Fort St. James – Water Master Plan

March 30, 2023

Prepared for:

District of Fort St. James
477 Stuart Dr W
Fort St James, BC V0J 1P0

Prepared by:

Stantec Consulting Ltd.
205-1302 7th Avenue
Prince George BC V2L 3P1

File: 115821021

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Prepared by ______________________
(signature)

Chirag Shah, P.E., P.Eng.

Process Engineer – Water Treatment Process Evaluation

PROFESSIONAL
PROVINCE OF
S. R. BILBROUGH
24902
BRITISH COLUMBIA
ENGINEER

Prepared by ______________________
(signature)

Scott Bilbrough, P. Eng.

Office Leader, Prince George, B.C.

PROFESSIONAL
PROVINCE OF
D. C. S. FONSECA
31753
BRITISH COLUMBIA
ENGINEER

Reviewed by ______________________
(signature)

Cristina Fonseca, P.Eng.

Regional Water Practice Lead – Water Treatment Process Evaluation

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1.0 INTRODUCTION

The communities of Fort St. James and Nak'azdli are serviced by a community water system. The system draws water from a confined artesian groundwater aquifer. The system is currently serviced by a single groundwater well and a single water supply pump that fills a 2,278 cubic meter reinforced concrete reservoir at the east end of Stuart Drive East. Water from the aquifer is not treated.

The residents of Fort St. James and Nak'azdli desire a higher standard / Level of Service than is provided by the existing water system. Known areas of the desired improvement include:

- Water quality.
- Reliability and redundancy.
- Fire Protection.

The District of Fort St. James (District) retained Stantec Consulting Ltd. (Stantec) to prepare this Water Master Plan (WMP). The objective of the WMP is to assess the existing water treatment and distribution system components and provide a recommendation to improve the quality of the water provided, to increase the level of reliability / redundancy of critical components of the system and to review opportunities to increase the level of service provided to its users.

1.1 BACKGROUND REPORTS

In preparing this WMP report, we have reviewed the following background reports:

- Largest Flowing Well in Canada at Fort St. James, Johnson Drillers Journal, 1970
- Village of Fort St. James, Concept Design Report, Water System Improvements, Associated Engineering, 1982
- Village of Fort St. James, Water Study, L&M Engineering Ltd. 1992.
- District of Fort St. James Water Supply and Treatment Study, L&M Engineering Ltd., 2001.
- Nak'azdli Whute'en, Feasibility Assessment of Water Quality and Systems, David Nairne – Associates Ltd., 2019.

2.0 EXISTING WATER SYSTEM

The existing water distribution system servicing the Fort St. James (FSJ) area is shown in **Figure 2.1**. The FSJ distribution system includes pipe sizes ranging from 50 mm to 300 mm. The FSJ water distribution system also supplies water to the Nak'azdli service area, located south of the Kwah Road. **Figure 2.1** also shows the locations of the water supply well and pump station at Birch Street and the reservoir at the end of Stuart Drive East.

Water supply, pump station and reservoir components of the system are located within the District of Fort St. James municipal boundaries and are operated by District of Fort St. James public works staff. Water is provided to the Nak'azdli community via a number of connections to the water distribution system. The provision of drinking water from Fort St. James to Nak'azdli is provided for by a Municipal Type Servicing Agreement (MTSA) which has been in place since the 1980s. Each community maintains its own water distribution system network / piping.

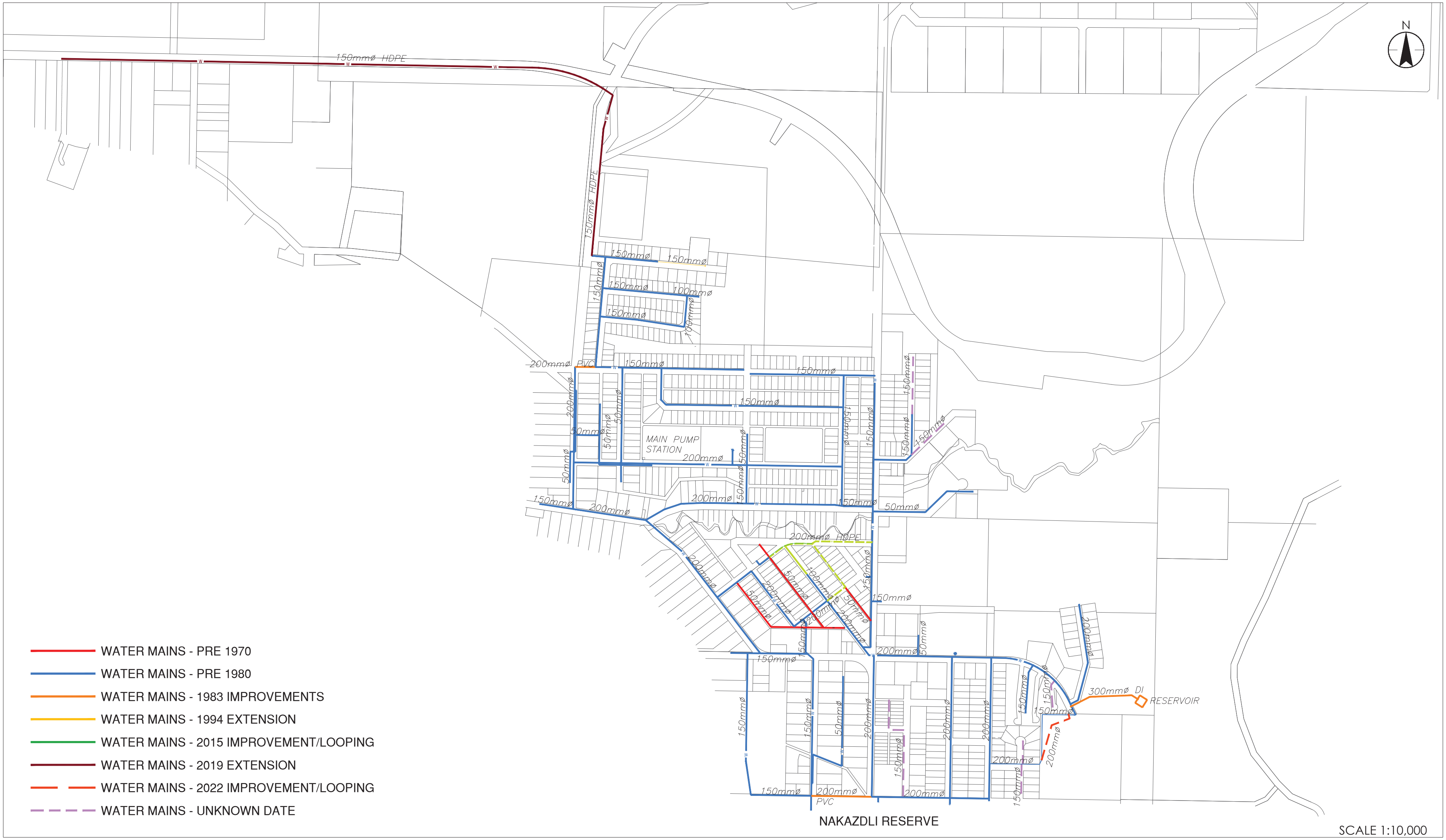
2.1 WATER SYSTEM HISTORY

Like many other communities in Northern BC, the area experienced significant growth and development in the 1960s, 1970s and early 1980s. This period included the design and construction of the District's municipal water supply and sewage treatment systems. Key milestones in the history of the water system include:

- 1939 – Drilling of multiple small diameter water wells / discovery of artesian aquifer
- 1940s / 1950s – 50 mm diameter low-pressure water distribution system
- 1969 – Drilling of Birch Street water supply well
- 1969 – Construction of the core water distribution system (150 mm and 200 mm diameter water mains)
- 1983 – Water system improvements (Birch Street pumphouse upgrade, Pineridge Way pump station, water storage reservoir construction)

Community growth has slowed since the 1980s. The slowing of growth has coincided with a reduced level of investment in water system infrastructure. Water system improvements / service extensions that have been implemented since the 1980s include:

- 2015 – Simon Fraser Avenue water main replacement / looping
- 2019 – Stones Bay Road water service extension
- 2003 – Birch Street Pumphouse Upgrading
- 2018 – Water System SCADA system installation



FT. ST. JAMES | FIGURE 2.1

EXISTING WATER SYSTEM

Stantec
Design with community in mind

115821021
May 21, 2021

In recent years, the communities of Fort St. James and Nak'azdli have confirmed a renewed desire for a higher standard / Level of Service than is provided by the existing water system. Known areas of the desired improvement include:

- Water quality;
- Reliability and redundancy; and
- Fire Protection.

2.2 WATER SUPPLY WELL

The existing water supply well was drilled in November 1969. The existing groundwater supply well has been evaluated a number of times in the past, including assessments by Golder, in 2001, and by Western Water Associates Ltd., in 2021 / 2022 as a part of this Water Master Plan. A copy of Western Water Associates report is included as **Appendix A** to this report.

Both assessments have found the existing artesian groundwater aquifer to be suitable as a long-term water supply for Fort St. James. Recommendations for drilling a new well include drilling a new well in relatively close proximity to the existing well (if it is to be used primarily for system redundancy / backup) or drilling a new well a distance away from the existing well if substantial additional water supply capacity is desired.

Given the substantial capacity of the existing well to provide for the forecasted water supply demands of Fort St. James, we expect that it may be most beneficial for a new well to be drilled at the same site as the existing well. In this manner, both wells can be connected to the water distribution system (and a proposed water treatment plant).

It is important to note that drilling a new water well into an artesian aquifer introduces risks that will be required to be managed via an engineered well drilling, testing, and development program. Recommendations for the proposed well drilling program are included in Western Water Associates report.

Western Water Associates report suggests that drilling and testing of a new well can be expected to cost approximately $0.5 million. Allowing for inflation, connection to the existing system and risks, the total cost of developing a new water supply well could approach $0.75 million by the time it is connected and ready for operation.

2.3 WATER DISTRIBUTION MAINS

The water distribution system includes mains of varying ages (and assumed conditions). The oldest pipes are 50 mm diameter plastic pipe that dates back to before 1969 (i.e.: 50 – 70 years of age). These pipes are too small / too old to be of any use. Over time, they have been replaced by larger-diameter mains. Where these pipes are still in service, they should be replaced as a part of a capital project replacement program.

The largest part of the system includes a network of 150 and 200-mm diameter asbestos cement water mains. Record drawings appear to suggest that these pipes were likely installed at or about the same time the main water supply well was drilled (1969). At nearly 55 years of age, these pipes are nearing the end of their useful service life. Determining the exact service life of buried infrastructure can be difficult. Given the importance of a reliable water system, and the significant capital cost required, we suggest that the District considers the planning process that is required to provide for replacing the existing water mains as a part of a phased capital project replacement program.

2.4 BIRCH STREET PUMP STATION

The original Birch Street pump station was constructed when the water supply well was developed in the late 1960s / early 1970s. It was upgraded in 1983, and again in 2003.

The station includes a single 30 Hp centrifugal pump that draws water from the head of the artesian well (static suction pressure approximately 50 psi, operating suction head approximately 42 psi @ 62 L/s) and pumps to the water storage reservoir via a connection to the water distribution system. The pump station has one pump connected to the station. A second pump is located within the pump station. Should the operating pump fail, the system will be out of operation while the public works staff attend to the site to switch out the replacement pump.

The pump is controlled via water level sensors at the water storage reservoir. As described in later sections of this report, the water distribution system relies on the pump being “on” during a fire event in order to provide increased volumes / pressure of water for fire protection. It may be advantageous to modify the mode(s) of operation to include controls that will automatically turn the pump “on” in the event of a fire.

2.5 STORAGE RESERVOIR

The existing reservoir was built in 1983 and is a 22.5-meter diameter reinforced concrete earth-covered tank with a capacity of 2,278 m^3. As described in later sections of this report, the existing reservoir is expected to provide a sufficient volume of storage for water demand balancing and fire protection (up to commercial flows of 150 L/s, not including allowances for higher industrial-type land use). Its single-cell design does not provide redundancy, should it need to be taken out of service for repair or maintenance.

The reservoir location, on the far east side of the District’s developed area, has the effect of limiting the water system hydraulic grade line. This is also a result of a system that includes a large number of smaller diameters (150 and 200 mm) water lines / no dedicated water transmission main.

As will be illustrated in concept design plans in later sections of this report, options to ensure adequate water (flow / pressure) for fire protection include:

- Increasing water main sizes in a number of locations across the community.
- Ensuring that any future improvements (water supply and treatment) include allowances for additional water storage and a fire pump at the water treatment plant.

For the purpose of this Water Master Plan and budgetary planning, the concept designs and cost estimates contained herein assume that a newly drilled water well and water treatment plant will include provisions for additional water storage and a water booster pump / fire pump at the water supply / water treatment plant site.

2.6 PINERIDGE WAY BOOSTER STATION

The Pineridge Way booster station was constructed in 1983, as a part of the 1983 Water System Improvements project. The station is required to ensure that adequate pressure (Peak Hour Demand) and water for fire protection can be provided to homes along Pineridge Way; these properties are located at elevations that are too high to be provided with water from the storage reservoir without pumps to booster pressure / flow.

Given its limited service area, improvements to the Pineridge Way booster station are expected to be of lower priority and may be limited to the continued operation and maintenance type efforts.

2.7 WATER QUALITY

The existing water source currently supplies untreated groundwater and fulfills the provincial requirements for potability based on the available information. The source water has been assessed as having a low risk of containing pathogens and is anticipated to remain so unless routine monitoring of bacteriological indicators or other relevant insights emerge to suggest otherwise. All chemical water quality parameters assessed meet the health-based limits of the *Canadian Guidelines for Drinking Water Quality*, prepared by Health Canada. The source is thus deemed potable without further treatment.

Notwithstanding its meeting health-based limits, the water does not meet the aesthetic objectives of the community. Treatment is desired; providing a water treatment feasibility / concept design and Class C Cost Estimates is a key objective of this Water Master Plan.

2.7.1 Raw Water Quality and Treated Water Quality Targets

Historically, it has been reported that many consumers have complained about the aesthetic quality of the water received. The reported raw water quality suggests that most parameters of concern affect aesthetics and do not pose a significant risk to consumer health; these parameters can be mitigated by appropriate water treatment to improve consumer satisfaction. **Table 2.1** summarizes the raw and treated water quality targets.

In addition to improving aesthetics and consumer satisfaction, treatment may offer additional benefits including improved preservation of water quality during distribution and storage, which benefits system maintenance.

Table 2.1 - Source Water Quality and Proposed Finished Water Objectives

Parameter	Unit	MAC/AO/OV[1]	Source Water	Finished Water Objective
Iron	mg/L	0.3	0.7	< 0.05
Manganese	mg/L	0.02	0.03	< 0.01
Hardness	mg/L as $CaCO_3$	None	330	100 to 125[2]
Calcium	mg/L as $CaCO_3$	None	125	-
Magnesium	mg/L as $CaCO_3$	None	205	-
TDS	mg/L	<500	370	<500
Ammonia	mg/L	<0.01	0.1	<0.01
H_2S	mg/L	<0.001	0.01	<0.001
Arsenic	mg/L	<0.010	0.007	<0.005
Turbidity	NTU	<0.3	5	<0.3
TOC	mg/L	<4	< 0.5	<4
Colour	TCU	<15	< 5	<15
pH	-	7 – 10.5	8.0	7 – 10.5
Alkalinity	mg/L as $CaCO_3$	>30	300	>30
Sodium	mg/L	< 50	14	< 50
Potassium	mg/L	-	3	-
Sulphate	mg/L	<500	55	<500
Chloride	mg/L	<250	< 0.5	<250

1. MAC – Maximum acceptable concentration; AO – Aesthetic Objective; OV – Other Value
2. Target applicable only If softening treatment is selected; under the Guidelines for Canadian Drinking Water Quality hardness does not have to be within a specific range.

Based on the available source water information, and end-user complaints, several potential issues are identified. These include potential water discoloration events, possible staining of plumbing fixtures and clothing, intermittent odour issues, and the buildup of scale on plumbing, particularly hot water surfaces. Based on the parameters listed in **Table 2.1,** these issues could be associated with the presence of iron, manganese, hydrogen sulphide, and moderate hardness.

The total dissolved solids (TDS) level is relatively high, and care should be made such that any treatment does not cause an unacceptable increase and impair the taste of treated water.

Other water quality parameters of note include a slight presence of ammonia, negligible total organic carbon (TOC) and colour, and significant alkalinity to buffer the water around pH 8. Elevated turbidity levels reported previously are suspected of being due to the oxidation of iron by air occurring between sampling and prior to analysis, i.e., during transport to the certified laboratory. The presence of particulate iron and its impact on turbidity requires verification.

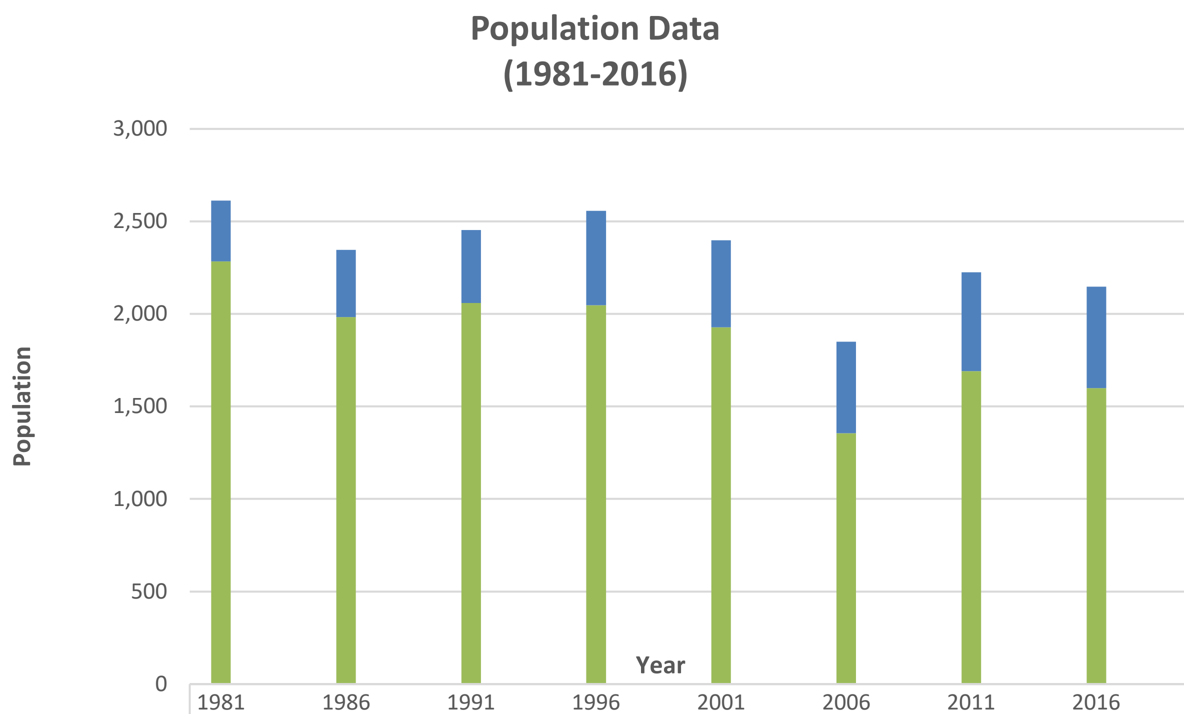
3.0 WATER DEMAND

A cost-effective water distribution system catering to the community's needs requires an accurate estimate of the water usage, population forecast, and their respective water demands. For this purpose, analysis of population data for the District and Nak'azdli and the water well pump records received from the District have been performed and presented in the following sections.

3.1 EXISTING POPULATION

Stantec analyzed the population data obtained from Statistics Canada for the past 35 years, presented in **Figure 3.1**. As it generally happens with the growth of small communities, there is no consistent pattern in the District's population growth. The District has seen years of growth as well as a reduction in the population. The total population of Fort St. James and Nak'azdli as per the 2016 census was 2,146, which is about 16% less than the 1996 census's population data (2,556).

Figure 3.1- Historic Population for the District of Fort St. James and Nak'azdli



	1981	1986	1991	1996	2001	2006	2011	2016
Total Population	2,611	2,346	2,453	2,556	2,397	1,850	2,225	2,146
Necoslie 1	327	363	395	510	470	495	534	548
Fort St. James	2,284	1,983	2,058	2,046	1,927	1,355	1,691	1,598

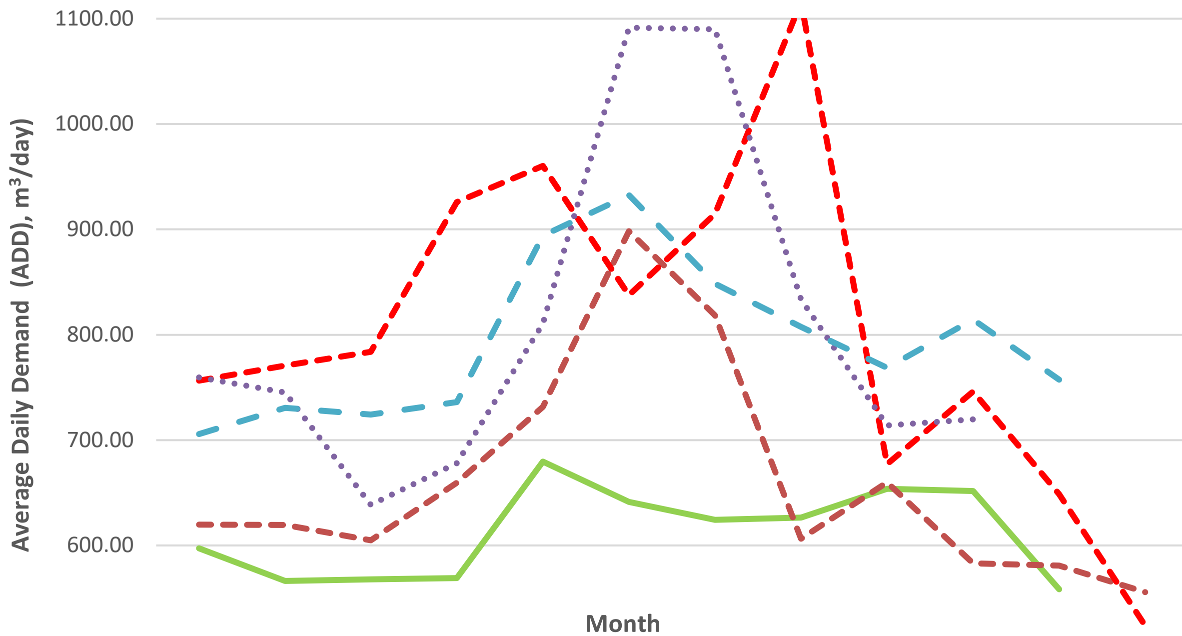
Source: Statistics Canada 1981-2016 Census

3.2 EXISTING WATER DEMAND

Stantec analyzed the water well pump data records from January 2016 to November 2020, provided by the District, to estimate the existing Average Day Demand (ADD) in m^3/day as shown in **Figure 3.2.**

It is noted that the water demand in the year 2020 has decreased significantly from the previous year's water demand. Water demand is a function of the population, seasonal demand, length of the rainy season and the number of fire incidents. Since the population record for the year 2020 and other data are currently unavailable, it is difficult to comment on the exact reason behind this change in water demand.

Figure 3.2 - Historic Water Demand



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2020	597.43	566.42	568.03	569.10	679.79	641.33	624.33	626.57	653.97	651.80	558.52	
2019	619.77	619.43	604.90	659.69	731.43	898.46	818.40	606.53	659.89	583.10	580.83	555.83
2018	756.53	770.57	783.72	925.71	960.06	837.52	914.76	1115.87	677.12	746.06	648.77	524.14
2017	759.43	745.11	638.65	678.08	810.48	1091.31	1089.77	833.60	713.87	719.72		
2016	705.77	730.69	724.19	735.97	893.83	932.62	848.29	807.40	768.69	814.42	757.40	

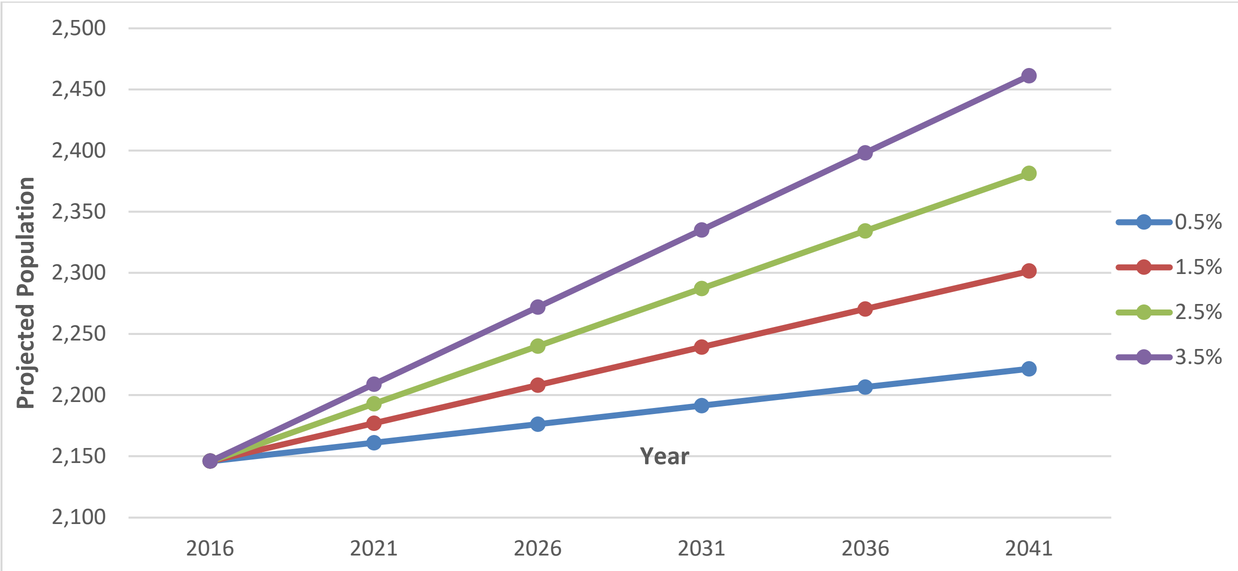
Source: Existing Water Well Pump Records (Jan 2016 – Nov 2020)

ADD for 2016 from the water well record data is found to be 793 m^3/day. The total population in the same year was 2,146, hence per capita demand in 2016 is estimated to be 369 or ~370 Liters per capita per day (Lpcd).

3.3 POPULATION FORECAST

Past census population data has been used to estimate the future population of the District at the end of the 20-year design period of the water distribution system. For the purpose of forecasting future population, growth rates varying from 0.5% to 3.5% for the District and a growth rate of 1.3% for the Nak'azdli have been considered in the calculation. Considering the year 2016 as the base year, population forecast calculations are presented in **Figure 3.3**.

Figure 3.3 - Population Forecast



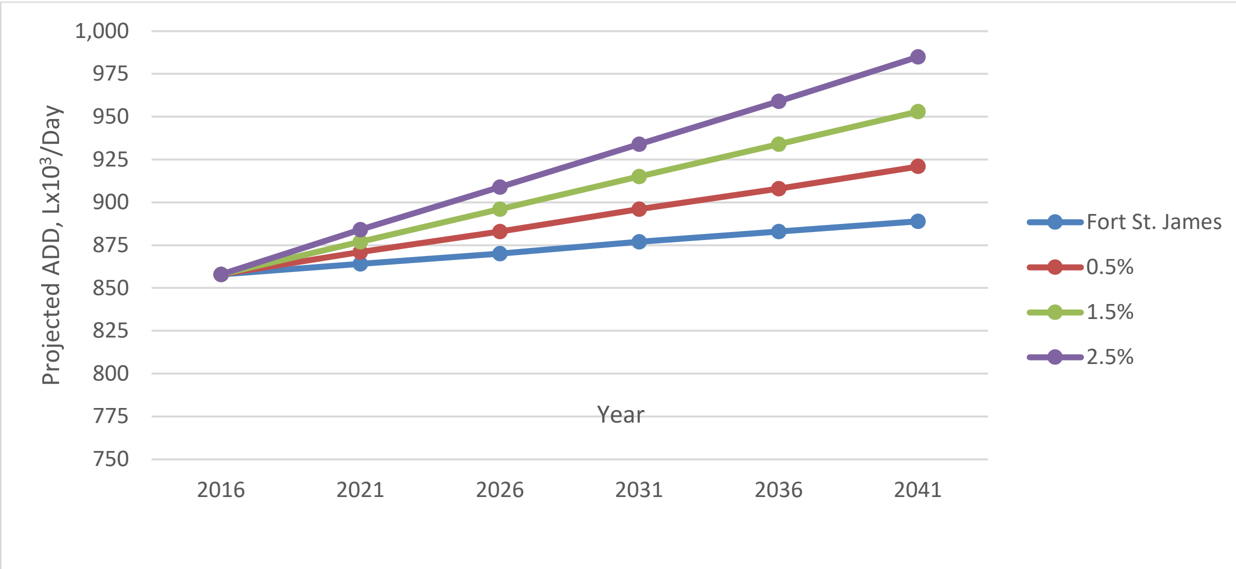
Population Forecast (Base Year - 2016)							
Growth Rate		Year					
Fort St. James	Necoslie 1	2016	2021	2026	2031	2036	2041
0.5%	1.3%	2,146	2,161	2,176	2,191	2,206	2,222
1.5%	1.3%	2,146	2,177	2,208	2,239	2,270	2,301
2.5%	1.3%	2,146	2,193	2,240	2,287	2,334	2,381
3.5%	1.3%	2,146	2,209	2,272	2,335	2,398	2,461

With a high growth rate of 3.5%, the estimated population in 2041 will be 2,461 whereas a low growth of 0.5% will lead to 2,222.

3.4 WATER DEMAND FORECAST

Based on the growth rates assumed in the population forecast section and a per capita demand of 400 Lpcd, water demand has been calculated and presented in **Figure 3.4.**

Figure 3.4 - Water Demand Forecast



Projected Average Day Demand (ADD), m³/day (Base Year - 2016)								
Growth Rate		**Water Demand**	**Year**					
Fort St. James	Nak'azdli	Lpcd	2016	2021	2026	2031	2036	2041
0.5%	1.3%	400	858	864	870	877	883	889
1.5%	1.3%		858	871	883	896	908	921
2.5%	1.3%		858	877	896	915	934	953
3.5%	1.3%		858	884	909	934	959	985

With a low growth rate of 0.5%, ADD in 2041 is estimated to be 889 m³/day whereas a growth of 3.5% will lead to 985 m³/day.

For the water distribution analysis, the upper growth rate of 3.5% has been used as the pipe sizing requirements will be governed by the fire flow requirements. For the Maximum Day Demand (MDD) and Peak Hour Demand (PHD), peaking factors of 2.5 and 4.0 have been used, respectively. The projected water demands for the year 2041 for the high and low growth requirements are summarized in **Table 3.1**.

Table 3.1 - Projected Water Demands for 2041

Demand	Low Growth Rate (0.5%)		High Growth Rate (3.5%)	
	m^3/d	L/s	m^3/d	L/s
ADD	889	10.3	985	11.4
MDD	2,223	25.7	2,463	28.5
PHD	3,556	41.2	3,940	45.6

4.0 DRINKING WATER TREATMENT EVALUATION

4.1 PRIORITIZATION OF TREATMENT TARGETS

Table 4.1 prioritizes treatment based on impact on the community and the water quality summary presented in **Table 2.1**; the presumed priority is based on engineering judgment and requires confirmation from community insight and historical experience.

Table 4.1 - Prioritization of Treatment Targets to Mitigate Aesthetic and Nuisance Effects

Priority	Parameter	Aesthetic and Nuisance Effect
1	Iron	Red-brown water, staining of plumbing fixtures and laundry
2	Manganese	Black, brown, or greyish water, staining of plumbing fixtures and laundry
3	Hydrogen Sulphide	Rotten egg-like smell
4	Hardness	Buildup of scale in boilers, water heaters, washing machines, and appliances that use or come into contact with hot water. Higher consumption of soaps and detergents.

Iron occurs at a level slightly over twice the aesthetic objective of 0.3 mg/L and is anticipated to cause, or potentially cause, consumer complaints due to discoloration of the water and staining, as well as nuisance effects related to the growth of biofilm largely composed of iron-reducing bacteria in the distribution system. It is thus given the highest priority in terms of treatment.

Manganese occurs at a level slightly above the aesthetic objective of 0.02 mg/L and well below the Health Canada health-based guideline of 0.12 mg/L. However, the removal of manganese is advised based on the tendency for manganese to accumulate in the distribution system due to settling of particulates, adsorption to dissolved manganese on pipe surfaces and scale, and the incorporation and buildup of manganese into biofilm on pipe surfaces. Subsequent changes in hydraulic and / or chemical conditions within the distribution system may periodically result in the release of manganese and increased consumer complaints due to potentially intense water discoloration events. Such events may occur even when source levels are low, due to the continuous loading of the distribution system and premise plumbing with significant accumulation over time. Such deposits are relatively cohesive and not easily removed by conventional flushing, even at high velocities. More aggressive cleaning is required to remove manganese deposits, such as foam pigging. Manganese removal is thus given the second highest priority following iron.

It should be noted that any treatment applied to remove iron and manganese will reduce the loading of these metals to the system but will not address the accumulated legacy deposits. Legacy iron and manganese must be removed from the distribution and storage system by appropriate cleaning procedures prior to a significant change in finished water chemistry and flow patterns.

The occurrence of periodic customer complaints of odour described as “rotten egg” is commonly associated with hydrogen sulphide (H_2S), potentially produced by sulphate-reducing bacteria in the absence of dissolved oxygen. In addition to the odour, such bacterial growth may initiate the corrosion of metal surfaces and react with iron to form black deposits. This issue may originate in the source water or within the distribution system and may be mitigated by modifications to the operation and maintenance of both the well and the distribution system. In addition to well and distribution system considerations, common treatment processes addressing iron and manganese will also serve as a barrier to H_2S formation. For this reason, the removal of H_2S by treatment is given priority after iron and manganese. It should be noted that the occurrence and causes of H_2S in the finished water should be confirmed and mitigated to the fullest extent possible by modification of operation and maintenance practices.

The fourth priority treatment objective addresses water hardness. Commonly, in North America, water with a hardness of 300 mg/L as $CaCO_3$ or greater is classified as “very hard”. While the current source for Fort St. James may be considered very hard, in the context of public health protection, there is no evidence that hard water causes any adverse effects or provides any benefits. Hard water is just as satisfactory for human consumption as soft water. Reasons for providing community-wide softening at a central treatment facility are based primarily on economics and to a lesser extent aesthetic grounds.

Two reasons are most commonly cited for removing hardness. The first is to reduce the precipitation of scale on surfaces in contact with hot water, such as boilers, water heaters, and appliances, thereby improving their efficiency and extending their useful life. The second is to reduce the consumption of soaps and detergents. While households may benefit from softening in large metropolitan areas, due to the size of the community, softening is not anticipated to provide an economic benefit for either motive in Fort St. James. The average life of a conventional hot water heater in the United States was reported by the U.S. Department of Energy in 2010 to be approximately 13 years, with replacements not necessarily due to failure. On a net present value basis, the cost of centralized softening is predicted to be at least an order of magnitude higher than the cost of community-wide water heater replacement over the same period assuming an average useful life of 13 years. The useful life of a water heater depends not only on the hardness of the water but on the type of heater and the maintenance it receives. For these reasons, community-wide removal of hardness has been given the lowest priority for treatment.

Though ammonia is neither a health nor aesthetic concern in the finished water, it should be removed for operational reasons, specifically if a free chlorine residual is maintained in the distribution system, in order to minimize interference with residual stability. Ammonia will be oxidized by sodium hypochlorite during normal treatment and thus should not present an issue.

To summarize, based on this preliminary evaluation the removal of iron, manganese, and H_2S is given the highest priority. Verification of source water quality is recommended as a preliminary step before any further detailed studies are conducted. Discussion and confirmation of treated water objectives with stakeholders are recommended to align community needs and known historical issues with the assumed prioritization of **Table 4.1**.

4.2 WATER TREATMENT DESIGN FLOW

The design of a treatment plant facility must provide adequate treatment capacity to accommodate anticipated demands. Additional capacity is required to compensate for operational water needs or mechanical downtime that does not contribute to the production of finished water, such as backwashing of filters or maintenance activities. The size of the treatment facility is dictated by process capacity. The minimum process capacity is based on meeting the maximum day demand forecasted for the planning horizon of 2041, with an assumed value of 28.5 L/s. The planning level capacity must include treatment options that are able to supply at least 28.5 L/s (2.5 ML/d) of treated water to the community. The raw water needs will vary depending on the residuals generated within the process, i.e., filter backwashing.

The treated water design flows assumed for the sizing of our water treatment system are summarized below and were developed based on the evaluation summarized in **Section 3.0**.

- Average Day Demand (ADD): 15 L/s
- Maximum Day Demand (MDD): 30 L/s
- Peak Hour Demand (PHD): 45.6 L/s
- Fire Flow (FF): 90 L/min for 120 minutes

4.3 TREATMENT PROCESS OPTIONS

Treatment processes potentially capable of addressing water quality priorities and finished water objectives are briefly reviewed in this section and prioritize iron, manganese, and hardness. The dedicated removal of iron and manganese is a common treatment involving groundwater systems and may be achieved by the use of granular media filtration using one or more modes of operation. The common modes include oxidation / filtration, sorption / filtration, biological and reverse osmosis.

The most common method is oxidation / filtration, involving the addition of an oxidant to convert the soluble metal to a less soluble particulate form, followed by the capture of the particulate in the filter. Common oxidants include air (oxygen), sodium hypochlorite, and potassium permanganate; anthracite and silica sand are common filter media utilized. The oxidation rate for manganese is relatively low when compared to that of iron, often requiring the use of stronger oxidants (such as potassium permanganate) and extended reaction times prior to filtration to achieve adequate removal efficiencies. To overcome the low reaction rate of manganese, sorption / filtration is used, whereby filter media is selected such that soluble manganese readily sorbs to the surfaces of grains where it is effectively immobilized. The addition of an oxidant to the feed water, usually continuously, may then achieve oxidation of manganese to the less soluble oxide which is retained by the filter. This process is also known as “catalytic filtration”. Catalytic media for such an application is often referred to as “greensand”, though may involve proprietary media under other trade names. A top layer of anthracite is usually used to remove particulates not requiring catalytic oxidation, thereby reducing the load to the catalytic media, and increasing filter capacity and filter run times.

Process similarities readily allow the combination of oxidation and sorption into a single unit and will be referred to as oxidation / sorption / filtration. Such a process will generally remove iron by oxidation / filtration and manganese by sorption / filtration. The oxidant of choice is chlorine, in the form of bulk sodium hypochlorite, which avoids problems with overdosing permanganate and also provides a disinfectant residual for water entering the distribution system. Backwash wastewater is normally disposed of via the sanitary sewer.

Another common process for the removal of iron and manganese involves biological filtration, a common approach in Europe that has been increasingly used in North America over the past twenty years. This process offers simplicity and low cost. Although biological filtration is known to be sensitive to temperatures below 5^{o} C, a system already exists in Burns Lake that is meeting water quality targets for arsenic and manganese. Discussions with the Village Superintendent indicate this process to be extremely successful at removing target water quality compounds.

The use of ion exchange allows the removal of hardness (calcium and magnesium), as well as iron and manganese when using cation exchange resin. Two types are possible, including strong cation exchange resin for the exchange of divalent cations with sodium, and weak cation exchange resins that exchange the divalent metals using an acid. In both cases, the resin must be periodically regenerated and the spent regenerant solution properly disposed of. In the case of strong cation exchange resins, the TDS of the softened water is increased by the addition of sodium or potassium, with a possible adverse effect on the taste of the finished water. In the case of weak cation exchange resins, large quantities of strong acid are required for resin regeneration and large quantities of base are required to neutralize the spent regenerant. In addition to the high costs of these chemicals, very large and specially lined tanks are required for their storage, and operators will be exposed to an additional hazard during bulk handling, supply, and storage. Initial contact with local suppliers resulted in considerable volumes of salt (i.e., NaCl) being required to regenerate commonly used resins making it impractical for treatment at Fort St. James. For these reasons, ion exchange at this scale is not recommended for a municipal water treatment application and it will not be considered further for Fort St. James, as part of the potential treatment options.

An alternative treatment approach that could be used at Fort St. James includes nanofiltration (NF) or reverse osmosis (RO) membranes. These membranes are very efficient at removing iron, manganese, hardness, as well as most other parameters listed in **Table 4.2**. In the case of RO, depending on the type of membranes, most dissolved contaminants present in the water will also be removed. However, these membranes generate up to 25-30% residuals requiring disposal (normally to the sanitary sewer). Also, because they remove most water compounds the treated water is rendered unstable requiring blending with water treated to a lesser extent prior to final disinfection.

Although there are reported facilities using NF or RO as a direct treatment to groundwater, i.e., without upstream pre-treatment, this application must be verified for Fort St. James; the fouling potential of the water and the capacity of commonly used antifoulants to work efficiently must be confirmed with bench-scale tests and pilot-scale testing. It is too risky to assume NF or RO can be used as direct treatment to groundwater, otherwise.

A summary of the removal capabilities for the highlighted treatment processes, and the key benefits and disadvantages are provided in **Table 4.2**. Treatment options are subject to verification by on-site pilot testing and related investigations.

Table 4.2 - Process removal capabilities for finished water quality parameter priorities

Treatment Process	Fe	Mn	H_2S	Hard.	Key Benefits	Key Disadvantages
Oxidation/ Sorption/Filtration	F	F	F	N	• Simplicity • Low cost	• No softening
Biological Filtration	F	F	N	N	• Simplicity • Low energy • Minimal residuals production • Low cost	• No softening • Cold water temperatures may impact performance.
Ion Exchange	F	F	N	F	• Simplicity	• Added cost compared to media filtration • Increase in TDS Levels • Challenges with brine disposal
RO/NF Membrane Filtration	F	F	P	F	• Removes all targeted parameters	• Higher cost and complexity • Challenges with concentrate disposal • All contaminants must be dissolved to avoid fouling

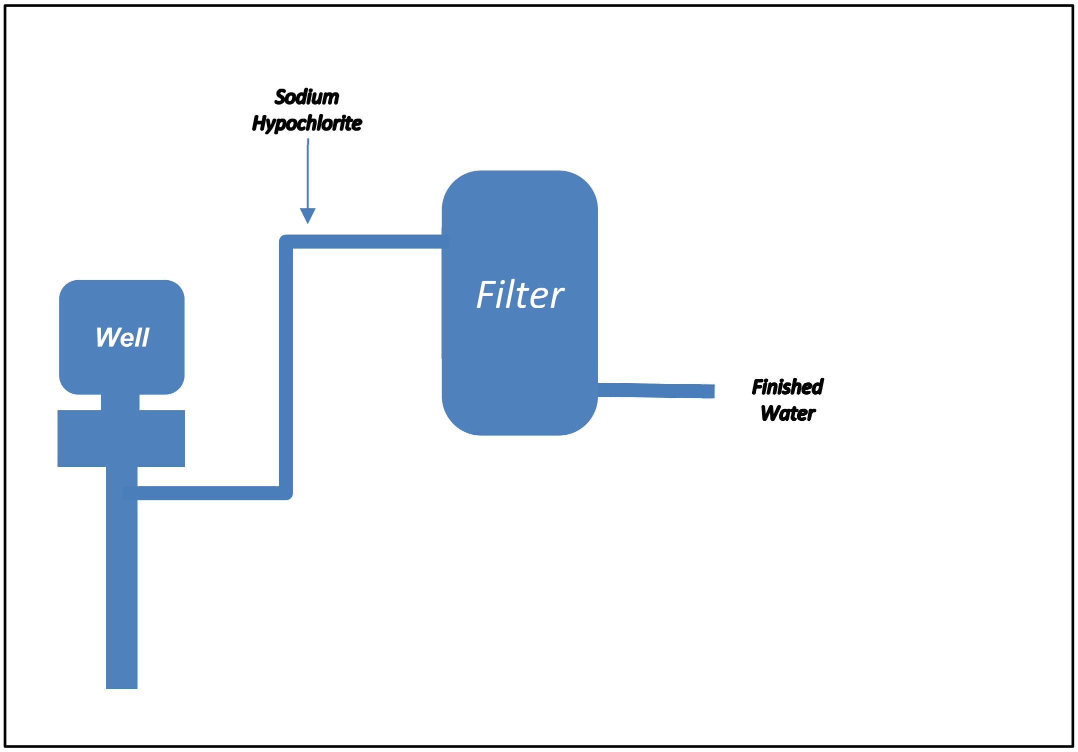
* F = full or substantial removal, P = partial removal, N = no or negligible removal

Three treatment options are suggested for consideration based on presumed finished water quality priorities, cost, and technical complexity.

4.3.1 Option 1 – Pressure Filtration

Pressure filtration is provided using dual media of anthracite on top of greensand or equivalent catalytic media or similar type media. Chlorine, in the form of bulk sodium hypochlorite, is provided as a pre-oxidant with a residual maintained in the effluent. Filters are backwashed at a frequency of approximately once per day. Air can also be applied to each filter during a backwash to assist in media cleaning. Catalytic media is anticipated to be replaced at a frequency of approximately once per ten years. A conceptual process diagram is provided in **Figure 4.1**.

Figure 4.1 – Oxidation / Sorption / Filtration Treatment (Option 1)



Based on its advantages and disadvantages, this option should be further considered as part of the treatment alternatives for Fort St. James.

4.3.2 Option 2 – Pressure Media Filtration and RO Membrane Filtration

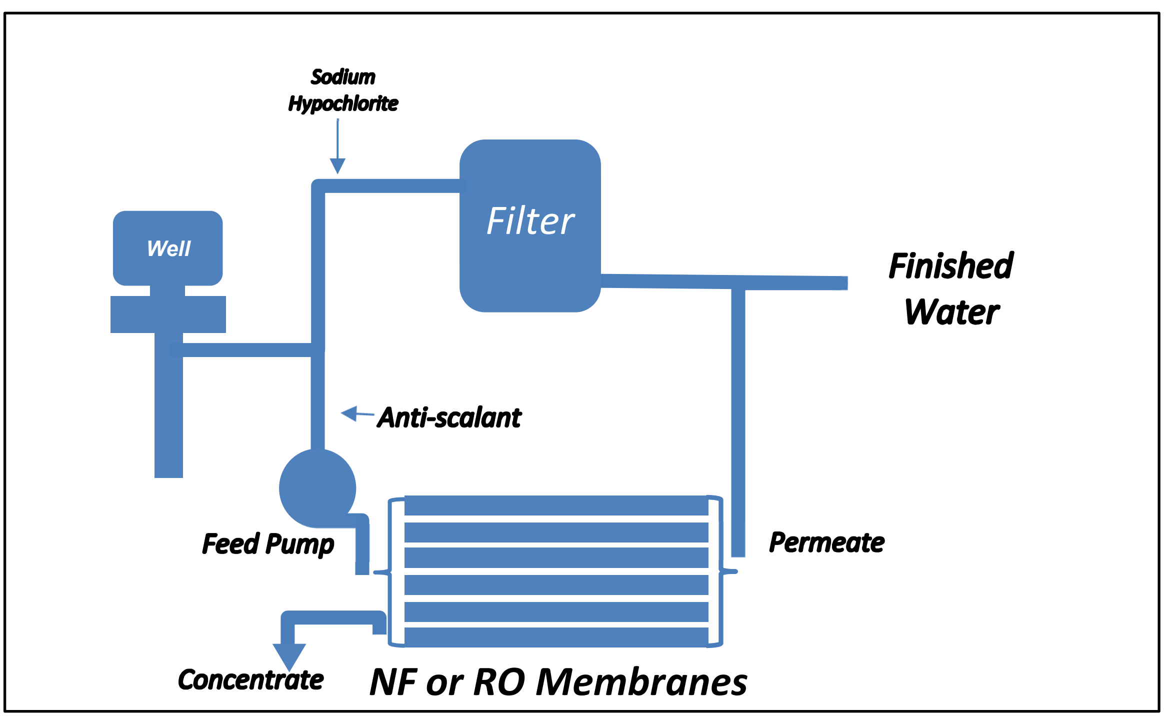
Option 2 assumes independent treatment of two streams as summarized below:

- Stream 1 – Pressure filtration targeting iron and manganese removal.
- Stream 2 – Direct RO membrane filtration targeting the removal of most water constituents.

The blend of the two systems is calculated based on treated water quality targets assuming most contaminants are removed through the RO membrane treatment process. Pressure filtration was briefly summarized under Option 1. Normally, RO membranes for systems of this size are skid-mounted and arranged in trains with a system recovery of around 75%, which results in about 25% of the raw water being disposed of to the sanitary sewer; this water contains all the rejected contaminants and can be somewhat difficult to dispose of as often they exceed discharge limits. The disposal of this stream must be evaluated in the next project phase if this option is selected.

Periodic cleaning will be conducted using built-in clean-in-place (CIP) procedures using a chemical treatment to remove reversible foulants from the membrane surfaces. The minimum frequency of CIPs will depend on the raw water quality, treated water production rates, antiscalant efficacy, and CIP procedures. An initial CIP frequency of once per month is assumed, subject to pilot investigation and optimization. A conceptual process diagram is provided in **Figure 4.2** - NF / RO Softening with Oxidation / Sorption bypass (Option 2)

Figure 4.2 - NF / RO Softening with Oxidation / Sorption bypass (Option 2)



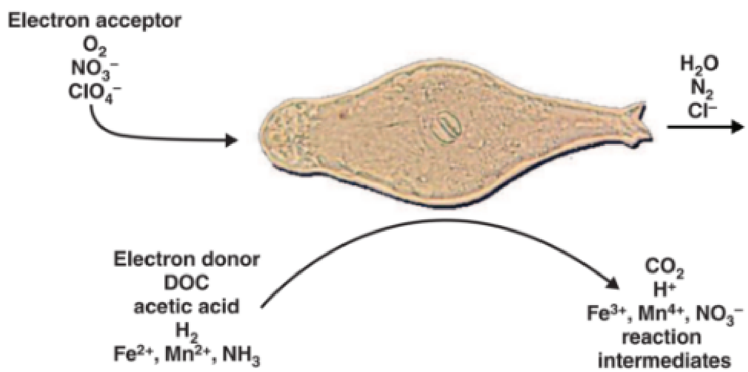
Although more complex, this process train offers advantages the others do not, and will thus be considered as part of the treatment options evaluated.

4.3.3 Option 3 – Biofiltration for Iron and Manganese removal

Biofiltration is an alternative treatment process that requires lesser chemicals and process control than conventional filtration processes (refer to Section 4.3.1). The process relies on autotrophic bacterial activity for the oxidation of iron and manganese.

Iron, manganese, and ammonia can be biologically oxidized under the correct pH, redox potential, temperature, dissolved oxygen (DO), micronutrients, and residence time conditions. **Figure 4.3** graphically illustrates the biological oxidation capacity of different species.

Figure 4.3 - Bacteria Mediation of Electron Transfer Between Reduced and Oxidized Compounds

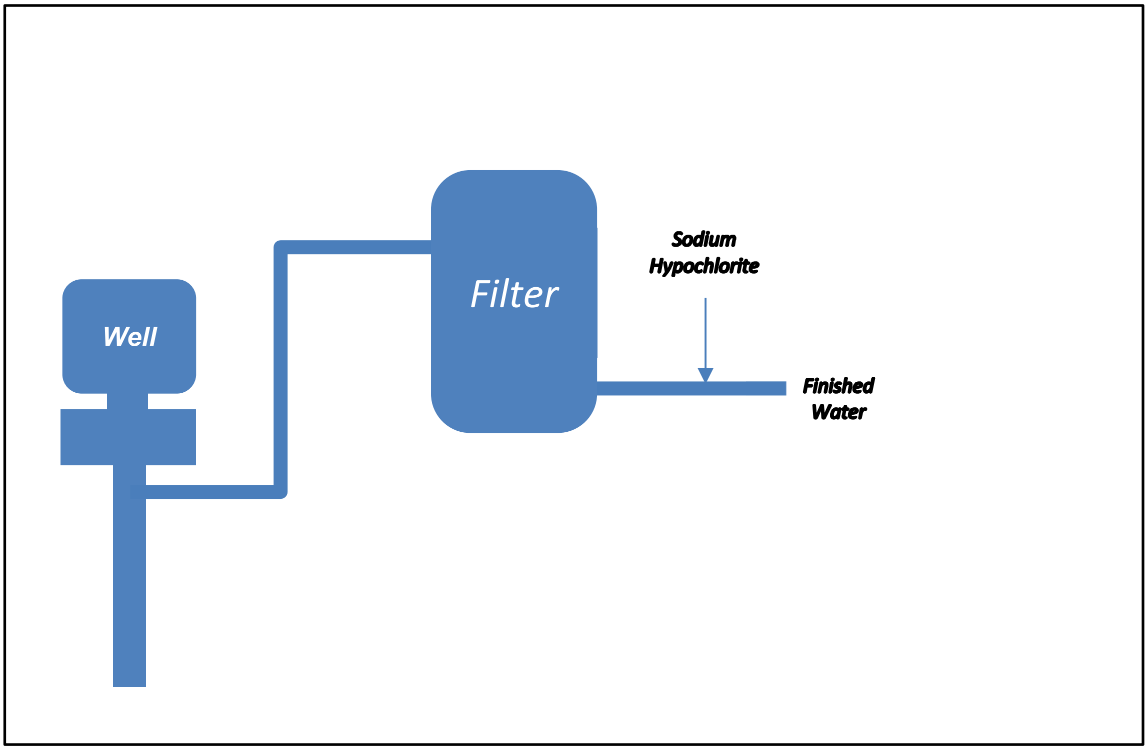


ClO_4^-—perchlorate, Cl^-—chlorine, CO_2—carbon dioxide, DOC—dissolved organic carbon, Fe^{2+}—iron(II), Fe^{3+}—iron(III), H^+—hydrogen ion, H_2—hydrogen, H_2O—water, Mn^{2+}—manganese(II), Mn^{4+}—manganese(IV), N_2—nitrogen, NH_3—ammonia, NO_3^-—nitrate, O_2—oxygen

Although co-removal has been reported, normally optimal results require multi-stage biofiltration; filtration occurs in steps, each targeting specific pollutants. Optimal bacterial oxidation of iron and manganese occurs at slightly different conditions. For example, an upper pH limit of 7 is normally reported for iron bacteria, while manganese prefers pH values above 7.2 to 7.5. Optimal iron removal takes place between 200 mV and 300 mV, while manganese is preferably removed at redox conditions above 300 mV and DO above 5 mg/L.

In the case of Fort St. James, the prime contaminant of concern is iron, and the secondary contaminant is manganese, although levels of manganese are below the prescribed guidelines. Available literature suggests iron and ammonia can be removed within 15 minutes, while longer times may be required for manganese depending on site-specific conditions. **Figure 4.4** shows a conceptual process diagram for this option.

Figure 4.4 - Biological Filtration (Option 3)



While it appears that biofiltration is an option that may be feasible for this project, further pilot testing and investigations may be required to ensure performance guarantee to meet the iron removal requirements.

We note that a similar biofiltration system has been installed recently at the nearby Village of Burns Lake. Discussions with the Village of Burns Lake Public Works staff suggest that it is satisfactorily functioning and meeting their performance requirements. Based upon this information, and the advantages listed in **Table 4.2**, this is an option that should be considered further for water treatment at Fort St. James.

4.4 PROCESS DESIGN AND REDUNDANCY

Treatment systems are sized to meet MDD expected for 2041 with appropriate redundancy to account for treatment downtime and failures.

A summary of the operational process configurations, firm capacity, and total capacity is provided in **Table 4.3**. The firm capacity is defined as the production capacity with one unit of each process, not in service, and total capacity involving all available units in service.

Table 4.3 - Process Configurations and Capacities (Treated Water Flows)

Options	Description	Firm Capacity (L/s)	Total Capacity (L/s)	Arrangement
1	3 Media filters at 15 L/s each	30	45	2 + 1
2	3 Biofilters at 15L/s each	30	45	2 + 1
3	3 Media filters at 5 L/s each 3 RO membranes units at 10 L/s each	30	45	2 + 1

4.5 OPINION OF PROBABLE COSTS

This section provides a summary of the Class C opinion of probable costs (OPC) for the treatment options evaluated at a conceptual level. These estimates are based on the best available information to date, budgetary pricing from suppliers, and Stantec's experience with recently tendered projects, and contain a contingency to account for details yet to be designed and other uncertainties. The costs are based on 2023 dollars excluding GST.

Table 4.4 presents the cost comparison between Options 1, 2, and 3. Detailed cost breakdowns can be found in **Appendix B**.

Table 4.4 – Treatment Options OPC Comparison

	Option 1	Option 2	Option 3
	GSF only	GSF and RO	Biofiltration as a replacement to GSF
General Requirements (7 % of Project Subtotal)	$ 450,000	$ 450,000	$ 450,000
Site Civil	$ 698,000	$ 698,000	$ 698,000
Process Mechanical	$ 2,123,000	$ 3,519,000	$ 1,652,100
Electrical and Instrumentation Cost	$ 580,000	$ 580,000	$ 580,000
Structural and Building	$ 2,520,000	$ 3,045,000	$ 2,520,000
Subtotal	**$ 6,371,000**	**$ 8,292,000**	**$ 5,901,000**
Geotech Site Investigation	$ 20,000	$ 20,000	$ 20,000
Site Survey	$ 25,000	$ 25,000	$ 25,000
Engineering Fees - 15% of Construction Sub-Total	$ 956,000	$ 1,243,800	$ 885,150
Contingency 35%	$ 2,229,850	$ 2,902,200	$ 2,065,350
Total Opinion of Probable Construction Costs	***$ 9,601,850***	***$ 12,483,000***	***$ 8,897,000***

4.6

4.6 RECOMMENDED WATER TREATMENT PROCESS

Options 1 and 3 show the most promise in terms of target water quality, complexity, and cost. Option 1 is the most commonly used in North America while Option 3 is gaining popularity; there is a water treatment plant using biofiltration located in the Village of Burns Lake operating extremely well. It would be prudent to pilot test biological filtration side-by-side with greensand (or similar type media) filtration to account for site-specific conditions such as temperature, pH, and oxidation-reduction potential (ORP). The Village of Burns Lake underwent a similar process and would be a good source of information.

If hardness proves to be a more critical parameter than considered in this report, then membrane filtration would have to be considered, i.e., Option 2. Any direct use of these membranes, i.e., without pre-treatment, must be carefully tested. A more costly alternative would be to filter all water and treat a fraction of that water with membranes to meet the target water quality.

4.7 DISTRIBUTION SYSTEM CONSIDERATIONS

Prior to the commissioning of a new treatment facility, particularly the introduction of a chlorine residual, it is of paramount importance to properly address the issue of legacy iron and manganese deposits that may have accumulated within the system over time. Following an initial assessment, the removal of legacy deposits is accomplished by an appropriate cleaning and flushing program that is capable of removing the typically adhesive biofilms often associated with the retention of these metals.

4.8 RELIABILITY AND RESILIENCY

In addition to the usual redundancy requirement for process equipment and critical elements, backup power generation capabilities are assumed as part of the electrical system, including a diesel generator sized to power the raw water well pumps and filter backwash and MTU feed pumps. Additional features would include process monitoring and alarms, emergency shutdown, and raw water bypass capabilities.

4.9 RESIDUALS DISPOSAL

The primary waste streams include the spent filter backwash water and rinse water (Options 1, 2, and 3), and the MTU concentrate and spent membrane CIP solutions (Option 2). These streams are assumed to be discharged to the sanitary sewer collection system and constitute approximately 5% of the average daily plant flow or approximately 50,000 L on an annual average basis.

4.10 POTENTIAL TREATMENT PLANT SITES

The site of the existing water well and pump house structure is recommended for consideration as a site for a new water well and a treatment facility, as shown in **Figure 4.5**.

DISTRICT OF FT. ST. JAMES | FIGURE 45

PROPOSED WATER TREATMENT PLANT

LOCATION PLAN

Stantec
Design with community in mind

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March 27, 2023

DISTRICT OF FT. ST. JAMES | FIGURE 46

PROPOSED WATER TREATMENT PLANT

SITE PLAN

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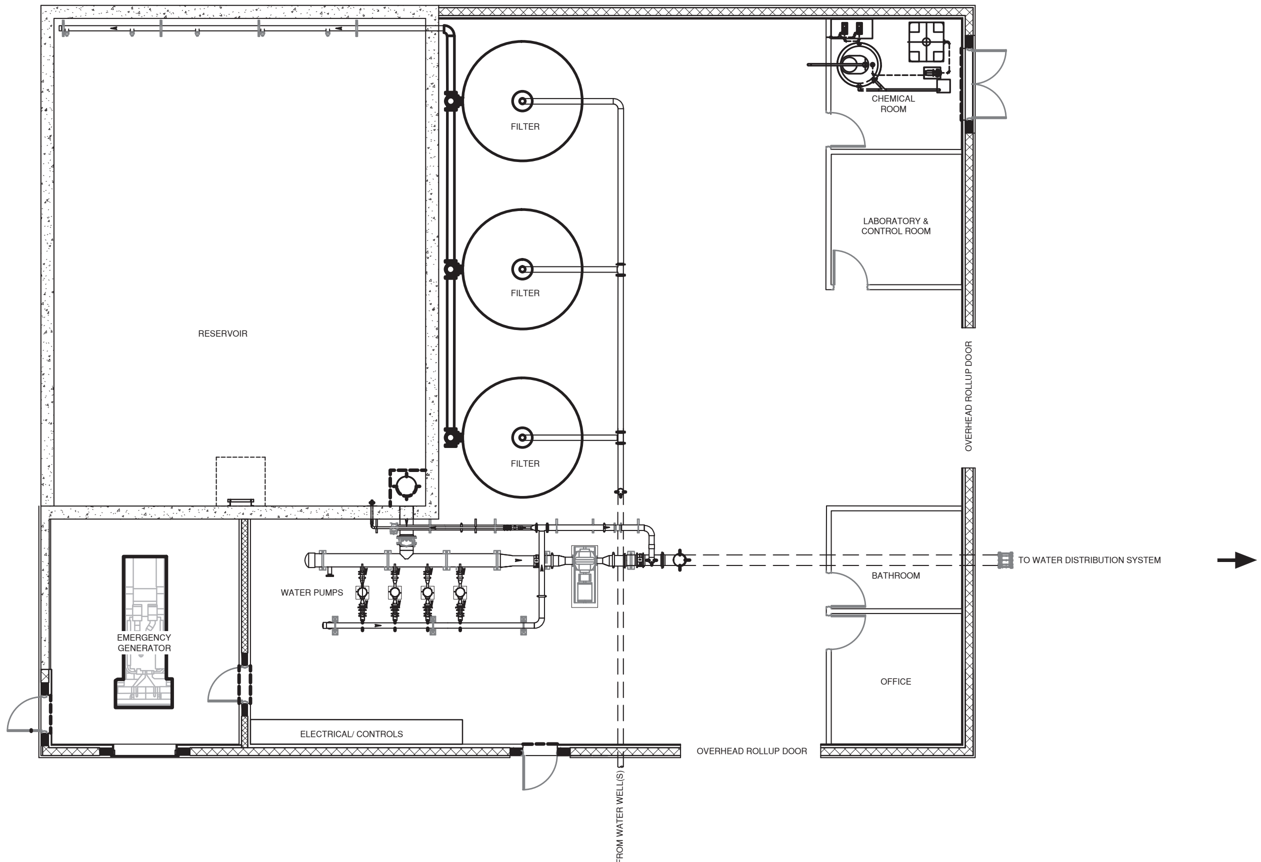

DISTRICT OF FT. ST. JAMES | FIGURE 47

PROPOSED WATER TREATMENT PLANT

GENERAL ARRANGEMENT

CONCEPT PLAN

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Design with community in mind
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March 27, 2023

5.0 WATER STORAGE AND DISTRIBUTION SYSTEM

As described in **Section 2.0** of this report, record drawings suggest significant components of the existing water system were constructed in the 1960s and 1983, along with other additions / improvements in the period since 1983. Issues with the water storage / distribution system may be related to its age and / or the size of pipes as follows:

- Many of the pipes are 150 mm and 200 mm diameter, which in some cases results in reduced water supply capacity for fire protection.
- Watermains in 1969 were constructed with Asbestos Cement pipe. The District's experience with this pipe matches that of other communities; it will start to fail at an increasing rate in the future now that its age exceeds 50 years.

In order for the District to be able to consider options / opportunities for improvement of the water distribution system, Stantec developed a water model that can be used to evaluate the performance of the water distribution system under various operating conditions / scenarios.

For the purpose of this Water Master Plan, the model was used to evaluate existing conditions under Peak Hour Demand and Maximum Day Demand plus Fire Flow.

5.1 WATER DISTRIBUTION SYSTEM DESIGN CRITERIA

The Fort St. James District water supply and distribution system requirements have been determined based on the following guiding documents:

- Master Municipal Construction Documents (MMCD) Design Guidelines 2022, published by the Master Municipal Construction Documents Association
- Fire Underwriters Survey Water Supply for Public Fire Protection – A Guide to Recommended Practice in Canada, 2020

The following is a summary of the relevant design parameters based on the above guidelines for the water distribution system design and analysis:

Master Municipal Construction Documents (MMCD – Design Guidelines 2022, published by the Master Municipal Construction Documents Association)

- Per Capita Demand:
  - o demand (ADD) = 400 L/c/d
  - o Maximum day demand = 900 L/c/d
  - o Peak hour demand = 1350 L/c/d

  - – In absence of reliable bulk meter and water consumption records and/or specific municipal requirements, use 15% of ADD to estimate unaccounted-for or non-revenue water demand.

- Non-Residential Demand (based on residential population equivalent)
  - – Institutional – 50 p/ha
  - – Commercial – 90 p/ha
  - – Industrial – 90 p/ha

- Minimum fire flow requirements for developments without sprinklers:
  - – Single Family Residential = 60 L/s
  - – Apartment/Townhouses = 90 L/s
  - – Institutional = 150 L/s
  - – Commercial = 150 L/s

- Design Flows (system design flow should be based on the Official Community Plan (OCP) unless instructed by the local authority):
  - – Q_{design} = MDD + FF
  - – Q_{design} = PHD

- Water Pressure:
  - – Maximum allowable pressure = 850 kPa*
  - – Minimum pressure at PHD = 300 kPa
  - – Minimum pressure in system during MDD+FF = 150 kPa

- Maximum allowable velocities:
  - – PHD condition = 2.0 m/s
  - – MDD+FF condition = 3.5 m/s

Fire Underwriters Survey Water Supply for Public Fire Protection – A Guide to Recommended Practice in Canada, 2020

- Fire flow requirements
- Reservoir storage requirements

5.2 WATER FOR FIRE PROTECTION

Firewater demands, as per the MMCD's 2022 Design Guideline for developments without sprinklers, are as follows:

- Single Family Residential = 60 L/s
- Apartment / Townhouses = 90 L/s
- Institutional / Commercial = 150 L/s

The MMCD Design Guideline recommends the above values be confirmed based on the Fire Underwriters Survey (FUS) requirements. In order to verify fire flow requirements, a few major facilities within the District were selected, and a fire flow requirement for each selected facility was computed based on assumed facility information based on air photo review. For the computation of the fire flow, the FUS provides preprogrammed spreadsheets that use the method in the guideline document.

The fire flow requirement is the water flow rate at a minimum residual pressure of 150 kPa for a specific duration necessary to control a major fire in a specific structure or a group of structures utilizing manual firefighting measures along with the built-in safety features of the building. The fire flow duration aspect is addressed in the design of reservoir storage.

The base fire flow requirement is computed based on the following formula:

$$RFF = 220\, C\sqrt{A}$$

Where RFF = the Required Fire Flow in litres per minute
C = the Construction Coefficient related to the type of construction of the building
A = the Total Effective Floor Area (effective building area) in square meters

The construction coefficient value ranges from 1.5 for wood frame construction to 0.6 for fire-resistive construction. A building is considered a fire-resistive construction when all structural elements are constructed with a 2-hour fire resistance rating. For the District facilities, no fire flow requirement adjustment was made based on the assumption of all building construction having ordinary construction material (mixture of combustible and non-combustible material).

The fire flow requirements can be further adjusted by as much as a 25% reduction for occupancies having a low fire hazard and increased by as much as 25% for occupancies having a high fire hazard. For the District facilities, no fire flow requirement adjustment was made based on the assumption of all occupancies having combustible contents.

In addition, the fire flow requirements can be further reduced by up to 50% for facilities having a complete automatic sprinkler fire protection system. The fire flow requirement adjustment for sprinkler systems was assigned based on the assumption of having an adequate building sprinkler system in each facility.

Based on the above assumptions, the calculated fire flows required are summarized for the selected facilities in **Table 5.1.** Based on the required fire flow values, it is recommended to use a maximum of 150 L/s fire flow rate for all significant non-residential facilities and 90 L/s fire flow rate for all smaller non-residential and multi-family residential areas. These assigned fire flow requirement values are then used in subsequent sections to identify any existing system deficiencies and to establish the upgrade requirements.

Table 5.1 – Estimated Fire Flow Requirements for Commercial / Institutional Facilities

Building Name	Building Area (m^2)	Building Construction Materials	Sprinkler System	Calculated Required Fire Flow (L/s)	Assigned Required Fire Flow (L/s)
FSJ Secondary School	4,900 m^2	MSB, WD	Yes	167	150
David Holy Elementary School	3,300 m^2	MSB, WD	Yes	117	150
Stuart Lake General Hospital	2,200 m^2	MSB, WD	Yes	100	150
Fort Forum Arena	3,200 m^2	MSB, WD	Yes	100	150
North Strip Mall (Highway 27 & Manson Street)	4,800 m^2	MSB, WD	Yes	167	150
South Strip Mall (Highway 27 & Manson Street)	2,500 m^2	MSB, WD	Yes	133	150
Notes: WD = Wood Stud Walls / Roof Components MSB = Masonry Blocks					

5.3 STORAGE RESERVOIR SIZE

As per Design Guidelines for Rural Residential Community Water Systems (2012), as well as the MMCD Design Guideline (2022), reservoir capacity is calculated as per the following equation:

Reservoir Capacity = A + B + C

Where:
A = Fire Storage
B = Equalization Storage 25 % of projected MDD
C = Emergency Storage 25% of (A +B)

According to the FUS guidelines, for a 150 L/s fire flow requirement, fire storage needs to be provided for a duration of 2.0 hours. Based on these requirements, the required storage for the District for 2041 with a 3.5% growth rate is summarized in **Table 5.2.**

Table 5.2 - Required Water Reservoir Storage Volume

Storage Description	Required Storage Volume (m^3)	Remarks
Fire Flow Storage (A)	1,080	150 L/s fire flow for 2.0-hour duration
Equalization Storage (B)	620	MDD = 2,463 m^3/d
Emergency Storage (C)	425	25% of (A+B)
Total Storage (m^3)	**2,125**	

At 2,278 m^3, the existing reservoir provides sufficient volumes of water for fire protection, theoretically. Water modelling, however, has demonstrated that the reservoir location, on the far east side of the District's developed area, has the effect of limiting the amount water (rate / pressure) that is available for fire protection in areas that are located further away from the reservoir.

Options to ensure adequate water (flow / pressure) for fire protection may include:

- Increasing water main sizes / available water main looping in a number of locations across the community.
- Ensuring that any future improvements (water supply and treatment) include allowances for additional water storage and a fire pump at the water treatment plant.

For the purpose of this Water Master Plan and budgetary planning, the cost estimates contained herein assume that a newly drilled water well and water treatment plant will include provisions for additional water storage and a water booster pump / fire pump at the water supply / water treatment plant site. Details of this design concept will need to be revisited during the design stage of the project.

5.4 WATER DISTRIBUTION SYSTEM ANALYSIS

Modelling and analysis of the water distribution system were completed to determine the adequacy of the existing water main sizes. The water distribution model was used to assess the peak hour and maximum day plus fire flow demand conditions.

The water model was developed based on the as-built records provided by the District for this study. The existing water distribution system model primarily consists of nodes or junctions, water mains, booster pump and reservoir representing boundary / supply conditions. The node / junction is assigned to represent either a junction where multiple pipes meet or simply a point along the water main. The water mains or links are used to assign pipe size, pipe material represented by Hazen-William's roughness coefficient, and pipe lengths between nodes. The node points are used to assign water demand for adjacent facilities and system topography. For this model, the fire hydrants and hydrant links are not included; instead, the fire flow is simulated at the junction node to assess the distribution system's capability. The Nak'azdli service area distribution system is also not included in the model. However, for the Nak'azdli service area, the water demands are included in the model.

The maximum day and peak hour water demands at nodes were assigned based on 2041 demand with an assumed 3.5% growth rate. The ground / surface elevations at nodes were assigned based on the available topographic contour information.

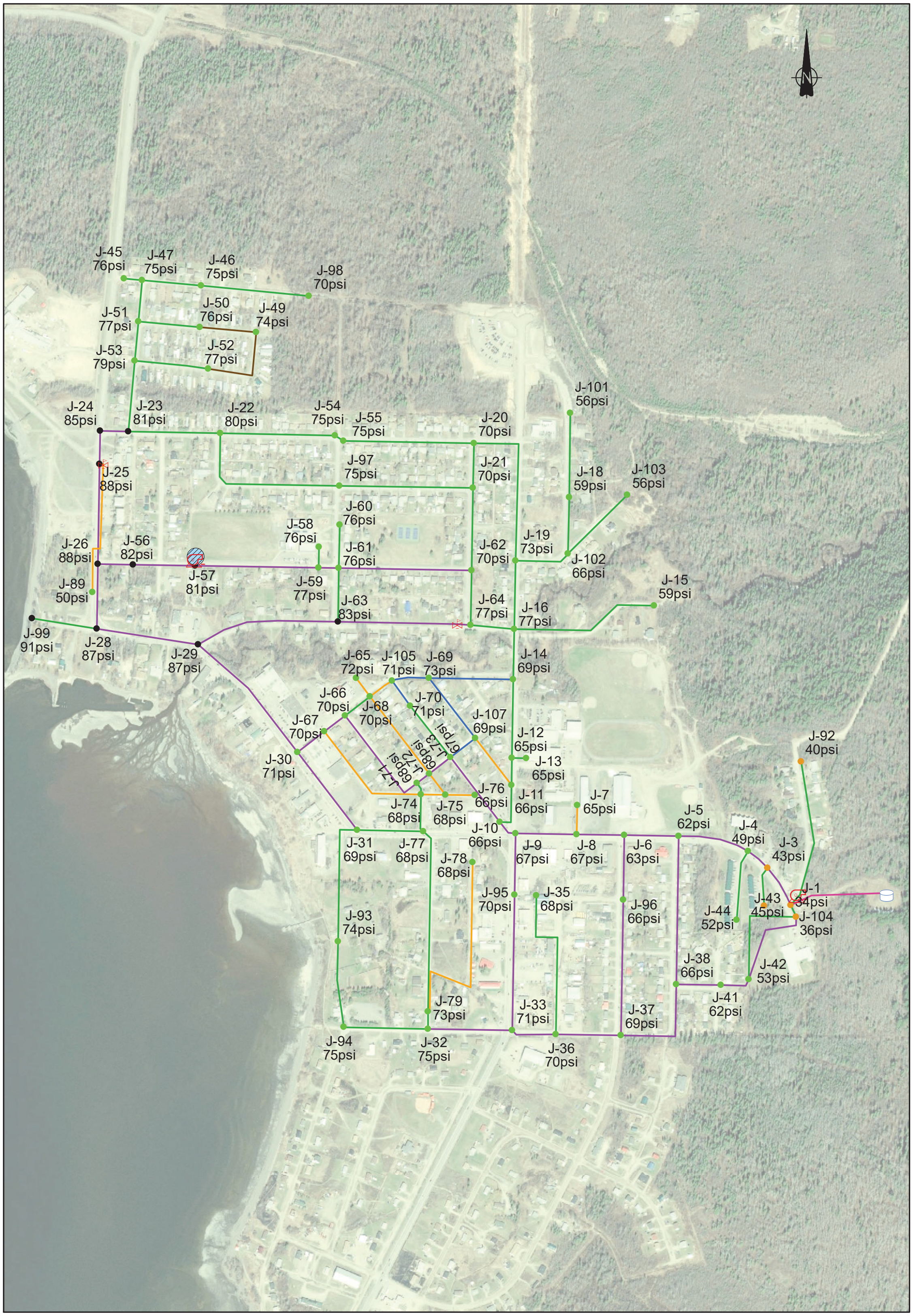
As the water distribution system is primarily supplied from the storage reservoir by gravity, the minimum water level in the reservoir of 740.5 m was used for the boundary condition for all model demand scenarios. When the reservoir is full, the water level can reach 745 m.

Existing distribution system pipe materials include PVC, asbestos cement, ductile iron, and HDPE, and therefore, are assigned a Hazen-William's roughness coefficient of 120. The roughness coefficients are generally calibrated using hydrant flow tests, but these were not available, and as a result, the model is not calibrated. For the water distribution system to function properly, all water distribution system valves need to be fully opened, and it is assumed that all valves are fully opened for the analysis.

5.4.1 Peak Hour Demand

For the PHD, available simulated pressures for the existing distribution system are shown in **Figure 5.1.** The required total PHD for the District is 45.6 L/s for the 2041 development condition.

The existing distribution systems can meet the PHD required minimum residual pressure of 300 kPa at most of the node locations. The minimum required pressure is not met at a few nodes in the vicinity of the storage reservoir due to higher ground elevation. This impact has recently been noted as a part of the development of Seniors housing on Pineridge Way and the re-development of the Stuart Lake Hospital; both of these projects have required booster pumps to ensure adequate domestic water pressure and water pressure / flow for sprinklers.



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Revised: 2023-03-14 By: abnabi

March 2023
Project No. 115821021

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Legend

Tank
PRV
Pumps
Reservoir

Available Pressure
- <30 psi
- 30 psi - 45 psi
- 45 psi - 80 psi
- >80 psi

Existing Pipe Diameter
- 50 mm
- 100 mm
- 150 mm
- 177 mm
- 200 mm
- 300 mm

Client/Project

FORT ST JAMES

District of Fort St. James
Water Distribution System Analysis

Figure No.
5.1

Title
Existing System
Available Pressure During PHD
Water Supply Pump On

5.4.2 Maximum Day Demand Plus Fire Flow

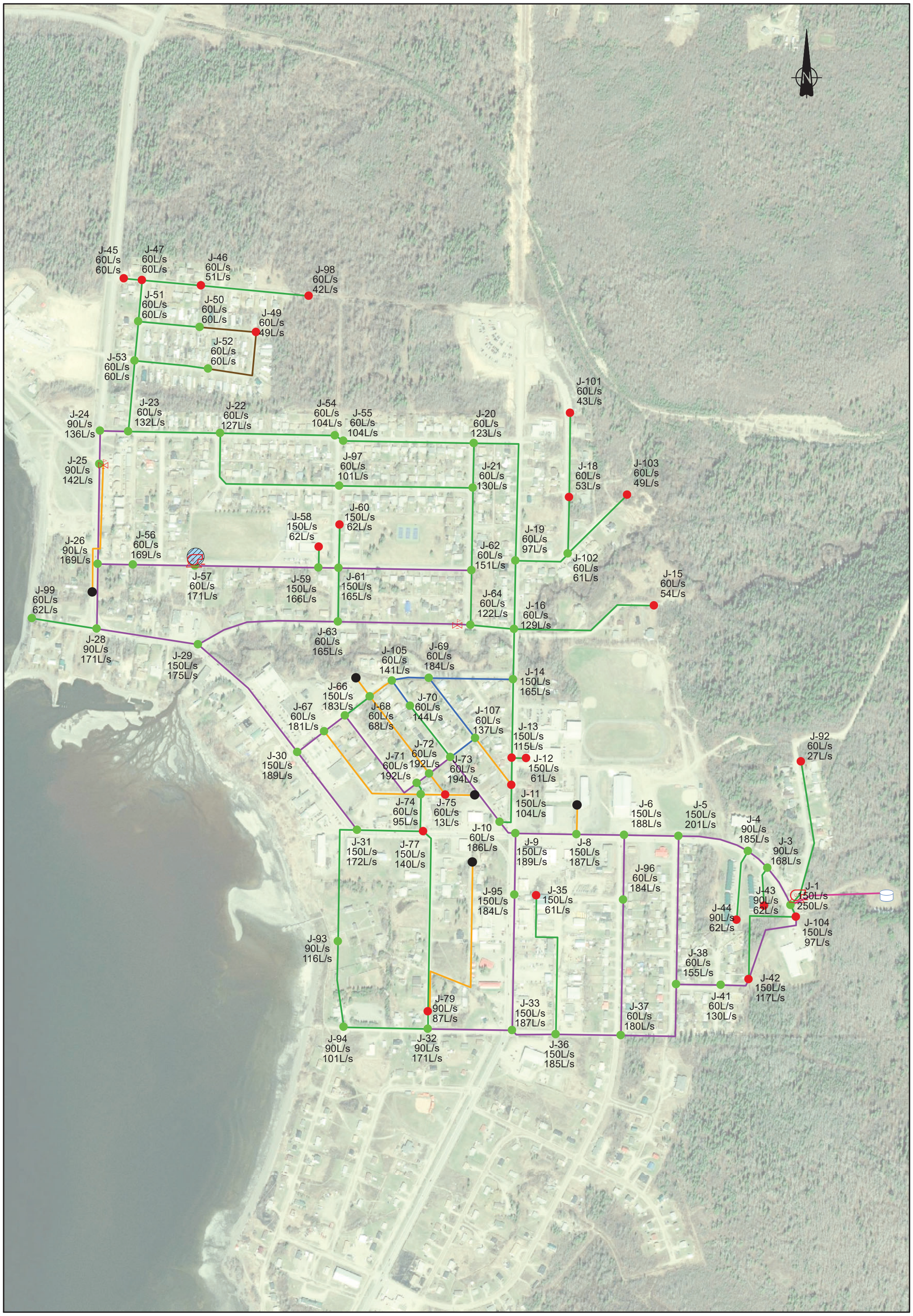
The existing water distribution system analysis results for the MDD plus 60, 90, and 150 L/s fire flow model results are shown in **Figure 5.2 and Figure 5.3**, with and without the supply pump on. The total required MDD for 2041 is 28.5 L/s based on the 3.5% growth rate.

The water model suggests that the existing system cannot meet the MDD plus required fire flow at many locations with the minimum residual pressure of 150 kPa at all nodes, particularly when the main water supply pump (Birch Street Water Pump Station) is not operating.

5.4.3 Upgraded Water Distribution System Analysis

An upgraded water distribution system analysis for the FSJ area was completed to identify the existing system upgrade requirements. As the existing distribution system has many locations where fire flow requirements cannot be met (see **Figure 5.3**), the proposed system improvements are limited to the gradual upsizing of the distribution system.

In general, the fire flow deficiencies in the distribution system are due to the minimum system pressure requirement in the higher ground area distribution system.



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Revised: 2023-03-14 By: abnabi

March 2023
Project No. 115821021

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Legend

- Tank
- PRV
- Pumps
- Reservoir
- FF Req't Not Met
- FF Req't Met
- FF Not Simulated

Existing Pipe Diameter

- 50 mm
- 100 mm
- 150 mm
- 177 mm
- 200 mm
- 300 mm

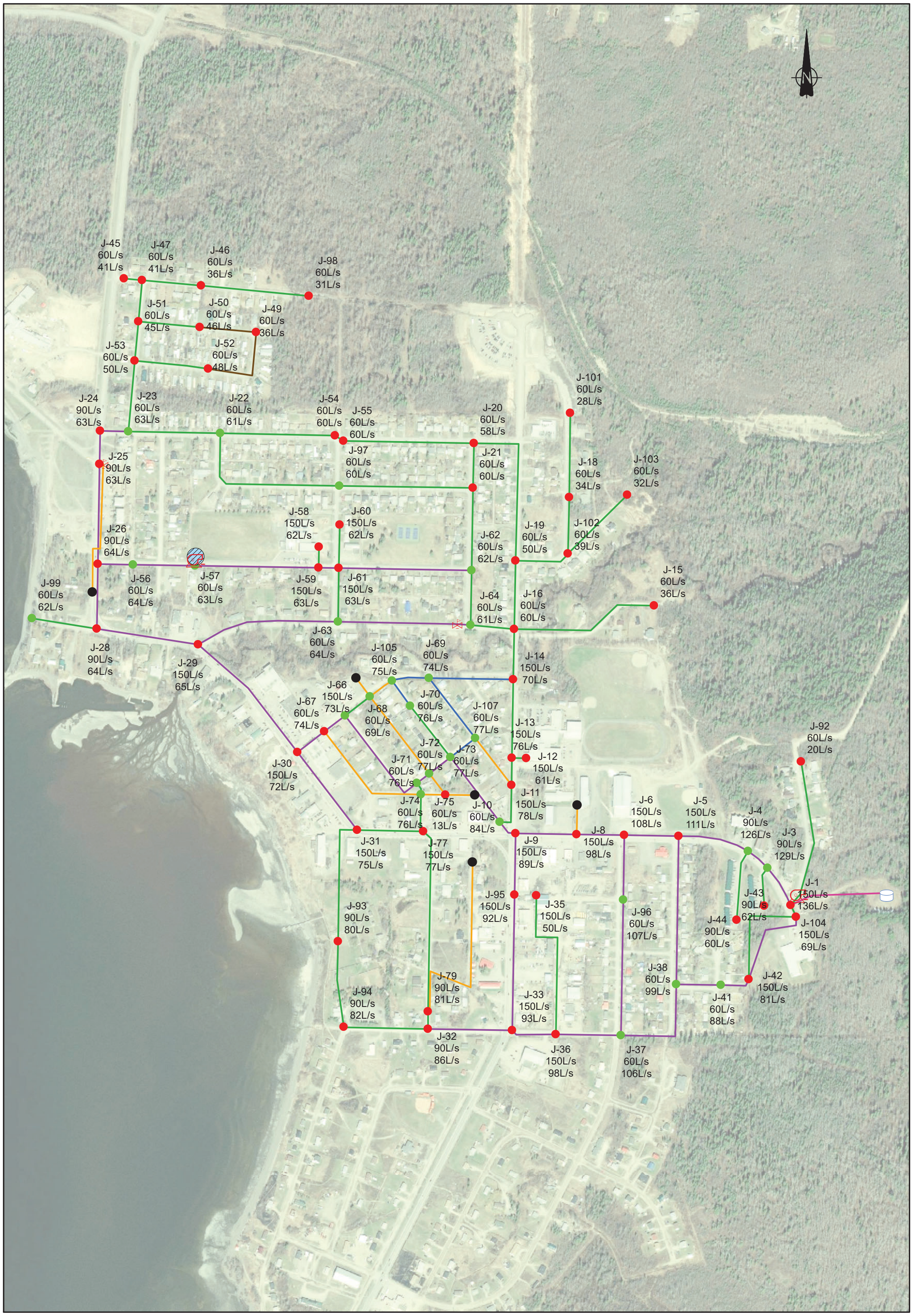
Client/Project

FORT ST JAMES

District of Fort St. James
Water Distribution System Analysis

Figure No.
5.2

Title
Existing System
MDD Plus Fire Flow Scenario
Water Supply Pump On



Legend

Tank

PRV

Pumps

Reservoir

FF Req't Not Met

FF Req't Met

FF Not Simulated

Existing Pipe Diameter

50 mm

100 mm

150 mm

177 mm

200 mm

300 mm

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Client/Project

FORT ST JAMES

District of Fort St. James
Water Distribution System Analysis

Figure No.
5.3

Title
Existing System
MDD Plus Fire Flow Scenario
Without Supply Pump

6.0 CAPITAL COST ESTIMATES

Previous reports have estimated the cost of implementing water treatment in Fort St. James at approximately $4 million (Class D Estimate, Option 1A, 2019 Feasibility Study, David Nairne + Associates Ltd.). We understand that the new water treatment plant in Burns Lake cost approximately $5 million in 2021 dollars. The exact scope of the Burns Lake project is not fully understood and may not be an "apples to apples" comparison, but it is useful for a high-level comparison and project planning.

As described in **Section 4.5** and detailed in **Appendix B,** our Class C cost estimates suggest that the cost of Option 3 (including drilling a new / backup water supply well) could be approximately $5.9 million, not including Engineering and Contingency (E&C). Adding allowances for E&C, the total cost of Option 3 is estimated to be in the order of magnitude of $8.9 million in 2023 dollars.

The next phase of the project should include detailed design that will allow for improved definition of the project scope of work and confirmation of funding budgets via higher accuracy cost estimates.

7.0 FUNDING OPPORTUNITIES

The communities of Fort St. James and Nak'azdli have expressed a desire to share in the costs of works that are proposed to improve the reliability and quality of water provided to residents of both communities.

In the past, the framework for cost sharing has been provided for in a Municipal Type Service Agreement (MTSA) that has been in effect since the 1980s. The MTSA is dated and may not adequately provide for the sharing of costs of capital improvements of the magnitude / type currently proposed.

Regardless of the details of the MTSA, both communities will be expected to investigate various funding opportunities that may include, but not necessarily be limited to:

- Indigenous Services Canada (ISC) funding.
- Investing in Canada Infrastructure Program (ICIP) funding.

It is our understanding that ISC has previously supported Nak'azdli with the 2019 feasibility assessment. Prior to either community approaching ISC, ICIP or other funding agencies, it will be important for the two communities to review the water system improvement concept proposal contained herein.

8.0 RECOMMENDATIONS

The District engaged Stantec to confirm the feasibility and costs of implementing water treatment and improving water system redundancy. This study has found that it is feasible to drill a second (back–up) water supply well and to construct a new water treatment plant at the site of the existing water supply well and pump house. Potentially, the water treatment plant could be quite similar to the new plant that is operating in Burns Lake, BC.

Next steps could include:

- Stakeholder engagement within the communities of Fort St. James and Nak'azdli, approving agencies such as First Nations Health Authority and the Public Health Engineers at Northern Health and funding agencies such as ISC and ICIP.
- A site visit to view operation of the Burns Lake Water Treatment Plant.

Ideally, this process of engagement and discussion would lead to the implementation of the next phases of the project, including:

- Phase 1 - Pilot Testing of specific water treatment processes.
- Phase 2 - Drilling and testing of a backup well / Preliminary Design and Class B Cost estimates for a water treatment plant.
- Phase 3 - Detailed design / Class A cost estimates for a water treatment plant.
- Phase 4 - Construction of a water treatment plant.

Given that the community of Fort St. James has not had a water treatment plant, the construction phase should include a planned process for operator training and system commissioning. The process of commissioning the water treatment plant will likely need to include a substantial program for water distribution system cleaning and flushing and communication with residents regarding expectations for water quality disruptions during cleaning and flushing.

9.0 REFERENCES

Design Guidelines for Rural Residential Community Water Systems, 2012
https://www2.gov.bc.ca/assets/gov/environment/air-land-water/water/water-rights/water-utilities/design_guidelines_rural_residential_water_mar2012.pdf

Ministry of Health Design Guidelines for Drinking Water Systems in British Columbia, Draft March 2021

Design Guidelines for First Nations Water Works
https://www.sac-isc.gc.ca/eng/1100100034922/1533666798632

APPENDICES

Appendix A WESTERN WATER ASSOCIATES LTD.

Backup Groundwater Supply Feasibility Assessment

western water
ASSOCIATES LTD
Consultants in Hydrogeology and Environmental Science

District of Fort St. James Backup Groundwater Supply Feasibility Assessment

Prepared for:

Stantec Consulting Ltd.
205-1302 7th Ave.
Prince George, B.C.
V2L 3P1

Stantec

January 2023
Project: 21-035-01PG

Executive Summary

Western Water Associates Ltd. (WWAL) was retained by Stantec to conduct a feasibility study with respect to the development of a backup drinking water supply well for the District of Fort St. James. At present the District utilizes one high-pressure flowing artesian well (PW2), which is reportedly capable of supplying the District's water requirement. The well was drilled in the 1970's and has a shut-in wellhead pressure of 50-52 PSI and reportedly free flows at up to 4200 US gpm (259 L/sec). Several studies have been completed in the past with the intention of increasing the total water supply to the system at a time when the District population was expected to increase. However, based on a Maximum Daily Demand (MDD) of 2463 m^3/day determined by Stantec, a well capable of supplying approximately 450 US gpm has been determined to be sufficient for use as a secondary water source. As a new well would solely be for the purpose of adding redundancy to the system and will not be utilized while operating the existing well, a nominal 203 mm (8-inch) production casing well drilled near PW2 is recommended. To attain an 8-inch production casing, drilling the surface casing down to the confining layer above the flowing artesian aquifer would be done at a larger diameter. During the development of the well, significant volumes of turbid water will be produced that will require the development of a control plan. As the existing well is located in the corner of the secondary school field, sufficient space should be available for a new well. In B.C. there are only a handful of well drillers with the equipment and experience to successfully drill a high pressure artesian well and for this reason, it is imperative that careful consideration should be made to select a qualified well driller during any future tendering process.

With respect to water quality, a well drilled near PW2 will likely exhibit similar water quality requiring treatment for iron and manganese and possibly antimony. Several recommendations including the collection of additional water samples and pumping pressure data are discussed in section 5 of this report.

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1. INTRODUCTION

Western Water Associates Ltd. (WWAL) is pleased to provide this hydrogeological assessment of groundwater supply potential for the District of Fort St. James ("the District)." The feasibility assessment was completed to investigate the potential for developing an emergency back-up water supply for the District in the event a sudden loss of the District's main water supply from registered groundwater well WTN22987.

1.1 Project Background

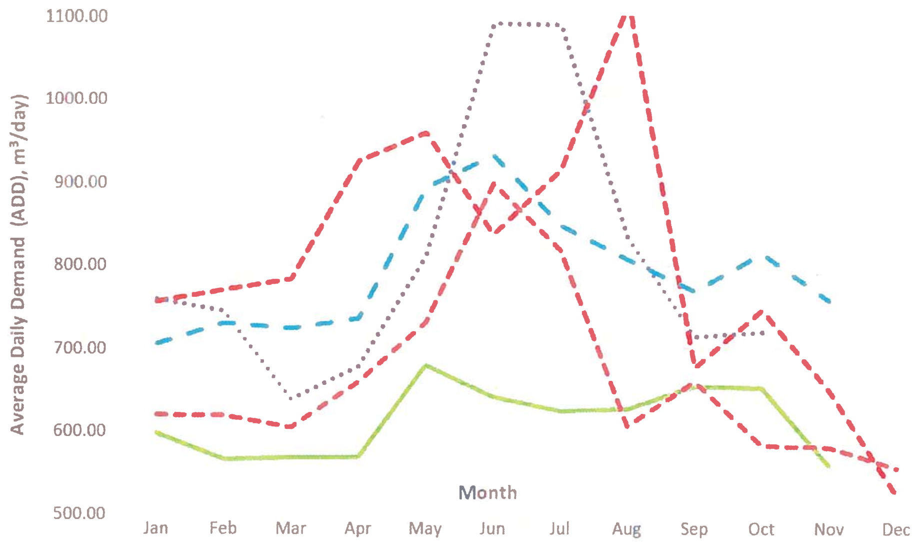
The District's operates a community water system that services the majority of residences, businesses and institutions in the municipality and on Nak'azdli First Nations IR1, which borders Fort St. James to the south. Potable water is supplied from one well, with WTN22987 referred to as Production Well 2 (PW2). A population of approximately 1,818 people is supplied with potable water by the District, with 655 connections located in the municipality and 220 connections on Nak'azdli IR1. Water from the well is not treated and is pumped to a reservoir form where it is distributed throughout the municipal water system. Future water treatment is planned to address water quality concerns related to odour as well as elevated iron, arsenic and manganese (District of Fort St. James, 2020). The District's well is capable of meeting the current water demand; however, the District does not have a back-up supply that could provide water in the event of a loss of supply from WTN22987.

WWAL hydrogeologist Warren Grafton, P.Geo., visited Fort St. James in April 2021 to view the community water supply well and surrounding areas.

1.2 Water Demands

Stantec calculated current water demand for the District based on well pump data from January 2016 to November 2020. Pumping data included daily totals and were recorded manually by the District from totalizers. Average Day Demand (ADD) for the 2016-2020 period was calculated at 793 m^3/day and is summarized below in Figure 1-1.

Water demand forecasts were calculated by Stantec for population growth rates of 0.5-3.5% for the District and 1.5% for Nak'azdli. Based on the current volume of 400 Liters/capita/day (Lpcd) and the highest projected population growth rate of 3.5%, a daily volume of 985 m^3/day was determined. Any future wells must be capable of supplying a Maximum Day Demand (MDD) considered to be 2.5 times the ADD equating to 2463 m^3/day. Based on discussion with the engineering team, WWAL has determined a target well yield of 28.5 L/s (451.7 USgpm). For the purposes of a suitable well design for a second well, we have rounded this number up to a nominal flow rate of 500 US gpm (31.5 L/s).



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2020	597.43	566.42	568.03	569.10	679.79	641.33	624.33	626.57	653.97	651.80	558.52	
2019	619.77	619.43	604.90	659.69	731.43	898.46	818.40	606.53	659.89	583.10	580.83	555.83
2018	756.53	770.57	783.72	925.71	960.06	837.52	914.76	1115.87	677.12	746.06	648.77	524.14
2017	759.43	745.11	638.65	678.08	810.48	1091.31	1089.77	833.60	713.87	719.72		
2016	705.77	730.69	724.19	735.97	893.83	932.62	848.29	807.40	768.69	814.42	757.40	

Figure 1-1: Summary of Average Daily Demand (ADD) 2016-2020 (Stantec 2021).

1.3 Scope of Services

WWAL provided the following services for this project:

1) Compiled and reviewed pertinent background information such as original drilling reports, aquifer hydrogeological mapping, area well logs, climate information, hydrological data and available hydrogeological and engineering reports.
2) Met with the District's Engineering consultant to confirm project requirements and reporting timelines.
3) Conducted a field reconnaissance of the District's PW2 well site area to view District-owned properties and visually assessed and documented land use surrounding potential well locations. Options for water management during drilling and other possible site constraints related to well construction were also assessed at the time of the field reconnaissance.
4) Produced a high-level cost estimate to drill a new backup well specific to the area's flowing artesian conditions for budgeting purposes.
5) Prepared this report assessing the feasibility of establishing an emergency back-up groundwater supply for the District of Fort St. James.

2. SITE DESCRIPTION

2.1 Physiography

Both Fort St. James and Nak'azdli IR1 are located within the valley located at the southwest end of Stuart Lake. Nak'azdli IR1 borders Fort St. James to to the south. Mountains with elevations up to 1400 m above sea level (asl) are located to the north and east of Fort St. James. The majority of the townsite is located in relative proximity to Stuart Lake and is relatively flat to west sloping towards the lake. PW2 is located to the northeast of the intersection of Birch Street West and 7th Avenue West at an elevation of approximately 695 m above sea level (asl). The well site is located approximately 320 m east of Stuart Lake. The nearest surface water body to PW2 is Nahounli Creek, which is located approximately 220 m south of the well. Nahounli creek flows through Fort St. James from the mountains to the east and discharges into Stuart Lake southwest of PW 2. The Stuart River originates at the south end of Stuart Lake from where is flows south/southwest. The Necoslie River flows through the southwest portion of Nak'azdli IR1 and joins the Stuart River near Stuart Lake. A general location map is provided as attached Figure 1 and a map showing the location of PW2 is provided in Figure 2.

2.2 Geologic Setting

Bedrock underlying the Fort St. James area is mapped as sedimentary rock of the Cache Creek Complex. Fort St. James and the northeast of Nak'azdli IR1 are underlain by limestone, marble, calcareous sedimentary rocks from the early Pennsylvania to middle Triassic (323 – 235 million years before present). Areas to the north and west of Fort St. James and the south and west of Nak'azdli IR1 Mudstone, siltstone, shale and fine clastic sedimentary rocks early Permian to late Jurassic in age (290 – 146.5 million years before present) underly areas to the north and west of Fort St. James and the south and west of Nak'azdli IR1.

Surficial geology underlying the Fort St. James area was mapped by A. Plouffe in 2000 (Plouffe, A., 2000). The majority of Fort St. James as well as Nak'azdli IR1 are mapped as underlain by glaciolacustrine deposits comprised of well sorted, stratified sand, silt and clay interpreted to be deposited in deep water of former glacial lakes. The sands, silts and gravels are overlain sporadically by sand and gravel near the limits of former glacial lakes. Areas along Nahounli Creek are mapped as alluvial gravel and sand with minor silt and clay. Fluvial terrace deposits comprised of stratified sand and gravel overlain by a veneer of sand and silt are mapped along the Nekoslie and Stuart Rivers.

2.3 Hydrogeologic Setting

2.3.1 Aquifer 373 (Fort St. James Aquifer)

Provincial aquifer mapping shows Aquifer 373 as underlying Fort St. James. Aquifer 373 is a confined glaciofluvial sand and gravel aquifer. The provincially mapped aquifer is described as having low vulnerability to contamination, high productivity and is mapped as covering 14.1 km^2 (Figure 3). The aquifer is confined by a thick clay and silt aquitard with a lower boundary comprised of bedrock (ENV, 2021). The Fort St James Aquifer was discovered in the 1930s by Alex Nicholson, a local well driller, while drilling a 3 inch diameter well for the former Fort St James Hotel. Area well logs indicate that the well drilled at the Fort St James Hotel is likely associated with WTN1833. Two supply wells (WTNs 44535 and 44522) were drilled in 1980 for a fish hatchery north of the District's PW2. WTN44535 was reportedly drilled 300 m north of WTN45522, encountered bedrock at 135 ft and the aquifer was reportedly not encountered, whereas WTN44522 was reported to be free flowing at 40-68 L/s with a shut-in pressure of 39.7 psi. This indicates that Aquifer 373 is likely relatively steep walled and potentially a channel deposit extending to the northeast (Golder, 2001). Recharge to the Aquifer is inferred to be from higher elevations to the northeast including possible losses from a shallower unmapped aquifer (Golder, 2001).

2.3.2 Aquifer 373 Wells

WWAL completed a search of the ENV GWELLS application to identify reported wells in proximity to Fort St. James (ENV, 2021). Twelve wells were noted as completed in Aquifer 373 on the respective well logs. WWAL reviewed well logs for wells mapped within the mapped extent of the aquifer, which resulted in an additional eleven wells interpreted as being installed in Aquifer 373. Depth of wells varied from 41-334 ft with bedrock noted on the well log for the District's PW2 at 334 ft and two other wells at depths of 284 ft and 311 ft. Water yield ranged from 2 US gpm to 4200 US gpm and flowing artesian conditions were reported for PW2 and ten other wells. Aquifer mapping reports describe the aquifer as having a relatively high well density; however, considering the aquifers high productivity groundwater diversion appears to be relatively low. A summary of well construction details for wells installed in Aquifer 373 is provided in Table 2-1 and well locations are shown on Figure 3.

Table 2-1: Wells registered as installed in Aquifer 373.

Well Tag Number	License Status	Intended Water Use	Finished Well Depth (ft)	Water Depth (ft)	Bedrock Depth (ft)	Well Yield (US gpm)
22987 (PW 2)	Licensed	Water Supply System	334	flowing	334	4200
22350	Unlicensed	Private Domestic	117	100	-	10
36391	Unlicensed	Unknown Well Use	200	115	-	-
40138	Unlicensed	Private Domestic	82	30	-	10
44996	Unlicensed	Private Domestic	235	135	-	8
47185	Unlicensed	Private Domestic	295	60	-	10
35171	Unlicensed	Private Domestic	41	flowing	-	20
53827	Unlicensed	Private Domestic	120	106	-	10
53727	Unlicensed	Private Domestic	162	106	-	5
13518	Unlicensed	Private Domestic	294	-	-	50
4961	Unlicensed	Water Supply System	319	flowing	-	500
1876	Unlicensed	Unknown Well Use	284	flowing	284	180
12036	Unlicensed	Private Domestic	200	flowing	-	2
4950	Unlicensed	Private Domestic	295	flowing	-	50
118300	Unlicensed	Water Supply System	115	-	-	8
80805	Unlicensed	Private Domestic	216	125	-	26
44522	Unlicensed	Unknown Well Use	307	flowing	311	1200
56929	Unlicensed	Private Domestic	273	40	-	15

Well Tag Number	License Status	Intended Water Use	Finished Well Depth (ft)	Water Depth (ft)	Bedrock Depth (ft)	Well Yield (US gpm)
59715	Unlicensed	Private Domestic	108	95	-	8
1824	Unlicensed	Private Domestic	200	flowing	-	2
2380	Unlicensed	Unknown Well Use	200	flowing	-	2
2042	Unlicensed	Unknown Well Use	150	flowing	-	2
1833	Unlicensed	Private Domestic	200	flowing	-	2

2.3.3 Groundwater Licensing

WWAL completed a search of the provincial water license database, and one groundwater license is present for Aquifer 373 (MFLNRORD, 2021). The license belongs to the District of Fort St. James and the point of diversion is the District's well PW2. Select license details are included for reference in Table 2-2 below. As any future well drilling would be for redundancy purposes and not for an increase in volume a new source groundwater license would not be required. However, the existing license would require an amendment to include additional points of diversion including a new well. Since the District is licenced as an existing user under the Water Sustainability Act, given the purpose of the secondary well, it is likely that the well would be permitted as "additional works" and would not require a new licence but rather an existing licence amendment.

Table 2-2: Summary of water license search.

Water License	Owner	Date of Precedence	Quantity	Source Aquifer
500046	District of Fort St. James	Nov, 19, 1969	292,000 m^3/year	373

2.3.4 Production Well 2 (WTN22987)

WTN22987 was drilled in 1969 by Western Water Wells and Hamelin Drilling Ltd. with a cable tool drill rig under the oversight of hydrogeologists from Robinson, Roberts and Brown Ltd. As it was known that several wells installed in the aquifer underlying the Fort St. James area were flowing artesian, WTN22987 was reportedly drilled with a 20 inch diameter surface casing to a depth of 30.5 m (100 ft) grouted into the ground, 16 inch casing was installed within the surface casing to drill through the aquifer until bedrock was reached, drill mud was used and a 4 ft plug was kept in the casing to control the pressure. After installation of the 16 inch casing, 12 inch casing with 6.4 m (21 ft) of 100 slot screen welded to it was installed within the 16 inch casing as a long-string production assembly. The screen was exposed between 94.5 -100.9 m (310 – 331 ft). The well was developed for 20 hours by bailing and by alternating between shutting the well in and letting it flow freely. Development reportedly increased the rate of free flow from 200 USgpm at the start of development to 4,200 USgpm at the end of development (Robinson, Roberts and Brown Ltd., 1970). Well construction details for PW2 are summarized below in Table 2-3 and the well driller's log for PW2 is provided in Appendix A.

Table 2-3: PW2 Select Construction Details.

Diameter	305 mm (12 inch PS)
Screen Depth	94.5-100.9 m
Screen Length	6.4 m
Slot Size	100 Slot
Free Flow Volume	4200 US gpm
Shut in Pressure	52 PSI
Partial Flow Pressure	40 PSI flowing at 942 US gpm

2.3.5 Previous Hydrogeological Reports

Previous hydrogeological studies reviewed include the well completion report by Robinson, Robert and Brown Ltd. describing the drilling of PW2 and a water supply evaluation completed by Golder Associates (Golder) in 2001.

The well completion report for PW2 prepared by Robinson, Roberts and Brown Ltd. indicates that PW2 was drilled in response to the existing village well producing sand and gravel. Due to the existing well producing sand and gravel it could not meet the District's water requirements without danger of the well collapsing as it was not constructed properly. The existing well was drilled in 1965 with an open ended slotted 8 inch casing and was over excavated during drilling, which led to the well being in danger of collapsing. The existing well reportedly flowed freely at 1500 US gpm and had a shut-in pressure of 48 Psi. As previously mentioned, the artesian aquifer underlying Fort St James was discovered in the 1930s by Alex Nicolson while drilling a 3 inch diameter well. According to Robinson, Roberts and Brown, Mr. Nicholson would advance 4 inch diameter surface casing approximately 30 to 50 ft into the clays overlying the aquifer, then drill a hole into the top of the aquifer in which 3 inch diameter casing was placed. A total of twelve wells were reportedly drilled throughout Fort St James using this methodology, one of which became uncontrolled and created a large crater (or "sinkhole"). The crater phenomenon is typical of some out-of-control flowing wells were the flow to surface causes significant erosion, piping and transport of formation material to surface, causing instability, and ultimately, subsidence at the ground surface in the affected area. The crater location was inferred by Golder to be located at the present-day District office however, based on a review of historical drilling records WWAL has confirmed the crater location to be on Lot 5, the present-day location of Spirit Square Park (MFLNRORD, 1938).

The construction of PW2 was designed in anticipation of encountering relatively large artesian pressures and construction details as outlined by Robinson, Roberts and Brown are described above in Section 2.3.4. After commissioning of PW2, the well was reportedly operated under a system pressure of 50 Psi at a flow rate of 157 USgpm (10 L/s). It was concluded that if the water system was maintained at 40 Psi the well will deliver 942 US gpm (59 L/s). Water from PW2 reportedly has encrusting tendencies and the well may need to be cleaned if deterioration is noted at a level where District water needs are no longer met. Robinson, Roberts and Brown concluded that the Aquifer 373 is capable of substantial development much greater than PW2's capacity

In 2001 Golder completed a groundwater evaluation study to assess the potential for increasing the District's groundwater withdrawal rates from 14.6 L/s (232 USgpm) to 61 L/s (967 USgpm) Three options were evaluated by Golder to increase the District's water supply volumes which were to increase the existing well's withdrawal rate, install a second well in the same aquifer as the existing well or develop other aquifers in the area. Golder concluded that there was no indication for a decline in flow rates from the District's well over time and that there may have been a minor reduction in pressure in the aquifer due to the well's operation but that this reduction in pressure was likely only localized. Golder estimated that the well could provide the estimated Maximum Day Demand (MDD) for 2015, which at the time was thought to be 55.6 L/s, without depleting the aquifer. Considering screen design, Golder estimated the maximum withdrawal rate of the well to be 75 L/s with the exception of fire flows during which they concluded that the well's free flow rate of up to 265 L/s would be acceptable. Golder recommended that a step testing program be conducted on the well and that an additional well would only be needed to provide a back-up supply for the District (Golder, 2001).

2.3.1 Well and Aquifer Performance

A review of previous hydrogeological studies conducted for the Fort St James Aquifer and the District's well indicates that the well and aquifer performance likely have not changed significantly over the years. Although some lower shut-in pressures were noted in 1981, 1997 and 2001, it is likely that shut in pressure has remained relatively stable as constant use of the well does not allow sufficient time for the shut in pressure to fully develop. Reported well and aquifer performance from 1970 to 2021 is summarized in Table 2-4

Table 2-4: Well and Aquifer Performance.

Year	1970[1]	1980[2]	1981 and 1997[2]	2001[2]	2021[3]
Shut in pressure	52 psi	52 psi	40 psi	44	51 psi
Free Flow Rate	4200 (265 L/s) US gpm	264 L/s	-	-	-
Flow Rate	942 Us gpm (59 L/s)at 40 psi	364 US gpm (23 L/s) at 40 psi	1188 US gpm (75 L/s) at 20 psi	184 US gpm (11.6 L/s) at 38 psi	120-182 US gpm (7.6-11.5 L/s) m3/day at 41 psi

1- (Robinson, Roberts and Brown Ltd., 1970)

2 -(Golder, 2001)

3- WWAL Site visit June, 2021

2.4 Water Chemistry

WWAL reviewed the results of six historical water samples and one recent comprehensive water sample to assess if water quality has remained consistent throughout the use of PW2. Water quality parameters analyzed vary between samples and the earliest available water sample was collected in 1969. General water quality parameters including hardness and alkalinity remained relatively consistent in the samples reviewed and indicators of anthropogenic contamination or septic influence on groundwater (chlorides, nitrate/nitrite) also remained low or less than the detection limit. Laboratory turbidity was consistently high (5.82-8.9 NTU) which exceeds recommended treatment objectives. Although turbidity appears higher than recommended, further assessment is recommended including the collection of field turbidity measurements. Metals concentrations appear relatively consistent over time with a slightly increasing boron concentration, still well below guidelines, and a significant decrease in antimony observed in the 2021 sample. Iron and manganese exceed present day aesthetic objectives in all samples and although not a health concern, treatment is typically desired for aesthetic reasons. Elevated iron and manganese are common in confined aquifers in the British Columbia interior. Bacteria has been consistently absent in the reviewed samples and in sampling reported by Northern Health (Northern Health Authority, 2021) which is expected in a deep confined aquifer. Any future wells drilled in Aquifer 373 in proximity to PW2 will likely have similar water quality. A summary of reviewed water quality data is provided in Table 2-5.

Table 2-5: Summary of Reviewed Water Quality Data for PW2.

			Production Well 2						
Date			Nov 1969	Apr 1980	Nov 1989	Mar 1997	Feb 1998	Jan 2000	Jul 2021
Parameter	Units								
pH (field / lab)	pH units	AO = 7 – 10.5	7.8	7.8	7.95	7.57	7.6	7.65	7.97
General Parameters		Guideline							
Turbidity (lab)	NTU	Varies	<0.005	-	-	8	6	8.9	5.82
Total Dissolved	mg/L	AO ≤500	545	-	-	360	379	387	377
Hardness	mg/L		-	-	450	291	289	288	310
Alkalinity	mg/L		354	390.4	317	315	373	321	301
Fluoride	mg/L	MAC = 1.5	-	-	0.3	0.2	0.2	0.1	<0.1
Nitrate, N	mg/L	MAC =10	-	0.0121	0.006	<0.003	0.016	<0.003	<0.1
Nitrite, N	mg/L	MAC = 1	-	-	0.002	<0.003	<0.003	<0.003	<0.1
Chloride	mg/L	AO ≤ 250	1.4	1.39	-	<0.5	1.6	<0.5	<0.5
Sulfate	mg/L	AO ≤ 500	60	46.2	44	50	46	38	53
Selected Ions and Metals									
Aluminum	mg/L	OG < 0.1	-	-	-	<0.05	<0.05	<0.05	<0.001
Antimony	mg/L	MAC = 0.006	-	-	-	0.19	005	0.04	<0.00002
Arsenic	mg/L	MAC = 0.01	-	-	-	<0.001	<0.001	<0.01	0.0068
Barium	mg/L	MAC = 2	-	-	-	<0.08	<0.08	0.04	0.051
Boron	mg/L	MAC = 5	-	-	-	<0.01	<0.01	3.98	0.040
Cadmium	mg/L	MAC = 0.005	-	-	-	<0.0006	<0.0005	<0.0005	<0.00001
Chromium	mg/L	MAC = 0.05	-	-	-	<0.004	<0.004	<0.004	<0.00005
Copper	mg/L	AO ≤ 1	-	-	-	<0.005	<0.005	0.005	<0.0005
Iron	mg/L	AO ≤ 0.30	-	0.613	0.63	0.66	0.37	0.675	0.68
Lead	mg/L	MAC = 0.01	-	-	-	<0.001	<0.001	<0.001	<0.00001
Manganese	mg/L	AO ≤ 0.02 MAC = 0.12	-	-	-	0.031	0.02	0.027	0.030
Selenium	mg/L	MAC = 0.05	-	-	-	<0.01	<0.01	<0.01	<0.0002
Sodium	mg/L	AO ≤ 200	-	-	-	13.3	14	13.3	13
Uranium	mg/L	MAC = 0.02	-	-	-	-	-	-	0.00013
Zinc	mg/L	AO ≤ 5	-	-	-	0.006	<0.002	0.002	0.0005
Microbiological Parameters									
Total Coliforms	CFU/10	MAC < 1	-	-	-	-	-	-	<1
E. Coli	CFU/10	MAC < 1	-	-	-	-	-	-	<1

3. PROPOSED BACK-UP WELL DRILLING AND TESTING PROGRAM

Based on available information regarding PW2, the aquifer (Aquifer 373) in which PW2 is installed in and wells installed within Aquifer 373, we prepared a high-level cost estimate for the drilling and testing of a back-up well. As a new well would solely be for the purpose of adding redundancy to the system and will not be utilized while operating the existing well, a nominal 203 mm (8-inch) production casing well drilled near PW2 is recommended. As the existing well is located in the corner of the secondary school field, sufficient space should be available for a new well.

As flowing artesian conditions with a shut-in pressure in the 50 psi range and a free flow rate of up to 4,200 US gpm are expected to be encountered during drilling and testing of the back-up well. Our cost estimate is based on a well completed with 8-inch diameter production casing to supply the required well yield of 500 US gpm and takes into consideration flowing artesian conditions expected to be encountered during drilling and testing of the back-up well with a shut-in pressure of 50 psi.

The conceptual well design is as follows:

- To fully control artesian flow, our recommended approach would be to drill a 16- inch (400 mm) diameter temporary casing into a suitable confining unit to an approximate depth of 100 ft (30.5 m).
- A12-inch (300 mm) diameter casing would be telescoped inside the 16- inch (400 mm) diameter temporary casing and cemented into place, becoming the "control" or conductor casing providing the long-term control of artesian pressure inside the well. . The 16-inch temporary casing would then be removed after the 12-inch (300 mm) diameter casing is cemented in place.
- After the cement grout has cured, 10-inch (250 mm) intermediate conductor casing would be advanced into bedrock to a depth of approximately 335 ft (102.1 m), followed by installation of 8-inch (200 mm) long-string production casing within the 10-inch (250 mm) casing. The long string assembly would be field-welded on installation and would include the screen section near the bottom of the assembly.
- To control pressure when drilling into the aquifer, drilling would be completed with mud (barite weighted) below a depth of approximately 280 ft (85.3 m). To achieve a yield of 500 US gpm, we included a 30 ft long well screen in our cost estimate.
- Following well drilling and development, we recommend conducting a pumping test program consisting of a step test followed by a constant rate test along with water quality sampling. The constant rate test would either be a long-term artesian flow test, or a pumping test. The details and advantages to each approach would be discussed with the team during final design and tendering. Our cost estimate for planning and budgeting purposes includes a pumping test.

Due to the large pressure and flow expected, significant amounts of water and cuttings will be produced throughout the drilling and testing program. Cuttings and water will need to be managed and this will likely be associated with considerable cost and logistics. We have included a rough estimate of costs associated with cuttings and water management in the budget. Such costs would depend on the details developed at the time of final design and tendering. This approach to managing cuttings and drilling/development water would likely include the construction of permanent 12 inch piping from the well site to Stuart Lake along with sump pits dug in the adjacent field for sediment control; possibly combined with large containers to manage cuttings and water prior to discharge.

The Table below provides high level cost estimates and assumptions for the back-up well program. Also, few drillers with sufficient experience necessary to drill the above recommended well are licensed to drill in British Columbia. Careful consideration should be made during any tendering process to allow the selection of the appropriate driller as only a few in the province have the equipment and experience necessary for drilling in this type of formation.

Table 3-1: High Level Cost Estimate to Drill and Test a Back-up Well

Task	Cost Estimate	Comments
Drill 8-inch diameter back-up well	$300,000	Does not include management of cuttings and water during drilling
Cuttings and water management	$50,000	May include piping to Stuart Lake, settling ponds, Baker Tanks
Flow Testing, water quality samples	$25,000	
Hydrogeology (tendering, field oversight, initial permitting, reporting)	$50,000	
Contingency (15%)	$63,750	
Total	**$488,750**	

4. CONCLUSIONS

C1 The District of Fort St. James currently relies on one well (PW2 or WTN22987) to provide potable water to connections located within the District as well as on Nak'azdli IR1. The District does not have a back-up water supply to draw from in the event of a sudden loss of PW2.

C2 PW2 sources water from provincially mapped Aquifer 373, a highly productive, confined glaciofluvial sand and gravel aquifer with low vulnerability to contamination. Flowing artesian conditions have been reported in several wells installed within Aquifer 373 including in PW2. The shut-in pressure of PW2 following drilling was reported at 52 psi and the free flow rate was reported at 4,200 US gpm.

C3 Based on our review of the District's current water supply well, provincially mapped Aquifer 373 as well as other wells installed within this aquifer, we believe that Aquifer 373 is capable of providing sufficient capacity to establish a back-up well for the District. The indicated flow requirement of approximately 500 US gpm (31.5 L/s) is feasible from a well with an 8 inch diameter (200 mm) production casing.

C4 Available water quality data indicate that groundwater from PW2 exceeds the GCDWQ AO for iron and manganese. Elevated iron and manganese are common in confined aquifers in the British Columbia interior. Antimony has historically been noted in excess of the GCDWQ MAC; however, appears to have declined to below the laboratory detection limit in 2021. Bacteria have been consistently absent in the reviewed samples and in sampling reported by Northern Health Metals. Laboratory turbidity was consistently high (5.82-8.9 NTU) and exceeds recommended treatment objectives. Although turbidity appears higher than recommended, further assessment is recommended including the collection of field turbidity measurements. Any future wells drilled in Aquifer 373 in proximity to PW2 will likely have similar water quality.

5. RECOMMENDATIONS

R1 We recommend the installation of a nominal 8-inch diameter back-up well in proximity to PW2 to provide redundancy for the District's water system.

R2 A back-up well should be designed to account for high pressures and flow rates encountered within Aquifer 373 during drilling of PW2. We recommend that a water and cuttings management plan is developed and implemented as part of the back-up well drilling program.

R3 We recommend the collection of an additional water quality sampling to confirm the sudden decrease in antimony observed in the 2021 sample. Field turbidity samples are also recommended to determine if turbidity is actually high or as a result of precipitation of metals prior to laboratory analysis.

R4 We recommend collection of well head pressures from PW2 while the District booster pump is operating including well head pressure following booster pump shut off. Regular collection of well head pressure and booster pump flow rate may allow for the development of pressure relief strategies during the drilling of future nearby wells.

6. CLOSURE

We trust the information provided in this report is sufficient for your needs at this time. If there are questions or you wish to discuss the results presented, please contact WWAL at (250)-614-6645.

Western Water Associates
(EGBC Permit to Practice # 1001419)

PROFESSIONAL GEOSCIENTIST
PROVINCE OF BRITISH COLUMBIA
W. M. GRAFTON
40751

Jan 17, 2023

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
L. A. GARDINER
51401

Jan 17, 2023

Senior Review:

Warren Grafton, P.Geo.
Hydrogeologist

Lisa Gardiner, P.Eng.
Environmental Engineer

Douglas Geller, M.Sc., P.Geo.
Principal Hydrogeologist

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Figures

District of Fort St James – Groundwater Supply Assessment
WWAL Ref: 21-035-01PG

Figure 1: Site Plan			
Date:September 2021	Image Source: BC Water Resources Atlas		WWAL Project: 21-035-01PG
Drawn by: LG	Checked by: WG	Client: District of Fort St. James	Client Project: N/a

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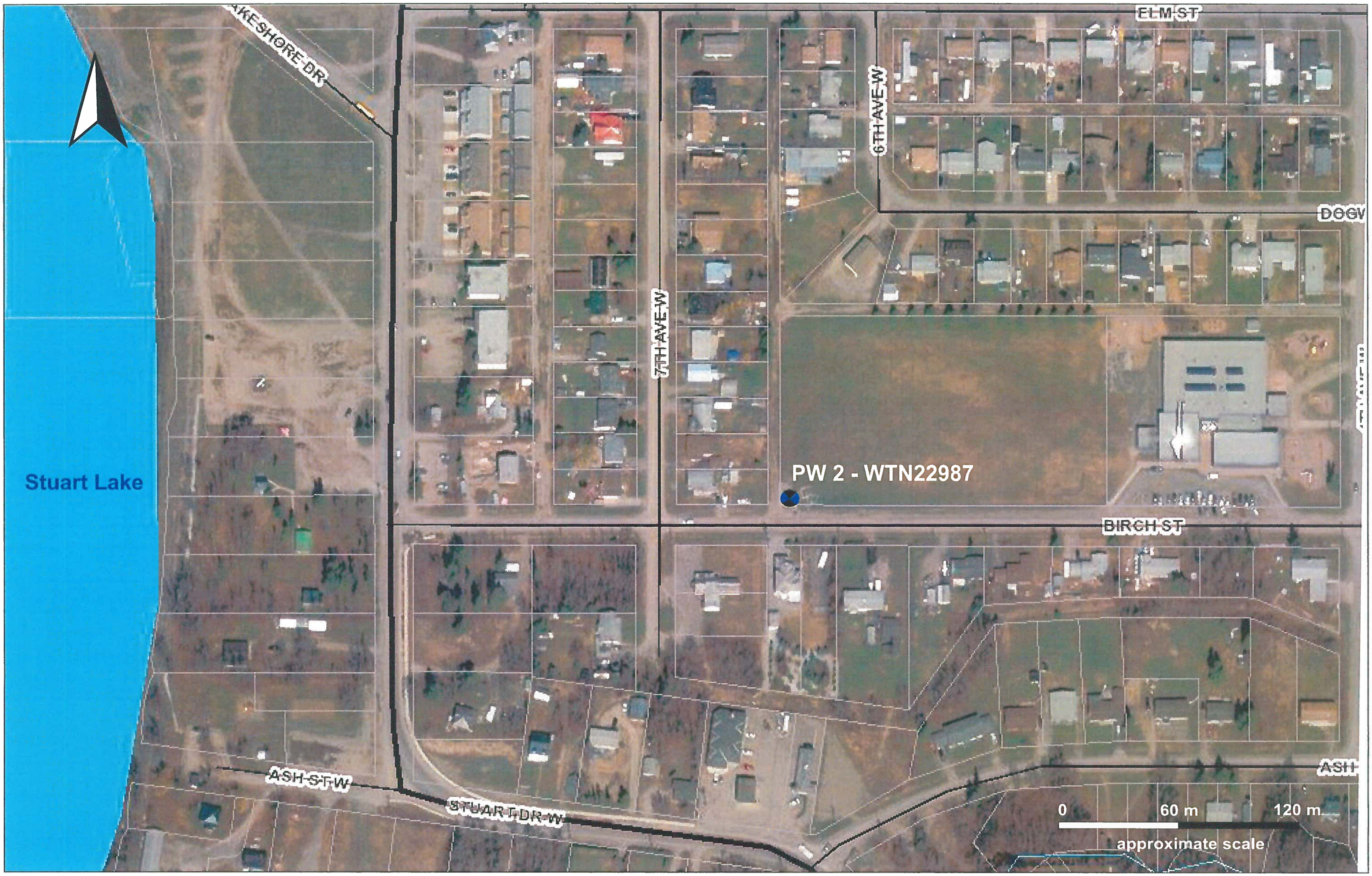

Figure 2: Site Plan			
Date: September 2021	Image Source: RDBN Mapping		WWAL Project: 21-035-01PG
Drawn by: LG	Checked by: WG	Client: District of Fort St. James	Client Project: N/a

western water ASSOCIATES LTD
Consultants in Hydrogeology and Water Resources Management

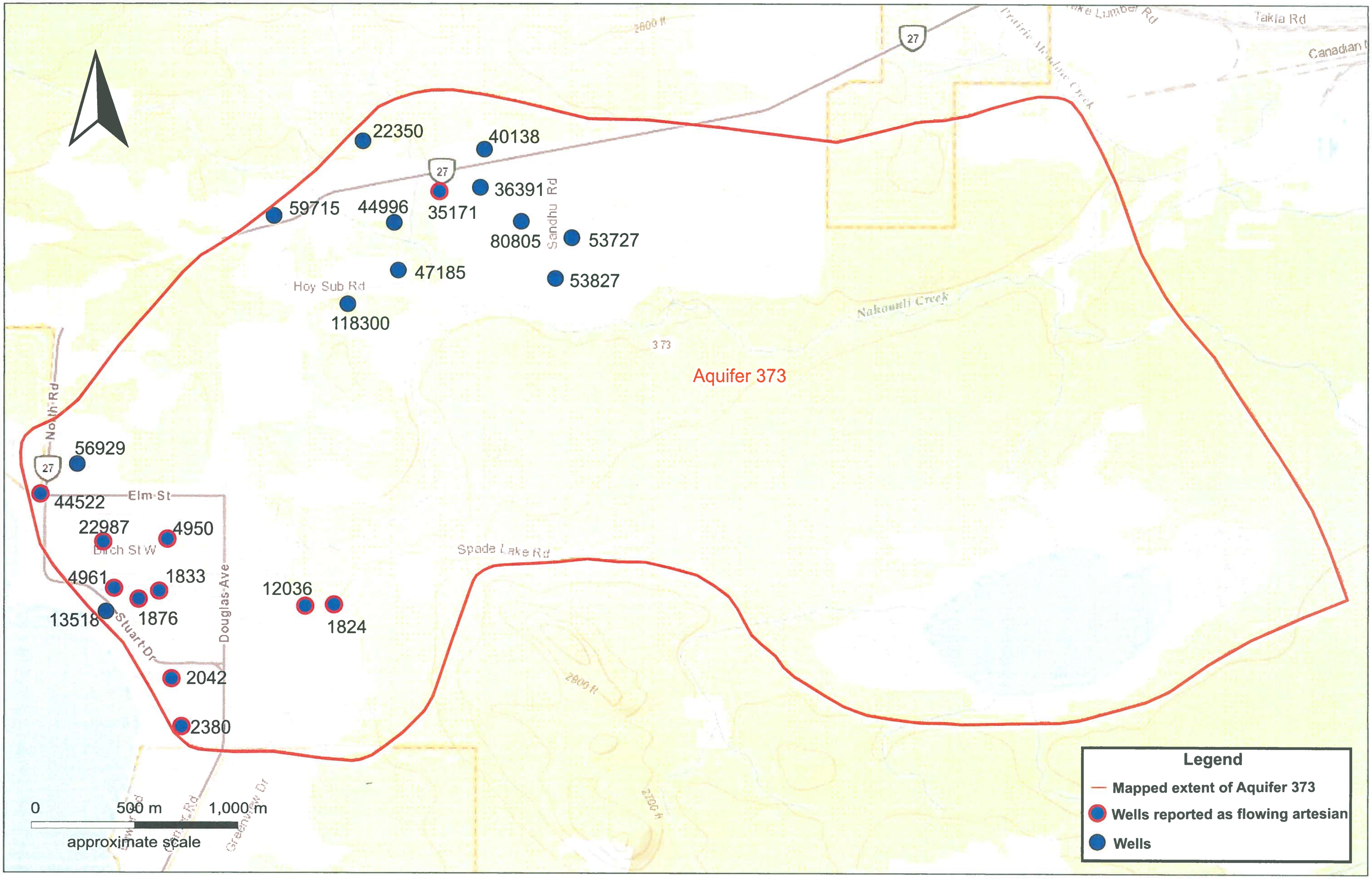

Figure 2: Aquifer 373 and Wells (identified by WTN) interpreted to be installed in Aquifer 373

Date: September 2021	Image Source: BC Water Resources Atlas		WWAL Project: 21-035-01PG
Drawn by: LG	Checked by: WG	Client: District of Fort St. James	Client Project: N/a

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Appendix A

Well Driller's Log

PW2 (WTN22987)
District of Fort St James – Groundwater Supply Assessment
WWAL Ref: 21-035-01PG

Well Tag Number: 22987	Drilling Method: Cable Tool	Development Method: Cycle of closing and allowing to flow
	Driller: Western Water Wells	Artesian Flow: 4,200 US gpm
Informal Well Name: Fort St. James Well 2	Construction Date: November 1969	Shut in Pressure: 52 PSI

Generalized Lithology

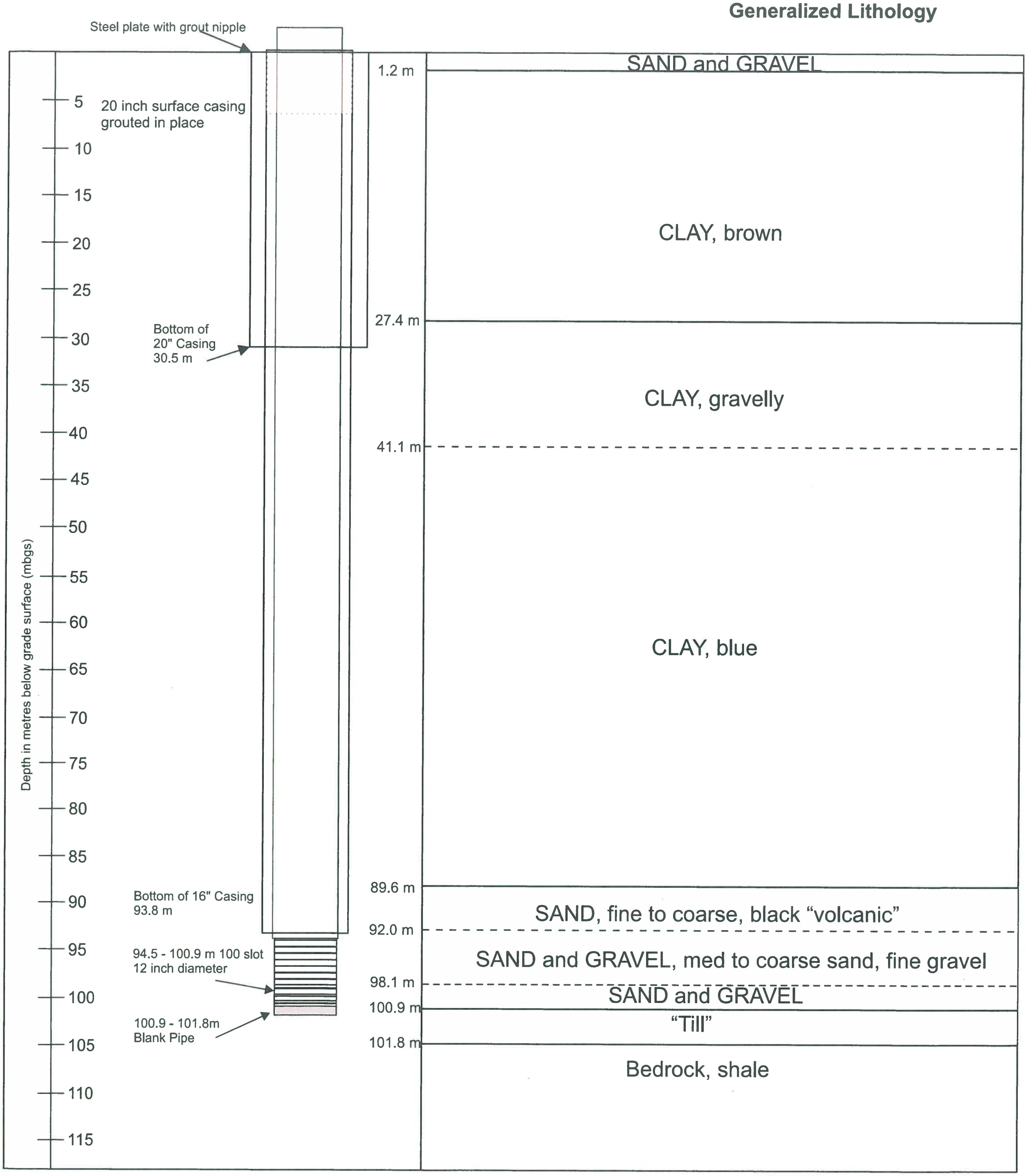

Figure 1: Completion Schematic WTN 22987		
Date: April 19, 2021	Project: 21-035-01PG	western water ASSOCIATES LTD
Drawn by: WG	Checked by: WG	Consultants in Hydrogeology and Water Resources Management

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Groundwater Supply Development and Management

Source Water Assessment and Protection

Well Monitoring & Maintenance

Environmental & Water Quality Monitoring

Storm & Wastewater Disposal to Ground

Groundwater Modeling

Aquifer Test Design and Analysis

Geothermal Heat Exchange Systems

Policy and Guideline Development

Applied Research

Expert Witness Services

Environmental Assessment & Permitting

Appendix B CLASS C ENGINEER'S OPINION OF PROBABLE CONSTRUCTION COST

Detailed cost breakdowns, along with vendor quotes

Fort St. James WTP
Project No: 115621021

Class C Engineer's Opinion of Probable Construction Cost - Option 1 - Green Sand Filters

Item No.	Description	Unit	QTY	Unit Price (CAD)	Total Costs
1.0	**General Requirements (7 % of Project Subtotal)**				
	Mobilization and Demobilization	LPSM	1	$ 150,000	$150,000
	Bonding and Insurance	LPSM	1	$ 150,000	$150,000
	Environmental Management Plan and Erosion Control	LPSM	1	$ 150,000	$150,000
	General Sub Total				*$450,000*
2.0	**Site Civil**				
	New Water Supply Well	LPSM	1	$ 488,750	$488,750
	Stripping	Sq. m	1800	$ 10	$18,000
	Water Main	l.m.	65	$ 400	$26,000
	200mm Water Tie-in	l.m.	1	$ 20,000	$20,000
	200 Gate Valve	EA	4	$ 5,500	$22,000
	200mm SDR35 Sanitary	l.m.	21	$ 400	$8,400
	Excavation to Waste - Birch Street	cu. m	430	$ 25	$10,750
	600mm SGSB	cu. m	260	$ 50	$13,000
	150mm WGB	cu. m	65	$ 40	$2,600
	65mm asphalt	Sq. m	430	$ 80	$34,400
	600mm Culvert	l.m.	30	$ 25	$750
	Excavation to Waste - Alley Lane	cu. m	370	$ 25	$9,250
	600mm SGSB	cu. m	225	$ 50	$11,250
	150mm WGB	cu. m	60	$ 40	$2,400
	65mm asphalt	Sq. m	375	$ 80	$30,000
	Civil Sub Total				*$698,000*
3.0	**Process Mechanical**				
	Raw Water Supply Pumps	EA	2	$ 17,600	$ 35,200
	Green Sand Filters (Includes Ancilliary Equipment)	EA	3	$ 600,000	$ 1,800,000
	Hypochlorite Dosing Pumps Duplex Skid (For Disinfection)	EA	1	$ 37,500	$ 37,500
	Finished Water Distribution Pumps	EA	4	$ 25,000	$ 100,000
	Fire Pump	EA	1	$ 75,000	$ 75,000
	Chemical Storage Tanks (2 * 2000 USG)	EA	2	$ 37,500	$ 75,000
	Equipment Sub Total				*$ 2,123,000*
4.0	**Electrical and Instrumentation Cost**				
	BC Hydro	LPSM	1	$ 30,000	$ 30,000
	Instrumentation	LPSM	1	$ 100,000	$ 100,000
	Generator (200kVA)	LPSM	1	$ 150,000	$ 150,000
	MCC	LPSM	1	$ 120,000	$ 120,000
	VFD	EA	5	$ 8,000	$ 40,000
	PLC	LPSM	1	$ 50,000	$ 50,000
	SCADA	LPSM	1	$ 20,000	$ 20,000
	Junction Box, Conduits, Cable, Misc., General Power, Receptices	LPSM	1	$ 50,000	$ 50,000
	Commissioning and Startup	LPSM	1	$ 20,000	$ 20,000
	EIC Sub Total				*$ 580,000*
5.0	**Structural and Building**				
	450 sq.m Building	sq.m	450	$ 3,500	$ 1,575,000
	Finished Water Storage Tank	cu.m	270	$ 2,500	$ 675,000
	Base Slab and Foundation	cu.m	180	$ 1,500	$ 270,000
	Structural Sub Total				$ 2,520,000
	Sub Total				***$ 6,371,000***
	Geotech Site Investigation	LPSM		$ 20,000	$ 20,000
	Site Survey	LPSM		$ 25,000	$ 25,000
	Engineering Fees - 15% of Construction Sub-Total	LPSM		$ 956,000	$ 956,000
	Contingency 35%	LPSM		$ 2,229,850	$ 2,229,850
				Total	**$ 9,602,000**

Fort St. James WTP
Project No: 115621021

Class C Engineer's Opinion of Probable Construction Cost - Option 2 - GSF and RO

Item No.	Description	Unit	QTY	Unit Price (CAD)	Total Costs
1.0	**General Requirements (15% of Project Subtotal)**				
	Mobilization and Demobilization	LPSM	1	$ 150,000	$150,000
	Bonding and Insurance	LPSM	1	$ 150,000	$150,000
	Environmental Management Plan and Erosion Control	LPSM	1	$ 150,000	$150,000
	General Sub Total				*$450,000*
2.0	**Site Civil**				
	Drill New Water Supply Well	LPSM	1	$ 488,750	$488,750
	Stripping	Sq. m	1800	$ 10	$18,000
	Water Main	l.m.	65	$ 400	$26,000
	200mm Water Tie-in	l.m.	1	$ 20,000	$20,000
	200 Gate Valve	EA	4	$ 5,500	$22,000
	200mm SDR35 Sanitary	l.m.	21	$ 400	$8,400
	Excavation to Waste - Birch Street	cu. m	430	$ 25	$10,750
	600mm SGSB	cu. m	260	$ 50	$13,000
	150mm WGB	cu. m	65	$ 40	$2,600
	65mm asphalt	Sq. m	430	$ 80	$34,400
	600mm Culvert	l.m.	30	$ 25	$750
	Excavation to Waste - Alley Lane	cu. m	370	$ 25	$9,250
	600mm SGSB	cu. m	225	$ 50	$11,250
	150mm WGB	cu. m	60	$ 40	$2,400
	65mm asphalt	Sq. m	375	$ 80	$30,000
	Civil Sub Total				*$698,000*
3.0	**Process Mechanical**				
	Raw Well Water Supply Pumps	EA	2	$ 17,600	$ 35,200
	Green Sand Filters (Includes Ancilliary Equipment)	EA	3	$ 525,000	$ 1,575,000
	Reverse Osmosis	EA	3	$ 550,000	$ 1,650,000
	Hypochlorite Dosing Pumps Duplex Skid (For Disinfection)	EA	1	$ 37,500	$ 37,500
	Finished Water Distribution Pumps	EA	4	$ 17,600	$ 70,400
	Fire Pump	EA	1	$ 75,000	$ 75,000
	Chemical Storage Tanks (2 * 2000 USG)	EA	2	$ 37,500	$ 75,000
	Equipment Sub Total				*$ 3,519,000*
4.0	**Electrical and Instrumentation Cost**				
	BC Hydro	LPSM	1	$ 30,000	$ 30,000
	Instrumentation	LPSM	1	$ 100,000	$ 100,000
	Generator (200kVA)	LPSM	1	$ 150,000	$ 150,000
	MCC	LPSM	1	$ 120,000	$ 120,000
	VFD	EA	5	$ 8,000	$ 40,000
	PLC	LPSM	1	$ 50,000	$ 50,000
	SCADA	LPSM	1	$ 20,000	$ 20,000
	Junction Box, Conduits, Cable, Misc., General Power, Receptices	LPSM	1	$ 50,000	$ 50,000
	Commissioning and Startup	LPSM	1	$ 20,000	$ 20,000
	EIC Sub Total				*$ 580,000*
5.0	**Structural and Building**				
	600 sq.m Building	sq.m	600	$ 3,500	$ 2,100,000
	Underground Finished Water Storage Tank 650 m3	cu.m	270	$ 2,500	$ 675,000
	Base Slab and Foundation	cu.m	180	$ 1,500	$ 270,000
	Structural Sub Total				$ 3,045,000
	Sub Total				***$ 8,292,000***
	Geotech Site Investigation	LPSM		$ 20,000	$ 20,000
	Site Survey	LPSM		$ 25,000	$ 25,000
	Engineering Fees	LPSM		$ 1,243,800	$ 1,243,800
	Contingency 35%	LPSM		$ 2,902,200	$ 2,902,200
				Total	**$ 12,483,000**

Fort St. James WTP
Project No: 115621021

Class C Engineer's Opinion of Probable Construction Cost - Option 3 - BioFiltration

Item No.	Description	Unit	QTY	Unit Price (CAD)	Total Costs
1.0	**General Requirements (15% of Project Subtotal)**				
	Mobilization and Demobilization	LPSM	1	$ 150,000	$150,000
	Bonding and Insurance	LPSM	1	$ 150,000	$150,000
	Environmental Management Plan and Erosion Control	LPSM	1	$ 150,000	$150,000
	General Sub Total				*$450,000*
2.0	**Site Civil**				
	Drill New Water Supply Well	LPSM	1	$ 488,750	$488,750
	Stripping	Sq. m	1800	$ 10	$18,000
	Water Main	l.m.	65	$ 400	$26,000
	200mm Water Tie-in	l.m.	1	$ 20,000	$20,000
	200 Gate Valve	EA	4	$ 5,500	$22,000
	200mm SDR35 Sanitary	l.m.	21	$ 400	$8,400
	Excavation to Waste - Birch Street	cu. m	430	$ 25	$10,750
	600mm SGSB	cu. m	260	$ 50	$13,000
	150mm WGB	cu. m	65	$ 40	$2,600
	65mm asphalt	Sq. m	430	$ 80	$34,400
	600mm Culvert	l.m.	30	$ 25	$750
	Excavation to Waste - Alley Lane	cu. m	370	$ 25	$9,250
	600mm SGSB	cu. m	225	$ 50	$11,250
	150mm WGB	cu. m	60	$ 40	$2,400
	65mm asphalt	Sq. m	375	$ 80	$30,000
	Civil Sub Total				*$698,000*
3.0	**Process Mechanical**				
	Raw Well Water Supply Pumps	EA	2	$ 17,600	$ 35,200
	BioFiltration	EA	3	$ 453,000	$ 1,359,000
	Hypochlorite Dosing Pumps Duplex Skid (For Disinfection)	EA	1	$ 37,500	$ 37,500
	Finished Water Distribution Pumps	EA	4	$ 17,600	$ 70,400
	Fire Pump	EA	1	$ 75,000	$ 75,000
	Chemical Storage Tanks (2 * 2000 USG)	EA	2	$ 37,500	$ 75,000
	Equipment Sub Total				*$ 1,652,100*
4.0	**Electrical and Instrumentation Cost**				
	BC Hydro	LPSM	1	$ 30,000	$ 30,000
	Instrumentation	LPSM	1	$ 100,000	$ 100,000
	Generator (200kVA)	LPSM	1	$ 150,000	$ 150,000
	MCC	LPSM	1	$ 120,000	$ 120,000
	VFD	EA	5	$ 8,000	$ 40,000
	PLC	LPSM	1	$ 50,000	$ 50,000
	SCADA	LPSM	1	$ 20,000	$ 20,000
	Junction Box, Conduits, Cable, Misc., General Power, Receptices	LPSM	1	$ 50,000	$ 50,000
	Commissioning and Startup	LPSM	1	$ 20,000	$ 20,000
	EIC Sub Total				*$ 580,000*
5.0	**Structural and Building**				
	600 sq.m Building	sq.m	450	$ 3,500	$ 1,575,000
	Finished Water Storage Tank	cu.m	270	$ 2,500	$ 675,000
	Base Slab and Foundation	cu.m	180	$ 1,500	$ 270,000
	Structural Sub Total				$ 2,520,000
	Sub Total				***$ 5,901,000***
	Geotech Site Investigation	LPSM		$ 20,000	$ 20,000
	Site Survey	LPSM		$ 25,000	$ 25,000
	Engineering Fees	LPSM		$ 885,150	$ 885,150
	Contingency 35%	LPSM		$ 2,065,350	$ 2,065,350
				Total	**$ 8,897,000**