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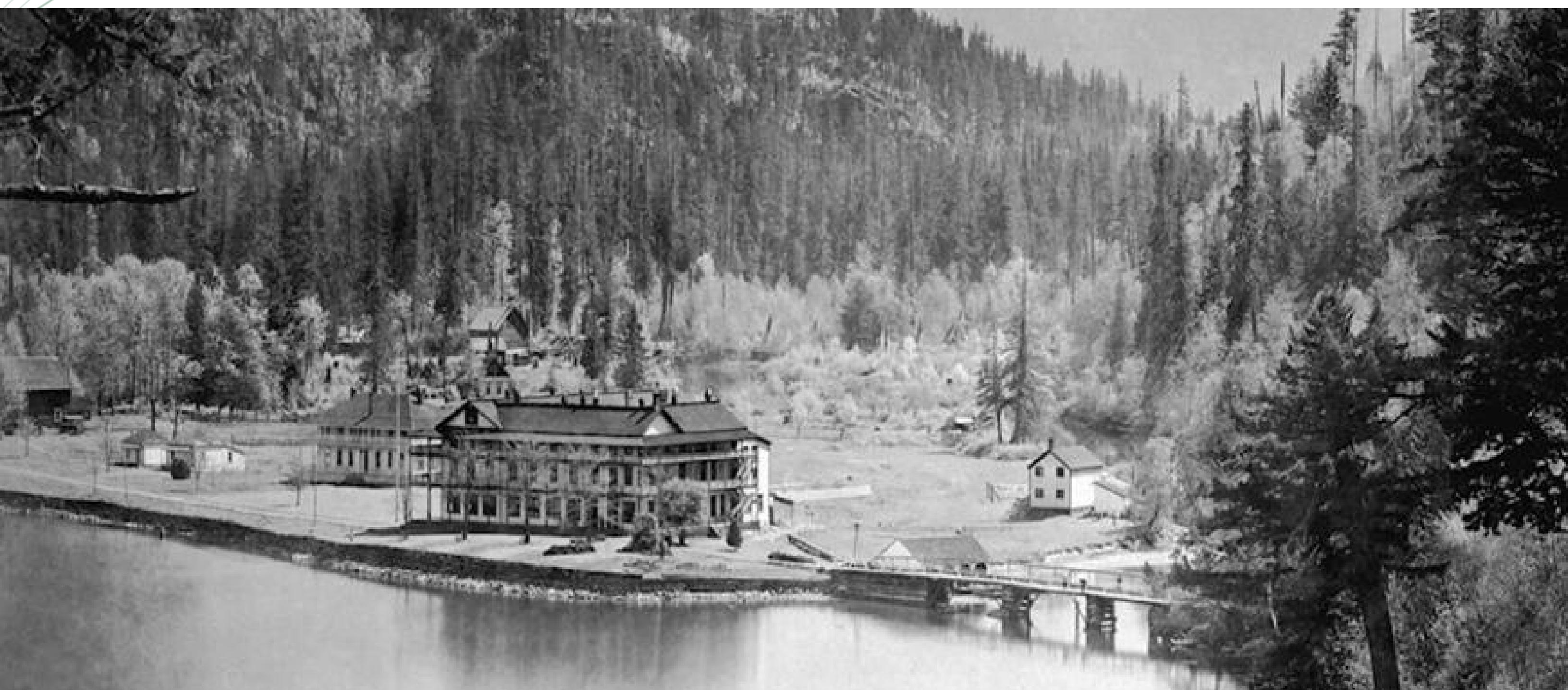


Harrison Hot Springs

Wastewater Treatment Plant - Master Plan

FINAL – Rev 1

July 15, 2026

Submitted to: Village of Harrison Hot Springs
Prepared by: McElhanney Ltd.

Contact
Mike Canning, P.Eng.
Project Manager
603-424-4867
mcanning@mcelhanney.com

Address
100 – 8837 201 Street, Langley
BC Canada V2Y 0C8

Prepared by
Michael Panidisz, P.Eng.
Design Engineer

Reviewed by
Karen Sutherland, M.Eng., P.Eng.
Wastewater Engineer

Our file: 2241-25029-01

YOUR CHALLENGE.

OUR PASSION.

Executive Summary

INTRODUCTION

As part of their objective of maintaining and improving their wastewater treatment infrastructure both now and for the future, the Village of Harrison Hot Springs (VHHS) commissioned McElhanney to prepare a Master Plan for their Wastewater Treatment Plant (WWTP). Key findings are summarized below.

PERMIT AND EFFLUENT QUALITY

The wastewater treatment plant effluent quality meets and exceeds anticipated permit requirements.

Permit requirements are noted as anticipated since the permit was never amended after the 2012 upgrades; however, staff have been operating the WWTP under the effluent permit requirements that would likely be imposed based on regulations.

PLANT FLOWS

Influent Flows

Influent Lift Station #1 (LS#1) pumps all the collection system flows to the wastewater treatment plant. Until the LS#1 upgrade was completed in the spring of 2025, the only flow measurement was on the plant outfall, after the attenuation of flows in the bioreactor tank. The 2025 upgrades included the addition of a flow meter to track influent flows.

Tracking plant flows using the effluent meter was not ideal as influent peaks and flow patterns were not well understood due to the attenuation of flows in the bioreactor. In addition to flow attenuation, the bioreactor tank floor is equipped with pressure relief valves which are designed to open when the water table reaches a certain elevation to avoid structural damage to the bioreactor. The pressure relief set point is unknown, the opening of the valve cannot be recorded, and the volume of water into the bioreactor cannot be estimated. Staff have observed the bioreactor level rising during high lake levels.

Consequently, the existing influent minimum, average, maximum day and peak flows, all parameters used to size process equipment, could not be confirmed at the time of preparing this report.

Inflow and Infiltration

High inflow and infiltration (I&I) in the collection system is suspected due to high effluent flows during the winter season, when the number of residents and tourists is low, but rainfall is high. The high I&I continues into the spring and early summer, when the lake level and surrounding water table rises.

There is a drop in plant effluent flows just after the peak of tourist season in September, when there is also little rainfall. The effluent increases again in conjunction with rainfall.

To delay more costly treatment plant capacity upgrades by reducing influent flows, every effort should be made to reduce I&I in the collection system.

Future Flow Data Analysis

With the addition of the LS#1 flow meter, influent flow data should be analysed for the next few years to better understand flow patterns and estimate how much water might be relieved into the bioreactor during high lake level events, increasing treatment costs. The flow analysis should also be used to understand the impact of I&I reduction measures.

Understanding the flows will also facilitate confirming timing of process upgrades based on existing capacity.

CONDITION AND CAPACITY ASSESSMENT

Generally, the capacity of the treatment plant components meets current flows, and most components can meet future flows. There are a number of smaller recommended projects, many of which can be taken on by operations staff. However, drivers for larger projects and upgrades recommended in the short to medium term are outlined below.

Regulatory Drivers

To meet the Municipal Wastewater Regulations (MWR) redundancy requirements, there should be two biological treatment tanks, allowing for isolation and maintenance of one tank during low flow periods.

The existing bioreactor is oversized, even for use as an extended aeration tank, and there would be sufficient volume to split the bioreactor in three, with two bioreactor zones and one equalization zone.

However, as the existing bioreactor bottom is approximately 1.0m below the 1/5-year water table, simply dividing the existing bioreactor is difficult and costly. In addition, based on current I&I and high-water table occurrences, maintaining an equalization tank is recommended.

Of the redundancy options considered and priced, the preferred option based on ease of construction was the construction two small biological process tanks outside the existing bioreactor, allowing the existing bioreactor to be refurbished and re-used as an equalization tank.

Another option might be to construct within the footprint of the existing bioreactor. This would include damming a portion of the bioreactor and adding rock fill to raise the bioreactor floor and constructing new process tanks on the rock fill. This would allow the remaining portion to continue to serve as a treatment tank. Once the bioreactor tanks are commissioned, flow would be diverted through these new tanks, and the remaining area would be refurbished, as required, to serve as an equalization tanks. As the condition of the existing bioreactor base is unknown, there are more risks related to the 2nd option, but a key advantage is that the footprint of the bioreactor is not lost space.

Regardless, to meet MWR redundancy requirements, eventually replacing the existing bioreactor with smaller optimized treatment processes, resulting in lower operating costs is recommended. Options would have to be developed in more detail to assess the feasibility of construction and weigh other pros and cons before a preferred solution could be recommended.

Capacity and Condition Upgrade Drivers

MBRs: The third Membrane Reactor (MBR), downstream of the bioreactor should be installed. The current capacity of the two MBRs is approximately equal to the maximum effluent flows, while the capacity of one MBR is slightly below average day flows. However, there is no redundancy in the event that an MBR needs to be taken out of service.

Centrifuge: The existing centrifuge is adequate, however there are on-going maintenance issues and replacement parts are difficult to find. Funding has been set aside to replace the centrifuge as there is no redundancy. When replacing the centrifuge, the opportunity should be taken to increase the capacity and improve the layout.

Outfall Upgrades: Outfall capacity upgrades will be required, and the timing will be a function of influent flows and lake levels.

Treatment Process: There are some opportunities to optimize the existing process and most of this work can be done in-house.

PROJECT COST ESTIMATE AND POTENTIAL FUNDING SOURCE

Upgrades were identified and grouped into two major categories based on funding source:

1. Annual operating reserves. Some projects would be undertaken in-house (by Operations Staff) to reduce overall costs, whereas some project might require engineering design services.
2. Grant funding candidate. These projects are typically larger complex projects which require engineering design services.

Some projects in both categories might also be candidates for DCC funding, if upgrades also result in treatment capacity increases.

Note that if grant funding is not available, some projects may require financing if the timing of the project cannot be delayed, either due to the process or component being at capacity due to increased influent flows or equipment being at the end of its life.

Assumptions

- Capital cost estimates were prepared for all projects.
- Based on the level of detail at the time of preparing the estimates, the cost estimate should be considered Class D. Thus, 40% should be added as a contingency to the capital cost to cover unknown details.
- Engineering fees were estimated at 15% of the capital costs and are identified separately.
- Projects that VHHS deemed could be undertaken in-house did not included engineering fees.
- Escalation costs were not included in the estimates as the exact timing of most projects is unknown and will depend on other factors such as I&I reduction, influent flows, and funding availability. An annual 3% escalation fee should be added to the project costs on top of the 40% contingency and 15% engineering, depending on the final timing of the project.
- All costs and fees are rounded up to the closest $1000.

Schedule

The projects were also grouped into three (3) time frames as follows:

- Short-term: 0-5 years
- Medium-term: 5-10 Years
- Long-term: 10-20 Years

The proposed cost estimates and project schedule based on these time frames is presented in Figure E.1 below. The detailed cost estimates are provided in Appendix B of the report.

Long-Term Project Considerations

Although some of the recommended upgrades are relatively straightforward, advances in technology and construction methodologies, new permit requirements, and layout options not investigated herein may impact the selection of equipment or processes in the future.

The suggested technologies, such as the headworks equipment and new biological treatment (MBBR) trains, should be considered as a guideline until implementation is imminent, and additional options can be evaluated and cost estimates refined. With this in mind, allowances to investigate biological treatment options, and resulting refurbishments needs and associated costs have been included rather than providing a fixed estimate.

Note that at the time of finalizing this report, additional layout and construction options were already being investigated by Operations Staff,

Table E.1 Summary of Project Capital Cost, Funding Source and Anticipated Implementation Timeline

Capital Costs and Project Schedule

Village of Harrison Hot Spring

WWTP - Master Plan - Capital Improvements

Project No.	TASK DESCRIPTION	DCC Eligibility	Short-Term 0 - 5 years		Medium-Term 5 - 10 years		Long-Term 10 - 20 years	
	Wastewater Treatment Plant							
	ANNUAL OPERATING PROJECTS		Engineering	Capital	Engineering	Capital	Engineering	Capital
S.1	Influent Flow Analysis		✓					
S.3	New MWR Permit - *Allowance*		$ 200,000					
S.4	Chemical Storage Upgrades -Ventilation		$ 8,000	$ 35,000				
S.5	Process Building - Floor Repair - In-House			$ 17,000				
S. 6	Process Building - Chain at Overhead Door for Safety, In-House			$ 300				
S.7	Addition of 3rd MBR Frame - In-House	✓		$ 62,000				
S.8	MBR and WAS Tank Concrete Repair, In-House			$ 126,000				
S.9	New Centrifuge and Building and Assoc. Equipment	✓	$ 160,000	$ 750,000				
S.10	Process Building - Blower Room - Air Intake Filters				$ 3,000	$ 10,000		
S.11	Addition of 3rd MBR Cassette, Piping, Controls, EI&C, In-House	✓		$ 800,000				
S.12	Replace / Improve Trains 1 & 2 MBR Modules, In-House *Assumed 3% inflation over 9 years (Repair & Maintenance)*					$ 1,290,000		
S.13	Operations Building							$ 1,240,000
	SUBTOTAL		**$**	**2,158,300**	**$**	**1,303,000**	**$**	**1,240,000**
	GRANT OPPORTUNITY PROJECTS							
L.1	Influent Forcemain Twinning - M.R. Pump Station to WWTP *Incl. power, water supply, fibre and spare conduit* *Costs could be lower if incorporated with the Dyke Upgrade contract*	✓	$ 210,000	$ 1,000,000				
L.2	Plant Electrical Upgrades		$ 80,000	$ 373,000				
L.3	New Outfall				$ 80,000	$ 334,000		
L.4	Investigate New Biological Treatment Options - *Allowance* *Capital Cost TBD based on current technologies*						$ 100,000	TBA
L.5	Investigate Desludging and Repair of Existing Bioreactor for EQ Tank Use - *Allowance* *Costs TBD based on extent of reactor reused and current construction methodologies. Combine with New Biological Treatment Options Investigation*	✓					$ 50,000	TBA
L.6	Headworks Upgrades (Screen and Grit Removal)						$ 230,000	$ 1,100,000
	SUBTOTAL		**$**	**1,663,000**	**$**	**414,000**	**$**	**1,480,000**
	TOTAL ESTIMATED COSTS PER TIME FRAME		**$**	**3,821,300**	**$**	**1,717,000**	**$**	**2,720,000**
	FURTHER INVESTIGATION REQ'D							
A.1	New UV Disinfection System *Confirm new permit requirments*							
	Assumptions (See Report for more details)							
	Inflation: Inflation is not included. Suggest using 3% / year after 2025 or actual value, if known.							
	Engineering: 15% for projects which require engineering. In-house projects do not include engineerig							
	Class D Estimate: 40% contingency to be added to all projects							

Contents

LIST OF FIGURES

LIST OF TABLES

1. Introduction

As part of their objective of maintaining and improving their wastewater treatment infrastructure both now and for the future, the Village of Harrison Hot Springs (VHHS) has commissioned McElhanney to prepare a Master Plan for their Wastewater Treatment Plant (WWTP).

In discussions, VHHS has expressed its commitment to assessing the resiliency of the existing wastewater treatment infrastructure and prioritizing future upgrades. Numerous upgrades for the long-term betterment of the Wastewater Water Treatment Plant were identified by VHHS with a focus on the optimizing the process, addressing power supply issues, upgrading the centrifuge and evaluating the hydraulic capacity of the outfall.

This Master Plan is intended to be a roadmap to address existing WWTP concerns and future treatment capacity requirements. The Master plan includes a prioritized project list, suggested implementation schedule and Class D cost estimate for each project, based on the implementation date. Key recommendations or findings are highlighted in teal text throughout for ease of reference.

In addition, this document may be used in discussions with the BC Ministry of Environment and Climate Change Strategy (MoECCS) for an amendment to VHHS's waste discharge permit.

2. Background

2.1. SYSTEM OVERVIEW

The original WWTP was built circa 1978 and included off-site lift station No. #1, an equalization basin and outfall to the lake. In 2008, upgrades included the addition of an activated sludge system and related sludge processing components.

A major upgrade was undertaken in 2012, including the addition of a Membrane Bioreactor (MBR) and UV disinfection system, a centrifuge and all ancillary systems and components. The resulting treatment process consists of the major process elements listed below, in addition to ancillary systems such as compressed air:

- Off-Site Lift Station #1
- Aerated Bioreactor and transfer pumps
- 2mm drum screen
- Two MBR trains (four cassettes) plus allowance for a 3rd train (2 additional cassettes)
- Permeate holding tank for backwashing
- UV System
- Final effluent flow meter
- Gravity outfall to Harrison Lake
- Waste activated sludge (WAS) tanks
- Centrifuge
- Aerations Blowers
- Chemical systems (citric acid, sodium hydroxide, alum, polymer, sodium hypochlorite)

Treatment plant performance is reviewed in Section 3.0 as the performance will help to inform the upgrade or retrofit recommendations. A more detailed description and assessment of the treatment components is discussed in Section 4.0.

2.2. SITE VISITS

On February 26, 2025, McElhanney's Christina Saxvik met with Tyler Simmonds and conducted a site visit to discuss current operational concerns and in preparation of the RFQI.

A second site visit was conducted by Karen Sutherland and Michael Barnes April 24, 2025. The aim of this visit was to review the current system condition, obtain additional staff input, and collect any additional information such as drawings not provided during the scoping phase.

2.3. REFERENCE INFORMATION

The following documentation has been made available to the McElhanney team to complete this assessment:

- Permit PE-116 (March 12, 1965) amended April 3, 1970
- Population, Base Sanitary Flows and Base Water Demands, Technical Memo, (Water Street Engineering, June 17, 2025)

- National Hydrological Service (NHS) of Canada, Lake elevations for the Harrison Lake site (ID# - 08MG012)[1]
- MBR Wastewater Treatment Upgrade Investigations for the Village of Harrison Hot Springs (AWC Water Solutions, 2019)
- Village of Harrison Hot Springs - Liquid Waste Management Plan (CTQ, December 2016)
- Federal Wastewater Systems Effluent Regulations (WSER, 2015)
- BC Municipal Wastewater Regulation (MWR, 2012)
- Harrison Hot Springs - Operational, Equipment and Maintenance (O&M) Manuals (2012)
- Harrison Hot Springs WWTP – Phase 1 - As-Built Drawings (Wilson Contractor Set, 2011)
- Village of Harrison Hot Springs Sewer Regulation Bylaw No. 980, 2011 (Consolidated)
- Harrison Hot Springs Wastewater Treatment Plant Feasibility Study (Dayton & Knight Ltd., 2008)
- Sewage Treatment Plant Environmental Study (Dayton & Knight Ltd., 2003)
- VHHS MBR Upgrade Report (AWC, 2019)
- Technical Memo – Population, Base Sanitary Flows, and Base Water Demands (Water Street Engineering, 2025)

[1] https://wateroffice.ec.gc.ca/report/real_time_e.html?stn=08MG012

3. Design Criteria

The regulations and design criteria applicable to the WWTP are discussed in the following sections. Current and projected flows and loads over the span of the WWTP Master Plan are included to establish design criteria.

3.1. HISTORICAL POPULATIONS AND PROJECTED GROWTH

Typically, wastewater flow projections are based on historical populations, historical per capita wastewater production rates during dry weather (litres per capita per day - lpcd), and projected community growth rate over a 20-to-25-year time frame.

The historical population data from Statistics Canada and Census data are presented in Table 1 and Table 2, respectively, for reference.

Table 1. Historical Populations and Average Annual Growth Rate Between Census

Source / Year	2011	2016	2020	2021	2022	2023	2024
Statistics Canada (refer to image below)			1889	1975	1999	2038	2051
Avg. Annual Growth (%)				*4.6%*	*1.2%*	*2.0%*	*0.6%*
Census	1248	1468		1905			
Avg. Annual Growth (%)		*3.5%*		*6.0%*			

Table 2. Statistics Canada Population Estimates in the Fraser Valley (Statistics Canada, 2024)

Statistics Canada's newly released data include estimates for 2024 and revisions to the previous year's figures. Those revisions reveal that last year Statistics Canada significantly overestimated population growth in Chilliwack while undercounting new arrivals in Abbotsford. Population growth in Mission was also lower than expected. In Langley, meanwhile, the stats agency confirmed its previous estimates, and determined that more people had moved to the community than it had previously estimated.

More people: 2024.

Population: Fraser Valley municipalities

Place	2020	2021	2022	2023	2024*
Abbotsford	159,139	160,759	164,237	169,440	175,087
Langley Township	135,885	138,426	146,455	154,833	162,269
Chilliwack	95,818	97,222	98,350	100,203	102,306
Mission	42,906	43,340	43,652	45,230	46,226
Langley City	29,510	30,333	32,162	33,620	35,316
Kent	6,616	6,540	7,040	6,958	7,115
Hope	6,747	6,911	6,950	6,736	6,842
Harrison Hot Springs	1,889	1,975	1,999	2,038	2,051

**2024 populations are estimated as of July 1 and subject to revisions*

Based on this information, the base population has been increasing over time and theoretically, a community growth rate could be projected. The exodus from large centres to smaller communities during COVID is evident in the VHHS growth around this time, which skews long-term analysis. Since COVID, community growth has dropped significantly. But the base resident population is only one consideration in estimating flow rates in VHHS.

The influx of seasonal tourists is very high due in part to VHHS' proximity to a Metro Vancouver and the Lower Mainland. Thus, the following is a broad list of categories of residents and visitors, each having different water demands and resulting wastewater production:

- Year-round residents
- Seasonal residents
- Hotel / motel guests – laundry services
- Campers – full RV setups, with some sites having pools and showers
- Day tourists

This seasonal influx is highest in the summer during the dry weather and when combined with the activities catering to tourists such as restaurants, estimating the wastewater generation in lpcd during dry weather flows based on population becomes very difficult.

Further complicating the analysis, are the high inflow and infiltration (I&I) rates during the winter months. As the piping system is reportedly very old with some sections in better condition than others, the I&I rates are higher than typical. Consequently, the impact of I&I likely lasts beyond the actual rain event.

Lastly, the available flow measurement at the WWTP is located on the effluent line, downstream of the buffering provided by the bioreactor, which provides a retention time measured in days, rather than hours. To estimate the plant influent flows, the run times of the pumps that were replaced in April 2025 would have to be analyzed in detail. However, as the pumps were just installed, there is insufficient data to confirm annual trends.

As Water Street Engineering was in the midst of conducting a detailed analysis of the wastewater flow generation and projected flows for the Collection System Master Plan (CSMP) when this report was being prepared, re-iterating this work was not deemed beneficial for VHHS. Rather, once the CSMP is complete, the results will be used as a cross check to the project treatment capacity recommendations in this WWTP Master Plan.

In Summary: Rather than trying to estimate projected population and tourist growth individually, while considering the impact of I&I, the expansion of the system will be based on total flows to the plant. This is discussed in detail in Section 3.2.2 Flow Projections.

3.2. WASTEWATER FLOWS

3.2.1. Historical and Current Flows

Annual reports were provided from 2020 to March 2025, which includes effluent daily flows. As noted above, there is no influent flow data available for system.

The total effluent monthly flow and effluent average flow for each month are presented in Figure 3-1 and 3-2.

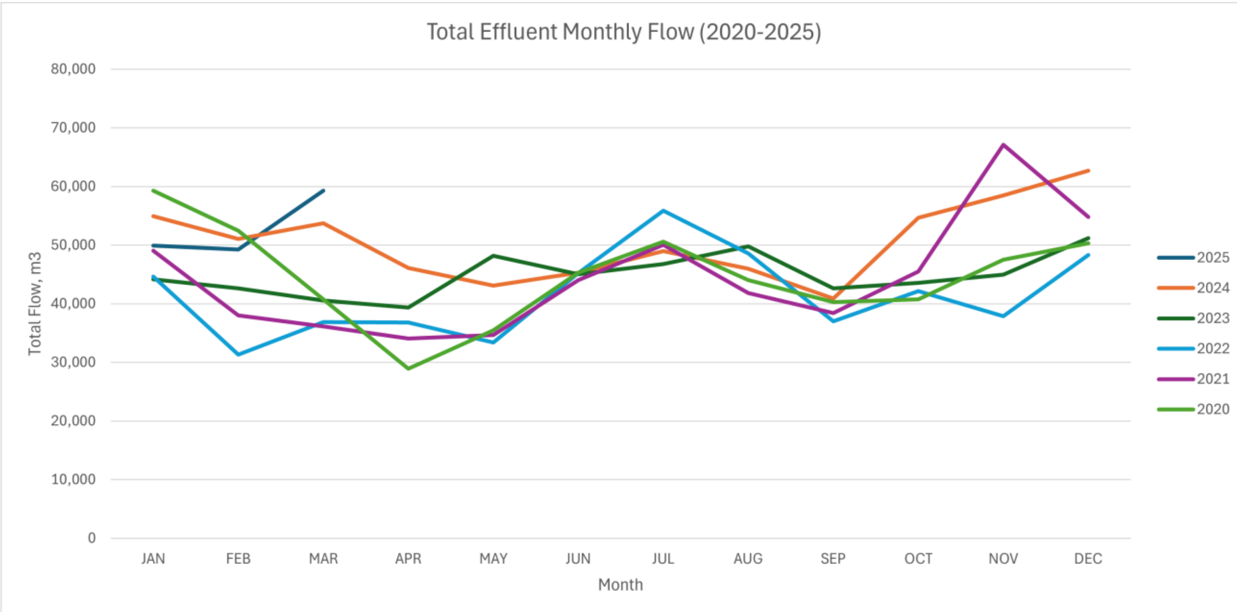


Figure 1 Total Effluent Monthly Flow 2020 to March 2025

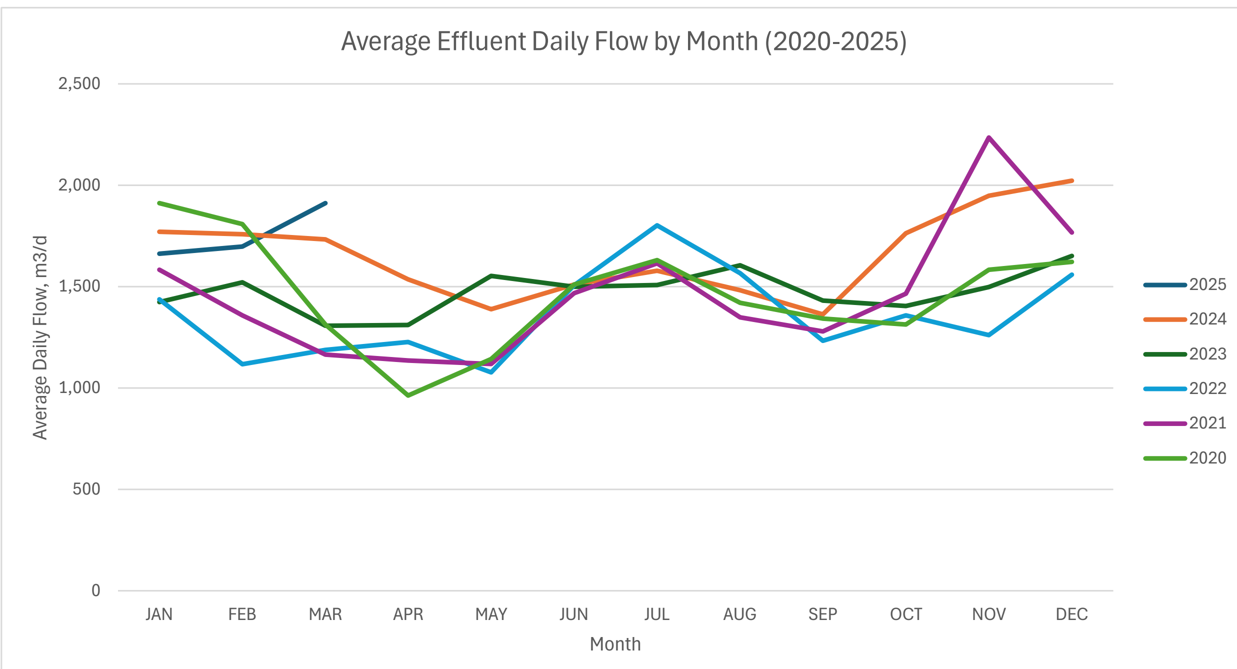


Figure 2 Average Effluent Flow by Month (2020 to March 2025)

Based on Figure 1, there is more variability in the winter months from, from approximately October to April, which corresponds with the wet months of the year. The total monthly flow is more consistent between June and September, which correspond to the dry months. There is also a peak in flow in July which corresponds with the peak of freshet, when the river rises to between 10.9 and 11m, and likely tourist season. The average effluent flow (Figure 2) which mirrors the total monthly flow, is provided for reference as the units are similar to those used to evaluate process and equipment capacity, i.e.m3/d.

The daily effluent flows are presented in Figure 3. A line indicating the 3,000 m3/d capacity of the existing MBR and current permit maximum discharge rate of 2,400 m3/d are shown for reference.

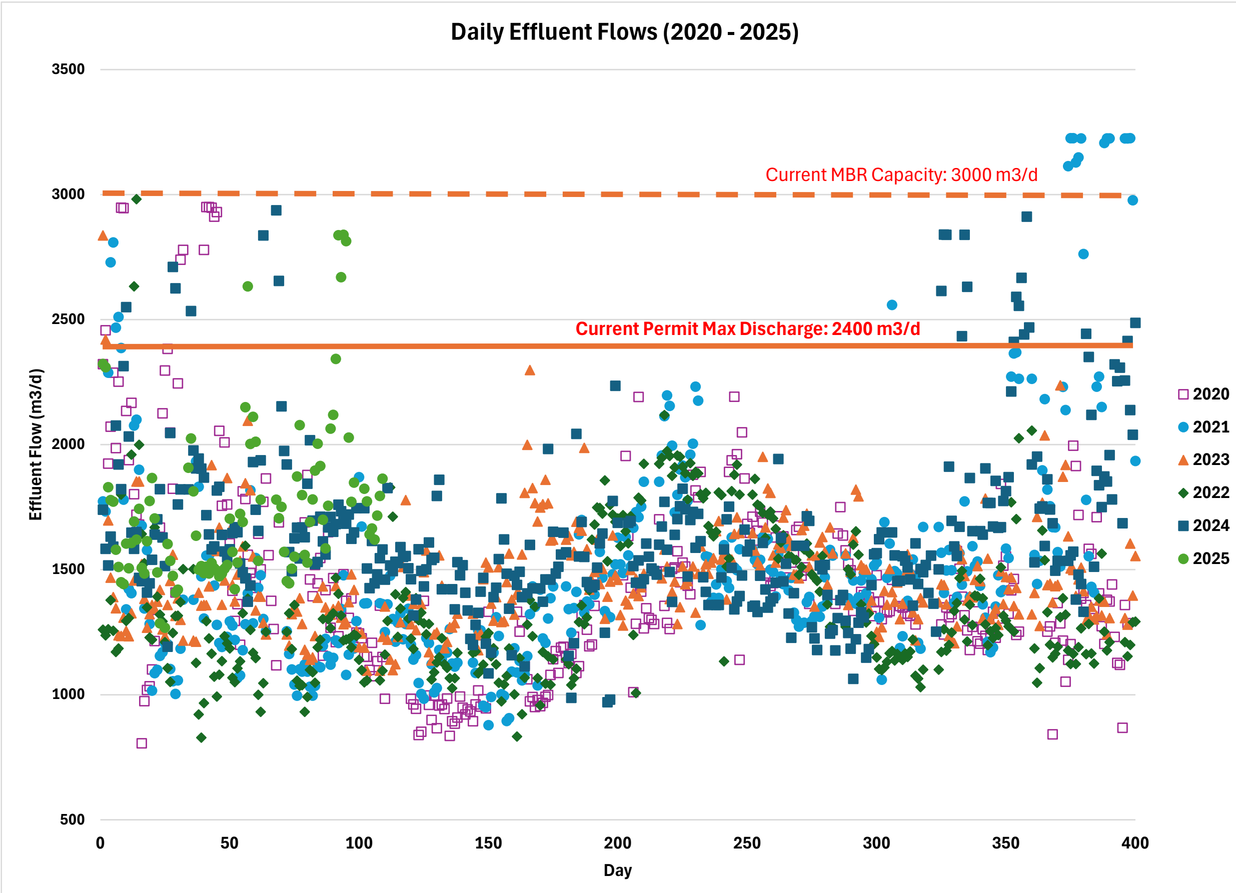


Figure 3. Daily Effluent Flows from 2020 to April 2025

As seen in Figure 3, during the winter months, the daily effluent flow is close to or exceeds the current rated capacity of the MBR system and the current permitted maximum discharge rate. For each permit exceedance since 2020, staff have prepared a report for the Ministry of Environment explaining the event and water quality parameters.

It is important to note that these are the effluent flows measured just before discharge into the outfall line. As the bioreactor provides several days of retention and thus buffering capacity, it is reasonable to assume that the influent flows are higher than presented in this graph. Hence the need to update the permit and allowable discharges.

The effluent annual minimum day, daily average, maximum day and total annual flow from 2020 to 2024 are presented in Table 3.

Table 3. Effluent Minimum, Annual Average and Maximum Day, and Total Annual Flow (2020 – 2024)

Annual Flow	2020	2021	2022	2023	2024	Average
Min Day, m3/d	806	440	809	1,096	970	825
Date	*Jan 15*	*Jul 17*	*Feb 3*	*Mar 31*	*Jun 26*	
Daily Average, m3/d	**1,463**	**1,462**	**1,364**	**1,476**	**1,660**	**1,485**
Max Day, m3/d	2,677	3,223	2,976	2,826	2,912	2,922
Date	*Nov 14*	*Dec 2*	*Jan 13*	*Dec 5*	*Nov 18*	
Total Annual Flow, m3	**535,534**	**533,575**	**497,827**	**538,588**	**605,790**	**542,263**

The annual daily average for the five years was 1,485 m3/day with a drop in 2022 due to lower winter flows, and a noticeable spike in 2024 with flows consistently higher all year long.

The maximum day flows all occurred during the winter rainy season, between mid-November and mid-January, with the highest maximum day recorded in 2021.

Interestingly, the recorded minimum day flows occurred between January 15 and July 17, although the low flow in 2021 was likely due to a low tourist season in the middle of COVID.

Staff noted the I&I component of the incoming wastewater is very high, with wet flows up to three times higher than high flows during the tourist season. These large dilute flows can impact biological treatment processes and will need to be considered in proposed upgrade options as the existing over-sized bioreactor attenuates flows, thereby reducing the impact on the MBR.

3.2.2. Flow Projections

The most recent plant upgrade was the addition of the MBR system in 2012, which increased the installed treatment capacity to 3,000 m3/d, with an allowance for a 3rd MBR tank. Assuming an equivalent MBR module was installed, the plant capacity would be 4,500 m3/d. Plant components were assessed based on this flow rate. Just before submitting the final report, manufacturers advise of a newer slightly more expensive MBR module with a capacity of 1,800 m/3/d. **Installing the newer MBR modules in the 3rd tank would provide an MBR installed capacity of 4,800 m3/d; other components would have to be re-assessed and upgraded accordingly.**

Based on the current maximum day effluent flows already exceeding the installed 3,000 m3/d capacity of the MBR system, and if system modifications include any reduction in buffering capacity currently provided by the oversized bioreactor, it reasonable to assume that the capacity of the treatment system should be increased.

As it is anticipated that while the population and number of visitors will continue to increase, VHHS will simultaneously work on reducing I&I. Consequently, projected flow rates and corresponding timelines are more difficult to estimate. However, increasing the treatment system capacity to 4,800 m3/d should provide sufficient treatment capacity for the foreseeable future.

Different areas of the treatment plant are sized based on different flow rates due to retention times. For example, the headworks screens must be sized based on the peak wet weather flows as retention time is measured in seconds. However, other processes will be based on maximum day and maximum month as the retention time is measured in hours, which is the case for the biological treatment and MBRs.

The average max. day flow over the last five years was 2992 m3/d, approximately 3,000 m3/d. This is also the rated capacity of the existing MBR system.

Growth scenarios: Assuming upstream buffering to attenuate peak flows is provided, low (1%), medium (2%), high (3%) and extremely high (5%) constant growth rate scenarios were assumed to determine how many years until a flow rate of approximately 4,500 m3/d would be reached, which is the equivalent of a 3rd MBR train equivalent to the two original trains.

I&I Scenarios: These four growth scenarios were considered based on two I&I scenarios:

- **Maintain existing conditions**: Minimal upgrades to the existing collection system, resulting in continued high I&I
- **Reduced I&I**: Over time, I&I would be reduced so that the result would be an equivalent Max Day Flow of 2,700 m3/d **today**. Note that this is approximately the current Avg Annual Flow of 1,485 m3/d * 1.85.

Flow Scenarios and approximate number of years to reach 4,500 m3/d based on population growth rates are presented in Table 4.

Table 4. Flow Scenarios and Approximate Number of Years to reach 4500 m3/d

	Current Max Day Flow (m3/d)	Assumed Annual Growth Rate	Number of Years to Reach Target Max Day Flow	Future Max Day Flow (m3/d)
MBR Max Day Capacity	**3000**			**4500**
Current Max Day Flow Based on the Current System with High I&I				
Scenario 1a - Low Growth	3000	1%	40	4467
Scenario 2a - Med Growth	3000	2%	20	4458
Scenario 3a - High Growth	3000	3%	14	4538
Scenario 4a - Extreme Growth	3000	5%	8	4432
Long-Term Reduced Max Day Flow Assuming I&I Reductions Achieved				
Scenario 1b - Low Growth	2700	1%	51	4485
Scenario 2b - Med Growth	2700	2%	26	4518
Scenario 3b - High Growth	2700	3%	17	4463
Scenario 4b - Extreme Growth	2700	5%	10.5	4507

Communities can experience population growth rates of up to 10%; however, these growth rates are not typically sustained. And although VHHS has seen high resident population growth rates in the past, it is unlikely that VHHS would experience a sustained population and tourist growth of 5% over 15 to 20 years.

Assuming the high growth case scenario of 3% average growth, increasing the treatment capacity to 4500 m3/d would result in the system capacity being adequate for the next 14 to 17 years, depending on how well I&I can be addressed.

Recommendation: It is recommended that influent flows being logged by the new flow meter be analyzed to confirm estimated design parameters, and so that impacts of I&I reduction strategies can also be assessed and monitored. At the time of issuing this report, this process has been started, and will be continued, in-house by operations staff.

In the meantime, and for the purposes of this report, the current effluent and proposed future influent design flow rates are listed in Table 5 below.

Table 5. Current Effluent and Proposed Future Influent Flows Rates

	Current Effluent m3/d	Future Influent m3/d
Average Day Flow	1,485	2,250
Maximum Day Flow	2,922	4,500
Peak Wet Weather Flow	3,200 (buffered)	9,000

3.3. WASTEWATER LOADS

The influent wastewater characteristics and typical average municipal wastewater[2] concentrations are presented in Table 6

Table 6. Annual Influent Wastewater Data

PARAMETER	Typical municipal Range (mg/l)	Annual minimum (mg/l)	Average (mg/L)			Annual maximum (mg/l)
			Avg. Winter	Avg. Annual	Avg. Summer	
Biochemical Oxygen Demand (BOD_5)	250 - 300	17.6	69.7	71.8	73.8	150
Total Suspended Solids (TSS)	250 - 300	3.3	68.8	70.2	71.5	304
Total Kjeldahl Nitrogen (TKN)	50 - 60	---	---	---	---	---
Ammonia Nitrogen	35 - 40	4.2	13.2	15.7	18.3	28.7
Total Phosphorus	≤ 10	0.8	2.2	2.6	2.9	4.9
Grease	≤ 100	---	---	---	---	---
pH						

The annual influent wastewater concentrations are generally much lower than average municipal wastewater concentrations.

[2] Metcalfe and Eddy, 6th edition

Also shown are the average winter (October – March) and summer (April – September) influent data to determine if there are significant changes in the influent throughout the year, as generally, wet winter flows contribute to a more diluted influent and therefore lower BOD, TSS, Nitrogen, and Phosphorus.

However, both average winter and summer are similar to the annual average and are still lower than industry average municipal wastewater concentrations. This may be indicative of I&I throughout the year. These numbers may change once I&I reduction strategies are implemented.

3.4. REGULATORY REQUIREMENTS

3.4.1. Overview

In Canada, municipal wastewater treatment facilities and effluent discharges are regulated by Federal, Provincial/Territorial, Municipal, and Regional governments. Federal and Provincial, governments are responsible for regulating the wastewater treatment facility operations and setting wastewater effluent discharge limits. Municipalities and Regional Districts are responsible for the operation of municipal wastewater treatment facilities. Local governments also regulate the substances that are discharged into the sewer system through sewer use bylaws.

The average daily flow for this facility is approximately 1,485 m^3/d and falls within the *Municipal Wastewater Regulation (MWR, BC COR 2012)* guidelines.

In summary, the regulations and standards that may have bearing on the discharge of treated effluent to the receiving environment from the Study Area are:

- *Federal Wastewater Systems Effluent Regulations (2015) (WSER)*
- *BC Municipal Wastewater Regulation (MWR, 2012).*

These regulations are discussed in more detail hereunder.

3.4.2. Federal Wastewater Systems Effluent Regulations

The Federal *Wastewater Systems Effluent Regulations (2015)* (WSER) under the *Fisheries Act*, establishes minimum effluent criteria for all discharges to water bodies with average flows greater than 100 m^3/day. The WSER does not apply to ground discharges and discharges to water bodies with average flows less than 100 m^3/day. Part 1 – Authorization to Deposit of the WSER outlines the effluent requirements for discharges.

3.4.3. BC Municipal Wastewater Regulation

In BC, the *Municipal Wastewater Regulation* (MWR) authorized under the *Environmental Management Act*, is a comprehensive regulation that governs all aspects of municipal wastewater management. The BC MWR has set minimum standards for effluent discharges exceeding 22.7 m^3/day into different receiving bodies (environment), as well as minimum requirements for STP quality monitoring, design and construction standards, and management and operations of STPs. MWR specifies that domestic wastewater discharges to ground or surface waters shall receive, as a minimum, primary and secondary treatment, respectively.

However, the level of treatment is dependent on the method and location of the effluent discharge, and on the findings of an Environmental Impact Study (EIS). An EIS is typically undertaken to determine if a more stringent effluent quality is required.

Possible environmental options for disposal include marine, lake, creek, groundwater, and reclaimed water use for various applications. Part 6 of the BC MWR specifies minimum requirements for effluent quality criteria for lake discharges.

3.4.4. Effluent Quality Criteria- Current Permit and Future Requirements

The effluent criteria stipulated in the current 2008 Permit (PE-116) dating from before the 2012 WWTP upgrade, the Federal WSER and Part 6 of the BC MWR are outlined in Table 7 below.

Since the 2012 upgrade, the Operations Staff have been running the treatment plant based on meeting the WSER and MWR limits. The amendment of the Permit to account for the upgrades was not completed due to staff turnover.

Table 7. Effluent Quality Criteria for Discharge to Lake

Parameter	Existing 2008 Permit PE-116	WSER Annual Avg. limit if daily value >2500 m3/d	MWR Lake Discharge Max Daily Flow >50 m3/d and Daily Flows < 2x ADWF
Maximum Discharge Rate, m3/d	2,400		
Annual Avg. Discharge Rate, m3/d			
$cBOD_5$ (mg/L) (Note 1)	≤45	≤25 avg..	≤45 (note 4)
TSS (mg/L) (Note 2)	≤45	≤25 avg.	≤45 (note 4)
Total phosphorus (mg/L)			≤1
Ortho phosphate (mg/L)			≤0.5
Un-ionized ammonia (mg/L) (Note 3)		≤1.25	
Total residual chlorine (mg/L)		≤0.02	
pH			6-9
Fecal Coliform, MPN / 100 mL			<200 (note 5)

Notes:
Note 1: cBOD – Carbonaceous Biochemical Oxygen Demand
Note 2: TSS – Total Suspended Solids
Note 3: expressed as nitrogen (N), at 15˚C ± 1˚C,
Note 4: For daily flows ≥ 2x ADWF, interim BOD5 and TSS is ≤130 mg/L
Note 5: limit if discharging to recreational use waters, at the edge of the dilution zone.

As observed in Figure 3 above, the maximum permitted discharge rate of 2,400 m3/d is exceeded regularly during the winter months, and thus the WSER and MWR are applicable.

As the average dry weather flows occur during the summer tourist season, the maximum daily flows which occur during the winter rainy season remain less than two times ADWF, slightly obscuring the high I&I rates in the data analysis.

Recommendation: The existing permit must be amended as current flow are exceeding the discharge permit, and the treatment process has changed significantly.

3.4.4.1. Disinfection

Division 2 – Section 52 (1) of the MWR states: A discharger must disinfect municipal effluent if necessary to ensure that receiving water or groundwater used for domestic or agricultural water extraction, recreational uses or aquatic food production meets water quality guidelines.

As the wastewater is discharged into a lake, disinfection would be required as a back-up in the event that the MBR is not working, unless authorization from a director is obtained that disinfection is not required.

Based on recent report, included in Appendix D, there are numerous documented cases of Canadian and American municipalities no longer requiring disinfection after MBRs. The need for disinfection should be discussed and confirmed with BC Ministry of Environment and Climate Change Strategy (MoECCS).

3.4.4.2. Reclaimed Water

Given the distance of the WWTP to the closest potable water source, approximately 900 m away, treatment requirements for water re-use are included herein, as reclaimed water could potentially displace potable water demands on site.

In addition, as a dyke will be constructed along the shoreline from the WWTP to the town, there may be an opportunity to run a water reuse line back to town with minimal additional cost to the construction of the dyke.

Based on the current regulations, reuse water must be disinfected, in addition to meeting the criteria outlined in Table 8 (Table 13 of the MWR[3]), based on the reuse scenario and exposure potential.

Table 8. Municipal Effluent Quality Requirements for Reclaimed Water (MWR - Table 13)

Table 13 — Municipal Effluent Quality Requirements For Reclaimed Water

Parameters	Indirect Potable Reuse	Greater Exposure Potential	Moderate Exposure Potential	Lower Exposure Potential
pH	site specific	6.5 to 9	6.5 to 9	6.5 to 9
$CBOD_5$, TSS	$CBOD_5$ 5 mg/L TSS < 5 mg/L	10 mg/L	25 mg/L	45 mg/L
turbidity	maximum 1 NTU	average 2 NTU, maximum 5 NTU	n/a	n/a
fecal coliform (/100 mL)	median < 1 CFU or < 2.2 MPN; maximum 14 CFU	median < 1 CFU or < 2.2 MPN; maximum 14 CFU	median 100 CFU; maximum 400 CFU	median 200 CFU; maximum 1 000 CFU

Once the reuse scenario has been confirmed, any additional treatment requirements can be established. At a minimum, a hypochlorite dosing system to maintain chlorine residuals in distribution piping would be anticipated.

[3] https://www.bclaws.gov.bc.ca/civix/document/id/complete/statreg/87_2012

3.4.5. Reliability Requirements

The following section provides brief explanations and results of the design aspects that will be considered when assessing the WWTP.

MRW component reliability requirements are laid out in Part 3 – General Design and Construction Requirements, Division 1 – Table 1 of Section 35 (Table 9 below).

Table 9. Component and Reliability Requirements for WWT Facilities (MWR, Part 3, Div. 1-35 – Table 1)

	Reliability Category					
	I		II		III	
Components	**Treatment System**	**Power Source**	**Treatment System**	**Power Source**	**Treatment System**	**Power Source**
blowers or mechanical aerators	multiple units	yes	multiple units	optional	2 minimum	no
aeration basins	multiple units[b]	yes	multiple units[b]	optional	single unit	no
disinfection basins	multiple units[b]	yes	multiple units[a]	yes	multiple units[a]	no
trickling filters	multiple units[b]	yes	multiple units[b]	optional	no backup	no
primary sedimentation	multiple units[a]	yes	multiple units[a]	yes	2 minimum[a]	yes
chemical sedimentation	multiple units[b]	optional	no backup	optional	no backup	no
final sedimentation	multiple units[b]	yes	multiple units[a]	optional	2 minimum[a]	no
degritting	n/a	optional	n/a	no	n/a	no
chemical flash mixer	2 minimum or backup	optional	no backup	optional	no backup	no
flocculation	2 minimum[a]	optional	no backup	optional	no backup	no
aerobic digesters	2 minimum[a]	yes	2 minimum[a]	optional	single unit	no
anaerobic digesters	2 minimum[a]	yes	2 minimum[a]	optional	2 minimum	no
effluent filters	2 minimum[b]	yes	2 minimum[b]	yes	2 minimum[b]	yes
facultative lagoons	2 cells[b]	n/a	2 cells	n/a	2 cells	n/a
aerated lagoons	2 cells[b]	yes	2 cells	optional	2 cells	no
package treatment plants	multiple unitsb or ability to repair within 48 hours	yes	2 units or ability to repair within 48 hours	yes	single unit	no

Notes: The remaining capacity with the largest unit out of service must be at least

- 50% of the design maximum flow where the notation "a" appears, or
- 75% of the design maximum flow where the "b" appears

For the purposes of this assessment, it was assumed that this facility falls into a reliability category I. A facility in reliability category I is defined as[4]:

"wastewater facilities

(i) that discharge to ground or water, and

(ii) *in respect of which short term effluent degradation could cause permanent or unacceptable damage to the receiving environment, including discharges near drinking water sources, shellfish waters or recreational waters in which direct human contact occurs*."

Although most components have multiple units, there is only one bioreactor, which does not meet the reliability requirements of the MWR.

[4] https://www.bclaws.gov.bc.ca/civix/document/id/complete/statreg/87_2012#division_d2e2922

3.5. TREATED EFFLUENT COMPLIANCE ASSESSMENT

Monthly effluent quality data was provided from 2020 to 2025.

Although the Permit was not updated after the 2012 upgrade to reflect the new process and capacity and integrate effluent quality limits based on the WSER and MWR, VHHS has been sampling based on the effluent quality limits that would likely be applicable for a new permit.

The maximum recorded, average and minimum recorded test results over the five-year period are summarized in Table 10. The permit, WSER and MWR limits are provided for reference at the top.

Table 10. Reported Effluent Quality from 2020 to 2024: Typical Range and Exceedances Lake

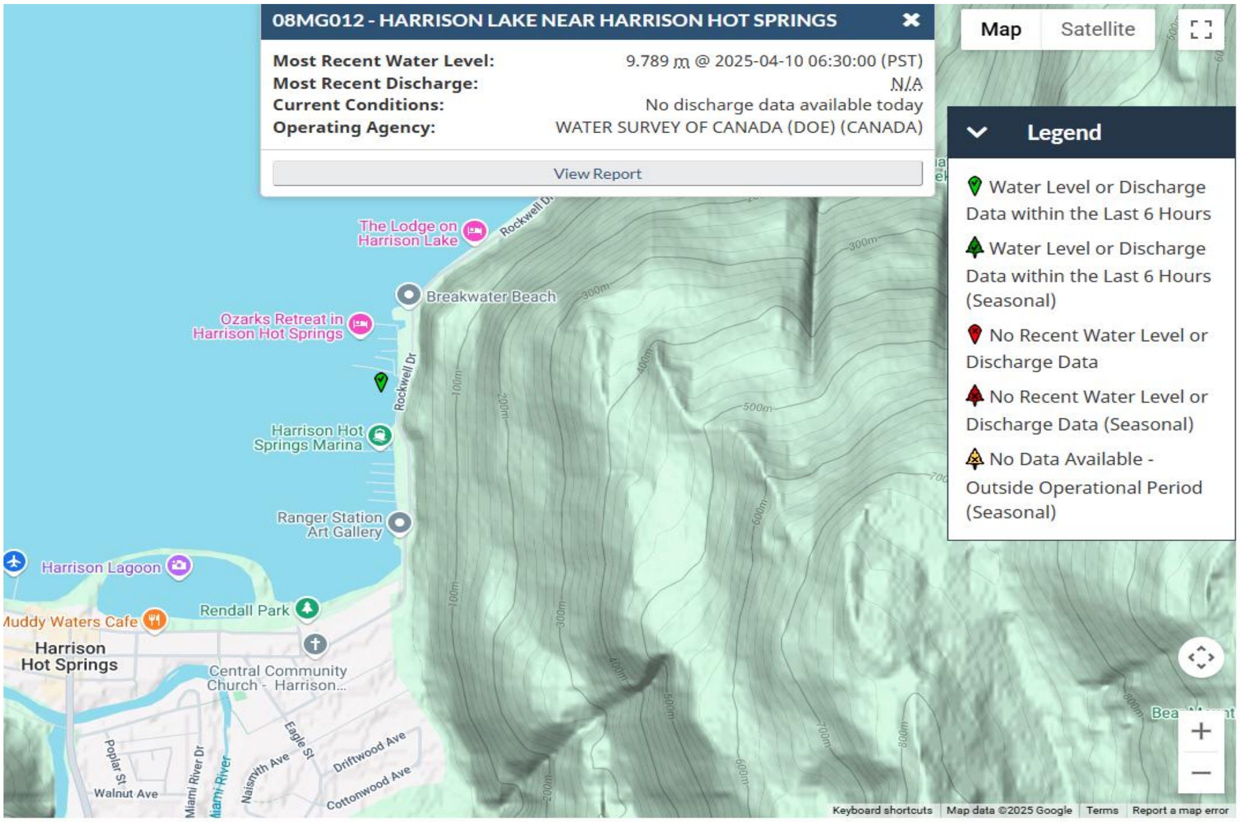
Date	CBOD, (mg/L)	TSS, (mg/L)	Un-ionized ammonia NH3-N (mg/L)	Orthophosphate PO4-P (mg/L)	Total P (mg/L)	Fecal Coliform MPN/100	Toxicity Test (Pass/Fail)
Permit 116 Effluent Limit	**45**	**45**					
WSER Limit	**25**	**25**	**1.25**				
MWR Limit	**45**	**45**		**0.5**	**1**	**200**	
Effluent Data							
Max. effluent concentration	2.4	11.9	1.1	0.44	0.45	<1	No Fails
Avg. effluent concentration	2.01	3.2	0.16	0.19	0.21	<1	60 Passes
Min. effluent concentration	<2	<2	0.007	0.033	0.044	<1	

All parameters were consistently below the Permit, WSER and MWR limits.

3.6. LAKE ELEVATIONS

Historical and real time lake levels can be obtained through the National Hydrological Service (NHS) of Canada. Data from the Harrison Lake site (ID# - 08MG012)[5] located along the shore of Harrison Hot Springs (refer to Figure 4) was used to select a lake design level for outfall discharge design.

[5] https://wateroffice.ec.gc.ca/report/real_time_e.html?stn=08MG012



Displaying 1 stations.

Figure 4. Hydrometric Data Collection point for Harrison Lake (ID# 08MG012)

Daily data was downloaded from 1933 to April 2025 and analysed. The daily maximum, daily mean, daily median, and daily minimum are presented in Figure 5.

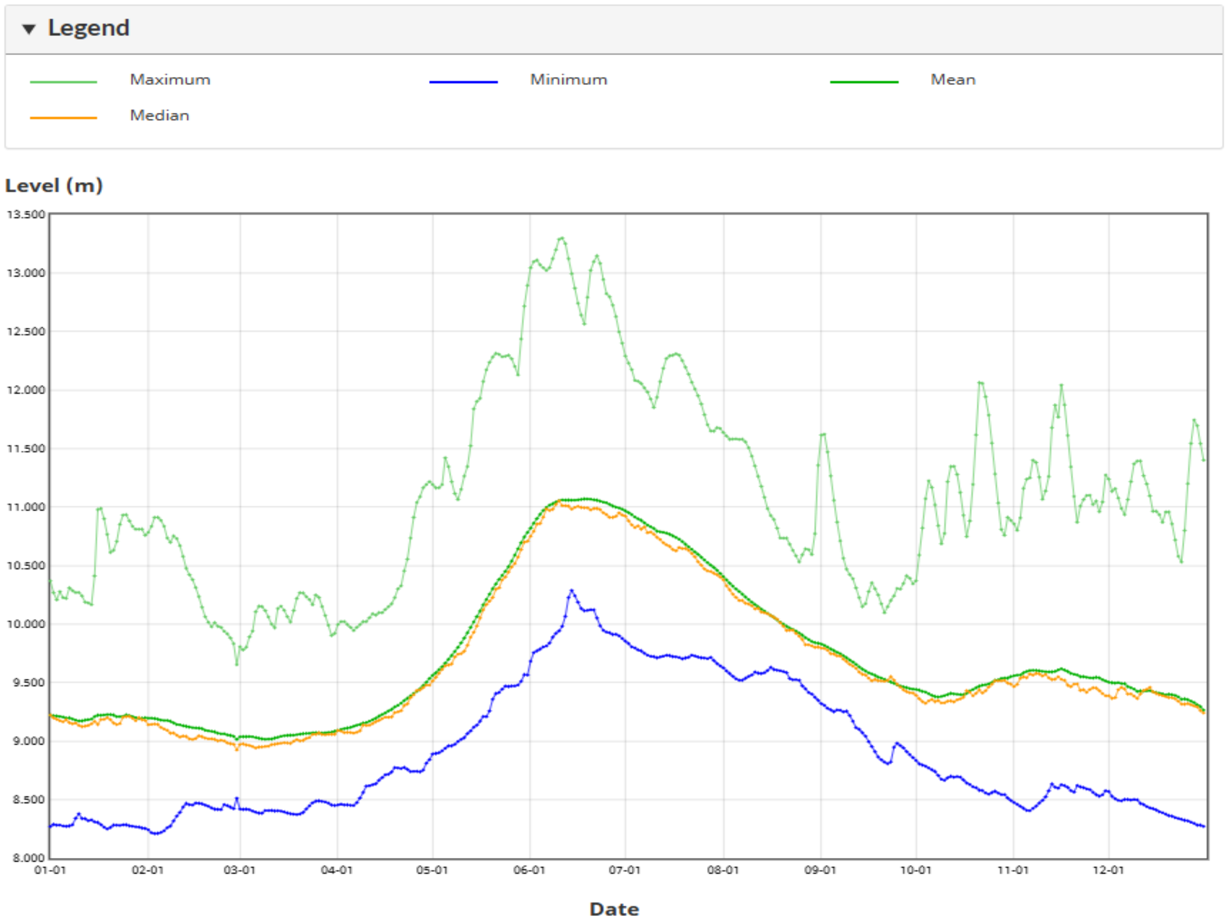


Figure 5. Daily Water Level Graph for Harrison Lake Near Harrison Hot Springs (08MG012)

Lake levels are highest during June, just after the spring snowmelt. Thereafter, lake levels drop during the dry summer season and start to rise again in the fall during the rainy season. During the coldest period, likely when there is little rain coming off the mountains, the lake level is at it's lowest.

As seen in Table 11, the 99th percentile of Harrison Lake's water level is significantly less than the Extreme case that occurred on June 11, 1948 and less than the 2012 1/5-Year design level.

Table 11. Daily Water Levels: Max, 99th & 98th Percentiles

Parameter	Water Level (m)
Maximum (Extreme) Occurred in 1948	13.298
1/5-Yr Design Level used in 2012	12.300
99th Percentile	11.903
98th Percentile	11.619

Statistically, 99% of the time lake levels should be below 11.903 m, which would be an adequate high-water level to consider in design of lake-side related infrastructure.

3.7. PERMITTING

Recently, VHHS staff applied for a minor amendment to their waste discharge permit under the Environmental Management Act to increase the capacity from 2,400 m3/d to 4,500 m3/d.

However, the BC MoECCS deemed that a major amendment would be required based on the proposed treatment system changes.

As this Master Plan is to include recommendations regarding a phased expansion to increase system capacity, this information will likely form part of the document submission for the amendment and ultimate the permit.

If a 3rd MBR is installed with a capacity of 1,800 m3/d, VHHS may want to revisit the requested future maximum discharge rate.

4. Existing System Assessment

Currently, the existing installed capacity of the treatment plant is 3,000 m3/d based on capacity of the membranes; the membranes were designed to allow for expansion to approximately 4,500 m3/d which exceeds the current Permit-116 maximum discharge flow of 2,400 m3/d.

However, individual components may have different capacity ratings based on time of construction.

The components of the existing system listed in Section 2.1 are described below with any deficiencies or recommendations listed in the relevant section and also consolidated at the end of Section 4 for ease of reference.

4.1. PLANT INFLUENT

4.1.1. Lift Station #1

Rehabilitation and upgrading of Lift Station (LS1) was completed in April 2025. The two pumps operate in a lead/lag configuration and have a duty point rating of 48 l/s at 28m total dynamic head (TDH) and static lift of 12.0 m, per the curves provided by Xylem for this application. Pump details are summarized in Table 11.

Table 12. Summary of Lift Station #1 Pumps

Equipment	Manufacturer	Configuration	Preliminary Duty Point	HP
Lift Station Pump	Flygt (Xylem) NP 3171 HT3-454	Lead / Lag	48.2 L/s @ 28m TDH	26.4 HP

Based on calculations by McElhanney, the bottom of the lift station is 6.0 m below the top of bioreactor tank at the WWTP, and the static lift based on operation levels and discharge elevation is approximately 5.2 m. Based on 3rd party review comment of this report, the pumps were originally designed to pump to the highest elevation in the plant to allow for modifying the discharge point in the future; this resulted in a static lift of 12.0 m and this elevation was carried through in the design and pump selection.

Condition: As the lift station and pumps are new as of April 2025, there are no condition concerns.

Capacity: Refer to the discussion at the end of section 4.1.2.

4.1.2. Influent Forcemain

The forcemain from the Lift Station to the WWTP consists of twin 200 mm dia. lines approximately 882 m long according to records.

- The first section of forcemain, constructed in 1994, is approximately 484 m of PVC. It runs through a parking lot and then under a walking path along the shoreline to the Miami River Pump Station.

- The section of the twinned forcemain from the Miami River Pump Station to the WWTP was upgraded in 2012 and is HDPE DR17, approximately 398 m long.

- The influent forcemain discharges into a channel equipped with a 5mm fine screen at the head of the plant. Refer to Figures 6 and 7.

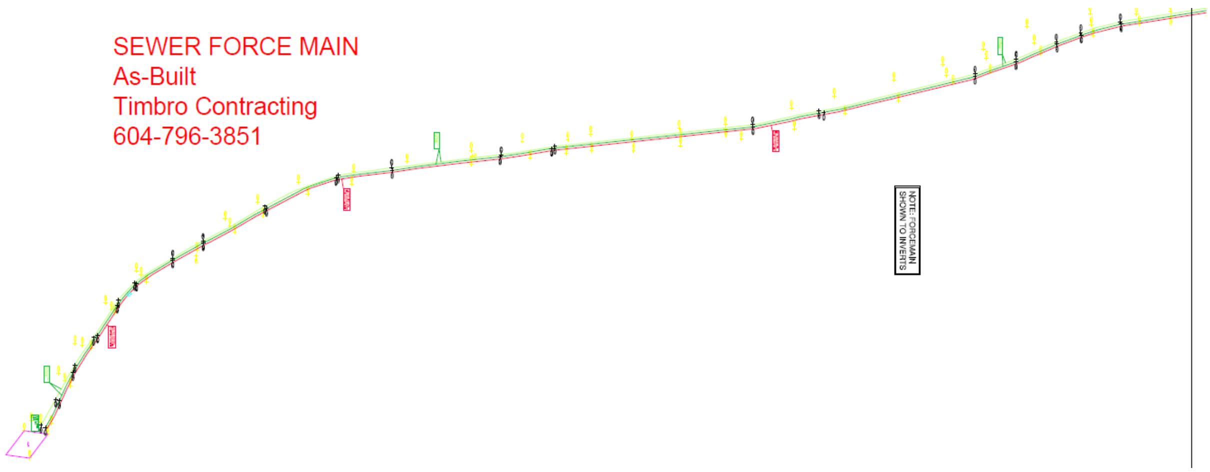


Figure 6. Record drawing of the Forcemain from Miami River Pump Station towards the WWTP

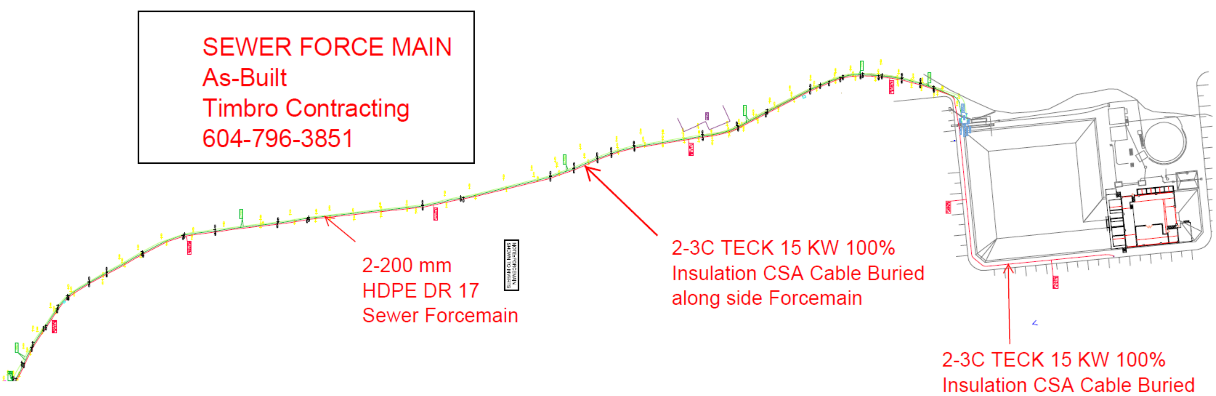


Figure 7. Record drawings of the Forcemain from the connection to the WWTP discharge point.

Condition comments

The first section of the twinned forcemain constructed in 1994 between the new lift station LS#1 and the Miami River Pump Station requires rehabilitation. In addition, VHHS would like to relocate the above power lines below ground, similar to the second section of forcemain piping up to the WWTP.

The second section of piping was upgraded in 2012, in conjunction with the WWTP upgrades. There are no concerns with the condition of this section of piping.

Figure 8. 2012 Forcemain Twinning from Miami River Pump Station to WWTP. a) Twinning along the pathway to the WWTP. b) Connection of twinned lines at the WWTP before the influent fine screen channel.

Plant Influent System Capacity

As the pumps were slightly oversized, when the first pump is called to run, the speed is limited to a flow rate of approximately 45 L/s to limit cycling of the pumps. If the pump runs for more than 300 seconds (5 minutes), the pump is ramped up to full speed, resulting in a flow rate of approximately 76 L/s, which typically draws down the well pump in the summer. In the winter, during the higher flow rates, the 2nd pump will start automatically. This scenario has not yet been tested as the pumps were installed spring 2025.

Operations staff ran the pumps based on different operating conditions to confirm the existing system capacity. Refer to Table 13 for a summary of the recorded flow rates.

Table 13. Measured Flows of Lift Station #1 Pumps in May 2025

Pump in Operation	Connecting Valve (Open/ Closed)	One Pump Reduced Flow, (L/s)	One Pump Max Flow (L/s)	Two Pumps Max Flow each Pump (L/s)
Pump 1	Open	43.8	76.3	48.5
Pump 2	Open	45.0	76.3	49.7
Total Avg Flow		**45**	**76.6**	**97.2**
Pump 1	Closed	29.8	47.3	48.5
Pump 2	Closed	31.6	48.5	49.7
Total Avg Flow		**30.7**	**47.9**	**97.2**
Influent Capacity				**8,398 m3/d**

Currently, the maximum influent pumping capacity is 97.2 l/s or 8,393 m3/d. By increasing the MBR capacity to 4,500 m3/d, based on Max. Day flow, the plant influent and headworks should be sized based on the PWWF which is approximately 2x Max Day. Thus, the influent pumping system should have a capacity of 9,000 m3/d. The current system is just at this estimated PWWF.

Recommendation: To further increase the influent capacity, the original sections of twinned pipe could be upsized to 250 mm. Based on the existing piping configuration, this would provide a theoretical flow rate of 123 L/s or 10,627 m3/d.

If the headworks equipment and piping are modified, including raising the discharge elevation of the influent piping, then the existing flow rate could be more easily be maintained.

4.2. INFLUENT FINE SCREEN

The influent wastewater discharges into a channel equipped with a Parksons 5mm Hycor Helisieve Model HLS400 which starts based on the liquid level upstream of the screen, as measured by a level sensor. The screened effluent is discharged directly into the southwest corner of the bioreactor.

The screenings are collected in a garbage bin. There is no wash water connected to this screen and there were no freezing concerns during winter operation.

Operations staff have added a short weir at the front of the channel to try to capture any larger solids. An overflow at the influent discharges directly into the bioreactor.

Capacity: The HLS 400 has a rated capacity of 1.8 MDG (8,183 m3/d) average day flow to 3.0 MGD (11,360 m3/d) peak flow. This exceeds the estimate future PWWF of 9,000 m3/d.

Condition: The channel appears to be in adequate condition. However, signs of corrosion, likely due to hydrogen sulphide (H_2S) are visible as the bioreactor railing which parallels the influent channel was heavily corroded up to the end of the channel and all the grating above the channel is corroded. See Figure 9.

The operation of the fine screen was acceptable. However, weekly manual de-ragging of the screen is required to reduce torque on the auger.

In addition, there is no grit removal system, consequently grit is being conveyed into the bioreactor and potentially through remainder of the treatment system via the bioreactor transfer pumps, accelerating the wear on pumps, equipment, in-line instruments, and piping.

Figure 9. Influent Channel with Helisieve. Corroded grating and bioreactor railing.

Recommendations:

- The grating and bioreactor handrailing should be replaced or recoated for safety reasons.
- Addition of a grit removal system should be considered to reduce wear on downstream equipment and solids settling in the bioreactor.

- Process optimization will influence timing and upgrades of the fine screen system, due to compatibility and layout considerations.

4.3. AERATED BIOREACTOR

Raw wastewater is discharged into the south-west corner of an aerated bioreactor.

The aerated bioreactor consists of a repurposed concrete equalization basin, whereby the leg of the L-shaped basin was filled in 2012 and is used as part of the process building.

The tank is now relatively square, with a design working volume 6,487 m3 and maximum 4.0m operating depth. The actual operating depth is maintained at approximately 3.6m which allows for buffering of the influent during the winter rain events. This equates to an operating volume of approximately 5,490 m3.

There are seven floating aeration laterals, each equipped with several EDI Flexair diffusers grids. Refer to Figure 10 for a typical aeration diffuser. Alternating duty/standby Delta process air rotary lobe blowers provide air to the aeration system. There is a stub for a 3rd blower.

Figure 10. Bioreactor Aeration Diffuser (typ.)

Effluent is transferred from the bioreactor to the fine screen drum and MBR tanks via the submersible Flygt model NP 3153.185-1513 submersible transfer pumps located in the north-east corner of the bioreactor. The layout includes for the addition of a 3rd transfer pump if required; piping stubs are in place.

Return activated sludge (RAS) from the MBR mixed liquor effluent channel is discharged in this Southwest corner of the bioreactor. Mixing of the influent stream and RAS relies on mixing achieved through the aeration system, which is minimal in this corner of the tank.

Sodium hydroxide (NaOH) for pH adjustment is discharged into the mixed liquor effluent manhole to provide contact time and mixing time before the mixed liquor (RAS) discharges into the bioreactor.

In the original design, hydrostatic relief valves were installed at the bottom of the bioreactor, but their current condition is unknown. Refer to Figure 11.

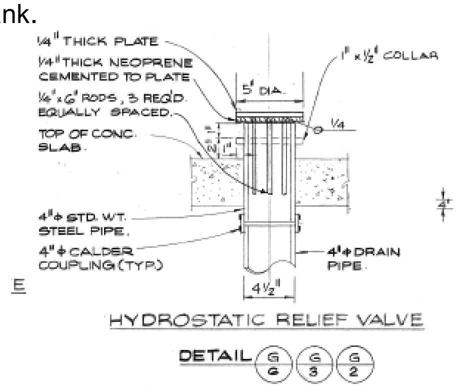


Figure 11. Original pressure relief valve at the bottom of the bioreactor.

In 2012, the bioreactors were only partially desludged during the construction of the new facility to allow for re-use and conversion of the north section of the bioreactor. Since then, sludge removal occurs through wasting of sludge settled in and removed from the MBR tanks. If bioreactor mixing is not adequate, there is likely a lot of sludge at the bottom of the tank. **As there is only one tank, the tank cannot be taken out of service for inspection or repair.**

The key components of the aerated bioreactor are summarized in Table 14.

Table 14. Summary of Bioreactor Pumps

Equipment	Manufacturer	Configuration	Preliminary Design Duty Point	HP
Transfer Pumps to 2mm Screen	Flygt Submersible Pump	Two duty / Room for 3rd pump	44.7 L/s @ 4.0m TDH	20.0 hP
Process Blowers	Delta Aerzen Positive Displacement	One duty / one standby	1,251 to 2,500 SCFM @ 7.5 PSIG	Installed: 100 hP
Aeration Laterals	EDI Flex Air Diffusers	Seven duty	---	---
Air Compressors	Quincy	One duty / one standby	22.6 CFM @ 175 psig	Installed: 7.5 hP

Reportedly, the original bioreactor liquid level design operating level was 4.3m, higher than operating elevation of approximately 3.6 m which was targeted for many years. In the fall of 2025, the target operating level was further reduced to 2.50m. The impact is that the aeration system layout is likely inadequate, resulting in insufficient mixing and short-circuiting; in addition, the oxygen transfer efficiency is also likely reduced further exacerbating treatment issues in the bioreactor. Refer to key bioreactor elevations in Table 15.

Table 15. Key Bioreactor Elevations

	Elevation (m)
Bioreactor Top Elevation	14.48
Bioreactor Bottom Elevation	9.14
Bioreactor Height	5.34
Target Bioreactor Operating Level at the time of writing this report	2.50

4.3.1. Bioreactor - Extended Aeration Process

Based on average day and maximum MBR flow rates of 1,500 m3/d and 3,000 m3/d, and a historical operating level of 3.6 m, the hydraulic retention times (HRTs) are 3.66 days (88 hrs) and 1.83 days (44 hours) respectively. These HRT are closer to an extended aeration activated sludge (EA) process, which have HRTs of between 24 and 48 hours based on average day flows and solids retentions times (SRT) of up to 30 days.

The HRTs based on the current (Fall 2025) operating level of 2.5 m and proposed future scenarios are summarized in Table 16.

Table 16. Bioreactor HRTs Based on Different Flow Scenarios Compared to Typical Design Values

Scenario	Influent Flow Rate, m3/d	Bioreactor HRT @ 3.6 m – Historical Operating Height	Bioreactor HRT @ 2.5 m – Current Operating Height
Typical Extended Aeration Design		*24 – 48 hrs*	*24 – 48 hrs*
Current Average Day	**1,500**	88 hrs / 3.66 d	56 hrs / 2.32 d
Current Max Day and MBR capacity	**3,000**	44 hrs / 1.83 d	28 hrs / 1.16 d
Future Max Day	**4,500**	29 hrs / 1.22 d	19 hrs / 0.78 d

The historical operating volume exceeds typical design HRTs based on average day flows and would likely be at the upper limit of future Average Day flows. Thus, the bioreactor could continue to operate as an EA process, but optimization of the aeration system and other parameters is required. Note that current operating height to 2.5 m results in an HRT of 56 hours at 1,500 m3/d, but the aeration system would not be effective, and other operating issues may ensue.

This EA treatment systems operates in the endogenous respiration phase, whereby microorganisms will consume their own cellular mass for energy, after more easily accessible "food" (BOD) has been consumed. This sludge digestion can result in a decrease in overall sludge volume.

In addition, at VHHS, the aerated WAS tank is acting as an aerobic sludge digester, further contributing to a reduction in sludge which must be processed in the centrifuge. Combined, the overall sludge volume is lower than typical biological system.

Several factors must be optimized when operating EA treatment systems including:

- **Food to microorganism ratio (F:M):** This is the relationship between the influent BOD (food) to recycled microorganisms in the return activated sludge (RAS).
- **RAS rate**: This should typically be between 0.8 – 1.2 of the influent flow rate based on average day flows. Currently, the MBRs overflow based on flow into the MBR; this mixed liquor stream (the RAS) is returned to the front of the bioreactor. There is no control over this rate and in the winter, when the influent flows are higher due to I&I resulting in more dilute influent, the RAS rate is also higher. Thus, the F:M is off balance and with insufficient food, the filamentous micro-organisms will proliferate.
- **Aeration**. Sufficient air must be provided to ensure full nitrification.
    - Whereas degradation of carbon typically requires 0.9- 1.2kg O_2 / kg of BOD, an EA system can require up to 1.5 kg O_2/kg BOD.
    - Nitrification requires an additional 4.6 kg O_2 / kg TKN; an EA system can require even more.
    - A dissolved oxygen concentration of 2.0 is typically maintained in bioreactors to ensure nitrification.

Observations at VHHS:

- Noticeable volumes of floating sludge was observed on the top of the lagoon (refer to Figure 12) indicative of filamentous growth. There are several options available to control the growth of filamentous organisms:

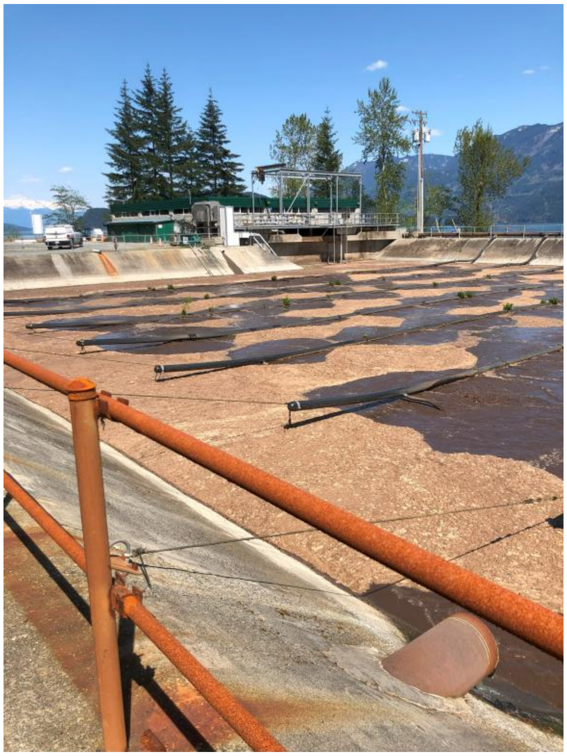


Figure 12. View of Bioreactor from South-West corner. Noticeable floating sludge.

  - Controlling F:M ratio to ensure sufficient food is available for the microorganism.
  - Small doses of chlorine could be added to the RAS stream to kill the filamentous organisms.
  - Currently, the aeration system is cycled on and off for 90 minutes at a time; the air is automatically turned off if the dissolved oxygen reached 1.5 mg/L. Modifying this on/off cycle would result in different oxygen conditions and thereby promote growth of different microorganism. However, this would require trial and error, based on seasonal variations.
  - Adding an anoxic zone at the front of the aerobic cell, where the RAS discharges. Although aeration would not be required in this zone, a mixer would be required to prevent settling. This option may be of interest if Total Nitrogen limits are anticipated to be more stringent in the future as this zone would promote denitrification.

Although the capacity of the bioreactor, which exceeds EA HRTs does provide for equalization and dampening of the influent flows, some of the negative impacts of the size of the bioreactor are noted below:

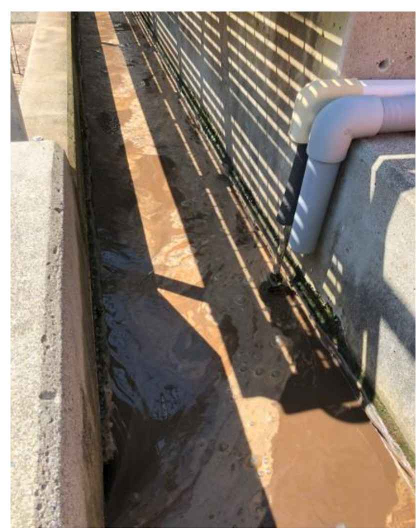


Figure 13. Mixed Liquor Effluent Channel. Promotes recirculation of floating sludge.

- The long retention times in the bioreactor result in extremely high sludge age which is evidenced by the floating sludge. When the Food to Microorganisms ratio (F:M) ratio is less than optimal, i.e., there is insufficient food, filamentous bacteria tend to proliferate. These long, thread-like structures can cause sludge bulking and flotation, which was evident in the bioreactor during the site visit.
- As the filamentous bacteria float, they are not removed during the settling phase when the MBR blowers are turned off, just before wasting. Consequently, the bacteria are returned to the Bioreactor via the mixed liquor overflow channel and compete for incoming food with the other bacteria. Potentially consider installing a weir at the discharge into the channel forcing the under flow (cleaner

wastewater) into the channel. The floating sludge would remain in the MBR. A means of transfering floating sludge to WAS tank, such as a floating decanter / skimmer (SKIMOIL) should also be considered.

- The mass of either filamentous or dead bacteria is continuously recycled back to the MBR and is likely reducing the efficiency of the MBR system.

- The blower system is controlled based on a timer (on/off cycle) and on bioreactor oxygen concentration; however, the on/off duration could likely be optimized.

- The chemical dosing is not flow paced and could likely be optimized.

- The mixing of RAS and plant influent is not optimal.

- There is also likely short-circuiting in the bioreactor, as the aeration system was likely not designed for the lower operating levels.

- Theoretically bioreactors upstream of MBRs should be operated at higher Mixed Liquor Suspended Solids (MLSS) concentrations compared to conventional activated sludge systems which operate at concentration of 2,000 to 6,000 mg/, and EAs which typically target an MLSS concentration of 2,000 mg/L. This higher MLSS is why the footprint of these bioreactor/MBR systems is typically smaller than conventional systems. The average concentration in the bioreactors followed by MBRs is 8,000 – 12,000 mg/L, but can range from 6,000 – 20,000 mg/L.

  After running the bioreactors in the 8,000 – 12,000 mg/L range, Operations staff have indicated that targeting a concentration of 6,000 mg/L to limit the long sludge ages has improved treatment at VHHS and reportedly aligns with target concentrations in similar facilities.

- **Recommendation**:

  - If EA treatment is maintained, models such as GPX-S or EnviroSim BioWin could be used to optimize the treatment system to provide operations staff better guidelines to operate the plant.

  - Improved sludge removal systems are recommended.

4.3.2. Bioreactor Components Capacity

Capacity – Bioreactor. The bioreactor can meet future flows. However, the lack of redundancy is an issue.

Capacity – Transfer Pumps. There are currently no capacity concerns and when the flows increase, the infrastructure is in place to add an additional pump. Thus, capacity is not a concern. The timing of adding a pump will depend on other plant upgrades.

Capacity - Blower. There are currently no capacity concerns and when the flows increase, the infrastructure is in place to add an additional blower. Thus, capacity is not a concern. The timing of adding a pump will depend on other plant upgrades.

4.3.3. Bioreactor Components Condition

Bioreactor: There are several concerns related to the bioreactor, as follows:

- The caulking along the sides of the bioreactor was visibly degraded in some areas
- Pressure relief valves were installed at the bottom of the bioreactor to relieve high water pressures. The condition of these relief valves is unknown as the bioreactor has never been desludged. However, Staff have noted that the liquid level rises when the lake level rises, indicating that ingress through the valves or potentially the bioreactor floor is likely occurring. This is further supported by visible floor leaks in the basement of the operations building.
- The railings along the sides are corroded in some areas.

Transfer Pump: The Flygt transfer pumps were installed in 2019, replacing the ABS pumps, and there is room for a 3rd transfer pump and associated discharge piping to the drum screen; as such, there are no concerns with the condition of the transfer pumps.

Blowers: There are no concerns with the operation of the blowers or their capacity. However, the air quality in the blower room is a concern and filters on the air inlet should be provided.

Aeration System: There are several concerns related to the bioreactor, as follows:

- Air was noticeably leaking from some of the laterals during the site visit
- Vegetation was growing on the laterals
- Maintenance of the existing floating laterals is difficult

Treatment options are discussed in Section 5.0 and depending on the process option selection, the existing aeration system would either have to be replaced, modified, or made redundant.

4.4. ROTARY DRUM SCREEN

A 2mm drum screen is located between the aerated bioreactor and the MBR tanks. This location ensures that any material > 2mm which may have landed on the bioreactors and or is drawn into the pumps is removed prior to the MBR tanks to prevent damage of the membranes.

The rotary drum screen model is an IPEC IFM 4872 which has a flow capacity of 2,875 gpm or 15,638 m3/d (181 L/s). The conveyor is a PLB 984, which is a 9” shaftless screw press with a capacity of 1.9 m3/h.

Alum for phosphorous removal is dosed in the rotary drum screen effluent, just before the MBR tanks.

When looking into the drum, it is clear that grit is getting into the bioreactor which is causing wear on the transfer pumps and any downstream piping. As material smaller than 2mm can pass through this screen, there is likely additional wear on downstream components due to abrasive nature of the material.

Figure 14. Inside of 2mm Drum Screen. Grit particles observable.

Condition: According to operations, after moving the drive motor from the bottom to the top of the shaft, there have been no operational concerns. Staff have however noted that there are concerns related to freezing of the auger in the winter.

A quote was requested from the original manufacturer for a winterization package to provide insulation around the auger, drum screen and potentially heat tracing. However, they do not provide such package.

Recommendation: A custom winterization package could be installed by a local contractor or in-house staff, starting with insulating covers and potentially a heater under the covers. A higher cost option would be the addition of a heated enclosure.

Capacity: At the time of inspection, the drum was relatively empty. However, operations staff indicated that the bin is filled approximately once per day, indicating that the drum screen is keeping up and has sufficient capacity to meet the system needs.

4.5. MEMBRANE BIOREACTOR (MBR)

The existing MBR system is comprised of two tanks. The membranes were replaced in 2020 while the LEAP aeration system was added to the cassettes in 2024. The system includes 192 installed GE ZeeWeed 500D 370 ft2 membranes and is rated for 3,000 m3/d. There is a third installed tank and related pipe stubs which would allow for the addition of another 104 membranes based on the new 500D 430ft2 model, thereby expanding the capacity to 4,500 m3/d. The system could be expanded slightly more using the latest model ZW500EV – 460ft2. But for simplicity, discussions will focus on an expansion to 4,500 m3/d.

The system consists of two outdoor concrete tanks (3,048mm L x 4,572 mm W x 3,658mm H); each tank is equipped with two cassettes, and 48 membranes per cassette and two cassettes per tank.

The treated effluent passes through the membrane and is collected in a permeate tank.

Mixed liquor discharges into a common collector channel and is conveyed by gravity back to the front of the bioreactor (RAS).

As the operation of the MBR tanks was modified so that it is based on liquid level (pressure) rather than influent flow rate, Operations staff noted that some of the original piping in the MBR tank is now redundant. Piping modifications would have to be reviewed in detail when placing the 3rd MBR tank into operation.

Membrane Cleaning

To maintain membrane performance, the membranes are cleaned using air scouring, back pulsing and chemical cleaning.

The MBR air scouring system consists of two duty blowers with a stub for an additional future blower. Air is bubbled through the membrane to dislodge solids.

Treated effluent collected in the permeate (clean water) tank, is also backpulsed through the membranes at a higher pressure to dislodge solids. A clean-in-place (CIP) cleaning solution using citric acid or hypochlorite can also be back-pumped through the membranes.

Waste from the bottom of the MBR tank is pumped to the waste active sludge (WAS) holding tanks. There is one WAS tank for each MBR tank. Table 16 summarizes MBR Pump and Blower design values.

Table 17. Summary of MBR Pumps and Blowers

Equipment	Manufacturer	Configuration	Preliminary Design Duty Point	hP
Process Blowers	Delta Aerzen Positive Displacement	One Duty / One Standby	372 to 600 SCFM @ 5.0 PSIG	Installed: 20 hP
MBR Transfer Pump	Goulds GL54BF1J6E0	One duty / One spare		

Condition:

- Based on observations during maintenance, the existing MBR tanks are reportedly spalling.
- If kept online, membranes do not experience freezing, but ice does accumulate around the water line. Freezing could occur if the trains were put offline during cold winter conditions. Temperatures as low as 10°C have been recorded in the bioreactor.
- No specific concerns were made regarding the membranes themselves.
- The piping and pumping systems located in the Process building appear to be in good condition. Past leaks were relatively easily addressed.
- The blowers are reportedly in good condition and no concerns were reported.

Capacity: As the effluent flow has exceeded 3,000 m3/d, the approximate capacity of the two MBR tanks, it would be prudent to install a 3rd MBR train in the near future to for maintenance of one train during low flows while maintaining 3,000 m3/d capacity and treat peak flows.

In the short-term, the frame for the 3rd train could be installed and the MBR cassette moved to the 3rd MBR tank. This, along with some minor piping upgrades and implementation of controls would allow for the concrete in the two MBR tanks currently in operation to be repaired, sequentially, during low flows.

The reduction of I&I in the collection system and concrete repairs to the various components will dictate the when the 3rd MBR should be brought on-line. However, to facilitate maintenance, installation of the 3rd MBR cassettes is recommended.

4.6. PERMEATE SYSTEM

The permeate holding tank, also referred to as the backpulse tank, is made of low-density polyethylene (LDPE). Tank dimensions are approximately 3251mm H x 2286mm D with a working volume of 7,900 L.

Two bi-directional permeate pumps, one duty pump per MBR tank, are used to draw effluent from the MBR to fill the permeate holding tank and used to draw permeate from the holding tank for the back-pulsing process to clean the membranes. There is one mag meter located on each pump.

When in the back-pulse mode, citric acid and / or sodium hypochlorite can be added to the permeate return to improve and optimize membrane cleaning. Table 18 summarizes Permeate Pump design values.

Table 18. Summary of Permeate Pumps

Equipment	Mode	Configuration	Preliminary Duty Point	hP
Permeate Pumps	Treatment	Two duty / one per MBR tank	Avg. 10 L/s @ 8.8 m Peak. 39 L/s @ 11.3 m	15 hP
	Back-Pulse		Build-Out. 29.6 L/s @ 10 m	

Permeate overflows the permeate tank and is discharged to the UV system.

Condition: A new LDPE permeate holding tank was installed in 2025.

Capacity: There are no capacity concerns with the existing holding tank. The addition of a 3rd membrane will require the addition of a 3rd dedicated permeate pump and flow meter.

4.7. UV SYSTEM

The UV system, originally supplied by VOLTREX, was added in 2012 and consists of three 150mm diameter UV reactors installed in parallel. The AWC report (2019) indicated that there were concerns finding replacement parts, as one of the reactors had failed.

Since then, VHHS staff retrofitted the system using new ballasts and blubs to restore it to working order and replace deteriorating pipes. VHHS staff noted effluent quality meets existing targets without the use of the UV system.

Downstream of the UV lamps, the piping runs outside and discharges into the Outfall Manhole.

Capacity: Reportedly, the flow capacity of the original UV reactors was 12-13 L/s. No information on the original UV system design was available at the time of writing the report. After the retrofit by VHSS, staff estimated 22 L/s could flow through the reactors.

Condition: Although staff have increased the flow capacity, the UV dose, i.e. UV intensity (mW/cm2) x exposure time (s) has not been validated by a 3rd party based on the new configuration, thus the maximum dosing capacity is unknown.

Recommendation: Although not reliant on the UVs system to meet target effluent quality requirements when the MBR is operation, a UV system is currently required as part of the permit and per Division 2 – Section 52 (1) of the MWR, disinfection is required in the event of unforeseen circumstances. Key recommendations are as follows:

- VHHS should pursue discussions with BCMoE regarding the need for a UV system downstream of the MBRs. Research indicates that a UV system maybe be redundant, and some authorities are no longer stipulating the need for a UV system post MBR. Refer to Appendix D for a paper on this topic.
- Including an allowance for replacing the UV system would be prudent until a permit amendment is obtained indicating otherwise.

4.8. EFFLUENT DISCHARGE AND OUTFALL

The existing outfall which discharges into Harrison Lake was built in 1978 (D&K, 2008). Effluent flows via gravity from the outfall manhole through a 250 mm dia 553m long asbestos cement line. The discharge into Harrison Lake is equipped with diffusers and is located at 49° 18.712N and 121° 48.019W.

The capacity of the existing gravity outfall is reportedly 3,630 m3/d (D&K, 2000). No information is available regarding treated discharge configuration and elevations, and lake elevations considered at the time these calculations were made.

The liquid level in the outfall manhole dictates the pressure available to push effluent through the outfall line. An in-house air vacuum/release valve was constructed to avoid siphoning of the chamber contents.

Condition: The lake outfall was upgraded approximately 15 years ago and is reportedly in good condition and is video inspected by divers every two years.

Capacity: The discharge is reportedly reduced during high lake levels. In addition, as the VHHS is considering upgrading the WWTP capacity, the capacity of the existing gravity outfall system also needs to be increased.

4.9. WASTE ACTIVATED SLUDGE SYSTEM

Two waste/drain pumps, one duty and one shelf spare, transfer waste from the membrane bioreactor tanks to one of the two waste-activated sludge (WAS) tanks. An alternate discharge allows for redirection of the sludge to the bioreactor.

The outdoor WAS tanks are on either side of the MBR tanks and were constructed using the sloped sides of the original bioreactor to form one of the side walls. The tanks are 4,600mm W x 3,658mm H x

5800mm L at the top. As the exterior wall slopes down at a 1.5:1 ratio, the bottom is only approximately 0.8 m wide. The interior walls are vertical, forming part of the MBR tank walls.

The relatively large surface area of the tanks likely promotes the continued floatation of the sludge as the tanks are not drawn down to the bottom to remove all the sludge. Due to the configuration of the tanks with a very small base, they cannot be repurposed for use as process tanks.

The Process Blowers also supply air to the WAS tanks whenever the blowers are running. The air is only shut off during the settling of the WAS, just before pumping to the centrifuge is started.

There is a new 4 tonne gantry crane which was installed above the MBR and WAS tanks to facilitate equipment removal for maintenance and installation of the 3rd MBR cassettes. Table 19 summarizes Pump design values.

Table 19. Summary of Waste / Drain Pumps

Equipment	Manufacturer	Configuration	Preliminary Duty Point	hP
Waste / Drain Pumps	Goulds GL54BF1J6E0	One duty / one shelf spare	25.2 L/s @ 4.9m TDH	5 hP

Condition: There were no visible concerns with the WAS tanks. These tanks were built from new concrete placed inside the repurposed bioreactor, with new sealant. Operators have drained and inspected the tank and did not observe visible cracks or leakage.

Capacity:

- **WAS tank**: The capacity of the existing WAS tanks currently meets the needs of the system. As sludge wasting from the membrane tanks is not continuous and alternates between two MBR tanks, the existing system could accommodate the addition of a third MBR tank. Automated distribution of sludge from three MBRs to two WAS tanks and the dewatering regime will have to be optimized in the future.

- **WAS Pump**: There are no concerns with the capacity of the existing pumps as they do not run continuously. The addition of a 3rd MBR tank would only require an adjustment to pump run time and controls.

4.10. CENTRIFUGE AND SOLIDS DISPOSAL

WAS is thickened to approximately 1.5% in the WAS tank. The thickened WAS is pumped daily to the centrifuge for dewatering using a progressing cavity pump located in the basement of the process building. See Figure 15.

Figure 15. WAS transfer pump to the Centrifuge

Thickened WAS pump design values are provided in Table 20.

Table 20. Summary of Thickened WAS Transfer Pump

Equipment	Manufacturer	Configuration	Preliminary Duty Point	hP
Thickened WAS Transfer Pump	Allweiler Tecflow 201 Eccentric Sprial Pump	One duty pump	1.1 L/s	4 HP

A centrifuge enclosed in a seacan is used for sludge dewatering; the seacan also houses the polymer metering pumps and related controls system. The seacan, cake conveyor, and disposal bin are covered with a canopy but not enclosed in a building (see Figure 16).

Figure 16. Centrifuge System. Roof cover over solids disposal bin (left); centrifuge in seacan (right).

The centrifuge is a model GEA Westfalia UCD 305-00-32, 20 HP, with a maximum capacity of 88 gal/min (5.6 L/s). Depending on the WAS solids concentration, the flow rate to the centrifuge is between 15-18 gal/min (1.1 L/s or 3.96 m3/hr); the centrifuge runs approximately 6 hrs/day five days a week.

The WAS concentration is reportedly around 1.5% solids into the centrifuge and a cake concentration of approximately 18% is achieved.

Figure 17. Centrifuge System. Two cake conveyors (left); limited maintenance room around the centrifuge (right).

Staff would like to simplify the sludge handling process. Currently two conveyor / augers are required to convey the cake from the centrifuge into the bin (see Figure 17). Staff would prefer to directly discharge into the cake bin. The cake conveyor is an Askania TSFS 250 (254-277 / 440-480 V, 2.4 HP, 60 Hz).

Condition: The existing centrifuge and conveyors are from Germany; parts and on-going operations support are difficult to obtain. In addition, freezing concerns were noted. VHHS would like the centrifuge to be replaced with a locally supported centrifuge and the freezing concerns addressed.

Capacity: The operation of the centrifuge is continuous during normal operating hours. There is minimal spare capacity to address increased loads unless the system were to be completely automated, including start-up and shutdown. Although possible, this would also require automation of several other system components such as shut down of the aeration system before starting the thickened was pumping system, and automation of the polymer system, etc.

Recommendation: Given the component supply issues, it is recommended that the centrifuge be upsized to accommodate future flows and that the centrifuge and conveyor layout be improved to facilitate operations, maintenance and controls. The seacan could be relocated and used to store equipment.

4.11. CHEMICAL SYSTEMS

There are five chemical systems which are reportedly all operating without any major concerns. The chemical totes are stored in a corrugated steel shed adjacent to the west wall of the Process Building.

Each chemical area can house either one or two totes and is equipped with lighting. Each chemical system is provided with individual secondary containment. There are no low-level switches for alarming within the secondary containment.

There is no heating or ventilation of the sheds as previously 50% hydroxide was used.

The following pumps are located inside the Process Building and draw directly from the totes:

- The sodium hypochlorite and citric acid pumps are panel mounted and include related ancillaries such as foot valves, isolation valves, back pressure sustaining valves, relief valves, check valves, and/or calibration column.

- The alum and sodium hydroxide pumps are equipped with foot valves, pressure relief valves and isolation valves.

  - The polymer day tank and pumping system are located in the Centrifuge seacan and is discussed Section 4.10.

The use and design flow rates are summarized Table 21 below.

Table 21. Chemical Systems Summary

Chemical	Use	No. of Pumps	2012 Design Flow Rate	Storage Tank
Citric Acid 50% (note 1, 2)	MBR chemical clean; pH control	One duty	Maintenance: 2.2 l/min Recovery: 6.1 l/min	1525 kg
Sodium Hydroxide (NaOH) (Note 2)	pH control	One duty	25 L/h	1453 kg
Alum (coagulant)	Phosphorus removal	One duty	34 L/h	1640 kg
Sodium Hypochlorite – 10.8% (Note 2)	MBR Chemical clean	One duty	Maintenance: 1.78 l/min Recovery: 16.28 l/min	1230 L
Polymer	Centrifuge dewatering	One Duty		1040 kg Day Tank

Notes:
1. Citric acid is reduced to 50% concentration with potable water.
2. Each tank include a ¼ hP motor mixer.

Condition: Ventilation of the storage units should be considered to reduce the potential for gas build-up. Vent discharge locations should consider compatibility with other compounds stored on site and proximity to other air intakes.

Capacity: There are no capacity concerns related to the chemical storage volumes or capacity of the metering pumps. The metering pumps are all running at the lower end of their operating range.

Recommendation: The existing centrifuge should be replaced and increased in size; the capacity and controls of the polymer system should also be reviewed in more detail to ensure integration of the polymer system controls with the operation of the centrifuge.

4.12. ONSITE POWER SUPPLY AND GENERATOR

Power Imbalances

The WWTP experiences phase voltage imbalances originating on the BC Hydro distribution system due to lengthy single phase supply line in the vicinity of Harrison Hot Springs. The voltage imbalances are worst at peak highs and peaks lows in winter and summer (heating and cooling loads). From data recorded and captured by VHHS staff from January-2024 to July-2025, the worst recorded voltage imbalances were:

- 115V between Phase B & C on 27 February 2024
- 98V between Phase A & B on 23 February 2025

Typically, a 10% voltage difference is considered tolerable, so for this 600V service, a 60V imbalance could be expected. In addition to the two above noted imbalances, there were a total of five additional events in 2024 and one other event in 2025 with imbalances greater than 10%. This is therefore an isolated and infrequent occurrence and also not easily mitigated at the WWTP. The power meter at the service entrance to the WWTP was or is being connected to the SCADA system for continuous logging and recording of voltage and power and this may reveal more frequent imbalances.

Potential Solutions

A potential solution explored by McElhanney through suppliers was a form of inline “UPS” system whereby the incoming service is rectified and then goes through an inverter supplying a balanced and controlled output voltage. It is a relatively costly and “large” solution but would conceivably mitigate the concerns, equipment damage and protect the WWTP equipment from further damage because of the grid imbalances.

Other solutions, if process conditions and treatment capacities allow, would be to utilize the power meter to inter-lock equipment from operating or even tripping circuit breakers if imbalances are extreme; this would protect equipment from damage but would interrupt treatment processes during times of extreme voltage imbalances and may require operator and electrical personnel to reset the plant after the situation returns to normal.

At the time of finalizing this report, VHHS had engaged a 3rd party to undertake a detailed electrical study to further assess the issues and recommend a solution.

Generator

In the event of a power outage, the power supply is automatically switched to an onsite generator. There were not concerns related to the operation of the generator or it’s size to meet current needs.

4.13. PROCESS BUILDING

The original L-shaped bioreactor was repurposed in 2012. The leg of the bioreactor was sectioned off and now contains the outdoor MBR and WAS tanks; the MBR piping and pumps are housed in the bottom of the original bioreactor tank inside the process building. The permeate, backwash, UV, and chemical pumping systems, blower room and electrical rooms are housed at an intermediary level in the old bioreactor tank, also inside the process building.

The bioreactor and process building elevations relative to lake levels are summarized in Table 22.

Table 22. Key Elevations and Components

Floor Reference	Elevation (m)	Key Equipment Installed at this Elevation
Bioreactor Top Elevation	14.48	Chemical Containment and Metering Pumps
Operations Building Operating Floor Elevation	13.56	
Extreme Lake Level	13.30	
Bioreactor Mid-Elevation / Process Bldg. Mid-Elevation	12.97	Permeate system, blower and electrical rooms
Maximum Monthly Mean	12.62	Recorded in June 1948
Operations Building Exterior Original Ground Elevation	12.59	
1/5 Year Lake Level used for 2012 Design	**12.30**	
Lake 99th Percentile Elevation	**11.90**	
Process Building Basement	11.55	MBR sludge wasting (MBR to WAS tank) WAS pumps (WAS tank to Centrifuge)
Bioreactor Bottom	9.14	Transfer Pumps

As seen from the elevations above, the Process building basement is 0.75m below the 1/5-year lake level used for the 2012 design. Ingress of water into the basement during the summer months when the lake level is high is evident as water comes in through cracks in the floor and along joints of the walls added to the bioreactor to support the mid-elevation operating floor in the building. There have been several instances of the basement floor pumps being flooded.

These events raise concerns regarding the long-term integrity of the bioreactor floor.

Recommendations:

- Review the equipment layout in the basement to determine if the pumps can be raised.
- Fill in the cracks and line the basement floor and walls with a hydrophilic compound to prevent moisture and water ingress.

4.13.1. Electrical Room

The electrical room is located at the mid-level elevation and houses the main service disconnects, generator automatic transfer switch (ATS), plant control panels, pump and blower Variable Frequency Drives (VFDs).

Condition: The equipment was in good condition, well maintained and in no urgent need or replacement or upgrade (barring process and other upgrades which necessitate electrical changes).

Equipment and spare parts (VFDs) are stored on the ground and around the electrical equipment. A consolidated storage location of spares and spare VFDs is recommended to both avoid accidental damage to inappropriately stored equipment, but also for organization, accessibility and knowledge of spares holding.

4.13.2. Blower Room

The blower room is located at the mid-level elevation and houses the Process Air Blowers, the Membrane Blowers and the two compressed air systems. The room walls are covered with black felt, presumably for sound attenuation. There is also some shelving to store equipment and spare parts.

Condition: There were no concerns related to the room itself. However, dust on the air intake screen of the blowers and compressors was visible. Recommendation: A means of filtering the air coming into the room or on the intake of each blower and compressor should be provided to optimize the efficiency of the equipment and reduce wear.

4.13.3. Mid-Level Central Area

The main zones in the mid-level central area include:

- The permeate pumping system, holding tank and UV system.
- A workbench and work area
- Two manholes in the middle of the mid-level floor.
    - **Groundwater Manhole**: One manhole is not sealed and collects groundwater. As the lake level rises, so does the liquid level in this manhole. A line equipped with a check valve discharges into the second manhole (right in Figure 18).
    - **Sanitary Manhole**: The drain lines around the facility discharge into the second manhole which is equipped with a Flygt submersible pump. Manhole contents are pumped back to the bioreactor.
- A section of the original bioreactor side wall which forms a ramp sloped at 1.5:1 up to the ground level garage door. A crane has been installed above the ramp as equipment and material cannot be safely moved up/down the ramp due to the steep slope. (see Figure 18). A platform could be installed over the slope; as a minimum, a safety barrier is recommended.

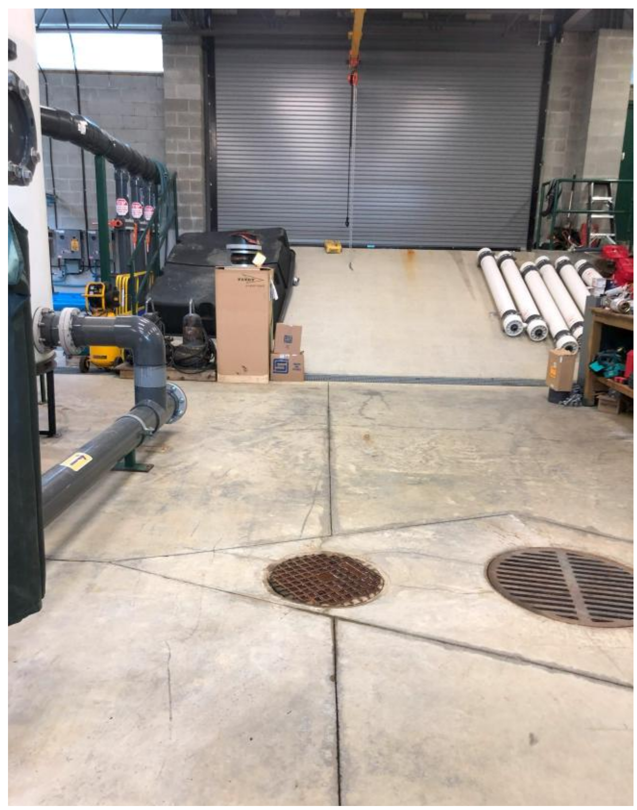


Figure 18. Process Building Mid-Level. Groundwater MH (left), sanitary MH (right), and Bioreactor Side (1.5:1 Slope) poured in 20121 in front of roll-up door. Note east and west slopes are original.

4.13.4. Storage for Equipment

Equipment is stored in several locations on site including the electrical room, the blower room, between the process building and the centrifuge seacan, and under the roof of the centrifuge bin.

A list of spare VFDs already stored on-site includes the following:

- 8700: Two new VFDs were installed in fall 2025.
  Two existing VFDs will become spares.
- P3500: Two old VFDs are spares.
- RAS pump: One old VFD is a spare.
- Centrifuge Bowl: One old VFD is a spare.

4.13.5. Space

As the existing process building was laid out considering the addition of a future MBR which would require addition equipment and piping within the building, there are no concerns related to space for the process equipment.

4.13.6. Building Condition

The building was constructed in 2012, and Operations staff did not note any concerns with major building infrastructure. A detailed structural analysis was not included as part of this work.

4.14. OPERATIONS BUILDINGS

An old process blower building was repurposed in 2012 for use as an Operations Building and includes a lunchroom, washroom, lab area and office.

The building floor elevation is several feet below the elevation of the top of the bioreactor. In addition, the building sits in the low point of the site and is prone to flooding. Refer to the original building drawing (Figure 19) and key elevations relative to the bioreactor elevation (Table 23).

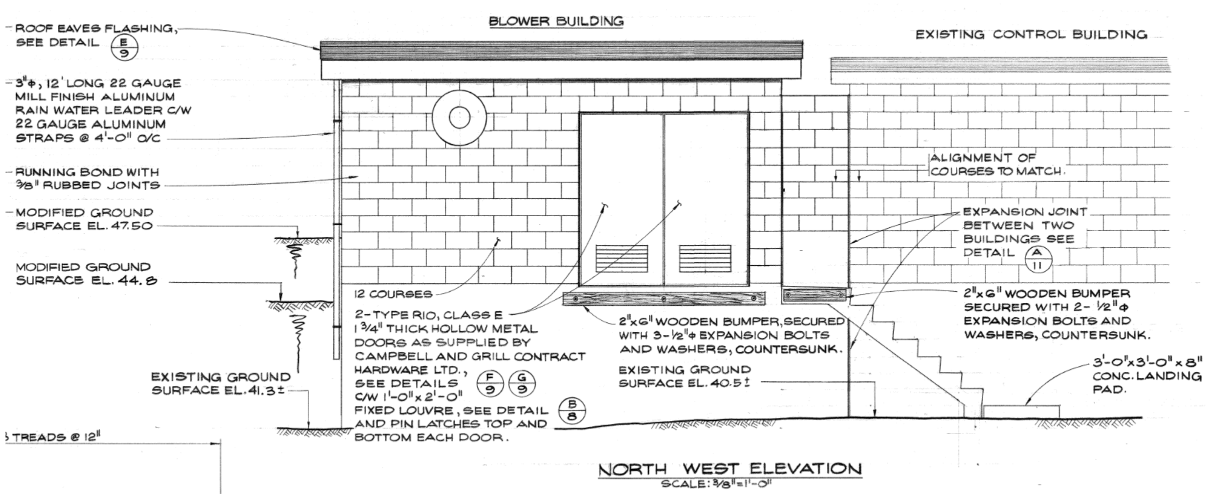


Figure 19. Original Blower Building Drawings Showing Key Elevations (Dayton & Knight)

Table 23. Key Elevations of the Operations Building, Bioreactor and Lake Flood Elevation

Floor Reference	Elevation (m)	Elevation (ft)	Notable Items at this Elevation
Bioreactor Top Elevation	14.48	47.5	North & East ground elevations
Top of Slab Elevation	13.78	45.2	Lunchroom, washroom, lab, etc.
Exterior Original Ground Elevation	13.65	44.8	Mid level of gravel stairway
Exterior Original Ground Elevation	12.59	41.3	West ground elevation
Exterior Original Ground Elevation	12.34	40.5	South ground elevation
1/5 Year Lake Level (2012 Design)	**12.30**		

Condition: As a structural condition assessment was not included as part of this scope, comments are limited to observations and anecdotal information.

- Water damage to the baseboards is visible.
- Operations also noted that there is asbestos in the cinder block.

A new operations building located at the north end of the site, on higher ground, was noted by operations as desirable, as this would facilitate access during floods.

4.15. POTABLE WATER

A 25 mm diameter potable water line supplies potable water to the site; potable water is used for the safety showers, lab, and kitchen.

The potable water line enters the Process Building adjacent to the final discharge, downstream of the UV lamps. A potable water line also runs to the Operations Building.

Supply: Potable water is supplied from the Resort. Ideally, when the forcemain is upgraded, a new water supply line should be installed in the same trench.

Capacity: Since all potable water demands are intermittent, it is assumed that the existing on-site potable water supply system will meet the needs of any future system. .

Condition: As the existing lines below the office building are reportedly old and unreliable, water in the office is not considered as potable as a precaution. Recommendation: These lines should be relocated and replaced.

4.16. COMPONENT RELIABILITY ANALYSIS

In Table 24, the number of components for the VHHS treatment process are compared to the required number of components as per the reliability requirements for a Category I WWTP presented in Section 3.5 of this report.

Table 24.Reliability Analysis of VHHS Based on Reliability Category 1

Component	Reliability Category 1 Required No. Components	VHHS Actual No. of Components	Current Configuration
Aeration Basins	multiple powered units (b)	1 Bioreactor	One Duty
Effluent Filters	2 powered units (b)	2 MBR's	Two duty each rated for ≈50% of current flows
Blowers	multiple powered units	2 Bioreactor Blowers 2 MBR Blowers	One duty / one standby
Disinfection Basins / System	multiple powered units (b)	3 UV reactors	Two duty / One Standby Maximum capacity not confirmed.
Aerobic Digesters (WAS Tanks)	2 powered units (a)	2 WAS tanks; Same Blowers as MBRs	WAS tanks can be configured duty/standby
Notes: The remaining capacity with the largest unit out of service must be at least ➢ 50% of the design maximum flow where the notation "a" appears, or ➢ 75% of the design maximum flow where the notation "b" appears			

Based on the deficiencies highlighted in red in the configuration column, the following must be considered for the proposed upgrades discussed in Section 5.0.

1. **Aeration Basin:** Either the existing bioreactor must be partitioned or new tankage added to meet the requirement of multiple units. This will facilitate maintenance while maintaining a minimum of treatment.

2. **MBRs:**

   a) Add the 3rd MBR train using the new technology, providing an additional 1,800 m3/d for a total capacity of 4,800 m3/d, based on the higher capacity of the new MBR cassettes.

   b) With the new MBR out of service, the capacity will be 3,000 m3/d or 66% of the design maximum flow rate in the short-term.

   c) When flows increase so that the max day of 4,500 m3/d is reached, replace the two original MBRs with newer technology MBRs rated at 20% higher at 1800 m3/d each. With one bioreactor out of services, the capacity will be 3,600 m3/d which is 80% of the maximum design flow rate.

3. **UV Disinfection:** If required, replace the existing UV Lamps with 2 reactors, each rated for the future full plant capacity. Operating in a duty / standby configuration, full capacity of the future system, i.e.4,500 m3/d or 4,800 m3/d depending on which MBR modules are installed, is maintained with one unit out of service.

5. Process Upgrades and Optimization

Based on the capacity and condition assessment in Section 4.0, some of the existing components are at the end of their useful life while others should be rehabilitated for continued use. In addition, the capacity of several components of the existing system should eventually be increased to meet future flows and there are opportunities to optimizing the existing system.

An options analysis, if relevant, and the proposed upgrades for each process or area of the plant are summarized in this section.

5.1. COLLECTION SYSTEM

Although not part of the scope of this Master Plan, reducing the collection system I/I should be a priority for VHHS, as this will delay the need for capacity increases throughout the rest of the treatment system. Due to the nature of these projects, this is considered an on-going activity.

5.2. HEADWORKS

As modifications to the fine screen and grit removal systems impact the lift station and forcemain hydraulics, a discussion of the headworks will precede recommendations related to the forcemain.

5.2.1. Fine Screen and Grit removal

Screened Influent Discharge into the Bioreactor

The screened effluent discharge piping into the bioreactor should be extended towards the RAS discharge into the bioreactor to promote mixing of the two streams.

Alternative: add a mixier or create an anoxic zone.

Grit Removal Improvements

A grit removal system should be added to the existing headworks to reduce grit accumulation in the bioreactor to maximize treatment volume and reduce silting in this tank and reduce wear on the downstream equipment.

Options for adding a grit removal system are as follows:

1. Keep the existing fine screen and install a pump chamber downstream to raise the screened influent up to an at grade grit system, then discharge into the bioreactor;

2. Keep the existing fine screen configuration and install a below grade grit tank that would discharge by gravity into the bioreactor; or

3. Modify the influent piping, raise the elevation of the fine screen, and install a fine screen and grit removal system at grade.

Several suppliers were contacted for information on equipment sizing, headlosses and budget costs. Equipment configuration options include:

- At grade fine screen (Claro) + at grade grit removal systems (Grit King).
- Combined at grade fine screen / grit removal system (Veolia).

An at grade grit removal system and grit decanters are depicted in Figure 20.

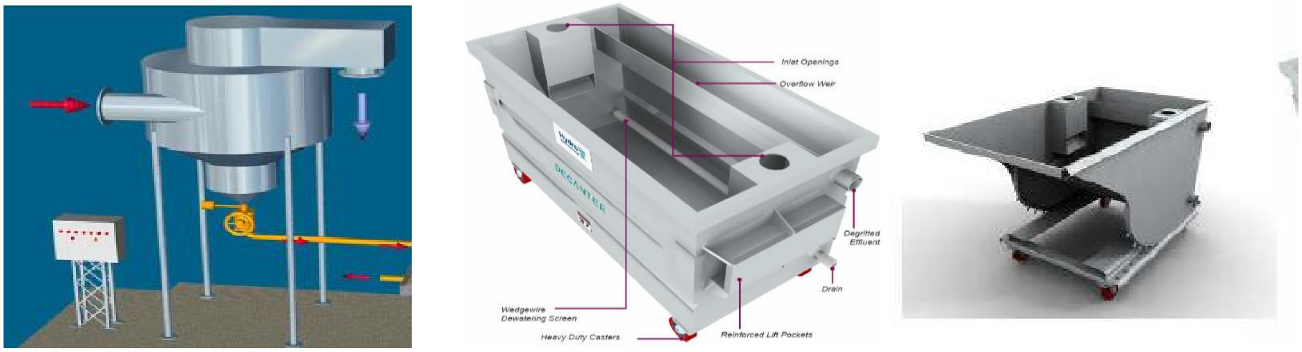


Figure 20. Above grade grit removal (left); batch dewatering decanter options (centre and right)

Based on preliminary sizing for a 9,000 m3/d system at PWWF, the headloss through the equipment would range from 50-110 mm at 0% to 30% blinding, respectively. In addition, the forcemain discharge would need to be raised 1.9 m above grade. This information was used in reviewing forcemain upgrade requirements.

When the existing fine screen needs to be replaced, it is recommended that a grit removal system be added to the equipment train. Installation of a finer screen and grit system can significantly reduce downstream operational issues, as testified by the District of Muskoka (refer to letter in Appendix E).

<u>Optional Equipment Roof:</u>

Budget permitting, the addition of a roof over the equipment would improve operator comfort and reduce equipment exposure to UV rays. Examples of roof design for an influent screen system on Vancouver Island by McElhanney is presented in Figure 21.

Figure 21. Covered Headworks Equipment by McElhanney (Claro Fine Screens)

5.2.2. Influent Forcemain

The existing section of twinned 200mm forcemain from LS #1 to the Miami River pump station could be replaced with the same size of piping.

However, replacing this section with twinned 250 mm piping would reduce the headlosses in the system.

1. If no changes to the headworks are made, the estimated maximum flow rate would increase from the current 98.2 L/s (8484 m3/d) with two pumps in operation up to 123 L/s (10,627 m3/d) with two pumps in operation.

2. Alternatively, if the existing fine screen was replaced with an at grade fine screen and grit chamber, increasing the static lift by approximately 2.4 m (0.5 m up to grade + 1.9 m above grade for Veolia) and the headlosses increased by approximately 590 mm, the maximum flow with two pumps in operation would be approximately 109 L/s.

Recommendations:

- Given the nominal additional cost to increase the pipe size from 200 mm to 250 mm, it is recommended that the existing twinned 200mm piping be replaced with 250 mm piping to allow for greatest flexibility in the future.

- In addition, VHHS should consider running a section of pipe for water reuse in the future. As the construction of a dyke along the shoreline to the WWTP had been approved, the reuse line could potentially be run adjacent to the dyke. Refer to Section 5.5 for additional comments.

5.3. INFLUENT EQUALIZATION AND BIOLOGICAL TREATMENT

Influent equalization and biological treatment are closely tied due to the current use of an extended aeration system which doubles as an equalization. Thus, this section discusses options related to improving both processes.

Influent Equalization

Given the high influent flows during the winter rainy season, an influent equalization (EQ) tank allows for reduced treatment system capacity and infrastructure downstream. To date, the bioreactor has doubled as an EQ tank due to its size. Ensuring sufficient equalization volume can be achieved by:

1. Rehabilitating the existing bioreactor and using it as an EQ tank only
2. Rehabilitating the existing bioreactor and splitting it into an EQ tank and Bioreactor tank
3. Building a new EQ tank

Two Bioreactors to Meet Regulations

To meet regulations, two bioreactor trains are required for redundancy during maintenance. This can be achieved either by:

1. Splitting the existing bioreactor with partition walls
2. Building new bioreactor

Treatment Process Options

To improve the biological treatment system, there are two options:

1. Optimize the existing EA process.

2. Replace the EA process with a different biological treatment process.

a. **Conventional Activated Sludge System (CAS).** This is similar to the existing EA process with shorter HRTs, typically between 16-24 hours; although the HRTs can be reduced upstream of an HRT. Wastewater and activated sludge are mixed with air in a tank. This process could be implemented within the existing bioreactor or externally.

b. **Moving Bed Biological Reactor (MBBR).** Specialized plastic carriers (media) create a large surface area for biofilm growth, increasing biomass concentration compared to the CAS system. The media are suspended in the tankage by the mixing energy of the aeration system. These processes deal better with variable influent loads and flows than CAS as biofilm is attached to the media. HRTs upstream of an MBR are typically around 4-8 hrs. Due to existing bioreactor configuration with steep slopes, this option could only be considered if external tankage was used to reduce air required to maintain media in suspension and reduce short circuiting.

c. **Integrated Fixed Film Activated Sludge (IFAS).** Synthetic media is fixed in place to maximize the surface area available for biofilm growth, resulting in higher biomass concentration than CAS. Typically, less energy is also required for aeration as the air can be directed under the media but is not required to maintain the media in suspension. These processes deal better with variable influent loads and flows than CAS as biofilm is attached to the media. HRTs upstream of an MBR are typically around 4-8 hrs. This process could be implemented withing the existing tankage although external tankage would allow for footprint optimization.

Quotes were obtained for external MBBR and IFAS systems for comparison purposes.

<u>Summary of Options</u>

A summary of options considered at a high level include:

1. Optimizing the existing AE process and using the excess volume for equalization
2. Partitioning the bioreactor to provide redundancy and optimizing the existing AE process; when required, the bioreactors could provide buffering of high flows as per current operation
3. Partitioning the bioreactor to create an EQ zone and biological zones; the treatment zones could be operated as an AE or CAS process
4. Partition the bioreactor to create two biological tanks; build an external EQ tank
5. Convert bioreactor to EQ tank and add external treatment process (IFAS and MBBR evaluated)

As rehabilitating and partitioning the Bioreactor is key to several of these options, this is discussed in more detail in Sections 5.3.1 and 5.3.2 below. Potential process options are discussed in Sectio 5.3.3.

5.3.1. Bioreactor Concrete Rehabilitation

When considering constructability and the need to maintain treatment, the bioreactor would have to be rehabilitated in sections, while flow is maintained through the live section. Work would have to be completed during the dry season.

Each section would need to be isolated for approximately two months, allowing time for desludging, cleaning, concrete preparation, coating application and curing during the low flow season when the water table is also below the bioreactor floor. If walls or baffles are being installed, this would be undertaken at the same time.

Several suppliers were contacted for coating recommendations. Based on the age of concrete, leakage concerns and high-water table concerns, the two coating systems recommended are presented in Table 25 along with cost estimate for the coating only. These costs do not include desludging, cleaning, coating application, or other contractor mark-ups for mobilization and demobilization.

Table 25. Bioreactor Concrete Coating Systems and Costs for Coating only

Manufacturer	Coating System	Cure Time	Unit Cost	Total Cost
Sika	Sika EWL Elastic Waterproof Lining for Concrete		$260	$721,500
Spec Roc System	RJ Watson Zedseal + Sauereisen 220 Joint Filler		$450	$1,248,750
	Sauereisen 210S SewerGard sprayable Liner			

The cost to rehabilitate the existing bioreactor is significant. The desired bioreactor use and level of rehabilitation should carefully be considered by VHHS.

5.3.2. Feasibility of Partitioning the Bioreactor Tank

Permanent Partition

Options 2, 3, and 4 would require a permanent partition wall, allowing for isolation of one bioreactor train or the EQ zone for maintenance. Structural input was solicited to review the feasibility of the construction of a permanent partition wall.

Based on a preliminary analysis, installation of a permanent partition wall would require major structural work. The bioreactor cell is only lined with a non-structural concrete slab, which is too thin to act as foundation for a partition wall. A cantilever wall would probably be 400 mm (16”) thick with a footing of at least the same thickness, but the existing concrete slab is only 100 mm (4”) thick. The entire cell would have to be emptied, the slab cut, the foundation built below the slab, the wall built, and the slab filled in.

Construction of the above would likely take over a month because the concrete would have to cure to gain its strength.

Temporary Partition

Systems used for temporary dewatering and floor protection might be feasible; however, the need to provide a 3 m high partition, allowing for 0.5 of buffering above the existing operating level, requires a sturdy structure. Temporary systems typically are not 100% waterproof, and would still require some pumping, which would not be ideal.

Baffle Walls

As long as the water level remains equal on both sides of the wall, baffle walls are non-structural walls which can be used to improve the flow path and reduce short-circuiting. These walls are not 100% waterproof either, allowing leaking below and/or along the sides, however most of the water in the cell is directed in the preferred flow path.

These walls would be ideal to improve mixing of the screened influent and the RAS as costs and installation complexity is less onerous than a fixed partition. However, they could not be used to create two bioreactor trains or a bioreactor and an EQ zone.

Summary

Budget cost estimates to split the bioreactor in two using either concrete or a baffle wall are presented in Table 26. These costs do not include emptying and cleaning the tank, excavation for concrete footings, backfilling, or concrete slab repair.

Table 26. Estimated Cost to Split Bioreactor in Two Based on Partition Type

Partition Type to Split Tank in Two	Complete Isolation	Description	Total Cost
Concrete Wall	Yes	Reinforced concrete wall and footing	$400K to $500K
Baffle Wall	No	Enduro Composite FRP baffle walls, and concrete footings, incl. install	$660,000

Based on the above, baffle walls are hard to implement quickly, are not ideal in an emergency and cost more than concrete walls. Consequently, baffle walls would only be considered for portioning a small area to create an anoxic zone.

However, additional process tanks and infrastructure will also be costly and complexity to the system. Thus, if redundancy is to be provided, costs considerations will be important.

Reducing the need to enter the bioreactor by replacing the existing aeration equipment would be required if bioreactor redundancy is not provided.

5.3.3. External Biological Process – MBBR

Process Description

The MBBR technology has provided a robust treatment approach for both full plant design and to augment ammonia removal for lagoon treatment. The MBBR process originated in the 1980’s as a relatively simple process that staff with lagoon-based experience can manage. It was first developed as a complete mechanical plant to treat raw sewage after screening. The technology has subsequently been used to augment lagoon-based treatment to enable more effective ammonia removal. The treatment has proven ammonia reduction with low cBOD levels anticipated after the lagoon. The process uses a plastic media that hosts microbes that are required to meet low ammonia levels - even in cold temperatures.

Micro-organisms grow on the media surfaces that are submerged in an aerated basin. The small media provides an extremely large surface area relative to volume to which a vast quantity of biomass will attach. This moving biomass provides a significant amount of treatment surface in a small footprint. Treatment expansion can also be as simple as adding more media. There are no moving wear parts in the bioreactor. The MBBR tanks require a screen to keep the media in the bioreactor from moving into the storage lagoon after the bioreactor.

Figure 22. MBBR Media

The media is kept in constant motion throughout the bioreactor. An aeration system provides air for supporting aerobic digestion by the bacteria plus bioreactor mixing; blowers are controlled using a feedback loop for optimal control and efficiency. From an operator's standpoint, the flow-through principle of this technology is like the existing lagoons.

The biomass adapts to influent conditions. Higher strength influent promotes growth. Lower strength will see a loss of biomass that falls from the media that will settle into subsequent storage cells. A high concentration of media is added to increase the opportunity for the attached biofilm to contact the influent nutrients (sewage). The process has proven performance for communities with high I&I. The high concentration of media and the likelihood of contacting nutrients make this process less sensitive to hydraulic changes, unlike most other biological processes which are sensitive to the lower concentration of feed that results from I&I issues.

Concept

Hydraulics permitting, to reduce on-site construction costs, the MBBR system could be supplied within a factory supplied structure including MBBR tanks, piping, and blowers, piping and electrically connected. Alternatively, tanks could be shallow bury concrete which would provide some ground insulation but also avoids high water table issues, while facilitating access. The elevation of the tanks would have to be coordinated with the influent screens and the existing MBR tank operating levels, to ensure gravity flow.

A quote was received for a two-train system, which would meet the MWR redundancy requirements. The design parameters for Phase 1 and Phase 2 flows are summarized in Table 27; the key differences between the phases which are highlighted in light blue are the size of the concrete tanks, the volume of medio and the aeration requirements.

Table 27. Design Parameters for Phase 1 and Phase 2 MBBR Systems

Description	Units	Phase 1 – 3,000 m3/d	Phase 2 – 4,500 m3/d
Number of Process Trains	-	2	2
Number of BOD reactors per train	-	1	1
BOD reactor dimensions (each)	m	4.57 L x 3.66 W x 3.66 SWD	**4.57 L x 4.57 W x 3.66 SWD**
Number of nitrification reactors per train	-	2	**2**

Description	Units	Phase 1 – 3,000 m3/d	Phase 2 – 4,500 m3/d
Nitrification reactor #1 dimensions (each)	m	9.14 L x 3.66 W x 3.66 SWD	**10.67 L x 4.57 W x 3.66 SWD**
Nitrification reactor #2 dimensions (each)	m	4.57 L x 3.66 W x 3.66 SWD	**4.57 L x 4.57 W x 3.66 SWD**
Total reactor volume (all reactors, all trains)	m3	489.7	662.7
Aeration system type	-	Medium Bubble Aeration System	Medium Bubble Aeration System
Residual D.O. concentration	mg/L	2.5-5	2.5-5
Total air requirement - design	Nm3/hr	**1,271**	1,841
Total Media Provided of 650 m2/m3 media	m3	**210**	314
Note: Existing blowers would be used for the air supply.			

To reduce costs in the short-term while facilitating long-term expansion, if the MBBR were implemented, the Phase 2 concrete tanks could be constructed while only adding sufficient media and aeration for Phase 1. When the flows increase, media could be added (thrown into the tank) and the aeration system expanded accordingly. The existing blowers have sufficient capacity to meet the required air flow for the MBBRs.

Flows from the headworks screen and grit removal to an MBBR and straight to the MBR tank would negate the need for the 2mm drum screen. However, if overflows were stored in the existing bioreactor / extended aeration tank and returned to the MBBR for treatment, installing the 2 mm drum screen upstream of the MBBR line would be prudent to avoid additional debris from the EQ tank.

5.3.4. External Biological Process – Fixed Film Bioreactor

Process Description

Fixed film bioreactor modules include fixed media with integrated aeration systems which can be installed into existing lagoon cells or tanks to augment the system treatment capacity for both BOD and Ammonia removal by providing a greater surface area for biological growth. ***Figure 5-5*** depicts a typical module with the aeration line connection depicted rising above the module. .

Figure 23. Fixed Media Module

Similar to MBBRs, these modules can be added incrementally as influent flows increase or other parameters change, thus requiring smaller initial capital investment.

Concept

Although these systems can be installed in lagoons, due to the concerns with the existing bioreactor watertightness, investigation of use in external tanks was deemed of greater interest. Similar to the MBBR, this system could be installed in concrete tanks. A quote was received from Ecofixe for the equipment and tank sizing. Refer to Table 28.

Table 28. Design Parameters for Phase 1 and Phase 2 MBBR Systems

Description	Units	Phase 2 – 4,500 m3/d
Number of Process Trains	-	2
Total Number of reactors, ea./ with 4 aerators	-	37
Dimension of each reactor	m	5,0 m x 3.3.m x 3.4 m
Minimum Footprint	m2	350
Minimum Side Water Depth	m	3.0 m
Total reactor volume (all reactors, all trains)	m3	1050
Volume of media / reactor	m3	36
Total Volume of Media	m3	1,307
Residual D.O. concentration	mg/L	2.5
Total air requirement - design	Nm3/hr	2417
Note: Existing blowers would be used for the air supply. One 100 hP blower required		

As this option requires slightly more air and a slightly larger bioreactor volume, it was not pursued any further.

5.3.5. Summary of Options Considered

The five options considered are summarized Table 29. Major additional equipment and upgrades required for these options are listed in the top half of the table. Pros and Cons for each option are listed at the bottom of the table.

Table 29. Summary of Equalization Tank and Treatment Process Configurations

Description	Option 1: Optimize Existing EA Process	Option 2: Partition Bioreactor to allow for Redundancy + Optimize EA Process Use Bioreactor for EQ	Option 3: Split Bioreactor into EQ Zone and Two Bioreactor Zones + Use AE or CAS process	Option 4: Partition Bioreactor to two Bioreactor Zones (AE or CAS), Build External EQ tank	Option 5: Convert Existing Bioreactor to EQ Tank + Build External Process (MBBR)
Required Upgrades					
Improve Influent + RAS mixing	✓	✓	✓	✓	✓
Desludge Bioreactor Tank	✓	✓	✓	✓	✓
Refurbish Bioreactor Tank – (Sika Coat)	✓	✓	✓	✓	✓
Upgrade Existing Bioreactor Aeration System	✓ $175K	✓ $175K	✓ $175K	✓ $175K	incl. in MBBR quote; minimal cost for additional header
Filamentous control: Add weir plate to MBR mixed liquor effluent or add Cl2 to RAS	✓	✓	✓	✓	✓
RAS Control: Add Flow meter and Flow Control Valve	✓	✓	✓	✓	✓
Split Bioreactor - Partition(s) Walls	X	$700K for one partition	$1.4M for 2 partitions	$700K for one partition	X
Add Mixer to EQ zone or tank			✓	✓	✓
Add New Transfer Pump (EQ to Bioreactor)			✓	✓	Reuse some existing pumps
Add New External Tank(s)				$4,7M One 2500 m3 EQ	$3,0M Two Bioreactor Tanks and Equipment
Reroute Bioreactor Effluent Transfer Line					✓
Increased Sludge Production			✓ CAS option	✓ CAS option	✓
Evaluation					
Meets Bioreactor Redundancy Req'mt	**X Ruled Out**	✓	✓	✓	✓
Pros	▪	▪ Improved operational flexibility ▪ Easier maintenance	▪ Improved operational flexibility ▪ Easier maintenance	▪ Improved operational flexibility ▪ Easier maintenance	▪ Better process control ▪ No concerns with leaking process tank

Description	Option 1: Optimize Existing EA Process	Option 2: Partition Bioreactor to allow for Redundancy + Optimize EA Process Use Bioreactor for EQ	Option 3: Split Bioreactor into EQ Zone and Two Bioreactor Zones + Use AE or CAS process	Option 4: Partition Bioreactor to two Bioreactor Zones (AE or CAS), Build External EQ tank	Option 5: Convert Existing Bioreactor to EQ Tank + Build External Process (MBBR)
					▪ Ease of Expansion ▪ Reduced Air needs (lower O&M) ▪ Allows for repairs of ex. Bioreactor during dry season. Can reduce required EQ tank volume and area requiring repairs ▪ Can replace Bioreactor Aeration with mixers; run as req'd (lower O&M)
Cons	▪	▪ **Rehabilitation Logistics**: Keeping plant operational while constructing partitions – a small treatment system would potentially be required	▪ EQ zone still needs coarse aeration at low water levels and/or mixers at higher levels ▪ Higher capital and O&M costs with little operational advantage ▪ **Rehab. Logistics**	▪ Large EQ tank required ▪ Extra transfer pumps and mixers ▪ Higher capital and O&M costs with little operational advantage ▪ **Rehab. Logistics**	▪ Extra equipment ▪
Major Costs Differences Compared to Option 2 – Base Case	**Ruled Out**	**Not Recommended**	▪ Transfer Pumps ▪ Mixer ▪ Same BOD = Similar Aeration req'd ▪ Some potential savings in diffusers **Not Recommended**	▪ EQ Tank ▪ Mixer ▪ Transfer Pumps **Not Recommended**	▪ MBBR quote ▪ MBBR Tanks ▪ Transfer Pumps ▪ Mixers **Recommended**

Based on the above, the following options were ruled out or not recommended:

- **Option 1** should be ruled out as it does not meet MoE redundancy requirements. There will likely be some system modifications that should ideally be implemented in the short-term if the funding is not available to implement the preferred option for several years.

- **Option 2, 3 and 4** are not recommended due to the complexities related to construction. In 2012, reportedly the existing bioreactor was never properly desludged and the bioreactor was never properly refurbished. There have been on-going operational issues since then, including unnecessary treatment of wastewater and consequently unnecessary operational costs.

- The use of a CAS process for **Options 3 and 4** is not recommended due to the increased in sludge production, which increases all desludging system related O&M costs (sludge pumping, WAS aeration, WAS pumping to centrifuge, centrifuge operation, polymer addition).

- **Option 4** is not recommended as using external tanks for EQ rather than the unused bioreactor volume results in higher capital costs and O&M costs with little operational advantage.

Summary and Recommendation

In summary, the preferred option would be to construct two external biological treatment tanks (MBBRs) and use the existing biological tank exclusively as an EQ tank during high flow events (overflows).

Overflows could be returned to the treatment train using the existing transfer pumps once the high flows have subsided. The existing drum screen could be used upstream of the MBBRs on the overflow return line. The drum screen would likely have to be relocated. (Refer to Figure 24)

As the existing influent station pumping system is limited in its capacity to elevate the influent sewage and the operating level of the MBR is fixed, it is anticipated that pumps would be required either downstream of the new headworks or downstream of the MBBR tanks to lift the wastewater up to the MBR. A detailed hydraulic analysis would be required to determine the best location and size of the pumps.

Options

1. Denitrification Zone: Add an anoxic zone for denitrification if future regulations limit total effluent nitrogen

2. A cover can be added for all options to improve winter operation for heat retention. The cost of a cover over the existing bioreactor is expensive. However, if implementing an MBBR, a cover could be added at a later date over the MBBR.

Figure 24. Site Plan Showing Proposed MBBRs, Relocated Drum Screen and Major Piping from Fine Screens and Return EQ Line through Drum Screen.

5.4. MBR

As there are no concerns with the performance of the MBR system, recommendations below are related to long-term treatment capacity and eventual equipment replacement:

1. Add a 3rd MBR frame, piping and related controls. Transfer two MBR cassettes from one tank to the 3rd tank.

2. MBR tanks should be inspected, refurbished as required, and coated with Sika Top Seal 107 or similar to prevent potential leakage. Add a scum removal system to facilitate removal of filamentous organisms (See figure 24).

3. Sequentially retrofit the MBR tanks and then the WAS tank

4. Add a 3rd MBR now using the new technology, providing a total capacity of 4,800 m3/d.

Figure 25. Rotating Scum Removal System

5. With the new MBR out of service, the capacity will be 3,000 m3/d or 66% of the design maximum flow rate in the short-term.

6. When flows increase such that the max day of 4,500 is reached, replace the two original MBR cassettes with newer technology rated at 20% higher (1800 m3/d each MBR). With one MBR out of services, the capacity will be 3,600 m3/d.

5.5. UV SYSTEM AND WATER REUSE

5.5.1. Treated Effluent

Unless authorized by a director and included in a permit that disinfection is not required after an MBR system, a validated UV disinfection system is required.

To meet the disinfection reliability requirements, the existing UV Lamps would have to be replaced with UV reactors each rated for 4,500 m3/d or 4,800 m3/d, pending MBR equipment selection. Operating in a duty / standby configuration, full capacity of the future system can be maintained.

5.5.2. Reclaimed Water

Based on the current regulations, reuse water must be disinfected, in addition to meeting the criteria outlined in the MWR, depending on the use. Refer to Section 3.4.5 of this report.

If water were re-used within the WWTP for equipment washdown, the addition of hypochlorite would be required to prevent microbial growth in the distribution lines.

Treatment requirements for other re-uses such as irrigation, should be further investigated. As discussed in Section 5.2, a water reuse pipe could potentially be run adjacent to the proposed dyke along the shoreline line.

5.6. EFFLUENT DISCHARGE AND OUTFALL

As discussed in Section 4, the capacity of the existing outfall is reportedly 3,600 m3/d, based on the original design. To accommodate an increase in treatment capacity, the capacity of the outfall system must also be increased.

As the cost of either increasing the outfall pipe diameter or adding a 2nd outfall line would be extremely expensive in addition to likely requiring environmental assessments and more complex permitting, the only options which would not affect the portion of the outfall line in the lake were considered. The options include:

Option 1: Increase the Operating Level. As the discharge pressure is dictated by the liquid level in the Outfall manhole, one option would be to raise the operating level in the outfall manhole.

To achieve gravity discharge at a flow rate of 4,500 m3/d, 2.10 m of head is required assuming the least amount of pipe restrictions: i.e., a straight pipe from the existing outfall manhole to the discharge point, with no elbows or diffusers. The operating level in the manhole would be 2.10 m higher than the lake level.

- At a 1/5-year lake level of 12.30 m, the operating liquid level in the outfall manhole would have to be 14.40 m.
- Theoretically, 12.50 m would be the maximum lake elevation that allows for a gravity discharge at a flow rate of 4,500 m3/d is. The resulting operating level in the effluent manhole would be approximately 14.60 m, just below the Manhole Rim elevation of 14.64 m but higher than the top of the bioreactor (14.48 m).

Although the average elevations in June and July are around 11.00 m, and the 99th percentile is 11.903m, based on the recorded Lake Levels (See Figure 26), the lake level can exceed 12.50 m from late May to early July.

Thus, this option is not recommended, and other options must be considered.

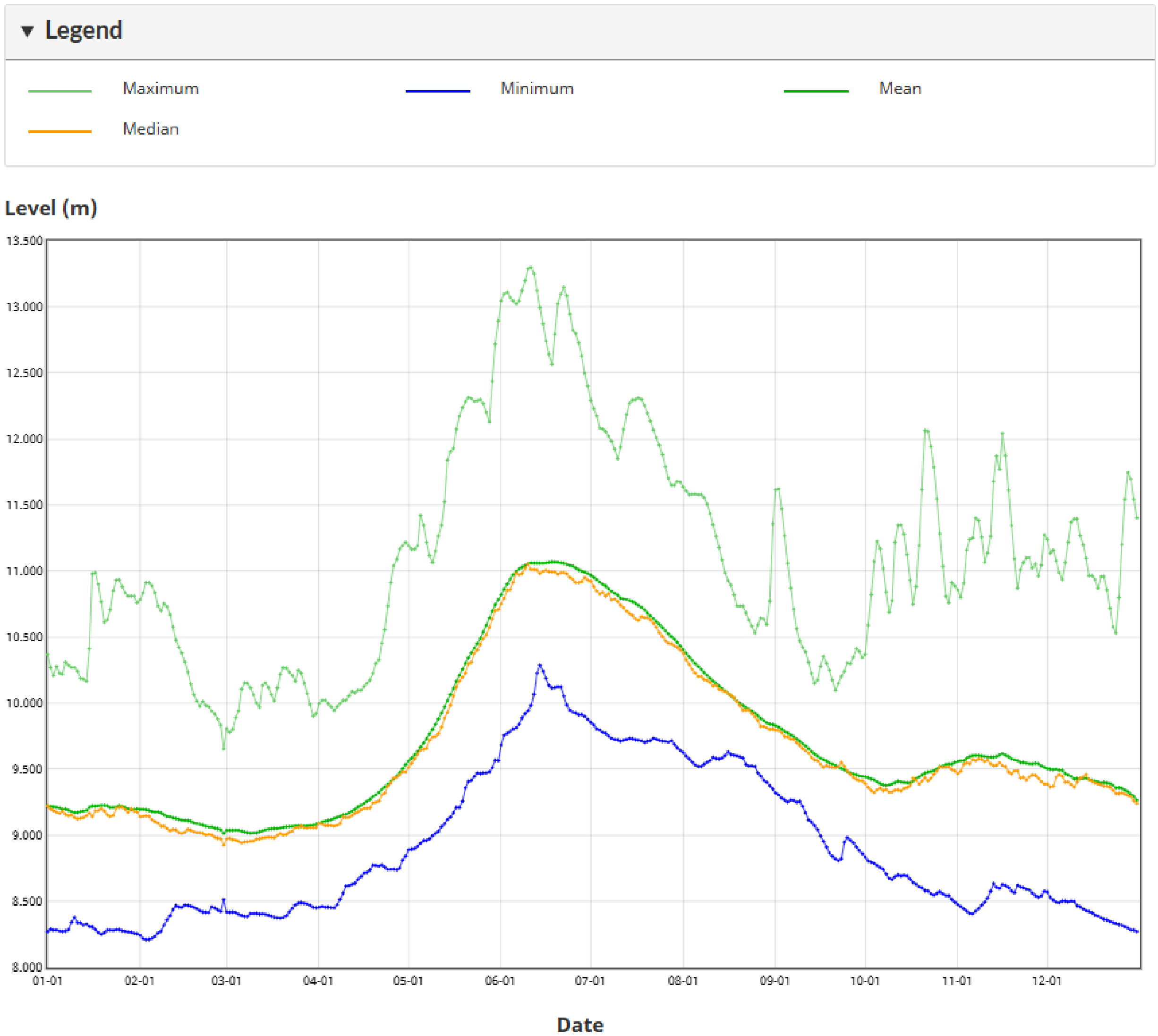


Figure 26. Daily Water Level Graph for Harrison Lake Near Harrison Hot Springs (08MG012)

Option 2: **High Flow By-Pass Line.** This option involves installing a check valve in the outfall piping, just at the edge of the outfall manhole and tapping into the existing line to create a pressurized high flow by-pass. This by-pass line could only be "opened" in case of emergency.

Operations staff noted that the existing permeate pumps are limited to a maximum pumping rate of 17 L/s. However, when one MBR tank is in backwashing mode, the second pump can ramp up to a maximum effluent rate of 29 L/s (2505 m3/d). Allowing the permeate pumps to pump in the forward direction at their maximum pumping rate could increase the effluent discharge significantly. Veolia would likely have to be contacted to implement the programming changes under emergency conditions.

This option would also require additional piping modifications to ensure a flow rate of 4,500 m/d could be achieved through the system.

A more detailed hydraulic analysis of the piping system to determine the system curve (flow vs headloss) would be required, which would also need to include any changes due to replacement of the UV equipment.

Option 3: **Booster Pumps.** Rather than relying on the permeate pumps, a booster pump system could be added. The pumps could be installed in the outfall manhole. The pumps could either run continuously or only based on high incoming influent flows:

- If running continuously, the pump discharge would be tied into the existing piping.
- If only operated during a high influent event, similar to the high-flow by-pass option, a check valve could be installed on the existing piping, allowing for continued gravity discharges, although slightly limited due to the higher headlosses through the check valve. And the pump discharge would be tied into the outfall line just downstream of the check valve.

Recommendation: As Option 1 would leave VHSS at risk of overflows, Option 3 is recommended since it would be completely independent from the MBR system and the need for complicated controls based on the operating status of the MBR tanks.

5.7. SLUDGE DIGESTION (WAS SYSTEM)

As with all repurposed sections of the original bioreactor, the WAS tanks should be inspected regularly and refurbished as required. Based on recent inspections, there are currently no concerns.

Further hydraulic analysis is required to determine if the existing WAS pump needs to be replaced when the centrifuge is replaced. Typically, progressing cavity pumps can push against higher head and only the capacity (volume) may be deficient. Currently the pump is typically running at minimum speed, so it is anticipated that there the capacity would meet short-term needs.

5.8. DEWATERING (CENTRIFUGE)

The existing centrifuge runs 6 hrs / day five days a week. Allowing for some start-up and shut-down tasks, the centrifuge is running full time. Increasing the treatment system capacity 1.5 times, results in the need to increase the capacity of the sludge thickening and dewatering processes.

The capacity of the existing centrifuge approximately 20 m3/hr, but due to the low WAS concentration, it is fed at approximately 3.4 m3/h to maximize solids concentration of the cake. There would be two options available to increasing WAS throughput:

- Increase the WAS concentration by adding thickening equipment
- Upsize the centrifuge.

As the existing centrifuge is old, and difficult to service and obtain parts, the recommendation is to replace the existing centrifuge. In addition, this will allow for optimization of the layout.

A cost estimate for a HAUS centrifuge model DDE3542 with a maximum capacity of ~ 12 m3/h is the basis of pricing for the cost estimate.

5.9. PROCESS BUILDING

A few upgrades and modifications in the process building include:

1. Repair the basement floor

2. Raise the pumps by modifying the piping layout.

3. Provide air filtration in the blower room. Either add individual air filters to each blower or add air filtration to the blower room intake. Individual blower intake air filters was assumed for costing purposes.

4. The addition of a platform in front of the roll up door could be considered to prevent people and objects from rolling down the steep incline.

5. A chain link safety barrier should be considered for staff safety; see Figure 27 for suggestions.

Figure 27. Sample Safety Barriers: Retractable gate (left) and Chain Link (right)

5.10. OPERATIONS BUILDING

Allowances for a new one-story building have been included in the cost estimate so that the operations building is no longer sitting at the low point of the site and can be more easily accessed in the event of a high rain event when the grounds are flooding.

It is anticipated that the building be approximately 80 m2 and include the following rooms:

- Two offices (4m x 3m each)
- A lunchroom (4 m x 6m),
- A washroom and shower (1.5m x 2.5m),
- A small lab (3m x 5m),
- Storage (4mx4m),
- Utility room for HVAC, Hot Water and electrical equipment (4mx4m).

Figure 28. Approximate Area of 80 m2 Operations Building in Northwest Corner of the Site

The 80 m2 footprint of the proposed operations building is shown on the site plan for reference in Figure 28. The layout and required amenities would need to be optimized.

6. Summary of Recommendations

The key recommendations suggested throughout the technical memo are consolidated in this section. The projects have generally been group into two categories:

- **Small Projects** based on value and complexity and projects that can be completed in-house.
- **Large Projects** and more complex projects

Key drivers for each project are listed. Key drivers are a combination of operator safety, capacity, end-of-life expectancy, system optimization and operability, maintenance concerns, funding availability.

Based on the key drivers, the project timeline was categorized as short-term (0-5 years), medium-term (5–10 years) and long-term (10-20 years) for simplicity. Refer to Table 30 for a summary of the projects, key drivers and anticipated project timeline.

Table 30. List of Recommended Replacements, Upgrades and Activities, including Key Drivers and Anticipated Timing

Key Drivers and Project Timing			
Village of Harrison Hot Spring			
WWTP - Master Plan - Capital Improvements			
Project No.	Project Description	Key Drivers	Project Timing
	SMALLER AND/OR IN-HOUSE PROJECTS		
S.1	Influent Flow Analysis	Understand flow patterns, impact of I&I reduction and when capacity upgrades are required	On-going
S.2	Bioreactor - Replace corroded handrails	Operator Safety - Complete	Short-term
S.3	New MWR Permit	Regulatory	Short-term
S.4	Chemical Storage Upgrades -Ventilation	Operator Safety	Short-term
S.5	Process Building - Floor Repair - In-House	Operations - Building Integrity	Short-term
S.6	Process Building - Chain or Gate at Overhead Door for Safety	Operator Safety	Short-term
S.7	Addition of 3rd MBR Frame - In-House	Operations - Reliability / Flexibility for Maintenance	Short-term
S.8	MBR and WAS Tank Concrete Repair, In-House	Operations - Building Integrity	Short-term
S.9	New Centrifuge and Building and Assoc. Equipment	Reliability (Next Major Failure): *Lack of parts and maintenance support* Funding is available	Short-term
S.10	Addition of 3rd MBR Cassette, Piping, Controls, EI&C, In-House	Capacity - Reliability	Short-term
S.11	Process Building - Blower Room - Air Intake Filters	Process Optimization	Medium-term
S.12	Upgrade MBR Modules in Trains 1&2, In-House	End of Life *Replaced in 2019 - replace in 10 -15 yrs*	Medium-term
S.13	Operations Building	Operations	Long-term
	LARGER AND MORE COMPLEX PROJECTS		
L.1	Influent Forcemain Twinning - M.R. Pump Station to WWTP *Incl. power, water supply, fibre and spare conduit*	Integrate with Dyke Upgrade if Possible to Reduce Costs Capacity - PWWF ≈ 8,000 m3/d	Short-Term
L.2	Plant Electrical Upgrades	Power Surges & Phase Imbalance Funding	Short-Term
L.3	New Outfall	Effluent Max Month Flows ≈ 8,000 m3/d and/or Lake Levels rise	Medium-Term
L.4	New Biological Process Trains (MBBR)	Regulatory Redundancy Reduced Operating Costs Grant Funding	Long-Term
L.5	Desludging and Repair of Existing Bioreactor for EQ Tank Use	May be combined with the MBBR project	Long-Term
L.6	Headworks Upgrades (Screen and Grit Removal)	End of Life or Capacity - PWWF ≈ 8,000 m3/d	Long-Term
	FURTHER INVESTIGATION REQ'D		
A.1	New UV Disinfection System	Regulatory - Confirm new permit requirements	

6.1.1. Implementation Schedule

Although a rough implementation schedule has been indicated in Table 29, some of the projects presented are stand-alone projects and can be undertaken when funding is available or based on other key drivers, whereas other projects should be undertaken sequentially to facilitate on-going operation during project implementation. Several projects should also include some allowance for future piping tie-ins to facilitate work.

6.1.1.1. Stand-Alone Projects (SA)

Stand-Alone (SA) projects include the following:

- **S.1** Flow should be monitored and analysed in detail in two years to determine diurnal, seasonal and other trends *- on-going at the time of writing the report.*

- **S.2** The process for a new permit should be completed to reflect the current process. The process was not completed during the 2012 upgrades.

- **S.3** Corroded Bioreactor Handrail Replacement – *on-going at the time of writing the report*.

- **S.4** Chemical Storage Ventilation Upgrades

- **S.5** Floor Repair in the Process Building Basement

- **S.6** Add a chain or gate in front of the roll-up door,

- **S.7 and S.8 Addition of 3rd MBR Frame and MRB / WAS Tank Concrete Repair.** The three MBR tanks reportedly all require repair; for now, the WAS tanks do not. Repairs can only be undertaken when the tanks are empty; a minimum of two MBR trains and one WAS tank must remain operational, consequently the repairs must be undertaken sequentially. Fast-curing products will be essential to reduce down time for each tank.

    - The empty MBR tank concrete should be repaired first. Then the components required for the operation of a 3rd MBR tank should be installed including piping, valves, aeration system, permeate/backpulse pump, and controls upgrades. The MBR cassette from Tank No. 1 could then be transfer to the new tank, allowing Tank No. 1 to be taken out of serviced and refurbished. Following a similar sequence, the refurbishment of the 2nd MBR Tank could then be undertaken.

    - If required, the WAS tanks would also have to be taken out of service sequentially to allow the tanks to be refurbished. Once the piping is in place, all MBR sludge would be directed to one WAS tank; thus, centrifuge dewatering rates would have to increase to accommodate the smaller retention volume.

- **S.9 Replacement of the Existing Centrifuge:** Upsizing the existing centrifuge would allow more sludge to be processed during normal operating hours. This would facilitate consolidation of

WAS in one WAS tank, resulting in reduced retention time, while the 2nd tanks is being refurbished.

- **S.10 Addition of 3rd MBR train.** Once the concrete has been repaired in the three MBR tanks and WAS tanks, then a 3rd MBR cassette could be added, providing redundancy.

- **S.11 Process Building Upgrades**. Add air intake filters to the blowers, expand the platform in front of the roll-up door.

- **S.12** Two new MBR cassettes. Future replacement of the two MBR cassettes will be required. Although these are asset management costs, they are included herein to highlight cost considerations and a reminder that the replacement of the existing would increase the capacity of the system and reduce the blower costs, based on the new technology that is currently available.

- **L.1 Twinning of the Influent Forcemain**. Timing of this project would likely be dictated by timing of the dyke project, to reduce costs related to digging up the same stretch of ground.

- **L.2 Plant Electrical Upgrades -** *electrical study ongoing to identify preferred solution*

- **L.3 New Outfall**: The key driver for this project will a combination of plant influent flows and rising lake levels.

- **L.6 Headworks Upgrades** – New Screen and grit removal systems. The key driver for this project will likely be the lifespan and maintenance issues related to the existing screen system. Ideally this project would be undertaken before the addition of the MBBR tanks to reduce the amount of grit which is conveyed through the remainder of the treatment facility.

6.1.1.2. Sequential Projects (SP)

The sequential projects refers only to the projects in that group. The sequential project groups do not need to be undertaken in order, e.g., the first project in group SP.2 (addition of the centrifuge) can be undertaken before both projects in group SP.1 (MBBR addition and Bioreactor refurbishment).

These two projects should be undertaken sequentially:

- **S.13 Addition of a new operations building** The operations should be relocated to the north end of the site. This would allow construction of the MBBR trains adjacent to the existing bioreactor without disrupting operations.

- **L.4 Addition of two MBBR trains**. After relocation of the Operations building, process equipment could be installed closer to the MBBR trains and transfer pumps can be constructed.

- **L.5 Refurbishment of the Bioreactor**. Once the treatment process is operational, refurbishment of the existing bioreactor can be undertaken during the dry season and when the lake water levels are low to avoid infiltration of lake water during desludging of the tank and curing of the concrete repairs.

These projects should be undertaken sequentially:

- **SP.3a Negotiation of New Permit:** The new permit requirements should be confirmed with the authorities. Based on recent permits in other jurisdictions, permits have not required disinfection post-MBR systems. A third standby MBR train may be required first before authorities consider a possible dispensation of a UV system.

- **SP.3b UV System Upgrade:** The need for the UV system should be confirmed first.

Based on the above, a proposed sequence of work was developed. Costs for each project were broken out between engineering and capital, assuming the engineering design would be undertaken the year before the construction works.

The schedule is presented in Figure 29, and the annual cost are summarized in Figure 30. The cost estimates, based on 2025 dollars, were escalated by 3% annually, based on the proposed timing of the engineering phase. Costs exclude operating and maintenance costs.

Based on funding opportunities and other considerations, the schedule can easily be modified to suit unforeseen changes or opportunities.

7. Cost Estimate

The following section describes the expected capital construction costs associated with the proposed upgrades.

7.1. CONSTRUCTION COSTS

McElhanney has prepared an opinion of probable capital cost estimate based on historical pricing and current trends in pricing work and 2025 equipment quotes. The definition of each class of estimate is described as:

- Class “A” estimate: this is a detailed estimate based on the quantity take-off from final drawings and specifications. It is used to evaluate tenders or as a basis of cost control during day-labour construction. It should be noted that even 3-month-old pre-tender Class A cost estimates are falling short of keeping up with the current market price increases.

- Class “B” estimate: this is prepared after site investigations and studies have been completed and the major systems are defined. It is based on the project brief and preliminary design. It is used for obtaining effective project approval and for budgetary control.

- Class “C” estimate: this is prepared with limited site information and is based on probable conditions affecting the project. It represents the summation of all identifiable project elemental costs and is used for program planning to establish a more specified definition of client needs and to obtain preliminary project approval.

- Class "D" estimate: this is a preliminary estimate which, due to limited site information in respect to the project focus, indicates the approximate magnitude of cost of the proposed project, derived from lump sum or unit costs for a similar project. It may be used in developing long term capital plans and for preliminary discussion of proposed capital projects.

For these project estimates, based on available information and level of accuracy, a Class "D" cost estimate was prepared. To each cost summary, as an allowance for unforeseen elements of cost that generally covers small design changes, small quantity changes, central bank target inflation rate, and small estimating changes which cannot be identified prior to construction, a contingency should be applied. Normally a 40% contingency would be applied to the Class D cost estimate.

Based on Client instructions, McElhanney has not applied a 40% contingency and therefore reminds the reader that a 40% contingency should be applied to all capital cost estimates.

7.2. ENGINEERING ALLOWANCE

The engineering allowance applied to each project is an estimate of the level of effort required to further develop the preferred option through detailed design to issuance of construction drawings and includes allowances for the typical levels of effort involved with contract management, construction supervision, record drawings and commissioning, which are typically completed by the owner/contractor with support from the engineering firm of record. The allowance is based on approximate costs for similar projects, as well as the Association of Consulting Engineering Companies British Columbia Fee Guidelines.

Typical engineering fee allowance as a percentage of project capital cost can vary from 20% for smaller projects to 13.3% for large scale water and sewer projects. A 15% Engineering Fee Allowance should be added to any project not being undertaken in-house.

7.3. PROJECT COST ESTIMATE

The projects identified in this report are summarized in Table 30. Projects were divide based on funding with small projects being funded through the annual operation reserves and larger complex projects likely to be funded through grants. In addition, projects likely to be eligible for DCC funding where identified.

Cost estimates in Table 31 do not include a contingency allowance; a 40% contingency allowance should be added to all costs plus a suggested annual 3% escalation fee, depending on the final timing of the project. If engineering is required for the project, a separate column is included based on an estimated fee of 15% engineering. All costs and fees are rounded up to the closest $1000.

The detailed cost estimates for the projects are provided in Appendix B based on 2025 estimates.

Key assumptions related to the cost estimate are as follows:

- Quotes were solicited from equipment suppliers for key process components in 2025.
- Other estimates were based on recent tenders for similar projects and pro-rated based on flows
- Including the forcemain twinning as part of the dyke upgrade may allow for some costs savings.

Table 31. Estimated Cost Summary Excl. Contingency, Engineering, and Escalation costs.

Capital Costs and Project Schedule

Village of Harrison Hot Spring

WWTP - Master Plan - Capital Improvements

Project No.	TASK DESCRIPTION	DCC Eligibility	Short-Term 0 - 5 years		Medium-Term 5 - 10 years		Long-Term 10 - 20 years	
	Wastewater Treatment Plant							
	ANNUAL OPERATING PROJECTS		Engineering	Capital	Engineering	Capital	Engineering	Capital
S.1	Influent Flow Analysis		✓					
S.3	New MWR Permit - *Allowance*		$ 200,000					
S.4	Chemical Storage Upgrades -Ventilation		$ 8,000	$ 35,000				
S.5	Process Building - Floor Repair - In-House			$ 17,000				
S. 6	Process Building - Chain at Overhead Door for Safety, In-House			$ 300				
S.7	Addition of 3rd MBR Frame - In-House	✓		$ 62,000				
S.8	MBR and WAS Tank Concrete Repair, In-House			$ 126,000				
S.9	New Centrifuge and Building and Assoc. Equipment	✓	$ 160,000	$ 750,000				
S.10	Process Building - Blower Room - Air Intake Filters				$ 3,000	$ 10,000		
S.11	Addition of 3rd MBR Cassette, Piping, Controls, EI&C, In-House	✓		$ 800,000				
S.12	Replace / Improve Trains 1 & 2 MBR Modules, In-House *Assumed 3% inflation over 9 years (Repair & Maintenance)*					$ 1,290,000		
S.13	Operations Building							$ 1,240,000
	SUBTOTAL		**$**	**2,158,300**	**$**	**1,303,000**	**$**	**1,240,000**
	GRANT OPPORTUNITY PROJECTS							
L.1	Influent Forcemain Twinning - M.R. Pump Station to WWTP *Incl. power, water supply, fibre and spare conduit* *Costs could be lower if incorporated with the Dyke Upgrade contract*	✓	$ 210,000	$ 1,000,000				
L.2	Plant Electrical Upgrades		$ 80,000	$ 373,000				
L.3	New Outfall				$ 80,000	$ 334,000		
L.4	Investigate New Biological Treatment Options - *Allowance* *Capital Cost TBD based on current technologies*						$ 100,000	TBA
L.5	Investigate Desludging and Repair of Existing Bioreactor for EQ Tank Use - *Allowance* *Costs TBD based on extent of reactor reused and current construction methodologies. Combine with New Biological Treatment Options Investigation*	✓					$ 50,000	TBA
L.6	Headworks Upgrades (Screen and Grit Removal)						$ 230,000	$ 1,100,000
	SUBTOTAL		**$**	**1,663,000**	**$**	**414,000**	**$**	**1,480,000**
	TOTAL ESTIMATED COSTS PER TIME FRAME		**$**	**3,821,300**	**$**	**1,717,000**	**$**	**2,720,000**
	FURTHER INVESTIGATION REQ'D							
A.1	New UV Disinfection System *Confirm new permit requirments*							
	Assumptions (See Report for more details)							
	Inflation: Inflation is not included. Suggest using 3% / year after 2025 or actual value, if known.							
	Engineering: 15% for projects which require engineering. In-house projects do not include engineerig							
	Class D Estimate: 40% contingency to be added to all projects							

Long-Term Project Considerations

Although some of the recommended upgrades are relatively straightforward, advances in technology and construction methodologies, new permit requirements, and layout options not investigated herein, may impact the selection of equipment or processes in the future.

The suggested technologies, such as the headworks equipment and new biological treatment (MBBR) trains, should be considered as a guideline until implementation is imminent, and additional options can be evaluated and cost estimates refined. With this in mind, allowances to investigate biological treatment options, and resulting refurbishments needs and associated costs have been included rather than providing a fixed estimate.

Note that at the time of finalizing this report, additional layout and construction options were already being investigated by Operations Staff.

8. Closure

McElhanney is pleased to provide this technical memorandum for the proposed upgrade plan to meet current issues and future effluent requirements to the Village of Harrison Hot Springs.

If there are any questions or concerns, please contact the undersigned.

Sincerely,

Prepared by:

Michael Panidisz, P. Eng
Design Engineer
mpanidisz@mcelhanney.com
778-646-2522

Reviewed by:

Karen Sutherland, M. Eng, P. Eng
Wastewater Engineer
ksutherland@mcelhanney.com
778-476-8136

Prepared by:

Michael Barnes, P. Eng
Electrical Engineer
mbarnes@mcelhanney.com
778-693-2220

Appendix A

Statement of Limitations

Statement of Limitations

Use of this Report. This report was prepared by McElhanney Ltd. ("**McElhanney**") for the particular site, design objective, development, and purpose (the “**Project**”) described in this report and for the exclusive use of the client identified in this report (the “**Client**”). The data, interpretations and recommendations pertain to the Project and are not applicable to any other project or site location and this report may not be reproduced, used, or relied upon, in whole or in part, by a party other than the Client, without the prior written consent of McElhanney. The Client may provide copies of this report to its affiliates, contractors, subcontractors, and regulatory authorities for use in relation to and in connection with the Project provided that any reliance, unauthorized use, and/or decisions made based on the information contained within this report are at the sole risk of such parties. McElhanney will not be responsible for the use of this report on projects other than the Project, where this report or the contents hereof have been modified without McElhanney’s consent, to the extent that the content is in the nature of an opinion, and if the report is preliminary or draft. This is a technical report and is not a legal representation or interpretation of laws, rules, regulations, or policies of governmental agencies.

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Information from Client and Third Parties. McElhanney has relied in good faith on information provided by the Client and third parties noted in this report and has assumed such information to be accurate, complete, reliable, non-fringing, and fit for the intended purpose without independent verification. McElhanney accepts no responsibility for any deficiency, misstatements or inaccuracy contained in this report as a result of omissions or errors in information provided by third parties or for omissions, misstatements or fraudulent acts of persons interviewed.

Effect of Changes. All evaluations and conclusions stated in this report are based on facts, observations, site-specific details, legislation and regulations as they existed at the time of the report preparation. Some conditions are subject to change over time, and the Client recognizes that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site may substantially alter such evaluations and conclusions. McElhanney should be requested to re-evaluate the conclusions of this report and to provide amendments as required prior to any reliance upon the information presented herein upon any of the following events: a) any changes (or possible changes) as to the site, purpose, or development plans upon which this report was based, b) any changes to applicable laws subsequent to the issuance of the report, c) new information is discovered in the future during site excavations, construction, building demolition or other activities, or d) additional subsurface assessments or testing conducted by others.

Independent Judgments. McElhanney will not be responsible for the independent conclusions, interpretations, interpolations and/or decisions of the Client, or others, who may come into possession of this report, or any part thereof. This restriction of liability includes decisions made to purchase, finance or sell land or with respect to public offerings for the sale of securities.

Construction Cost Estimates. This construction cost estimate has been prepared using the design and technical information currently available, and without the benefit of Survey, Geotechnical, Environmental, or other disciplines, information. Furthermore, McElhanney cannot predict the competitive environment, weather or other unforeseen conditions that will prevail at the time that contractors will prepare their bids. The cost estimate is therefore subject to factors over which McElhanney has no control, and McElhanney does not guarantee or warranty the accuracy of such estimate.

Appendix B

Detailed Cost Estimates

Item	Description	No. units	Unit	Unit cost	Base Cost
	Small Projects				
S.4	***Chemical Storage Upgrades - Safety***				
	Ventilation Upgrades	1	LS	$ 35,000	$ 35,000
	SubTotal				***$ 35,000***
	Contingency 40%	***40%***			*$ 14,000*
	Commissioning 5%	***5%***			*$ 3,000*
	Chemical Storage Upgrades - Safety Capital Costs				***$ 52,000***
	Engineering 15%	***15%***			*$ 8,000*
	Chemical Storage Upgrades - Safety Construction Costs				***$ 60,000***
	Estimated Construction Cost (Inflation 0%/yr over 2 yrs)	***0%***	***2 yrs***		***$ 60,000***
S.5	***Process Building - Integrity***				
	Basement Floor Repair and Sealing	80	m2	210	$ 16,800
	SubTotal				***$ 16,800***
	Contingency 40%	***40%***			*$ 7,000*
	Commissioning 0%	***0%***			*$ -*
	Process Building - Integrity Capital Costs				***$ 23,800***
S.8	***MBR and WAS Tanks Refurbishment***				
	MBR and WAS Tanks Refurbishment	386	m2	$ 260	$ 101,000
	Cleaning / Desluding of Tank - Allowance for Sequential Work - 5 tanks	5	ea	$ 5,000	$ 25,000
	SubTotal				***$ 126,000***
	Contingency 40%	***40%***			*$ 51,000*
	Commissioning 0%	***0.0%***			*$ -*
	MBR and WAS Tanks Refurbishment Capital Costs				***$ 177,000***
	Engineering 0%	***0%***			*$ -*
	MBR and WAS Tanks Refurbishment Construction Costs				***$ 177,000***
	Estimated Construction Cost (Inflation 0%/yr over 9 yrs)	***0%***	***9 yrs***		***$ 180,000***
S. 6, S.10	***Process Building - Operational and Safety Improvements***				
	Blower Room Air Intake	4	LS	$ 2,000	$ 8,000
	Chain or Gate at Overhead Door	1	LS	$ 300	$ 300
	SubTotal				***$ 10,000***
	Contingency 40%	***40%***			*$ 4,000*
	Commissioning 5%	***5%***			*$ 1,000*
	Process Building - Operational and Safety Improvements Capital Costs				***$ 15,000***
	Engineering 15%	***15%***			*$ 3,000*
	Process Building - Operational and Safety Improvements Construction Costs				***$ 18,000***
S.11	***Process Expansion - Medium-Term***				
	Add 3rd MBR Cassettes - ZW500EV 460ft2	1	LS	$ 713,000	$ 713,000
	Addtion of 3rd backpulse pump + Piping modifications	1	LS	$ 85,000	$ 85,000
	SubTotal				***$ 800,000***
	Contingency 40%	***40%***			*$ 320,000*
	Commissioning 2%	***2%***			*$ 23,000*
	Process Expansion - Medium-Term Capital Costs				***$ 1,143,000***
S.12	***Process Upgrade - Long Term***				
	Replace MBR Cassettes - ZW500EV 460ft2 - Two Tanks	1	LS	$ 713,000	$ 713,000
	Installation	35%		$ 713,000	$ 249,550
	SubTotal				***$ 962,550***
	Contingency 0%	***0%***			*$ -*
	Commissioning 2%	***2.0%***			*$ 20,000*
	Process Upgrade - Long Term Capital Costs				***$ 982,550***
	Engineering 0%	***0%***			*$ -*
	Process Upgrade - Long Term Construction Costs				***$ 982,550***
	Estimated Construction Cost (Inflation 0%/yr over 9 yrs)	***3%***	***9 yrs***		***$ 1,290,000***

Item	Description	No. units	Unit	Unit cost	Cost
	S.13 New Operations Bldg				
1	***Front End***				***$ 140,000***
	Mobilization and Demobilization - 10% Construction	10%	%	$ 1,100,000	$ 110,000
	Insurance	1.5%	%	$ 1,100,000	$ 20,000
	Traffic Control	0.0%	%	$ 1,100,000	$ -
	Environmental Plan	1.0%	%	$ 1,100,000	$ 10,000
2	***Civil Site Works - Common***				***$ 132,000***
	Site Preparation (Clearing, Grubbing, and Grading)	300	m3	$ 50	$ 15,000
	Landscaping - Allowance	1	LS	$ 15,000	$ 15,000
	Road Improvements and Walkways btw Buildings	1	LS	$ 20,000	$ 20,000
	Base Preparation - Operations Building (100m2)	100	m^2	$ 40.00	$ 4,000
	Concrete Base - Operations Building + MBBR	30	m^3	$ 2,600.00	$ 78,000
3	***Piping to / from New Building***				***$ 137,000***
	Water and Sanitary piping, instruments and valves	1	LS	$ 100,000	$ 100,000
	Delivery @ 2%	2%	LS	$ 100,000	$ 2,000
	Equipment Installation @ 35%	35%	LS	$ 100,000	$ 35,000
4	***Operations Building***				***$ 810,000***
	Bldg: Lab, Office, Lunchroom, Washroom, Storage, includes HVAC and lighting.	100	m2	$ 6,000	$ 600,000
	Roll-Up Door for Storage Room	1	LS	$ 10,000	$ 10,000
	Washroom/Sanitary System	1	LS	$ 100,000	$ 100,000
	Furnishings - Allowance	1	LS	$ 100,000	$ 100,000
5	***Electrical, Instrumentation and Controls***				***$ 16,000***
	Power Supply To Building	1.5%	%	$ 1,079,000	$ 16,000
	SubTotal (1) "Front End"				***$ 140,000***
	SubTotal (2 - 5)				***$ 1,100,000***
	SubTotal				***$ 1,240,000***
	Contingency 40%	***40%***			*$ 500,000*
	Commissioning 0%	***0%***			*$ -*
	S.13 New Operations Bldg Capital Costs				***$ 1,740,000***
	Engineering 15%	***15%***			*$ 261,000*
	S.13 New Operations Bldg Construction Costs				***$ 2,001,000***

Item	Description	No. units	Unit	Unit cost	Cost
	Forcemain Replacement - LS#1 to M.R. Pump Station				
1					***$ 150,000***
	Mobilization and Demobilization	10%	%	$ 850,000	$ 85,000
	Insurance	1.5%	%	$ 850,000	$ 13,000
	Traffic & Pedestrian Control	3.0%	%	$ 850,000	$ 26,000
	Environmental Plan	3.0%	%	$ 850,000	$ 26,000
2	***Twin Forcemain***				***$ 614,486***
	2-250mm DR17 HDPE Pipe (joint trench)	483.6	LM	$ 600	$ 290,160
	Water supply line, 50 mm	483.6	LM	$ 100	$ 48,360
	Water reuse line, 75 mm	483.6	LM	$ 125	$ 60,450
	Blowdown Chamber	1	LS	$ 40,500	$ 40,500
	Air Release Valve	1	LS	$ 25,000	$ 25,000
	250mm 90° Bend c/w Thrust Block & Restraints	6	each	$ 3,500	$ 21,000
	250mm Gate Valve c/w Thrust Block & Restraints	4	each	$ 5,500	$ 22,000
	200x250mm Reducer c/w Restraints	4	each	$ 3,500	$ 14,000
	Tie-in to Ex. 2-200mm Forcemain	2	LS	$ 5,000	$ 10,000
	Abandon Ex. Forcemain + Disconnections	1	LS	$ 15,000	$ 15,000
	Asphalt Sawcut & Removal c/w Offsite Disposal	300	m2	$ 40	$ 12,000
	50mm Asphalt Pavement	300	m2	$ 75	$ 22,500
	150mm 19mm- Granular Base	300	m2	$ 35	$ 10,500
	Landscaping	1150.8	m2	$ 20	$ 23,016
3	***Electrical - Power Line Relocate Below Ground***				***$ 226,437***
	2-100mm PVC Electrical Conduit	483.6	Lm	$ 141	$ 68,227
	Electrical Wiring	483.6	Lm	$ 165	$ 79,794
	Optic Fibre	483.6	Lm	$ 60	$ 29,016
	Pull Boxes	4	EACH	$ 2,600	$ 10,400
	Tie-in to Power Pole	1	LS.	$ 12,000	$ 12,000
	Tie-in to Miami River Pump Station	1	LS.	$ 17,000	$ 17,000
	Remove old power lines	1	LS.	$ 10,000	$ 10,000
	SubTotal (1) "Front End"				***$ 150,000***
	SubTotal (2 - 3)				***$ 850,000***
	SubTotal				***$ 1,000,000***
	Contingency 40%	***40%***			*$ 400,000*
	Commissioning 0%	***0%***			*$ -*
	Forcemain Replacement - LS#1 to M.R. Pump Station Capital Costs				***$ 1,400,000***
	Engineering 15%	***15%***			*$ 210,000*
	Forcemain Replacement - LS#1 to M.R. Pump Station Construction Costs				***$ 1,610,000***

Option	Description	No. units	Unit	Unit cost	Cost
	L.2 Plant Electrical Upgrades				
1	***Front End***				***$ 23,000***
	Mobilization and Demobilization - Calculated	3.0%	%	$ 350,000	$ 11,000
	Insurance	1.5%	%	$ 350,000	$ 6,000
	Traffic Control	0.5%	%	$ 350,000	$ 2,000
	Environmental Plan	1.0%	%	$ 350,000	$ 4,000
3	***Electrical and Controls Upgrades***				***$ 350,000***
	Site Power Supply Upgrades	1	LS.	$ 264,000	$ 264,000
	Electrical Upgrades - Power Supply to Equipment	1	LS.	$ 66,000	$ 66,000
	Control Upgrades – Tie into Scada	1	LS.	$ 20,000	$ 20,000
	SubTotal (1) "Front End"				***$ 23,000***
	SubTotal (2 - 3)				***$ 350,000***
	SubTotal				***$ 373,000***
	Contingency 40%	***40%***			*$ 149,200*
	Commissioning 2%	***2%***			*$ 11,000*
	L.2 Plant Electrical Upgrades Capital Costs				***$ 540,000***
	Engineering 15%	***15%***			*$ 81,000*
	L.2 Plant Electrical Upgrades Construction Costs				***$ 621,000***

Item	Description	No. units	Unit	Unit cost	Cost
	L. 3 Outfall Upgrades				
1	***Front End***				***$ 72,000***
	Mobilization and Demobilization	8.0%	%	$ 261,266	$ 21,000
	Insurance	4.0%	%	$ 261,266	$ 11,000
	Traffic Control	0.0%	%	$ 261,266	$ -
	Environmental Plan	15.0%	%	$ 261,266	$ 40,000
2	***Outfall Upgrades***				***$ 252,266***
	Chamber Modifications - New 1.8m ID Manhole (3-4m depth) c/w connection to discharge & outfall lines, guide rails, piping, fittings	1	LS	$ 27,000	$ 27,000
	Air Release / Vacuum Valve	1	LS	$ 4,000	$ 4,000
	NP3102LT 423 Pumps (Duplex system)	2	Each	$ 17,000	$ 34,000
	Controls, level sensor, electronics	1	LS	$ 43,200	$ 43,200
	Flow Monitoring Station (River Discharge)?	1	LS	$ 25,000	$ 25,000
	Need for By-Pass Pumping During Construction	1	LS	$ 50,000	$ 50,000
	Delivery @ 2%	0.02	LS.	$ 183,200	$ 3,664
	Installation @ 35%	0.35	LS.	$ 186,864	$ 65,402
3	***Electrical Works***				***$ 9,000***
	Electrical Upgrades - Power Supply to Equipment	1.0%	%	$ 252,266	$ 3,000
	Control Upgrades – Tie into Scada	2.5%	%	$ 252,266	$ 6,000
	SubTotal (1) "Front End"				***$ 72,000***
	SubTotal (2 - 3)				***$ 261,266***
	SubTotal				***$ 334,000***
	Contingency 40%	***40%***			*$ 133,600*
	Commissioning 3%	***3%***			*$ 15,000*
	L. 3 Outfall Upgrades Capital Costs				***$ 490,000***
	Engineering 15%	***15%***			*$ 80,000*
	L. 3 Outfall Upgrades Construction Costs				***$ 570,000***

Item	Description	No. units	Unit	Unit cost	Cost
	L. 4 Secondary Treatment Redundancy Upgrades - MBBR				
1	***Front End***				***$ 530,000***
	Mobilization and Demobilization - 10% Construction	10%	%	$ 4,300,000	$ 430,000
	Insurance	1.5%	%	$ 4,300,000	$ 60,000
	Traffic Control	0.0%	%	$ 4,300,000	$ -
	Environmental Plan	1.0%	%	$ 4,300,000	$ 40,000
2	***Civil Site Works - Common***				***$ 179,000***
	Site Preparation (Clearing, Grubbing, and Grading)	300	m3	$ 50	$ 15,000
	Base Preparation - MBBR (200m2)	200	m²	$ 40.00	$ 8,000
	Concrete Base - MBBR	60	m³	$ 2,600.00	$ 156,000
3	***Process - Secondary Treatment Upgrades Based on MBBR Upgrade***				***$ 3,928,500***
	Extend Aeration Header to MBBR Tanks	80	lm	$ 700	$ 56,000
	Improve RAS control - flow meter and flow control valve	1	LS	$ 25,000	$ 25,000
	Reroute RAS piping to MBBR	80	lm	$ 650	$ 52,000
	New pumps from MBBR to MBR	2	LS	$ 50,000	$ 100,000
	Existing Transfer Pump discharging piping modifications to new bioreactor	50	lm	$ 650	$ 32,500
	MBBR equipment quote	1	LS	$ 1,216,000	$ 1,216,000
	MBBR Tanks (Reinforced Concrete) Process Tanks and Slab, Wet Well	350	m3	$ 2,600	$ 910,000
	New mixing system in EQ tank	1	LS	$ 175,000	$ 175,000
	Piping, instruments and valves	1	LS	$ 100,000	$ 100,000
	Misc. Steel, Platforms, supports	1	LS	$ 200,000	$ 200,000
	Delivery @ 2%	2%	LS	$ 2,866,500	$ 58,000
	Equipment Installation @ 35%	35%	LS	$ 2,866,500	$ 1,004,000
4	***Electrical, Instrumentation and Controls***				***$ 124,000***
	Site Power Supply Upgrades	1.5%	%	$ 4,107,500	$ 62,000
	Electrical Upgrades - Power Supply to Equipment	0.5%	%	$ 4,107,500	$ 21,000
	Control Upgrades	1.0%	%	$ 4,107,500	$ 41,000
	SubTotal (1) "Front End"				***$ 530,000***
	SubTotal (2 - 5)				***$ 4,300,000***
	SubTotal				***$ 4,830,000***
	Contingency 40%	***40%***			*$ 1,940,000*
	Commissioning 1%	***1%***			*$ 68,000*
	L. 4 Secondary Treatment Redundancy Upgrades - MBBR Capital Costs				***$ 6,838,000***
	Engineering 15%	***15%***			*$ 1,026,000*
	Construction Costs				***$ 7,864,000***

Item	Description	No. units	Unit	Unit cost	Cost
L.5 Desludging and Repair of Existing Bioreactor for Use as EQ Tank					
1	***Front End***				***$ 180,000***
	Mobilization and Demobilization - 10% Construction	10%	%	$ 1,269,000	$ 130,000
	Insurance	1.5%	%	$ 1,269,000	$ 20,000
	Traffic Control	0.0%	%	$ 1,269,000	$ -
	Environmental Plan	2.0%	%	$ 1,269,000	$ 30,000
2	***Bioreactor Cleaning and Refurbishment***				***$ 133,000***
	3rd Party Sludge Removal - Desludging Aerated Bioreactor	1	LS	$ 47,000	$ 47,000
	Decommissioning of Aeration Equipment	1	LS	$ 25,000	$ 25,000
	French Drain around Bioreactor	1	LS	$ 61,000	$ 61,000
3	***Bioreactor Repairs***				***$ 1,136,000***
	Bioreactor Base and Sides Waterproofing *RJ Watson Zedseal + Sauereisen 220 Joint Filler +* *Sauereisen 210S SewerGard sprayable Liner*	2275	m2	$ 450	$ 1,024,000
	Addition of Mixers	3	LS	$ 35,000	$ 105,000
	Electrical Modifications	1	LS	$ 6,500	$ 7,000
	SubTotal (1) "Front End"				***$ 180,000***
	SubTotal (2 - 3)				***$ 1,269,000***
	SubTotal				***$ 1,449,000***
	Contingency 40%	***40%***			*$ 579,600*
	Commissioning 1%	***1%***			*$ 21,000*
	L.5 Desludging and Repair of Existing Bioreactor for Use as EQ Tank Capital Costs				***$ 2,050,000***
	Engineering 15%	***15%***			*$ 308,000*
	L.5 Desludging and Repair of Existing Bioreactor for Use as EQ Tank Construction Costs				***$ 2,360,000***

Item	Description	No. units	Unit	Unit cost	Cost
	L. 6 Headworks Upgrades				
1	***Front End***				***$ 70,000***
	Mobilization and Demobilization	4.0%	%	$ 1,030,000	$ 42,000
	Insurance	2.0%	%	$ 1,030,000	$ 21,000
	Traffic Control	0.0%	%	$ 1,030,000	$ -
	Environmental Plan	1.0%	%	$ 1,030,000	$ 11,000
2	***Process - Headworks***				***$ 902,210***
	Abandon and remove existing equipment	1	LS.	$ 10,000	$ 10,000
Option 1	Option 1 - Fine Screen Package	1	LS.	$ 242,000	$ 242,000
Option 1	Option 1 - Grit Removal Package	1	LS.	$ 310,000	$ 310,000
Option 2	*Veolia Combination Fine Screen & Grit Removal Chamber*	*0*	*LS.*	*$ 424,000*	*$ -*
Option 2	*1/2" PEX Water Service for Wash Water c/w Valves & Fittings*	*0*	*LS.*	*$ 15,000*	*$ -*
	Inlet and outlet piping modifications	1	LS.	$ 10,000	$ 10,000
	Grating over Inlet Channel Replacement	12	lm	$ 560	$ 6,800
	Base Preparation	21	m^2	$ 40	$ 1,000
	Concrete Slab	4	m^3	$ 2,600	$ 10,400
	Steel Support Frame for Equipment	1	LS.	$ 25,000	$ 25,000
	Structured Roof	1	LS.	$ 40,000	$ 40,000
	Delivery @ 2%	0.02	LS.	$ 655,200	$ 13,104
	Installation @ 35%	0.35	LS.	$ 668,304	$ 233,906
3	***Electrical and Controls Upgrades***				***$ 121,798***
	Site Power Supply Upgrades	6%	%	$ 902,210	$ 54,133
	Electrical Upgrades - Power Supply to Equipment	4%	%	$ 902,210	$ 36,088
	Control Upgrades – Tie into Scada	3.5%	%	$ 902,210	$ 31,577
	SubTotal (1) "Front End"				***$ 70,000***
	SubTotal (2 - 3)				***$ 1,030,000***
	SubTotal				***$ 1,100,000***
	Contingency 40%	***40%***			*$ 440,000*
	Commissioning 1%	***1%***			*$ 16,000*
	L. 6 Headworks Upgrades Capital Costs				***$ 1,556,000***
	Engineering 15%	***15%***			*$ 234,000*
	L. 6 Headworks Upgrades Construction Costs				***$ 1,790,000***

Appendix C

Biosolids Summary

As can be seen from the table below, the sludge processed from the Harrison Hot Springs WWTP is classified as Class B Biosolids. Exceedances to trace metals *Arsenic*, *Selenium*, *Zinc* and occasionally *Molybdenum* would need to be measured and managed to improve the sludge quality to Class A. Potable water data provided by VHHS does not indicate notably high levels of these parameters. WWTP staff have noted possible sources of these metals could be the water from Hot Springs Source, campground septic waste, or debris entering the wastewater.

Wastewater Parameters	Units	Biosolids Sample 2025-02-26	Land Application Class A Biosolids/ Compost	Land Application Biosolids Growing Medium	Land Application Class B Biosolids/ Compost
Chemical Oxygen Demand	mg/L	-	Must be Tested	Must be Tested	Must be Tested
Biochemical Oxygen Demand (5-day)	mg/L	73.6	Must be Tested	Must be Tested	Must be Tested
Nitrate (as N)	mg/L	-	Must be Tested	Must be Tested	Must be Tested
Total Phosphorus	mg/L	2.55	Must be Tested	Must be Tested	Must be Tested
Total Kjeldahl Nitrogen (TKN) < 0.6	% wt	2.55		< 0.6%	
Carbon to Nitrogen Ratio (C:N)	g/g	2.64		> 15:1	
Organic Matter	% wt	-		< 15%	
Total Recoverable Metals					
Arsenic	µg/g	**16.2**	13	13	75
Cadmium	µg/g	1.01	3	1.5	20
Chromium	µg/g	12.8	100	100	1,060
Cobalt	µg/g	2.43	34	34	150
Copper	µg/g	390	400	150	2,200
Lead	µg/g	16.2	150	150	500
Mercury	µg/g	0.39	2	0.8	15
Molybdenum	µg/g	**4.91**	5	5	20
Nickel	µg/g	15.5	62	62	180
Selenium	µg/g	**3.66**	2	2	14
Zinc	µg/g	**718**	500	150	1850
Dissolved Metals					
Hexavalent Chromium	µg/g	-	75	75	75
Hot Water-Soluble Boron	µg/g	-	20	20	20
Fecal Coliform	MPN/ 100 mL	**6030**	1,000	2,000,000	2,000,000

Notes:

1. **Red** values indicate exceedances of OMRR Class A Biosolids

Appendix D

MBR Effluent Quality

MBRs Ability to Meet Effluent Discharge Requirements without Downstream Disinfection

Kathleen Peach[1], Stephen Katz[2]
Veolia Water Technologies, Oakville, Canada
[1]Email: Katie.Peach@veolia.com
[2]Email : Stephen.Katz@veolia.com

Abstract:

Data collected from 20 MBR facilities to provide a benchmark for the long term microbial and pathogen removal performance of full scale MBR installations. The facilities all use Veolia's ZeeWeed 500* ultrafiltration membranes and were selected to include varying membrane age, configuration, hydraulic capacity and geographical location.

The geometric mean concentrations in the effluent prior to any additional disinfection across all the plants examined in this study for *E. coli*, fecal coliforms, and total coliforms were 1.2 CFU/100mL, 1.3 CFU/100mL, and 1.6 CFU/100mL, respectively. 95th–percentile values were 4.0 CFU/100mL for *E. coli*, 4.0 CFU/100mL for fecal coliforms, and 14CFU/100mL for total coliforms. These results demonstrate the ability of MBR technology to produce effluent that meets typical regulatory limits for surface water discharge and some non-potable reuse.

Case studies of several facilities are shared that operate without any further disinfection such as UV or chlorination, relying on the MBR to achieve the required effluent discharge requirements.

Keywords

Membrane bioreactor, disinfection, pathogen removal, wastewater treatment

Introduction

Microbial parameters are generally included in treated wastewater effluent discharge regulations to safeguard public health from pathogenic microorganisms in water bodies. Conventional activated sludge (CAS) processes in municipal wastewater treatment require a separate disinfection stage to meet microbial standards, because the CAS process cannot reliably remove indicator bacteria. This is because microbial removal is dependent on particles settling due to gravity, which is a process that is subject to variability in effectiveness. However, membrane bioreactor (MBR) systems equipped with ultrafiltration (UF) membranes offer a more reliable solution through size exclusion. In MBRs, coliform removal occurs either through direct membrane filtration or when bacteria attach to mixed liquor solids that are too large to pass through the membrane. The membrane's effectiveness is further enhanced by the formation of a dynamic filtration layer on its surface (Oppenheimer et al., 2012).

In a 2015 study, detailed in The Case for Removing Post-Disinfection from MBR Treatment (Katz et. al, 2015), 10 full-scale MBR installations were evaluated on their ability to provide microbial removal, testing for *E. coli* bacteria, total coliforms, and fecal coliforms in the MBR effluent. An extensive dataset was analyzed and demonstrated effluent geometric mean concentrations of 1.1 CFU/100mL, 1.6 CFU/100mL, and 2.6 CFU/100mL of *E. Coli*, fecal coliforms, and total coliforms, respectively. This paper expands the dataset to include 10 additional MBR facilities creating a larger data set to be analyzed for MBR’s efficacy of removal of these parameters.

Both U.S. and European authorities have established guidelines for acceptable pathogen levels in public recreational waters. The U.S. EPA's 2012 recreational water quality criteria (RWQC) sets limits for primary contact recreation: *E. coli* should not exceed 100 CFU/100mL (30-day geometric mean) and 320 CFU/100mL (30-day 90th percentile), while Intestinal Enterococci should remain below 30 CFU/100mL (30-day geometric mean) and 110 CFU/100mL (30-day 90th percentile) (USEPA, 2012). The European Union’s 2006 Bathing Water Directive implements similarly stringent standards, particularly for coastal and transitional waters, requiring *E. coli* levels below 250 CFU/100mL and Intestinal Enterococci below 100 CFU/100mL (95th percentile values) for excellent quality classification (European Commission, 2006). For the facilities examined in this study where permits were known, surface water discharge permits typically specify monthly geometric mean or average limits between 23 and 284 CFU/100mL for *E. coli,* fecal or total coliform

Raw wastewater exhibits significant variability in coliform bacteria populations, influenced by multiple factors including daily flow patterns, weather changes, seasonal shifts, and treatment plant operational efficiency (Water Environment Research Federation, 2009). The 2015 study by Katz et. al, analyzed coliform levels in 281 raw wastewater samples, and showed a range of several orders of magnitude, with geometric mean values of 166,000 CFU/100mL, 251,000 CFU/100mL, and 498,000 CFU/100mL for *E. coli*, fecal coliform, and total coliform, respectively. Given that regulatory compliance for microbial contaminants is based on absolute values rather than removal percentages, the treatment system must maintain consistent effectiveness regardless of influent contamination levels.

While this research doesn't examine intestinal enterococcus levels in MBR effluent specifically, previous full-scale studies reveal that enterococcus bacteria were removed to levels similar to, or lower than, *E. coli* in MBR permeate. Furthermore, these studies have shown that fecal enterococcus bacteria tend to be less prevalent in municipal wastewater than *E. coli*. (Zanetti et al. 2010, De Luca et al. 2013).

Methodology

MBR Plant Selection

The original study in 2015 (Katz et. al, 2015) included ten (10) MBR facilities, and this expanded study adds an additional ten (10) facilities to the dataset. This study also adds additional data collected from one of the original facilities. All of the facilities use Veolia's ZeeWeed membrane, in a variety of module and cassette configurations, including the 500C, 500D and 500EV products. The MBR facilities represent a range of geographic locations, hydraulic capacity, and membrane ages, ranging from brand new to 12 years during the sampling period. Plant details are summarized in Table 1.

Sample Collection and Analysis

Samples of MBR permeate prior to any additional disinfection were collected by facility staff members and analyzed using accredited laboratories and standard testing protocols for *E. coli,* fecal coliform, and total coliform. Specific bacteriological sampling at each facility varied depending on the regulatory criteria required for their specific permit.

The detection limit for all analyses was one (1) colony-forming unit (CFU)/100mL or one (1) most probable number (MPN)/100mL of sample, with the following exceptions:

- North Liberty, IA tested fecal coliform with a detection limit of 1.8CFU/100mL, and *E. coli* with a detection limit of 10CFU/100mL
- Linwood, GA tested fecal coliform with a detection limit of 2 CFU/100mL

Most samples were collected as colony forming units (CFU) methods however some data was collected as most probable number (MPN). Although CFU and MPN methods use different approaches to determine the bacterial concentrations in the samples, both have been widely accepted as comparable indicators of the microbial load in a sample.

Table 1: Summary of MBR Study Microbial Sampling Quantities. "n/a" = Not Analyzed, "TC" = Total Coliforms, "FC" = Fecal Coliforms, "EC" = E. coli

SITE	Capacity ADF (MGD)	Membrane Product	Startup Year (replacement year)	Sampling start/end date (dd/mm/yyyy)	# OF SAMPLES ANALYZED		
					PERMEATE (n = 8657)		
					TC	FC	EC
				Total # of Results for each Parameter	***958***	***6211***	***1438***
North Kent (MI, USA)	6	500D	2008 (2018)	11/5/2008 - 4/30/2025	n/a	4429	n/a
Euclid (OH, USA)	22	500D	2020	5/1/2022 - 5/31/2025	n/a	n/a	401
Cranberry (PA, USA)	7.2	500D	2019	7/8/2021 - 4/30/2025	6	40	10
Lloydminster (SK, Canada)	5.5	500EV	2023	1/8/2024 - 12/16/2024	24	n/a	24
Vermilion (AB, Canada)	0.6	500D	2021	1/4/2023 - 5/2/2025	582	584	n/a
North Liberty (IA, USA)	3.3	500D	2009 (2021-2023)	1/13/2013 - 4/17/2025	n/a	123	100
Harrison Hot Springs (BC, Canada)	0.4	500D	2012 (2020)	1/25/2017 - 3/12/2025	n/a	95	n/a
Montechiari (Italy)	2.5	500D, 500EV	2011 (2023)	2/2/2023 - 10/12/2023	n/a	n/a	18
Verziano (Italy)	12.7	500C	2002 (2016)	2/28/2023 - 9/26/2024	n/a	n/a	19
MBR Facility (Belgium)	17.3	500D	2018	1/1/2022 - 6/1/2023	n/a	n/a	15
Beaver Creek (TN, USA)	6	500D	2009	1/29/2019 - 12/23/2019 14/09/2010 - 26/10/2010	n/a 12	n/a n/a	104 12
Bonita Springs (FL, USA)	4.1	500D	2007	10/06/2010 - 22/06/2011	13	73	73
Hutchinson (MN, USA)	1.3	500D	2008	30/03/2010 - 07/11/2011	23	152	151
Kilworth (ON, Canada)	0.34	500D	2008	02/11/2009 - 04/10/2010	30	43	43
Linwood (GA, USA)	4.6	500D	2007	06/07/2010- 22/02/2012	n/a	167	167
Marco Island (FL, USA)	3	500D	2007	14/04/2010- 31/10/2011	227	227	227
Oroszlány (Hungary)	1.32	500D	2004	14/10/2010 - 23/08/2011	41	40	41
Oxford (ON, Canada)	3.6	500D	2008	15/09/2010 - 07/10/2010	n/a	8	8
Peoria (AZ, USA)	10/20	500D	2008	04/05/2010 - 31/05/2011	n/a	107	25
Schofield (HI, USA)	4.2/4.2	500D	2006	10/09/2010 - 29/12/2011	n/a	123	n/a

Results

Summary of *E. coli* in MBR Permeate

Among the total coliform bacteria group, Escherichia coli (EC) stands unique as it is found solely in the intestinal tract of humans and other warm-blooded animals. Its detection in water serves as a reliable indicator of recent fecal contamination and suggests the possible presence of disease-causing organisms including bacteria, viruses, and protozoa. *E. coli* testing provides more accurate risk assessment of pathogenic contamination compared to total or fecal coliform measurements (Stevens *et* al., 2003). Many jurisdictions are reconsidering the approach of using FC as the indicator of pathogenic contamination and changing this to use *E. coli* instead. North Liberty in Iowa is an example of this, changing their testing from FC to *E. coli* in 2020. Notably, both EU and US EPA regulations now require *E. coli* monitoring as a mandatory parameter in drinking water assessment.

In this study, 1438 samples were evaluated for *E. coli* in MBR permeate from 16 full-scale facilities. Figure 1 below summarizes the results for each of these facilities.

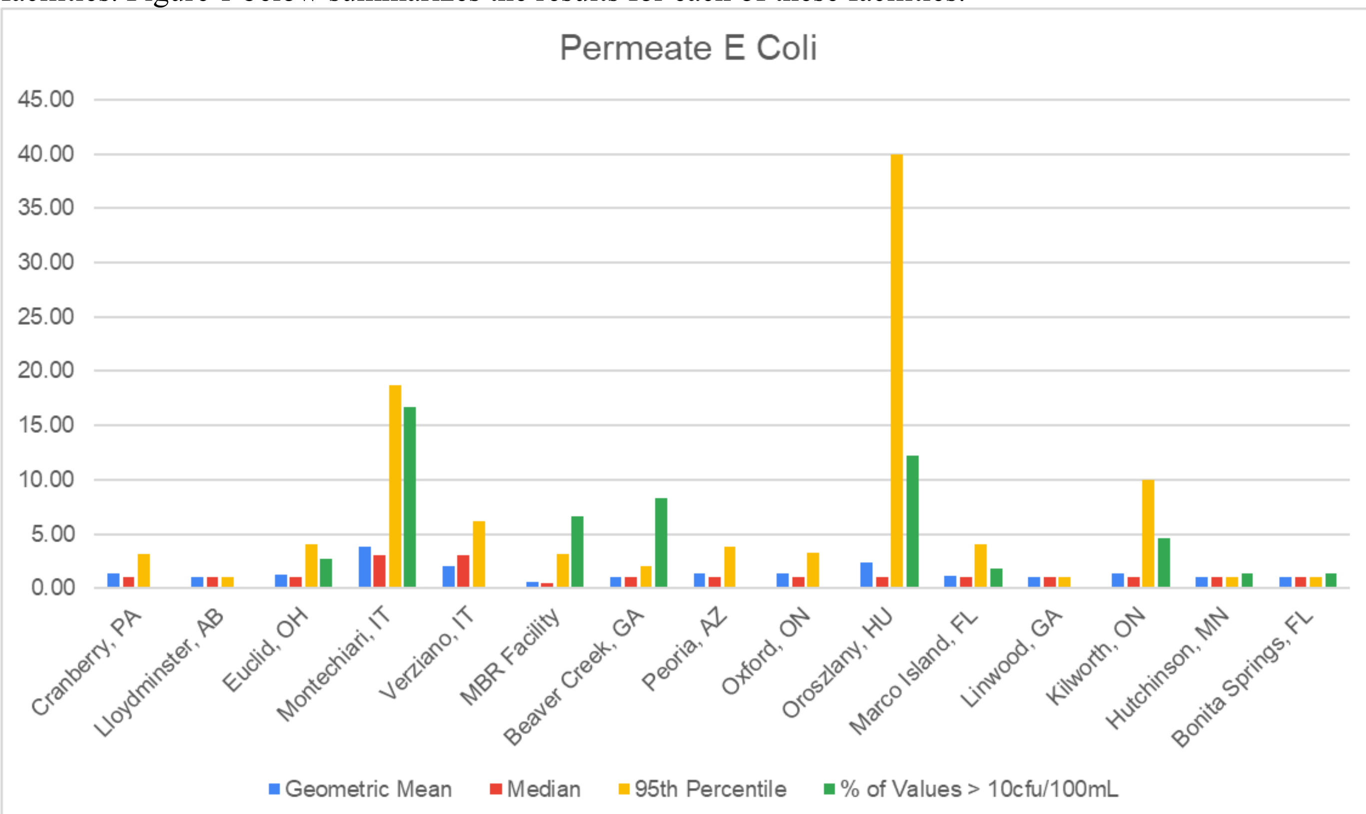

Figure 1: E. *Coli results are displayed for the MBR permeate of 15 MBR facilities.*

Note that North Liberty data has been excluded from this dataset as the detection limit for the majority of the samples was 10 CFU/100mL limiting the relative interpretation of the dataset. The geometric mean for this facility was <9.8 CFU/100mL.

Highlights of the *E. coli* (EC) dataset include:

- The range of geometric mean concentration of EC across the 15 facilities was <1.0 - 3.84 CFU/100mL.

- The range of 95th percentile concentration of EC across the 15 facilities was <1.0 – 40 CFU/100mL
- The maximum 99th percentile EC across all of the facilities was 84 CFU/100mL, with an absolute maximum sample value of 100 CFU/100mL.
- Of the 15 datasets evaluated, 14 had <10% of all *E. coli* samples measured at >10 CFU/100mL, with 8 facilities never having any samples exceeding 10 CFU/100mL.

Summary of Fecal Coliform in MBR Permeate

Fecal coliform (FC) bacteria represent a subset of total coliforms within the Enterobacteriaceae family, including bacteria from the genera *Escherichia, Klebsiella, Enterobacter, and Citrobacter*. These bacteria are commonly referred to as 'thermotolerant coliforms' due to their ability to grow optimally at higher temperatures (44.5°C) compared to other total coliforms (35°C). This temperature tolerance reflects their origin in warm-blooded animal intestines, making fecal coliform measurements a more reliable indicator of fecal contamination in water sources than total coliform measurements (Stevens et. al., 2003).

In this study, 6211 results of fecal coliform were analyzed in the MBR permeate from 14 full-scale facilities. Figure 2 summarizes the results for each plant.

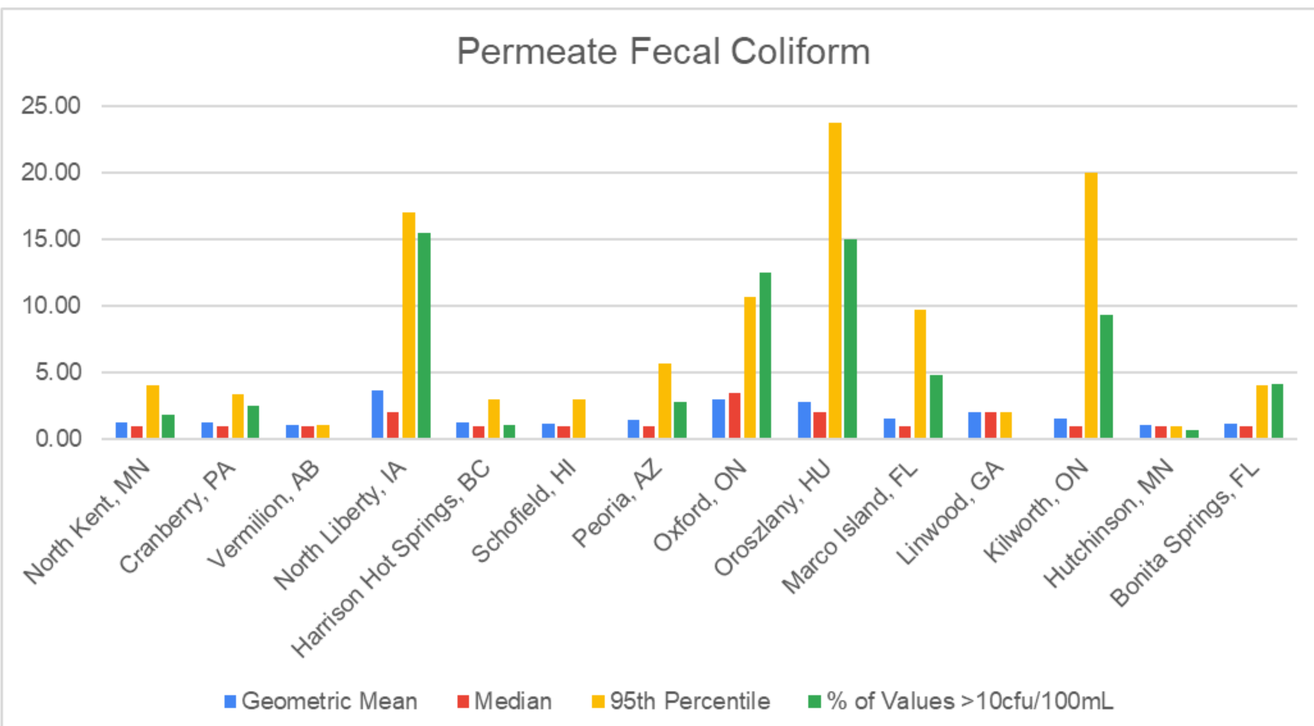

Figure 2: 14 MBR facilities were analyzed for the fecal coliform in their MBR permeate.

Highlights of the fecal coliform (FC) data include:

- The range of geometric mean concentration of FC across the 9 facilities was 1.02 - 3.63 CFU/100mL.
- The range of 95th percentile concentration of FC across the 9 facilities was <1.0 - 23.7 CFU/100mL
- The maximum 99th percentile FC across all of the facilities was 49.8 CFU/100mL, with an absolute maximum value of 180 CFU/100mL.

Summary of Total Coliforms in MBR Permeate

Total coliform tests detect bacteria from the Enterobacteriaceae family, including species from *Escherichia, Klebsiella, Enterobacter, Citrobacter, Serratia*, and others (Health Canada, 2012). While some of these bacteria come from feces, others are naturally present in soil, water, and plants. Among the three bacterial groups studied (total coliforms, fecal coliforms, and E. coli), total coliforms provide the least specific indication of fecal contamination.

In this study, 958 results were obtained from the MBR permeate prior to any disinfection from 9 full-scale MBR facilities. The results from these facilities are displayed in Figure 3 below, noting that a 95th percentile for Cranberry, PA could not be calculated due to an insufficient number of total results.

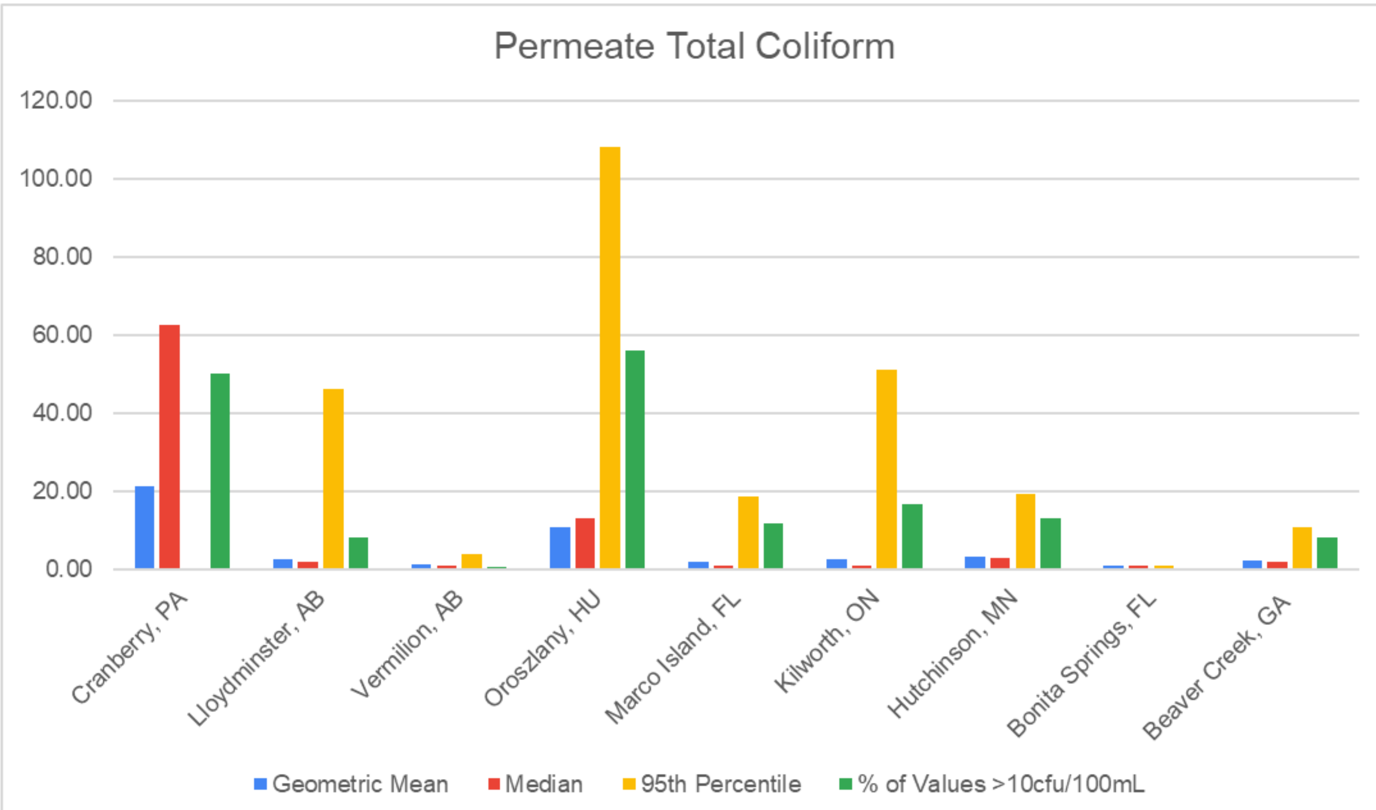

Figure 3: Nine MBR facilities were analyzed for the Total Coliform in their MBR permeate.

Highlights of the total coliform data are as follows:

- The range of geometric mean concentration of TC across the 9 facilities was <1.0 - 21.2 CFU/100mL.
- The range of 95th percentile concentration of TC across the 9 facilities was <1.0 – 100 CFU/100mL
- The maximum concentration of TC in the MBR permeate was 201 CFU/100mL at one facility, with all other facilities ≤ 200 CFU/100mLin all samples. Note that a single data point of 201 CFU/100mL likely points to a contaminated sample rather than an actual

measured value, as the following measurements at the facility returned to single digit values.

Overall Summary of Aggregate Data

When the substantial dataset of more than 8,000 microbial sample results is analyzed for the bacteriological parameters, it is clear that the MBR permeate quality as it relates to disinfection capabilities is very high, with data summarized in Figure 4 below. The median values for all parameters were found to be at the detection limit, or 1.0 CFU or MPN/100mL for all of *E. coli,* fecal coliform and total coliform. The geometric means for all three parameters were 1.21 CFU/100mL, 1.28 CFU/100mL, and 1.63 CFU/100mL, respectively, and the 95th percentile values were 4.0 CFU/100mL, 4.0 CFU/100mL, and 14.0 CFU/100mL, respectively, signifying excellent removal of bacteriological pathogens through the MBR system.

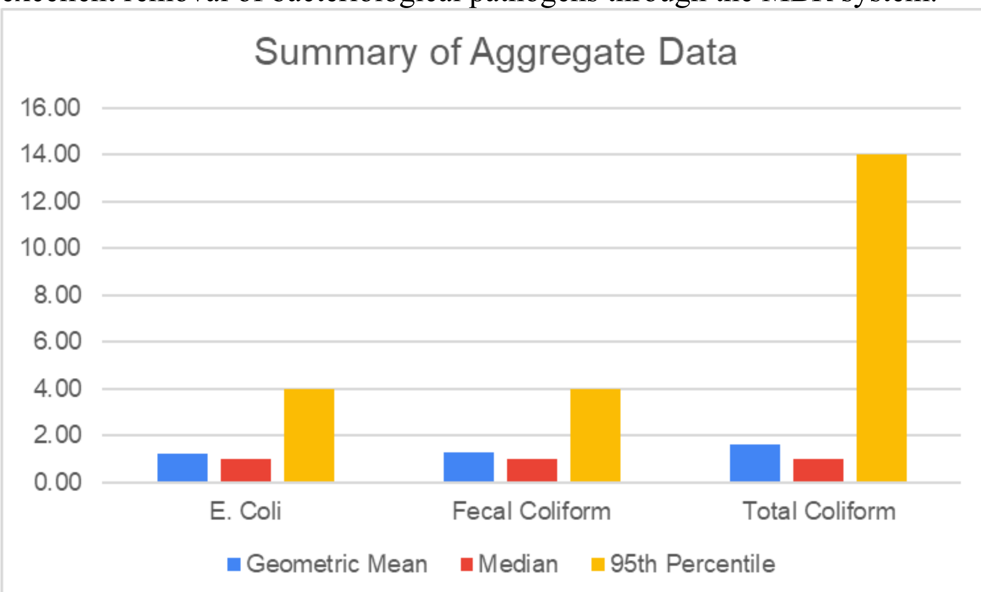

Figure 4: Data for all 20 facilities was combined into an aggregate dataset and analyzed.

Table 2 below includes additional highlights of the aggregate data sample. While the absolute maximum measured value is listed, the 99th percentile for each bacteriological parameter is also included to account for sampling and measurement error, as a more accurate representation of the maximum value.

Table 2: Summary of analytical parameters for each bacteriological pathogen.

	% of Non-Detect Values	**% of Values ≤10 CFU/100mL**	**99th Percentile (CFU/100mL**	**Absolute Max Value (CFU/100mL)**
E Coli	74%	98%	19.2	100
Fecal Coliform	72%	98%	21.0	210

Total Coliform	66%	93%	100	201

MBR Permeate by Membrane Age

A common concern raised when planning for the full lifecycle of a facility is how the efficacy of bacteriological removal may change over time, depending on the age of the membranes in question. Data was compiled based on the membrane age at the time of sampling and grouped into ranges of membrane age. The split of sample points across the age ranges for each microorganism is displayed in Table 3 below. This analysis was not included for total coliforms due to a lack of available data across all of the age ranges.

Table 3: Number of samples points in each membrane age grouping

Membrane Age	*E. coli* samples	Fecal Coliform samples
0-3 years	371	2148
3-6 years	787	2528
6-9 years	79	1277
9-12 years	104	296

Figure 5 below shows the geometric mean of the coliform data for each age range. This analysis shows consistent performance over the life of the membranes, with the geometric mean for all ranges <2.0 CFU/100mL for both *E. coli* and fecal coliforms.

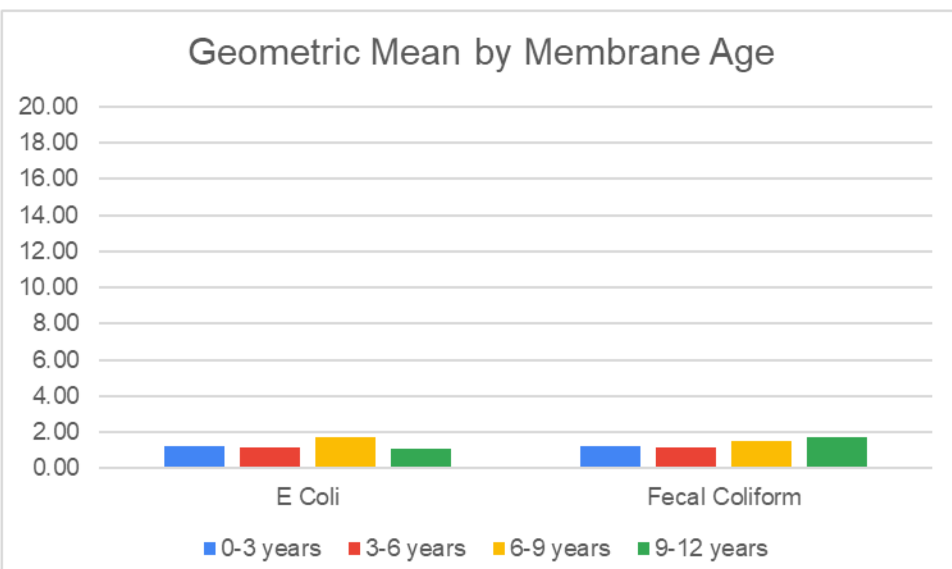

Figure 5: Comparing geometric mean of permeate coliform at various membrane ages.

Figure 6 below shows the 95th percentile of the coliform data for each age range. This analysis shows consistent performance over the life of the membranes, with the 95th percentile for all ranges <20.0 CFU/100mL for both *E. coli* and fecal coliforms.

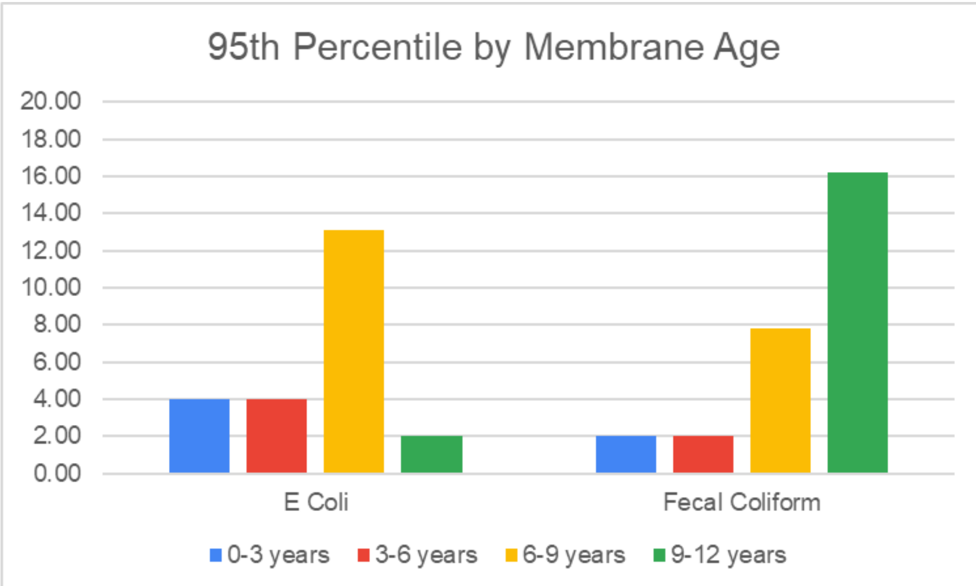

Figure 6: Comparing 95th percentile of permeate coliform at various membrane ages.

Interpretation of Results

MBR Permeate Meets Discharge Requirements without Additional Disinfection

As previously discussed, the recreational water quality criteria established by the US EPA in 2012 recommend that levels for *E. coli* are not to exceed a 30-day geometric mean of 100 CFU/100mL, and a 30-day 90th percentile value of 320 CFU/100mL as the most stringent recommendation for primary contact recreational use water. The data collected and analyzed in this study demonstrate that MBR effluent clearly meets these requirements without requiring further disinfection, as the 95th percentile (an even more stringent analysis than the recommended 90th percentile) value for *E. coli* at all facilities ranged from <1.0 to 40 CFU/100mL, with a total aggregate 95th percentile of 4.0 CFU/100mL, significantly below the recommended limits.

Regulatory Acceptance for Removing Downstream Disinfection

Specific permit requirements for bacteriological parameters in plant effluent varies, depending on factors such as the location and regulatory body, the receiving body of water, etc. Some regulatory bodies only dictate specific limits for effluent discharge, while others specifically state that disinfection equipment such as chemical or ultraviolet (UV) disinfection must be included as part of the process. This data analysis demonstrates that the latter approach, where disinfection is required regardless of water quality, is perhaps outdated as technologies mature and our understanding of their capabilities evolve. Technologies such as MBR can meet and exceed water quality requirements without further treatment.

There are a growing number of MBR facilities that are permitted to operate without further disinfection downstream of the MBR system, allowing these facilities to benefit from substantial savings on both capital and operational costs, as well as maintenance labour and time. Several of these plants are listed in Table 4 below, covering a range of plant capacities, age, locations, and permit requirements.

Table 4: ZeeWeed MBR facilities across North America have been permitted to discharge without additional disinfection of MBR permeate.

Plant Name	**State /Province**	**ADF (MLD)**	**ADF (MGD)**	**Start Year**	**Permit Requirements**
Vermilion WWTP	AB	2.1	0.55	2021	Monthly Average, sampling 5x / week FC < 200 CFU/100mL TC < 1000 CFU/100mL
Henderson WWTP	NV	30.3	8.00	2011	Weekly sampling FC < 2.2 MPN/100mL Monthly Mean FC < 23 MPN/100mL Daily Max
Hutchinson WWTF	MN	4.8	1.27	2008	FC < 200CFU/100mL average
North Liberty WWTP	IA	11	2.91	2009	**Original Permit:** Monthly Geometric Mean, Mar 15 - Nov 15 FC < 200 CFU/100mL **Updated Permit:** Monthly Geometric Mean, Mar to Nov EC < 126 CFU/100mL
Lloydminster WWTP	SK	20.9	5.52	2023	**Outfall #1:** EC < 200 CFU/100mL average, <400 CFU/100mL as single sample **Outfall #2** Monthly Geo mean of EC <200 CFU/100mL No sample to exceed 1000 CFU/100mL
Brandon WWTP	MB	30.4	8.03	2010	EC < 200 MPN/100mL, 3 consecutive days in 7 day period
City of Euclid	OH	83.3	22	2020	EC < 284 CFU/100mL weekly max EC < 126 CFU/100mL monthly max

Case Studies

North Liberty, IA

The 3.3MGD North Liberty ZeeWeed MBR facility in Iowa started operations in 2008. The facility was built and started up with UV disinfection, however it has been operating since 2009 without running the UV system and meeting all permit requirements.

At the time of startup in 2008, the National Pollutant Discharge Elimination System (NPDES) requirements for the facility were based on fecal coliform and required a monthly geometric mean < 200 CFU/100mL from March 15th through November 15th. After startup, the operators of the facility collected fecal coliform data from the MBR permeate and submitted this to the Iowa Department of Natural Resources (IDNR) along with a request to have the permit amended to remove the requirement for the UV system. The IDNR approved this request, and the permit was amended such that operation of the UV system could be suspended. It was required for many years that the UV system be maintained so that it could be reactivated within 24 hours if the system was no longer meeting the coliform limits. However, since the plant has never needed to re-activate the disinfection system, it has since been permitted to fully remove the UV equipment during upcoming plant upgrades. An additional change to the permit in recent years was to update the bacteriological discharge limit from Fecal Coliform <200 CFU/100mL to *E. coli* < 126 CFU/100mL.

The North Liberty facility has undergone various upgrades and expansions to the membrane system over the life of the plant, including upgrades to the aeration system for improved efficiency, and the replacement of membranes over a period from 2021-2023. Throughout all of these changes, and over an extended plant life, the system has operated without any additional disinfection with no permit exceedances.

Lloydminster, SK

The Lloydminster Wastewater Treatment Plant is a 5.5MGD MBR facility in SK, Canada, commissioned in 2023. The facility uses the ZeeWeed 500EV product, the latest module and cassette configuration of the ZeeWeed 500 membrane product family, adapted to further improve surface area density, efficiency, and product longevity.

The MBR facility is designed to discharge to one of two outfalls, with a permit for each outfall. The Neale Edmunds outfall requires an average *E.coli* < 200 CFU/100mL, and a maximum of 400 CFU/100mL as a single sample, while the North Saskatchewan River outfall has a monthly geometric mean limit of *E.coli* < 200 CFU/100mL, and a single sample maximum limit of 1000 CFU/100mL.

The permit does not specify the type of treatment required but instead focuses on the water quality that must be met for discharge purposes. As such, the flowsheet and design of the wastewater treatment plant does not include a disinfection step, but rather the system relies on the MBR to meet the permit requirements, saving approximately 5% in capital expenditures by not installing a UV system. With potential opportunities to use the treated effluent for water recycling purposes in the future, the facility was built with provisions to add a UV disinfection system in the future if required.

Data collected from one full year of plant operation reported 23 of 24 samples of *E. coli* as non-detect, or <1.0 CFU/100mL, with a single sample reporting 1 CFU/100mL, demonstrating excellent removal of *E. coli* through the membrane bioreactor system.

Euclid, OH

The City of Euclid Wastewater Treatment Plant in Ohio is a ZeeWeed MBR that started operations in 2020. With an average day capacity of 22MGD, and a peak capacity of 66MGD, it is one of the largest MBR facilities in North America.

Similar to the Lloydminster MBR, the City of Euclid permit does not mandate specific treatment technologies, but rather focuses on the required discharge water quality, with a weekly maximum average of *E. coli* < 284 CFU/100mL, and a monthly maximum average of *E. coli* < 126 CFU/100mL. As such, the facility was designed without post-disinfection equipment, using the MBR to meet effluent bacteriological requirements. Operating without any additional disinfection, the data reviewed in this paper included 401 total samples from May 2022 - May 2025 demonstrated a total geometric mean of 1.29 CFU/100mL, a maximum monthly geometric mean of 2.4 CFU/100mL, and a maximum weekly geometric mean of 6.8 CFU/100mL, orders of magnitude below the limits.

City of Hutchinson, Minnesota

In 2008, the Hutchinson Wastewater Treatment Facility completed a major expansion which included a ZeeWeed MBR filtration system running in parallel to the existing oxidation ditch activated sludge process. The MBR system added 1.27 MGD of capacity to the plant for a total treatment capacity of 3.67 MGD with the ability to treat a peak wet weather flow of 9.62 MGD. The MBR system consists of two aerobic tanks followed by two ZeeWeed* filtration trains. The MBR effluent is directed to the UV disinfection facility. The City's permit requirements for fecal coliforms are a monthly mean value of <200 MPN per 100 mL based on grab samples 3 times per week.

A testing protocol was initiated in July and concluded in October of 2008 to review the MBR system's ability to independently meet the discharge requirements without post-disinfection. During the testing, sampling was done as per the permit requirements from the MBR effluent and was tested for fecal coliform and *E. coli*. The results of the testing demonstrated that the amount of fecal coliform and *E. coli* in the MBR effluent was negligible for all tests.

The four months of testing highlighted the MBR's ability to readily meet the permit requirements. The City of Hutchinson requested that the Minnesota Pollution Control Agency (MPCA) deem the MBR effluent as having met the discharge requirements without further disinfection (Donohue & Associates, 2009)

The MPCA granted the City of Hutchinson permission to bypass the UV system. As of November 2014, the Hutchinson facility continues to bypass their UV system. The MBR system continues to produce water within the permit requirements, and the UV system has not been required for MBR permeate. As a result, the city saved several hundred thousand dollars not having to purchase previously planned additional UV facilities as well as additional savings attributed to operational, electrical and maintenance costs.

City of Henderson, Nevada

The Southwest Water Reclamation Facility (SWRF) in Henderson, Nevada consists of a biological treatment system designed to include nutrient removal followed by an 8 train ZeeWeed* MBR system. The MBR effluent is directed to the UV facility followed by post-chlorination. The SWRF is designed to treat annual average daily flow of 8.0 MGD and peaking to a peak hourly flow rate of 13.6 MGD. The city's permitted requirements for bacteriological parameters, driven by the reclamation of the water, requires the effluent to meet <2.2 MPN of fecal coliforms per 100 mL as the mean of all samples taken in the month and a daily maximum of 23 MPN of fecal coliforms per 100 mL. Sampling is conducted weekly.

A study was initiated in November of 2012 and concluded in April of 2013 with the goals of demonstrating the treatment facility's ability to remove various virus and bacteria without the use of disinfection. During this time samples of the MBR permeate were taken during various phases of operation (i.e. normal operation, high flux operation, post citric acid cleaning and post sodium hypochlorite cleaning). The samples were tested for total coliforms, fecal coliform, and *E. coli*, as well as for viruses including Human Adenovirus (HAdV), Norovirus G1 and G2, F Specific (MS2) and Somatic Coliphage (SC). The study concluded that the MBR system was very effective at reducing virus and bacteria concentrations to very low levels and in most cases to non-detect levels without disinfection. (Erdal *et al.*, 2014)

The City of Henderson was given permission by the Nevada Department of Environmental Protection to bypass the UV system. Post-chlorination is still maintained to prevent bio growth in the reclaimed water system. As of November 2014, the Henderson facility continues to bypass their UV system and the MBR system continues to produce water within the permit requirements. As a result, the City of Henderson is currently saving an estimated $93,000 a year in operational costs.

Conclusions

Twenty full-scale MBR facilities sampled non-disinfected MBR permeate to evaluate the efficacy of ZeeWeed MBR technology at removing bacteriological pathogens. In total, 8,657 analytical results for *E. coli*, fecal coliforms and total coliforms were gathered and analyzed. Additionally, several facilities that are permitted to discharge effluent without further disinfection were reviewed. Based on the data, the following conclusions can be made:

- The permeate quality at all twenty (20) facilities met surface-water discharge standards, and United States and European Union public health guidelines for recreational use of water bodies.
- The maximum geometric mean of *E. coli* for all facilities was 3.84, with a maximum 95th percentile of *E. coli* of 18.75 CFU/100mL, well within all guidelines.
- The geometric mean concentrations across all permeate samples were 1.21 CFU/100mL, 1.28 CFU/100mL, and 1.63 CFU/100mL for *E. coli,* fecal coliform, and total coliform,

respectively, and the corresponding 95th-percentile values were 4.0 CFU/100mL, 4.0 CFU/100mL, and 14 CFU/100mL

- The length of time that membranes have been in operation has no impact on the microbial removal of the ZeeWeed MBR systems.
- Several regulatory bodies across the United States and Canada have allowed MBR permeate to be discharged without requiring downstream disinfection, based on demonstrated pathogen removal in the MBR process. This includes facilities of different sizes operating with different evolutions of the ZeeWeed membrane product.
- The ability to not install or operate downstream disinfection after and MBR system offers facilities significant savings in both capital and annual operating expenditures.

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Appendix E

Fine Screen - Reference Letter

THE DISTRICT MUNICIPALITY OF MUSKOKA

April 30, 2024

To whom it may concern,

The Bracebridge WWTP influent works building, commissioned in January 2021, features two fine screening and grit removal trains installed in stainless steel tankage. The screens, 3mm Claro fine step screens, each equipped with an associated wash press, are rated for 32,000 m3/day (100% redundancy). The grit system consists of 3000 mm dia. forced vortex systems, each with a dedicated wet suction grit pump and grit classifier. The Claro mechanical process equipment controls system also includes logic for influent works building auxiliary systems such as Summer & Winter HVAC, a high lift pump station, a back-up overflow pumping station, odour control systems, gas detection, and booster pumps.

Since installing the 3mm step screens, the two existing 1mm drum screens immediately upstream of the MBR system have ceased discharging screenings debris. Operations have not needed to empty the common 3 yard receiving bin in the last 2 years, which previously required weekly maintenance. The 1mm screens now capture a fine pulpy organic material that dissolves when subjected to the washing jet filter cleaning system. There is no evidence of larger fibers or bypass. The step screens, in conjunction with the screenings filter mat, effectively filter debris without fouling the hollow fiber MBR since startup in 2021. Consequently, decommissioning the 1mm rotary drum screens is under consideration due to their maintenance and energy costs.

The step screens require minimal maintenance and boast low runtimes since installation. Their enclosed design isolates operators from odors and contamination hazards inherent to wastewater treatment. The clean and dry screening plugs produced by the wash presses are reliably bagged. Grit discharged from the classifiers is clean, granular, and dry, contributing to a quiet and conducive working environment.

Claro's after-sales support, system knowledge, and availability have been exceptional. Their personnel demonstrate a genuine interest in system operation and effectiveness.

We strongly endorse Claro and their step screening and grit removal system.

Philip Moesker
Chief Operator

The District Municipality of Muskoka
70 Pine Street, Bracebridge ON P1L1N3

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Contact
Mike Canning, P.Eng.
mcanning@mcelhanney.com