Appendix V3-2E

Hope Bay Project: Geotechnical Design Parameters and Overburden Summary Report

Hope Bay Project
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Prepared for

TMAC Resources Inc.

Prepared by

SRK Consulting (Canada) Inc.
1CT022.004
November 2016

Hope Bay Project Geotechnical Design Parameters and Overburden Summary Report

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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 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 Project Certificate, and Phase 2, which includes the permitting and mining of the Madrid and Boston deposits. Phase 1 includes mining and infrastructure at Doris Mine only, while Phase 2 includes mining and infrastructure at Madrid (Madrid North and Madrid South mines) and Boston (Boston Mine) located approximately 10 and 60 km due south from Doris Mine respectively (Figure 1).

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)

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: NRCC 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 be 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 (2016a).

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, 2016a). 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, 2016b).

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.

Program	Description
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(1).
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, 1010)	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 (2016c).

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 (~−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 (~−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

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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 (°)	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)

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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

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Table 12: Typical Sand Overburden Engineering Properties

Element		Value/Comment	Source
Unfrozen	Apparent Cohesion, c'	0 kPa	Estimated
	Friction angle, ɸ (°)	35	Estimated (SRK, 2003)
Frozen	Apparent Cohesion, c'	4.5 MPa	Ladanyi and Morel (1990)
	Friction angle, ɸ (°)	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, ϕ	38 to 40°	Field Observations
Frozen	Apparent Cohesion, c'	5 kPa	Estimated
	Friction Angle, ϕ	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/m³)	Bulking Factor	Loose Density (Mg/m³)	Shrinkage Factor	Compacted Density (Mg/m³)
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

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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, 2006 and 2016d).

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

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)

Element	Unit	Value/Comment	Source
	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)

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**	
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	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.

This final report, "Hope Bay Project Geotechnical Design Parameters and Overburden Summary Report", was prepared by SRK Consulting (Canada) Inc.

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

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

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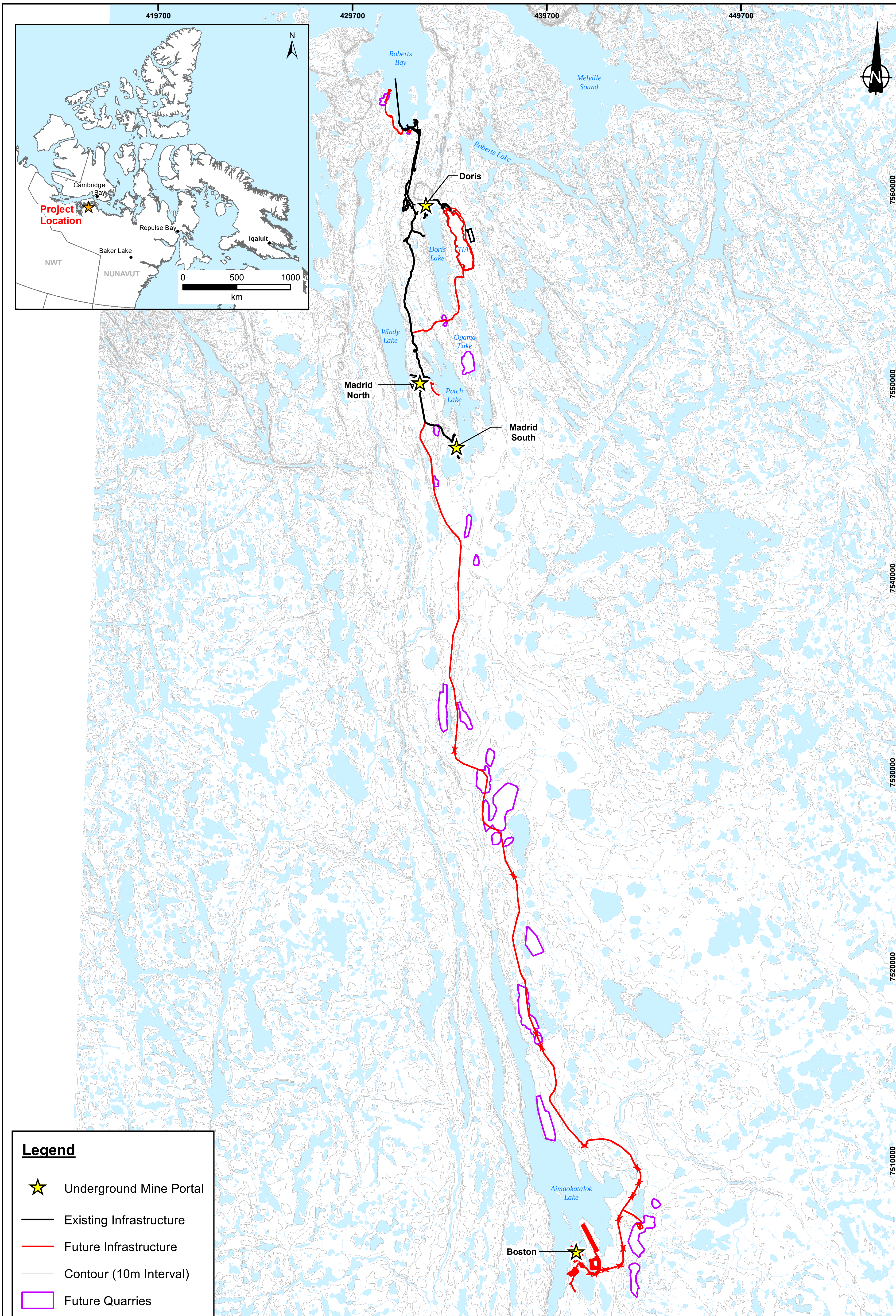
Figures



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Kilometres

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

srk consulting

Project No: 1CT022.004.200.10

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TMAC RESOURCES

HOPE BAY PROJECT

Overburden Summary Report

Site Location Map

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