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Package P5-26

Boston Tailings Management Area Preliminary Design, Hope Bay Project

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Prepared for

TMAC Resources Inc.

Prepared by

SRK Consulting (Canada) Inc.
1CT022.013
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 V3-2F 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-DEIS-56		No change. All technical analysis will be revisited when more site characterization is completed.
INAC-IR38	6.2	Under upset conditions, dewatered tailings will be recycled to the Process Plant or weather permitting, placed in thin lifts and allowed to dry.
KIA-IR162	6.2	The active tailings deposition area will be cleared of snow.
KIA-IR164	7.2.3	Sampling and testing will be completed to confirm compliance of run-off with water quality criteria prior to decommissioning of the contact water ponds.
KIA-IR166	4.9	Additional clarity on seepage estimate.
INAC-IR41	4.3 6.3 7.2.3	a) Seepage analysis to be completed at detailed engineering b) Residence period extended to two weeks c) FoS of 1.5 for long-term conditions. d) Similar design at various facilities at Doris e) Detailed in AEMP

1 Introduction

1.1 Background

1.1.1 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.

The mine plan at Boston consists of underground mining of 5.1 Mt of ore, over an approximately 8-year mine life (TMAC 2017a). Ore processing will be completed at a maximum rate of 2,400 t/d, with all stages of beneficiation being completed at the Boston Mill to produce a gold doré. Two tailings products are produced; flotation tailings comprising about 94% of the ore mass and detoxified tailings comprising the remaining 6% of the ore mass. Both tailings streams will be dewatered (filtered) separately, with the detoxidied tailings disposed of underground with waste rock backfill and the flotation tailings trucked to the Boston Tailings Management Area (TMA) for deposition in a dry stack. The dry stack is conservatively designed to contain the full ore reserve, not accounting for the detoxified tailings being deposited underground.

Operational environmental containment for the Boston TMA is provided by a series of contact water ponds. At closure, the TMA will be covered with a low-infiltration cover consisting of a geosynthetic liner and a protective rock layer.

This report documents the preliminary design of the proposed Boston TMA.

1.2 Scope of Work

SRK Consulting (Canada) Inc. was retained by TMAC to carry out the preliminary design of the Boston TMA for the Phase 2 Hope Bay 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, 2017) and publications and bulletins published by the International Commission of Large Dams (ICOLD).

In addition, in response to the 2014 Mt. 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 (BC Reg. 40/2016) and the guidelines for Site Characterization for Dam Foundations in BC (APEGBC 2016).

1.3 Report Structure

A brief description of the TMA concept is described in Section 2 while the TMA design criteria are presented in Section 3. Details of the TMA design and detailed descriptions of the supporting analyses are provided in Section 4. Section 5 lists the TMA construction details, including construction material take-off quantities. The TMA operational plan which includes the deposition plan is described in Section 6, while TMA closure is described in Section 7 and includes a brief discussion on monitoring and maintenance.

2 Tailings Management System Concept

2.1 Tailings Storage Requirements

The Boston ore reserve is 5.1 Mt and this has been considered as the design capacity for the Boston TMA.

The current Hope Bay production plan (TMAC 2017a) considers 365,000 tonnes of Boston ore will be hauled to Doris and processed at the Doris process plant during the first three years of mining at Boston. The tailings produced by processing of ore at Doris will be contained within the Doris tailings impoundment area (TIA) (SRK 2017a). The design of the Boston TMA conservatively considers the full Boston ore reserve of 5.1 Mt to be processed at Boston and does not consider the portion planned to be transported to Doris, nor the volume of detioxified tailings deposited underground.

2.2 Selection of Preferred Tailings Management System

A comprehensive tailings disposal alternatives assessment was completed for the Boston deposit in the form of a multiple accounts analysis (MAA). It was prepared in accordance with the Environment Canada guideline for disposal of mine waste (EC 2011). The alternatives assessment took into consideration technical, operational, environmental, socio-economic, and project economic factors. It also considered tailings disposal technologies, containment dam technologies, and tailings disposal sites (SRK 2017b).

The analysis concluded that the most favorable methodology is to place filtered tailings into a free-standing dry stack facility located about 1 kilometer east of the Boston processing facility, directly south of the proposed new Boston Airstrip (Appendix A, drawing BTMA-02). A portion of the contact water pond berms required to retain the run-off contact water will double as the access road to the proposed new airstrip.

The dewatered filtered tailings will be trucked to the dry-stack facility, where it will be spread in thin lifts (0.3 m thick) and compacted. The facility is continuously built up in this fashion to reach a maximum height of about 26 m, with 5 m high intermediate benches (Appendix A, drawing BTMA-04). The inter-bench slope will be 3H:1V, with an overall slope of about 3.9H:1V.
The footprint occupied by the proposed tailings facility is about 19.8 hectares, and the location offers further expansion capacity to the north.

Contact water from the TMA will be retained by a series of contact water berms (Appendix A, drawing BTMA-03) (SRK 2017c). At closure the TMA will receive a low permeability cover (Appendix B) to mitigate long term water quality concerns associated with neutral metal leaching of the tailings (SRK 2017d).

3 Tailings Management Area Design Criteria

3.1 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 practices. The foremost guidance documents in this regard 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.

The Boston TMA is however not a dam, and in absence of an appropriate hazard classification system for filtered tailings facilities, the CDA guidelines were applied.

A key component of the guidelines is classifying the dams into hazard categories (dam class) that establish appropriate geotechnical and hydro-technical design criteria. Table 1 is a reproduction of the recommended dam classifications as presented in the CDA guidelines. 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 1: Dam Hazard Classification as per CDA (2013)

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 (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; however, in the context of the TMA this was applied as the likely zone of run-out in the event of a slope failure.

The “*Population at Risk*” has been generously selected as “*Temporary Only*” due to very infrequent need for personnel to monitor the contact water pond berms in the likely flow debris run-out zone in the case of catastrophic failure. The “*Loss of Life*” has again conservatively been selected as “Unspecified” to reflect that there will be short and infrequent periods of time where persons will be present in the likely run-out zone.

The “*Environment and Cultural*” impacts associated with a breach of the TMA will be associated with a finite quantity of tailings immediately downstream of the failure zone. This run-out will likely be captured by the contact water pond berms and therefore tailings run-out reaching the Aimaokatalok Lake is not expected. If the contact water ponds were to be completely full at the time of the breach, it is conceivable that this contact water may overtop the ponds entering Aimaokatalok Lake. Although Aimaokatalok Lake is considered significant habitat, restoration of that habitat under this scenario 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 TMA hazard classification are summarized in Table 2, with “Significant” being the hazard rating adopted as the design guideline.

Table 2: Boston TMA Hazard Classification

Population at Risk	Loss of Life	Environmental and Cultural Values	Infrastructure and Economics	Overall Hazard Classification
SIGNIFICANT	SIGNIFICANT	SIGNIFICANT	LOW	SIGNIFICANT

3.2 Design Life

Ore production at Boston will be for 8 years, with the processing facility operating for 7 years. The dry-stack will therefore have an active design life of 7 years, followed by a one year closure period during which the closure cover will be constructed, and the contact water ponds breached. Post-closure monitoring is assumed to span another 10 years (SRK 2017e). The closed TMA will however remain in perpetuity. Thermal analysis of the TMA considers climate change up to the year 2100 (SRK 2017f).

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 (SRK 2017g) and are summarised in Table 3.

Table 3: Summarized Tailings Geotechnical Properties

Parameter	Value
Specific gravity	2.85
% Fines (<0.075 mm)	65%
% Silt	52%
% Clay	13%
Void ratio (e) for filtered tailings	0.6
Deposited dry density (Tonnes/m^3) for filtered tailings	1.8
Internal angle of friction (degrees)	40
Cohesion (kPa)	0
Gravimetric moisture content (%)	20.5
Hydraulic conductivity (m/s)	1.3×10^{-7}

3.1 Tailings Geochemical Properties

Detailed geochemical characterization of the Boston flotation tailings (SRK 2017h) confirms that the tailings are not potentially acid generating but have the potential for neutral metal leaching. Collection and treatment of contact water may therefore be required, contingent on the water quality predictions for the leachate (SRK 2017d, SRK 2017i).

3.2 Tailings Storage Requirement

The total quantity of ore mined at Boston is in the order of 5.1 Mt. In the first three years of operations a portion of the ore mined at Boston will be hauled to Doris for processing. Thereafter the Boston processing facility will produce separate streams of flotation tailings and detoxified tailings. The expected total reduction in flotation tailings mass produced at Boston due to ore transfer to Doris and underground disposal of detoxified tailings is in the order of 0.7 Mt. Therefore, the actual quantity of tailings sent to the TMA will be limited to about 4.4 Mt.
For planning purposes these reductions have not been considered, and the design storage capacity remains 5.1 Mt. Assuming an average density of 1.8 t/m^3 for the filtered tailings, this translates to about 2.8 Mm^3. Complete Boston tailings storage requirements are summarized in Table 4.

Table 4: Tailings Storage Requirements

Component	Value	Source
Tailings storage requirement	2.8 Mm^3 (5.1 Mt)	Quantity based on TMAC mine plan and conservative asusmptions; volume conversion based on dry density listed below in this table
Tailings production	400 tpd for first year; 1,800 tpd in second year; 2,400 tpd for remaining mine life	Supplied by TMAC.
Tailings production period	7 years	Supplied by TMAC.
Ice entrainment allowance	None	Tailings will be placed in an unsaturated state such that there is no excess water. Snow removal will be done before new tailings placement.
Run-off and contact water allowance	Provided by separate contact water ponds	Additional deisgn storage capacity not required as contact water will be temporarily contained in separate contact water ponds and pumped back to the process plant or water treatment plant.
Deposited tailings dry density	1.8 t/m^3	SRK (2017g)

3.3 Stability Criteria

The minimum factors of safety (FOS) that are applicable to, and required to be achieved for the TMA, are defined by the Canadian Dam Safety Guidelines applied specifically to tailings dams (CDA 2014), and are reproduced in Table 5.

Table 5: Minimum Required Factors of Safety in Accordance with CDA (2014)

Stability Condition	Minimum Factor of Safety
Static Assessment	
During, or at end of construction	Greater than 1.3 depending on risks assessed during construction
Long-term (steady-state seepage, normal reservoir level)	1.5
Seismic Assessment	
Pseudo-static	1.0
Post-earthquake	1.2

Note: This table is summarized from Tables 3-4 and 3-5 in CDA (2014) dam

3.4 Design Earthquake

Corresponding to the “Significant” hazard classification, the guidelines (CDA 2014) specifiy the design earthquake with annual exceedance probability (AEP) of between 1/100 and 1/1,000 years for the construction and operations stage. For long-term scenarios, i.e. post-closure, the design seismic event must be increased to AEP of 1/2,475 years. A detailed analysis of the site-specific seismic factors was completed for the Project (SRK 2017j), with a resultant Peak Ground Acceleration (PGA) value of 0.018 g for the 1/2,475 years event (Appendix C).

3.5 Inflow Design Flood (Contact Water Ponds)

For dams with the hazard classification of “Significant”, the Inflow Design Flood (IDF) is defined to be an event half way between the 1/100 and 1/1,000 years rainfall (CDA 2014). The TMA however does not require containment of water and therefore this IDF does not apply.

Contact water running off from the TMA is collected in contact water ponds with a combined IDF of the 1:100 year return period, 24 hour duration storm event (55 mm) plus the maximum daily snowmelt of 18 mm, for a total of 73 mm (SRK 2017k). The 1:100 year storm event includes allowances for climate change predicted to the year 2040 (SRK 2017f).

Based on the dry stack and contact water berms layout, three ponds will be formed (Appendix A, drawing BTMA-03). The volume to be stored in each of the ponds was determined by modelling the sub-catchments within the facility footprint and then determining the final water elevation using a combination of Global Mapper and Muck3D software. The storage capacity of each pond is summarised in Table 6. It is important to note that storage capacity in the north-west pond is less than the required storage; however, water will overflow into the south-west pond which has ample excess storage capacity.

3.6 Design Freeboard (Contact Water Ponds)

A detailed freeboard analysis was not completed at this time for the Boston TMA; however, the normal freeboard (wind setup + wave action) for the Doris TIA was found to be in the order of 1.1 m (SRK 2017a) accounting for a pond surface much larger than the Boston Contact Water Ponds.

A conservative total freeboard of 1.3 m was assumed for the contact water berms, to prevent overflow by wind setup and wave action. This total freeboard extends from the maximum elevation of the water resulting in each of the ponds from the combined IDF (full supply level) to the crest of the berm (Appendix A, drawing BTMA-06). The top elevation of the geomembrane in each of the containment berms was determined as the full supply level plus 0.3 m hydraulic freeboard.

3.7 Summary of TMA Design Criteria

A complete summary of the TMA design criteria is listed below (Table 6), and is consistent with Best Management Practices, including the Canadian Dam Association (CDA 2013, 2014) guidelines.

Table 6. Summary of TMA Design Criteria

Component	Criteria
Hazard Classification	SIGNIFICANT
Design Life • Active deposition period • Assumed Post-closure monitoring period • Long-term design basis	 • 7 years • 10 years • Up to year 2100
Tailings Production Rate	400 tonnes per day for first year; 1,800 tonnes per day in second year; 2,400 tonnes per day for remaining mine life
Tailing Moisture Content	20.5% (by weight)
Tailings Dry Density	1.8 t/m^3
Tailings Storage Capacity • By mass • By volume	 • 5.1 Mt • 2.8 Mm^3
Tailings Deposition Method	Load, haul, dump, place, and compact filtered tailings
Maximum Design Earthquake	1:2,475 seismic event; PGA of 0.018 g
Contact Water Pond(s) Inflow Design Flood	1:100 year return period, 24 hour duration storm event (55 mm) plus maximum daily snowmelt of 18 mm, for a total of 73 mm; includes climate change allowance to 2040
Contact Water Pond(s) Storage Requirement	North-east Pond: 9,957 m^3 North-west Pond: 1,984 m^3 South-west Pond: 8,762 m^3 Total: 20,703 m^3
Contact Water Pond(s) Freeboard	1.3 m normal
Dry Stack Stability Factors of Safety (Static)	1.3 during construction 1.5 during operation and closure
Dry Stack Stability Factors of Safety (Pseudo-Static)	1.0 during earthquake 1.2 post earthquake

Note: Contact Water Pond Stability assessment is provided in SRK 2017c.

4 TMA Design

4.1 Foundation Conditions

Numerous geotechnical investigations have been performed at the Project site. A surficial geology and permafrost investigation was carried out at Boston in 1996 (EBA 1996).
The investigation included air photo interpretation followed by ground truthing and completion of six onshore drill holes, followed up by laboratory testing of select geotechnical samples.
The investigation found the proposed Boston area is characterized mostly by marine deposits of silty-clay with trace sand. Small pockets of glaciofluvial deposits of coarse sand and some gravel are also present.

Project-wide overburden consists of permafrost soils which are mainly marine clays, silty clay, and clayey silt, with pockets of moraine till underlying these deposits. The marine silts and clays contain ground ice ranging from 10 to 30% by volume on average, but occasionally as high as 50%. The till typically contains low to moderate ice contents ranging from 5 to 25%. Overburden soil pore water is typically saline due to past inundation of the land by seawater following deglaciation of the Project area. Salinity measurements in the EBA (1996) investigation ranged from 3 to 48 parts per thousand, which depresses the freezing point and contributes to higher unfrozen water content at below freezing temperatures.

Permafrost at the Project area extends to depths of about 565 m, with an average geothermal gradient of 0.021°C/m. Active layer depth in overburden soil averages 0.9 m, with a range from 0.5 to 1.4 m (SRK 2017j).

Isopach maps developed from seismic surveys and exploration and geotechnical drill holes indicate that depth of overburden under the infrastructure is expected to range from 0 to 10 m, with most areas having less than 6 m of overburden. General foundation conditions, material properties for geotechnical analysis, and development of the overburden isopach surface are described in more detail in SRK (2017f).

4.2 Dry Stack Components

4.2.1 Layout

The dry stack facility will occupy a flat area just east of the Aimaokatalok Lake extension, south of the proposed new Boston airstrip. This area is separated from the mining infrastructure (SRK 2017l) by the extension of the Aimokatalok Lake and the outflow creek from Stickleback Lake (Appendix A, drawing BTMA-02).

The footprint of the dry stack facility is in the shape of an irregular heptagon, with a footprint of about 19.8 hectares. The dimensions are about 410 m in east-west direction and 530 m in north-south direction with a final height of 26 m. The height of the facility is limited by the proximity to the airstrip, in order to avoid encroachment into the airstrip exclusion zone (SRK 2017m).

The facility will be constructed in lifts of 0.3 m, spread and compacted successively over the life of the mine and 5 m high intermediate benches with side slopes of 3H:1V. Setback benches of 5 m will result in an overall slope configuration of about 3.9H:1V. The top of any given lift will be graded at 2% toward the perimeter of the facility, to prevent standing water.

Access to the facility will be gained via the Madrid-Boston all weather road, then following the Airstrip access road which doubles as the contact water pond berms in select locations (Appendix A, drawing BTMA-03). An access ramp with a nominal grade of about 8% will provide continuous access to the rising dry stack.

4.2.2 Underdrain

In temperate and very wet climates it is best practice to construct underdrainage for dry stack facilities, to preclude buildup of a phreatic surface and thereby reduce the risk of static liquefaction and slope instability.

Constructing an underdrain for the Boston dry stack facility is not practical nor necessary.
The Boston dry stack is founded on permafrost soils, and complete freeze-back of the tailings is expected within the first winter season following deposition (Appendix D). Correspondingly, an underdrain will also freeze and once the tailings thickness exceeds the active zone depth will remain frozen indefinitely.

4.2.3 Seepage Collection

The dry stack foundation is frozen, and the tailings will freeze back soon after placement (Appendix D), save for the active layer. Therefore there is no concern related to potential deep groundwater seepage. Shallow groundwater seepage emerging from the active layer will be collected in the contact water ponds. Post-closure seepage through the active layer will be limited to what may infiltrate through the low permeability cover (Appendix E). This volume of flow is considered negligible, and as a result no post-closure seepage collection is planned or required.

4.2.4 Operational Erosion Protection

As far as practical progressive reclamation of the dry-stack facility will be completed; however, at any given time there will be exposed tailings that might be susceptible to overland runoff erosion. Should this occur all eroded sediments will end up in the contact water ponds, with no risk of an uncontrolled environmental discharge. The volume of sediment trapped in the contact water ponds will be monitored and if it compromises the pond design capacity, the sediment will be removed, or the contact water ponds capacity will be increased.

4.3 Contact Water Ponds

The catchment area, which includes the dry stack facility and the surface area of the three contact water ponds is about 28.0 ha. Water retention of these contact water ponds is provided by inclusion of a geosynthetic liner tied into permafrost, i.e. a frozen foundation dam design. For this stage of the design a high density polyethylene (HDPE) membrane is assumed as the geosynthetic product to be used, but the choice of liner will be analysed in detail at the later stages of the design. Seepage analysis was not completed at this stage of the design, but will be required at the detailed engineering stage. Similar berms with geosynthetic liners at 2H:1V slope angle are in operation at the Doris Diversion Berm, Doris Sedimentation and Pollution Control Ponds, or the Doris Quarry#2 Landfarm.

The contact water pond berms were modelled and the location and geometry refined to provide the necessary containment to all run-off originating from the dry stack. The geometry of the contact water ponds was modelled to confirm that sufficient storage capacity is available. Further modeling will be completed at the more advanced engineering design stages, to optimize the shape of the ponds, maximize storage capacity, and determine the exact location of the sumps.

Design criteria and design details for the contact water ponds are provide in the Boston Water Management Report (SRK 2017c) and the Contact Water Pond Berm Design Report (SRK 2017n). A detailed stability analysis of a critical section through the berms was completed, concluding that the berms meet the FoS criteria of 1.5 for long-term conditions.

4.4 Monitoring Instrumentation

Ground temperature cables to verify the foundation thermal response will be installed below the containment berms, as well as along specific cross-sections of the contact water pond containment berms.

Deformation of the crest and slopes of the dry stack tailings will be monitored during construction and into the initial post-closure period to provide an early indication of possible instability. Monitoring will be performed through a network of survey prisms placed at appropriate intervals along the interbench berms and the crest of the facility. The prisms will be installed in large boulders imbedded within the final ROQ cover.

4.5 Dry Stack Stability Analysis

4.5.1 Foundation and Slope Stability Analysis

A comprehensive stability analysis was carried out to confirm whether the dry stack meets the appropriate design requirements as stipulated in Section 3.3. Complete details of the analysis are presented in Appendix C and the results are summarized in Table 7. The analysis considered staged construction of the facility according to the five bench heights, and the ultimate long-term stability was assessed at the end of construction, i.e. the full height of the facility.

Table 7: Dry Stack Minimum Factor of Safety

<table>
<tr><th>Analysis Method</th><th>Construction Stage</th><th>Short-term FOS (Undrained Loading Conditions)</th><th>Long-term FOS (Drained Loading Conditions)</th><th>Pseudo-Static FOS</th></tr>
<tr><td colspan="2">Minimum Required FOS</td><td>1.3</td><td>1.5</td><td>1.1</td></tr>
<tr><td rowspan="5">PLAXIS (FE)</td><td>1st Stage (Height 6 m)</td><td>1.4</td><td>1.8</td><td>1.3</td></tr>
<tr><td>2nd Stage (Height 5 m)</td><td>1.4</td><td>1.9</td><td>1.3</td></tr>
<tr><td>3rd Stage (Height 5 m)</td><td>1.4</td><td>1.9</td><td>1.3</td></tr>
<tr><td>4th Stage (Height 5 m)</td><td>1.4</td><td>1.9</td><td>1.3</td></tr>
<tr><td>5th Stage (Height 5 m)</td><td>1.4</td><td>1.9</td><td>1.3</td></tr>
<tr><td rowspan="2">SLOPE/W (LE)</td><td>1st Stage (Height 6 m)</td><td>1.4</td><td rowspan="2">2.5</td><td>1.3</td></tr>
<tr><td>5th Stage (Height 5m)</td><td>1.4</td><td>1.3</td></tr>
</table>

The dry stack meets all the required minimum slope stability FOS as prescribed by CDA (2014).

Given the low seismicity of the Project area and the results of the pseudo-static analysis, deformation of the dry stack during the design earthquake is expected to be negligible. As a result, further numerical analysis of the dry stack facility post-earthquake was not deemed necessary.

Additional analyses with SLOPE/W were completed to assess the long-term creep effects on the stability of the dry-stack. The procedure consisted in back-calculating the friction angle of the frozen marine silt and clay required to meet the stability criterion for the long-term condition (i.e., FOS=1.5). It was found that a maximum friction angle of 20° is required to meet the long-term stability criterion, compared to the 26° friction angle observed in these soils (SRK 2017j) and it was therefore concluded that creep is unlikely to compromise the stability of the dry stack.

4.5.2 Liquefaction Analysis

Liquefaction is a process by which all strength is temporarily lost from a saturated soil, and the soil behaves like a fluid. Liquefaction is normally associated with loose sandy soils, as suggested by the process commonly being referred to as "quicksand". Liquefaction is trigered by a sudden increase in pore pressure, which cannot dissipate fast enough and results in the effective stress becoming near-zero (Holtz and Kovacs 1981). In the context of the Boston TMA, liquefaction could theoretically affect the foundation and the tailings deposit; however, it is extremely unlikely to occur for the reasons described below.

In the case of the foundation, the soils are mostly comprised of marine-type silty clay deposits, with traces of sand. These types of soils are finer than the particle size distribution commonly associated with liquefaction, and are thus not susceptible to liquefaction. In addition, the foundation soils are frozen and will remain frozen indefinitely. In the worst-case scenario of the foundation becoming unfrozen, any thawing would be progressing slowly from the outside of the facility toward the middle and thus would allow timely dissipation of any excess porewater pressures. The dry stack facility will be built gradually with an average rate of rise of about 3.7 m/year (Appendix A). The tailings will be laid out in thin lifts and compacted, thus eliminating the loose state required for liquefaction. In addition, the tailings deposited in previous years will freeze over the subsequent winter, eliminating any possibility of pore pressure fluctuations except for the top 2.5 m representing the active layer thickness in exposed tailings (see Section 4.7).

4.6 Deformation Analysis

Deformation of the dry-stack will be due to two mechanisms: consolidation settlement and creep.

4.6.1 Consolidation Settlement

Settlement of the dry stack facility is the apparent displacement of the facility as a whole and is limited to consolidation of the foundation soils. The foundation will however remain frozen (Appendix D), preventing any settlement settlement due to consolidation. Therefore the dry stack facility will not experience settlement due to consolidation.

4.6.2 Creep

A detailed creep analysis was completed (Appendix F) with the objective of predicting if long-term strains occurring over the dry-stack design life will affect the performance or compromise the stability of the Boston TMA. The analysis also confirms whether the integrity of the underlying saline foundation will be affected by creep deformations. Tailings are not expected to experience creep since they are not ice-rich materials.

No creep strains were predicted below the selected threshold stress of 30 kPa. The predicted shear strain rates for the 40kPa stress (equivalent ot the weight of the dry-stack) are very low with a maximum of 3.0E-08 year^{-1} 80 years after the dry-stack completion and the maximum shear strain is 0.03 m/m (3.3%) for the same period. Maximum shear strain and shear strains rates are predicted to occur in points within the shear localization zone (i.e., inside the frozen foundation). Displacements due to creep 80 years after construction were predicted to reach a maximum of 0.15 m in the vertical direction and 0.25 m in the horizontal direction.

A long-term ductile behavior is predicted for the frozen marine silt and clay. Creep shear strains in this layer will occur very slowly and remain below the strain rate for brittle failure.

4.7 Thermal Analysis

Tailings are expected to freeze completely during the first winter season following placement, therefore a tailings freeze-back model was not completed. Seasonal thaw of the upper-most layers of tailings will create an active layer of variable thickness, which was assessed in a detailed thermal model (Appendix D) which includes consideration for climate change.

Active layer thickness of exposed tailings located outside of areas of active material placement is estimate to average 2.5 m. Once the closure cover is constructed, active layer thickness is predicted to be between 2.7 m and 3.2 m depending on tailings saturation.

4.8 Cover Seepage Analysis

Seepage through the tailings in the TMA is considered negligible due to the high placed density and the fact that tailings will freeze back and remain frozen for the foreseeable future (other than the active layer).

Although no seepage is expected through the geomembrane, a worst case scenario was analysed to provide an upper bound in case seepage does materialize (Appendix E).
The analysis concludes a potential upper bound leakage rate of 0.64 m^3/day from the TMA after closure. This leakage is only possible for about 60 days per year, from the time the top 1 m of cover thawed (assumed early August) to the time when the surface starts to freeze back (typically early October). The stated seepage rate of 0.64 m^3/day is not a constant seepage rate, but rather an event driven seepage rate triggered by precipitation events (SRK 2017i).
This seepage occurs for up to 4 days after a rainfall event during the open water season.
During the operational phase this water is collected in the contact water berms, and following closure after the contact water berms has been breached this water will revert to natural overland drainage to Aimaokatalok Lake.

4.9 Cover stability analysis

The dry stack facility will be closed by installation of a low permeability cover incorporating a geosynthetic liner. At this stage of preliminary design, a 60 mil textured HDPE is assumed as the liner component. The liner will be placed directly on the tailings surface and a 0.3 m thick protective layer of crushed gravel will be placed onto the liner. Non-woven geotextile will be used to separate the liner and the gravel, providing added protection against puncture.

In considering the stability of the cover system, a conservative analysis was completed using the limit equilibrium method on an infinite slope (Appendix C). Based on the slope grade of 3H:1V (26°) and the cover thickness of 1 m, the factor of safety against the cover material sliding off the geomembrane was found to be 1.5.

5 Construction

5.1 Construction Materials

The dry stack facility will be built entirely of filtered tailings.

Construction material for the closure cover and contact water ponds consist of crushed rock (bedding), and run of quarry (ROQ) material. The granular fill will be produced on site from one of the local approved quarries. Geological, mineralogical and geochemical details on these quarry sites are documented in (SRK 20170).

Other materials that will be used to construct these facilities include HDPE liner and geotextile. Complete details of all these materials are provided in the Technical Specifications (SRK 2011).

5.2 Construction Equipment

Typical earth moving equipment will be used for the construction of the dry stack, the cover and the contact water ponds. Tailings deposition will be completed with a dedicated fleet consisting of a front end loader, one or two articulated dump trucks (30 or 40 tonne), one bulldozer and one smooth drum 10 tonne compactor.

Construction of the contact water ponds and the closure cover will be completed using a contractor fleet of loaders, articulated haul trucks, bulldozers, and compactors. Hydraulic excavators may be used for special tasks as required. Drilling and blasting, if required, will be done 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 the Technical Specifications (SRK 2011). Quality Control will be the responsibility of the Contractor, and/or the equipment and materials manufacturer. The Engineer of Record, which will be a Registered Professional Engineer in the Nunavut Territory, will carry out Quality Assurance. Complete documentation of all QA/QC data will be provided in the relevant As-Built Reports.

5.4 Construction Schedule

Construction of the dry stack will be done year-round. The dry stack tailings material will be placed directly on the tundra, with no removal of vegetation or excavation of overburden prior to tailings placement. To ensure the permafrost foundations remain frozen, the first lift of filtered tailings should, if practical, be placed in the winter when the ground is frozen. If tailings placement must start when the ground is thawed, a layer of ROQ may be required for trafficability.

The closure cover should ideally be constructed during the warmer seasons to facilitate geomembrane seaming and welding. The gravel bedding layer protecting the integrity of the geomembrane must be constructed immediately after geomembrane installation is complete. The final ROQ layer can be placed any time of the year.

Construction of the containment berms of the contact water ponds must be done 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.

5.5 Material Quantities

Includes materials for the construction of the closure cover and the contact water containment berms.

Material quantities for the construction of the TMA are summarized in Table 8. All fill and excavation volumes represent neat volumes, i.e., “in place”, with no allowance for swelling and compaction. The liner quantities are neat quantities, with no allowance for seams and waste.

Table 8: Summary of Material Quantities

Material	Quantity
Closure Cover	
Liner Bedding Material(m^3)	60,850
Geomembrane (m^2)	202,800
Geotextile (m^2)	202,800
ROQ Fill (m^3)	142,000
Contact Water Pond Containment Berms	
Liner Bedding Material (m^3)	16,495
Geomembrane (m^2)	32,700
Geotextile (m^2)	60,095
ROQ Fill (m^3)	71,750
Transition Fill (m^3)	33,660
Key Trench Excavation (m^3)	5,580

6 Tailings Management System Operations

6.1 Operations, Maintenance and Surveillance (OMS) Manual

A standalone Operations, Maintenance and Surveillance (OMS) Manual has been developed for the Boston TMA (TMAC 2017b). This OMS Manual has been based on the existing Doris Water Licence, the Mining Association of Canada's (MAC) guideline (MAC 2011), as well as the Canadian Dam Association's Dam Safety Guideline (CDA 2014). Prior to Boston tailings deposition, this OMS Manual will need to be updated to reflect the conditions and requirements of the new applicable Water Licence yet to be issued.

6.2 Dry Stack Tailings Deposition Plan

The tailings produced by the Boston process plant will be filtered to a water content amenable to handling by typical earth moving equipment (loaders, trucks, bulldozers) and stockpiled in the mill building. When sufficient tailings accumulated to provide several truckloads, a loader will load the tailings into 30 or 40 tonne trucks which will then transport the tailings to the dry stack facility. Tailings will be end-dumped by the dump trucks and spread to a thin lift (0.3 to 0.5 m) by a bulldozer dedicated to this operation. Once spread, the tailings will be compacted to achieve the target density. For the purposes of this preliminary design, a target density of 1.8 t/m^3 was selected; however, this may change in the more advanced phases of the design based on specific testing.

The facility is built up in this fashion to reach a maximum height of about 26 m, with 5 m high intermediate benches (Appendix A, drawing BTMA-04). The inter-bench slope will be 3H:1V, with the overall slope of about 3.9H:1V.

If for any reason the filtered tailings cannot achieve the specified minimum density, those tailings will be recycled to the mill and temporarily stored in dedicated tanks until adequate filtration can be resumed. Alternatively, if weather conditions allow, the non-compliant tailings will be spread in a lift as thin as possible and allowed to dry before final compaction is completed.

During winter operations the active deposition surface will be kept clean as much as possible. Any snow blanket exceeding 10 cm in thickness will be removed prior to placement and compaction of a tailings lift. In freezing conditions the tailings will be spread and compacted immediately after, to prevent the freezing in place of un-compacted tailings.

The footprint occupied by the tailings facility is about 19.8 hectares, but the location offers the possibility of expanding this area to the north if required in the future.

6.3 Contact Water Management

Contact water from the tailings area will be retained by a series of containment berms, surrounding the facility on three sides forming contact water ponds. The east portion of the berm will double as the access road to the proposed airstrip. The north side is open as the topography is rising in this area and a containment berm is not necessary (Appendix A, drawings BTMA-03, BTMA-05).

Contact water will be collected in the contact water ponds and pumped to the surge pond for use in the process plant or pumping to the water treatment plant for treatment and discharge. The contact water ponds were sized to retain the IDF of 1/100 year rainfall plus the maximum daily snowmelt. The ponds will be emptied within two weeks of the storm event and operated normally empty.

6.4 Dust Management

A comprehensive assessment of possible dust management practices for the tailings surface is presented in Appendix G. The tailings stacking plan will be developed to, as far as practical, minimize the area of exposed inactive tailings surface that might be prone to dusting.
Beyond such mitigation by design, the primary dust control measure of the TMA 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 depending on the stacking sequence.

7 TMA Closure and Reclamation

7.1 Closure Concept

At closure, a low permeability cover will be constructed to reduce the amount of seepage expected. The geomembrane will be placed in direct contact with the tailings and will be protected by a granular cover consisting of 0.3 m of crushed rock (bedding) and 0.7 m of ROQ. Construction of the cover will be done in stages or at the end of the active deposition.

The contact water containment berms will be breached and the liner will be cut to prevent collecting any water. Several breaches may be required and will be done at the topographic lows. The balance of the berms will be left in place, as removal of the ROQ fill will could result in localised permafrost degradation.

7.2 Closure Components

7.2.1 Landform Design

The tailings facility will be built in 5 m high benches. The inter-bench slope of 3H:1V and bench width of 5 m results in an overall slope of 3.9H:1V. This slope configuration will be created during active deposition, and no resloping is anticipated to be required at closure.

7.2.2 Cover System

Water quality for combined run-off and seepage from the TMA will meet the discharge criteria (SRK 2017i). Although the thermal model indicates the majority of the tailings will be perennially frozen, the seepage resulting from the active layer will exceed the water quality guidelines for closure. To mitigate this issue, a low permeability cover will be required to reduce seepage to essentially zero. This is achieved by constructing a low permeability cover including a geomembrane (Appendix B). The geomembrane is assumed to be HDPE for the scope of this report, but a detailed assessment will be completed at a later stage of the design to confirm the most suitable geomembrane alternative. The geomembrane will be laid directly onto the tailings surface and covered by a protective non-woven geotextile and a 0.3 m thick crushed rock (bedding) layer. The final erosion protection layer of the cover will be constructed of a 0.7 m ROQ layer.

7.2.3 Water Management

Conveyance Channels

The top surface of the tailings deposit will be graded to shed water and the final cover will assume the same configuration. This water will be collected and conveyed off the top of the dry stack by appropriately designed conveyance channels. A detailed hydraulic and geotechnical design of these channels will be completed at later stages in the project planning. As the final cover layer is ROQ which is not prone to erosion, no intermediate channels are required.

Contact Water Ponds

The contact water ponds are required to temporarily detain the contact run-off water from the dry stack. Once the closure cover is constructed, there will no longer be any contact water. Compliance of the non-contact run-off with the water quality criteria will be confirmed through sampling and testing. The berms will then be breached and the contact water ponds will be decommissioned.

Discharge Criteria

Water quality from the tailings pore water will not meet Canadian Council of Ministers of the Environment (CCME) guidelines; therefore, the very low permeability cover will be constructed to eliminate as much as possible any seepage from the tailings. Water quality will be sampled at several sites within the Aimaokatalok Lake as part of the annual Phase 2 Aquatic Effects Monitoring Plan (AEMP). Additional water quality sampling could be indeintified during the permitting application process for the Type A Water Licence.

7.3 Monitoring and Maintenance

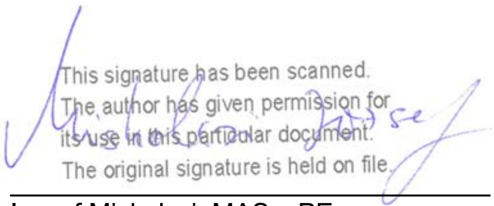
7.3.1 Monitoring

Throughout the operational phase of the Project, the contact water berms and the dry stack will be subject to rigorous monitoring to evaluate their performance. This will include thermal, settlement and other general deformation monitoring. In addition, thermal monitoring of the tailings profile will be carried out to confirm tailings freeze-back assumptions. All of the above will be subject to annual inspections by a qualified professional engineer as part of routine annual inspections. The frequency of these inspections may be reduced as time progresses in accordance with the inspection engineer’s recommendations.

7.3.2 Maintenance

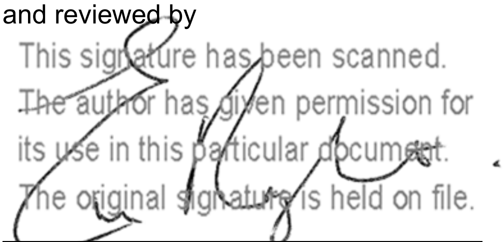
The geomembrane encapsulated in the closure cover will require maintenance and repairs. Periodic geotechnical inspections will be completed to inform of necessary maintenance work. Replacement of the geomembrane may be required and is assumed to be no more frequent than 100 years.

This final report, “Boston Tailings Management Area Preliminary Design, Hope Bay Project”, was prepared by SRK Consulting (Canada) Inc.



Iozsef Miskolczi, MASc, PEng
Senior Consultant

and reviewed by



Maritz Rykaart, PhD, PEng
Principal Consultant

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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Appendices

Appendix A – Engineering Drawings for the Boston Tailings Management Area Preliminary Design

Engineering Drawings for the Boston Tailings Management Area Preliminary Design, Hope Bay Project, Nunavut, Canada

ACTIVE DRAWING STATUS

DWG NUMBER	DRAWING TITLE	REVISION	DATE	STATUS
BTMA-01	Engineering Drawings for the Boston Tailings Management Area Preliminary Design, Hope Bay Project, Nunavut, Canada	A	Nov. 29, 2017	Issued for Discussion
BTMA-02	General Site Plan	A	Nov. 29, 2017	Issued for Discussion
BTMA-03	Tailings Management Area Plan	A	Nov. 29, 2017	Issued for Discussion
BTMA-04	Section AA and BB	A	Nov. 29, 2017	Issued for Discussion
BTMA-05	Centerline Profile Contact Water Berms	A	Nov. 29, 2017	Issued for Discussion
BTMA-06	Water Berm and Drystack Typical Details	A	Nov. 29, 2017	Issued for Discussion
BTMA-07	Details and Material Quantities	A	Nov. 29, 2017	Issued for Discussion

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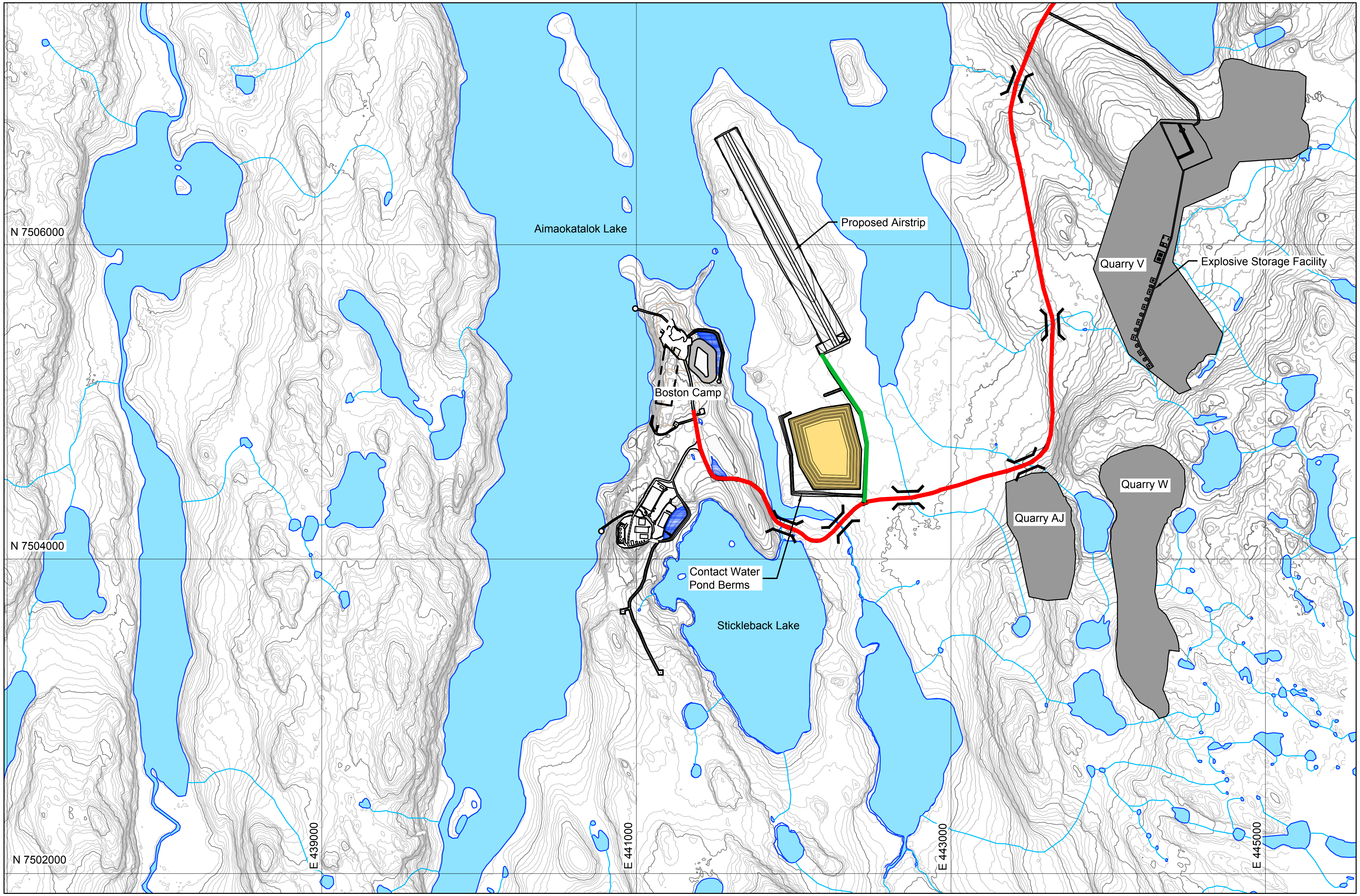
PROJECT NO: 1CT022.013-135
Revision A
November 29, 2017
Drawing BTMA-01



LEGEND

- Lakes
- Tailings Area
- Quarry Footprint
- Creeks and Rivers
- Airstrip Access Road
- Madrid-Boston Road
- Stream Crossing

NOTES

1. Contours shown at 1.0m Intervals.
2. NAD 83 UTM Zone 13

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

DRAWING NO. | DRAWING TITLE — REFERENCE DRAWINGS

REVISIONS

PROFESSIONAL ENGINEERS STAMP

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DESIGN: IM | DRAWN: NV/TH | REVIEWED: IM
CHECKED: CH | APPROVED: IM | DATE: 17/11/29
FILE NAME: 1CT022.013-137_2.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-135

Boston TMA Preliminary Design

DRAWING TITLE:

General Site Plan

DRAWING NO.: BTMA-02 | SHEET: 2 OF 7 | REVISION NO.: A

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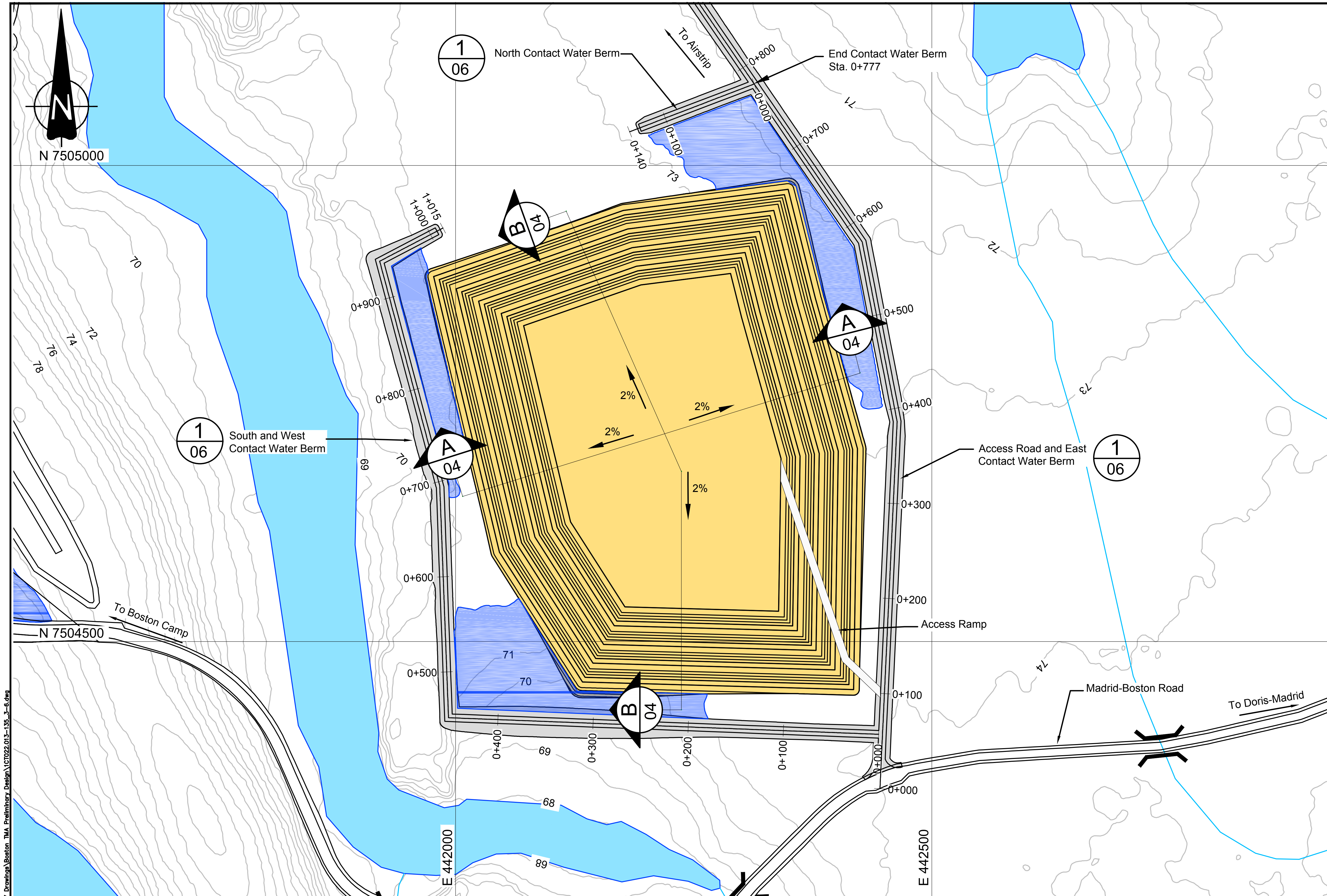

LEGEND

- Lakes
- Tailings Area
- Contact Water Pond
- Creeks and Rivers

NOTES

1. Contours shown at 1.0m Intervals.
2. NAD 83 UTM Zone 13

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

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MAC RESOURCES

DESIGN: IM DRAWN: NV/TH REVIEWED: IM
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HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-135

Boston TMA Preliminary Design

DRAWING TITLE

Tailings Management Area Plan

DRAWING NO. BTMA-03 SHEET 3 OF 7 REVISION NO. A

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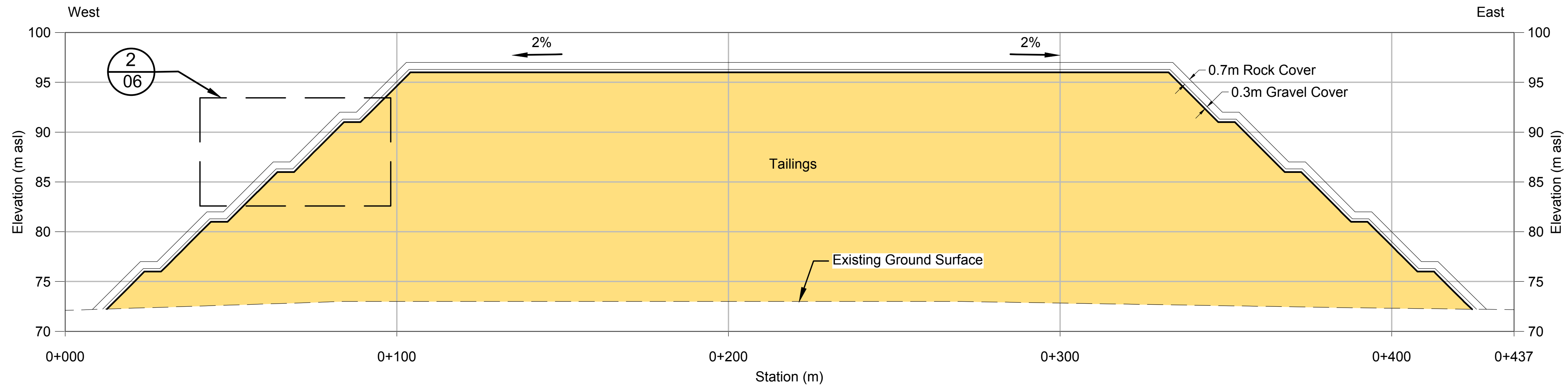

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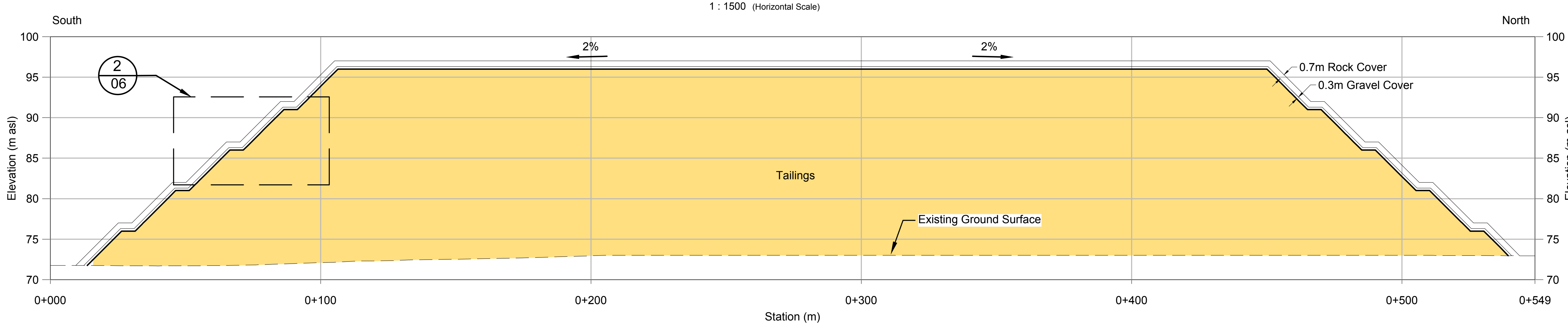

SECTION BB / 03

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REVISIONS

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DESIGN: IM | DRAWN: NV/TH | REVIEWED: IM

CHECKED: CM | APPROVED: IM | DATE: 17/11/29

FILE NAME: 1CT022.013-135_3-6.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-135

Boston TMA Preliminary Design

DRAWING TITLE

Sections AA and BB

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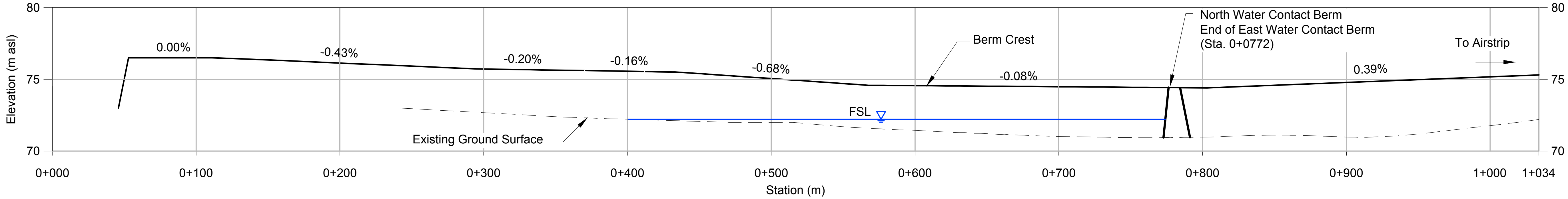

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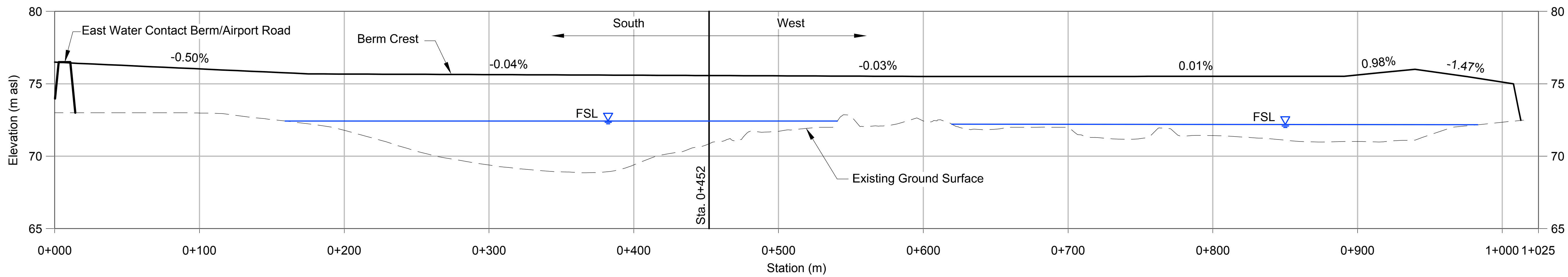

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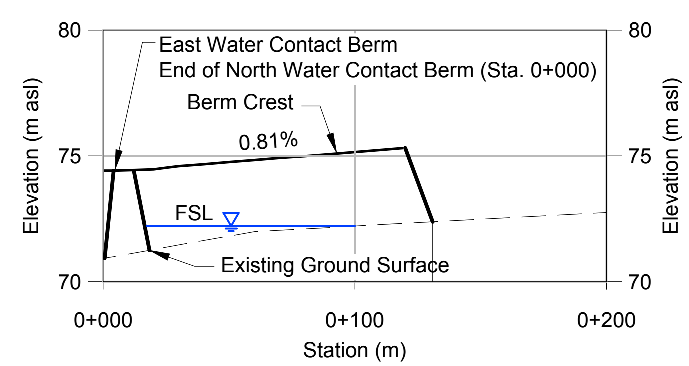
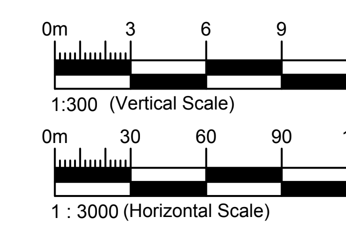


CENTRELINE PROFILE NORTH CONTACT WATER BERM



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REFERENCE DRAWINGS

REVISIONS

PROFESSIONAL ENGINEERS STAMP

srk consulting

DESIGN: MDDS	DRAWN: NV/TH	REVIEWED: IM
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TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013.135

Boston TMA Preliminary Design

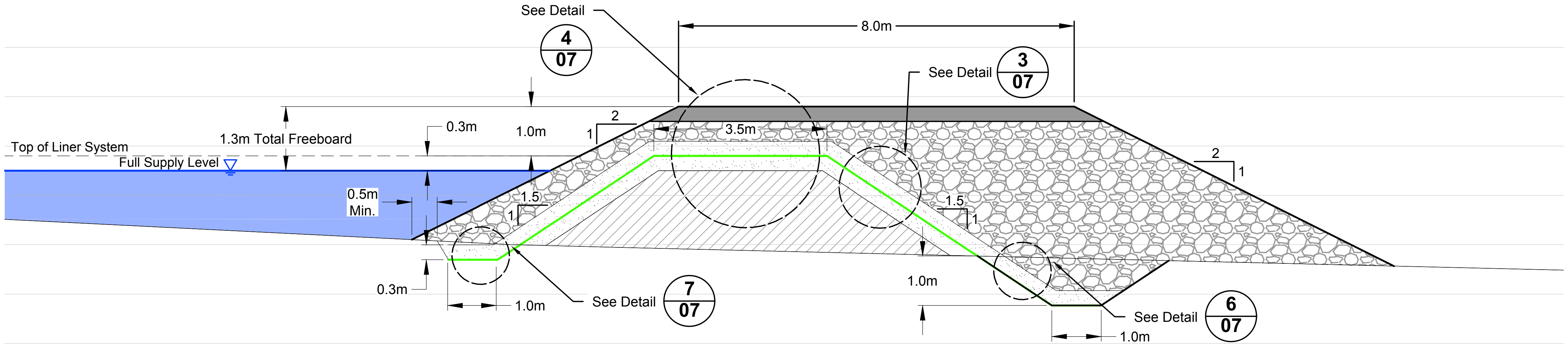
DRAWING TITLE

Centerline Profile Contact Water Berms

DRAWING NO.	SHEET	REVISION NO.
BTMA-05	5 OF 7	A

LEGEND

- Surfacing Material
- Bedding Material
- Run of Quarry Material
- Crushed Rock Protection Layer
- Transition Material
- Tailings
- HDPE Liner
- Geotextile
- Liner System



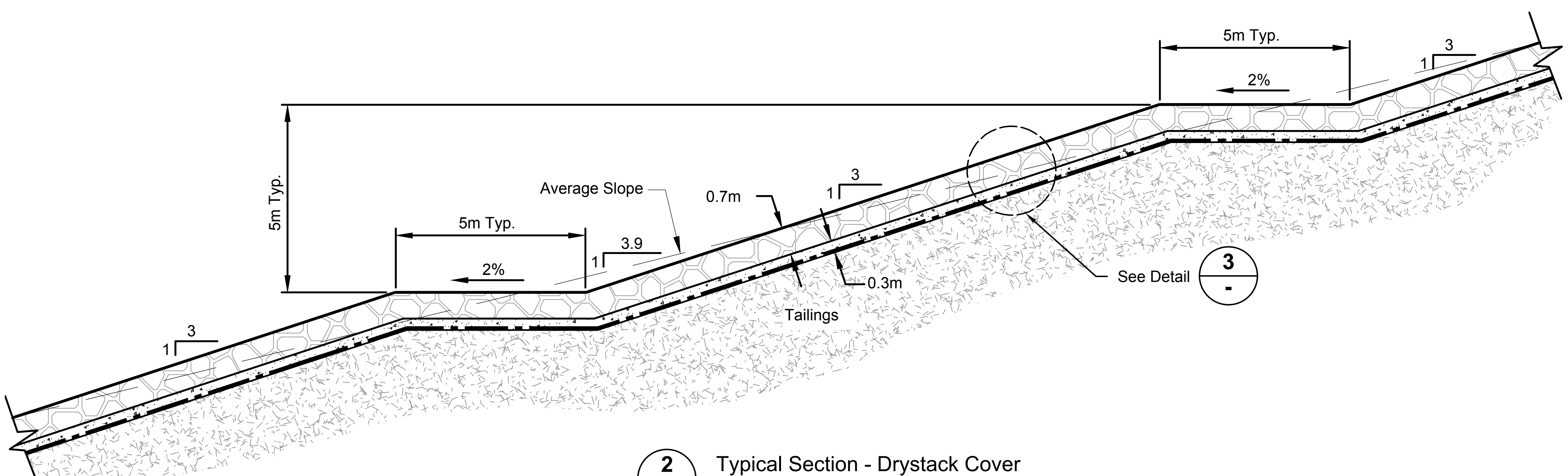
1 / 03 Typical Section - Contact Water Berm

0m 0.5 1.0 1.5 2.0 2.5 3.0

1 : 75 (Vertical Scale)

0m 1 2 3 4 5 6

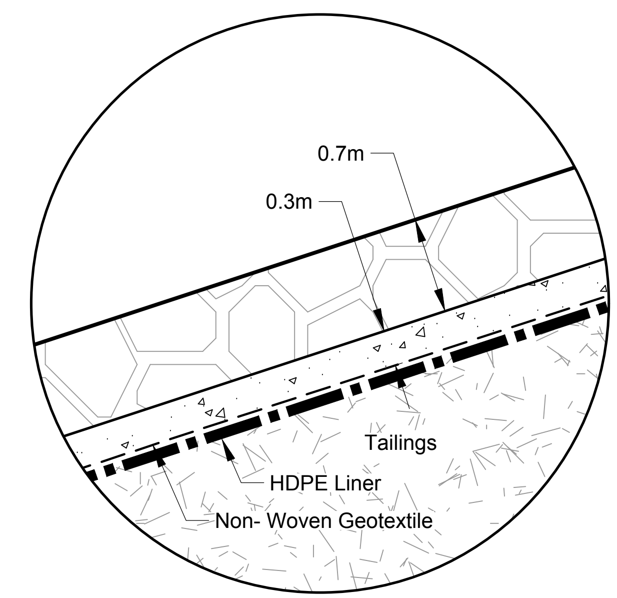
1 : 150 (Horizontal Scale)



2 / 04 Typical Section - Drystack Cover

0m 1 2 3 4 5 6

1 : 150



3 / - Typical Cover System

NOT TO SCALE

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FILE NAME: 1CT022.013-135_3-6.dwg

TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-135

Boston TMA Preliminary Design

DRAWING TITLE

Water Berm and Drystack Typical Details

DRAWING NO. BTMA-06 | SHEET 6 OF 7 | REVISION NO. A

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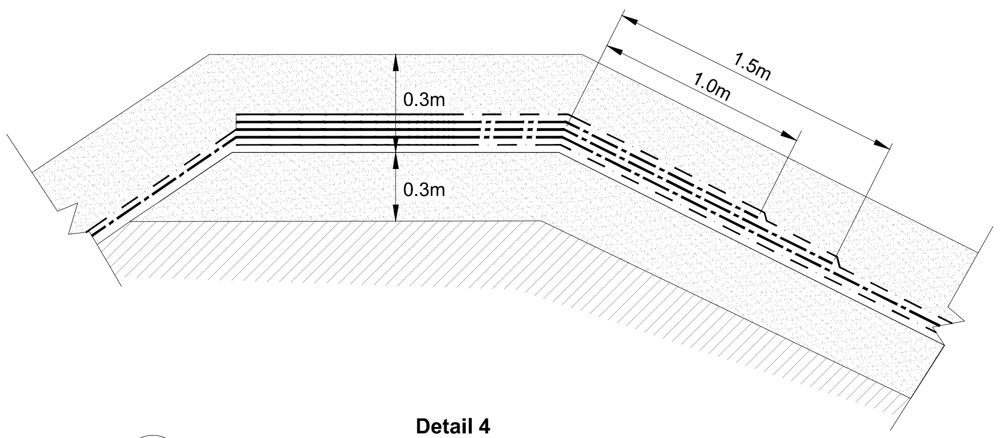
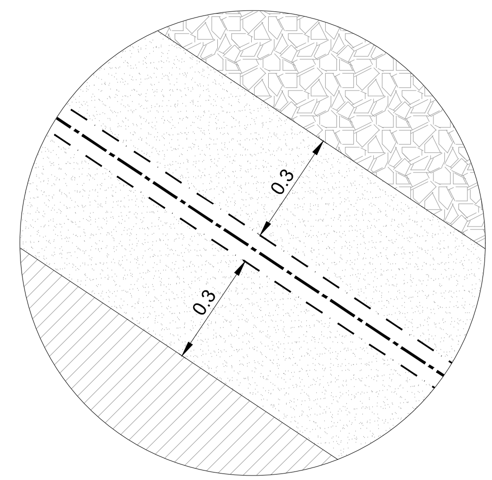
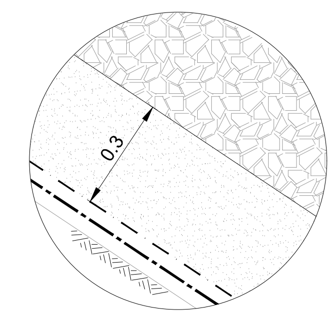


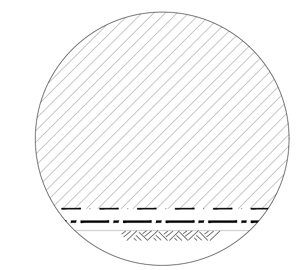
4 / 06 — Detail 4
Typical Liner System Under Driving Surface
NTS



5 / 06 — Detail 5
Typical Liner System in Berm Core
NTS



6 / 06 — Detail 6
Typical Liner System in Keytrench
NTS



7 / 06 — Detail 7
Typical Liner System in Liner Anchor Area
NTS

Materials List and Quantities for Closure Cover

Item	Quantity / Area / Volume		Description
Liner Bedding Material (m³)	Closure Cover	60,850	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses and/or tundra embedment
HDPE Liner (m²)	Closure Cover	202,800	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses
Geotextile (m²)	Closure Cover	202,800	
ROQ Fill (m³)	Closure Cover	142,000	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses and/or tundra embedment

Materials List and Quantities for Containment Berms

Item	Quantity / Area / Volume		Description
Liner Bedding Material (m³)	Containment Berms	16,495	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses and/or tundra embedment
HDPE Liner (m²)	Containment Berms	32,700	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses
Geotextile (m²)	Containment Berms	60,095	
ROQ Fill (m³)	Containment Berms	71,750	Approximate In-Place Neat-Line Volume 3D Volume based on Civil3D Surfaces - No allowance has been made for losses and/or tundra embedment
Transition Fill (m³)	Containment Berms	33,660	
Key Trench Excavation (m³)	Key Trench	5,580	

LEGEND

- Surfacing Material
- Bedding Material
- Run of Quarry Material
- Transition Material
- 12oz. Non-woven Geotextile
- Textured HDPE Liner

NOTES

1. Dimensions in metres unless noted otherwise.

DRAWING NO.	DRAWING TITLE	DRAWING NO.	DRAWING TITLE
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REVISIONS

PROFESSIONAL ENGINEERS STAMP

srk consulting

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TMAC RESOURCES

HOPE BAY PROJECT

SRK JOB NO.: 1CT022.013-135

Boston TMA Preliminary Design

DRAWING TITLE:

Details and Material Quantities

DRAWING NO. BTMA-07 | SHEET 7 of 7 | REVISION NO. A

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Appendix B – Hope Bay Project: Boston TMA Detailed Cover Design

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Memo

To:	John Roberts, PEng, Vice-President Environment 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:	Hope Bay Project: Boston TMA Detailed 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 consists of two phases: Phase 1 (Doris deposit) and Phase 2 (Madrid and Boston deposits).

Ore processing at Boston includes milling and processing to produce gold doré. Flotation tailings will be dewatered using a filter press and trucked to the Tailings Management Area (TMA) and placed in thin compacted lifts (SRK 2017a). Cyanide leach tailings will be detoxified and used as underground backfill.

The flotation tailings placed within the TMA will not be acid generating; however, the tailings will have the potential for neutral metal leaching (SRK 2017a), which may result in runoff water exceeding discharge guidelines if mitigation measures are not applied (SRK 2017d).

2 Background

2.1 Boston TMA Closure Objectives

The closure objectives for the TMA have been extracted from the Boston Conceptual Closure and Reclamation Plan (SRK 2017b) and are listed below:

- Ensure long-term physical stability of tailings;
- Prevent direct contact of the tailings by humans and wildlife;
- Ensure chemical stability by minimizing water ingress and release of neutral metal leaching to the receiving environment; and
- Restore natural drainage, to prevent the need for long term water management.

2.2 Strategies for Meeting TMA Closure Objectives

To meet the stated TMA closure objectives the following strategies could be adopted:

- Relocation of tailings;
- Collection and treatment of drainage water not meeting discharge criteria; or
- Cover the TMA with an infiltration reducing cover.

Relocation and perpetual collection and treatment are not viable strategies and therefore covering the TMA is the only reasonable strategy. Because the tailings in not acid generating, there is no benefit in preventing oxidation, and therefore any cover should be focussed towards reducing infiltration.

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 3.1 summarize the cover types considered for the Boston TMA.

Table 3.1: Overview of Cover Types

Cover Type	General Description	Best Suited to	Boston TMA 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	Although oxidation control is not required, infiltration control is required. This cover is therefore not suitable
Water	Permanent water cover to prevent oxidation	Sites where net positive climatic water balance exist and material is strongly acid generating	Above ground TMA cannot be practically flooded 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
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	Viable means of constructing a high-quality cost effective low permeability cover
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
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

The primary function of the Boston TMA cover is to limit infiltration, and therefore an isolation cover is not suitable. Similarly, a water cover is not necessary, not would be it be practical. Since there are no suitable low permeability natural soils, or natural soils to construct a capillary break, neither Natural Barrier, nor Capillary Break covers will be suitable for the Boston TMA.
Finally, although the site is well suited towards use of a frozen cover, there are no suitable natural cover materials, and a coarse quarry or waste rock cover would require a cover thickness of about 5 m. Since there is a deficit in mine backfill material necessitating all waste rock supplemented by quarry rock to be used for mine backfill, construction of a frozen cover is not practical.

As a result, the only viable cover type for the Boston TMA is to use a synthetic liner.
The geosynthetic materials considered technically feasible for the Boston TMA are polyethylene geomembranes (HDPE and LLDPE), bituminous geomembranes (BGM), and geosynthetic clay liners (GCL). Final selection of the liner type will be completed at the detailed design stage and will consider longevity and constructability.

Polyethylene geomembranes comprise of high-density polyethylene (HDPE) or linear low-density polyethylene (LLDPE), both of which are widely used in industry, including arctic applications due to their low cost and long-term performance. Benefits of HDPE geomembranes include high resistance to puncturing while offering good resistance to chemical weathering. However, HDPE can be challenging to install in cold weather due to its rigidity. LLDPE geomembranes are like HDPE, but is less rigid and thus is less prone to cold weather installation challenges. Both HDPE and LLDPE installation require a protective bedding layer (above and below the liner) consisting of either fines or a geotextile, and both liners require specialized seaming of joins under hoarding if temperatures fall below -25°C (Layfield 2011).

BGMs are composite liners consisting of a non-woven geotextile impregnated with bitumen, a fibreglass structural reinforcing layer, and an additional layer of oxidized bitumen. BGMs are heavier than HDPE and provide greater resistance to puncturing, tearing, and stress cracking. BGMs are easy to seam under any temperature, and has slightly less restrictive bedding layer requirements than polyethylene liners.

GCLs are also composite liners consisting of a layer of bentonite encapsulated between two geotextiles or bonded to a geomembrane. To function, a GCL must be permanently confined, which typically requires at least 1 m of cover material over the GCL. Ideally GCLs must also be permanently hydrated to get the maximum benefit of the swelling clay. If the GCL is not allowed to hydrate prior to installation, construction under arctic conditions is simple requiring no specialized equipment.

GCLs have stated lifespans of about 30 years (Thies *et al*. 2002, MEND 2009), with failure related to the geotextile if it is exposed to the environment (i.e. ultraviolet light). The clay layer can effectively remain functional in perpetuity and research has shown that its effectiveness as an infiltration layer is not impacted by freeze-thaw cycling (Podgorney and Bennett 2006).

HDPE and LLDPE geomembranes have predicted lifespans exceeding 600 years (Rowe 2005, Koerner 2005), while BGMs are expected to have lifespans approaching 1,000 years (Nilex 2017).

4 Preliminary Cover Design

4.1 Cover Profile

The primary function of the Boston TMA cover is to limit infiltration. To that end the preliminary cover design is assumed to consist of a textured HDPE geomembrane placed directly on the dry-stack tailings surface, overlain by a non-woven geotextile, a 0.3 m thick gravel bedding layer, followed by a 0.7 m thick Run of Quarry (ROQ) layer. The geotextile and bedding layers serve to protect the liner against damage form the overlying ROQ material, and the ROQ material serves to protect the liner from ultraviolet exposure and ensures long-term erosional (i.e. physical) stability. Both the bedding and ROQ material will be geochemically suitable quarry rock or waste rock.

4.2 Cover Construction

Because the dry-stack tailings consist of a compacted material, it is trafficable at any time. Therefore, the cover can be constructed at any time of the year. Liner placement during the winter season will be more time consuming than summer construction due to the need to hoard all seams, however this does not mean that winter construction cannot be considered.

4.3 Infiltration Analysis

Seepage through the geomembrane will be negligible, and will only occur because of manufacturing and installation defects. (SRK 2016e) provides an upper bound of seepage due to these defects. Defects that were analyzed were 0.02 m diameter circular pinholes from manufacturing defects, and punctures through installation represented by square defects with side lengths of 0.01 m. The corresponding seepage through the liner was subsequently estimated as 0.64 m^3/d for the entire facility. In practice, however, this seepage is only present for approximately 60 days per year, due to freezing and thawing of the upper rock layer.

4.4 Stability Analysis

An infinite slope stability analysis was completed to confirm stability of the geomembrane liner on the TMA. The analysis confirmed that at the design grades in question the liner will remain stable (SRK 2016a).

5 Cover Failure Modes and Effects Analysis

A preliminary failure modes and effects analysis (FMEA) was completed for the preliminary Boston TMA 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 poor execution because of inadequate quality control and quality assurance (QC/QA). This would result in increased defects which in turn would translate into increased infiltration which subsequently could result in discharge of water to the environment that does not meet discharge criteria. Appropriate mitigation would be to implement a suitably rigorous QC/QA program.

6 Monitoring and Maintenance

Post construction monitoring and maintenance will be required for the Boston TMA cover. This will include geotechnical inspections by a qualified geotechnical engineer. No in-situ instrumentation is recommended or required. Water quality form 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 TMA cover mitigation may be required.

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Appendix A – Cover FMEA for the Boston TMA

FMEA Worksheet - Hope Bay Boston TMA 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 construction methods	Imperfections in geomembrane welds leading to neutral metal leaching	M	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	L	L	H	Mo-H	Weld seams will be inspected by qualified persons to ensure welds meet specifications.
	Geomembrane anchoring not completed to specification and cover slides, leading to neutral metal leaching	L	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	Mo	L	L	H	Mo	Anchoring of geomembrane will be inspected by qualified persons.
Poor execution of construction QA/QC program and/or inexperienced personnel supervising construction	Defects in geomembrane from manufacturer and imperfections in installation go unnoticed, leading to neutral metal leaching	M	Mi	Mo	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	L	L	H	Mo-H	Robus QA/QC program detailed in technical specifications.
	Damage to integrity of geomembrane during installation leads to neutral metal leaching	M	Mi	Mo	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	Mo	Mo-H	L	L	H	Mo-H	Geomembrane will be installed by qualified personnel and checked for defects and imperfections prior to installation.
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	L	L	L	L	L	L	L	Mo	Mo	L	L	L	L	H	Mo	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 fill materials not meeting design specs, leading to constructcion delays and/or higher capex costs	L	L	L	L	L	L	L	Mi	L	L	L	L	L	H	L	Geotechnical testing of borrow materials will be conducted to ensure borrow materials meet design specifications prior to commencement of construction.
Excessive rill/gully formation due to higher than expected runoff	Formation of gullies in cover system material exposes underlying geomembrane, leading to degradation of geomembrane material	NL	Mi	L	Mi	L	Mo	L	L	L	Mi	L	L	L	H	L	Coarse textured ROQ material is not prone to formation of gullies

FMEA Worksheet - Hope Bay Boston TMA 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
Freeze / thaw cycling of the cover system	Differential heaving leads to tearing of the geomembrane	NL	Mi	L	Mi	L	Mo	L	L	L	Mo	L	L	L	H	L	Not likelhy that structural changes resulting from feezee/thaw cylcling will lead to significant differential heaving. Geomebrane flexibility considered at design analysis.
	Differential frost heave results in standing water on the reclaimed landforms	M	L	L	Mi	Mo	L	L	L	L	L	L	L	L	M	Mo	Although possible, it is not likely that structural changes resulting from frost heaving will lead to standing water. Mitigation methods are monitoring and maintenance and surface water management.
Differential settlement of fill materials beyond tolerance levels	Differential settlement results in standing water on the reclaimed landforms	M	L	L	Mi	Mo	L	L	L	L	L	L	L	L	M	Mo	Geomembrane flexibility considered as design criteria. Compaction as part of tailings will minimize settlement.
	Differential settlement results in tearing of geomembrane	M	L	L	Mi	Mo	L	L	L	L	L	L	L	L	M	Mo	Geomembrane flexibility considered as design criteria. Compaction as part of tailings will minimize settlement.
Burrowing animals create substantial holes / macropores in the cover system profile	Formation of holes/voids in the cover leads to neutral metal leaching	L	Mi	L	Mi	L	Mi	L	L	L	Mi	L	L	L	M	L	Due to geographical area and climate, the likelihood of burrowing animals is considered low. Use of well-graded ROQ will make it difficult for animals to dig through.
Substantial damage to integrity of cover system due to anthropogenic activities	Formation of voids and/or tears leads to neutral metal leaching	NL	Mi	L	Mi	L	Mo	L	L	L	M	Mo	L	L	M	Mo	The likelihood of people digging around on the cover and puncturing the geomembrane over the assessment is considered low due to the isolated area and institutional controls like signage
Climate change leading to wetter conditions than anticipated in design	Increased net percolation into tailings mass	NL	Mi	L	Mi	L	Mo	L	L	L	Mo	L	L	L	L	L	This cover has low sensitivity to the volume of precipitation. The head on the cover is not likely to be exceeded.

Risk Matrix

Likelihood	Consequence Severity: *Low (L)*	*Minor (Mi)*	*Moderate (Mo)*	*Major (M)*	*Critical (C)*
Expected (E)	Moderate	Moderately High	High	Critical	Critical
High (H)	Moderate	Moderate	Moderately High	High	Critical
Moderate (M)	Low	Moderate	Moderately High	High	High
Low (L)	Low	Low	Moderate	Moderately High	Moderately High
Not Likely (NL)	Low	Low	Low	Moderate	Moderately High

Intolerable Region

ALARP Region

Broadly Acceptable Region

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

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.

Appendix C – Hope Bay Project: Boston Tailings Management Area Stability Analysis

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Director Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Cameron Hore, CPEng, PEng	**Project No:**	1CT022.013
Reviewed:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project — Boston Tailings Management Area Stability Analysis		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-2F, 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-IR36	Figure 1 3.1 3.2	Upper 1 m of overburden and all tailings are unfrozen. Figure showing assumed frozen and unfrozen materials for stability analysis. Analysis method updated to eliminate the unnecessary analysis of excess pore pressures in frozen material.
INAC-IR37	3.3 Fig. 3 to 5, 4.2	Additional explanation on the consideration of inter-bedded pure ice. Material properties shown in figures Cohesion vanishes because of low strain rates
INAC-IR39	4.2	Inclusion of creep analysis
KIA-IR165	4.2	Inclusion of creep analysis
INAC-TRC11	4.2	Inclusion of creep analysis

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.

Tailings deposition at Boston will be in the form of dewatered (i.e. filtered) tailings placed in a compacted dry-stack. This tailings management area (TMA) is located approximately 1.2 km east of the proposed Boston camp and processing facilities, and is accessed via the Boston-Madrid all-weather road. At closure, the dry-stack will be covered with a geosynthetic low permeability infiltration reducing cover.

1.2 Objectives

This memo documents the methods, assumptions, and results of the stability analyses completed for the Boston TMA. The analysis considers overall stability along a critical cross-section of the dry-stack, as well as stability of the proposed geosynthetic closure cover.

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). Although the Boston TMA contains tailings, the dry-stack is not a dam, but more closely represents a waste rock dump. The most notable design guidelines for waste rock dumps are those published by the British Columbia Mine Waste Rock Pile Research Committee (BCMWRPRC, 1991).

Table 1 summarizes the recommended minimum design FOSs in accordance with the CDA (2014), while Table 2 summarizes the recommended minimum design FOSs in accordance with BCMWRPRC (1991). These values are used as guidelines for the stability of the dry stack. A long-term FOS against the cover material sliding on the geomembrane of 1.5 was also adopted for the cover analysis.

Table 1: Minimum Factors of Safety Used for Slope Stability Analysis (CDA, 2014)

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

Table 2: Minimum Factors of Safety Used for Slope Stability Analysis (BCMWRPRC, 1991)

Stability Condition	Factor of Safety	
	Case A	Case B
Stability of Waste Rock Pile Surface • Short term (during construction) • Long term (reclamation – abandonment)	 1.0 1.2	 1.0 1.1
Overall Stability (Deep Seated Stability) • Short term (static) • Long term (static) • Pseudo-static	 1.3 – 1.5 1.5 1.1 – 1.3	 1.1 – 1.3 1.3 1.0
Case A: • Low level of confidence in critical analysis parameters • Possibly unconservative interpretation of conditions, assumptions • Severe consequences of failure • Simplified stability analysis method (charts, simplified method of slices) • Stability analysis method poorly simulates physical conditions • Poor understanding of potential failure mechanism(s)		
Case B: • High level of confidence in critical analysis parameters • Conservative interpretation of conditions, assumptions • Minimal consequences of failure • Rigorous stability analysis method • Stability analysis method simulates physical conditions well • High level of confidence in critical failure mechanism(s)		

Recognizing the fact that the recommended minimum design FOS for a tailings dry-stack is not truly captured by either CDA (2014) or BCMWRPRC (1991), the most conservative design values were used; i.e. 1.3 for short-term static stability, 1.5 for long-term static stability, and of 1.1 for pseudo-static stability.

2.2 Seismic Design Parameters

The CDA (2014) provides recommended minimum seismic design criteria based on the hazard classification assigned to the structure. Assuming a hazard classification of Significant, the CDA (2014) specifies the design earthquake with AEP of between 1/100 and 1/1,000 years for the construction and operations stage. For long-term scenarios, i.e. post-closure, the design seismic event must be increased to 1/2,475 year event.

The BCMWRPRC (1991) recommends using the seismic coefficient prediction (i.e. 10% probability of exceedance in 50 years) outlined in Weichert and Rogers (1987). In this report, the recommended Peak Ground Acceleration (PGA) with this return period is 0.04 g.

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, 2016a). 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 criterion was thought to be appropriately conservative. The horizontal seismic coefficients for the Boston dry-stack facility was determined to be 0.018 g, resulting from a 1/2,475 year return period earthquake.

3 Stability Analysis

3.1 Conceptual Model

Figure 1 illustrates the design cross-section of the dry-stack and associated model domain used in the stability analysis. A single critical cross-section of the facility was assessed for overall stability. Since the facility foundation is not expected to show significant variability, the critical section was deemed to be where the dry-stack would be at its maximum overall height of 26 m. The dry-stack will be constructed in lifts, each approximately 5 m in height with inter-bench slope angles of 3H:1V. Inter-bench ramps will be constructed to allow for an overall regraded slope of 3.9H:1V at closure.

The model includes foundation soil layers (bedrock and overburden), a phreatic surface, filtered tailings, a geosynthetic liner, and a run-of-quarry (ROQ) protective shell. The foundation profile was assumed to consist of 7 m of permafrost overburden soils (marine silt and clay) overlying competent bedrock. The upper 1 m of the overburden profile, immediately beneath the first bench, was assumed to be thawed and the entire dry-stack was conservatively assumed to be thawed.

The closure cover consists of the textured High-Density Polyethylene (HDPE) liner placed directly on the tailings surface and a protective heavy duty non-woven geotextile, covered with 0.3 m of crushed and screened overliner (bedding), followed by 0.7 m of ROQ rock.

3.2 Methods of Analysis

Stability of the critical section was assessed using two methods: a) the strength reduction method as applied in the finite element code PLAXIS (Plaxis, 2016), b) the limit equilibrium method using Slope/W (Geoslope, 2012), and c) the infinite slope analysis (Koerner 2005). The analyses with the first two methods were carried out using a Mohr-Coulomb failure criterion for all the materials including the ice-rich frozen soils. The third method relied on a forces equilibrium calculation.

3.2.1 PLAXIS Analysis

The 2D (plain strain) stability analysis with PLAXIS was completed with 15-node elements. The generated finite element model shown in Figure 1 consisted of 2,210 soil elements, 18,900 nodes, and an average element size of 2.8 m. The stability was assessed for five construction steps represented by the individual lifts of the dry-stack facility. Step 1 construction considered a 6 m lift. The remaining steps had 5 m lifts.

The analysis with PLAXIS was completed for both static and pseudo-static conditions with a predefined phreatic surface. Below is a summary of the conditions considered in the analysis.

Static Analysis

The static analysis considered the following conditions:

- Long-term condition using drained shear strength properties of the materials. Stiffness and strength were defined in terms of effective stress properties.
- Short-term condition using undrained shear strength properties of the materials. The stiffness of the materials was defined in terms of effective stress properties; the strength was defined as the undrained shear strength.

Pseudo-Static Analysis

The pseudo-static analysis was completed under partially frozen (see Figure 1) undrained foundation conditions considering a horizontal seismic coefficient of 0.018 g (Section 2.2).

A Post-Earthquake Analysis was not completed. The mainly frozen foundation conditions (Figure 1), coupled with the low seismicity of the area suggest that large deformations of the dry-stack and/or the foundation are unlikely to occur following the design earthquake. Furthermore, the material properties of the foundation and tailings material, and the water content of the tailings material if thawed is such that tailings and/or foundation liquefaction is unlikely.

3.2.2 Slope/W Analysis

The results using the Finite Element (FE) method with PLAXIS were compared with results obtained with SLOPE/W using the Limit Equilibrium (LE) method. The analysis with Slope/W was performed for the first and fifth construction stages of the cross-section shown in Figure 1, and the groundwater level used in PLAXIS. The constitutive parameters for the analysis are included in Table 3.

SLOPE/W was also used to assess the long-term creep stability of the dry-stack considering reduced shear strength parameters of the frozen marine silt and clay.

3.2.3 Infinite Slope Analysis

The stability of the closure cover was assessed as an infinite slope. Table 3 includes properties of the textured HDPE used in the analysis. The interface friction angles between the various layers of the cover system were obtained from the pertinent literature (Koerner 2005) and are presented in Table 4.

The analysis consists of computing the factor of safety against failure based on the equilibrium of forces along the plane with the lowest interface friction angle, or 26° in our case representing the interface between the geotextile and the overlying protective crushed rock. For the purposes of this analysis a homogeneous rock cover 1 m thick was assumed.

3.3 Material Properties

Sub-surface investigations within the footprint of the proposed TMA has not been carried out. Material properties for the analysis was therefore based on the site wide geotechnical design properties (SRK, 2017) which have been used for previous designs on site and been through numerous review processes. Table 3 summarizes the main properties used in the method of analysis.

Table 3: Material Properties

Property	Symbol	Unit	ROQ (Thawed)	Marine Silt and Clay (Frozen)	Marine Silt and Clay (Thawed)	Filtered Tailings	HDPE Liner
Unit Weight	γ	kN/m^3	20.0	17.0	17.0	17.5	9.2
Young's Modulus at Reference Level	E'	MPa	175	150	5	100	800
Poisson's ratio	V	-	0.3	0.3	0.3	0.3	0.3
Cohesion	c'	kPa	0	112	0	0	15
Friction Angle	φ'	º	40	26	30	40	0
Dilatancy Angle	Ψ	º	0	0	0	0	0
Undrained Shear Strength at Reference Level	S_u	kPa	-	-	13	-	-

There is the potential for inter-bedded pure ice within the foundation material. The cohesion of pure ice at the typical temperature of permafrost at Hope Bay, in the range of -6 to -8°C, was estimated to exceed 1,500 kPa using the equations from Nater et al. (2008). This value far exceeds the strength of the frozen soils used in the stability model, and therefore it is unlikely to represent the path of least resistance.

Table 4: Interface Friction Angles

	Bedding – fine sand (tailings)	Geotextile – Nonwoven needle-punched	Protective cover – crushed rock
HDPE - Textured	26°	32°	--
Geotextile - Nonwoven needle-punched	--	--	26°

All values from Koerner (2005).

4 Results

4.1 Stability of the Dry-stack

The results from the FE and LE analysis are summarized in Table 5, and complete details are presented in Attachment 1.

Table 5: Slope Stability Analysis Results

Analysis Method	Construction Stage	Short-term FOS (Undrained Loading Conditions)	Long-term FOS (Drained Loading Conditions)	Pseudo-Static FOS
Minimum Required FOS		**1.3**	**1.5**	**1.1**
PLAXIS (FE)	1st Stage (Height 6 m)	1.4	1.8	1.3
	2nd Stage (Height 5 m)	1.4	1.9	1.3
	3rd Stage (Height 5 m)	1.4	1.9	1.3
	4th Stage (Height 5 m)	1.4	1.9	1.3
	5th Stage (Height 5 m)	1.4	1.9	1.3
SLOPE/W (LE)	1st Stage (Height 6 m)	1.4	2.5	1.3
	5th Stage (Height 5m)	1.4		1.3

The SLOPE/W analysis yields similar or greater FOS compared to the PLAXIS analysis, confirming that the results are consistent. In all cases, the calculated FOS met the stability criteria (1.3 for short-term static stability, 1.5 for long-term static stability, and of 1.1 for pseudo-static stability).

Figures 2 and 3 show the critical failure surfaces determined using the PLAXIS and SLOPE/W methods respectively. These analyses clearly show that in all cases the calculated failure surface is near the toe of the facility, within the first construction stage. Under no scenarios is large deep-seated foundation failures induced.

4.2 Long-Term Creep Effects on the Stability of the Dry-Stack

Additional analyses with SLOPE/W were completed to assess the long-term creep effects on the stability of the dry-stack. The procedure consisted in back-calculating the friction angle of the frozen marine silt and clay required to meet the stability criterion for the long-term condition (i.e., FOS = 1.5). In the absence of specific creep parameters of the frozen soils, the back-analysis considered that the strain rate dependent cohesion vanishes due to the long-term creep, and that the friction angle remains independent of the strain rate during the long-term creep deformation process (Andersland and Al-Nouri, 1970 and Ladanyi, 1972); i.e., the strength in the frozen marine silt and clay tends to a long-term strength when the strain rate tends to zero.

The back-analyses were performed for two configurations of the dry stack: a) at the first construction stage, and b) at the end of the construction. The latter considered a deep-seated failure mechanism. The analysis also considered 1 m of non-frozen foundation soils beneath the slope of the first bench with the shear parameters c' = 0 kPa and φ' = 30°.The results are summarized in Table 6. Figures 4 and 5 presents the details of the failures surfaces from the back-analysis.

Table 6: Slope Stability Back-Analysis Result

Model	c' = 0 kPa; Required φ'
1st Stage (Height 6 m)	20°
Final Configuration – Deep Seated Failure Surface	15°

According to Table 6, the frozen marine silt and clay requires a maximum friction angle of 20° to meet the long-term stability criterions (FOS = 1.5) when the cohesion of the frozen soil vanishes due to very low creep strain rates. Since the ice-content dependent friction angle of the frozen marine silt and clay was estimated in 26° (Table 3), the designed dry-stack meets the long-term stability criterion considering creep effects on the shear strength parameters. In other words, with increasing volumetric ice content, the friction angle decreases (Nater *et al*. 2008). A friction angle of 20° corresponds to a volumetric ice content of 0.76, which is much higher than the observed site-wide volumetric ice content of 0.45 (SRK 2016).

4.3 Stability of the Closure Cover

In considering the stability of the cover system, a conservative analysis was completed using the limit equilibrium method on an infinite slope. Based on the slope grade of 3H:1V (18°) and the cover thickness of 1 m, the factor of safety against the cover material sliding off the geomembrane was found to be 1.5.

5 Conclusion

Stability analyses of the dry-stack were completed using finite element and limit equilibrium codes. The calculated FOS met the stability criteria (1.3 for short-term static stability, 1.5 for long-term static stability, and of 1.1 for pseudo-static stability) for all the cases analyzed (construction stages and at the end of the construction).

Additional limit equilibrium analyses were completed to assess the long-term stability of the dry-stack considering the effect of long-term creep strain rates on the shear strength parameters of the frozen marine silt and clay. If the cohesion of the frozen soil vanishes because of low creep strain rates, it will require a maximum friction angle of 20° to meet the long-term stability criteria. Since the friction angle of the frozen marine silt and clay is 26° the designed dry-stack met the long-term stability criteria for the case of zero cohesion due to long-term creep.

The cover system consisting of a textured HDPE geomembrane overlain by a nonwoven geotextile and protective granular fill of crushed rock and ROQ is stable on a 3H:1V slope, with a calculated FOS of 1.5.

6 References

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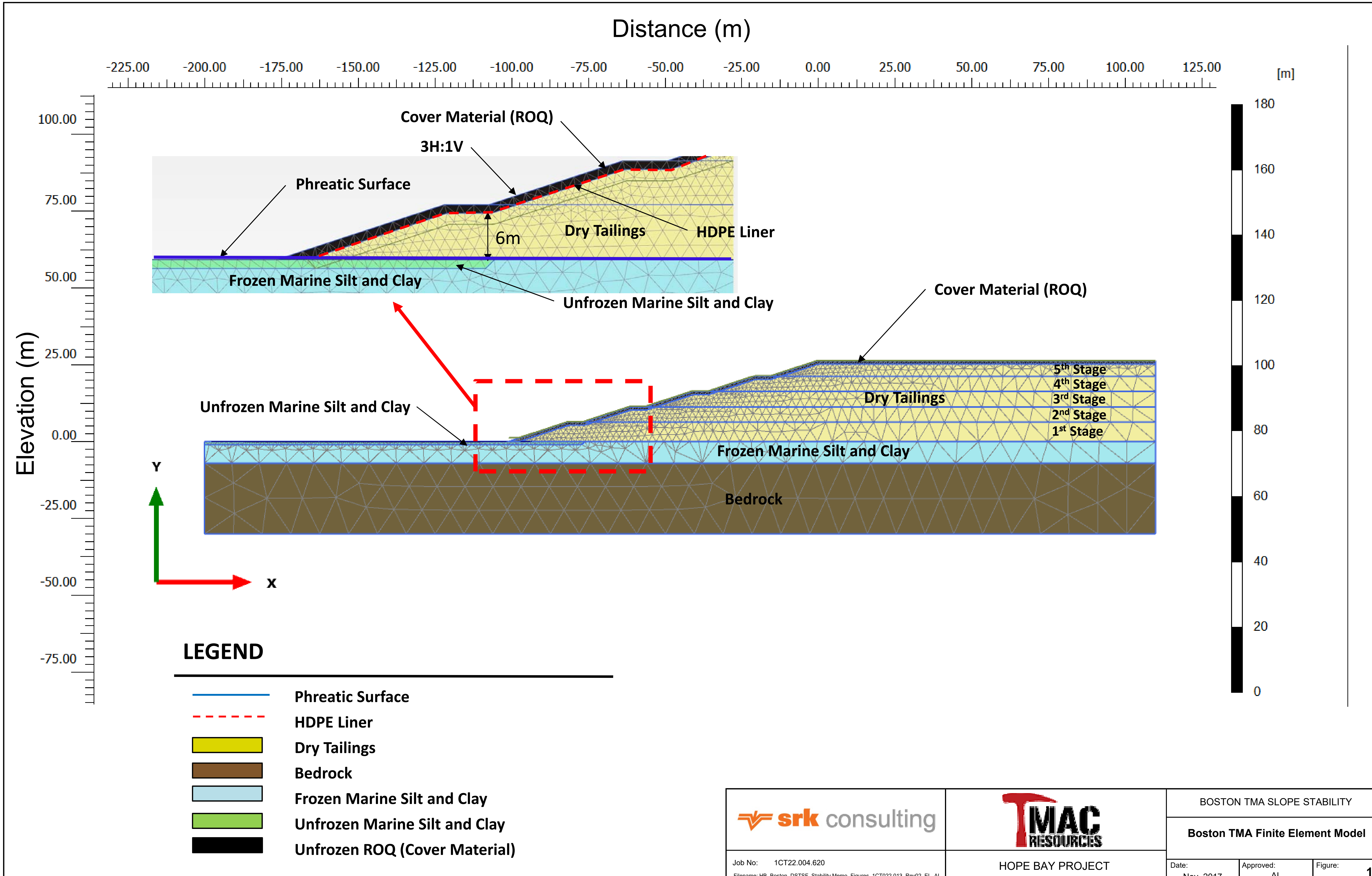
Figures



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-200.00
-175.00
-150.00
-125.00
-100.00
-75.00
-50.00
-25.00
0.00
25.00
50.00
75.00
100.00
125.00
[m]
Elevation (m)
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75.00
50.00
25.00
0.00
-25.00
-50.00
-75.00
180
160
140
120
100
80
60
40
20
0
Cover Material (ROQ)
3H:1V
Phreatic Surface
6m
Dry Tailings
HDPE Liner
Frozen Marine Silt and Clay
Unfrozen Marine Silt and Clay
Cover Material (ROQ)
5th Stage
4th Stage
3rd Stage
2nd Stage
1st Stage
Dry Tailings
Unfrozen Marine Silt and Clay
Frozen Marine Silt and Clay
Bedrock
Y
X
LEGEND
Phreatic Surface
HDPE Liner
Dry Tailings
Bedrock
Frozen Marine Silt and Clay
Unfrozen Marine Silt and Clay
Unfrozen ROQ (Cover Material)
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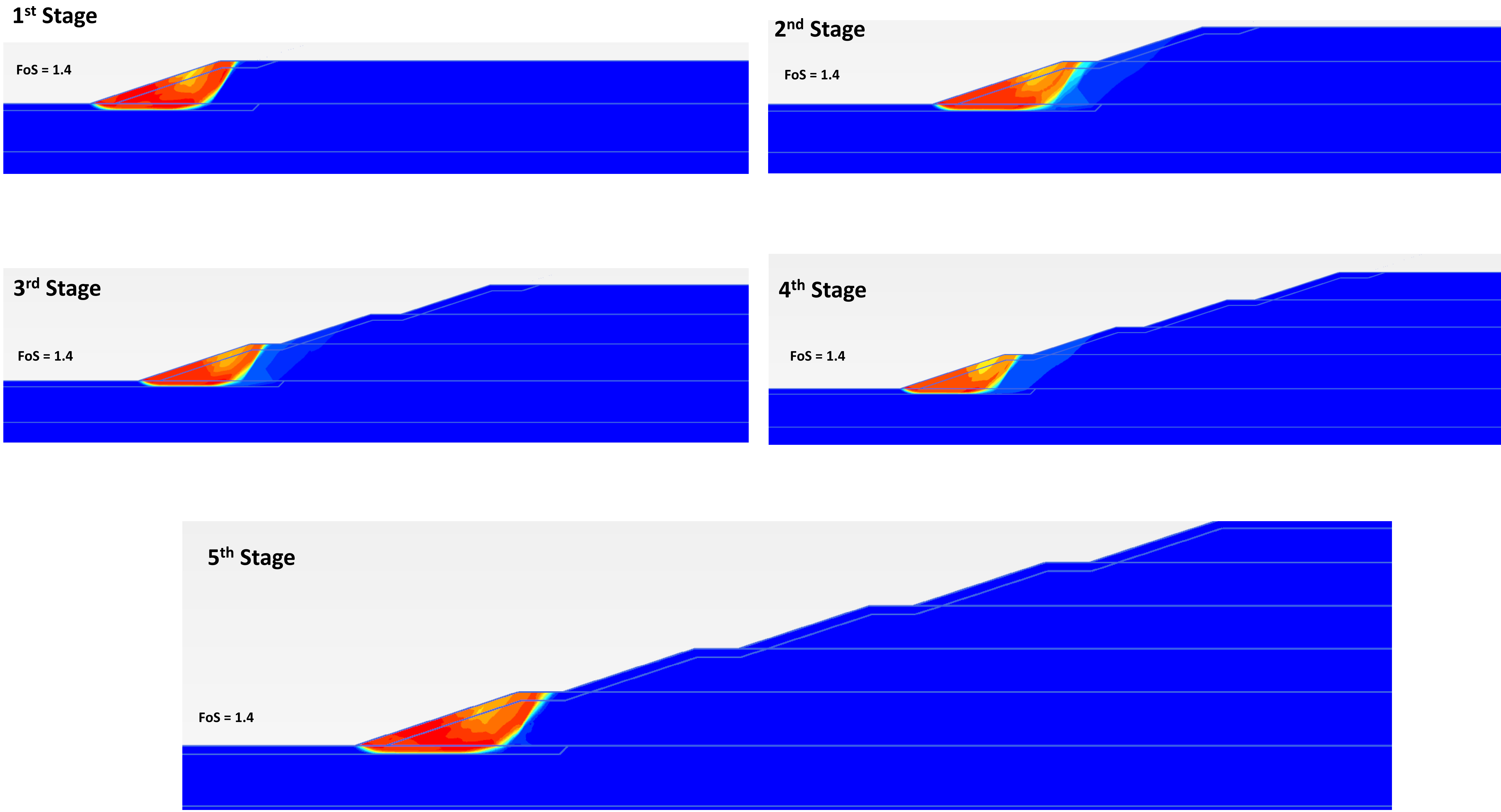
BOSTON TMA SLOPE STABILITY
Boston TMA Finite Element Model
Job No: 1CT22.004.620
Filename: HB_Boston_DSTSF_Stability Memo_Figures_1CT022.013_Rev02_EL_AL
HOPE BAY PROJECT
Date: Nov. 2017
Approved: AL
Figure: 1



srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
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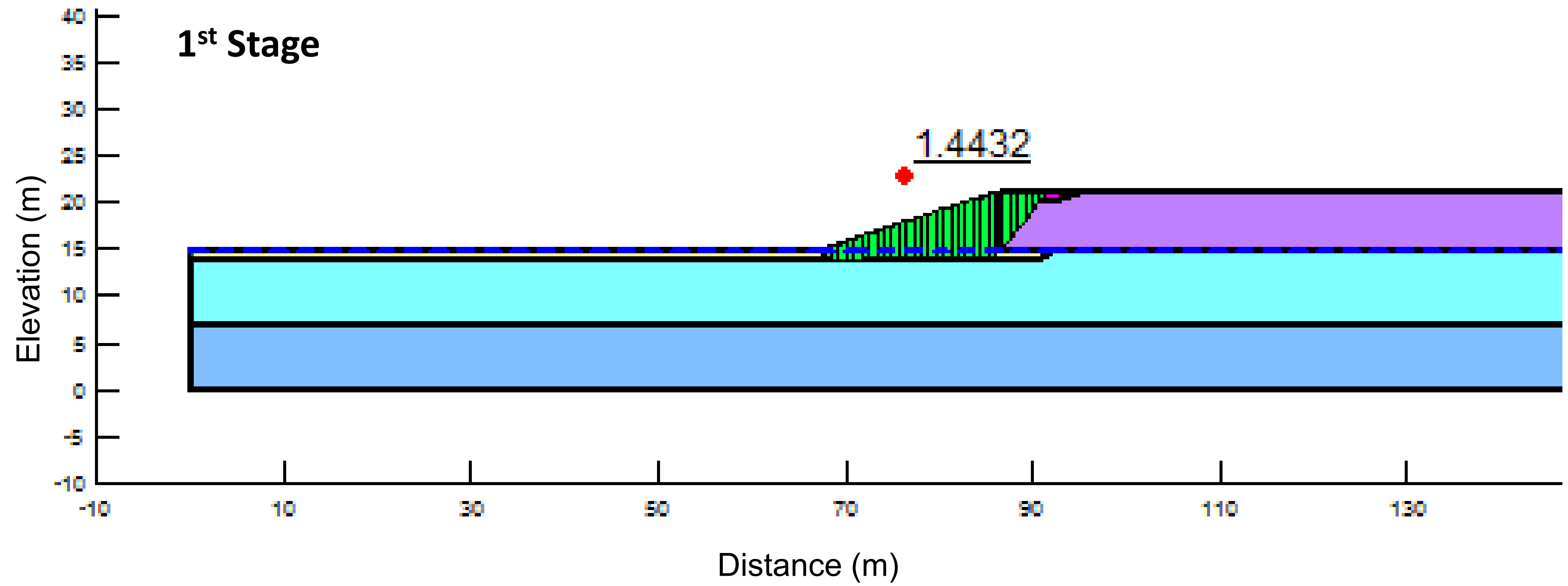
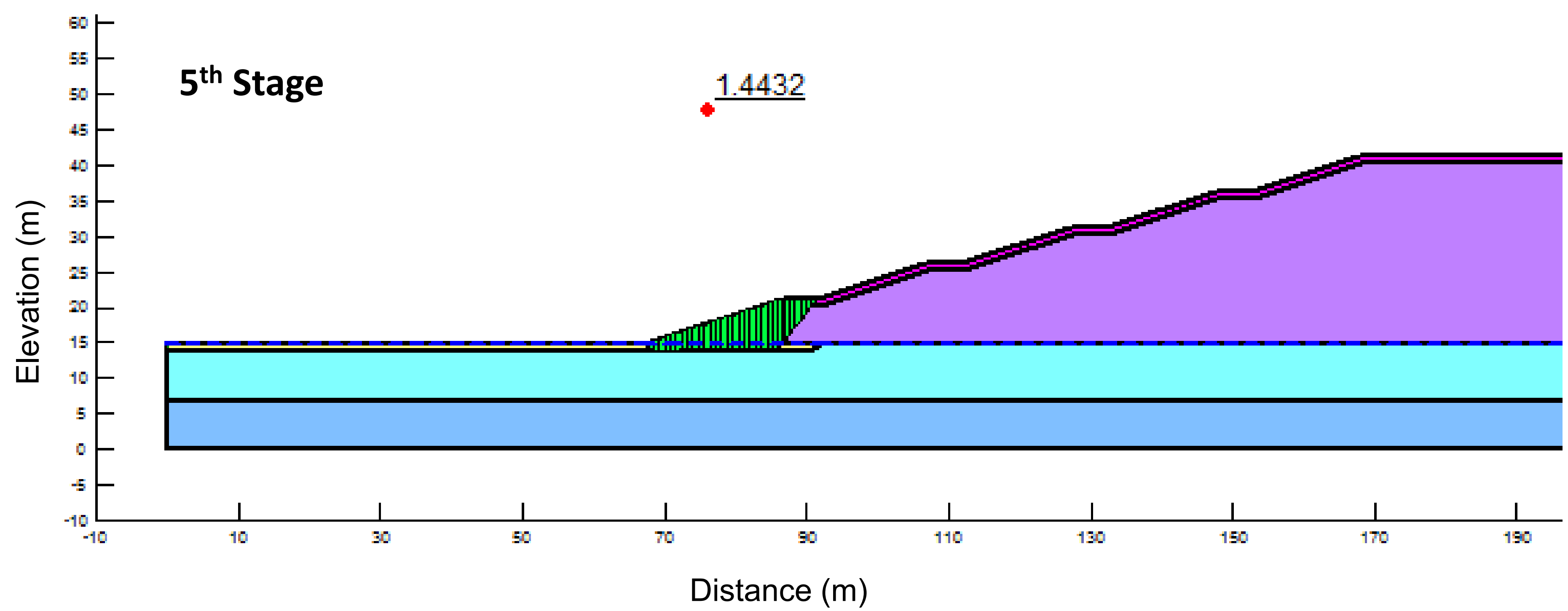
Job No: 1CT22.004.620 Filename: HB_Boston_DSTSF_Stability Memo_Figures_1CT022.013_Rev02_EL_AL	HOPE BAY PROJECT	Date: Nov. 2017 / Approved: AL / Figure: 2





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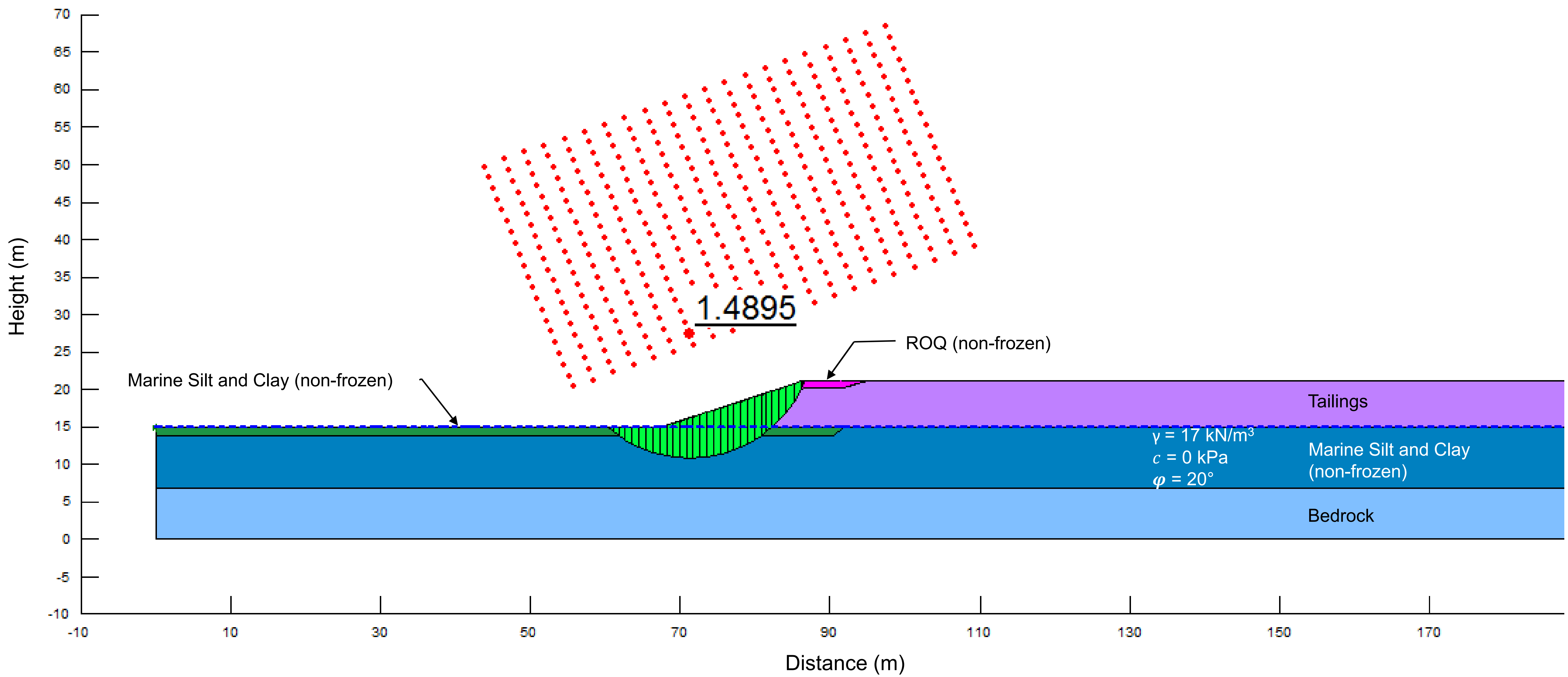
Job No: 1CT22.004.620 Filename: HB_Boston_DSTSF_Stability Memo_Figures_1CT022.013_Rev02_EL_AL	HOPE BAY PROJECT	Date: Nov. 2017 | Approved: AL | Figure: 3

Dry-stack: 1st Bench – Cohesion = 0 kPa

Friction Angle required = 20°



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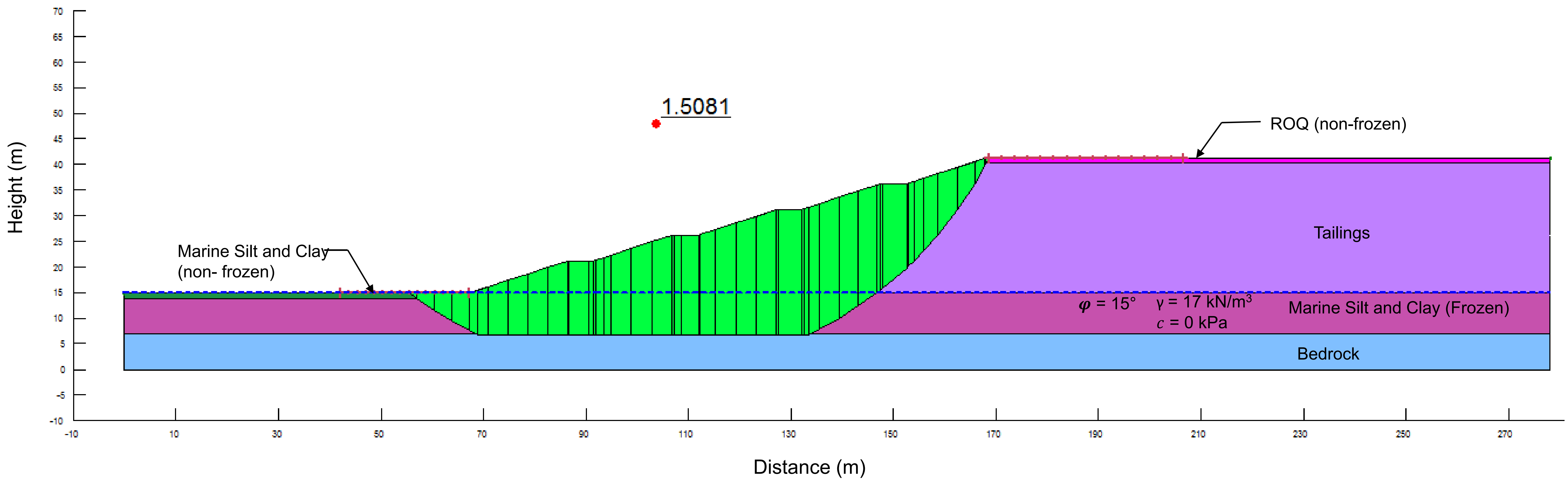
		Dry-stack: 1st Bench Stability Long Term Creep Effect
Job No: 1CT22.004.620 Filename: HB_Boston_DSTSF_Stability Memo_Figures_1CT022.013_Rev02_EL_AL	HOPE BAY PROJECT	Date: Nov. 2017 / Approved: AL / Figure: 4

Dry-stack: Overall Failure – Cohesion = 0 kPa

Friction angle required = 15°



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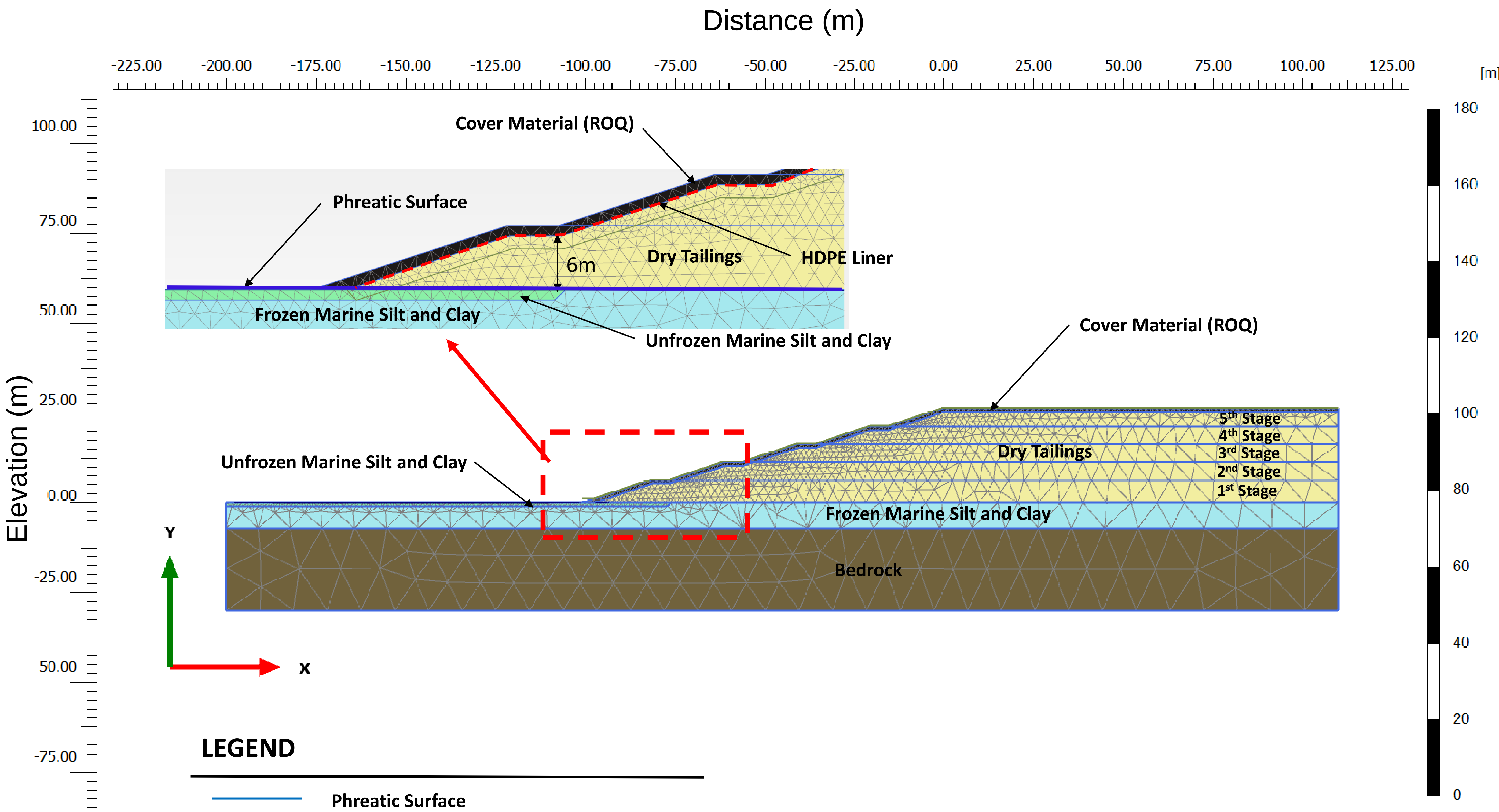
		Dry-stack Deep-Seated Failure Long-Term Creep Effect
Job No: 1CT22.004.620 Filename: HB_Boston_DSTSF_Stability Memo_Figures_1CT022.013_Rev02_EL_AL	HOPE BAY PROJECT	Date: Nov. 2017 · Approved: AL · Figure: 5

Attachment 1: Slope Stability Result



LEGEND

- Phreatic Surface
- HDPE Liner
- Dry Tailings
- Bedrock
- Frozen Marine Silt and Clay
- Unfrozen Marine Silt and Clay
- Unfrozen ROQ (Cover Material)

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		Analyzed Cross-Section
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 / Approved: CH / Figure: A1 - 1

Boston TMA Slope Stability Results – Undrained Condition – Plaxis

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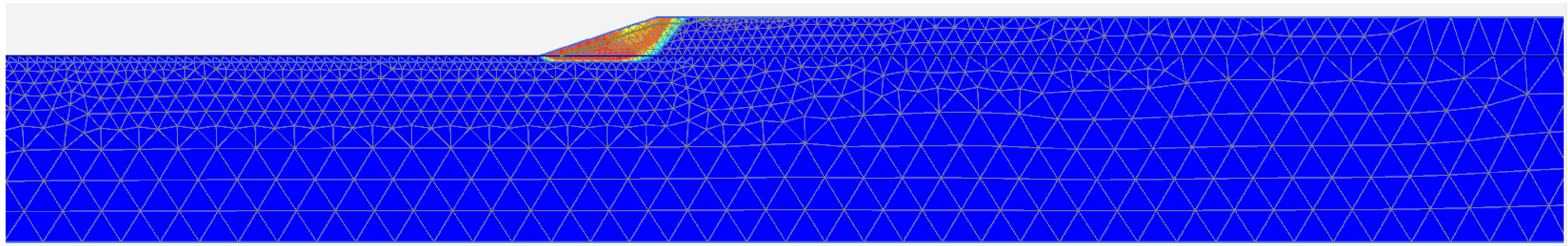

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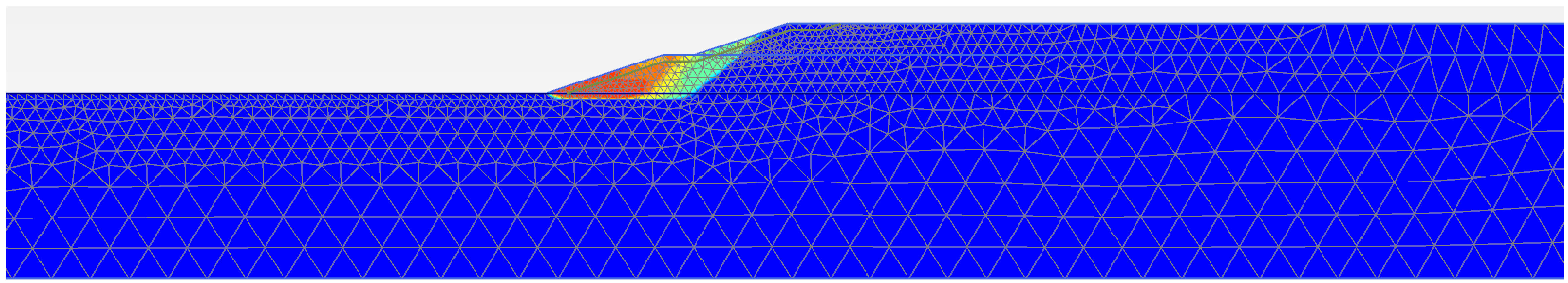

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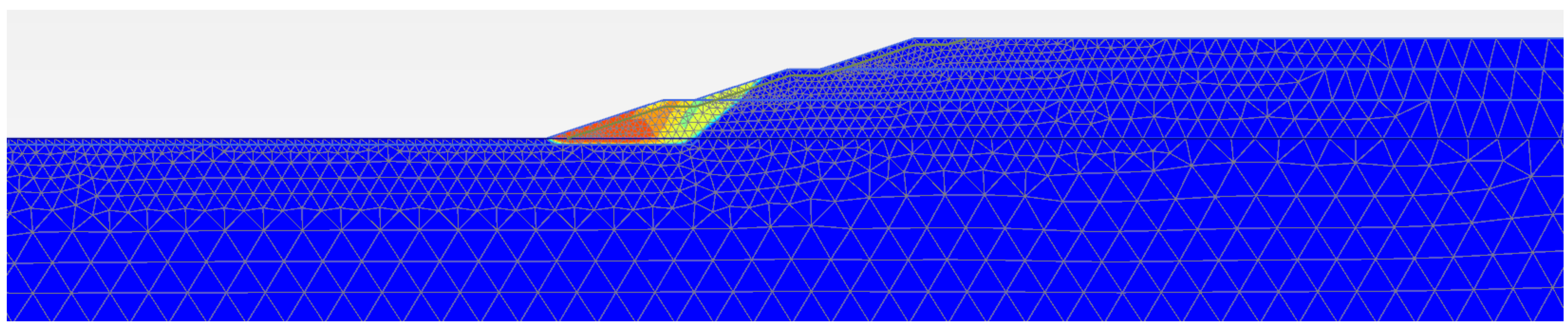

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY **Analyzed Cross-Section**
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 · Approved: CH · Figure: A1 - 2

Boston TMA Slope Stability Results – Undrained Condition – Plaxis

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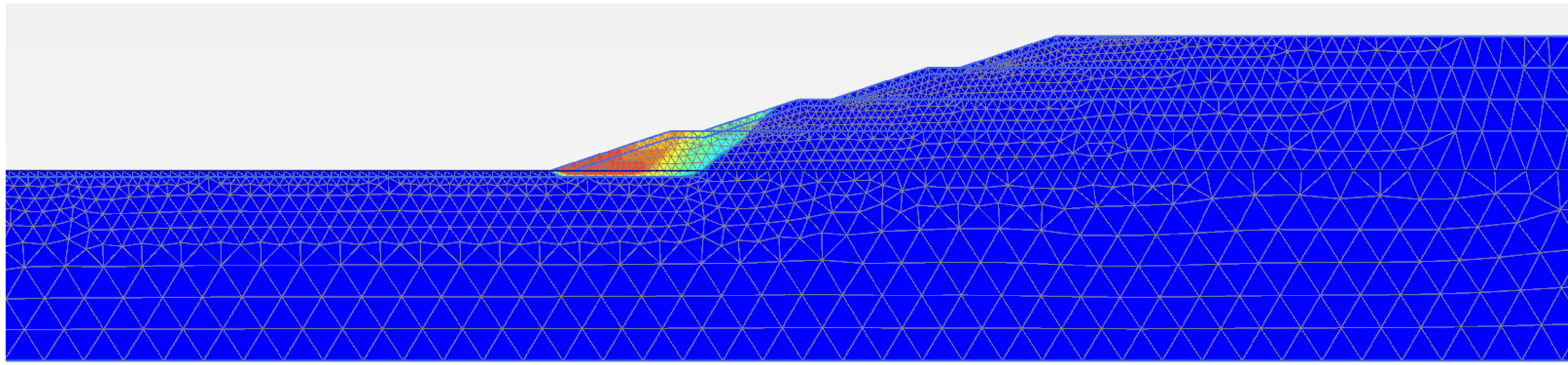

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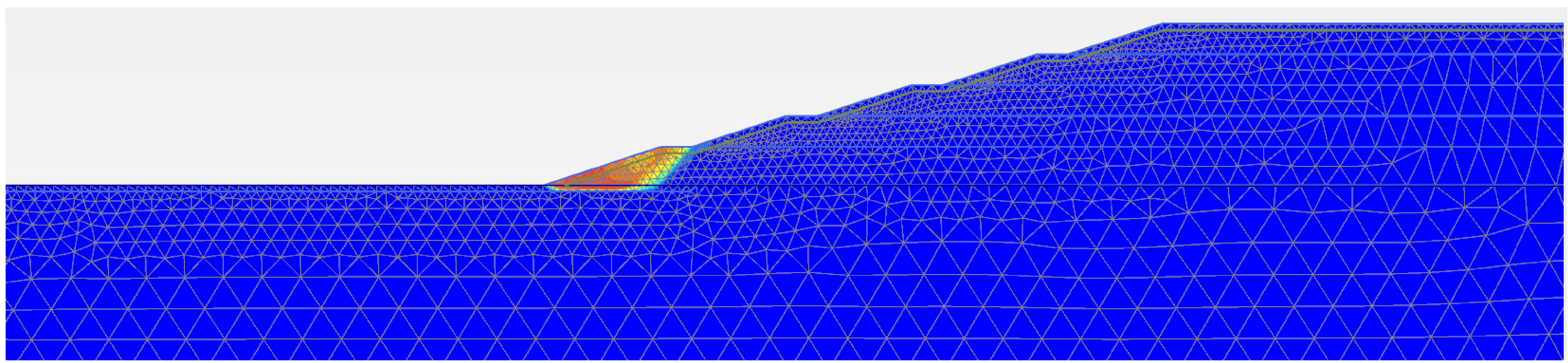

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Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 | Approved: CH | Figure: **A1 - 3**

Boston TMA Slope Stability Results – Drained Condition – Plaxis

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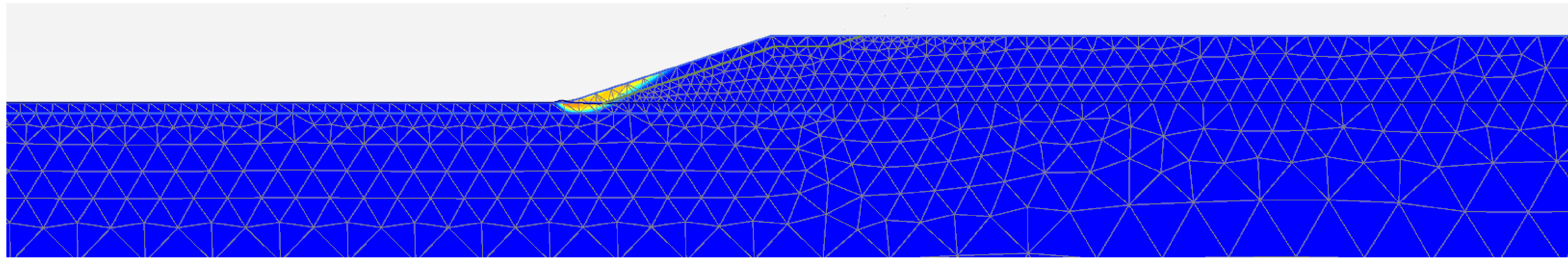

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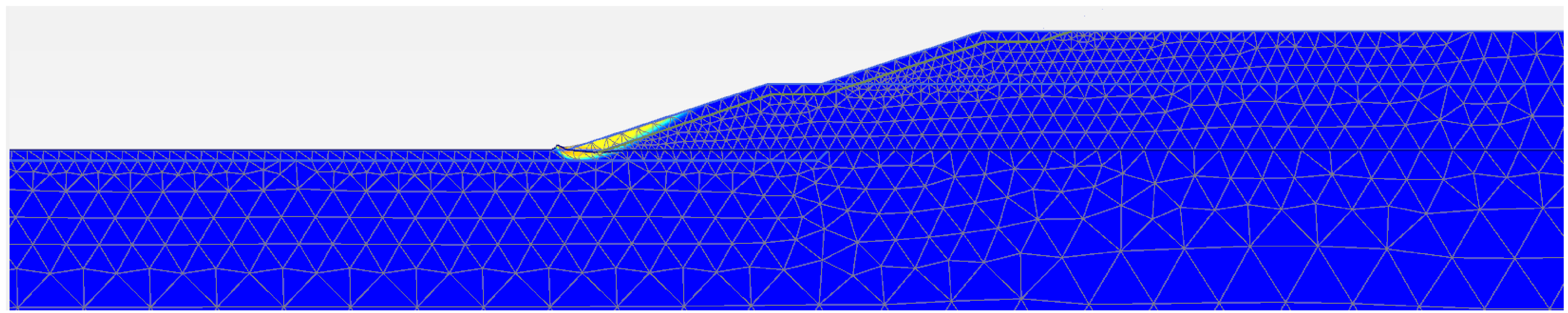

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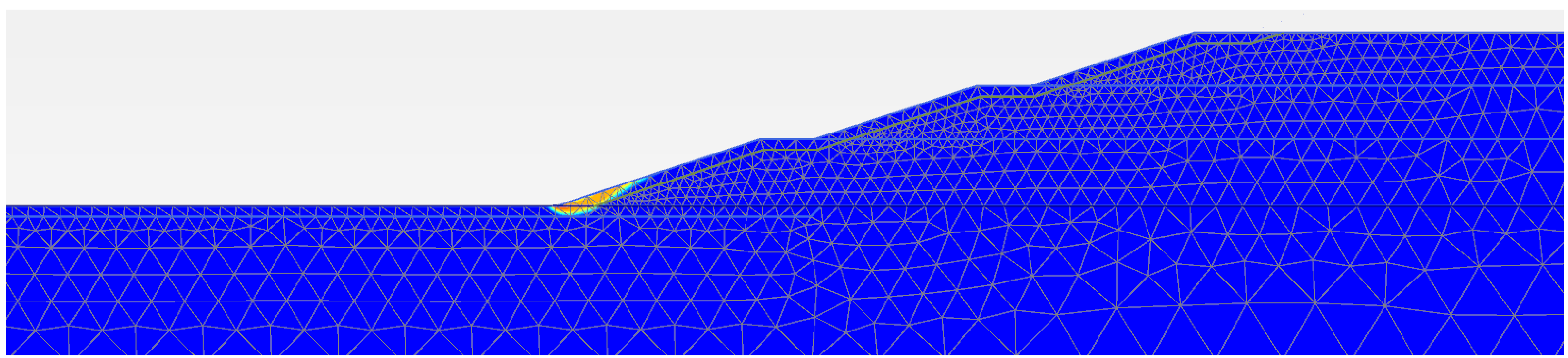

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Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 · Approved: CH · Figure: A1 - 4

Boston TMA Slope Stability Results – Drained Condition – Plaxis

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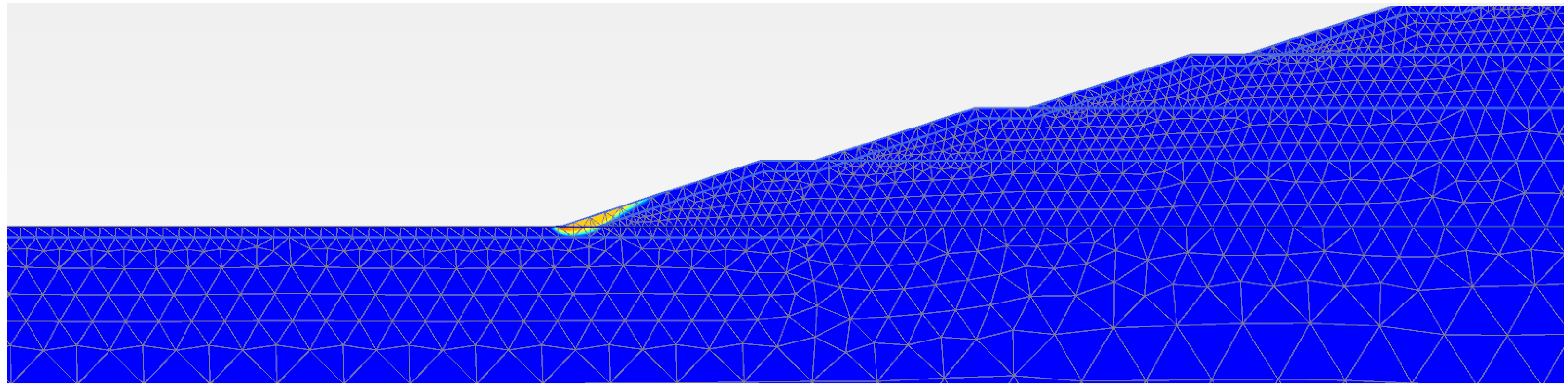

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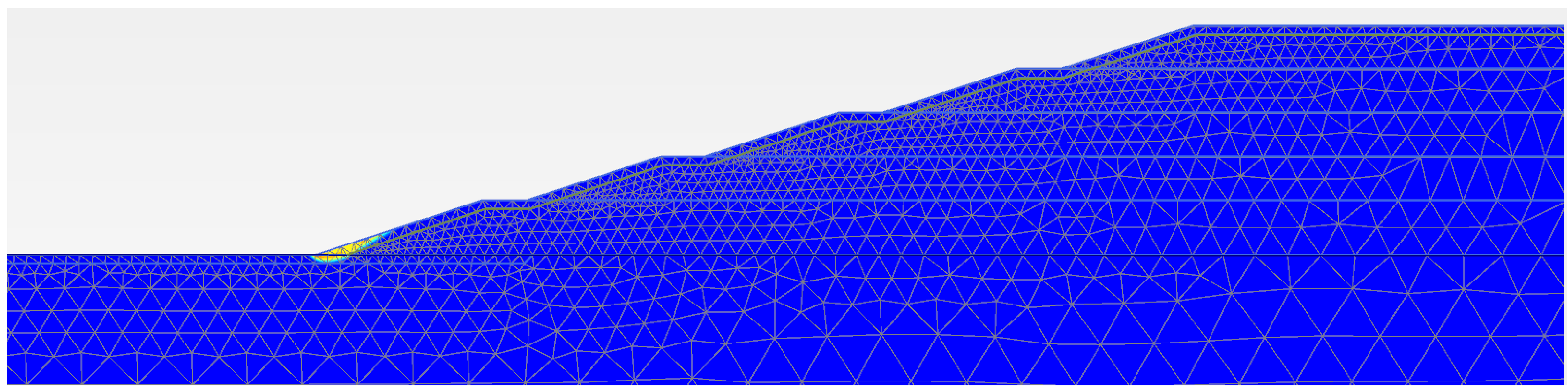

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY **Analyzed Cross-Section**
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 / Approved: CH / Figure: A1 - 5

Boston TMA Slope Stability Results – Pseudo-Static – Plaxis

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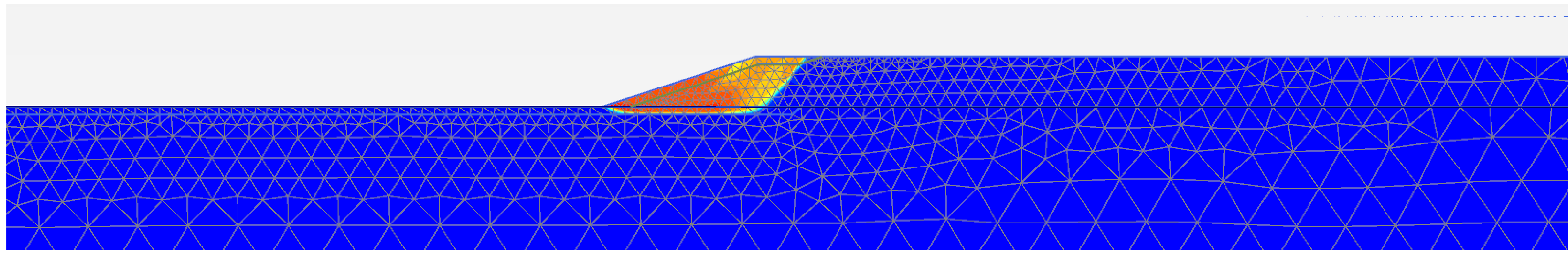

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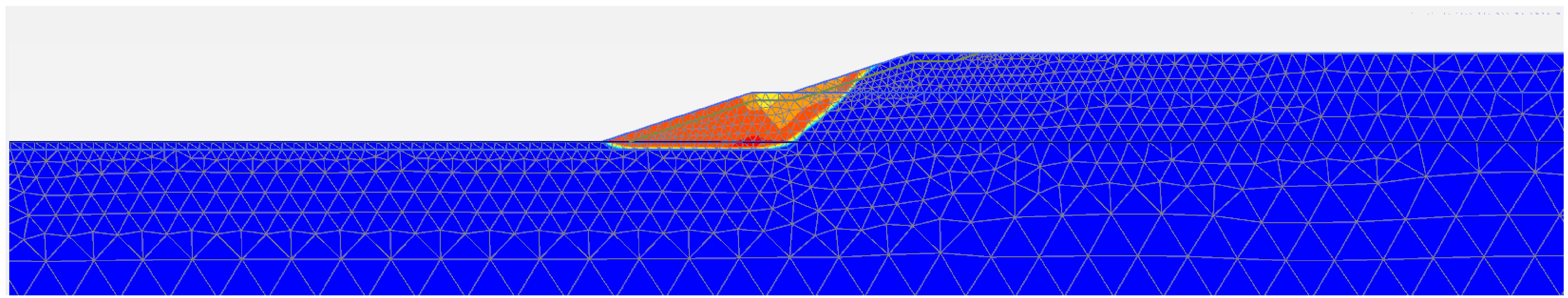

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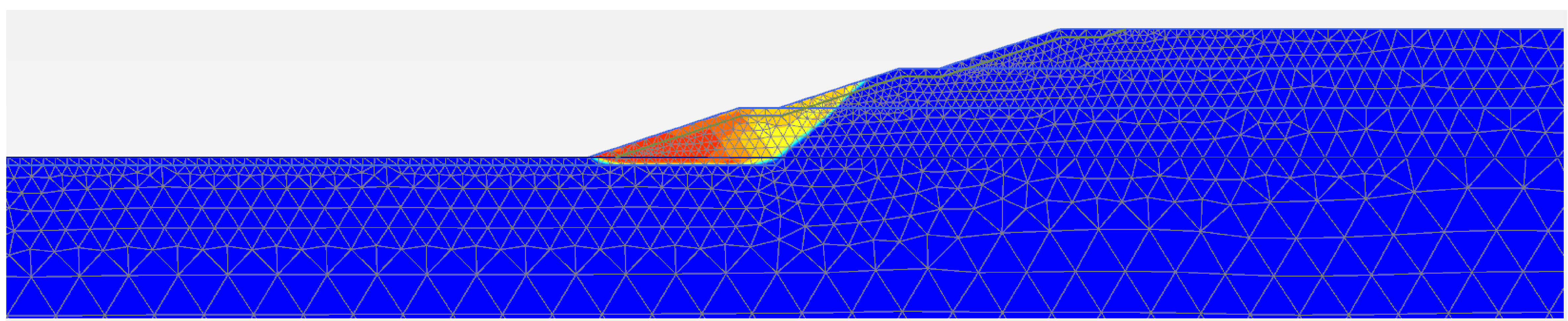

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TMAC RESOURCES

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

Job No: 1CT022.013

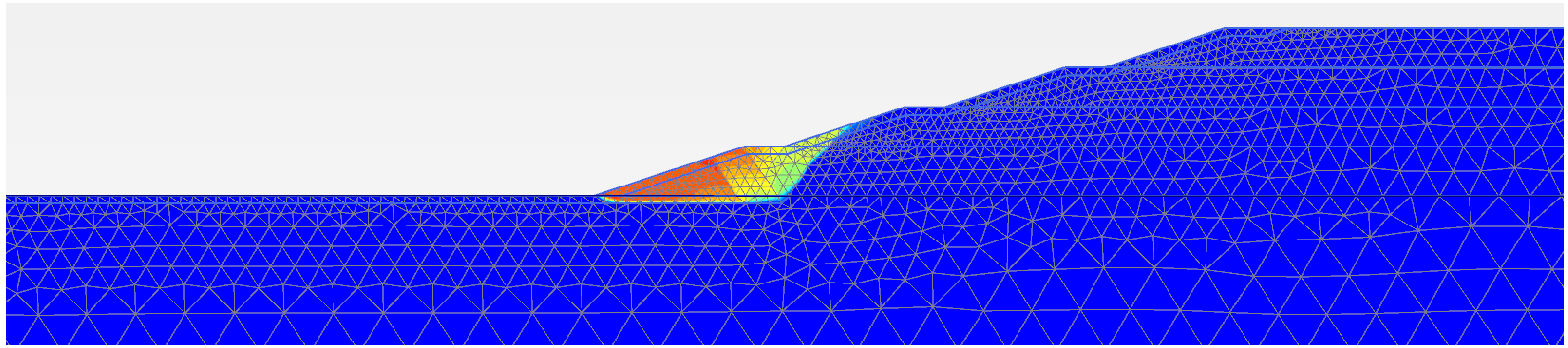
Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB

HOPE BAY PROJECT

Date:	Approved:	Figure:
November 2017	CH	A1 - 6

Boston TMA Slope Stability Results – Pseudo-Static – Plaxis

Fos = 1.25



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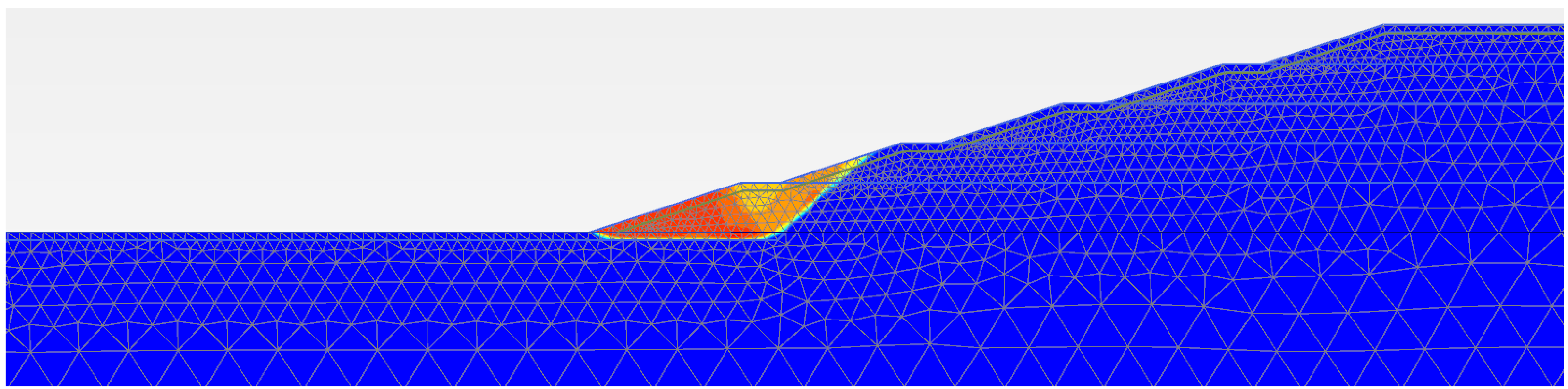

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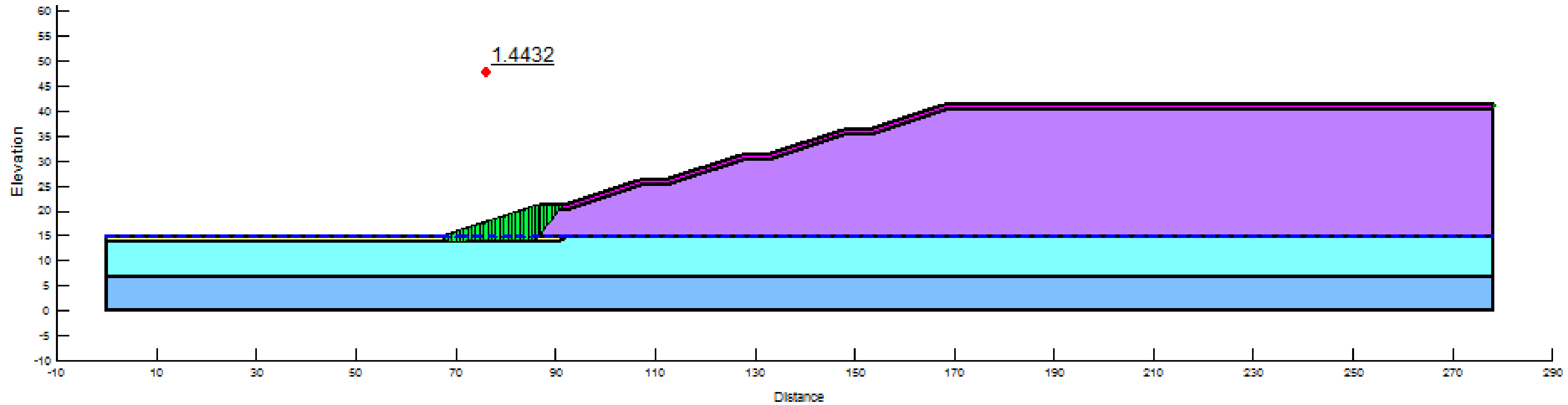
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 / Approved: CH / Figure: A1 - 7

Boston TMA Slope Stability Results – Undrained Condition – Slope/W



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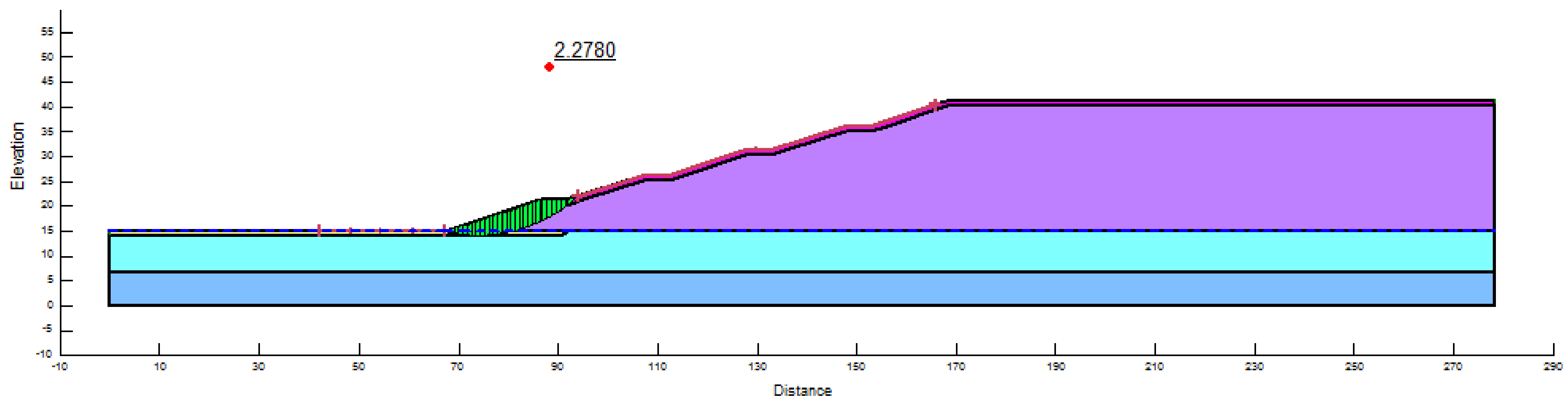

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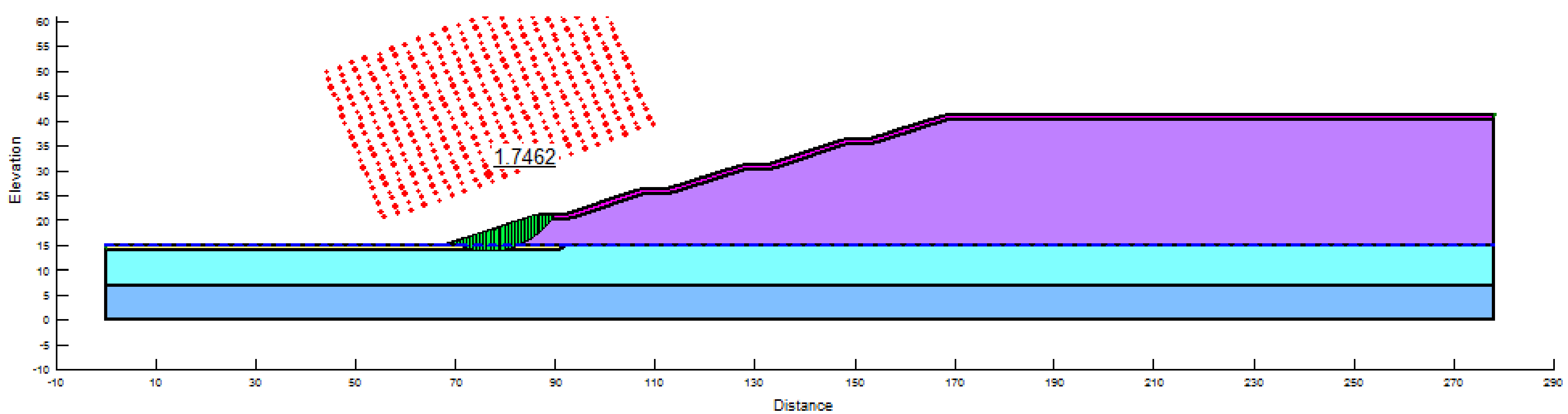
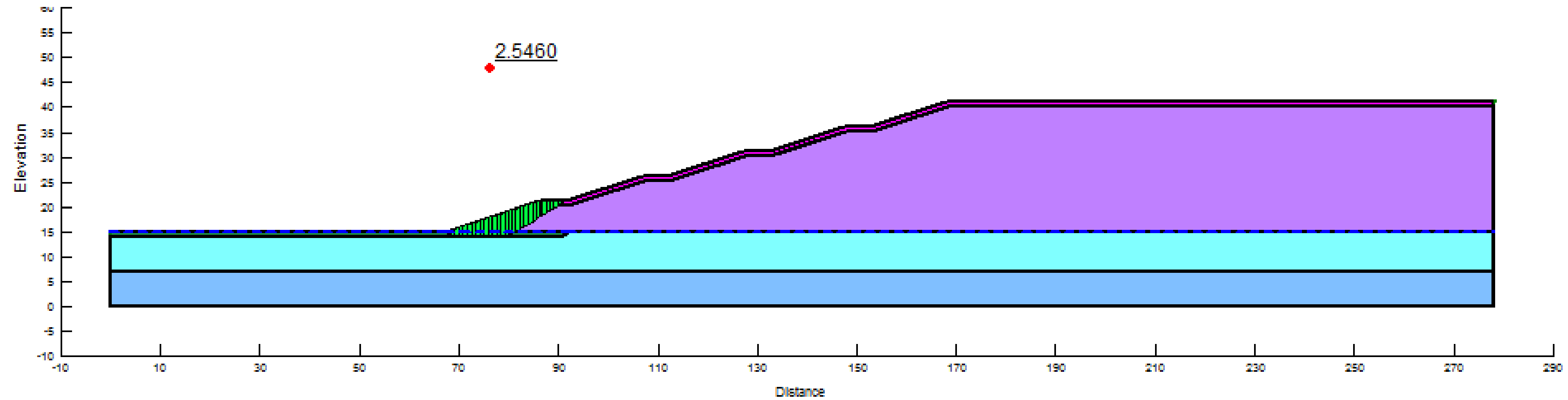


Grid & Radius

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 | Approved: SA | Figure: **A1 - 8**

Boston TMA Slope Stability Results – Drained Condition – Slope/W



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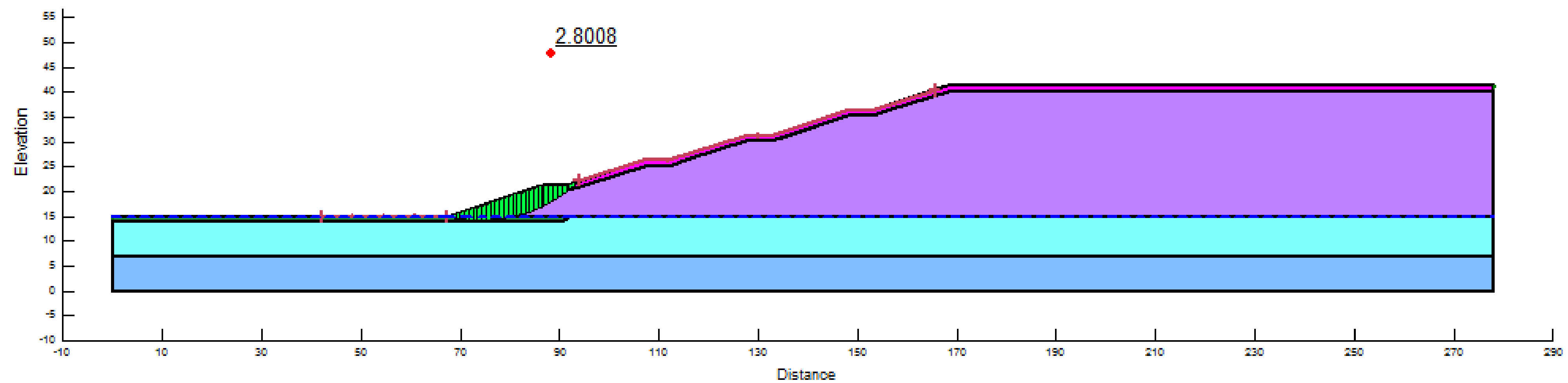

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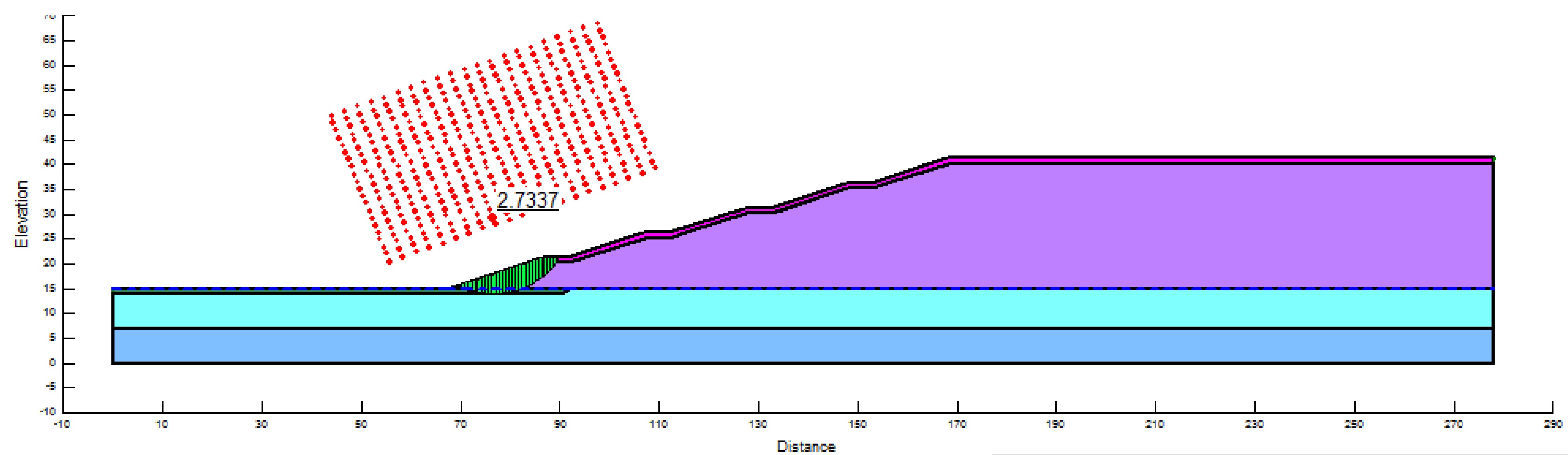

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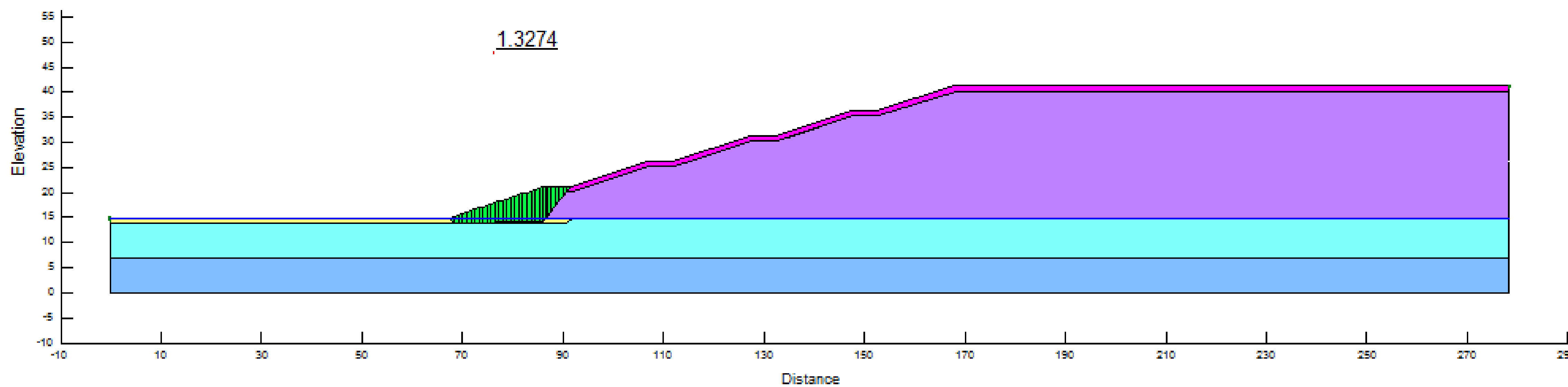
		Analyzed Cross-Section
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 | Approved: SA | Figure: **A1 - 9**

Boston TMA Slope Stability Results – Pseudo Static – Slope/W



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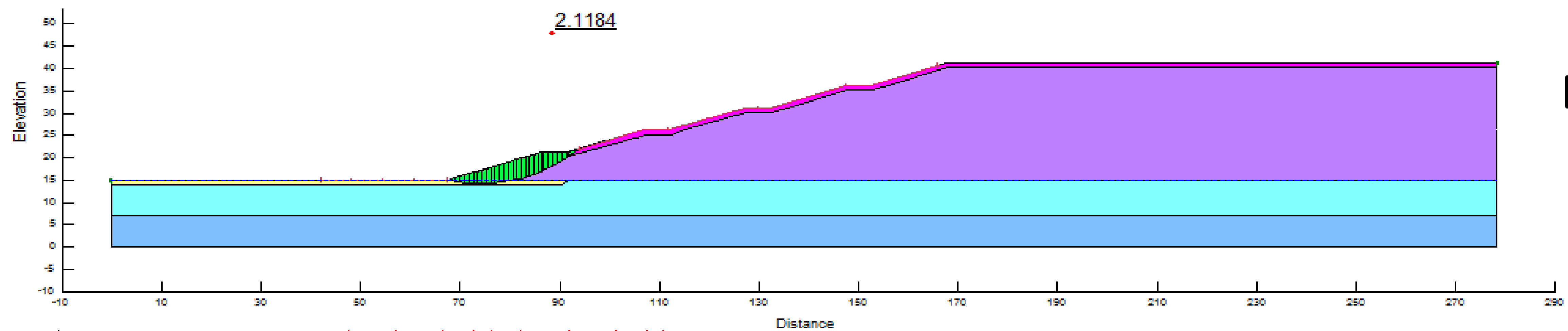

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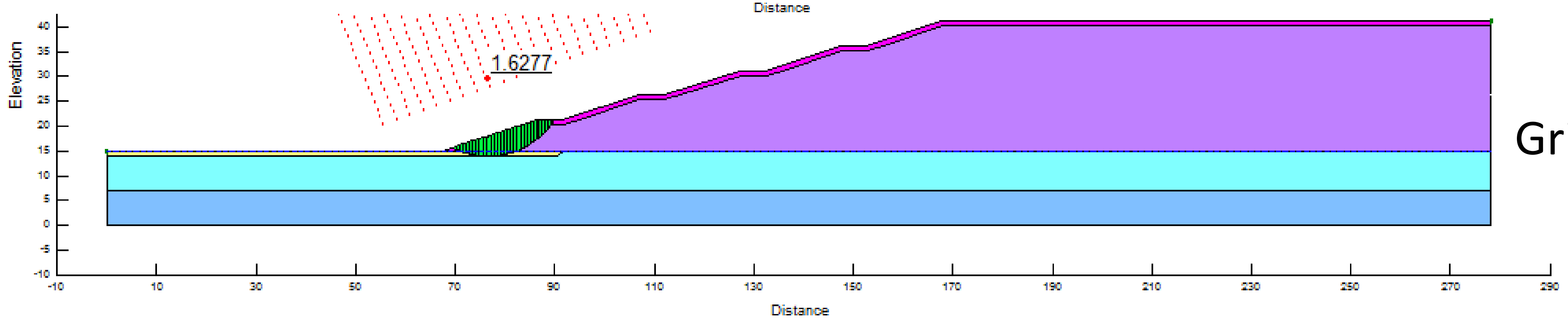

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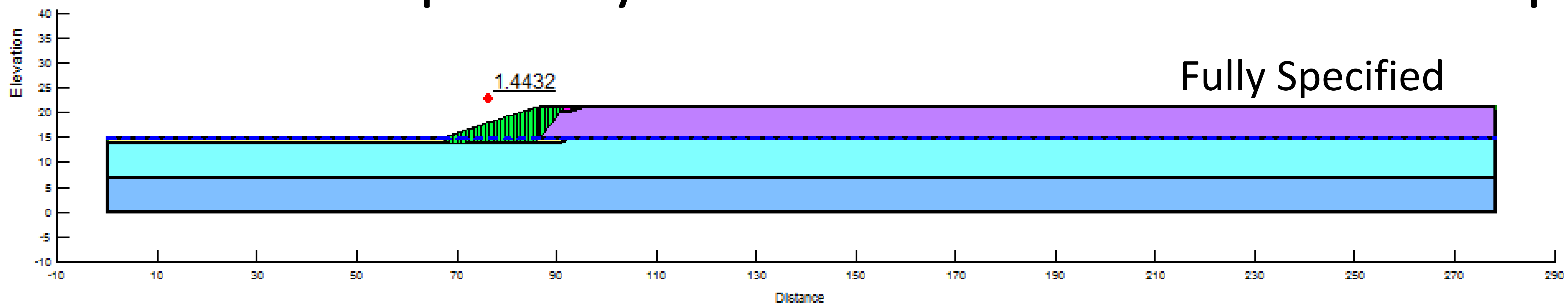
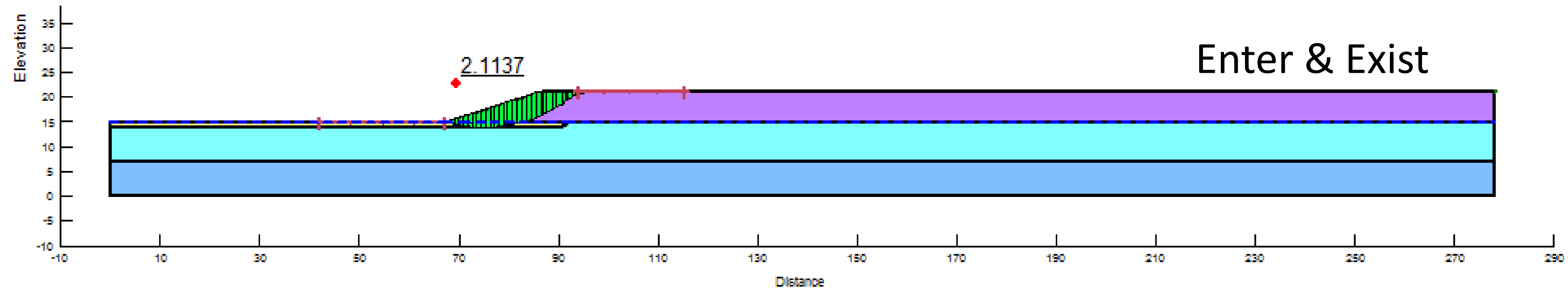
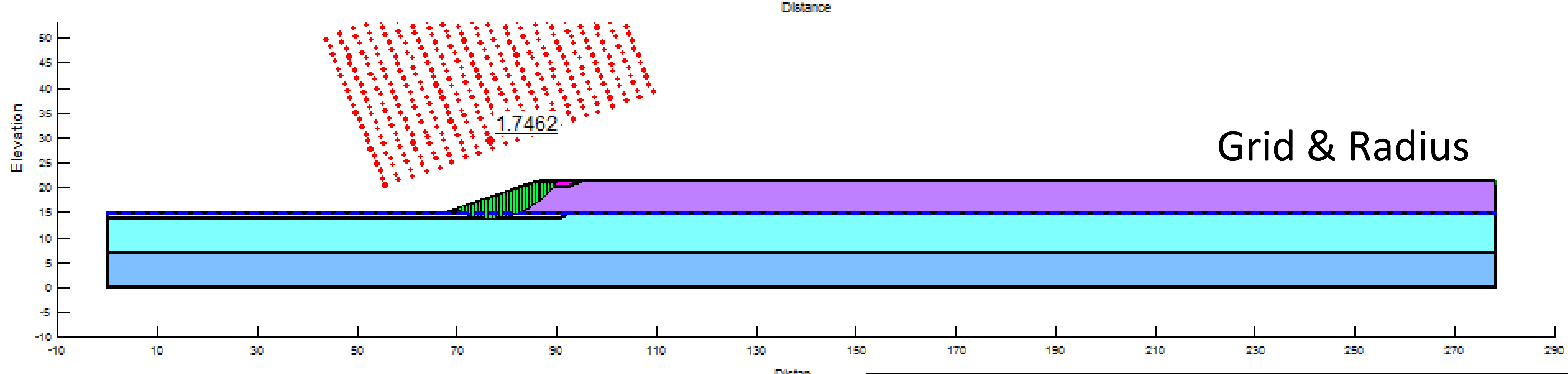
		Analyzed Cross-Section		
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017	Approved: SA	Figure: A1 - 10

Boston TMA Slope Stability Results – 1st Bench - Undrained Condition – Slope/W







srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
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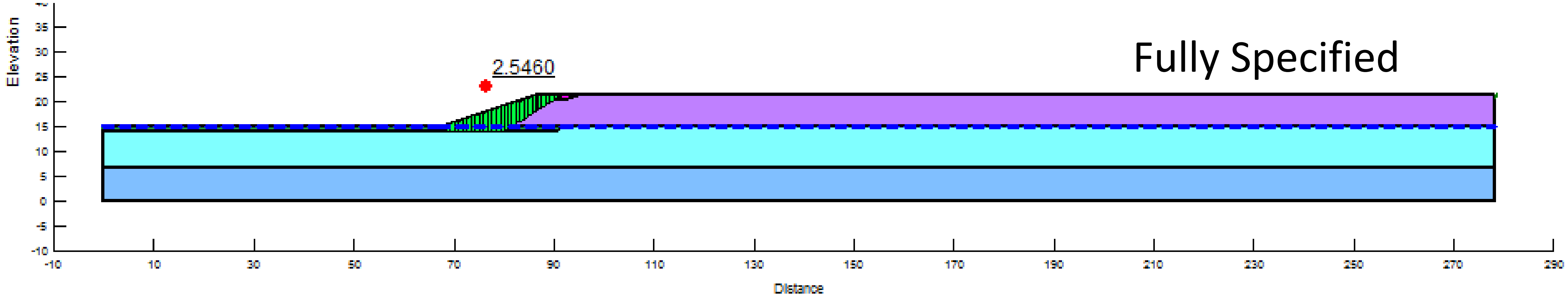
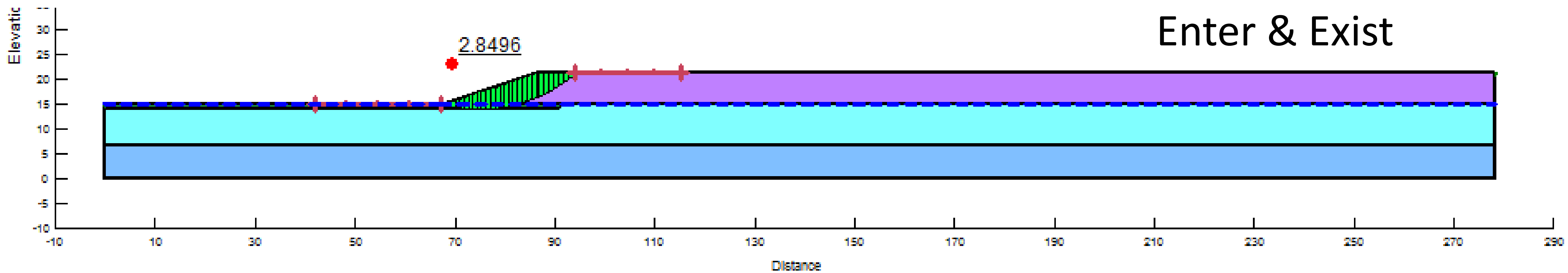
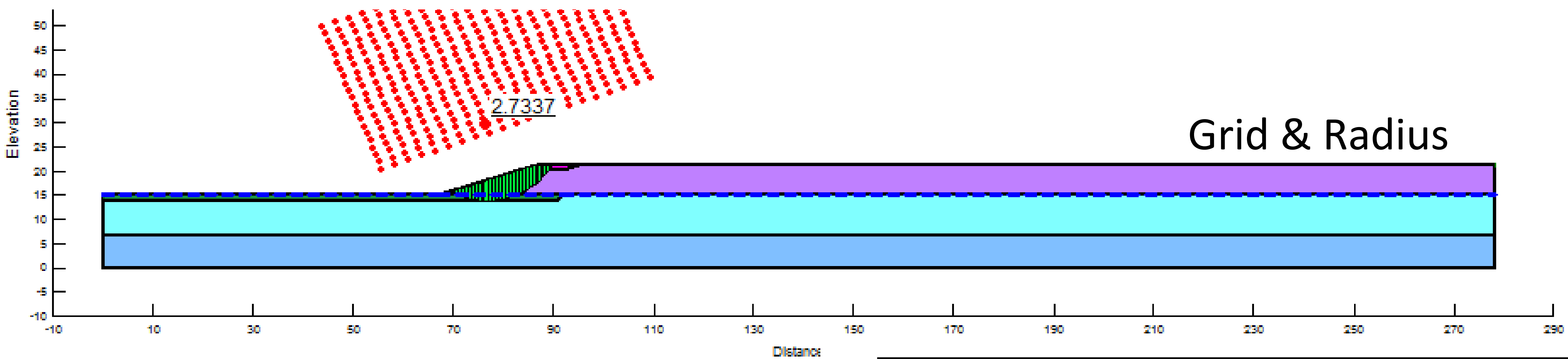
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 / Approved: SA / Figure: A1 - 11

Boston TMA Slope Stability Results – 1st Bench - Drained Condition – Slope/W







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		Analyzed Cross-Section
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 · Approved: SA · Figure: **A1 - 12**

Boston TMA Slope Stability Results – 1st Bench – Pseudo Static – Slope/W

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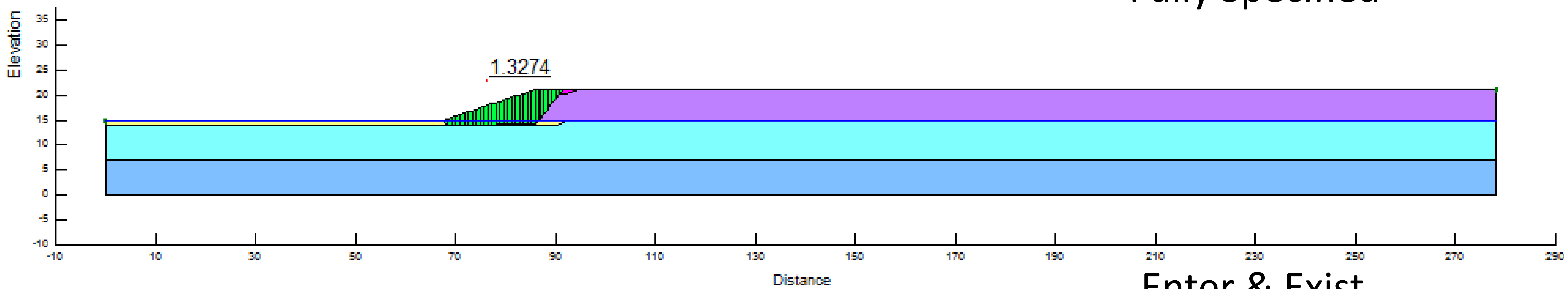

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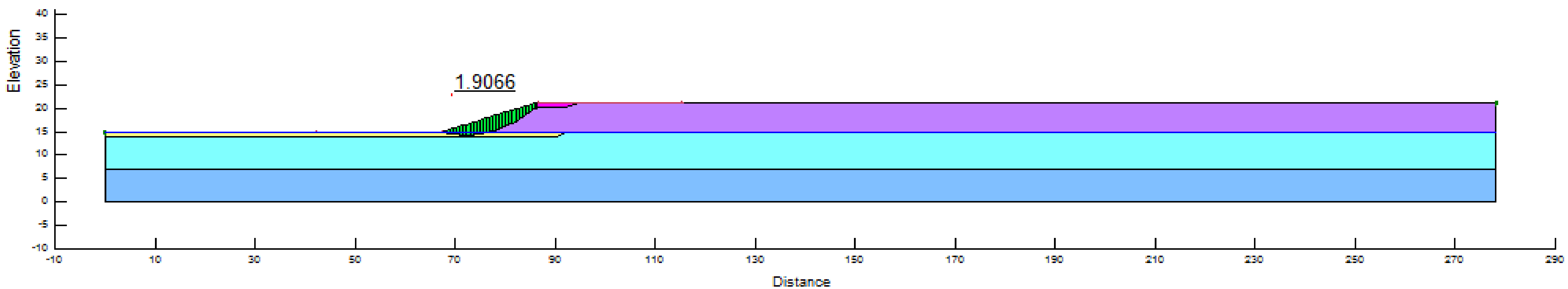

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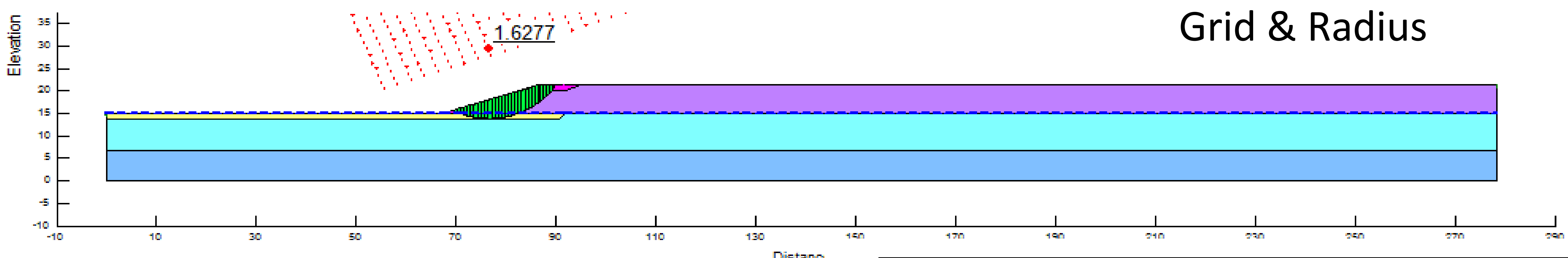

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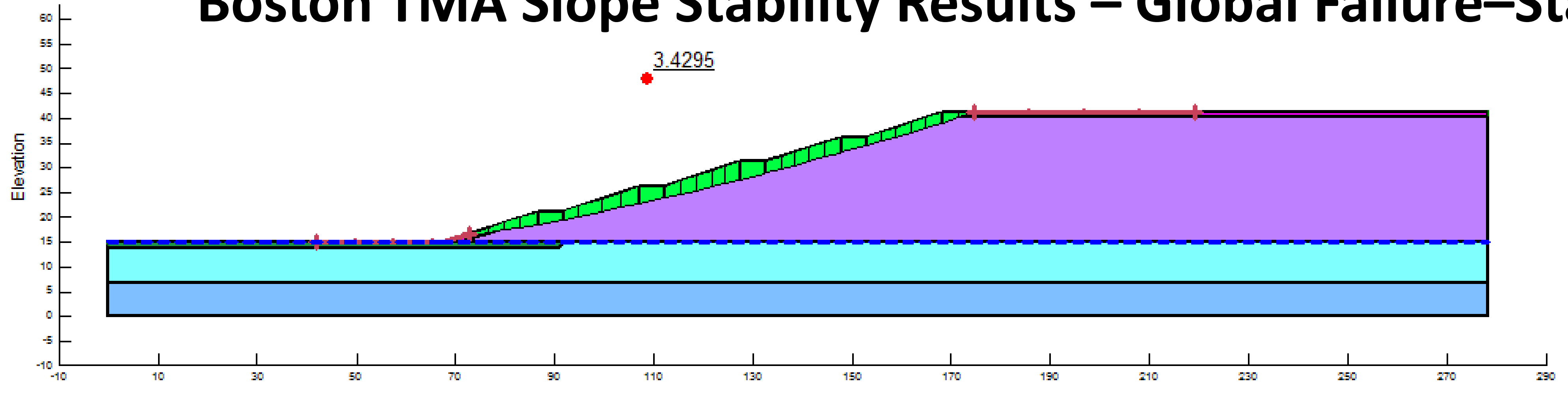
		Analyzed Cross-Section		
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017	Approved: SA	Figure: A1 - 13

Boston TMA Slope Stability Results – Global Failure–Static – Slope/W



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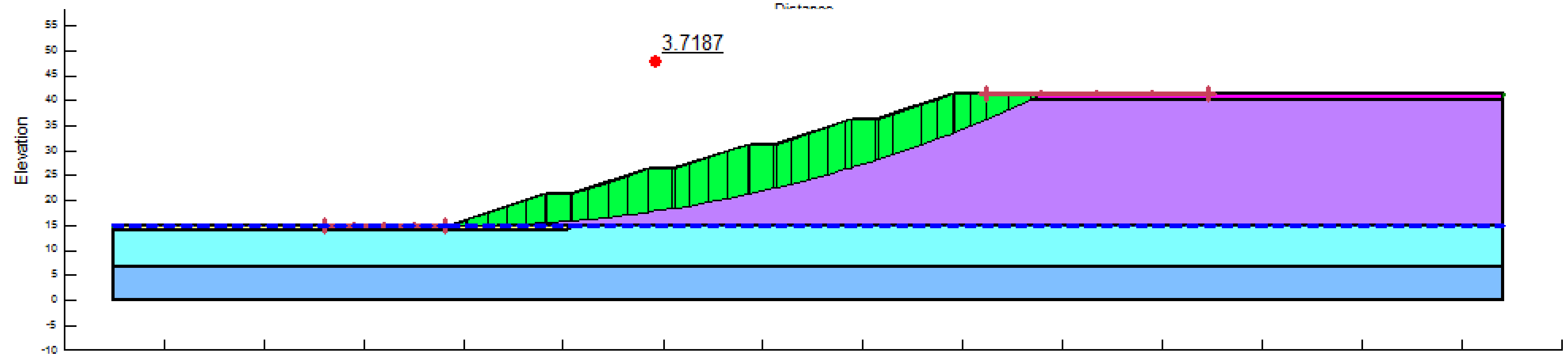

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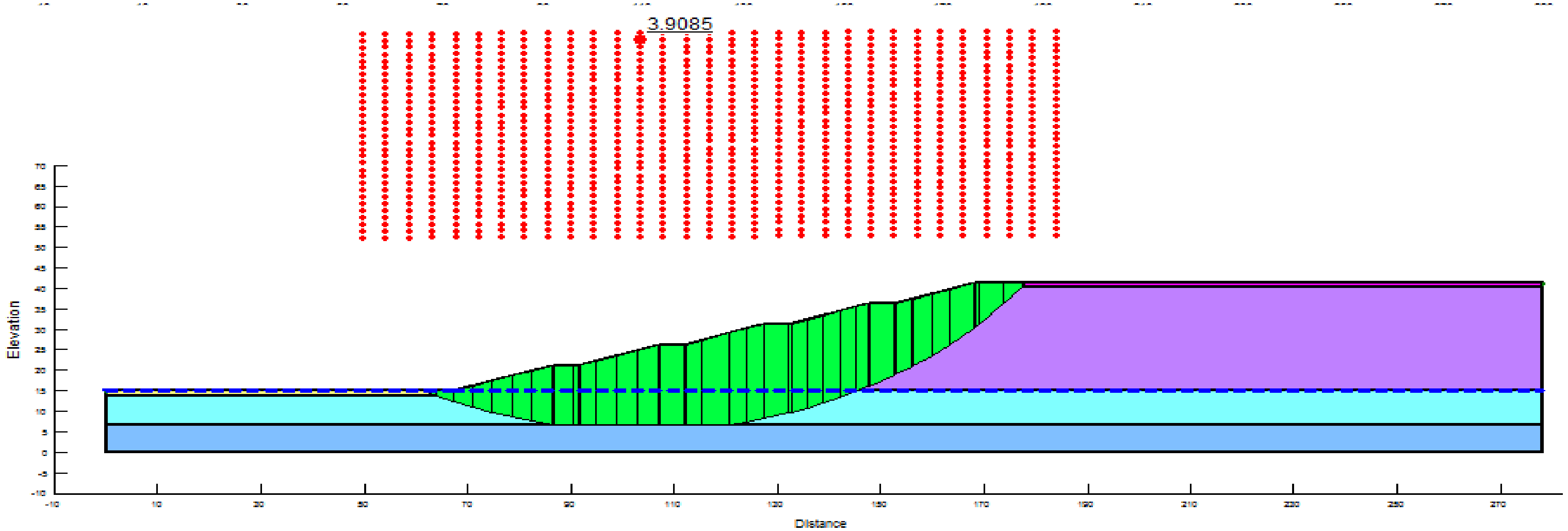

Grid & Radius

srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT022.013 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevB	HOPE BAY PROJECT	Date: November 2017 / Approved: SA / Figure: **A1 - 14**

Appendix D – Hope Bay Project: Boston Tailings Management Area Thermal Modelling

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, 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: Boston Tailings Management Area Thermal Modelling		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-2F, 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
Thermal model results for Year 85 added	Section 3	Model results used in stability analysis

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.

Tailings deposition at Boston will be in the form of dewatered (i.e. filtered) tailings placed in a compacted dry-stack. This tailings management area (TMA) is located approximately 1.2 km east of the proposed Boston camp and processing facilities, and is accessed via the Boston-Madrid all weather road.

1.2 Objectives

The objective of the modelling was to estimate active layer thickness in dry stack tailings in support of the long-term geochemical load balance for the Boston tailings management area (TMA). The model assumptions and results are summarized in this memo.

2 Methods

2.1 Model Setup

Modelling 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.

The final 2.3 Mm^3 configuration of the dry stack was used for modelling active layer thickness. The final configuration includes 3H:1V slopes with 5 m wide benches and a cover consisting of 0.3 m of gravel and 0.7 m of run of quarry (ROQ) material (Figure 1). The model assumes the cover consists entirely of ROQ material.

2.2 Model Inputs

2.2.1 Material Properties

Three material units were considered: native soil foundation (overburden clay), a cover consisting of ROQ material, and dry stack tailings (Figure 1). Table 1 presents a summary of the material properties.

Table 1: Material Thermal Properties

Material	Degree of Saturation (%)	Porosity	Thermal Conductivity (kJ m^{-1} day^{-1} °C^{-1})		Volumetric Heat Capacity (kJ m^{-3} °C^{-1})	
			Unfrozen	Frozen	Unfrozen	Frozen
Cover - ROQ Material	30	0.30	104	117	1,697	1,509
Tailings[1]	49	0.37	117	132	2,974	2,300
Tailings[1], Saturated	100	0.37	169	255	3,200	2,414
Overburden Clay [1,2]	85	0.52	150	185	2,178	1,801

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

The thermal properties for ROQ material were taken from previous work completed by SRK for granular pad design (SRK, 2017a). 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, 2017b). 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).

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 (2.85) and a dry density (1.8 g cm^{-3}) (SRK, 2017c). At complete saturation (100%), the gravimetric water content of the tailings is estimated to be 20.4%. The tailings properties with a gravimetric water content of 10% and a saturation of 49% was also assessed in the model. The most conservative active layer thickness based on these reasonable end-member water contents was subsequently used to estimate the water and load balance for the dry stack.

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 the surface. The boundary is defined by sinusoidal function of temperature and time based on Equation 1 and the parameters shown in Table 2.

$$T = \max(nf * \left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi + (t+182.5)}{365}\right)\right], \mathrm{nt}\left[MAAT + (C_A * t) + Amp * Sin\left(\frac{2\pi + (t+182.5)}{365}\right)\right] \quad \text{Eq.1}$$

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* $^{-1}$
- 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°C
ROQ Surface, Thawing n-factor *(nt)*	1.52
ROQ Surface, 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, 2017d). 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, unless otherwise specified in the memo. These values are based on average published values (SRK, 2017c) and considered to be reasonable base case conditions for the Project site. N-factors for natural overburden was applied to the model using values calibrated to ground temperatures measured at the Project site (SRK, 2017e).

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 (2017d). The model simulations are performed to the year 2100, which is 85 years beyond the year 2015.

2.2.3 Initial Conditions

The initial conditions were defined for each material region in the model. The tailings and ROQ material were assumed to be +2°C. The value is considered a conservative initial temperature for the tailings which is based on temperature measurements from a dry stack operating in a considerably warmer permafrost environment in Alaska. Tailings temperatures from this facility ranged from 0°C to -1°C (Neuffer et al., 2014).

The 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, 2017c). The model assumes continuous permafrost exists beneath the dry stack. Bedrock below the clay overburden was not considered in the model and would not influence estimation of active layer thaw at the top of the dry stack.

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 m^{-2} day^{-1} $°C^{-1}$ which was calculated from the average geothermal gradient (0.021°C m^{-1}) and the thermal conductivity of the clay overburden (SRK, 2017a).

Table 3: Summary of Boston Air Climate Change Factors Based on Climate Change Models

Year	Rate (°C $decade^{-1}$)	Air Climate Change Factor (°C day^{-1})
2015 - 2040	0.58	0.000160
2041 - 2070	0.54	0.000148
2071 - 2100	0.61	0.000167

3 Results

3.1 Period of Operation

Figure 2 shows active layer thickness for a five-year period of operation. The model assumes an exposed tailings surface with no active placement of material over the five-year period. The active layer thaw depth ranges from 2.7 m to 2.3 m, with an average of 2.5 m. Active layer thaw decreases as the tailings thermally equilibrate to the surface climate forcing boundary applied to the top surface of the model. As the near-surface ground temperature decreases over the five-year period, a greater amount of energy is required to seasonally warm and thaw the material.

3.2 Period of Closure

Active layer thickness for the period of closure was modelled for 85 years from 2015 to 2100. The model was based on final configuration of the dry stack and ROQ cover. Active layer thickness is reported as the total estimated thaw from the ROQ cover surface. Thaw of tailings below the cover is also provided, and calculated as:

$$T_{thaw} = ALT - CT$$

Where:
T_{Thaw} *is the thickness of seasonally thawed tailings below the ROQ cover (m)*
ALT *is the total active layer thickness from the ROQ cover surface (m)*
CT *is the ROQ cover thickness (m)*

Figures 3 and 4 show active layer thaw for tailings at 100% and 49% saturation, respectively. For tailings with a saturation of 100% (gravimetric water content of 20.4%), the maximum active layer thickness is 2.7 m (Figure 3). For tailings with a saturation of 49% (gravimetric water content of 10%), the maximum active layer thickness is 3.2 m (Figure 4). The thickness of seasonally thawed tailings below the ROQ cover is 1.7 m and 2.2 m, respectively.

The increase in active layer thickness for tailings with a reduced water content results from a lower heat capacity which causes more rapid warming of the tailings and a lower amount of latent heat required to change phase of the pore-ice to water. The model shows an overall increase in active layer thickness over time which relate to the increase in air temperature from climate change.

Figure 5 shows the ground temperature at the end of year 85. Foundation temperatures are predicted to be less than the -2°C thawing point depression of overburden clay.

4 Conclusions

The Boston TMA active layer thickness has been estimated for the period of operation and following placement of a ROQ cover, with consideration for climate change. Over the period of operation, the active layer thickness for exposed tailings located outside of areas of active material placement is estimated to average 2.5 m. Active layer thickness of tailings at 100% saturation (moisture content of 20.4%) and 49% saturation (moisture content of 10%) were modelled for an 85 year period (from 2015 to 2100) to estimate long-term thaw with a ROQ cover. The maximum long-term active layer thickness for tailings at 100% saturation and 49% saturation is 2.7 m and 3.2 m, respectively. The thickness of seasonally thawed tailings below the ROQ cover is 1.7 m and 2.2 m, respectively. Active layer thickness increase for tailings with a lower moisture content due to changes in the thermal properties of the material, mainly the reduced heat capacity and latent heat requirements at lower saturation.

5 References

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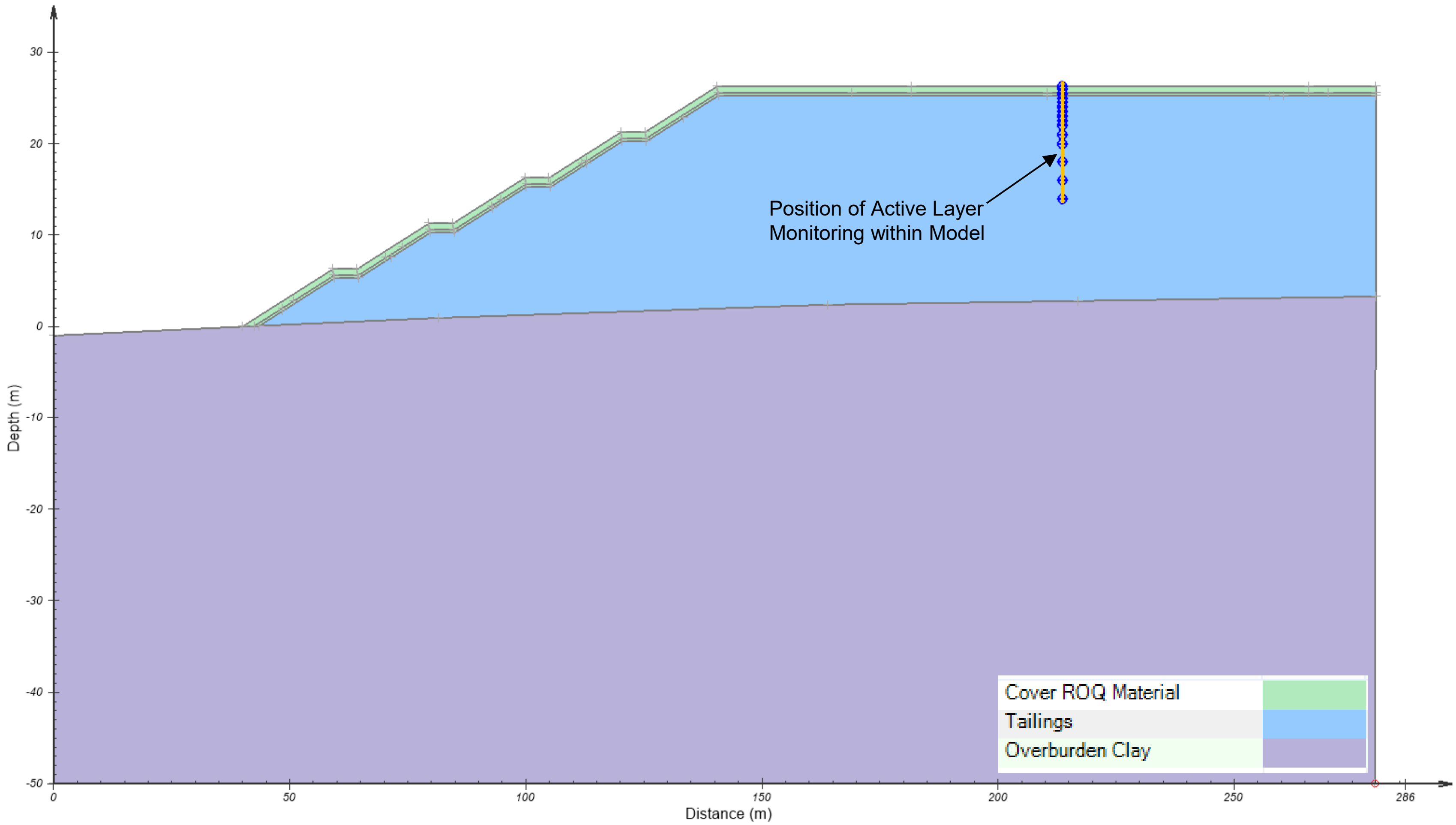
Figures



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MAC RESOURCES

Boston Dry Stack Thermal Modeling

Boston Dry Stack Model Domain

Job No: 1CT022.013

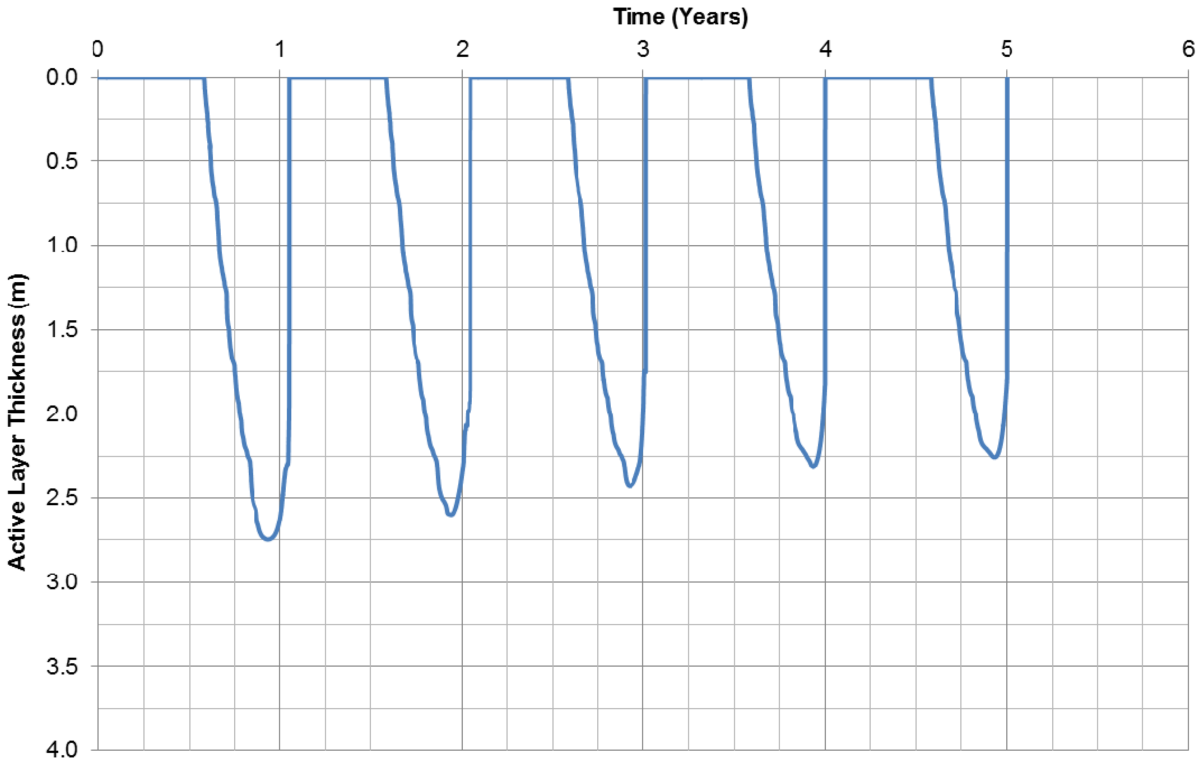
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Date: 11/29/2017

Approved: cws

Figure: 1



Year	Active Layer, Thaw of Tailings (m)
1	2.7
2	2.6
3	2.4
4	2.3
5	2.3

Notes:
1. Active layer thickness based on 0°C isotherm and measured from tailings surface
2. Model assumes thaw beneath an exposed tailings surface with no additional placement of material or ROQ cover

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MAC RESOURCES

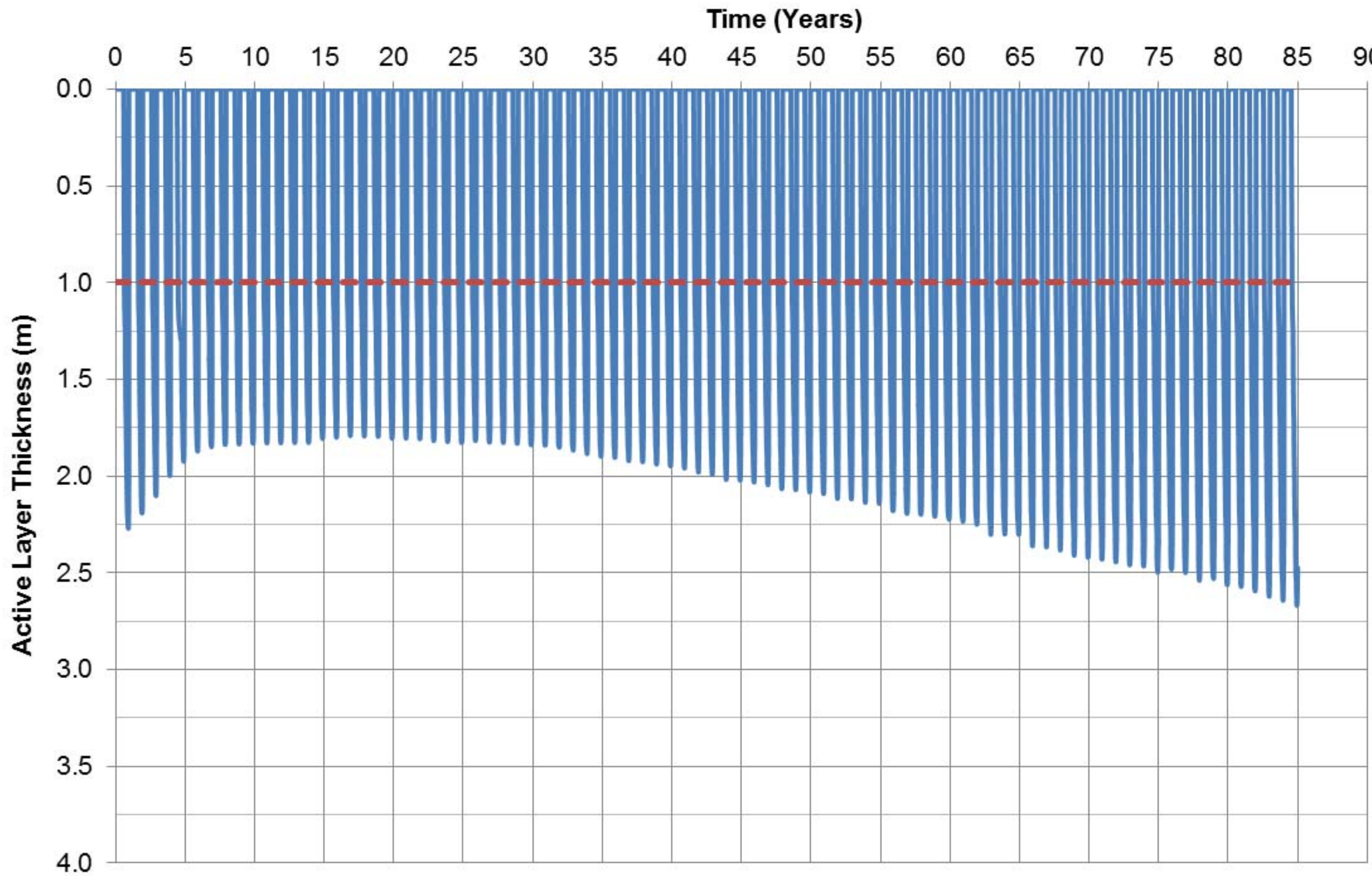
Job No: 1CT022.013
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Boston Dry Stack Thermal Modeling

Active Layer Thickness – Tailings Saturation 49% (Period of Operation)

Date: 11/29/2017 | Approved: cws | Figure: 2



Year	Active Layer Thickness (m)	Thaw of Tailings (m)
1	2.3	1.3
5	1.9	0.9
10	1.8	0.8
15	1.8	0.8
20	1.8	0.8
25	1.8	0.8
30	1.8	0.8
35	1.9	0.9
40	1.9	0.9
45	2.0	1.0
50	2.1	1.1
55	2.1	1.1
60	2.2	1.2
65	2.3	1.3
70	2.4	1.4
75	2.5	1.5
80	2.6	1.6
85	2.7	1.7

Notes:

1. Active layer thickness based on 0°C isotherm (solid blue line)
2. Base of ROQ cover indicated with dashed red line
3. Active layer depth measured from surface of ROQ cover
4. Thaw of tailings below base of ROQ cover

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MAC RESOURCES

Boston Dry Stack Thermal Modeling

Active Layer Thickness – Tailings Saturation 100% (Year 0 to 85)

Job No: 1CT022.013

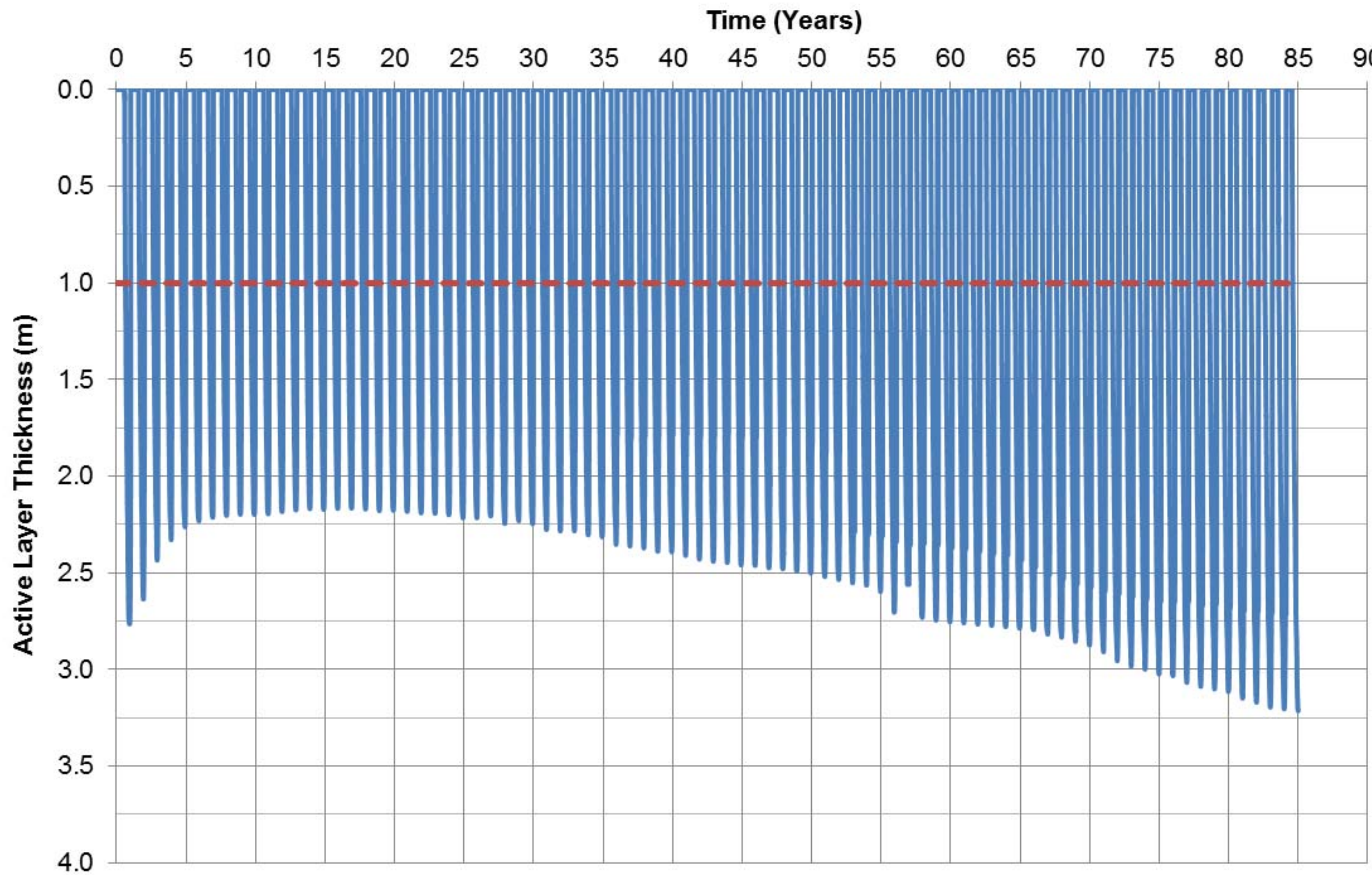
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Date: 11/29/2017

Approved: cws

Figure: 3



Year	Active Layer Thickness (m)	Thaw of Tailings (m)
1	2.8	1.8
5	2.3	1.3
10	2.2	1.2
15	2.2	1.2
20	2.2	1.2
25	2.2	1.2
30	2.2	1.2
35	2.3	1.3
40	2.4	1.4
45	2.5	1.5
50	2.5	1.5
55	2.6	1.6
60	2.8	1.8
65	2.8	1.8
70	2.9	1.9
75	3.0	2.0
80	3.1	2.1
85	3.2	2.2

Notes:
1. Active layer thickness based on 0°C isotherm (solid blue line)
2. Base of ROQ cover indicated with dashed red line
3. Active layer depth measured from surface of ROQ cover
4. Thaw of tailings below base of ROQ cover

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MAC RESOURCES

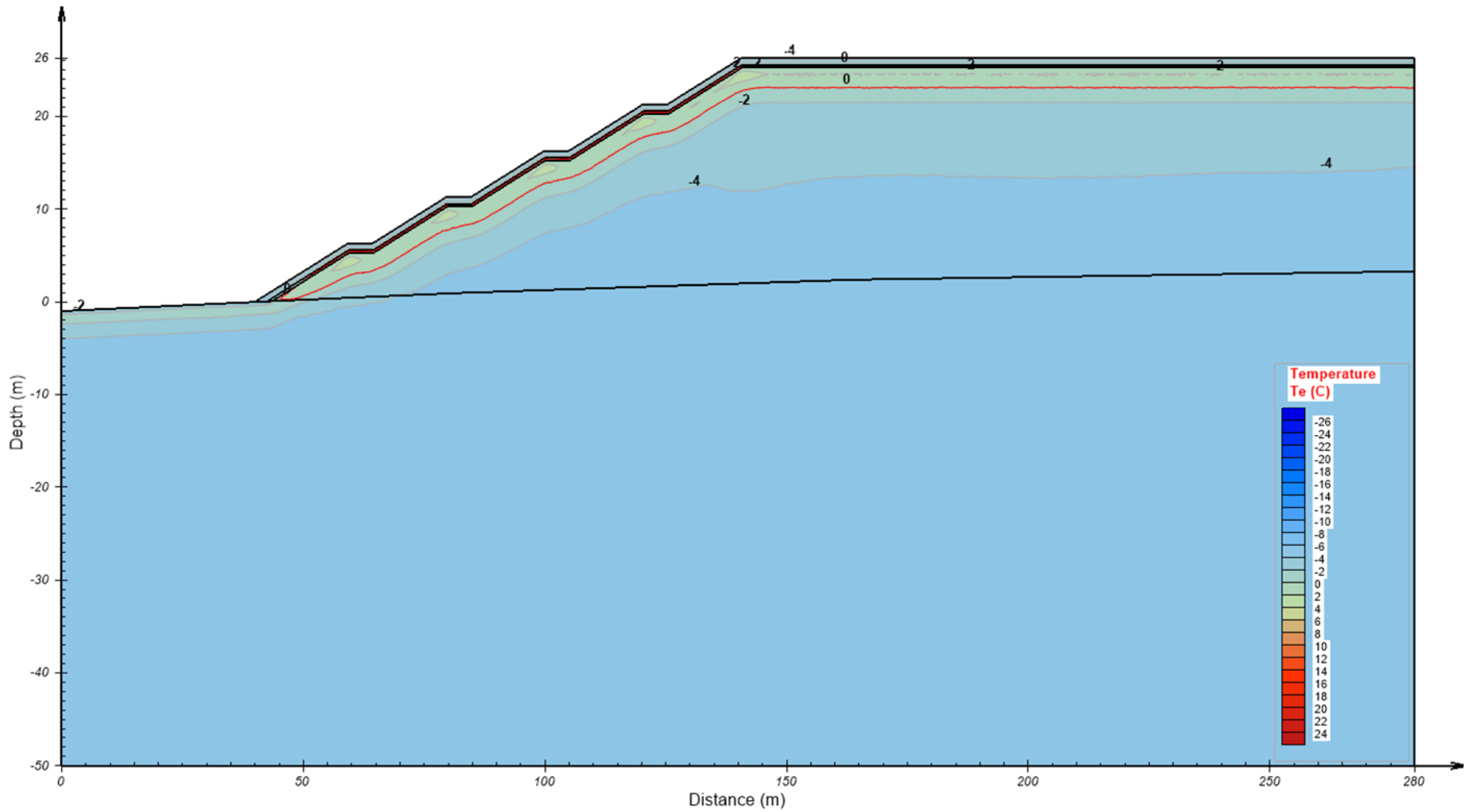
Boston Dry Stack Thermal Modeling

Active Layer Thickness – Tailings Saturation 49% (Year 0 to 85)

Job No: 1CT022.013
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Date: 11/29/2017
Approved: cws
Figure: 4



Notes:
1. Model result for model year 85
2. Solid red line shows the position of the -2°C isotherm

srk consulting	TMAC RESOURCES	Boston Dry Stack Thermal Modeling
		Model Result – Year 85
Job No: 1CT022.013 Filename: BostonDryStack.pptx	HOPE BAY PROJECT	Date: 11/29/2017 · Approved: cws · Figure: 5

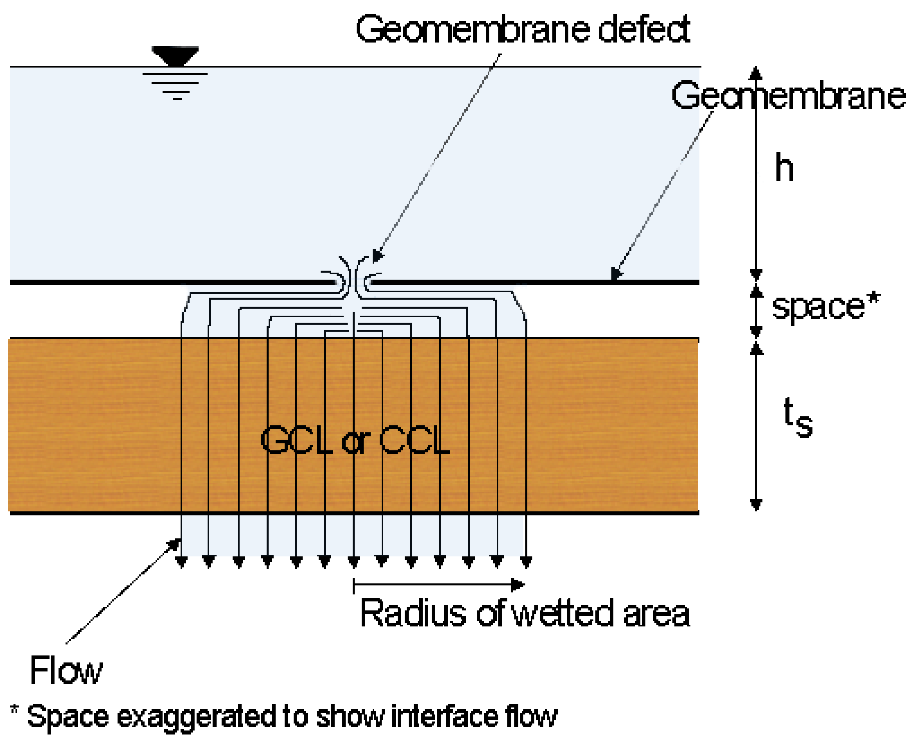
Appendix E – Hope Bay Project: Boston TMA Geomembrane Leakage Assessment

Leakage Rate Through a Composite Liner

Problem Statement

This calculator computes the rate of leakage through defects in a composite liner, i.e. geomembrane/CCL or geomembrane/GCL. The thickness of a CCL is between 0.3 to 1.5 m whereas the thickness of a hydrated GCL depends on the compressive stress applied during hydration. Typical values are between 5 and 10 mm; or in the order of 100 times less than the thickness of a CCL. Field evaluation, sponsered by USEPA, of leakage rate for double-lined landfills indicates that GM/GCL composite liners outperform GM/CCL liners (Othman et al.,1998.)

The rate of leakage through a geomembrane liner due to geomembrane permeability is negligible compared to the rate of leakage through defects in the geomembrane (Giroud and Bonaparte 1989.) Hence, only leakage through defects will be considered. If there is a defect in the geomembrane, the liquid first passes through the defect, then it flows laterally some distance between the geomembrane and the low-permeability soil, and, finally it infiltrates in the low permeability soil.



Flow between geomembrane and low-permeability soil is called interface flow, and is highly dependent upon the quality of contact between the two components (Bonaparte et al., 1989.) Contact conditions are defined as follows:

- **Good contact conditions** correspond to a geomembrane installed, with as few wrinkles as possible, on top of a low-permeability soil layer that has been adequately compacted and has a smooth surface.

- **Poor contact conditions** correspond to a geomembrane that has been installed with a certain number of wrinkles, and/or placed on a low-permeability soil that has not been well compacted and does not appear smooth.

Table 1

	Contact quality factor (C_{qo}) (circular, square, rectangular)	Contact quality factor ($C_{q\infty}$) (infinite length)
Good contact	0.21	0.52
Poor contact	1.15	1.22

The Help model provides guidance for estimating the defect densities (Schroeder et al., 1994). Some useful information on the Help model is given in the Technical Note on Using HELP Model (ver 3.07). There are mainly two types of defects, manufacturing defects and installation defects. Typical geomembranes may have about 0.5 to 1 (1 to 2 per hectare) pinholes per acre from manufacturing defects (Pinholes are defects with a diameter equal or smaller than the geomembrane thickness. The density of installation defects is a function of the quality of installation, testing, materials, surface preparation, equipment, and QA/QC program. Representative installation defect densities as a function of the quality of installation are given in Table 2 for landfills being built today with the state of the art in materials, equipment and QA/QC.

Table 2

Installation quality	Defect density (number per acre)	Frequency (percent)
Excellent	Up to 1	10
Good	1 to 4	40
Fair	4 to 10	40
Poor	10 to 20*	10

*Higher defect densities have been reported for older landfills with poor installation operations and materials; however, these high densities are not characteristic of modern practice.

Studies by Giroud and Bonaparte (1989) have shown that for geomembrane liners installed, with strict construction quality assurance, could have one to two defects per acre (4000 m^2) with a typical defect diameter of 2 mm (i.e., a defect area of 3.14 * 10^{-6} m^2).

Typical for liner performance evaluation one defect per acre (4000 m^2) is considered with a defect area of 0.1 cm^2 (equivalent to defect diameter of 3.5 mm), for a conservative design a defect area of 1 cm^2 (equivalent defect diameter of 11 mm) can be considered (Giroud et al., 1994)

Problem Solution

Different geomembrane defect shapes will be considered:

Circular defect with diameter of d

$$\frac{Q}{A} = n \cdot 0.976\, C_{qo} \cdot [1 + 0.1 \cdot (h/t_s)^{0.95}] \cdot d^{0.2} \cdot h^{0.9} \cdot k_s^{0.74}$$

Square defect with side length b

$$\frac{Q}{A} = n \cdot C_{qo} \cdot [1 + 0.1 \cdot (h/t_s)^{0.95}] \cdot b^{0.2} \cdot h^{0.9} \cdot k_s^{0.74}$$

Infinitely long defect with width of b

$$\frac{Q^*}{A} = n \cdot C_{q\infty} \cdot [1 + 0.2 \cdot (h/t_s)^{0.95}] \cdot b^{0.1} \cdot h^{0.45} \cdot k_s^{0.87}$$

Rectangular defect with width of b and length of B

$$\frac{Q}{A} = n \cdot C_{qo} \cdot [1 + 0.1 \cdot (h/t_s)^{0.95}] \cdot b^{0.2} \cdot h^{0.9} \cdot k_s^{0.74}$$

$$+ n \cdot C_{q\infty} \cdot [1 + 0.2 \cdot (h/t_s)^{0.95}] \cdot (B - b) \cdot b^{0.1} \cdot h^{0.45} \cdot k_s^{0.87}$$

Q	Leakage rate through the considered geomembrane defect (m^3/s)
Q*	Leakage rate per unit length of geomembrane defect ($m^3/s.m$)
A	Considered geomembrane area (m^2)
n	Number of defects per considered geomembrane area (A)
Co or $C_{q\,\infty}$	Contact quality factor (see above table 1)
h	Hydraulic head on top of the geomembrane (m)
t_s	Thickness of the low-permeability soil component of the composite liner (m)
d	Diameter of circular defect (m)
b	Width of defect (m)
B	Length of rectangular defect (m)

Limitation of the equations presented (Giroud et al. 1997):

- If the effect is circular, the defect diameter should be no less than 0.5 mm and not greater than 25 mm. In the case of the defects that are not circular, it is proposed to use these limitations for the defect width.
- The liquid head on top of the geomembrane should be equal to or less than 3 m.

Input Values

Geometry of circular defect

Considered geomembrane area (A)	203000	m^2
Hydraulic head on top of the geomembrane (m)	0.1	m
Thickness of the low-permeability soil (m)	2 >	m
Permeability of the low-permeability soil (m/s)	1.3E-7	m/s

Properties of circular defect

Contact (good or poor)	Good ▼	
Number of defects (n)	51	
Diameter of defect (d)	0.002	m

Geometry of square defect

Considered geomembrane area (A)	203000	m^2
Hydraulic head on top of the geomembrane (m)	0.1	m
Thickness of the low-permeability soil (m)	2	m
Permeability of the low-permeability soil (m/s)	1.3E-7	m/s

Properties of square defect

Contact (good or poor)	Good ▼	
Number of defects (n)	51	
Side length of defect (d)	0.01	m

Geometry of Infinitely Long Defect

Considered geomembrane area (A)	4000	m^2
Hydraulic head on top of the geomembrane (m)	0.3	m
Thickness of the low-permeability soil (m)	2	m
Permeability of the low-permeability soil (m/s)	1.00E-7	m/s

Properties of Infinitely Long Defect

Contact (good or poor)	Good ▼	
Number of defects (n)	0	

Width of defect (b)	0.0002	m

Geometry of Rectangular Defect

Considered geomembrane area (A)	4000	m^2
Hydraulic head on top of the geomembrane (m)	0.3	m
Thickness of the low-permeability soil (m)	2	m
Permeability of the low-permeability soil (m/s)	1.00E-7	m/s

Properties of Rectangular Defect

Contact (good or poor)	Good ▼	
Number of defects (n)	0	
Width of defect (b)	0.002	m
Length of defect (B)	0.01	m

Calculate

Solution

Circular Defect

Leakage Rate	1.509E-011	$(m^3/s)/m^2$
	44.0640	lphd (liter per hectare per day)
		$1\ (m^3/s)/m^2 = 8.64 \cdot 10^{11}$ lphd
	1.37708	gpad (gallons per acre per day)
		1 lphd =0.1056 gpad

Square Defect

Leakage Rate	2.134E-011	$(m^3/s)/m^2$
	44.0640	lphd (liter per hectare per day)
		$1\ (m^3/s)/m^2 = 8.64 \cdot 10^{11}$ lphd
	1.94672	
		1 lphd =0.1056 gpad

Infinitely Long Defect

Leakage Rate per unit length	0.000E+000	$(m^3/s)/m^2.m$
	0.0000	lphd/m (liter per hectare per day per meter)
		$1\ (m^3/s)/m^2 = 8.64 \cdot 10^{11}$ lphd
	0.00000	gpad/ft (gallons per acre per day per feet)

1 lphd =0.1056 gpad

Rectangular Defect

Leakage Rate	0.000E+000	$(m^3/s)/m^2.m$
	0.0000	lphd (liter per hectare per day)
		$1\ (m^3/s)/m^2 = 8.64 \cdot 10^{11}$ lphd
	0.00000	gpad (gallons per acre per day)
		1 lphd =0.1056 gpad

Assistance

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Appendix F – Dry Stack Creep Deformation Analysis

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Memo

To:	John Roberts, PEng, Vice President Environment Oliver Curran, MSc, Environmental Affairs	**Client:**	TMAC Resources Inc.
From:	Eric Lino	**Project No:**	1CT022.004
Reviewed By:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Boston Tailings Management Area – Dry Stack Creep Deformation Analysis		

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 only, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Tailings deposition at Boston will be in the form of dewatered (i.e., filtered) tailings placed in a compacted dry-stack. This tailings management area (TMA) is located approximately 1.2 km east of the proposed Boston camp and processing facilities, and is accessed via the Boston-Madrid all-weather road. At closure, the dry-stack will be covered with a geosynthetic low permeability infiltration reducing cover.

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 dry-stack design life can affect the performance or compromise the stability of the Boston TMA. The analysis also confirms whether the integrity of the underlying saline foundation will be affected by creep deformations.

2 Boston Dry Stack Details

2.1 General

The dry stack facility will occupy a flat area in the east of the Aimaokatalok Lake extension, south of the proposed new Boston airstrip. This area is separated from the mining infrastructure (SRK 2017a) by the extension of the Aimokatalok Lake and the outflow creek from Stickleback Lake. The footprint of the dry stack facility is in the shape of an irregular heptagon, with an area of about 19.8 hectares as shown in Figure 1.

The facility will have 5 m high intermediate benches with side slopes of 3H:1V. Setback benches of 5 m will result in an overall slope configuration of about 4.0H:1V and a final height of 25 m. A detailed cross section is shown in Figure 2.

2.2 Foundation materials

The foundation profile was assumed to consist of 7 m of frozen overburden soils (marine silt and clay) overlying competent bedrock. The upper 1 m of the overburden profile, immediately beneath the first bench, is assumed to be thawed. The average temperature in the dry-stack foundation is -4.0°C (see Section 2.3).

In absence of specific data within the footprint of the dry-stack, it is assumed that the foundation has an average pore water salinity of 37 parts per thousand (ppt) based on site wide salinity measurements (SRK 2017b).

2.3 Thermal Modeling

A thermal modelling for the Boston dry-stack was completed for the years 25, 50 and 85 after the end of construction (SRK 2017c). The results are included in Figures 3 to 5. The modeling predicts temperatures in the dry-stack foundation varying between -6°C and -4°C. For year 85 specifically, the modeling predicts temperatures between -4.6°C and -4.7°C. An average foundation temperature of -4°C at the year 50 was selected as representative for the frozen ice-rich marine silt and clays to perform the creep deformation analysis.

According to the results of the thermal modeling, it is also predicted that the tailings will freeze. However, tailings are not expected to experience creep since they are poor-ice materials.

2.4 Creep Deformation Evaluation Criteria

Creep deformation evaluation criteria for the frozen foundation establish limits to insure long-term integrity. 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 that the frozen foundation underneath the dry stack 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 $year^{-1}$).

3 Creep Deformations Analysis

3.1 Model description

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 typical cross section included in Figure 2. The thermal modelling outlined in Section 2.3 was completed for the same cross section.

Figure 6 presents the 2D finite difference model (FDM) of the typical cross section developed for the analysis with FLAC. The model considered four material regions: dry tailings, thawed foundation, frozen foundation and bedrock. The 1 m closure cover was not represented in the model.

3.2 Basis for the Assessment

Secondary creep (i.e., constant creep strain rate) were assumed for the frozen marine silt and clay in the foundation. This type of soil 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:

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

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

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

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 the frozen foundation 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 in triaxial condition in the laboratory (Arenson 2002). The assumed stress was a threshold for the deviatoric part of the stresses as introduced by Norton (1929); i.e., the deviatoric part of the stresses ($\bar{\sigma}$) in Equation (2) is reduced by σ_{th}, or $\dot{\varepsilon} = A(\bar{\sigma} - \sigma_{th})^n$. Likely thresholds for other creep mechanisms in frozen soil (e.g., temperature) were not considered in the analysis.

Equation (2) can 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:

$$\sigma_r = 0.323(1 - T)^2 \left(\frac{49.505 - S}{8.425 + S}\right) \tag{5}$$

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 the reference stress 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 calculated with Equation (4) for a temperature of -4°C (see Section 2,3) and the reported average salinity of 37 ppt (Section 2.2)

3.3 Methodology

The thermal conditions used in the creep analysis were predicted by the thermal modelling at the typical section. It is expected that the creep behavior of the frozen 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 fifty years after dam construction (Figure 4). This time interval is considered as representative for the long-term creep deformation in the Boston dry-stack.

The analysis followed the following steps:

1. Initial state: The initial stresses of the bedrock and foundation (frozen and thawed) was achieved in the FDM by using elastic properties for all materials and turning gravity on.
2. Elasto-plastic phase: bedrock and foundation (frozen and thawed) were changed from elastic to Mohr-Coulomb materials, and the FDM was brought again to equilibrium.
3. Construction phase with creep behavior: Five construction stages were simulated. Each stage was analyzed as follows:
   (a) The first lift was placed using the Mohr-Coulomb constitutive model for all the materials, and then the FDM was brought to equilibrium;
   (b) Temperature dependent elastic and creep properties were assigned to the frozen marine silt and clays and the FDM was allowed to deform for six months;
   (c) After six months, all materials were changed back to Mohr-Coulomb materials, the next lift was placed and the FDM was brought to equilibrium.
   (d) The placement of the remaining lifts followed the steps (b) and (c).
4. Creep phase after construction: At the end of dry-stack construction, elastic and creep properties were kept in the frozen soils and the FDM was allowed to deform for 80 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., EBA (2006)), published data in the literature, and engineering judgment.

3.4.1 Elastic Properties

Table presents the material elastic properties used for achieving the initial state in the FDM.

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

Geotechnical Unit	**Unit Weight** (kN/m^3)	**Elastic Modulus** (kPa)	**Poisson's Ratio** (-)
Tailings	17.5	1.0E+05[1]	0.30[1]
Thawed foundation	17.0	5.0E+03[1]	0.30[1]
Frozen foundation	17.0	1.5E+05[1]	0.30[1]
Bedrock	26.0	1.0E+08[2]	0.25[2]

Notes:
1. Source: Dry-Stack Stability Memo (SRK 2017d)
2. Source: Creep Deformation Memo North Dam (SRK 2017e)

3.4.2 Shear Strength Properties

The Mohr-Coulomb properties suitable for the elasto-plastic phase of the analysis are included in Table 2 based on SRK 2017b and SRK 2017e.

Table 2: Shear Strength Properties

Model Region	**Cohesion (**kN/m^2)	**Friction Angle** (º)
Tailings	0	40[1]
Thawed foundation	0	30[1]
Frozen foundation	112[1]	26[1]
Bedrock	1000[2]	0[2]

Notes:
1. Source: Stability Memo (SRK 2017d)
2. Source: Creep Deformation Memo North Dam (SRK 2017e)

3.4.3 Creep Parameters

The parameter n for the frozen marine silt and clay (Equation (2)) was estimated to be 3 based on published laboratory testing results from saline fine-grained soils (Nixon and Lem 1984 and Wijeweera and Joshi 1991). The temperature-dependent value A used in the analysis is 9.1E-06 kPa^{-3}year^{-1} and was calculated with equations (3) and (4) for a constant salinity of 37 ppt. For reference, SRK 2017e plotted equations (4) and (5) for different temperatures and salinities.

The temperature dependent elastic moduli of the frozen foundation soils required for the elastic strains was estimated to be 3.2E+04. Since the elastic part of creep is considered to be a constant volume process (undrained process), the analysis used a Poisson's ratio of 0.5 for the frozen foundation soils.

4 Results

4.1 Shear Strain Rates and Shear Strains

The results of the creep analysis in terms of maximum shear strains and shear strain rates are presented in Figures 7 and 8. The analysis predicts shear strain localization in the frozen marine silt and clay. The strain localization zone is almost planar. This surface can be considered as a likely failure surface if the material strength is mobilized along this surface. To assess the stability of the dry-stack, an limit equilibrium analysis was performed considering the strain localization surface as a potential failure surface. The results are included in Section 4.5.

In general, the predicted shear strain rates are very low in all zones of the model 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 rate is 2.4E-07 year^{-1} 80 years after the dry-stack completion (Figure 7). The maximum shear strain is 6.0E-01 m/m (60%) for the same period (Figures 8). Maximum shear strain and shear strains rates are predicted to occur in points within the shear localization zone (i.e., inside the frozen foundation).

The predicted creep strain rates in the frozen marine silt and clay in the foundation meets the design criteria for ductile material behavior (Section 2.4), while the shear strains *themselves* exceed the criteria. However, a ductile material behavior is expected because the maximum rate of shear strain is predicted to be very low (~ 2.4E-07 $year^{-1}$). 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 dry-stack weight. Maximum principal stresses differences of around 50 kPa are predicted to be almost constant in the frozen foundation, throughout the 80 years period of analysis after the end of the dry-stack construction. Figure 9 presents the principal stress difference at the year 80.

The predicted stress differences in the frozen foundation can be considered as low compared with the expected peak deviatoric stress of ice-rich frozen soils under typical triaxial conditions in the laboratory (Arenson 2002).

4.3 Shear stresses

Figure 10 includes the results of the shear stresses at year 80. In general, the shear stresses in the frozen marine silt and clay are predicted to be relatively low throughout the period of analysis compared with the expected shear strengths of ice-rich frozen soils (Arenson 2002).

4.4 Displacements

Figures 11 and 12 show the distribution of the vertical- and horizontal creep displacements predicted at year 80. Figures 13 and 14 present vertical- and horizontal displacement histories, respectively, of the control points 1 to 4 shown in Figure 6. At year 80, the maximum displacement and related velocity predicted at the control points are as follows:

- Maximum vertical displacement: 2.9 m
- Maximum average vertical velocity: 0.036 m/year
- Maximum horizontal displacement: 5.7 m
- Maximum average horizontal velocity: 0.071 m/year

4.5 Stability Assessment

Limit equilibrium back-analysis was performed to assess the effect of the long-term creep on the stability of the dry-stack. The strain localization surface described in Section 4.1 and presented in Figure 7 was considered as a potential failure surface. The analysis consisted in back calculate friction angle of the frozen marine silt and clay required to met the long-term stability criterion (FOS=1.5). The analysis considered that the strain rate dependent cohesion of the frozen marine silt and clay vanishes because of the very low predicted strain rates. The friction angle, on the other hand, was considered dependent on the ice content and independent of the strain rate.

In other words, the strength of the frozen marine silt and clays tends to the long-term strength when the strain rate tends to zero.

Based on the limit equilibrium analysis, the frozen marine silt and clay requires a friction angle of 8.6° to meet the long-term stability criterion. Figure 15 shows the result of the stability analysis.

5 Conclusions

Main conclusions from deformation assessment of the Boston dry-stack due to creep of the frozen marine silt and clays are as follows:

- The deformation analysis was completed using the ‘Two Component Power Law model’ implemented in FLAC. In absence of constitutive parameters of the frozen soil within the footprint of the Boston dry-stack, the analysis was performed with the best estimate creep parameter considering material type, temperature and salinity and using engineering judgment. No creep strains were predicted below a selected threshold stress of 30 kPa.

- A long-term ductile behavior is predicted for the frozen marine silt and clay. Creep shear strains in this layer will occur very slowly and remain below the strain rate for brittle failure mode.

- Shear strains are predicted to localize in the frozen marine silt and clay layer. Eighty years after dry-stack completion, high shear strains (~ 60%) with very low strain rates (~1.0E-07 year^{-1}) are predicted within the localization zone.

- A minimum friction angle of 8.6° is required to met the long-term stability criterion when the shear strain localization surface is considered a potential failure surface, and the cohesion vanishes because of the predicted very low strain rate. The designed dry-stack meet the long-term stability criterion since the friction angle of the frozen marine silt and clay is 26°.

- The maximum calculated vertical and horizontal displacements in the analyzed cross section are predicted to be 2.9 and 5.7 m, respectively, eighty years after the end of the dry-stack construction.

6 References

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SRK Consulting (Canada) Inc. 2017c. Hope Bay Project, Boston Tailings Management Area Thermal Modelling. Memo prepared for TMAC Resources Inc. Project No.: 1CT022.013

SRK Consulting (Canada) Inc. 2017d. Hope Bay Project, Boston Tailings Management Area Stability Analysis. Memo prepared for TMAC Resources Inc. Project No.: 1CT022.013

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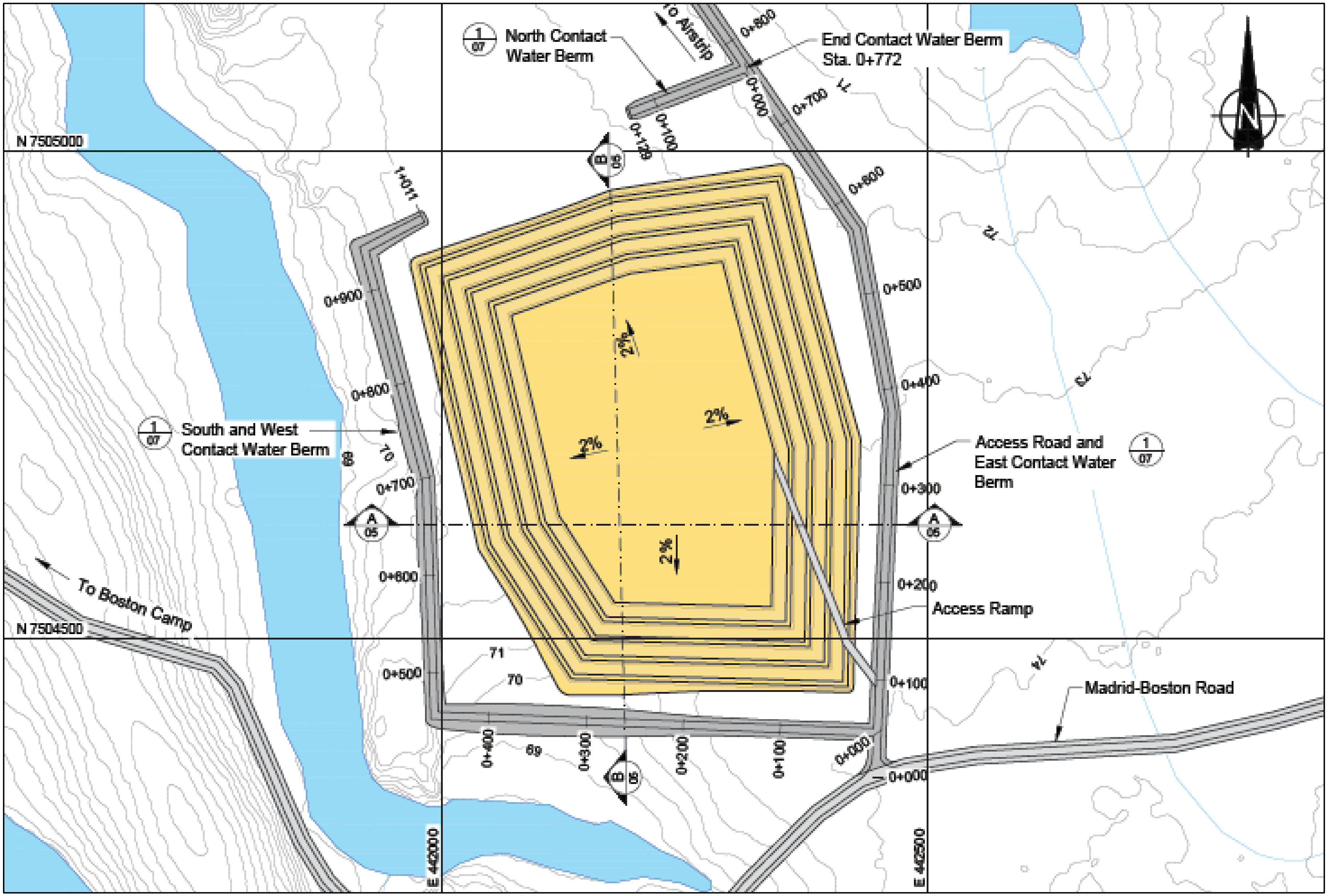
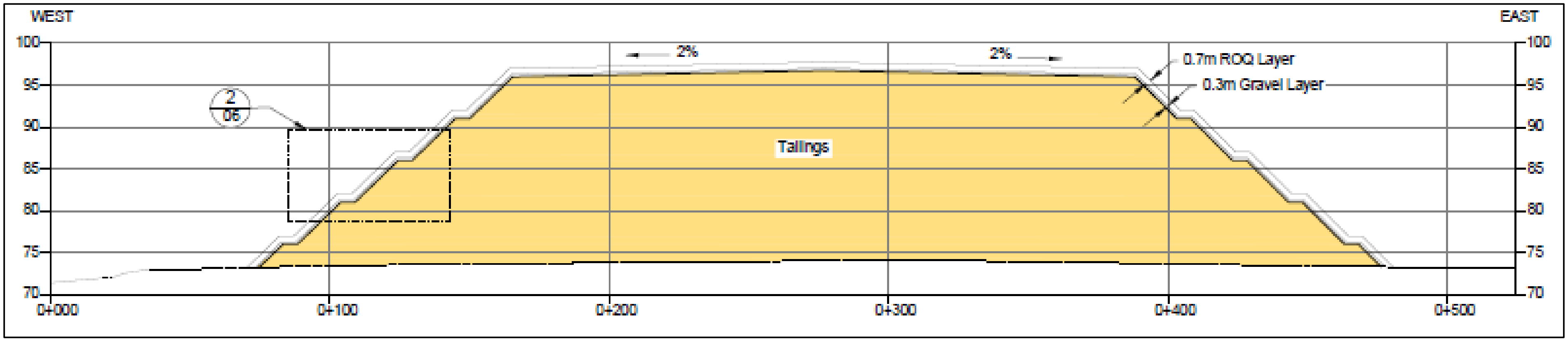
Figures

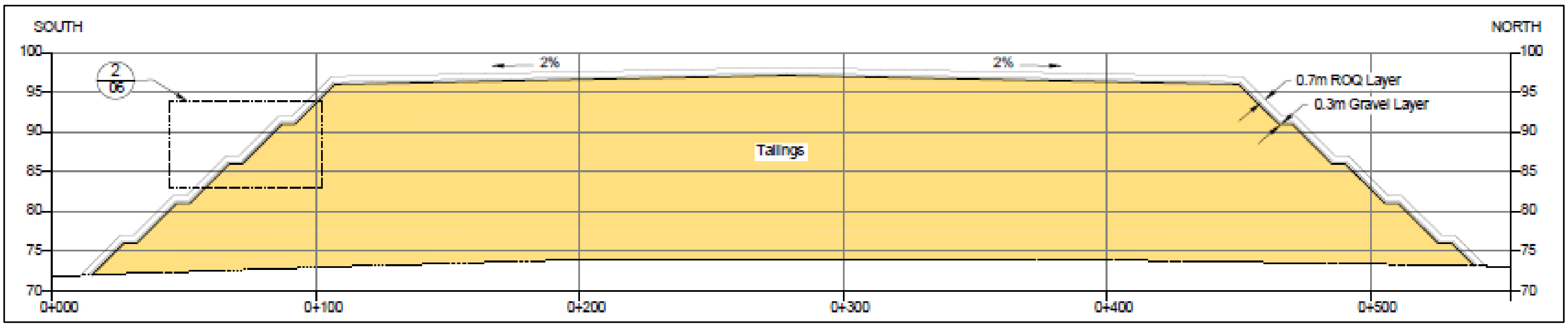


srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Boston TMA General Arrangement
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 | Approved: EL | Figure: 1

Section A-A



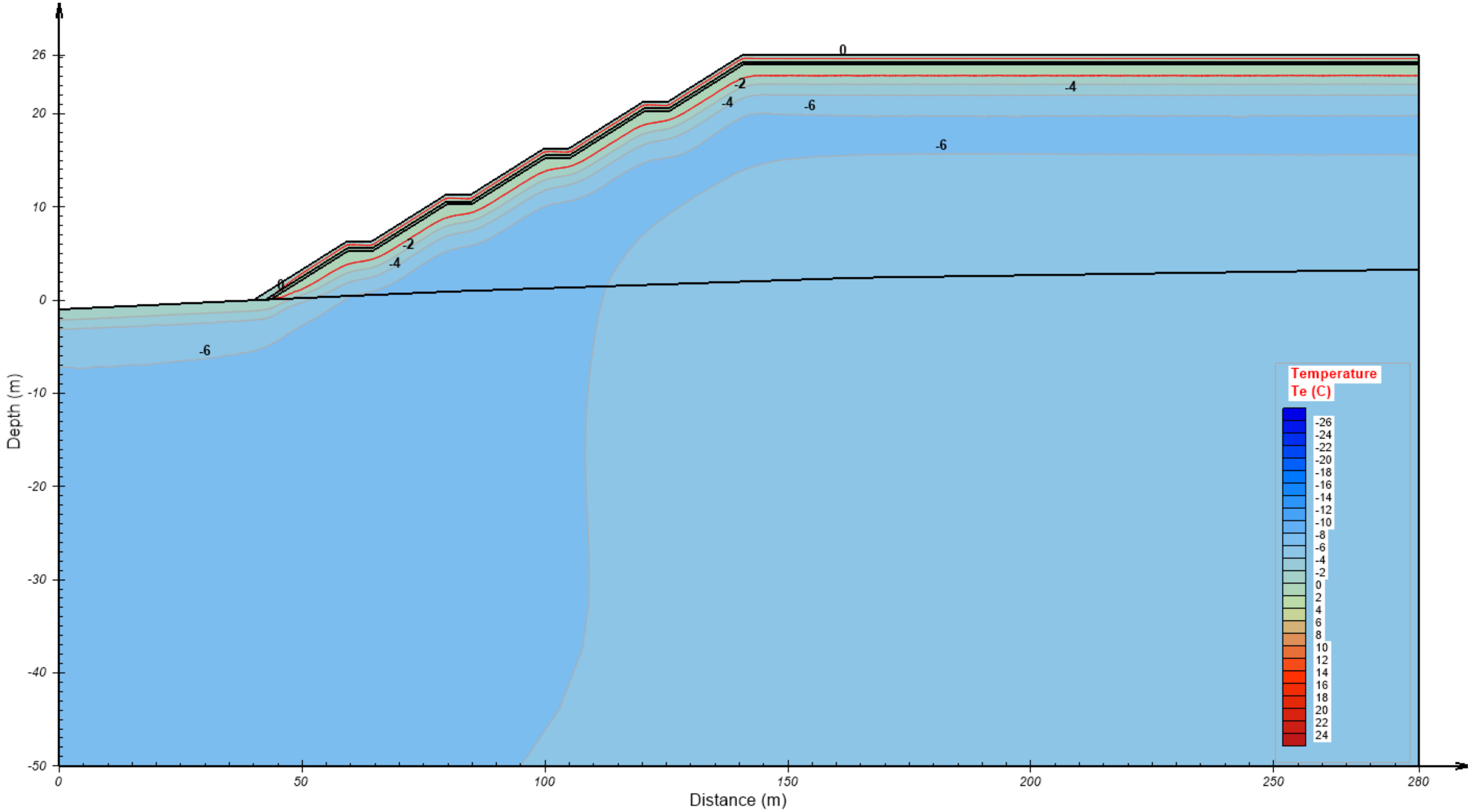
Section B-B



Notes:

1. Sections have 3x Vertical Exaggeration

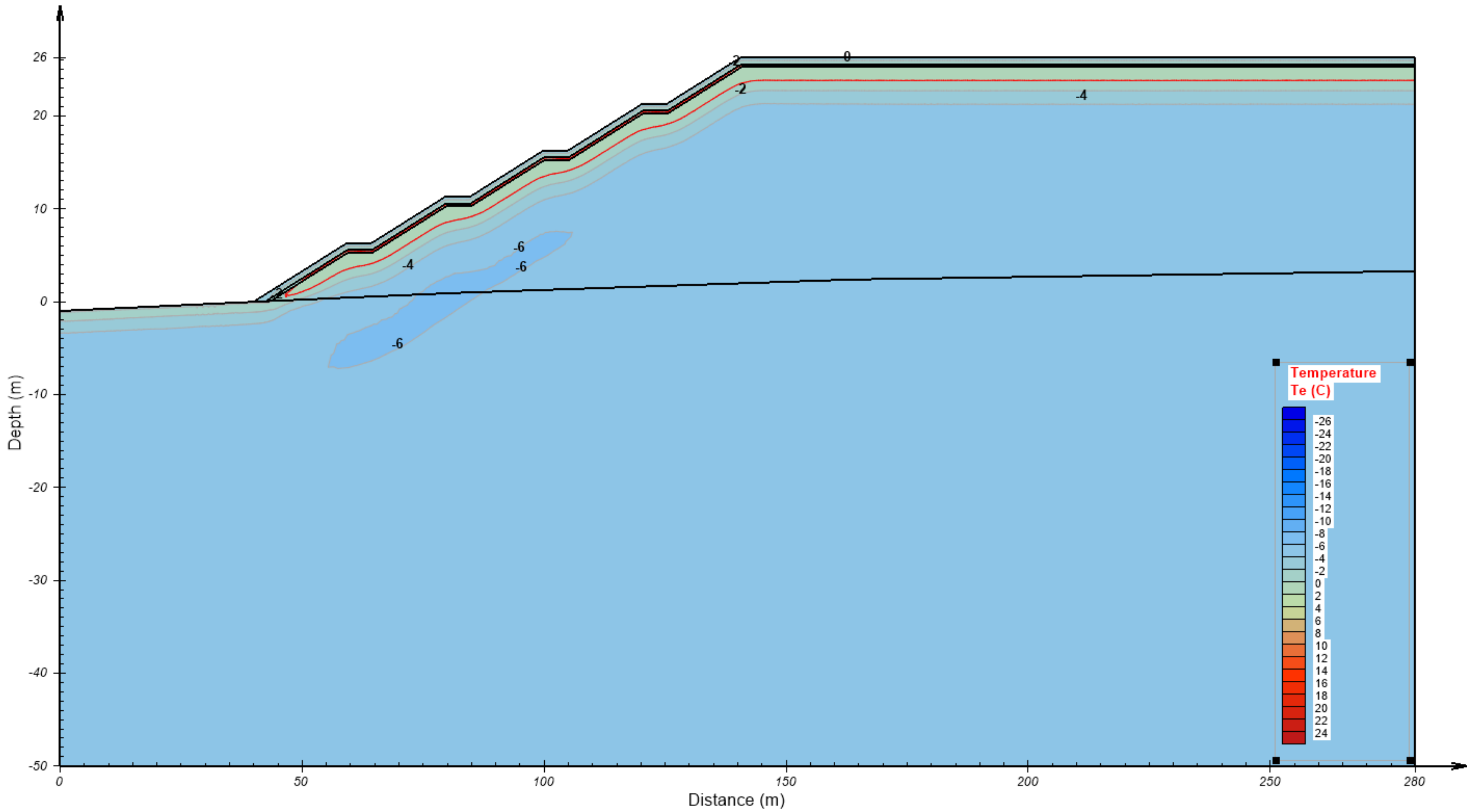
srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Boston TMA Typical section
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 2



Notes:

1. Model section represents maximum position of -2°C isotherm (solid red line) during Year 25

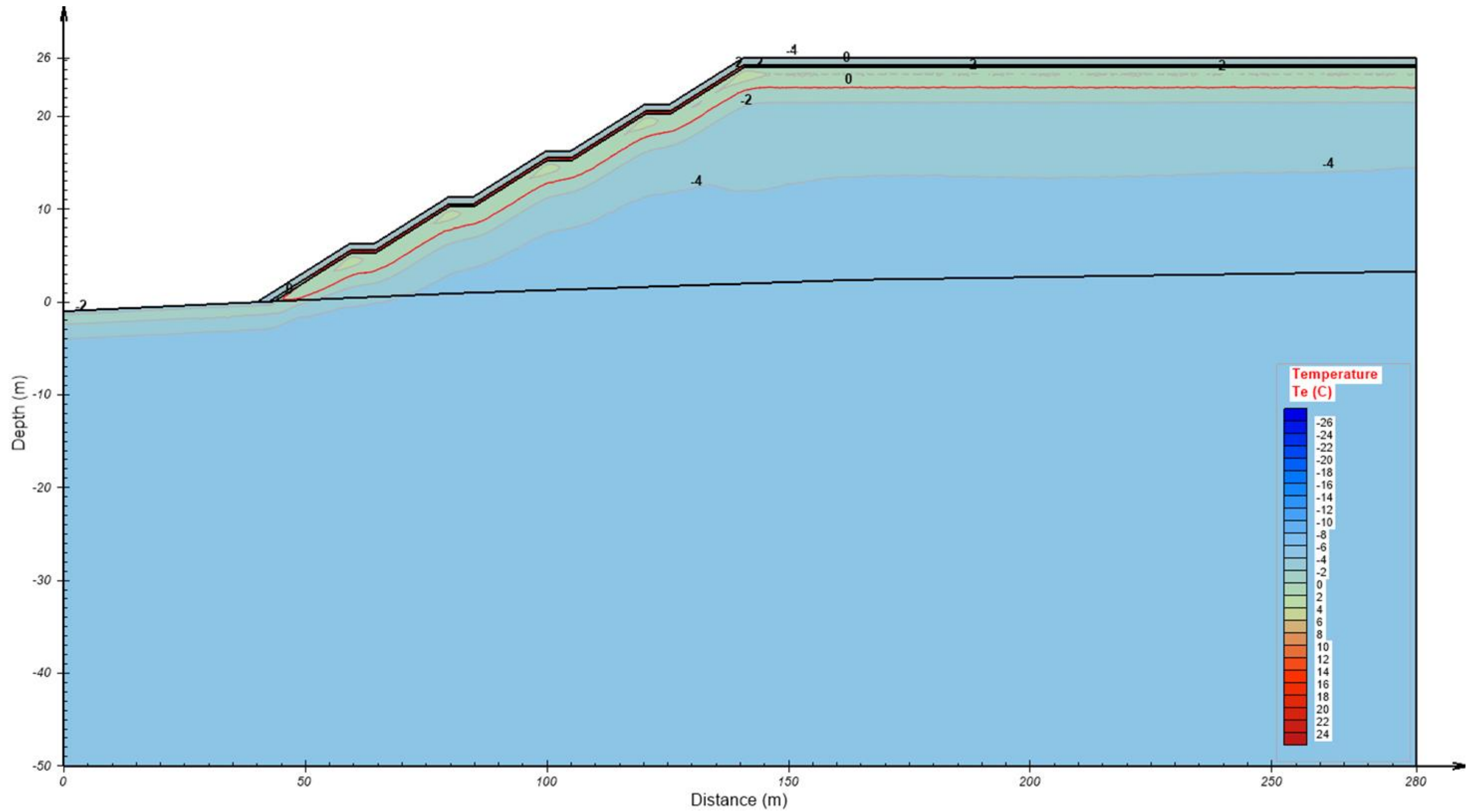
srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Thermal Model Results – Boston TMA (Year 25)
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: **3**



Notes:

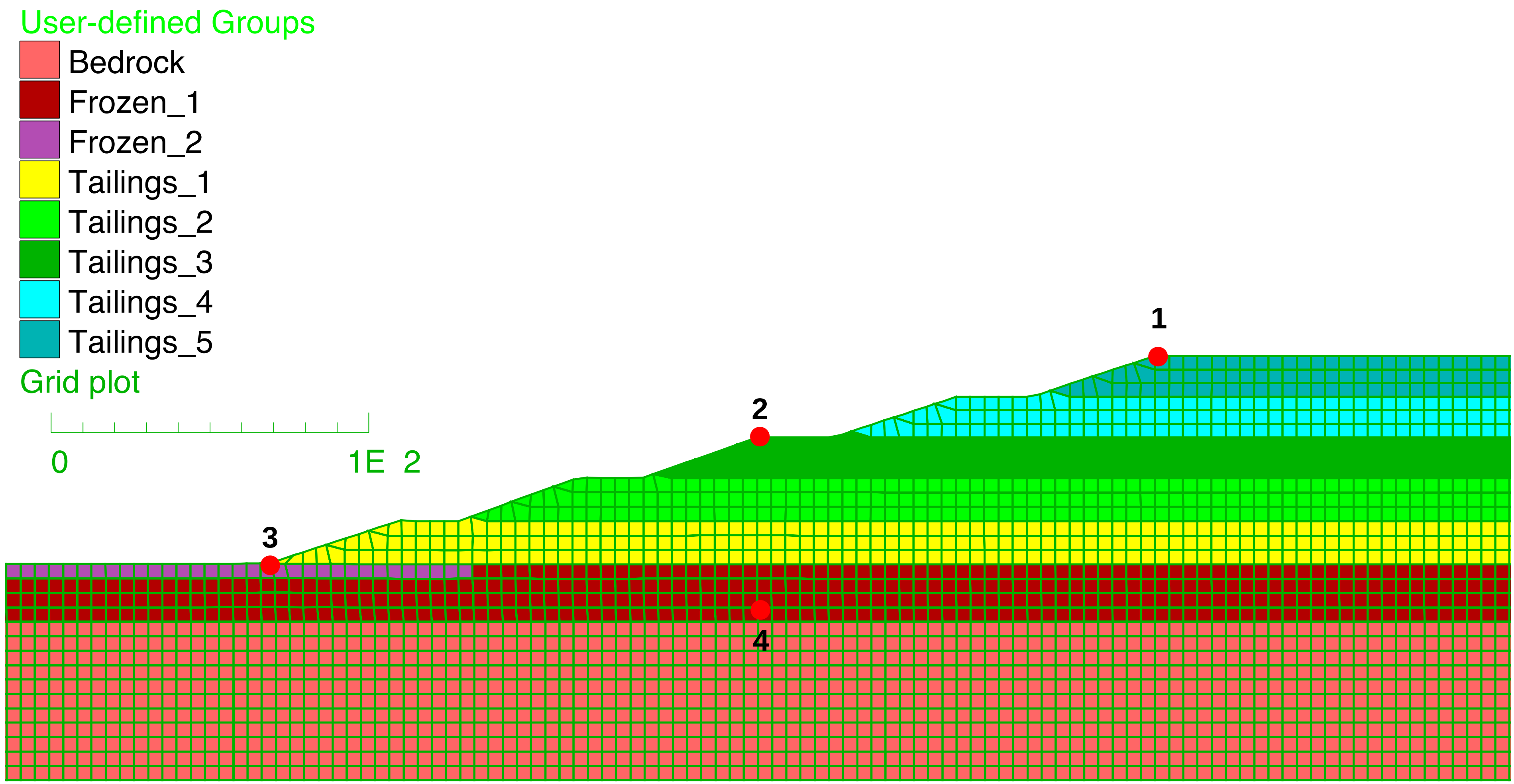
1. Model section represents maximum position of -2°C isotherm (solid red line) during Year 50

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Thermal Model Results – Boston TMA (Year 50)
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 4



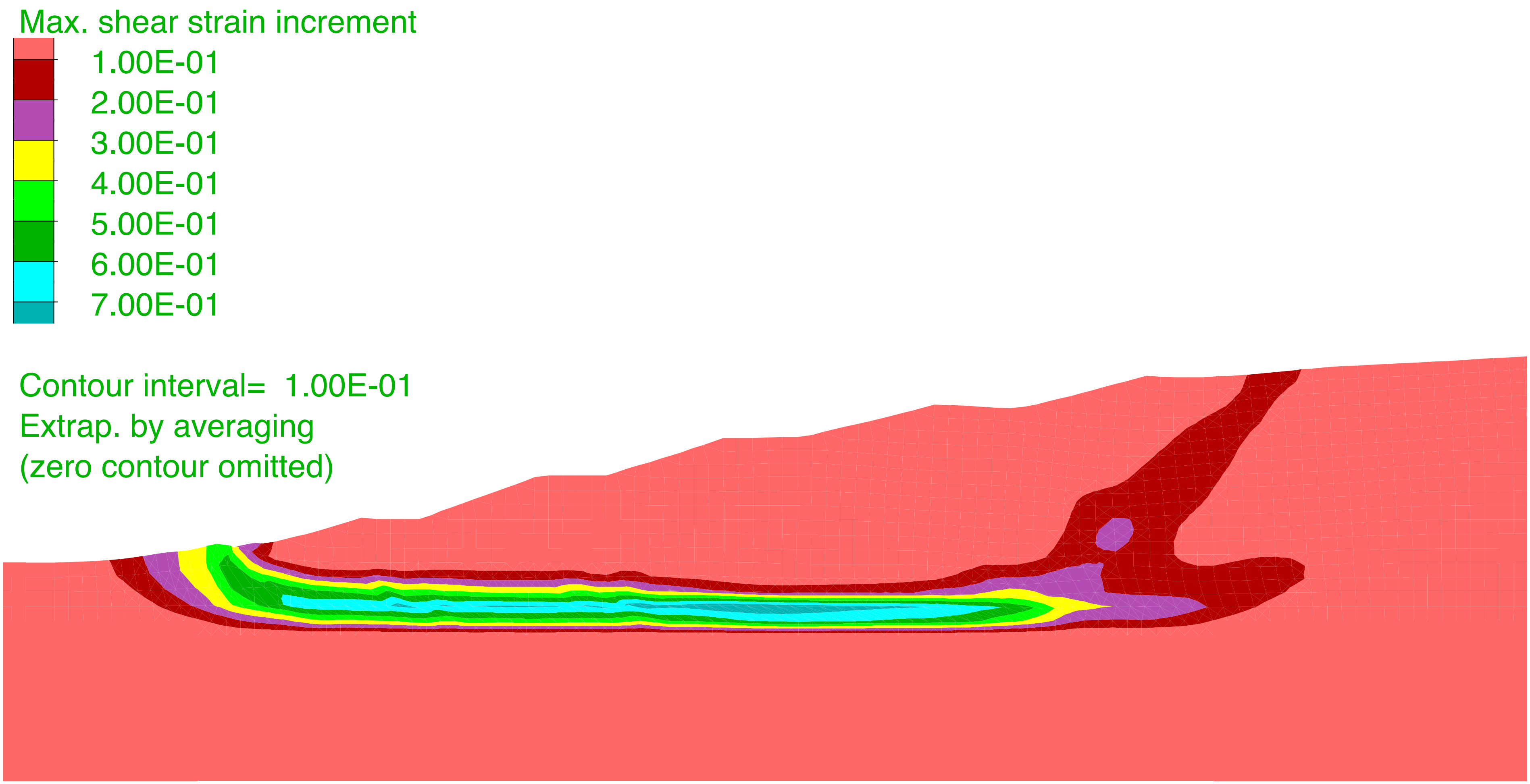
Notes:
1. Model section represents maximum position of -2°C isotherm (solid red line) during Year 85

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis **Thermal Model Results – Boston TMA (Year 85)**
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 5



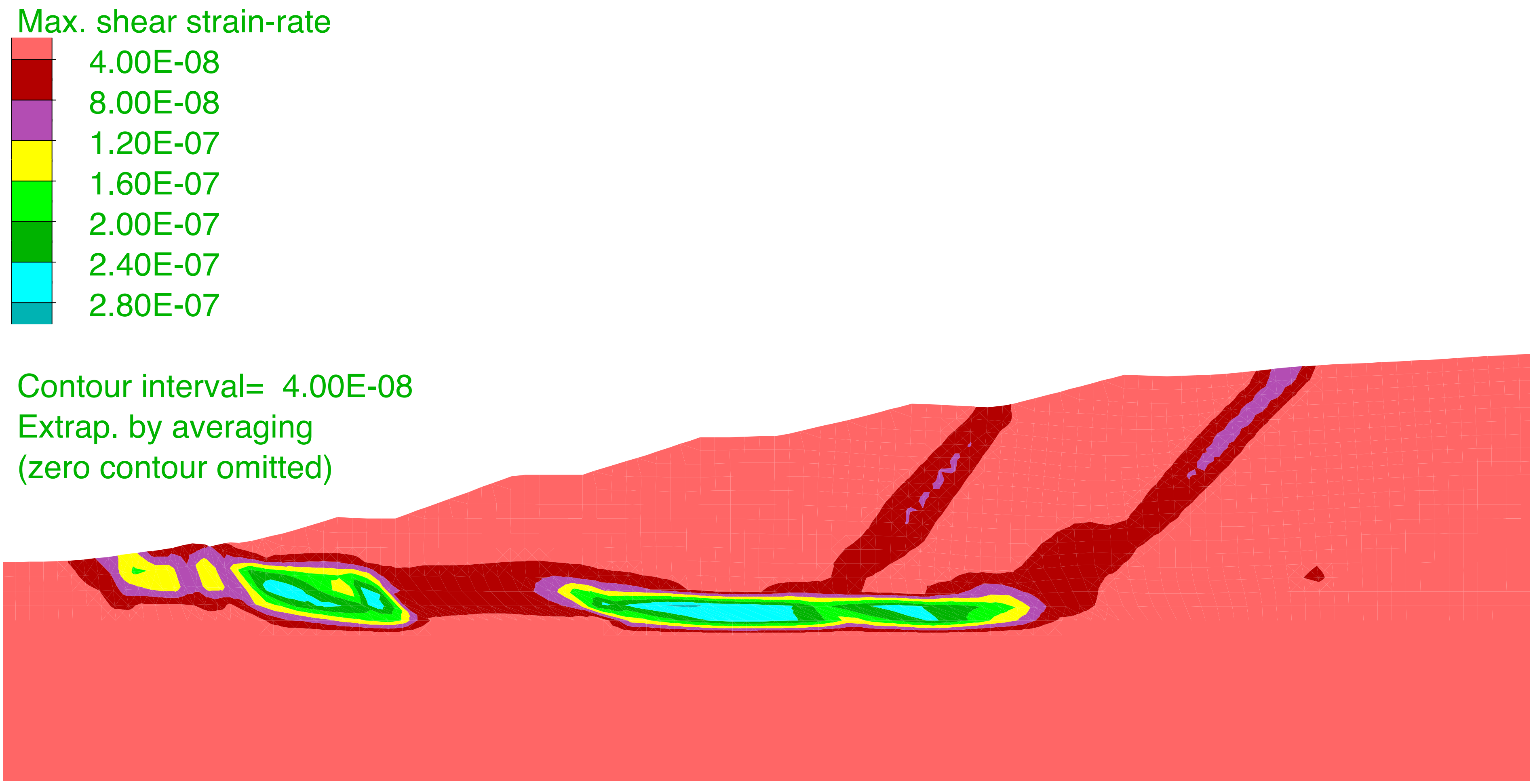
Notes:
1. Bedrock is included in the Figure
2. Points 1, 2, 3 and 4 are displacements points
3. The group ‘Frozen 2’ represents the thawed foundation.

srk consulting	MAC RESOURCES	Boston TMA Creep Deformation Analysis **Model Set Up – Typical Section**
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 6



Notes:
1. Units in meters/meters
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Maximum Shear Strain 80 Years After Dam Construction
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 7



Notes:
1. Units in year^{-1}
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

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TMAC RESOURCES

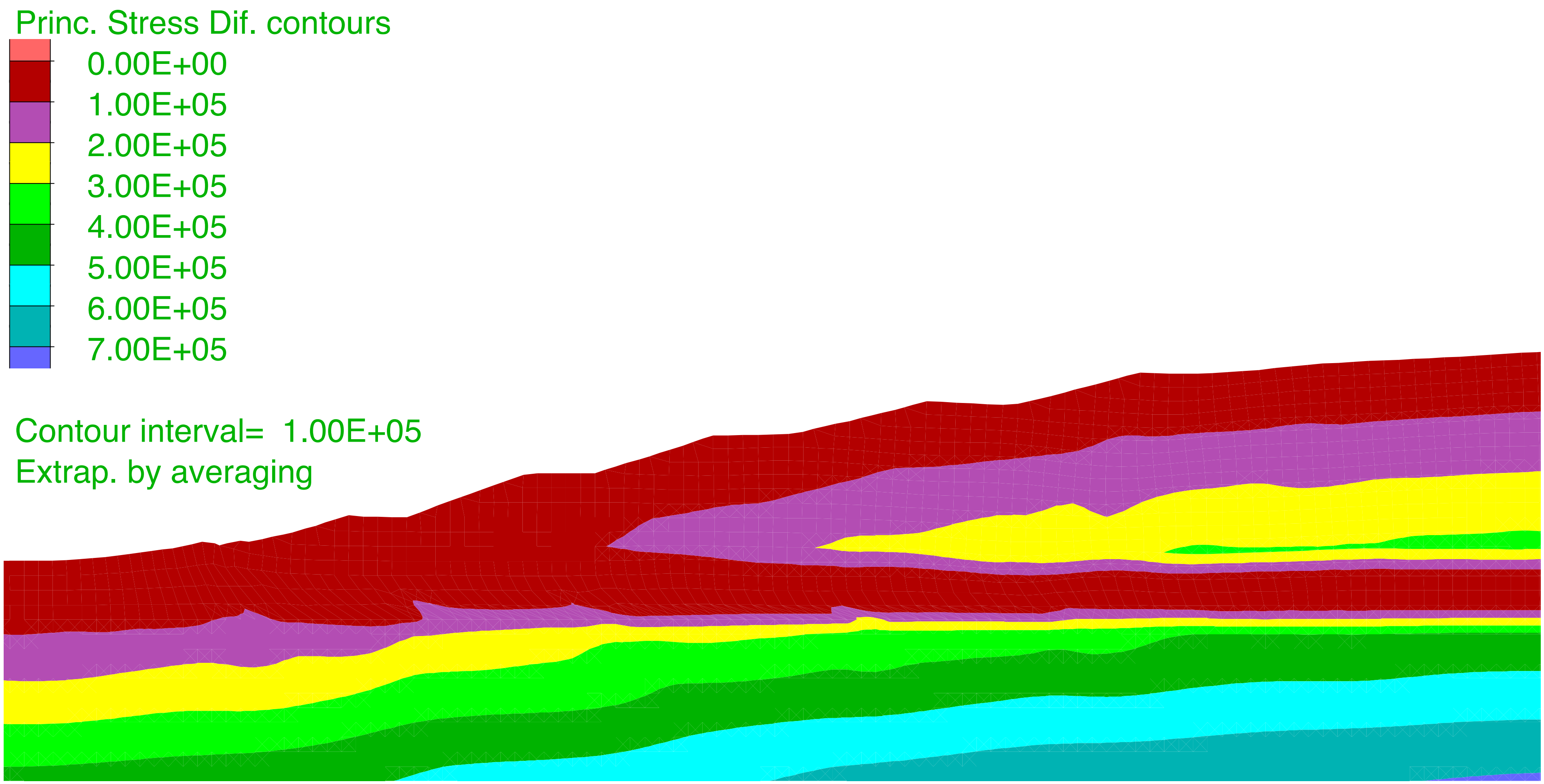
Boston TMA Creep Deformation Analysis

Shear Strain Rate
80 Years After Dam Construction

Job No: 1CT022.013
Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx

HOPE BAY PROJECT

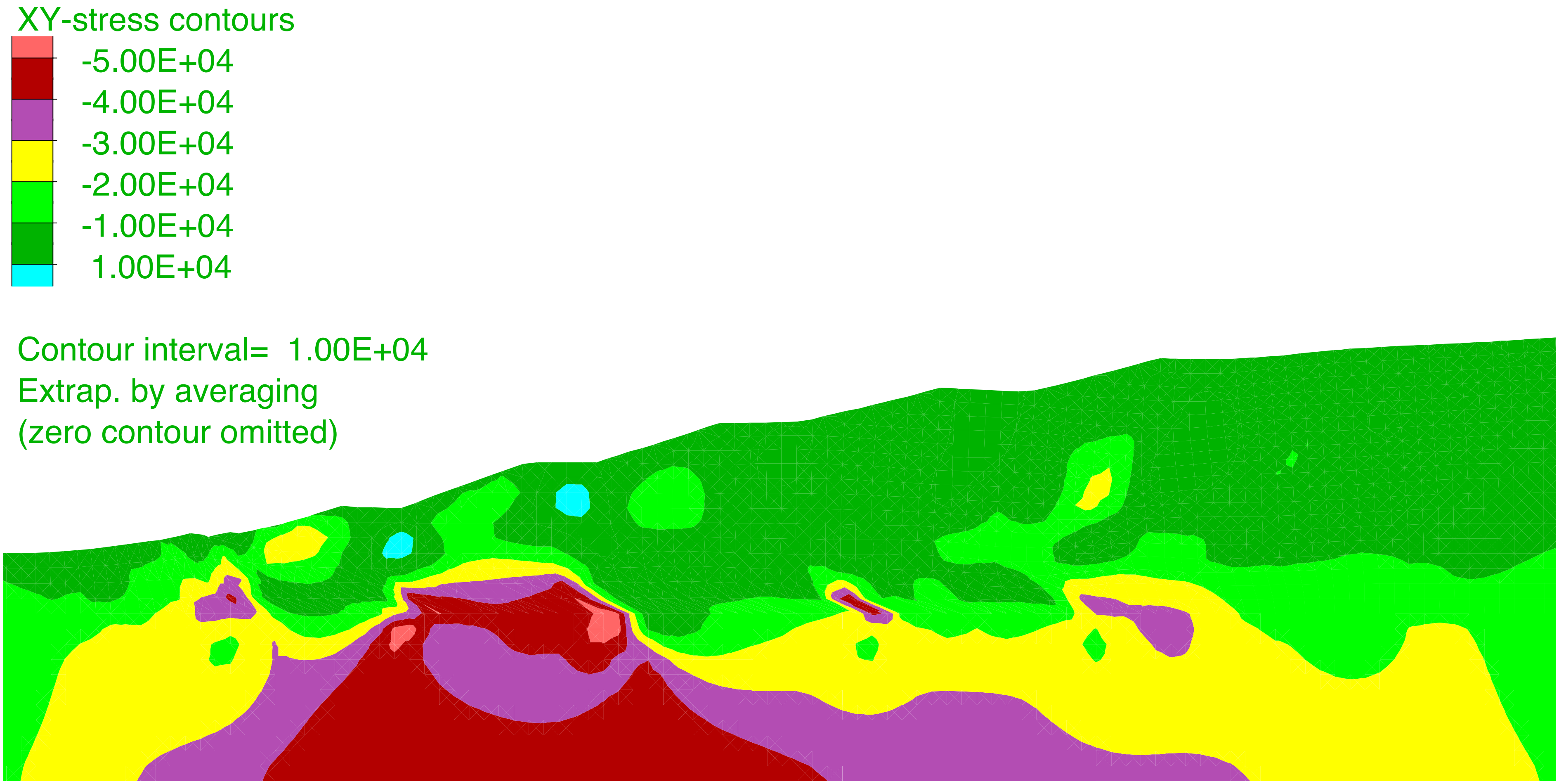
Date: November 2017
Approved: EL
Figure: 8



Notes:

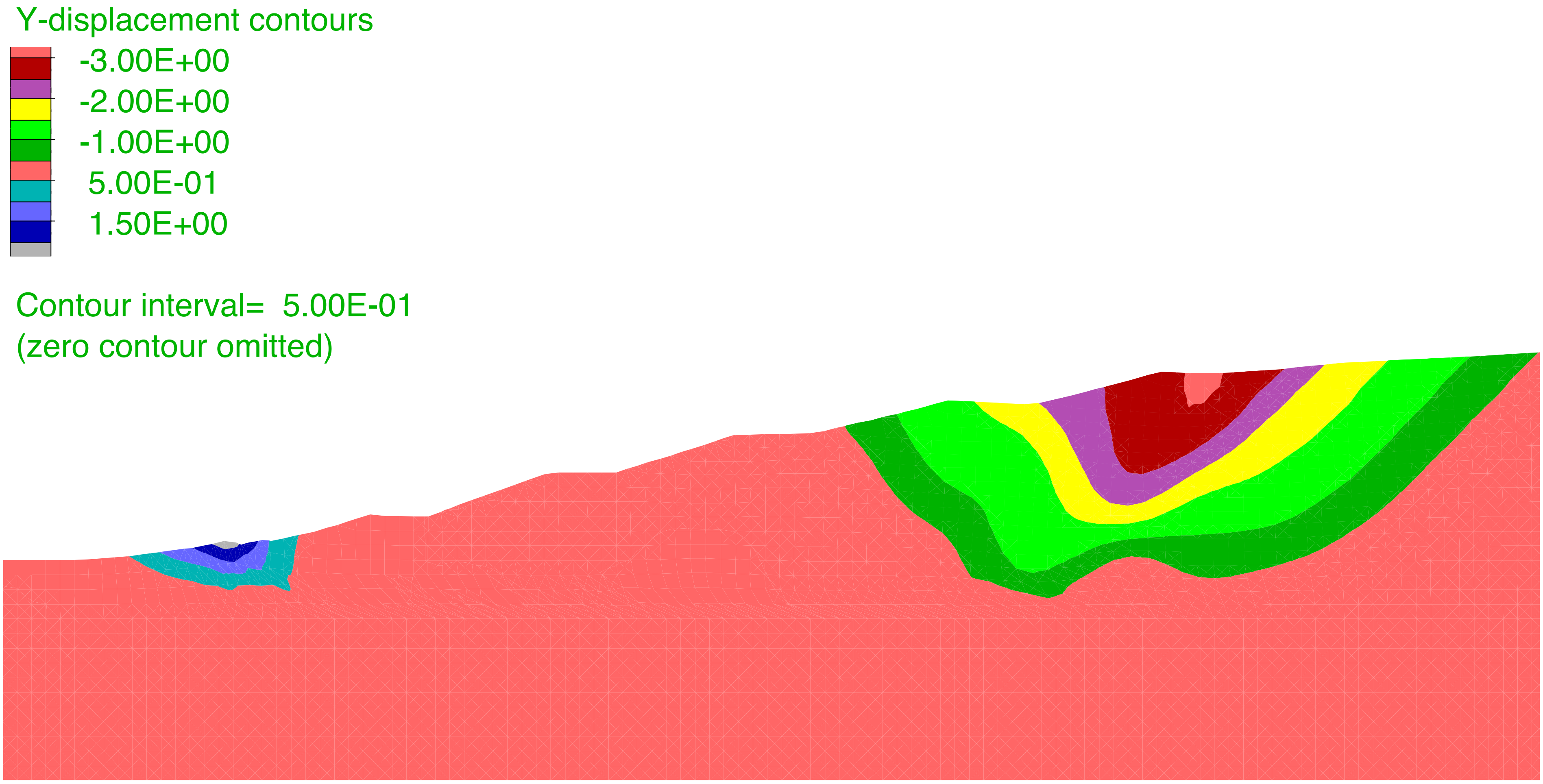
1. Units in Pascals
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis **Principal Stress Difference** **80 years After Dam Construction**
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: **9**



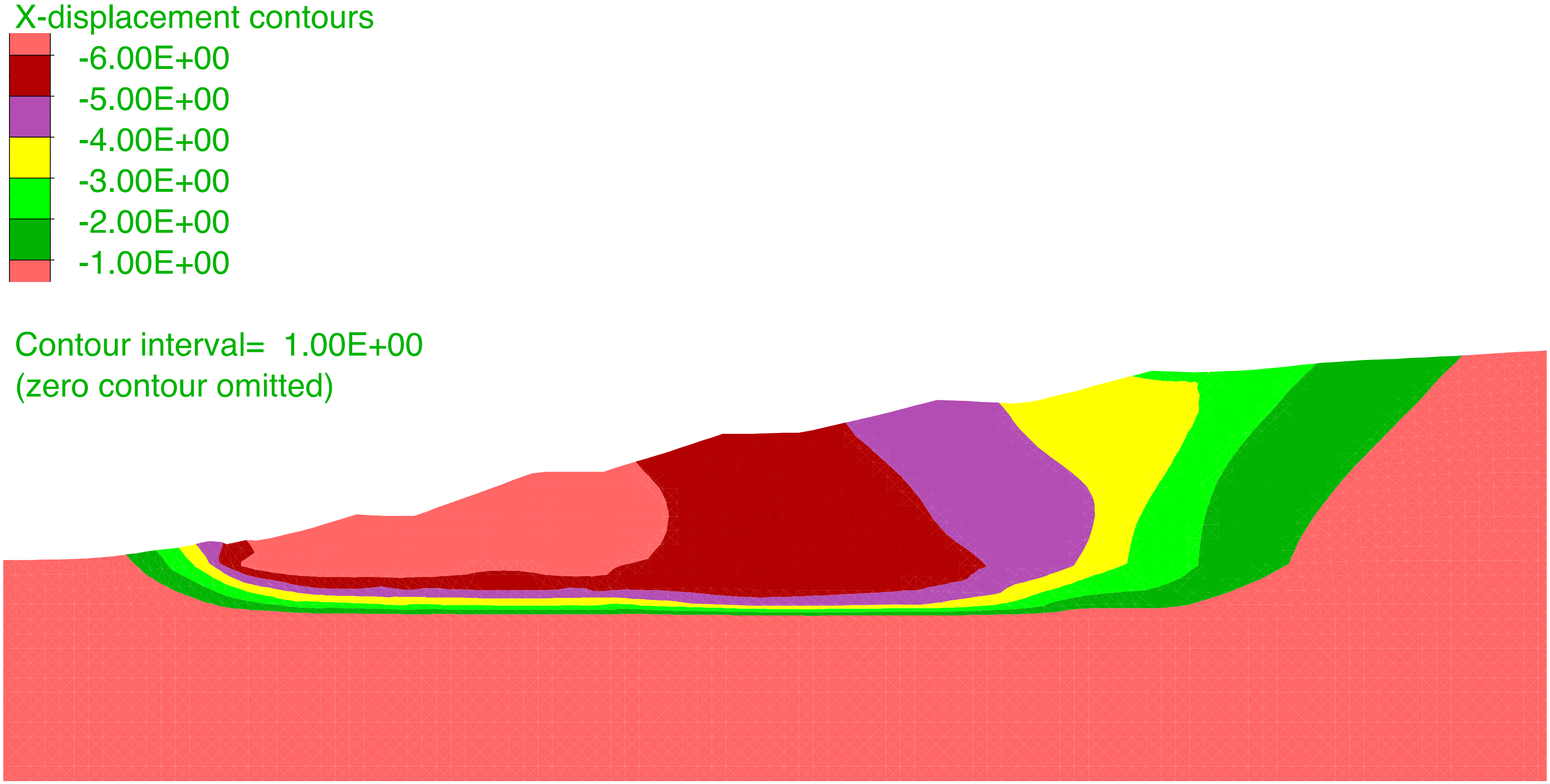
Notes:
1. Units in Pascals
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Shear Stresses 80 years After Dam Construction
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: **10**



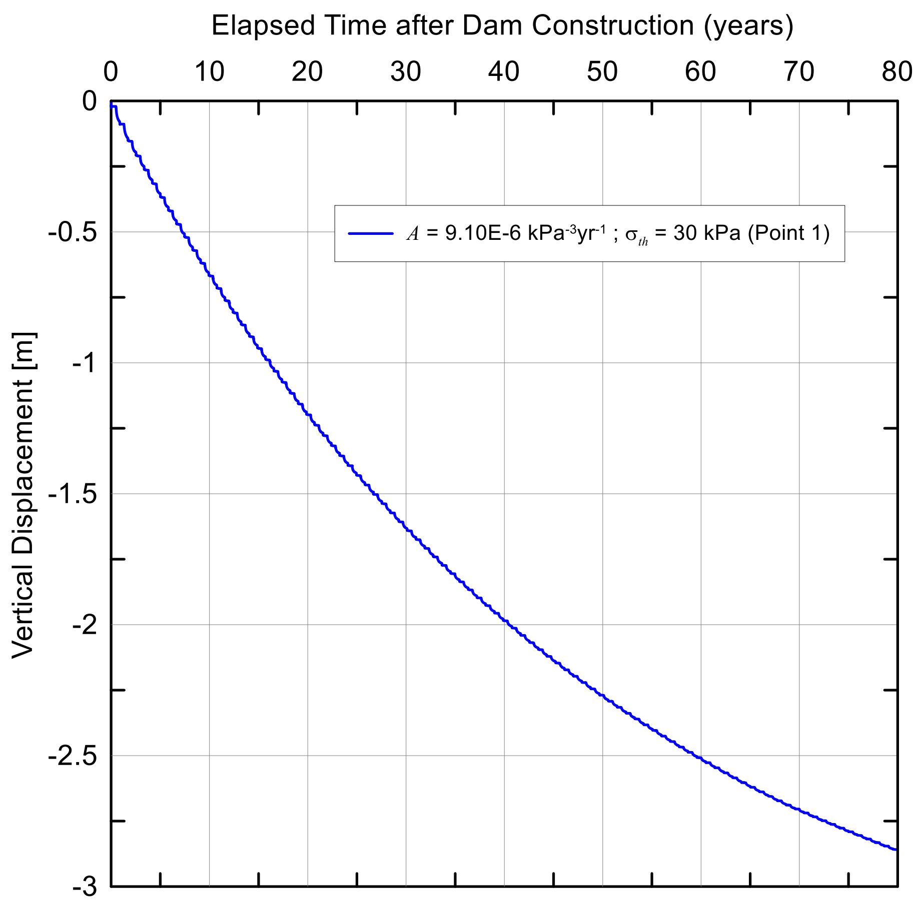
Notes:
1. Units in meters
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

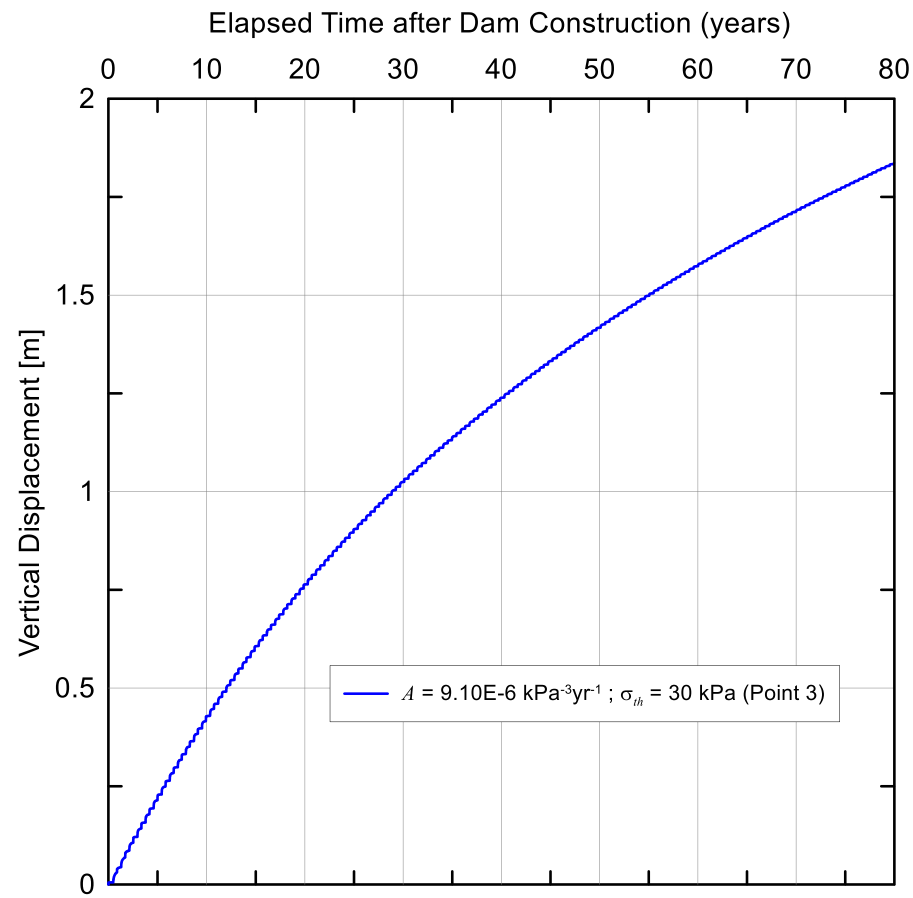
srk consulting	MAC RESOURCES	Boston TMA Creep Deformation Analysis
		Vertical Displacements 80 years After Dam Construction
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 / Approved: EL / Figure: 11



Notes:
1. Units in meters
2. Bedrock is included in the Figure
3. Results for a salinity of 37 ppt in the frozen foundation and a threshold stress of σ_{th} =30 kPa

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Horizontal Displacements 80 years After Dam Construction
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017	Approved: EL	Figure: 12

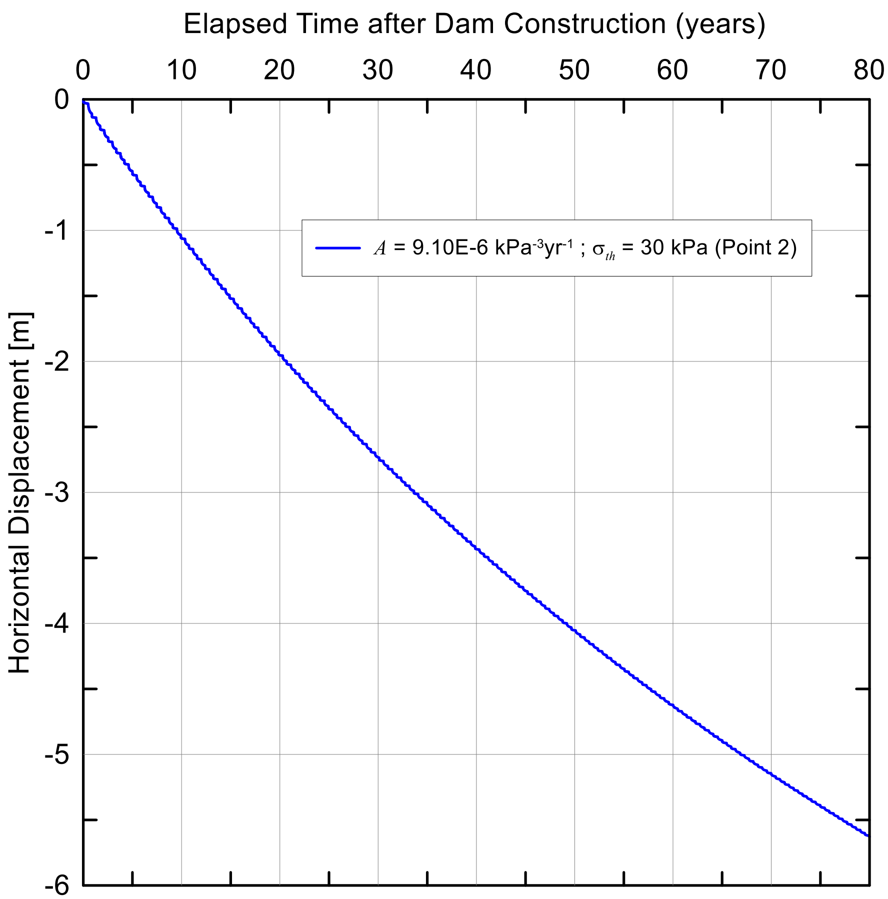


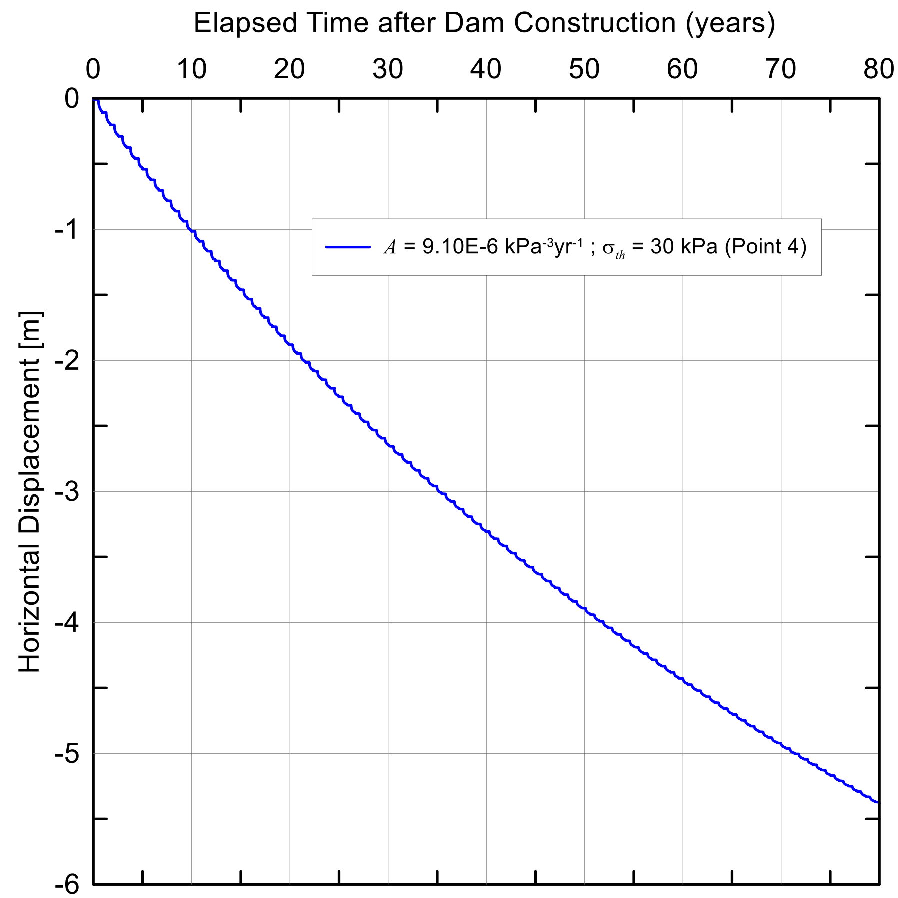


Notes:

1. See Figure 6 for reference (Points 1 and 3)

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Vertical Displacements History Points 1 and 3
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 · Approved: EL · Figure: **13**

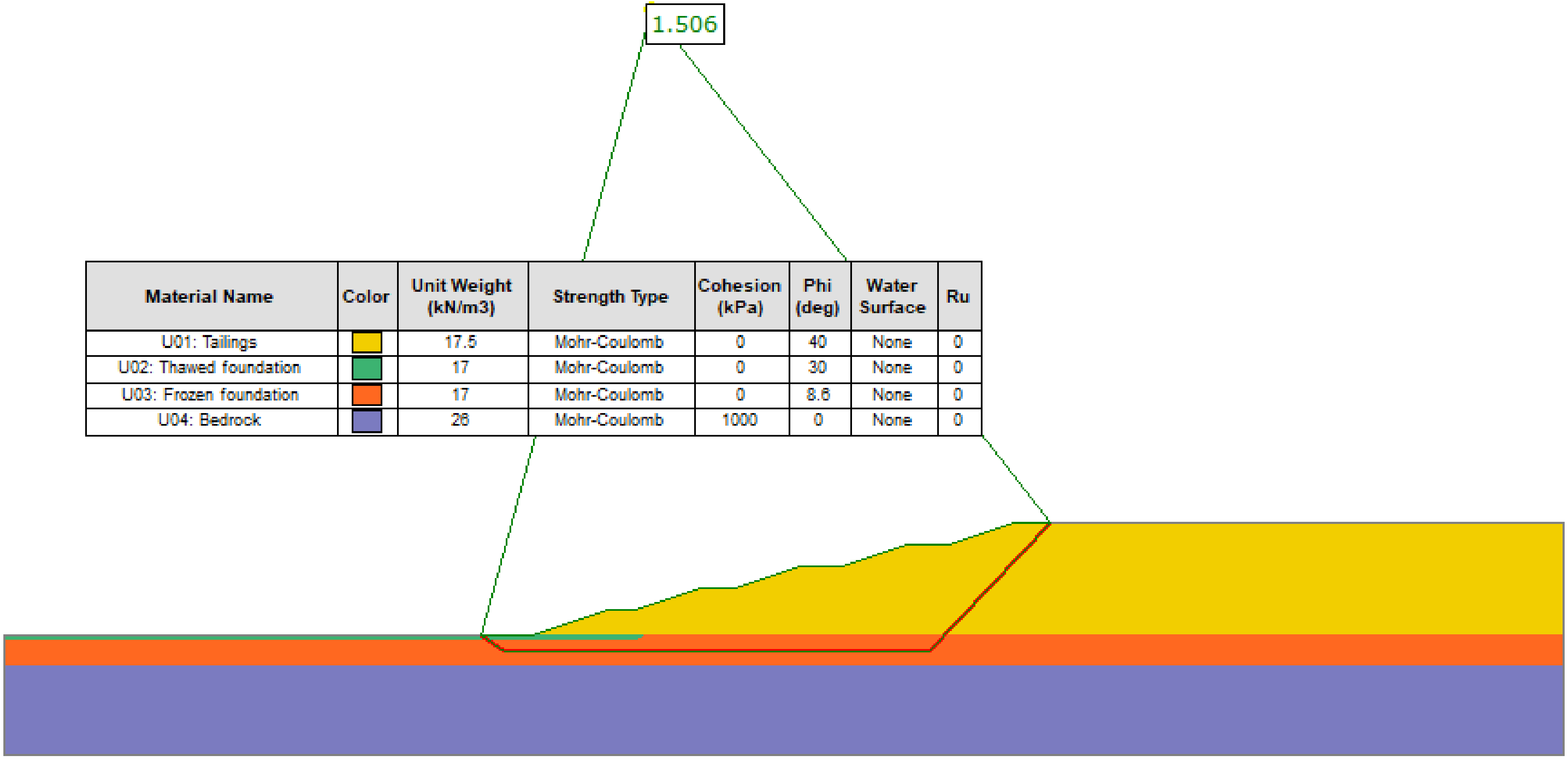




Notes:

1. See Figure 6 for reference (Points 2 and 4)

srk consulting	TMAC RESOURCES	Boston TMA Creep Deformation Analysis
		Horizontal Displacements History Points 2 and 4
Job No: 1CT022.013 Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx	HOPE BAY PROJECT	Date: November 2017 — Approved: EL — Figure: **14**



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
U01: Tailings		17.5	Mohr-Coulomb	0	40	None	0
U02: Thawed foundation		17	Mohr-Coulomb	0	30	None	0
U03: Frozen foundation		17	Mohr-Coulomb	0	8.6	None	0
U04: Bedrock		26	Mohr-Coulomb	1000	0	None	0

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TMAC RESOURCES

Boston TMA Creep Deformation Analysis

Stability Analysis

Job No: 1CT022.013

Filename: Boston_DSTSF_CreepDeformationAnalysis_Memo_1CT022-013_Rev05.pptx

HOPE BAY PROJECT

Date: November 2017

Approved: EL

Figure: 15

Appendix G – Hope Bay Project: Tailings Area Dust Control Strategy

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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, 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 Boston TMA		

Change Log

The following table provides an overview of material changes to this report from the previous version issued as Appendix V3-2F, 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

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 Phase 2. The existing Doris tailings impoundment area (TIA) will be expanded, and a new Boston tailings management area (TMA) will be developed comprised of filtered tailings developed as a dry-stack. This memo is addressing dust management strategies for the Boston TMA facility.

Two tailings streams will be produced; flotation tailings, comprising approximately 94% of the overall volume, and detoxified leach tailings (following cyanidation, and subsequent cyanide destruction), comprising about 6% of the overall volume. Only flotation tailings will be deposited in the Boston TMA. The detoxified leach tailings will be filtered, mixed with mine waste rock and used for underground mine backfill.

The dry stack within the Boston TMA will be closed by construction of a low permeability cover incorporating a geosynthetic liner. The liner will be protected by a 0.3 m thick layer of gravel overlain by 0.7 m of ROQ as final erosion layer. The cover could be constructed in stages, as each 5 m high bench is completed, or at the end of the operations. In any case, the top surface of the facility will be exposed throughout operations, being the active stacking site, while the side slopes may be exposed for various time periods depending on the chosen closure schedule.

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 memorandum 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. Tailings in the Boston TMA will have relatively low moisture content (but still wet), allowing the surface to dry quickly.

Fugitive dust from the tailings surface could be generated when equipment and personnel operate on, or travel across areas where the surface layer of the deposited tailings has dried out. This activity is expected because of standard operating and maintenance activities, as well as routine safety inspections. Additional fugitive tailings dust will 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. The Boston facility will be a dry stack so the tailings will be filtered before being stacked at moisture contents near optimal. This means that the surface will under the right conditions rapidly dry. As a result, at any given time, most of the outer Boston tailings surface will be exposed dry tailings. Aeolian tailings dust is therefore 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 because 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 packed by machinery. 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.

No tailings should be stacked over any areas of compacted snow. If the compacted snow does not melt during the subsequent summer season due to the insulating blanket of the overlying tailings, then an outer ice lenses may form over the tailings TMA dry stack surface. This may lead to possible instability and settlement issues as well as occupy space that is not accounted for.

There is sufficient snowfall at the Project site that this dust control method could be effectively used. In addition, there is a requirement on site for snow removal in specific areas. Snow that is removed could be hauled to the TMA and used specifically for 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 complementary dust control method.

3.2.2 Ice Cover

An ice cover would work for more conventional slurry tailings deposition, but as the dry stack will not have any free water, a full ice cover forming naturally over the TMA surface cannot occur and therefore this strategy is not deemed practical.

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 mill or can be hauled via water truck from the site contact water ponds. As a contingency option water could be pumped or trucked from Aimaokatalok Lake. The tailings surface is however too steep for wheeled trucks 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 in an unsaturated state in an above-grade dry stack facility.

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. To facilitate placement of a final dust cover as expediently as possible, any tailings stacking plan should be designed taking into consideration all opportunities for progressive reclamation.

The Boston TMA will be constructed by placing the tailings in thin layers, i.e. “stacking”. The top surface of any given layer becomes the operational base of the subsequent layer, thus it is not amenable to intermediate dust covers. The side slopes could be covered under a progressive reclamation scenario, but the joining of the subsequent sections of geosynthetic membrane becomes challenging in a staged approach like this.

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 stacking might resume at any location may be extensive. When tailings stacking eventually returns to the covered area, these materials are not removed and tailings stacking proceeds to overtop the sacrificial cover. This however can be very cost intensive and will take up valuable storage space in the facility.

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 TMA.

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 Hope Bay 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 TMA. 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 the Project’s prevalent wind directions to provide a comprehensive dust management system for the TMA.

Given the very high wind speeds and the multiple wind directions, experienced at the Project’s TMA, engineered wind barriers are not be considered a viable dust control strategy for the Project.

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 minus 10°C, and can be spread in a thin layer over exposed frozen tailings surfaces during the shoulder seasons. 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 unlined contact water ponds which may result in vegetation die-back, permafrost degradation, and additional environmental concerns. This is therefore not a viable dust control strategy for the Project TMA.

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 TMA. 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 Boston TMA

The primary dust control measures of the Boston TMA 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 dry stack construction progresses 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 areas 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.