



ATTACHMENT 15

LTWP Detailed Design Report

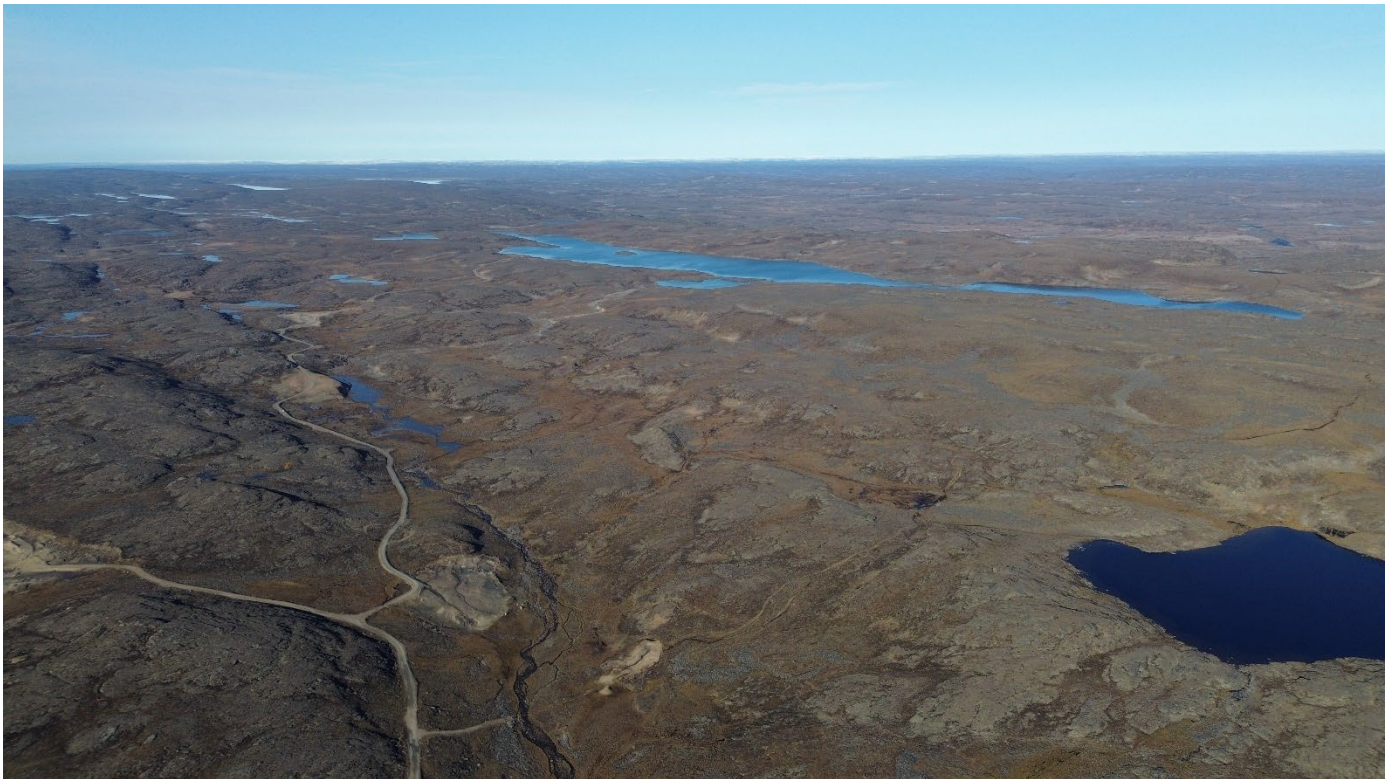
City of Iqaluit

Detailed Design Report

Long Term Water Program – Supply and Storage

DRAFT REPORT

January 2026



Detailed Design Report

Long Term Water Program – Supply and Storage

January 2026

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- H - Liner Material Technical Memorandum
- I - Stability Analysis Technical Memo
- J - Preliminary Blast Assessment Report
- K - Preliminary Systems Operation Manual
- L - Basis of Estimate
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1 Introduction

1.1 Background

The City of Iqaluit (City) is the newest Capital City in Canada and is rapidly developing into a regional center for the Territory with many northern businesses in Nunavut making it their base of operations. This has led to a rapid growth in population (3% - 4% growth rate expected annually). As a result, the Lake Geraldine raw water reservoir is no longer sufficient to supply or store the required amount of water needed to support potable water needs for the projected growth rate of the City. The project water requirement to the Year 2050 was determined to be 1,824,500 m³.

Additional water resources and storage are needed to sustain expected future demand. Based on the funding agreement between Infrastructure Canada and the City, Arcadis understands that the new reservoir storage must hold 1.5 times the current capacity of Lake Geraldine (noted as 1,095,500 m³ in the pre-feasibility study performed by others), which represents an additional 1,643,250 m³ of over-winter raw water storage required.

Over the last few years, the City has pumped water from the Apex (Niaqunngut, Apex River is used throughout) River (AR) to Lake Geraldine as an emergency measure to augment the over-winter storage requirements for the City. In 2019 the City also used Lake Qikiqtalik (LQ) for emergency supply. A system was put in place to use LQ in 2022 although it was ultimately not required.

Based on a final hydrological report in 2025 (by others) regarding available volume of water in the catchment areas to both LQ and the AR, it was determined following the completion of the original Preliminary Design Report (PDR) in 2024, that both LQ and AR would be required water sources to supply the City's needs until 2050. The long-term water supply project will utilize water from LQ and AR in a new reservoir adjacent to Lake Geraldine to augment storage in Lake Geraldine. The planned approach is to take advantage of the greater amount of water available during the Spring freshet followed by continuous pumping for an additional period during the open water season to fill the new reservoir in preparation for winter.

1.2 Project Scope

Arcadis is currently completing professional engineering and environmental services for the City's Long Term Water Program Raw Water Supply and Storage (LTWP – S&S). The project will be completed in several phases, as follows:

- Phase I: Project Definition and Concept Design.
- Phase II: Preliminary Design Report (30% Submission).
- Phase III: Design Development (50% Submission).
- Phase IV - Final Design (90 % and 100 % Submission Milestones).
- Permitting Requirements.
- Phase V-Tendering Support.
- Phase VI - Contract Administration, Site Inspection and Engineering Support Services, during construction.
- Phase VII - Closeout Phase.

The project scope includes:

- New raw water intake and pump station at LQ.
- New raw water intake and pump station at AR.

- Water conveyance pipeline:
 - Approximately 4 km two pipelines from the LQ pumping station to the new Reservoir.
 - Approximately 500 m two pipelines from AR pumping station to the new Reservoir – Route 1 and Route 2 – two short routes.
 - Another pipeline section 500 m from a junction chamber at one of the pipelines to LG and the total length approximately 1.1 km for this long route from AR to LG.
- New reservoir for raw water storage constructed using dams/dykes adjacent to the existing Lake Geraldine reservoir.
- Interconnection between the reservoirs with a service corridor including automated level control/flow.
- Access roads using existing roadways where possible and construction of new access roads to allow for maintenance and operations of all constructed components of the program.
- Utility power construction to the new facilities including pump station and reservoir (by the local utility QEC).
- Completion of background studies to support the design and construction including geotechnical and environmental.
- Permitting submittals to the Nunavut Impact Review Board (NIRB), Nunavut Planning Commission (NPC), Nunavut Water Board (NWB).
- Stakeholder consultation.

This report presents the results of 50% Detailed Design. Figure 1-1 provides an overview of the proposed infrastructure for this project. Pipelines, pump stations, new reservoir and existing water bodies are shown.

1.3 Project Objectives

The objectives of the LTWP – S&S are as follows:

- Provide long term sustainable raw water source for potable water for the City to the year 2050.
- Design and construct an infrastructure system that is durable to the conditions of the north.
- Design and construct an infrastructure system that is mindful of ease of operation and maintenance.
- Meet the requirements of funding stakeholders.
- Meet the requirements of permitting agencies.
- Engage stakeholders to educate and affirm the objectives of the program to obtain acceptance.

1.4 Functional Design Requirements

The LTWP – Supply and Storage program will be designed to provide a sustainable source of water for the City based on planning projections to 2050. There have been a number of background studies completed including feasibility of different water sources (e.g., Sylvia Grinnell River, Apex River, Unnamed Lake). These studies included water availability/quantity assessments, habitat assessments, bathymetry, water quality, and other evaluations based on both technical and socio-economic factors. Climate change was considered as well, including its impact on water supply and constructability (i.e., thawing permafrost impacts on foundations, roadways, etc.).

A general site plan of the project is provided in **Appendix A**, Drawing G101. Design requirements at the time of preparation of this report were:

- Total storage of raw water volume in the new reservoir = 2,080,100 m³
- Minimum useable overwinter storage volume = 1,643,250 m³
- Available useable overwinter storage of raw water volume in the new reservoir = 1,646,000 m³ (i.e. exceeds the minimum required)
- Reservoir filling time approximately 3-4 months. This will be achieved with peak pumping during the Spring freshet and lower pumping rates during subsequent months until the reservoir volume is reached. Pumping rates at LQ will be 556 L/s (two pumps running) during Spring freshet and approximately 278 L/s (one pump running) during lower flow months. As well as additional 400 L/s (two pumps running at 200 L/s) from the AR.

The new raw water supply will require pumping during the open water season to fill the new reservoir. Winter operations will include transfer of water from the new reservoir to maintain volume in Lake Geraldine. The new reservoir will be constructed adjacent to Lake Geraldine and will require a connection to Lake Geraldine, which will be conveyed via two pipes in a heated service corridor. Considerations for this connection include cold weather operation, access, and maintenance. The new reservoir will be designed to allow for storage of water in a manner that optimizes the intake of water during the freshet and larger rain events and maximizes the ability of the entire reservoir system throughout the year. Level monitoring will be implemented in Lake Geraldine and will be used for control of actuated valves on the connection pipes between the two reservoirs to maintain water level in Lake Geraldine.

A route for piping to convey raw water from LQ and was selected considering topography, access, constructability, expected cost, and environmental factors. Winter operations and maintenance (O&M) will include draining the conveyance pipeline.

Frazil ice build-up during winter is a consideration and the new lake intake into LQ from the pump station will be equipped with a compressed air system. A compressor and associated equipment such as pneumatic tank will be housed in the pump station for this requirement. The intake pipe will be insulated, and heat traced. For the AR

pump station, given the unique requirements for intakes in the river, and freezing temperatures, an intake is proposed that includes a screen that can be mechanically lifted out of the water for the winter season.

Power for the new pump stations and at the new reservoir (for control valves, heat tracing, etc.) has been planned assuming it will be installed by the local utility (QEC). Backup power considerations are also described in this report which will be needed to support freeze protection for the pump stations and pipes.

1.5 Environmental, Permitting and Consultation

Environmental, permitting and consultation aspects of this project are being completed by Arcadis. Reporting on these topics is provided under separate cover and therefore details are not included here. Reporting will include the following.

1.5.1 Environmental

The following environmental reports will be completed to support the project:

- Environmental Management Plan (EMP)
- Environmental Protection Plan (EPP)
- Erosion and Sediment Control Plan (ESCP)
- Environmental Site Assessment Phase 1
- Physical/Biological/Socio-Economic Impact Assessment
- Infrastructure Canada Greenhouse Gas Emissions Assessment
- Stakeholder and Permitting Communication Plan (SPCP)

1.5.2 Permitting

The following permits are required from the territory of Nunavut:

- Nunavut Planning Commission (NPC) - Project Proposal – application submitted December 2025
- Nunavut Impact Review Board (NIRB) - Screening Report
- NIRB Review Report – A three phase environmental assessment process consisting of:
 - Phase 1 Scoping and Guidelines from the NIRB.
 - Phase 2 Draft EIS from Proponent – All environmental studies to be completed at this milestone. The draft shall incorporate the designs identified at the Phase III 50% submission.
 - Phase 3 Final Environmental Impact Statement – The final EIS shall incorporate feedback from stakeholders.
- Nunavut Water Board (NWB) Water License – The City currently holds a Water Use License for its water use needs. A new permit will be required for the LTWP and an amendment for the water treatment facility to increase its maximum water intake.

Following consultations in September 2023 with the NIRB, it is likely that only a Screening Report will be required for the LTWP. This confirmation has been given as part of the NPC Project Proposal which was approved in 2025.

A Draft Physical, Biological and Socio-Economic Impact Assessment (PBSEIA) was been submitted for review by the City of Iqaluit and has been included in the NIRB submission made in December of 2025. It provides the environmental baseline of the LTWP site and, based on the information collected to date and designs available,

provides an impact classification, and recommend mitigation actions for the construction and operations and maintenance phases of the LTWP. Its format is based on the information requirements outlined in the *Proponent's Guide- NIRB Technical Guide Series* dated February 2020.

Design approvals and/or permits that may also be required include:

- City of Iqaluit Building Permit
- City of Iqaluit Site Plan or Rezoning
- Department of Fisheries and Oceans (DFO) for work in waterways and intake screen design criteria
- Local utility permits (e.g. power)

Other project permits may be required further to those listed above as consultations proceed and final field studies are undertaken.

1.5.3 Consultation

Consultation activities are outlined in our Stakeholder and Permitting Communication Plan.

Stakeholders that have already been consulted by the City are as follows:

- Nunavut Department of Culture and Heritage
- Nunavut Department of Environment
- Nunavut Department of Health
- Qikiqtani Inuit Association (QIA)
- Hunters and Trappers Association (HTA)
- Nunavut Tunngavik Incorporated (NTI)
- Department of Fisheries and Oceans (DFO)
- Crown Indigenous Relations and Northern Affairs Canada (CIRNAC)
- Infrastructure Canada (INFC)
- Environment and Climate Change Canada (ECCC)

Public Information Sessions were held in November of 2024 and March of 2025.

1.6 Background Documentation

Table 1-1 outlines background documentation provided by the City for this project.

Table 1-1 Reference Documentation

Documentation Received
RFP
Municipal Design Guidelines, City of Iqaluit – 2005
Good Building Practices Guideline, Government of Nunavut – 2020
Iqaluit Water Storage Pre-Feasibility Study (EXP 2020)
Comparative Evaluation of Sylvia Grinnell River and Unnamed Lake as Long-Term Water Supply for the City of Iqaluit (Nunami Stantec 2022)

Documentation Received
Water Balance Assessment for Unnamed Lake – Modelling Report (Golder 2021)
Review of Golder Associates Ltd. Unnamed Lake Water Balance Assessment Draft Report (Stantec Memorandum 2021)
Options Evaluation for Raw Water Supplementation from the Sylvia Grinnell River (Nunami Stantec 2019)
Conceptual Design Advancement for Raw Water Supplementation from the Sylvia Grinnell River (Nunami Stantec 2019)
Unnamed Lake Fish and Fish Habitat Assessment Technical Report (WSP 2021)
Fish and Fish Habitat Assessment of the Niaqunguk (Apex) River, Lake Geraldine, and the Lake Geraldine Drainage Channel (Nunami Stantec 2017)
Iqaluit DFO Bathymetric Lake Surveys (Tetra Tech 2019)
UNL Lidar Report (Aethon Aerial Solutions 2019)
Long-Term Water – UNL Water Quality Sampling Memorandum (Nunami Stantec 2019)
Long-Term Water – UNL Data Collection Summary Memorandum (Nunami Stantec 2021)
Iqaluit SCADA System User Manual – Reduced – Stantec 2020
Iqaluit Type A Water Licence – No 3AM-IQA1626
Iqaluit Type A Water Licence – No 3AM-IQA1626 (Amendment No.4)
RFI-001
Supplementary Lake Geraldine Water Balance Modelling for 2022 (Golder 2022)
Apex Supply Line As Built
Lake Geraldine Resupply (Apex River Supplementary Pumping Program) Report of Activities – Nunami Stantec 2022
Lake Geraldine Resupply (Apex Pumping) Report of Activities – Nunami Stantec 2021
Emergency Water Supply Project: Draft Report of Activities (Nunami Stantec 2019)
Lake Geraldine Resupply 2019 Field Review (Nunami Stantec, Aug 18, 2019)
Lake Geraldine Resupply 2019 Field Review (Nunami Stantec, Aug 23, 2019)
Lake Geraldine Resupply 2019 Field Review (Nunami Stantec, Sept 2, 2019)
Lake Geraldine Resupply 2019 Field Review (Nunami Stantec, Nov 15, 2019)
RFI-002
National Computer Aided Design and Drafting Standard (Public Services and Procurement Canada)
RFI-004
Iqaluit Wastewater Treatment Plant Upgrades IFC - General (Nunami Stantec 2018)
Iqaluit Wastewater Treatment Plant Upgrades IFC – Civil Site Plan (Nunami Stantec 2018)
Iqaluit Wastewater Treatment Plant Upgrades IFC – Architectural (Nunami Stantec 2018)
Iqaluit Wastewater Treatment Plant Upgrades Issued for Generator As-Built - Electrical (Nunami Stantec March 10, 2022)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT– Civil (Stantec 2023)

Documentation Received
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Architectural (Stantec 2023)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Structural (Stantec 2023)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Process (Stantec 2023)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Building Mechanical (Stantec 2023)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Electrical (Stantec 2023)
Lift Station No.1, Septage Receiving Station and Lower Iqaluit Sewer IFT – Instrumentation (Stantec 2023)
Generator O&M Manual
Apex Pumping Spill Contingency Plan (Qikiqtaaluk Environmental, Tower Arctic, May 22, 2020)
Apex River Erosion and Sediment Control Plan (Qikiqtaaluk Environmental, Tower Arctic, May 28, 2020)
Apex River Fish and Fish Habitat Plan (Qikiqtaaluk Environmental, Tower Arctic, June 5, 2020)
Addendum No.1 Watermain Replacement Details As Recorded (Trow Associates Inc., October 10, 2008)
Lake Geraldine Dam Underwater Repair Site Plan (Concentric, January 24, 2018)
OTHER
Lake Geraldine Dam Operations, Maintenance & Surveillance Manual (Meco, July 16, 2020)
Lake Geraldine Dam 2021 Dam Safety Review (Concentric, 16 June 2022)
Bridge Inspection Report – Final (Concentric, November 17, 2020)
Apex Bridge and Helen Maksagak Drive Culverts (Concentric, March, 2022)
Road to Nowhere Bridge (Concentric, March 2022)
Analysis of Fisheries and Hydrologic Information of Apex River (Nunami Stantec, April 21, 2023)
Supplementary Lake Geraldine Water Balance Modelling (Golder, July 2, 2019)
Supplementary Lake Geraldine Water Balance Modelling (Golder, May 15, 2020)
Supplementary Lake Geraldine Water Balance Modelling (Golder, May 7, 2021)
Supplementary Lake Geraldine Water Balance Modelling (WSP, May 2, 2023)
Iqaluit Hydrometric Monitoring (Tetra Tech, September 2023, updated March 2024)
Technical Memo – Desktop Study of Discharge in the Apex River (Tetra Tech, July 24, 2025)
Qikiqtaalik Lake Water Balance for Withdrawals – Final Report (Tetra Tech, July 24, 2025)

2 Field Investigations

This section summarizes field investigations that have been completed or will be completed in future to provide input to the design. These include:

- Site Topographical Survey (completed 2023-and updated 2024).
- Geotechnical and Hydrogeological Investigation (ongoing as of December 2025).
- Archaeological Investigation (completed by others in 2023).
- Biological Field Review (completed 2024).
- Hydrological Investigation (by Tetra Tech), including flow monitoring (2025).

2.1 Topographic Survey

Arcadis sub-consultant Adaptive Baseline Geotechnical (ABG) conducted topographic surveys of the existing site and environs in September – November 2023. The survey was used to prepare site plans of existing conditions and design base plans. ABG’s field engineer experienced with topographic surveys completed the topographic survey in the field using Topcon Hiper VR equipment. ABG’s engineering and field team worked remotely with a professional land surveyor to ensure the survey met all standard practices and is properly tied into multiple local survey monuments for Geodetic location and elevation. The survey covers the entire area of interest for the preferred pipeline and access road routes, and the reservoir area, based on input from the Arcadis team and an approximate 5 to 10 m grid spacing, extending at least 10 meters beyond the areas of interest in all directions. High/low points were picked up and the grid spacing densified or spread out to incorporate any existing line features (i.e., top/bottom of slopes, edge of gravel roadway areas, etc.). The topographical survey plan covered the area and the road with both contours at 0.25-meter intervals and surveyed points with their Northing, Easting and Elevation. The survey includes rock outcrops, watercourses, boulder fields, existing culverts/pipes (including sizes and inverts), ditches (including the bottom and top of slopes), power/communication poles/guys, road centerline/edges, existing buildings, and additional site features of interest to design/construction.

The field engineer who completed the topographic survey was well experienced with geotechnical investigations in the North, such that the topographic survey task was also site reconnaissance for the geotechnical field work component of the project. Areas of potential interest for the geotechnical boreholes and potential borehole locations identified as part of our survey. Photos of the areas of interest were captured and shared with the design team for consideration.

Further to this initial topographic survey work, Arcadis conducted an airborne LiDAR (Light Detection and Ranging) survey of the site in September 2024. This LIDAR survey generated a high-resolution, georeferenced digital elevation dataset, including both a classified point cloud and derived terrain model, using a Remotely Piloted Aircraft System (RPAS) controlled by a licensed operator at Arcadis. The survey included and expanded on the area covered as part of the ground-survey to verify topographic data and obtain data for the expanded reservoir footprint and access road design. The high-resolution LiDAR data was combined with the ground-survey data to generate the topographical surfaces used to inform detailed designs outlined in this report.

Several established coordinate control monuments were used as benchmarks to verify the accuracy of all survey data collected. Arcadis compared the ground-based and LiDAR survey data to publicly available 2022 contour GIS data from the Department of Community and Government Services of Nunavut to further verify its accuracy. Survey monuments and benchmarks are shown on detailed design drawings.

The final topographic survey report is included in **Appendix B**.

2.2 Geotechnical Investigation

The geotechnical investigation has been conducted over multiple mobilizations, from 2024 to 2025. A summary of the programs completed are presented the following table.

Table 2-1 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 in bedrock at the site.
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.
October 2025	Test pitting program of borrow sources, proposed pipeline corridor	Test pits were advanced by excavator to assess bedrock and permafrost depth and obtain further samples for testing at potential borrow sources and along the proposed pipeline corridor. At-depth soil samples were collected for geotechnical analysis to confirm soil properties. Test pit depths were used to improve soil volume estimates of potential borrow source areas across the Site. Additionally, soil samples were collected from the potential borrow source occupied by the Shooting Range to assess soil for potential contamination resulting from range activities.

The findings of the geotechnical investigation programs are outlined in the *Preliminary Geotechnical Investigation Report* by Arcadis dated 19 August 2025 in **Appendix C**. The geotechnical investigation programs are ongoing with future work planned for 2026. The report noted lists the further recommended geotechnical work.

A Blast Assessment Report for the project was completed by Explotech Engineering Limited (Explotech). The report was finalized in 2025 and is included in **Appendix J**.

2.3 Archaeological Investigation

An Archaeological Overview Assessment (AOA) and an Archaeological Impact Assessment (AIA) were completed by AECOM in 2023. The AIA investigation was focused within a 100 m radius of the preliminary Project area which at the time included two pipeline options (1 and 2), two access roads (Eastern and Western), and the new reservoir.

Archaeological Sites were identified as follows:

- KkDn-54 is a single stone meat cache approximately 130 cm by 180 cm consisting of 45 stones. No faunal remains or other culturally significant features exist within the cache. It is located approximately 40 m northeast of the Eastern Access Road along the Apex River.
- KkDn-55 consists of two stone cairns containing 20+ stones on a slightly raised beach. The cairns are approximately 3 m apart from each other and are around 150 cm to 180 cm in diameter each. KkDn-55 is located to the southeast of KkDn-54 along the Apex River and is approximately 45 m northeast of the Eastern Access Road.
- KkDn-56 is a campsite consisting of a tent ring and hearth that may have been recently disturbed by activity on a nearby ATV trail. This site is in between Pipeline Option 1 and Option 2 on a ridge east of the access road to Unnamed Lake.
- KkDn-58 is a single stone cairn consisting of 10 boulders and is 1.25 m in diameter. Archaeologists are unsure of the exact age of the feature. However, the presence of a large amount of moss on the stones suggests that it is prehistoric in age. No faunal remains or other culturally significant features exist within or around the cairn. KkDn-58 is located within the footprint of the new reservoir.

Contemporary Land Use Sites:

- One contemporary land use site was noted within the buffer for the new reservoir to the north. It consists of a stone meat cache that currently contains remnants of caribou such as ribs and vertebrae.
- A wind break was noted within the buffer between Pipeline Option 1 and the new reservoir, north of the intersection between Pipeline Option 1 and the Western Access Road.

No paleontological resources were identified in the reports provided by AECOM.

Sites that will be in the footprint of construction activities will require assessment by a professional archaeologist so they can be altered, relocated, or removed. It is likely that a permit from the Government of Nunavut to relocate or record the site prior start of construction will be needed. The City's archaeological consultant will be applying for this permit. It is recommended that the assessment be completed in the summer of 2024.

2.4 Initial Site Reconnaissance

A site reconnaissance was undertaken from September 19-23, 2023, by Arcadis subject matter experts (SME). These included:

- Conveyance design lead.
- Pumphouse design specialist.
- Geotechnical engineer.
- Terrestrial biologist, and
- Permitting specialist.

Prior to arrival at the site a desktop analysis was undertaken of the literature available to locate into the project GIS database the various design options provided in the RFP annexes. Also include were the publicly available datasets including areal images, elevation contours, water bodies and land use. The preliminary pipeline locations were included for evaluation and the available centrelines of the planned new reservoir dams and dykes. This information was brought in printed form into the field for evaluation.

Arcadis SMEs toured the LTWP mostly by foot in the accessible areas of the LTWP which included:

- Lake Qikiqtalik.
- The Emergency Road and its culverts.
- The Road to Nowhere and its culverts.
- The Bridge to Nowhere.
- The emergency pump station at the Apex River.

- The path of the temporary pipeline from the Apex River to Lake Geraldine.
- The circumference of the two lakes forming the New Reservoir, and
- The circumference of Lake Geraldine.

Other areas of interest that were toured in the reconnaissance visit were:

- Quarries west of the City.
- Borrow material locations near the City.
- Lake Geraldine Dam.
- General locations of municipal services (hospital, RCMP, fire station).

Photos were georeferenced and uploaded to the project GIS platform for storage. The information collected onsite was used in the development of the LTWP.

2.5 Initial Biological Field Review

As part of the site reconnaissance from September 19-23, 2023, an Arcadis permitting specialist and terrestrial biologist made the following high-level observations:

- Generally, it was too late in the season to observe birds, terrestrial wildlife, presence of fish and any flowering vegetation.
- Arctic ground vegetation is present across the undisturbed areas of the project area. This vegetation is typically slow growing and may take decades to regenerate following disturbance. Important to the environmental protection planning will be to limit ground disturbance caused by the construction activities when possible.
- Various berries and other edible terrestrial plant species were observed growing on the site. Consultations will focus on the use of these areas for traditional uses however the City noted they are not aware of any specific traditional foraging areas.
- Bones (likely from Caribou) were observed at the area of the planned pumphouse, pipeline paths and new reservoir. No Caribou were observed during the site reconnaissance.
- Wetlands were observed at various locations where construction is planned. In areas where construction will impact the wetland mitigation measures will require review. Of importance will be to maintain the hydrologic functions of these areas.

Final field work and consultation was completed in July 2024 to review vegetation, nesting birds, wildlife, fish and fish habitat assessment, species at risk, and traditional land uses (Inuit Qaujimagatuqangit). Separate reports were submitted under separate cover documenting findings and conclusions. In general, no major impacts or mitigations are recommended other than requirements for seasonal protection of certain species as noted in the report. These reports were “Fish and Fish Habitat Report (July 2025)” and “Terrestrial Wildlife and Habitat Report (May 2025)”.

2.6 Hydrological Field Investigation

Depth estimates of the ponds onsite have been modelled using the LiDAR and photogrammetry data obtained during the drone surveys conducted in 2024. Surface water volume estimates and dewatering recommendations are discussed in Arcadis’ Preliminary Geotechnical Investigation Report dated 19 August 2025 in **Appendix C**.

Site hydrology is discussed in detail in Arcadis Hydrology Technical Memorandum, included as **Appendix G**.

3 Raw Water Volume and Sources

3.1 General

To meet the City's water needs to the year 2050 up to an additional $\approx 1,824,500 \text{ m}^3$ of raw water will be necessary annually beyond what can currently be stored in Lake Geraldine. The new reservoir will have total storage of $2,080,100 \text{ m}^3$, with $1,643,250 \text{ m}^3$ minimum over-winter volume (i.e. useable volume assuming a 2-m ice surface in winter). It is assumed that some water from the new reservoir will be utilized to fill Lake Geraldine as necessary in summer, while the majority of the new reservoir storage will be used to maintain level/volume in Lake Geraldine in the winter months.

The total volume of 1.8M m^3 water will be supplied by both LQ and AR. It will be pumped during the annual open water season which can last up to five months each year or be as short as three months. The water will be stored in the new reservoir.

Tetra Tech (TT) has estimated in their report Qikiqtalik Lake Water Balance for Withdrawals¹, that LQ is projected to have a median value of $1,681,000 \text{ m}^3$ available for withdrawal and a mode value of $1,284,184 \text{ m}^3$ available. Based on the report's probability distribution function, the 0th percentile volume was $767,189 \text{ m}^3$ and the 10th percentile was $1,114,609 \text{ m}^3$. These low percentiles represent unusually low lake volumes, which would largely be caused by low precipitation and would thus result in less available water for pumping – i.e. a conservative estimate using a "low flow" year. The 0th percentile value was taken and rounded to $800,000 \text{ m}^3$. This number represents what could be withdrawn from LQ in a low-flow condition, without depleting the lake's storage.

As the LQ volume is not sufficient to supply the City's long term needs an additional volume of water is required to be taken from AR of $\approx 1,024,500 \text{ m}^3$. Based on our analysis, AR should be able to provide $\approx 1,000,000 \text{ m}^3$ in a low flow year. Because both bodies of water are geographically close they are subject to the same weather conditions; if one water body experiences a low flow year it is very likely the other one will too.

3.2 Lake Qikiqtalik

The pump station design assumes a conservative scenario of open water season length (i.e. three months or 92 days between June and September). To accommodate the required volume the pump station would require a minimum rated capacity of 139 L/s. However, the pumped flow capacity was further evaluated based on the need to take advantage of the spring freshet. Thus, the station was sized to pump 60% of the potential $1,100,000 \text{ m}^3$ of raw water ($660,000 \text{ m}^3$) during the month of June (30 days) with the remainder of the water ($440,000 \text{ m}^3$) pumped during the open water season.

To accommodate the $660,000 \text{ m}^3$ during the month of June (30 days) the pump station would require a rated capacity of 255 L/s. The design capacity of 255 L/s will be achieved using two pumps running in parallel, with a third standby pump.

During the remainder of the season, the pumps will be turned down to 170 L/s to accommodate the $440,000 \text{ m}^3$ in the 62 remaining days. Should less water be available in a given year, the pumps would be turned down accordingly to meet the DFO requirements.

Further explanation of the hydraulics for the pipeline and the required flow is outlined in Section 4. Table 3-1 summarizes the design criteria for the RWPS.

¹ TetraTech, Qikiqtalik Lake Water Balance for Withdrawals, July 24 2025

Table 3-1 Lake Qikiqtalik Design Criteria

Description	Value	Reference
Pump Capacity (Design)	255 L/s	Design review meetings with the Project Team

3.3 Apex River

For the AR pump station to meet the approximate 1,000,000 m³ desired, a design maximum flow of 400 L/s was determined by analysis of the AR flow from historic data, and pumping limitations imposed on the project by regulatory bodies (NWB, DFO).

3.3.1 Apex River’s Flow

The Apex River is a wide, shallow river. During the spring melt, it experiences very high flows which reduce until the water is very low in late summer. In the coldest temperatures, the river freezes to the bottom. It flows north to south and discharges to the ocean near Apex. Pumping will be located approximately 500 metres upstream of the Road to Nowhere river crossing.

To understand the river’s flow in detail there two Water Survey Canada (WSC) water monitoring stations.

- 10UH015: this station is located very close to the proposed pumping location, and has 6 years of real time data, 2019-2025. The catchment area is approximately 3,600 ha.
- 10UH002: this station is located near Apex, and therefore several kilometres downstream of station 10UH015. Its daily data spans from 1973-2021, and its real time data from 2011-2024. The catchment area is approximately 5,400 ha.

In order to understand the flows at 10UH015 for a longer time period, a relationship was established between the two flow monitoring stations using the overlapping time range of 2019-2024. The real time data were used for both for consistency. The objective was to get scaling factors that could be applied to the longer historical data for 10UH002, meaning that daily scaling factors were required. Real time data was aggregated into daily data by taking the average for each day. For each day of the year, a scaling factor for the relative flow of 10UH015/10UH002i was computed. The average scaling factor for each month was also calculated, as noted in Table 3-2.

Table 3-2 Scaling Factors for Relative Flow upstream and downstream of proposed pumping location at Apex River

Month	Scaling Factor	Variance
June	0.89	0.085
July	0.67	0.036
August	0.62	0.030
September	0.56	0.051

The scaling factors were applied to the daily data set from 10UH002 to create a modified “synthetic” dataset of relative flows for 10UH015 that spanned from 1973-2021. The 10UH002 data was entirely missing 1984 and 1996-2005. In addition, many years were missing large amounts of data. Due to the missing data noted, the final dataset included the years noted in Table 3-3.

Table 3-3 Years of Data included in Synthetic Dataset

1980s	1990s	2000s	2010s
1982	1990	2007	2010
1983	1991	2008	2011
1985	1992	2009	2012
1986	1994		2013
1988			2014
1989			2015
			2016
			2017

In 1989 the WSC transitioned from manual readings to digital readings. It is possible that the manual readings prior to 1989 may have been less accurate.

3.3.2 Pumping Constraints

The main pumping constraint is the amount of water that can be withdrawn from the AR, based on regulatory requirements in place to protect the river.

The current water license, as of 2024, has the following stipulation:

May draw up to **10%** of instantaneous flows, and only when the flow in the river is greater than **30%** of the mean annual discharge.

The TT technical memo Desktop Study of Discharge in Apex River² found that these limitations would not allow sufficient withdrawals to meet Iqaluit’s water flows. A DFO memo stated that the following conditions would be acceptable:

May draw up to **20%** of the instantaneous flows, when the flow in the river is greater than **33%** of the mean annual discharge.
 May draw up to **10%** of instantaneous flows, when the flow in the river is greater than **30%** of the mean annual discharge.

TT used the DFO conditions for their assessment and Stantec is currently using the DFO condition for the emergency pumping.

The City will be requesting the latter DFO conditions in the application for an amended water license going forward.

A key part of the limitations is the definition of the mean annual discharge, or MAD. Various sources state different values for the MAD. This discrepancy would theoretically cause concern, because a lower MAD would allow for pumping at lower river flows. However, the realistic lower bound for pumping at the pump station will always be greater than the MAD threshold, meaning that changes in the MAD will not affect the pumping configuration.

² Tetrattech, Desktop Study of Discharge in Apex River, July 24, 2025

For the purposes of pumping, the main factor is the 10% or 20% of instantaneous flow that can be withdrawn. The river flows and the pumping limitations were overlaid to run pumping scenarios.

3.3.3 Pumping Scenarios

Pumping scenarios were run for the entirety of the synthetic dataset. They were conducted by comparing each day's flow against the pumping limitations and getting an allowable pump rate for each day. Various pumping configurations were tested by setting hypothetical upper and lower bounds on the pump rate, which better mimics real-world operations; in practice, a pump station will likely not be capable of pumping both 5 L/s and 1000 L/s without excessive cost. The configurations run were:

- 50-150 L/s
- 100-200 L/s
- 100-250 L/s
- 100-300 L/s
- 100-350 L/s
- 100-400 L/s
- 100-500 L/s

The 100-350 L/s scenario only had one year which would not have resulted in the desired annual withdrawal volume of 1,000,000 m³, which was 1983 (also note that TT considered this year to be low quality data). 1983 yielded 983,790 m³.

To balance the desire for an appropriately sized pump, while limiting oversizing, and to include a factor of safety, a design maximum pump flow of 400 L/s was selected. The maximum flow will be achieved using two duty pumps rated at 200 L/s each, with one standby pump available. Based on an expected maximum turndown of a variable frequency drive (VFD), a lower pumping rate of 60 L/s (per pump) is possible. With a low end pump rate of 60 L/s each year analyzed had sufficient annual withdrawal.

A final consideration was then added to the pumping scenarios, which is the pumping season. Due to the cold climate conditions and the screens required to be submerged in the river for fish protection a winterization (and subsequent Spring startup) process will be required. In selecting probable dates for the pumping season the end of the season was chosen to be September 30, after which the river is expected to begin freezing. For the start of the season, two dates were reviewed; June 1 and June 16. By June 1 the river will likely have begun flowing, and data showed high flows as a result of the spring freshet (spring melt). However, based on anecdotal feedback we understand the water flows begin on the surface of the river as the sun melts the ice and snow in the catchment reaches the river. Practically, these flows are inaccessible. June 16 was selected because according to a report regarding Lake Geraldine³, June 16 is the latest date after which the ice will have melted. It is possible that after June 16 there may still be floes of ice going down the Apex River, however, we have assumed that generally the entire cross section of the river is expected to be flowing by then, allowing for placement of equipment and drawing of flows at any depth. June 16 - September 30 was selected as the design pumping duration for these reasons.

For a pump range of 60 L/s to 400 L/s, from June 16 to September 30, the volumes that would be pumped based on historical data are shown in Figure 3-1 and Table 3-4.

³ Technical Memorandum, Supplementary Lake Geraldine Water Balance Modelling for 2021; May 7, 2021, Golder (member of WSP)

Figure 3-1 Pump Volumes, by Month

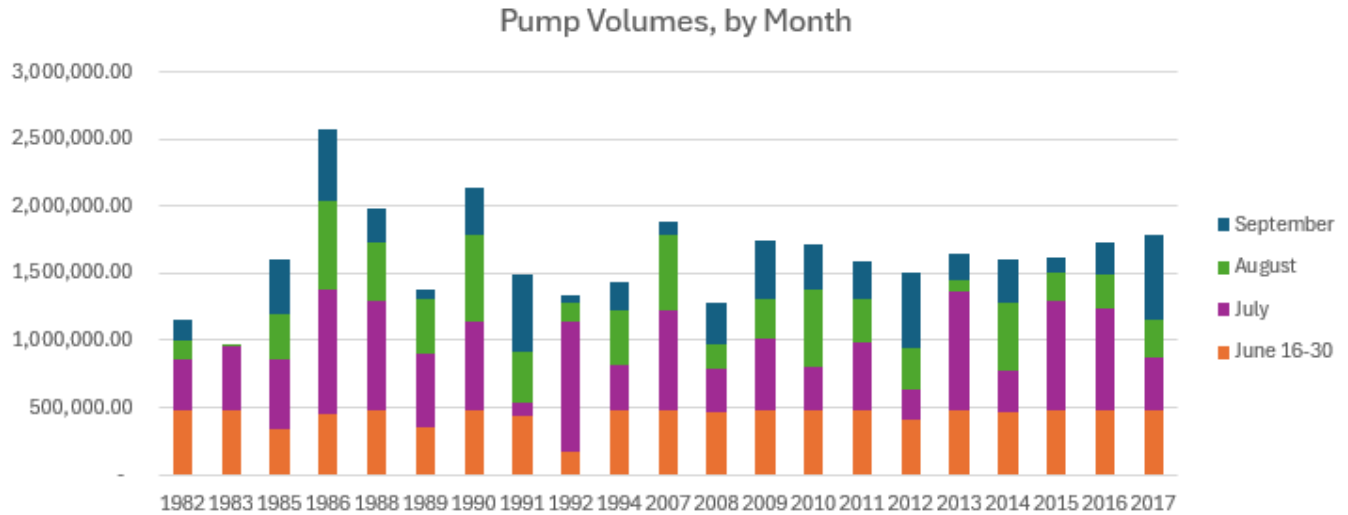


Table 3-4 Pump Volumes by Year

Year	Pumped Volume, L/s
1982	1,158,196
1983	965,491
1985	1,598,425
1986	2,567,615
1988	1,976,540
1989	1,376,357
1990	2,144,566
1991	1,493,017
1992	1,338,777
1994	1,433,403
2007	1,882,149
2008	1,286,906
2009	1,739,976
2010	1,719,685
2011	1,586,657
2012	1,509,276
2013	1,641,491
2014	1,601,089
2015	1,615,165
2016	1,726,163
2017	1,791,206

The majority of the pumped volume will be in June and July, which reflects the high river flows due to the freshet. The estimated percentages of pumped volume that could be drawn in each month, based on historical data, are displayed in Table 3-5. The preliminary AR design criteria for pumping is noted in Table 3-6.

Table 3-5 Monthly Average of Pump Volume

Month	Average Pumped Volume (m ³)	%, month
June 16-30	381,343.76	30%
July	392,812.33	31%
August	264,293.39	21%
September	217,763.91	17%
Total	1,256,213.40	100%

Table 3-6 Apex River Design Criteria

Description	Value	Reference
Pump Capacity (Design)	400 L/s	Design review meetings with the Project Team

4 Lake Qikiqtaalik Raw Water Pump Station

4.1 General

The LQ Raw Water Pump Station (RWPS) is designed as a wet well pump station that includes a building structure that houses the following:

- A raw water intake well (wet well) with three submersible pumps (two duty and one standby), discharge piping, ultrasonic level sensor, high-level and low-level float.
- A slide gate on the intake pipe entering the wet well to isolate the inlet flow.
- A valve room designed to house station piping, valves, and flow monitoring equipment.
- A wet well access room, above the wet well, with a two-tonne monorail and manual trolley and hoist for the pumps.
- An electrical and control room for the motor control centre (MCC), electrical panels and instrumentation and control (I&C) panels, network closet, compressors and air receiver tanks, and a washroom.

4.2 Pump Station Design Features

The RWPS was designed within the context of the site and the location in Iqaluit. The site is remote from the City and therefore remote control (through SCADA), durable building features, systems to maintain temperature for equipment, and ease of maintenance were in the forefront.

The main design features of the RWPS include:

- Raw water intake to LQ, insulated and heat traced.
- Sub-grade wet well for three submersible centrifugal pumps (two duty and one standby).

- Ultrasonic level system that monitors the wet well water levels.
- Back-up float alarm system to alert operations staff in the event of low wet well water level.
- Isolated valve room adjacent to the wet well to house valves, piping, and electromagnetic flow meter on the discharge header.
- Wet well will have lighting, heating and ventilation and a manually operated two tonne hoist and trolley to remove the pumps.
- Valve room with lighting, heating and ventilation. The valves can be lifted and moved to the floor of the valve chamber using the monorail, then lifted to the electrical and control room using a portable hoist, with a hook in the ceiling over the hatch.
- At grade Electrical and Control Room for the MCC panels, Programmable Logic Controller (PLC) and control panels, network closet, variable frequency drives (VFDs) for the raw water pumps, compressors, electric heating units, ventilation, and lighting.
- Outdoor standby diesel-powered generator set with automatic transfer switch, and the ability to connect a portable emergency generator in the event that backup power fails.
- Three phase primary power supply to the pump station building complete with a pole mounted transformer (to be determined by Utility) for 600 V supply to the motor control centre.
- A washroom consisting of a waterless toilet and portable handwashing unit (washroom will be accessed from electrical room).
- An intake screen air burst system comprised of two air compressors (duty/standby) in an acoustic enclosure and a pressure tank for the intake. The Electrical and Control Room will house electrical for the intake heat tracing.
- An indoor air handling unit will provide ventilation air into all spaces except the wet well, serviced by a separate blower heater and exhaust fan.
- Additional electric heaters will be provided in each space, and one baseboard heater will be provided inside the washroom.

Refer to pump station drawings in **Appendix A**. Draft specifications are included in **Appendix D**.

Items that are critical to station operations will be identified as a list of recommended spare parts (and included in the design specifications) for storage at a City works facility.

4.3 Intake

4.3.1 Pipe

The intake at Lake Qikiqtalik comprises of a pump station on the west shore of Lake Qikiqtalik and a nominal 600 mm High Density Polyethylene (HDPE) DR 11 raw water inlet line extending out into Lake Qikiqtalik, into a minimum of eight metre water depth and running along the lake bed to the wet well. The water intake is set two metres from the bottom of Lake Qikiqtalik, to minimize silt and sand from entering the pipe. The intake is at a minimum depth of six metres below the water surface allowing for two metres of ice and four metres of water over the intake when the water level in the lake is at the lakes' outlet channel invert of 202.10 m.

A bypass connection will be provided in the event that the intake is out of service or requires maintenance. This would allow dropping of submersible pumps in the lake and a connection to a blind flange at the valve chamber downstream of the station for temporary pumping.

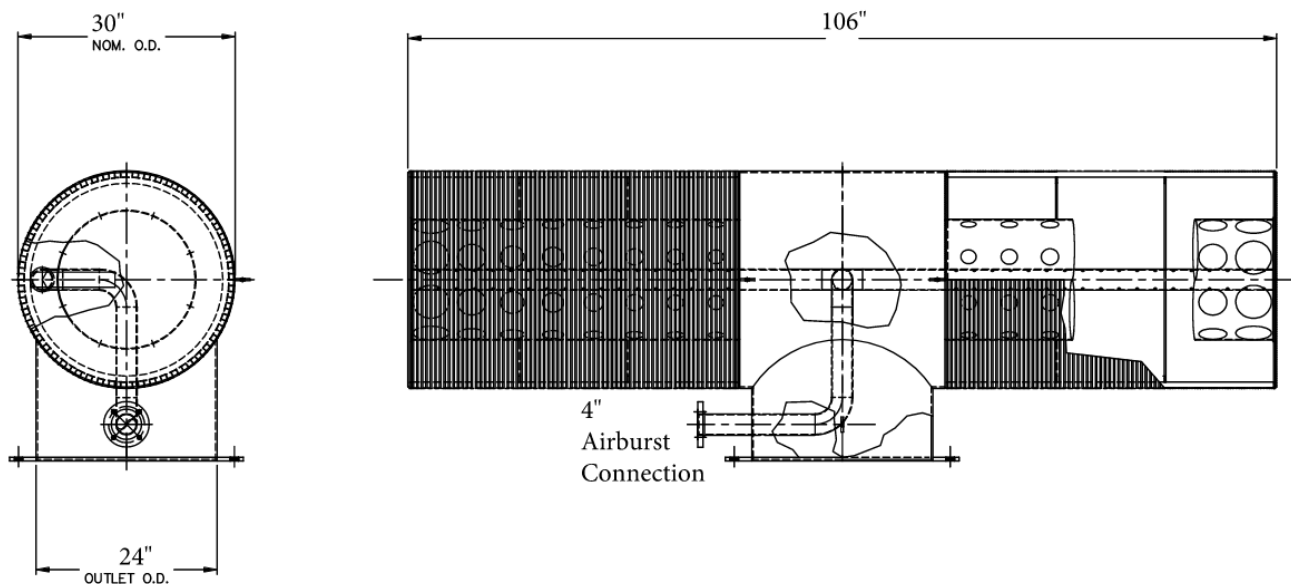
4.3.2 Screen

The intake screen is sized on accordance with the Department of Fisheries and Oceans (DFO) Freshwater Intake End-of-Pipe Fish Screen Guidelines for the fish that may be present in Lake Qikiqtalik. Previous studies identified the presence of Arctic Char in Lake Qikiqtalik however, Ninespine Stickleback may also be present. Arctic Char and Ninespine Stickleback have a subcarangiform mode of swimming. Using a screen approach velocity of 0.11 m/s per the DFO guidelines, a screen with a clear area of 2.38 m² is required for a flow rate of 255 L/s to prevent entrainment and impingement. The screen openings will be no larger than 2.54 mm in accordance with the DFO guidelines.

The screen is fitted with an air scour that is operated by an airline running in parallel with the raw water line and connected to a compressor located in the electrical and control room of the pump station. The air scour is designed to operate at programmed, adjustable intervals in winter to mitigate frazil ice or in bursts to control biofouling.

The raw water intake screen is designed for a flow of 255 L/s with an inlet flow velocity of 0.11 m/s (0.36 fps). The screen is a “TEE” type, 762 mm (30 in.) in diameter, with an overall length of approximately 2.69 m (106 in.) long with #69 wire screen providing a slot size of 2.54 mm (0.1 in.). The screen is equipped with a 600 mm (24 in.) flanged outlet and a 100 mm (4 in.) airburst connection. All material is 316 stainless steel (SS). Refer to Figure 4-1 for a general layout of the intake screen.

Figure 4-1 Intake Screen



4.3.3 Airburst System

The airburst system consists of two (one duty, one standby) 60.4 L/s (128 cfm) rotary screw compressors each with 575/3/60, 22.37 kW (30 hp) TEFC motor, aftercooler, oil/moisture separator, graphic controller, and sound attenuating enclosure. Each compressor is sized to charge the 4013 L (1060 gallon) receiver from 0 kPa to 1035 kPa (0 psi to 150 psi) in 9 minutes. The receiver is equipped with a pressure switch, pressure gauge, safety valve and auto drain valve and one – 18.9 L (five gallon) 1379 kPa (200 psi) rated horizontal receiver with check valve, pressure gauge, safety valve and manual drain valve. The receiver will have a 150 mm (6") lug style butterfly valve ductile iron body, SS disc & stem, BUNA rubber seat with pneumatic rack and pinion fails close actuator with limit switches, visual position cone, direct mounted solenoid with manual override.

The air system package includes a control panel that will be connected to the RWPS control system for monitoring.

4.4 Process Mechanical

4.4.1 Piping

4.4.1.1 Intake Size and Class

The intake is a 600 mm diameter HDPE DR 11 (inside diameter of 529.1 mm) pipe. The flow velocity in the intake with a pump flow of 255 L/s is 1.2 m/s.

4.4.1.2 Internal Station Piping

Each pump discharge line is 300 mm in diameter and is equipped with a swing flex check valve and a knife gate valve. The common discharge header is equipped with a 350 mm diameter magnetic flow meter, pressure sensor and transmitter and combination air valve. All piping within the building is 304L stainless steel with stainless steel flanges.

4.4.1.3 Discharge Pipe (Forcemain) Size and Class

The pumps are sized for the design flow of 255 L/s. The discharge pipe is sized as a 400 mm diameter HDPE DR 11 (inside diameter of 356.8 mm). The flow velocity with a pump flow of 255 L/s is 2.55 m/s.

4.4.1.4 Discharge Pipe Drain Line

In the case of pipe failure, it may be necessary to drain a portion of the discharge pipe back into the wet well. The portion of the discharge pipe within the valve chamber is equipped with a 150 mm diameter drain line complete with an isolation valve to drain the discharge pipe back into the wet well. The isolation drain valve is equipped with a manual handwheel operator accessed from the valve chamber.

4.4.1.5 Discharge Pipe Route and Installation

The discharge pipe directs the raw water pumped from Lake Qikiqtalik to the reservoir located next to Lake Geraldine, approximately 3.9 km southwest of the pump station. The plan and profile drawings are provided in **Appendix A**.

4.4.1.6 Surge Pressure Calculations

A preliminary transient analysis was performed (see Section 6.5 for details). The surge pressure at the pump station is approximately 903 kPa (132 psi). Evaluation of surge pressure valves is ongoing and if necessary will be added to the station during the next phase of design.

4.4.1.7 Combination Air and Vacuum Release Valve

Short-lived full-vacuum or sub-atmospheric pressure may occur at the discharge side of the pump station.

A 25mm diameter sewage combination air and vacuum valve (SCAV) for raw water is installed on the 350 mm diameter common pump discharge header to release entrapped air and provide pipeline vacuum protection. The vacuum release valve allows air into the pipe to facilitate draining the piping during maintenance and the fall shutdown. Due to the nature of raw water, the SCAV must be cleaned and maintained regularly by the operation staff. The SCAV operation and maintenance manual indicates that the SCAV should be scheduled for regular

inspection and backwash monthly. Based on Operation Staff service experience, this backwash regimen can be adjusted to suit actual in use experience.

4.4.2 Wet Well

4.4.2.1 Design

The supportive design calculations for the pump station are enclosed in **Appendix E**. Section 4.4.4 provides a summary of the design calculations and pump selection.

4.4.2.2 Inlet Pipe Slide Gate

An inlet slide gate is provided on the wall of the wet well to isolate the wet well from the intake pipe in Lake Qikiqtalik for ease of maintenance of the wet well.

4.4.3 Valve Room

The valve room is located below grade and houses the discharge piping and valving for the pumps. Each pump is equipped with a swing check valve to prevent backflow, and a knife gate valve for isolation. A magnetic flow meter is located on the discharge pipe prior to exiting the station. A monorail is provided for lifting of the valves for maintenance along with an equipment access hatch to the ground floor.

4.4.4 Raw Water Pumps

4.4.4.1 Head Loss Calculations

The system head curve is developed by calculating the total dynamic head (TDH) as the sum of static head and friction losses in pipes and fittings. The following is a summary of the calculation methodology for the pump station. Refer to **Appendix E** for the detailed design calculations.

4.4.4.1.1 Static Head

Static head values are based on the proposed operating levels in the wet well (between 201.26 m and 202.96 m) with two pumps running in parallel and on the highest point in the discharge pipe. The invert of the pipe at the highest point in the discharge line at VC1 is 212.25 m.

Three different static heads were considered in developing the system head curve: the lowest static lift, the mean static lift, and the highest static lift. These values represent the range of static lifts against which the pump will operate. The lowest static lift is associated with the high-water level in the wet well, the mean static lift is associated with the average liquid level and the highest static lift is associated with the low water level. Refer to **Appendix E** for the detailed design calculations.

4.4.4.1.2 Friction Losses

Friction losses are calculated using the Hazen-Williams equation as follows:

$$S = \frac{h_f}{L} = \frac{10.67 Q^{1.85}}{C^{1.85} d^{4.87}}$$

Where:

- S = Hydraulic slope
- h_f = head loss in meters (water) over the length of pipe
- L = length of pipe in meters

- Q = volumetric flow rate, m^3/s (cubic meters per second)
- C = pipe roughness coefficient
- d = inside pipe diameter, m (meters)

Note: pressure drop can be computed from head loss as $h_f \times$ the unit weight of water (e.g., 9810 N/m^3 at 4°C).

The following three 'C' values were used to develop system head curves and assess system characteristics at various flows throughout the system design life:

- $C = 120$ in conjunction with the highest static lift (HSL) to calculate the worst-case system scenario, as may occur in the future after long term use of the discharge piping.
- $C = 140$ in conjunction with the mean static lift (MSL) to calculate the system curve that is most likely to occur during normal pumping operation.
- $C = 150$ in conjunction with the lowest static lift (LSL) to determine system characteristics during initial start-up. This will ensure that the pump can operate at low head and avoid run-out conditions.

The system curve with $C = 150$ was used to select a suitable pump. However, the pump performance curve must cross all three curves to ensure adequate operation under different hydraulic conditions.

In addition to friction losses in the pipes, local losses that occur in valves and fittings are calculated using the following equation:

$$\text{Minor Losses in Piping} = h_m = k (V^2/2g)$$

Where:

h_m = Minor Losses in Piping because of fittings

K = friction constant

V = Velocity of fluid in the pipe

g = acceleration due to gravity = 9.807 m/s^2

4.4.4.1.3 Total Dynamic Head (TDH)

The TDH is equal to the sum of the static lift, friction loss in pipes as well as local losses. The three scenarios that define the hydraulic envelope of the system are:

Worst-case scenario with maximum static lift: $\text{TDH} = \text{HSL} + h_f (@ C=120) + h_L$

Normal operating conditions with average lift: $\text{TDH} = \text{MSL} + h_f (@ C=130) + h_L$

Start-up conditions with lowest static lift: $\text{TDH} = \text{LSL} + h_f (@ C=140) + h_L$

Calculations were completed for the 350 m diameter discharge pipe at different static lifts and a system curve was generated. Please refer to **Appendix E** for the detailed design calculations.

4.4.4.2 Pump Selection

The pump design information is provided in **Appendix E**. Based on the system curve and duty requirements, the required pump capacity is set at 128 L/s each.

Preliminary pump performance curves and operation and maintenance manual from Flygt are enclosed in **Appendix F**.

Pump criteria can be seen in Table 4-1.

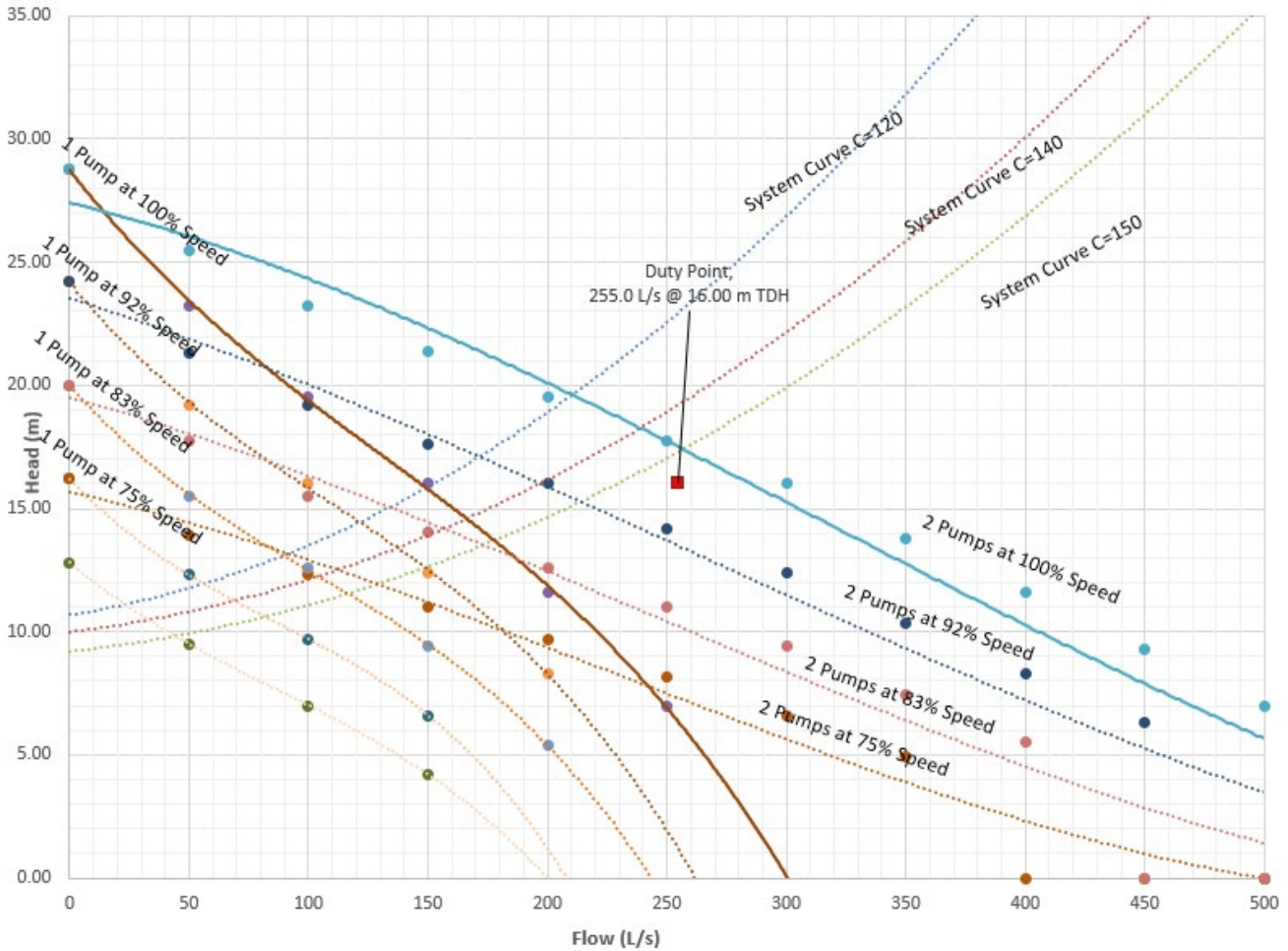
Table 4-1 Pump Criteria

Parameter	Criteria
Performance - Pump Duty Point	128 L/s @ 16 m TDH (with VFD pumps will operate near the best efficiency point.
Duty Pumps	2
Standby Pumps	1
Type	Submersible centrifugal pumps with non-clog impellers
Cost Effectiveness	Submersible pumps are cost effective for this application. Alternative pump types (e.g. vertical turbine pumps) are not recommended for this application and remote location due to the need for a crane to remove the pumps through the roof.
Power Requirement - Power Rating	45 kW (60 hp)
Variable Frequency Drive (VFD)	Yes
Reliability of Operation	Flygt submersible pumps have historically been very reliable.
Serviceability and Service Life	Service Life is typically 15 years
Maintenance and Safety	Pump maintenance schedule is recommended at intervals of 12,000 hrs or three (3) years. Refer to the Installation, Operation and Maintenance Manual and Service and Repair Instructions enclosed in Appendix F.

4.4.4.3 System and Pump Curve

Figure 4-3 outlines the calculated system curve for two duty pumps discharging into a 400 mm discharge pipe, with the duty point of 128 L/s at 16 m of TDH. The hydraulic calculations for the pump station, along with system curves, are presented in **Appendix E**. The pump to meet the design operating conditions is rated at 45 kW (60 hp) (600 V/60 Hz/3 ph).

Figure 4-2 System and Pump Curves



4.4.4.4 Pump Control Philosophy

Control of the raw water pumps is by a programmable logic controller (PLC) using local flow meter as its primary input, with warning alarms for high and low pressure.

A discharge header pressure transmitter is used to provide the discharge pipe pressure input. A discharge header flow meter is used to provide the flow input.

Pumps are started manually by operation staff to transfer the water from Lake Qikiqtalik to the reservoir. The pumps will then run automatically until operations staff turn them off or a low level in the wet well is detected. If a low level in the wet well is detected the pump will automatically shut down.

The pumps do not automatically rotate. Rotation is expected to occur between pumping seasons. If one of the duty pumps fails or needs to be removed from service for maintenance, the stand-by pump will be activated.

During the commissioning phase, the Systems Integrator along with Arcadis and operations staff will adjust the VFD control loop parameters and the start and stop setpoints for the pumps to reflect actual flow requirements and conditions in the pipeline to achieve the desired flow rate (i.e., to maintain full flow and/or to reduce the pipe

velocity in the pipeline system). The pumps will then operate continuously at the determined setpoint (from commissioning) within 75% - 100% of rated speed.

4.5 Civil

The site and civil works associated with the RWPS at Lake Qikiqtalik site include the following:

- Site grading and drainage modifications
- Facility access road
- Construction vehicular access
- Site plan approval
- Underground piping and utilities

Refer to further drawings in **Appendix A**.

4.5.1 Site Grading and Drainage Modifications

The RWPS site is vacant land, with an elevation range of 210.5 m to 210.0 m. The site slopes naturally from west to east towards Lake Qikiqtalik. The finished site will be graded to drain the runoff towards Lake Qikiqtalik. The finished first floor of the pump station is to be set at 210.50 m. This is approximately 6.7 m above expected high water level in LQ therefore there is no risk of flooding of the station. Should the lake level rise, the pumps are equipped with VFD therefore pump control can be adjusted to future water level changes.

The RWPS will be located approximately 62 m east of Lake Qikiqtalik with overland drainage, running from west to east and south to north. Site grading will generally follow the natural slope of the land with adjacent areas graded away from the new facility. The additional roof areas will be directed via a roof drainage system.

4.5.2 Facility Access Road

An upgraded access to the pump station from the 'Access Road to Unnamed Lake' will be constructed to allow for proper operation, maintenance, and access to equipment in the new facility. A gravel surface access road will be constructed all around the building and parking spaces will be provided for operation and maintenance vehicles.

4.5.3 Construction Access and Staging Area

The main vehicle access during construction will be via the existing access gravel road (the 'Access Road to Unnamed Lake') to the new facility. There is ample space around the RWPS construction area to set up laydown and storage areas as needed.

4.5.4 Site Plan Approval

Site plan approval documents will be prepared prior to tendering and submitted to the City Planning Department if required.

4.5.5 Underground Piping and Utilities

As no potable water or wastewater servicing from a central system will be servicing the RWPS there will be no underground piping for these systems. The raw water intake and conveyance pipeline are discussed in separate sections.

Power feed to the RWPS is expected to be overhead and will be determined by the local utility provider.

Control and electrical conduits from the pump station to the nearby valve chamber (VC1) will be constructed underground with adequate cover.

4.5.6 Fire Protection

Given the remote location of the RWPS and the expected materials of construction, no fire protection will be provided other than smoke/heat detection. Fire protection is not required by code for this facility (NBC Article 3.2.2.80). Extinguishers will be located onsite if a situation occurs when an operator is present that can be safely extinguished. Building temperature and smoke alarms will be monitored by the facility SCADA system.

4.5.7 Landscaping

No trees, shrubs or other landscaping improvements are planned. Site grading will be done in a manner which promotes the movement of surface water away from the pump station.

4.6 Architectural

The Lake Qikiqtaaluk pump station will be constructed on an undeveloped site with no other structures in the vicinity. The building will be designed to 'sit well' in the current context with the following in mind:

- The design will follow regulations under the latest National Building Code of Canada (NBC 2020).
- Given the function of the building being an unmanned small industrial facility, simplicity in design of the exterior will be one of the driving factors of the final shape, details, and materials.
- The design shall strive to minimize the gross area and volume required to accommodate the functions of the station. Simple forms such as square or rectangular forms is considered to minimize the exterior perimeter.
- The building envelope will be designed to suit to the climate in the area and will be using the rain screen principle with thermal resistance adapted to both the region's climate and the nature of the building with it being not intended to be occupied on continuous basis. The envelope insulation will be designed to latest codes in effect (NBC 2020).
- Exterior door frames and doors will be of metal and provided with appropriate thermal resistance.
- Consideration for introducing natural light into the building will be further studied during detailed design with proposed windows or translucent panels at a high elevation (clerestory) The potential for vandalism will require consideration.
- Materials for the interior of the building will be selected to be durable, requiring little maintenance. Interior finishes will be kept to minimal such as concrete or epoxy floors and/or walls.
- The roof will be designed with a slope in all directions to divert snow loads from accumulation onto the structure.
- With respect to accessibility (barrier free design) this building is not intended to be occupied on daily or full-time basis and based on the function of the building, it is not required to be designed to barrier-free standards.

Material selection for all building elements (walls, roof, floors) will be coordinated and developed in the next phase(s) of the design.

4.7 Structural

4.7.1 RWPS

The RWPS is a one-story structure designed with cast in place concrete. The Wet Well Access room will house a monorail with hoist and access hatch for servicing submersible pumps in the Wet Well. The Valve room will have a monorail with hoist for servicing valves and other equipment, and access to this room is from the Electrical and Control Room with floor hatch and access ladder. The below grade structure is still at the preliminary stage waiting for completion of geotechnical investigations and subsequent report from the geotechnical consultant.

Due to existence of permafrost at this location, major attention will be given to the design of structure below grade and the superstructure will have conventional design matching the site and process requirements. Detail structural design of the foundation system and the wet well will be carried out on the basis of the results of geotechnical investigations and recommendations in the geotechnical report. The design of the below grade structure will require mitigation of the effects of permafrost characteristics of the existing subsurface medium. Rock anchors will be provided at the underside of base slab of the wet well to resist buoyancy as required by the design.

An intentional offset between the cast-in-place concrete walls and the undisturbed bedrock was created during the design of the Wet Well to facilitate external leak testing. This gap provides access to the exterior surfaces of the Wet Well walls for visual inspection, formwork installation, application of waterproofing, backfilling, and quality control. It also allows identification and repair of potential leaks on the outside of the structure before final backfilling and finishing. Additionally, the offset accommodates irregularities and potential overbreak in the rock face, ensuring structural integrity and quality of the finished walls. Reducing or eliminating this offset may be considered if internal leak testing methods (such as hydrostatic or vacuum testing conducted from inside the wet well) are accepted by the Client and Regulatory authorities, and if testing confirms that the structure remains watertight when constructed in direct contact with the bedrock. Such an approach could result in reduced excavation volume, lower construction time, and decreased labor and costs. This alternative requires written approval from the Client and permitting authorities, formal specification of the leak test methods, acceptable leakage limits, test duration, and acceptance criteria, along with additional geological investigations to confirm the suitability of direct concrete-to-bedrock contact. Applicable standards include CSA A257 and CSA A23 for watertightness and concrete quality, Nunavut Capital Standards and Criteria, Iqaluit Municipal Design Guidelines, and Good Building Practices for Nunavut in alignment with CSA and NBCC standards.

The use of bored pile foundations combined with a base slab is based on the recommendations of section 10.0 of the "Geotechnical Investigation Apex River LTWP," Final Report, dated June 27, 2025. This approach is driven by the presence of ice-rich soils in the upper soil layers. Final design will be based on the results of additional geological investigations and subsequent recommendations and approvals from the Geological Team. After the final geotechnical results, the design foundations, which bear directly on bedrock without the need for piles at different levels, will be evaluated.

As per architectural design the finished roof of the building will be sloped to reduce the snow accumulation on the roof. The roof structure will comprise of reinforced cast-in-place concrete slab with prefabricated engineered wooden roof truss to provide the base for metal cladding and attaining the roof slope as specified by architectural design. The roof attic space will be accessible through an opening in the roof slab with access ladder and access hatch.

Cast-in-place concrete structures shall be designed to comply with ACI 350 "*Code Requirements for Environmental Engineering Concrete structures*" and CSA A23.3 "Design of concrete structure". Durability of the concrete elements shall be addressed through implementing quality concrete mix design, proper detailing of concrete joints, adequate reinforcement to resist stresses due to thermal expansion and contraction, and crack control measures by minimizing crack widths and depths. Backfill will be done with engineered backfill material with free draining characteristics according to geotechnical recommendations.

Aluminum access ladders will be designed inside both the Wet Well and the Valve Room to provide access to operation staff. Aluminum access hatches will be designed with secondary fall protection grating and have four-sided guard protection at floor level. Secured and convenient lockdown mechanisms will be included in the hatch design. Design live load of these hatches will be 12 kPa matching the floor design live load. Cast in floor slab davit socket will be provided near access hatches to allow the installation of a davit to assist operation staff to stay tethered during access into the wet well.

As per information available at the time of this report, there might not be any "Ready-mix" concrete batching plant available in the area to supply ready mix concrete for this project. If that is the final scenario before construction starts, then an onsite concrete batching plant will be required for producing concrete as per approved mix-design. Also, for coarse aggregates local excavated rocks can be tested in a CSA approved laboratory for design parameters for validation before being used in concrete mix design.

4.7.2 Design Criteria

The following are the structural design criteria for the LTWP.

- Site Location - Iqaluit, Nunavut
- Importance Category - Post Disaster
- Building Code - NBC2020
- Design Loads:
 - Live Load on main floor = 12 kPa
 - Superimposed Dead Load on main floor= 3.6 kPa + moving load from monorail in Valve Room
- Climatic data:
 - Wind Load:
 - $q(1/10) = 0.51$
 - $q(1/50) = 0.65$
 - Snow Load parameter:
 - $S_s = 2.9$
 - $S_r = 0.2$
- Seismic Load Design parameters (based on 2% / 50 years probability):
 - $S_a(0.2) = 0.202$
 - $S_a(0.5) = 0.225$
 - $S_a(1.0) = 0.143$
 - $S_a(2.0) = 0.0709$
 - $S_a(5.0) = 0.0191$
 - $S_a(10.0) = 0.00609$
 - $PGA = 0.112$
 - $PGV = 0.144$

Note: Design seismic parameters will be obtained from NBC 2020 seismic hazard tool online based on latitude-longitude values of Pumping Station and site designation defined by geotechnical engineer. Above seismic values are taken from the online tool with D class site designation at nearby location of Pumping Station with Latitude (63.784) and longitude (-68.458) and provided for reference at this stage.

- Applicable Design Codes:
- CSA A23.1&2:24 - Concrete materials and methods of concrete construction.
- CSA A23.3:24 - Design of concrete structures.
 - CSA S16:19 - Design of steel structure.
 - CSA A23.4-16 - Precast concrete – Materials and construction.
 - ACI 350-06 - Code Requirements for Environmental Engineering Concrete structures.

4.8 Building Services

4.8.1 General

The building mechanical design for this project will include plumbing and drainage systems for the new pump station, including electrical room, wet well access room, and the valve room. Heating and ventilation systems for the spaces will be provided.

Equipment, material, and installation will be in accordance with the noted edition of the following Standards, Codes and Recommended Practices:

- National Building Code of Canada (NBC) 2020
- National Fire Code of Canada (NFC) 2020

- National Plumbing Code of Canada (NPC) 2020
- Nunavut Consolidation of Building Code Act
- Nunavut Good Building Practices Guideline 2005
- The Government of Nunavut Energy Strategy
- Nunavut Electric Code (CSA C22.1-15)
- National Fire Protection Association (NFPA) Standards
- ASHRAE standards

4.8.2 Area Classification

All spaces at the pumping facility will be non-classified areas.

4.8.3 Heating and Ventilation

The Electrical and Control Room will be ventilated through an indoor air handling unit using heat recovery from the heat generated by the air compressors. Make-up air will be provided through the same air handling unit. A storm intake louvre will be provided for the unit intake connection. This unit will pre-heat the outdoor air by reclaiming the heat in the exhaust air stream. An electric duct system will be added to warm up the makeup air to prevent over cooling the space by cold outdoor air in winter. Additional electric space heating system will be provided for each room to maintain room temperatures as listed below. Outdoor air intake ductwork will be thermally insulated. All exhaust air connections to outdoors will be thermally insulated.

The wet well access room and valve chamber will be heated and ventilated from the same air handling unit and additional electric unit heaters will be provided to maintain room temperature. A wall mounted exhaust fan will be provided for the valve room and wet well access room and the air will be discharged to outdoors with an exhaust air louvre.

The wet well will be ventilated by a blower heater to bring fresh air into the space and will be heated by additional electric unit heater. A dedicated exhaust fan for the wet well will be provided, and the air will be discharged to outdoors with an exhaust air louvre.

The HVAC systems will be designed based on the following outdoor design criteria, in accordance with the National Building Code as follows:

- Winter: -41°C
- Summer: 17°C Dry Bulb, 12°C Wet Bulb

Room maximum and minimum design temperatures are in accordance with the room function and occupancy requirements as noted in Table 4.

Table 4-2 Indoor Space Design Basis

Description	ACH	Minimum Indoor Air Temperature	Maximum Indoor Air Temperature ¹	Space Humidity Level
Electrical Room	4	15°C	30°C	Not controlled
Wet Well Access Room	4	15°C	30°C	Not controlled
Wet Well	3	7°C	30°C	Not controlled
Valve Room	3	15°C	30°C	Not controlled

Description	ACH	Minimum Indoor Air Temperature	Maximum Indoor Air Temperature ¹	Space Humidity Level
Washroom	N/A	15°C	30°C	Not controlled
Note1: ACH based on good engineering practices and design guidelines for drinking water systems/pumping facilities.				

4.8.4 Controls

Standalone controls will be provided for mechanical equipment. There will be no building automation systems provided. Each electric heater will be controlled by a remote room temperature sensor. The ventilation system will be controlled by a room temperature sensor located in the electrical/control room, valve room and wet well respectively.

The exhaust fan for the washroom will be controlled by a manual switch.

Room temperature monitoring for minimum temperature values, will be included with the SCADA system to alert operations staff of a low temperature/heating failure.

Ventilation and heating equipment general alarms will be also included with the SCADA system to alert operations staff of system failures.

4.8.5 Seismic Considerations

As indicated in the structural section, the building will be a post disaster building, all building mechanical services will be seismically braced in accordance the National Building Code and Sheet Metal and Air Conditioning Contractors' National Association (SMACNA) standards.

Our specifications will require the mechanical contractor to conduct detailed seismic analysis and calculations for a complete design of equipment restraints system (e.g., hangers and supports) and provide submittals by a Professional Engineer. The seismic restraints design package will be provided at the shop drawings submittals phase.

4.8.6 Water Supply and Plumbing

4.8.6.1 Water Supply

There will be no municipal utility services available for the pump station. A portable handwashing unit will be provided and manually filled. The use of the water in the portable unit will be strictly for handwashing.

4.8.6.2 Plumbing and Drainage

Floor drains will be provided in each space with drain line connections to a sump pit located in the Valve Room. The sump pit will be provided with a submersible pump, floats and vendor supplied pump control panel. A water and airtight cover for pump access will be provided. The sump pit will be vented to the outdoors.

A venting system will be provided for all floor drains and plumbing fixtures and vented outdoors.

A trap priming system will be provided to all floor drains.

All water and drainage lines will be insulated, and no heat tracing will be required in the heated spaces.

All roof drainage will be provided with roof gutters and exterior mounted and heat traced rain leaders, drained to grade and completed with discharge concrete pads. Refer to architectural section for further details.

4.8.6.3 Washroom

A washroom with a portable handwashing unit and a waterless toilet will be provided in the pump station. All waste from the waterless toilet will be stored in the toilet and will need to be periodically emptied.

4.8.6.4 Emergency Eyewash and Deluge Shower Stations

There will be no emergency eyewash and/or deluge shower stations provided for this facility.

4.8.7 Fire Protection

There will be no overhead sprinklers, fire hose or fire alarms provided for this facility. Heat/smoke monitoring will be provided and monitored via the SCADA system. Portable fire extinguishers suitable for the construction and conforming to requirements of the National Fire Code and City of Iqaluit will be provided.

4.9 Electrical

4.9.1 General

Arcadis and the local utility provider (QEC) will carry out coordination efforts to plan and secure supply power to the site. Electrical power supply to the pump station is to be a utility supplied three-phase 347/600 V pad-mounted transformer; final type and size of transformer will be decided by the utility. The transformer pad and underground duct to the building will be included in this contract. Design will be done as per the latest edition of the Canadian Electrical Code.

Refer to the electrical single line drawing in **Appendix A**.

4.9.2 Utility Power

The pump station will require an electrical service rated at 400 A, 600 V, 3 phase. A utility supplied and installed transformer will provide 600 V, three-phase power feed to the station service entrance breaker located in the MCC. The service entrance section of the MCC will include a main service breaker with shunt trip capability, surge suppression, utility meter Current Transformers (CT) and Potential Transformers (PT) (to suit customer metering). The location of utility metering cabinet will be discussed with the utility. A standalone 400 A 600 V Automatic Transfer Switch will be located next to the MCC for the generator connection.

Utility power will be provided by QEC. They have confirmed that they can install power to the project for an approximate cost of \$2.5-3M. Construction of utility power is likely to take 2-3 years in order for QEC to complete design, procure poles and cabling, and transformers (which can take 2 years for delivery to Iqaluit based on input from QEC).

QEC requires one-year advance application for utility power in order to complete their design and begin procurement.

Temporary power will be required for construction until utility power is available.

4.9.3 Motor Control Centre

The station will be equipped with a new free standing Motor Control Center (MCC) which will include a service entrance type main breaker with shunt trip capability as well as utility meter CTs and PTs, and feeder breakers. This would then feed the main pumps and other 600 V loads.

4.9.4 Power and Lighting

General 120/208 VAC power and lighting for the pump station will be provided by a 30 kVA, 600-120/208 VAC station service transformer. The primary feed for the service transformer will be provided by the 600 V MCC. The 120/208 VAC secondary of the service transformer will feed a 120/208 VAC distribution panel located in the electrical room and designated as Panel LPA. All station lighting, receptacles and 120/208 VAC auxiliary equipment will be fed from Panel LPA. This distribution panel is in the electrical room and all wiring inside the pump station will be installed in PVC, rigid conduits, or TECK cable.

All emergency lighting will be LED and will have its own battery pack.

4.9.5 Equipment Loads

There are four overall large loads in the RWPS:

- One 22.37 kW (30 hp) air compressor.
- One 22.37 kW (30 hp) air compressor (not considered for overall loading since only one compressor runs at one time).
- Two 45 kW (60 hp) duty raw water pumps.
- One 45 kW (60 hp) standby raw water pump (not considered for overall loading as only two pumps run at one time).

Overall, this represents 112 kW (150 hp) of large motor loads and consumes a third of the nominal connected electrical capacity of the station.

4.9.6 Heat Tracing

Heat tracing will be required from within the wet well to the water intake in Lake Qikiqtalik to prevent freezing of the pipe year-round. The linear distance is approximately 160 m plus verticals. It is estimated that 20 kW of heat tracing load is required. Given the distances involved 600 V heat tracing is expected to be required to prevent voltage drops or excess sizing of conductors. The air burst line is also required to be heat traced. Final design calculations are underway to optimize the cable size and insulation required on the pipe.

4.9.7 Standby Generator

A permanent pad mount diesel generator will be installed as electrical backup for the facility. This will fully backup every piece of equipment in the event of a power failure. The generator will be equipped with a integral tank with the capacity of 48 hours of backup under full load. The generator will be connected to the building via an underground duct bank and connect to the MCC via an Automatic Transfer switch. The transfer switch will be a CSA C22.2 rated electrical device to suit power transfer operation from a primary source to a secondary power source (pad mount generator or plug in generator). The generator size is estimated to be 300 kW, 600 V, 3 phase.

The pump station will operate on utility power while such power is available. When operating on utility power, operation of pumps and motors will be controlled by the pump control system. During a failure of utility power, the station will operate with the diesel standby generator.

4.9.8 Miscellaneous Site Requirements

A power connection from the main building is required to feed a motorized valve in a chamber approximately 100 m from the pump building. A 600 V 3 phase connection is allocated to the chamber via an underground conduit. In addition, a conduit will be provided for any instrumentation connections required in the valve chamber.

4.10 Instrumentation and Control/SCADA System

4.10.1 New SCADA Network Communication

The new Supervisory Control and Data Acquisition (SCADA) system will consist of a freestanding PLC control panel housing Allen Bradley CompactLogix hardware, a redundant wide area network (WAN) consisting of single mode fibre optic and radio communication links, and an Inductive Automation Ignition Human Machine Interface (HMI).

The LQ RWPS will be connected to the SCADA WAN through network communication links designed to provide redundancy by utilizing separate network technologies and using separate physical paths. The primary WAN network link will be a six strand single mode fibre optic cable connecting the RWPS to the Water Treatment Plant. The fibre cable will be strung overhead utilizing the hydro poles providing power to the RWPS. An evaluation during detailed design will be conducted to assess the modifications required at the Water Treatment Plant in order to support the connectivity of the fibre optic cable. A new fibre optic patch panel may be required for the termination of the six fibre optic strands. A media converter from fibre to CAT6 may be required if the current network switch does not support the installation of a fibre optic transceiver.

At the RWPS a fibre optic patch panel will be installed in network cabinet to support the termination of the six strand fibre optic cable. Two strands will be active, the remaining four strands will be terminated at both ends to a fibre optic patch panel and will be spares. A network switch in the network cabinet will be equipped with a fibre optic transceiver which will be connected to the fibre optic patch panel through a prefabricated patch cord.

The secondary WAN network will be based on a radio network. At the RWPS an external mast will be installed on the roof to support the installation of a yagi directional antenna. The transmission cable and lightning surge protector will be installed in the new PLC control panel. The antenna will be connected to the radio antenna located at the Water Treatment Plant. A radio path profile analysis will need to be performed to confirm the required antenna mast height at the RWPS and to confirm the existing radio infrastructure at the Water Treatment Plant can support the new radio network link. Modifications to the Water Treatment Plant will be required and could include adjusting the height of the current antenna mast, realigning the existing antenna or the installation of a new mast and antenna complete with new grounding, transmission cable, radio, and lightning surge protector.

The local area network within the RWPS will consist of a network switch installed in the network cabinet. The network switch will support the termination of the WAN radio modem and fibre optic cable, CompactLogix PLC, HMI Interface and SCADA Client PC. All network active devices will be powered from the PLC control panel UPS.

4.10.2 PLC Control Panel

A new free standing PLC control panel will be installed at the RWPS and will house Allen Bradley CompactLogix PLC hardware from active product cycle. An ELO 1593L 15.6in LCD Touchscreen Monitor will be installed on the PLC control panel front face and will be connected to a Axiomtek Fanless eBOX560-880-FL-5010U industrial computer running Windows 10 Professional (or as determined at the time of installation) in the interior of the cabinet. The industrial computer will run Inductive Automation's Ignition software and will utilize the Allen Bradley Ethernet/IP driver. An Ignition Gateway will be configured at the RWPS and will forward information to the Central Gateway at the WTP through the radio/fibre wide area network connections.

4.10.3 PLC Control Panel Power Distribution

The PLC control panel will be supplied power from a new 120 VAC lighting panel connected to the standby generator for backup power in the event of utility power fail. A surge protector device will be installed in the PLC control panel for protection of the internal electrical components. A UPS will be installed complete with an external maintenance bypass switch to facilitate the removal and maintenance of the UPS without disrupting power within the control panel. All instrumentation installed in the RWPS will be supplied power from the UPS. Dual

redundancy 24 VDC power supplies will be installed to supply power to any 24 VDC devices. The status of the UPS will be monitored by the PLC through a UPS relay interface card and include the following the signals:

- UPS Fault Alarm
- UPS Battery Low Alarm
- UPS Online

The status of the UPS/maintenance switch will be monitored by the PLC and include the following the signals:

- UPS Bypass Switch in Bypass Mode

The status of the UPS Supply Power will be monitored by the PLC and include the following the signals:

- UPS Supply Power Failure Alarm

Additional Power related signals terminated to the PLC include:

- 24VDC Power Supply 1 Fault Alarm
- 24VDC Power Supply 2 Fault Alarm
- Surge Protection Fault Alarm
- PLC Control Panel Power Failure Alarm

For the standby generator and automatic transfer switch the following signals will be terminated to the PLC:

- Automatic Transfer Switch Utility Power Available
- Automatic Transfer Switch Emergency Power Available
- Automatic Transfer Switch On Utility Power
- Automatic Transfer Switch On Emergency Power
- Generator Running Status
- Generator Fault Alarm
- Generator In Auto Status
- Generator Battery Charger Fault Alarm
- Generator Battery Voltage Low Alarm

4.10.4 Instrumentation

All instrumentation connected to the PLC control panel in the RWPS will be backed up by UPS power from the PLC panel that the instrument is terminated at. Remotely connected instruments will be connected to the PLC network by industry standard fieldbus protocols (ie. Modbus TCP/IP). Remotely connected instruments that are located at a significant distance from the PLC panel will be connected by fibre optic cable and field mounted ethernet switches and/or media converters. The SCADA system will provide alarming and trending functions for all relevant process conditions being monitored.

4.10.5 Control Philosophy

The raw water pumps will have three modes of control: local, remote-manual, and remote-automatic. Mode selection will occur at the MCC through a local-off-remote selector switch. While in remote, Operations staff can select remote-manual or remote-automatic through the HMI. In local mode, the pumps can be started/stopped through start and stop pushbuttons at the MCC, and interface with the VFD through a keypad. Pilot lights for

pump running and hardwired alarm conditions including VFD fault and general fault will be provided accompanied by a reset pushbutton. An emergency mushroom style stop will be provided at the MCC to terminate power to the device and interlock it from starting.

The raw water pumps will have the following I/O signals terminated to the PLC:

- Power On Status
- Remote Status
- Emergency Stop
- Running Status
- General Fault Alarm
- VFD Fault Alarm
- Leak Detection Alarm
- Motor Thermal Overload Alarm
- Start Command
- Speed Setpoint
- Speed Indication

In remote-automatic mode, Operations staff will assign the pump duty arrangement through the HMI by selecting a pump to be either duty 1, duty 2 or standby. If a pump fails or is unavailable the duty rotation logic will automatically place the pump into the standby position and promote duty 2 to duty 1 and the standby pump to duty 2. Duty rotation based on duration (hours) can be enabled through the HMI to equalize the runtimes of the pumps. Once the duration setpoint has been reached the duty 1 pump will be automatically stopped, duty 2 will be promoted to duty 1 and the standby pump will be automatically started after a brief time delay has expired provided the Operator has selected the option to run two pumps on the HMI. To start the pumps in remote-automatic mode, Operations staff can press either the "Start Duty 1 Pump" pushbutton or "Start Duty 2 Pump" pushbutton on the HMI and stop the pumps by pressing either the "Stop Duty 1 Pump" pushbutton or "Stop Duty 2 Pump" pushbutton. To select the "Stop Duty 1 Pump" pushbutton the Duty 2 Pump must be stopped. The raw water pumps will be hardwired interlocked with the wet well low-level float and will not be permitted to run in local or remote modes of operation if a wet well low-level event is active. In addition, in remote mode, the virtual low-level alarm from the wet well level transmitter will be used as a software interlock in order to prevent the pumps from running during a wet well low-level event. The following level transmitter and float switch signals are wired to the PLC:

- Level Indication
- Level Loss of Echo Alarm
- Liquid Level High Alarm
- Liquid Level Low Alarm

The pumps operate continuously, modulating between 75% - 100% of its rated speed, based on a pre-set pressure at VC 1 and/or flow rate from the flow meter at the PS in order to achieve the desired flow velocity in the pipeline. The setpoints will be adjusted during commissioning to achieve the required hydraulics in the pipeline (to avoid excessive velocity) such that the pump operates always at a constant speed while running in local or remote mode.

Discharge pressure will be monitored through a pressure transmitter with the following signals wired to the PLC:

- Pressure Indication

- Pressure Transmitter Fault

Discharge flow will be monitored through a flow transmitter with the following signals wired to the PLC:

- Flow Indication
- Flow Transmitter Fault Alarm
- Flow Totalized Flow Indication

A sump pump is installed in the valve room sump pit. The pump starts and stops automatically based on the built-in adjustable float switches. A separate level float in the sump pit is wired to the PLC for flood alarm monitoring. The PLC will monitor the sump pump running status. The following signals are wired to the PLC:

- Valve Chamber Flood Alarm
- Valve Chamber Sump Pump Running Status

4.10.6 Backup Level Control

Under normal conditions the level in the wet well is maintained at an elevation well below the high level float tipping point. In the event that the level in the wet well rises high enough to trip the high level float the float backup mode will be activated. In float backup mode pump 1 starts automatically and runs until the low level float is tipped. If pump 1 does not stop after 15 minutes of running then pump 2 starts. If the low level float does not tip after both pumps are running for another 15 minutes then it is assumed that the low level float has malfunctioned. Subsequently, the pumps will automatically stop, and an alarm will be generated. All timer durations described above will be tested and optimized during commissioning.

4.10.7 Air Burst System

The air burst system consists of two rotary screw type compressors, primary and auxiliary air tanks, pneumatic discharge valve, and related air control valves and monitoring sensors. The complete system will be controlled by a vendor supplied control system consisting of an Allen Bradley Compact Logix PLC. The airburst system PLC will be housed in a vendor supplied control panel located in the RWPS. The airburst system PLC will be equipped with Ethernet communications that will be connected to the Ethernet switch in the RWPS PLC panel. Detail on the IO signals to SCADA will be determined during detailed design.

4.10.8 Building Services

The access doors to the pump station control room and wet well access room will be monitored by the PLC to provide security and alarm when there is unauthorized intrusion. High and low temperature switches and a smoke detector will be installed in the control room and connected to the PLC to provide monitoring and alarming. The PLC will be connected to the plant SCADA and Ignition Alarm Notification Module for remote monitoring and alarm annunciation. The following signals are wired to the PLC:

- Control Room Door Opened Alarm
- Wet Well Access Room Door Opened Alarm
- Room Temperature High Alarm
- Room Temperature Low Alarm
- Room Smoke Detection Alarm

4.10.9 Lake Level Monitoring

The water levels at the new reservoir, Lake Geraldine and LQ will be monitored by pressure-based level sensors. The power and signal wiring from the sensors will be wired to a small enclosure located nearby pump station or intake station infrastructure and connected by conduit for power and instrumentation cables. Signal converters will be used to convert the analog (4-20mA) sensor signals to Modbus TCP/IP. A network switch in the enclosure connects the sensors to the PLC network via fibre optic cable back to the pump stations or access building at the service corridor.

5 Apex River Raw Water Pump Station

5.1 General

The Apex River (AR) Raw Water Pump Station (RWPS) is designed as a wet well pump station that includes:

- A raw water intake well (wet well) with three submersible pumps (two duty and one standby), discharge piping, ultrasonic level sensor, high-level and low-level float.
- Two slide gates on the intake pipes entering the wet well to isolate the inlet flow.
- A valve room, designed to house station piping, valves, and flow monitoring equipment.
- A wet well access room, above the wet well, with a two-tonne monorail and manual trolley and hoist for the pumps.
- An electrical and control room for the motor control centre (MCC), electrical panels and instrumentation and control (I&C) panels, and a washroom.

5.2 Pump Station Design Features

The RWPS was designed within the context of the site and the location in Iqaluit. The site is remote from the City and therefore remote control (through SCADA), durable building features, systems to maintain temperature for equipment, and ease of maintenance were in the forefront.

The main design features of the AR RWPS include:

- Raw water intake to AR, insulated and heat traced.
- Sub-grade wet well for three submersible centrifugal pumps (two duty and one standby).
- Ultrasonic level system that monitors the wet well water levels.
- Back-up float alarm system to alert operations staff in the event of low or high wet well water level.
- Isolated valve room adjacent to the wet well to house valves, piping, and electromagnetic flow meter on the discharge header.
- Wet well will have lighting, heating and ventilation and a manually operated two tonne hoist and trolley to remove the pumps.
- Valve room with lighting, heating and ventilation. The valves can be lifted and moved to the floor of the valve chamber using the monorail, then lifted to the electrical and control room using a portable hoist, with a hook in the ceiling over the hatch.
- At grade Electrical and Control Room for the MCC panels, Programmable Logic Controller (PLC) and control panels, network closet, variable frequency drives (VFDs) for the raw water pumps, electric heating units, ventilation, and lighting.

- Outdoor standby diesel -powered generator set with automatic transfer switch, and the ability to connect a portable emergency generator in the event that backup power fails.
- Three phase primary power supply to the pump station building complete with a pole mounted transformer (to be determined by Utility) for 600 V supply to the motor control centre.
- A washroom consisting of a waterless toilet and portable handwashing unit with a baseboard heater.
- An intake screen brush cleaning system.
- The Electrical and Control Room will house electrical for the intake heat tracing.
- HVAC equipment to provide heat and ventilation air into all spaces.

Refer to pump station drawings in **Appendix A**.

As detailed design progresses items that are critical to station operations will be identified as a list of recommended spare parts (and included in the design specifications) for storage at a City works facility.

5.3 Intake

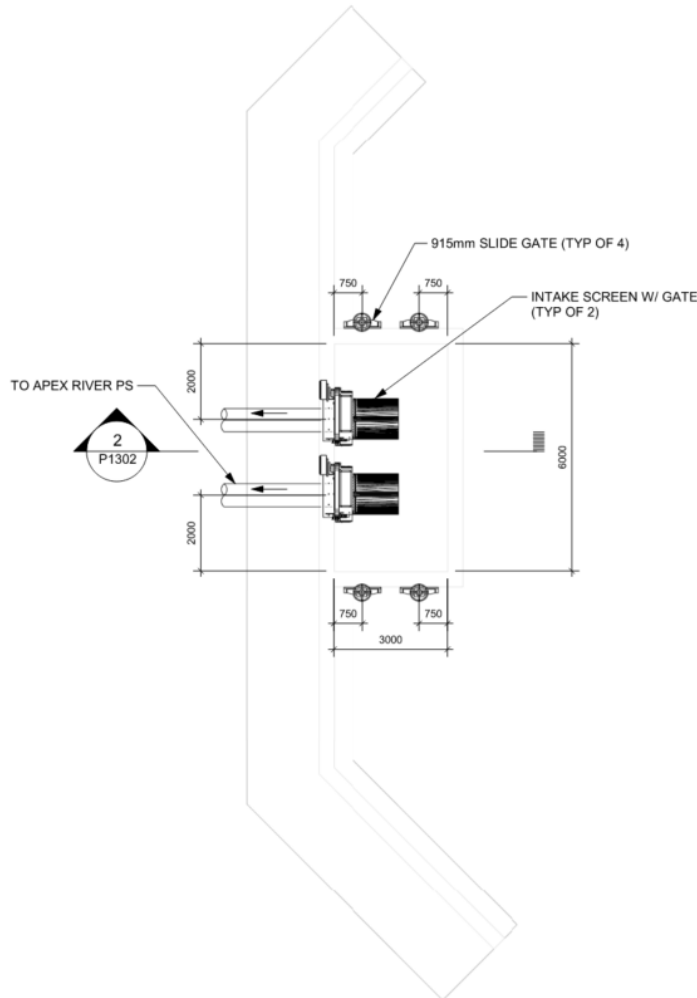
5.3.1 Crib

The Apex River flows in a north-to-south direction. To effectively manage and capture water flow in its shallow environment, an intake crib is proposed to be constructed adjacent to the riverbank. Water will enter the intake crib through two slide gates located on the north side. Once inside, the flow will follow one of two paths:

- Screened Flow Path: Water passes through intake screens and is directed to the wet well for further processing.
- Bypass Flow Path: Water bypasses the screens and exits the crib via two downstream slide gates positioned on the south side.

During non-pumping seasons, all slide gates will remain closed to minimize ice formation within the crib structure thereby protecting internal components and maintaining operational integrity. Refer to Figure 5-1 for a general layout of the intake crib and screens.

Figure 5-1 Intake Crib Layout



5.3.2 Screen

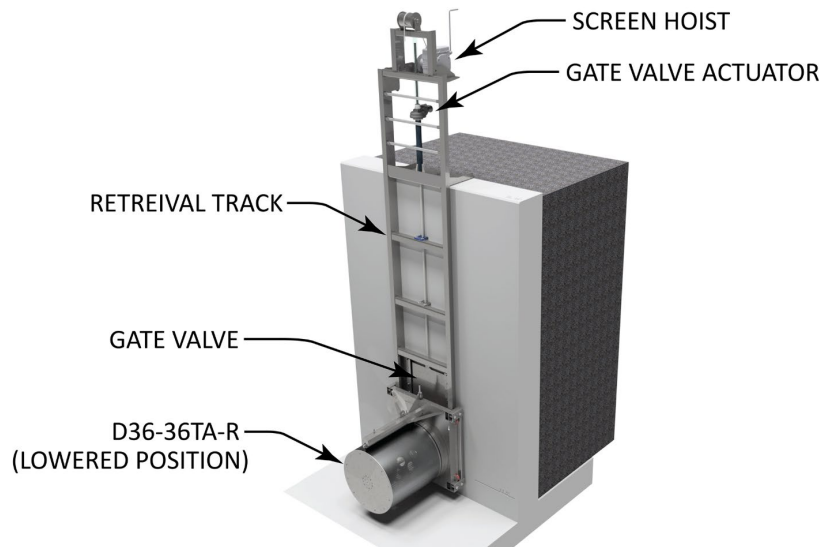
The intake screen is sized in accordance with the Department of Fisheries and Oceans (DFO) Freshwater Intake End-of-Pipe Fish Screen Guidelines for the fish that may be present in the Apex River. Previous studies identified the presence of Arctic Char. Arctic Char have a subcarangiform mode of swimming. Using a screen approach velocity of 0.11 m/s per the DFO guidelines, a screen with a clear area of 3.64 m² is required for a flow rate of 400 L/s to prevent entrainment and impingement. The screen openings will be no larger than 2.54 mm in accordance with the DFO guidelines.

The intake itself consists of two retractable self cleaning cylinder screens. The cleaning is accomplished by rotating a screen cylinder between internal and external brushes. The internal mechanism that turns the barrel can either be electrically driven, or propeller driven (using the river's flow through the barrel to turn the screen). These options will be explored further during detailed design.

During the non pumping seasons, the intake screens can be removed from the water to prevent damage via built-in lifting mechanisms. The screens would then need to be lifted onto a crib and taken in to be stored. There is sufficient space in the ARPS to store the screens.

The cylinder screens are also equipped with blanking plates to seal off the openings when the screens are lifted during the non pumping seasons. Refer to Figure 5-2 for a general depiction of the intake screen.

Figure 5-2 Intake Screen and Track



5.4 Process Mechanical

5.4.1 Piping

5.4.1.1 Intake Size and Class

The intake is two 600 mm diameter HDPE DR 11 (inside diameter of 499 mm) pipes. The flow velocity with a total flow of 400 L/s is 1.0 m/s per intake pipe.

5.4.1.2 Internal Station Piping

Each pump discharge line is 350 mm in diameter and is equipped with a swing flex check valve and a knife gate valve. The common 600 mm discharge header is equipped with a 450 mm diameter magnetic flow meter, pressure sensor and transmitter and combination air valve. All piping within the building is 304L stainless steel with stainless steel flanges.

5.4.1.2.1 Discharge Pipe Drain Line

In the case of pipe failure, it may be necessary to drain a portion of the discharge pipe back into the wet well. The portion of the discharge pipe within the valve chamber is equipped with a 150 mm diameter drain line complete with an isolation valve to drain the discharge pipe back into the wet well. The isolation drain valve is equipped with a manual handwheel operator accessed from the valve chamber.

5.4.1.3 Conveyance Pipeline Size and Class

There are two main pipelines. One of them discharges directly to the New Reservoir (Route 1). The other pipeline (Route 2) can discharge either to the New Reservoir, or to Lake Geraldine. All pipe segments are 400 mm HDPE DR 11 (inside diameter of 356 mm). The flow velocity with a design flow of 200 L/s is 2 m/s.

5.4.1.4 Conveyance Pipeline Route and Installation

The Route 1 pipeline directs the raw water pumped from the Apex River to the reservoir located next to Lake Geraldine, approximately 0.5 km southwest of the pump station. The plan and profile drawings are provided in **Appendix A**.

The Route 2 pipeline directs the raw water pumped from the Apex River either to the reservoir located next to Lake Geraldine (0.5 km southwest of the pump station), or to Lake Geraldine (1.1 km southwest of the pump station). The plan and profile drawings are provided in **Appendix A**.

5.4.1.5 Surge Pressure Calculations

A preliminary transient analysis was performed. The surge pressure at the pump station is approximately 860 kPa (125 psi). Due to the high surge pressure in the analysis, we are planning to install a surge anticipating valve at the pump station that will release energy back into the wet well during surge events.

5.4.1.6 Combination Air and Vacuum Release Valve

Short-lived full-vacuum or sub-atmospheric pressure may occur at the discharge side of the pump station.

A 25 mm diameter sewage combination air and vacuum valve (SCAV) for raw water is installed on the 450 mm diameter common pump discharge header to release entrapped air and provide pipeline vacuum protection. The type and size of SCAV is to be reviewed during the detailed design stage. The vacuum release valve allows air into the pipe to facilitate draining the piping during maintenance and the fall shutdown. Due to the nature of raw water, the SCAV must be cleaned and maintained regularly by the operation staff. The SCAV operation and maintenance manual indicates that the SCAV should be scheduled for regular inspection and backwash monthly. Based on Operation Staff service experience, this backwash regimen can be adjusted to suit actual in use experience.

5.4.2 Wet Well

5.4.2.1 Design

The supportive design calculations for the pump station are enclosed in **Appendix E**.

5.4.2.2 Inlet Pipe Slide Gate

An inlet slide gate is provided on the wall of the wet well to isolate the wet well from the intake pipe in the Apex River for ease of maintenance of the wet well.

5.4.3 Valve Room

The valve room is located below grade and houses the discharge piping and valving for the pumps. Each pump is equipped with a swing check valve to prevent backflow, and a knife gate valve for isolation. A magnetic flow meter is located on the discharge pipe prior to exiting the station. A monorail is provided for lifting of the valves for maintenance along with an equipment access hatch to the ground floor.

5.4.4 Raw Water Pumps

5.4.4.1 Head Loss Calculations

The system head curve is developed by calculating the total dynamic head (TDH) as the sum of static head and friction losses in pipes and fittings. The following is a summary of the calculation methodology for the pump station. Refer to **Appendix E** for the detailed design calculations.

5.4.4.1.1 Static Head

Static head values are based on the proposed operating levels in the wet well (between 94.1 m and 95.5 m) with two pumps running in parallel.

Three different static heads were considered in developing the system head curve: the lowest static lift, the mean static lift, and the highest static lift. These values represent the range of static lifts against which the pump will operate. The lowest static lift is associated with the high-water level in the wet well, the mean static lift is associated with the average liquid level and the highest static lift is associated with the low water level. Refer to **Appendix E** for the detailed design calculations.

5.4.4.1.2 Friction Losses

Friction losses are calculated using the Hazen-Williams equation as follows:

$$S = \frac{h_f}{L} = \frac{10.67 Q^{1.85}}{C^{1.85} d^{4.87}}$$

Where:

- S = Hydraulic slope
- h_f = head loss in meters (water) over the length of pipe
- L = length of pipe in meters
- Q = volumetric flow rate, m³/s (cubic meters per second)
- C = pipe roughness coefficient
- d = inside pipe diameter, m (meters)

Note: pressure drop can be computed from head loss as $h_f \times$ the unit weight of water (e.g., 9810 N/m³ at 4°C).

The following three 'C' values were used to develop system head curves and assess system characteristics at various flows throughout the system design life:

- C = 120 in conjunction with the highest static lift (HSL) to calculate the worst-case system scenario, as may occur in the future after long term use of the discharge piping.
- C = 140 in conjunction with the mean static lift (MSL) to calculate the system curve that is most likely to occur during normal pumping operation.
- C = 150 in conjunction with the lowest static lift (LSL) to determine system characteristics during initial start-up. This will ensure that the pump can operate at low head and avoid run-out conditions.

The system curve with C = 150 was used to select a suitable pump. However, the pump performance curve must cross all three curves to ensure adequate operation under different hydraulic conditions.

In addition to friction losses in the pipes, local losses that occur in valves and fittings are calculated using the following equation:

$$\text{Minor Losses in Piping} = h_m = k (V^2/2g)$$

Where:

h_m = Minor Losses in Piping because of fittings

K = friction constant

V = Velocity of fluid in the pipe

g = acceleration due to gravity = 9.807 m/s²

5.4.4.1.3 Total Dynamic Head (TDH)

The TDH is equal to the sum of the static lift, friction loss in pipes as well as local losses. The three scenarios that define the hydraulic envelope of the system are:

Worst-case scenario with maximum static lift: $TDH = HSL + h_f (@ C=120) + h_L$

Normal operating conditions with average lift: $TDH = MSL + h_f (@ C=130) + h_L$

Start-up conditions with lowest static lift: $TDH = LSL + h_f (@ C=140) + h_L$

Calculations were completed for the 400 m diameter discharge pipe at different static lifts and a system curve was generated. Please refer to **Appendix E** for the detailed design calculations.

5.4.4.2 Pump Selection

The pump design information is provided in **Appendix E**. Based on the system curve and duty requirements, the required pump capacity is set at 200 L/s each.

Preliminary pump performance curves from Flygt are enclosed in **Appendix F**.

The preliminary pump criteria can be seen in Table 5-1.

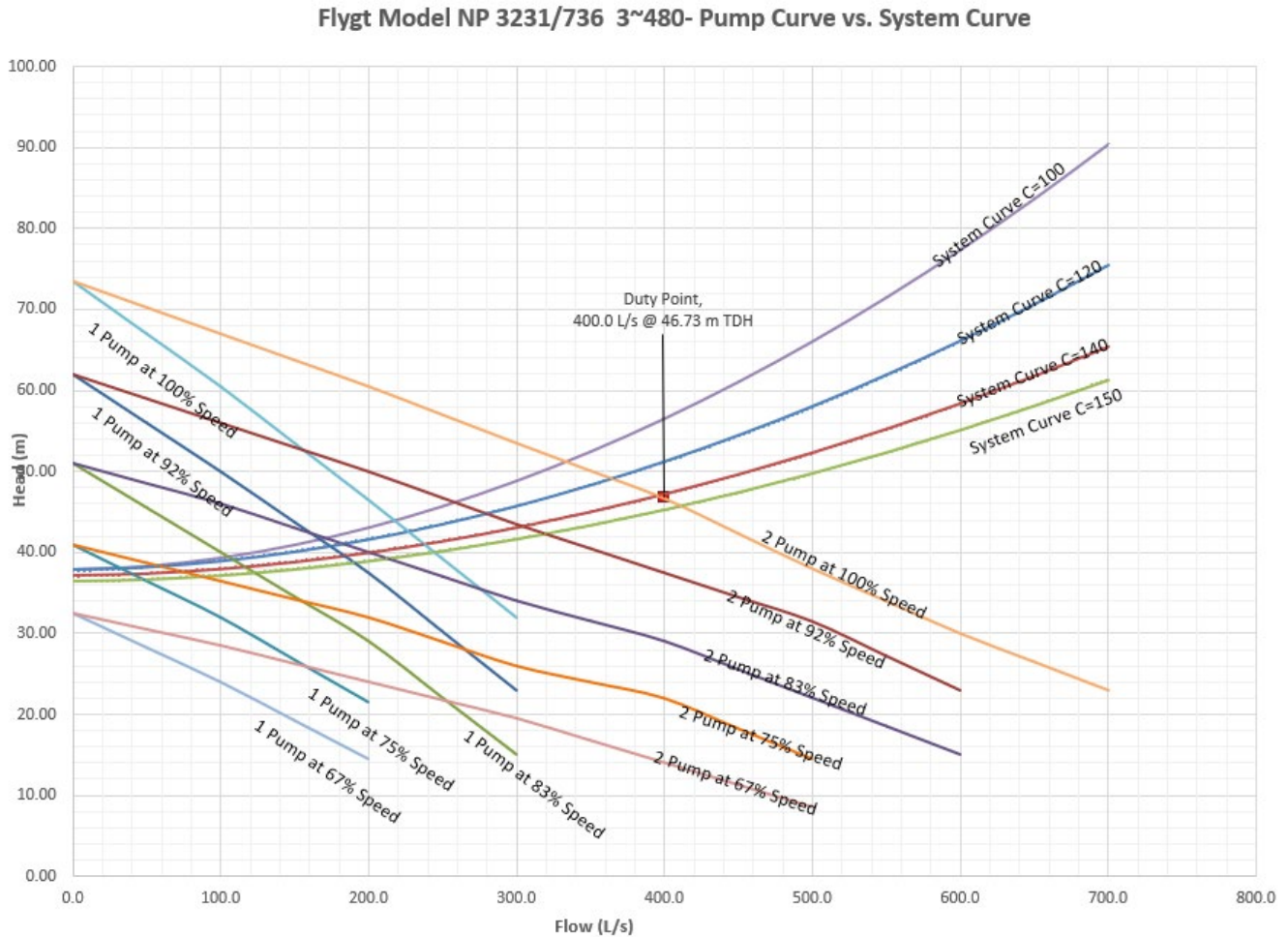
Table 5-1 Pump Criteria

Parameter	Criteria
Performance - Pump Duty Point	200 L/s @ 39 m TDH (with VFDs, pumps will operate near the best efficiency point)
Duty Pumps	2
Standby Pumps	1
Type	Submersible centrifugal pumps with non-clog impellers
Cost Effectiveness	Submersible pumps are cost effective for this application. Alternative pump types (e.g. vertical turbine pumps) are not recommended for this application and remote location due to the need for a crane to remove the pumps through the roof.
Power Requirement - Power Rating	186 kW (250 hp)
Variable Frequency Drive (VFD)	Yes
Reliability of Operation	Flygt submersible pumps have historically been very reliable.
Serviceability and Service Life	Service Life is typically 15 years
Maintenance and Safety	Pump maintenance schedule is recommended at intervals of 12,000 hrs or three (3) years.

5.4.4.3 System and Pump Curve

Figure 5-3 outlines the calculated system curve for two duty pumps discharging into two 400 mm discharge pipes, with the duty point of 370 L/s at 46 m of TDH. The hydraulic calculations for the pump station, along with system curves, are presented in **Appendix E**. The pump to meet the design operating conditions is rated at 186 kW (250 hp) (600 V/60 Hz/3 ph).

Figure 5-3 Apex River PS System and Pump Curves



The system curves presented above correspond to the scenario involving pumping to the newly constructed reservoir. In the alternative scenario of pumping to Lake Geraldine, the current pump configuration is capable of delivering approximately 200 L/s. It was agreed in a workshop Oct 2, 2025 that this was acceptable given that pumping to Lake Geraldine is an emergency scenario.

5.4.4.4 Pump Control Philosophy

Control of the raw water pumps is by a programmable logic controller (PLC) using local flow meter as its primary input, with warning alarms for high and low pressure.

A discharge header pressure transmitter is used to provide the discharge pipe pressure input. A discharge header flow meter is used to provide the flow input.

Pumps are be started manually by operation staff to transfer the water from the Apex River to the reservoir. The pumps will then run automatically until operations staff turn them off or a low level in the wet well is detected. If a low level in the wet well is detected the pump will automatically shut down.

The pumps do not automatically rotate. Rotation is expected to occur between pumping seasons. If one of the duty pumps fails or needs to be removed from service for maintenance, the stand-by pump will be activated.

During the commissioning phase, the Systems Integrator along with Arcadis and operations staff will adjust the VFD control loop parameters and the start and stop setpoints for the pumps to reflect actual flow requirements to achieve the desired flow rate that is based on the daily river flow. The pumps will then operate continuously at the determined setpoint for that day within 75% - 100% of rated speed.

5.5 Civil

The site and civil works associated with the RWPS at Apex River site include the following:

- Site grading and drainage modifications
- Facility access road
- Construction vehicular access
- Site plan approval
- Underground piping and utilities

Refer to drawings in **Appendix A**.

5.5.1 Site Grading and Drainage Modifications

The AR RWPS site is vacant land, with an elevation range of 95.50 m to 107.70 m. The site slopes naturally from east to west towards Apex River. The finished site will be graded to drain the runoff towards Apex River. The finished first floor of the pump station is to be set at 105.65 m. This is approximately 11.15 m above expected high water level in Apex River at 96.50 therefore there is no risk of flooding of the station. Should the river level rise, the pumps are equipped with VFD therefore pump control can be adjusted to future water level changes.

The RWPS will be located approximately 60 m west of Apex River with overland drainage, running from east to west and south to north. Site grading will generally follow the natural slope of the land with adjacent areas graded away from the new facility. The additional roof areas will be directed via a roof drainage system.

5.5.2 Facility Access Road

An upgraded access to the pump station from the existing road will be constructed to allow for proper operation, maintenance, and access to equipment in the new facility. A gravel surface access road will be constructed all around the building and parking spaces will be provided for operation and maintenance vehicles. An additional 6 m wide access road will be provided to the new intake structure.

5.5.3 Construction Access and Staging Area

The main vehicle access during construction will be via the existing access gravel road (the 'Access Road to Apex River ') to the new facility and intake structure. There is ample space around the RWPS construction area to set up laydown and storage areas as needed.

5.5.4 Site Plan Approval

Site plan approval documents will be prepared prior to tendering and submitted to the City Planning Department if required.

5.5.5 Underground Piping and Utilities

As no potable water or wastewater servicing from a central system will be servicing the RWPS there will be no underground piping for these systems. The raw water intake and conveyance pipeline are discussed in separate sections.

Power feed to the RWPS is expected to be overhead and will be determined by the local utility provider.

Control and electrical conduits from the pump station to the nearby intake facility will be constructed underground with adequate cover.

5.5.6 Fire Protection

Given the remote location of the RWPS and the expected materials of construction, no fire protection will be provided other than smoke/heat detection. Fire protection is not required by code for this facility (NBC Article 3.2.2.80). Extinguishers will be located onsite if a situation occurs when an operator is present that can be safely extinguished. Building temperature and smoke alarms will be monitored by the facility SCADA system.

5.5.7 Landscaping

No trees, shrubs or other landscaping improvements are planned. Site grading will be done in a manner which promotes the movement of surface water away from the pump station.

5.6 Architectural

The AR pump station will be constructed on an undeveloped site with no significant structures in the vicinity. The building will be designed to 'sit well' in the current context with the following in mind:

- The design will follow regulations under the latest National Building Code of Canada (NBC 2020).
- Given the function of the building being an unmanned small industrial facility, simplicity in design of the exterior will be one of the driving factors of the final shape, details, and materials.
- The design shall strive to minimize the gross area and volume required to accommodate the functions of the station. Simple forms such as square or rectangular forms is considered to minimize the exterior perimeter.
- The building envelope will be designed to suit to the climate in the area and will be using the rain screen principle with thermal resistance adapted to both the region's climate and the nature of the building with it being not intended to be occupied on continuous basis. The envelope insulation will be designed to latest codes in effect (NBC 2020).
- Exterior door frames and doors will be of metal and provided with appropriate thermal resistance.
- Consideration for introducing natural light into the building will be further studied during detailed design, with windows or translucent panels placed at a high elevation (celestory).
- Materials for the interior of the building will be selected to be durable, requiring little maintenance. Interior finishes will be kept to minimal such as concrete or epoxy floors and/or walls.
- The roof will be designed with a slope in all directions to divert snow loads from accumulation onto the structure.
- With respect to accessibility (barrier free design) this building is not intended to be occupied on daily or full time basis and based on the function of the building, it is not required to be designed to barrier-free standards.

Material selection for all building elements (walls, roof, floors) will be coordinated and developed in the next phase(s) of the design. The exterior design and finishes at this station will be similar to the Lake Qikiqtalik pumping station with possible variations in texture and colour to slightly distinguish the two buildings/sites.

5.7 Structural

5.7.1 RWPS

The RWPS is a one-story structure designed with cast in place concrete. The Wet Well Access room will house a monorail with hoist and access hatch for servicing submersible pumps in the Wet Well. The Valve room will have a monorail with a hoist for servicing valves and other equipment, and access to this room is from the Electrical and Control Room with a floor hatch and an access ladder. The below grade structure is still at the preliminary stage waiting for completion of geotechnical investigations and subsequent report from the geotechnical consultant.

Due to the presence of permafrost in the underlying soil medium at this location, significant attention will be given to the design of the structure below grade, and the superstructure will have a conventional design that matches the site and process requirements. A detailed structural design of the foundation system and the wet well will be carried out based on the results of geotechnical investigations and recommendations outlined in the geotechnical report. The design of the below-grade structure will require mitigation of the effects of permafrost characteristics of the existing subsurface medium. Rock anchors will be provided at the underside of the base slab of the wet well to resist buoyancy as required by the design.

The use of bored pile foundations combined with a base slab is based on the recommendations of section 10.0 of the Geotechnical Report. The presence of ice-rich soils in the upper soil layers drives this approach. Based on the results of additional geological investigations and subsequent recommendations and approvals from the Geological Team, the design foundations, which bear directly on bedrock without the need for piles at different levels, will be evaluated.

Cast-in-place concrete structures shall be designed to comply with ACI 350 “*Code Requirements for Environmental Engineering Concrete structures*” and CSA A23.3 “*Design of concrete structure*”. Durability of the concrete elements shall be addressed through implementing quality concrete mix design, proper detailing of concrete joints, adequate reinforcement to resist stresses due to thermal expansion and contraction, and crack control measures by minimizing crack widths and depths. Backfill will be done with engineered backfill material with free-draining characteristics according to geotechnical recommendations.

Aluminum access ladders will be designed inside both the Wet Well and the Valve Room to provide access to operating staff. Aluminum access hatches will be designed with secondary fall protection grating and have four-sided guard protection at floor level. Secured and convenient lockdown mechanisms will be included in the hatch design. Design live load of these hatches will be 12 kPa matching the floor design live load. Cast in floor slab davit socket will be provided near access hatches to allow the installation of a davit to assist operation staff to stay tethered during access into the wet well.

As per information available at the time of this report, there might not be any “Ready-mix” concrete batching plant available in the area to supply ready mix concrete for this project. If that is the final scenario before construction starts, then an onsite concrete batching plant will be required for producing concrete as per approved mix-design. Also, for coarse aggregates local excavated rocks can be tested in a CSA approved laboratory for design parameters for validation before being used in concrete mix design.

5.7.2 Steel Bridge

The project involves constructing a steel bridge across the Apex River, designed to support four utility pipes encased in steel. Its primary purpose is to ensure the safe and efficient transport of these utilities across the river while maintaining structural integrity and resilience against environmental conditions.

The main structural system consists of a steel superstructure, selected for its strength, durability, and ability to withstand dynamic forces. The foundation system uses concrete caissons, chosen for their capacity to provide robust support within the riverbed and resist lateral forces. Ice protection measures are implemented through concrete ice breakers installed in concrete pile caps to reduce the impact of ice flows. In areas subject to high water levels, concrete is extended to the underside of the steel superstructure to enhance stability and protect against erosion.

The design accounts for key load considerations, including gravity, wind, seismic activity, snow, ice, and water flow. Emphasis is placed on long-term durability and serviceability, maintaining architectural continuity, and minimizing environmental impact throughout the structure's lifespan.

5.7.3 Concrete Intake

The concrete structure will serve dual purposes: supporting intake systems and functioning as retaining walls to manage soil and water pressures effectively. This structure is designed with reinforced concrete to ensure durability, resistance to hydrostatic pressure, and long-term serviceability. In areas prone to high water levels, the concrete will be extended to the underside of the steel superstructure to enhance stability and protect against erosion. The retaining walls will be anchored into the foundation system to ensure stability under lateral earth and water loads.

5.7.4 Design Criteria

The following are the structural design criteria for the LTWP.

- Site Location - Iqaluit, Nunavut
- Importance Category - Post Disaster
- Building Code - NBC2020
- Design Loads:
 - Live Load on main floor = 12 kPa
 - Superimposed Dead Load on main floor= 3.6 kPa + moving load from monorail in Valve Room and Dry Well
- Climatic data:
 - Wind Load:
 - $q(1/10) = 0.51$
 - $q(1/50) = 0.65$
 - Snow Load parameter:
 - $S_s = 2.9$
 - $S_r = 0.2$
- Seismic Load Design parameters (based on 2% / 50 years probability):
 - $S_a(0.2) = 0.202$
 - $S_a(0.5) = 0.225$
 - $S_a(1.0) = 0.143$
 - $S_a(2.0) = 0.0709$
 - $S_a(5.0) = 0.0191$
 - $S_a(10.0) = 0.00609$
 - $PGA = 0.112$
 - $PGV = 0.144$

Note: Design seismic parameters will be obtained from the NBC 2020 seismic hazard tool online based on the latitude-longitude values of the Pumping Station and site designation defined by the geotechnical engineer. The

above seismic values are taken from the online tool with D class site designation at the nearby location of Pumping Station with Latitude (63.784) and longitude (-68.458), and provided for reference at this stage.

- Applicable Design Codes:
 - CSA A23.1&2:24- Concrete materials and methods of concrete construction.
 - CSA A23.3:24 - Design of concrete structures.
 - CSA S16:19 - Design of steel structure.
 - CSA A23.4-16 - Precast concrete – Materials and construction.
 - ACI 350-06 - Code Requirements for Environmental Engineering Concrete structures.

5.8 Building Services

5.8.1 General

The building mechanical design for this project will include electrical room, wet well access room, and dry well room. Heating and ventilation systems for the spaces will be provided. A washroom system for the new pump station will be confirmed by the City in the future design development.

Equipment, material, and installation will be in accordance with the noted edition of the following Standards, Codes and Recommended Practices:

- National Building Code of Canada (NBC) 2020
- National Fire Code of Canada (NFC) 2020
- National Plumbing Code of Canada (NPC) 2020
- Nunavut CONSOLIDATION OF BUILDING CODE ACT
- Nunavut Good Building Practices Guideline 2005
- The Government of Nunavut Energy Strategy
- Nunavut Electric Code (CSA C22.1-15)
- National Fire Protection Association (NFPA) Standards
- ASHRAE standards

5.8.2 Area Classification

All spaces at the pumping facility will be non-classified areas.

5.8.3 Heating and Ventilation

The Electrical and Control Room will be ventilated through an exhaust fan system to remove the heat that is dissipated from the electrical equipment. An outside air intake opening is furnished with a motorized louvre which is equipped with steel mesh, snow baffles, and self-regulating heating cables. An electric space heater with adjustable built-in thermostat will be installed to maintain room temperatures as listed in Table 5-2.

The Wet Well Access Room will be ventilated from a blower heater system which brings outside air into the space continuously. An exhaust fan system will be installed to discharge the exhaust air from the room to the exterior. An electric space heater will be installed to maintain room temperatures.

The Wet Well Room will be ventilated from a blower heater system which bring outside air into the space continuously. An exhaust fan system will be installed to discharge the exhaust air from the room to the exterior. An electric space heater will be installed to maintain room temperatures.

The HVAC systems will be designed based on the following outdoor design criteria, in accordance with the National Building Code as follows:

- Winter: -41°C
- Summer: 17°C Dry Bulb, 12°C Wet Bulb

Room maximum and minimum design temperatures are in accordance with the room function and occupancy requirements as noted in Table 5-2.

Table 5-2--Indoor Space Design Basis

Description	ACH	Minimum Indoor Air Temperature	Maximum Indoor Air Temperature ¹	Space Humidity Level
Electrical Room	2	15°C	30°C	Not controlled
Wet Well Access Room	2	15°C	30°C	Not controlled
Wet Well	2	7°C	30°C	Not controlled
Washroom	N/A	15°C	30°C	Not controlled
Note1: ACH based on good engineering practices and design guidelines for drinking water systems/pumping facilities.				

5.8.4 Controls

There will be no building automation systems provided.

Each electric space heater will be controlled by an adjustable built-in thermostat.

Each blower heater will be controlled by its built-in control system with three heating stages and freezing self-stop feature. The blower heater is interlocked with the associated exhaust fan.

The air volume of exhaust fan in the electrical and control room is controlled by a VFD system to satisfy the requirements of various working conditions. The outside air intake louvre will be interlocked with the exhaust fan.

Room temperature monitoring for minimum temperature values, will be included with the SCADA system to alert operations staff of a low temperature/heating failure.

Ventilation and heating equipment general alarms will be also included with the SCADA system to alert operations staff of system failures.

5.8.5 Seismic Considerations

As indicated in the structural section, the building will be a post disaster building, all building mechanical services will be seismically braced in accordance the National Building Code and Sheet Metal and Air Conditioning Contractors' National Association (SMACNA) standards.

Our specifications will require the mechanical contractor to conduct detailed seismic analysis and calculations for a complete design of equipment restraints system (e.g., hangers and supports) and provide submittals by a Professional Engineer. The seismic restraints design package will be provided at the shop drawings submittals phase.

5.8.6 Water Supply and Plumbing

5.8.6.1 Water Supply

There will be no municipal utility services available for the pump station. A portable handwashing unit will be manually filled.

5.8.6.2 Plumbing and Drainage

A hose bibb will be supplied within the pump station for space washdown. This will be confirmed with the City in future design development.

5.8.6.3 Washroom

A washroom with a portable handwashing unit and a waterless toilet will be provided in the pump station.

All waste from the waterless toilet will be stored in the toilet and will need to be periodically emptied. Details of the handwashing/wastewater system will be further discussed and confirmed with the City.

5.8.6.4 Emergency Eyewash and Deluge Shower Stations

There will be no emergency eyewash and/or deluge shower stations provided for this facility.

5.8.7 Fire Protection

There will be no overhead sprinklers, fire hose or fire alarms provided for this facility. Heat/smoke monitoring will be provided and monitored via the SCADA system. Portable fire extinguishers suitable for the construction and conforming to requirements of the National Fire Code and City of Iqaluit will be provided.

5.9 Electrical

5.9.1 General

Arcadis and the local utility provider (QEC) will carry out coordination efforts to plan and secure supply power to the site. Electrical power supply to the pump station is to be a utility supplied three-phase 347/600 V pad-mounted transformer; final type and size of transformer will be decided by the utility. The transformer pad and underground duct to the building will be included in this contract. Design will be done as per the latest edition of the Canadian Electrical Code.

Refer to the electrical single line drawing in **Appendix A**.

5.9.2 Utility Service Entrance

The pump station will require an electrical service rated at 500 A, 600 V, 3 phase. A utility supplied and installed transformer will provide 600 V, three-phase power feed to the station service entrance breaker located in the MCC. The service entrance section of the MCC will include a main service breaker with shunt trip capability, surge suppression, utility meter Current Transformers (CT) and Potential Transformers (PT) (to suit customer metering). The location of utility metering cabinet will be discussed with the utility. A standalone 400 A 600 V Automatic Transfer Switch will be located next to the MCC for the generator connection.

5.9.3 Motor Control Centre

The station will be equipped with a new free standing Motor Control Center (MCC) which will include a service entrance type main breaker with shunt trip capability as well as utility meter CTs and PTs, and feeder breakers. This would then feed the main pumps and other 600 V loads.

5.9.4 Power and Lighting

General 120/208 VAC power and lighting for the pump station will be provided by a 30 kVA, 600/120/208 VAC station service transformer. The primary feed for the service transformer will be provided by the 600 V MCC. The 120/208 VAC secondary of the service transformer will feed a 120/208 VAC distribution panel located in the electrical room and designated as Panel LPA. All station lighting, receptacles and 120/208 VAC auxiliary equipment will be fed from Panel LPA. This distribution panel is in the electrical room and all wiring inside the pump station will be installed in PVC, rigid conduits, or TECK cable.

All emergency lighting will be LED and will have its own battery pack.

5.9.5 Equipment Loads

There are four overall large loads in the RWPS:

- Two 186 kW (250 hp) duty raw water pumps.
- One 186 kW (250 hp) standby raw water pump (not considered for overall loading as only two pumps run at one time).
- The station has a baseline of 85kw of electrical heating loads connected

Overall, the pumps represent (500 hp) of large motor loads and consumes approximately 60% of the nominal connected electrical capacity of the station.

5.9.6 Standby Generator

A permanent pad mount diesel generator will be installed as electrical backup for the facility. This will fully backup every piece of equipment in the event of a power failure. The generator will be equipped with a integral tank with the capacity of 48 hours of backup under full load. The generator will be connected to the building via an underground duct bank and connect to the MCC via an Automatic Transfer switch. The transfer switch will be a CSA C22.2 rated electrical device to suit power transfer operation from a primary source to a secondary power source (pad mount generator or plug in generator). The generator size is estimated to be 600 kW, 600 V, 3 phase.

The pump station will operate on utility power while such power is available. When operating on utility power, operation of pumps and motors will be controlled by the pump control system. During a failure of utility power, the station will operate with the diesel standby generator.

5.10 Instrumentation and Control/SCADA System

5.10.1 New SCADA Network Communication

The new Supervisory Control and Data Acquisition (SCADA) system will consist of a freestanding PLC control panel housing Allen Bradley CompactLogix hardware, a redundant wide area network (WAN) consisting of single mode fibre optic and radio communication links, and an Inductive Automation Ignition Human Machine Interface (HMI).

The Apex River RWPS will be connected to the SCADA WAN through network communication links designed to provide redundancy by utilizing separate network technologies and using separate physical paths. The primary

WAN network link will be a six strand single mode fibre optic cable connecting the RWPS to the Water Treatment Plant. The fibre cable will be strung overhead utilizing the hydro poles providing power to the RWPS. An evaluation during detailed design will be conducted to assess the modifications required at the Water Treatment Plant in order to support the connectivity of the fibre optic cable. A new fibre optic patch panel may be required for the termination of the six fibre optic strands. A media converter from fibre to CAT6 may be required if the current network switch does not support the installation of a fibre optic transceiver.

At the RWPS a fibre optic patch panel will be installed in the PLC control panel to support the termination of the six strand fibre optic cable. Two strands will be active, the remaining four strands will be terminated at both ends to a fibre optic patch panel and will be spares. The network switch within the PLC control panel will be equipped with a fibre optic transceiver which will be connected to the fibre optic patch panel through a prefabricated patch cord.

The secondary WAN network will be based on a radio network. At the RWPS an external mast will be installed on the roof to support the installation of a yagi directional antenna. The transmission cable and lightning surge protector will be installed in the new PLC control panel. The antenna will be connected to the radio antenna located at the Water Treatment Plant. A radio path profile analysis will need to be performed to confirm the required antenna mast height at the RWPS and to confirm the existing radio infrastructure at the Water Treatment Plant can support the new radio network link. Modifications to the Water Treatment Plant will be required and could include adjusting the height of the current antenna mast, realigning the existing antenna or the installation of a new mast and antenna complete with new grounding, transmission cable, radio, and lightning surge protector.

The local area network within the RWPS will consist of a network switch installed in the PLC control panel. The network switch will support the termination of the WAN radio modem and fibre optic cable, CompactLogix PLC, HMI Interface and SCADA Client PC. All network active devices will be powered from the PLC control panel UPS.

5.10.2 PLC Control Panel

A new free standing PLC control panel will be installed at the RWPS and will house Allen Bradley CompactLogix PLC hardware from Active product cycle. An ELO 1593L 15.6in LCD Touchscreen Monitor will be installed on the PLC control panel front face and will be connected to a Axiomtek Fanless eBOX560-880-FL-5010U industrial computer running Windows 10 Professional (to be confirmed at the time of installation) in the interior of the cabinet. The industrial computer will run Inductive Automation's Ignition software and will utilize the Allen Bradley Ethernet/IP driver. An Ignition Gateway will be configured at the RWPS and will forward information to the Central Gateway at the WTP through the radio/fibre wide area network connections.

5.10.3 PLC Control Panel Power Distribution

The PLC control panel will be supplied power from a new 120 VAC lighting panel connected to the standby generator for backup power in the event of utility power fail. A surge protector device will be installed in the PLC control panel for protection of the internal electrical components. A UPS will be installed complete with an external maintenance bypass switch to facilitate the removal and maintenance of the UPS without disrupting power within the control panel. All instrumentation installed in the RWPS will be supplied power from the UPS. Dual redundancy 24 VDC power supplies will be installed to supply power to any 24 VDC devices. The status of the UPS will be monitored by the PLC through a UPS relay interface card and include the following the signals:

- UPS Fault Alarm
- UPS Battery Low Alarm
- UPS Online

The status of the UPS/maintenance switch will be monitored by the PLC and include the following the signals:

- UPS Bypass Switch in Bypass Mode

The status of the UPS Supply Power will be monitored by the PLC and include the following the signals:

- UPS Supply Power Failure Alarm

Additional Power related signals terminated to the PLC include:

- 24VDC Power Supply 1 Fault Alarm
- 24VDC Power Supply 2 Fault Alarm
- Surge Protection Fault Alarm
- PLC Control Panel Power Failure Alarm

For the standby generator the following signals will be terminated to the PLC:

- Automatic Transfer Switch Utility Power Available
- Automatic Transfer Switch Emergency Power Available
- Automatic Transfer Switch On Utility Power
- Automatic Transfer Switch On Emergency Power
- Generator Running Status
- Generator Fault Alarm
- Generator In Auto Status
- Generator Battery Charger Fault Alarm
- Generator Battery Voltage Low Alarm

5.10.4 Instrumentation

instrumentation connected to the PLC control panel in the RWPS will be backed up by UPS power. Remotely connected instruments will be connected to the PLC network by industry standard fieldbus protocols (i.e. Modbus TCP/IP). Remotely connected instruments that are located at a significant distance from the PLC panel will be connected by fibre optic cable and field mounted ethernet switches and/or media converters. The SCADA system will provide alarming and trending functions for all relevant process conditions being monitored.

5.10.5 Control Philosophy

The raw water pumps will have three modes of control: local, remote-manual, and remote-automatic. Mode selection will occur at the MCC through a local-off-remote selector switch. While in remote, Operations staff can select remote-manual or remote-automatic through the HMI. In local mode, the pumps can be started/stopped through start and stop pushbuttons at the MCC, and interface with the VFD through a keypad. Pilot lights for pump running and hardwired alarm conditions including VFD fault and general fault

will be provided accompanied by a reset pushbutton. An emergency mushroom style stop will be provided at the MCC to terminate power to the device and interlock it from starting.

The raw water pumps will have the following I/O signals terminated to the PLC:

- Power On Status
- Remote Status
- Emergency Stop
- Running Status
- General Fault Alarm

- VFD Fault Alarm
- Leak Detection Alarm
- Motor Thermal Overload Alarm
- Start Command
- Speed Setpoint
- Speed Indication

In remote-automatic mode, Operations staff will assign the pump duty arrangement through the HMI by selecting a pump to be either duty 1, duty 2 or standby. If a pump fails or is unavailable the duty rotation logic will automatically place the pump into the standby position and promote duty 2 to duty 1 and the standby pump to duty 2. Duty rotation based on duration (hours) can be enabled through the HMI to equalize the runtimes of the pumps. Once the duration setpoint has been reached the duty 1 pump will be automatically stopped, duty 2 will be promoted to duty 1 and the standby pump will be automatically started after a brief time delay has expired provided the Operator has selected the option to run two pumps on the HMI. To start the pumps in remote-automatic mode, Operations staff can press either the “Start Duty 1 Pump” pushbutton or “Start Duty 2 Pump” pushbutton on the HMI and stop the pumps by pressing either the “Stop Duty 1 Pump” pushbutton or “Stop Duty 2 Pump” pushbutton. To select the “Stop Duty 1 Pump” pushbutton the Duty 2 Pump must be stopped. The raw water pumps will be hardwired interlocked with the wet well low-level float and will not be permitted to run in local or remote modes of operation if a wet well low-level event is active. In addition, in remote mode, the virtual low-level alarm from the wet well level transmitter will be used as a software interlock in order to prevent the pumps from running during a wet well low-level event. The wet well level will be monitored through a level transmitter. The following level transmitter and float switch signals are wired to the PLC:

- Level Indication
- Level Loss of Echo Alarm
- Liquid Level High Alarm
- Liquid Level Low Alarm

The pumps operate continuously, modulating between 75% - 100% of its rated speed, based on a pre-set pressure at VC 12 and/or flow rate from the flow meter at the PS in order to achieve the desired flow velocity in the pipeline. The setpoints will be adjusted during commissioning to achieve the required hydraulics in the pipeline (to avoid excessive velocity) such that the pump operates always at a constant speed while running in local or remote mode.

Discharge pressure will be monitored through a pressure transmitter with the following signals wired to the PLC:

- Pressure Indication
- Pressure Transmitter Fault

Discharge flow will be monitored through a flow transmitter with the following signals wired to the PLC:

- Flow Indication
- Flow Transmitter Fault Alarm

Flow Totalized Flow Indication

A sump pump is installed in the valve room sump pit. The pump starts and stops automatically based on the built-in adjustable float switches. A separate level float in the sump pit is wired to the PLC for flood alarm monitoring. The PLC will monitor the sump pump running status. The following signals are wired to the PLC:

- Valve Chamber Flood Alarm

- Valve Chamber Sump Pump Running Status

5.10.6 Backup Level Control

Under normal conditions the level in the wet well is maintained at an elevation well below the high level float tipping point. In the event that the level in the wet well rises high enough to trip the high level float the float backup mode will be activated. In float backup mode pump 1 starts automatically and runs until the low level float is tipped. If pump 1 does not stop after 15 minutes of running then pump 2 starts. If the low level float does not tip after both pumps are running for another 15 minutes then it is assumed that the low level float has malfunctioned. Subsequently, the pumps will automatically stop, and an alarm will be generated. All timer durations described above will be tested and optimized during commissioning.

5.10.7 Building Services

The access doors to the pump station control room and wet well access room will be monitored by the PLC to provide security and alarm when there is unauthorized intrusion. High and low temperature switches and a smoke detector will be installed in the control room and connected to the PLC to provide monitoring and alarming. The PLC will be connected to the plant SCADA and Ignition Alarm Notification Module for remote monitoring and alarm annunciation. The following signals are wired to the PLC:

- Control Room Door Opened Alarm
- Wet Well Access Room Door Opened Alarm
- Room Temperature High Alarm
- Room Temperature Low Alarm
- Room Smoke Detection Alarm

5.10.8 River Level and Flow Monitoring

The water levels at the Apex River will be monitored by pressure based level sensors. The power and signal wiring for the sensor at Apex River will be wired directly to the pump station PLC control panel. Signal converters will be used to convert the analog (4-20mA) sensor signals to Modbus TCP/IP. A network switch in the enclosure connects the sensors to the PLC network via fibre optic cable routed to the pump station.

AR flow monitoring will be done by either tie-in to the WSC flow stations or by installation of an area-velocity meter in the river. This is being confirmed firstly through discussions with WSC and will be finalized by the 90% design stage. Flow monitoring is required to maintain flows within the approved daily water taking limits.

6 Conveyance

In this project, the term "conveyance" refers to the transfer of water from Lake Qikiqtalik or Apex River to the new reservoir and to Lake Geraldine via the pipeline system. The primary goal remains the establishment of a robust and sustainable conveyance system that not only meets water demand but also emphasizes technical parameters such as energy efficiency, environmental impact mitigation, and the enduring operational dependability of the pipeline network.

6.1 General

Design of the elements of this project are in accordance with the following criteria and standards:

- City of Iqaluit Municipal Design Guidelines.

- Applicable OPSS elements (as design references given these standards do not exist in Nunavut).
- Canadian Standards Association (CSA) standards.
- American Waterworks Association (AWWA).

Design drawings are included in **Appendix A**. Draft specifications are included in **Appendix D**.

6.2 Existing Conditions

The existing roadway network in the area consists primarily of unpaved roads. Notably, the “Access Road to Lake Qikiqtaalik” connects the lake to the “Road to Nowhere,” which in turn links to an access road adjacent to the Iqaluit Shooting Range and ultimately leads to the Apex River.

These roads incorporate multiple culverts designed to manage rain and stormwater runoff, as well as the freshet, effectively, minimizing water accumulation along the roadside.

Two high-density polyethylene (HDPE) pipelines are currently in place:

- A 300 mm HDPE pipe pumps water from the Apex River to Lake Geraldine.
- A 400 mm HDPE pipe transfers water from Lake Qikiqtaalik and discharges it upstream of the Apex River. This pipeline is not currently used.

Both pipelines are laid directly on the ground surface and remain exposed to environmental conditions. A condition assessment of these pipelines is outside the scope of this report.

The pipeline alignments traverse hilly, rocky terrain characterized by continuous permafrost and several wetland areas.

6.3 Design Criteria

6.3.1 Watermain Location, Length and Sizing

The new Raw Water Transmission pipes are outlined as followed.

LQ RWPS:

- A 400 mm HDPE DR11 Raw Water Transmission Main (RWTM) is proposed to convey raw water from Pumping Station PS1 to a valve chamber.
- At the valve chamber, the flow will be split via a wye connection into two parallel 300 mm HDPE DR11 RWTM.
- The 400 mm main is approximately 65 meters in length, while each of the 300 mm parallel mains will extend for approximately 3.9 kilometers.

AR RWPS:

- Route 1: A single 400 mm HDPE DR11 RWTM is proposed to run from the AR pump station to the new reservoir, with an approximate length of 286 meters.
- Route 2: A 400 mm HDPE DR11 RWTM is proposed to convey raw water from AR RWPS to a valve chamber. At this chamber, the flow can be redirected based on operational requirements to either:
 - the New Reservoir via approximately 500 meters of 400 mm HDPE DR11 RWTM.
 - Lake Geraldine, via approximately 800 meters of the same pipe type.

The proposed pipelines have been aligned along existing road corridors to enhance accessibility and reduce environmental disturbance. Installation will involve minor sub-excavation of the existing ground surface, following clearing and grubbing as necessary. The pipelines will be placed on prepared bedding and backfilled with selected material to form a protective berm. At the Apex River crossing, the four pipelines will be supported on a shared utility bridge comprising a platform mounted on structural piers.

Refer to design drawings in **Appendix A**.

6.3.2 Pipe Material

The proposed pipe material for the RWTMs complies with the municipal design guidelines established by the City of Iqaluit. In accordance with these guidelines, the watermain will be constructed using High-Density Polyethylene (HDPE) DR11 (Series 160, 1100 kPa). All pipe materials shall conform to CSA B137.1 and ASTM standards F714, D3035, and D3350.

For fittings, molded components must meet ASTM D2683 or D3261 specifications. Fabricated fittings will be manufactured from pipe of the same series as the main system to ensure material compatibility and performance consistency.

Although the City’s guidelines specify cast iron gate valves, we recommend the use of ductile iron gate valves for improved strength and durability, aligning with current industry best practices.

A comparative table of pipe material options is provided below for reference.

Material \ Criteria	HDPE	PVC	CPP	DI
Strength	Medium	Medium	Very high	High
Handling negative pressure	Good	Moderate	Excellent	Good
Corrosion resistance	Excellent	Excellent	Good	Moderate
Cost (Including FOB site)	Moderate	Moderate	High	Moderate
Installation	Moderate	Easy	Easy	Moderate
Durability	High	Good	Very high	High
Weight	Light	Light	Heavy	Medium
Freeze/ Thaw resistance	Excellent	Good	Good	Moderate
Flexibility	Very high	High	Low	Moderate
Surge capacity	High	Moderate	Moderate	Moderate
Hydraulic efficiency	High	High	Moderate	Moderate

City pipe material experience	High	Good	Low	Low
Preferred	Less Preferred	Least Preferred		

Abbreviations: HDPE: High Density polyethylene, PVC: Polyvinyl chloride, CPP: Concrete Pressure Pipe. DI: Ductile Iron.

6.3.3 Design Flow

A C-factor of 120 for HDPE pipe aligns with the municipal design guidelines of the City. Literature review indicates that a typical C-factor of 150 is suggested for plastics and/or HDPE pipes. Due to uncertainty of friction losses along the RWTM pipelines, the design C-factors of 120 and 150 have been considered. As noted in Section 3.3, flow rate has been determined keeping in mind the spring freshet – and the need to pump a greater volume of water during this time (e.g., June). A balance between pump flow rate and the hydraulics of the pipeline is needed to address surge pressure in the pipeline. To meet the hydraulic requirements, the design C-factors of 120 and 150 are used.

Using the C-Factor of 150, the design flow 556 L/s for LQ system and 400 L/s for AP, the system will be commissioned to maintain pipe full-flow and/or to reduce the pipe velocity due to the steep slope of the pipeline in the initial reaches of the pipe following the pump station. Sensitivity to the C-factor will affect the final achievable flow.

6.3.4 Design Velocity

The RWTM velocities aligns with the municipal design guidelines of the City which states a maximum velocity of 3.5 m/s. Pipe velocity based on the design to date indicates pipe velocities could reach 3 m/s. As per the City’s design guidelines, special design provisions to stabilize the line shall be incorporated when design velocity exceeds 3.0 m/s. For the LQ system, the required maximum operational flow and hydraulic conditions will be checked if the flow can be slightly reduced by adjusting the sustaining pressures at upstream of the flow control valves at RWTM outlets (e.g., pipe velocity less than 3.0 m/s) at the commissioning stage. VFD pump control will be pre-set during commissioning to maintain the desired maximum flow. This VFD setpoint can also be modified as the system ages if the C-factor changes.

The projected RWTM velocities for the AR system are less than 2.5 m/s and meets a maximum velocity of 3.5 m/s as suggested by the City.

6.3.5 Design Pressure

For the RWTMs and the discharge pipeline, the maximum Field Test Pressure (also known as hydrostatic test pressure) will be the higher of 1.5 times the system design working pressure or the Maximum Allowable Working.

For the LQ system, the Field Test Pressure should be at least 1378 kPa (200 psi) at the lowest point in the test sections of the RWTMs. For the AR system, the Field Test Pressure should be at least 1034 kPa (150 psi).

The procedure and test duration will follow the appropriate AWWA manual as well as the pipeline manufacturer’s recommendations. Note that the RWTM pipelines may need isolation of test sections not to exceed pipe material maximums due to the pipeline profile and the inherent pressure head. All pipe materials provide an allowance for surge pressures over and above the design working or operating pressure. The allowance for a recurring surge during normal operation is 50% (over and above) of the pipe material (HDPE) pressure class, and the allowance for an occasional surge such as a system interruption or failure is 100% (over and above) of the pipe material (HDPE) pressure class.

The pipe and fittings are designed and specified to be suitable for the Working Pressure, Maximum Transient/Surge Pressure, and Field Test Pressure.

Pressure monitoring devices will be located within the valve chambers along the LQ RWTMs to monitor pressure. Specifically, automated pressure transmitters are recommended at four locations (VC1 and VC10 for both RWTMs). Due to the absence of power along the pipe alignment to support the pressure transmitters, manual pressure gauges will be located at the additional valve chambers.

Pressure monitoring devices will be located within the valve chambers along the AR RWTMs to monitor pressure. Specifically, automated pressure transmitters are recommended at minimum four locations (VC10, VC 11, VC12, and VC15). Due to the absence of power along the pipe alignment to support the pressure transmitters, manual pressure gauges will be located at the remaining valve chambers.

6.3.6 Depth of Cover

In accordance with the City of Iqaluit’s municipal design guidelines, the standard minimum burial depth for watermains is 2.5 m below finished grade. However, due to the unique operational conditions of this project—where the Raw Water Transmission Main (RWTM) will function only during the summer months—this requirement is not applicable.

For this specific case, the pipe will be installed near the existing ground elevation along the alignment. To provide additional protection, a berm will be constructed above the pipe. This berm will consist of approximately 1.0 m of cover over the pipe and will extend 2.5 m laterally on either side.

The berm will be composed of two layers:

- An internal layer of 300 mm of sand, wrapped in a geotextile membrane.
- An external layer of 700 mm of crushed stone with a nominal size of 150 mm (6").

This layered design enhances structural stability and provides resistance against wind and other environmental factors. The berm’s side slopes will be constructed at a 2:1 ratio, facilitating easier crossing for wildlife and snowmobiles while improving overall stability.

The depth of the active layer in the project area is approximately 2.0 m, and the pipe will be installed within this layer. Since the pipe will be drained and left empty at the end of each operational season, it will freeze during the winter months. As a result, the presence of permafrost does not pose a concern for this installation, and no additional measures are required to mitigate its effects.

Although the berm may help moderate temperature fluctuations around the pipe, the HDPE material will still undergo thermal expansion and contraction between seasons. To mitigate potential axial and lateral movement, anchor blocks will be installed along the pipe.

6.3.7 Crossings

Table 6-1 provides design requirements for crossings.

Table 6-1 Design Criteria for Crossings

Type of Crossing	Authority	Criteria	Remarks
Utility Crossings	City of Iqaluit	0.3 m minimum clearance if crossing above utility, or 0.5 m clearance if crossing below utility.	The only anticipated utility to be crossed is the drainage culverts. (Road cross culverts)

Type of Crossing	Authority	Criteria	Remarks
Apex River Crossing	Nunavut Impact Review Board (NIRB) Department of Fisheries and Oceans (DFO) Nunavut Water Board (NWB)	The RWTM will utilize an aerial crossing to pass over the river, enclosed in a steel casing and supported by piers 2.4 m above the water level.	Permitting agencies will be able to comment on the design of the river crossing as part of permitting submissions.
Rural Roads Crossings	City of Iqaluit	RWTM will cross rural roads at a minimum depth of 600 mm, following best engineering practices.	Construction equipment loading will also be considered during construction/live loads.

6.3.8 Thrust Restraints

The recommended pipeline material is HDPE, which supports various thrust restraint methods. These include butt fusion joints, bolted flange connections, and mechanical joint (MJ) adaptors equipped with appropriate restraint rings and tie-rod assemblies. A wide range of couplings is available from multiple manufacturers and distributors.

The design accounts for thermal expansion and contraction, as well as the Poisson effect during pipeline pressurization in both testing and operational phases. Certain sections may require external bracing, such as concrete thrust blocks or reinforced anchor structures with tie-downs, to manage anticipated forces.

Restrained length calculations have been performed based on project-specific parameters, including depth of cover (from top of berm to top of pipe), soil envelope, and other relevant conditions. These calculations follow guidance from AWWA manuals and manufacturer recommendations for the selected pipe and coupling systems. The results inform the placement of components such as expansion/contraction joints and the use of fully restrained couplings at critical locations. These components will be added as design progresses, indicating final locations.

It is anticipated that fusion joints will be used for the majority of pipeline connections.

6.3.9 Drain Time

City of Iqaluit municipal design guidelines do not specify a drain time for watermains. Manually operated drain valves will be strategically placed along the RWTM using best judgment and engineering practices, in such a way that the drain time of the pipe section in each chamber is less than or equal to 15 hours (based on best judgement and engineering practice).

For LQ system, RWTMs slope downhill except for two sections near VC8 (and/or above the Apex River) plus another four sections near VC3 and VC5 for both RWTMs. The water in RWTMS continues flowing downstream and discharging at the outlets to the reservoir. Six drain valves (two-150 mm at VC8 and four-100 mm (at VC3 and VC5) will be located at these drain valve chamber locations for both RWTMs. The type and size of SCAV are to be revisited during the detailed design stage. The draining time via the drain valve at each VC location and discharging outlet to reservoir is less than 1 hour. The main pipeline drain location will include chambers complete with person access and appropriate drain discharge size and configuration.

For AR system, the section of RWTM near LQ (or section downstream of VC14) slopes downhill and/or above the Lake Geraldine). The water in RWTM continues flowing downstream and discharging at the outlets to the LG.

Four drain valves (two-150 mm at VC9 and VC11) will be located at these drain valve chamber locations. The draining time via the drain valve at each VC location and discharging outlet to Apex River or ground is less than 5 hours. The main pipeline drain location includes chambers complete with person access and appropriate drain discharge size and configuration.

6.3.10 Watermain Horizontal Alignment

The horizontal alignment of the raw water transmission main plays a critical role in ensuring efficient and reliable water conveyance. Alignment design must incorporate a comprehensive assessment of topographic features, environmental constraints, and existing infrastructure to determine the most feasible routing. The objective is to minimize directional changes, such as bends and curves, which can lead to increased head loss, reduced hydraulic efficiency, and maintenance challenges. The alignment must also balance direct routing with practical limitations, including environmental sensitivities and constructability considerations. All design decisions should conform to applicable industry standards and municipal guidelines to ensure long-term system integrity and regulatory compliance.

The alignment of the raw water transmission pipeline from LQ Pump Station will primarily follow existing roadways wherever feasible. This approach facilitates access during construction and future maintenance, while minimizing the need for new road development and reducing environmental impacts.

The proposed route begins adjacent to the access road leading to LQ and extends southward until it intersects with the Road to Nowhere. From there, the pipeline continues along the access road, passes the shooting range, and terminates at the east bank of the Apex River.

At this location, the raw water transmission pipelines from AR Pump Station will join the same alignment and cross the Apex River via a proposed utility bridge. The bridge will consist of a platform supported by piers, elevated 2.4 meters above the defined high-water level.

On the west bank of the Apex River, the alignment of the two pipelines from LQ Pump Station and one of the pipelines from PS2 will continue westward until reaching the new reservoir.

The second pipeline from AR Pump Station will follow the same alignment as the other three pipelines for approximately 200 meters westward, up to a proposed chamber. At this chamber, it will split into two branches: one will continue to the new reservoir alongside the other pipelines, while the other will divert westward, running along the southern edge of the new reservoir before reaching Lake Geraldine.

Horizontal and vertical alignment of the pipeline will continue to be refined as detailed design progresses to ensure compliance with manufacturer tolerances and to optimize constructability, hydraulic performance, and long-term operational integrity. Temporary construction access roads, each with a minimum width of 4.0 meters, will be established parallel to pipeline segments located beyond the reach of existing access infrastructure. These roads will be designed to accommodate the anticipated load and turning radii of construction vehicles and equipment, thereby facilitating uninterrupted access for excavation, pipe installation, and inspection activities throughout the construction period.

6.3.11 Transient Analysis and Pressure Rating

The software program Bentley HAMMER was used to undertake the hydraulic transient analysis for the RWTMs for both LQ and AR systems. The worst-case transient condition occurs for the RWTMs upon pump trip (e.g., power failure). This has been analyzed and the results are summarized below.

6.3.11.1 LQ Pump Station

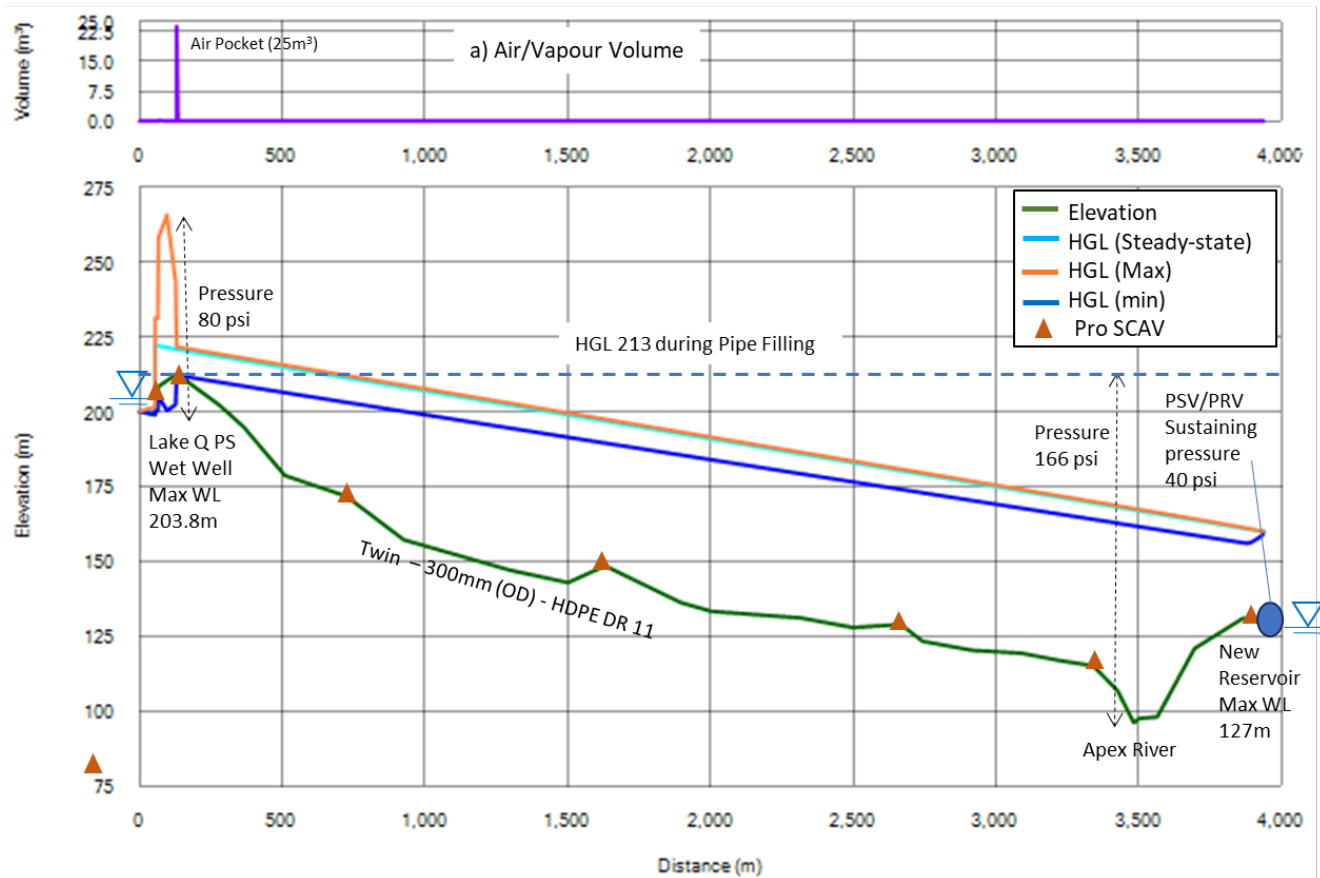
The following assumptions have been made:

- Two duty pumps are in place and initially operational and then trip simultaneously.
- A check valve is present at each of the pumps to limit reverse flow at each pump.

- A 150 mm Surge Anticipated Valve (SAV) is provided at the LQ PS
- A pressure wave speed of 350 m/s (for HDPE DR11 pipe) and 1100 m/s for SS pipe.
- A flow of 255 L/s via the RWTM system.
- The pumps each will be completed with VFD.
- A C-Factor of 150 (for a conservative design in term of transient pressure).
- Near low water level (e.g., 200 m with ice) at Lake Qikiqtaalik (supply source).
- RWTMS (HDPE DR11) Pressure: Maximum Allowable Working Pressure is 200 psi and Maximum Allowable Total Pressure (including Surge Pressure) is 300 psi as per the manufacture's information.
- PS Discharge Header (316 SS) Pressure: Maximum Allowable Working Pressure is 150 psi and Maximum Allowable Total Pressure (including Surge Pressure) is 190 psi as per the manufacture's information.
- Two Pressure Sustaining/Pressure Reducing Valves (PSV/PRVs) - one is provided at each discharge outlet. A strainer with automatic flushing device is recommended to provide at upstream of each PSV/PRV, as the pipeline will contain raw water that may have solids.
- Two 100 mm by-pass lines with two small PSV/PRVs – one is provided at each discharge outlet and parallel to the main PSV/PRVs
- Surge protection devices: A total of 13 sewage combination air valves (SCAVs) is proposed for the RWTM system. One small SCAV (25 mm) at the discharge side of the LQPS and 12 SCAVs (150 mm) at six VC locations (VC 1, VC2, VC4, VC6, VC7 and VC9) for both RWTMs. The type and size of SCAV are to be revisited during the detailed design stage. The SCAV attached with automatic flushing device is recommended.

Figure 6-1 shows the elevation profile and minimum/maximum hydraulic grade lines (HGLs) for the RWTM from the LQ pump station to the discharge outlet, upon pump emergency shutdown and restart slowly (e.g., pump ramp up time at least 2 minutes) with the SCAV protections.

Figure 6-1 Hydraulic Profile of LQ System



The key findings for the RWTMs are summarized as follows:

- The maximum steady-state (or working) pressure along the entire RWTM route is approximately 63 psi under normal operation. The system pressure is approximately 166 psi for the low point sections near the Apex River for filling the RWTM(s) during the spring system restart.
- The maximum transient pressures (including surge) reach 75 psi under normal operations and 166 psi during RWTM(s) filling and are below the Maximum allowable Working Pressure and/or Field Test Pressure of 200 psi.
- Short-lived sub-atmospheric pressure occurs along the entire both RWTMs and at the discharge side of the pump station.
- Three SCAVs including one SCAV at the PS discharge header and two each at VC1 for both RWTMs that trip open to allow air entrainment and minimize the vacuum pressure for the system. The other SCAVs don't activate since vacuum pressures is not detected at these locations.
- Up to 25 m³ air pocket are detected at the high points at both VC1 of RWTMs.

The subject RWTMs should be capable of withstanding short-lived full vacuum and/or negative pressure conditions. It is recommended to install two flow meters one for each RWTM to measure the flow to each RWTM.

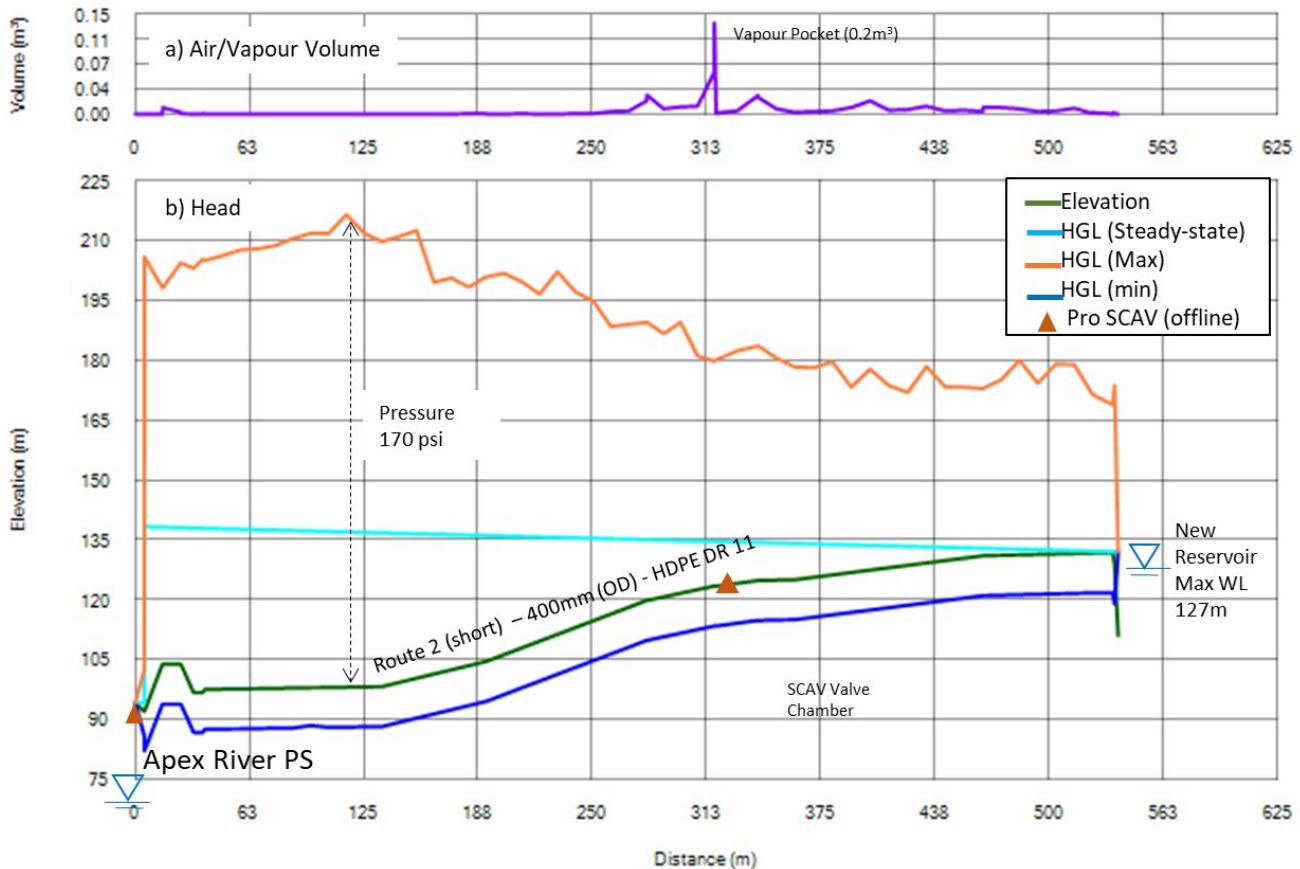
6.3.11.2 AR Pump Station

For the AR system, a hydraulic and transient analysis was performed as shown below.

The following assumptions have been made (refer to Figure 1-1 for summary of the piping routes):

- Two primary routes ((Route 1 and short route of Route 2 (Route 2-short)), from AR PS to the new reservoir are proposed and to be operated under normal operation – see **Figure 6-2** for the pipeline profile.
- Another Route (Route 2-long) is proposed and to be operated occasionally by closing the isolation valve at VC 14 to switch Route 2-short to Route 2-long. Route 1 is offline under this condition. See **Figure 6-3** for the pipeline profile.
- Route 2-long will be operated independently (i.e., without connection to Route 2-short or diversion flow to Geraldine Lake), a PSV/PRV does not be required at each outlet.
- Two duty pumps are in place and initially operational and then trip simultaneously.
- A check valve is present at each of the pumps to limit reverse flow at each pump.
- A 150 mm Surge Anticipated Valve (SAV) is provided at the ARPS.
- A pressure wave speed of 350 m/s (for HDPE DR11 pipe) and 1100 m/s for SS pipe.
- A total flow of 400 L/s - 200 L/s via each short route.
- Approximately 180 L/s via long route of Route 2 (Route 2-long).
- The pumps each will be completed with VFD.
- A C-Factor of 150 (for a conservative design in term of transient pressure).
- Near low water level (e.g., 94 m) at Apex River (supply source).
- Pipelines (HDPE DR11) Pressure: Maximum Allowable Working Pressure is 200 psi and Maximum Allowable Total Pressure (including Surge Pressure) is 300 psi as per the manufacture's information.
- PS Discharge Header (316 SS) Pressure: Maximum Allowable Working Pressure is 150 psi and Maximum Allowable Total Pressure (including Surge Pressure) is 190 psi as per the manufacture's information.
- One Pressure Sustaining/Pressure Reducing Valve (PSV/PRV) is provided at the Route 2-long discharge outlet. A strainer with automatic flushing device is recommended to be provided upstream of the PSV/PRV, as the pipeline will contain raw water that may have solids.
- A 100 mm bypass line with one small PSV/PRV is provided at discharge outlet and parallel to the main PSV/PRV.
- Surge protection devices: A total of 8 sewage combination air valves (SCAVs) are proposed for the AR pipeline system. One SCAVs at the discharge side of the ARPS and 7 SCAVs (150 mm) at other VC locations for both routes. The type and size of SCAV are to be revisited during the detailed design stage. The SCAV attached with automatic flushing device is recommended.

Figure 6-2 Hydraulic Profile of AR System for short Twin Routes from ARPS to New Reservoir



The key findings for the AR short routes (Route 1 and Route 2-short) are summarized as follows:

- **Figure 6-2** shows the hydraulic profile of AR system for Route 1 and Route 2-short with 200 L/s via each route upon pump emergency shutdown without surge protections
- The elevation profile of the pipeline (Route 1 and Route 2-Short) from Apex River PS to the new reservoir indicates a generally rising slope.
- The maximum steady-state (or working) pressure along the entire AR short routes is approximately 65 psi under normal operation. The system pressure is approximately 170 psi for the low point sections, approximately 100 m from ARPS and are below the Maximum allowable Working Pressure and/or Field Test Pressure of 200 psi.
- Short-lived sub-atmospheric/full vacuum pressure occurs along the entire both Routes and at the discharge side of the ARPS.
- Up to 0.2 m3 vapour pocket are detected at the high points at both Route 1 and Route 2-short.

Figure 6-3 shows Route 2-long for flow diversion from ARPS to Geraldine Lake, a PSV/PRV is needed to remain full flow for the elevated section along Rotue 2-long.

Figure 6-3 Hydraulic Profile of AR System for Long Route 2 from ARPS to LG

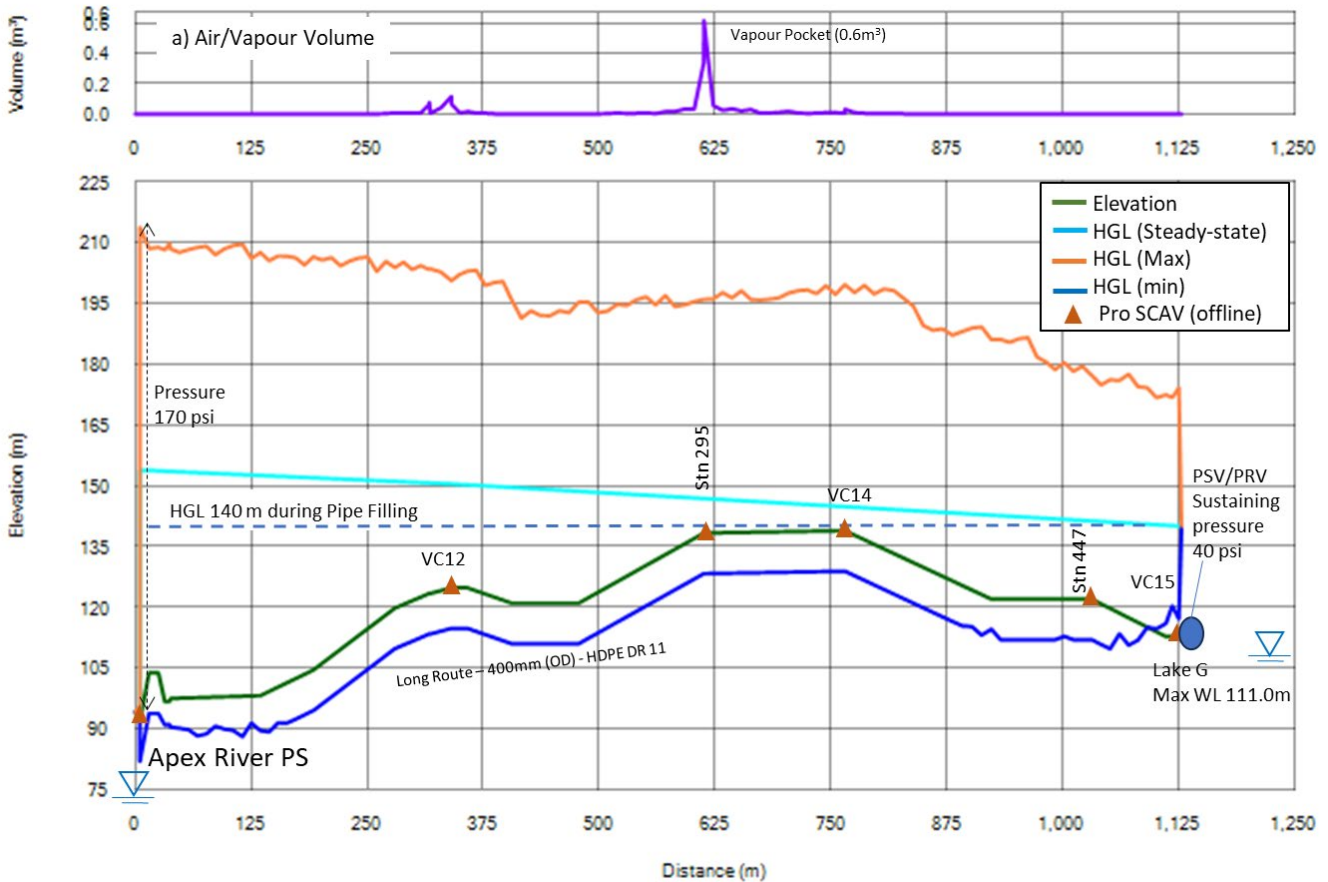


Figure 6-3 shows the hydraulic profile of AR system for Route 2-long upon pump emergency shutdown without surge protections. The key findings Route 2-long are summarized as follows:

- A downhill-sloped and/or elevated section of watermain has been identified along Route 2-Long from ARPS to LG. The recommended pressure sustaining valve setting is 40 psi, equivalent to a HGL of 141 m at the outlet.
- The maximum steady-state (or working) pressure along the entire AR short routes is approximately 65 psi under normal operation. The system pressure is approximately 170 psi at the APPS discharge are below the Maximum allowable Working Pressure and/or Field Test Pressure of 200 psi.
- Short-lived sub-atmospheric/full vacuum pressure occurs along the entire both routes and at the discharge side of the ARPS.
- Up to 0.6 m³ vapour pocket are detected at the local high point at approximately 600 m from ARPS
- The subject pipeline should be capable of withstanding short-lived full vacuum and/or negative pressure conditions. It is recommended to install two flow meters at APRPS to measure the flow to each route.

6.4 Valves and Valve Chambers

Based on the pipe layout, vertical alignment of the pipe and the requirements of the city, the RWTM will be provided with main line, drain and SCAVs (or other surge protection option as described in Section 4.6.4). Valves are shown on the design drawings.

6.4.1 Valve Chambers

The following valve chamber design assumptions have been made:

- Steel access vaults similar to those currently in place in Iqaluit will be utilized.
- Access vaults must be sized to allow adequate space to house equipment and for operations and maintenance personnel to access equipment for servicing.
- Steel vaults must be located to allow for easy access for operations and maintenance crews, including access paths/roads where required.
- Confined space issues and lock-out requirements may be required based on the City's health and safety requirements.
- Prefabricated structures will be used, when possible, to minimize on-site construction work.
- Valves will be operable from grade level using valve stems.

6.4.2 Emergency Pumping Valve Chamber

The valve chamber on the Apex River conveyance pipeline closest to the ARPS (VC 9) will also feature a blind flange on each pipeline, which will enable emergency pumps to be placed in the Apex River that can be connected to the conveyance piping.

6.4.3 Main Line Valves

Automated ball valves will be located along the pipeline as outlined in our design drawings. Also, for operational purposes, pressure-sustaining valve will be located at the pipe outlets to regulate flow and facilitate the filling of the pipe with water.

The purposes of line valves along the RWTM route are to:

- Isolate pipe sections for maintenance and repair.
- Control the volume of water wasted when attempting to repair a problem.
- Control the "down time" associated with isolating a problem, draining the impacted area, repairing affected sections, and returning the line back into service.

Although gate valves are recommended per the City's guidelines for mainline valves, the design proposes the use of ball valves instead. This choice is driven by their superior performance in raw water systems, particularly in the presence of grit. Ball valves exhibit increased resilience, being less susceptible to clogging and damage by grit, and they require less maintenance.

6.4.4 Drain Valves and Structures

Following the design criteria of the City, the installation of drain valves is required at the low points of all watermains. Water inside the RWTMs is required to be drained for operation and maintenance purposes. The pipes have a substantial length, enabling them to store a significant volume of water. This amount of water needs to be drained within a reasonable period of time. The topography of the area naturally allows the RWTMs to drain by gravity along its whole length. A significant portion of the RWTMs will naturally drain from north to south, flowing from Lake Qikiqtaaluk southwards to the Apex River. Simultaneously, another segment will naturally drain in the west to east direction, moving from the new reservoir to Apex River.

A significant portion of the pipe will be drained into the reservoir, limited by the available head. The remaining drainage will be facilitated by drain valves strategically placed along the pipes.

6.4.5 Air Valves and Structures

In adherence to the City's Design Criteria, air release valves are mandated to be installed at the high points of all watermains. These valves play a crucial role on watermains, strategically placed at significant elevations facilitating the release of trapped air. The presence of air pockets within the system can lead to a substantial reduction in flow capacity, increased energy consumption, and the risk of pressure surges and water hammer in pipelines.

Air release valves are instrumental in preventing these issues. During the filling of the line, they exhaust air, ensuring optimal operation. In the event of line drainage or surge pressure problems, vacuum valves respond to pressure loss by opening, allowing air to re-enter the pipelines, and averting potentially damaging vacuum conditions.

For the LQ system 150 mm Sanitary Combination Air Valves (SCAV) have been strategically placed within the valve chambers, specifically VC1, VC2, VC4, VC6, VC7, and VC9. The selection of VC1, VC4, VC6, and VC9 corresponding to high points in the pipeline to effectively address air pocket concerns. Air valves in VC2 and VC7 are placed based on hydraulic analysis requirements, particularly to mitigate the issue of vacuum in the pipe due to its sharp slope. The choice of SCAV is recommended due to the nature of the raw water and the presence of grit, aiming to prevent clogging and minimize required maintenance, thereby optimizing the long-term performance of the air valve system.

For the AR system 150 mm Sanitary Combination Air Valves (SCAV) have been strategically placed within the valve chambers, specifically VC11, VC14, VC15 and VC16 (at the short Route 2 outlet) corresponding to high points in the pipeline to effectively address air pocket concerns, particularly to mitigate the issue of vacuum in the pipe due to its sharp slope. The choice of SCAV is recommended due to the nature of the raw water and the presence of grit, aiming to prevent clogging and minimize required maintenance, thereby optimizing the long-term performance of the air valve system.

6.5 Trenchless Aerial Crossings

The RWTMs are proposed to cross the Apex River within steel casings, supported by a utility bridge. The bridge will consist of a platform/beam mounted on piers spaced at 6 m intervals.

The river channel and floodplain span approximately 45 m at the crossing location. To maintain alignment with existing ground elevations, the steel casings will extend across a total length of 90 m.

Figure 6-4 shows the proposed river crossing location. Design drawings of the crossing are included in **Appendix A**.

Figure 6-4 Apex River Crossing Location



6.6 Hydroelectric Power Generation

Given the significant elevation difference from the RWPS to the new reservoir there is a potential for microelectric power generation (i.e. power generation less than 100 kW). This potential has not yet been fully analyzed, and will be completed prior to completing the design documents. It is likely that capital funding available will not be sufficient to include this component of the project, even if feasible.

6.7 Drainage, Erosion and Sediment Control

The drainage plan for the RWTM route optimizes the natural topography, using the ground slope to guide the discharge of water from the pipe along the terrain contours. Water discharged from drain valves will be directed through the pipe and onto rip rap. This controlled discharge onto rip rap serves the purpose of energy dissipation and erosion prevention. The channeled water, following the existing topography slope will be directed towards the nearest watercourse or culvert.

The berm will intersect with existing culverts under current access roads. These culverts will undergo necessary extensions or realignments to maintain proper stormwater drainage. A thorough evaluation of their condition will be conducted to ensure they can withstand the live load of construction machinery and the additional dead load due to increased fill. Additionally, new culverts will be introduced along the alignment as needed to uphold effective stormwater drainage and prevent ponding.

6.8 Instrumentation and Control

6.8.1 Lake Qikiqtalik Flow Control and Pressure Monitoring

Discharge flow from LQ RWPS is supplied by two pipelines to the new reservoir approximately 4.2 km away. Motorized ball valves and pressure sensors are installed on each of the pipelines in the two valve chambers located at approximately 80 m and 4 km downstream of the pump station, herein referred to as Valve Chamber 1 (VC1) and Valve Chamber 10 (VC10), respectively. The valve actuators and pressure sensors will be equipped with remote control capability via Ethernet comms to the PLC using standard industrial communications protocols. The equipment connected to SCADA in each valve chamber is listed below.

VC1:

- BV03 Ball valve
- BV04 Ball valve
- PT01 Pressure sensor
- PT02 Pressure sensor

VC10:

- BV23 Ball valve
- BV24 Ball valve
- PT03 Pressure sensor
- PT04 Pressure sensor
- PSV01 Pressure sustaining valve
- PSV02 Pressure sustaining valve

The motorized ball valves will each have the following signals to the PLC:

- Local/remote status
- Valve fault
- Valve fully open status
- Valve fully closed status
- Valve open command
- Valve close command

The motorized pressure sustaining valves will each have the following signals to the PLC:

- Local/remote status
- Valve fault
- Valve fully open status
- Valve fully closed status
- Valve position feedback
- Valve position setpoint

The pressure sensors will each have the following signals to the PLC:

- Pressure transmitter fault
- Pressure indication

VC1 – The valve actuators and pressure sensors in VC1 will connect to the PLC IO rack in the RWPS. The PLC IO rack will be installed in a free-standing enclosure. The valve actuators and pressure sensors will be connected by copper patch cables to an Ethernet switch inside valve chamber 1 which is located approximately 80 m from the RWPS. The Ethernet switch with fibre transceivers will connect to the main PLC IO rack via a new fibre optic link between the valve chamber 1 and RWPS. The fibre link will be routed along the hydro poles installed alongside the pipeline. The fibre cable will be 6-strand single mode and terminated at each end on fibre patch panels. A media converter installed in the PLC enclosure will be used to enable a connection to the fieldbus communications module on the PLC IO rack. The PLC IO rack will be connected to the SCADA WAN, as described in Section 4.10.1. Inside the valve chamber there will be an enclosure housing the Ethernet switch with fibre transceivers, fibre patch panel, 24VDC power supply, and a panel heater controlled by a thermostat. The panel will be powered from a 120VAC source with backup generator power.

VC10 – The valve actuators and pressure sensors in VC10 will connect to the PLC IO rack. The valve actuators and pressure sensors will be connected by copper patch cables to an Ethernet switch inside VC10 which is located approximately 4.1 km from the RWPS. The Ethernet switch with fibre transceivers will connect to the PLC IO rack via a new fibre optic link between the valve chamber 10 and the RWPS. The fibre link will be routed along the hydro poles installed alongside the pipelines.

6.8.2 Apex River Flow Control and Pressure Monitoring

Discharge flow from Apex River Raw Water Pump Station (RWPS) is supplied by two separate pipelines. One line to Lake Geraldine, approximately 600 m away, and the other to the new reservoir, approximately 300 m away. Motorized ball valves and pressure sensors are installed along the pipelines in some of the valve chambers. The discharge to Lake Geraldine routes through valve chambers VC12 and VC15. Diversion valves in valve VC12 provides an alternate route to the new reservoir instead of Lake Geraldine through VC11. The other pipeline from the pump station discharges to the new reservoir routing through VC11. The valve actuators and pressure sensors will be equipped with remote control capability via Ethernet comms to the PLC using standard industrial communications protocols. The equipment connected to SCADA in each valve chamber is listed below.

VC11:

- BV11 Ball valve
- BV13 Ball valve
- PT05 Pressure sensor
- PT06 Pressure sensor
- PSV04 Pressure sustaining valve
- PSV05 Pressure sustaining valve

VC12:

- BV04 Ball valve (diverts flow to Lake Geraldine)
- BV05 Ball valve (diverts flow to the reservoir)
- PT07 Pressure sensor
- PSV01 Pressure sustaining valve

- PSV02 Pressure sustaining valve

VC15:

- BV09 Ball valve
- PT08 Pressure sensor
- PSV03 Pressure sustaining valve

The motorized ball valves will each have the following signals to the PLC:

- Local/remote status
- Valve fault
- Valve fully open status
- Valve fully closed status
- Valve open command
- Valve close command

The motorized pressure sustaining valves will each have the following signals to the PLC:

- Local/remote status
- Valve fault
- Valve fully open status
- Valve fully closed status
- Valve position feedback
- Valve position setpoint

The pressure sensors will each have the following signals to the PLC:

- Pressure transmitter fault
- Pressure indication

VC11 – The valve actuators and pressure sensors in VC1 will connect to the PLC IO rack in the RWPS. The PLC IO rack will be installed in a free-standing enclosure. The valve actuators and pressure sensors will be connected by copper patch cables to an Ethernet switch inside valve chamber 11 which is located approximately 300 m from the RWPS. The Ethernet switch with fibre transceivers will connect to the main PLC IO rack via a new fibre optic link between the valve chamber 11 and RWPS. The fibre link will be routed along the hydro poles installed alongside the pipeline. The fibre cable will be 6-strand singlemode and terminated at each end on fibre patch panels. A media converter installed in the PLC enclosure will be used to enable a connection to the fieldbus communications module on the PLC IO rack. The PLC IO rack will be connected to the SCADA WAN, as described in Section 5.10.1. Inside the valve chamber there will be an enclosure housing the Ethernet switch with fibre transceivers, fibre patch panel, 24VDC power supply, and a panel heater controlled by a thermostat. The panel will be powered from a 120VAC source with backup generator power.

VC12 – The valve actuators and pressure sensors in VC12 will connect to the PLC IO rack. The valve actuators and pressure sensors will be connected by copper patch cables to an Ethernet switch inside VC12 which is located approximately 250m from the RWPS. The Ethernet switch with fibre transceivers will connect to the PLC IO rack via a new fibre optic link between the valve chamber 12 and the RWPS. The fibre link will be routed along the hydro poles installed alongside the pipelines.

VC15 – The valve actuators and pressure sensors in VC15 will connect to the PLC IO rack. The valve actuators and pressure sensors will be connected by copper patch cables to an Ethernet switch inside VC15 which is

located approximately 600m from the RWPS. The Ethernet switch with fibre transceivers will connect to the PLC IO rack via a new fibre optic link between the valve chamber 15 and the RWPS. The fibre link will be routed along the hydro poles installed alongside the pipelines.

7 Reservoir

As described in earlier sections of this report, the City is projected to require an additional $\approx 1,643,250 \text{ m}^3$ raw water storage (useable) capacity to meet the demands of an increasing population. The new storage reservoir has been designed to function in conjunction with the existing storage volume in Lake Geraldine, with the conveyance systems described in Section 6. This section of the report presents the reservoir design with relevant site assumptions, hydrological analyses, stability analyses, and construction quantities.

The scope of the reservoir design includes:

- Evaluation of current site conditions and identification of construction materials.
- Desktop assessment of subsurface conditions based on site visits and publicly available data.
- Confirmation of the relevant codes, standards, and guidelines to be considered for design. This includes the initial Dam Classification.
- Hydrological and water management assessments.
- Geotechnical stability analyses.
- Seepage analysis of dam and dykes.
- 50% dam and dyke design.

Design calculations and analyses have been based on conservative estimates of stratigraphy and material properties. Revisions will be made to design and analyses based on site-specific data obtained from further geotechnical investigations to be completed in 2026, including stratigraphic information from boreholes, test pits, and trenches advanced onsite; permeability data from packer testing, piezometer and thermistor data; and any updated material parameters obtained from further laboratory analyses.

7.1 General

The reservoir dam and dykes have been designed in accordance with the following guidelines:

- Dam Safety Guidelines, Canadian Dam Association (CDA) (2013).
- Technical Bulletin: Inundation, Consequences, and Classification for Dam Safety, Canadian Dam Association (2007).
- Technical Bulletin: Surveillance of Dam Facilities, Canadian Dam Association (2007).
- Technical Equipment: Flow Control Equipment for Dam Safety, Canadian Dam Association (2007).
- Technical Bulletin: Dam Safety Analysis and Assessment, Canadian Dam Association (2007).
- Technical Bulletin: Hydrotechnical Considerations, Canadian Dam Association (2007).
- Technical Bulletin: Seismic Hazard Considerations for Dam Safety, Canadian Dam Association (2007).
- Technical Bulletin: Geotechnical Considerations for Dam Safety, Canadian Dam Association (2007).
- Technical Bulletin: Structural Considerations for Dam Safety, Canadian Dam Association (2007).

7.2 Existing Conditions

The reservoir site is located immediately north of Lake Geraldine and comprises the footprints of two existing shallow surface water bodies and the immediately surrounding area. The location of the reservoir was selected as part of previously completed feasibility studies (by others) to take advantage of the existing terrain, topography, and water supply infrastructure. There is no development within the project area and the site is currently only accessible overland by foot or all-terrain equipment. The current site conditions and the extent of the new reservoir area are shown in Drawing B101 in **Appendix A**.

7.2.1 Subsurface Conditions

Refer to the *Preliminary Geotechnical Investigation Report* by Arcadis dated 19 August 2025 for a detailed description of subsurface conditions.

Quaternary geology in the area consists of no cover or thin layers of glaciofluvial drift, with sporadic pockets of organic materials (e.g., sphagnum moss) encountered across the site. Based on the site visits and initial investigations conducted to date, surficial units consist of boulder fields or exposed rock with little to no soils present in the reservoir footprint.

According to the Canadian Geological Survey maps (Geology of Nunavut, 2006), and supported by core samples obtained from the site, bedrock in the area consists of Proterozoic granulite-facies granitoids. This rock is highly competent. It is common for the near surface zone to be frost-fractured.

7.3 Design Overview and Criteria

7.3.1 Reservoir System

The reservoir system comprises eight retention structures in addition to peripheral utilities (e.g., access road). The retention structures are shown on Drawing B101 included in **Appendix A**, and the basic characteristics of each are summarized in Table 7-1.

Table 7-1 Retention Structure Characteristics

Structure	Number	Max. Height* (m)	Approximate Length (m)
Dam	1	15.5	400
Dyke	2	7	150
Dyke	3	7	150
Dyke	4	8	150
Dyke	5	8	50
Dyke	6	12	200
Dyke (Spillway)	7	6	150
Dyke	8	13	200

*Note: maximum heights are estimates. Structure height may be greater or lesser depending on overburden thickness and foundation preparation requirements.

7.3.2 Dam Classification

The Dam Classification for the system focuses predominantly on Dam 1 (the reservoir system, structures, and numbering are shown on Drawing B101); the largest structure to be built as part of this system. This dam has the largest risk for the system, downstream infrastructure, and the population at risk (PAR). As per CDA guidelines, cascade effects must also be considered when a drainage basin has a series of dams. Given that Lake Geraldine would act as the immediate receptor of any volume released by a failure or overtopping of Dam 1, the failure assessment scenarios must consider the impacts on the Lake Geraldine dam and infrastructure downstream of that structure. This includes critical infrastructure such as the water purification plant, power plant and hospital in addition to residential areas of the City.

Based on Table 2-1 of the CDA Dam Safety Guidelines, Dam 1 of the reservoir system would be classified as an 'Extreme' risk. This is based on a) a permanent PAR with the potential loss of life in the event of a catastrophic dam failure estimated at more than 100 lives and b) the probability of extreme losses affecting critical infrastructure or services (e.g., potable water facilities, power plant, hospital). As such, the calculated required elevation for Dam 1 sets the required elevation for all retention structures. Table 2-1 of the CDA Dam Safety Guidelines is included below.

Table 2-1: Dam Classification

Dam class	Population at risk [note 1]	Incremental losses		
		Loss of life [note 2]	Environmental and cultural values	Infrastructure and economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of <i>important</i> fish or wildlife habitat Restoration or compensation in kind highly possible	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of <i>critical</i> fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of <i>critical</i> fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)
<p>Note 1. Definitions for population at risk:</p> <p>None— There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.</p> <p>Temporary— People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).</p> <p>Permanent— The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).</p> <p>Note 2. Implications for loss of life:</p> <p>Unspecified— The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.</p>				

The Dam Classification has ramifications in terms of minimum required Factors of Safety (FoS) to be achieved in stability models, dam instrumentation requirements, inspection frequency, and more. It also affects the required inundation studies – which inform the Emergency Response Plan (internal procedures) and Emergency Preparedness Plan (shared with external stakeholders). Further measures on dam safety management; operations, maintenance, and inspection/surveillance; and analysis and assessment will be completed during the final design phase (for example, the results of the inundation study). The risk classification of each retention structure, based on the CDA criteria above, is summarized in Table 7-2.

Table 7-2 Retention Structure Classification

Number	Structure	Classification	Relevant Criteria
1	Dam	Extreme	Population at Risk, Infrastructure and Economics
2	Dyke	High	Population at Risk, Infrastructure and Economics
3	Dyke	High	Population at Risk, Infrastructure and Economics
4	Dyke	Low	All
5	Dyke	Low	All
6	Dyke	Low	All
7	Dyke (Spillway)	Low	All
8	Dyke	Low	All

7.3.3 Design Criteria

Considering the 'Extreme' Dam Classification and as per Table 6-1B in the CDA guidance documents, the following events have been used as the design basis for the reservoir system:

- Inflow Design Flood (IDF): equal to the Probable Maximum Flood (PMF).
- Design Earthquake: Earthquake with an Annual Exceedance Probability (AEP) of 1/10,000.
- Due to the classification of 'Extreme', the Environmental Design Flood (EDF) does not apply and the PMF levels are used for analysis scenarios.

The dam and dykes are to be designed considering all applicable loadings under end of construction, operating, and seismic conditions. As per Table 6-2 and 6-3 of the CDA guidelines, the required minimum Factors of Safety (FoS) are outlined in Table 7-3 and Table 7-4.

Table 7-3 Factors of Safety for Slope Stability - Static Assessment

Loading Condition	Minimum FoS	Slope
End of construction (prior to filling)	1.3	Upstream and downstream
Long- term (steady state, normal reservoir level)	1.5	Upstream and downstream
Full or partial rapid drawdown	1.2 – 1.3	To be completed once material properties have been obtained through laboratory testing.

Table 7-4 Factors of Safety for Slope Stability - Seismic Assessment

Loading Condition	Minimum FoS
Pseudo-static	1.0
Post-earthquake	1.2 – 1.3

7.3.4 Design Considerations

The following information and limitations have also been considered in the design:

- The new reservoir and all its features are to be confined within the project boundaries.
- Construction is expected to be a staged process over several years.
- The new reservoir will have a relatively small catchment area (see **Appendix G** for the Hydrology Technical Memo), the majority of inflow will be from the pipeline conveyance from Lake Qikiqtalik and Apex River, and thus controlled.
- New reservoir filling will be conducted during the ice-free months, any inputs outside this period will be solely from precipitation events and the spring freshet.
- The reservoir will be steadily discharged over the course of the winter to Lake Geraldine to prevent it falling below a target elevation. Thus, the reservoir will be at a low water level for the spring freshet.
- The new reservoir will likely not reach full operating level in the first year of operation – filling is anticipated to be a staged process.
- Spillway locations and designs are to direct flow away from the Lake Geraldine catchment area.
- Seepage through the rock-fill dam and dykes will be controlled via the application of an impermeable liner on the upstream slopes, anchored to the bedrock (see **Appendix H** for the Geosynthetic Liner Technical Memo).
- The crest of the dam and dykes will be used for vehicle travel. Crest width has been specified at 10 m. Vehicle traffic will occur only on areas without geosynthetic liner (i.e., no vehicle traffic over key trenches or areas underlain by liner).
- The dam and dykes will be founded on bedrock, with founding surface preparation as necessary.
- Suitable on-site and/or locally available materials can be used for the dam, dyke and access road construction. Blasted rock from the excavation will need to be further crushed and/or graded to meet material specifications for the internal cushion layers if necessary.
- Bedrock surfaces under the geosynthetic liner contact zone will be appropriately treated with slush grout or similar surface dressing. Surface irregularities, cavities, or overhangs will be treated by reshaping through the use of dental concrete, as required.
- A seepage channeling system will be installed at the downstream toe of all dams and dykes, with the exception of Dyke 7 (Spillway).
- Snow capture measures have not been considered as the potential volume is not expected to be significant. A severe wind speed of 216 km/h is noted for the Region and has been used in design calculations (e.g., wave height calculations). CSA S505:20 for considerations for high winds and snow drifting has been reviewed and will continue to be referenced during the subsequent design phases.

7.4 Hydrology Analysis

Given the nature of the new reservoir operation, the available storage volume will increase over the winter months and be at the annual maximum by the time of the spring freshet. As noted in the design considerations, the reservoir is expected to be drawn down steadily from September – October to April – May of any one year. As per the current model, the sub-catchment area of the new reservoir has been calculated to be 29.87 ha (refer to **Appendix G** for the Hydrology Technical Memorandum). Given that the reservoir is expected to have discharged the majority of its volume over the course of the winter, no difficulties are anticipated accommodating the spring freshet volume.

Given that the flow into the new reservoir is almost entirely from a controlled source (pipeline from Lake Qikiqtalik and Apex River), the greater hydrologic concern stems from a flood event once the reservoir has been filled in preparation for the winter.

Based on the performed hydrologic and water management assessments for the subject area, the PMF is estimated to be 240 mm (extrapolated from existing meteorological data), and a conservative runoff coefficient of 1.0 (100%) has been applied. This results in a peak volume of roughly 71,705 m³ per event. Given a normal operating reservoir level at elevation 129.5 m, the PMF volume would increase the reservoir level to 129.825 masl, based on the stage-storage curve for the reservoir system area. Further considering wind setup and wave height, a crest elevation of 131.5 m for the system is considered adequate. For greater detail on hydrology analyses and a discussion of wave-run up factors, etc., refer to **Appendix G**.

The dimensions of the emergency spillway are a 50 m wide channel, with 3H:1V side slopes. This channel will be lined with rip rap similar to the native boulder fields currently present. The initial water level has been conservatively assumed to be at the same elevation as the emergency spillway invert, at elevation 129.5 m. The predicted peak flow, given the SCC type II for rainfall hyetograph, is 17.94 m³/s. Given the dimensions of the spillway, this results in a peak flow depth of roughly 0.325 m in the spillway channel. This confirms that the design crest elevation of 131.5 m for the retaining structures is adequate.

7.5 New Reservoir Design

Detailed Design drawings of the new reservoir are included in **Appendix A**. Draft specifications are included in **Appendix D**. One dam (Dam 1), seven dykes (one of which doubles as the spillway), and the site access roads comprise the structures. The retention structures work with the natural terrain to retain water in the reservoir footprint.

7.5.1 Dam/Dykes

Dam and dyke structures are similar, 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 Lake Geraldine. The general dam/dyke structure is a homogenous rockfill embankment with internal, filter-graded cushion layers to support the geosynthetic liner, and riprap armouring on the upstream side to protect against ice impacts. Cross-sections of each structure and of the reservoir footprint are included as part of the Detailed Design drawings in **Appendix A**.

Key design features and dimensions include:

- Rock-fill body placed in 0.5 m thick lifts and compacted using heavy equipment (not less than five passes).
- Design crest elevation: 131.5 m
- Crest width: 10 m.
- Downstream exterior slope: 3H:1V.
- Upstream exterior slope: 3H:1V.
- 1 m thick filter-graded layer between rock fill and liner cushion layer (Type 1: General Fill).
- 1 m thick cushion layer (bedding sand) on either side of the geosynthetic liner assembly (Type 2: Subcushion). To be laid in 300 mm lifts and compacted to 95% SPMDD.
- Non-woven Terrafix 600R or equivalent geotextile between bedding sand and geosynthetic liner (both sides).
- Non-woven Terrafix 600R or equivalent geotextile between bedding sand and riprap armour on upstream slope.
- Upstream slope protected by 1.0 m rip rap.

- Bedrock founding surfaces to be cleared and appropriately treated with slush grout or similar surface dressing. Surface irregularities, cavities, or overhangs will be treated by reshaping through the use of dental concrete, as required.
- Grout curtains to be installed via injection in fractured bedrock zones as required. The need for this feature will be assessed after the completion of further geotechnical investigation.
- Seepage channels and drainage control to be constructed at the rear of structures as required. A v-notch weir will be installed in the combined drainage channel at the rear of the dam.

7.5.2 Spillway

A spillway is incorporated into the reservoir rim (Dyke 7) at the location noted on Drawing B101. The spillway is designed to safely pass the PMF away from the Lake Geraldine watershed to avoid additional risk to the cascading reservoir systems. The key design features of the spillway are:

- Constructed as a stand-alone structure in what appears to be an existing surface water flow channel down to the Apex River.
- Spillway channel with a 50 m wide base, 3H:1V side slopes and longitudinal slopes built into the natural grade on either side.
- Lined with bedding sand and geosynthetic liner to achieve the design invert elevation of 129.5 m. Upper layer to consist of riprap armour as on retaining structures.
- Will be made to match existing grade downstream.
- A mountable crossing will be installed as part of the ring road construction downstream of the spillway embankment channel.

Please refer to Drawing B306 for spillway composition, features, and dimensions. The natural drainage capacity will be further assessed as part of the detailed design, after the geotechnical investigation has been completed.

7.5.3 Blasting and Excavation

The majority of the rock fill required for reservoir construction is planned to be obtained by blasting and excavation inside the reservoir footprint. A plan view of the reservoir, with cross-section lines and topographic detail of the excavation, are shown in the Detailed Design drawings in **Appendix A**.

To achieve the desired storage volume, the entire reservoir footprint will be excavated down to elevation 116.0 m. In some locations this will be achieved through overburden removal alone. In other areas, excavation will extend well into the competent bedrock underlying the site through blasting or removal using heavy machinery.

The reservoir has been designed to meet the required over-winter water storage while minimizing bedrock excavation volumes and excess aggregate generation (and thus capital costs). The estimated cut volume predicted by the design is approximately 950,000 m³ (this includes overburden to be stripped). Excavated material volumes, required amounts for construction, and excess excavated material amounts are discussed in Section 9.

Drawing B101 shows the full excavation extents with the retention structures indicated. Drawing B308 shows the latitudinal cross-section A-A', across the long axis of the reservoir. Drawing B309 shows the latitudinal cross-section B-B' at the Dam 1 midpoint, and Drawing B310 shows the longitudinal cross-section C-C' stretching the length of the reservoir from the north and ending at Dam 1.

For a preliminary assessment of blasting conditions and requirements, refer to the preliminary Blasting Assessment attached as **Appendix J**.

7.5.4 Stability Analysis

Slope stability analyses were carried out using the computer software program SLOPE/W 2022.1, Version 11.4.0.18., developed by Seequent Ltd. The Morgenstern-Price method of slices was used, which is based on limit equilibrium mechanics with a half-sine function for modelling of inter-slice side forces. The typical section representing the deepest section of Dam 1 was used for analyses.

The material properties of the overburden and dam fill zones use conservative estimates based on experience with similar materials. Design material properties will be revised using laboratory test results from site-specific materials after the geotechnical investigations are carried out. The material properties used in the stability analyses are summarized in Table 5-5.

Table 7-5 Material Properties Used for Slope Stability Analyses

Material	Bulk Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (Phi)
Type 3: Rock Fill	21	0	41
Type 1: General Fill	20	0	35
Type 2: Subcushion	19	0	32
Type 4: Riprap	21	0	41

Iqaluit, and specifically the reservoir site, is located at approximately 63.76N, -68.49E, within a relatively low hazard seismic zone. Based on the NBC probabilistic ground motion parameters and the Dam Classification of 'Extreme', peak ground acceleration of 0.053 g was chosen for the seismic stability assessment. Refer to **Appendix I** for the Stability Analysis Technical Memorandum in its entirety. A horizontal load equivalent to 0.143 g was used in the pseudo-static stability analysis. Seismic testing will be carried out on site as a part of the planned geotechnical investigation to take place next year. Seismic stability analyses will be revised with actual site-specific parameters once obtained.

Stability analyses were carried out for all applicable loading conditions, as recommended by the CDA guidance documents. The results of the analyses are tabulated below. The slope stability analyses cross-sections and graphical results are presented in full in **Appendix I**. The results show that the design configuration satisfies all slope stability criterion as recommended by the CDA guidance documents. As Dam 1 is the only "Extreme" consequence structure, the results have been presented here in the report. For the stability analyses results for all other structures, please refer to **Appendix I**.

Table 7-6 Calculated Factors of Safety for Dam 1

Analysis Condition	Minimum Required FoS	Upstream Slope FoS	Downstream Slope FoS
End of Construction	1.3	2.06	2.72
Long Term Steady State (normal reservoir level)	1.5	2.12	2.69
Earthquake Condition (pseudo-static)	>1.0	1.08	NA
PMF Loading Condition	1.3	2.164	2.69

7.6 Dam Surveillance and Monitoring

Surveillance of dam facilities includes both visual inspection and monitoring of installed instrumentation. Dams and dykes are unique structures and require unique surveillance requirements to ensure safe performance.

Catastrophic dam failures are often preceded by warning signs; properly planned and executed inspections, combined with instrumentation monitoring and suitable follow-up, can be highly effective in identifying developing dam performance concerns.

Performance-based surveillance programs require inspection personnel to understand: the hazards that are present; potential failure modes; early signs of failure to look for; and what inspection or monitoring measures could be used to detect a developing failure. The goal of any surveillance program is to identify deviations in performance conditions so that corrective action or risk mitigation measures can be implemented before adverse consequences result.

A recommended surveillance program would be prepared as part of the Operations, Maintenance and Surveillance Manual, to be developed during the detailed design process (e.g., site-specific inspection forms identify specific areas to be inspected, specific characteristics, etc.). At a minimum, the inspection process should include:

- Review of relevant files and reports of dam performance (inspection and instrumentation records) prior to on-site inspection.
- Communication with on-site staff, where applicable, to flag performance issues or observations from the last inspection.
- On-site visual inspection.
- Interpretation and preparation of report of inspection findings.
- Presentation of report findings to suitable authority (regulator or owner).
- Follow-up on inspection findings and recommendations.

The main dam structure will be relatively large and will require detailed and frequent inspections; this would consist of visual checklist inspections conducted once a month, initially. As a basis of comparison, it will be significantly larger than the Lake Geraldine dam and will require commensurately greater surveillance. As with the current Lake Geraldine structures, the main dam would be subject to annual Dam Safety Reviews, performed by a qualified third-party engineering firm. These annual DSRs would recommend the inspection frequency – either more or less frequent depending on the noted performance of the structures.

Site access is expected to be through the access and ring road(s) to be constructed as part of the reservoir construction effort. Periodic visual inspection by a qualified professional engineer is integral, but there are many possible supplementary measures (e.g., the use of drones for photographic surveys).

7.6.1 Instrumentation

Dam instrumentation is not intended to be a replacement for regular visual inspections, but an aid to ongoing assessment of dam performance. The need for instrumentation is defined systematically, based on expected dam performance and identification of parameters to be measured quantitatively. Instrument locations are provided for Dam 1 on Drawing B102 and Dyke 8 on Drawing B107. These are typical of the instrumentation to be included at each structure. Formal instrumentation plans will be included in a technical memorandum to follow the results of further geotechnical investigation in 2026, namely the onsite trenches to be advanced at dam/dyke locations. Details on the location and configuration of instrumentation on Dykes 2 to 7 will be presented in the formal instrumentation plan.

Factors potentially affecting reservoir structures (i.e., dams, dykes and spillways) include the presence and volumes of seepage and groundwater flows, embankment deformation, movement or settlement, and the presence and extent of permafrost underneath and inside structures. An instrumentation network for the new reservoir structures would be expected to include seepage weirs, horizontal and vertical control points, piezometers, wire-line inclinometers and thermistors. The detailed designs include number, spacing, and construction specifications for each set of instruments. Construction considerations are important; introducing an instrument into a dam may alter the local area surrounding the instrument, creating non-conformity. The potential

impacts of drilling through an embankment dam or altering the compaction methods during construction in the immediate area were considered.

The design for an instrumentation network at the new reservoir system includes:

- Vertical and horizontal control points to monitor embankment creep or movement.
- Piezometers to monitor groundwater and/or seepage levels.
- V-notch weirs were appropriate to measure seepage and runoff.
- Inclinerometers to monitor slump, creep or movement.
- Thermistors to monitor the thermal regime and quantify freezeback into structures or talik formation under the new reservoir.

The type and approximate number of instruments on each retention structure are outlined in Table 7-7.

Table 7-7 Instrumentation of Reservoir Retention Structures

Retention Structure	Number of Instruments						
	Survey Control	Thermistor	Piezometer	Inclinometer	Remote Camera	Staff Guage	V-notch Weir (seepage)
Dam 1	20	19	19	8	4	1	2
Dyke 2	3	1	1	1	0	0	1
Dyke 3	3	2	2	2	0	0	1
Dyke 4	3	1	1	1	0	0	1
Dyke 5	2	1	1	1	0	0	1
Dyke 6	5	3	2	2	0	0	1
Dyke 7	2	2	2	2	1	1	0
Dyke 8	6	3	3	3	2	1	1

Installed instrumentation must be maintained and monitored on a regular schedule, and the data must be reduced, plotted, and interpreted by qualified staff on a regular basis. A range of values indicating normal behaviour should be established for all instrumentation.

Instruments in Dam 1 and Dyke 8 are intended to be linked to an alarm system in SCADA. Depending on inundation modelling other structures may also require real-time alarm systems. The final Systems Operation Manual, Emergency Preparedness Plan and Emergency Response Plans will be prepared as part of the construction process. These plans will speak to operator alerts and required response measures.

7.6.2 Seepage Considerations

Initial design seepage tolerances for the reservoir as a whole are to be less than 10% of the total reservoir volume per annum, pro-rated over measured intervals when water is flowing.

Seepage effects on founding surfaces will be calculated for each retention structure using worst-case scenario estimates. Acceptable limits for each structure will be based on detailed design models constructed using the data obtained from the geotechnical investigations. Detailed seepage analysis will be completed after the further recommended geotechnical investigations in 2026.

Given the expected rock mass characteristics (massive, no faulting, relatively low occurrence of non-continuous, isotropic fractures) the current design does not include grout curtains around the circumference of the reservoir. Grout certain injection in a permafrost environment can be difficult and highly variable. The need for grout curtains will be assessed based on characteristics observed and data obtained from further geotechnical investigations.

7.6.3 Permafrost Considerations

Permafrost will need to be monitored long-term. Initial permeability in the bedrock is expected to be zero, as permafrost conditions currently exist below an estimated 2 mbgs (i.e., even if there are fractures bearing water, they are currently frozen year-round). As the talik is gradually formed under the reservoir footprint, any existing fractures may thaw and begin to conduct flow – hence the requirement for continual monitoring not just in the dam/dykes, but in the bedrock underneath as well. Based on the rock type (typically very competent with seemingly few to no continuous fractures at depth), this is not anticipated to be a concern. Further recommended geotechnical investigation, namely packer testing, will help quantify this risk.

Please refer to the Climate Lens Assessment (**Appendix M**) for more details on permafrost conditions and expected impacts to design.

7.7 Connection to Lake Geraldine – Service Corridor

Drawing C202 in **Appendix A** provides an overview of the connection of the new reservoir and Lake Geraldine. Two 750 mm HDPE pipes will be used to connect the reservoirs and to provide redundancy. To facilitate personnel access the two 750 mm pipes will be housed within a 3000 mm steel pipe with a thickness of 19 mm. Part of the bottom of the steel pipe will be filled with grout or grating (to facilitate installation of drainage and other utilities) to provide a level surface to allow for personnel access. This entire assembly will be positioned within a trench and encased with concrete or covered with large rock from the reservoir excavation to reinforce the steel pipe, ensuring it can effectively withstand both live and dead loads. A detailed section illustrating this arrangement can be found in Drawing C202, **Appendix A**. The connection between reservoirs is referred to as a “service corridor”. The service corridor will be maintained at a minimum temperature to prevent freezing.

Ventilation will be required prior to entering the service corridor for inspection or maintenance. At seven air changes per hour (ACH) a ventilation volume of approximately 3400 m³/h (2044 cfm) at an air velocity of 3.81 m/s (750 fpm) has been determined for ventilation. A 600 mm (24 inch) diameter supply air pipe or equivalent will be required. Hazardous gas detectors systems will be installed and connected to SCADA. Ventilation will be required to be continuous when there are personnel in the corridor.

Power will be brought to the site, most likely from the nearby subdivision (per comments from QEC during early consultation). An emergency standby generator will be provided at this site similar to the pump station to provide power in the event of a failure of utility power.

The access building that is incorporated into the service corridor will contain an electrical panel from which the feed for the heat tracing will be fed. Heating will be sized to maintain above 5C ambient temperature. Motorized valves will be heat traced to ensure no freezing occurs. The motorized valve will need to be heat traced at the end of the conveyance system going into the reservoir. The source of power for this valve will be from the access building.

Manual isolation valves inside the service corridor will serve as backup and double isolation for maintenance purposes.

Level monitoring will be provided for both Lake Geraldine and the new reservoir. Actuated valves in the service corridor access building will be used to control filling of Lake Geraldine based on the water level readings, with the

goal to maintain the reservoir within a working range to be finalized with the City, but no greater than 111.3 m. Level monitoring and valve control will be similar to that of the pump station and will be accessible through the City's SCADA system. Drain valves will also be included in the enclosure.

A diffuser pipe arrangement will discharge water into Lake Geraldine and will be situated below expected freezing level to protect the pipe from being damaged. An intake structure in the new reservoir will support the inlet location.

The service corridor has been designed with maintenance of the pipe and valves in mind. Specifications will include a trolley and needed pipe maintenance tools for the City to keep in storage should any repairs be required. The service corridor will be considered a confined space.

7.8 Instrumentation and Control/SCADA System

Motorized gate valves and flow meters will be installed in the service corridor and controlled by the PLC controller in the access building. The valves and flow meters are as follows:

- GV01 Gate valve
- GV02 Gate valve
- GV03 Gate valve
- GV04 Gate valve
- FT01 Flow meter
- FT02 Flow meter
- Lake Geraldine level sensor
- Reservoir level sensor

The motorized gate valves will each have the following signals to the PLC:

- Local/remote status
- Valve fault
- Valve open status
- Valve closed status
- Valve open command
- Valve close command

The flow meters will each have the following signals to the PLC:

- Flow transmitter fault
- Flow indication
- Totalized flow

The level sensors will each have the following signals to the PLC:

- Level transmitter loss of echo alarm
- Level indication

Design of the service corridor is continuing and will be updated as detailed design progresses.

8 Site Drainage

Site grading will generally follow the natural slope of the land with adjacent areas graded away from the new facility. As noted in Section 4, the drainage plan for the RWTM route optimizes the natural topography, using the ground slope to guide the discharge of water from the pipe along the terrain contours. The RWTMs will be equipped with strategically positioned drain valves, as detailed in Section 4.6.3. The discharged water from these valves will be directed through the pipe and onto rip rap. This controlled discharge onto rip rap serves the purpose of energy dissipation and erosion prevention. The channeled water, following the existing topography slope will be directed towards the nearest watercourse or culvert.

The berm will intersect with existing culverts under current access roads. These culverts will undergo necessary extensions or realignments to maintain proper stormwater drainage. Additionally, new culverts will be introduced along the alignment as needed to uphold effective stormwater drainage and prevent ponding.

With respect to the reservoir area stormwater management for the reservoir involves several key strategies to ensure the stability of the embankment, prevent erosion, and maintain the water quality in the reservoir. Key components of drainage are outlined in the following sections. Drainage design will continue as detailed design progresses.

8.1.1.1 Surface Drainage

Surface drainage will include the following components.

- **Crest drains:** It is recommended to install surface drains along the crest (top) of the embankment to collect and channel rainwater away from the embankment. These can be shallow channels lined with drainage stone or riprap to prevent erosion.
- **Crest Berms:** Berms along the crest can help direct water toward designated drains, preventing water from flowing down the slopes and causing erosion.

Slope drainage will include the following components

- **Surface Channels:** It is recommended to incorporate drainage channels along the embankment slopes to intercept surface runoff. These channels should be lined with erosion-resistant material and designed to convey water to safe discharge points.
- **Chutes or Downdrains:** Steep chutes or downdrains, often lined with concrete or riprap, can be used to safely convey water down the embankment slopes to the base, preventing erosion.
- Toe drainage will include the following components.
- **Toe Drains:** It is recommended to use surface swales or trenches filled with gravel or perforated pipes placed at the base (toe) of the embankment to collect and drain surface water. They prevent water from pooling at the toe, which can lead to instability.
- **Swales:** Shallow ditches at the toe of the embankment can help collect and channel runoff away from the embankment to designated discharge areas.

8.1.1.2 Subsurface Drainage

Internal drains will include the following components:

- **Drainage Blanket:** A layer of permeable material, such as sand or gravel, will be placed within the embankment to intercept and drain seepage water. This is typically placed near the downstream face or within the embankment core. The primary construction media (rock fill) has been specified to have relatively high permeability.

- **Filter Zones:** Filter materials (sand, gravel) will be placed between the drainage blanket and embankment fill to prevent fine particles from clogging the drainage system as necessary.

Toe drains and collector drains will include the following components:

- **Toe Drain System:** Either a network of surficial drainage swales (in instances of bedrock foundation) or a network of perforated pipes or gravel drains installed along the downstream toe of the embankment to collect seepage water from within the embankment and foundation will be used. This system helps to lower the phreatic surface (the water table) within the embankment.
- **Collector Drains:** These drains are connected to the toe drain system and carry the collected water to a safe discharge location..
- The following are components that will handle outfall of drainage systems:
- **Energy Dissipation:** Discharge points will be equipped with energy dissipation structures, such as stilling basins or riprap aprons, to reduce the velocity of water exiting the drainage system and prevent downstream erosion.
- **Outlet Channels:** Outlet channels will carry water from the embankment drainage system to a receiving body of water (e.g., river, creek, existing drainage channel). These channels will be designed to handle the peak flow rates expected during storm events.

Drainage and stormwater management features and outfall locations are noted on the drawings in **Appendix A**.

9 Construction Considerations

9.1 Site Access

Initial construction works will focus on the staging and laydown area adjacent the Road to Nowhere and the site access road. A small quarry for aggregate to construct the initial access road may be required. This small quarry would only be used to level and construct the initial site access; once blasting and excavation begin in the reservoir footprint all rock fill and produced materials will be sourced from there.

A ring road to allow access around the rim of the reservoir area is planned across the tops of the retaining structures and the spillway will have a mountable crossing inset downstream of the structure to allow vehicle transit. Various construction roads, staging areas and embankments will be constructed to allow equipment access to areas of the excavation floor, structure footprints, and surrounding areas. Please see Drawings T101 through T401 for a detailed layout of planned roadways.

Access roads and laydown areas will be constructed of excavation-run rock fill, with a surface course to protect vehicle tires as necessary. Material gradations will be the same as those for the reservoir structures. Access road structure will consist of a variable thickness of 6" excavation run material – expected to be an average 1.0 m thick, but as much as 2.0 m or greater, depending on surface grades after clearing, grubbing and excavation – topped with 15 cm of 2" minus processed excavation rock/borrow pit material. Road edges and curves to be delineated using larger riprap/boulders produced by blasting, bollards protecting instrument locations, etc. Refer to Section 9.6 for material gradation intentions.

Rock removal will be required to construct the intake and wet well. The wet well will be designed as a watertight structure able to withstand the buoyancy forces. Details for construction shoring and dewatering will be determined by the Contractor prior to commencement of construction activities on site.

The site location for the pump station has adequate space to carry out construction work including lay down. Materials used and produced during construction will be stored on-site. Overall project construction coordination will be required to determine where excess materials will be located following construction.

Some structures were observed near the Road to Nowhere and Apex River bridge, likely cabins. Two structures are located on the path of the access road to the new reservoir and will likely need to be relocated prior to the start of construction.

9.2 Dewatering

As a part of access road construction, two small ponds (Ponds D and E) will be dewatered and discharged to the drainage channels leading to the Apex River. Drainage will use existing flow channels to minimize disturbance to the existing drainage configuration of the terrain. These small ponds currently occupy the most level route to the reservoir area. Setbacks from the Apex River to the north and variable topography further south make this the most viable access route.

Three existing ponds (Ponds A, B and C) inside the reservoir footprint will similarly be dewatered in preparation for excavation and construction access to the area. Discharge of Ponds A and B is anticipated to be through the rock saddle in which spillway (Dyke 7) will be constructed, then along existing drainage channels to the Apex River. Pond C will be discharged to drainage channels leading to the Apex River similar to Ponds D and E.

Anticipated pond water volumes based on LiDAR survey data are presented in Table 9-1.

Table 9-1 Estimated Pond Water Volumes for Dewatering of Reservoir and Access Road Footprints

Pond ID	Location	Water Elevation (masl)	Estimated Water Volume (m ³)
Pond A	Reservoir Footprint - North	120.8	45,000
Pond B	Reservoir Footprint – South	119.1	32,000
Pond C	Reservoir Footprint – East	121.5	4,000
Pond D	Access Road to Reservoir - West	119.0	4,500
Pond E	Access Road to Reservoir - East	118.8	6,000

All pond water will receive the necessary treatment before discharge to the ground to meet water quality objectives. At a minimum, total suspended solids (TSS) in discharge water will be treated using dewatering bags (geotubes) or similar technology. Discharge of pond water will occur at least 30 m from the high-water mark of the Apex River. Erosion and sediment suppression measures will be implemented downstream of the dewatering to prevent sediment from entering the Apex River. Erosion prevention measures may include berms, rock check dams, and/or silt fences.

9.3 Intake and Outlet Pipes

For the intake pipe at the RWPSs a coffer dam construction is proposed in the water to allow for installation of the screens and connected pipe (at flanged fitting) at LQ and to build the intake structure at AR. The coffer dam is planned to be constructed out of “supersacs” (aka meter bags) that will be filled with sand from the onsite borrow and placed in a horseshoe to develop the coffer dam. These supersacs will be stacked two high. For the shore area they will be installed via an excavator. Bags required in the deeper section of LQ will be placed from a barge and moved into place via the excavator on the barge. The screen is expected to be installed via barge as well, with an underwater connection to the flange on the intake pipe.

For the inlet and outlet pipes (at the new reservoir and LG) a coffer dam construction is proposed similar to the pump station intake. The coffer dam is planned to be constructed out of “supersacs” (aka meter bags) that will be filled with sand from the onsite borrow and placed in a horseshoe to develop the coffer dam. These supersacs will

be stacked two high. For the shore area they will be installed via an excavator. Bags required in the deeper sections of the water will be placed from a barge and moved into place via the excavator on the barge. The area inside the coffer dam will be dewatered with pumps as necessary to complete the installation.

9.4 Excavation and Blasting

As stated above, reservoir excavation limits are below the interpolated surface of competent bedrock in some areas, and blasting will be needed to reach the required depths. Surface blasting and excavation will also be required at areas along planned roadways and pipeline conveyance berm construction.

Blasting and excavation plans will be updated upon the completion of the final Blast Assessment Report, by a qualified contractor retained by the City for this purpose. Blasting will be conducted in a manner that limits the impact on adjacent infrastructure and on the surrounding rock mass (the integrity of the rock mass around the reservoir footprint is assumed to be such that grouting is not required). The blast program (i.e. number of lifts and spacing of blast holes) will be designed in such a manner so as to minimize the noise and vibration on the community and City infrastructure. In addition, blast patterns and methods will be selected in order to minimize the impact on placed embankments while the remainder of the blasting operations are conducted. A Blast Assessment Report has been prepared by Explotech Engineering Limited and has been included in **Appendix J**.

9.5 Quarry and Borrow Materials

As described above, excavated rock may need to be sourced initially from the access road path area adjacent the Road to Nowhere, and then from the reservoir excavation. Excavated rock will be transported to crushing stations if necessary, with stockpiles of the various produced aggregates stored in laydown areas constructed for that purpose. The initial crushing will occur in the laydown area adjacent the Road to Nowhere, with crushing activities moving closer to the reservoir excavation once suitable laydown areas can be constructed.

Bedding materials (i.e., cushion layers for the geosynthetic liner) will be obtained from the previously used borrow areas along the Road to Nowhere/access to Lake Qikiqtalik. Please see Arcadis' report titled *Granular Material Requirements and Local Sources Report*, dated May 2025, for visual observations, deposit extents, initial volume estimates, and test results including material gradation curves and Proctor. Quantity estimates will be further refined after the planned geotechnical investigation confirms depths to bedrock at potential borrow locations.

While a large volume of material is required for the construction of the reservoir structures, a large volume of excess material is expected. The estimated rock excavation from the reservoir footprint is on the order of 800,000 m³, not including the volume of overburden and existing ponds onsite. The total requirements for dams and peripheral structures, as shown in Table 9-2 and Table 9-3 below, are estimated at 349,000 and 392,500 m³, respectively. Excavation volumes generated over the full course of the project are expected to exceed material requirements or construction. These excess stockpiles will be stored at temporary staging areas close to the reservoir area during construction, then moved to the larger laydown area adjacent the Road to Nowhere for use in other City projects.

9.6 Reservoir Structures

Formal construction specifications will be provided once design revisions have been made based on material parameters established by site-specific geotechnical investigation and sample laboratory analyses. The current reservoir structure design models are described above and presented in the design drawings in **Appendix A**. The general description and requirements for the various fill zones to be used in reservoir structures are described below.

- Type 1: General Fill – this will be a filter graded material (gravel and sand) designed to retain the subsequent bedding layers from the rock fill. May also be used as a surface course for access roads and laydown areas. Intended to be produced onsite from borrow source sand, this material may need to be screened to achieve the filter grade required, but it is intended that local material be used as is. Material typically to be placed in

300 mm vertical lifts and compacted at $\pm 2\%$ optimum moisture content using a smooth drum vibratory roller to 95% Standard Proctor Maximum Dry Density (SPMDD). In the event that a coarser material is selected (e.g., 50 mm minus grading), a process compaction method will be specified.

- Type 2: Subcushion – bedding material for the geosynthetic liner. This sandy material will be obtained from local borrow sources and screened as necessary to protect the geosynthetic liner according to the manufacturer’s recommendations. Material typically to be placed in 300 mm vertical lifts and compacted at $\pm 2\%$ optimum moisture content using a smooth drum vibratory roller to 95% SPMDD.
- Type 3: Rock Fill – this material will form the bulk of the retention structures. Intended to be sourced primarily from the bedrock excavation of the reservoir itself, this will be produced onsite as run of mine material (i.e., blast rock with minimal material preparation). Blasts will be designed to produce a roughly 150 mm minus material. The maximum material size used will be $< 0.5\text{m}$. Material to be placed in 0.5 m lifts and compacted using heavy equipment tracks.
- Geosynthetic Liner: liner selection is discussed in the Technical Memorandum included in **Appendix H**. The liner is to be a flexible membrane geosynthetic which is considered reasonably robust and appropriate for the level of settlement/deformation expected. The liner will be protected on the downstream and upstream sides by the bedding material (Subcushion and General Fill, respectively) and sandwiched between layers of non-woven Terrafix 600R or equivalent geotextile.
- Type 4: Riprap – will act as erosion control and protection from ice effects on the upstream slope and crests, and for the spillway. Material is to be placed using an excavator and tamped using the excavator bucket. The preparation of this material will be done as part of the material gradation work done concurrently with the preparation of the Type 1: General Fill material.

An estimate of the earthwork quantities for construction of the reservoir system structures is summarized in Table 9-2.

Table 9-2 Estimated Material Quantities - Retention Structures

STRUCTURE	TOTAL QUANTITIES					
	VOLUME (M ³)				AREA (M ²)	
	TYPE 3 ROCKFILL	TYPE 1 GENERAL FILL	TYPE 2: SUBCUSHIO N	TYPE 4: RIPRAP	GEOMEMBRANE	GEOTEXTILE
Dam 1	198398	32826	13626	18177	21777	43554
Dyke 2	10071	4006	1339	2260	2693	5386
Dyke 3	13557	6811	2492	3947	4579	9159
Dyke 4	4077	3089	986	1657	1854	3711
Dyke 5	1457	1325	392	720	803	1606
Dyke 6	48822	11683	4486	6029	7297	14594
Dyke 7 (Spillway)	2766	2193	667	1148	2844	5689
Dyke 8	31303	10106	3805	5559	2844	5689
Total	310457	72044	27797	39505	44696	89394

Note the volumes presented above for the rockfill will be amended once we understand the limits of the boulder and cobble fields present within the limits of the dam and dykes. Furthermore, at present we are assuming no grouting of the bedrock will be required beneath the dam or dykes. This information will be updated upon completion of the geotechnical investigation programs.

In addition to retention structures, the pipeline bedding and protection berm; construction access roads, ramps, and embankments; laydown areas; and the reservoir ring road will be required as part of the reservoir system construction. An estimate of the required material quantities for these structures is summarized in Table 9-3.

Table 9-3 Estimated Material Quantities - Peripheral Structures

Construction Item	Length (m)	Width (m)	Depth (m)	Volume
Access Road	1400	12	3	50400
Ring Road	3000	12	3	108000
Laydown Areas	100	100	2	20000
Pipeline Berm LQ	4000	4.5	2	36000
Pipeline Berm Apex	580	4.5	2	5220
Pipeline Berm Apex 2	1200	4.5	2	10800
LQ Pump Station	50	50	2	5000
Apex Pump Station	50	50	2	5000
Road Improvement	4000	8	1	32000
Culverts / Repair Stockpile	120	100	10	120,000
Total				392,420

Based on the location of the project, and the weather impacts that are expected during construction, the majority of construction for this scope of work – and certainly the liner placement and welding – is planned to take place outside of the colder winter seasons.

9.7 Terrain and Permafrost

Iqaluit and southern Baffin Island are located within the continuous permafrost zone. As such, all exposed land will be underlain by permafrost. This is irrespective of the soil or bedrock type, understanding that permafrost is a thermal state. Only under deeper water bodies (typically greater than about 2 m water depth) will unfrozen (talik) conditions exist. The water supply reservoir and conveyance pipelines are situated in areas dominated by shallow bedrock, typically Precambrian granites. Where unconsolidated soils overlie the bedrock, it is usually present as a thin veneer or blanket of till or glaciolacustrine deposits, generally being coarse grained in texture.

The active layer in undisturbed ground will be typically 1 m to 2 m deep, reaching a maximum depth in late September (even as the ground surface starts to freeze back). The depth of this seasonal thawing will depend on the nature of the material (soil versus bedrock), water content and vegetation cover. Greater thawing is expected in dry soils or bedrock with no organic cover.

The mean annual ground temperature (typically measured at 10 m to 15 m below ground surface) is likely in the order of -8°C.

9.7.1 Implications of Permafrost to Construction

There are few significant issues related to the presence of permafrost relative to the construction of the reservoir and conveyance pipeline. Environmental and climatic issues are addressed elsewhere. The following issues are presented for consideration.

9.7.1.1 Pipeline

It is recommended that the conveyance pipelines be constructed using “above-ground burial” where the pipeline is laid on a levelling pad of granular material placed on the ground surface and then covered with a combination

of finer granular material grading to 200 mm to 300 mm minus rock, with a total cover thickness of about 0.75 m to 1.0 m. Materials used in the pipeline foundation and protection berm must not be susceptible to frost, and the pipeline should not be buried so deep that freezeback occurs around the pipe.

For above-ground burial the ground surface should be prepared prior to pipeline installation. The ground surface should be smoothed using appropriate equipment to remove highpoints that may act as stress concentrators on the pipeline and to infill any significant depressions along the route. A leveling course of gravel should then be placed on which the pipeline will be laid. Backfill over the pipeline should consist of fine to coarse gravel grading to cobble materials with a maximum particle size of 200 mm to 300 mm at the outer surface of the cover. The backfill cover should be a minimum of 750 mm thick. The presence of permafrost should not impact the construction and installation of the conveyance pipeline for above-ground burial.

To allow for appropriate material dimensions for the constructed containment berms, blasted Precambrian bedrock will be transported to the material process laydown area, where if necessary material will be placed through a cone, jaw, and screener assembly spread for resizing to meet the design criteria of the containment berms. Ideally, excavated material can be used without processing beyond screening (i.e., rebar grill).

Figure 9-1 presents a photograph of typical rock processing equipment used to produce finer graded granular materials.

Figure 9-1 Photograph of Rock Processing System



(Photograph credit: <https://im-mining.com/tag/jaw-crusher/>)

9.7.1.2 Reservoir

As the storage reservoir includes both excavations to deepen the reservoir site and the construction of containment berms to increase storage capacity, there are several applicable permafrost considerations. Relative to deepening the reservoir, all excavation will be in permafrost. If unconsolidated materials are present, they will be frozen and difficult to excavate by conventional mechanical equipment. It is likely that large ripper teeth or blasting will be required to loosen and breakup the soil. Bedrock will also be frozen but should behave no differently than unfrozen Precambrian bedrock relative to blasting and excavation.

The constructed perimeter containment berms used to increase storage capacity will be constructed of granular materials with an interior impermeable liner to provide containment. While the perimeter containment berms will be constructed of unfrozen soils, the majority of the fill structure will freeze in the years following construction. That is, the permafrost table that was near the original ground surface will aggrade into the constructed dams, dykes, and berms.

It should be noted that a redundant, buried conveyance between the new reservoir and Lake Geraldine was considered but ultimately rejected as not feasible due in part to permafrost considerations. The length of blasting/excavation of bedrock in a permafrost condition means that 1) initial construction costs would be

excessive; 2) likewise maintenance measures (e.g., heat-tracing) and 3) such a buried pipeline outside the monitored dam footprint is considered highly vulnerable and potentially unreliable.

9.8 Environmental Management and Impacts

Several reports have been prepared to meet permitting requirements and environmental protection during construction. These include the Environmental Management Plan (EMP), the Environmental Protection Plan (EPP), and Erosion and Sediment Control Plan (ESCP). Draft reports are included in **Appendix M**. These reports will be updated once the details of the final project design are known.

A climate lens report has also been prepared and included in **Appendix M**. Section 3 of the Climate Lens report outlines the requirements for proponents to identify the climate-related hazards given the project’s location. Based on the initial assessment by Arcadis’s Subject Matter Experts and Climate Risk assessment team, climate hazards identified at the LTWP site are;

- Extreme Heat
- Extreme Cold
- Extreme Precipitation
- Short Duration High-Intensity Precipitation
- Drought
- Frost Season
- Frost Free Season
- Permafrost Depth
- Freeze Thaw Cycles
- Wind
- Tundra Fires

These environmental hazards were reviewed by the Arcadis SMEs to determine the potential risk mitigation required in the project designs. The results are summarized in Table 1-5 of the Climate Lens report. The table is provided herein as Table 9-4 for reference of design mitigation measures that will be included for infrastructure within this project.

Table 9-4 Elements, Climate Parameters and Mitigation

Element	Climate Parameter	Mitigation
Raw Water Pumping Station (RWPS)	Wind	The building will be designed to have appropriate structural integrity according to the National Building Code of Canada. Although scientific research regarding changes to wind frequency and intensity are inconclusive, building designers should consider the structural sensitivity to increased wind loads and if warranted consider adjusting design parameters such as a factor of safety or other wind load design calculations.

Element	Climate Parameter	Mitigation
Electrical Distribution to RWPS + Backup Generator	Wind	Electrical Provider to design with electrical code standards with consideration for wind combined with freezing rain. Although scientific research regarding changes to wind frequency and intensity are inconclusive, electrical distribution designers should consider the structural sensitivity to increased wind loads and freezing rain, and if warranted consider adjusting design parameters such as factor of safety or other wind load design calculations.
	Tundra Fires	Power poles vulnerable to tundra fire may be designed and built with corrugated steel bases filled with sand to act as firebreaks. Other alternatives and best practices to mitigate the risk from tundra fires may be considered.
Two Parallel Pipelines with Overburden	Extreme Heat	Anchor blocks will be included to hold the pipe in place to counter the effects of expansion and contraction due to temperature changes. The design also incorporates the placement of a berm of soil over the pipes which moderates temperature and mitigates ultraviolet degradation to the pipe.
	Extreme Precipitation	The berm of soil over the pipes will be armoured with blast rock to prevent erosion from extreme precipitation. Culverts will be placed under the pipe to mitigate damming of water and flooding, which could otherwise potentially cause pipe flotation if the pipes are empty at the time of a flood. Regular inspections will serve to inform periodic maintenance and mitigate risks from the long-term effects of erosion on the berm.
	Short Duration High-Intensity Precipitation	See mitigation from Extreme Precipitation.
	Permafrost Depth	The organic mat under the pipes will be stripped away and the pipes will be found on crushed rock. Thermal characteristics between the pipe interactions with permafrost will be accounted for in the design process for long-term stability based on the assumption of depths to the permafrost layer increasing over time.
	Wind	The berm over pipes will be surfaced with shot rock to mitigate wind erosion.
Reinforcement of the Road parallel to the pipeline	Extreme Precipitation	The road will be designed to manage precipitation loads. Road design criteria involving climate variables such as for rain intensity, duration, and frequency (IDF) will consider projected changes to the end of the century (ie. climate change-adjusted IDF curves for drainage design).

Element	Climate Parameter	Mitigation
	Short Duration High-Intensity Precipitation	See mitigation from Extreme Precipitation.
	Permafrost Depth	Road design best practices for permafrost will be applied. Additional aggregate will be added in permafrost-sensitive areas to insulate the permafrost and mitigate the rate of permafrost melting.
Pipeline Water Crossing at Niaqunnguk (Apex) River (vertical columns supporting a crib)	Extreme Heat	The pipeline will be designed by heat parameters projected to the end of the century.
	Wind	The water crossing will be designed for wind loads. Although scientific research regarding changes to wind frequency and intensity are inconclusive, structural designers should consider the structural sensitivity to increased wind loads and if warranted consider adjusting design parameters such as a factor of safety or other wind load design calculations.
Retrofit of Bridge at Niaqunnguk (Apex) River	Extreme Heat	The bridge will be designed by heat parameters projected to the end of the century.
	Short Duration High-Intensity Precipitation	Bridge design will account for high water. High water calculations shall consider precipitation parameters projected to the end of the century.
New Reservoir - eight retention structures (dams, dykes, berm)	Extreme Cold	The dams will be designed to withstand ice sheets and waterside exposure to freezing temperatures as water is drained from the lake for consumption.
	Short Duration High-Intensity Precipitation	The dam will be designed for a Probably Maximum Flood scenario, which accounts for projections beyond the end of the century (greater than a 1 in 1000-year storm), due the high impact the dam's failure would have given its proximity to a city. Potential Acid Generating (PAG) rock will not be used in the construction of the dams. Testing to date shows none of the bedrock as PAG rock.
	Permafrost Depth	All overburden will be stripped from beneath the footprint of the dams and dykes, allowing the dams to be founded on bedrock. Thermal monitoring will be conducted within the dams to assess the variability of the permafrost layer.
	Wind	The dams will be designed with industry best practices to withstand ice sheet movements.

Element	Climate Parameter	Mitigation
Spillway	Short Duration High-Intensity Precipitation	The spillway will be designed to account for precipitation conditions as required by the Canadian Dam Guidance documentation. Calculations shall consider precipitation parameters projected to the end of the century.
	Wind	The spillway will be designed to account for wind conditions as required by the Canadian Dam Guidance documentation. Although scientific research regarding changes to wind frequency and intensity are inconclusive, spillway designers should consider the sensitivity to increased wind loads and if warranted consider adjusting design parameters such as a factor of safety or other wind load design calculations.
New Access Road to New Reservoir, and Ring Road partially atop the New Reservoir	Extreme Precipitation	The road and associated dam spillway will be designed to manage precipitation loads. Design criteria involving climate variables such as for rain intensity, duration, and frequency (IDF) will consider projected changes to the end of the century (ie. climate change-adjusted IDF curves for drainage design).
	Short Duration High-Intensity Precipitation	See mitigation from Extreme Precipitation.
	Permafrost Depth	Road design best practices for permafrost will be applied. Additional aggregate will be added in permafrost-sensitive areas to insulate the permafrost and mitigate the rate of permafrost melting.
Valve Access and Control Building at New Reservoir	Permafrost Depth	The building will be designed to withstand thermal impacts due to permafrost.
	Wind	The building will be designed to have appropriate structural integrity according to the National Building Code of Canada. Although scientific research regarding changes to wind frequency and intensity are inconclusive, building designers should consider the structural sensitivity to increased wind loads and if warranted consider adjusting design parameters such as a factor of safety or other wind load design calculations.
Conveyance Pipeline Between New Reservoir and Lake Geraldine (requires heat tracing)	Permafrost Depth	The pipeline will be designed to mitigate thermal impacts from permafrost.
Electrical Distribution + Backup Generator at Control Building and Buried Conveyance Pipeline	Wind	Electrical Provider to design in accordance with electrical code standards with consideration for wind combined with freezing rain. Although scientific research regarding changes to wind frequency and intensity are inconclusive, electrical distribution designers should consider the structural sensitivity to increased wind loads and freezing rain, and if warranted consider adjusting design parameters such as factor of safety or other wind load design calculations.

Element	Climate Parameter	Mitigation
	Tundra Fires	Power poles vulnerable to tundra fire may be designed and built with corrugated steel bases filled with sand to act as firebreaks. Other alternatives and best practices to mitigate the risk from tundra fires may be considered.

10 System Operation

A preliminary operation manual has been started for the LTWP – Supply and Storage system. The final manual will outline required ongoing maintenance suggestions and operating procedures. This does not take place of the Operations and Maintenance Manual (O&M) that the construction contractor will be required to provide. That manual will detail specific equipment operation and maintenance including shop drawings and manual from equipment manufacturers.

Considerations for cleaning of the RWTM, and thawing procedures (should this ever be necessary) will be included in the detailed design documents and the system operation manual procedures.

The preliminary system operation manual is included in **Appendix K**. An updated manual is underway during detailed design.

Design life for the main components of the LTWP – S&S system are outlined in Table 10-1.

Table 10-1 Design Life for Project Components and Recommended Equipment Spares

System Component	Equipment	Expected Design Life	Critical Component/Spares Recommended
RWPS	Structure	50+ years	Roof/waterproofing 15-20 replacement
RWPS	Raw water pumps	15 years	Standby pump provided for redundancy, also keep a “shelf spare” in storage.
RWPS	HVAC equipment	15 years	
RWPS	Compressed air system	15 years	
RWPS	Valves	25 years	Keep one type of each valve in storage
RWPS	Electrical – MCC, transformer	20 years	
RWPS /Reservoir	Electrical – Standby generator	25 years	City to have a portable generator that can be connected to the manual transfer switch
RWPS – Intakes	Screens	25 years	Two screens provided.

System Component	Equipment	Expected Design Life	Critical Component/Spares Recommended
RWPS – Intakes	Heat tracing	20 years	
RWPS	Instrumentation and Control – Instruments (pressure, temperature, etc.).	15-20 years	Replace instrumentation on failure. Watch for drift in instrumentation values through SCADA trend screens to identify potential issues and requirement for replacement. For network equipment one spare for each active network device i.e. network switch. Network equipment make/model should be consistent with other facilities such that the spare can be used for multiple facilities outside of this project.
RWPS	Instrumentation and Control – UPS	3-5 years	UPS batteries will require replacement. City to have a spare UPS that can be used as a replacement. UPS make/model and size should be consistent with other facilities, such that the spare can be used for multiple facilities outside of this project.
RWPS	Instrumentation and Control – Control/PLC panels	15-20 years	City to have one spare the same make/model for PLC DI, DO, AI and AO cards. One spare PLC CPU, power supply, chassis and communication cards as well. PLC hardware should be standardized across multiple facilities, such that the spares identified above can be used at multiple facilities outside of this project.
RWTM	HDPE Pipe	50+ years	Two pipes for redundancy. Keep two sections of pipes and coupling in storage.

System Component	Equipment	Expected Design Life	Critical Component/Spares Recommended
RWTM	Valves (SCAV, drain, etc.)	20 years	Keep 2-3 spare valves of each type in storage.
Reservoir	Liner	100+	Liner stockpile for patches/repairs. Bolts and brackets for key trench repairs to be kept onsite.
Reservoir	Instrumentation	15 years	Battery replacement every year. Five spares of each type to be kept onsite.
Reservoir – Service Corridor	Valves	25	Keep one of each type of valve in stores
Reservoir	Dams/Dykes	100+	Aggregate stockpiles at each dam/dyke location for repairs.

11 Cost Estimate and Construction Schedule

this section to be updated in February

The cost estimate for the LTWP – Supply and Storage, at the preliminary design stage, is \$XXXM including a 20% contingency. Our basis of estimate (BOE) document outlines the approach to estimating and more detail regarding the costing of the project components. The basis of estimate is found in **Appendix L**.

Suggested construction sequencing and schedule, including the recommendation for an early works contract package is described in the BOE.

The cost breakdown for the various components of the project is as follows:

Project Components	Labour	Materials	Equipment	Subcontractors	Hotel / Travel	Total	Contingency %	Total Amount with Contingency
LQ Pump Station			Note 1	Note 1	Note 2	\$12,490,000	20%	\$14,990,000
AR Pump Station			Note 1	Note 1	Note 2	\$14,450,000	20%	\$17,340,000
Raw Water Intake - LQ	\$1,240,000	\$140,000	\$890,000	\$210,000	Note 2	\$2,460,000	20%	\$2,950,000
Raw Water Intake - AR								TBD
Conveyance Pipeline – LQ	\$2,760,000	\$2,290,000	\$1,580,000	\$3,220,000	Note 2	\$9,830,000	20%	\$11,800,000
Conveyance Pipeline – AR						\$5,660,000	20%	\$6,792,000
New Reservoir	\$17,500,000	\$2,900,000	\$19,200,000	\$27,030,000	Note 2	\$66,620,000	20%	\$79,940,000
Service Corridor	\$1,250,000	\$210,000	\$770,000	\$2,480,000	Note 2	\$4,700,000	20%	\$5,640,000
							Total	\$139,452,000

Note 1: Equipment, subcontractors and indirect costs included in labour/indirect costs.

Note 2: Hotel/travel included in contractor indirect costs.

12 Next Steps

A design review workshop for the 50% detailed design submission will be held in Iqaluit, in January 2026. Following this and receipt of consolidated City comments, the design will proceed to 90%. Documents such as the Operations Manual, Commissioning Plan, and others will continue as design details progress to be incorporated as final deliverables and in tender documents as applicable.

Appendix A

Detailed Design Drawings

Appendix B

Survey Report

Appendix C

Geotechnical Investigation Report

Appendix D

Draft Specifications

Appendix E

Pump Calculations

Appendix F

Pump Curves

Appendix G

Hydrology Technical Memorandum

Appendix H

Liner Material Technical Memo

Appendix I

Stability Analysis Technical Memo

Appendix J

Preliminary Blast Assessment Report

Appendix K

Preliminary Systems Operation Manual

Appendix L

Basis of Estimate

Appendix M

Environmental Reports

Environmental Management Plan (EMP)

Environmental Protection Plan (EPP)

Erosion and Sediment Control Plan (ESCP)

Climate Lens Report