



**Conceptual Design Advancement
for Raw Water Supplementation
from the Sylvia Grinnell River,
Iqaluit, NU**

FINAL

April 2019

Prepared for:
City of Iqaluit
Iqaluit, Nunavut

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Project Number: 144930114

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Abbreviations

AZR	Airport Zoning Regulations
City	City of Iqaluit
CRA.....	Commercial, Recreation, or Aboriginal
DFO.....	Fisheries and Oceans Canada
GPS.....	global positioning system
LGHC	Lake Geraldine Headwater Creek
NIRB.....	Nunavut Impact Review Board
NLCA.....	Nunavut Land Claims Agreement
NPC.....	Nunavut Planning Commission
NWB.....	Nunavut Water Board
Nunami.....	Nunami Stantec Ltd.
NuPPAA	Nunavut Planning and Project Assessment Act
WSC.....	Water Survey of Canada

1 INTRODUCTION

The City of Iqaluit, Nunavut (the City) is investigating options to supplement the City's current potable water supply, which is the Lake Geraldine Reservoir. Nunami Stantec Ltd. (Nunami) was retained by the City to develop and evaluate options for withdrawing water from the Sylvia Grinnell River to supplement the current supply.

This report outlines the findings and analyses conducted by Nunami to evaluate two options for the withdrawal of raw water from the Sylvia Grinnell River, and its transport to the Lake Geraldine reservoir. Nunami's evaluation included desktop assessment of river morphology and pipeline route reconnaissance; field study of river morphology and fish use and habitat; and, cost estimation and conceptual design for the water intake structure and pipeline route associated with withdrawal at two locations in the Sylvia Grinnell River. The regulatory implications and requirements of proceeding with the selected option(s) are also evaluated.

1.1 Background

The City currently withdraws potable water from a constructed reservoir impoundment at Lake Geraldine (see Figure 1-1). The 50th percentile historic probability of rainfall runoff and snowfall accumulation yields for the Lake Geraldine is 977,000 cubic metres (m³) per year (Golder 2013). The Lake Geraldine impoundment was initially constructed in 1958 by the Department of National Defence, when Iqaluit (then Frobisher Bay) was a hub for DEW Line¹ construction operations, and the dam has been raised several times since then: in 1979, 1985, and 1995, and most recently in 2006 (Concentric 2014). The current dam spillway elevation is 111.33 m above sea level (masl; Concentric 2014). It has been estimated that the reservoir could service a community population up to 12,800 (City of Iqaluit 2010); however, the quantity of water within the Lake Geraldine watershed (i.e., basin yield and reservoir volume) is estimated sufficient to service a population up to 8,300 only (ibid.). For a population above 8,300, a secondary water source will be needed to supplement the Lake Geraldine reservoir to meet the community's ongoing potable water supply needs.

Iqaluit has been ranked as the fastest growing community in Nunavut, and between 2001 and 2006, was among the top 15 fastest growing communities in Canada (City of Iqaluit 2010). The Nunavut Bureau of Statistics (2014) provides population projections, currently based on the 2011 Canadian census, and estimates a 5.5% population increase in Iqaluit over the next five years, from 2018 to 2022 (from 7,881 individuals, to 8,318). However, based on the 2016 Canadian census, Iqaluit has already experienced a 15.5% population increase between 2011 and 2016, from 6,699 individuals (2011) to the current estimate of 7,740 individuals (Statistics Canada 2017).

¹ DEW Line refers to the Distant Early Warning Line radar system, which was constructed in the 1940s and 1950s across the Canadian Arctic



Sources: Base Data - Government of Canada; Thematic Data - Stantec Ltd.

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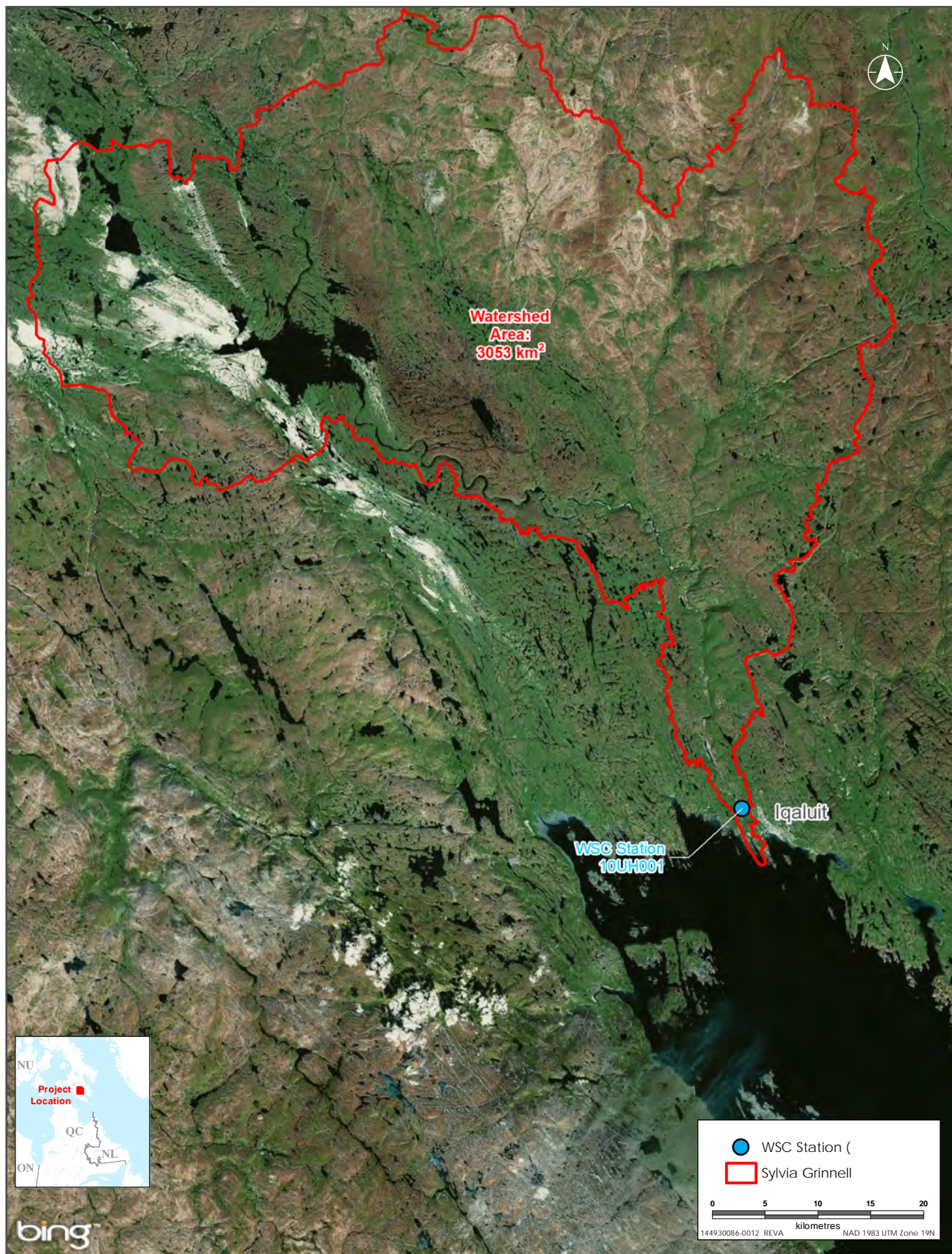
City of Iqaluit - Raw Water Supplementation Project Area

Iqaluit's population is fast approaching the estimated limits of water supply from the Lake Geraldine watershed (i.e., population of 8,300). Given the current population estimates, and population projections for the community, the need for a secondary, supplemental water source for the City is apparent. In their General Plan (City of Iqaluit 2010), the City identified the Apex (Niaqunguk) River as a potential secondary water source for seasonal resupply (i.e., resupply operated in summer only) and included both the Lake Geraldine and the Apex River watersheds within their Watershed Protection Area to limit development within the basins to protect the water resource.

Exp (2014) suggested that the Apex (Niaqunguk) River could meet the supplementation requirements of the City if there were to be no requirement to maintain a minimum stream flow for protection of fish; and that all water flowing in the river would be available for withdrawal, if needed. This assessment was completed in advance of any field studies, and in 2016, Nunami (2017a) identified a resident population of Arctic char (*Salvelinus alpinus*) in the Apex River.

As the Arctic char may be considered a Commercial, Recreational, or Aboriginal (CRA) fishery, as defined under Section 2(1) of the federal *Fisheries Act*, minimum flow requirements would need to be considered for the Apex River to prevent serious harm to fish, and a limit would likely be placed on the volume of water available for withdrawal. Minimum flow requirements would need to consider the Fisheries and Oceans Canada (DFO) *Framework for Assessing the Ecological Flow Requirements to Support Fisheries in Canada* (DFO 2013a), which recommends that, to avoid serious harm, diversion from a riverine ecosystem should not exceed $\pm 10\%$ of instantaneous flow, or that any diversions should not reduce instantaneous flow below 30% of the mean annual discharge. Nunami (2017a) completed a desktop assessment of the Apex River historical flow data, based on DFO (2013a), and determined that under these restrictions, the supplementation requirements for the City could not have been met every year over the 32-year historical flow record, and average withdrawal rates would need to range from 13 to 18% of available flow (i.e., greater than the 10% limit). Bakaic et al. (2017) identified similar limitations with the Apex River water supply using 20-year forecast modeling based on the primary source of the Apex River water (i.e., rainfall), and multiple climate scenarios. Therefore, proceeding with the Apex River as the secondary, supplemental water source for the community would not provide a long-term solution for seasonal resupply without compromising the resident char population.

As a potential alternative to the Apex River, Nunami (2017b) conducted a desktop assessment of historical flow data of the Sylvia Grinnell River, with consideration of DFO (2013a), and determined that the supplementation requirements could have been met every year of the 37-year historical flow record, with average withdrawal requirements ranging from 1.05 to 2.30% of available flow during open water periods. As a result, the City is investigating the potential for the Sylvia Grinnell River as a secondary, supplemental water source for seasonal resupply.



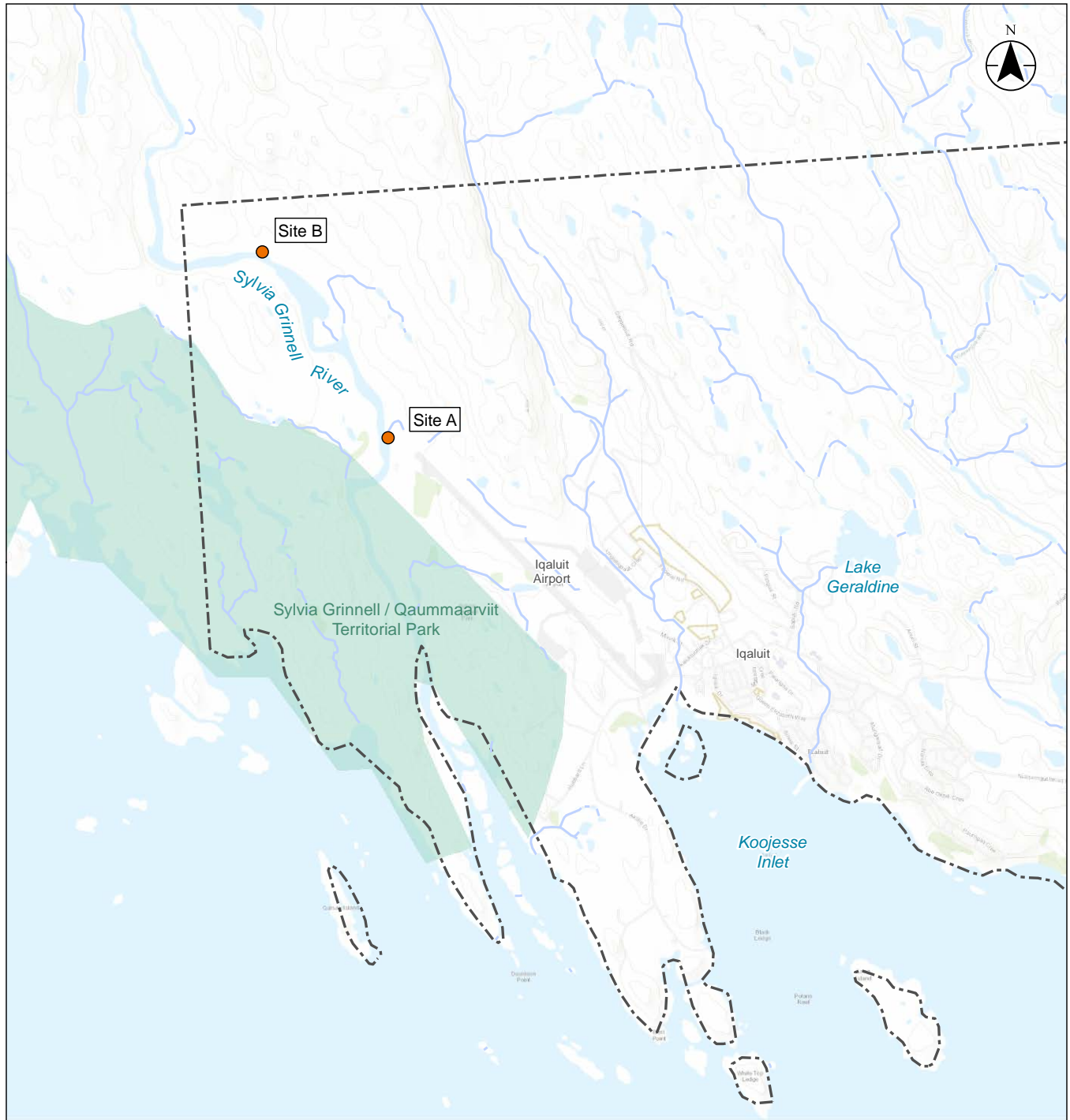
Sources: Base Data - Government of Canada; Thematic Data - Stantec Ltd.
Service Layer Credits: © 2018 Microsoft Corporation Earthstar Geographics. SIO

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Sylvia Grinnell River Watershed

2 SYLVIA GRINNELL RIVER INTAKE SITE OVERVIEWS

Nunami's 2018 report "Options Evaluation for Raw Water Supplementation from the Sylvia Grinnell River" (Nunami, 2018) identifies five possible intake sites that were compared as possible river intake locations. Following that report, two sites were selected for further evaluation as possible intake locations. The selected sites included Site A and Site B as show in Figure 2-1. Hydraulic modeling was completed for each site to further evaluate their potential to serve as a location of reliable water supply during the annual open water season on the Sylvia Grinnell River. Annual open water season can last up to 5-months each year; however, it can be as short as 3-months. For this reason, we are providing conceptual design for pumping during the worst-case scenario of open water season length.



Notes
 1. Coordinate System: NAD 1983 UTM Zone 19N
 2. Data Sources: Natural Resources Canada; ESRI World Topo Map

- Conceptual Intake Location
- City of Iqaluit
- Sylvia Grinnell / Qaummaarviit Territorial Park

0 500 1,000 1,500 2,000
 1:50,000 (at original document size of 8.5x11)



Project Location: Iqaluit, Nunavut
 Project Number: 14430114
 Prepared by: LTRUDEL on 20190206
 Discipline Review by: DJONES on 20190206

Client/Project/Report:
 City of Iqaluit
 Conceptual Design Advancement for Raw Water
 Supplementation from the Sylvia Grinnell River

Figure No.

2-1

Title

Overview of **Intake** Site Options

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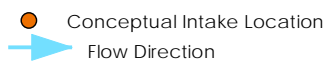
2.1 Site A

Site A (Figure 2-2) is immediately west of the Iqaluit Airport (YFB) and near the northern boundary of the Sylvia Grinnell Territorial Park. Site A is the closest to the City and has the shortest distance for piping, access, power, and communications. Site A is adjacent to the existing Water Survey of Canada (WSC) flow monitoring station 10UH001.

The site is frequented by people travelling upstream from the secondary parking area of the Territorial Park. The site is a deep run following a riffle and located on a gradual bend to the west (see Photo 1 and Photo 2 in Appendix A). The channel is approximately 150 m wide and the river's thalweg is poorly pronounced but running in the middle of the channel. Substrates consist of large coarse material, primarily large cobble and boulders underlain by cobbles and gravels. A bedrock outcrop is present and is the site of the WSC hydrometric station 10UH001. The outcrop protrudes into the flow path slightly and it was noted that this protrusion was not enough to induce scour in the bed substrate. Erosion of the left bank is minimized by the presence of the outcrop. The shallow profile and presence of the point bar downstream suggests that ice jamming here is possible and ice floes appear to dominate geomorphic processes. Scarring from ice movement was observed along the banks. Frazil ice is likely present here during freeze-up, though the upstream riffles likely freeze to bed early in the season and are likely not a major generator of that frazil.

The site is sufficiently upstream to not be affected by rising tides of Koojesse Inlet and salt intrusion into the raw water supply is not a concern at this most downstream site. The site is upstream of the ditch that drains from the airstrip to the east, but it is possible that runoff from the north side of the airstrip enters the river. A review of runoff and water quality may be warranted should Site A proceed to engineering design.

The channel and hydraulic conditions at the site will be a challenge for an intake type that requires deep water. At the time of the site visit there was an average of 1 m of water depth with 1.5 m depth observed in select locations around the WSC station. The site is on a gradual outside bend and it is expected that some increased depth could be achieved with river training or encroachment into the main channel conveyance path, but the evidence provided by the bedrock outcrop suggests the potential to achieve deep scour is low.



Notes
1. Coordinate System: NAD 1983 UTM Zone 19N
2. Data Sources: Natural Resources Canada; Microsoft Bing
imagery



Project Location Iqaluit, Nunavut Project Number 14430114
Prepared by LTRUDELL on 20190206
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Figure No.

2-2

Site Intake Option A Detail

2.1.1 Fish and Fish Habitat

The Sylvia Grinnell River supports an anadromous population of Arctic char (Gallagher and Dick 2010). Arctic char (*Salvelinus alpinus*) exhibit anadromous and freshwater resident populations and are found in rivers, lakes, estuaries, and marine environments throughout their life cycle (Evan et al. 2002). Anadromous Arctic char that are part of the fishery in the area migrate to marine waters during the summer to feed and return to freshwater habitats to overwinter and spawn. Arctic char use the Sylvia Grinnell River for migration, overwintering, spawning and rearing; however, it is believed that most char likely overwinter and spawn in Sylvia Grinnell Lake. Personal communication with DFO (C. Lewis, 21 August 2017) indicate that large deep pools in the Sylvia Grinnell River are also used for overwintering.

Field observations of fish habitat at Site A were completed in 2018 as reported in Nunami Stantec (2018). Site A has a riffle flowing into a deep run associated with backwater formed from gravel deposits upstream of a bedrock intrusion on the east bank. The river channel is approximately 150 m wide at the bedrock intrusion. Water depth is primarily less than 1 m, with depths exceeding 1 m in the backwater and likely in locations in the main channel. Substrates consist of large coarse material, primarily large cobble and boulders with bedrock intrusions. Substrates in the backwater had a deposition layer of fine sediment. Deep backwater pool habitat along the east bank, upstream of the WSC hydrometric station may provide habitat for downstream migrating young Arctic char and the backwater pool may provide rearing habitat for Arctic char and habitat for small fish, such as stickleback species. Site A is unlikely to provide overwintering habitat to adult Arctic char due to shallow water depths.

2.2 Site B

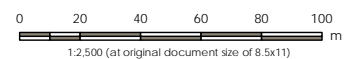
Field observations of fish habitat at Site B were completed in 2018 as reported in Nunami Stantec (2018). Site B is a run located on a pronounced outside bend of the river channel. The left (north) bank at Site B is actively eroding and would require stabilization for any infrastructure placed at this site. A bedrock outcrop is present downstream, and that outcrop runs under the thalweg at a shallow depth. It is anticipated that the outcrop provides some limit to the progression of erosion at left bank. Bed substrates consist of cobble and large gravel with the occasional boulder. The high banks are comprised of loose gravel and sand and the extent of the bedrock in the left bank is unclear. The river's thalweg is moderately pronounced and runs near the outside of the bend (see Photo 3 and Photo 4 in Appendix A). At the time of the site visit, a depth of 1.5 m was measured near the thalweg. Its potential depth may also be limited by the downstream bedrock. The channel is 130 m wide but the shallow slip-off slope on the inside of the bend (right [south] bank) suggests flood flow is not confined at this location and may further limit the deepening of the thalweg by existing processes. The profile of the bend suggests the thalweg may never run up against the toe of the bank and the observed depth of the thalweg may be at its limit, without the addition of training structures.

Ice floes at Site B likely hang-up on the bedrock outcrop during break-up and could induce jamming at this site. The lack of confinement from the shallow slip-off slope on the right side of the channel, and the evidence of a historic avulsion route through the slip-off, suggests flow through an ice jam may splay out across the larger active channel, rather than concentrate up against the left bank. The rapids upstream of Site B run intermittently over a 600 m stretch and if they maintain open water leads then they could be a significant source of frazil ice generation for this site during freeze-up. Site B is shown in Figure 2-3.



Notes
 1. Coordinate System: NAD 1983 UTM Zone 19N
 2. Data Sources: Natural Resources Canada; Microsoft Bing Imagery

● Conceptual Intake Location
 ➡ Flow Direction



Project Location
 Iqaluit, Nunavut



Project Number 14430114
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City of Iqaluit
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 Supplementation from the Sylvia Grinnell River

Figure No.

2-3

Title

Site Intake Option B Detail

2.2.1 Fish and Fish Habitat

As with site A, the Site B fish habitat is discussed in relation to the predominant fish species noted in the Sylvia Grinnell; the Arctic char.

Site B is run habitat along the east bank, turning into a riffle, then a rapid formed at the downstream end where a bedrock intrusion narrows the channel. The channel is approximately 130 m wide. Along the middle and upstream portions of the east bank, substrates consist of cobble and large gravel, with high loose gravel banks. Occasional boulders provide instream cover. Site B is unlikely to provide overwintering habitat due to low water depth, nor act as a holding pool for upstream migratory Arctic char. Nearshore habitat would provide cover for downstream moving fish, but rearing habitat is poor because of the absence of cover.

2.3 Hydrology – Site A and Site B

Key hydrologic characteristics of Sites A and B were detailed in Nunami (2018). The data used to estimate monthly river flow rates at each site was taken from WSC flow monitoring station 10UH001 (Sylvia Grinnell River Near Iqaluit). Although it is typical to scale the river flows reported by WSC station data to a site based on drainage area, the discharges through Sites A and B were assumed to be equal to the discharge at the WSC station for two reasons. First, Station 10UH001 is located at proposed intake location Site A so the WSC data is reflective of flows through the Site A reach. Secondly, Site B is only 2 km (as the river flows) upstream of Site A, and the differences in drainage areas between Sites A and B were deemed too small to warrant modifications to estimate flows at Site B. The following provides a summary of the site hydrology and design flows through the reach of each proposed intake site.

2.3.1 Monthly Flows

Open water flows through the lower reaches of the Sylvia Grinnell are governed by the break-up and freeze-up cycles each year. Historical records from WSC flow monitoring station 10UH001 show that the open water season typically spans from the end of May to the end of October each year. To evaluate the feasibility of an intake for each proposed site, Nunami used the estimated mean monthly flows for June to October as the basis for the hydraulic modeling of each proposed intake reach. Tables 2-1 and 2-2 show the WSC reported minimums and mean monthly flows for each month. These flows became the design basis for the intake's seasonal serviceability.

Table 2-1: Sylvia Grinnell River Minimum Monthly Flows (Source: WSC)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Discharge (m ³ /s)	0	0	0	0	0	0	11.6	17.6	12.7	1.45	0	0

Table 2-2: Sylvia Grinnell River Mean Monthly Flows (Source: WSC)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Discharge (m ³ /s)	0.7	0.2	0.1	0.0	0.7	100.4	149.3	69.6	50.0	19.1	5.5	1.9

2.3.2 Regulatory Flow Limits

DFO (2013a) guidelines recommend that water withdrawal rates from a fish bearing watercourse are limited to no more than 10% of instantaneous flow and that withdrawals are limited to times when flows are greater than 30% of the mean annual discharge (MAD); which is approximately 10.11 m³/s at the Sylvia Grinnell River (Nunami, 2018). These guidelines are provided to limit the possibility of serious harm to fish.

2.3.3 Flood Flows

To inform the general arrangements of the infrastructure required at each site and the design for flood resiliency, the 5-year and 350-year return period flood flows were modeled at each proposed intake site (see Table 2-3). The flow rates for each return period flow were estimated from WSC flow data for the Sylvia Grinnell as discussed in Nunami (2018).

Table 2-3: Sylvia Grinnell River Design Flood Flows

Return Period	Discharge (m ³ /s)
5-Year	513
350-Year	757

2.3.4 Withdrawal Capacity

Golder (2013) reported the annual peak freshwater volume to needed meet future water needs for the city of Iqaluit to be 1,853,000 m³ under a combined climate change and projected population growth scenario (Golder, 2013).

The hydrometric records, and observations on site, suggest the Silvia Grinnell freezes to its bed in winter and any flow that is present would be shallow, and splayed out through its flat, broad channel bottom. For this reason, a year-round withdrawal scheme was deemed not feasible and the design basis was to focus on withdrawal during the open water period (Nunami, 2018).

Open water flows through the intake sites typically occur from June through October each year. WSC records show that break-up and freeze-up can vary by up to a month earlier or later in the year. Several years on record show open water periods of only 3 to 4 months. Years with shorter open water periods would likely strain the Lake Geraldine water supply and, in these years, it would be important to replenish the reservoir rapidly. Our design basis assumed a minimum open water withdrawal window of 3-months; spanning July, August, and September of each year. To meet the City's current and future water needs (1.8 Million m³) through a 3-month supplementation window, a withdrawal rate of 233 L/s is needed (Nunami, 2018). In consideration of a potentially wider withdrawal window, the advancement of the intake concept

designs for each site considered modeled water levels at mean monthly flows, minimum monthly flows, and recommended minimum withdrawal flow limits over 5-months spanning June through October.

2.3.4.1 Intake Screen Requirements

DFO has published a freshwater fish screen guideline to assist with intake design and help protect fish against impingement or entrainment into an intake system where freshwater is being withdrawn from fish-bearing waters (DFO, 1995). The Sylvia Grinnell hosts a seasonal population of Arctic Char which migrate upstream and downstream during their annual spawning period. We have assumed that the Arctic Char are the design species for the purposes of fish screening concept at the proposed intake sites.

The Arctic Char is a subcarangiform fish (fish that swim like salmon or trout). The design approach velocity (V) at the surface of a fish screen for subcarangiform type fish is 0.11 m/s and the design withdrawal rate (Q) is 0.233 m³/s (233 L/s). Using the relationship that $Q = \text{Velocity} \times \text{Area}$, the required open area for the fish screen is about 2.2 m². As fish screens are not 100% open area, the total required screen area to achieve 2.2 m² of open area is determined by the following:

$$A_{\text{screen}} = A_{\text{open}} / \%_{\text{eff}} ; \text{ where}$$

$$A_{\text{screen}} = \text{total screen area}$$

$$A_{\text{open}} = \text{required open area of the screen}$$

$$\%_{\text{eff}} = \% \text{ open area of the screen}$$

Common screen types have an open area that ranges from 51% to 69%. Percent open area of screen depends on the material and screen type chosen. Using these ranges, the total intake screen area will need to be 3.2 m² - 4.3 m² in size to both protect fish and maintain the desired withdrawal rate of 233 L/s. The conceptual designs in this report assume a total screen area of 4 m².

3 SITE SELECTION AND CONCEPTUAL DESIGN ADVANCEMENT

3.1 Basis for Assessment and Conceptual Design

The advancement of the conceptual designs for the raw water intakes and conveyance pipelines were based on the following:

- A site visit by David Luzi, Paul Harper, and Matt Wood, of Nunami, to:
 - The study area over the period August 21 to 23, 2017
 - The potential pipeline routes on August 22, 2017, plus discussions with staff members at the Government of Nunavut, Department of Environment, Fisheries and Sealing Division, and the local DFO office on the same day
 - The Lake Geraldine reservoir and water treatment plant on August 23, 2017, and discussions with City staff members (Mike Hatfield, Bob Brouillet, and Maria Karveli) on the same day;
- Regional LiDAR based digital elevation model (DEM) provided by City of Iqaluit;
- A total required withdrawal volume of 1,853,000 m³ to address projected water supply shortages under climate change and projected population growth scenarios (Golder, 2013)
 - assumed minimum 3-month pumping regime for a design peak withdrawal rate of 233 L/s
- Site A and Site B bathymetry collected via Z-boat bathymetric survey by Stantec on September 22 and 24, 2018;
- Frontier Geosciences Inc. Seismic Refraction Survey, September 27 to October 1, 2018.
- Nunami report, "Options Evaluation for Raw Water Supplementation from the Sylvia Grinnell River", April 2018
- Assumed maximum 5-month pumping regime with a design withdrawal peak rate of 233 L/s

Site specific hydraulic modeling, and the results from the Frontier Geosciences Seismic Refraction Survey Report (Frontier GeoSciences Inc., 2018; Appendix E) were used to inform the advancement of the intake concepts for Sites A and B. One-dimensional flow models were constructed for each proposed intake location using HEC-RAS² modeling software. For the purpose of modeling, cross sections from each site were reconstructed using a combination of the site bathymetry for the main channel and LiDAR based digital elevation model (DEM) for the overbank regions. Due to discrepancies in reference elevations between the DEM (from City of Iqaluit) and site bathymetry (from Stantec), the DEM, geophysical survey data, and bathymetric model had to be manually rectified in CAD prior to running the hydraulic models. As a result, the elevations associated with the concept designs are relative only and do not necessarily reflect the actual elevations at site. Additional survey would be required prior to detailed design to confirm site elevations in relation to a local monument or benchmark. Figures 3-1 and 3-2 show the modeled intake reaches and extracted hydraulic cross-sections in relation to the geophysical survey for each site.

² U.S. Army Corps of Engineers. HEC-RAS River Analysis System, Version 5.0. February 2016

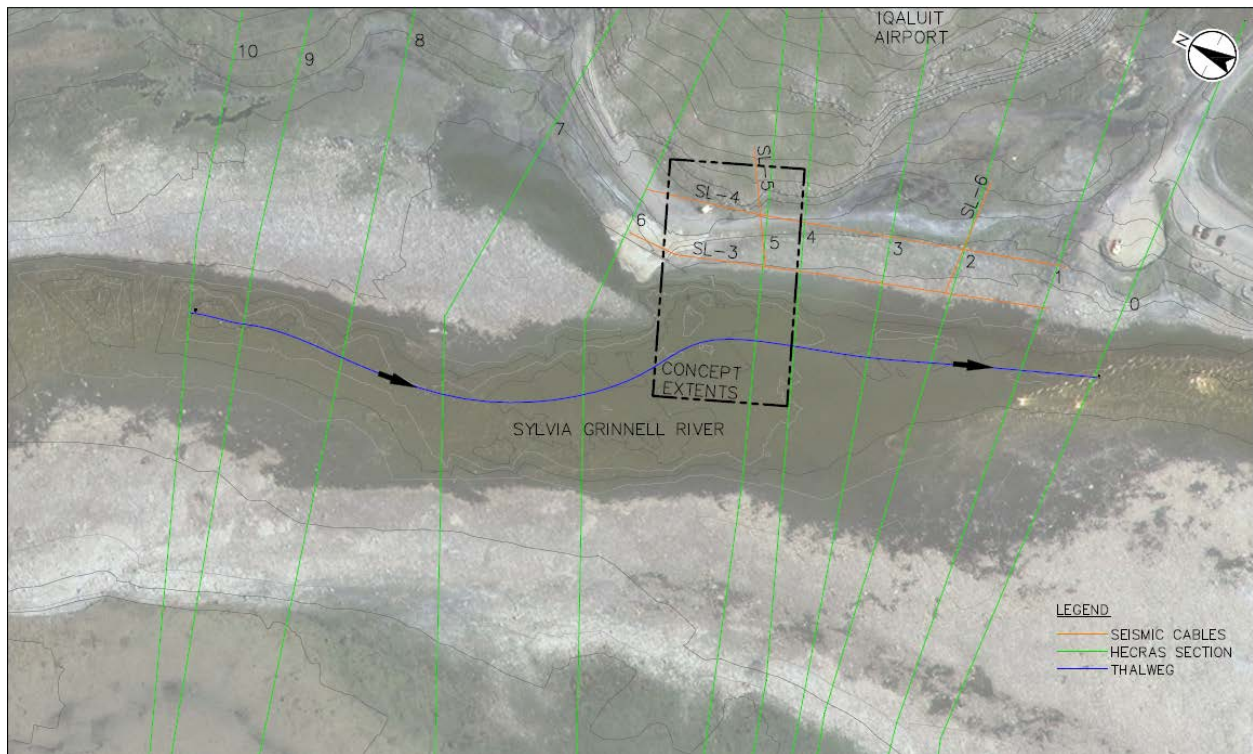


Figure 3-1: Site A - Hydraulic Model Cross Sections

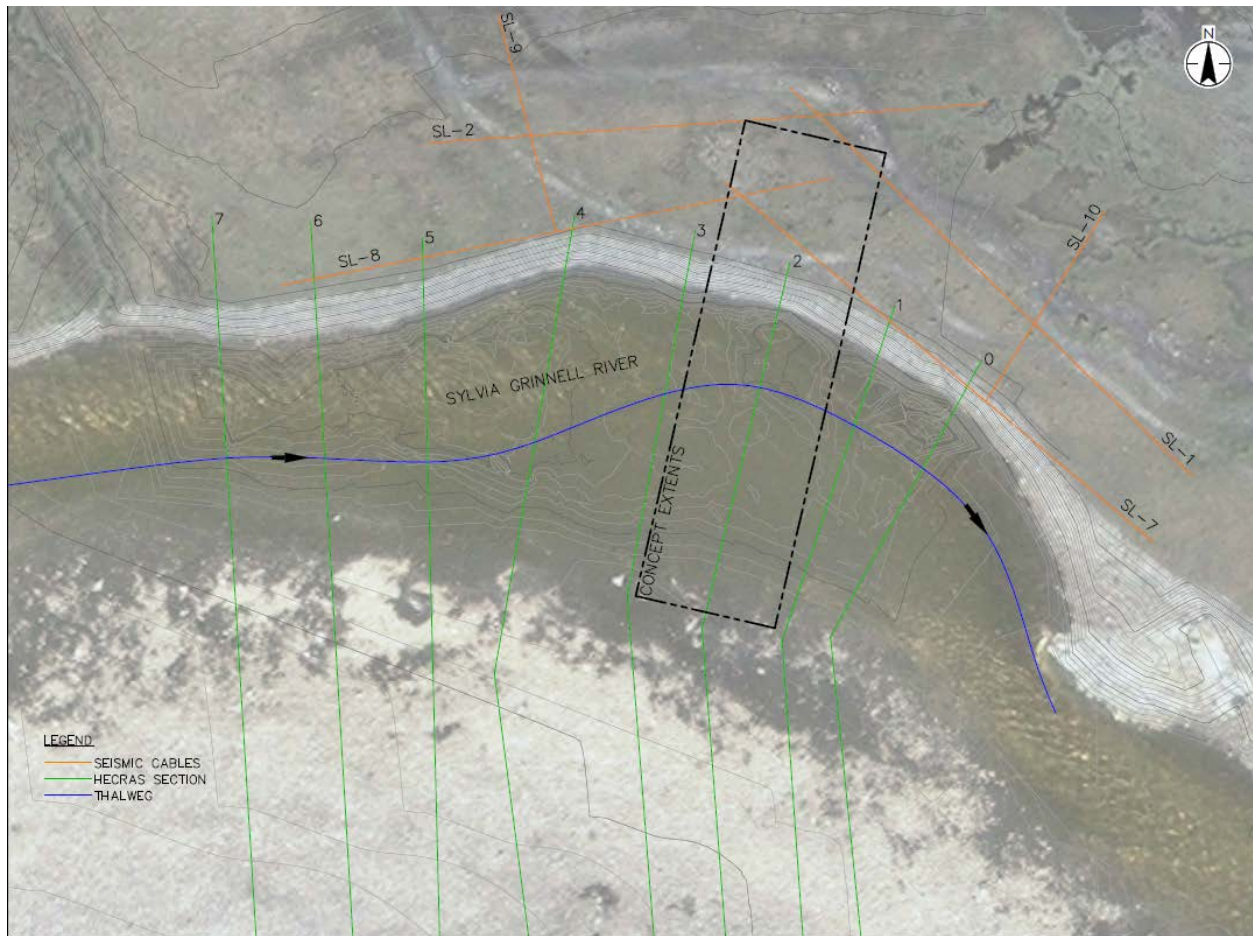


Figure 3-2: Site B - Hydraulic Model Cross Sections

3.2 Site A Intake and Pipeline Routing

3.2.1 Site A - River Hydraulics

To model a hydraulic section, a roughness coefficient (Manning's 'n') must be applied to the main channel and overbank regions of the watercourse. At Site A, the channel is characterized by a large cobbles and boulders with a more pronounced presence of gravels and some bedrock outcropping along the banks. The Manning's 'n' values were selected according to recommendations provide by Chow (1959) and with reference to United States Geological Survey's (USGS) "Verified Roughness Characteristics of Natural Channels" (USGS, 2019). The channel through Site A was assumed as "clean and winding, some pools and shoals" (Chow, 1959).

The slope through the proposed Site A intake reach was estimated from a profile section through the reach that was extended both upstream and downstream of Site A. Table 3-1 summarizes the model inputs for Site A.

Table 3-1: Site A River Hydraulic Model Inputs

Manning's 'n'			Channel Slope (m/m)
Left Overbank	Channel	Right Overbank	
0.045	0.040	0.045	0.0029

Hydraulic models were run for mean and minimum monthly flows, the low-flow limit recommended by DFO guidelines and as reported in Nunami (2018), and for the 5-year and 350-year return period flood flows. Table 3-2 below summarizes the hydraulic modeling results. The DFO low-flow limit for withdrawals (i.e. 30% mean annual discharge) was the limiting low flow condition during the proposed withdrawal window. The concepts at each site were advanced with the goal of being able to withdraw down to the low flow limit in the river.

The 350-return period flow estimate was considered the design flood for resiliency as noted in Nunami (2018).

Table 3-2: Site A River Hydraulic Model Results

Flow Condition	Discharge (m ³ /s)	Max Depth (m)	Hydraulic Depth in the Thalweg (m)	Mean Channel Velocity (m/s)
Mean June	100	2.08	1.66	0.71
Mean July	149	2.27	1.85	0.91
Mean August	69.6	2.01	1.59	0.56
Mean September	50.0	1.95	1.53	0.42
Mean October	19.1	1.55	1.18	0.22
DFO Low Flow	10.1	1.31	1.01	0.14
5 Year Flood	513	3.69	2.97	2.01
350 Year Flood	757	4.02	3.3	2.51

3.2.2 Site A Geophysical Characteristics

Frontier Geosciences Inc. completed a geophysical survey of the proposed intake sites in the fall of 2018 utilizing seismic refraction. The seismic refraction survey provided an approximation of the overburden layering at each site and depth to bedrock.

A total of five seismic refraction traverses were completed at Site A. The seismic refraction survey consisted of laying out seismic cables with geophones located at regular intervals along each traverse. Seismic energy, provided by firing 8-gauge blank shotgun shells or sledgehammer striking a steel plate, was utilized at up to seven (7) locations for each traverse; one at each end of the geophone array, two at intermediate locations and one off each end of the traverse to ensure adequate coverage of the basal layer. The arrival times were recorded digitally using a seismograph for each geophone. The estimated extents of the overburden and depth to bedrock were used to inform our intake type selections. As an example, shallow bedrock could limit the potential for a wet well style intake. The installation of a well and supply pipe coming from an intake would require excavation and/or drilling and shallow bedrock can make these processes expensive and time consuming.

Based on the seismic refraction results, Site A was characterized by three distinct velocity layers. Extending from the ground surface, survey results suggested a surficial layer of unconsolidated sands and gravel overlying a zone of saturated sands and gravels. Below this, the geophysical survey results suggested a layer of compacted overburden with a high content of coarse material (glacial till) likely progressing to discontinuous permafrost and/or weathered bedrock. A distinct signal consistent with competent bedrock was noted across Site A at depths of 1.7 m to 7.5 m below the existing grade.

The Site A concept drawing as shown in figure B1, Appendix E, shows the reported bedrock surface as a solid purple line (layer "L4"). Near the bank of the Sylvia Grinnell at the proposed intake, the bedrock layer appeared to dip to approximately 4 m from the existing grade. To advance the intake concept at this site, an assumed bedrock profile was projected to the main channel of the Sylvia Grinnell and is represented by the dashed purple line extending to the thalweg at Site A.

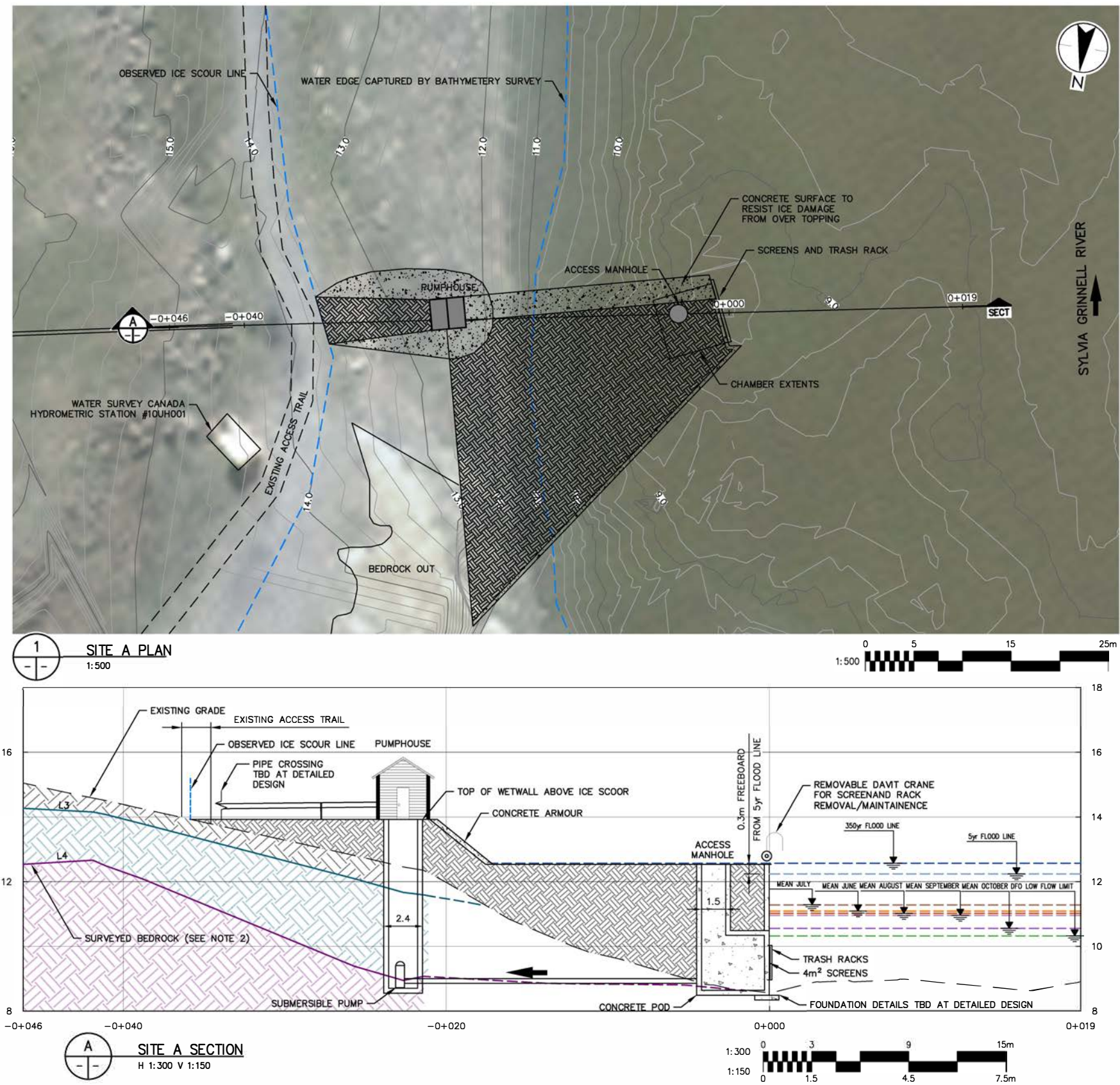
The actual extents of bedrock, both at the bank and within the channel, should be confirmed using a borehole geotechnical investigation program. Relevant geotechnical investigations should be completed prior to, or in conjunction with, preliminary design.

3.2.3 Site A - Intake Type Selection

At Site A, the projection of the bedrock into the channel suggests that the bedrock might daylight within the main channel. As noted, boreholes in the channel would be required to confirm bedrock depth as part of preliminary engineering. The projection of the geophysical findings suggests that excavation for a bed infiltration gallery may not be practical and limits the feasibility of a bed infiltration gallery at Site A. Bed infiltration galleries also have serviceability and maintenance issues due to clogging and it is not common for large municipalities to use infiltration galleries style intakes for this reason. The use of a bed infiltration gallery was eliminated from further consideration at Site A.

The recommended intake concept type Site A consists of a protruding intake structure, wet well, and separate instrumentation and control building. Figure 3-3 illustrates the Site A intake design concept. The rationale for this concept is discussed below and considered the river hydraulic model results for low flows and flood flows, the reported bedrock depth at this location, and the estimated extents of ice scour along the bank at Site A.

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- EXISTING GRADE
- SURVEYED SURFICIAL MATERIAL
- L3 - SURVEYED COARSE MATERIAL/GLACIAL TILL OR BROKEN WEATHERED BEDROCK
- ASSUMED L3
- L4 - SURVEYED BEDROCK SURFACE
- ASSUMED L4

- 1) Reported elevations are relative and may not represent precise elevations at the site; LiDAR and Seismic data were raised or lowered to match Bathymetry survey, completed by Nunami.
- 2) As per "Seismic Refraction Survey Report Iqaluit Water Intake Project" by Frontier Geosciences Inc. dated November 2018
- 3) Pumphouse building to be placed above both design flood and observed ice scour elevations.

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City of Iqaluit

Sylvia Grinnell River Intake

Project No.

144930114

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3-3

3.2.4 Site A Intake Concept Advancement

3.2.4.1 General Arrangement

The Site A concept proposes a protruding wall structure be built into the river channel. The concept assumes the wall to be concrete, but the wall could be formed of sheet piles or alternate materials that can resist ice forces and which are compatible with a suitable foundation for the structure. Selection of the wall material would be dependent upon geotechnical investigation and foundation design as part of preliminary engineering.

The face of the protruding wall structure intersects a deeper run of the river close to the left bank and which is part of the channel's thalweg. The structure protrudes such that it promotes scour off its screen face during flood and ice break-up events. This scour is desirable as it promotes self-cleaning of the screens and limits the potential for blockage by sediment. The upstream face of the protruding wall structure is angled to promote the shedding of ice floes during break-up and helps to limit the forces that can push on the structure, should an ice jam form. Because of the bank geometry, the protruding wall structure's height is limited by what is practical to fill with connection to the shore. Given the concept geometry, the protruding wall structure would be overtopped in floods greater than a 5-year event and may be overtopped during ice break-up.

The Site A concept has the pump house and wet well separated from the intake screen chamber. This separation is a consequence of the bank geometry, and the ice and flood hydraulics. These have been arranged in consideration of access and serviceability described further in Sections 4.1.4.2 and 4.1.4.3.

The intake chamber would be incorporated into wall structure with maintenance access provided by a manhole. The intake chamber is fitted with two (2), 2 m x 1 m intake screens for a total screen area of 4 m². The actual dimensions will be dependent upon the available opening area of the screens as selected as part of detailed design. The screens, trash rack, and any protective plates could be raised and lowered along slotted guides using the davit shown on the concept drawings.

The wet well uses a standard manhole and was positioned where, according to the geophysical survey, the bottom of the wet well could reach enough depth to permit a gravity fed connection between the intake pod and the wet well, without significant intrusion into the bedrock. A pipe from the intake pod feeds water from the screen chamber to the wet well by gravity where it would be pumped to the first booster station location. Physical confirmation of the extents of the bedrock elevation would need to be made through a geotechnical field investigation (e.g. borehole drilling program) as part of preliminary design.

The Site A concept proposes a submersible pump. A submersible pump would require a simpler building layout, would be less prone to priming and/or cavitation issues, and is simple to install and remove given the proposed seasonal pumping regime when compared to alternative pump types. The submersible pump would be lowered into the wet well and secured to the discharge pipe seasonally. Some submersible pumps have the option of being installed on a rail guide with a self-latching mechanism to secure the pump discharge to the discharge pipe. This latching mechanism can also be disengaged using a chain link or similar connection so that the pump can be disengaged remotely and removed for maintenance or storage.

At the end of each annual withdrawal window the pump would be removed from the wet well for winterization and protection against ice that might form in the well.

The pumphouse, including instrumentation, was placed above an ice scar line that was observed along the right bank in August 2018. This building would house the electrical and communication components and could serve as storage for items like the screens and submersible pump.

3.2.4.2 Access

Site access would come from the south via an access road. The concept presented in this report puts the instrumentation building adjacent to the existing trail that appears to be providing access to the WSC station and informal ATV access up the Sylvia Grinnell River. The access trail should be upgraded to ensure the required operations and maintenance traffic can reach the intake and pumphouse with room to maneuver to complete any maintenance or equipment replacement. The estimate of probable cost includes a rough cost for required access trails. It should be noted that this cost will be refined as design advances.

Once at the intake site, the pumphouse and wet well, and intake pod would be accessed via appropriately sloped surface as noted in the concept drawings. The wet well and pumphouse have been raised above the observed ice scar to avoid ice damage.

3.2.4.3 Resiliency and Serviceability

The current Site A concept was designed to provide resilience in operability to low flow periods while resisting damage from flood events and ice floes. The low flow limit was assumed to be equal to the DFO low flow limit presented in Nunami (2018); 10.1 m³/s. As noted in the concept drawing (Figure 3-3), the intake screen was positioned in a run adjacent to the left bank at Site A. According to the hydraulic model, the placement of the intake screens in this area would allow for withdrawals at the shoulders of the open water season (June and October), if conditions permit, and would also allow withdrawals to the proposed DFO low flow limit. Based on the current model, the intake's 1 m tall screens would remain fully submerged to the DFO low flow limit.

The estimated 5-year and 350-year return period flood were used as the design basis for flood events. The protruding wall structure lies 300 mm above the 5-year return period flood to avoid frequent inundation with sediment laden flood waters. This design elevation for the protruding wall is at the estimated 350-year flood water elevation and is unlikely to be fully overtopped given a 350-year flood flow on the Sylvia Grinnell. It appears that the main risk to the structure is posed by ice.

During Nunami's field visit in 2018, an ice scar was observed along the left bank at Site A and the approximate location of this ice scar is shown in the Site A concept drawings. This ice scar suggests that ice could overtop the protruding wall structure during break-up, the concept does not have a fully hardened surface but includes provision for some extra concrete surfacing at interfaces that could be damaged by the overtopping ice. The protruding wall structure, and its foundation, would need to be designed to resist ice forces as part of preliminary engineering. The concept includes some concrete armor around the raised wet well structure to prevent its damage from overtopping ice.

To account for the potential impact of ice floes, the instrumentation building has been positioned at a higher elevation to limit the likelihood of ice impacting and damaging the electrical equipment needed to operate the proposed pump station.

3.2.4.4 Maintenance

The intake concept for Site A included maintenance considerations related to screen clogging, sediment accumulation in the wet well, seasonal operation, and preliminary measures to protect against ice floes or ice jams that might occur at the intake site.

It is likely that some sediment will build-up in the intake chamber and wet well through operation during periods of high turbidity. Sediment can be readily removed from the intake chamber using a hydrovac unit, but the wet well depth must be shallow enough for a hydrovac to suck up the sediment to be removed.

The screen face and trash racks could be designed with some provision for ice resistance but there would be residual risk of damage over winter. The City may wish to remove the screens at the end of the withdrawal season, prior to freeze-up and replace them with solid steel plates. Removing the screens would prevent them from being damaged by ice, and the steel plates would provide some protection to the structure and the intake chamber. The Site A concept includes a removable davit for screen and trash rack removal. The davit also facilitates the removal of the screens for cleaning as part of regular maintenance. Regular cleaning of sediment, debris and biological fouling will help to maintain the flow rate across the screen and maintain overall system performance.

In the Site A general arrangement, access to the screens may not be available during ice break-up. Should the City wish to pursue Site A with provision to begin withdrawal during ice break-up, then consideration for leaving the screens in throughout winter must be included in preliminary engineering design.

Pushing operations into spring or fall open water shoulder seasons could increase the chances of frazil ice build-up on the intake screens. Frazil ice can form when temperatures suddenly drop, and supercooled water nucleates into ice that adheres to conductive surfaces, like the metal on a water intake. The build-up of frazil ice on the intake screen could reduce its performance significantly. To limit the potential impacts of frazil ice, the protruding wall intake could be designed with an air bubbler or air scour system to reduce the likelihood of frazil ice build-up on the intake screens. Teflon coated screens and trash racks can also reduce frazil build-up. Since withdrawal is planned for the open water season, provision for frazil management has not been included in the concept, but mitigation measures like those mentioned above, could be further considered in preliminary engineering.

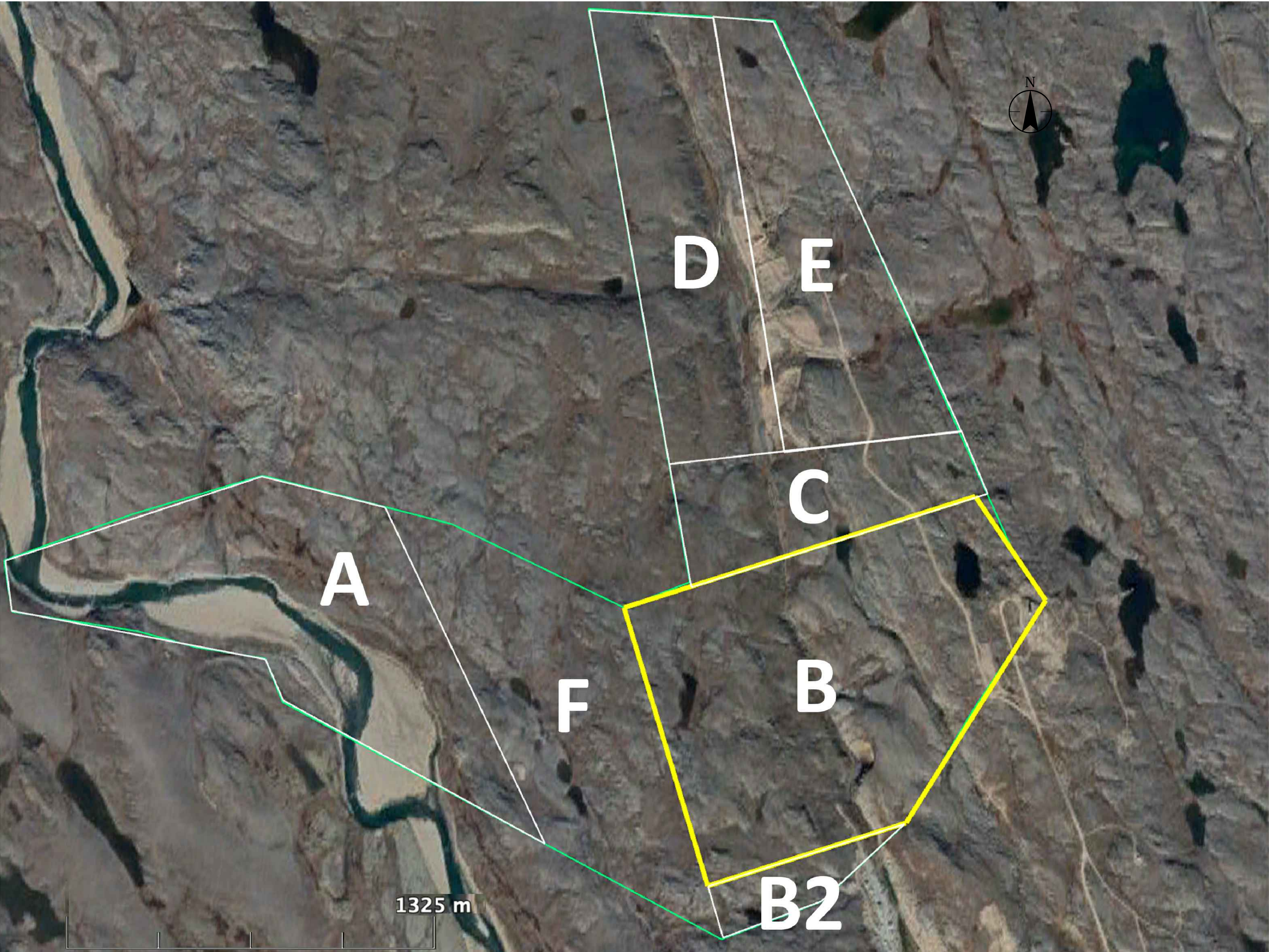
3.2.5 Site A Pipeline Concept Advancement

As mentioned in the intake descriptions, Site A has partial road and power access. Following the lidar topographic survey (partially completed, to date), one optimal route from Intake Site A to Lake Geraldine was selected. The complete survey sections and the missing section are shown in Figure 3-4 (Drawing C-001). Route A runs from Sylvia Grinnell River overland past the north side of the runway. From here, the route runs south past the east side of the runway and bordering the airport lands between the asphalt plant and the quarry. Where applicable, the pipeline has been offset from the river to accommodate the 30 m federal reserve on navigable waters.

Figure 3-5 (drawing C-100) shows a plan view of Route A from intake to discharge. The 500 mm diameter pipeline will span a total length of 4,443 m across craggy terrain to the discharge location at Lake Geraldine Headwater Creek (LGHC). High and low points can be smoothed with a cut and fill plan during detailed design, however, not eliminated. As a result, staged pumping and drains will be required to ensure adequate pressure and relief can be maintained. The elevation gain from Site A to LGHC is 125 m.

Running north of the airport lands, this option is easily accessed through Kudlik Construction Ltd. (Kudlik) yards and along access roads north of Federal Road. The pipeline continues along Qaqqamiut Street toward Upper Base and behind the residential housing on the Plateau toward LGHC. A service road or trail will be required in sections between existing road sections and from the intake site past the eastern side of the runway and as the pipeline approaches LGHC. Approximately 3.1 km of trail could be required.

Profiles for the proposed Route A are shown in Figures 3-6 (drawing C-101) and 3-7 (drawing C-102). Here, estimated locations for pumps, drains, and road and ditch crossings. Additionally, Nunami is recommending a portion of the pipeline be buried between stations 23+00 and 29+00 (Kudlik and City yards and new Waste Transfer Station site) to avoid interference with activities in this part of town and to offer additional protection to the pipe material.

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LIDAR SURVEY AREAS LOCATION PLAN

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3.3 Site B Intake and Pipeline Routing

3.3.1 Site B - Intake Hydraulics

Site B was modelled in the same manner as Site A. Table 3-3 outlines the hydraulic model input for the Site B analysis.

Table 3-3: Site B Hydraulic Model Inputs

Manning's 'n'			Channel Slope (m/m)
Left Overbank	Channel	Right Overbank	
0.045	0.040	0.045	0.0027

As with site A, site B modeled flow regimes included mean and minimum monthly flows, and the DFO recommended low flow limit for withdrawals. Table 3-4 summarizes the hydraulic modeling results. The DFO low flow limit was also the limiting flow conditions for the concept design advancement at Site B.

Table 3-4: Site B Hydraulic Model Results

Flow Condition	Discharge (m ³ /s)	Max Depth (m)	Hydraulic Depth (m)	Mean Channel Velocity (m/s)
Mean June	100	2.34	1.51	1.09
Mean July	149	2.54	1.71	1.35
Mean August	69.6	2.13	1.30	0.91
Mean September	50.0	1.94	1.11	0.79
Mean October	19.1	1.49	0.76	0.49
DFO Low Flow	10.1	1.30	0.60	0.35
5 Year Flood	513	3.50	2.67	2.08
350 Year Flood	757	3.90	3.04	2.41

3.3.2 Site B - Geophysical Characteristics

A total of six seismic refraction traverses were completed at Site B. The seismic refraction survey was completed in accordance with the method previously described, and the full report is provided in Appendix E.

It should be noted that the six seismic refraction traverses were completed on the ridge at Site B, both above and not immediately adjacent to the river.

Based on the results of the seismic refraction survey, Site B was characterized by up to four distinct velocity layers. Extending from the ground surface the survey results suggested a surficial layer of unconsolidated sands and gravel across the surveyed area. Across the full survey area, with exception of an area along seismic refraction traverse SL-2, velocities consistent with discontinuous permafrost, glacial till or fractured bedrock were measured. This is consistent with site observations of the exposed material within the bluff, which suggest glacial till soils. Along seismic refraction traverses SL-1 and SL-2, a discontinuous

intermediary layer between the surficial sands and gravel and discontinuous permafrost/glacial till/fractured bedrock was identified. The lower layer returned compressional wave velocities consistent with competent bedrock at depths of 2 m to 5.7 m below the existing grade.

The geophysical survey results reported a return signal consistent with a competent bedrock surface 3m to 4 m deep approximately 20 m north of the left bank at the proposed Site B intake location. As this bedrock layer was not noted along the bank at the proposed intake location during Nunami's August 2018 site visit, it was assumed that the bedrock surface dips below the observed water line, or to a shallow depth below the thalweg at Site B and as shown in Figure 3-8.

The reported bedrock surface is shown as a solid purple line in Figure B2, Appendix E. An assumed bedrock profile was projected to the main channel of the Sylvia Grinnell as a part of the conceptual design. The assumed bedrock depth is represented by a dashed purple line. Coarse overburden material is represented by the blue line (surface) and hatching.

The extents of bedrock along the Site B bank and main channel should be confirmed using a borehole geotechnical investigation program. Relevant geotechnical investigations should be completed prior to, or in conjunction with preliminary design.

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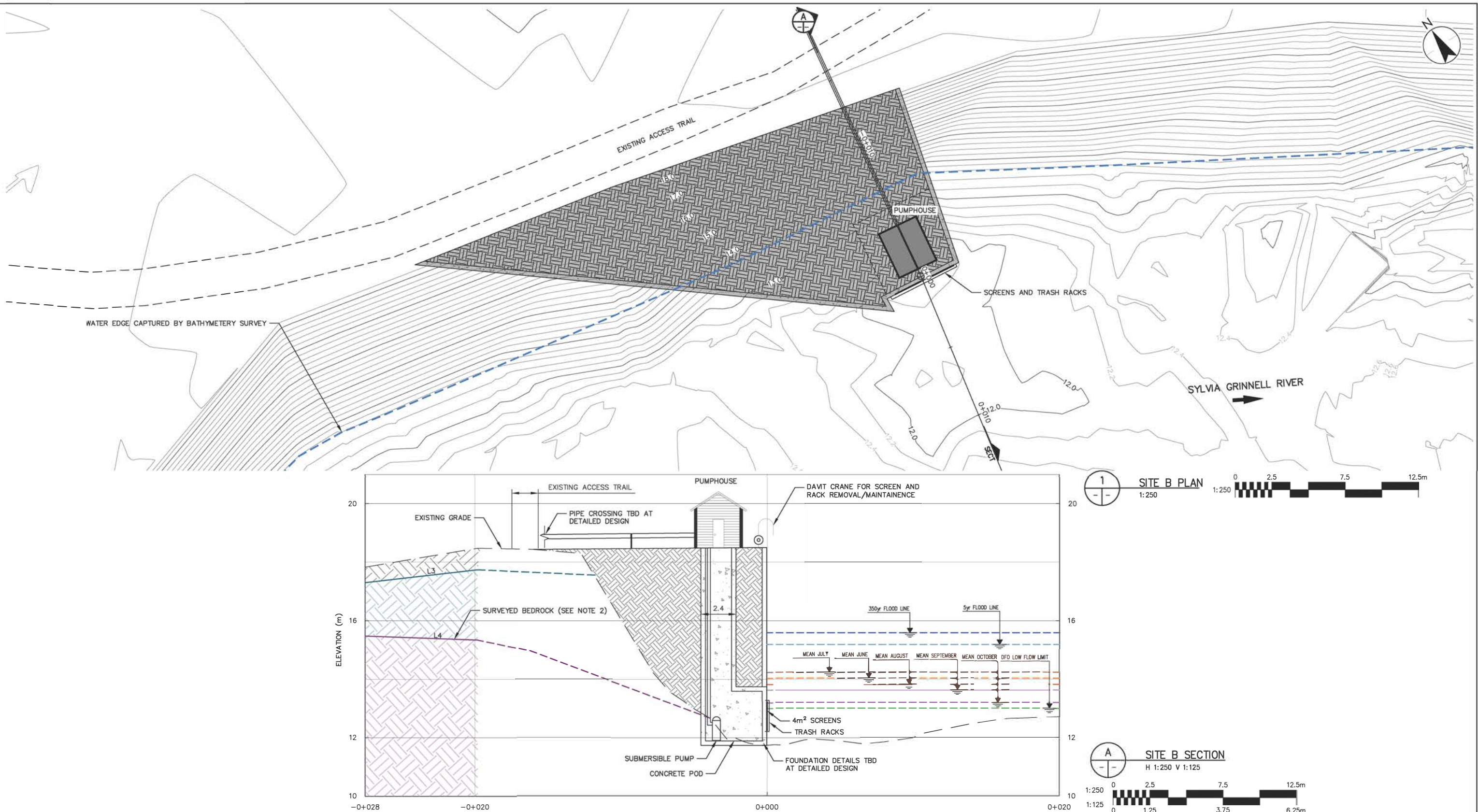


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- SURVEYED SURFICIAL MATERIAL
- L3 - SURVEYED COARSE MATERIAL/GLACIAL TILL OR BROKEN WEATHERED BEDROCK
- ASSUMED L3
- L4 - SURVEYED BEDROCK SURFACE
- ASSUMED L4

- 1) Reported elevations are relative and may not represent precise elevations at the site; LiDAR and Seismic data were raised or lowered to match Bathymetry survey, completed by Nunami.
- 2) As per "Seismic Refraction Survey Report Iqaluit Water Intake Project" by Frontier Geosciences Inc. dated November 2018
- 3) Pumphouse building to be placed above design flood elevation.



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Figure No. 3-8

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3.3.3 Site B - Intake Type Selection

The Site B intake concept considered an intake pod inside a riprap spur as described in Nunami (2018). Concept advancement revealed that riprap may become mobile in flood or dislodged by ice if it is not grouted. The grouting would complicate installation and limit the options for infiltration through the spur. This resulted in a revision in the selected intake type to a protruding wall structure, as with the Site A concept.

The presence of the bedrock in the bank limits the feasibility of a wet well structure within the bank. The protruding wall structure at Site B takes advantage of the high bank and allows the wet well to be located within the structure.

Because of the wet well limitations introduced by the bedrock and the ice resiliency risks associated with a riprap-based structure, the proposed intake type at Site B is a protruding wall structure with its wet well incorporated into the structure. The concept for the selected intake type is provided in Figure 3-8. The rationale for this concept is discussed below.

3.3.4 Site B Intake Concept Advancement

3.3.4.1 General Arrangement

The Site B concept shown in Figure 3-8 involves a protruding bank structure as at Site A concept. The difference in the Site B general arrangement is that the structure takes advantage of the bank's natural geometry and the access platform of the protruding wall structure can be placed well above the estimated flood and ice elevations. As a result, there is no need for separate screen chamber and wet well infrastructure and there is opportunity for the instrumentation building to be placed on top of the wet well to serve as a combined instrumentation and mechanical building. Though not designed for all-season withdrawal, this intake arrangement has improved access to the intake infrastructure during all season when compared to Site A. This could be an important consideration if the City wishes to withdraw during ice break-up.

Like Site A, the Site B concept uses concrete to form the wall of the protruding structure. Pending sub-surface conditions, the wall could be formed of sheet pile, or an alternate material that can resist ice forces. The structure would require a suitable foundation. Selection of the wall material would be dependent upon geotechnical investigation and foundation design as part of preliminary engineering.

The Site B intake chamber would be incorporated into the structure with maintenance access provided by a manhole. The chamber is abutted to the wall and has an opening to the river that is fitted with two (2), 2 m x 1 m intake screens. The screens, trash rack and any protective plates could be raised and lowered along slotted guides using the davit shown on the concept drawings.

The Site B concept also proposes a submersible pump. Pump type alternatives were considered as part of the concept design advancement. A submersible style pump was deemed the most appropriate. The proposed depth of the wet well at Site B is a disadvantage for conventional centrifugal skid mounted pumps because the high suction head requirements can create issues with pump priming and increase the chances

for cavitation at the pump's impeller. Cavitation can lead to pump damage and significantly more maintenance.

Although vertical turbine pumps can provide reliable water withdrawals from a wet well, they are difficult to remove where pumping is seasonal and where winterization is needed. Vertical turbine pumps also require a significantly taller building structure, more precise wet well design, a customized hoist system, and/or large overhead doors or roof openings for their removal or access for maintenance. All these factors could increase both the construction and maintenance costs associated with using vertical turbines at either Site A or Site B. Lastly, if vertical turbine pumps were to be left in the wet well year-round without a heating system, ice formation at the pump intake or within one or more of the bowl assemblies could result in damage to the impellers.

The submersible pump would be lowered into the wet well and secured to the discharge pipe seasonally. At the end of each annual withdrawal window the pump would be removed from the wet well for winterization and protection against possible ice formation in the well.

3.3.4.2 Access

At the intake site, the pumphouse building and wet well would be accessed using the gravel pad formed by the protruding wall structure.

3.3.4.3 Resiliency and Serviceability

The Site B geometry also intersects the river's thalweg, but this site has the added benefit of the hydraulic control provided by the bedrock outcrop located approximately 50 m downstream. The hydraulic control provided by the bedrock outcrop provides added assurance to the hydraulic stability of the site at low flow.

In this geometry, the Site B protruding wall structure is not overtopped by flood events and though ice scars were not seen during the site visit, it is expected that this structure would not be overtopped by ice.

The protruding wall structure puts the intake screens into the channel's thalweg as shown by bathymetric survey. Hydraulic modeling results show that have the screens placed in this location would allow for withdrawals at the shoulders of the open water season (June and October), if conditions permit, and would also allow withdrawals to the proposed DFO low flow limit.

3.3.4.4 Maintenance

The Site B concept considered maintenance requirements related to screen clogging, sediment accumulation in the wet well, seasonal operation, and preliminary measures to protect against ice floes or jams that might occur at the intake site.

A potential disadvantage of the general arrangement of this concept is that the wet well approaches 6 m deep. Any sediment build-up in the wet well would likely need to be removed using a hydrovac. At this depth (6 m) it may difficult for a hydrovac to remove the sediment. It is possible that the site could be graded down 1 to 2 m to create a shallower wet well. However, any lowering of the proposed infrastructure would have

to be evaluated against potentially exposing the infrastructure to flood flows and/or ice and debris. A potential lowering of the wet well access at Site B for sediment removal using a hydrovac, should be further reviewed during preliminary design.

The screen face and trash racks at Site B could also be designed with some provision for ice resistance but there would be residual risk of damage over winter. The City may wish to remove the screens at the end of the withdrawal season, prior to freeze-up and replace them with solid steel plates. Removing the screens would prevent them from being damaged by ice, and the steel plates would provide some protection to the structure and the intake chamber. The Site B concept includes a removable davit for screen and trash rack removal. The davit also facilitates the removal of the screens for cleaning as part of regular maintenance. Regular cleaning of sediment, debris and biological fouling will help to maintain the flow rate across the screen and maintain overall system performance.

Unlike Site A, the general arrangement of Site B allows for access during spring break-up. Should the City wish to pursue Site B with provision to begin withdrawal during ice break-up, then consideration for leaving the screens in throughout winter should be included in preliminary engineering design.

The Site B intake can be designed with an air bubbler or air scour system to reduce the likelihood of frazil ice build-up on the intake screens. Teflon coated screens and trash racks can also reduce frazil build-up. Because withdrawal is planned for the open water season, we have not included provision for frazil management in the concept, but mitigation measures like those mentioned above, could be further considered in preliminary engineering.

3.3.5 Site B Pipeline Concept Advancement

As an intake location Site B has some advantages, however, it presents some additional complications for pipeline routing. For this reason, pipeline routing from Site B has been presented as two options. Following the lidar topographic survey (partially completed, to date), two optimal routes from Intake Site B to Lake Geraldine were selected. The complete survey sections and the missing section were shown in Figure 3-4 (drawing C-001). Option 1 (Route B) follows a less direct path to LGHC, east from the intake site past upper base to discharge. Option 2 (Route B-A) follows a path from the intake site toward the proposed intake at Site A, where it then follows the same path as Route A. Route B options are shown in plan view on drawing C-100.

Route B

Route B has partial road and power access, requiring additional trail extension from Upper Base Road to the intake site of approximately 3.2 km. It would require approximately 6 km of power line extension, assuming the preferred pumps will be electrically operated. While inaccessible from Upper Base Road to Intake Site B, Upper Base Road is an existing and well-defined road and provides economy to Route B.

Figure 3-5 (drawing C-100) shows a plan view of Route B from intake to discharge. The 500 mm diameter pipeline will span a total length of 7,170 m across craggy terrain and a northern section of Airport Creek, past Upper Base Road to the discharge location at LGHC. High and low points can be smoothed with a cut and fill plan during detailed design, however, not eliminated. As a result, staged pumping and drains

will be required to ensure adequate pressure and relief can be maintained. The elevation along Route B to LGHC is 146 m.

Profiles for the proposed Route B are shown in Figures 3-9, 3-10 and 3-11 (drawings C-103, C-104, and C-105). Here, estimated locations for pumps, drains, and road and ditch crossings. Nunami believes that much of this pipeline routing option can remain above ground, except for some short road crossings.

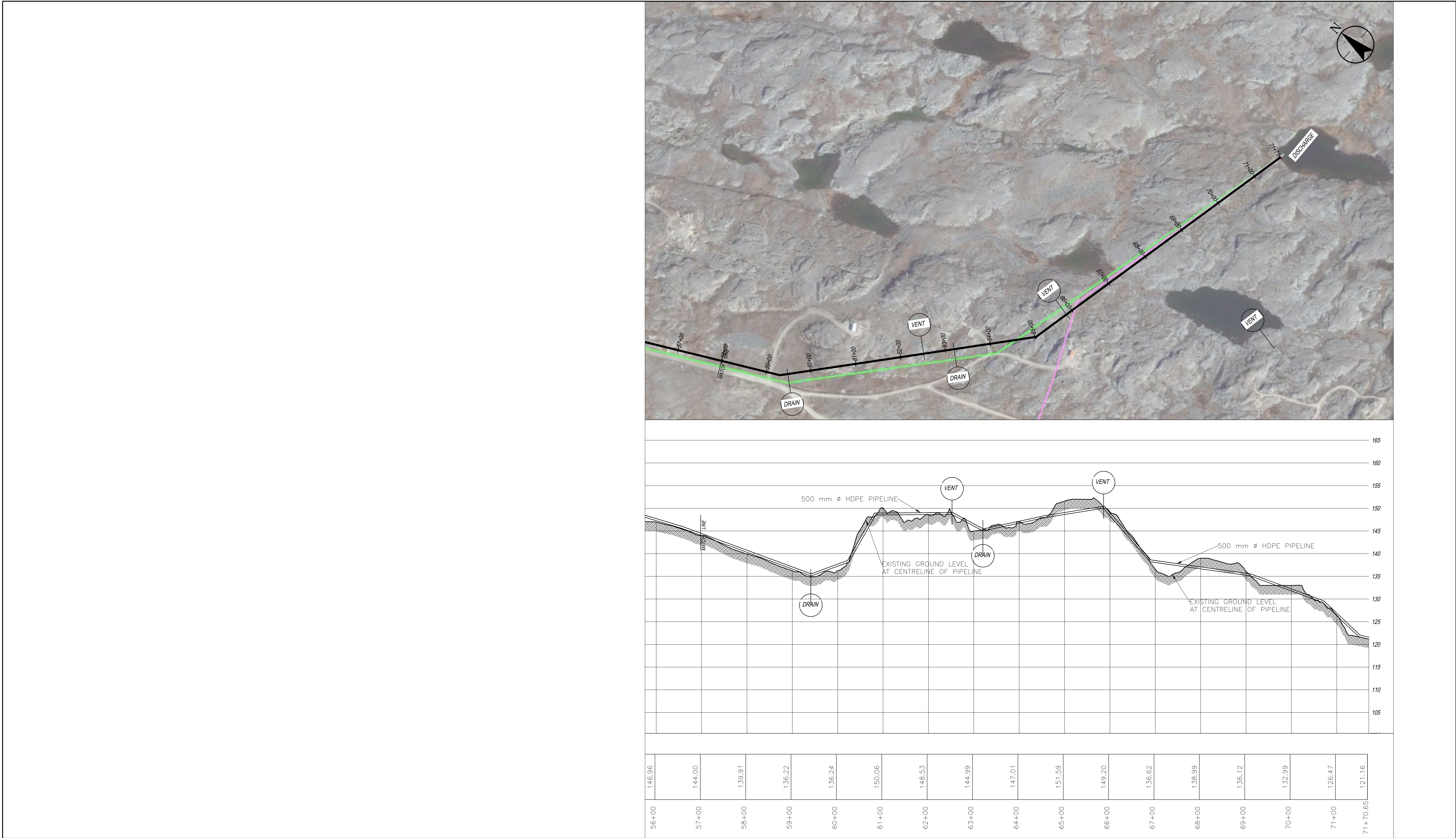
Route BA

Route BA runs from Sylvia Grinnell River Intake Site B to the general location of proposed Intake Site A. From here, it utilizes the route described in Route A, following overland past the north side of the runway. From here, the route runs south past the east side of the runway and bordering the airport lands between the asphalt plant and the quarry. Where applicable, the pipeline has been offset from the river to accommodate the 30 m federal reserve on navigable waters.

Figure 3-5 (drawing C-100) shows a plan view of Route BA from intake to discharge. The 500 mm diameter pipeline will span a total length of 2,057 m to Intake Site A in addition to the 4,443 m from Route A (combined 6,500 m) across craggy terrain. Running north of the airport lands, this option is easily accessed through Kudlik yards and along access roads north of Federal Road. The pipeline continues along Qaqqamiut Street toward upper base and running behind the residential housing on the Plateau toward LGHC. A service road or trail will be required in sections between existing road sections and from the intake site past the eastern side of the runway and as the pipeline approaches LGHC. A service trail and power would also need to be constructed from Intake Site A to Intake Site B along this route. In total, approximately 5.15 km of trail could be required.

Profile for the proposed Route BA are shown in Figure 3-12 (drawing C-106). These profiles are combined with Route A (Figures 3-6 and 3-7, drawings C-101 and C-102) for a complete profile to LGHC. We have estimated locations for pumps, drains, air release, and road crossings. Nunami is recommending a portion of the pipeline be buried between stations 23+00 and 29+00 (Kudlik and City yards and new Waste Transfer Station site) to avoid interference with activities in this part of town and to offer additional protection to the pipe material.

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1 Revision	DS	WO	19.04.12
0 Initial Submission	IM	WO	19.02.13
Revision	By	Appd.	YY.MM.DD
A Conceptual Design	IM	WO	19.02.13
Issued	By	Appd.	YY.MM.DD

General Notes

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CONSTRUCTION

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Client/Project



IQALUIT SYLVIA GRINNELL
OVERLAND FILL LINE
City of Iqaluit, Nunavut

File Name:

DS

WO

19.04.12

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Title

ROUTE B PLAN & PROFILE
57+00 TO 71+70.65

Project No.
144930114

Scale
1:2,500 H

Drawing No.

Sheet

Revision

C-105

5 of 5

A/1

3.4 Pipeline Design Summary

3.4.1 Pipeline Routing Options

In conjunction with the assessment of potential intake locations, Sites A and B, as described in this report, Nunami has provided one optimal routing option from Intake Site A (Route A) and two possible pipeline routing options from Intake Site B (Route B and Route BA). Each of the options were developed in considerations of the overall head and pipeline length best suited for transmission of water from Sylvia Grinnell River to the Lake Geraldine reservoir. Included within the concept advancement is the discharge location at LGHC. This idea was originally presented in the feasibility report (Nunami, 2018) and involves pipeline discharge in the small headwater creek at the north end of the Lake Geraldine reservoir. Discharge to the LGHC would slightly reduce the overall length of pipe required for this service while also providing the most direct discharge to the Lake Geraldine reservoir. Although there is likely capacity for optimization of the discharge location at different points within the watershed, Nunami believes the LGHC discharge point is suitable for this conceptual level of design. For other discharge locations within the watershed, additional factors, such as evaporation and ground permeation, must be assessed. Nunami does not believe that a discharge location at a point in the watershed farther from Lake Geraldine is suitable, considering the additional water and pumping requirements that would result from this action.

A summary table outlining information from the pipeline routing options is provided in Table 3-5 depicts each of the routing options considered.

Table 3-5: Pipeline Routing Options Summary Table

Water Intake Site	Route Option	Pipe Length (km)	Road Access	Discharge Point	Max Head to Discharge Point (m)	Encroaching on Airport Lands
A	A	4.443	Partial	LGHC	138.5	Yes
B	B	7.170	Partial	LGHC	168.0	No
	BA	6.500	Partial	LGHC	138.5	Yes

The routing options presented follow the natural topography of the route. While there is capacity for some route optimization, such as determining the cut and fill quantities, blasting would likely not be performed for this work. Instead, slight adjustments to the alignment and pipe cribbing would be employed. Even after optimization, any selected route between Sylvia Grinnell and LGHC will cross many high and low points. Due to a steep valley present north of Federal Road, the pipeline routes presented represent the best options from each intake site for this project. Routing a water pipeline directly through this steep valley would create additional pumping challenges. Given the topography of the site (i.e., intermediate valley peaks of almost 50 m and maximum head to discharge of up to 165 m), the staged pumping proposed within this report provides the most optimal pipeline routing from Sylvia Grinnell River to the LGHC. To follow a direct, straight-line route to LGHC would require additional and substantial increases and decreases in elevation, thus, reducing the efficiency of the design and further increasing complexity and cost of construction and operation. For the purposes of this report, Nunami believes that the pipeline routes presented within this document represent the most economical and constructible routes for this project.

3.4.2 Pipeline Pumping and Pipe Considerations

Nunami has selected the accelerated open-water withdrawal rate of 233 L/s to provide adequate resupply of Lake Geraldine over a 92-day period of July, August and September.

A suitable pipeline design will run primarily above-ground from a pumping station at the selected intake location to the LGHC. As discussed in the previous sections, year-round withdrawal was not considered feasible, so the pipeline design considered only summer withdrawal. The pipe will consist of fused, uninsulated high-density polyethylene (HDPE) pipe. Buried sections will be required at road crossings and to avoid interference with activities at Kudlik yards. An intake pumphouse will be required for each option and up to four additional booster pumps, depending on the which routing option is selected. Head and frictional losses contribute to a higher pressure within the system.

It is good practice where possible that each additional booster pump be coupled with an atmospheric tank or drain to prevent unsafe conditions within the pipeline. An atmospheric tank, or a break pressure tank, is one possible solution to provide pressure relief of the pipe water is pumped to a higher elevation. During the selection of pumps for each routing option, a 63 m³ atmospheric tank could be used to release pressure at stages within the pipeline route. If this option is selected for detailed design, the number of required tanks would be one less than the required number of pumps. In situations requiring only one pump, no atmospheric tank would be needed.

$$No. Tanks = No. Pumps - 1$$

That also been included within the estimate of probable cost.

Details of the required pipe sizes and required pump information are provided in Table 3-6, including the required number of pumps for each scenario. While a larger pipe size is paired with a higher construction cost, it has lower pumping operational costs. Power poles and an access trail will need to be constructed for booster and pumping station operation and maintenance, which have been computed in the estimate of probable cost. Two larger but standard pipe sizes were chosen as practical options in the design. Following computation of pressures resulting from head loss and frictional losses, the required pump size, configuration, and required horsepower were determined.

Table 3-6: Pipe Sizes and Pumping Information for the Raw Water Intake Sites

Water Intake Site	Routing Option	Pipe Size (mm)	Required Pump Pressure (psi)	Number of Pumps Required	Design Pump Horsepower (hp)	No. Atmospheric Tanks
A	Route A	500	222	1	650	0
B	Route B	500	300	2	850	1
	Route BA	500	260	2	700	1

Route A

Route A from site A to upstream of Lake Geraldine running along the airport property, through the North 40 portion of town, and then uphill along Qaqqamiut road would produce a pipeline of approximately 4450 m in length, with approximately 500 m below grade through the North 40.

It is possible to construct pipeline Route A with a single pump located at the intake pumphouse.

Based upon a 500mm HDPE pipe diameter and computation of head and frictional pressures, a single pump (250 psi normal operation) with a 650 hp motor will be sufficient to lift water between Intake Site A and LGHC. This is possible without the use of an atmospheric tank. Route A builds a pressure of 222 psi (511 ft Total Dynamic Head, TDH). This pressure is less than the capacity of one pump is required. Air releases will be placed at high points or every 150 m (500 ft) and drains at low points. This configuration is shown in Figure 3-6 (drawing C-101) and Figure 3-7 (drawing C-102).

Alternate configurations would include two 300 hp pumps with a single booster station would also meet the requirements of Route A and may be more economical. The location of the single booster station is shown on C102. In addition, this route would allow a smaller pipe, down to 400 mm diameter to be used with slightly higher pump power.

Pressure and Elevation Curves showing the above options have been included in **Appendix C**.

Route BA

Pumping from Site B to Site A and then along the route previously identified from Site A to Lake Geraldine would produce a pipeline of approximately 6200 m in length, with similarly approximately 500 m below grade through the North 40.

The additional frictional and head losses from Intake Site B to tie-in with Route A increase pressure to 260 psi (600 ft TDH), resulting in the requirement of two 320 HP pumps and probably 1 atmospheric tank at a booster station located in the North 40.

Pressure and Elevation Curves showing the above options have been included in **Appendix C**.

Route B

Pumping from Site B to Lake Geraldine directly would produce a pipeline of approximately 7200 m in length, with only road crossings below grade.

The frictional and head losses from Intake Site B to the north side of Upper Base Road increases pressure to 300 psi (692 ft TDH), resulting in the requirement of two or three pumps and one or two atmospheric tanks. A total pumping HP of approximately 825 HP would be required, which could be split evenly, or more installed at the pumphouse. Reasonable configurations could include a 275 HP installed at the intake pumphouse and two 275 HP booster stations, or have 550 HP installed at the Site B pumphouse, and 275 HP installed adjacent to the creek crossing at the westernmost extension of Qaqqamiut road.

Pressure and Elevation Curves showing the above options have been included in **Appendix C**.

3.4.3 Redundancy

Considering the critical nature of this project, it was previously requested that redundant backup pumps be installed at each of the pumping sites. These additional pumps will be factored into the opinion of probable cost (OPC). The OPC, including pipeline material, power routing, access trail, pump, pumping station(s) and intakes, and operations has been performed. An estimate of pumping station cost has also been included and is based upon costs from a similar Nunavut project in 2015-2016. A summary of Nunami's Opinion of Probable Cost (OPC) is included within Nunami's site evaluation in Section 4. Cut sheets of a recommended pump type and manufacturer are provided in Appendix C.

4 SITE EVALUATION AND PROBABLE COST

4.1 Qualitative Site Evaluation

Nunami completed an evaluation of the two intake site options, and pipeline routing, for seasonal (open water) withdrawal, based on identified conditions in the following categories:

- Hydraulic conditions (flood, low flow, and ice)
- Stability (bank and bed)
- Land ownership
- Serviceable intake type (e.g., permanent or removable options)
- Instream isolation complexity (e.g., complexity of construction for intake types)
- Other intake engineering and operations considerations (e.g., bedrock presence)
- Fish and fish habitat presence and type
- Site security
- Intake site and pipeline land conflicts
- Pipeline and access road length
- Estimated construction and annual operation costs for 500mm pipeline

The suitability of each intake site and sub-site was assessed qualitatively, relative to each other, and identified as generally good, likely challenging, or very challenging. This assessment is provided in Table D1 in Appendix D, where conditions were assigned colours for visual identification of site suitability (i.e., green for good, yellow for likely challenging, and red for very challenging).

4.2 Opinion of Probable Costs

Based on the identified site conditions, anticipated serviceable intake types, and pipeline options with associated infrastructure (e.g., access roads, power, pump stations), Nunami has developed a (AACE No. 18R-97) Class 5 estimate of probable infrastructure construction costs per site to allow overall cost comparison intake, pumping, and pipeline routing from Site A and Site B. This summary is included in Table 4-1, with more detail provided in **Appendix B**. Costs are inclusive of access, power supply, and instrumentation and controls.

Table 4-1: Summary of Class 5 Probable Project Costs for Raw Water Supplementation from Intake Sites at the Sylvia Grinnell River

Cost Item	Site A	Site B	
	Route A	Route B	Route BA
Intake (including Pumphouse and Equip.)	\$3,955,400	\$3,661,150	\$3,661,150
Power Supply, Access Trail and Pipe	\$2,804,751	\$6,160,890	\$5,073,622
Pumps and Tanks (incl. Redundancy)	\$300,000	\$700,000	\$700,000
Additional Booster Pumphouses	\$0	\$550,000	\$550,000
Sub-total (no contingency)	\$7,060,151	\$11,072,040	\$9,984,772
Total Construction Costs (incl. 30% contingency)	\$9,178,196	\$14,393,652	\$12,980,204
Annual Operation Cost (incl. 30% contingency)	\$1,200,000	\$1,600,000	\$1,300,000

An additional 30% contingency is also included for construction and operational costs. Estimated costs for construction reviews and inspection are not included as these can vary considerably between projects depending on contractor time and client preference. Other pre-construction costs such as public engagement and permitting are also not included.

4.3 Discussion

4.3.1 Intake Sites

Operationally, intakes at Site A and Site B are likely to be very similar and both should provide reliable water withdrawal given the concept designs to date. However, there are a few factors that support an intake situated at proposed Site B versus Site A.

Site B has channel characteristics that support an intake at this location. There is a bedrock outcropping that immediately downstream of Site B. This outcropping will likely stabilize the downstream control of the water flowing through the intake reach and is likely to create a stable backwater effect during low flows. Site A does not have such a hydraulic control. Additionally, Site B is located on an outside meander bend which will promote scour at the toe of the intake structure, and which is also likely to minimize sediment build-up at the screens which decreases screen maintenance requirements. At Site A, the presence of the upstream side channel/backwater area could result in additional sediment being deposited near the screens resulting in increased maintenance requirements.

Site A is situated near the end of the main runway at the Iqaluit airport and may be susceptible to contamination from runway runoff from the airport. Water quality sampling would be needed to confirm if this is a concern or not when comparing sites.

Site B is situated at a higher elevation due to the natural geography at the site. The proposed location provides better access to the site from the access road and the high elevation provides additional mitigation against exposure to ice floes and water from flood events.

Construction at Site B is assumed to have lower fill requirements and the ability to combine the intake with the wet well into a single structure. This constructability results in an estimated construction cost reduction of 10-15%. The main disadvantage of the intake Site B concept is the elevation change from the base of the wet well to the top rim. Nearing 6 metres, the depth of the wet well could make it difficult for maintenance equipment, such as a hydrovac, to remove sediment during routine maintenance. This could be mitigated by grading the site down and decreasing the elevation change in the wet well; however, this would need to be evaluated against potentially exposing the wet well and pumphouse to ice or flood flows.

Further to these comparisons, the site location will need to consider the ancillary requirements for the intake structure; including road access, power, and the conveyance system requirements.

4.3.2 Pipeline Routes

Operationally, Pipeline Route A is superior to Route B and Route BA. It will require less pumping and power considerations and the entire route is more accessible for maintenance and construction crews. In addition to this, Route A is a less visible option for users of the Sylvia Grinnell River. Though the intake will be in a more accessible area and widely used area of the park, Routes B and BA will have a larger impact on the land, requiring roads, power to Intake Site B. Preliminary discussions with the Hunters Trappers Association (HTA) identified a sharp resistance to this project. The roads, infrastructure, and above ground pipeline will have a much larger visual impact than routing from Intake Site A, which is largely already developed.

Prior to construction, an archaeological assessment of the approved pipeline route will be required.

4.3.3 Permitting and Engagement

Site A and Site B intake sites are within the Sylvia Grinnell River, but outside of the Sylvia Grinnell Territorial Park. Access to intake sites and overland pipeline routes are within the municipal boundaries of the City of Iqaluit. The construction and operation of a new intake site for water withdrawal will require review, approvals and/or authorizations from:

- Nunavut Planning Commission (NPC) and Nunavut Impact Review Board (NIRB) (Screening)
- Fisheries and Oceans Canada (Request for Review / Fisheries Authorization)
- Nunavut Water Board (Amendment to Water Licence)
- Iqaluit Airport Manager and Transport Canada (Review)

Parties that may have an interest in the project include:

- Residents of the City of Iqaluit
- Amaruq Hunters and Trappers Association
- Qikiqtani Inuit Association
- Crown-Indigenous Relations and Northern Affairs Canada – Water Resources Division
- Government of Nunavut – Department of Health, Department of Environment, Department of Culture and Heritage, Nunavut Parks and Special Places

4.3.3.1 NPC/NIRB Screening

The requirements to obtain an authorization from one or more regulators or agencies for the project triggers the requirement for a NPC/NIRB screening of the proposed project under the provisions of the *Nunavut Agreement* and *Nunavut Planning and Project Assessment Act*. This process requires submission of a project proposal to the agencies, including a description of the project works and activities, evidence of engagement with affected communities, and a preliminary assessment of impacts of the project on the environment. The outcome of this public screening is a determination by the NIRB whether the project can proceed to permitting, or whether additional review is required. The NPC/NIRB screening for this project should be expected to take approximately 3 months.

4.3.3.2 Amendment to City of Iqaluit Water Licence

The City of Iqaluit currently holds a type “A” water licence issued by the Nunavut Water Board (NWB). The licence permits water withdrawal from specified sources and up to specified amounts. The withdrawal of more than 300 m³ water per day from a new source – the Sylvia Grinnell River, requires approval by the NWB and an amendment to the licence in accordance with the *Nunavut Waters Regulations*. An application for such amendment requires that the application supporting documents include 100% complete design drawings. The NWB’s process to amend a type A water licence includes a public technical review and requires a public hearing to be held. The NWB review process should be expected to take approximately 12 months.

4.3.3.3 Fisheries Authorization

A *Fisheries Act* Authorization is likely required due to instream works related to the proposed water intake structure and local importance of the Sylvia Grinnell River fishery. There are two approaches which can be taken. One is to submit a Request for Review by Fisheries and Oceans Canada (DFO) to confirm an authorization will be required or secondly apply for authorization with the understanding that it is likely there would be serious harm to fish as defined under the *Fisheries Act*. A Request for Review by DFO typically takes 6- 8 weeks for a decision whether an Authorization is required.

If an Authorization is required under Section 35(2) of the *Fisheries Act* an application for Authorization is required to be submitted to DFO. The application also requires the submission of an Offsetting Plan and letter of credit. DFO prefers that offsetting be completed prior to construction of a project, however this

sometimes can be waived and the offsetting can be conducted at the same time as project construction. DFO has 60 days to respond whether the application is complete. Once DFO considers the application to be complete they have 90 days to decide on whether the Authorization is approved or denied. Depending on the circumstances (generally information requirements) the timelines can be stopped and restarted.

Both the Authorization and Offsetting Plan require engagement with affected communities and Indigenous groups. Two engagement sessions in Iqaluit should be held. The objective of the first session would be to describe the project and to seek potential offsetting measures. The second engagement session would be to confirm the offsetting measured to be used. The amount of offsetting would be based on the degree of serious harm that is likely to occur and is usually transformed into square meters of habitat. Offsetting would require an engineered design which would require a field survey of the area to be offset to inform the engineering design. Preparing and obtaining an Authorization should be expected to take approximately 6 – 12 months.

4.3.3.4 *Transportation Zoning Review*

The City of Iqaluit Airport Authority has jurisdiction over developments conducted on airport lands, and Transport Canada has authority to approve certain types of developments within Airport Zoning Regulations (4,000 m radius of airport). A review of the proposed development against these authorities' requirements should be conducted in discussion with the Authority.

4.3.3.5 *Permitting and Engagement Timelines*

The NPC/NIRB screening usually precedes the NWB and DFO review processes. All three require evidence of engagement and the incorporation of feedback into project design and mitigations. 100% complete designs are required to support NWB and DFO applications. A plausible timeline for engagement, design and permitting are included in Table 4-2.

4.3.4 *Project Risks*

Some project risks identified are summarized in the list below.

1. Approval from HTA and Inuit Organizations
 - a. During two engagement meetings with the Amaruq HTA (fall 2018; winter 2019), the City presented updates on studies being done on the Sylvia Grinnell River to advance the understanding of the suitability of the river for water withdrawal. The concept of withdrawing water from the Sylvia Grinnell River was met with serious concern from the HTA based on its importance as a fishery. The project should consider lengthy discussion and engagement, including a presentation of alternate water source options.
2. Schedule
 - a. A sample schedule is presented in Section 4.3.5. Nunami believes this to be a reasonable schedule to minimize risks pertaining to design time, tendering, long lead items, and remote delivery of off-site construction materials (e.g. precast vaults, pumps, specialized piping).

3. Procurement

- a. While Iqaluit has access to several great contractors with piping and municipal experience, this will be the first project of this nature built in Iqaluit. Exact and direct experience of bidding contracts will be limited.
- b. The number of contractors expected to bid on this is limited.

4. Constructability

- a. This design will be partially constructed in a remote and inaccessible site. Constructing access will be the first step of construction.
- b. This project will involve an intake in the Sylvia Grinnell River, which is a major food source for Iqaluit. Potential limits to comply with regulatory bodies may exist and may be impacted by contractor experience (isolations and water management) and permitting (see Section 4.3.4)

4.3.5 Schedule

A reasonable schedule for this project is presented in the table below and considers design, regulatory approvals and permitting, tendering, procurement, and construction. The schedule was developed assuming design would be awarded by early summer 2019.

Table 4-2: Estimated Schedule to Project Completion

Task	Date
Selection of Engineering Consultant	Summer 2019
Preliminary Design	Summer 2019
Engagement	Fall 2019
Geotechnical Investigations	November – December 2019
NPC/NIRB Screening	January – March 2020
Detailed Design	Fall 2019
100% Detailed Design	February 28, 2020
Water Licence Amendment Application DFO Authorization Application	March 2020
Water Licence Approval DFO Authorization	Prior to March 31, 2021
Tender	March 2021
Materials Shipping	Sealift 2021
Construction	Intake: Fall 2021 Pipeline: Summer 2022
Commissioning	Late Summer 2022

4.3.6 Funding Opportunity

We understand where and how facilities and infrastructure obtain capital funding, and therefore we can provide this practical, current, and region-specific knowledge to develop a funding plan. Through Nunami-Stantec and Stantec's North America Funding Program (NAFP), we can also access Canada's most comprehensive source of funding information. With this affiliation, we can help clients search, access, and apply for over \$28 Billion of Canadian public and private funding sources from the Funding Portal's database. This tool is highly applicable for finding all (both public and private) grants and funding programs you could apply for to develop this new project.

As well as identifying the applicable funding sources and grants you are eligible to apply for, Stantec can help you evaluate which ones are most appropriate for you and then complete the paper or on-line applications to improve your chances of success.

While not the focus of this study, some sources of funding that may be applicable to this project include:

1. Small Communities Fund (PTIC-SCF_ Canada. Infrastructure Canada)
2. Green Municipal Fund (Water Quality and Conservation Canada. Federation of Canadian Municipalities)

4.4 Recommendation

While Intake Sites A and B are operationally similar, some slight advantages can be seen with construction costs for Intake Site B. Both pipeline routes from Intake Site B require additional pumping, access, and servicing requirements. In consideration of cost, impact, constructability, and servicing, Intake Site A and Pipeline Route A are the preferred option.

4.5 Next Steps

4.5.1 Additional Field Components

Following the review of this report and acceptance of the intake location and pipeline route, further field data collection will be required. Currently, Nunami believes the following two items are the major field programs remaining.

1. Geotechnical investigation
 - Geotechnical drilling program should be performed at the selected intake site.
2. Topographic survey
 - Due to the vast area limits to analyze pipeline routing options, UAV survey was used to gather information at concept-level as a tool to further develop pipeline routing options.
 - Once the pipeline routing option has been selected, more detailed topographic information will be required moving into detailed design. Now that extents of the survey are limited, traditional GPS survey is appropriate.

4.5.2 Design Advancement

Table 4-2 outlines a schedule estimate for completion of this project. An assumption for that schedule was that engineering design would be awarded in the summer 2019. Following selection of the concept-advancement options, the required engineering submissions will include preliminary design, detailed design (50%, 90%, 99%), Issued for Tender (IFT), and Issued for Construction (IFC, stamped) design packages.

5 CLOSURE

Nunami Stantec Ltd. has prepared this report for the sole benefit of the City of Iqaluit (the City) for the purpose of advancing the conceptual design for raw water supplementation infrastructure at two sites on the Sylvia Grinnell River. The report may not be relied upon by any other person or entity, other than for its intended purposes, with the express written consent of Nunami Stantec Ltd. and the City. Any use of this report by a third party, or any reliance on decisions made based upon it, are the responsibility of such third parties.

The information provided in this report was compiled from existing documents and data provided by the City, and by field data compiled by Nunami Stantec Ltd. This report represents the best professional judgement of our personnel available at the time of its preparation. Nunami Stantec Ltd. reserves the right to modify the contents of this report, in whole or in part, to reflect any new information that becomes available. If any conditions become apparent that differ significantly from our understanding of conditions presented in this report, we requested that we be notified immediately to reassess the conclusions provided herein.

Respectfully Submitted,

NUNAMI STANTEC LIMITED

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6 REFERENCES

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APPENDIX A

Site Photographs

Conceptual Design Advancement for Raw Water Supplementation from the Sylvia Grinnell River, Iqaluit, NU
Appendix A: Site Photographs

April 2019



Photo 1: Site A, Looking Across the Channel from the WSC Station on the Left (East) Bank



Photo 2: Site A, Looking Across the Channel from the Left (East) Bank

Conceptual Design Advancement for Raw Water Supplementation from the Sylvia Grinnell River, Iqaluit, NU
Appendix A: Site Photographs

April 2019



Photo 3: Site B Looking Downstream from the Left (East) Bank



Photo 4: Site B Looking Downstream from the Left (East) Bank

APPENDIX B

Opinion of Probable Cost

Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
CONCEPT DESIGN: OPINION OF PROBABLE COST
Intake Site A, Pipeline Route A

Item	Description	Unit	Quantity	Unit Price	Total
Intake					
1 General Requirements					
1.1	Mobilization and Demobilization	ls	1	\$ 300,000.00	\$ 300,000.00
1.2	ECO Plan & Environmental Monitoring	ls	1	\$ 50,000.00	\$ 50,000.00
1.3	Utilities Coordination	ls	1	\$ 25,000.00	\$ 25,000.00
2 Site Preparation					
2.1	Top Soil Stripping and Sub-grade Preparation	m2	450	\$ 20.00	\$ 9,000.00
2.2	Supply and Install Access Pad Granular Base	m2	50	\$ 30.00	\$ 1,500.00
3 Wet Well, Intake Installation, and Backfill					
3.1	Supply and Install Isolation, Including Water Management	ls	1	\$ 50,000.00	\$ 50,000.00
3.2	Bed Substrate Excavation and Stockpile (Top 400 mm)	m3	170	\$ 20.00	\$ 3,400.00
3.3	Common Excavation & Stockpile Onsite	m3	300	\$ 10.00	\$ 3,000.00
3.4	Supply and Install Pre-Cast Intake Structure c/w Fish Screens	ls	1	\$ 250,000.00	\$ 250,000.00
3.5	Supply and Install 2.4 m x 2.4 m Precast Concrete Well Structure	ls	1	\$ 225,000.00	\$ 225,000.00
3.6	Supply and Install NPS 24 Intake Pipe (Material TBD)	m	35	\$ 1,200.00	\$ 42,000.00
3.7	Backfill with Onsite Material	m3	250	\$ 8.00	\$ 2,000.00
3.8	Replaced and Regrade Salvaged Bed Substrate	m2	100	\$ 25.00	\$ 2,500.00
3.9	Remove Isolation	ls	1	\$ 5,000.00	\$ 5,000.00
3.10	Bank Regrading	m2	250	\$ 10.00	\$ 2,500.00
4 Wing Wall, Access Pad, Pumphouse, and Controls					
4.1	Base Preparation (Granular Top Fill 400 mm and Grading)	m2	500	\$ 35.00	\$ 17,500.00
4.2	Granular Fill - Intake Structure	m3	1,200	\$ 35.00	\$ 42,000.00
4.3	Supply & Install Concrete Wing Wall Structure Tied to Intake Pod, c/w Lifting Davit	ls	1	\$ 1,500,000.00	\$ 1,500,000.00
4.4	Supply and Install Prefabricated Pumphouse	ls	1	\$ 550,000.00	\$ 550,000.00
4.5	Supply and Install Submersible Pump c/w installation/removal railing system c/w back-up pump for redundancy	ls	1	\$ 250,000.00	\$ 250,000.00
4.6	Provision for ICE	ls	1	\$ 250,000.00	\$ 250,000.00
4.7	Provision for Supply and Install Generator c/w Fuel Storage	ls	1	\$ 350,000.00	\$ 350,000.00
4.8	Supply and Install Secondary Containment	ls	1	\$ 25,000.00	\$ 25,000.00

Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
CONCEPT DESIGN: OPINION OF PROBABLE COST
Intake Site A, Pipeline Route A

Item	Description	Unit	Quantity	Unit Price	Total
Pipeline					
5 Conveyance System					
5.1	Pipe	m	4,443	\$ 557.00	\$ 2,474,751
5.2	Road / Access Trail	m	0	\$ 396.00	\$ -
5.3	Power Supply	No. poles	44	\$ 7,500.00	\$ 330,000
5.4	Pumphouse (in addition to 4.4)	Is	0	\$ 550,000.00	\$ -
5.5	Booster Pumps (including redundant)	Is	2	\$ 150,000.00	\$ 300,000
5.6	Atmospheric Tanks	Is	0	\$ 100,000.00	\$ -
Contingency (30%)					\$ 2,118,045
<u>CONSTRUCTION</u> OPINION OF PROBABLE COST @ Concept Design					\$ 9,178,196
6 Estimated Annual Operational Costs					
6.1	Power	KWH	1,779,836	\$ 0.5204	\$ 926,227
Contingency (30%)					\$ 277,868
<u>ANNUAL OPERATION</u> OPINION OF PROBABLE COST @ Concept Design					\$ 1,204,095

Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
CONCEPT DESIGN: OPINION OF PROBABLE COST
Intake Site B, Pipeline Route B

Item	Description	Unit	Quantity	Unit Price	Total
Intake					
1 General Requirements					
1.1	Mobilization and Demobilization	ls	1	\$ 300,000.00	\$ 300,000
1.2	ECO Plan & Environmental Monitoring	ls	1	\$ 50,000.00	\$ 50,000
1.3	Utilities Coordination	ls	1	\$ 25,000.00	\$ 25,000
2 Site Preparation					
2.1	Top Soil Stripping and Sub-grade Preparation	m2	300	\$ 20.00	\$ 6,000
2.2	Supply and Install Access Pad Granular Base	m2	20	\$ 30.00	\$ 600
3 Wet Well, Intake Installation, and Backfill					
3.1	Supply and Install Isolation, Including Water Management	ls	1	\$ 50,000.00	\$ 50,000
3.2	Bed Substrate Excavation and Stockpile (Top 400 mm)	m3	100	\$ 20.00	\$ 2,000
3.3	Common Excavation & Stockpile Onsite	m3	100	\$ 10.00	\$ 1,000
3.4	Supply and Install Intake Structure c/w Wet Well and Fish Screens	ls	1	\$ 250,000.00	\$ 250,000
3.5	Backfill with Onsite Material	m3	100	\$ 8.00	\$ 800
3.6	Replaced and Regrade Salvaged Bed Substrate	m2	50	\$ 25.00	\$ 1,250
3.7	Remove Isolation	ls	1	\$ 5,000.00	\$ 5,000
3.8	Bank Regrading	m2	250	\$ 10.00	\$ 2,500
4 Wing Wall, Access Pad, Pumphouse, and Controls					
4.1	Base Preparation (Granular Top Fill 400 mm and Grading)	m2	300	\$ 35.00	\$ 10,500
4.2	Granular Fill - Intake Structure	m3	900	\$ 35.00	\$ 31,500
4.3	Supply & Install Concrete Wing Wall Structure Tied to Intake Pod, c/w Lifting Davit	ls	1	\$ 1,500,000.00	\$ 1,500,000
4.4	Supply and Install Prefabricated Pumphouse	ls	1	\$ 550,000.00	\$ 550,000
4.5	Supply and Install Submersible Pump c/w installation/removal	ls	1	\$ 250,000.00	\$ 250,000
4.6	railing system c/w back-up pump for redundancy	ls	1	\$ 250,000.00	\$ 250,000
4.7	Provision for ICE	ls	1	\$ 250,000.00	\$ 250,000
4.8	Provision for Supply and Install Generator c/w Fuel Storage	ls	1	\$ 350,000.00	\$ 350,000
4.9	Supply and Install Secondary Containment	ls	1	\$ 25,000.00	\$ 25,000

Continued on next page

Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
CONCEPT DESIGN: OPINION OF PROBABLE COST
Intake Site B, Pipeline Route B

Item	Description	Unit	Quantity	Unit Price	Total
Pipeline					
5 Conveyance System					
5.1	Pipe	m	7,170	\$ 557.00	\$ 3,993,690
5.2	Road / Access Trail	m	3,200	\$ 396.00	\$ 1,267,200
5.3	Power Supply	No. poles	120	\$ 7,500.00	\$ 900,000
5.4	Pumphouse (in addition to 4.4)	ls	1	\$ 550,000.00	\$ 550,000
5.5	Booster Pumps (including redundant)	ls	4	\$ 150,000.00	\$ 600,000
5.6	Atmospheric Tanks	ls	1	\$ 100,000.00	\$ 100,000
Contingency (30%)					\$ 3,321,612
<u>CONSTRUCTION</u> OPINION OF PROBABLE COST @ Concept Design					\$ 14,393,652
6 Estimated Annual Operational Costs					
6.1	Power	KWH	2,327,478	\$ 0.5204	\$ 1,211,220
Contingency (30%)					\$ 363,366
<u>ANNUAL OPERATION</u> OPINION OF PROBABLE COST @ Concept Design					\$ 1,574,586

Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
CONCEPT DESIGN: OPINION OF PROBABLE COST
Intake Site B, Pipeline Route BA

Item	Description	Unit	Quantity	Unit Price	Total
Intake					
1 General Requirements					
1.1	Mobilization and Demobilization	ls	1	\$ 300,000.00	\$ 300,000.00
1.2	ECO Plan & Environmental Monitoring	ls	1	\$ 50,000.00	\$ 50,000.00
1.3	Utilities Coordination	ls	1	\$ 25,000.00	\$ 25,000.00
2 Site Preparation					
2.1	Top Soil Stripping and Sub-grade Preparation	m2	300	\$ 20.00	\$ 6,000.00
2.2	Supply and Install Access Pad Granular Base	m2	20	\$ 30.00	\$ 600.00
3 Wet Well, Intake Installation, and Backfill					
3.1	Supply and Install Isolation, Including Water Management	ls	1	\$ 50,000.00	\$ 50,000.00
3.2	Bed Substrate Excavation and Stockpile (Top 400 mm)	m3	100	\$ 20.00	\$ 2,000.00
3.3	Common Excavation & Stockpile Onsite	m3	100	\$ 10.00	\$ 1,000.00
3.4	Supply and Install Intake Structure c/w Wet Well and Fish Screens	ls	1	\$ 250,000.00	\$ 250,000.00
3.5	Backfill with Onsite Material	m3	100	\$ 8.00	\$ 800.00
3.6	Replaced and Regrade Salvaged Bed Substrate	m2	50	\$ 25.00	\$ 1,250.00
3.7	Remove Isolation	ls	1	\$ 5,000.00	\$ 5,000.00
3.8	Bank Regrading	m2	250	\$ 10.00	\$ 2,500.00
4 Wing Wall, Access Pad, Pumphouse, and Controls					
4.1	Base Preparation (Granular Top Fill 400 mm and Grading)	m2	300	\$ 35.00	\$ 10,500.00
4.2	Granular Fill - Intake Structure	m3	900	\$ 35.00	\$ 31,500.00
4.3	Supply & Install Concrete Wing Wall Structure Tied to Intake Pod, c/w Lifting Davit	ls	1	\$ 1,500,000.00	\$ 1,500,000.00
4.4	Supply and Install Prefabricated Pumphouse	ls	1	\$ 550,000.00	\$ 550,000.00
4.5	Supply and Install Submersible Pump c/w installation/removal railing system c/w back-up pump for redundancy	ls	1	\$ 250,000.00	\$ 250,000.00
4.6	Provision for ICE	ls	1	\$ 250,000.00	\$ 250,000.00
4.7	Provision for Supply and Install Generator c/w Fuel Storage	ls	1	\$ 350,000.00	\$ 350,000.00
4.8	Supply and Install Secondary Containment	ls	1	\$ 25,000.00	\$ 25,000.00

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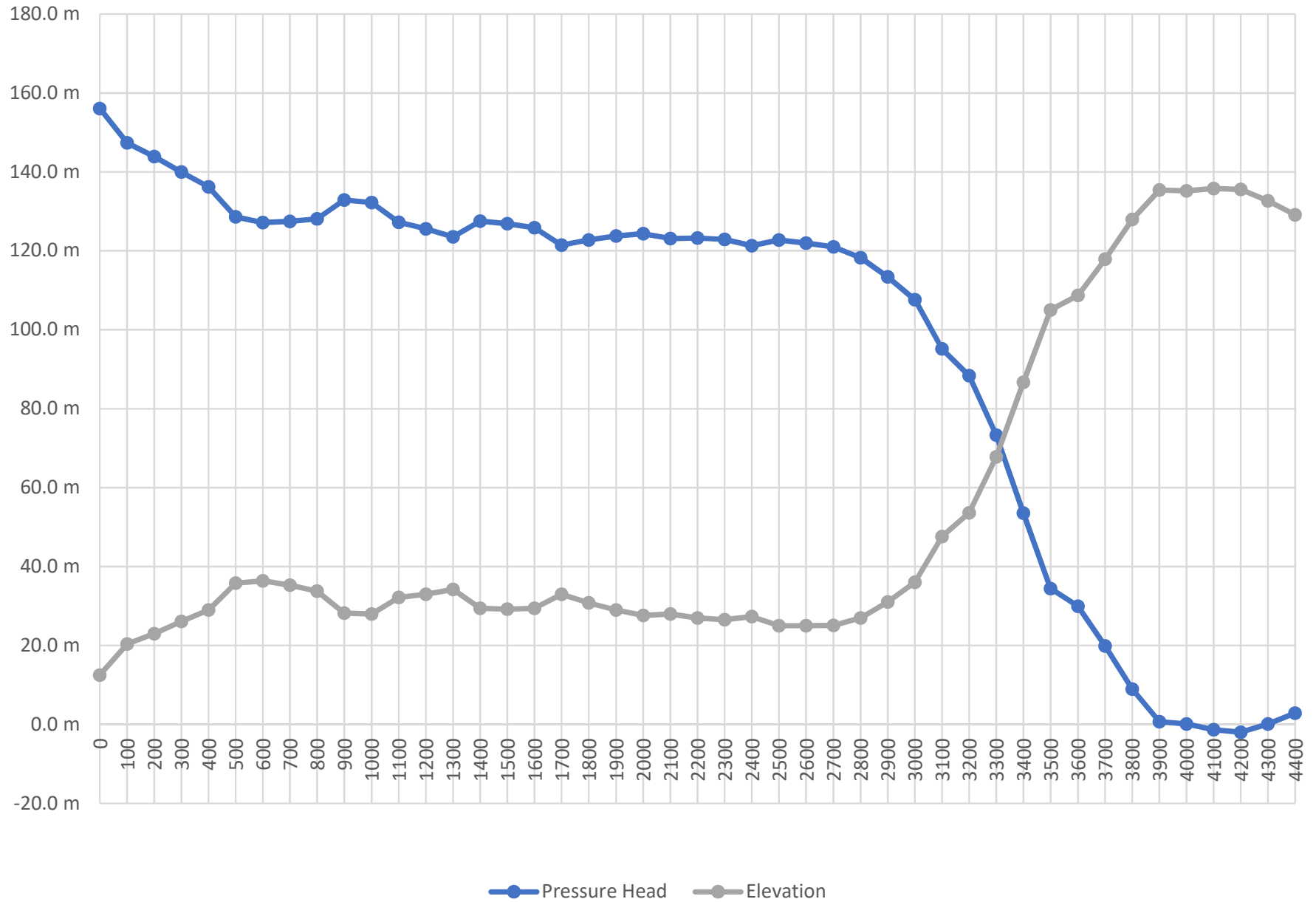
Iqaluit Supplemental Water Supply - Sylvia Grinnell Intake Concepts
 CONCEPT DESIGN: OPINION OF PROBABLE COST
 Intake Site B, Pipeline Route BA

Item	Description	Unit	Quantity	Unit Price	Total
Pipeline					
5 Conveyance System					
5.1	Pipe	m	6,500	\$ 557.00	\$ 3,620,500.00
5.2	Road / Access Trail	m	2,057	\$ 396.00	\$ 814,572.00
5.3	Power Supply	No. poles	85	\$ 7,500.00	\$ 638,550.00
5.4	Pumphouse (in addition to 4.4)	ls	1	\$ 550,000.00	\$ 550,000.00
5.5	Booster Pumps (including redundant)	ls	4	\$ 150,000.00	\$ 600,000.00
5.6	Atmospheric Tanks	ls	1	\$ 100,000.00	\$ 100,000.00
Contingency (30%)					\$ 2,995,431.60
CONSTRUCTION OPINION OF PROBABLE COST @ Concept Design					\$ 12,980,203.60
6 Estimated Annual Operational Costs					
6.1	Power	KWH	1,916,747	\$ 0.5204	\$ 997,475.11
Contingency (30%)					\$ 299,242.53
ANNUAL OPERATION OPINION OF PROBABLE COST @ Concept Design					\$ 1,296,717.65

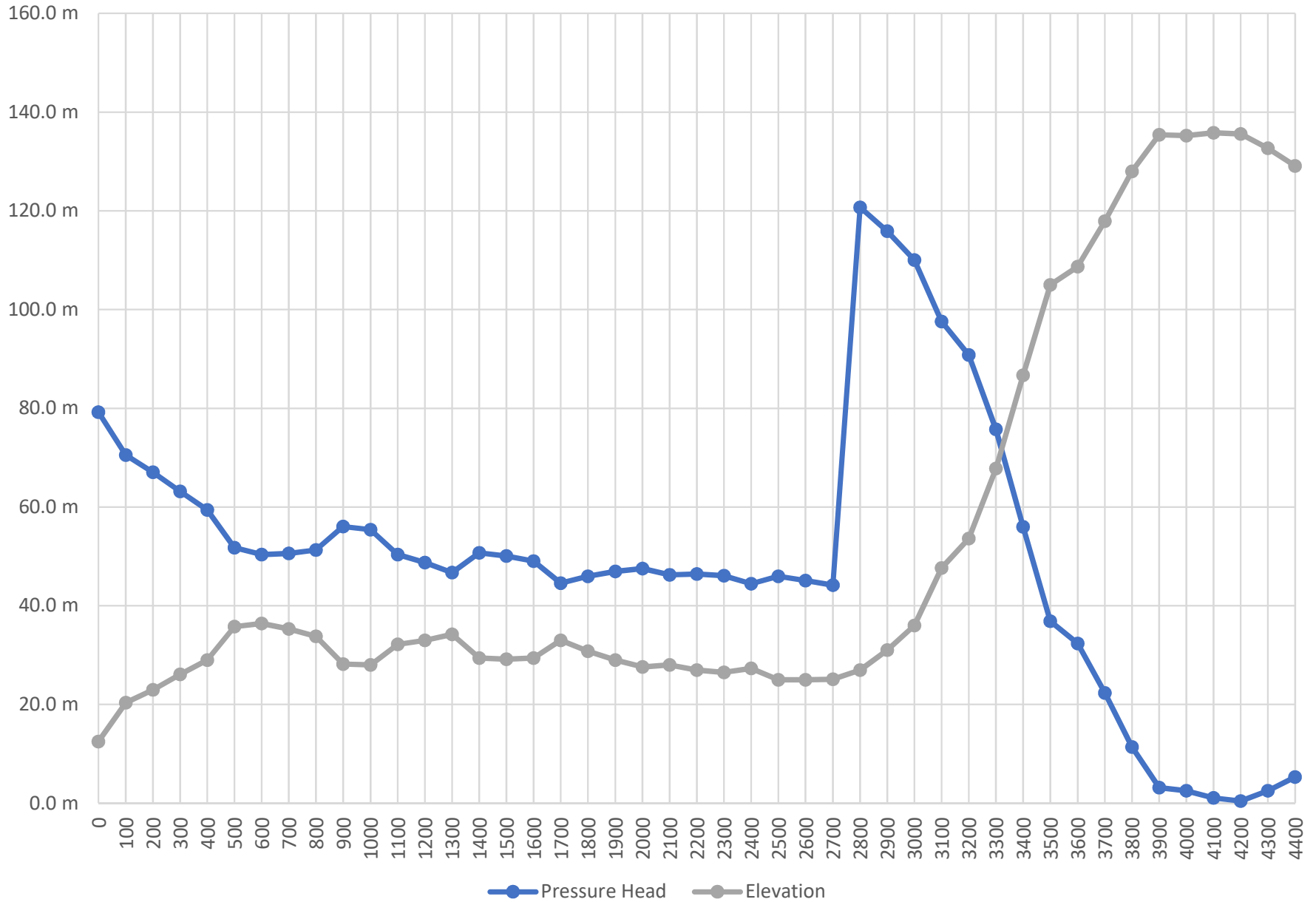
APPENDIX C

Recommended Pump Specifications

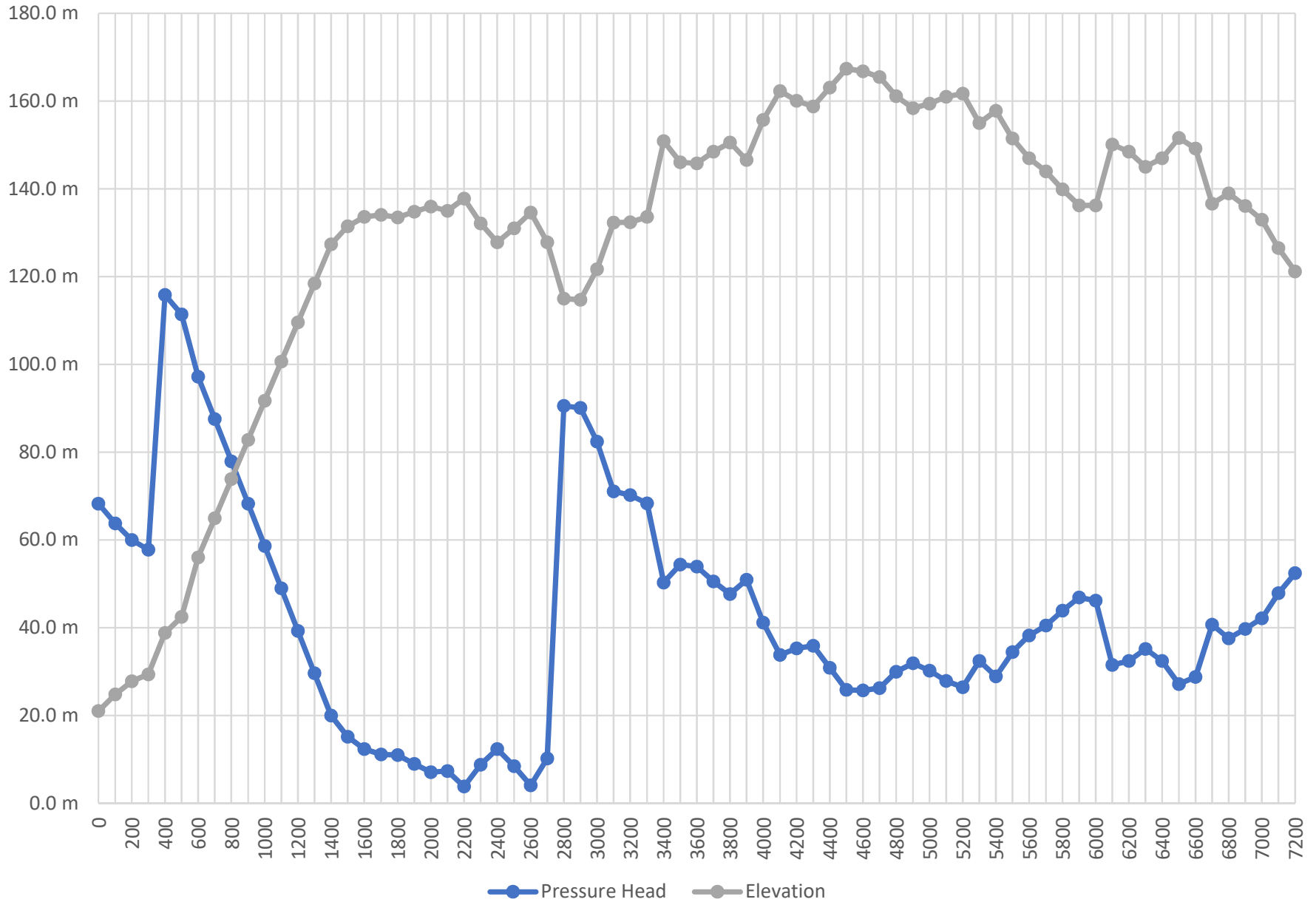
Site A to Grinnell Route A Overland - 500 mm Pipeline Single Pump



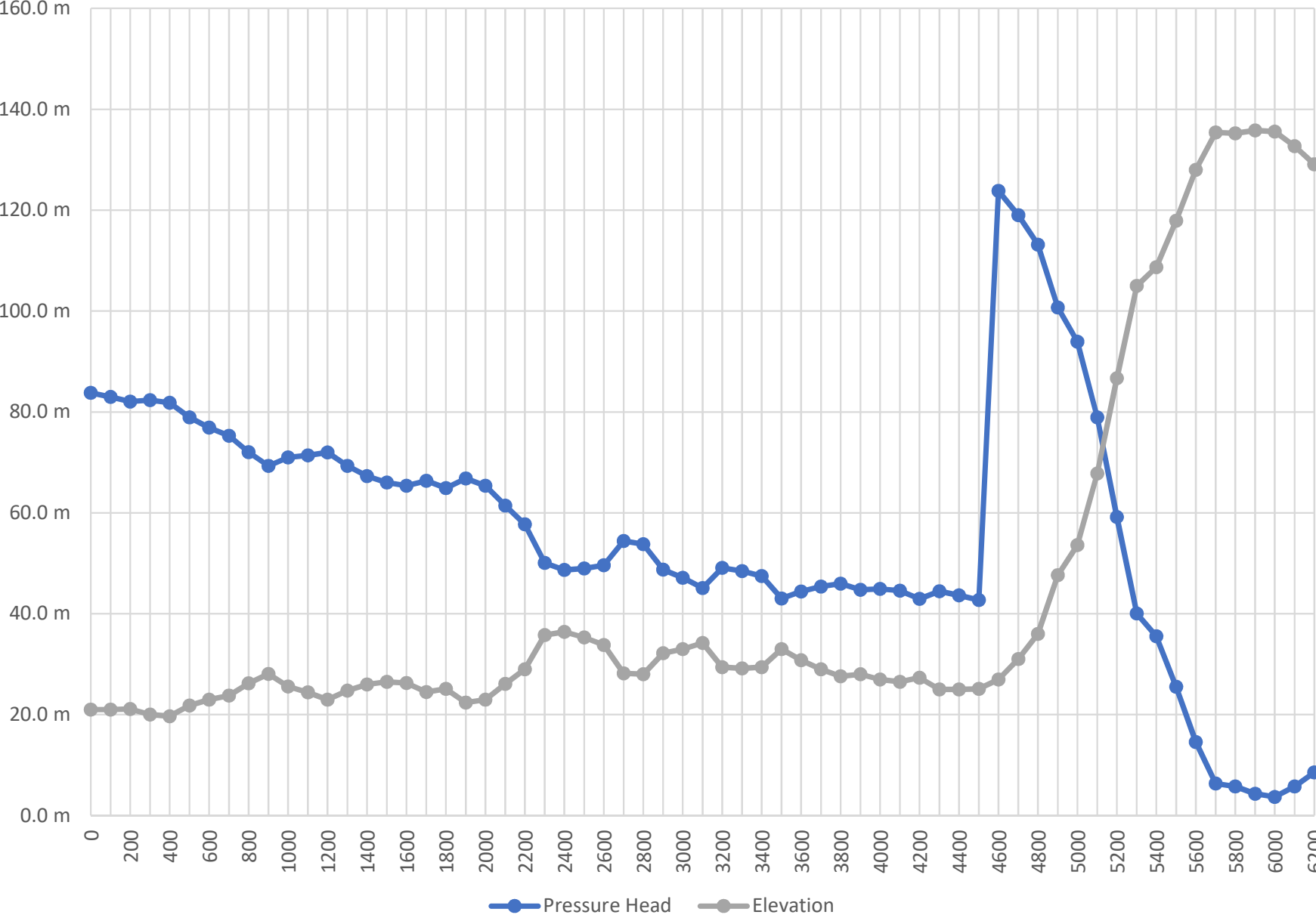
Site A to Grinnell Route A Overland - 500 mm Pipeline - 2 pumps



Site B to Grinnell Route B1 Overland - 500 mm Pipeline



Site B to Grinnell Route B2 Via Route A - 500 mm Pipeline





Priming Assisted Centrifugal Pump

Model PAH10A60C-B

Size 12" x 10"



PUMP SPECIFICATIONS

Size: 12" x 10" (305 mm x 254 mm) Flanged.

Casing: Ductile Iron 65-45-12.

Maximum Casing Pressure 351 psi (2420 kPa).*

Maximum Operating Pressure 250 psi (1723 kPa) at Temperatures up to 100° F (37° C) Based on System Component Limitations.*

Enclosed Type, Four Vane Impeller: Ductile Iron 80-55-06.

Handles 2" (50.8 mm) Diameter Spherical Solids.

Suction Spool: Gray Iron 30.

Impeller Shaft: Alloy Steel 4150M.

Replaceable Wear Rings: Ductile Iron 65-45-12.

Pedestal: Gray Iron 30.

Seal Plate: Ductile Iron 65-45-12.

Shaft Sleeve: Stainless Steel 303/304.

Priming Chamber: Gray Iron 30 Housing w/Stainless Steel Float and Linkage.

Discharge Check Valve: Ductile Iron Housing w/Buna-N Flapper.

Radial Bearing: Open Single Row Ball.

Radial and Thrust Bearing: Open Double Row Ball.

Bearing and Seal Cavity Lubrication: SAE 30 Non-Detergent Oil.

Gaskets: Resistant Synthetic Rubber, Vegetable Fiber w/Compressed Synthetic Fibers.

O-Rings: Buna-N.

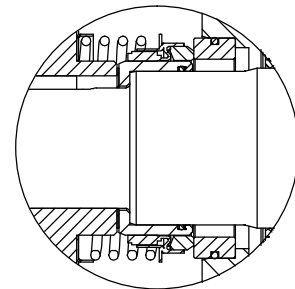
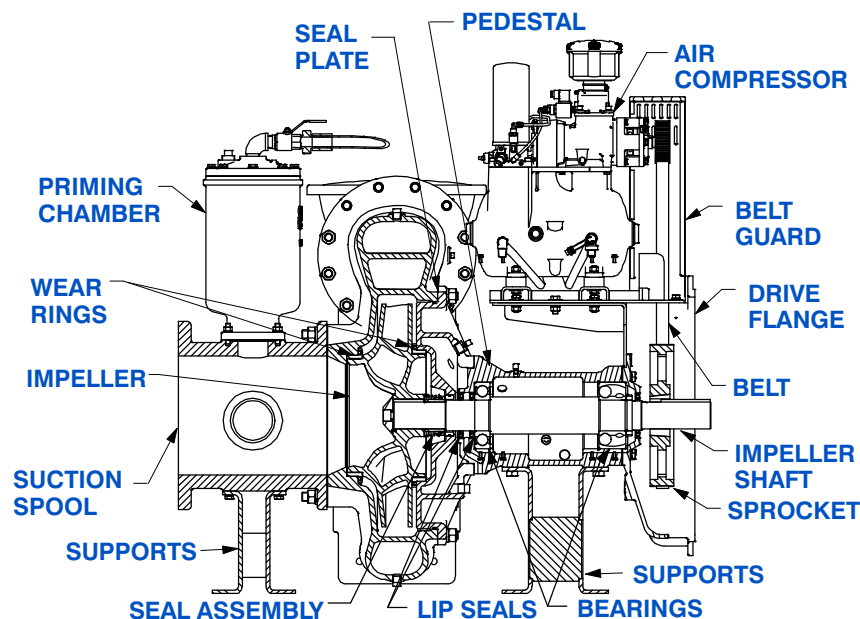
Hardware: Standard Plated Steel.

Bearing and Seal Cavity Oil Level Sight Gauges.

**Consult Factory for Applications Exceeding Maximum Pressure and/or Temperature Indicated.*

Standard Equipment: Belt Driven Air Compressor With Electric Motor Driven Cooling Fan (12 Volt Power Supply Required to be Supplied by End User). Strainer.

Optional Equipment: Base. NPT Threaded Flange Kit. Coupling. TEFC Motor. Suction Vacuum and Discharge Pressure Gauge Kit.



SEAL DETAIL

Mechanical, Oil-Lubricated. Silicon Carbide Rotating Face and Stationary Seat. Fluorocarbon Elastomers (DuPont Viton® or Equivalent). Stainless Steel 316 Cage and Stainless Steel 18-8 Spring. Maximum Temperature of Liquid Pumped 160°F (71°C).*



GORMAN-RUPP PUMPS

www.grpumps.com

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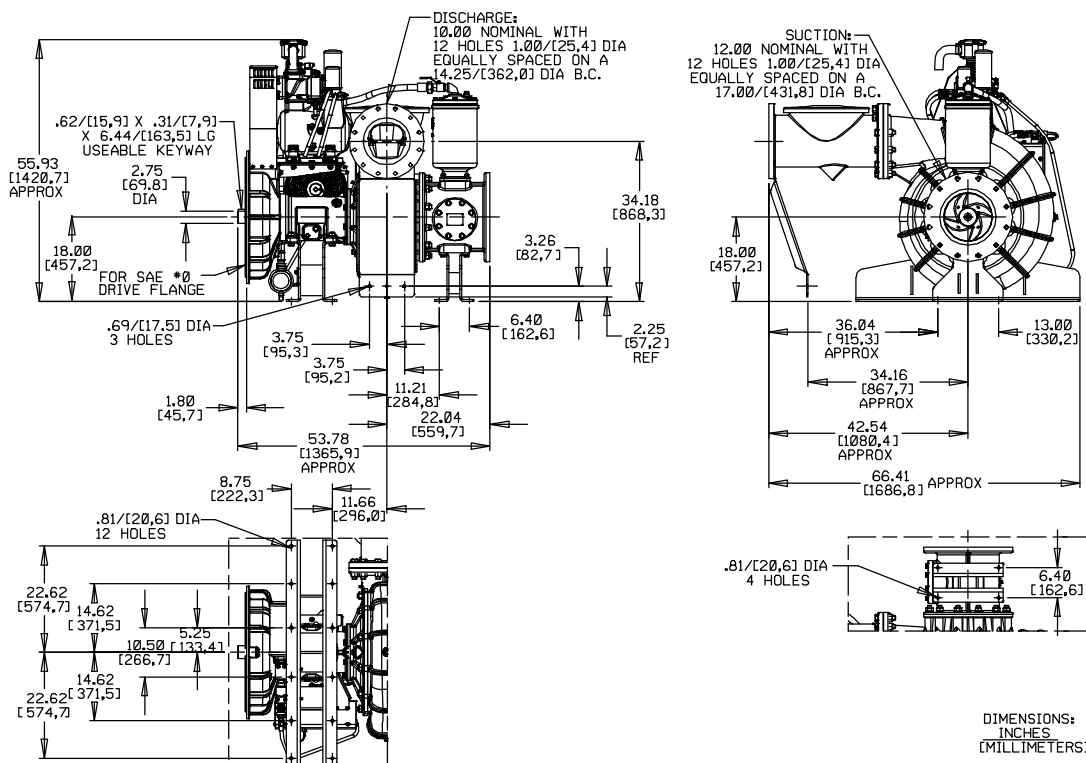
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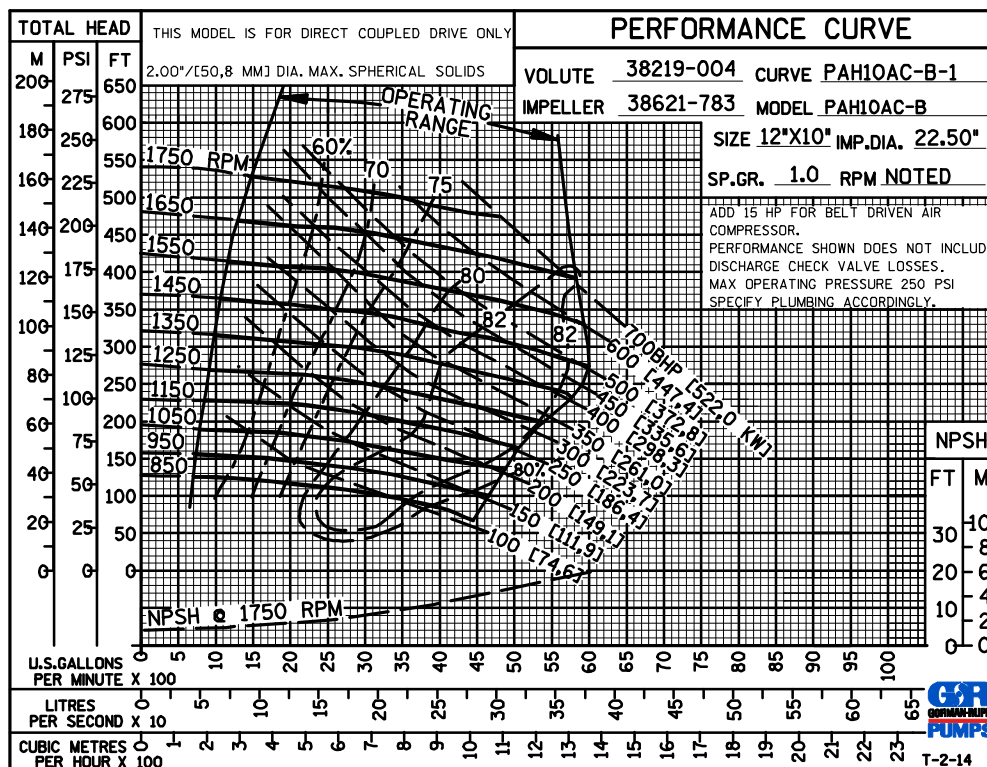
SECTION 43, PAGE 520C

APPROXIMATE
DIMENSIONS and WEIGHTS

NET WEIGHT: 2540 LBS. (1152 KG.)
SHIPPING WEIGHT: 2900 LBS. (1315 KG.)
EXPORT CRATE SIZE: 156 CU. FT. (4,4 CU. M.)



**MULTI-SPEED
CURVE
PERFORMANCE
BASED ON
WATER**



GORMAN-RUPP PUMPS

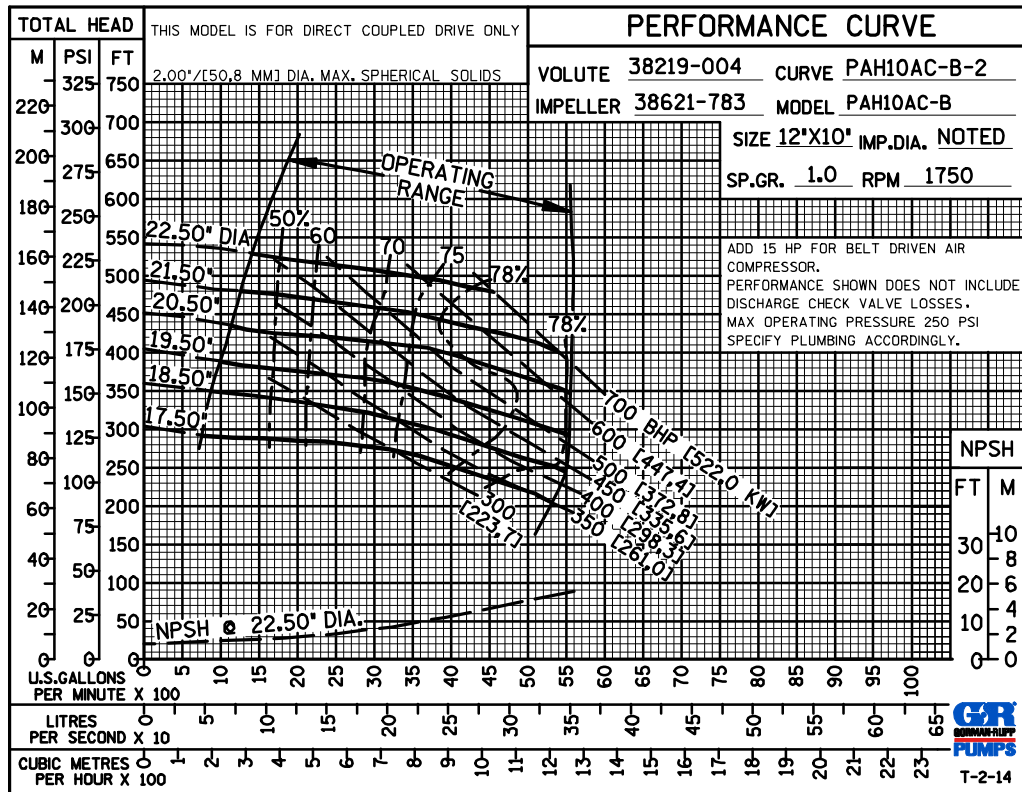
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60 HERTZ
PERFORMANCE
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Priming Assisted Centrifugal Pump w/Autostart

Model PAH10A60C-C18

Size 12" x 10"



Total Head		Capacity of Pump in U.S. Gallons per Minute (GPM) at Continuous Performance				
P.S.I.	Feet					
242.8	560	N/A	1500	◆	◆	◆
238.4	550	N/A	2400	◆	◆	◆
216.8	500	N/A	3075	◆	◆	◆
195.1	450	N/A	3285	◆	◆	◆
173.4	400	2410	3390	4250	4250	◆
151.7	350	2420	3400	4675	5590	◆
130.0	300	2425	3400	4690	5670	4000
108.4	250	2430	3400	4700	5695	5950
86.7	200	2450	3400	4725	5700	6100
65.0	150	2460	3400	4750	5700	6200
43.4	100	2475	3400	4780	5720	6300
21.7	50	2495	3400	4795	5740	6400
Suction Lift		±25'	20'	±15'	±10'	±±6'

‡ Suction Lift Based On 1600 RPM Engine Speed.

‡‡ Suction Lift Based On 1400 RPM Engine Speed.

◆ Flow Limitation due to Available Engine Horsepower.

PUMP SPECIFICATIONS

Size: 12" x 10" (305 mm x 254 mm) Flanged.

Casing: Ductile Iron 65-45-12.

Maximum Casing Pressure 375 psi (2586 kPa).*

Maximum Operating Pressure 250 psi (1723 kPa) at Temperatures up to 100° F (37° C) Based on System Component Limitations.*

Enclosed Type, Four Vane Impeller: Ductile Iron 80-55-06.

Handles 2" (50.8 mm) Diameter Spherical Solids.

Impeller Shaft: Alloy Steel 4150M.

Replaceable Wear Ring: Ductile Iron 65-45-12.

Bearing Pedestal: Gray Iron 30.

Seal Plate: Ductile Iron 65-45-12.

Seal: Mechanical, SAE 30 Non-Detergent Oil-Lubricated. Silicon Carbide Rotating Face and Stationary Seat. Fluorocarbon Elastomers (DuPont Viton® or Equivalent). Stainless Steel 316 Cage and Stainless Steel 18-8 Spring. Maximum Temperature of Liquid Pumped, 160° F (71° C).*

Shaft Sleeve: Stainless Steel 303/304.

Priming Chamber: Gray Iron 30 Housing w/Stainless Steel Float and Linkage.

Discharge Check Valve: Ductile Iron Housing w/Buna-N Flapper.

Radial Bearing: Open Single Ball.

Thrust Bearing: Open Double Ball.

Bearing Cavity Lubrication: Lithium EP2 Grease.

Suction Spool: Gray Iron 30.

Gaskets: Resistant Synthetic Rubber, Vegetable Fiber w/Compressed Synthetic Fibers.

O-Rings: Buna-N.

Hardware: Standard Plated Steel.

Seal Cavity Oil Level Sight Gauges.

*Consult Factory for Applications Exceeding Maximum Pressure and/or Temperature Indicated.

Standard Equipment: Belt-Driven Air Compressor. Hoisting Bail. Combination Skid Base w/Fuel Tank. Strainer. **Single Ball Type Float Switch.** **

**50 Ft. (15 m) Standard Length; Dual Switches and Alternate Cable Lengths Available From the Factory.

Optional Equipment: Batteries (2 Req'd). Heated Priming Chamber Kit. G-R Hard Iron Impeller. NPT Threaded Flanges. Over-the-Road Trailer (Meets D.O.T. and Transport Canada Requirements) Available w/Either Electric or Hydraulic Surge Brakes, Running Lights, Trailer Jack Stands and Safety Chains/Cables. Automatic Air Release Valve. Suction Vacuum and Discharge Pressure Gauge Kit. **Full Feature Control Panel** For Use w/Submersible Transducer Liquid Level Sensor (50 Ft. [15 M] Cable Standard, Alternate Lengths Available).



ENGINE SPECIFICATIONS

Model: Caterpillar C18 ACERT™.

EPA Tier: Interim Tier 4.

Type: Six Cylinder, Turbocharged AfterCooled, Liquid Cooled Diesel Engine.

Displacement: 1104.5 Cu. In. (18.1 liters).

Lubrication: Forced Circulation.

Air Cleaner: Dry Type.

Fuel Tank: 218 U.S. Gals. (825 liters).

Full Load Operating Time: 6.3 Hrs.

Starter: 12V Electric.

Standard Features: Muffler w/Weather Cap.

Optional: Electronic Fuel Level Sensor.

Engine Control Features: Padlockable Box with Throttle Control, Tachometer, Coolant Temperature, Oil Pressure, Voltage and Overstart Indicators/Shut-downs. Manual/Stop/Auto Keyswitch. Audible Startup Warning Delay. Fuel Level Display/Alarm/Shutdown (For Use With Optional Fuel Level Sensor).

CATERPILLAR PUBLISHED PERFORMANCE:

Maximum Gross Industrial BHP (Intermittent)
755 (563 kW) @ 1800 RPM



GORMAN-RUPP PUMPS

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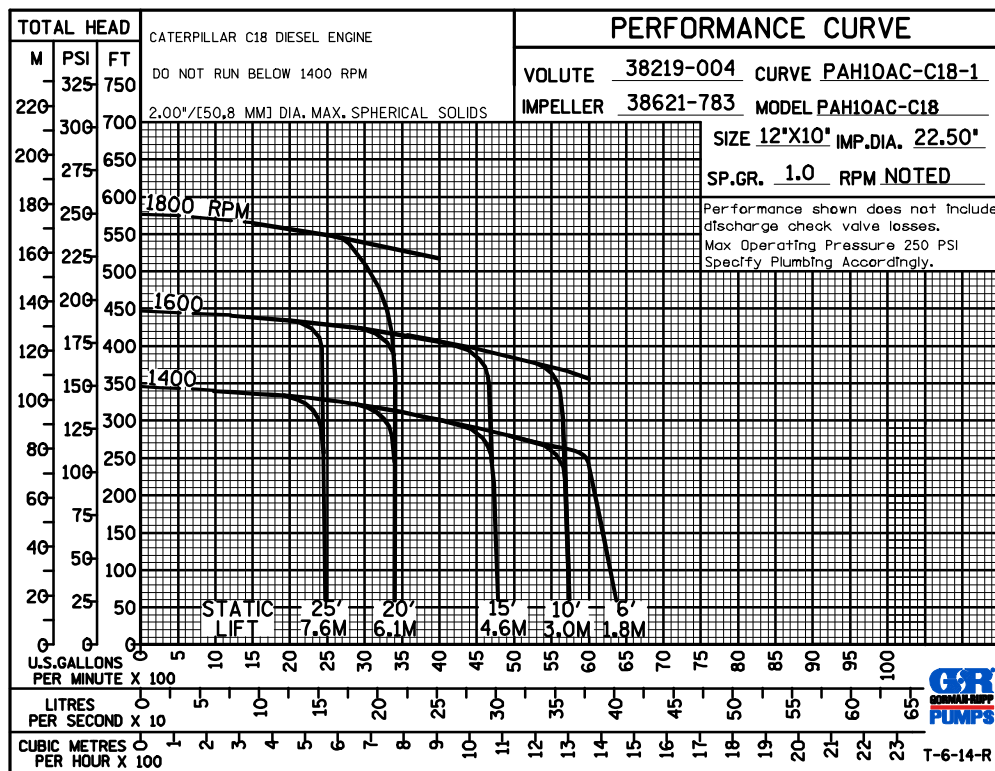
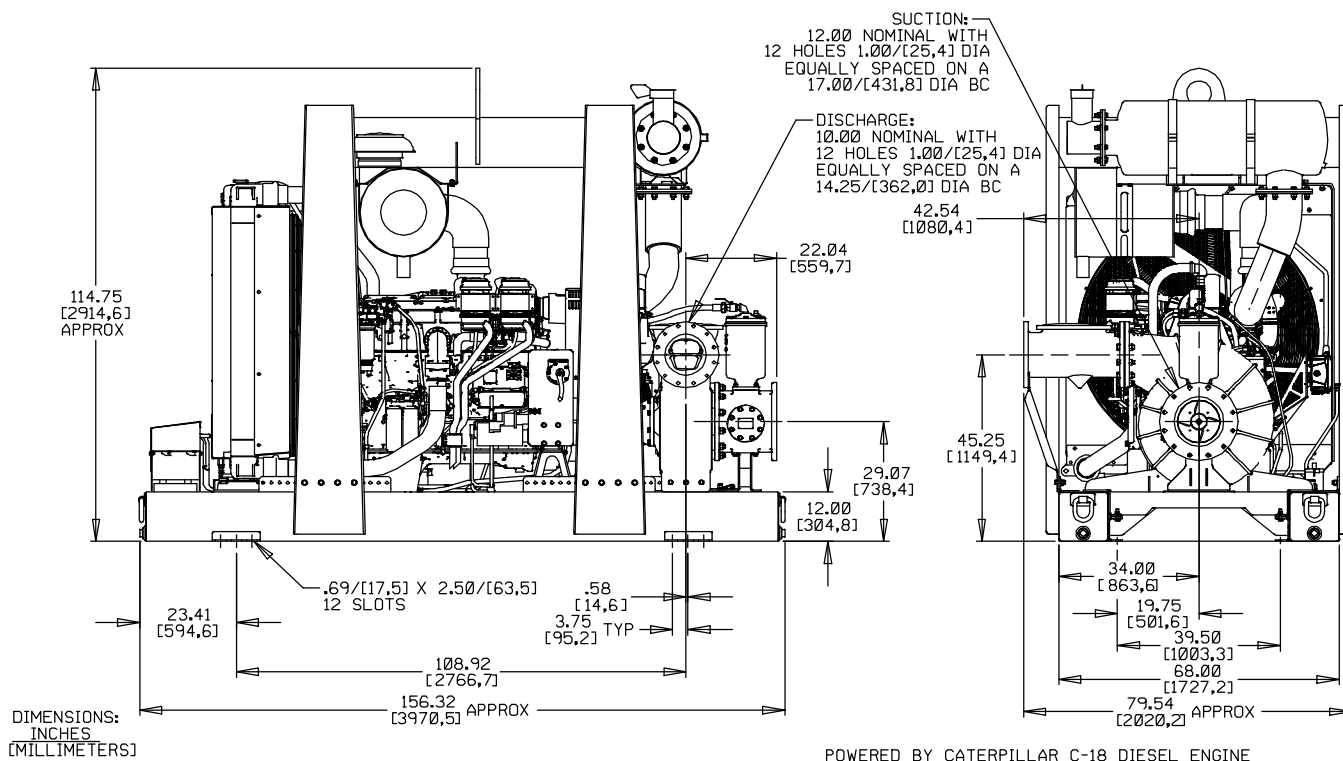
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SECTION 43, PAGE 510

APPROXIMATE DIMENSIONS and WEIGHTS

NET WEIGHT:
SHIPPING WEIGHT:
EXPORT CRATE SIZE:

12600 LBS. (5715,2 KG.)
13600 LBS. (6168,9 KG.)
970 CU. FT. (27,5 CU. M.)



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No.:

Date: 17-JUL-2017

Customer :

TECHNICAL PROPOSAL

Three-phase induction motor - Squirrel cage rotor

Product line : TEFC - W22 NEMA Premium Efficiency

Catalog Number :

List Price : \$

Notes:

Performed by:

Checked:



No.:

Date: 17-JUL-2017

DATA SHEET

Three-phase induction motor - Squirrel cage rotor

Customer :
Product line : TEFC - W22 NEMA Premium Efficiency

Frame : 588/9T
Output : 750 HP
Frequency : 60 Hz
Poles : 4
Full load speed : 1790 rpm
Slip : 0.56 %
Voltage : 575 V
Rated current : 672 A
Locked rotor current : 4700 A
Locked rotor current (I_L/I_n) : 7.0
No-load current : 256 A
Full load torque : 2171 lb.ft
Locked rotor torque : 240 %
Breakdown torque : 250 %
Design : B
Insulation class : F
Temperature rise : 80 K
Locked rotor time : 29 s (hot)
Service factor : 1.00
Duty cycle : S1
Ambient temperature : -20°C - +40°C
Altitude : 1000 m
Degree of Protection : IP55
Approximate weight : 4952 lb
Moment of inertia : 346.86 sq.ft.lb
Noise level : 81 dB(A)

	D.E.	N.D.E.
Bearings	6322 C3	6319 C3
Regreasing interval	6000 h	8000 h
Grease amount	60 g	45 g

Load	Power factor	Efficiency (%)
100%	0.85	96.7
75%	0.80	96.6
50%	0.70	96.3

Notes:

Performed by

Checked

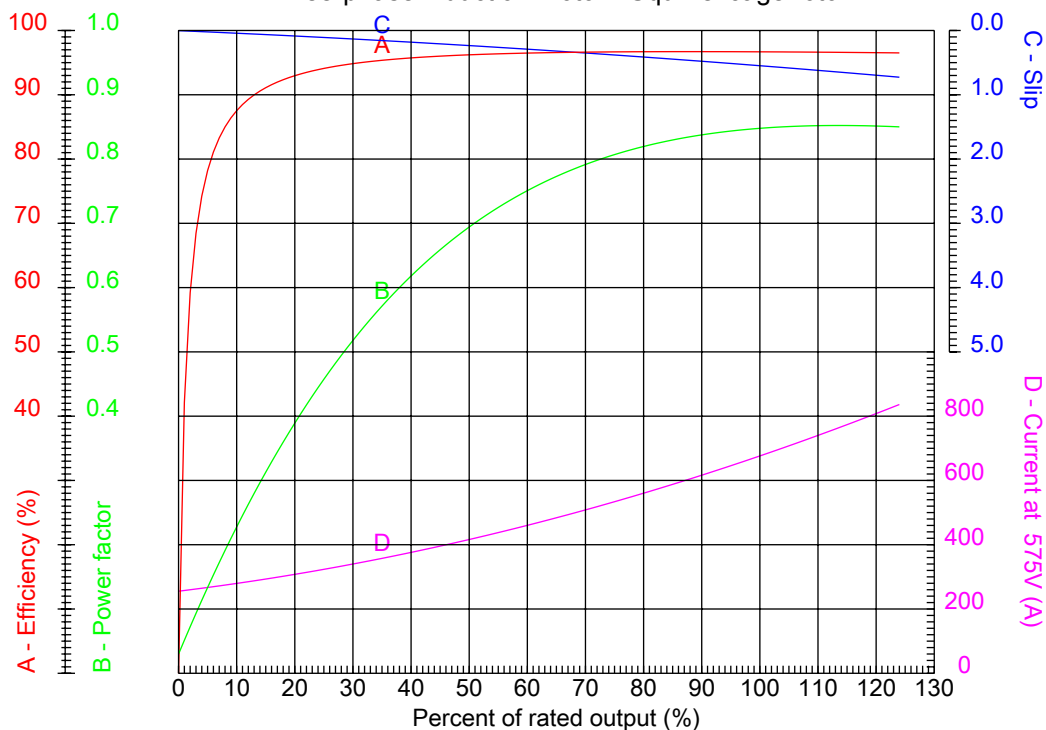


No.:

Date: 17-JUL-2017

PERFORMANCE CURVES RELATED TO RATED OUTPUT

Three-phase induction motor - Squirrel cage rotor



Customer :
Product line : TEFC - W22 NEMA Premium Efficiency

Frame	: 588/9T	Locked rotor current (I _L /I _n)	: 7.0
Output	: 750 HP	Duty cycle	: S1
Frequency	: 60 Hz	Service factor	: 1.00
Full load speed	: 1790 rpm	Design	: B
Voltage	: 575 V	Locked rotor torque	: 240 %
Rated current	: 672 A	Breakdown torque	: 250 %
Insulation class	: F		

Notes:

Performed by

Checked

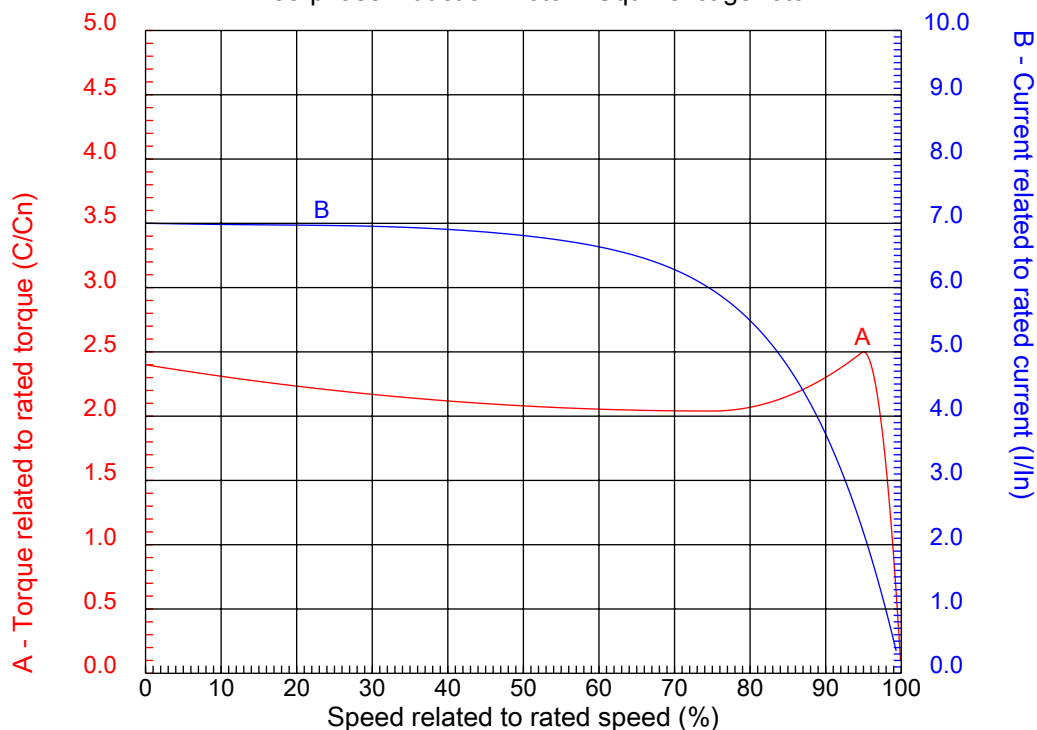


No.:

Date: 17-JUL-2017

CHARACTERISTIC CURVES RELATED TO SPEED

Three-phase induction motor - Squirrel cage rotor



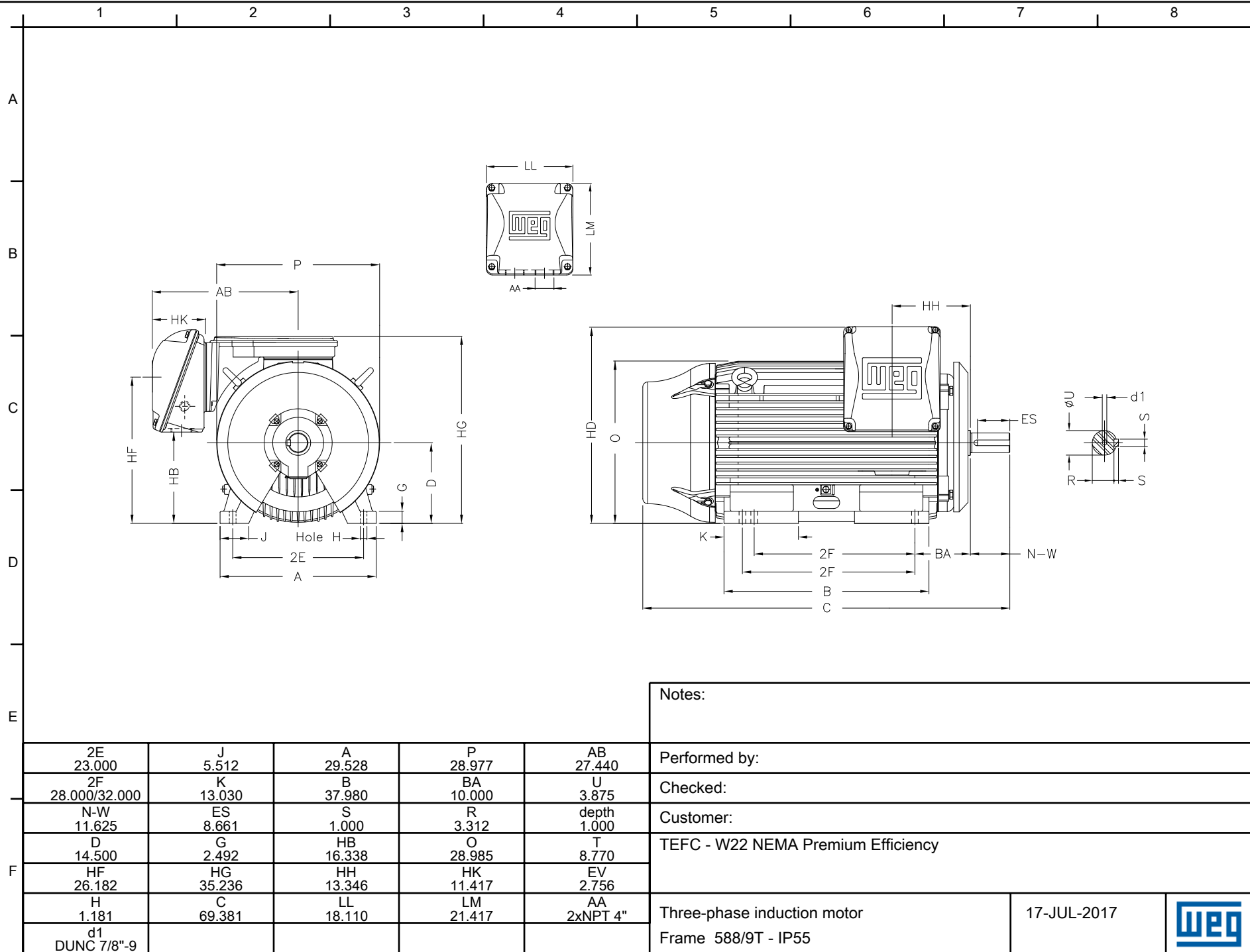
Customer :
Product line : TEFC - W22 NEMA Premium Efficiency

Frame	: 588/9T	Locked rotor current (I _L /I _n)	: 7.0
Output	: 750 HP	Duty cycle	: S1
Frequency	: 60 Hz	Service factor	: 1.00
Full load speed	: 1790 rpm	Design	: B
Voltage	: 575 V	Locked rotor torque	: 240 %
Rated current	: 672 A	Breakdown torque	: 250 %
Insulation class	: F		

Notes:

Performed by

Checked



APPENDIX D

Qualitative Site Assessment and Comparison

Table D1: Evaluation of Intake Site and Pipeline Route Options for Seasonal (Open-Water) Supplemental Water Withdrawal from the Sylvia Grinnell River

Intake Site	Location (UTM NAD83)	Pipeline Route	Location Description	Hydraulic Conditions			Stability		Land Owner-ship	Serviceable Intake Types ^{2,4}	Instream Isolation Complexity	Fish and Fish Habitat	Security	Site and Pipeline Route Land Conflict	Pipeline Length (m)	Pipeline (500 mm diam)	
				Flood ¹	Low Flow ¹	Ice ¹	Bank	Bed								Est. Const. 1st	Est. Op. Cost
A	520650 E 7071051 N	A	Intake: End of YFB runway and adjacent to WSC Station 10UH001 Pipeline: From Site A along runway, past quarry and Kudlik yards, toward Upper Base and discharge into LGHC	Moderate velocity and low debris. Pumphouse should be placed above elevation of WSC Station	Shallow, ow-pronounced thalweg. Protruding wall structure may promote some scour.	Ice floes. Forces may be moderate. Some frazil.	High	High	Municipal	Walled protruding bank structure. Submersible pump.	Modest cofferdam and no diversion.	Of the sites, least important for fish habitat. Unlikely to be overwintering habitat. Unlikely to be holding habitat for migrating fish.	Close to the airport.	Site and approx.2 km within municipal Transportation zone (airport). Remainder of pipeline within Populated Area.	4,443	\$9.2M	\$1.2M annually
B	519657 E 7072562 N	B	Intake: Left bank outside bend, outcrop Pipeline: From Site B running north east toward the north side of Upper Base Road. From here it runs south along Upper Base toward LGHC.	Moderate velocity and moderate debris. High bank allows pumphouse to be above flood and ice levels.	Low flow will confine to a thalweg and backwatered slightly by the bedrock. Protruding wall structure should promote some scour.	Ice floes grinding against left bank and may get hung up by bedrock outcrop. Jam forces may be high. Frazil generated in riffles upstream	Low. Armouring is warranted.	High and controlled by bedrock downstream	Municipal	Walled protruding bank structure. Submersible pump.	Modest cofferdam required. Could consider diversion through opposite bank slip of but dependent upon extent of bedrock.	Unlikely to be overwintering habitat. Unlikely to be holding habitat for migrating fish. Poor rearing habitat.	Remote site.	Site and approx. 3.5 km of pipeline within the Nuna land use area. Approx. 3.6 km of pipeline route within municipal. Pipeline not within transportation zone (airport) or populated area	7,170	\$14.4M	\$1.6M annually
		BA	Intake: Left bank outside bend, outcrop Pipeline: From Site B along Sylvia Grinnell toward Site A. From here, follows path of Route A.							Walled protruding bank structure. Submersible pump.			Partial remote site.	Site and approx. 2 km of pipeline within the Nuna land use area. Site and approx.2 km within municipal Transportation zone (airport). Remainder of pipeline within Populated Area.	6,500	\$13M	\$1.3M annually

APPENDIX E

**Seismic Refraction Survey Report –
Frontier Geosciences Inc.**

FRONTIER GEOSCIENCES INC.

SEISMIC REFRACTION SURVEY REPORT IQALUIT WATER INTAKE PROJECT IQALUIT, NU

Submitted to:
Stantec Consulting Ltd.
November 2, 2018

Authors:
Beth Galambos, P.Geo.
Cliff Candy, P.Geo.

Project: FGI-1574

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2. Seismic Refraction Survey	2
2.1 Survey Equipment	2
2.2 Survey Procedure	2
2.3 Seismic Refraction Interpretive Method	3
3. Geophysical Results	4
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3.2 Discussion	4
4. Limitations	6

Illustrations

	Location
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Figure 2 Site Plan A	Appendix
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Figure 4 Interpreted Seismic Profile SL-4	Appendix
Figure 5 Interpreted Seismic Profile SL-5	Appendix
Figure 6 Interpreted Seismic Profile SL-6	Appendix
Figure 7 Site Plan B	Appendix
Figure 8 Interpreted Seismic Profile SL-1	Appendix
Figure 9 Interpreted Seismic Profile SL-2	Appendix
Figure 10 Interpreted Seismic Profile SL-7	Appendix
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Figure 12 Interpreted Seismic Profile SL-9	Appendix
Figure 13 Interpreted Seismic Profile SL-10	Appendix

1. Introduction

During the period September 27 to October 1, 2018, Frontier Geosciences Inc. carried out a seismic refraction investigation for Stantec Consulting Ltd., at the Iqaluit Water Intake Project in Iqaluit, NU. A Survey Location Plan of the area is shown at a scale of 1:50,000 in Figure 1 in the Appendix.

The purpose of the geophysical survey was to classify overburden layering and depth to bedrock and identify possible permafrost, or weathered bedrock at two sites, each about 200 m by 200 m. Approximately 973m of detailed seismic refraction data was collected along 10 separate seismic lines. The line locations are presented at a scale of 1:1,500 in Site Plan A (Figure 2) and Site Plan B (Figure 7) in the Appendix.



Project site, View of Sylvia Grinnell River

2. Seismic Refraction Survey

2.1 Survey Equipment

The seismic refraction investigation was carried out using a Geometric Geode, 24 channel, signal enhancement seismograph and Oyo Geospace 10 Hz geophones. Geophone intervals along the multicored seismic cable were maintained at either 2.5 or 5 metres in order to ensure high resolution data on subsurface layering. Seismic energy was provided from a Buffalo gun, shotgun source firing 8 gauge, blank, shotgun shells into hand-excavated shotholes or a sledgehammer striking a steel plate. Shot initiation or zero time was established by metal to metal contact of a striking hammer contacting the firing pin of the shotgun or the hammer striking the plate.

2.2 Survey Procedure

For each spread, the seismic cable was stretched out in a straight line and the geophones implanted in the soil. Up to seven separate 'shots' were then initiated: one at either end of the geophone array, two at intermediate locations along the seismic cable, and one off each end of the line, to ensure adequate coverage of the subsurface. The shots were triggered individually and arrival times for each geophone were recorded digitally in the seismograph. For quality assurance, field inspection of raw data after each shot was carried out, with additional shots recorded if first arrivals were unclear.

Throughout the survey, notes were recorded regarding seismic line positions in relation to topographic and geological features. Relative elevations along the seismic lines were recorded by chain and inclinometer and referenced to handheld GPS measurements.

2.3 Seismic Refraction Interpretive Method

The final interpretation of the seismic data was arrived at using the method of differences technique. This method utilizes the time taken to travel to a geophone from shotpoints located to either side of the geophone. Velocities are calculated as the slope of first break pick times and geophone distances. When there is a significant change in slope a new velocity is calculated and assigned to the new layer. Basal velocities are calculated by the arrivals of off-end shots, where picked arrivals are refracted from the basal layer. Each geophone is assigned a velocity and time for each layer. Using the total time, a small vertical time is computed which represents the time taken to travel from the refractor up to the ground surface. This time is then multiplied by the velocity of each overburden layer to obtain the thickness of each layer at that point. The thicknesses are splined along the seismic line to create a continuous boundary between layers.



Seismic survey setup

3. Geophysical Results

3.1 General

The interpreted results of the seismic refraction lines are illustrated in Figures 3 to 6 and 8 to 13, at a scale of 1:500, in the Appendix. The seismic velocity layer interfaces are marked on the seismic profiles in green, blue and red. The interface line colours are not a specific velocity contour, but rather the interpreted discrete boundary above which velocities are defined within a certain range and below which velocities are within a significantly increased velocity range.

3.2 Discussion

For Site A, the results of the seismic refraction survey indicate the area is underlain by three distinct velocity layers. The surficial layer has a range of compressional wave velocities between 400 m/s and 1650 m/s. The majority of values fall between 400 m/s and 850 m/s, which is indicative of unconsolidated sands and gravels with a moderately high water content. The values above 850 m/s are found along SL-3 and the eastern end of SL-6. This may be due to the close proximity of the Sylvia Grinnell River, which would cause a more water saturated surficial layer resulting in higher compressional wave velocities.

Underlying the surficial layer is an intermediate layer with compressional wave velocities ranging from 1700 m/s to 2875 m/s. The lower end of these velocities is indicative of compacted overburden and/or a higher content of coarse materials. The higher end of the range represents possible discontinuous permafrost, a glacial till, or possible highly jointed, broken or weathered bedrock. Should continuous permafrost have been present, it is expected that velocities of the order of 3000 m/s would have been encountered. SL-3 shows has the largest variation in thicknesses, ranging from to 1.2 m to almost 6 m at the north end of the line.

The basal layer with compressional wave velocities of 5400 m/s to 5800 m/s is the interpreted bedrock surface. The velocity range is relatively small and uniformly distributed, suggesting the bedrock is competent. The location showing the most consistent thicker layer of material overlying the competent bedrock is located on line SL-2 at station 100E.

For Site B, the interpreted results of the seismic refraction data indicate that the area is underlain by up to four distinct velocity layers. All six seismic lines have a surficial layer with a range of compressional wave velocities between 400 m/s and 450 m/s, which is consistent with unconsolidated overburden such as sand and silt and the occasional area of rock fill.

SL-1 and SL-2 show an upper and lower intermediate layer. The thin upper intermediate layer has an interpreted velocity range of 850 m/s to 1350 m/s, which is indicative of an increasingly compact overburden found commonly with increasing depth. This layer is discontinuous and pinches out, as shown along SL-1 (Figure 8).

The underlying, lower intermediate layer has an interpreted velocity range of 1430 m/s to 2550 m/s and is shown in all seismic lines at Site B; although this layer also becomes discontinuous in SL-2 (Figure 9). The highest velocities range between 2100 m/s to 2550 m/s and are found along SL-1, SL-2, SL-7 and SL-10. Lower velocities of under 2100 m/s are found along the eastern section of Site B, along SL-8 and SL-9. As with Site A, this layer does not show the elevated velocities expected of continuous permafrost of approximately 3000 m/s. The lower range is consistent with compact granular materials, with the higher velocity range raising the possibility of discontinuous permafrost, glacial till or fractured and weathered bedrock. This layer varies in thickness from as low as 0.8 m to 3.3 m, with the thinner layers often correlating with lower compressional wave velocities.

The basal layer exhibits compressional wave velocities of 5400 m/s to 5750 m/s. The higher velocity range represents the competent bedrock surface with an almost identical range as found for the basal layer at Site A. The most consistent thicker layer of material overlying the competent bedrock is located on line SL-3 between approximately stations 40N and 80N.

4. Limitations

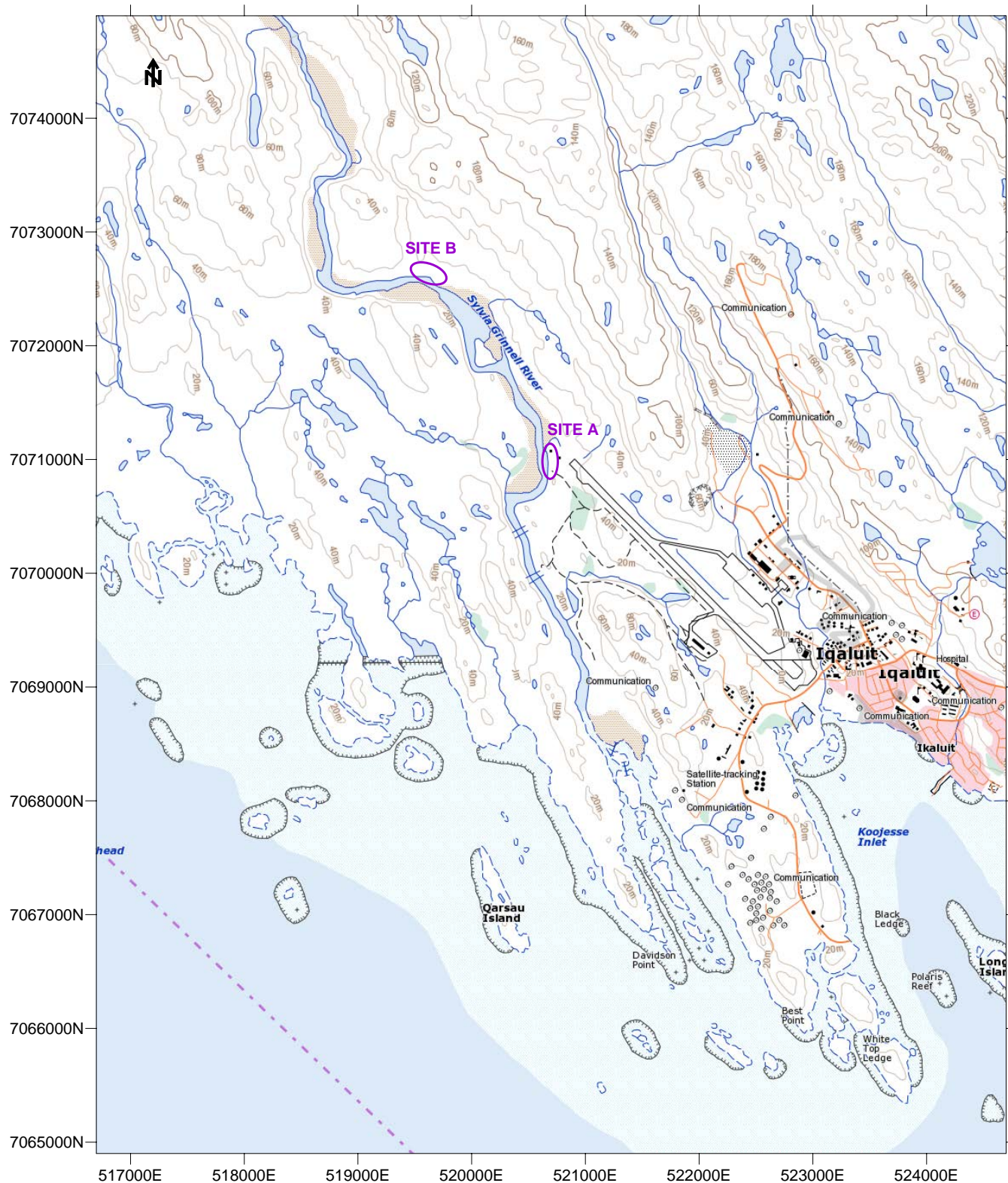
The depths to subsurface boundaries derived from seismic refraction surveys are generally accepted as accurate to within fifteen percent of the true depths to the boundaries. In some cases, unusual geological conditions may produce false or misleading data points with the result that computed depths to subsurface boundaries may be less accurate. In seismic refraction surveying difficulties with a 'hidden layer' or a velocity inversion may produce erroneous depths. The first condition is caused by the inability to detect the existence of a layer because of insufficient velocity contrasts or layer thicknesses. A velocity inversion exists when an underlying layer has a lower velocity than the layer directly above it. The interpreted depths shown on drawings are to the closest interface location, which may not be vertically below the measurement point if the refractor dip direction departs significantly from the survey line location. Structural discontinuities occurring on a scale less than the geophone spacing or isolated boulders would go undetected in the interpretation of the data. The seismic refraction method may not detect a narrow canyon-like feature incised into bedrock, if the canyon width is narrow relative to the depth of burial of the feature.

The information in this report is based upon geophysical measurements and field procedures and our interpretation of the data. The results are interpretive in nature and are considered to be a reasonably accurate representation of existing subsurface conditions within the limitations of the seismic refraction method.

For: Frontier Geosciences Inc.

Beth Galambos, P.Geo.

Cliff Candy, P.Geo.



0 500 1000 1500 2000
METRES

NATURAL RESOURCES CANADA
TOPOGRAPHIC MAPSHEETS
25N09, 25N10, 25N15 & 25N16

STANTEC CONSULTING LTD.
IQALUIT WATER INTAKE PROJECT

SEISMIC REFRACTION SURVEY

SURVEY LOCATION PLAN

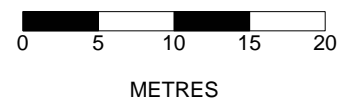
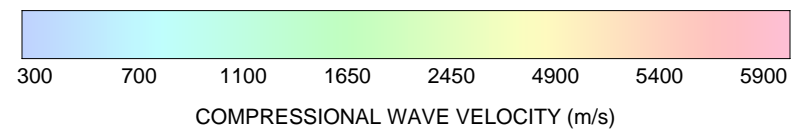
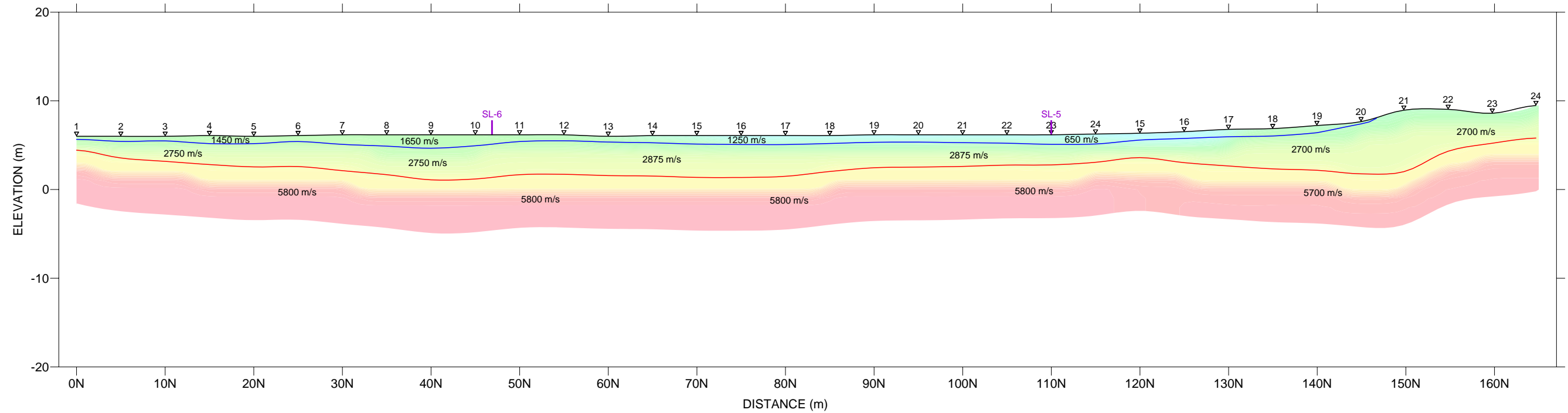
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DATE: OCT. 2018

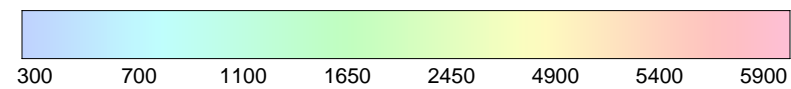
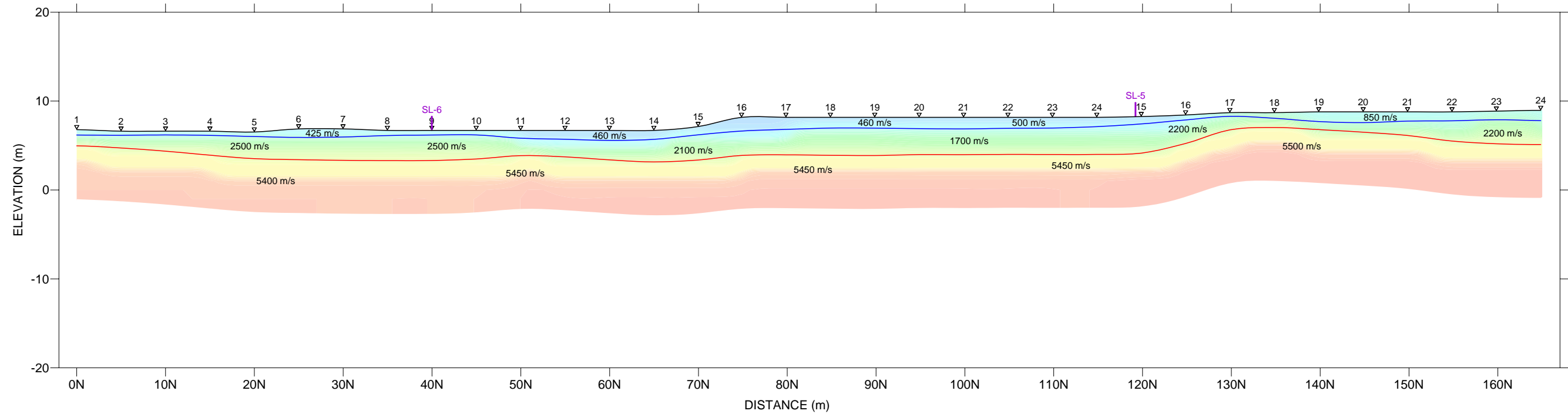
SCALE 1:50,000

FIG. 1

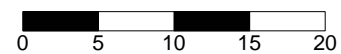




STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-3		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 3



COMPRESSIONAL WAVE VELOCITY (m/s)



METRES

STANTEC CONSULTING LTD.
IQALUIT WATER INTAKE PROJECT

SEISMIC REFRACTION SURVEY

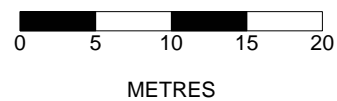
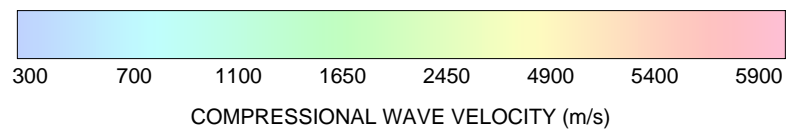
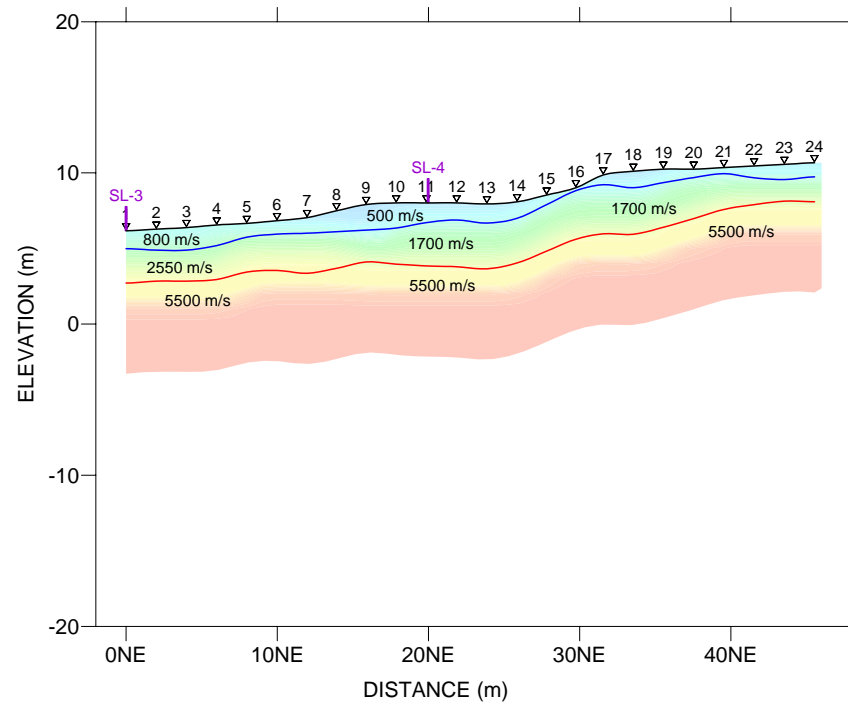
INTERPRETED SEISMIC PROFILE
SL-4

FRONTIER GEOSCIENCES INC.

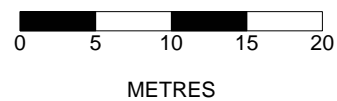
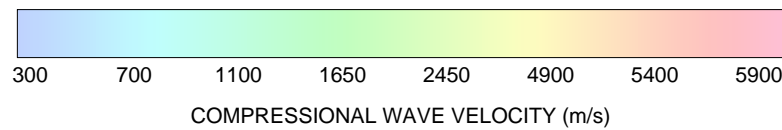
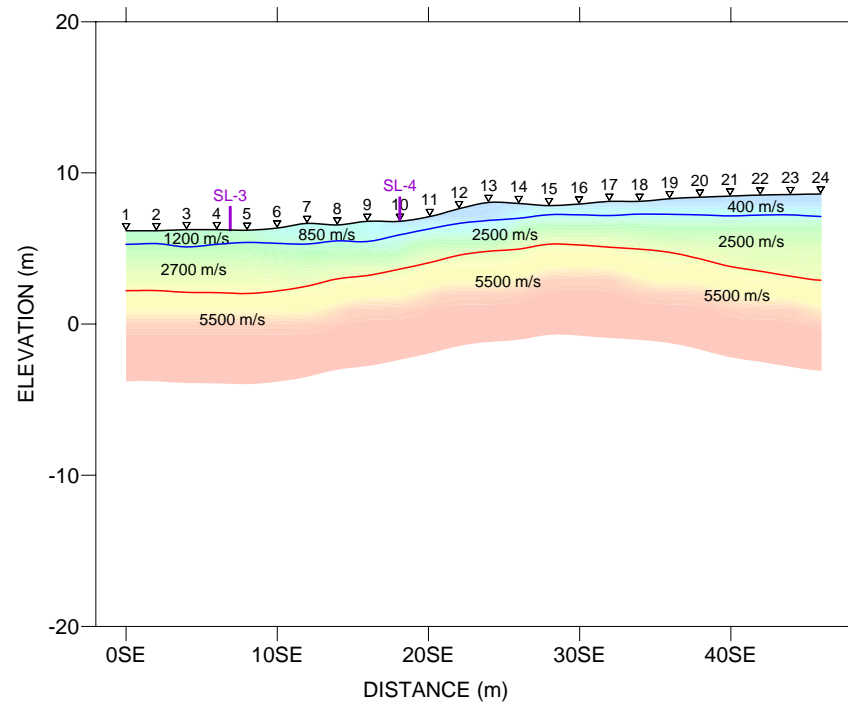
DATE: OCT. 2018

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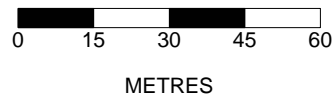
FIG. 4



STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-5		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 5



STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-6		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 6



STANTEC CONSULTING LTD.
IQALUIT WATER INTAKE PROJECT

SEISMIC REFRACTION SURVEY

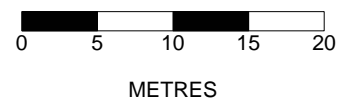
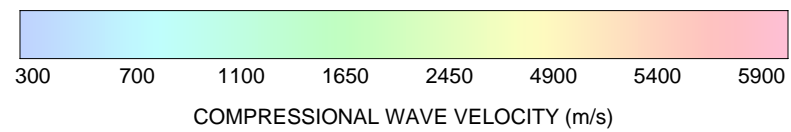
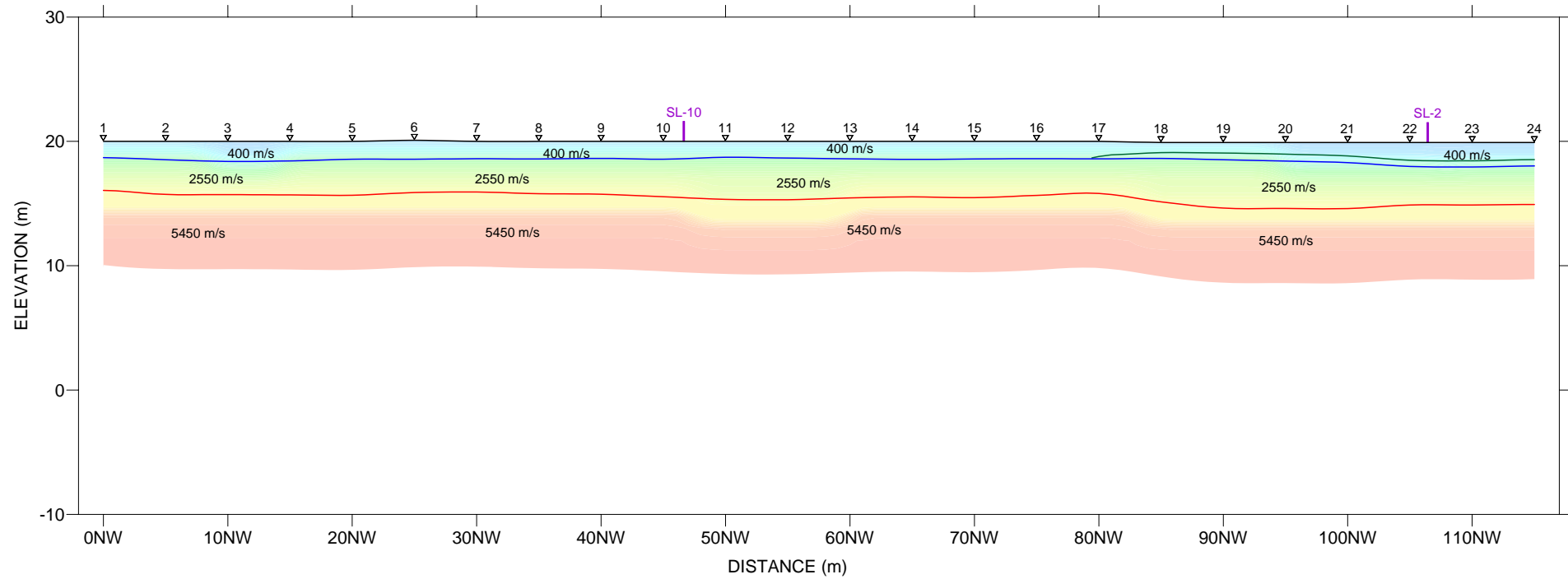
SITE B
PLAN

FRONTIER GEOSCIENCES INC.

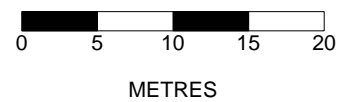
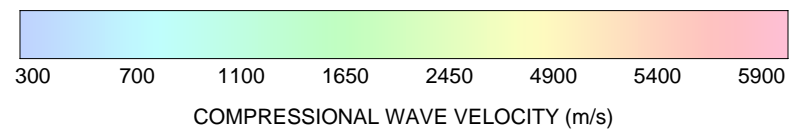
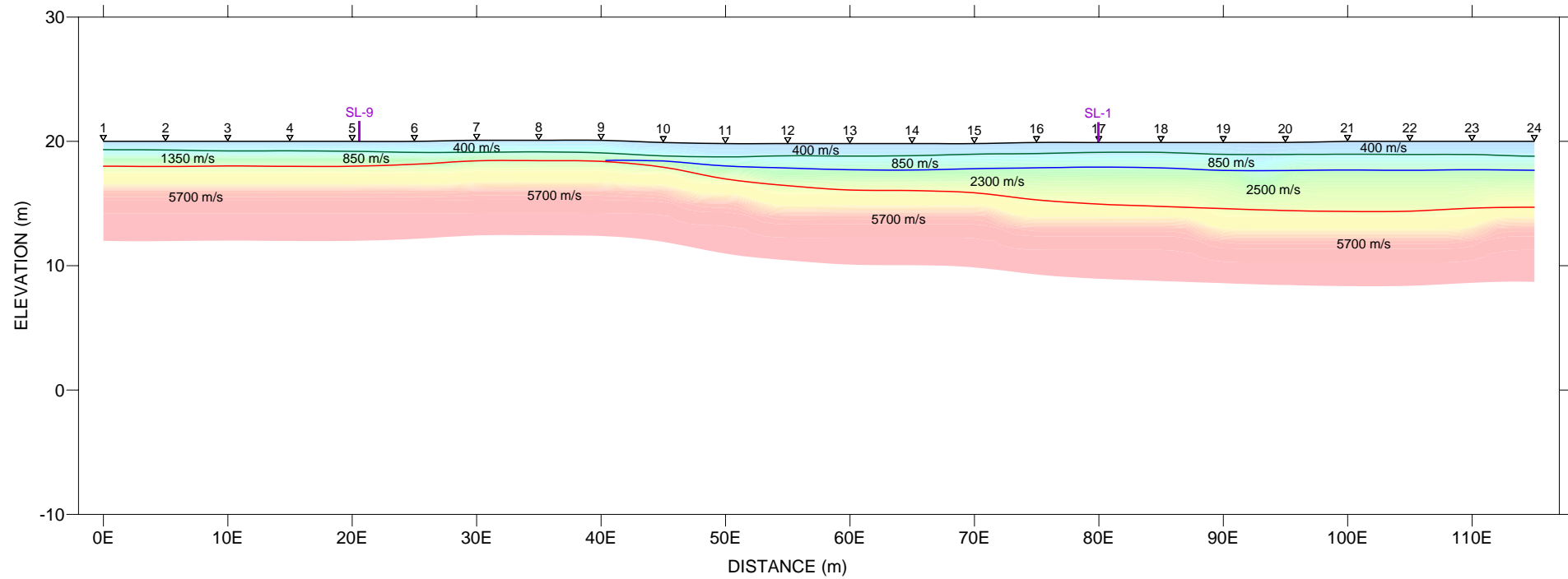
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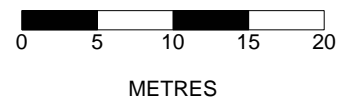
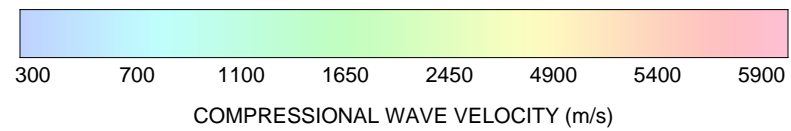
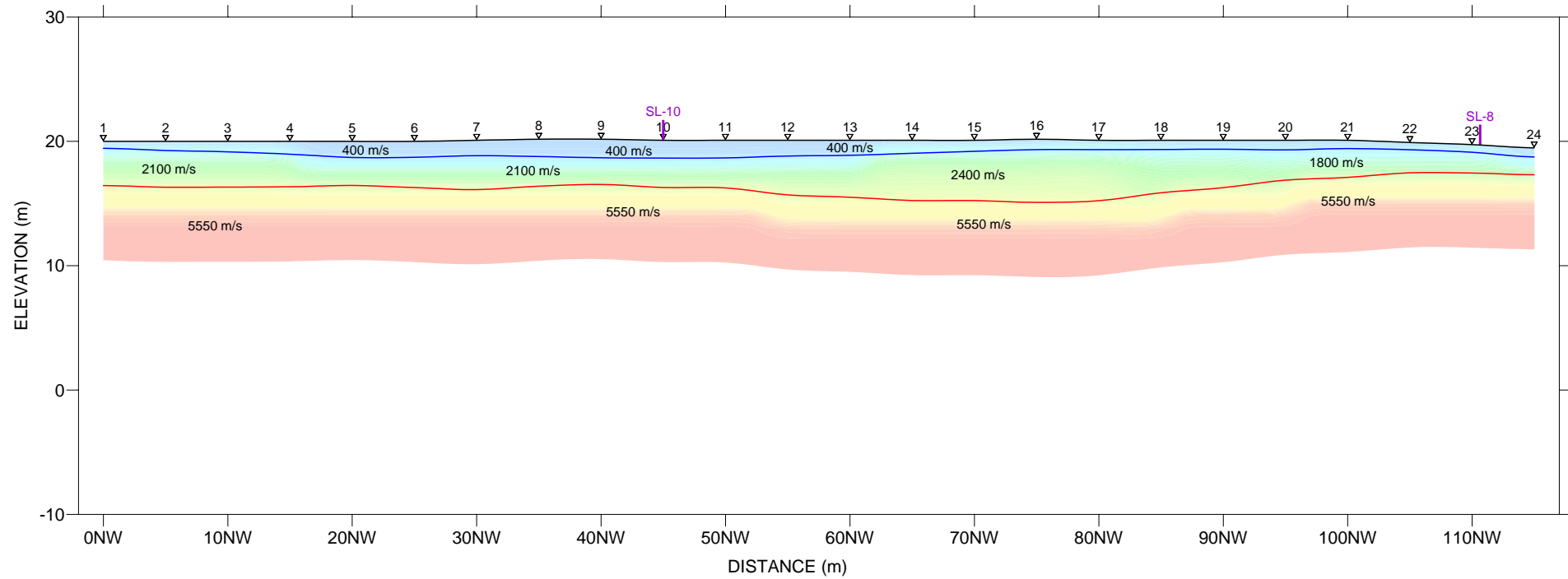
FIG. 7



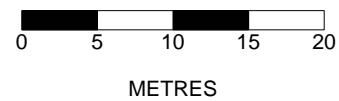
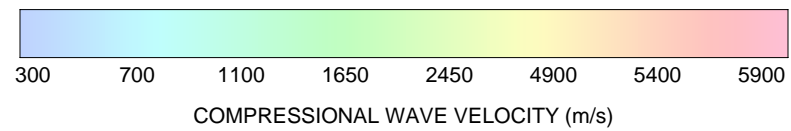
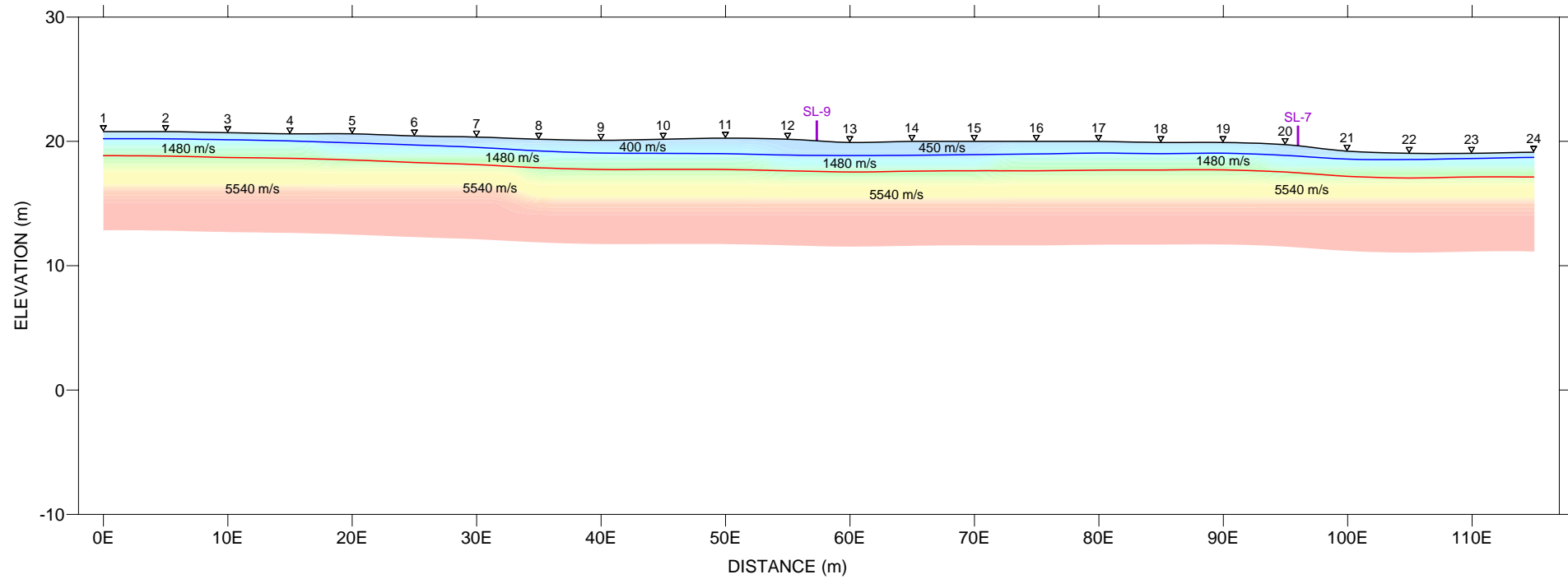
STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-1		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 8



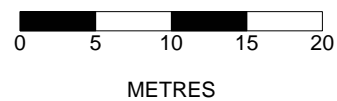
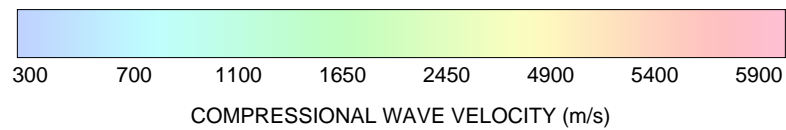
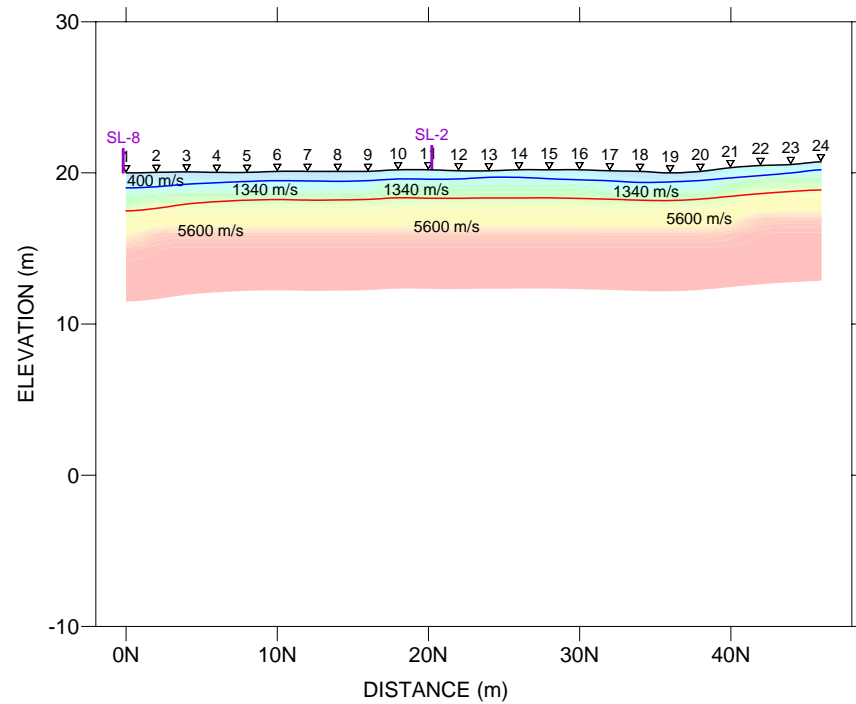
STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-2		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 9



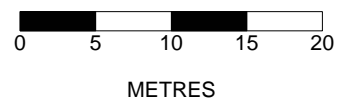
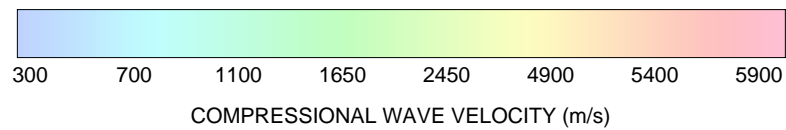
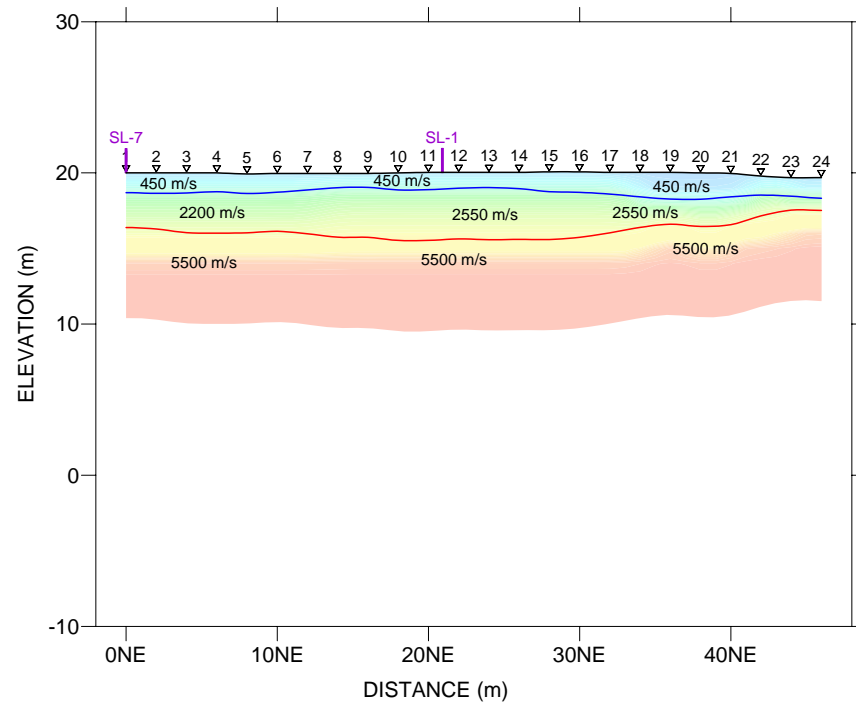
STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-7		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 10



STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-8		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 11



STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-9		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 12



STANTEC CONSULTING LTD. IQALUIT WATER INTAKE PROJECT		
SEISMIC REFRACTION SURVEY		
INTERPRETED SEISMIC PROFILE SL-10		
FRONTIER GEOSCIENCES INC.		
DATE: OCT. 2018	SCALE 1:500	FIG. 13