Appendix A

Engineering Study for the Wastewater Treatment Plant and Water Treatment Plant at CFB Gagetown (Draft), CBL, 2012

Engineering Study for the Wastewater Treatment Plant and Water Treatment Plant at CFB Gagetown

Draft Report

DND Project File: 183524

CBCL Project #123252 ● Draft Report ● December 12, 2012

ISO 9001
Registered Company

Prepared for:

Department of National Defense and Defense Construction Canada

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12 December 2012

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Dear Ms. McCoy and Mr. Thomson:

Please find four (4) copies of the above named Draft Report for your review and comments.

We look forward to meeting with you on December 19, 2012 to review the contents of this Draft Report and receive any feedback.

Yours very truly,

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A Water Treatability Study

Glossary

ADF	Average Daily Flow
BNR	Biological Nutrient Removal
BOD/ BOD_5	5-day Biochemical Oxygen Demand (non-Nitrification-inhibited)
^oC	Degree Celsius
$CaCO_3$	Alkalinity (expressed as calcium carbonate equivalent)
CCME	Canadian Council of the Minsters of Environment
CGD	Canadian Geodetic Datum
COD	Chemical Oxygen Demand
DAF	Dissolved Air Floatation
DBPs	Disinfection By Products
DCC	Defence Construction Canada
DND	Department of National Defence
DO	Dissolved Oxygen
EC	Environment Canada
F:M	Food-to-Microorganisms Ratio
GCDWQ	Guidelines for Canadian Drinking Water Quality
GWWTP	Gagetown Wastewater Treatment Plant
GWTP	Gagetown Water Treatment Plant
ha	Hectare
HP	Horsepower
HRT	Hydraulic Retention Time
I&C	Instrumentation and Control
kg	Kilogram
kW	Kilowatt
kWh	Kilowatt-hour
L	Litre
LPCD	Litres Per Capita Per day
$M	Million Dollars
m	Metre
m^2	Square Metre
m^3	Cubic Metre
MF	Microfiltration Membrane
mg	Milligram
mL	Millilitre
ML	Million Litres
MLD	Million Litres per Day
MLSS	Mixed Liquor Suspended Solids
mm	Millimetre
MPN	Most Probable Number
MWWE	Municipal Wastewater Effluent
N	Nitrogen
NBDOE	New Brunswick Department of Environment

NF	Nanofiltration Membrane
NH_3	Un-ionized Ammonia
NH_3-N	Un-ionized Ammonia-Nitrogen
NH_4+	Ionized Ammonia or Ammonium
NH_4+-N	Ammonium-Nitrogen
NO_3-	Nitrate
NO_3-N	Nitrate-Nitrogen
NTU	Nephelometric Turbidity Units, measure of Turbidity
NOM	Naturally Occurring Organic Matter
O_2	Oxygen
O&M	Operations and Maintenance
OUR	Oxygen Uptake Rate
P	Phosphorus
PO_4^-	Ortho-phosphate
PLC	Programmable Logic Controller
MMF	Max Month Flow
PF	Peak Flow
Q	Flow
RAS	Return Activated Sludge
RFP	Request for Proposals
SCADA	Supervisory Control and Data Acquisition
SRT	Solids Retention Time (or sludge age or mean cell residence time)
SVI	Sludge Volume Index
SWD	Side Water Depth
TCU	True Colour Units
THM	Trihalomethanes
TKN	Total Kjeldahl Nitrogen
TSS	Total Suspended Solids
UF	Ultrafiltration Membrane
USmgd	US million gallons per day
UV	Ultraviolet
VFD	Variable Frequency Drive
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge

EXECUTIVE SUMMARY

ACKNOWLEDGEMENTS

CHAPTER 1 INTRODUCTION

This Predesign study for the upgrade and expansion of the CFB Gagetown Water Treatment (WTP) and CFB Gagetown Wastewater Treatment Plant (WWTP) has been conducted for the Department of National Defence (DND) and Defence Construction Canada (DCC), under DCC File No. GA 183524.

DND has expressed interest in transferring ownership of the WTP and WWTP to the Town. Defence Construction Canada (DCC) issued a Request for Proposals to study the existing treatment plants, outline recommended upgrades or replacement and associated costs to assist in the decision making process. This project was initiated after a meeting between CBCL Limited, the Town of Oromocto, DCC and representatives from C.F.B. Gagetown in July of 2012. The meeting was used to examine the existing facilities, coordinate data recovery, and discuss the general scope of work.

Both treatment plants were constructed with the original Military Base and would have been considered "State of the Art" treatment facilities when they were commissioned some 50 to 55 years ago. Today these facilities, and the process technologies operating within, are nearing the end of their intended service life and are thus due for major upgrade or replacement. DND has decided to investigate options for the complete replacement of each facility.

The Predesign Report has been separated into two sections, Part A for the Water Treatment Plant and Part B for the Wastewater Treatment Plant.

1.1 Service Area for Water Supply and Wastewater Collection

The CFB Gagetown Water Treatment Plant provides potable water to all residents in the Town of Oromocto, the First Nations Reserve and Canadian Forces Base Gagetown. The Gagetown Wastewater Treatment Plant provides service to about 80% of the Town's population and to CFB Gagetown. Within the Town of Oromocto there is small wastewater treatment plant (Oromocto West WWTP) that currently provides service to the remainder of the Town's population; however, that facility will be decommissioned and sewage will be pumped to the new treatment plant that is to be constructed as part of this project.

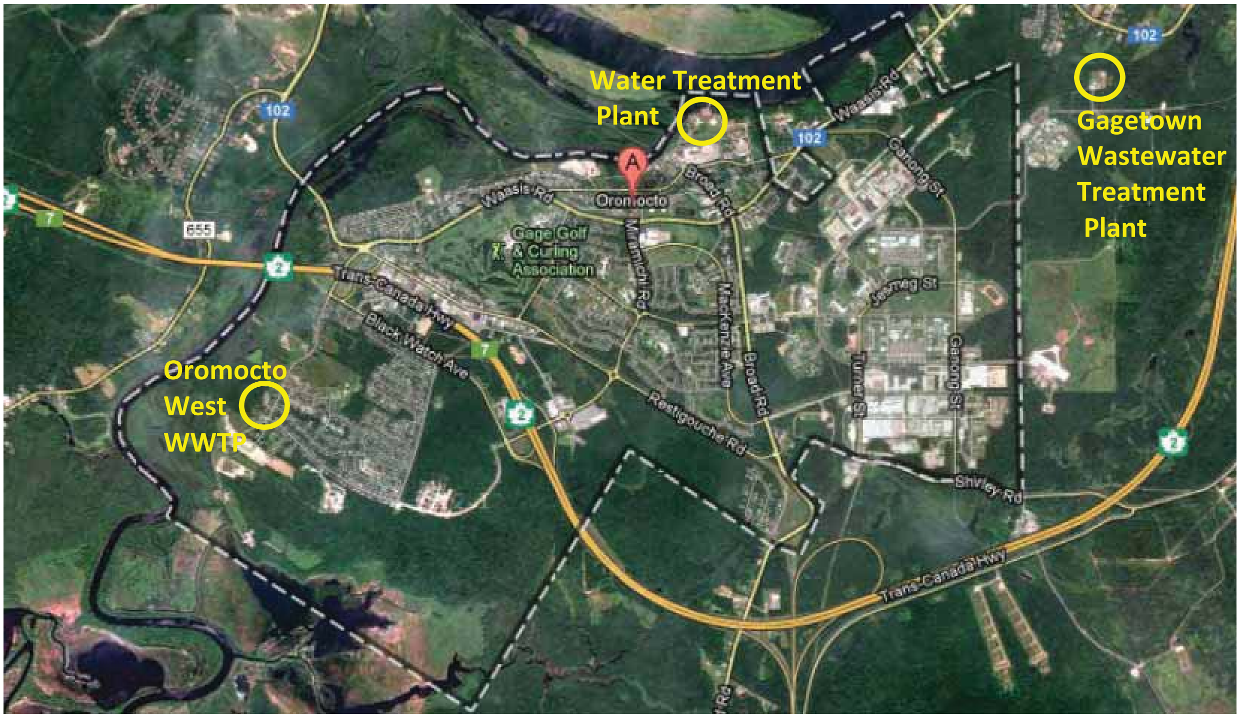


Figure 1.1: Town of Oromocto and CFB Gagetown Service Area

1.1.1 Service Area Population

According to Statistics Canada, the census population for the Town or Oromocto was 8,932 for the year 2011. The Town population fluctuates due mainly to the mobility of between 3500 and 5000 military personnel posted at CFB Gagetown with their families. The total military strength has increased slightly over the years; however, the trend has been for families to leave Military Housing and purchase private properties within the Town. This has been a driver for development especially in the Oromocto West Area (Municipal Plan for the Town of Oromocto, 2006).

Table 1.1: Canadian Census Data for Town and Oromocto Reserve

Year	Town of Oromocto Population	Oromocto IR26 Population	Total Population	% Annual Increase	% Increase
2011	8932	286	9218	1.20%	6.1%
2006	8402	284	8686	-0.91%	-4.5%
2001	8843	249	9092	-0.77%	-3.8%
1996	9194	256	9450	-0.14%	-0.7%
1991	9325	190	9515	-0.57%	-2.8%
1986	9655	135	9790	1.31%	6.7%
1981	9064	110	9174	-4.30%	-19.7%

Overall since 1986, the population within the Town has been steady or decreased slightly with the exception being the last 5 years during which there has been 1.2% annual growth. Growth is attributable to retirees returning to the Town and possibly people who live in Oromocto and commute to the City of Fredericton for employment.

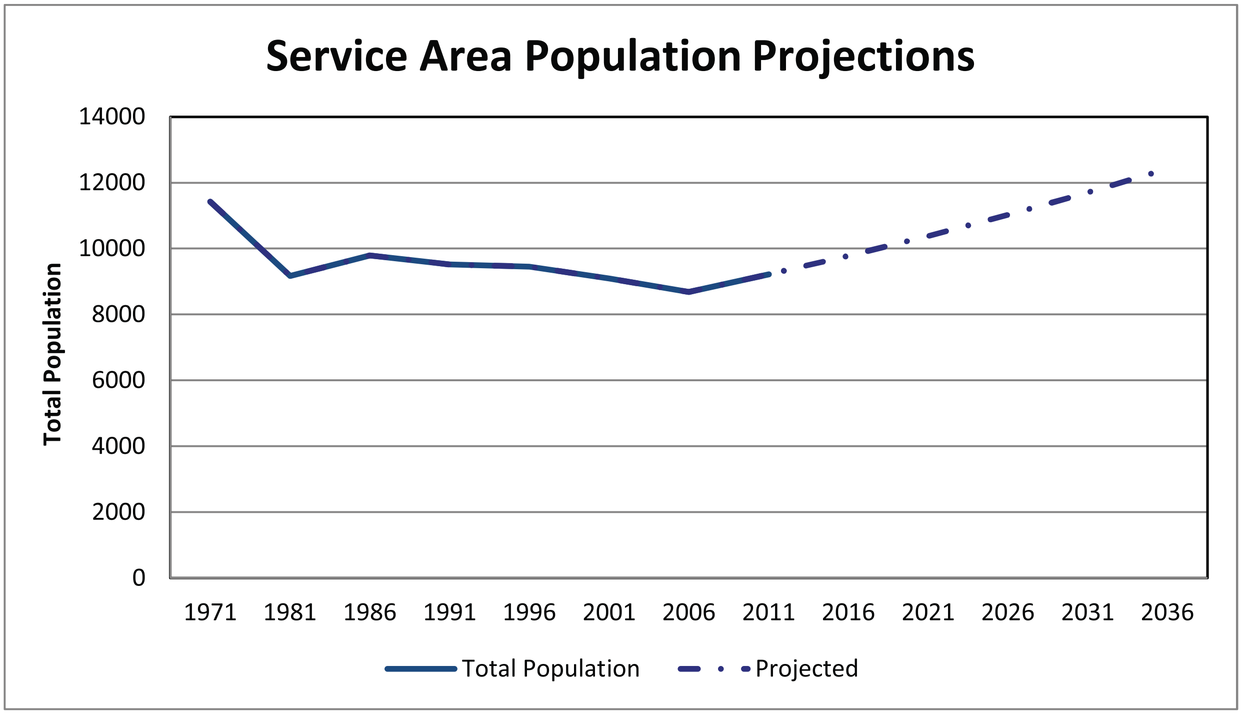


Figure 1.2: **Service Area Population Projections for the Town of Oromocto and Native Reserve**

For the purpose of this predesign study, we have chosen to project population growth 25-years into the future from 2011 making the year 2036 the design year. In doing so, the average annual growth rate over the past 5 years has been assumed. Therefore, the design population for the new water and wastewater treatment facilities will be 12,421 people assuming 25 years of growth at 1.2%.

The assumption for low growth (1.2% annually) is consistent with the Municipal Plan for the Town of Oromocto. The major risk in using Census data and Municipal plans would be major unforeseen changes with the Military base at CFB Gagetown. The service population projection for the water and wastewater treatment facilities assumes continued military presence at the base of between 3500 and 5000 personnel depending on the season.

CHAPTER 2 PART A – WATER TREATMENT PLANT

2.1 Introduction

The Department of National Defence operates the C.F.B. Gagetown Water Treatment Plant for the purpose of providing potable water to the Town of Oromocto and C.F.B. Gagetown. The existing water treatment plant (WTP) was constructed about 50 years ago and has seen upgrades to the treatment process and controls over the years. Although the current WTP meets the guideline requirements, the plant is aging and will require substantial upgrades in the near future.

As described in Chapter 1, DND has expressed an interest in transferring ownership to the Town. Defence Construction Canada (DCC) issued an RFP for a study to determine recommended upgrades and associated costs to assist in the decision making process regarding potential transfer of ownership.

2.2 Part A Study Objectives – Water Treatment Plant

After review of the Consultant Briefing document for the project (GA 183524), CBCL Limited responded with a proposal outlining the following objectives:

1. Establish Design Flows for New Water Treatment Facilities.
2. Characterize the Existing Source Water Quality (Saint John River).
3. Preliminary Evaluation of Water Treatment Options.
4. Develop Preliminary Design for Water Treatment Facility.
5. Review the existing water treatment plant intake and recommend further action, as required.
6. Review of Plant Siting Requirements.
7. Provide Budgetary Cost Estimates.

2.3 Background

2.3.1 Watershed

The system is supplied with water from the convergence of the Oromocto and Saint John Rivers, located in the Saint John River Basin. The drainage area of the Saint John River Basin extends from the Bay of Fundy to Quebec, and Maine, with a total area of approximately 55,000km^2. The population residing in the Saint John River Basin was estimated to be 513,000 in 2001. Some of the larger communities located in the Saint John River Basin are; Edmundston, NB, Fredericton, NB, Fort Kent, Maine, Presque Isle, Maine, and Cabano, Quebec.

2.3.2 Existing Water Treatment Facility

The existing facility is fed from a 600mm steel intake pipe located in the Saint John River. Water flows through a screen chamber and low lift pump well via gravity before being pumped into the treatment facility. Aluminum sulfate and activated silica are added to the raw water in flash mixers prior to the flocculation stage consisting of two tanks, installed in parallel. Floc removal is achieved by gravity settling in a quiescent 877 m^3 sedimentation basin. Filtration is achieved with a high rate sand filter comprising a 75cm deep bed of sand supported by 15cm of gravel. Chlorine is added to the treated water before being pumped into the 2500 m^3 clearwell. Water flows from the clearwell to a 518 m^3 high lift pump well prior being pumped into the distribution system. The original plant design capacity was 13MLD, although typical plant production is 6.5 to 8.5MLD.

2.3.3 Transmission, Distribution, and Storage Systems

Flow from the high lift pumps is metered prior to a flow split. Approximately 80% of the flow is diverted to a Town owned 1400 m^3 reservoir, water from this reservoir is designated for the Town of Oromocto and can't be directed to the base in emergency situations. The remaining 20% of the flow is destined for the Base distribution system. The Base distribution system consists of a 3400 m^3 reservoir and distribution piping. The transmission main from the WTP to the Base reservoir has service connections along its length leading to some uncertainty with respect to hydraulics and effective mixing in the Base reservoir.

The Base distribution system was installed in the 1950's and appears to be showing some signs of its age. Preliminary discussions regarding system rehabilitation have occurred with no definitive plan or timeline in place. The Town distribution system is of similar age with the exception of newer real estate developments. Currently there is no leak detection program currently in place. Major leaks in the distribution system are indicated by visible signs of leaks in the road or by spikes in demand of the system. Minor system leaks are not detectable by either of the above methods and may account for more loss than major leaks given they may occur for much longer durations.

CHAPTER 3 WTP DESIGN CRITERIA

3.1 Water Consumption

The key flows that must be determined when examining the long-term demand requirements for a water supply and distribution system are the Average Day and Maximum Day demands. Average Day demand is defined as the average amount of water supplied to the system on a daily basis, calculated over one calendar year. Maximum Day demand is defined as the maximum amount of water supplied to the system on any given day within a calendar year.

The source supply must be capable of producing a safe yield equal to that of the average-day demand while the treatment plant will be required to produce the maximum-day demand.

3.1.1 Flow Records

Water production is measured at the water treatment plant using a flow meter and the total daily flow is logged using SCADA programming. Four years of daily flow data from 2008 to 2011 were available for the purpose of determining the system demands and are shown in Figure 3.1. These were reviewed and the current average and maximum daily demands were determined and are summarized in Table 3.1.

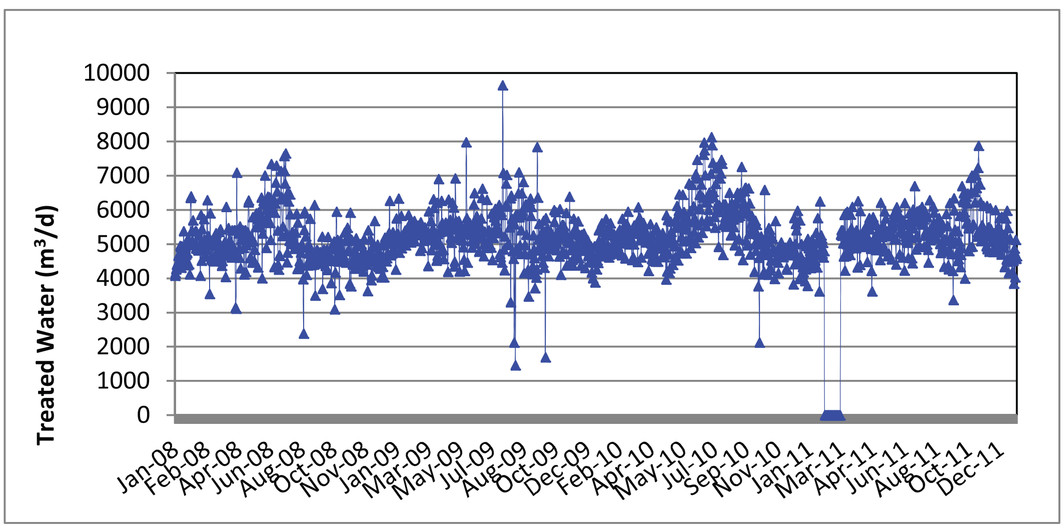


Figure 3.1: CFB Gagetown Flow Data (2008-2011)

Table 3.1: Observed Water Demand Average and Maximum Day

Year	2008	2009	2010	2011	All Data
Average Day, (m^3/day)	4,990	5,244	5,327	5,274	5,209
Maximum Day, (m^3/day)	7,652	9,639	8,124	7,874	8,322
Peaking Factor	1.53	1.84	1.53	1.49	1.60

As shown in the table, the average day and maximum day demands are 5,209 m^3/day and 8,322 m^3/day, respectively over the past four years. A review of Figure 3.1 and plant records reveals the maximum day demand of 9,639 m^3/day occurred on July 9, 2009. Plant records indicate that distribution system maintenance and flushing was conducted on that day and so this isolated data point is considered an outlier for design purposes. Instead, the average maximum day demand over the past 4 years has been chosen as a more representative design parameter.

3.2 Water Treatment Plant Capacity

As stated previously in Chapter 1, the military presence at CFB Gagetown is assumed to be constant during the design horizon and that the growth rate for the Town will be 1.2% as per the last 5 years. CBCL has chosen a 25-year design horizon, making 2036 the design year.

Based on the values in Table 3.2, the new Water Treatment Plant design will be based on water system demand plus an allowance for service and backwash water within the treatment facility. The average day capacity of 7,500 m^3/day and a maximum day capacity of 12,000 m^3/day has been selected for the new WTP.

Table 3.2: Design Capacity for the Water Treatment Plant

	Year 2011	Year 2036	Design Capacity
Design Population[1]	9,218	12,420	12,420
Average Day Demand (m^3/day)	5,209	7,030	7,500
Maximum Day Demand, (m^3/day)	8,322	11,250	12,000
Peaking Factor	1.60	1.60	1.60

Notes:

1. Population is based on 2011 Census Data which includes military personnel who have their primary residence in the service area.
2. Plant Design Capacity comprises demand plus an allowance for 5% wastage due to backwash.

It is interesting to note the original plant capacity was reportedly 13,000 m^3/day; however, process changes over the years have altered the actual plant capacity to approximately 9000 m^3/day.

3.3 Raw Water Quality

3.3.1 Existing Information

Raw water quality data from the Saint John River has been recorded by treatment plant staff at the existing WTP. Pertinent data are presented in Table 3.3 below.

Table 3.3: Saint John River Raw Water Characteristics from 2008 to 2011

Parameter	Units	Sample Type	Min.	Avg.	Max.	GCDWQ
pH		Continuous	6	7	8	6.5-8.5
Alkalinity	mg/L	Grab	9	31	54	
Turbidity	NTU	Continuous	1	4	50	0.1
Colour	TCU	Grab	12	68	783	<15
TOC	mg/L	Grab	0.4	7.5	20.1	

The raw water average condition is characterized by its relatively low alkalinity, periods of high colour and high turbidity. Periodically (commonly during spring runoff/flood) extreme water quality events are encountered, referred to by Plant staff as the *"Oromocto Effect"*. The Oromocto Effect refers to water quality conditions that occur when high water levels combine with high tides to cause the Oromocto River to flow upstream and enter the intake pipe to the Water Treatment Plant. Colours over 700 TCU have been recorded, which is an extremely large number. Although the maximum recorded turbidity is 50 NTU, it is suspected that actual peaks are higher due to the limitations with the turbidity instrumentation. The raw water then, is similar to many others under average conditions but experiences extreme peaks (Oromocto Effect), which pose a significant challenge for operators and for process technology.

3.3.2 Giardia and Cryptospridium

Source water quality results from sampling conducted between 2003 and 2008 confirmed the presence of *Cryptosporidium* oocysts and *Giardia* cysts (Acer, 2010). *Cryptosporidium* oocysts were found through sampling and analysis to range between 0.5 and 38 oocysts per 100mL with an average of 4 oocysts per 100mL over the sampling period. Giardia counts were even higher during this study period ranging 0.9 cyst/100mL to 160 cysts/100mL with an average count of 45 cysts/100mL. It is well known that *Cryptosporidium* and *Giardia* are resistant to chlorine disinfection and so UV disinfection will also be part of the treatment process in addition to chlorine disinfection. As described in the following chapters, clarification, filtration, UV disinfection and chlorine disinfection will form the basis for the multi-barrier approach to providing safe drinking water for the service area.

3.3.3 Treatability Study – Sample Testing

During October of 2012, river water elevation was high and high colour/turbidity conditions associated with an "Oromocto Effect" was suspected. Several cans of source water were collected and delivered to the Laboratory at Dalhousie University with instructions from CBCL pertaining to prescribed treatability study. The results of this study were pending at the time of this writing with analytical results expected shortly. The raw water colour for the sample collected was 56 TCU, below the average colour on record.

The extreme poor water quality events on record usually occur during spring freshet and spring tides when river levels are at their highest. Collection of water samples for treatability testing during these extreme events should be a priority for future work. This may help to determine with a greater degree of confidence the preferred water treatment process.

3.4 Treated Water Requirements

The new treatment facility will meet all water quality objectives set forth in the *Guidelines for Canadian Drinking Water Quality* (GCDWQ). The GCDWQ includes Health-Based Guidelines, Aesthetic Objectives, and Maximum Acceptable Concentrations for a range of parameters including organic and inorganic compounds, metals, minerals, and other identified water contaminants. These guidelines are provided by Health Canada through the federal government. The enforcement of particular standards is the responsibility of individual provincial agencies. New Brunswick requires utilities to adhere to the CDWQG.

3.4.1 Disinfection Byproducts

The new treatment facility must produce water that forms limited amounts of disinfection byproducts (DBPs) while ensuring protection against water borne pathogens. Trihalomethanes (THMs) and haloacetic acids (HAAs) are disinfection byproducts (DBP) of concern that are typically monitored for in the distribution system. Although DBPs can be measured as treated water leaves the treatment plant, this will not give a representative reading of the levels encountered in the system since DBPs are more a function of the condition and operation of the distribution system and contact time. The treatment system must produce water with a low enough level of naturally occurring organic material (NOM) such that DBP levels in the system remain under the limit. Through extensive experience with surface water throughout Atlantic Canada over the last 20 years, CBCL has found that a target treated water TOC level of 2 mg/l will produce DBPs that meet the guideline limits. The use of UV 254 absorbance as an indicator of organic content can also be helpful as this is a parameter than can be measured on a continuous flow thru basis in treatment plants.

CHAPTER 4 WTP PROCESS ALTERNATIVES

4.1 Existing Process Overview

The existing WTP is based on a conventional treatment process consisting of flocculation, sedimentation and sand filtration. Widely applied for many years, conventional treatment is known to have difficulty removing the low density floc particles created when treating soft, coloured water such as that found in the Saint John River, and indeed many surface waters throughout Atlantic Canada. Though conventional treatment has improved over the years through the use of inclined plates and tubes, the process still has difficulty with removal of low density particles, particularly in cold water. This manifests itself in significant floc carry over to the filters during times of poor water quality, resulting in reduced filter runs, poor effluent quality, increased backwashing requirements, and reduced plant output.

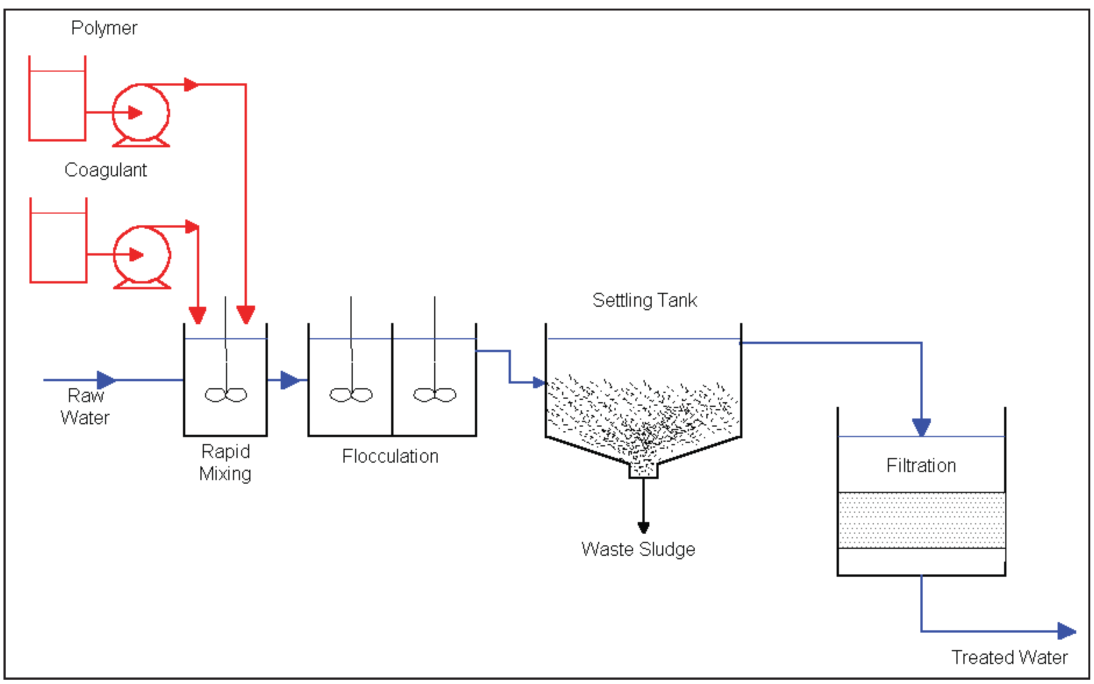


Figure 4.1: Conventional Treatment Process Schematic

More modern treatment processes are better able to deal with the challenges of treating soft coloured water. Newer technologies are capable of producing superior quality water in a smaller footprint typically resulting in a more cost effective treatment system. As a result, we have not included conventional treatment in the evaluation of candidate processes.

During average flow conditions, raw water quality in the Saint John River is generally similar to many other surface waters in Atlantic Canada in that it is soft (low mineralization) and coloured. It is during high river conditions coupled with high tides that extremely poor water quality events occur that are associated with the Oromocto Effect previously described. Water quality during these periods is exceptionally poor and difficult to treat.

4.1.1 Coagulation and Flocculation

Removal of naturally occurring organic matter in the treatment plant is the key to ensuring that DBP levels in the system are maintained at the lowest possible levels. Various parameters can be used as a measure of organic content in the water including colour, UV254 absorbance, total organic carbon and dissolved organic carbon. Reduction of organic matter will therefore be a primary goal of the treatment facility and will require the addition of a coagulant followed by flocculation to grow floc particles to a larger size such that they can be more easily removed. The coagulation-flocculation process is therefore common to all processes under consideration for this facility.

4.2 Options for Clarification and Filtration

As noted previously, floc particles created by soft coloured water with low turbidity are very slow to settle. This condition is exacerbated by the cold water temperatures experienced for several months of the year in Atlantic Canada. Clarification processes such as dissolved air floatation or ballasted flocculation are preferred processes when trying to separate low density particles in potable water treatment.

Filtration with granular media filters (typically anthracite and sand) are used as a final particle removal treatment process following the clarification step. This type of filter can achieve the particle removal efficiency and effluent turbidity required to meet the applicable standards provided they are paired with an effective clarification process such as DAF or ballasted flocculation.

Membrane technology is another filtration technique and it also requires clarification for removal of some of these particles to reduce loading rate on the membranes. Without the pretreatment in the clarification step, there will be excessive membrane fouling and eventually failure of the membrane filters would occur. Membranes are able to achieve superior particle removal and effluent turbidity compared to granular media filtration.

Based on the clarification and filtration options listed above, CBCL recommends the following three(3) treatment process themes be considered further:

- Dissolved Air Flotation with Granular media filtration;
- Ballasted Flocculation with Granular Media Filtration; and
- Dissolved Air Flotation with Membrane Filtration.

4.3 Dissolved Air Flotation

The Dissolved Air Floatation (DAF) process uses coagulation and flocculation to condition colour and turbidity in raw water for removal. Historically in North America flocculation particles resulting from coagulation have been removed by settling in sedimentation basins. However in recent years, DAF treatment systems have become the preferred choice for many new facilities treating light colour floc that results from cold, soft, coloured raw waters, such as are common in Atlantic Canada. The first DAF plant in Canada was installed in Port Hawksbury, NS in 1996. There are now many others in the region as well as throughout Canada and the U.S.

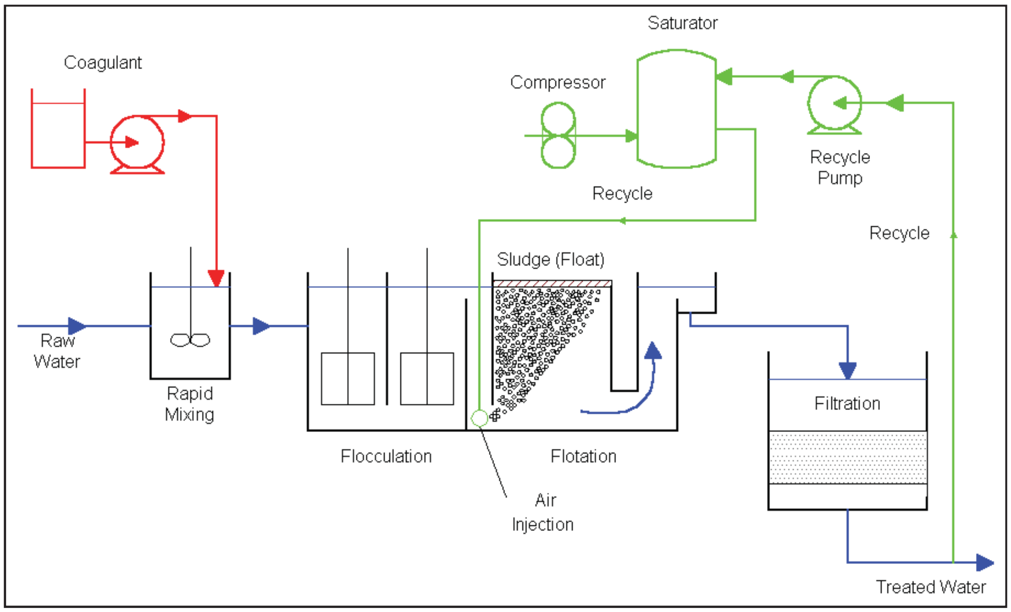


Figure 4.2: Typical Dissolved Air Flotation Treatment Process

In the dissolved air flotation process, fine bubbles injected into the water attach to the flocculated particles and cause them to float to the water surface where they are collected and removed. The dissolved air flotation process is usually designed to remove sufficient solids to provide the desired length of filter runs during maximum day flow. Following DAF pre-treatment, the clarified water is typically filtered through conventional granular media beds comprised of anthracite and fine sand. Recent adaptations include DAF pre-treatment upstream of membrane filtration, which enhances the filtered water quality, particularly for pathogen removal.

DAF processes can remove iron and manganese after the soluble manganous and ferrous ions are oxidized to manganese dioxide and ferric hydroxide. Manganese dioxide and ferric hydroxide will precipitate out of the water as a floc and will be removed by settling or flotation.

Removal of particles and floc by settling or flotation will remove enteroviruses and pathogens that become attached to the floc. Together with filtration, these processes are an integral part of pathogen removal and public health protection.

As with conventional sand filtration, regular backwashing of filters results in the production of wastewater in bulk amounts. In a DAF process this filter backwash water is combined with the top layer of DAF "float" which is mechanically and continuously scraped from the top of the DAF clarifier basins. The total volume of backwash water and "float" results in a process recovery typically in the range of 90 - 95%.

The ability of DAF to remove low density floc particles in cold water conditions means that it is typically preferred over Conventional Treatment for soft coloured raw waters. Some other advantages include:

- Very resistant to systems upset due to plant stop/stars or water quality changes;
- An ability to handle algae laden raw waters;
- Higher loading rate resulting in reduced clarification footprint requirements;
- Less flocculation requirements;
- Lower chemical dosages as a result of the smaller floc particle created; and
- Consistent removals at low water temperatures.

DAF treatment equipment is available from a variety of vendors within Canada.

4.4 Ballasted Floc Clarifiers

Ballasted floc clarifiers use a continuously recycled inert carrier such as silica sand (ballast) to increase the settling properties of the suspended floc in the clarifier. The settled material is recycled through a cyclone separator for separation of the residual waste sludge and the ballast material. The sludge goes to waste and the ballast is reintroduced to the clarifier inlet for subsequent settling. Gravity filtration with granular media (anthracite and sand) is commonly used after the ballasted clarification.

The ballasted flocculation clarification process addresses one of the primary concerns in treating coloured water; the slow settling rate of the resulting floc. The sand acts as a settling agent and greatly improves the settling velocity of the floc/sand particles thereby reducing footprint requirements for settling. This process is appropriate for a wide range of water qualities including low turbidity, coloured waters. Its use on algae laden waters is not well documented. There are some concerns regarding the service life of the sand handling equipment which could result in some high long term maintenance costs. Make-up sand is also required routinely as a result of loss through the sludge. Ballasted flocculation equipment is offered by one vendor in Canada.

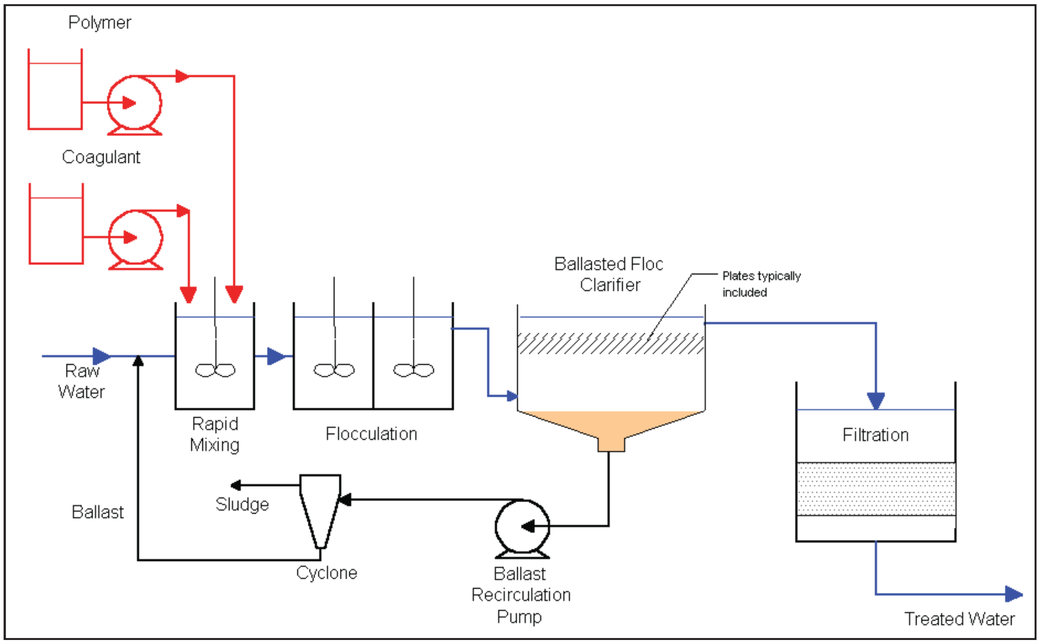


Figure 4.3: Ballasted Floc Clarification

4.5 Membrane Filtration Processes

Membranes are available in a variety of configurations. The most commonly used classifications in municipal water treatment are microfiltration (MF), ultrafiltration (UF), and nanofiltration (NF). Each classification is applicable to source waters of varying quality using both chemical and chemical-free processes. MF and UF membranes filter particles based on size exclusion. They have a rated pore size and any particles larger than the pore will not pass through the membrane. NF membranes remove particles using a combination of size exclusion and molecular filtration. The net result is that MF and UF will remove suspended particles, generally larger than 0.1-0.2 microns, while NF will remove suspended particles and some dissolved species such as divalent ions and long chain molecular compounds.

4.5.1 Microfiltration and Ultrafiltration Membranes

MF and UF membranes are both very effective at treating water with high levels of turbidity. Depending on the composition of the source water and TOC/DOC partitioning, coagulation/flocculation may be required for sufficient DOC removal and DBP formation potential reduction. Since both classifications act as absolute barriers to passage of pathogens and particulate matter, treated water will not fluctuate with variance in feed water quality. The removal of dissolved substances including colour, DOC, and metals is variable with water quality and often requires coagulation to achieve sufficient reductions. The term flux (units of flow per unit area per unit time) is applied to the loading rate of membranes. Increased flux under given conditions will result in a faster membrane fouling rate, which is comparable to head loss in a conventional filter, but will not cause a change in finished water quality.

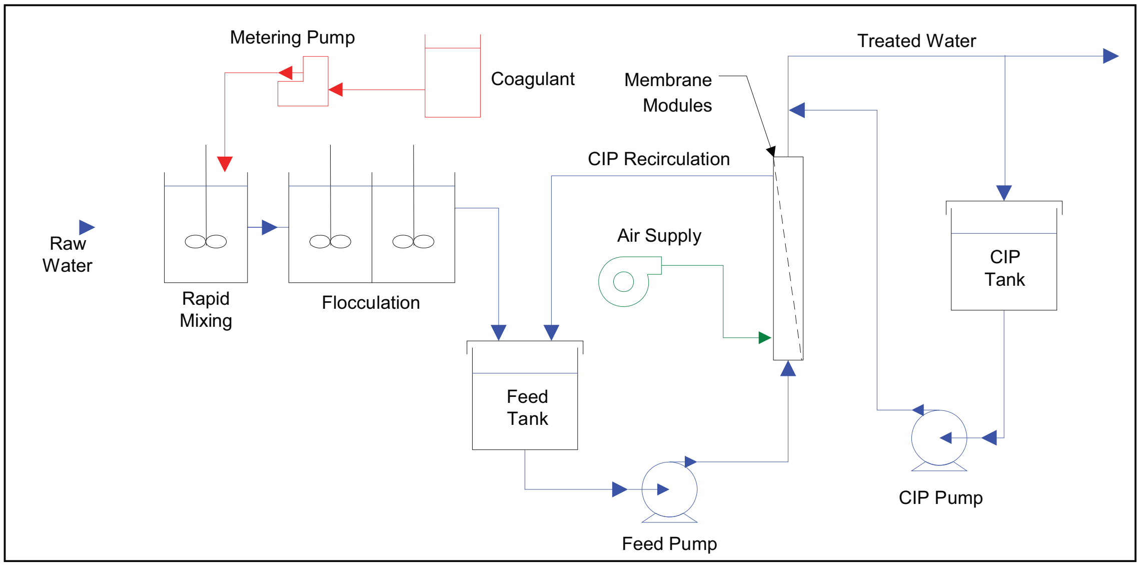


Figure 4.4: Typical Coagulation Membrane Treatment Process

In general, MF membranes will not remove a significant amount of colour when applied in direct filtration mode. Direct filtration with UF will achieve a higher level of colour removal than MF. However, in highly coloured source water, this will typically not result in finished water colour of an acceptable level. If a coagulant is applied upstream of a MF or UF membrane, micro-flocculation (or maintenance dosing) may be sufficient to allow measurable removal of DOC and colour compounds (DBP precursors). Micro-flocculation refers to inline coagulation without the use of large contact basins for the formation of floc particles which would be suitable for settling. For water sources where colour and TOC removal requirements are substantial, multi-stage flocculation with larger tanks is applied. Clarification can be applied in larger size applications for additional solids removal prior to filtration. The amount of floc formed affects the flux rate that a membrane can operate at. Higher amounts of floc formation require a lower operating flux and more membrane area for a given capacity.

4.5.2 Nanofiltration Membranes

NF is often capable of DOC and colour removal without the addition of a coagulant as a result of filtration at a molecular level. The limitations of NF are approached when using feed water containing suspended solids. Since NF membranes usually have a spiral wound configuration, solids in feed water cause plugging and premature failure. As a result, these types of membranes employ pre-treatment by media filtration (or other membrane filtration) and cartridge filtration upstream to ensure adequate runtime.

4.5.3 Other Parameters of Concern

Membranes are also capable of iron and manganese removal when in suspended form. Pre-treatment with aeration and/or an oxidant such as potassium permanganate prior to filtration increases metal removal rates, particularly when the source water contains dissolved species.

All three of MF, UF and NF provide an effective barrier to bacteriological contaminants including *cryptosporidium* and *Giardia.* Most jurisdictions grant a 3-5 log *giardia* removal for membrane filtration, which can increase with challenge testing. UF and NF also provide 3-4 log removal of viruses. A disinfectant is still required for any additional contact time and maintenance of safe water quality throughout the distribution system.

4.5.4 Membrane Recovery Rate

In addition to flux, the other critical operating parameter when examining membranes is the recovery rate. The recovery rate refers to the ratio of treated water volume to total water fed to a membrane system. Residual waste from a membrane system is comparable to backwash water in a conventional system, particularly if coagulation is being used.

MF and UF systems are often capable of operating comfortably at 95% recovery and can go as high as 97%. This is due to hollow-fibre filtration technology which allows high solids water to be in contact with the membranes. In circumstances of challenging water quality or enhanced coagulation, the recovery may be lower, at 90-92 %.

Spiral wound NF membranes operate at much lower recoveries due to the packed nature of the thin-film sheets and the need to avoid concentration/ precipitation of solids in the feed channel spaces. A typical NF recovery is 60-70% which results in greater raw water pumping volumes and increases residual waste handling requirements.

4.5.5 Membrane Option for Gagetown WTP

For the application of membrane filtration at C.F.B. Gagetown, a high level of colour and organics removal is required. Some removal of metals and other compounds are needed, but not enough to warrant treatment to the NF level since it has considerably higher capital and operating costs. For Gagetown UF or MF membranes preceded by a coagulation-based pre-treatment such as DAF, is the most appropriate membrane technology.

4.6 Process Evaluation

4.6.1 Process Selection

As per section 4.1 the 3 process themes identified as candidate processes for the new WTP are:

- Dissolved Air Flotation with Granular media filtration;
- Ballasted Flocculation with Granular Media Filtration; and
- Dissolved Air Flotation with Membrane Filtration.

While the membrane system will produce water with lower finished water turbidities, all three process themes have the capability of meeting all water quality objectives. The DAF clarification system is a superior particle separation technique for low density floc particles. However, we suspect that peak raw water turbidities have not been accurately recorded and have exceeded the instruments range on occasion. If there are many higher density particles in the raw water during these extreme water quality events, a ballasted flocculation system may have some advantages as compared to a DAF system as it may be easier to settle these particles than to float them. For this reason, and the fact that the building footprint is very similar for all 3 candidate processes, we have chosen to evaluate the costs of all of the candidate processes rather than select one preferred process at this stage. It would be beneficial to collect, analyse and conduct treatability testing on at least one extreme water quality event prior to making the final treatment process selection.

Table 4.1: Comparison of Water Treatment Process Options

Process	Advantages	Disadvantages
Dissolved Air Flotation with Granular media filtration	• Simple, cost effective technology; • Good at removal of low density particles; and • Well established technology with equipment available from a variety of vendors.	• Electrical energy associated with recycle loop; and • Not as effective with higher density particles.
Ballasted Flocculation with Granular Media Filtration	• Good at removal of low density particles and higher density particles; and • Small footprint.	• Proprietary process with one vendor; • Use of sand for ballast can cause premature equipment wear; and • Polymer and make-up sand use can be significant.
Dissolved Air Flotation with Membrane Filtration	• Highest quality finished water; • Lower filter effluent turbidities; and • More effective barrier for pathogen removal	• Cost , complexity; and • Replacement of membranes in 5 -10 years.

CHAPTER 5 WTP SITING AND CONCEPTUAL LAYOUT

5.1 Development of WTP Siting

In the preliminary stages on the project, it was decided that the new WTP would be located in a ball field on property owned by the Town of Oromocto, adjacent the existing WTP. The site is favourable due to the close proximity to the existing intake structure, distribution mains, access roads, and adequate power supply. The site is located near the shore of the Saint John River and finish floor elevation would have to be high enough to reduce the return period of a major flood event.

5.1.1 Utilization of Existing Infrastructure

The existing water intake was underwent refurbishment in the 1990's and was replaced with a 600mm pipe which feeds water into a low lift pump well. This would be an ideal location for the new plant to connect to the raw water intake and the existing connection to this low lift pump well would be abandoned once construction and commissioning was complete. No other existing infrastructure related to the treatment process would be reused for this project.

5.1.2 Waste Stream Disposal

The wastewater from the treatment process and other plant operations will be directed to the municipal sewer. A new 8" gravity sewer main with excess capacity will be installed 90m south to a connection to the gravity system. At this intersection a pumping station may be required to pump the wastewater along Onondaga Street and tie into the municipal system. This would potentially occur at a location prior to entry into the wastewater pumping station along Onondaga Street.

5.2 Conceptual Water Treatment Plant Layout and Design

5.2.1 Site Layout

The water treatment plant site at the existing ball field with the treatment plant is shown in the Proposed WTP Site Layout, Sketch 4. The WTP building will include a loading/unloading area, storage/work areas, offices and meeting space in addition to the basic requirements of the treatment plant itself. This will allow the building to serve as a centralized location for municipal staff included in the water utility operation. Access to the site and WTP can be via the existing access road which runs parallel to the river.

5.2.2 Building Layout

Plant layout drawings are presented in Sketches 1 - 3. Apart from the chlorine contact tank located beneath the process area; the entire treatment facility will be located on one level. This simplifies the design and associated cost of construction.

The new treatment plant building will be approximately 1400m^2 in footprint and will house treatment equipment, high lift pump station, cleaning equipment, chemical storage and dosing facility, new electrical room, office, washroom facilities, storage and sub grade tanks. Normal access to the treatment plant building will be from an access road through the office door on the northeast side of the building.

The treatment equipment, pre-treatment system and ancillary equipment are located in the equipment room which is approximately 1000 m^2 in size. Access to the process area can be gained via a 3.65 m overhead door for loading and unloading of equipment or via man-doors on the southwest and southeast sides of the building. The clear height of the building is anticipated to be 4.6 m (15 feet). This clearance will be more than adequate to service the treatment equipment and all tanks and other equipment inside the building. The final configuration of the building height and profile will be determined during detailed design.

The chlorination facility and chemical storage and dosing equipment are located in a separate storage and dosing room that is accessed by double doors near the back of the building. The chemical room and chemical delivery would be through an overhead door in the workshop area on the southeast side of the building.

The finished water pumps include three new pumps and a common flow meter that are open to the main process area and located in the north corner of the building. The finished water pump system includes visual access to the wet well through an access hatch for maintenance.

Mechanical, storage, and air compressor/blower rooms are located in the northeast area of the building. The mechanical room will house backflow preventers, hot water heating, fire system controls, and HVAC controls. The air compressor/blower room is separated from the main process area to limit noise in the process area. The storage area will provide space for general maintenance items and spare parts.

Offices and a lunch room are located in the southern corner of the building and include windows facing the road. The washroom facilities are along a hallway leading to a man door on the southeast side. This hallway can also be used to access the onsite lab facilities. As per code requirements, space for a separate motor control centre/electrical room that is fire rated has been allowed for. The exact dimensions and layout for this room will be finalised during detailed design.

A vehicle parking area is located to the southeast side of the building and allows sufficient room to maneuver trucks or other large equipment.

5.2.3 Architectural

The characteristics of a “Standard” industrial building for a water treatment plant as follows:

- Reinforced concrete foundations;
- Concrete slab on grade;
- Pre-cast concrete roof panels;
- Load bearing concrete block with brick or split faced block wall systems;
- Internal concrete block partitions; and
- Membrane roof system.

For CFB Gagetown, conditions inside the building such as open tanks and chemicals make the use of pre-cast concrete roof units applicable for corrosion resistance. Alternative roof structures could be considered during detailed design.

The wall system will be constructed of concrete block walls with an exterior split faced brick chosen for durability and aesthetics.

5.2.4 Mechanical

Building heat will be provided by electric unit heaters in the various personnel and equipment areas. Ventilation is provided with two heat recovery ventilators, one for the equipment areas and one for office, MCC, and auxiliary rooms. Plant service water will come from finished water in the storage reservoir. Sanitary sewage from the laboratory sink and washroom will be discharged to the wastewater equalization tank where it will mix with process wastewater.

5.2.5 Electrical

A new service entrance complete with automatic transfer switch will be brought into the new Treatment Plant Building just in front of the MCC room. Permanent stand-by power via diesel generator will be provided for the facility as per the Atlantic Canada Guidelines.

An assessment of the existing electrical feed will be conducted during detailed design.

5.2.6 Process Design

Although three treatment options have been considered for this project, there is still considerable variation in the exact configuration of the equipment that must be determined during detailed design. These factors include the number of treatment trains or redundant equipment, the amount of pre-treatment, the required potential future expansion factor (typically 10% - 20%), and the equipment supplier.

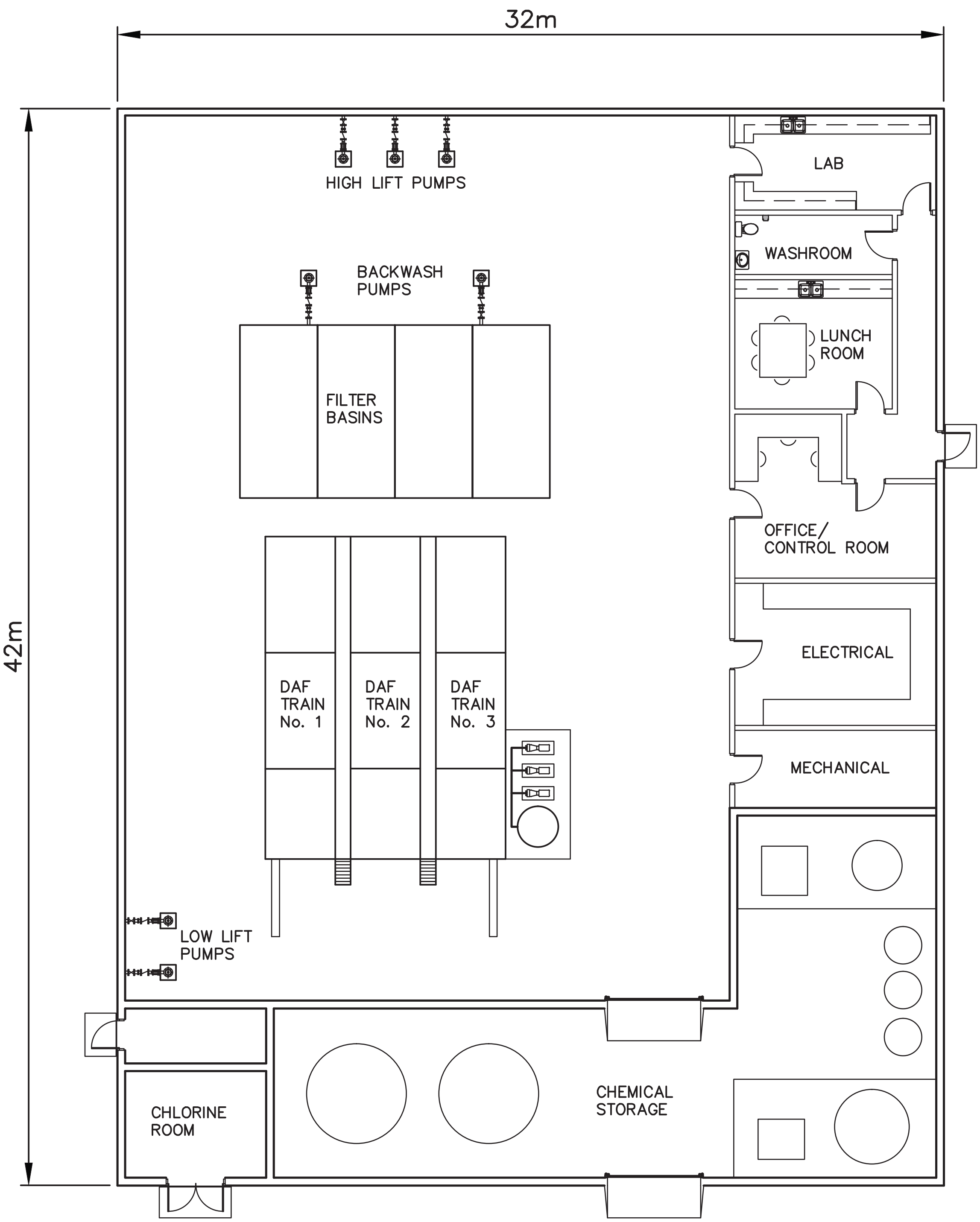
5.2.7 Treated Water Disinfection

Primary disinfection through UV disinfection has been allowed for in the process design. The presence of *cryptosporidium* and *Giardia* in the source water make UV disinfection an important part of the multi-barrier approach to providing clean and safe drinking to the system users.

In addition to UV, chlorine disinfection will be required in the new plant. Sizing of chlorine contact chambers is determined by assigning a value from the treatment standards to the CT value and establishing a hydraulic retention time needed adequately disinfect the water. In this design a CT value of 33 is required along with a chlorine residual of 0.5mg/L. A baffling system would be installed in the tank which increases the contact between the chlorine and water hence reducing the overall size of the tank by 0.7. The overall tank size will need to be a minimum of 850m^3 allowing for a minimum contact time of 94 minutes at peak flow.

5.2.8 Treated Water Storage and Fire Flows

Treatment plant production abilities are buffered from system peak demands by reservoirs located in the distribution system. This buffering allows peak water demands to be met while maintaining smaller peak production values in the plant. In addition to the existing distribution system reservoirs, there is a clearwell located within the treatment plant. It is recommended that a new plant has a clearwell which will help balance the system demands and increase high lift pump efficiency. A clearwell sized to hold 2500m^3 of storage would be adequate to meet the system needs.



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ENGINEERING STUDY FOR WATER AND WASTEWATER TREATMENT PLANTS, CFB GAGETOWN

PROPOSED WTP LAYOUT – DAF AND GRAVITY FILTRATION

Sketch
1

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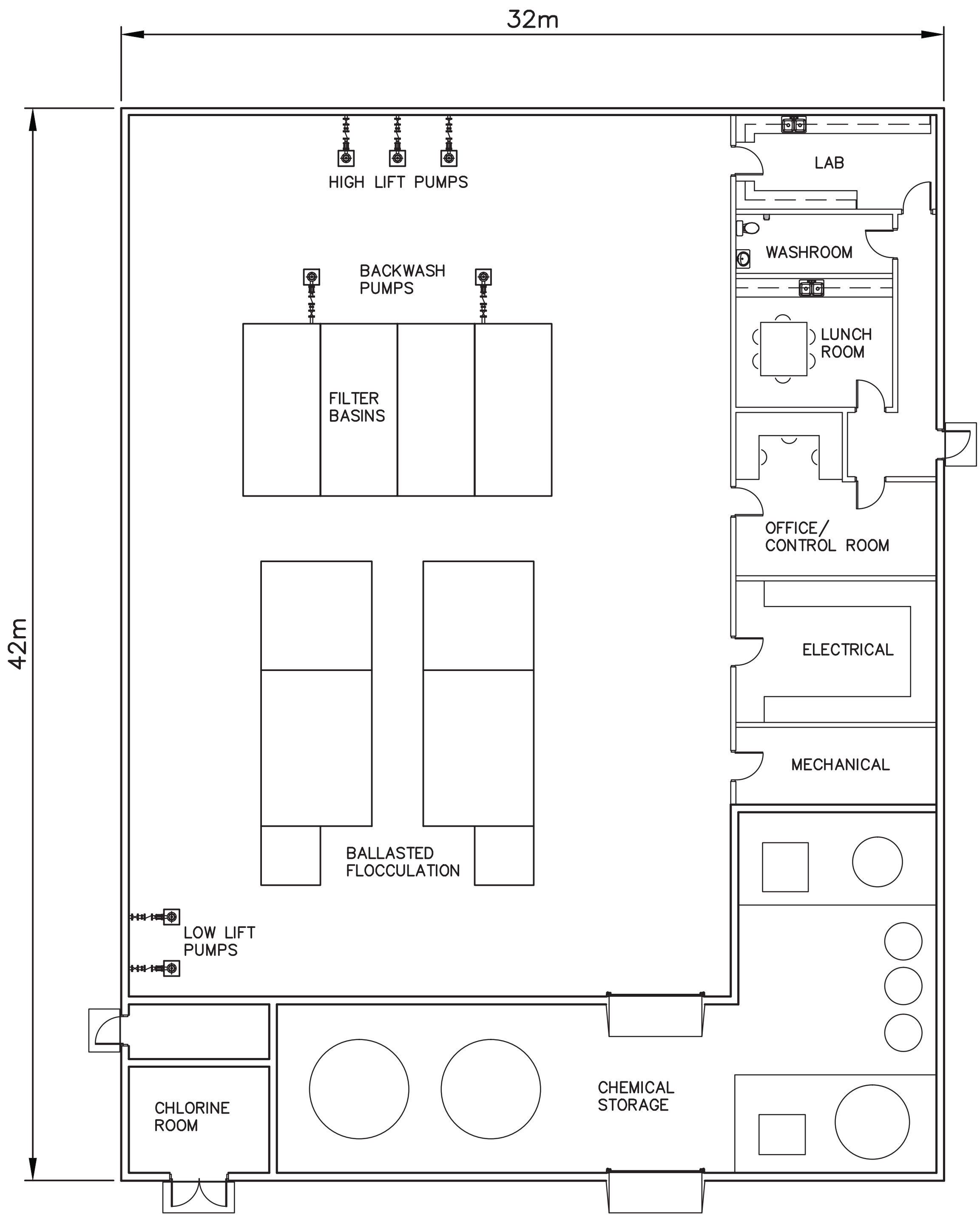

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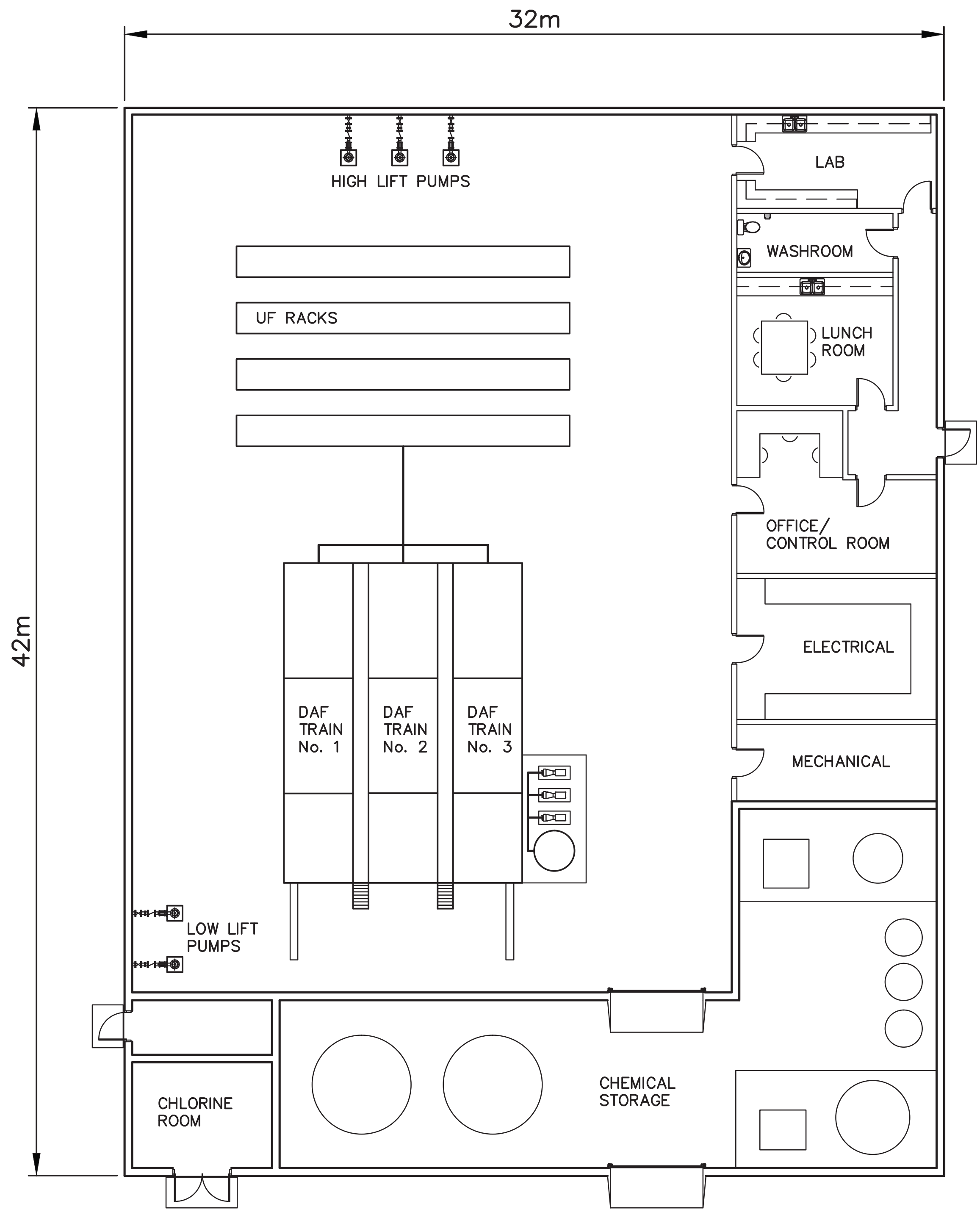
PROPOSED WTP LAYOUT – BALLASTED FLOCCULATION WITH GRAVITY FILTRATION

Sketch
2

DRAWING NAME: L:\123252 OROMOCTO\123252-50.DWG LAYOUT NAME: WTP PLOT DATE:December-06-12 1:50:05 PM CAD OPERATOR: MATTM



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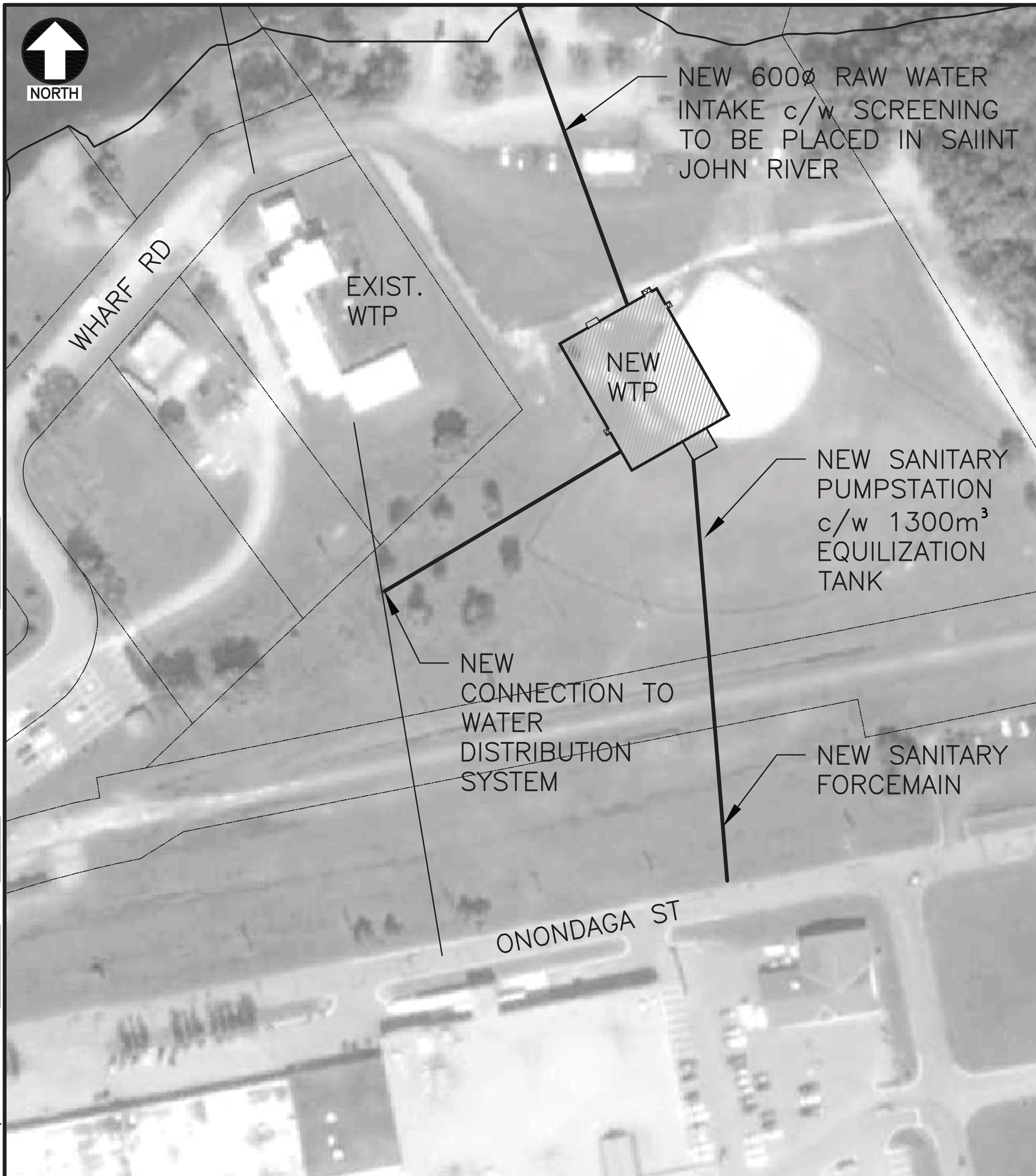
PROPOSED WTP LAYOUT – DAF AND MEMBRANE FILTRATION

Sketch
3



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ENGINEERING STUDY FOR WATER AND WASTEWATER TREATMENT PLANTS, CFB GAGETOWN

PROPOSED WTP SITE LAYOUT

Sketch **4**

DRAWING NAME: L:\123252 OROMOCTO\123252-50.DWG LAYOUT NAME: WTP PLOT DATE:December-07-12 3:20:42 PM CAD OPERATOR: MATTM

CHAPTER 6 PART A - WTP COST ESTIMATES

The tables below provide a summary of the probable construction costs for the treatment options being considered. Given the predesign nature of the estimates and the level of design development that has been carried out, these estimates can be considered to be very close in capital cost. Having said that, it is expected the DAF- Granular Media Filtration process to have some slight cost advantage. However, we are not prepared to recommend that this be selected as the preferred process at this time due to uncertainty associated with the extreme water quality events. As noted previously, the nature and extent of turbidity that occurs during these events in not well defined. Further testing should be conducted to determine the treatability of these extreme water quality events.

The cost estimates provided below are based on concept sketches and are thus considered Class D estimates that are conceptual in nature. Project documents (sketches) are in the initial stages but are sufficient to provide an indication of probable cost and allow ranking of options being considered. When preparing more accurate cost estimates, design drawings for civil, structural, process, mechanical and electrical designs are used to develop equipment lists and arrive at more accurate estimates (Class B or Class A). At this stage in the project, sketches are available for cost estimating and so quantities are approximate even if unit prices may be accurate.

Table 6.1: Predesign Opinion of Probable Cost for DAF and Gravity Filtration
CLASS D Project Budget - Option #1

Item Description		Total
Division 2 - Site Civil		
Site Preparation		
Excavation and Site Grading		
Rock Excavation		
Imported Fill and Granular Material		
Pavement (roads, parking areas)		
Gravel Access Roads		
Yard Pipework, MH's, Pipe Connections, Etc.		
Site Fencing		
Site Finishes & Reinstatement		
Environmental Protection & Testing		
Subtotal - Division 2		**$ 1,481,050**

Item Description		Total
Division 3 - Concrete & Waterproofing		
Clearwell Backwash Equilization Tank Chlorine CT Tank Intake Well & Process Tanks		
Subtotal - Division 3		**$ 887,473**
Division 4 to 10 - Buildings		
Water Treatment Plant		
Subtotal - Division 4 to 10		**$ 1,815,000**
Division 11 & 14 - Process Equipment Supply		
Process Equipment Supply U.V. Disinfection System Pretreatment Chemical Systems Low Lift Pumps High Lift Pumps Backwash System Material Handling Equipment Diesel Generator		
Subtotal - Division 11 & 14		**$ 2,550,000**
Division 15 - Mechanical		
Process Mechanical Pipe, Valves, Etc., Supply Process Mechanical Install Plumbing, Heating & Ventilation		
Subtotal - Division 15		**$ 2,865,000**
Division 16 - Electrical & Instrumentation		
Power Distribution and General Electrical		
Instrumentation and Control		
Subtotal - Division 16		**$ 1,276,000**
Subtotal		**$ 10,874,523**
Design Development Contingency	25%	$ 2,718,631
Construction Contingency	10%	$ 1,359,315
Total Estimated Construction Cost (not including taxes)		**$ 14,952,468**
Full Engineering Svc (10%)	10%	$ 1,495,247
Total Estimated (not including taxes)		**$ 16,447,715**

Table 6.2: Predesign Opinion of Probable Cost for Ballasted Flocculation and Gravity Filtration CLASS D Project Budget - Option #2

Item Description		Total
Division 2 - Site Civil		
Site Preparation		
Excavation and Site Grading		
Rock Excavation		
Imported Fill and Granular Material		
Pavement (roads, parking areas)		
Gravel Access Roads		
Yard Pipework, MH's, Pipe Connections, Etc.		
Site Fencing		
Site Finishes & Reinstatement		
Environmental Protection & Testing		
Subtotal - Division 2		**$ 1,481,050**
Division 3 - Concrete & Waterproofing		
Clearwell		
Backwash Equilization Tank		
Chlorine CT Tank		
Intake Well & Process Tanks		
Subtotal - Division 3		**$ 887,473**
Division 4 to 10 - Buildings		
Water Treatment Plant		
Subtotal - Division 4 to 10		**$ 1,815,000**
Division 11 & 14 - Process Equipment Supply		
Process Equipment Supply		
U.V. Disinfection System		
Pretreatment Chemical Systems		
Low Lift Pumps		
High Lift Pumps		
Backwash System		
Material Handling Equipment		
Standby Generator		
Subtotal - Division 11 & 14		**$ 2,750,000**
Division 15 - Mechanical		
Process Mechanical Pipe, Valves, Etc., Supply		
Process Mechanical Install		
Plumbing, Heating & Ventilation		
Subtotal - Division 15		**$ 3,005,000**
Division 16 - Electrical & Instrumentation		
Power Distribution and General Electrical		
Instrumentation and Control		
Subtotal - Division 16		**$ 1,376,000**

Item Description		Total
Subtotal		**$ 11,314,523**
Design Development Contingency	25%	$ 2,828,631
Construction Contingency	10%	$ 1,414,315
Total Estimated Construction Cost (not including taxes)		**$ 15,557,468**
Full Engineering Svc (10%)	10%	$ 1,555,747
Total Estimated (not including taxes)		**$ 17,113,215**

Table 6.3: Predesign Opinion of Probable Cost for DAF and Membrane Filtration CLASS D Project Budget - Option #3

Item Description		Total
Division 2 - Site Civil		
Site Preparation		
Excavation and Site Grading		
Rock Excavation		
Imported Fill and Granular Material		
Pavement (roads, parking areas)		
Gravel Access Roads		
Yard Pipework, MH's, Pipe Connections, Etc.		
Site Fencing		
Site Finishes & Reinstatement		
Environmental Protection & Testing		
Subtotal - Division 2		**$ 1,481,050**
Division 3 - Concrete & Waterproofing		
Clearwell		
Backwash Equilization Tank		
Chlorine CT Tank		
Intake Well & Process Tanks		
Subtotal - Division 3		**$ 887,473**
Division 4 to 10 - Buildings		
Water Treatment Plant		
Subtotal - Division 4 to 10		**$ 1,815,000**
Division 11 & 14 - Process Equipment Supply		
Process Equipment Supply		
U.V. Disinfection System		
Pretreatment Chemical Systems		
Low Lift Pumps		
High Lift Pumps		
Backwash System		
Material Handling Equipment		
Diesel Generator		
Subtotal - Division 11 & 14		**$ 3,100,000**

Item Description		Total
Division 15 - Mechanical		
Process Mechanical Pipe, Valves, Etc., Supply		
Process Mechanical Install		
Plumbing, Heating & Ventilation		
Subtotal - Division 15		**$ 3,250,000**
Division 16 - Electrical & Instrumentation		
Power Distribution and General Electrical		
Instrumentation and Control		
Subtotal - Division 16		**$ 1,550,000**
Subtotal		**$ 12,083,523**
Design Development Contingency	25%	$ 3,020,881
Construction Contingency	10%	$ 1,510,440
Total Estimated Construction Cost (not including taxes)		**$ 16,614,843**
Full Engineering Svc (10%)	10%	$ 1,661,484
Total Estimated (not including taxes)		**$ 18,276,328**

CHAPTER 7 PART B – WASTEWATER TREATMENT PLANT

7.1 Introduction

The Department of National Defence (DND) operates the CFB Gagetown Wastewater Treatment Plant (GWWTP) serving Canadian Forces Base Gagetown and most of the sanitary sewage generated in the Town of Oromocto. The original plant was built between 1953 and 1956 and has seen upgrades and expansion to the treatment process over the years. The existing facility consists of raw sewage screening and a bypass channel located in the headworks building, primary clarifiers, aeration basins, secondary clarifiers, chlorine contact tanks, dechlorination, anaerobic sludge digesters, and a geotextile bag dewatering system located on the old sludge drying beds. Effluent is released into the Saint John River via an outfall pipe that discharges approximately 1.5 kilometres north of the WWTP. On average, the plant discharges 5,500 m^3/day of treated wastewater into the Saint John River.

7.2 Part B Study Objectives – Wastewater Treatment Plant (WWTP)

After review of the Consultant Briefing document for the project (GA 183524), CBCL Limited responded with a proposal outlining the following objectives:

1. Establish the Design Flows for New Wastewater Treatment Facility.
2. Characterize the Existing Raw Wastewater Quality.
3. Preliminary Evaluation of Wastewater Treatment Options.
4. Develop Preliminary Designs for Wastewater Treatment Facilities.
5. Review of Plant Siting Requirements.
6. Carry out Cost Estimates of Preferred (recommended) Alternative.

CHAPTER 8 WWTP DESIGN CRITERIA

8.1 Gagetown Wastewater Treatment Plant Flow and Loads

All sanitary sewage from Canadian Forces Base Gagetown is collected and conveyed to the Gagetown WWTP for treatment. The majority of sanitary sewage from the Town of Oromocto is also treated at the Gagetown WWTP. The remainder flows to the Oromocto West WWTP, which is currently part of a separate collection system.

Flow is recorded inside the headworks building at the plant and consists of treated flow plus bypass flow as shown in the following chart. The graph depicts treated flow in the lighter shaded area (green) and bypass flow in the darker area on top (red).

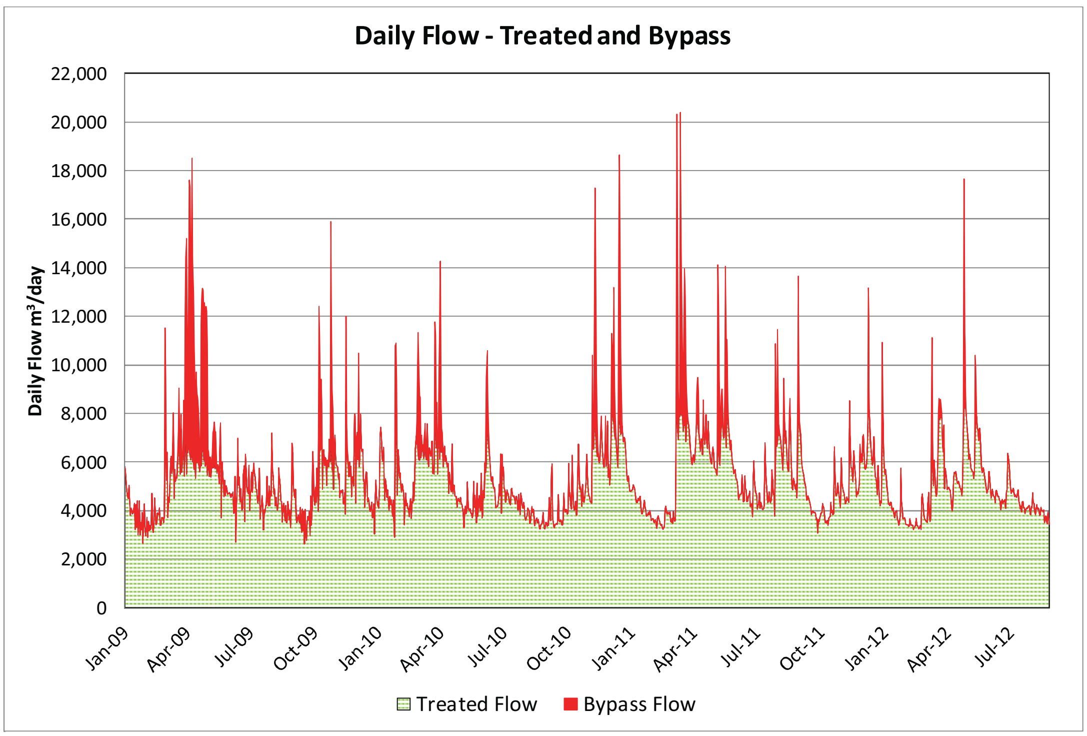


Figure 8.1: Daily Flow and Bypass Flow Arriving at the GWWTP

As indicated in Figure 8.1, bypass flows of high intensity but short duration frequently occur during spring and fall. The average daily flow (ADF) arriving at the plant over the last three-and-a-half years was 5540 m^3/day. This includes Bypass Flows which occurred on 322 days out of the 1334 days of recorded data. A review of the chart and the data reveals that over this period, the maximum daily flow arriving at the GWWTP has twice exceeded 20,000 m^3/day (236 L/sec); occurring on March 8th and March 13th of 2011. The plant experiences exceptionally high peak flows that approach 400% of the average daily flow.

Since most of the sanitary collection system dates back to the original founding of the Military Base and the Town's infrastructure, most sewer pipe is 60 years old. It is not uncommon for a collection system of that vintage, to experience high inflow and Infiltration (I&I). Reducing I&I can save operating and maintenance costs for pumping and treatment, as well as capital costs for future capital works by reduce demand and capacity.

8.1.1 Town of Oromocto Sewage Flow Treated at Gagetown WWTP

Figure 8.2 depicts sewage flow measured at Manhole 96P, which represents sewage flow from the Town of Oromocto prior to treatment at the Gagetown WWTP. Flow from the Town of Oromocto reportedly makes-up 80% of the flow that arrives at the GWWTP.

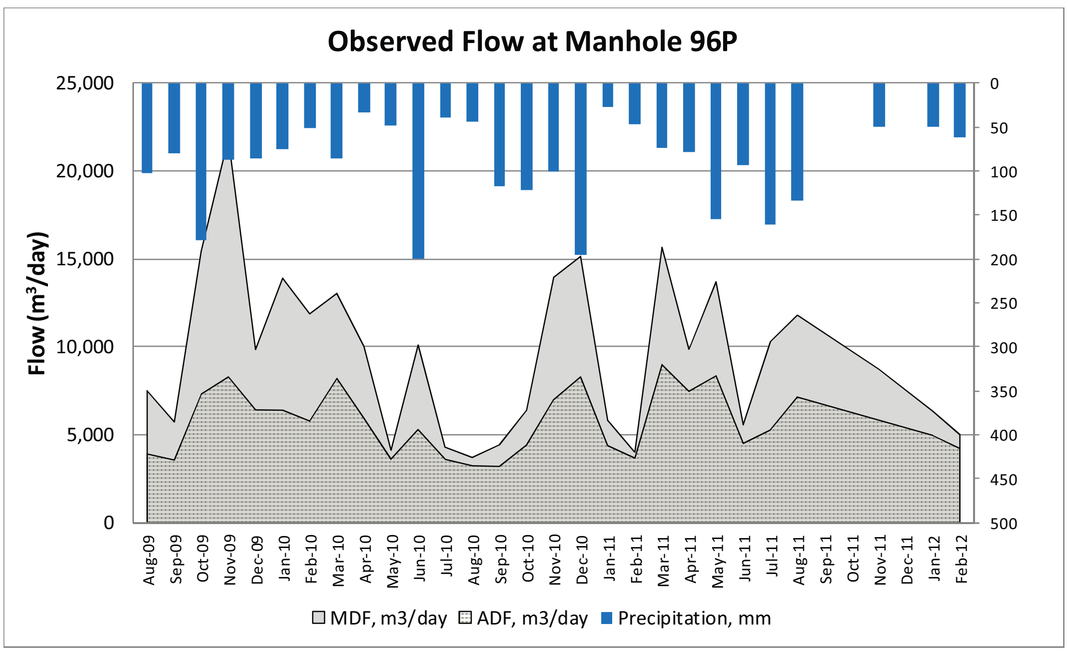


Figure 8.2: Summary of Average Day, Max Day and Precipitation from Flow Monitoring Study (RV Anderson)

Flow recordings are taken on 5-minute intervals using an area-velocity flow meter set up in Manhole 96P. Average daily flow, maximum daily flow and peak instantaneous flow can be calculated from this flow monitoring work. The chart above depicts maximum daily flow (MDF) and average daily flow (ADF) with monthly precipitation (snowmelt) also shown on the secondary axis to the right. As one would expect, months with high precipitation correlate directly with periods of high flow at the monitoring manhole. A few other things can be learned from the data and this chart:

1. The Town of Oromocto's collection system is susceptible to inflow and infiltration. Inflow occurs when rainfall or snow melt enters the collection system from the surface as evidenced by a rapid increase in measured flow in the pipes. This is apparent during dry periods when a rainfall event with high-intensity and short-duration occurs.
2. Infiltration occurs when water from a high ground water table enters a collection system that has been compromised by cracks, faulty joints etc. Infiltration can persist for days after a rainfall or snowmelt because the ground water table was high preceding the event.

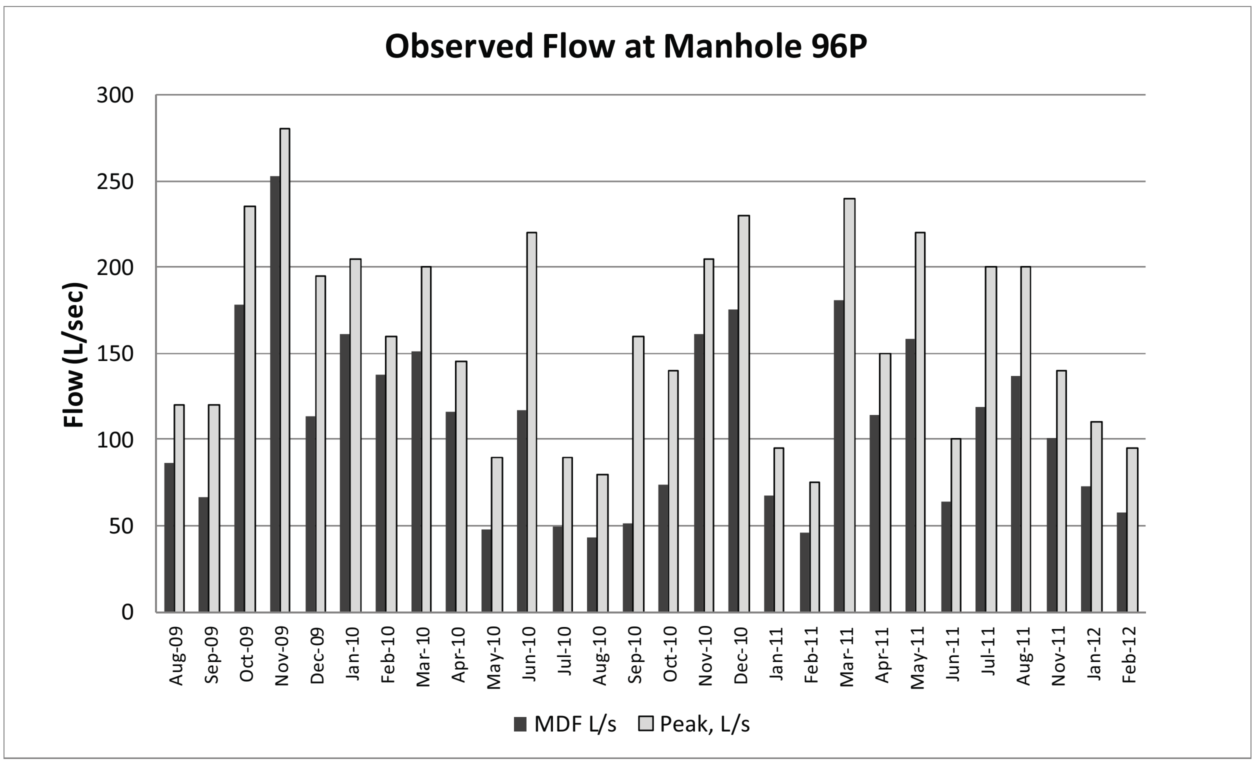


Figure 8.3: Maximum Daily Flow versus Peak Instantaneous Flow in L/sec

A comparison of maximum daily flow (MDF) to peak instantaneous flow (PF) is provided in Figure 8.3. The PF was 280 L/s on November 20, 2009 and the MDF was 250 L/s on that same day. The magnitude of these recordings is suspect; however, as the pipe was flowing full representing a problem for this type of flow meter. A few other PFs of note were recorded in the 220 to 240 L/s range.

It is evident that MDF and PF are pretty close especially during wet months in the spring and fall when the ground water table is high. During the period between August 2009 and February 2012, the average ratio of Peak Flow to Maximum Daily Flow (PF to MDF) within the Oromocto collection system was 1.6.

During periods of wet weather in spring and fall the peaking factor was 1.3. This information will be useful in predicting the combined Peak Flow to be used for design of plant hydraulics.

8.2 Raw Sewage Characteristics at Gagetown WWTP

Raw sewage characteristics for chemical oxygen demand (COD) and total suspended solids (TSS) are shown in Figure 8.4 for the period between January of 2009 and August of 2012. There were 1311 observed TSS results and 645 COD results recorded. Biochemical oxygen demand (BOD) was not measured during the last three-an-a-half years. Thirty-one raw sewage ammonia (NH_3) samples were recorded. Raw sewage entering the GWWTP could be characterized as weak in strength and consists almost entirely of residential and commercial sources with little or no known industrial component. The average raw sewage characteristics are presented in Table 8.1.

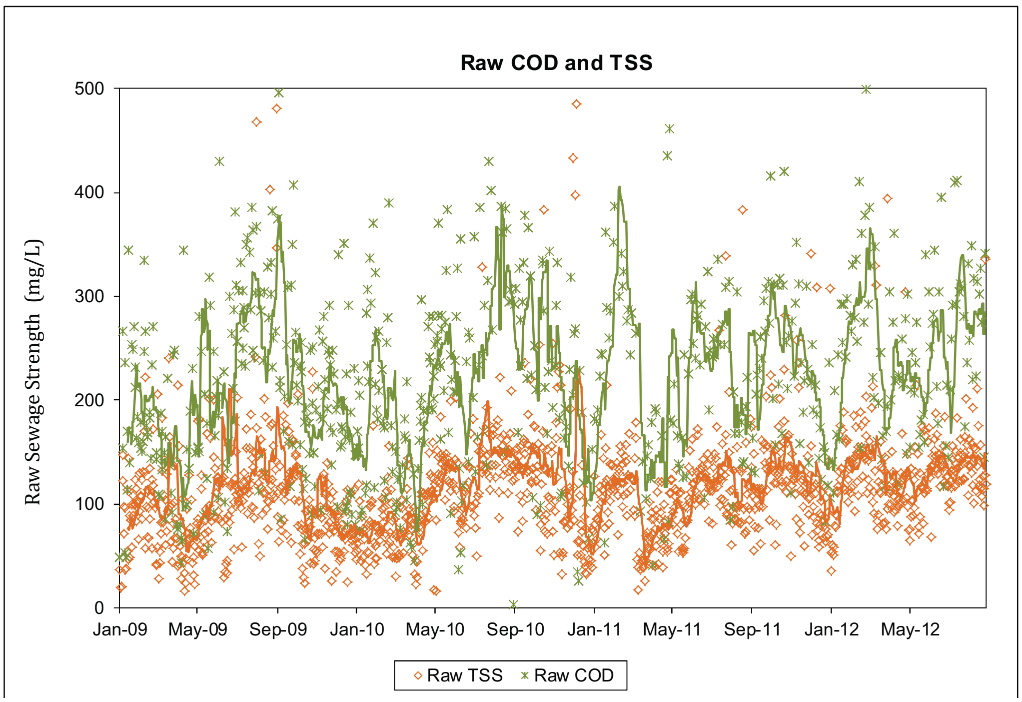


Figure 8.4: Raw Sewage Characteristics at Gagetown WWTP 2009 to 2012

Table 8.1: Gagetown WWTP Raw Sewage Characteristics

Parameter	Average Concentration, mg/L	# Observations
TSS	117	1311
COD	223	645
BOD_5	not available	0
TKN	not available	0
NH_3	14	31
TP	not available	0
PO_4	not available	0

Prior to detailed design, additional raw sewage data should be collected in a raw sewage composite sampling program. As a minimum, the raw sewage parameters listed in Table 8.2 should be characterized over a two month period.

8.3 Flows and Loads at the Oromocto West WWTP

Oromocto West is located on the south-west side of the Trans Canada Highway upriver from the Gagetown WTP. Sanitary sewage from that sewershed is captured and treated at a small packaged plant, called the Oromocto West WWTP. Approximately 2000 people live in the service area. Most of the Town's growth is now occurring in Oromocto-West, and so the treatment plant has seen increasing stress on its operations especially during wet weather events. To provide relief from this situation, a pumping station and forcemain was commissioned in late 2011 to divert a portion of the Oromocto-West sewage to the much larger wastewater treatment plant at CFB Gagetown. Flow data suggests that this project was successful in diverting as much as 25% of average daily flow. The plan going forward is to combine the Oromocto-West collection system with the Town's main collection system and then decommission the Oromocto West Facility.

The average daily flow recorded from Oromocto-West was 885 m^3/day between 2008 and 2009. More recently in 2012, the average daily flow was reduced to 660 m^3/day, as a result of the sewer diversion project mentioned above. A conservative approach will be to assume that the ADF flow from Oromocto West will contribute 885 m^3/day to the average flow at the new GWWTP once the collection systems are joined. Maximum daily flows, peak flows and wastewater characteristics are presented in the following sections.

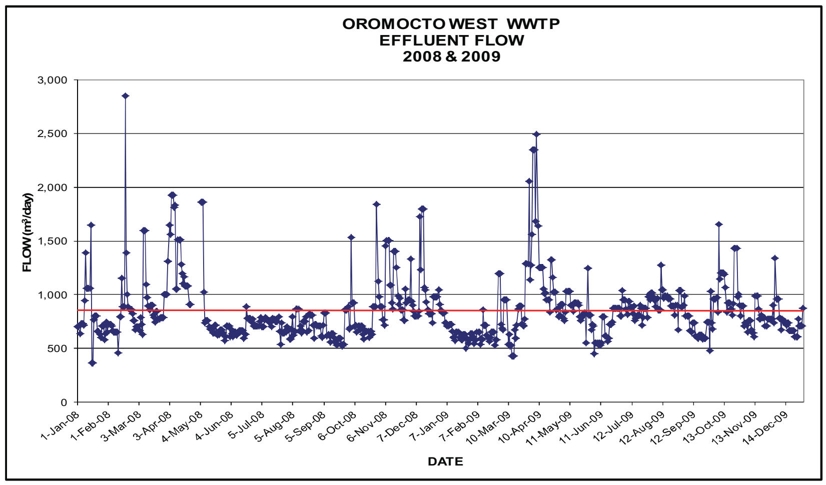


Figure 8.5: Recorded Flow between 2008 and 2009

8.3.1 Raw Sewage Characteristics at Oromocto West

The raw sewage from Oromocto West is residential in nature with characteristics as shown in Table 8.2. The organic strength is measured in terms of chemical oxygen demand (COD), Biochemical Oxygen Demand (BOD) and total suspended solids (TSS).

Table 8.2: Oromocto West WWTP Raw Sewage

Parameter	COD, mg/L	TSS, mg/L	BOD[1], mg/L
Average Day	366	169	166
Average per capita generation rate		0.07 kg/capita/day	0.06 kg/capita/day

Note: [1.] Raw sewage BOD is estimated based on an established COD/BOD ratio of 2.2

8.4 Design Sewage Flow

An important part of the Wastewater Treatment Plant Predesign study is the development of design criteria for a new facility that can provide service to the Base and the Town of Oromocto in its entirety. Table 8.3 is a summary of the present flows and the projected flows for the design year in 2036.

For design of biological processes within the treatment plant, average daily flow is typically referred to for process tankage, maximum daily flow and loads are applied for process aeration requirements and maximum month flows and loads are typically used for overall process design since effluent requirements are typically based on monthly averages.

Peak instantaneous flow is necessary for design of plant hydraulics. Actual recorded peak flows at the GWWTP could be suspect because of reported incidences of influent channels overflowing indicating that hydraulic capacity at the plant is undersized. Peak flow data measured at the plant can be augmented by peak flow to maximum daily flow (PF/MDF) peaking factor from the ongoing monitoring program at manhole 96P.

Table 8.3: WWTP Design Flow Summary

	Gagetown WWTP	Oromocto West WWTP	Present Day Combined Flow	Design Year 2036[1]
Average Daily Flow (m^3/day)	5,540	855	6,395	8600
Max Day Flow (m^3/day)	20,391	2,500	22,891	30,900
Max Month Flow (m^3/day)	10,200	1,370	11,570	15,600
Peak Instant Flow (L/sec)	330[2]	40	370[2]	500

Notes:

1. Population growth is assumed to be 35% over the next 25 years. Design Year flows are arrived at by inflating Present Day flows by 35%.
2. The capacity of the influent line to the plant is 330 L/s when flowing full.

A maximum daily flow of 236 L/s (20,390 m^3/day) was observed at the GWWTP. If we apply the peak flow to maximum daily flow (PF/MDF) ratio of 1.3 determined in the 96P flow monitoring study, then a peak flow of 307 L/sec (26,508 m^3/day) would be anticipated. This peak flow corresponds with the capacity of the influent line that to the treatment plant. If the collection system is expanded to include the addition of Oromocto West, the influent line to the plant may become a bottleneck and pipes will flow full in that part of the system. With that being the case, lower portions of the collection system will in fact be used to store instantaneous peak flows which is not uncommon but can cause problems for service connections in those locations.

8.5 Summary of Design Criteria

The raw sewage design criteria for the new wastewater treatment plant upgrade is summarized below.

Table 8.4: Gagetown WWTP Design Criteria - Raw Sewage Flows and Loads

Year	2012	Design Year 2036
Design Service Population	9,329	12,421
	Annual Average Day	
Flow, m^3/day	5,540	8,600
Average Strength		
COD, mg/L	223	223
BOD_5, mg/L [a]	-	-
TSS, mg/L	117	117
TP, mg/L [a]	-	-
PO_4, mg/L[a]	-	-
TN, mg/L [a]	-	-
NH_3, mg/L	15	15
	Design - Maximum Month	
Flow, m^3/day	10,200	15,600
MM Load[b]		
COD, kg/day	2275	3479
BOD_5, kg/day	-	-
TSS, kg/day	1193	1825
TP, kg/day	-	-
PO_4, kg/day	-	-
TN, kg/day	-	-
NH_3, kg/day	153	234
Temperature, °C		
Min	8	8
Max	22	22
Peak Hourly Flow, L/sec [e]	370	500
(m^3/day)	(32,000)	(43,200)

Notes:

a. Concentrations in raw sewage to be determined in composite sampling program
b. Maximum month design loads based on max month flow x average concentration of individual parameters

8.5.1 Effluent Requirements

Effluent requirements for the Gagetown Wastewater Treatment Plant will be dictated by three separate regulatory requirements.

1. *The Wastewater Systems Effluent Regulations published in Canada Gazette Part II on July 18, 2012.*

According to the Canada-wide Strategy, effluent for all facilities must meet the National Performance Standards (NPSs) and site-specific Effluent Discharge Objectives (EDOs). The National Performance Standards are minimum performance requirements for effluent quality from wastewater facilities that discharge to surface water. The NPSs under the Canada-wide Strategy include the following guidelines:

- Five day carbonaceous biochemical oxygen demand (CBOD5) < 25 mg/L;
- Total suspended solids (TSS) < 25 mg/L;
- Total residual chlorine (TRC) < 0.02 mg/L; and
- Unionized Ammonia < 1.25 mg/L; nontoxic effluent

In addition to NPSs, the following guidelines apply to the wastewater effluent quality as it exits the CFB Gagetown WWTP and are included in this report for comparison purposes:

2. *1976 Guidelines: The Environment Canada Guidelines for Effluent Quality and Wastewater Treatment at Federal Establishments (April 1976).*
   The 1976 Guidelines, apply to all effluents discharged from land based establishments under the direct authority of the Federal Government. The 1976 Guidelines give specific limits for concentrations of several water quality parameters including biochemical oxygen demand (BOD), suspended solids, fecal coliforms, chlorine residual, pH, phenols, oils and greases, phosphorus and temperature.

3. *Provincial Approvals to Operate*
   Both the CFB Gagetown WWTP and the CFB Gagetown Collection System must comply with provincial approvals to operate (S-C19-P1-05 and S1215, respectively, NBENV). Within the WWTP Approval to Operate are effluent standards for the effluent from the wastewater treatment facility for BOD5 (20 mg/L), suspended solids (20 mg/L) and total residual chlorine in the final discharge (0 mg/L). These standards apply to daily averages from May to October, inclusive, with winter operation based on good and efficient operation.

Table 8.5: Proposed Effluent Parameters

Parameter	Value
$cBOD_5$	20 mg/L
TSS	20 mg/L
Total residual chlorine (TRC)	< 0.02 mg/L
Ammonia-N	< 1.25 mg/L; Not acutely toxic
Fecal Coliforms and E. Coli	<200 MPN/100 mL

CHAPTER 9 WWTP PROCESS UPGRADE OPTIONS

9.1 Secondary Treatment Processes

This chapter deals with the heart of the wastewater treatment plant, the secondary treatment or biological treatment process. A list of secondary process options is presented in Table 8.1 and candidate processes are pre-screened. A description and evaluation of the processes that are short-listed is then provided.

There are numerous biological treatment processes with proven track records in this region that can achieve the effluent requirements. Biological or secondary treatment processes can be distinguished as either suspended growth, attached growth or combined. For a facility similar in size to the Gagetown WWTP, suspended growth systems are most common in Atlantic Canada and use aeration and mixing to keep microorganisms or biomass in suspension. A list of candidate secondary treatment processes is presented in Table 9.1.

All of the treatment processes in Table 9.1, except lagoons, are capable of meeting or exceeding effluent requirements for BOD, TSS and un-ionized ammonia year-around for sewage at the GWWTP. Nitrification and nutrient removal is more commonly achieved with suspended growth systems such as activated sludge, Biological Nutrient Removal (BNR) or Sequencing Batch Reactors (SBRs). Attached growth systems such as trickling filters and RBCs can be designed for nutrient reduction; however, they are usually operated in conjunction with some form of suspended growth process.

Of the process technologies listed above, Sequencing Batch Reactors, Membrane Bioreactors and Biological Nutrient Removal are discussed further.

Table 9.1: Candidate Secondary Treatment Processes for the Gagetown WWTP Upgrade

Process	Advantages	Disadvantages
Extended Aeration. *i.e.* *New Glascow, NS*	• No primary clarifiers; and • Simple to operate.	• Large bioreactors (18hr HRT); • High energy cost; • Will nitrify but does not have nitrogen removal without selectors; • Waste sludge may be difficult to dewater; and • Requires Chemical P-Removal.
Sequencing Batch Reactor (SBR) *i.e. Truro, NS* ***Considered Further***	• No primary or secondary clarifiers; • Process is automated; and • Can be operated in storm flow mode.	• Completely dependent on PLC controls; and • Large bioreactors (18hr HRT). • Proprietary equipment and controls; • Waste sludge may be difficult to dewater; and • Requires Chemical P-Removal.
Membrane Bioreactor (MBR) *i.e. Bridgewater, NS* ***Considered Further***	• Fine screens in lieu of primary clarifiers; • Membrane filtration in lieu of secondary clarifiers; • Highest quality effluent possible; • Process is automated; and • Smaller footprint.	• Typically highest Capital and Operating cost; • Proprietary process, equipment and controls; • Will require chemical for phosphorus removal; and • Completely dependent on instrumentation and PLC controls.
Biological Nutirient Removal (BNR) *i.e. Summerside, PEI* ***Considered Further***	• Same as activated sludge with addition of un-aerated cells; • Non-proprietary technology; • Can achieve N and P removal without chemicals; and • Uses less energy.	• Requires more knowledgeable operators; and • Requires submersible mixers and recycle pumps.
Lagoons *i.e. Quispamsis,* *Mirimachi, NB* *Kings County, NS*	• Simple to operate; • Low cost to operate at small scale; and • Are very common in NB.	• Prohibitive land requirements; • Difficulty meeting effluent requirements; • Minimal control over treatment process; and • High energy cost at large scale. 4000m^3/day and greater.
Activated Sludge *i.e. Gagetown,* *Fredericton, Saint John,* *NB*	• Same treatment process as existing; and • Smaller aeration basins.	• Will nitrify but does not have nitrogen removal without addition of unaerated zones; and • Requires Chemical P-Removal.

Process	Advantages	Disadvantages
Rotating Biological Contactor (RBC) *i.e. Halifax, NS*	• Simple to operate; and • Low energy usage.	• High maintenance cost, shaft and bearings; • Inflexible for upgrades; • Difficulty meeting effluent requirements; and • Will require chemical P-removal.
Trickling Filter - Solids Contact *i.e. Saint John, NB*	• Smaller footprint; and • More resistant to shock loads.	• Capital cost; • Reduced ability to control process sludge age and effluent quality; and • Requires chemical P-removal.

9.2 Sequencing Batch Reactors

Sequencing batch reactor (SBRs) technology is considered further because it can achieve the required effluent on a consistent basis, requires a relatively small footprint and at the lowest capital cost.

SBR technology is based on a fill-and-draw batch process without the need for clarifiers. It differs from continuous flow systems in that the biological treatment and separation of solids occur in sequence within the same tanks. As a result the process is considered batch in nature and so multiple tanks (process trains) are often provided to smooth influent and effluent flows. Figure 9.1 is a schematic depicting process operation, which consists of the following phases:

- Fill: Influent fed to the reactor; rise in liquid level;
- React (120 minutes): Aeration and mixing of the reactor contents;
- Settle (60 minutes): Quiescent period with solid-liquid separation; and
- Draw or Decant (60 minutes): Discharge of clear treated effluent.

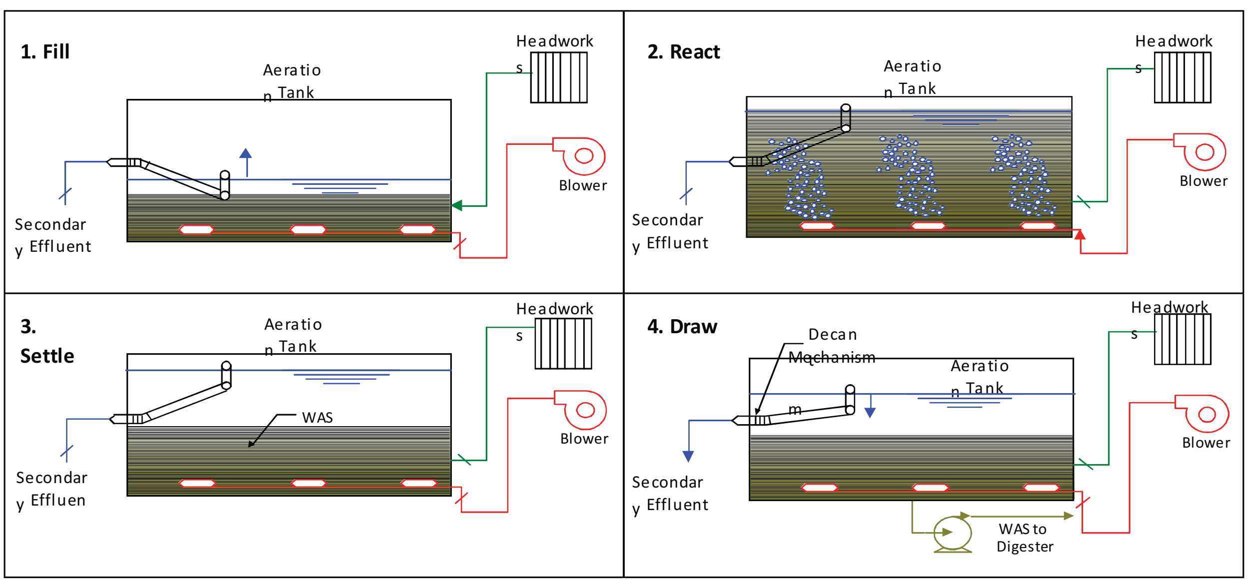


Figure 9.1: SBR Process Sequence

The example cycle time described will take 4hrs (240 minutes) to complete and will be repeated 6 times per day. That means that each basin will discharge treated effluent for 6 hrs per day (6 x 60 minutes). If there are 3 SBR process trains, then there will be 18hrs of treated effluent discharged (3 x 6hrs).

SBR's are operated at long solids and hydraulic retention times, resulting in large reactor volumes. Since there are no clarifiers, the result is often reduced overall area requirements especially for small to medium sized facilities. For larger plants, capital costs can be higher than activated sludge processes since more equipment and larger bioreactors are required.

The various process phases are initiated and terminated on a timed basis by a programmable logic controller (PLC). Phase durations can be adjusted to match flow and loading conditions. As well, by modifying the reaction times, nitrification or nitrogen removal can usually be accomplished. Table 9.2 is a summary of some typical design parameters for an SBR process sized for Gagetown.

Table 9.2: Sequencing Batch Reactor (SBR) Process Design

Parameter	Typical Design Standard	Design Year 2036
No. of Reactors	2 to 4	3
Basin Length (m)	-	39
Basin Width (m)	-	13
Top Water Depth (m)	-	6
Total Reactor Volume (m^3)	-	9,126
Average / Max Month HRT (hr)	24 / 6	25 / 14
Cycles per Reactor per Day	4 – 6	6
MLSS (mg/L)	1500 – 5000	3000
SRT (d)	15 – 45	30
Sludge Dry Solids (kg/d)	-	1800
Sludge Cake at 20% TS (kg/d)	-	9,000

Waste sludge from SBR processes can be more difficult to dewater, since there is no primary sludge component. This will limit some of the sludge handling alternatives compared to process configurations that include the use of the existing primary clarifiers. Furthermore, liquid and solid discharges from SBRs are intermittent, resulting in larger and more complex downstream facilities.

SBR processes require proprietary controls and equipment. Two of the larger manufacturers with numerous installations in North America include Seimens (Omniflo) and Xylem (Sanitaire). The conceptual layout has been developed based on the projected design flows, loads, and design parameters. A three-train SBR site plan and plant layout is provided in Sketch 2 and Sketch 4 at the end of Chapter 10.

9.3 Membrane Bioreactors

Membrane bioreactor (MBR) technology is considered further because it can achieve the best effluent quality in terms of BOD, TSS and NH_3 and MBR does not require clarification.

Membrane Bioreactors (MBR) combine an aeration tank and a secondary clarifier into one unit, similar to a sequencing batch reactor. The difference is that process flow is continuous and footprint requirements for an MBR are smaller. Membranes are essentially filters that permit separation of solids from treated liquids. Because solids can be actively separated from liquids with a membrane, the need for gravity settling of solids is eliminated. The replacement of gravity clarification with membrane filtration allows the bioreactor to be operated with a shorter hydraulic retention time and longer solids retention time. These two characteristics combine to reduce tankage requirements for MBRs over conventional activated sludge systems operating at lower rates.

GE manufactures a proprietary MBR process that uses a system of hollow core membranes, flexible fibres 1 to 2 metres in length, which are designed to be operated under vacuum. The hollow fibres are configured in cartridges and are completely submerged in the bioreactor mixed liquor. A vacuum is applied to the cartridge header resulting in filtered wastewater being drawn through the membrane walls. The filtered water is of tertiary effluent quality and is suitable for UV disinfection and direct discharge. A typical MBR process schematic is shown in Figure 9.2.

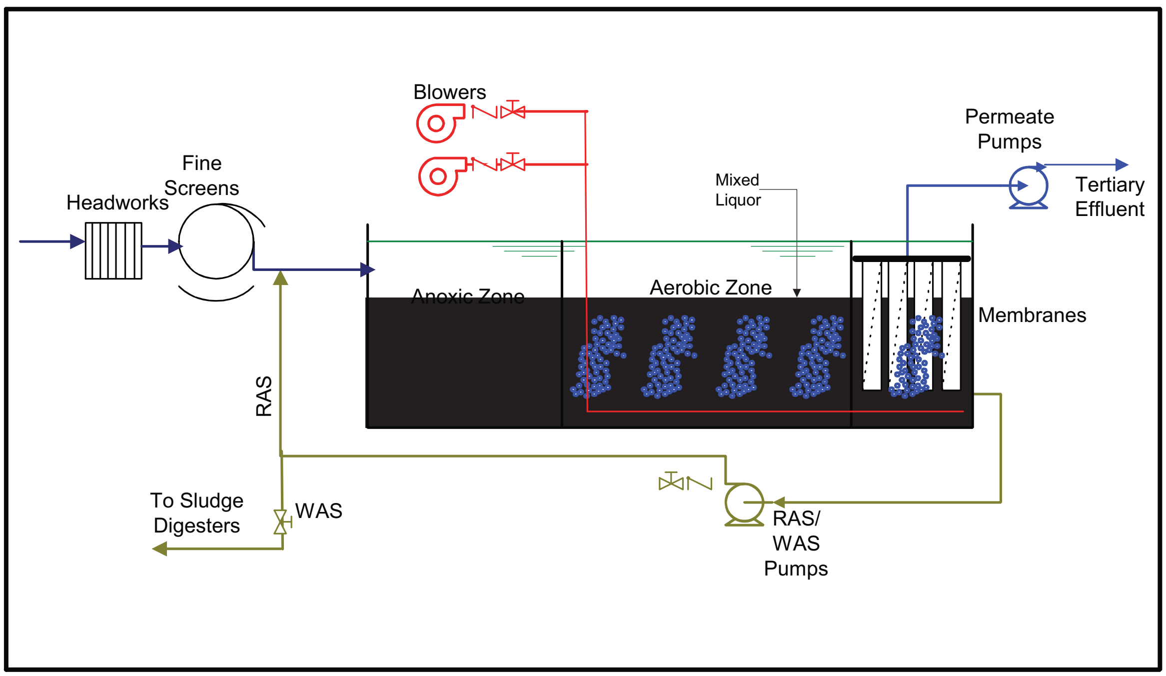


Figure 9.2: MBR Process Schematic with Anoxic Zone

A summary of design criteria for the GE MBR process is listed below:

Table 9.3: Membrane Bioreactor (MBR) Process Design

Parameter	Typical Design Standard	Design Year 2036
No. of Reactors	2	2
Basin Length (m)	-	26
Basin Width (m)	-	13
Top Water Depth (m)	-	6
Total Reactor Volume (m^3)	-	4050
Average / Max Month HRT (hr)	15 / 6	11.3 / 6.2
MLSS (mg/L)	8,000 – 15,000	10,000
SRT (d)	30 – 50	40
Sludge Dry Solids (kg/d)	-	1000
Sludge Cake at 20% TS (kg/d)	-	5,000

9.4 Biological Nutrient Removal

Biological nutrient removal (BNR) technology is considered further because this process is very similar to the existing activated sludge process and it can achieve the required effluent quality without the use of chemicals for phosphorus reduction.

Nutrient removal in wastewater treatment refers to phosphorus and nitrogen removal in addition to BOD and TSS removal. Biological nutrient removal (BNR) is a logical progression from the activated sludge process provided that provisions for such upgrades are included in the plant design. With BNR, the aim is to reduce nutrient loads to the receiving waters and minimize operating costs utilizing biological processes instead of chemicals. Figure 9.3 is one such BNR configuration designed to provide good Phosphorus and Nitrogen removal in addition to BOD and TSS removal rates.

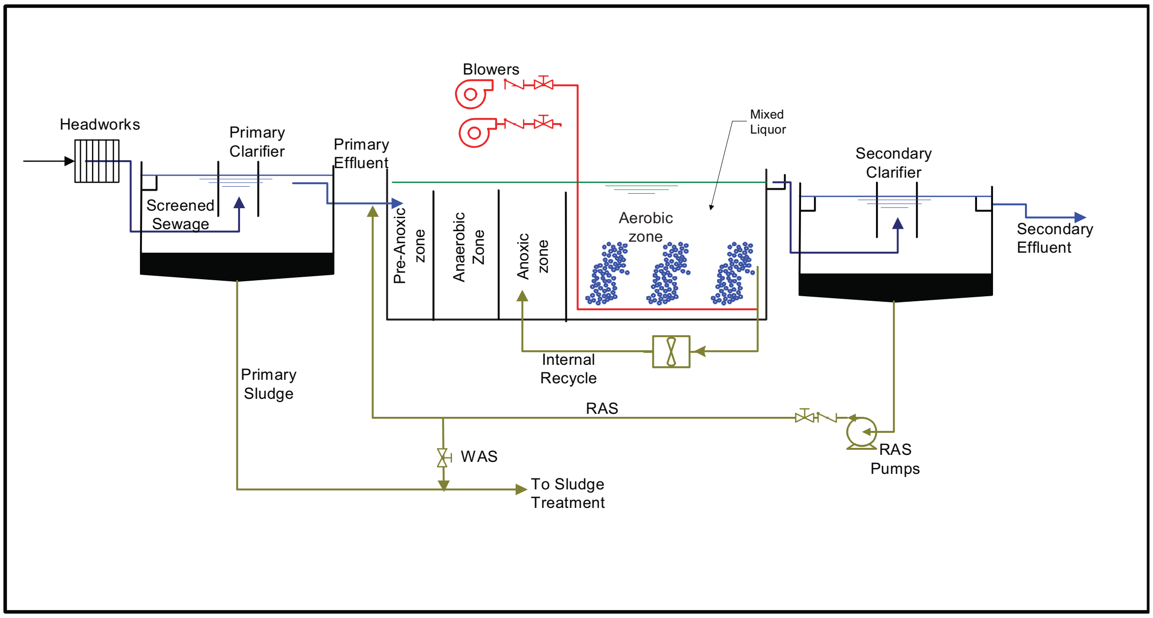


Figure 9.3: BNR Process Schematic Based on the Summerside Process

9.4.1 Bioreactor Zones and Description

In addition to the aerobic zone, which is common to the activated sludge process, selector cells are simply created by using internal baffles, recycle pumps and submersible mixers in lieu of aeration. The function of each zone is described below.

Pre-Anoxic Zone

- Un-aerated, completely mixed using submersible mixers;
- Short HRT < 1hr designed to remove nitrates (denitrify) present in the RAS and incoming sewage;
- The pre-anoxic zone functions to protect the anaerobic zone from nitrates; and
- Provides some removal of readily biodegradable BOD.

Anaerobic Zone

- Un-aerated, completely mixed using submersible mixers. HRT is < 1hr;
- Phosphorus accumulating organisms (PAOs) are conditioned here;
- An exchange of biodegradable carbon and stored polyphosphate results in a phosphorus release from the phosphorus accumulating organisms. This conditions the PAOs to recover additional phosphate in subsequent stages of the process;
- Main function of this zone is to induce the release of phosphate; and
- Provides some removal of readily biodegradable BOD.

Main Anoxic Zone

- Un-aerated, completely mixed using submersible mixers. HRT is typically 20% of the total;
- Contents (MLSS) from the end of the aerobic zone are recycled to the main anoxic zone. This recycle stream has a high concentration nitrates;
- The addition of nitrates to the anoxic zone results in a substitution of nitrate-oxygen for dissolved oxygen, which drives respiration and results in denitrification;

- The use of nitrates as an oxygen source and BOD results in the release of nitrogen gas to the atmosphere and thus the removal of nitrogen from the system; and
- Significant removal of BOD without requirement for aeration energy.

Aerobic Zone

- Aerated and completely mixed using fine bubble diffusers. The aerobic zone represents two-thirds of the total bioreactor volume;
- The addition of oxygen results in the completion of BOD removal and the initiation of ammonia oxidation;
- Nitrification (conversion of ammonia to nitrate) is completed in this zone once BOD has been removed;
- The uptake of phosphate is completed with the remaining dissolved phosphorus being acquired for storage in the cells of the PAOs; and
- Phosphorus is ultimately removed from the process when sludge is wasted form the system.

There will be a total of two process trains required for the Gagetown WWTP, with each one being a mirror image of the other. Two process trains are required, so that if one is taken out of service, adequate treatment with a single train can still be maintained for short periods of time.

Table 9.4: Biological Nutrient Removal Process Design

Parameter	Typical Design Standard	Design Year 2036
No. of Reactors	-	2
Basin Length (m)	-	26
Basin Width (m)	-	15
Side Water Depth (m)	-	6
Total Reactor Volume (m^3)	-	4,680
Average / Peak HRT (hr)	15/ 6	13 /7.2
MLSS (mg/L)	2500 – 4000	3000
SRT (d)	15 – 30	25
Sludge Dry Solids (kg/d)	-	1,200
Sludge Cake at 20% TS (kg/d)	-	6,000

A twin bioreactor plant layout is shown in Sketch 1 and Sketch 3 at the end of Chapter 10. Each bioreactor shown is plug-flow design with each pass being 26m long. The preliminary design for the first pass in each bioreactor is 6m width and contains the anoxic and anaerobic zones. The second pass in each bioreactor is 9m wide and is aerobic. The process reactor sizing and configuration should be confirmed in detailed design through biological process modelling.

9.5 Process Selection Summary

In this chapter a number of biological processes were listed and SBR, MBR and BNR processes were shortlisted as possible treatment process alternatives.

Some of the benefits attributable to membrane bioreactors are not key issues for the Gagetown WWTP process selection. For example, MBR technology is often chosen if tertiary level effluent (5/5 for BOD/TSS) is required or if site restrictions require a very small footprint. Neither of these benefits are driving factors for Gagetown. Since MBR technology will be the highest capital cost and the highest operating cost, it is not considered further at this time.

SBR technology and BNR technology are carried forward in Chapter 10 where the proposed plant site and conceptual layouts are presented.

CHAPTER 10 WWTP SITING & CONCEPTUAL LAYOUT

The existing wastewater treatment plant is almost 60 years old. In developing the predesign for the upgrade and expansion, CBCL consulted with DND/DCC before deciding that a new site and process layout was in the best interests of the project moving forward. This decision to build a new WWTP on a new site will yield the following benefits:

- Allows for treatment at existing plant to be maintained during construction;
- New site will simplify design, construction, commissioning and start-up;
- New plant facilitates ownership transfer to the Town;
- A new process is more flexible with regard to upgrade alternatives; and
- Better scope, schedule and cost estimates and ultimately a more cost effective solution.

This Chapter presents two options for site layouts incorporating the process options under consideration.

10.1 BNR Treatment Plant Siting and Layout

A Biological Nutrient Removal plant adjacent to the existing wastewater treatment plant site will offer some advantages including similar process operation to the existing treatment plant, non-proprietary process controls and equipment and phosphorus removal without the need for Alum addition.

A preliminary site plan depicting process tankage and buildings is provided in Sketch 1. This sketch provides a view of the proposed new BNR plant relative to the existing facility as well as space for future expansion. Sketch 3 shows a preliminary BNR process layout. Primary clarifiers remain rectangular but are much longer and of standard length. There are two process bioreactors each 15m x 26m x 6m depth with internal baffles to provide selector cell configuration (anoxic and anaerobic zones) and plug flow treatment. Secondary clarifiers are circular instead of rectangular as this geometry has superior settling characteristics for process MLSS. The proposed buildings include a new headworks building with space for workshop and a new sludge blending and disinfection building. It is proposed that the existing administration building and laboratory remain in service.

10.2 SBR Treatment Plant Siting and Layout

A Sequencing Batch Reactor plant adjacent to the existing wastewater treatment plant site will offer some advantages including biological treatment and clarification (separation of solids from liquids) all in the same basin. SBR processes can be designed to occupy a compact footprint with common wall construction. Because it is a hybrid process, the basins are not ideal for clarification or for biological treatment; however, good quality effluent on a consistent basin has been proven achievable at many facilities in Canada.

A preliminary site plan depicting process tankage and buildings is provided in Sketch 2. This sketch provides a view of the proposed new SBR plant relative, comprising three (3) parallel process trains with space for expansion to a fourth tank if required. Sketch 4 shows an SBR process layout. The preliminary dimensions for the three (3) SBR process reactors are 42m x 14m x 6m max depth. As with the BNR option, new buildings include a headworks building, and a new sludge thickening and disinfection building. There will be some additional chemical handling facilities as well. The existing administration building and laboratory will remain in service.

10.3 BNR versus SBR Process

Table 10.1 provides a qualitative comparison of the SBR and BNR options.

Table 10.1: Comparison of new SBR Versus New BNR Adjacent to the Existing WWTP

Characteristic	Sequencing Batch Reactors	Biological Nutrient Removal
Impact on existing treatment process	Little or no impact	Little or no impact
Comparison to existing process	Batch process versus continuous flow. Proprietary equipment, PLC controls, SCADA and HMI.	Similar to existing. Continuous flow process with non proprietary controls and equipment.
Phosphorus removal	Needs chemical P-removal; Alum feed system.	P-removal can be achieved biologically. Alum as backup.
Equipment cost	Less equipment cost. Oversize UV system due to batch discharge	Usually more equipment cost due to clarifiers and PS pumps.
Land requirement	Requires less land. Bigger bioreactors which serve as clarifiers during settle phase.	Requires more land owing to primary clarifiers and secondary clarifiers.
Sludge production	All biological sludge (waste activated sludge), more difficult to digest and dewater.	Contains primary sludge and biological sludge. Easier to digest and dewater.
Operating cost	More operating cost due to chemical and aeration energy. Aerated volume is 10,500m^3	Lower operating cost. Aerated volume is 4500m^3
Manpower and skilled operators	Similar manpower requirements.	Knowledgeable operators required for BNR operation.

10.3.1 Recommended Treatment Process Selection

Although both process options for the new WWTP are presented here, BNR is recommended for further consideration because of similarities to the existing treatment process and because it will require less chemical, less energy and therefore reduced operating cost. The BNR process is also more amenable to the sludge digestion and dewatering process that is in operation and is recommended for continued service.

10.4 Sludge Treatment and Processing

Continued use of the existing sludge digesters and dewatering process is assumed in the predesign for the new treatment process. Anaerobic digestion is a proven and reliable method for achieving sludge stabilization, solids destruction and pathogen reduction. In addition to being capable of treating raw sludge to Class B Biosolids treatment objectives or higher, anaerobic digestion offers the added benefit of being a net energy producer if digester gas (methane) is used for heating or plant energy requirements.

CBCL has recently been involved with anaerobic digester upgrades at the Charlottetown Pollution Control Plant which is 40 years old. In Charlottetown, sludge digestion is preceded by sludge pasteurization which achieves additional pathogen reduction (Class A) and aids in digestion and solids destruction. Digester gas is used for boiler energy and heating requirements in Charlottetown.

Continued use of the existing anaerobic digestion process at the Gagetown WWTP is assumed for all liquid process upgrade options. In the estimate of capital cost, allowances have been made for cleaning out the existing digesters, relining and recoating the tanks, and upgrading digester gas collection system to be code compliant. One Million, six hundred thousand ($1.6M) has been allowed for rehabilitation of the digesters. Recent design and construction experience for this type of work will be helpful in renovating and upgrading existing anaerobic digesters at Gagetown.

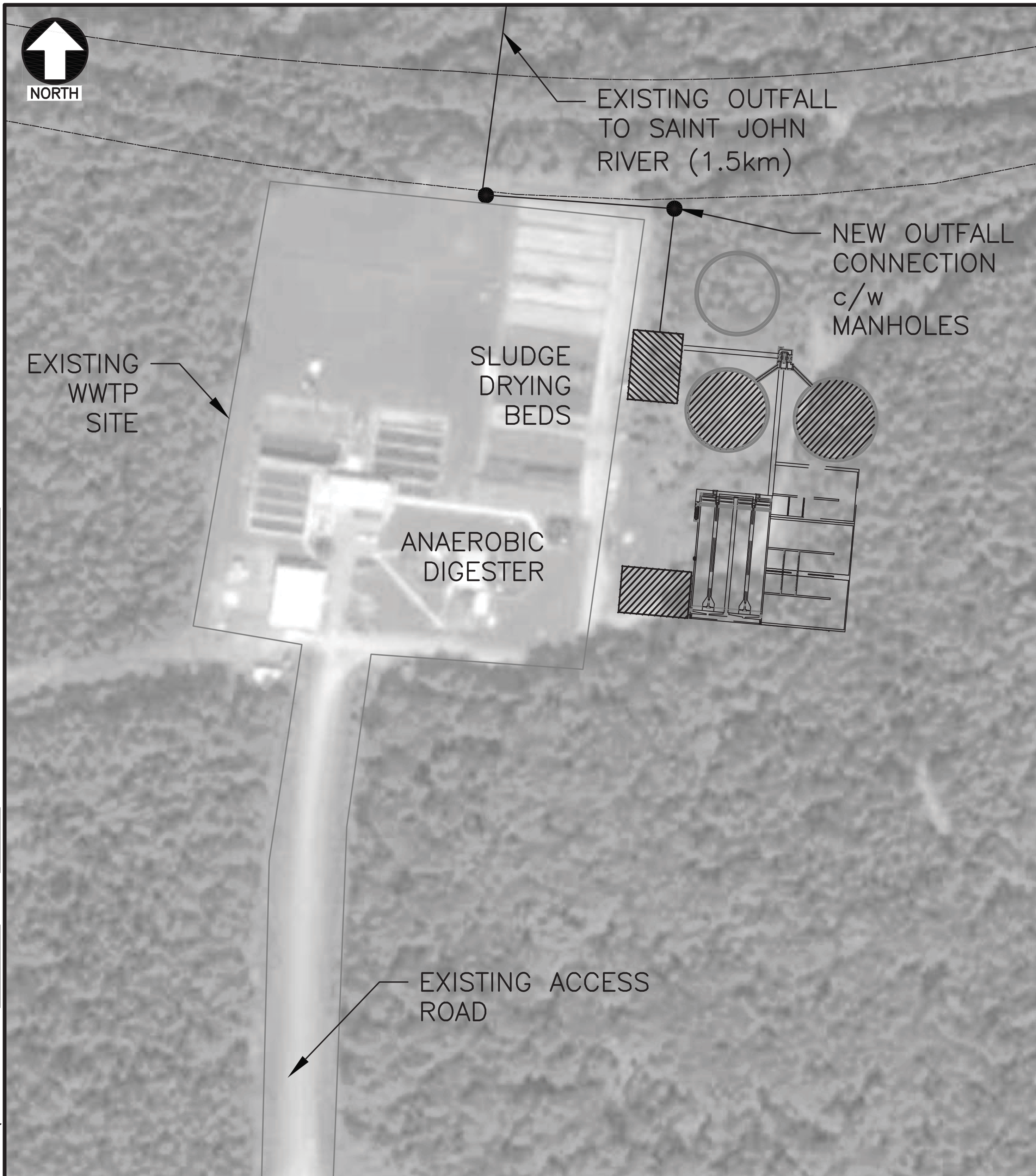
Continued use of the sludge drying beds and geotextile bags for dewatering of digested sludge is assumed. This has proven to be the most cost effective sludge dewatering method available and achieves the objectives for handling processed biosolids without creating odour problems at the plant or at neighbouring properties. At present dewatered sludge cake is hauled to a private commercial composting facility operating in the region for further processing and beneficial re-use.

10.5 Outfall Upgrade Requirements

The existing outfall extends to about 20m away from shore to a depth of about 0.5m at low water. The Environmental Risk Assessment (Jacques Whitford, 2010) recommended that the existing outfall pipe be extended to about 100m away from the river bank to a depth of 2.5m below low water level. The outfall extension was recommended to improve mixing and dilution and prevent shoreline attachment of treated effluent.

For the purpose of this predesign report, we have assumed 100m of outfall replacement within the river to a depth of 2.5m below low water. We have also assumed continued use of the existing 1500m +/- of discharge pipe that conveys treated effluent from the site of the new plant to the river bank.

A condition and capacity assessment of the existing 1500m +/- discharge pipe should be conducted to confirm that it is suitable for continued use.



Date	Scale	Designed	Drawn	Checked	Approved	Contract
NOV. 16/2012	1:1500	DD	MM			120828.01

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ENGINEERING STUDY FOR WATER AND WASTEWATER TREATMENT PLANTS, CFB GAGETOWN

PROPOSED WWTP SITE PLAN – BNR OPTION

Sketch
1

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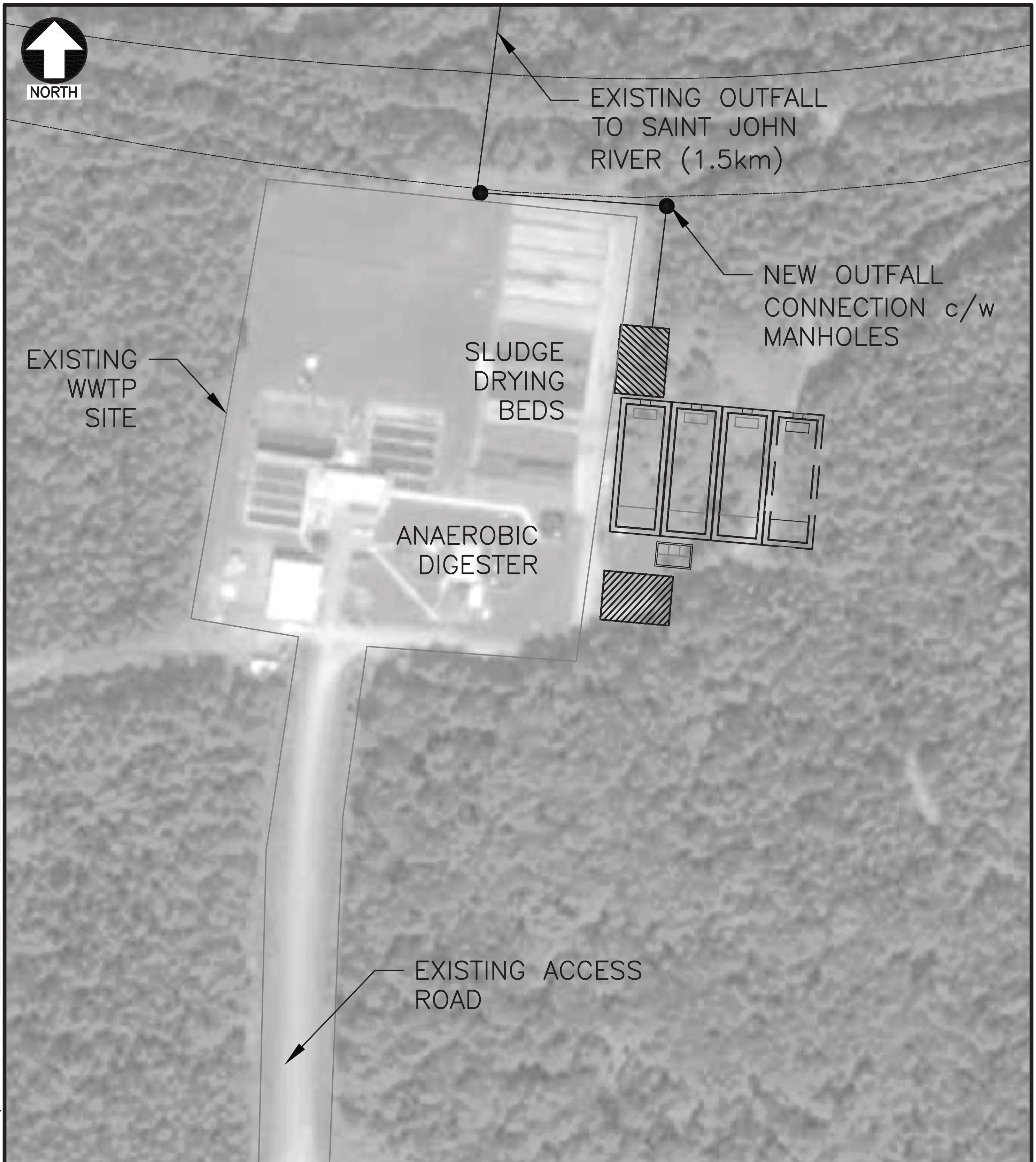

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PROPOSED WWTP SITE PLAN – SBR OPTION

Sketch
2

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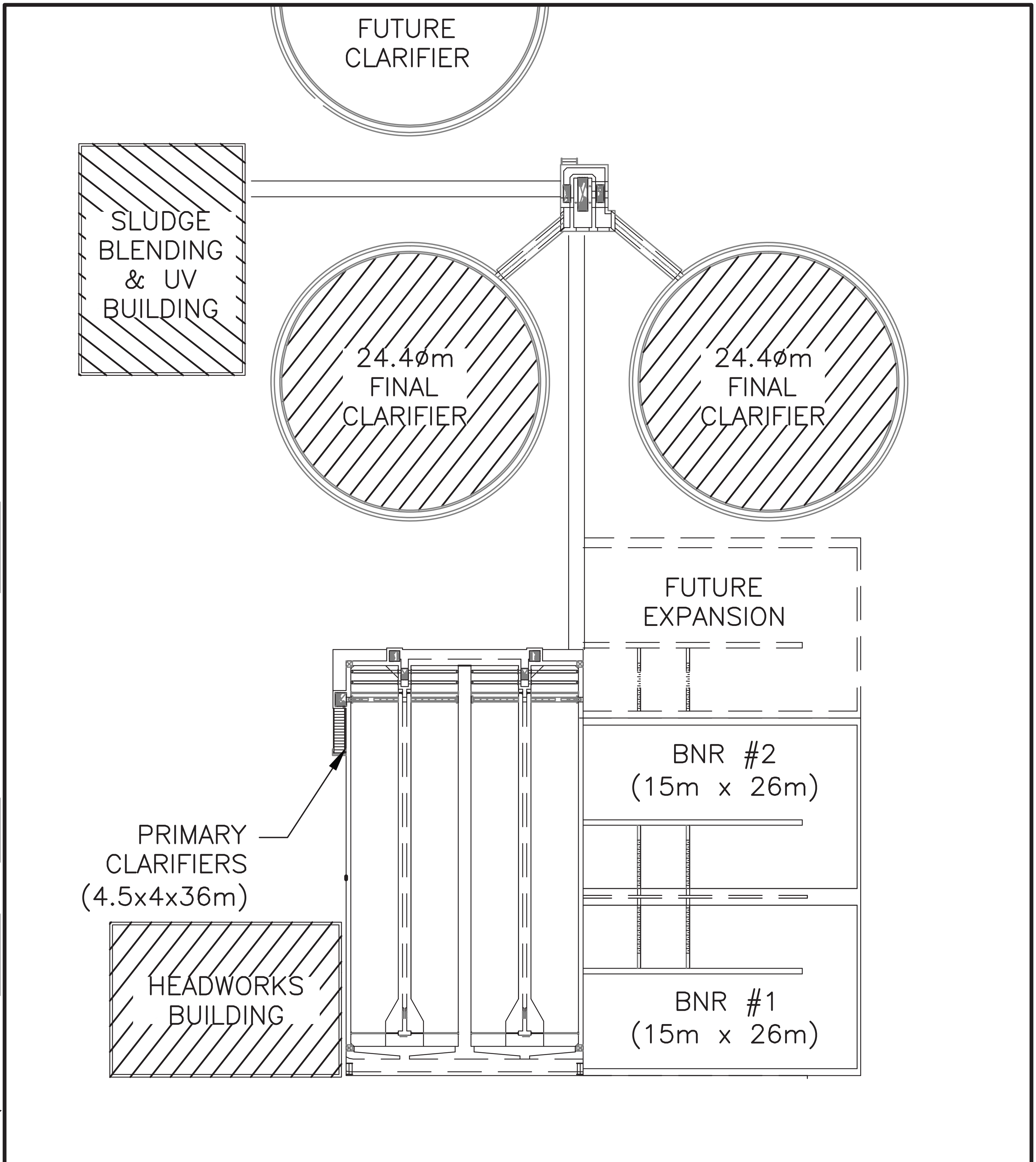

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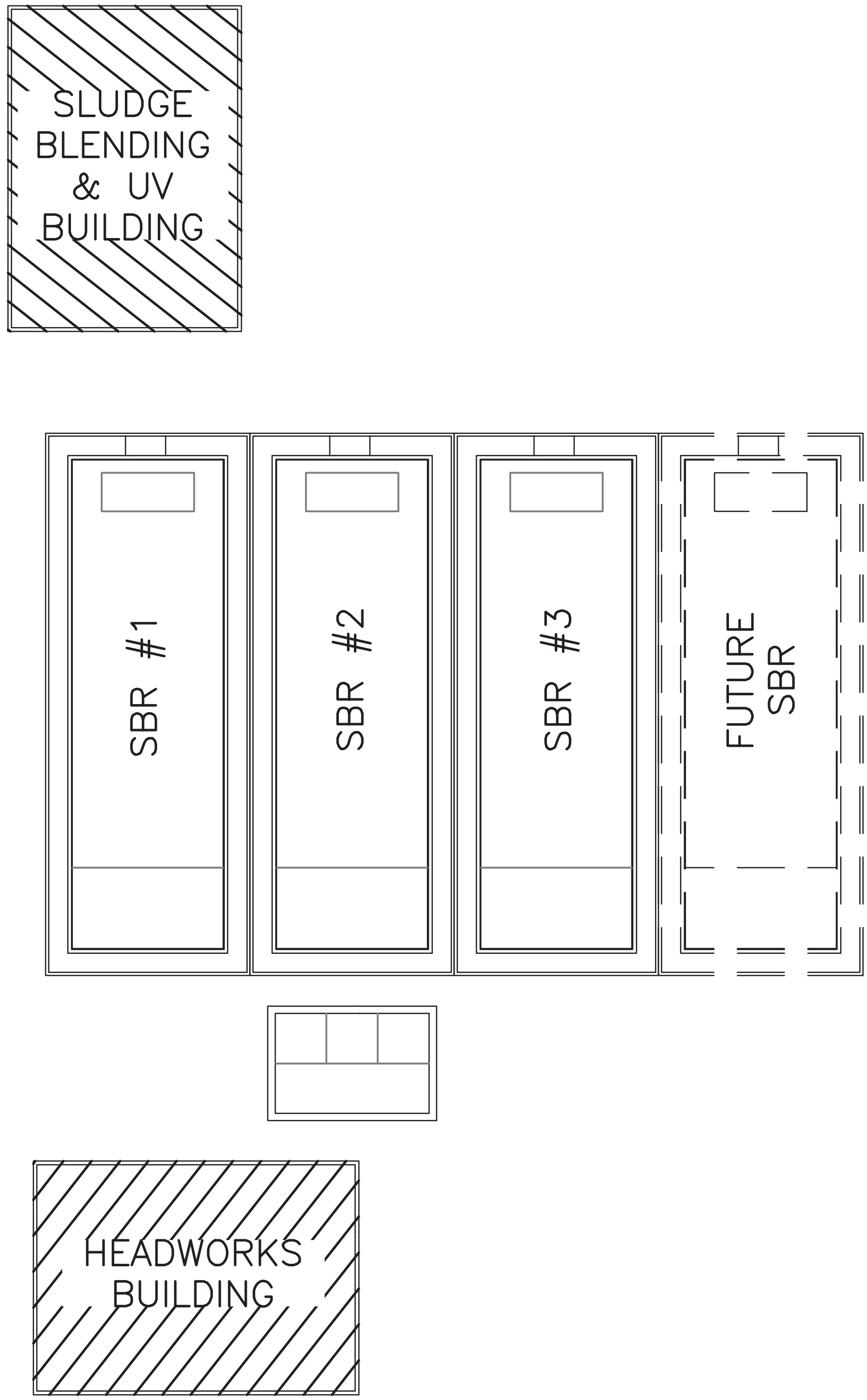
PROPOSED WWTP PLANT LAYOUT – BNR OPTION

Sketch
3

DRAWING NAME: L:\123252 OROMOCTO\123252-50.DWG LAYOUT NAME: WWTP PLOT DATE:December-06-12 2:39:08 PM CAD OPERATOR: MATTM



Date	Scale	Designed	Drawn	Checked	Approved	Contract
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PROPOSED WWTP PLANT LAYOUT – SBR OPTION

Sketch
4

DRAWING NAME: L:\123252 OROMOCTO\123252-50.DWG LAYOUT NAME: WWTP PLOT DATE:December-06-12 1:47:37 PM CAD OPERATOR: MATTM

CHAPTER 11 PART B - WWTP COST ESTIMATES

11.1 WWTP Capital Cost Estimate

A pre-design cost estimate (Class D), not including taxes, is presented below. Tables 11.1 and 11.2 include estimated opinion of capital costs for a new wastewater treatment plant based on BNR technology and SBR technology. The cost estimate includes retrofit of the existing anaerobic digesters ($1.6M), extension of the outfall into the Saint John River ($0.57M), demolition of the old WWTP and a renovation allowance for the existing administration building.

Equipment suppliers for major unit processes were contacted and budgetary proposals for supply were received. Equipment prices and unit prices from other treatment plants in the region (Fredericton WPCC and the Eastern WWTF in Saint John) recently designed by CBCL were used in the cost estimate. For the SBR Option #2, budgetary quotations were received from two equipment suppliers for their package supply. Despite the availability of recent and local prices, the cost estimate is still very conceptual in nature.

A Class D estimate is an order of magnitude conceptual estimate used to screen various alternatives. Project documents are in the initial stages but are sufficient to provide an indication of probable cost and allow ranking of options being considered. When preparing more accurate cost estimates, drawings for civil, structural, process, mechanical and electrical design are used to develop equipment lists and arrive at accurate quantities. At this stage in the project, only sketches are available and so quantities are very approximate even if unit prices may be accurate.

Table 11.1: Predesign Opinion of Probable Cost – BNR Option #1
CLASS D Project Budget

Item Description		Total
Division 2 - Site Civil		**2 Train BNR Process**
Site Preparation		
Excavation and Site Grading		
Rock Excavation		
Imported Fill and Granular Material		
Pavement (roads, parking areas)		
Gravel Access Roads		
Yard Pipework, MH's, Pipe Connections, Etc.		
Site Fencing		
Site Finishes & Reinstatement		
Environmental Protection & Testing		
Replace Outfall from River Bank		
Subtotal - Division 2		**$ 2,634,106**
Division 3 - Concrete & Waterproofing		
Primary Clarifiers		
Bioreactors		
Secondary Clarifiers		
Sludge Blending and Pump Station		
Headworks Bldg		
Channels		
Subtotal - Division 3		**$ 3,708,140**
Division 4 to 10 - Buildings		
Headworks & Blower Building		
Sludge Blending and Pump Station UV Bldg		
Existing Control Building		
Anaerobic Digester Coatings and Rehab		
Main Process Tankage (misc. metal fabrication)		
Demolition of Existing Infrastructure		
Subtotal - Division 4 to 10		**$ 2,943,696**
Division 11 & 14 - Process Equipment Supply		
WAS PreThickener (RDTs)		
U.V. Disinfection System		
Bar Screens		
Grit Removal System		
Grit Classifier Equipment		
Primary Clarifier Internals		
Aeration Diffusers (membranes)		
Aeration Blowers (centrifugal)		
Secondary Clarifier Mechanism		
Sludge Blending Equipment		
Primary Sludge Pumps		
RAS Pumps (Horizontal, end suction)		
WAS Pumps (Horizontal, end suction)		

Item Description		Total
Service Water Pumps (Vert. Centrifugal)		
Bioreactor Mixers		
Bioreactor Recycle Mixers		
Liquid Polymer System for RDTs		
Misc. Process Equipment		
Air Compressor System		
Sump Pumps		
Material Handling Equipment		
Parshall Flume		
Odour control		
Diesel Generator		
Subtotal - Division 11 & 14		**$ 2,860,000**
Division 15 - Mechanical		
Process Mechanical Pipe, Valves, Etc., Supply		
Process Mechanical Install		
Plumbing, Heating & Ventilation		
Subtotal - Division 15		**$ 2,803,400**
Division 16 - Electrical & Instrumentation		
Power Distribution and General Electrical		
Instrumentation and Control		
Subtotal - Division 16		**$ 1,430,000**
Subtotal		**$ 16,379,341**
Design Development Contingency	20%	$ 3,275,868
Construction Contingency	10%	$ 1,965,521
Full Engineering Services (10%)	10%	$ 2,162,073
SCADA		$ 150,000
BNR Option #1Total Estimated Cost (not including taxes)		**$ 23,932,803**

Table 11.2: Predesign Opinion of Probable Cost – SBR Option #2
CLASS D Project Budget

Item Description		Total
Division 2 - Site Civil		**3 Train SBR Process**
Site Preparation		
Excavation and Site Grading		
Rock Excavation		
Imported Fill and Granular Material		
Pavement (roads, parking areas)		
Gravel Access Roads		
Yard Pipework, MH's, Pipe Connections, Etc.		
Site Fencing		
Site Finishes & Reinstatement		
Environmental Protection & Testing		
Replace Outfall from River Bank		
Subtotal - Division 2		**$ 2,495,020**
Division 3 - Concrete & Waterproofing		
Bioreactors		
Sludge Blending and Pump Station		
Headworks Bldg		
Channels		
Subtotal - Division 3		**$ 3,030,740**
Division 4 to 10 - Buildings		
Headworks & Blower Building		
Sludge Blending and Pump Station UV Bldg		
Existing Control Building		
Anaerobic Digester Coatings and Rehab		
Main Process Tankage (misc. metal fabrication)		
Demolition of Existing Infrastructure		
Subtotal - Division 4 to 10		**$ 2,943,696**
Division 11 & 14 - Process Equipment Supply		
WAS Rotary Drum Thickeners		
U.V. Disinfection System		
Bar Screens		
Grit Removal System		
Grit Classifier Equipment		
SBR Process Equipment Package		
Sludge Blending Equipment		
Service Water Pumps (Vert. Centrifugal)		
Liquid Polymer System for RDTs		
Chemical Feed System for P-Removal		
Misc. Process Equipment		
Air Compressor System		
Sump Pumps		
Material Handling Equipment		
Parshall Flume		

Item Description		Total
Odour control Diesel Generator **Subtotal - Division 11 & 14**		**$ 2,755,000**
Division 15 - Mechanical		
Process Mechanical Pipe, Valves, Etc., Supply Process Mechanical Install Plumbing, Heating & Ventilation **Subtotal - Division 15**		**$ 2,729,900**
Division 16 - Electrical & Instrumentation		
Power Distribution and General Electrical Instrumentation and Control **Subtotal - Division 16**		**$ 1,377,000**
Subtotal		**$ 15,331,356**
Design Development Contingency	20%	$ 3,066,271
Construction Contingency	10%	$ 1,839,763
Estimated Construction Cost (not including taxes)		**$ 20,237,389**
Full Engineering Svc (10%)	10%	$ 2,023,739
SCADA		$ 150,000
SBR Option #2 Total Estimated (not including taxes)		**$ 22,411,128**

These opinions of probable costs are based on concept level design development and are presented on the basis of experience, qualifications, and best judgement. Cost estimates have been prepared in accordance with acceptable principles and practices. Market trend changes, non-competitive bidding situations, unforeseen labour and material adjustments, and the like are beyond the control of CBCL Limited, and as such we cannot warrant or guarantee that actual costs will not vary from the opinion provided.

In total, the SBR Option #2 is estimated to be $1.5M less capital cost than the BNR Option #1 owing to less concrete tankage and reduced civil costs. The process equipment supply is also slightly less with the SBR Option.

11.2 WWTP Operating and Maintenance Cost

Most of the operating and maintenance cost items for the new treatment plant will be similar to the existing wastewater treatment plant. The BNR plant will utilize blowers, submersible mixers, recycle pumps and a UV disinfection system. The SBR plant will utilize blowers, submersible mixers a chemical feed system for P-removal and UV disinfection. The existing plant depends on chemical feed systems for phosphorus removal and chlorine disinfection of final effluent where as the new plant will not require these chemical feed systems as part of the daily operation. Sludge processing, dewatering, hauling and disposal costs will be about the same.

Operating costs presented in Table 10.3 and 10.4have been developed based on conditions at the design flows and loads in the year 2036. It is expected that operating costs for maintenance, power, hauling and chemicals will be less during the initial years of operation.

Table 11.3: Annual Operating Cost Estimate at Design Flows for BNR Option #1

Item	Cost/Year
Salaries (6 fulltime staff)	$300,000
Maintenance	$100,000
Power (blowers, UV, pumps, lights etc.)	$150,000
Testing	$5,000
Biosolids for Beneficial Reuse - ($100/wet tonne at 15%TS)	$150,000
Chemicals (soda ash, polymer)	$50,000
Totals	**$755,000**

Operating costs for the SBR Option #2 are estimated to be $170,000 per year more than BNR Option #1 due to power cost and chemical cost. With BNR, a significant portion of the process reactor volume is unaerated, yet BOD removal still occurs in these anoxic and anaerobic zones. The result is reduced blower energy which is offset slightly by power required to operate submersible mixers n these zones.

Table 11.4: Annual Operating Cost Estimate at Design Flows for SBR Option #2

Item	Cost/Year
Salaries (6 fulltime staff)	$300,000
Maintenance	$100,000
Power (blowers, UV, pumps, lights etc.)	$170,000
Testing	$5,000
Biosolids for Beneficial Reuse - ($100/wet tonne at 15%TS)	$150,000
Chemicals (soda ash, polymer, alum)	$200,000
Totals	**$925,000**

The chemical cost difference between the two options is due to the estimated Alum requirements for the SBR option. It is estimated that 800kg/day of Alum will be required at design conditions to achieve the effluent phosphorus limits that are required through chemical phosphorus removal. For the BNR option, it is assumed that effluent phosphorus limits can be achieved without the use of chemicals.

CHAPTER 12 REFERENCES

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APPENDIX A

Water Treatability Study