# ATTACHMENT 9

## LTWP Preliminary Design Report

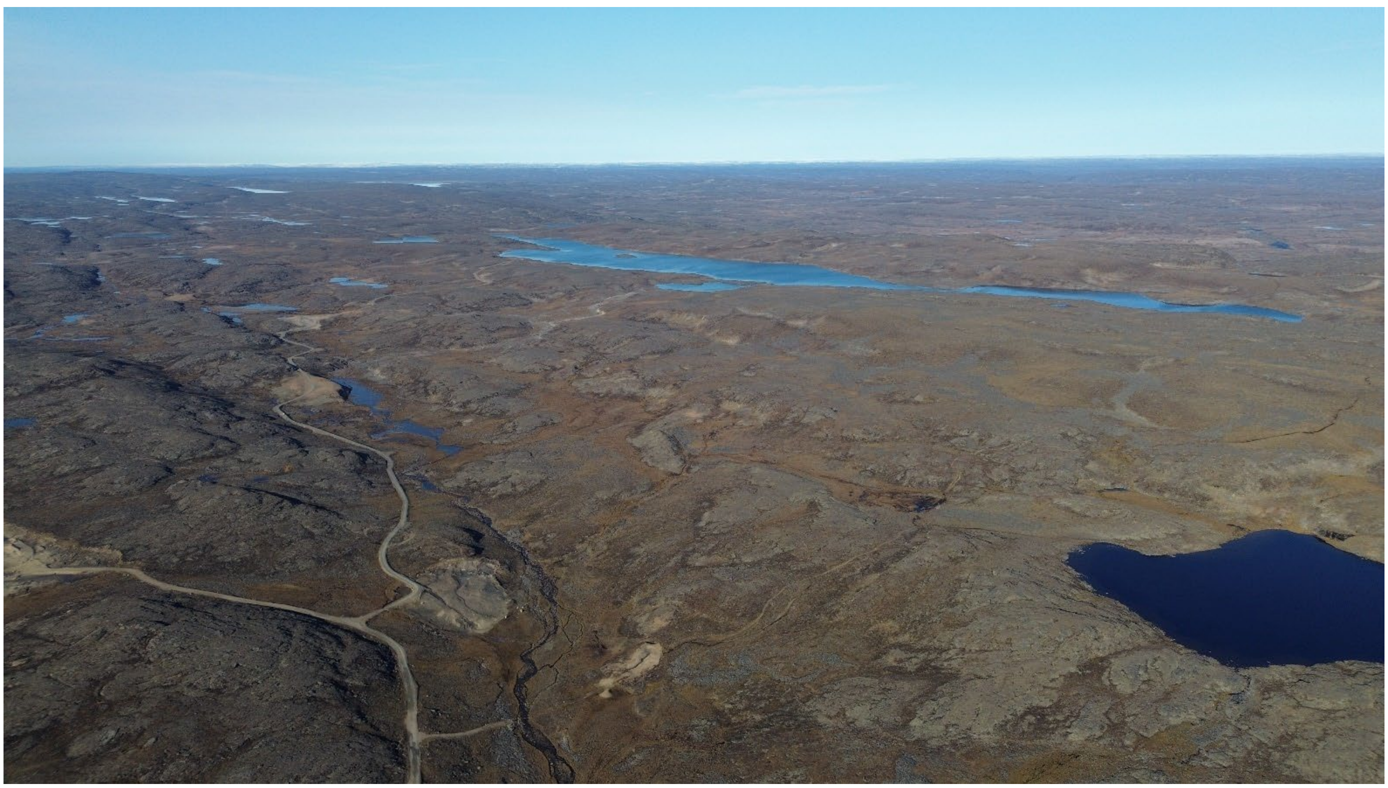
City of Iqaluit

# Preliminary Design Report

## Long Term Water Program – Supply and Storage

DRAFT REPORT

September 2025



# Preliminary Design Report

**Long Term Water Program – Supply and Storage**

September 2025

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# Version Control (optional)

| Issue | Revision No. | Date Issued | Page No. | Description | Reviewed By |
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| 1 | 0 | March 25, 2024 | N/A | Draft – for client review | Various |
| 2 | 1 | September 20, 2024 | Various per comment sheet | Final – for client record | Various |
| 3 | 2 | September 26, 2025 | Updated to include the Apex River pumping station and conveyance pipeline | Draft – for client review | Various |

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# 1 Introduction

## 1.1 Background

As the City of Iqaluit (City) is considered 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. Additional water resources and storage are needed to sustain expected future demand. Based on feasibility studies (by others) for the new reservoir, the amount of water required by the City is expected to increase by 65% which represents an additional 1,824,500 $m^3$ of raw water storage required.

Over the last few years, the City has pumped water from the Apex 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 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.

Additional project scope was added to the project tin 2025 to include pumping from AR. This preliminary design report has been updated to include the additional pump station at AR and the associated conveyance pipeline. Reservoir changes and optimization will be provided the 50% design report.

## 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 the Apex River

- Water conveyance pipeline of:
  - approximately 4 km two pipelines from the LQ pumping station to the new Reservoir
  - approximately 500 m two pipelines from Apex River 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 Apex River 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 Phase II Preliminary Design and outlines the final recommended design basis for the project.

## 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 = 1,832,760 $m^3$
- Minimum useable overwinter storage volume = 1,247,500 $m^3$
- Available useable overwinter storage of raw water volume in the new reservoir = 1,389,970 $m^3$ (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) from the Apex River.

It is noted that as the project moves into detailed design, the volume of useable overwinter storage will be required to meet the conditions of the federal funding agreement, which is 1.5 times 1,095,000 $m^3$, for a total of 1,642,000 $m^3$. This will be addressed during the next design phase. The new storage volumes will be:

- Total storage of raw water volume in the new reservoir = approximately 2,080,000 $m^3$
- Minimum useable overwinter storage volume = 1,643,250 $m^3$

Design documents reflecting this update will be included in the 50% design deliverable.

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 AR to a new reservoir has been selected with preliminary design presented in this report. The pipeline route 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 Lake Qikiqtalik 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. The potential to generate hydroelectric power through a micro generator will be reviewed by our electrical and process staff. This potential exists due to the significant hydraulic head difference between Lake Qikiqtalik and Lake Geraldine/new reservoir. This will be reviewed during detailed design.

# 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
- 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 will be given as part of the NPC Project Proposal to be submitted in 2024. A Draft Physical, Biological and Socio-Economic Impact Assessment (PBSEIA) has been submitted for review by the City of Iqaluit. 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)

A Public Information Session will be held in future to present the design to the community for input.

## 1.6 Background Documentation

Table 1-1outlines 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) |
| 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) |

| Documentation Received |
|---|
| 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) |
| 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) |

| Documentation Received |
|---|
| 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) |
| Qikiqtalik 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 2023-2024).
- Geotechnical and Hydrogeological Investigation (ongoing as of January 2024).
- Archaeological Investigation (completed by others in 2023).
- Initial 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 meets 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 is also site recognisance 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, we will revisit the site after the spring freshet of 2024 if required (late June or early July) to fly an expanded area of interest with a LiDAR system (reported accuracy of 3 cm). The purpose of this work will be confirmation of the “boots on the ground” work in 2023, to pick up any new areas of interest following winter design activities, and to capture high resolution photographic and digital elevation data overlay of the entire area.

Boreholes, survey monuments and benchmarks will be shown on detailed design drawings.

The draft survey report is included in **Appendix B**.

## 2.2 Geotechnical Investigation

Based on the project award date and permitting delays, the phased geotechnical investigation has been reorganized. The first phase of the program started in February 2024 and consisted of air rotary boreholes using Canadrill’s local drill rig via winter access. The purpose of these boreholes was to verify overburden consistency and bedrock depth at key areas within the footprint of the proposed reservoir and potential quarry locations. Piezometers were installed in select air-rotary boreholes to assess groundwater conditions. Permeability testing (slug testing) was conducted by Arcadis in late August 2024 on all piezometers. Seismic testing at select air-rotary boreholes is scheduled for November 2024 to obtain the shear and compressional wave depth vs. velocity profiles for the bedrock at site.

The second phase of the program will consist of bedrock coring and multi-bead thermistor cable installations. This phase is anticipated to begin in mid September 2025.The overall purpose of the bedrock coring program will be a preliminary evaluation of the bedrock type and quality (extent of fracturing/seams) along the liner key trench of the reservoir (around the ponds onsite). This information is considered critical to assessing seepage potential and ground improvement requirements beneath/around the key trench as part of this project. A multi-bead thermistor string will then be installed to the bottom of select boreholes and backfilled to surface with imported sand, receiving a proper housing above grade to protect and help identify the location of each thermistor string for future readings. The thermistor information will provide existing ground temperature profiles along the key trench (baseline conditions for geothermal modelling purposes, if desired during as part of design of long-term thaw considerations). Readings will be obtained from the thermistors on an occasional basis as our representatives are in Iqaluit for other work and we will be sure to obtain readings at key times, including the onset of winter (warmest ground temperature profile) and the onset of spring (coldest ground temperature profile) once this phase of work is completed.

A draft copy of the Preliminary Blast Assessment Report by Explotech Engineering Limited (Explotech) is included in **Appendix J**. Based on Explotech’s assessment of the project scope, a visit to the site is not necessarily anticipated to be necessary to complete the preliminary blasting assessment report – pending the results of the geotechnical investigations (notably, the geophysical testing results). Explotech is expected to update and finalize their preliminary report to address client comments before the end of December 2024.

A LIDAR survey was completed September 2024 to obtain additional topographical data at the site and will include the proposed reservoir extension and access roads. The proposed reservoir extension involves the relocations of Dyke 8 and resulted from a redesign of the reservoir following an optimization exercise conducted by Arcadis.

Two geotechnical reports will be prepared: 1) the investigation report which will provide factual data results from field work, and 2) the geotechnical baseline report which will include data analysis, interpretations and recommendations for design and construction. All sites investigated (test pits, boreholes, monitoring wells, etc.) will be shown using geodetic coordinates on our design drawings.

The previously proposed geotechnical workplan is included in **Appendix C**. The scope of work outlined in the workplan will be limited in 2024 to only meet the requirements established in the RFP. The additional necessary work outlined in the workplan that goes beyond the RFP scope will be recommended for completion at a later date.

## 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 Qaujimajatuqangit). A separate report will be used with report findings and conclusions.

## 2.6 Hydrological Field Investigation

Based on instruction from the City, the field investigation scope will be limited to items detailed in the RFP and the full geotechnical program proposed in Arcadis' Geotechnical Workplan (provided under separate cover) will not be conducted this year. As such, visual depth estimates of the five ponds within the footprint of the proposed reservoir and access road will be attempted from shore in during the second phase of geotechnical program when approved.

# 3 Flow Analysis

## 3.1 General

To meet the City's water needs to the year 2050 up to an additional ≈1,650,050 m$^3$ of raw water will be necessary annually beyond what can currently be stored in Lake Geraldine. This represents the over-winter (i.e. useable volume assuming a 2-m ice surface in winter).

This 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 a new reservoir to be constructed adjacent to Lake Geraldine.

Tetra Tech (TT) has estimated in their report Qikiqtalik Lake Water Balance for Withdrawals[1], that Lake Qikiqtalik is projected to have a median value of 1,681,000 m$^3$ available for withdrawal and a mode value of 1,284,184 m$^3$ available. Based on the report's probability distribution function, the 0$^{th}$ percentile volume was 767,189 m$^3$ and the 10$^{th}$ percentile was 1,114,609 m$^3$. These low percentiles represent unusually low river 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 0$^{th}$ percentile value was taken and rounded to 800,000 m$^3$. This number represents what could be withdrawn from Lake Qikiqtalik in a low-flow condition, without depleting the lake's storage. This volume is not sufficient to supply the City's needs, and therefore, an additional volume of water is required to be taken from AR of ≈800,000 m$^3$. Based on our analysis, AR should be able to provide this approximately 1,000,000 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

LQ was originally determined to be the only source of future raw water for the City's new reservoir, however, final analysis by TT indicated that there would be insufficient volume to rely solely on this source. As noted, the final determination of minimum available water for LQ was 800,000 m$^3$.

Preliminary design was advanced in 2024 assuming LQ would supply the entire source of volume up to 1,824,500 m$^3$. At this time the pump station design assumed 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 230 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 required

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

1,824,500 m$^3$ of raw water (1,094,700 m$^3$) during the month of June (30 days) with the remainder of the water (729,800 m$^3$) pumped during the open water season.

To accommodate the 1,094,700 m$^3$ during the month of June (30 days) the pump station would require a rated capacity of 423 L/s. However, hydraulic requirements of the raw water pipeline were modeled and a flow of 556 L/s was determined to be the recommended value based on pipe velocities. The design capacity of 556 L/s will be achieved using two pumps running in parallel, with a third standby pump.

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.

The water withdrawal rates and timeframe for LQ will require review following preliminary design, in conjunction with the addition of the AR pump station. Pump capacities may change as the volumes available/required from LQ to fill the new reservoir have been reduced. This will be addressed during the 50% design phase.

*Table 3-1 Lake Qikiqtalik Design Criteria*

| Description | Value | Reference |
|---|---|---|
| Pump Capacity (Design) | 556 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 m$^3$ 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 10U015i/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 River[2] 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:

[2] Tetratech, Desktop Study of Discharge in Apex River, July 24, 2025

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.

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 $m^3$, which was 1983 (also note that TT considered this year to be low quality data). 1983 yielded 983,790 $m^3$.

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 1and 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 Geraldine[3], 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.

*Figure 3-1 Pump Volumes, by Month*

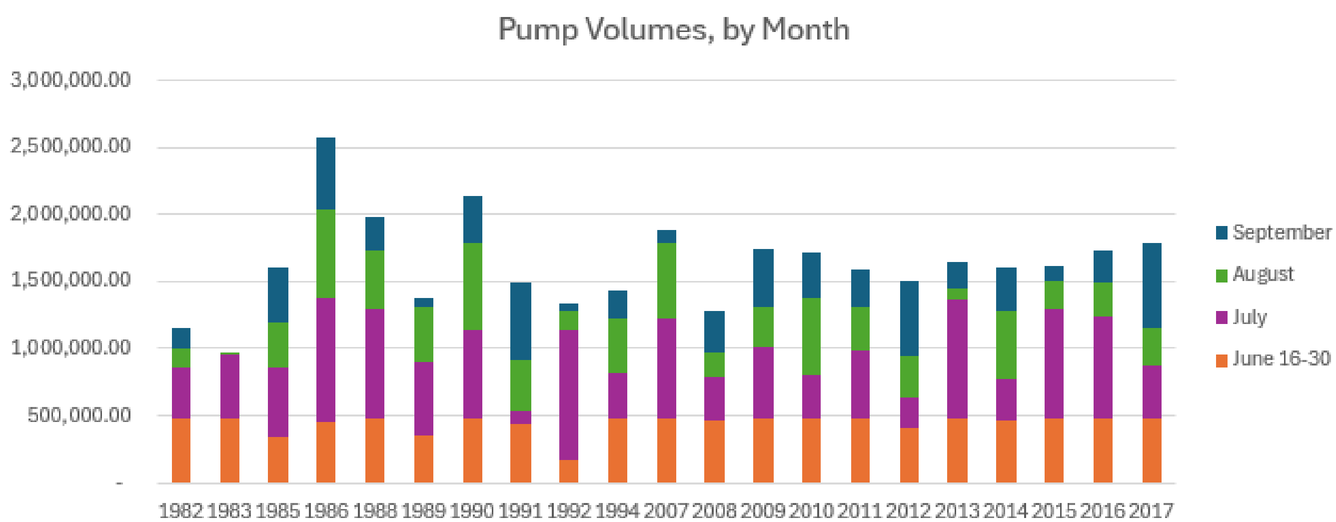



*Table 3-4 Pump Volumes by Year*

| 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 |

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

| Year | Pumped Volume, **L/s** |
|---|---|
| 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 Qikiqtalik 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 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, compressors and air receiver tanks, and a washroom.
- A gas detection system with door horn and strobe lights for High H2S, LEL and O2 alarms.

## 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 and a sump with a submersible pump to lift the wastewater to the wastewater tank at grade level inside electrical room. 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, 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 potable water tank with booster pump will service the washroom fixtures and hose bibs (washroom will be accessed from electrical room). An electric point of use water heater will be provided for the hand sink hot water connection.
- A wastewater tank including grinder/pump to service the washroom fixtures and floor drains in each space.
- 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 blow heater and exhaust fan.
- Additional electric heaters will be provided in each space, one baseboard heater will be provided inside the washroom.

Refer to pump station drawings in **Appendix A**. A draft specification list is included in **Appendix D**.

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.

# 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 750 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 lakebed 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.

At present a single intake is recommended, with bypass pumping connection provided. The expected cost should the City wish to provide a second intake would be $400,000 for pipe installation, plus the cost of modifying the pump station to allow for a new inlet gate. The cost for a second screen would be approximately $150,000. It may be possible to use the same airburst system which would need to be determined with the supplier. If this is not possible, a second airburst system would be approximately $640,000.

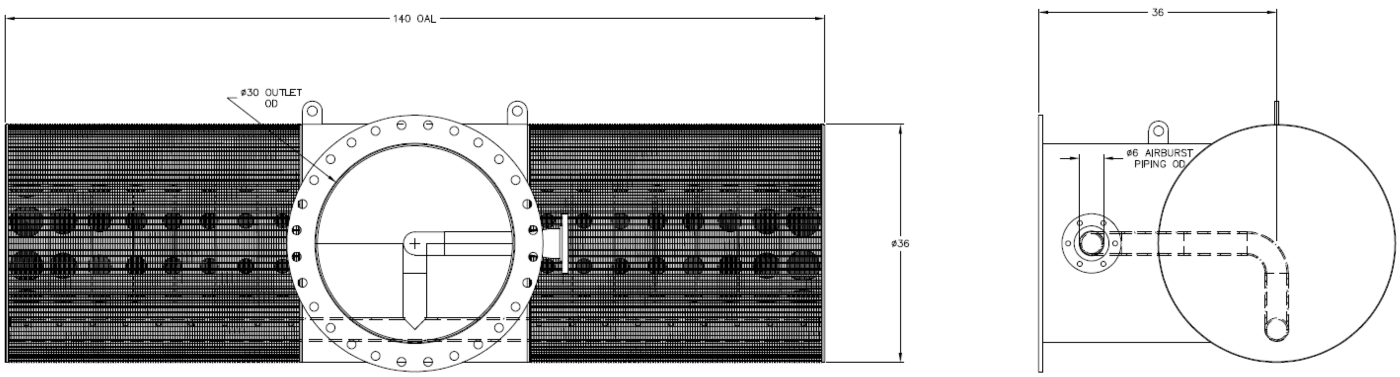
## 4.3.2 Screen

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 4.82 $m^2$ is required for a flow rate of 556 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 556 L/s with an inlet flow velocity of 0.11 m/s (0.36 fps). The screen is a "TEE" type, 914 mm (36 in.) in diameter, with an overall length of approximately 3.56 m (140 in.) long with #69 wire screen providing a slot size of 2.54 mm (0.1 in.). The screen is equipped with a 750 mm (30 in.) flanged outlet and a 150 mm (6 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) 141.0 L/s (299 cfm) rotary screw compressors each with 575/3/60, 55.9 kW (75 hp) TEFC motor, aftercooler, oil/moisture separator, graphic controller, and sound attenuating enclosure. Each compressor is sized to charge the 8330 L (2200 gallon) receiver from 0 kPa to 1035 kPa (0 psi to 150 psi) in 10 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 750 mm diameter HDPE DR 11 (inside diameter of 615.1 mm) pipe. The flow velocity in the intake with a pump flow of 556 L/s is 1.9 m/s.

#### 4.4.1.2 Discharge Pipe (Forcemain) Size and Class

The pumps are sized for the design flow of 556 L/s. The discharge pipe is sized as a 600 mm diameter HDPE DR 11 (inside diameter of 492.1 mm). The flow velocity with a pump flow of 556 L/s is 2.92 m/s.

#### 4.4.1.3 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.

#### 4.4.1.4 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**.

#### 4.4.1.5 Internal Station Piping

Each pump discharge line is 450 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 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. The header transitions from 450 mm stainless steel after Pump 3 to 600 mm SS.

#### 4.4.1.6 Surge Pressure Calculations

A preliminary transient analysis was performed (see Section 4.5 for details). The surge pressure at the pump station is approximately 310 kPa (45 psi). Since signature surge pressure is not predicted to occur no surge protection devices, except for a check valve for each pump, are provided at the pump station.

#### 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 100 mm diameter sewage combination air and vacuum valve (SCAV) for raw water is installed on the 600 mm diameter common pump discharge header to release entrapped air and provide pipeline vacuum protection. The type and size of SCAV is to be revisited 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.

### 4.4.2 Wet Well

#### 4.4.2.1 Design

The supportive design calculations for the pump station are enclosed in **Appendix E**. Section 3.5.4 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.

The valve room is considered a confined space. Based on input from the City the potential to eliminate this as a confined space will be investigated in detailed design, including the possibility of utilizing a staircase for access as opposed to an access ladder as shown in a design sketch, Figure 4-2.

## 4.4.4 Raw Water Pumps

### 4.4.4.1 Head Loss Calculations

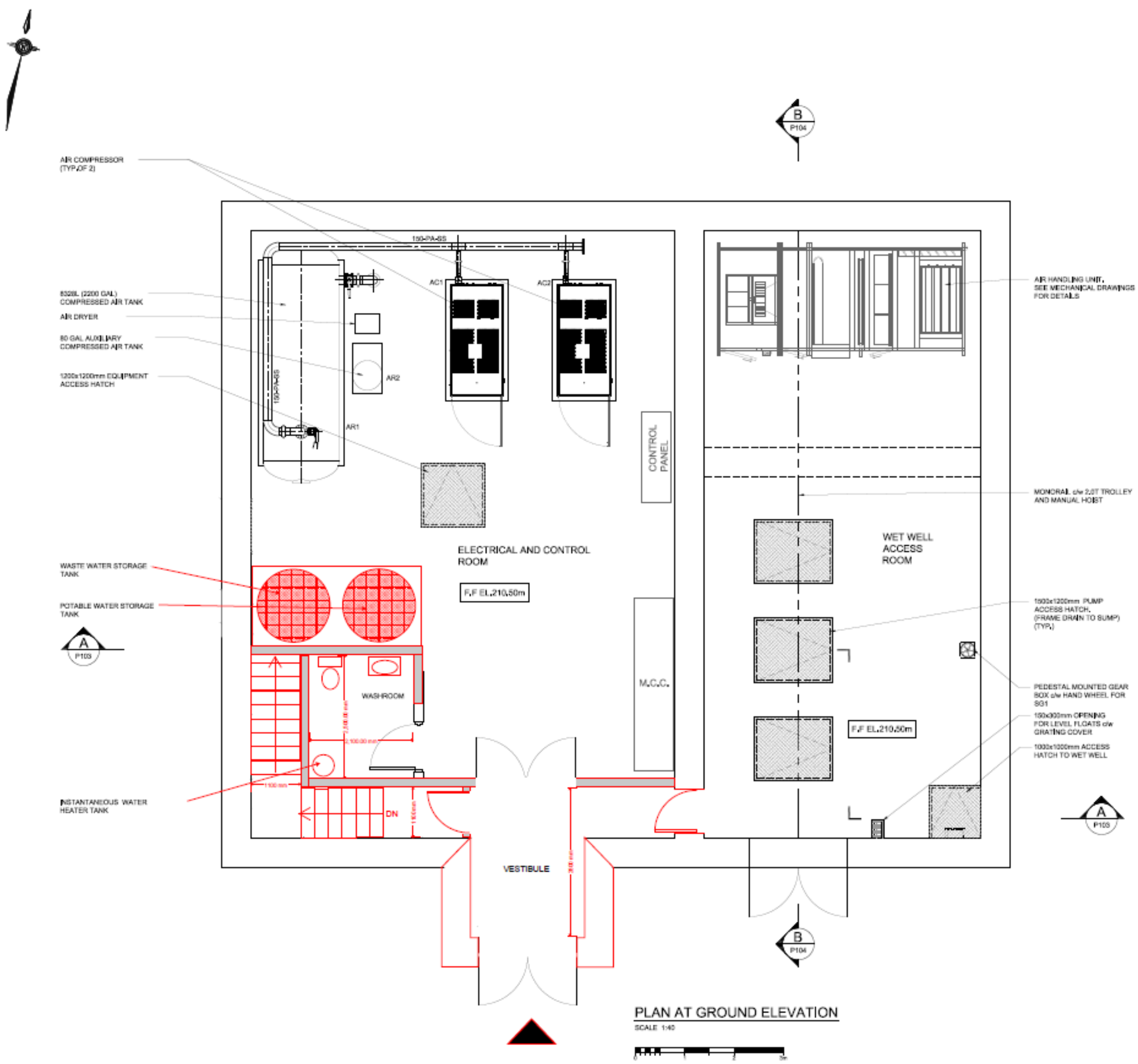
The system head curve is developed by calculating the total dynamic head (TDH) as the sum of static head and friction losses in pipes and fittings. The following is a summary of the calculation methodology for the pump station. Refer to **Appendix E** for the detailed design calculations.

#### 4.4.4.1.1 Static Head

Static head values are based on the proposed operating levels in the wet well (between 200.69 m and 202.39 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. The pump is operated at pre-set pressure of 3.33 m higher than obvert of the pipe or HGL 215.91 m at VC1).

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.

*Figure 4-2 Proposed Stairwell Modification to Pump Station to eliminate confined space*




## 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 $h_f$ × 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:

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: 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 600 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 278 L/s each.

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

The preliminary pump criteria can be seen in Table 4-1. As noted previously, the pump selection and capacity may be revisited following preliminary design given the addition of the AR station.

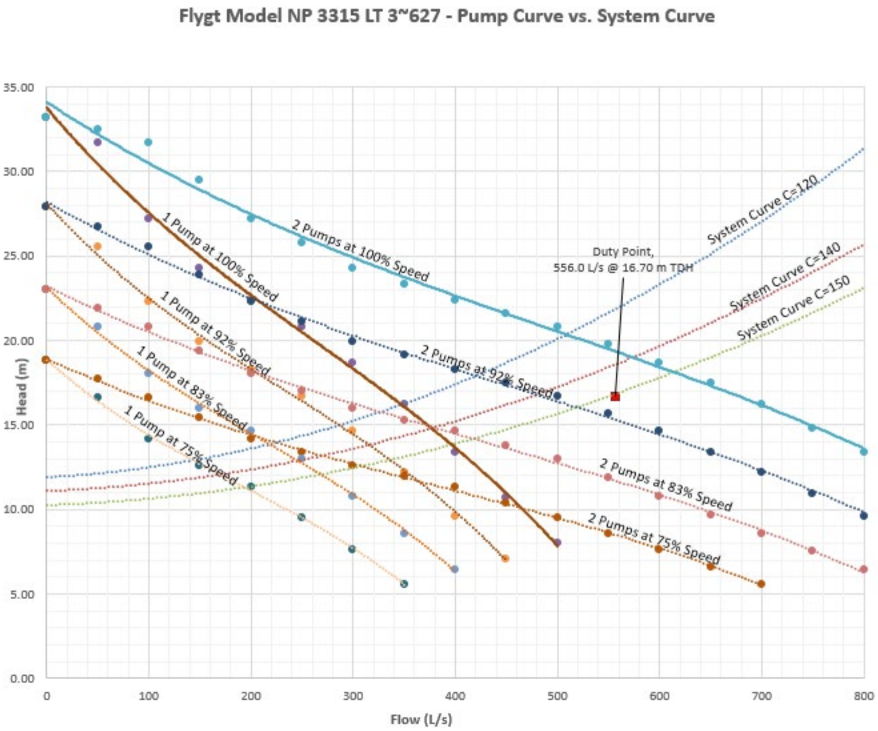
*Table 4-1 Pump Criteria*

| Parameter | Criteria |
|---|---|
| Performance - Pump Duty Point | 278 L/s @ 16.7 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 | 82 kW (110 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 600 mm discharge pipe, with the duty point of 278 L/s at 16.7 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 82 kW (110 hp) (600 V/60 Hz/3 ph).

*Figure 4-3 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 and system pressure as its primary input.

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 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 operate with rotating duty/stand-by cycles to maintain similar operating hours for each pump. 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 Qikiqtalik 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 doors 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 General

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 in the underlying soil medium of the ground 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. 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 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. A ceiling mounted exhaust fan will be provided in the washroom. An electric baseboard heater will be provided for the washroom, and a transfer air grille will bring fresh air from electrical room into the washroom space. 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 4-2.

*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 |
| Washroom | N/A | 15°C | 30°C | Not controlled |
| Note1: Mechanical ventilation will be provided to maintain space temperature at 10°C above ambient temperature.; 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 room.

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.

Additional SCADA alarms for plumbing and drainage systems will be provided as follows:

- Low water level alarm - potable water tank
- High level alarm - wastewater tank
- General alarm - water heater

### 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. The water supply for the plumbing fixtures will be provided via an indoor storage tank sized at 3600 L (960 usgal) capacity suitable for potable water. The tank will be manually filled. Since the station will not be frequently used, the water within the water tank will not be maintained as drinkable water and will be marked as such (i.e., is shown on the design drawings as “NPW" non-potable water”). The use of the water will be strictly for handwashing. Frequency of filling the water tank will need to be discussed with the City to ensure it is maintained safe for handwashing. The tank will be provided with a filling port on the exterior wall of the station and drain valve. The tank will be floor mounted on a galvanized steel frame to allow for a pumping system installation below the tank.

#### 4.8.6.2 Plumbing and Drainage

A water booster pumping system will be provided to transfer the water from the water storage tank to all plumbing fixtures and water heater. The booster pump will provide sufficient pressure for the water so that all plumbing fixtures and water heater can be operated properly.

An electric point of use water heater will be provided to provide hot water to the hand sink.

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

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, one washroom floor drain will be trapped from the toilet flushing system.

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 hand sink and a toilet will be provided in the pump station. Both hand sink and the toilet will be barrier free type. To reduce water consumption, all plumbing fixtures selected for this project will be low consumption fixtures.

All wastewater generated from the washroom will be transferred to a wastewater holding tank with a sanigrind (pump/grinder) local system located inside the washroom between toilet and hand sink. Drainage lines from fixtures will be connected to this system and the discharge line will be connected to the wastewater storage tank. The holding tank, 3600 L (960 usgal capacity), will be suitable for wastewater and will be emptied via a sanitary tanker truck as is common in Iqaluit. The tank will be located in the Electrical and Control room in the proximity of the washroom area on a concrete pad. The tank will not have a drain line provided and there will be no provisions for containment. A double wall tank can be considered. The tank will be vented to the outdoors and an quick connect outlet will be provided on the exterior wall of the station for truck access. Minimum required distances between raw sewage and potable water services will be maintained.

Details of the handwashing/wastewater system will be further discussed and confirmed with the City.

#### 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 Service Entrance

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.

### 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 56 kW (75 hp) air compressor.
- One 56 kW (75 hp) air compressor (not considered for overall loading since only one compressor runs at one time).
- Two 82 kW (110 hp) duty raw water pumps.
- One 82 kW (110 hp) standby raw water pump (not considered for overall loading as only two pumps run at one time).

Overall, this represents 220 kW (295 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. Exact requirements for electrical loading will be developed during detailed design. 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 will be done in detailed design 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.

Confirmation of feasibility for construction for power (by the local utility QEC) has been carried out. QEC confirmed that they can install power to the project for an approximate cost of $2.5-3M. A one-year notice for construction along is requested. 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 being procurement.

### 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 be designed based on the City's SCADA System User Manual Revision 1. The 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 Lake Qikiqtalik 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 during preliminary design 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.

### 4.10.2 PLC Control Panel

The SCADA system will be designed based on the City's SCADA System User Manual Revision 1. 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 Inducive Automation's Ignition software and will utilize the Allen Bradley Ethernet/IP driver. Ignition will be configured following the Enterprise Architecture documented in the SCADA System User Manual. 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.

The Ignition SCADA software will be programmed based on the guidelines established in the SCADA System User Manual.

### 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/maintenance switch will be monitored by the PLC through a UPS relay interface card and include the following the signals:

- UPS Fault
- UPS Battery Low
- UPS On Bypass
- UPS On Utility
- UPS On Battery
- UPS Bypass Switch in Bypass Mode

Additional Power related signals terminated to the PLC include:

- DC Power Supply #1 DC Voltage Fault Indication
- DC Power Supply #2 DC Voltage Fault Indication
- Surge Protection Device Surge Indication
- PLC Control Panel AC Power OK

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 Run Status
- Generator Fault Status
- Generator Auto Status
- Generator Battery Charger Fault
- Generator Battery Voltage Low
- Generator Enclosure Temperature

### 4.10.4 Instrumentation

New instrumentation for the project will comply with the City's Approved Product and Equipment List. 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). 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.

The water levels at the reservoir and Lake Geraldine will be monitored by pressure based level sensors. The power and signal wiring from the sensors will be wired to a small enclosure located in the intake station. Signal converters will be used to convert the analog (4-20mA) sensor signals to Modbus TCP. A network switch in the enclosure connects the sensors to the PLC network via fibre optic cable back to the pump station.

### 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:

- Local/Remote Status
- Emergency Stop
- Running
- General Fault
- VFD Fault
- Motor Leak Detection
- Motor Thermal Overload Fault
- Start/Stop Command
- Speed Setpoint

- Speed Feedback

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 be 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
- High Level Float
- Low Level Float

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
- Flow Totalizer

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..

### 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 funning 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 house 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.

# 5 Apex River Raw Water Pump Station

## 5.1 General

The Apex River (AR)The AR 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, compressors and air receiver tanks.
- Washroom facilities (if needed) will be determined during detailed design.

## 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, 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.
- 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

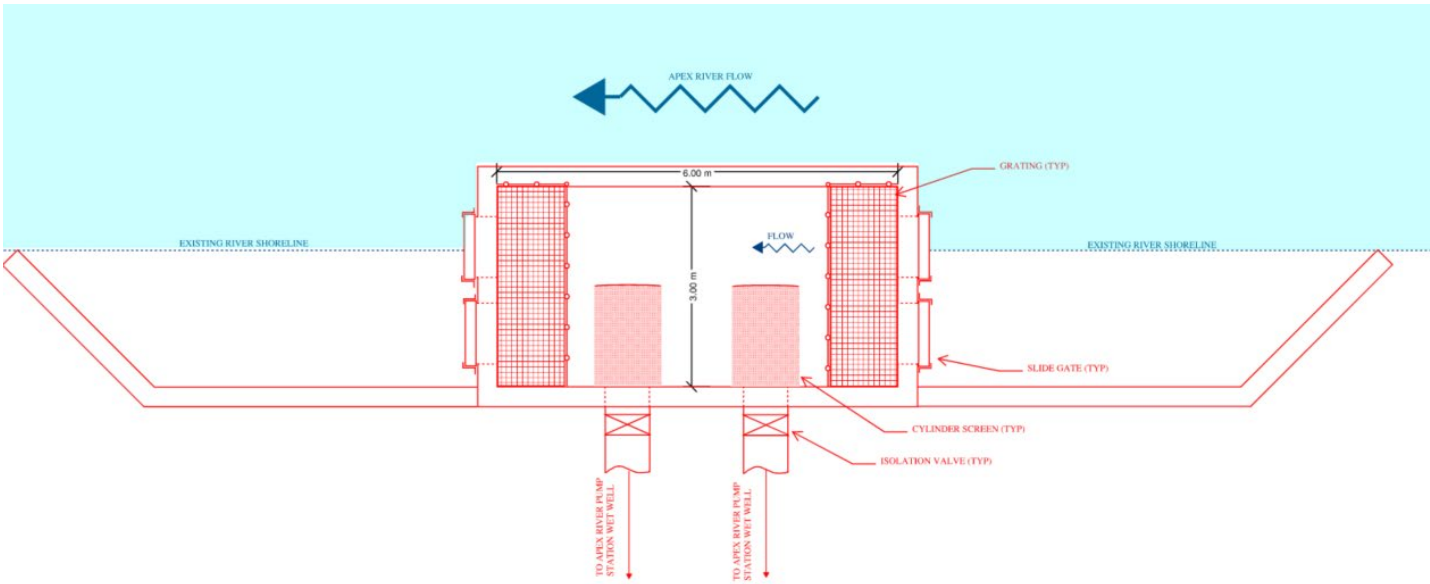
## 5.3.1 Crib

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 prevent 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

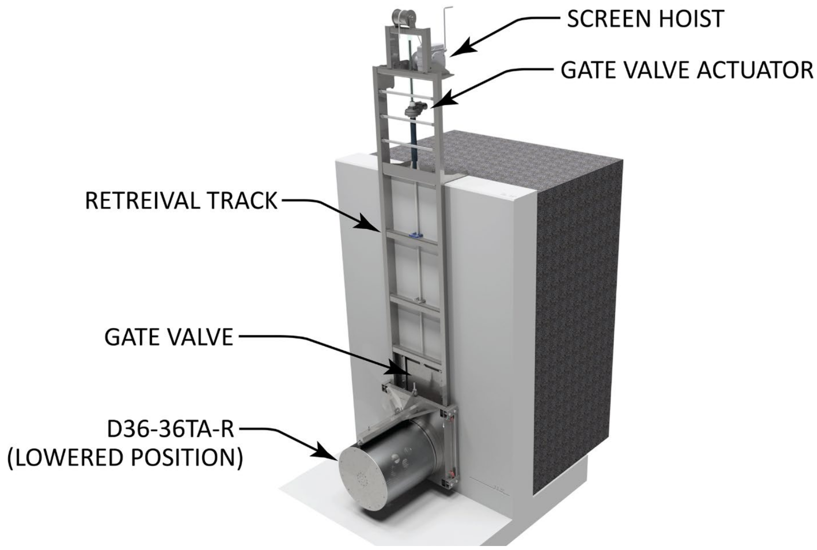
The intake screen is sized in accordance with the Department of Fisheries and Oceans (DFO) Freshwater Intake End-of-Pipe Fish Screen Guidelines for the fish that may be present in the Apex River. Previous studies identified the presence of Arctic Char. Arctic Char have a subcarangiform mode of swimming. Using a screen approach velocity of 0.11 m/s per the DFO guidelines, a screen with a clear area of 3.64 $m^2$ 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.

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 100 mm diameter sewage combination air and vacuum valve (SCAV) for raw water is installed on the 600 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, $m^3/s$ (cubic meters per second)
- $C$ = pipe roughness coefficient
- d = inside pipe diameter, m (meters)

Note: pressure drop can be computed from head loss as $h_f$ × 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:

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$^2$

#### 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.

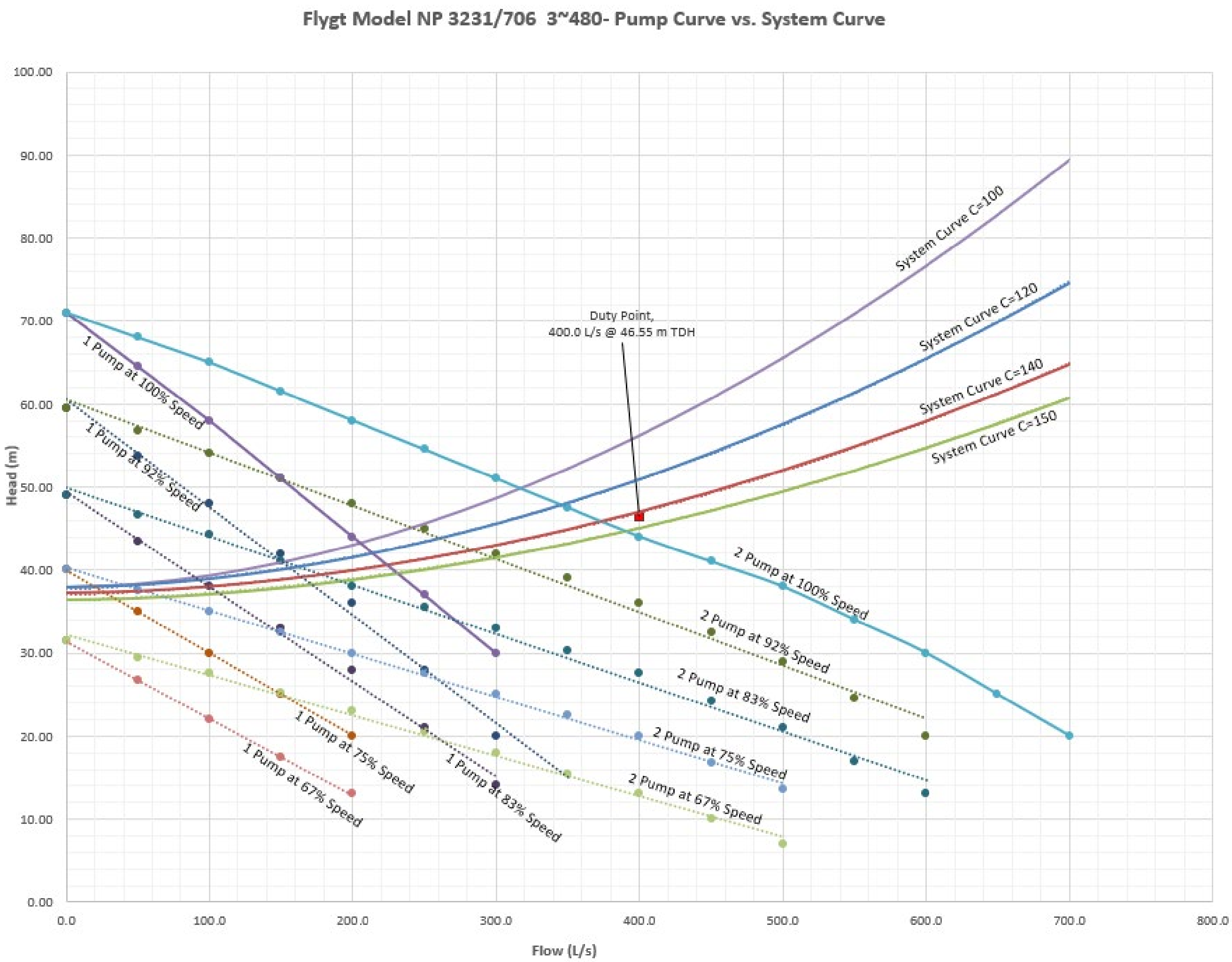
*Table 5-1 Pump Criteria*

| 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 | 138 kW (185 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 138 kW (185 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 100 L/s. However, this flow rate may be deemed insufficient for the operational requirements of this scenario. Should this limitation be confirmed, the feasibility of deploying larger-capacity pumps will be investigated to ensure adequate performance.

### 5.4.4.4 Pump Control Philosophy

Control of the raw water pumps is by a programmable logic controller (PLC) using local flow meter and system pressure as its primary input.

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 operate with rotating duty/stand-by cycles to maintain similar operating hours for each pump. 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 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 doors 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 (celerestory).
- 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 General

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 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: Mechanical ventilation will be provided to maintain space temperature at 10°C above ambient temperature. 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. Determination of requirements for sanitary facilities will be completed during detailed design.

#### 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

Details for washroom facilities will be determined with the City during detailed design.

#### 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, 600120/208 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 138kW (185 hp) duty raw water pumps.
- One 138 kW (185 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 (370 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.

Confirmation of feasibility for construction for power (by the local utility QEC) has been carried out. QEC confirmed that they can install power to the project for an approximate cost of $2.5-3M. A one-year notice for construction along is requested. 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 being procurement.

## 5.10 Instrumentation and Control/SCADA System

### 5.10.1 New SCADA Network Communication

The new Supervisory Control and Data Acquisition (SCADA) system will be designed based on the City's SCADA System User Manual Revision 1. The 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 during preliminary design 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

The SCADA system will be designed based on the City's SCADA System User Manual Revision 1. 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 Inducive Automation's Ignition software and will utilize the Allen Bradley Ethernet/IP driver. Ignition will be configured following the Enterprise Architecture documented in the SCADA System User Manual. 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.

The Ignition SCADA software will be programmed based on the guidelines established in the SCADA System User Manual.

### 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/maintenance switch will be monitored by the PLC through a UPS relay interface card and include the following the signals:

- UPS Fault
- UPS Battery Low
- UPS On Bypass
- UPS On Utility
- UPS On Battery
- UPS Bypass Switch In Bypass Mode

Additional Power related signals terminated to the PLC include:

- DC Power Supply #1 DC Voltage Fault Indication
- DC Power Supply #2 DC Voltage Fault Indication
- Surge Protection Device Surge Indication
- PLC Control Panel AC Power OK

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 Run Status
- Generator Fault Status
- Generator Auto Status
- Generator Battery Charger Fault
- Generator Battery Voltage Low
- Generator Enclosure Temperature

### 5.10.4 Instrumentation

New instrumentation for the project will comply with the City's Approved Product and Equipment List. All 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). 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.

The water levels at the reservoir and 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. At the reservoir the power and signal wires will be wired to an enclosure located in the intake station at Lake Geraldine along the service corridor for Lake Qikiqtalik. Signal converters will be used to convert the analog (4-20mA) sensor signals to Modbus TCP. A network switch in the enclosure connects the sensors to the PLC network via fibre optic cable routed to the pump station.

### 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:

- Control Power On
- Local/Remote Status
- Emergency Stop
- Running
- General Fault
- VFD Fault
- Motor leak detection fault
- Motor thermal overload fault

- Start/Stop Command
- Speed Setpoint
- Speed Feedback

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 be 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
- High Level Float
- Low Level Float

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
- Flot Totalizer

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.

### 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 funning 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.

# 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 the Lake Geraldine via the pipeline system. The preliminary design phase builds upon the initial conceptual groundwork, delving into more detailed design technical considerations. This involves further optimizing flow rates and managing pressure dynamics, pipe size, and hydraulic analyses including transients. 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).

The Preliminary Design Plan and Profile Drawings are included in **Appendix A.** A draft specification list is included in **Appendix D** (to be updated during detailed design).

## 6.2 Existing Conditions

The existing roadway network in the area consists primarily of unpaved roads. Notably, the “Access Road to Lake Qikiqtalik” 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 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 Qikiqtalik 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 alignment traverses hilly, rocky terrain characterized by continuous permafrost and several wetland areas. The topography includes a short ascent from Lake Qikiqtalik to a local high point, followed by a descent toward the Apex River, and then another ascent toward the new reservoir and Lake Geraldine.

# 6.3 Design Criteria

## 6.3.1 Watermain Location, Length and Sizing

The new Raw Water Transmission pipes are outlined as followed.

LQ Pump Station:

- A 600 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 400 mm HDPE DR11 RWTMs.

The 600 mm main is approximately 65 meters in length, while each of the 400 mm parallel mains will extend for approximately **3.9 kilometers**.
AR Pump Station:
**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 Pump Station 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

Where feasible, the proposed pipelines will be 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 preliminary 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.

As the design progresses, pipeline appurtenances will be evaluated for quality and suitability to minimize future maintenance requirements.

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

| Material | HDPE | PVC | CPP | DI |
|---|---|---|---|---|

| Criteria | | | | |
|---|---|---|---|---|
| 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 system for maintaining 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., 500 L/s and/or 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. 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 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 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 will be 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 VC9 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 another eight locations (VC2, VC3, VC5, and VC8 for both RWTMs).

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, VC13, VC 14 and VC15 at outlet chambers and one at the other locations at VC9 at VC14. Due to the absence of power along the pipe alignment to support the pressure transmitters, manual pressure gauges will be located at one location (VC9).

### 6.3.5 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. Specific details regarding the size and spacing of these blocks will be finalized during the detailed design phase.

### 6.3.6 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) |
| Apex River Crossing | Nunavut Planning Commission (NPC)<br>Nunavut Impact Review Board (NIRB)<br>Department of Fisheries and Oceans (DFO) | 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. (Pier spacing will be determined during detail design and informed by geotechnical). | NIRB, DFO & NPC to be contacted for further consultation meetings (meetings have been ongoing) to finalize permitting requirements. |
| 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. |

### 6.3.7 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. During detailed design, these options will be reviewed for availability, size compatibility, pressure rating, and ease of installation and maintenance.

The design will account 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.

During the detailed design phase, restrained length calculations will be 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 will follow guidance from AWWA manuals and manufacturer recommendations for the selected pipe and coupling systems. The results will inform the placement of components such as expansion/contraction joints and the use of fully restrained couplings at critical locations.

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

### 6.3.8 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. Smaller localized drains may also be required, this will be determined during detail design.

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 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 Apex River or ground is less than 5 hours. The main pipeline drain location will include chambers complete with person access and appropriate drain discharge size and configuration. Smaller localized drains may also be required, this will be determined during detail design.

## 6.4 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, comprising twin 400 mm diameter pipes, 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 Lake Qikiqtalik 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 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, running parallel to an existing 400 mm HDPE pipe, 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.

During the detailed design phase, horizontal and vertical alignment of the pipeline will be refined 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.

## 6.5 Transient Analysis and Pressure Rating

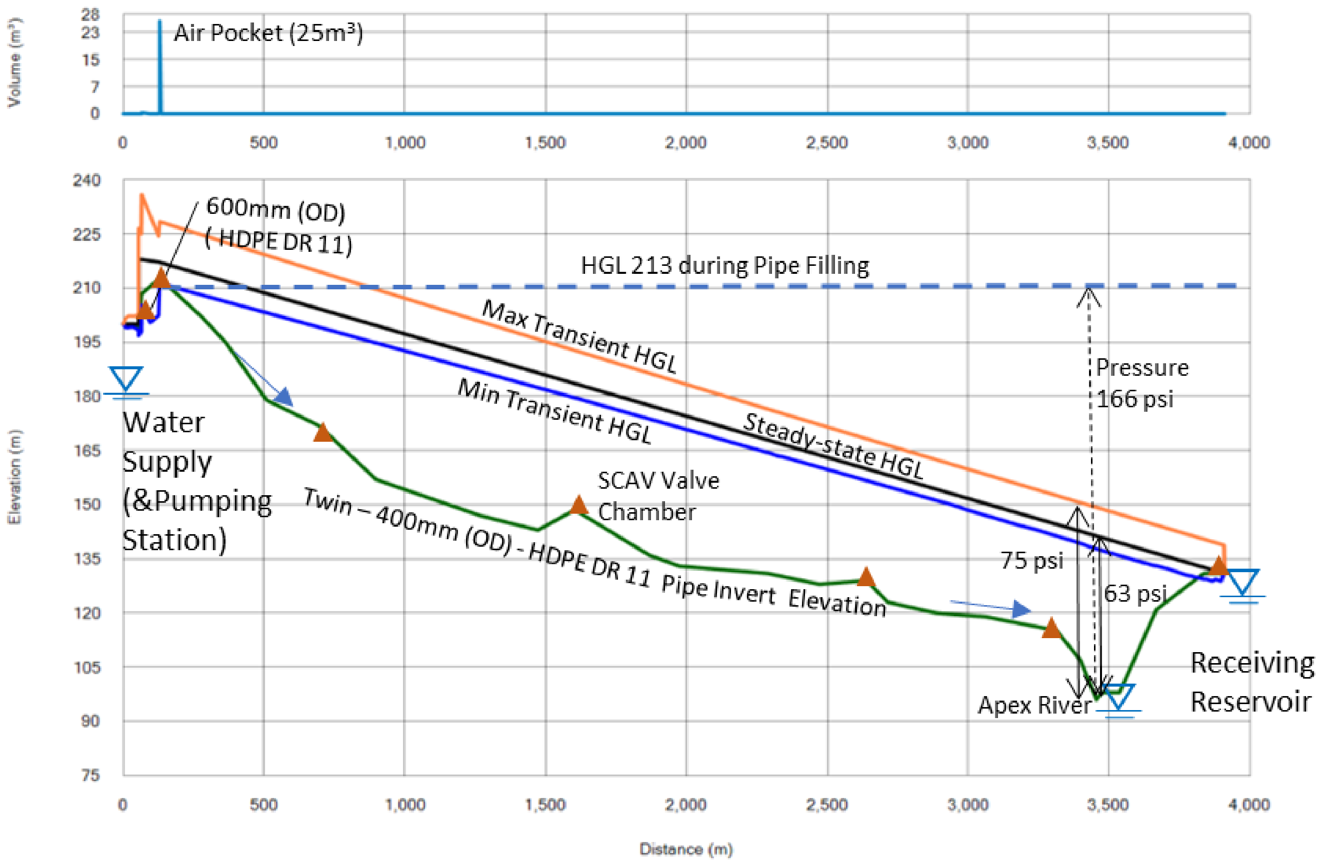
For the LQ system, the software program Bentley HAMMER was used to undertake the hydraulic transient analysis for the RWTM. The worst-case transient condition occurs for the RWTMs upon pump trip (e.g., power failure). This has been analyzed and the results are summarized below.

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 pressure wave speed of 350 m/s (for HDPE DR11 pipe) and 1200 m/s for SS pipe.
- A flow of 580 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 Qikiqtalik (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.
- Surge protection devices: A total of 13 sewage combination air valves (SCAVs) are proposed for the RWTM system. One small SAVs (25 mm) at the discharge side of the PS 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 CAV are to be revisited during the detailed design stage. The SCAV is recommended as the pipeline will contain raw water that may have solids.

Figure 6-1 shows the elevation profile and minimum/maximum hydraulic grade lines (HGLs) for the RWTM from the 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 surge 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 170 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 170 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 30 $m^3$ 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.

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

*Figure 6-2 Hydraulic Profile of AR System*

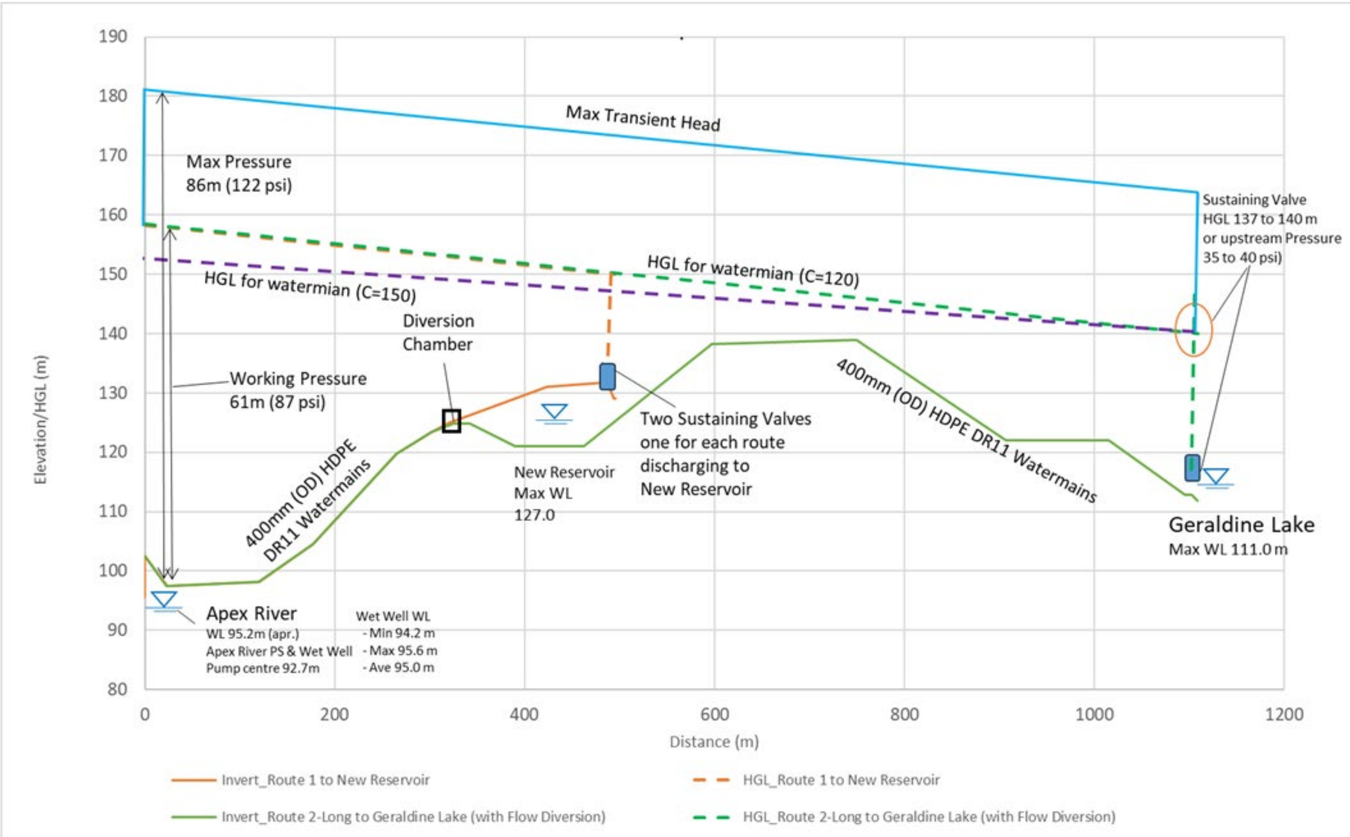



**Figure 6-2** shows the hydraulic profile of AR system for Route 1 and the long Route 2 with 200L/s via each route. Pressure Sustaining Valves (PSVs) each will be provided at the outlets of Route 1, Route 2-Short (to new reservoir) and Route 2-Long (to LQ).

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. If the system can be operated independently (i.e., without connection to or diversion flow to Geraldine Lake), a Pressure Sustaining Valve (PSV) may not be required at each outlet.

A downhill-sloped and/or elevated section of watermain has been identified along Route 2-Long from Apex River Pump Station to Geraldine Lake. The recommended pressure sustaining valve setting is 40 psi, equivalent to a HGL of 141 m at the outlet.

Route 1 and Route 2- Short (and Route 2-Long) are supplied via the common discharge header of Apex River PS. The system is interconnection and Pressure Sustaining Valves (PSVs) would also be required on Route 1 and Route 2-Short to maintain minimum upstream pressure when flow is being diverted to Geraldine Lake. However, under conditions where there is no flow diversion to Geraldine Lake (Route 2-Long), the sustaining valves for Route 1 and/or Route 2-Short do not need to maintain a minimum pressure and may remain fully open during operation.

The required pump total dynamic head (TDH) ranges from 40 m to 65 m, depending on system operating conditions.

The guidelines suggested that a transient surge be considered with a velocity of 2ft/s (0.6m/s) for large transmission watermains. The watermain pipes and joints will conform to AWWA C900, C905 and are designed toa pressure of 1034kPa (150psi).

The proposed watermains within the subject site are HDPE DR11. The typical wave speed for a HDPE pipe is approximately 400m/s and the maximum surge can be calculated using the equation:

Hs = av/g (m),

• Hs - Pressure rise due to Surge (m)

• a - Pipe Wave speed (m/s)

• v - 0.6m/s, instantaneous flow velocity change (as suggested by MECP)

• g - 9.8m/s2 acceleration of gravity

Estimate Hs = 24.5m (240kPa, or 35psi)

A water column moving at 0.6m/s will produce a transient pressure of 240kPa (35psi). The detected static pressure is approximately 90psi (620kPa) from modelling for the area within the subject site. The projected total

pressure is 850 kPa (122 psi) is less than the proposed watermains are designed to withstand. All pipes, valves

It is recommended to install surge protection devices (e.g., Surge Anticipating Valves, SAV) and a Pressure Relief Valve (PRV) with a by-pass line at the AR Pump Station. A detailed Technical Memorandum on transient analysis for both LQ and AR systems will be prepared during the detailed design stage of the project.

## 6.6 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). Preliminary locations of valves are shown on the preliminary design drawings.

### 6.6.1 Valve Chambers

The following valve chamber design assumptions have been made:

- Concrete vaults must be sized to allow adequate space to house equipment and for operations and maintenance personnel movement when inside.
- Concrete 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 when servicing equipment must be accounted for.
- Precast structures will be used, when possible, to minimize on-site construction work.

Valves will be operable from grade level using valve stems.

### 6.6.2 Main Line Valves

Four 400 mm automated ball valves will be located along the alignment, with two situated in the chamber VC1 and the remaining two in the chamber VC9 at the reservoir outfall. Also, for operational purposes, a pressure-sustaining valve will be located within VC9 at the pipe outlet to regulate the flow and facilitate the filling of the pipe with water. This valve will control the pressure to ensure that the pipeline reaches its desired operating conditions (Refer to design drawings in **Appendix A**). Additionally, valves shall be the same size as the corresponding main. It shall be noted that mainline valves may need to be limited along the negative slope alignment in order to mitigate operator error and significant transient occurrence during normal operation. More details will be provided during detail design.

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.

### 6.6.3 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 Qikiqtalik 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.

Additional details on the drain valves will be provided in accordance with established standards during the detailed design phase.

### 6.6.4 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. Preliminary location of Air Structures/ valves are illustrated within the preliminary design drawings (Refer to **Appendix A**).

For the LQ system four of 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 four of 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.

## 6.7 Trenchless Aerial Crossings

-- Four 400 mm Raw Water Transmission Mains (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 2.4 m-high piers, spaced at 6 m intervals.

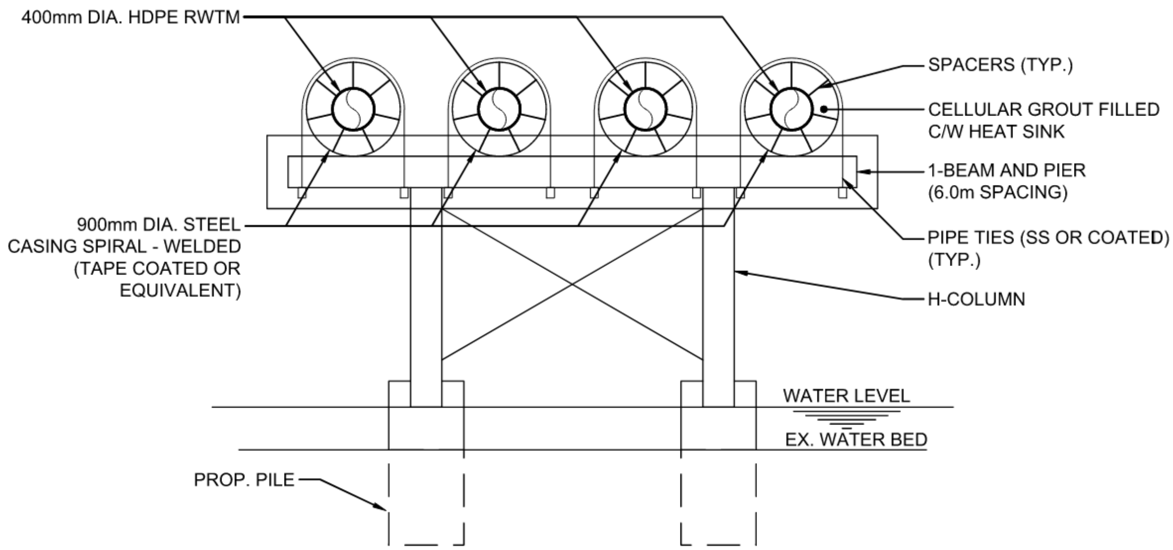
The river channel and floodplain span approximately 45 m at the crossing location. To maintain alignment with existing ground elevations, the steel casings will extend across a total length of 90 m.

Final specifications—including pier spacing, dimensions, and casing details—will be confirmed during the detailed design phase, following the results of the geotechnical investigation.

Figure 6-3 shows the proposed river crossing location, while Figure 6-4 provides a preliminary detail of the aerial crossing configuration.

*Figure 6-3 Apex River Crossing Location*

Figure 6-4 Apex River Crossing Aerial Detail




# 6.8 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 will be reviewed during detailed design.

# 6.9 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. 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. 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. More details will be addressed in the detailed design.

# 6.10 Instrumentation and Control

## 6.10.1 Lake Qikiqtalik Flow Control and Pressure Monitoring

Discharge flow from Lake Qikiqtalik Raw Water Pump Station (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:

- BV3 Ball valve
- BV4 Ball valve
- PT1 Pressure sensor
- PT2 Pressure sensor

VC10:

- BV23 Ball valve
- BV24 Ball valve
- PT3 Pressure sensor
- PT4 Pressure sensor
- PSV1 Pressure sustaining valve
- PSV2 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 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 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.

### 6.10.2 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
- PT5 Pressure sensor
- PT6 Pressure sensor
- PSV4 Pressure sustaining valve
- PSV5 Pressure sustaining valve

VC12:

- BV4 Ball valve (diverts flow to Lake Geraldine)
- BV5 Ball valve (diverts flow to the reservoir)
- PT7 Pressure sensor
- PSV1 Pressure sustaining valve
- PSV2 Pressure sustaining valve

VC15:

- BV9 Ball valve
- PT8 Pressure sensor

- PSV3 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 ≈1,600,000 $m^3$ 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.
- Preliminary geotechnical stability analyses.
- Preliminary 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 geotechnical investigations to be completed in 2024, including stratigraphic information from boreholes advanced onsite; piezometer and thermistor data; and material parameters obtained from 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 C301.

### 7.2.1 Subsurface Conditions

There is no readily available geotechnical information for the subject site. Inferences with regards to groundwater, soil stratigraphy, rock mass types and characteristics have been made based on publicly available literature, correspondence with contractors operating in the area, and visual site inspections. Design revisions will be made once site-specific geotechnical work is carried out and representative material samples obtained for laboratory testing. The geotechnical investigation program is currently scheduled to be completed in autumn 2024.

According to the preliminary information obtained, 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 conducted by Arcadis staff in September 2023, 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), bedrock in the area consists of Proterozoic granulite-facies granitoids. This rock is expected to be 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 C301 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 | 12.0 | 370 |
| Dyke | 2 | 5.0 | 100 |
| Dyke | 3 | 5.0 | 130 |
| Dyke | 4 | 4.0 | 90 |
| Dyke | 5 | 3.5 | 45 |
| Dyke | 6 | 10.0 | 190 |
| Dyke (Spillway) | 7 | At existing grade | 75 |
| Dyke | 8 | 9.0 | 170 |

*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 C301); 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<br>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<br>Loss of marginal habitat only<br>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 *important* fish or wildlife habitat<br>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 *critical* fish or wildlife habitat<br>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 *critical* fish or wildlife habitat<br>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) |

**Note 1.** Definitions for population at risk:

**None**—There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.

**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).

**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).

**Note 2.** Implications for loss of life:

**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.

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 detailed design (for example, the inundation study is already underway). 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 dams 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 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 Selection Technical Memo).
- The crest of the dam and dykes will be used for vehicle travel. Crest width has been specified at 10 m.
- 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.
- 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 collection system will be installed at the downstream toe of Dam 1, but not for other dykes.
- 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, it is expected that the storage volume will be relatively empty 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 29.77 ha (refer to **Appendix G** for the Hydrology Technical Memorandum). Given that the reservoir is expected to have discharged roughly 1.2 M $m^3$ 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), 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 71,500 $m^3$ per event. Given a normal operating reservoir level at elevation 127 m, the PMF volume would increase the reservoir level to 127.42 masl, based on the stage-storage curve for the reservoir system area. Further considering wind setup and wave height, a crest elevation of 129.0 m for the system is considered adequate. For greater detail on hydrology analyses and a discussion of wave-run up factors, etc., please refer to **Appendix G**.

The dimensions of the emergency spillway are a 25 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 127.25 m. The predicted peak flow, given the SCC type II for rainfall hyetograph, is 25.18 $m^3/s$. Given the dimensions of the spillway, this results in a peak flow depth of roughly 1.0 m in the spillway channel. This confirms that the design crest elevation of 129.0 m for the retaining structures is adequate.

# 7.5 New Reservoir Design

Preliminary design drawings of the new reservoir are included in **Appendix A**. A draft specification list is included in **Appendix D**. One dam (Dam 1), six dykes, one spillway and the site access road 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 rock-fill 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 preliminary 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: 129.0 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 (Cushion 1).
- 1 m thick cushion layer (bedding sand) on either side of the geosynthetic liner assembly (Cushion 2). 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 the 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 C301. 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 spillway is essentially an overflow swale. 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 25 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 127.25 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.

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 expected 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 preliminary design drawings in **Appendix A**.

The dam, dyke and spillway structures can be seen as well. To achieve the desired storage volume, the entire reservoir footprint is to be excavated down to elevation 117.0 m. It is anticipated that in some locations this will be achieved through overburden removal alone.  In other areas, this excavation will extend well into the competent bedrock underlying the site and will be achieved through blasting and removal using heavy machinery.

Currently, the retention structures have been designed so as to minimize their impact on the excavation footprint, allowing for a potential capacity of 1,834,300 $m^3$ if desired. The further design development will discuss reservoir storage capacity and any recommendations for overcapacity contingency. The actual reservoir footprint and volume will be quantified during construction and documented on as-built drawings.

The estimated cut volume predicted by the model is 1,126,000 $m^3$ minus reductions in the excavation footprint for construction access or areas not excavated should the overcapacity contingency not be required. This includes overburden to be stripped and water volume to be removed (there is no bathymetric data for the existing water bodies); the actual excavated volumes will be somewhat lower than this number. Excavated material volumes, required amounts for construction, and excess excavated material amounts are discussed in Section 6.

Drawing C301 shows the full excavation extents with the retention structures indicated. Drawing C315 shows the latitudinal cross-section A-A', across the long axis of the reservoir. Drawing C316 shows the latitudinal cross-section B-B' at the Dam 1 midpoint, and Drawing C317 shows the longitudinal cross-section C-C' stretching the length of the reservoir from the north and ending at Dam 1.

Blasting programs and designs will be influenced by the results of the seismic testing to be performed as a result of the geotechnical investigation program.  For a preliminary assessment of blasting conditions and design strictures, please refer to the preliminary Blasing 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 were conservative estimates based on experience with similar materials. Design estimates 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) |
|---|---|---|---|
| Rock Fill | 21 | 0 | 41 |
| Cushion 1 | 20 | 0 | 35 |
| Cushion 2 (sand bedding) | 19 | 0 | 32 |
| 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. A full instrumentation plan will be developed as part of Phase II, during the detailed design process. Tentative instrument locations are provided for Dam 1, on Drawing C302. The results of the geotechnical investigation are required to design final instrumentation plans.

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 thus be expected to include seepage weirs, horizontal and vertical control points, piezometers, wire-line inclinometers and thermistors. The detailed design plan will 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 will be considered. Tentative instrument locations are provided for Dam 1, on Drawing C302. The results of the geotechnical investigation are required to design final instrumentation plans.

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.
- Inclinometers to monitor slump, creep or movement.
- Thermistors to monitor the thermal regime and quantify freezeback into structures or talik formation under the new reservoir.

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 are intended to be linked to an alarm system in SCADA. Depending on inundation modelling (to be performed as part of detailed design) other structures may also require real-time alarm systems. Detailed instrumentation plans will be determined during detailed design phases once the geotechnical investigations have been completed. The 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

Eleven (11) monitoring wells were installed during the February/March 2024 mobilization, and slug testing will be performed on each during the August/September 2024 mobilizations. Seepage estimates will be calculated using these results, observed rock mass characteristics (via camera in air track holes and samples from cored holes), and depth of permafrost as measured by piezometers installed during the August/September mobilizations. Based on the variability of results and confidence based on field observations, packer testing may be recommended as part of subsequent investigations, if necessary, to provide more accurate estimates.

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 during the geotechnical investigations.

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 during the 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 2mbgs (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 few to no continuous fractures at depth), this is not anticipated to be a concern. The geotechnical investigation will help quantify this risk.

Please refer to the Climate Lens Assessment 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 to provide a level surface, facilitating the placement of the pipes inside and improving personnel access. This entire assembly will be positioned within a trench and encased with concrete 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**. Further details and a comprehensive evaluation will be provided as part of the detailed design phase. The connection between reservoirs is referred to as a “service corridor”. The service corridor will be maintained at a minimum temperature to prevent freezing. Heat trace may be required on portions of the pipe (i.e., at entrance/exit), which will be further evaluated.

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 $m^3$/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 intake structure that is incorporated into the service corridor will contain an electrical panel (Intake Structure Panel) from which the feed for the heat tracing (if determined to be needed)/heating will be fed from. Heating will be sized to maintain above 5C ambient temperature. Motorized valves on either end 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 Intake Structure panel. This needs to be coordinated with the utility to see if poles will be provided in the vicinity to run cabling to the valve chamber.

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 Intake Structure 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 be required to support the inlet location.

The service corridor has been designed with maintenance of the pipe and valves in mind. A concrete floor will be included and specifications will include a trolley and needed pipe maintenance tools for the City to keep in storage should any repairs be required (see drawings for design of corridor). The service corridor will be considered a confined space.

## 7.8 Instrumentation and Control/SCADA System

Raw water from the reservoir flows through two pipelines to Lake Geraldine. Motorized gate valves and flow meters will be installed in the service corridor and controlled by the PLC controller in Lake Qikiqtalik RWPS. The valves and flow meters are as follows:

- GV1 Gate valve
- GV2 Gate valve
- GV3 Gate valve
- GV4 Gate valve
- FT1 Flow meter
- FT2 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

The valve actuators and flow meters in the service corridor will connect to the PLC IO rack located in the Lake Qikiqtalik RWPS.  The PLC IO rack will be installed in a free-standing enclosure. The valve actuators and flow meters will be connected by copper patch cables to an Ethernet switch inside the intake station for Lake Geraldine  which is located approximately 4.6 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 intake station and the RWPS. The fibre link will be routed along the hydro poles installed alongside the pipelines. 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 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 intake station 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.

The water levels at the reservoir and Lake Geraldine will be monitored by pressure based level sensors. The power and signal wiring from the sensors will be wired to a small enclosure located in the intake station. Signal converters will be used to convert the analog (4-20mA) sensor signals to Modbus TCP. A network switch in the enclosure connects the sensors to the PLC network via fibre optic cable back to the pump station.

# 8 Site Drainage

As noted in Section 3, the RWPS site slopes naturally from west to east towards Lake Qikiqtalik.

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. We will provide a drainage plan in the next submission to address both surface water and subsurface drainage to prevent erosion, manage seepage, and reduce pressure on the embankment. Below are the key components of a drainage plan.

#### 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 concrete 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 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.

**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.

Toe drains and collector drains will include the following components:

**Toe Drain System**: 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, such as a nearby watercourse or stormwater management facility.

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) or a stormwater management facility. These channels will be designed to handle the peak flow rates expected during storm events.

A drainage map in indicating stormwater management features and outfall locations will be provided as part of detailed design.

# 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 will be required. This small quarry will 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.

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 7.3 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, several small ponds will be dewatered and discharged to the drainage channels leading to the Apex River. Drainage will use existing flow channels for a minimum of 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.

The existing ponds inside the reservoir footprint will similarly be dewatered in preparation for excavation and construction access to the area. Discharge is anticipated to be through the spillway, through the existing drainage channel to the Apex River. As there is no bathymetry currently available for these ponds, the anticipated discharge volume is unknown.

Based on instruction from the City, field investigation scope will be limited to items detailed in the RFP and the full geotechnical program proposed in Arcadis' Geotechnical Workplan (provided under separate cover) will not be conducted in 2024. As such, visual depth estimates of the five ponds within the footprint of the proposed reservoir and access road will be attempted from shore in September 2024 during the geotechnical program. It is anticipated the information collected in 2024 will be incorporated into the PBSEIA report which is part of the NWB submission. A detailed bathometric survey is recommended in 2025 to improve water volume estimates and bedrock excavation requirements.

## 9.3 Intake and Outlet Pipes

For the intake pipe at the RWPS a coffer dam construction is proposed in LQ to allow for installation of the screens and connected pipe (at flanged fitting). 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 is also required at a small initial quarry to a) achieve a viable access route to the reservoir site and b) obtain the required material for the initial site access road construction.

Blasting and excavation plans will be updated upon the completion of the final Blast Assessment Report. 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 preliminary draft Blast Assessment Report dated December 15, 2023 has been prepared by Explotech Engineering Limited and has been included in **Appendix J.** This assessment will be revised pending the results of the geotechnical investigation onsite.

The planned geotechnical program (to be conducted this year) includes bedrock coring, which will provide samples for rock mass strength testing, fracture mapping, RQD and ABA testing, as well as a downhole geophysics program. These parameters will inform the blasting strategy.

## 9.5 Quarry and Borrow Materials

As described above, excavated rock will be sourced initially from the small quarry adjacent the Road to Nowhere, and then from the reservoir excavation. Excavated rock will be transported to crushing stations, 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 LLDPE liner) will be obtained from the previously used borrow areas along the Road to Nowhere/access to Lake Qikiqtalik. A technical memorandum with material quantity estimates will be produced after the planned geotechnical investigation confirms depths to bedrock, deposit extents, and provides samples for preliminary material gradation curves.

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 845,000 $m^3$, less the volume of overburden and existing ponds onsite. The total requirements for dams and peripheral structures, as shown in Table 6-1 and Table 6-2 below, are estimated at 228,500 and 295,400 $m^3$, respectively. That leaves an excess of roughly 320,000 $m^3$, generated over the full course of the project. 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.

A more accurate model and quantities will be generated using the results of the geotechnical investigation, when some measurements of overburden/water depths can be quantified. Future design models will provide more accurate quantity assessments.

## 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.

- 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 6” minus material. The maximum material size used will be <0.5m. Material to be placed in 0.5 m lifts and compacted using heavy equipment tracks.
- Cushion 1: 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 excavated bedrock, this material will be crushed and screened to achieve the filter grade required. Material to be placed in 300 mm vertical lifts and compacted at ±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., 2" minus grading), a process compaction method will be specified.

- Cushion 2: 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 to be placed in 300 mm vertical lifts and compacted at ±2% optimum moisture content using a smooth drum vibratory roller to 95% SPMDD.
- Geosynthetic Liner: liner selection is discussed in the Technical Memorandum included in **Appendix H**. The liner is a linear-low density polyethylene (LLDPE) geomembrane, 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 (Cushion 2) and sandwiched between layers of non-woven Terrafix 600R or equivalent geotextile.
- 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 Cushion 1 material.

An estimate of the earthwork quantities for construction of the reservoir system structures is summarized in Table 9-1.

*Table 9-1 Estimated Material Quantities - Retention Structures*

| STRUCTURE | TOTAL QUANTITIES | | | | | |
|---|---|---|---|---|---|---|
| | VOLUME ($M^3$) | | | | AREA ($M^2$) | |
| | ROCKFILL | CUSHION 1 | CUSHION 2 | RIPRAP | GEOMEMBRANE | GEOTEXTILE |
| Dam 1 | 100788 | 16676 | 6922 | 9234 | 11063 | 22126 |
| Dyke 2 | 5116 | 2035 | 680 | 1148 | 1368 | 2736 |
| Dyke 3 | 6887 | 3460 | 1266 | 2005 | 2326 | 4653 |
| Dyke 4 | 2071 | 1569 | 501 | 842 | 942 | 1885 |
| Dyke 5 | 740 | 673 | 199 | 366 | 408 | 816 |
| Dyke 6 | 24802 | 5935 | 2279 | 3063 | 3707 | 7414 |
| Dyke 7 (Spillway) | 1405 | 1114 | 339 | 583 | 1445 | 2890 |
| Dyke 8 | 15902 | 5134 | 1933 | 2824 | 1445 | 2890 |
| **Total** | **157715** | **36599** | **14121** | **20069** | **22706** | **45413** |

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 9-2.

Please note that a quantity of stone for Lake Qikiqtalik road enhancement has been included; this was not considered part of the original scope of work but road upgrades/maintenance are considered prudent. The design of these road upgrades/reinstatement would be considered an additional item requiring a change order.

*Table 9-2 Estimated Material Quantities - Peripheral Structures*

| Structure | Volume ($m^3$) | | ESTIMATED LENGTH / AREA |
|---|---|---|---|
| | Rockfill | Cushion 1 (Vehicle Surface) | |
| Access Road | 16,000 | 1,600 | 1,600 m |
| Ring Road | 20,000 | 2,000 | 2,000 m |
| Laydown Areas (Various) | 75,000 | 25,000 | 50,000 $m^2$ |
| Pipeline Berm | 18,000 | 9,000 | 4,200 m |
| Qikiqtalik Road | 24,000 | 4,800 | 6,000 m |
| Repair Stockpile | 100,000 | - | - |
| **Total** | **253,000** | **42,400** | |

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, or basalts. 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.

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 material will be placed through a cone, jaw, and screener assembly spread for resizing to meet the design criteria of the containment berms.

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/](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 9-3 for reference of design mitigation measures that will be included for infrastructure within this project.

*Table 9-3 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. |
| 2 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.<br>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.<br>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 shot 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 - 8 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.<br>Potential Acid Generating (PAG) rock will not be used in the construction of the dams. |
| | Permafrost Depth | All overburden will be stripped from beneath the footprint of the dams and dykes, allowing the dams to be founded on bedrock.<br>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**.

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.<br>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.<br>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

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

50% detailed design is underway. For the AR pump station – comments from the City will be received before proceeding.

## A – Preliminary Design Drawings

# B – Survey Report

# C – Geotechnical Workplan

## D – Draft Specification Table of Contents

# E – Pump Calculations

## F – Pump Curves

# G – Hydrology Technical Memorandum

# H – Liner Material Technical Memo

# I – Stability Analysis Technical Memo

# J – Preliminary Blast Assessment Report

## K – Preliminary Systems Operation Manual

# L – Basis of Estimate

## M – Environmental Management Plan (EMP), Environmental Protection Plan (EPP), Erosion and Sediment Control Plan (ESCP), Climate Lens Report