# Attachment 8.4

## North Railway Bridges Geotechnical Recommendations

(23 Pages)

# Baffinland Iron Mines Corporation
# Mary River Expansion Project

# Foundation Recommendations for Rail Bridges

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# 1. Introduction and Background

This report provides geotechnical recommendations for the design of deep foundations to support four separate railway bridges, associated with the development of a 110 km long rail line on Baffin Island. The rail line is developed by Baffinland Iron Mines Corp. (BIM) to facilitate the increased production from the Mary River Iron Mine.

It is understood from preliminary designs that the rail bridges will consist of multi-span, steel through plate girders supported on multiple deep foundations, specifically prefabricated steel pipe adfreeze piles, at the bridge abutment locations. All bridges are crossing existing waterways and a separate hydrotechnical scour analysis is being prepared by Hatch at all bridge sites.

# 2. Available Subsurface Information

In 2018, a geotechnical investigation was conducted by Hatch on behalf of BIM in support of the rail line development. Table 2-1 outlines the available boreholes relevant to the bridge locations.

**Table 2-1: Summary of Available Boreholes**

| Bridge | Available BHs | Ground Surface Elevation (m) | Depth (m) |
|---|---|---|---|
| No. 15-1 | BH18-BR15-1 | 78.0 | 17.4 |
| | BH18-BR15-2 | 78.0 | 16.8 |
| No. 70-2 | BH18-BR70-1 | 124.0 | 32.5 |
| | BH18-BR70-2 | 124.0 | 28.5 |
| No. 86-3 | BH18-BR86-1 | 142.9 | 39.6 |
| | BH18-BR86-2 | 143.0 | 39.6 |
| | BH18-BR86-3 | 143.4 | 39.6 |
| No. 102-4 | BH18-BR102-1 | 168.0 | 9.7 |
| | BH18-BR102-1 | 166.0 | 16.9 |

# 3. Foundation Recommendations

## 3.1 Rock-Socketed Piles

Rock-socketed steel pipe piles are the preferred foundation type, where competent bedrock can be practically and economically reached. These foundations have a higher axial capacity and are not typically susceptible to long-term settlement, as compared to adfreeze piles.

Rock-socketed steel pipe piles are constructed by predrilling installation holes to the specified depth and about 100 mm wider than the diameter of the pile. Prior to inserting the steel pipe piles, the holes and hole bottoms should be free of loose soil, rock or ice. Within the rock-socket zone the steel pipes are perforated with slot holes. Grout is poured through the centre of the piles and flows through the slots to fill the annulus space in the rock socket. The grout should be placed to a height of at least 2.0 m above the design bedrock elevation. After grouting, the pile should be lightly vibrated to consolidate the grout and set the surface contact with the pile and bedrock. The remaining annulus space from the grout to the active zone should be backfilled with sand-slurry, and within the active zone with dry sand or approved drill cuttings, as outlined in Section 3.2.

The capacity of rock-socketed steel pipe piles is typically governed by the bond strength at the steel-grout and rock-grout interfaces, i.e., shaft resistance. End bearing resistance should not be relied upon, unless the following conditions are met 1) the base of the pile socket is visually inspected (i.e., CCTV camera) and confirmed to be free of water, mud, ice, or loose material, and 2) the piles are seated in placed before grouting using several strikes from a pile driving hammer. End bearing has not been considered in these recommendations. For a 610 mm pile, and in accordance with the Canadian Foundation Engineering Manual, 4$^{th}$ Ed., Chapter 18, the elastic settlement of the shear socket is estimated to be less than 1 mm per 1000 kN of applied axial load, which is considered acceptable for this project. Significant long-term settlement is not anticipated.

The grout used in the rock-socket should have a minimum compressive strength of 25 MPa. The grout should be specifically designed for arctic applications, with a high early strength and capable of properly curing when placed on a substrate with a minimum temperature of -10 °C.

Prior to installation, the surface of the steel piles within the socket zone should be clean and free of grease, paint, varnish, or similar coatings. However, the upper portion of the piles within the active soil zone should be treated to prevent shear stress development on the piles due to frost heave uplift and thawing-induced down-drag forces. The recommended approach is to wrap the piles with at least three layers of polyethylene sheets. In areas where significant scour is anticipated, active zone treatment may not be practical.

## 3.2 Adfreeze Piles

Adfreeze piles, or slurry piles, installed in permafrost may be considered where the use of rock-socketed piles is not practical (e.g., very deep bedrock surface).

Adfreeze piles are constructed by first drilling the installation hole to the specified depth. The steel pile is inserted and temporarily braced while the annulus space is backfilled with a water-sand slurry. The slurry is consolidated in place by either inserting a manual ‘pencil’ vibrator or vibrating the pile. The bracing is left in place while the slurry freezes. The diameter of the drilled installation holes should be 100 mm to 200 mm larger than the pile diameter. In general, larger installation holes may be required for deeper installations. The sand used in the slurry should be clean and well graded with a maximum particle size of 2.0 mm and no more than 5% (by weight) of particles passing 75μm. The water should be clean, fresh water. At the time of placement, the temperature of the slurry should be between 5 °C and 10 °C. If the slurry is too cold, it may prematurely freeze during placement; if too warm, the heat from the slurry may cause excessive disturbance to the permafrost. The slurry in the annulus should extend from the pile toe to a depth of 2.0m below the final grade. The upper 2.0m annulus space should be backfilled with dry sand or approved drilling cuttings. At a minimum, the bottom 1.0 m of the inside of the pile should be filled with slurry, the remaining space inside the piles may be left empty or backfill with soil cuttings. Depending on the location and time of year, a temporary steel casing may be required in areas of thawed soil to prevent the ingress of water into the installation hole.

The axial capacity of adfreeze piles is governed by the interface bond between steel and the frozen slurry. This bond is susceptible to both short-shear failure as well as continuous long-term creep deformation under sustained loads. The appropriate load combinations used for short-term and long-term conditions should be selected by the structural engineer. The allowable long-term pile settlement should be selected based on project requirements.

Prior to installation, the surface of the steel piles within the slurry filled adfreeze zone should be clean and free of grease, paint, varnish, or similar coatings. Although not considered in this design, higher short-term capacities could be obtained by sandblasting the outside of the pile or installing steel ring flanges around the perimeter at regular intervals.

Similar to the rock-socketed piles, the pile surface within the active soil zone should be sufficiently treated to prevent frost heave or down-drag forces.

## 3.3 Scour Protection

Adequate scour protection should be provided where ever significant scour is anticipated, in accordance with AREMA standards. Please refer to the associated Hydraulic Report for details regarding the scour assessment and recommendations for scour protection (H353004-35000-220-230-0002).

## 3.4 Thermal Analysis

Site specific, 1-D thermal analysis has been performed and presented in Appendix A. The objective of the thermal analysis is to define the thickness of the active zone and the maximum temperature of the permafrost. It is noted that heat transport due to advection in the overlying water bodies, which is expected to increase the active zone thickness, has not been accounted for in the thermal analysis. The recommended design thickness for the active zone has been increased to accounted for advective heat transport, based on available temperature data and site experience.

## 3.5 Lateral Resistance

Lateral pile analyses were performed using the software GROUP (Version 2016.10.13, by Ensoft) to estimate the maximum displacements of the pile caps and the maximum bending moment, shear force, and axial force within the individual piles. Pile group configurations and loading conditions were provided by the structural designer. All structural loads were unfactored in the analysis. The structural capacity of the pile sections must be confirmed by the structural designer.

Pile sections were modeled as short-column, linear elastic elements and Table 3-1 summarizes the section parameters considered for all locations. At the pile-cap connection, pinned conditions were considered at all pier locations, whereas fixed conditions about the lateral axis were considered at the abutment locations. The pile cap was considered as a rigid body.

**Table 3-1: Steel Pipe Pile Section Details for the Lateral Analysis.**

| Parameter | Value |
|---|---|
| Outside Diameter | 610 mm |
| Wall Thickness | 19 mm |
| Elastic Modulus | 200 GPa |

A total of 12 cases were analyzed and the results are summarized below in Table 3-2. presents the primary input parameters and detailed output results for each case. The elevation of the base of the pile caps was determined from the bridge design drawings. Ground surface elevations for piers (Regular Bents and Super Bents) follows the post scour ground elevations recommended in the Hydraulic Report, assuming worst-case conditions. At the abutments, a design ground elevation of 1.0 m below the pile cap base was selected.

It is noted that Case 4.1 involves a large distance between the post scour ground surface and the base of the pile cap (11.2m). No structural bracing was considered between piles above the ground surface. This ground clearance is considered excessive for the modelling methodology used and special caution should be used by the structural designer for this case.

**Table 3-2: Summary of Lateral Analysis Results**

| Case | Bridge | Pile Group Type | Loading Type[1] | Max Pile Cap Displacement (mm) | | | Individual Pile Structural Loads | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | Longitudinal | Lateral | Vertical | Max Bending Moment (kN-m) | Max Shear Force (kN) | Max Axial Force (kN) |
| 1.1 | 15-1 | Regular Bent | Regular | 11.7 | 2.9 | 0.8 | 212.9 | 279.3 | 841.5 |
| 1.2 | 15-1 | Regular Bent | Elevated | 15.6 | 2.9 | 0.8 | 277.8 | 322.7 | 859.6 |
| 1.3 | 15-1 | Super Bent | Regular | 12.7 | 2.9 | 0.7 | 194.5 | 268.1 | 797.6 |
| 1.4 | 15-1 | Abutment | Regular | 5.8 | 0.3 | 0.1 | 211.5 | 252.4 | 1150.1 |
| 2.1 | 70-2 | Regular Bent | Regular | 20.8 | 4.4 | 1.0 | 255.5 | 230.7 | 867.9 |
| 2.2 | 70-2 | Regular Bent | Elevated | 27.8 | 4.4 | 1.0 | 334.2 | 299.7 | 886.9 |
| 2.3 | 70-2 | Super Bent | Regular | 18.8 | 3.9 | 0.8 | 221.8 | 207.9 | 807.9 |
| 2.4 | 70-2 | Abutment | Regular | 6.6 | 0.3 | 0.3 | 242.8 | 192.4 | 941.3 |
| 3.1 | 86-3 | Regular Bent | Regular | 33.0 | 4.2 | 1.0 | 427.1 | 405.4 | 906.8 |
| 3.2 | 86-3 | Abutment | Regular | 7.2 | 0.2 | 0.3 | 251.8 | 218.7 | 1011.6 |
| ***4.1[2]*** | ***102-4*** | ***Regular Bent*** | ***Regular*** | ***147.3*** | ***8.0*** | ***1.2*** | ***788.5*** | ***1168.9*** | ***1005.2*** |
| 4.4 | 102-4 | Abutment | Regular | 6.6 | 0.3 | 0.1 | 250.9 | 227.4 | 1211.6 |

Notes:

[1] Loading type refers to the design longitudinal load.

[2] Clearance between the ground surface and the bottom of the pile cap is considered excessive for the analysis method used, values should be considered with caution.

### 3.6 Bridge Abutments/Backwalls

It is understood that the bridge approaches will be supported laterally at the abutments using precast concrete backwalls. The surface of the backwall should be treated with an adfreeze bond-breaker consisting of a thin layer of cold-temperature grease followed by at least three (3) layers of 6 mils polyethene sheeting. To minimize seasonal frost heave forces, the underside of the abutment and the soil-facing side of the backwall should be lined with a 150mm thick polyethylene void form. The void form will provided a secondary function as insulation. The abutments should be backfilled with free-draining, crushed granular fill, such as Type 25 Fill. The lateral earth pressure acting on the backwall may be determined using a triangular earth pressure profile, with a lateral earth pressure coefficient of $K_a$ = 0.24.

## 4. Site Specific Recommendations

### 4.1 Bridge No. 15-1

**Table 4-1: Summary of Foundation Design Recommendations for Bridge No. 15-1**

| | |
|---|---|
| Overburden Soils | Silty Sand to Sandy Silt<br>Occasional Cobbles and Boulders<br>Ice-poor to Ice-rich |
| Bedrock | Gneiss |
| Design Top of Permafrost Elevation | Piers: 72.8 m<br>Abutments: 75.0 m |
| Design Top of Bedrock Elevation | 66.5 m |
| Depth of Frost Treatment | 2.5 m |

Rock-socketed steel pipe piles are recommended for Bridge No. 15-1. The piles may be designed with an allowable shaft capacity of 500 kPa at the pile-grout interface, for the length of socket installed below the design top of bedrock elevation. The socket length should be a minimum of 2.0 m and a maximum of 6.0 m in length. The structural capacity of the piles must be confirmed by the structural engineer.

In accordance with Table 9-1-6 of the 2018 AREMA Manual, Bridge No.15-1 should be considered Site Class B for seismic design.

## 4.2 Bridge No. 70-2

**Table 4-2: Summary of Foundation Design Recommendations for Bridge No. 70-2**

| | |
|---|---|
| Overburden Soils | Silt to Sandy Silt to Silty Sand to Sand, Occasional Cobbles and Boulders, Ice-poor to Ice-rich |
| Bedrock | Siltstone to Dolomitic Limestone |
| Design Top of Permafrost Elevation | Piers: 118.9 m<br>Abutments: 122.3 m |
| Design Top of Bedrock Elevation | 100.5 m |
| Depth of Frost Treatment | 2.5 m |

Bridge No. 70-2 may be supported on adfreeze piles installed into the overburden soils. The adfreeze piles may be designed with a factored short-term allowable shaft capacity of 150 kPa at the pile-slurry interface for the portion installed below the design active zone, or a minimum of 2.5 m below the final ground surface. The long-term shaft capacity is dependent on the permitted pile settlement, as outlined in Table 4-3. The adfreeze piles shall have a minimum embedment length of 7.0 m and a maximum embedment of 25.0 m below the design top of permafrost elevation. The structural capacity of the piles must be confirmed by the structural engineer.

**Table 4-3: Allowable Long-term Adfreeze Shaft Capacities for Bridge No. 70-2**

| | | | |
|---|---|---|---|
| Allowable Long-Term Settlement | 50 mm / 25 years | 30 mm / 25 years | 15 mm / 25 years |
| Allowable Long-Term Shaft Stress (unfactored) | 51 kPa | 42 kPa | 31 kPa |

As an alternative, rock-socketed steel pipe piles may be considered for Bridge No. 70-2. The piles may be designed with an allowable shaft capacity of 500 kPa at the pile-grout interface, for the length of socket installed below the design top of bedrock elevation. The socket length should be a minimum of 2.0 m and a maximum of 6.0 m in length. The structural capacity of the piles must be confirmed by the structural engineer.

In accordance with Table 9-1-6 of the 2018 AREMA Manual, Bridge No.70-2 should be considered Site Class C for seismic design.

## 4.3 Bridge No. 86-3

**Table 4-4: Summary of Foundation Design Recommendations for Bridge No. 86-3**

| | |
|---|---|
| Overburden Soils | Sand,<br>Ice-poor to Ice-rich |
| Bedrock | Not Available |
| Design Top of Permafrost Elevation | Piers: 138.2 m<br>Abutments: 141.4 m |
| Design Top of Bedrock Elevation | Not Available (below 103.3m) |
| Depth of Frost Treatment | 2.5 m |

Bridge No. 86-3 may be founded on adfreeze pile installed in the underlying permafrost soils. The adfreeze piles may be designed with a factored short-term allowable shaft capacity of 150 kPa at the pile-slurry interface for the portion installed below the design active zone, or a minimum of 2.5 m below the final ground surface. The long-term shaft capacity is dependent on the permitted pile settlement, as outlined in Table 4-5. The adfreeze piles shall have a minimum embedment length of 7.0 m and a maximum embedment of 25.0 m below the design top of permafrost elevation. The structural capacity of the piles must be confirmed by the structural engineer.

**Table 4-5: Allowable Long-term Adfreeze Shaft Capacities for Bridge No. 86-3**

| | | | |
|---|---|---|---|
| Allowable Long-Term Settlement | 50 mm /<br>25 years | 30 mm /<br>25 years | 15 mm /<br>25 years |
| Allowable Long-Term Shaft Stress<br>(unfactored) | 55 kPa | 45 kPa | 33 kPa |

In accordance with Table 9-1-6 of the 2018 AREMA Manual, Bridge No.86 should be considered Site Class C for seismic design.

## 4.4 Bridge No. 102-4

**Table 4-6: Summary of Foundation Design Recommendations for Bridge No. 102-4**

| | |
|---|---|
| Overburden Soils | Gravelly Sand to Sand and Gravel<br>Occasional Cobbles and Boulders,<br>Ice-poor to Ice-rich |
| Bedrock | Gneiss |
| Design Top of Permafrost Elevation | Piers: 156.7 m<br>Abutments: 164.9 m |
| Design Top of Bedrock Elevation | 156.5 m |
| Depth of Frost Treatment | 2.5 m |

Rock-socketed steel pipe piles are recommended for Bridge No. 102-4. The piles may be designed with an allowable shaft capacity of 500 kPa at the pile-grout interface, for the length of socket installed below the design top of bedrock elevation. The socket length should be a minimum of 2.0 m and a maximum of 6.0 m in length. The structural capacity of the piles must be confirmed by the structural engineer.

In accordance with Table 9-1-6 of the 2018 AREMA Manual, Bridge No. 102-4 should be considered Site Class B for seismic design.

# 5. References


- AREMA Manual, 2018
- H353004-10000-229-230-0005, Rev. 2, 2016-2017-2018 Rail Geotechnical Investigation Factual Data Report.
- H353004-35000-220-230-0002, Rev. 0, Rail Bridge Design Hydraulic Report
- H353004-00000-229-210-0001, Rev. 0, Geotechnical Design Basis.
- EBA 2008, Memorandum: Mary River – Pile Foundations Revision C, File: E14101009.001
- Andersland, B.A., Ladanyi, B., 2004, "Frozen Ground Engineering", Second Edition, ASCE, John Wiley and Sons, Inc.
- Nixon, J.F., 1978, “Foundation Design Approaches in Permafrost Area”, Canadian Geotechnical Journal, 1978, Vol.15, pp.96-112.
- Weaver, J.S, Morgenstern, N.R. “Pile Design in Permafrost”, Canadian Geotechnical Journal, 1981, Vol. 18, 357-370.

- Canadian Foundation Engineering Manual, 4th Ed.

# Appendix A
# Ground Temperature Modelling

## A.1 Approach

The ground temperature profile was modelled using the Finite Element Method (Temp/W, (GeoStudio 2016, version 8.16) for each bridge site based on site-specific subsurface conditions. The primary objective of this analysis is to support the foundation design of the four rail bridges, which are underlain by permafrost. The variation in the seasonal fluctuation ground temperatures with depth has been estimated, including the thickness of the active zone, for each bridge site.

### *A.1.1 Soil Model*

All subsurface layers (soil and bedrock) have been modelled using the Simplified Thermal Model, which considers two physical states: frozen (≤ 0°C) and unfrozen (> 0 °C) each with unique thermal properties. The model accounts for the heat released/absorbed due to the latent heat of fusion of the pore water. Instantaneous freezing is considered to occur at 0 °C. This soil approach is considered acceptable for modelling granular soils and rock.

The model only considers heat transport in the in the soils through conduction only.

### *A.1.2 Boundary Conditions and Initial Conditions*

The upper thermal boundary was considered as a defined temperature condition (i.e. prescribed ground surface temperature). The mean monthly air temperature from Pond Inlet, NU (as published by the Government of Canada) has been used as the basis of the site temperatures. As discussed in the Geotechnical Design Basis report, it is recommended that the considered mean daily air temperature be increased to account for the anticipated effects of global over the design life of the railway bridges. For this analysis, the mean air temperature has been increased by 50% of this recommended temperature increase between 2010 and 2039, as outlined in Table A-1. The ground surface temperature was equated to the mean daily air temperature using a freezing factor ($n_f$) and thawing factor ($n_t$) where appropriate. These parameters are intended to account for factors such as snow cover and radiated heat transport and are typically a function of the ground surface conditions. For the purposes of this analysis, the following factors were conservatively selected for all four bridge sites: $n_f$ = 0.9 and $n_t$ = 2.0 (Andersland and Ladanyi, 2004).

The lower thermal boundary (depth of 25.0m) was modelled as a unit-flux boundary with a constant value of 0.06 J/s/m$^2$. The sides boundaries of the model were considered as zero-flux, leading to 1D conditions for this analysis.

The initial temperature of the model was a uniform value of -10 °C, with a ground surface temperature corresponding to January. The upper and lower boundary conditions were applied for a total of 10 years (cycling between years) and the final year (January to December) was used to determine the minimum and maximum temperature profiles for each site. After 10 years, the temperature at the lower boundary condition varied between -10.0 °C and -10.5 °C across all sites.

**Table A-1: Considered Mean Air Temperature**

| Month | Mean Daily Average Temperature,<br>Pond Inlet NU (1981 – 2010)<br>(°C) | Recommended Mean Temperature Increase from 2010 to 2039<br>[Geotechnical Design Basis]<br>(°C) | Considered Mean Daily Air Temperature,<br>[100% Mean Daily Average Temperature +<br>50% Mean Temperature Increase]<br>(°C) |
|---|---|---|---|
| January | -33.4 | 3.8 | -31.5 |
| February | -33.7 | 3.8 | -31.8 |
| March | -30.0 | 2.7 | -28.7 |
| April | -21.9 | 2.7 | -20.6 |
| May | -9.3 | 2.7 | -8.0 |
| June | 2.4 | 1.9 | 3.4 |
| July | 6.6 | 1.9 | 7.6 |
| August | 4.8 | 1.9 | 5.8 |
| September | -0.8 | 3.5 | 1.0 |
| October | -9.7 | 3.5 | -8.0 |
| November | -21.7 | 3.5 | -20.0 |
| December | -28.2 | 3.8 | -26.3 |

## A.2 Site Specific Temperature Profiles

Site specific ground temperature profiles were determined for each bridge site based on the observed subsurface conditions, as outlined in the Rail Geotechnical Investigation Factual Data Report. Thermal properties were interpreted based on the reported information, the Geotechnical Baseline Report, and published values (Andersland and Ladanyi, 2004; GEO-SLOPE), where appropriate.

### *A.2.1 Bridge No. 15-1*

**Table A-2: Temp/W Model Input Parameters, Bridge No. 15-1**

| Stratigraphic Unit | Silty Sand | Gneiss (Bedrock) |
| --- | --- | --- |
| Depth Below Ground Surface | 0.0 m – 10.7 m | 10.7 m – 25.0 m |
| Frozen Thermal Conductivity | 2.8 J/s/m/°C | 2.5 J/s/m/°C |
| Unfrozen Thermal Conductivity | 1.8 J/s/m/°C | 2.5 J/s/m/°C |
| Frozen Volumetric Heat Capacity | 2.6e6 J/m$^3$/°C | 2.7e6 J/m$^3$/°C |
| Unfrozen Volumetric Heat Capacity | 1.9e6 J/m$^3$/°C | 2.7e6 J/m$^3$/°C |
| Volumetric Water Content | 0.41 m$^3$/m$^3$ | 0.03 m$^3$/m$^3$ |

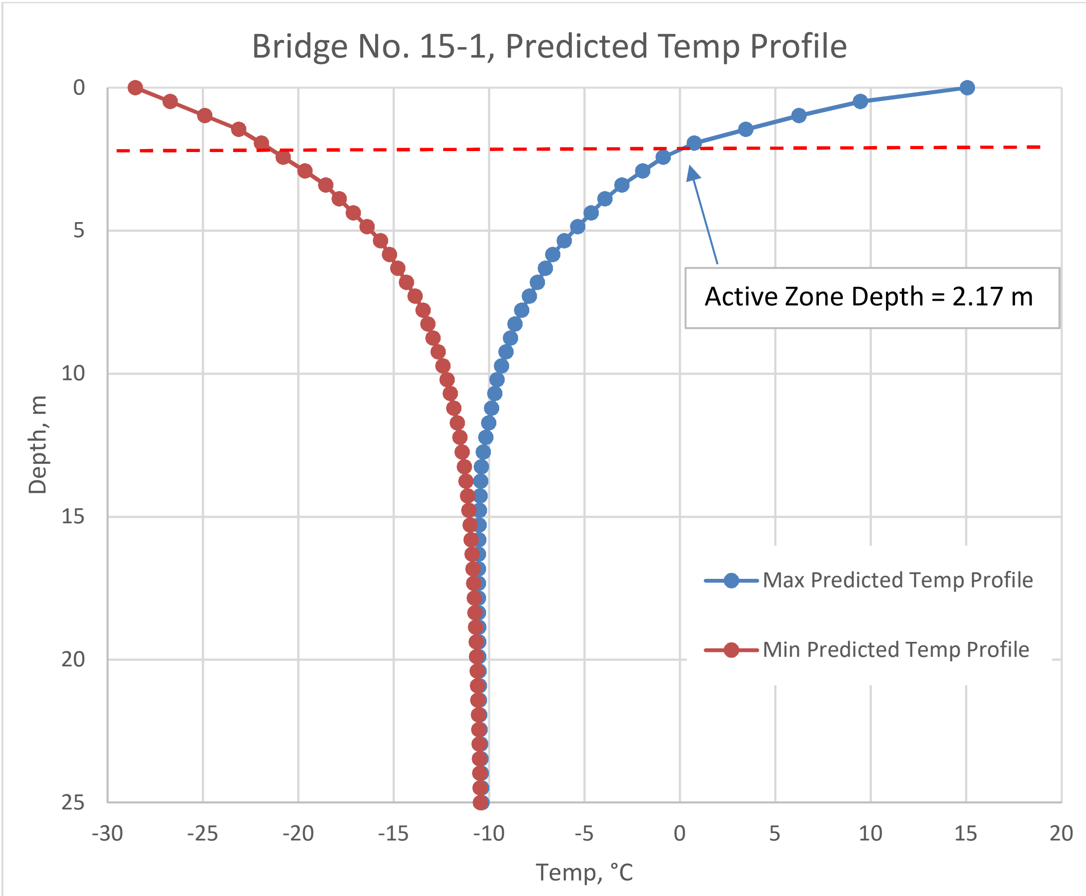


**Figure A-1: Predicted Ground Temperature Profile at Bridge No. 15-1**

### *A.2.2 Bridge No. 70-2*

**Table A-3: Temp/W Model Input Parameters, Bridge No. 70-2**

| Stratigraphic Unit | Sandy Silt |
|---|---|
| Depth Below Ground Surface | 0.0 m – 25.0 m |
| Frozen Thermal Conductivity | 2.2 J/s/m/°C |
| Unfrozen Thermal Conductivity | 1.4 J/s/m/°C |
| Frozen Volumetric Heat Capacity | 2.5e6 J/m³/°C |
| Unfrozen Volumetric Heat Capacity | 1.9e6 J/m³/°C |
| Volumetric Water Content | 0.41 m³/m³ |

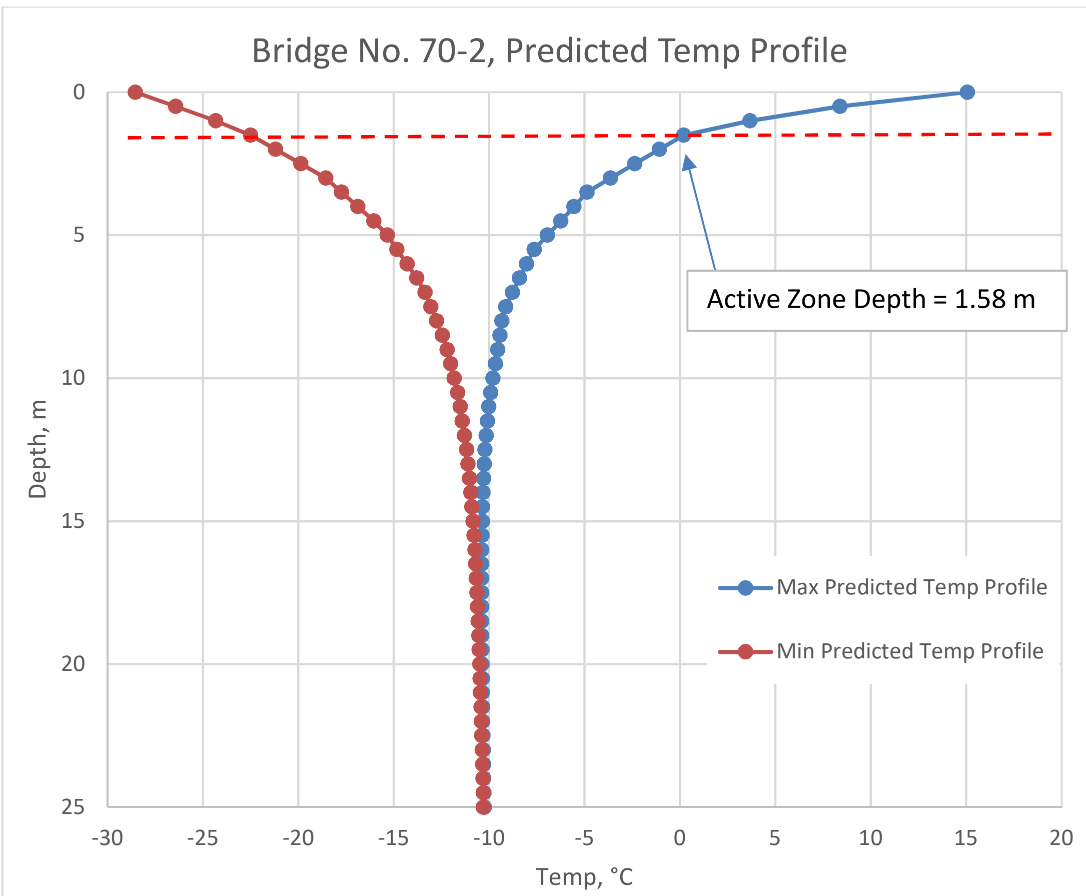


**Figure A-2: Predicted Ground Temperature Profile at Bridge No. 86-3**

### *A.2.3 Bridge No. 86-3*

**Table A-4: Temp/W Model Input Parameters, Bridge No. 86-3**

| Stratigraphic Unit | Sand to Sand and Gravel |
|---|---|
| Depth Below Ground Surface | 0.0 m – 25.0 m |
| Frozen Thermal Conductivity | 3.0 J/s/m/°C |
| Unfrozen Thermal Conductivity | 1.5 J/s/m/°C |
| Frozen Volumetric Heat Capacity | 2.8e6 J/m³/°C |
| Unfrozen Volumetric Heat Capacity | 1.9e6 J/m³/°C |
| Volumetric Water Content | 0.45 m³/m³ |

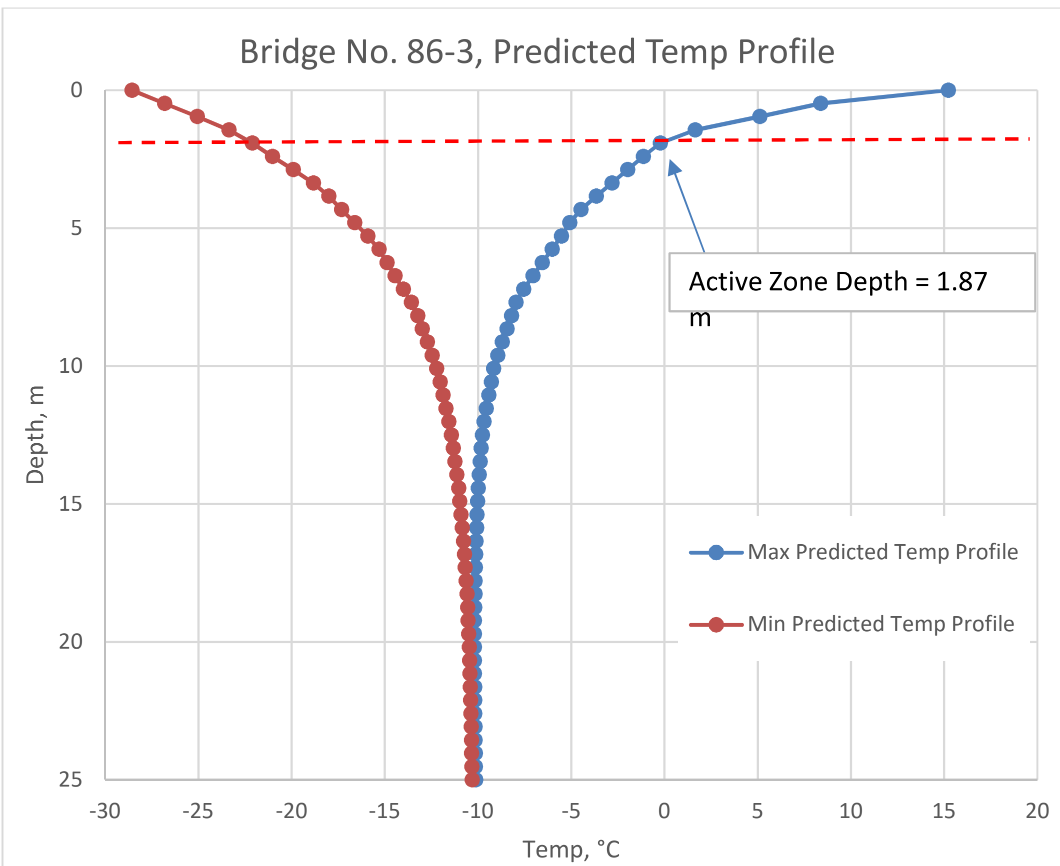


**Figure A-3: Predicted Ground Temperature Profile at Bridge No. 86-3**

### *A.2.4 Bridge No. 102-4*

**Table A-5: Temp/W Model Input Parameters, Bridge No. 102-4.**

| Stratigraphic Unit | Sand to Sand and Gravel | Gneiss (Bedrock) |
|---|---|---|
| Depth Below Ground Surface | 0.0 m – 8.5 m | 8.5 m – 25.0 m |
| Frozen Thermal Conductivity | 3.0 J/s/m/°C | 2.5 J/s/m/°C |
| Unfrozen Thermal Conductivity | 1.5 J/s/m/°C | 2.5 J/s/m/°C |
| Frozen Volumetric Heat Capacity | 2.8e6 J/m³/°C | 2.7e6 J/m³/°C |
| Unfrozen Volumetric Heat Capacity | 1.9e6 J/m³/°C | 2.7e6 J/m³/°C |
| Volumetric Water Content | 0.45 m³/m³ | 0.03 m³/m³ |

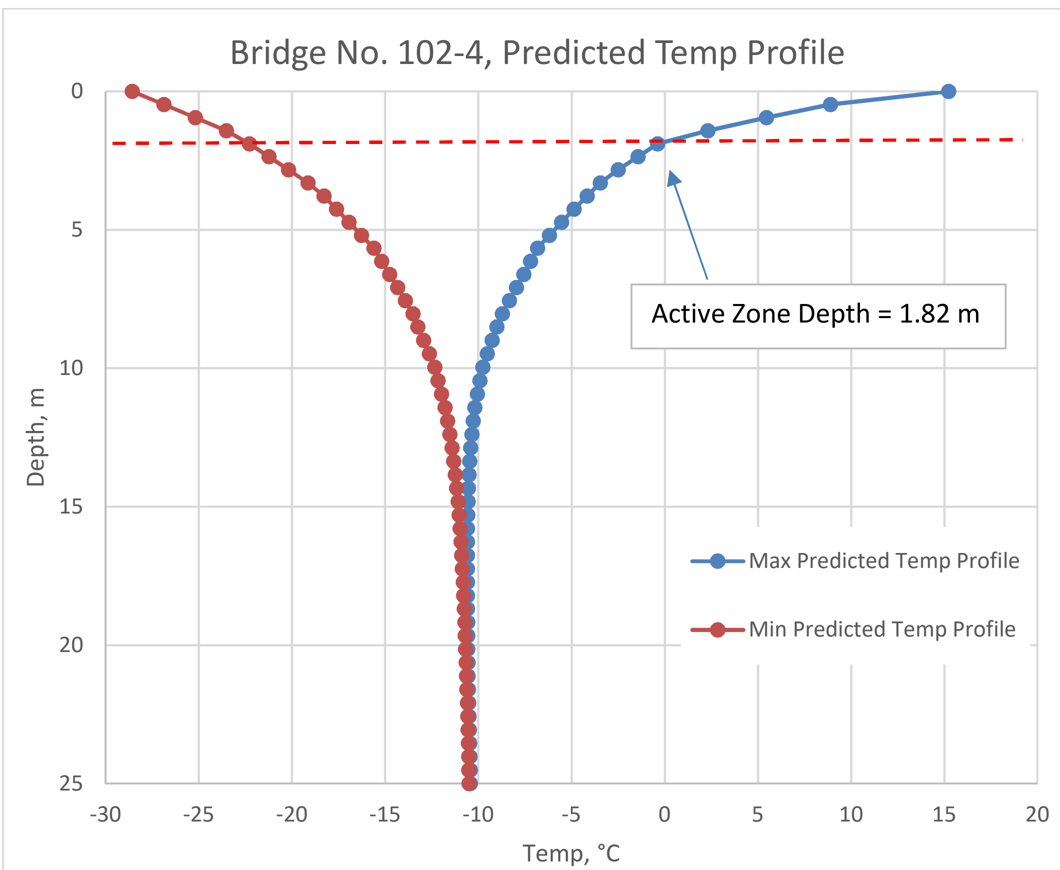


**Figure A-4: Predicted Ground Temperature Profile at Bridge No. 102-4**

## A.3 References


- Andersland, B.A., Ladanyi, B., 2004, "Frozen Ground Engineering", Second Edition, ASCE, John Wiley and Sons, Inc.
- H353004-10000-229-230-0005, Rev. 2, 2016-2017-2018 Rail Geotechnical Investigation Factual Data Report.
- H353004-00000-229-210-0001, Rev. 0, Geotechnical Design Basis.
- GEO-SLOPE International Inc., 2014, “Thermal Modelling with TEMP/W – An Engineering Methodology”.

# Appendix B
# Lateral Pile Analysis

# Appendix C
# Reference Borehole Logs