KERR WOOD LEIDAL
consulting engineers

Greater Vancouver
200 - 4185A Still Creek Drive
Burnaby, BC V5C 6G9
T 604 294 2088
F 604 294 2090

Water Treatment Investigation

Final Report
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KWL Project No. 0743.016-300

Prepared for:

Village of PEMBERTON

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1. Introduction

This report has been prepared for the Village of Pemberton (Village) and summarizes the completed water treatment investigation regarding three (3) groundwater wells that provide water to the Village. Water is currently supplied by two groundwater wells (Wells 2 and 3) that are connected to the Pemberton Creek Fan Aquifer. A third well (Well 1) is no longer connected to the system. Water quality data collected from 2009 to 2020 indicate periods in which iron and manganese levels in the well water exceeded the Health Canada guidelines for aesthetic parameters. In May of 2019, Health Canada lowered the aesthetic limit for manganese and introduced a new health-based limit for manganese. The health-based limit was the outcome of recent research and peer reviewed studies.

The purpose of this investigation was to review up to three available water treatment options that would provide Village residents with potable water that meets Canadian Drinking Water Quality (GCDWQ) guidelines. This report addresses the following tasks:

1. Investigate potential treatment options to address water quality concerns in the Village water system;
2. Evaluate different treatment options and recommend the most suitable option for the application;
3. Identify proposed water treatment plant (WTP) configurations based on locations and access to sanitary system; and
4. Provide a Capital and Operation and Maintenance (O&M) cost opinion for the recommended options.

2. Background/Justification

A review of the existing water system with emphasis on the performance of the source aquifer was performed by the Village in 2020. Water quality results from the Village wells report iron and manganese levels exceeding the Aesthetic Objectives of 0.3 mg/L for iron (Wells 1 and 2) and 0.02 mg/L for manganese stated in the GCDWQ (Wells 1, 2, and 3). Manganese levels in Wells 1 and 2 also exceed the Maximum Allowable Concentration (MAC) of 0.12 mg/L.

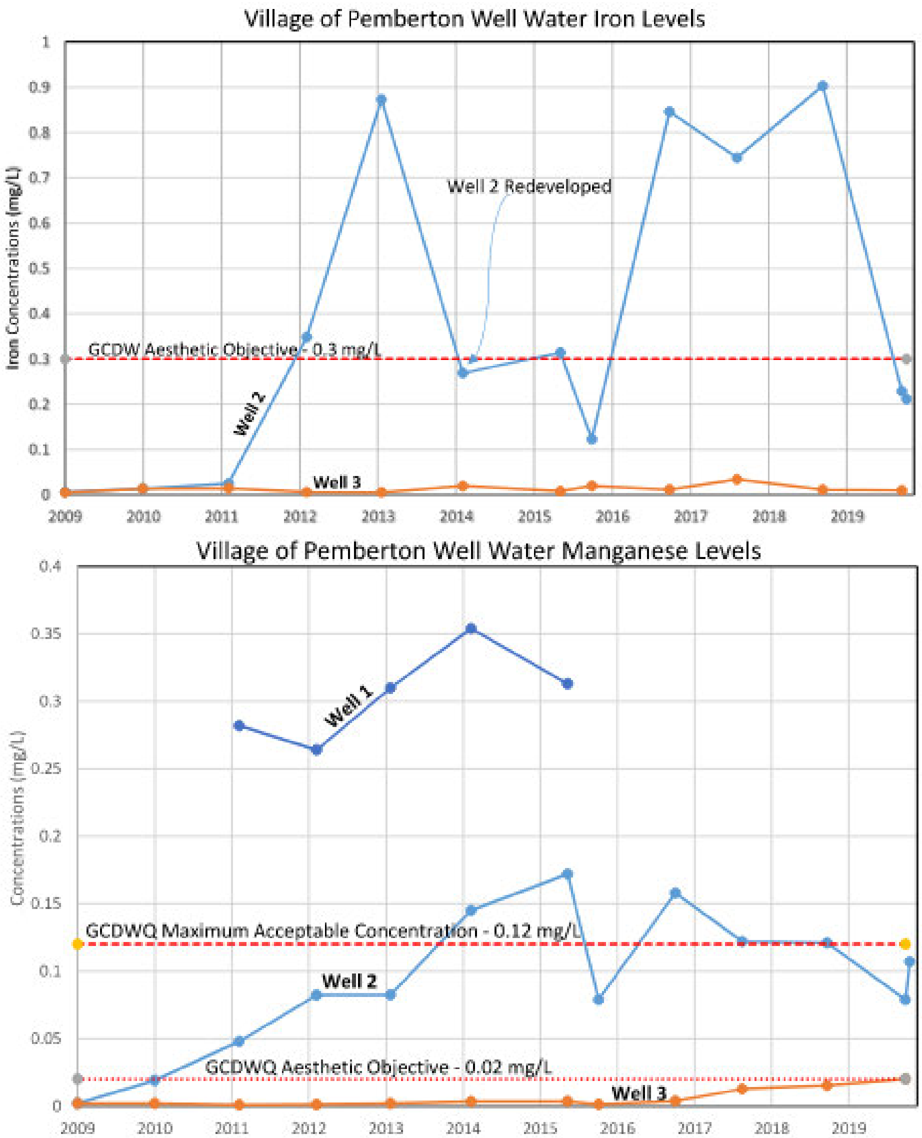


Figure 2-1: Water Quality Data for Wells 1, 2, and 3 [1]

[1] Village of Pemberton, *Water System Performance Assessment*, 2020. Well 1, not shown on the iron level graph due to scale, had a test result of 16.7 mg/L in 2013, the last time it was tested.

3. Existing System

The existing groundwater system consists of two (2) wells (Wells 2 and 3) that distribute water, through mostly 50 to 300 mm PVC piping, from the Benchlands reservoirs, throughout the Village core and adjacent neighbourhoods, and eastwards to Pemberton Farm Road East and Airport Road, and North towards PNWS. Sections of existing piping are comprised of asbestos cement that are scheduled to be replaced. The existing system provides soda ash conditioning to increase the pH from 6.5 to 6.8, and chlorination for both primary disinfection and to maintain a minimum free chlorine residual of 0.2 mg/L at the farthest ends of the distribution system. There are no other treatment processes in place with respect to reduction of iron or manganese in the raw water. The tables and enclosed information presented in this section are extracted from the Village of Pemberton's *Water System Performance Assessment* completed in 2020.

A third well (Well 1) is no longer connected to the distribution system due to high levels of iron and manganese.

The Per Capita Demand (in litres per capita per day, or LPCD) and Design Pressures are summarized in Table 3-1.

Table 3-1: Per Capita Demand

Per Capita Demand	
Average Daily Domestic Flow	455 LPCD
Maximum Daily Domestic Flow	910 LPCD
Peak Hour Domestic Flow	1,820 LPCD
Design Pressures	
Minimum Pressure at Peak Demand	300 kPa (44 psi)
Maximum Allowable Pressure	850 kPa (123 psi)
Minimum Pressure for Fire Flow Plus Max Day Demand	150 kPa (22 psi)

There are three (3) reservoirs totaling 4,511 m^3 of storage. The total required storage is 2,506 m^3 which leaves 2,045 m^3 for future expansion. Table 3-2 summarizes the relevant information for each reservoir. This calculation was completed in Section 2.3 of Village of Pemberton's *Water System Performance Assessment* (2020).

Table 3-2: Existing Reservoir Details

Reservoir	Year Constructed	Type	Capacity (m^3)	Top Water Level Elevation (m)
Benchlands Reservoir 1	2002	Circular Concrete Tank	1,640	290.5
Benchlands Reservoir 2	2014	Circular Steel Tank	1,490	290.5
Ridge Reservoir	2017	Circular Steel Tank	1,421	357.6

Table 3-3 summarizes the year of construction, diameter, depth, rated flow, location, and general notes of the three wells in the Village.

Table 3-3: Existing Well Details

Well	Construction Year	Diameter (mm)	Depth (m)	Rated Flow (L/s)	Location	Notes
1	1992	200	29	28.8	Well house	Isolated from distribution due to declining yield and poor quality.
2	1997	300	42	68	Foughberg Park	Current backup well.
3	2007	300	46	52	Pioneer Park	Current duty well.

4. Future Development

Skénkenam Development Limited Partnership applied to develop certain lands within the Pemberton Benchlands (known as the Nkukwma project), as referenced in the Village of Pemberton Official Community Plan (OCP). The plan indicates Block A DL 8556 (10.6 ha), Block J DL 202 (9.69 ha), and Block I DL 202 (11 ha) will potentially be developed into single family homes, duplexes, townhomes, and apartments. However, Block K 8410, DL 202, and DL 2297 are not included due to concerns with historical contamination of these sites. It is possible that these sites will be remediated to a standard suitable to residential development, while development of these blocks will continue over the next 30+ years.

The Water Distribution plan for Pemberton Benchlands includes construction of a new reservoir on the upper limits of the Block J DL 202 development boundary, a pump station adjacent to the existing reservoirs, a second pump station next to the newly proposed reservoir, supply of relevant back-up generators, and relocation of the existing reservoir's supply and distribution mains to follow the new proposed alignment of the collector road.

There are other potential future developments in the village which include 40-120 more units in the Ridge/Sunstone Area, 30-90 more units in the downtown area, and 60-70 more in the Glen area. These projects total 130 to 280 more units. The previously mentioned Nkwukwma project totals 200 to 400 projects over the next 10 years. Combining all future developments totals 330 to 680 potential additional units over the next 20 years.

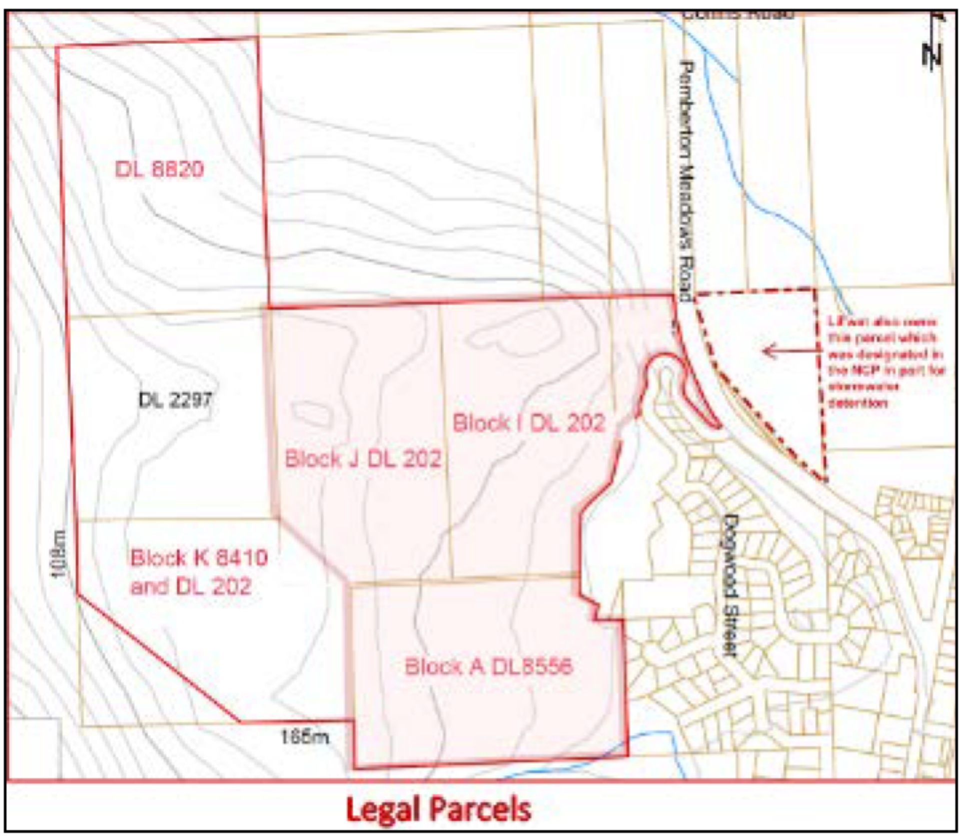


Figure 4-1: Developer Plans

The north side of the reservoirs is surrounded by proposed townhomes. On the east side of the reservoirs, there is a proposed park. There is also a proposed pump station located in this park that would service zone 3 (a small number of lots within the current development as well as the whole extent of the future development to the west).

The development will increase the population by approximately 1,252 people. For the proposed and future developments, this will create an average daily domestic flow of 569,660 L/day (6.6 L/s), a maximum daily domestic flow of 1,139,320 L/day (13.2 L/s) and a peak domestic flow of 2,258,640 L/day (26.1 L/s) for the new development alone.

Skénkenam Development Limited Partnership is funding the Village's engineers to update/develop existing village infrastructure, water/sewer/storm models to determine capacity.

5. Water Treatment Investigation

Three water treatment options to reduce the iron and manganese levels in the system were considered. This section briefly summarizes these options.

5.1 Option 1: Oxidation and Filtration using Catalytic Media

Most iron and manganese removal treatment processes require oxidation as the first step of treatment to precipitate the iron and manganese dissolved in the water. Normally this is done by injecting the source water with an oxidant such as sodium hypochlorite or potassium permanganate. Once oxidized, the precipitates can be settled or filtered out. Sufficient oxidant must also be added to ensure the adsorption characteristics of the GreenSandPlus™ are regenerated to continue to attract any dissolved manganese.

Media filtration with GreenSandPlus™ media is an effective and proven means for reducing both iron and manganese in dissolved or precipitated form in raw water. In a GreenSandPlus™ media filter the media acts as a catalyst for the iron and manganese oxidation process. As water passes through the filter bed, the oxidized iron and manganese are retained by the filter media and their concentration in the water reduces as water progresses downward through the filter. The filter would require periodic backwashing to remove the accumulated iron and manganese precipitate.

GreenSandPlus™ media can remove both iron and manganese but removal efficiency of each parameter varies depending on the pH of the water as well as the concentrations of other constituents in the water. Pilot testing is usually completed to establish the removal efficiency of iron and manganese in a specific water. As a minimum, bench scale testing with the actual water should be completed prior to full-scale implementation.

5.2 Option 2: Oxidation and Media Filtration

This treatment process incorporates oxidation of iron and manganese in the water to convert the dissolved forms of the metals to a solid. Often exposure to air is sufficient for oxidizing iron, but for manganese, a stronger oxidant such as ozone or potassium permanganate is used in the oxidation process. Following the oxidation process, water passes through sand media filters to filter out the formed precipitate. Sodium hypochlorite is then dosed to provide virus inactivation and secondary chlorine residuals.

Sand media filters are either gravity or pressure type. The filters are backwashed periodically for removing the precipitated material on the surface of the filters.

5.3 Option 3: Biological Treatment

Biological filters are designed to remove soluble iron and manganese from the water supply by the biological activity and uptake of impurities by the naturally occurring bacteria retained in the filter media. Unlike Options 1 and 2, biological treatment does not require any chemical oxidants and relies on usually two stages of biological filters.

The process consists of raw water passing through the biological filters, where conditions are established to promote the growth of specific bacteria for iron removal, and a different type of bacteria for manganese removal. Soluble particles will build up and be retained in the filter media and form dense and compact precipitates. Over time, insoluble particles build up in the filters and backwashing is required to remove the build up. Due to the compaction of precipitates and longer filter times, the biological treatment process has a longer retention time and therefore allows the system to achieve longer filter run times. Air is injected into the raw water prior to entry into the biological filters to foster bacteria growth.

For application related to the biological treatment of iron and manganese, the process system will require individual treatment (or two stages in a series) to meet the required environmental conditions for biological removal of iron and manganese. This requires controlled aeration and filtration for biological iron removal and intensive aeration and filtration for biological manganese removal. [2] Biological treatment can be applied in gravity or pressure filters, where pressure filters are designed for high-rate operations.

[2] Sharma K.K. Petruseveski B, & Schippers J.C. 2005 *Biological Iron Removal from Groundwater: A Review*.

6. Treatment Options Discussion

Table 6-1 lists the advantages and disadvantages of the treatment options for the existing groundwater source.

Table 6-1: Comparison of Technologies to Treat Existing Groundwater Source

Water Treatment Technology	Advantages	Disadvantages	O&M Requirements
Oxidation and Catalytic Media Filtration	• Can effectively remove both iron and manganese in combination with oxidation. • Relatively simple operation. • Media is readily available and can be ordered ahead of time. • Chlorination provides continuous insitu media regeneration and primary and residual disinfection.	• Generation of backwash wastewater.	• Periodic backwashing of catalytic media. • Oxidant chemical usage.
Oxidation and Media Filtration	• Relatively simple operation. • Media is readily available and can be ordered ahead of time.	• Not as effective at removing dissolved manganese, compared to catalytic media. • Need for strong pre-oxidant and hypochlorite. • Generation of backwash wastewater.	• Periodic backwashing or replacement of filter media. • Multiple chemicals used.
Biological Treatment	• Can effectively remove both iron and manganese in combination with air oxidation. • No strong oxidants required. • Lower backwash requirements and reduced backwash water quantities.	• Higher initial cost due to the requirement of oxidation using an air compressor. • Relies on naturally occurring bacteria and appropriate environment to consume iron and manganese. • Usually requires two stage filtration step for removal of iron and then manganese. • Always a risk of a biological process upset that results in poor water quality that takes time to resolve.	• More effort and skill required to maintain and operate the system. • Complex maturation for new filters.

6.1 Recommended Water Treatment Process

Oxidation and catalytic (GreenSandPlus™) filtration for the specific removal of iron and manganese is the preferred treatment option for the existing source based on the information summarized in Table 6-1.

Options 1 and 2 are similar in process and configuration; however, the primary process difference is that Option 1 only uses chlorination process as the pre-oxidant with GreendSandPlus™. The chlorination pre-treatment completes two steps; step one allows for continuous regeneration of the GreenSandPlus™, and step 2 provides for 4-log virus inactivation and a secondary chlorine residual of the treated water.

For Option 2, a stronger pre-oxidant other than chlorine is required and involves the introduction of another chemical (i.e., potassium permanganate or ozone) to fully oxidize the dissolved iron and manganese. A conventional sand media filter is then used to remove the precipitated iron and manganese. Option 2 still requires disinfection with chlorine and contact time for virus inactivation and a chlorine residual.

Option 1 uses chlorination for two requirements, Option 2 uses chlorination for only one requirement, but also requires a stronger pre-oxidant like ozone prior to the sand media filter. Option 1 is a more efficient and cost-effective process and is easier to operate than Option 2.

With respect to Option 3 biological treatment, benefits such as longer filter times and less backwashing as well as the need for no chemical oxidants are considered favourable, but biological treatment can still be considered an option with many unknowns that can be influenced by the source water. Limitations with biological treatment are summarized below:

1. High reliance on bacteria formation at start of the process. This may require additional adjustments and trial periods at the start of the project resulting in a duration that provides inefficient treatment. Maturation of bacteria for full efficiency may last up to 50 to 60 days for a new filter; [3]
2. Chance of bacterial die out resulting in treatment stoppages;
3. Process may be influenced by substances such as ammonia, hydrogen sulfide, and zinc; [4]
4. Need for experienced operators that understand the system and requirements to operate biological treatment;
5. Formation of anaerobic conditions in the filter bed resulting in elevated iron concentrations in the filtrate; and
6. Need for specific conditions for iron and manganese oxidising bacteria (i.e., may required two-stage filtration).

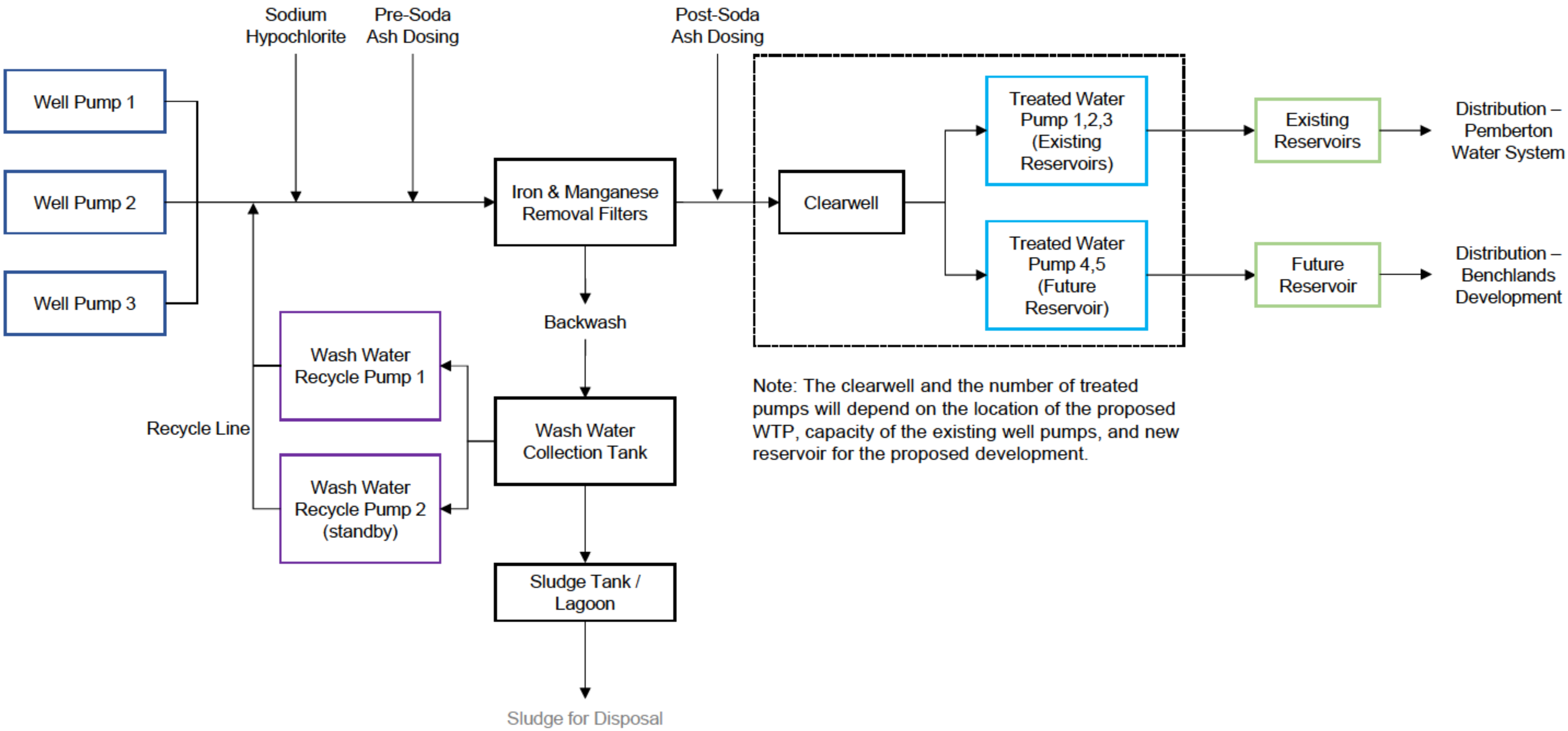
Based on the above, a more conventional approach with oxidation by chlorination and catalytic media filtration is recommended.

The recommended treatment process is portrayed in the block flow diagram shown in Figures 6-1 and 6-2. These figures show similar details, but the main difference is whether the proposed WTP includes access to a sanitary collection system. These figures can be used as a guide or refences as information is described in the report.

[3] Stevenson, D. G. 1997 Water Treatment Unit Processes. World Scientific, Singapore, pp. 261–266, 275–293.

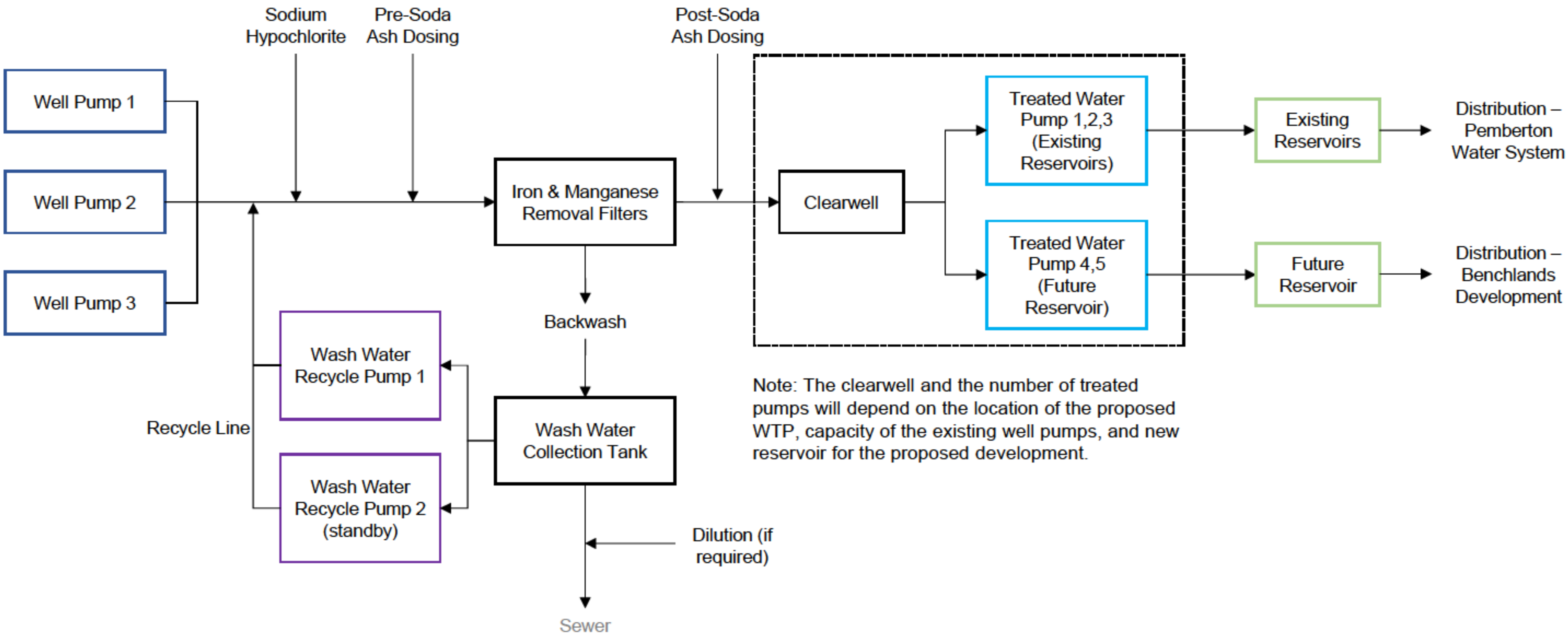
[4] Twort, A. C., Ratnayaka, D. D. & Brandt, M. J. 2000 Water Supply, 5th ed. Arnold, London.

Description of Option 1: WTP Without Access to Sanitary Line



Note: The clearwell and the number of treated pumps will depend on the location of the proposed WTP, capacity of the existing well pumps, and new reservoir for the proposed development.

Description of Option 2: WTP With Access to Sanitary Line



Note: The clearwell and the number of treated pumps will depend on the location of the proposed WTP, capacity of the existing well pumps, and new reservoir for the proposed development.

7. Water Treatment Location and Facility

Based on the Village's expected population growth (with the future development included), the flow of 60 L/s was chosen for the maximum day design flow of the water treatment plant to meet 2040 demands. Table 7-1 summarizes the anticipated flow rates based on village population growth until 2040 as per the *Village Water System Performance Assessment Report*, (2020).

Table 7-1: Anticipated Flow Rate Based on Population Growth

Year	Village Population	ADD (m^3/day)	ADD (L/s)	MDD (m^3/day)	MDD (L/s)
2020	3,100	1,880	22	3,700	43
2025	3,510	2,067	24	4,073	47
2030	3,925	2,255	26	4,451	52
2035	4,335	2,442	28	4,824	56
2040	4,750	2,631	31	5,203	60

7.1 Proposed Water Treatment Building

There are two proposed locations for the water treatment building; the north side of the existing reservoirs (Location 1), and southeast of the reservoirs where the developer proposes a new pump station (Location 2). The current existing treatment location was not considered as the existing infrastructure would not be large enough to include the required equipment. If a new water treatment plant were to be added at this existing location, it would take up a significant portion of the park and would not be acceptable to park users. Figure 7-1 below provides the approximate location of the both the proposed locations.

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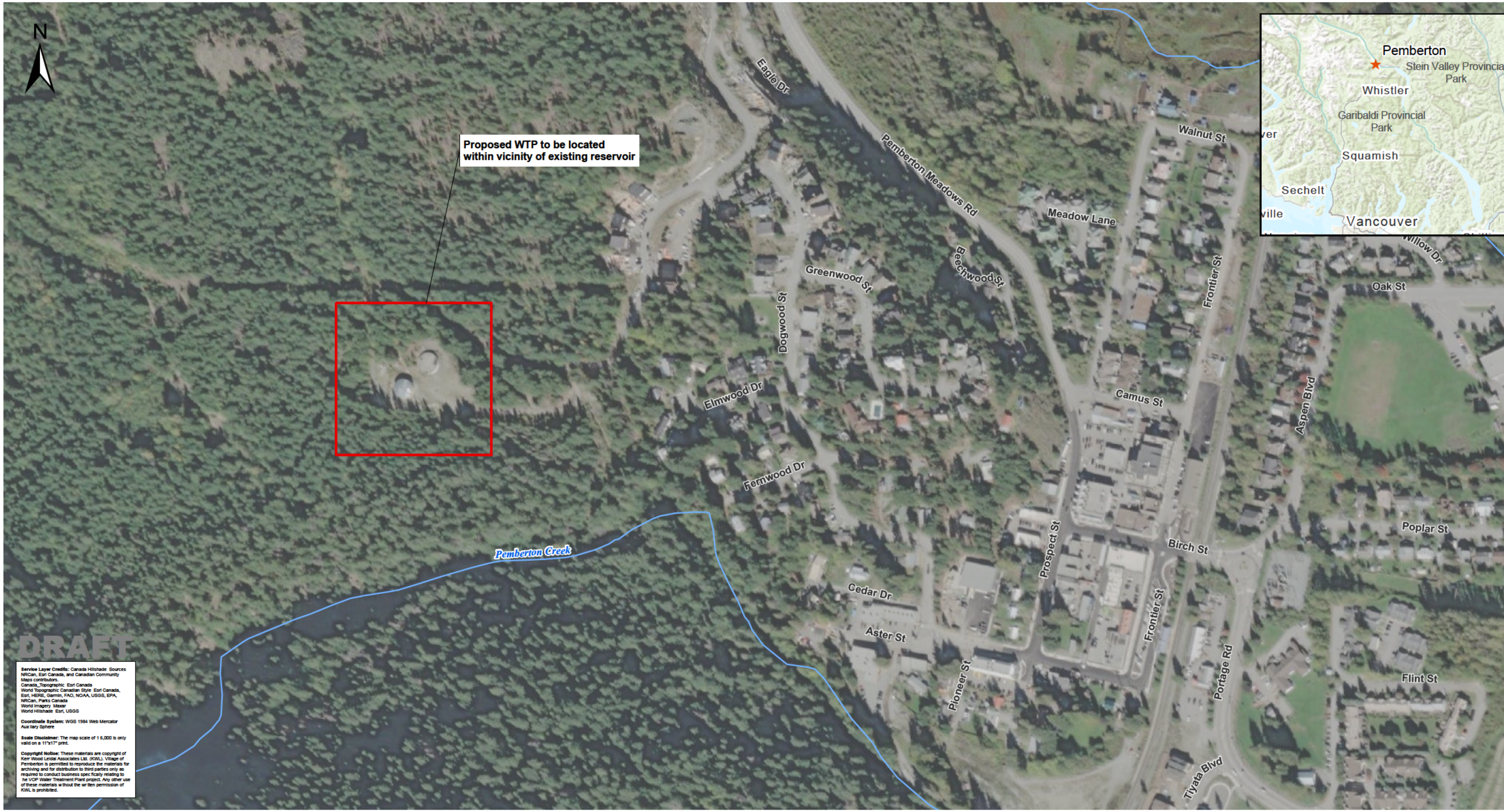

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Date	January 2022
Scale	1:6,000

0 25 50 100 Metres

Site Plan

Figure 7-1

7.2 Location 1: Behind Existing Reservoir

At Location 1, the proposed WTP will be positioned behind, and elevated above, the existing reservoirs. Siting the proposed WTP at this location provides both benefits and drawbacks. The major benefit of having the proposed WTP at Location 1 is the option to have treated water gravity fed to the existing reservoirs. This would eliminate the need for a clearwell and domestic pump(s) to provide treated water to the existing reservoirs. The removal of clearwell and domestic pump(s) would reduce capital and operation costs, as well reduce maintenance associated with pump operation and future replacement.

Drawbacks related to Location 1 include increased capital costs associated with increased sitework related to locating the proposed WTP to the north of the existing reservoirs. Existing well pumps would likely need to be updated or replaced as the well pumps will need to pump to a higher elevation and will need to account for added pressure associated with treatment.

Design and construction considerations to locate the proposed WTP north of the reservoirs will include the following:

1. Environmental and permit applications related but not limited to tree removal, bird surveys, and working within set back of creeks;
2. Increased work associated with archaeological and geotechnical assessments near Location 1;
3. Review of elevation details related to site location and top water level (TWL) of the existing reservoirs. Additional pump(s) may still be required if elevation difference between the proposed WTP and TWL of the reservoirs is not achievable;
4. Increased sitework preparation such as clearing, excavation, backfill, and compaction, as well as increased construction related to access roads and parking spaces to allow for access to the proposed facility;
5. Additional routing of buried utilities such as raw, treated, backwash, and recycle lines will need to be designed and constructed;
6. Upgrades to existing well pumps to increase head pressures to allow well water to reach higher elevation of the WTP and capacity to pump through the proposed WTP; and
7. Will likely require a future pump station to provide water to a proposed future development. Should the proposed development move forward, a clearwell with domestic pump(s) could be constructed at the proposed WTP at Location 1. This would eliminate any benefits associated with gravity fed treated water to the existing reservoirs as described above. This would provide an opportunity for cost sharing with the developer. It is assumed the cost of the clearwell and pump(s) would be the responsibility of the developer should the proposed WTP be located at Location 1 and gravity feed of treated water is achievable.

Based on information provided by Skénkenam Development Limited Partnership (refer to Section 4), the Village will need to discuss with the developer the proposed location of the proposed WTP which may result in the overall reduction of lots or units located near the reservoir. The Village will also need to discuss with the developer regarding future pump station and reservoir requirements, as a clearwell at the WTP could be constructed to perform the duties of a future pump station. This would save additional space near the reservoirs by reducing the need for a separate pump station building and would be more cost effective for both parties.

7.3 Location 2: Front Existing Reservoir

The second proposed location for the WTP is southeast of the existing reservoirs. Location 2 will be at an elevation lower than the TWL of the reservoirs, so a clearwell and domestic pump(s) would be required to feed treated water to existing reservoirs. Based on field reconnaissance of the existing reservoir site, Location 2 will likely require less site modification, reducing the capital cost of the proposed WTP.

Drawbacks of Location 2 include higher costs associated with constructing the clearwell and installation of domestic pump(s). Additional pump(s) also increase operational and maintenance requirements and adds additional complexity should pump issues (faults, failures, power outages, etc.) become frequent in the future.

If the proposed WTP were to be developed at Location 2, there is an opportunity to incorporate the design for future domestic pump(s). This would provide an opportunity to combine both the clearwell and the future pump station building for the proposed development into a single footprint. Cost-sharing opportunities would be made available as discussed above.

Design considerations for Location 2 will be similar those noted above but are noted as follows:

1. Environmental and permit applications related to tree removal, bird surveys, and working within set back of creeks;
2. Complete archaeological and geotechnical assessments;
3. Review of elevation details related to site location and TWL of the existing reservoirs to confirm domestic pump(s) sizing;
4. Upgrades to existing well pumps to increase head pressures to allow well water to be pumped through the proposed WTP; and
5. Design and construction of a clearwell and domestic pump(s) to provide water to the existing reservoir. Provisions can be made to include additional space for future domestic pump(s) for future reservoir.

Similar cost sharing opportunities will need to be discussed with the developer.

7.4 Potential Well Pump Upgrades

To provide treated water to the existing reservoirs, the existing well pumps will require approximately 11 m (15 psi) of pressure to pump water through the filters of the WTP. Should the proposed WTP be located at Location 1, additional pressure will also be required to lift raw water above the existing TWL of the reservoirs. This section is a high-level analysis of the existing well pumps based on pump curve drawings provided by the Village. The analysis assumes the following:

1. The TWL of the existing reservoirs is 290.50 m;
2. The existing dedicated water main has a 300 mm diameter with an approximate length of 1.57 km;
3. The elevation of well pumps is approximately 188.3 m based on Well 3 drawings;
4. Hazen-Williams coefficient of 130 was used to determine the major head loss; and
5. Loss associated with pipe fittings were not included in the calculation.

Table 7-2 summarizes preliminary well pump requirements. System curve calculations will need to be refined during detail design to account for exact dedicated water main lengths and diameters, losses with pipe fittings, and exact well pump elevations.

Table 7-2: Well Pump Requirements for Proposed WTP

Well #	Design for Well Pump [1]	Design Head [1]	Major Head Loss	Assumed Minor Head Loss	Assumed Pressure Loss through WTP [2]	Estimated Pressure Required [3]	Upgrades Required to Meet Flow Demand
2	68 L/s	107 m	5 m	2 m	11-15 m	120-124 m (171-176 psi)	Yes
3	52 L/s	108 m	3 m	2 m	11-15 m	118-122 m (168-173 psi)	Yes

1. Based on pump curves provided by VOP. Well 2: Warson Pump 9WH-1C (stage 2). Well 3: 825 GPM 10" SSI Sub-Pump
2. Additional pressure required to push water through proposed WTP and lift water to Location 1 (north of existing WTP).
3. Estimated well pump requirements based on high level calculations.

Based on the estimates presented in Table 7-2, the well pumps will eventually need to be upgraded to meet design flows and pressures to pump raw water through the proposed WTP. It should be noted that existing well pumps could continue to operate until the pumps are replaced but will operate at a reduced flow to meet increased pressure requirements. Operating the existing well pumps in this manner will be less efficient and will require longer duration to fill the existing reservoirs.

Existing infrastructure such as piping, fittings, and flanges near the well pump will need to be evaluated and rated for pressures above 173 psi prior to initiating design of the new well pumps. If rated pressure for piping connections are unable to maintain high pressure requirements for new well pumps, another option such as a inline booster pump at the front end of the proposed WTP would need to be considered and would allow existing well pumps to remain in use.

7.5 Dedicated Watermain to WTP and Existing Chemical Dosing

An existing dedicated water main provides water from Wells 2 and 3 to the existing reservoir. Sodium hypochlorite and soda ash are currently dosed along Aster Street near Pioneer Park. Based on discussion with operators, there do not appear to be issues with the sodium hypochlorite dosing system. The current soda ash dosing system is located at the Well 1 pump station. Operation staff have noted concerns and higher staff requirements with operating the soda ash dosing system and include increased labour requirements associated with preparing soda ash solution and scaling issues when injecting rates decrease during low demand periods.

The Village has requested KWL review past dosing requirements and testing procedures to determine whether the existing soda dosing requirements should be changed. Findings from this investigation are out of scope for this report but will be summarized in a separate technical memorandum.

The cost estimates presented in Section 8 will include a new soda ash dosing system and a bulk bag feeder for comparative purposes. The bulk bag feeder system should reduce operation requirements related to preparation of soda ash solution. Additional information related to chemical dosing is described in the following section and dosing related to the recycle line.

7.6 Soda Ash Dosing and pH Adjustment

Based on water quality parameters discussed in Section 2, pH levels are adjusted with soda ash from 6.0 to 6.8 pH. It should be noted that pH greater than 6.8 may cause some iron precipitation issues in the proposed media filters (GreenSand Plus^{TM}). If the required target pH is higher than 6.8, a two-stage dosing process will need to be implemented to restrict formation of the precipitation in the filters and to meet corrosion control requirements. These stages would involve the following:

1. Stage 1 (pre-dosing), pH can be increased to 6.8 (via dosing with soda ash) for efficient filtration; and
2. Stage 2 (post dosing), pH can be increased with soda ash (or caustic based on confirmation of enough alkalinity in water after Stage 1 pH adjustment) for corrosion control.

Stage 1 and 2 pH adjustments will need to be further investigated during pre-design based on the technical memorandum to be issued on soda ash dosing, testing, and sampling.

A recycle line (to be discussed in later section) will be piped to the front of the treatment process from the wash water collection tank to reduce the amount of water that would be disposed to the sanitary system. The recycle water will mix with unprocessed groundwater prior to entering the treatment filters. Depending on the recycle water's time spent in the wash water collection tank, chlorine and pH adjustment may need be injected into the recycle line prior to being blended with unprocessed groundwater.

Due to this arrangement, it is recommended that the existing soda ash systems be relocated to the newly constructed WTP. New soda ash dosing systems can replace the existing system once the system is unable to keep up with demands. It is proposed, a new chlorine dosing system should be installed at the proposed WTP to limit the risk involved with relocating the existing chlorination equipment.

Chemical dosing systems will be sized to meet full buildout system so adequate sizing of these systems can be fitted into the proposed WTP.

7.7 Tie-ins

The developer has proposed new routes for the existing watermains to align with the proposed roads, as shown in Figure 7-2. It is assumed two tie-ins for the inlet and outlet piping would need to be installed upstream of the existing reservoirs to service the proposed WTP.

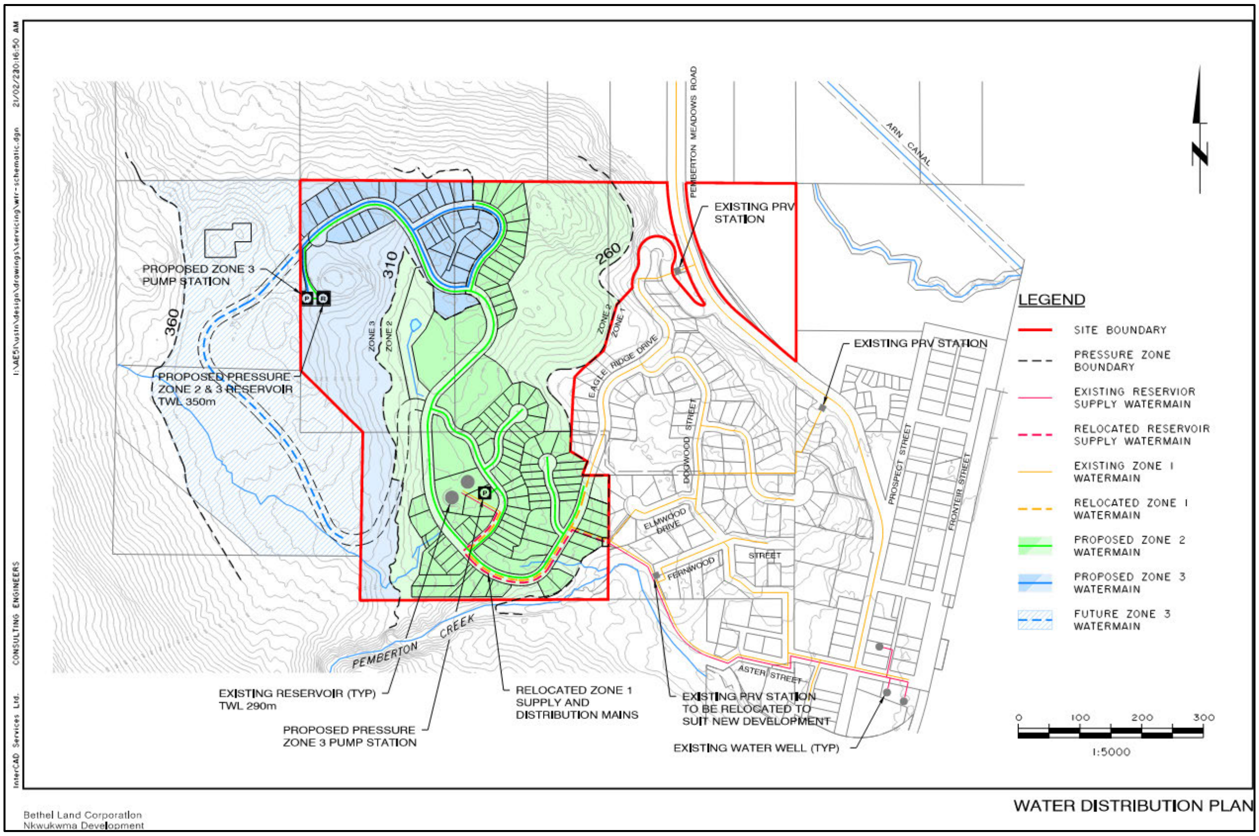


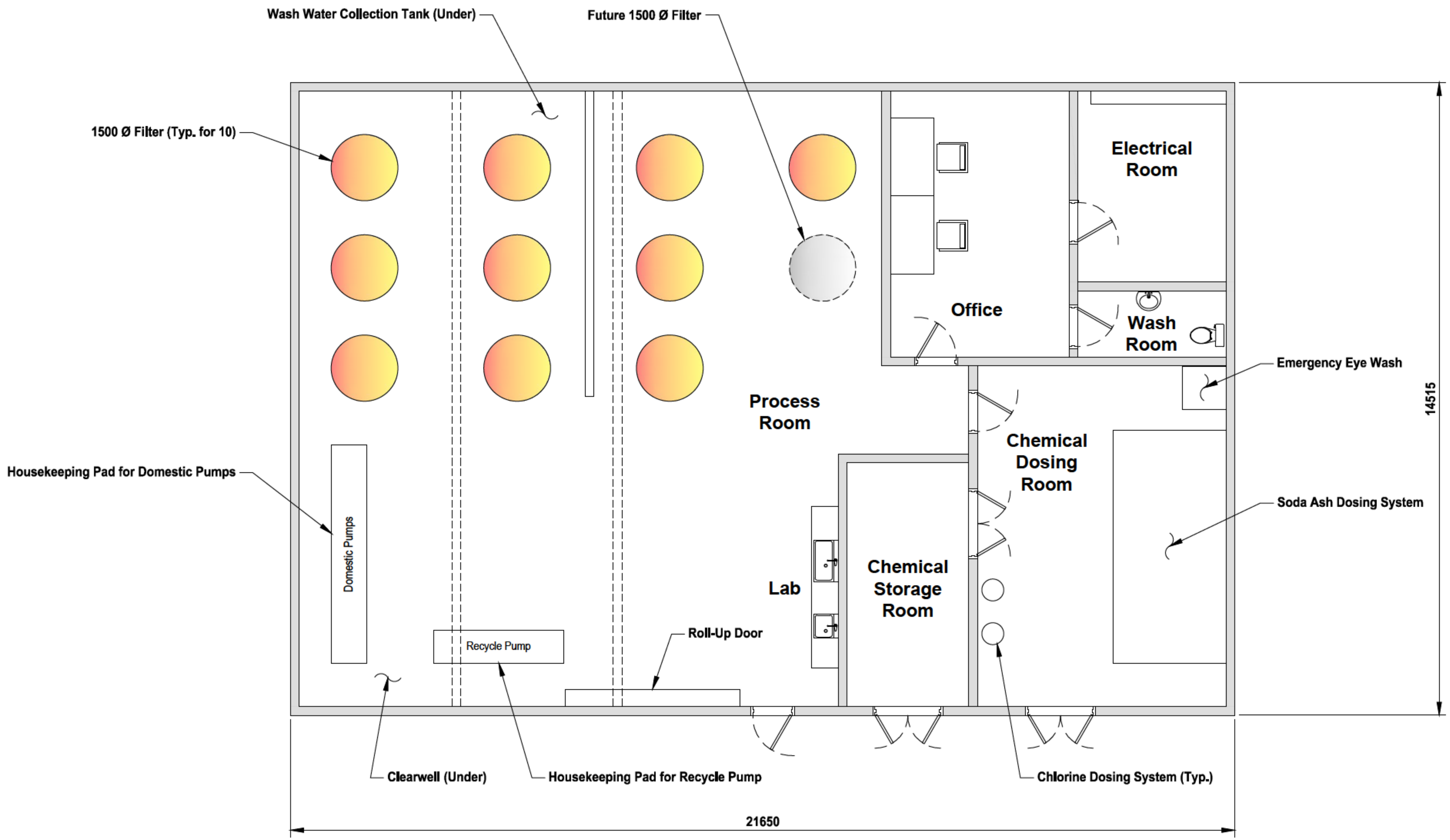
Figure 7-2: Proposed Alignment for Intake Pipes

7.8 Facility Layout

As a post-disaster designed building, it is anticipated that the building structure would consist of a mixture of reinforced concrete, potentially concrete block with wood or steel stud framing for interior walls. Surface finishing in process rooms is likely to be cement board and/or chemical resistant fibreglass wall panels. Concrete provides a more durable aesthetic look and provides an opportunity to customize the building layout to suit site and treatment conditions as well as any operator preferences. Based on the design requirement, room layout (soda ash and chlorine dosing), and future development, it is recommended a customized post-disaster concrete building be constructed to house the treatment equipment. The proposed water treatment plant layout is shown in Figure 7-3. It should be noted, the layout is for discussion purposes only, as items may be omitted based on Village preferences, locations of the proposed WTP, and access to sanitary services.

The proposed water treatment plant has a concrete foundation which includes 1.5 m pony walls for raw water, backwash supply, and treated water pipe anchoring. This also provides for solid anchoring points for the filter vessels. The dimensions of the proposed water treatment plant are approximately 21.7 m x 14.5 m, or 314 m^2. The height of the building will be approximately 5 m in height to allow for pump and filter removal and spacing for chemical dosing equipment. A clearwell and backwash tank (if required) would be placed below the WTP floor as shown below. The following lists the major components of the proposed WTP layout that were considered:

1. Three (two duty, one standby) vertical turbine pumps adequately spaced centre to center;
2. Two (duty and standby) vertical turbine pumps adequately spaced. Pumps to be installed when proposed Pressure Zone 2 and 3 reservoir is constructed;
3. Ten 1,500 mm (60”) diameter filters spaced approximately 3.5 m apart center to center. Filters are oriented so that operators have easy access to control valves and other components for easy operation and maintenance;
4. Spacing for one future 1,500 mm (60”) filters to act as a spare treatment filter;
5. Separate electrical room comprising of VFDs, control panels (MCC), and other electrical equipment. A clearance allowance of 1 m to meet code requirements and additional spacing for operations was provided;
6. One 3 m roll up door located in the process room of the proposed WTP to allow for removal of pumps and filters for maintenance, repairs, or equipment replacements;
7. Several access doors located throughout the building to allow for operator ingress and emergency egress;
8. Laboratory area to allow for water collection and sampling work;
9. Chemical dosing and storage rooms to house sodium hypochlorite and soda ash;
10. Standard washroom with water closet, lavatory sink, and faucets; and
11. A 50 m^3 clearwell and a backwash water collection tank located below WTP.



Proposed Water Treatment Plant Layout

Figure 7-3

7.9 Infrastructure Requirements

Electrical

The proposed WTP will require 3-phase 600 VAC power with a minimum of 200 A service but will likely require less amperage if domestic pump(s) are not required.

Based on the site visit, 3-phase power is available off Eagle Ridge Drive, near the road that enters the reservoirs. The village has requested that some electrical equipment stationed near the reservoirs be moved into the water treatment plant. Building service electrical requirements and any additional services need to be evaluated in a subsequent detail design phase.

The Village should discuss with the developer whether 3-phase power will be extended from Eagle Ridge Drive up to the proposed Zone 3 Pump Station. Should extension of 3-phase power proceed up to the proposed pump station, the Village should negotiate responsibility and conditions as part of the development.

Sanitary Systems and Filter Backwash Collection

Use of filters to remove iron and manganese will require periodic backwash to remove accumulated solids in filters. When the filters are backwashed, the generated backwater will head to the wash water collection tank. To conserve water and reduce volumes of backwash water, water from the top of the wash water collection tank will be recycled to the front of the proposed WTP for treatment.

The recycle pump will be programed to pump the recycled water on a pre-determined intervals (after allowing approximately two hours after backwash for solids to settle down in the wash water collection tank). Based on similar facilities, typical backwash volumes are approximately 5 to 7% of the total water treated by the facility. It is safe to assume more than half of the backwash volume can be recycled to the front of WTP.

Depending on the access and sewer capacities, the settled sludge will be pumped to a sludge tank. The sludge tank will need to be cleaned out and haul away on a regular basis. If access to sewer is available, settling and recycling can still be used to conserve water as previously mentioned.

Currently there are no existing sanitary mains near the existing reservoir locations that would be able to accept any backwash wastewater from the proposed WTP; therefore, wastewater generated from backwashing of media filters and other maintenance procedures will need to be captured and collected in tanks for disposal.

Should a sanitary line be installed for the future development, portion of the backwash from the proposed WTP can be disposed via the sanitary line. Access to a sanitary system would eliminate the need for the backwash settling, and recycle, but a solids collection tank would still be used to reduce the solids loading to the wastewater collection system. This option should be further evaluated in preliminary design phase and should be evaluated against proposed development requirements and wastewater treatment plant capacities.

Based on anticipated filter backwash volumes, a wash water collection tank will be approximately 50 m^3 in size which is equivalent to four backwash volumes plus room for freeboard. A 30 m^3 tank would be used for sludge collection.

Domestic Booster Pumps Clearwell

Domestic booster pumps may also need to be installed to provide the required pressure to pump water to the reservoir TWL height. Technical requirements along with Village preferences and location will need to be reviewed during the design phase.

At Location 1 (north side of the existing reservoirs), the treatment plant would not require a clearwell or pumps as the water would be gravity fed to the existing reservoirs. At Location 2 (southeast of the reservoirs), a clearwell approximately 12.8 x 3.5 x 1.5 m (50 m^3) would be required and would be located below the floor of the proposed WTP. Should the Village include provision to have future domestic pumps installed in the clearwell to provide treated water to a future reservoir, spacing (i.e., concrete pad) could be included into the design and future pumps can be installed when the development is being built. The Village will need to discuss cost share details with the developer.

Based on the above, if a clearwell were to be installed at the proposed WTP, the following pump configurations or a combination of both could exist:

1. Three pumps (two duty, one standby) to pump water to the existing reservoirs; and
2. Two pumps (one duty, one standby) to pump water to the future pressure zone 2/3 reservoir.

Since it is considered a benefit for both the Village and developer to have a clearwell with provisions for future pumps, all WTP option discussed below will include a clearwell. Clearwell size will differ based on gravity fed and pumping requirements.

Allowances for well pump replacements and domestic pump installation have been included in the Class D cost estimate.

7.10 Summary of Proposed Water Treatment Plant Options

Based on the details discussed above, there are several options or configurations that the proposed WTP can be constructed. These options would depend on the Villages preferences related to the location of the proposed WTP, access to sanitary system, and the configuration of chemical dosing systems.

Table 7-3 summarizes major details for each of the proposed WTP options. A breakdown of costs is provided in Section 0 of the report.

Table 7-3: Proposed WTP Options

Parameters	Option 1	Option 2	Option 3	Option 4
Location	Location 1	Location 1	Location 2	Location 2
# Filter Tanks (dia. 60")	9	9	9	9
Gravity Fed WTP to Existing Reservoir	Yes	Yes	No	No
Clearwell Tank Volume	~ 20 m^3	~ 20 m^3	50 m^3	50 m^3
Domestic Pump(s) to Existing Reservoirs	None	None	3 (2 duty, 1 standby)	3 (2 duty, 1 standby)
Domestic Pumps(s) to Future Reservoir	2 (1 duty, 1 standby)	2 (1 duty, 1 standby)	2 (1 duty, 1 standby)	2 (1 duty, 1 standby)
Wash Water Collection Tank	50 m^3	50 m^3	50 m^3	50 m^3
Access to Sanitary Line	Yes	No	Yes	No
Sludge Holding Tank	None	30 m^3	None	30 m^3
Capital Cost ($)	$8,159,000	$8,024,000	$8,013,00	$7,877,000
O&M Cost ($/year)	$248,000	$285,000	$248,000	$285,000

7.11 Alternative Source and Post Expansion

The Village has conducted desktop assessment of alternative sources to supplement water supply should the existing groundwater source be unable to meet demands of the community while meeting the sustainable recharge rate of 30 L/s for the Pemberton Creek Fan Aquifer. Such alternative sources include both surface sources and other groundwater sources not tied to the Pemberton Creek Fan Aquifer.

As discussed in Section 7 and Table 7-1, the capacity of the proposed WTP is 60 L/s and is based on 2040 population growth with the intention to meet the sustainable recharge rate of the Pemberton Creek Fan Aquifer. Should an alternative source be added to supplement the water supply the Village has the option to expand the overall capacity of the WTP by adding addition space/structure for more filters or treatment equipment.

Based on discussion with the Village, a likely additional water source would be drilling new well(s) into the Lillooet River Aquifer. The Village plans to complete sampling of this potential well source, but past water quality samples have suggested the Lillooet River Aquifer may have higher iron and manganese and may be susceptible to the influence of surface water. Should this alternative source be added in the future, the proposed WTP can be expanded to include additional treatment process such as UV disinfection to meet treatment requirements. Where a UV disinfection system could be installed after the filters or downstream of the clearwell prior to entering the existing reservoirs. The addition of a UV disinfection system can be investigated during detail design.

8. Class D Cost Estimate

This section summarizes the cost opinions for various options discussed above. Options will be split based on the locations of the proposed WTP and access to the sanitary line for backwashing of the filters.

8.1 Limitations

The projected capital costs presented in this report are based on Class D Capital Cost Opinions. These costs opinions are order-of-magnitude level costs prepared with brief site information and should be used for planning purposes only. The costs may be subject to change upon receipt of significant new site or other information. A 60% allowance (40% contingency and 20% engineering) has been applied to the cost options to reflect their high-level nature.

8.2 Assumptions

The selection, sizing, and projected costs of the proposed WTP are based on the following:

1. All options will meet design flow target of 60 L/s by 2040 and will have provisions to include additional filters and pumps by 2040;
2. All options include a clearwell and pump(s) to provide treated water to the existing reservoir or future Pressure Zone 2 and 3 reservoirs;
3. A revised soda ash system to be installed for all options;
4. Electrical supply is available at existing site;
5. Includes contractor overhead and profit mark up (30%) and PST (7%); and
6. Cost escalation uncertainty with supply chain issues, pandemic fallout or recent provincial flooding could result in a cost escalation of 20 to 30%. This cost escalation has not been included in the cost opinion presented below.

8.3 Summary of Capital Cost Opinions

A summary of the proposed WTP located at Location 1 (north of the existing reservoirs) cost opinions are provided in Table 8-1.

Table 8-1: Summary of Cost Opinions for Proposed WTP located North of Existing Reservoirs

Item	Option 1, No Access to Sanitary Water Line (North of Reservoirs)	Option 2, Available Access to Sanitary Water Line (North of Reservoirs)
General Requirements	347,600	342,100
Site Work	529,000	485,900
Concrete	638,200	602,400
Building	492,500	492,500
Equipment	2,316,900	2,316,900
Mechanical & Piping	314,000	314,000
Electrical	461,000	461,000
Sub-Total	5,099,200	5,014,800
Engineering (20%)	1,020,000	1,003,000
Contingency (40%)	2,040,000	2,006,000
Total	**8,159,000**	**8,024,000**

A summary of the proposed WTP located at Location 2 (south of the existing reservoirs) cost opinions are provided in Table 8-2.

Table 8-2: Summary of Cost Opinions for Proposed WTP located South of Existing Reservoirs

Item	Option 3, No Access to Sanitary Water Line (South of Reservoirs)	Option 4, Available Access to Sanitary Line (South of Reservoirs)
General Requirements	341,600	336,100
Site Work	320,300	277,200
Concrete	638,200	602,400
Building	492,500	492,500
Equipment	2,439,900	2,439,900
Mechanical & Piping	314,000	314,000
Electrical	461,000	461,000
Sub-Total	5,007,500	4,923,100
Engineering (20%)	1,002,000	985,000
Contingency (40%)	2,003,000	1,969,000
Total (Rounded)	**8,013,000**	**7,877,000**

8.4 O&M Cost Estimate

The Operation and Maintenance, O&M costs are allowances based on similar WTP projects completed by KWL and are intended to be for comparison purposes between the various treatment processes evaluated. It is anticipated that the estimate for O&M costs would be refined with subsequent phases of work such as pilot testing, preliminary design, final design.

At this stage, the proposed WTP O&M Cost estimate is split among six categories which include electrical operating charges, staffing, water monitoring, consumables, waste management, and facility maintenance. The following summarizes how each category were calculated:

1. Electrical charges are based on typical electrical requirements of major equipment (i.e., pumping) and anticipated duty cycle. Electrical cost rates at $0.10/kwh reflect industrial averages;
2. Staffing charges are based on typical hours required to maintain and operate the WTP. Assumes an hourly charge rate of $40/hour. Staffing generally relates to routine labour, filter replacement, sludge disposal, and after-hour response;
3. Water monitoring is assumed to be completed at the proposed WTP to measure overall performance of the treatment process and to confirm the distribution is receiving treated water that meets guideline requirements. It is anticipated samples will be collected quarterly at the proposed WTP;
4. Consumables are based on the recommended treatment process which include filter media replacement and removal as well as chemical consumptions;
5. Waste management is the cost associated with disposal of accumulated sludge from the backwashing of media filters. A disposal fee of $1,500/haul was applied at a rate of two hauls per month for the proposed WTP. The cost associated with hauling could be eliminated if a sanitary line is accessible; and
6. Maintenance are costs associated with the maintenance and replacement of equipment at the WTPS.

Based on the assumptions above, the estimated O&M costs for the proposed WTP ranges between $248,000 and $285,000 per year. Lower range cost would be related to WTP options that have access to sanitary system and would not require the need for sludge disposal services. It should also be noted, approximately $110,000 of the O&M costs are related to replacement and maintenance costs which would most likely be put aside to allow for equipment to age and be replaced.

9. Overall Discussion and Summary

Four proposed WTP options were presented in the information above to treat existing groundwater wells to meet requirements of the GCDWQ. The proposed WTP should be designed to operate at 60 L/s and will have provisions to supply any future adjacent development with the addition of domestic pumps dedicated for that development. The proposed WTP will consist of ten 1,500 mm (60") diameter filters with GreenSandPlus™ media to remove iron and manganese and will be fitted with domestic pumps, recycle pumps, and sludge pumps.

The treatment process at the proposed WTP will consist of oxidation by chlorination and filtration by GreenSandPlus™ media. Oxidations by chlorine disinfection will promote precipitate formation of iron and manganese and provide primary and secondary disinfection of the water. Catalytic media filtration with GreenSandPlus™ will further react with dissolved manganese to promote absorption to the filter media.

Periodic backwash of the GreenSandPlus™ filter media will be required to remove the accumulated iron and manganese in the filter. The backwash water will be sent to wash water collection tank where settling of solids will occur. To reduce the amount of liquid waste for disposal, a recycle pump will pump the supernatant liquid to the front of the WTP for treatment. Solids in the backwash collection tank will settle to the bottom, where a sludge pump will transfer the solids to a sludge collection tank or lagoon for storage and disposal. It is anticipated, disposal of solid waste will occur approximately one to two times per month at the proposed WTP but will depend on the water quality of the raw water and actual volume of water being treated.

The recycle line will be piped to the front of the proposed WTP from the wash water collection tank, where chemicals will need to be injected upstream or downstream of the filters. Due to the proposed arrangement, it is recommended, the existing soda ash systems be moved to the constructed WTP and a new chlorine system be installed at the proposed WTP. A new pH adjustment system can replace the existing systems once the system is unable to keep up with future demands.

The capital cost of the proposed WTP options range between $7.9 to $8.2 million depending on the location, pumping requirements, and access to sanitary systems for sludge disposals. O&M costs estimated to be $248,000 to $285,000 per year.

10. Conclusion and Recommendations

10.1 Conclusions

Based on the scope of this water treatment investigation, several conclusions have been reached and are listed below:

1. The existing Wells 2 and 3 groundwater sources, will eventually have elevated iron and manganese water levels that do not meet the requirements of the Guideline for Canadian Drinking Water Quality;
2. Anticipated flow rates based on village population growth until 2040 are summarized in the table below:

Table 10-1: Summary of Flow Rates

Year	Village Population	ADD (m^3/day)	ADD (L/s)	MDD (m^3/day)	MDD (L/s)
2020	3,100	1,880	22	3,700	43
2025	3,510	2,067	24	4,073	47
2030	3,925	2,255	26	4,451	52
2035	4,335	2,442	28	4,824	56
2040	4,750	2,631	31	5,203	60

3. The proposed water treatment process of oxidation with chlorine injection and catalytic media filtration (GreenSandPlus™) will provide adequate treatment and disinfection to the water from the wells;
4. The proposed WTP should be designed to operate at 60 L/s and will have provisions to supply any future adjacent development with the addition of domestic pumps dedicated for that development;
5. Two proposed locations were identified. Location 1 would have the proposed WTP located north of the existing reservoirs at an elevation above the reservoirs TWL to allow for gravity feed. Location 2 will be located southeast of the existing reservoirs and will require a clearwell and additional pumps to provide treated water to the reservoir. Both locations have advantages and drawbacks;
6. A separate technical memorandum related to soda ash dosing is currently being prepared by KWL and will provide additional insight into the design of future soda ash dosing system for corrosion control of the treated water;
7. To conserve water use and produce reduce volumes of wash waste for disposal at the proposed WTP, a pump from the wash water collection tank will recycle settled water to the front of the WTP for treatment. Hypochlorite injection and pH adjustment systems will need to be installed downstream of the tie-in point of the raw water and recycle line;
8. Four WTP configurations were presented based on the location of the proposed WTP and access to sanitary system. Costs of the WTP range from $7.9 to $8.2 Million and include 40% contingency;
9. O&M costs to operate the WTP is estimated to be $248,000 to $285,000 per year. Costs will be impacted by access to sanitary system and sludge disposal; and

10.2 Recommendations

Based on the conclusions of this study, a list of recommendations is provided below:

1. The Village to review the proposed WTP options and determine which configuration best suits their needs and requirements;
2. Conduct bench scale testing with water from Well #2 and #3 to confirm Oxidation and Catalytic Media Filtration with GreenSandPlus™ is able to meet treatment requirements;
3. Proceed with pre-liminary design of the preferred WTP option;
4. The Village, supported by KWL, apply for Investing in Canada Infrastructure Program Green Infrastructure Grant by February 23, 2022;
5. The Village to confirm existing wastewater treatment plant capacity and determine whether additional volumes as a result of the proposed WTP and future development would impact the treatment facility, or the conveyance of sewage to that facility; and
6. Complete additional water quality samples of potential additional water sources to determine if additional treatment process may need to be included at the proposed WTP.

11. Report Submission

KERR WOOD LEIDAL ASSOCIATES LTD.

Prepared by:

[redacted]

Alfred Louie, P.Eng.
Project Engineer/Project Manager

Kimberley Brown, EIT
Junior Engineer

Reviewed by:

[redacted]

Venkat Raman, P.Eng.
Technical Reviewer

Reviewed by:

[redacted]

Irfan Gehlen, P.Eng.
Principal in Charge

AL/sk

Statement of Limitations

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Revision History

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A	November 26, 2021	Draft		AL/KSB
B	December 10, 2021	Draft		AL/KSB
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KERR WOOD LEIDAL
consulting engineers

Vancouver Island
201 - 3045 Douglas Street
Victoria, BC V8T 4N2
T 250 595 4223
F 250 595 4224

Technical Memorandum - Draft

DATE: January 31, 2022

TO: Tom Csima, Manager, Operation and Projects
Village of Pemberton

FROM: Brandon Johnson, P.Eng.

RE: **Village of Pemberton**
Water Conservation Plan
Our File: 0743.018-300

Background

Kerr Wood Leidal Associates Ltd. (KWL) has been retained by the Village of Pemberton (the Village) to provide a Water Conservation Plan (the Plan). The Village has a higher per capita water use than the Canadian average, indicating potential for reducing consumption through water conservation and leak detection efforts.

The Village's water source derives from two active wells that withdraw from the Pemberton Creek Fan Aquifer which supply the Village population of approximately 3,100 as well residents of the Squamish-Lillooet Regional District who live in the Pemberton North Improvement District. Demands from the Pemberton North Water System (PNWS) comprise approximately 17% of the total demand.

The goal of the Plan is to identify both where conservation efforts should be made, and tools and work needed to reduce water use and leakage to achieve an overall reduction in per capita water use of 15% in the next 10 years.

Incentives to conserve water are both economic and environmental. Economically, the Village is significantly invested in its current source. Alleviating capacity constraints will defer infrastructure replacement costs, reduce operational costs, reduce water treatment costs, and maximize the time that the Pemberton Creek Fan Aquifer can be utilized before needing a new source. Environmentally, reducing the extraction of water from the aquifer will subsequently also reduce the volume of wastewater released, minimizing the impact to the environment.

Per-Capita Water Demands and Component Analysis

Average per capita water use is approximately 600 L/capita/day which included residential, industrial, and commercial use but excludes water demand from the Pemberton North Water System. In 2017, Canadian average per capita water use is 427 L/capita/day which also includes industrial, commercial, and other uses.

The village's current maximum day per capita demands are estimated at 1,190 L/cap/day which is high.

Per-capita water use is often used as a metric for assessing residential use and conservation efforts, however, it is affected by the type and quantity of industrial and commercial (ICI) use. High ICI water use combined with a relatively low population can inflate the average per capita water use metric. Equally, higher than average system leakage can have the same effect.

Water conservation initiatives should put focus and effort where improvements can be made based on analysis of the components of water use. It is therefore necessary to understand where the greatest reductions are possible by measuring other metrics such as leakage levels, the leakage infrastructure index, residential base (indoor use), and seasonal water use.

The recommended process for determining the components of water use, including leakage, are as follows:

1. ICI Use: Quantified by customer water meter billing database and estimates for unmetered commercial and industrial use. It is noted that all businesses located in the Village's industrial park have water meters installed and it is estimated that metered ICI customers account for roughly half of the total ICI use[1] in the Village. Average, base, and seasonal usage for industrial and commercial users may be estimated if all meters are read for billing at set times marking the normal transition from winter to summer usage.
2. Water System Leakage: Quantified by zone night flow analysis. It is noted that the Village is working towards providing SCADA monitoring for zone metering to allow leakage assessments to be completed.
3. Base Demand and Base Residential water use: Review of average winter demand data. Base demand is average winter demand. Base residential water use is calculated by deducting the estimated industrial and commercial use and estimated system leakage from average winter demand.
4. Seasonal water use: Review of yearly flow data. The yearly quantity of seasonal demand and period in which it occurs can be quantified by review of daily flow records.

Components of Water Demand

Water billing data from 2010 through to 2020 was reviewed along with source flow data from 2020 and 2021. In 2015, a major leak was identified and repaired. The leak was responsible for a daily loss upwards of 500 m^3 or 5.79 L/s, which accounted for roughly 20% of the water demand at the time.

A water balance was completed using billing data from 2016 to 2021 to categorize water use and applying the breakdown to 2020 source flow data. Water demands have decreased by approximately 10% since 2016. Seasonal water use in 2021 was significantly higher than 2020 (+32%); however, 2021 was an uncharacteristically hot year, leading to many municipalities observing record water usage and therefore was not considered in this analysis.

The following assumptions were made to complete the water balance:

1. Indoor residential water use is estimated to be 230 L/cap/day.
2. Unmetered ICI demands are assumed as approximately one-third of the total ICI demand. Approximately half of the total ICI customers are metered, and it is assumed that these include the larger water users.
3. Total base usage is calculated from the average day winter demand multiplied by 365 days.

[1] Village of Pemberton Water System Performance Assessment

4. Seasonal usage is all water usage above the average winter base demand that occurs from May through September.

The components of water demand are presented in Table 1 below.

Table 1: Estimated Annual Water Use Breakdown

Period	Usage by Demand Type (m^3)					Total	% of Annual Total
	Residential	ICI	Metered Outside Boundary	PNWS	Water Losses		
Base Usage	220,000	74,000	7,000	32,000	70,000	403,000	66%
Seasonal Usage (% of Total for Demand Type)	143,000 (39%)	24,000 (24%)	4,000 (36%)	36,000 (53%)	NA	207,000	34%
Annual Total	363,000	98,000	11,000	68,000	70,000	610,000	100%
% of Annual Total	60%	16%	2%	11%	11%		

The following is noted with regards to the estimated annual water use breakdown:

1. Seasonal water use is high and accounts for 33% of total yearly demand and is estimated to account for 39% of the total non-metered residential demand. By comparison, in the lower mainland, seasonal demand accounts for 33% of total residential demand. The difference in seasonal use between the Village and the Lower Mainland is greater than these numbers signal since the assumed residential base use for the Village (230 L/cap/day) is approximately 30% greater than Lower Mainland base residential usage. On average each residential account uses 142 m^3 of seasonal (outdoor) water between May 1 and September 30 or 934 L/property/summer-day.
2. Water losses are moderate, estimate at 70,000 m^3/year which is roughly 11% of total annual water use or a leakage rate of 2.2 L/s. It is noted that actual loss levels may be higher than reported as loss levels are calculated based on a relatively conservative estimates of legitimate residential base demand. The accuracy of the audit would be improved by determining water loss through minimum night flow analysis once zone metering is completed and connected to SCADA.
3. The water loss total includes leakage within PNWS. PNWS water losses are high. The total annual water supplied to PNWS is approximately 120,000 m^3; comprised of an estimated 68,000 m^3 of legitimate usage and 52,000 m^3 of water loss. PNWS water losses are estimated to be 74% of the Village's total water loss of 70,000 m^3.

Population and Growth

The village's current population serviced by the Village's water system is estimated at approximately 3,100. The following is noted with regards to serviced population, current development plans, and future growth.

1. On average between 1991 and 2016, the Village has grown at a rate of 80 people per year.
2. The Village population is estimated to increase by 686 people according to the several developments that are either under construction or have recently been completed.

3. Over the next 5 to 10 years, there are significant residential plans approved, housing approximately another 1,763 people.
4. Extrapolating the population best fit line into the year 2040, the population for the Village is estimated for the years 2020 to 2040[2] as shown in Table 2 below.

Table 2: Projected Water Service Population

Year	Population
1991 Water Study	550
2001 Census	1,637
2006 Census	2,192
2011 Census	2,369
2016 Census	2,574
2020 Estimated	3,100
2025 Estimate	3,510
2030 Estimated	3,925
2035 Estimate	4,335
2040 Estimated	4,750

Water Supply Capacity

The Pemberton Creek Fan Aquifer is unconfined and primarily recharged via Pemberton Creek at a rate of approximately 30 L/s. Production wells 1, 2, and 3 are located in the central portion of the aquifer.

1. Well 1 is inactive due to excessive iron and manganese concentrations.
2. Well 2 is the backup well constructed in 1997 at a depth of 41.8 m. It has a diameter of 300 mm and a rated flow of 76 L/s.
3. Well 3 is the current duty well constructed in 2007 at a depth of 46 m. It has a diameter of 200 mm and a rated flow of 50 L/s.

During the summer months water is consumed quicker than the aquifer's recharge rate. The aquifer water levels typically recover each winter as the Village's water usage drops. The sustainable use of the Pemberton Creek Fan Aquifer requires the Average Daily Demand (ADD) to remain below 30 L/s (2,600 m^3/day). Currently, ADD is approximately 21 L/s.

Climate Change Adaptation and Mitigation

The following is noted with regards to climate change adaptation and mitigation:

1. In general, weather is likely to become wetter in the winter and drier in the summer in the future in the Squamish-Lillooet region. According to the Pacific Climate Impacts Consortium (PCIC; plan2adapt.ca), by the 2050s, precipitation in the region is expected to change from current normal as follows (median of forecasts, and range of 10th to 90th percentiles):

[2] Village of Pemberton Water System Performance Assessment

   a. Annual +2.4% (-1.7% to +7%)
   b. Summer -5.9% (-30% to 5.3%)
   c. Winter +2.9% (-1.9% to +8%)
2. As well, the Pacific Climate Impacts Consortium (PCIC; plan2adapt.ca), estimates that by the 2050s annual average temperatures in the region will increase by +3.1˚C (+2.1˚C to +4.2˚C).
3. Extreme weather events (temperature and precipitation, drought, and flooding) are expected to increase in frequency. The impact on water service may include increased storage requirements for balancing peak flows.
4. The physical capacity of the Pemberton Creek Aquifer is considered a constraint into the future with climate change worsening the effects of a growing population on demand.
5. Benefits of water conservation (mitigation and adaptation):
   a. Reducing the extraction of water from the aquifer will reduce the volume of wastewater released, minimizing the impact to the environment.
   b. Reducing costs and carbon emissions of expanding the infrastructure to accommodate growth (e.g., manufacturing, transporting, and installing larger watermains).
   c. Reducing carbon emissions associated with trucking water to overcome capacity constraints.
   d. Maintaining more water storage in reserve for emergencies such as wildfires or extreme drought, which may increase due to climate change.

Water Demand Targets

The following water conservation targets are recommended:

1. Overall water supply flow (annual total or average) at WTP: Maintain below 25 L/s through year 2040.
2. Maximum day demand at WTP: Maintain below 50 L/s through year 2040.
3. End user demand (L/cap/day): Reduce to 900 L/cap/day Maximum Daily Demand (MDD) and 450 L/cap/day ADD at WTP by year 2040.

Achieving a per-capita reduction in water demands of approximately 25% over the next 20 years will rely on a combination of educational and regulatory measures to reduce water demands in existing buildings, water-efficient new construction, and implementation of a water distribution loss management program. If these measures are implemented, the targets are achievable with a water service area population of 4,750 in year 2040.

Current and Planned Water Conservation Measures

A planned adaptive strategy enables conservation measures to be tailored to meet the changing needs of the community over time. The following conservation measures are currently undertaken or are planned for implementation as required.

1. **Regulation** (current): In 2015 the Village established an Outdoor Water Use Regulation Bylaw (Bylaw No. 792), which includes four water conservation levels as shown on Figure 1 below.

Figure 1: Village of Pemberton Outdoor Water Use Levels

The Village advertises the current level of watering restrictions in social media and on its website. The Village also provides details about best practices for reducing indoor water use on its website to educate the public.

2. **Retail metering program** (feasibility study completed 2007, prioritized implementation in progress: The Village's zoning and building code bylaws require all new developments to install water meters and low-flow toilets and urinals (residential and ICI). Approximately half of all ICI connections are currently metered.
3. **Consumption based billing** (current): Metered customers are billed for water based on water consumption and a two-tiered inclining block rate structure where consumption over 65 m^3 per quarter is penalized with a higher water rate (2.25 times). The Village has developed appropriate non-metered and metered rate structures that achieve stable revenues and appropriate incentives to reduce base and peak demands.
4. **Water Loss Management** (current and planned): The Village has implemented zone metering and is in the process of connecting zone meters to SCADA for the purpose of monitoring nighttime flows and leakage levels. Distribution losses are estimated to average 2.2 L/s, which is approximately 11% of annual demand. Ongoing recommended measures include minimum overnight flow monitoring, keeping records of leaks found and repaired, and sounding for leaks at line valves and curb stops when they are exercised or located. Losses are the greatest in the Pemberton North Water System, comprising approximately three quarters the total loss value.
5. **Demand Management Program** (Current): Providing information to customers through print and electronic media has been a major component of the Village's conservation program since its inception. Print media has included bill stuffers, flyers and brochures that address indoor and outdoor water conservation practices. This information has also been posted on the Village's website and published in its bi-monthly e-newsletter. The Village also ensures responsible 'water wise' irrigation for all civic properties in accordance with their bylaw. The Village will continue to implement a program to reduce peak and annual water use as needed to defer capacity upgrades and meet the planned conservation targets, including a community awareness campaign aimed at water efficient lawn and landscape maintenance.
6. **Reporting usage and water budgets on water bills** (current and future): Displaying information about water use on water bills is completed to raise customer awareness about their water use. Comparing each customer's water use to a system average, or to a water use budget based on

system constraints will enable customers to make informed and timely decisions about how they use water.

7. **Water Conservation Plan Renewal** (planned for 2026, and every five years thereafter): A review of this plan will be conducted every five years to update forecasts and targets, consider new information, and adjust program activities as required to meet targets.

Program Implementation Responsibility, Cost and Schedule

The Director of Public Works will have overall responsibility for the water conservation program. Aspects of the program may be delivered by public works (e.g., water-loss management), finance (rates), and Community Development and Planning (bylaw administration, forecasting and public engagement). The program is budgeted under the water fund. Planned measures will proceed within the next five years (subject to budget approvals), or as necessary to achieve targets and avoid premature infrastructure capacity upgrades where it is cost-effective.

Linkages to Other Plans and Policies

This Plan supports the Official Community Plan; outdoor water use bylaw; Water Rates Bylaw; PNWS water rates study; Water and Sewer Asset Management Plan; and Corporate Asset Management Policy.

KERR WOOD LEIDAL ASSOCIATES LTD.

Prepared by:

[redacted]

PROFESSIONAL ENGINEER
PROVINCE OF BRITISH COLUMBIA
B. L. JOHNSON
54481

2022-01-31

Brandon Johnson, P.Eng.
Project Engineer

Permit to Practice #1000696

Reviewed by:

[redacted]

Ryan Lesyshen, M.Sc., P.Eng.
Technical Reviewer

BLJ/aah

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Revision History

Revision #	Date	Status	Revision Description	Author
A	January 10, 2022	DRAFT	Initial Draft	BLJ/RYL
B	January 17, 2022	DRAFT	Updated based on received data	BLJ/RYL
C	January 31, 2022	FINAL	Minor updates based on feedback from client	RLJ/RYL

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