ATTACHMENT 27

LTWP Geotechnical Report

City of Iqaluit

Preliminary Geotechnical Investigation Report – DRAFT

Long Term Water Program – Supply and Storage

Iqaluit, Nunavut

August 2025

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Long Term Water Program – Supply and Storage

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Acronyms and Abbreviations

Arcadis	Arcadis Canada Inc.
ABA	Acid Base Accounting
ABG	Adaptive Baseline Geotechnical Limited
ALS	ALS Canada Limited
ARD	Acid Rock Drainage
Canadrill	Canadrill Limited
CIRNAC	Crown–Indigenous Relations and Northern Affairs Canada
City	City of Iqaluit
cm	centimetres
CSA	Canadian Standards Association
DFO	Fisheries and Oceans Canada
DS	Downhole Seismic
ft	foot
GBR	Geotechnical Baseline Report
g/cm^3	grams per cubic centimeter
GPa	Gigapascal
GPR	Geophysics GPR International
GPS	Global Positioning System
ha	hectares
HASP	Health and Safety Plan
LG	Lake Geraldine
LQ	Lake Qikiqtalik
LTWP	Long Term Water Program
masl	metres above sea level
mbgs	metres below ground surface
m	metre
mm	millimetres
MOE	Modulus of Elasticity
MPa	megapascal
NPR	neutralization potential ratio
OMC	optimum moisture content
PDR	Preliminary Design Report
PNJ	PNJ Engineering Inc.
RQD	Rock Quality Designation
RtN	Road to Nowhere
SPMDD	Standard Proctor Maximum Dry Density
UCS	Uniaxial Compressive Strength

Executive Summary

Arcadis Canada Inc. (Arcadis) has prepared this draft preliminary geotechnical investigation report for the City of Iqaluit (the City) to provide a summary of the geotechnical results obtained as of August 2025 for the Long-Term Water Program – Supply and Storage project. The geotechnical investigations outlined in this report were completed to inform the design and the potential construction of a raw water reservoir and conveyance system.

The geotechnical investigations were conducted on a parcel of untitled City land located northeast of the surveyed portion of the community in Iqaluit, Nunavut (the Site). The Site is approximately 1,150 hectares in size and is mostly undeveloped. Its terrain comprises rolling hills with rocky outcrops and tundra valleys, with finger lakes oriented northwest-southeast. The Niaqunnguk (Apex) River traverses the Site from its northwest boundary to its southern corner. The Site resides in a zone of continuous permafrost.

The geotechnical investigation work conducted at the Site as of July 2025 has consisted of the following:

- Drilling of 37 boreholes into bedrock using a combination of air-rotary and coring methods in the proposed reservoir and access road areas of the Site;
- Drilling of 5 boreholes into bedrock using an air-rotary drill method at the proposed pump station and pipe crossing at the Apex River;
- The logging of the subsurface conditions encountered at each borehole by field observations and downhole camera assessment;
- Installation of 12 piezometers and 10 thermistors in boreholes onsite;
- Survey of all boreholes and piezometer locations to obtain UTM coordinates and elevation data;
- Grain size analysis of soil fractions encountered at boreholes within native overburden cobble fields;
- Acid base accounting and shake flask testing on four bedrock samples;
- Groundwater level measurements and flow direction interpretation;
- Bedrock permeability testing (slug tests in existing piezometers);
- Groundwater sample collection and chemistry analyses;
- Rock Quality Designation (RQD) assessment based on extracted rock cores;
- Rock strength characterization by laboratory testing; and,
- Geophysical testing to obtain site-specific seismic response values to inform structure design.

Air-rotary drilling and piezometer installations were conducted at in the proposed reservoir area between 6 February and 25 March 2024 under the supervision of Adaptive Baseline Engineering. Additional air-rotary drilling was conducted in May 2025 at the proposed pump station and pipeline crossing area along the Apex River while the river was frozen to full depth. Boreholes were advanced to depths ranging from 4.0 to 13.7 metres below ground surface (mbgs).

Native ‘soil’ was encountered at most air-rotary boreholes advanced in the proposed reservoir area. This ‘soil’ is considered an interstitial matrix of the cobble beds present across the Site – the native overburden is considered to be a predominantly gravel/cobble/boulder veneer. The interstitial soil was encountered at depths ranging between 0.75 and 3.9mbgs, and consisted of a brown silty sand with gravel and frequent cobbles and boulders within the proposed reservoir area. At the Apex River near the proposed pipeline crossing, the surficial veneer ranges in depth between 2.4 and 12.5mbgs and consists of cobbles and boulders with interstitial sand/silty sand.

Grey granitoid bedrock was encountered at all borehole locations advanced as of May 2025. Poor to excellent quality bedrock based on field observation and downhole camera assessment was encountered at all air track boreholes. In several locations, a frost-riven weathered zone of poor to fair quality bedrock was encountered near

the bedrock surface and ranged from 0.7 to 4.6m thick. Good to excellent quality bedrock was observed to underlie all poor to fair quality bedrock zones in all boreholes.

Core holes were advanced between October 2024 and May 2025 in order to confirm bedrock type, verify previous observations, determine RQD values, and obtain samples for Uniaxial Compressive Strength (UCS) testing. RQD values observed for the core holes that were advanced to a maximum depth of 5.2mbgs ranged from 22 – 100%, with the rock being generally of fair to excellent quality. However, discrete zones of very poor to poor RQD up to 1.5 to 2m thick were also encountered, notably in the narrow throat area between the new reservoir footprint and Lake Geraldine (referred to as the ‘throat’ area). Core samples from boreholes advanced in the throat area were reported to have UCS values between 163 and 272 megapascals (MPa). These values are characteristic of very strong granitoid bedrock.

The acid rock drainage and metal leaching potential of encountered bedrock at the Site was assessed by conducting acid-base accounting and shake flask testing on four bedrock samples. The bedrock at the Site was classified as not-potentially net acidic material as the neutralization potential ratios for all bedrock samples were greater than 2.

Groundwater level monitoring and permeability testing was conducted in August and October 2024 at all 12 monitoring wells. The results indicate groundwater is present between 0.4 and 2.8mbgs at most locations. Most wells (8 of 12 wells) were dry or frozen during the August monitoring program and four wells continued to remain dry or frozen during the October monitoring program. Interpretation of groundwater flow was completed using linear interpolation between measured water elevations and showed groundwater flow is closely tied to the undulating bedrock topography across the Site. Groundwater is estimated to flow from regions of high ground surface elevation to low-lying regions, primarily near waterbodies and watercourses. The *in situ* permeability test results were found to increase from August to October, with hydraulic conductivity ranging from approximately 3.0 $x10^{-10}$ to 1.0 x 10^{-5} m/s, potentially as a result of active layer thaw.

Downhole seismic surveys were completed at two borehole locations by Geophysics GPR during the December 2024 fieldwork. The Geophysics GPR investigation report is included as **Appendix E**. ReflexW software was used to interpret velocity layer contacts and layer velocities for S- and P-waves. Interpreted velocities are characteristic for gravel/boulder overburden in the upper layers, with high velocities indicating sound bedrock below, consistent with the Conceptual Site Model.

The geotechnical work outlined in this report was completed to assist with the ongoing development of the reservoir, conveyance, and pump station designs. As design parameters change and further geotechnical data is obtained, this report will be updated accordingly. Further site investigation is recommended to support detailed design.

1 Introduction

Arcadis Canada Inc. (Arcadis) was retained by the City of Iqaluit (the City) to complete a Preliminary Geotechnical Investigation Report for the parcel of untitled land located northeast of the surveyed portion of the community in Iqaluit, Nunavut (the Site). The field investigations and laboratory testing outlined in this report were carried out to provide geotechnical information to guide the design and the potential construction of a raw water reservoir and conveyance system associated with the City's Long-Term Water Program (LTWP) – Supply and Storage project.

This Geotechnical Report has been prepared in accordance with the requirements outlined in Appendix G – Supplementary Scope of Work of the Request for Proposal 2023-RFP-048 issued by the City.

This document is an interim version of the Preliminary Geotechnical Investigation Report and presents the results of the geotechnical work completed as of May 2025. Further geotechnical investigation and testing are recommended; once complete this report will be updated and expanded.

1.1 Site Description

The Site lies northeast of the residentially developed portion of Iqaluit and is roughly square in shape, with aerial coverage of approximately 1,150 hectares (ha), please refer to **Figure 1** located at the rear of this report. The northeast half of Lake Geraldine (LG) is included in the Site, as is the southwest half of Lake Qikiqtalik (LQ). The Road to Nowhere leads from a residential area in the southeast portion of Iqaluit to the approximate centre of the Site. An access road splits from the Road to Nowhere north of the Niaqunnguk (Apex) River and leads to the southwest shore of LQ. These site features are shown in **Figure 2** at the rear of this report.

The Site is mostly undeveloped. Topography varies from rolling to rough, with some major and numerous minor ridges and scarps, with tundra valleys and finger lakes oriented northwest-southeast in the direction of glacial scouring. Vegetation is continuous to absent, low Arctic to mid-Arctic. The Niaqunnguk River traverses the Site from its northwest boundary to its southern corner. Water in the Niaqunnguk River flows across the Site and south to Frobisher Bay.

The Site is within the municipal boundary of Iqaluit but outside of the surveyed area and is therefore classified as Untitled Municipal Land by the City's Planning and Development Department.

1.2 Study Area

The Study Area of the Site is based on the raw water reservoir and conveyance system configuration outlined in the Preliminary Design Report (PDR) prepared by Arcadis, dated March 2024. The preliminary reservoir design consists of eight retention structures (Dam 1 and Dykes 2 to 8 – Dyke 7 is designated as the reservoir spillway). These retention structures are to be constructed predominantly of blast rock obtained from the bedrock within the footprint of the proposed reservoir. An access road is proposed from the southeast side of the reservoir to the Road to Nowhere. Several rock quarry sites have been selected along the proposed access road to provide initial construction material. The transfer of water from LQ to the reservoir is to occur through a water conveyance pipeline that generally follows the existing access road before it turns west towards the reservoir. Two intake pump stations are proposed – at LQ and the Apex River – to draw water from the lake and provide flow through the pipeline. These proposed features are shown on **Figure 2**.

In accordance with preliminary designs, the Study Area of the Site has been restricted to key areas associated with the proposed structures and borrow areas.

1.3 Local Geology

A review of Canadian Geoscience Map 64 indicates that the surficial geology at the Site consists predominantly of a till veneer containing sand, gravel, cobbles, and boulders in a silty sand matrix; generally 0.0 – 2.0 metres (m) thick. Over the reservoir area, the surficial geology is roughly >40% till, <60% rock ledges, knobs, and rubble. The bedrock topography is evident, and very minor colluvium (including talus, colluvial fans, and solifluction lobes) is theoretically present among washed-till boulder fields. The majority of the Site appears to be intact and frost-riven outcrop with a discontinuous cover of rubble, boulders, gravel, and sand; all of which has been glacially scoured.

Approximately central through the Site, generally along the Niaqunnguk (Apex) River basin, the till veneer becomes a thicker till blanket of similar composition 1 – 10m thick, with instances of glaciofluvial outwash deposits including stratified gravel and sand, proglacial floodplains, channeled deltas, terraces, and fans. The till blanket can mask bedrock structure, appears in end moraines, and is affected by periglacial processes such as solifluction lobes, frost boils, and sorted patterns. The till blanket is reportedly also susceptible to thaw slumping on slopes or in excavations. Adjacent to, and intermittent within, the till blanket are areas of glaciofluvial subaerial outwash containing stratified gravel and sand which may be up to 30m thick. These areas may contain ice wedges and massive ice bodies.

As noted, bedrock topography is evident across the Site. A review of the Geological Survey of Canada Map 1860A indicates that bedrock conditions at the Site consist of intrusive igneous rock, further classified as an undivided granitoid (consist predominantly of quartz, plagioclase, and alkali feldspar) from the Paleoproterozoic Cumberland batholith (2100 – 1800 million years ago).

The Site is in a zone of continuous permafrost that generally begins at a depth between 1 to 2 metres below ground surface (mbgs). It is expected that the larger water bodies across the Site (including LG and LQ) serve as heat sinks and depress permafrost formation to greater depths in their immediate area. Taliks, areas of unfrozen ground surrounded by permafrost, are likely to form in areas near or under water bodies at the Site.

The topography of the Site comprises rolling hills to maximum elevations ranging between approximately 205 and 250m above mean sea level (amsl) along the northeast side of the Site near LQ based on GIS data from the Department of Community and Government Services of Nunavut. Surface elevation generally reduces from Qikiqtalik Lake toward LG. The minimum surface elevations along the southwest side of the Site range between approximately 80 and 135masl.

2 Scope of Work

As per the Request for Proposal 2023-RFP-048 issued by the City, the scope of work of the geotechnical investigations were to include the following:

- If permafrost is encountered, its ice content will be recorded.
- If unfrozen soil is present, perform Shelby Tube tests, Atterberg limits, sieves, and soil chemistry as necessary.
- Determine Rock Quality Designation (RQD) of recovered rock core.
- Collect and preserve samples of intact bedrock core for determination of unconfined compressive strength, shear strength and associated parameters.
- Carry out permeability tests at the borehole locations.
- Determine in-place unit weights for soils and rock samples.
- Install monitoring wells in boreholes and collect information on stabilized groundwater levels to infer groundwater flow direction.
- Obtain samples of each soil type encountered, the first sample being at a depth not greater than 750 millimetres (mm) and succeeding samples at not more than 1500mm increments of depth. Record whether samples are dry, moist, or wet.
- Record penetration values of Standard Penetration Test at the top of each soil stratum commencing at 750mm depth and at increments not greater than 750mm down to appropriate bearing stratum.
- Restore the Site to its original state upon completion of on-Site work.
- Develop a geotechnical investigation report which includes but is not limited to: (a) subsurface profiles showing rock and soil materials and geological formations, including presence of faults, buried channels, and weak layers or zones where encountered; and (b) characteristics and properties of soils and the weaker types of rock.
- Remove monitoring wells per applicable regulations upon completion of the project.

It has been further requested that laboratory test results be submitted to the Nunavut Impact Review Board (NIRB) as well as Crown–Indigenous Relations and Northern Affairs Canada (CIRNAC) regarding the acid rock drainage and metal leaching potential of encountered bedrock.

In order to meet the objectives specified and obtain other information required for the proposed structure and system designs, the following items have been completed as of May 2025 for the preliminary geotechnical investigation program:

1. Prepared a site-specific Health and Safety Plan (HASP).
2. Incorporated publicly available information (e.g., Geological Survey of Canada maps) obtained during the Conceptual Design Phase and observations from the preliminary site visit conducted in September 2023 into the strategy for site-specific geotechnical investigation.
3. Advanced 24 boreholes at 22 locations in the proposed reservoir area using an air-rotary drilling method. The air rotary drilling was completed in February and March 2024 while the Site was covered in snow to a) limit the impact the drilling equipment had on the tundra and b) allow the equipment to traverse the rough terrain while accessing the remote borehole locations.

4. Advanced 5 boreholes in the proposed pump station and pipeline crossing area at the Apex River using an air-rotary drilling method. This air rotary drilling was completed in May 2025 when the Apex River was frozen to its bottom. Approval from the Department of Fisheries and Oceans Canada was obtained.
5. Installed 12 piezometers in the boreholes advanced using the air-rotary drill method.
6. Installed three PVC casings in designated holes to allow for geophysical testing – conducted in December 2024 – to obtain site-specific response values to inform structure design.
7. Monitored groundwater in existing wells in August and October 2024 to confirm groundwater levels and infer groundwater flow direction.
8. Performed slug tests to obtain estimates of rock mass permeability at each monitoring well location.
9. Advanced 13 cored bedrock boreholes between October 2024 and May 2025.
10. Calculated Rock Quality Designation for extracted rock cores.
11. Completed laboratory testing (including unconfined compressive strength testing) to characterize rock strength.
12. Installed nine thermistor strings within the footprints of the proposed reservoir retention structures and one thermistor string within the footprint of the proposed pump station at the Apex River to measure ground temperatures.

Additional geotechnical investigation has been recommended to further inform the design and reduce uncertainty.

A draft Geotechnical Baseline Report, as specified in RFP documents, will be provided under separate cover to be included as part of the project's construction tender documents.

All drilling and excavating activities were conducted in general accordance with Canadian National Standard CAN/BNQ 2501-500: Geotechnical Site Investigations for Building Foundations in Permafrost Zones, Canadian Standards Association (CSA) PLUS 4011:19 Technical guide: Infrastructure in permafrost: A guideline for climate change adaptation, and CSA W205:19 Erosion and sedimentation management for northern community infrastructure. Borehole logging was completed in general accordance with ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes and ASTM D4083 Standard Practice for the Description of Frozen Soils.

3 Investigation Method

3.1 General

The geotechnical investigation has been conducted over multiple mobilizations, from 2024 to 2025. A summary of the programs completed are presented in **Table 3-1**.

Table 3-1: Summary of Geotechnical Programs and Timelines

Dates	Program	Description
September 2023	Initial reconnaissance	Initial site visit. Note features, inspect proposed pipeline, road routes, observe terrain and surficial materials, etc.
February to March 2024	Air-rotary drilling associated with proposed reservoir	A total of 24 boreholes were drilled at 22 locations primarily within the proposed reservoir and under proposed dam/dike locations. Boreholes were advanced to assess overburden and bedrock depth and condition, install piezometers, and collect rock samples for acid base accounting.
August 2024	Preliminary borrow source assessment	Surface soil sampling and preliminary soil volume estimation were conducted at potential borrow source areas across the Site.
August and October 2024	Groundwater monitoring and permeability testing	Groundwater monitoring including groundwater level measurements, permeability testing, and sampling was conducted at 12 piezometers associated with proposed reservoir.
October 2024 to May 2025	Bedrock coring associated with proposed reservoir	Thirteen (13) core holes were advanced using a hand-held exploration coring drill at proposed dam/dike locations. Core holes were advanced to further assess bedrock quality, secure core samples for strength testing, and install nine thermistors.
December 2024	Geophysics testing with proposed reservoir	Geophysics testing was conducted inside the proposed reservoir footprint to obtain seismic velocities and other physical properties.
April to May 2025	Air-rotary drilling associated with proposed pump station and pipeline crossing at Apex River	Five (5) boreholes were drilled using air-rotary drilling equipment near the proposed pump station and pipeline crossing area at the Apex River. Boreholes were advanced to install a thermistor and assess overburden and bedrock depths.

3.2 Drilling Activities

3.2.1 Air-Rotary Boreholes

The initial mobilization consisted of boreholes advanced by air-rotary with downhole hammer drilling equipment. Boreholes were advanced at 22 locations (AT2023-2a to AT2023-21, AT2023-25, and AT2023-27a) to depths of 4.0 to 11.3mbgs to assess the veneer/soil and bedrock conditions onsite. Descriptions of the veneer and interstitial soil stratigraphy are presented on the borehole logs in **Appendix A**. UTM coordinates for all borehole locations are shown on the borehole logs and in **Table 1**.

Drilling and sampling operations were conducted with the use of an air-rotary attachment on a CAT 320D excavator and a sled-mounted compressor, supplied and operated by Canadrill Limited (Canadrill). The boreholes advanced had a diameter of either 102 or 190mm, depending on tooling used. Prior to drilling, snow was removed from above the borehole location. Each hole was then advanced incrementally with 1.22m (4 ft) rods. All boreholes were drilled under the supervision of Adaptive Baseline Engineering Limited (ABG) field personnel.

Soil and rock-chip samples were collected in discrete intervals as each borehole progressed. Overburden samples were collected beginning at a depth of less than 0.75mbgs and succeeding samples, where applicable, were collected in succeeding intervals less than 1.5m apart. The samples were examined at the time of collection for general soil classification purposes (including type, texture, colour, and moisture characteristics). Borehole logs were prepared on the basis of sample and drilling process observations in the field describing the encountered strata. Following field logging, samples were placed into sealed plastic bags and shipped to the ABG laboratory for further inspection and grain size analysis. Sample locations and soil fraction results are shown on the borehole logs in **Appendix A**.

The subsurface interstitial soil and bedrock conditions, as encountered in the boreholes completed during the air-rotary drilling program, are documented in the detailed logs provided in **Appendix A**. A summary of the soil and bedrock conditions encountered at sampling locations is provided in **Section 5**. The reader is cautioned that conditions between and beyond boreholes may vary. It is also noted that the quantity of large rock in the overburden is likely underrepresented by the soil descriptions presented in the air-rotary borehole logs, as the air-rotary with downhole hammer method crushes and pulverizes material as the drill head is advanced, and only interstitial soils were collected for grain size analyses (coarse gravel, cobbles, and boulders left *in situ* and not included).

3.2.2 Diamond Drill (Coring) Boreholes

A total of 13 core holes were advanced by ABG using a hand-held exploration coring drill. The core holes were located within the footprint of the proposed reservoir retention structures (dam/dykes) as shown on **Figure 3**.

The initial phase of bedrock coring occurred between October and December 2024. In this period, a total of six core holes (BH2023-2b, BH2023-4b, BH2023-5b, BH2023-7b, BH2023-17b and BH2023-19b) were advanced. Four of the core holes (BH2023-2b, BH2023-4b, BH2023-5b, and BH2023-7b) were advanced under the proposed footprint of Dam 1, and a thermistor string was installed in BH2023-4b. Core hole BH2023-17b and BH2023-19b were advanced under the footprint of proposed Dyke 6 and Dyke 7, respectively.

The second phase of bedrock coring occurred between April and May 2025. During this period, an additional seven core holes (BH2023-1b, BH2023-9b, BH2023-12b, BH2023-13b, BH2023-15b, BH2023-27b, and BH2023-28b) were advanced. Two of the core holes (BH2023-1b and BH2023-9b) were advanced under the proposed footprint of Dam 1. Core holes BH2023-12b, BH2023-13b BH2023-15b, BH2023-28b and BH2023-27b were advanced under the footprint of proposed Dyke 2, Dyke 3, Dyke 4, Dyke 6 and Dyke 8, respectively. Several boreholes required multiple attempts to reach target depth due to adverse environmental and drilling conditions.

Core hole locations had to be accessed using all-terrain vehicles and/or snowmobiles due to the rough terrain of the Site and snow conditions at the time of the coring activities. The maximum diameter of the core holes was 40mm while the diameter of the extracted rock cores ranged between 25 and 36mm. The core holes were co-located with air hammer holes in areas where bedrock was at or near surface, to enable comparison and confirmation of air track observations. Core hole depths ranged between 3.1 and 5.2m, as documented on the Core Logs presented in **Appendix B**.

Distances between natural fractures in the core were measured on each core run extracted for RQD assessment. RQD was calculated at core run intervals of 1.5m or less using the following equation.

$$RQD = \frac{\sum Length\ of\ core\ pieces > 100\ mm}{Total\ core\ run\ length} \text{ x } 100\%$$

Under the RQD classification system, rock quality is described as ‘excellent’ when RQD is 90 - 100%, ‘good’ when RQD is 75 -100%, ‘fair’ when RQD is 50 - 75%, ‘poor’ when RQD is 25 - 50%, and ‘very poor’ when RQD is 0 - 25%. Core photos and lengths for RQD assessment are documented in the Core Logs presented in **Appendix B**.

A total of 11 rock core samples were submitted to PNJ Engineering Inc. (PNJ) for unit weight, tensile strength, and unconfined compressive strength testing along with modulus of elasticity (MOE) calculation. Tensile strength and unconfined compressive strength tests were conducted in accordance with ASTM D3967and ASTM D7012, respectively. Geotechnical laboratory certificates of analyses have been included in **Appendix C**.

3.3 Piezometer Installation and Groundwater Assessment

During the initial air-rotary drilling program, 12 boreholes (drilling method described in Section 3.2.1) were completed as piezometers. The piezometers comprised 50mm diameter Schedule 40 PVC riser pipes with a No. 10 slot intake zone (well screen) of varying length. Silica sand was placed around the piping to a height of at least 300mm above the top of the well screen as porous sand pack. The remaining annular space was filled with a bentonite seal or a combination of bentonite and silica sand. A protective steel stick-up monument casing was then sealed in place with bentonite and/or concrete over the top of the well riser.

Boreholes outfitted with piezometers are indicated in **Table 1** and are shown on **Figure 3** and **Figure 4**, at the rear of this report**.**

3.3.1 Groundwater Level Measurements

Groundwater levels and well depths were recorded at all 12 piezometer wells on 14 August and 2 October 2024. Water level measurements were taken using a Heron interface probe. Measurements were taken from the top edge of the PVC riser at each well. If water was not detected, the probe head was examined for ice crystals. Groundwater elevations were then calculated using the survey data elevation from each well. Groundwater level measurements are presented in **Table 2**, at the rear of this report.

3.3.2 Bedrock Permeability Testing

Slug tests were conducted on all 12 piezometers in August and again in October 2024. The screens in all wells were installed in bedrock; therefore, the derived K values are considered to be representative of shallow bedrock permeability over the screen intervals.

Prior to conducting the test, the existing water level or depth to bottom was measured at the well using an interface probe. Next, a known volume of water (1 litre, 2 litres, or 8 litres – the 'slug') was quickly injected into the well and the water level was measured and recorded at specific times using the interface probe. When possible, the water level in the well was monitored until 90% of the head change had dissipated. Slug tests were completed a minimum of two times per well, after allowing a return to equilibrium between tests.

Slug test data was interpreted using the Hvorslev calculation method. The Hvorslev expression for hydraulic conductivity is:

$$K = \frac{r^2 \ln (L/R)}{2\, LT_0} \;\; \text{for} \; \frac{\mathrm{L}}{\mathrm{R}} > 8$$

where K is hydraulic conductivity, r is casing radius, L is length of the open screen, R is the filter pack radius, and T_0 is the time at which the water level has recovered 37% of the initial head change. This method assumes a homogenous, isotropic medium in which the rock and water are incompressible.

Slug test data and hydraulic conductivity calculations have been included in **Appendix D**.

3.3.3 Groundwater Sampling

A groundwater sample was collected from two wells (MW2023-4a and MW2023-5a) located in the footprint of the dam area (Dam 1) of the proposed reservoir to assess general water chemistry for future concrete considerations. The groundwater samples were collected using a HDPE plastic bailer on 3 October 2024 and placed immediately in sterile, laboratory-provided bottles.

Samples were placed into an insulated cooler with ice for sample preservation. The cooler containing the samples was delivered to Bureau Veritas Laboratories in Ottawa, Ontario, by Arcadis. Bureau Veritas Laboratories is accredited by the Standards Council of Canada and has Canadian Association for Laboratory Accreditation.

3.4 Thermistor Installation

Thermistors were installed by ABG in nine core holes located within the proposed footprints of the reservoir retention structures, as shown on **Figure 3**. Multi-bead thermistor stringers were installed in each hole to a maximum depth of 5m. Thermistors were finished with either a plastic or metal stick-up monument.

Another thirteen-bead thermistor was installed to a depth of 9.1m within the footprint of the proposed pump station at Apex River, as shown on **Figure 4**.

Additional thermistor installations are proposed, pending the verification of the maximum thaw depth. Thermistors will be installed in additional boreholes at areas of interest: near the Apex River bridge along the Road to Nowhere (scheduled for replacement), at the proposed pump station location at LQ; and potentially along the access road to LQ, at one or more major culvert replacements. At each location, one multi-bead thermistor will be installed downhole to allow for ground temperature monitoring. Thermistors will be installed in the open boreholes and allowed to freeze. They will be finished with concrete into the bedrock surface with a locking roadbox or monument.

Multiple thermistor locations are intended to provide evaluation of the ground temperature profile around the proposed reservoir area as a whole, and at areas of interest as noted above. The goal is to provide a baseline and ongoing temperature profiles to assess the presence of a talik in the immediate area, assess the potential for freezeback inside retention structures, and verify the permafrost regime at areas with deeper design features (e.g., pump stations, bridge piles). Local temperature profiles will also influence dam/dyke design.

Further thermistor networks are planned as part of dam/dyke construction and must be monitored continuously over the life of the reservoir.

3.5 Down-Hole Seismic Testing

Geophysics GPR International (GPR) conducted downhole seismic (DS) surveys at the Site between 12 and 14 December 2025. The purpose of the testing was to measure the in-situ seismic velocity profiles. Seismic velocity profiles are required for earthquake design analyses for the proposed structures, and to help inform blasting program designs. The seismic surveys were completed in accordance with ASTM D7400-14 Down-hole Seismic Testing test method.

The DS testing was completed at boreholes AT2023-11A and AT2023-14A, located in the middle and north end of the proposed reservoir respectively. Geophones (tri-axial and vertical) were secured to the PVC casings at both boreholes. The seismic source was located 0.3 and 0.7m from the borehole AT2023-11A and AT2023-14A, respectively. A sledgehammer was used as an energy source. Three seismic records were recorded at each one-meter interval with the tri-axial geophone beginning at the bottom of the borehole and ending at one meter depth.

For additional details on the testing methodology refer to the full geophysical report prepared by GPR included in **Appendix E**.

3.6 Elevation Surveying

The boreholes advanced on the Site were surveyed by ABG. The surveys were conducted using Global Navigation Satellite System (GNSS) observations via a Topcon HiPer-VR Real-Time Kinematic Global Positioning System (RTK GPS) receiver. All observations were on the grid referenced to the 6-degree Universal Transverse Mercator (UTM) projection (Zone 19), central meridian 69-degrees West, based on the horizontal reference frame NAD83 (CSRS), Epoch 2010.0. All elevations are geodetic, referenced to the Canadian Geodetic Vertical Datum of 2013 (CGVD2013).

Coordinate Control Monument No. 28 (CCM28) and control points established in the initial survey of the Site were used as benchmarks to verify the accuracy of the borehole survey data collected. GPS coordinates and elevations of the boreholes can be found in **Table 1** and on the borehole logs provided in **Appendix A** located at the rear of this report. This survey effort and the subsequent information are thoroughly presented Arcadis' *Topographic Survey Report*, provided under separate cover in April 2024.

In September of 2024, the expanded reservoir footprint and Niaqunnguk River Valley was surveyed via drone. Further details can be found in Arcadis' updated *Topographic Survey Report* dated June 2025..

Boreholes, test pits, additional site features, and further controls will be surveyed as work progresses.

3.7 Surficial Test Pit Program (Potential Borrow Sources)

Three test pits were manually advanced at each potential borrow source (as outlined on **Figure 2**) in August 2024. Three test pits per potential source area were advanced to a maximum of 1mbgs with composite samples taken from each location. Borrow source areas were measured and delineated based on surficial observations. These activities were conducted as part of the aggregate assessment program, the results of which are provided under separate cover.

The goal of this program was to obtain soil samples for grain size analyses, assess the suitability of local borrow materials, and provide preliminary volumetric estimates for design and construction. Please refer to Arcadis' *Aggregate Assessment Report*, provided under separate cover, for greater detail.

4 Laboratory Analysis Program

4.1 Geotechnical Testing

Geotechnical laboratory testing was carried out on representative samples recovered from the boreholes and test pits as well as rock core samples. The purpose of the geotechnical laboratory analyses was to classify the various soil strata and rock encountered.

Testing on recovered soil samples included:

- Natural moisture content;
- Grain size analyses;
- Standard Proctor testing;
- Unit weight determination; and
- Interface friction and shear box testing.

Testing/measurement on recovered rock core samples included:

- RQD determination;
- Unconfined compressive strength testing; and
- Unit weight testing.

4.2 Acid Rock Drainage Testing

Acid Base Accounting (ABA) and shake flask testing was conducted on four bedrock chip samples obtained during the initial air-rotary drilling. Samples were collected from four boreholes: AT2023-03, AT2023-14, AT2023-21, and AT2023-25. ABA was completed by the Minerals Engineering Laboratory at Dalhousie University using the modified Sobek method and a sericite schist source material (KZK-1). Shake flask testing to assess leachable metals was conducted by ALS Canada Ltd. (ALS). Certificates of Analysis have been included in **Appendix F**.

4.3 Groundwater Testing

Two groundwater samples (MW2023-4 and MW2023-5) were collected and submitted for laboratory analysis of pH, dissolved sulphate, and dissolved chloride. Analyses were completed by Bureau Veritas Mississauga located at 6740 Campobello Road, Mississauga, Ontario. Certificates of Analysis have been included in **Appendix F**.

5 Subsurface Conditions and Results

A summary of the subsurface conditions encountered in the study area is presented below. The subsurface soil conditions, as encountered in the boreholes completed during this investigation, are documented in the detailed borehole logs provided in **Appendix A**. The reader is cautioned that conditions between and beyond boreholes may vary. It is also noted that the quantity of large rock in the overburden is underrepresented by the soil descriptions presented in the borehole logs as the air-rotary with downhole hammer method crushes and pulverizes material as the drill head is advanced, and only interstitial soil fractions were collected for grain size analyses (coarse gravel, cobbles, and boulders left in-situ). The overburden material across the reservoir site is not considered a ‘soil’, it is considered a cobble/boulder washed-till veneer. Glaciofluvial soils were encountered in the potential borrow pit areas along the RtN, and analyses carried out on samples retrieved from BPs are considered representative of the overburden material as a whole in that area, unless otherwise stated.

5.1 Surficial Geology Overview

The Study Area of the Site is comprised of rolling hills with boulder fields and exposed bedrock. In areas where an overburden veneer is present, it generally consists of unconsolidated cobbles and boulders with rare interstitial silty sand or gravel, generally to a depth less than 4mbgs. The underlying bedrock is classified as an undivided granitoid (monzogranite of the Paleoproterozoic Cumberland batholith) and is reported to consist predominantly of quartz, plagioclase, and alkali feldspar.

The Site is in an area of continuous permafrost. Only under deeper water bodies (typically greater than about 2m water depth) are unfrozen (talik) conditions expected to exist. The active layer in undisturbed ground is typically 1 to 2m deep, reaching a maximum depth in September/October (even as the ground surface starts to freeze back).

5.2 Fill Soils

No fill soils were encountered during the geotechnical investigation. This was expected, as the Site is predominantly undeveloped with the exception of the roadways, shooting range, Rotary Park and the interim pumping station areas.

Soils used on access roadways are observed to be primarily reworked native soils taken from local borrow sources. Please refer to Arcadis’ *Aggregate Assessment Report*, provided under separate cover, for more background information on local borrow sources and soils.

5.3 Native Soils

5.3.1 Cobble/Boulder Veneer

Native overburden was encountered at 18 of the 22 air-rotary borehole locations advanced within the proposed reservoir as of March 2024 – the others were bare bedrock at surface. Overlying interstitial soils at eight locations was a 0.2 to 1m thick layer of cobbles and boulders with significant void space, likely caused by freeze/thaw and/spring melt action (washed till resulting in the boulder fields referenced). Cobbles/boulders were typically covered in vegetation (moss) at surface. The rare interstitial overburden fraction was encountered at depths ranging between 0.75 and 3.9mbgs, and was characterized in the field as a brown silty sand with gravel, along with the predominant gravel, cobble, and boulder fractions.

Moisture content analyses conducted on the interstitial silty sand fractions ranged from 1.4 to 14.1%. No Standard Penetration Testing was possible in this unit due to it being predominantly gravel, cobbles and boulders.

Sieve analyses were conducted on 34 recovered interstitial soil fractions. Grain size analyses indicate that the interstitial media comprises a sand or gravelly sand, with some to trace fines. The results of the grain size analyses on interstitial media are summarized on the borehole logs included in **Appendix A** at the rear of this report.

5.3.2 Organic Soils

No organic soils were encountered at borehole locations. At several locations in the boulder fields a layer of moss was present, but only on the upper surface of boulders/cobbles and less than 10cm thick. Observations and grain size analyses in the reservoir area indicate a thoroughly washed till with very few or no fines remaining.

5.3.3 Glaciofluvial Deposits (Sand)

Sand soils were encountered in test pits advanced at the potential borrow sources (shown on **Figure 2**). These borrow source materials would potentially be used in dam/dyke construction, and were subjected to a variety of tests to obtain parameters required for design. In BP1, BP2, and BP3, soils were described as brown to dark brown sand, homogeneous, and dry to moist. Samples from BP4 were described as a sand till, brown to dark brown, heterogenous, and moist, with frequent instances of cobbles and boulders.

5.3.3.1 Grain Size Analyses

Moisture content in samples from BP1, BP2 , and BP3 ranged from 2.8% to 18%. Grain size analyses show the samples to be relatively poorly graded, with a sand fraction of 80% or above. The one exception was sample BP1-A3 (taken from south of the access road), which was described as sandy silt and found to have a silt fraction of 81%. The natural moisture content in this sample was 23% with trace ice noted.

Samples from BP4 were classified in the field as glacial till, with gravel and cobbles (heterogeneous). Grain size curves show the soils were relatively well graded, with a much flatter grain size curve and sand fraction of 64 – 70%. Moisture content ranged from 10 – 18%.

The results of these grain size analyses are presented in Table C at the rear of this report and the laboratory certificates in Appendix C. For a more detailed discussion please refer to Arcadis Aggregate Assessment Report, provided under separate cover.

5.3.3.2 Standard Proctor Testing

Standard Proctor tests were performed on the 12 soil samples from potential borrow sources. Samples from BP1, BP2, and BP3 showed a Standard Proctor Maximum Dry Density (SPMDD) ranging from 1642 to 1840kg/m^3, with optimum moisture content from 11.2 to 15.5%.

Samples from BP4 showed SPMDD from 2118 to 2209kg/m^3, with corresponding OMC ranging from 4.4 to 7.1%.

The results of the Proctor testing are presented in **Table D** at the rear of this report and the laboratory certificates in **Appendix C**. For a more detailed discussion please refer to Arcadis *Aggregate Assessment Report*, provided under separate cover.

5.3.3.3 Soil Hydraulic Conductivity Testing

Seven samples were submitted for hydraulic conductivity testing. The average *k* ranged from 1.25E-02 (BP2-A1) to 6.46E-05 (BP4-A1).

The results of the hydraulic conductivity testing are presented in **Table E** at the rear of this report, and the laboratory certificates in **Appendix C**. For a more detailed discussion please refer to Arcadis *Aggregate Assessment Report*, provided under separate cover.

5.3.3.4 Direct Shear Testing

Direct shear testing was performed on one sample from each potential borrow source, and index values were determined via graphical analyses of the results. Cohesion values for all soils tested was 0, as expected for granular soils.

Peak internal friction angle ranged from 33 to 38 degrees. The ultimate internal friction angle for samples from BP1, BP2, and BP3 was 27 degrees, and for the BP4 sample was 32 degrees.

The results of the direct shear tests are presented in **Table F** at the rear of this report and the laboratory certificates in **Appendix C**. For a more detailed discussion please refer to Arcadis *Aggregate Assessment Report*, provided under separate cover.

5.3.4 Riverbed Soils

As per the ABG Geotechnical Investigation Report included in **Appendix H**, the soils encountered at the Apex River pump station/crossing location consisted of surficial cobbles and boulders with an interstitial matrix of brown to grey sand to silty sand. The overburden materials here differ somewhat from those in the reservoir areas in that there appears to be a larger percentage of actual soil (sand and smaller particulate – though the cobble/boulder fractions are still high enough to preclude SPT measurements), and the ubiquitous presence of ice. The material was noted to vary between well-bonded with no excess ice, to zones of visible excess ice, to zones of pure ice at depth. Moisture content for samples obtained during drilling ranged from 6.0 to 43.3%.

Grain size analyses performed on the interstitial material show sand fractions generally in excess of 70% with few exceptions, with varying gravel and silt fractions. One outlier sample was measured to have 44% silt. The results of the grain size analyses results are tabulated in ABG's report.

5.4 Bedrock

Grey granitoid bedrock was encountered at all borehole locations advanced as of May 2025 in the proposed reservoir area. Poor to fair quality bedrock based on field observation and downhole camera assessment was encountered at 10 of the 22 borehole locations (AT2023-4a, AT2023-5a, AT2023-8, AT2023-11B, AT2023-13a, AT2023-15a, AT2023-17a, AT2023-18, AT2023-25, and AT2023-27a). The poor to fair quality bedrock typically started near the bedrock surface and was between 0.7 and 4.6m thick. However, at one borehole (AT2023-11B) located near the centre of the proposed reservoir, an approximately 1m thick zone of poor-quality bedrock was observed at a depth of 6mbgs with a 3.8m thick zone of sound bedrock observed above the poor-quality zone. Good to excellent quality bedrock appears to underlie all poor to fair quality bedrock zones in all boreholes based on core sample and/or camera assessment. Bedrock depths and elevations are shown on **Figures 3** and **Figure 4** at the rear of this report.

5.4.1 Coring Conditions

Coring conditions within the proposed reservoir area were observed to be difficult, given the generally fair – excellent quality, high-strength rock encountered onsite. Core samples retrieved to date confirm the presence of a monzogranite unit of the Cumberland batholith across the proposed reservoir area. Core drilling progress was generally on the order of 0.5 – 1.5m per day, indicating a very hard, competent rock mass, often immediately at surface.

5.4.2 Rock Quality Designation

Core samples retrieved from the proposed reservoir area show RQDs ranging from 51% to 96% over the lengths advanced, with rock quality generally designated as fair to good. RQD results for the 13 core holes advanced to date are presented in **Table 5-5**.

Table 5-1: Rock Quality Designations (RQDs) at Proposed Reservoir Retention Structures

BH No.	Associated Structure	Run 1 RQD	Run 2 RQD	Run 3 RQD	Run 4 RQD	Total Length RQD	Hole Depth (mbgs)
BH2023-1b	Dam 1	78% Good	82% Good	68% Fair	62%* Fair	74% Fair	5.11
BH2023-2b	Dam 1	82% Good	97% Excellent	67% Fair	84%* Good	82% Good	5.00
BH2023-4b	Dam 1	94% Excellent	31% Poor	76% Fair	23%* Very Poor	63% Fair	5.00

BH2023-5b	Dam 1	88% Good	51% Fair	61% Fair	22%* Very Poor	55% Fair	4.81
BH2023-7b	Dam 1	93% Excellent	87% Good	65% Fair	99%* Excellent	79% Good	5.11
BH2023-9b	Dam 1	98% Excellent	97% Excellent	88% Good	100%* Excellent	96% Excellent	5.04
BH2023-12b	Dyke 2	87% Good	100% Excellent	95% Excellent	85%* Good	94% Excellent	5.22
BH2023-13b	Dyke 3	41% Poor	49% Poor	50% Fair	65%* Fair	50% Fair	5.21
BH2023-15b	Dyke 4	66% Fair	80% Good	57% Fair	90%* Excellent	71% Fair	5.18
BH2023-17b	Dyke 6	79% Good	61% Fair	-	-	62% Fair	3.10
BH2023-19b	Dyke 7	96% Excellent	61% Fair	53% Fair	76%* Good	70% Fair	4.98
BH2023-27b	Dyke 8	75% Good	43% Poor	89% Good	100%* Excellent	72% Fair	5.11
BH2023-28b	Dyke 6	54% Fair	67% Fair	91% Excellent	16%* Very Poor	57% Fair	5.03

* Run 4 length was typically less than 1.5m and at the bottom of the hole, making this interval subject to mechanical difficulty, increased risk of breakage, and making the RQD calculation vulnerable to small sample size.

5.4.3 Rock Strength

Uniaxial compressive strength of the eleven rock core samples submitted to date ranged between 163 and 272MPa, indicating very strong rock. Two samples did have a compressive strength test result lower than 163MPa, but those results were considered to be due to pre-existing fractures in the rock, as noted by visual observations and failure behaviour measured by the strain gauges. Tensile and compressive strength results along with unit weights and modules of elasticity for the rock core samples are presented in **Table G** at the rear of this report, and laboratory certificates in **Appendix C**.

5.4.4 Acid Rock Drainage and Metal Leaching Results

The neutralization potential ratios (NPRs) of the four bedrock samples analyzed ranged between 4.35 and 19.58. NPRs were calculated by dividing the neutralization potential result by the acid generation potential result for each sample. The pH of the four samples ranged between 8.05 and 8.55. ABA and leachable metals results have been summarized in **Table J** and **Table K**, respectively, at the rear of this report. Certificates of Analysis have been included in **Appendix F**.

5.5 Groundwater Conditions

Groundwater conditions at the Site were investigated using the 12 piezometers installed between February and March 2024. Water level measurements indicate that groundwater, when present, is between 0.4 and 2.8mbgs. Groundwater was detected in four of the 12 wells in August 2024 and seven of the 12 wells in October 2024, indicating the presence and fluctuation of the active layer (permafrost). Water level measurements for each well are presented in **Table B**, at the rear of this report.

Groundwater flow directions were inferred using linear interpolation between groundwater level measurements collected at the wells in August and October 2024. At wells where groundwater was not detected, shallow groundwater was assumed to be confined to the active layer at a depth of 1.5mbgs. Interpreted groundwater flow directions for the Site are shown on **Figure 5** and **Figure 6**, at the rear of this report.

5.5.1 Bedrock Hydraulic Conductivity Results

The hydraulic conductivity in bedrock at each well location as determined by slug testing is presented in **Table 5-7** below.

Table 5-2 Hydraulic Conductivity Results at Well Locations

Well No.	K (m/s)	Well Status	K (m/s)	Well Status
	August 2024		October 2024	
MW2023-2a	3.06×10^{-8}	Dry/Frozen	2.42×10^{-9}	Dry/Frozen
MW2023-4a	8.41×10^{-8}	Dry/Frozen	1.47×10^{-5} 1.96×10^{-5}	Groundwater Present
MW2023-5a	5.86×10^{-6}	Groundwater Present	8.51×10^{-6}	Groundwater Present
MW2023-7a	8.25×10^{-7}	Groundwater Present	1.94×10^{-6}	Groundwater Present
MW2023-9a	9.30×10^{-6}	Dry/Frozen	3.96×10^{-8}	Dry/Frozen
MW2023-12a	1.39×10^{-8}	Groundwater Present	6.23×10^{-6}	Groundwater Present
MW2023-13a	4.72×10^{-8}	Groundwater Present	5.14×10^{-6}	Groundwater Present
MW2023-15a	1.42×10^{-7}	Dry/Frozen	4.08×10^{-9}	Dry/Frozen
MW2023-17a	6.37×10^{-6}	Dry/Frozen	1.04×10^{-5}	Groundwater Present
MW2023-19a	1.60×10^{-7}	Dry/Frozen	2.94×10^{-10}	Dry/Frozen

Well No.	K (m/s)	Well Status	K (m/s)	Well Status
MW2023-21a	4.16×10^{-8}	Dry/Frozen	1.54×10^{-5}	Groundwater Present
MW2023-27a	3.71×10^{-6}	Dry/Frozen	8.42×10^{-9}	Dry/Frozen

5.5.2 Groundwater Chemistry Results

The analytical results for pH, dissolved sulphate, and dissolved chloride in groundwater samples collected from wells MW2023-4a and MW2023-5a are presented in **Table I** at the rear of this report. pH values were between 7.0 and 7.5, and all ion concentrations, where detected, are considered low (of no concern).

5.6 Ground Temperature Measurements

Thermistors were installed during the 2024/2025 coring work, and ground temperatures were measured in each thermistor after installation. Several thermistors have been monitored multiple times.

Initial thermistor readings generally show temperatures of -6 or -7°C at depths of 5m in the April/May monitoring period. That time of year is expected to be roughly maximum freeze depth, to be verified by subsequent measurements.

Measurements to note occur in T1 (BH2023-4b) during December, which initially showed readings of 0°C at 5mbgs. This reading could potentially be a result of the string being slow to equilibrate, or indicate the depth of ground thaw. More readings are required at all locations.

T4 (APEX2025-05) shows a different thermal profile than the thermistors in the reservoir area. The lowest temperature recorded here is -2°C, at 9mbgs. The thermal profile is expected to be different due to the proximity to the Apex River (large water body).

Temperature readings from thermistors will continue to be taken periodically to measure the ground temperature seasonally. Existing thermistor readings are considered at or close to maximum ground freeze (April/May); thermistor readings from maximum thaw are required to measure seasonal variation. Thermistor reports are presented in **Appendix G**.

5.7 Geophysical Investigation

The summarized results of the DH seismic testing completed at the Site are presented in **Table H** at the rear of this report.

Overburden/frost-riven intervals (0 – 2m in BH2023-11B, and 0 – 4mbgs in BH2023-14B) show S-wave velocities ranging from 497 to 1301m/s. In the sound bedrock intervals underlying the overburden, S-wave velocities are all in excess of 2500m/s. The corresponding assigned shear moduli range from 494 to 4,232MPa for overburden intervals, and from 15,638 to 20,938MPa for bedrock intervals.

Both S-wave and P-wave Velocity Inversion Models at boreholes AT2023-11A and AT2023-14A are presented in the geophysical report prepared by GPR included in **Appendix E**. Please refer to that report for more detailed results and commentary.

6 Geotechnical Discussion and Recommendations

It is understood that the Site is to be developed to meet the objectives of the LTWP. The primary objectives of the LTWP are to:

- Establish a new long-term water source and the necessary infrastructure to address the City's present and future water demands, ensuring that the water supply system supports population growth; and,
- Construct a new reservoir to secure sufficient year-round water storage capacity by adding a minimum 1.5 increase in the over-winter storage capacity and meeting the current and projected needs of the City (overwinter capacity of 1,643,250m^3 needed).

Currently under consideration is the raw-water reservoir and conveyance system configuration outlined in the PDR prepared by Arcadis, dated March 2024. The preliminary reservoir design consists of eight retention structures (Dam 1 and Dykes 2 to 8, including Dyke 7 which is designated as the reservoir spillway), as shown on **Figure 2**. These retention structures are to be founded directly on bedrock and will be constructed predominantly of blast rock obtained from the bedrock within the footprint of the proposed reservoir. Dam and dyke structures are similar in design, the differences being: the size of the structures; the presence of an additional buttress at the rear of the dam; and the water conveyance under the dam allowing water transit from the new reservoir to LG. The general dam/dyke structure is a homogenous rock-fill embankment with internal, filter-graded cushion layers to support the waterproof geosynthetic liner, and riprap armouring on the upstream side to protect against ice impacts.

A reservoir ring road and an access road constructed of crushed rock material is proposed from the southeast side of the reservoir to the Road to Nowhere. Several potential rock quarry sites, shown on **Figure 2**, have been selected near the proposed access road to provide the needed initial construction material.

The transfer of water from LQ to the reservoir is to occur through a water conveyance pipeline that generally follows the existing access road before it turns west to towards the reservoir. This pipeline must cross multiple water courses between LQ and the proposed reservoir, the largest being the Niaqunnguk (Apex) River.

A pump station is proposed at LQ to draw water from the lake and provide flow through the pipeline. A second pump station is proposed at Niaqunnguk (Apex) River to supplement the water draw from LQ. Both these stations will likely be constructed directly on bedrock at the locations shown on **Figure 2**.

Recommendations are provided below to guide building foundation design and construction. Further geotechnical engineering analyses may be required if there are any changes to the final configuration of the proposed development, or if unexpected conditions are encountered during construction.

6.1 Foundation Considerations

The building comments and recommendations presented below apply to the structure footprint zone presented on the PDR drawings. Should the proposed structure be located outside of the zones established in the PDR, or building parameters change, revisions to these recommendations may be required.

6.1.1 Dam and Dykes

The dam and dyke structures are to be founded directly on dressed, debris-free, competent, native granitoid bedrock. Overburden and frost-riven material are to be excavated and sidecast to expose competent rock. Given the estimated strength of the rock, structure loads are not anticipated to be cause for concern.

Angular blast rock with freshly exposed facets is expected to have a higher internal friction angle than existing boulder field material, and is thus recommended for use in the dam and dyke structures at all founding surfaces.

6.1.1.1 Foundation Drainage and Backfill

The recommended blast rock fill is expected to be free-draining and frost-resistant. Founding surfaces must direct water away from retention structure faces and toes (i.e., towards the rear of structures).

Drainage swales, either constructed of blast rock or adequately dressed bedrock surfaces, must be constructed such that surficial runoff, seepage and precipitation is directed away from retention structures. Slopes at the rear of retention structures are to be not less than 1.5%, with drainage swales constructed at greater angles as specified by hydrology event models.

6.1.1.2 Rock Surface Dressing

Founding surfaces should be ripped (i.e., with an excavator outfitted with a ripping tooth or hydraulic hammer attachment) and cleared of deleterious weathered and/or frost-riven material. It is anticipated that in the throat area (i.e., within the footprint of Dam 1) overburden and weathered bedrock will require extensive excavation and removal.

Founding surfaces are to be inspected and approved prior to the placement of concrete or engineered fill. Cobbles and boulders larger than those noted in the dam and dyke material specifications should be removed.

6.1.1.3 Dental Concrete

A certain amount of dental concrete is expected at founding surfaces, and especially around key trenches for liner systems, etc. Analytical results indicate that general use cement is appropriate for the Site.

Any large fractures or open features at surface should be sealed and backfilled using appropriate materials. Any concrete emplacements must be allowed to properly cure before loading or further construction.

6.1.1.4 Liner Embedment

Blasting is anticipated at the toe of each retention structure to construct a key trench for liner embedment. Backfill concrete and appropriate materials must be allowed to cure around the liner toe prior to loading of any kind on the liner segments.

Stepwise construction will be required, given the height of the dams and the nature of design. Underlying layers must be placed and compacted appropriately prior to liner placement. Compaction of upper layers is to be performed carefully and only after an initial settlement period has elapsed. Liner welding and friction hold will proceed concurrently with backfill activities, given the height of the structures.

6.1.1.5 Grout Curtains

A wide range of hydraulic conductivities has been measured onsite, from 1.04 x 10^{-5} to 1.94 x 10^{-10}m/s. The combination of excavation of poor quality and frost-riven zones with surface dressing, key trenches and dental concrete, may be expected to mitigate seepage at the retention structure locations. Rock RQD values generally increase with depth, and thus while some immediate areas may experience a limited amount of seepage the bulk of the reservoir volume is expected to be effectively watertight.

Given the goals of the project, additional confirmation of hydraulic conductivity and connectivity over distance is recommended.

The 'throat' area under Dam 1 is noted to have instances of poor to fair quality rock. Given the conveyance underneath Dam 1, larger amounts of grout and/or concrete are expected in order to waterproof this structure and the liner key trench. Overburden and/or poor rock quality appears to be more prevalent in this area than in others, and will require special attention during design.

6.1.2 Water Conveyance Pipeline

The water pipeline is expected to be used during the summer months and use the existing topography (generally sloping down from LQ to the Niaqunnguk River) to its advantage. Pipe material selection should consider the loading requirements and material type used to bury the pipe.

6.1.2.1 Foundation Drainage and Backfill

Backfill material is expected to be a combination of local borrow material (i.e., sand) and the same blast rock used to construct the retention structures. These materials are expected to be free-draining and frost-resistant.

As with roadways, founding surfaces should be sloped to promote drainage away from the pipe and fill structures. Drainage swales along the pipe alignment will be required where the natural topography does not slope away from ROW.

6.1.2.2 Pipe Embedment and Cover

Backfill material is expected to be a combination of local borrow material (i.e., sand) and the same blast rock used to construct the retention structures. Granular material compaction should be performed using hand tools immediately around the pipe itself. The blast rock material is expected to be self-compacting and should only need to be placed atop the pipe cushion layers.

Dedicated crossings using culverts are recommended to avoid heavy equipment loading the angular blast rock material on top of the pipe once in place.

6.1.2.3 River Crossing

The river crossing is expected to incorporate a pipe pile/pier design, as is typical in the region. Borehole data along the proposed river crossing indicates bedrock ranges from 7.0 to 12.5mbgs across the Niaqunnguk (Apex) River bed with the bedrock surface rising to 2.4mbgs along the northern shore.

Design parameters and considerations for the river crossing using a Rock Socket Steel Pipe Pile design are presented in ABG's Geotechnical Investigation report in **Appendix H**.

6.1.3 Pump Station (Niaqunnguk River)

The overburden material in and around the Apex River differs somewhat from the washed cobble/boulder veneer present in the reservoir area. Bulk material characteristics are similar, with sand/silty sand forming the interstitial matrix between cobbles/boulders, but with the added component of water and ice throughout. Please refer to the ABG Geotechnical Investigation Report provided in **Appendix H** for greater detail and discussion.

6.1.3.1 Building Foundation

A detailed discussion of recommended pile foundations is included in the aforementioned Geotechnical Investigation Report. Shallow foundations are not recommended in the area due to the presence of ice-rice soils and relatively warmer ground temperatures in proximity to the Apex River.

Concrete footings on bedrock may be possible. Further investigation (test pits) are recommended in the area to verify the depth of overburden across the area of the pump station footprint.

6.1.3.2 Slab Foundations

Seasonal thaw and freezeback of the active layer with result in seasonal movement of any exterior slabs and/or unheated ancillary equipment founded directly on grade, necessitating flexible connections. Slabs should also be sufficiently rigid to accommodate potential differential movement.

6.1.3.3 Rock Surface Dressing

Founding surfaces should be ripped (i.e., with an excavator outfitted with a ripping tooth or hydraulic hammer attachment) and cleared of deleterious weathered and/or frost-riven material. It is anticipated that overburden and weathered bedrock will require extensive excavation and removal. Unlike the dam/dyke foundations, all cobbles and boulders should be removed.

Founding surfaces are to be inspected and approved prior to the placement of concrete or engineered fill.

6.1.3.4 Dental Concrete

A certain amount of dental concrete is expected at founding surfaces, and especially around wet wells, trenches for water conveyances, etc. Analytical results indicate that general use cement is appropriate for the Site.

Any large fractures or open features at surface should be sealed and backfilled using appropriate materials. Any concrete emplacements must be allowed to properly cure before loading or further construction.

6.1.3.5 Grading, Foundation Drainage and Backfill

Final site grades must eliminate the potential for ponding water and ensure drainage away from structures. It is further recommended that cuts into the native soil be avoided, and design grades reached by building above native grades where possible.

Parking areas/roadways are anticipated to be gravel surfaces. A minimum of 200mm of surface/base course material, underlain by at least 300mm of sub-base material, is recommended across all driveway, parking, and roadway areas. Heavy-duty areas (e.g., heavy construction equipment access) should receive an additional 300mm of sub-base material, unless founded directly on sound bedrock. Recommendations for backfill components, (including Type I and II material specifications), thicknesses, required compaction, etc., are all included in ABG's Geotechnical Investigation Report (**Appendix H**).

All founding surfaces must be inspected by a geotechnical engineer prior to placement of fill or concrete. Construction planning should consider the potential for underlying permafrost soils to thaw/soften during excavations. Excavation should be limited to only those areas deemed necessary, and be carried out in stages to minimize the length of time excavations remain open/below previous grade. Please see **Appendix H** for greater detail and discussion.

6.2 Seismic Considerations

Geophysics GPR International Inc. (GPR) was retained to perform downhole seismic analyses in order to obtain site-specific values from the site. Please refer to their report, provided in **Appendix E**, for more details on methodology and results.

6.2.1 Seismic Site Classification

Given the presence of competent rock at/near surface, the site class for seismic response in the reservoir area considered to be is Class A (hard rock) for the foundations bearing on competent rock. Site surveys confirm this evaluation, with V_{s30} well over 1500 m/s.

The Apex River Pump Station site can be classified as "X_c" for seismic site response, given the presence of a larger soil fraction. Soils are not considered liquefiable.

6.2.2 Seismic Hazard

The Iqaluit area is, historically, an area of low seismic activity. No notable seismic activity has been reported in or around the entire Frobisher Bay area from 1627 – 2015, as best as can be determined.

The seismic loading for designing new dams is determined by the intensity of the ground shaking expected at the dam site, and is not governed by the magnitude of the quake alone. Existing reports (e.g., the LG DSR, Concentric 2021) report using estimated ground motion values as extrapolated by the NBCC seismic estimating tool for the area. These reports typically rationalize using the extrapolated PGA values instead of carrying out S(Ta) analyses, in order to avoid carrying out analyses with overly conservative assumptions for a very stiff structure (the dam is concrete, founded directly on bedrock).

Both methods will be examined during the new structure design process, using the site-specific values obtained in December 2024.

6.2.3 Seismic Modelling

The CDA technical guidance documents specify seismic modelling guidelines for the differing dam risk classes. The existing LG dam and proposed Dam 1 are both classified as Extreme Risk, and thus require definition of earthquake design ground motion (EDGM) parameters using both a) local and regional geotectonic information (of which the seismic parameters obtained in 2024 are a part), and b) statistical analyses of historical earthquakes experienced in the region.

These analyses will be performed as part of the design process.

6.3 Subgrade

The stratigraphy in the area consists mainly of discontinuous frost-riven rubble and washed-till blankets from 0.5 to 2.0m thick. The subgrade is essentially bedrock, of varying condition, and that is the assumed founding surface for all structures/utilities.

6.3.1 Site Grading

Given the majority of the reservoir footprint is to be blasted and excavated, site grading is expected to be fairly rough. Grading inside the reservoir footprint is not of large importance. The rock fill structures should easily incorporate rough grading, which will add a level of friction and stability to founding surfaces, provided they are adequately drained.

Cut and fill structures are expected to be required, in different locations, for access roads and laydown areas. As with other structures, ROW founding surfaces should have deleterious and/or frost-riven material removed and be graded to promote drainage away from footprints. Surficial drainage courses are expected to be required at some sections around access roads, and water discharge points (culverts) will be required at select locations.

6.3.2 Engineered and Native Fill

6.3.2.1 Sand Fill

Sand fill is to be obtained from local borrow pits onsite. Please refer to Arcadis' *Aggregate Assessment Report*, provided under separate cover, for more details regarding these potential sources.

The material encountered at potential borrow sources BP1, BP2, and BP3 during this investigation is predominantly well-graded sand, with the rare exception, and suitable for the intended uses, i.e., liner/pipeline cushion layers, roadway surface courses. Some stockpiling and sorting is expected to be required, depending on exact material specifications for different applications.

Soils from BP4 may be acceptable for use as well, depending on design specifications. Field characterization and laboratory testing show a material is typical of a glacial moraine (till) feature, albeit a relatively well-drained one: compact, heterogeneous, poorly sorted, and with a relatively high density and internal friction angle. Several layers in the dam construction design call for coarser gradations, and BP4 materials may have adequate drainage, shear strength, and internal friction angles to be of potential use. It is anticipated that some processing would be required to render these materials acceptable for use in construction, but with a potentially very large volume available, this resource may be worth developing.

A more extensive test pit program is recommended to more fully define available volumes and confirm material gradation and extents.

Recommendations for backfill components, (including Type I and II material specifications), thicknesses, required compaction, etc., are all included in ABG's Geotechnical Investigation Report (**Appendix H**).

6.3.2.2 Crushed Rock Fill

Multiple gradations of crushed rock fill may be required. It is understood that the blasting program will be designed to produce 150mm minus material, which is appropriate for the intended uses as detailed above, subject to design specification.

The produced material is expected to be angular, rough, free-draining and frost- and erosion-resistant. The majority of the structures will be able to incorporate various particle sizes, to a degree. Sorting after excavation is expected to be minimal. Larger gradations up to 1m in diameter will have applications as riprap, armour stone and vehicular guard obstacles. Very large boulders may need to be crushed or split before use.

6.3.3 Slab on Grade Considerations

Slab-on-grade construction could be appropriate for certain components onsite. Details are not known regarding the floor grade(s); however, topsoil, humus, and the existing surficial silt soils are not considered suitable for support of building floor slabs and should be excavated and removed, along with any other weak/deleterious material (none encountered at borehole locations). Exposed surfaces of the native sand and gravel are considered suitable for slab-on-grade-support, after being proof-rolled to identify soft spots – which should be repaired through excavation and backfilled with an appropriate engineered fill as discussed in **Section 6.3.2**.

Any building floor slabs should be constructed to be independent of the building foundation walls and any other parts of the structure that will be supported by different soils to minimize differential settlement issues.

A minimum 150mm thick layer of compacted, free-draining granular or clear crushed stone material should be placed between any prepared soil subgrade and the building floor slab to provide sub-slab drainage, moisture migration control and support. If any native/reworked material options are used, given the variable subgrade soil potentially present it is recommended that a layer of a non-woven geotextile be placed to separate the crushed stone from the subgrade.

Proof-rolling and geotechnical inspection is required to ensure that founding surfaces are of acceptable, undisturbed native soils prior to placing crushed stone, engineered fill, or concrete.

6.3.4 Groundwater Interpretation

Groundwater at the Site was investigated using piezometers installed across the proposed footprint of the reservoir. Most boreholes, and therefore wells, were advanced in regions directly under proposed dam and dyke structures. The well screens were installed in the surficial bedrock formation to a maximum depth of 8.7mbgs. Data collected as of October 2024 indicates that groundwater, when present, is found between 0.4 and 2.8mbgs. Interpretation of groundwater levels suggest groundwater flow at the Site is closely connected to topography. Groundwater flows from regions of high elevation around the proposed reservoir to the nearby waterbodies and watercourses, as shown in **Figure 5** and **Figure 6**.

Ice formation in the ground strongly influences groundwater behavior at the Site. Thawing of the ground over the summer months results in increased groundwater activity in shallow bedrock as the depth of the active layer above permafrost increases. Preliminary data indicates that groundwater is present in shallow bedrock under the footprint of proposed Dam 1 and Dykes 2 and 3 as early as August. The presence of groundwater in shallow bedrock can be expected to increase into the fall season and deepen in areas where overburden material is removed.

Bedrock permeability at the Site was found to be higher during October than August, expected to be due to fluctuations in the active layer. Preliminary permeability testing indicates that horizontal hydraulic conductivity in thawed bedrock above 5.5mbgs can be expected to range between 1.47 x 10^{-5} and 8.51 x 10^{-6} m/s.

6.3.5 Groundwater Considerations

If groundwater is encountered in shallow excavations, it may be able to be managed by pumping from a system of drainage swales and sumps inside the excavation locations.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium. Discharged water is expected to be directed toward natural drainage courses that do not feed into LG, unless the appropriate approvals are obtained.

The finished surface grades should be sloped away from the proposed structures to prevent surface ponding and infiltration immediately adjacent to any construction. Backfill adjacent to structures should comprise compacted, free-draining granular materials.

6.3.5.1 Initial Dewatering

The surface water in the reservoir footprint will need to be evacuated prior to drilling and blasting. Given the source and native providence of the water, it is expected that discharge overland to the Niaqunnguk River is an acceptable solution. It is understood that these surface water bodies are filled via the freshet, and refill via groundwater after evacuation is expected to be minimal.

6.3.5.2 Dewatering During Construction

Inferred groundwater flow generally follows the bedrock surface and appears to be primarily driven by the interflow of precipitation.

Given the low amount of precipitation in the area, the presence of permafrost, the quality of bedrock and proximity to the surface, and the depth of excavation required, groundwater infiltration is expected to be minimal. Precipitation events and the annual freshet will require dewatering, although the catchment area is relatively small. Adequate dewatering is expected to be achieved via a series of sumps across the reservoir footprint until excavation targets are reached and structures have been fully constructed.

6.3.6 Permafrost Considerations

The active layer appears to extend from 1.5 to 5.5mbgs within the proposed reservoir area , based on the freeze/thaw of groundwater noted in the monitoring wells placed onsite and initial thermistor readings. This must be verified by subsequent seasonal measurements. This is expected to impact blasting, excavation, and dewatering onsite. Continued monitoring is recommended at the Site to measure active layer fluctuations, timing, speed, and long-term trends.

All founding surfaces must be inspected by a geotechnical engineer prior to placement of fill or concrete. Construction planning should consider the potential for underlying permafrost soils to thaw/soften during excavations. Excavation should be limited to only those areas deemed necessary, and be carried out in stages to minimize the length of time excavations remain open/below previous grade.

Continual monitoring of the active layer and deeper permafrost will be required over the lifespan of the project as the expected talik under the new reservoir expands. Some freezeback may be expected in the retention structures given their size. Free drainage is thus important in these areas. Founding surface drainage and appropriate materials are required for all structures.

Additional thermistors and a long-term monitoring program are recommended.

6.3.7 Bedrock Acid Drainage Potential

The bedrock at the Site is classified as not-potentially net acidic (Non-PAG) material. Based on the criteria established in the Prediction Manual for Drainage Chemistry from Sulphidic Geologic Materials (Price, 2009), the Non-PAG classification was considered appropriate as the NPRs for all bedrock samples were greater than 2.

6.4 Future Recommendations

6.4.1 Additional Investigation Work

Further recommendations may be made following the ongoing investigation onsite.

6.4.1.1 Additional Borehole Locations

Given the size and extent of the project construction, additional boreholes are recommended to confirm the findings and data obtained thus far. Additional data will allow for smaller tolerances and less uncertainty in the Geotechnical Baseline Report, to be provided under separate cover. Combined with the recommended test pit program, additional boreholes will provide greater definition of the overburden layer types and depths, and confirm rock types across the reservoir footprint. Additional boreholes are also recommended within the footprints of the proposed retention structures, along the access road to LQ, and at the proposed pump station location at LQ to inform structure design. Added boreholes and greater site coverage will ultimately allow for design refinement, a more precise specification and tender package, and greater certainty and cost control during construction.

Existing and new boreholes would be used to perform the packer testing discussed below. Additional, deeper boreholes advanced via air track would also allow for additional thermistors to be installed. Thermistor installation is planned at each structure location onsite. Maximum ground thaw depths have not yet been verified – seasonal thermistor data is still required – but the active layer, as measured in T1 in December and inferred from frozen monitoring wells onsite, may be deeper than anticipated. Pending seasonal results and verification of the maximum thaw depth, additional thermistors to greater depths are recommended in the reservoir footprint area. This will provide a greater understanding of the current permafrost configuration and allow for more detailed modelling of the active layer, permafrost and potential talik progression over time. A long-term thermistor monitoring program is also recommended.

6.4.1.2 Packer Testing

In order to refine estimates of underlying bedrock permeability, packer testing is recommended at select core holes advanced at each dam, dyke or spillway locations. Given the range of hydraulic conductivities found at different locations and depths onsite, packer testing is recommended to isolate particular intervals of interest at potential problem locations.

Specific intervals at each borehole (identified through coring) will be isolated using inflatable bladders on an apparatus called a packer system; when lowered to the target depth, the bladders are hydraulically inflated using fluid in drill rods to seal against the edges of the borehole. Permeability testing is then performed on the isolated interval (pressure injections of clean water). Pressure and flow rates are recorded during a test and analytical methods (e.g., Thiem equation) are used to determine the hydraulic conductivity of the unit and transmissivity of the zone. Packer testing also allows for the simulation of increased hydraulic head pressures in the targeted intervals.

Initial permeability calculations have allowed the design to progress to the current stage. Targeting testing is recommended at the areas where seepage could potentially be an issue (e.g. in the throat area) as the design is refined.

6.4.1.3 Test Pit Program

A test pit program is recommended for the summer/fall of 2025, when ground thaw is at the largest extent. Test pits are recommended:

- to assess the depth and extent of potential borrow sources while obtaining samples for further geotechnical testing;
  - o Additional testing would include Standard Proctor, column permeability, shear box testing and material interface friction tests, all required to finalize design.
- at locations of interest along the existing roadway/planned pipeline conveyance to aid in road upgrades and pipeline foundation design; and
- in and around the proposed footprints of the pump stations at Apex River and LQ to assess overburden, depth to bedrock, and aid in foundation design.

Four potential borrow sources, previously used during the construction of the road to LQ, have been identified, are shown on **Figure 2**, and are further detailed in Arcadis' *Aggregate Assessment Report*.

A second testpit program is recommended for the summer/fall of 2026 once an access road to the proposed reservoir has been constructed as part of the Early Works construction activities. Test pits are recommended:

- to assess the depth and composition of till veneer in the reservoir area;
- to expose and assess bedrock uniformity along the proposed dam and dyke alignments; and,
- to assess excavation strategies for weathered/frost-riven bedrock at structure footprint locations (constructability).

ABG will mobilize a local excavator and supervise excavation and sampling activities. Test pits will be terminated at either bedrock/permafrost or the maximum reach of the excavator (3m). All test pits will be backfilled and compacted using the bucket of the excavator to closely match the existing grade at tes tpit locations.

Soil in the testpits will be examined by the Arcadis field technician upon recovery for purposes of soil characterization, which will include noting texture, colour, odour, moisture content as well as evidence of environmental impacts (odour and staining).

Samples from representative stratigraphy at each proposed borrow source will also be obtained. The first sample will be collected at a depth not greater than 750mm and succeeding samples at not more than 1500mm increments of depth. Select soil samples will be submitted to accredited laboratories for geotechnical testing.

6.4.2 Geotechnical Consultation During Design Process

The geotechnical recommendations provided herein to assist preliminary foundation and building design are general in nature as the design of the proposed structures has not yet been finalized. The recommendations will be reviewed by Arcadis prior to final design and construction to assess their applicability to the proposed structure. Site-specific foundation design recommendations will be required for components of the proposed structure.

6.4.3 Geotechnical Supervision During Construction

Development of the Site will require movement of a variety of soil and rock types as well as specialized foundation installations. A qualified geotechnical engineer must be retained to inspect and approve the subgrade prior to placement of any engineered media and to supervise the installation of foundations. Geotechnical supervision should also be provided to ensure that engineered fill placed beneath floor slabs, roadways and other applications is properly compacted and that any weak soil layers are properly removed. Geotechnical inspection of the bearing conditions for the proposed foundation system must also be carried out.

Geotechnical site supervision and review is required during future construction activities. It is recommended that the following material testing and observation program be performed by a licensed geotechnical engineering consultant during construction operations:

- Observation of all bearing surfaces prior to the placement of concrete/crushed stone/engineered fill;
- Sampling and testing of any concrete and fill materials used;
- Periodic observation of the condition of unsupported excavation side slopes, if applicable;
- Observation of all subgrades prior to backfilling;
- Field density tests to determine the level of compaction achieved, as applicable;
- Sampling, testing and verification of construction materials; and
- Verification of spec compliance and any necessary amendments to design specifications.

A report confirming that these construction works have been conducted in general accordance with geotechnical recommendations would then be issued following the completion of a satisfactory material testing and observation program by the geotechnical consultant. It is recommended that all footing excavations be inspected by competent geotechnical personnel to ensure that a proper bearing surface has been attained and that foundation designs are suited to site conditions.

7 Statement of Limitations

This draft iteration of this geotechnical report is not to be distributed, reproduced, or relied upon by any party. Further data and site investigation is expected, and the report will be expanded.

This report, prepared for City of Iqaluit, does not provide certification or warranty, expressed or implied, that the investigation conducted by Arcadis uncovered all potential geotechnical constraints at the Site. The conclusions and recommendations presented in this geotechnical investigation report are based on the information determined at the borehole locations. The information contained within this report in no way reflects the environmental aspect of the Site or soil, unless specifically reported upon. Subsurface and groundwater conditions between and beyond the test locations may differ from those encountered at the specific locations tested, and conditions may be encountered during construction which were not detected and could not be anticipated at the time of the site investigation. It is recommended that Arcadis be retained during construction to confirm that the subsurface conditions throughout the Site do not differ materially from those conditions encountered at the test locations.

The design recommendations provided in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not have been available at the time this report was prepared, it is recommended that Arcadis be retained during future stages of the design process to verify that the design is consistent with the recommendations of this report, and that the assumptions made in the analyses contained in this report are still valid. The need for additional subsurface investigation work and laboratory testing should be reviewed by Arcadis during the course of the detail design work.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer. The number of boreholes/ groundwater monitoring wells may not be sufficient to determine all of the factors that may affect construction methods and costs (e.g., the thickness of surficial topsoil and fill layers can vary markedly and unpredictably). Contractors bidding on the project or undertaking the construction should, therefore, make their own interpretations of the factual information in this report and draw their own conclusions as to how the subsurface conditions may affect their bid or work.

The material in this report reflects the best judgement of Arcadis based on the information available at the time of preparation, June 2025. Changes to soil and/or groundwater quality in the areas investigated can occur following the date of testing. Any use which a third party makes of the report, or reliance on, or decisions to be based on it, is the responsibility of such third parties. Arcadis accepts no liability, whether in negligence, contract or arising on any other basis for damages or from indemnification arising from decisions or actions by others based on this report.

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G – Thermistor Reports

Appendix H

H – Apex River Pump Station and Pipeline Crossing Investigation Report

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