

GRAHAM LAKE IMPROVEMENT DISTRICT  
REPORT NUMBER:

# WATER TREATMENT SYSTEM OPTIONS ASSESSMENT

NOVEMBER 15, 2019

CONFIDENTIAL





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GRAHAM LAKE IMPROVEMENT DISTRICT

FINAL  
CONFIDENTIAL

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# 1 INTRODUCTION

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## 1.1 SETTING

The Graham Lake Improvement District (GLID) obtains its water from Graham Lake, located in the middle of the southern end of Denman Island. The water system was originally constructed around 1970 and has been upgraded a number of times.

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## 1.2 DESCRIPTION OF EXISTING SYSTEM

The Graham Lake Improvement District (GLID) consists of:

- A dam on Graham Lake;
- a submerged raw water intake;
- approximately 750 m of 150 mm diameter AC supply main;
- a water treatment and booster pumping system building;
- a concrete reservoir (clearwell) storage of 4,500 Igal (20.5 m<sup>3</sup>);
- distribution watermains consisting of 100 mm diameter and 150 mm diameter AC watermains and hydrants.

The submerged intake structure consists of six vertical 0.9 m long 150 mm (6") PVC pipes, with 0.04 inch slots, supported by an aluminum frame.

Water flows by gravity from the lake to the treatment/pumping building, where it is filtered by two parallel sand filters, each with a reported capacity of 8.9 L/s (117 Igpm), and then passes through two parallel banks of Ultra-Violet (UV) lights, also rated at 8.9 L/s. The water is then injected with 12% sodium hypochlorite and passes through a stainless steel 3.2 m<sup>3</sup> baffled chlorine contact tank. As the water flows out of the chlorine contact tank, it is injected with ammonia which combines with the chlorine to create chloramine for secondary disinfection, and then flows into the concrete storage reservoir.

Treated water is pumped into the distribution system by a continuously operating 3 hp pump, pressurizing the system to 50 psi. A second pump automatically engages when the system demand exceeds 55 USgpm. A third pump is a back-up and is used if one of the others is out of service.

The water distribution system has a flush-out system that automatically operates for one hour each day, between 2 am and 3 am, turning over the entire volume of water in the main between the pumphouse and the flush-out point, a volume of approximately 4,000 USgal.

A propane generator with an automatic transfer switch backs-up the treatment and pumping system during power outages.

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## 1.3 OBJECTIVES

This assessment is being undertaken as a step by GLID towards becoming compliant with VIHA's *Health Protection and Environmental Services Policy 3.3 Treatment Objectives for Surface Water Supplies and the Drinking Water Treatment Objectives (Microbiological) for Surface Water Supplies in British Columbia (SWTOMSWSBC)*. This policy requires that potable water derived from surface water have at least two processes of operation acceptable to the Vancouver Island Health Authority that together will achieve a 4-log removal/inactivation of viruses and a 3-log removal/inactivation of *Giardia* cysts and *Cryptosporidium* oocysts, and produce a finished water with less than 1 NTU turbidity and 0 coliforms.

Water from Graham Lake has high levels of organics and is subject to disinfection by-products (DBP) formation caused by the reaction between the chlorine used for disinfection and the naturally occurring organic matter in the water. DBP formation also leads to loss of chlorine residual for secondary disinfection. Because of these concerns, an ammonia addition system was installed at the treatment facility in 2014. The addition of ammonia converts the chlorine used for disinfection to a less reactive chloramine that provides residual disinfection while reducing DBP formation.

Although filtration is required for compliance with the SWTOMSWCBC, if the driving factor for filtration is the removal of disinfection by-products forming organics, the necessary turbidity reduction will also be achieved. The technologies designed with the primary purpose of removing organics provide overall high levels of treatment, as organics are smaller than turbidity and are often in a dissolved state. Therefore, most processes that remove organics also remove turbidity.

There is no sanitary sewer close to the treatment facility, so treatment residuals disposal is an important consideration in the selection of a treatment process.

We note that in addition to becoming compliant with VIHA requirements, the GLID systems faces the challenge of being supported by a small community, and that the proposed treatment upgrades will be entirely funded by GLID member households.

# 2 RAW WATER QUALITY AND TREATMENT OBJECTIVES

## 2.1 RAW WATER QUALITY

Table 2-1 below summarizes the key water quality parameters and target water quality objectives, based on water quality records provided by GLID. The number of values provided for each parameter is dependent on FLID water testing frequency for that parameter.

**Table 2-1: Raw Water Quality and Treatment Objectives**

| PARAMETER                       | VALUE                                    | TARGET  |
|---------------------------------|--|---------|
| Turbidity (NTU)                 | 1.76, 2.8, 0.72, 1.59, 0.88              | <1      |
| Alkalinity                      | 17.2, 15.6, 20.5, 15.9                   |         |
| UV Transmissivity (%)           | 59.4, 70.1, 66.6                         | N/A     |
| Total Organic Carbon (mg/L)     | 5.32, 3.67, 4                            | < 3     |
| Dissolved Organic Carbon (mg/L) | 4.93, 3.36, 3.7                          | <3      |
| True Colour (Colour units)      | 36.2                                     | < 15 CU |
| Apparent Colour (Colour Units)  | 40, 23.4                                 | < 15 CU |
| pH                              | 7.52, 7.35, 7.07, 7.29, 7.43, 7.61, 7.42 | 7.5-8.0 |
| Iron (µg/L)                     | 284, 266, 164,                           | <300    |
| Manganese (µg/L)                | 6.8, 4.5, 58.5                           | <50     |

## 2.2 TREATMENT OBJECTIVES

British Columbia regulates municipal drinking water quality through its *Drinking Water Protection Act* (DWPA) and *Drinking Water Protection Regulation* (DWPR). The Act and Regulation on Vancouver Island are administered by VIHA who mandate that the “4-3-2-1-0” treatment objective for surface water supplies as follows:

- 4-log (99.99%) reduction or inactivation in viruses, normally achieved through chlorine disinfection with contact time.
- 3-log (99.9%) reduction or inactivation in protozoa (Giardia cysts and Cryptosporidium oocysts), typically achieved through filtration, or UV disinfection, or both.
- 2 treatment processes for surface water; combining more than one process for treatment allows for a multi-barrier approach against a range of microorganisms.
- 1 NTU turbidity or less; well established filtration technologies can consistently reduce turbidity in the water to <0.1 to 1 NTU.

- No detectable E.coli, fecal coliforms, and total coliforms, typically achieved through disinfection (such as chlorination and/or UV disinfection) or a combination of disinfection and filtration.

In addition to the 4-3-2-1-0 objective, the treatment system must also address the potential for elevated concentrations of disinfection by-products in the water following chlorination.

Therefore, the treatment objectives for the Graham Lake Water System are the following:

- 1 Organics reduction (or reduction of potential for formation of DBPs).
- 2 Turbidity reduction.
- 3 Pathogen reduction: 4-log (99.99%) removal/inactivation of viruses; 3-log (99.9%) removal/inactivation of protozoa (Giardia cysts and Cryptosporidium oocysts); 0 cfu/100 mL of total coliforms, and 0 cfu/100 mL of E.coli.
- 4 Secondary disinfection.

# 3 DESIGN PARAMETERS

## 3.1 WATER DEMAND

GLID requested that the project be based on the following customer base:

- 67 existing GLID connections
- 14 potential additional GLID connections (future)
- 23 potential DIWLSA connections
- Up to 10 potential future connections on vacant land
- BC Ferries Denman East Terminal connection (future).

Based on water records provided for the current connections, Table 3-1 below summarizes the water demand information based on the data provided for three years (2014-2016). We note that additional data for 2017 – 2018 was provided but were not complete. This data was used to confirm the 2014 – 2016 data and was also useful to determine the maximum day demand.

**Table 3-1: Current Average Water Demands**

### WATER DEMANDS

|                                       |            |          |
|---------------------------------------|------------|----------|
| # existing connections                | 56         |          |
| Total Demand per year                 | 6,455,400  | USG/year |
| Flushed volume per year               | 1,362,667  | USG/year |
| DIWLSA consumption per year           | 445,795    | USG/year |
| Demand excl. flushed water and DIWLSA | 4,646,939  | USG/year |
| Demand excl. flushed water and DIWLSA | 17,588,664 | L/year   |
| Demand excl. flushed water and DIWLSA | 48,188     | L/day    |
| Demand excl. flushed water and DIWLSA | 0.56       | L/s      |
| Demand per connection per year        | 82,981     | USG/year |
| Demand per connection per day         | 227        | USG/day  |
| Demand per connection per day         | 861        | L/day    |

The following table shows the maximum day demand from records provided for 2016-2018:

**Table 3-2: Current Maximum Water Demands**

**WATER DEMANDS**

|                                       |         |         |
|---------------------------------------|---------|---------|
| # existing connections                | 56      |         |
| Maximum Day (2016)                    | 50,290  | USG/day |
| Flushed volume per day                | 3,200   | USG/day |
| DIWLSA consumption on maximum day     | 4,933*  | USG/day |
| Demand excl. flushed water and DIWLSA | 42,157  | USG/day |
| Demand excl. flushed water and DIWLSA | 159,565 | L/day   |
| Demand excl. flushed water and DIWLSA | 1.85    | L/s     |
| Demand per connection per day         | 2,849   | L/day   |

\*based on MDD to ADD ratio of 3.5

Because GLID suspects that their flowmeter is not accurate, the demand from the 56 GLID connections was increased by 10% to account for the inaccuracy. Projecting to the ultimate number of connections (113 connections) gives the following future projected average daily demand and maximum day demand:

**Table 3-3: Future Water Demands**

**ESTIMATED FUTURE WATER DEMANDS**

|  |         |       |
|--|---------|-------|
| <b>Average Daily Demand</b>                      |         |       |
| Ultimate # of connections                        | 113     |       |
| Average demand per day per connection            | 982     | L/day |
| Total average demand per day for all connections | 106,960 | L/day |
| BC Ferries connection                            | 2,271   | L/day |
| Flushing water                                   | 4,864   | L/day |
| Total average demand per day                     | 114,095 | L/day |
| Total average demand per day                     | 1.32    | L/s   |
| <b>Maximum Day Demand</b>                        |         |       |
| Maximum Day Demand per connection                | 3,134   | L/day |
| Maximum day demand per day for all connections   | 354,177 | L/day |
| BC Ferries connection                            | 7,949   | L/day |
| Flushing water                                   | 4,864   | L/day |

## ESTIMATED FUTURE WATER DEMANDS

|                          |         |       |
|--------------------------|---------|-------|
| Total maximum day demand | 366,990 | L/day |
| Total maximum day demand | 255     | L/min |
| Total maximum day demand | 4.25    | L/s   |

\*based on MDD to ADD ratio of 3.5 applied

Peak hour can be estimated by applying a factor to average day demand. For communities with a population below 500 people, peak hour factors can be expected to range between 5 and 7.5 applied to average daily demand, with higher factors being applied to smaller communities. Assuming a peak hour factor of 7.5 results in a peak hour demand as follows:

$$7.5 \times 1.32 \text{ L/s} = 9.9 \text{ L/s for future expansion}$$

The current treatment system has a capacity of 8.9 L/s, well above the current and projected average day demand but close to the estimated future peak hour demand.

### DESIGN PARAMETER

**Future Average Daily Demand = 1.3 L/s**

**Future Maximum Day Demand = 4.3 L/s**

**Future Peak Hour Demand = 9.9 L/s**

## 3.2 WATER STORAGE

Providing storage of treated water will balance peak demands, allowing the treatment system capacity to be sized to meet maximum day demand with peak demands provided by the balancing storage. (For the same reason, water storage could also be used for firefighting.)

As well, the treatment system can operate with fewer flow fluctuations, and fewer stop-start cycles. Without storage, a treatment and pumping system can have added complexity to address highly variable demands.

The minimum recommended storage capacity for systems that do not provide fire protection is the average daily consumption, which equates to a storage volume of 114,095 L for the future expanded system. The existing reservoir/clearwell has a capacity of 20,500 L and, therefore, the storage shortfall is 93,595 L.

This capacity needed can be reduced when the source and treatment can meet the peak hour demand. This is the current strategy at GLID where the treatment capacity at 8.9 L/s is well above the peak hour demand.

### DESIGN PARAMETER

**Treated Water Storage Volume Required = 114,095 L**

## 3.3 PUMPING

Presently, water is pumped from the clearwell to the distribution system via three pumps each with a separate suction line from the storage reservoir.

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## 3.4 CHLORINE CONTACT

Chlorination and contact time are required by VIHA to provide 4-log inactivation of viruses. For water at 0.5 °C temperature, a minimum concentration·time (CT) value of 12 mg·min/L is needed to provide 4-log inactivation of viruses. Chlorine disinfection will also provide a residual chlorine concentration in the distribution system for suppression of bacterial growth. Chlorine residuals in the distribution system must be above 0.2 mg/L, but residuals should not be above 0.8 mg/L for aesthetic reasons.

If the disinfection system will provide a minimum target concentration of 0.8 mg/L for treated water (at the discharge from the CT tank), and using a baffling factor of 0.7 for Superior Baffling (the factor for serpentine basin baffles), to achieve a CT of 12.0 mg·min/L, a chlorine contact time of 21.4 minutes is required:

$$12.0 \text{ mg}\cdot\text{min}/\text{L} = 0.8 \text{ mg}/\text{L} \times 0.7 \times t \text{ minutes}$$

$$t = 21.4 \text{ minutes}$$

At a design flow rate of 4.3 L/s, the volume of the contact tank needed is 5,521 L, calculated as follows:

$$4.3 \text{ L}/\text{s} \times 60 \text{ s}/\text{min} \times 21.4 \text{ minutes} = 5,521 \text{ L}$$

The existing CT tank volume is 3,200 L. An additional 958 L of contact time is available in the distribution pipe before the first connection (info provided by GLID). Therefore, the existing contact time in the system is insufficient for the maximum day flow of 4.3 L/s.

A portion of the existing clearwell, which has a volume of 20,500 liters, could also be dedicated to additional CT.

The chlorine residual should be measured downstream of the chlorine contact tank and be maintained above the value of 0.8 mg/L.

### DESIGN PARAMETER

**Minimum Chlorine Contact Volume = 5,521 L (assuming a residual of 0.8 mg/L chlorine at discharge)**

# 4 TREATMENT OPTIONS

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## 4.1 FILTRATION SYSTEM OPTIONS

Historically, water treatment facilities for surface waters were designed to remove turbidity. This led to the implementation of coagulation based media filtration and membrane technologies. Over time, as organics reduction came to be recognized as a significant health issue due to the formation of disinfection by-products when combined with chlorine, conventional chemical treatment processes were modified to include organics removal. This technology became known as enhanced coagulation.

Conventional filtration technologies for organics reduction include sedimentation, dissolved air flotation (DAF), direct filtration, and ultrafiltration (UF) membranes, following the injection of a chemical coagulant and a coagulation/flocculation step.

The use of chemical coagulants for organic reduction is typically limited to around 50% of the raw organic level. DAF and UF membrane processes are typically able to achieve the GCDWQ objectives, but are highly dependent on coagulant dosing for effective organics reduction to reduce DBP's to below GCDWQ limits. Furthermore, DAF and UF are both relatively complex treatment technologies, requiring an experienced operator.

Granulated Active Carbon (GAC) is also effective at removing organics and is very simple to implement. However, both Opus<sup>1</sup> and BIPurewater<sup>2</sup> previously reviewed the option of GAC for organics reduction and both concluded that the amount of GAC media that would be needed would incur unreasonably high operational costs.

As well, high levels of organics lower UVT values which impact the effectiveness of the UV disinfection system.

There are other newer and innovative technologies that focus on organics removal that are becoming more readily available that don't use chemical coagulants.

The primary considerations for selecting a treatment system for GLID, aside from meeting the water treatment objectives, are cost and residuals handling and disposal. The size of the treatment equipment is also a consideration, as it is desirable to install the equipment within the existing building, or within an addition to the building, which will reduce building construction costs, and because there is limited available land within the current easement.

We have selected the following treatment technologies to review:

- 1 Nanofiltration membranes + UV
- 2 Ceramic Ultra-Filtration Membrane + UV
- 3 Ion Exchange Resin + UV
- 4 Direct filtration (coagulation/flocculation/filtration) + UV, with dewatering for backwash water
- 5 Ozone + biofiltration + UV
- 6 Cartridge filtration + UV

These technologies have the potential for achieving organics and turbidity removal with less dependency on chemical usage than that of DAF and UF filtration. Reductions in residuals waste streams over other technologies are also expected through these technologies.

All technologies would be sized to meet the maximum day demand and would, therefore, require reservoir storage to be built, except for the direct filtration and cartridge filtration options which could be sized to meet peak hour demand without significant space requirements compared to a system size for maximum day demand.

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<sup>1</sup> [Denman Island Disinfection By-product Removal](#), Opus DaytonKnight Consultants Ltd, Nay 7, 2014

<sup>2</sup> [Engineering Report](#), BIPurewater, May 3, 2017

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## 4.2 HOLLOW FIBRE NANOFILTRATION MEMBRANE (HFNF)

BIPurewater's report for GLID provides a detailed proposal for this option using Pentair HFW1000 nanofiltration membranes.

Membrane filtration is an established water treatment process that uses a physical barrier to retain any particulates greater than the barrier's pore size. Membrane filtration is a pressure (or vacuum) driven sieve process that removes particles and microorganisms by forcing water through a semi-porous surface. Modern membranes can remove silt and clay particles, bacteria, and protozoa such as *Giardia* and *Cryptosporidium*. Some membranes can remove viruses as well. Membranes can be classified based on their pore size as follows:

**Table 4-1: Types of Membrane Filtration**

| MEMBRANE TYPE        | PORE SIZE | CONSTITUENTS REMOVED  |
|----------------------|-----------|---|
| Microfiltration (MF) | >50 nm    | Silt, protozoa and some bacteria and viruses                  |
| Ultrafiltration (UF) | 2-50 nm   | Large molecules, clay, most bacteria and some viruses         |
| Nanofiltration (NF)  | <2 nm     | Small molecules, viruses and some dissolved metals (hardness) |
| Reverse Osmosis (RO) | <2 nm     | Very small molecules, colour, hardness, ions                  |

For organic reduction, nanofiltration is the minimum barrier pore size to filter organics without the use of a coagulant. Nanofiltration membranes have extremely small pore sizes which filter large molecular weight organic molecules as well as suspended solids in one process step, without the need for any chemical coagulation. It is anticipated that the retained organics and solids can be discharged to the environment as there are no chemical additions.

Pentair HFW1000<sup>3</sup>, proposed by BIPurewater, is a hollow fibre nanofiltration technology composed of Polyethersulfone (PES) and modified PES material. This membrane can reduce feed water turbidity from a maximum value of 25 NTU to less than 0.1 NTU and reduce colour to less than 5 TCU with typical removal of 80% to 90% of dissolved organic carbon (DOC).

Since no chemicals are required for pre-treatment, neither the chemical nor the mineral composition of the water is altered. Feed water is pumped through the NF membrane fibres' interior (inside-out) in a crossflow arrangement to minimize fouling occurrence on the membrane surface. A small reject stream is continuously wasted from the membrane system to control solids concentration and to optimize permeate quality.

Cross-flow filtration is different from dead-end filtration, in which the feed is applied perpendicular through the filter, and particles smaller than the effective pores size pass through as filtrate, and the larger particles build up as a cake layer on the filter. In cross-flow filtration, most of the feed flow travels tangentially *across* the surface of the filter, rather than into the filter. As the feed flows across the membrane surface, filtrate passes through while concentrate accumulates at the opposite end of the membrane. The principal advantage of this is that the filter cake is washed away during the filtration process, increasing the length of time that a filter unit can be operational. It also prevents irreversible fouling of the membrane which can potentially result in shorter membrane lifespan.

To be able to reject the organic molecules, the membranes have a very tight pore structure which results in low membrane flux (filtrate flow per unit membrane surface area) and permeability (rate of flow through the membrane for a given pressure), thus requiring a large number of membranes to treat the required water demand. This results

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<sup>3</sup> Engineering Report, BIPurewater, May 3, 2017

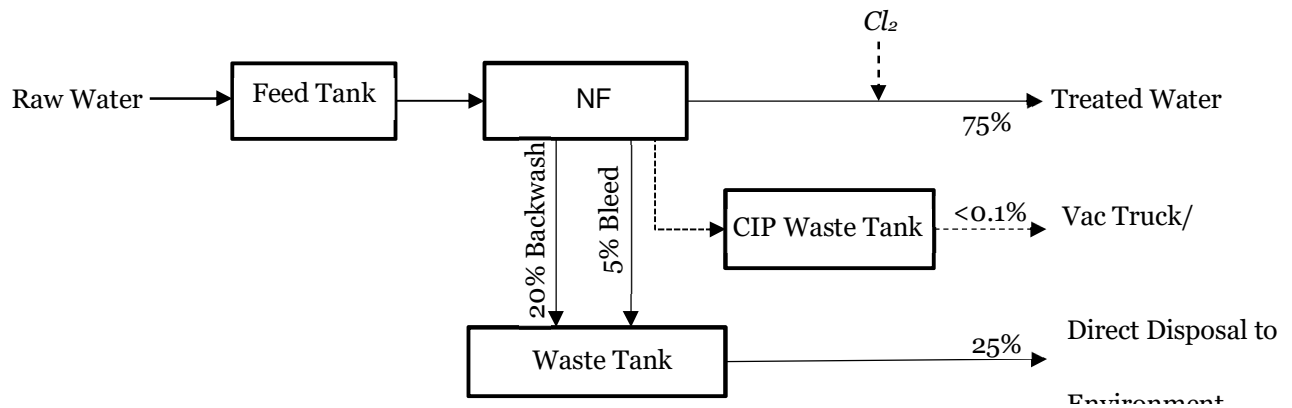
in large system footprint and high system cost. As well, the membranes are prone to breakage and frequent repair and have an expected life of about 5 to 10 years (typically 7 years) after which they must be replaced at a high cost.

The membranes are maintained by frequent backwashing where treated water is applied to the membrane in the reverse filtration direction (outside-in) to dislodge any retained particles in the membrane pores. Aside from the elevated solids and colour concentrations, the backwash stream is free of chemicals and environmental discharge may be viable. The recovery ratio (the volume of filtrate water produced per unit feed water supplied) is limited to about 75% requiring 25% of the feed water to be discharged from the WTP as reject waste water. This large volume of waste (1/3 of the volume of treated water) will likely have significant impact if discharged to ground as currently done for the sand filter backwash.

The membranes periodically must be chemically cleaned using a chlorine, sodium hydroxide and/or hydrochloric acid solution. High pH chemically enhanced cleaning is typically performed every 3 to 4 days. Low pH cleans are also required about every 13 weeks. The resultant chemical wastes constitute less than 0.1% of the total treatment flow and are the only waste stream that requires special handling and disposal. It may be possible to neutralize these cleaning solutions and then discharge them blended with the reject stream to the environment.

If the membranes are verified by daily membrane integrity tests, then UV-disinfection is not required as 4-log reduction of *Giardia* and *Cryptosporidium* is credited through the NF process and would then be considered a disinfection method and provide the 2<sup>nd</sup> barrier of the required dual barrier approach. BIPurewater's proposal includes UV disinfection, and does not provide for daily integrity testing, but states that integrity testing can be provided.

Figure 4-1 below shows a typical process flow diagram of a NF membrane process.



**Figure 4-1: Typical Process diagram of nanofiltration membrane system**

Post NF treatment, the filtered water would be chlorinated and discharged to the existing contact chamber. Chlorine disinfection will provide 4-log removal of viruses and a chlorine residual for secondary disinfection and delivered to the distribution network via the existing distribution pumps. Ammonia addition to produce chloramines will no longer be needed and can be removed as the organics would be reduced.

The amount of chlorine addition is expected to be minimized as the organics present in the water after filtration will be substantially reduced.

The main power demands in this process are the NF circulation pumps, backwash pumps and forward flush pumps. Other operational costs would be cleaning chemicals and membrane replacements.

The treatment system would consist of the following processes:

- 1 Strainer remove large particles
- 2 Pentair nanofiltration system
- 3 UV disinfection (if integrity testing not provided)
- 4 Chlorination + contact time

We note that BIPurewater's proposal is for a system with a capacity of 1.2 L/s, which is approximately equal to the current average day demand.

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### 4.3 CERAMIC ULTRAFILTRATION MEMBRANE (CUF)

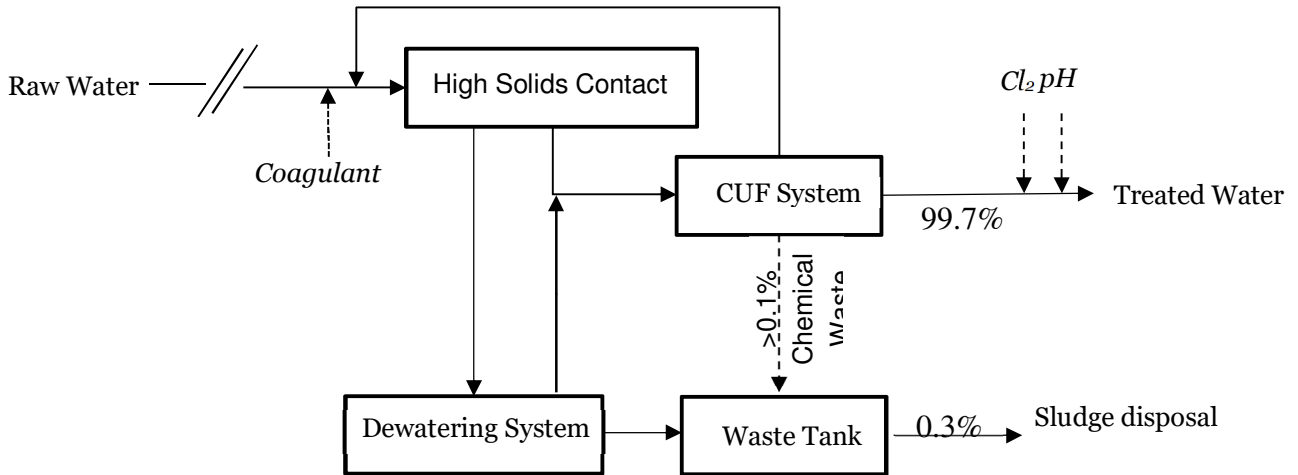
Ceramic ultrafiltration (CUF) is a water treatment technology that combines treatment ideologies from ceramic filtration and membrane ultrafiltration (UF). In ceramic ultrafiltration, the ceramic barrier is manufactured to have a pore size similar to a UF membrane. The ceramic media is typically 100% silicon carbide (SiC), which makes it very resistant to abrasion as well as chemical and biological reactions. The fine UF pore size in the media allows it to reject particles, colloidal material, bacteria, and pathogens. Due to these characteristics, CUF also has the highest operational flow rate (flux) of all UF membrane systems and lowest footprint requirements per volume of water treated. The robust material of the membrane allows it to have a membrane lifespan in excess 25 years.

The use of ceramic membranes in municipal drinking water application is still emerging as ceramic membrane costs are becoming competitive with polymeric membranes. Presently, there are two operating drinking water ceramic membrane plants in the United States that use a Purifics Ceramic Ultrafiltration system, in Delaware (3,800 m<sup>3</sup>/day) and Mississippi (3,300 m<sup>3</sup>/day). One other ceramic membrane plant is being designed for the Cache Creek Casino in Brooks, CA using the Kruger Ceramic Membrane (KCM) of Kruger. Two CUF pilot studies are currently in operation, the 180 MLD Choa Chung Kang Waterworks Plant in Singapore and 9.5 MLD City of Watsonville Water Treatment plant in California.

CUF treatment requires pre-screening of the raw water to remove any coarse particles. Following the screening, coagulant is added at the inlet to the high solids contact reactor (HSCR), where rapid mixing is used for effective mixing. Coagulation is required for the removal of organics and improved turbidity reduction.

From the HSCR tank, water is pumped into the membrane module in a cross flow arrangement. After passing through the membrane, the filtered water would be disinfected and sent to distribution. A waste stream is generated during membrane cleaning through an automatic maintenance cleaning cycle. A percentage of the waste stream water is circulated back to the HSCR in order to reduce the volume of wastewater and to produce a high concentrate solids, which would then be dewatered to a 3% to 10% solids sludge using a dewatering system. Waste volume is anticipated to be 0.3% of the overall process volume, compared to 10% for DAF/ultrafiltration. As such, the produced sludge can be locally stored for off-site disposal by vacuum truck. Overall system efficiency is therefore expected to exceed 99.7%. Figure 4-1 shows a typical process flow diagram of a CUF membrane WTP.

A frequent automatic maintenance cleaning of the CUF is conducted to dislodge any foulants that are attached to the membrane. Occasionally, a full maintenance cycle is activated through a combination of heat, high cross-flow flux (similar to a backwash), as well as chemical applications of acid and/or caustic which will scrub and dissolve residual foulants from the membrane. Overall, this chemical waste constitutes less than 0.1% of the total treatment flow. Disposal of this waste can be combined with the waste from the dewatering system and contained for vacuum truck disposal. Alternatively, a neutralization stage can be added to treat the chemical waste which will enable the plant to directly discharge to the environment, such as through a rock pit.



**Figure 4-2: Typical process diagram of ceramic ultrafiltration membrane**

Post CUF treatment, chlorine disinfection is required to provide 4-log removal of viruses and chlorine residual for secondary disinfection. However, the amount of chlorine addition is expected to be minimized as the organics present in the water after filtration will be substantially reduced. pH adjustment would likely be required post-treatment due to the reduction in alkalinity following coagulation.

A feed pump will likely be required. The spent TMP rinse solutions from the CUF will be neutralized inside the CUF unit and can then be discharged to the existing backwash disposal system. The solids discharge from the DeWRS could be discharged to a 1200 L covered bin for periodic disposal (truck away or local land application). The treated water from the CUF/DeWRS would be discharged into a small bin that can be hauled out by hand using a hand-dolly. The treated water from the CUF/DeWRS would be discharged to the existing contact chamber and then delivered to the distribution network via existing distribution pumps.

The CUF and dewatering system is fully automated and will start and stop automatically when required to meet the varying water demand and will perform its periodic membrane cleaning function without operator intervention or attention. The system performance is logged on its SCADA operator interface and can be remotely accessed via high speed internet connection (either land-line or cellular service is required).

The waste sludge from the WTP may need to be hauled off the island for disposal. However, during Bowen Island pilot testing at Grafton Lake (similar water quality as Graham Lake) an environmental analysis of the waste sludge was performed and found to be suitable for disposal by local ground application.

The CUF unit will perform an automatic membrane cleaning every 1 to 2 weeks that will generate approximately 500 L of neutralized waste water that could be disposed of to the existing backwash drain manhole. Otherwise it could be hauled off the island with the waste sludge for disposal.

The system will require a larger back-up generator to provide power to their pumps, air compressor and controls.

While the CUF & DeWRS system can be operated at reduced flow rates, it is typically run at or near its full design capacity to provide for its most energy efficient operation. Reductions in daily flow consumption is met by running the system intermittently to fill a reservoir volume between some start and stop levels.

The operator(s) will be required to perform the following tasks:

- Remove the collected waste sludge from the DeWRS (swap-out full sludge bin with an empty sludge bin); there is expected to be between 10 to 100 L/day of waste sludge produced.
- Clean-out the raw feed water strainer, estimated once per week.
- Perform a manual membrane Direct Integrity Test (DIT) once per week. This entails initiating the DIT procedure on the SCADA interface and adding a small amount of marker fluid (containing titanium dioxide particles) to the CUF unit.

- Replace the 210 L (55 gallon) ACH coagulant drum when it gets used up; each 210 L ACH drum is expected to last from between 1 to 6 months.
  - Replace the four (4) x 19 L (5 gallon) chemical carboy jugs (caustic soda, sulfuric acid, sodium hypochlorite and calcium thiosulfate) used for membrane cleaning; these chemical jugs are each expected to last 6 months to a year.
  - Perform manual membrane cleaning of the DeWRS ceramic plate membranes possibly once or twice per year; this entails wiping the easily accessible ceramic plates with a cloth and cleaning solution.
  - Provide regular maintenance such as; air compressor air filter replacement and lubrication, instrument calibration and sensor & reagents replacements, pumps/valve/actuator lubrication & seals replacements, general cleaning and housekeeping.
  - Respond to alarm conditions & notifications and provide trouble shooting to resolve.
  - Periodic monitoring and review of system performance.
- 

## 4.4 ION EXCHANGE RESIN

There are many ion exchange (IX) resins available that are designed specifically for the adsorption of Natural organic matter (NOM) from drinking water supplies and can achieve up to 90% removal of TOC. These resins are typically regenerated with a sodium chloride brine solution. The source water is first filtered to remove suspended solids that could foul (plug) the ion exchange resin media beads. The filtered water is then passed through the media where organic molecules are adsorbed onto the media surface by exchange with chloride ions from the media to the water. When the media's capacity for organics adsorption has been exhausted, it is regenerated using a concentrated salt brine solution during which chloride ions replace the organic molecules on the media surface, releasing the organics for discharge with the waste brine.

Small ion exchange systems are similar to home water softeners and are very easy to operate and maintain as their operation, including regeneration, can be automated. However, they generate a large volume of waste brine solution that needs to be disposed of. The ion exchange system will also require periodic backwashing to remove fine suspended solids that may collect on the media, followed by a short period of treated water rinse and discharge to waste each time the filter is started, to flush the water that was 'standing' in the media. The system should also have post filtration to ensure the treated water turbidity remains below 1 NTU, and UV disinfection for pathogen reduction.

The following are the operational tasks for the system:

- Load bags of salt into the brine saturation tank; during peak summer demand, approximately 3 to 5 bags (20 kg each) of salt per week; during the minimum winter demand 1 bag of salt every 1 or 2 weeks.
- Cartridge filter cleaning or replacements for the pre-filter and post filter (estimate one time per week for each filter).
- Transfer of waste brine collected in a waste tank discharge to waste truck. 7,000 L transfer to waste truck, one time every three months (winter) to three times per month (summer).
- Clean-out the raw feed water strainer, maybe once per week.
- Provide regular maintenance such as; instrument calibration and sensor & reagents replacements, pumps/valve/actuator lubrication & seals replacements, general cleaning and housekeeping.
- Respond to alarm conditions & notifications and provide trouble shooting to resolve.

The system would consist of the following processes:

- 1 Pre-filtration
- 2 Ion exchange filters
- 3 Post filtration (cartridge filter)
- 4 UV disinfection
- 5 Chlorination + contact time

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## 4.5 DIRECT FILTRATION

Direct filtration consists of passing water through sand or other media filter by gravity or pumping. A preliminary coagulation and flocculation stage is required where process chemicals are added to destabilize the surface charge of suspended solids and dissolved organics allowing them to combine into larger particles prior to filtration. The filter media may consist of specially graded sand or be a layered multi-media system.

Organics removal of between 30% to 50% and effective colour removal can be achieved using coagulation followed by direct dual media filtration. The larger organic molecules (such as those that cause colour) are more readily bound to floc formed by coagulation which are then captured by the media filtration. Media filtration can be expected to produce water with turbidity < 0.3 NTU and colour < 15 CU. Media filters are very easy to operate and maintain, like the existing sand filters and can have their backwash sequence automated.

To maintain efficient operation, these filters must be backwashed periodically and thus produce a high volume of dirty backwash water containing the coagulation chemicals. Because this backwash water can't be disposed to ground, we propose providing a Purifics DeWRS dewatering recovery system as described in the ceramic membrane section which may produce a sludge solid enough for local land disposal<sup>4</sup>, or which can be stored and disposed of periodically. To minimize waste water generation, the 'filter-to-waste' water produced during the initial stage of filter operation should be recycled back to the head of the treatment system along with any treated backwash recovery water. The system should also have cartridge filtration after the media filters to catch any media fines.

One advantage of direct filtration systems is that it is a well-established technology. Operating costs are typically low. The primary disadvantage is that flocculation and coagulation are most effective in a continuously operated system. Also, the formation of adequate floc is highly sensitive to raw water quality, and turbidity spikes and other changes in water quality and temperature must be managed. Frequent monitoring by an experienced operator would be required to adjust the chemical feed rates as required.

Based on the GCDWQ supporting documentation, direct filtration systems are credited with 2.5-log reduction of *Giardia* and *Cryptosporidium*. Additional protection with a UV disinfection system is required to achieve the required total 3-log reduction of cysts and oocysts. Direct filtration will also provide 2-log removal of viruses.

Residuals from this process are backwash water containing coagulation chemicals.

For the GLID system, we recommend installing the coagulant injection system near the lake, requiring a small shed with power, and replacing the existing sand filters. The travel time between the lake and the treatment/pumping building will provide time for the coagulation and flocculation of the organics and turbidity to take place, however, the effectiveness of this arrangement for organics removal will need to be confirmed through pilot testing.

The chlorination system would not be needed as the organics would be reduced.

The major O&M costs associated with direct filtration treatment are coagulation chemicals and pumping of backwash water. Operator(s) will be required to perform the following tasks:

- Remove the collected waste sludge from the DeWRS (swap-out full sludge bin with an empty sludge bin); between 20 to 100 L/day of waste sludge is estimated to be produced.
- Replace the 210 L (55 gallon) ACH coagulant drum when used up; each 210 L ACH drum is expected to last from between 2 weeks to 4 months.
- Cartridge filter replacements and cleaning for the post filter (estimated once per week).
- Clean-out the raw feed water strainer, estimated once per week.

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<sup>4</sup> During pilot testing, testing of Grafton Lake water (with similar water quality as Graham Lake), for the new Bowen Island treatment plant, an environmental analysis of the waste sludge was performed and found to be suitable for disposal by local ground application.

- Provide regular maintenance such as; instrument calibration and sensor & reagents replacements, pumps/valve/actuator lubrication & seals replacements, general cleaning and housekeeping.
- Respond to alarm conditions & notifications and provide trouble shooting to resolve.

The system would consist of:

- 1 Coagulation/flocculation
- 2 Filtration
- 3 UV disinfection
- 4 Chlorination + contact time
- 5 Dewatering of backwash

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## 4.6 OZONE + BIOFILTRATION

This option is an emerging technology that is becoming more common in North America and has been in use for some time in Europe. This technology uses ozone to break down the organic compounds in the water which is then passed through GAC filters where bacterial growth in the filters is promoted and maintained. The bacteria in the filter will assimilate the organic carbon. The system works well for low alkalinity waters, which is the case for the water from Graham Lake.

Ozone oxidation of colour and organics followed by biofiltration is in use in many small and large treatment plants, mainly in Europe and North America. An on-site ozone generator is used to dose ozone gas into the raw feed water, followed by an ozone contact tank to provide reaction time for the ozone with the organics in the water. Ozone when dissolved in water produces highly reactive hydroxyl free radicals which readily oxidize the colour and larger organic molecules of natural organic matter (NOM). The NOM typically found in surface waters primarily consists of large organic molecules that are not ready bio-available (recalcitrant organics that have not been consumed by microbes in the environment). Oxidation of these molecules by ozone transforms them into smaller organic molecules that are readily bio-available.

The water and oxidized organics leaving the ozone contact tank is then passed through a media filter (usually GAC media) which is allowed to become colonized by naturally occurring bacteria from the source water. These bacteria consume the bio-available organics (BDOC). The bio-film formed on the media also provides an excellent medium for capture of suspended solids as the water filters through it. The result is that the treated water leaving the biofilters will have highly effective removal of colour (< 15 TCU), reduction in TOC (20% to 30% reduction) and reduced turbidity (< 0.3 NTU). The treated water from a biofilter will be biologically stable and thus reduce regrowth potential in the distribution system.

As the bio-mass grows and suspended solids are captured on the media in the biofilters, the void spaces between the media become reduced resulting in increased pressure drop through the filter. Periodically, excess bio-mass and captured suspended solids will need to be backwashed from the filters. Under normal operation the filters need to be backwashed with un-chlorinated water, however, occasionally a biofilter may need to be backwashed with chlorinated water to suppress the growth of filamentous (bulking) bacteria that could cause excessive filtration head-loss. An air scour step is also commonly required in the backwash sequence to dislodge bacteria colonies and biofilm from the media surface before backwashing the loosened material out.

Granular Active Carbon (GAC) is commonly used as the biofilter media because it provides a very high surface area to support a large number of bacteria providing good biofilter performance in cold water. It typically takes several months for a biofilter to become colonized with bacteria to the point where they can consume the BDOC. During this bacteria establishment phase, the GAC will typically provide good adsorption of the organics from the water to produce good treated water quality.

The ozone-biofiltration process is typically easy to operate. The ozone generators and dosing can be automated and do not typically require much attention and the biofilters are operated (and backwashed) much like typical media filters. However, maintaining the correct conditions to promote the growth of preferred types of bacteria (and avoid problematic bacteria) will likely be an on-going challenge. The biofilters will need to be maintained in continuous operation by modulating the plant capacity to meet the required daily demand. If the filters are stopped for an

extended time, the bacteria can turn anaerobic and consume each-other, degrading the filter performance and imparting foul odour to the water.

To avoid anaerobic conditions, the biofilters must be provided with sufficient supply of dissolved oxygen at all times. This can be accomplished by recirculation of aerated water through the biofilters during times of little to no water demand. In some instances, biofilters may require addition of nutrients to control the Carbon:Nitrogen:Phosphorus balance and promote useful bacteria growth to maintain optimum biofilter performance. While control of the biofilter operating conditions, or addition of nutrients is not at all difficult or costly, the analysis and evaluation of the biofilter, the make-up of its biomass and the determination of which course of actions may be required could be challenging (ie. how to figure out what the bacteria need when the process is not operating well). It may take some time to develop an understanding and history of how to best maintain the biofiltration system operation through changing operating conditions (like WTP demand, raw water quality and temperature).

Ozone is an unstable gas, and therefore, ozone must be generated on-site, typically by corona discharge using concentrated oxygen as feed gas. Natural organic matter is partly oxidized when treated with ozone and becomes more easily biodegradable. The combined treatment of ozone with a biofilter can result in DOC reductions of 40-60%.

The biofilters must be periodically backwashed to remove accumulated suspended solids and bio-mass growth. Since no pre-treatment chemicals are required, the backwash water can likely be discharged to the environment. However, the volume of dirty backwash water from biofiltration could be a challenge. The backwash water is expected to be quite dirty containing suspended solids, carbon media fines and bacteria colonies and will most likely not be suitable for discharge into the existing backwash pit. Biofilter backwash water is typically easily settleable or filterable so it would likely be possible to filter-out most of the dirt from the backwash water to either reclaim the water (or some portion of it) back to the head of the WTP, or to discharge the filtered backwash water to the existing backwash pit. Settling or filtration of the backwash water may require the addition of a polymer flocculant aid chemical and would require disposal of the solids waste sludge collected.

Ozone-biofiltration is highly conducive to incremental expansion of the plant capacity. Additions to the plant capacity could be accommodated simply by adding additional biofilter vessels as required to meet an expected increase in demand with the rest of the process equipment sized for the full build-out capacity.

The biofilters should include a gravel bed and sand layer under the carbon media to filter-out any bacteria that slough-off the media during operation.

As this is an emerging technology, likely post cartridge filtration and additional protection with a UV disinfection system will be required to assure total 3-log reduction of *Giardia* cysts and *Cryptosporidium* oocysts.

The chloramination system would not be needed as the organics would be reduced.

For an ozone-biofiltration system the operator(s) will be required to perform the following tasks:

- Regular monitoring and review of system performance.
- Remove the collected waste sludge from the backwash recovery bag filter (ie swap-out filter bags and dispose of solids). There is expected to be between 2 to 20 kg/day of waste solids produced.
- Clean-out the raw feed water strainer, maybe once per week.
- Initiate backwash of the biofilters, maybe once per week to once per month each.
- Provide regular maintenance such as; oxygen concentrators air filter replacement and lubrication, instrument calibration and sensor & reagents replacements, pumps/valve/actuator lubrication & seals replacements, general cleaning and housekeeping.
- Respond to alarm condition & notifications and provide trouble shooting to resolve.

Backwash waste management (filtering or settling, recirculation or discharge, solids collection and disposal) will likely need to be determined during the plant operation due to uncertainty in the volume and character of the biofilter backwash waste.

The system would consist of:

- 1 Ozonation
  - 2 Bio-Filtration
  - 3 UV disinfection
  - 4 Chlorination + contact time
  - 5 Backwash filtering
- 

## 4.7 CARTRIDGE FILTRATION

Cartridge filtration can be used to reduce turbidity in some water supplies and is well suited for small systems because it has a low capital cost, is operationally simple, does not produce any residuals to be disposed of, and has a compact footprint. When cartridges become clogged, they must be replaced, so the operational cost of replacing cartridges must be considered. Replacement frequency of cartridges can be anywhere from days to months, depending on the quality of the source water. Considering the characteristics of Graham lake and its water quality analysis, it is highly recommended that a pilot test of the performance of cartridge filtration should be performed in order to determine the achievable treated water turbidity reduction as well as the rate of solids loading of the cartridge filters.

Washable filter cartridges are available in 5 micron or larger filtration size as pleated surface filtration media. These can be used as pre-filters to reduce load and increase the service life of downstream finer filters that are not washable (are discarded after each use).

Cartridge filtration will not remove the organics from the water so formation of disinfection by products and loss of disinfectant residual would continue to be a problem in the distribution system.

Cartridge filters are rated by one of two systems: nominal or absolute. An absolute particle size rating does not allow any particles larger than the specified size to pass, whereas a nominal rating refers to the average particle size retained. For turbidity particles smaller than 1 microns, the absolute particle size rated filters are required.

There are no prescriptive federal or BC provincial guidelines for cartridge filtration. However, under Alberta guidelines, 1-micron absolute cartridge filters are credited with 2-log removal of *Giardia lamblia* cysts and *Cryptosporidium* oocysts. Some reviewing authorities may require higher log removals, so this must be discussed with VIHA.

For pathogen reduction, the Alberta Waterworks Standards require cartridge filters to undergo challenge testing, and require a treated water turbidity below 0.3 NTU, allowing up to 1.0 NTU for a maximum of 15 minutes per day. Since any particles breaching the 1-micron absolute filter are smaller than *Giardia* and *Cryptosporidium* cysts, protection of cysts by suspended particles is not considered to be a concern.

UV disinfection must be added to the treatment system and will provide 3-log removal of *Giardia* and *Cryptosporidium*, which meets the pathogens reduction target, and then the filtration system would not be relied upon for pathogen reduction. A minimum UVT of 70% is required for UV disinfection to be effective, and Graham Lake water UVT has been measured (3 samples) with values ranging from 60% -70%. It is possible that the cartridge filters would improve UVT, but this can only be confirmed by pilot testing. It would also be prudent to conduct pilot testing to determine how frequently the cartridges would need to be changed out.

Cartridge filtration provides some protection against viruses and bacteria, but the UV disinfection and subsequent chlorination and contact time would be relied upon for virus and bacteria inactivation.

Chlorination and ammonia addition to create chloramines for residual disinfection would still be required.

Cartridge filtration will have the lowest capital cost and smallest footprint that would easily fit into the existing WTP building but will not realize the treatment goal of meeting Island Health's drinking water objectives.

The primary operational costs for this system would be the cost of replacing cartridges.

The system would consist of the following processes:

- 1 Pre-treatment to remove large particles
- 2 1 micron (absolute) cartridge filtration

- 3 UV disinfection
- 4 Chlorination + contact time
- 5 Ammonia addition (to create chloramine for secondary disinfection).

## 4.8 TREATMENT OPTIONS ASSESSMENT

The following matrix provides a comparison of the filtration options:

**Table 4-2: Comparison of Filtration Options**

| CRITERION                 | HFNF<br>PENTAIR  | CERAMIC<br>ULTRA-<br>FILTRATION                              | ION<br>EXCHANGE  | DIRECT<br>FILTRATION  | OZONE +<br>BIOFILTER   | CARTRIDGE<br>FILTER     |
|---------------------------|--|--|--|---|--|-------------------------|
| System Complexity         | Moderate   | Moderate   | Low  | Moderate  | Moderate   | Low                     |
| Technology                | Established  | Established  | Established  | Established   | Emerging   | Established             |
| Process Risk              | Negligible   | Negligible   | Low  | Need to establish and adjust coagulation and assess                   | May need regular attention to assess bio activity and adjust | High                    |
| Maintenance               | Low (highly automated)   | Low (highly automated)                                       | Low  | High  | High   | Low                     |
| Reliability               | High likelihood of frequent membrane breakage & repair                     | Robust   | High   | Proven reliability, but requires attention                            | High   | Robust                  |
| Residuals                 | 25% reject water direct discharge to environment, spent cleaning chemicals | Thickened sludge, dispose at landfill, or possibly to land   | Backwash and rinse water / Brine waste from regeneration | Thickened sludge with DeWRS, dispose at landfill, or possibly to land | Backwash with concentrated water and biomass                 | None                    |
| Disinfection Requirements | Chlorine (+ UV if no integrity testing)                                    | Chlorine + UV  | Chlorine + UV  | Chlorine + UV   | Chlorine + UV  | Chlorine + Ammonia + UV |
| Chemicals                 | — Membrane cleaning chemicals  | — Requires coagulant dosing<br>— Membrane cleaning chemicals | Uses salt for regeneration                               | Requires coagulant dosing   | None   | None                    |

| CRITERION                 | HFNF<br>PENTAIR   | CERAMIC<br>ULTRA-<br>FILTRATION   | ION<br>EXCHANGE           | DIRECT<br>FILTRATION | OZONE +<br>BIOFILTER  | CARTRIDGE<br>FILTER   |
|---------------------------|---|---|---------------------------|----------------------|---|---|
| Expandability             | No  | No  | Yes                       | Yes                  | Yes   | Yes   |
| Footprint                 | Large, significant addition needed  | Small (will fit in existing building)   | Small                     | Small                | Moderate  | Small (will fit in existing building)   |
| Reservoir storage needed  | Yes   | Yes   | Yes                       | No                   | Yes   | No  |
| Other                     | <ul style="list-style-type: none"> <li>– Large power req'mt</li> <li>– Will need larger backup generator</li> </ul> | <ul style="list-style-type: none"> <li>– Large power req'mt</li> <li>– Will need larger backup generator</li> </ul> |                           |                      | <ul style="list-style-type: none"> <li>– Need to maintain continuous flow with fresh oxygen supply to biofilters to prevent going anaerobic</li> <li>– Occasional replacement of GAC</li> </ul> | <ul style="list-style-type: none"> <li>– Pilot testing recommended to determine effectiveness and filter replacement frequency</li> <li>– Highly unlikely will meet VIHA requirements due to low UVT</li> </ul> |
| Capital Cost              | \$1,860,000   | \$1,180,000   | \$360,000<br>(\$480,000)  | \$800,000            | 690,000   | \$220,000   |
| Monthly operational costs | \$2,050   | \$1,636   | \$2,860<br>(\$470)*       | \$1,170              | 460   | \$1,970**   |
| 20-year Life Cycle Costs  | \$2,167,000   | \$1,425,000   | \$788,000<br>(\$550,000)* | \$975,000            | 759,000   | \$515,000**   |

\*Numbers in brackets are for system that has ocean disposal of brine water

\*\*could increase substantially if more frequent cartridge replacement required.

# 5 COST ESTIMATES

The Class 'D' cost estimate for each option including 40% contingency, as well as O&M estimates and life cycle costs are detailed in Appendix A attached.

# 6 LAYOUTS AND SCHEMATICS

Layouts for ion exchange, direct filtration and ozone and biofiltration are attached in Appendix B. Schematics for ceramic ultra-filtration, ion exchange, direct filtration and ozone-biofiltration are attached in Appendix C.

# APPENDIX





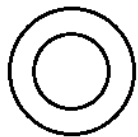
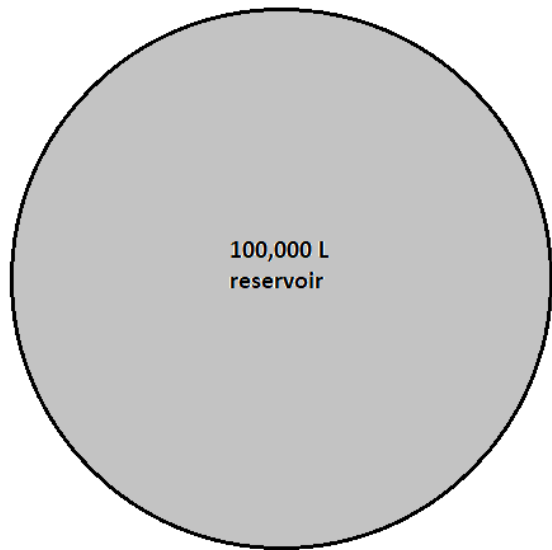
APPENDIX A  
Cost Estimates  
(Capital, O&M and Life Cycle Costs)

In keeping with procurement best practices, the detailed cost estimates for each of the treatment options have been removed from this document.



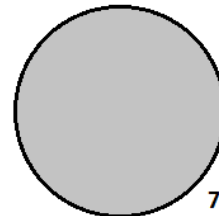
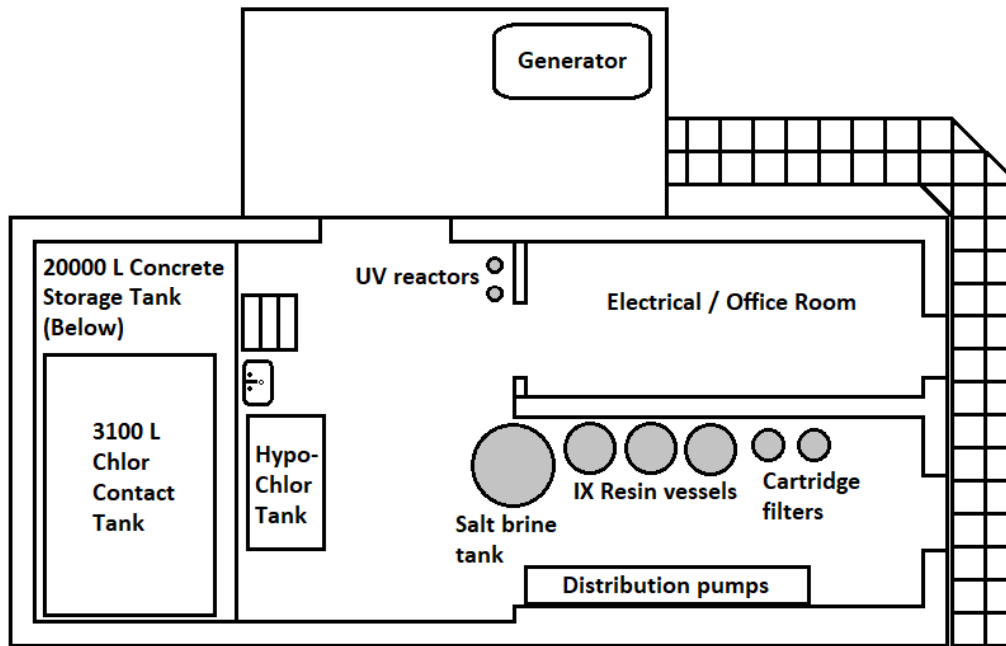
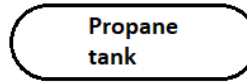
## APPENDIX B

### Layouts



BW drain manhole

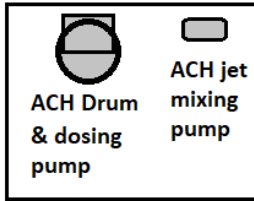
270 LPM Ion Exchange Organic Resin Trap



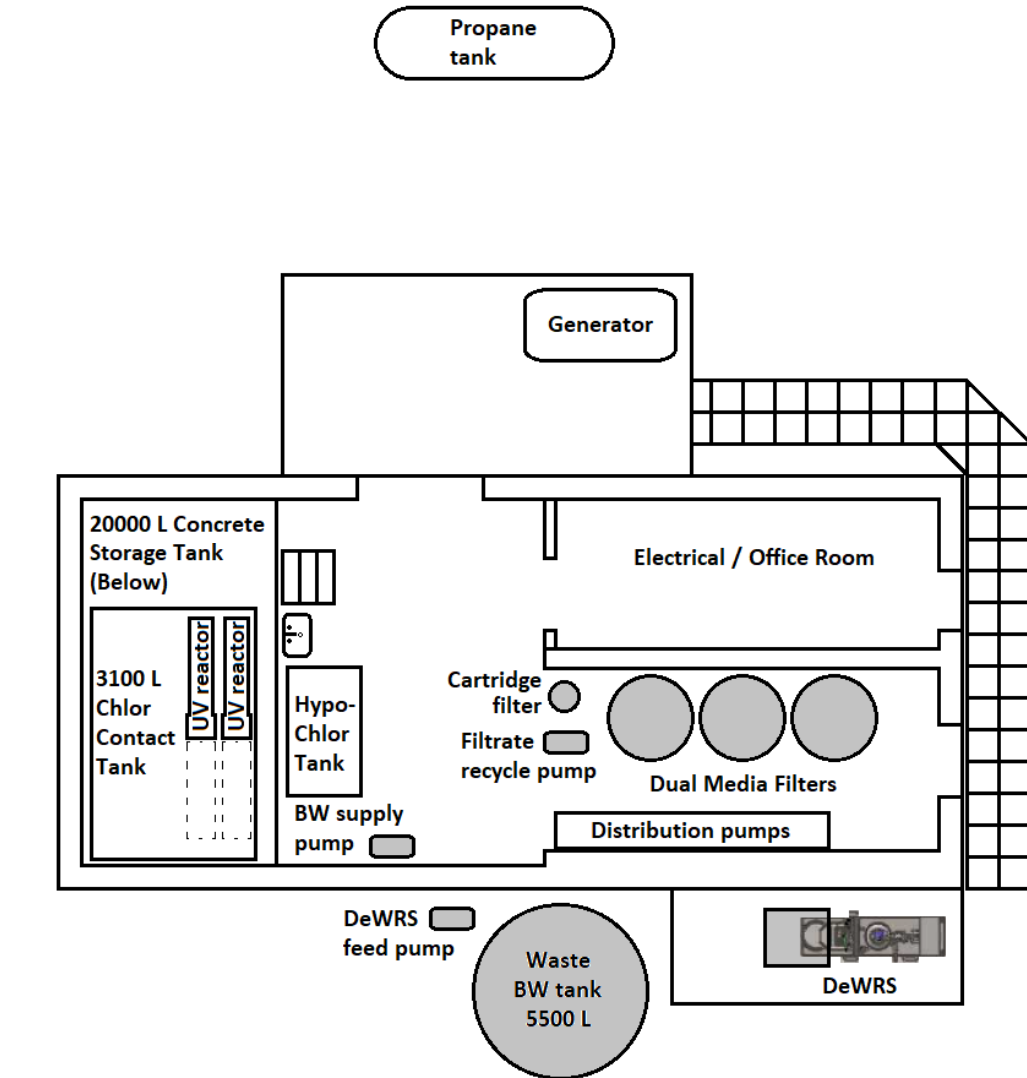
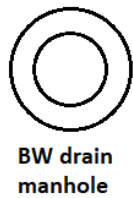
Waste brine pump

7500 L Waste brine holding tank

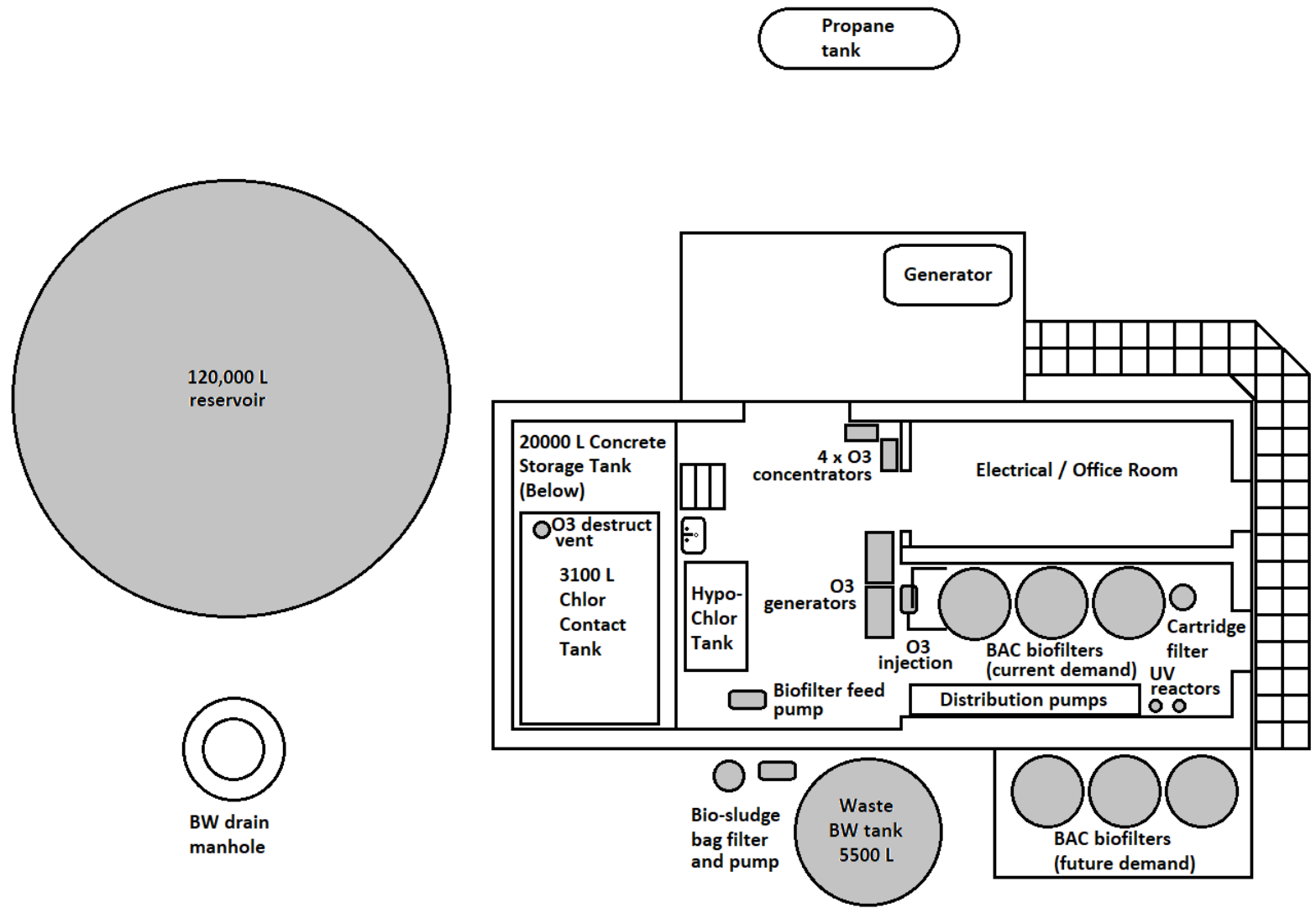




New coagulant dosing shed (at lake or along the raw water pipeline)



680 LPM Direct Filtration with DeWRS M8

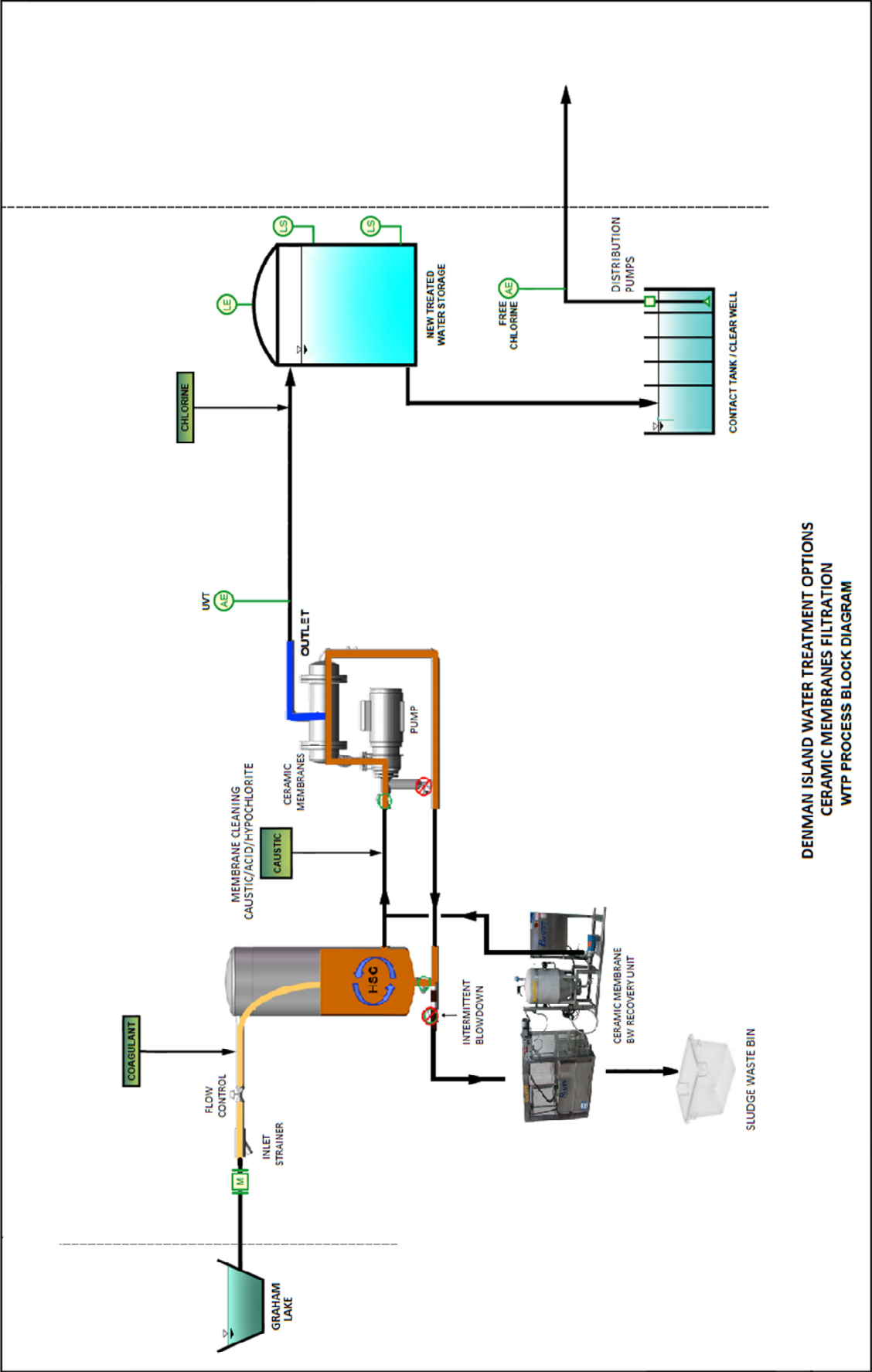


270 LPM Ozone Oxidation with Bio-Filtration

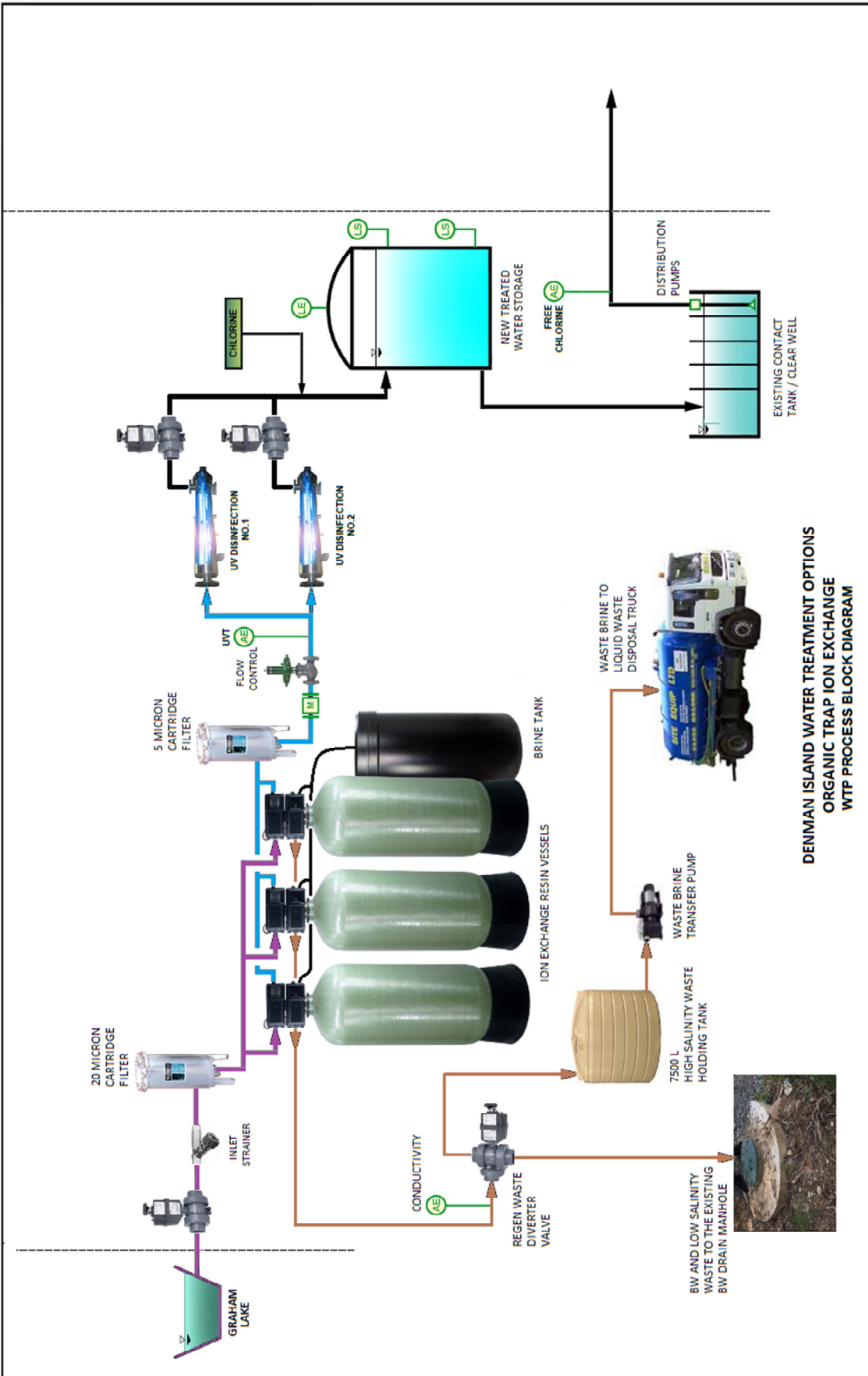




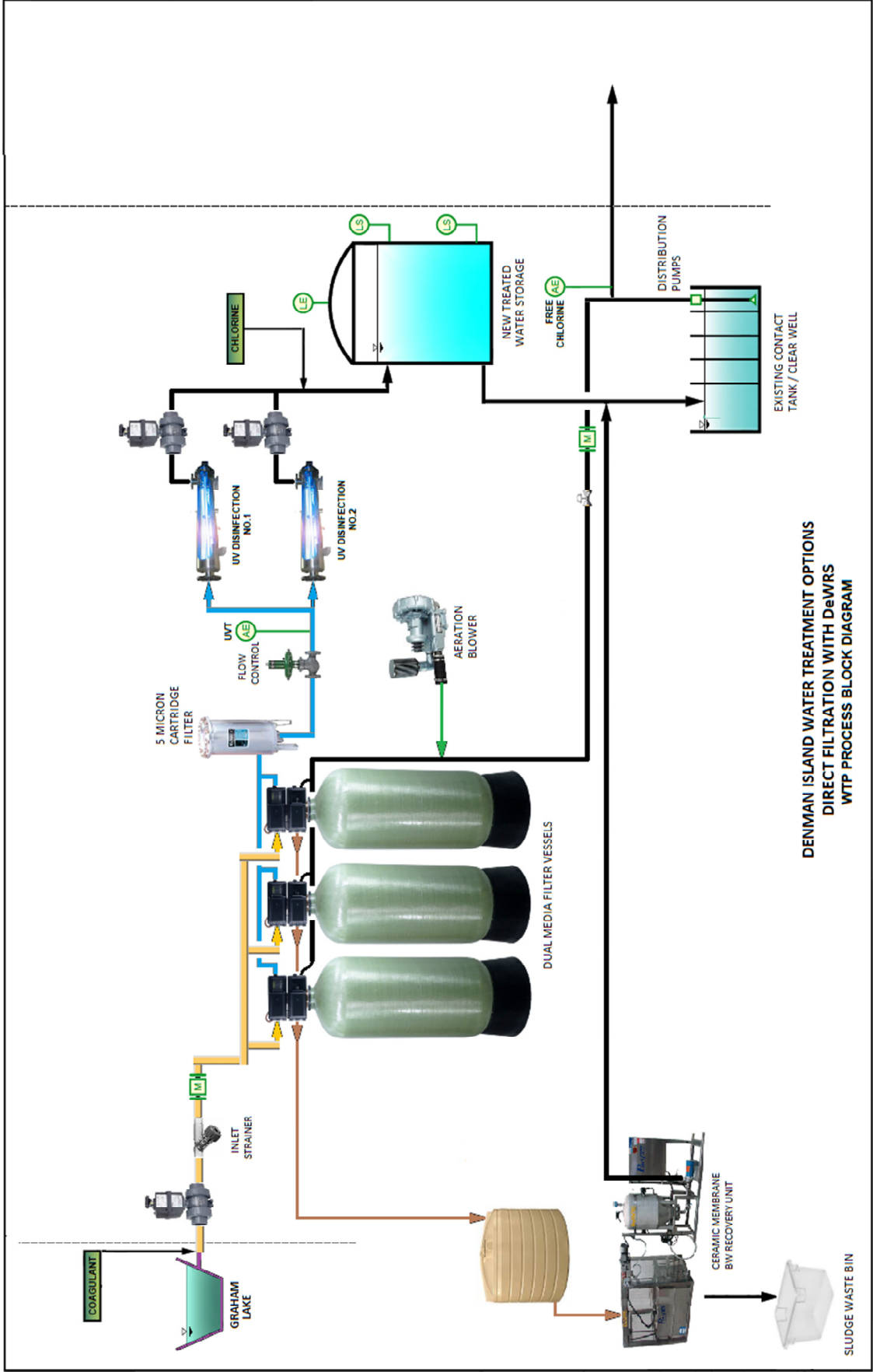
APPENDIX C  
Schematics



DENMAN ISLAND WATER TREATMENT OPTIONS  
 CERAMIC MEMBRANES FILTRATION  
 WTP PROCESS BLOCK DIAGRAM



**DENMAN ISLAND WATER TREATMENT OPTIONS  
ORGANIC TRAP ION EXCHANGE  
WTP PROCESS BLOCK DIAGRAM**



**DENMAN ISLAND WATER TREATMENT OPTIONS  
DIRECT FILTRATION WITH DeWRS  
WTP PROCESS BLOCK DIAGRAM**

