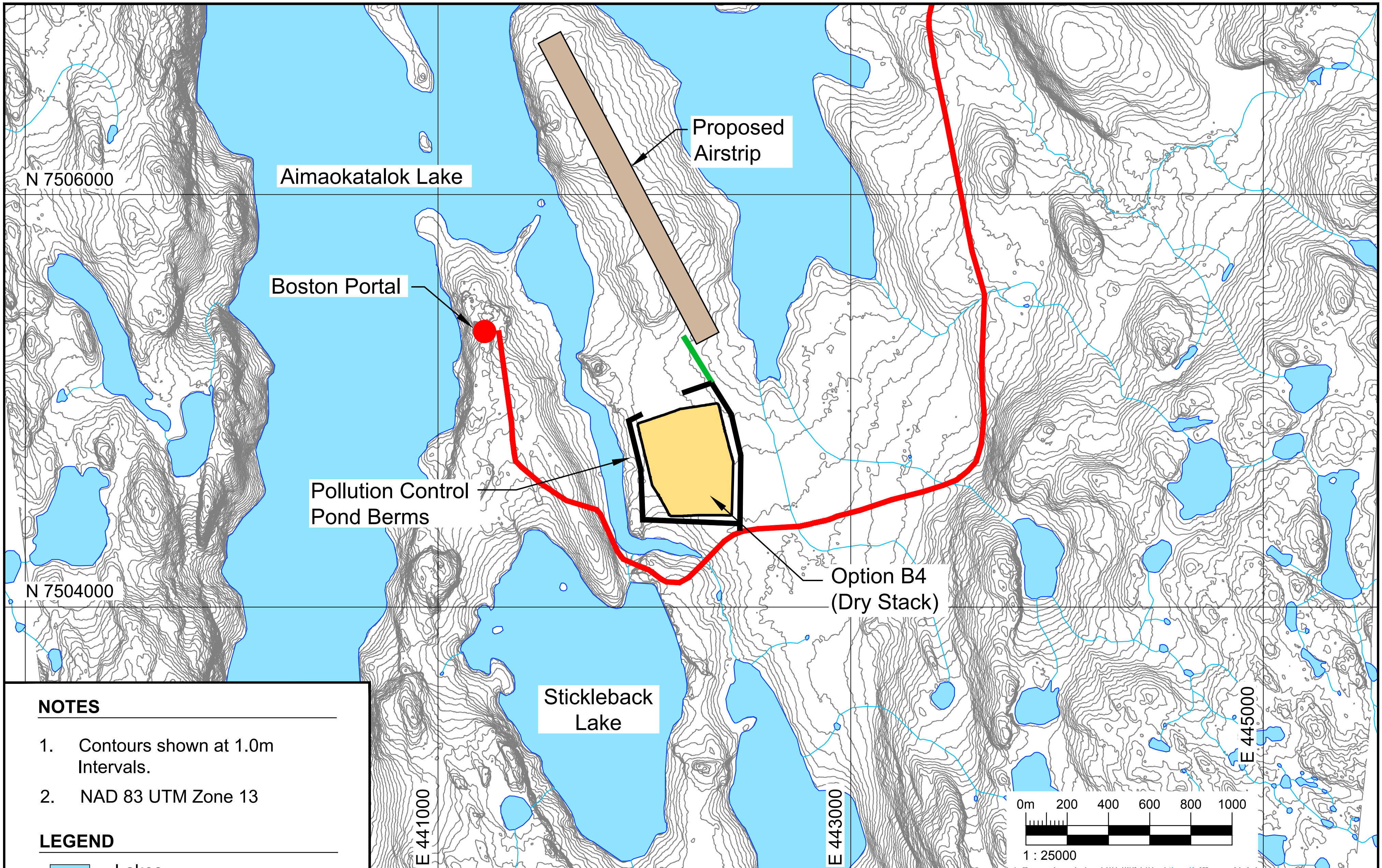

NOTES

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

LEGEND

- Lakes
- Tailings Area
- Creeks and Rivers
- Proposed Access Road
- Madrid-Boston Road

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TMAC RESOURCES

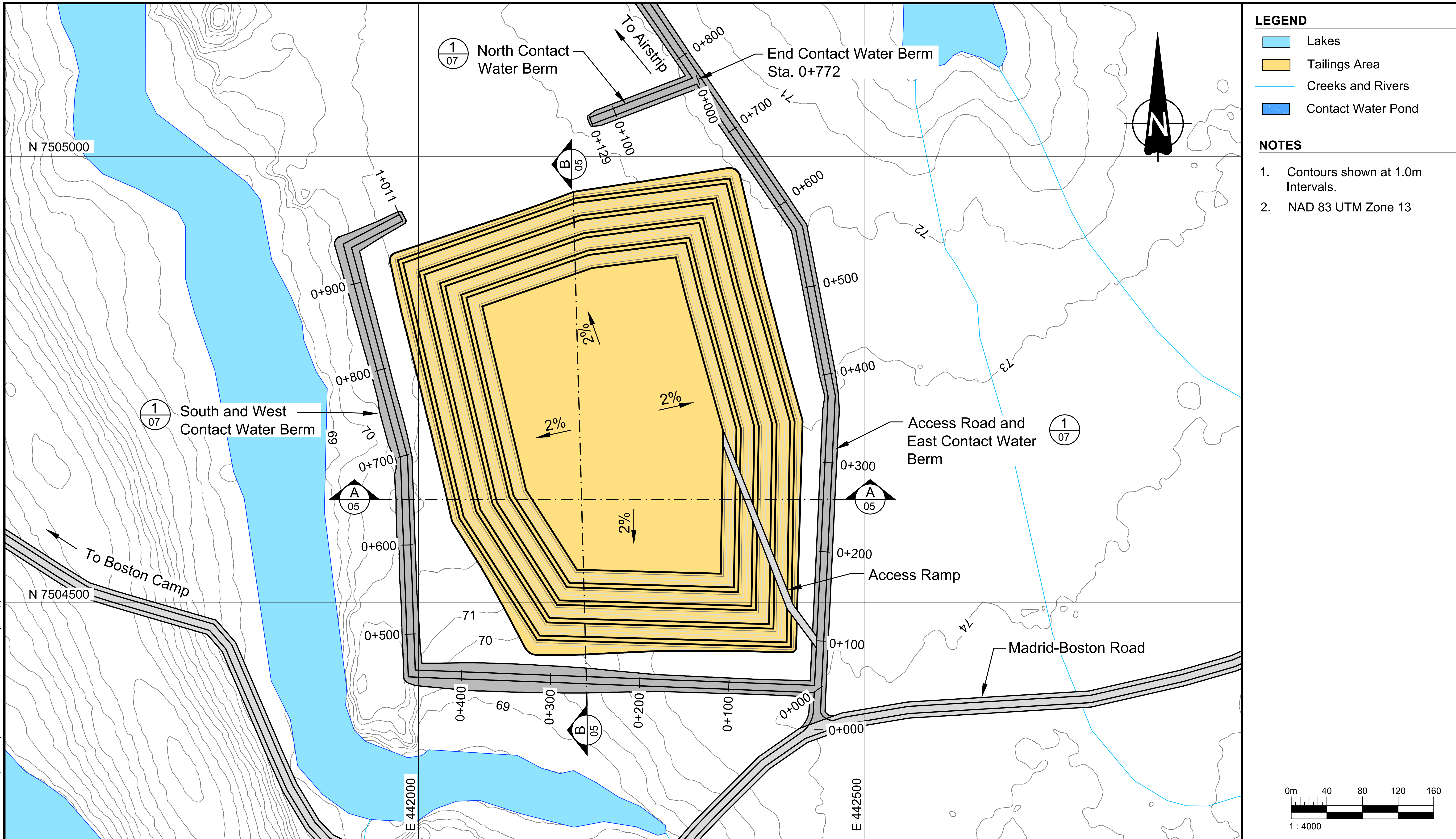
HOPE BAY PROJECT

Boston TMA Preliminary Design

General Site Plan

SRK JOB NO.:	1CT022.004.600.20
FILE NAME:	1CT022.004T600.20-FIGURE 3 - Drystack.dwg

DATE:	APPROVED:	FIGURE:
Nov. 2016	IM	03



Boston TMA Preliminary Design

Drystack Plan View

srk consulting

MAC RESOURCES

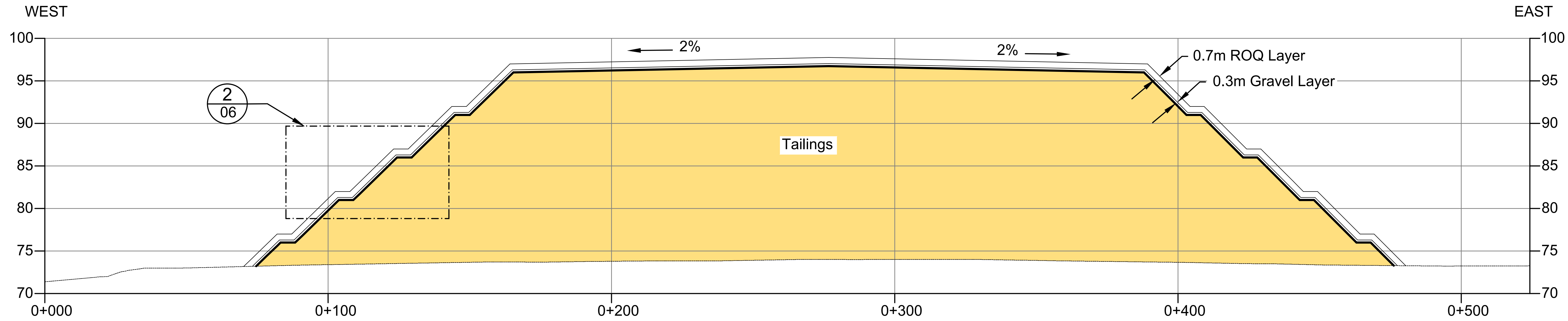
HOPE BAY PROJECT

SRK JOB NO.: 1CT022.004.600.20

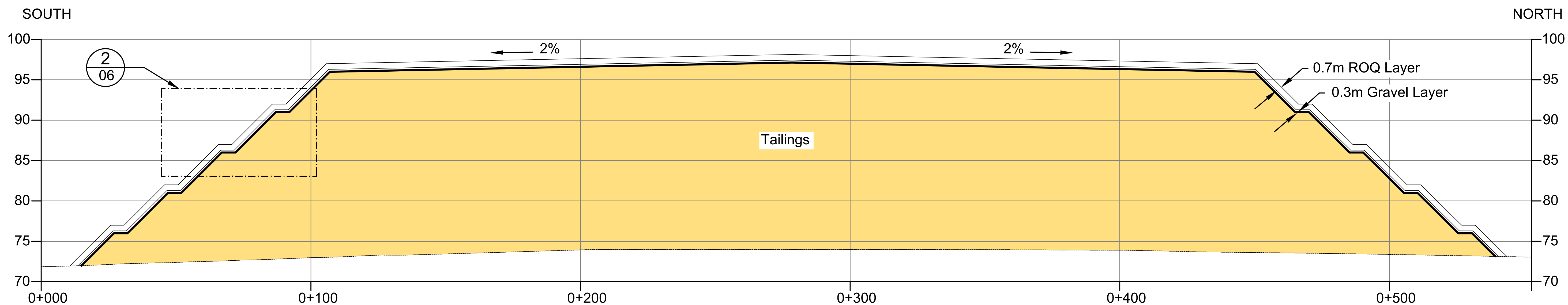
FILE NAME: 1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg

DATE: NOV. 2016 | APPROVED: IM | FIGURE: 4

C:\MDSINC\CLIENTS\SRK\2016\1CT022.004.600.20 BOSTON DRYSTACK\1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg



AA
04
SECTION
3x Vertical Exaggeration



BB
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SECTION
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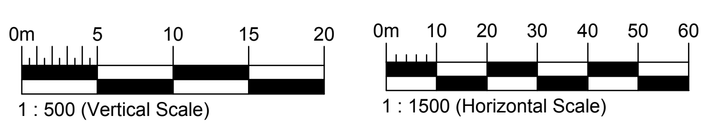


srk consulting

SRK JOB NO.: 1CT022.004.600.20

FILE NAME: 1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg

TMAC RESOURCES

HOPE BAY PROJECT

Boston TMA Preliminary Design

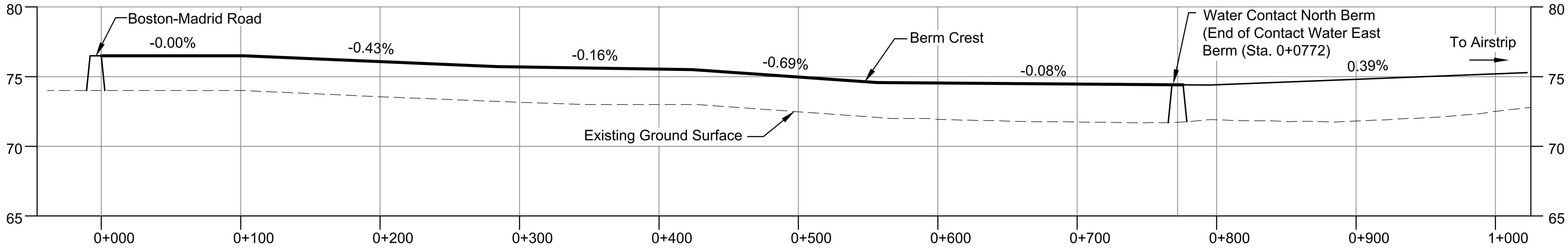
Sections AA and BB

DATE: NOV. 2016

APPROVED: IM

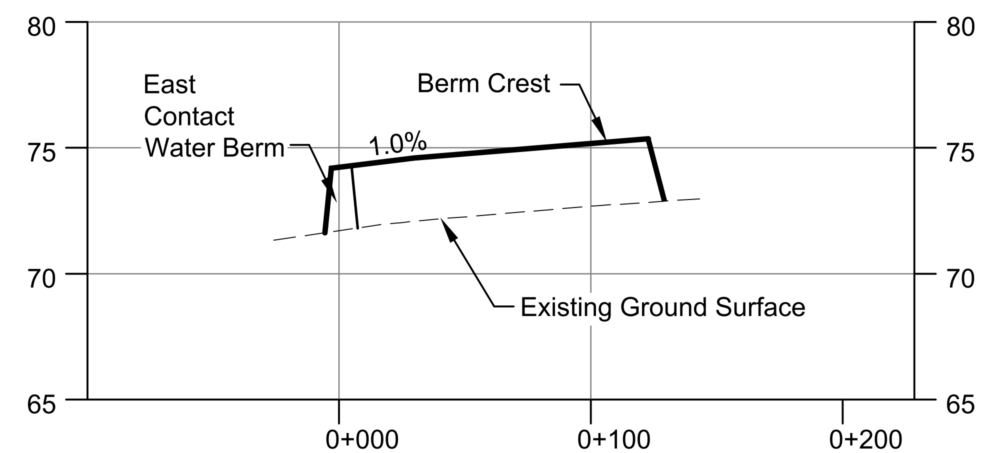
FIGURE: 5

C:\MDDSINC\CLIENTS\SRK\2016\1CT022.004.600.20 BOSTON DRYSTACK\1CT022.004T600.20-FIGURE 4-7 – Drystack.dwg



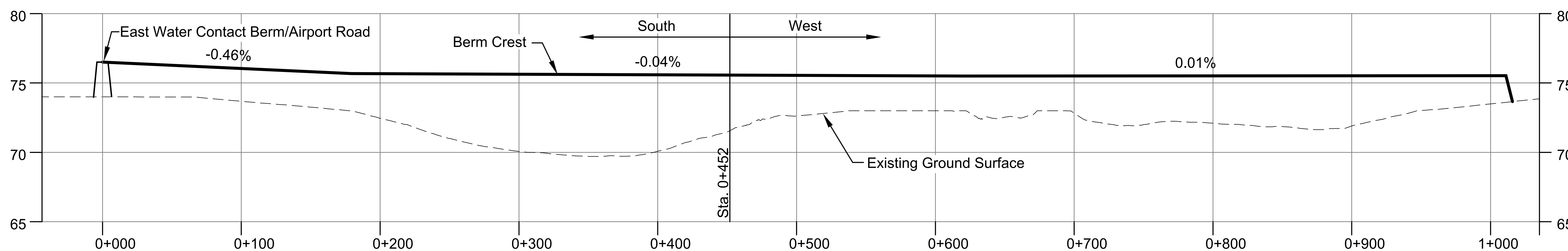
CENTERLINE PROFILE EAST CONTACT WATER BERM

10x Vertical Exaggeration



CENTERLINE PROFILE NORTH CONTACT WATER BERM

10x Vertical Exaggeration



CENTERLINE PROFILE SOUTH AND WEST CONTACT WATER BERM

10x Vertical Exaggeration

0m 3 6 9 12

1:300 (Vertical Scale)

0m 30 60 90 120

1 : 3000 (Horizontal Scale)

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MAC RESOURCES

Boston TMA Preliminary Design

Centerline Profile Contact Water Berms

SRK JOB NO.: 1CT022.004.600.20

FILE NAME: 1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg

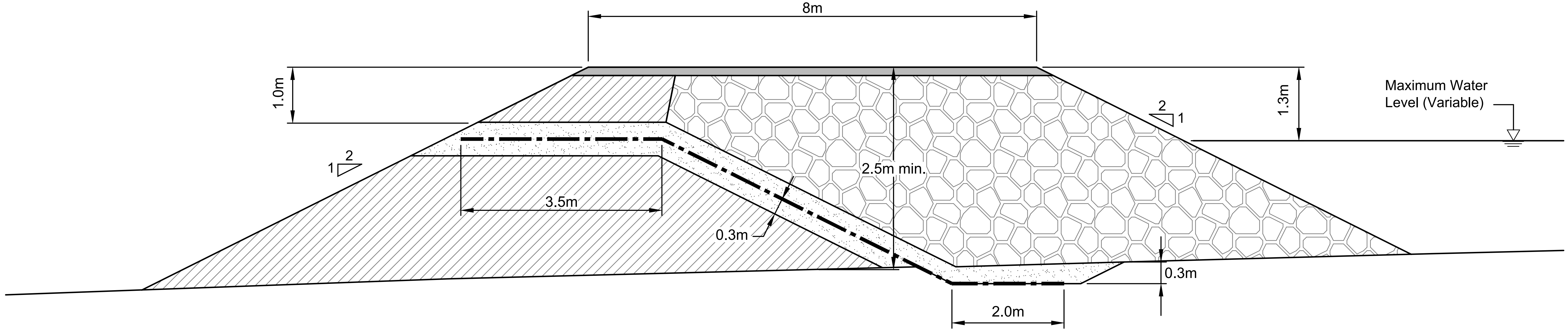
HOPE BAY PROJECT

DATE: NOV. 2016

APPROVED: IM

FIGURE: 6

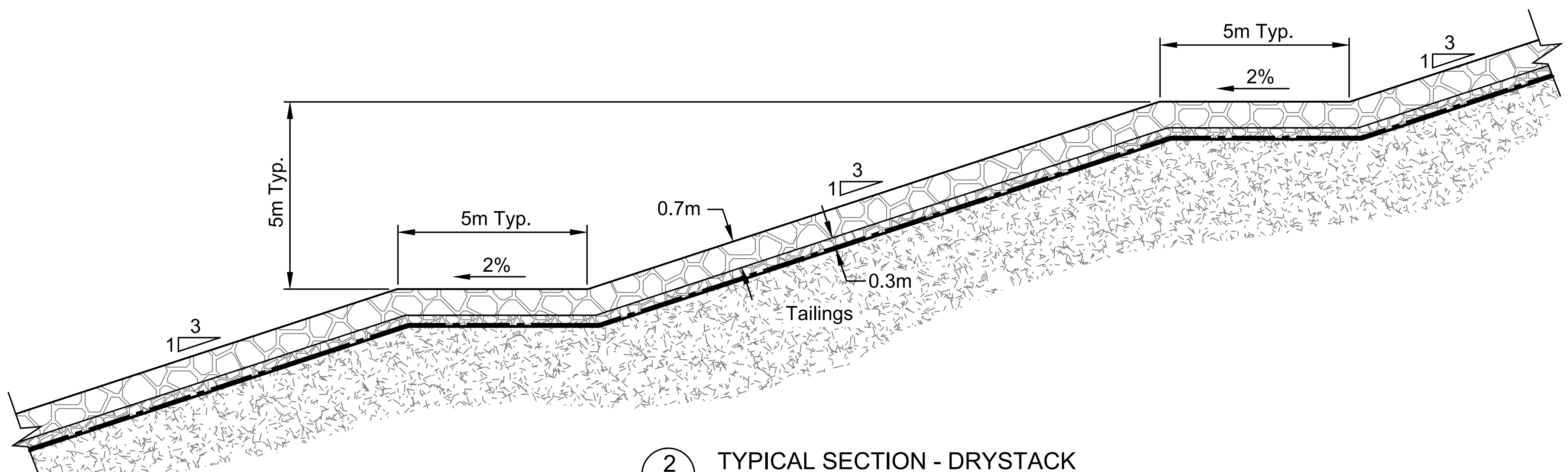
C:\MDDSINC\CLIENTS\SRK\2016\1CT022.004.600.20 BOSTON DRYSTACK\1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg



1/04 TYPICAL SECTION - CONTACT WATER BERM

0m 0.5 1.0 1.5 2.0 2.5 3.0

1 : 75



2/05 TYPICAL SECTION - DRYSTACK

0m 1 2 3 4 5 6

1 : 150

LEGEND

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

srk consulting

TMAC RESOURCES

SRK JOB NO.: 1CT022.004.600.20

FILE NAME: 1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg

HOPE BAY PROJECT

Boston TMA Preliminary Design

Contact Water Berm and Closure Cover Typical Details

DATE: NOV. 2016

APPROVED: IM

FIGURE: 7

C:\MDOSINC\CLIENTS\SRK\2016\1CT022.004.600.20 BOSTON DRYSTACK\1CT022.004T600.20-FIGURE 4-7 - Drystack.dwg

Appendices

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

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Cameron Hore	**Project No:**	1CT022.004
Reviewed:	Arcesio Lizcano, PhD Maritz Rykaart, PhD, PEng	**Date:**	December 1, 2016
Subject:	Hope Bay Project — Boston Tailings Management Area 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 which is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris only, while Phase 2 includes additional 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).

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)	1.0	1.0
• Long term (reclamation – abandonment)	1.2	1.1
Overall Stability (Deep Seated Stability)		
• Short term (static)	1.3 – 1.5	1.1 – 1.3
• Long term (static)	1.5	1.3
• Pseudo-static	1.1 – 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 for long-term scenarios, i.e. post-closure, the design seismic event must be increased to halfway between the 1:100 and 1:1,000 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 criteria 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 Analysis Method

3.1 Conceptual Model

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 25 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 4H:1V at closure.

The model includes foundation soil layers (bedrock and overburden), a phreatic surface, filtered tailings, a geosynthetic cover (High Density Polyethylene, HDPE), 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 HDPE liner placed directly on the tailings surface, covered with 0.3 m of graded liner protection gravel, followed by 0.7 m of ROQ rock. Figure 1 illustrates the design cross-section and associated model domain.

3.2 Modeling Method

Stability of the critical section was assessed using two procedures; the advanced strength reduction method as applied in the finite element code PLAXIS (Plaxis, 2016), followed by a standard limit equilibrium analysis using Slope/W (Geoslope, 2012).

3.3 PLAXIS Analysis

3.3.1 Approach

The PLAXIS analysis was completed using 2D plane-strain conditions with 15-node elements. The generated model consisted of 2,210 soil elements, 18,900 nodes, and an average element size of 2.8 m. Both static and pseudo-static loads were considered for drained and undrained loading conditions, with a predefined phreatic surface.

The model was analyzed with the Mohr-Coulomb constitutive model, using the fully coupled deformation-flow analyses, which includes:

- Different construction stages and rates;
- Generation and dissipation of excess pore water pressure;
- Water pressure and flow induced by compression, and consolidation of all overburden materials, solved simultaneously with the soil and rock mass stress field; and
- Computation of the FOS using the strength reduction technique.

3.3.2 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, 2016b). Table 3 summarizes the properties used in the analysis.

The initial stresses in the model domain prior to any construction activities were calculated using the foundation self-weight and associated hydrostatic water pressure.

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
Initial Void Ratio	e_{init}	-	1.0	0.5	1.0	0.6	0.2
Young's Modulus at Reference Level	E'	MPa	175	150	5	100	800
Cohesion	c'_{ref}	kPa	0	112	0	0	15
Friction Angle	φ '	º	40	26	30	40	0
Dilatancy Angle	ψ	º	0	0	0	0	0

Property	Symbol	Unit	ROQ (Thawed)	Marine Silt and Clay (Frozen)	Marine Silt and Clay (Thawed)	Filtered Tailings	HDPE Liner
Undrained Shear Strength at Reference Level	Su_{ref}	kPa	-	-	13	-	-
Poisson's ratio	v	-	0.3	0.3	0.3	0.3	0.3
Horizontal Permeability	K_x	m/s	5.0×10^{-3}	4.6×10^{-10}	4.6×10^{-10}	1.3×10^{-7}	Non-porous
Vertical Permeability	K_y	m/s	5.0×10^{-3}	4.6×10^{-10}	4.6×10^{-10}	1.3×10^{-7}	Non-porous
K_0 Determination	Type	-	Auto	Auto	Auto	Auto	Auto
Lateral Earth Pressure Coefficient	$K_{0,x}$	-	0.38	0.56	0.5	0.36	1.0

3.3.3 Staged Construction

Stability was assessed for five construction stages represented by the individual lifts of the dry-stack facility. The overarching assumptions include:

- Stage 1 construction considers a 6 m lift, with the remaining stages having 5 m lifts;
- Construction of each lift is assumed to be continuous over the designated time period;
- The construction period for each lift was estimated based on the design volume and provided production rates with a 25% ramp up factor to allow for inconsistencies in the lift construction timing. The applied construction periods are presented in Table 4; and
- Stages are constructed sequentially, with no time period between stages.

Table 4: Staged Construction Periods

Stage	Construction Period (days)
1	691
2	726
3	586
4	463
5	303

3.3.4 Analysis Conditions

Static Analysis

The following loading conditions are considered in the static analysis:

- Drained (i.e. long-term) behaviour where the stiffness and strength are defined in terms of effective properties; and
- Undrained (i.e. short-term) behaviour where stiffness is defined in terms of effective properties and strength is defined as undrained shear strength.

Pseudo-Static Analysis

The pseudo-static analysis was completed under partially frozen (see Figure 1) undrained foundation conditions to evaluate the stability of the facility during an earthquake.

Post-earthquake Analysis

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 moisture content of the tailings material if thawed is such that tailings and/or foundation liquefaction is unlikely. As a result, a post-earthquake analysis was not completed.

3.4 Slope/W Analysis

The Slope/W analysis assumes the same cross-section, phreatic level and material properties as the PLAXIS analysis; however, as this was simply a confirmation analysis, only the 1st and 5th construction stages were assessed.

4 Results

The analysis results are summarized in Table 4, and complete details are presented in Attachment 1.

Table 5: Slope Stability Analysis Results

Analysis Method	Construction Stage	Short-term (Undrained) FOS	Long-term (Drained) FOS	Pseudo-Static FOS
Minimum Required FOS		**1.3**	**1.5**	**1.1**
PLAXIS	1st Stage (Height 6 m)	1.4	1.8	1.25
	2nd Stage (Height 5 m)	1.4	1.9	1.25
	3rd Stage (Height 5 m)	1.4	1.9	1.25
	4th Stage (Height 5 m)	1.4	1.9	1.25
	5th Stage (Height 5 m)	1.4	1.9	1.25
Slope/W	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 and applicable. In all cases, the required minimum FOS was met (1.3 for short-term static stability, 1.5 for long-term static stability, and of 1.1 for pseudo-static stability).

In addition to determining the facility FOS, the PLAXIS results provide information as to the physical behavior of the facility. Figure 2 shows the excess pore water pressure distribution in the foundation at the end of the first construction stage. The zero excess pore pressure within the tailings and the thawed marine silt and clay indicates that construction of the first stage occurs under drained loading conditions in both these zones. This conclusion is plausible considering the construction method which entails building the dry-stack up in 0.3 m compacted lifts, with the average rate of rise for Stage 1 of about 6 cm per week.

Figures 2 and 3 illustrate excess pore water pressure distribution at the end of the construction of the first and final stages of the facility. The lower excess pore water pressure values for the toe region shown in Figure 3, indicates that the rate of pore water pressure dissipation is faster than the increased loading accumulation in the thawed layer during the construction of the facility. It should be noted that pore water pressures were calculated (in PLAXIS) based on the assumption that the phreatic surface is located at the ground surface.

Figures 4 and 5 show the critical failure surfaces determined using the PLAXIS and Slope/W methods respectively. These analyses clearly show that in all cases the failure surface is near the toe of the facility, within the first construction stage. Under no scenarios is large deep seated foundation failures induced.

The vertical deformation distribution of the facility at the end of its construction is shown in Figure 6. The maximum vertical deformation, which includes the elasto-plastic deformation and consolidation, was determined to be 6.3 cm along the top surface of the facility. This deformation assumes fully thawed tailings, which based on thermal analysis is not expected.

5 References

Canadian Dam Association (CDA). 2014. Technical Bulletin: Application of Dam Safety Guidelines to Mining Dams. 2014.

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PLAXIS bv 2015. PLAXIS 2D 2015 (Basic, Dynamics, and Plaxflow).Delft, Netherlands.

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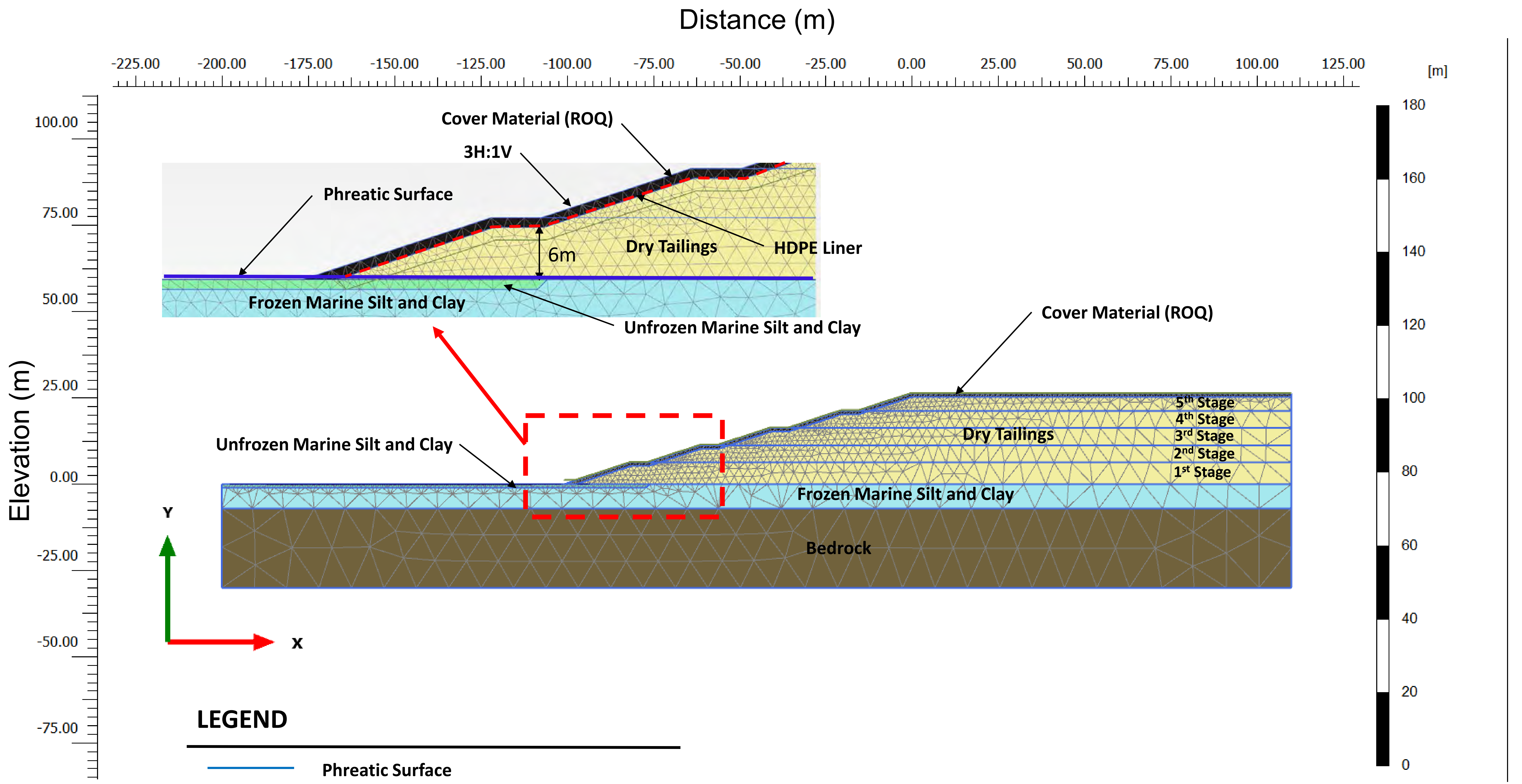
SRK Consulting (Canada) Inc. 2016b. Hope Bay Project, Geotechnical Design Parameters and Overburden Summary Report. Report prepared for TMAC Resources Inc. Project No.: 1CT022.004. 2016.

Weichert, D.H. and Rogers, G.C., 1987. "Seismic Risk in Western Canada." Symp. Earthquake Eng., Van. Geotech. Soc., Vancouver. May 1987.

Figures



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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MAC RESOURCES

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

Job No: 1CT22.004.620

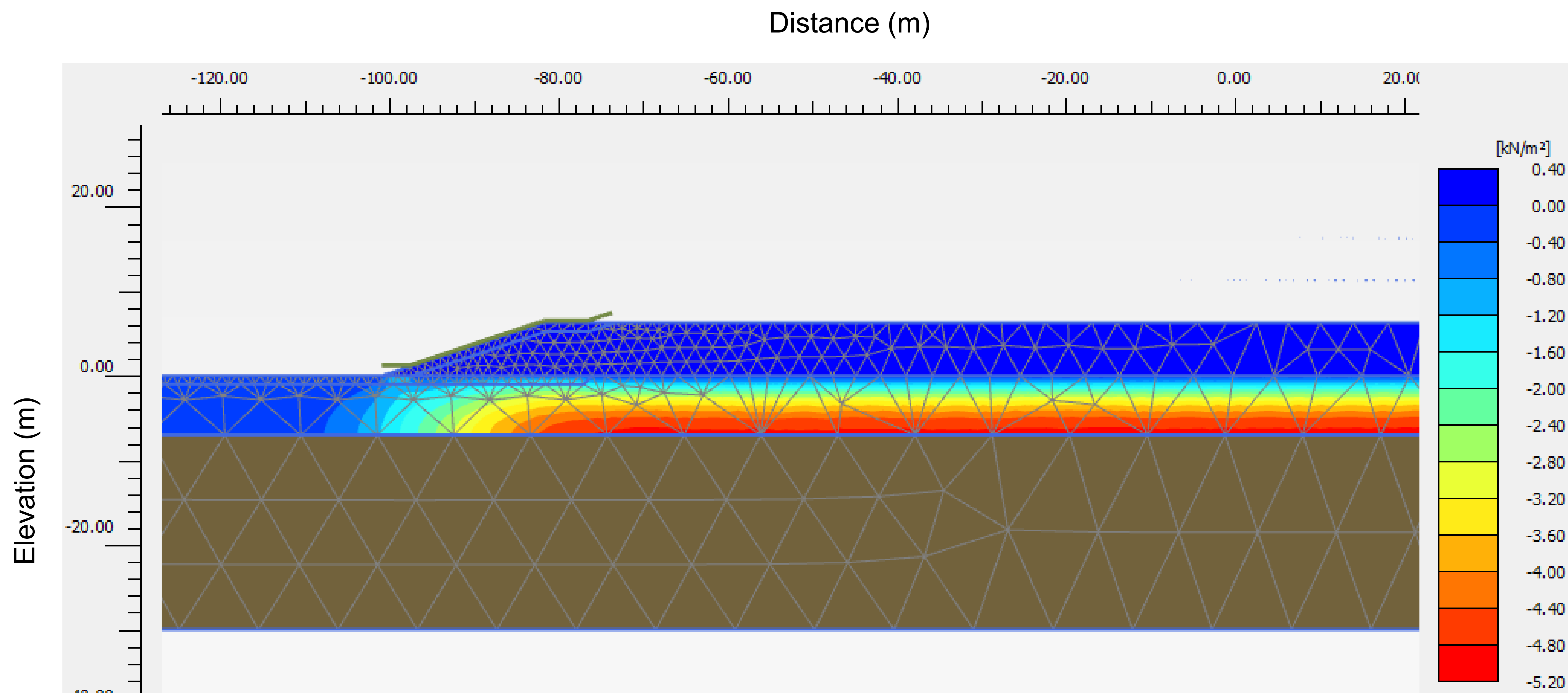
Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA

HOPE BAY PROJECT

Date: November 2016

Approved: CH

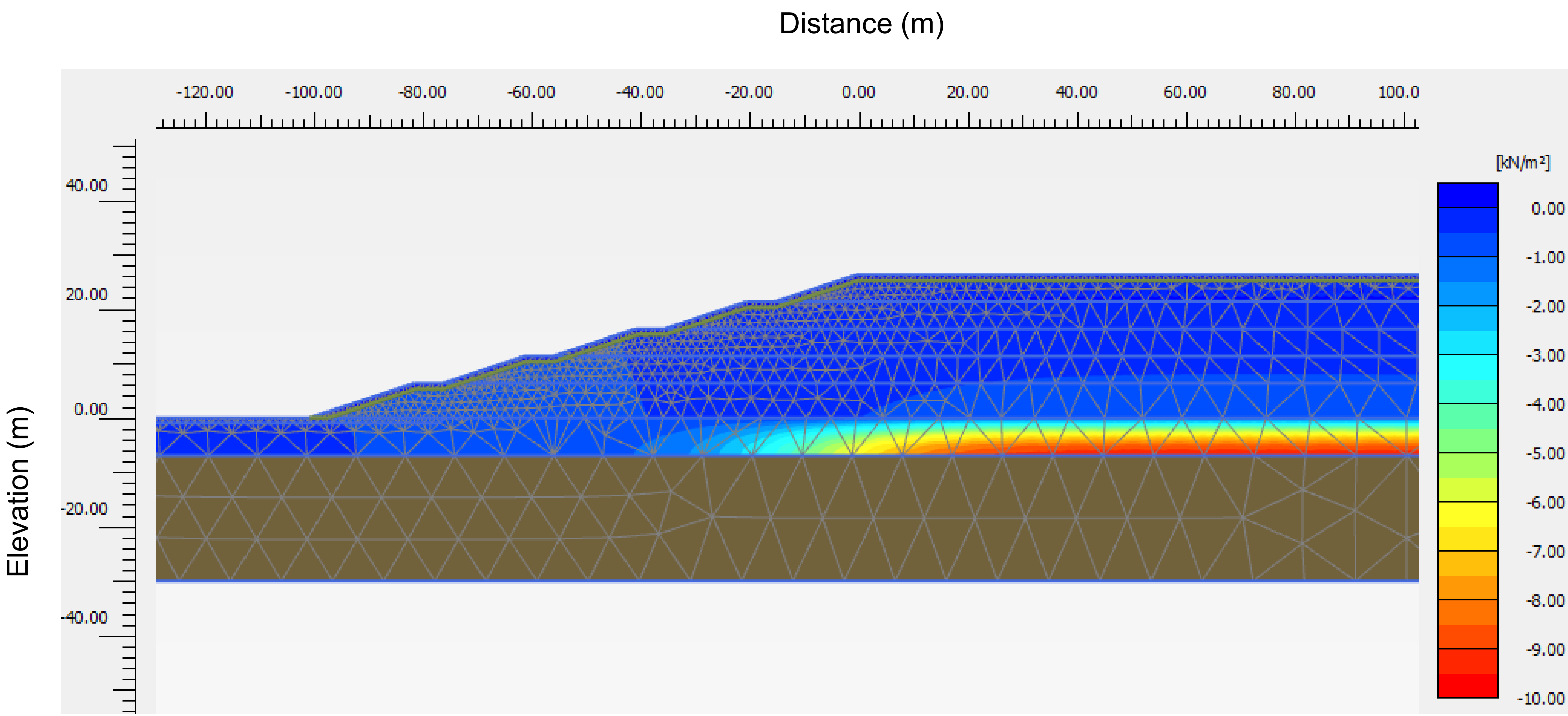
Figure: 1



Note:

- The stress distribution shown in this figure corresponds to the end of first stage construction.
- The excess pore water pressures are shown in negative values.

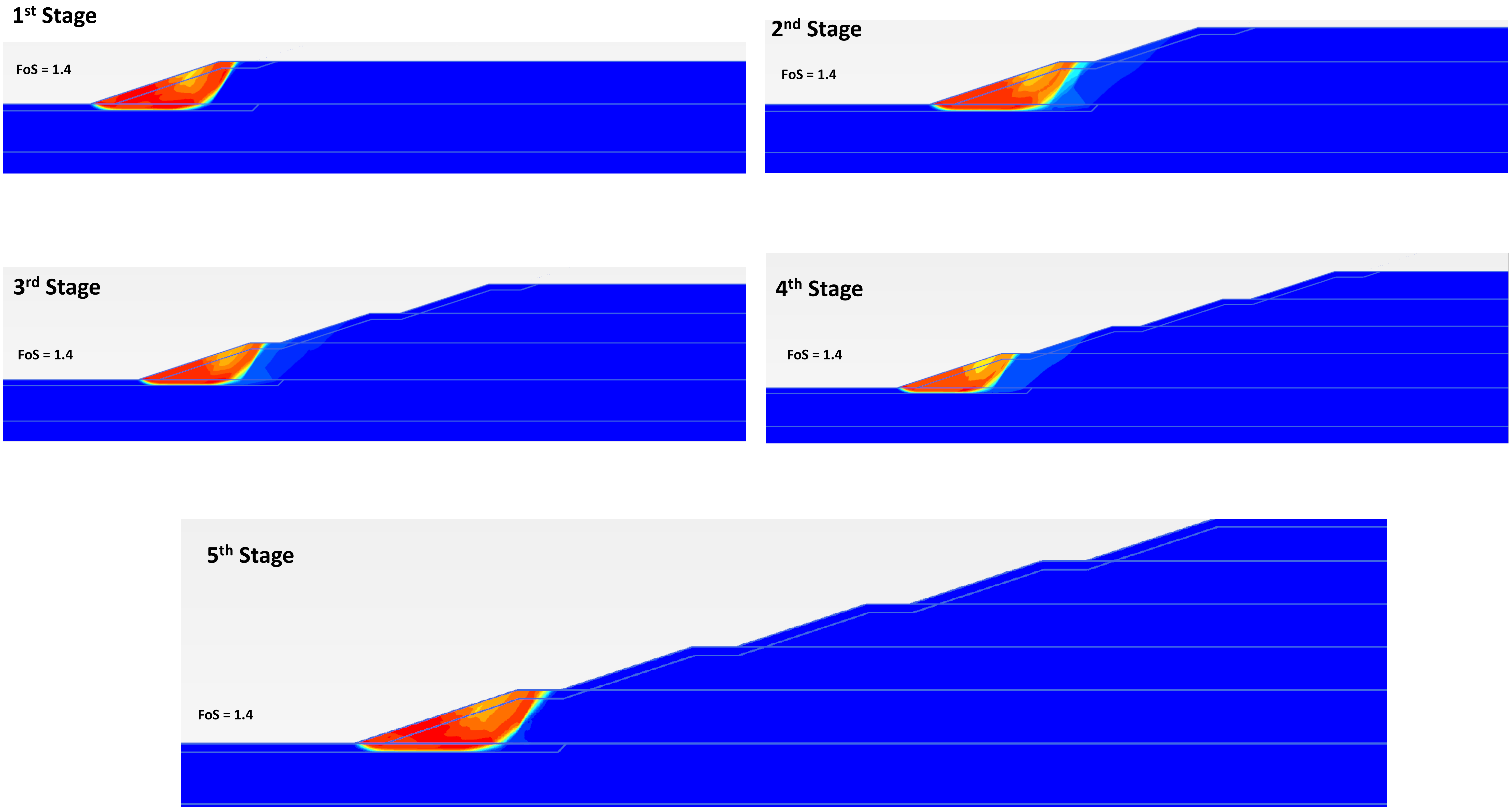
srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Excess Pore Water Pressure 1st Stage
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA	HOPE BAY PROJECT	Date: November 2016 · Approved: CH · Figure: 2



Note:

- The stress distribution shown in this figure corresponds to the end of final stage construction.
- The excess pore water pressures are shown in negative values.

srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY		
		Excess Pore Water Pressure 5th Stage		
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA	HOPE BAY PROJECT	Date: November 2016	Approved: CH	Figure: 3



srk consulting

MAC RESOURCES

BOSTON TMA SLOPE STABILITY

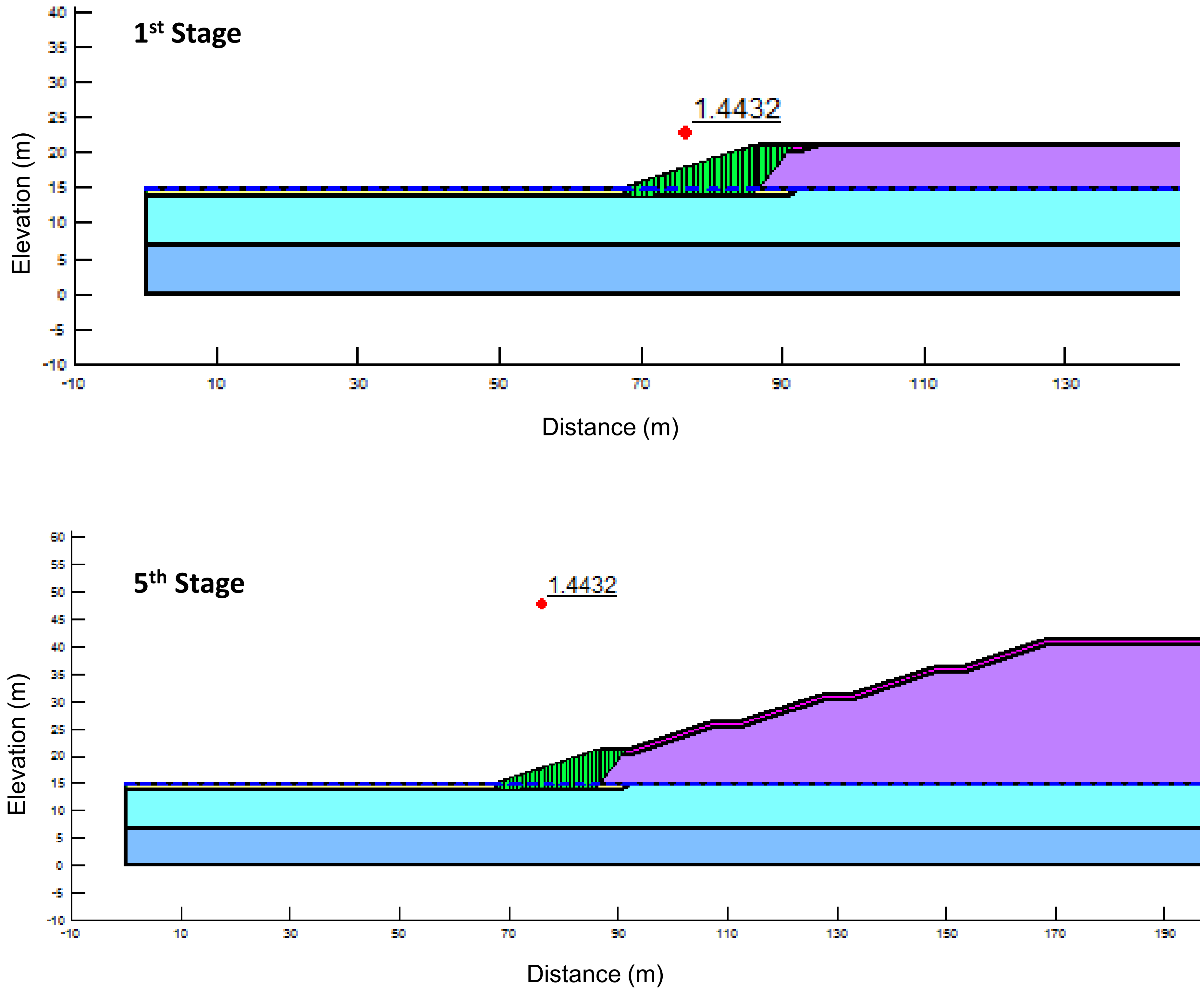
Finite Element Analysis Slope Stability Results (Plaxis)

Job No: 1CT22.004.620

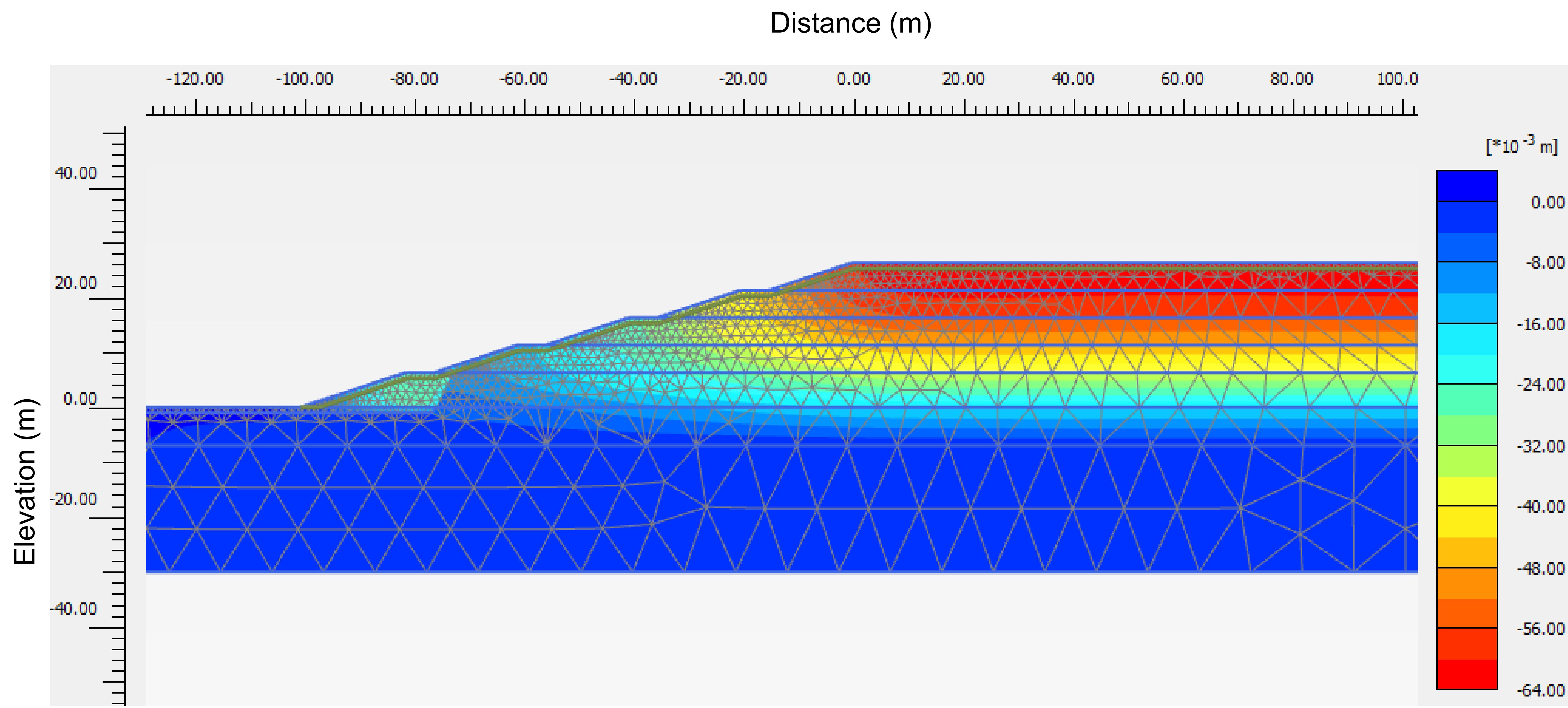
Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA

HOPE BAY PROJECT

Date:	Approved:	Figure:
November 2016	CH	4



srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Limit Equilibrium Analysis Slope Stability Results (Slope/w)
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA	HOPE BAY PROJECT	Date: November 2016 / Approved: SA / Figure: 5



srk consulting

MAC RESOURCES

BOSTON TMA SLOPE STABILITY

Boston TMA
Vertical Deformation

Job No: 1CT22.004.620

Filename: HopeBay_Boston_DSTSF_SlopeStability_Figures_CH_RevA

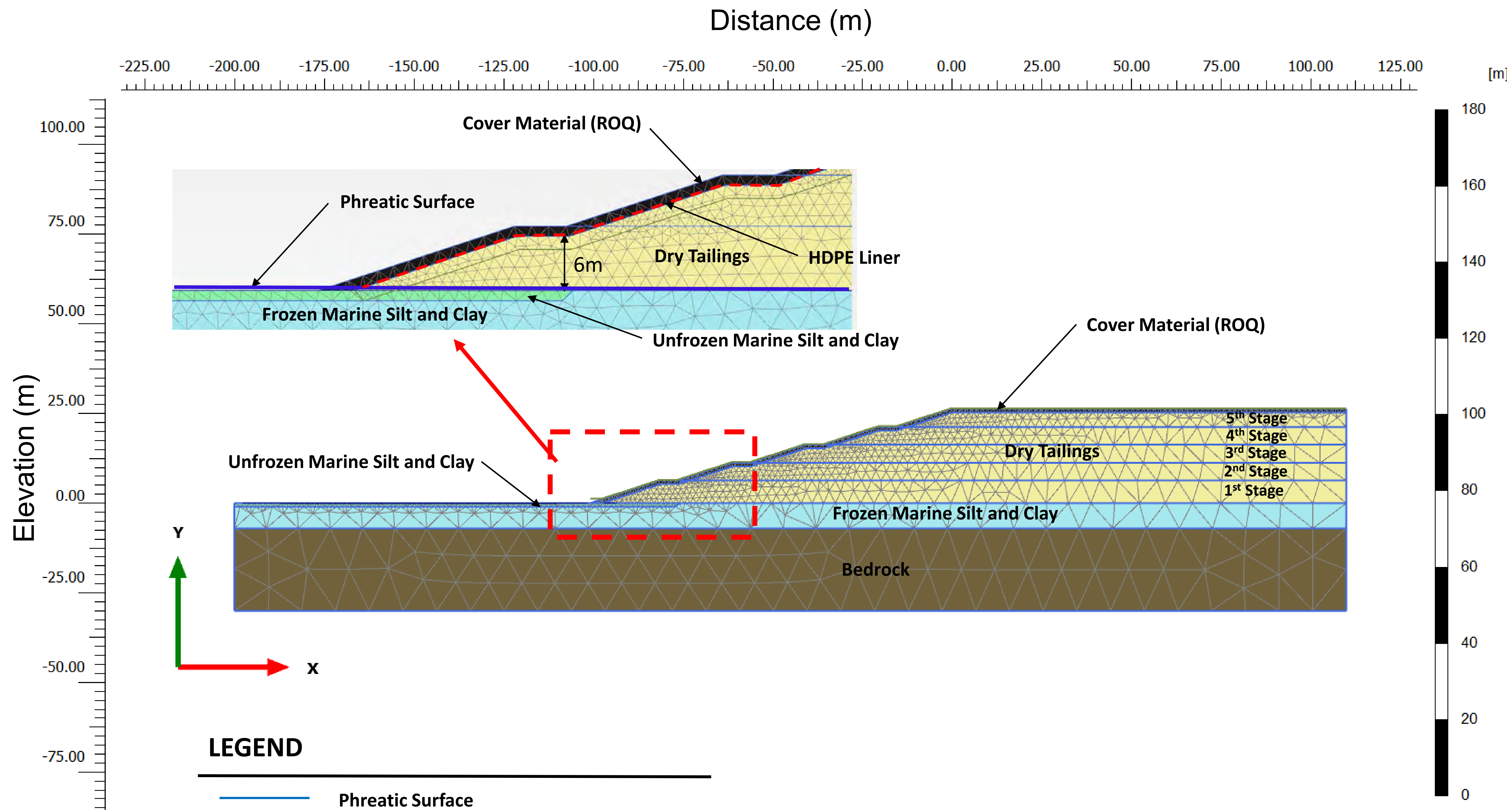
HOPE BAY PROJECT

Date: November 2016

Approved: CH

Figure: 6

Attachment 1: Slope Stability Result



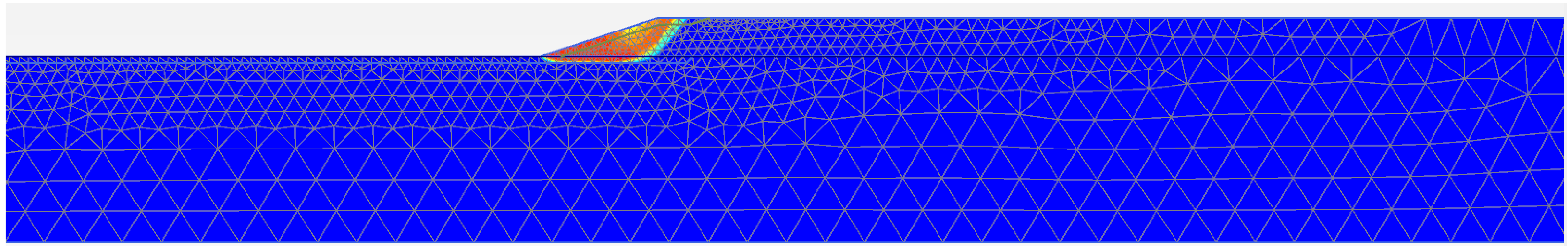
LEGEND

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

srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 / Approved: CH / Figure: A1 - 1

Boston TMA Slope Stability Results – Undrained Condition – Plaxis

Fos = 1.4



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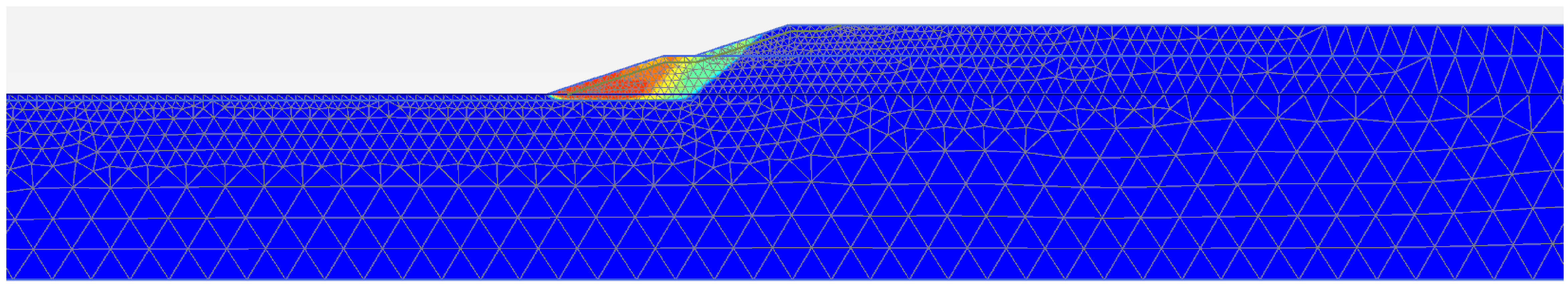

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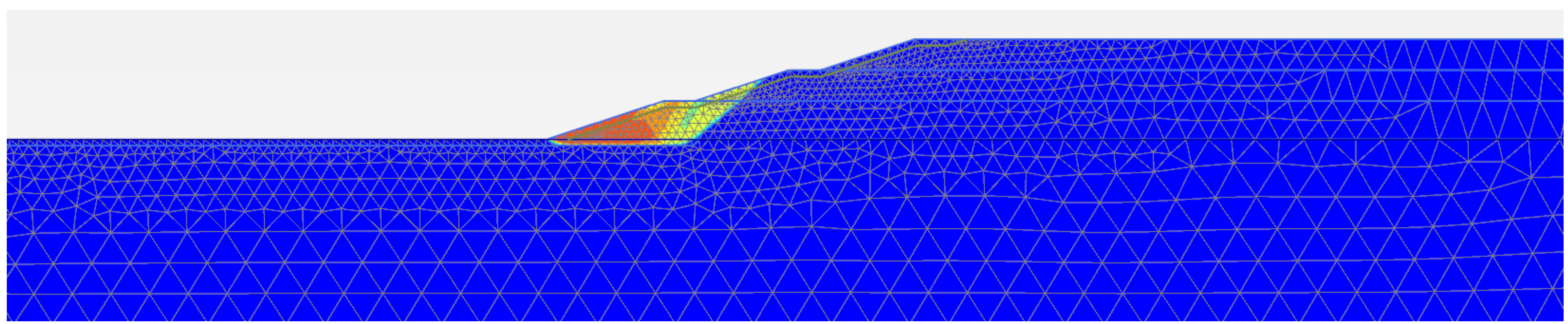

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY		
		Analyzed Cross-Section		
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016	Approved: CH	Figure: **A1 - 2**

Boston TMA Slope Stability Results – Undrained Condition – Plaxis

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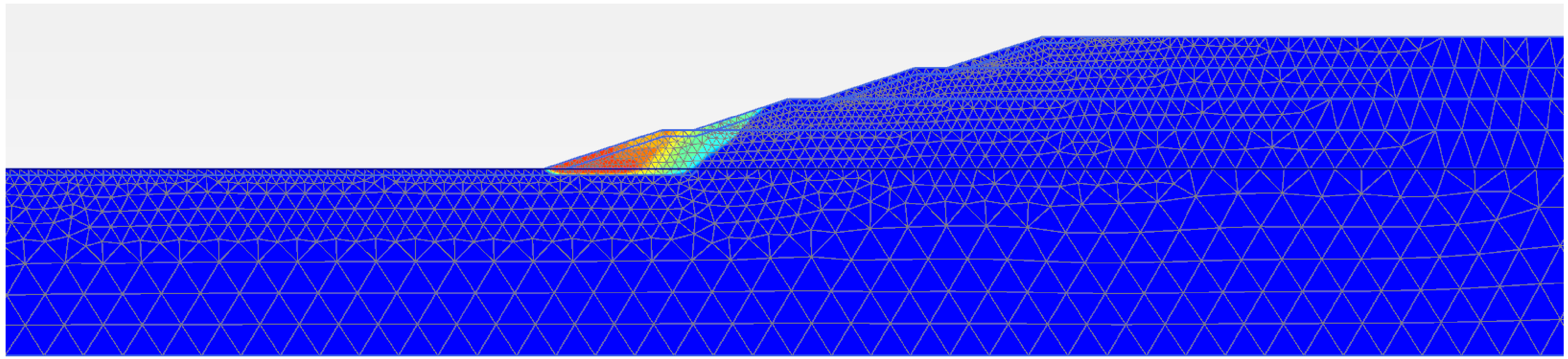

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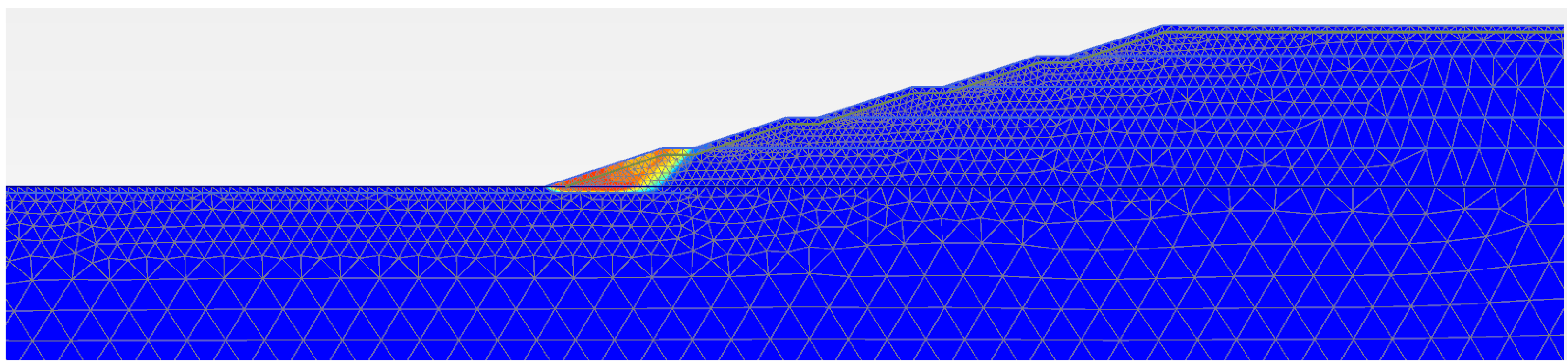

srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 | Approved: CH | Figure: **A1 - 3**

Boston TMA Slope Stability Results – Drained Condition – Plaxis

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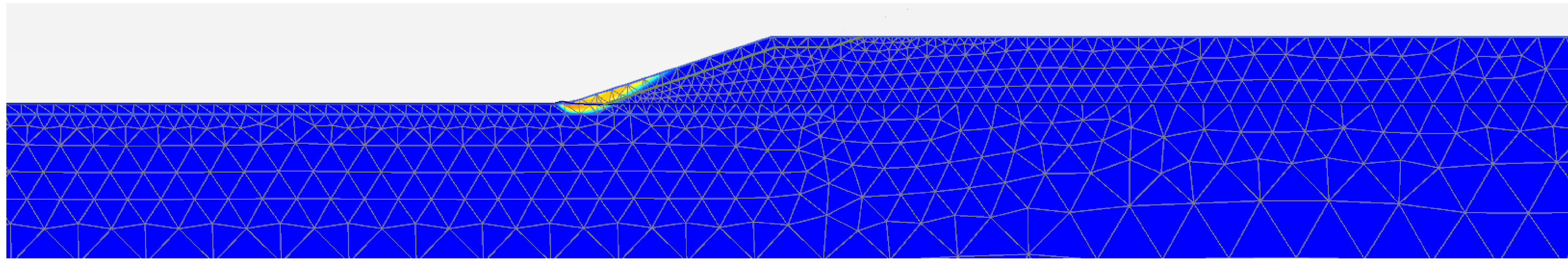

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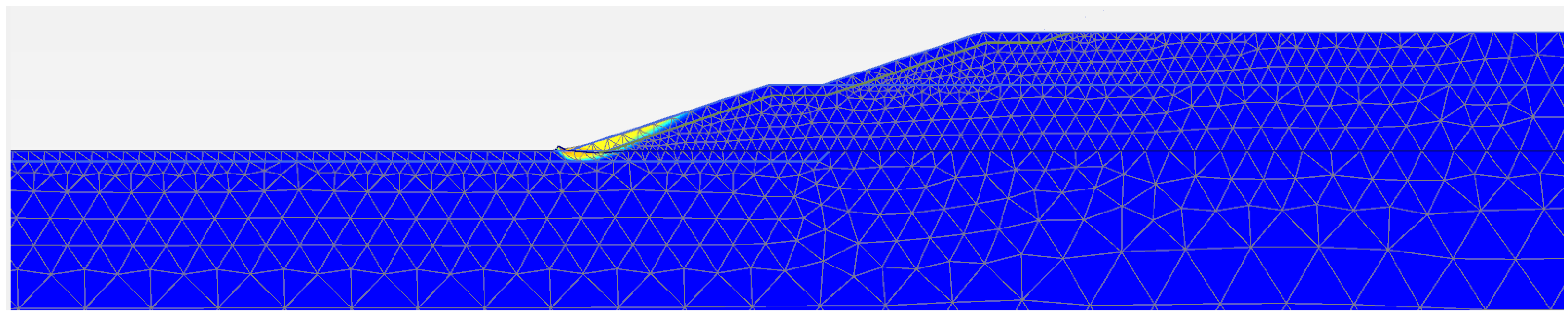

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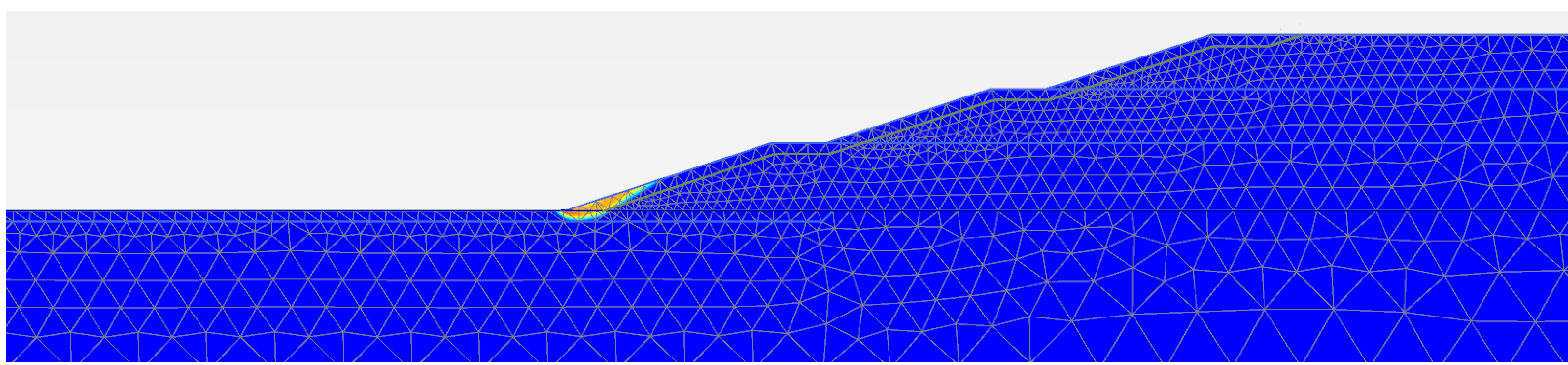

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	**Analyzed Cross-Section** Date: November 2016 · Approved: CH · Figure: A1 - 4

Boston TMA Slope Stability Results – Drained Condition – Plaxis

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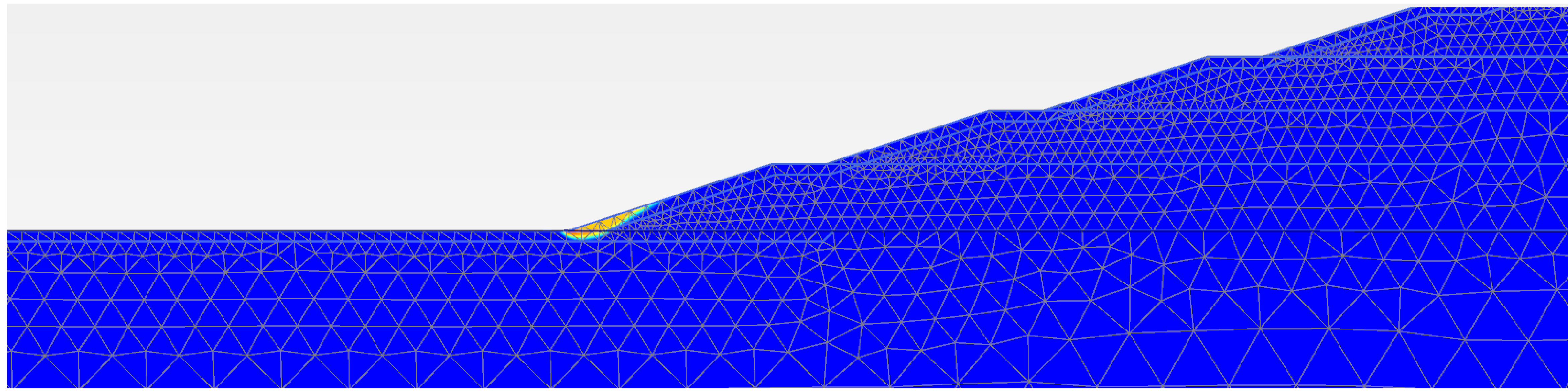

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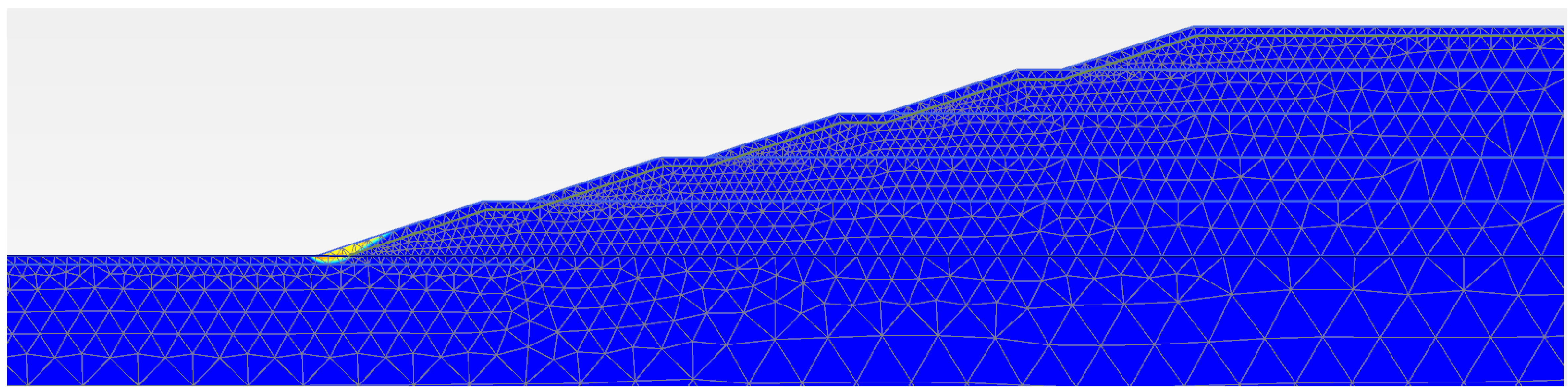

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 · Approved: CH · Figure: A1 - 5

Boston TMA Slope Stability Results – Pseudo-Static – Plaxis

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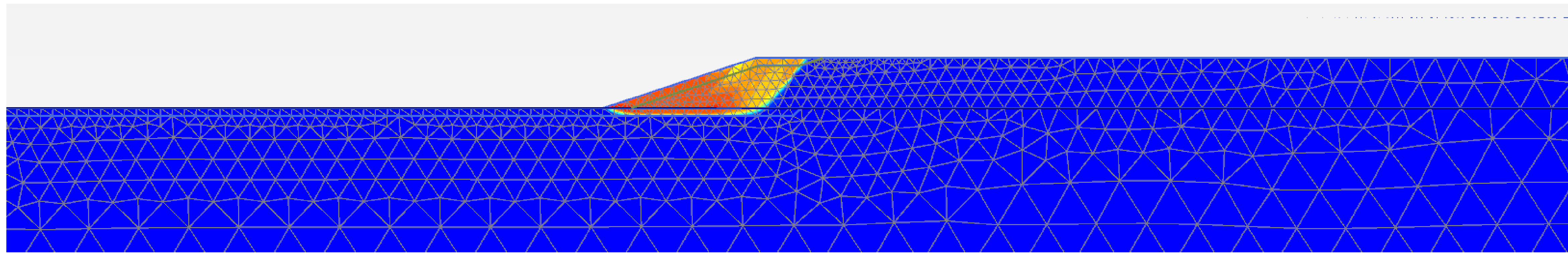

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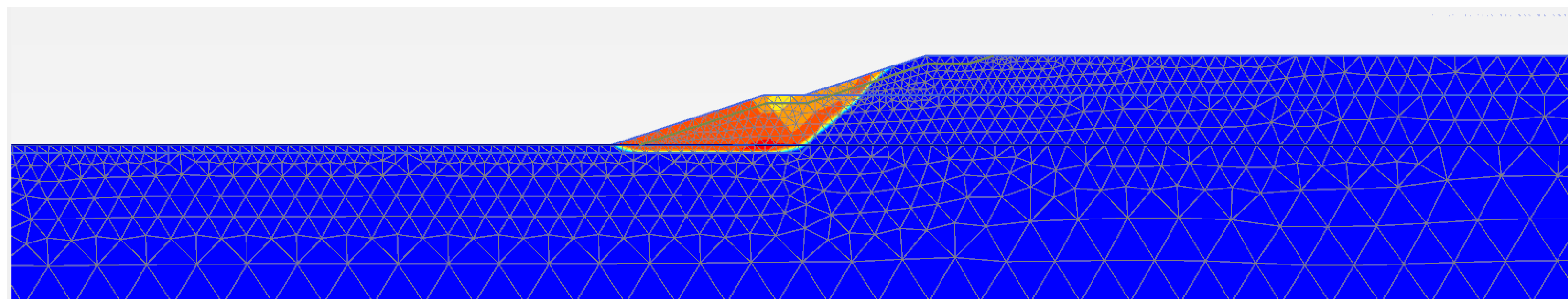

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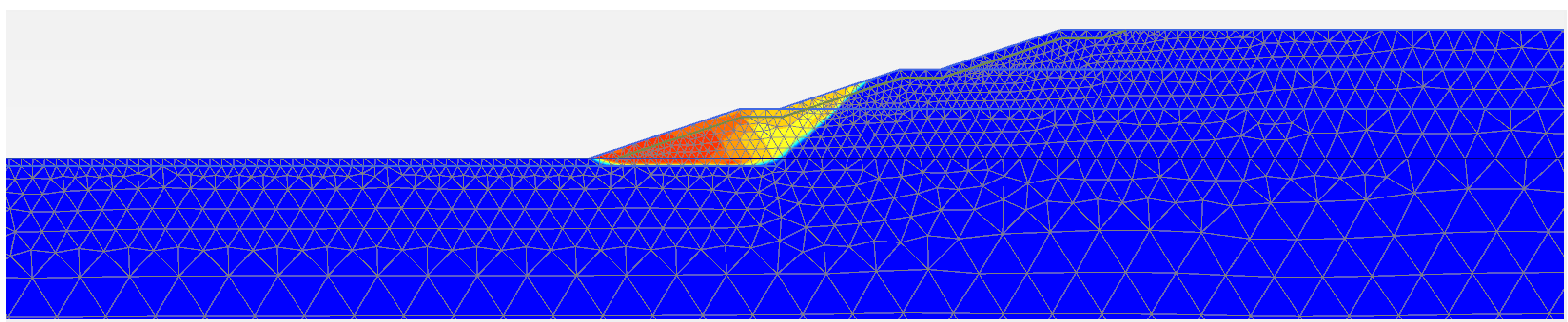

srk consulting

TMAC RESOURCES

Job No: 1CT22.004.620

Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA

HOPE BAY PROJECT

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

Date:	Approved:	Figure:
November 2016	CH	A1 - 6

Boston TMA Slope Stability Results – Pseudo-Static – Plaxis

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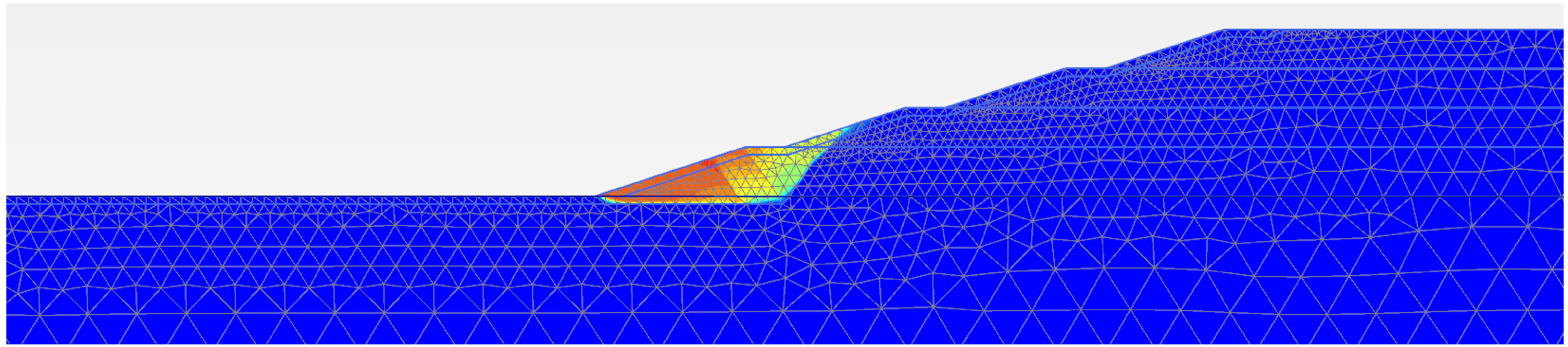

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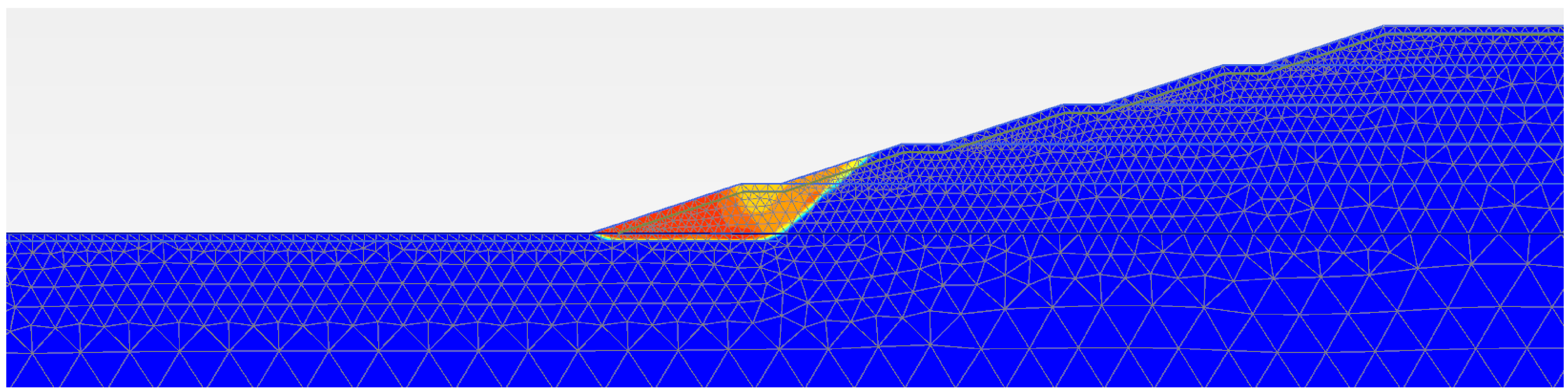
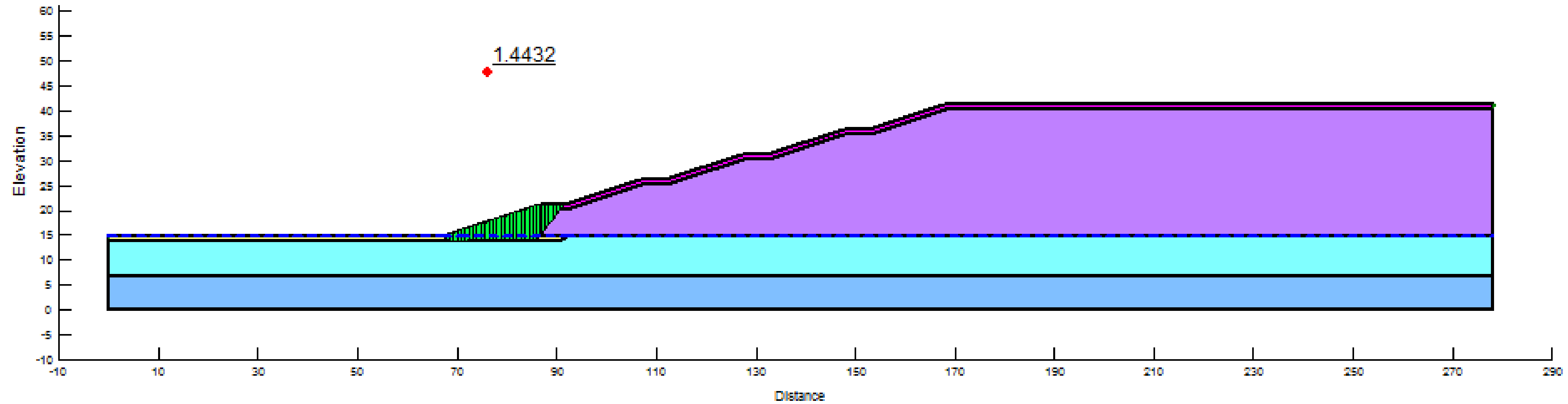


srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 / Approved: CH / Figure: **A1 - 7**

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



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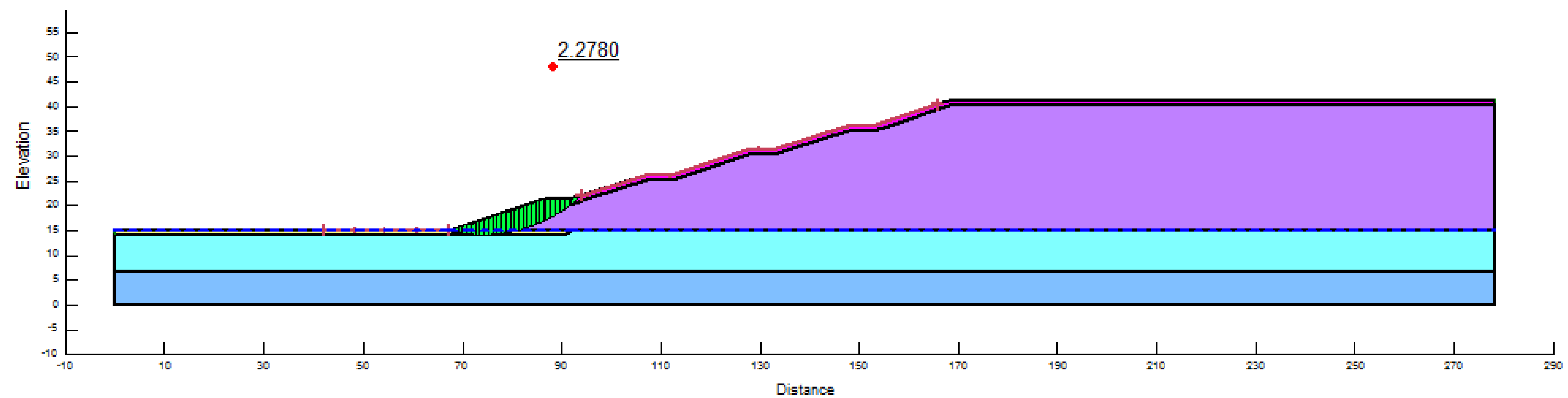

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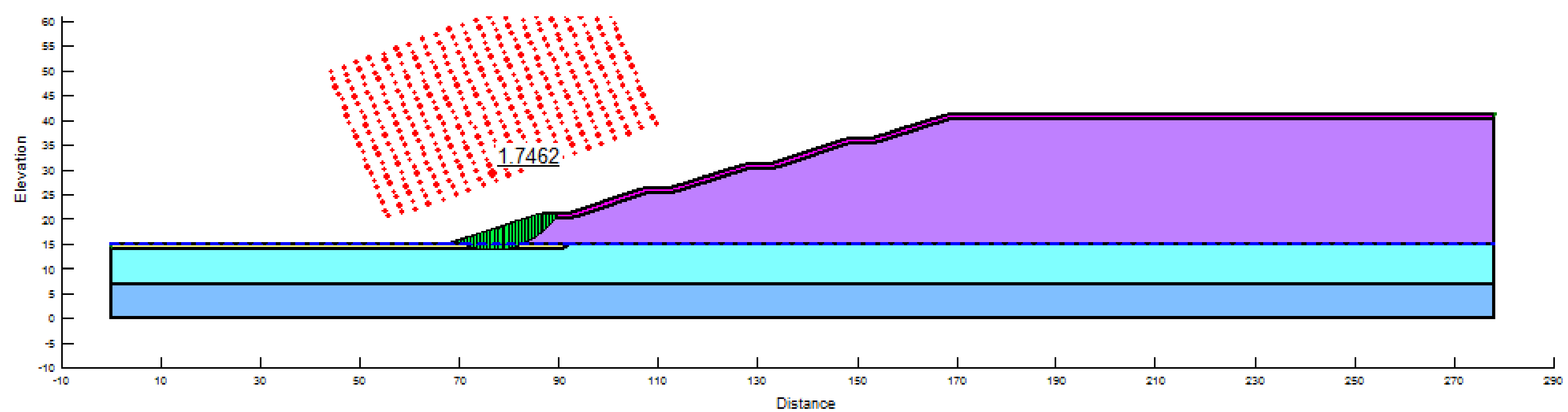
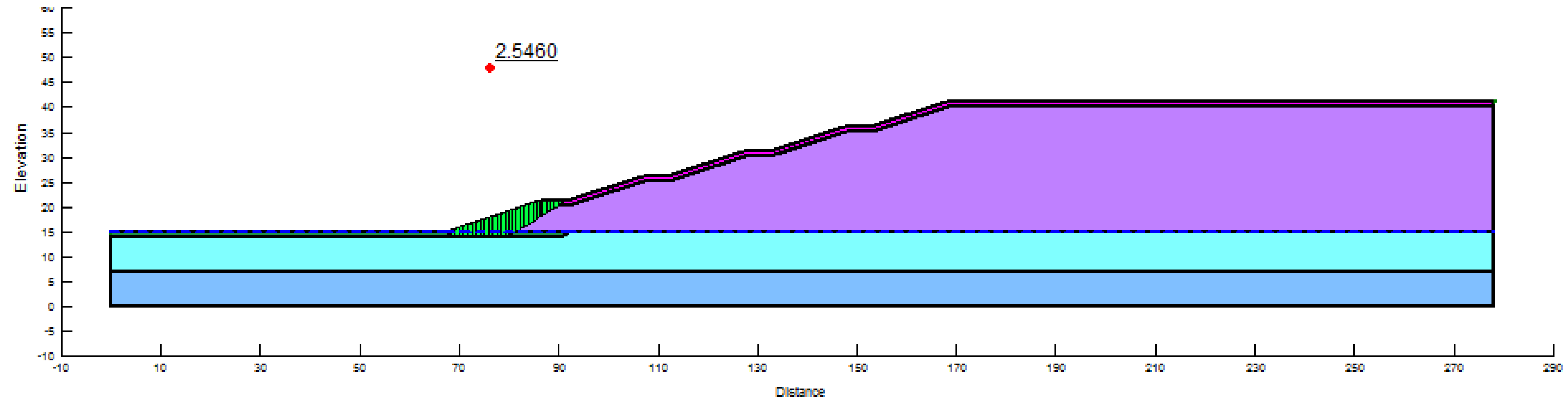


Grid & Radius

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 / Approved: SA / Figure: **A1 - 8**

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



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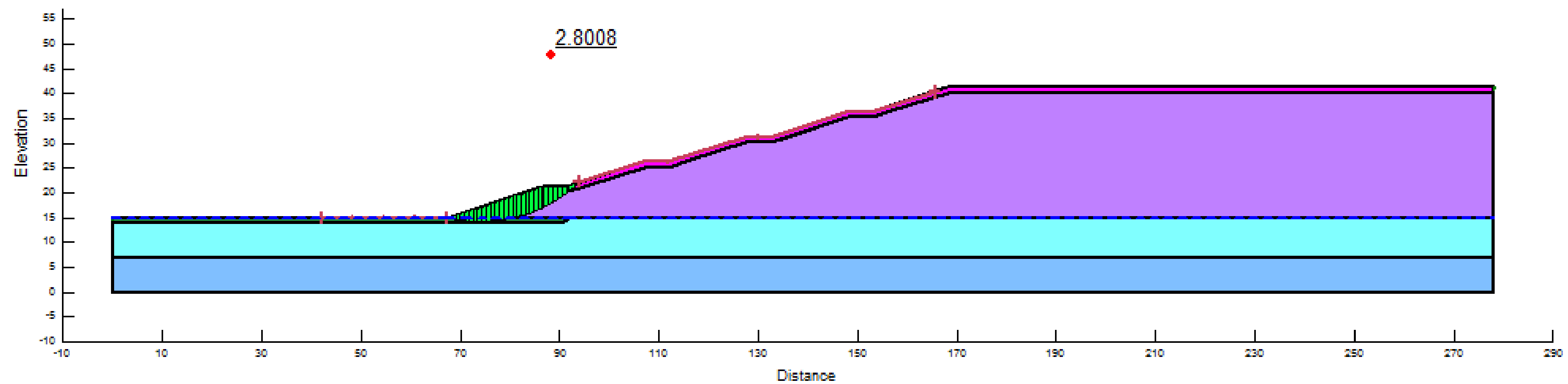

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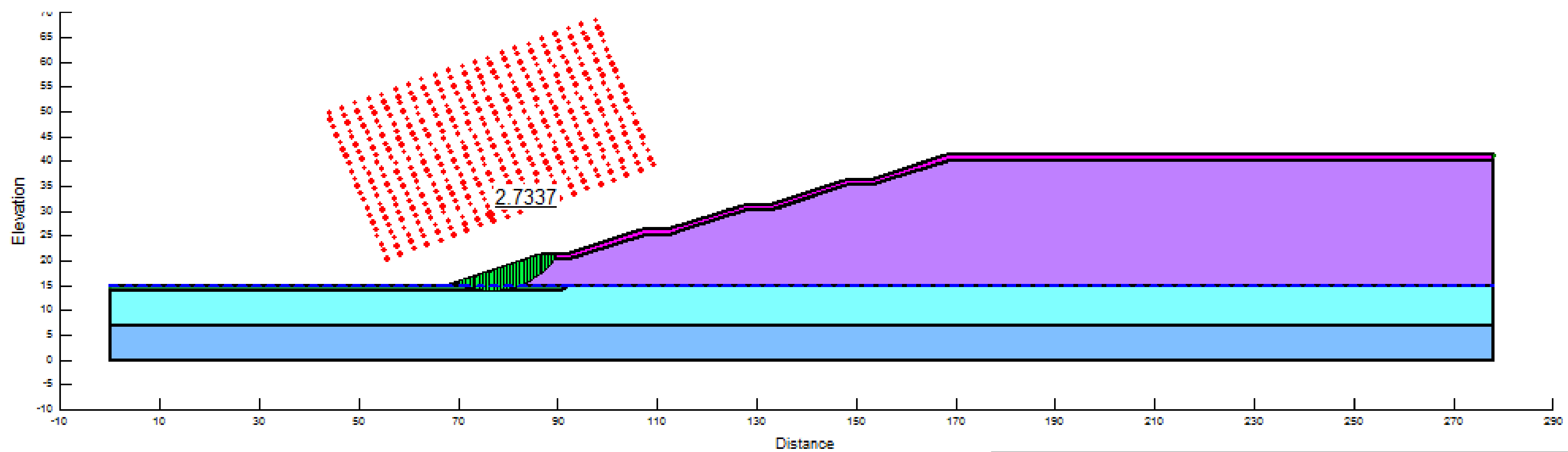


Grid & Radius

srk consulting

MAC RESOURCES

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

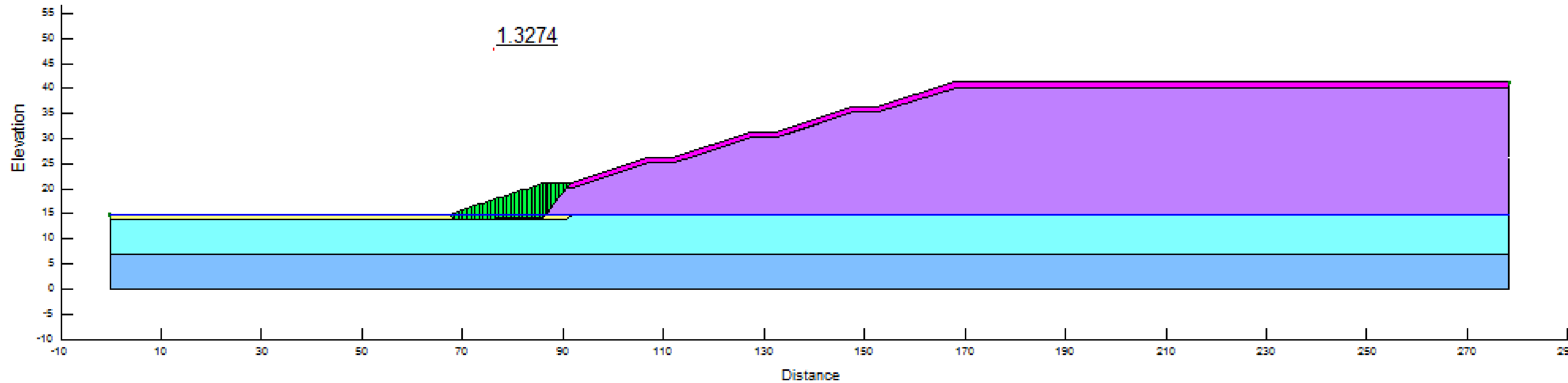
Job No: 1CT22.004.620

Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA

HOPE BAY PROJECT

Date:	Approved:	Figure:
November 2016	SA	A1 - 9

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



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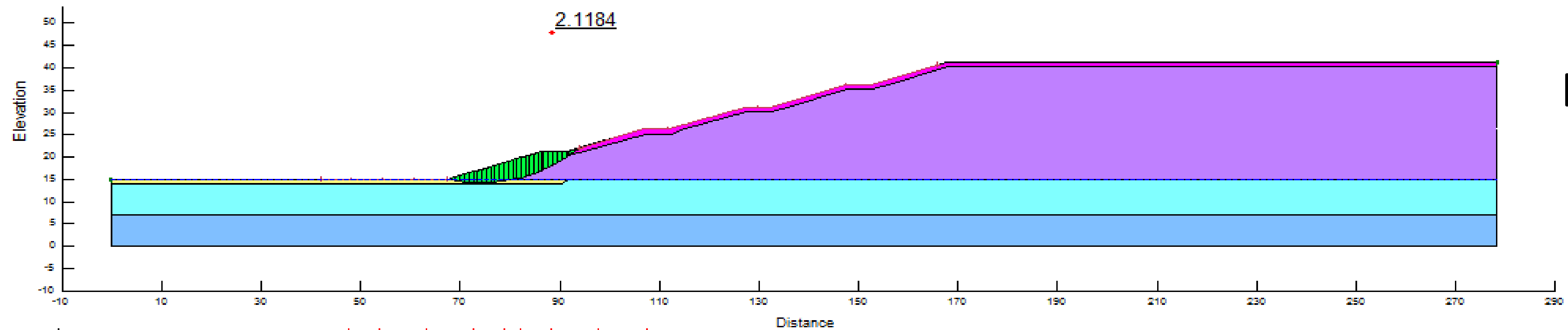

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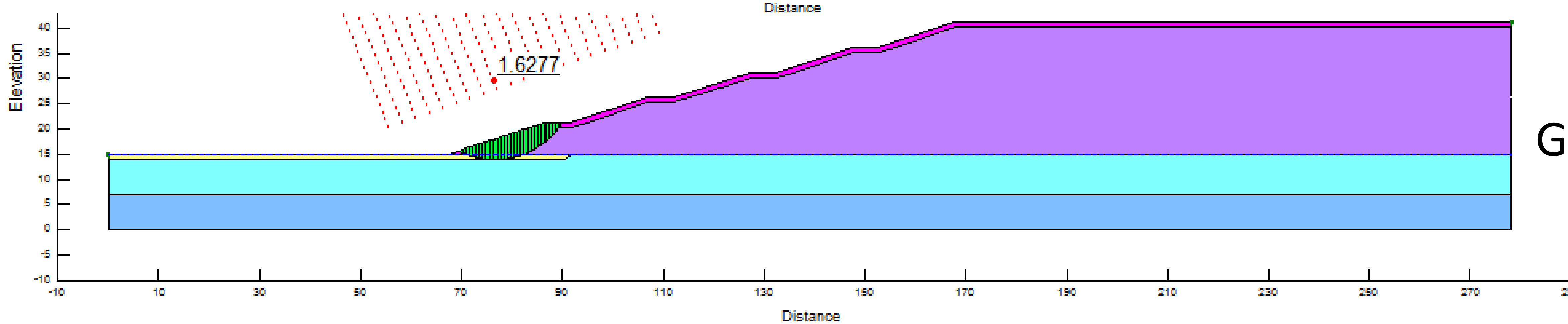
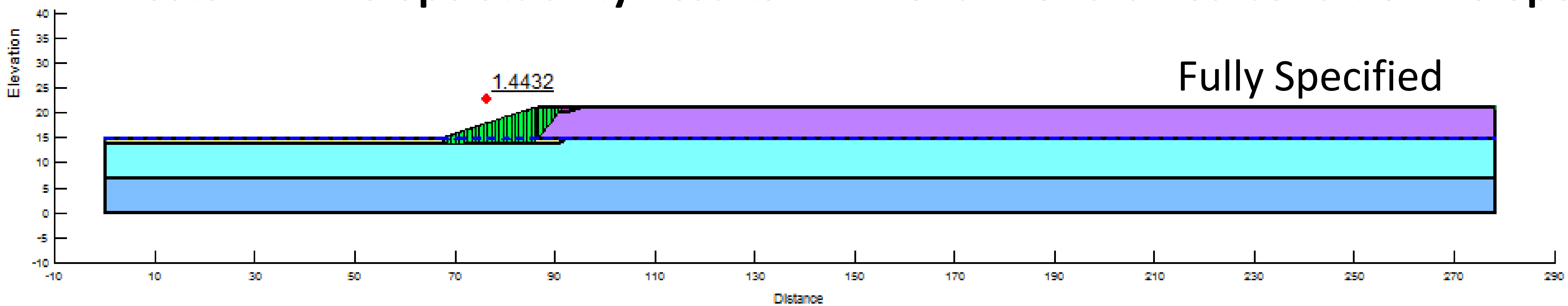
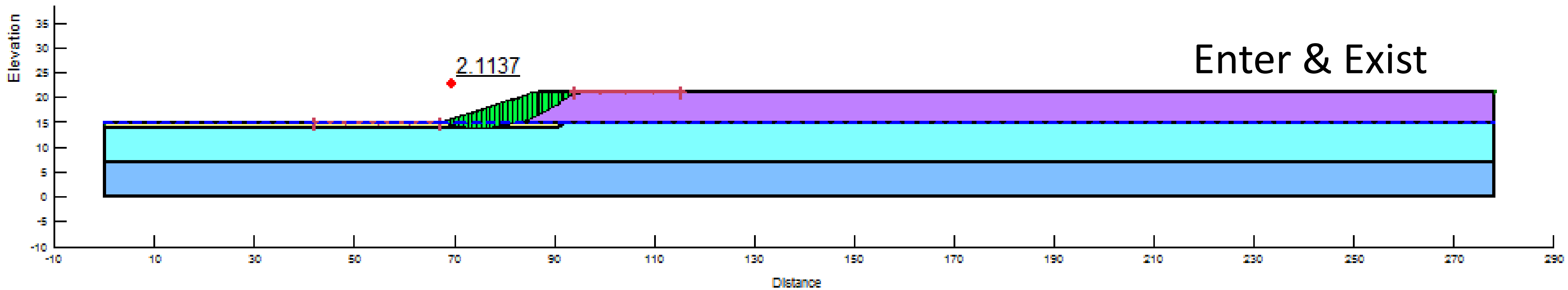


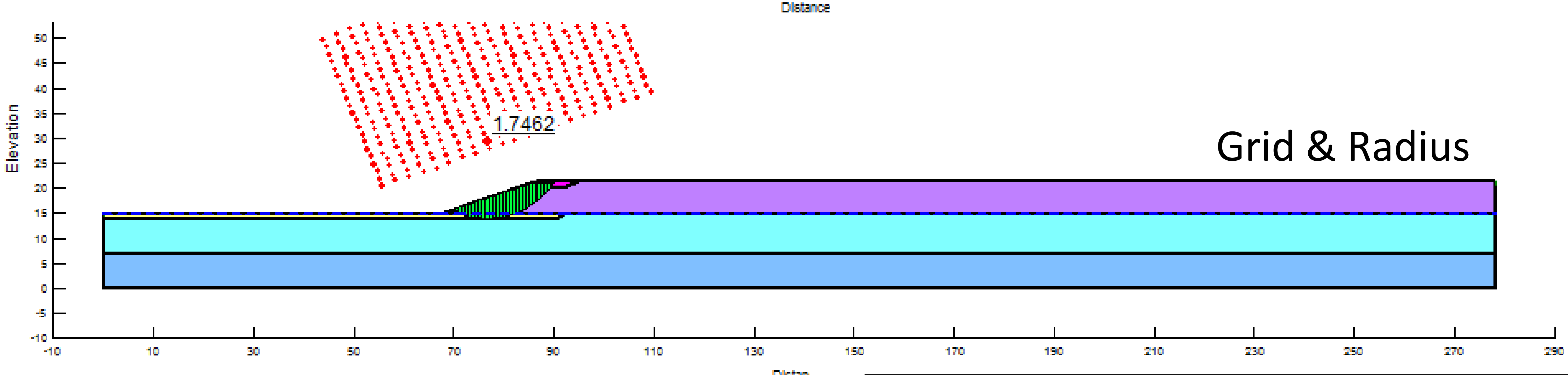
Grid & Radius

srk consulting	TMAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 · Approved: SA · Figure: **A1 - 10**

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

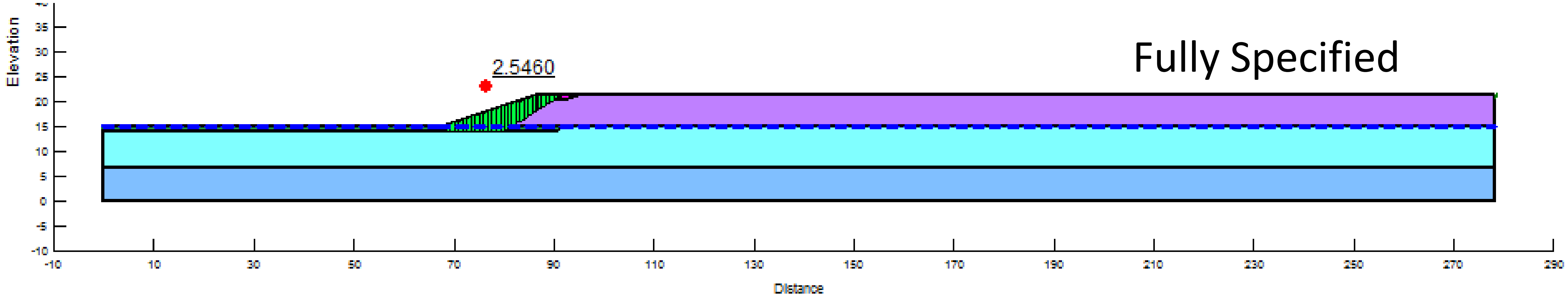


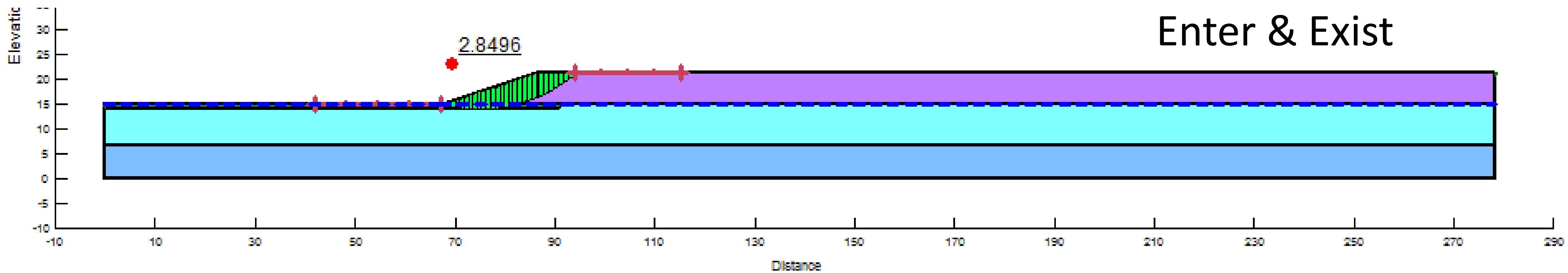


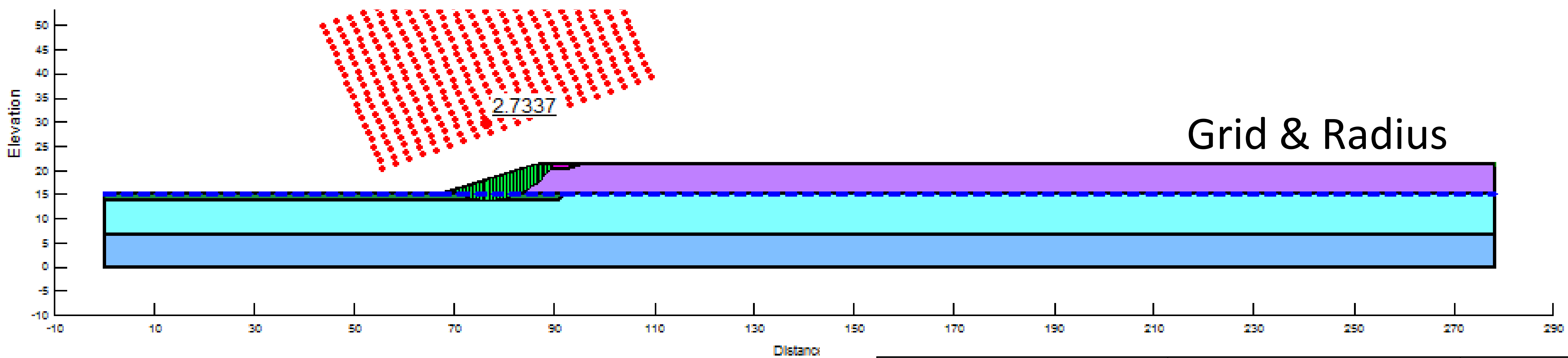


srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Analyzed Cross-Section
		Date: November 2016 / Approved: SA / Figure: A1 - 11

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



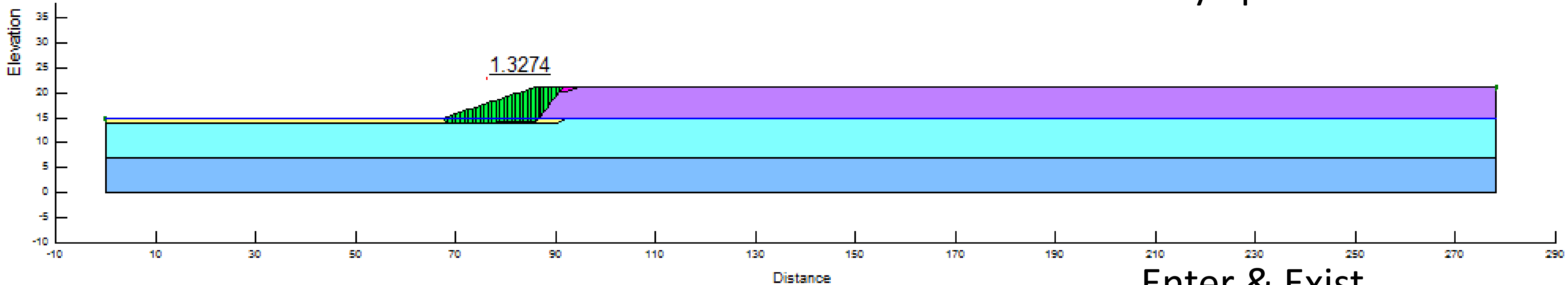




srk consulting	MAC RESOURCES	BOSTON TMA SLOPE STABILITY
		Analyzed Cross-Section
Job No: 1CT22.004.620 Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA	HOPE BAY PROJECT	Date: November 2016 / Approved: SA / Figure: **A1 - 12**

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

Fully Specified



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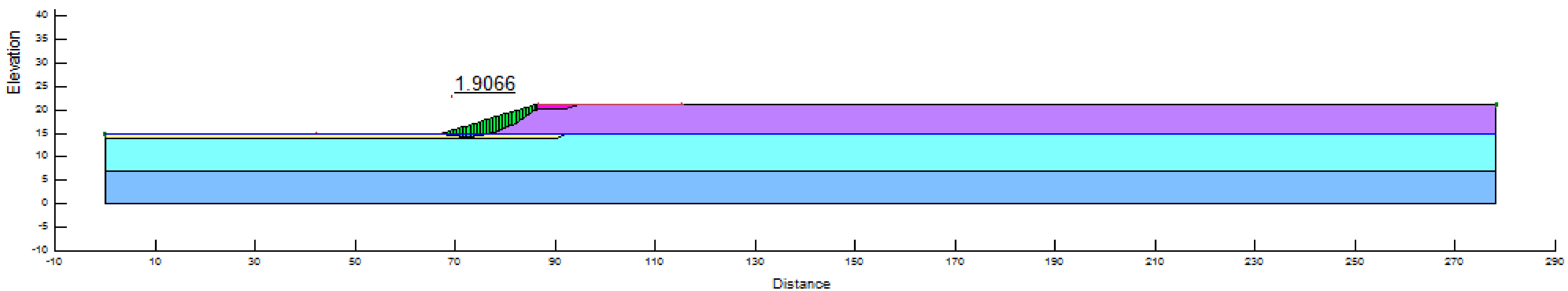

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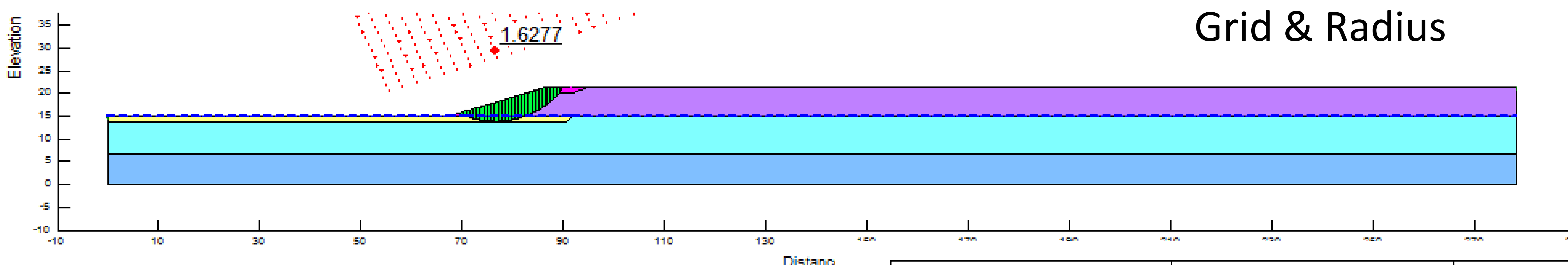

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MAC RESOURCES

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

Job No: 1CT22.004.620

Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA

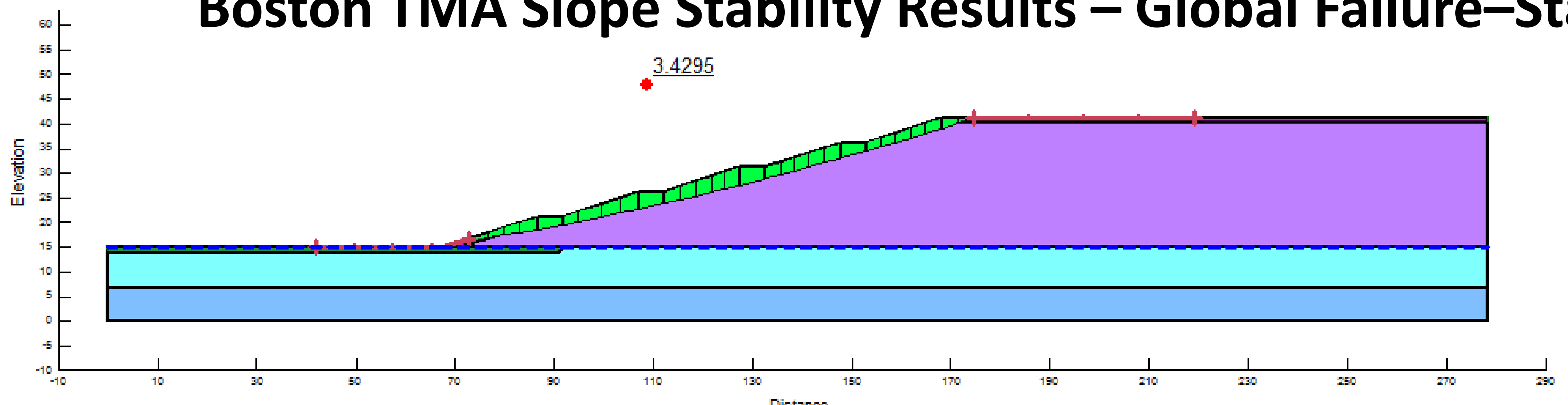
HOPE BAY PROJECT

Date: November 2016

Approved: SA

Figure: **A1 - 13**

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



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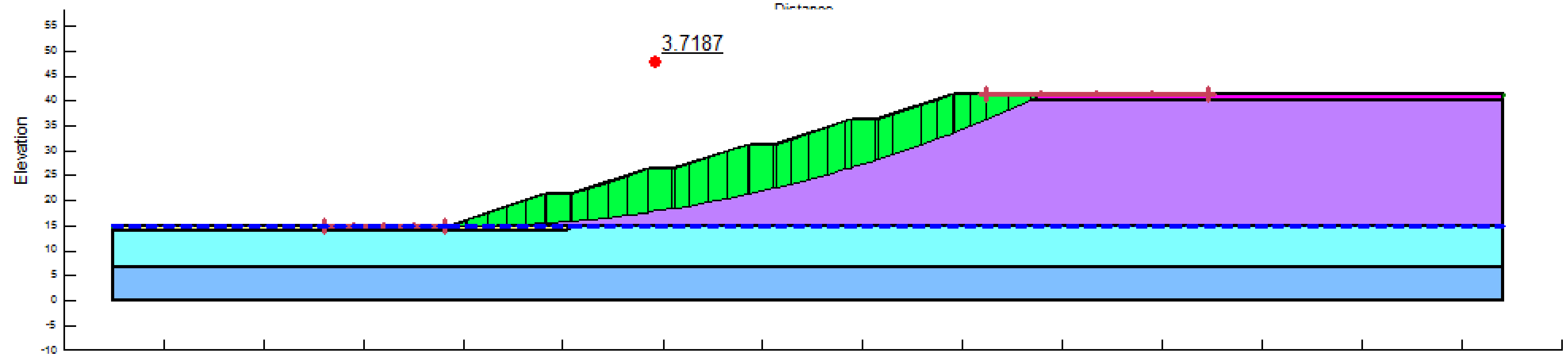

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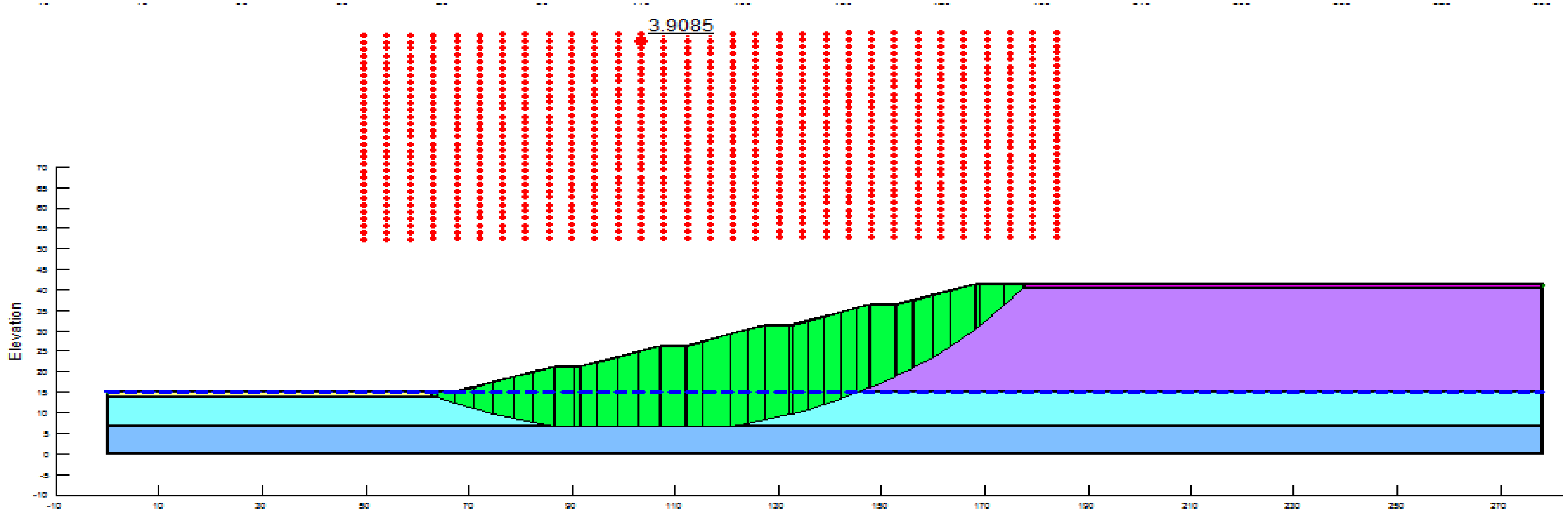

Grid & Radius

srk consulting

MAC RESOURCES

BOSTON TMA SLOPE STABILITY

Analyzed Cross-Section

Job No: 1CT22.004.620

Filename: HopeBay_Boston_DSTSF_SlopeStability_Results_CH_RevA

HOPE BAY PROJECT

Date:	Approved:	Figure:
November 2016	SA	A1 - 14

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

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Christopher W. Stevens, PhD	**Project No:**	1CT022.004
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	December 1, 2016
Subject:	Hope Bay Project: Boston Tailings Management Area Thermal Modelling		

1 Introduction

1.1 General

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

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

1.2 Objectives

The objective of the modelling was to estimate active layer thickness in the 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, 2016a). 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, 2016b). 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, 2016c). 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, 2016d). 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, 2016c) 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, 2016e).

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 (2016d). 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 consider to be 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 facilities range 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, 2016c). 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, 2016a).

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 five years during the 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 of the climate boundary applied to the model. As the near-surface 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 thaw from the surface of the ROQ cover. Thaw of tailings below the cover is also provided, and calculated as:

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

Where:

T_{Thaw} *is the 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 which relate to the increase in air temperature from climate change.

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 estimate 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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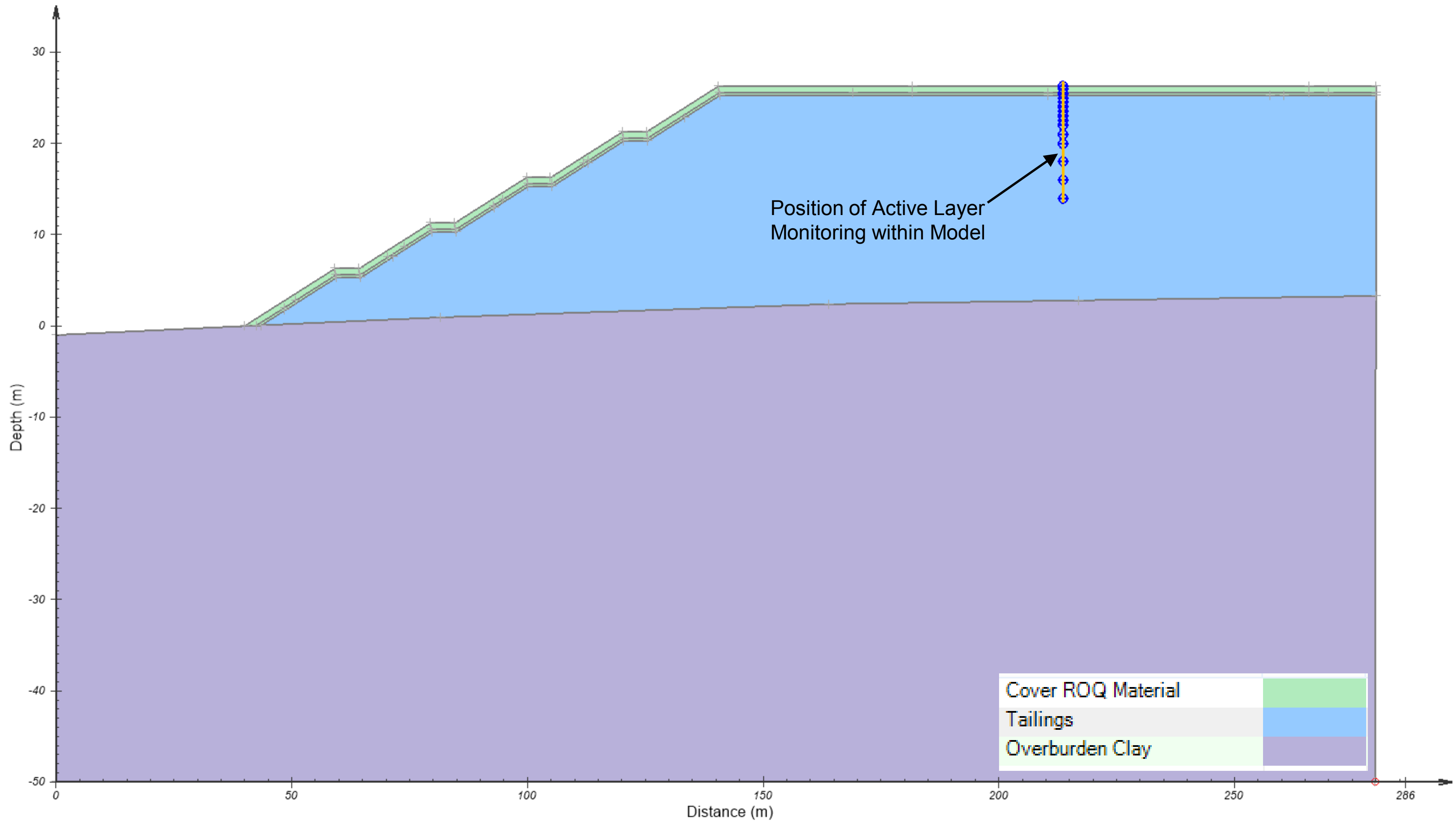
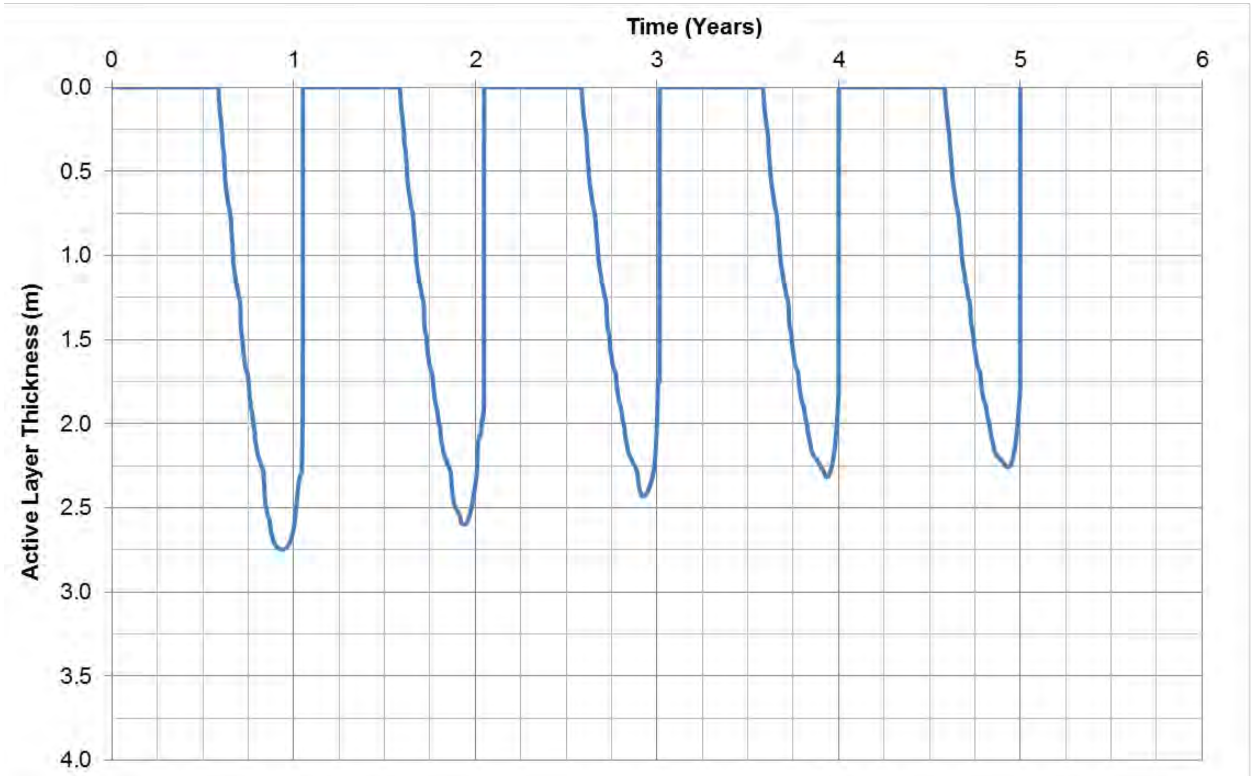
Figures



srk consulting	TMAC RESOURCES	Boston Dry Stack Thermal Modeling
		Boston Dry Stack Model Domain
Job No: 1CT022.004 Filename: BostonDryStack.pptx	HOPE BAY PROJECT	Date: 6/14/2016 — Approved: cws — Figure: 1

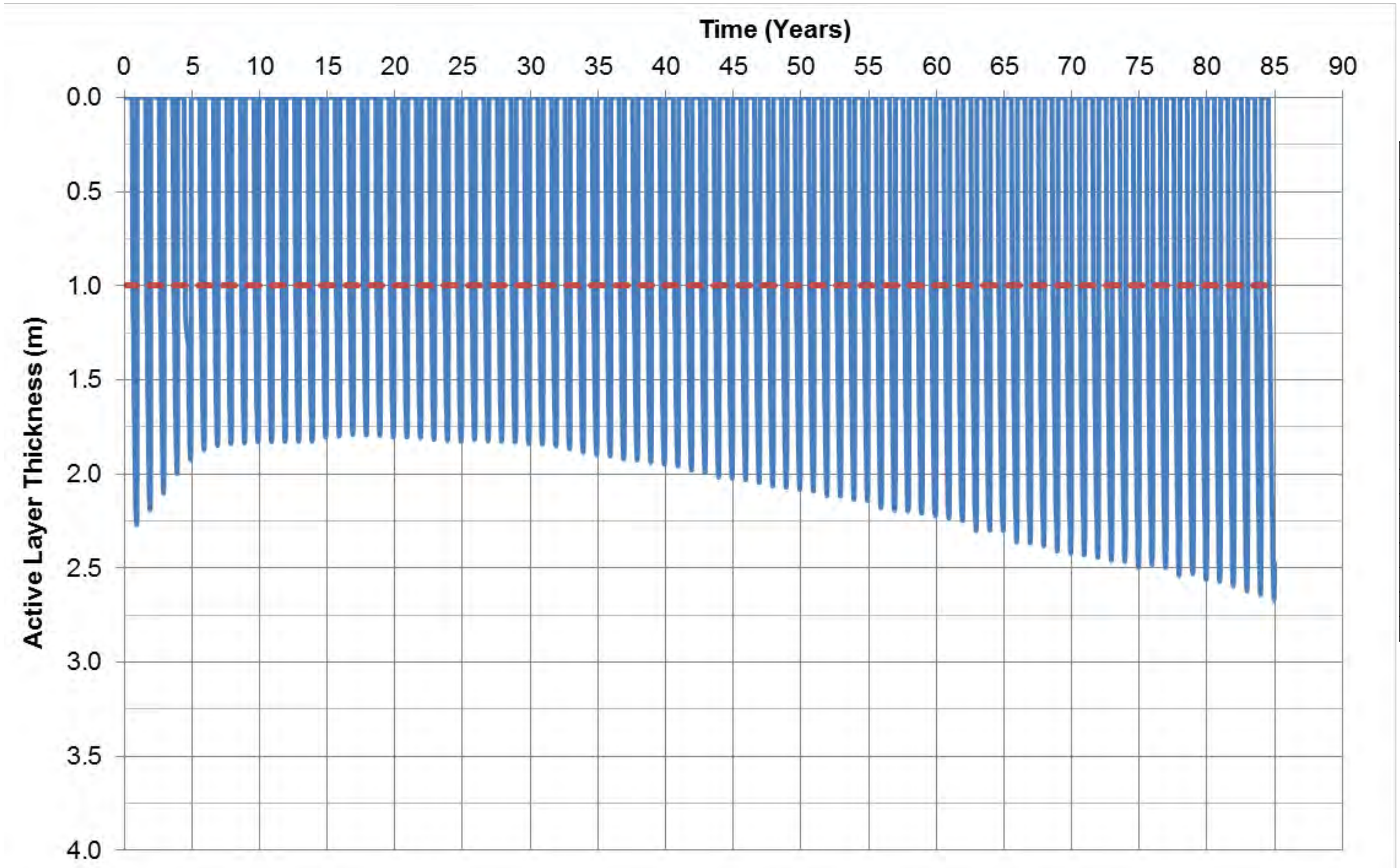


Year	Active Layer, Thaw of Tailings (m)
1	2.7
2	2.6
3	2.4
4	2.3
5	2.3

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

srk consulting	TMAC RESOURCES	Boston Dry Stack Thermal Modeling
		Active Layer Thickness – Tailings Saturation 49% (Period of Operation)
Job No: 1CT022.004 Filename: BostonDryStack.pptx	HOPE BAY PROJECT	Date: 6/14/2016 · 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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TMAC RESOURCES

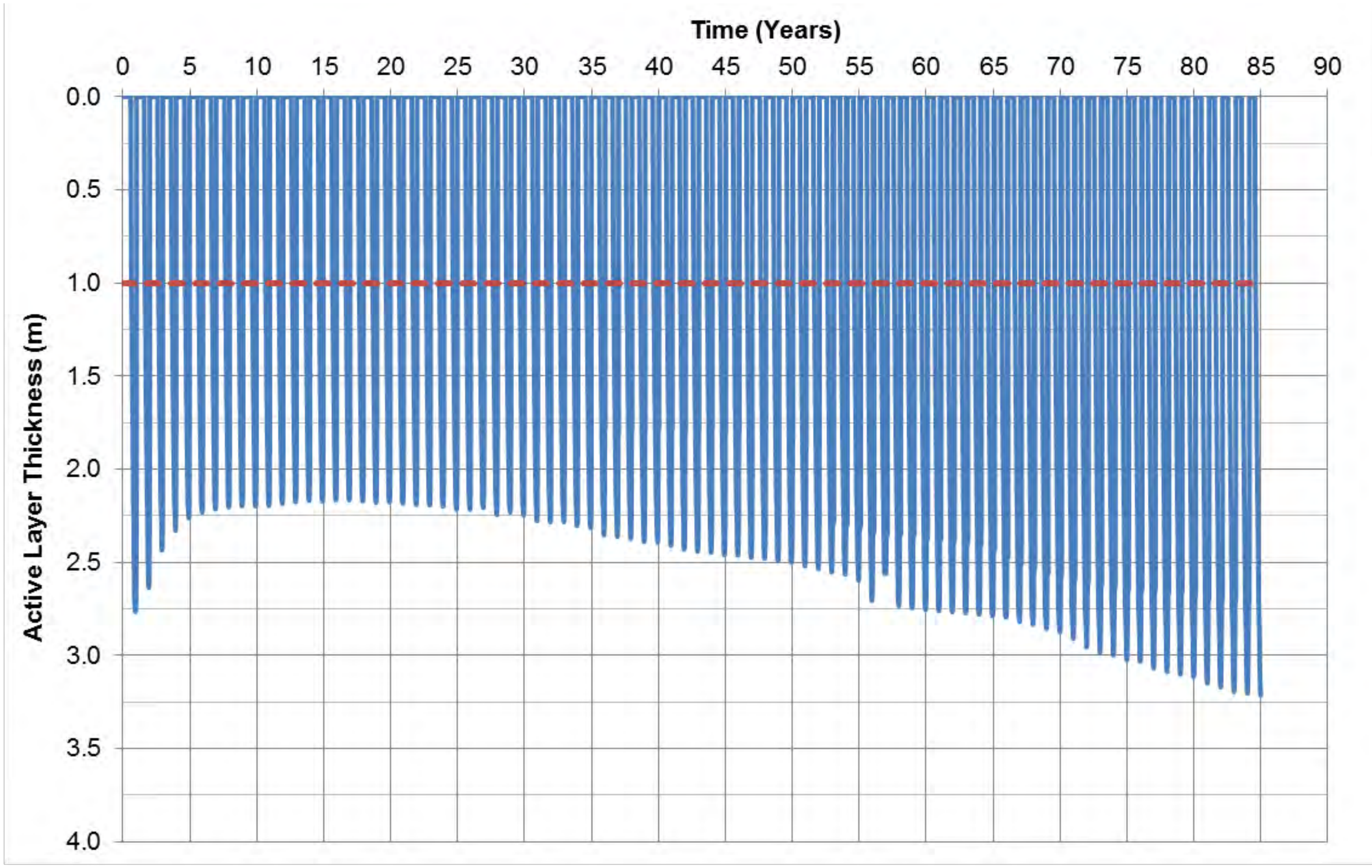
Boston Dry Stack Thermal Modeling

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

Job No: 1CT022.004
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Date: 6/14/2016 | 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

srk consulting | TMAC RESOURCES | Boston Dry Stack Thermal Modeling

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

Job No: 1CT022.004
Filename: BostonDryStack.pptx

HOPE BAY PROJECT

Date: 6/14/2016 | Approved: cws | Figure: 4

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

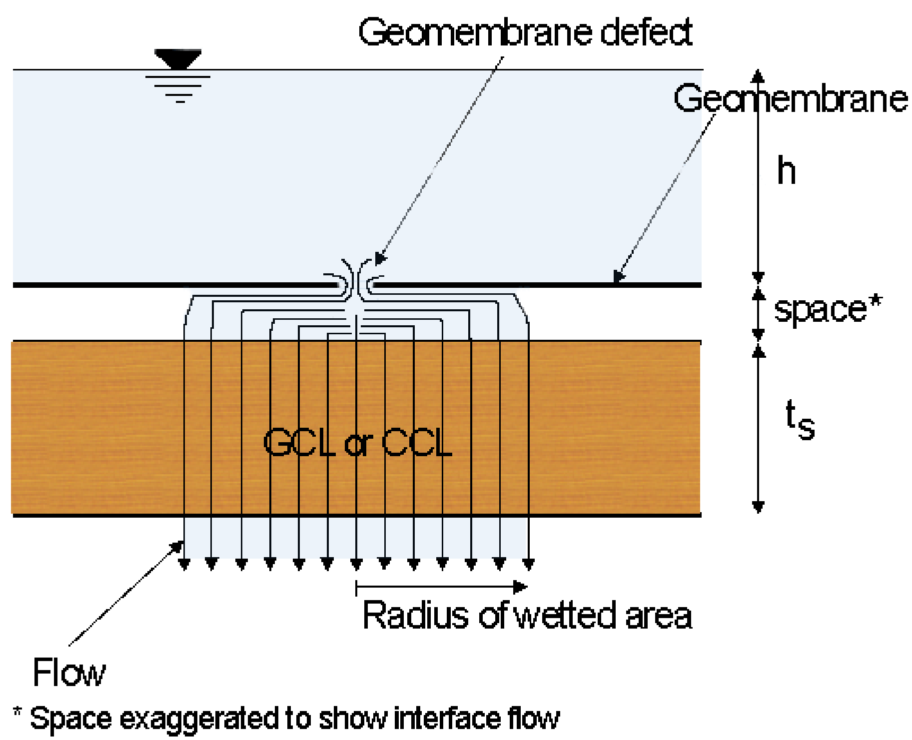
Design Calculator

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 D – Hope Bay Project: Tailings Area Dust Control Strategy

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Memo

To:	Project File	**Client:**	TMAC Resources Inc.
From:	Iozsef Miskolczi, PEng	**Project No:**	1CT022.004
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	December 13, 2016
Subject:	Hope Bay Project: Tailings Area Dust Control Strategy for Boston TMA		

1 Introduction

The Hope Bay Project (the Project) is a gold mining and milling undertaking of TMAC Resources Inc. (TMAC). 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), currently being carried out under an existing Water Licence, and Phase 2 is in the environmental assessment stage. Phase 1 includes mining and infrastructure at Doris, while Phase 2 includes mining and infrastructure at Madrid and Boston located approximately 10 and 60 km due south from Doris, respectively.

Two tailings 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 92-94% of the overall volume, and detoxified leach tailings (following cyanidation, and subsequent cyanide destruction), comprising about 6-8% of the overall volume. Only flotation tailings will be deposited in the 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 deposition 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 as a result 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. Although tailings are discharged wet, the surface eventually dries out as a result of evaporation or freezing of the tailings surface. As a result, at any given time, large areas of the tailings surface would expose dry tailings. Aeolian tailings dust is expected because the Project site is prone to high winds and the moderate surrounding topography does not offer effective protection from wind.

3 Typical Dust Control Methods

3.1 State of Practice

Dust control from operating and closed tailings impoundments is a significant concern in the mining industry, and as a result, the state of practice is quite advanced. There are three primary dust control strategies for fugitive and aeolian dust from exposed tailings areas: natural dust control, physical dust control and chemical dust control. Natural dust control specifically relies on maximizing the benefits offered by nature in the form of precipitation (rain and snow). While highly effective, these benefits are opportunistic and may not always be available at the times when it may be needed.

Physical dust control is by far the most effective strategy, as it relies on creating a physical barrier, such as a cover, that would preclude dusting. This may however not be a cost efficient strategy for an operating tailings impoundment, since any interim cover would occupy space within a tailings impoundment that would otherwise be required for tailings.

Chemical dust control relies on modification of the tailings surface that generates the dust. The effectiveness of this method is temporary, but its application is typically simple, making it a very good alternative for managing dust from an operating tailings impoundment.

The sections that follow provide a detailed description of all the dust control methods that are currently being used in the industry, with a specific focus towards their potential applicability for this Project.

3.2 Natural Methods

3.2.1 Snow Cover

If early in the fall season, wet snow falls directly on the exposed tailings surface and subsequently freezes, it will remain in place all winter protecting the tailing surface from dusting. Snow that falls later in the season is typically drier and more powdery and it tends to be subject to wind transport and redistribution (drifting). This means that portions of the tailings surface will become exposed and opportunity for dust release increases. This is exacerbated by the fact that during the winter the tailings surface gets extremely dry as a result of freezing, making it highly susceptible to dusting.

To maximize the potential benefits offered by snow as a natural dust control method, any snow that does fall on the tailings surface can be track. By mechanically compacting the snow, it will stay in place longer and will melt at a much slower rate in the spring, extending the useful life of the snow as a dust control method.

It is however important to minimize the amount of tailings that gets deposited over the compacted snow. If the compacted snow does not melt during the subsequent summer season due to the insulating blanket of the overlying tailings, ice lenses within the tailings impoundment are created which result in a loss of tailings storage space and possible instability.

There is sufficient snowfall at the Project site that this dust control method could be effectively used. In addition, there is a requirement on site for snow removal in specific areas. Snow that is removed could be hauled to the TMA and used specifically for the purpose of creating a compacted snow cover over any temporarily inactive tailings surface areas. Due to the temporary nature of this dust control method, it will not be a complete solution, but would be a practical complementary dust control method.

3.2.2 Ice Cover

Similar to compacted snow, an ice cover will remain in place for the duration of the winter and thus temporarily mitigate dust migration. Ice cover on exposed tailing surfaces can be achieved by various methods, including ponding water during freezing weather and mechanical placement of ice blocks imported from a different source (contact water ponds).

Water can be held back in specified locations and retained there during the shoulder seasons when freezing weather will create an ice cap. Once the ice cap is achieved the open water beneath the ice can be drained off, leaving an ice cap.

The ice cap can also be created mechanically by loading ice from contact water ponds (or fresh water streams) into haul trucks and dumping the ice on the tailings surface.

Similar to compacted snow, care must be taken to ensure that the amount of tailings deposited over an ice cover is limited to avoid entraining long-term ice in the TMA.

There are several contact water ponds throughout the Project site all of which must be managed such that they are normally empty. Contact water ponds are, therefore, unable to provide a reliable source of water to use to create an ice cover. Fresh water cannot be readily hauled to the TMA, to create an ice cover, as the use of fresh water is governed by the Water License. Therefore creating an ice cover for dust control is not considered a viable practical alternative for application at the Project.

3.3 Physical Methods

3.3.1 Water – Surface Wetting

Water is by far the most common temporary dust control measure used in areas where water shortage is not of concern. The exposed surface is wetted up, preventing particles from becoming airborne. Since the water rapidly evaporates (in a matter of hours or days), it needs to be reapplied at a frequent interval to be effective. The surface wetting can be done using a conventional water truck, a water cannon fitted to a water truck, or a stationary sprinkler system. Naturally this dust control method is only applicable during non-freezing periods of the year.

For the Project, water could readily be obtained from the mill or can be hauled via water truck from other site contact water ponds. 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 such time as the tailings surface has reached its final elevation. In order to facilitate placement of a final dust cover as expediently as possible, any tailings deposition plan should be designed taking into consideration all opportunities for progressive reclamation.

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 deposition might resume at any particular spot may be extensive. When tailings deposition eventually returns to the covered area, these materials are not removed and tailings deposition proceeds to overtop the sacrificial cover. This 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 of the Project’s prevalent wind directions so as 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 Tailings

The primary dust control measures of the Project site tailings facilities will be the use of environmentally suitable chemical dust suppressants. The application of these suppressants will be reviewed on an ongoing basis to ensure that any areas that may be at risk will be adequately covered. Generally, annual application of chemical suppressants will be applied; however it is recognized that more frequent applications may be required as 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.