – Below Key Trench (0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **8**



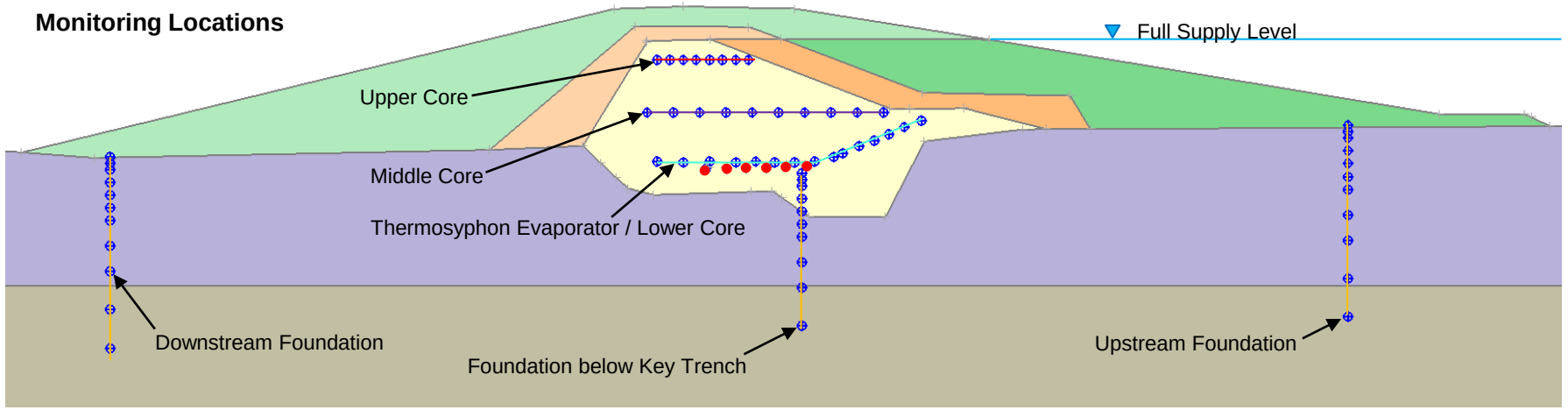
Notes:
1. Ground temperature measurements from thermistor string ND-VTS-085-US (see Figure 4)
2. Data shown for select thermistor nodes for graphical clarity
3. Depth referenced from top thermistor node

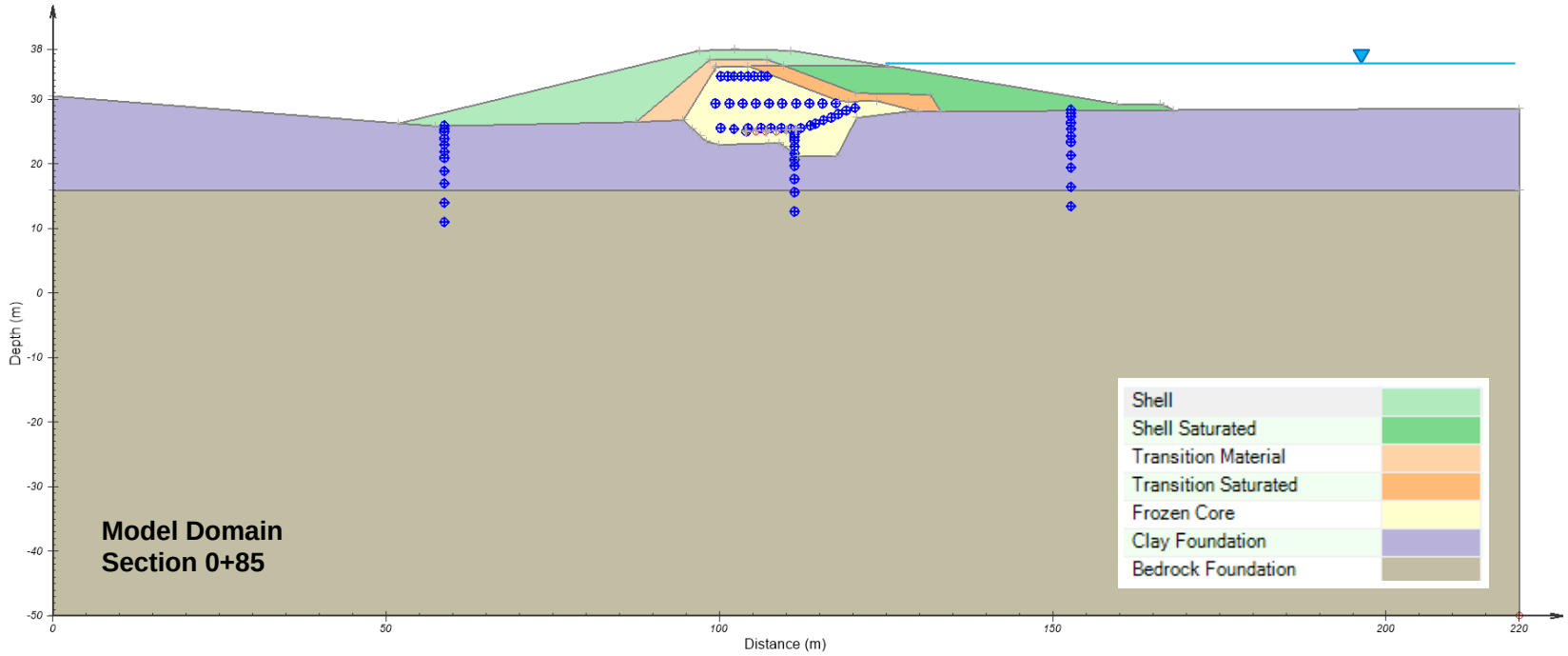
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Measured Temperature – Upstream Foundation (0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **9**



Notes:
1. Ground temperature measurements from thermistor string ND-VTS-085-DS (see Figure 4)
2. Data shown for select thermistor nodes for graphical clarity
3. Depth referenced from top thermistor node

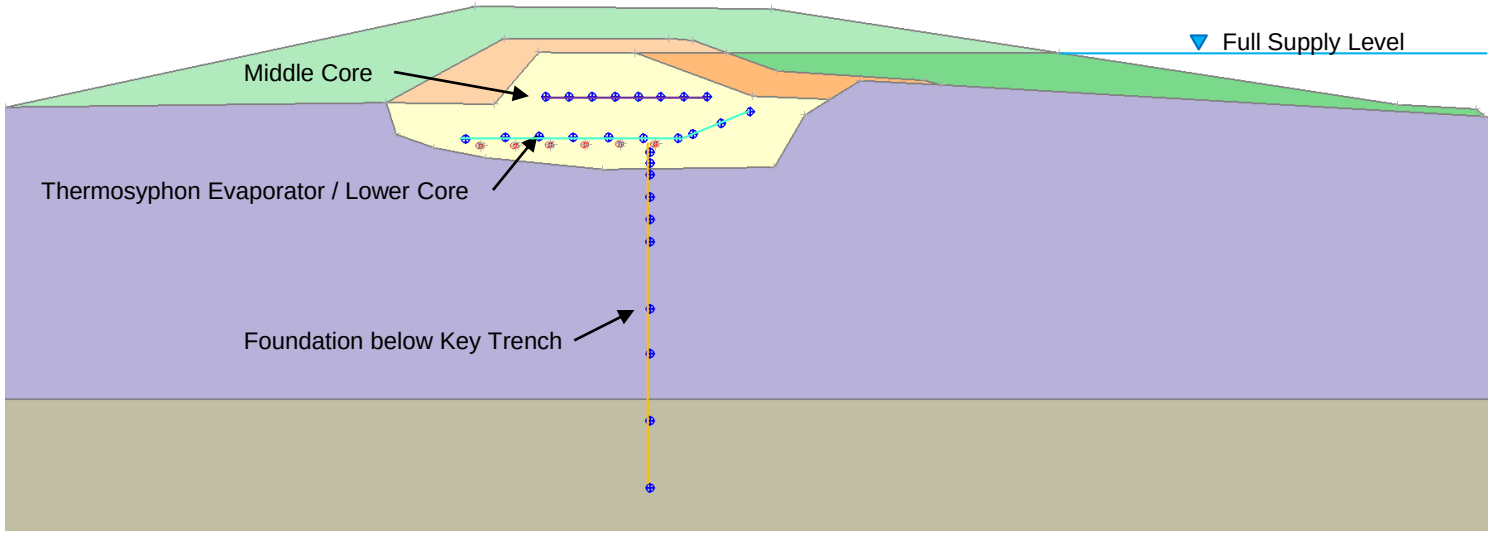
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Measured Temperature – Downstream Foundation (0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **10**

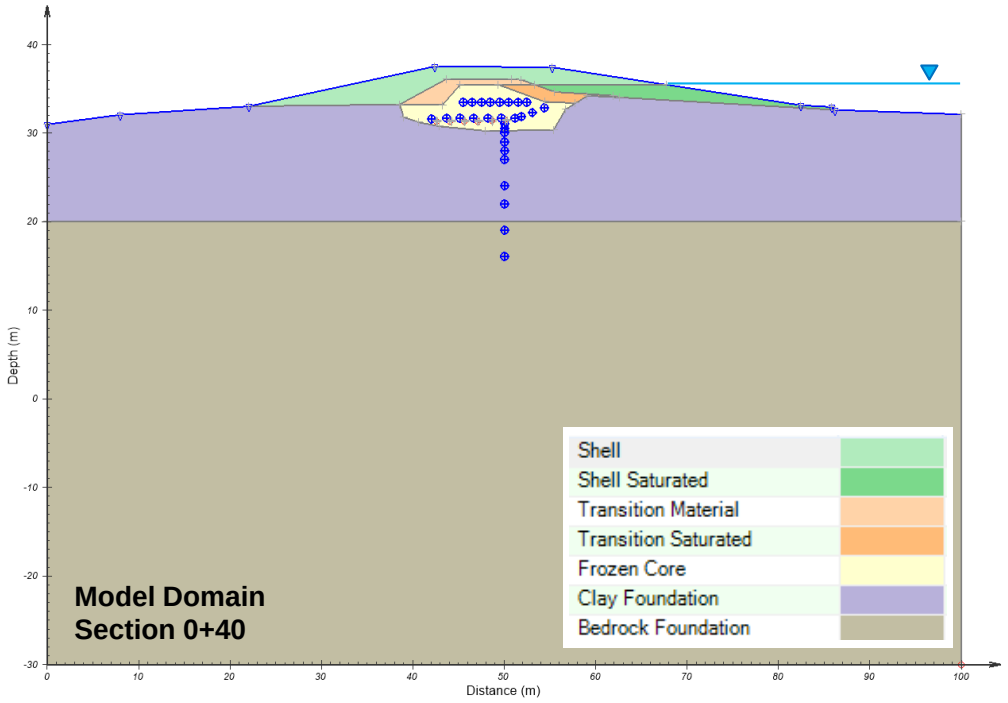




Notes:
1. Red dots indicate evaporator pipes
2. Blue dots indicate thermistor nodes

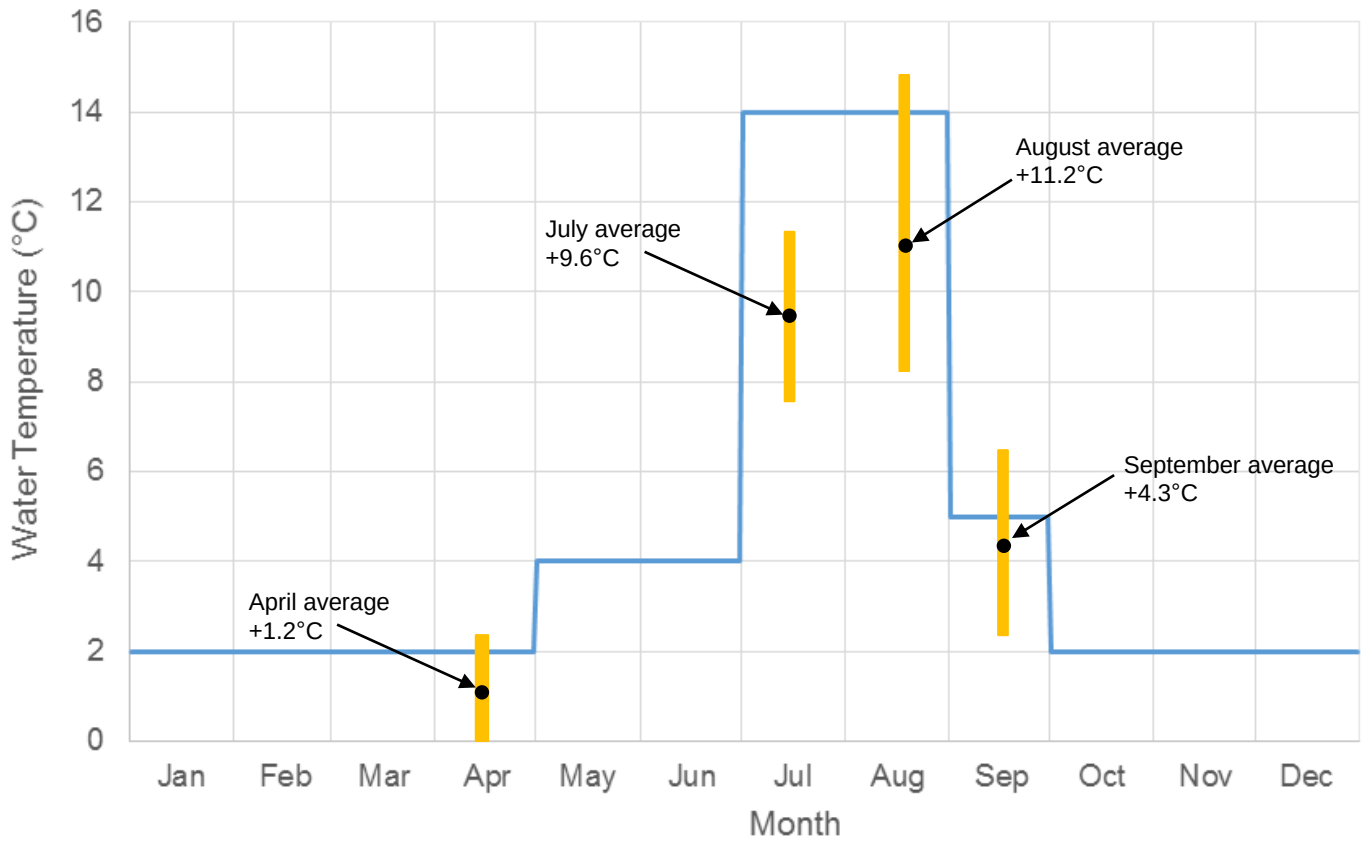
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**North Dam Model Domain (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **11**





Notes:
1. Red dots indicate evaporator pipes
2. Blue dots indicate thermistor nodes

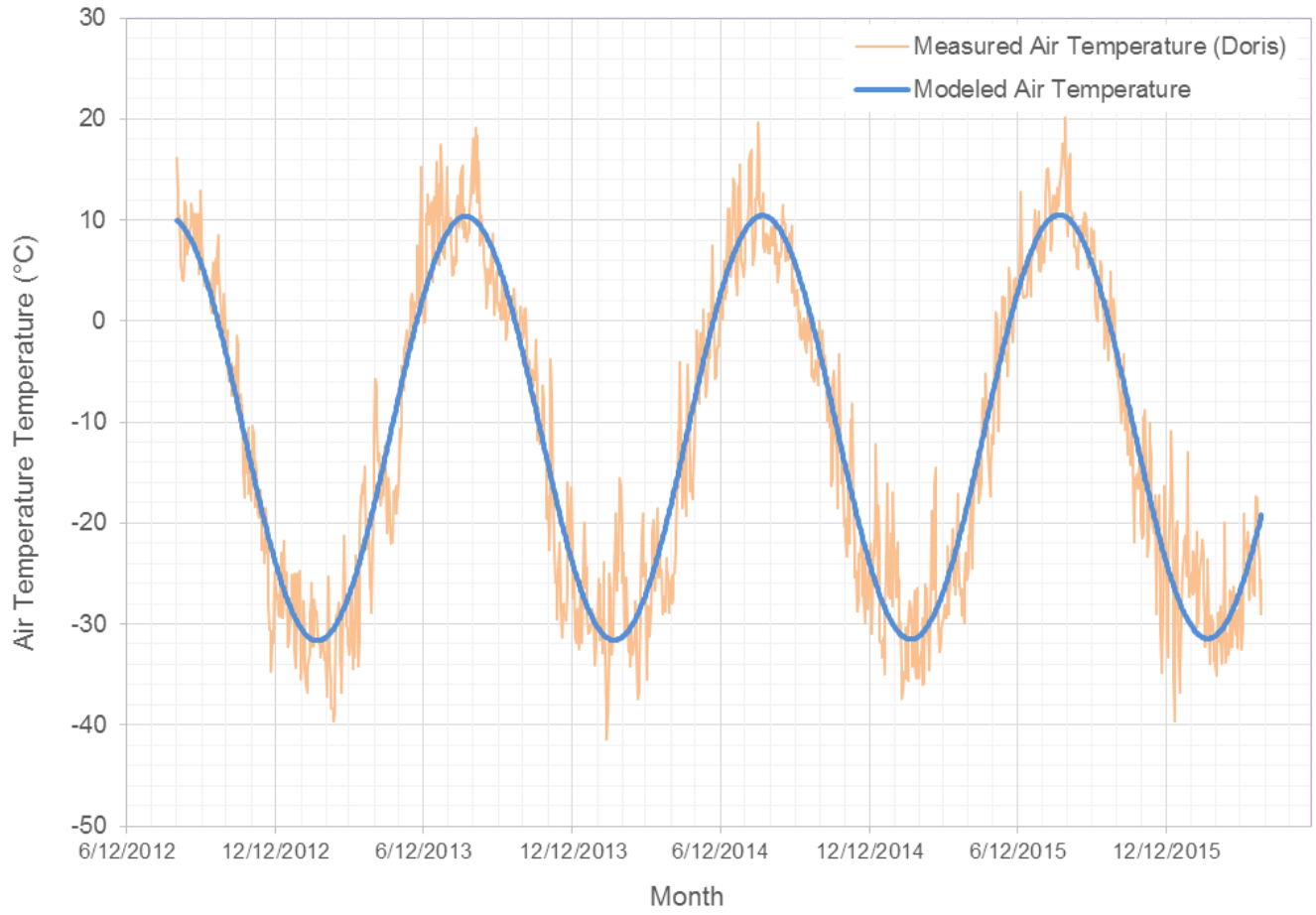
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam Model Domain (0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **12**



Notes:

1. Solid blue line represents water temperature used in the thermal model
2. Yellow bars represent range of lake water temperature measured at Property

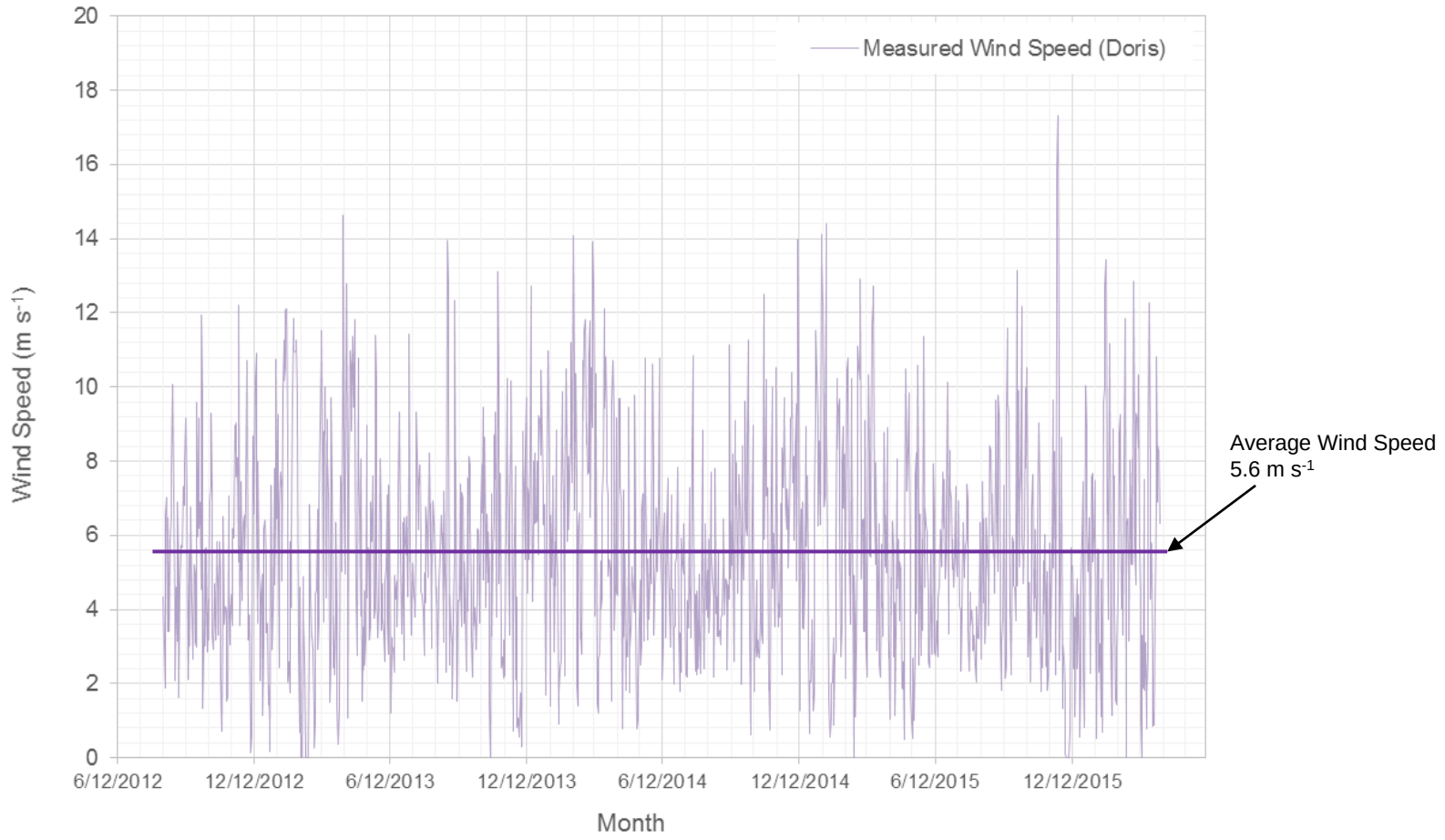
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		Model Input – Water Temperature
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **13**



Notes:

1. Daily air temperature measured at Doris Meteorological Station
2. Modeled air temperature used in the thermal model

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Model Input – Air Temperature
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **14**

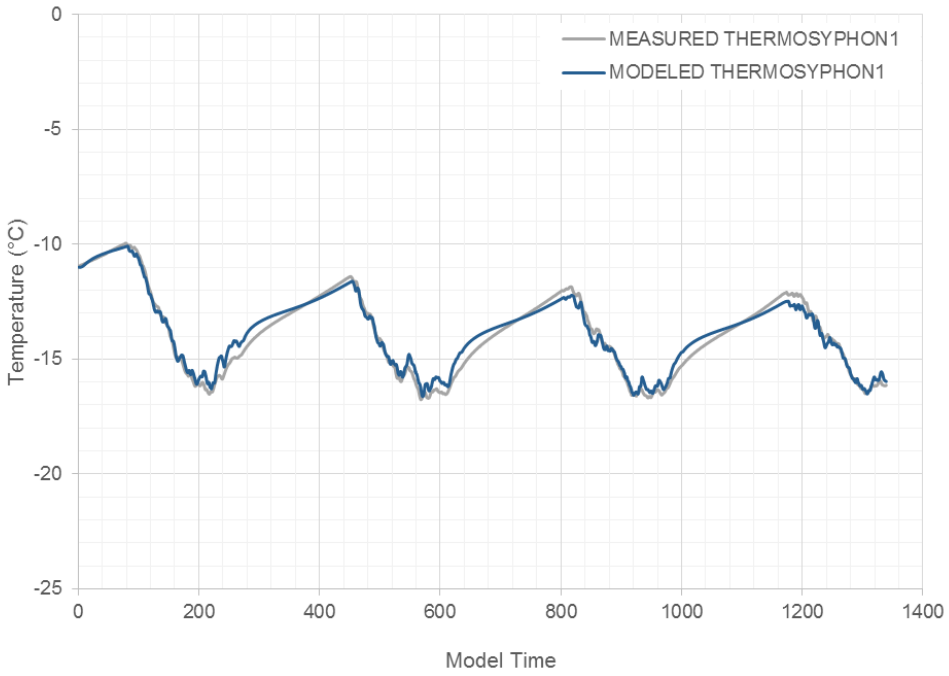


Notes:

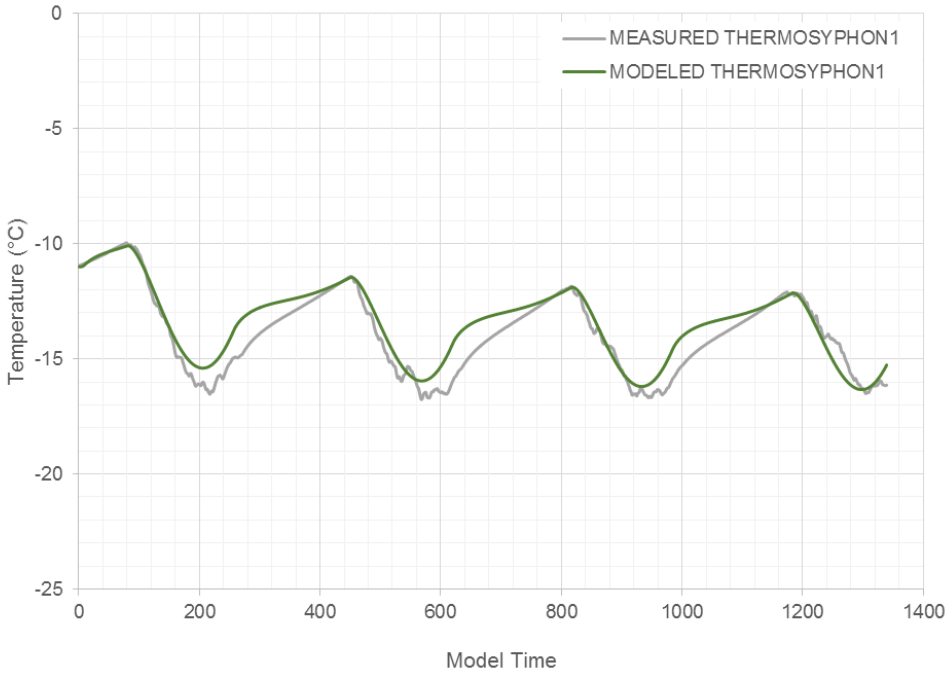
1. Daily wind speed measured at Doris Meteorological Station
2. Wind speed data used for model calibration

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		Model Input – Wind Speed
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **15**

Model 1: Measured Air Temperature and Wind Speed



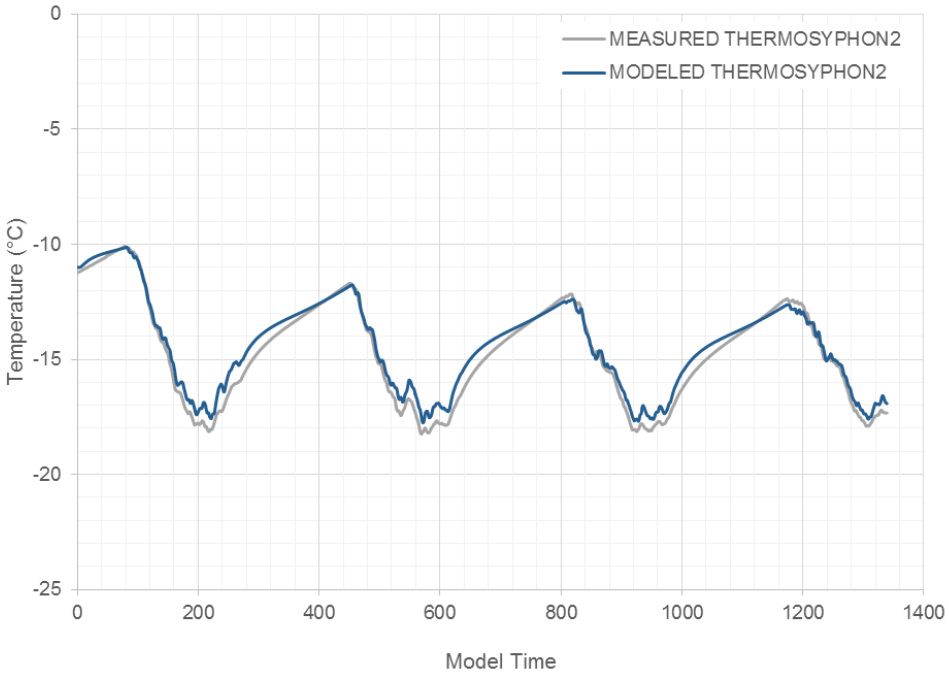
Model 2: Climate Boundary with Average Wind Speed



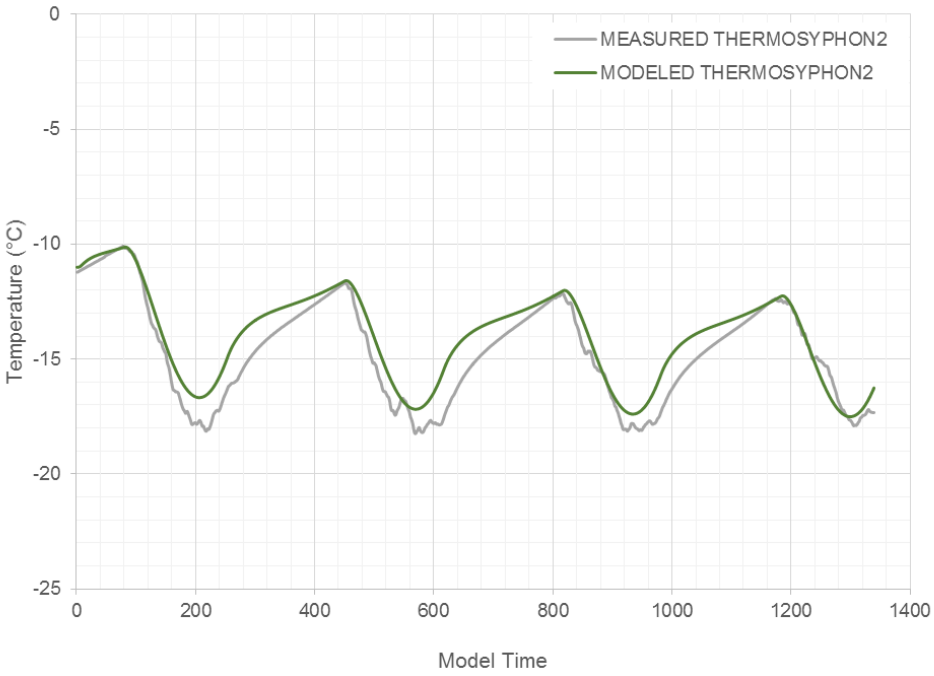
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 1, Dam Section 0+85

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 1 (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **16**

Model 1: Measured Air Temperature and Wind Speed



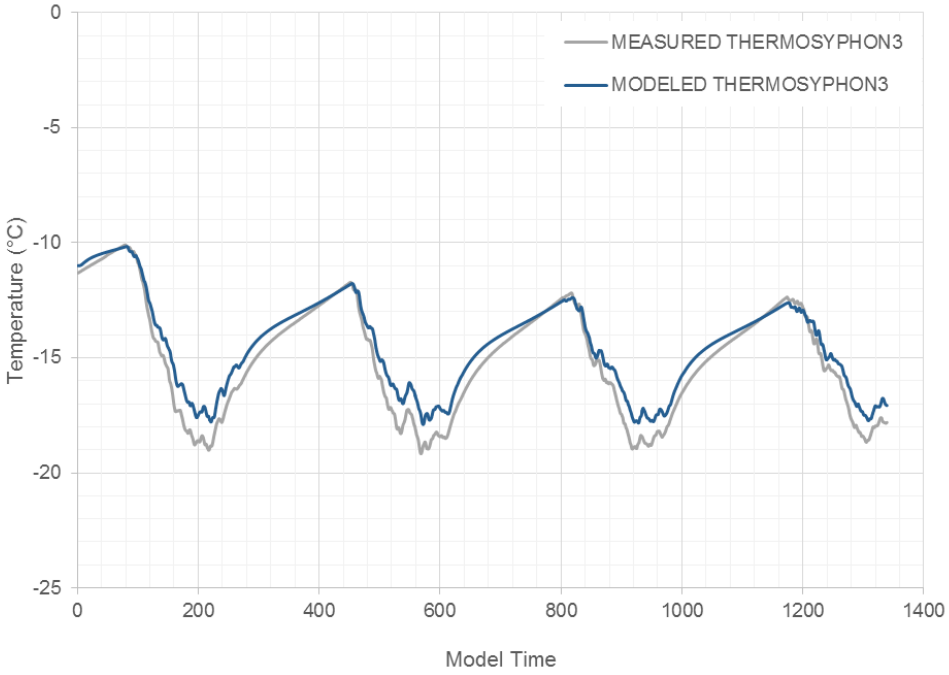
Model 2: Climate Boundary with Average Wind Speed



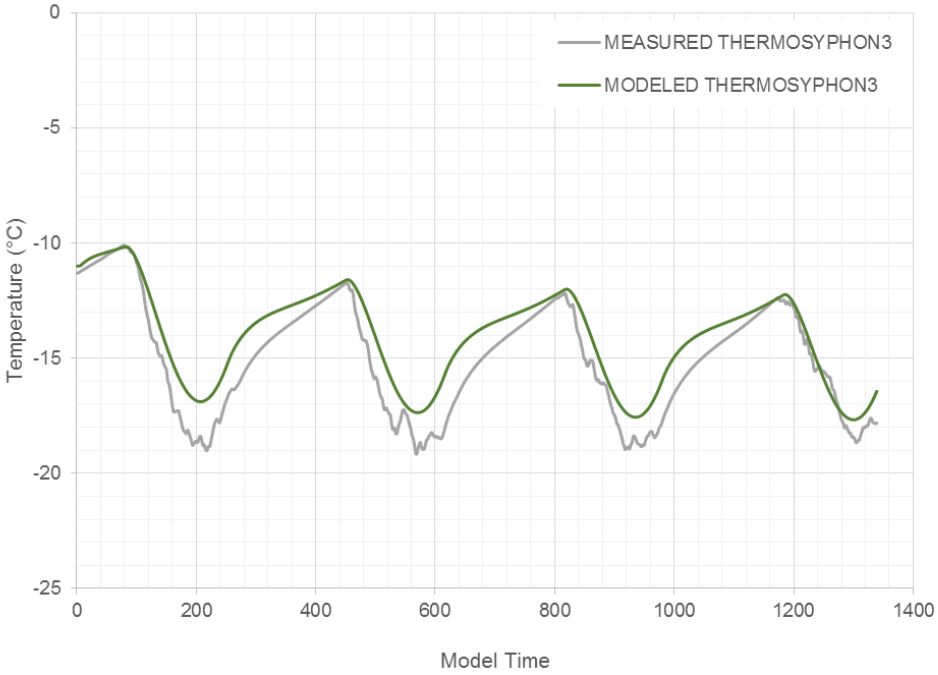
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 2, Dam Section 0+85

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling		
		Model Calibration – Thermosyphon Evaporator 2 (0+85)		
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017	Approved: cws	Figure: **17**

Model 1: Measured Air Temperature and Wind Speed



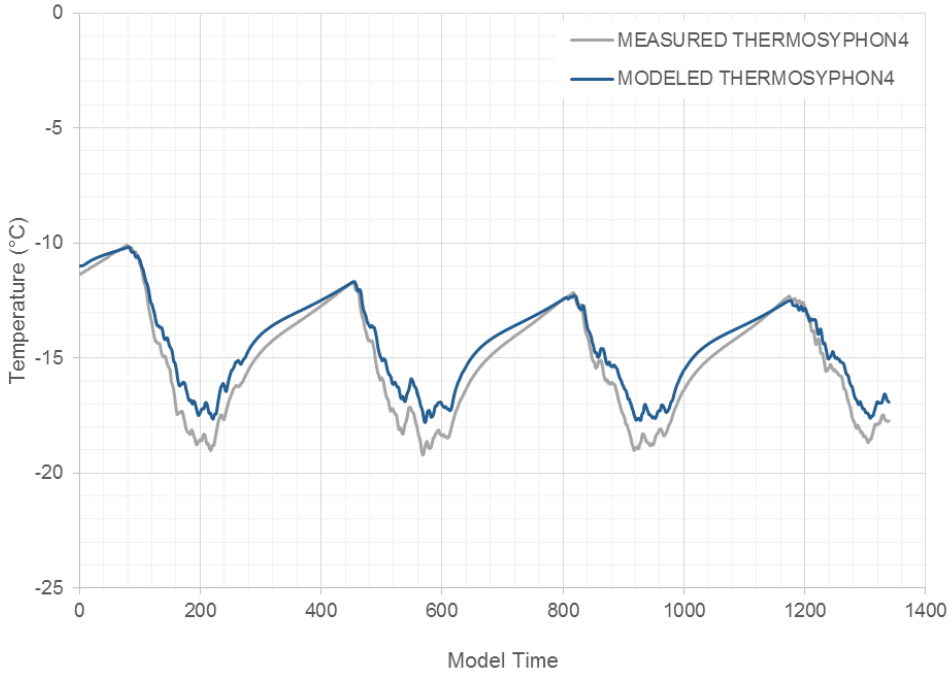
Model 2: Climate Boundary with Average Wind Speed



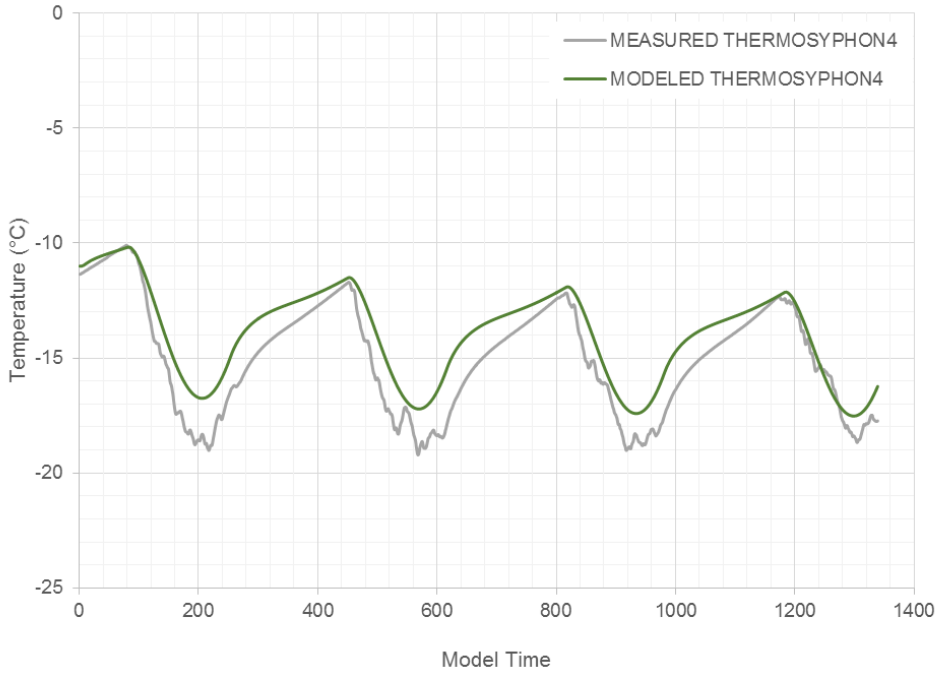
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 3, Dam Section 0+85

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 3 (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **18**

Model 1: Measured Air Temperature and Wind Speed



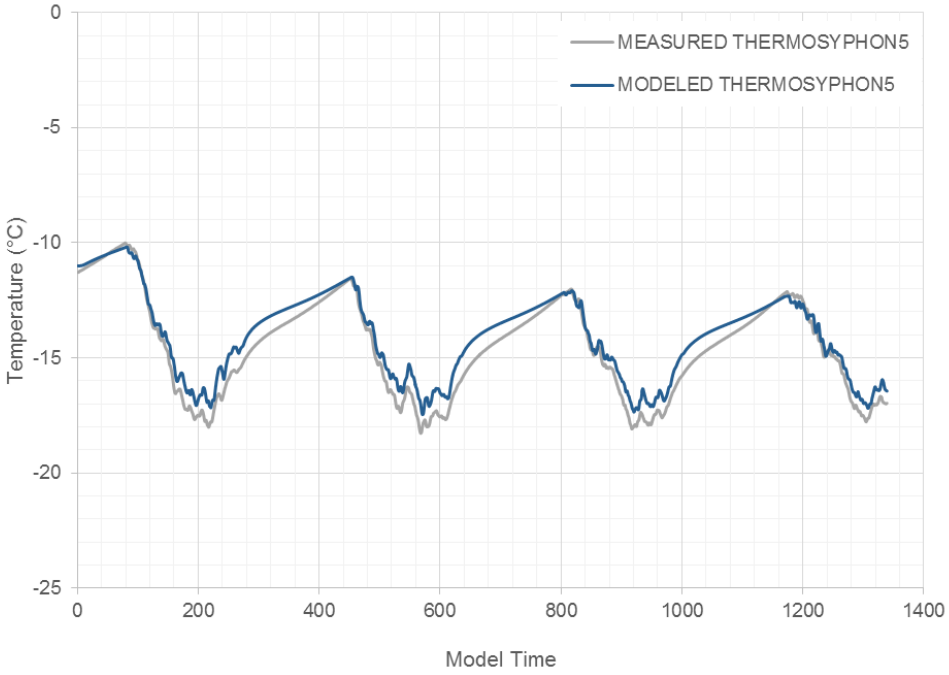
Model 2: Climate Boundary with Average Wind Speed



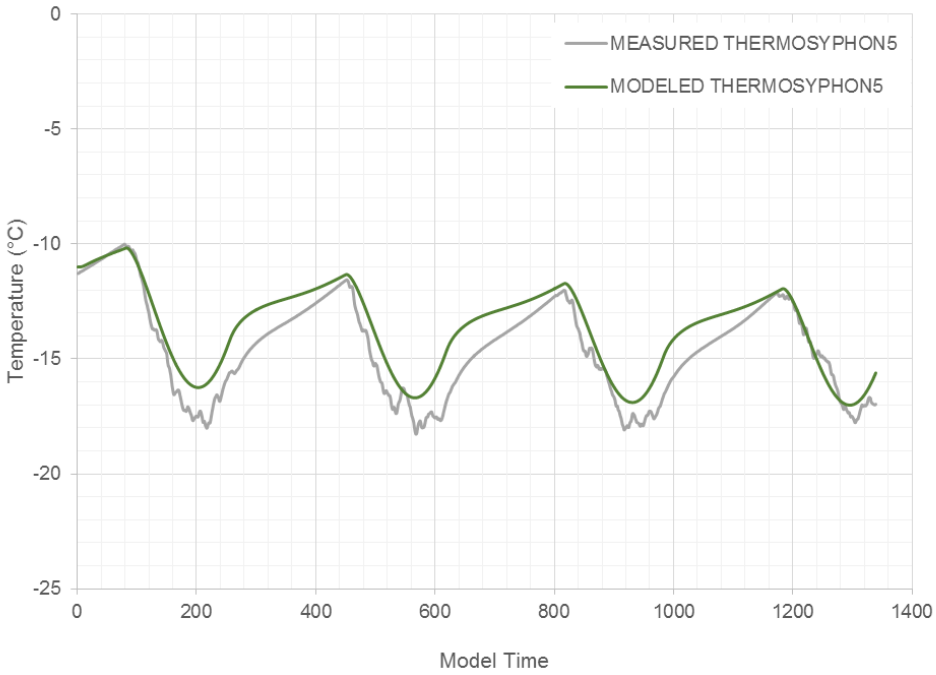
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 4, Dam Section 0+85

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 4 (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **19**

Model 1: Measured Air Temperature and Wind Speed



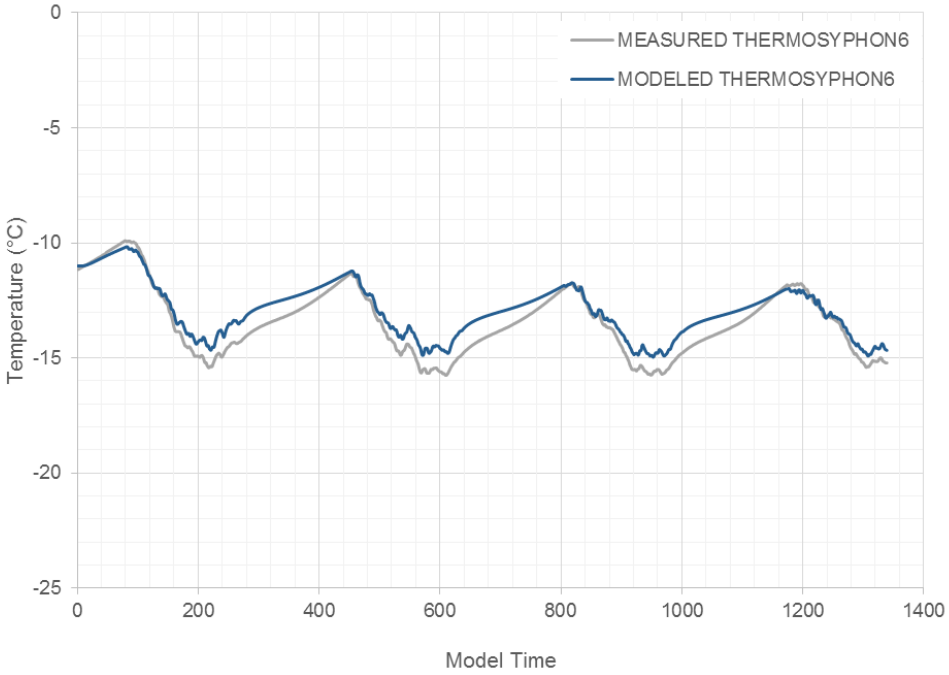
Model 2: Climate Boundary with Average Wind Speed



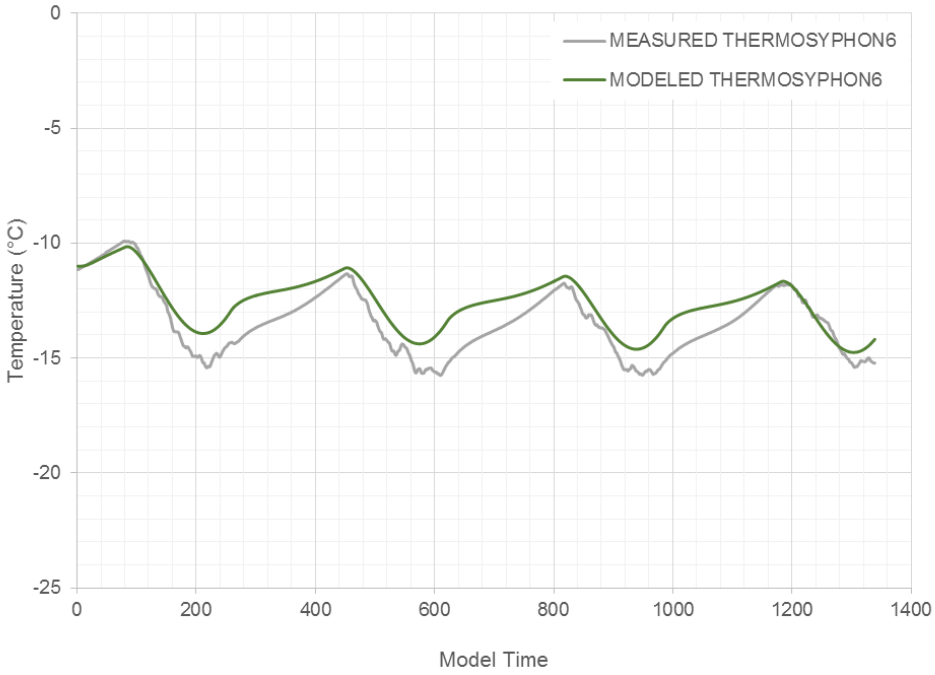
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 5, Dam Section 0+85

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 5 (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **20**

Model 1: Measured Air Temperature and Wind Speed



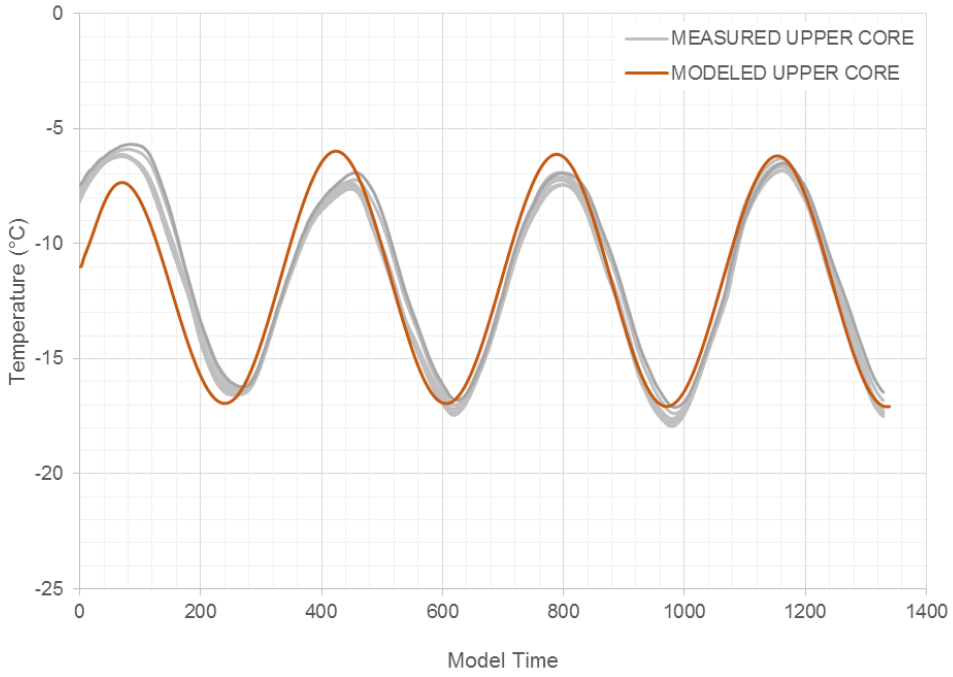
Model 2: Climate Boundary with Average Wind Speed



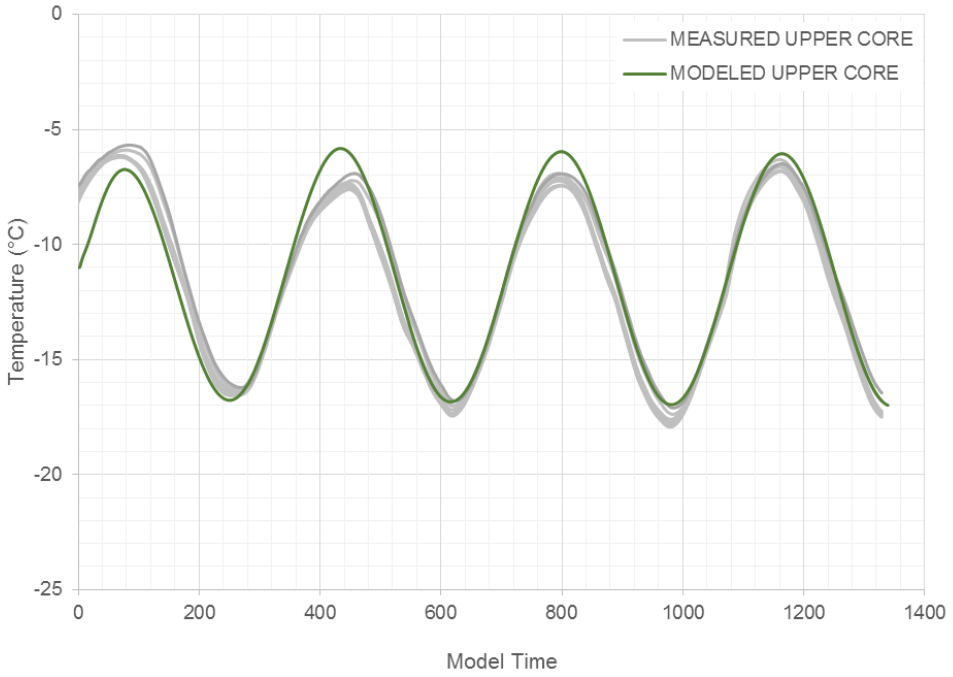
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-25.3
2. Thermistor node located near thermosyphon evaporator 6, Dam Section 0+85

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 6 (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **21**

Model 1: Measured Air Temperature and Wind Speed



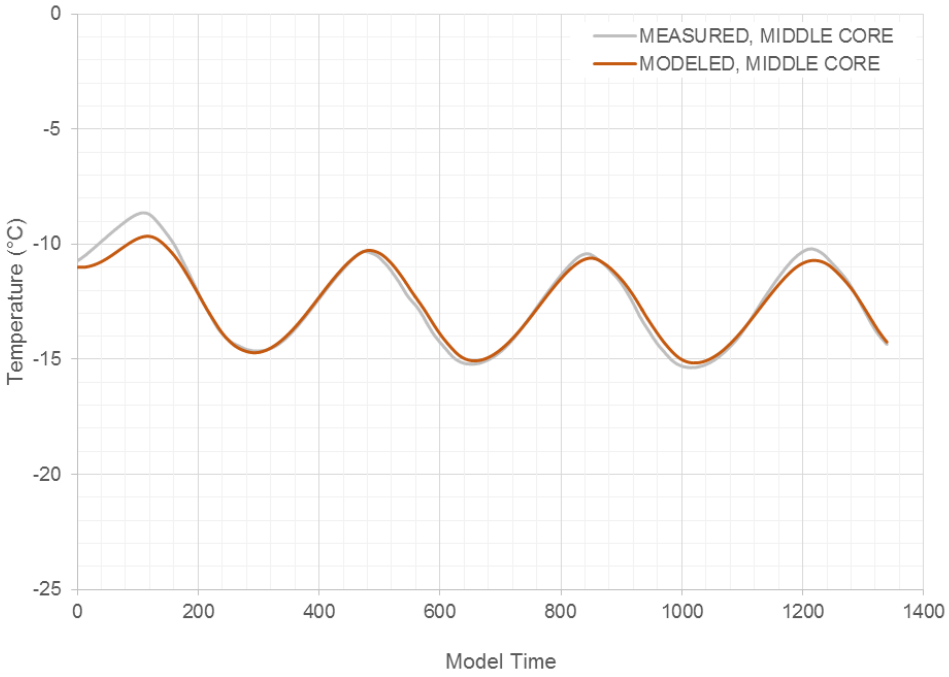
Model 2: Climate Boundary with Average Wind Speed



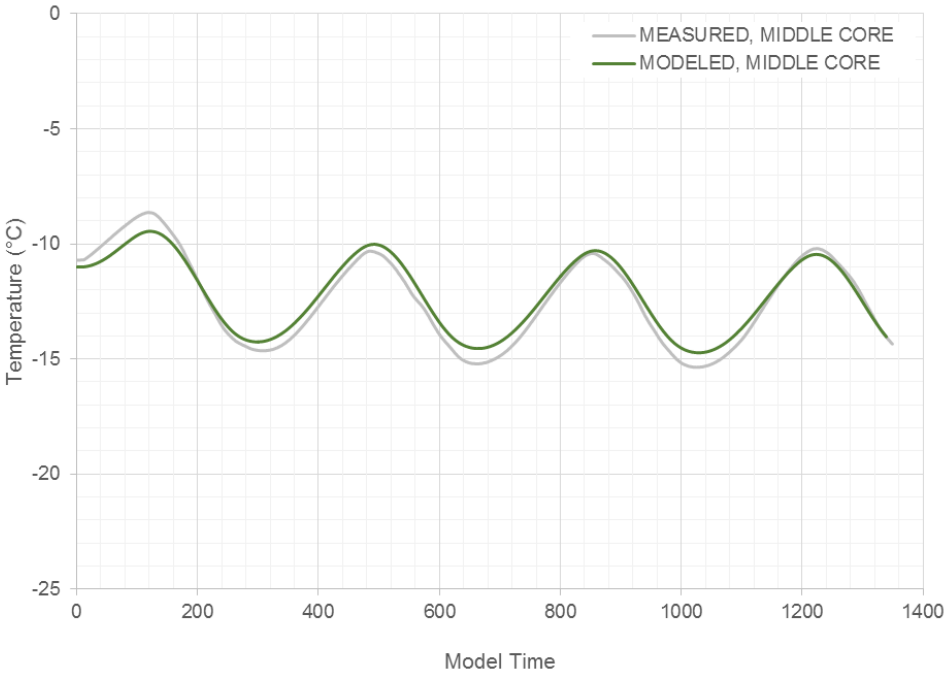
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-33.5
2. Thermistor cable is located near the top of the frozen core, Dam Section 0+85

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Upper Core (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **22**

Model 1: Measured Air Temperature and Wind Speed



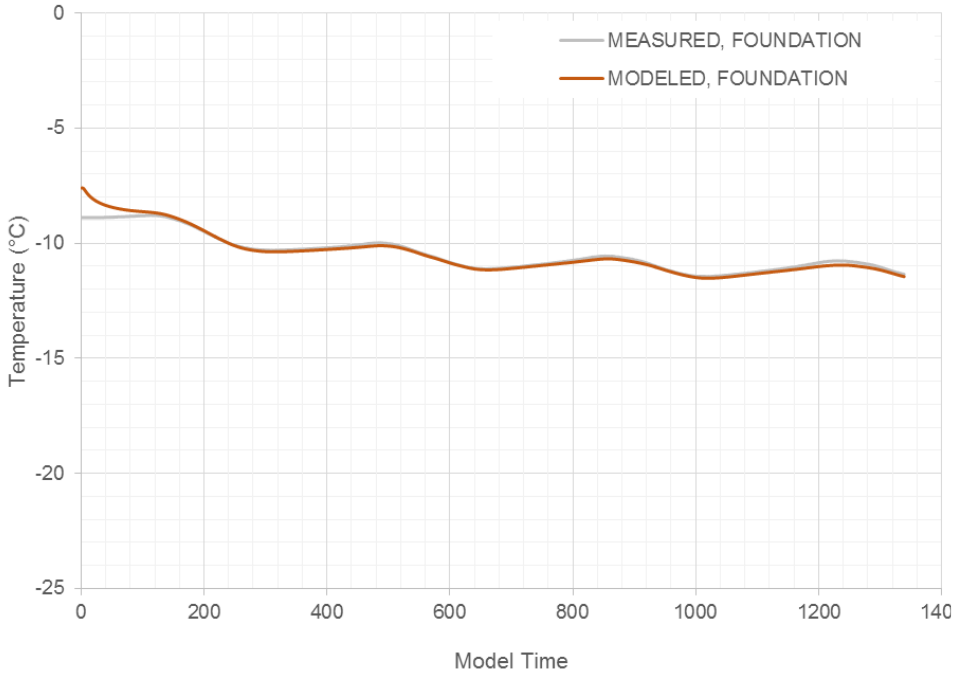
Model 2: Climate Boundary with Average Wind Speed



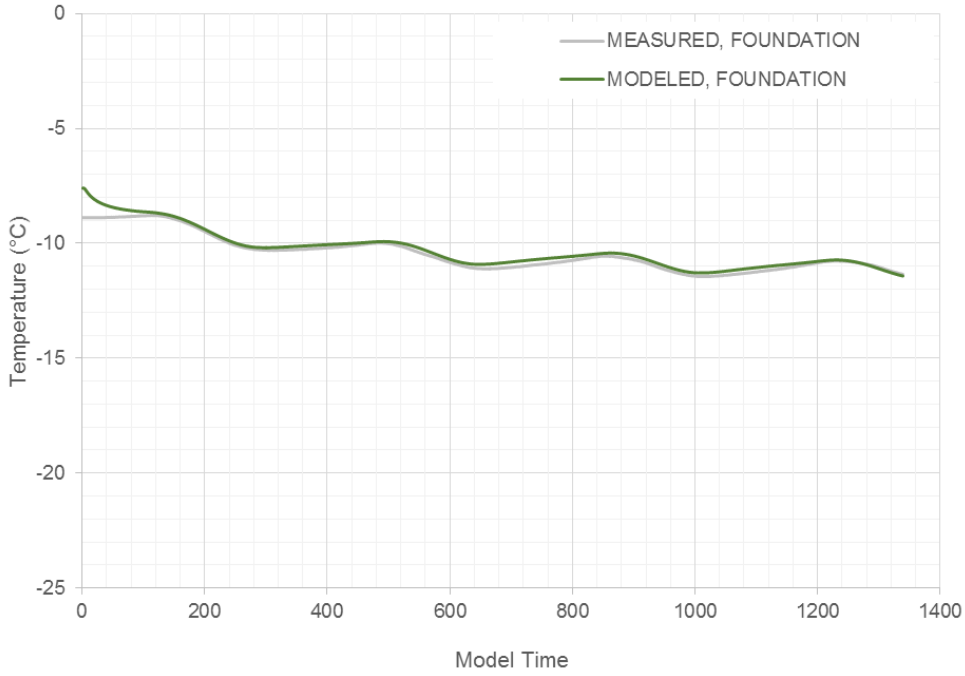
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-085-29.4
2. Thermistor cable is located near the middle of the frozen core, Dam Section 0+85

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling		
		Model Calibration – Middle Core (0+85)		
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017	Approved: cws	Figure: **23**

Model 1: Measured Air Temperature and Wind Speed



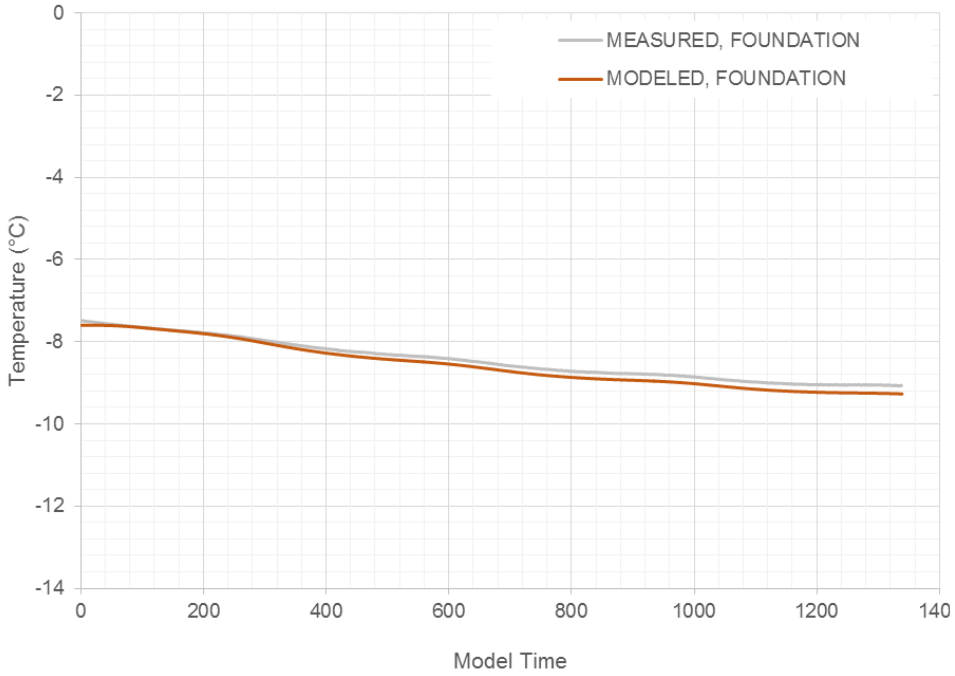
Model 2: Climate Boundary with Average Wind Speed



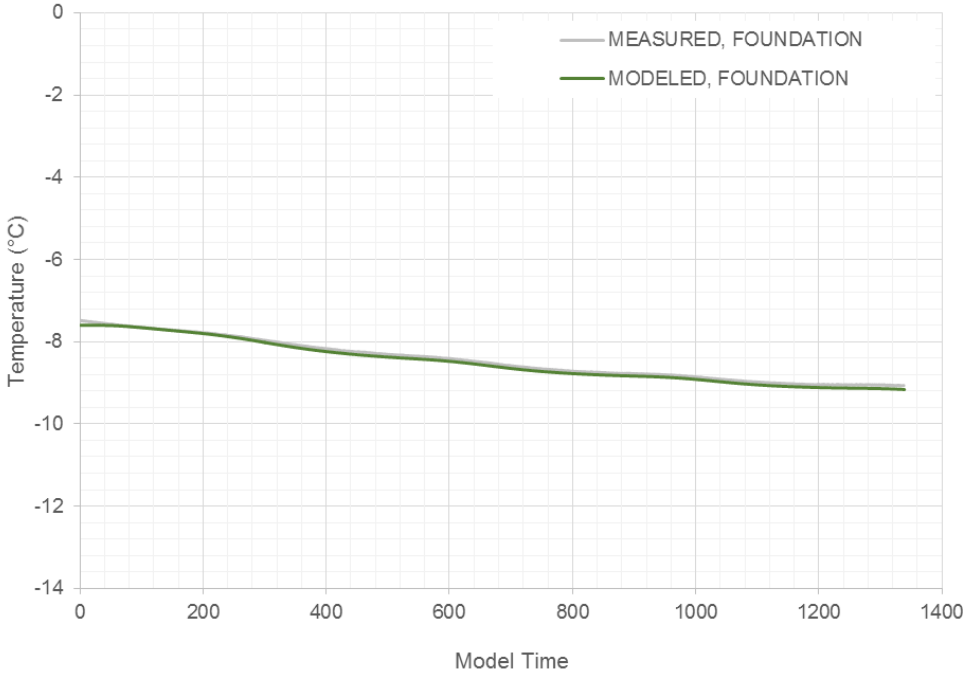
Notes:
1. Ground temperature measurements from thermistor string ND-VTS-085-KT
2. Thermistor node located 2 m below base of key trench, Dam Section 0+85
3. Depth referenced to uppermost thermistor node at base of key trench

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Foundation 2 m Below Key Trench (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **24**

Model 1: Measured Air Temperature and Wind Speed



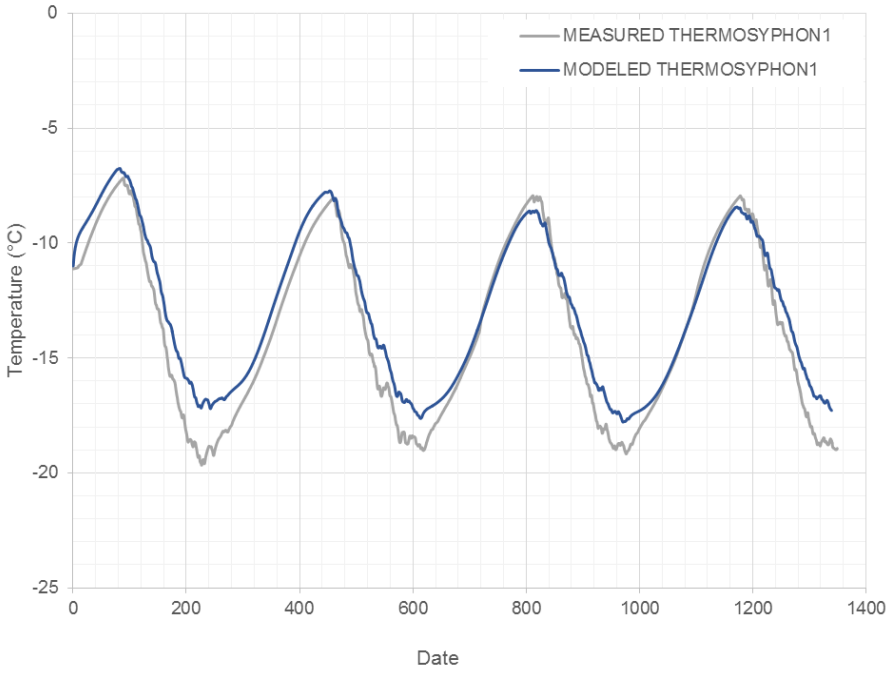
Model 2: Climate Boundary with Average Wind Speed



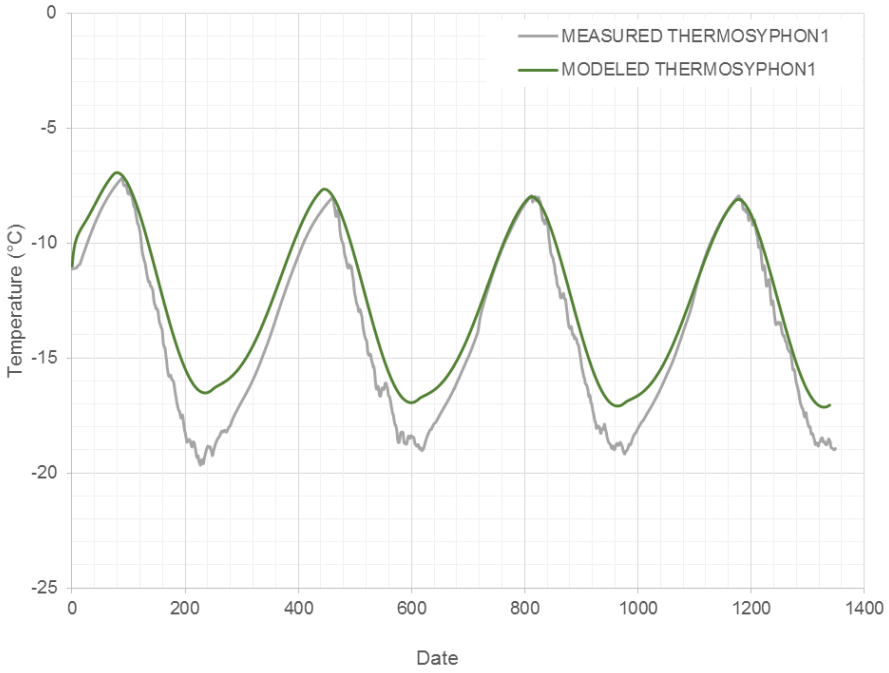
Notes:
1. Ground temperature measurements from thermistor string ND-VTS-085-KT
2. Thermistor node located 9 m below base of key trench, Dam Section 0+85
3. Depth referenced to uppermost thermistor node at base of key trench

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Foundation 9 m Below Key Trench (0+85)**
		Date: 8/15/2017 | Approved: cws | Figure: **25**

Model 1: Measured Air Temperature and Wind Speed



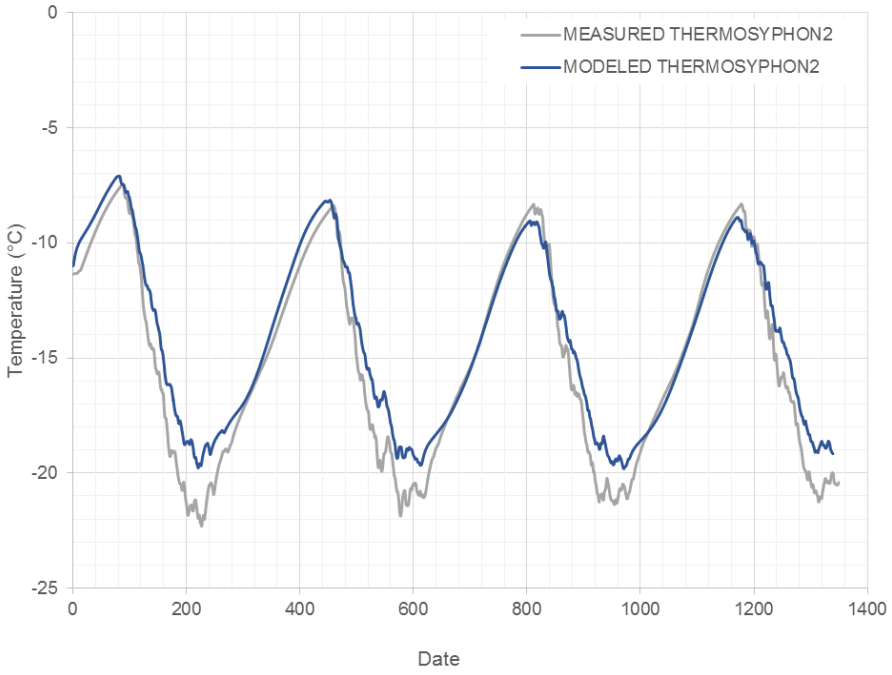
Model 2: Climate Boundary with Average Wind Speed



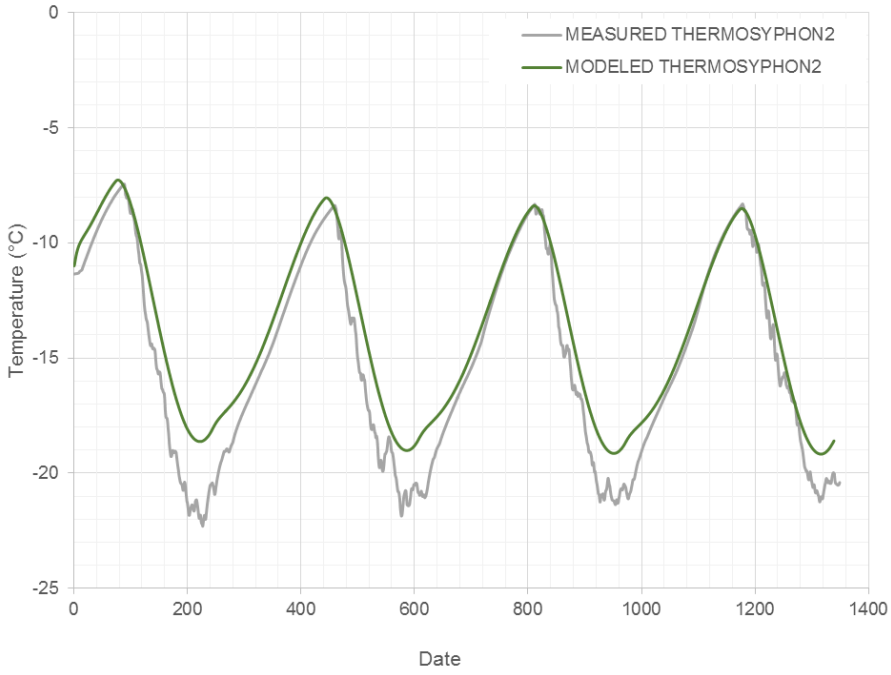
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 1, Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 1 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **26**

Model 1: Measured Air Temperature and Wind Speed



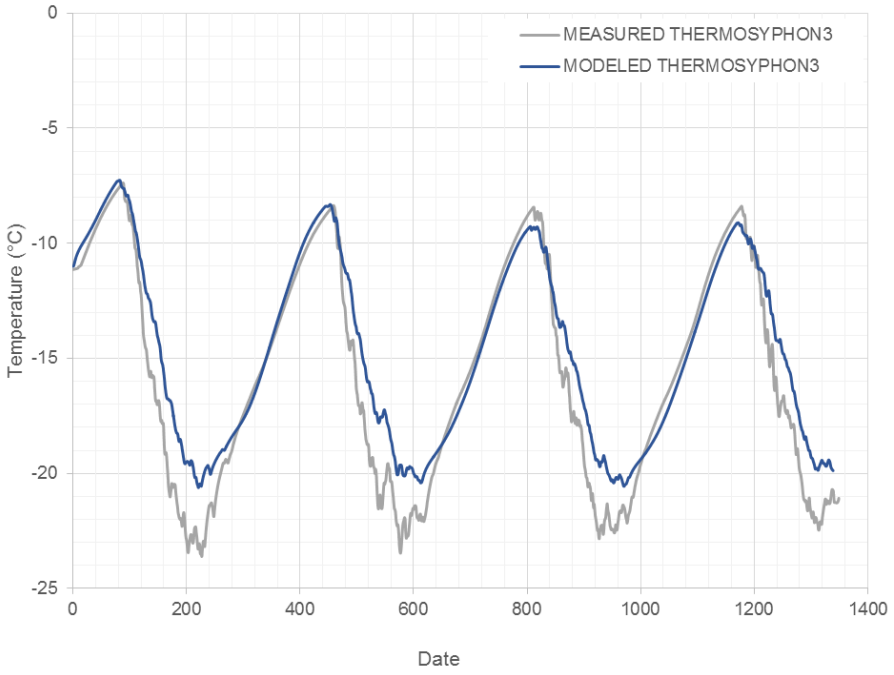
Model 2: Climate Boundary with Average Wind Speed



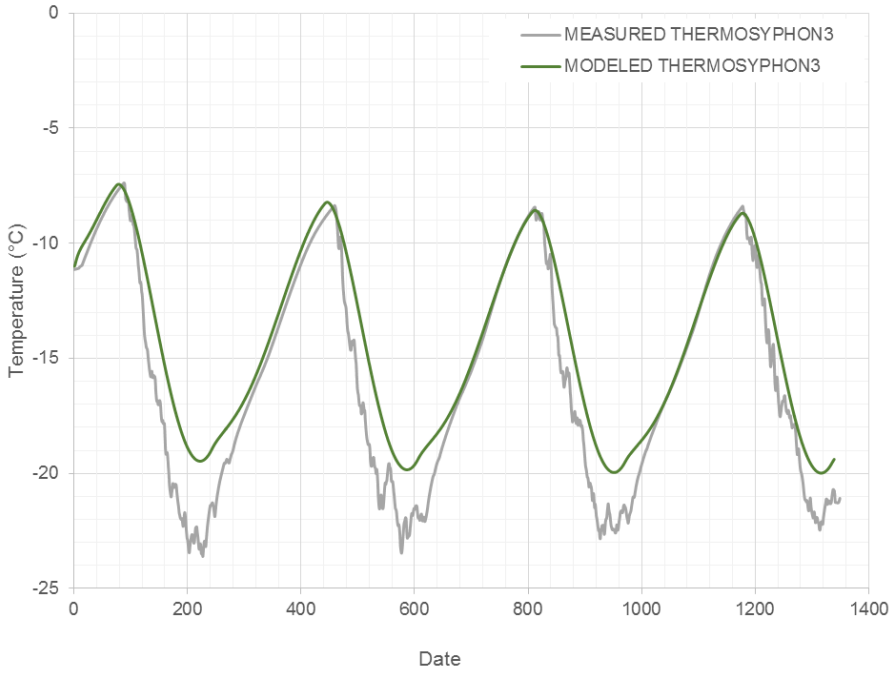
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 2, Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 2 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **27**

Model 1: Measured Air Temperature and Wind Speed



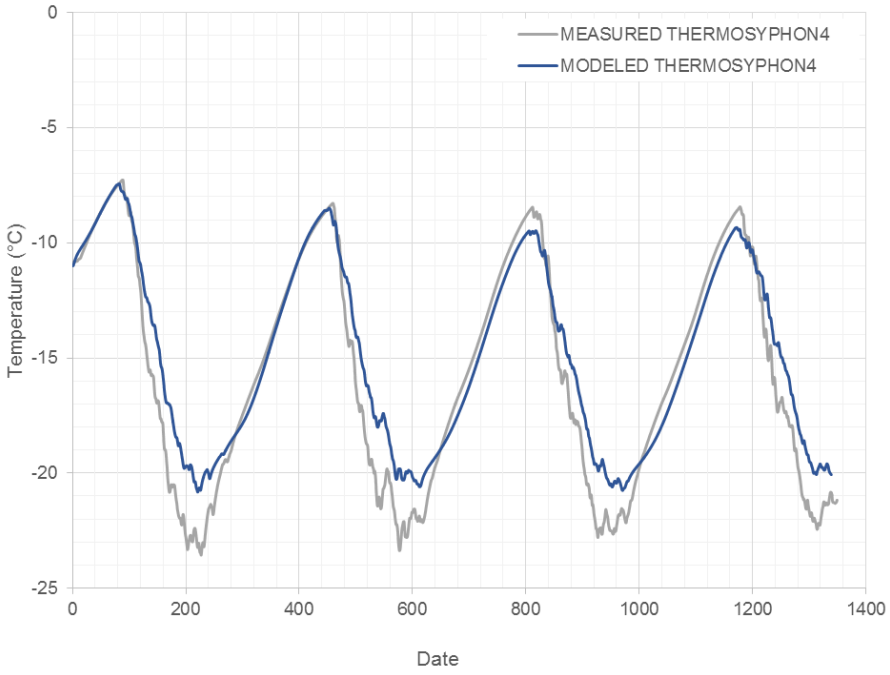
Model 2: Climate Boundary with Average Wind Speed



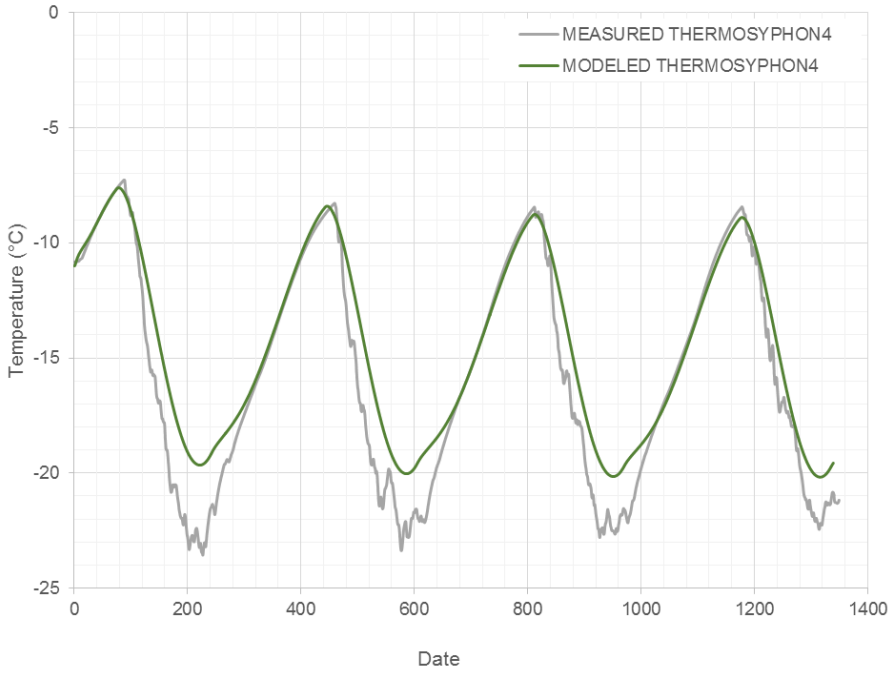
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 3, Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 3 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **28**

Model 1: Measured Air Temperature and Wind Speed



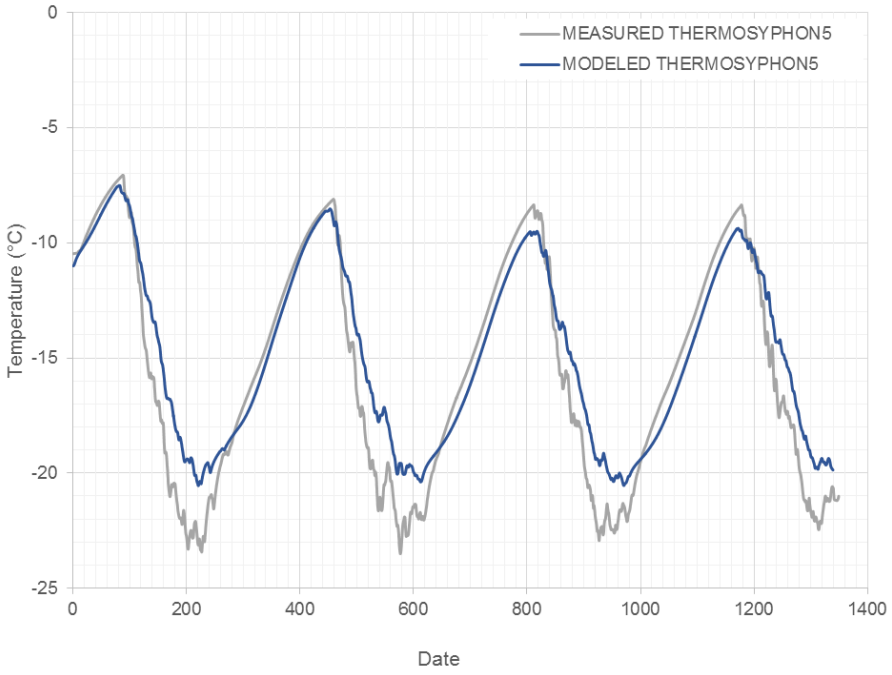
Model 2: Climate Boundary with Average Wind Speed



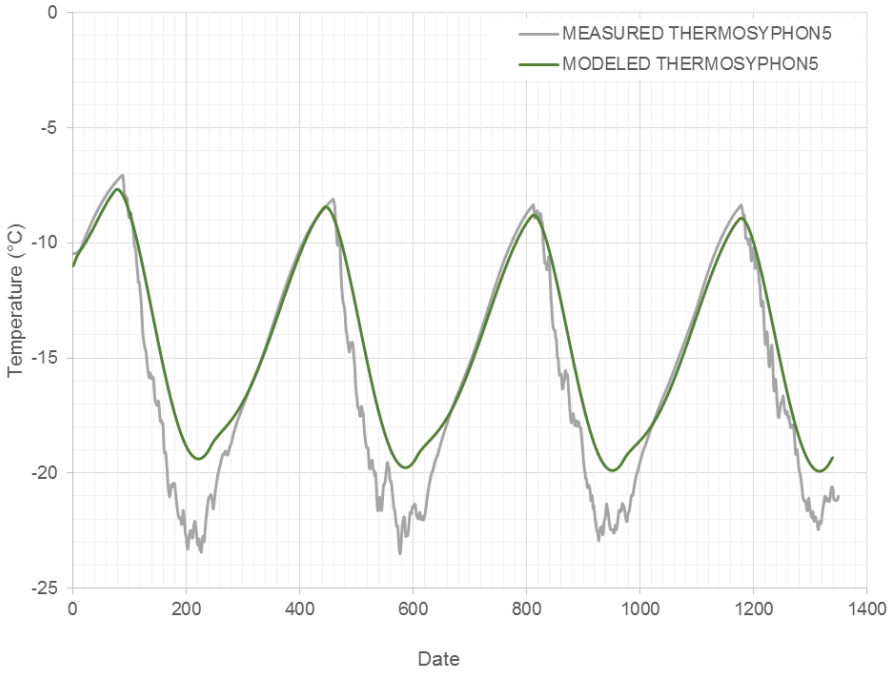
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 4, Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 4 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **29**

Model 1: Measured Air Temperature and Wind Speed



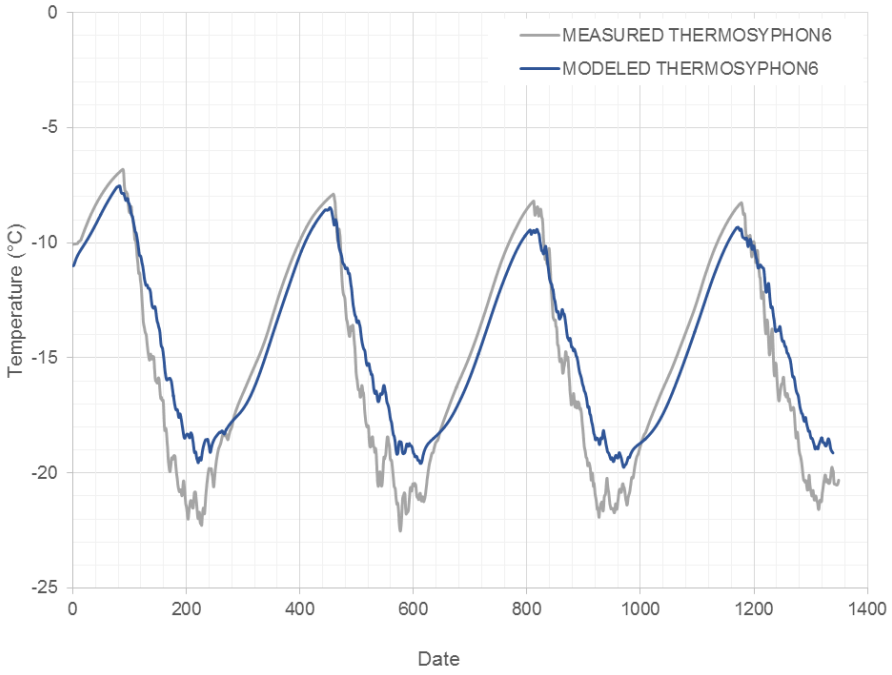
Model 2: Climate Boundary with Average Wind Speed



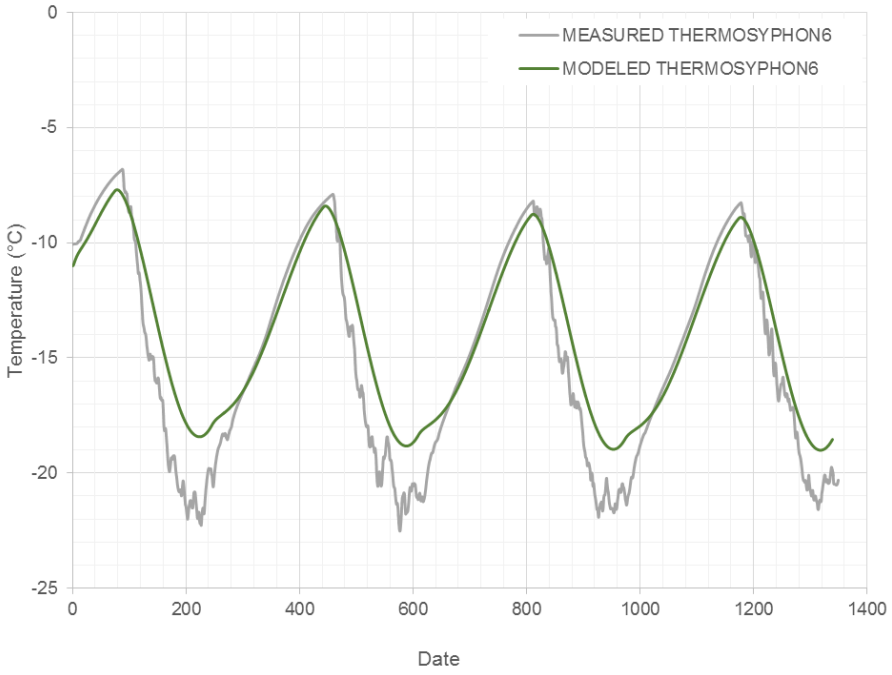
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 5, Dam Section 0+40

srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 5 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **30**

Model 1: Measured Air Temperature and Wind Speed



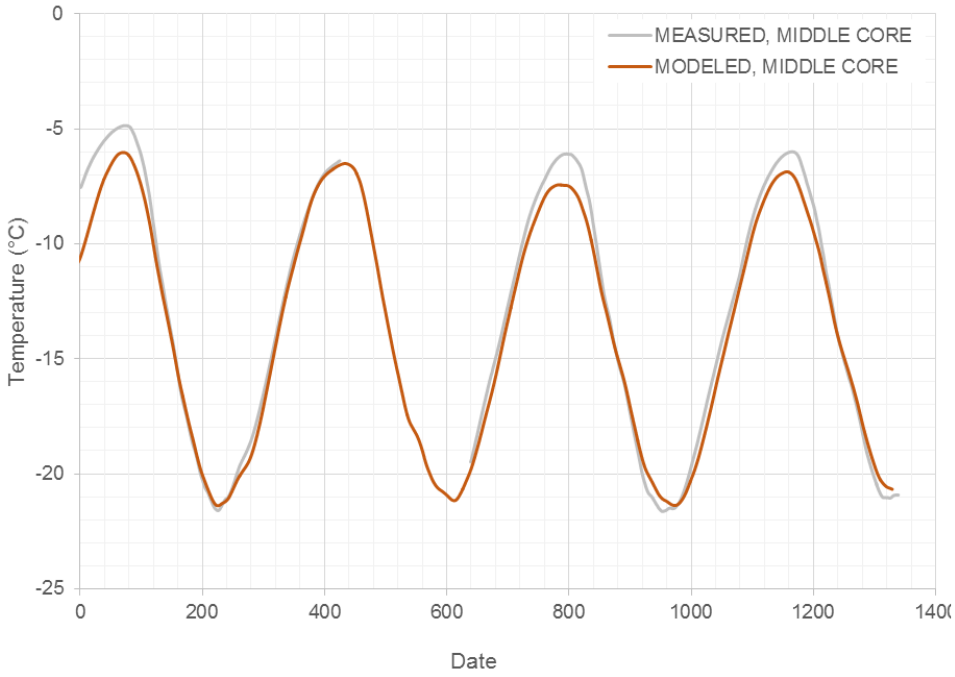
Model 2: Climate Boundary with Average Wind Speed



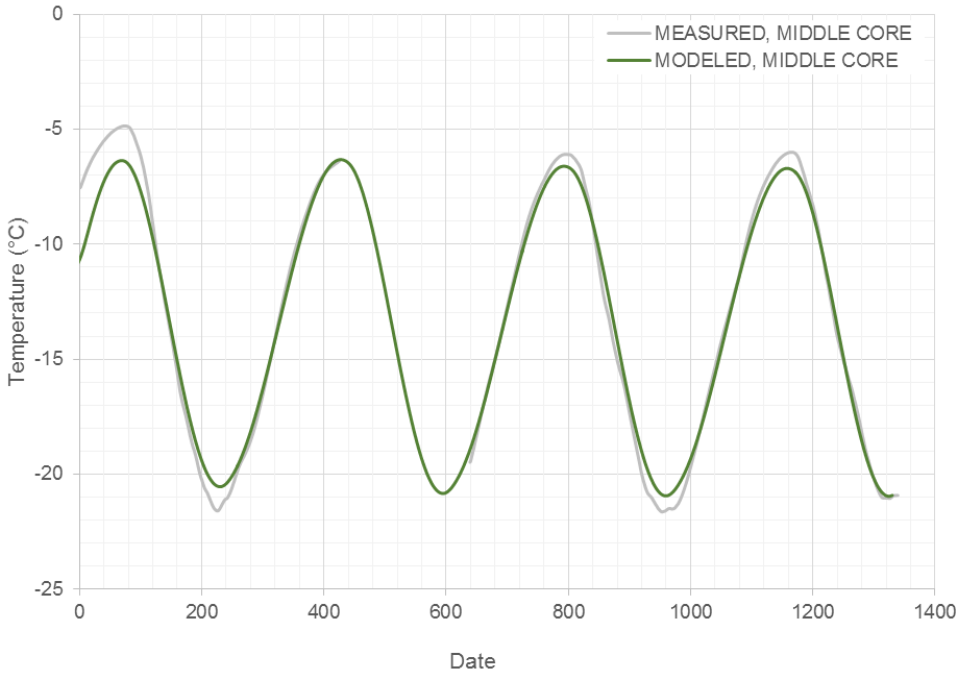
Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-31.5
2. Thermistor node located near thermosyphon evaporator 6, Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	**Model Calibration – Thermosyphon Evaporator 6 (0+40)**
		Date: 8/15/2017 | Approved: cws | Figure: **31**

Model 1: Measured Air Temperature and Wind Speed

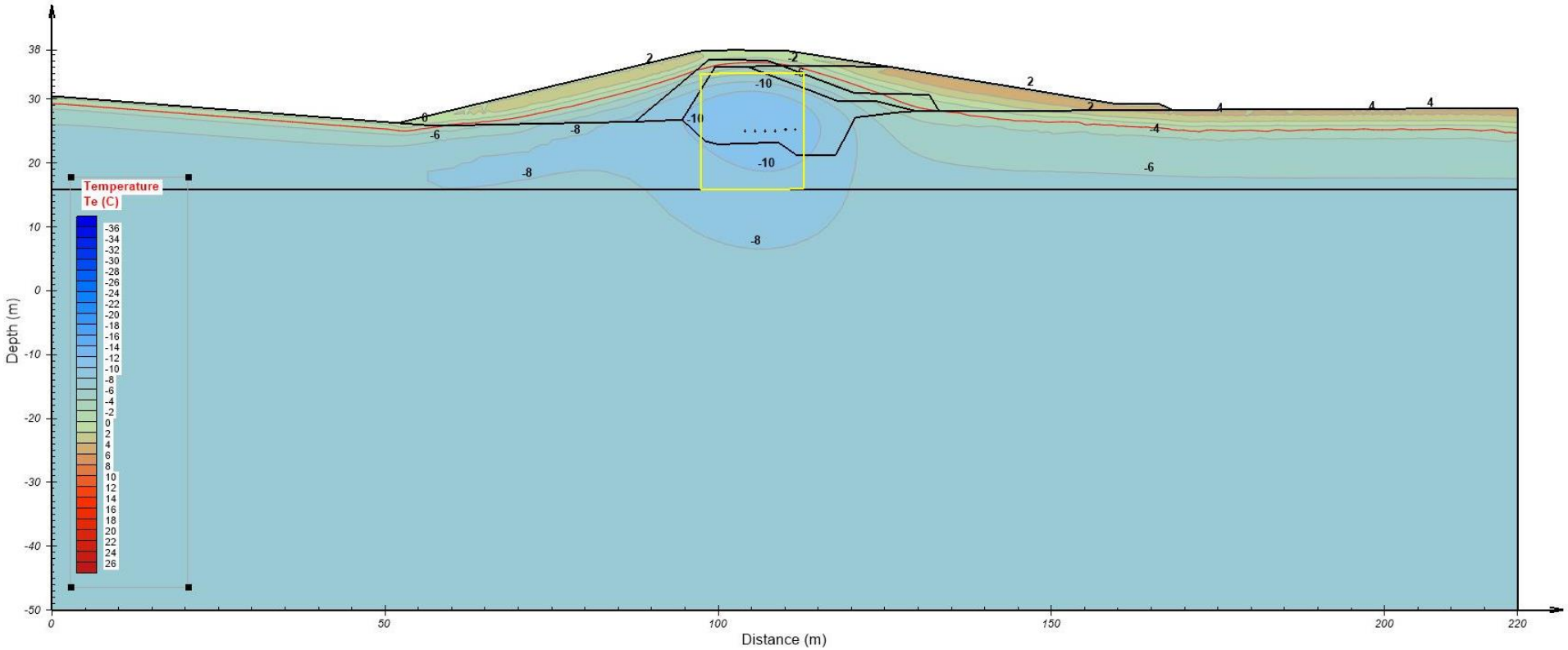


Model 2: Climate Boundary with Average Wind Speed



Notes:
1. Ground temperature measurements from thermistor string ND-HTS-040-33.5
2. Thermistor string is located near the middle of the frozen core, Dam Section 0+40

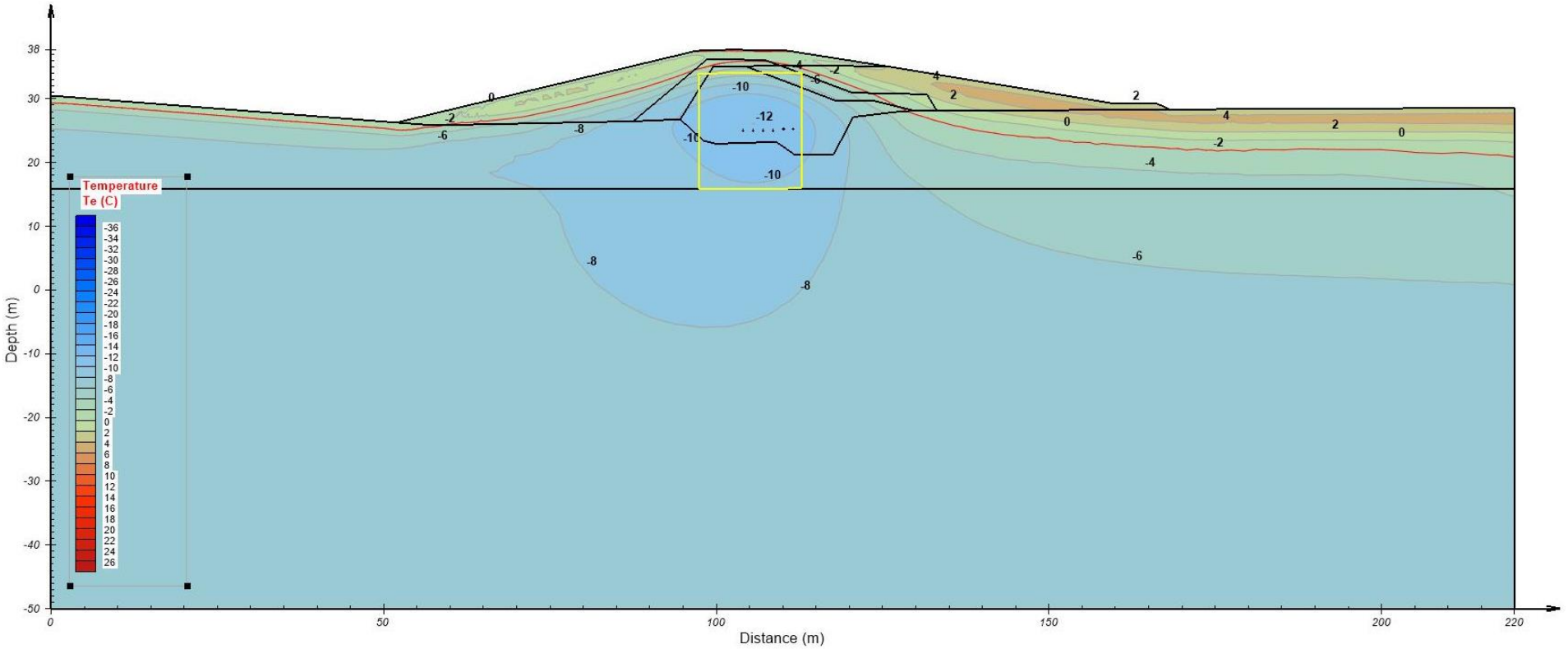
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		Model Calibration – Middle Core (0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **32**



Notes:

1. Model results for year 2, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

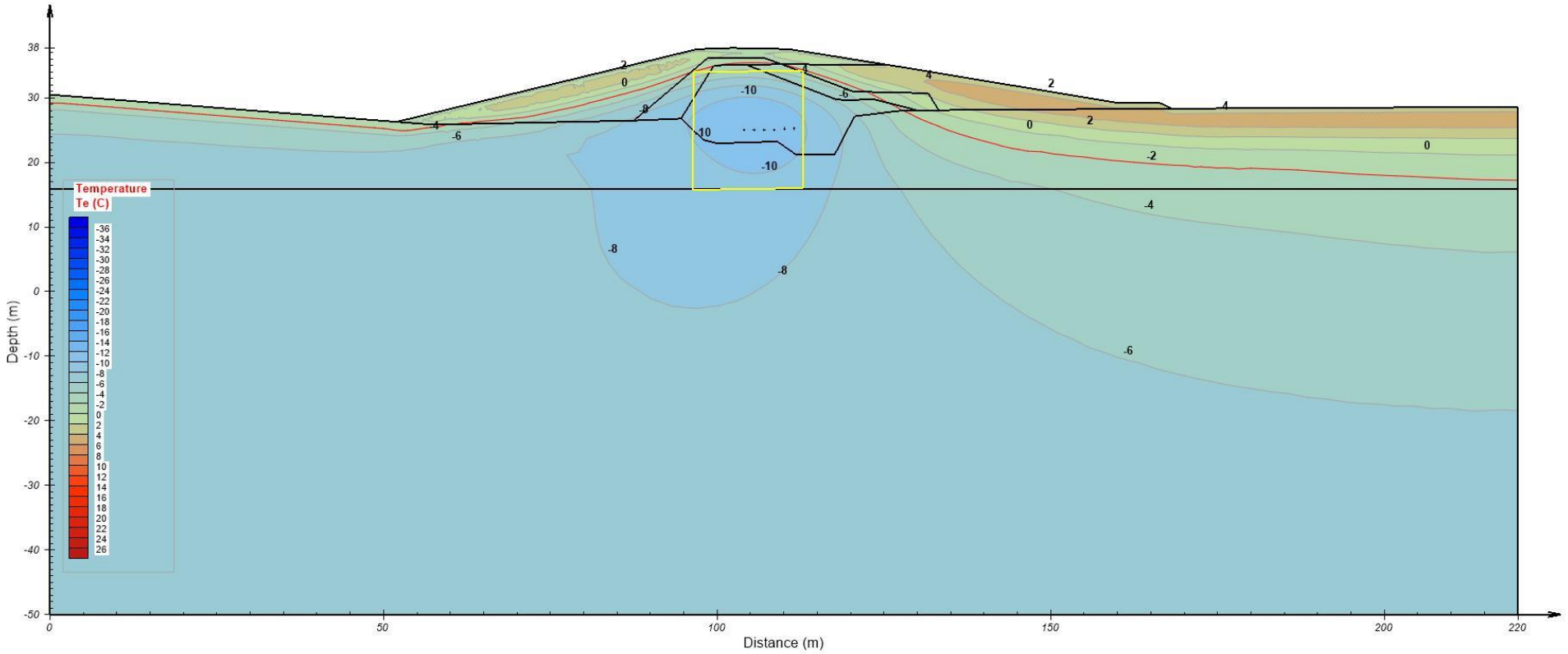
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 2 (Section 0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **33**



Notes:

1. Model results for year 10, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

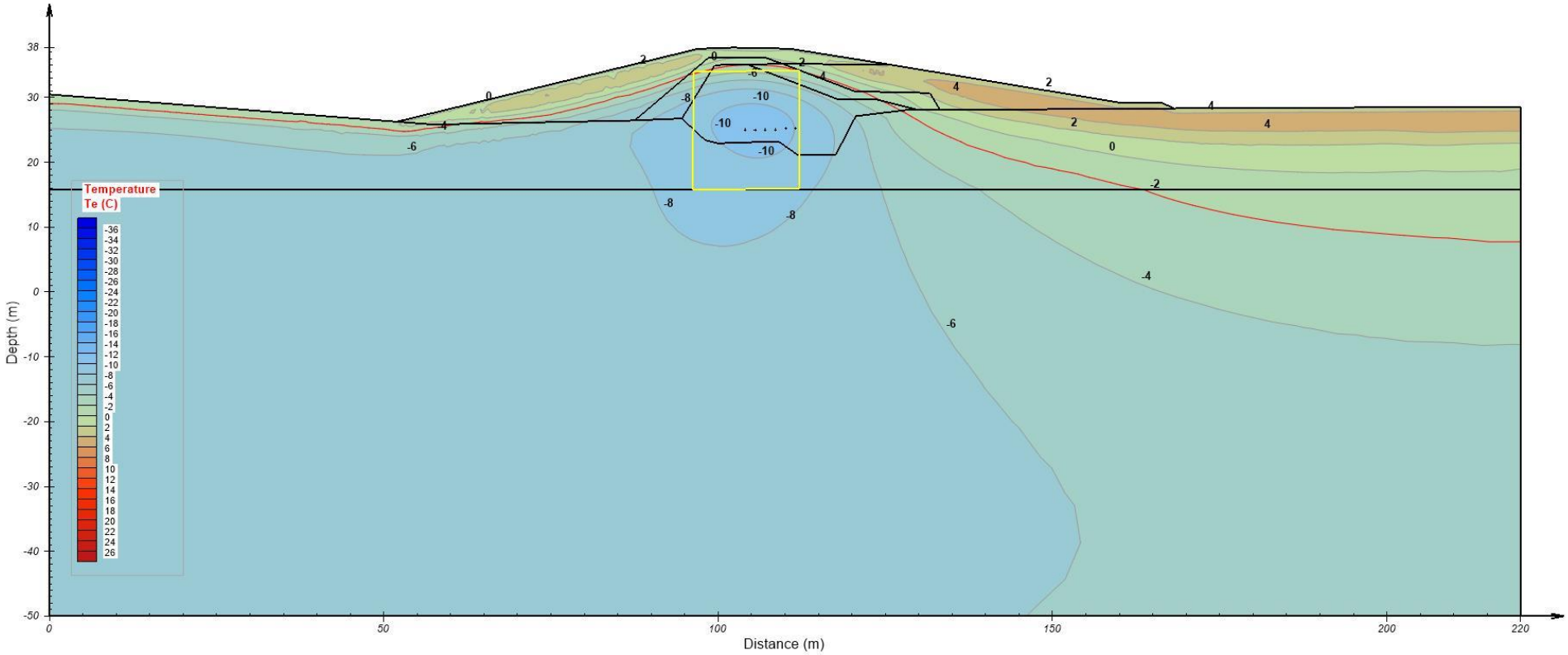
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 10 (Section 0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **34**



Notes:

1. Model results for year 20, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

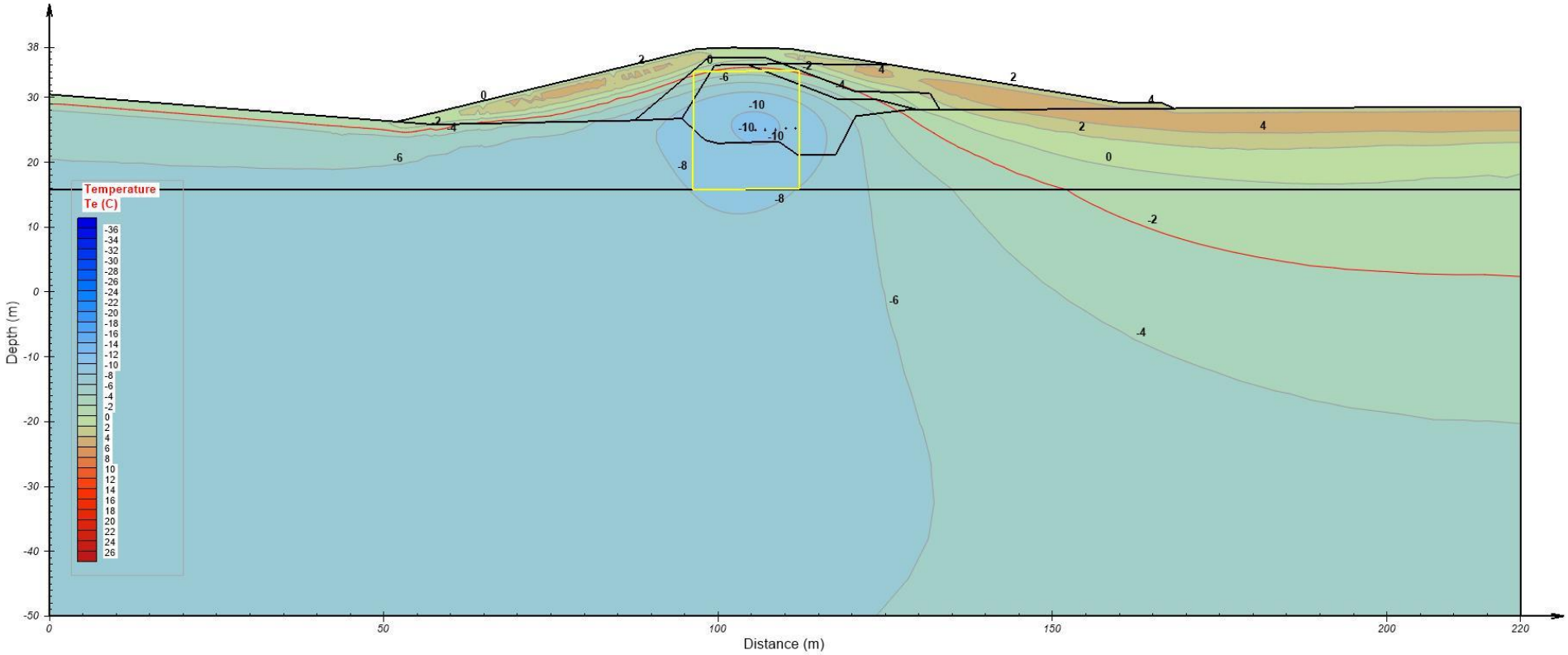
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 20 (Section 0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **35**



Notes:

1. Model results for year 30, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

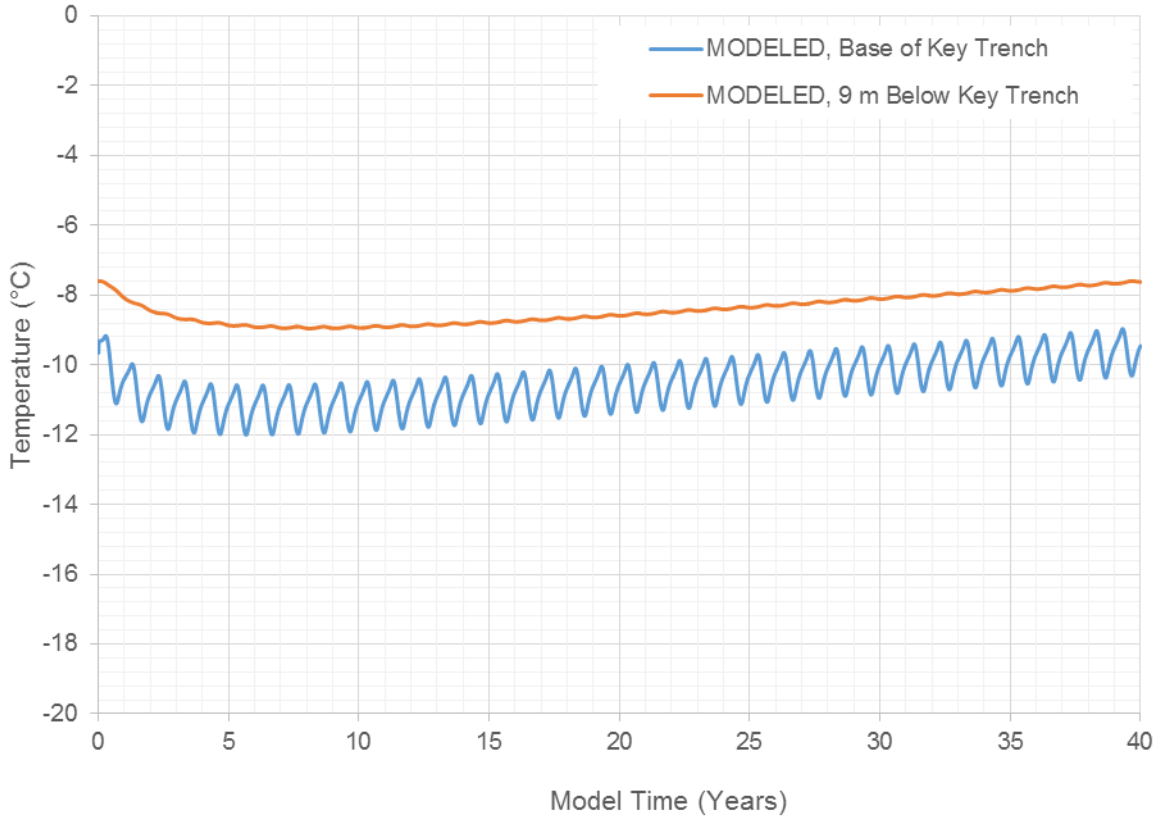
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 30 (Section 0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **36**



Notes:

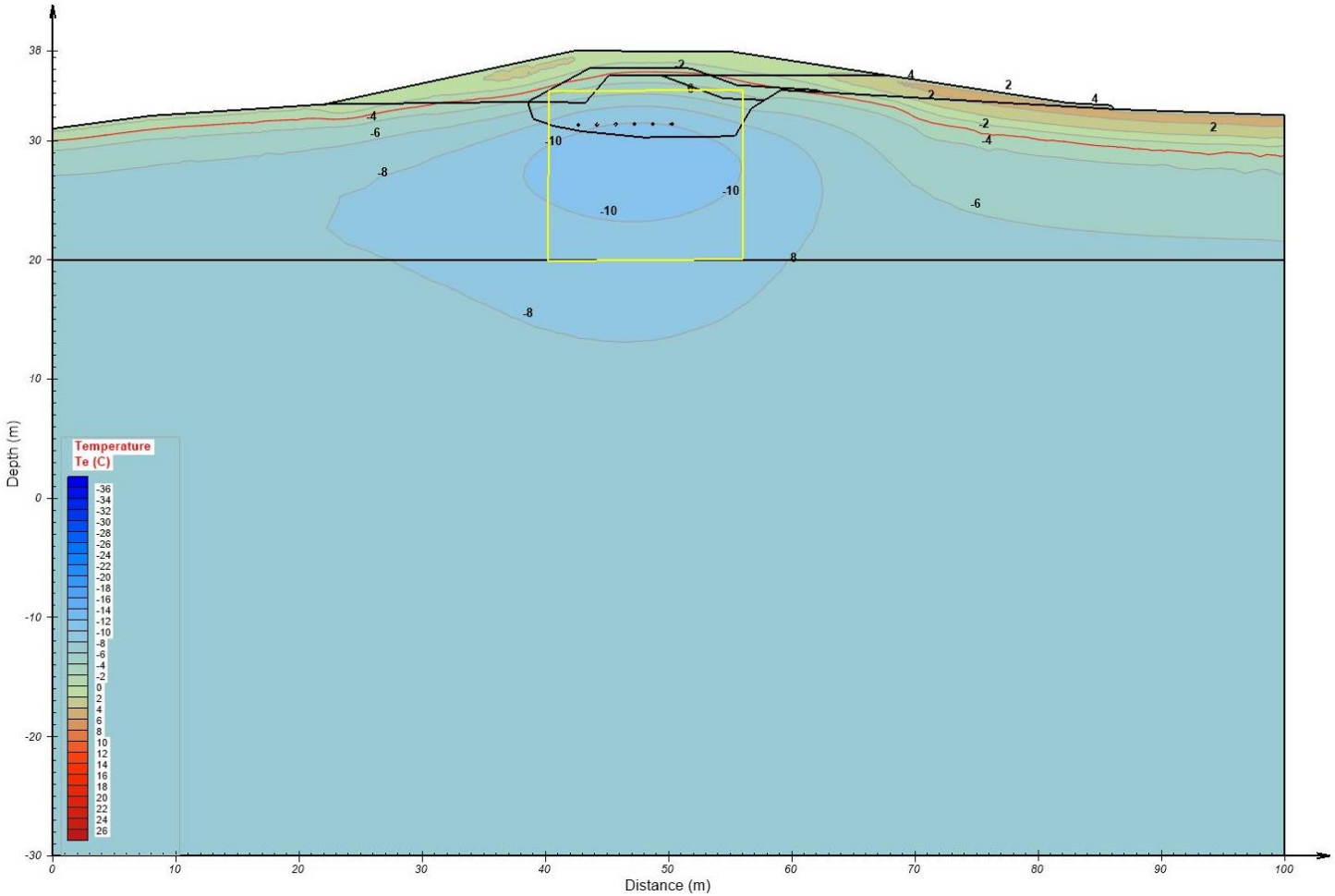
1. Model results for year 40, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – End of Design Life (Section 0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **37**



Notes:
1. Model results for base of key trench and 9 m below key trench within the foundation
2. Dam Section 0+85

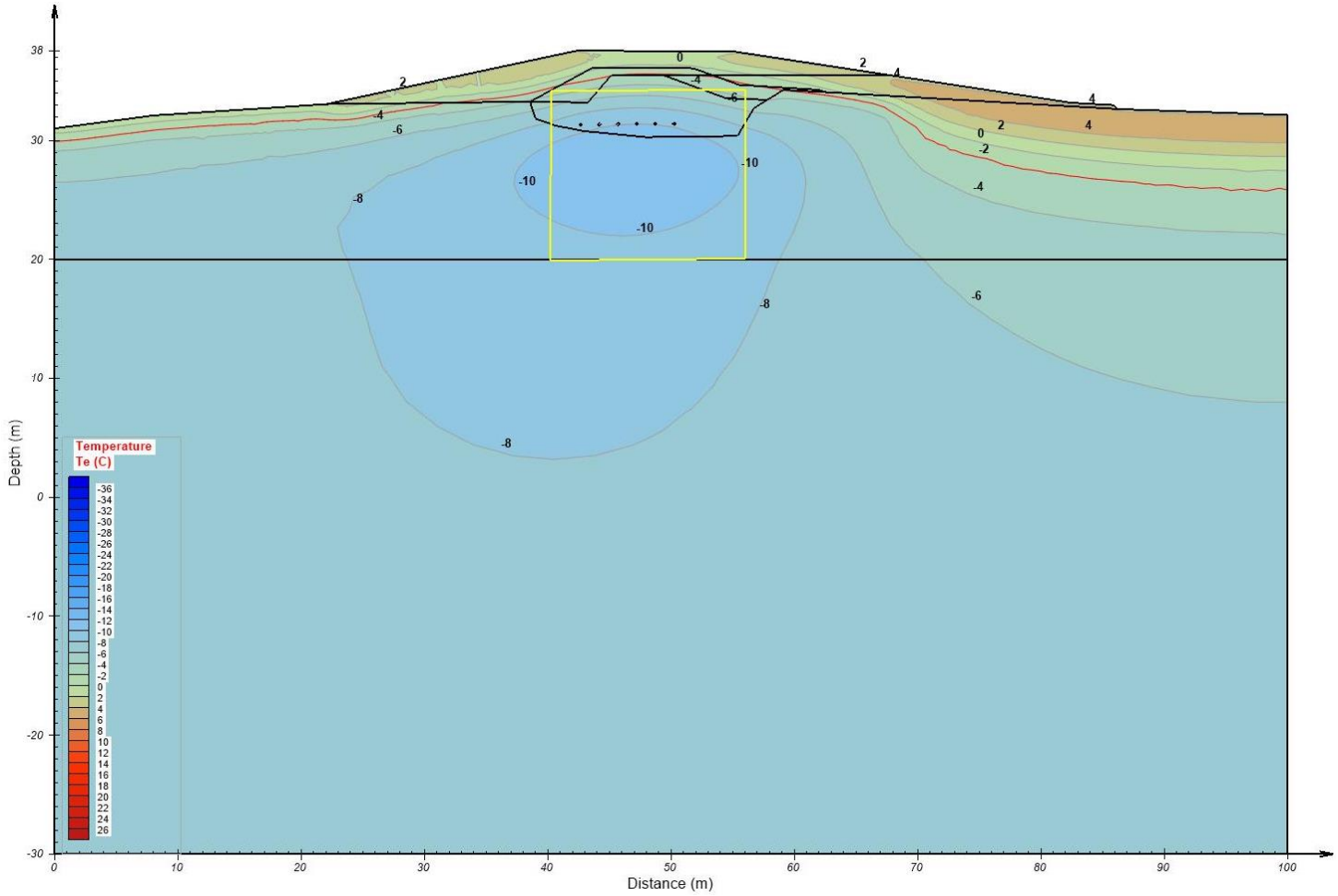
srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Design Life, Key Trench and Foundation (0+85)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **38**



Notes:

1. Model results for year 2, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

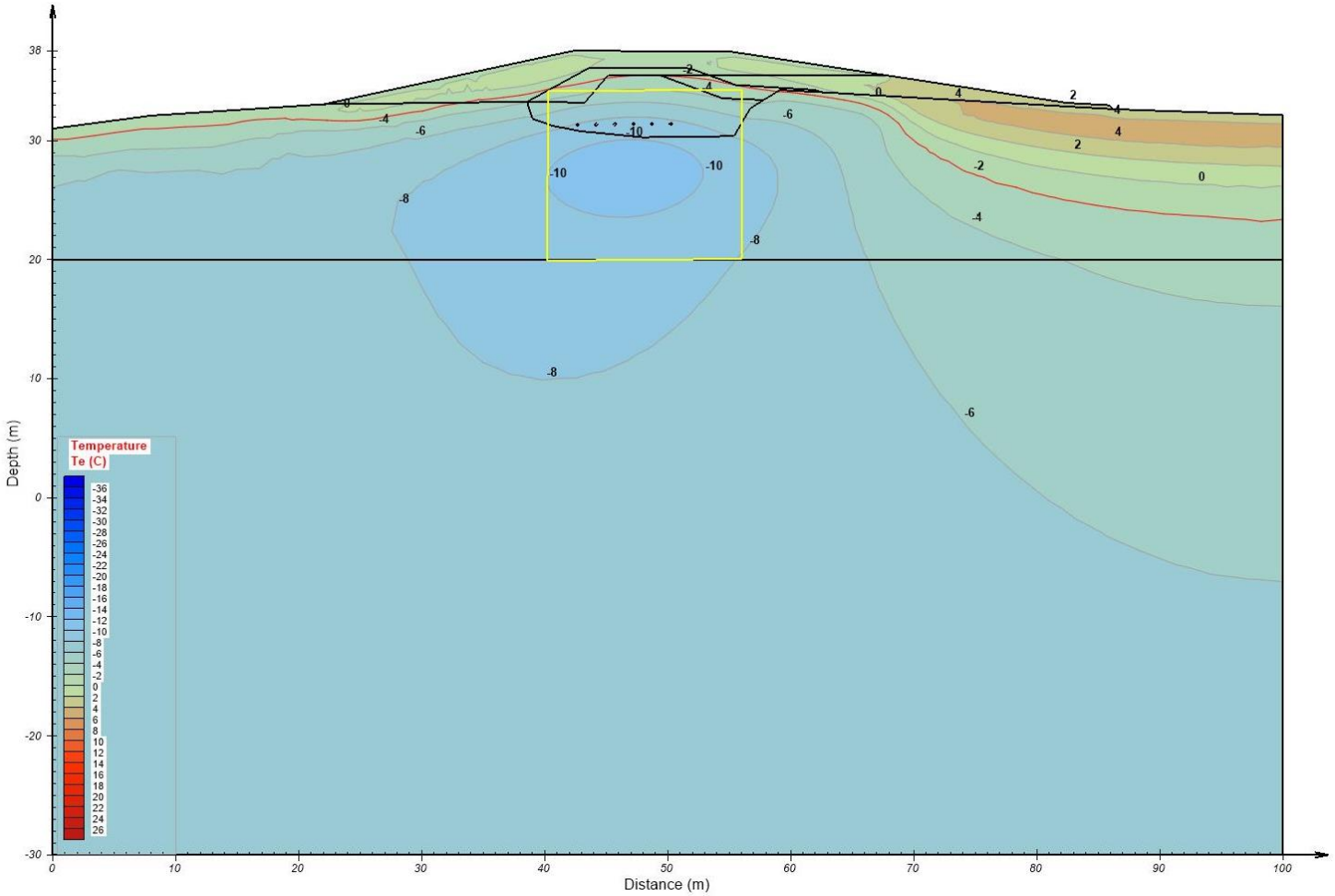
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 2 (Section 0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **39**



Notes:

1. Model results for year 10, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

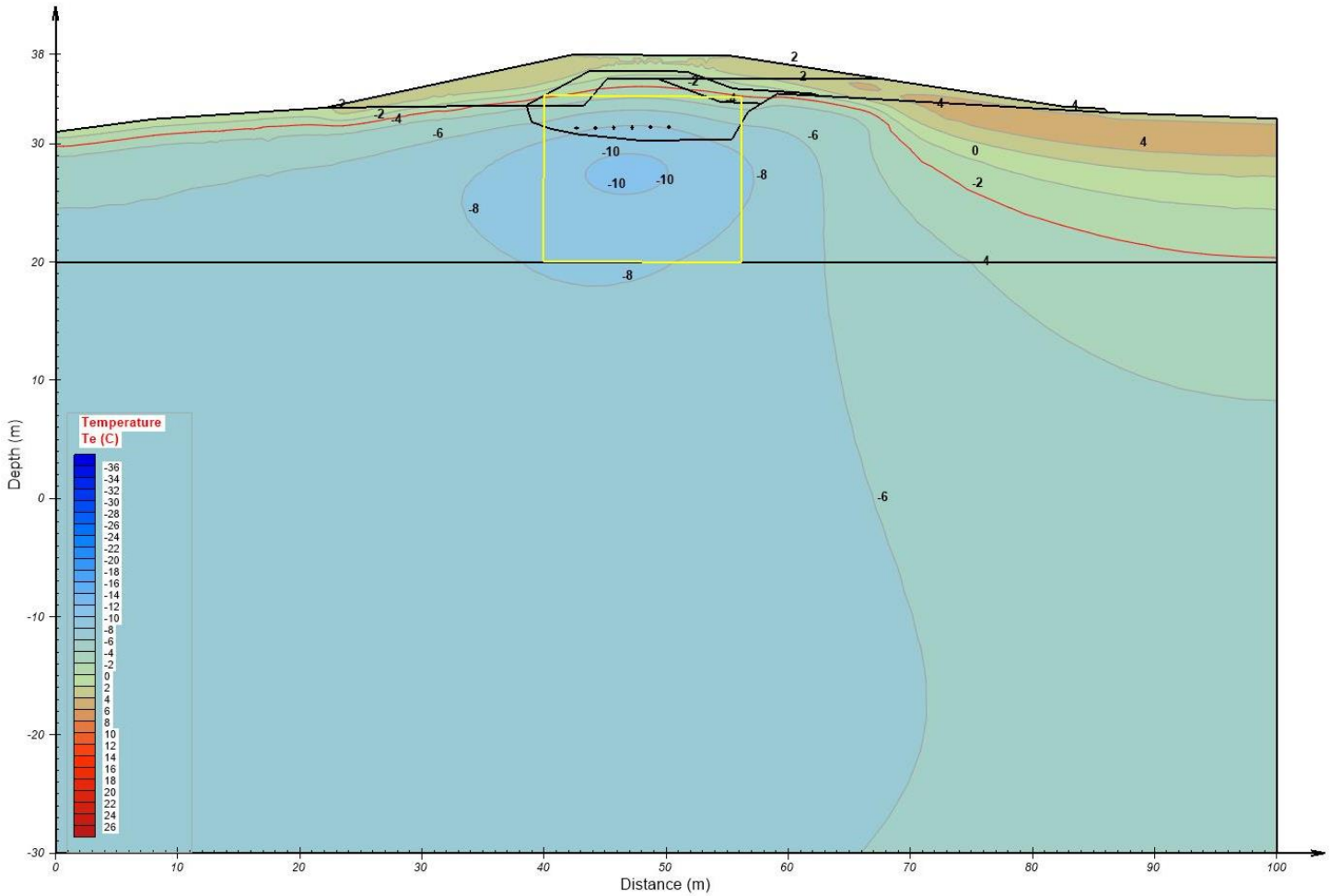
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 10 (Section 0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **40**



Notes:

1. Model results for year 20, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

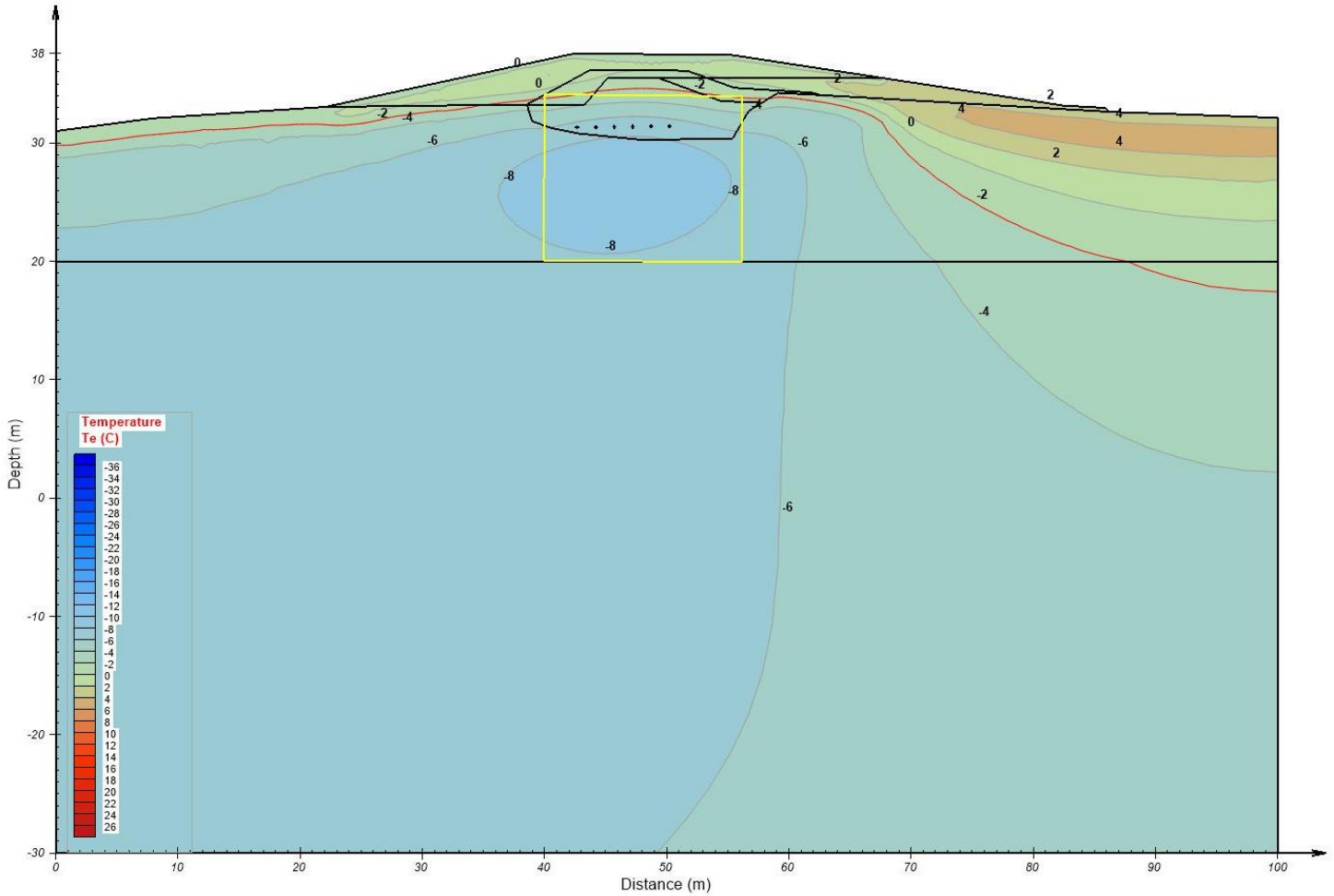
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 20 (Section 0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **41**



Notes:

1. Model results for year 30, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

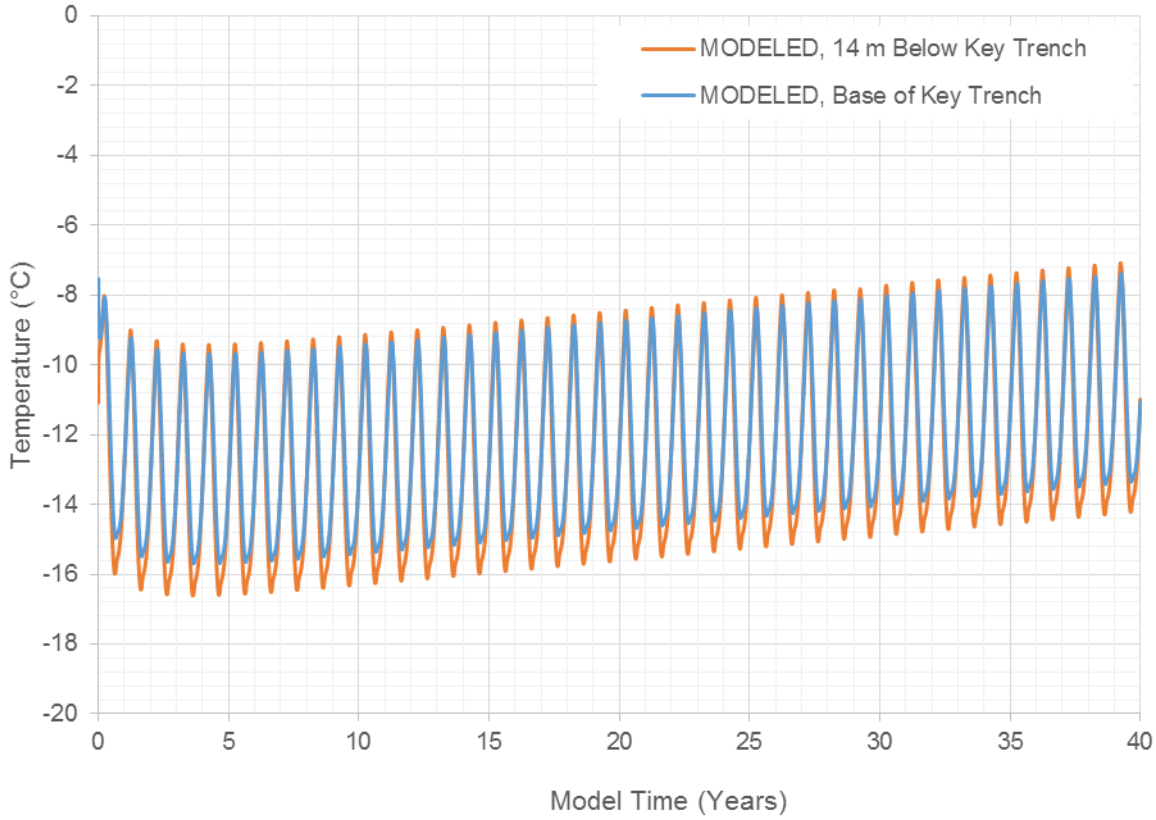
srk consulting	TMAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Year 30 (Section 0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **42**



Notes:

1. Model results for year 40, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – End of Design Life (Section 0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **43**



Notes:
1. Model results for base of key trench and 14 m below key trench within the foundation
2. Dam Section 0+40

srk consulting	MAC RESOURCES	Doris North Dam Thermal Modeling
		North Dam – Design Life, Key Trench and Foundation (0+40)
Job No: 1CT022.013 Filename: NorthDam.pptx	HOPE BAY PROJECT	Date: 8/15/2017 | Approved: cws | Figure: **44**

Appendix E – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area Creep Analysis

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Arcesio Lizcano, PhD	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Doris Tailings Impoundment Area North Dam Creep Deformation Analysis		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix E as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material change		

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases: Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2, which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris only, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Phase 1 tailings are deposited subaerially in the Doris tailings impoundment area (TIA), formerly Tail Lake, located approximately 5 km from the Doris Mill. Containment would be provided by three retention structures: a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam), and an Interim Dike situated at approximately the midpoint of the facility. Tailings would be deposited subaerially between the South Dam and Interim Dike, and the Reclaim Pond will be contained between the Interim Dike and the North Dam.

The North Dam was constructed over two winters (2011 and 2012) and has impounded water since 2011 (Figure 1). The South Dam and Interim Dike are scheduled for construction in 2017. Phase 2 tailings deposition would include a continuation of the Doris TIA with raising of the South Dam and construction of a new West Dam (SRK 2016a). Thermal analysis completed for the North Dam is reported in SRK (2016b).

The North Dam relies on a frozen ice-saturated core and foundation to achieve the required water retention properties for a water full supply level of 33.5. A geosynthetic clay liner (GCL) was installed along the upstream side of the frozen core to provide secondary water-retaining capability in case cracks develop in the core caused by thermal expansion or creep deformation. To ensure adequate performance of the dam, it is imperative to maintain the frozen state of the core and foundation over the design life.

The original design life of the North Dam was 25 years (SRK 2007). As part of Phase 2 tailings deposition, the North Dam design life would be extended to 2041 (30 years from 2011). This timeline assumes a period of nominal water impoundment prior to the start of tailings deposition in 2017, active tailings deposition between 2017 and 2032 and a one-year post-closure period prior to breaching the dam in 2033.

1.2 Objective of the Creep Deformation Analysis

The objective of the creep deformation analysis is to anticipate if long-term strains occurring over the dam design life can affect the performance or compromise the stability of the North Dam. The analysis also confirms whether the integrity of frozen ice-saturated core and underlying saline foundation will be affected by creep deformations occurring in these two zones, and if the level of the core crest remains above the full supply level (FSL) throughout the dam design life.

2 North Dam Details

2.1 As-Built Overview

2.1.1 General

The North Dam is located across the Tail Lake outlet and extends approximately 200 m long and 11 m high, with upstream and downstream slopes of 6H:1V and 4H:1V, respectively (Figure 1 through Figure 3). The dam as-built report, drawings, and quality control and quality assurance documentation is provided in SRK (2012).

2.1.2 Foundation

The overburden soils are up to 20 m thick at the base of the valley and thin out at the dam abutments. About two-thirds of the dam longitudinal section is characterized by ice-saturated sand of approximately 10 to 15 m thick. The sand deposit is overlain by a silt and clay layer less than 3 m thick. The remaining one-third portion of the dam alignment is characterized by marine clayey silt that is up to 15 m thick. The fine-grained materials are also ice-saturated and contain excess ground ice. At the North Dam, the average pore water salinity is 39 parts per thousand (ppt), with a freezing point depression of −2.2°C (geometric mean of 30 ppt and −1.7°C). The average site-wide freezing point depression is −0.8°C for sand and −2.2°C for silt and clay, with an average of −2.1°C for all samples collected at the Project site (SRK 2016c). Bedrock is generally competent basalt.

2.1.3 Dam Construction

The North Dam was constructed over the winters of 2011 and 2012 and consists of three major regions: the frozen core, transition zone, and dam shell (Figure 3).

The key trench was excavated in 2011 using drill and blast methods. A hyper-saline zone comprised of clayey silt with an average pore water salinity of 45 ppt and a freezing point of −2.6°C was encountered between key trench Station 1+00 and Station 1+20. The trench was over excavated to remove as much hypersaline material as practical. Further detail of key trench excavation, testing, and conditioning of the surface for material placement is provided in SRK (2012).

The central frozen ice-saturated core was constructed of a 2:3 blend of 20 mm minus material to fines (SRK 2012). This blend was tested on-site to obtain the moisture retention required for placement during construction. The blend material was moisture conditioned in the frozen core mixing plant using freshwater sourced from Doris Lake, with routine testing to ensure no elements in the water would affect the frozen material. A GCL was installed over the upstream side of the frozen core to function as a secondary water retention system. The crest of the frozen central core is at an elevation of 35 m.

The transition zone was constructed of 150 mm (6 inch) minus crushed material and placed over the top of the frozen core and GCL (SRK 2012). The transition material was observed to be clean with little fines and no sand and gravel.

The external dam shell (Shell) was constructed of run-of-quarry material. Finer crushed rock transition material was placed over a portion of the dam crest to serve as an access road.

2.1.4 Ground Temperature Cables

A total of 24 ground temperature cables (*aka* thermistor strings) were installed within the North Dam during construction. They monitor temperature every six hours to ensure the dam core and foundation remain within the design operating temperature. The cables include horizontal thermistor strings installed in the upper (Upper Core), middle (Middle Core), and lower (Lower Core) regions of the frozen core. Details of ground temperature cable locations in the North Dam are included in SRK (2016b).

2.1.5 Thermosyphons

Thermosyphon evaporator pipes located at the base of the key trench provide passive cooling during the winter to ensure that the core and foundation remain frozen throughout the year.

Thermosyphons are pressurized sealed pipes, charged with a two-phase working gas that vaporizes and condenses to move heat without the need of a mechanical pump. A typical passive thermosyphon consists of an evaporator pipe buried in the ground and radiator exposed at the surface. The radiator section is manufactured with fins attached to the radiator pipe to enhance heat transfer with the atmosphere.

North Dam thermosyphons were procured and installed by Arctic Foundations of Canada Inc. Thermosyphon installation included one series of six evaporator pipes installed from the north end of the key trench and another six installed from the south end. The evaporator pipes extend to Section 0+85 which is the lowest point of the key trench. The north and south evaporator pipes are sloped at 4.6° and 8°, respectively.

Two thermosyphon radiators were attached to each evaporator pipe, with a total radiator surface area of 39 m^2. The North and South radiators are exposed at the surface and unobstructed by surface infrastructure to allow for effective heat loss from the radiator (Figure 1). Each pipe is charged with a carbon dioxide working gas and considered to have similar performance. General function of the thermosyphons is assessed in the winter by comparing the temperature differential between the air and the evaporator pipe directly below the ground surface.

2.1.6 Survey Monitoring

A series of 14 crest survey monitoring points, 3 deep settlement points, and 18 surface survey points were installed in the North Dam upon completion (Figure 4). These monitoring points were installed to monitor any surface movement of the dam crest, downstream face, and deep settlement of the downstream foundation of the dam.

2.2 Creep Deformation Evaluation Criteria

Creep deformation evaluation criteria for the frozen core and foundation establish limits to insure long-term integrity of the frozen core and foundation. The criteria guarantee long-term strains occur slowly and in a ductile manner. The criteria are based on the original design criteria proposed by EBA (2006) and require:

- The frozen core maintains the long-term shear strains at or below 2% and the maximum shear strain rate at or below 1.0E-05 sec^{-1} (3.2E+02 $year^{-1}$).
- The frozen foundation underneath the core maintain the long-term shear strain at or below 10% and the maximum shear strain rate at or below 1.0E-05 sec^{-1} (3.2E+02 $year^{-1}$).

2.3 Current Conditions

Annual inspections and review of monitoring data suggest the dam is performing in accordance with the design expectations. The North Dam has had impounded water since the first winter of construction in 2011. The operating water level impounded against the upstream face of the dam has averaged 29.0 m, with a maximum level of 29.5 m over the period from September 2011 to September 2015. The original water level of Tail Lake prior to construction of the North Dam was 28.3 m. Core and foundation temperatures have been below the design temperatures of −2ºC and −8°C, respectively.

Annual review of displacement monitoring data collected at the North Dam indicates (SRK 2016e):

- The maximum vertical displacement at the crest is 0.07 m measured at ND-SMP-120-DS, with a change of 0.02 m in the last three years.
- The maximum horizontal displacement measured at the crest is 0.10 m measured at the ND-SMP-120-US, with a change of 0.06 m in the last three years.
- The maximum vertical displacement measured at the downstream face of the dam is 0.21 m measured at ND-SSP-065-2, with a change of 0.01 m in the last three years.
- The maximum horizontal displacement measured at the downstream face of the dam is 0.20 m at ND-SSP-110-3, with a change of 0.07 m in the last three years.

3 Creep Deformations Analysis

3.1 Model Setup

Creep deformations were assessed by plane strain conditions using the two-dimensional non-linear finite difference code, Fast Lagrangian Analysis of Continua (FLAC 2-D), by Itasca (2012). The analysis was carried out along the cross sections 0+85 located at the thickest section of the dam (Figure 2). Thermal modelling was completed for the same cross section (SRK 2016b).

As-built survey information was used for the 2-D model sections of the dam. Section 0+85 was modelled with a 14 m wide crest, 11 m height, upstream slope of 6H:1V, and downstream slope of 4H:1V. Five material regions were considered in the model: shell, transition zone, core, foundation, and bedrock. The 5 mm GCL liner was not represented in the model, which is presented in Figure 5.

3.2 Basis for the Assessment

The ice-saturated granular material in the frozen core is a dense material with a void ratio around 0.35 (SRK 2016b). The deviatoric strength (peak or residual) of the frozen core material is expected to be above 1 MPa based on published results from laboratory tests conducted on frozen sand samples under constant temperatures around −5°C and constant strain rates around 1E-07 sec^{-1} (Bragg and Andersland 1981 and Arenson 2002). Considering the height of the North Dam (11 m), the level of deviatoric stresses within the frozen core is anticipated to be low relative to the expected deviatoric strength of the frozen material in the core. According to Andersland and Landanyi (2004), medium to high-density ice-saturated sands under low stress levels exhibit only primary creep (i.e., decreasing strain rates). Therefore, the creep deformation analysis assumed the frozen core will exhibit only primary creep.

Secondary creep (i.e., constant creep strain rate) were assumed for the frozen marine clayey silt in the foundation. This type of soils exhibits a short primary-creep period and a prolonged secondary-creep phase (Andersland and Landanyi 2004).

Based on the Bailey-Norton law (Norton 1929 and Bailey 1935), creep strains rates ($\dot{\varepsilon}$) of frozen soils due to the deviatoric part of the stresses ($\bar{\sigma}$) can be described by the following general equation:

$$\dot{\varepsilon} = (A\bar{\sigma}^n) \cdot mt^{m-1} \quad (1)$$

where A is a creep parameter that depends on soil type and temperature, n and m can be considered temperature independent parameters, and t is the elapsed time after load application.

Secondary creep is commonly described by Equation (1) with $m = 1$. In this case, the equation can be rewritten as

$$\dot{\varepsilon} = A\bar{\sigma}^n \quad (2)$$

With Equation (2), frozen soils are always predicted to creep for any given deviatoric stress. Even for very small stresses, frozen soil will be predicted to creep. This may lead to overestimating actual long-term displacements. A threshold stress (σ_{th}) for frozen soils likely exists, as for metals (Norton 1929), below which creep cannot be measured and Equation (2) no longer applies. Equation (2), as most constitutive equations for creep, is however formulated without a threshold stress.

In the performed analysis, creep strains were evaluated using a constitutive relation represented by Equation (2) implemented in FLAC, described as "The Two-Components Power Law" (Itasca 2012). For the analysis, a temperature independent threshold stress of 30 kPa was selected for all frozen materials based on published laboratory testing results (Landanyi 1971, Nixon and Lem 1984, Wijeweera and Joshi 1991, and Arenson, 2002) and engineering judgment. No creep strains were predicted ($\dot{\varepsilon} = 0$) for $\bar{\sigma} < \sigma_{th} = 30$ kPa. This stress is considered to be low relative to the expected peak deviatoric strength. The assumed stress was not a threshold for the deviatoric part of the stresses as introduced by Norton (1929). In this latter case, the deviatoric part of the stresses ($\bar{\sigma}$) in Equation (2) is reduced by σ_{th}, or $\dot{\varepsilon} = (\bar{\sigma} - \sigma_{th})^n$. Likely thresholds for other creep mechanisms in frozen soil (e.g., temperature) were not considered in the analysis.

Equation (2) can therefore be written as follows:

$$\frac{\dot{\varepsilon}}{\dot{\varepsilon}_r} = \left(\frac{\bar{\sigma}}{\sigma_r}\right)^n \tag{3}$$

where $\dot{\varepsilon}_r$ and σ_r are reference values for the strain rate and stress. According to Equation (3), the creep parameter A in Equation (2) is:

$$A = \frac{\dot{\varepsilon}_r}{(\sigma_r)^n} \tag{4}$$

Based on the experimental work from Nixon and Lem (1984) on saline fine grained frozen soils, Andersland and Landanyi (2004) proposed the following empirical expression for σ_r in kPa as a function of temperature and salinity:

$$\sigma_r = 0.323(1 - T)^2 \left(\frac{59.505 - S}{8.425 + S}\right) \tag{5}$$

where T is the temperature in Celsius degrees and S is the salinity in ppt.

The parameter A ($\mathrm{kPa}^{-n} \cdot \mathrm{year}^{-1}$) can be then calculated with Equation (4) as a function of temperature and salinity using the Equation (5) for σ_r and a reference strain rate of $\dot{\varepsilon}_r = 10^{-4}\ \mathrm{year}^{-1}$ (Anderson and Landanyi 2004). For the analysis, the parameter A was determined with Equation (4) at different temperatures for the reported average salinity of 39 ppt (Section 3.4.3).

3.3 Methodology

The creep analysis used in the ground thermal conditions were predicted by thermal modelling at cross section 0+85 (SRK 2016b). It is expected that the creep behavior of the frozen core and foundation changes as the temperature changes over the dam design life. An accurate prediction of long-term creep deformations therefore requires a thermomechanical coupled constitutive model. However, an efficiently implemented coupled thermo-mechanical model is not available in commercial codes. Hence, long-term creep behavior was evaluated for the ground temperature distribution predicted ten years after dam construction (Figure 6). This time interval is considered as representative for the long-term creep deformation in the North Dam.

The analysis followed the following steps:

1. Initial state: The initial stresses of the dam embankment and foundation was achieved in the model by using elastic properties for all materials and turning gravity on.
2. Elasto-plastic phase: Dam shell, transition zone, and foundation zone over the isotherm −2°C (thawed clayey silt) predicted by the thermal modelling (Figure 6) were changed from elastic to Mohr-Coulomb materials, and the model was brought again to equilibrium.
3. Creep phase: Temperature dependent elastic and creep properties were assigned to the frozen core and foundation based on the predicted temperature ten years after dam construction. The model was allowed to deform for 30 years.

3.4 Material Properties

Elastic and creep material properties from laboratory tests are not available. Elastic and creep properties used in the deformation analysis were estimated based on previous reports (e.g., in EBA (2006)), published data in the literature, and engineering judgment.

3.4.1 Elastic Properties

Elastic properties for the initial state were taken from EBA (2006). SRK adjusted the elastic modulus of the frozen foundation. Table 1 presents the material elastic properties used for achieving the initial state in the model.

Table 1: Elastic Properties for the Initial State[1]

Model Region	**Material**	**Unit Weight** (kN/m^3)	**Elastic Modulus** (kPa)	**Poisson's Ratio** (-)
Shell	Run-of-Quarry	22	1.0E+05	0.35
Transition	150 mm minus	21	1.0E+05	0.30
Core	20 mm minus: 5 mm minus (2:3 blend by volume)	22	1.0E+05	0.25
Foundation	Clayey Silt	17	6.4E+05[2]	0.35
Bedrock	Basalt	26	1.0E+08	0.25

Notes:
1. Source: EBA 2006
2. Adjusted by SRK. EBA Elastic Modulus: 5×10^{-4} kPa

3.4.2 Shear Strength Properties

Shear strength properties were taken from EBA (2006). SRK believes these Mohr-Coulomb properties are suitable for the elasto-plastic phase of the analysis. Table 2 includes the shear strength properties.

Table 2: Shear Strength Properties

Model Region	Cohesion (kN/m^2)	Friction Angle (º)
Shell	-	40
Transition	-	35
Foundation (Thawed Clayey Silt)	40	-
Bedrock	1000	-

3.4.3 Creep Parameters

Table 3 summarizes the parameter used for the creep phase of the analysis. Parameters n, m, and A (Equation (1)) for the frozen core were estimated based on the laboratory testing results from Ottawa sand (Sayles 1968) and an average temperature of −9˚C in the core ten years after dam construction. For the frozen foundation, n, (Equation (2)) was estimated based on published laboratory testing results from saline fine grained soils (Nixon and Lem 1984 and Wijeweera and Joshi 1993). Temperature dependent A values for the frozen foundation were calculated with equations (3) and (4) for a constant salinity of 39 ppt. For reference, Figure 7 plotted equations (3) and (4) for different temperatures and salinities. The figure includes values from Nixon and Lem (1984) for a salinity of 35 ppt and those used by EBA (2006) for a salinity of 45 ppt.

Table 3 includes the estimated temperature dependent elastic moduli of the frozen core and foundation required for the elastic strains. Since the creep is considered to be a constant volume process, the analysis used a Poisson's ratio of 0.5 for the frozen core and foundation.

Table 3: Creep and Elastic Properties of the Frozen Core and Foundation[1]

Model Region	***b*** (-)	***n*** (-)	A	**Elastic Modulus** (kPa)
	(-)	(-)	$\mathrm{kPa}^{-n}\mathrm{year}^{-b}$	(kPa)
Core (-9ºC)	0.26	1.32	2.0E-07	3.0E+05
Foundation[2]				
-3ºC	1	3	9.6E-05	1.0E+04
-4ºC	1	3	2.5E-05	3.2E+04
-5ºC	1	3	8.4E-06	6.6E+04
-6ºC	1	3	3.3E-06	1.4E+05
-7ºC	1	3	1.5E-06	2.8E+05
-8ºC	1	3	7.4E-07	5.7E+05
-9ºC	1	3	3.9E-07	1.2E+06
-10ºC	1	3	2.2E-07	2.4E+06

Notes:
1. Constant volume deformation; Poisson's ratio $\nu = 0.5$
2. Salinity 35 ppt

4 Results

Table 4 summarizes the predicted creep strains and stresses in the frozen core and underlying frozen foundation 10 and 30 years after dam completion. Table 5 presents the predicted displacements for the same time intervals.

Table 4: Creep Strains and Stresses in the Frozen Core and Foundation

	Maximum Shear Strain Rate (Fig. 8 and 9)		**Maximum Shear Strain** (Figs. 10 and 11)		**Maximum Deviatoric Stress** (Figs. 12 and 13)		**Shear Stress** (Figs. 15 and 16)	
	(year^{-1})		(m/m)		(kPa)		(kPa)	
	Core[1]	Foundation[2]	Core[1]	Foundation[2]	Core[1]	Foundation[2]	Core[1]	Foundation[2]
10 years after dam construction	5.0E-08	1.0E-07	5.0E-02	~1.0E-01	100[3] (300)[4] (400)[5]	50	20 (50)[5]	20
30 years after dam construction	2.0E-08	4.0E-08	1.0E-01	~2.0E-02	50[3] (300)[4] (450)[5]	50	20 (70)[5]	20

Notes:
1. Within the frozen core
2. Underneath the frozen core
3. At the top of the frozen core
4. At the bottom of the frozen core
5. Localized at the upstream lower corner of the core

Table 5: Creep Displacements in the Frozen Core and Foundation

	Maximum Horizontal Displacement (Fig. 18 and 19)		**Maximum Vertical Displacement** (Figs. 20 and 21)		**Vertical Displacement** (Fig. 22)
	(m)		(m)		(m)
	Core[1]	Foundation[2]	Core[1]	Foundation[2]	Core Crest
10 years after dam construction	0.4	0.4	0.6	0.6	0.24
30 years after dam construction	0.8	0.6	1.0	1.0	1.0

Notes:
1. Within the frozen core
2. Underneath the frozen core

4.1 Shear Strain Rates and Shear Strains

The analysis predicts shear strain localization at the downstream side of the dam mainly (Figures 9 to 11). The shear localization zone is almost circular and goes along the transition zone and through the saline frozen foundation. This surface can be considered as a likely failure surface in the event that the material strength is mobilized along this surface.

In general, the predicted shear strain rates are very low in all zones of the dam and foundation compared with strain rates usually used in laboratory tests with frozen soils (Sayles 1968, Wijeweera and Joshi 1991 and Arenson 2002). The maximum shear strain rates are 3.5E-07 $year^{-1}$ and 1.E-07 $year^{-1}$, 10 and 30 years after dam completion, respectively (Figures 8 and 9). The maximum shear strains are 4.0E-01 m/m (40%) and 6.0E-01 m/m (60%) for the same periods of time (Figures 10 and 11). Maximum shear strain rates and shear strains are predicted to occur in points within the shear localization zone (i.e., outside the frozen core and underlying foundation).

In the frozen sandy core and underlying foundation (Table 4), the maximum rate of shear strain meets the design criteria for ductile material behavior (Section 2.2), while the shear strains *themselves* exceed the criteria. However, for the frozen sandy core, ductile material behavior is expected because the maximum rate of shear strain is predicted to be very low (~ 1E-08 $year^{-1}$). Based on Bragg and Andersland (1981) and Arenson (2002), a brittle mode of failure can be excluded in frozen sands that deform under a shear strain rate below < 1E-05 sec^{-1} (3.2E+02 $year^{-1}$). Nevertheless, the lower the strain rate, the lower the strength of the frozen material.

4.2 Principal stresses Difference

Creep strain rates were evaluated as a response to induced deviatoric stresses by the dam weight. Maximum principal stresses differences of around 75 and 300 kPa are predicted to be almost constant at the crest and bottom of the frozen core, respectively, throughout the dam design life (Table 4). In the frozen saline foundation, underneath the frozen core, an almost constant principal stress difference below 50 kPa is predicted over the design life (Figures 12 and 13). Figure 14 presents histories of principal stresses difference for two points located at the crest and within the low zone of the frozen core (Points A and B, Figure 5).

The predicted stress differences at the bottom of the frozen core can be considered as intermediate compared with the expected peak deviatoric stress. In the remaining areas of the frozen core and underlying foundation, low stresses difference will prevail.

4.3 Shear stresses

In general, the shear stresses in the frozen core and foundation are predicted to be relatively low over the design life of the dam compared with the expected shear strengths of these materials (Figures 15 and 16). Figure 17 shows shear stress histories for two points located at the core crest and within the low zone of the frozen core (Points A and B, Figure 5).

4.4 Displacements

Based on the thermal modelling, a greater frozen area is predicted in the foundation at the downstream side than at the upstream side (Figure 6 and SRK 2016b). Therefore, the maximum creep displacements due to the frozen foundation will be expected at the downstream side of the dam.

Figure 18 and 19 show the distributions of horizontal creep displacements. Maximum horizontal displacements of 2.8 and 4.2 m are predicted to occur 10 and 30 years after dam construction, respectively, in a small zone of the foundation at the downstream side. Within and underneath the frozen core, the horizontal displacements remains under 0.8 m over the design life of the dam (Table 5).

The distribution of the vertical displacements 10 and 30 years after dam construction are presented in Figures 20 and 21. Figure 22 shows the vertical displacement history of a point located on the frozen core crest determined for a threshold stress of 30 kPa and pore water salinity of 30 ppt. At the end of the planned design life, the core crest is predicted to settle around 1.0 m.

As a reference for the predicted vertical displacements, Figure 22 includes the maximum measured vertical displacements. These displacements were measured at the downstream part of the dam crest in station 1+20 (ND-SMP-120-DS in Figure 4) over the first four years after dam completion.

To show the impact of threshold stresses and salinities on the creep deformations analysis, Figure 22 also includes predictions of core crest settlements obtained with thresholds stresses of 0 and 35 kPa, and salinities of 39 and 35 ppt. The lower the threshold stress and higher the salinity, the greater the core crest settlement will be. For a salinity of 39 ppt and without threshold stress ($\sigma_{th} = 0$ kPa), the core crest is predicted to settle around 1.4 m at the end of the dam design life, i.e., the core crest will settle up to the water full supply level of 33.5 m. However, this result is considered an overestimation of vertical displacements for the reasons outlined in Section 3.2.

5 Conclusions

Main conclusions from the creep deformation assessment are as follows:

- A long-term ductile behavior is predicted for the materials in the ice-saturated frozen core and underlying frozen foundation. Creep shear strains in these zones will occur very slowly and remain below the strain rate for brittle failure modes.

- Vertical creep displacements will not compromise the long-term integrity of the frozen core and underlying foundation. Shear and deviatoric stresses in these zones caused by creep strains will remain well below the expected peak strengths of the materials. No shear strain localization is predicted within and underneath the frozen core.

- Long-term performance of the frozen core is not expected to be compromised throughout the dam design life. Thirty years after dam construction, the total settlement of the core will be around 1.0 m, i.e., 0.5 m above the full supply level (33.5).

- Shear strains are predicted to localize at the downstream side of the dam. Thirty years after dam completion, high shear strains (~ 60%) with very low strain rates (~1.0E-08 year^{-1}) can be expected in few points within the localization surface. However, shear stresses and principal stress differences will remain well below the expected peak deviatoric stress of the materials (> 1 MPa).

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

6 References

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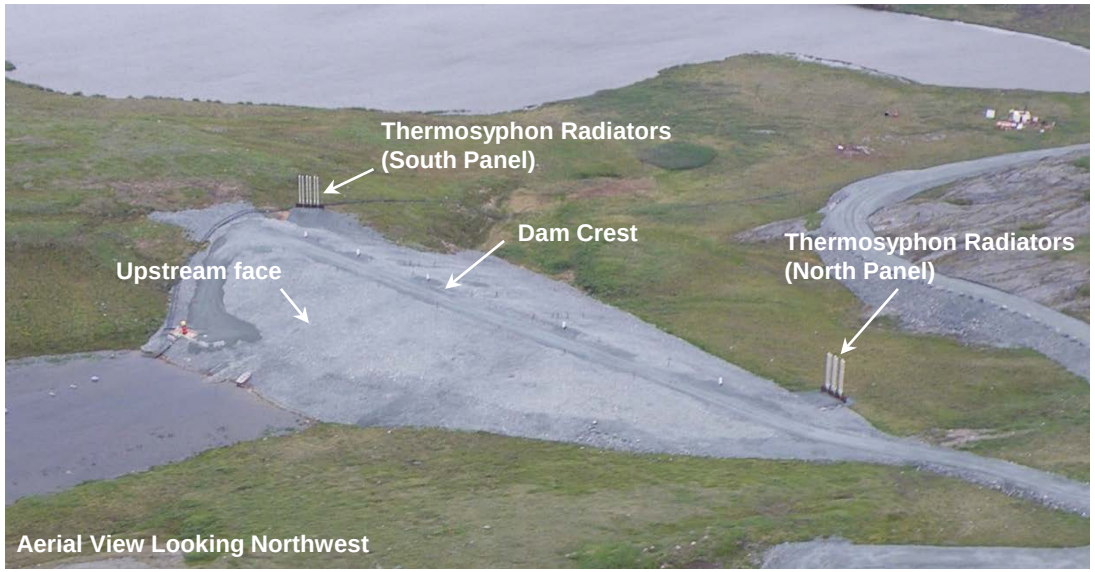
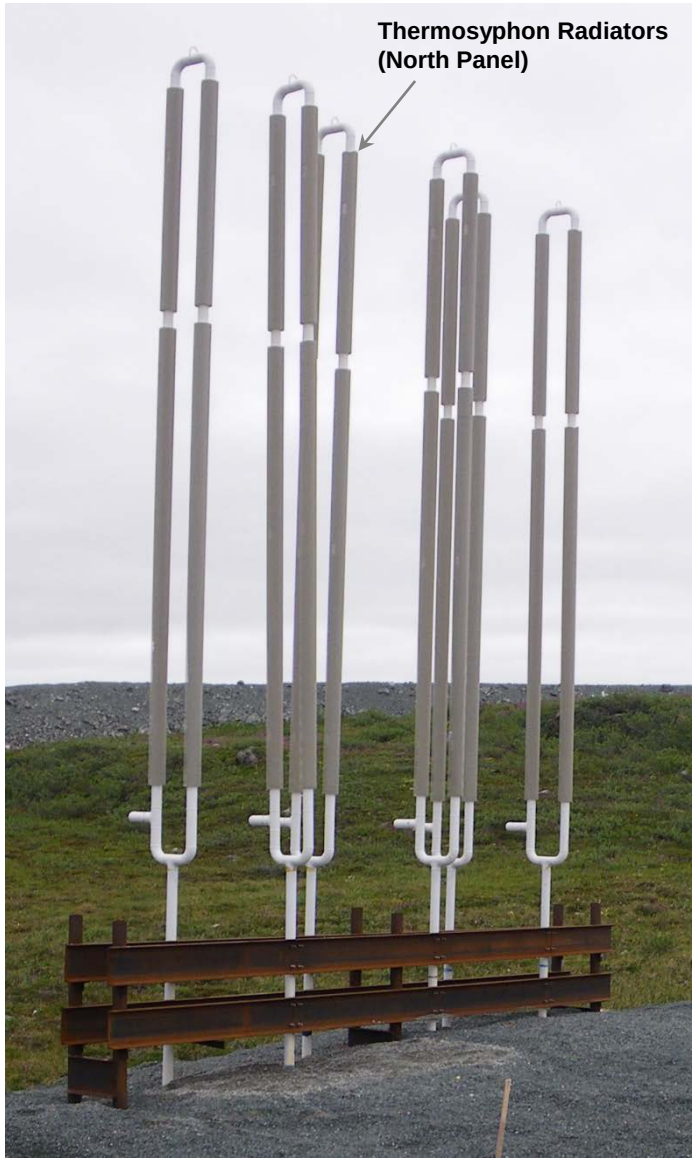
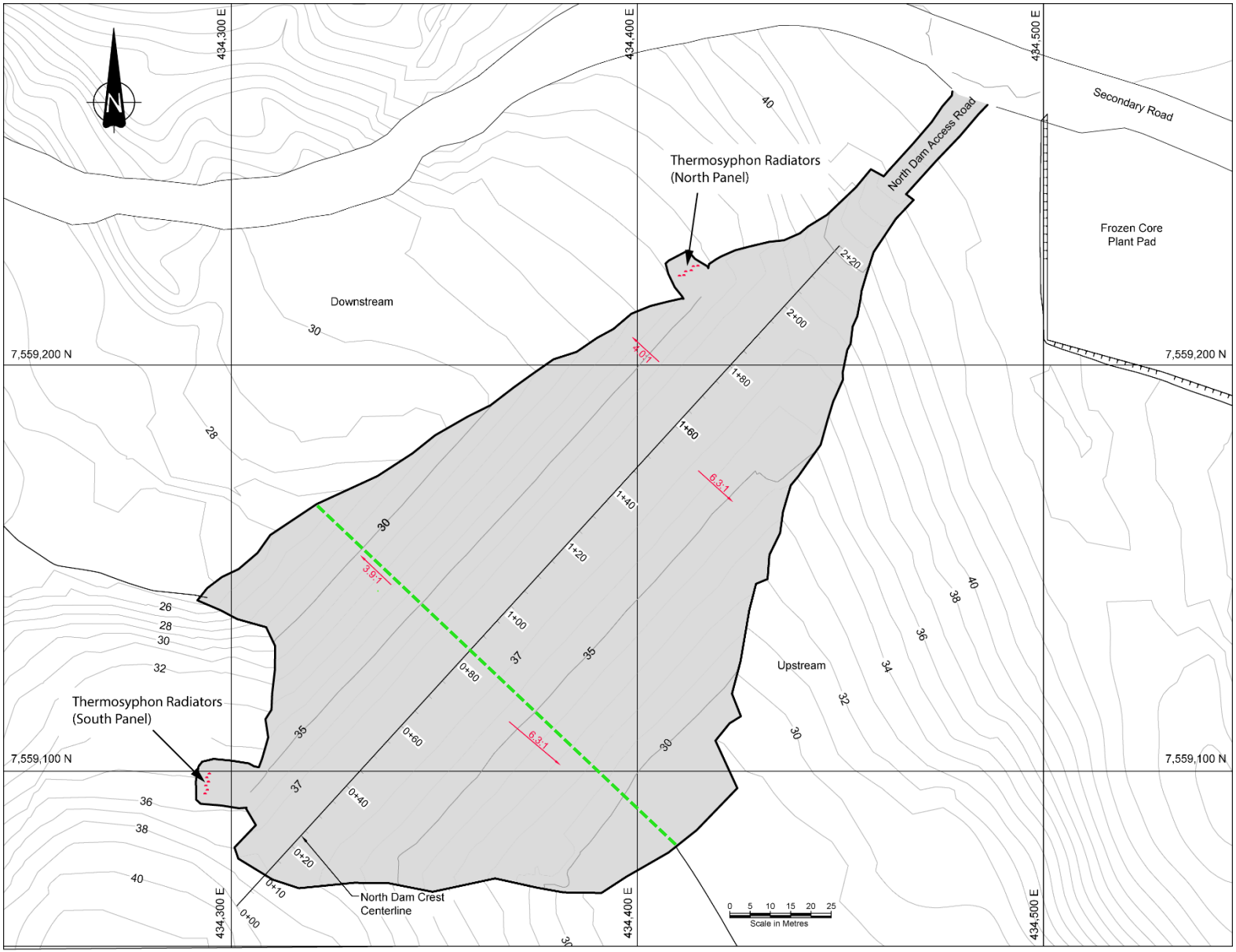
Figures





Notes:
1. Field photographs taken July 18th and July19th of 2014

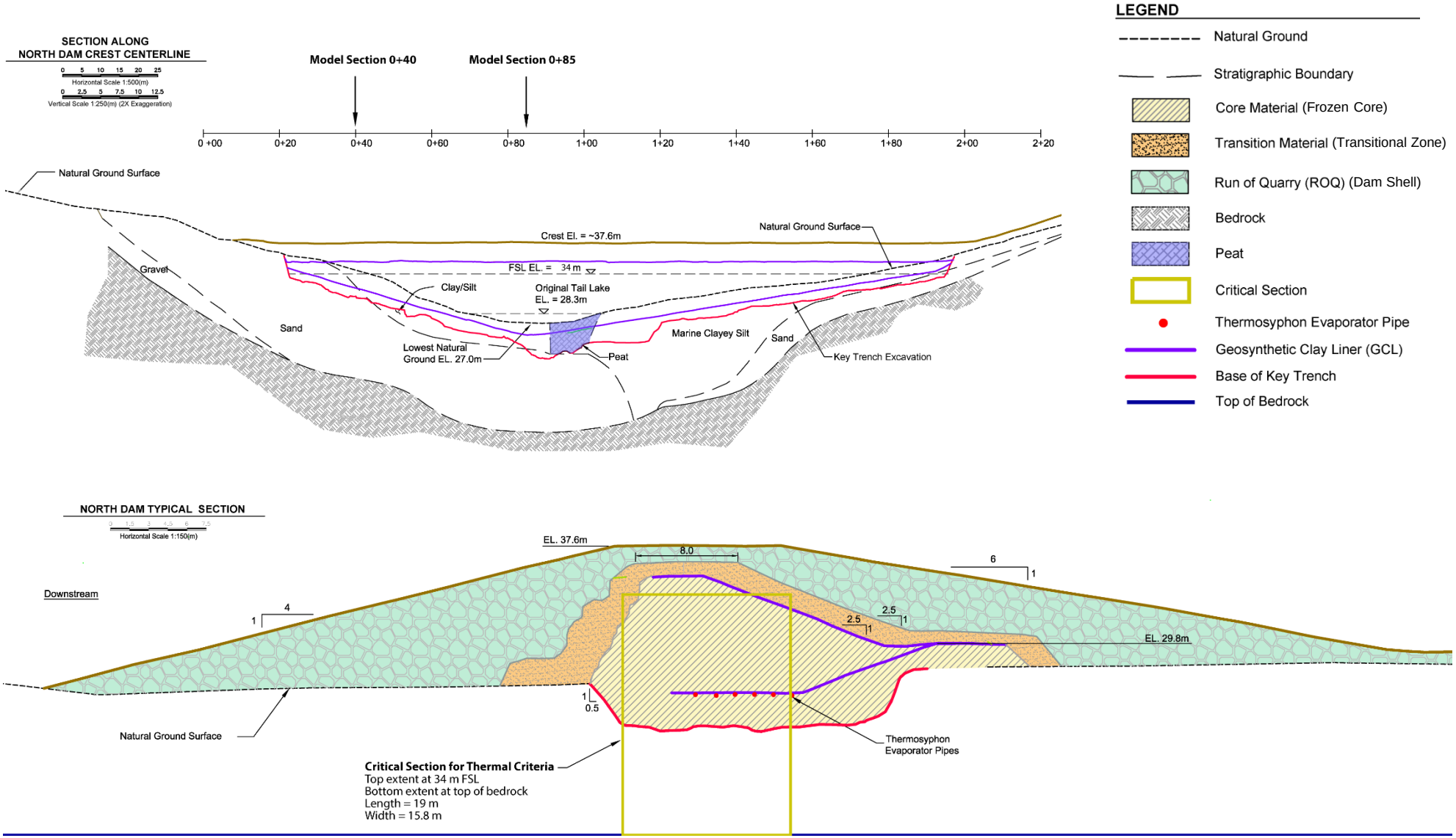
srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**North Dam – Field Photograph**
		Date: 8/17/2016 | Approved: AL | Figure: **1**



Notes:

1. Dashed green line indicates Dam Section 0+85

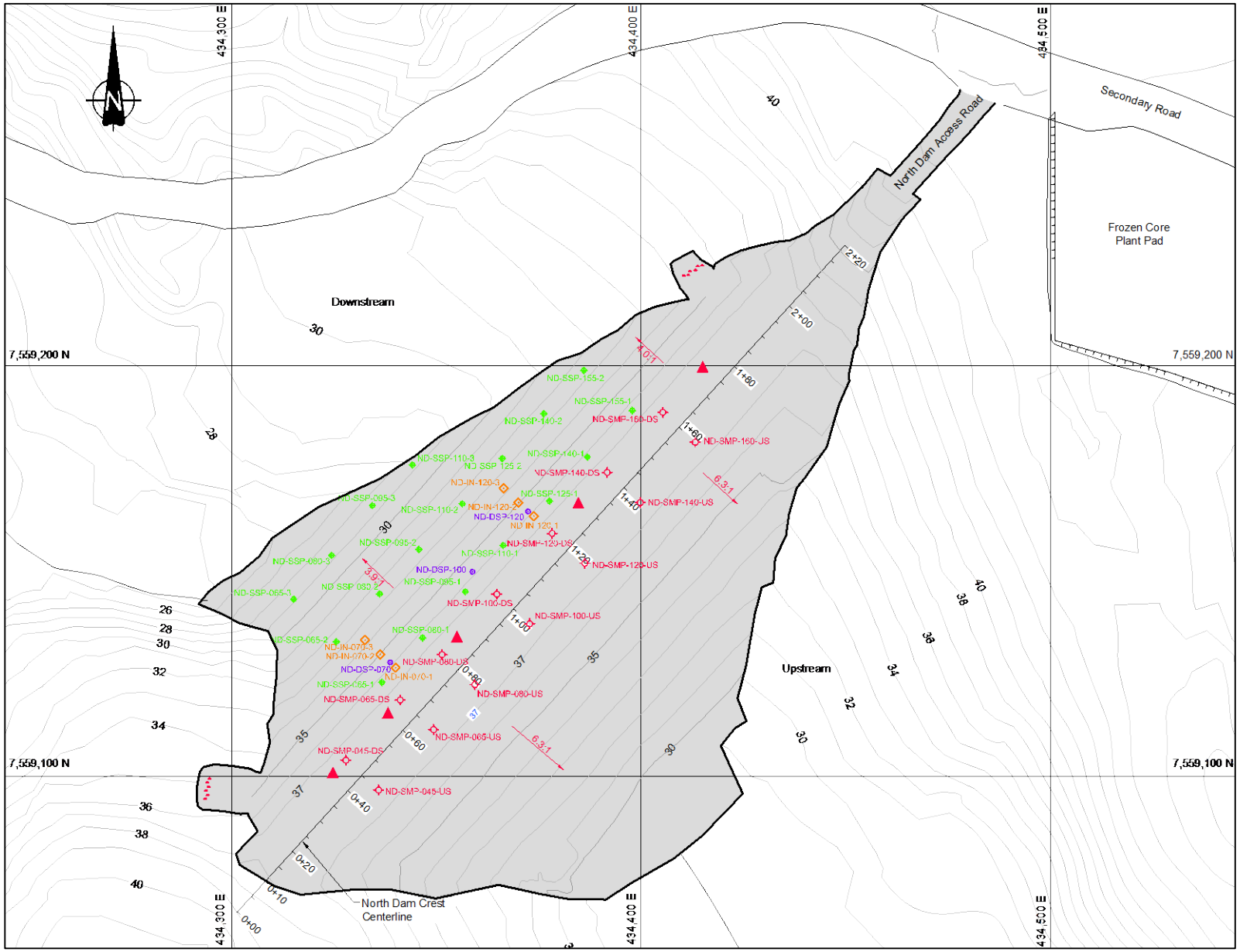
srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**North Dam General Arrangement**
		Date: 8/17/2016 | Approved: AL | Figure: **2**



Notes:

1. The subsurface geology has been extrapolated from a series of geotechnical investigations and geological unit contacts are therefore likely to vary somewhat.

srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**North Dam** **Typical As-Built Section**
		Date: 8/17/2016 | Approved: AL | Figure: **3**



LEGEND

- Survey Monitoring Point
- Deep Settlement Point
- Surface Survey Point
- Inclinometer Location
- Major Contour
- Final Surface Extents
- Thermosyphon Radiator
- Datalogger Support Post

NOTES

1. Topographic contour data for the terrain model was provided by the Contractor.
2. The co-ordinate system is UTM NAD 83, Zone 13.

srk consulting

TMAC RESOURCES

Job No: 1CT022.004

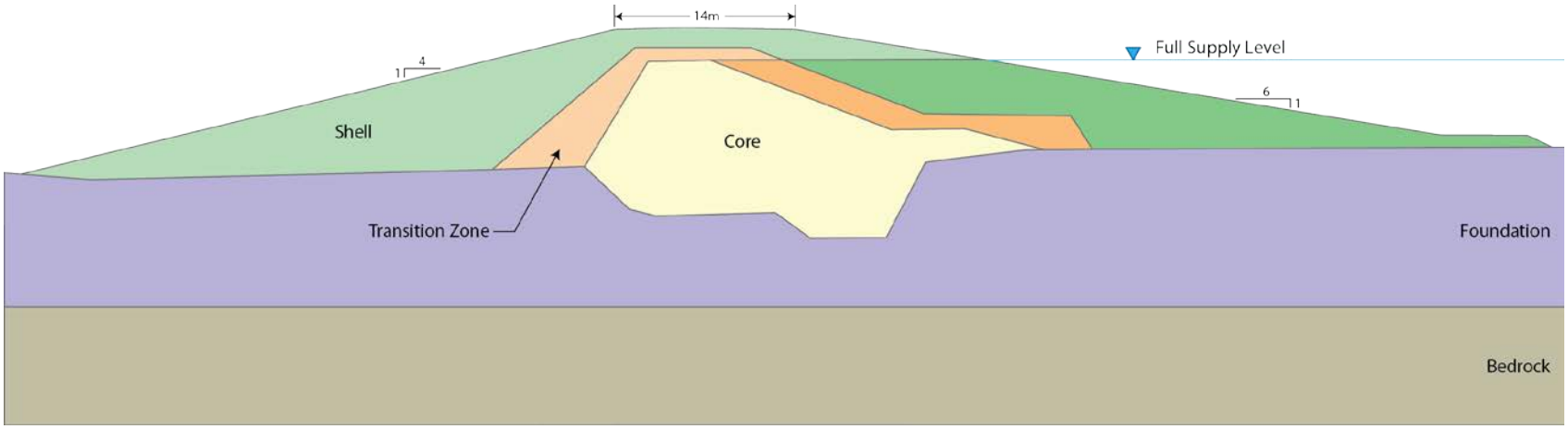
Filename: NorthDam_CreepAnalysis.pptx

HOPE BAY PROJECT

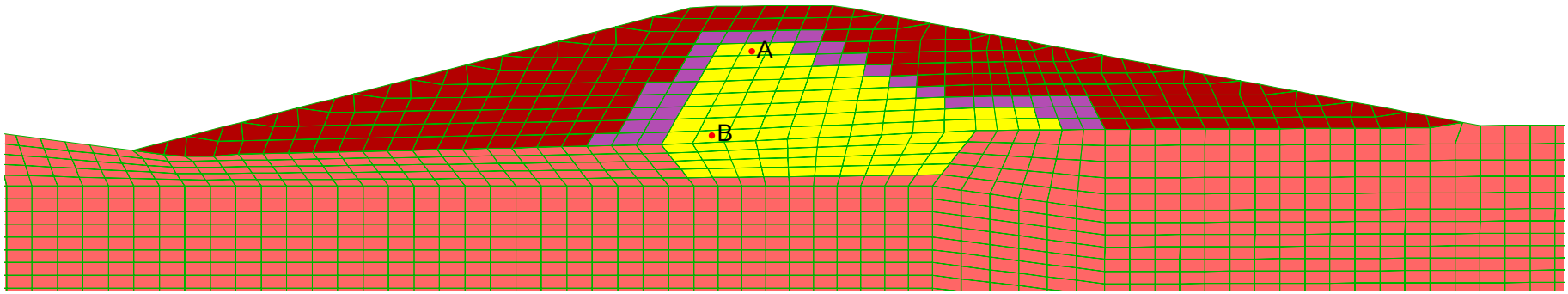
North Dam Creep Deformation Analysis

North Dam
Survey Monitoring Points

Date:	Approved:	Figure:
8/17/2016	AL	4



Model Regions

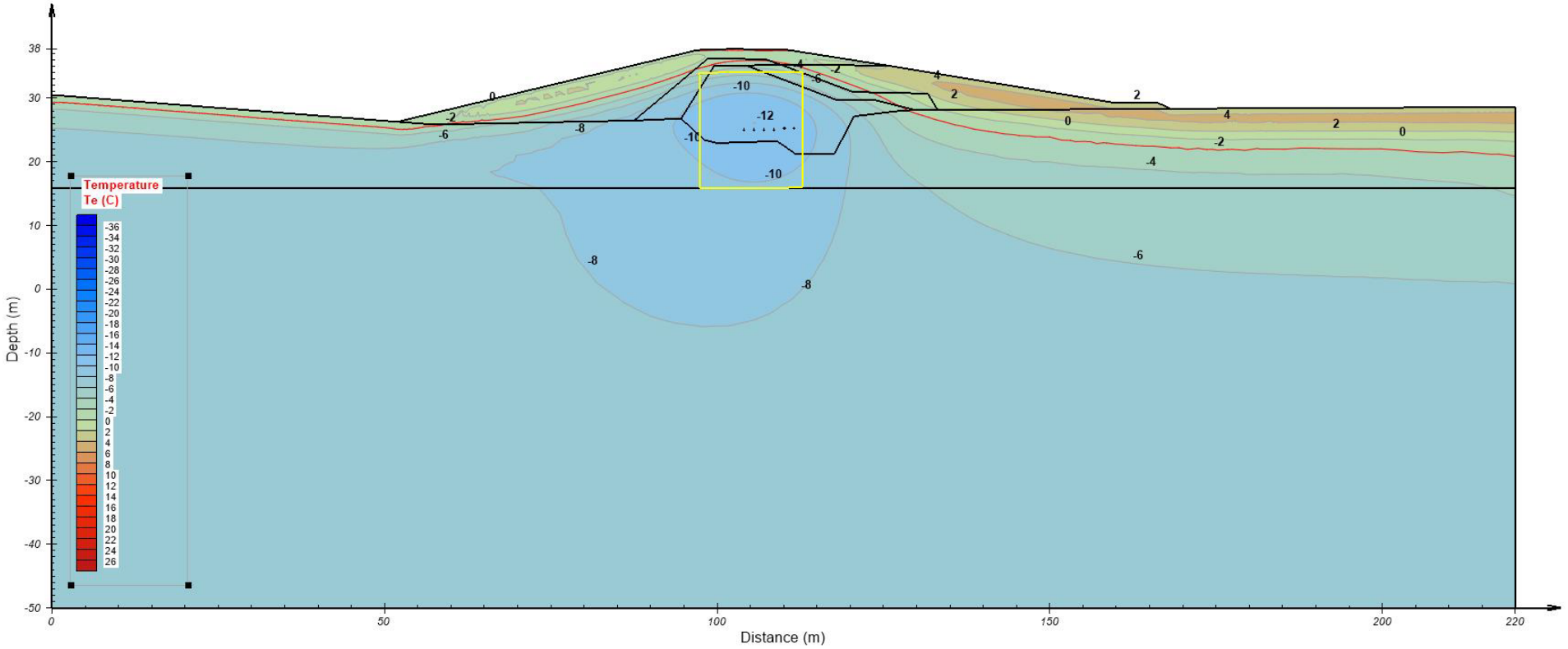


Finite Difference Mesh

Notes:

1. Foundation layer in the FLCA mesh is shown until bedrock
2. Points A and B for stress history; see Figures 14 and 17

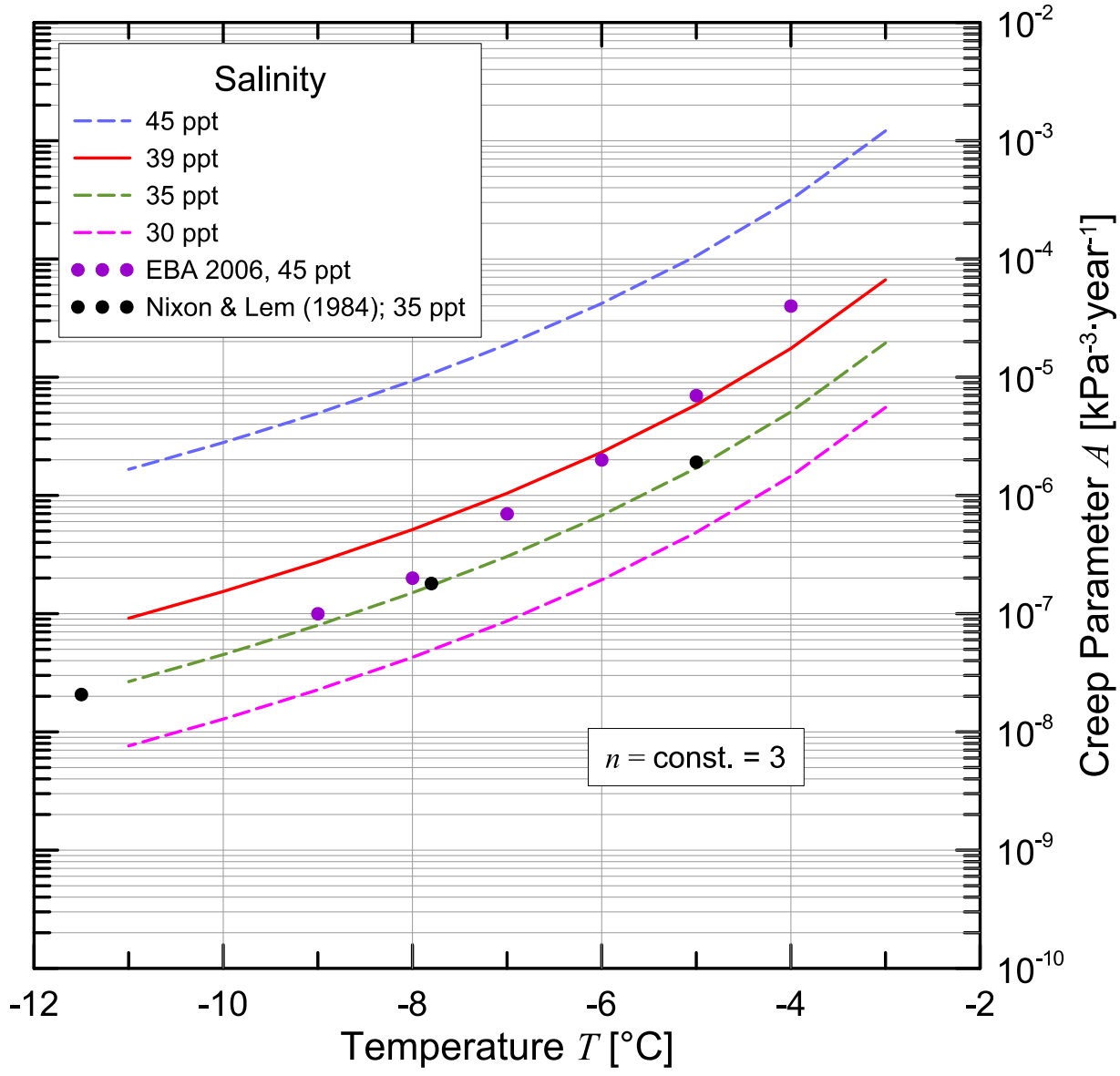
srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Model Set up – Section 0+85
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **5**



Notes:

1. Model results for year 10, maximum position of -2°C isotherm
2. Yellow bounding box indicates critical section based on thermal design criteria
3. Dam Section 0+85 with clay foundation
4. Five working thermosyphons
5. Source: SRK 2016b

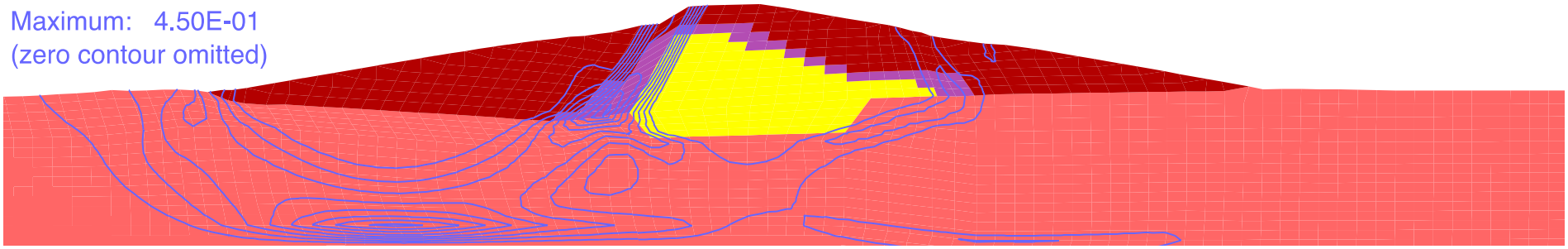
srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**North Dam – Year 10 (Section 0+85)**
		Date: 8/17/2016 | Approved: AL | Figure: **6**



srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Creep Parameter A for the Marine Clayey Silt
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: 7

Max. shear strain increment

Contour interval= 5.00E-02
Minimum: 0.00E+00
Maximum: 4.50E-01
(zero contour omitted)



Contour interval lines

Max. shear strain-rate

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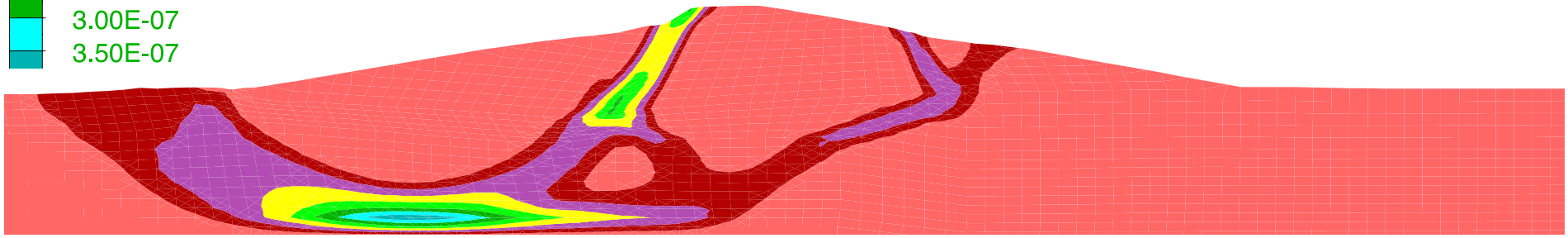
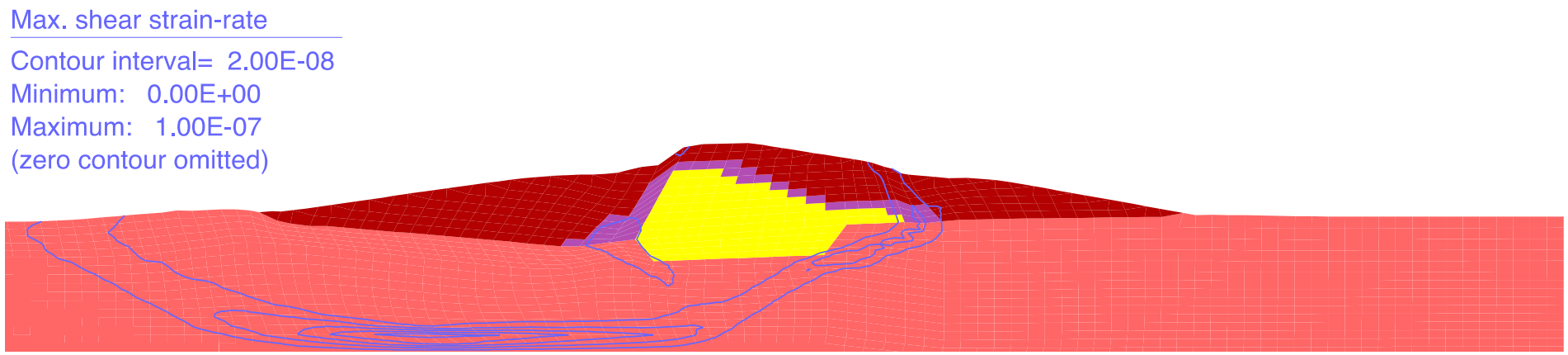
1.00E-07
1.50E-07
2.00E-07
2.50E-07
3.00E-07
3.50E-07



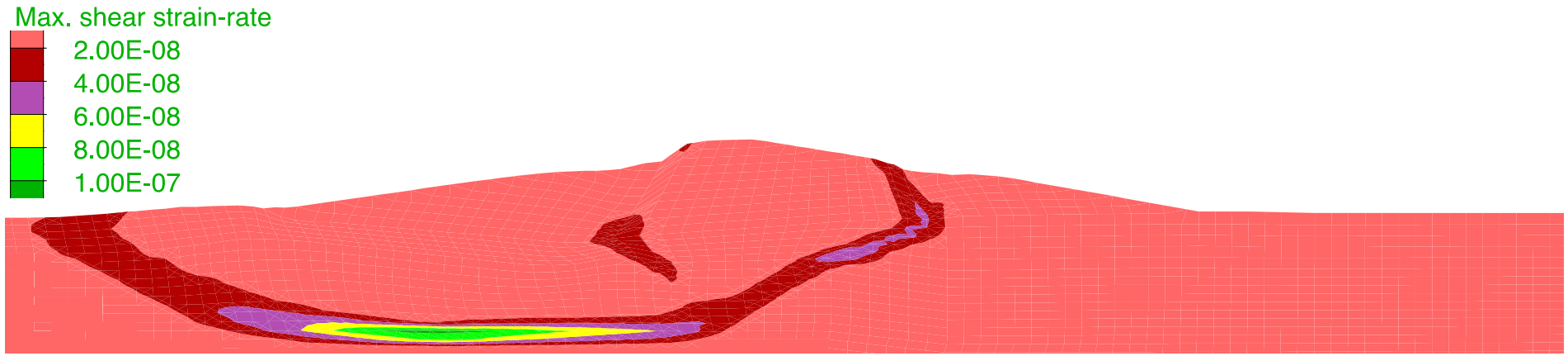
Contour intervals

Notes:
1. Units in year^{-1}
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**Shear Strain Rates Ten Years After Dam Construction**
		Date: 8/17/2016 | Approved: AL | Figure: **8**



Contour interval lines



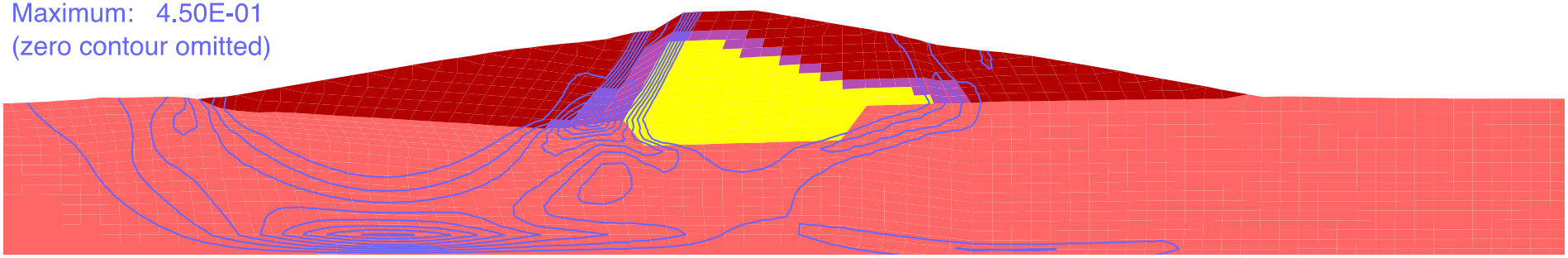
Contour intervals

Notes:
1. Units in year^{-1}
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**Shear Strain Rates 30 Years After Dam Construction**
		Date: 8/17/2016 | Approved: AL | Figure: **9**

Max. shear strain increment

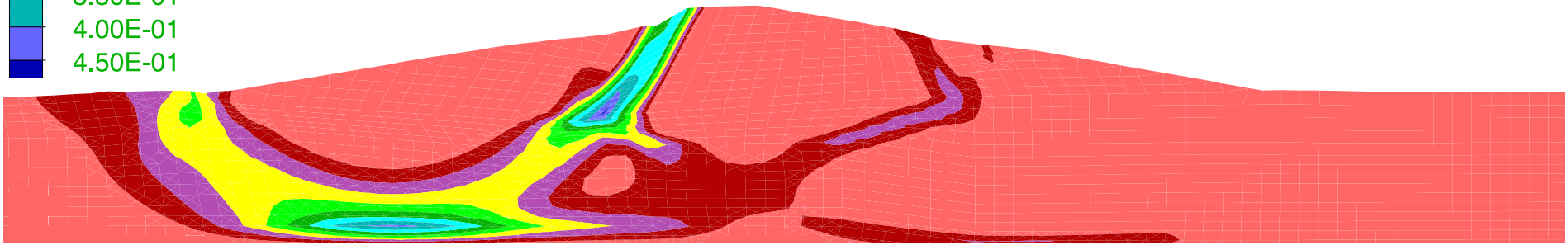
Contour interval= 5.00E-02
Minimum: 0.00E+00
Maximum: 4.50E-01
(zero contour omitted)



Contour interval lines

Max. shear strain increment

- 5.00E-02
- 1.00E-01
- 1.50E-01
- 2.00E-01
- 2.50E-01
- 3.00E-01
- 3.50E-01
- 4.00E-01
- 4.50E-01



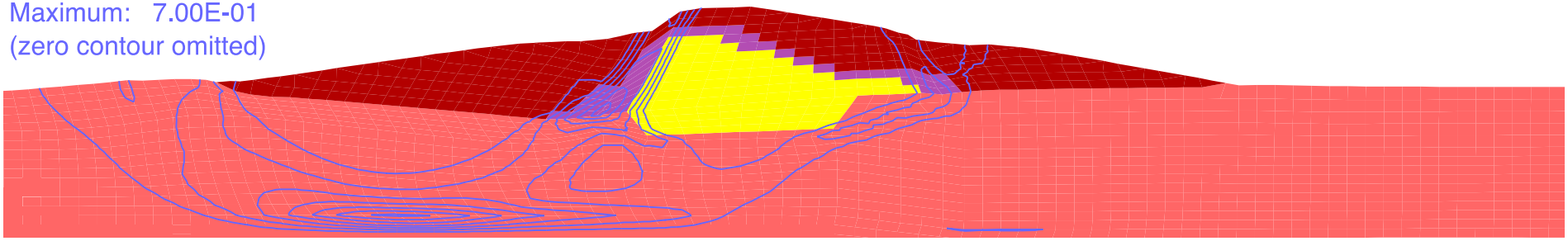
Contour intervals

Notes:
1. Units in meters/meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Shear Strains Ten Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **10**

Max. shear strain increment

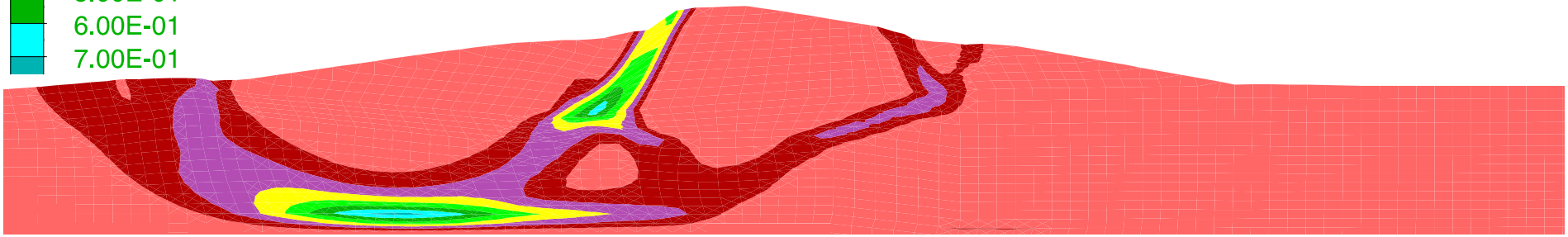
Contour interval= 1.00E-01
Minimum: 0.00E+00
Maximum: 7.00E-01
(zero contour omitted)



Contour interval lines

Max. shear strain increment

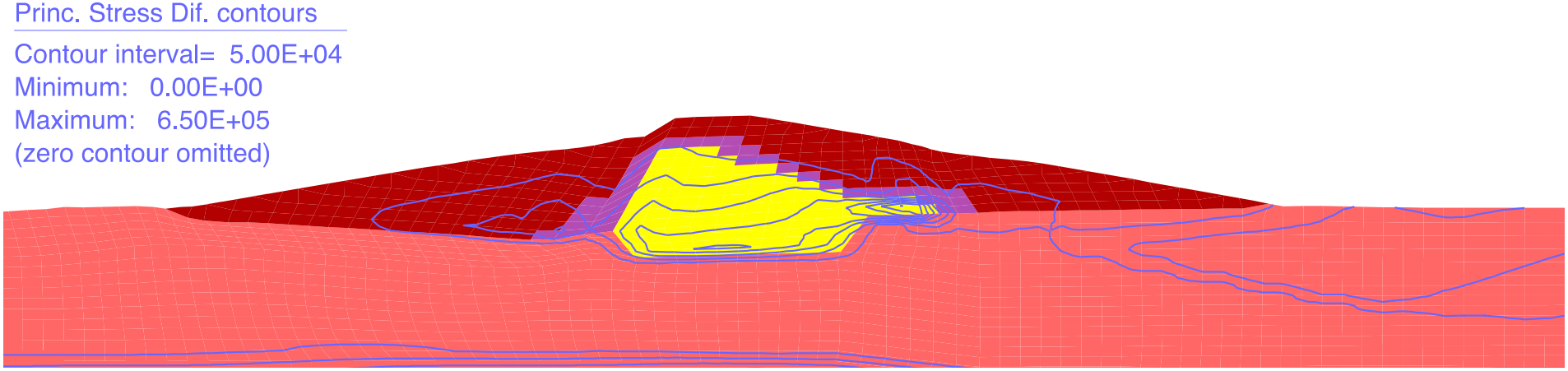
1.00E-01
2.00E-01
3.00E-01
4.00E-01
5.00E-01
6.00E-01
7.00E-01



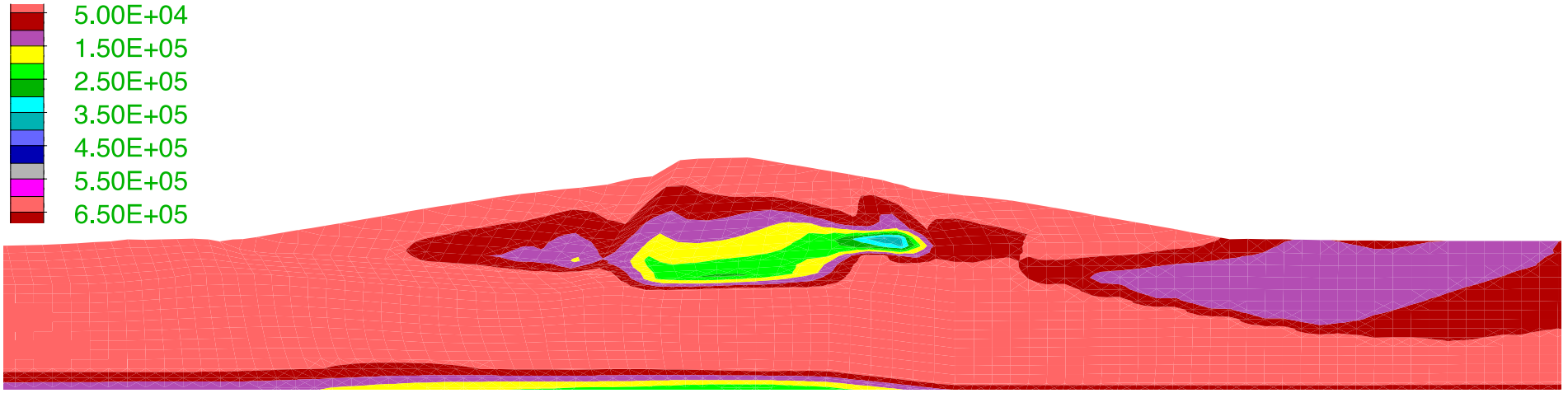
Contour intervals

Notes:
1. Units in meters/meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Shear Strains 30 Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **11**



Contour interval lines



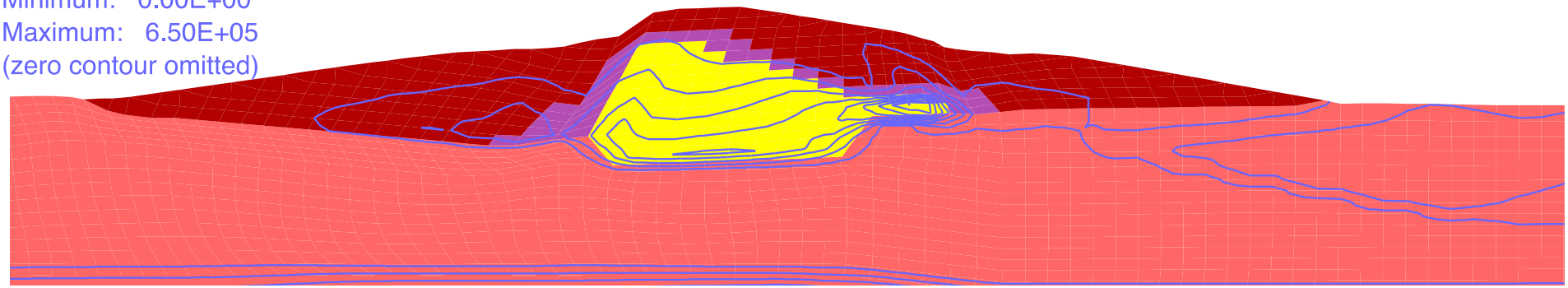
Contour intervals

Notes:
1. Units in Pascals
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**Principal Stresses Difference Ten Years After Dam Construction** Date: 8/17/2016 · Approved: AL · Figure: **12**

Princ. Stress Dif. contours

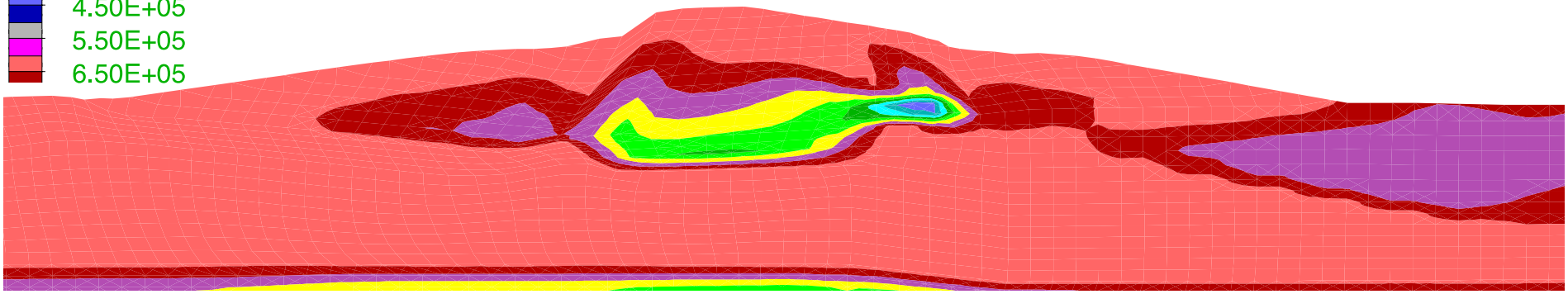
Contour interval= 5.00E+04
Minimum: 0.00E+00
Maximum: 6.50E+05
(zero contour omitted)



Contour interval lines

Princ. Stress Dif. contours

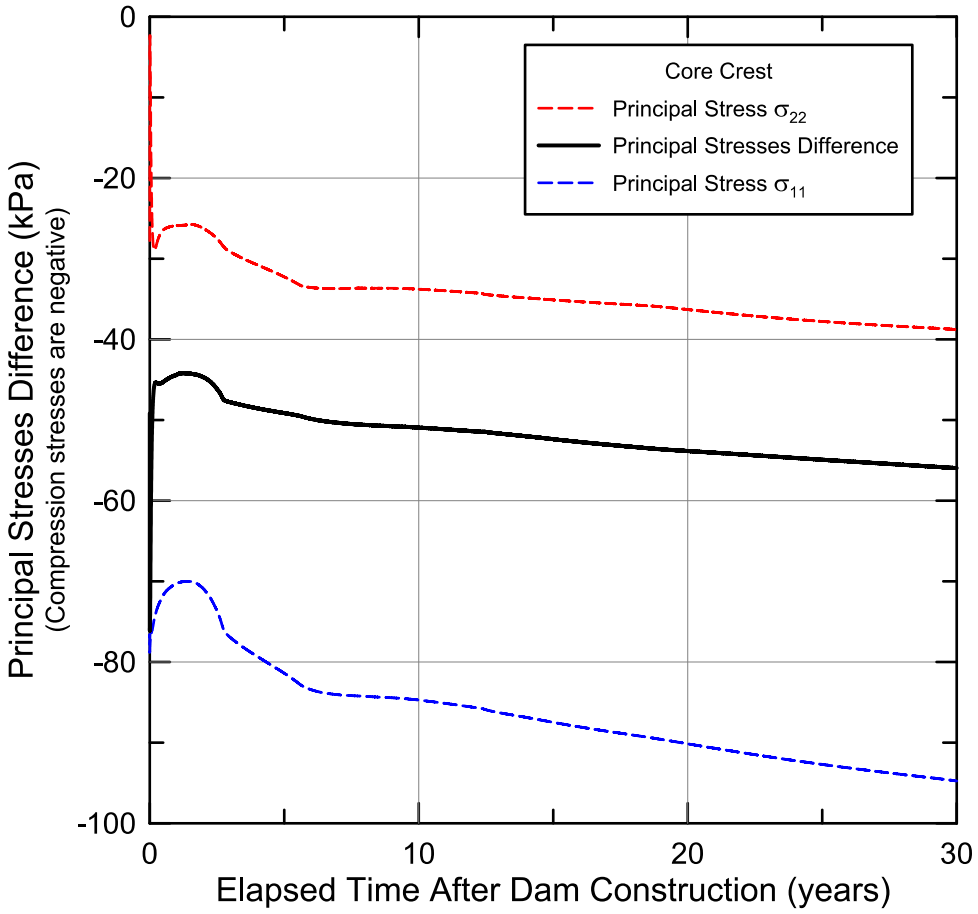
5.00E+04
1.50E+05
2.50E+05
3.50E+05
4.50E+05
5.50E+05
6.50E+05

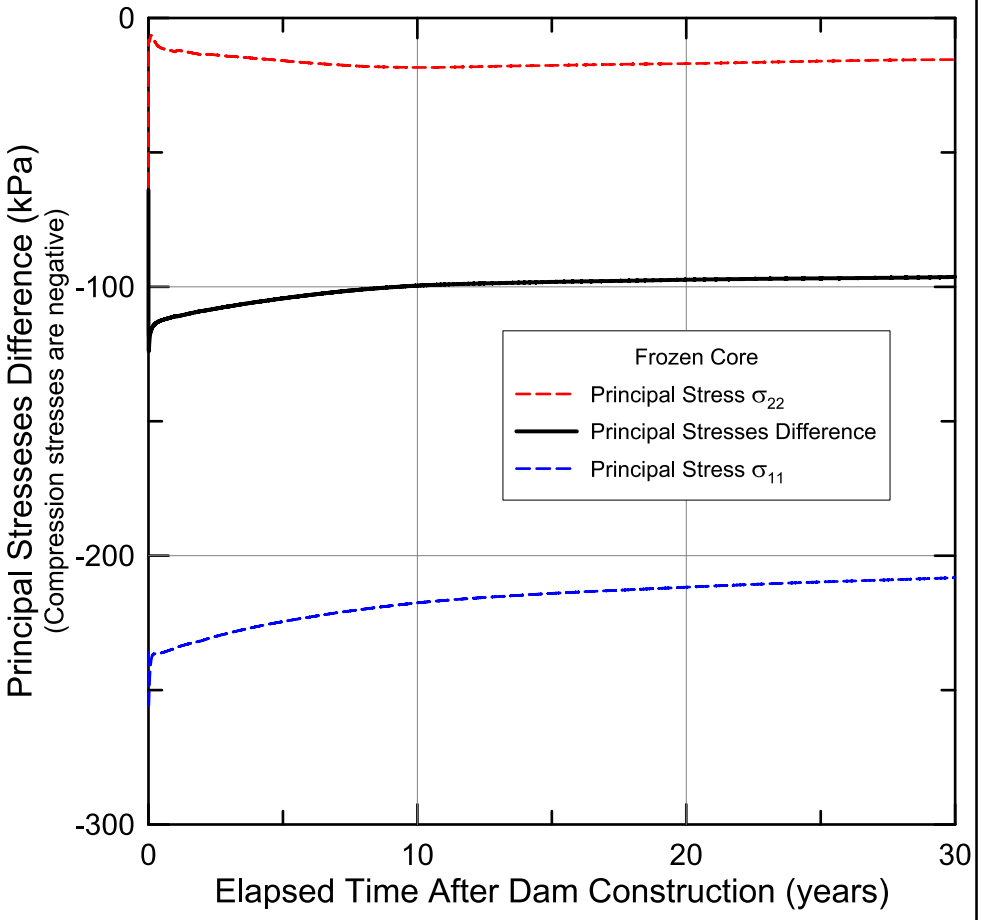


Contour intervals

Notes:
1. Units in Pascals
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Predicted Stresses Difference 30 Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **13**



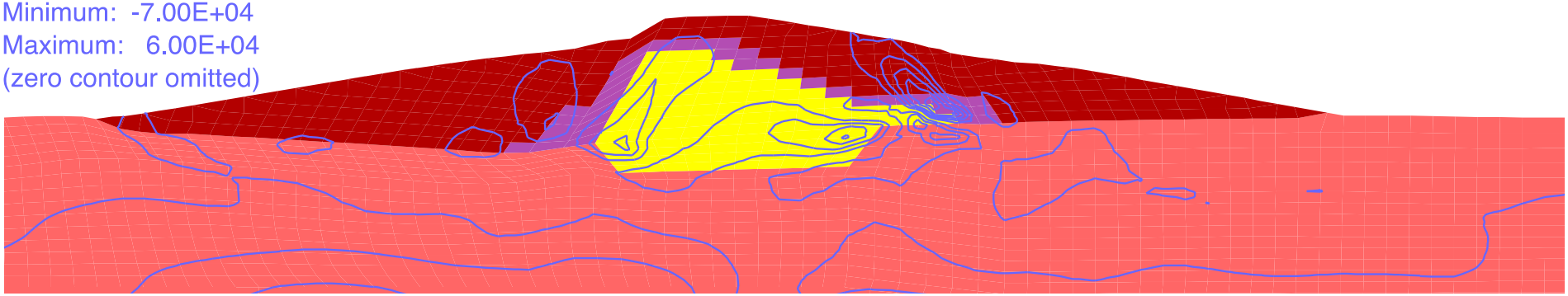


Notes:
1. Core crest is point A in Figure 5
2. Frozen core is point B in Figure 5
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

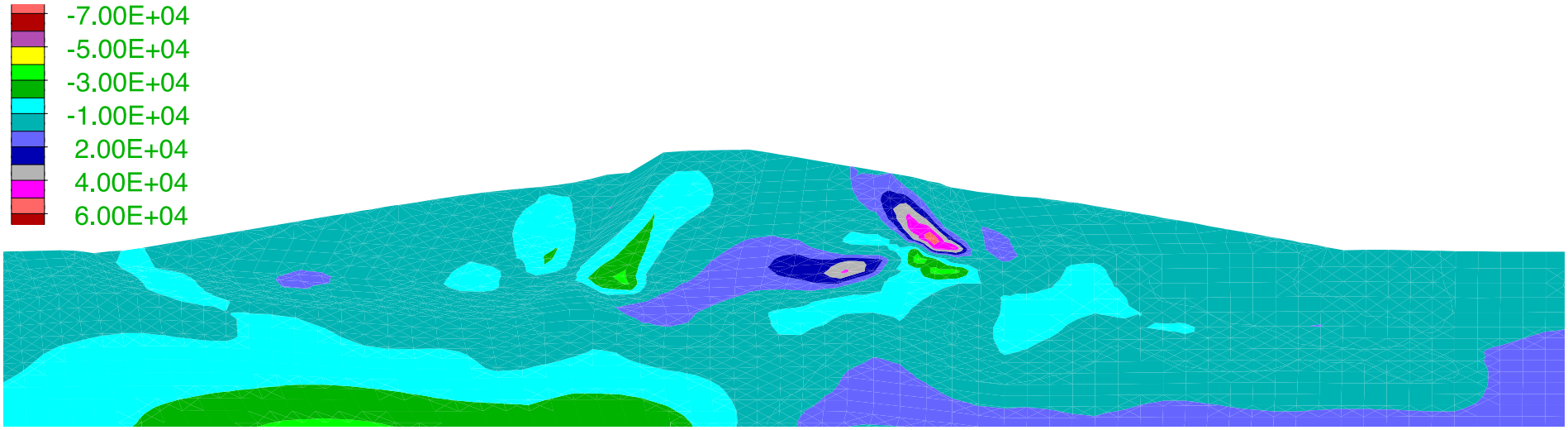
srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	**History of Principal Stresses Difference of two Points in the Core**
		Date: 8/17/2016 | Approved: AL | Figure: **14**

XY-stress contours

Contour interval= 1.00E+04
Minimum: -7.00E+04
Maximum: 6.00E+04
(zero contour omitted)



Contour interval lines



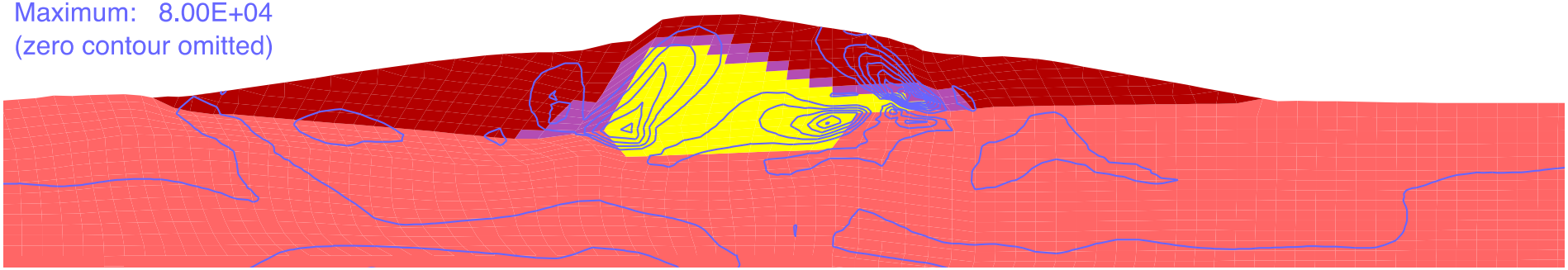
Contour intervals

Notes:
1. Units in Pascals
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

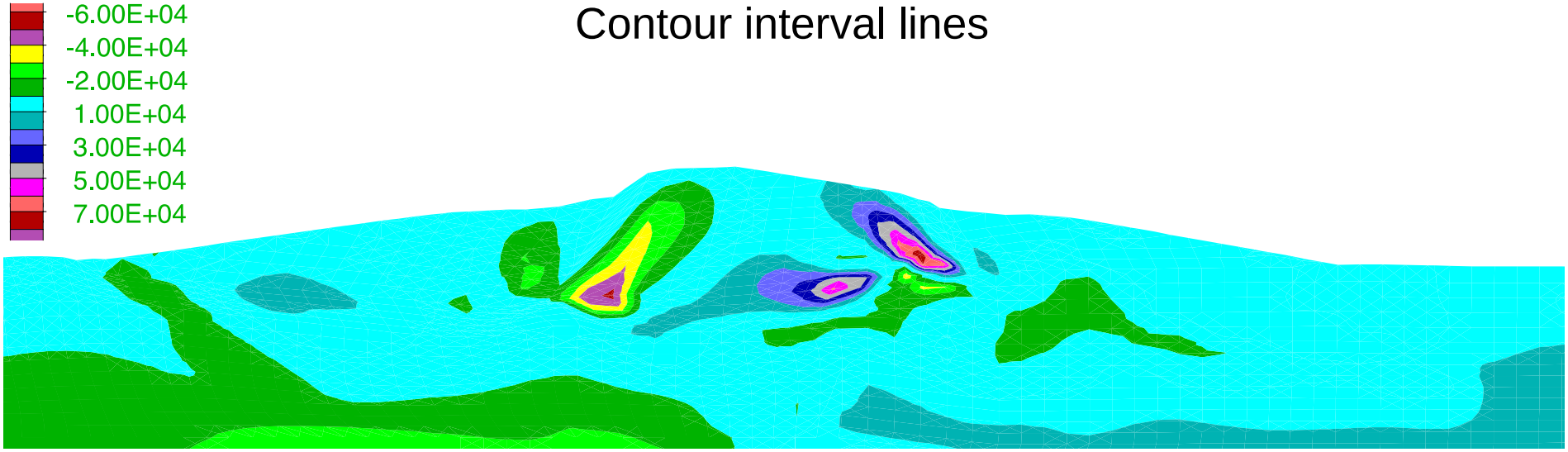
srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Shear Stresses Ten Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **15**

XY-stress contours

Contour interval= 1.00E+04
Minimum: -6.00E+04
Maximum: 8.00E+04
(zero contour omitted)



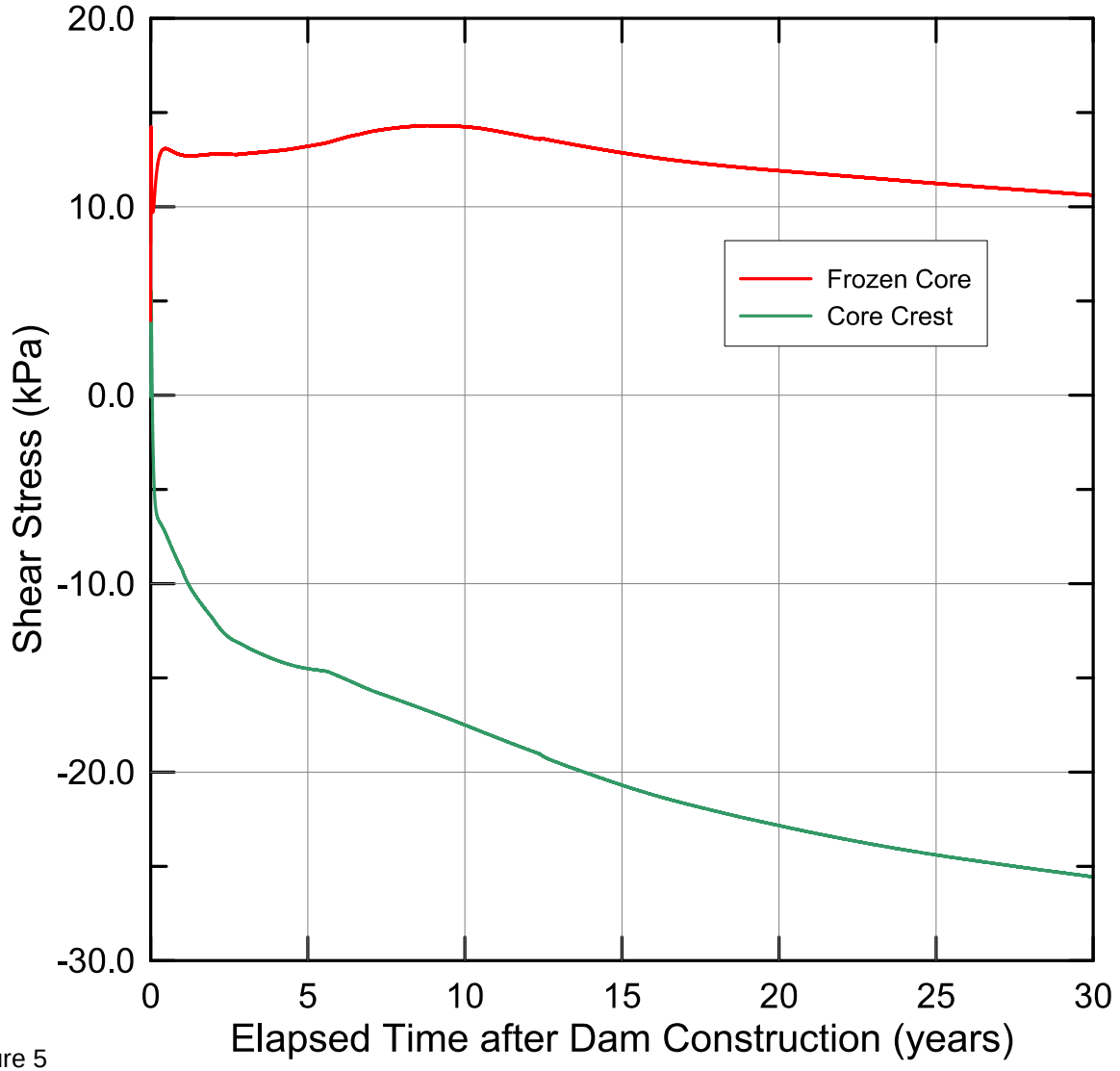
Contour interval lines



Contour intervals

Notes:
1. Units in Pascals
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
		Shear Stresses 30 Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **16**



Notes:
1. Core crest is point A in Figure 5
2. Frozen core is point B in Figure 5
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Shear Stresses History of two Points in the Core
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **17**

X-displacement contours

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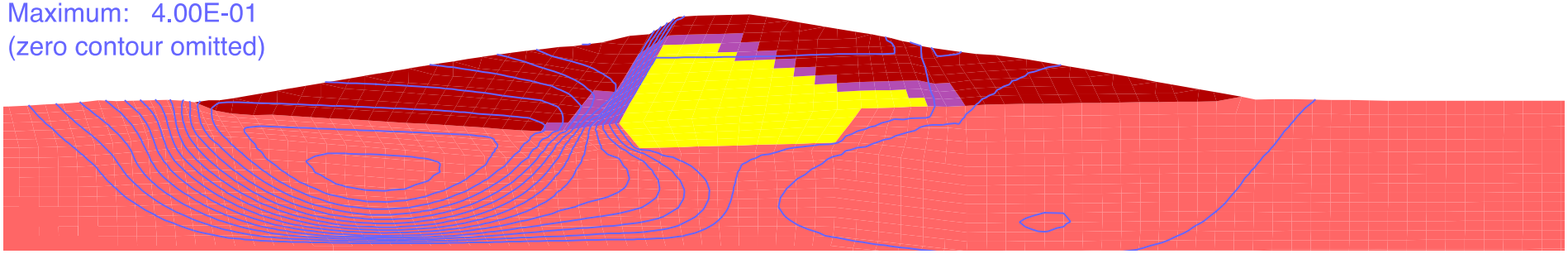
(zero contour omitted)



Contour interval lines

X-displacement contours

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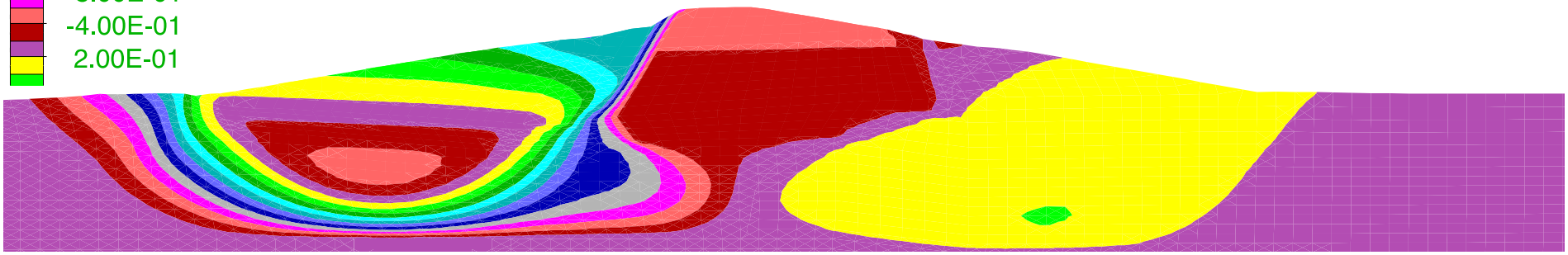
-2.40E+00
-2.00E+00
-1.60E+00
-1.20E+00
-8.00E-01
-4.00E-01
2.00E-01



Contour intervals

Notes:
1. Units in meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Horizontal Displacements Ten Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **18**

X-displacement contours

Contour interval= 2.00E-01
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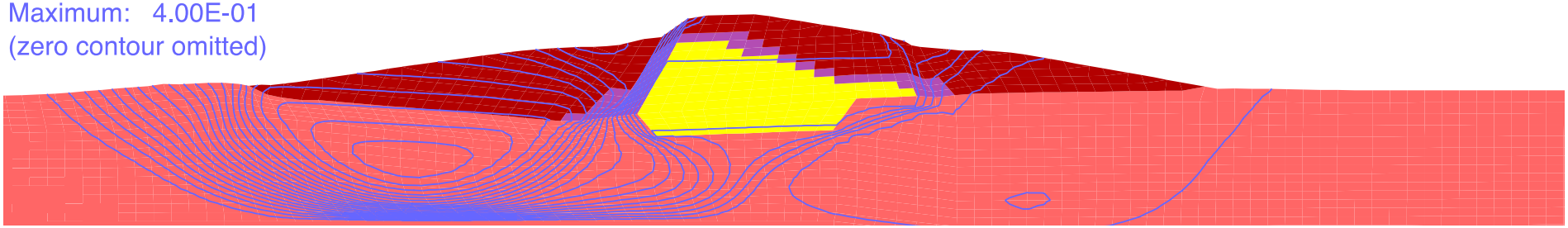
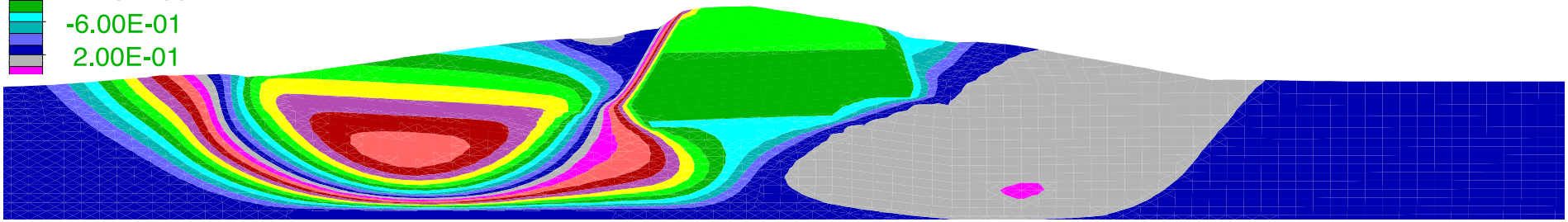
Maximum: 4.00E-01
(zero contour omitted)



Contour interval lines

X-displacement contours

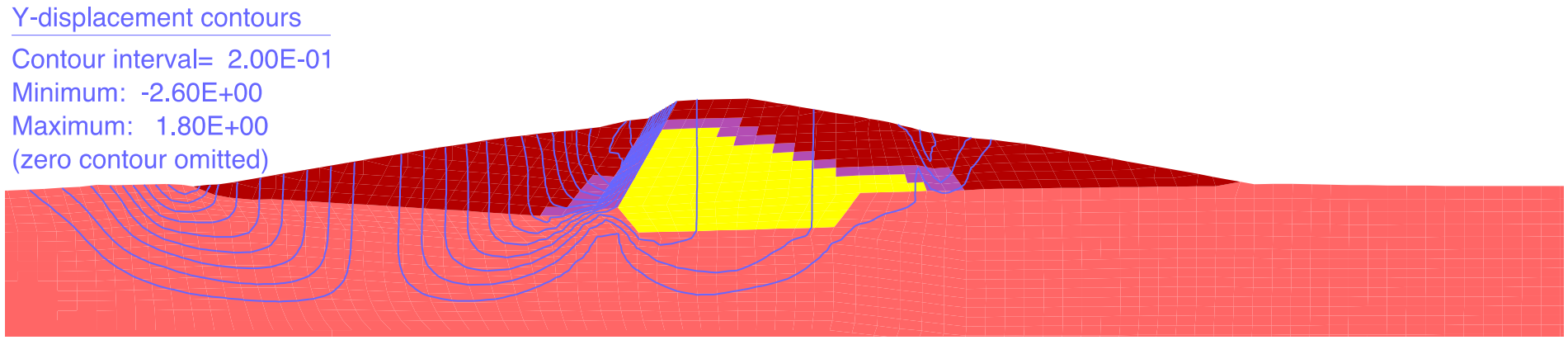
-4.20E+00
-3.60E+00
-3.00E+00
-2.40E+00
-1.80E+00
-1.20E+00
-6.00E-01
2.00E-01



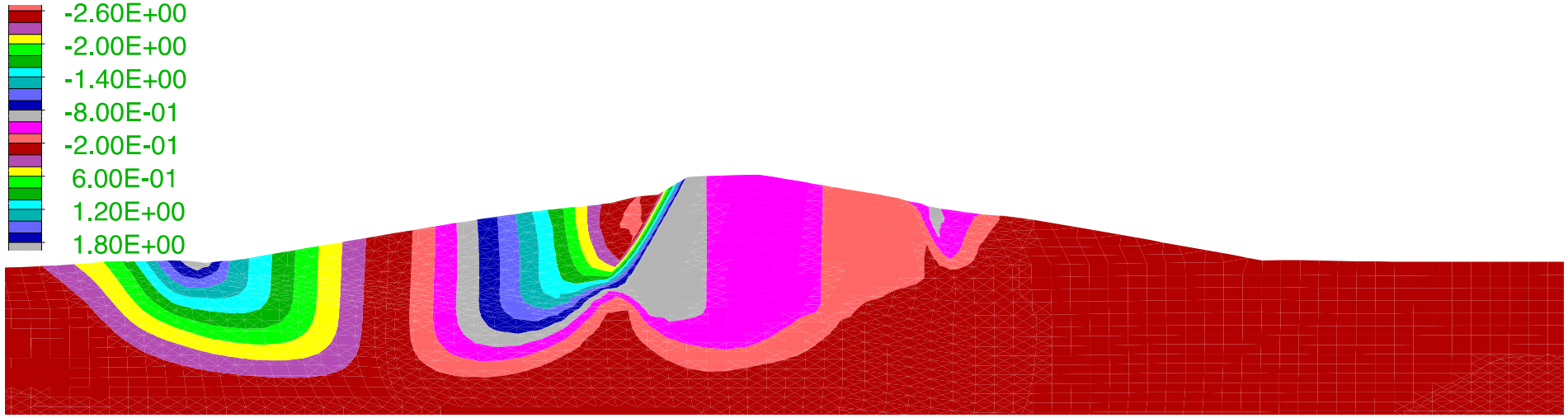
Contour intervals

Notes:
1. Units in meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Horizontal Displacements 30 Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **19**



Contour interval lines



Contour intervals

Notes:
1. Units in meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Vertical Displacements Ten Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **20**

Y-displacement contours

Contour interval= 2.00E-01
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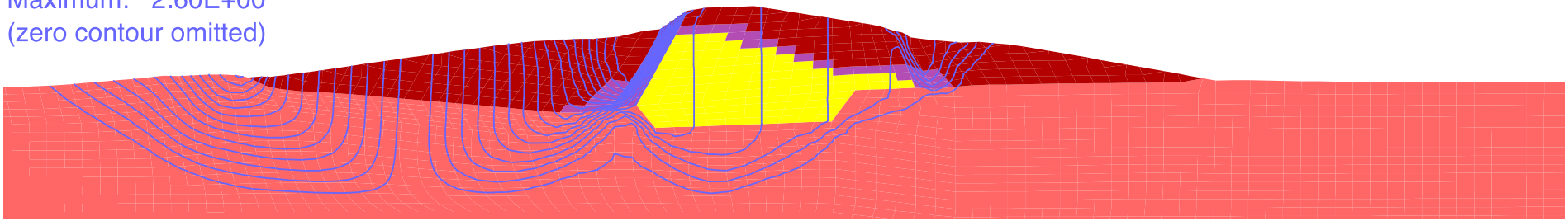
Maximum: 2.60E+00
(zero contour omitted)



Contour interval lines

Y-displacement contours

-3.40E+00
-2.60E+00
-1.80E+00
-1.00E+00
-2.00E-01
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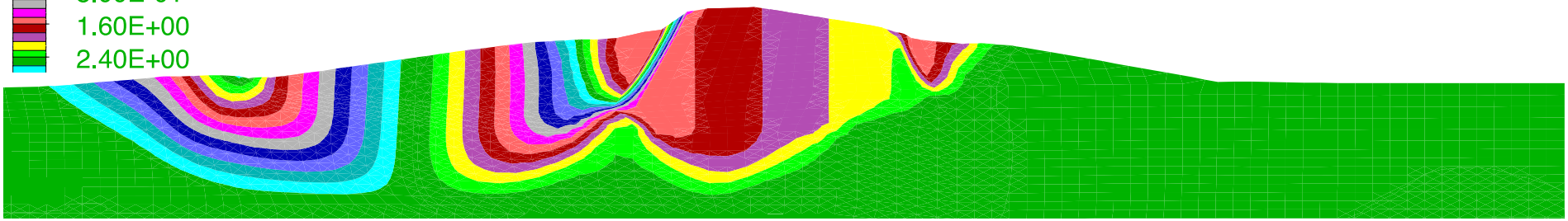
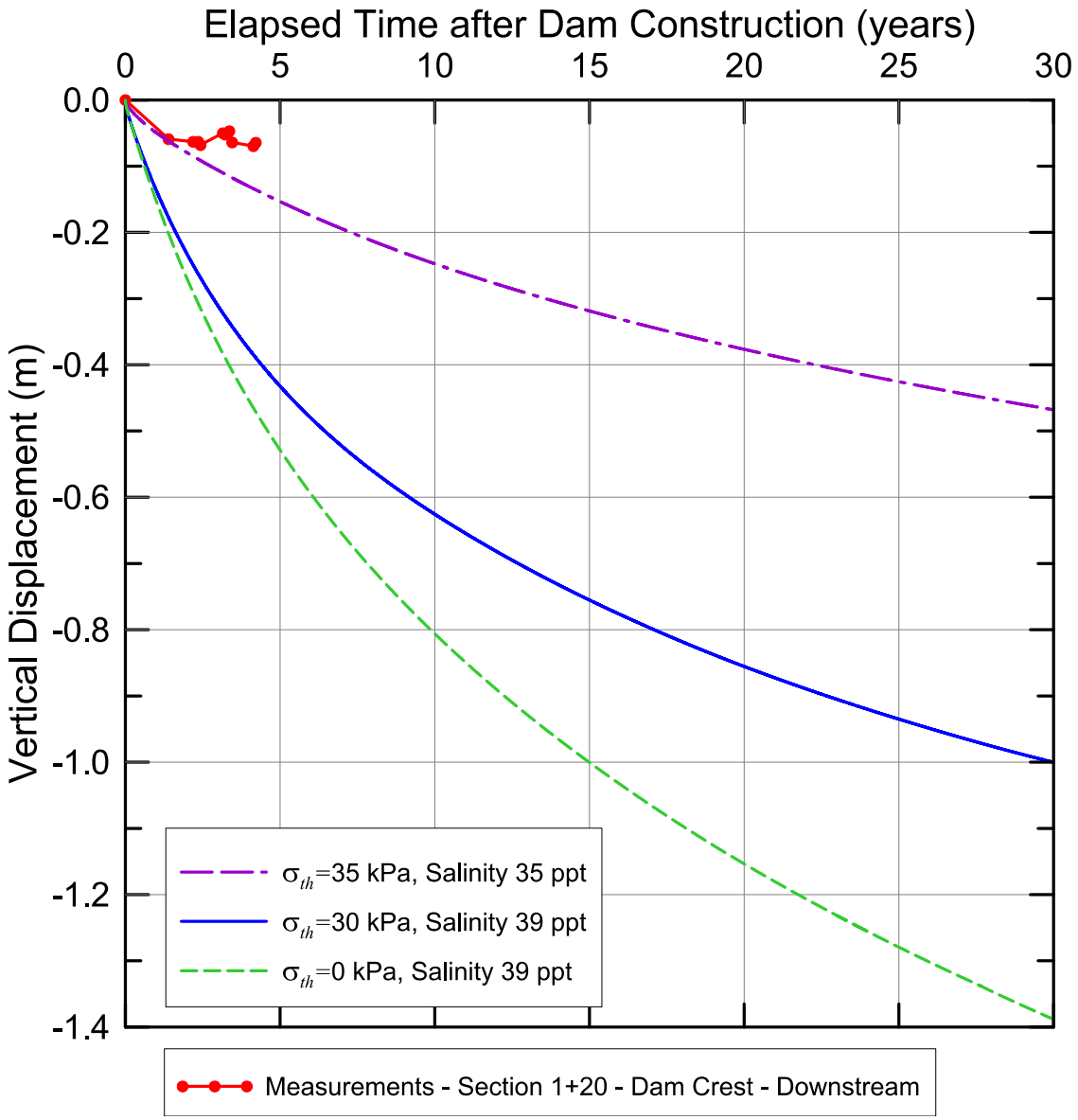
1.60E+00
2.40E+00



Contour intervals

Notes:
1. Units in meters
2. Foundation layer is shown until bedrock
3. Results for a salinity of 39 ppt in the clayey silt foundation and a threshold stress of σ_{th}=30 kPa

srk consulting	MAC RESOURCES	North Dam Creep Deformation Analysis
		Vertical Displacements 30 Years After Dam Construction
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **21**



srk consulting	TMAC RESOURCES	North Dam Creep Deformation Analysis
		Vertical Displacement History of the Core Crest
Job No: 1CT022.004 Filename: NorthDam_CreepAnalysis.pptx	HOPE BAY PROJECT	Date: 8/17/2016 | Approved: AL | Figure: **22**

Appendix F – Hope Bay Project: Phase 2 Tailings Impoundment Area North Dam Freeboard Hydraulic Assessment

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Samantha Barnes, EIT Victor Muñoz, PEng	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project - North Dam Freeboard Hydraulic Assessment		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix F as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes		

1 Introduction

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Phase 1 tailings are deposited sub-aerially in the Doris tailings impoundment Area (TIA), formerly Tail Lake, located approximately 5 km from the Doris mill. Containment is provided by three retention structures; a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam), and an Interim Dike situated at approximately the midpoint of the facility. Tailings would be deposited sub-aerially between South Dam and Interim Dike, and the Reclaim Pond will be contained between the Interim Dike and the North Dam. The North Dam was constructed over two winters (2011 and 2012) and has impounded water since 2011. The South Dam and Interim Dike are scheduled for construction in 2017.

Phase 2 tailings deposition would include a continuation of the Doris TIA with raising of the South Dam and construction of a new West Dam (SRK, 2017a). Under the current tailings management plan, all flotation tailings from Doris, Madrid, and Boston would be deposited in the TIA, with cyanide leach tailings being detoxified and used as structural backfill in the underground workings. Tailings would be deposited sub-aerially starting at spigot points along the crest of the raised South Dam and the West Dam, as well as additional spigot points along the eastern perimeter.

The deposition plan makes it such that final tailings surface will create a contiguous beach draining consistently to the north at a 1% slope starting at the South Dam, while the South and West dams will not be retaining any water with the Reclaim Pond developing against the North Dam. As tailings are deposited, the surface of the Reclaim Pond is gradually decreasing. The calculations of the freeboard presented here were completed for the smallest Reclaim Pond footprint at the end of tailings deposition, this being the most conservative scenario.

This memo summarizes the inflow design flood (IDF) and hydraulic freeboard calculations for the North Dam.

2 Methodology

2.1 Supporting Information

Topographical information for the Project site consists of LiDAR, provided by TMAC. The dam alignment and proposed tailings surface bathymetry was used to estimate fetch length and average water depth across the Reclaim Pond.

Required hydrological inputs such as rainfall depth, snow pack, wind speed and spring freshet were obtained from SRK (2017b). The month with the highest snowmelt is June, and this is incorporated into the spring probable maximum flood (PMF) estimation. The IDF table from the SRK (2017b) report was also used in the spring Probable Maximum Precipitation (PMP) calculation.

2.2 Approach

A wind and wave analysis was performed to ensure that the crest of the dam is protected against the most critical of the following two cases (CDA, 2007, updated 2013):

- Normal freeboard: no overtopping by 95% of the waves caused by the most critical wind with a frequency of 1 in 1,000 year when the reservoir is at its full supply level (FSL); and

- Minimum freeboard: no overtopping by 95% of the waves caused by the most critical wind associated with the annual exceedance probability (AEP) event, when the reservoir is at its maximum extreme level during the occurrence of the IDF.

In accordance with the most recent Canadian Dam Safety Association Guidelines (CDA, 2014) the possibility of high water level occurring at the same time of the storm and high wind events must also be considered. Therefore a third scenario was analysed, combining the normal and the minimum freeboard values described above.

The following sections describe the estimations for the IDF, the normal freeboard and the minimum freeboard, as well as the results of the combined freeboard scenario.

3 Inflow Design Flood

3.1 Method

For a dam with a HIGH consequence classification, the IDF is defined to be 1/3 between the 1,000-year event and the PMF (CDA, 2014). The TIA was however designed without a spillway, thus requiring the retention of the full PMF event. The IDF was therefore increased to account for the PMF, which corresponds to a dam with an EXTREME consequence classification.

Both scenarios described above were modelled in HEC-HMS, developed by the U.S. Army Corps of Engineers (USACE, 2015). Additional inputs for the model included the TIA catchment area and associated lag time, and a Curve Number to account for rainfall losses. The results of the model were a range of total flood volumes (Section 3.4).

3.2 Hydrologic Definitions

3.2.1 Probable Maximum Flood

The PMF is considered the most severe flood event that may reasonably be expected at the Project site. It is generated by the PMP, which is the maximum precipitation that may reasonably occur at the Project site, plus snowmelt.

Two PMF cases were considered, based on guidance from the Canadian Dam Association (CDA, 2007 updated 2013):

1) Summer PMF, which is generated by the summer PMP; and

2) Spring PMF, which is defined as the maximum of the following two cases:

    a. Rainfall dominated event: PMF computed with the spring PMP and snowmelt from a 1/100-year snow accumulation; and

    b. Snowmelt dominated event: PMF computed with the 1/100-year rainfall and the snowmelt from the probable maximum snow accumulation.

The probable maximum snow accumulation can be simplified as two times the 100-year snowpack, (Alberta Transportation, 2004).

The snowmelt in each case is calculated by applying a 1/100-year temperature sequence to the design snow depth for an appropriate duration. Since the North Dam will not have a spillway, the selected duration was the full snowmelt period to ensure the dam does not overtop. This simplified the snowmelt calculations to a total snowpack calculation, in snow-water-equivalent (SWE).

3.2.2 1/1,000-Year Flood

The 1/1,000-year flood is the flood resulting from the larger of two events, similar to the PMF, as defined by CDA:

1. Summer 1/1,000-year rainfall; and
2. Spring 1/1,000-flood, which is defined as the maximum of two cases:
    a. Rainfall dominated event: A 1/1,000-year spring rainfall event combined with average snowmelt; and
    b. Snowmelt dominated event: A 1/1,000-year snowmelt combined with an average spring rainfall event.

The snowmelt in this case is also simplified to the total snowpack of each frequency, in SWE.

3.3 Modelled Flow Conditions

Based on the definitions above, a total of five hydrologic conditions were prepared in the HEC-HMS model, which are outlined below:

1) Summer PMF: generated by the summer PMP event;
2) Spring PMF Rainfall-Dominated: generated by the 100-year SWE and the spring PMP event;
3) Spring PMF Snowmelt-Dominated: generated by the 100-year spring rainfall and the probable maximum snow accumulation SWE.
4) 1/1,000-year Summer Flood: generated by the summer 1/1,000 rainfall event;
5) 1/1,000-year Rainfall-Dominated Spring Flood: generated by the spring 1/1,000 rainfall event and the average spring snowmelt; and
6) 1/1,000-year Snowmelt-Dominated Spring Flood: generated by the 1/1,000 SWE and the average spring 24-hour rainfall.

Inputs to the five hydrologic conditions are summarized in Table 1 and defined in SRK (2017b). The total precipitation is equal to the sum of the snowmelt and the rainfall, under each case.

Table 1: IDF Hydrologic Calculation Inputs

Hydrologic Condition	Rainfall (mm)	Snowmelt, in SWE (mm)	Total Precipitation (mm)
1. Summer PMF	180	0	180
2. Spring PMF: Rainfall-Dominated	115	126	241
3. Spring PMF: Snowmelt-Dominated	49	252	301
4. 1/1,000 Summer Flood	118	0	118
5. 1/1,000 Rainfall-Dominated Spring Flood	75	56	131
6. 1/1,000 Snowmelt-Dominated Spring Flood	7	202	209

3.4 Hydrologic Model

3.4.1 Catchment Delineation

The TIA catchment was delineated in Global Mapper™ (Blue Marble, 2016) using site topography, and was divided into three categories: tailings beach, Reclaim Pond, and undisturbed (i.e. natural catchment) area (

Table 2).

3.4.2 Transformation

The SCS Unit Hydrograph method was used to transform precipitation into an outflow hydrograph for each sub-area. HEC-HMS uses a single input parameter, the lag time (T_{lag}), defined as the time between the centroid of precipitation mass to the peak for the resulting hydrograph. The lag time for each catchment was calculated using the National Resource Conservation Service (NRCS) transformation from time of concentration. The NRCS transformation is T_{lag} = 0.6*Tc. Time of concentration and resulting lag times are relatively short (<15 minutes). This results in peak discharges near the time of the storm peak. The time of concentration was estimated with the methods cited in (Li *et al*., 2008). A minimum time of concentration of ten minutes was applied to all catchments. Each catchment's lag time is presented in

Table 2.

To best approximate the storm type, the geographical region closest to the Project site should be used to estimate the storm characteristics. The Type 1 rainfall distribution is typical across Alaska (Chow *et al*. 1988) and because of the close weather similarities between Alaska and the Project site, based on Peel MC *et al.* (2007), the SCS Type I rainfall distribution was selected for use in the HEC-HMS model. This distribution approximates an intense, short-duration storm event.

3.4.3 Curve Number Selection

The SCS Curve Number (CN) for the tailings beach was estimated based on typical tables, based on hydrologic group A, for sands, and an Antecedent Moisture Condition 3 (AMC III) which assumes the soil is saturated. For the Reclaim Pond area, no losses are expected and a CN of 100 was used.

The CN for the undisturbed (i.e. natural catchment) area was calibrated using the 1/100 year flow event for the nearby hydrometric station 10TF001 Freshwater Creek near Cambridge Bay (EC, 2015). The lag time and catchment area were modelled in HEC-HMS, and the CN was adjusted until the modelled peak flow matched the historically estimated 1/100 year peak flow. The final CN for the undisturbed areas was calibrated to be 72. A factor of 1.2 was applied to the CN value to convert it from AMC II to AMC III, giving a CN value of 86 for undisturbed areas.

A summary of catchment CNs and other characteristics are presented in

Table 2.

Table 2: Catchment Characteristics

Catchment Description	Catchment Area (km^2)	Curve Number	Lag Time (min)	Source
Tailings Beach Area	1.33	80	6	Natural desert landscaping (pervious area only), Group A, AMC III
Reclaim Pond Area	0.43	100	6	Pond surface – no losses
Undisturbed (i.e. Natural Catchment) Area	2.54	86	6	Calibrated by SRK, AMC III

3.5 Hydrologic Model Results

Using the input parameters listed in Table 1 and Table 2, the results presented in Table 3 were determined.

Table 3: Hydrologic Model Results

Hydrologic Condition	Rainfall Volume (m^3)	Snowmelt Volume (m^3)	Total Volume (m^3)
1. Summer PMF	588,600	-	588,600
2. Spring PMF: Rainfall-Dominated	401,037	439,863	840,900
3. Spring PMF: Snowmelt-Dominated	178,718	915,382	1,094,100
4. 1/1,000 Summer Flood	339,000	-	339,000
5.1/1,000 Rainfall-Dominated Spring Flood	223,454	166,846	390,300
6. 1/1,000 Snowmelt-Dominated Spring Flood	23,720	684,480	708,200

Based on the results presented in Table 3, the critical storm for the PMF was found to be Condition No. 3, the snowmelt-dominated Spring PMF, and the critical hydrologic condition for the 1/1,000-year event was found to be Condition No. 5, i.e.. the Snowmelt-Dominated Spring Flood.

These results were subsequently used to calculate the IDF volume for both the HIGH and EXTREME consequence classifications (Table 4). The HIGH consequence classification is based on an interpolation between the critical 1/1,000-year event and the critical PMF (CDA, 2014).

Table 4: IDF Volume Results for Dam Classifications High and Extreme

Dam Consequence Classification	Corresponding IDF	Critical Hydrologic Condition	Rainfall Volume (m^3)	Snowmelt Volume (m^3)	Total IDF Volume (m^3)
HIGH	1/3 between 1/1,000-year event and the PMF	3, 6	75,386	761,448	836,833
EXTREME	PMF	2	178,718	915,382	1,094,100

At the end of the mine life, when the Reclaim Pond is at its smallest, the IDF's listed in Table 2 translates to water level increases over the design FSL of 33.5 m for the HIGH and EXTREME consequence classifications of 1.7 m (35.2 m) and 2.1 m (35.6 m) respectively.

4 Freeboard

4.1 Input Parameters and Design Criteria

Inputs for calculating the normal and minimum freeboard for the North Dam include wind speed, initial water level, average Reclaim Pond water depth, and fetch length as presented in Table 5. Fetch length was updated based on the initial water level.

Table 5: Freeboard Design Criteria and Input Parameters

Parameter	Normal Freeboard		Minimum Freeboard HIGH Consequence Classification		Minimum Freeboard EXTREME Consequence Classification	
	Design Criteria	Value	Design Criteria	Value	Design Criteria	Value
IDF	n/a		1/3 between 1/1,000-year event and the PMF		PMF	
Wind Speed (m/s)	1/1,000 year event	35.3	1/2 year event	25.6	1/2 year event	25.6
Initial Water Level (masl)	FSL	33.5	FSL + IDF	35.2	FSL + IDF	35.6
Average Water Depth (m)	FSL	4.4	FSL + IDF	6.1	FSL + IDF	6.5
Fetch Length (m)	FSL	950	FSL + IDF	1,100	FSL + IDF	1,160
Corrected Wind Speed Over Water[1](m/s)	Correction Factor = 1.09	38.5	Correction Factor = 1.10	28.3	Correction Factor = 1.11	28.4
Significant Wave Height (m)[2]	m	0.80	m	0.75	m	0.75
Specific Wave Height (m)	5% exceeding specific wave height[4]	1.12	5% exceeding specific wave height	1.05	5% exceeding specific wave height	1.05
Run-up Ratio[3]	6:1 (H:V)	0.95	6:1 (H:V)	0.95	6:1 (H:V)	0.95

Note:

1. Correction factors are based on fetch length (USACE 1997). See Appendix A-3.
2. The significant wave height was estimated based on the maximum of: wave height figures presented by SANCOLD (1990) in Appendix A-1 and A-2, and the Design Standards No. 13 for Embankment Dams, USBR (2012).
3. Run-up ratio is based on a relationship between the wave height, the embankment slope and the embankment protection (SANCOLD, 1990). Appendix A-4.
4. Average wave height of the highest 5% of waves in a given spectrum, based on Design Standards No. 13 for Embankment Dams, USBR (2012).

4.2 Normal Freeboard

4.2.1 Definition

Normal freeboard is the sum of the wind set-up and the wave run-up, based on the design parameters listed in Table 5. Figure 1 illustrates the parameters contributing to the normal freeboard calculation.

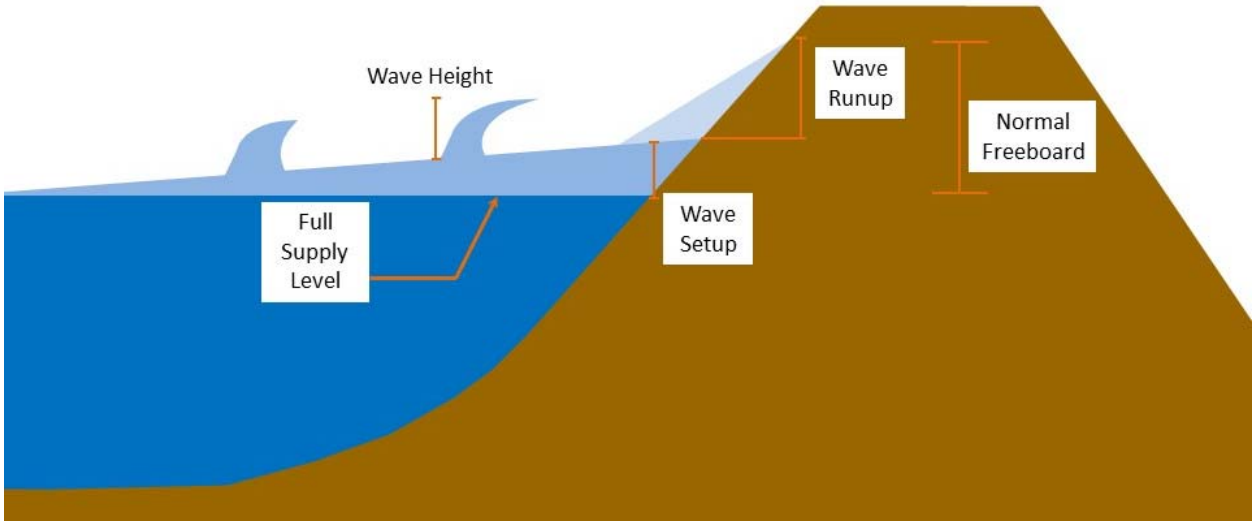

Figure 1: Normal Freeboard Schematic

4.2.2 Wind Setup

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) (which for the purpose of these calculations is equivalent to the FSL) using the following expression:

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

Where:

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

Based on the input parameters presented in Table 5, the resultant wind setup for the normal freeboard is 0.07 m.

4.2.3 Wave Run-up

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 was based on the method presented by the South African National Committee on Large Dams (SANCOLD, 1990). SANCOLD (1990) presents a relationship between the wave height, the embankment slope and the embankment protection. The wave run-up is calculated using Equation 2 below.

$$Wave\ Runup = Wave\ Height \times Runup\ Ratio \quad \text{(Eq. 2)}$$

Based on the 6 to 1 slope (horizontal to vertical), and a combination of rough stone and riprap, the run-up ratio was approximated to be 0.95 (Table 5) while the wave run-up for the 1/1,000 wind event was calculated to be 1.06 (Equation 2).

This value and methodology was compared with the methodology presented by USBR (2012); this reference suggests slightly lower values than SANCOLD (1990). As a conservative approach, the values selected utilize the SANCOLD (1990) methodology.

4.2.4 Normal Freeboard Estimate

The normal freeboard for the North Dam was subsequently determined to be 1.13 m (wind setup of 0.07 m + wave run-up of 1.06 m).

4.3 Minimum Freeboard

4.3.1 Definition

Minimum freeboard is described as the required freeboard to protect against the IDF and the wave run-up from a 1 in 2-year wind storm event. This is schematically illustrated in Figure 2, based on the design parameters listed in Table 5.

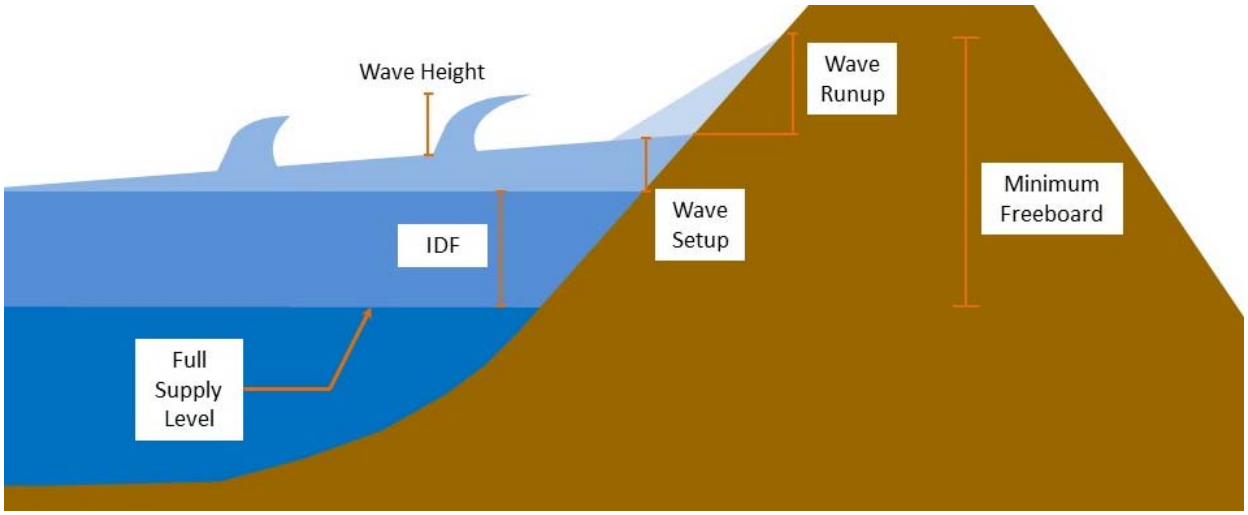

Figure 2: Minimum Freeboard Schematic

4.3.2 Wind Setup

Based on Equation 1 and the input parameters presented in Table 5, the wind setup for the minimum freeboard was calculated to be 0.03 m for both the HIGH and EXTREME consequence classification.

4.3.3 Wave Run-up

Based on the input parameters in Table 5, and Equation 2, the wave run-up for the minimum freeboard was calculated to be 1.00 m for both the HIGH and EXTREME consequence classification.

4.3.4 Minimum Freeboard Estimate

Subsequently the minimum freeboard for the North Dam is 1.03 m, irrespective of the consequence classification.

4.4 Combined Freeboard

4.4.1 Definition

The combined freeboard is the required freeboard to protect against the IDF and the wave run-up from a 1 in 1,000 year wind storm event. This is essentially a combination of the normal freeboard and minimum freeboard criteria for the EXTREME dam classification, where an extreme pond level and high wind are considered, based on the design parameters listed in Table 5.

4.4.2 IDF

As described in Section 3.5, at the end of the mine life when the Reclaim Pond is at its smallest, the IDF's listed in Table 2 translates to 2.1 m water level increases over the design FSL of 33.5 m.

4.4.3 Wind Setup

Based on Equation 1 and the input parameters presented in Table 5, the wind setup for the combined freeboard was calculated to be 0.06 m.

4.4.4 Wave Run-up

Based on the input parameters in Table 5, and Equation 2, the wave run-up for the combined freeboard was calculated to be 1.16.

4.4.5 Combined Freeboard Estimate

Subsequently the combined freeboard for the North Dam is 3.3 m (IDF of 2.1 m + wind setup of 0.06 m + wave run-up of 1.16 m), designed to retain the IDF without the need for a spillway and withstand the wave action resulting from strong winds.

5 Results

The normal and minimum freeboard results for the North Dam are presented in Table 6.

Table 6: Freeboard Results

Freeboard Condition	Initial Water Level (m)	Wind Setup (m)	Wave Run-up (m)	Freeboard (m)	Final Freeboard Elevation (m)
Normal Freeboard	33.5	0.07	1.06	1.13	34.6
Minimum Freeboard HIGH Consequence Classification	35.2	0.03	1.00	1.03	36.2
Minimum Freeboard EXTREME Classification	35.6	0.03	1.00	1.03	36.6
Combined Freeboard EXTREME Classification	35.6	0.06	1.16	1.22	36.8

These freeboard results are consistent with typical values suggested by USBR (2012) where for a fetch smaller than 1,600 m (1 mile) a normal freeboard of 1.2 m (4 ft.) and minimum freeboard of 0.9 m (3 ft.) should be expected.

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

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Appendix A: Reference Material

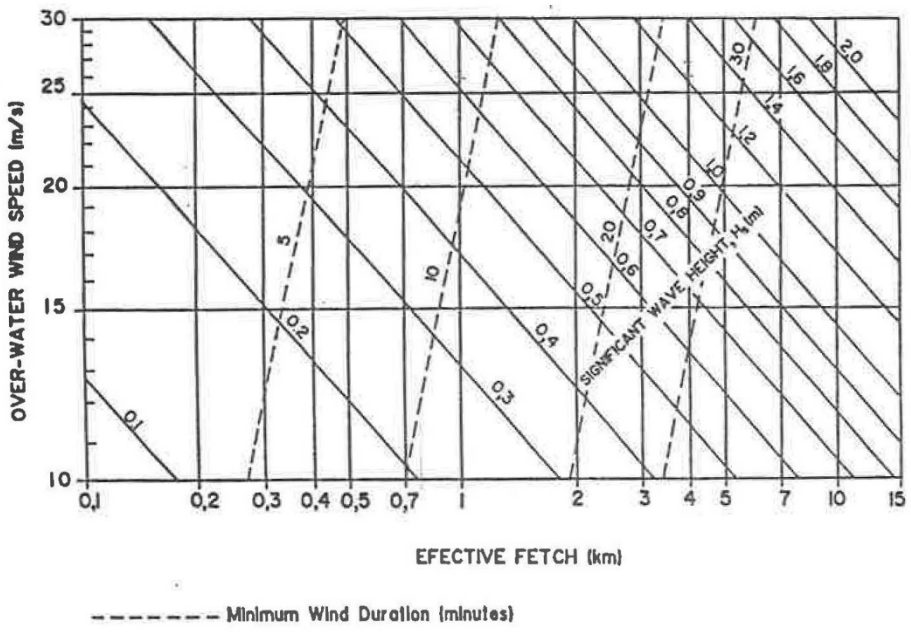


FIGURE A4 : SIGNIFICANT WAVE HEIGHTS

Reference A-1: Significant Wave Height (SANCOLD, 1990)

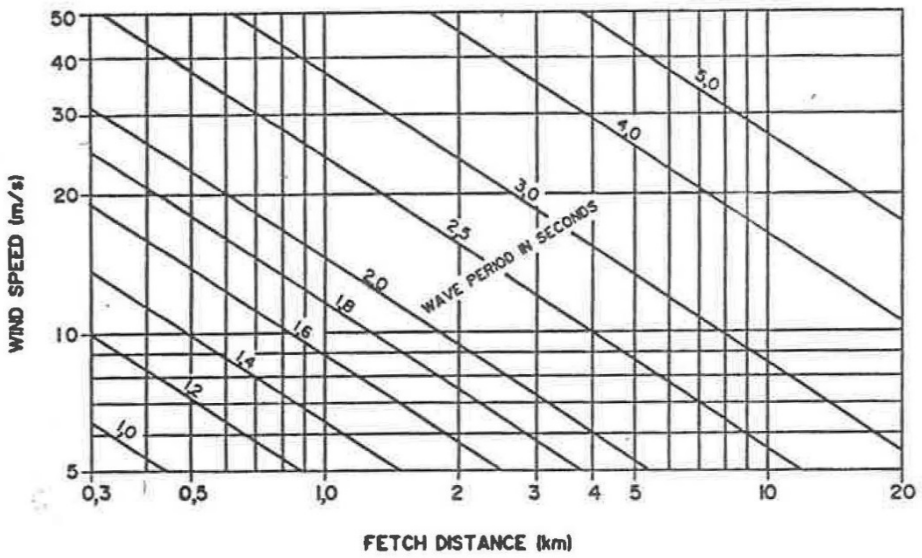
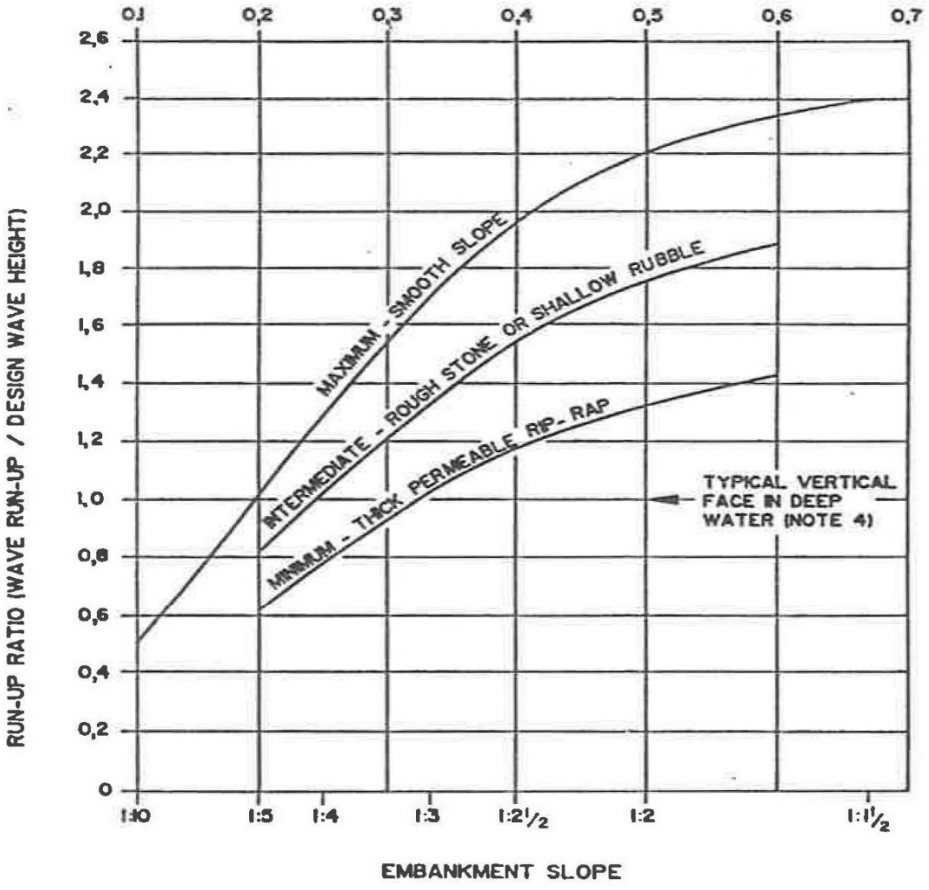


FIGURE A3 : WIND WAVE PERIODS

Reference A-2: Significant Wave Period (SANCOLD, 1990)

Fetch, km (miles)	Wind Ratio Over Water/Over Land
0.8 (0.5)	1.08
1.6 (1)	1.13
3.2 (2)	1.21
4.8 (3)	1.26
6.4 (4)	1.28
8.0 (5)	1.30

Reference A-3: Wind Over Water Correction Factors Based on Fetch Length (USACE 1997).



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.

Reference A-4: Wave run-up estimation based on Run-up Ratio and Embankment Slope (SANCOLD, 1990)

Appendix G – Hope Bay Project: Doris, Madrid and Boston Tailings Geotechnical Properties

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Erik Ketilson, PEng Trevor Podaima, PEng	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project - Doris, Madrid and Boston Tailings Geotechnical Properties		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix G as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes.		

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Two tailings areas are planned for Phase 2. The existing Doris tailings impoundment area (TIA) will be expanded, and a new Boston tailings management area (TMA) will be developed. Doris TIA tailings deposition will consist of subaerial tailings deposition, while the Boston TMA will be comprised of filtered tailings developed as a dry-stack. Two tailings streams will be produced; flotation tailings, comprising approximately 92-94% of the overall volume, and detoxified leach tailings (following cyanidation, and subsequent cyanide destruction), comprising about 6-8% of the overall volume. Only flotation tailings will be deposited in the Doris and Boston tailings areas. The detoxified leach tailings will be filtered, mixed with mine waste rock and used for underground mine backfill.

1.2 Objective

Dating back to 2003, there have been multiple campaigns of tailings geotechnical testing carried out for the Project. This memo summarizes all this information, and serves to provide definitive geotechnical design data for the Project with respect to tailings properties.

2 Tailings Property Test Programs

Three separate tailings geotechnical test campaigns have been carried out between 2003 and 2009, as outlined in Table 2.1.

Table 2.1: Hope Bay Tailings Geotechnical Test Programs

Testing Date	Testing Agency	Tailings Tested		
		Area	Type	Method
2003	AMEC Earth Engineering Pty. Limited in Perth Australia	Doris	Mixed detoxified leach / flotation	Slurry
2009	Knight Piésold Consulting (Denver)	Doris Central	Flotation only Mixed detoxified leach / flotation Detoxified leach only	Slurry
		Boston	Flotation only Mixed detoxified leach / flotation Detoxified leach only	
		Madrid	Flotation	
	Pocock Industrial Inc. (Denver)	Doris Central	Flotation only Mixed detoxified leach / flotation Detoxified leach only	Thickened
		Boston	Flotation only Mixed detoxified leach / flotation Detoxified leach only	
		Madrid	Floatation	

Table 2.2 summarizes the specific testing completed during the 2003 test campaign (SRK 2003). A single combined tailings sample (mixture of flotation and detoxified leach tailings) was prepared by Bateman Engineering from pilot scale metallurgical tests.

Table 2.2: 2003 Laboratory Tests on Single Combined Tailings Sample

Test	Test Method (Australian Standards)
Grain Size Distribution	Sieve and Hydrometer (AS 1289.3.6.2)
Plastic Properties	Casagrande (AS 1289.3.1.1,.3.2.1,.3.3.1,.3.4.1,.2.1.1)
Particle Density	AS 1289.3.5.1
Triaxial Test	Consolidated Undrained Test with Pore Pressure Measurement (AS 1289.6.4.2)
Consolidation Test	One-Dimensional Consolidation (AS 1289.6.6.1)
Undrained Settling Test	SRC-WI-4.8.3
Drained Settling Test	SRC-WI-4.8.2

Table 2.3 summarizes the laboratory testing completed by Knight Piésold Consulting in 2009 (KP 2009). These tailings samples were prepared by Newmont Metallurgical Services (NMS) as part of pilot scale testing in their Denver laboratories.

Table 2.3: 2009 Laboratory Tests Conducted by Knight Piésold Consulting

Test	Test Method	Number of Tests
Grain Size Distribution	ASTM D 422	9
Plastic Properties	Not Specified	9
Specific Gravity	ASTM D 854	8
Undrained Settling Test	Lab specific procedure – no specific ASTM	12
Drained Settling & Falling Head Permeability Test	Lab specific procedure – no specific ASTM	12
Settling and Drying Test	Lab specific procedure – no specific ASTM	12

NMS also submitted samples to Pocock Industrial Inc. for additional laboratory testing associated with flocculent screening, gravity sedimentation, pulp rheology, pressure filtration, and vacuum filtration (Pocock 2009).

3 Tailings Properties Used Historically

Different tailings management strategies have been assessed for the Project between 2003 and 2015, and many different geotechnical parameters for tailings were used in these designs. Some of these were based on the test data listed in Section 2, while others were based on engineering judgement. Table 3.1 summarizes these values.

Table 3.1: Tailings Geotechnical Properties Used Historically

Parameter		Reported Values					
		2003 & 2005	**2007**	**2009**	**2011**	**2012**	**2015**
Reference(s)		SRK (2003, 2005)	SRK (2007)	SRK (2009)	SRK (2011)	SRK (2012)	SRK (2015)
Depositional Strategy		Sub-Aqueous	Sub-Aqueous	Thickened	Sub-Aqueous	Not Specified	Sub-Aerial
% Solids		36.1%	36.1%	57.0%	65.0%	65.0%	36.1%
Specific Gravity		2.7	2.7	-	-	2.9	2.7
Fines content (<75 micron)		56%	56%	-	-	-	-
Clay content (< 2 microns)		11%	11%	-	-	-	-
Plasticity		Non-plastic	Non-plastic	-	-	-	-
Moist Unit Weight (kN/m^3)		-	-	-	-	-	17.5
Void Ratio	Sub-Aqueous	1.20	1.20	-	-	1.60	-
	Sub-Aerial	-	-	-	-	1.35	-
	Paste/thickened	-	-	-	-	1.10	-
	Filtered	-	-	-	-	0.90	-
In-situ Dry Density	Sub-Aqueous (t/m^3)	1.23	1.23	-	1.29	1.12	-
	Sub-Aerial (t/m^3)	-	-	-	-	1.24	1.29
	Paste/thickened (t/m^3)	-	-	1.36	-	1.39	-
	Filtered (t/m^3)	-	-	-	-	1.53	-
Hydraulic conductivity (m/s)		1 x 10^{-7}	1 x 10^{-7}		-	-	5.4 x 10^{-7}
Friction Angle (degrees)		-	-	-	-	-	40º
Cohesion (kPa)		-	-	-	-	-	0

4 Tailings Geotechnical Test Results

4.1 Index Properties

Index properties include specific gravity (Table 4.1), particle size distribution (Table 4.2) and Atterberg limits (Table 4.3).

Table 4.1: Tailings Specific Gravity

Test Date	Source	Tailings Type	Specific Gravity		
			AMEC (2003)	KP (2009)	Pockock (2009)
2003	Doris North	Flotation (34% Solids)	2.74	-	-
2009	Doris Central	Flotation (38% Solids)	-	2.76	2.82
	Boston		-	2.90	2.89
	Madrid (Suluk)		-	2.90	2.75
	Madrid (Naartok East)		-	2.84	2.80
	Madrid (Naartok West)		-	2.87	2.86
Average Flotation Tailings			**2.83**		
2009	Doris Central	Mixed Detoxified Leach / Flotation	-	2.85	2.87
	Boston		-	2.97	2.90
Average Mixed Detoxified Leach / Flotation Tailings			**2.90**		
Average of Flotation and Mixed Detoxified Leach / Flotation Tailings			**2.85**		
2009	Doris Central	Detoxified Leach Tailings	-	3.46	3.42
	Boston		-	3.22	3.26
Average Detoxified Leach Tailings			**3.34**		

Table 4.2: Particle Size Distribution Testing Results

Test Date	Source	Tailings Type	USCS Classification[1]	Fines (< 0.075 mm)	%Silt	%Clay	D_{15} (mm)	D_{50} (mm)
2003	Doris North	Flotation (34% Solids)	ML	56%	46%	10%	0.011	0.068
2009	Doris Central	Flotation (38% Solids)	ML	63%	51%	11%	0.003	0.040
	Boston		ML	55%	43%	12%	0.003	0.061
	Madrid (Suluk)		CL-ML	74%	59%	15%	0.002	0.020
	Madrid (Naartok East)		SM	75%	61%	14%	0.002	0.022
	Madrid (Naartok West)		ML	69%	56%	13%	0.003	0.034
Average Flotation Tailings			**ML**	**65%**	**53%**	**13%**	**0.004**	**0.041**
2009	Doris Central	Mixed Detoxified Leach / Flotation	ML	65%	52%	13%	0.003	0.030
	Boston		ML	63%	50%	13%	0.002	0.035
Average Mixed Detoxified Leach / Flotation Tailings			**ML**	**64%**	**51%**	**13%**	**0.003**	**0.033**
Average of Flotation and Mixed Detoxified Leach / Flotation Tailings			**ML**	**65%**	**52%**	**13%**	**0.004**	**0.039**
2009	Doris Central	Detoxified Leach Tailings	ML	99%	84%	15%	0.002	0.010
	Boston		CL	100%	80%	20%	0.001	0.009
Average Detoxified Leach Tailings			**CL-ML**	**100%**	**82%**	**18%**	**0.002**	**0.010**

Notes:
1. USCS: ML = Silt, CL = Clay, SM = Silty Sand

Table 4.3: Atterberg Limits

Test Date	Sample	Tailings Type	Plasticity Limit (PL %)	Liquid Limit (LL %)	Plasticity Index (PI %)
2003	Doris North	Flotation (34% Solids)	NP	ND	NP
2009	Doris Central	Flotation (38% Solids)	NP	16	NP
	Boston		NP	17	NP
	Madrid (Suluk)		15	19	4
	Madrid (Naartok East)		NP	17	NP
	Madrid (Naartok West)		NP	17	NP
	Doris Central	Mixed Detoxified Leach / Flotation	14	17	3
	Boston		14	17	3
	Doris Central	Detoxified Leach Tailings	22	22	NP
	Boston		20	28	8

Notes:
1. NP = Non-Plastic
2. ND = Not able to be determined

4.2 Consolidation, Saturated Hydraulic Conductivity and Strength

Consolidation and triaxial testing completed on the single 2003 sample (AMEC 2003) are summarized in Table 4.4 and Table 4.5, respectively. After measuring the saturated hydraulic conductivity at 400 kPa confining stress, within the triaxial cell, the tailings sample was axially loaded to failure under undrained conditions to measure the frictional strength. These results are summarized in Table 4.7.

Falling head permeability testing (saturated hydraulic conductivity) was completed in 2009, concurrently with drained settling tests (Section 4.3), with the results summarized in Table 4.6.

Table 4.4: One-Dimensional Consolidation Properties of Tailings (AMEC 2003)

Pressure (kPa)	Void Ratio, e (-)	Coefficient of Consolidation, C_v (m^2/year)	Coefficient of Volume Compressibility, M_v, (m^2/kN)	Saturated Hydraulic Conductivity, K (m/sec)
0	0.839	-	-	-
20	0.836	0.648	8.170×10^{-5}	1.69×10^{-11}
40	0.830	0.529	8.183×10^{-5}	1.37×10^{-11}
100	0.833	0.488	2.732×10^{-5}	0.42×10^{-11}
200	0.824	0.455	3.289×10^{-5}	0.47×10^{-11}
300	0.811	0.439	7.178×10^{-5}	9.98×10^{-12}
400	0.797	0.441	7.791×10^{-5}	1.11×10^{-12}
200	0.798	-	-	-
40	0.803	-	-	-

Notes:
1. Initial dry density = 1.49 t/m^3; final dry density – 1.54 t/m^3
2. Initial moisture content = 22.6%, final moisture content = 13.4%
3. Initial saturation = 73.8%; final saturation = 47.3%

Table 4.5: Undrained Triaxial Compression Test Data (AMEC 2003)

Stage (Confining Stress, σ_3 (kPa))	Coefficient of Consolidation, C_v (m^2/year)	Coeff. of Volume Compressibility, M_v, (m^2/kN)	Hydraulic Conductivity, K (m/sec)	Cohesion, c (kPa)
1 (250 kPa)	113,890	0.153	5.4×10^{-7}	0.825
2 (300 kPa)	3,388	0.108	1.1×10^{-7}	0.010
3 (400 kPa)	1,822	0.040	2.2×10^{-8}	0.007

Table 4.6: Hydraulic Conductivity Testing (KP 2009)

Test Date	Source	Tailings Type	Falling Head Permeability, $k_{ave.}$ (m/sec)
2003	Doris North	Flotation - 34%	NT
2009	Doris Central	Flotation - 38%	2.1×10^{-7}
	Boston		2.0×10^{-7}
	Madrid (Suluk)		7.8×10^{-8}
	Madrid (Naartok East)		1.4×10^{-7}
	Madrid (Naartok West)		1.3×10^{-7}
Average Flotation Tailings			**1.5×10^{-7}**
Maximum Flotation Tailings			**2.1×10^{-7}**
Minimum Flotation Tailings			**7.8×10^{-8}**
2009	Doris Central	Mixed Detoxified Leach / Flotation (3 Samples)	9.2×10^{-8}; 8.0×10^{-8}; 5.4×10^{-8}
		Average of 3 Samples	*7.5×10^{-8}*
	Boston	Mixed Detoxified Leach / Flotation (3 Samples)	7.9×10^{-8}; 6.8×10^{-8}; 4.6×10^{-8}
		Average of 3 Samples	*6.4×10^{-8}*
Average Mixed Detoxified Leach / Flotation Tailings			**7.0×10^{-8}**
Maximum Mixed Detoxified Leach / Flotation Tailings			**7.5×10^{-8}**
Minimum Mixed Detoxified Leach / Flotation Tailings			**6.4×10^{-8}**
Average of Flotation and Mixed Detoxified Leach / Flotation Tailings			**1.3×10^{-7}**
Maximum of Flotation and Mixed Detoxified Leach / Flotation Tailings			**2.1×10^{-7}**
Minimum of Flotation and Mixed Detoxified Leach / Flotation Tailings			**6.4×10^{-8}**
2009	Doris Central	Detoxified Leach Tailings	Not Reported
	Boston		9.2×10^{-8}

Notes:
1. NT = Not Tested

Table 4.7: Tailings Shear Strength (AMEC 2003)

Parameter	Stage 1	Stage 2	Stage 3
Confining Stress, σ_3 (kPa)	250	300	400
Porewater Pressure, U (kPa)	222	137	134
Effective Confining Stress, (σ_3 – U) (kPa)	28	163	266
Deviator Stress, (σ_1 - σ_3) (kPa)	128	706	1,163
Shear Stress (σ_1 - σ_3)/2 (kPa)	64	353	582
Internal Friction, Φ (degrees)	43.2		
Cohesion, c (kPa)	1.0		
Moisture Content (%)	22.5		
Dry Density (t/m^3)	1.49		
Void Ratio	0.839		
Saturation (%)	73.0		

4.3 Tailings Settling Properties

Undrained and drained settling tests were completed in 2003 (AMEC 2003), while undrained, drained, settling and evaporation tests were completed in 2009 (KP 2009). Settling tests are typically performed in support of determining the amount of fluid released and/or retained by the tailings, and the time required for particles to settle. Undrained settling tests restrict drainage through the base of the sample, while drained settling tests allow water to drain through the base of the sample. The tests also provide an indication of the unconsolidated dry tailings density. Complete summarized results are presented in Table 4.8, Table 4.9, and Table 4.10. Settling times are not discussed in this memo (data provided in AMEC 2003 and KP 2009).

Table 4.8: Tailings Undrained Settling Properties

Testing Date	Sample Source	Tailings Type	Undrained Settling Test							
			Initial			Final			Total Water Recovery (%)	Portion of Initial Water Retained in Tailings prior to Evap (%)
			Solids Content (%)	Slurry Dry Density (t/m^3)	Moisture Content (%)	Solids content (%)	Slurry Dry Density (t/m^3)	Moisture Content (%)		
2003	Doris Tailings	Flotation - 34%	33.8%	0.428	195.6%	68.1%	1.19	46.9%	76%	24.1%
2009	Doris Central	Flotation - 38%	38.7%	0.515	158.6%	67.9%	1.20	47.4%	70%	29.9%
2009	Boston	Flotation - 38%	38.0%	0.505	162.7%	66.2%	1.16	51.2%	69%	31.4%
2009	Madrid - Suluk	Flotation - 38%	37.9%	0.503	163.7%	69.7%	1.27	43.5%	73%	26.6%
2009	Madrid - Naartok East	Flotation - 38%	38.3%	0.512	161.0%	70.1%	1.30	42.6%	74%	26.5%
2009	Madrid - Naartok West	Flotation - 38%	38.0%	0.614	162.6%	68.0%	1.22	47.0%	71%	29.0%
Average Flotation Tailings			**37.5%**	**0.513**	**167.4%**	**68.3%**	**1.22**	**46.4%**	**72%**	**27.9%**
2009	Doris Central	Mixed tails (CND/Flot.) - Sample 1	65.1%	1.112	53.7%	75.0%	1.44	33.3%	38%	61.9%
		Mixed tails (CND/Flot.) - Sample 2	69.8%	1.263	43.2%	75.5%	1.48	32.4%	25%	75.2%
		Mixed tails (CND/Flot.) - Sample 3	74.7%	1.436	33.9%	78.4%	1.58	27.5%	19%	81.1%
		Average	*69.8%*	*1.270*	*43.6%*	*76.3*	*1.50*	*31.1%*	*27%*	*72.7%*
2009	Boston	Mixed tails (CND/Flot.) - Sample 1	64.7%	1.114	54.7%	74.2%	1.44	34.9%	36%	63.6%
		Mixed tails (CND/Flot.) - Sample 2	70.1%	1.282	42.5%	76.5%	1.53	30.8%	28%	72.4%
		Mixed tails (CND/Flot.) - Sample 3	75.0%	1.481	33.3%	78.0%	1.60	28.2%	15%	84.5%
		Average	*69.9%*	*1.292*	*43.5%*	*76.2%*	*1.52*	*31.3%*	*26%*	*73.5%*
Average Mixed Detoxified Leach / Flotation Tailings			**69.9%**	**1.281**	**43.6%**	**76.3%**	**1.51**	**31.2%**	**27%**	**73.1%**
Average of Flotation and Mixed Detoxified Leach / Flotation Tailings[3]			**45.6%**	**0.705**	**136.4%**	**7**	**1.30**	**42.6%**	**61%**	**39.2%**
2009	Doris Central	Detox tailings	39.9%	NR	NR	NR	1.20	NR	65%	35.1%
2009	Boston	Cyanidation - 45%	45.0%	0.647	122.4%	61.6%	1.06	62.4%	49%	51.0%
Average Detoxified Leach Tailings			**42.5%**	**0.647**	**122.4%**	**61.6%**	**1.13**	**62.4%**	**57%**	**43.1%**

Notes:
1. NT = Not Tested
2. NR = Not Recorded
3. Average is based on the flotation samples and the average as reported for the Mixed tailings

Table 4.9: Tailings Drained Settling Properties

Testing Date	Sample Source	Tailings Type	Drained Settling Test & Falling Head Permeability								
			Initial			Final			Total Water Recovery (%)	Portion of Initial Water Retained in Tailings prior to Evap (%)	$k_{ave.}$ (m/sec)
			Solids Content (%)	Slurry Dry Density (t/m^3)	Moisture Content (%)	Solids content (%)	Slurry Dry Density (t/m^3)	Moisture Content (%)			
2003	Doris Tailings	Flotation - 34%	33.8%	0.044	195.6%	77.9%	1.43	28.4%	85%	14.5%	
2009	Doris Central	Flotation - 38%	37.9%	0.501	164.0%	71.2%	1.31	40%	75%	24.7%	2.1x10^{-7}
2009	Boston	Flotation - 38%	37.8%	0.501	164.4%	75.5%	1.23	33%	80%	19.7%	2.0 x10^{-7}
2009	Madrid - Suluk	Flotation - 38%	38.2%	0.505	161.9%	73.3%	1.40	36%	78%	22.2%	7.8 x10^{-8}
2009	Madrid - Naartok East	Flotation - 38%	38.0%	0.502	163.6%	74.1%	1.43	35%	79%	21%	1.4 x10^{-7}
2009	Madrid - Naartok West	Flotation - 38%	37.9%	0.503	163.9%	72.6%	1.38	38%	77%	23%	1.3 x10^{-7}
		Average Flotation Tailings	**37.3%**	**0.426**	**168.9%**	**74.1%**	**1.36**	**35%**	**79%**	**21%**	**1.5 x10^{-7}**
2009	Doris Central	Mixed tails (CND/Flot.) - Sample 1	65.2%	1.116	53.4%	77.2%	1.52	30%	44%	56.2%	9.2 x10^{-8}
		Mixed tails (CND/Flot.) - Sample 2	69.9%	1.260	43.2%	78.1%	1.57	28%	35%	64.7%	8.0 x10^{-8}
		Mixed tails (CND/Flot.) - Sample 3	74.8%	1.432	33.8%	79.5%	1.63	26%	22%	77.6%	5.4 x10^{-8}
		Average	*69.9%*	*1.269*	*43.5%*	*78.3%*	*1.57*	*28%*	*34%*	*66.2%*	*7.5 x10^{-8}*
2009	Boston	Mixed tails (CND/Flot.) - Sample 1	64.9%	1.128	54.0%	77.0%	1.54	30%	44%	55.5%	7.9 x10^{-8}
		Mixed tails (CND/Flot.) - Sample 2	69.9%	1.289	42.9%	78.9%	1.61	27%	36%	63.6%	6.8 x10^{-8}
		Mixed tails (CND/Flot.) - Sample 3	74.9%	1.462	33.6%	79.4%	1.66	26%	23%	77.1%	4.6 x10^{-8}
		Average	*69.9%*	1.293	*43.5%*	*78.4%*	*1.60*	*28%*	*34%*	*65.4%*	*6.4 x10^{-8}*
		Average Mixed Detoxified Leach / Flotation Tailings	**69.9%**	**1.281**	**43.5%**	**78.4%**	**1.59**	**28%**	**34%**	**66%**	**7.0 x10^{-8}**
		Average of Flotation and Mixed Detoxified Leach / Flotation Tailings (3)	**45.4%**	**42.6%**	**168.9%**	**74.1%**	**1.42**	**35.1%**	**68%**	**32%**	**1.3 x10^{-7}**
2009	Doris Central	Detox tailings	40.2%	NR	NR	NR	1.37	NR	70.8%	29.2%	NR
2009	Boston	Cyanidation - 45%	44.9%	0.629	122.7%	66.1%	1.19	51%	59%	41.3%	9.2 x10^{-8}
		Average Detoxified Leach Tailings	**42.6%**	**0.629**	**122.7%**	**66.1%**	**1.28**	**51%**	**65%**	**35.3%**	**9.2 x10^{-8}**

Notes:
1. NT = Not Tested
2. NR = Not Recorded
3. Average is based on the flotation samples and the average as reported for the Mixed tailings

Table 4.10: Tailings Settling and Drying Properties

Testing Date	Sample Source	Tailings Type	Settling and Drying Test				
			Initial	Final			Total Evaporation (mm)
			Solids Content (%)	Solids Content (%)	Slurry Dry Density (t/m^3)	Moisture Content (%)	
2003	Doris Tailings	Flotation - 34%	NT	NT	NT	NT	NT
2009	Doris Central	Flotation - 38%	38.1%	99.1%	1.88	0.9%	27.3
2009	Boston	Flotation - 38%	37.9%	90.0%	1.84	11.1%	21.9
2009	Madrid - Suluk	Flotation - 38%	37.6%	98.2%	1.69	1.8%	28.1
2009	Madrid - Naartok East	Flotation - 38%	37.8%	87.1%	1.98	14.8%	18.4
2009	Madrid - Naartok West	Flotation - 38%	37.8%	99.1%	1.74	0.9%	32.4
		Average Flotation Tailings	**37.8%**	**94.7%**	**1.83**	**5.9%**	**25.6**
2009	Doris Central	Mixed tails (CND/Flot.) - Sample 1	64.9%	96.7%	1.76	3.4%	32.8
		Mixed tails (CND/Flot.) - Sample 2	69.8%	98.2%	1.83	1.8%	32.8
		Mixed tails (CND/Flot.) - Sample 3	74.7%	97.0%	1.85	3.1%	32.8
		Average	*69.8%*	*97.3%*	*1.81*	2.77%	*32.8*
2009	Boston	Mixed tails (CND/Flot.) - Sample 1	64.7%	96.2%	1.85	4.0%	38.6
		Mixed tails (CND/Flot.) - Sample 2	69.8%	91.5%	1.88	9.3%	30.6
		Mixed tails (CND/Flot.) - Sample 3	74.8%	92.7%	1.92	7.9%	30.6
		Average	*69.8%*	*93.5%*	*1.88*	7.1%	*33.3*
		Average Mixed Detoxified Leach / Flotation Tailings	**69.8%**	**95.4%**	**1.85**	**4.9%**	**33.0**
		Average of Flotation and Mixed Detoxified Leach / Flotation Tailings (3)	**47.0%**	**94.7%**	**1.83**	**5.9%**	**27.7**
2009	Doris Central	Detox tailings	39.7%	NR	1.72	NR	46.7
2009	Boston	Cyanidation - 45%	44.7%	98.6%	1.66	1.4%	49.2
		Average Detoxified Leach Tailings	**42.2%**	**98.6%**	**1.69**	**1.4%**	**48.0**

Notes:
1. NT = Not Tested
2. NR = Not Recorded
3. Average is based on the flotation samples and the average as reported for the Mixed tailings

4.4 Vacuum and Pressure Filtration Data

Vacuum and pressure filtration work tests (Pocock 2009) are summarized in Table 4.11. It should be noted that in many of the scenarios tested, the test objective was to optimize tailings filtration production, with subsequent sizing of the filtration plant. As such, the reported dry densities and moisture contents are conservative. It is also important to consider that the reported data is representative of the material as it comes out of the filtration plant, and it can be assumed that typically lower moisture contents and higher in-situ densities are achieved following placement and compaction of the material.

Table 4.11: Vacuum and Pressure Filtration Data

Sample Source	Tailings Type	SG	Filtration Tailings (from Vacuum Filter Testing)					Filtration Tailings (from Pressure Filter Testing)		
			Filter Feed Solids (%)	No Flocculent		With Flocculent		Filter Feed Solids (%)	Dry Density (t/m^3)	Cake Moisture (%)
				Cake Dry Density (t/m^3)	Cake Moisture (%)	Cake dry Density (t/m^3)	Cake Moisture (%)			
Boston	Flotation	2.89	59.7%	1.93	18%	1.52	22%	59.1%	1.89	12.5 / 13.4 / 15.1
		2.89	38.6%	1.68	21%	1.52	24%	NR	NR	NR
Doris Central	Flotation	2.82	59.7%	1.78	18%	1.59	21%	58.5%	1.83	12.3 / 13.0 / 14.4
		2.82	38.9%	1.78	22%	1.59	23%	NR	NR	NR
Madrid - Suluk	Flotation	2.75	58.5%	1.84	18%	1.6	23%	38.5%	1.71	14.7 / 15.4 / 16.8
		2.75	38.1%	1.84	19%	1.6	26%	59.1%	1.83	13.7 / 14.6 / 16.4
Madrid - Naartok East	Flotation	2.8	59.0%	1.88	18%	1.46	22%	42.3%	1.72	13.1 / 13.7 / 15.0
		2.8	38.6%	1.88	19%	1.46	23%	59.2%	1.84	13.4 / 14.2 / 15.8
Madrid - Naartok West	Flotation	2.86	59.4%	1.91	18%	1.66	22%	39.4%	1.7	13.5 / 14.1 / 15.0
		2.86	38.6%	1.91	20%	1.66	24%	60.4%	1.91	12.9 / 13.6 / 15.0
Average Flotation Tailings		**2.82**	**48.9%**	**1.84**	**19%**	**1.57**	**23%**	**25.4%**	**1.8**	**14.2%**
Boston	Mixed tails (CND/Flot)	2.9	59.8%	1.83	19%	1.49	23%	39.5%	1.74	14.2 / 14.9 / 16.1
		2.9	39.1%	1.83	20%	1.49	27%	60.1%	1.74	13.2 / 14.0 / 15.5
Doris Central	Mixed tails (CND/Flot)	2.87	59.8%	1.84	19%	1.52	23%	41.4%	1.75	14.0 / 15.0 / 16.7
		2.87	39.5%	1.84	21%	1.52	24%	60.7%	1.75	12.4 / 13.5 / 15.6
Average Mixed Detoxified Leach / Flotation Tailings		**2.89**	**50.0%**	**1.84**	**20%**	**1.51**	**24%**	**50.4%**	**1.75**	**14.6%**
Average Flotation and Mixed Detoxified Leach / Flotation Tailings[(3)]		**2.84**	**49.1%**	**1.84**	**19%**	**1.55**	**23%**	**17.1%**	**1.78**	**14.4%**

Table 4.11: Vacuum and Pressure Filtration Data (Continued)

Sample Source	Tailings Type	SG	Filtration Tailings (from Vacuum Filter Testing)					Filtration Tailings (from Pressure Filter Testing)		
			Filter Feed Solids (%)	No Flocculent		With Flocculent		Filter Feed Solids (%)	Dry Density (t/m^3)	Cake Moisture (%)
				Cake Dry Density (t/m^3)	Cake Moisture (%)	Cake dry Density (t/m^3)	Cake Moisture (%)			
Boston	CIL Residue	3.26	NR	NR	NR	NR	NR	41.4%	1.79	19.0%
		3.26	NR	NR	NR	NR	NR	61.0%	1.79	18.0%
	Cyanidation	NR	NR	NR	NR	NR	NR	42.1%	1.69	19.0%
		NR	NR	NR	NR	NR	NR	61.3%	1.69	18.0%
Doris Central	CIL Residue	3.42	NR	NR	NR	NR	NR	41.2%	1.84	17.0%
		3.42	NR	NR	NR	NR	NR	59.2%	1.84	17.0%
	Cyanidation	NR	59.6%	1.76	24%	1.52	29%	53.4%	1.76	18.0%
		NR	39.7%	1.76	24%	1.52	32%	NR	NR	NR
Average Detoxified Leach Tailings		**3.34**	**49.7%**	**1.76**	**24%**	**1.52**	**31%**	**51.4%**	**1.77**	**18.0%**

Notes:
1. NT = Not Tested
2. NR = Not Recorded
3. Average is based on the flotation samples and the average as reported for the Mixed tailings

4.5 Deposited Tailings Densities

The deposited tailings density is dependent on both the specific gravity of the particles and the void ratio of the resulting deposited material. The densities measured in the undrained settling tests result in an average of 1.3 t/m^3 (Table 4.8), and based on an average specific gravity of 2.85 (Table 4.1), the resulting void ratio is 1.2. This is consistent with typical void ratios for slurry deposited gold tailings to be between 1.1 and 1.2 (Vick 1990), and is consistent with the observed dry density versus void ratio relationship as illustrated in Figure 4.1. The data reported in Figure 4.1 is representative of all reported dry densities in Table 4.8, Table 4.9, Table 4.10, and Table 4.11. This approach does not factor the potential increase in dry density due to consolidation of the tailings mass, which is considered appropriate as much of the tailings is anticipated to freeze shortly after deposition, and consolidation of the tailings may not be possible. Therefore an average dry density of placed slurried tailings of 1.3 t/m^3 is deemed appropriate for the Project tailings.

The available pressure filtration data of the Boston ore indicates an average dry density of 1.8 t/m^3 for the tailings coming from the filtration plan (Table 4.11). Based on the reported relationship in Figure 4.1, the resulting void ratio would be approximately 0.6, which results in a gravimetric water content of approximately 20.5%, and correlates with a degree of saturation of 100%. As the drystack will be constructed with a degree of saturation less than 100%, these values are considered conservative.

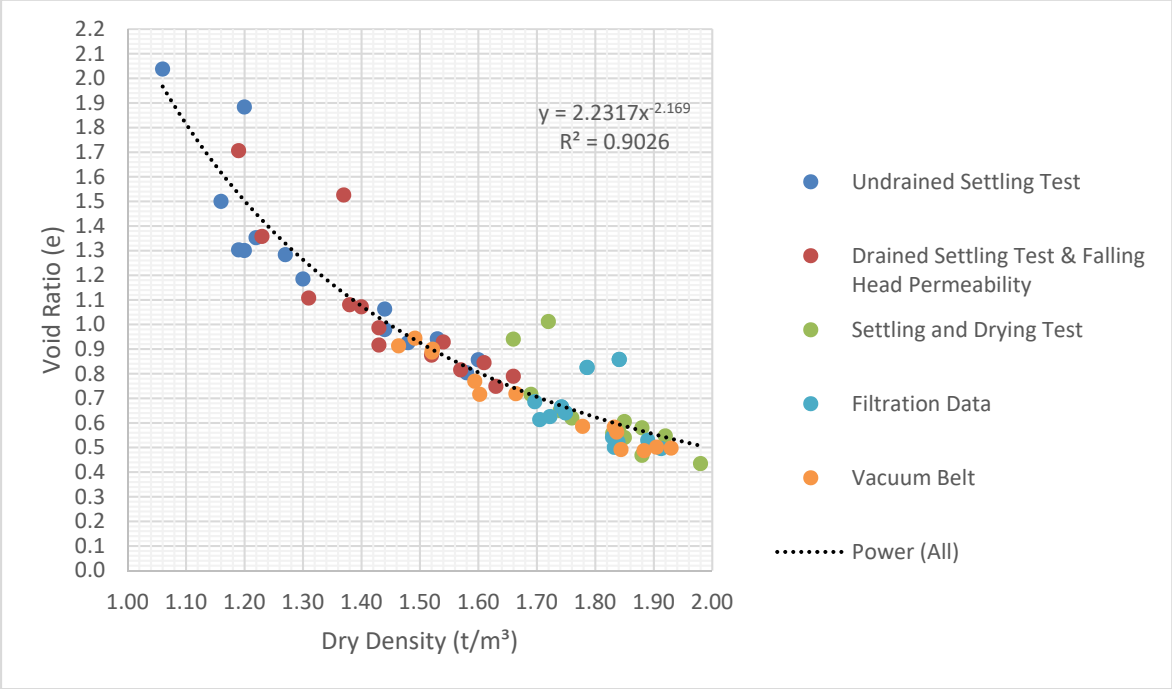

Figure 4.1: Measured Dry Density versus Void Ratio

Figure 4.2 and Figure 4.3 demonstrate the observed relationship between dry density and gravimetric moisture content, and gravimetric moisture content and void ratio, respectively. It can be observed that the obtained final results from the settling and drying tests are not consistent

with the otherwise observed trends in the data. This is consistent with our expectations, as the settling and drying tests evaluate the maximum amount of water that can be drained and evaporated from the tailings samples, which can result in an appropriately low moisture content. The data from the settling and drying tests has been included on Figure 4.2, and Figure 4.3 in the interest of reporting all available data.

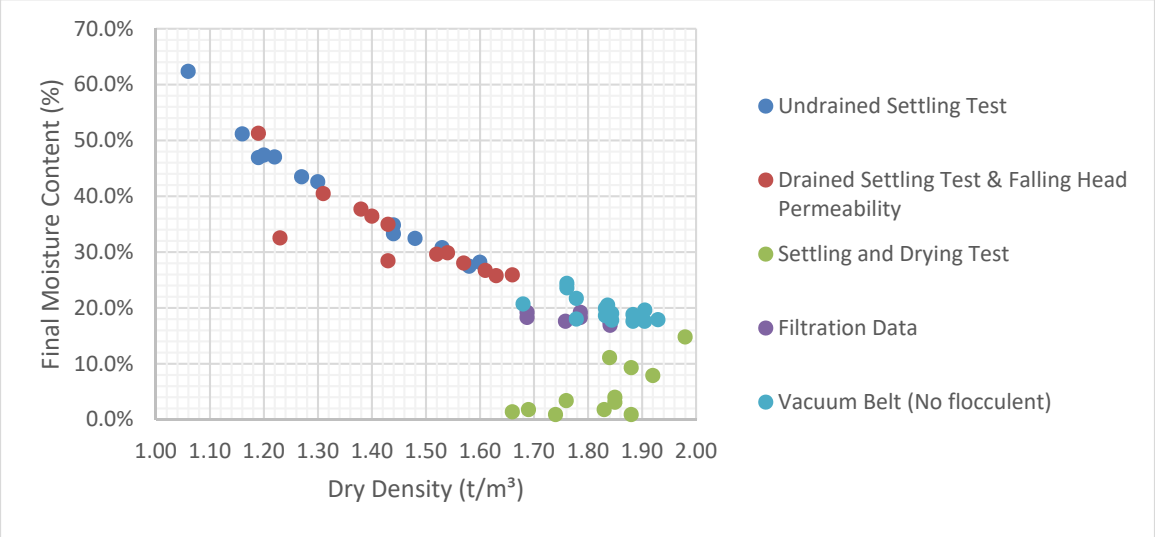

Figure 4.2: Measured Dry Density versus Measured Moisture Content (Final)

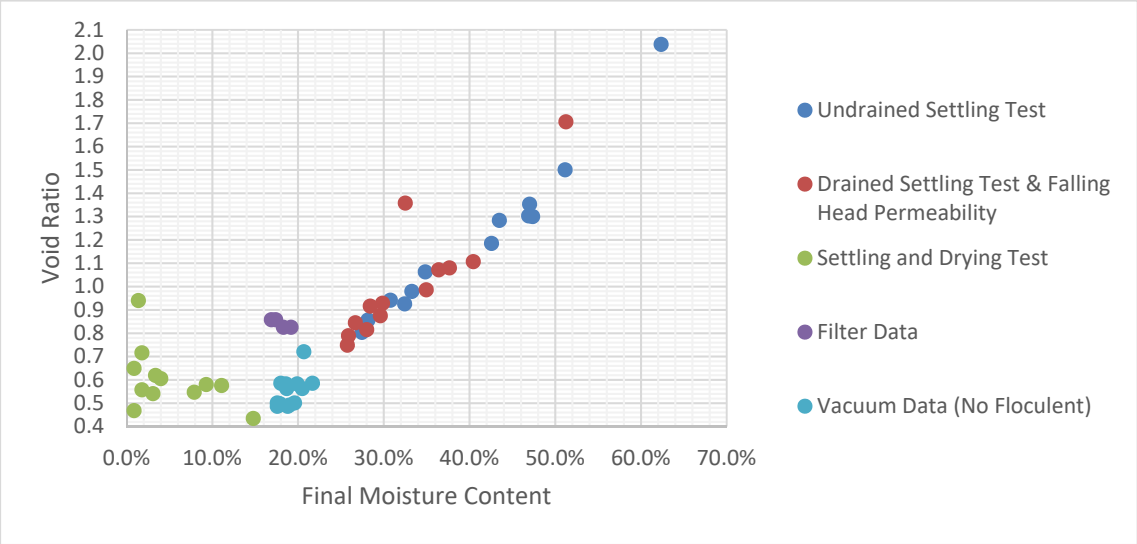

Figure 4.3: Measured Final Moisture Content versus Void Ratio

5 Summary

As previously stated, the deposited tailings density is dependent on both the specific gravity of the particles and the void ratio of the resulting deposited material. Based on a the average specific gravity of 2.85 (Table 4.1), and a void ratio of 1.2 (Vick 1990), the resulting dry density is 1.3 t/m^3, which is consistent with the observed average dry density for the tailings from the undrained settling tests (Table 4.8), and the observed relationship in Figure 4.1. Therefore, an average dry density of placed tailings equal to 1.3 t/m^3 is appropriate for the Project tailings.

Similarly, the dry density of the drystack tailings is recommended to be 1.8 t/m^3, with a void ratio of 0.6, and a gravimetric water content of 20.5%. These values are representative of the tailings testing in laboratory conditions and appropriately conservative for use as design parameters. Under operational conditions the void ratio may change significantly due to active compaction, resulting in increased density and lower moisture content. Further testing will be required to determine the maximum density and the optimum moisture content.

An internal angle of friction of 43 degrees may be considered high for these tailings; therefore, an angle of 40 degrees has been used in stability modelling to date. Also, the measured value for cohesion is low (1 kPa), and its effectiveness in supporting the soil structure is highly dependent on how the stress path for failure is mobilized. Therefore, cohesion is not considered in stability analysis.

Based on the geotechnical testing results, and the design understanding, the values as presented in Table 5.1 are recommended to be used on a go-forward basis.

Table 5.1: Summarized Tailings Geotechnical Properties

Parameter	Value	Source
Specific gravity	2.85	Average of lab testing (Table 4.1)
% Fines (<0.075 mm)	65%	Average of lab testing (Table 4.2)
% Silt	52%	Average of lab testing (Table 4.2
% Clay	13%	Average of lab testing (Table 4.2)
Void ratio (e) – slurried tailings	1.2	Assumed & supported with available data (Figure 4.1)
Void ratio (e) – drystack tailings	0.6	Assumed & supported with available data (Figure 4.1)
Deposited dry density (tonnes/m^3) – slurried tailings	1.30	Based on void ratio of 1.2 and SG of 2.85, correlates with available data (Figure 4.1).
Deposited dry density (tonnes/m^3) – drystack tailings	1.8	Based on void ratio of 0.6 and SG of 2.85, correlates with available data (Table 4.11).
Internal angle of friction (degrees)	40	Engineering judgement based on available data (Table 4.7)
Cohesion (kPa)	0	Engineering judgement based on available data (Table 4.7)
Gravimetric moisture content (%) – slurried tailings	42.6%	Average based on undrained settling tests (Table 4.8) (Note: a gravimetric moisture content of 42.6% results in a degree of saturation > 100%)
Gravimetric moisture content (%) – drystack tailings	20.5%	Based on observed relationships in Figure 4.2 and Figure 4.3. (Note: a gravimetric moisture content of 20.5% results in a degree of saturation equal to 100%)
Hydraulic conductivity (m/s)	$1.3x10^{-7}$	Average (Table 4.6)

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The opinions expressed in this report have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. Whilst SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

6 References

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Appendix H – Borehole Log (SRK 39)

PROJECT: HOPE BAY DORIS NORTH PROJECT
PROJECT NO: 1CM014.01
LOCATION: Tail Lake South
SURFACE (COLLAR) ELEVATION: 41.14 m
NORTHING: 7556391 **EASTING:** 435164
LOGGED BY: D.B.M.

HOLE NO: SRK 39
HOLE DIAMETER: 76 mm (NQ)
DATE AND TIME STARTED: 2003-07-31, 01:00
DATE AND TIME FINISHED: 2003-08-01, 06:00
DRILL CONTRACTOR: Major Midwest
DRILLING METHOD: Diamond Core

RQD ROCK QUALITY(%)
100 x core lengths 100 mm and longer / length of run

Fracture spacing (mm)
<30
30 - 100
100 - 300
300 - 1000
>1000

GRADE	ROCK MASS WEATHERING	ROCK MASS HARDNESS	ROCK MASS FABRIC	DISCONTINUITY ROUGHNESS	DISCONTINUITY SEPARATION	DISCONTINUITY DIP
1	unweathered	v. hard	v. fine	smooth	closed	vertical
2	slightly	hard	fine	sl. rough	v. narrow	steep
3	medium	medium	medium	medium	narrow	medium
4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
0		OVERBURDEN Grey silty CLAY, wet, with coarse sand-sized pieces of hard dry clay which can be broken down with finger pressure. Material contains lenses of ice.		CL									1	100%	Sample 1 0.0-3.0	
3													2	100%	Sample 2 3.5-5.6	
6													3	100%		
6													4	100%		
7	7.25	BEDROCK Green foliated BASALT with calcite veining and infilling parallel to foliation, with occacional quartz veins			49%								5	100%		
9					63%								6	100%		

PROJECT: HOPE BAY DORIS NORTH PROJECT
PROJECT NO: 1CM014.01
LOCATION: Tail Lake South
SURFACE (COLLAR) ELEVATION: 41.14 m
NORTHING: 7556391 **EASTING:** 435164
LOGGED BY: D.B.M.

HOLE NO: SRK 39
HOLE DIAMETER: 76 mm (NQ)
DATE AND TIME STARTED: 2003-07-31, 01:00
DATE AND TIME FINISHED: 2003-08-01, 06:00
DRILL CONTRACTOR: Major Midwest
DRILLING METHOD: Diamond Core

RQD ROCK QUALITY(%)
100 x core lengths 100 mm and longer / length of run

Fracture spacing (mm)
<30
30 - 100
100 - 300
300 - 1000
>1000

GRADE	ROCK MASS WEATHERING	ROCK MASS HARDNESS	ROCK MASS FABRIC	DISCONTINUITY ROUGHNESS	DISCONTINUITY SEPARATION	DISCONTINUITY DIP
1	unweathered	v. hard	v. fine	smooth	closed	vertical
2	slightly	hard	fine	sl. rough	v. narrow	steep
3	medium	medium	medium	medium	narrow	medium
4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
11					92%								7	100%		
12					81%								8	97%		
13																
14																
15					90%								9	100%		
					100%											
16													10	100%		
17																
18					81%											
19													11	100%		
20																

PROJECT: HOPE BAY DORIS NORTH PROJECT
PROJECT NO: 1CM014.01
LOCATION: Tail Lake South
SURFACE (COLLAR) ELEVATION: 41.14 m
NORTHING: 7556391 **EASTING:** 435164
LOGGED BY: D.B.M.

HOLE NO: SRK 39
HOLE DIAMETER: 76 mm (NQ)
DATE AND TIME STARTED: 2003-07-31, 01:00
DATE AND TIME FINISHED: 2003-08-01, 06:00
DRILL CONTRACTOR: Major Midwest
DRILLING METHOD: Diamond Core

RQD ROCK QUALITY(%)
100 x core lengths 100 mm and longer / length of run

Fracture spacing (mm)
<30
30 - 100
100 - 300
300 - 1000
>1000

GRADE	ROCK MASS WEATHERING	ROCK MASS HARDNESS	ROCK MASS FABRIC	DISCONTINUITY ROUGHNESS	DISCONTINUITY SEPARATION	DISCONTINUITY DIP
1	unweathered	v. hard	v. fine	smooth	closed	vertical
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4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
21					95%											
22													12	100%		
23																
24					93%											
25																
26													13	99%		
27					89%											
28																
29													14	100%		
30					88%											

PROJECT: HOPE BAY DORIS NORTH PROJECT
PROJECT NO: 1CM014.01
LOCATION: Tail Lake South
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4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
31													15	98%		
32																
33					75%											
34													16	97%		
35																
36					68%											
37													17	99%		
38																
39					84%											
40																

PROJECT: HOPE BAY DORIS NORTH PROJECT
PROJECT NO: 1CM014.01
LOCATION: Tail Lake South
SURFACE (COLLAR) ELEVATION: 41.14 m
NORTHING: 7556391 **EASTING:** 435164
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HOLE NO: SRK 39
HOLE DIAMETER: 76 mm (NQ)
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RQD ROCK QUALITY(%)
100 x core lengths 100 mm and longer / length of run

Fracture spacing (mm)
<30
30 - 100
100 - 300
300 - 1000
>1000

GRADE	ROCK MASS WEATHERING	ROCK MASS HARDNESS	ROCK MASS FABRIC	DISCONTINUITY ROUGHNESS	DISCONTINUITY SEPARATION	DISCONTINUITY DIP
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3	medium	medium	medium	medium	narrow	medium
4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
41													18	99%		
42					73%											
43													19	88%		
44																
45					71%											
46													20	100%		
47																
48					95%											
49													21	100%		
50																

PROJECT: HOPE BAY DORIS NORTH PROJECT
HOLE NO: SRK 39
PROJECT NO: 1CM014.01
HOLE DIAMETER: 76 mm (NQ)
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DRILLING METHOD: Diamond Core

RQD ROCK QUALITY(%)
100 x core lengths 100 mm and longer / length of run

Fracture spacing (mm)
<30
30 - 100
100 - 300
300 - 1000
>1000

GRADE	ROCK MASS: WEATHERING	ROCK MASS: HARDNESS	ROCK MASS: FABRIC	DISCONTINUITY: ROUGHNESS	DISCONTINUITY: SEPARATION	DISCONTINUITY: DIP
1	unweathered	v. hard	v. fine	smooth	closed	vertical
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4	highly	soft	coarse	rough	wide	shallow
5	completely	v.soft	v. coarse	v. rough	v. wide	horizontal

Depth (m)	Contact (m)	Material Description	Lithology	Soil Class	RQD	Fracture Spacing	Weathering	Hardness	Fabric	Roughness	Separation	Dip (degrees)	Run	Recovery	Sample	Installations
51	51 EOH															
52																
53																
54																
55																
56																
57																
58																
59																
60																

Appendix I – Hope Bay Project: Phase 2 Doris Tailings Impoundment Area South and West Dam Thermal Modeling

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Christopher W. Stevens, PhD	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Doris Tailings Impoundment Area South and West Dam Thermal Modeling		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix I as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes		

1 Introduction

1.1 General

The Madrid-Boston Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris only, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Phase 1 tailings will be deposited sub-aerially in the Doris tailings impoundment area (TIA), formerly Tail Lake, located approximately 5 km from the Doris Mill. Containment will be provided by three retention structures: a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam), and an Interim Dike situated at approximately the midpoint of the facility. Tailings will be deposited sub-aerially between the South Dam and Interim Dike, and the Reclaim Pond will be contained between the Interim Dike and the North Dam. The North Dam was constructed over two winters (2011 and 2012) and has impounded water since 2011 (SRK 2012). The South Dam and Interim Dike are scheduled for construction in 2017.

Phase 2 tailings deposition will include a continuation of the Doris TIA with raising of the South Dam and construction of a new West Dam (SRK 2017a). Phase 2 tailings deposition will see a return of the discharge spigots to the raised South Dam to form a continuous beach against the upstream side. Once the final beach elevation is reached at the South Dam, the tailings deposition will shift to the West Dam to form a similar beach. Tailings beach development will result in the supernatant pond, located directly against the North Dam. The Interim Dike will be completely inundated by this deposition strategy. At Closure, the North Dam will be breached to allow drainage of the pond, and a run-of-quarry (ROQ) or geochemically suitable run-of-mine (ROM) cover will be constructed over the tailings beach.

The South and West dams will rely on an impermeable liner incorporated in the dam fill and keyed into the permafrost foundation to achieve the required water retention properties. In order to ensure adequate performance of the dam, it is imperative to maintain the frozen state of the key-in trench. The North Dam will continue to operate as a frozen core dam until closure.

1.2 Objectives

This memo summarizes the assumptions and the results of the numerical modeling completed to predict the thermal performance of the Doris TIA South Dam and West Dam to support the Phase 2 tailings deposition. The thermal modeling completed for the North Dam is presented in SRK (2017b).

The objective of the modeling was to determine whether or not the proposed designs were suitable to maintain the liner within a frozen key-in trench. The foundation was considered to be

valid if the temperature within the dam key-in trench remained colder than -2°C, which accounts for the average site-wide porewater freezing point depression of -2.1°C within natural overburden clay at the Project site (SRK 2017a). There is a negligible difference between the position of these two isotherms, and -2°C is presented for clarity of the results. The model also predicts tailings freezeback and active layer development to understand long-term geochemical loading from the facility.

2 Methods

2.1 Model Setup

Modeling was completed in a two-dimensional domain by solving for conductive heat movement in the soil, using SoilVision's SVHeat (SoilVision 2011) software package in combination with FlexPDE (FlexPDE 2014). SVHeat was utilized for the problem setup, while FlexPDE 6.35 solver was used to complete the calculation.

For the purpose of modeling, the South and West dams are divided into two periods of time; the period of active tailings deposition (*active deposition at dam*) when tailings are deposited from the dam and the period following tailings deposition (*deposition complete at dam*) when deposition is complete and a stable beach is present against the dam (Figure 1). The timeline for thermal modeling is based on the Phase 2 – Doris TIA Tailings Deposition Plan (SRK 2017c).

The South Dam was modeled with a 10 m crest width, 15 m maximum height, upstream slope of 4H:1V, and downstream slope of 2H:1V. The key-in trench into the in situ overburden material has a 6 m wide base with upstream slope of 2H:1V and downstream slope of 1H:1V. The key-in trench excavation is located near the upstream toe of the dam. Drill holes completed at the site have characterized silt and clay to a depth of 24 m which is underlain by a 10 m thick gravelly till (SRK 2017d). For simplicity, the model assumes a clay foundation. The model geometry for the South Dam during the period of active tailings deposition and following the completion of deposition is presented in Figure 2 and Figure 3, respectively.

The West Dam was modeled with a 10 m crest width, 5 m maximum height, upstream slope of 4H:1V, and downstream slope of 2H:1V. The key-in trench excavated into overburden material has a 4 m wide base with upstream slope of 2H:1V and downstream slope of 1H:1V. The key-in trench excavation is located near the centre of the dam. The foundation conditions at the West Dam are inferred to consist of silt and clay based on a borehole log (SRK 2017d), and therefore the model assumes clay. Given the location of the dam alignment, it is assumed that the entire alignment of the West Dam is underlain by permafrost. The model geometry for the West Dam during the period of active tailings deposition and following the completion of deposition is presented in Figure 4 and Figure 5, respectively.

2.2 Model Inputs

2.2.1 Material Properties

Three material units were considered: native soil (overburden clay), dam fill (ROQ material), and tailings. Table 1 presents a summary of the material properties. No peat or organic layer was considered during the modeling. This omission is reasonable given the fact that removal of this layer from the model makes the analysis more conservative. If organic layers were to be

considered in the model, the rate of thaw would be less due to latent heat required to change phase of ice to water.

The thermal properties for the ROQ material were taken from previous work completed by SRK for the granular pad design (SRK 2017e). The ROQ material was modified to account for the unfrozen saturated thermal conductivity and heat capacity of the material located on the upstream side of the liner. The unfrozen properties for this material (*ROQ Material, Unfrozen Saturation 100%*) were estimated for 100% saturation (Table 1).

The thermal properties for natural overburden clay were based on average physical properties of the soil and a porewater freezing point depression of -2°C (SRK 2017d). An unfrozen water content curve for overburden clay was included in the model with consideration for the freezing point depression in accordance with Banin and Anderson (1974). The thermal conductivity was calculated in accordance with Cote and Konrad (2005).

Table 1: Material Thermal Properties

Material	Degree of Saturation (%)	Porosity	Thermal Conductivity, kJ/(m·day·°C)		Volumetric Heat Capacity, kJ/(m³·°C)	
			Unfrozen	Frozen	Unfrozen	Frozen
ROQ Material	30	0.30	104	117	1,697	1,509
ROQ Material, Unfrozen Saturation 100%	100	0.30	141	117	2,576	1,509
Overburden Clay [1,2]	85	0.52	112	187	2,842	2,038
Tailings[2]	100	0.55	123	232	3,502	2,350

Notes:

Overburden clay includes a porewater freezing point depression of -2°C

Unfrozen water content curve based on grain size

The tailings thermal properties were based on physical samples of tailings sourced from the Project site. The physical properties measured in the laboratory included an average specific gravity (2.87) and a dry density (1.3 g cm^{-3}) (SRK 2017f). The measured specific gravity, dry density, and a saturation of 100% were used to calculate the porosity (55%) and gravimetric water content (42% based on dry weight of solids) of the tailings. These physical properties were in turn used to derive the tailings thermal properties shown in Table 2. The tailings thermal properties include an unfrozen water content curve based on silt size mineral soil particles. Tailings process water is not expected to have an appreciable level of dissolved ions which contribute to a freezing point depression, and no allowance was made in the model.

2.2.2 Climate Boundary Conditions

A ground surface temperature curve was developed for the Project site to represent the ground temperature immediately below surface. The boundary is defined by the sinusoidal function of temperature and time based on Equation 1 and the parameters shown in Table 2.

Equation 1

$$T = \max(nf * \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi + (t + 182.5)}{365}\right)\right], \text{nt} \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi + (t + 182.5)}{365}\right)\right]$$

Where:

T *is the ground temperature measured in °C*
nf *is the surface freezing n-factor*
nt *is the surface thawing n-factor*
$MAAT$ *is the mean annual air temperature measured in °C*
Amp *is the air temperature amplitude measured in °C*
C_A *is the air climate change factor in °C d*$^{-1}$
t *is time measured in days*

Table 2: Current Climate Boundary Parameters

Model Parameter	Value
Mean annual air temperature *(MAAT)*	-10.7°C
Air temperature amplitude *(Amp)*	21.0°C
ROQ Surface, Thawing n-factor *(nt)*	1.52
ROQ Surface, Freezing n-factor *(nf)*	0.86
Tailings Beach, Thawing n-factor *(nt)*	1.52
Tailings Beach, Freezing n-factor *(nf)*	0.86
Natural Overburden, Thawing n-factor *(nt)*	0.55
Natural Overburden, Freezing n-factor *(nf)*	0.65

Mean annual air temperature and amplitude are based on average values for the baseline period of 1979-2005 (SRK 2017g). Seasonal n-factors are applied as multipliers of air temperature to estimate the ground surface temperature at the ground surface. The ROQ and tailings surfaces included a freezing n-factor (nf) of 0.86 and thawing n-factor (nt) of 1.52. These values are based on average published values (SRK 2017e) and considered to be reasonable base case conditions for the Project site. N-factors for natural overburden were applied downstream of the dams using values calibrated to ground temperatures measured at the Project site (SRK 2017h).

Climate change is considered in Equation 1 using the air climate change factor. This factor allows for a daily increase in air temperature within the model. Table 3 shows the daily increase in air temperature in the model which is based on the work of SRK (2017g). The model simulations are performed to the year 2100, which is 85 years beyond the initial model year of 2015.

Table 3: Summary of Doris Air Climate Change Factors Based on Climate Change Models

Year	Rate (°C decade^{-1})	Air Climate Change Factor (°C day^{-1})
2015 - 2040	0.74	0.000203
2041 - 2070	0.71	0.000195
2071 - 2100	0.69	0.000192

2.2.3 Initial Conditions

The initial conditions were defined by each material region in the model. The dam fill (ROQ material) was assumed to thermally adjust to the ambient air temperature during the period of winter construction. The initial temperature of the ROQ material was set to -6°C. The temperature of the ROM material would likely be colder during the winter construction period, and therefore represents a conservative input to the model. Initial ground temperature for the clay overburden was set to -7.6°C which is representative of average permafrost temperatures at the Project site (SRK 2017e). The model assumes continuous permafrost exists beneath the dam alignment prior to construction.

The vertical sides of the model space were set to a zero flux boundary and the lower boundary set to a constant flux 3.93 kJ/(m2·day·°C) which was calculated from the average geothermal gradient (0.021°C m-1) and the thermal conductivity of the clay overburden (SRK 2017e).

2.2.4 Transient Conditions

Active Deposition at Dam

For the period of active deposition, a constant temperature of +6°C was applied along the upstream face of the dam to the maximum elevation of the liner and along to the bottom of the TIA. The +6°C temperature used in the model is similar to the mean annual water temperature associated with Arctic lakes. This value is greater than the mean annual temperature resulting from heat transfer between the atmosphere and the exposed tailings surface. The boundary is considered to be conservative since a tailings beach formed against the upstream dam face will result in seasonal cooling of the tailings and gradually increase in height over the period of deposition.

The period of active deposition spans 2.97 years for the South Dam and 6.42 years for the West Dam (Figure 1). It is assumed that the West Dam's construction is delayed until immediately prior to the period of active deposition.

Beached Tailings

At each dam, a stable sub-aerial beach is specified in the model after the period of active deposition is complete (Figure 1). The climate boundary with average freezing and thawing n-factors is applied to the uppermost surface of the tailings beach (Equation 1; Table 2). An exposed tailings beach surface without a material cover is assumed in the model. The initial internal temperature of the tailings is specified to be +6°C, which is conservative when considering that a portion of Phase 1 tailings will be exposed to atmospheric conditions prior to the short period of Phase 2 deposition. The initial conditions for all other regions were defined by the last time step of the Active Deposition Model.

After the maximum beach elevation is achieved at each dam, the tailings deposition will shift to adjacent positions (spigots). Lateral heat flow from areas of active deposition is not expected to thermally influence the dam and key-in trench.

3 Results

3.1 South Dam

Figure 6 shows the predicted temperature of the South Dam and foundation at the end of active deposition (Year 2.97). The constant temperature applied to the model contributes to thaw along the upstream face of the dam and bottom of the TIA. The temperature of the model at the end of the design life is shown in Figure 7.

Over the 25 year design, the South Dam key-in trench and liner remains frozen at the critical position of Monitoring Point 1 (Figure 8). The maximum temperature predicted for this location is -3.6°C over the design life. Gradual cooling is observed shortly after the tailings deposition is complete and the beached tailings are sub-aerially exposed to heat transfer with the atmosphere.

At the South Dam, the tailings are observed to freezeback over a 13-year period following completion of deposition. During this period of time, seasonal thaw exceeds freezeback. Figure 9 shows the tailings active layer which increases to a maximum thickness of 2.07 m by the year 2100.

3.2 West Dam

Figure 10 shows the predicted temperature for the West Dam and foundation at the end of active deposition (Year 9.39). Similar to the South Dam, the constant temperature applied to the upstream face of the dam and bottom of the TIA contributes to thaw. Modeled temperature at the end of the design life is shown in Figure 11. The South Dam key-in trench and liner remains frozen at the critical position of Monitoring Point 1, with a maximum temperature of -5.1°C over the 25-year design life (Figure 12).

The tailings are observed to freezeback over a two (2) year period near the West Dam. The reduced time for freezeback of the tailings at this location is related to the decreased thickness when compared to tailings deposited near the South Dam. By the year 2100, tailings active layer thickness is estimated to be 2.00 m (Figure 13).

4 Conclusions

Thermal modeling of the South and West TIA dams suggests that the thickness of the bulk ROM material of the dam, acting as thermal protection over the key-in trench, is adequate to maintain the base of the key-in trench and liner below -2°C. The thermal results confirm that the impermeable liner will provide the intended water retaining capacity over the 25-year design life.

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The opinions expressed in this document have been based on the information available to SRK at the time of preparation. SRK has exercised all due care in reviewing information supplied by others for use on this project. While SRK has compared key supplied data with expected values, the accuracy of the results and conclusions from the review are entirely reliant on the accuracy and completeness of the supplied data. SRK does not accept responsibility for any errors or omissions in the supplied information, except to the extent that SRK was hired to verify the data.

5 References

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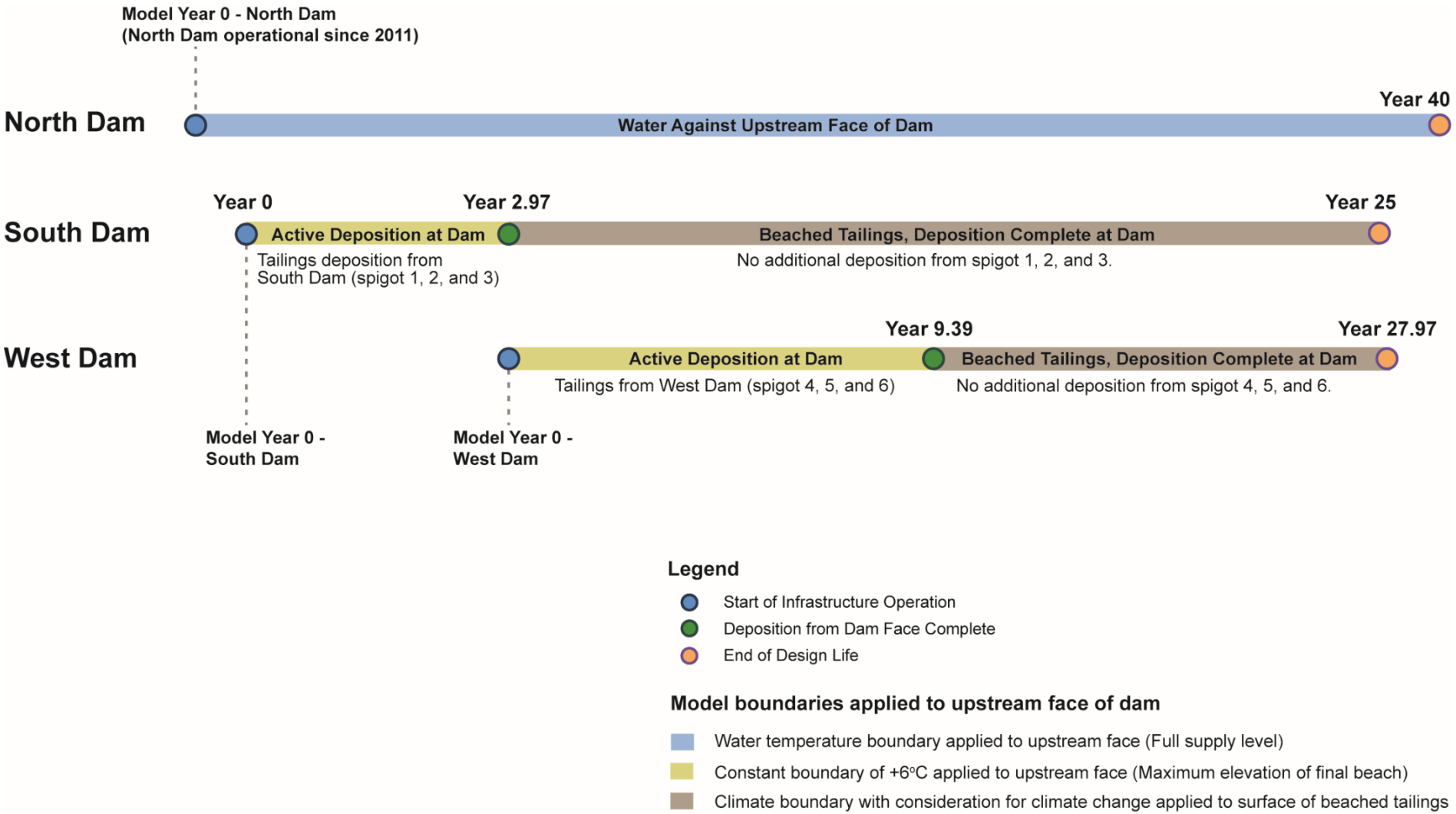
Figures



Notes:

1. Spigot numbers refer to the location of tailings deposition described in SRK (2016d)

srk consulting	TMAC RESOURCES	Doris TIA Thermal Modeling
		Timeline for Thermal Models
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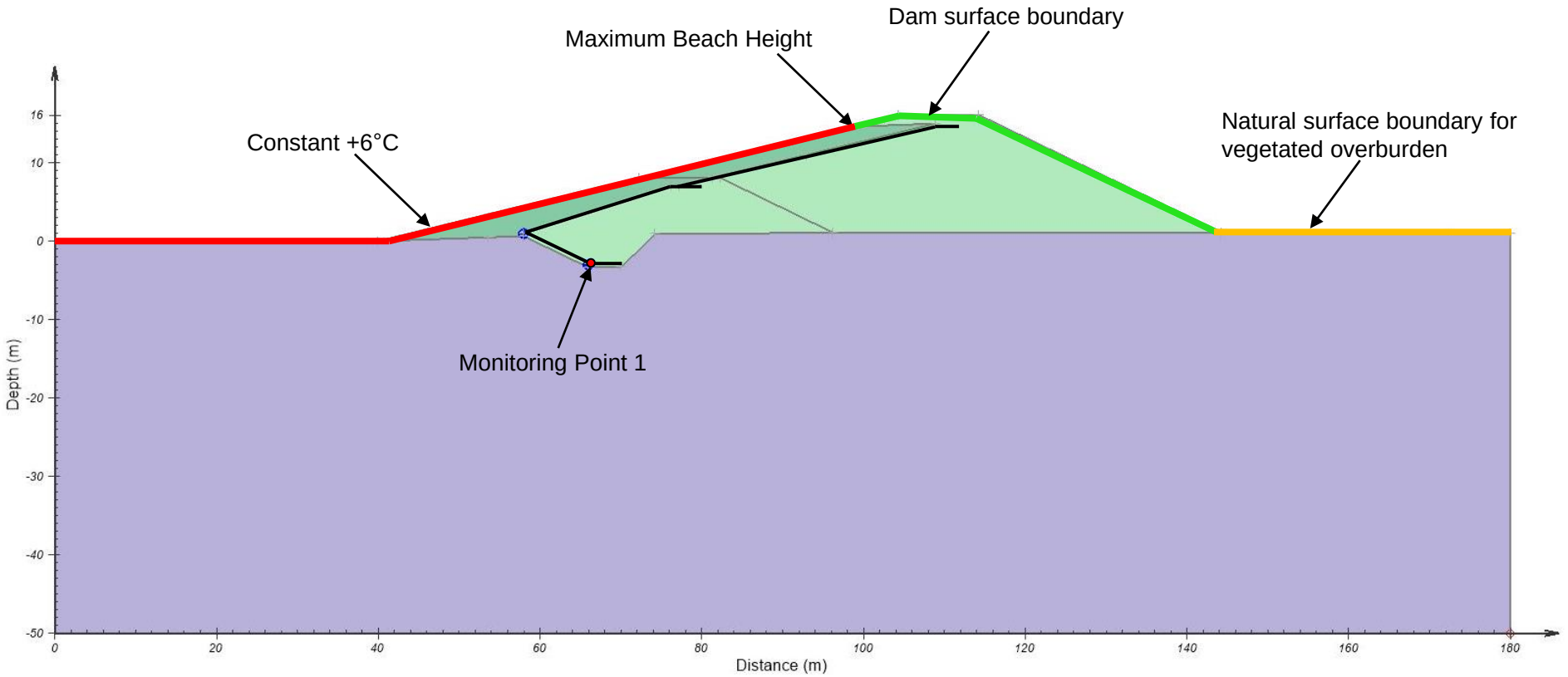


Notes:
1. Model geometry for period of active tailings deposition from South Dam
2. ROQ – Run-of-Quarry material or geochemically suitable run-of-mine (ROM) waste rock

Materials
- ROQ
- Clay
- ROQ100Saturation

srk consulting

MAC RESOURCES

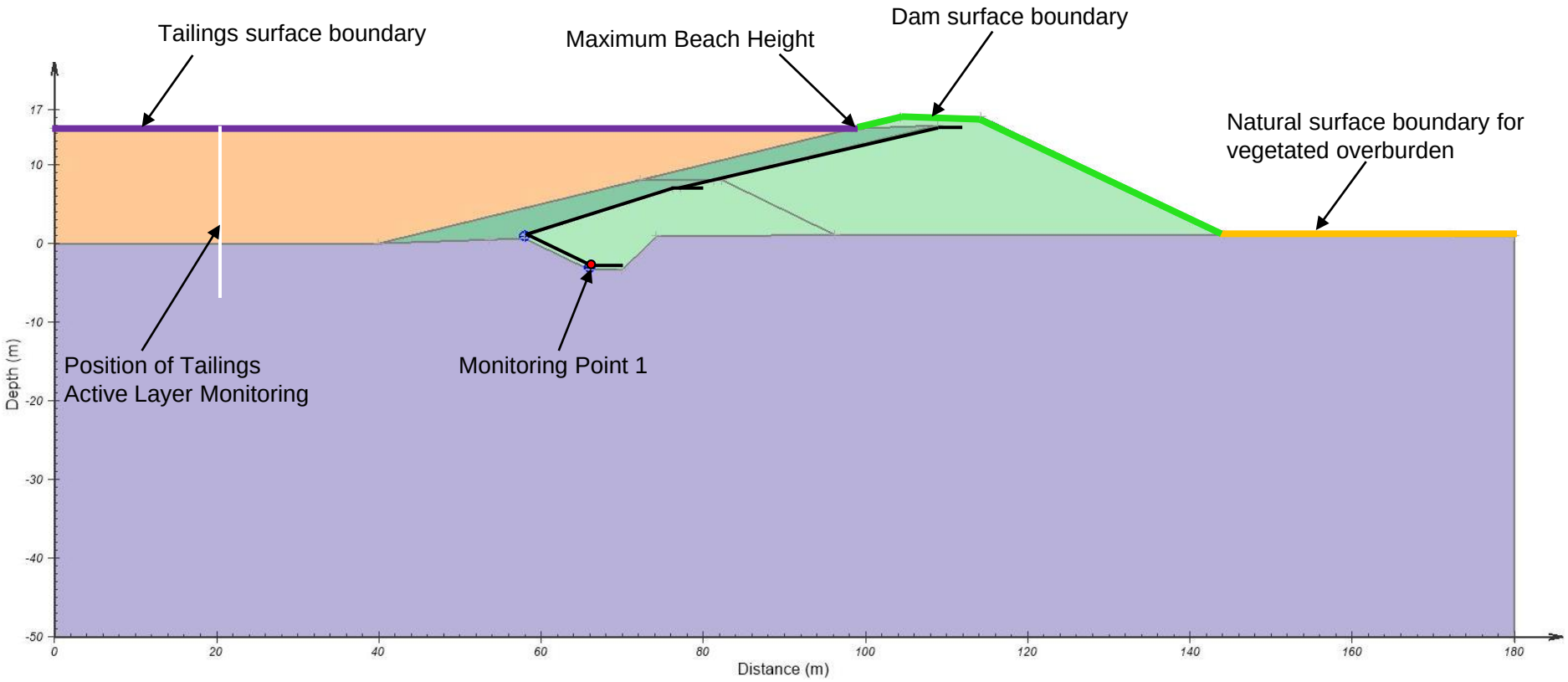
Doris TIA Thermal Modeling

South Dam Model Geometry – Active Deposition

Job No: 1CT022.004
Filename: ModelSWDams.pptx

HOPE BAY PROJECT

Date:	Approved:	Figure:
6/2/2016	cws	2



Materials

- ROQ
- Clay
- ROQ100Saturation
- Tailings

srk consulting

TMAC RESOURCES

Doris TIA Thermal Modeling

South Dam Model Geometry – Deposition Complete

Job No: 1CT022.004

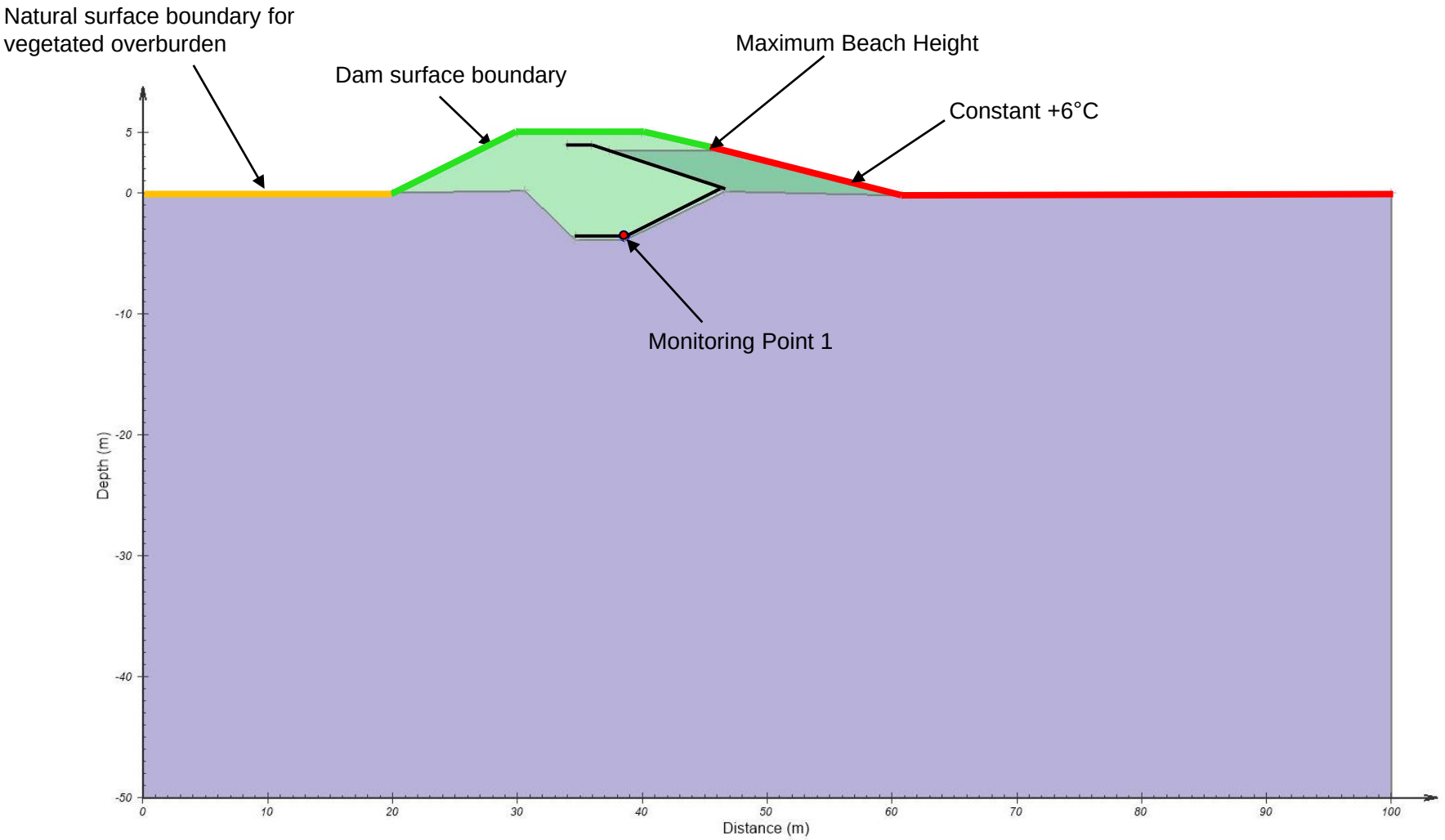
Filename: ModelSWDams.pptx

HOPE BAY PROJECT

Date: 6/2/2016

Approved: cws

Figure: **3**



Notes:
1. Model geometry for period of active tailings deposition from West Dam
2. ROQ – Run-of-Quarry material or geochemically suitable run-of-mine (ROM) waste rock

Materials

- ROQ
- Clay
- ROQ100Saturation

srk consulting

MAC RESOURCES

Doris TIA Thermal Modeling

West Dam Model Geometry – Active Deposition

Job No: 1CT022.004

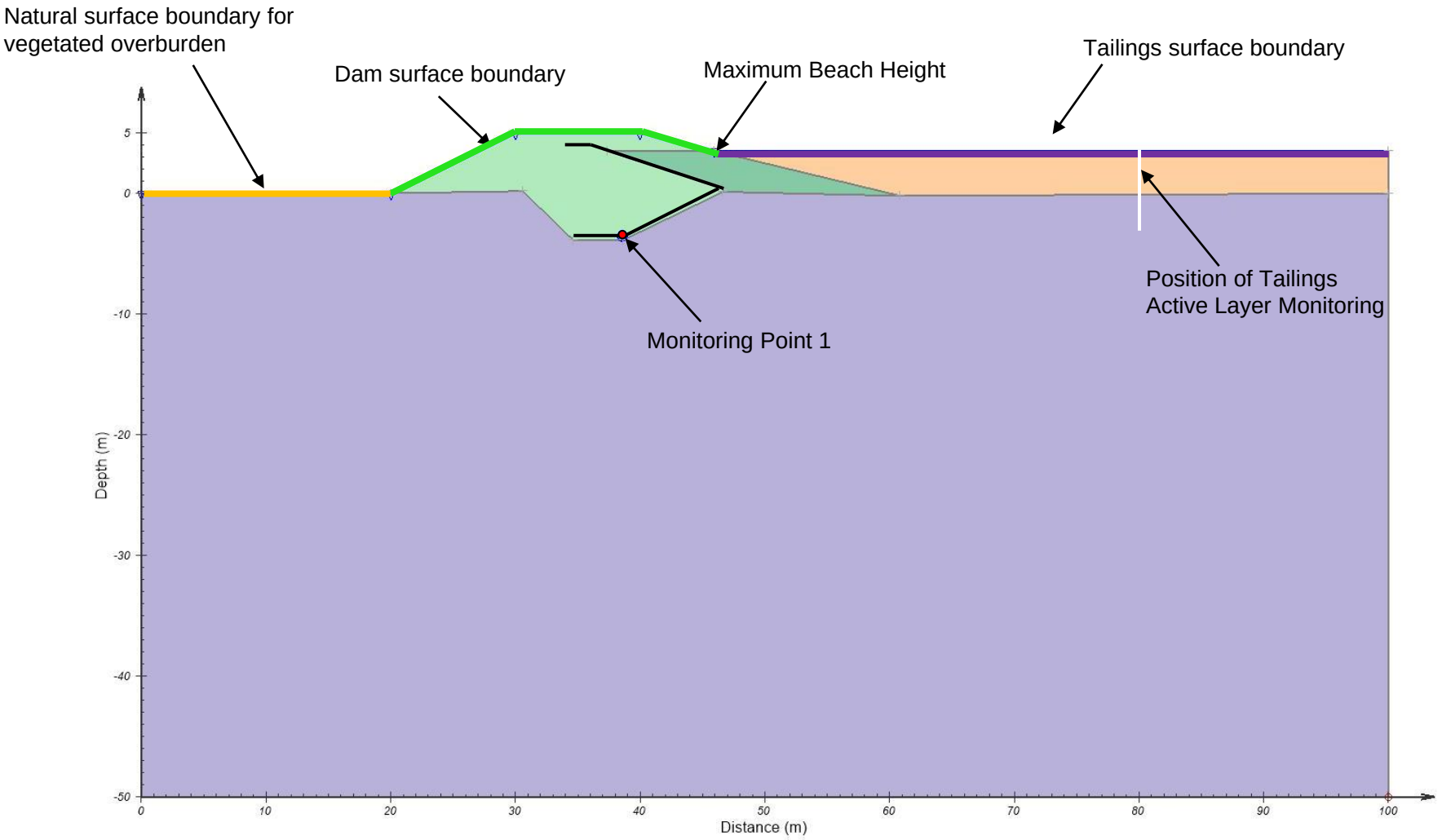
Filename: ModelSWDams.pptx

HOPE BAY PROJECT

Date: 6/2/2016

Approved: cws

Figure: 4



Notes:

1. Model geometry period of beached tailings following completion of deposition from the West Dam
2. ROQ – Run-of-Quarry material or geochemically suitable run-of-mine (ROM) waste rock

Materials

- ROQ
- Clay
- ROQ100Saturation
- Tailings

srk consulting

TMAC RESOURCES

Doris TIA Thermal Modeling

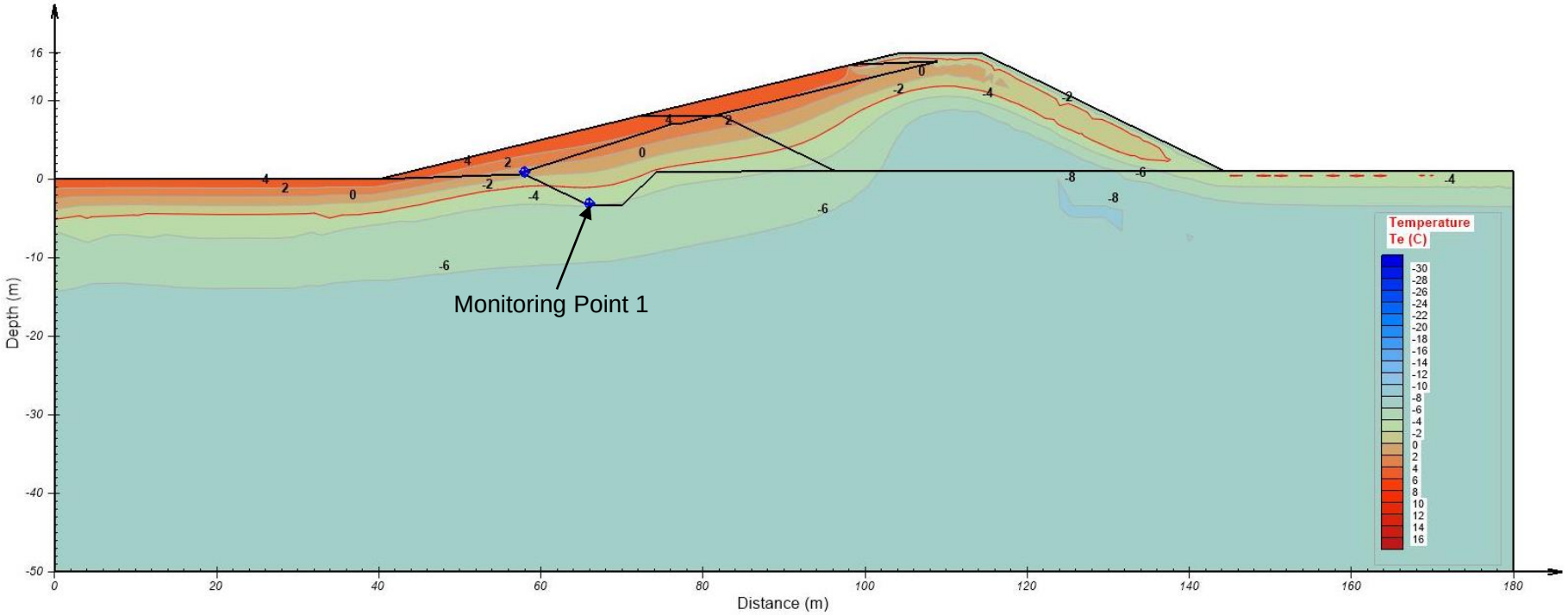
West Dam Model Geometry – Deposition Complete

Job No: 1CT022.004

Filename: ModelSWDams.pptx

HOPE BAY PROJECT

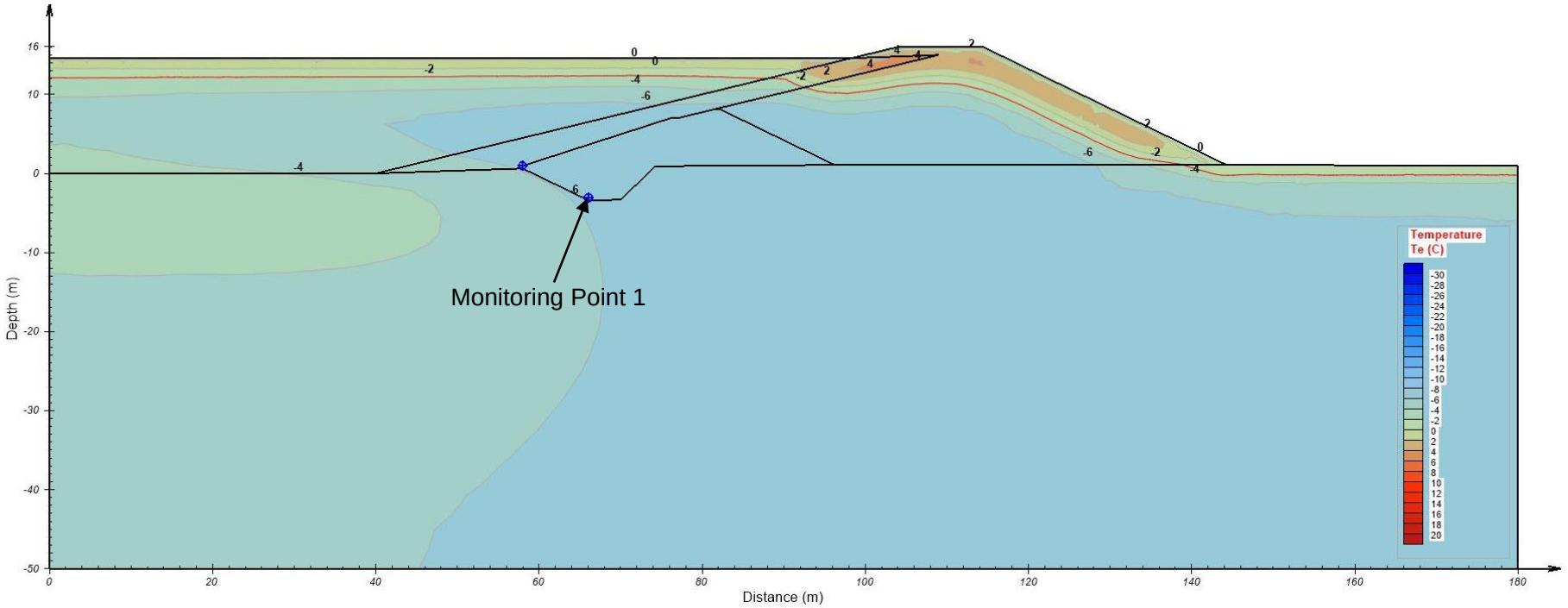
Date:	Approved:	Figure:
6/2/2016	cws	5



Notes:

1. Model results for Year 2.97 at the end of active deposition from the South Dam
2. Solid red line shows -2°C isotherm

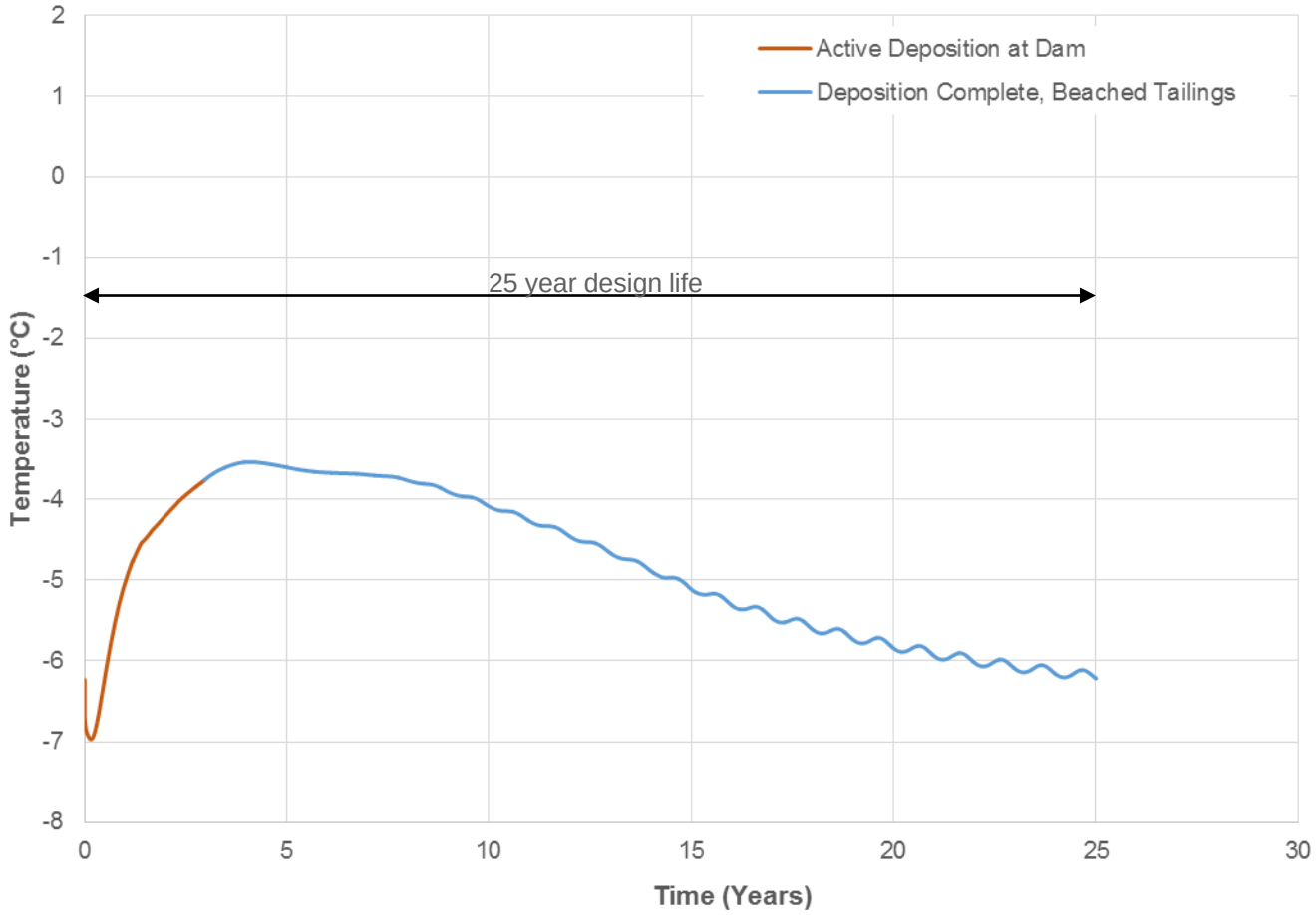
srk consulting	TMAC RESOURCES	Doris TIA Thermal Modeling		
		South Dam – Model Year 2.97		
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016	Approved: cws	Figure: **6**



Notes:

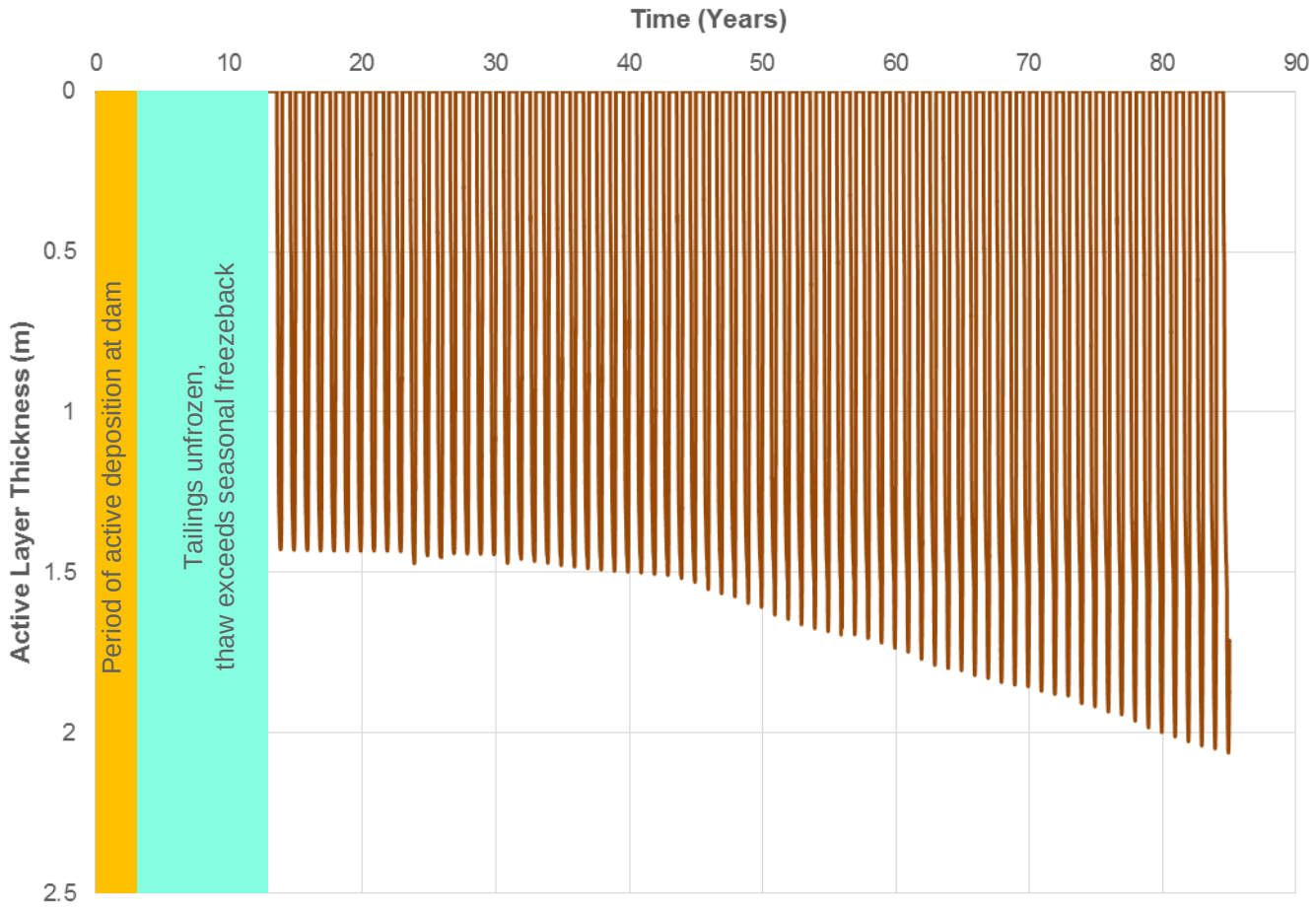
1. Model results for maximum position of seasonal thaw at the end of the 25 year design life
2. Solid red line shows -2°C isotherm

srk consulting	TMAC RESOURCES	Doris TIA Thermal Modeling
		South Dam – End of Design Life
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016 | Approved: cws | Figure: **7**



Notes:
1. Location of monitoring point 1 shown on Figures 2 and 3
2. Time express as number of years from start of South Dam operation

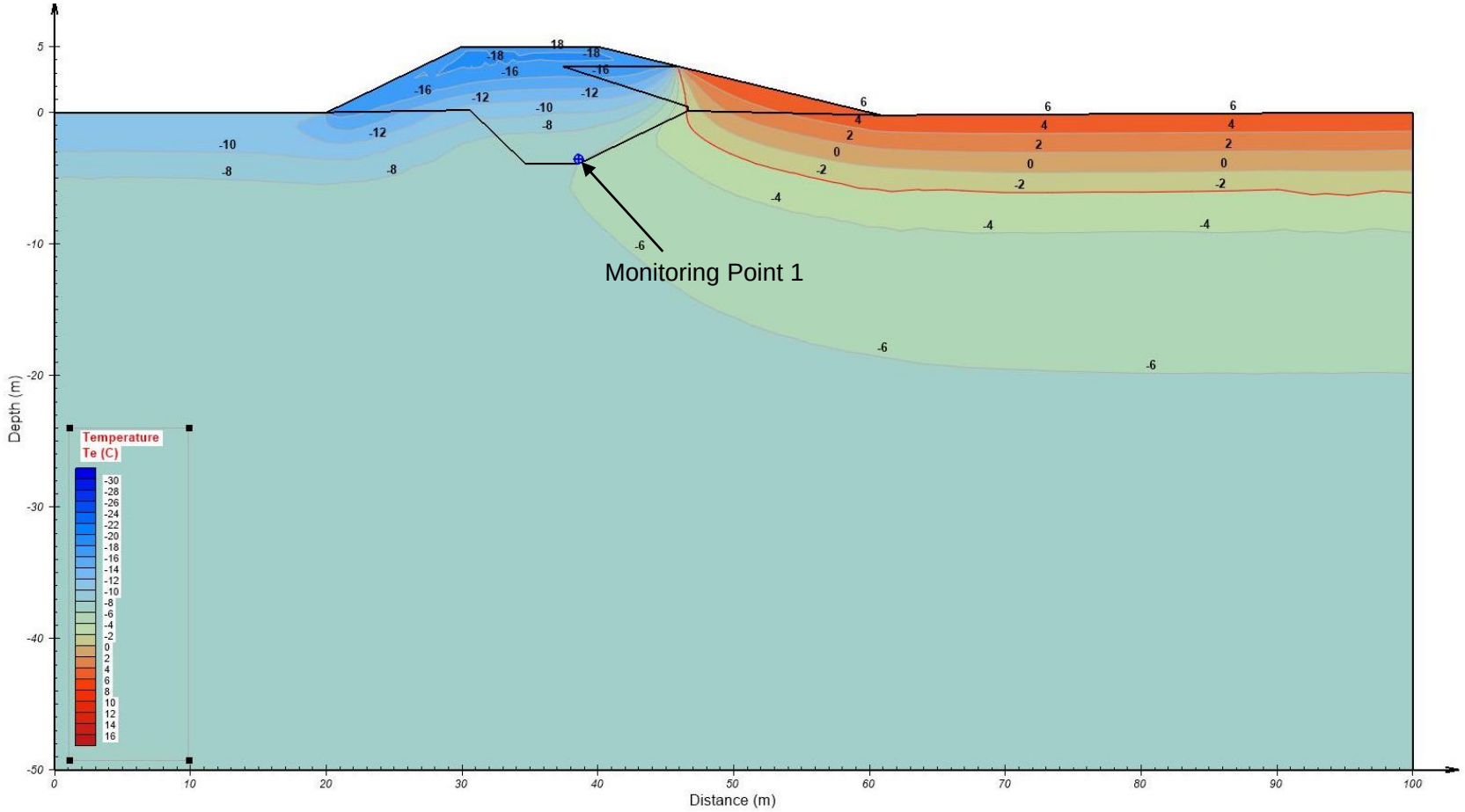
srk consulting	MAC RESOURCES	Doris TIA Thermal Modeling
		South Dam – Temperature at Monitoring Point 1
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016 | Approved: cws | Figure: **8**



Year	Max depth of thaw (m)	Comment
3	17.61	Thaw exceeds freezeback
5	15.72	Thaw exceeds freezeback
10	13.22	Thaw exceeds freezeback
15	1.43	Active Layer
20	1.43	Active Layer
25	1.45	Active Layer
30	1.47	Active Layer
35	1.48	Active Layer
40	1.50	Active Layer
45	1.55	Active Layer
50	1.63	Active Layer
55	1.69	Active Layer
60	1.75	Active Layer
65	1.82	Active Layer
70	1.87	Active Layer
75	1.93	Active Layer
80	2.01	Active Layer
85	2.07	Active Layer

Notes:
1. Active layer thickness based on 0°C isotherm
2. Location of active layer thickness shown on Figure 3
3. Time express as number of years from start of South Dam operation

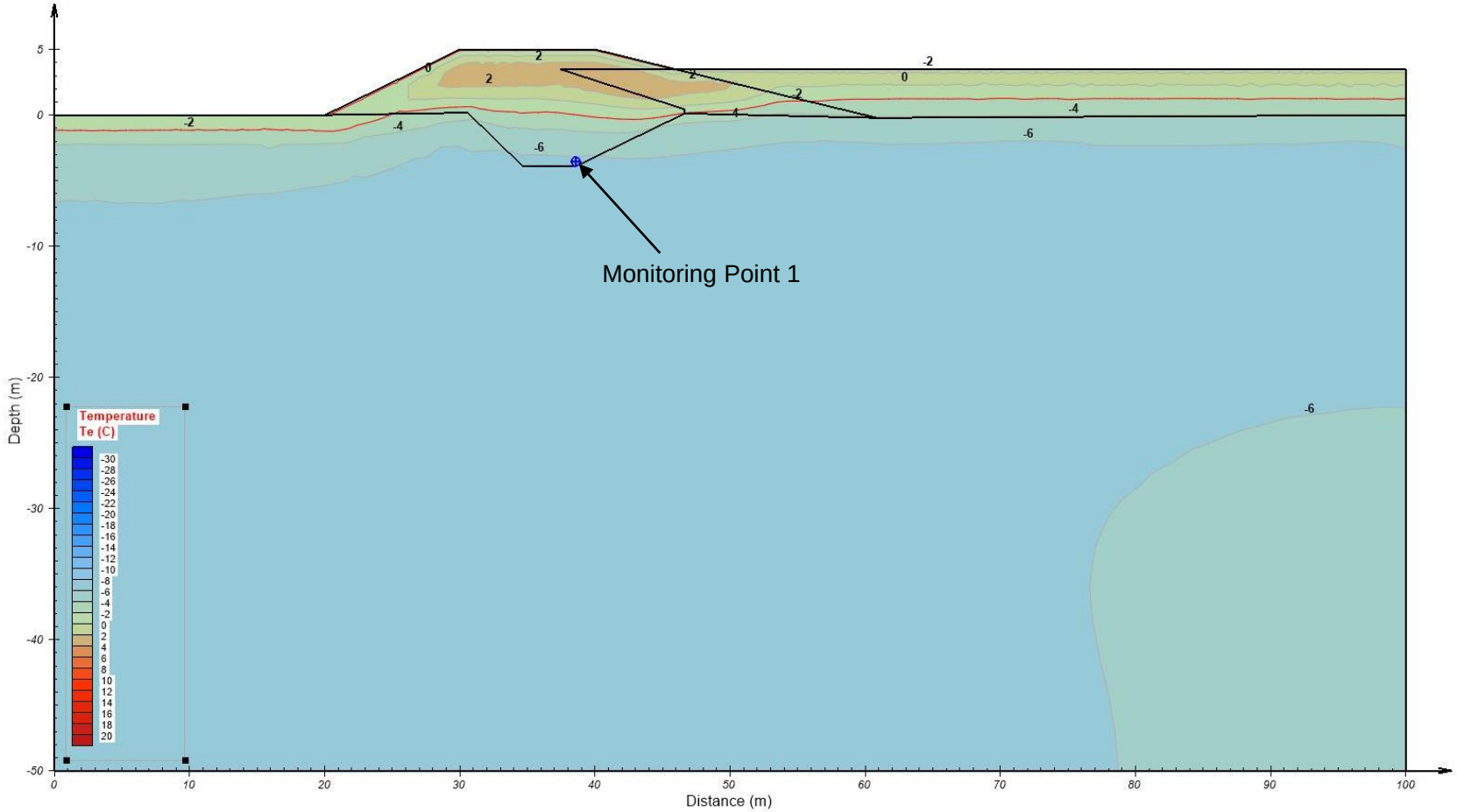
srk consulting	MAC RESOURCES	Doris TIA Thermal Modeling
		South Dam – Tailings Active Layer (Year 0 to 85)
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016 | Approved: cws | Figure: **9**



Notes:

1. Model results for Year 9.39 at the end of active deposition from the West Dam
2. Solid red line shows -2°C isotherm

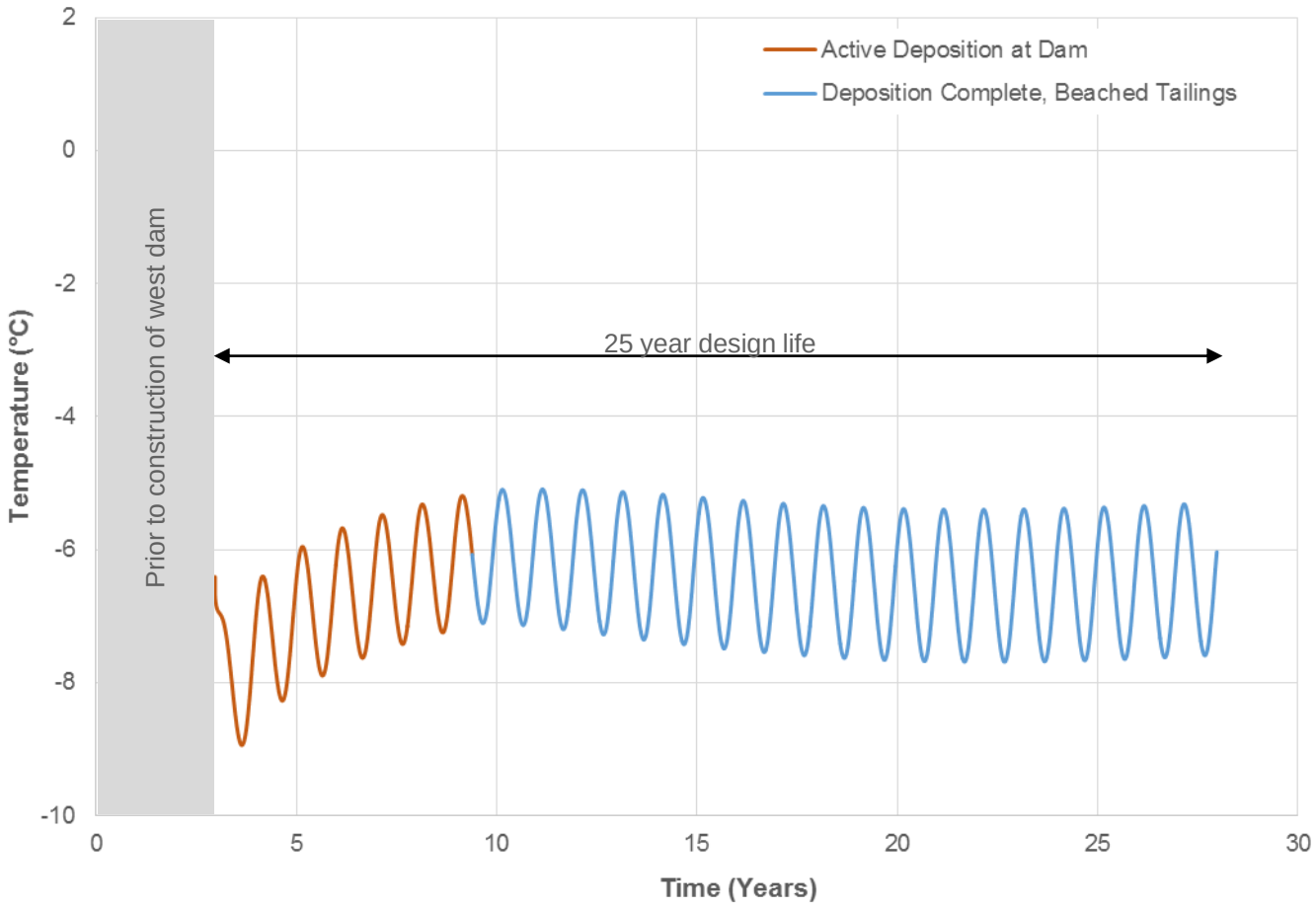
srk consulting	TMAC RESOURCES	Doris TIA Thermal Modeling		
		West Dam – Model Year 9.39		
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016	Approved: cws	Figure: **10**



Notes:

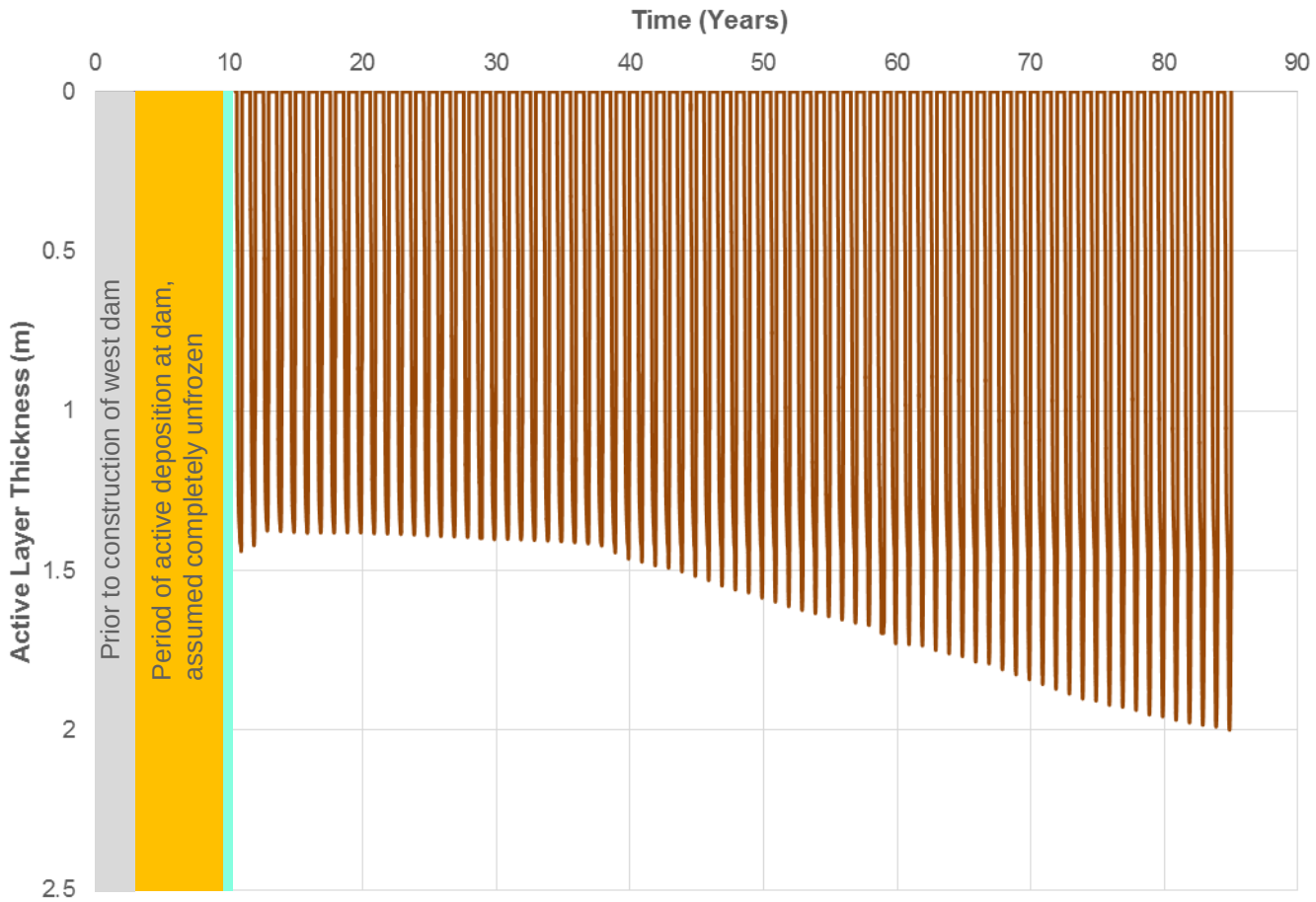
1. Model results for maximum position of seasonal thaw at the end of the 25 year design life
2. Solid red line shows -2°C isotherm

srk consulting	TMAC RESOURCES	Doris TIA Thermal Modeling		
		West Dam – End of Design Life		
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016	Approved: cws	Figure: **11**



Notes:
1. Location of monitoring point 1 shown on Figures 4 and 5
2. Time express as number of years from start of South Dam operation

srk consulting	MAC RESOURCES	Doris TIA Thermal Modeling
		West Dam – Temperature at Monitoring Point 1
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	Date: 6/2/2016 | Approved: cws | Figure: **12**



Year	Max depth of thaw (m)	Comment
10	8.07	Thaw exceeds freezeback
11	5.90	Thaw exceeds freezeback
15	1.38	Active Layer
20	1.38	Active Layer
25	1.39	Active Layer
30	1.40	Active Layer
35	1.41	Active Layer
40	1.46	Active Layer
45	1.52	Active Layer
50	1.59	Active Layer
55	1.64	Active Layer
60	1.73	Active Layer
65	1.77	Active Layer
70	1.84	Active Layer
75	1.91	Active Layer
80	1.96	Active Layer
85	2.00	Active Layer

Notes:
1. Active layer thickness based on 0°C isotherm
2. Location of active layer thickness shown on Figure 11
3. Time express as number of years from start of South Dam operation

srk consulting	MAC RESOURCES	Doris TIA Thermal Modeling
Job No: 1CT022.004 Filename: ModelSWDams.pptx	HOPE BAY PROJECT	**West Dam – Tailing Active Layer (Year 0 to 85)**
		Date: 6/2/2016 | Approved: cws | Figure: **13**

Appendix J – Hope Bay Project: Phase 2 Tailings Impoundment Area, South Dam and West Dam Settlement, and Tailings Frost Heave Evaluation

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Tia Shapka-Fels, EIT Peter Luedke, EIT Iozsef Miskolczi, MASc, PEng	**Project No:**	1CT022.013
Reviewed:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Phase 2 TIA, South and West Dam Settlement and Tailings Frost Heave Evaluation		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix J as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
KIA-IR173	Section 4.2.1	Freezing point depression

1 Introduction

1.1 General

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Phase 1 tailings are deposited sub-aerially in the Doris Tailings impoundment Area (TIA), formerly Tail Lake, located approximately 5 kilometres from the Doris Mill. Containment is provided by three retention structures; a water retaining frozen core dam (North Dam), a frozen foundation tailings containment dam (South Dam); and an Interim Dike situated at approximately the midpoint of the facility.

Phase 2 tailings deposition would include a continuation of the Doris TIA with raising of the South Dam and construction of a new West Dam. Tailings would be deposited sub-aerially starting at spigot points along the crest of the raised South Dam and the West Dam, as well as additional spigot points along the eastern perimeter.

At closure, once water quality discharge criteria are met within the Reclaim Pond, the North Dam would be breached returning the pond water level to the pre-mining elevation of 28.3 m. The tailings surface will be covered with a nominal geochemically suitable waste rock cover of 0.3 m thickness. The function of the cover is to prevent dust and to minimize direct contact by animals and humans.

1.2 Objectives

1.2.1 Settlement Evaluation

The South and West dams will be constructed on frozen foundation soils and are designed as frozen foundation dams. Thermal modeling shows that the key trench and liner of both dams remain frozen at critical elevations over their 25 year design life (SRK 2017a). This technical memorandum provides details of the settlement that would occur should the foundation sediments thaw under the entire structure.

1.2.2 Frost Heave Evaluation

Tailings will be deposited from spigot points along the South and West Dams, and along the eastern perimeter. The maximum thickness of the final tailings deposit will be about 20 m, tapering off around the perimeter and toward the North Dam. This memorandum provides details on the expected magnitude of frost heave due to talik freeze-back and the effect of frost heave on the integrity of the proposed closure cover.

2 Site Conditions

2.1 South and West Dam Geometry

The South and West Dams are frozen foundation dams with a geosynthetic clay liner (GCL) keyed into the permafrost overburden foundation. The dams will not have ponded water against them as tailings will be beached from the crest of the dams pushing the supernatant pond water, i.e. Reclaim Pond, away from the dams. To accommodate Phase 2 tailings, the Phase 1 South Dam will be raised, and the West Dam will be a new structure.

The Phase 2 raise of the South Dam will be a downstream raise following the same geometry as the Phase 1 design. The final crest width will be 10 m, the upstream slope 4H:1V, and the downstream slope 2H:1V (Figure 1). The crest elevation is raised 8 m from 38.0 to 46.0 m and results in a maximum dam height of 15 m and total dam length of 520 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 previously installed as part of the Phase 1 South Dam will be extended vertically to reach an elevation of 45.0 m at a slope of 4H:1V.

The West Dam was designed with the same cross section as the South Dam. This dam will be constructed in a single stage, and will be about 470 m long with a maximum height of 5 m.

2.2 Foundation Conditions

2.2.1 South Dam

Sub-surface investigations has been completed along the foundation of the Phase 1 South Dam (SRK 2003, 2005, 2007), but not along the footprint of the Phase 2 raise. It is however reasonable to assume that foundation conditions under the raise would be similar to those under Phase 1.

Foundation conditions are variable, with the overburden at its maximum thickness along the center of the proposed alignment, thinning towards the abutments. 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 thick overlying basalt bedrock.

The entire profile is cold permafrost (-8˚C near-surface temperature), with an active layer thickness of about 1 m. The marine silt and clay are ice rich with clear ice lenses present. Salinity results from samples collected in the footprint of the South Dam foundation indicate salinity ranges from 6 ppt to 86 ppt, with an average of about 47 ppt. This results in a depressed freezing point of about -2.6˚C (Andersland 2004).

Thermal modeling of the South Dam (SRK 2017a) confirms that the foundation soils beneath the dam will remain frozen. At the end of the design life, the active layer downstream of the dam (-2°C isotherm) is approximately 1 m below the ground surface. The isotherm roughly follows the profile of the dam, and as such, rises toward the crest into the dam fill. With this thermal profile, the critical foundation beneath the key trench and crest does not thaw, thus any thaw effects will be limited and localised to the downstream toe (SRK 2017a).

2.2.2 West Dam

Sub-surface characterization at the West Dam is limited to a single drill hole (SRK-39), completed in 2003 (SRK 2003). This drill hole suggests a layer about 7 m thick of marine silty clay overlying basalt bedrock. Bedrock outcrops at both abutments suggests thinning of the overburden towards the abutments, similar to the South Dam. Ice content in the overburden soil formations was assumed to be consistent with the ice content of the South Dam foundation soils.

Similarly, at the end of the design life, the active layer (based on the -2°C isotherm) was assumed to be approximately 1 m below the ground surface near the downstream toe of the West Dam. The isotherm roughly follows the profile of the dam, and as such, rises towards the crest, into the dam fill. With this thermal profile, the critical foundation beneath the key trench and crest does not thaw and any thaw effects will be limited and localized to the downstream toe (SRK 2017a).

2.2.3 Tailings Area

Site characterization and thermal analysis (SRK 2017a) suggest a closed talik beneath the TIA (formerly Tail Lake). The overburden profile underneath the TIA can be inferred from site characterization under the North and South Dams, as well as drill hole completed through the TIA (SRK 2017b). The overburden thickness ranges from about 15 m at the North Dam to about 34 m at the South Dam, and consists of predominantly marine silt and clay, overlying gravelly till and basalt bedrock.

Thermal modelling (SRK 2017a) confirms that tailings freeze back will occur and that permafrost will extend into the foundation soils underlying the deposited tailings, ultimately removing the closed talik.

2.3 Material Properties

Material properties for the analysis was based on the North Dam (SRK 2007), and Phase 1 South Dam (SRK 2015) designs, supplemented with the site wide geotechnical design properties (SRK 2017b). Table 1 summarize the properties used in the analysis.

Table 1: South and West Dam Foundation and Material Properties

Parameter	Run-of-Quarry Rock (Dam Fill)	Thawed Marine Silt and Clay	Tailings
Dry Unit Weight (kN/m³)	19.12[1]	12.67[1]	12.75[9]
Saturation (%)	30.0[2]	84.7[3]	100.0[7]
Gravimetric Water Content (%)	4.6[1]	34.2[2]	41.8[9]
Moist Unit Weight (kN/m³)	20.0[2]	17.0[4]	18.1[9]
Specific Gravity	2.80[5]	2.70[2]	2.85[8]
Void Ratio	0.44[1]	1.09[1]	1.20[8]
Compression Index	-	0.3[6]	0.1[7]

Notes:

[1]Calculated value based on sources 2,4,5 below.
[2]SRK 2011
[3]Assumed value based on location and high excess ice content prior to thaw.
[4]SRK 2015
[5]EBA 2011
[6]Assumed value based on range of laboratory testing data and expected average value of silts and clays.
[7]Assumed at deposition
[8]SRK 2017c
[9]Calculated based on sources 7 and 8

3 Settlement Evaluation

3.1 Approach

3.1.1 Analysis Assumptions

The settlement evaluation assumes that settlement at each dam site will only occur if the foundation thaws. If that happens the settlement will occur as a result of consolidation in the foundation soils, and as a result of the ice content in the foundation soils. The analysis assumes the following:

- Marine silt and clay underlie the structures;
- The partial thaw depth considered in this analysis was 1 m, and was assumed to occur uniformly across the entire dam footprint. 1 m is a conservative depth as the predicted thaw is only 1 m at the toe of the dams (SRK 2017d);
- The maximum thaw depth considered in this analysis was 7 m, and was assumed to occur uniformly across the entire dam footprint (SRK 2017d);
- Following foundation thaw, the marine silt and clay is in a normally consolidated state prior to the application of the full load induced by the entire dam structure;
- After foundation thaw, the excess of pore water pressure is equal to the contact pressure at the ground surface exerted by the entire dam structure; and
- Consolidation of the underlying frozen marine silt and clay is negligible.

3.1.2 Settlement due to Foundation Consolidation

Consolidation settlement, δ_c, was evaluated using the one-dimensional consolidation theory from Terzaghi (1943):

$$\delta_c = \frac{H}{1+e_0} C_c \log\left(\frac{\sigma' + \Delta\sigma'}{\sigma'}\right) \quad (1)$$

where:

C_c = compression index of the marine silt and clay (dimensionless);
e_0 = initial void ratio of the marine silt and clay (dimensionless);
H = thickness of the consolidation layer (m);
σ'= Average in-situ effective stress in the consolidations layer (kPa); and
$\Delta\sigma'$= Stress increment in the consolidation layer due to the weight of the dam (kPa).

3.1.3 Settlement due to Foundation Ice Content

Settlement due to the foundation ice content considers both excess (i.e. massive) and non-visible ice. Andersland (2004) attributes settlement from thaw of non-visible ice to 9% of the total thaw depth. Settlement associated with thaw of excess ice is calculated as the percentage of excess ice in the foundation soil multiplied by the thaw depth.

3.2 Results

The settlement results are summarized in Table 2. To account for uncertainty associated with the actual foundation conditions under the structures, a sensitivity analysis was completed with foundation ice content values of 10%, 20% and 30%.

Table 2: Total Settlement with Foundation Thaw of 1 m and 7 m

	Excess Ice (%)	Excess Ice Settlement (m)		Non-Visible Ice Settlement (m)		Consolidation Settlement (m)		Total Settlement (m)	
Structure / Thaw Depth		**1 m**	**7 m**	**1 m**	**7 m**	**1 m**	**7 m**	**1 m**	**7 m**
South Dam	10%	0.10	0.70	0.09	0.63	0.28	1.12	0.47	2.45
	20%	0.20	1.40	0.09	0.63	0.28	1.12	0.57	3.15
	30%	0.30	2.10	0.09	0.63	0.28	1.12	0.67	3.85
West Dam	10%	0.10	0.70	0.09	0.63	0.21	0.70	0.40	2.03
	20%	0.20	1.40	0.09	0.63	0.21	0.70	0.50	2.73
	30%	0.30	2.10	0.09	0.63	0.21	0.70	0.60	3.43

This simplified analysis provides a conservative range of settlement that could occur over time. Based on the thermal modelling completed by SRK, the dam settlement by the year 2100 will be less than the 1 m case as the foundation is not expected to fully thaw. The 7 m thaw case represents a long term potential settlement case that is not plausible under the expected long term field conditions, and is only used as a theoretical absolute limit (on a millennial time scale).

4 Frost Heave Analysis

Frost heave is a common phenomenon in cold regions resulting in a rise in elevation of a structure or facility due to massive ice lens formation or volume change associated with the freezing of pore water. Frost heave could occur in the foundation or within the rock fill structures. If not managed frost heave has the potential to cause damage to structures and facilities.

4.1 Approach

4.1.1 Analysis Assumptions

The South and West Dams will be constructed on frozen overburden foundations, well outside of the former Tail Lake footprint, and thus outside of the influence of the lake talik. Frost heave of the dams associated with foundation freeze back will therefore not occur. Furthermore, the dams are constructed with non-frost susceptible materials and therefore frost heave within the dam structure will not occur.

The Doris TIA will be constructed in the basin of the former Tail Lake, and the tailings will be displacing the lake water and occupy the former lake bed. The thermal modelling completed for the TIA (SRK 2017a) indicates that complete talik freeze-back will occur. Freeze-back of the pore water in the lake sediments and foundation soils within talik will cause frost heave of the tailings deposit and the overlying closure cover. Likewise the tailings are placed in a thawed and saturated state, and as the tailings freezes back frost heave will be experienced.

4.1.2 Heave Assessment of Tailings

Two mechanisms of frost heave were identified as potentially impacting the structures associated with the Doris TIA, as follows:

- Freezing of pore water; and
- Segregated ice formation.

Differential heave resulting from massive ice lens formation could compromise the physical integrity of the cover, while generalized uniform heave over large areas as in the case of pore water freeze-back would not necessarily cause undue differential deformations.

A relative upper bound of deformation due to pore water freeze-back was calculated based on the fact that a unit volume of liquid water will experience an increase in volume of about 9%. The one-dimensional deformation was computed using the following equation:

$$s = \frac{H}{1+e_0}\Delta e \quad (2)$$

where:

s: Magnitude of deformation (m)
H: Thickness of the active zone (m); $H = 2.0$ m
e_0: Initial void ratio in the active zone by the time of cover construction (unitless); $e_0 = 1.2$
Δe: Void ratio increase due to freezing of tailings pore water (unitless); $\Delta e = 0.09e_0$

Estimating the potential and the magnitude for segregated ice is difficult, and due to lack of relevant data was not completed for this stage of project planning; however, a qualitative analysis is presented in this section. Segregated ice causing frost heave can form in fine-grained soils if a constant supply of water is available. At the beginning of the cover construction, the talik under the TIA will be bounded on three sides (east, south, and west) and will have an ample water supply from the unfrozen portion of the talik underlying the Reclaim Pond.

4.2 Results and Discussion

4.2.1 Frost Heave due to Freezing of Pore Water

Tailings pore water will freeze at 0˚C relatively fast following deposition, and mostly within the period of active tailings deposition. Therefore, by the time the closure cover will be constructed, there will be no further frost heave associated with the actual tailings freeze back. The active layer within the tailings will initially be about 1.5 m, and by the year 2100, accounting for climate change, the tailings active layer would have increased to about 2.0 m (SRK 2017a). Freeze and thaw of this active layer will result in minor frost heave and thaw settlement changes on an annual cycle.

Under the conservative assumption that the tailings of the active layer, before cover construction, have the same void ratio as at time of deposition and are completely saturated, the maximum magnitude of frost heave was calculated to be 9.8 cm, considering pore water in the foundation soils will freeze in time as the talik closes. Detailed soil properties for the soils underlying the TIA are not available; therefore, soil properties obtained from Doris Lake samples will be used for this assessment. Assuming a conservative upper bound overburden thickness of 30 m, and an average void ratio of 1.14 (SRK 2009), the magnitude of the frost heave is estimated to not exceed 1.5 m.

The upper bound of the heave expected due to pore water freezing is expected to be about 1.6 m. Furthermore, the heave is expected to be widely distributed, with little differential surface deformation and gentle transition zones.

A freeze point depression (below 0°C) will not increase the predicted frost heave. In this case the magnitude of frost heave would reduce proportionally with the amount of liquid water remaining in the frozen soil and due to the freeze point depression. This liquid water does not go through the phase change, and therefore the 9% volume increase. In the specific case of the Doris TIA no tailings freezing point depression is expected because the process water will not have an appreciable level of dissolved ions which could cause a freezing point depression. Furthermore, a prediction of the freezback for the tailings based on a phase change of porewater at 0°C results in the largest prediction of the magnitude of frost heave.

The predicted freezeback for the tailings was therefore based on phase change of porewater at 0˚C which in turn resulted in the magnitude of predicted heave being the largest that can be expected.

4.2.2 Frost Heave due to Segregated Ice

Because complete tailings freeze back is expected prior to cover construction, the active layer is a closed system, i.e. excess water is not available for segregated ice formation. Frost heave is therefore limited to the volumetric change of the tailings pore water in the active layer, with little segregated ice development.

Although the properties of foundation soils may exhibit spatial variations, the significant properties with respect to segregated ice lens formation (segregation potential, hydraulic conductivity and the thermal conductivity) are expected to be similar for the deposit based on drill holes completed in other lakes on the Hope Bay property (SRK 2009). This in turn will cause frost heaves of similar magnitude, resulting in little differential heave transmitted to the tailings surface. It is expected that heave will be most pronounced in the deepest part of the tailings deposit, tapering to zero toward the edges.

4.2.3 Effect of Frost Heave on the Closure Cover

The magnitude of the frost heave due to freezing of pore water and segregated ice lenses that could potentially form in the foundation soils is anticipated to be of similar magnitude over relatively large area of the tailings surface. Therefore differential frost heave due to foundation freeze-back will result in gentle undulations of the tailings surface, rather than step-type transitions.

The low-gradient vertical displacement of the tailings surface can be easily accommodated by the protective cover due to its granular nature. The particles of the cover will rearrange to adjust to the gentle changes caused by frost heave in the tailings surface. The function of the cover as protection against wind and run-off erosion will therefore be maintained. Additionally, the low-gradient vertical displacement of the tailings surface will not compromise the stability of the cover as the friction angle in the cover-tailings interface is at least 40° (SRK 2017b), sufficient to avoid the slide of the cover over a tailings surface slope of 1.2H:1V.

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Appendix K – Tailings Impoundment Area Dust Control Strategy

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Memo

To:	John Roberts, PEng, Vice President Environmental Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Iozsef Miskolczi, PEng	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Tailings Area Dust Control Strategy for Doris TIA		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-3F, Appendix K as part of the DEIS for Phase 2 of the Hope Bay Project dated December 2016.

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
No material changes		

1 Introduction

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases; Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Two tailings storage areas are planned for the project. The existing Doris tailings impoundment area (TIA) will be expanded, and a new Boston tailings management area (TMA) will be developed. The Doris TIA tailings deposition will consist of subaerial tailings deposition, while the Boston TMA will be comprised of filtered tailings developed as a dry-stack. This memo is addressing dust management strategies for the Doris TIA.

Two tailings streams will be produced; flotation tailings, comprising approximately 92-94% of the overall volume, and detoxified leach tailings (following cyanidation, and subsequent cyanide destruction), comprising about 6-8% of the overall volume. Only flotation tailings will be deposited in the Doris TIA. The detoxified leach tailings will be filtered, mixed with mine waste rock and used for underground mine backfill.

Upon closure, the tailings surface of the Doris TIA will be covered with a nominal waste rock cover of about 0.3 m thick. The function of the cover is to prevent dust and to minimize direct contact by terrestrial animals. Once the water quality in the Reclaim Pond has reached the required discharge criteria, the North Dam will be breached allowing the TIA to return to its pre-mining elevation of 28.3 m.

Throughout the operational phase, portions of the tailings surface will be exposed, and sufficiently inactive such that they would dry out and pose a dusting risk. This memo describes alternative dust management strategies that have been considered and presents the rationale for selection of the preferred strategy.

2 Definition of Dust

2.1 Fugitive Dust

Fugitive dust is particulate matter suspended in air by wind action and human activities. Within the Doris TIA, tailings will be deposited by hydraulic placement of a tailings slurry which does not generate any fugitive dust. Fugitive tailings dust will however be generated during the period when the tailings closure cover is being constructed.

2.2 Aeolian Dust

Aeolian dust is defined as particles that are transported as suspended load due to wind action on a surface. Although tailings are discharged wet, the surface eventually dries out as a result of evaporation or freezing of the tailings surface. As a result, at any given time, large areas of the tailings surface would expose dry tailings. Aeolian tailings dust is expected because the Project site is prone to high winds and the moderate surrounding topography does not offer effective protection from wind.

3 Typical Dust Control Methods

3.1 State of Practice

Dust control from operating and closed tailings impoundments is a significant concern in the mining industry, and as a result, the state of practice is quite advanced. There are three primary dust control strategies for fugitive and aeolian dust from exposed tailings areas: natural dust control, physical dust control and chemical dust control. Natural dust control specifically relies on maximizing the benefits offered by nature in the form of precipitation (rain and snow). While highly effective, these benefits are opportunistic and may not always be available at the times when it may be needed.

Physical dust control is by far the most effective strategy, as it relies on creating a physical barrier, such as a cover, that would preclude dusting. This may however not be a cost efficient strategy for an operating tailings impoundment, since any interim cover would occupy space within a tailings impoundment that would otherwise be required for tailings.

Chemical dust control relies on modification of the tailings surface that generates the dust. The effectiveness of this method is temporary, but its application is typically simple, making it a very good alternative for managing dust from an operating tailings impoundment.

The sections that follow provide a detailed description of all the dust control methods that are currently being used in the industry, with a specific focus towards their potential applicability for this Project.

3.2 Natural Methods

3.2.1 Snow Cover

If early in the fall season, wet snow falls directly on the exposed tailings surface and subsequently freezes, it will remain in place all winter protecting the tailing surface from dusting. Snow that falls

later in the season is typically drier and more powdery and it tends to be subject to wind transport and redistribution (drifting). This means that portions of the tailings surface will become exposed and opportunity for dust release increases. This is exacerbated by the fact that during the winter the tailings surface gets extremely dry as a result of freezing, making it highly susceptible to dusting.

To maximize the potential benefits offered by snow as a natural dust control method, any snow that does fall on the tailings surface can be track compacted in areas where the tailings surface is trafficable. By mechanically compacting the snow, it will stay in place longer and will melt at a much slower rate in the spring, extending the useful life of the snow as a dust control method.

It is however important to minimize the amount of tailings that gets deposited over the compacted snow. If the compacted snow does not melt during the subsequent summer season due to the insulating blanket of the overlying tailings, ice lenses within the tailings impoundment are created which result in a loss of tailings storage space and possible instability.

There is sufficient snowfall at the Project site that this dust control method could be effectively used. In addition, there is a requirement at the Project site for snow removal in specific areas. Snow that is removed could be hauled to the TIA and used specifically for the purpose of creating a compacted snow cover over any temporarily inactive tailings surface areas. Due to the temporary nature of this dust control method, it will not be a complete solution, but would be a practical and complementary method.

3.2.2 Ice Cover

Similar to compacted snow, an ice cover will remain in place for the duration of the winter and thus temporarily mitigate dust migration. Ice cover on exposed tailing surfaces can be achieved by various methods, including ponding water during freezing weather and mechanical placement of ice blocks imported from a different source (contact water ponds).

Water can be held back in specified locations and retained there during the shoulder seasons when freezing weather will create an ice cap. Once the ice cap is achieved the open water beneath the ice can be drained off, leaving an ice cap.

The ice cap can also be created mechanically by loading ice from contact water ponds (or fresh water streams) into haul trucks and dumping the ice on the tailings surface.

Similar to compacted snow, care must be taken to ensure that the amount of tailings deposited over an ice cover is limited to avoid entraining long-term ice in the TIA.

There are several contact water ponds throughout the Project site all of which must be managed such that they are normally empty. Contact water ponds are; therefore, unable to provide a reliable source of water to use to create an ice cover. Fresh water cannot be readily hauled to the TIA to create an ice cover as the use of fresh water is governed by the Water License (2AM-DOH1323); therefore, creating an ice cover for dust control is not considered a viable practical alternative for application at the Project.

3.3 Physical Methods

3.3.1 Water – Surface Wetting

Water is by far the most common temporary dust control measure used in areas where water shortage is not of concern. The exposed surface is wetted up, preventing particles from becoming airborne. Since the water rapidly evaporates (in a matter of hours or days), it needs to be reapplied at a frequent interval to be effective. The surface wetting can be done using a conventional water truck, a water cannon fitted to a water truck, or a stationary sprinkler system. Naturally this dust control method is only applicable during non-freezing periods of the year.

For the Project, water could readily be obtained from the Reclaim Pond or can be hauled via water truck from other site contact water ponds. The tailings surface is however not expected to be trafficable in the short term and the only viable means of frequent tailings wetting would be via a water cannon, or a sprinkler system. While both of these methods are viable, the short useful life of every wetting cycle makes this a very labor intensive dust control method which is not preferred. This method will however be reserved as a last line of defence should any of the other dust control methods prove to be ineffective.

3.3.2 Water – Flooding

Flooding the tailings surface will naturally preclude any dust concerns. This is however not a viable strategy for the Project since the objective is to place tailings subaerially. At Doris, TIA portions of the tailings may be seasonally flooded as the water level in the Reclaim Pond rises; however, the water level will be managed such that a perpetual water cover will not be present.

3.3.3 Permanent Dry Cover

The most effective permanent dust control system is a permanent physical dust cover. Typically this is in the form of a layer of soil, or other suitable readily available cover material. This is however not practical until the tailings surface has reached its final elevation. In order to facilitate placement of a final dust cover as expediently as possible, any tailings deposition plan should be designed taking into consideration all opportunities for progressive reclamation.

In the context of the Doris TIA, the tailings deposition plan provides limited opportunity for progressive reclamation during the early Project life. This is predominantly driven by the surface topography and as a result there are no practical means to improve the design. The only viable permanent dust cover would be geochemically suitable waste rock, or quarry rock. Since all the Project waste rock is designated for use as structural underground backfill, only quarry rock can be considered a viable source for a permanent dust cover. While this will be the final closure dust control method, it is not considered a viable method during the operational phase of the Project.

3.3.4 Sacrificial Dry Cover

In extreme cases, nominal sacrificial covers such as a layer of sand or gravel are used to manage tailings dust when the final tailings surface has not yet been reached, but the period until tailings deposition might resume at any particular spot may be extensive. When tailings deposition eventually returns to the covered area, these materials are not removed and tailings deposition proceeds to overtop the sacrificial cover. This can be very cost intensive and will only be practical if the tailings surface is readily trafficable.

There are no suitable natural sacrificial cover materials readily available at the Project site. Gravel could be produced from quarry rock; however, at great cost. This is therefore not considered a viable dust control strategy for the Project TIA.

3.3.5 Biodegradable Cover

Biodegradable material such as hay, wood mulch or sewage treatment sludge can be applied over exposed tailings surfaces to mitigate dust for a limited period (i.e. requiring occasional reapplication). Naturally this option is only economically viable if the organic source is readily available. The tailings surface must also be sufficiently trafficable to allow equipment to spread these materials. As these materials biodegrade and dry out, they themselves become prone to being part of the dust hazard.

There is no viable source of biodegradable materials at the Project site, and therefore this is not considered a viable dust control strategy for the Project.

3.3.6 Wind Barriers

A wind barrier (aka windbreak or shelterbelt) is a physical structure used to reduce the wind speed, which will reduce tailings from being re-mobilized from the TIA. Typically, a wind barrier consists of one or more rows of trees or shrubs. Trees and shrubs don't grow at the Project site (at least not to the size where they would be effective wind barriers), therefore, any wind barriers would have to be engineered structures. The efficiency of wind barriers is also a function of wind speed, and often, at very high wind speeds, wind barriers can fail since it is simply not cost effective to design and build these structures to withstand large wind velocities. As well, wind barriers only work effectively over a very narrow range of wind directions. Multiple wind barriers would need to be installed to cover all of the Project's prevalent wind directions so as to provide a comprehensive dust management system for the TIA.

Given the very high wind speeds and the multiple wind directions, experienced at the Project's TIA, engineered wind barriers are not be considered a viable dust control strategy for the Project's TIA.

3.3.7 Vegetation

Revegetating an exposed tailings surface is a very effective way to mitigate dust. In an arctic setting such as at the Project site, this is not a practical option since the growth season is simply too short to allow for rapid onset of effective vegetation. In addition, the tailings material may not be amenable to supporting vegetation without the addition of supplemental nutrients, which might preclude establishment of natural successional vegetation species. This is therefore not a viable dust control method for the Project.

3.4 Chemical Methods

3.4.1 Salt (Calcium Chloride)

"Salted" sand will not freeze at temperatures above -10˚C, and can be spread in a thin layer over exposed frozen tailings surfaces during the shoulder seasons when frost penetration is enough to support the spreader truck (or other suitable spreader mechanism). The calcium chloride in the sand acts to melt the frost on the exposed tailing surface and stops the fine particulate dust particles from becoming airborne.

There are no sources of sand at the Project site, requiring that both sand and salt would have to be imported at great cost. As runoff occurs from the tailings surface, the salt will dissolve reducing the efficiency; however, since this mitigation method is best used during freezing conditions this risk is limited. However, during freshet the salt is washed off towards the Reclaim Pond which results in an increased salt load to the TIA, which may limit the use of TIA reclaim water to the mill. This is therefore not a viable dust control strategy for the Project TIA.

3.4.2 Chemical Suppressants

There are many environmentally safe commercial chemical dust suppressants on the market. Although originally developed for other forms of fugitive dust management, they are routinely used for dust control on tailings surfaces. These products work in different ways, but principally they all either chemically bind dust, or alternately facilitate towards development of a crust to prevent particles from separating and becoming airborne.

The chemical suppressants are normally supplied in concentrated liquid form in containers of various sizes. They are typically water based and are diluted before application at a ratio of about nine parts water to one part suppressant. The solution is applied by means of a spray cannon mounted on a modified water truck, but can also be done via hand held sprayers. The application rate is typically about four liters per square metre.

Chemical suppressants have a useful life which is dependent on the concentration applied and local weather conditions. Normally, products are applied at a concentration which would render a useful life of approximately one year.

Of all the dust control methods, chemical suppressants offer the greatest flexibility for application at the Project TIA. The concentrated liquid can be shipped to site on an annual basis and solution can be mixed an applied on site as required. The relatively long useful life limits the amount of effort that needs to be exerted and therefore makes the dust control method practical.

4 Dust Control Procedures for Tailings

The primary dust control measures of the Project site tailings facilities will be the use of environmentally suitable chemical dust suppressants. The application of these suppressants will be reviewed on an ongoing basis to ensure that any areas that may be at risk will be adequately covered. Generally, annual application of chemical suppressants will be applied; however it is recognized that more frequent applications may be required as discharge locations are changed throughout any year.

In addition to chemical dust suppressants, natural dust control in the form of packed snow when available will be used as far as practical. Again, the effectiveness will vary on a year by year basis depending on how deposition points vary for any given winter season.

Finally, if for any reason, any of the above dust control methods prove to be temporally ineffective, a suitable water cannon will be available to allow for dust suppression in the form of spraying of the areas of concern.

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Appendix L – Detailed Cover Design

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Memo

To:	John Roberts, PEng, Vice-President Environmental Affairs Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Kyle Scale, PhD, EIT Iozsef Miskolczi, PEng	**Project No:**	1CT022.013
Reviewed by:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Preliminary Doris Tailings Impoundment Area (TIA) Cover Design		

1 Introduction

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. The Project is located 705 km northeast of Yellowknife and 153 km southwest of Cambridge Bay in Nunavut Territory, and is situated east of Bathurst Inlet. The Project comprises of three distinct areas of known mineralization plus extensive exploration potential and targets. The three areas that host mineral resources are Doris, Madrid, and Boston.

The Project consists of two phases: Phase 1 (Doris project), which is currently being carried out under an existing Water Licence, and Phase 2 (Madrid-Boston project) which is in the environmental assessment and regulatory stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Ore processing at Doris includes milling and processing to produce gold doré. Processing at Madrid North includes producing a concentrate for further processing at Doris. Flotation tailings at both Doris and Madrid will pumped to the Tailings Impoundment Area (TIA) as a low solids content slurry for subaerial hydraulic deposition (TMAC 2017). Cyanide leach tailings at Doris will be detoxified and used as underground backfill.

The flotation tailings deposited within the TIA will not be acid generating, and although there are some potential for neutral metal leaching (SRK 2015, 2017a), water quality and tailings freeze back modeling has show that at closure mitigation in terms of oxidation or infiltration control is not required (SRK 2017b, 2017c).

2 Background

2.1 Doris TIA Closure Objectives

The closure objectives for the TIA have been extracted from the Interim Closure and Reclamation Plan (SRK 2017d) and are listed below:

- Ensure long-term physical stability of tailings surface and tailings retaining structures;
- Dust levels are safe to humans, wildlife, and vegetation;
- Minimize the risk of catastrophic or chronic release of tailings and/or tailings water into the environment;
- Minimize the risk of Acid Rock Drainage (ARD) and metal leaching potential into the surface water system to the limits acceptable by the Canadian Council of Ministers of Environment (CCME) water quality guidelines; and
- Tailings area blends in with the topography as much as possible.

At closure, as part of the decommissioning of the Doris TIA, all water from the Reclaim Pond will be pumped to the Marine Mixing Box and onward to Roberts Bay during the open water season (SRK 2017b). The small residual basin of the Reclaim Pond will be backfilled to prevent accumulation of any water. A portion of the North Dam will be removed to re-establish, as much as possible, the natural run-off drainage patterns. Runoff through the Tail Lake catchment will flow to Doris Lake and subsequently to Doris Creek.

3 Cover Options Analysis

Site-specific climate and material availability are the primary driver towards what covers may be practical at a given site. Table 1 summarize the cover types considered for the Doris TIA.

Table 1: Overview of Cover Types

Cover Type	General Description	Best Suited to	Doris TIA Applicability
Isolation	Nominal single layer of soil or rock to prevent direct contact (including dust) of with humans and wildlife	Sites where oxidation and/or infiltration control is not required	This cover will meet the stated objectives
Water	Permanent water cover to prevent oxidation	Sites where net positive climatic water balance exist and material is strongly acid generating	TIA can only practically be flooded if perpetual dams are in place and oxidation control is not required
Natural Barrier	Low permeability natural or amended soil layer that reduces infiltration	Sites where natural low permeability materials are readily available	No natural low permeability materials available and infiltration control not required
Synthetic Barrier	Geosynthetic liner as a means to reduce infiltration	Sites where natural low permeability materials are not readily available, or where a high degree of infiltration control is required	Infiltration control not required
Capillary break	Multi-layer cover to reduce infiltration and/or saturation	Sites where appropriate contrasting natural soils and gravels are readily available	No suitable contrasting natural materials available and infiltration control not required
Frozen / Thermal	Use cold climate to ensure perpetually frozen state in waste material	Very cold environment with abundance of suitable cover materials	No abundance of suitable cover materials since all available waste rock is required for backfill and infiltration control not required

4 Preliminary Cover Design

4.1 Cover Profile

The primary function of the Doris TIA cover is to preclude human and wildlife contact with the tailings. Therefore, a simple isolation cover is all that is required. The cover will consist of a 0.3 m thick Run of Quarry (ROQ) layer placed directly onto the tailings surface. The ROQ material will be geochemically suitable quarry rock or waste rock.

4.2 Cover Construction

The beach tailings, i.e., areas near the West and South Dams will consist of coarse, well-drained tailings and is expected to be trafficable, allowing for simple placement of the isolation cover using conventional earth-moving equipment at any time. Areas nearer to the North Dam will comprise finer, poorly drained tailings and will not be trafficable during summer months when the tailings is thawed. Construction of the cover over these areas will have to be completed in the winter season when the tailings is completely frozen.

4.3 Infiltration and Water Quality

A technical evaluation of the depth of infiltration into the vadose zone of the Doris TIA tailings was completed in SRK (2017c) as part of an investigation into the depth of oxidation into the tailings. Transport of metals from the pore water of the tailings to the receiving environment occurs through lateral flow of pore water, or by mixing and dilution into the surface run-off. Since the tailings have a very low hydraulic conductivity, the contaminant pathway is likely to be dominated by run-off transport. The results of this analysis indicate that lateral transport cannot occur while the tailings are frozen, and a thin frozen surface effectively seals off the tailings regardless of whether some of the tailings below are not yet frozen.

4.4 Stability and Deformation Analysis

A stability analysis for the isolation cover at the Doris TIA was completed in SRK (2017e). This analysis assumed the magnitude of the frost heave due to freezing of pore water and segregated ice lenses that could potentially form in the foundation soils was of similar magnitude over a relatively large area of the tailings surface. As a result, differential frost heave due to foundation freeze-back is expected to result in gentle undulations of the tailings surface, rather than step-type transitions. This low-gradient vertical displacement of the tailings surface can be easily accommodated by the cover due to its non-cohesive granular nature. The particles of the cover will rearrange to adjust to the gentle changes caused by frost heave in the tailings surface. The function of the cover as protection against wind and run-off erosion will therefore be maintained. Additionally, the low gradient vertical displacement of the tailings surface will not compromise the stability of the cover as the friction angle at the cover-tailings interface is at least 40° (SRK 2017f).

4.5 Boil Analysis

In the context of the cover system at the Doris TIA, boils would be the upwards migration of solids and/or water from the underlying tailings through the overlying cover material. Should this occur the cover will no longer serve as an isolation cover.

Boils develop at critical hydraulic gradients when the seepage force of water in the cover material is greater than the unit weight of the cover material. The seepage of water at the surface (boils) or erosion of solids at the surface (sand boils) therefore occurs when the upwards pressure of the water within the cover material is greater than the downwards force of the cover material.

Factors that increase the likelihood of boils forming are associated with increased precipitation, traffic on the cover, and rapid cover construction. These conditions lead to excess pore pressures, which leads to greater hydraulic gradients and seepage forces.

Means to mitigate against formation of boils include increasing the thickness of cover material, placement of the cover material over frozen tailings, and placing the cover material at a slow rate. Increasing the thickness of the soil cover result in a reduction in the hydraulic gradient and resulting seepage forces. Placement of a cover material on frozen tailings means that there will be no upward hydraulic gradient, and thus no seepage forces. Placing cover material at a slower rate will provide time for excess pore pressures (and the associated hydraulic gradient) to dissipate.

5 Cover Failure Modes and Effects Analysis

A preliminary failure modes and effects analysis (FMEA) was completed for the preliminary Doris TIA cover design in accordance with (MEND 2012). The complete analysis is documented in Attachment A. The failure mode identified as carrying the highest risk is constructability of the cover on the finer, poorly drained tailings. This could result in mixing of the cover and tailings materials which would negate the principles of isolation, which in turn would result in the need for additional cover material. Appropriate mitigation would be to implement a suitably rigorous construction schedule that ensures cover in those areas are placed during the winter when the surface is appropriately frozen and trafficable.

6 Monitoring and Maintenance

Post construction monitoring and maintenance will be required for the Doris TIA cover. This will include geotechnical inspections by a qualified geotechnical engineer. No in-situ instrumentation is however recommended or required. Water quality from the facility will be monitored as part of routine post-closure monitoring and should water quality not meet discharge criteria it can be concluded that additional TIA cover mitigation may be required.

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Attachment A: Failure Modes and Effects Analysis

FMEA Worksheet - Hope Bay Doris TIA Cover Design

Failure Mode Description	Effects and Pathways	Likelihood	Environmental Impact		Special Considerations		Legal and Other Obligations		Consequence Costs		Community / Media / Reputation		Human Health and Safety		Level of Confidence	Highest Risk Rating	Mitigation / Comments
Poor execution of construction QA/QC program and/or inexperienced personnel supervising construction	Thickness of cover layer not built to specification leading to erosion of tailings	L	Mi	L	Mo	Mo	Mo	Mo	Mo	Mo	M	Mo-H	L	L	H	Mo-H	Design isn't complicated. Therefore likelihood is considered low.
	Cover is placed when tailings are not frozen and ROQ material mixes with underlying softer tailings, compromising effectivenss of cover to limit erosion of tailings	M	Mi	Mo	Mo	Mo	Mo	Mo-H	Mo	Mo-H	M	H	L	L	H	H	Cover placement will be carried out while tailings are frozen such that the underlying tailings can support the cover system with unintended mixing of the cover material and tailings.
Insufficient volume of cover material available to complete cover system construction	Modification to the design required due to lack of suitable materials leading to construction delays and/or higher capex costs	M	L	L	L	L	L	L	Mo	Mo-H	L	L	L	L	H	Mo-H	Quantities of borrow materials will be calculated based on surveying and geotechnical testing to ensure adequate material is available prior to commencement of construction.
	Modification to the design required due to insufficient volumes of materials meeting design specs (or due to needs from other site aspects) leading to constructcion delays and/or higher capex costs	M	L	L	L	L	L	L	Mo	Mo-H	L	L	L	L	H	Mo-H	Quantities of borrow materials will be calculated based on surveying and geotechnical testing to ensure adequate material is available prior to commencement of construction.
Excessive rill/gully formation due to higher than expected runoff	Loss of material due to erosion from cover resulting in erosion of the tailings	L	Mo	Mo	Mi	L	Mo	Mo	L	L	M	Mo-H	L	L	M	Mo-H	Although tailings surface is landscaped to allow free drainage, the tailings are susceptible to hydraulic erosion, which will mobilize tailings towards the Reclaim Pond. Therefore, a tailings cover that functions to prevent wind and water erosion will be constructed.
	Loss of material due to erosion of cover system leading to unacceptable TSS levels in Doris Lake	H	L	Mo	Mi	Mo	Mi	Mo	L	Mo	Mo	Mo-H	L	Mo	M	H	Although tailings surface is landscaped to allow free drainage, the tailings are susceptible to hydraulic erosion, which will mobilize tailings towards the Reclaim Pond. Therefore, a tailings cover that functions to prevent wind and water erosion will be constructed.

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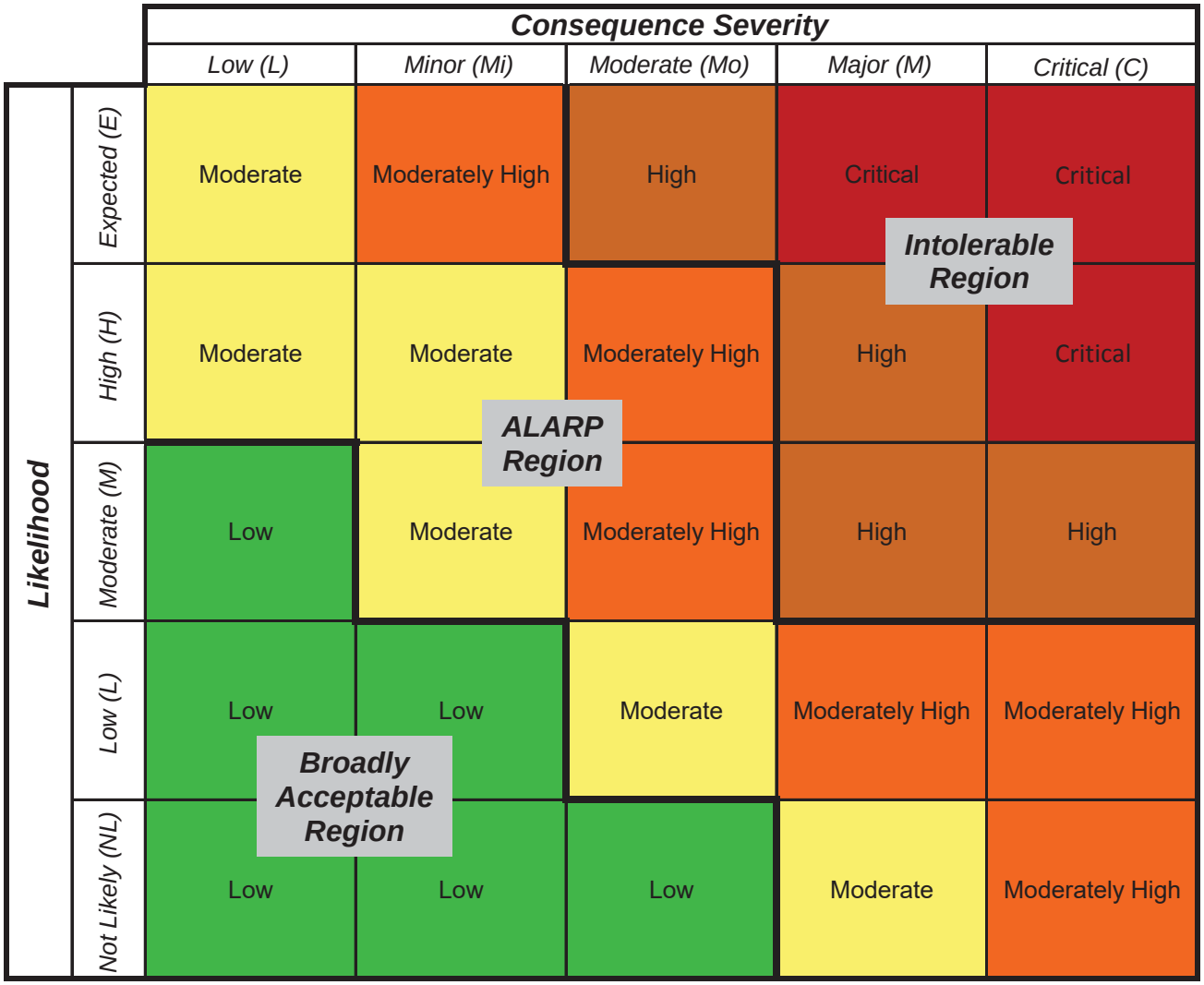
Freeze / thaw cycling of the cover system	Formation of cracks in the cover leads to exposure of the tailings	L	Mo	Mo	Mi	L	Mo	Mo	L	L	M	Mo-H	L	L	M	Mo-H	Not likelhy that structural changes resulting from feezee/thaw cylcling will lead to exposure of tailings
	Differential frost heave results in standing water on the reclaimed landforms	L	L	L	Mi	L	L	L	Mi	L	L	L	L	L	M	L	Not likelhy that structural changes resulting from feezee/thaw cylcling will lead to standing water. Mitigation methods are monitoring and maintenance and surface water management.
	Frost boils result in migration of tailings pore water and solids to surface	L	Mi	L	Mi	L	Mo	Mo	Mi	L	M	Mo-H	L	L	M	Mo-H	Likelihood of frost boils is considered low because cover material will be placed on frozen tailings.
Differential settlement or frost heave	Differential settlement or frost heave results in standing water on the reclaimed landforms	M	Mi	L	Mi	L	L	L	Mi	L	Mi	L	L	L	M	L	Granular material used for cover will conform to gentle changes resulting from differential settlement or frost heave without compromising functionality of cover.
Burrowing animals create substantial holes / macropores in the cover system profile	Formation of cracks/holes/voids in the cover leads to exposure of tailings	L	Mo	Mo	Mi	L	Mo	Mo	L	L	Mi	L	L	L	M	Mo	Due to geographical area and climate, the likelihood of burrowing animals is considered low.
Substantial damage to integrity of cover system due to anthropogenic activities	Formation it cracksof/voids system due to leads to exposure of tailings	L	Mo	Mo	Mi	L	Mo	Mo	L	L	M	Mo-H	L	L	M	Mo-H	The likelihood of people digging around on the cover over the assessment is considered low due to the isolated area and institutional controls like signage

Risk Matrix


Consequence Severity
Low (L)
Minor (Mi)
Moderate (Mo)
Major (M)
Critical (C)
Likelihood
Expected (E)
High (H)
Moderate (M)
Low (L)
Not Likely (NL)
Moderate
Moderately High
High
Critical
Critical
Moderate
Moderate
Moderately High
High
Critical
Low
Moderate
Moderately High
High
High
Low
Low
Moderate
Moderately High
Moderately High
Low
Low
Low
Moderate
Moderately High
Intolerable Region
ALARP Region
Broadly Acceptable Region

Level of Confidence

Confidence	Description
Low (L)	Do not have confidence in the estimate or ability to control during implementation.
Medium (M)	Have some confidence in the estimate or ability to control during implementation, conceptual level analyses.
High (H)	Have lots of confidence in the estimate or ability to control during implementation, detailed analyses following a high standard of care.

Likelihood of Risk

Likelihood Class	Likelihood of Occurrence for Environmental and Public Concern Consequences over Assessment Period (500 yrs)
Not Likely (NL)	< 0.1% chance of occurrence
Low (L)	0.1 - 1% chance of occurrence
Moderate (M)	1 - 10% chance of occurrence
High (H)	10 - 50% chance of occurrence
Expected (E)	> 50% chance of occurrence