Geotechnical Design Basis

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HATCH						**Client**

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1. Introduction

Baffinland Iron Mines (BIM) plans to increase the Mary River Mine production to 12 Mtpa, shipping the increased output through Milne Port. This will be achieved by upgrades including the construction of a 110 km long rail line connecting the mine site to the Milne Port, a new crushing and screening facility at the port, larger ore stockpiles and a second ore dock for ship loading.

Hatch Ltd. (Hatch) was retained by BIM to conduct geotechnical drilling investigations and to provide geotechnical design support for the rail line and the facilities to be constructed in the mine site and Milne Port. This document summarizes the geotechnical design basis for the design scope.

2. Thermal Modelling

Hatch's scope of work includes conducting thermal analyses for the cut sections along the proposed rail line and the foundations of the proposed port structures including an ore stockpile reclaim tunnel, crushing and screening plants, a dumper and dumper loadout tunnel, and stacker reclaim berm. Accordingly, Hatch's geotechnical team has reviewed documents, reports and published literature regarding soil conditions, climate data, and thermal properties of the materials, along with the documents related to the field investigation and design of the rail line and port structures. From these information sources, representative material parameters, as well as initial and in service boundary conditions were selected and/or calculated to be used in all thermal modelling for this project.

The objective of the thermal analyses is to determine the potential subsurface temperature change and disturbance from construction activities. The results from the analyses will be reviewed to ensure that the threshold temperature in critical locations is not exceeded. The threshold temperature is the limit above which unacceptable subsurface disturbance/ deformation will occur due to thawing of the permafrost, and it varies among the proposed structures. For the rail line foundation, it is assumed that significant thaw effects can be minimized to acceptable levels if:

- the subsurface material in the zone 1 meter below the base of the rail embankment is protected by not allowing this layer to reach a temperature above -3°C (Line A).
- the temperature in a zone 1 meter below the cut faces can be maintained below -2°C for ice-rich layers (Line B) and below 0°C for ice-poor layers (Line C).

The reference lines are illustrated in Figure 2-1. The temperature threshold for structure foundations will be provided in the structure-specific memorandums.

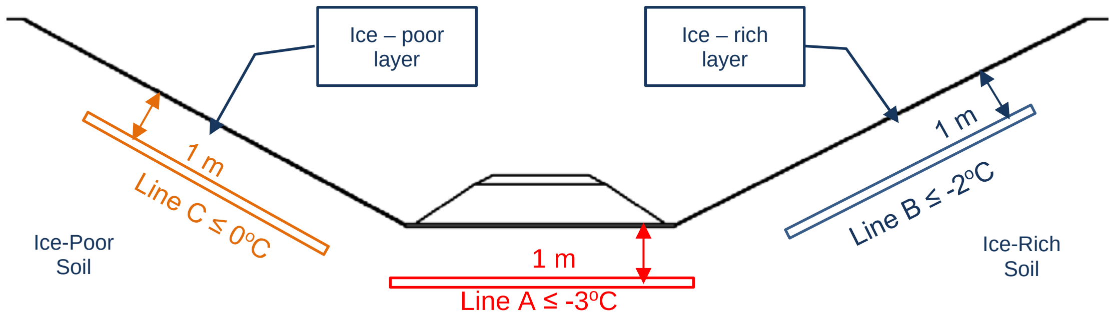


Figure 2-1: Temperature Threshold Zones Below Typical Railway Cut

This design basis provides the summary of the material parameters and the boundary conditions used in thermal modelling along with the list of reference documents used for this selection.

2.1 Methodology

GeoStudio (SLOPE, SIGMA/W, TEMP/W) version 2012, a two-dimensional Finite Element (FE) software developed by Geo-Slope International Ltd, will be used for this study. The selection of material parameters, boundary and initial conditions used in the analysis are discussed in the following sections.

2.2 Material Properties

Following the review of the information provided in the 2016/2017 geotechnical investigation reports (Hatch 2017a, 2017b), the subsurface soil layers were categorized into “Silt” and “Sand/Gravel” units. Other units to be modelled in the thermal study include “Fill” (Types 5, 25, 8 and 12 specified in Quarried Fill Materials Requirements specifications), “Bedrock”, “Insulation” (assumed to be Extruded Polystyrene Foam), “Concrete” and “Steel” units. The thermal properties of these units were selected based on the values recommended in technical references (by Andersland and Ladanyi, 2004; Johnston, 1981). These thermal properties are summarized in Table 2-1.

Table 2-1: Recommended Thermal and Physical Properties for Design

Material	Frozen Thermal Conductivity (J/s/m/°C)	Unfrozen Thermal Conductivity (J/s/m/°C)	Frozen Volumetric Heat Capacity (J/m³/°C)	Unfrozen Volumetric Heat Capacity (J/m³/°C)	Insitu Water content (%)	Insitu Volumetric Water content (m³/m³)
Snow, compacted	0.7[1]	-	1,045,000[1]	-	-	-
Fill (Types 5, 25, 8 and 12)	4.5[1]	3.0[1]	2,400,000[1]	3,000,000[1]	2	3.6
Silt[4,5]	2.0[1]	1.3[1]	2,200,000[3]	2,200,000[3]	30	45
Sand/Gravel[4,5]	3.0[1]	2.0[1]	2,600,000[3]	2,600,000[3]	15	25.5
Bedrock, limestone	2.37[2]	2.37[2]	2,403,000[2]	2,403,000[2]	1	2.2
Bedrock, Gneiss	2.41[2]	2.41[2]	2,754,000[2]	2,754,000[2]	1	2.2
Insulation	0.035	0.035	37,500[1]	37,500[1]	0	0
Concrete	1.5[1]	1.5[1]	2,000,000[1]	2,000,000[1]	1	2.4
Steel	43.0[1]	43,0[1]	3,750,000[1]	3,750,000[1]	0	0

[1](Andersland and Ladanyi, 2004)
[2](Eppelbaum, Kutasov, and Pilchin. 2014)
[3](Johnston, 1981)
[4](Hatch, 2017a)
[5](Hatch, 2017b)

2.3 Boundary Conditions

Ground surface boundary: It was assumed that the top of the soil profile and edge of excavation, rail embankment or insulation boundaries experience a temperature which is related to the air temperature. The air temperature is assumed to be the mean monthly air temperature from Pond Inlet, NU (1981-2010) extracted from the Government of Canada website and proposed in Table 2-2 and Figure 2-2.

Table 2-2: Mean Monthly Temperatures – Pond Inlet, NU, 1981-2010 Climate Normals

Month	Mean Daily Average Temperature (°C)
January	-33.4
February	-33.7
March	-30.0
April	-21.9
May	-9.3
June	2.4
July	6.6
August	4.8
September	-0.8
October	-9.7
November	-21.7
December	-28.2

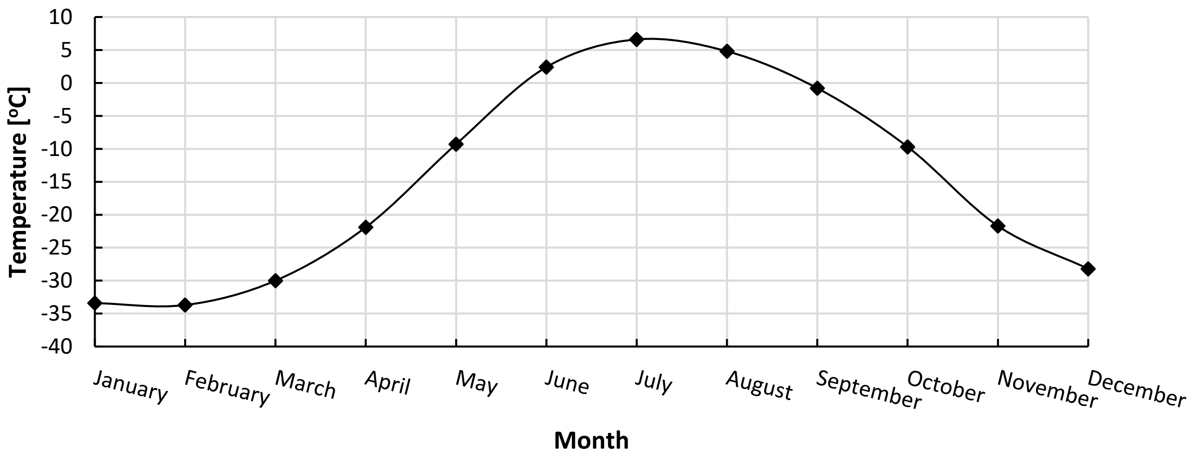


Figure 2-2: Mean Monthly Temperatures for Pond Inlet, NU (1981-2010)

There is a non-linear relationship between mean air temperatures and mean ground surface temperatures, which was accounted for by correlating the ground surface boundary conditions with the air temperature using an empirically determined function coefficients called the freezing factor (n_f) and thawing factor (n_t) for freezing and thawing seasons, respectively.

As large ranges were provided for the two factors (mainly for n_f) in the reference documents, a calibration was completed to ensure that the calibrated n-factors represent the specific conditions encountered at the Mary River site. The temperature profiles were reviewed for thermistors installed in 2006, 2007, 2008, and 2011 as presented in reports by Knight Piesold (2007, 2008, 2010) and (Hatch, 2012). One representative profile was selected for a thermistor installed within the mine site (BH2007–10) with temperature curves spanning throughout the year. There is no thermistor information at the port, or along the proposed rail line, or the Tote Road.

Temperature profiles were generated for an undisturbed ground surface and modelled based on the stratigraphy found in BH2007-10 borehole report by Knight Piesold (2008). The material parameters were assigned from Table 2-1. The ambient air temperature was sourced from (Government of Canada, 2017), for the time duration from 2008 to 2011 chosen to correspond with the thermistor measurement dates in BH2007-10. An n_t of 1.2 was chosen (based on Andersland and Ladanyi, 2004) and the n_f was varied between 0.5 and 0.9 (Andersland and Ladanyi, 2004) to produce numerical temperatures profiles which best matched the thermistor recorded temperature profiles. Comparing the numerical results with factual temperature profiles, an n_f of 0.7 was chosen from this calibration exercise. Temperature profiles from both numerical modelling and recorded thermistor data are presented in Figure 2-3.

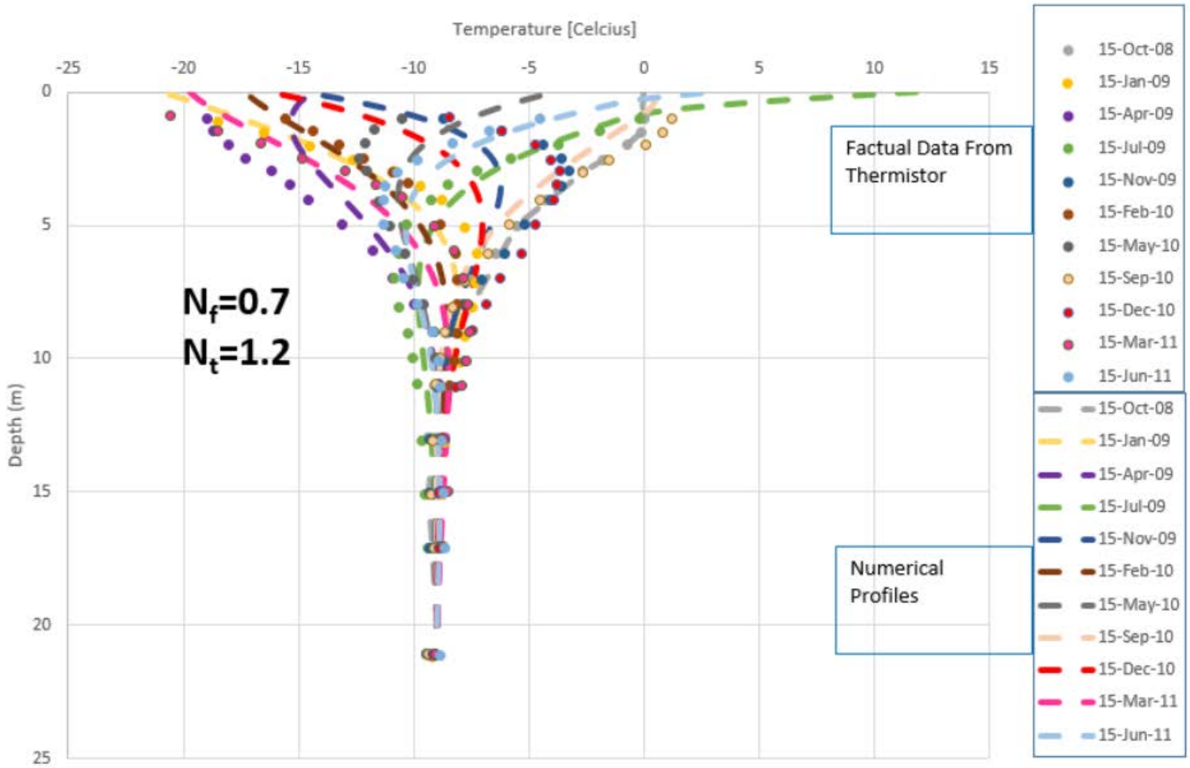


Figure 2-3: Temperature Profiles containing Numerical and Recorded Data Overlain

As the available temperatures profiles are from the thermistors predominantly installed in Sand and Gravel material, the N-Factors for other materials (silt and fill) have been sourced from reference documents. A summary of the n-factors used for the thermal modelling can be found in Table 2-3.

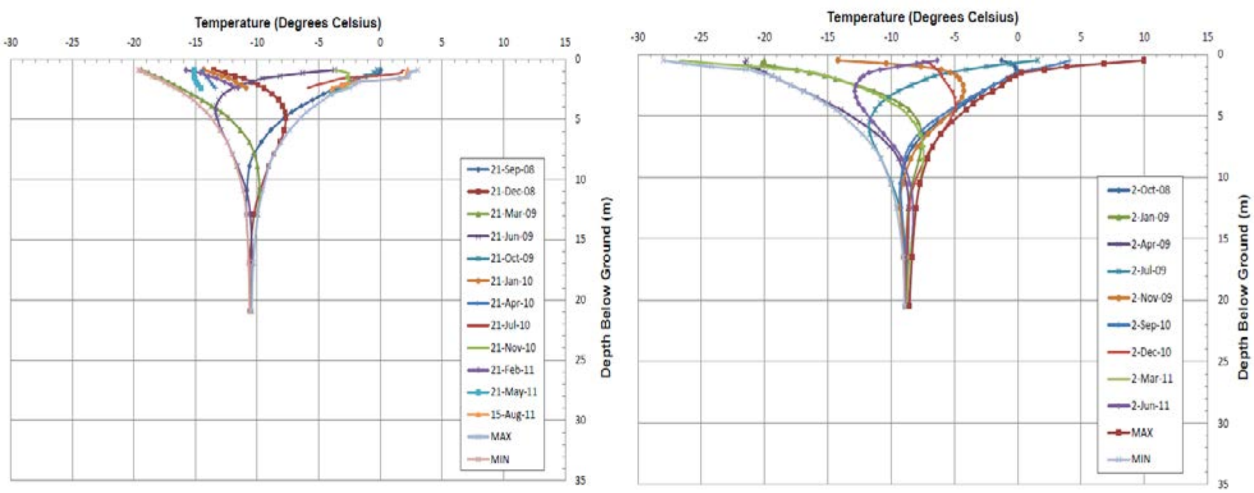
Table 2-3: N-factors to be used in Modelling

Material	N – factors	
	Freezing (n_f)	Thawing (n_t)
Sand and Gravel	0.7	1.2
Silt	0.5(1)	1.2(1)
Rockfill	0.8(1)	1.5(1)

Note:(1) N-Factors sourced from (Andersland and Ladanyi, 2004)

Bottom boundary: Based on the data from the thermistors installed at the Mine site, the temperatures at the depths from 15 to 20 m below existing ground surfaces range between -8°C and -11°C, depending on locations and specific soil conditions (referring to Figure 2-4 for sample thermistor data). To simplify the thermal analyses, the bottom boundary temperature is set at -10°C at 15 m or 20 m depth, depending on the depth of excavation, below the ground surface. A sensitivity analysis was carried out on the bottom boundary by extending the depth of a representative thermal model. The temperature changes compared to the original model were negligible indicating the low sensitivity of the modelling to the depth of the bottom boundary beyond that selected for this study.

Left and right boundaries: These were assumed to be no-flow boundaries, which is the default boundary condition in the TEMP/W software (i.e., heat neither enters nor exits through these boundaries). A sensitivity analysis also carried out on the horizontal boundary by extending the width of a representative thermal model. The temperature changes compared to the original model were also found to be negligible indicating that the modelling is not sensitive to the horizontal boundaries beyond that selected for this study.



(a) (b)

Figure 2-4: Sample Thermistor Data (a) BH4-2008-66 (b) BH4-2008-178

2.4 Climate Change

The mean monthly temperature database is established based on the data recorded between 1981 to 2010 at the climatic station located at Pond Inlet. However, it is predicted that there is a trend of increasing temperatures over time due to the effects of climate change. While the mean monthly temperatures are applicable to the current time, significant temperature changes may occur by the end of mine life at 2039 according to the Intergovernmental Panel on Climate Change (IPCC) long term climate change studies.

To account for the potential climate change, the data from the IPCC 2014 report titled *Climate Change 2014 Synthesis Report* by IPCC (Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change, 2014), and the IPCC 2003 report titled *Future Climate in World Regions* by Ruosteenoja et al., 2003, were analyzed.

The 2014 IPCC report adopts the Representative Concentration Pathway (RCP) criteria for modelling various future Greenhouse Gas Concentration (GHC) trajectories. The RCPs include a stringent mitigation scenario (RCP2.6), two intermediate scenarios (RCP4.5 and RCP6.0) and one scenario with very high GHG emissions (RCP8.5). A graphical representation of the four RCPs are shown in Figure 2-5. The climate change results from an RCP of 4.5 was chosen to correspond with the mean concentration trajectory.

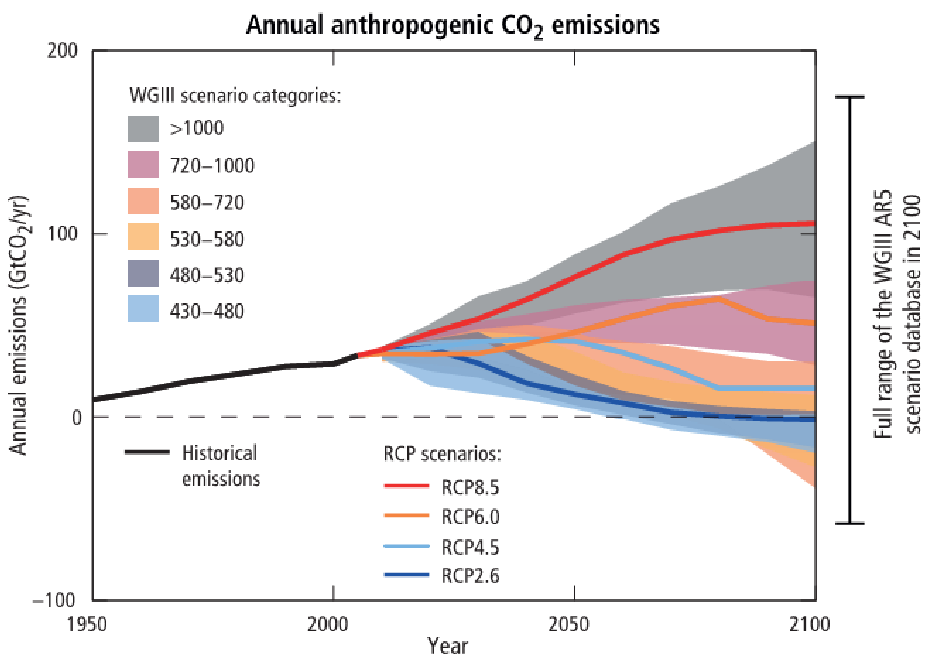


Figure 2-5: Four RCP Scenarios Corresponding to Predicted Future GHC Concentrations (IPCC 2014)

The 2014 IPCC prediction is relative to the 1986 to 2005 records. A graphical output of the temperature change from 2016 to 2035 in the Arctic region from the 2014 IPCC report is shown below in Figure 2-6.

The 50th percentile modelling results were chosen as it corresponds to the mean output results. Potential temperature increases of 1.0 – 1.5°C and 2.0 – 3.0°C are predicted for the June – August and December – February periods, respectively.

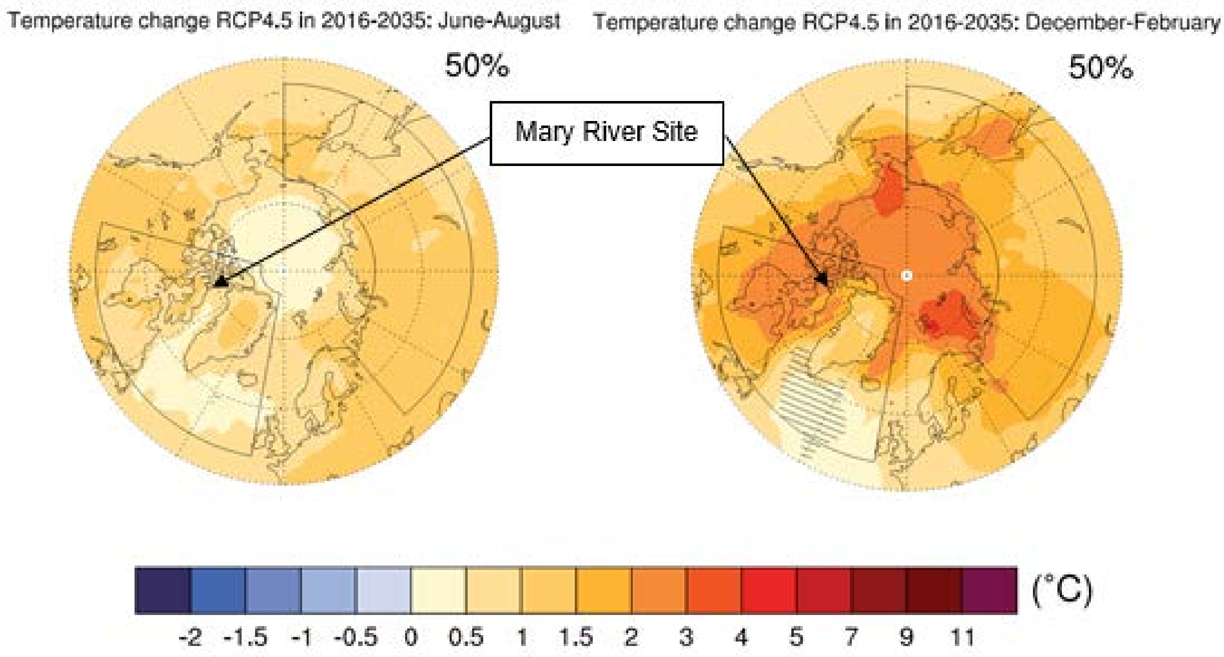


Figure 2-6: Predicted Mean Temperature Change in Arctic Region from 2016 to 2035 (Based on IPCC 2014)

Figure 2-7 presents the climate change predicted from different climate change models for the Arctic Region from the 2003 IPCC report. The model data presented in Figure 2-7 correlates the change in temperature (x-axis) to the change in precipitation (y-axis) for the period from 2010 to 2039, split up into four periods (Dec–Feb, Mar–May, Jun–Aug, Sept–Nov). Among the temperature change predicted in the different models, the mean values were highlighted for all periods by the red dash line.

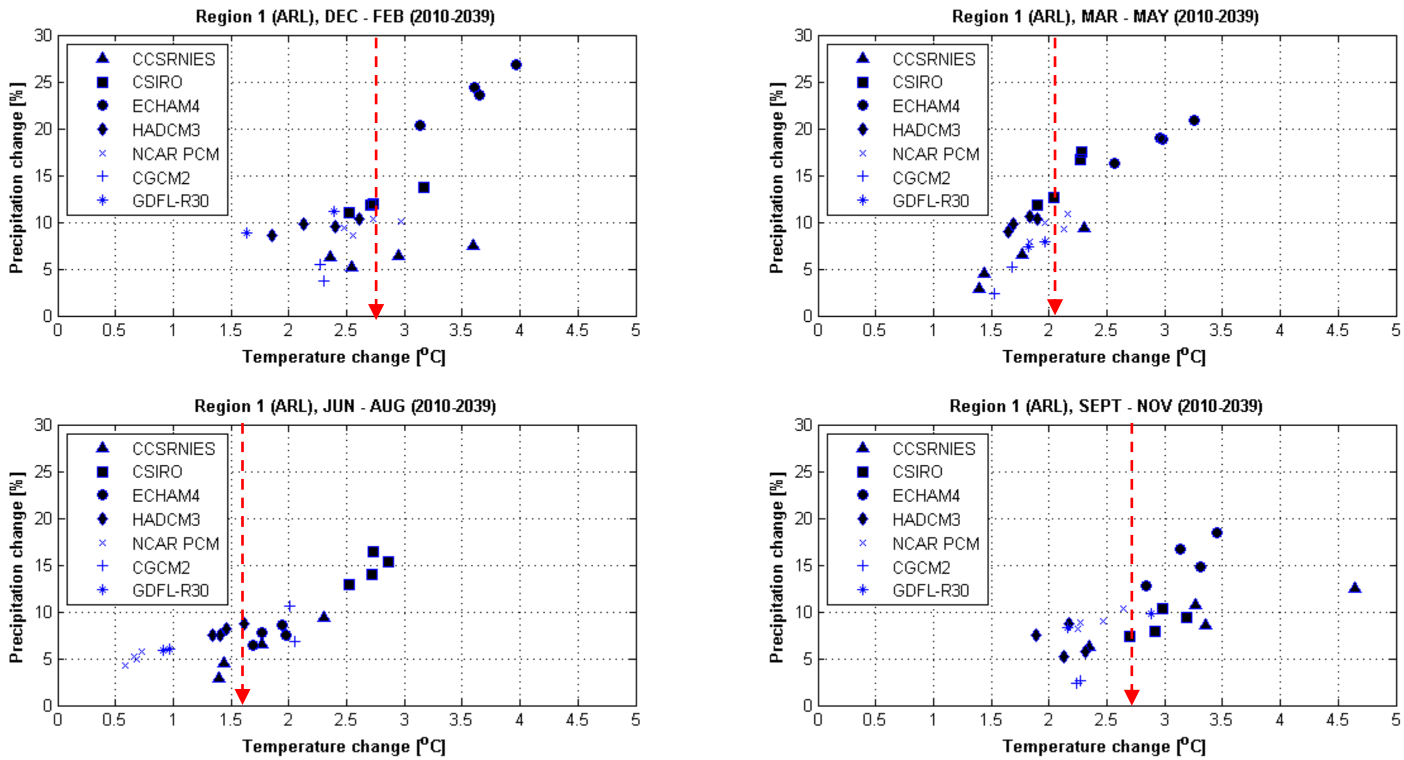


Figure 2-7: Predicted Mean Temperature Change from 2010 to 2039 (Based on IPCC 2003)

A comparison of the temperature change from the two sources are summarized in Table 2-4. IPCC 2014 predicted temperature increase from 2016 to 2035. These increments range from 1.0 to1.5°C, and from 2.0 to 3.0°C predicted for the June – August and December – February periods, respectively. As the IPCC 2014 output spans a shorter time period (from 2016 to 2035) compared to that required for this thermal analysis (from 2010 to 2039), a linear extrapolation was applied to the IPCC 2014 predication to estimate the temperature increase from 2010 to 2039. The estimated mean temperature increases are summarized in Table 2-4.

Table 2-4: Summary of Potential Mean Temperature Increase from 2010 to 2039

Period	IPCC 2003 - Temperature increase (°C)	IPCC 2014 RCP4.5 - Temperature increase (°C)
Dec – Feb	2.7	3.1 - 4.6
Mar – May	2.1	Not Reported
Jun – Aug	1.6	1.5 - 2.3
Sept – Nov	2.7	Not Reported

The mean value of the temperature increase ranges estimated from the 2014 IPCC report for the December – February and June – August periods were chosen for this study. Comparing the predicted mean temperature increases from 2003 IPCC and 2014 IPCC, an increase of 20% to 40% can be identified from 2003 to 2014 prediction. Since the temperature increase for March – May and September – November periods are not reported in 2014 IPCC, 30% increase were applied to the temperature increase predicted in 2003 IPCC and used for these two periods.

The predicted mean temperature increase rates are to be considered along with the base mean monthly air temperature function from 2010, to estimate the climate condition between 2010 and 2039. The temperature increments to be used in the Hatch thermal study are summarized in Table 2-5. It should be noted that these temperatures are potential temperature increases based on scientific studies.

Table 2-5: Recommended Mean Temperature Increase from 2010 to 2039 Used in Thermal Modelling

Period	Temperature increase (°C)
Dec – Feb	3.8
Mar – May	2.7
Jun – Aug	1.9
Sept – Nov	3.5

2.5 Initial Conditions

As the initial conditions will vary depending on the foundation type and the schedule for construction activities (excavation, backfilling, insulation placement, etc.), this input will be defined for each structure within the site specific design memo. Therefore, no set initial conditions are presented in this design basis document.

3. Strength and Deformation Parameters

The strength and deformation parameters in this section only applies for the foundation soil in the area of Bulk Material Handling System at the Milne Inlet site. presents the mechanical parameters for the materials including ore, rockfills, native soils of silt, sand in unfrozen state.

Two basic granular soil types were encountered during geotechnical site investigations for the Mary River Project. These were either (1) silt with trace to some sand and trace gravel, or (2) sand to sand and gravel with trace to some silt. Hereafter, these are referred to silty and sandy soils, respectively. Glacial till was also encountered.

In general, the silty permafrost has a higher ice content than the sandy soil. The investigations indicate that the silty permafrost is well bonded with trace of thin ice lenses and has a ice content typically ranging from 20% to 40%, on a ice-to-dry-soil weight basis. The sandy permafrost has a ice content typically ranging from 10% to 20%. The till permafrost is typically well graded and has a low ice content below 10%. Figure 3-1 summarizes the ice content measurements for the silty and sandy permafrost.

Table 3-1 and Table 3-2 provide the design parameters for the typical native fine-grained permafrost (i.e., silt) and native coarse-grained permafrost (i.e., sand).

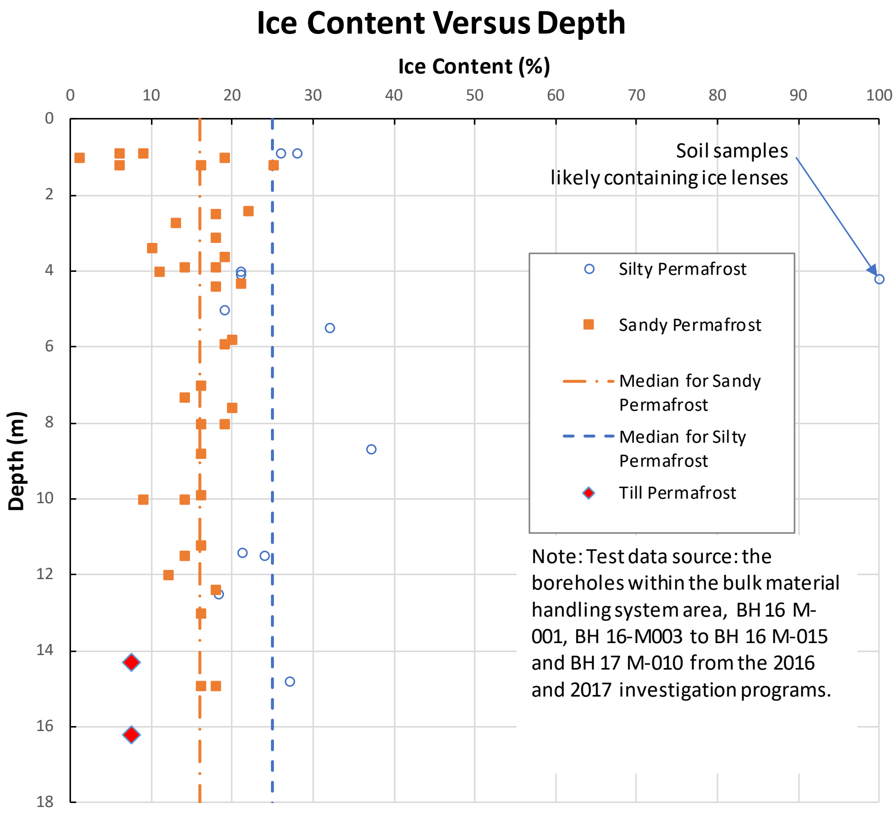


Figure 3-1: Ice Content in Permafrost with Depth

3.1 Foundation Bearing Capacity of Frozen Ground

The site is in a continuous permafrost zone with a mean annual air temperature of about -15°C. The bulk material handling facilities are not heated structures. The foundation design for the structures will attempt to control the temperature of the underlying native permafrost by using rockfill mat and/or insulation pad to control the thermal regime in the ground and to achieve the required bearing capacity.

For foundation design, the design loading pressure at permafrost should be based on the allowable long-term bearing capacity of 380 kPa for the temperature from -4°C to -7°C, and 560 kPa for the temperature of -7°C and below. These values are based on the ultimate resistance bearing capacity of frozen ground from SNiP (Russian Code, SNIP_2.02.04-88) with a factor of safety of 2.5.

The Unified Facilities Criteria (TM 5-852-4/AFM 88-19) reported that Hanover silt was tested to have an allowable bearing capacity of 550 kPa at -9.4°C, which is comparable with the values based on SNiP(1988). USSR (1969) recommends an allowable bearing capacity of about 720 kPa for the soil with ice contents from 20% to 40% at the temperature -7°C.

3.2 Creep Deformation

The creep deformation of permafrost is a highly complex process governed by the deformation of all of the permafrost components – i.e., the air voids, unfrozen water, ice and soil particles. For design, the long-term creep deformation was evaluated using equivalent long-term (i.e., creep adjusted) deformation modulus values reported by Tsytoich et al. (1973), which were developed based on oedometer creep compression tests (see Appendix A for details). The test conditions in the studies by Tsytoich et al. (1973) are considered applicable for the foundation soil at the site and the expected loading conditions for the structures. Summarizing:

- the oedometer tests properly account for the impact of confining pressure,
- the frozen soils in the tests had ice contents typically from 20% to 40%, and
- the loading pressure in the tests ranged from 0 to 800 kPa.

In addition, the estimated long-term deformation modulus is consistent with the deformation modulus at a very low strain rate inferred from a series of uniaxial compression tests at various strain-rates (the test data from Zhu and Carbee, 1984, as shown in Appendix A).

A Mohr-Coulomb envelope was established for design using the long-term strength parameters as recommended by Johnston (1981). For stresses below the long-term strength envelope, the creep deformation of frozen soil is expected to attenuate with time and is evaluated using the equivalent long-term deformation modulus values listed in Table 3-2 and Table 3-3.

For the silty permafrost at the site, Table 3-2 provides the design parameters for the long-term strength and deformation. For sandy permafrost at the site, Table 3-3 provides the design parameters for long-term strength and creep deformation. For the permafrost till containing less than 10% ice content, the creep deformation is considered negligible for design purposes, as the direct grain-to-grain contacts governs the deformation mechanism, similar to the unfrozen soil.

In order to verify the strength and deformation properties utilized, a case study was back-analyzed using the monitoring data from the Doris dam. The Doris dam is about 10m high and is founded on a 12m thick frozen saline marine silt and sand deposit (Miller and Rykaart, 2016). It is located in the cold permafrost area, similar to the Mary River Project site, with a mean annual air temperature of -12.1°C. The temperature in the Doris foundation below the core is below -8°C. Hatch's back- analysis of the Doris Dam settlement using the equivalent deformation modulus values listed in Table 3-2 and Table 3-3 gave calculated settlements that agreed with the monitored settlement (actual observations) of the Doris dam foundation (i.e., less than 10 mm after 4 year of construction). In comparison, the Doris Dam was designed using the Vyalov model, which predicted a much larger settlement of 300 mm after 4 years and 1200 mm of long-term settlement. For the Mary River Project, the equivalent deformation modulus approach is considered to be more realistic for the cold permafrost site than the power-law creep model (Vyalov 1959, Andersland and Ladanyi 2004).

It is noted that the creep adjusted strength and deformation parameters recommended for use on the Mary River Project were developed from creep test data reported in the literature. As such, site specific testing has not been done for the project due to the implementation schedule and the complex logistics involved with sampling the permafrost and testing it in the laboratory. As a result, an observation approach has been adopted.

Given the uncertainty associated with the design parameters, either in-situ tests such as plate load tests with temperature monitoring or temperature controlled triaxial creep tests should be conducted early during the construction phase of this project to verify the design assumptions.

Table 3-1: Design Parameters for Unfrozen Materials

Materials	Elastic Young's Modulus	Poisson's Ratio	Unit Weight	Strength Parameters	
	Es, (MPa)		kN/m³	c' (kPa)	ϕ' (Degrees)
Crushed Ore	30	0.33	26 Note 2	0	40°
TY 5 Rockfill (32 mm minus)	70 Note1	0.33	22	0	40°
TY 8 & TY 12 Rockfill (150 mm minus and run of mine)	70 Note1	0.33	20	0	40°
Pit-run Fill (compacted)	30	0.33	20	0	34°
Ballast	30	0.33	20	0	40°
Sub-Ballast	30	0.33	20	0	40°
Native Silt (unfrozen condition)	8	0.33	18	0	30°
Native Sand / Gravel (unfrozen condition)	15	0.33	18	0	32°

Notes:

1. Type 5 and 8 rockfills are crusher-run materials. Type 5, 8, and 12 should be placed and well compacted as per the construction specifications. The modulus of rockfill ranges from 40 MPa for poorly compacted state and 100 MPa for well compacted state. 70 MPa was selected as the design value.
2. Data from Primary Crushed Ore Data Sheet, H353004-00000-210-206-0001, Rev. 0, 2016.

Table 3-2: Design Parameters for Silty Permafrost (Typical Ice Content from 20% to 40%)

Temperature	Long-term Equivalent Deformation Modulus	Poisson's Ratio	Unit Weight	Strength Parameters For Creep Analyses (20-year design life)	
	Ec, (MPa)		kN/m³	c'_{LT} (kPa)	ϕ'_{LT}
- 0.4° C to - 7° C	22	0.33	18	0 Note 1	30°
Below -7° C	44	0.33	18	0 Note 1	30°

Notes:

1. A zero cohesion intercept (c'_{LT} =0 kPa) serves a lower bound for the creep strength envelope.
2. The long-term deformation modulus was based on the experiment test data published by Tsytoich et al. (1973) and Zhu and Carbee (1984), see Appendix A for details.
3. The parameters in this table are only applicable for the Silty Permafrost with the ice content typically ranging from 20% to 40%.

Table 3-3: Long-term Deformation Modulus for Sand Permafrost (Ice Content Less than 20%)

Temperature	Long-term Equivalent Deformation Modulus (Applicable for 10% to 10% ice content)	Poisson's Ratio	Unit Weight	Strength Parameters For Creep Analyses (20-year design life)	
	Ec, (MPa)		kN/m³	c'_{LT} (kPa)	ϕ'_{LT}
-0.4° C to - 7° C	80	0.33	18	0 Note 1	32°
Below -7° C	160	0.33	18	0 Note 1	32°

Notes:

1. See Table 2-1 for the note 1.
2. The long-term deformation modulus for permafrost sand is inferred from laboratory data by Tsytoich et al. (1973) and Zhu and Carbee (1984), see Appendix A for details. Design parameters other than listed in this table are same with those for Permafrost Silt listed in Table 3-2.

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Appendix A
Design Parameters – Long-Term Deformation Modulus

Mary River – Calculation Reference Sheet - 1
Long-term Deformation Modulus
Design Basis Value (Data By Tsytovich 1973)

HATCH

Dec. 21, 2017
By Greg Qu,
Reviewed by H. Warren

Oedometer Compression Test Data by Tsytovich 1973 and the calculated Deformation Modulus of Frozen Soils

Raw Test Data								Long-term Deformation Modulus				
Soil type	w [%]	T [°C]	mv, [cm^2/kgf x 10^-4]					E Mpa (0-100kPa)	E Mpa (100-200kPa)	E Mpa (200-400kPa)	E Mpa (400-600kPa)	E Mpa (600-800kPa)
			0 to 1	1 to 2	2 to 4	4 to 6	6 to 8					
Medium Sand	21	-0.6	12	9	6	4	3	55	74	110	165	221
	27	-4.2	17	13	10	7	5	39	51	66	95	132
	27	-0.4	32	26	14	8	5	21	25	47	83	132
Medium loam, silty, massive structure	35	-4	8	15	26	28	24	**83**	**44**	**25**	**24**	**28**
	32	-0.4	36	42	37	21	14	18	16	18	32	47
Medium loam, silty, reticulate structure	42	-3.8	5	10	18	42	32	132	66	37	16	21
	38	-0.4	56	59	39	24	16	12	11	17	28	41
Medium loam, silty, layered structure	104	-3.6	54	54	59	44	34	12	12	11	15	19
	92	-0.4	191	137	74	36	18	3	5	9	18	37
	Avg.	-2.0										

Summary of Modulus (Mpa) from Test Data		Frozen Sand	Frozen Silty Loam
Average	(As per tests @ average temperature of -2° C)	100	25
Standard Deviation	(As per tests @ average temperature of -2° C)	56	13
Average - SD*1/4	(As per tests @ average temperature of -2° C)	**86**	**22**

Design Parameters for Deformation Modulus (Mpa)

Deformation Modulus	Frozen Sand	Frozen Silt
Deformation Modulus (for permafrost above -7 °C)	80	22
Deformation Modulus (for permafrost below -7 °C)	160	44

Note: Poisson's Ratio 0.33

P:\REDLEAF\335458\SPECIALIST_APPS\02 Marry River\001 Report\01 Calculation Package\[Ref - Deformation Modulus - Coefficient_Compressibility v10.xlsx]Table Summary

Design Parameter Select and Basis

22 Mpa was selected for permafrost silt (ice/water content 20% to 40%, temperature @ -2C (average), applicable for -0.4C to -7C)

Design basis

- 22MPa is close to the value of (the average - 1/4* standard deviation), which, statistically, represents about 60% confident level , given the data scatter.

- This value is conservatively selected, to address (1) the uncertainty in the assumption that the tested soil by Tsytovich 1973 is representative to the permafrost silt at the site, and (2) data scatter.

-The data from 0 to 100 kPa was not used in the assessment due to the impact of initial sitting loading.

44 Mpa was selected for permafrost silt (ice content 20% to 40%, temperature -7C and below)

Design basis

- This value = 2 x 22 Mpa, as per a correlationship between E and temperature from the data by Zhu and Carbee, 1984, E(at -7C) = 2 x E (at -2C).

80 Mpa was selected for permafrost sand (ice content 10% to 20%, temperature @ -2C (applicable for -0.4C to -7C)

Design Basis - Data selected for frozen, based on the value of (the average - 1/4* standard deviation) in the table above.

160 Mpa was selected for permafrost sand (ice content 10% to 20%, temperature -7C and below)

Design basis

- This value = 2 x 80 Mpa, as per a correlationship between E and temperature from the data by Zhu and Carbee, 1984, E(at -7C) = 2 x E (at -2C), average

Additional Note:

- The lower bound of temperature applicable to the tested data was selected as -7C instead of -4.2 C considering possible high salinity at the site

Long-term Deformation Modulus

(Interpretation of the strain rate dependent test data by Zhu and Carbee 1984 for frozen silt with 30% to 50% ice content by weight)

Dec. 21, 2017
By Greg Qu,
Reviewed by H. Warren

The test data by Zhu and Carbee (1984) supports the design parameters developed from the test data by Tsytovich 1973 and suggests that the parameters were reasonably conservative.

Notes
- In general, deformation E (50% modulus) is temperature-dependent. From the Figure below, it is estimated that E(at -7C) is about two times of E (at -2 C).
- The data from this study was based on unconfined tests. They are comparable with the long-term deformation modulus from the test data by Tsytovich 1973

From the Figure, the inferred long-term deformation modulus (using a long-term strain rate, 1E-13 /s, which corresponds to 1 mm settlement for a 15 m deposit for 20 years) is about 33 MPa for -2 C, which is comparable with the design value (22 Mpa) selected as per test data by Tsytovich 1973.

Long-term Deformation Modulus (Mpa) Inferred From Literature Test Data

Deformation Modulus	Frozen Silt	Silt Permafrost
	(inferred from the test data by Zhu and Carbee 1984)	Design Parameter
Deformation Modulus @ -2° C	33	22
Deformation Modulus @ -7° C	70	44