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Package P5-5

Hope Bay Project Geotechnical Design Parameters and Overburden Summary Report

TMAC RESOURCES

Hope Bay Project Geotechnical Design Parameters and Overburden Summary Report

Prepared for

TMAC Resources Inc.

Prepared by

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

Hope Bay Project
Geotechnical Design Parameters and Overburden Summary Report

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Change Log

This following table provides an overview of material changes to this report from the previous version issued on Appendix V3-2E 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 made		

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.

1.1.2 Site History

The Hope Bay Greenstone Belt was first discovered in 1962 by geologists from the Geological Survey of Canada. Exploration of the area by various companies has been ongoing ever since.

BHP Minerals Canada Inc. performed exploration from 1988 to 1999, which included 2,300 m of underground development at the Boston mining area for bulk sampling and underground exploration. Hope Bay Mining Limited started underground development of the Doris Mine in 2011. TMAC acquired the property in 2012, and is currently developing the Doris Mine.

1.1.3 Proposed Site Development

Phase 2 will consist of the development of underground mines at Madrid and Boston, continued use of infrastructure at Doris and expansion of some infrastructure at Doris. Phase 2 infrastructure will include:

- Doris Tailings Impoundment Area (TIA), which includes the North, South and West dams;
- Boston Tailings Management Area (TMA);
- Ore and waste rock piles and associated water management infrastructure;
- Madrid-Boston All-weather Road;
- Infrastructure pads, and access roads;
- Boston airstrip; and
- Quarries, to support development.

1.2 Scope of Work

The scope of this document is to provide a comprehensive listing of all overburden, permafrost and borrow investigations performed on site and summarize typical overburden and borrow material properties and permafrost characteristics. Additionally, the report will lay out general geotechnical design principals for the proposed infrastructure and discuss the expected foundation conditions. Fundamental parameters for geotechnical design are also provided.

The fundamental parameters contained in this report are independent of the intended use by other engineering disciplines. For example, estimates of settlement are not provided as settlement is a function of the geometry and load, which is unknown to SRK. However, the geotechnical parameters provided will allow calculation of settlement once the structure geometry and loads are known.

While this report is intended to be a comprehensive summary of material properties and design parameters, these values should only be used when site specific data is not available.

1.3 Report Layout

This report is broken down into four sections including this introduction. Section 2 summarizes general site conditions and provides a comprehensive history of characterization programs that have been performed on site. Section 3 describes the geotechnical design principals associated with the proposed site development, and summarizes foundation conditions expected under the proposed infrastructure components. Section 4 provides geotechnical parameters to be used in design.

2 General Site Conditions

2.1 Regional Geology

The Hope Bay Volcanic Belt (the Belt) is a mafic volcanic dominated greenstone belt located in the northeast portion of the Slave Structural Province. The Belt is typified by massive to pillowed tholeiitic flows interbedded with calc-alkaline felsic volcanic and volcaniclastic rocks, clastic sedimentary rocks, and rarely synvolcanic conglomerate and carbonates. Rock outcrop mapping can be seen in Figure 2 and Figure 3.

During the last Quaternary Period, the region was subjected to multiple glaciations. Ice flows were predominantly towards the north-northwest and north, and the melting ice sheets left an extensive blanket of basal till. Immediately following the de-glaciation, the entire region was submerged approximately 200 metres below present mean sea level (Dyke and Dredge, 1986). Fine sediment, derived from meltwater (rock flour), was deposited onto the submerged Hope Bay shelf as marine clays and silts onto the basal tills. The greatest thicknesses accumulated in the deeper water zones, now represented by valleys.

Isostatic rebound after the de-glaciation resulted in emergent landforms and reworking of the unconsolidated marine sediments and tills along the prograding shoreface (EBA, 1996). Sediments were easily stripped off the uplands and redeposited in valleys, leaving relatively continuous north-northwest trending bedrock ridges and elongate lakes.

2.2 Regional Seismicity

The Project is located in the lowest category seismic hazard zone of Canada in accordance with the 2015 National Building Code of Canada seismic hazard maps (NRC, 2015). The seismic hazard is described by spectral-acceleration (Sa) values at periods of 0.05, 0.1, 0.2, 0.3, 0.5, 1.0, 2.0, 5.0 and 10.0 seconds, as well as the peak ground acceleration (PGA) and peak ground velocity (PGV). Spectral acceleration is a measure of ground motion that takes into account the sustained shaking energy at a particular period; however, PGA is the parameter considered for foundation design.

Ground motions for the Project are presented in Table 1 for probabilities associated with return periods of 1:100 years, 1:476 years, 1:1,000 years and 1:2,475 years. These ground motions are the values in the National Building Code (NRC, 2015), and need to be adjusted for site specific ground type, prior to being used in design. This analysis is provided in Appendix B, and described in Section 4.7.

Table 1: National Building Code Ground Motions for the Project(1)(2)

Spectral Period (s) or Peak Parameter	Ground Accelerations (g)			
	1:100 year	1:476 year	1:1000 year	1:2475 year
Sa(0.05)	0.0034	0.012	0.021	0.042
Sa(0.1)	0.0056	0.019	0.031	0.059
Sa(0.2)	0.0069	0.021	0.032	0.056
Sa(0.3)	0.0065	0.019	0.029	0.047
Sa(0.5)	0.0051	0.017	0.025	0.038
Sa(1.0)	0.0026	0.0096	0.015	0.023
Sa(2.0)	0.0010	0.004	0.0064	0.011
Sa(5.0)	0.0004	0.0009	0.0014	0.0023
Sa(10.0)	0.0003	0.0006	0.0008	0.0011
PGA	0.0033	0.011	0.017	0.032

Source: NRC 2015

Note(s):

(1) Ground motions provided are for Site Class C (very dense soil and soft rock), ground motions for other material types should be calculated as described in the National Building Code of Canada (NRC, 2015).

(2) Ground motions for the Doris, Boston and Madrid mining areas were the same; therefore, these ground motions apply to the entire site.

2.3 Overburden Characteristics

2.3.1 Overburden Characterization Studies

Numerous overburden studies have been conducted on the site. Table 3 provides details on drilling and testing pitting programs, while Table 2 summarizes surficial mapping studies and provides information on laboratory testing. Surficial geology and permafrost features can be seen in Figure 4 and Figure 5. Locations of geotechnical drill holes and thermistor installations can be seen in Figure 6 through Figure 9.

Table 2: Summary of Historic Reconnaissance and Surficial Mapping Studies

Program	Area	Description
Summer 1992 (Ryder, 1992)	Boston Mining Area	Terrain analysis and surficial geology mapping (air photo interpretation and field verification) to support exploration work. The geographical extent of study is not clear as the mapping has been completed by hand and unique geographical identifiers were not provided.
Summer 1993 (EBA, 1993)	Hope Bay Belt	Field reconnaissance (samples of esker sand collected and submitted for petrographic analysis), to support early development of winter and all-weather road route selections between the Boston mining area and Roberts Bay. Borrow sources for road construction was identified.
Summer 1996 (EBA, 1996)	Boston Mining Area	Surficial geology and permafrost feature mapping (air photo interpretation and field verification) to support mine infrastructure development. Work focused on peninsula where the Boston deposit is located. This map has been recreated in SRK (2002a).
Summer 1997 (EBA, 1998)	Hope Bay Belt	Field reconnaissance and air photo interpretation. A continuation of the Summer 1993 work to support development of a road between Boston and Roberts Bay. One all-weather road route and two candidate winter-road routes were identified. Portages and potential quarries were identified, inspected and mapped.
Summer 2002 (SRK, 2002a)	Doris Mining Area	Field reconnaissance, to evaluate ground features at the Doris mining area, including the planned port, camp and Tail Lake area.
2001 & 2002 (Sherlock, 2002)	Doris and Madrid Mining Areas	Bedrock mapping (bedrock outcrop and major structural features mapping) to support mineral exploration.
Fall 2003 (Thurber, 2003)	Doris Mining Area	Surficial geologic mapping (air photo interpretation and field verification) to support mine infrastructure development engineering.
Winter 2008 (SRK, 2009)	Doris and Madrid Mining Areas	Surficial geologic mapping (air photo interpretation), an extension of the mapping carried out in the Fall of 2003, to support infrastructure development.

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Table 3: Summary of Drilling and Test Pitting Programs

Program	Area	Description
Winter 1996 (EBA, 1996)	Boston	Six onshore drill holes One offshore drill hole (Stickleback Lake) Three 15 m long thermistor strings installed One 250 m long thermistor string installed Laboratory testing performed(1)
Winter 1997(2) (EBA, 1997)	Roberts Bay	Six onshore drill holes Four offshore drill holes(2), and eight offshore probe holes (Roberts Bay) Six 15 m long thermistor strings installed Laboratory testing performed(1)
2002 (SRK, 2002b)	Doris TIA and Doris	Ten onshore drill holes Six offshore drill holes (former Tail Lake) Three 10 m long thermistor strings installed One standpipe piezometer installed Falling head permeability tests were performed in three boreholes Laboratory testing performed(1)

Program	Area	Description
2003 (SRK, 2003)	Doris TIA	Five onshore drill holes, six hand dug test pits and three hand auger holes, North Dam footprint Seven onshore drill holes around the former Tail Lake Six onshore drill holes, South Dam footprint Twelve thermistors strings installed Falling head permeability tests were performed in four holes Laboratory testing performed(1)
Summer 2004 (SRK, 2005a)	Doris TIA	One onshore drill hole, North Dam footprint Three onshore drill holes around the former Tail Lake Four thermistor strings installed Laboratory testing performed(1)
Winter 2004 (SRK, 2005a)	Doris TIA	Two hand dug bulk samples at North Dam footprint One 200 m deep onshore drill hole Laboratory testing performed(1)
Winter 2004 (SRK, 2004)	Roberts Bay	Four offshore drill holes (Roberts Bay) Laboratory testing performed(1)
Winter 2005 (SRK, 2005b and 2005e)	Doris TIA	Four onshore drill holes North Dam footprint One onshore drill hole at alternate North Dam location Three onshore drill holes around the former Tail Lake Six thermistor strings installed Laboratory testing performed(1)
Winter 2005 (SRK, 2005c)	Roberts Bay	Six offshore borings with vane-shear apparatus (5 tests per hole)
Winter 2006 (SRK, 2006a)	Doris TIA	Two onshore drill holes, South Dam footprint Laboratory testing performed(1) Geophysics performed around entire perimeter of lake (former Tail Lake)
Winter 2006 (SRK, 2006b)	Roberts Bay	Seven offshore drill holes Laboratory testing performed(1)
Winter 2007 (SRK, 2009)	Patch Lake	Nine offshore drill holes, Patch Lake Four onshore drill holes, Madrid area
Winter 2008 (SRK, 2009)	Doris, Patch and Aimaokatalok Lakes	Four offshore CPT holes, Doris Lake Nine offshore CPT holes, Patch Lake Five offshore CPT holes, Aimaokatalok Lake Vane shear testing in select holes adjacent to the CPT holes Pore pressure dissipation testing performed Laboratory testing performed(1)
Winter 2010 (SRK, 2010)	Doris / Roberts Bay	Thirty onshore drill holes to identify depth to bedrock (holes not logged) Eight offshore drill holes (Roberts Bay) to determine depth to bedrock
Winter 2011 (SRK, 2012a)	Doris TIA	Twenty-three onshore percolation drill hole within North Dam footprint Laboratory testing performed(1)
May 2016(3) (SRK)	TIA	Five offshore CPT holes with four twinned drill holes (Former Tail Lake) Laboratory testing will be performed(3)

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Note(s):

(1) Details of laboratory testing provided in Table 4.

(2) It should be noted that when looking at this report that laboratory testing data and borehole logs for offshore holes assign the mudline a depth of 0 m.

(3) Drill program included for completeness, results from this program (including field report, CPT results, and laboratory testing) are not available at the time of writing this report.

Table 4: Summary of Laboratory Testing

Program	Moisture Content	Particle Size Distribution	Specific Gravity	Atterberg Limit	Unconsolidated Undrained Triaxial	Multi - Stage Consolidated	Salinity	Thermal Conductivity	Other
Winter 1996 (EBA, 1996)	40	17		6			5		
Winter 1997 (EBA, 1997)	15	11		9	3				Intact bulk density
2002 (SRK, 2002b)	13			5					
2003 (SRK, 2003)	28			16			10		
Summer 2004 (SRK, 2005a)	17		3	9			6	2	Porosity, saturation, bulk density, minerology, unfrozen water content
Winter 2004 (SRK, 2004)	6	5		5					
Winter 2005 (SRK, 2005b)	43		6	9			16	3	Intact bulk density, saturation, unfrozen water content
Winter 2005 (SRK, 2005c)									No lab testing, but vane shear testing performed
Shoreline Erosion Study (SRK, 2005d)	4			4					
Winter 2006 (SRK, 2006a)	7			6			7		
Winter 2006 (SRK, 2006b)	13	13	1	11	2		1		Bulk density, saturation, consolidation
Winter 2007 (SRK, 2009)	11	10		7					
Winter 2008 (SRK, 2009)	6	6	3	3		3			Direct shear test, bulk density, shrinkage limit, shrinkage ratio, consolidation
Winter 2011 (SRK, 2012)(1)	232						54		
Winter 2016 (SRK)(2)	9	9		9	2				Consolidation

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Note(s):

(1) The winter 2011 laboratory testing described in the table only includes the testing performed on overburden materials.

(2) Drill program included for completeness, results from this program (including field report, CPT results, and laboratory testing) are not available at the time of writing this report.

Table 5 summarizes the on-site installations, excluding the thermistor strings. Due to the large number of thermistor strings installed, a summary table is provided in Appendix A. Though a standpipe piezometer was installed in 2002, no data is available from this installation.

Table 5: Summary of Installations(1)

Program	ID	Instrument	Area	Installation Date	Northing (m)	Easting (m)
2002 (SRK, 2002b)	SRK12	Standpipe Piezometer	Doris Mining Area	2002	7559154	434380
Westbay Program (SRK, 2011a)	10WBW001	Westbay multi-port monitoring well	Doris Mining Area	2010	7557537	433778
	10WBW002	Westbay multi-port monitoring well	Doris Mining Area	2010	7559375	433913
	10WBW004	Westbay multi-port monitoring well	Boston Mining Area	2010	7505665	441018

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Note(s):

(1) Installations listed above do not include the thermistor cables installed throughout the site. Thermistor cable installation details are provided in Appendix A.

2.3.2 Onshore Overburden Characteristics

The overburden profile consists of a thin veneer of poorly drained, hummocky organic soil covered by tundra heath vegetation. Under this organic zone is a layer of marine clay and silt (silty clay and clayey silt) typically between 5 and 20 m thick, underlain with pockets of gravelly moraine till. Since the terrain is glaciated with significant bedrock control, overburden thickness can range from less than 5 m to over 30 m. The marine silts and clays contain significant ground ice (10 to 30% by volume on average, but occasionally as high as 50%), whilst the till contain low to moderate ice contents (5 to 25%). The till contains small to moderate amounts of cobbles and boulders. The bedrock contact zone generally consists of a small rubble zone ranging from a few centimetres to up to 2 m in thickness.

The overburden soil pore water typically has high salinity concentrations, often exceeding that of seawater. This has the effect of depressing the freezing point, as well as contributing towards high unfrozen water content. Notwithstanding, in-situ hydraulic conductivity of these soils is low, both in the frozen and unfrozen state. The overburden soils are normally consolidated, and the state of the soil, i.e. whether it is frozen or thawed defines its apparent strength. The expected salinity on site is described in more detail in Section 2.4.3, and the permafrost characteristics are described in Section 2.4.2.

2.3.3 Offshore (Lake) Overburden Characteristics

The overburden soils under Doris, Patch and Aimaokatalok Lakes are of the same origin as the onshore overburden soils (i.e. marine silty clays and clayey silts). In all three lakes there is a layer of unconsolidated sediments ranging from a few centimetres to 2 m thick. Under the Doris and Patch lakes the remainder of the overburden consists of a normally consolidated layer of marine silty clay and clayey silts between 10 and 20 m thick (Figure 10 and Figure 11). Beneath Aimaokatalok Lake, the layer of unconsolidated material is generally less than 10 m thick, and in places the bedrock contact zone, which consists of boulder sized frost shattered bedrock fragments, extends to near surface. The permafrost characteristics under these lakes is described in more detail in SRK (2017a).

2.3.4 Offshore (Roberts Bay) Overburden Characteristics

The sediments in Roberts Bay appear to be of similar origin to those in the rest of the Hope Bay Belt. Some submarine permafrost is present along the southern shore of Roberts Bay, where a 100 m long shallow shelf is present. Drilling data suggests that submarine permafrost is present to a distance of about 60 m from the shore, which corresponds to an average water depth of about 1 m. Submarine permafrost does not exist in the rest of the bay, and the total depth of Roberts Bay sediments is up to 20 m. In-situ vane shear testing confirms that these sediments have low strength, and exhibit properties of moderately sensitive clay, not dissimilar to the offshore conditions under Doris, Patch and Aimaokatalok Lakes (SRK, 2009).

2.3.5 Overburden Isopachs

Overburden isopachs were developed for the Doris, Madrid and Boston mining areas; these isopachs can be seen in Figure 10, Figure 11 and Figure 12 respectively. These isopachs were developed using all available drill hole data, including depth of overburden from TMAC's exploration drill hole database. Three lines of seismic data from Frontier Geosciences Inc. (1998) were also used to develop the Boston isopach. Where overburden isopachs are under lakes, the 2006 bathymetric survey data was used as the top surface (Golder, 2006).

In 2010 a drill program was conducted in the Doris Camp area to refine and prove the existing overburden isopach, a similar drill program has not been performed in the Boston or Madrid areas.

2.4 Permafrost

2.4.1 Permafrost Characterization Studies

Permafrost characterization of the Project includes permafrost mapping (Table 2, Figure 4 and Figure 5), numerous field programs (Table 3), and the installation of thermistor cables. A full listing of the thermistor cables installed at the Project, including their location, status and range of available data is provided in Appendix A.

2.4.2 Permafrost Characteristics

The Project site is located within the continuous zone of permafrost. The permafrost at the Project site is estimated to be approximately 570 m thick, with permafrost 500 m thick in the Doris mining area, 570 m thick in the Madrid mining area and 565 m thick in the Boston mining area (SRK, 2017a). The geothermal gradient is estimated to be 0.021°C m^{-1}. The active layer in overburden soils ranges between 0.5 m and 1.4 m, with an average depth of 0.9 m. Baseline ground temperatures collected at the Project indicates a range of permafrost temperatures from −5.6°C to −9.8°C, with an average temperature of −7.6°C (Appendix C). The typical ground temperature trumpet curve for the Project is shown in Figure 13.

Climate change and the predicted increase in air temperatures at the Project are expected to affect permafrost characteristics. While the Project is predicted to stay in the zone of continuous permafrost (ACIASC, 2005), the region is predicted to be thermally sensitive to climate change (Smith and Burgess, 2004). Climate change, specifically warmer summer temperatures, are expected to increase the active layer thickness. By 2100, the active layer of clay overburden is estimated to increase by 0.93 m (SRK, 2017b).

2.4.3 Salinity and Freezing Point Depression

The freezing point of the permafrost overburden on site is depressed due to the high salinity of the overburden porewater. Table 6 provides a summary of the salinity measurements and associated freezing point depressions, without differentiating for material type. Site wide salinity measurements range from 1 to 162 ppt, with an average of 37 ppt. These salinities correspond to freezing point depressions ranging from −10.5°C to 0.0°C, with an average of −2.1°C.

Typically the salinity of the marine silt and clay deposits is much higher than that measured in the sand deposits (Table 7). Measured salinity for the silt and clay overburden ranges from 162 to 0.5 ppt, with an average of 39 ppt, while the measured salinity of sand overburden ranges from 2 to 89 ppt, with an average of 14 ppt.

While the salinity and associated freezing point depression can be shown to vary based on location and material type, a single freezing point depression value of −2.1°C associated with the site wide average has been selected for all thermal modelling of overburden soils. While the average values for the South and North Dam freezing point depressions are lower than −2.1°C, −2.1°C is lower than the geometric mean values for both dam locations. For sand deposits, the use of the −2.1°C freezing point depression will account for any migration of saline material from the surrounding marine deposits.

For groundwater modelling, a freezing point depression of −1.9°C should be used based on the lowest freezing point depression calculated from connate groundwater concentrations (SRK, 2011a).

Table 6: Salinity Measurements and Freezing Point Depressions by Area

	North Dam		South Dam		Site Wide	
	Salinity (ppt)	Freezing Point	Salinity (ppt)	Freezing Point	Salinity (ppt)	Freezing Point
Arithmetic mean	39	−2.2°C	47	−2.6°C	37	−2.1°C
Geometric mean	30	−1.7°C	36	−2.0°C	25	−1.4°C
Max	162	−10.5°C	86	−5.1°C	162	−10.5°C
Min	4	−0.2°C	6	−0.3°C	1	0.0°C
Standard deviation	24	−1.3°C	25	−1.4°C	25	−1.4°C
Count	69	-	12	-	99	-

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Table 7: Salinity Measurements and Freezing Point Depressions by Material Type

	Silt and Clay		Sand		All Materials	
	Salinity (ppt)	Freezing Point	Salinity (ppt)	Freezing Point	Salinity (ppt)	Freezing Point
Arithmetic mean	39	−2.2°C	14	−0.8°C	37	−2.1°C
Geometric mean	26	−1.5°C	7	−0.4°C	25	−1.4°C
Max	162	−10.5°C	89	−5.3°C	162	−10.5°C
Min	0.5	0.0°C	2	−0.1°C	1	0.0°C
Standard deviation	29	−1.6°C	23	−1.3°C	25	−1.4°C
Count	51	-	14	-	99	-

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2.4.4 Unfrozen Water Content

The high porewater salinity concentrations in the overburden also contribute to high unfrozen water contents. Unfrozen water content testing of clay and silt performed in 2004 and 2005 (SRK, 2005a and 2005b) indicates that the unfrozen water content at −5°C ranges from 31% to 90%; this decreases to 23% to 63% at −10°C. Unfrozen water content curves are shown in Figure 14.

Unfrozen water content testing has not been performed on the sandy overburden; however, literature suggests that sandy soils generally exhibit low unfrozen water content below freezing temperatures (Hivon and Sego, 1995). Since the sand overburden has been found to have lower salinity than the clay and silt overburden (Table 7), a lower unfrozen water content is also expected.

2.5 Borrow Characteristics

2.5.1 Borrow Source Investigations

The extent of borrow pit and quarry investigations for general construction fill, concrete aggregate, road surfacing material, liner bedding material, and low permeability material are summarized in Table 8.

Table 8: Summary of Borrow Source and Quarry Investigations

Program	Description
Summer 1993 (EBA, 1993)	Field reconnaissance and potential borrow material (esker sand) collected for petrographic analysis. Potential borrow sources for road construction identified.
Summer 1997 (EBA, 1998)	Field reconnaissance and air photo interpretation, identified and mapped potential quarry locations.
Thurber (2003)	Surficial geological mapping, and four samples collected to identify potential borrow sources for granular material.
Summer 2003 (SRK, 2003)	Air photo interpretation to identify potential borrow sources. Three potential quarries identified. Six hand augured holes/test pits to investigate potential borrow source for fine grained material. Geotechnical laboratory testing performed on fine grained samples.
MHBL (2003)	Geochemical testing of quarries.
Summer 2006 (SRK, 2007)	Eight diamond drill holes for geochemical testing of quarry materials.
Summer 2008 (SRK, 2008)	Nine drill holes for geochemical testing of quarry materials.
Winter 2010 (SRK, 2010)	Ten drill holes for geochemical testing of quarry materials.
Summer 2010 (SRK, 2011b)	Five shallow drill holes for geochemical testing of quarry materials.
Summer 2010 (SRK, 2015)	Two shallow drill holes for geochemical testing of quarry materials.
Summer 2011 (SRK, 2011c)	Seventy-six shallow drill holes (~1.5 m) for geochemical testing of quarry material. Desktop study to identify potential quarries. Site reconnaissance.

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Previous work considered the use of low permeability materials for the impermeable layer within tailings dams (Thurber, 2003 and SRK, 2003). However, development of these borrow sources would be extremely challenging and the risk of permafrost degradation and increasing total suspended solids in water bodies would be significant. Therefore, this concept was not carried forward.

Smaller eskers which would yield good surfacing, bedding and aggregate materials are present in the Boston area. However, eskers are considered high quality wildlife habitat in the arctic, making them unsuitable for development. Shallower sandy beach deposits are present throughout the site. However, exploitation of these deposits is not recommended because of the environmental and engineering challenges associated with developing borrow pits in ice-rich permafrost.

Where excavations of overburden soils are required for infrastructure development, the excavated marine silts and clays could be used as bedding or low permeability construction material. The practicality of properly conditioning these materials (i.e. thawing and subsequent moisture control) to be used as engineered fill makes reliance of this potential borrow source undesirable. Not to mention the high salinity content of this material, and its relative low availability.

Waste rock could be considered to be a source of construction material, provided it is geochemically suitable; however, the current mine plan has all waste rock earmarked for mine backfill.

As a result, construction materials will be obtained from locally developed rock quarries. Different material types will be created through appropriate blasting and crushing operations. Quarry rock may only be used once it has been confirmed to be geochemically suitable (including blast residue). To date six quarries have been developed on the Project site, and there is good understanding of the geochemical properties associated with the rock type in these quarries. Details on quarry geochemistry are provided in SRK (2017c).

2.5.2 Bedrock Characteristics

The most prevalent rock type with surface exposure on the Project are mafic volcanics, predominantly basalt. High ridges on the Project site usually consist of Diabase (Proterozoic Rocks). In isolated areas there are small amounts of gabbro, felsic volcanic and granitoids. These rock types are typically competent and exhibit well-defined foliation. Site wide outcrop mapping can be seen in Figure 2.

3 Geotechnical Design Principles

3.1 Overburden Stripping and Disposal

Stripping of overburden soils on site will be very limited, as open pits are not planned and overburden, organics and vegetation is not stripped prior to road, pad and airstrip construction. Overburden will be stripped from quarry rock during quarry development, contact water pond berm key trenches, and from tailings dam key trenches. Stripped overburden material will be placed in overburden stockpiles constructed at each quarry and mining area.

Whenever possible, overburden stripping should be performed in winter as the sensitive nature of thawed overburden soils could pose construction challenges. Clay sensitivity is expected to range from 4 to 22 with an average of 8, which is quite low, and could indicate that the soil may not be trafficable. While clay sensitivities are quite low, samples of the same material did not liquefy during handling and testing, and the stress-strain curves from consolidated undrained triaxial testing does not suggest rapid structural breakdown if the soil is disturbed.

Based on the data available, SRK believes that thawed overburden soils (including clays) can be stripped using conventional truck and excavator methods. Construction of temporary run-of-quarry (ROQ) access roads may be required for excavation of thawed overburden.

Frozen overburden soils will be excavated using drill and blast techniques; therefore, the resultant overburden pile will consist of blocky frozen material with significant amounts of ice. Compaction to consolidate this frozen material is not practical; therefore, significant thaw settlement is to be expected seasonally. As the overburden pile thaws, trafficability is expected to be challenging.

Water release from the overburden stockpiles is expected to be significant, and this water will likely have a high total suspended solids and possibly high salinity and ammonia. Appropriate water management measures will therefore be integral to the design of these stockpiles, sedimentation berms will be required or possibly contact water ponds.

Based on the material properties of the overburden soils, SRK recommends that the overburden piles be designed with overall slopes angles that do not exceed 11° (5H:1V), and a maximum height of 10 m. Buttressing may also be required. Foundation requirements for overburden piles are similar to those for waste rock piles; which are discussed in Section 3.4.

3.2 Doris Tailings Impoundment Area and Dam Foundations

The Doris TIA, located in a former natural lake (Tail Lake), is the designated tailings impoundment for Phase 1, and will be expanded to contain Phase 2 tailings. The TIA will consist of three dams, the North, South and West dams. These dams will ensure containment of the tailings and associated supernatant water. The North Dam is a frozen core water retaining dam which was constructed in 2011 and 2012 (SRK, 2012a), under the existing Doris Mine permits and licenses. The South and West dams are designed to be frozen foundation rock fill dams. The South dam starter dam will be designed and constructed in Phase 1, under the existing Doris

Mine permits and license, with the final dam being designed and constructed in Phase 2. Figure 15 displays the Doris TIA layout.

Foundation conditions in the general TIA area and under North and South dams, are well characterized from numerous field investigations and thermal monitoring as described in Table 3, Table 4, and Table 5. Foundation conditions under the West Dam are not as well understood as only one geotechnical drill hole exists under the dam alignment, and geophysics was not performed in the area; however, based on site knowledge and the location of surrounding bedrock outcrops, a conceptual stratigraphic profile was developed. Figure 16 and Figure 17 show the interpreted stratigraphic profiles of the dams.

3.2.1 North Dam Foundation

The North Dam is located approximately 200 m downstream of the north most extent of the former Tail Lake, and runs perpendicular to a narrow valley, over the former lake discharge point. The entire dam alignment is located on cold permafrost (approximately −8°C), and no talik was encountered under the discharge point.

The stratigraphy under the dam has two distinct zones, the southwest side is dominated by ice-saturated sand deposits 10 to 15 m thick, overlain by up to 3 m of silt and clay, while the northwest side is dominated by ice-saturated marine clayey silt with a maximum thickness of 15 m. A thin layer of sand and gravel overlies the bedrock surface in the upper portions of the valley (Figure 16). A peat unit was encountered in the center of the dam, in the area of the lake discharge point. This material was removed during dam construction.

A zone of high salinity material was also encountered during construction of the dam. This zone was characterized by unfrozen silty clay material which could be removed with the excavator. All high salinity material at the base of the key trench was removed during construction (SRK, 2012a).

3.2.2 South Dam Foundation

The South Dam is located on the south end of the former Tail Lake on the watershed boundary that separates the TIA from Ogama Lake. The proposed alignment is along a well-drained flat valley section, with bedrock outcrops present on both sides of the valley. Ground temperature measurements of the alignment indicate that cold permafrost (approximately −8°C) is present for the entire dam alignment.

Figure 16 displays the overburden profile along the centerline of the South Dam starter dam; detailed characterization under the South Dam raise has not yet been completed. The overburden foundation are thickest near the center of the dam alignment and thin significantly towards the abutments. The upper 5.5 m of the overburden profile consists of ice rich, saline, marine silt, which transitions to ice rich, saline marine silt and clay to a depth of approximately 24 m. The marine silt and clay is underlain by an approximately 10 m thick layer of gravelly till, which overlies the basalt bedrock. As shown in Table 6, the measured salinity at the South Dam ranges from 6 to 86 ppt, with an average value of 47 ppt.

The South Dam is designed to be a geosynthetic clay liner lined frozen foundation dam, constructed with rock fill (ROQ or geochemically suitable waste rock). The dam will be constructed directly on tundra with no excavation of vegetation or organic material, except in the location of the key trench. Any snow or ice will be removed prior to fill material placement. The key trench will be excavated into the overburden soils, and overburden and vegetation removed from the key trench excavation will be placed in a nearby overburden pile. To ensure a competent key trench foundation any peat, massive ice or hypersaline zones encountered during key trench excavation will be removed.

3.2.3 West Dam Foundation

The West Dam is located in a saddle between two bedrock outcrops to the west of the southern end of the TIA. The foundation conditions for the West Dam are not well understood, as only a single borehole has been completed within the alignment of the dam. Surficial geology and permafrost mapping of the area indicated that the dam foundation will consist of marine blanket material made up of undifferentiated clay, silt and sand (Figure 4). The single borehole log for the area (SRK 39) suggests that the overburden consist of silty clay to a depth of 7 m (SRK, 2003). The overburden profile under the dam is assumed to thin out gradually following the surface topography, from 7 m at the centre of the dam tapering off at the bedrock outcrops (Figure 17). Given the location of the dam alignment, and the ground temperature measurements from SRK 39, it is assumed that the entire alignment of the west dam is in permafrost.

The foundation preparation for the West Dam will be the same as for the South Dam.

3.3 Boston Tailings Management Area Foundations

Foundation conditions for the Boston TMA are not well understood, as only limited geotechnical drilling has been performed in the Boston mining area and none of these drill holes are within the TMA footprint (Figure 9). However, surficial geology and permafrost features mapping performed by EBA (1996) indicates that the TMA will be founded on marine deposits, silty clay with trace sand, and that the area is generally free of frost polygons and permafrost features (Figure 5). Air photo analysis supports the conclusion that the footprint area is relatively free of permafrost features. Based on the nearby drill holes, it is expected that the marine deposits will be 1.5 to 8 m thick (EBA, 1996).

The dry stack tailings material will be placed directly on the tundra, with no excavation of vegetation or overburden prior to tailings placement. To ensure the permafrost foundations remain frozen, the first lift of filtered tailings should be placed in the winter when the ground is frozen. If tailings placement must start when the ground is thawed, a layer of rock fill (ROQ or geochemically suitable waste rock) is likely needed for trafficability.

The contact water pond berms surrounding the tailings will also be placed directly on the tundra with no excavation of vegetation or organic material, except for the key trench portion of the berm. Overburden materials and vegetation excavated from the key trench should be placed in the designated overburden pile.

3.4 Waste Rock Pile Foundations

Waste rock piles are assumed to be constructed on a 1 m thick rock fill (ROQ or geochemically suitable waste rock) pad, constructed directly on permafrost soils, with no excavation of vegetation or organic material. Permafrost soils will provide suitable foundation conditions for waste rock piles, provided the foundation remains frozen. To ensure the foundations remain frozen the underlying ROQ pad should be constructed in the winter, and the first lift of all new waste rock piles should, whenever practical, be placed during the winter. Thermal analysis, using a depressed freezing point of −2.1°C, suggests that a 2.7 m thick layer of ROQ or waste rock placed in the winter provides sufficient insulation to ensure that the active layer remains above original ground level in subsequent summers (Appendix C).

If the rock fill pad and the first lift of a waste rock have to be placed during the summer, the pile will be subject to differential settlement due to consolidation of the active layer. The amount of settlement will vary but will likely be between 10 and 30% of the active layer thickness (i.e., between 0.1 and 0.3 m), which is less than the settlement expected from a typical free-dumped waste rock pile with 10 m lifts. This settlement will only occur during the first summer provided the foundation freeze during the following winter. Should placement of the first lift of waste rock occur during the summer, thermal analysis will be required to determine the maximum thickness that can be placed in the summer to ensure freezing of the foundation materials the following winter.

In all cases, whether waste rock pile construction is started in summer or winter, once active layer freeze-back has been achieved, and the active layer is demonstrated to remain within the base of the waste rock pile, there will likely be few restrictions on maximum lift thickness (subject to confirmation analysis). Overall maximum height of the waste rock piles should be limited to 100 m, unless analysis to confirm otherwise is carried out.

Provided the foundation remains frozen, the only foundation deformation expected is creep. Creep is a long-term process, whereby foundation materials slowly move and permanently deform

Stability analysis for site waste rock piles is presented in Appendix E.

3.5 Permafrost Foundations

When frozen, the overburden soils have sufficient bearing capacity to support infrastructure and associated loads; however, when thawed these soils have little strength. Furthermore, due to high ice contents, overburden soils will undergo significant differential settlement under thawing conditions. Founding surface infrastructure on overburden, under thawed or thawing conditions should be avoided as far as practical, and care must be taken to ensure that heat generated from buildings does not result in foundation thaw.

Structures that are particularly sensitive to differential settlement, such as mills, powerhouses and fuel storage areas, should be founded on competent bedrock wherever possible. Competent bedrock foundations can be obtained by drilling and blasting exposed bedrock ridges, or stripping overburden to expose underlying shallow bedrock. Should bedrock foundations not be available, these sensitive structures could be founded on load bearing piles which extend to bedrock.

Should overburden be stripped to expose bedrock, the design criteria for the overburden slopes are:

- Overall slope of 18° (3.1H:1V).
- The toe of the overburden slope should be set back at least 13 m from structures.
- Minimum 2 m thick rock fill cladding of slope to provide thermal protection. Ideally the thermal protection should be placed before average ambient daily air temperature are above 0°C degrees. However, should this not be practical, the slopes can likely be left exposed for one summer, with the expectation that there will be significant surficial sloughing.
- Stability assessment of the slope design should be carried out to ensure that the offset from infrastructure is adequate.
- The factor of safety (FOS) for this slope should exceed 1.3.

Structures and linear surface infrastructure elements (i.e. roads, pipeline corridors, and airstrips) that are not sensitive to differential settlement can be founded on the overburden soils, provided an appropriate thermal protection layer is constructed. Thermal analysis suggests that a 2 m thick ROQ material pad should maintain the 0°C isotherm within the base of the pad, when not thermally impacted by heated buildings or other surface infrastructure (Appendix C). Performance monitoring of existing pads, roads and airstrip on the Doris site suggest that a minimum 1 m fill thickness is sufficient to prevent differential settlement, when not thermally impacted.

A greater pad thickness, and foundation insulation would be required to maintain the 0°C isotherm within the base of the pad for areas thermally influenced by heated buildings. For large heated buildings, it is likely that additional preventative measures are required to prevent permafrost degradation, such as raising buildings above the pad surface to allow circulation of cold air or the placement of thermosyphons. This is discussed in more detail in Appendix C.

Should thaw consolidation and settlement occur, due to thinner pads and 0°C isotherm being within the overburden soils, it will be short-lived (e.g. one or two seasons) after which no further settlement should be experienced. This is assuming that no heat is generated by a structure resulting in an increase in the active layer thickness.

Due to the shallow active layer thickness and cold permafrost temperatures, the use of geosynthetics (geotextile and geogrid) to increase the foundation strength is not required. Adfreeze piles can be used for smaller structures such as radio towers, small bridge crossings, culvert footings etc. Adfreeze pile design is described in Section 4.4.2.

3.6 Talik Foundations

Construction of facilities founded on talik zones may be necessary. These soils have low bearing and frictional strength. Construction on this material poses significant challenges including substantial settlement and possible foundation bearing failure. Settlement can be compensated for via overbuilding; however, foundation bearing failure is a more challenging problem, and may require pre-consolidation and/or design of foundation strengthening elements, such as load distributing foundation pads. The design thickness of these pads should be calculated based on required geometry and load requirements, but if necessary, geosynthetics (geogrid and geotextile) can be used to optimize the fill requirements of these pads. Load-bearing piles extending to bedrock driven through the talik overburden soils can also be used under these conditions.

3.7 Surface Water Management Facilities

Surface water management facilities such as diversion ditches, culverts, sedimentation, and contact water ponds will be required as part of the Project. Where these facilities are located on permafrost, above ground solutions must be sourced. Excavation of channels and/or ditches into the overburden soils must be avoided, and if absolutely necessary, excavated ditches and channels will have to be over-excavated and lined with a thermal blanket to protect the permafrost. Appropriate thermal and hydraulic assessment of these channels will be required.

Ponded water on permafrost areas should also be avoided, except in specifically designed and constructed water containment dams and ponds. Uncontrolled ponding of water on permafrost will result in vegetation dieback, followed by permafrost thaw. Permafrost thaw may result in erosion and fine-grained silts and clays being released into the receiving environment and water, which would increase total suspended solids.

Even within engineered containment structures, areas where permafrost will be flooded may be subject to shoreline erosion, and appropriate mitigation measures will be required. Source mitigation would likely consist by blanketing the area in question with a layer of rock fill, which includes a filter layer to prevent fines from being released.

3.8 Infrastructure Preparation Recommendations

Considering the number of the conditions listed in the preceding section, the specific foundation preparation recommendations for the Project are summarized below.

- Overburden and organic material is not to be stripped prior to construction;
- Bedrock foundations or end-bearing piles are required for critical structures such as fuel storage facilities and processing plants and powerhouses;
- Whenever possible pad, airstrip and road construction should take place in the winter;
- Minimum fill thickness for roads and non-critical, unheated infrastructure pads is 1.0 m; and
- Minimum fill thickness for critical, unheated infrastructure (e.g., airstrip, bridge abutments) is 2.0 m.

4 Geotechnical Design Parameters

4.1 Typical Overburden Properties

Typical overburden properties for the Project are given in Table 9 through Table 12. These properties are intended to be used for general geotechnical design where site specific characterization is not available. Only minimal laboratory testing has been performed on the sand overburden; therefore, the engineering properties presented are based on literature values and engineering judgment.

It should be noted that all undisturbed samples, CPT and vane shear tests used for development of engineering properties are from talik overburden located beneath lakes (Doris, Patch and Aimaokatalok lakes) and Roberts Bay. When these properties are used, appropriate engineering judgement must be applied to account for uncertainties.

Table 9: Typical Clay/Silt Overburden Indicator Properties

Element	Value/Comment	Source
Natural Moisture Content	variable, but typically greater than 50%	Laboratory testing results
Degree of Saturation	97%	Laboratory testing results
Porosity, n	0.52 to 0.63	Laboratory testing results
Volumetric Water Content	0.952	Average laboratory testing results
Volumetric Fraction of Unfrozen Water at -5°C	0.3 to 0.9	Laboratory testing results
Plastic Limit	13 to 37	Laboratory test results
Liquid Limit	18 to 58	Laboratory test results
Plasticity Index	3 to 41	Laboratory test results
Clay Fraction	7.3 to 62%	Laboratory testing results
Silt Fraction	27 to 70%	Laboratory testing results
Clay Mineralogy	Illite, chlorite, albite, kaolin, quartz, plagioclase	South East of former Tail Lake SRK-54 (SRK, 2005a)
Clay Sensitivity	4 to 22	Laboratory testing results
Primary Soil Type	Clay (CL)	
Specific Gravity	2.7	Average of laboratory test results
Bulk Density	1,190 to 2,380 kg/m^3	Laboratory testing results
Moist Unit Weight	15.2 to 23.3 kN/m^3 Design: 17.0 kN/m^3	Laboratory testing results Based on engineering judgement
Shrinkage Limit	17 to 23	Laboratory testing results
Shrinkage Ratio	1.5 to 1.8	Laboratory testing results

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[LabResultsSummary_20160608.xlsx]

Table 10: Typical Clay/Silt Overburden Engineering Properties

Element		Value/Comment	Source
Unfrozen	Peak Undrained Shear Strength	8.5 to 27 kPa Design: 13 kPa	Laboratory testing, and vane shear testing results. Value for design selected based on engineering judgement.
	Remoulded Shear Strength	0 to 4 kPa	Vane shear testing results
	Total Strength, cohesion	3 to 10 kPa	Triaxial test results
	Total Strength, friction angle	12 to 15°	Triaxial test results
	Effective Strength, cohesion	6 to 8 kPa	Triaxial test results
	Effective Strength, friction angle	26 to 31°	Triaxial test results
	Apparent Cohesion, c	0 kPa	Normally consolidated material does not have cohesion
Frozen	Apparent Cohesion, c'	112 kPa	Calculated
	Friction angle, ɸ (º)	26 kPa	Calculated
Coefficient of Consolidation		0.59 to 1.27 m^2/year	Laboratory results at 100 kPa
Saturated Hydraulic Conductivity		3.55 to 4.61 x 10^{-10} m/s	From consolidation testing (SRK, 2009)

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[LabResultsSummary_20160608.xlsx]

Table 11: Typical Sand Overburden Indicator Properties

Element	Value/Comment	Source
Natural Moisture Content	Variable, but typically around 26	Laboratory testing results
Degree of Saturation	38%	Laboratory testing results
Porosity, n	0.63	Laboratory testing results
Volumetric Water Content	70%	Calculated
Specific Gravity	2.68	Laboratory testing result
Bulk Density	1,849 kg/m^3	Laboratory testing results
Moist Unit Weight	18	Laboratory testing results

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[LabResultsSummary_20160608.xlsx]

Table 12: Typical Sand Overburden Engineering Properties

Element		Value/Comment	Source
Unfrozen	Apparent Cohesion, c'	0 kPa	Estimated
	Friction angle, ɸ (º)	35	Estimated (SRK, 2003)
Frozen	Apparent Cohesion, c'	4.5 MPa	Ladanyi and Morel (1990)
	Friction angle, ɸ (º)	26 to 32	Ladanyi and Morel (1990)

4.2 Borrow Properties

Table 13 outlines the recommended material properties for engineered fill, ROQ material. Other than the California Bearing Ratio, these properties have not been measured, but are based on comparison of the materials with similar materials as reported in literature and SRK's internal database. The material properties apply only to engineered fills that have been placed in accordance with SRK's Technical Specifications (SRK, 2011d).

Table 13: Typical in Place Run-of-Quarry Properties

Element		Value	Source
Moist Unit Weight		20 kN/m^3	Estimated
Degree of Saturation		30%	Estimated
Porosity, n		0.3	Estimated
Volumetric Water Content		0.09	Estimated
California Bearing Ratio(1)		42.1 to 78.6%	SRK (2011e)
Unfrozen	Apparent Cohesion, c'	0 kPa	Estimated
	Friction Angle, ɸ	38 to 40°	Field Observations
Frozen	Apparent Cohesion, c'	5 kPa	Estimated
	Friction Angle, ɸ	38 to 40°	Field Observations

Note(s):

(1) Due to testing methodology 20 mm minus material was used for the California bearing ratio testing.

4.3 Bulking and Shrinkage Factors

Bulking and shrinkage factors to use for the various geotechnical materials on the Project are provided in Table 14. The bulking and shrinkage factors are based on material properties reported in literature and SRK's internal database. Bulk and compacted densities are based on laboratory testing values, or literature values. Compacted densities from laboratory testing results are assumed to be 90% of the maximum bulk density of the Standard Proctor results.

Table 14: Bulking and Shrinkage Factors for the Project's Geotechnical Materials

Material	Bulk Density (Mg/m^3)	Bulking Factor	Loose Density (Mg/m^3)	Shrinkage Factor	Compacted Density (Mg/m^3)
ROQ	1.96	1.1	1.78	1.0	1.96
Transition Material	1.99	1.1	1.81	1.0	1.99
Surfacing Material	2.01	1.1	1.83	1.0	2.01
Bedding Material	2.20	1.1	2.00	1.0	2.20
Waste Rock	1.96	1.1	1.78	1.0	1.96
Clay/Silt Overburden, Frozen	1.70	1.3	1.31	1.0	1.72
Clay/Silt Overburden, Unfrozen	1.70	1.3	1.31	1.0	1.72
Sand Overburden	1.77	1.1	1.61	0.9	1.88

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\!080_Deliverables\OverburdenSummaryReport\020_Tables\[OVBSummaryReport_Tables_1CT022-004_Rev00_mmm_ts.xlsx]

4.4 Foundation Bearing Capacity

In unfrozen soils, the allowable bearing pressure for a shallow foundation is usually based on the FOS against general soil failure and on the tolerable foundation settlement. Similar criteria are applicable to shallow foundations in frozen soils, but the strength of frozen soils is temperature dependant, and the main source of frozen foundation settlement is typically creep rather than consolidation (Andersland and Ladanyi, 2004). The cold saline ice-rich marine silt and clay permafrost found in the Project area will be subject to creep deformation which will impact excavation slopes and infrastructure foundations in the long-term. Creep testing has not been carried out, and to-date modelling has been limited to analysis of tailings structures (SRK, 2017d and 2017e).

Ultimate bearing capacity (UBC) takes into consideration fundamental soil characteristics, footing geometry, loads and drainage. SRK cannot provide definitive values of ultimate bearing capacity as footing geometry and loads are not know; however, Table 15 lists typical ranges of ultimate bearing capacity for the geotechnical conditions that may be encountered in the Project area. It will be up to the designer to select appropriate values within these suggested ranges based on site specific conditions.

Assuming limit state design, it is up to the designer to adopt an appropriate FOS to determine the allowable bearing capacity of a structure. Allowable bearing capacity is defined as the ultimate bearing capacity divided by FOS. Guidance on selecting FOS should be obtained from design guidelines, such as the Canadian Foundation Engineering Manual (CGS, 2006).

For the Project, spread footings on an appropriate thermal pad of competent engineered fill are recommended for structure foundations. Notwithstanding, additional analysis will be recommended.

Table 16, Table 17 and Table 18 list further important geotechnical design parameters that are required to complete foundation, settlement and retaining wall design. The choice of appropriate design values within the specified range is left up to the designer; however, SRK recommends sticking to the lower end of the range given the uncertainty and lack of site specific data.

Table 15: Summary of Bearing Capacity Characteristics[(1)]

Element	Unit	Value/Comment	Source
Ultimate Bearing Capacity Pressure[(2)]	Competent Bedrock (Hard) (Sound Igneous or Metamorphic Rock)	7,500 kPa	USACE (1992)
	Competent Bedrock (Medium Hard) (Sound Sedimentary Rock to Foliated Metamorphic Rock)	1,000 to 3,000 kPa	USACE (1992)
	Bedrock (Soft to Medium Hardness) (Weathered or Broken Rock, RQD typically <25)	950 to 1,000 kPa	USACE (1992)
	Silt/Clay Overburden (Frozen)	200 to 400 kPa	Estimated
	Silt/Clay Overburden (Unfrozen)	50 to 75 kPa	USACE (1992)
	Sand Overburden (Frozen)	300 to 700 kPa	Sebastyan (1962)
	Sand Overburden (Unfrozen)	140 to 280 kPa	USACE (1992)
	Engineered Fills (Crushed Rock 1 to 4 m Thick)	200 to 600 kPa	USACE (1992), Oloo et al (1997)
Allowable Bearing Capacity Pressure[(3)]	Competent Bedrock	500 to 2000 kPa	Calculated
	Silt/Clay Overburden (Frozen)	100 to 200 kPa	Calculated
	Silt/Clay Overburden (Unfrozen)	25 to 37.5 kPa	Calculated
	Sand Overburden (Frozen)	150 to 350 kPa	Calculated
	Sand Overburden (Unfrozen)	70 to 140 kPa	Calculated
	Engineered Fills > 1 m (Crushed Rock 1 to 4 m Thick)	100 to 300 kPa	Calculated

Notes:

(1) Values presented should only be used as presumed preliminary design bearing pressures. Field inspection of work site should be completed before final design to adjust design bearing pressures.

(2) Ultimate bearing capacity should be calculated based on dimensions (i.e. Ultimate bearing capacity = ultimate bearing capacity pressure * foundation width * foundation lateral length, for simple rectangular shape).

(3) FOS of 2 has been utilized in determining the maximum allowable bearing capacity pressures presented

Table 16: Summary of Poisson's Ratio[1]

Unit	Value/Comment	Source
Silt/Clay Overburden (Frozen)	0.3 to 0.4[(1)]	Estimated
Silt/Clay Overburden (Unfrozen)	Saturated, undrained: 0.5 Partially saturated: 0.3 to 0.4	Coduto (1999)
Sand Overburden (Frozen)	0.3 to 0.4[(1)]	Estimated
Sand Overburden (Unfrozen)	0.1 to 0.35	Coduto (1999)
Engineered Fills (Crushed Rock)[(2)]	0.15 to 0.35	Coduto (1999) and Das (2005)

Notes:

(1) Poisson's ratio for ice is 0.33 at −5°C (Schulson 1999).

(2) When compacted to SRK's Technical Specifications (SRK 2011d).

Table 17: Summary of Modulus of Subgrade Reaction

Unit	Value/Comment	Source
Silt/Clay Overburden (Frozen)	1,380 to 1,515 kPa	Estimated
Silt/Clay Overburden (Unfrozen)	345 to 690 kPa	ISUDS (2009)
Sand Overburden (Frozen)	1,380 to 1,515 kPa	Estimated
Sand Overburden (Unfrozen)	1,030 to 1,375 kPa	ISUDS (2009)
Engineered Fills (Crushed Rock)	1,380 to 1,515 kPa	ISUDS (2009)

Table 18: Summary of Select Elastic Moduli, E

Unit	Value/Comment	Source
Silt/Clay Overburden (Frozen)	70 to 150 MPa	Estimated
Silt/Clay Overburden (Unfrozen)	4 to 25 MPa	USACE (1990)
Sand Overburden (Frozen)	14 to 34 GPa	Andersen et al. (1995)
Sand Overburden (Unfrozen)	14 to 95 MPa	USACE (1990)
Engineered Fills (Crushed Rock)	50 to 175 MPa[(1)]	USACE (1990)

Notes:

(1) Depending on degree of compaction.

4.4.1 Typical Thermal Properties

Typical material properties to be used in thermal modelling are summarized in Table 19, these values are obtained from a combination of literature values, calculated values and laboratory testing results. The thermal properties of rigid polystyrene insulation were obtained from Andersland and Ladanyi (2004), while the thermal properties for peat were obtained from Romanovsky and Osterkamp (2000). The thermal properties of the granular pad were calculated using the method by Johansen (1975) and Cote and Konrad (2005).

The thermal properties for clay/silt overburden was based on laboratory measurements, and a porewater freezing point depression of −2°C. Local variability in the freezing point depression is expected to have minor effect on the predicted temperatures; therefore, an average value is presented.

Table 19: Typical Thermal Properties

Material	Degree of Saturation (%)	Porosity	Thermal Conductivity (kJ m^{-1} day^{-1} °C^{-1})		Volumetric Heat Capacity (kJ m^{-3} °C)		Source
			Unfrozen	Frozen	Unfrozen	Frozen	
ROQ	30	0.30	104	117	1,697	1,509	Calculated using method by Johansen (1975)
ROQ, Saturated	100	0.30	141	117	2,576	1,509	Calculated using method by Johansen (1975) and Cote and Konrad (2005)
Transition(1)	40	0.21	172	174	1,821	1,646	Calculated using method by Cote and Konrad (2005)
Transition(1) Saturated	100	0.21	208	274	2,347	1,911	Calculated using method by Cote and Konrad (2005)
Core(1) Saturated	88	0.26	184	231	2,827	2,351	Calculated using method by Cote and Konrad (2005), using material properties from SRK (2012a)
Polystyrene Insulation	0	-	3	3	38	38	Andersland and Ladany (2004)
Peat	100	0.65	48	138	2,600	2,200	Romanovsky and Osterkamp (2000)
Silt/Clay Overburden(2)	85	0.52	112	187	2,842	2,038	Laboratory testing results, Newman (1995) and Cote and Konrad (2005)
Bedrock (Basalt)	100	0.05	260	260	2,380	2,133	Estimated SRK (2003)

Notes:

(1) Transition material is 150 mm minus material and core material is frozen core material used in North Dam Construction.

(2) Clay/Silt overburden includes a freezing point depression of −2°C and an unfrozen water content curve.

4.4.2 Adfreeze Piles

Critical infrastructure, or infrastructure subject to heavy loads or vibrations, should typically be founded on bedrock or load bearing piles extending to bedrock. Adfreeze piles can be used for smaller structures if they cannot be founded on rockfill pads.

Adfreeze piles derive most of their load-bearing capacities from adfreeze bonds which develop between the soil or backfill and the pile surface. Only a small fraction of an adfreeze piles capacity is due to end bearing unless the pile extends to bedrock or dense thaw stable granular material (Andersland et al., 2004). Appropriate adfreeze strengths for use in adfreeze pile design are presented in Appendix D. The provided adfreeze strengths take into account the freezing point depression and Project ground temperature profile.

The annulus of adfreeze piles should be backfilled with non-saline sand slurry or arctic grout with strength greater than 30 MPa, such as cold SET 45 or Arctic 100. The selection of an appropriate backfill material will depend on the structure, and the expected loads.

4.5 Lateral Earth Pressures

Lateral earth pressures are needed for the design of retaining wall and bridge foundations. Table 20 provides expected values of lateral earth pressures, assuming a long, smooth vertical wall where the lateral pressures increase linearly with depth and no frost action. These values are provided to give a reference point of expected values, structure specific analysis should be performed if conditions differ from those assumed.

Retaining wall backfill and bridge foundations should be constructed of well drained granular fill to limit frost heave. If overburden or other frost susceptible material was utilized for retaining wall construction, lateral frost forces acting on the retaining walls should be included into calculations.

Table 20: Summary of Select Lateral Earth Pressure Parameters

Element	Unit	Value/Comment	Source
Rankine Passive (K_p) Soil Pressure Coefficient	Overburden	2.56 to 3.12	Calculated from Rakine's theory of active and passive soil pressures. Assumes a long smooth wall and linear distribution of lateral pressure
	Engineered Fills >1 m	4.20	
Rankine Active (K_a) Soil Pressure Coefficient	Overburden	0.32 to 0.39	
	Engineered Fills >1 m	0.24	
Coefficient of at Rest Earth/ Soil Pressures (K_o)[1]	Overburden	0.49 to 0.56	Calculated based on Jaky empirical equation, assuming normally consolidated soils that exhibit zero cohesion during drained shear
	Engineered Fills >1 m	0.38	
Angle of Repose	Engineer Fills	1H:1V to 1.2H:1V	Field observations during infrastructure construction support 2010 to 2012
Allowable Slopes	Engineered Fills <2 m	1.5H:1V	SRK (2011d)
	Engineered Fills >2 m	1V:2H	SRK (2011d)
Coefficient of Friction	Between Concrete Wall and ROQ Material	25 to 27°	Estimated, based on ROQ friction angle and Coulomb equation

Notes:

(1) Simplification to be reassessed on a site by site case

4.6 Corrosion Potential

To prevent corrosion of wall materials, the backfill material for mechanically stabilized earth (MSE) walls must meet certain standards for chloride concentration, resistivity, sulfur concentration and pH. Table 21 summarizes the results of corrosion resistance testing performed on ROQ, waste rock and overburden samples.

Table 21: Corrosion Resistance Parameters

Material	Chloride (mg/L)	Resistivity (ohm cm)	Sulfur, as SO_4 (mg/L)	Paste pH	Source
Waste Rock	5370	2,200	184	7.11	SRK (2011e)
ROQ	54 to 385	720 to 7,400	28 to 159	7.70 to 8.37	SRK (2011e)
Overburden	<20 to 76	5,750 to 6,100	27 to 69	7.04 to 7.84	SRK (2011e)

4.7 Seismicity

The site specific ground motions presented in Table 1, were adjusted to the soil class and then the Limit Equilibrium Pseudo Static Stability Analysis method (FHWA, 2011) to obtain seismic coefficients for design. Details on the development of the seismic coefficients are presented in Appendix B.

Table 22 presents the horizontal seismic coefficients for the infrastructure requiring stability analysis during Phase 2 of the Project, assuming a FOS of 1.1. The vertical seismic coefficients are assumed to be negligible. Should analysis of other infrastructure be required, the horizontal seismic coefficients for analysis can be obtained from Table 23, assuming that the infrastructure is founded on a minimum of 3 m of marine silt and clay overburden.

Table 22: Seismic Coefficients for Various Infrastructure on the Project

Structure	Critical Section Height (m)	Seismic Event	Seismic Coefficient (g)
Operations			
South Dam	15	1:2,475	0.021
West Dam	5	1:2,475	0.025
Madrid South Waste Rock	20	1:476	0.0075
Madrid North Waste Rock	100	1:476	0.0075
Boston Waste Rock	25	1:476	0.0072
Boston Dry Stack	26	1:2,475	0.018
Contact Water Pond Berms	2.5	1:476	0.0086
Closure			
South Dam	15	Halfway between 1:2,475 year and 1:10000 year	0.036
West Dam	5	Halfway between 1:2,475 year and 1:10,000 year	0.043
Boston Dry Stack	25	1:2,475	0.018

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\Seismic Hazard Analysis\[HopeBay_SeismicCoefficentCalculation_1CT022.004_20160510_mmm.xlsm]Summary

Table 23: Horizontal Seismic Coefficient Geotechnical Design of Infrastructure Founded on Clay/Silt Overburden(1)

Dam / Embankment Height (m)	Seismic Coefficient (g)				
	1:100 year	1:476 year	1:1,000 year	1:2,475 year	1: 10,000(2) year
≤ 5	0.0026	0.0086	0.013	0.025	0.061
10	0.0024	0.0083	0.013	0.023	0.056
15	0.0023	0.0079	0.012	0.021	0.051
20	0.0021	0.0075	0.012	0.020	0.046
25	0.0020	0.0072	0.011	0.018	0.041
30	0.0018	0.0068	0.011	0.016	0.036
≥ 35	0.0018	0.0067	0.011	0.016	0.035

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\Seismic Hazard Analysis\[HopeBay_SeismicCoefficentCalculation_1CT022.004_20160510_mmm.xlsm]Summary

Note(s)

(1) These seismic coefficients apply whenever there is more than 3 m of clay and silt overburden within the overburden profile.

(2) The 1:10,000 year seismic coefficient is extrapolated.

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

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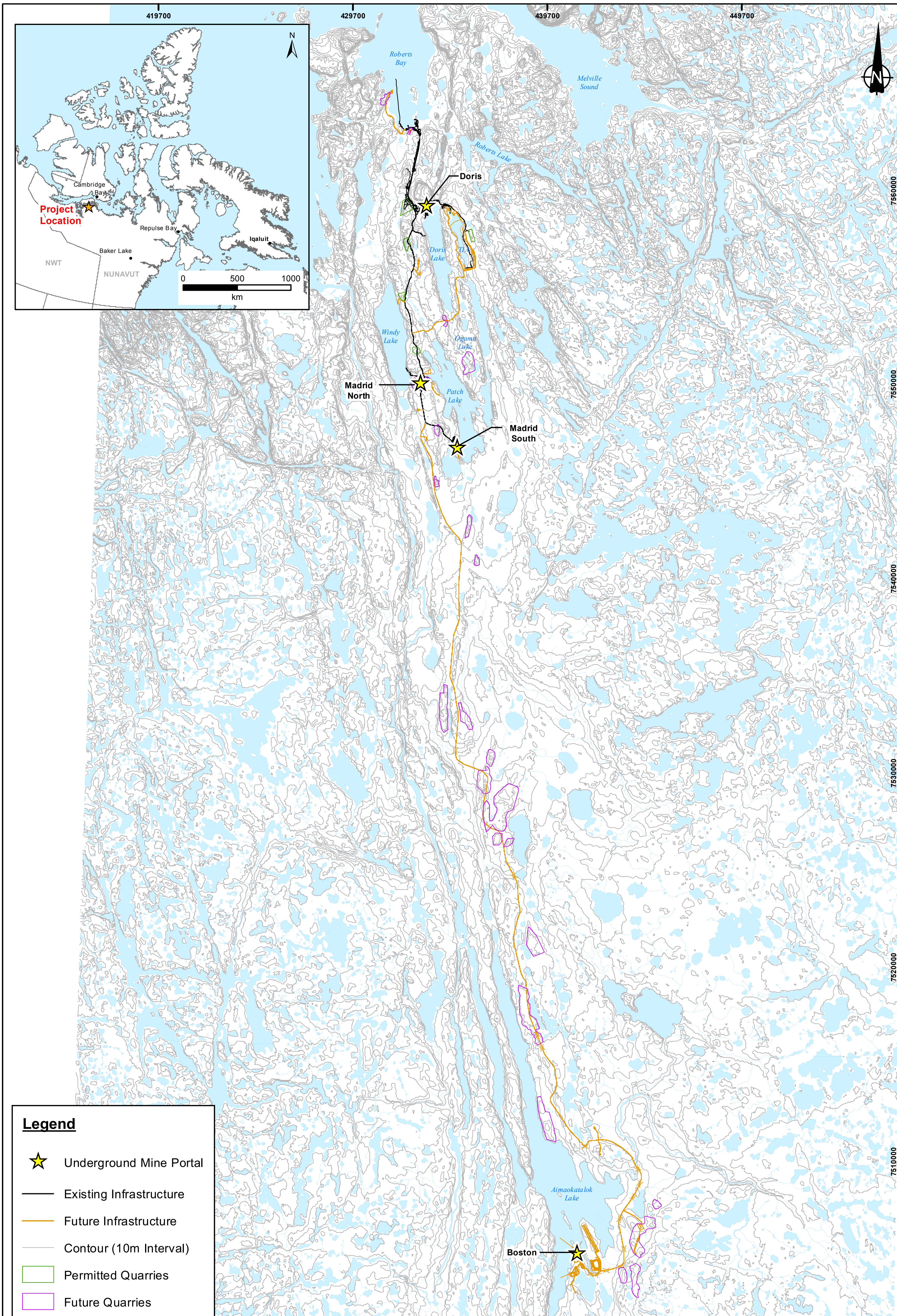
Figures



Notes:
1. Coordinate System: NAD 1983 UTM Zone 13N
2. Base Topo Data: CanVec, Natural Resources Canada

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TMAC RESOURCES

Overburden Summary Report

Site Location Map

Project No: 1CT022.013

Filename: 1CT022.013_Fig_1_SiteLocationMap_mzs_RevC

HOPE BAY PROJECT

Date:	Approved:	Figure:
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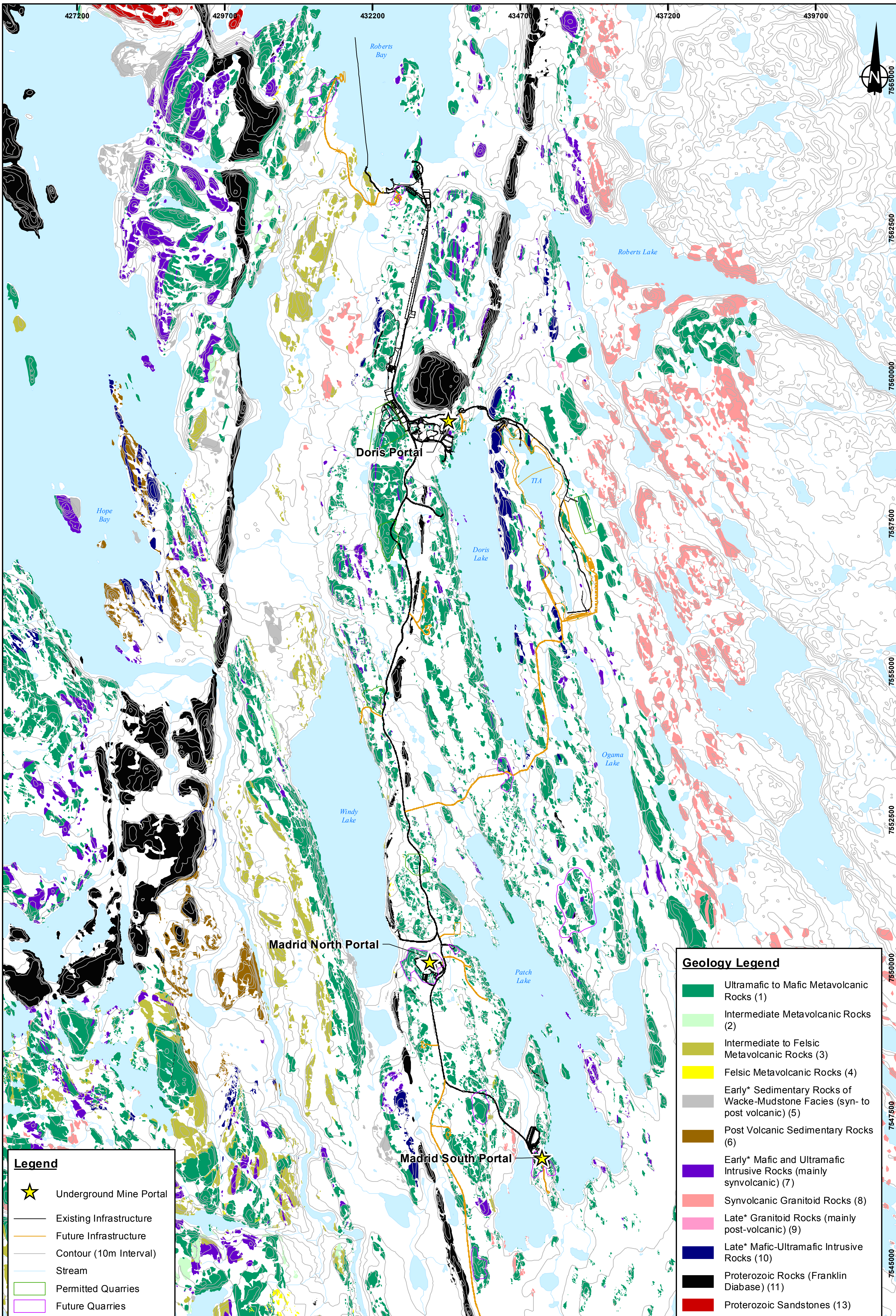

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Notes:
1. *early and late used in a relative sense only.
2. Coordinate System: NAD 1983 UTM Zone 13N
3. Base Topo Data: CanVec, Natural Resources Canada

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Project No: 1CT022.013

Filename: 1CT022.013_Fig_2_OutcropMapping_mzs_RevB

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HOPE BAY PROJECT

Overburden Summary Report

Doris and Madrid Mining Area
Outcrip Mapping (After Sherlock 2002)

Date:	Approved:	Drawing:
Dec 2017	CM	2

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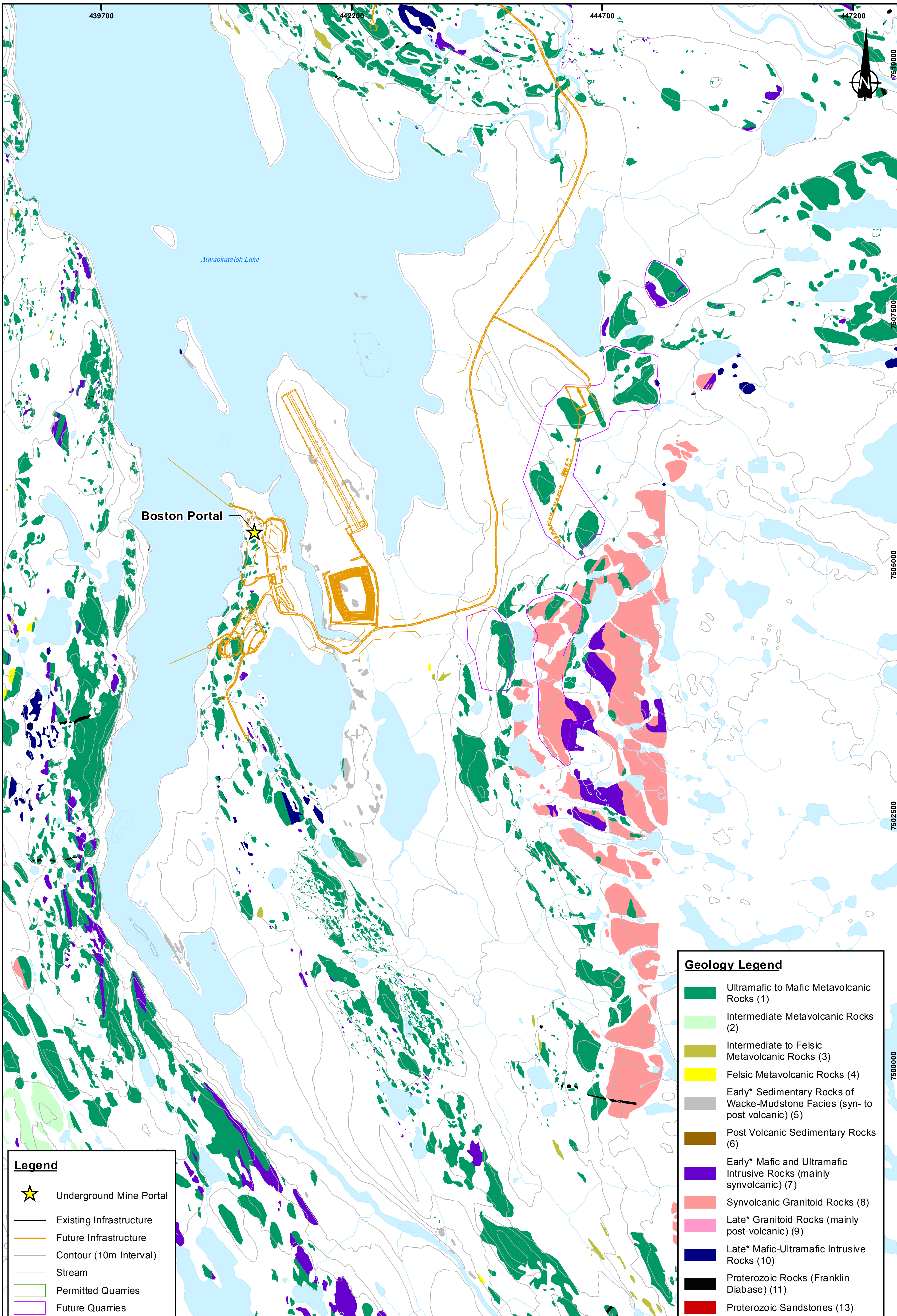

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Notes:
1. *early and late used in a relative sense only.
2. Coordinate System: NAD 1983 UTM Zone 13N
3. Base Topo Data: CanVec, Natural Resources Canada

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Project No: 1CT022.013

Filename: 1CT022.013_Fig_3_BostonMiningArea_mzs_RevB

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HOPE BAY PROJECT

Overburden Summary Report

Boston Mining Area Outcrop Mapping (After Sherlock 2002)

Date:	Approved:	Drawing:
Dec 2017	CH	3

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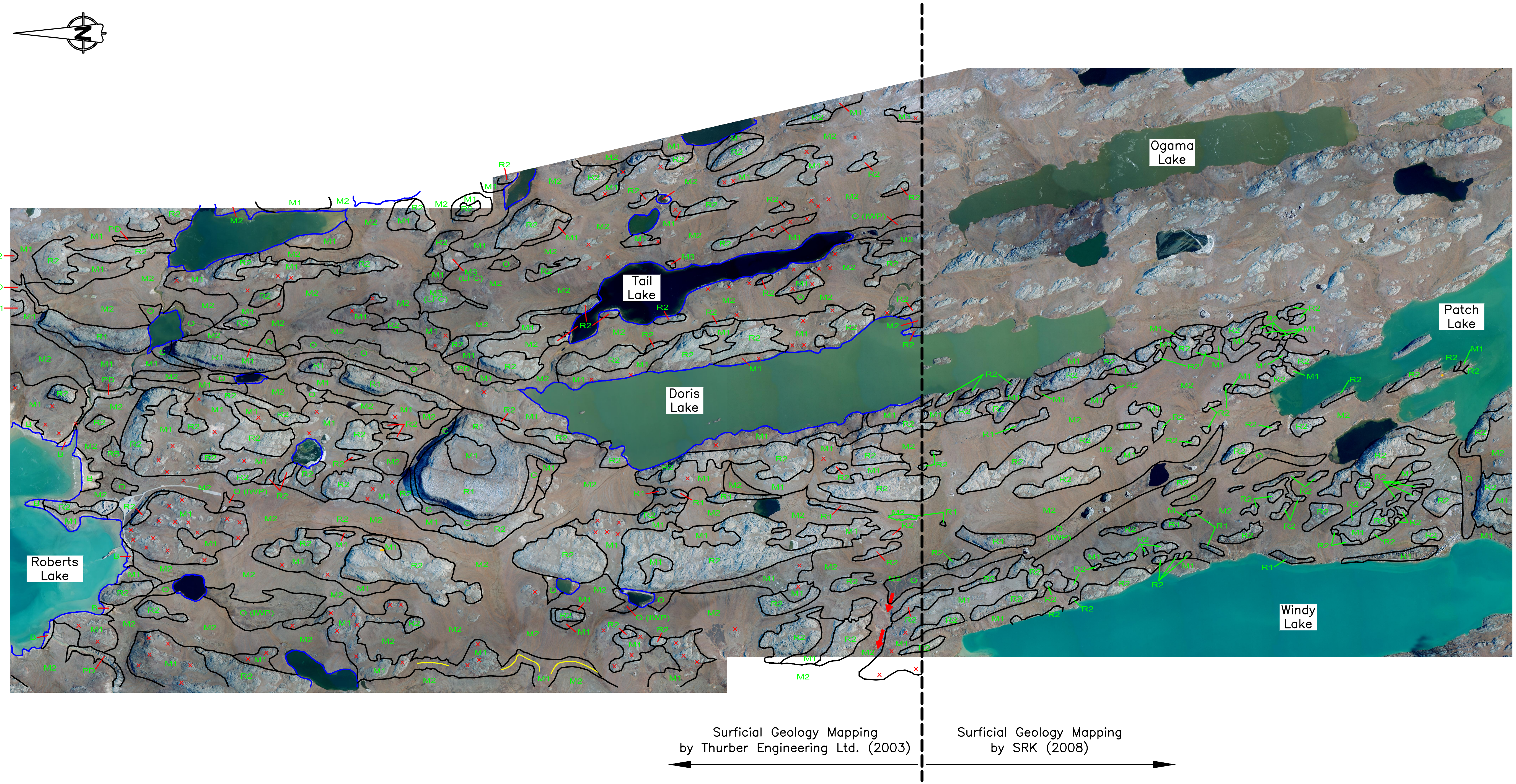


LEGEND

Quaternary Holocene Deposits

- B Active Marine Beach Deposits
- C Colluvium
- O Organic Deposits: Peat and organic mud
- PD Areas of Permafrost Degradation
- M Inactive Marine Beach Deposits
  - M1 Littoral Deposits: Sand, gravel, cobbles and boulders
  - M2 Marine Blanket: Undifferentiated clay, silt and sand judged to be 2.5 to 20m thick
  - M3 Marine Veneer: Same as M2 but generally judged to be less than 2.5m thick

Pre-Cambrian Bedrock

- R1 Neoproterozoic-Age Diabase Dykes and Sills
- R2 Folded, northward striking, metamorphosed and foliated Archean volcanics and lesser sedimentary rocks

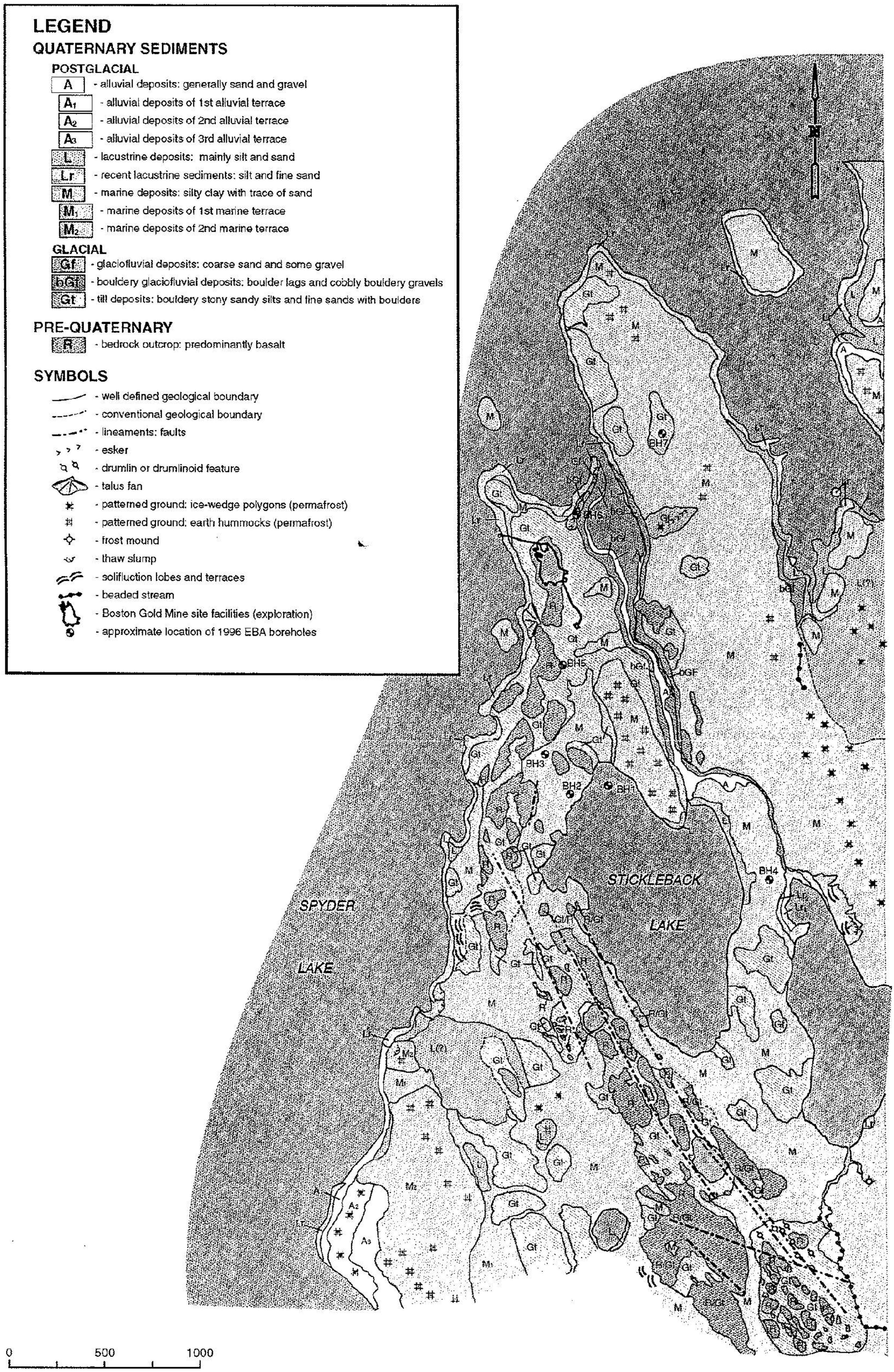
On-Site Symbols

- Geologic Boundaries Interpreted from Topographic Map and Aerial Photos (all boundaries are approximate)
- (IWP) Ice Wedge Polygons
- (LFC) Linear Frost Cracks
- Δ Frost Shattered Rock
- Gully
- Marine Wave Eroded Slope
- RB Raised Marine Beach
- × Small Bedrock Outcrops

0 300 600 900 1200 1500 Metres

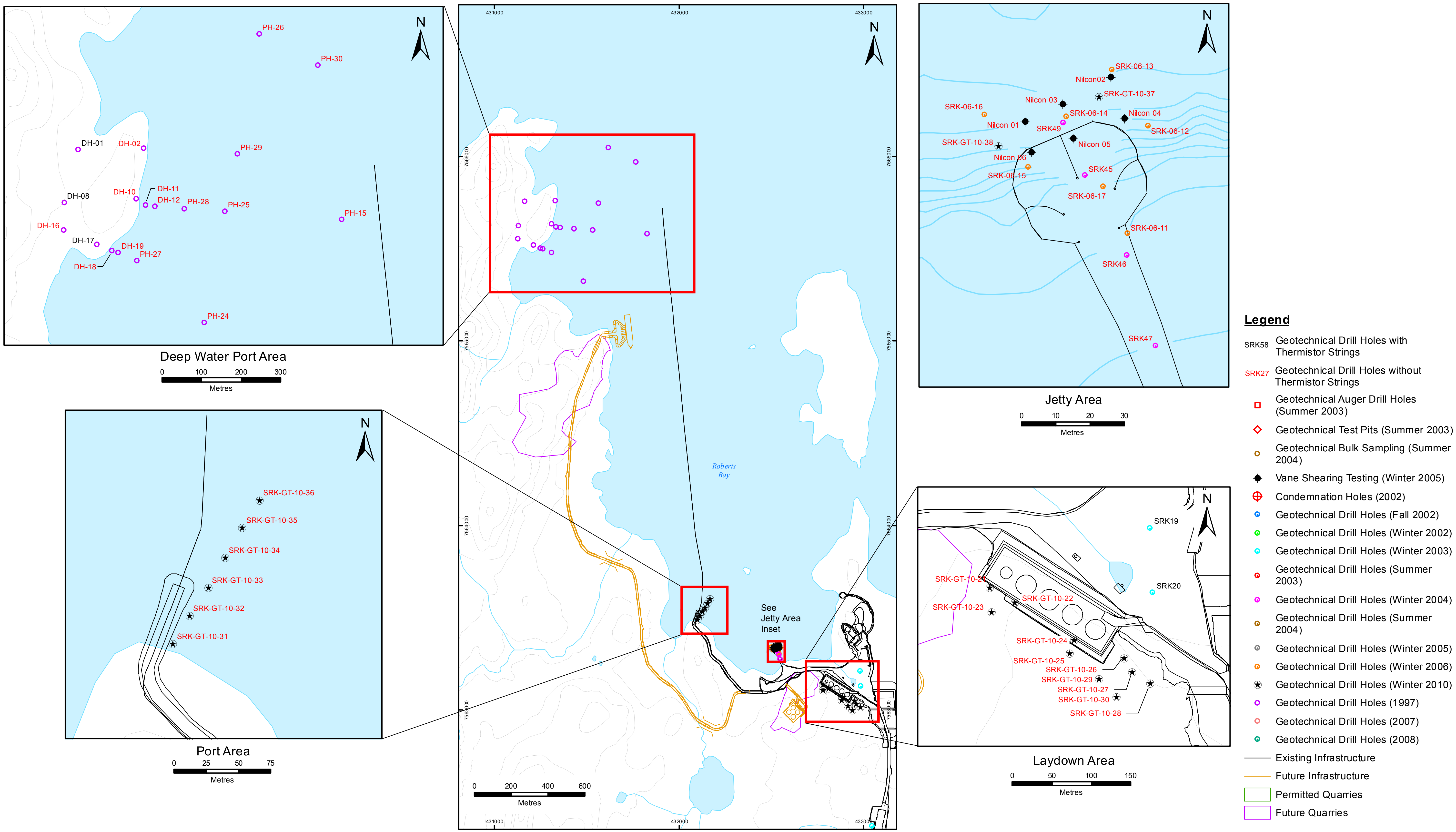
1:40,000

srk consulting	MAC RESOURCES	Overburden Summary Report
		Surficial Geology and Permafrost Features at Doris and Madrid Mining Areas
SRK JOB NO.: 1CT022.013	HOPE BAY PROJECT	DATE: Nov. 2017
FILE NAME: 1CT022.013_Fig_4_SurficialGeo.dwg		APPROVED: MMM
		FIGURE: 4



Note: Spyder Lake is now called Aimaokatalok Lake

srk consulting	MAC RESOURCES	Overburden Summary Report
		Geology and Permafrost at Boston Mining Area
Job No: 1CT022.013	HOPE BAY PROJECT	Date: November 2017 / Approved: MMM / Figure: 5
Filename: 1CT022.013_Fig_5.pptx		



Notes:
Coordinate System: NAD 1983 UTM Zone 13N

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Overburden Summary Report

Overburden Characterization Holes at the Doris North Mining Area (Roberts Bay)

Job No: 1CT022.013	HOPE BAY PROJECT	Date: Dec 2017	Approved: CH	Figure: 6
Filename: 1CT022.013_Fig_6_RobertsBay_mzs_RevB				

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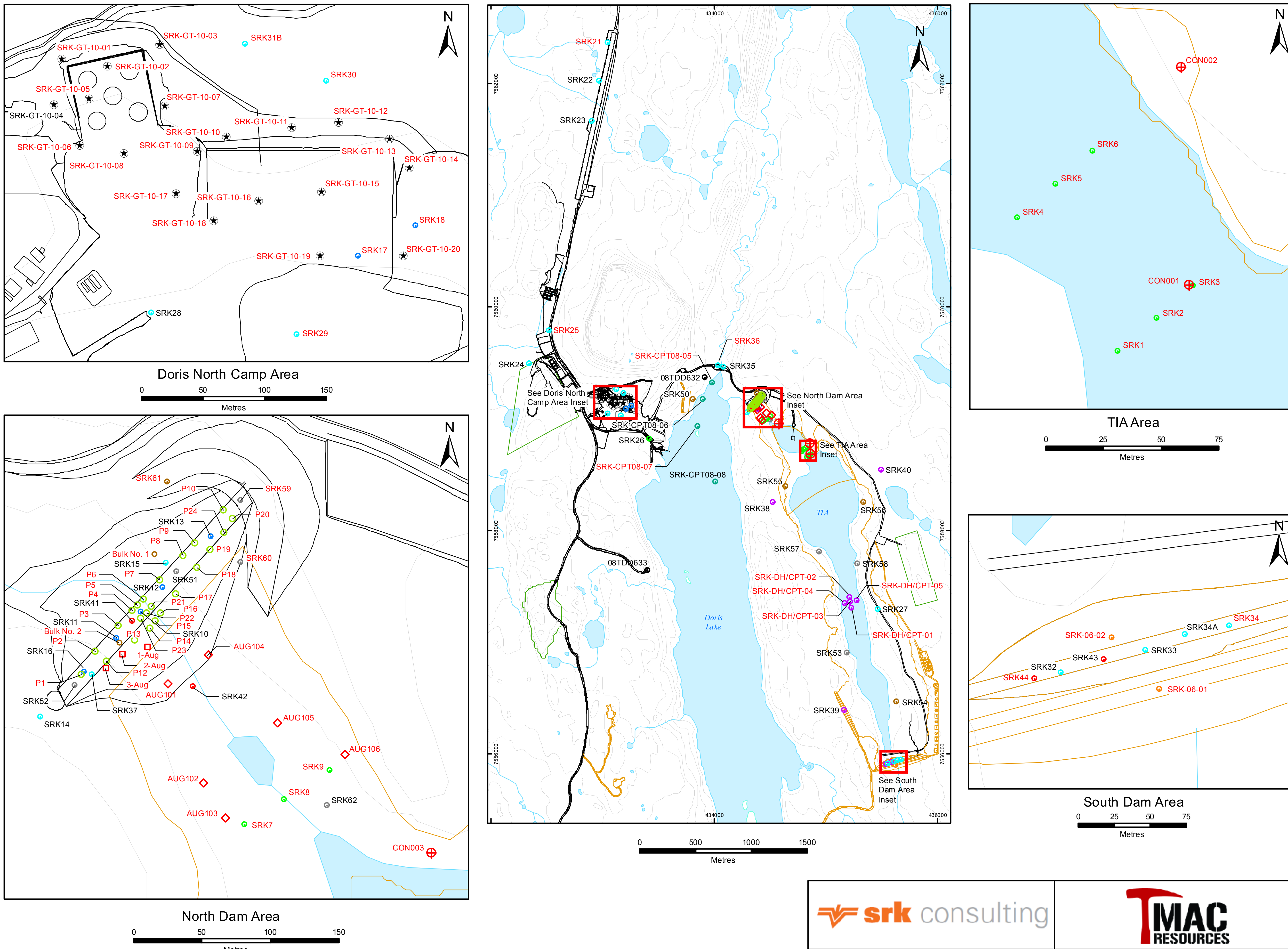

Legend

SRK58	Geotechnical Drill Holes with Thermistor Strings
SRK27	Geotechnical Drill Holes without Thermistor Strings
	Geotechnical Auger Drill Holes (Summer 2003)
	Geotechnical Test Pits (Summer 2003)
	Geotechnical Bulk Sampling (Summer 2004)
	Vane Shearing Testing (Winter 2005)
	Condemnation Holes (2002)
	Percolation Drill Holes (Winter 2011)
	Geotechnical Drill Holes (Fall 2002)
	Geotechnical Drill Holes (Winter 2002)
	Geotechnical Drill Holes (Winter 2003)
	Geotechnical Drill Holes (Summer 2003)
	Geotechnical Drill Holes (Winter 2004)
	Geotechnical Drill Holes (Summer 2004)
	Geotechnical Drill Holes (Winter 2005)
	Geotechnical Drill Holes (Winter 2006)
	Geotechnical Drill Holes (Winter 2010)
	Geotechnical Drill Holes (1997)
	Geotechnical Drill Holes (2007)
	Geotechnical Drill Holes (2008)
	Geotechnical Drill holes (Winter 2008)
	Geotechnical Drill Holes (Winter 2016)
	Existing Infrastructure
	Future Infrastructure
	Permitted Quarries
	Future Quarries

Notes:
Coordinate System: NAD 1983 UTM Zone 13N

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MAC RESOURCES

Job No: 1CT022.013
Filename: 1CT022.013_Fig_7_DorisCamp_TIA_mzs_RevB

HOPE BAY PROJECT

Overburden Summary Report

Overburden Characterization Holes at the Doris North Mining Area (Doris Camp and TIA)

Date:	Approved:	Figure:
Dec 2017	CH	7

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Legend

SRK58 Geotechnical Drill Holes with Thermistor Strings

SRK27 Geotechnical Drill Holes without Thermistor Strings

Geotechnical Drill Holes (2007)

Geotechnical Drill Holes (Winter 2008)

Geotechnical Drill Holes (2014)

Existing Infrastructure

Future Infrastructure

Permitted Quarries

Future Quarries

0 500 1000 1500
Metres

Notes:
Coordinate System: NAD 1983 UTM Zone 13N

srk consulting

TMAC RESOURCES

Overburden Summary Report

Overburden Characterization Holes at the Madrid Mining Area

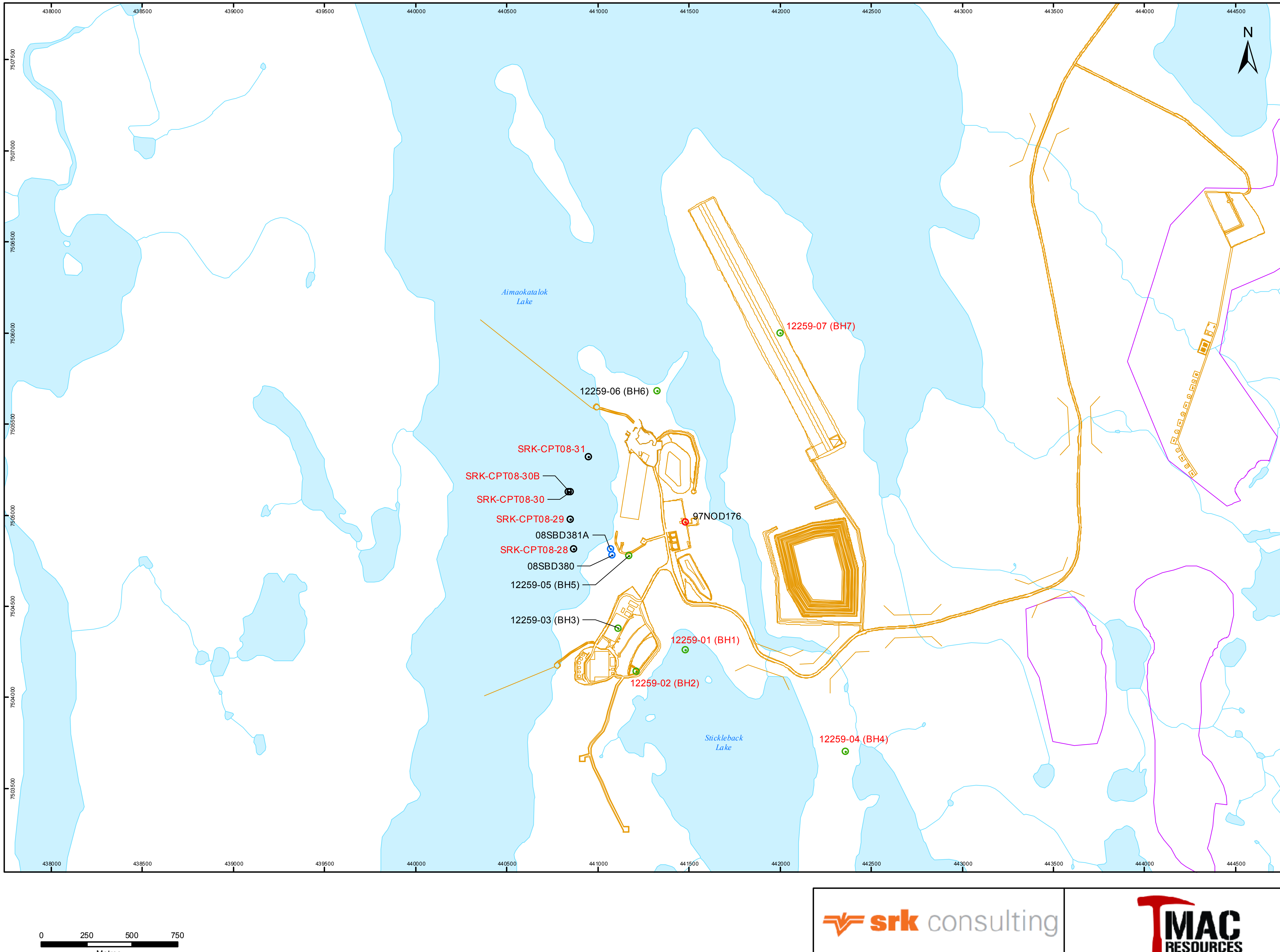
Job No: 1CT022.013

Filename: 1CT022.013_Fig_8_MadridArea_mzs_RevB

HOPE BAY PROJECT

Date:	Approved:	Figure:
Dec 2017	CH	8

\\van-svr0\projects\01_SITES\Hope.Bay\1CT022.013_Phase_2_FEIS_Water_Licence_Submission\!040_AutoCAD\GIS_MXD\Overburden_Summary_Dec2017\1CT022.013_Fig_8_MadridArea_mzs_RevB.mxd



Legend

SRK58 Geotechnical Drill Holes with Thermistor Strings

SRK27 Geotechnical Drill Holes without Thermistor Strings

Geotechnical Drill Holes (Winter 1996)

Geotechnical Drill Holes (Winter 1997)

CPT Holes (Winter 2008)

Geotechnical Drill Holes (Winter 2008)

Existing Infrastructure

Future Infrastructure

Permitted Quarries

Future Quarries

Notes:
Coordinate System: NAD 1983 UTM Zone 13N

srk consulting

TMAC RESOURCES

Overburden Summary Report

Overburden Characterization Holes at the Boston Mining Area

Job No: 1CT022.013

Filename: 1CT022.013_Fig_9_BostonMiningArea_mzs_RevB

HOPE BAY PROJECT

Date:	Approved:	Figure:
Dec 2017	CH	9

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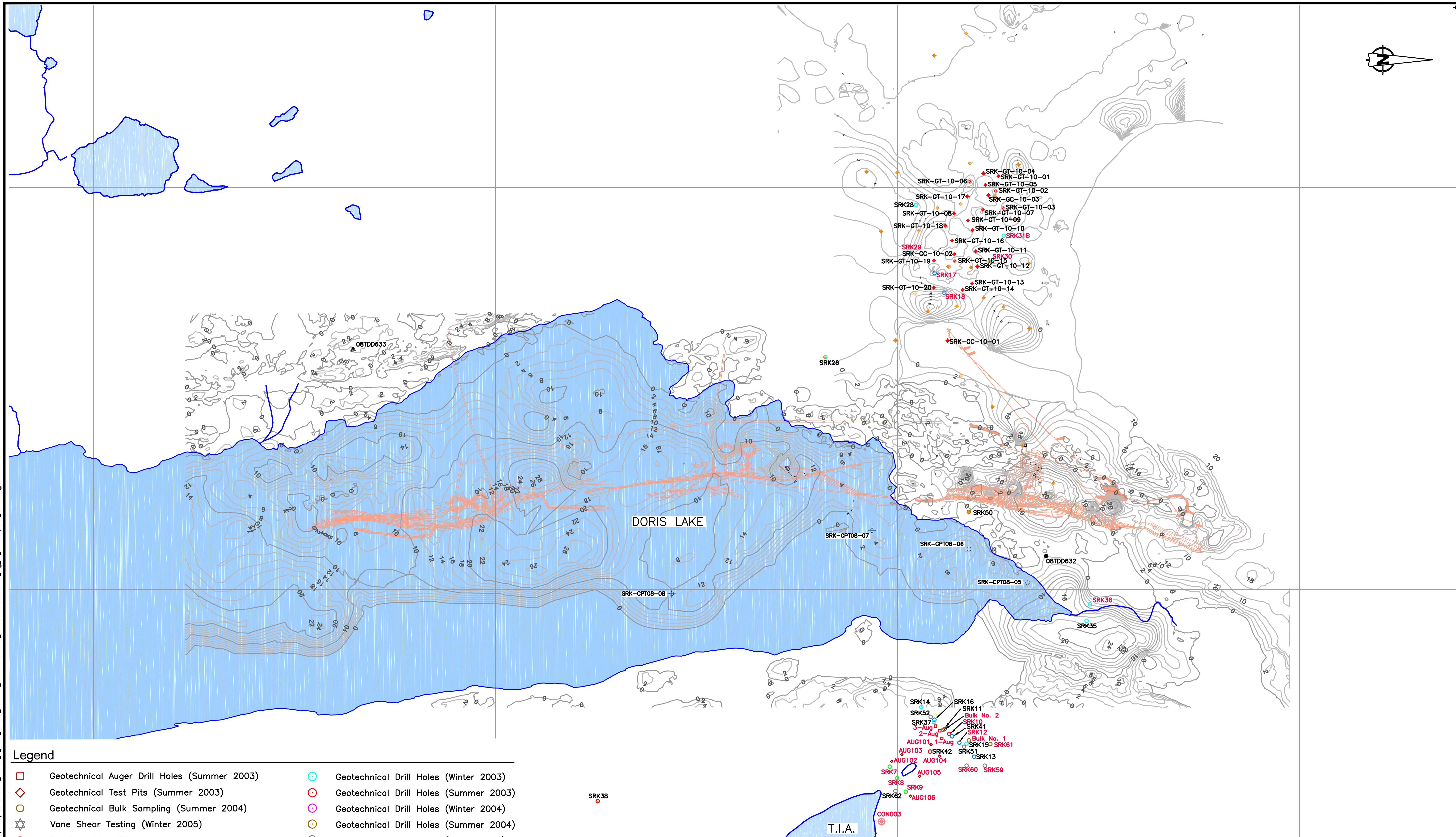

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SRK JOB NO.: 1CT022.013

FILE NAME: 1CT022.013_Fig_10_OvbIsopach_Doris.dwg

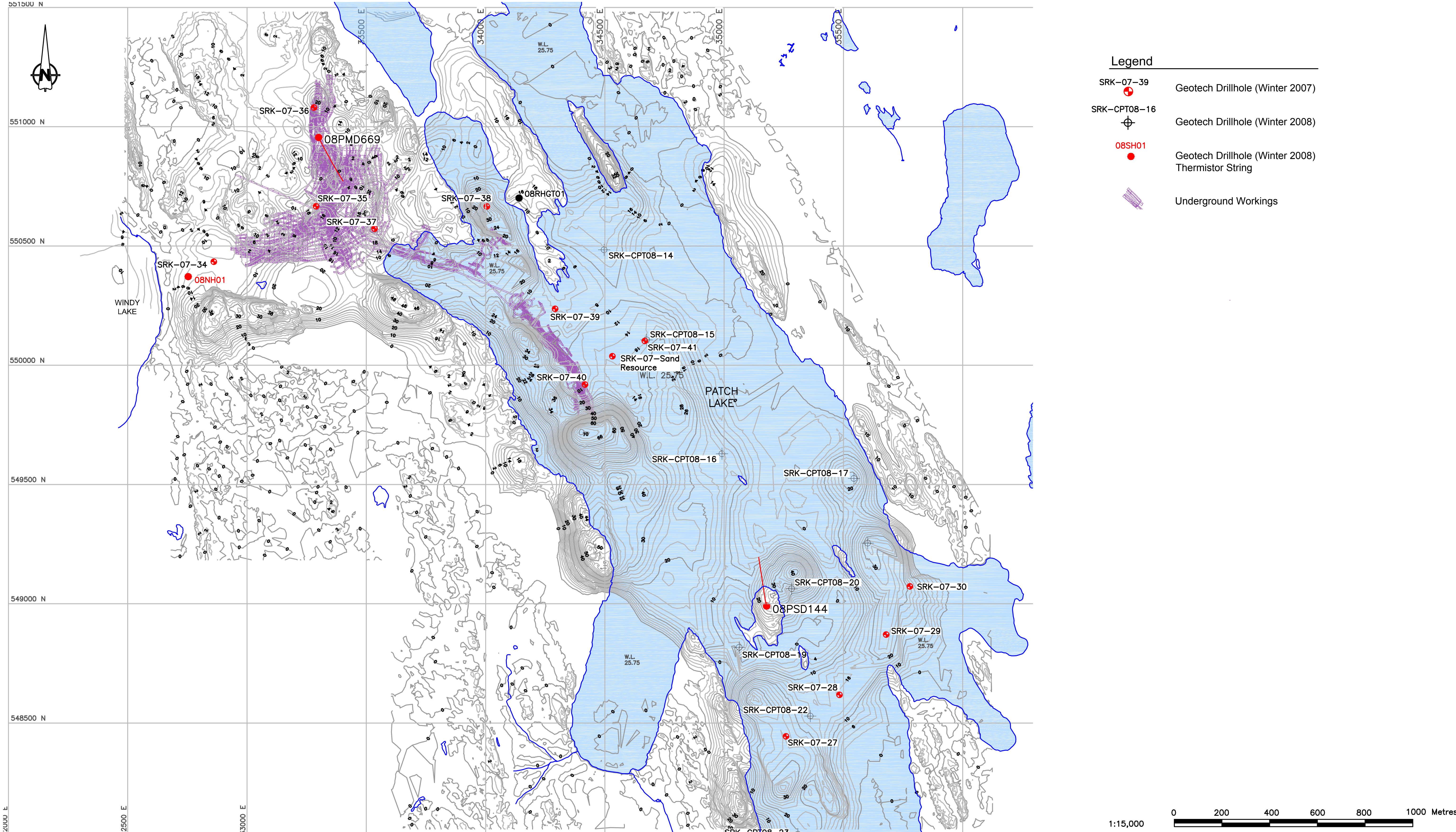
TMAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Overburden Isopach at the Doris Mining Area

DATE:	APPROVED:	FIGURE:
Nov. 2017	MMM	10



srk consulting

SRK JOB NO.: 1CT022.013

FILE NAME: 1CT022.013_Fig_11_OvbIsopach_Madrid.dwg

MAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Overburden Isopach at the Madrid Mining Area

DATE:	APPROVED:	FIGURE:
Nov. 2017	MMM	11

\\VAN-SVR0\Projects\01_SITES\Hope.Bay\1CT022.013_Phase_2_FEIS_Water_Licence_Submission\040_AutoCAD\1CT022.013_Fig_11_OvbIsopach_Madrid.dwg

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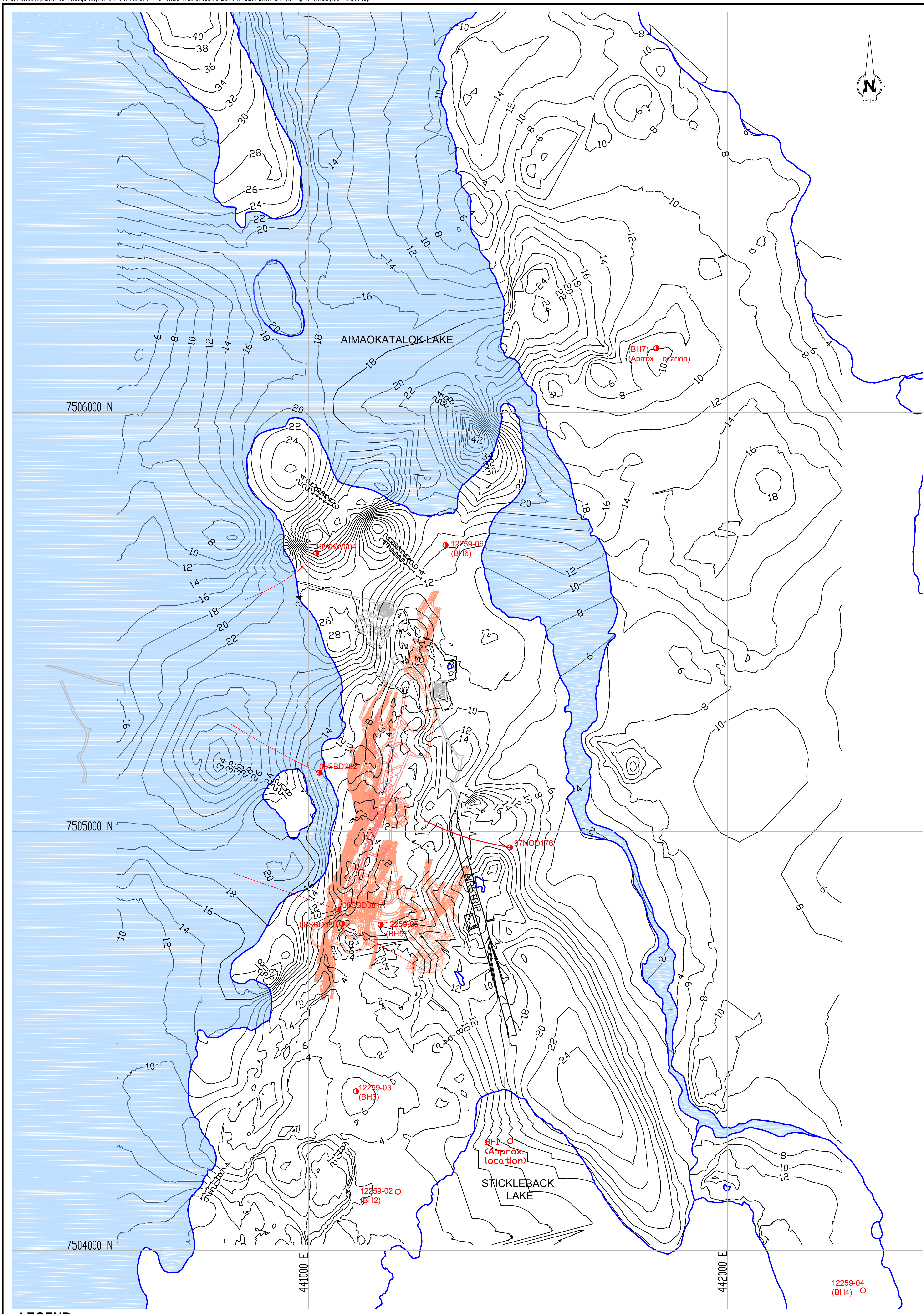

LEGEND

12259-03	Thermistor Installed
(BH1)	Thermistor Not Installed
08SBD380	Angled Holes
28	Ispoch 2m thickness Contour
	Underground Workings

0 50 100 150 200 250
Scale in Metres

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SRK JOB NO.: 1CT022.013
FILE NAME: 1CT022.013_Fig_12_OvbIsopach_Boston.dwg

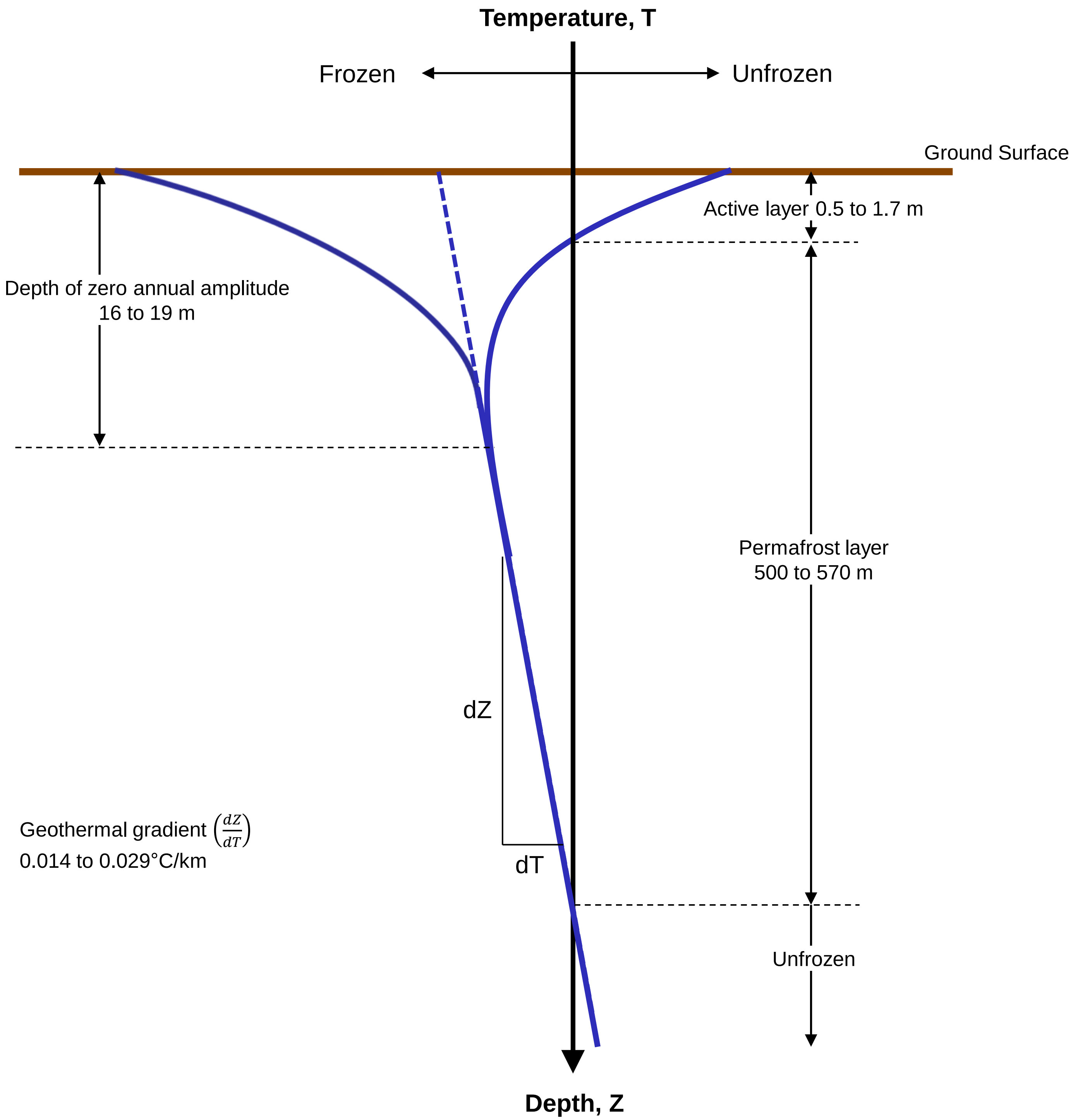
MAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Overburden Isopach at the Boston Mining Area

DATE:	APPROVED:	FIGURE:
Nov. 2017	MM	12



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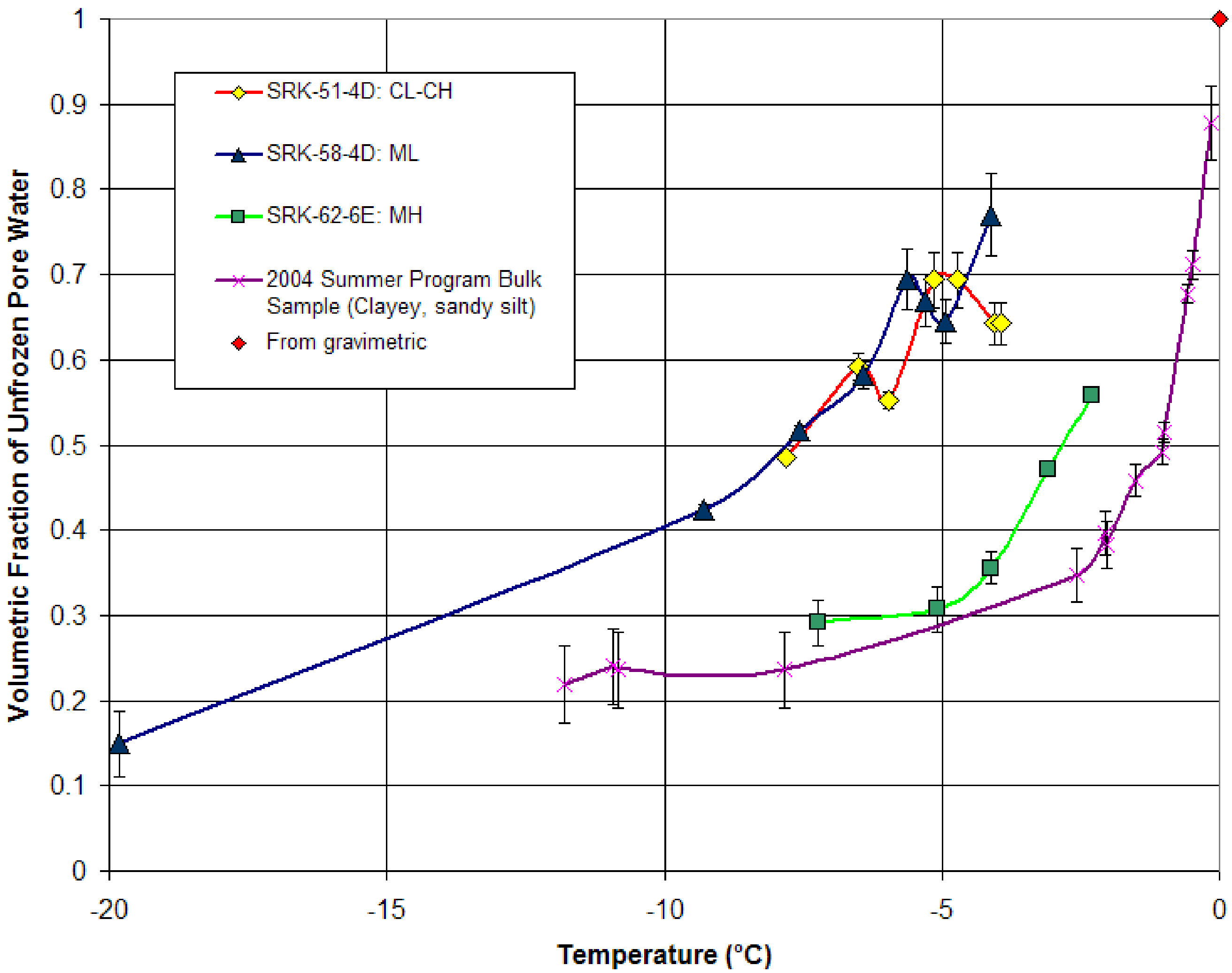
srk consulting	MAC RESOURCES	Overburden Summary Report **Ground Temperature and Permafrost Characteristics**
Job No: 1CT022.013 Filename: Fig13_PermafrosChar_1CT022.013_Rev00.pptx	Hope Bay Project	Date: Nov 2017 / Approved: MMM / Figure: **13**



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MAC RESOURCES

Overburden Summary Report

Unfrozen Volumetric Water Content versus Temperature

Job No: 1CT022.013

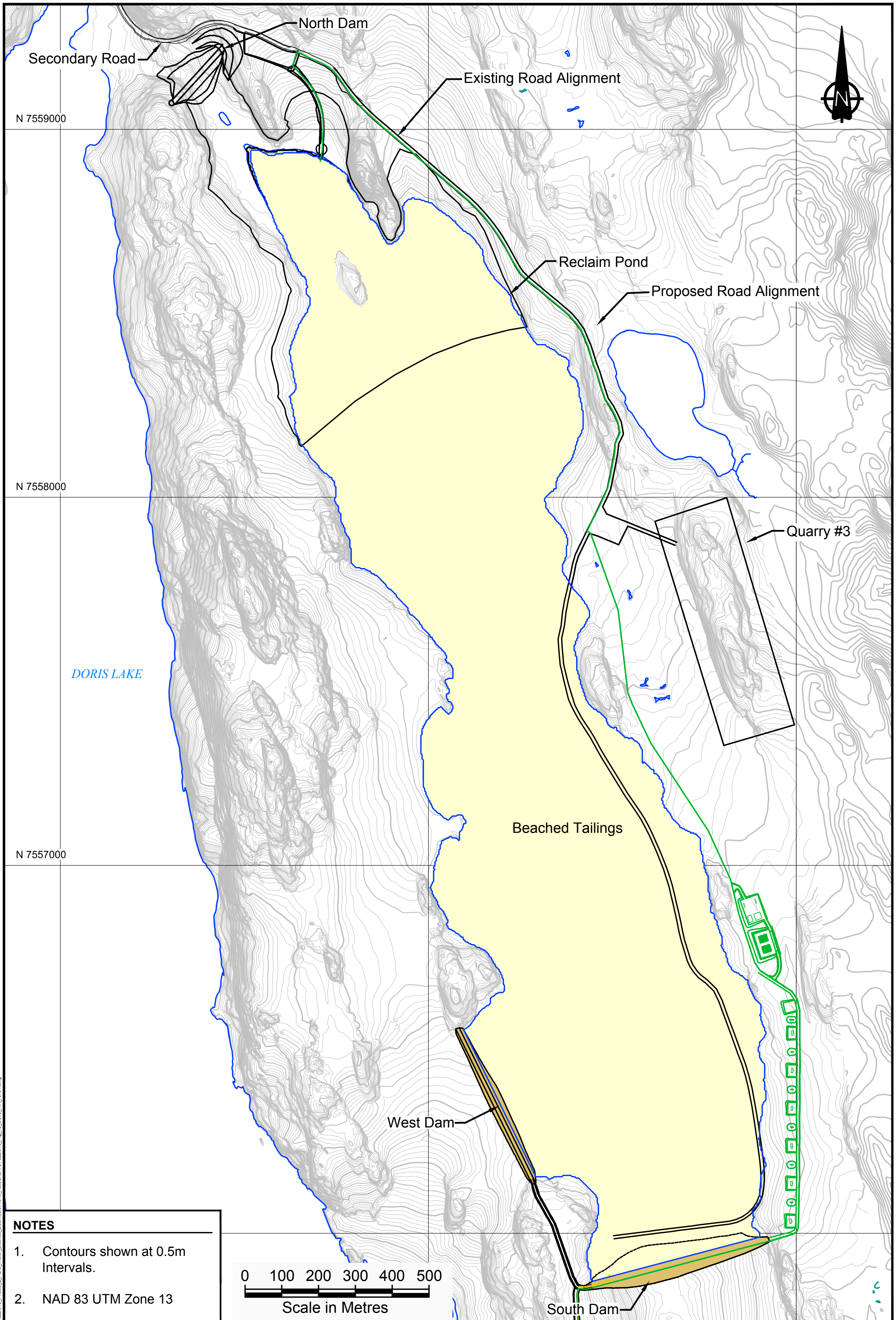
Filename: Fig14_UnfrozenWaterContent.pptx

Hope Bay Project

Date: Nov 2017

Approved: MMM

Figure: 14



NOTES

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

LEGEND

Proposed Tailings Facility
Proposed Dam/Dikes

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SRK JOB NO.: 1CT022.013
FILE NAME: 1CT022.013_Fig_15_Doris_TIA_rev01.dwg

MAC RESOURCES

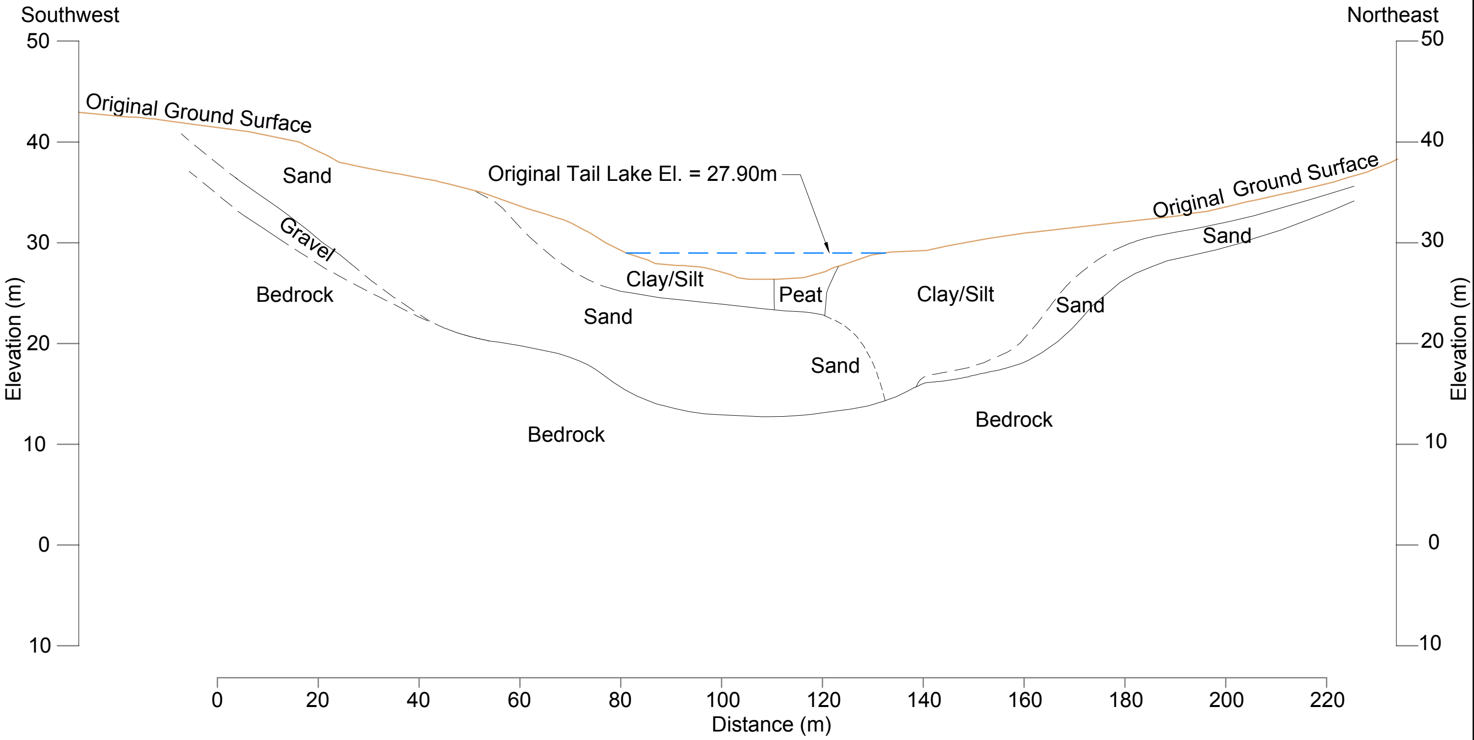
HOPE BAY PROJECT

Overburden Summary Report

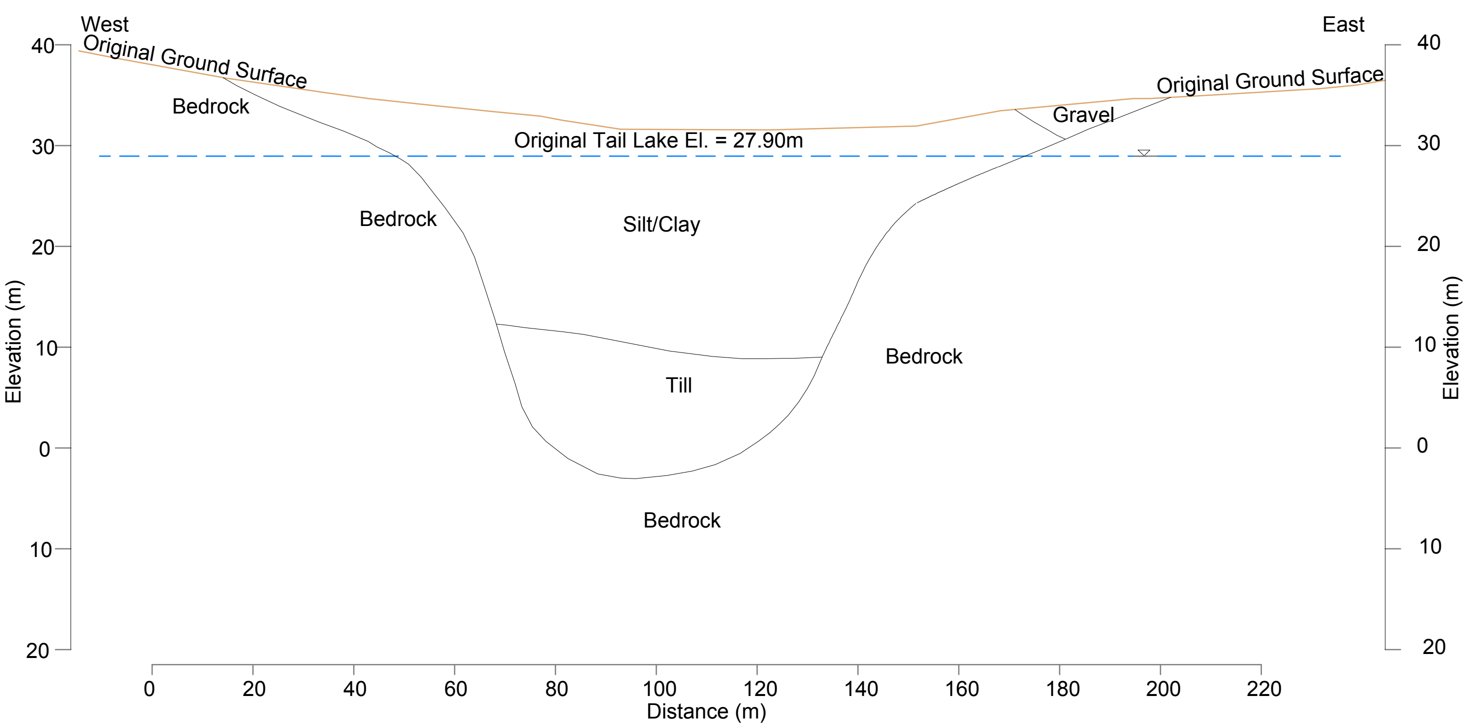
Doris TIA Layout

DATE:	APPROVED:	FIGURE:
Dec 2017	CH	15

\\VAN-SVR0\Projects\01_SITES\Hope.Bay\1CT022.013_Phase_2_FEIS_Water_Licence_Submission\!040_AutoCAD\1CT022.013_Fig_15_Doris_TIA_rev01.dwg



North Dam



South Dam

0 10 20 30 40 50
Horizontal Scale in Metres
Vertical Exaggeration x2

\\VAN-SVR0\Projects\01_SITES\Hope.Bay\1CT022.013_Phase_2_FEIS_Water_Licence_Submission\040_AutoCAD\1CT022.013_Fig_16_OVB_Stratographic.dwg

srk consulting

SRK JOB NO.: 1CT022.013
FILE NAME: 1CT022.013_Fig_16_OVB_Stratographic.dwg

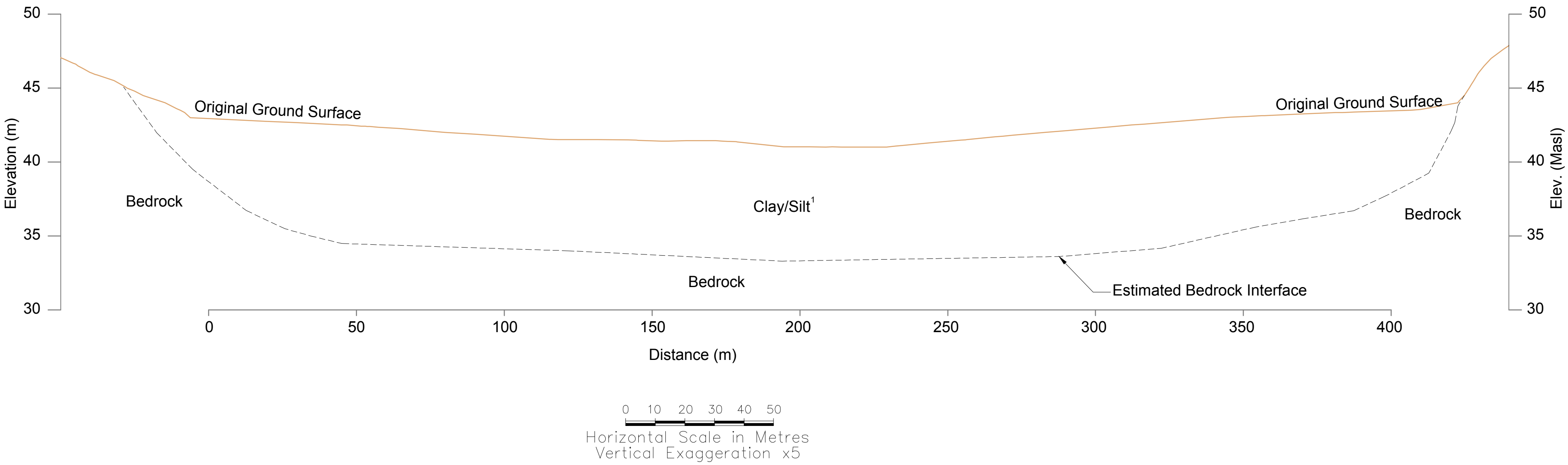
TMAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Interpreted Stratographic Profiles of the North and South Dams

DATE:	APPROVED:	FIGURE:
Nov. 2017	MMM	16



NOTES

1. Stratigraphic profile based on a single borehole log. Bedrock contact and overburden material assumed.

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SRK JOB NO.: 1CT022.013

FILE NAME: 1CT022.013_Fig_17_OVB_Stratographic_WDam.dwg

TMAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Interpreted Stratographic Profile of the West Dam

DATE:	APPROVED:	FIGURE:
Nov. 2017	MMM	17

Appendix A – Thermistor String Summary

SRK	Installation Date	Northing	Easting	Status	Area	Data Summary	Serial Number	Notes
SRK-11	Sep 2002	7559117.00	434347.00	Inactive	Doris	Sep 2002 - Jul 2010	00577-2	
SRK-13	Sep 2002	7559171.72	434383.32	Inactive	Doris	Sep 2002 - Aug 2003	00577-1	
SRK-14	Apr 2003	7559059.45	434291.66	Active	Doris	Apr 2003 - Sep 2015	690007	
SRK-15	Mar 2003	7559171.62	434383.00	Inactive	Doris	Apr 2003 - Oct 2008, Jul 2010	690012	
SRK-16	Sep 2002	7559092.00	434323.20	Inactive	Doris	Sep 2002 - Jul 2010	0577-3	
SRK-19	Apr 2003	7563211.92	432983.69	Inactive	Roberts Bay	Apr 2003 - Oct 2008	690006	
SRK-20	Apr 2003	7563129.78	432986.02	Inactive	Roberts Bay	Apr 2003 - Oct 2008	690009	
SRK-22	Apr 2003	7562026.69	432971.94	Active	Roberts Bay	Apr 2003 - Feb 2016	690003	
SRK-23	Apr 2003	7561665.77	432901.86	Inactive	Roberts Bay	Apr 2003 - Oct 2008	690008	
SRK-24	Apr 2003	7559493.64	432344.49	Active	Doris	Apr 2003 - Sep 2015	690001	
SRK-26	Apr 2003	7558819.91	433422.37	Inactive	Doris	Apr 2003 - Oct 2008	690002	
SRK-28	Apr 2003	7559046.27	433043.30	Inactive	Doris	Apr 2003 - Jul 2009	690011	
SRK-32	Apr 2003	7555914.51	435554.73	Inactive	Doris	Apr 2003 - Aug 2013	690010	
SRK-33	Apr 2003	7555930.36	435613.59	Active	Doris	Apr 2003 - Oct 2014	690005	
SRK-34A	Apr 2003	7555941.61	435640.69	Inactive	Doris	Apr 2003 - Oct 2008	690004	
SRK10-DCB2 Doris Bridge East	Jul 2011	7559478.35	434036.99	Active	Doris	Jul 2011 - Present	TS3017	Installed in bridge abutment.
SRK10-DCB1 Doris Bridge West	Jul 2011	7559475.15	434067.76	Active	Doris	Jul 2011 - Present	TS3016	Installed in bridge abutment.
SRK-35	Apr 2003	7559477.53	434035.64	Inactive	Doris	Apr 2003 - Nov 2010	690013	
SRK-37	Mar 2003	7559090.54	434328.97	Inactive	Doris	Apr 2003 - Jul 2010	690013	
SRK-38	Aug 2003	7558254.33	434525.84	Active	Doris	Aug 2003 - Sep 2015	TS0015	
SRK-39	Aug 2003	7556391.33	435164.13	Active	Doris	Aug 2003 - Oct 2014	TS0011	
SRK-40	Aug 2003	7558546.86	435492.39	Inactive	Doris	Aug 2003 - Oct 2008	TS0014	
SRK-41	Aug 2003	7559129.11	434358.55	Inactive	Doris	Aug 2003 - Oct 2010	TS0012	
SRK-42	Aug 2003	7559081.34	434402.62	Inactive	Doris	Aug 2003 - Jul 2010	TS0013	
SRK-43	Aug 2003	7555923.82	435584.52	Inactive	Doris	Aug 2003 - Oct 2008	TS0010	
SRK-50	Aug 2004	7559177.00	433807.00	Active	Doris	Aug 2004 - Nov 2014	TS1618	
SRK-51	Apr 2005	7559165.54	434390.70	Inactive	Doris	Apr 2005 - Jul 2010	TS2048	
SRK-52	Apr 2005	7559082.73	434316.33	Inactive	Doris	Apr 2005 - Jul 2010	TS2047	
SRK-53	Apr 2005	7556906.93	435184.24	Inactive	Doris	Apr 2005 - Aug 2013	TS1625	
SRK-54	Sep 2004	7556467.00	435632.00	Inactive	Doris	Sep 2004 - Jul 2010	TS1626	
SRK-55	Sep 2004	7557813.27	434935.95	Inactive	Doris	Sep 2004 - Sep 2004	TS1621	
SRK-56	Sep 2004	7558258.00	435334.00	Inactive	Doris	Sep 2004 - Oct 2005	TS1621	
SRK-57	Apr 2005	7557812.13	434937.72	Active	Doris	Apr 2005 - Oct 2014	TS1623	
SRK-58	Apr 2005	7557704.54	435284.89	Inactive	Doris	Apr 2005 - Aug 2012	TS1622	
SRK-62	Apr 2005	7558994.93	434500.74	Inactive	Doris	Apr 2005 - Jul 2010	TS2046	
SRK-JT1-09	Mar 2009	7563297.00	432534.00	Active	Roberts Bay Jetty	Mar 2009 - Present	TS2667	
SRK-JT2-09	Mar 2009	7563264.00	432550.00	Inactive	Roberts Bay Jetty	Mar 2009 - Nov 2011	TS2668	
SRK-JT2-12	May 2012	7563264.00	432550.00	Inactive	Roberts Bay Jetty	May 2012 - Sep 2012	TS3019	
08SBD380	Jul 2008	7504780.24	441079.71	Unknown	Boston	Jul 2008 - Aug 2010	VW8891/TS2717	
08SBD381A	Aug 2008	7504813.94	441070.40	Unknown	Boston	Aug 2008 - Sep 2009	VW8887/TS2713	
08SBD382	Aug 2008	7505140.53	441025.86	Unknown	Boston	Aug 2008 - Oct 2014	VW8888/TS2717	
08PMD669	Jul 2008	7550955.12	433300.23	Unknown	Madrid	Jul 2008 - Aug 2010	VW8847/TS2711	
08PSD144	Oct 2008	7548989.92	435177.97	Unknown	Madrid	Sep 2008 - Aug 2010	VW8890/TS2716	
08TDD632	Jun 2008	7559369.75	433915.20	Inactive	Doris	Jun 2008 - Jul 2010	VW8826/TS2706	
08TDD633	Jun 2008	7557646.05	433402.21	Inactive	Doris	No Data	VW8846/TS2710	
SRK-12-GTC-DH01	Apr 2012	7558917.20	433169.18	Active	Doris	Apr 2012 - Present	TS3260	Installed in pollution control pond berm.
SRK-12-GTC-DH02	Apr 2012	7558912.96	433225.25	Active	Doris	Apr 2012 - Present	TS3261	Installed in pollution control pond berm.
SRK-12-GTC-DH03	Apr 2012	7558930.81	433225.25	Active	Doris	Apr 2012 - Present	TS3262	Installed in pollution control pond berm.
SRK10-DWB1	Apr 2012	7555673.50	432703.40	Active	Madrid	Apr 2012 - Present	TS3021	Installed in bridge abutment.
SRK10-DWB2	Apr 2012	7555644.40	432708.20	Active	Madrid	Apr 2012 - Present	TS3025	Installed in bridge abutment.
SRK10-DWB3	Apr 2012	755615.00	432712.80	Active	Madrid	Apr 2012 - Present	TS3020	Installed in bridge abutment.
SRK10-DWB4	Apr 2012	7554860.30	432444.00	Active	Madrid	Apr 2012 - Present	TS3024	Installed in bridge abutment.
SRK10-DWB5	Apr 2012	7554831.30	732437.00	Active	Madrid	Apr 2012 - Present	TS3023	Installed in bridge abutment.
ND-HTS-040-31.5	Apr 2011	7559100.71	434324.01	Active	Doris	Aug 2012 - Present	TS3091	Installed in North Dam.
ND-HTS-040-33.5	Mar 2012	7559100.71	434324.01	Active	Doris	Aug 2012 - Oct 2013, May 2014 - Present	TS3102	Installed in North Dam.
ND-VTS-040-KT	Mar 2011	7559100.71	434324.01	Active	Doris	Aug 2012 - Present	TS3080	Installed in North Dam.
ND-VTS-060-DS	Feb 2011	7559115.28	434337.72	Active	Doris	Aug 2012 - Present	TS3086	Installed in North Dam.
ND-HTS-060-33.5	Mar 2012	7559115.28	434337.72	Active	Doris	Aug 2012 - Present	TS3099	Installed in North Dam.
ND-HTS-060-31.0	Feb 2012	7559115.28	434337.72	Active	Doris	Aug 2012 - Present	TS3096	Installed in North Dam.
ND-HTS-060-28.8	Apr 2011	7559115.28	434337.72	Active	Doris	Aug 2012 - Present	TS3092	Installed in North Dam.
ND-VTS-060-KT	Mar 2011	7559115.28	434337.72	Active	Doris	Aug 2012 - Present	TS3081	Installed in North Dam.
ND-VTS-060-US	Feb 2011	7559106.54	434346.46	Inactive	Doris	No Data	TS3085	Damaged during construction.
ND-VTS-085-DS	Feb 2011	7559133.96	434353.91	Active	Doris	Aug 2012 - Present	TS3088	Installed in North Dam.
ND-HTS-085-25.3	Apr 2011	7559133.96	434353.91	Active	Doris	Aug 2012 - Present	TS3093	Installed in North Dam.
ND-HTS-085-29.4	Feb 2012	7559133.96	434353.91	Active	Doris	Aug 2012 - Present	TS3097	Installed in North Dam.
ND-HTS-085-33.5	Mar 2012	7559133.96	434353.91	Inactive	Doris	No Data	TS3100	Damaged during construction.
ND-VTS-085-KT	Mar 2011	7559133.96	434353.91	Active	Doris	Aug 2012 - Present	TS3082	Installed in North Dam.
ND-VTS-085-US	Feb 2011	7559125.08	434363.23	Active	Doris	Aug 2012 - Present	TS3087	Installed in North Dam.
ND-VTS-130-DS	Feb 2011	7559167.23	434384.47	Active	Doris	Aug 2012 - Present	TS3090	Installed in North Dam.
ND-HTS-130-28.8	Apr 2011	7559167.23	434384.47	Active	Doris	Aug 2012 - Present	TS3094	Installed in North Dam.
ND-HTS-130-31.0	Feb 2012	7559167.23	434384.47	Active	Doris	Aug 2012 - Present	TS3098	Installed in North Dam.
ND-HTS-130-33.5	Mar 2012	7559167.23	434384.47	Active	Doris	Aug 2012 - Present	TS3101	Installed in North Dam.
ND-VTS-130-KT	Mar 2011	7559167.23	434384.47	Active	Doris	Aug 2012 - Present	TS3083	Installed in North Dam.
ND-VTS-130-US	Feb 2011	7559158.49	434393.93	Active	Doris	Aug 2012 - Present	TS3089	Installed in North Dam.
ND-HTS-175-32.5	Apr 2011	7559200.63	434414.72	Active	Doris	Aug 2012 - Present	TS3095	Installed in North Dam.
ND-HTS-175.33.5	Feb 2012	7559200.63	434414.72	Active	Doris	Aug 2012 - Present	TS3103	Installed in North Dam.
ND-VTS-175-KT	Mar 2011	7559200.63	434414.72	Active	Doris	Aug 2012 - Present	TS3084	Installed in North Dam.
12259-97-01 / DH#1	May 1997	7565767.60	431164.80	Inactive	Roberts Bay	Jun 1997, Apr 2003	1135	
12259-97-8 / DH#8	1997	7565625.00	431129.60	Inactive	Roberts Bay	No Data	1136	Damaged
12259-97-17 / DH#17	May 1997	7565519.50	431211.40	Inactive	Roberts Bay	Jun 1997	1134	
12259-03	May 1996	7504380.00	441113.00	Inactive	Boston	May 1996 - Sep 2001	1049	
12259-05	May 1996	7504778.00	441172.00	Inactive	Boston	May 1996 - Jun 1996	1050	
12259-96-06	May 1996	7505683.00	441327.00	Inactive	Boston	May 1996 - Sep 2001	1051	
97NOD176	Jun 1905	7504962.00	441481.00	Unknown	Boston	Oct 1997 - Sep 2001	1130	
TM00141	Jul 2014	7546691.1	435141.3	Active	Madrid	Apr 2015 - Jan 2016	TS3787	
TDD-242	May 2000	15549.98(1)	5067.82(1)	Inactive	Doris	Aug 2000 - Sep 2001	#2	
TDD-261	2000	15224.89(1)	4917.00(1)	Inactive	Doris	No Data	#1	
CX3(2)-13	1997	N/A	N/A	Inactive	Boston	No Data	N/A	Mentioned in Golder 2001. Drilled in Boston Underground, last face at 3935 m level cross cut.
DB#27-H	1997	N/A	N/A	Inactive	Boston	No Data	1142	Mentioned in Golder 2001. Drilled in Boston Underground, Drill bay #27.
DB#27-V	1997	N/A	N/A	Inactive	Boston	No Data	1142	Mentioned in Golder 2001. Drilled in Boston Underground, Drill bay #27.
DB#36	1997	N/A	N/A	Inactive	Boston	No Data	N/A	Mentioned in Golder 2001. Drilled in Boston Underground, Drill bay #36.

Notes:
(1) Local mine grid, conversion to UTM from this grid is unknown
(2) Coordinate system is UTM NAD83, Zone 13

Appendix B – Seismic Hazard Analysis

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Vancouver, BC V6E 3X2

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Megan Miller, PEng	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Horizontal Seismic Parameters for Pseudo-Static Modelling		

Change Log

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

Changes by Section

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

1 Introduction

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

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

The Phase 2 project has several components that require slope stability analysis, including:

- North, South and West dams at the Doris Tailings Impoundment Area (TIA);
- Waste rock piles at Madrid North, Madrid South and Boston; and
- Dry stack tailings in the Boston Tailings Management Area (TMA).

All of these facilities will be founded on permafrost overburden of varying thickness. The overburden on site is comprised of ice rich marine clays and silts, with an active layer thickness of approximately 1 m (SRK, 2017a).

This memo presents the methodology for determining horizontal and vertical seismic parameters to be used in pseudo static slope stability analysis on the Project site. The values presented herein are site specific and dependant on foundation conditions, and embankment height.

2 Seismic Parameter Calculations

2.1 Site Ground Motions

Ground motions for the Boston and Madrid mining areas were obtained from the 2015 National Building Code of Canada seismic hazard calculator (NRC, 2016). Both mining areas are expected to have the same ground motions (Attachment 1), and are summarized in Table 2.1.

Ground motions for spectral periods of 0.05 s to 10.0 s and the peak ground acceleration (PGA) for the 1:10,000 year event were estimated by plotting the annual probability of exceedance and spectral acceleration (Sa(T)) values of the 1:476 and 1:4,275-year events on a log-log scale and extending the line to the annual probability of exceedance for the 1:10,000-year event, as outlined by NRC (2016). The extrapolated ground motions are a very rough estimation, which likely over estimates hazard, and a site specific hazard assessment is recommended (NRC, 2016). However, since the site is located in a low seismic zone, slope stability modelling with the extrapolated 1:10,000-year event can be used as a screening tool to determine if additional seismic analysis is required for the closure scenario.

The ground motions obtained from the seismic hazard calculator are for soils classified as Site Class C: very dense soil and soft rock. Ground motions for other material types are obtained by converting the Site Class C values to the average material type over the top 30 m of the soil profile using formulas and factors provided in Humar (2015) and NRCC (2015). Table 2.2 provides the properties used to define the different site classes.

Assuming thawed conditions, the overburden foundations under the specified infrastructure are Soft Soils (Site Class E) due to the natural moisture content (>40%), and undrained shear strength (11 to 25 kPa) expected in the marine silts and clays (Table 2.2). Even if other types of overburden or bedrock are present within the infrastructure foundations, these foundations would be classified as Soft Soils (Site Class E) because the marine clays and silts are likely more than 3 m thick (Table 2.2). Permafrost soils could likely be considered Site Class B or Site Class C (Table 2.2); however, since the Site Class E soils amplify ground accelerations using Site Class E for all analysis was adopted as a conservative approach.

Table 2.1: Site Class C Ground Motions for the Project

Spectral Period or Peak Parameter	Ground Accelerations (g)			
	1:100 year	1:476 year	1:1,000 year	1:2,475 year
Sa(0.05)	0.0034	0.012	0.021	0.042
Sa(0.1)	0.0056	0.019	0.031	0.059
Sa(0.2)	0.0069	0.021	0.032	0.056
Sa(0.3)	0.0065	0.019	0.029	0.047
Sa(0.5)	0.0051	0.017	0.025	0.038
Sa(1.0)	0.0026	0.0096	0.015	0.023
Sa(2.0)	0.0010	0.0041	0.0064	0.011
Sa(5.0)	0.0004	0.0009	0.0014	0.0023
Sa(10.0)	0.0003	0.0006	0.0008	0.0011
Peak Ground Acceleration (PGA)	0.0033	0.011	0.017	0.032

Source: NRCC 2016

Table 2.2: Site Classification for Seismic Site Response

Site Class	Ground Profile Name	Average Properties for Top 30 m of Profile		
		Average Shear Wave Velocity, $\bar{V}_s$(m/s)	**Average Standard Penetration Resistance, $\bar{N}_{60}$**	**Soil Undrained Shear Strength, s_u**
A	Hard Rock(2)	$\bar{V}_s > 1{,}500$	N/A	N/A
B	Rock(2)	$760 < \bar{V}_s \leq 1{,}500$	N/A	N/A
C	Very Dense Soil and Soft Rock	$360 < \bar{V}_s \leq 760$	$\bar{N}_{60} > 50$	s_u > 100 kPa
D	Stiff Soil	$360 < \bar{V}_s \leq 760$	$\overline{15 \leq N_{60}} \leq 50$	50 kPa < s_u ≤ 100 kPa
E	Soft Soil	$\bar{V}_s < 180$	$\bar{N}_{60}$<15	s_u < 50 kPa
		Any soil with more than 3 m of soil with the following characteristics: 1. Plasticity Index: PI ≥ 20 2. Moisture content: w ≥ 40% 3. Undrained shear strength: s_u < 25 kPa		
F(1)	Other Soils	Other soils include: 1. Liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, and other soils susceptible to failure or collapse under seismic loading 2. Peat and/or highly organic clays greater than 3 m thickness 3. Highly plastic clays (PI>75) more than 8 m thick 4. Soft to medium stiff clays more than 30 m thick		

Source: Adapted from National Building Code of Canada 2015 Table 4.1.8.4-A (NRCC 2015)

Notes:

(1) Site specific evaluation required.

(2) Site Classes A and B are not to be used if there is more than 3 m of softer materials between the rock and the underside of the footing or mat foundations. If more than 3 m of softer materials exist the Site Class is determined based on the average properties of the softer materials.

2.2 Horizontal Seismic Parameters

The horizontal seismic parameters were calculated from the site adjusted ground motions using the Limit Equilibrium Pseudo Static Stability Analysis method presented in Section 6.2.2 of the LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Reference Manual (FHWA, 2011).

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 factor of safety (FOS) of 1.1. While a larger allowable deformation is unlikely to affect the stability of the waste rock piles, dry stack tailings and Doris TIA dams, this criteria is conservative.

As the horizontal seismic parameter is dependent on slope height, soil properties, and design earthquake it was calculated separately for each component. The horizontal seismic parameter values are provided in Section 3.

2.3 Vertical Seismic Parameters

For most earthquakes the horizontal acceleration component is much greater than the vertical acceleration component; therefore, the vertical seismic coefficient is commonly assumed to be zero (Seed and Whitman 1970; FHWA 2011 and Anderson *et al*., 2008).

3 Results

Table 3.1 presents the horizontal seismic coefficients for the Phase 2 infrastructure requiring stability analysis, assuming a FOS of 1.1. The vertical seismic coefficients are assumed to be negligible. The selection of the seismic event for each structure is based on Canadian Dam Association design guidelines (CDA, 2014) and the Mined Rock and Overburden Piles Investigation and Design Manual (Piteau, 1991). The selection of the appropriate seismic event from the design guidelines is described in the design documents of the infrastructure components (SRK 2017b, c, d, e and f).

Should analysis of other infrastructure be required, the horizontal seismic coefficients can be obtained from Table 3.1 assuming that the infrastructure is founded on a minimum of 3 m of marine silt and clay overburden (Site Class E).

Table 3.1: Horizontal Seismic Coefficients for Various Infrastructure at the Project

Structure	Critical Section Height (m)	Seismic Event	Horizontal Seismic Coefficient (g)
Operations			
North Dam	10	1:2,475	0.023
South Dam	15	1:2,475	0.021
West Dam	5	1:2,475	0.025
Madrid South Waste Rock Pile	20	1:476	0.0075
Madrid North Waste Rock Pile	20	1:476	0.0075
Boston Waste Rock Pile	25	1:476	0.0072
Boston TMA	25	1:2,475	0.018
Contact Water Pond Berms	2.5	1:476	0.0086
Closure			
South Dam	15	Halfway between 1:2,475 year and 1:10,000 year	0.036
West Dam	5	Halfway between 1:2,475 year and 1:10,000 year	0.043
Boston TMA	25	1:2,475	0.018

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\Seismic Hazard Analysis\[HopeBay_SeismicCoefficentCalculation_1CT022.004_20160510_mmm.xlsm]Summary

Table 3.2: Horizontal Seismic Coefficient for the Project, Soil Class E

Dam / Embankment Height (m)	Seismic Coefficient (g)				
	1:100 year	1:476 year	1:1,000 year	1:2,475 year	1: 10,000 year
≤ 5	0.0026	0.0086	0.013	0.025	0.061
10	0.0024	0.0083	0.013	0.023	0.056
15	0.0023	0.0079	0.012	0.021	0.051
20	0.0021	0.0075	0.012	0.020	0.046
25	0.0020	0.0072	0.011	0.018	0.041
30	0.0018	0.0068	0.011	0.016	0.036
≥ 35	0.0018	0.0067	0.011	0.016	0.035

Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\Seismic Hazard Analysis\[HopeBay_SeismicCoefficentCalculation_1CT022.004_20160510_mmm.xlsm]Summary

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SRK Consulting (Canada) Inc., 2017f. Hope Bay Project: Madrid North Surface Infrastructure Preliminary Design. Memo Prepared for TMAC Resources Inc. 1CT022.013. November 2017.

Attachment 1: 2015 National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

March 11, 2016

Site: 67.6578 N, 106.3849 W User File Reference: Boston Camp

Requested by: Megan Miller, SRK Consulting

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	**Sa(0.2)**	Sa(0.3)	**Sa(0.5)**	**Sa(1.0)**	**Sa(2.0)**	**Sa(5.0)**	**Sa(10.0)**	**PGA (g)**	**PGV (m/s)**
0.042	0.059	**0.056**	0.047	**0.038**	**0.023**	**0.011**	**0.0023**	**0.0011**	**0.032**	**0.027**

Notes. Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. ***These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0034	0.012	0.021
Sa(0.1)	0.0055	0.019	0.031
Sa(0.2)	0.0068	0.021	0.032
Sa(0.3)	0.0063	0.019	0.029
Sa(0.5)	0.0050	0.017	0.025
Sa(1.0)	0.0025	0.0094	0.015
Sa(2.0)	0.0010	0.0040	0.0063
Sa(5.0)	0.0004	0.0009	0.0013
Sa(10.0)	0.0003	0.0006	0.0008
PGA	0.0033	0.011	0.017
PGV	0.0028	0.010	0.017

References

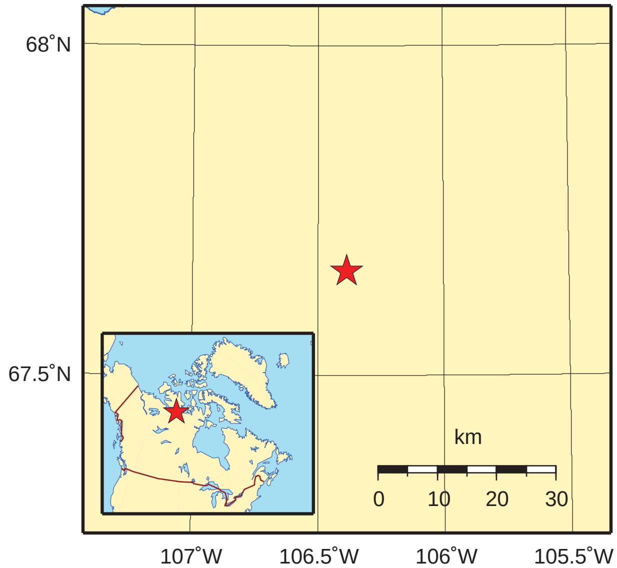
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



Appendix C – Thermal Modelling to Support Run-of-Quarry Pad Design

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Memo

To:	John Roberts, PEng, Vice President Environment	**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: Thermal Modelling to Support Run-of-Quarry Pad Design		

Change Log

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

The Project site is located in the continuous permafrost region of Canada, and the overburden soils consist of marine clay, which in some areas are ice rich. These soils, if thawed, may not have sufficient bearing capacity to support important surface infrastructure such as roads or building foundations. Therefore, these structures must be founded on bedrock with the excavation of the overburden soils, or alternately the overburden soils must be kept frozen.

This memo presents thermal modelling carried out to estimate the minimum run-of-quarry (ROQ) (or geochemically suitable run-of-mine (ROM) waste rock) pad thickness required to ensure that the underlying overburden soils remain frozen. This includes consideration of heated buildings and a depressed freezing point as a result of pore water salinity.

The thermal modelling was performed for an operating design life of 20 years with consideration for climate change. At closure the ROQ pads will remain; however, since they no longer have to functionally perform as a structural foundation, thaw settlement and consolidation is acceptable.

2 Ground Conditions

2.1 Overburden

Laboratory and in-situ testing on disturbed and undisturbed geotechnical samples collected during previous drilling campaigns confirm that onshore overburden soils are comprised mainly of marine clays, silty clay and clayey silt, with pockets of moraine till underlying these deposits. Soils in the region are overlain by a thin veneer of hummocky organic soil (SRK 2017a). The marine clay (silty clay and clayey silt) is typically between 5 and 20 m thick, with variable pore water salinity typically between 37 and 47 parts per thousand (ppt.) (SRK 2017a). Ground ice is typically 10 to 30% by volume, but occasionally as high as 50%. Local till typically contains ice contents ranging from 5 to 25%.

The most prevalent rock type on site with surface exposure is mafic volcanics, predominantly basalt. In isolated locations there are small amounts of gabbro, felsic volcanic and granitoids.

2.2 Permafrost

Ground temperature measured at the Project site indicates an average permafrost temperature of −7.6°C, with a range from −5.6°C to −9.8°C (Figures 1 –4). These statistics are based on temperature measurements near the depth of zero annual amplitude from 37 baseline sites located in the Doris Mine, and the Madrid and Boston mining areas. The baseline ground temperature sites do not permit for separate assessment of permafrost temperatures at each of the three mining areas.

Average active layer thickness was calculated to average 1.0 m (range from 0.5 m to 1.4 m) for suitable measurements (Figures 1 and 2). The base of permafrost was calculated from 11 instrumented sites to average 398 mbgs, with a range from 78 mbgs to 570 mbgs depending on proximity to waterbodies (Figures 1 and 2). The geothermal gradient from deeper extents of permafrost was calculated to average 0.021°C m^{-1} (Figures 1 and 2).

3 Thermal Modelling

3.1 Approach

Thaw depth estimates were based on analytical and numerical models. Numerical simulation of conductive heat transfer under transient conditions was completed using the finite element model SVHeat version 6 developed by SoilVision Systems Ltd. and the FlexPDE Version 6.34 solver developed by PDE Solutions Inc. (SoilVision Systems 2004).

SVHeat one-dimensional (1D) model simulations were used to estimate thaw depth for areas not impacted by heated buildings (Section 4.1). Thaw depths beneath non-insulated buildings were based on a steady-state thermal model (Section 4.2). Further details of the steady state model can be found in Andersland and Ladanyi (2004). Thaw depths beneath insulated buildings were estimated using SVHeat two-dimensional (2D) model simulations (Section 4.3).

Multiple foundation temperatures (0°C, −1°C, and −2°C) were analyzed to assess the sensitivity of the results to changes in ROQ pad and/or insulation thickness. For design purposes, foundations were considered to be valid if the base of the pads remained colder than 0°C. Subsidence that occurs during normal operations would be considered manageable.

3.2 Model Inputs

3.2.1 Material Parameters

The material properties used in the thermal modelling are summarized in Table 1. Properties for the ROQ pads were taken from previous work performed by SRK at the Project site as described in SRK (2017a). Thermal properties for ridged polystyrene insulation were obtained from Andersland and Ladanyi (2004). The thermal properties for natural overburden clay were based on average soil properties and a freezing point depression of −2°C. An unfrozen water content curve for clay was included in the model with consideration for the freezing point depression in accordance with Banin and Anderson (1974). The thermal properties for peat represent measured values presented by Romanovsky and Osterkamp (2000).

Table 1: Material Thermal Properties

Material	Degree of Saturation (%)	Porosity	Thermal Conductivity, kJ/(m·day·°C)		Volumetric Heat Capacity, kJ/(m³·°C)	
			Unfrozen	Frozen	Unfrozen	Frozen
Run of Quarry	30	0.30	104	117	1,697	1,509
Polystyrene Insulation	0	-	3	3	38	38
Peat	100	0.65	48	138	2,600	2,200
Overburden Clay[1]	85	0.52	112	187	2,842	2,038

Notes:

1. Overburden clay includes a freezing point depression of −-2°C and unfrozen water content curve

3.2.2 Climate Boundary

A ground surface response curve was developed for the Project site, representing the ground temperature immediately below ground surface. The boundary condition was applied to the model as a 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: Model Climate Boundary Parameters

Thermal Model Parameter	Base Case	Sensitivity Values
Mean annual air temperature	−10.7°C	−10.7°C
Mean annual ground temperature[1]	−7.6°C	−7.6°C
Air temperature amplitude	21.0°C	21.0°C
Air climate change factor (C_A)	0.000203°C d^{-1}	0.000203°C d^{-1}
ROQ surface thawing n-factor (*nt*)	1.52	1.25^C and 2.01^w
ROQ surface freezing n-factor (*nf*)	0.86	1.02^C and 0.60^W
Geothermal gradient	0.021°C/m	0.021°C/m

Notes:
1. Mean annual air temperature for 2015 based on “R” analysis climate change projection for Doris Mining Area
2. Mean annual ground temperature based on average temperature near the depth of zero annual amplitude
3. Superscript C indicated cold case n-factor scenario and W indicates warm n-factors scenario

Mean annual air temperature (−10.7°C) is based on average “R” analysis values for the baseline period of 1979 to 2005 and adjusted to 2015 values based on climate change predictions (SRK 2017b). This mean annual air temperature is consistent with the average measured Doris air temperature in 2015 (−10.8°C) (ERM 2016).Amplitude is based on average “R” analysis values for the baseline period (SRK 2017b).

Seasonal n-factors are applied as multipliers of air temperature to estimate the ground surface temperature at the pad surface. The ROQ n-factors were based on average published values (Table 3). A ROQ freezing n-factor (*nf*) of 0.86 and thawing n-factor (*nt*) of 1.52 is considered reasonable base case conditions for the Project site.

Table 3: Published N-factors for Gravel Surfaces

Surface Type	Site	Source	*nt*	*nf*
Sand and Gravel	Fairbanks, Alaska	US Army Corps. (1950)	2.00	0.90
Gravel	Fairbanks, Alaska	US Army Corps. (1950)	1.99	0.76
Gravel	Fairbanks, Alaska	US Army Corps. (1950)	2.01	0.63
Gravel	Fairbanks, Alaska	Carlson and Kersten (1953)	1.40	0.60
Gravel	Chitina, Alaska	Esch (1973)	1.47	1.00
Gravel - Dark color	Fairbanks, Alaska	Berg and Aitken (1973)	1.40	-
Gravel	Fairbanks, Alaska	US Army (1972)	1.50	-
Gravel - Dark color	Fairbanks, Alaska	US Army (1972)	1.27	-
Gravel	Inuvik, NWT, Canada	Johnston (1982)	1.33	0.96
Gravel	Inuvik, NWT, Canada	Johnston (1982)	1.49	0.94
Gravel	Inuvik, NWT, Canada	Johnston (1982)	1.33	0.88
Gravel	Inuvik, NWT, Canada	Johnston (1982)	1.36	0.91
Gravel	Inuvik, NWT, Canada	Johnston (1982)	1.48	1.02
Gravel	North Slope, Alaska	Klene et al. (2001)	1.25	-
		Average	**1.52**	**0.86**
		Minimum	**1.25**	**0.60**
		Maximum	**2.01**	**1.02**
		Standard Deviation	**0.26**	**0.14**
		Count	**14**	**10**

Notes:

1.Thawing n-factor (nt) and freezing n-factor (nf)

A conservative geothermal gradient of 0.021°C m^{-1} was applied to the lower boundary of the model which is consistent with average conditions measured at the Project site.

Climate change is considered in Equation 1 using the air climate change factor. This factor allows for a daily increase in air temperature. Air temperature is projected to increase by 2.6°C (0.74°C per decade) at Doris and 2.0°C (0.58°C per decade) at Boston between the period of 1979-2005 and 2011-2040, respectively. The rate of change projected for Doris was adopted as a more conservative input parameter to the model. The air climate change factor applied to Equation 1 in the model was 0.000203°C d^{-1} which is equivalent to an increase of 0.74°C per decade.

4 Model Results

4.1 Thaw Penetration Depth

A transient 1D model was constructed in SVHeat to estimate thaw penetration depth for ROQ pads for areas not thermally impacted by heated buildings and other surface infrastructure. The model was based on the input parameters outlined Table 2 and a sinusoidal surface ground temperature with average n-factors applied. All model runs consisted of 0.10 m peat underlain by clay which extended to 10 m below the base of the ROQ pad (Figure 5).

The model simulations are relatively simplistic as they do not account for lateral heat flow. Heat transported by surface water and near surface groundwater is also not accounted for in the model and would be expected to alter thermal conditions within and beneath the pad. However, at this level of design, the simplistic 1D model simulations are deemed appropriate.

Figure 5 summarizes the depth of the 0°C, −1°C, and −2°C isotherm for different ROQ pad thicknesses. The depths are relative to the base of the pad. The model estimates a minimum pad thickness of 1.9 m would be required to maintain the 0°C isotherm at the base of the pad assuming average n-factors. A minimum pad thickness of 2.2 m and 2.7 m were estimated to maintain the −1°C and −2°C isotherms at the base of the pad, respectively. For general design purposes, it is estimated that a minimum pad thickness of 1.9 m would be required.

The increase in active layer thaw below the pad for the 0°C, −1°C, and −2°C isotherms over the 20-year design life is shown in Figures 6 through 8. Seasonal thaw is estimated to increase over time due to increasing surface temperature described by Equation 1.

Figure 9 shows the sensitivity of thaw depth to changes in surface n-factors. A Cold Case and Warm Case scenario was modelled using the literature n-factors. The Cold Case represents the minimum thawing n-factor and maximum freezing n-factors presented in Table 3. The Warm Case represents the maximum thawing and minimum freezing n-factors presented in Table 3. The estimated pad thickness required to maintain the 0°C isotherm within the pad is 1.4 m and 2.9 m for the Cold Case and Warm Case, respectively.

4.2 Heated Buildings with Non-Insulated Foundation

The following section estimates thaw depth for heated buildings with non-insulated foundations constructed over a ROQ pad surface. Thaw depth calculations presented in this section were based on a steady-state heat strip method. Buildings are assumed to be rectangular with the plotted widths equal to the smallest dimension. Analyses were completed for buildings with the smallest dimension (width) ranging from 0 to 20 m and for interior temperatures from 5 to 30°C. The steady-state model assumes average interior temperature throughout the entire year and average ground temperatures.

The steady-state thaw depth for a heated building with no foundation insulation to maintain 0°C, −1°C, and −2°C isotherms within the ROQ pad is shown in Figures 10 through 12. The steady-state thaw depths are in general agreement with SVHeat numerical model simulations. The results show a linear relationship between the required pad thickness (thaw depth) and the minimum building dimension for buildings less than 20 m wide. As the building width increases, this relationship becomes non-linear with resultant increase in the required pad thickness. This analysis indicates that an insulated foundation is required for most heated buildings to maintain a foundation temperature below 0°C.

4.3 Heated Buildings with Insulated Foundations

The thaw depth for heated buildings with insulated foundations was analyzed using 2D SVHeat models. The transient models were based on the following:

- Polystyrene board insulation applied on top of the ROQ pad with a width equal to the building (thermal properties shown in Table 1);
- Simulations based on insulation ranging from 0 m to 0.5 m thick;
- Minimum building dimension of 20 m;
- Internal building heat at 5°C, 10°C, 15°C, 20°C, 30°C; and
- A sinusoidal surface ground temperature surrounding the building based on Equation 1 (see Table 2 for base case input parameters).

The results of the analysis are provided in Figures 13 through 5. The figures show combinations of ROQ pad and insulation thicknesses required to maintain the 0°C, −1°C, and −2°C isotherms within the pad material for different interior building temperatures.

The model results indicate that increasing the insulation thickness from 0.1 m to 0.2 m for a building heated to 30°C and assuming a thawing point of 0°C will reduce the required pad thickness from 16.8 m to 12.5 m (i.e. 100 mm insulation is equivalent to 4.3 m of ROQ pad). A further increase in insulation thickness from 0.2 m to 0.3 m will reduce the ROQ pad thickness by an additional 2.7 m from 12.5 m to 9.8 m.

The following example is provided to estimate the ROQ pad and insulation thickness requirements for a heated building with a minimum dimension other than 20 m.

Example: 10 m wide building heated to a constant internal temperature of 5°C and a thaw temperature of 0°C.

Step 1: Select a desired insulation thickness (0.1 m).

Step 2: From Figure 13 (thaw temperature of 0°C), a 0.1 m insulation layer for a 20 m wide building is estimated to require a ROQ pad thickness of 2.5 m.

Step 3: From Figure 10, a 10 m wide building requires approximately 50% of the ROQ pad thickness compared to a 20 m wide building (3.7 m vs. 7.5 m).

Step 4: Therefore, for a 0.1 m insulation layer, approximately 1.25 m of ROQ pad is required (50% of 2.5 m).

5 Conclusion

The analysis presented, which only includes thermal conduction, suggest that a ROQ (or geochemically suitable waste rock) pad design thickness of at least 1.9 m is required to maintain the 0°C isotherm at the base of the pad for areas not thermally impacted by heated buildings and other surface infrastructure. This assumes a 20-year design life with allowance for climate change. A minimum pad thickness of 2.2 m and 2.7 m are estimated to maintain the −1°C and −2°C isotherms at the base of the pad, respectively under the same conditions. For typical design purposes, the 0°C isotherm should be maintained within the pad to limit any potential subsidence to a manageable level.

A greater pad thickness as well as possible foundation insulation would be required to maintain thaw penetration within the pad for areas thermally influenced by heated buildings. For large heated buildings, it is likely that additional preventative measures are required to prevent permafrost degradation. These may include:

- Building footings or piles which raise the building from the ground surface and allow for circulation of cold air;
- Placement of thermosyphons beneath the buildings; or
- Placement of metal pipe ducts in the pads beneath the buildings to provide air circulation.

The conservatism built into the thermal analysis presented in this memo has been discussed. However to put into context, it is worth considering the performance of the ROQ rock fill roads, airstrip and general building pads that currently exist at the Project site, both at the Doris and Boston mining areas.

Underground development rock was used in 1996 and 1997 to construct a rock fill pad at the Boston site, as well as an airstrip and an all-weather access road to link these facilities. Because the material is mine development rock, it is predominantly 150 mm minus size material. The pads and the airstrip is nominally 1 m thick, and the all-weather road is less than 0.5 m thick.

At the Doris site, significant infrastructure has been constructed including almost 20 km of all-weather road, a 1.5 km long airstrip and multiple very large construction pads and laydown areas. All of these facilities were constructed with ROQ material with a maximum rock fill size of 1 m.

The roads, airstrip and various pads range in thickness from nominally 1 m to greater than 4 m; however, the predominant thickness is about 1 m. These facilities were constructed between 2007 and 2012, and consisted of both summer and winter construction.

Geotechnical inspections have been carried out annually at the Boston site since 2007 (SRK 2016a), and at the Doris site since 2009 (SRK 2016b). These inspections have confirmed that all pads, roads and airstrips have performed well and there have been no signs that suggest significant permafrost degradation has occurred, or are likely to start in the near term. Since many of these structures have thicknesses less than the recommended minimum design depth stated in this memo, it demonstrates that the calculated minimum design depths are conservative and that thinner pads can be constructed and evaluated using the observational approach over the life of the Project.

6 References

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Figures

Station ID	Northing	Easting	Location	Area	Data Exclusion and Limitations (See Notes)	ALT Average (m)	ALT Minimum (m)	ALT Maximum (m)	AL n	Geothermal Gradient (°C/m)	Base of Permafrost (mbgs)	Permafrost Temperature (°C)
SRK-11	7,559,117	434,347	North Dam	Doris Mining Area	1,4,5	<3.8	-	-	-	-	-	-7.9
SRK-13	7,559,172	434,383	North Dam	Doris Mining Area	6	-	-	-	-	-	-	-
SRK-14	7,559,059	434,292	Near North Dam	Doris Mining Area	1,5	1.3	1.0	1.4	11	-	-	-9
SRK-15	7,559,172	434,383	North Dam	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8.1
SRK-16	7,559,092	434,323	North Dam	Doris Mining Area	1,4,5	<3.3	-	-	-	-	-	-9.8
SRK-19	7,563,212	432,984	Beach Laydown Area	Roberts Bay	1,4,5	<0.9	-	-	-	-	-	-7.6
SRK-20	7,563,130	432,986	Beach Laydown Area	Roberts Bay	1,5	0.9	0.7	1.1	4	-	-	-7.3
SRK-22	7,562,027	432,972	East of Doris Airstrip	Roberts Bay	1,4,5	<0.7	-	-	-	-	-	-7.7
SRK-23	7,561,666	432,902	South Apron Doris Airstrip	Roberts Bay	1,4,5	<0.9	-	-	-	-	-	-7.9
SRK-24	7,559,494	432,344	Near crusher at Q2	Doris Mining Area	1,4,5	<0.7	-	-	-	-	-	-7.3
SRK-26	7,558,820	433,422	Junction Doris Rd and Tail Lk Road	Doris Mining Area	1,4,5	<0.8	-	-	-	-	-	-8.8
SRK-28	7,559,046	433,043	Camp Pad	Doris Mining Area	1,4,5	<0.8	-	-	-	-	-	-8.1
SRK-32	7,555,915	435,555	South Dam Area	Doris Mining Area	1,5	1.1	0.9	1.4	7	-	-	-8.4
SRK-33	7,555,930	435,614	South Dam Area	Doris Mining Area	1,4,5	<0.7	-	-	-	-	-	-8.7
SRK-34A	7,555,942	435,641	South Dam Area	Doris Mining Area	1,4,5	<0.9	-	-	-	-	-	-8.1
SRK10-DCB2	7,559,478	434,037	Doris Creek Bridge Abutment East	Doris Mining Area	2	-	-	-	-	-	-	-
SRK10-DCB1	7,559,475	434,068	Doris Creek Bridge Abutment West	Doris Mining Area	2	-	-	-	-	-	-	-
SRK-35	7,559,478	434,036	Doris Creek - West	Doris Mining Area	1,5	0.5	0.5	0.5	2	-	-	-6.2
SRK-37	7,559,091	434,329	North Dam	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8.2
SRK-38	7,558,254	434,526	Tail Lake West Side	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8.1
SRK-39	7,556,391	435,164	Tail Lake West Side	Doris Mining Area	1,3,5	-	-	-	-	-	-	-7.7
SRK-40	7,558,547	435,492	Tail Lake East Side	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8.7
SRK-41	7,559,129	434,359	North Dam Area	Doris Mining Area	1,3,5	-	-	-	-	-	-	-7.2
SRK-42	7,559,081	434,403	North Dam Area	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8
SRK-43	7,555,924	435,585	South Dam Area	Doris Mining Area	1,3,5	-	-	-	-	-	-	-8.7
SRK-50	7,559,177	433,807	Doris Lake North End	Doris Mining Area	1,3	-	-	-	-	0.019	394	-5.6
SRK-51	7,559,166	434,391	North Dam Area	Doris Mining Area	1,5	0.7	0.6	0.7	3	-	-	-
SRK-52	7,559,083	434,316	North Dam Area	Doris Mining Area	1,5	0.7	0.7	0.8	3	-	-	-
SRK-53	7,556,907	435,184	Tail Lake West Side	Doris Mining Area	1,5	1.0	0.8	1.2	6	-	-	-6.6
SRK-54	7,556,467	435,632	Tail Lake East Side	Doris Mining Area	1,5	1.2	1.1	1.3	4	-	-	-6.9
SRK-55	7,557,813	434,936	Tail Lake West Side	Doris Mining Area	6	-	-	-	-	-	-	-
SRK-56	7,558,258	435,334	Tail Lake East Side	Doris Mining Area	6	-	-	-	-	-	-	-
SRK-57	7,557,812	434,938	Tail Lake West Side	Doris Mining Area	1,4,5	<3.3	-	-	-	-	-	-8.1
SRK-58	7,557,705	435,285	Tail Lake East Side	Doris Mining Area	1,5	1.0	0.9	1.1	2	-	-	-7
SRK-62	7,558,995	434,501	Tail Lake North End	Doris Mining Area	1,5	0.9	0.9	1.0	4	-	-	-
TM00141	7,546,691	435,141	Patch Lake	Madrid Mining Area	1,3,5	-	-	-	-	0.023	346	-
SRK-JT1-09	7,563,297	432,534	Jetty	Roberts Bay Jetty	2	-	-	-	-	-	-	-
SRK-JT2-09	7,563,264	432,550	Jetty	Roberts Bay Jetty	2	-	-	-	-	-	-	-
SRK-JT2-12	7,563,264	432,550	Jetty	Roberts Bay Jetty	2	-	-	-	-	-	-	-
08SBD380	7,504,780	441,080	South of Boston Camp	Boston Mining Area	1,3	-	-	-	-	0.017	565	-7.1
08SBD381A	7,504,814	441,070	South of Boston Camp	Boston Mining Area	1,3	-	-	-	-	0.029	281	-6.1
08SBD382	7,505,141	441,026	South of Boston Camp	Boston Mining Area	1,3	-	-	-	-	0.027	302	-6.2
08PMD669	7,550,955	433,300	Between Patch and Windy Lakes (N)	Madrid Mining Area	1,3	-	-	-	-	0.018	570	-7.6
08PSD144	7,548,990	435,178	Patch Lake Island	Madrid Mining Area	5,8	-	-	-	-	-	78	-
08TDD632	7,559,370	433,915	West Side Doris Lake N	Doris Mining Area	1	-	-	-	-	0.024	445	-6.7
08TDD633	7,557,646	433,402	West Side Doris Lake	Doris Mining Area	6	-	-	-	-	-	-	-
10WBW001	7,557,537	433,778	Beneath Doris Lake	Doris Mining Area	7	-	-	-	-	-	-	-
10WBW002	7,559,375	433,913	Doris Site	Doris Mining Area	1,3,5	-	-	-	-	0.014	511	-7.1
10WBW004	7,505,665	441,018	Boston Site	Doris Mining Area	1,3	-	-	-	-	0.018	326	-6.1
12259-97-01 / DH#1	7,565,768	431,165	Onshore West Side of Roberts Bay	Roberts Bay	6	-	-	-	-	-	-	-
12259-97-8 / DH#8	7,565,625	431,130	Onshore West Side of Roberts Bay	Roberts Bay	8	-	-	-	-	-	-	-
12259-97-17 / DH#17	7,565,520	431,211	Onshore West Side of Roberts Bay	Roberts Bay	8	-	-	-	-	-	-	-
12259-03	7,504,380	441,113	Boston Site	Boston Mining Area	1,5	0.9	0.8	0.9	2	-	-	-7.0
12259-05	7,504,778	441,172	Boston Site	Boston Mining Area	6	-	-	-	-	-	-	-
12259-96-06	7,505,683	441,327	Boston Site	Boston Mining Area	1,5	1.7	1.6	1.7	2	-	-	-7.8
97NOD176	7,504,962	441,481	Boston Site	Boston Mining Area	1,3,5	-	-	-	-	0.019	556	-9.0

Notes:

1. Statistics shown on Figure 2
2. See Notes on Figure 3

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MAC RESOURCES

Infrastructure Thermal Modeling

Permafrost Characteristics from Baseline Measurements

Job No: 1CT022.013

Filename: PermafrostCharacteristics.pptx

HOPE BAY PROJECT

Date: 10/19/2017 | Approved: cws | Figure: 1

Station ID	Northing	Easting	Location	Area	Data Exclusion and Limitations (See Notes)	ALT Average (m)	ALT Minimum (m)	ALT Maximum (m)	AL n	Geothermal Gradient (°C/m)	Base of Permafrost (mbgs)	Permafrost Temperature (°C)
CX3(3)-13	-	-	Boston UG Workings	Boston Mining Area	6	-	-	-	-	-	-	-
TDD-242	-	-	-	Doris Mining Area	8	-	-	-	-	-	-	-
TDD-261	-	-	-	Doris Mining Area	6	-	-	-	-	-	-	-
DB#27-H	-	-	Boston UG Workings	Boston Mining Area	6	-	-	-	-	-	-	-
DB#27-V	-	-	Boston UG Workings	Boston Mining Area	6	-	-	-	-	-	-	-
DB#36	-	-	Boston UG Workings	Boston Mining Area	6	-	-	-	-	-	-	-
SRK-12-GTC-DH01	7,558,917	433,169	Pollution Control Pond	Doris Mining Area	2	-	-	-	-	-	-	-
SRK-12-GTC-DH02	7,558,913	433,225	Pollution Control Pond	Doris Mining Area	2	-	-	-	-	-	-	-
SRK-12-GTC-DH03	7,558,931	433,225	Pollution Control Pond	Doris Mining Area	2	-	-	-	-	-	-	-
SRK10-DWB1	7,555,674	432,703	Doris-Windy Road Bridge #2	Madrid Mining Area	2	-	-	-	-	-	-	-
SRK10-DWB2	7,555,644	432,708	Doris-Windy Road Bridge #2 / #3	Madrid Mining Area	2	-	-	-	-	-	-	-
SRK10-DWB3	7,556,150	432,713	Doris-Windy Road Bridge #3	Madrid Mining Area	2	-	-	-	-	-	-	-
SRK10-DWB4	7,554,860	432,444	Doris-Windy Road Bridge #4	Madrid Mining Area	2	-	-	-	-	-	-	-
SRK10-DWB5	7,554,831	732,437	Doris-Windy Road Bridge #4	Madrid Mining Area	2	-	-	-	-	-	-	-
ND-HTS-040-31.5	7,559,101	434,324	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-040-33.5	7,559,101	434,324	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-040-KT	7,559,101	434,324	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-060-DS	7,559,115	434,338	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-060-33.5	7,559,115	434,338	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-060-31.0	7,559,115	434,338	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-060-28.8	7,559,115	434,338	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-060-KT	7,559,115	434,338	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-060-US	7,559,107	434,346	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-085-DS	7,559,134	434,354	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-085-25.3	7,559,134	434,354	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-085-29.4	7,559,134	434,354	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-085-33.5	7,559,134	434,354	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-085-KT	7,559,134	434,354	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-085-US	7,559,125	434,363	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-130-DS	7,559,167	434,384	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-130-28.8	7,559,167	434,384	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-130-31.0	7,559,167	434,384	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-130-33.5	7,559,167	434,384	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-130-KT	7,559,167	434,384	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-130-US	7,559,158	434,394	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-175-32.5	7,559,201	434,415	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-HTS-175.33.5	7,559,201	434,415	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
ND-VTS-175-KT	7,559,201	434,415	North Dam	Doris Mining Area	2	-	-	-	-	-	-	-
					Average	**1.0**	**-**	**-**	**-**	**0.021**	**398**	**-7.6**
					Minimum	**-**	**0.5**	**-**	**-**	**0.014**	**78**	**-9.8**
					Maximum	**-**	**-**	**1.7**	**-**	**0.029**	**570**	**-5.6**
					n	**12**	**12**	**12**	**50**	**10**	**11**	**37**

Notes:

1. See Notes on Figure 3

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TMAC RESOURCES

Infrastructure Thermal Modeling

Permafrost Characteristics from Baseline Measurements

Job No: 1CT022.013

Filename: PermafrostCharacteristics.pptx

HOPE BAY PROJECT

Date: 10/19/2017

Approved: cws

Figure: 2

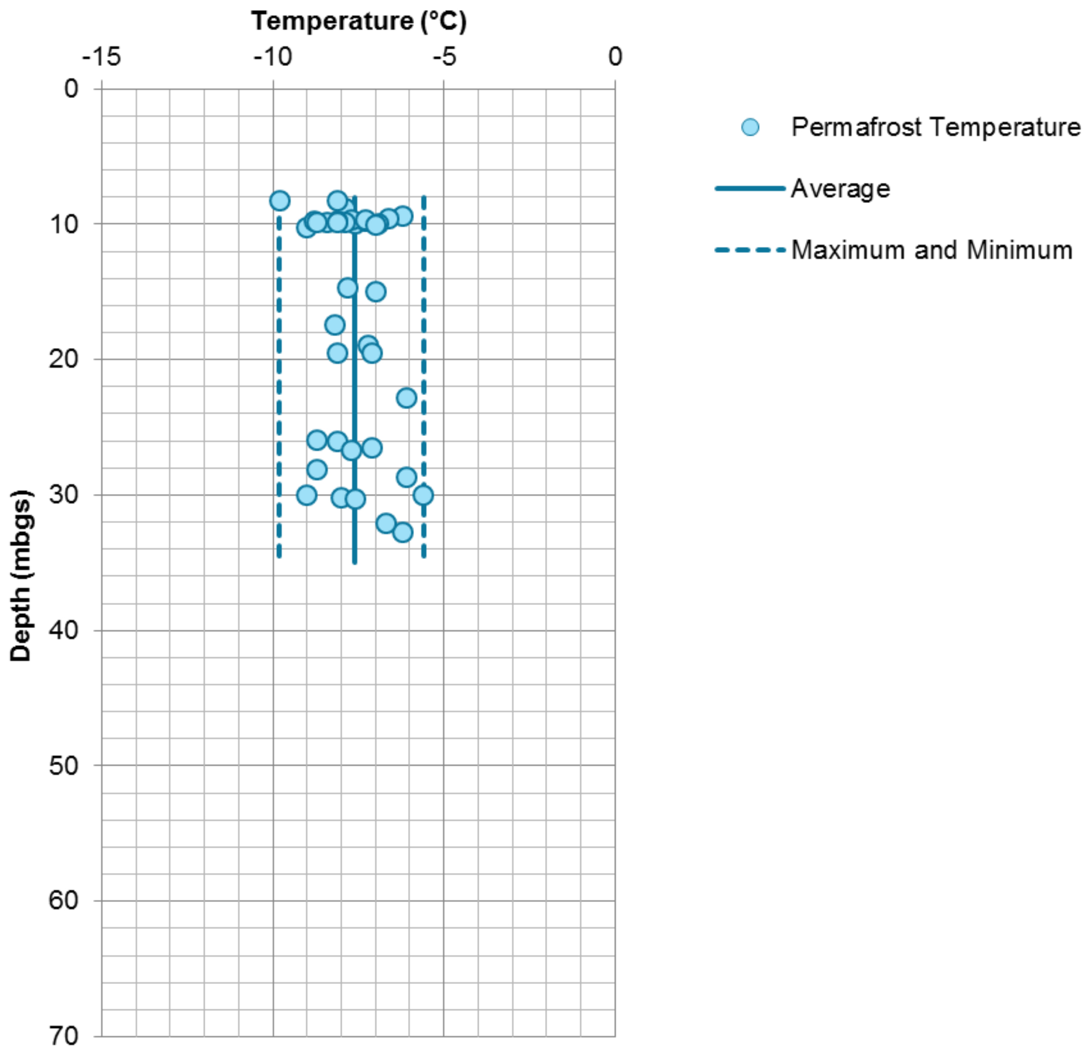
Notes:

Table Headings

1. Station ID – Thermistor cable identification
2. Northing and Easting – Universal Transverse Mercator coordinates for Stations (UTM NAD 83 Zone 13 N)
3. Location and Area provides descriptive location of the Station
4. Data Exclusion and Limitations index provided below
5. ALT Average – Average active layer thickness calculated for years with suitable data
6. ALT Minimum – Minimum active layer thickness calculated for years with suitable data
7. ALT Maximum – Maximum active layer thickness calculated for years with suitable data
8. AL n – Number of individual years with data suitable for active layer measurement
9. Geothermal Gradient – Calculated thermal gradient of deep permafrost for depths greater than 100 m
10. Base of Permafrost – Estimated bottom position of permafrost based on 0°C isotherm
11. Permafrost Temperature – Calculated between 8 and 33 m below ground surface based on data availability

Data Exclusion and Limitations Index

1. Baseline data used for permafrost characterization and statistics
2. Data excluded, monitoring site is not part of baseline monitoring
3. Measurement frequency, sensor position, and/or spacing not appropriate for estimation of active layer thickness
4. Active layer thickness constrained by upper sensor located within permafrost, value not included in statistics
5. Sensor position not appropriate for calculation of select permafrost characteristics
6. Insufficient data for analysis
7. Permafrost absent at instrumented site
8. Data in graphical form and not digital for exact calculation of temperature



Minimum Temperature	-9.8°C
Maximum Temperature	-5.6°C
Average Temperature	-7.6°C
Number of Stations	37

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Infrastructure Thermal Modeling

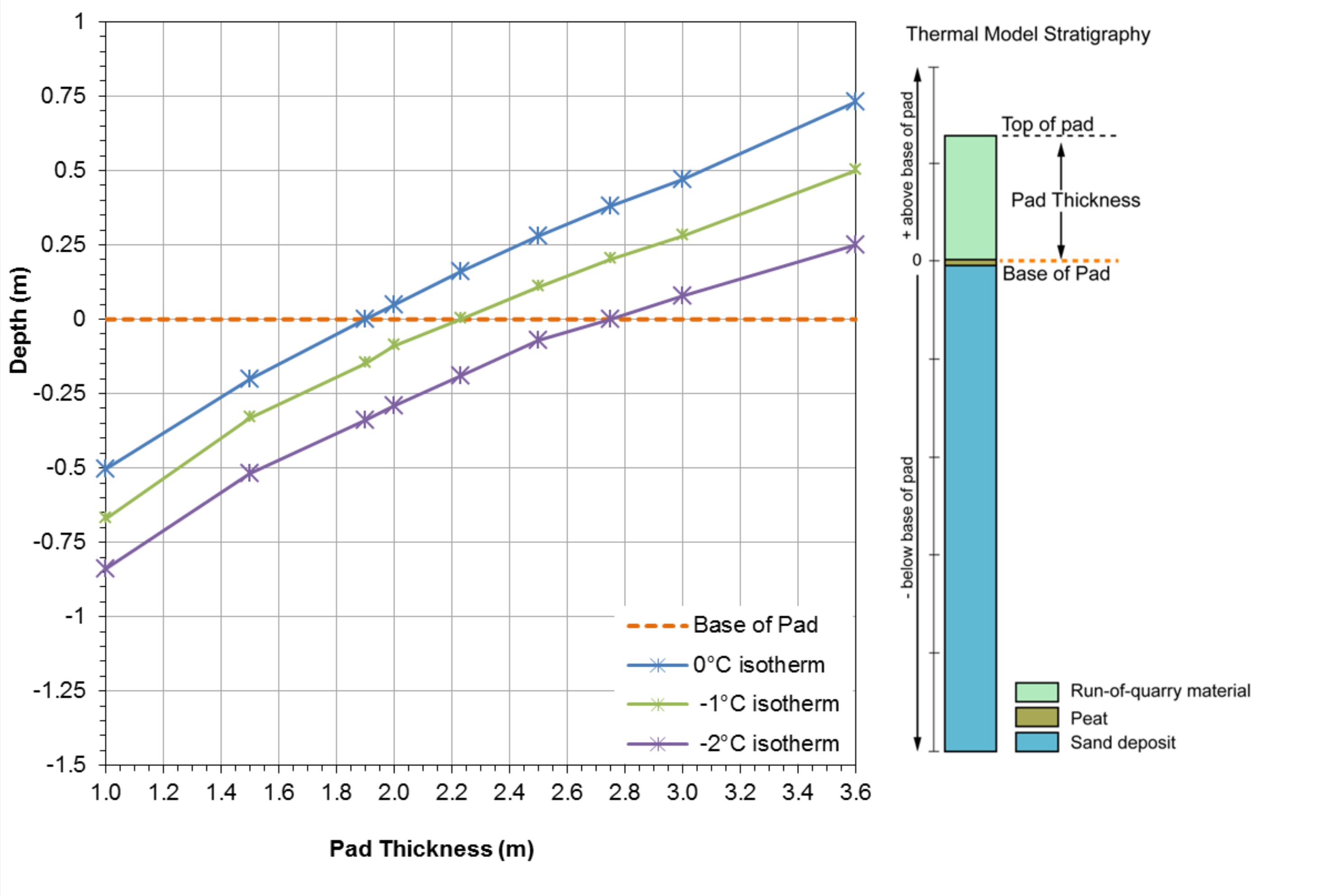
Permafrost Temperature

Job No: 1CT022.013

Filename: PermafrostTemperature_NearZeroAnnualAmp.pptx

HOPE BAY PROJECT

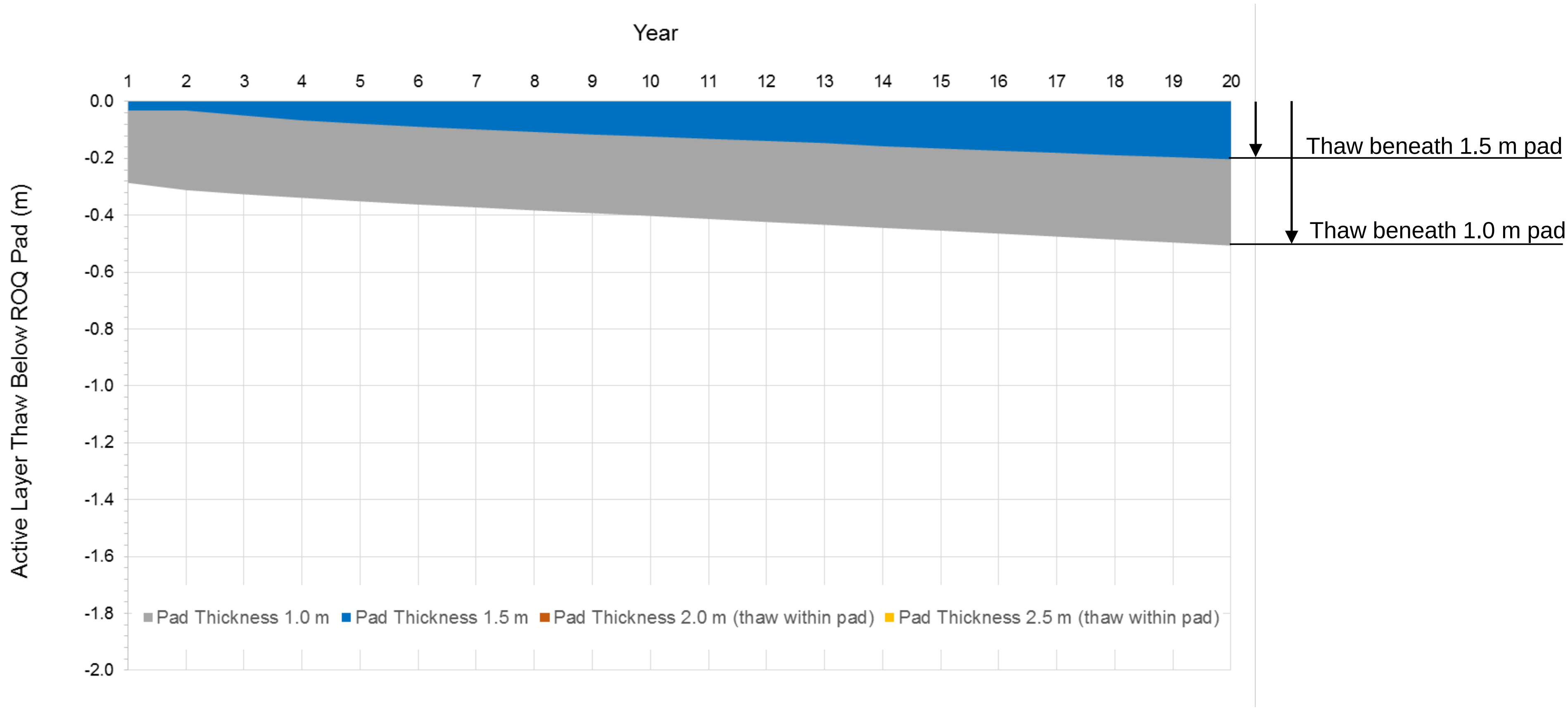
Date:	Approved:	Figure:
10/19/2017	cws	4



Notes:

1. ROQ pad active layer for model year 20
2. Surface n-factors, nf 0.86, nt 1.52, average literature values
3. Active layer depth referenced from base of ROQ Pad

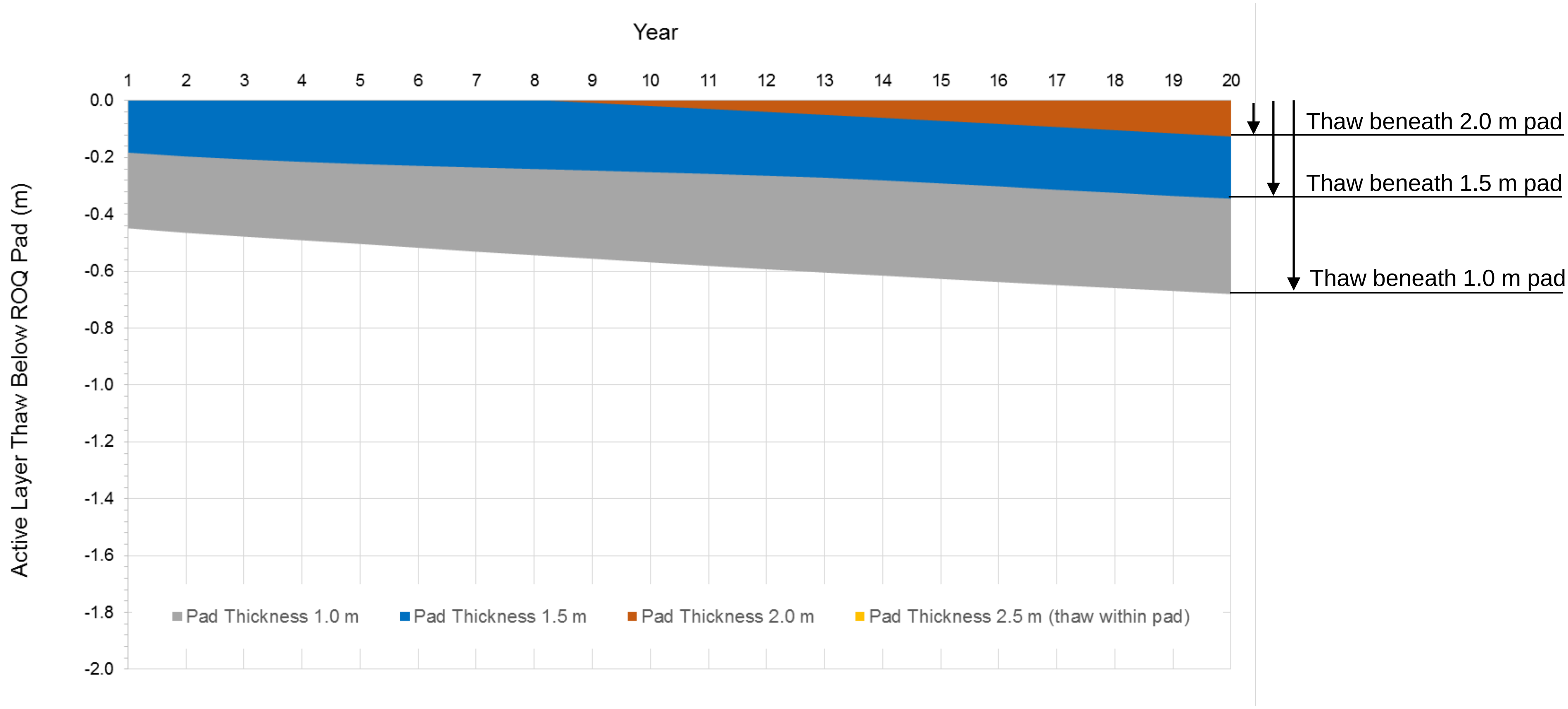
srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling
		ROQ Pad Active Layer – Model Year 20
Job No: 1CT022.013 Filename: ROQ_ActiveLayer_ModelYear20.pptx	HOPE BAY PROJECT	Date: 10/19/2017 / Approved: cws / Figure: 5



Notes:

1. Active layer thaw below base of ROQ pad
2. Thaw based on 0°C isotherm for model year 1 to 20
3. Surface n-factors, nf 0.86, nt 1.52, average literature values
4. Thaw is above the base of the pad for a ROQ pad >1.9 m thick

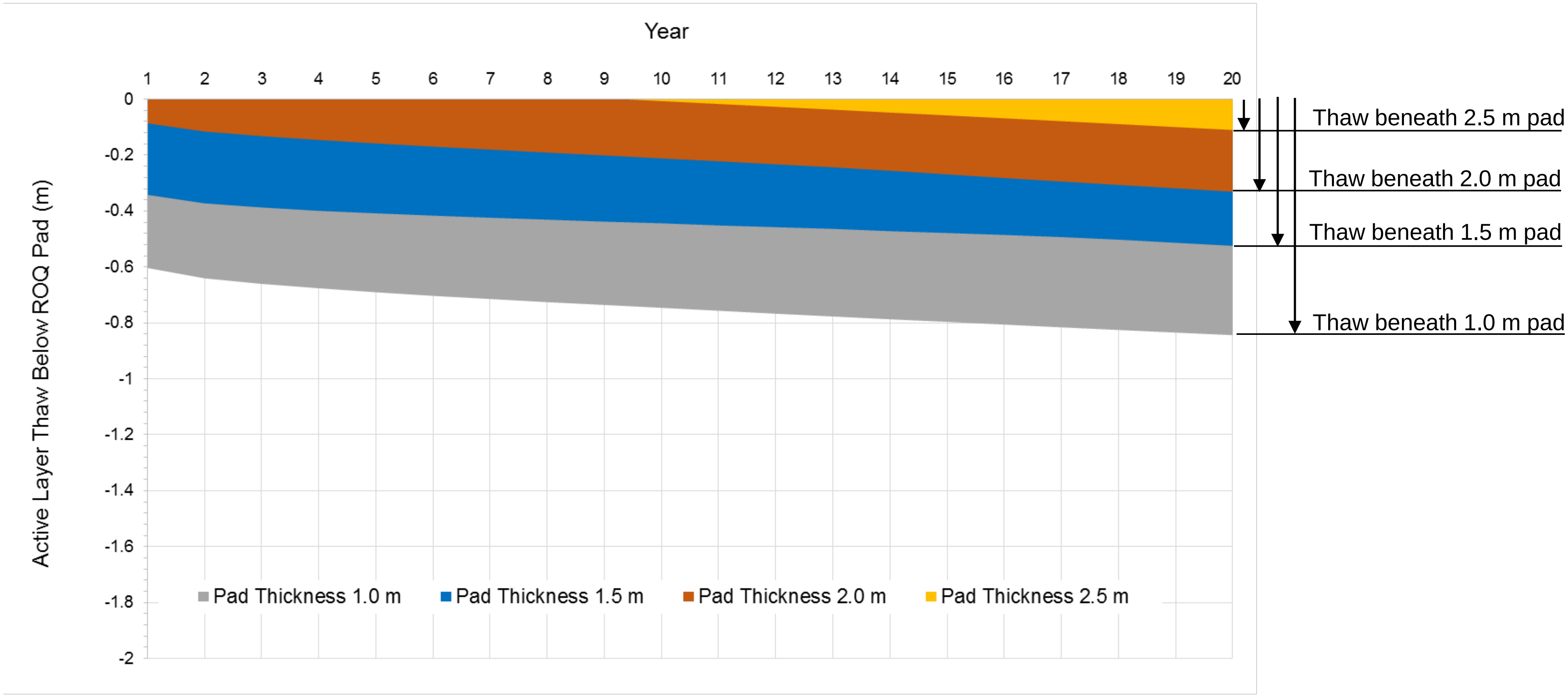
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling **ROQ Pad Active Layer, 0°C Isotherm – Model Year 1 to 20**
Job No: 1CT022.013 Filename: ROQ_ActiveLayer_ModelYear1to20.pptx	HOPE BAY PROJECT	Date: 10/19/2017 · Approved: cws · Figure: 6



Notes:

1. Active layer thaw below base of ROQ pad
2. Thaw based on -1°C isotherm for model year 1 to 20
3. Surface n-factors, nf 0.86, nt 1.52, average literature values
4. Thaw is above the base of the pad for ROQ pad >2.2 m thick

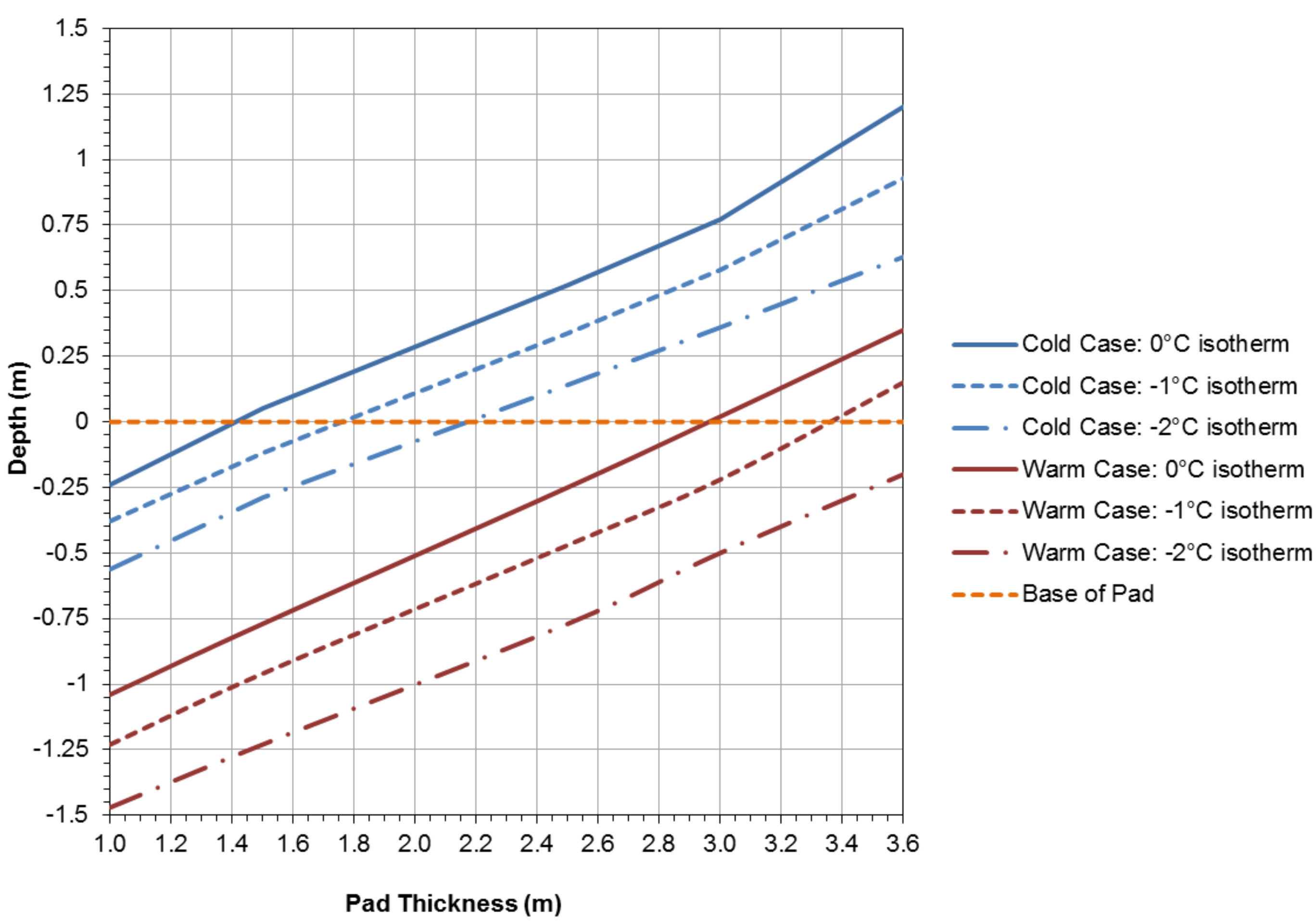
srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling
		ROQ Pad Active Layer, -1°C Isotherm – Model Year 1 to 20
Job No: 1CT022.013 Filename: ROQ_ActiveLayer_ModelYear1to20.pptx	HOPE BAY PROJECT	Date: 10/19/2017 | Approved: cws | Figure: 7



Notes:

1. Active layer thaw below base of ROQ pad
2. Thaw based on -2°C isotherm for model year 1 to 20
3. Surface n-factors, nf 0.86, nt 1.52, average literature values
4. Thaw is above the base of the pad for a ROQ pad >2.7 m thick

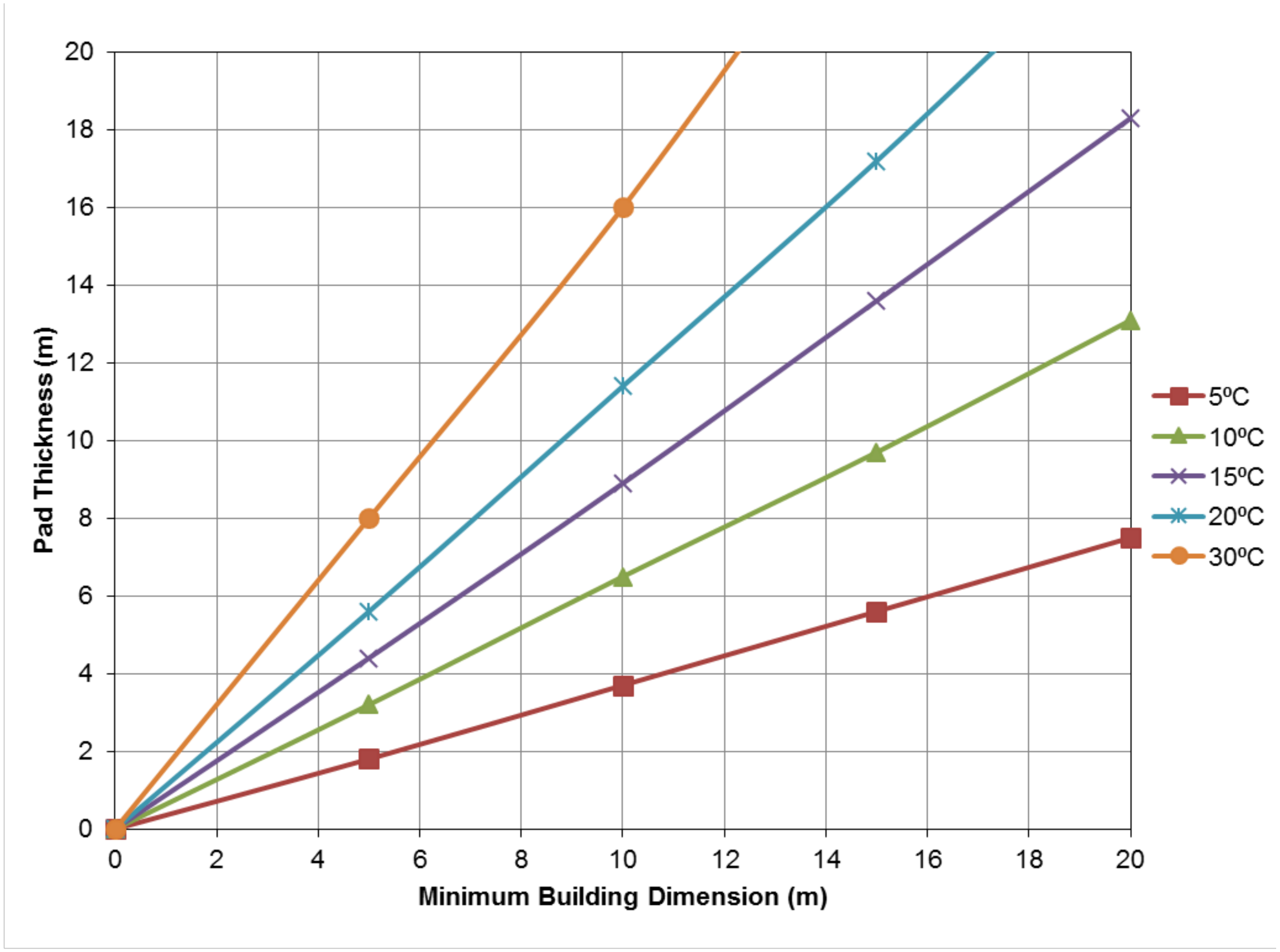
srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling
		ROQ Pad Active Layer, -2°C Isotherm – Model Year 1 to 20
Job No: 1CT022.013 Filename: ROQ_ActiveLayer_ModelYear1to20.pptx	HOPE BAY PROJECT	Date: 10/19/2017 · Approved: cws · Figure: 8



Notes:

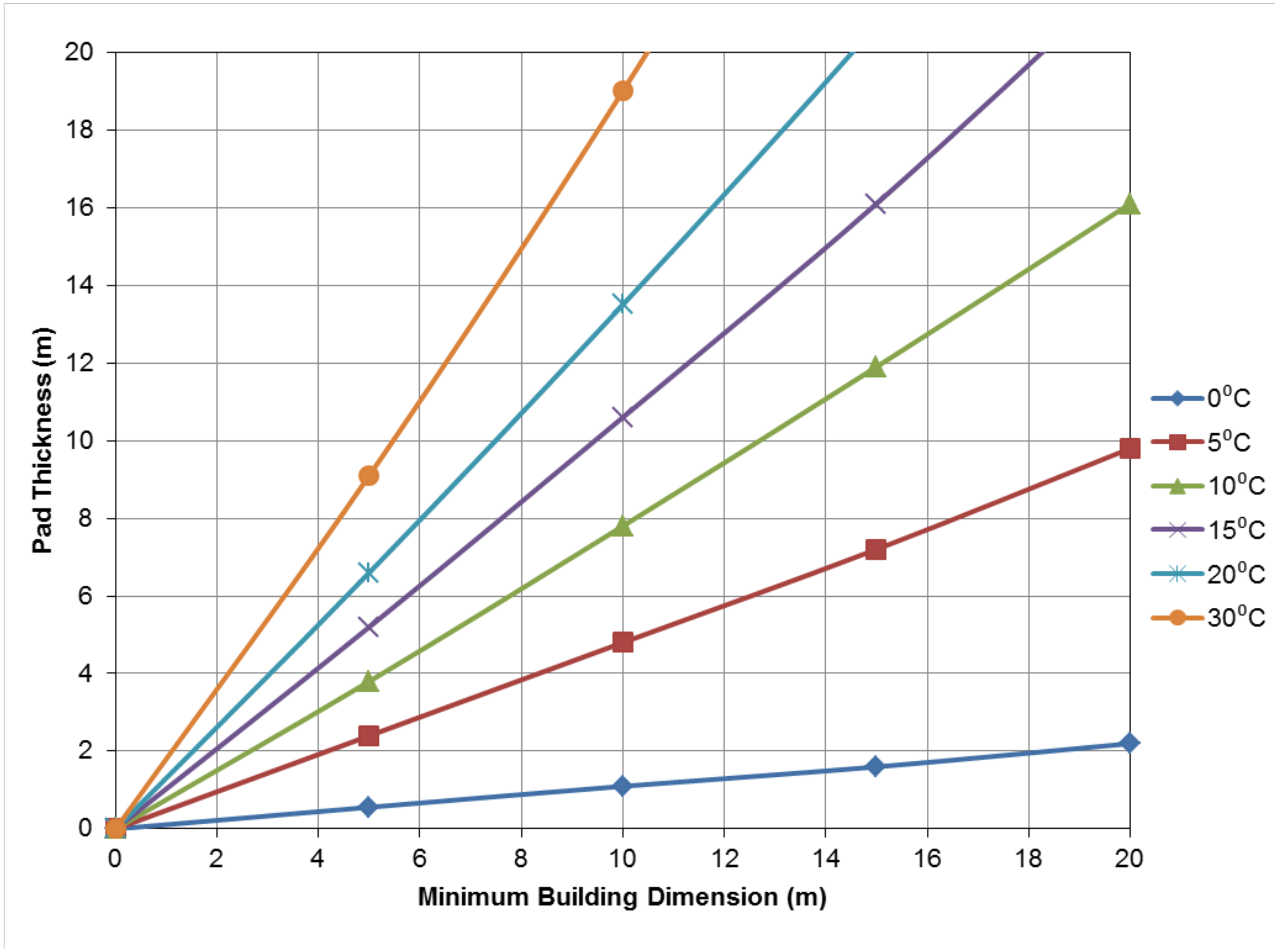
1. ROQ pad active layer for model year 20
2. Cold Case n-factors, nf 1.02, nt 1.25, literature values
3. Warm Case n-factors, nf 0.6, nt 2.01, literature values
4. Active layer depth referenced from base of ROQ Pad

srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling
		ROQ Pad Active Layer – Sensitivity to N-Factors
Job No: 1CT022.013 Filename: ROQ_ActiveLayer_ModelYear20.pptx	HOPE BAY PROJECT	Date: 10/19/2017 | Approved: cws | Figure: 9



Notes:
1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain 0°C isotherm within pad

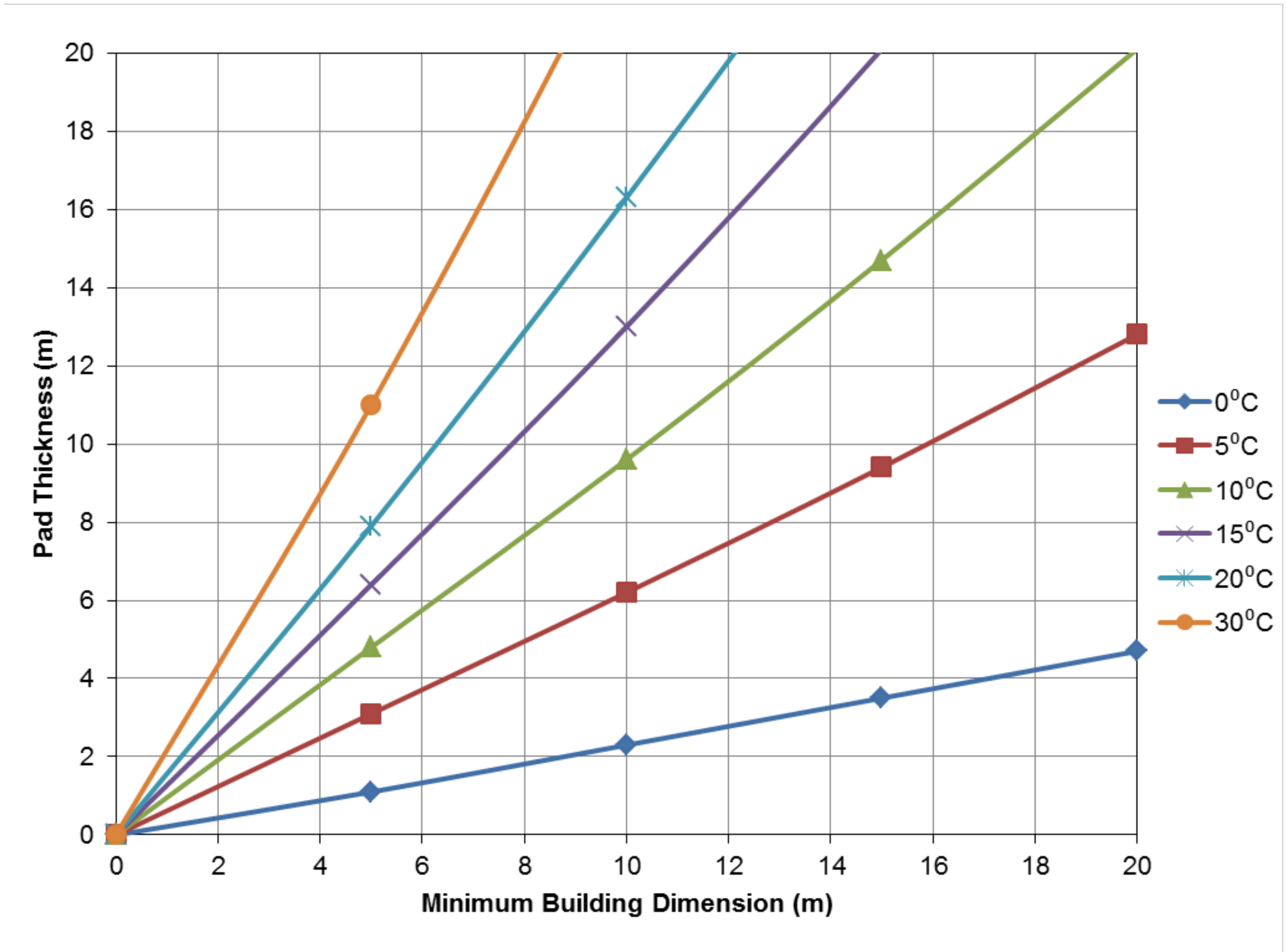
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building – Non-Insulated Pad (0°C Isotherm)
Job No: 1CT022.013 Filename: HeatedBuildingNonInsultatedPad_0C.pptx	HOPE BAY PROJECT	Date: 10/19/2017 | Approved: cws | Figure: **10**



Notes:

1. Heated building temperatures include 0°C, 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain -1°C isotherm within pad

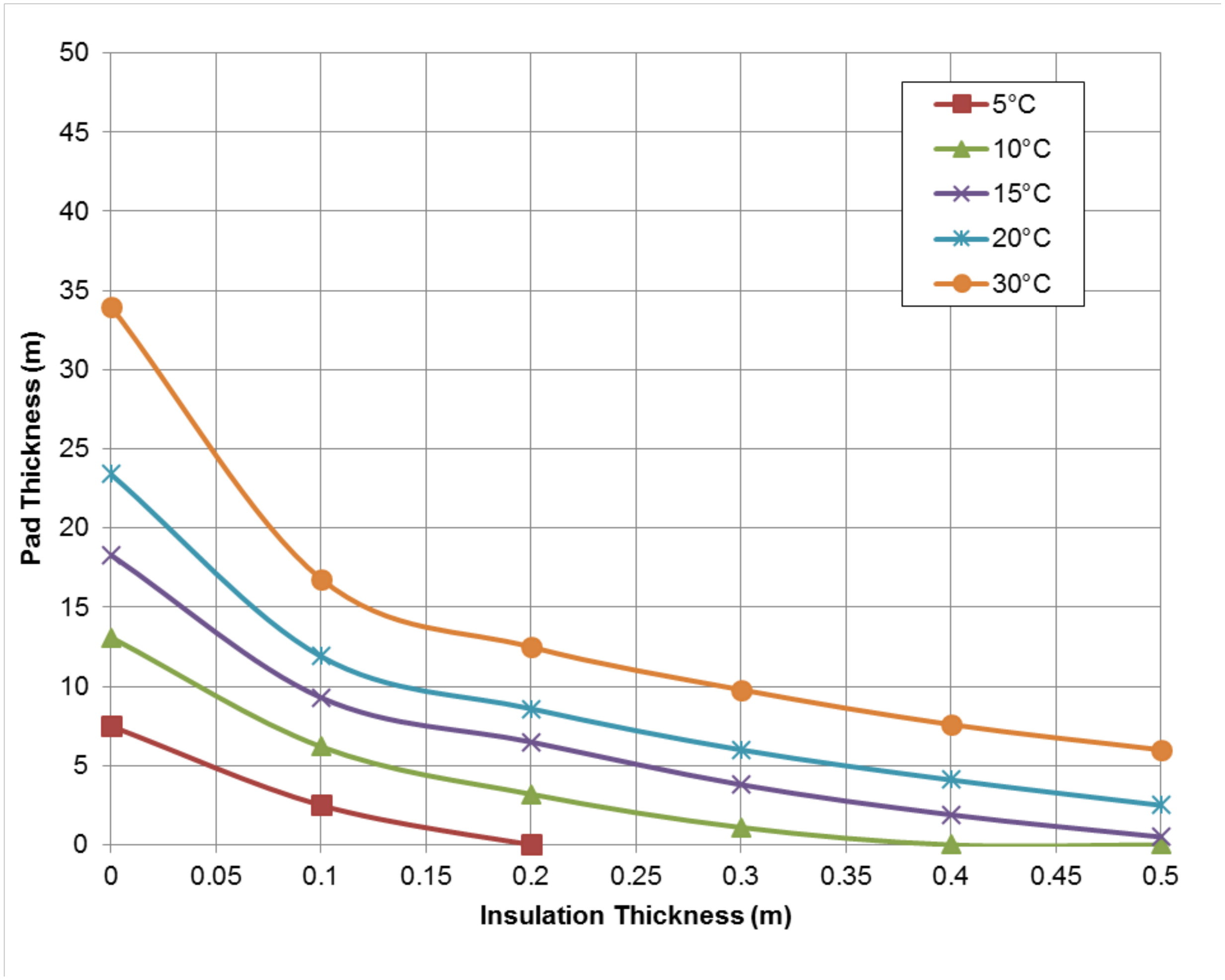
srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building – Non-Insulated Pad (-1°C Isotherm)
Job No: 1CT022.013 Filename: HeatedBuildingNonInsultatedPad_-1C.pptx	HOPE BAY PROJECT	Date: 10/19/2017 · Approved: cws · Figure: **11**



Notes:

1. Heated building temperatures include 0°C, 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum non-insulated pad thickness to maintain -2°C isotherm within pad

srk consulting	MAC RESOURCES	Infrastructure Thermal Modeling **Heated Building – Non-Insulated Pad (-2°C Isotherm)**
Job No: 1CT022.013 Filename: HeatedBuildingNonInsultatedPad_-2C.pptx	HOPE BAY PROJECT	Date: 10/19/2017 · Approved: cws · Figure: **12**



Notes:
1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain 0°C isotherm within pad

srk consulting

TMAC RESOURCES

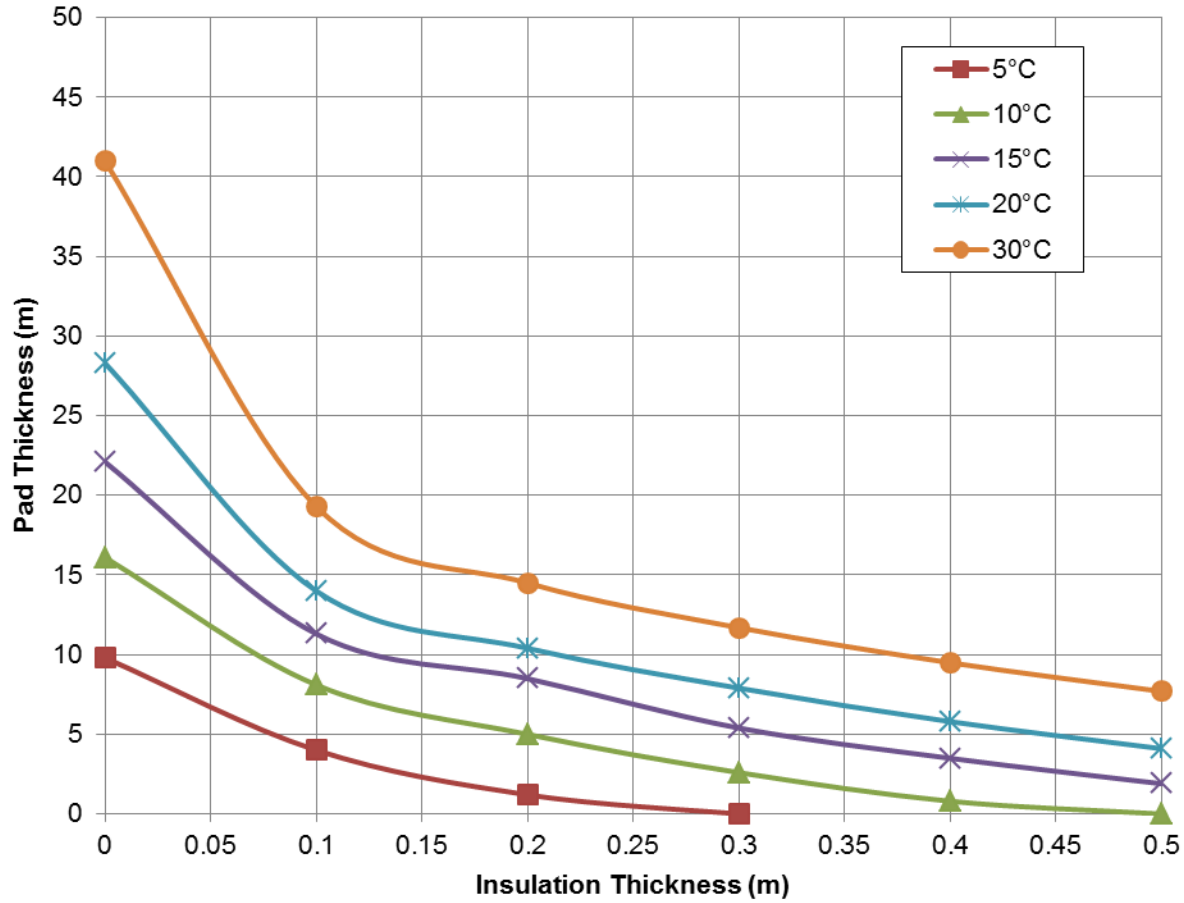
Infrastructure Thermal Modeling

Heated Building Pad Thickness – 0°C Isotherm

Job No: 1CT022.013
Filename: InsulatedPadThickness_0C.pptx

HOPE BAY PROJECT

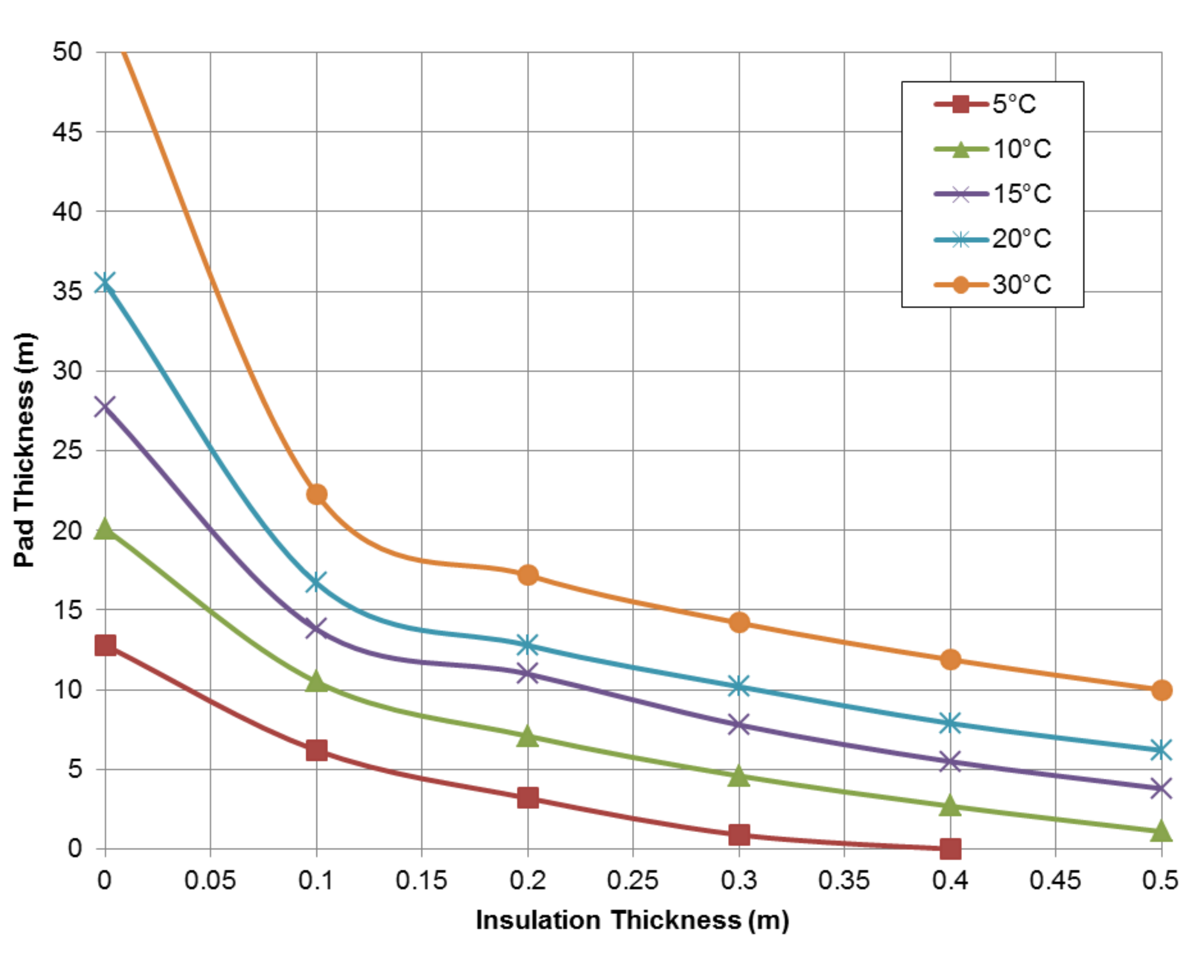
Date: 10/19/2017
Approved: cws
Figure: **13**



Notes:

1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain -1°C isotherm within pad

srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building Pad Thickness – -1°C Isotherm
Job No: 1CT022.013 Filename: InsulatedPadThickness_-1C.pptx	HOPE BAY PROJECT	Date: 10/19/2017 | Approved: cws | Figure: **14**



Notes:

1. Heated building temperatures include 5°C, 10°C, 15°C, 20°C, and 30°C
2. Minimum pad thickness and insulation to maintain -2°C isotherm within pad

srk consulting	TMAC RESOURCES	Infrastructure Thermal Modeling
		Heated Building Pad Thickness – -2°C Isotherm
Job No: 1CT022.013 Filename: InsulatedPadThickness_-2C.pptx	HOPE BAY PROJECT	Date: 10/19/2017 · Approved: cws · Figure: **15**

Appendix D – Adfreeze Pile Bond Strength

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Megan Miller, PEng	**Project No:**	1CT022.013
Reviewed By:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Adfreeze Pile Bond Strength		

Change Log

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

Changes by Section

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

1 Introduction

1.1 General

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

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

1.2 Objective

The objective of this memo is to provide a procedure to determine bond strength for adfreeze piles in permafrost foundations and adfreeze piles driven through engineered fill into permafrost foundations for the Project. The adfreeze pile's strengths developed with this method are only applicable to the Project.

While this memo is intended to provide adfreeze pile bond strengths to be used in design, these values should only be used when site specific data is not available.

2 Design Concept

2.1 Approach

Critical Project infrastructure should be founded on bedrock foundations or thermal pads which do not allow settlement. However, in some cases the use of piles may be required to meet the design objectives. In most cases these piles will be founded directly in permafrost, but in some case the piles will be driven through rockfill pads into permafrost.

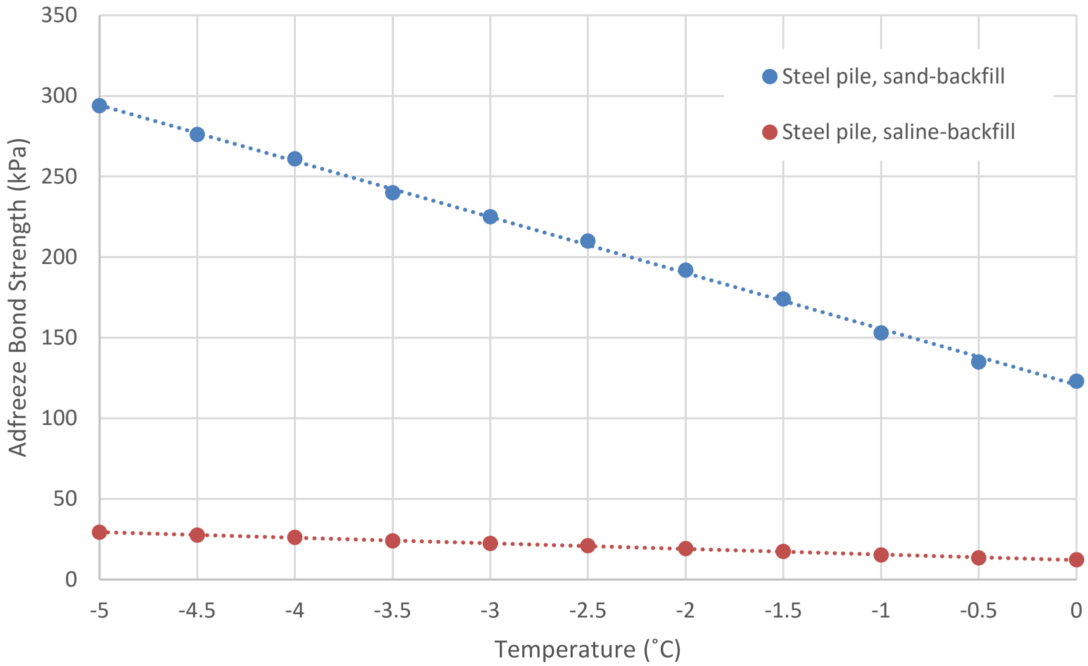
2.2 Foundation Conditions

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% (SRK 2017a). 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. The salinity of the marine silts and clays typically range from 37 to 47 parts per thousand which depresses the freezing point and contributes to higher unfrozen water content at below freezing temperatures.

2.3 Design Criteria

Based on measured and modelled ground temperatures (SRK 2017b) and literature adfreeze bond strengths (Weaver and Morgenstern 1981), SRK developed a series of graphs to estimate the strength of the adfreeze bond for steel piles backfilled with a non-saline sand slurry. The estimated adfreeze bond values presented are not valid if an overburden slurry is used for backfill as backfilling with saline permafrost cuttings greatly reduces adfreeze bond strength; a salinity of 15 ppt or greater reduces the bond strength by approximately 90% (Bigger and Sego 1993).

Figure 1 provides adfreeze bond strengths versus temperature for steel piles backfilled with a non-saline sand slurry, and a slurry of saline soil cuttings. The adfreeze bond strength is based on literature values provided in Weaver and Morgenstern (1981), and a saline soil reduction factor of 90% as described in Bigger and Sego (1993). These results show that under non-saline conditions, the maximum adfreeze bond strength at a temperature of −5°C is 294 kPa.



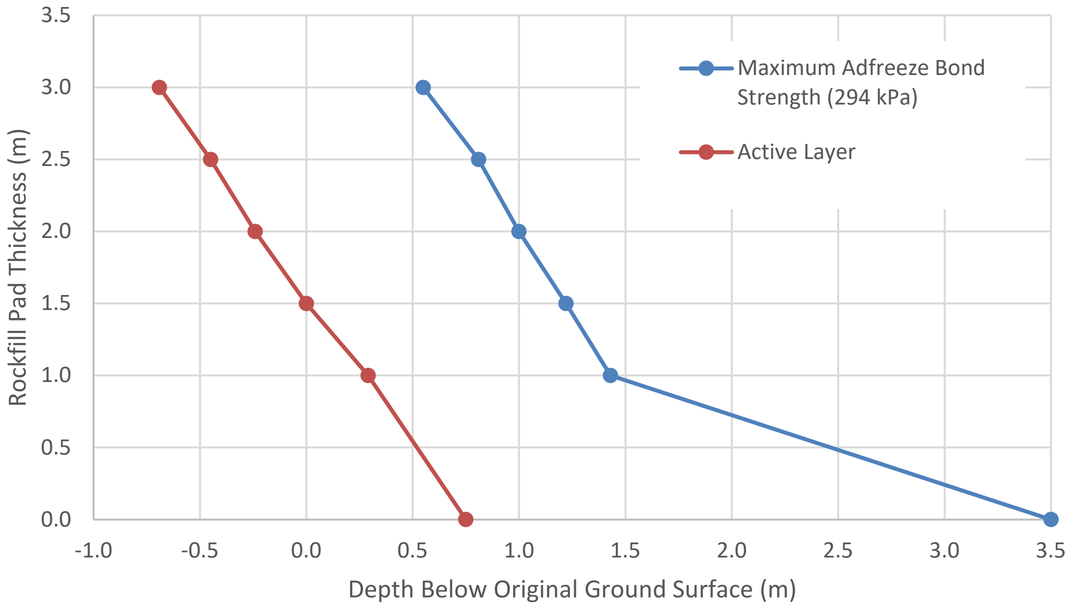
Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[AdfreezePile_Calculations_1CT022.004_rev01_mmm.xlsx]

Figure 1: Adfreeze Bond vs Temperature for Steel Pile Backfilled with Non- saline Sand Backfill, and Saline Backfill

The blue line in Figure 2 provides the depth below overburden surface where the maximum adfreeze bond strength of 294 kPa (at −5°C ground temperature) is first encountered, for various rockfill pad thicknesses. The adfreeze bond strength along the pile below this point remains at 294 kPa. Above the depth of maximum adfreeze pile bond strength (Figure 2, blue line) the adfreeze bond varies linearly from:

- The adfreeze bond strength at a temperature of 0°C (123 kPa, Figure 1), at the maximum depth of the active layer, to the maximum adfreeze bond strength at the depth shown in Figure 2 (for thin rockfill pads, or piles directly in overburden where there is an active layer within the overburden soils), or
- The bond strength associated with the maximum temperature at original ground surface (Figure 3) to the maximum bond strength (294 kPa) over the depth shown in Figure 2 (for thick pads where the overburden remains frozen all year round).

The adfreeze bond strength of 123 kPa at a temperature of 0°C is obtained from Figure 1, the maximum predicted depth of the active layer for the various fill thicknesses is provided in Figure 2 (red line). For thick pads where the overburden is expected to remain frozen year round (e.g., where the red line in Figure 2 has a negative depth below original ground surface) the adfreeze bond strength at the top of overburden can be obtained from Figure 3.



Source: \\srk.ad\dfs\na\van\Projects\01_SITES\Hope.Bay\1CT022.004_Phase 2 DEIS - Engineering Support\Task 210_Geotechnical_Overburden\[AdfreezePile_Calculations_1CT022.004_rev01_mmm.xlsx]

Figure 2: Active Layer and Maximum Adfreeze Bond Strength Depth for Various Thicknesses of Rockfill Pad

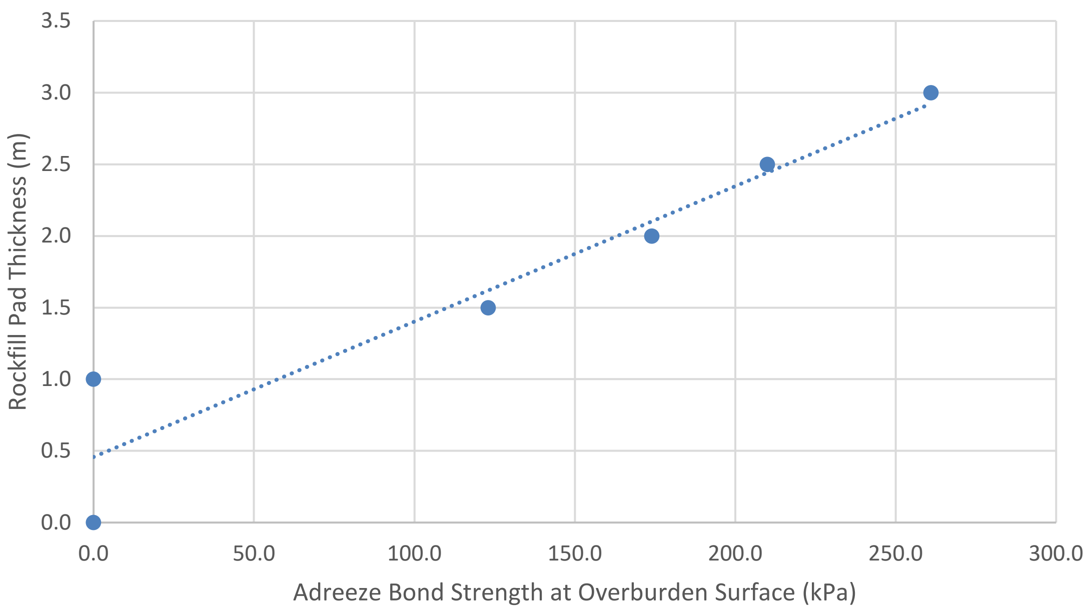


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Figure 3: Adfreeze Bond Strength at Overburden Surface

3 References

Bigger, K.W. and Sego, D.C. 1993. The Strength and Deformation Behavior of Model Adfreeze and Grouted Piles in Saline Frozen Soil. Canadian Geotechnical Journal. Volume 30. pp. 319-337.

SRK Consulting (Canada) Inc., 2017a. Geotechnical Design Parameters and Overburden Summary Report, Hope Bay Project. Report Prepared for TMAC Resources Inc. 1CT022.013. November 2017.

SRK Consulting (Canada) Inc., 2017b. Hope Bay Project: Thermal Modelling to Support Run-of-Quarry Pad Design. Memo Prepared for TMAC Resources Inc. 1CT022.013. November 2017.

Weaver, J.S., and Morgenstern, N.R. 1981. Pile Design in Permafrost. Canadian Geotechnical Journal. Volume 18. pp. 357-370.

Appendix E – Waste Rock Stability Analysis

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Memo

To:	John Roberts, PEng, Vice President Environment	**Client:**	TMAC Resources Inc.
From:	Megan Miller, PEng	**Project No:**	1CT022.013
Reviewed by:	Maritz Rykaart, PhD, PEng	**Date:**	November 30, 2017
Subject:	Hope Bay Project: Waste Rock Pile Stability Analysis		

Change Log

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

Changes by Section

Information Request, Technical Comment, or Other Change	Section	Comments
Other	Figures	Correct slope angle labels added

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.

Waste rock piles are planned at Madrid North (646,000 tonnes, 359,000 m^3), Madrid South (65,000 tonnes, 361,000 m^3), and Boston (628,000 tonnes, 349,000 m^3). These waste rock piles are however temporary, since all mine waste rock will be backhauled underground as structural mine backfill during the life of the Project.

1.2 Objectives

This memo presents the results of stability analysis completed for the most critical cross section (i.e. the highest) of the Madrid South waste rock pile to confirm its surficial and overall stability. The Madrid South waste rock pile was selected to be representative of the waste rock pile stability at all locations because it had the steepest foundation slope, and the pile height is only expected to be 3 m less than that of the Madrid North waste rock pile. To account for higher piles, or a future increase in pile height, a conceptual 100 m high pile located on the Madrid South pile location was also analyzed. The analysis results are used to define general design guidelines for all of the Project waste rock piles.

2 Stability Criteria

Federal or territorial (Nunavut) guidelines for waste rock pile designs do not exist. Therefore, the draft British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau, 1991), were used to suggest the design requirements of the waste rock piles. The suggested minimum design factors of safety (FOS) are presented in Table 1. The ranges in FOS for Cases A and B, reflect the different levels of confidence in understanding site conditions, material parameters, and consequences of instability.

The stability conditions mentioned in the Table 1 are described in detail in Section 3.4. Based on the current level of design and available information on foundation conditions, the Case A minimum FOS were used to assess the waste rock pile stability.

Table 1. British Columbia Mine Dump Factor of Safety Guidelines

Stability Condition	Suggested Minimum Design Values for Factor of Safety	
	Case A	Case B
Stability of Waste Rock Pile Surface		
Short-term (during construction) - (Stability Condition 1)	1.0	1.0
Long-term (reclamation – abandonment) – (Stability Condition 2)	1.2	1.1
Overall Waste Rock Pile Stability (Deep Seated Stability)		
Short-term (static) – (Stability Condition 3)	1.3 - 1.5	1.1 – 1.3
Long-term (static) – (Stability Condition 4)	1.5	1.3
Pseudo-static (Earthquake)[2]	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)		

Source: Piteau 1991

Notes:

1. A range of suggested minimum design values are given to reflect different levels of confidence in understanding site conditions, material parameters, consequences of instability, and other factors.
2. Where pseudo-static analyses, based on peak ground accelerations which have a 10% probability of exceedance in 50 years, yield FOS < 1.0, dynamic analysis of stress-strain response, and comparison of results with stress-strain characteristics of dump materials is recommended.

3 Slope Stability Assessment

3.1 Material Properties

3.1.1 Overburden Material Properties

Geotechnical investigations have not been performed within the proposed footprints of the waste rock piles. However, numerous geotechnical investigations have been performed on site which provide a general understanding of the foundation conditions to be expected under the waste rock piles.

The general overburden profile consists of a thin veneer of hummocky organic soil covered by tundra heath vegetation. Under this organic layer is a layer of marine silts and clays (i.e. silty clay and clayey silt) typically between 5 and 20 m thick. The bedrock contact zone consists of a relatively thin rubble zone of weathered blocky host rock (SRK, 2016a).

The waste rock piles will be constructed on a pad of run-of-quarry (ROQ) material overlaying the permafrost soils. The slope stability models were set up using the geotechnical properties

(SRK, 2016a) for marine silts and clays as the foundation soils. These material properties are summarized in Table 2.

The depth of the marine silts and clay layer under then Madrid South waste rock pile was estimated based on nearby geotechnical drill holes.

3.1.2 Waste Rock Pile Properties

The physical properties of the waste rock material for the Project have not been measured, but the physical properties used in the stability analyses are based on a comparison with Project's ROQ borrow material as reported in the literature and SRK's internal database. These properties can be seen in Table 2.

Table 2: Material Properties

Parameter		Marine Silt and Clay	Waste Rock Pile
Moist Unit Weight (kN/m^3)		17	20
Unfrozen	Apparent Cohesion c' (kPa)	0	0
Unfrozen	Friction Angle, φ ^{0}	30	40
Unfrozen	Undrain Shear Strength, Su	13	-
Frozen	Apparent Cohesion c' (kPa)	112	5
Frozen	Friction Angle, φ ^{0}	26	40

Source: SRK 2016a

A critical cross-section through the Madrid South waste rock pile, based on ultimate waste rock pile height, was selected to create the model used to run the analysis (Figure 1). A second model was created simply by increasing the height of the Madrid South critical cross-section to 100 m while keeping the slope and foundation conditions identical (Figure 2).

3.2 Seismic Coefficient

The British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau, 1991) recommends that a seismic event with a 10% probability of exceedance in 50 years (1:500 year) should be used to assess the waste rock piles (Table 1).

Horizontal seismic coefficient for the waste rock pile analysis were developed using the Limit Equilibrium Pseudo Static Stability Analysis method (SRK, 2016b). These seismic coefficients were developed specific to the waste rock pile geometry and recommended probability of exceedance, and are not applicable to other structures. The vertical seismic coefficients are assumed to be negligible. Table 3 provides the seismic coefficients used in the slope stability analysis.

Table 3: Seismic Coefficients used in the Waste Rock Stability Analysis

Waste Rock Pile Height (m)	Design Stage	Seismic Coefficient (g)
≤ 5	First bench	0.0086
20	Madrid South ultimate planned height	0.0075
100	Largest pile (theoretical case)	0.0067

(1) Source: SRK 2016b

3.3 Model Setup

The slope stability models were set up in SLOPE/W, a limit equilibrium slope stability analysis software tool developed by GEO-SLOPE International Ltd (Geoslope, 2012). The software is commonly used to compute the FOS of earth and rock slopes.

For the stability analyses, the waste rock piles are conservatively assumed to be unfrozen. This is conservative since freeze back may occur in both the foundation pad and the waste rock pile over the life of the structure. The thickness of the thawed foundation layer at the toe of the waste rock pile is assumed to be 1 m which is in line with the results of thermal analysis (SRK, 2016a).

3.4 Methodology

The stability, of the waste rock pile also took into consideration haul truck wheel loads applied near the crest of the waste rock pile. A loaded Sandvik TH540 was assumed to be the heaviest vehicle driving on the waste rock pile. The wheel loading calculation for the TH540 haul truck is included as Attachment 1. The minimum safe distance of the truck from the crest of the waste rock pile was determined to be 5.5 m satisfying the minimum recommended FOS.

The following two scenarios were considered and analyzed:

- Madrid South waste rock pile at the maximum planned design height (19 m); and
- Madrid South waste rock pile, assuming a theoretical maximum height of 100 m.

The slope stability of the waste rock piles were evaluated under five stability conditions (Table 1):

- Short-term (surficial/static) (Stability Condition 1): This stability case considers the stability of the waste rock pile surface with the truck loading applied at 5.5 m away from the crest of the waste rock pile.
- Long-term (surficial/static) (Stability Condition 2): This stability case considers the stability of the waste rock pile surface without the haul truck loads near the crest.
- Short-term (overall/static) (Stability Condition 3): This stability case considers the stability of the overall waste rock pile with the truck loading applied near the crest of the waste rock pile, and is only analyzed for deep seated stability by forcing the slip-surface to a particular path or certain depth.
- Long-term (overall/static) (Stability Condition 4): This stability case considers the stability of the overall waste rock pile without the truck loading applied, only is only analyzed for deep seated stability by forcing the slip-surface to a particular path or certain depth.
- Earthquake (overall/pseudo-static) (Stability Condition 5): This stability case considers the stability of the overall waste rock pile with the truck loading applied near the crest of the waste rock pile under seismic load.

The slope stability analyses were carried out using the Morgenstern-Price Method and were assessed for both static and pseudo-static conditions. To provide confidence in the results, the models were analyzed using three modes of searching for the failure surface:

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

The waste rock piles were assumed to be unsaturated, so pore water pressure conditions were not applied in the analyses.

4 Results

The lowest calculated FOS for the analyzed critical section of the Madrid South waste rock pile is presented in Table 4 while the results for all of the analyses are provided in Attachment 2. Although there is a good understanding of site conditions, material parameters and consequences of instability, which suggests the waste rock piles should meet the FOS required for Case B, as the calculated FOS was found to exceed those listed for Case A as well.

Table 4. Madrid South (Maximum Height) Waste Rock Pile Stability Analysis Results

Stability Analysis	Loading Condition[1]	Recommended FOS (Case A)	Calculated FOS
Short-term (Surficial Stability) (Stability Condition 1)	Undrained	1.0	1.1
Long-term (Surficial Stability) (Stability Condition 2)	Drained	1.2	1.3
Short-term (Overall Stability) (Stability Condition 3)	Undrained	1.3-1.5	1.8
Long-term (Overall Stability) (Stability Condition 4)	Drained	1.5	2.0
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	1.8

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

In order to confirm the stability of the Madrid South waste rock pile during construction, the stability of the pile at the end of the construction of the first bench was also analyzed. The main objective for the completion of this analysis was to check the stability of the pile through foundation failure assuming thawed foundation conditions. The results of this analysis are summarized in Table 5.

Table 5. Madrid South (1st Bench) Waste Rock Pile Stability Analysis Results

Stability Analysis	Loading Condition	Recommended FOS (Case A)	Calculated FOS
Short-term (Surficial Stability) (Stability Condition 1)	Undrained	1.0	1.1
Long-term (Surficial Stability) (Stability Condition 2)	Drained	1.2	1.3
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	1.1

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

The stability analysis results for a waste rock pile with a height of 100 m is summarized in Table 6. The surficial stability was not analyzed for this model, since the slope of this analyzed section is identical to the model which was analyzed for Madrid south waste rock pile.

Table 6. Stability Analysis Results for a Waste Rock Pile With a Height of 100 m

Stability Analysis	Loading Condition	Recommended FOS (Case A)	Calculated FOS
Short-term (Overall Stability) (Stability Condition 3)	Undrained	1.3-1.5	2.1
Long-term (Overall Stability) (Stability Condition 4)	Drained	1.5	2.1
Earthquake (Overall Stability) (Stability Condition 5)	Undrained	1.1-1.3	2.1

Note:

(1) Loading conditions refers to the overburden foundations, in all cases the waste rock is assumed to be drained.

5 Discussion

As shown in Table 4 to Table 6, the FOS computed by the models exceed the minimum FOS recommended by British Columbia Guidelines for Mined Rock and Overburden Piles (Piteau 1991) in all cases analysed. Therefore, the waste rock piles as design are expected to be stable under static, and pseudo-static conditions, provided the design haul truck remains 5.5 m away from the crest of the pile. It is assumed the haul tucks unload the waste rock at least 5.5 m from the crest and a bulldozer will push the material to the crest.

The waste rock pile geometry, and foundation conditions modeled in the analysis are consistent with the conditions expected under all waste rock piles planned site; therefore, all waste rock piles are expected to be stable.

6 References

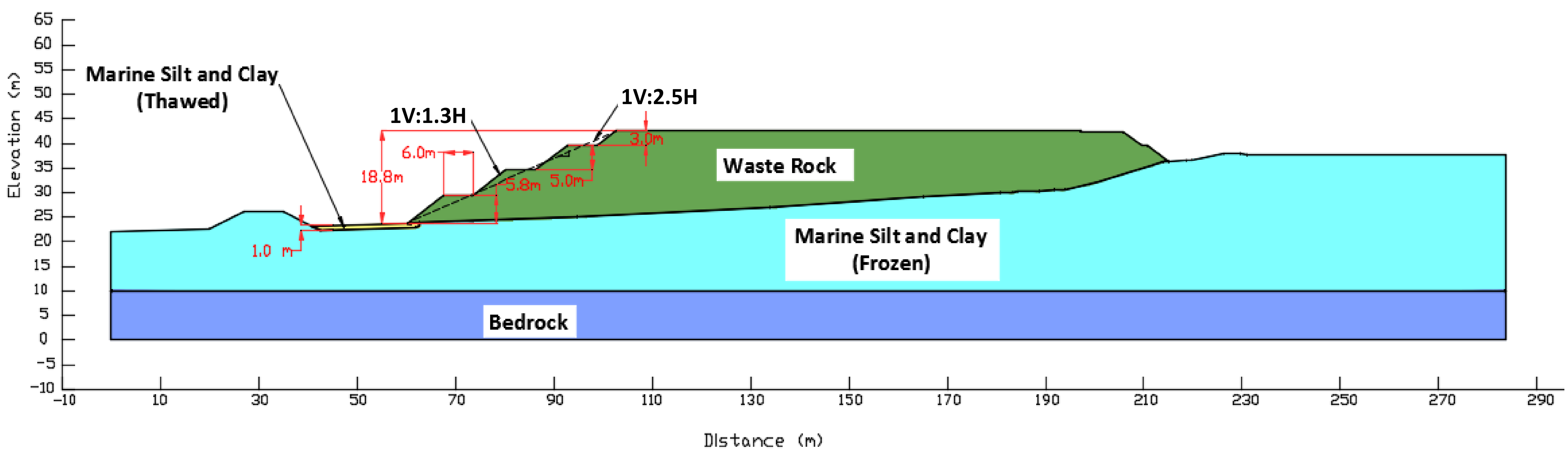
GEO-SLOPE International, Ltd. 2012. GeoStudio. 2012 (Version 8.12.2.7901). Calgary, Alberta.

[Piteau 1991] Piteau Associates Engineering Ltd. for British Columbia Mine Waste Rock Pile Research Committee. 1991. Mined Rock and Overburden Piles Investigation and Design Manual. Interim Guidelines. Prepared for the British Columbia Mine Dump Committee. May 1991.

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

SRK Consulting (Canada) Inc. 2016b. Horizontal Seismic Parameters for Pseudo-Static Modeling Memo prepared for TMAC Resources Inc. Project No.:1CT022.004. 2016.

Figures



LEGEND

- Waste Rock
- Marine Silt and Clay (Frozen)
- Marine Silt and Clay (Thawed, Undrained)
- Marine Silt and Clay (Thawed, Drained)
- Bedrock

Note: Only undrained loading condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same boundary.

srk consulting

Job No: 1CT22.004

Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa

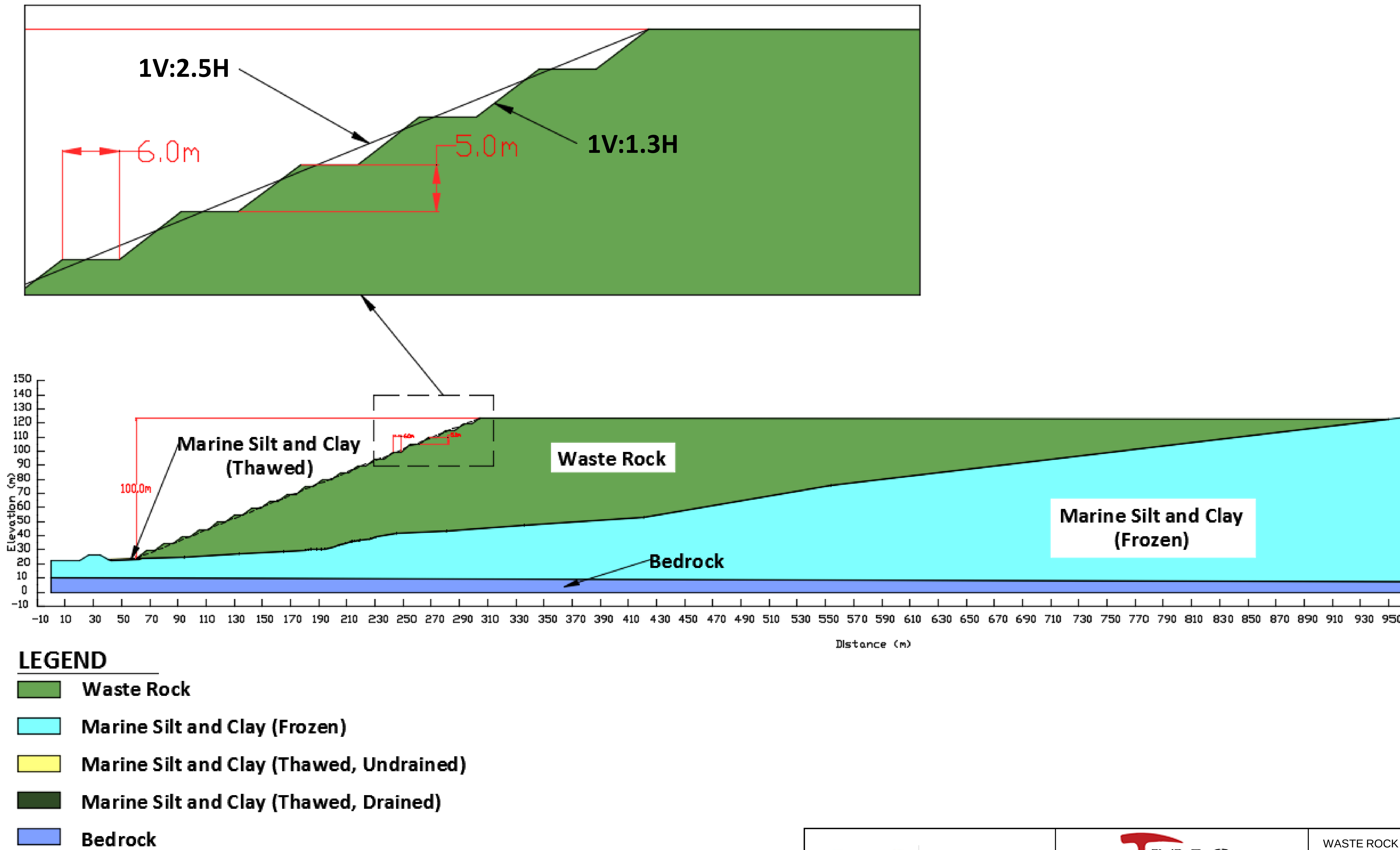
TMAC RESOURCES

HOPE BAY PROJECT

WASTE ROCK PILE STABILITY ANALYSIS

Madrid South Waste Rock Pile Analyzed Cross Section

Date:	Approved:	Figure:
April 2016	SA	1



Note: Only undrained condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same region.

srk consulting	MAC RESOURCES	WASTE ROCK PILE STABILITY ANALYSIS
Job No: 1CT22.004 Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa	HOPE BAY PROJECT	**Waste Rock Pile With a Height of 100 Analyzed Section** Date: April 2016 · Approved: SA · Figure: 2

Attachment 1: Truck Loading Calculations

Wheel load approximation for the Sandvik TH540			Reference
Operating Weight	34700	kg	(1)
Payload Capacity	40000	kg	(1)
Gross operating weight	74700	kg	(1)
Max operating weight	82700	kg	(2)
Loaded front axel weight	37200	kg	
% of gross operating weight	49.8%		
Loaded front rear weight	37500	kg	
% of gross operating weight	50.2%		
Front axel maximum weight	41184	kg	
Rear axel weight	41516	kg	
Load on each front tire	202.0072	kN	
Load on each rear tire	203.6363	kN	
Tire static loaded width	743	mm	(3)
Static loaded radius	784	mm	(3)
Assumed Contact length	743	mm	
Contact Area of one tire	0.552049	m2	
Ground pressure applied by each rear tire	368.87	kPa	

(1) Details on the Sandvik TH540 can be found in the Technical specs online

http://www.miningandconstruction.sandvik.com/sandvik/5100/SAM/Internet/cl01023.nsf/Alldocs/Products*5CLoad*and*haul*machines*5CUnderground*trucks*2ASandvik*40/$FILE/Sandvik%20TH540%20techspec.pdf

(2) 10-10-20 Payload Policy documents

Weight Calculation extracted from the Caterpillar 10-10-20 Payload Policy documents applied to the Sandvik Specs

Empty Chassis Weight (ECW) + Body and Liner = Empty Machine Weight (EMW) + Debris Fuel Attachments = Empty Operating Weight (EOW)

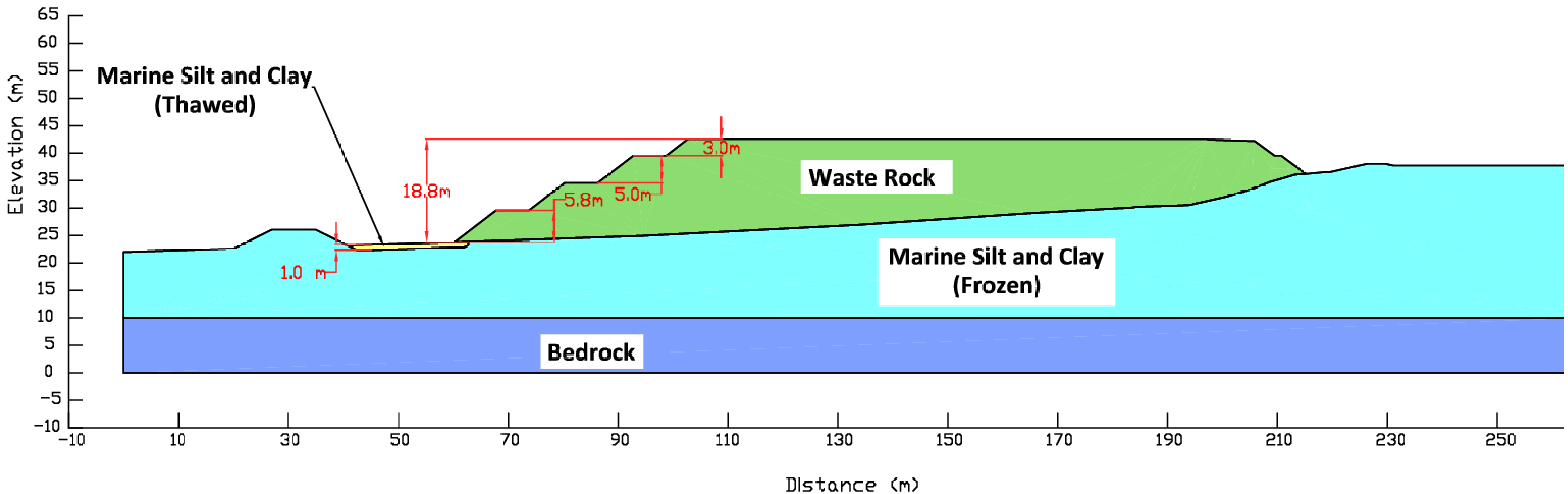
Target Gross Machine Weight (TGMW) -Empty Operating Weight (EOW) = Target Payload (TP)

Target Payload (TP) x 1.2 + Empty Operating Weight (EOW) < Maximum Gross Machine Weight (MGMW)

(3) Bridgestone tire specifications VLTS (26.5R25)

http://www.bridgestone.com/products/speciality_tires/off_the_road/products/pdf/brochure_earth_010.pdf

Attachment 2: Waste Rock Pile Stability Analysis

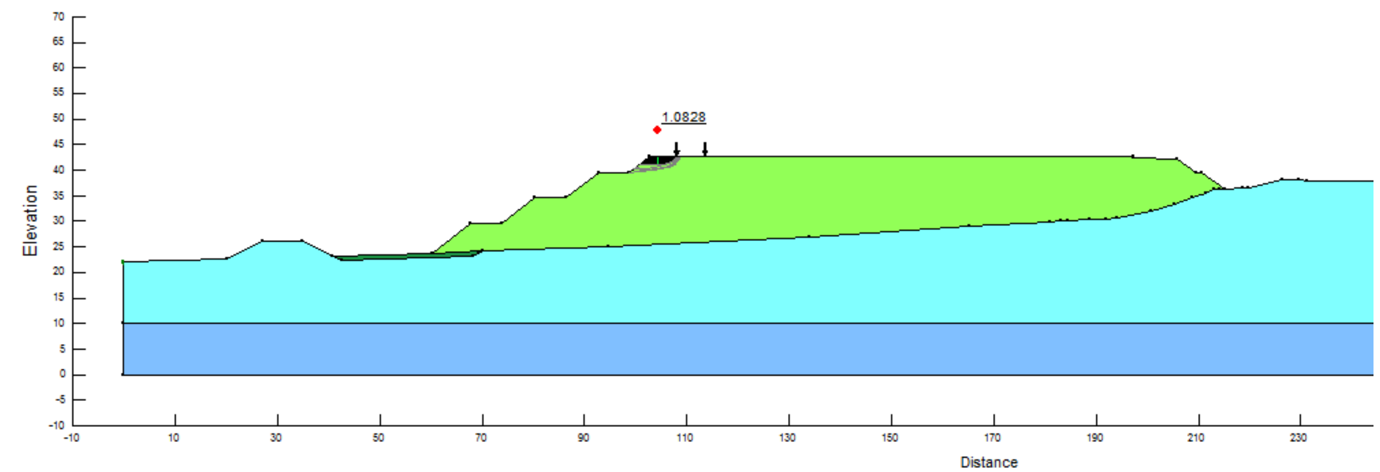
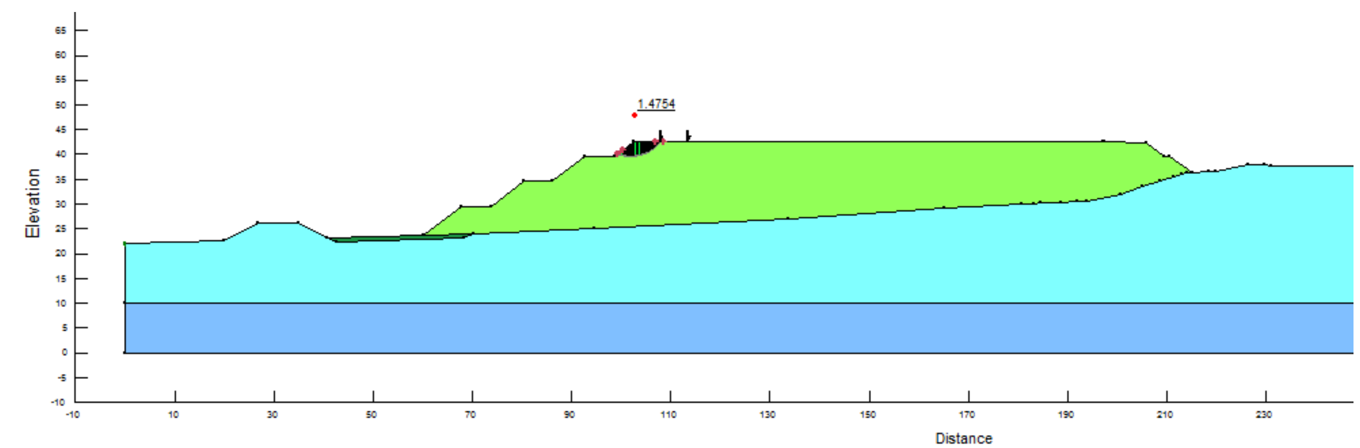
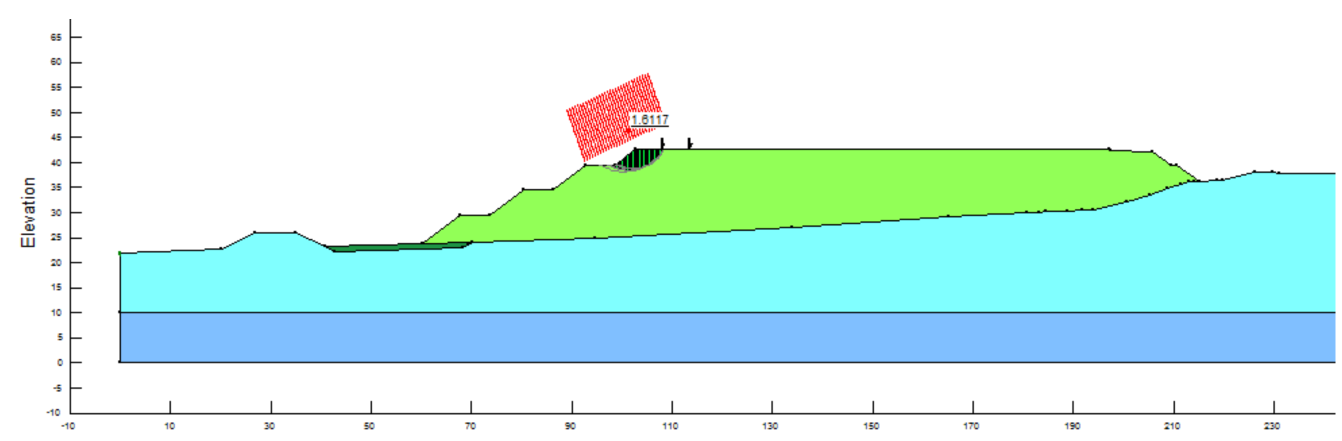


LEGEND

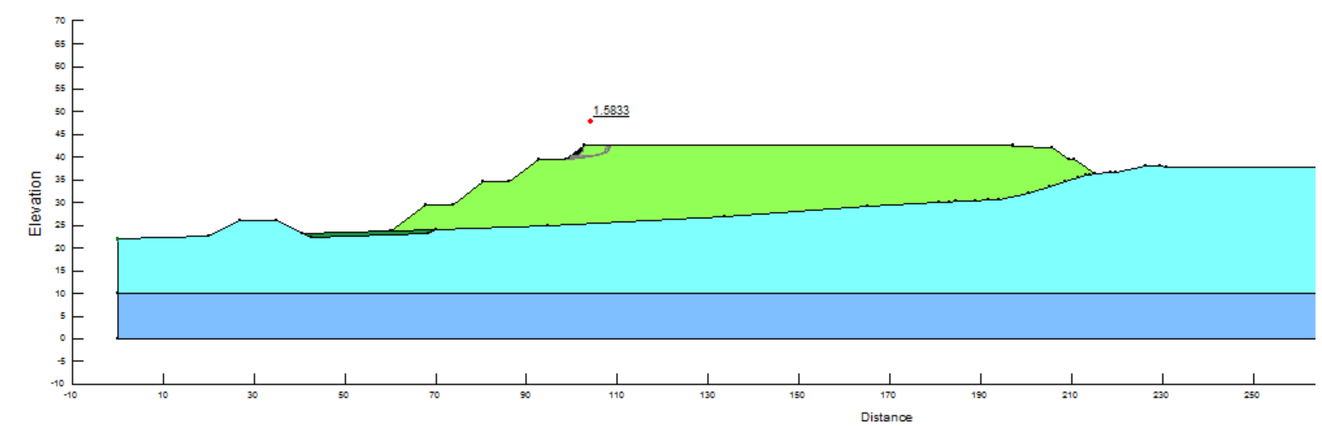
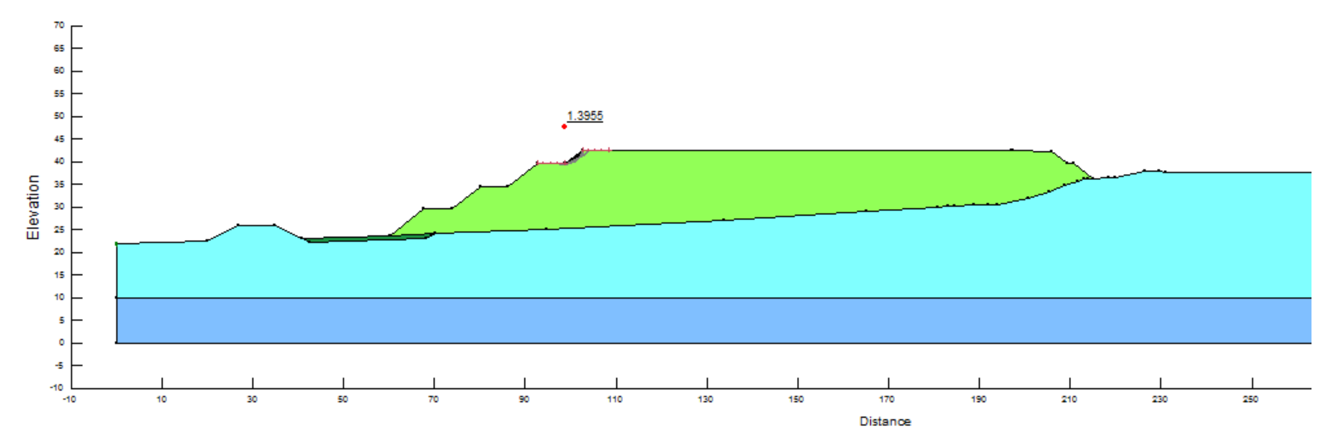
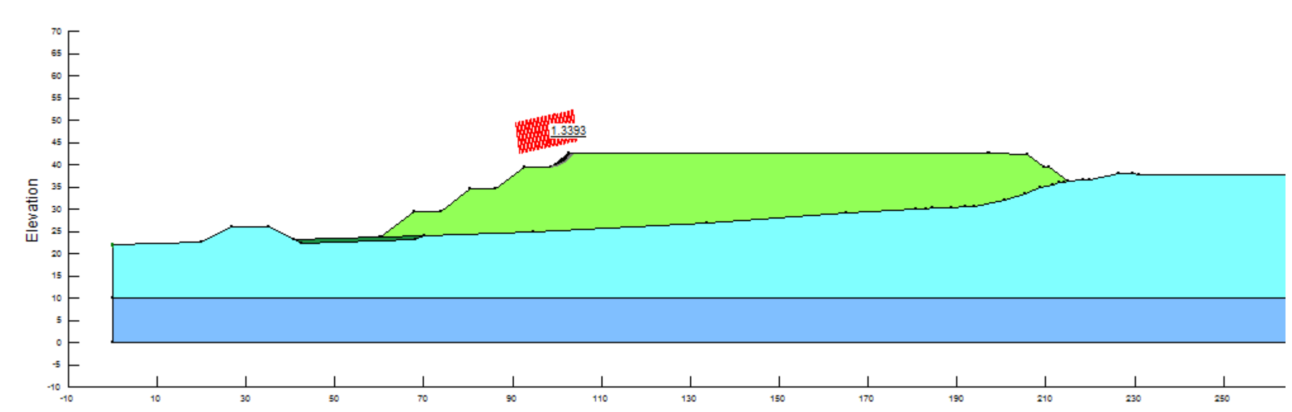
Note: Only undrained loading condition is shown in this figure. The drained analysis were completed by simply changing the material properties for the same boundary.

srk consulting	TMAC RESOURCES	WASTE ROCK PILE STABILITY ANALYSIS
Job No: 1CT22.004 Filename: HopeBay_MadridSouth_WRD_SlopeStability_160419_sa	HOPE BAY PROJECT	**Madrid South Waste Rock Pile Analyzed Cross Section**
		Date: April 2016 / Approved: / Figure: 1

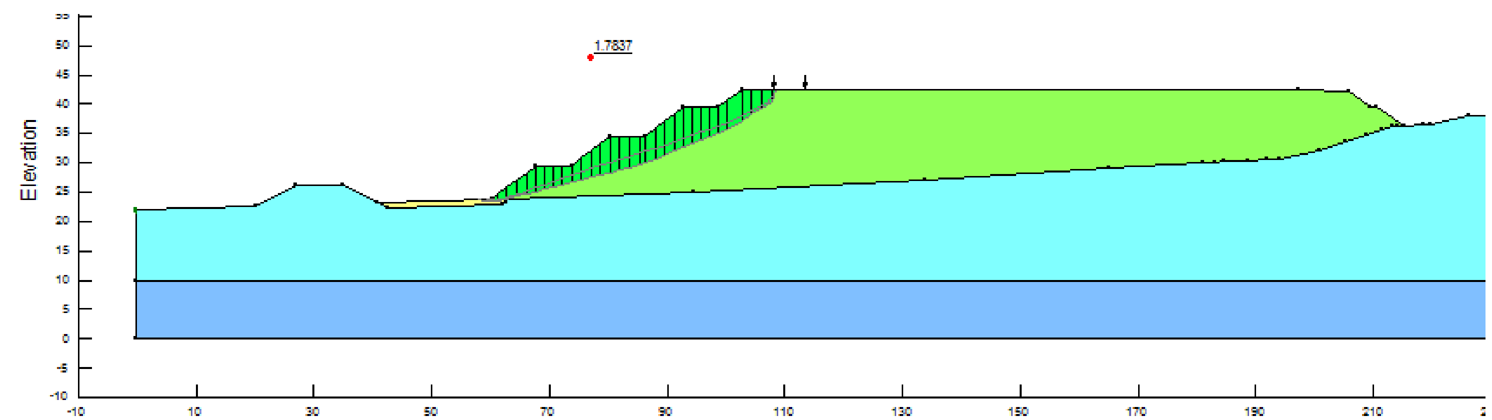
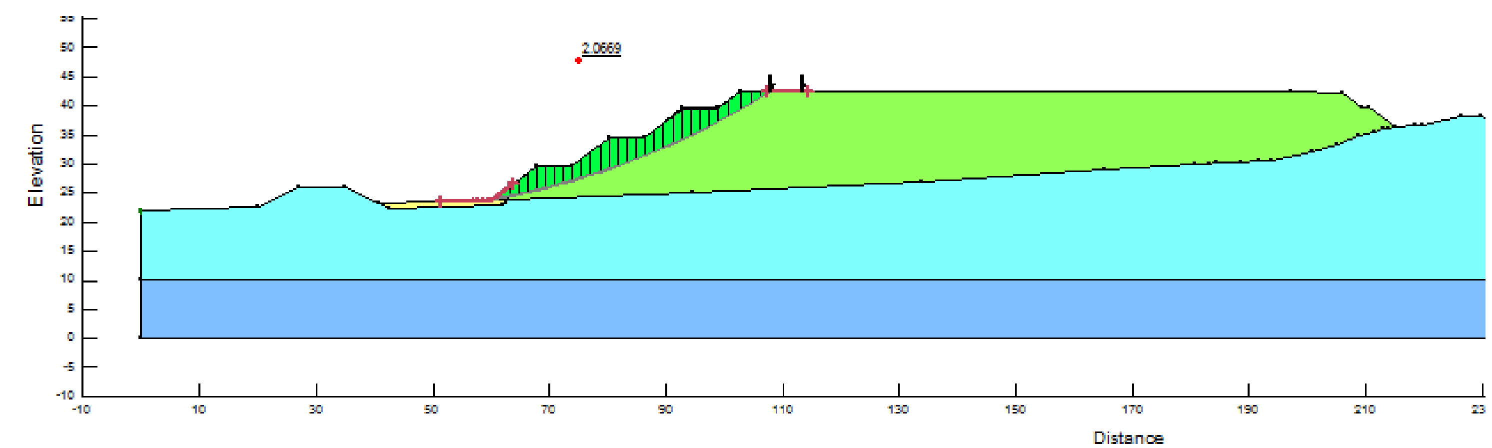
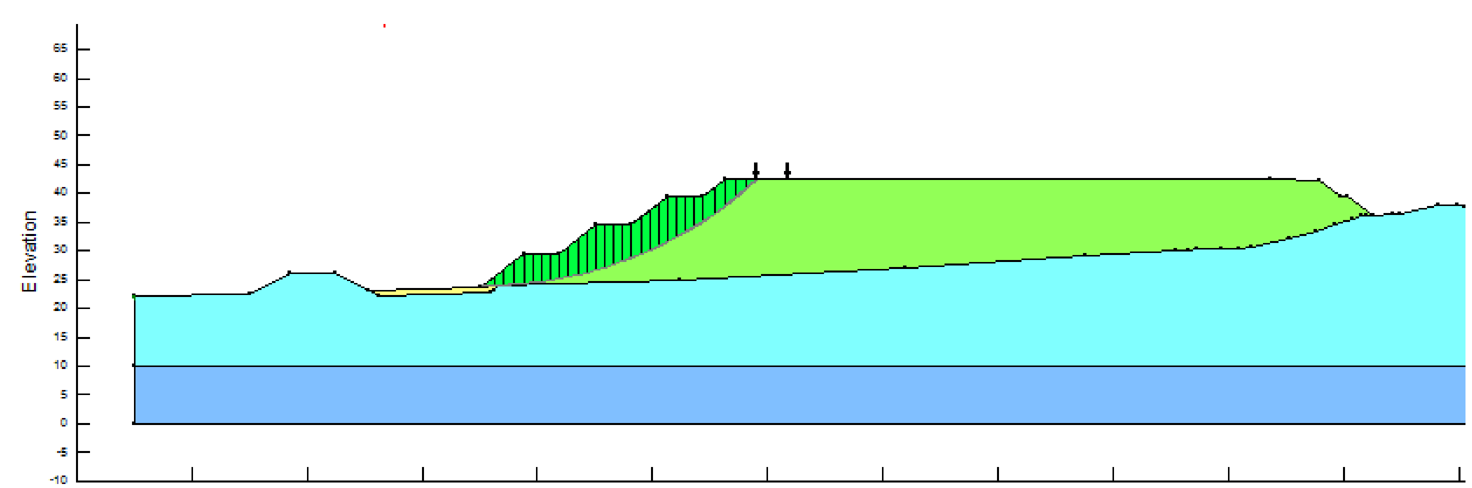
Madrid South Waste Rock Pile Stability Analysis Results – Stability of Dump Surface

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Short Term (Undrained Static Condition)	1.1	Fully Defined	
	1.5	Entry Exit	
	1.6	Grid and Radius	

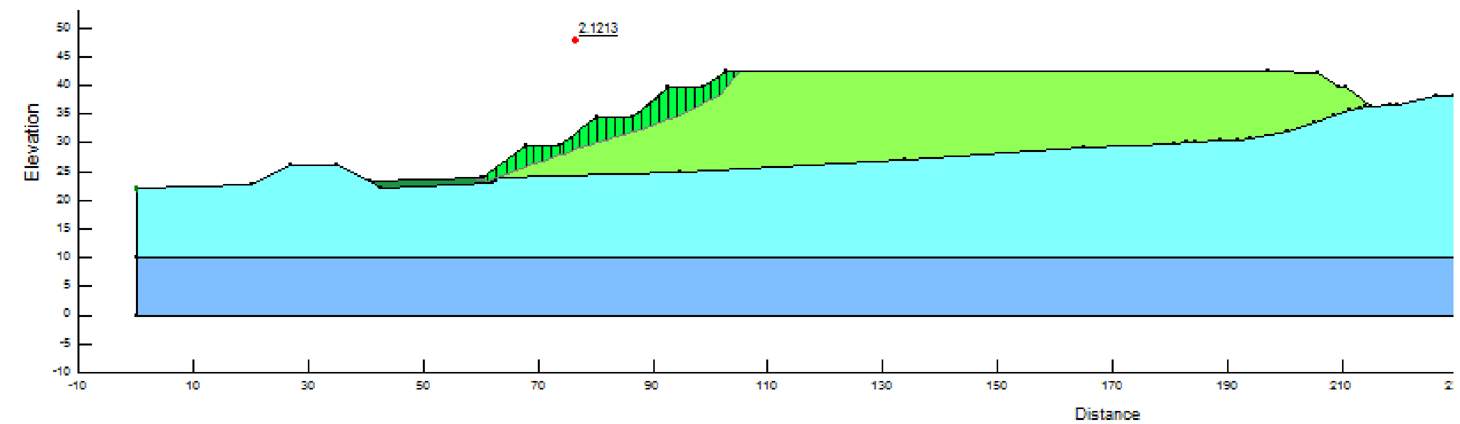
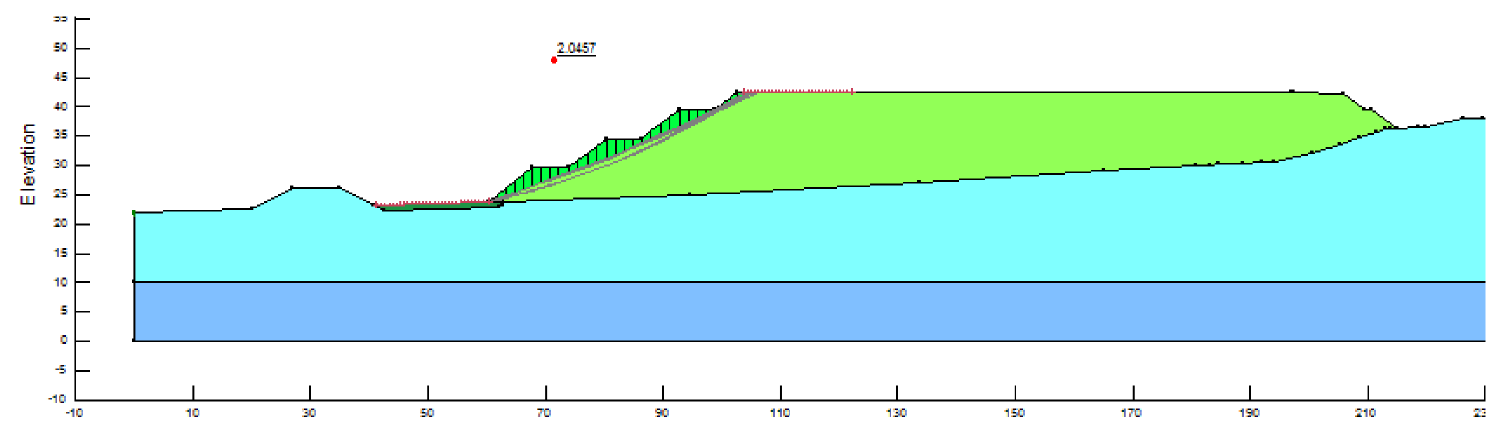
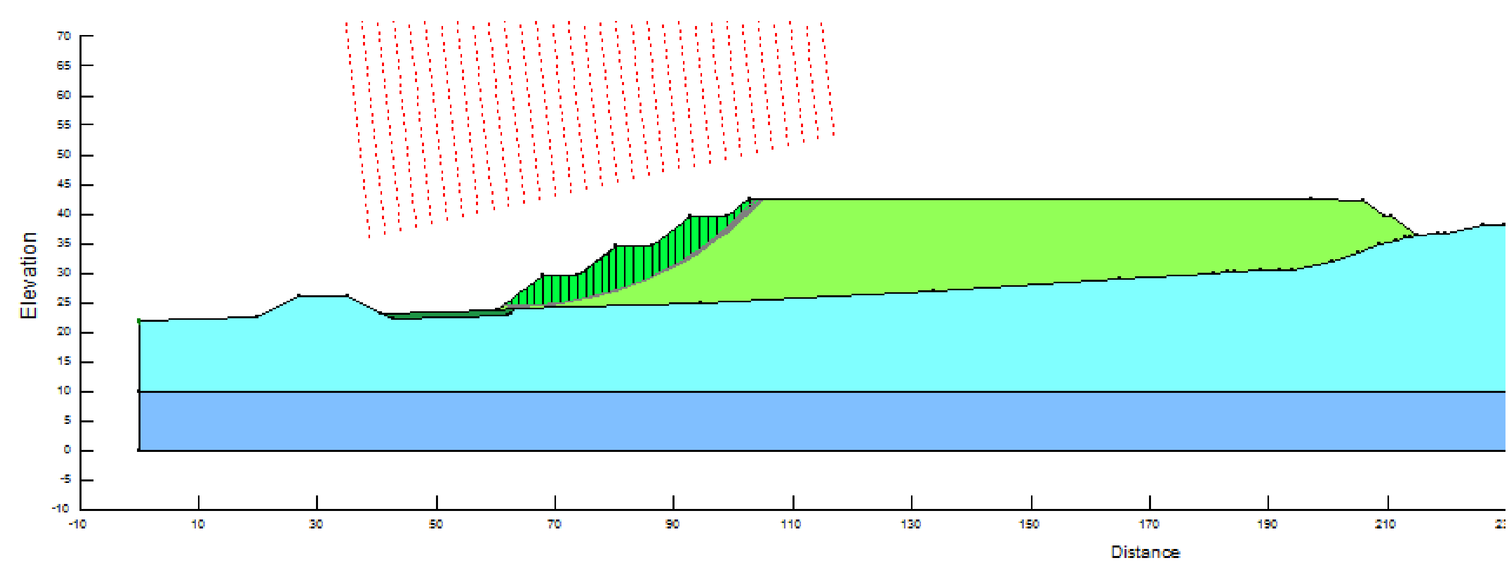
Madrid South Waste Rock Pile Stability Analysis Results – Stability of Dump Surface

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Long Term (Drained Static Condition)	1.6	Fully Defined	
	1.4	Entry Exit	
	1.3	Grid and Radius	

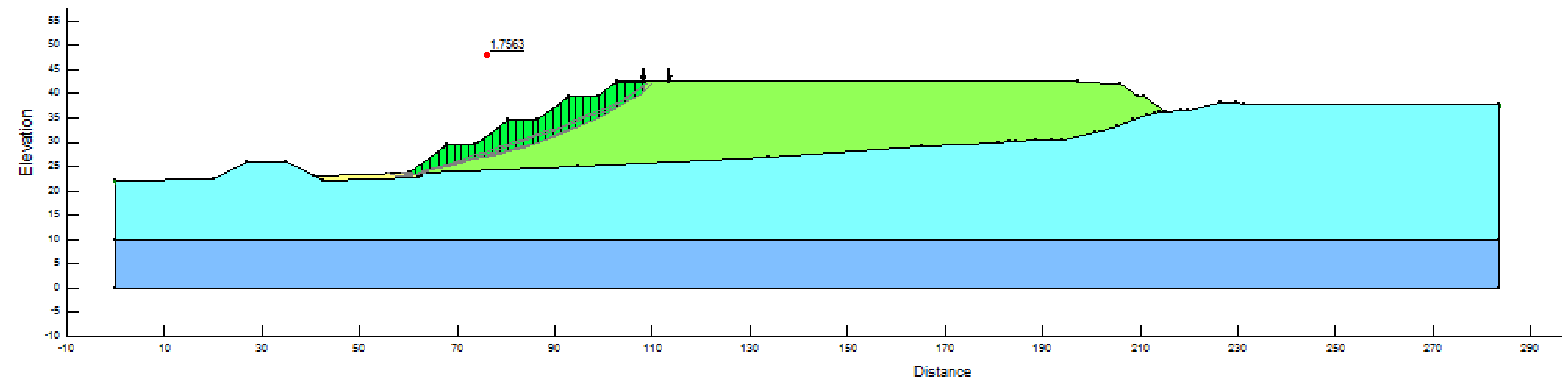
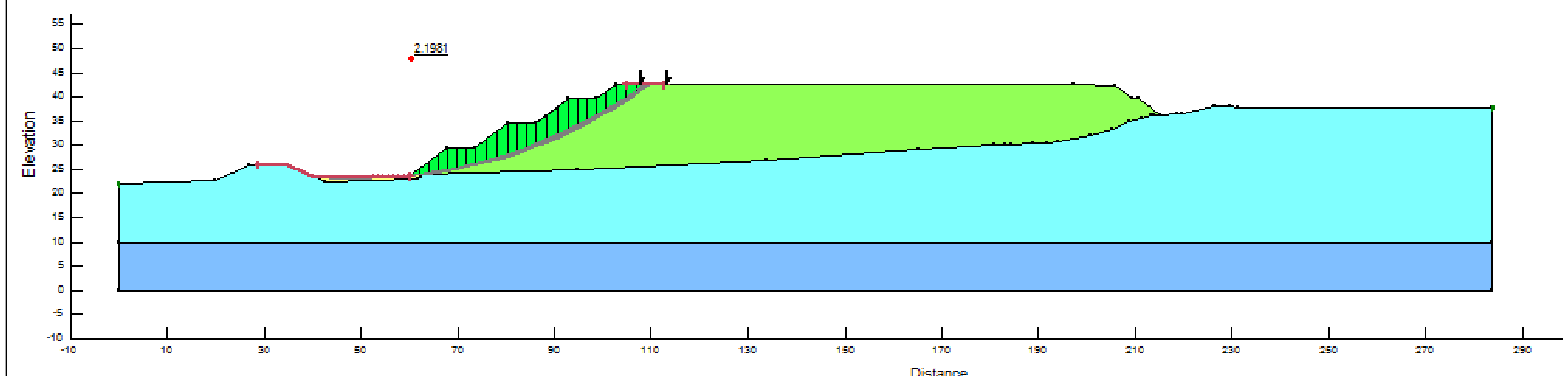
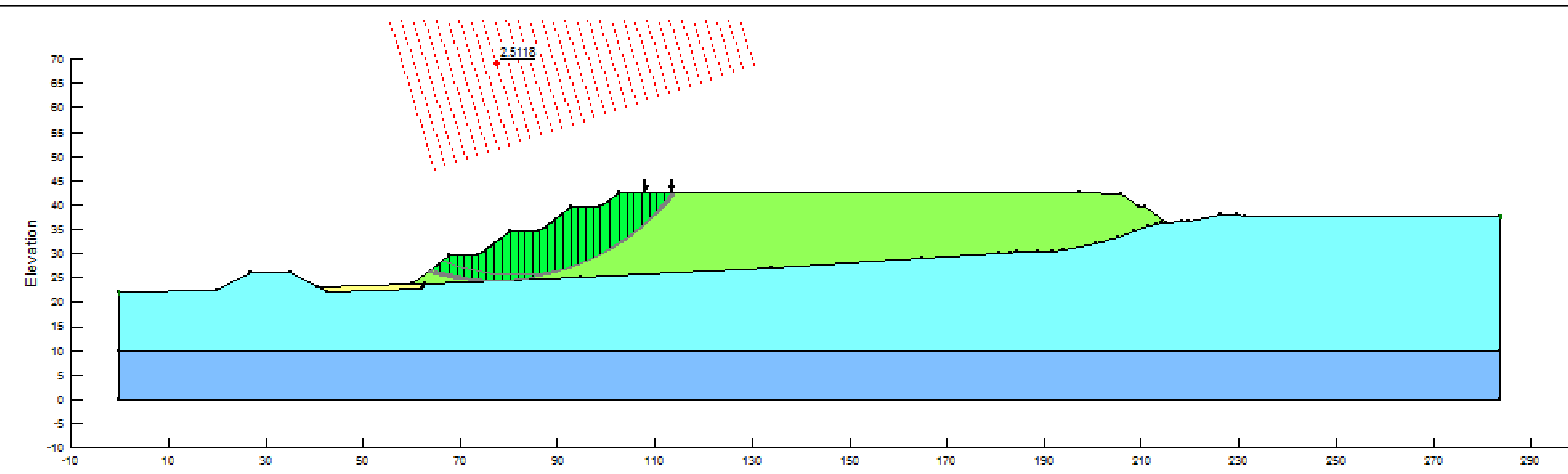
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Short Term (Undrained Static Condition)	1.8	Fully Defined	
	2.1	Entry Exit	
	2.2	Grid and Radius	

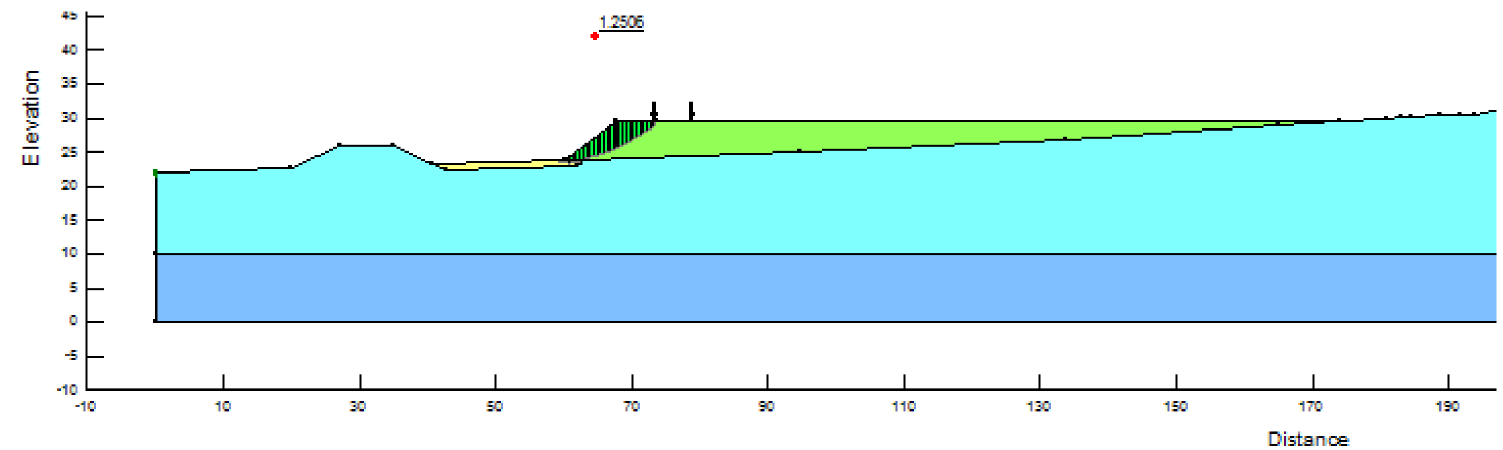
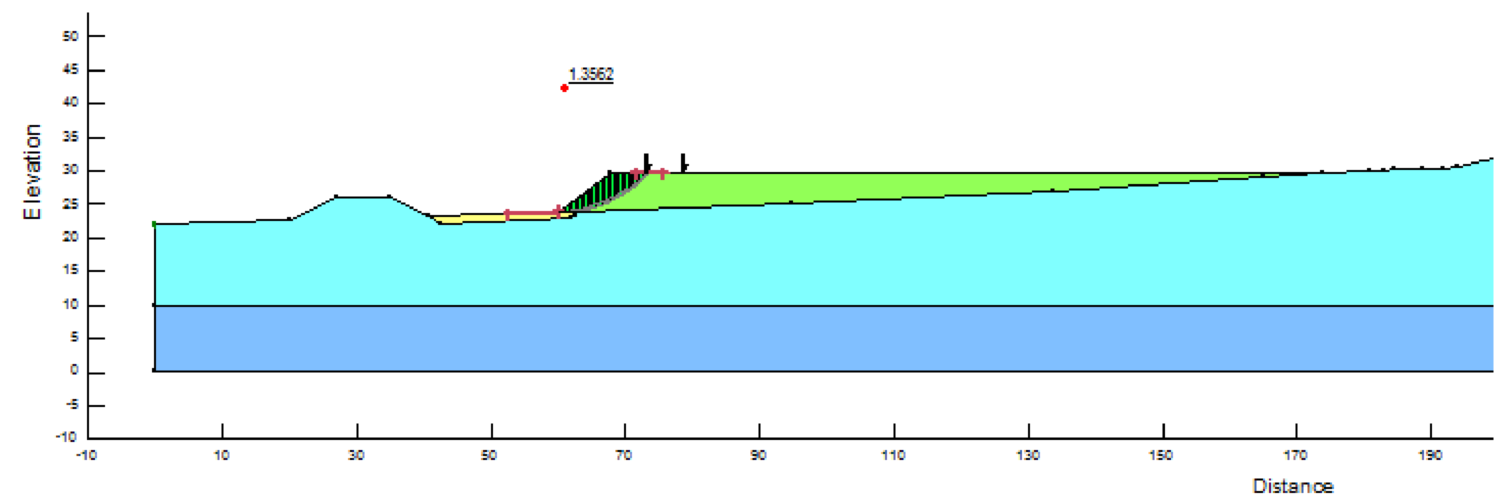
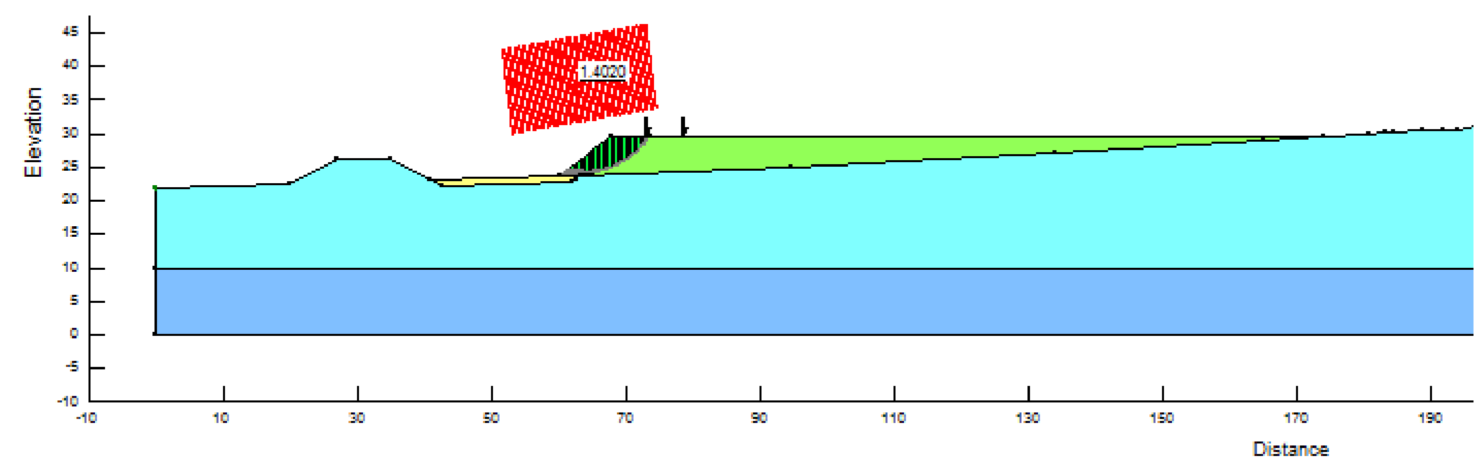
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/Long Term (Drained Static Condition)	2.1	Fully Defined	
	2.0	Entry Exit	
	2.3	Grid and Radius	

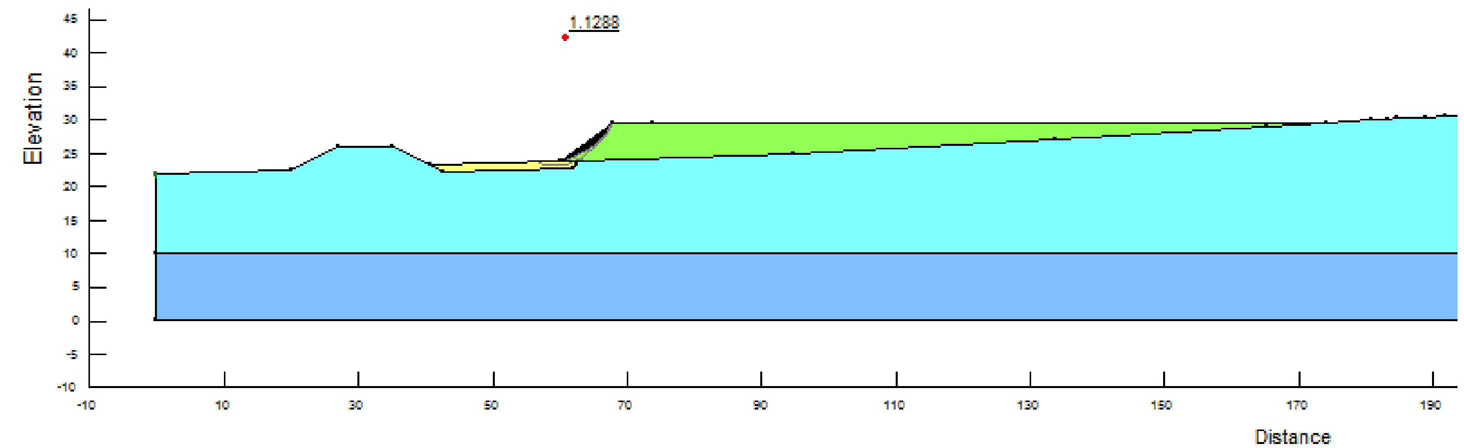
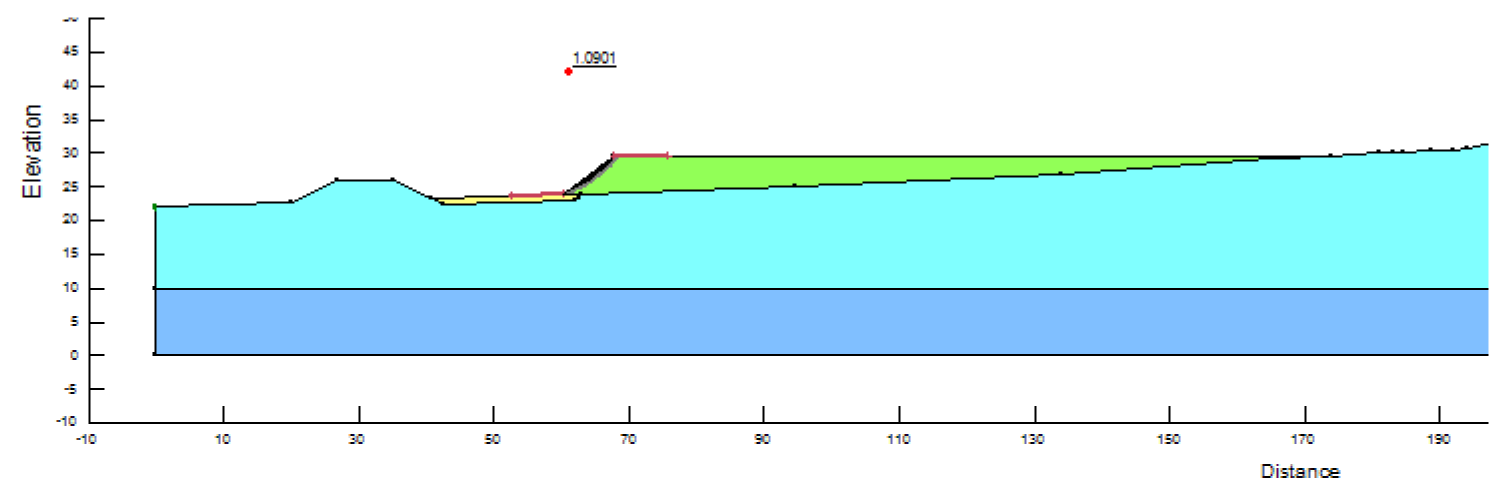
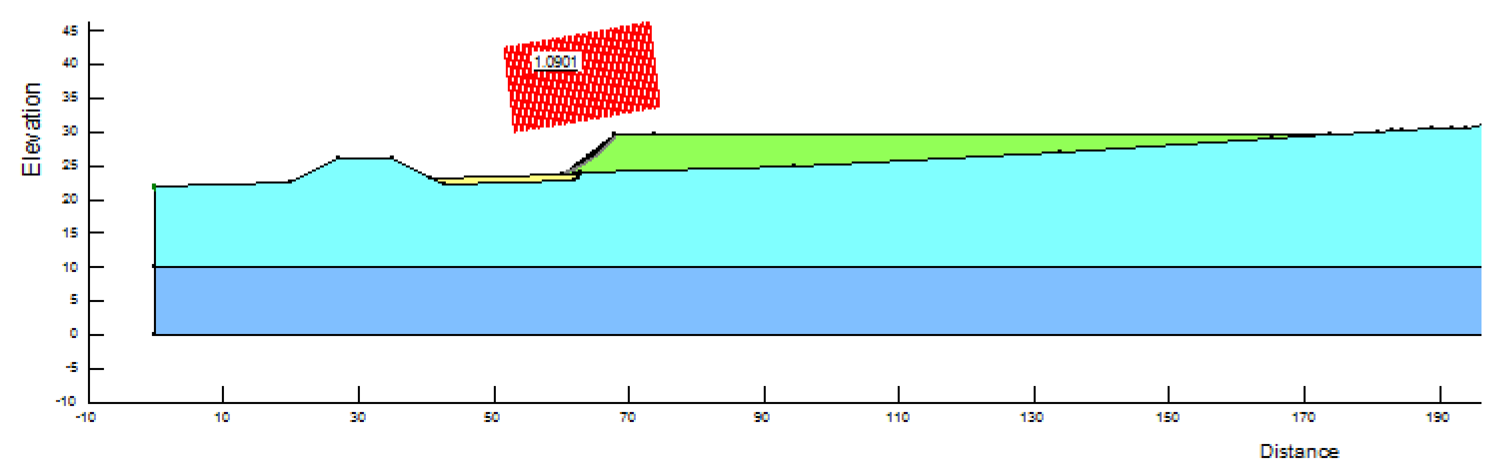
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition`	Factor of Safety	Slip Surface Option	Figure
Madrid South/Pseudo-Static (Earthquake)	1.8	Fully Defined	
	2.2	Entry Exit	
	2.5	Grid and Radius	

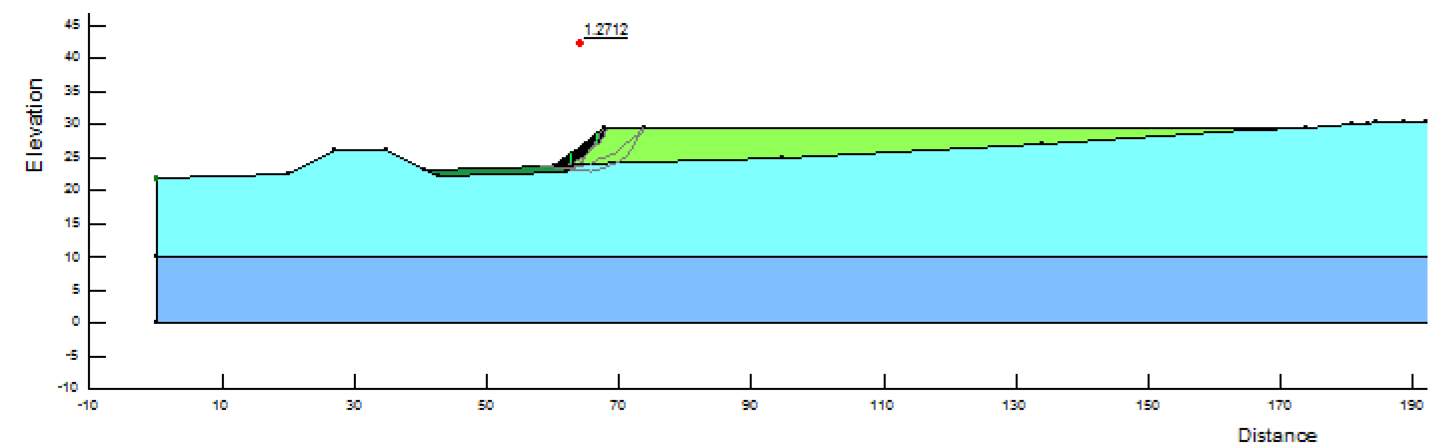
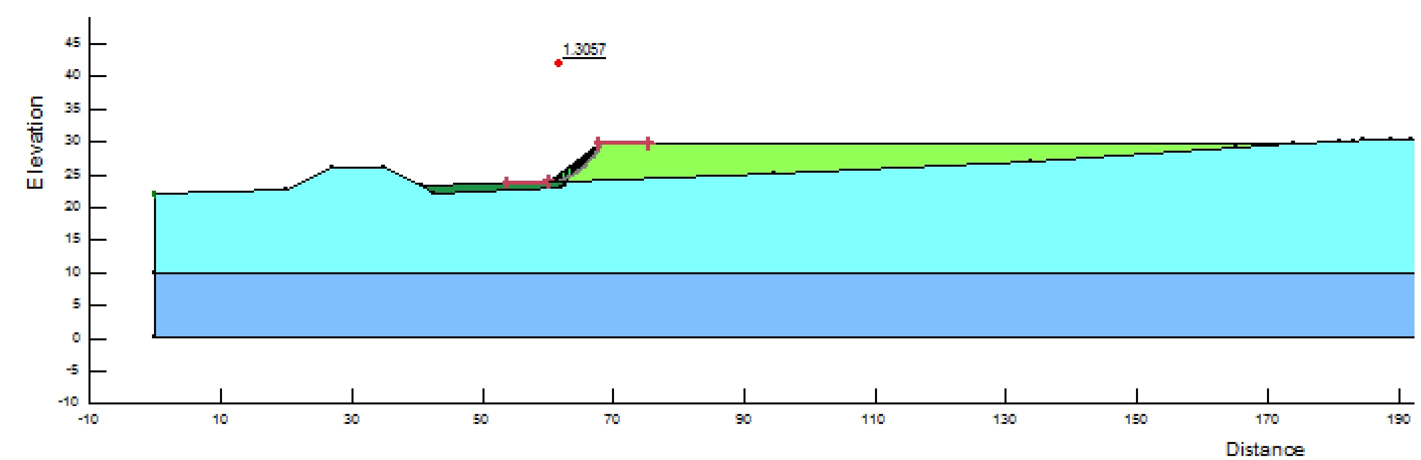
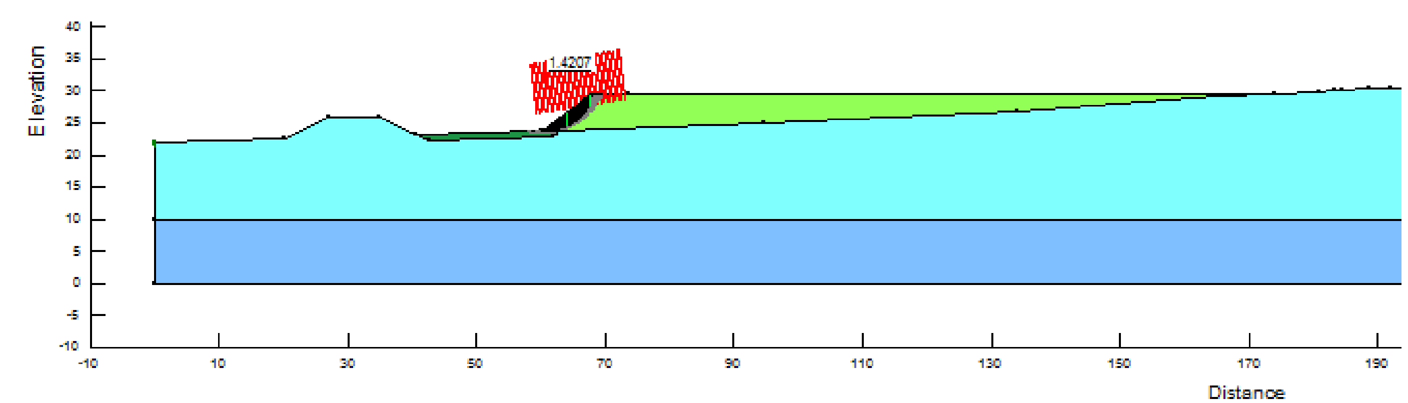
Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench – Short Term (Undrained Static Condition)	1.3	Fully Defined	
	1.4	Entry Exit	
	1.4	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench –Short Term (Undrained Static Condition) **Without Load**	1.1	Fully Defined	
	1.1	Entry Exit	
	1.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
Madrid South/1st Bench –Short Term (Drained Static Condition)	1.3	Fully Defined	
	1.3	Entry Exit	
	1.4	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Overall Stability

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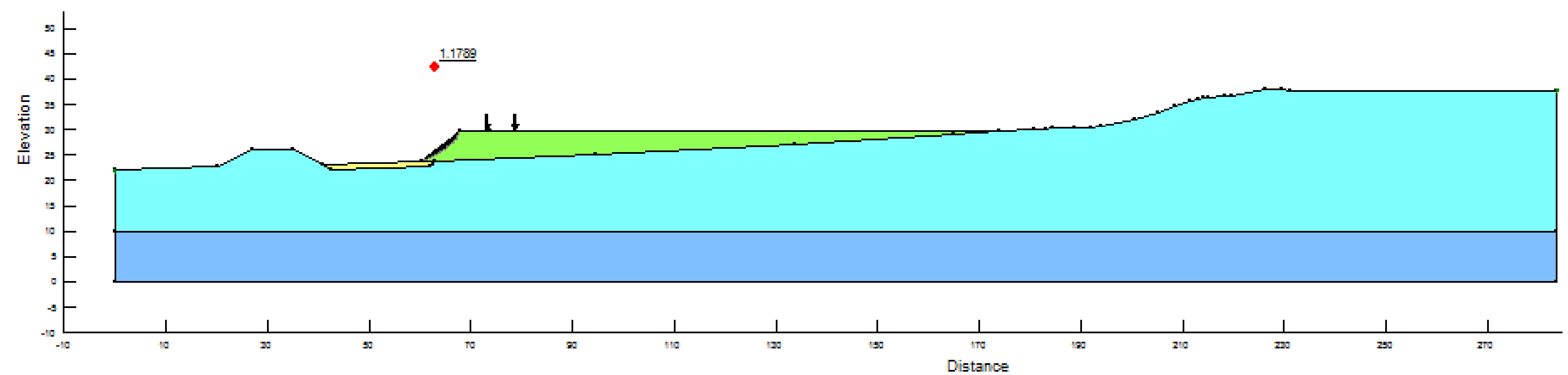
<tr><th>Model/Stability Condition</th><th>Factor of Safety</th><th>Slip Surface Option</th><th>Figure</th></tr>
<tr><td rowspan="3">Madrid South/1st Bench –Pseudo-Static (Earthquake)</td><td>1.2</td><td>Fully Defined</td><td>
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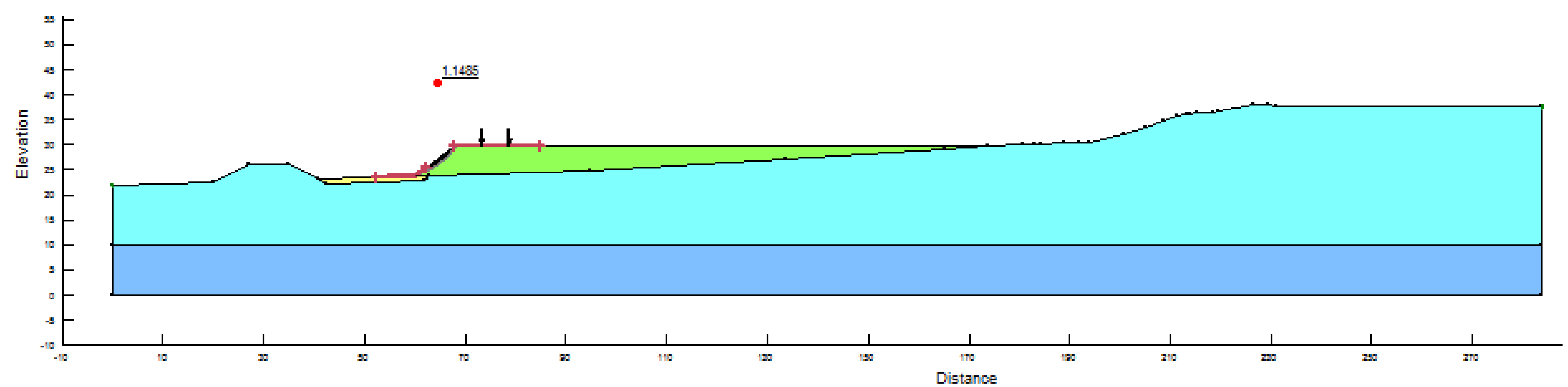
<tr><td>1.1</td><td>Entry Exit</td><td>
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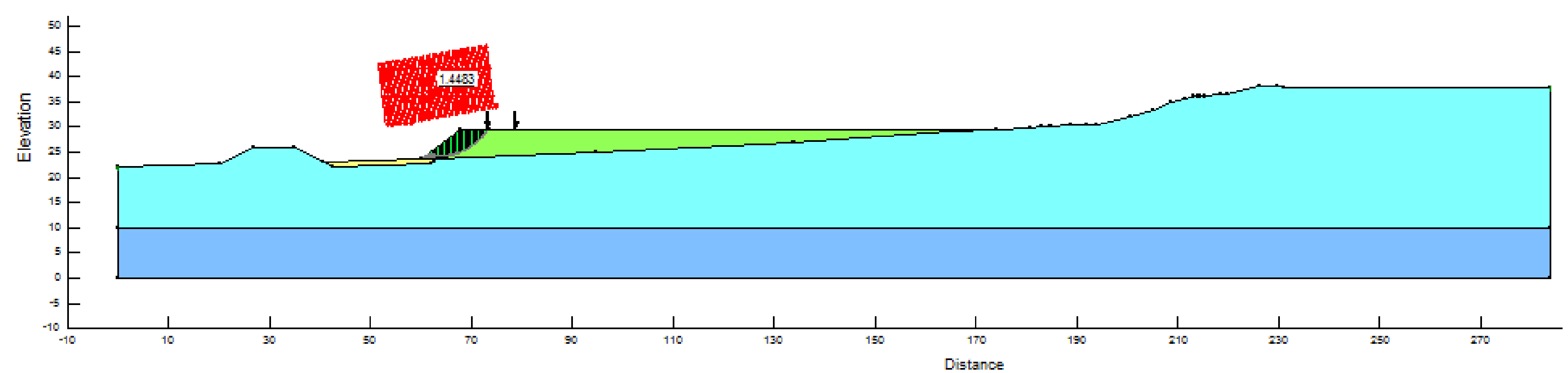
<tr><td>1.4</td><td>Grid and Radius</td><td>
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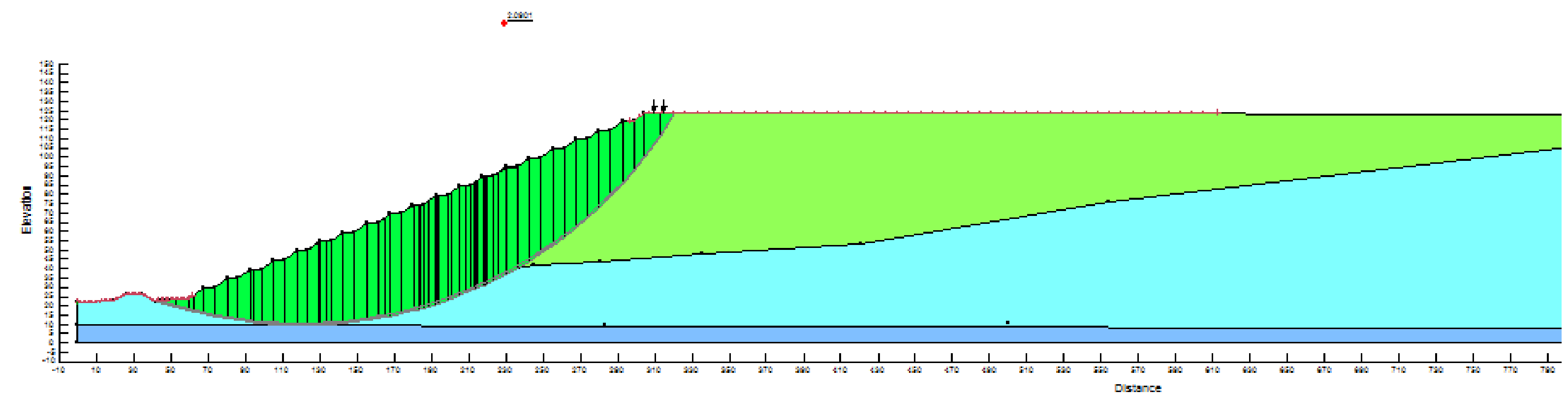
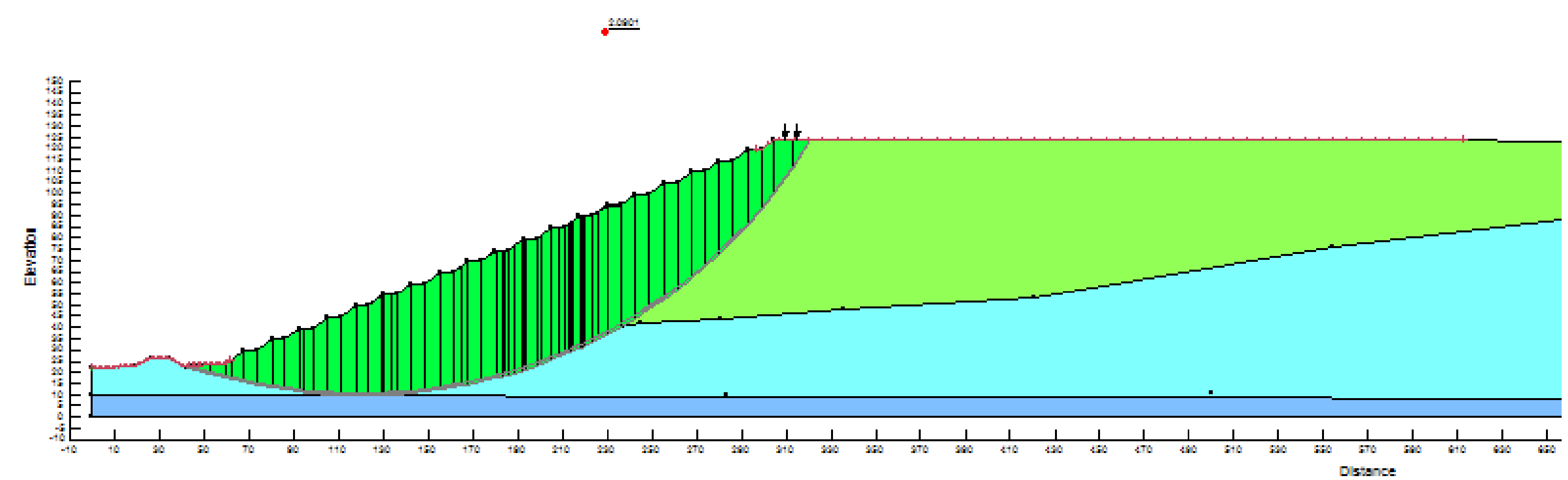
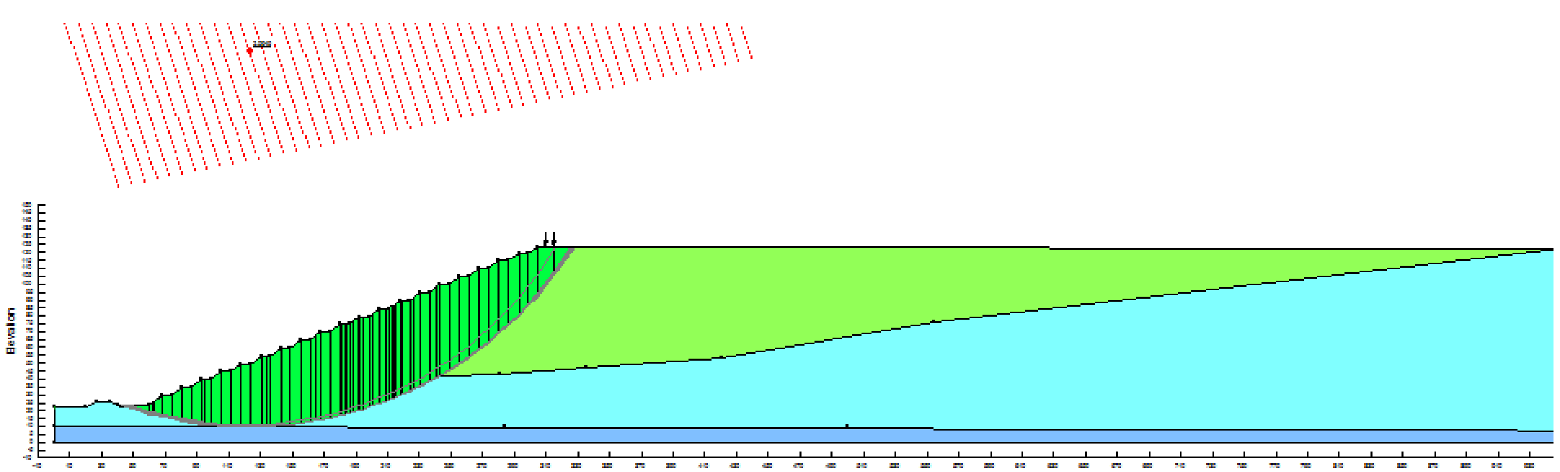
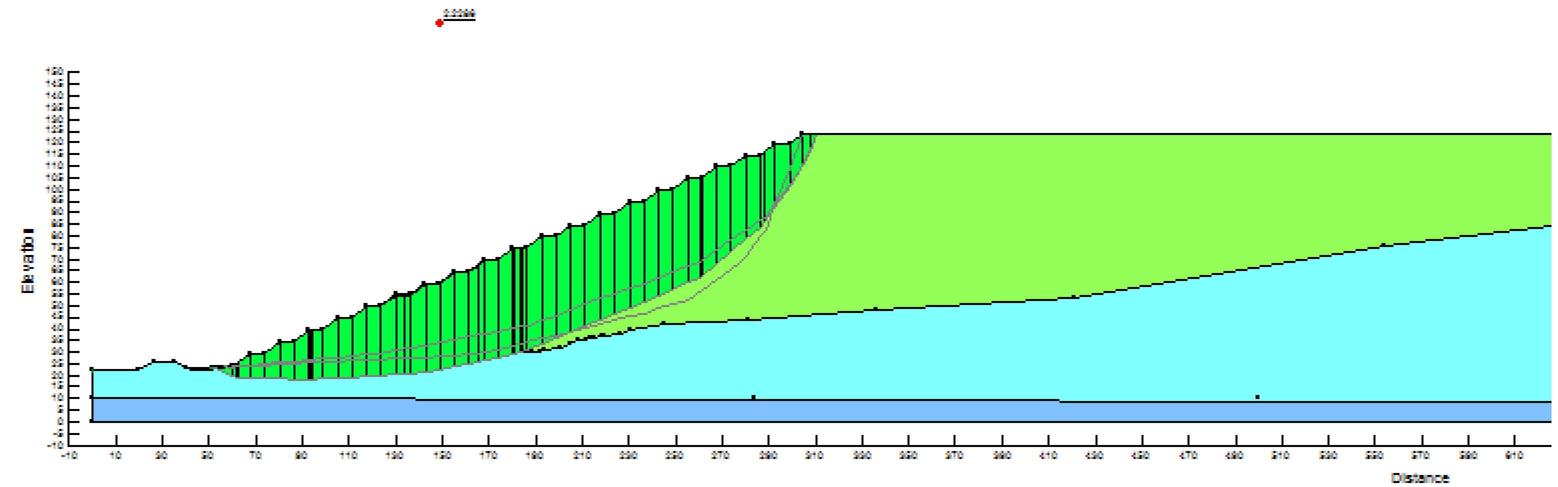
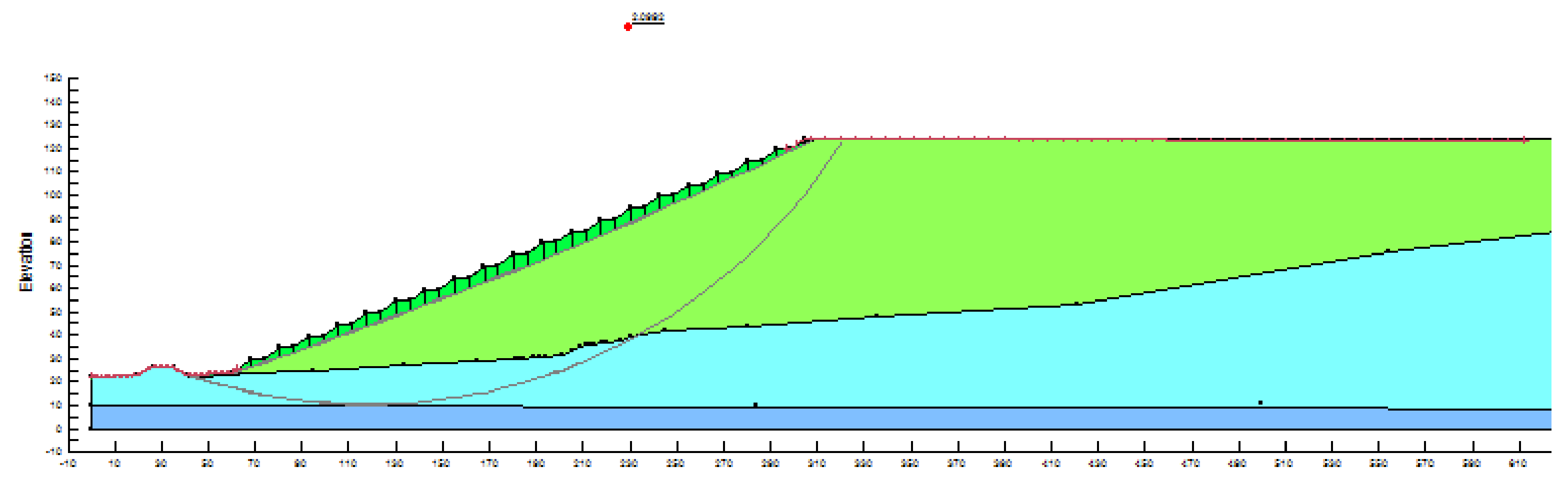
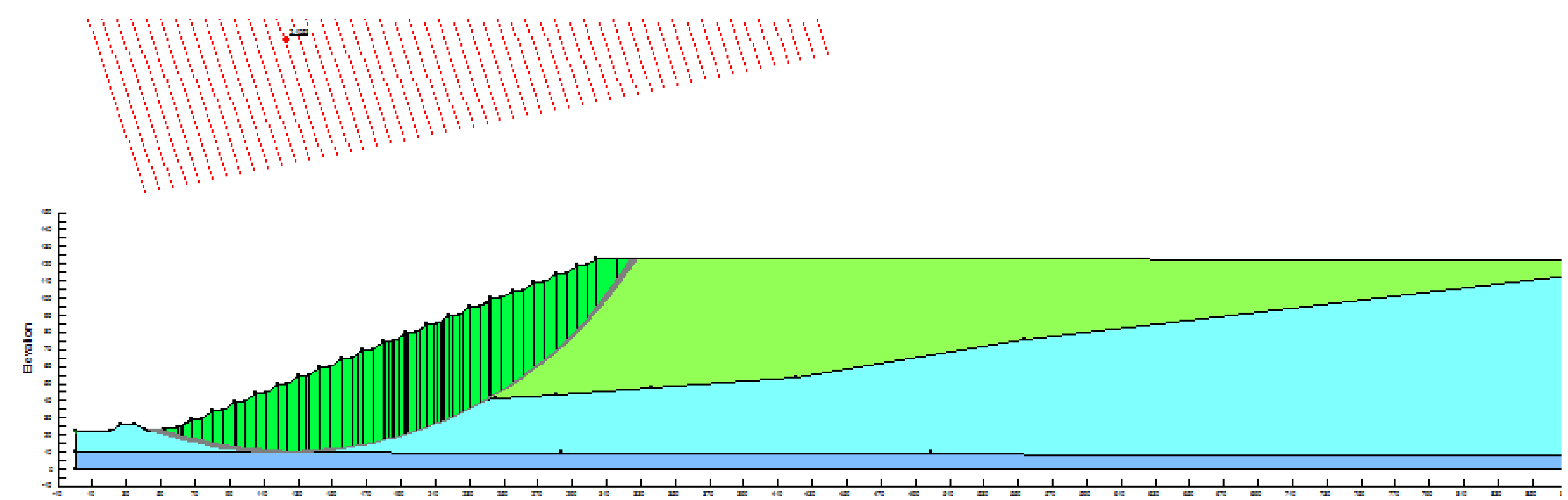
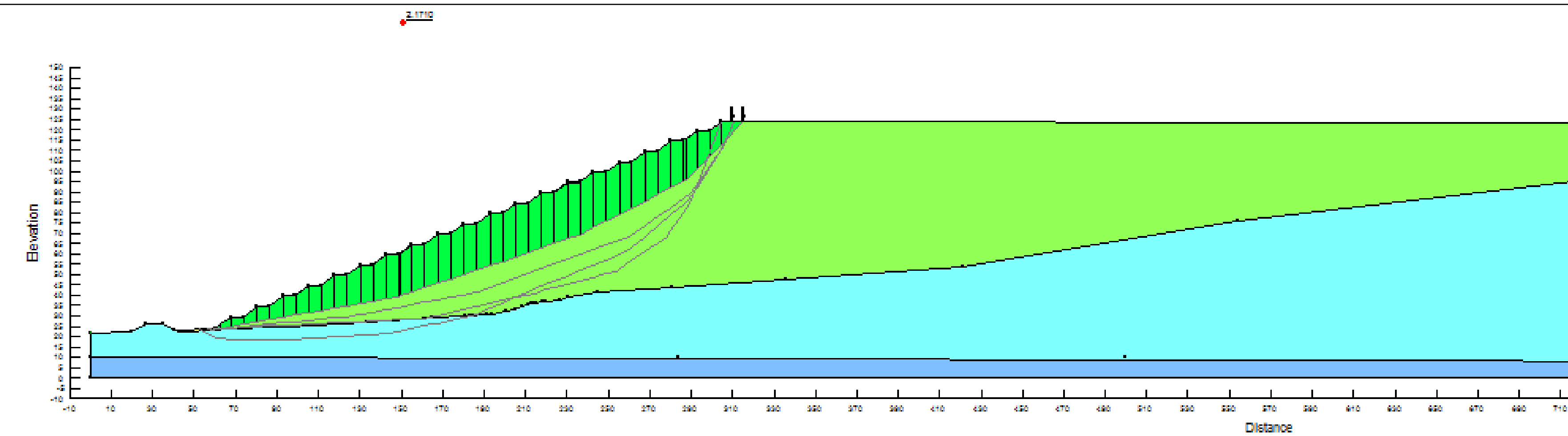
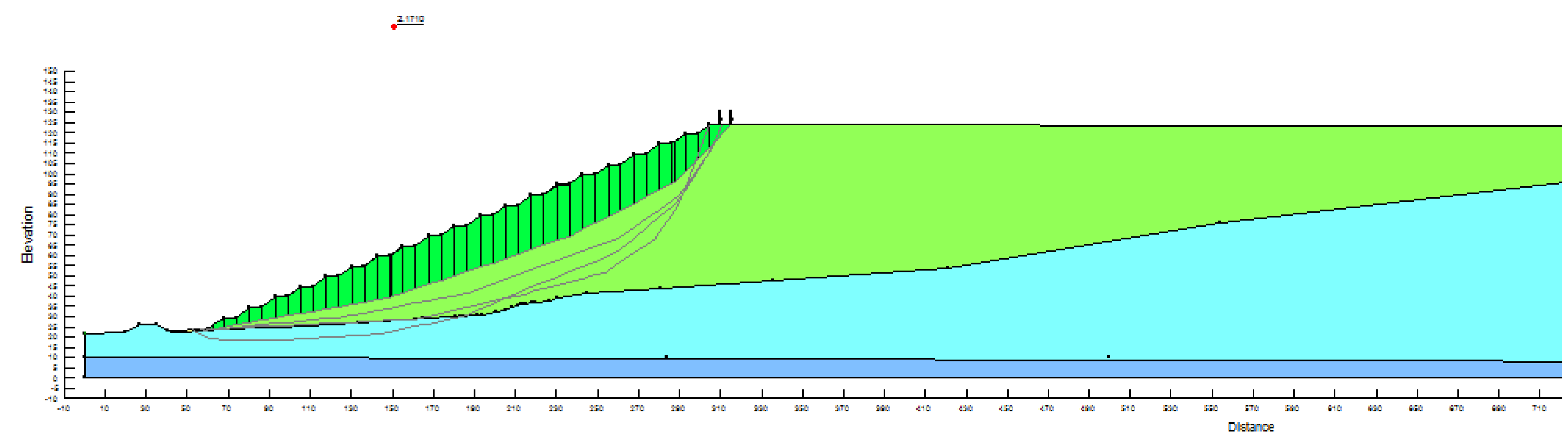
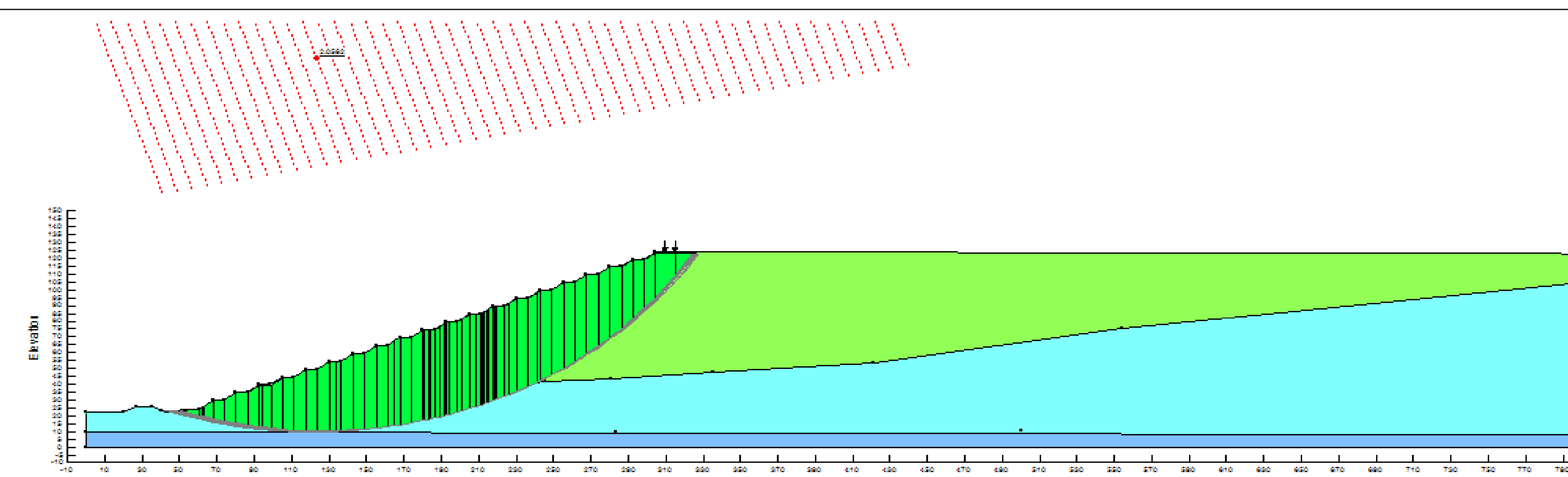
Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
100m High WRD - Short Term (Undrained Static Condition)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
100m High WRD – Long Term (Undrained Static Condition)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Madrid South Waste Rock Pile Stability Analysis Results – Max Height Overall Stability

Model/Stability Condition	Factor of Safety	Slip Surface Option	Figure
100m High WRD - Pseudo-Static (Earthquake)	2.2	Fully Defined	
	2.1	Entry Exit	
	2.1	Grid and Radius	

Hope Bay - Waste Rock Pile Stability Analysis Result Summary

Madrid South Waste Rock Pile (Maximum Height)

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
Stability of Dump Surface	Short Term	Undrain (with truck load)	1	1.1
	Long Term	Drain	1.2	1.3
Overall Stability of Dump	Short Term	Undrain (with truck load)	1.3-1.5	1.8
	Long Term	Drain	1.5	2.0
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	1.8

The distance of the truck load to the crest = 5.5m

Madrid South Waste Rock Pile (First Bench)

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
First Bench Stability of Dump Surface	Short Term	Undrain (with truck load)	1	1.1
	Long Term	Drain	1.2	1.3
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	1.1

100m High Waste Rock Pile

	Stability Condition	Loading Condition	Recommended FOS	Resulting FOS
WRD Height: 100m	Short Term	Undrain (with truck load)	1.3-1.5	2.1
	Long Term	Drain	1.5	2.1
	Pseudo-Static	Undrain (with truck load)	1.1-1.3	2.1

Mined Rock and Overburden Piles

Investigation and Design Manual

Interim Guidelines

May, 1991

British Columbia
Mine Waste
Rock Pile
Research
Committee

TABLE 6.4

INTERIM GUIDELINES FOR MINIMUM DESIGN FACTOR OF SAFETY [1]

STABILITY CONDITION	SUGGESTED MINIMUM DESIGN VALUES FOR FACTOR OF SAFETY	
	CASE A	CASE B
STABILITY OF DUMP 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 (earthquake) [2]	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)

NOTES:
1. A range of suggested minimum design values are given to reflect different levels of confidence in understanding site conditions, material parameters, consequences of instability, and other factors.
2. Where pseudo–static analyses, based on peak ground accelerations which have a 10% probability of exceedance in 50 years, yield F.O.S. < 1.0, dynamic analysis of stress–strain response, and comparison of results with stress–strain characteristics of dump materials is recommended.